CHAPTER 4. BRIDGE DESIGN

4.1 DESIGN CONDITIONS

4.1.1 Design Standard

The bridge design standard of the Project, including Bago River Bridge with the on-ramp bridge and flyover, complies with the JSHB. However, the calculations of the live load and the collision force were referred to the AASHTO LRFD Design Standard as a conventional bridge design load in Myanmar. Natural conditions related to the criteria such as meteorological issues were considered independently in this section.

4.1.2 Materials to be Used

Materials to be used for the Project are based on the Japanese Industrial Standard (JIS) since JSHB is based on JIS, and is applied to the design of the bridge.

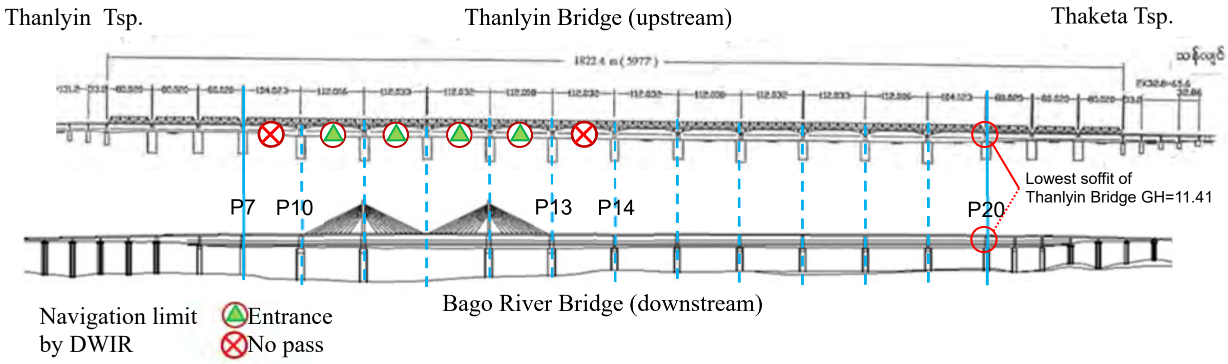
However, "equivalent" materials and/or products will be allowed in the technical specifications for the international procurement.

4.1.3 Span Arrangement in River Bridge Section

In consideration of the hydrological advantage and safety for the vessel, the pier arrangement of Bago River Bridge was allocated on the line-of-sight of the existing Thanlyin Bridge. Although Bago River is relatively shallow, middle-class vessel runs through the abyss near the Thanlyin side assigned by DWIR.

Four spans with green triangular signs indicated in Figure 4.1.1 are the current navigation route, which was designed to allocate space for the cable-stayed bridge for the new Bago River Bridge. Even though "no pass" was allocated for the other spans, the same span length is allocated for more than 100 m of the span of Thanlyin Bridge.

Navigation height is determined by the lowest soffit of Thanlyin Bridge at the P20 pier location of Bago River Bridge where the vertical alignment is lowest at navigation channel.



Source: JICA Study Team

Figure 4.1.1 Pier Arrangement of Bago River Bridge

4.1.4 Design Conditions for the Bridge Design

The design conditions are shown in the tables found in the next few pages.

Table 4.1.1 General Conditions

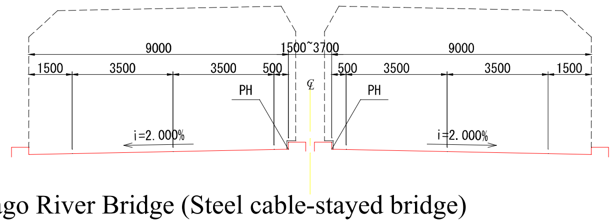
Item	Design Conditions			Remark
Design objective	Construction of new bridges and improvement of Thanlyin Chin Kat Road			
	Project length	3,644.341 m		
	River bridge	Length	2031.000 m	
		Superstructure	Steel cable-stayed bridge 448.000 m Steel box girder bridge 1,033.000 m (257 m, 776 m) PC box girder Bridge 550.000 m (250 m, 300 m)	
		Substructure	Wall pier, hammerhead pier, reverse-T abutment	
		Foundation	Steel pipe sheet pile (SPSP), cast-in-situ pile	
	Flyover	Length	602.000 m	
		Superstructure	Steel box girder bridge180.000 m Steel I girder bridge 122.000 m PC I girder bridge 300.000 m (60 m, 180 m, 60 m)	
		Substructure	Hammerhead pier, reverse-T abutment	
		Foundation	Cast-in-situ pile	
	On-ramp bridge	Length	115.200 m	
		Superstructure	PC I girder bridge 115.200 m	
		Substructure	Hammerhead pier, reverse-T abutment	
		Foundation	Cast-in-situ pile	
	Road improvement	Approach road Thanlyin side 357 m, Thaketa side 430 m		
		Arterial road 834.341 m		
	Intersection	Star City intersection, Shukinthar intersection, Yadanar intersection		
	Toll collection	Thaketa side (both northbound and southbound)		
Bridge name	Bago River Bridge			
Line name	Thanlyin Chin Kat Road			
Road design standards	Specifications for Road Design (Japan), June 2015, Japan Road Association (JRA) AASHTO A Policy on Geometric Design of Highways and Streets, 6th Edition (2011) for vertical clearance 5.0 m ASEAN Highway standard for traffic lane width 3.5 m Road Design Criteria in Myanmar, Department of Highway, Ministry of Construction (2015) for general reference			
Structural design standards	AASHTO LRFD Bridge Design 7th Edition (2014) for calculations of live load and collision force Specifications for Highway Bridge, March 2012, JRA Specifications for Earthwork for Road, June 2009, JRA Guidelines for Road Embankment, April 2010, JRA Guidelines for Road Revetment, July 2012, JRA Guidelines for Soft Soil Treatment, August 2012, JRA Guidelines for Design of Pile Foundations, March 2015, JRA Guidelines for Construction of Steel Pipe Pile Foundations, December 1997, JRA Other Relevant Standards and/or Documents			

Source: JICA Study Team

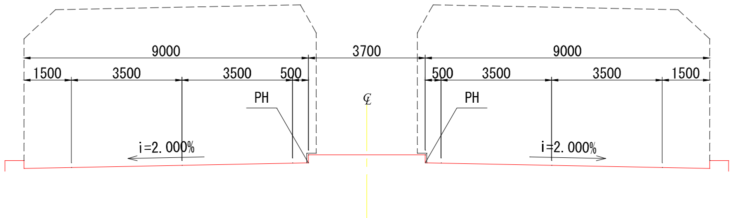
Table 4.1.2 Road Design Conditions

<table>
<tr><th>Item</th><th>Design conditions</th><th>Remark</th></tr>
<tr><td>Road classification</td><td>Bago River Bridge Equivalent to Class 2-1
Flyover Equivalent to Class 4-1
On-ramp Equivalent to Class C
Improvement of Thanlyin Chin Kat Road Equivalent to Class 4-1</td><td>Based on Japanese Road Structure Ordinance</td></tr>
<tr><td>Design speed</td><td>Bago River Bridge, Flyover 60 km/h
On-ramp 30 km/h
Thanlyin Chin Kat Road 40 km/h</td><td></td></tr>
<tr><td>Design traffic volume</td><td>Bago River Bridge
44,356 vehicle/day (northbound 25,352 v/d, southbound 19,004 v/d)
Trucks 6,173 vehicle/day (northbound 2,829 v/d, southbound 3,344 v/d)
Flyover
21,723 vehicle/day (northbound 12,061 v/d, southbound 9,662 v/d)
Trucks 3,639 vehicle/day (northbound 1,549 v/d, southbound 2,090 v/d)</td><td>Supplemental survey results, YUTRA Master Plan Case, 2035 time point</td></tr>
<tr><td>Planar road alignments</td><td>Bago River Bridge to Flyover
<table>
<tr><td>SP</td><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td></tr>
<tr><td>0+000.000</td><td>0+024.970</td><td>0+076.170</td><td>0+161.513</td><td>0+212.713</td><td>0+521.900</td></tr>
<tr><td>R=∞</td><td>A=160</td><td>R=-500</td><td>A=160</td><td>R=∞</td><td>R=-2000</td></tr>
<tr><td>6</td><td>7</td><td>8</td><td>9</td><td>10</td><td>11</td></tr>
<tr><td>0+857.522</td><td>2+627.420</td><td>2+680.992</td><td>2+724.080</td><td>2+777.651</td><td>2+782.486</td></tr>
<tr><td>R=∞</td><td>A=150</td><td>R=-420</td><td>A=150</td><td>R=∞</td><td>A=130</td></tr>
<tr><td>12</td><td>13</td><td>14</td><td>EP</td><td></td><td></td></tr>
<tr><td>2+835.298</td><td>2+961.571</td><td>3+014.383</td><td>3+644.341</td><td></td><td></td></tr>
<tr><td>R=320</td><td>A=130</td><td>R=∞</td><td>-</td><td></td><td></td></tr>
</table>
On-ramp
<table>
<tr><td>SP</td><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td></tr>
<tr><td>0+000.000</td><td>0+004.472</td><td>0+058.045</td><td>0+105.007</td><td>0+148.111</td><td>0+367.483</td></tr>
<tr><td>R=∞</td><td>R=-140</td><td>R=∞</td><td>A=50</td><td>R=-58</td><td>A=50</td></tr>
<tr><td>6</td><td>7</td><td>EP</td><td></td><td></td><td></td></tr>
<tr><td>0+410.587</td><td>0+535.778</td><td>0+643.083</td><td></td><td></td><td></td></tr>
<tr><td>R=∞</td><td>R=-1000</td><td>-</td><td></td><td></td><td></td></tr>
</table></td><td></td></tr>
<tr><td>Profiles</td><td>Bago River Bridge to Flyover
<table>
<tr><td>0+0.000</td><td>0+228.000</td><td>0+700.000</td><td>1+88.000</td><td>2+140.000</td><td>2+517.727</td></tr>
<tr><td>5.695</td><td>5.467</td><td>17.267</td><td>18.431</td><td>15.275</td><td>5.832</td></tr>
<tr><td>-0.100</td><td>2.500</td><td>0.300</td><td>-0.300</td><td>-2.500</td><td>3.000</td></tr>
<tr><td>2+830.000</td><td>2+960.000</td><td>3+160.000</td><td>3+475.000</td><td>3+500.000</td><td></td></tr>
<tr><td>15.200</td><td>15.850</td><td>14.420</td><td>4.970</td><td>4.895</td><td></td></tr>
<tr><td>0.500</td><td>-0.715</td><td>-3.000</td><td>-0.300</td><td>-</td><td></td></tr>
</table>
On-ramp
<table>
<tr><td>0+0.000</td><td>0+150.000</td><td>0.329.942</td><td>0+490.000</td><td>0+540.000</td><td></td></tr>
<tr><td>4.470</td><td>4.470</td><td>5.010</td><td>13.780</td><td>14.878</td><td></td></tr>
<tr><td>level</td><td>0.300</td><td>5.479</td><td>2.197</td><td>-</td><td></td></tr>
</table></td><td></td></tr>
<tr><td>Cant</td><td>Bago River Bridge 2% crossfall (Max. 4% camber)
Flyover 2% crossfall (Max. 6% camber)
On-ramp 2% camber (Max. 9% camber)</td><td></td></tr>
</table>

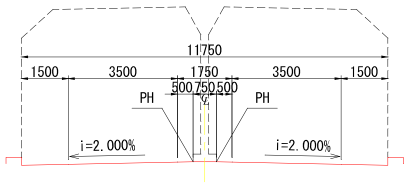
<table>
<tr><td>Cross section</td><td>Bago River Bridge (PC box girder and Steel box girder)

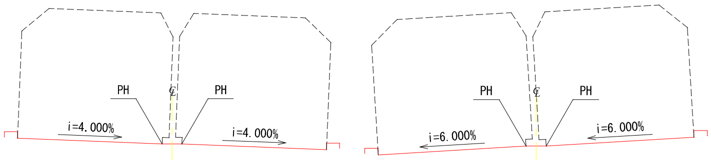


Bago River Bridge (Steel cable-stayed bridge)

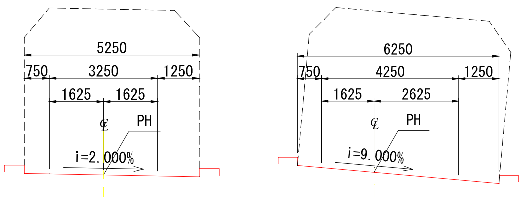


Flyover (crossfall, 4% camber and 6% camber)





On-ramp (2% camber and 9% camber)


</td><td></td></tr>
<tr><td>Widening</td><td>Bago River Bridge no widening but median
Flyover no widening
On-ramp 1.00 m widening at R = 58 section</td><td></td></tr>
<tr><td></td><td></td><td></td></tr>
</table>

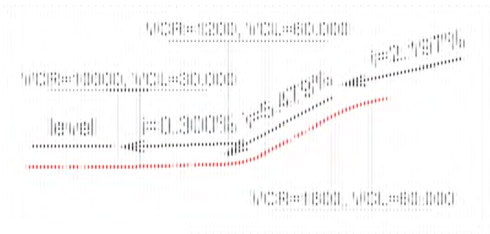
Source: JICA Study Team

Source: JICA Study Team

Figure 4.1.3 Planar Alignment

Source: JICA Study Team

Figure 4.1.4 Vertical Alignment (Main Road)



Source: JICA Study Team

Figure 4.1.2 Vertical Alignment (On-ramp)

Table 4.1.3 River Conditions

<table>
<tr><th>Item</th><th>Design Conditions</th><th>Remark</th></tr>
<tr><td>River name</td><td>Bago River</td><td></td></tr>
<tr><td>Navigation</td><td>Pier P10 to P13 will be the navigation after construction.
Pier P7 to P20 will also be the navigation in the future.</td><td>Agreement with DWIR</td></tr>
<tr><td>Clearance</td><td>Vertical height and width shall be secured between Pier P7 to P20 as Thanlyin Bridge</td><td>Agreement with DWIR</td></tr>
<tr><td>Design discharge</td><td>16,169 m³/s (100-year return period)</td><td></td></tr>
<tr><td>Design high water level (HWL)</td><td>
<table>
<tr><th>Load combination</th><th>Supposition</th><th>Water level (MSL＋m)</th><th>River flow (m/s)</th></tr>
<tr><td>Normal</td><td>Full/low tide of spring tide</td><td>+3.18／-2.39</td><td>0</td></tr>
<tr><td>Wind</td><td>Highest HWL</td><td>+4.99</td><td>0</td></tr>
<tr><td>Collision at navigation span</td><td>Full tide of spring tide</td><td>+3.18</td><td>0</td></tr>
<tr><td>Collision at side span</td><td>Maximum river flow at flood of 100year return period</td><td>+2.53</td><td>1.19</td></tr>
<tr><td>Earthquake</td><td>Normal water level</td><td>+0.29</td><td>0.60</td></tr>
<tr><td>During construction</td><td>5year return period</td><td>+4.34</td><td>0.65</td></tr>
</table>
</td><td></td></tr>
<tr><td>Design riverbed and scouring depth</td><td colspan="2">
<table>
<tr><th></th><th>P6</th><th>P7</th><th>P8</th><th>P9</th><th>P10</th></tr>
<tr><td>Riverbed height</td><td>0.41</td><td>-3.59</td><td>-5.35</td><td>-4.82</td><td>-4.55</td></tr>
<tr><td>Foundation height</td><td>-2.48</td><td>-6.38</td><td>-6.34</td><td>-6.35</td><td>-9.10</td></tr>
<tr><td>Maximum scouring depth</td><td>-3.41</td><td>-8.91</td><td>-9.42</td><td>-9.31</td><td>-11.27</td></tr>
</table>
<table>
<tr><th>P11</th><th>P12</th><th>P13</th><th>P14</th><th>P15</th><th>P16</th><th>P17</th><th>P18</th></tr>
<tr><td>-5.41</td><td>-7.96</td><td>-8.02</td><td>-6.28</td><td>-5.09</td><td>-5.26</td><td>-6.70</td><td>-6.99</td></tr>
<tr><td>-9.10</td><td>-9.10</td><td>-9.10</td><td>-8.06</td><td>-8.06</td><td>-8.06</td><td>-8.06</td><td>-8.06</td></tr>
<tr><td>-12.13</td><td>-13.67</td><td>-13.48</td><td>-11.43</td><td>-10.84</td><td>-10.36</td><td>-9.70</td><td>-10.00</td></tr>
</table>
<table>
<tr><th>P19</th><th>P20</th><th>P21</th><th>P22</th><th>P23</th><th>P24</th><th>P25</th></tr>
<tr><td>-6.88</td><td>-6.55</td><td>-6.15</td><td>-4.61</td><td>-0.05</td><td>4.11</td><td>4.04</td></tr>
<tr><td>-8.06</td><td>-7.28</td><td>-7.55</td><td>-7.59</td><td>-2.39</td><td>3.73</td><td>3.78</td></tr>
<tr><td>-9.78</td><td>-9.53</td><td>-8.56</td><td>-7.48</td><td>-2.07</td><td>3.98</td><td>3.92</td></tr>
</table>
Half of the maximum scouring depth is used for the seismic design of substructures and foundations.
</td></tr>
<tr><td>Reference height</td><td>Benchmark survey result at Monkey Point
MSL = CDL + 2.814 m
All the height in the Project will be expressed as the height from MSL</td><td></td></tr>
<tr><td></td><td></td><td></td></tr>
</table>

Source: JICA Study Team

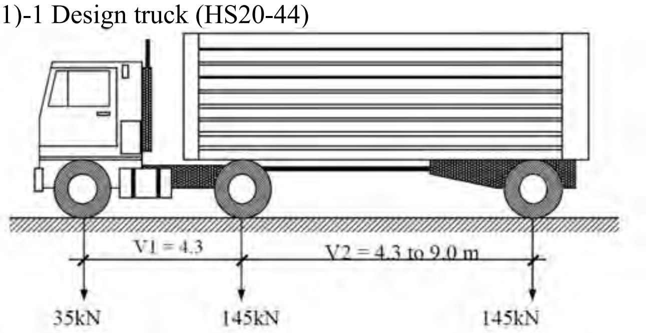
Table 4.1.4 Natural Conditions

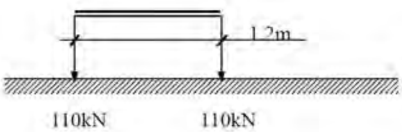
Item	Design Conditions	Remark
Temperature	39.2 to 11.3 (Celsius) at Kaba-Aye metrological station, 1991 to 2015	
Wind speed	42.9 m/s (Cyclone Nargis, 27 April 2008)	
Rainfall amount	149 mm/h (3-year return period, 10-minute rainfall intensity)	

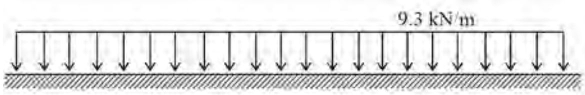
Source: JICA Study Team

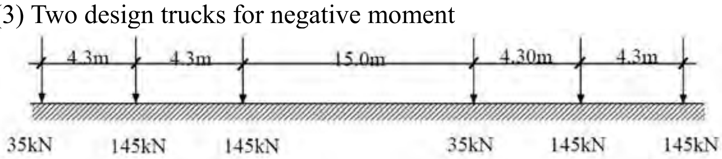
Table 4.1.5 Design Conditions

<table>
<tr><th>Item</th><th>Design Conditions</th><th>Remark</th></tr>
<tr><td>Dead load</td><td>These values are used for unit self-weight of the materials.
<table>
<tr><th>Materials</th><th>Unit Self-weight (kN/m^3)</th></tr>
<tr><td>Steels</td><td>77.0</td></tr>
<tr><td>Cast steel</td><td>71.0</td></tr>
<tr><td>Aluminum</td><td>27.5</td></tr>
<tr><td>Reinforced concrete</td><td>24.5</td></tr>
<tr><td>Prestressed concrete</td><td>24.5</td></tr>
<tr><td>Concrete</td><td>23.0</td></tr>
<tr><td>Mortar, cement</td><td>21.0</td></tr>
<tr><td>Timber</td><td>8.0</td></tr>
<tr><td>Bitumen</td><td>11.0</td></tr>
<tr><td>Asphalt concrete</td><td>22.5</td></tr>
</table></td><td>JSHB 2.2.1</td></tr>
<tr><td>Live load</td><td>1. AASHTO HL-93
Combination of these two different types of loads is considered.
(1) design truck or design tandem
(2) design lane load

(1)-1 Design truck (HS20-44)


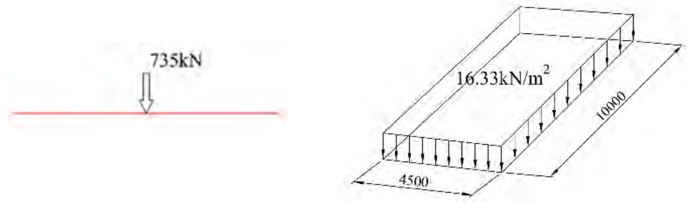
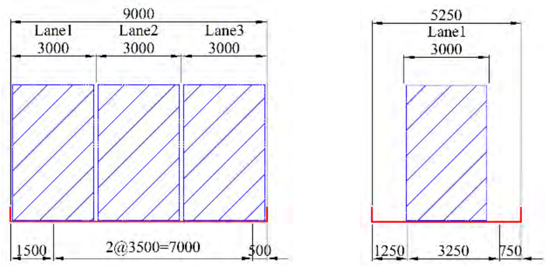
(1)-2 Design tandem


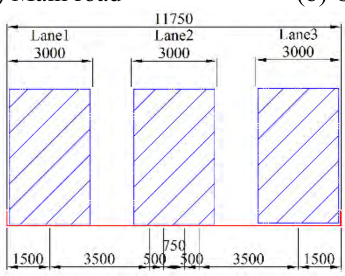
(2) Design lane load


(3) Two design trucks for negative moment

</td><td>AASHTO LRFD Bridge design specifications, 3.6.1

3.6.1.3

3.6.1.1</td></tr>
</table>

<table>
<tr><td></td><td>Types of combination 1) (1)-1 + (2) 2) (1)-2 + (2) 3) (3)×0.9 + (2)×0.9 Multiple presence factor **Table 3.6.1.1.2-1—Multiple Presence Factors, *m*** | Number of Loaded Lanes	Multiple Presence Factors, *m*
1	1.20
2	1.00
3	0.85
>3	0.65 | Nominal lane width shall be 3.0 m. 2. Special vehicular load (735kN concentrated load or equivalent distribution load) for main girder  (a) Concentrated load (b) Distribution load</td><td>MOC direction</td></tr>
<tr><td>Design lane</td><td>The width of the design lanes should be taken as 3.0m. The number of design lanes should be determined by taking the integer part of the ratio w/3.0, where w is the clear roadway width in feet between curbs and/or barriers.


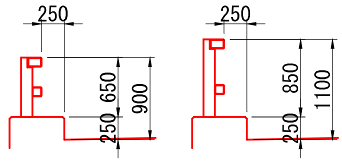
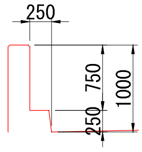
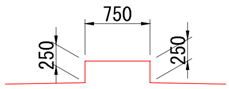
(a) Main road (b) Onramp


(c) Flyover (median is deemed as “clear roadway”)</td><td>AASHTO 3.6.1.1.1</td></tr>
<tr><td>Calculation method of inertia force</td><td>Calculation method of inertia force shall comply with JSHB.</td><td>JSHB V 6.3.2</td></tr>
<tr><td>Impact coefficient</td><td>Equivalent to L-load in JSHB.
Steel bridge $i = 20/(50+L)$
PC bridge $i = 10/(25+L)$
Impact coefficients of pylon and cable of the cable-stayed bridge are applied based on the result of experiments.</td><td>JSHB I 2.2.3</td></tr>
</table>

	pylon: i = 0.15, cable: i = 0.20	
Effect of temperature change	Reference temperature: 25 °C Main structure RC, PC: +10 °C to +40 °C (25 °C ± 15 °C), relative difference between members: 5 °C Steel: +10 °C to +40 °C (25 °C ± 15 °C), relative difference between members: 15 °C Bearings, expansion joints RC, PC: +5 °C to +45 °C (25 °C ± 20 °C) Steel: 0 °C to +50 °C (25 °C ± 25 °C)	
Effect on concrete	Prestressed force, Influence of creep and drying shrinkage shall be considered.	JSHB I 2.2.4, 2.2.5
Wind load	100 mph (44.7 m/s), Basic wind speed in Yangon City (This expression is “3-second gust wind speed”) $U_{10}=U_{max}/G=44.7/1.51=29.6$(m/s) -> <u>30.0 (m/s)</u> Here, U_{10}: 10-minutes average wind speed (m/s) U_{max}: 3-second gust wind speed (m/s) G: Gust factor $G=1+k(\sigma/U_{10})=1+3x(7.6/44.7)=1.51$ k: Peak factor, k=3 σ : Standard deviation of wind speed, $\sigma=7.6$	MOC instruction
Flowing water pressure	Flowing water pressure shall be considered.	JSHB I 2.2.7
Hydrodynamic pressure	Hydrodynamic pressure during earthquake shall be considered.	JSHB I 2.2.7
Collision force	Collision force by barge shall be considered.	
Effect of earthquake	Effect of earthquake shall be considered. $k_h = 0.30$ at project site, $k_{hgL0} = 0.24$	

Source: JICA Study Team

Table 4.1.6 Bridge Attachments

Item	Design Conditions	Remark
Railings	Bago River Bridge Steel railing Road side H = 1,100 mm Median side H = 900 mm Design force: more than 130 kJ (Class A) Shapes  Flyover Concrete barrier Roadside H = 1,000 mm Design force: more than 160 kJ (Class Sc) Median side H = 250 mm (raised median)   On-ramp Steel railing H = 900 (same as Bago River Bridge median)	
Noise barrier	Not considered	
Guard fence	Not considered	
Lighting	Considered	
Equipment	Bago River Bridge Water pipe (φ45 cm × 2 lanes) W = 6.0 kN/m 0.7 kN/m^2 for all width is considered as future installation plan Flyover and On-ramp bridge Not installed	YCDC water resources department
Inspection ladder	Bago River Bridge (steel girder) Installation of inspection ladder in steel box girder Flyover, On-ramp bridge, PC girder of Bago River Bridge Not installed	
Drainage	Steel catch pit (manufactured product) will collect surface water. Discharged water will be drained directly to the river where the drainage pipe is on the river, and will be gathered and drained to the channel where the drainage pipe is on land. Design rainfall intensity: 149 mm/h	
Pavement	Steel cable-stayed girder, steel box girder Polymer-modified asphalt pavement, t = 80 mm PC box girder, Flyover Normal asphalt, t = 80 mm	
Waterproofing layer	Install under pavement (liquid coating)	

Source: JICA Study Team

4.2 STUDY ON CABLE-STAYED BRIDGE

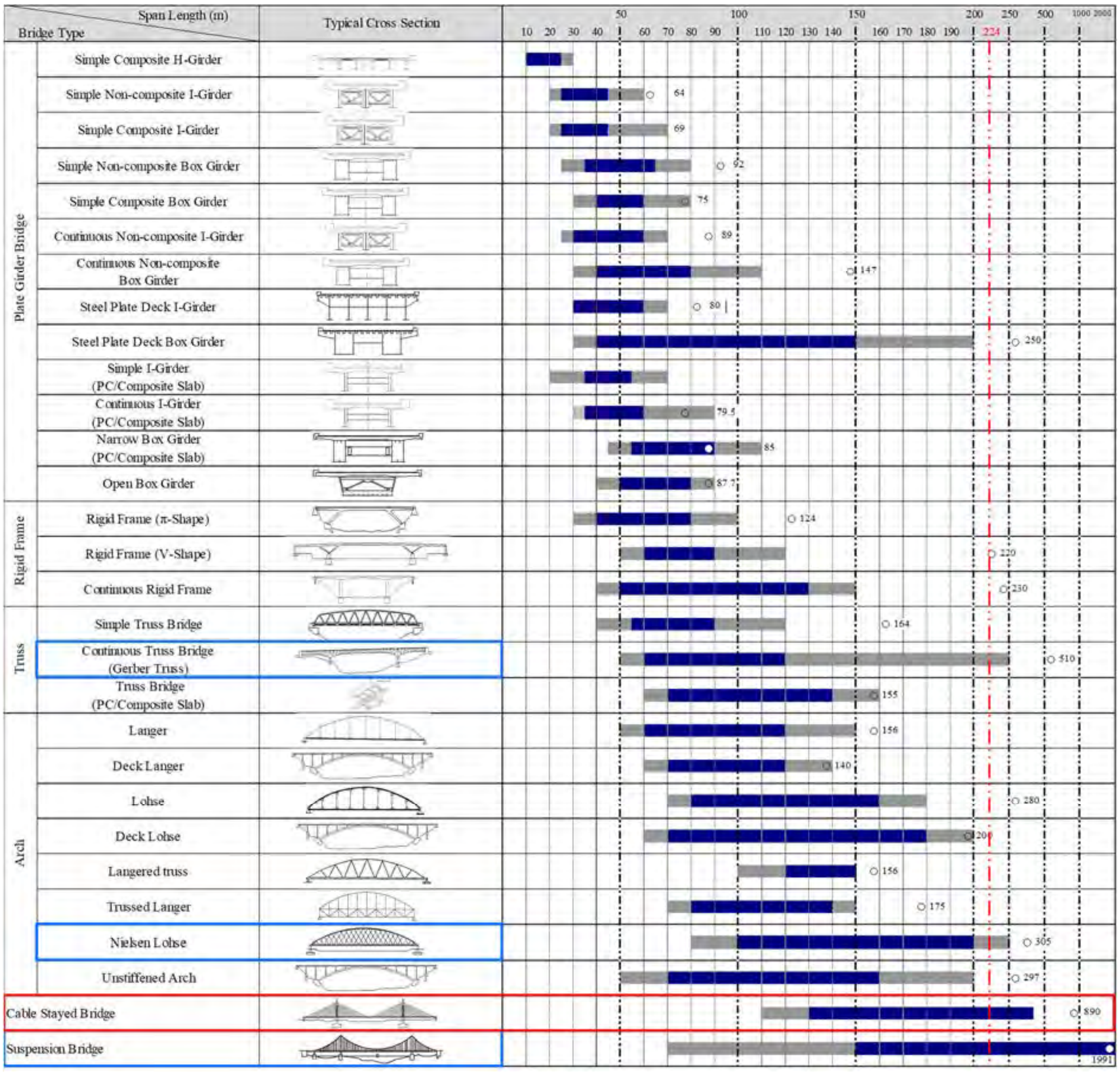
[Basic Design Stage]

4.2.1 Selection of Type of Cable-stayed Bridge

4.2.1.1 Review of the F/S Design

In the F/S, cable-stayed bridge was applied for the vessel operating route (span length = 224 m). The following table shows the applicable bridge types at each span.

Table 4.2.1 Applicable Span of Steel Bridge



: Ordinary Applicable Range : Applicable Range ○ : Maximum Span in Japan

Source: JICA Study Team

Based on the table, 1) Continuous Truss Bridge (Gerber Truss), 2) Nielsen Bridge, 3) Cable-stayed Bridge, and 4) Suspension Bridge can be applied for that span length. However, because of the following reasons, only the Cable-stayed Bridge can be applied in this Project:

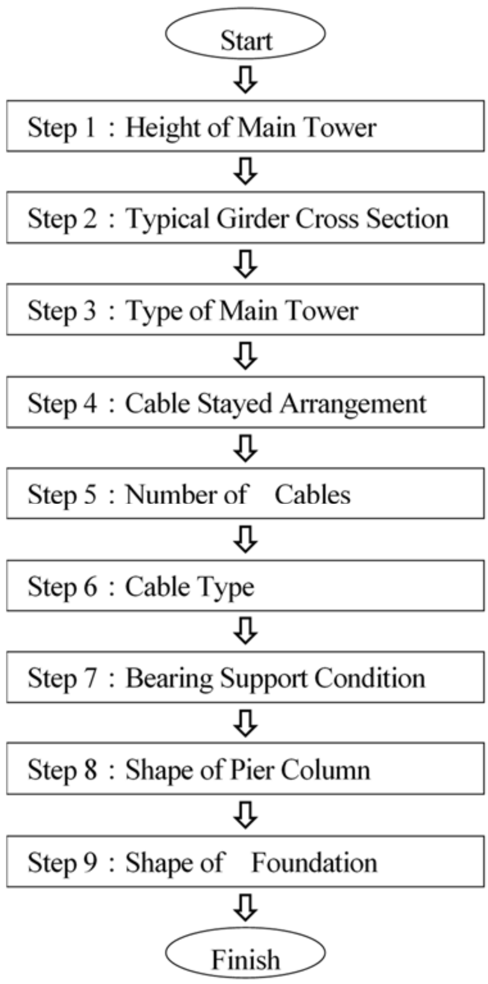
1) Continuous Truss Bridge (Gerber Truss): Usually, continuous truss bridge is applied for around 100 m span length. In case of more than 100 m span, Gerber Truss will be applied, but it is not good for maintenance and construction cost will become expensive.

2) Nielsen Bridge: In order to construct a Nielsen Bridge, cable construction method or large block erection method should be applied. However, both of the mentioned construction methods cannot be applied at the project site.

4) Suspension Bridge: Anchorage (anchor block for cable) is necessary for Suspension Bridge. However, there are no space available to construct the anchorage at the project site.

4.2.1.2 Flow Chart of Basic Design for Cable-stayed Bridge

In the B/D stage, the following items were considered and the best structure type was selected for each item:



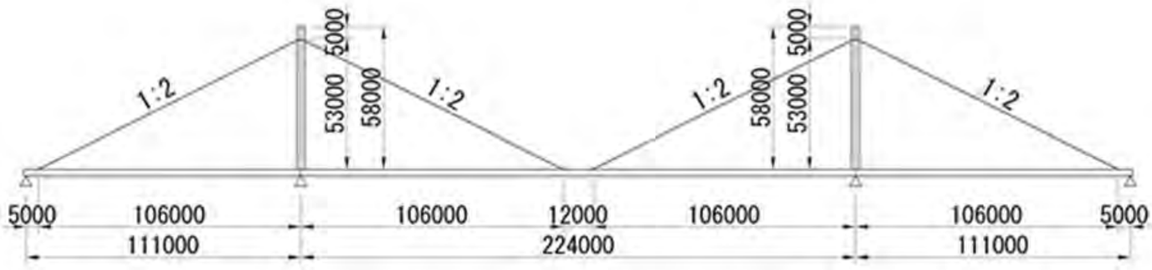
Source: JICA Study Team

Figure 4.2.1 Flow Chart of the Basic Design for Cable-stayed Bridge

4.2.2 Superstructure of Cable-stayed Bridge

4.2.2.1 Height of the Main Tower

Generally, the most economical gradient of the top cable of cable-stayed bridge is 1:2. In this Project, side span is 112 m (girder length :111 m) and top cable is fixed at 5 m from the end of the girder at the girder side. Therefore, considering the economical cable gradient (1:2), the height of the main tower is (111-5) / 2 = 53 m. Therefore, considering the work space at the top of the main tower for cable fixing, the total height of the main tower is decided as 53 + 5 = 58 m.



Source: JICA Study Team

Figure 4.2.2 Gradient of Top Cable

4.2.2.2 Typical Girder Cross Section

(1) Typical Girder Cross Section

For the typical girder cross section, three types of cross section (Wide Box Cross Section, Conventional Box Cross Section, Narrow Box Cross Section) were compared. Based on the comparison results, “Conventional Box Cross Section” was selected as the best type.

(2) Type of Rib for Slab

As for the type of rib for the steel deck slab, Flat Rib and U Rib can be applied. Based on the comparison results, U Rib was selected as the best type (as for the rib under median and barrier, plate rib will be applied).

(3) Height of Bracket

The height of the bracket at the cable-stayed bridge was changed from 1.2 m to 1.6 m, and the bracket weight was compared. Based on the comparison results, “Bracket Height = 1.3 m” was selected as the best type.

(4) Block Width

In order to transport the main girder from the factory to the project site, the main girder will be divided into blocks in the longitudinal direction and transverse direction.

The block width in the transverse direction was studied. Based on the comparison results, “Block Maximum Width = 3.06 m” was selected as the best type.

(5) Diaphragm Plate Thickness

The plate thickness of intermediate diaphragm was studied. As a result, “Diaphragm plate thickness = 9 mm” was enough for the outer cell and inner cell.

4.2.2.3 Types of Main Tower

(1) Comparison of Main Tower Types

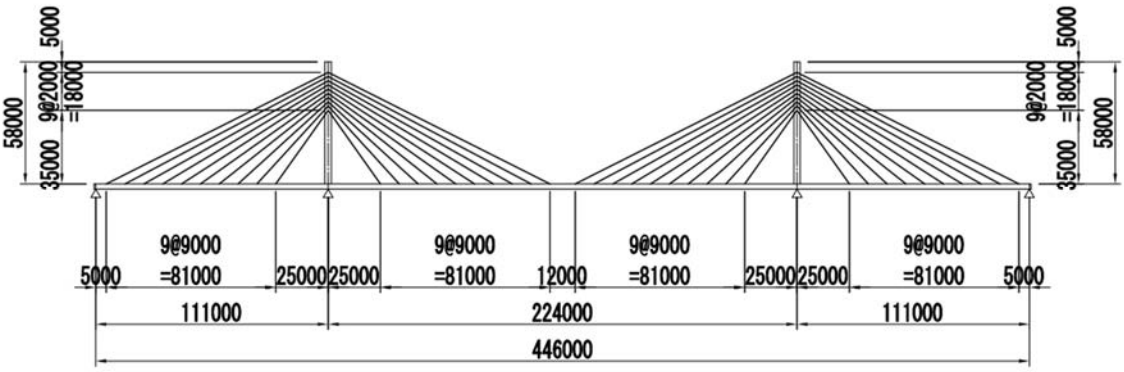
Three types of main tower (Single Tower, A-Shape Tower, Twin Tower) were compared. Based on the comparison results, “Single Tower” was selected as the best type.

(2) Pylon Width

The study of pylon width is based on the case of an anchor girder type as a typical anchor structure. The basic cross section to be studied is 2.5 m (width) x 3.0 m.

4.2.2.4 Cable-stayed Arrangement

Three types of cable-stayed arrangement (Harp Arrangement, Fan Arrangement, Semi Fan Arrangement) were compared. Based on the comparison results, <u>"Semi Fan Arrangement"</u> was selected as the best type.



Source: JICA Study Team

Figure 4.2.3 Semi Fan Arrangement

4.2.2.5 Number of Cables

Three types of the number of cables at the left (right) side of the pylon (11 Cables, 10 Cables, 9 Cables) were compared. Based on the comparison results, <u>"10 Cables (Total: 40 Cables)"</u> was selected as the best type.

4.2.2.6 Cable Type

(1) Comparison of Types of Cables

Three cable types (New Parallel Wire Strand Type, FUT-H Strand Type, Locked Coil Type) were compared. Based on the comparison results, <u>"FUT-H Strand Cable"</u> was selected as the best type.

(2) Anchor Study

1) Pylon Anchor Structure

Three types of pylon anchor structure (Anchor Girder, Anchor Plate, Saddle) were compared. Based on the comparison results, <u>" Anchor Girder"</u> was selected as the best type.

2) Girder Anchor Structure

Four types of girder anchor structure (Anchor Girder, Vertical Beam, Pipe Anchor, Vertical Girder) were compared. Based on the comparison results, <u>"Anchor Girder"</u> was selected as the best type.

4.2.2.7 Support Condition

Three types of support condition (M-F-M-M, M-F-F-M, E-E-E-E) were compared. Based on the comparison results, <u>"M-F-F-M Support"</u> was selected as the best type.

4.2.3 Substructure of Cable-stayed Bridge

4.2.3.1 Shape of Pier Column at P11 and P12

Three types of shape of pier column at the intermediate pier of cable-stayed bridge (P11, P12) (Round Shape, Oval Shape, Oval Shape without overhang section) were compared. Based on the comparison results, "Oval Shape" was selected as the best type.

4.2.3.2 Shape of Pier Column at P10 and P13

Three types of shape of pier column at the side pier of cable-stayed bridge (P10, P13) (Round Shape, Oval Shape, Oval Shape without overhang section) were compared. Based on the comparison results, "Oval Shape" was selected as the best type.

4.2.4 Foundation of Cable-stayed Bridge

4.2.4.1 Diameter of Steel Pipe Sheet Pile

(1) Study of Steel Pipe Sheet Pile Diameter (P11 and P12)

In the F/S stage, the diameter of the steel pipe sheet pile was planned to be 1000 mm. Here, a comparison of the diameters D = 1000 mm, 1200 mm and 1500 mm was conducted. Based on the comparison results, "φ1200 mm" was selected as the best type.

(2) Study of Steel Pipe Sheet Pile Diameter (P10 and P13)

Same as for P11 and P12, a comparison of the pile diameters at P10 and P13 was conducted. Based on the comparison results, "φ1200 mm" was selected as the best type.

4.2.4.2 Shape of Foundation at P11 and P12

In Section 4.2.4.1, pile diameter of φ1200 mm was selected. The shape of foundation was studied and three types (Rectangular Shape, Round Shape, Oval Shape) were compared. Based on the comparison results, "Oval Shape" was selected as the best type.

4.2.4.3 Shape of Foundation at P10 and P13

Same as for P11 and P12, comparison of the shape of foundation at P10 and P13 was conducted. Based on the comparison results, "Oval Shape" was selected as the best type.

4.2.5 Bridge Accessories

4.2.5.1 Bearing

(1) Edge Support Bearing

Structurally, since negative reaction forces normally act on the edge support points in cable-stayed bridges, a bearing structure which resist the reaction force is necessary. Also, since the edge support of the bridge needs a movable bearing due to temperature variation, a rocking bearing has generally been used for this occasion. The rocking bearing supports a vertical reaction force in both positive and negative directions, and is structured such that it can follow the movement of the girder in the longitudinal direction through a link structure provided above and below.

(2) Pylon Section Bearings

Applicable bearings under the pylon support point are shown below. The reaction forces in this point are big and the rotation movement should not be restricted, so pivot bearing was selected.

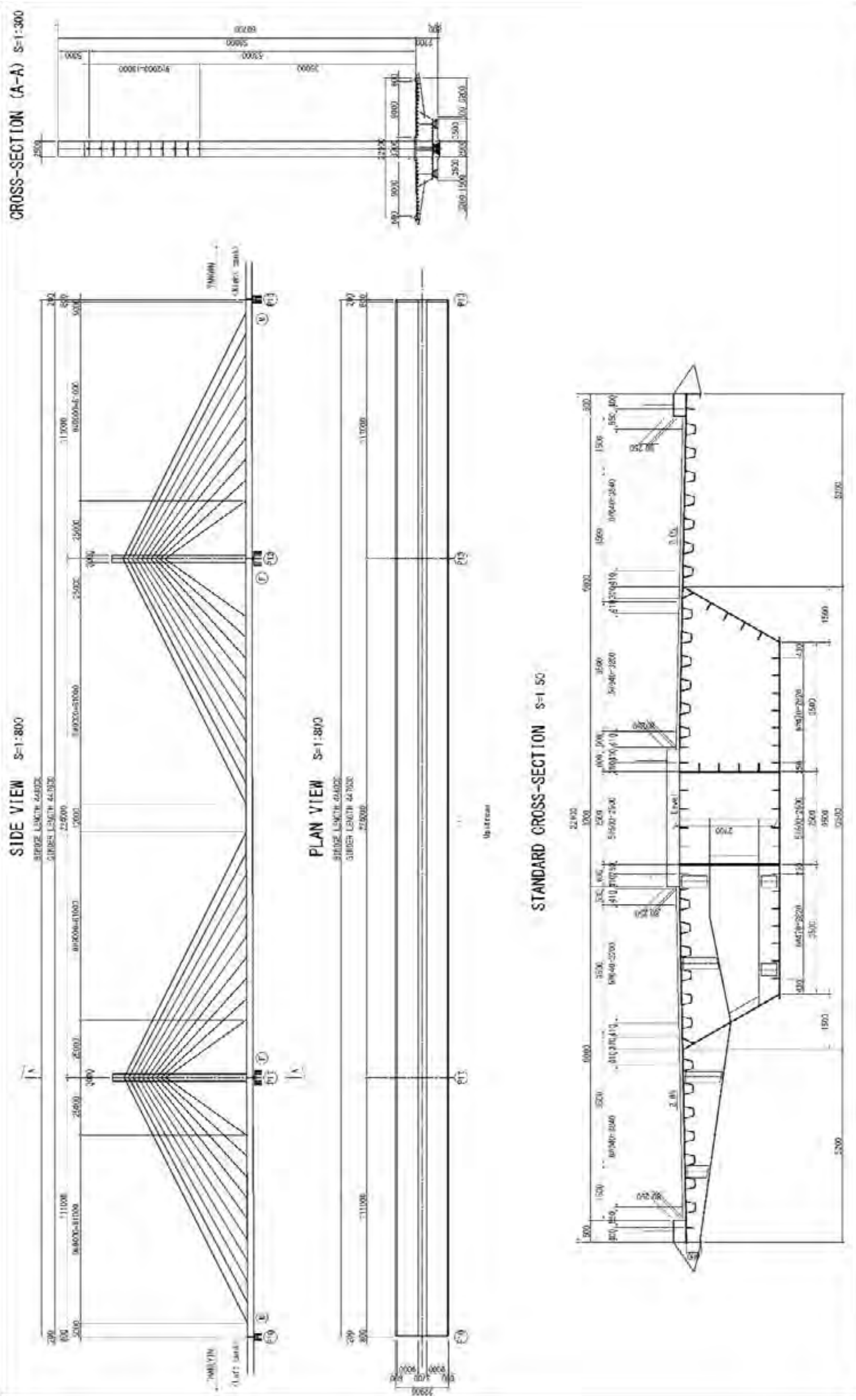
4.2.5.2 Expansion Joint

In long span bridges (similar to this bridge), expansion joints which can follow a big amount of expansion/contraction is necessary. The following types of expansion joints can be selected from a conventional construction record. In this bridge, based on the summary of expansion amount, Modular Expansion Joint was selected.

4.2.6 Basic Design Results

4.2.6.1 Superstructure Basic Design Results

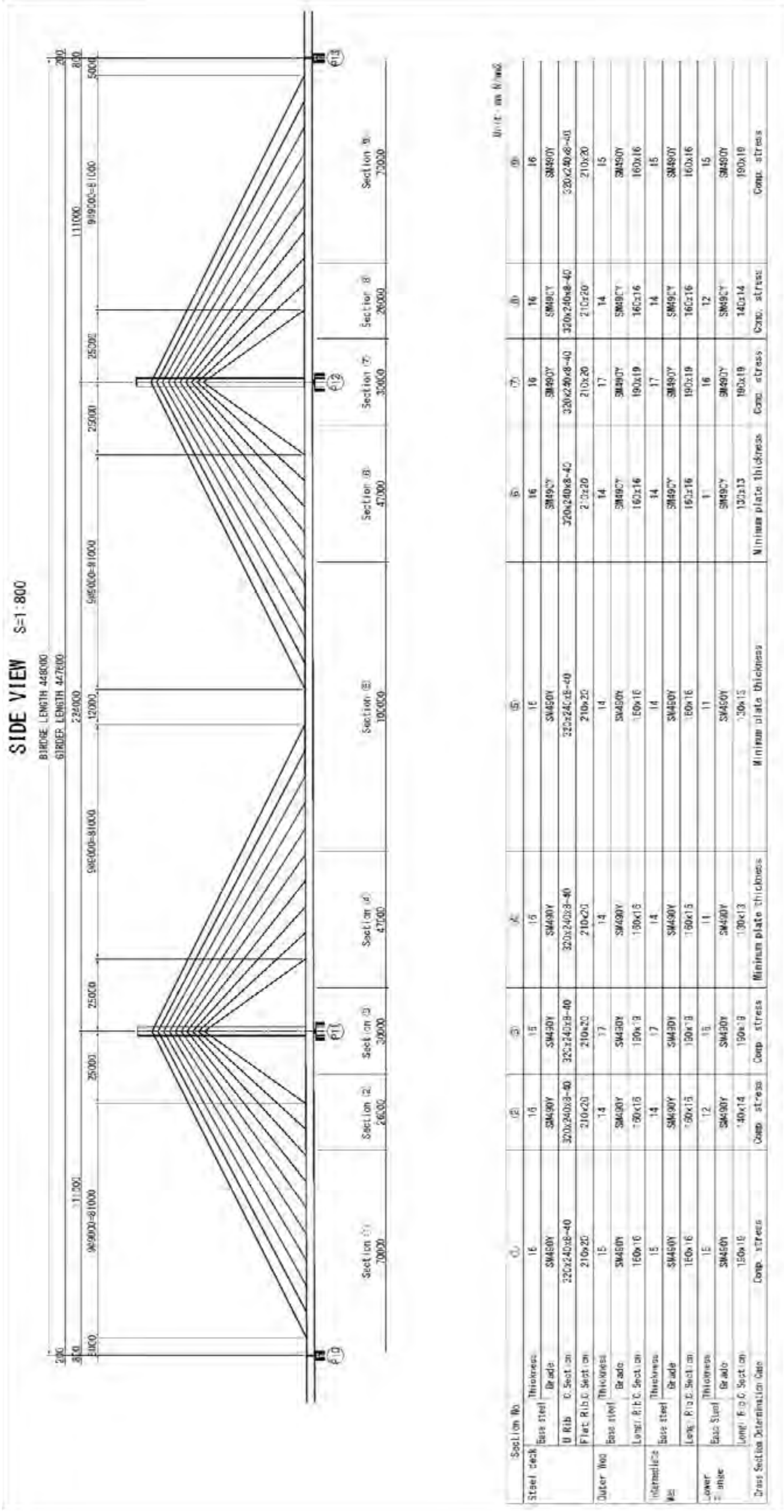
(1) Superstructure



Source: JICA Study Team

Figure 4.2.4 B/D Results for Superstructure

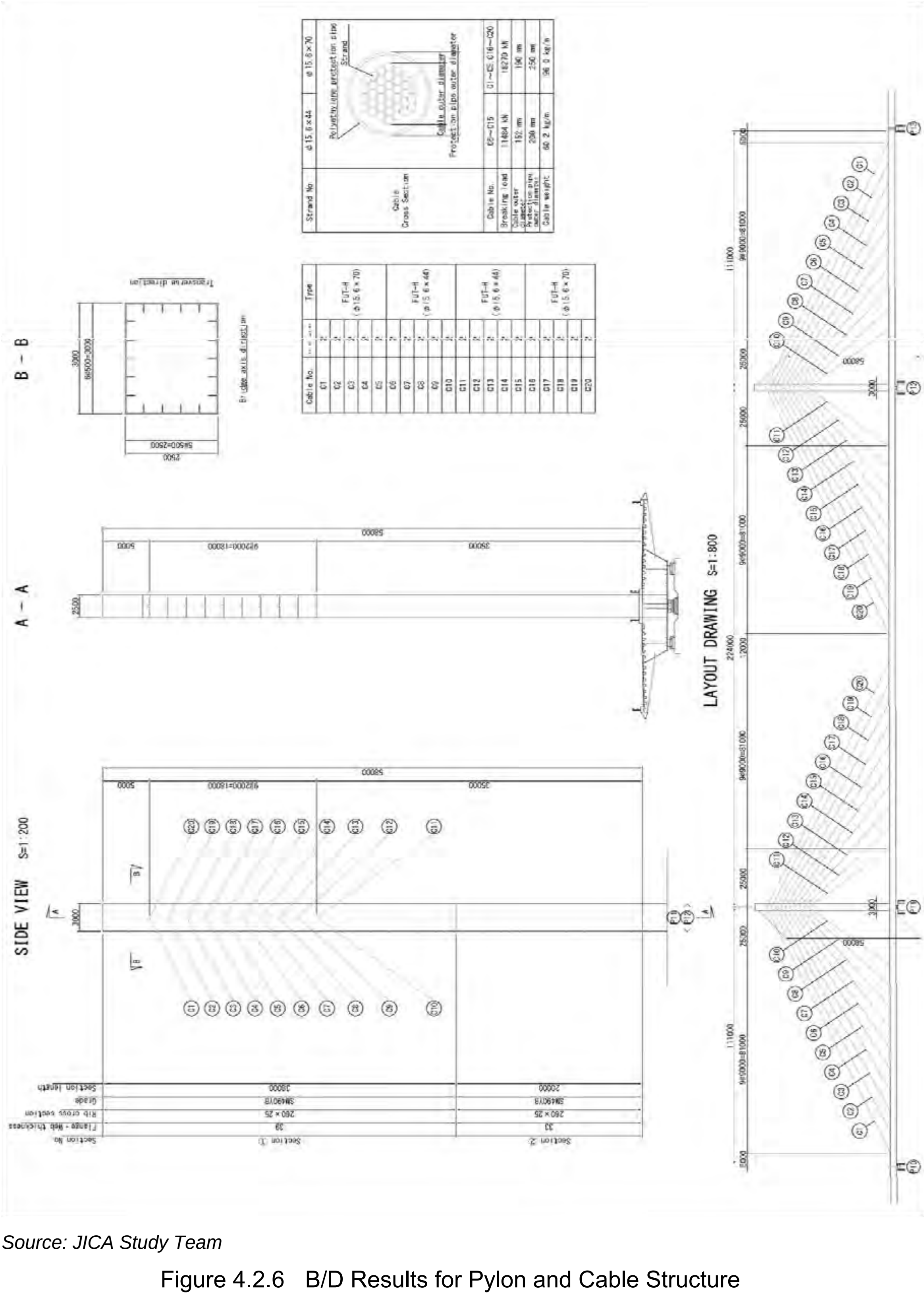
(2) Main Girder Cross Section



Source: JICA Study Team

Figure 4.2.5 B/D Results for Main Girder Cross Section

(3) Pylon and Cable Structure

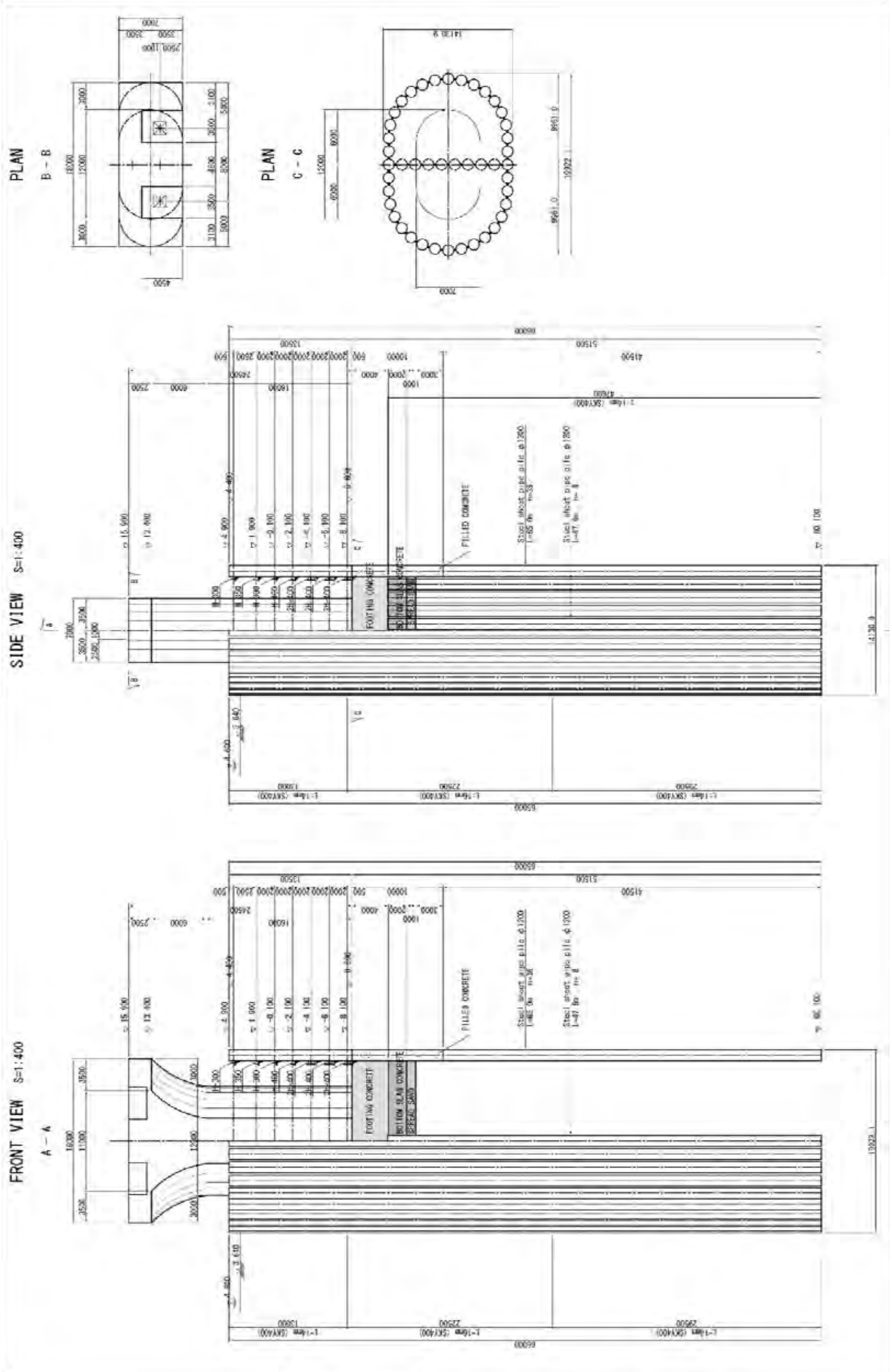


Source: JICA Study Team

Figure 4.2.6 B/D Results for Pylon and Cable Structure

4.2.6.2 Substructure Basic Design Results

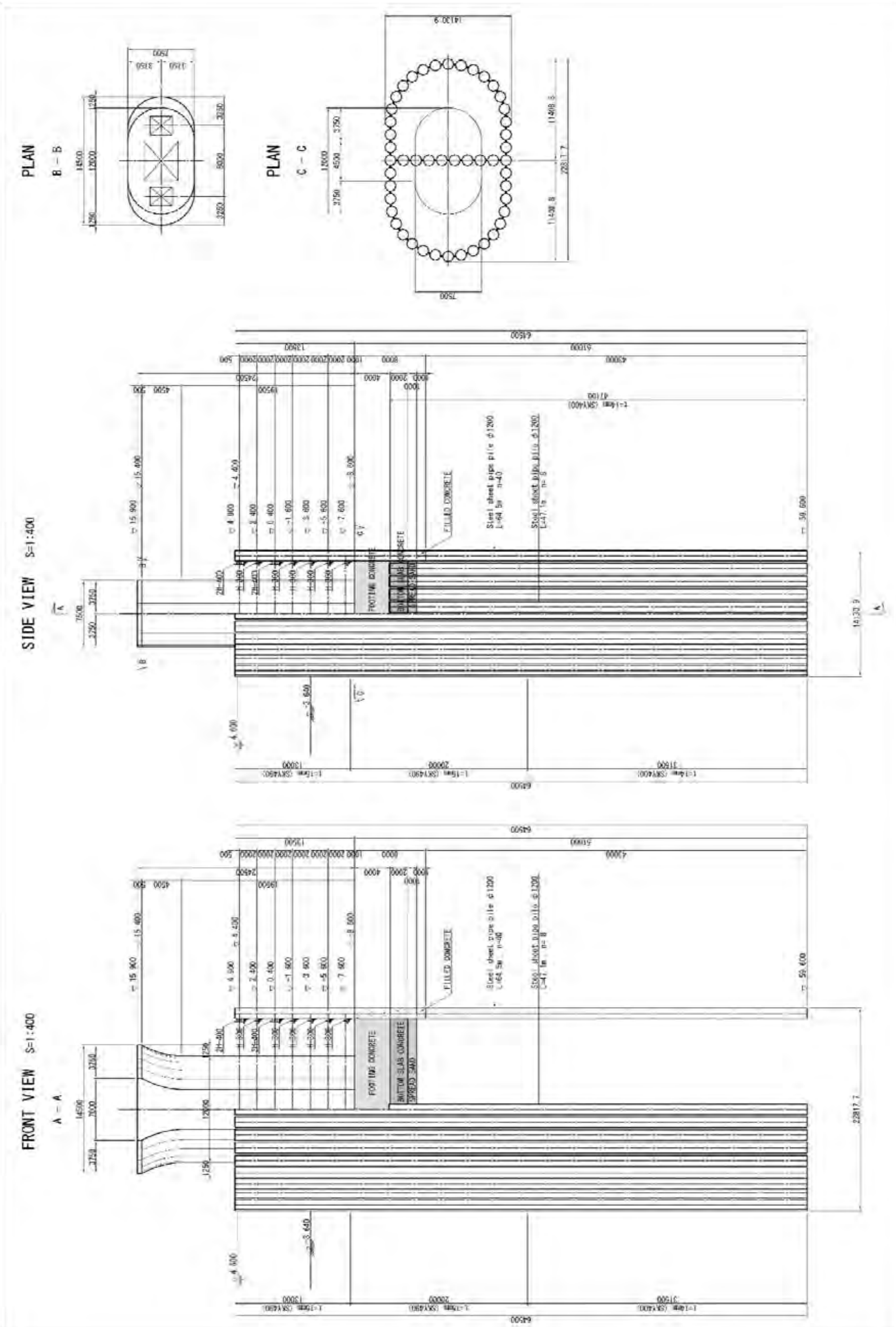
(1) P10 and P13



Source: JICA Study Team

Figure 4.2.7 B/D Results for Substructure of P10 and P13

(2) P11 and P12



Source: JICA Study Team

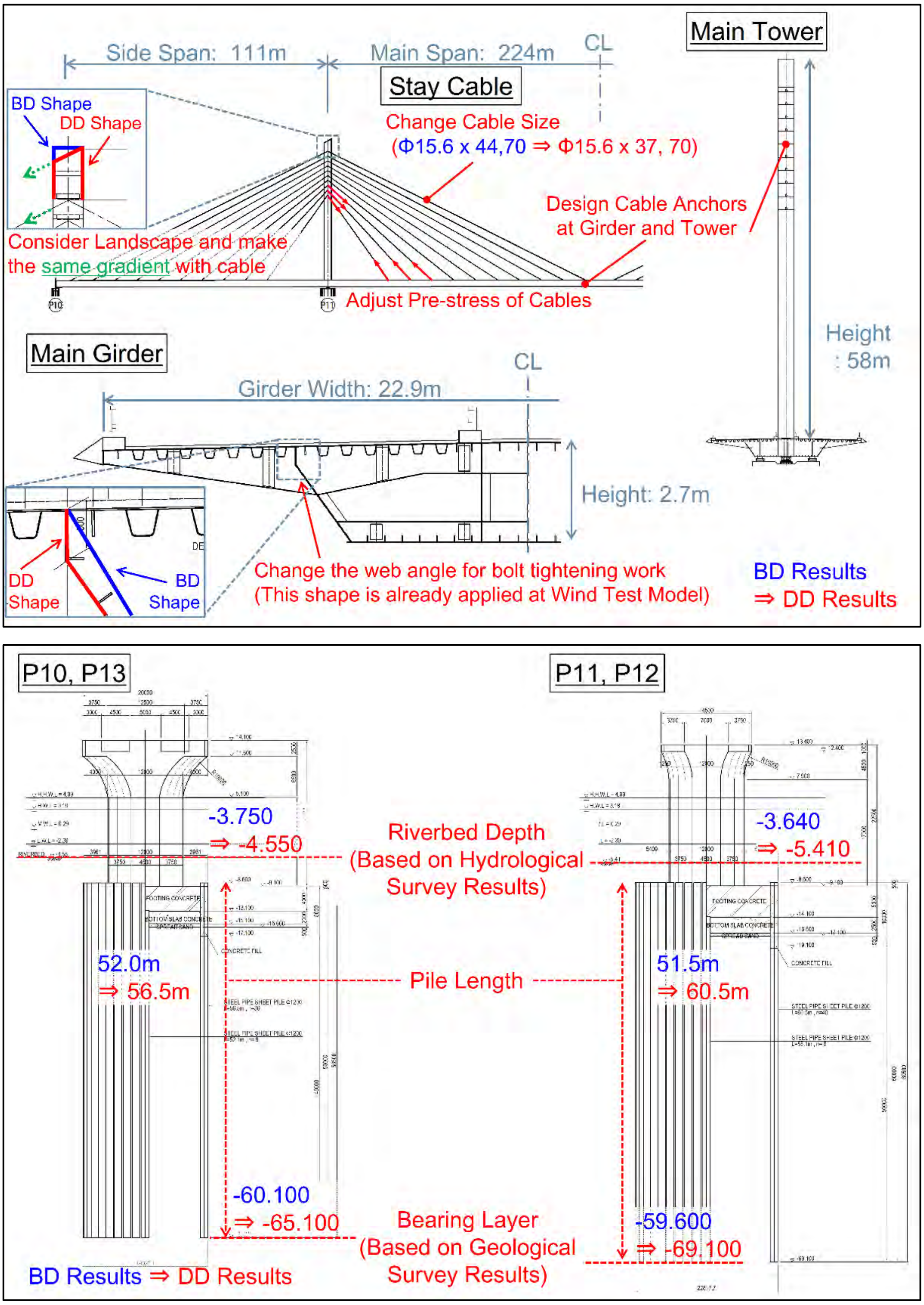
Figure 4.2.8 B/D Results for Substructure of P10 and P13

[Detailed Design Stage]

4.2.7 Summary of Detailed Design

4.2.7.1 Review of Design Conditions

Some design conditions were revised from the B/D to the D/D as shown in the figure below.

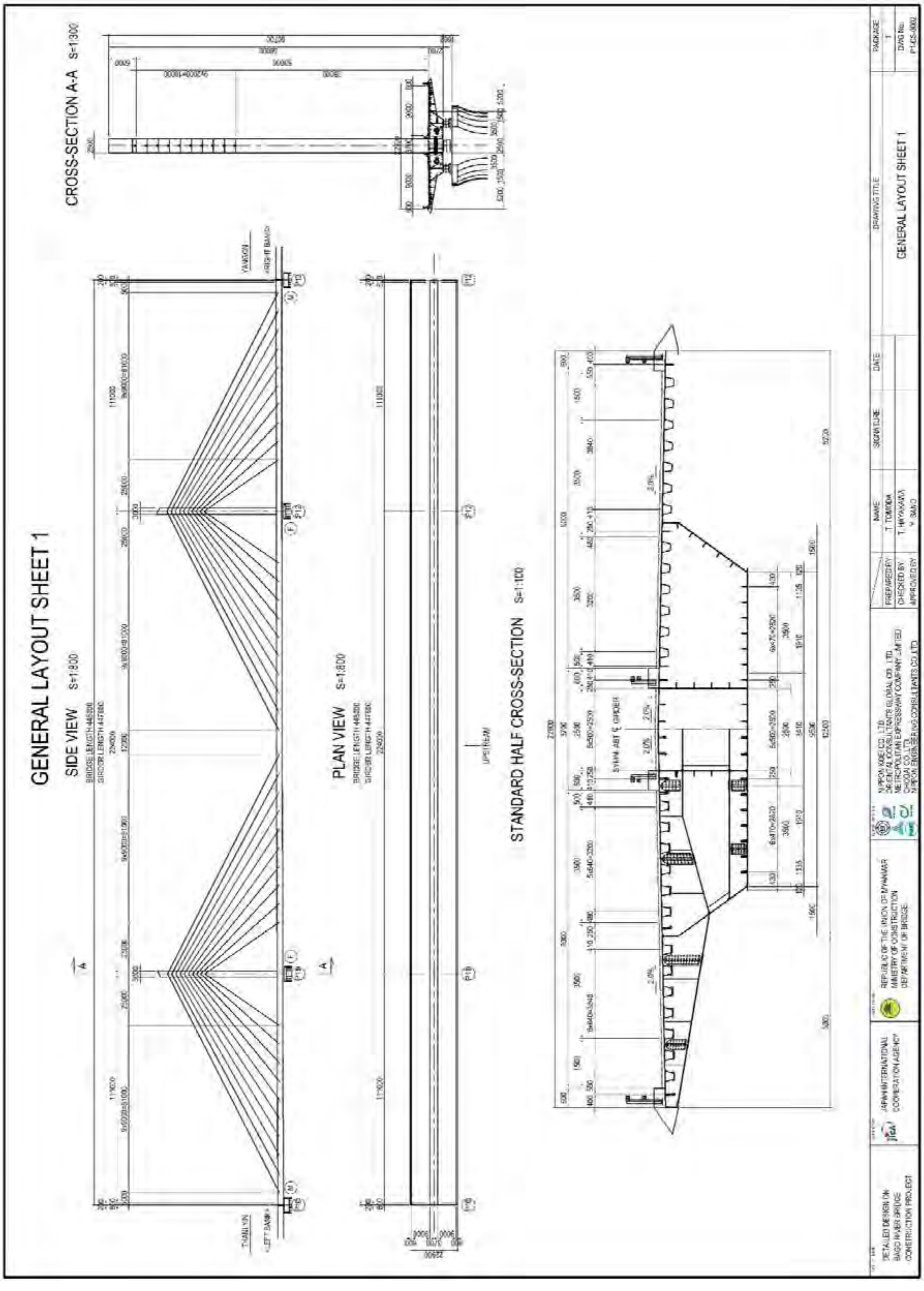


Source: JICA Study Team

Figure 4.2.9 Revised Design Conditions

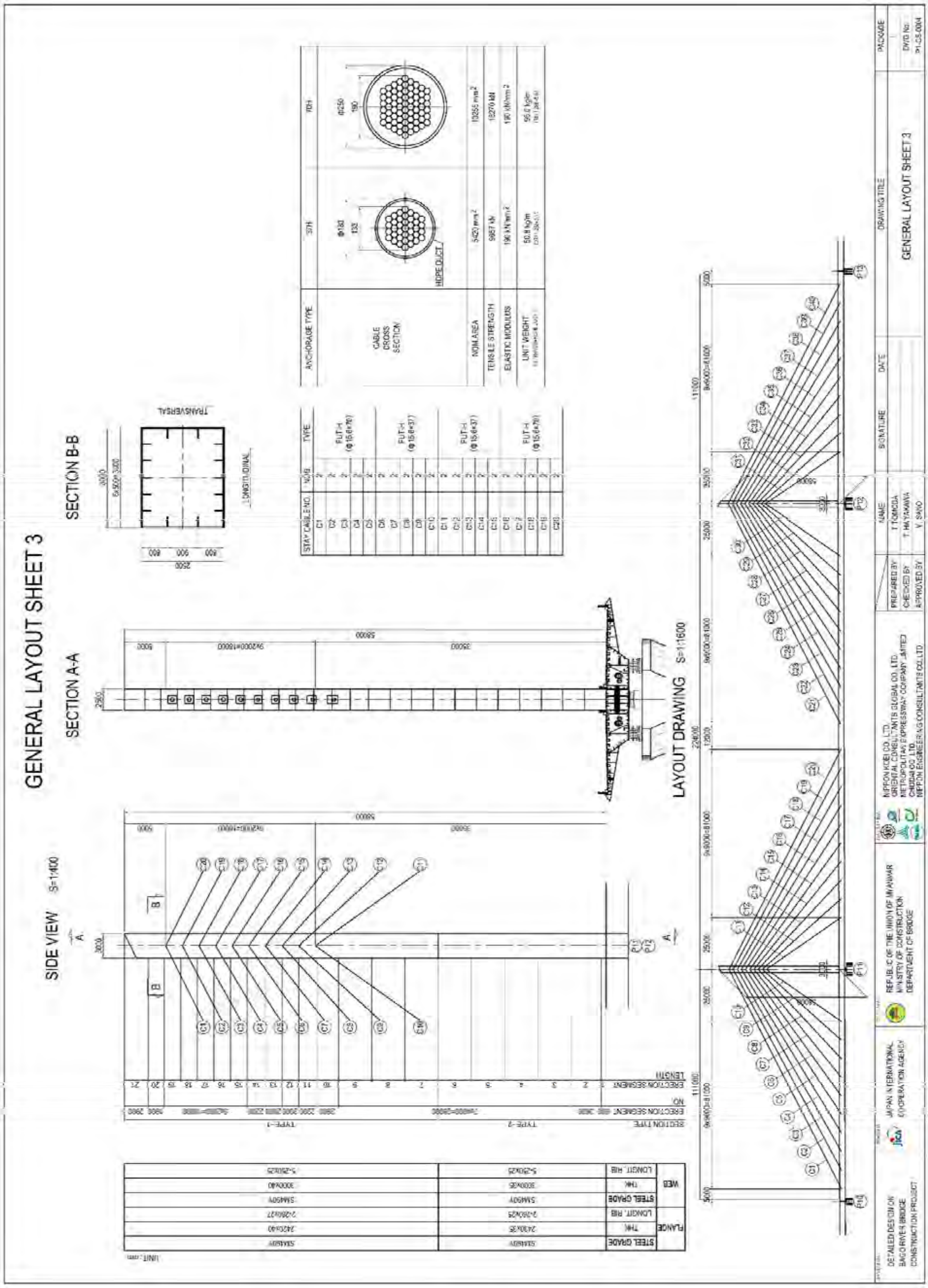
4.2.7.2 Detailed Design Results

The D/D results for the cable-stayed bridge are shown in the figure below.



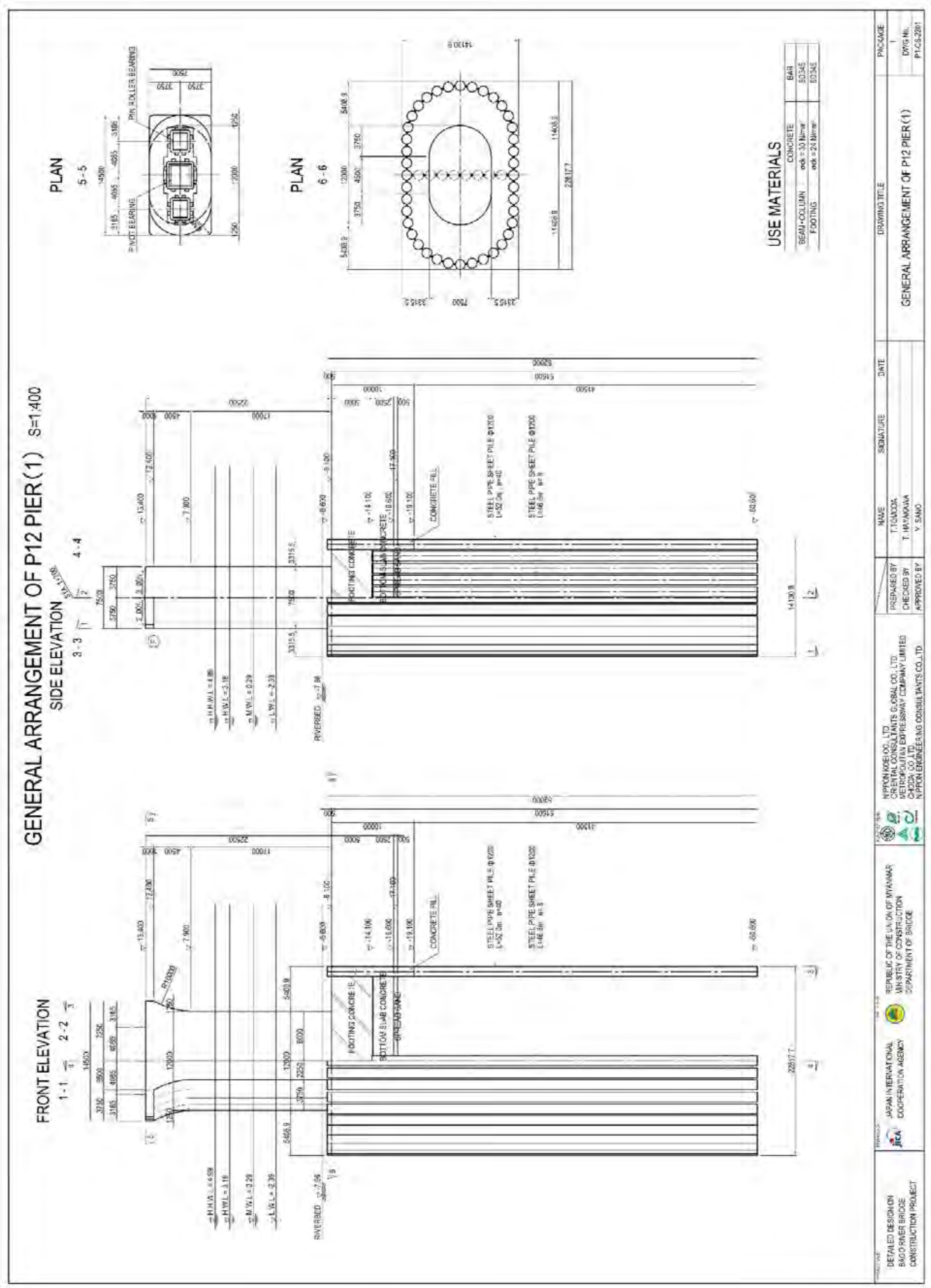
Source: JICA Study Team

Figure 4.2.10 Design Results for Cable-stayed Bridge (Superstructure: Girder)



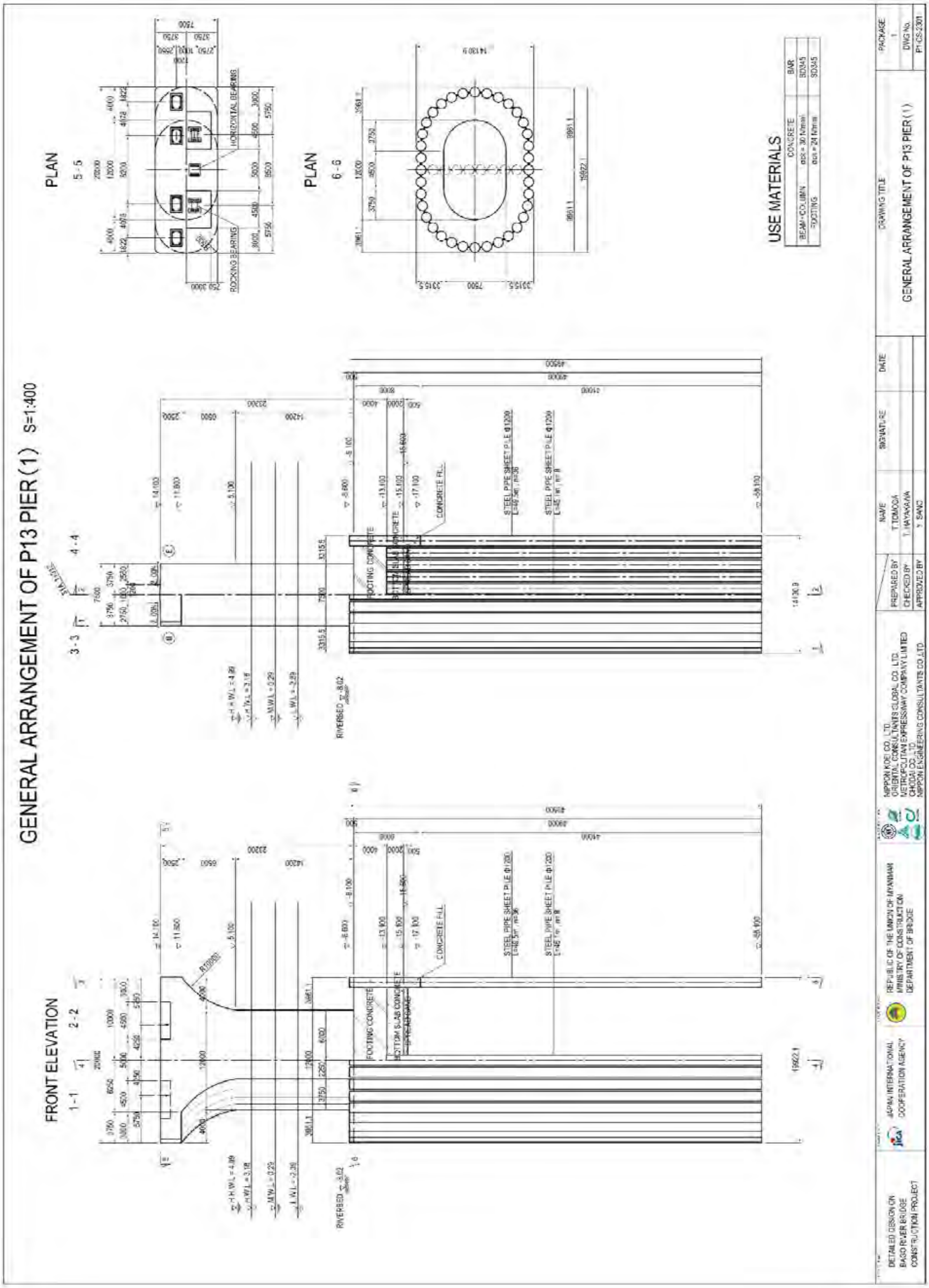
Source: JICA Study Team

Figure 4.2.11 Design Results for Cable-stayed Bridge (Superstructure: Tower and Cable)



Source: JICA Study Team

Figure 4.2.12 Design Results for Cable-stayed Bridge (Substructure: P11, P12)

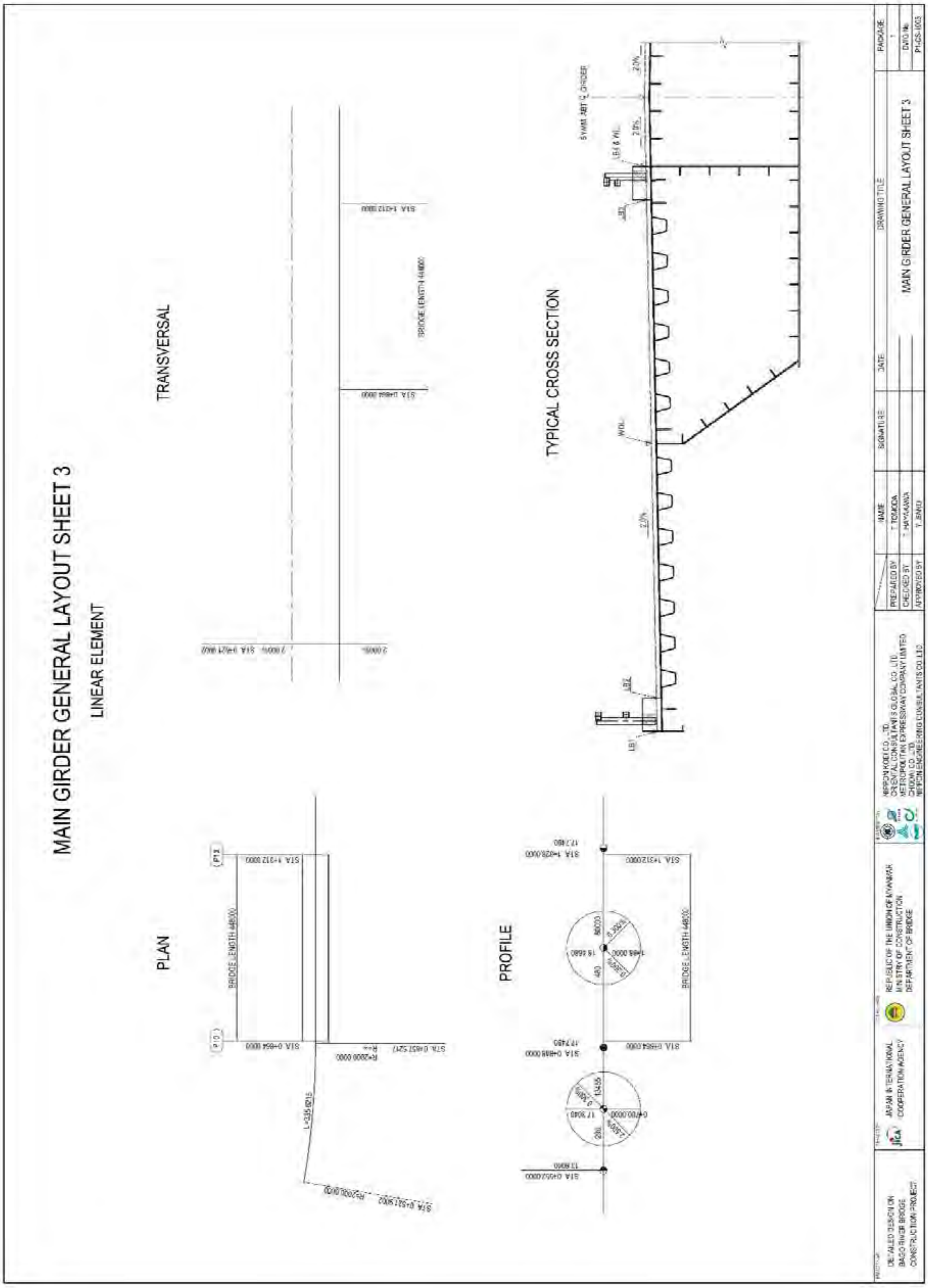


Source: JICA Study Team

Figure 4.2.13 Design Results for Cable-stayed Bridge (Substructure: P10, P13)

4.2.8 Alignment Calculation

The alignment of the cable-stayed bridge is as shown in the figure below.



Source: JICA Study Team

Figure 4.2.14 Alignment Information for Cable-stayed Bridge

4.2.9 Summary of Superstructure Design

4.2.9.1 Design Calculation of Steel Deck

- Design Results

The cross sections of the longitudinal rib, transverse rib, brackets, and vertical side girder were decided based on the maximum stress resultants of each member. (For reference, evaluation results based on the JSHB are shown in the following tables.)

Table 4.2.2 Design Results for Steel Deck (Transverse Rib)

Transverse Rib (Outer web - Inner Web)			AASHTO		JSHB
			Design Truck	Design Tandem	B Live Load
Section	Deck	Thickness: t	16	16	16
	Bottom flange	Width x t	240 x 10	240 x 10	240 x 10
	Web	Height x t	700 x 9	700 x 9	700 x 9
Material	Deck		SM400	SM400	SM400
	Bottom flange		SM490Y	SM490Y	SM490Y
	Web		SM490Y	SM490Y	SM490Y
Stress	Deck	Bending Stress	31	39	43
		Allowable Value	140	140	140
	Bottom flange	Bending Stress	-56	-71	-78
		Allowable Value	172	172	172
	Web	Shear Stress	26	34	49
		Allowable Value	120	120	120
	Composite	Composite Stress	0.12	0.19	0.30
	Defective Part	Vertical Shear	42	54	79
		Horizontal Shear	53	68	100
Deformation	Results	(mm)	0.78	1.02	1.2
	Allowable Value		10.0	10.0	10.0

Transverse Rib(Inner Web - Inner Web)			AASHTO		JSHB
			Design Truck	Design Tandem	B Live Load
Section	Deck	Thickness: t	16	16	16
	Bottom flange	Width x t	150 x 10	150 x 10	150 x 10
	Web	Height x t	350 x 9	350 x 9	350 x 9
Material	Deck		SM400	SM400	SM400
	Bottom flange		SM400	SM400	SM400
	Web		SM400	SM400	SM400
Stress	Deck	Bending Stress	3	3	8
		Allowable Value	140	140	140
	Bottom flange	Bending Stress	-9	-9	-22
		Allowable Value	131	131	131
	Web	Shear Stress	4	4	10
		Allowable Value	80	80	80
	Composite	Composite Stress	0.01	0.01	0.04
	Defective Part	Vertical Shear	11	11	27
		Horizontal Shear	4	4	11
Deformation	Results	(mm)	0	0	0
	Allowable Value		5.0	5.0	5.0

Source: JICA Study Team

Table 4.2.3 Design Results for Steel Deck (Bracket)

Bracket (at end)			AASHTO		JSHB
			Design Truck	Design Tandem	B Live Load
Section	Deck	Thickness: t	16	16	16
	Bottom flange	Width x t	370 x 15	370 x 15	370 x 15
	Web	Height x t	1300 x 10	1300 x 10	1300 x 10
Material	Deck		SM400	SM400	SM400
	Bottom flange		SM490Y	SM490Y	SM490Y
	Web		SM490Y	SM490Y	SM490Y
Stress	Deck	Bending Stress	37	43	51
		Allowable Value	140	140	140
	Bottom flange	Bending Stress	-109	-126	-149
		Allowable Value	160	160	160
	Web	Shear Stress	37	41	53
		Allowable Value	120	120	120
	Composite	Composite Stress	0.35	0.47	0.68
	Defective Part	Vertical Shear	46	52	66
		Horizontal Shear	75	84	106
Deformation	Results	(mm)	2.89	3.44	4.12
	Allowable Value		17.3	17.3	17.3

Bracket (at intermediate)			AASHTO		JSHB
			Design Truck	Design Tandem	B Live Load
Section	Deck	Thickness: t	16	16	16
	Bottom flange	Width x t	240 x 15	240 x 15	240 x 15
	Web	Height x t	1300 x 9	1300 x 9	1300 x 9
Material	Deck		SM400	SM400	SM400
	Bottom flange		SM490Y	SM490Y	SM490Y
	Web		SM490Y	SM490Y	SM490Y
Stress	Deck	Bending Stress	24	29	30
		Allowable Value	140	140	140
	Bottom flange	Bending Stress	-89	-105	-111
		Allowable Value	119	119	119
	Web	Shear Stress	28	34	41
		Allowable Value	120	120	120
	Composite	Composite Stress	0.23	0.32	0.38
	Defective Part	Vertical Shear	35	43	51
		Horizontal Shear	56	69	82
Deformation	Results	(mm)	2.44	2.96	3.07
	Allowable Value		17.3	17.3	17.3

Source: JICA Study Team

Table 4.2.4 Design Results for Steel Deck (Longitudinal Rib)

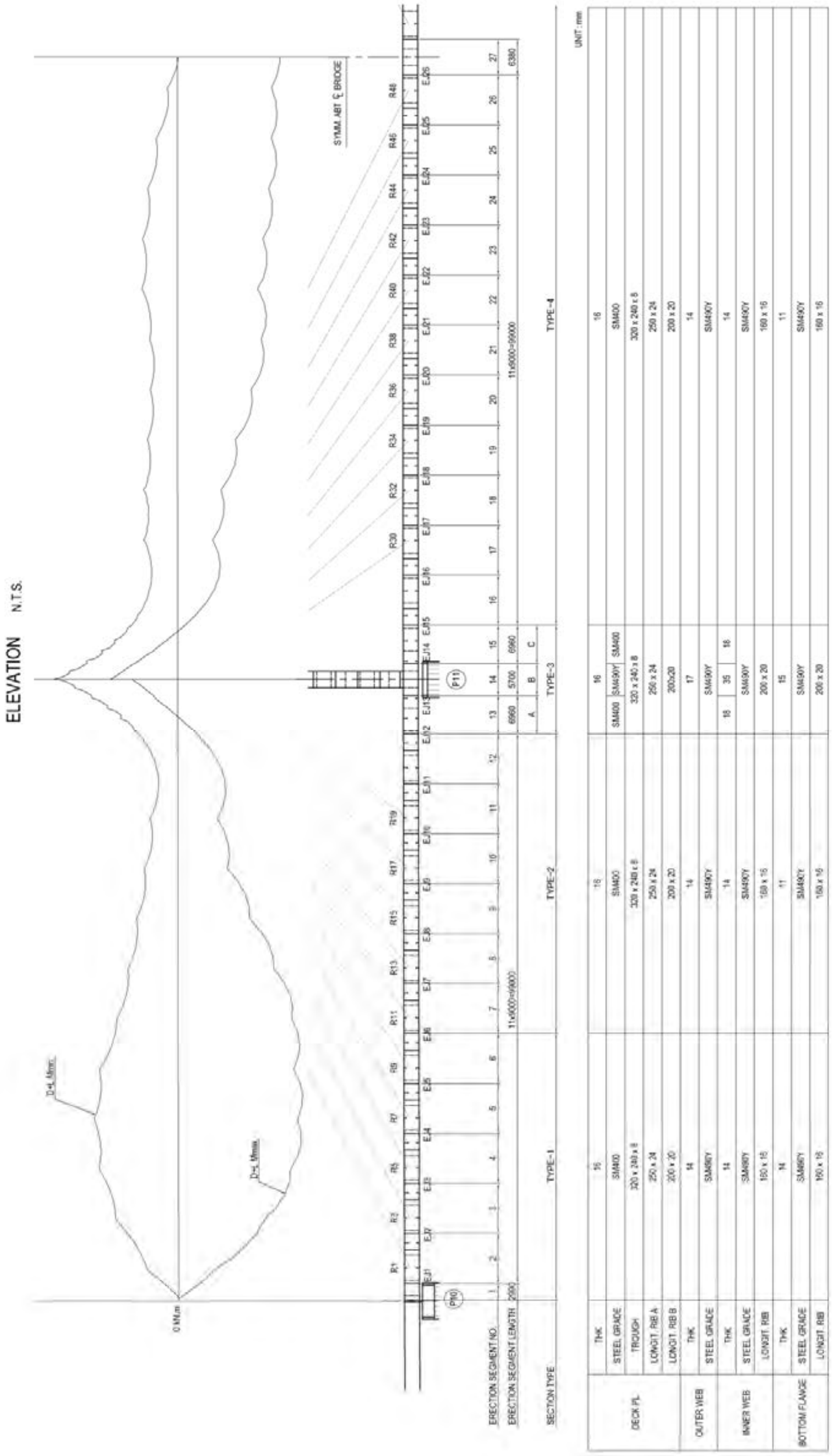
Longitudinal Rib			AASHTO		JSHB
			Design Truck	Design Tandem	B Live Load
Section	Deck	Thickness: t	16	16	16
	Longi. Rib	Shape	U-320x240x8	U-320x240x8	U-320x240x8
Material	Deck		SM400	SM400	SM400
	Longi. Rib		SM400	SM400	SM400
Stress	Deck	Bending Stress	-35	-32	-41
		Allowable Value	140	140	140
	Bottom Edge of Longi. Rib	Bending Stress	89	81	105
		Allowable Value	140	140	140
	Web of Longi. Rib	Bending Stress	12	11	16
		Allowable Value	80	80	80
	Composite	Composite Stress	0.43	0.35	0.60
Deformation	Results	(mm)	2.12	2.52	3.02
	Allowable Value		5.0	5.0	5.0

Source: JICA Study Team

4.2.9.2 Design Calculation for Main Girder

- Calculation Results for Cross Section of Main Girder

The calculation results for the cross section of the main girder are shown in the figure below.



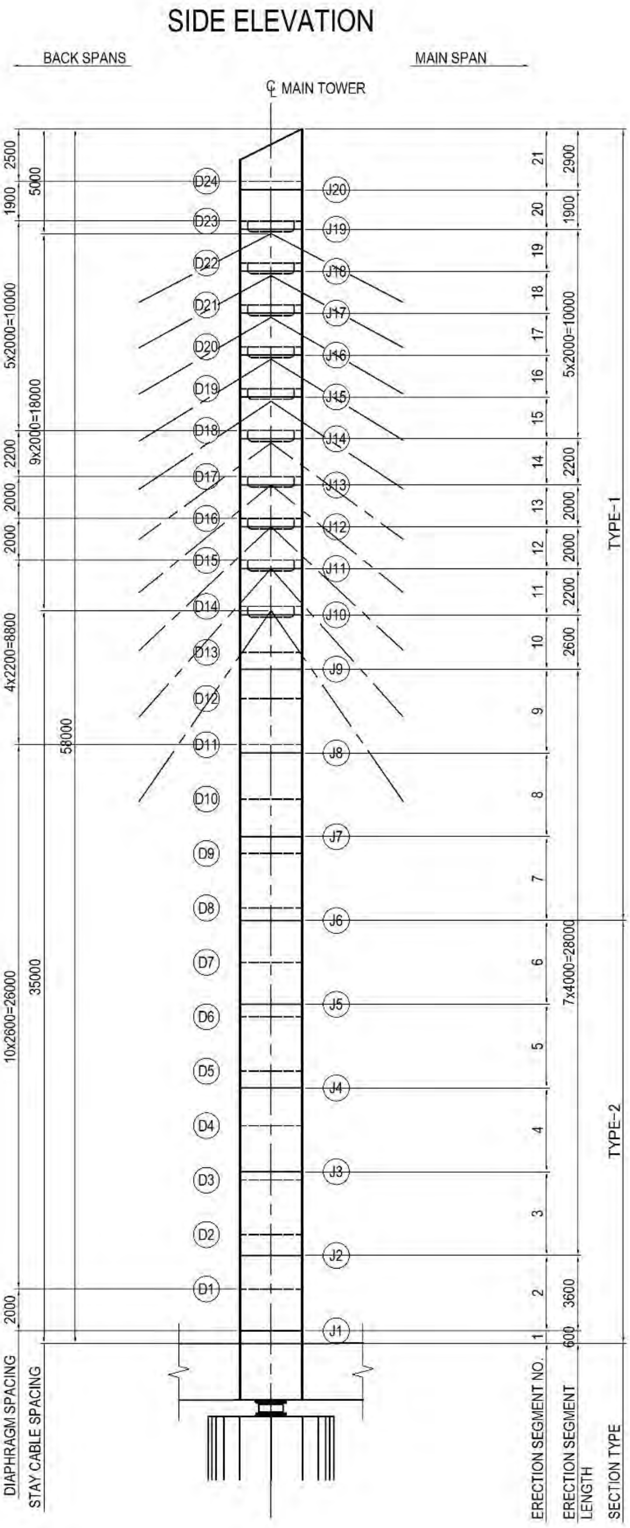
Source: JICA Study Team

Figure 4.2.15 Cross Sectional Diagram of Main Girder

4.2.9.3 Design Calculation for Main Tower

- Calculation Results for Cross Section of Main Tower

Calculation results for the cross section of the main tower are shown in the figure below.



UNIT : mm

		TYPE-2	TYPE-1
FLANGE	STEEL GRADE	SM490Y	SM490Y
	THK	2430x35	2420x40
	LONGIT. RIB	2-260x25	2-280x27
WEB	STEEL GRADE	SM490Y	SM490Y
	THK	3000x35	3000x40
	LONGIT. RIB	5-250x25	5-250x25

UNIT : kN・m, kN

SECTION TYPE		TYPE-2						TYPE-1													
SECTION		J1	J2	J3	J4	J5	J6	J7	J8	J9	J10	J11	J12	J13	J14	J15	J16	J17	J18	J19	J20
FLANGE	σ	-129.0	-127.3	-130.1	-135.7	-142.4	-149.7	-139.8	-146.7	-153.6	-158.5	-149.9	-138.7	-125.1	-109.7	-93.4	-70.5	-48.1	-27.7	-11.0	-0.2
	σa	210.0	204.7	204.7	204.7	204.7	204.7	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0
	LOAD CASE	D+ETR	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L
WEB	σ	-131.1	-127.3	-130.1	-135.7	-142.4	-149.7	-139.8	-146.7	-153.6	-158.5	-149.9	-138.7	-125.1	-109.7	-93.4	-70.5	-48.1	-27.7	-11.0	-0.2
	σa	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0	210.0
	LOAD CASE	D+ETR	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L

Source: JICA Study Team

Figure 4.2.16 Calculation Results for Cross Section of Main Tower

4.2.9.4 Design Calculation for Cable

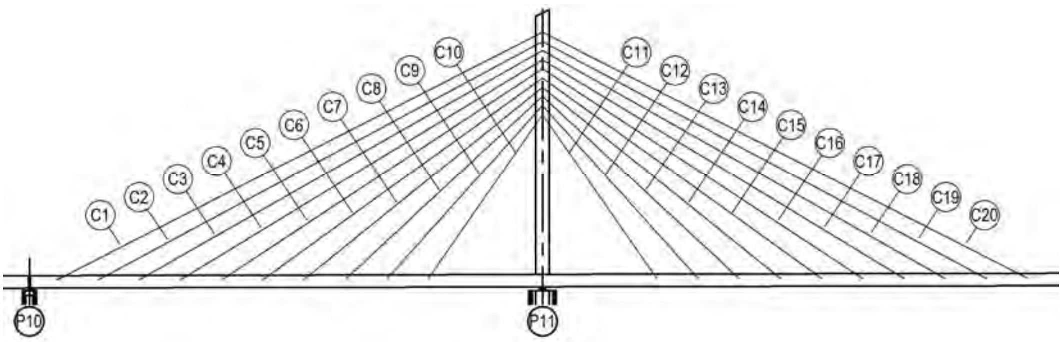
(1) Stay Cable

The calculated results of the cable tension and cross section of the stay cable are shown in the table below.

Table 4.2.5 Cable Tension and Cross Section

No.	Load	Tension (kN)	Cable Type
C1	Cable Tension max(all)	6616.6	φ15.6 * 70
C2	Cable Tension max(all)	5935.1	φ15.6 * 70
C3	Cable Tension max(all)	5322.2	φ15.6 * 70
C4	Cable Tension max(all)	5033.1	φ15.6 * 70
C5	Cable Tension max(all)	5291.6	φ15.6 * 70
C6	Cable Tension max(all)	3144.2	φ15.6 * 37
C7	Cable Tension max(all)	3457.4	φ15.6 * 37
C8	Cable Tension max(all)	3675.1	φ15.6 * 37
C9	Cable Tension max(all)	3752.1	φ15.6 * 37
C10	Cable Tension max(all)	3628.3	φ15.6 * 37
C20	Cable Tension max(all)	5622.5	φ15.6 * 70
C19	Cable Tension max(all)	5335.9	φ15.6 * 70
C18	Cable Tension max(all)	5150.0	φ15.6 * 70
C17	Cable Tension max(all)	5177.1	φ15.6 * 70
C16	Cable Tension max(all)	5488.0	φ15.6 * 70
C15	Cable Tension max(all)	3227.5	φ15.6 * 37
C14	Cable Tension max(all)	3521.6	φ15.6 * 37
C13	Cable Tension max(all)	3696.9	φ15.6 * 37
C12	Cable Tension max(all)	3738.0	φ15.6 * 37
C11	Cable Tension max(all)	3607.9	φ15.6 * 37

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.17 Cable Number

The safety ratio for the cable structure in a cable-stayed bridge is "2.5" in the JSHB. Evaluation result is as follows:

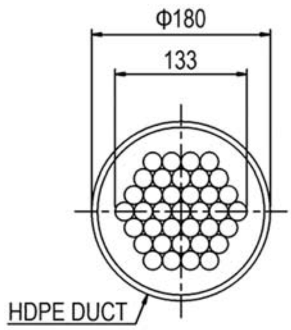
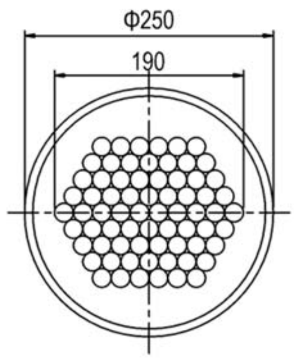
Table 4.2.6 Evaluation of Cable Tension

Cable No.	Max. Tension	Cable Strength	Safety Ratio
C1-C2, C16-C20 (70H)	6617 kN	18270 kN	2.76 > 2.5 (OK)
C6-C10, C11-C15 (37H)	3752 kN	9657 kN	2.57 > 2.5 (OK)

Source: JICA Study Team

The selected cable cross section is as follows:

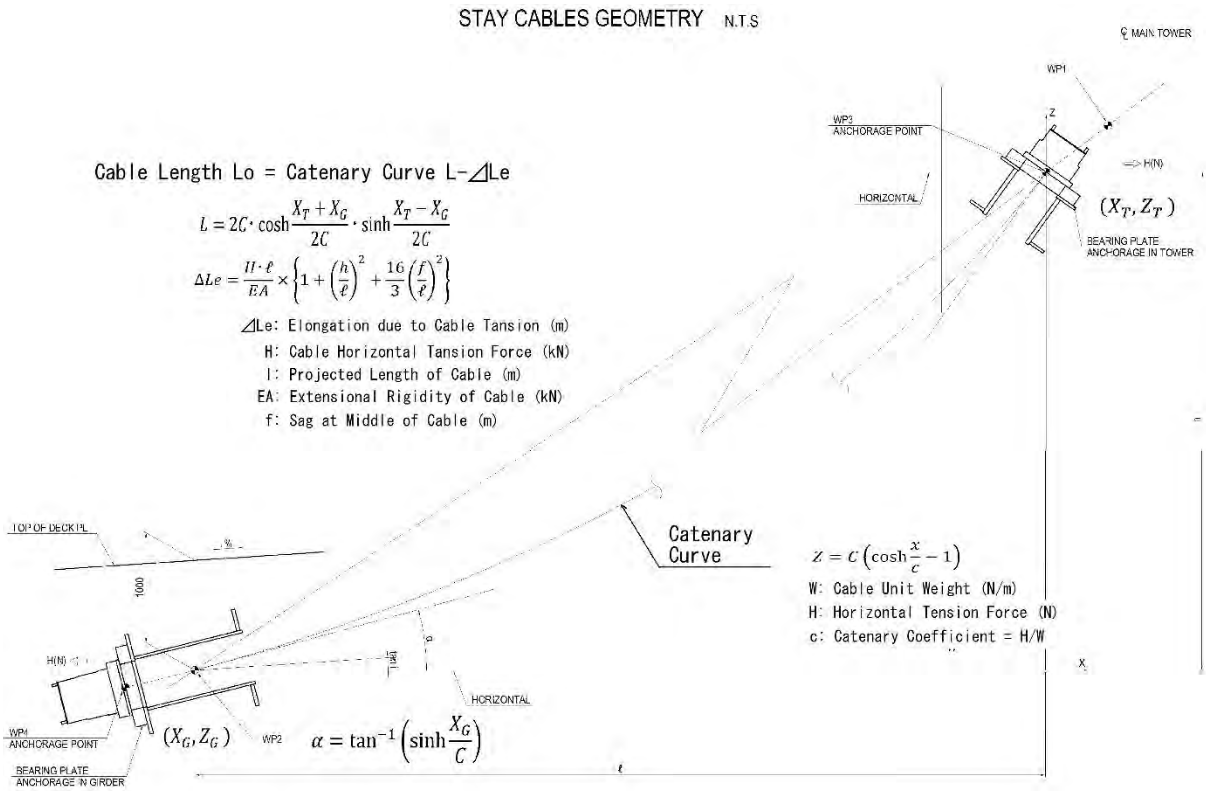
Table 4.2.7 Cross Section of Stay Cable

Items	37H	70H
Cable Cross Section		
Nominal Area	5420 mm^2	10255 mm^2
Tensile Strength	9657 kN	18270 kN
Elastic Modulus	190 kN/mm^2	190 kN/mm^2
Unit Weight (Strand + HDPE Coating)	50.8 kg/m	96.0 kg/m

Source: JICA Study Team

(2) Calculation of Stay Cable Length

The stay cable length is calculated by considering the “Catenary Curve”. The calculation method is shown in the figure below.



Source: JICA Study Team

Figure 4.2.18 Calculation Method of Catenary Curve

4.2.9.5 Study on Cable Pre-stressing Force

The study results are shown in the table below.

Table 4.2.8 Study Results for Cable Pre-stressing

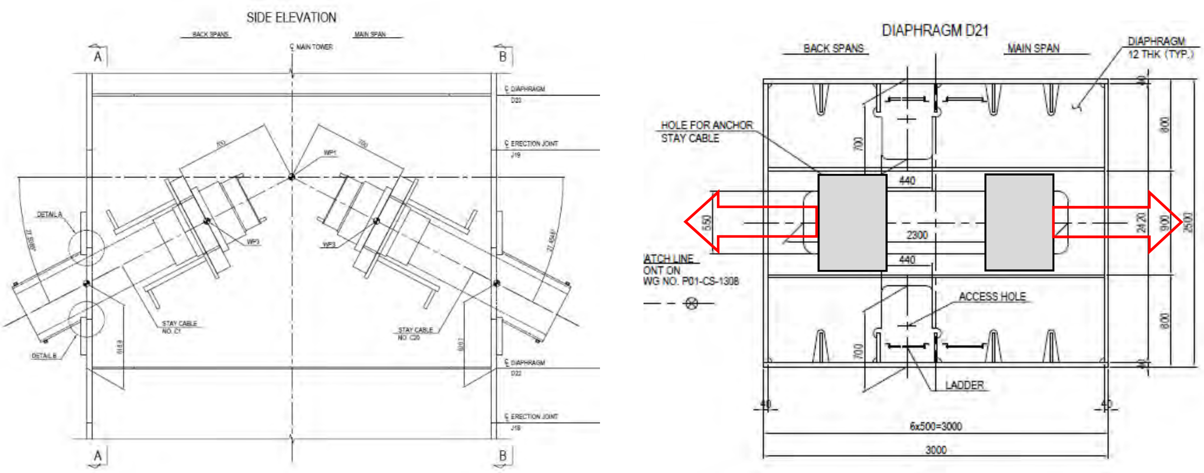
Section		Element	PS(kN)	Section		Element	PS(kN)	Section		Element	PS(kN)	Section		Element	PS(kN)
Side Span_P10	Upper	401	**720**	Main Span_P11	Upper	411	**1420**	Main Span_P12	Upper	421	**1420**	Side Span_P13	Upper	431	**720**
		402	**330**			412	**650**			422	**650**			432	**330**
		403	**0**			413	**20**			423	**20**			433	**0**
		404	**-170**			414	**-360**			424	**-360**			434	**-170**
		405	**-50**			415	**-400**			425	**-400**			435	**-50**
		406	**210**			416	**-20**			426	**-20**			436	**210**
		407	**470**			417	**220**			427	**220**			437	**470**
		408	**700**			418	**470**			428	**470**			438	**700**
		409	**1010**			419	**810**			429	**810**			439	**1010**
	Lower	410	**1450**		Lower	420	**1300**		Lower	430	**1300**		Lower	440	**1450**

Source: JICA Study Team

4.2.9.6 Study on Cable Anchorage Structure

(1) Anchor Structure on Main Tower

The Anchor Girder Structure, which transmits the differences of horizontal cable tensions and the vertical forces to the main tower from the anchor girder via a diaphragm and inner vertical plates, was selected.

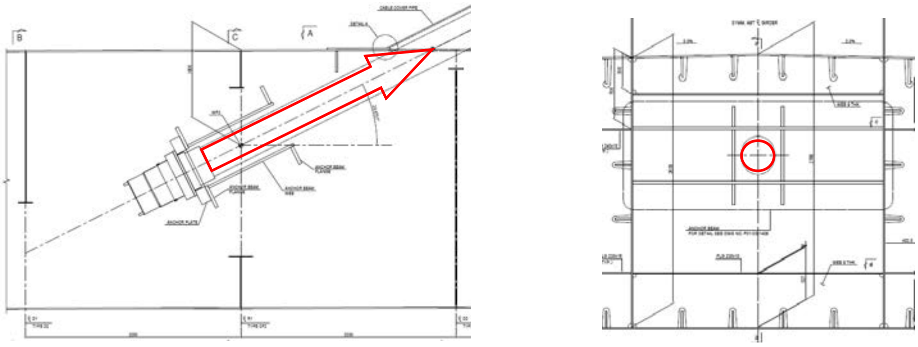


Source: JICA Study Team

Figure 4.2.19 Cable Anchor Structure on Main Tower Side

(2) Anchor Structure on Main Girder

The Anchor Girder Structure, which transmits the cable tension to the entire main girder via inner web, was selected. Although the Anchor Girder Structure tends to require a thicker web plate thickness, its physical characteristics are simple and clear.



Source: JICA Study Team

Figure 4.2.20 Cable Anchor Structure on Main Girder

(3) Evaluation Results for Web at the Main Girder and Tower

Evaluation results for the inner web where the additional stress is concerned are shown in the table below.

Table 4.2.9 Stress at Inner Web

Output Line (Inner Web)		Stress (N/mm^2)	Allowable Value (N/mm^2)	Results
Tower	A-A Line	50 ～ 160	210	OK
	B-B Line	50 ～ 150	210	OK
Girder	A-A Line	50 ～ 100	143	OK
	B-B Line	50 ～ 100	143	OK

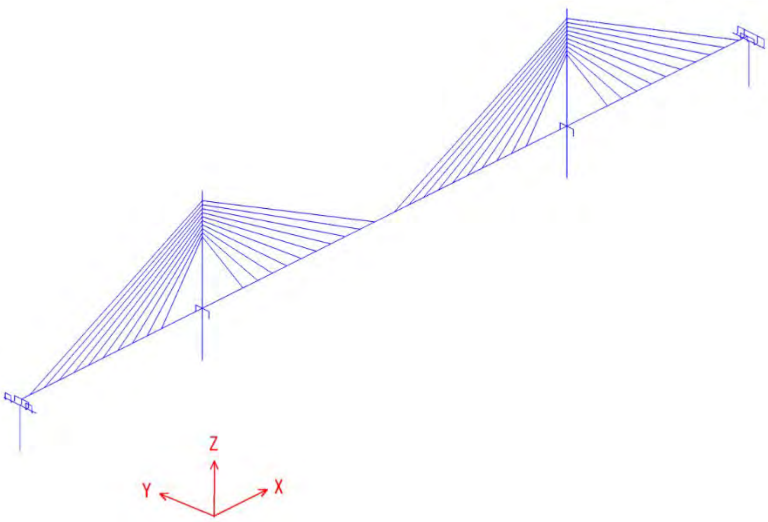
Source: JICA Study Team

In the design calculation, cross section of the web was decided by considering additional stress which was estimated by a simple calculation around the web. Furthermore, the safety performance of the web was confirmed through the FE analysis.

4.2.9.7 Static Structure Analysis

(1) Analysis Principle

The analysis model is shown in the figure below.



Source: JICA Study Team

Figure 4.2.21 Frame Analysis Model

(2) Loading Combinations

a) Design Section Force of Superstructure

Table 4.2.10 Loading Combination (Design Stress Resultants for Superstructure)

Case	Name	Increase Coefficient	Dead Load	PS	Live Load	Temperature	Wind		Seismic			
							Transverse		Longitudinal		Transverse	
							WTR ↑	WTR ↓	ELG→	ELG←	ETR ↑	ETR ↓
Dead Load	D[Db+Da+PS]: Dead Load+PS	1.00	○	○								
Normal	D+L	1.00	○	○	PICK UP							
Temperature	D+L+T	1.15	○	○	PICK UP	PICK UP						
Wind	D+WgTR ↑ (D+WtTR ↑)	1.25	○	○			○					
	D+WgTR ↓ (D+WtTR ↓)	1.25	○	○				○				
	D+L+WgTR ↑ (D+L+WtTR ↑)	1.25	○	○	PICK UP		○x0.5					
	D+L+WgTR ↓ (D+L+WtTR ↓)	1.25	○	○	PICK UP			○x0.5				
Wind + Temperature	D+WgTR ↑ +T (D+WtTR ↑ +T)	1.35	○	○		PICK UP	○					
	D+WgTR ↓ +T (D+WtTR ↓ +T)	1.35	○	○		PICK UP		○				
	D+L+WgTR ↑ +T (D+L+WtTR ↑ +T)	1.35	○	○	PICK UP	PICK UP	○x0.5					
	D+L+WgTR ↓ +T (D+L+WtTR ↓ +T)	1.35	○	○	PICK UP	PICK UP		○x0.5				
Seismic Performance Level 1	D+ELG→	1.50	○	○					○			
	D+ELG←	1.50	○	○						○		
	D+ETR ↑	1.50	○	○							○	
	D+ETR ↓	1.50	○	○								○

Source: JICA Study Team

b) Section Force for Bearing Supports

Table 4.2.11 Loading Combination (Design Stress Resultants for Bearing Support)

Case	Name	Dead Load	PS	Live Load	Temperature	Wind		Seismic			
						Transverse		Longitudinal		Transverse	
						WTR ↑	WTR ↓	ELG→	ELG←	ETR ↑	ETR ↓
Dead Load	D[Db+Da+PS]: Dead Load+PS	○	○								
Normal	D+L	○	○	PICK UP							
Temperature	D+L+T	○	○	PICK UP	PICK UP						
Wind	D+WgTR ↑ (D+WtTR ↑)	○	○			○					
	D+WgTR ↓ (D+WtTR ↓)	○	○				○				
	D+L+WgTR ↑ (D+L+WtTR ↑)	○	○	PICK UP		○x0.5					
	D+L+WgTR ↓ (D+L+WtTR ↓)	○	○	PICK UP			○x0.5				
Wind + Temperature	D+WgTR ↑ +T (D+WtTR ↑ +T)	○	○		PICK UP	○					
	D+WgTR ↓ +T (D+WtTR ↓ +T)	○	○		PICK UP		○				
	D+L+WgTR ↑ +T (D+L+WtTR ↑ +T)	○	○	PICK UP	PICK UP	○x0.5					
	D+L+WgTR ↓ +T (D+L+WtTR ↓ +T)	○	○	PICK UP	PICK UP		○x0.5				
Seismic Performance Level 1	D+ELG→	○	○					○			
	D+ELG←	○	○						○		
	D+ETR ↑	○	○							○	
	D+ETR ↓	○	○								○
Seismic Performance Level 2	D+SELG→	○	○					○x1.5			
	D+SELG←	○	○						○x1.5		
	D+SETR ↑	○	○							○x1.5	
	D+SETR ↓	○	○								○x1.5

Source: JICA Study Team

(3) Analysis Results

The analysis results are as follows:

Table 4.2.12 Section Force of Cables

Elem	Load	Force (kN)	Elem	Load	Force (kN)
401	D+L(max)	6616.58	421	D+L(max)	5622.46
402	D+L(max)	5935.07	422	D+L(max)	5335.81
403	D+L(max)	5322.17	423	D+L(max)	5149.99
404	D+L(max)	5033.07	424	D+L(max)	5177.10
405	D+L(max)	5291.65	425	D+L(max)	5488.03
406	D+L(max)	3144.18	426	D+L(max)	3227.47
407	D+L(max)	3457.40	427	D+L(max)	3521.63
408	D+L(max)	3675.10	428	D+L(max)	3696.88
409	D+L(max)	3752.13	429	D+L(max)	3738.00
410	D+L(max)	3628.30	430	D+L(max)	3607.91
411	D+L(max)	5622.50	431	D+L(max)	6616.53
412	D+L(max)	5335.85	432	D+L(max)	5935.03
413	D+L(max)	5150.02	433	D+L(max)	5322.14
414	D+L(max)	5177.12	434	D+L(max)	5033.05
415	D+L(max)	5488.04	435	D+L(max)	5291.63
416	D+L(max)	3227.47	436	D+L(max)	3144.17
417	D+L(max)	3521.63	437	D+L(max)	3457.40
418	D+L(max)	3696.88	438	D+L(max)	3675.10
419	D+L(max)	3738.00	439	D+L(max)	3752.14
420	D+L(max)	3607.90	440	D+L(max)	3628.30

Source: JICA Study Team

4.2.9.8 Fatigue Design

(1) Results of Fatigue Evaluation

The example of results of the fatigue evaluation is shown below.

Table 4.2.13 Example of Results of Fatigue Evaluation (1)

Position	No.	Point-1		Point-2		Point-3		Point-4	
		Grade D		Grade E		Grade E		Grade D	
		a)	b)	a)	b)	a)	b)	a)	b)
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j
Near P10	2	-----	-------	*****	*******	-----	-------	-----	-------
	3	0<85	-------	0<65	-------	*****	*******	0<88	-------
	4	-----	-------	*****	*******	0<81	-------	0<109	-------
Middle od Side Span	24	1<109	-------	*****	*******	1<81	-------	1<109	-------
	25	1<109	-------	1<81	-------	*****	*******	1<109	-------
	26	1<109	-------	*****	*******	1<81	-------	1<109	-------
Near P11	49	1<109	-------	1<81	-------	*****	*******	1<109	-------
	50	1<109	-------	*****	*******	1<81	-------	1<109	-------
	52	2<109	-------	*****	*******	2<81	-------	2<109	-------
Middle of Center	104	1<84	-------	*****	*******	1<62	-------	1<84	-------
	105	1<84	-------	1<62	-------	*****	*******	1<84	-------
	106	1<84	-------	*****	*******	1<62	-------	1<84	-------

Position	No.	Point-5		Point-6		Point-7		Point-8	
		Grade E		Grade E		Grade D		Grade G	
		a)	b)	a)	b)	a)	b)	a)	b)
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j
Near P10	2	*****	*******	10<68	-------	10<92	-------	*****	*******
	3	18<75	-------	*****	*******	18<101	-------	*****	*******
	4	*****	*******	24<71	-------	24<96	-------	*****	*******
Middle od Side Span	24	*****	*******	50<68	-------	50<93	-------	*****	*******
	25	49<68	-------	*****	*******	49<91	-------	*****	*******
	26	*****	*******	47<69	-------	47<94	-------	*****	*******
Near P11	49	15<81	-------	*****	*******	15<109	-------	*****	*******
	50	*****	*******	16<81	-------	16<109	-------	*****	*******
	52	*****	*******	19<81	-------	19<109	-------	19<42	-------
Middle of Center	104	*****	*******	33<62	-------	33<84	-------	*****	*******
	105	32<62	-------	*****	*******	32<84	-------	*****	*******
	106	*****	*******	33<62	-------	33<84	-------	*****	*******

Position	No.	Point-9		Point-10		Point-11		Point-12	
		Grade E		Grade E		Grade E		Grade E	
		a)	b)	a)	b)	a)	b)	a)	b)
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j
Near P10	2	*****	*******	-----	-------	*****	*******	10<68	-------
	3	0<65	-------	*****	*******	18<75	-------	*****	*******
	4	*****	*******	0<81	-------	*****	*******	24<71	-------
Middle od Side Span	24	*****	*******	1<81	-------	*****	*******	50<68	-------
	25	1<81	-------	*****	*******	49<68	-------	*****	*******
	26	*****	*******	1<81	-------	*****	*******	47<69	-------
Near P11	49	1<81	-------	*****	*******	15<81	-------	*****	*******
	50	*****	*******	1<81	-------	*****	*******	16<81	-------
	52	*****	*******	2<81	-------	*****	*******	19<81	-------
Middle of Center	104	*****	*******	1<62	-------	*****	*******	33<62	-------
	105	1<62	-------	*****	*******	32<62	-------	*****	*******
	106	*****	*******	1<62	-------	*****	*******	33<62	-------

Note: a) is the simple fatigue evaluation, b) is the detailed fatigue evaluation

Source: JICA Study Team

(2) Fatigue Evaluation of the Cable Anchorage Member

Similar to the main girder, an evaluation of fatigue is performed for the cable anchorage member.

4.2.9.9 Welding Design

(1) Calculation for Main Girder Welds

The results of the calculation for the main girder welds are listed below.

Table 4.2.14 Calculation Results for Fillet Welds (Outer Web)

Section	tu	tw	Stress		Allowable Value		Fillet Welding Size				
	tl	tw	τ	σ	τa	σa	S1	S2	Sreq	√(2・t)	S
	(mm)	(mm)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(mm)	(mm)	(mm)	(mm)	(mm)
EJ2	16	14	31.1	-27.7	120	210	2.57	2.36	2.57	5.66	6
	14	14	27.5	-51.5	120	210	2.27	2.12	2.27	5.29	6
EJ4	16	14	22.1	-51.2	120	210	1.82	1.71	1.82	5.66	6
	14	14	19.6	-91.8	120	210	1.62	1.61	1.62	5.29	6
EJ6	16	14	19.4	-60.5	120	210	1.60	1.51	1.60	5.66	6
	11	14	16.6	-92.2	120	210	1.37	1.36	1.37	5.29	6
EJ8	16	14	25.8	-55.2	120	210	2.13	2.00	2.13	5.66	6
	11	14	21.3	-73.3	120	210	1.76	1.69	1.76	5.29	6
EJ10	16	14	24.8	-48.9	120	210	2.05	1.91	2.05	5.66	6
	11	14	21.1	-62.0	120	210	1.74	1.65	1.74	5.29	6
EJ12	16	14	42.8	-43.8	120	210	3.53	3.28	3.53	5.66	6
	11	14	35.3	-80.5	120	210	2.91	2.84	2.91	5.29	6
EJ14	16	17	54.3	-21.6	120	210	5.44	4.99	5.44	5.83	6
	15	17	47.7	-110.9	120	210	4.78	4.98	4.98	5.83	6
EJ16	16	14	33.2	-53.2	120	210	2.74	2.57	2.74	5.66	6
	11	14	30.6	-67.1	120	210	2.52	2.41	2.52	5.29	6
EJ18	16	14	34.6	-51.8	120	210	2.85	2.67	2.85	5.66	6
	11	14	30.7	-65.3	120	210	2.53	2.41	2.53	5.29	6
EJ20	16	14	30.2	-54.5	120	210	2.49	2.34	2.49	5.66	6
	11	14	26.6	-55.0	120	210	2.19	2.06	2.19	5.29	6
EJ22	16	14	26.4	-48.3	120	210	2.18	2.03	2.18	5.66	6
	11	14	23.0	-55.4	120	210	1.90	1.78	1.90	5.29	6
EJ24	16	14	27.9	-42.9	120	210	2.30	2.14	2.30	5.66	6
	11	14	23.9	-37.9	120	210	1.97	1.82	1.97	5.29	6
EJ26	16	14	30.8	-28.4	120	210	2.54	2.34	2.54	5.66	6
	11	14	25.9	89.7	120	210	2.14	2.12	2.14	5.29	6

Source: JICA Study Team

Table 4.2.15 Calculation Results for Fillet Welds (Inner Web)

Section	tu	tw	Stress		Additional		Composite		Allowable Value		Fillet Welding Size				
	tl	tw	τ1	σ1	τ2	σ2	Στ	Σσ	τa	σa	S1	S2	Sreq	√(2・t)	S
	(mm)	(mm)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(mm)	(mm)	(mm)	(mm)	(mm)
EJ2	16	14	17.8	-30.6	-	-	17.8	-30.6	120	210	1.47	1.35	1.47	5.66	6
	14	14	14.4	-51.4	-	-	14.4	-51.4	120	210	1.19	1.11	1.19	5.29	6
EJ4	16	14	12.3	-55.9	-	-	12.3	-55.9	120	210	1.01	0.95	1.01	5.66	6
	14	14	10.0	-91.6	-	-	10.0	-91.6	120	210	0.82	0.82	0.82	5.29	6
EJ6	16	14	11.1	-65.6	-	-	11.1	-65.6	120	210	0.92	0.87	0.92	5.66	6
	11	14	8.5	-92.1	-	-	8.5	-92.1	120	210	0.70	0.70	0.70	5.29	6
EJ8	16	14	17.6	-58.8	-	-	17.6	-58.8	120	210	1.45	1.37	1.45	5.66	6
	11	14	13.3	-73.3	-	-	13.3	-73.3	120	210	1.10	1.06	1.10	5.29	6
EJ10	16	14	15.0	-51.1	-	-	15.0	-51.1	120	210	1.24	1.16	1.24	5.66	6
	11	14	11.4	-62.0	-	-	11.4	-62.0	120	210	0.94	0.89	0.94	5.29	6
EJ12	16	14	29.1	-44.9	-	-	29.1	-44.9	120	210	2.40	2.23	2.40	5.66	6
	11	14	22.0	-80.6	-	-	22.0	-80.6	120	210	1.81	1.77	1.81	5.29	6
EJ14	16	18	32.1	-20.2	-	-	32.1	-20.2	120	210	3.40	3.12	3.40	6.00	6
	15	18	25.9	-110.4	-	-	25.9	-110.4	120	210	2.75	2.86	2.86	6.00	6
EJ16	16	14	12.2	-55.1	-	-	12.2	-55.1	120	210	1.01	0.95	1.01	5.66	6
	11	14	9.8	-67.2	-	-	9.8	-67.2	120	210	0.81	0.77	0.81	5.29	6
EJ18	16	14	16.7	-54.1	-	-	16.7	-54.1	120	210	1.38	1.29	1.38	5.66	6
	11	14	13.0	-65.3	-	-	13.0	-65.3	120	210	1.07	1.02	1.07	5.29	6
EJ20	16	14	15.2	-57.9	-	-	15.2	-57.9	120	210	1.25	1.18	1.25	5.66	6
	11	14	11.8	-55.0	-	-	11.8	-55.0	120	210	0.97	0.92	0.97	5.29	6
EJ22	16	14	14.0	-50.7	-	-	14.0	-50.7	120	210	1.15	1.08	1.15	5.66	6
	11	14	10.8	-55.4	-	-	10.8	-55.4	120	210	0.89	0.84	0.89	5.29	6
EJ24	16	14	16.2	-46.4	-	-	16.2	-46.4	120	210	1.34	1.25	1.34	5.66	6
	11	14	12.4	-37.9	-	-	12.4	-37.9	120	210	1.02	0.95	1.02	5.29	6
EJ26	16	14	19.3	-4.2	-	-	19.3	-4.2	120	210	1.59	1.45	1.59	5.66	6
	11	14	14.6	89.4	-	-	14.6	89.4	120	210	1.20	1.19	1.20	5.29	6

Source: JICA Study Team

(2) Calculation for Main Tower Welds

The results of the calculation for the main tower welds are listed below.

Table 4.2.16 Calculation Results for Fillet Welds (Inner Web)

Section	Tu	tw	Stress		Additional		Composite		Allowable Value		Required Throat				Welds Size				Analysis					
	Tl	tw	τ1	σ1	τ2	σ2	Στ	Σσ	τa	σa	a1	a2	areq	areq*1.5	√(2・t)	S1'	S1"	S2	1	2	3	4	Design Throat a(mm)	
	(mm)	(mm)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)						
J20	40	40	0.1	-0.2	-	-	0.1	-0.2	120	210	0.03	0.03	0.03	0.05	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	0.1	-0.2	-	-	0.1	-0.2	120	210	0.03	0.03	0.03	0.05	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J19	40	40	4.3	-10.7	-	-	4.3	-10.7	120	210	1.43	1.31	1.43	2.15	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	4.3	-10.7	-	-	4.3	-10.7	120	210	1.43	1.31	1.43	2.15	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J18	40	40	7.1	-14.7	24.1	-25.1	31.2	-39.8	120	210	10.40	9.64	10.40	15.60	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	7.1	-27.2	31.6	-0.4	38.7	-27.6	120	210	12.90	11.86	12.90	19.35	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J17	40	40	8.3	-19.7	24.1	-25.1	32.4	-44.8	120	210	10.80	10.05	10.80	16.20	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	8.3	-47.7	31.6	-0.4	39.9	-48.1	120	210	13.30	12.42	13.30	19.95	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J16	40	40	8.3	-26.6	28.9	-20.6	37.2	-47.2	120	210	12.40	11.57	12.40	18.60	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	8.3	-70.0	34.0	-12.7	42.3	-82.7	120	210	14.10	13.79	14.10	21.15	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J15	40	40	8.1	-34.4	28.9	-20.6	37.0	-55.0	120	210	12.33	11.60	12.33	18.50	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	8.1	-93.0	34.0	-12.7	42.1	-105.7	120	210	14.03	14.42	14.42	21.63	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J14	40	40	6.6	-39.7	28.9	-20.6	35.5	-60.3	120	210	11.83	11.19	11.83	17.75	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	6.6	-109.5	34.0	-12.7	40.6	-122.2	120	210	13.53	14.58	14.58	21.87	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J13	40	40	5.3	-46.0	17.4	-10.7	22.7	-56.7	120	210	7.57	7.13	7.57	11.36	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	5.3	-125.1	19.7	-8.7	25.0	-133.8	120	210	8.33	9.35	9.35	14.03	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J12	40	40	3.7	-53.0	17.4	-10.7	21.1	-63.7	120	210	7.03	6.68	7.03	10.55	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	3.7	-138.8	19.7	-8.7	23.4	-147.5	120	210	7.80	9.28	9.28	13.92	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J11	40	40	2.2	-60.8	19.8	-1.7	22.0	-62.5	120	210	7.33	6.96	7.33	11.00	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	2.2	-150.3	17.5	-20.9	19.7	-171.2	120	210	6.57	8.97	8.97	13.46	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J10	40	40	2.6	-69.2	19.8	-1.7	22.4	-70.9	120	210	7.47	7.16	7.47	11.21	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	2.6	-159.2	17.5	-20.9	20.1	-180.1	120	210	6.70	9.83	9.83	14.75	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J9	40	40	2.6	-71.2	19.8	-1.7	22.4	-72.9	120	210	7.47	7.19	7.47	11.21	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	2.6	-154.4	17.5	-20.9	20.1	-175.3	120	210	6.70	9.45	9.45	14.18	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J8	40	40	2.7	-74.1	-	-	2.7	-74.1	120	210	0.90	0.87	0.90	1.35	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	2.7	-147.5	-	-	2.7	-147.5	120	210	0.90	1.07	1.07	1.61	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J7	40	40	2.7	-77.3	-	-	2.7	-77.3	120	210	0.90	0.87	0.90	1.35	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
	40	40	2.7	-140.7	-	-	2.7	-140.7	120	210	0.90	1.04	1.04	1.56	8.94	10	10	9	OK	OK	OK	OK	23.4	OK
J6	35	35	3.2	-90.5	-	-	3.2	-90.5	120	210	0.93	0.93	0.93	1.40	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
	35	35	3.2	-150.7	-	-	3.2	-150.7	120	210	0.93	1.13	1.13	1.70	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
J5	35	35	3.2	-96.3	-	-	3.2	-96.3	120	210	0.93	0.94	0.94	1.41	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
	35	35	3.2	-143.4	-	-	3.2	-143.4	120	210	0.93	1.09	1.09	1.64	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
J4	35	35	3.3	-102.8	-	-	3.3	-102.8	120	210	0.96	0.98	0.98	1.47	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
	35	35	3.3	-136.5	-	-	3.3	-136.5	120	210	0.96	1.09	1.09	1.64	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
J3	35	35	3.3	-109.6	-	-	3.3	-109.6	120	210	0.96	1.00	1.00	1.50	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
	35	35	3.3	-130.2	-	-	3.3	-130.2	120	210	0.96	1.07	1.07	1.61	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
J2	35	35	3.5	-116.8	-	-	3.5	-116.8	120	210	1.02	1.08	1.08	1.62	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
	35	35	3.5	-126.9	-	-	3.5	-126.9	120	210	1.02	1.12	1.12	1.68	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
J1	35	35	3.6	-123.6	-	-	3.6	-123.6	120	210	1.05	1.14	1.14	1.71	8.37	9	9	9	OK	OK	OK	OK	21.7	OK
	35	35	3.6	-127.7	-	-	3.6	-127.7	120	210	1.05	1.15	1.15	1.73	8.37	9	9	9	OK	OK	OK	OK	21.7	OK

Source: JICA Study Team

4.2.9.10 Evaluation of Ultimate Strength (1.7 x Design Loads)

(1) Evaluation Results for Girder

The evaluation results are shown in the table below. The stresses in all sections were less than the allowable value.

Table 4.2.17 Evaluation Results for Main Girder

Sider Span-Section 1

Stress	M-Max				M-Min				N-Max				N-Min			
(N/mm^2)	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa
DECK-L	-54	140	0	80	-21	140	7	80	14	140	19	80	-25	140	10	80
DECK-R	-54	140	0	80	-21	140	7	80	14	140	19	80	-25	140	10	80
WEB-1	-49	166	0	120	11	210	10	120	12	210	27	120	-14	200	15	120
WEB-2	-53	161	0	120	-95	147	10	120	-27	152	27	120	-94	146	15	120
WEB-3	-53	161	0	120	-95	147	10	120	-27	152	27	120	-94	146	15	120
WEB-4	-49	166	0	120	11	210	10	120	12	210	27	120	-14	200	15	120
WEB-L	-29	182	0	120	-96	150	10	120	-27	153	28	120	-94	150	15	120
LFLG	57	210	0	120	-96	147	7	120	-27	147	20	120	-94	147	11	120
WEB-R	-29	182	0	120	-96	150	10	120	-27	153	28	120	-94	150	15	120

Side Span-Section 2

Stress	M-Max				M-Min				N-Max				N-Min			
(N/mm^2)	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa
DECK-L	-59	140	2	80	-26	140	3	80	-55	140	4	80	-35	140	10	80
DECK-R	-59	140	2	80	-26	140	3	80	-55	140	4	80	-35	140	10	80
WEB-1	-53	167	3	120	-18	182	5	120	-49	167	5	120	-30	165	14	120
WEB-2	-57	162	3	120	-87	145	5	120	-54	163	5	120	-75	142	14	120
WEB-3	-57	162	3	120	-87	145	5	120	-54	163	5	120	-75	142	14	120
WEB-4	-53	167	3	120	-18	182	5	120	-49	167	5	120	-30	165	14	120
WEB-L	65	210	3	120	-87	149	5	120	64	210	5	120	-76	147	14	120
LFLG	65	210	3	120	-88	102	4	120	65	210	5	120	-76	102	12	120
WEB-R	65	210	3	120	-87	149	5	120	64	210	5	120	-76	147	14	120

Intermediate Pier(at Tower)-Section 3

Stress	M-Max				M-Min				N-Max				N-Min			
(N/mm^2)	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa
DECK-L	-30	210	17	120	52	210	59	120	-25	210	23	120	39	210	28	120
DECK-R	-30	210	17	120	52	210	59	120	-25	210	23	120	39	210	28	120
WEB-1	-17	201	18	120	42	210	62	120	11	210	24	120	30	210	30	120
WEB-2	-72	210	11	120	-149	210	39	120	-84	210	15	120	-130	210	19	120
WEB-3	-72	210	11	120	-149	210	39	120	-84	210	15	120	-130	210	19	120
WEB-4	-17	201	18	120	42	210	62	120	11	210	24	120	30	210	30	120
WEB-L	-72	177	18	120	-149	180	63	120	-85	179	24	120	-131	179	30	120
LFLG	-72	158	15	120	-150	158	51	120	-85	158	20	120	-131	158	25	120
WEB-R	-72	177	18	120	-149	180	63	120	-85	179	24	120	-131	179	30	120

Main Span-Section4

Stress	M-Max				M-Min				N-Max				N-Min			
(N/mm^2)	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa
DECK-L	-53	140	0	80	-11	140	11	80	-51	140	7	80	-32	140	0	80
DECK-R	-53	140	0	80	-11	140	11	80	-51	140	7	80	-32	140	0	80
WEB-1	-44	174	1	120	-23	175	16	120	-43	174	11	120	-35	160	0	120
WEB-2	113	210	1	120	-92	145	17	120	108	210	11	120	-50	139	0	120
WEB-3	113	210	1	120	-92	145	17	120	108	210	11	120	-50	139	0	120
WEB-4	-44	174	1	120	-23	175	16	120	-43	174	11	120	-35	160	0	120
WEB-L	113	210	1	120	-92	148	16	120	109	210	11	120	-50	145	0	120
LFLG	113	210	1	120	-92	102	14	120	109	210	9	120	-50	102	0	120
WEB-R	113	210	1	120	-92	148	16	120	109	210	11	120	-50	145	0	120

Source: JICA Study Team

(2) Evaluation Results for Tower

The evaluation results are shown in the table below. The stresses in all sections were less than the allowable value.

Table 4.2.18 Evaluation Results for Tower

Upper Cable Section

Stress	M-Max				M-Min				N-Max				N-Min			
(N/mm^2)	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa
Top	-81	210	6	120	9	210	7	120	0	210	0	120	-5	210	6	120
LWeb	-81	210	10	120	-104	210	12	120	0	210	0	120	-99	210	10	120
Rweb	-81	210	10	120	-104	210	12	120	0	210	0	120	-99	210	10	120
Bott	-1	210	6	120	-104	210	7	120	0	210	0	120	-99	210	6	120

Lower Cable Section

Stress	M-Max				M-Min				N-Max				N-Min			
(N/mm^2)	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa
Top	-124	210	0	120	-7	210	1	120	-87	210	5	120	-82	210	1	120
LWeb	-124	210	1	120	-149	210	2	120	-87	210	8	120	-99	210	1	120
Rweb	-124	210	1	120	-149	210	2	120	-87	210	8	120	-99	210	1	120
Bott	-18	210	0	120	-149	210	1	120	-7	210	5	120	-99	210	1	120

Bottom of Tower

Stress	M-Max				M-Min				N-Max				N-Min			
(N/mm^2)	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa	Σσ	σa	Στ	τa
Top	-125	205	2	120	-53	205	3	120	-125	205	2	120	-107	205	1	120
LWeb	-125	210	4	120	-143	210	5	120	-125	210	4	120	-107	210	1	120
Rweb	-125	210	4	120	-143	210	5	120	-125	210	4	120	-107	210	1	120
Bott	-55	205	2	120	-143	205	3	120	-55	205	2	120	-101	205	1	120

Source: JICA Study Team

4.2.9.11 Structural Analysis Considering Plasticity of Superstructure

(1) Safety Investigation Using Elasto-Plastic and Finite Displacement Analysis

By using the elasto-plastic and finite displacement analysis, safety evaluation under ultimate state of the designed bridge was performed based on the load coefficient design method. As a necessary parameter, the scale factor of load (α_{max}) at which the designed bridge reaches its ultimate state considering the elasto-plasticity was determined by gradually increasing the working force.

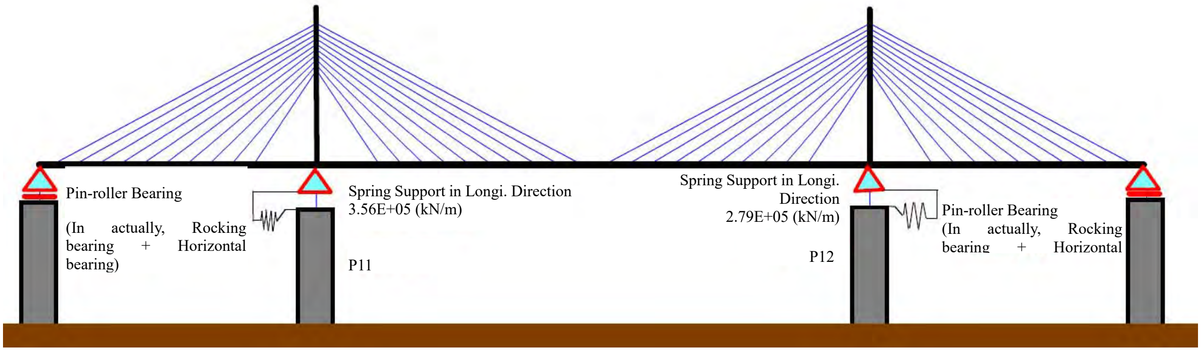
In order to evaluate the effect of loading range to the ultimate state, four cases of different loading conditions were assumed and employed in the analysis model shown in the figure below.

Table 4.2.19 Load Combination and Loading Range of Live Load

Load Combination / Load Scale Factor (α)	Loading Range of Live Load
α (D + L) + PS	L1: loading on the entire span L2: loading on the center span L3: loading on the half of center span L4: loading on the side span

Note: α: Load scale factor, D: Dead load, L: Live load, PS: Pre-stress

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.22 Analysis Model of Cable-stayed Bridge

(2) Analysis Result

The load scale factor when the girder, cable, and tower yielded and at the ultimate state (when the load scale factor becomes maximum) is shown below.

Table 4.2.20 Load Scale Factor α

Load Combination	Loading Range of Live Load	Load Scale Factor α Yield of Main Girder	Yield of Cable	Yield of Main Tower	Maximum (Ultimate State)
α (D + L) + PS	L1: loading on the entire span	2.07	2.51	2.84	2.98
	L2: loading on the center span	2.35	2.47	2.15	2.66
	L3: loading on the half of center span	2.31	2.47	2.26	2.72
	L4: loading on the side span	2.30	2.57	---	3.20

Source: JICA Study Team

From the analysis results, the following tendency was figured out regarding the process when the designed bridge reaches ultimate state.

Table 4.2.21 Processes to Ultimate State

Loading Range for Live Load	Process to Ultimate State
L1: loading on the entire span	Main girder (near the main tower) → Cable (center) → Main tower (base) → [Ultimate state]
L2: loading on the center span	Main tower (middle) → Main girder (near the main tower) → Cable (middle) → [Ultimate state]
L3: loading on the half of center span	Main tower (middle) → Main girder (near the main tower) → Cable (middle) → [Ultimate state]
L4: loading on the side span	Main tower (middle) → Cable (middle) → [Ultimate state]

Source: JICA Study Team

Based on the analysis results, the following conclusions can be stated:

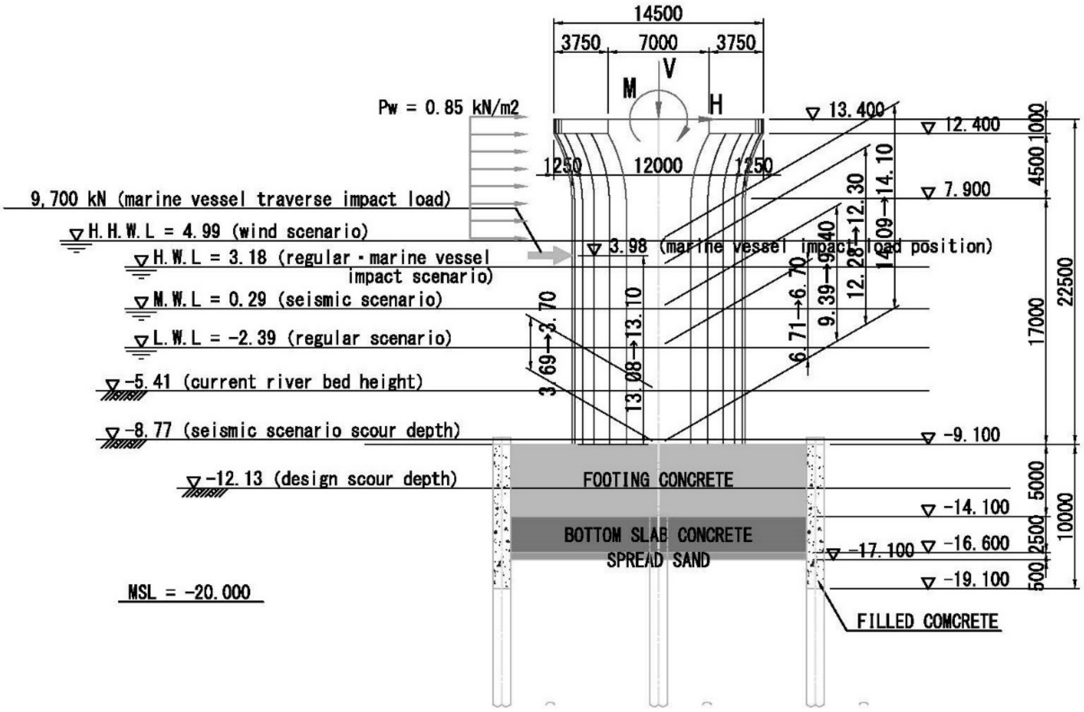
- The maximum load at ultimate state is about 2.7 times larger than D+L (dead load + live load). It means that the loading capacity of the designed bridge is high enough for the design load (D+L+PS).
- The designed bridge has sufficient loading capacity until the ultimate state. The relation between load and deflection at the center of the main girder does not change significantly even when the flange of the main girder or main tower is yielded.

4.2.10 Summary of Substructure Design

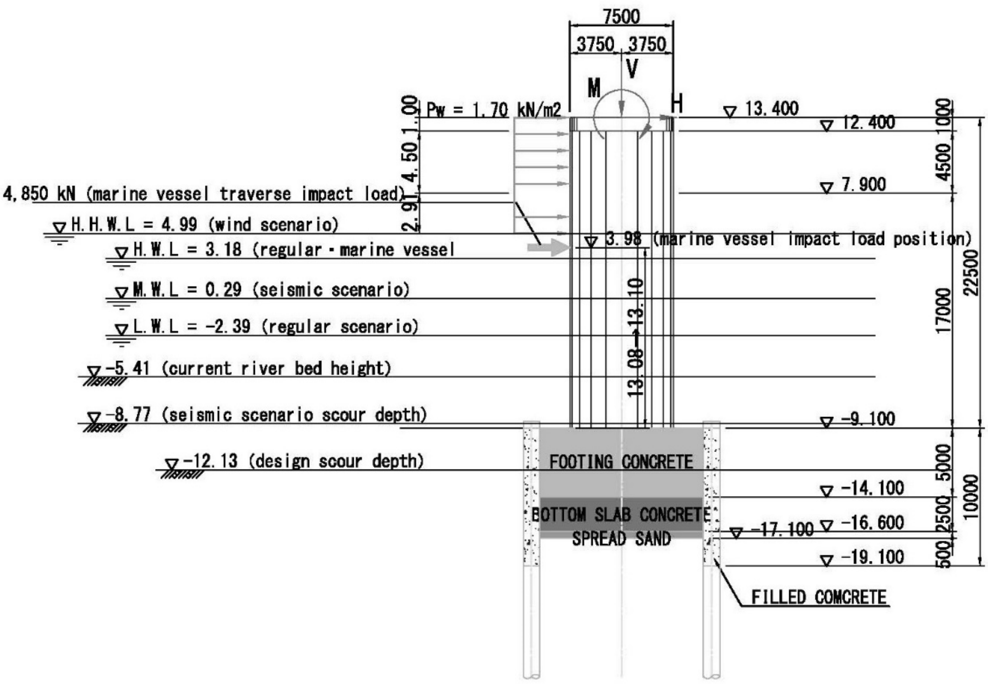
4.2.10.1 Calculation of Main Tower Pier/Foundation (P11 and P12)

(1) Figure of Design Condition

Front View



Side View



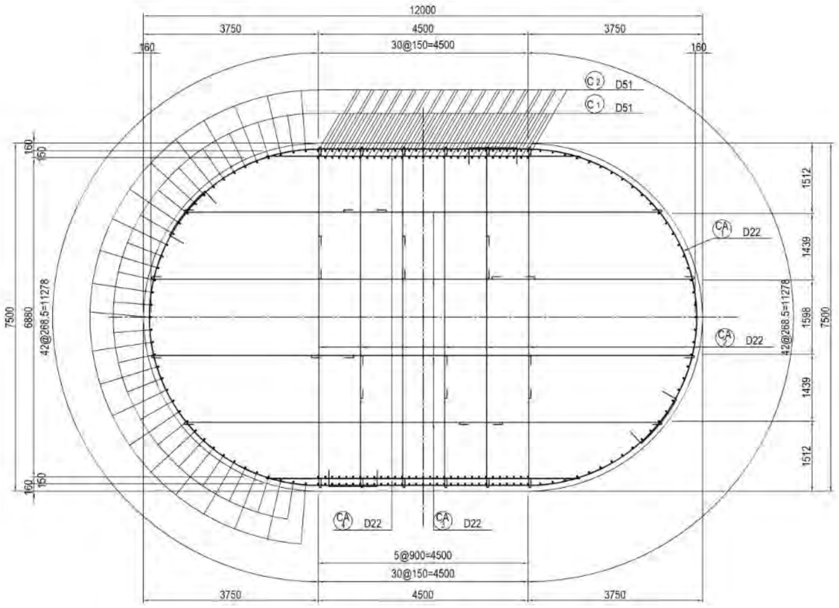
Source: JICA Study Team

Figure 4.2.23 Design Condition

(2) Design of Pier

1) Design of Column

The column shall be designed as a cantilever beam by treating the joint between the footing as a fixed end. The column cross section shall be designed against the most unfavorable combination of axial force and bending moment.



Source: JICA Study Team

Figure 4.2.24 Design Condition

<u>[Overview of Calculation Result]</u>

The following table shows the calculation results for beam.

Table 4.2.22 Calculation Result for Beam

				Longitude Direction				Transverse Direction			
Cross Section	Member Height		m	Elliptical Shape	;	12.000	×	7.500			
	Rebar	Main Rebar	1st block	D51	ctc	150		D51	ctc	269	
			2nd block	D51	ctc	150					
		Lateral Tie	---	D22	ctc	150		D22	ctc	150	
Cross Section Calculation	L1 Earthquake	σc	N/mm2	10.46	≦	15.00	○	8.85	≦	15.00	○
		σs	N/mm2	274.4	≦	300.0	○	200.2	≦	300.0	○
		τm	N/mm2	0.439	>	0.201	－	0.362	>	0.179	－
		Aw_req	mm2	1523.5	≦	3096.8	○	733.3	≦	2322.6	○

Source: JICA Study Team

2) Bridge Seat Design

a) Dimension of Bridge Seat Width

The distance between the bearing support edge and the top edge of the substructure was set in accordance with the Specifications for Highway Bridges IV 8.6.

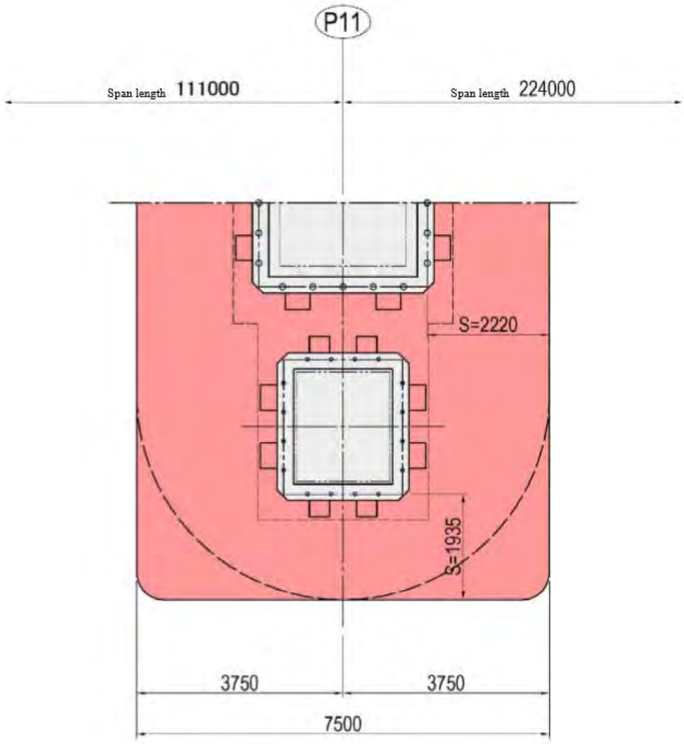


Figure 4.2.25 Bridge Seat Width

Source: JICA Study Team

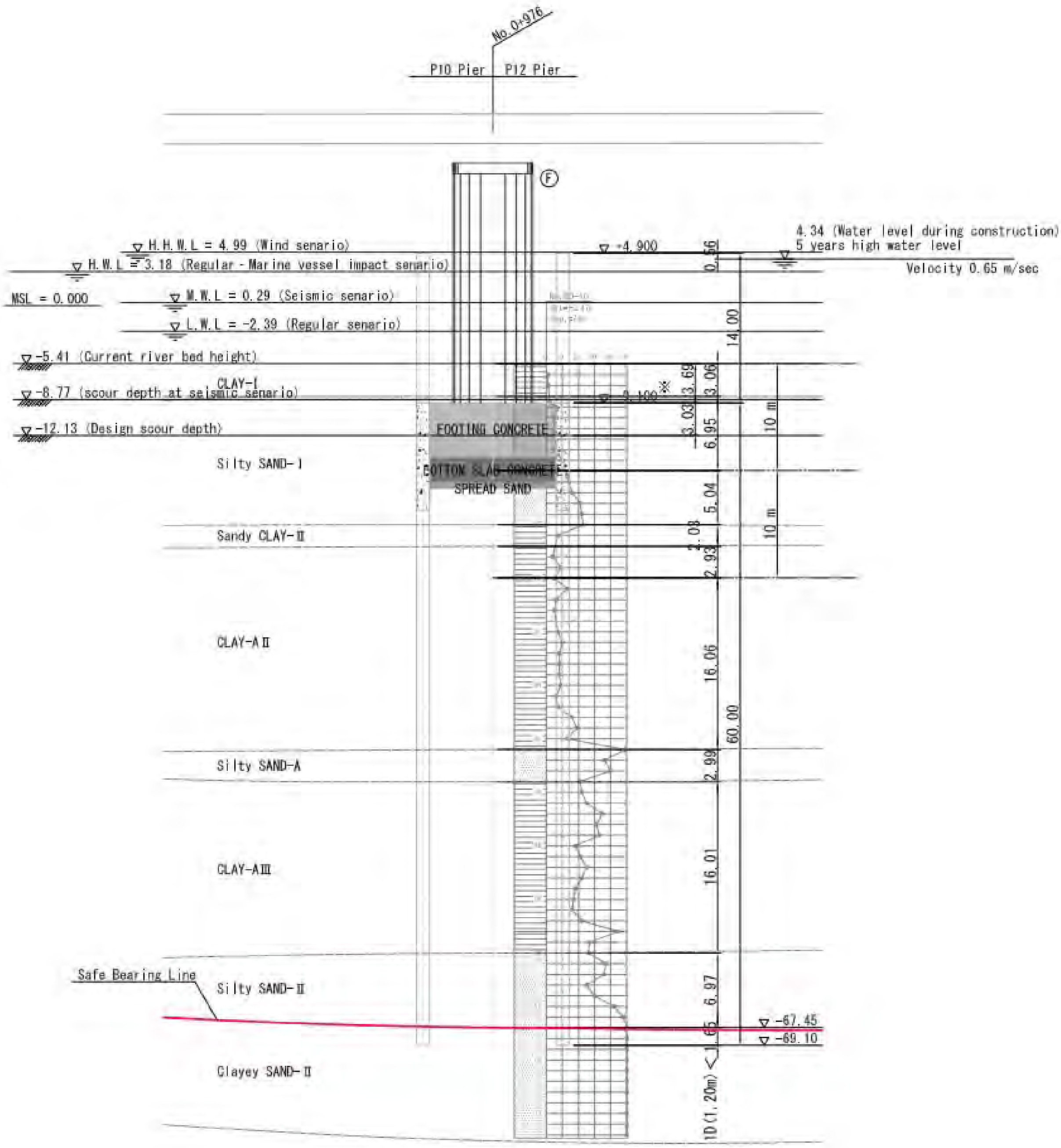
b) Evaluation of Bridge Seat Strength

Since the bridge seat has a function to support the superstructure via the bearing support, large horizontal force would act on it during an earthquake. For this reason, the bridge seat needs to be designed to have sufficient strength against design horizontal seismic force.

(3) Design of Foundation

1) Ground Conditions

The following figure shows the ground condition:

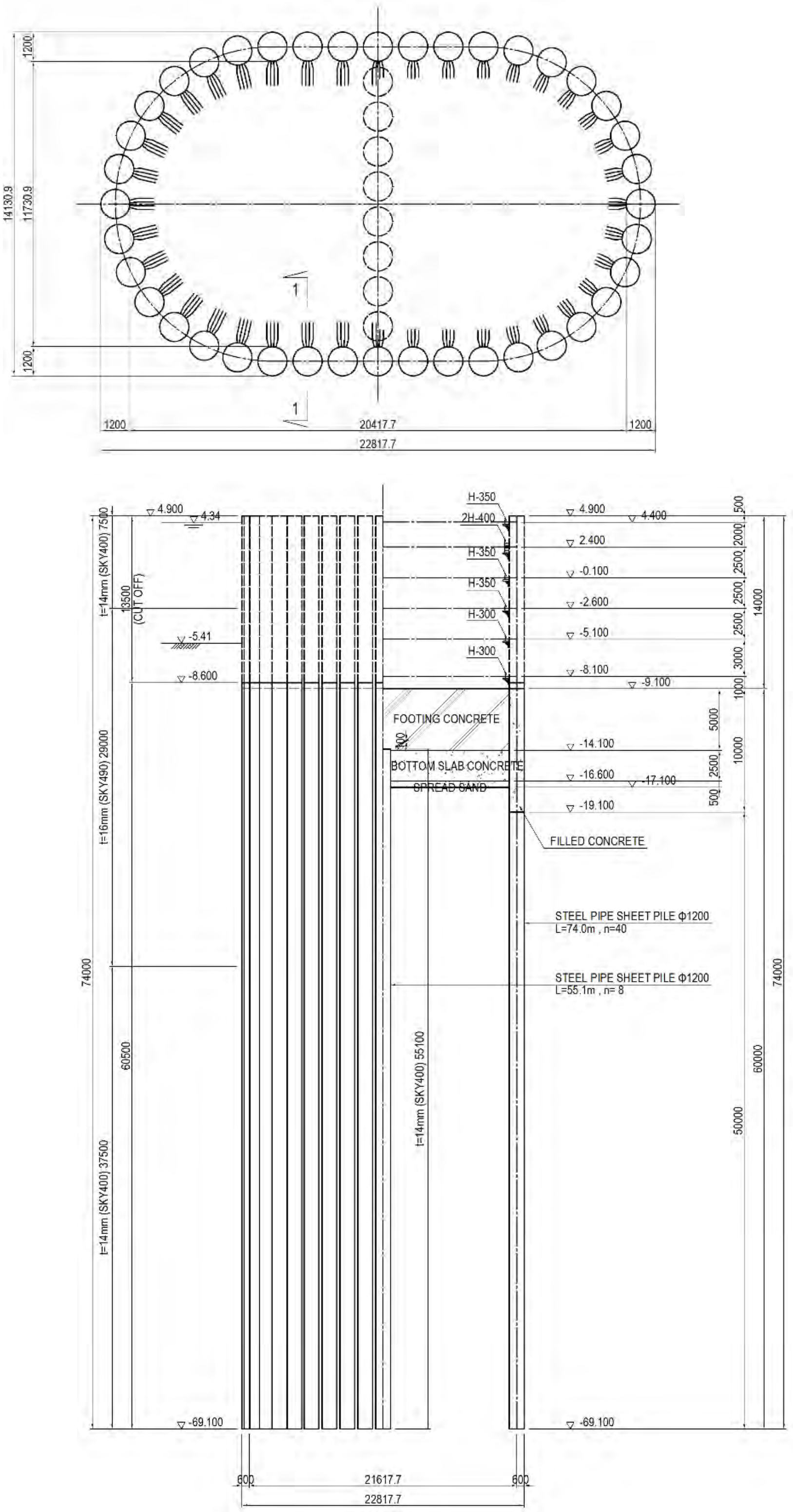


Source: JICA Study Team

Figure 4.2.26 Ground Condition

2) Foundation Shape (Steel Pile Sheet Pile Foundation)

The following figure shows the arrangement of the steel pile sheet pile foundation:



Source: JICA Study Team

Figure 4.2.27 Dimensional Drawing of Foundation Shape

[Calculation Result Table]

The table for the calculation results for the foundation is shown below.

Table 4.2.23 Calculation Results for Foundation

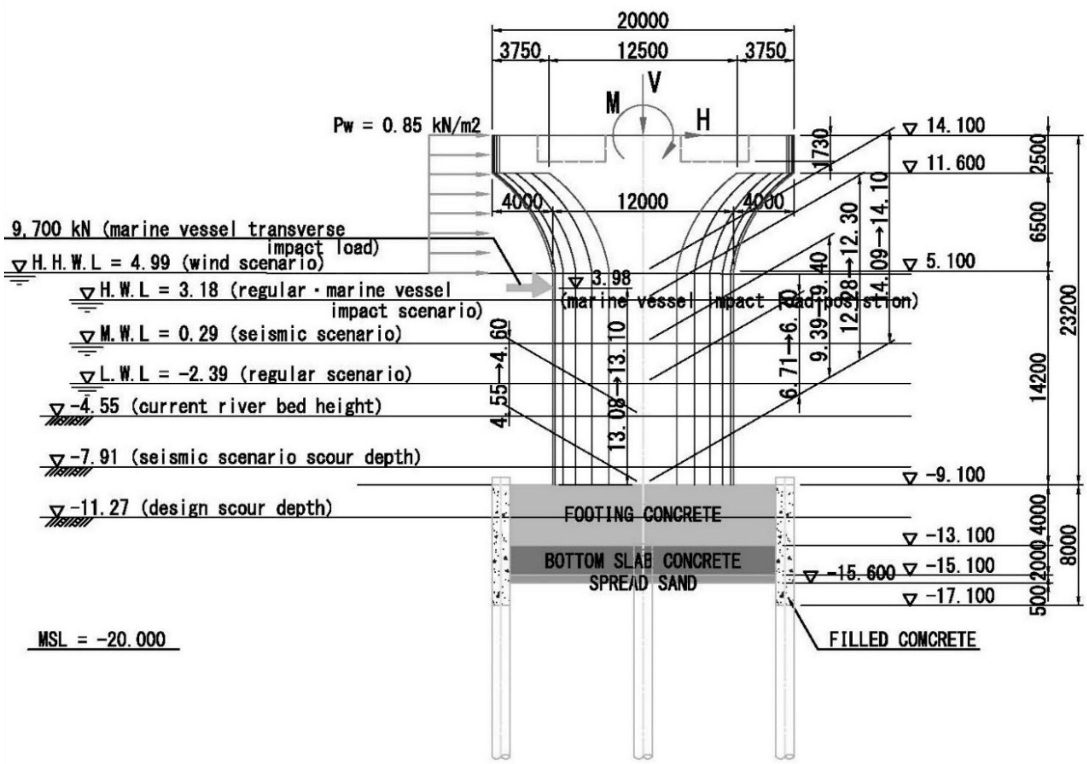
				Longitudinal Direction				Transverse Direction			
Pile	Size(mm)×Length(m)×Number			Outer peripheral sheet pile	;	φ1200	×	56.00	×	36 Piles	
				Partitioned sheet pile	;	φ1200	×	52.10	×	8 Piles	
	Steel Pipe Thickness	Outer peripheral sheet pile	Upper Pile	t = 14 mm (SKY490)							
			Lower Pile	t = 14 mm (SKY400)							
		Partitioned sheet pile	---	t = 14 mm (SKY400)							
Stability Calculation	Regular (Current River Bed)	δ	cm	0.41	≦	5.00	○	0.07	≦	5.00	○
		PNmax	KN/Number	2742	≦	3535	○	2740	≦	3535	○
		PNmin	KN/Number	2389	≧	-1865	○	2399	≧	-1865	○
	Seismic (Current River Bed)	δ	cm	2.68	≦	5.00	○	2.26	≦	5.00	○
		PNmax	KN/Number	2607	≦	5267	○	2623	≦	5267	○
		PNmin	KN/Number	2293	≧	-3092	○	2277	≧	-3092	○
Combined Stress (Seismic・Current River Bed)		SKY400	N/mm2	142.9	≦	210.0	○	156.4	≦	210.0	○
		SKY490	N/mm2	244.1	≦	277.5	○	242.1	≦	277.5	○

Source: JICA Study Team

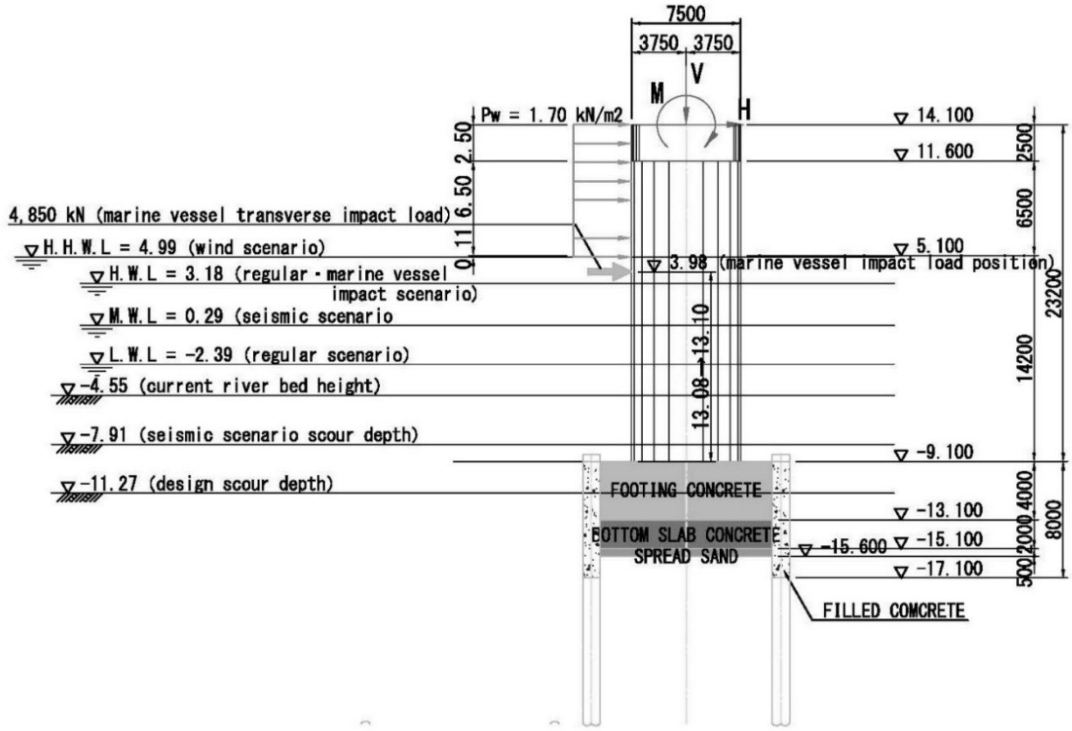
4.2.10.2 Calculation for Side Pier (P10 and P13)

(1) Figure of Design Condition

Front View



Side View



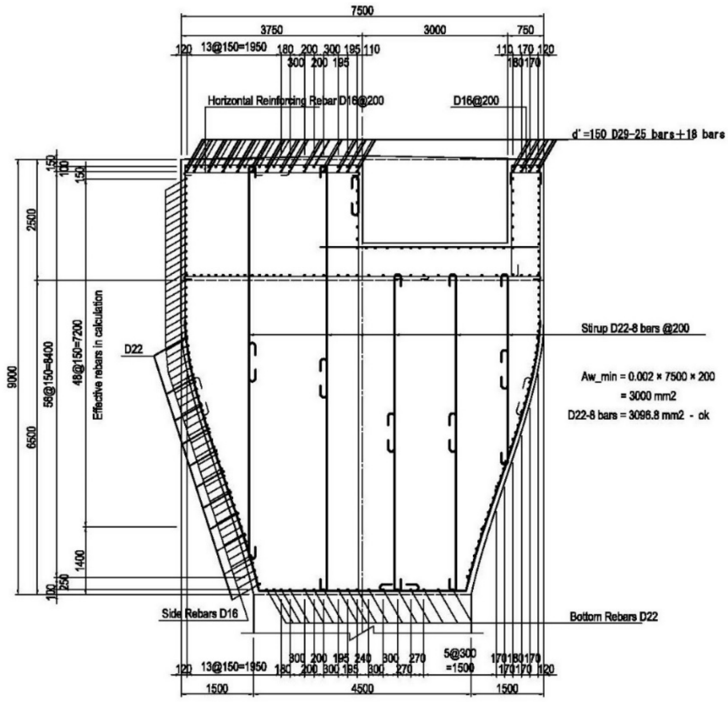
Source: JICA Study Team

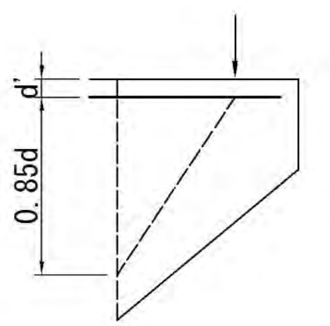
Figure 4.2.28 Design Condition

(2) Pier Design

1) Beam Design

The cross sectional shape of the beam and arrangement of steel reinforcement are shown below.





Note: Side reinforcement is effective in the range of 0.85 times the effective height.

Concrete cover for main steel reinforcement d' = 192 mm

Effective height d = 8808 mm

0.85d = 7487 mm

Effective range for side steel reinforcement = 192 + 7487 = 7679 mm

[Overview of Calculation Result]

The following table shows the calculation results for the beam.

Table 4.2.24 Calculation Results for Beam

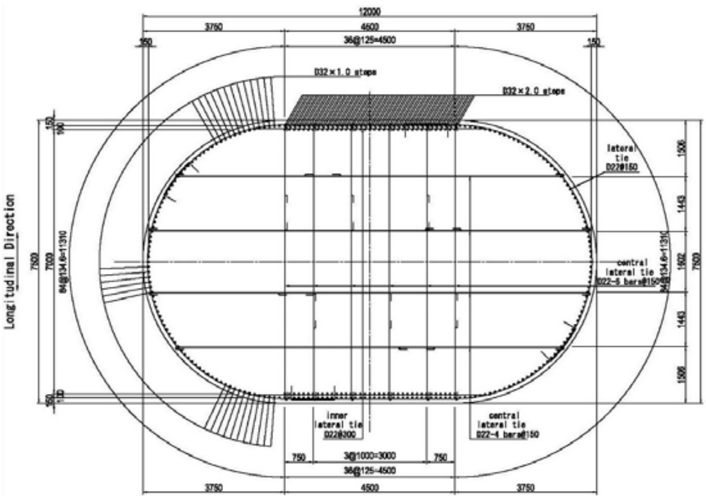
				Vertical Direction				Horizontal Direction			
Cross Section	Member Height		m	9.000				7.500			
	Rebar	Main Rebar	1st block	D29	─	25 rebars		D16	─	49本	
			2nd block	D29	─	18 rebars					
		Stirrup		D22-8rebars ctc200				D22-2rebars+D16-1rebars ctc200			
Bridge Seat Cracking		Required rebar amount	mm2		---				---		
Cobel		Required rebar amount	mm2	25,512	≦	27,623	○	11,049	≦	19,463	○
Cross Section Calculation	Bending Verification	Load Case		Dead Load				During Earthquake			
		σc	N/mm2	0.83	≦	10.00	○	0.71	≦	15.00	○
		σs	N/mm2	82.6	≦	100.0	○	99.5	≦	300.0	○
	Shear Verification	Load Case		Dead + Live Load				During Earthquake			
		τm	N/mm2	0.006	≦	0.143	○	0.047	≦	0.111	○
		Awreq < Aw	mm2								
	Verification for Earthquake Performance 2	M < My	KN･m		---			8,704	≦	21,371	○
		S < Ps	KN		---			3,636	≦	16,160	○

Source: JICA Study Team

2) Design of Column

The column shall be designed as a cantilever beam by treating the joint between the footing as a fixed end. The column cross section shall be designed against the most unfavorable combination of axial force and bending moment.

Note that the steel reinforcement in the column-axial direction was set by dynamic analysis evaluation.



Source: JICA Study Team

Figure 4.2.29 Cross Section of Column

[Overview of Calculation Result]

The following table shows the calculation results for the column.

Table 4.2.25 Calculation Result for Column

				Longitude Direction				Transverse Direction			
Cross Section	Member Height		m	Elliptical Shape	;	12.000	×	7.500			
	Rebar	Main Rebar	1st block	D32	ctc	125	※	D32	ctc	135	※
			2nd block	D32	ctc	125	※				
		Lateral Tie	---	D22	ctc	150		D22	ctc	150	
Cross Section Calculation	L1 Earthquake	σc	N/mm2	7.29	≦	15.00	○	4.96	≦	15.00	○
		σs	N/mm2	216.0	≦	300.0	○	100.3	≦	300.0	○
		τm	N/mm2	0.283	>	0.171	—	0.259	>	0.152	—
		Aw_req	mm2	721.6	≦	3096.8	○	431.4	≦	2322.6	○

Note: ※ was decided by dynamic analysis

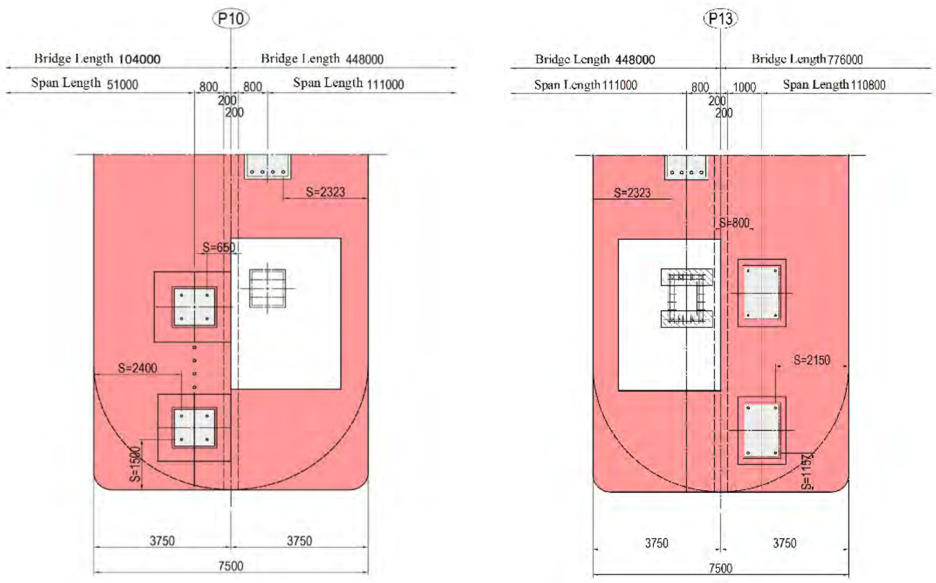
Source: JICA Study Team

3) Bridge Seat Design

a) Dimension of Bridge Seat Width

The distance between the bearing support edge and the top edge of the substructure was set in accordance with the Specifications for Highway Bridges IV 8.6.

[P10 Pier] [P13 Pier]



Source: JICA Study Team

Figure 4.2.30 Bridge Seat Width

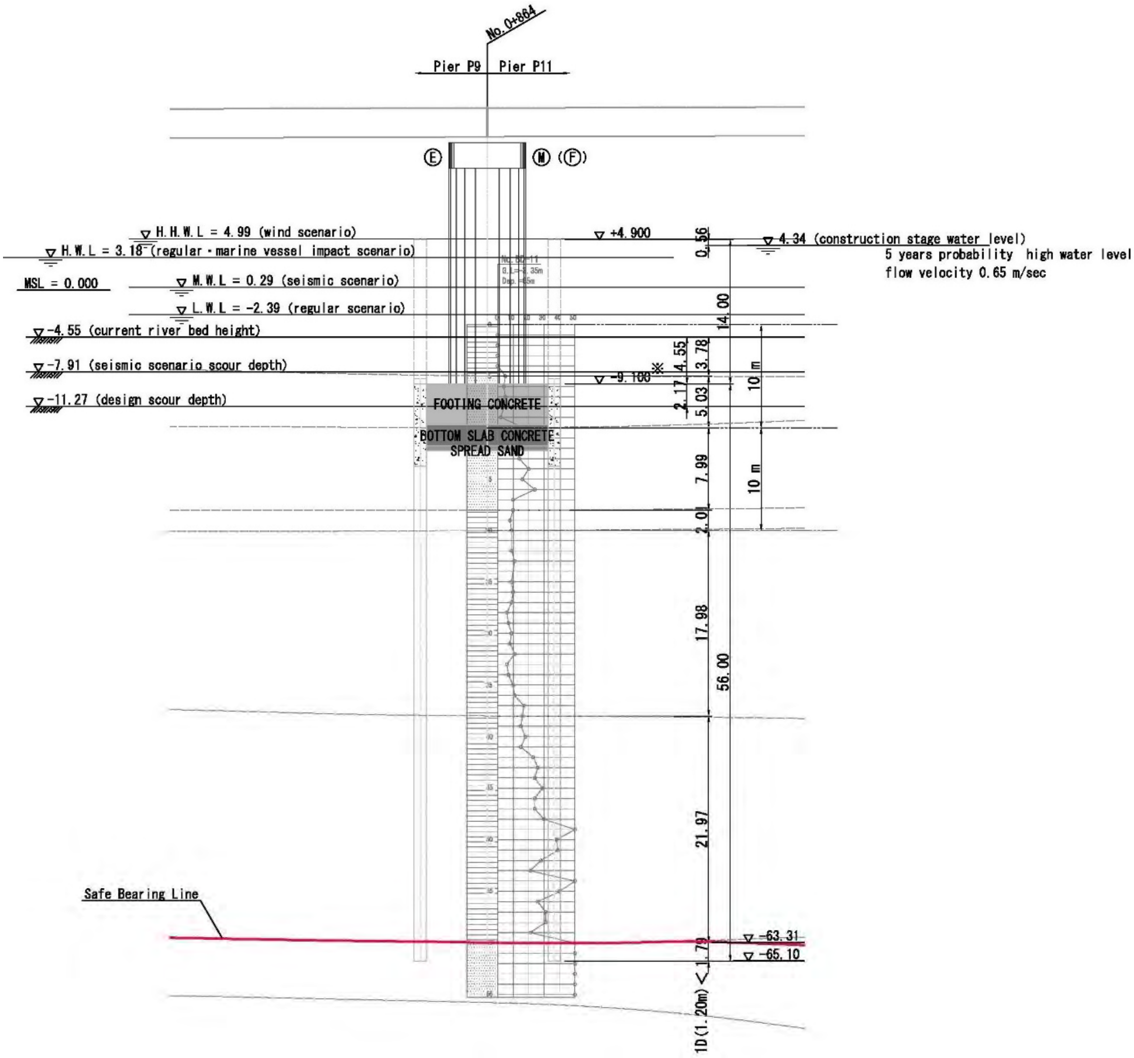
b) Evaluation of Bridge Seat Strength

Since the bridge seat has a function to support the superstructure via bearing support, large horizontal force would act on it during an earthquake. For this reason, the bridge seat needs to be designed to have sufficient strength against design horizontal seismic force.

(3) Foundation Design

1) Ground Conditions

The following figure shows the ground condition:

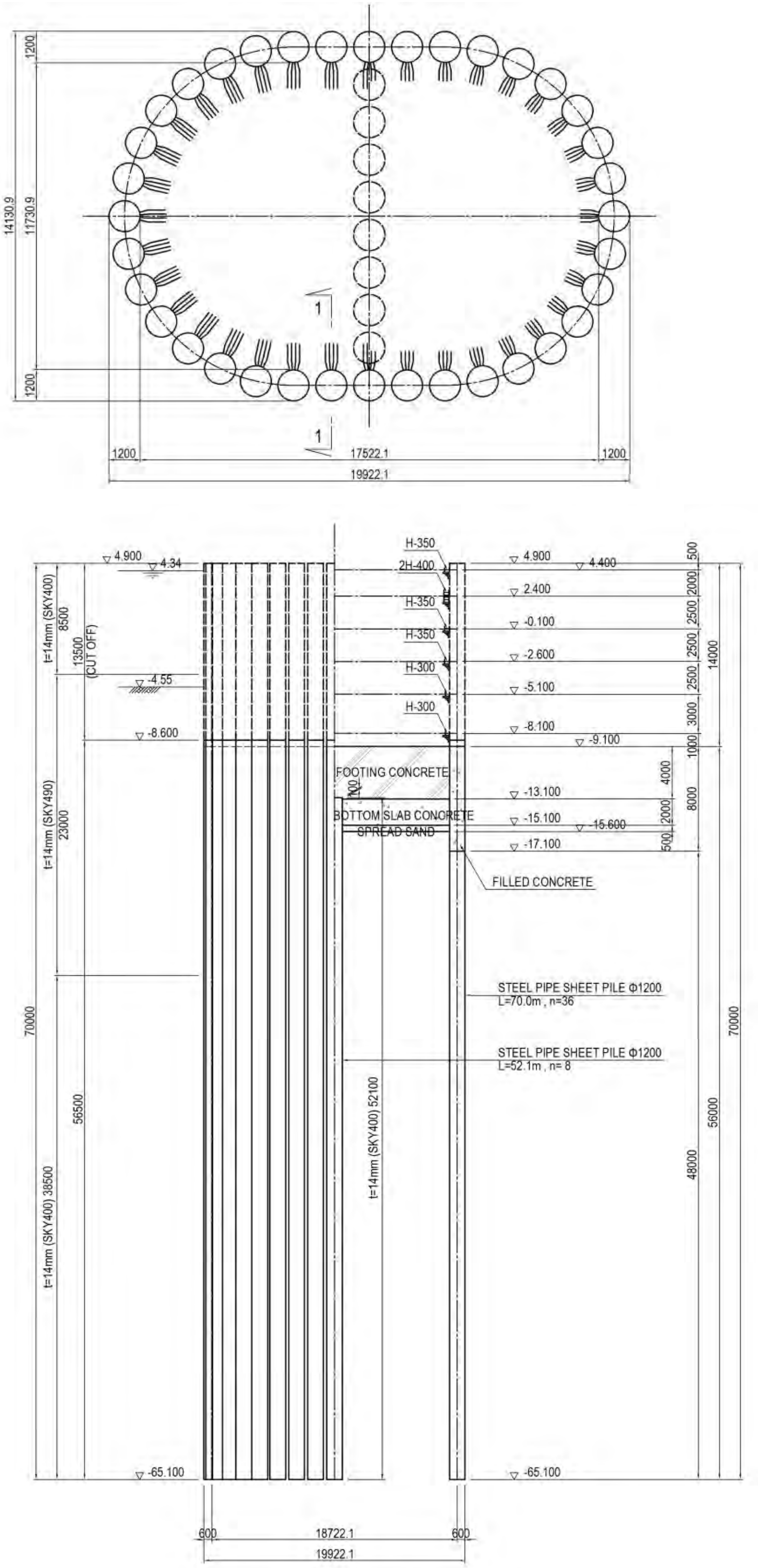


Source: JICA Study Team

Figure 4.2.31 Ground Condition

2) Foundation Shape (Steel Pile Sheet Pile Foundation)

The following figure shows the arrangement of the steel pile sheet pile foundation:



Source: JICA Study Team

Figure 4.2.32 Dimensional Drawing of Foundation Shape

[Calculation Result Table]

The table of the calculation results for the foundation is shown below.

Table 4.2.26 Calculation Results for Foundation

				Longitude Direction				Transverse Direction			
Pile	Size(mm)×Length(m)×Number			Outer peripheral sheet pile	;	φ1200	×	56.00	×	36 Piles	
				Partitioned sheet pile	;	φ1200	×	52.10	×	8 Piles	
	Steel Pipe Thickness	Outer peripheral sheet pile	Upper Pile	t = 14 mm (SKY490)							
			Lower Pile	t = 14 mm (SKY400)							
		Partitioned sheet pile	---	t = 14 mm (SKY400)							
Stability Calculation	Regular (Current River Bed)	δ	cm	0.11	≦	5.00	○	0.06	≦	5.00	○
		PNmax	KN/Number	1991	≦	3893	○	1990	≦	3893	○
		PNmin	KN/Number	1682	≧	-1959	○	1684	≧	-1959	○
	Seismic (Current River Bed)	δ	cm	2.51	≦	5.00	○	3.10	≦	5.00	○
		PNmax	KN/Number	1922	≦	5839	○	1924	≦	5839	○
		PNmin	KN/Number	1638	≧	-3344	○	1608	≧	-3344	○
Combined Stress (Seismic・Current River Bed)		SKY400	N/mm2	161.0	≦	210.0	○	194.3	≦	210.0	○
		SKY490	N/mm2	208.5	≦	277.5	○	239.6	≦	277.5	○

Source: JICA Study Team

4.2.11 Summary of Bridge Accessories Design

4.2.11.1 Design Calculation of Rocking Bearing and Bearing Support

(1) Design Conditions

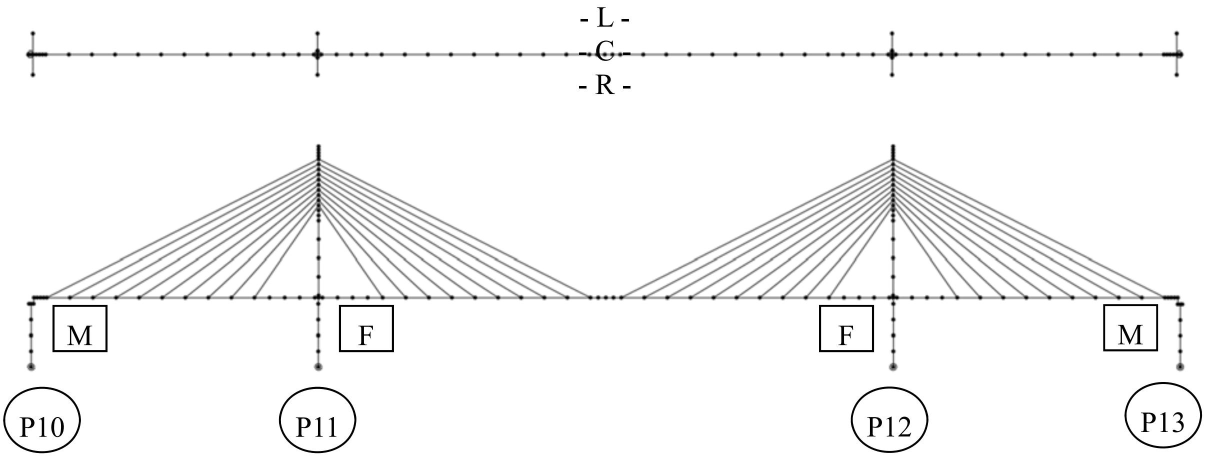
1) Support Conditions

The condition of the support in the cable-stayed bridge section is as listed in the table below.

Table 4.2.27 Condition of Support

	End Support Member: P10・P13				Center Support Member: P10・P13			
	Bearing Type	Bearing Condition			Bearing Type	Bearing Condition		
		Longitudinal	Transverse	Vertical		Longitudinal	Transverse	Vertical
L	Rocking Bearing	Movable	Movable	Fixed	Pin-Roller Bearing	Movable	Movable	Fixed
C	Horizontal Bearing	Movable	Fixed	Movable	Pivot Bearing	Fixed	Fixed	Fixed
R	Pendellosung	Movable	Movable	Fixed	Pin-Roller Bearing	Movable	Movable	Fixed

Source: JICA Study Team



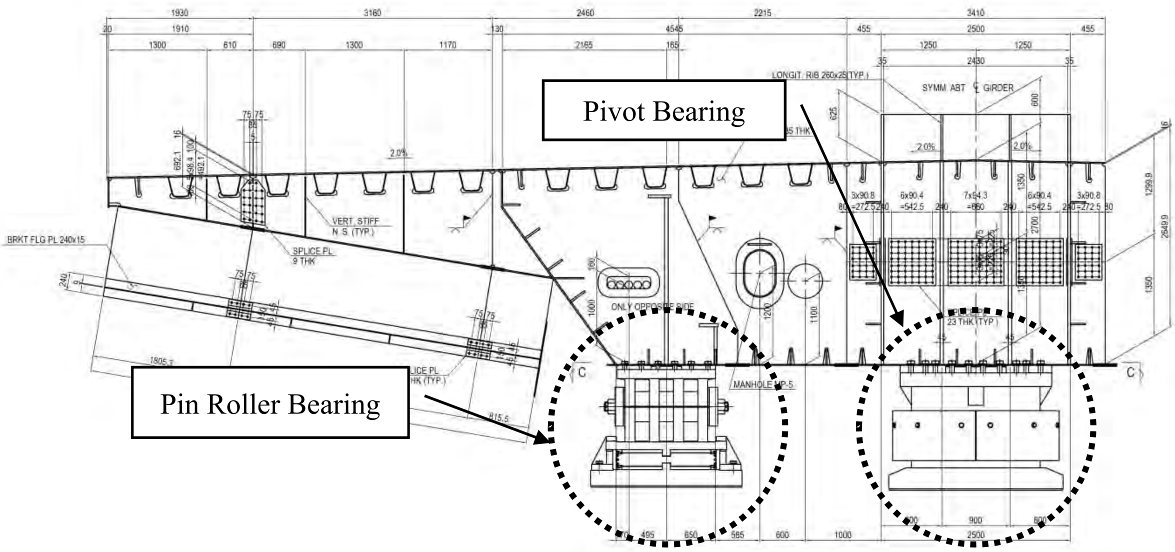
Source: JICA Study Team

Figure 4.2.33 Condition of Support

2) Structure of Bearings

The structure of the support section at each position is shown below.

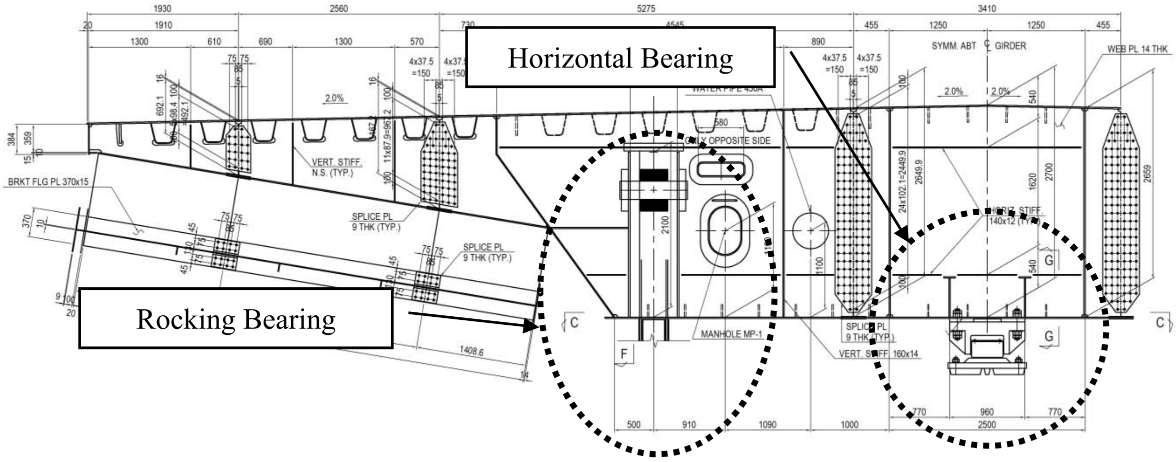
a) Support Section Underneath Main Tower



Source: JICA Study Team

Figure 4.2.34 Bearing Support under the Main Tower

b) Bearing at Ends



Source: JICA Study Team

Figure 4.2.35 Bearing Support at Girder End

3) Design Reaction Force of Bearing Support Section

The design reaction forces of the bearing sections are listed below.

(2) Design of Pivot Bearing

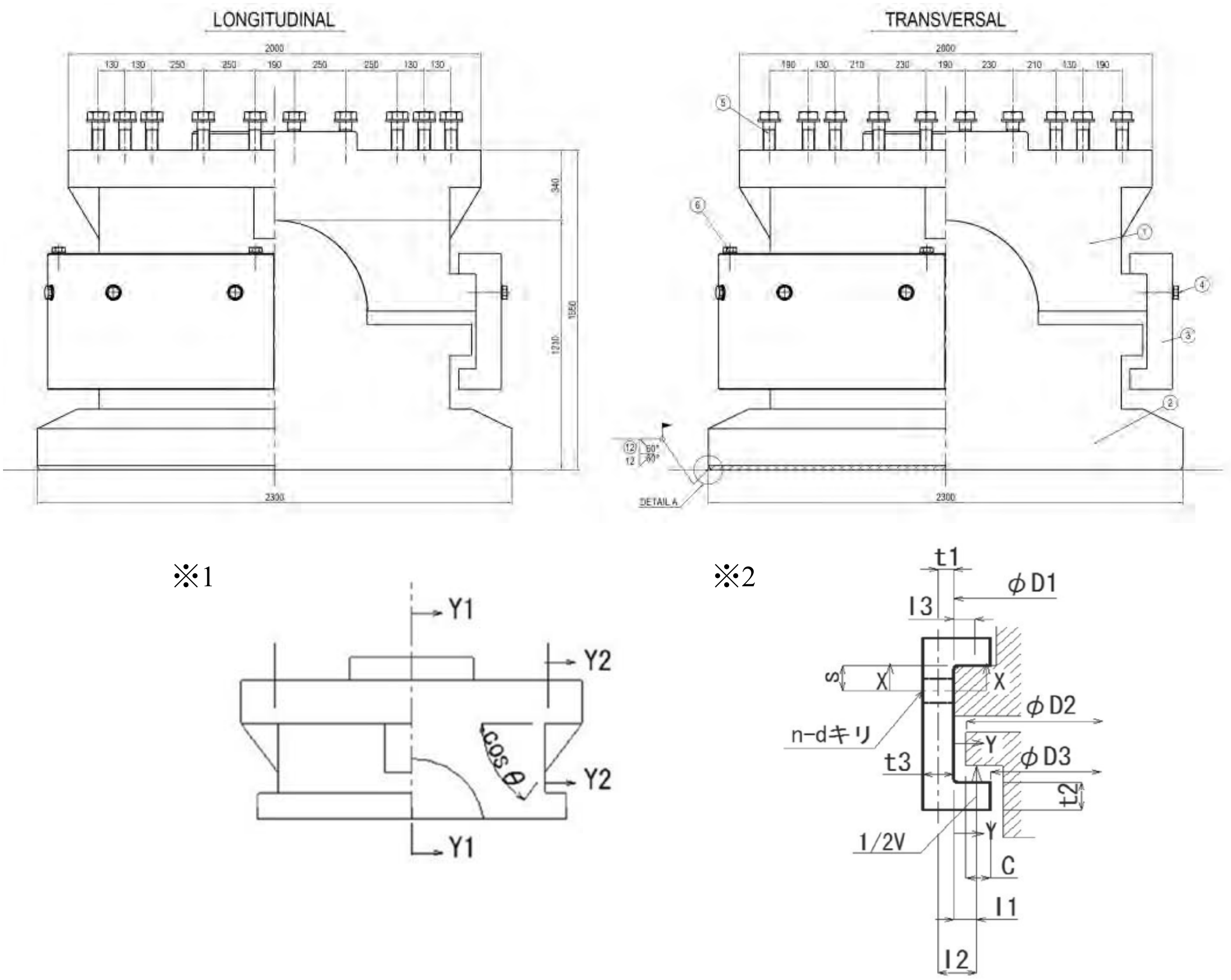
The results of the pivot bearing design are listed below.

Table 4.2.28 Design Calculation Results

Category				Units	Value		Allowable Value
Spherical Surface Section	Bearing Stress (Regular Scenario)			N/mm2	91.3	<	125.0
	Bearing Stress (Seismic Scenario)			N/mm2	97.5	<	425.0
Upper Shoe	Shear Stress Key	Bearing Stress		N/mm2	404.7	<	425.0
		Shearing Stress		N/mm2	51.5	<	170.0
	Bearing Stress between Supersturcutre	Bearing Stress (Regular Scenario)		N/mm2	16.5	<	250.0
		Bearing Stress (Seismic Scenario-Longitudinal)	Eccentricity	mm	485.3	>	381.7
			Bearing Stress	N/mm2	28.6	<	425.0
			Tensile Stress of Set Bolt	N/mm2	5.5	<	612.0
			Shearing Stress of Set Bolt	N/mm2	313.6	<	340.0
			Combined Stress of Set Bolt	N/mm2	0.9	<	1.2
		Bearing Stress (Seismic Scenario-Transverse)	Eccentricity	mm	369.1	<	381.7
			Bearing Stress	N/mm2	24.4	<	425.0
			Tensile Stress of Set Bolt	N/mm2	-	<	-
			Shearing Stress of Set Bolt	N/mm2	248.2	<	340.0
			Combined Stress of Set Bolt	N/mm2	-	<	-
	Bending Stress of Upper Shoe	Y1-Y1 Cross-Section (※1)	Bending Stress	N/mm2	127.9	<	153.0
		Y2-Y2 Cross-Section (※1)	Bending Stress	N/mm2	35.7	<	78.7
Lower Shoe	Bearing Stress between Substructure	Bearing Stress (Regular Scenario)		N/mm2	10.9	<	210.0
		Bearing Stress (Seismic Scenario- Longitudinal)	Eccentricity	mm	444.3	>	383.3
			Bearing Stress	N/mm2	18.2	<	315.0
			Shearing Stress from Tension on Weld	N/mm2	1.5	<	153.0
			Shearing Stress from Horizontal Force on Weld	N/mm2	135.4	<	153.0
			Combined Stress of Set Bolt	N/mm2	0.8	<	1.0
			Shearing Stress from Uplift Force	N/mm2	72.5	<	153.0
		Bearing Stress (Seismic Scenario-Transverse)	Eccentricity	mm	338.0	<	383.3
			Bearing Stress	N/mm2	16.4	<	315.0
			Shearing Stress from Tension on Weld	N/mm2	-	<	-
			Shearing Stress from Horizontal Force on Weld	N/mm2	107.0	<	153.0
			Combined Stress of Set Bolt	N/mm2	-	<	-
	Bending Stress of Lower Shoe			N/mm2	74.8	<	153.0
Ring	X-X Cross-Section (※2)	Tensile Bending Stress		N/mm2	234.6	<	289.0
	Y-Y Cross- Section(※2)	Bending Stress		N/mm2	135.7	<	289.0
		Shearing Stress		N/mm2	45.2	<	170.0
		Combined Stress		N/mm2	0.3	<	1.2
	C Member Bearing Stress	Bearing Stres		N/mm2	79.5	<	425.0
	Anchor Bolt	Tensile Stress		N/mm2	293.2	<	612.0
Set Bolt	Tensile Stress from Uplift Force			N/mm2	167.8	<	612.0

※Refer to the next page for cross-section position

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.36 Pivot Bearing Overview and Cross Section Location

(3) Design of Pin Roller Bearing

The results of the pin roller bearing design are listed below.

Table 4.2.29 Design Calculation Results - 1

Category				Unit	Value		Allowable value
Pin	Bearing Stress of Column Surface			N/mm2	72.7	<	125
	Stress by Horizontal Force in Transverse Direction	Tensile Stress		N/mm2	278.5	<	323
		Bearing Stress		N/mm2	362.2	<	425
		Shear Stress		N/mm2	138.5	<	170
Roller	Required Length			mm	589.6	<	1040
	Stress by Horizontal Force in Longitudinal Direction	Tensile Stress at Cutout Section		N/mm2	220.1	<	510
		Bearing Stress		N/mm	15318.3	<	25490
Upper Shoe	Projection of upper surface of upper shoe	Shear Stress Caused by Horizontal Force		N/mm2	44.6	<	170
		Shear Stress Caused by Horizontal Force		N/mm2	350	<	425
	Bearing Stress between Supersturcutre	Regular Scenario Bearing Stress		N/mm2	14.5	<	250
		Moving scenario bearing stress	Eccentricity	mm	25.5	<	216.1
			Bearing Stress	N/mm2	12.8	<	287.5
		Seismic Scenario Bearing Stress	Eccentricity	mm	232.2	<	233.3
			Bearing Stress	N/mm2	17.1	<	425
			Tensile Stress of Bolt	N/mm2	-	<	-
			Shear Stress	N/mm2	164.3	<	340
			Combined Stress	N/mm2	-	<	-
		Seismic Scenario （Transverse Direction）	Eccentricity	mm	2198	>	250
			Bearing Stress	N/mm2	92.7	<	425
			Tensile Stress of Bolt	N/mm2	549.76	<	612
			Shear Stress	N/mm2	164.3	<	340
			Combined Stress	N/mm2	1	<	1.2
	Bending stress	Center cross section	Bending Stress	N/mm2	149.9	<	153
		Ｙ2-Ｙ2 Cross section (※1)	Bending Stress	N/mm2	60.6	<	153
		Cross section in transverse direction	Bending Stress	N/mm2	169.8	<	289
Lower Shoe	Bensing Stress	Center cross section	Bending Stress	N/mm2	139.3	<	153
	Stopper	Stress by Horizontal Force in Transverse Direction	Bearing stress at Cutout Section	N/mm2	391.5	<	425
		Lower Shoe Bending Stress		N/mm2	176.6	<	289
		Lower Shoe Shear Stress		N/mm2	82.2	<	170
		Stress by Horizontal Force in Transverse Direction	Bending Stress	N/mm2	71.9	<	289
			Shear Stress	N/mm2	73.8	<	170
			Combined Stress	N/mm2	0.25	<	1.2
			Bearing Stress	N/mm2	326.3	<	425

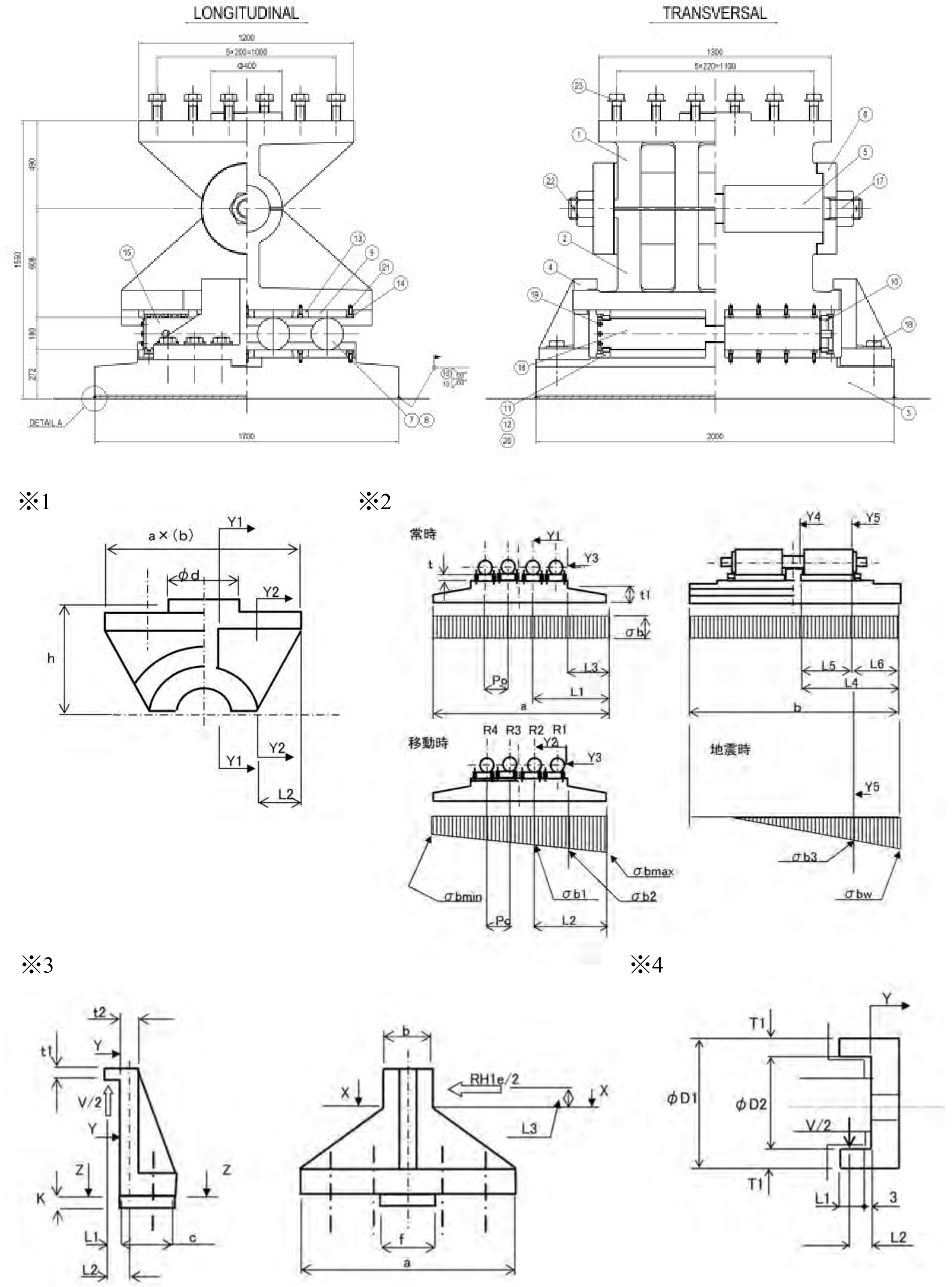
Source: JICA Study Team

Table 4.2.30 Design Calculation Results - 2

Category				Unit	Value		Allowable value
Bottom Board	Bearing Stress between Substurcutre	Regular Scenario Bearing Stress		N/mm2	6.12	<	210
		Moving scenario bearing stress		N/mm2	8.25	<	241.5
		Seismic Scenario Bearing Stress	Eccentricity	mm	587.6	>	283.3
			Bearing Stress	N/mm2	13.8	<	425
			Shear Stress Caused by Tension of Welded Section	N/mm2	11.6	<	136
			Shear Stress Caused by Horizontal Force	N/mm2	43.7	<	136
			Combined Stress	N/mm2	0.1	<	1
			Lift force Scenario	N/mm2	29	<	136
		Seismic Scenario （Transverse Direction）	Eccentricity	mm	2198	>	333.3
			Bearing Stress	N/mm2	21.7	<	357
			Shear Stress Caused by Tension of Welded Section	N/mm2	95.6	<	136
			Shear Stress Caused by Horizontal Force	N/mm2	43.7	<	136
			Combined Stress	N/mm2	0.6	<	1
	Bending Stress	Y1,2-Y1,2 Cross Section (※2)	Bending Stress	N/mm2	82.4	<	153
		Ｙ3-Ｙ3 Cross Section (※2)	Bending Stress	N/mm2	34.6	<	176
		Ｙ4,5-Ｙ4,5 Cross Section (※2)	Bending Stress	N/mm2	146.6	<	153
Side Block	Stress of Main Body	Stress on Y-Y Cross Section(※3)	Bending Stress	N/mm2	170.5	<	289
			Shear stress	N/mm2	62	<	170
			Combined Stress	N/mm2	0.48	<	1.2
		Tensile Bending Stress on X-X Cross Section(※3)		N/mm2	260.1	<	289
		Stress on X-X Cross Section (※3)	Bending Stress	N/mm2	88.9	<	289
			Shear Stress	N/mm2	98.8	<	170
			Combined Stress	N/mm2	0.43	<	1.2
		Stress on Z-Z Cross Section (※3)	Bending Stress	N/mm2	280.7	<	425
			Shear Stress	N/mm2	57.8	<	170
	Installing Bolt	Verification Considering Horizontal Force in Longitudinal Direction	Tensile Stress of Bolt	N/mm2	236.3	<	612
			Shear Stress	N/mm2	273.8	<	340
			Combined Stress	N/mm2	0.8	<	1.2
		Verification Considering Lift Force	Tensile Stress of Bolt	N/mm2	559.2	<	612
Cap	Bearing Stress			N/mm2	163.6	<	425
	Stress on Y-Y Cross Section (※4)	Bending Stress		N/mm2	180.5	<	425
		Shear Stress		N/mm2	91	<	161.5
		Combined Stress		N/mm2	0.71	<	1.2
Superstructure Installing Bolt	Tensile Force Caused by Lift Force			N/mm2	109.1	<	612
	Shear Stress			N/mm2	131.8	<	340

※See the next page for the cross-section position

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.37 Pin Roller Bearing Overview and Cross Section Location

(4) Design of Horizontal Bearing

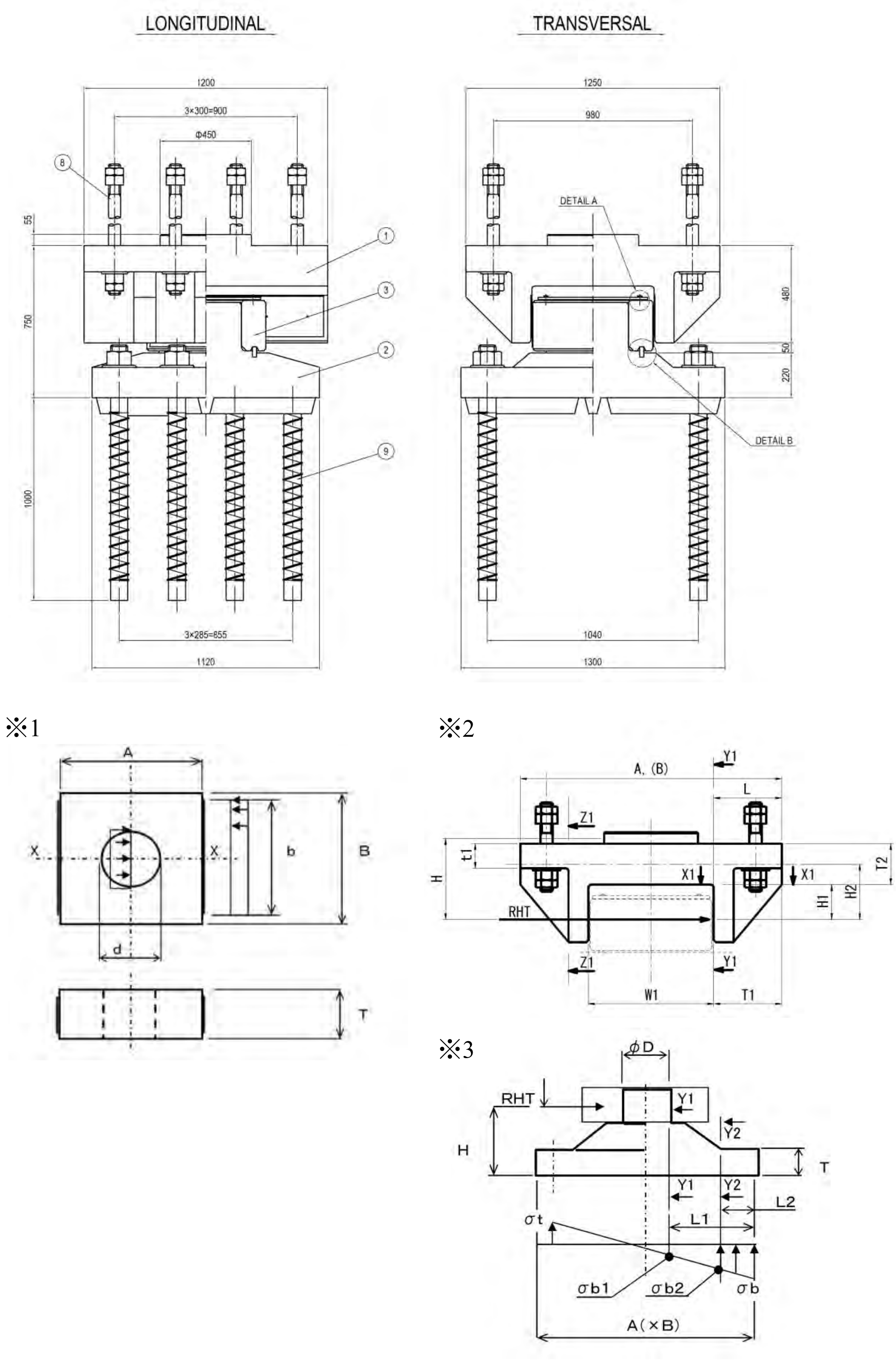
The results of the horizontal bearing design are listed below.

Table 4.2.31 Design Calculation Results

Category				Unit	Value		Allowable Value
Slide Slope	Bearing Stress			N/mm2	79.0	<	157.5
Collar	X-X Cross Section (※1)	Bending Stress		N/mm2	60.1	<	229.5
		Tensile Stress		N/mm2	225.2	<	229.5
Upper Shoe	Stress at Projection of upper shoe	Bearing Stress		N/mm2	297.8	<	375.0
		Shear Stress		N/mm2	42.1	<	150.0
	Stress of Main Body	Bearing Stress	Bearing Stress	N/mm2	49.3	<	323.0
			Tensile Stress of Bolt	N/mm2	532.0	<	799.0
		X1-X1 Cross Section (※2)	Bending Stress	N/mm2	234.9	<	255.0
			Shear Stress	N/mm2	56.8	<	150.0
			Combined Stress	N/mm2	1.0	<	1.0
		Y1-Y1 Cross Section (※2)	Bending Stress	N/mm2	199.3	<	255.0
		Z1-Z1 Cross Section (※2)	Bending Stress	N/mm2	216.7	<	255.0
Lower Shoe	Stress of Cylinder Section	Bearing Stress		N/mm2	93.4	<	375.0
		Foundation of Cylinder Section	Bending Stress	N/mm2	218.9	<	229.5
			Shear Stress	N/mm2	69.6	<	135.0
			Combined Stress	N/mm2	1.2	<	1.2
	Stress of Main Body	Bearing Stress	Bearing Stress	N/mm2	7.7	<	12.0
			Tensile Stress of Bolt	N/mm2	125.9	<	285.0
		Y1-Y1Cross Section (※3)	Bending Stress	N/mm2	96.7	<	229.5
		Y2-Y2 Cross Section (※3)	Bending Stress	N/mm2	57.0	<	230.0
Anchor Bolt	Shear Stress			N/mm2	147.6	<	165.0
	Bond Stress			N/mm2	2.3	<	2.4
	Combined Stress			N/mm2	1.0	<	1.2
Installing Girder Bolt	Tensile Stress			N/mm2	532.0	<	799.0
	Shear Stress			N/mm2	296.2	<	405.0
	Combined Stress			N/mm2	1.1	<	1.2

※See the next page for the cross-section position

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.38 Horizontal Bearing Overview and Cross Section Location

(5) Design of Rocking Bearing

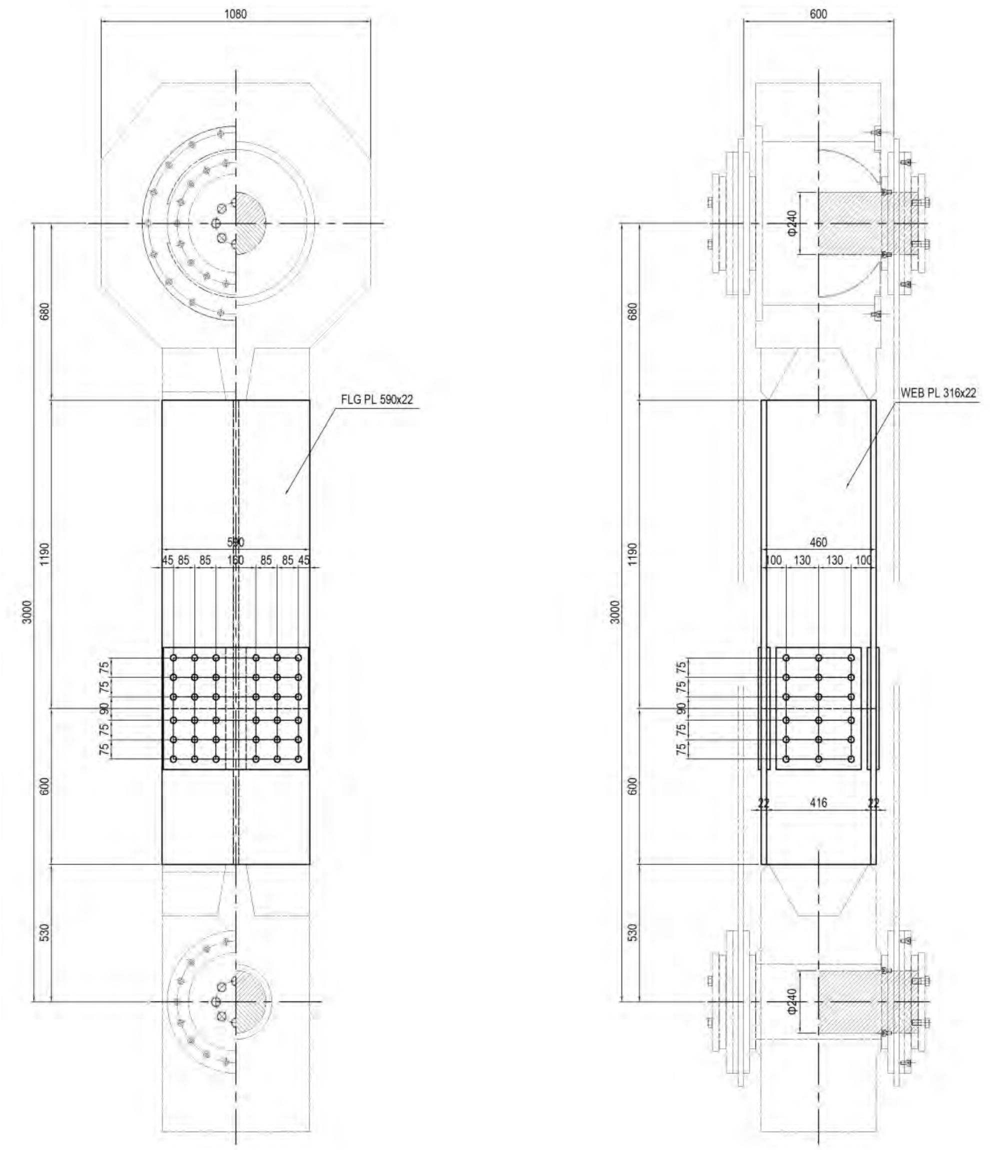
The results of the rocking bearing design are listed below.

Table 4.2.32 Design Calculation Results

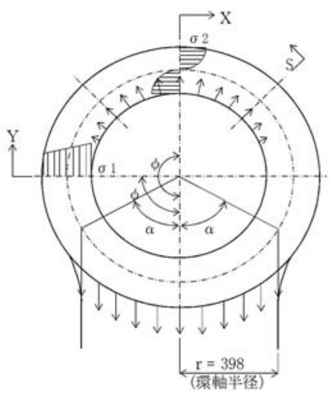
Category				Units	Value		Allowable Value
Endlink	Spherical Surface Bearing	Bearing Pressure（Internal Diameter）		N/mm2	41	<	50
	Spherical Bush Bearing	Maximum Bearing Stress at Center Cross Section		N/mm2	23	<	50
		Tensile Stress		N/mm2	55	<	290
	Pin	Bending Stress		N/mm2	262	<	290
		Shear Stress		N/mm2	57	<	160
	Anchor Structure at Upper Side	Curved Beam Calculation	Cross Section Y (※1)	N/mm2	132	<	153
			Cross Section X (※1)	N/mm2	80	<	153
		Shear Stress		N/mm2	37	<	90
	Anchor Structure at Lower Side	Curved Beam Calculation	Cross Section Y (※1)	N/mm2	101	<	153
			Cross Section X (※1)	N/mm2	49	<	102
		Shear Stress		N/mm2	40	<	60
Rocking Bearing	Tie Bar	Axial Compressive Stress		N/mm2	105	<	131
	Support Beam	Stress (Compression)	σu	N/mm2	73	<	207
			σl	N/mm2	85	<	210
		Stress (Tension)	σu	N/mm2	87	<	210
			σl	N/mm2	102	<	169
	Base of Beam Post	Design as Column	Axial Compressive Stress	N/mm2	47	<	210
			Bearing Stress	N/mm2	129	<	315
		Design as Beam	σ	N/mm2	3	<	210
			τ	N/mm2	9	<	120
	Base Plate	σ		N/mm2	155	<	210
		τ		N/mm2	21	<	120
	Anchor Bolt	σs		N/mm2	204	<	210
	Anchor Frame	Shear Stress of Web		N/mm2	37	<	120
		Compressive Stress of Diaphragm		N/mm2	108	<	210
		Stress of Flange	(A), (B) Panel Combined Stress	N/mm2	96	<	210
			(C), (D) Panel Combined Stress	N/mm2	27	<	210
			(E), (F) Panel Combined Stress	N/mm2	158	<	210

※See the next page for the cross-section position

Source: JICA Study Team



※1



Source: JICA Study Team

Figure 4.2.39 Rocking Bearing Overview and Cross Section Location

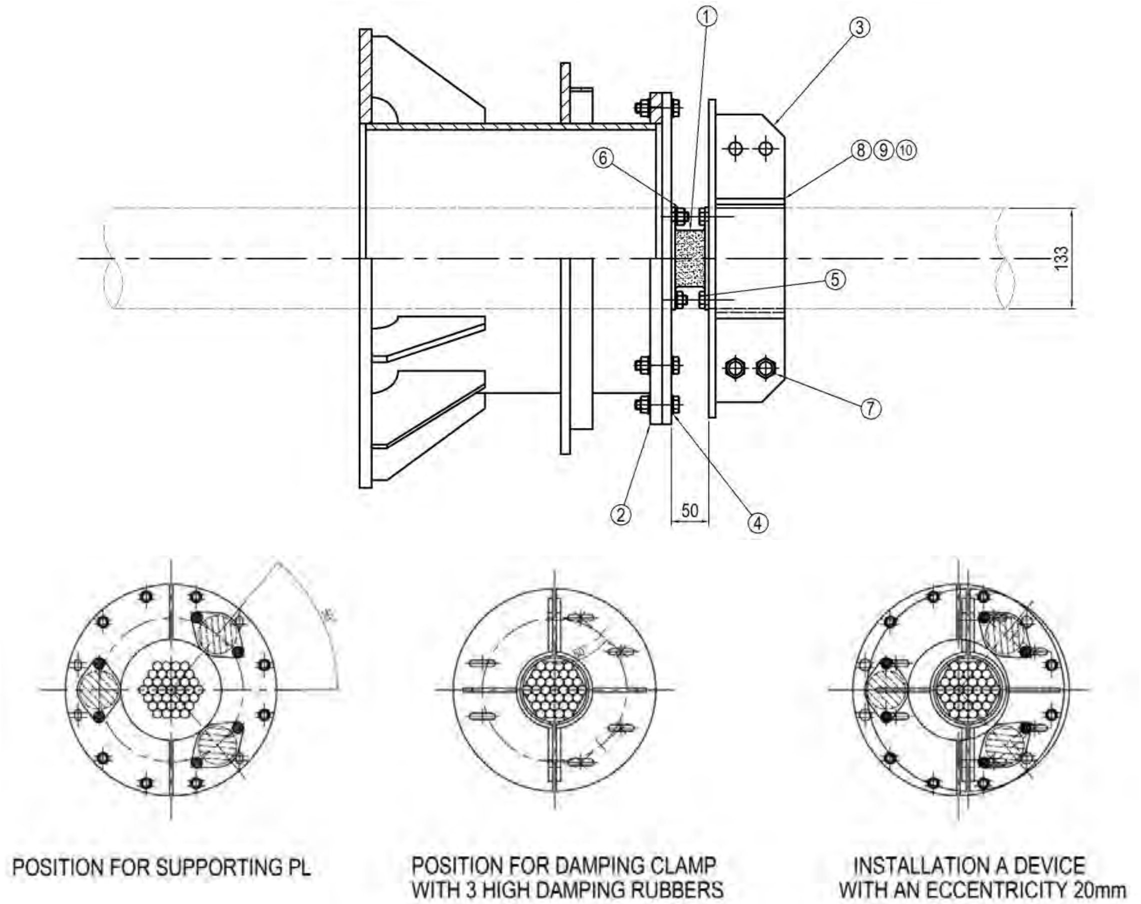
4.2.11.2 Study on Cable Damping Device

(1) Design Overview

Due to the exposure to constant winds, the cables of the cable-stayed bridge are said to be subjected to aerodynamically unstable oscillations, as stated below, which may lead to problems of fatigue at the cable ends. In this study, as a countermeasure for the aerodynamically unstable oscillation [1) Vortex induced vibration, 2) Rain-wind induced vibration] of the cable, the specifications of the apparatus and its damping effects when using high damping rubber damper were investigated.

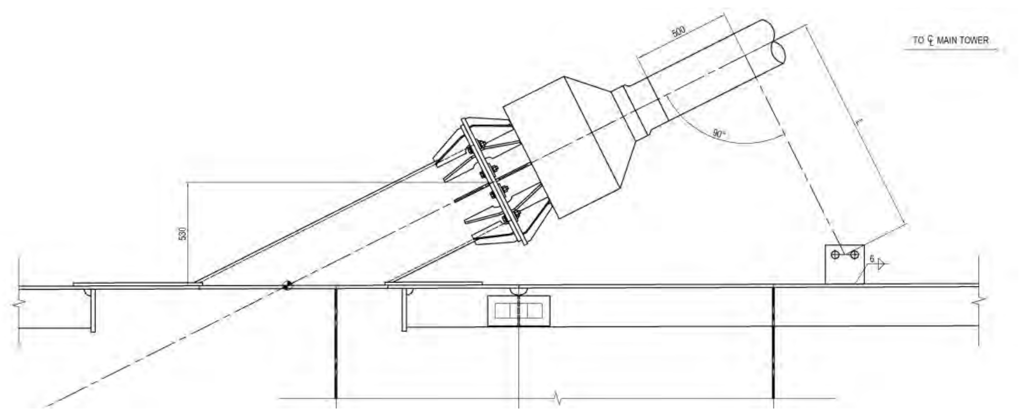
(2) Installation of Mitigation Apparatus

The mitigation apparatus was installed as shown in the figure below. The fitting metals for the rod-type vibration mitigation apparatus were attached on the girder in case of vibration after completion.



Source: JICA Study Team

Figure 4.2.40 Installed Mitigation Apparatus



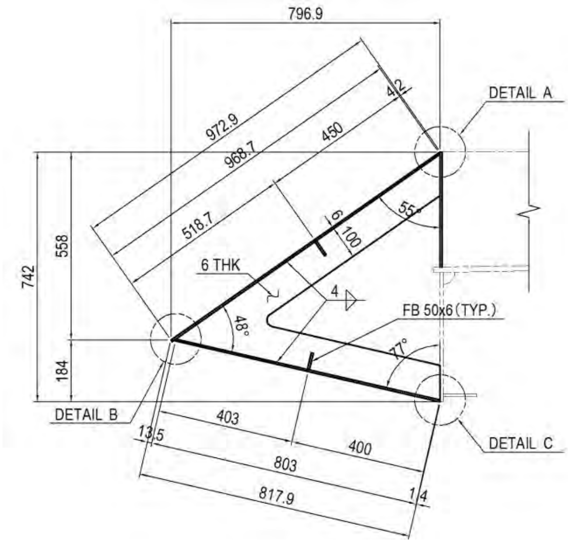
Source: JICA Study Team

Figure 4.2.41 Fitting Metal for Rod-type Mitigation Apparatus

4.2.11.3 Main Body Design of Fairing

(1) Fairing Shape

The fairing is installed at the girder in order to improve the wind resistance of the bridge. The fairing shape was referred from past cases and the wind stability was checked by wind tunnel test.



Source: JICA Study Team

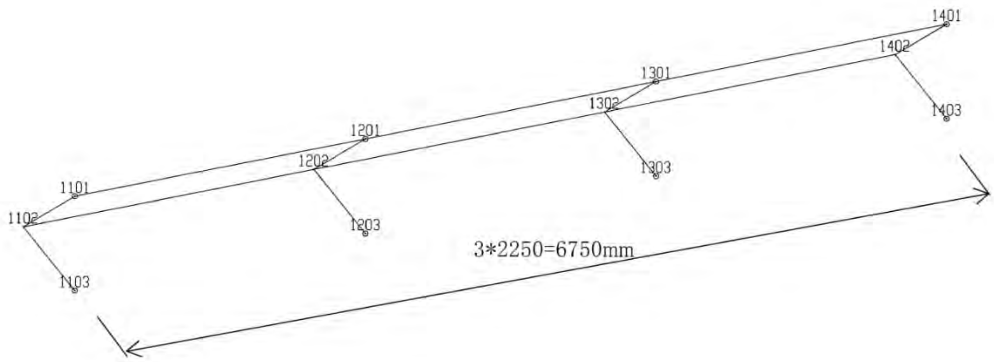
Figure 4.2.42 Fairing Shape

(2) Design Method

Design calculation is performed by applying wind and dead load.

The section force of the fairing member is calculated by applying the space frame model shown below.

By referring to past records of cable-stayed bridges, the fairing plate thickness is set to 6 mm.



Note: The interval of frame panel is 2250 mm of the maximum transverse rib interval

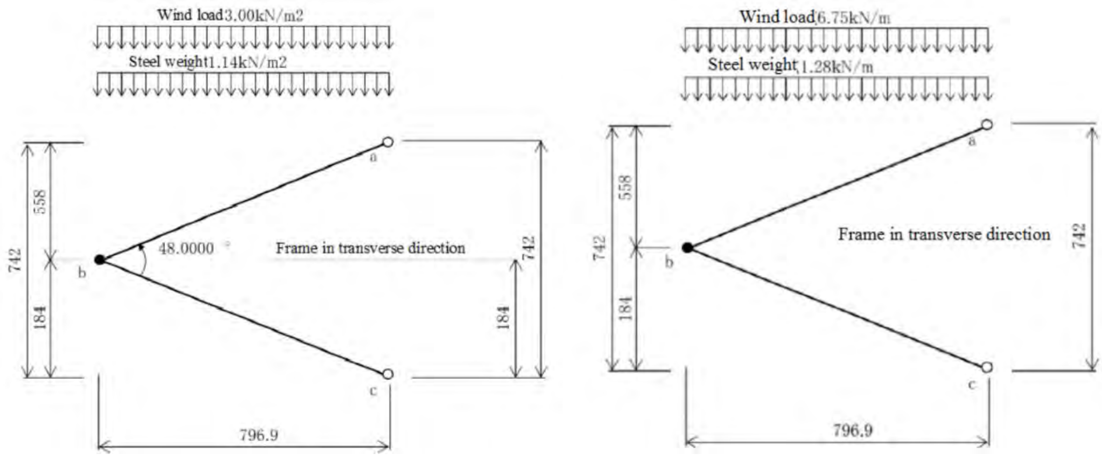
Source: JICA Study Team

Figure 4.2.43 Space Frame Model

(3) Design Load

The section force used for the design of the longitudinal member is determined by loading the surface load on the upper surface (a-b) of the space frame.

The section force used for the design of the transverse member is determined by loading the line load on the transverse frame of the space frame as shown below.



Source: JICA Study Team

Figure 4.2.44 Space Frame Model

The overdesign factor of allowable stress is 1.25 for the steel weight + wind load.

As the section force of the member against the wind load from the side is smaller than that from the perpendicular direction, the calculation of the section force of the member against the wind load from the side is omitted.

4.2.11.4 Design of Expansion Joint

(1) Design Conditions of EJ-1 (P10)

The design conditions for the expansion joint are listed in the table below.

Table 4.2.33 Design Conditions

Item	Left Girder（P9 side）	Right Girder（P11 side）
Type of bridge	Steel deck slab girder	Steel deck slab girder
Temp range	0 ℃～50 ℃	0 ℃～50 ℃
Load	72.5 kN for back wheel	

Source: JICA Study Team

(2) Selection of Expansion Joint Type

The design expansion amount is determined for the regular condition as:

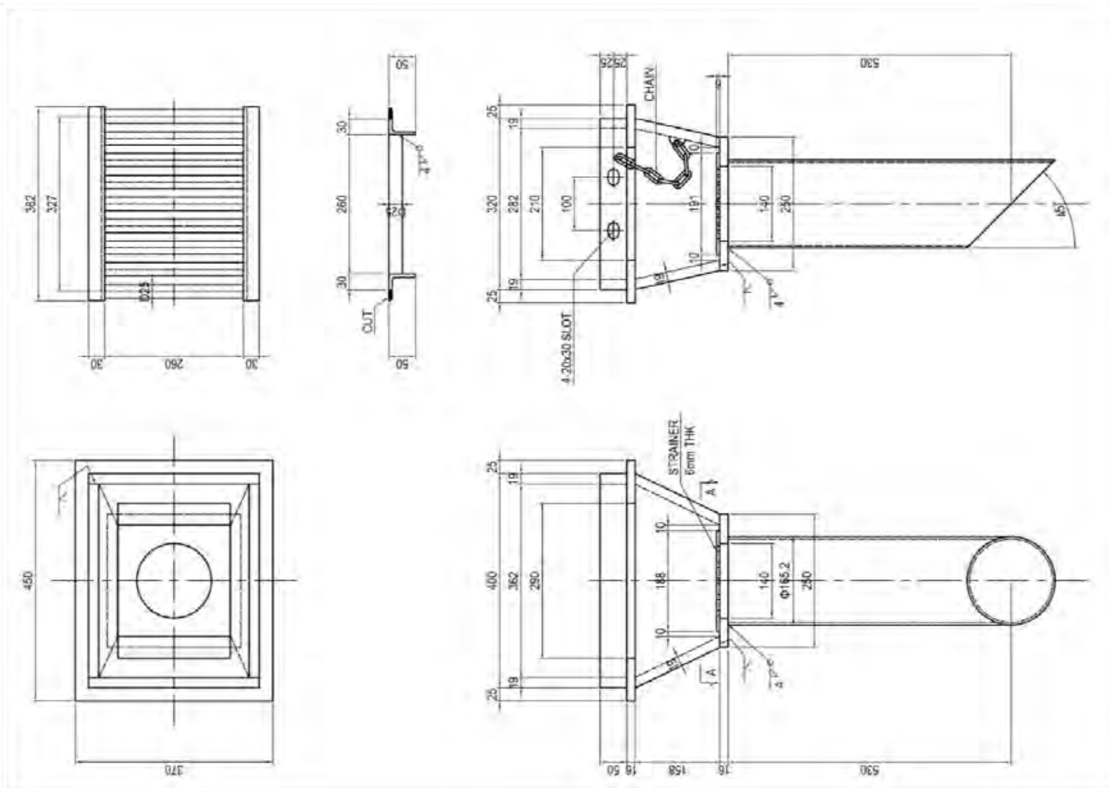
ΔLj：269 mm ＜ ΔLq：568 mm

Due to the design expansion amount, the modular type joint (maximum design movement of 640 mm) was selected.

4.2.11.5 Drainage Device

(1) Catch Basin Shape

The catch basin shape is shown in the figure below.

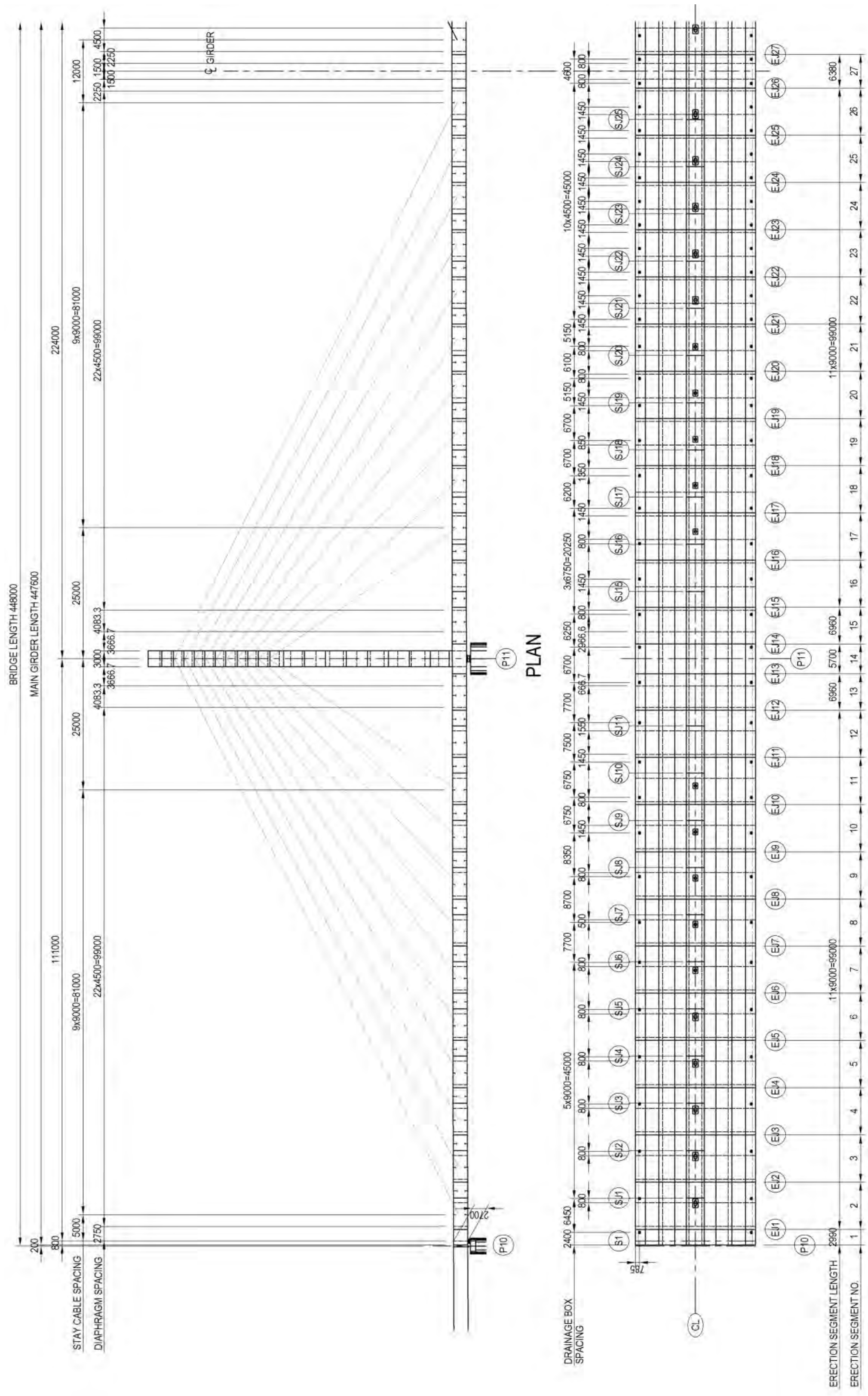


Source: JICA Study Team

Figure 4.2.45 Catch Basin Shape

(2) Catch Basin Arrangement

The position of the catch basin is shown below:



Source: JICA Study Team

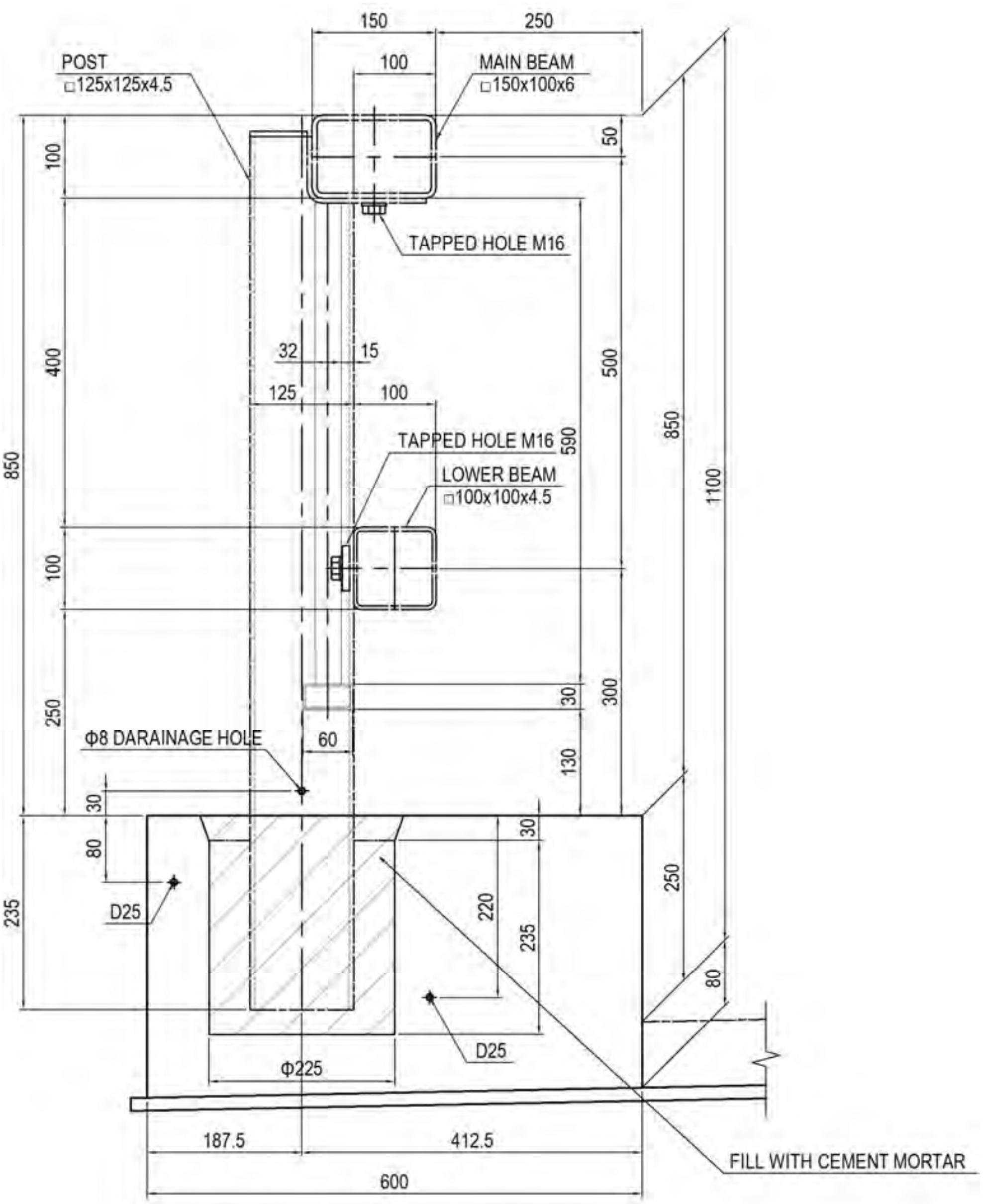
Figure 4.2.46 Catch Basin Location

4.2.11.6 Guardrail

(1) Specifications of Guardrail

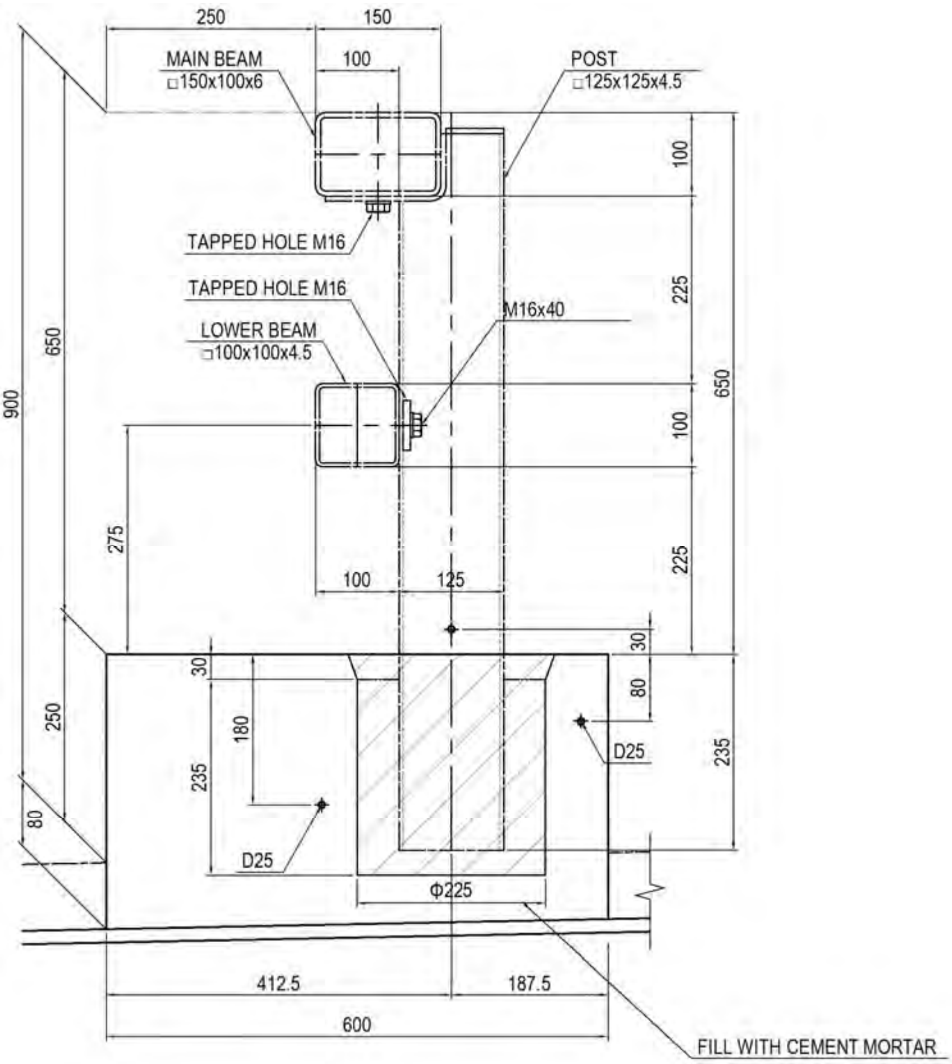
The Type-A combination railing (steel) which is shown in the standard drawings for Ministry of Land, Infrastructure, Transport and Tourism Hokuriku Regional Development Bureau was selected. The specifications of guardrail are as follows:

- Post interval : 2.0 m shall be set as the standard.
- Height of guardrail (Outer Side) : 1.1 m from bridge surface
  (Median Side) : 0.9 m from bridge surface



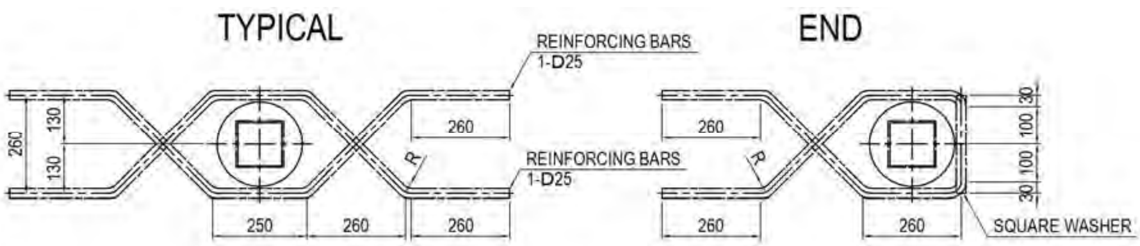
Source: JICA Study Team

Figure 4.2.47 Detailed Plan of Guardrail (Outer Side)



Source: JICA Study Team

Figure 4.2.48 Detailed Plan of Guardrail (Median Side)



Source: JICA Study Team

Figure 4.2.49 Reinforcing Steel

4.2.11.7 Design of Base for Miscellaneous Items

(1) Base for Road Lighting Pole

The road lighting pole weight was assumed as shown below, and the design of the base was performed. The calculation results are as follows:

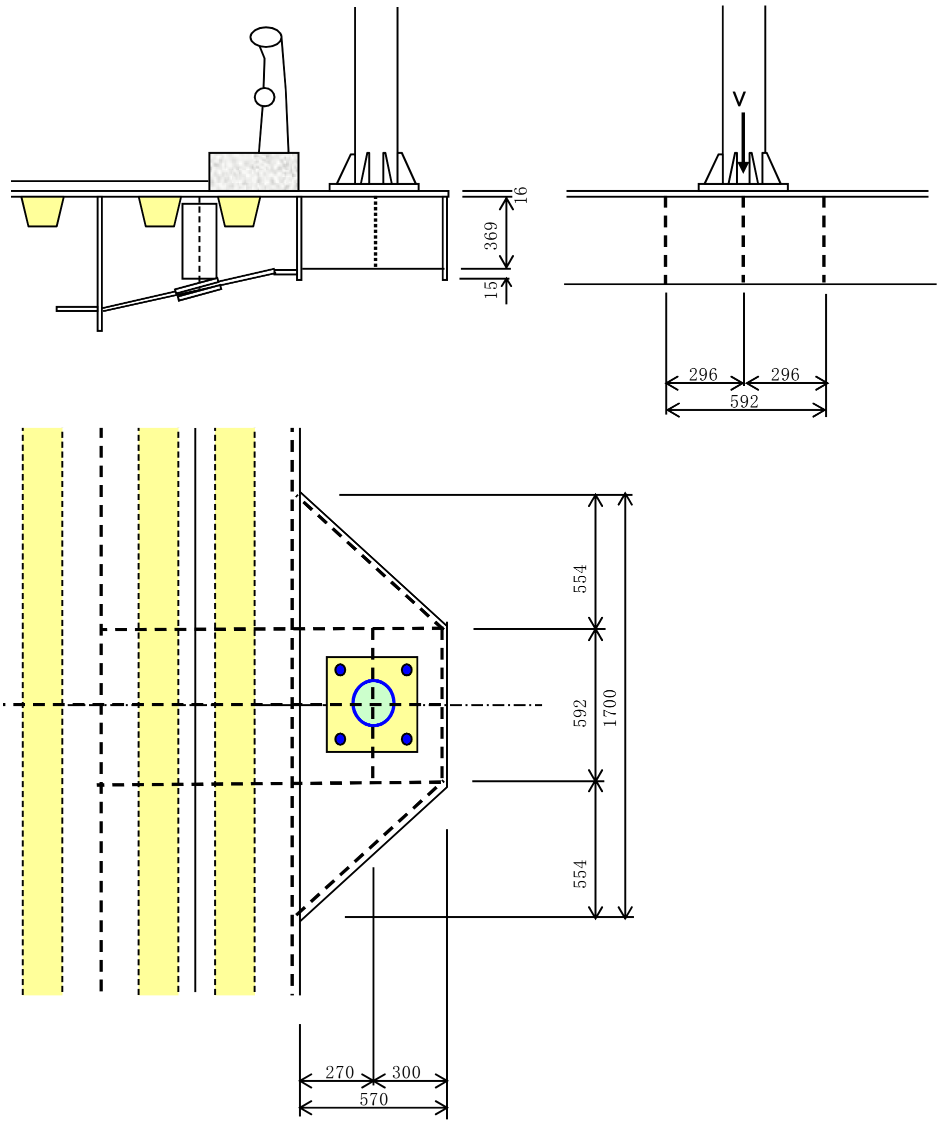
1) Design Load

The assumed weight of the road lighting pole is:

12 m lighting pole (assumed weight) V = 1.900 kN (about 190 kg)

2) Design of Base

$$M = 1.900 \times 0.270$$
$$= 0.513 \text{ kNm}$$
$$S = 1.900 \text{ kN}$$



Source: JICA Study Team

Figure 4.2.50 Base for Road Lighting Pole

(2) Base for Navigation Sign

The navigation sign weight was assumed as shown below, and the design of the base was performed. The calculation results are as follows:

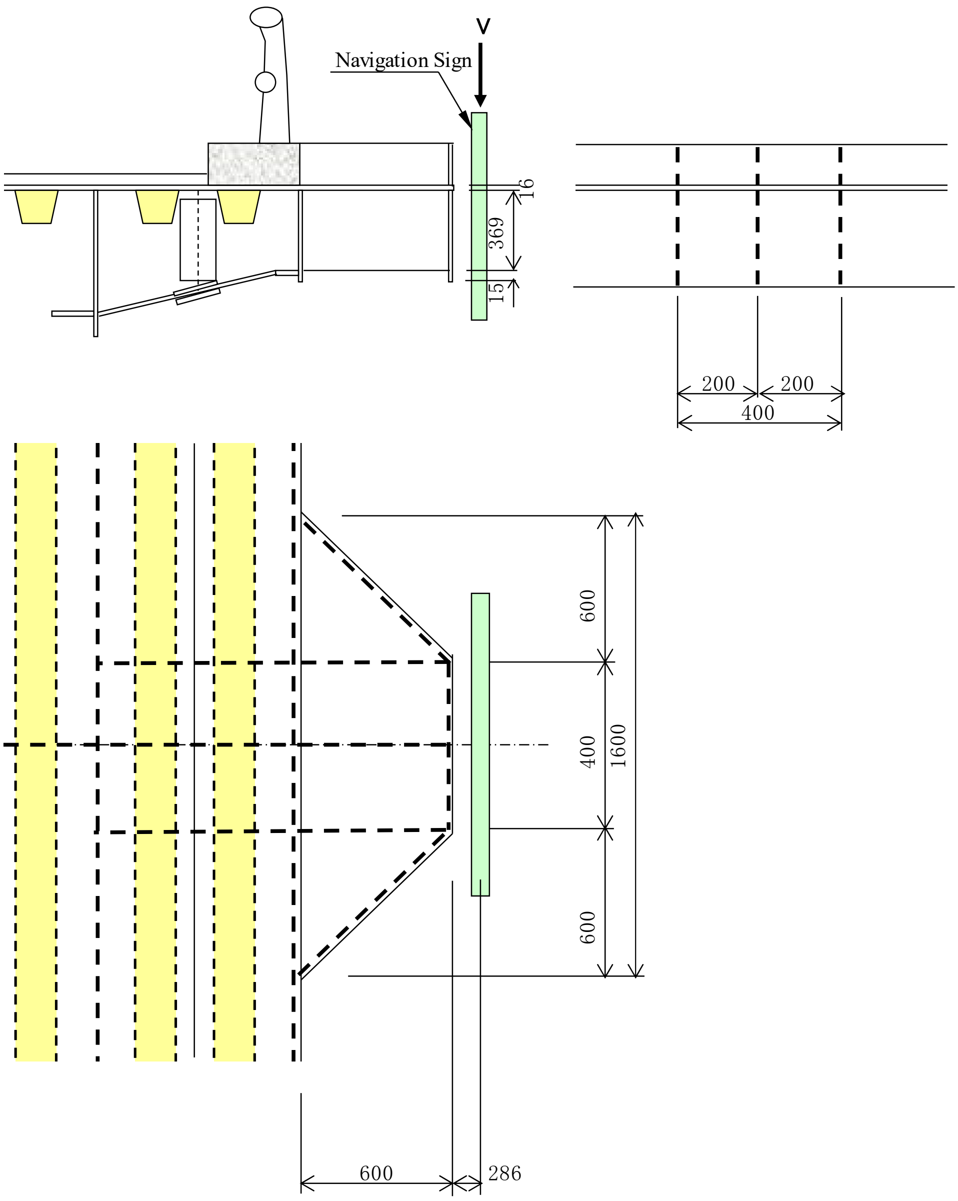
1) Design Load

The assumed weight of the navigation sign is:

Navigation sign (assumed weight) V = 1.000 kN (about 100 kg)

2) Design of Base

$$
\begin{aligned}
M &= 1.000 \times 0.886 \\
&= 0.886 \text{ kNm} \\
S &= 1.000 \text{ kN}
\end{aligned}
$$

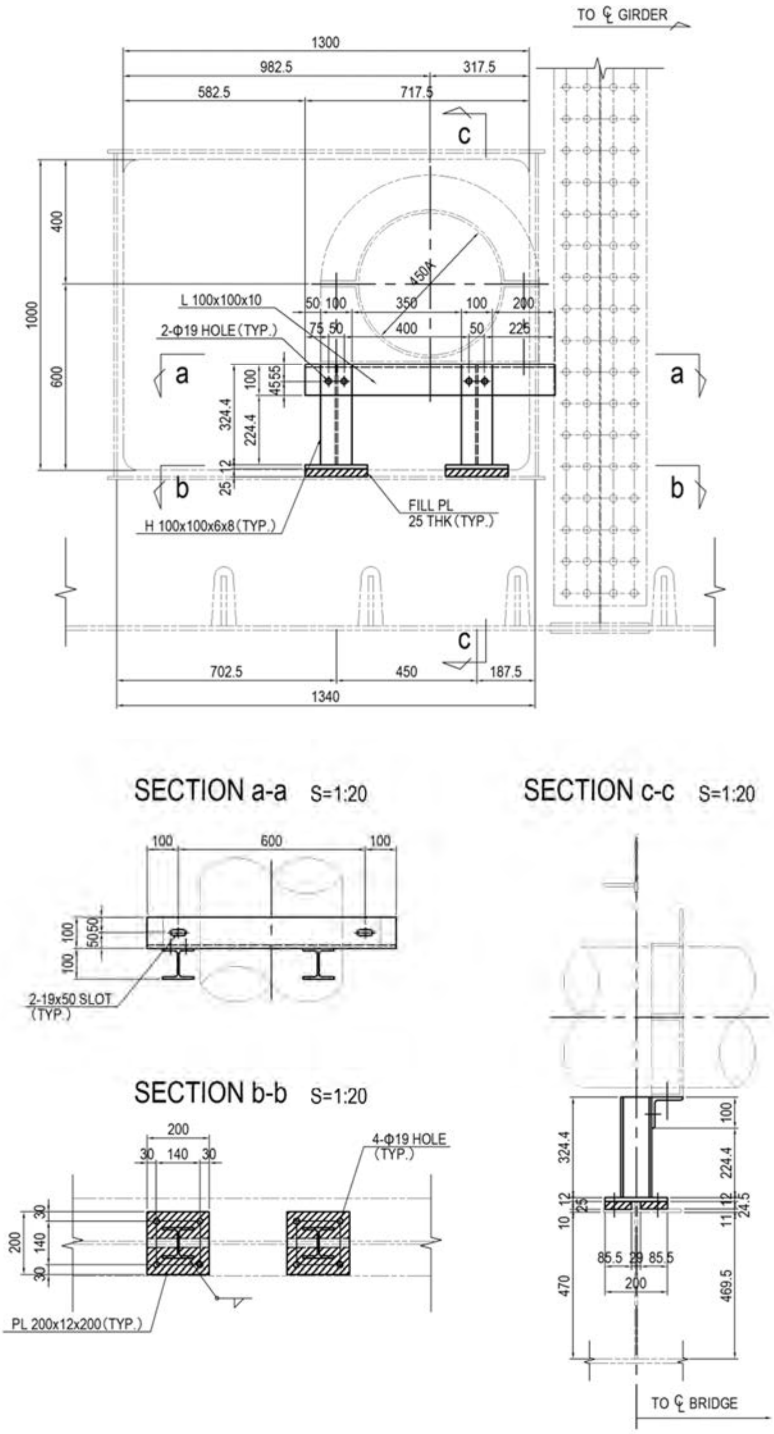


Source: JICA Study Team

Figure 4.2.51 Base for Navigation Sign

(3) Support for Water Pipe

The water pipe weight (full water) was assumed as shown below, and the design of the water pipe support was performed. The calculation results are as follows:

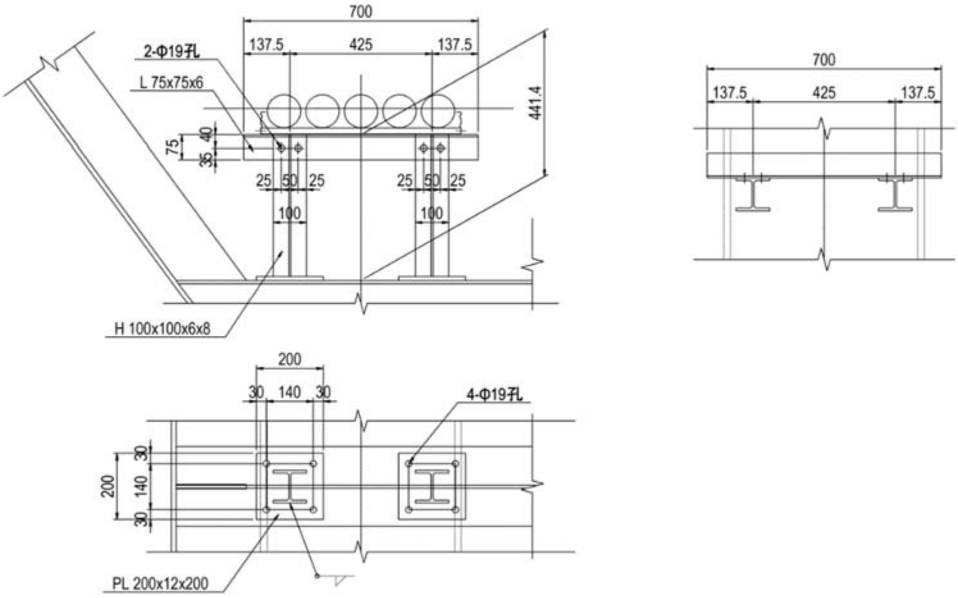


Source: JICA Study Team

Figure 4.2.52 Water Pipe Support

(4) Support for Electrical Cables

The electrical cable weight was assumed as shown below, and the design of the electrical cable support was performed. The calculation results are as follows:



Source: JICA Study Team

Figure 4.2.53 Electrical Cable Support

4.2.11.8 Maintenance Equipment

(1) Inspection Facility Plan

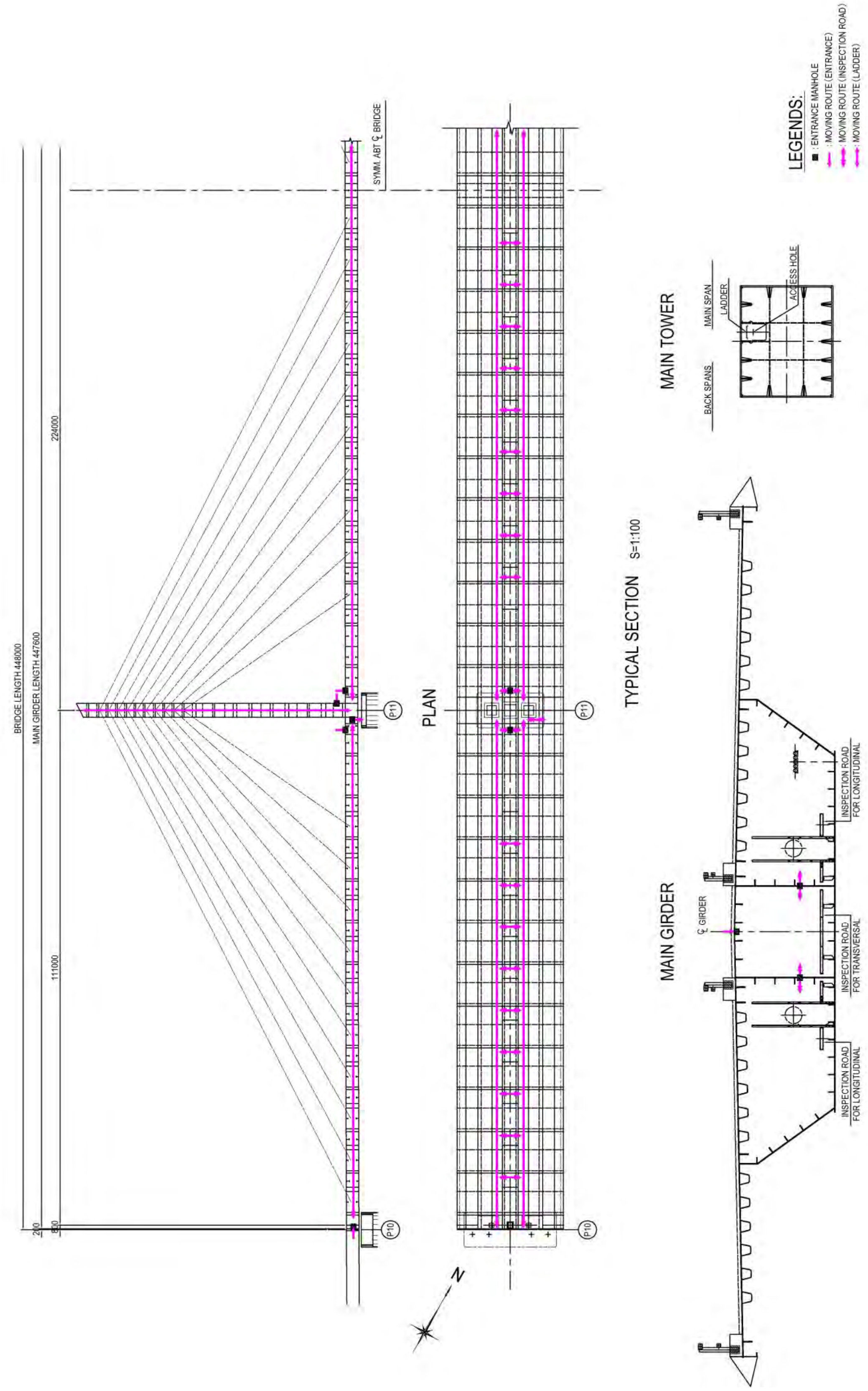
Based on the inspection facility arrangement plan listed below, the installation of the inspection facility at the necessary positions are examined.

Table 4.2.34 Inspection Facility Arrangement Plan

Inspection Point	Description	Note
Girder undersurface	Install inspection car rail for inspection and maintenance using girder undersurface inspection car*. Install scaffolding mountable temporary suspenders.	* Checked with assumed load for the inspection car
Inside girder	Install inspection roads and ladders. Install manholes at necessary positions.	
Tower outer surface	Install base plates for inspection and maintenance using gondola*.	* Checked with assumed load for the gondola
Inside tower	Install ladders. Install access ladders to link the inside of the girder to the inside of the tower. Install manholes at necessary positions.	
Top of pier	Install handrails at the top of the pier. Side pier*: Install access ladders from bridge face to pier top. Tower pier: Install access ladders from girder face to pier top.	* The access ladders at the side piers shall be installed at the adjacent bridge

Source: JICA Study Team

The maintenance route is shown in the figure below.



Source: JICA Study Team

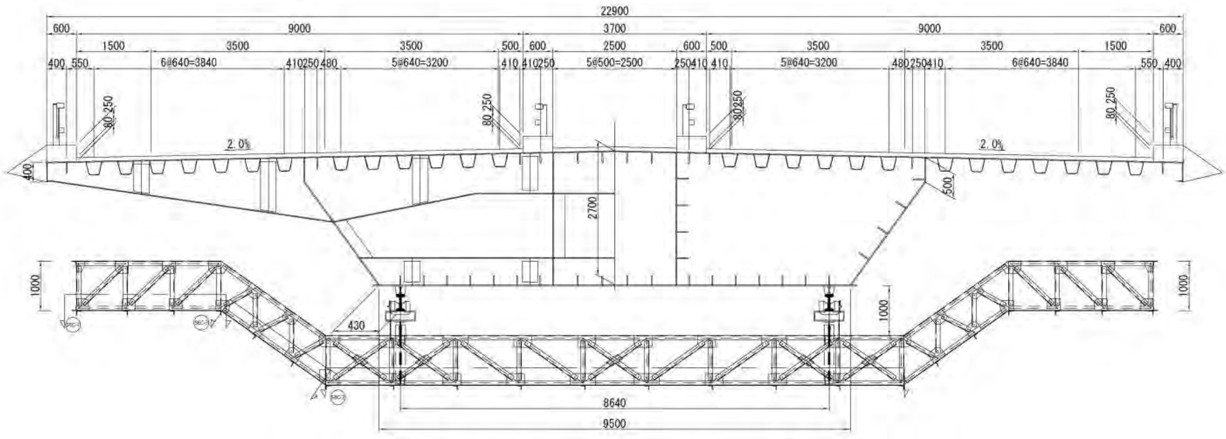
Figure 4.2.54 Maintenance Route

(2) Examination of Inspection Car Rail (Reference)

1) Examination Overview

The examination of the rail for the girder undersurface inspection car was performed.

The outline for the assumed inspection car is shown below.



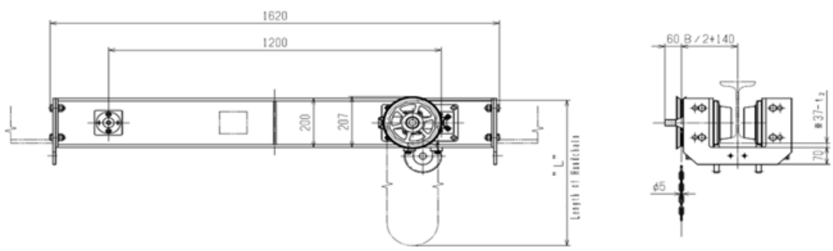
Source: JICA Study Team

Figure 4.2.55 Girder Undersurface Inspection Car

2) Design Conditions

- The self-weight of the inspection car was set by referring to the inspection car at the Thanlyin Bridge.
- The assumptions made for the saddle of the inspection car is as listed below.
  - Number of suspension points : 4 Points
  - Number of wheels per suspension point : 2 wheels
  - Suspension point interval Long. Direction : 1.2m

    Trans. Direction : 8.64m
  - Maximum load of one suspension point : 34.3kN (3.5t)
  - Sample image of saddle



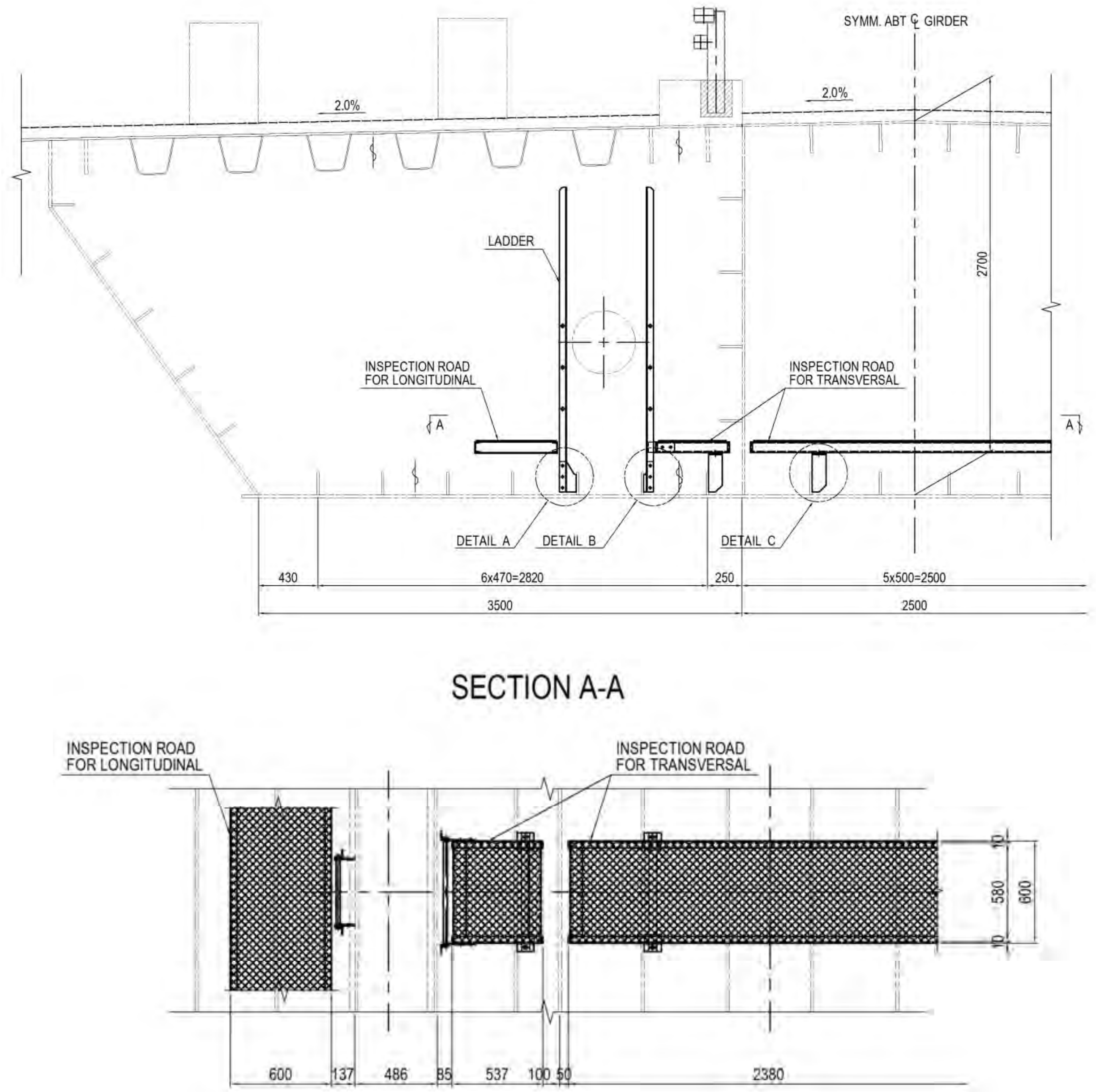
Note: The detailed figure of the saddle shall be treated as reference.

Source: JICA Study Team

Figure 4.2.56 Sample Image of Saddle

(3) Inspection Route inside Girder

The inspection route inside the girder is shown below.

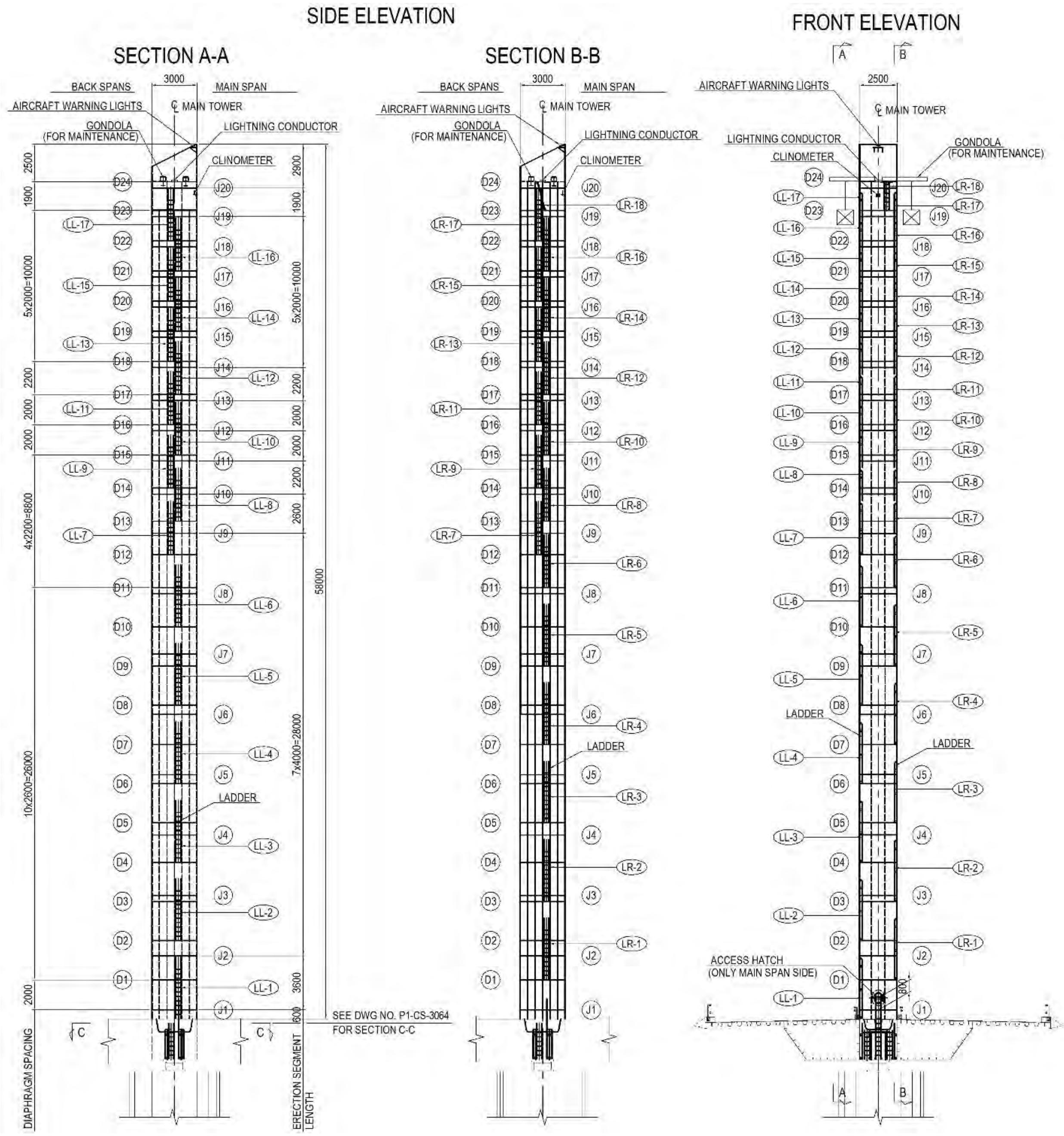


Source: JICA Study Team

Figure 4.2.57 Inspection Route inside Girder

(4) Inspection Route inside Tower

The shaft ladders inside the tower is as shown below.

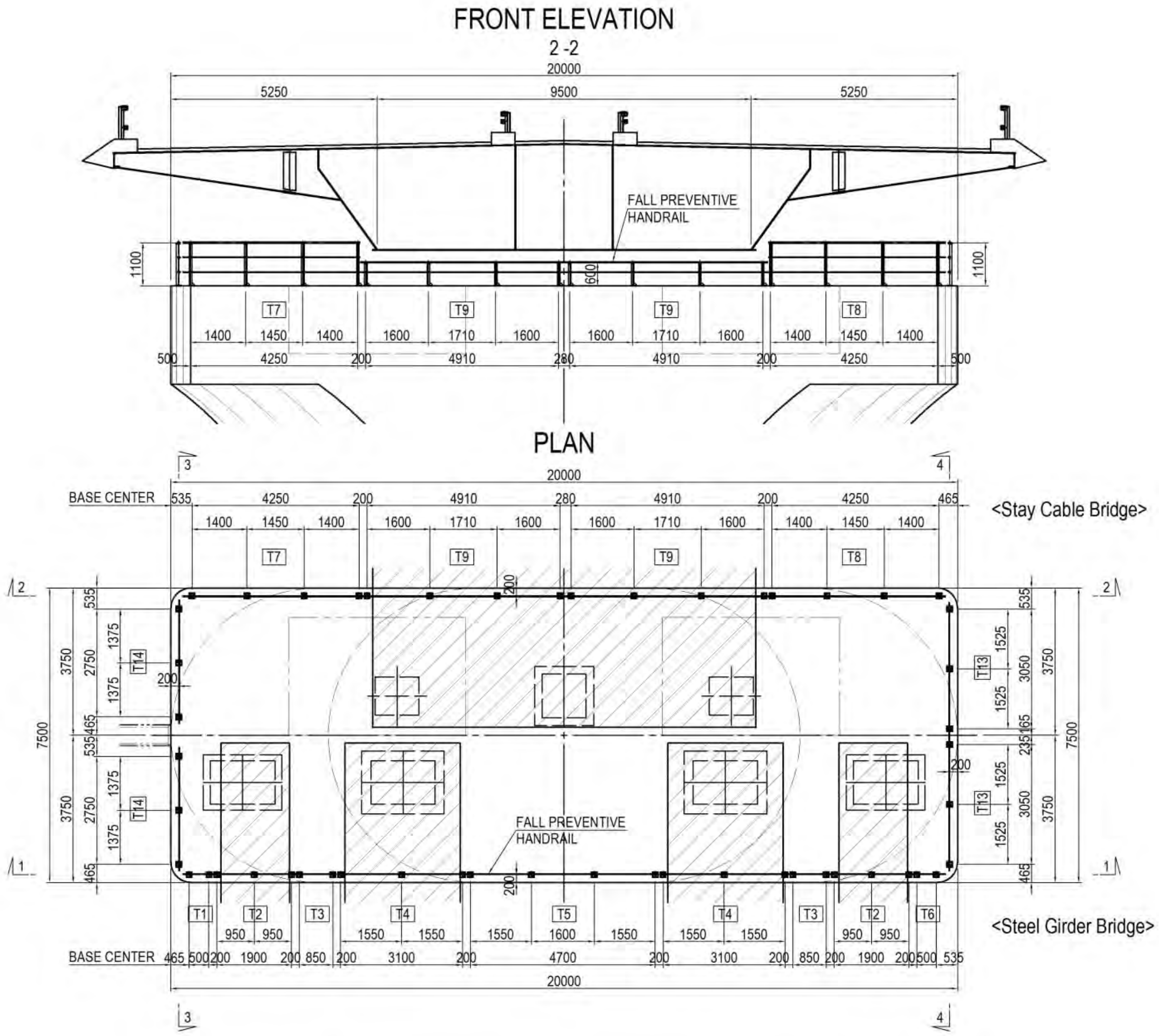


Source: JICA Study Team

Figure 4.2.58 Shaft Ladders inside Towers

(5) Fall Preventive Handrail at Pier Top

Fall preventive handrails for inspection and maintenance of bearings and pier top are installed.



Source: JICA Study Team

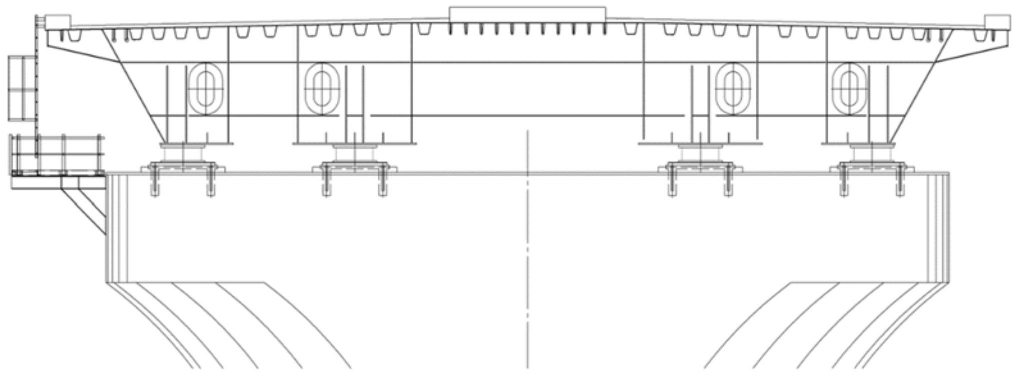
Figure 4.2.59 Fall Preventive Handrail at Pier Top

(6) Shaft Ladder

The shaft ladders to the top of the piers are installed as shown below.

1) Side Pier

The access ladders shall be installed linking the bridge face to the pier top as shown below. (The ladders shall be installed at the adjacent bridge).

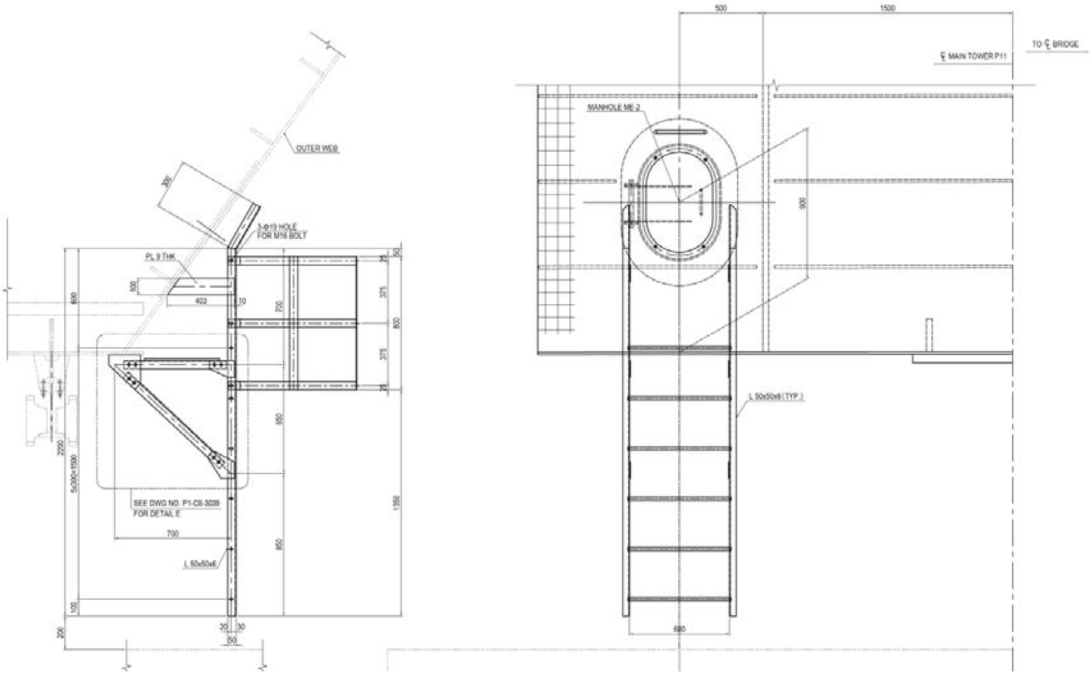


Source: JICA Study Team

Figure 4.2.60 Shaft Ladder (Side Pier)

2) Tower Pier

The access ladders shall be installed linking the inside of the girder to the pier top.



Source: JICA Study Team

Figure 4.2.61 Shaft Ladder (Tower Pier)

4.2.12 Summary of Seismic Analysis

4.2.12.1 Dynamic Analysis of Overall Structure

The purpose of the analysis is to observe the behavior of the main section, i.e., the cable-stayed bridge, during an earthquake. The static analysis of seismic design is shown in "Section 4.2.9.7 Static Structure Analysis" of this report.

(1) Non-Linear Dynamic Analysis Conditions

1) Outline of Structure

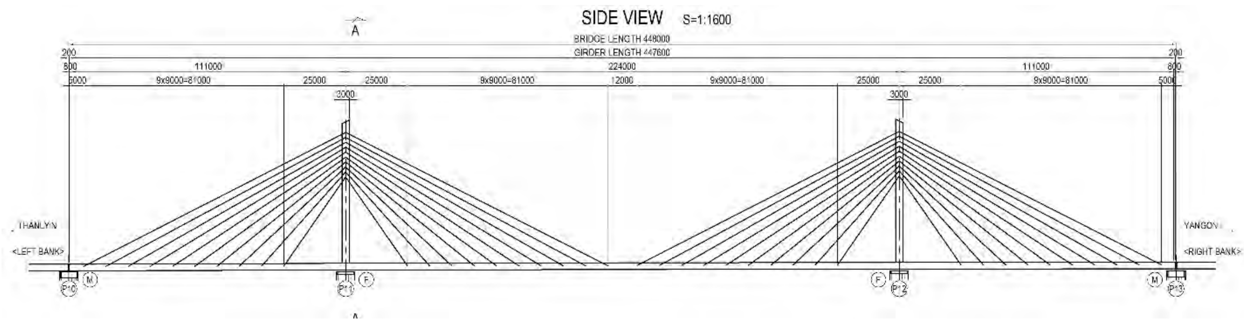
a) Structure Type

- Superstructure three-span continuous cable-stayed bridge
- Substructure Reinforced concrete single column type pier
- Foundation Type P10 Pier: Steel pipe sheet pile foundation
  P11 Pier: Steel pipe sheet pile foundation
  P12 Pier: Steel pipe sheet pile foundation
  P13 Pier: Steel pipe sheet pile foundation

b) Bearing Support Condition

- P10 Pier: Movable (Fixed on transverse direction) Rocking Bearing
- P11 Pier: Fixed (Fixed on transverse direction)
- P12 Pier: Fixed (Fixed on transverse direction)
- P13 Pier: Movable (Fixed on transverse direction) Rocking Bearing

c) Structural Plan

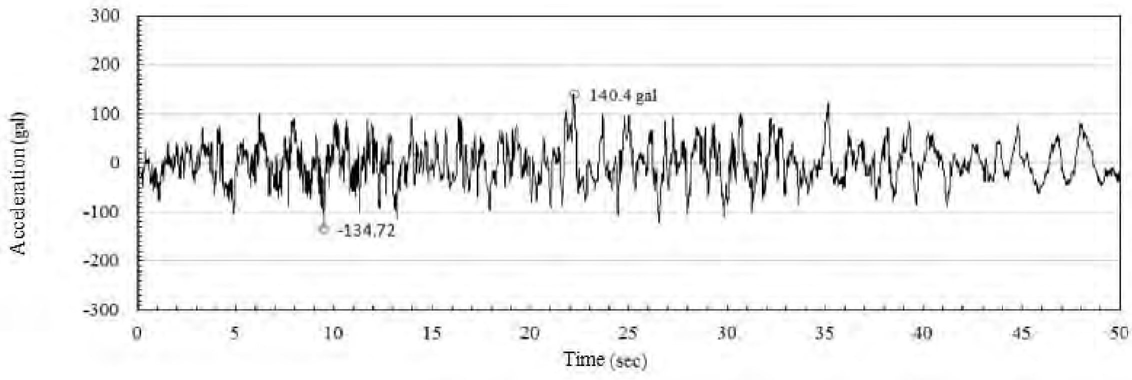


Source: JICA Study Team

Figure 4.2.62 Structural Plan of Superstructure

2) Design Seismic Wave

The design seismic wave used for the dynamic analysis shall use the waveform of the Specification of Highway Bridges Level 1 Seismic Motion (Type III Ground) corresponding to kh = 0.3 of the seismic coefficient method.

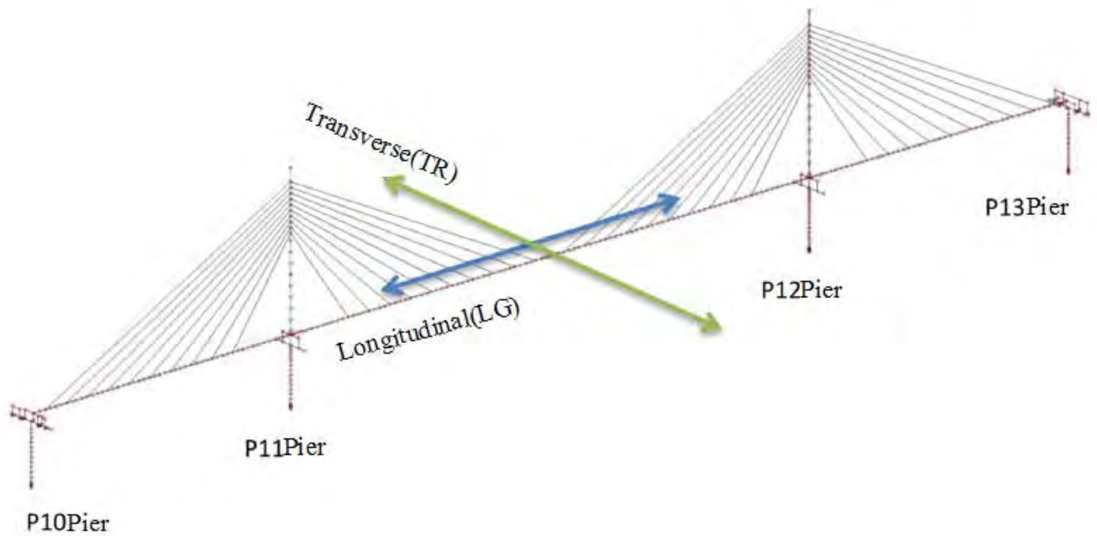


Source: JICA Study Team

Figure 4.2.63 Design Seismic Wave

3) Analysis Direction

The analysis of the bridge shall be performed in two directions, namely: the direction connecting the P10 and P13 pier, which is the Longitudinal Direction (LG), and the direction perpendicular to it, which is the Transverse Direction (TR), considering the bridge is straight.



Source: JICA Study Team

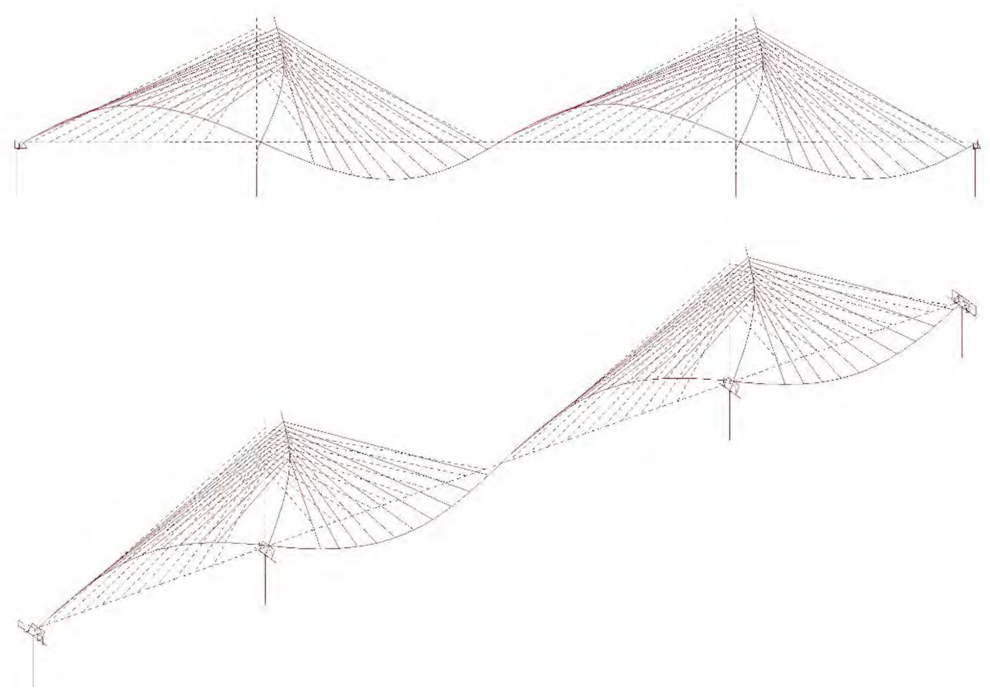
Figure 4.2.64 Analysis Direction for Dynamic Analysis

(2) Analysis Result

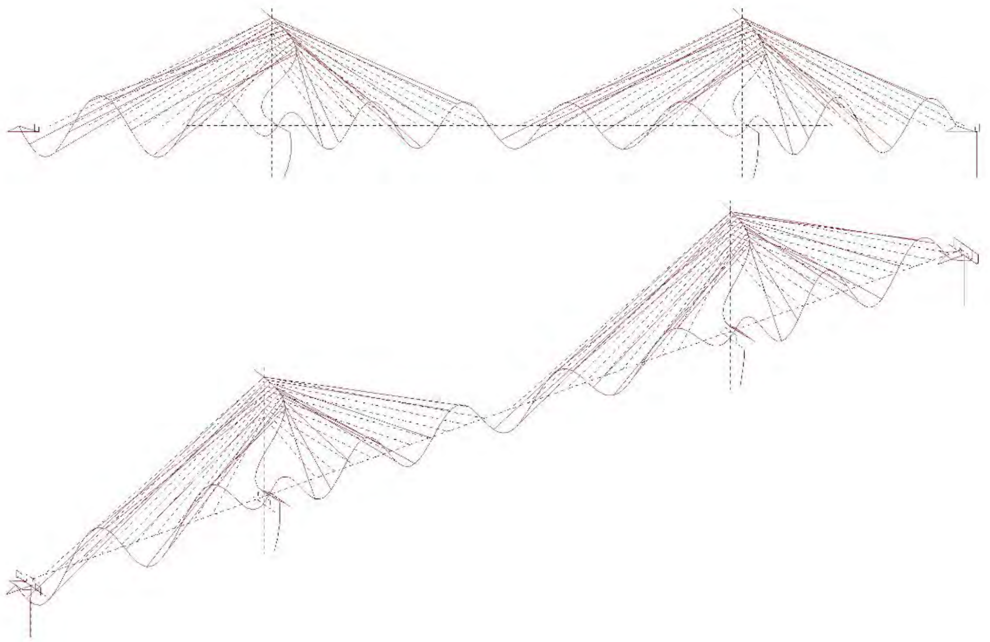
1) Natural Value Analysis

The fundamental natural frequency mode for each analysis model is shown below. It was found that the responded stress in all the structural members are less than the allowable stress.

4th Mode F = 0.527 [Hz]



40th Mode F = 8.118 [Hz]



Source: JICA Study Team

Figure 4.2.65 Example of Fundamental Vibration Mode (Whole Cross Section Stiffness – Transverse)

4.2.13 Superstructure Construction Stage Analysis

4.2.13.1 Construction Stage Analysis Overview

During the construction stage of temporary structures, the superstructure section force, cable tension, and bent reaction forces will be calculated to verify the safety and understand the deformation during erection.

4.2.13.2 Analysis Condition

The analysis condition for the construction stage analysis is listed in the table below.

Table 4.2.35 Analysis Condition

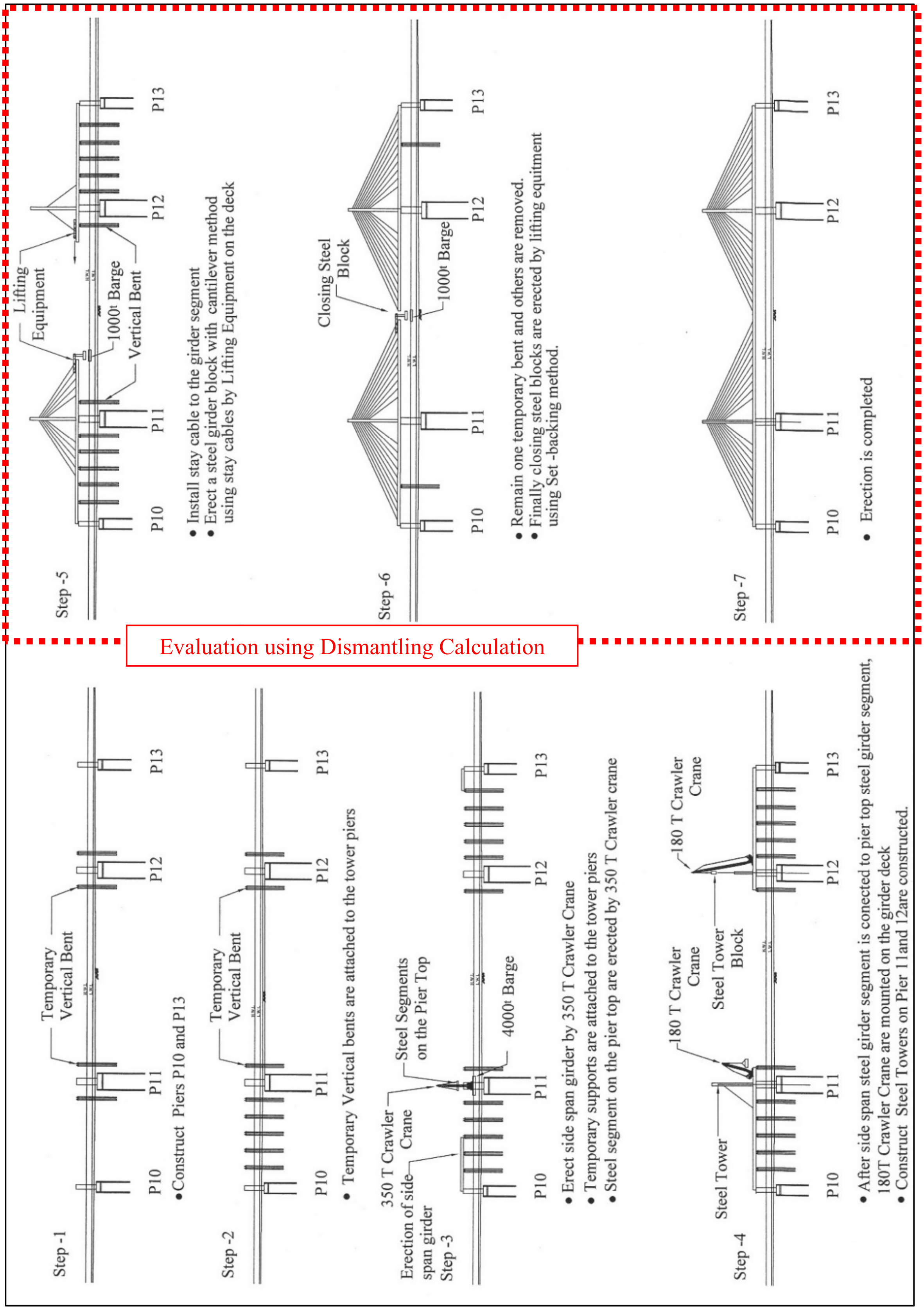
Item	Content
Analysis Theory	Linear structural analysis
Analysis Model	3d structure model
Considered Temporary Load	180 t considered Erection Machine W = 160 t Movement Protection Scaffolding W = 20 t
Analysis Stage	All 24 Stages （CS0～CS23）

Source: JICA Study Team

4.2.13.3 Construction Stage

The construction stages of the cable-stayed bridge is shown below.

For the construction stage surrounded by red dotted line in the image below, evaluation using the construction stage analysis is performed.



Source: JICA Study Team

Figure 4.2.66 Construction Steps for Cable-stayed Bridge

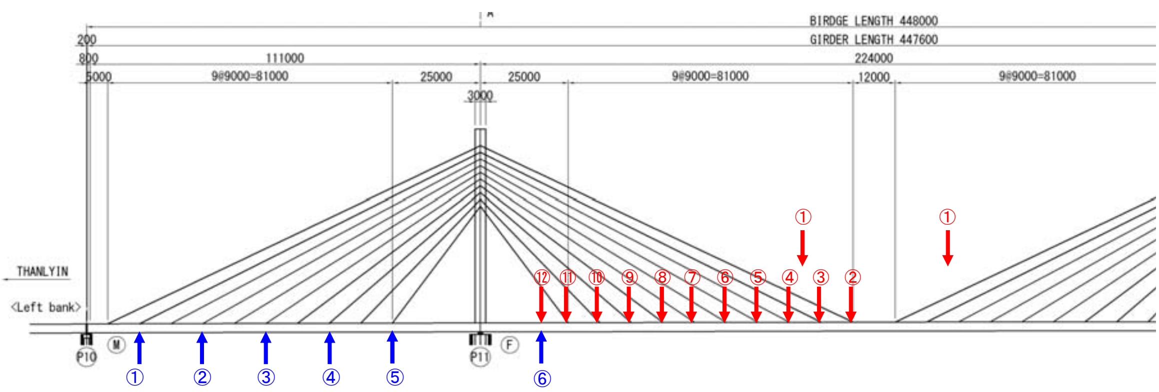
For the construction stage analysis, reversed order for the dismantling of temporary structure is configured as shown below.

Table 4.2.36 Dismantling Stages

Analysis Stage	Content	Crane Position	Bent
CS0	Competed (Front DL+Back DL+PS)	-	↑
CS1	Removal of Additional Dead Load	①	↑
CS2	Removal of Main Girder G27 (Closing Block)	② C 20 Anchor Position	↑
CS3	Removal of Cable C1,20	③ C 19 Anchor Position	↑
CS4	Removal of Main Girder G26	③ C 19 Anchor Position	↑
CS5	Removal of Cable C2,19	④ C 18 Anchor Position	↑
CS6	Removal of Main Girder G25	④ C 18 Anchor Position	↑
CS7	Removal of Cable C3,18	⑤ C 17 Anchor Position	↑
CS8	Removal of Main Girder G24	⑤ C 17 Anchor Position	↑
CS9	Removal of Cable C4,17	⑥ C 16 Anchor Position	Removal of Bent ①
CS10	Removal of Main Girder G23	⑥ C 16 Anchor Position	↑
CS11	Removal of Cable C5,16	⑦ C 15 Anchor Position	Removal of Bent ②
CS12	Removal of Main Girder G22	⑦ C 15 Anchor Position	↑
CS13	Removal of Cable C6,15	⑧ C 14 Anchor Position	↑
CS14	Removal of Main Girder G21	⑧ C 14 Anchor Position	↑
CS15	Removal of Cable C7,14	⑨ C 13 Anchor Position	Removal of Bent ③
CS16	Removal of Main Girder G20	⑨ C 13 Anchor Position	Removal of Bent ④⑥
CS17	Removal of Cable C8,13	⑩ C 12 Anchor Position	↑
CS18	Removal of Main Girder G19	⑩ C 12 Anchor Position	Removal of Bent ⑤
CS19	Removal of Cable C9,12	⑪ C 11 Anchor Position	↑
CS20	Removal of Main Girder G18	⑪ C 11 Anchor Position	↑
CS21	Removal of Cable C10,11	⑫	↑
CS22	Removal of Main Girder G17	-	↑
CS23	Removal of Main Girder G16	-	Install Bent ①～⑥

※Refer to the next page for cable number and main grider numbers

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.67 Bent Position and Crane Set Position

4.2.13.4 Dismantling Calculation Results

(1) Cross Section Evaluation

The maximum and minimum section forces and the results of the cross section evaluation are shown below.

Table 4.2.37 Main Girder Section Force Summary Table

Cross-Section Position \ Section Force	Bending Moment (kN・m)		Shear Force (kN)		Axial Force (kN)		Evaluation Result
	Max	Min	Max	Min	Max	Min	
EJ1	4325	-3607	1783	-1839	5	-5	OK
EJ2	15761	-22478	2725	-692	7	-3877	OK
EJ3	17452	-38031	2660	-1899	5551	-7463	OK
EJ4	20533	-44936	2678	-738	5554	-10812	OK
EJ5	18926	-47299	2791	-1053	5557	-14068	OK
EJ6	14205	-43778	2562	-676	5559	-17460	OK
EJ7	7998	-37889	2541	-862	4578	-19432	OK
EJ8	4820	-33121	2432	-658	4189	-21524	OK
EJ9	916	-30976	1348	-901	2747	-23586	OK
EJ10	1488	-22578	1038	-1304	746	-25496	OK
EJ11	14232	-10746	-342	-2204	-46	-27103	OK
EJ12	22777	-2323	1444	-1089	-44	-27100	OK
EJ13	24159	-14449	2825	-227	-43	-27098	OK
EJ14	23044	-20108	1214	-2961	261	-28675	OK
EJ15	22049	-11971	2077	-1579	262	-28673	OK
EJ16	12981	-23321	1919	-1605	0	-28671	OK
EJ17	1201	-29583	1068	-1755	0	-27074	OK
EJ18	0	-35244	578	-1744	0	-25172	OK
EJ19	689	-31912	346	-1984	0	-23113	OK
EJ20	2745	-26613	259	-2173	0	-21028	OK
EJ21	2561	-28750	319	-1410	0	-19056	OK
EJ22	697	-31618	0	-1795	0	-15667	OK
EJ23	1295	-28220	0	-1909	0	-12404	OK
EJ24	6258	-18198	512	-1905	0	-9054	OK
EJ25	4093	-5388	114	-1189	0	-5708	OK
EJ26	6087	-755	0	-634	0	-2337	OK

Source: JICA Study Team

Table 4.2.38 Main Tower Section Force Summary Table

Cross-Section Position \ Section Force	Bending Moment (kN・m)		Shear Force (kN)		Axial Force (kN)		Evaluation Result
	Max	Min	Max	Min	Max	Min	
Tower Base (Girder upper surface)	3136	-1903	31	-122	-3440	-43031	OK
J1	3098	-1913	31	-122	-3410	-43001	OK
J2	2868	-1976	31	-122	-3230	-42821	OK
J3	2614	-2045	31	-122	-3030	-42621	OK
J4	2360	-2177	31	-122	-2830	-42421	OK
J5	2105	-2367	31	-122	-2630	-42221	OK
J6	1851	-2556	31	-122	-2430	-42021	OK
J7	1596	-2745	31	-122	-2230	-41821	OK
J8	1342	-2935	31	-122	-2030	-41621	OK
J9	1087	-3124	31	-122	-1830	-41421	OK
C10	909	-3268	31	-122	-1690	-41281	OK
C9	671	-3447	71	-119	-1530	-36487	OK
C8	390	-3513	162	-140	-1370	-32040	OK
C7	173	-3425	243	-121	-1210	-28017	OK
C6	139	-3176	303	-74	-1050	-24449	OK
C5	102	-2765	341	-18	-890	-21406	OK
C4	61	-2106	332	-21	-730	-16709	OK
C3	36	-1292	407	-13	-570	-12499	OK
C2	13	-512	390	-11	-410	-8434	OK
C1	0	0	256	-7	-250	-4321	OK

Source: JICA Study Team

Table 4.2.39 Maximum Cable Tension

Cable Number	Maximum Cable Tension（kN）	Evaluation Result
C1	4356	OK
C2	4975	OK
C3	4854	OK
C4	4812	OK
C5	5172	OK
C6	3277	OK
C7	3392	OK
C8	3503	OK
C9	3338	OK
C10	3110	OK
C11	3113	OK
C12	3357	OK
C13	3515	OK
C14	3430	OK
C15	3365	OK
C16	5014	OK
C17	4571	OK
C18	4581	OK
C19	4783	OK
C20	4359	OK

Source: JICA Study Team

(2) Bent Reaction Force

The table of the bent reaction force for each analysis stage is shown below.

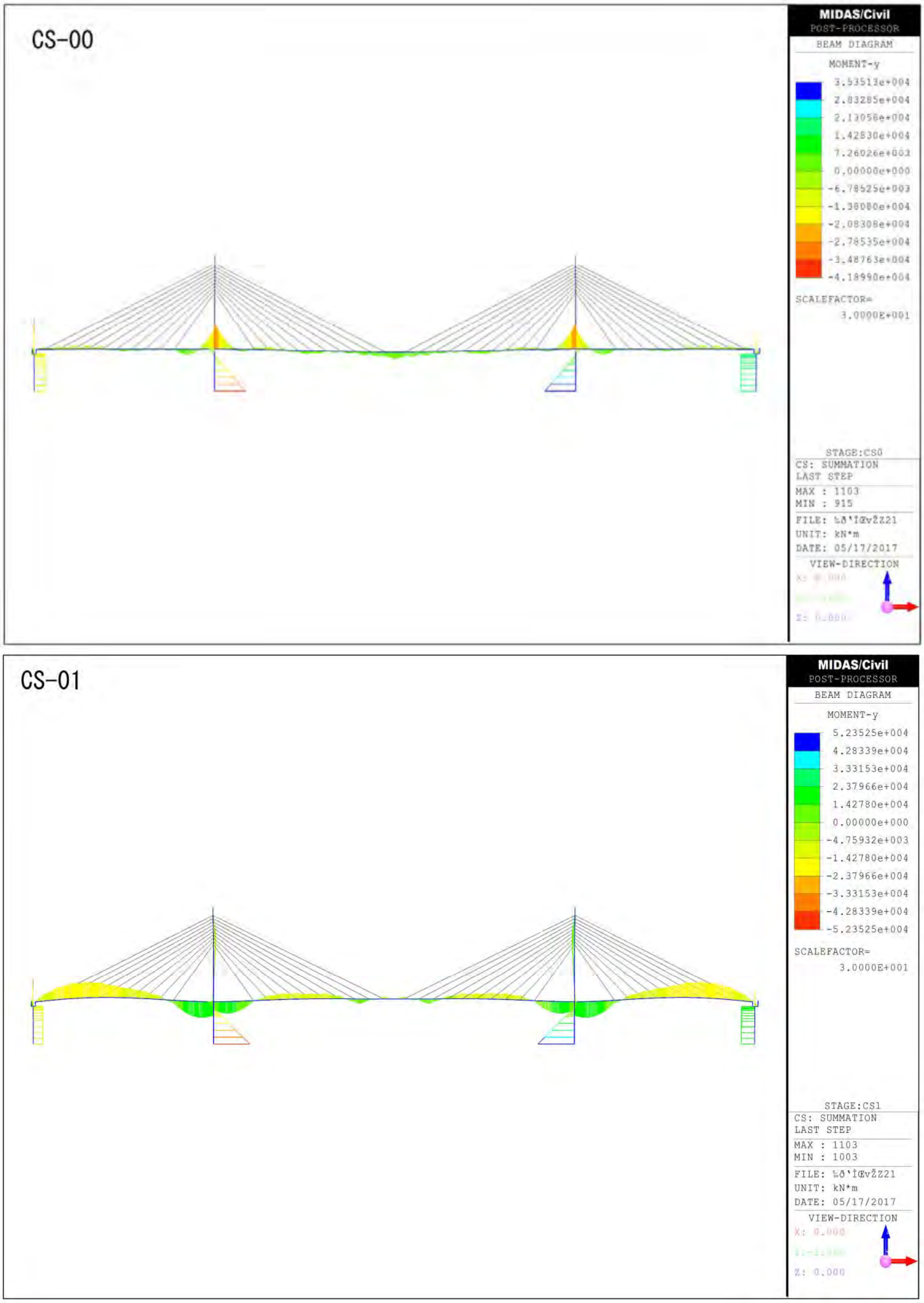
Table 4.2.40 Bent Reaction Table

Bent Position / Analysis Stage	B1	B2	B3	B4	B5	B6
CS10	2129.9	-	-	-	-	-
CS11	1952.3	-	-	-	-	-
CS12	3903.8	229.4	-	-	-	-
CS13	4306.6	189.3	-	-	-	-
CS14	2756.5	2330.6	-	-	-	-
CS15	2770.8	2659.5	-	-	-	-
CS16	2134.0	3159.5	852.0	-	-	-
CS17	2421.4	2109.1	2920.2	131.1	-	3659.8
CS18	2423.8	2100.4	2951.3	136.5	-	674.9
CS19	2381.9	2253.5	2401.4	2012.5	716.8	4369.6
CS20	2379.9	2261.0	2374.6	2110.3	466.6	1758.6
CS21	2445.9	2259.2	2380.8	2054.7	2955.6	4597.1
CS22	2444.8	2263.3	2366.4	2107.3	2810.3	2788.3
CS23	2379.3	2263.0	2367.5	2136.2	2816.9	-

Source: JICA Study Team

(3) Bending Moment Diagram

The examples of bending moment diagrams for each analysis stage are shown below.



Source: JICA Study Team

Figure 4.2.68 Examples of Bending Moment Diagram (Top: CS0, Bottom: CS1)

4.2.14 Revised Design of Side Pier (P10, P13) [Change from PC Box Girder to Steel Box Girder]

4.2.14.1 Pier Design

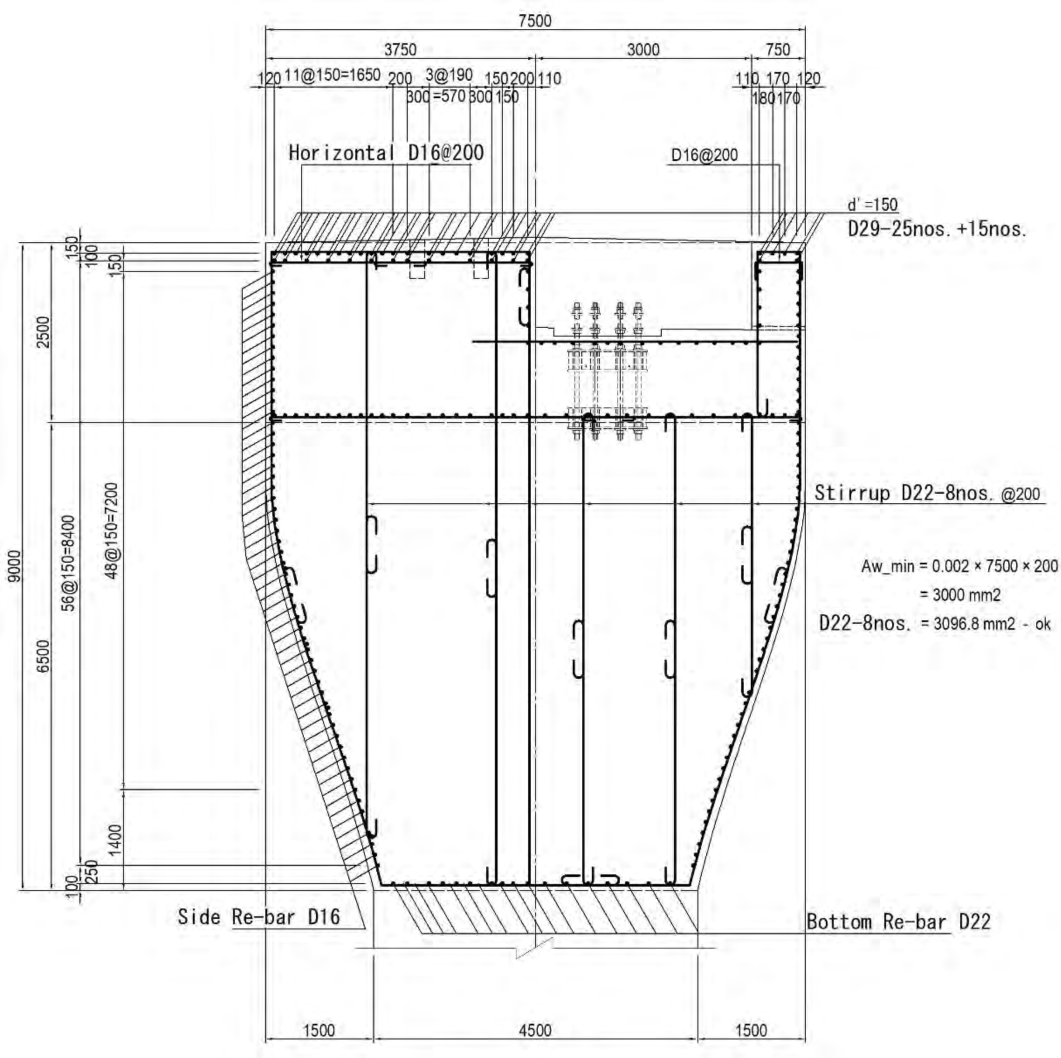
(1) Design Conditions

A superstructure type of the adjacent bridge at P10 was changed (from PC box girder to 3-span steel box girder), therefore revised design for the side pier was conducted.

(2) Pier Design

1) Beam Design

The cross sectional shape of the beam and arrangement of steel reinforcement are shown below.



Source: JICA Study Team

Figure 4.2.69 Cross Section of Beam

[Overview of Calculation Result]

The following table shows the calculation results for the beam.

Table 4.2.41 Calculation Results for Beam

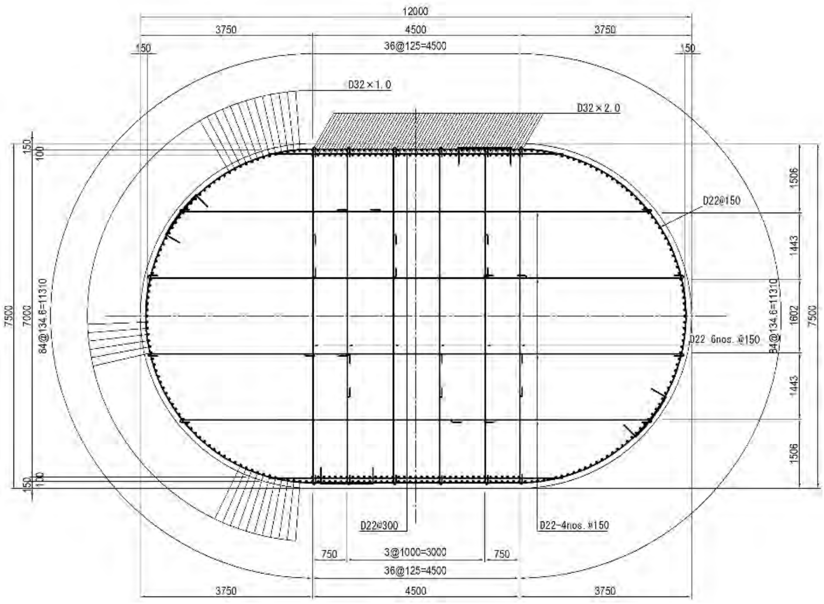
				Vertical Direction				Horizontal Direction			
Section	Height		m	9.000				7.500			
	Re-Bar	Main Re-bar	1st layer	D29	—	25nos.		D16	—	49nos.	
			2nd layer	D29	—	15nos.					
		Stirrup		D22-8nos. ctc200				D22-2nos.+D16-1no. ctc200			
Bridge Seat		Required Re-bar	mm2		---				---		
Corbel		Required Re-bar	mm2	23,101	≦	25,696	○	10,278	≦	19,463	○
Calculation	Bending Evaluation	Load Case		Dead Load				Seismic			
		σc	N/mm2	0.78	≦	10.00	○	0.70	≦	15.00	○
		σs	N/mm2	80.3	≦	100.0	○	97.1	≦	300.0	○
	Shear Evaluation	Load Case		Dead + Live Load				Seismic			
		τm	N/mm2	0.006	≦	0.140	○	0.045	≦	0.111	○
		Awreq < Aw	mm2								
	Evaluation for Seismic Performance 2	M < My	KN·m		---			7,560	≦	21,371	○
		S < Ps	KN		---			3,217	≦	16,160	○

Source: JICA Study Team

2) Design of Column

The column shall be designed as a cantilever beam by treating the joint between the footing as a fixed end. The column cross section shall be designed against the most unfavorable combination of axial force and bending moment.

Note that the steel reinforcement in the column-axial direction was set by dynamic analysis evaluation.



Source: JICA Study Team

Figure 4.2.70 Cross Section of Column

[Overview of Calculation Result]

The following table shows the calculation results for the column.

Table 4.2.42 Calculation Result for Column

				Longitudinal				Transverse			
Section	Height		m	Oval	;	12.000	×	7.500			
	Re-Bar	Main Re-bar	1st layer	D32	ctc	125	※	D32	ctc	135	※
			2nd layer	D32	ctc	125	※				
		Hoop	---	D22	ctc	150		D22	ctc	150	
Calculation	L1 Seismic	σc	N/mm2	7.43	≦	15.00	○	5.02	≦	15.00	○
		σs	N/mm2	231.0	≦	300.0	○	108.2	≦	300.0	○
		τm	N/mm2	0.279	>	0.171	—	0.258	>	0.152	—
		Aw_req	mm2	693.2	≦	3096.8	○	426.5	≦	2322.6	○

Note: ※ was decided by dynamic analysis

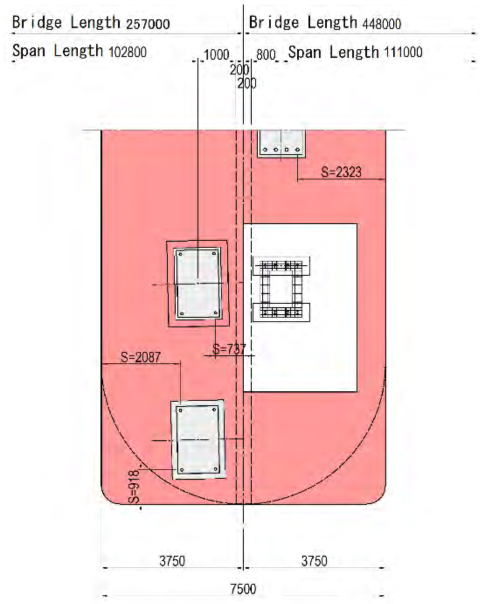
Source: JICA Study Team

3) Bridge Seat Design

a) Dimension of Bridge Seat Width

The distance between the bearing support edge and the top edge of the substructure was set in accordance with the Specifications for Highway Bridges IV 8.6.

[P10 Pier]



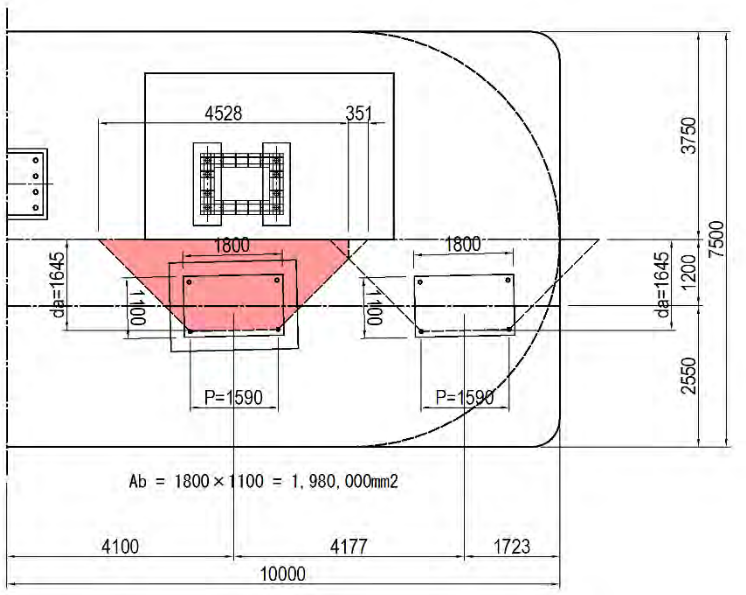
Source: JICA Study Team

Figure 4.2.71 Bridge Seat Width

b) Evaluation of Bridge Seat Strength

Since the bridge seat has a function to support the superstructure via bearing support, large horizontal force would act on it during an earthquake. For this reason, the bridge seat needs to be designed to have sufficient strength against design horizontal seismic force.

The resistance area of concrete against horizontal force is illustrated in the following drawings:



Source: JICA Study Team

Figure 4.2.72 Resistance Area of Concrete

4.2.14.2 Foundation Design

The foundation shape, steel pipe size, etc. were not changed based on the revised design. Therefore, only summary of the design results are shown in the table below.

Table 4.2.43 Suammary of Foundation Desing

				Longitudinal				Transverse			
Pile	Diameter(mm)×Length(m)×Number(no.)			Outer Pile	;	φ1200	×	56.00	×	36nos.	
				Diaphragm Pile	;	φ1200	×	52.10	×	8nos.	
	Thickness	Outer Pile	Top Pile			t = 14	mm	(SKY490)			
			Bottom Pile			t = 14	mm	(SKY400)			
		Diaphragm Pile	---			t = 14	mm	(SKY400)			
Calculation	Reguler (Existing River Bed)	δ	cm	0.04	≦	5.00	○	0.06	≦	5.00	○
		PNmax	KN/no.	1910	≦	4100	○	1912	≦	4100	○
		PNmin	KN/no.	1612	≧	0	○	1610	≧	0	○
	Seismic (Existing River Bed)	δ	cm	2.51	≦	5.00	○	3.10	≦	5.00	○
		PNmax	KN/no.	1922	≦	6200	○	1924	≦	6200	○
		PNmin	KN/no.	1585	≧	-3600	○	1604	≧	-3600	○
Composite Stress (Seismic・Existing River Bed)		SKY400	N/mm2	161.0	≦	210.0	○	194.3	≦	210.0	○
		SKY490	N/mm2	208.5	≦	277.5	○	239.6	≦	277.5	○

Source: JICA Study Team

4.2.15 Summary of Wind Tunnel Test

4.2.15.1 Introduction

This section is to summarize the conditions and the results of the wind tunnel tests to estimate the wind-resistant characteristics of the main girder and the towers of Cable-Stayed Bridge of Bago River Bridge in under-construction and after completion stages conducted by Bridge Engineering Laboratory and Structural Dynamics Laboratory, Department of Civil and Earth Resources Engineering, Kyoto University, Japan.

4.2.15.2 Wind Tunnel

The wind tunnel facility used for the test is the Eiffel type wind tunnel in Department of Civil and Earth Resources Engineering, Kyoto University, Japan. Width and height of working section is 1.0m and 1.8m for section model test. Wind velocity in the working section can be adjusted up to about 25m/s. Turbulent intensity in the empty working section isbless than 0.5(%).

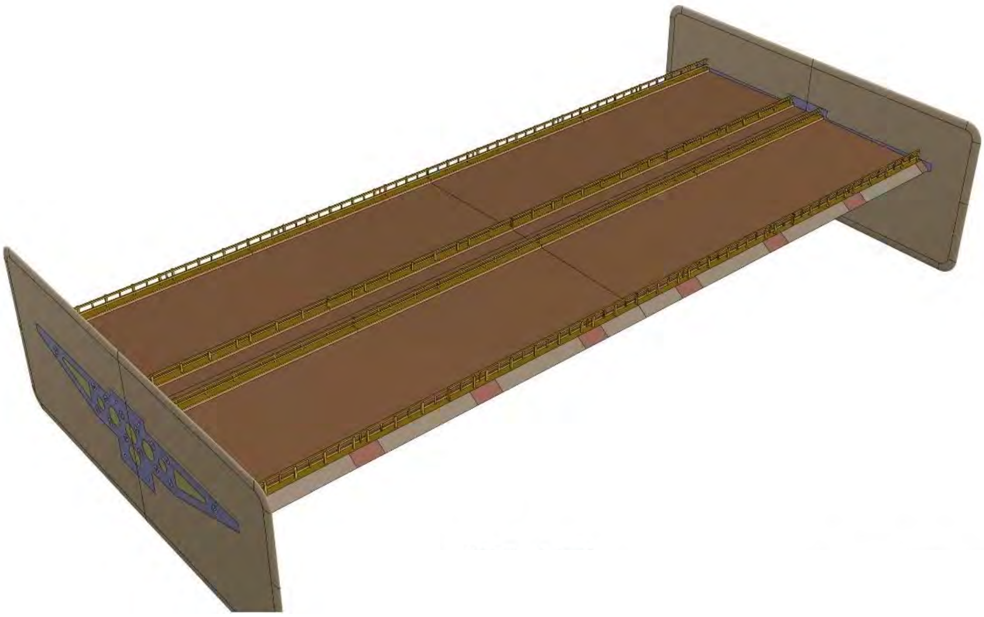
Source: Kyoto University

Figure 4.2.73 Wind Tunnel in Department of Civil and Earth Resources Engineering,Kyoto University

4.2.15.3 Models for Wind Tunnel Test

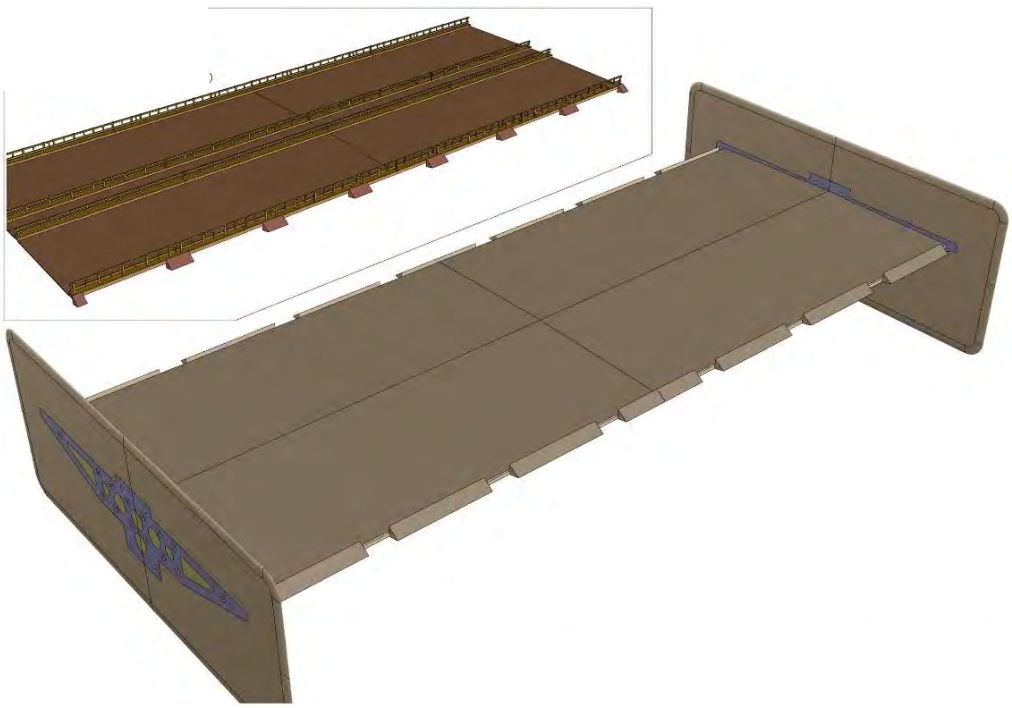
(1) Section model of main girder

The image of section model is shown in Figure 4.2.74 to Figure 4.2.75.



Source: JICA Study Team

Figure 4.2.74 3-D image of section model (for after-completion stage)



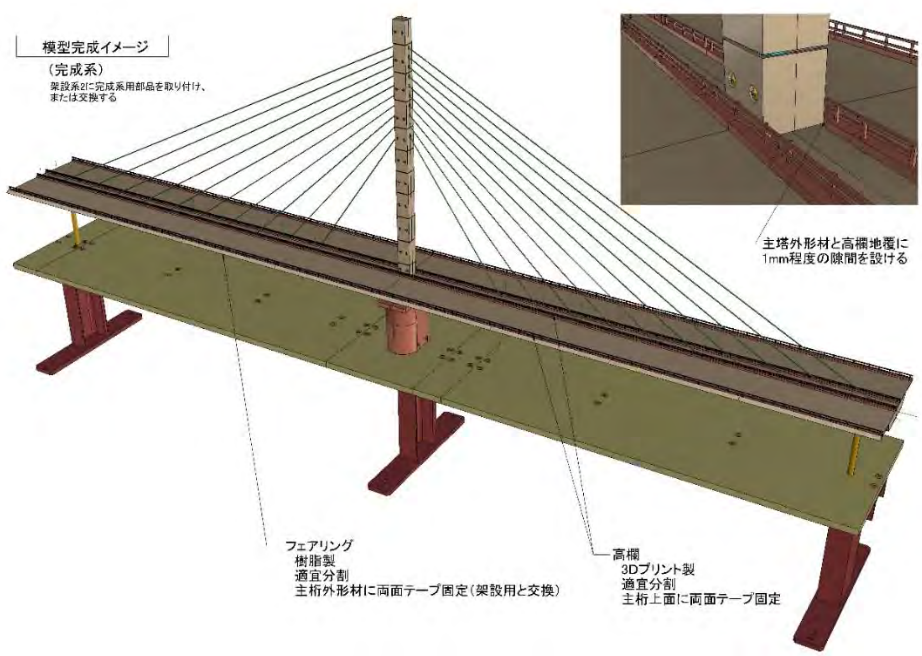
Source: JICA Study Team

Figure 4.2.75 3-D image of section model (for under-completion stage)

(The configuration is reproduced by taking out the top left piece from the section model.)

(2) 3-D elastic model of tower

Scale ratio of the model was determined as 1/120. The detail of the section model is shown in Figure 4.2.76.



Source: JICA Study Team

Figure 4.2.76 3-D image of the elastic model (for after-completion stage)

(The min girder part is a rigid model.)

4.2.15.4 Aerodynamic response of elastic tower model (scale ratio 1/120)

Aerodynamic vibration response of the main girder and the tower of Cable-stayed Bridge of Bago River Bridge was examined by wind tunnel tests.

For the under-construction stage, the following 2 stages were focused:

- Before the lowest cable being installed and just after the first segment of the main girder was installed. Heaving 1 DOF of the main girder dominates. The tower stands in isolated condition. Hence, both bending modes along/normal to cable plane may be possible. (Abbreviated as UC1)
- Just before the last segment of the main girder in the main span is installed. Heaving and torsional 2 DOF of the main girder dominates. Sine all cables are already installed, possible bending mode of the tower is normal to cable plane only. (UC2)

For after-completion stage,

- Heaving and torsional 2 DOF of the main girder dominates. Sine all cables are already installed, possible bending mode of the tower is normal to cable plane only. (AC)

The aerodynamic response of the main girder shows stable characteristics for the above 2 under-construction stages (UC1, UC2) in smooth and in turbulent flow conditions. Neither vortex-induced vibration (VIV) nor flutter was measured.

On the other hand, only vortex-induced vibration (VIV) of the main girder was measured in the after-completion stage (AC) at about 15 to 25 [m/s] for real bridge under the vertical incidence of angle 0, +3 and -3[deg.] in smooth flow condition. In case of turbulent flow condition, neither VIV nor flutter was measured.

Therefore, the main girder possesses stability to aerodynamic vibration, if turbulent flow condition is taking into account.

Vortex-induced vibration and galloping were observed in the tower for its original configuration.

For UC1 in smooth flow, VIV of bending mode normal to the cable plane (in y-direction) occurs at 16 to 19 [m/s] under wind direction of 0 [deg.] and 5 [deg.] (almost in parallel to the bridge axis), while galloping occurs from 60 [m/s] under 5 [deg.]. VIV of bending mode in parallel to the cable plane (in x-direction) was also observed in smooth flow under 80, 85 and 90 [deg.] (almost normal to bridge axis) at 14 to 23 [m/s]. Galloping occurs from 58 [m/s] for 80 [deg.] and from 43 [m/s] for 90 [deg.].

For AC in smooth flow, VIV was measured in smooth flow at 10 to 31 [m/s] for 0 [deg.] and 10 to 20 [m/s] for 5 [deg.]. For 10, 22.5, 45, 67.5 and 90 [deg.], stability for VIV was confirmed. The occurrence of galloping was confirmed from 28 [m/s] for 5 [deg.], while no galloping for other cases with different wind direction. The VIV and galloping in turbulent flow condition remains only for 0 [deg.]. VIV was observed at 17 to 20 [m/s] and galloping occurred from 23 [m/s]. The tower was stable for other wind directions, 5, 10, 22.5, 45, 67.5 and 90 [deg.].

From these results, the occurrence of galloping for the wind direction along bridge axis should be main concern.

In order to suppress the galloping as mentioned above, the L-shaped aerodynamic device is proposed to be attached nearby the corner of the tower cross section as shown in Figure 4.2.80.

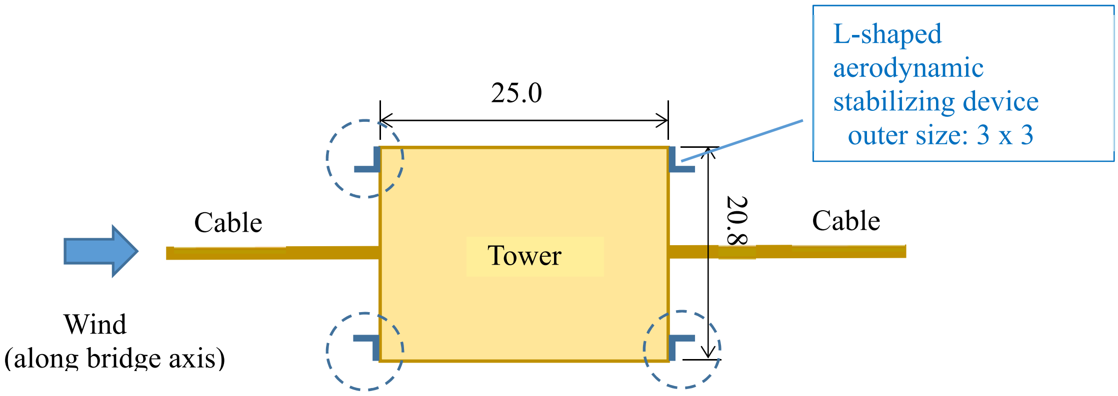
For UC1 in smooth flow, the response characteristics were examined by installing the device along 11.0 [m] from the top of the tower. VIV was still observed at 12 to 15 [m/s] for 0 [deg.] and 18 to 21 [m/s] for 90 [deg.], although no galloping was measured for both measurement cases. The response in turbulent flow condition was examined for 80, 85, 90 and 180 [deg.]. The tower was stable for all of these wind direction conditions. (There is no big difference in the response for 0 [deg.] and 180 [deg.], since the flow around the top of tower can be almost identical and no significant influence by the upstream elongation length of the main girder.

For UC2 in smooth flow with the L-shaped device installed along 11.0 [m], VIV was measured at 11 to 32 and 34 [m/s] for 0 [deg.], whereas, stable for 5 [deg.]. The tower showed stable for 0 and 5 [deg.] in turbulent flow.

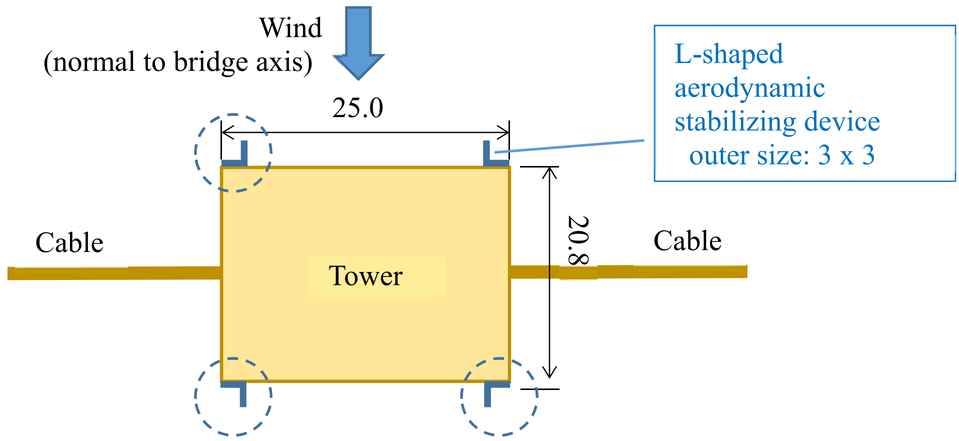
For AC in smooth flow with the L-shaped device installed along 11.0 [m], the tower was stable for 5 [deg.] but VIV occurred at 22 to 26 [m/s] for 0 [deg.]. No galloping was observed for both cases of wind direction. In turbulent flow, the tower was stable for 0 [deg.] and 5 [deg.].

Install length of the device was changed to 41.7 [mm] (= 5.0 [m] in real bridge), 141.7 [mm] (17.0 [m]), 191.7 [mm] (23.0 [m]) and 233.4 [mm] (28.0 [m]), respectively, in order to know its effect to stabilizing performance. Target wind direction was fixed to 0 [deg.] and 5 [deg.] only. In smooth flow, VIV was measured only for 0 [deg.] and the length of 141.7 [mm], while stable for 5 [deg.]. In turbulent flow, the tower showed stable response characteristics for all cases. No galloping was observed for all cases.

From these results, the response for 0 [deg.] with the length of the device 141.7 [mm] should be focused. This response was totally stabilized under turbulent flow condition. And the wind direction 0 [deg.] (along bridge axis) means the wind comes over the city of Yangon or the field in Thilawa. Moreover, this wind may be further disturbed by the existence of the cable in upstream of the tower. From these reasons, the wind resistant characteristics of the tower should be estimated under turbulent flow condition rather than in smooth flow. It was confirmed that, in turbulent flow condition, the tower is stabilized by installing the device longer than 141.7 [mm] (17.0 [m]) from the top. Therefore, it is recommended to install the device over 17.0 [m] from the top of the tower.



(a) For wind along bridge axis



(b) For wind normal to bridge axis

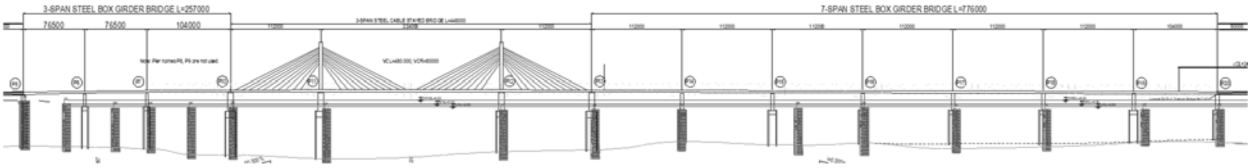
Source: JICA Study Team

Figure 4.2.77 L-shaped aerodynamic device (Model scale ratio: 1/120, unit: in mm)

4.3 STUDY ON STEEL BOX GIRDER BRIDGE

In this section, study results on the 3-span and 7-span of steel box girder bridges, as shown in Figure below, will be presented.

It is noted that 5-span PC box girder bridge (3@51m+2@52m) was originally designed instead of 3-span bridge. However, Pier No.9 was cancelled during this JICA study as requested by MOC because a navigation channel is possibility to be widened to the section between P8 and P10 in future. After due study, 3-span bridge was determined from better structural feature.



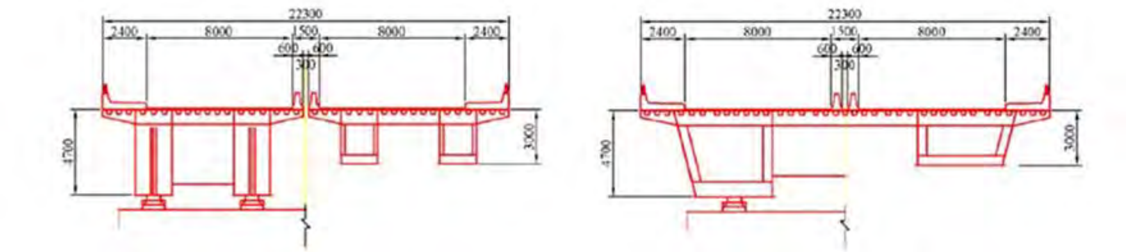
Source: JICA Study Team

Figure 4.3.1 Design Target Sections of Steel Box Girder Bridges

4.3.1 Basic Design for Superstructure of Steel Box Girder Bridge

4.3.1.1 Selection of Type of Steel Box Girder Bridge

In the F/S, separated bridge structure for up and down lanes was proposed taking account of the adjacent bridge structure types. In order to further reduce the construction cost and shorten the construction period, other arrangements of girders, including a combined structure type for up and down lanes was studied and compared. Items to be compared are steel weight, structural stability, construction plan (difficulty) and construction period and maintenance cost based on the structural analysis and preliminary cost estimate.



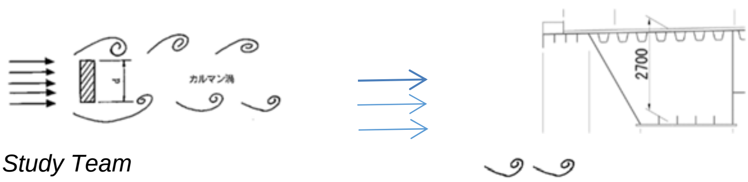
(i) F/S (up and down lanes separation structure) (ii) Alternative in B/D (up and down lanes combined structure)

Source: JICA Study Team

Figure 4.3.2 Type of Main Girder of Steel Box Girder with Steel Plate

(1) Structural Stability

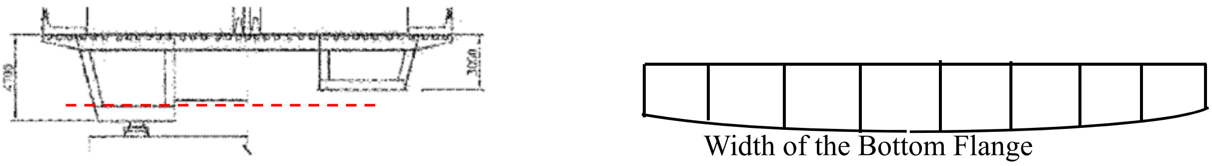
In general, a long span bridge with perpendicular tall web plate will be easy to be oscillated by wind (Karman vortex). One of the ways to avoid this influence is to adopt an inclined web. The inclination angle of approximately 60° of the outer web is common, taking account of fabrication.



Source: JICA Study Team

Figure 4.3.3 Image of the Karman Vortex

However, if the web height is changed into a curved form, as long as it keeps the inclination angle, width of the bottom flange will also change as shown in the figure below. This means that the diaphragm, which is an important element to ensure the accuracy of the box shape, shall have a different shape at each position, and so will increase the fabrication cost. Therefore, the girder of uniform height was decided to be more appropriate in terms of fabrication cost than the girder of changeable height planned in the F/S.

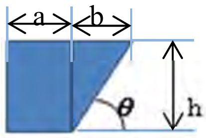


Source: JICA Study Team

Figure 4.3.4 Varying Width of Bottom Flange

The inclined web height will affect the width of the bottom flange. Therefore, web height and its inclination will be considered, taking account of its width, so that it will be possible to be transported.

Table 4.3.1 Relationship Between Girder Height and Width of the Bottom Flange



h(m)	θ°	Rad	tan θ	b(m)	a+b(m)	a(m)
2.7	70	1.222	2.747	0.98	3.00	**2.02**
2.7	61	1.065	1.804	1.50	3.00	**1.50**
2.7	60	1.047	1.732	1.56	3.00	**1.44**
2.7	50	0.873	1.192	2.27	3.00	0.73
3.0	70	1.222	2.747	1.09	3.00	**1.91**
3.0	63.5	1.108	2.006	1.50	3.00	**1.50**
3.0	60	1.047	1.732	1.73	3.00	**1.27**
3.0	50	0.873	1.192	2.52	3.00	0.48
3.3	70	1.222	2.747	1.20	3.00	**1.80**
3.3	65.5	1.143	2.194	1.50	3.00	**1.50**
3.3	60	1.047	1.732	1.91	3.00	1.09
3.3	50	0.873	1.192	2.77	3.00	0.23

Source: JICA Study Team

Considering the width of the bottom flange in the red cell above, the case where the web height is 2.7 m and web inclination is 61° was selected in the B/D as the most suitable one.

(2) Number of Main Girder and its Position

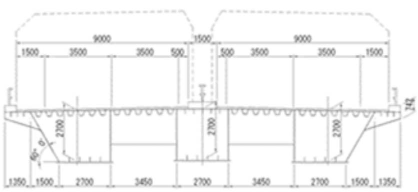
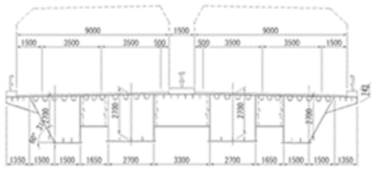
There are two options relating to the number of main girders; one is the 3-girder type and the other is the 4-girder type. Both types were compared from the following viewpoints:

- Steel Weight: This will depend on the number of web plates, and the effective total width of the bottom flange plate.
- Fabrication Cost: This depends on the number of segments and their self-weight.
- Transportation Cost: This depends on the dimension and weight that is possible to be transported.
- Erection Cost: This depends on the erection method and the required crane capacity and erection period.

The 4-girder type was recommended because it is superior in terms of cost and construction efficiency and there was no disadvantage found.

Table 4.3.2 Comparison of the Cross Section of the Steel Box Girder

Comparison of Cross Section of Steel Box Girder

		Alternative-1: 3-Girders	**Alternative-2**: 4-Girders (Option-1)
Profile			
	Outer Girder Steel Deck Thickness Bottom Flange Thickness	4440mm x19mm 2940mm x 44mm	3240mm x 16mm 1740mm x 60mm
Description		Outer Girder should be spliced to 2 parts for transportation. These parts shall be checked for its matching accuracy at shop.	All girders can be transported without division to small parts. Thickness of Bottom flange of the outer girder will be needed to use thicker plate, for instance abut 60mm, because the flange width is smaller.
Structural Aspect		To meet with requirements of stress and displacement. To be ensured durability of main girder because distance from wheel load to the web can be kept at 850mm.	To meet with requirements of stress and displacement. To be ensured durability of main girder because distance from wheel load to the web can be kept at 400mm.
Cost and Construction Efficiency	Estimated Weight	8,954 ton (Main Girder Only)	8,855 ton (Main Girder Only)
	Fabrication Cost (1)	**1.000**	**0.950**
	Transportation Cost*1/ (2)	**1.000**	**0.955**
	Averaged Weight for Erection	2.7 ton/m	2.4 ton/m
	Lifting Weight per 25m *2/	67.5	60.0
	Availability of Crane Capacity *3/	More than 250 ton C.C is required. If use 200 ton crane, the number of bent in the river should be increased.	200 ton CC is required.
	Erection Cost (3)	**1.000**	**0.864**
	Total Cost = i.+ii.+iii. i. Fabrication ((1) x 50%) ii. Transportation ((2) x 15%) iii. Erection ((3) x 35%)	1.000	0.920
Maintenance Aspect		Primary maintenance items will be corrosion or dirtiness of steel plates and bolted joints. Partial Visual inspection will be possible by looking from the man-hols that are prepared at main girder's web of about 25m pitch.	Primary maintenance items will be corrosion or dirtiness of steel plates and bolted joints. Partial Visual inspection will be possible by looking from the man-hols that are prepared at main girder's web of about 25m pitch.
Evaluation			
	Structural Aspect	◎	○
	Cost and Construction Efficiency	X	◎
	Maintenance Aspect	○	○
Other Aspects	Landscape View	○ Same inclination of web plate as Cable Stayed Bridge. Inclination of web plate does not match with PC girder.	○ Same inclination of web plate as Cable Stayed Bridge. Inclination of web plate does not match with PC girder.
	Efficiency against Wind Oscillation	○ Inclination 60°	○ Inclination 60°
Comprehensive Evaluation		**Less Recommended**	**Most Recommended**

*1/ Not only the number of unit but also size (volume) of one unit are affected to the transportation cost.

*2/ Erection of the girders by using 3 bents for one span 112m is assumed.

*3/ Crane capacity for construction of substructure is 200 ton. Therefore, no need to mobilize another crane if the options of 4-Girders is applied.

Source: JICA Study Team

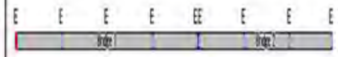
4.3.1.2 Study on the Number of Continuous Span and Supporting Condition

(1) 7-Span Bridge

To select the optimum option for the number of continuous span and support type, elastic support and fix support for the cases of 7-continuous span and 4+3 continuous span were compared in terms of the structural aspect, workability for superstructure erection and setting, economical aspect, travel comfort, and O&M. The following table shows the evaluation result.

After the evaluation, the fix support condition in a 7-continuous span bridge was selected because all items were ranked as superior.

Table 4.3.3 Study Results on the Number of Continuous Span and Support Condition (P13-P20)

Alternative	Alt-A	Alt-B	Alt-C	Alt-D
Continuous Span	7 Continuous Spans Bridge		4+3 Continuous Spans Bridge	
Support Condition/ Bearing Type	Elastic Support Condition  P13 P20	Fix Support Condition  P13 P20	Elastic Support Condition  P13 P17 P20	Fix Support Condition  P13 P17 P20
Structural Aspect/ Aseismicity	• It has a moderate risk of occurring resonance between soft ground and bridge because natural period of oscillation of the bridge is relatively longer, namely 1.7 second. (O)	• It has a low risk of occurring resonance between soft ground and bridge because natural period of oscillation of the bridge is relatively shorter, namely approx. 0.9 second. (◎)	• It has a moderate risk of occurring resonance between soft ground and bridge because natural period of oscillation of the bridge is relatively longer, namely 1.7 second. (O)	• It has low risk of occurring resonance between soft ground and bridge because natural period of oscillation of the bridge is relatively shorter, namely 1.0 second at P13-P17 and 0.9 second for P17-P20. (O)
	•Horizontal force due to temperature change and inertial force can be evenly distributed to all substructures P13 to P20 and resisted by both bearings and substructures. (◎)	•Horizontal force due to inertial force can be equally distributed to substructures of P14 to P19, and horizontal force due to temperature is slightly larger at P14 and P19. These forces are resisted by the substructures.(O)	•Horizontal force due to temperature change and inertial force can be evenly distributed to all substructures, and resisted by both bearings and substructures.(◎)	•Horizontal force due to temperature change and inertial force will be unevenly distributed to substructures of P14 to P19. (△)
Max. Inertial Force	6,000kN at P17	7,500kN at P19	5,960kN at P17	8,560kN at P15
Max. Horizontal Force due to temperature change	1,850k at P13	4,500kN at P19	1,319kN at P13	5,570kN at P19
Max. relative displacement between super- and sub-structure	177mm at P13	82mm at P13	190mm at P17	87mm at P13
Workability for Superstructure	•Required special attention in setting girders to adjust larger accumulated error of span length due to longer continuous spans. (O)	•Required special attention in setting girders to adjust larger accumulated error of span length due to longer continuous spans. (O)	•Easier to set the position of the girders because of smaller accumulated error of span length, and can earlier fix the anchor bolts by non-shrinkage mortal. (◎)	•Easier to set the position of the girders because of smaller accumulated error of span length, and can earlier fix the anchor bolts by non-shrinkage mortal. (◎)
Cost	•Substructure strength can be minimized. Instead, required large size of expansion joints and bearings to accommodate with larger displacement due to temperature change and inertial force. <Cost ratio 1.00> (◎)	•Smaller size of expansion joints and bearings can be used because forces from superstructure can be supported by mainly substructure not bearings. Instead, required larger size of re-bar for some parts of substructure. <Cost ratio 1.00> (◎)	•Thicker plate of girder is required to accommodate with larger bending moment, then steel weight increase at 163 ton. <Cost ratio 1.02> (O)	•Thicker plate of girder is required to accommodate with larger bending moment, then steel weight increase at 163 ton. •Required large size of re-bar for some parts of substructure at P14-P16. <Cost ratio 1.03> (△)
Travelling Comfortability	•More comfortable because of only 2 locations of expansion joint (◎)	•More comfortable because of only 2 locations of expansion joint (◎)	•Lower comfortable because of 3 locations of expansion joint (O)	•Lower comfortable because of 3 locations of expansion joint (O)
Operation & Maintenance	•Less maintenance work because of lower nos. of shoes (nos.32) and expansion joints (nos.2) (◎)	•Less maintenance work because of lower nos. of shoes (nos.32) and expansion joints (nos.2) (◎)	•More maintenance work because of larger nos. of shoes (nos.36) and expansion joints (nos.3) (O)	•More maintenance work because of larger nos. of shoes (nos.36) and expansion joints (nos.3) (O)
Evaluation	Less recommended	Most recommended	Less recommended	Less recommended

Note: ◎:Better O:Normal △: Worse

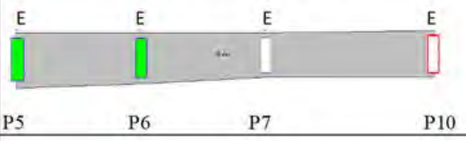
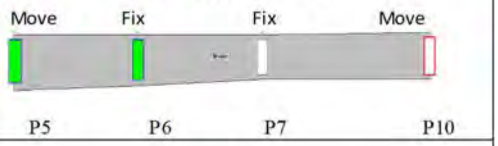
Source: JICA Study Team

(2) 3-Span Bridge

A 3-continuous span is applied in terms of structural and economical aspect. As for the support condition, two alternatives, namely elastic support (Alt-A) and fix support (Alt-B), are comparatively studied. Seismic horizontal force is evenly distributed to all piers in the elastic support condition, meanwhile 60% of inertial force is concentrated to one pier in the fix support condition which might be caused by unequal span length, different pier height and substructure rigidity. Since larger dimension and higher grade of rebar and steel sheet pipe are required for substructure in the fix support condition, the cost becomes 6% higher than Alt-A.

Accordingly, Alt-A is superior in terms of aseismicity and economic aspects.

Table 4.3.4 Study Results on the Support Condition (P5-P10)

Alternative	Alt-A	Alt-B
Continuous Span	3 Continuous Spans Bridge	
Support Condition/ Bearing Type	Elastic Support Condition 	Fix Support Condition 
Structural Aspect/ Aseismicity	• It has a moderate risk of occurring resonance between soft ground and bridge because natural period of oscillation of the bridge is relatively longer, it is 1.60 second. (O)	• It has a low risk of occurring resonance between soft ground and bridge because natural period of oscillation of the bridge is relatively shorter, it is 0.92 second. (◎)
	•Seismic horizontal force can be effectively distributed to all substructures P5 to P10 and resisted by both bearings and substructures. (◎)	•Seismic horizontal force is concentrated to one pier (P6) among 4 piers, which rate is around 60%. It has a risk to collapse superstructure in the case that the bearing of P6 would be damaged because remaining bearings would not have enough capacity to support superstructure. Accordingly, special attention for countermeasure to prevent collapse is required. (△)
Max. Inertial Force (Kh0.3)	4,600kN at P6	9,600kN at P6
Max. Horizontal Force due to temperature change(± 15°)	420kN at P10	730kN at P6,P7
Max. relative displacement between super- and sub-structure (earthquake)	±207mm at P13	±62mm at P5
Ditto (temperature change± 25°)	±44mm at P10	±43mm at P10
Economical Aspect (Cost)	•Substructure dimension can be minimized by pier thickness 3.0m using Dia.32 or Dia.38 as main rebar and normal grade of steel sheet with SKY400. Instead, required large size of expansion joints to accommodate with larger displacement due to inertial force. Rubber bearing cost is normally higher than other types such as steel/iron ones. <Cost ratio 1.00> (◎)	•Smaller size of expansion joints can be used, and steel/iron type of bearings can be applied and their costs are normally much lower than rubber bearing. Instead, larger substructure dimension is required with large size of main re-bar for pier column and footing (two-stage of Dia.51) and high grade of steel sheet with t16, SKY490. <Cost ratio 1.06> (△)
Evaluation	Most recommended	Less recommended

Note: ◎:Better O:Normal △: Worse

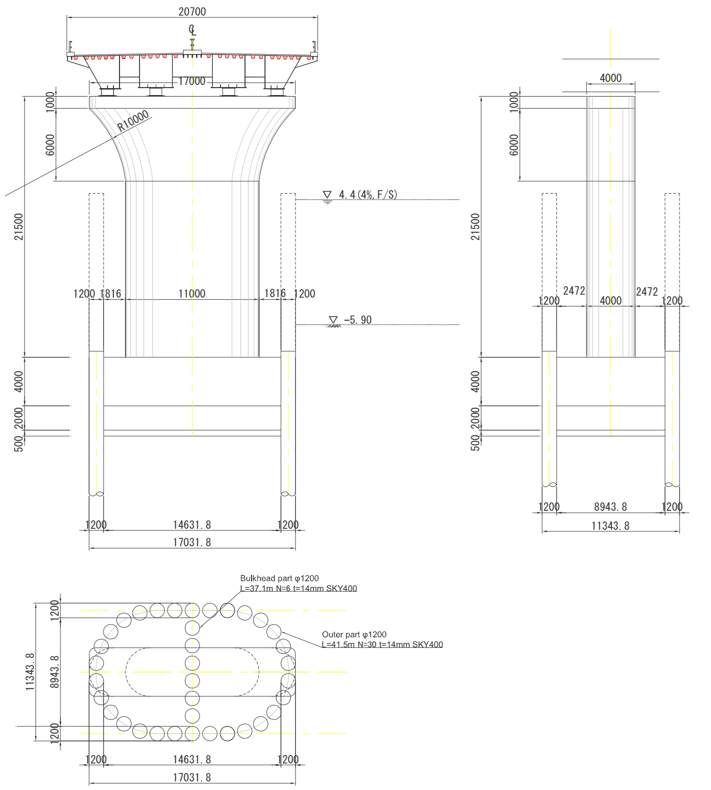
Source: JICA Study Team

4.3.1.3 Outline of the Proposed Substructure for Steel Box Girder Bridge in the B/D

Through the studies in Sections 4.3.2.3 to 4.3.2.7, the configuration of the substructures for steel box girder bridge is determined and are as shown in the table below.

Table 4.3.5 Structural Outline of the Substructure of P19 (Representing the Piers) for Steel Box Girder Bridge

Item	Description
Pier Column	
Shape:	Oval shape with an overhang
Size:	17 m width at top and 11 m at bottom Thickness is 4.0 m
Material:	Reinforced Concrete Class of concrete: 30 MPa Grade of rebar: SD345
Foundation	
Shape:	Oval shape
Size:	Dimension 17.0 m x 11.3 m Thickness of footing 4.0 m Thickness of bottom slab 2.0 m Thickness of Sand Mat 0.5 m Diameter of steel pipe 1.2 m Thickness of steel pipe: 14 mm Length of steel pipe: 41.5 m
Material	Grade of steel pipe: SKY400
Construction Method:	Foundation and Temporary Cofferdam Method



General View of the Substructure P19 Representing the Piers

Source: JICA Study Team

4.3.2 Detailed Design for Superstructure of the Steel Box Girder Bridge (7-Span Bridge)

4.3.2.1 Design Condition

(1) Profile

Span Length:

1.2 + 110.8 + 5@112.0 + 103.1 + *0.9* = 776.0 m (Bridge Length)

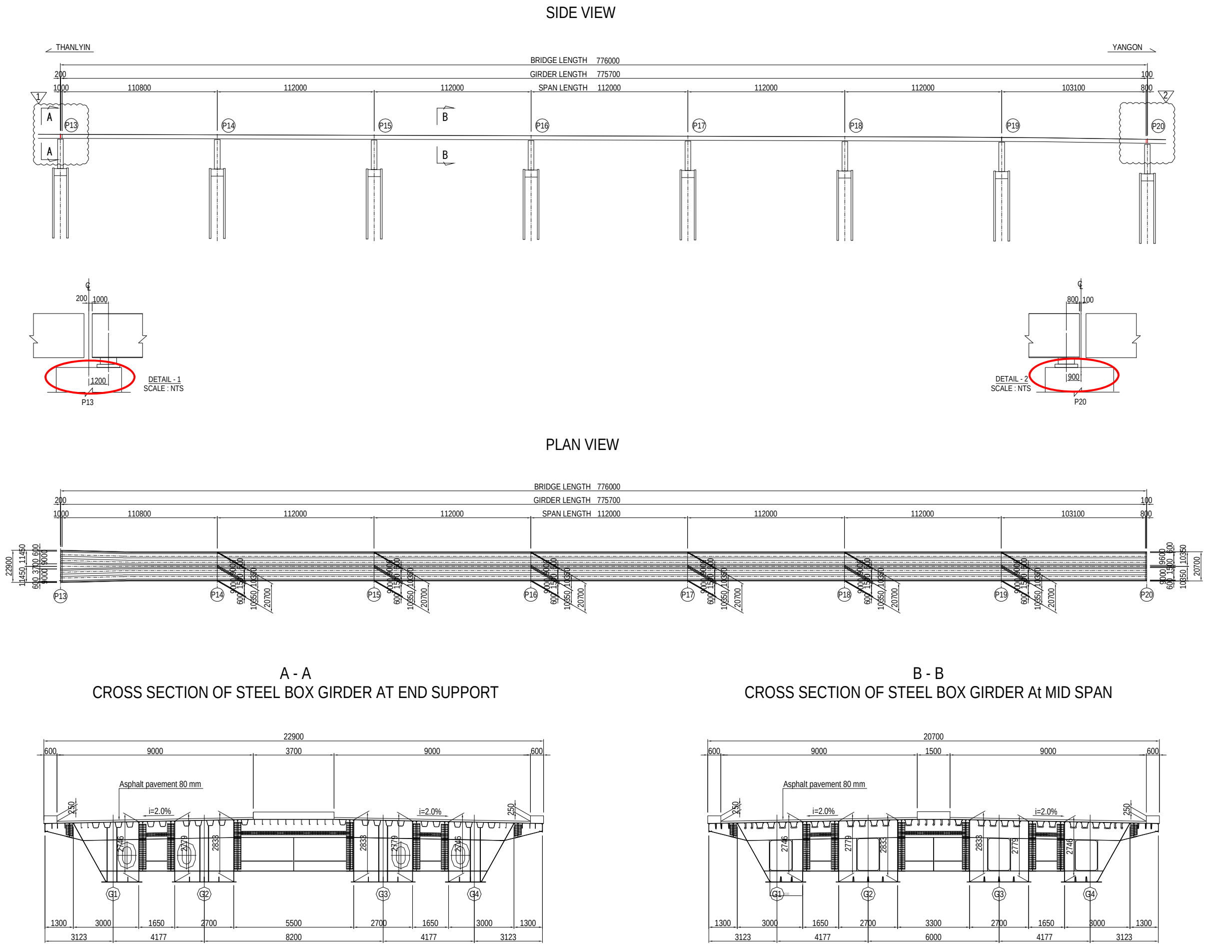
Italicized figures of 1.2 and 0.9 above show the combined length of the clearance and marginal length from the end girder to the bearing position. There has been a slight change for it is longer than the value on the B/D because of the displacement in consideration of the seismic behavior and temperature elongation.

The width composition is same as the B/D.

Normal Width *0.6* + 9.0 + *1.5* + 9.0 + *0.6* = 20.7 m

Widened Width *0.6* + 9.0 + *3.7* + 9.0 + *0.6* = 22.9 m

Italicized figures of 0.6, 1.5, and 3.7 above show the side barrier (coping) and median barrier (coping) width.

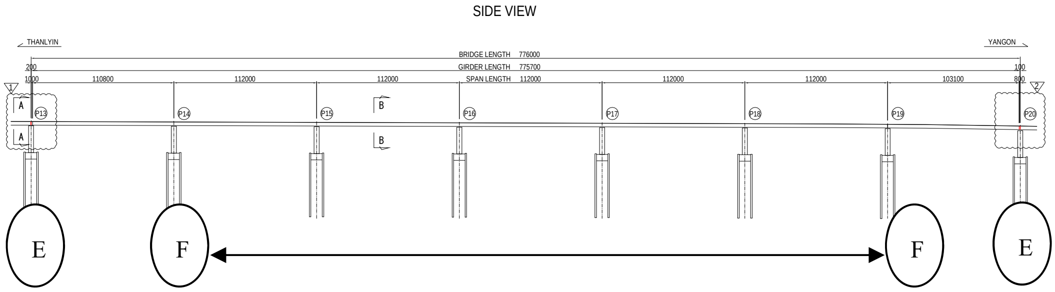


Source: JICA Study Team

Figure 4.3.5 General View

(2) Supporting Condition

- This bridge is supported by eight piers at the longitudinal road direction.
- Every girder has been assumed to be supported on elastic bearing that was rotatable and only longitudinally movable during the B/D.
- However, the end bearing capacity against rotation distortion due to live load was reviewed, and then it was decided that multi-fixed bearing system will be suitable in case that the substructure is built on soft foundation.
- Elasticity coefficient including flexibility of substructure on soft soil has been reviewed eventually at the design stage of substructure and bearing.

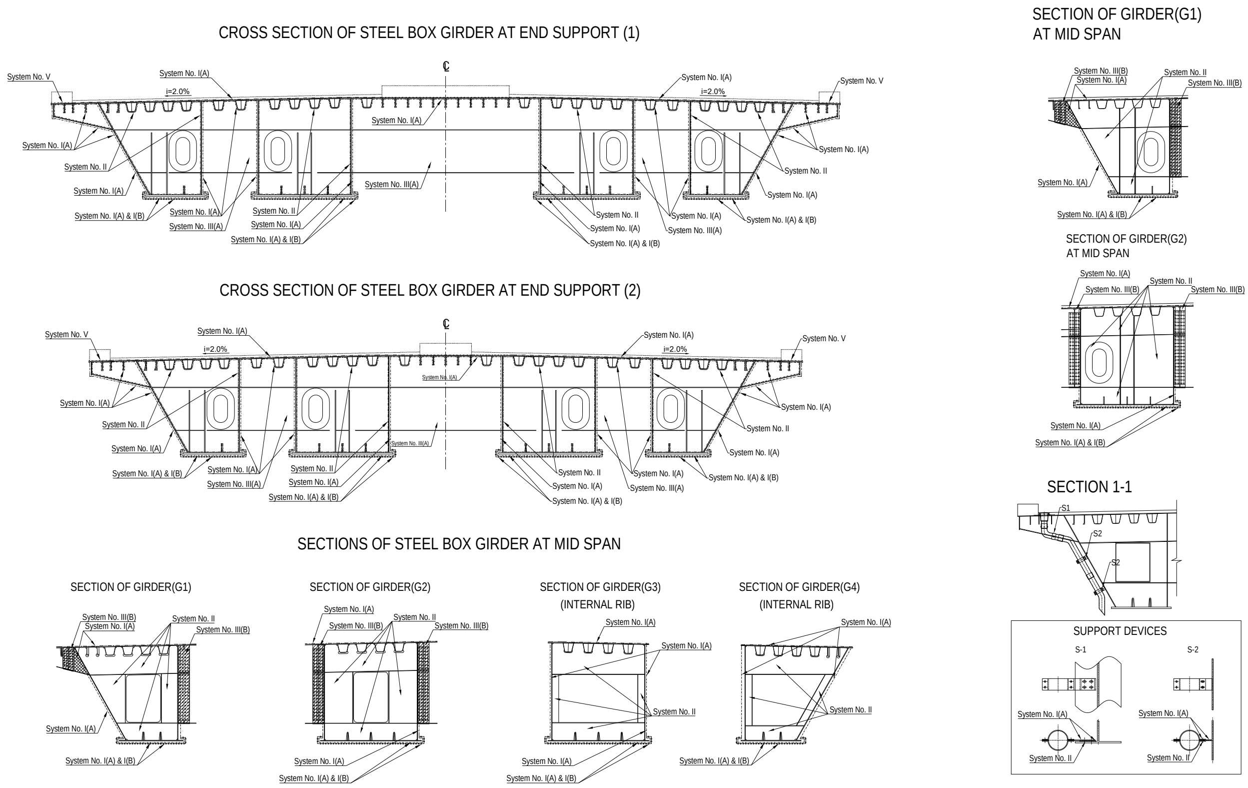


Source: JICA Study Team

Figure 4.3.6 Bearing Support Condition

(3) Sections of Girder

Cross sections of the girder are shown in following figure.



Source: JICA Study Team

Figure 4.3.7 Sections of Girder

4.3.3 Detailed Design for Superstructure of the Steel Box Girder Bridge (3-Span Bridge)

4.3.3.1 Design Condition

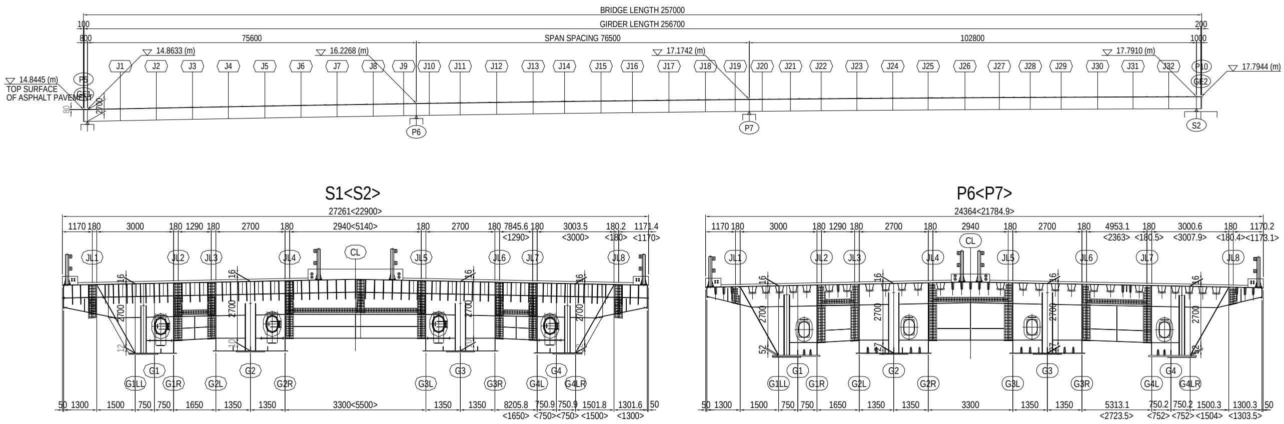
(1) Profile

Span Length:

0.9 m + 75.6 m + 76.5 m + 102.8 + *1.2 m* = 257.0 m (Bridge Length)

Italicized figures of 0.9 m and 1.2 m above show the combined length of the clearance and marginal length from the end girder to the bearing position.

The width composition is same as the B/D.



Source: JICA Study Team

Figure 4.3.8 General View

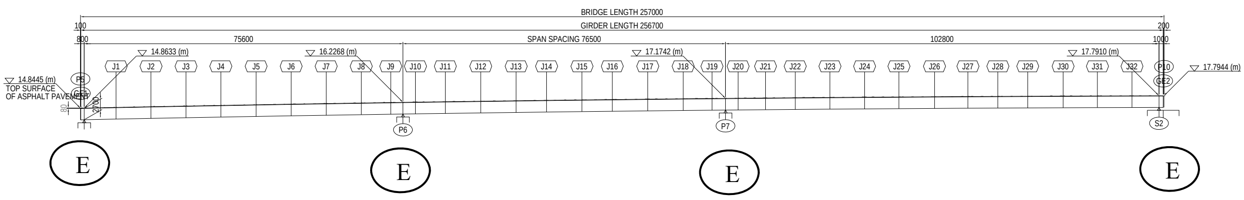
Normal Width (S1) *0.6* + 9.0 + *1.5* + 9.0 + *0.6* = 20.7 m

Widened Width (S2) *0.6* + 9.0 + *3.7* + 9.0 + *0.6* = 22.9 m

Italicized figures of 0.6, 1.5, and 3.7 above show the side barrier (coping) and median barrier (coping) width.

(2) Supporting Condition

- This bridge is supported by four (4) piers at the longitudinal road direction.
- Every girder has been assumed to be supported on elastic bearing that was rotatable and only longitudinally movable.
- Elasticity coefficient including flexibility of substructure on soft soil has been reviewed eventually at the design stage of substructure and bearing.

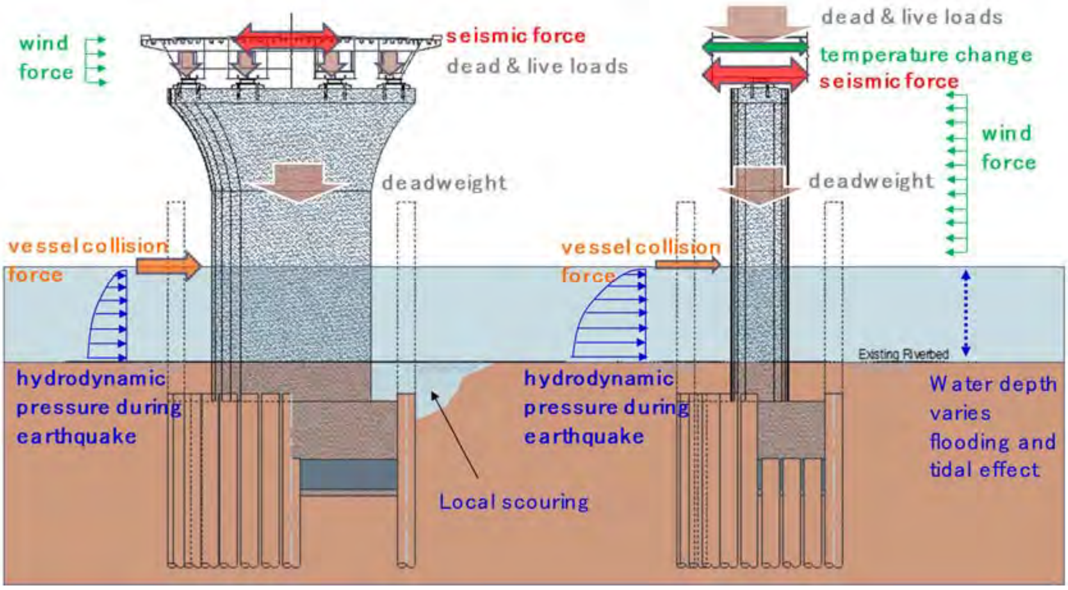


Source: JICA Study Team

Figure 4.3.9 Bearing Support Condition

4.3.4 Detailed Design for Substructure of Steel Box Girder Bridge (7-Span Bridge)

Based on the results of the B/D, which is presented in Section 4.3.2, further studies were carried out in the D/D for the piers from P14 to P19 taking into account the updated topographic, geological, and hydrologic conditions and loads from the superstructure.



Source: JICA Study Team

Figure 4.3.10 Image of External Forces to be Considered

4.3.4.1 SPSP Foundation Design

(1) Footing Top Elevation

Setting of the footing top elevation is very important because it will affect the stability of the structure in the long term and construction cost. For the design of the SPSP, in general, deeper setting of footing below the riverbed may require a thicker steel pipe and/or higher grade pile due to larger displacement and stress during construction.

Therefore, in this Project, footing top elevation is set to more than 1 m from the lowest elevation of existing riverbed among piers as shown in the table below, and projection of the footing above the riverbed after local scouring will be allowed and finally, the stability during ordinary and earthquake conditions will be considered in the design.

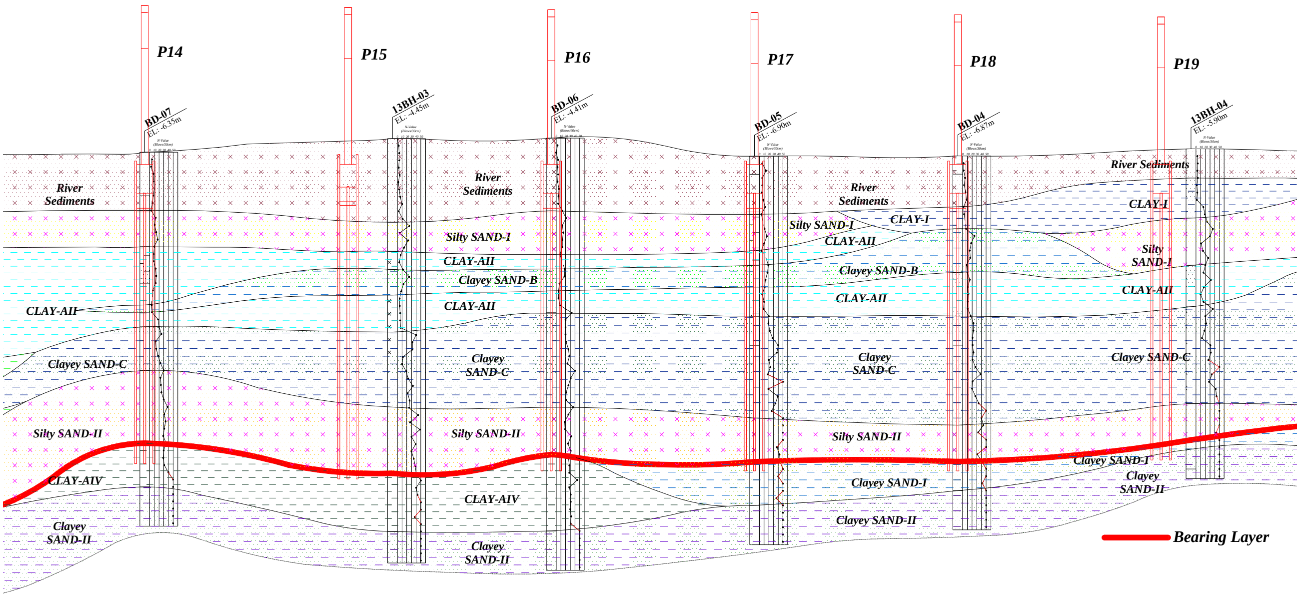
Table 4.3.6 Setting of Footing Top Elevation

Pier No.	Scour of Components				Riverbed Elevation (MSL+m)	Footing Top Elevation (MSL+m)	Scoured Level (MSL+m)
	Total Scour (m)	Scour for Pier (m)	Scour for Pile Cap (m)	Contraction Scour (m)			
P14	5.15	4.03	0.76	0.36	-6.28	-8.06	-11.43
P15	5.75	4.73	0.66	0.36	-5.09	-8.06	-10.84
P16	5.09	4.11	0.63	0.36	-5.26	-8.06	-10.36
P17	3.00	2.28	0.36	0.36	-6.70	-8.06	-9.70
P18	3.01	2.12	0.53	0.36	-6.99	-8.06	-10.00
P19	2.90	2.09	0.45	0.36	-6.88	-8.06	-9.78

Source: JICA Study Team

(2) Pile Tip Elevation

The tip of the steel pipe pile foundation of the well type in principle has to be supported by good soil ground layer, which assumes an N-value greater than 30 for sand soil and 20 for clay soil. In addition, the supporting layer must have a sufficient thickness not to be affected by the lower layers. Pile tip is set into the bearing layer to more than the length of the diameter of pile, namely, 1.2 m as shown in the figure below.

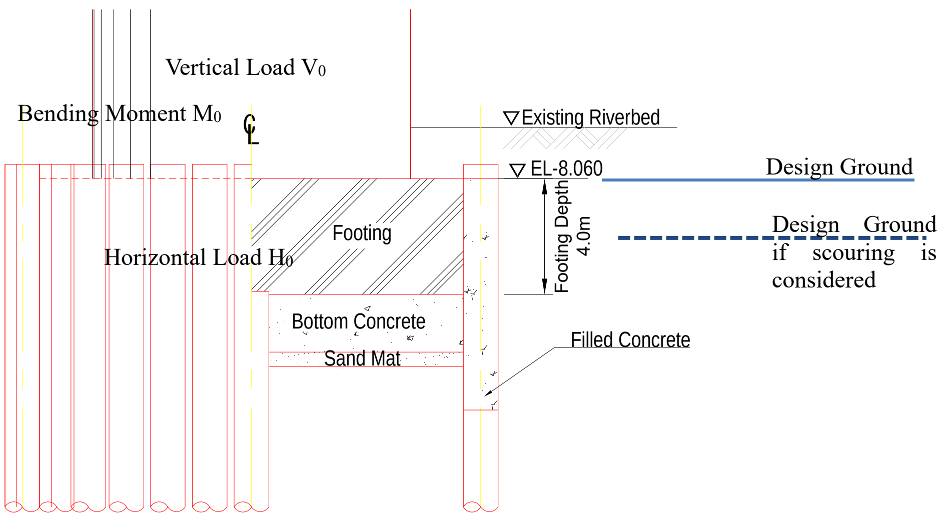


Source: JICA Study Team

Figure 4.3.11 Soil Profile and Pile Tip Position

(3) Design External Force

Design external force acting as point forces through the axis of the centroid on the center of the bottom of the footing is considered for the SPSP foundation design as shown in the figure below.



Source: JICA Study Team

Figure 4.3.12 Point of Loading of External Forces

For critical design load combination, the combined external forces during earthquake condition (Level-1) are summarized in the table below.

Table 4.3.7 Design External Force (V_0,H_0,M_0) at the top of Footing during Earthquake Condition

	Load Direction	V_0 (kN)	H_0 (kN)	M_0 (kN.m)
P14	Bridge axis direction	55,800	16,200	244,000
	Bridge axis perpendicular direction	55,800	15,100	267,500
P15	Bridge axis direction	51,700	15,600	238,900
	Bridge axis perpendicular direction	51,700	13,600	233,800
P16	Bridge axis direction	52,800	15,800	241,200
	Bridge axis perpendicular direction	52,800	13,900	238,300
P17	Bridge axis direction	51,800	16,000	240,500
	Bridge axis perpendicular direction	51,800	13,700	231,100
P18	Bridge axis direction	51,000	16,300	239,700
	Bridge axis perpendicular direction	51,000	13,600	223,700
P19	Bridge axis direction	53,100	16,300	240,600
	Bridge axis perpendicular direction	53,100	14,200	236,200

Source: JICA Study Team

(4) Verification of Foundation Dimension

1) Bearing Capacity and Displacement

Stability of the SPSP foundation is verified by bearing capacity and displacement and its results are summarized in the tables below.

Table 4.3.8 Verification of Bearing Capacity

Bride Axis Direction Unit: kN

Pier No.	Item	Ordinary Condition*1			Earthquake Condition*2		
		Vertical Reaction	Allowable Value	Judgement	Vertical Reaction	Allowable Value	Judgement
P14	Axial compression resistance	1,821 <	2,855	OK	1,553 <	4.259	OK
	Pulling-out resistance	1,821 >	-1,043	OK	1,546 >	-1,661	OK
P15	Axial compression resistance	1,729 <	2,007	OK	1,496 <	3,011	OK
	Pulling-out resistance	1,729 >	-1,006	OK	1,375 >	-1,566	OK
P16	Axial compression resistance	1,752 <	2,406	OK	1,521 <	3,609	OK
	Pulling-out resistance	1,752 >	-991	OK	1,408 >	-1,558	OK
P17	Axial compression resistance	1,693 <	1,763	OK	1,510 <	2,644	OK
	Pulling-out resistance	1,693 >	-893	OK	1,367 >	-1,359	OK
P18	Axial compression resistance	1,660 <	1,747	OK	1,491 <	2,621	OK
	Pulling-out resistance	1,660 >	-875	OK	1,342 >	-1,323	OK
P19	Axial compression resistance	1,724 <	1,791	OK	1,574 <	2,687	OK
	Pulling-out resistance	1,724 >	-850	OK	1,375 >	-1,290	OK

Bridge Axis Perpendicular Direction Unit: kN

Pier No.	Item	Ordinary Condition*1 Vertical Reaction		Allowable Value	Judgement	Earthquake Condition*2 Vertical Reaction		Allowable Value	Judgement
P14	Axial compression resistance	1,821	<	2,855	OK	1,801	<	4,259	OK
	Pulling-out resistance	1,821	>	-1,043	OK	1,299	>	-1,661	OK
P15	Axial compression resistance	1,729	<	2,007	OK	1,492	<	3,011	OK
	Pulling-out resistance	1,729	>	-1,006	OK	-1,379	>	-1,566	OK
P16	Axial compression resistance	1,752	<	2,406	OK	1,527	<	3,609	OK
	Pulling-out resistance	1,752	>	-991	OK	1,402	>	-1,558	OK
P17	Axial compression resistance	1,693	<	1,763	OK	1,481	<	2,644	OK
	Pulling-out resistance	1,693	>	-893	OK	1,396	>	-1,359	OK
P18	Axial compression resistance	1,660	<	1,747	OK	1,491	<	2,621	OK
	Pulling-out resistance	1,660	>	-875	OK	1,342	>	-1,323	OK
P19	Axial compression resistance	1,724	<	1,791	OK	1,528	<	2,687	OK
	Pulling-out resistance	1,724	>	-850	OK	1,421	>	-1,290	OK

Note: *1: ordinary condition at low tide in spring tide w/o local scouring

*2: earthquake condition at 1/2 of maximum local scouring

Source: JICA Study Team

Table 4.3.9 Verification of Displacement

Unit: cm

Pier No.	Item	Earthquake Condition*1 Displacement*2		Allowable Value	Judgement
P14	Bride Axis Direction	3.3	<	5.0	OK
	Bridge axis perp. direction	3.0	<	5.0	OK
P15	Bride Axis Direction	3.2	<	5.0	OK
	Bridge axis perp. direction	2.5	<	5.0	OK
P16	Bride Axis Direction	2.8	<	5.0	OK
	Bridge axis perp. direction	2.2	<	5.0	OK
P17	Bride Axis Direction	2.6	<	5.0	OK
	Bridge axis perp. direction	2.0	<	5.0	OK
P18	Bride Axis Direction	2.9	<	5.0	OK
	Bridge axis perp. direction	2.1	<	5.0	OK
P19	Bride Axis Direction	2.5	<	5.0	OK
	Bridge axis perp. direction	2.0	<	5.0	OK

Note: *1: earthquake condition at 1/2 of maximum local scouring

*2: displacement at design ground level

Source: JICA Study Team

2) Stress of Outer Steel Pipe Sheet Piles

In a steel pipe sheet pile foundation of the type that also serves as a temporary cofferdam, the steel pipe sheet piles are used as cofferdam walls during the work execution. Therefore, cofferdam walls shall be verified to be safe against the loads acting during temporary work.

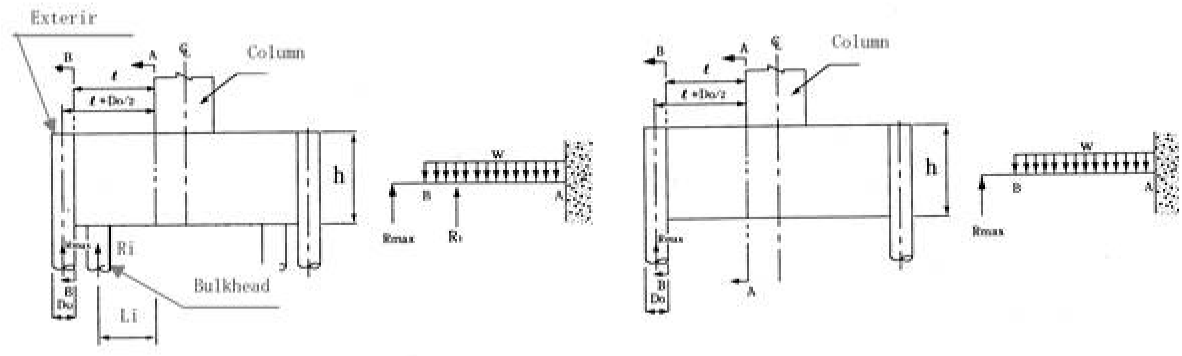
(5) Verification of Structural Members

1) Footing (Top Slab)

a) Design Sections

The footing of a steel pipe sheet pile foundation generally has a large rigidity and is rigidly connected to the steel pipe sheet piles. It can be calculated as a cantilever with the fixed end at the outer edge of the lower end of the body. Reaction by the soil under the footing inside the well will not be considered in the footing design for safety.

A verification of the sections of footing will be made at the section A-A for bending moment and section B-B for shear force as shown in figure below, and such section forces shall be calculated per unit width at the position of the steel pipe sheet pile that produces the maximum vertical reaction force.



(Bridge Axis Direction) *(Bridge Axis Perpendicular Direction)*

Source: Design and Construction Manual Published by the Japanese Association for Steel Pipe Piles

Figure 4.3.13 Section Calculation Model and Design Section of Footing

b) Design Conditions

- Width of footing for design b = 100.0 cm, thickness of footing h = 400.0 cm
- Concrete design strength: 24 N/mm^2
- Applied reinforcement bar: SD345 (underwater member)

c) Rebar Arrangement

P14 and P19

<u>Bridge Axis Direction</u>

Upper tension: cover 150 mm D32@260
cover 300 mm D32@260

Lower tension: cover 300 mm D51@183
cover 500 mm D51@302

<u>Bridge Axis Perpendicular Direction</u>

Upper tension: cover 118 mm D32@209
cover 268 mm D32@408

Lower tension: cover 230 mm D51@209
cover 430 mm D51@408

P15-P18

<u>Bridge Axis Direction</u>

Upper tension: cover 150 mm D32@260
cover 300 mm D32@260

Lower tension: cover 300 mm D51@183
cover 500 mm D51@370

<u>Bridge Axis Perpendicular Direction</u>

Upper tension: cover 118 mm D32@209
cover 268 mm D32@408

Lower tension: cover 230 mm D51@209
cover 430 mm D51@408

It is noted that shear reinforcement is arranged by D22@600 at chessboard patterns, which quantity is equal to approximately 0.15%, although it is not required in the calculation.

d) Verification of Stress in Footing and Content of Rebar

Design of bending moment is verified by tensile stress and content of rebar in the section as deep beam which has a deeper depth of the footing than 1/2 of design span that is the distance from the edge of pier column to the inside surface of the outer steel sheet pile.

Design of shear force is verified so that average shear stress should be within the allowable shear stress of concrete or allowable shear stress of concrete and shear reinforcement.

2) Connection between SPSP and Footing

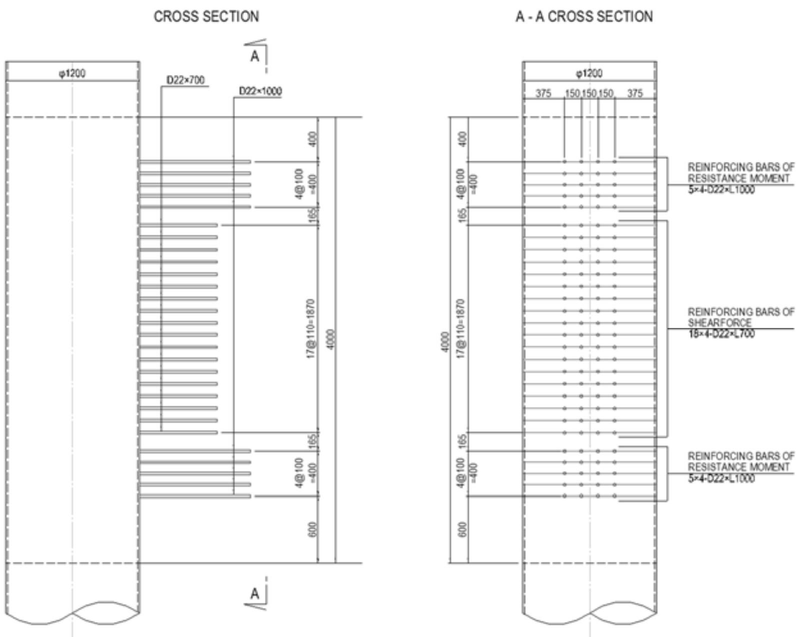
The required number of moment and shear reinforcement for connection between SPSP and footing by Reinforcement Stud Method is calculated as follows:

a) Design Condition

- Applied reinforcement bar: SD345 (underwater member), Diameter 22 mm
- Concrete design strength: 24 N/mm^2
- Material of SPSP: SKY490
- Joint method: Reinforcement Stud Method

b) Required Number of Moment and Shear Reinforcement

The required number of reinforcement is 16-17 for moment and it ranges between 54 and 72 for shear. Therefore, 20 studs for moment for all piers, 72 studs for shear for P15-P19 and 76 studs for shear for P14 are arranged as shown in the figure below and it was verified by the allowable stress summarized in the table below.



Source: JICA Study Team

Figure 4.3.14　Layout of Reinforcement Stud

3) Connection between Footing and Pile Head of Bulkhead Piles

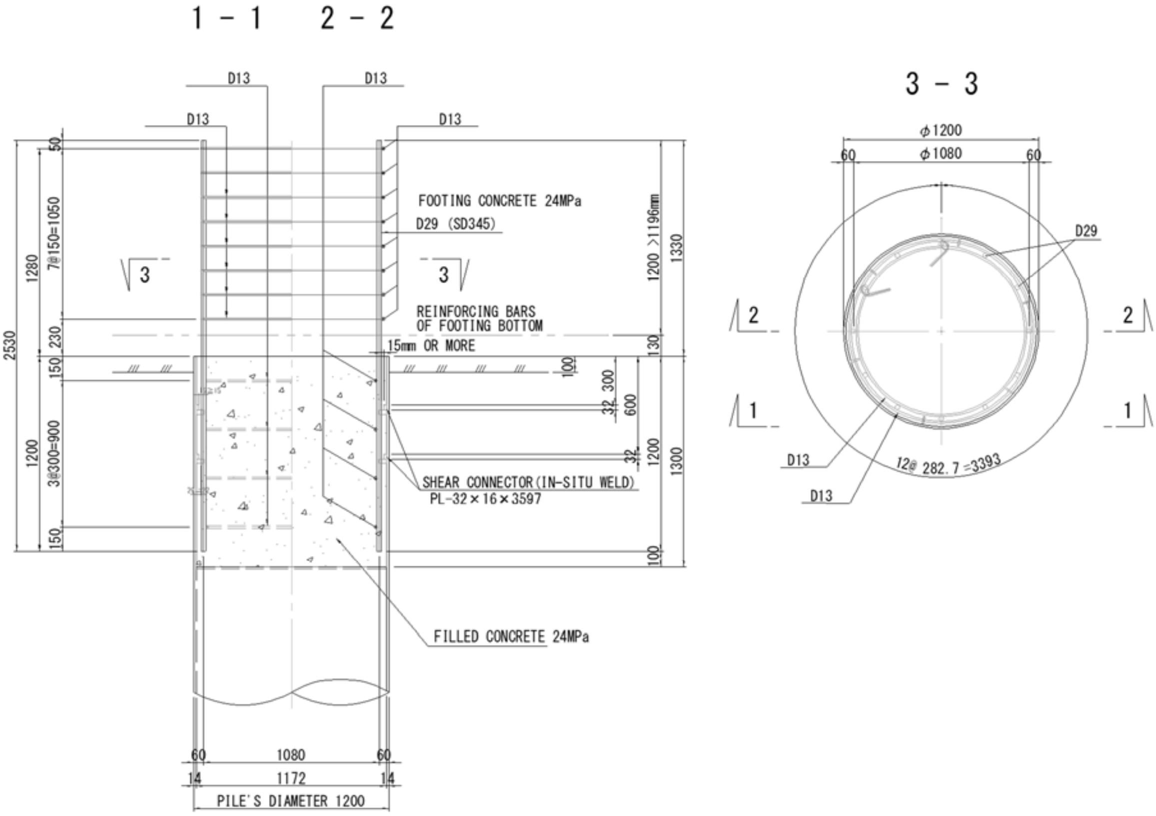
The pile head of the bulkhead part of the SPSP will be inserted and rigidly connected by reinforcing bars with the footing, and it has been verified in terms of stress and content of reinforcement as follows:

a) Design Condition

- Applied reinforcement bar: SD345 (underwater member)
- Concrete design strength: 24 N/mm^2

b) Rebar Arrangement

Steel pile is inserted at 100 mm to the footing and it is fixed by 12 numbers of main reinforcements of φ29 mm and filled concrete as shown in the figure below.



Source: JICA Study Team

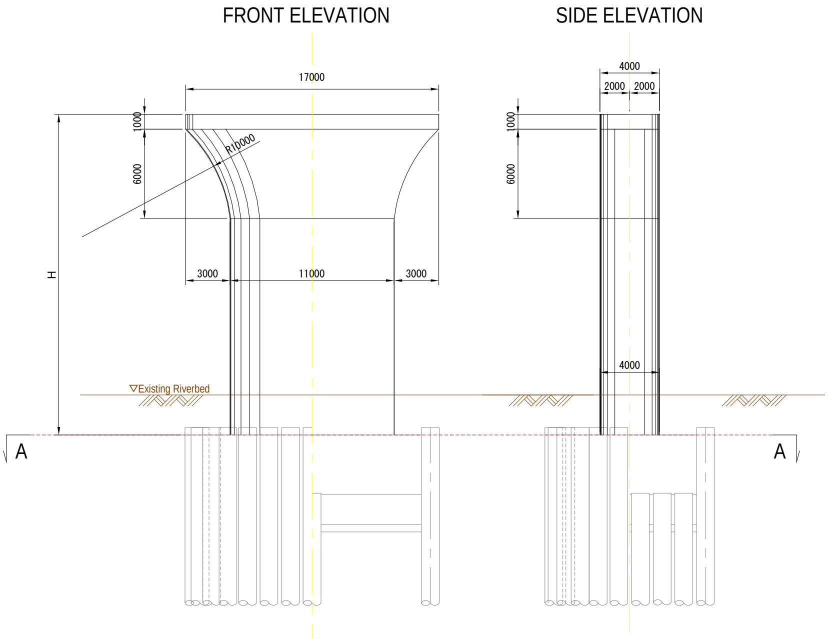
Figure 4.3.15 Detail of Pile Head Connection

4.3.4.2 RC Pier

(1) Verification of RC Pier Column

1) Design Section

A verification of the sections of pier column will be made at the section A-A against bending moment and shear force in each bridge axis and axis perpendicular direction as shown in the figure below.



Source: JICA Study Team

Figure 4.3.16 Design Section of Pier Column

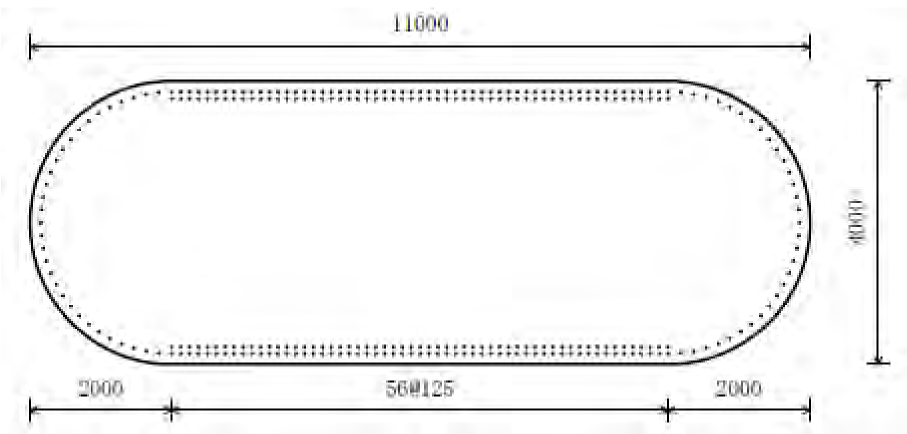
2) Rebar Arrangement

a) Main Reinforcement

- Main reinforcement is arranged as shown in the figure below, and no deduction of the rebar is made through the pier column.

<u>P14-P19</u>

Cover (mm)	Straight Section		Arc Section	
	Diameter	Arrangement	Diameter	Arrangement
150	D38	56@125	D38	2 x 32@182
250	D38	56@125	-	-



Source: JICA Study Team

Figure 4.3.17 Rebar Arrangement (Main Reinforcement)

b) Shear Reinforcement

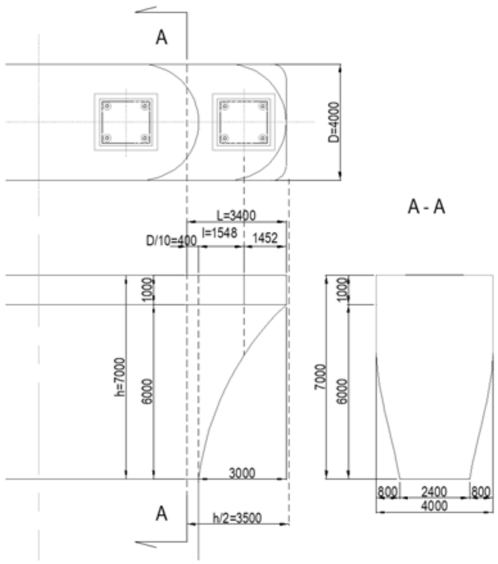
- Lateral tie to avoid the column from buckling due to shear force: D22, double reinforcement, 150 mm pitch through the column

- Intermediate hoop to avoid the lateral tie from jutting outside: 8 nos. for bridge axis direction and 4 nos. for bridge axis perpendicular direction per cross section, 150 mm pitch through the column

(2) Verification of Beam at Pier Head

1) Design Section

Since the distance from the front of the column to the loading point (bearing), l, is smaller than the height of beam, h, namely h/l=7000/1548=4.5≥1.0, this kind of beam will be designed as a corbel. And, design section (A-A) is set at 400 mm inside of column because of the oval column shape as shown in the figure below. It will be verified at A-A section in terms of bending moment and shear. The section at h/2 (=3500 mm) from A-A section is outside of the beam, so verification of shear force will be made only at A-A section.

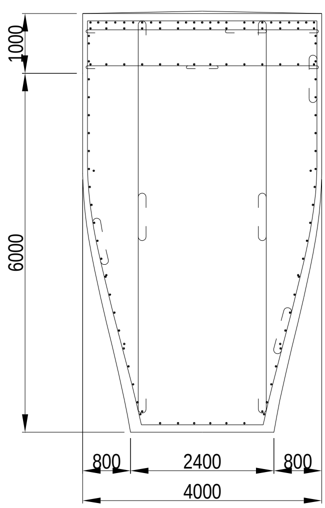


Source: JICA Study Team

Figure 4.3.18 Design Section of Pier Head Beam

2) Rebar Arrangement

Main reinforcement and stirrup at the design section (A-A) is arranged as shown in the figure below.



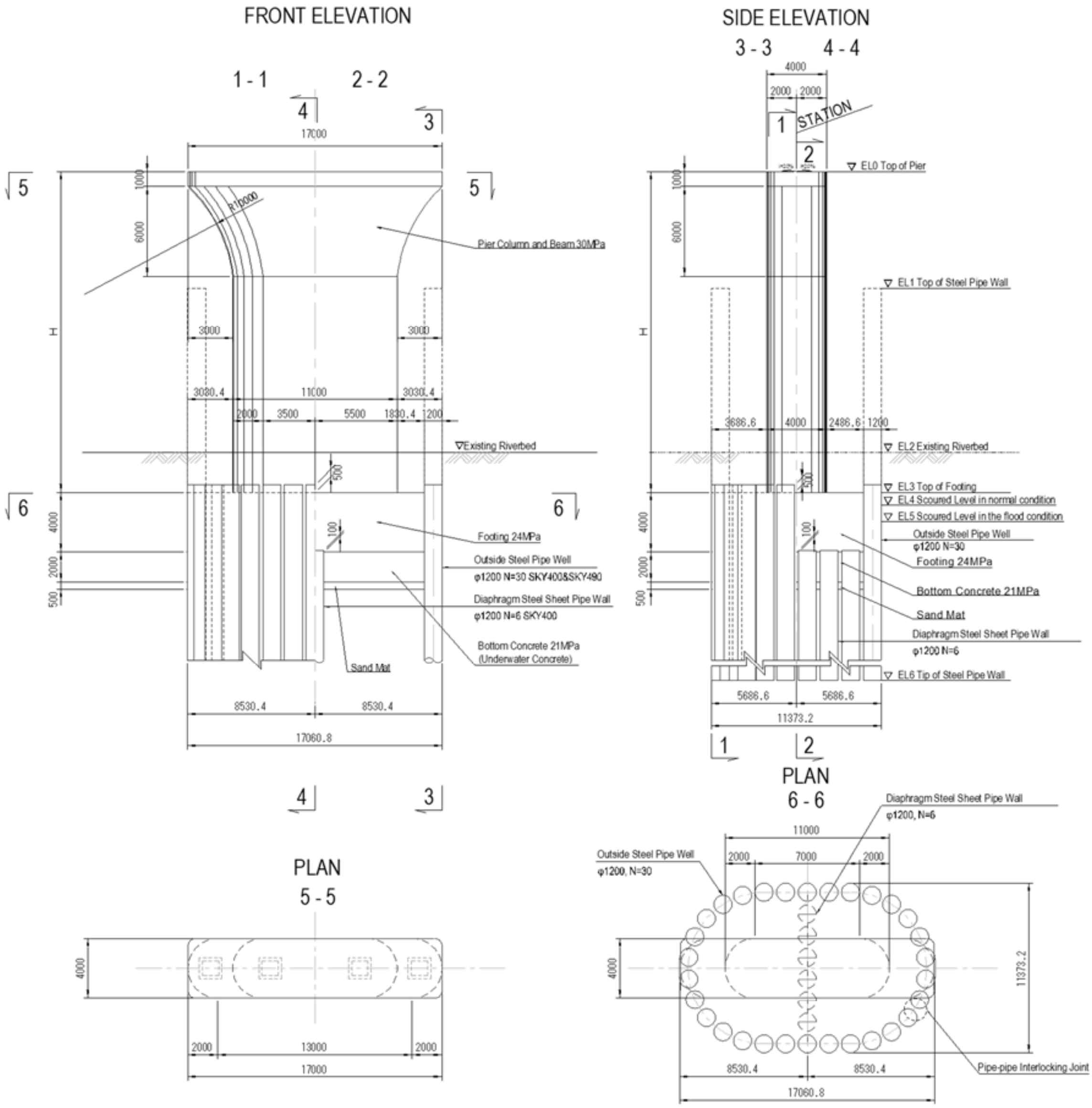
Location	Cover (mm)	Diameter	Arrangement
Upper (main reinforcement)	150	D32	24@155.8 in average
	250	D32	13@287.7 on average
Lower	150	D32	5@282 in average
Side	103	D22	(125+20@300+200) x 2 sides
Stirrup	-	D22	150mm pitch

Source: JICA Study Team

Figure 4.3.19 Rebar Arrangement (Main Reinforcement)

4.3.4.3 Structure Drawing

Through the basic design and the detailed design presented in Sections 4.3.2 and 4.3.4, the substructure of P14-P19 is designed in terms of economical and structural aspects. The structural drawing is shown in the figure below.



PIER No.	STATION	Elevation (m)						
		EL0 (H)	EL1	EL2	EL3	EL4	EL5	EL6
P14	1+424	+13.840m (21.9m)	+4.840m	-6.276m	-8.060m	-8.850m	-11.430m	-49.160m
P15	1+536	+13.540m (21.6m)	+4.840m	-5.091m	-8.060m	-7.970m	-10.840m	-52.160m
P16	1+648	+13.240m (21.3m)	+4.840m	-5.262m	-8.060m	-7.810m	-10.360m	-50.160m
P17	1+760	+12.840m (20.9m)	+4.840m	-6.695m	-8.060m	-8.200m	-9.700m	-50.160m
P18	1+872	+12.540m (20.6m)	+4.840m	-6.989m	-8.060m	-8.490m	-10.000m	-50.160m
P19	1+984	+12.240m (20.3m)	+4.840m	-6.878m	-8.060m	-8.330m	-9.780m	-48.660m

Source: JICA Study Team

Figure 4.3.20 Structure Drawing of Substructure of P14-P19

4.3.5 Detailed Design for Substructure of Steel Box Girder Bridge (3-Span Bridge)

4.3.5.1 SPSP Foundation Design

(1) Footing Top Elevation

Since it is located at the riverbank, footing top elevation is set to deeper one, of which more than 1 m from the elevation of existing riverbed or from the lowest water level (L.W.L.=-2.39m) to prevent projection of steel pipe above the water.

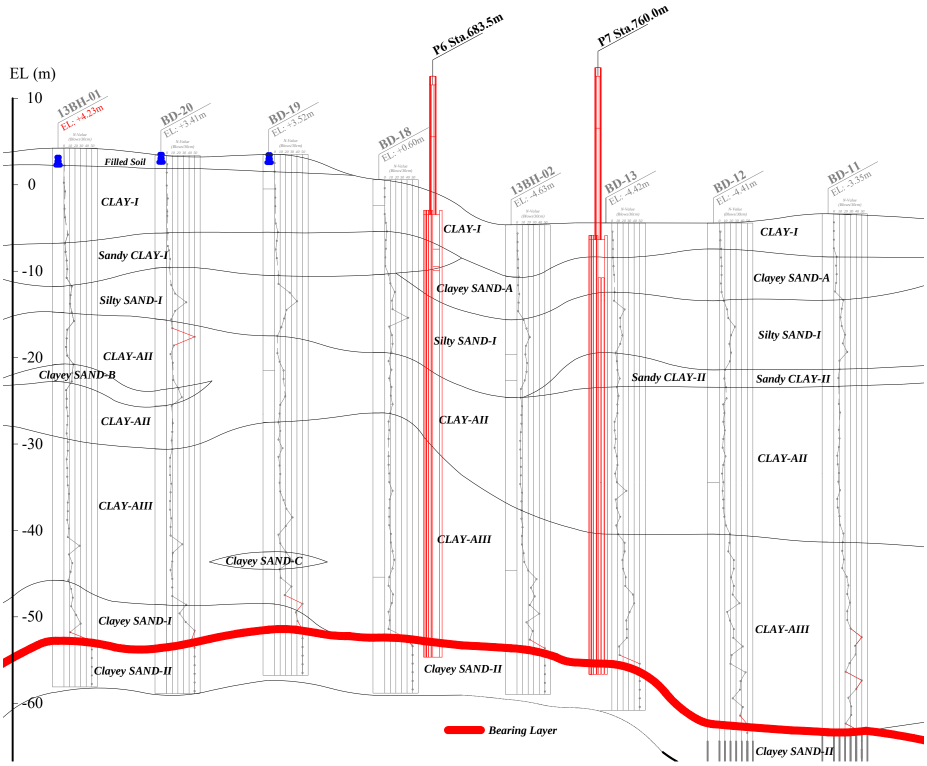
Table 4.3.10 Setting of Footing Top Elevation

Pier No.	Scour of Components				Riverbed Elevation (MSL+m)	Footing Top Elevation (MSL+m)	Scoured Level (MSL+m)
	Total Scour (m)	Scour for Pier (m)	Scour for Pile Cap (m)	Contraction Scour (m)			
P6	3.84	3.15	0.36	0.33	-1.72	-3.45	-5.56
P7	2.32	1.01	0.99	0.33	-5.35	-6.35	-7.67

Source: JICA Study Team

(2) Pile Tip Elevation

Pile tip is set into the bearing layer of Clayey Sand-II with N-value 50 (sand soil) to more than the length of the diameter of pile 1.2 m, and the pile tip elevation is EL-54.660m at P6 and EL-56.660m at P7 as shown in the figure below.



Source: JICA Study Team

Figure 4.3.21 Soil Profile and Pile Tip Position

(3) Design External Force

For critical design load combination, the combined external forces during earthquake condition (Level-1) are summarized in the table below.

Table 4.3.11 Design External Force (V_0,H_0,M_0) at the top of footing during Earthquake

	Load Direction	V_o (kN)	H_0 (kN)	M_0 (kN.m)
P6	Bridge axis direction	45,335	11,100	123,800
	Bridge axis perpendicular direction	45,335	10,800	146,600
P7	Bridge axis direction	48,932	11,700	153,600
	Bridge axis perpendicular direction	48,932	13,100	219,800

Source: JICA Study Team

(4) Verification of Foundation Dimension

1) Bearing Capacity and Displacement

Stability of the SPSP foundation is verified by bearing capacity and displacement and its results are summarized in the tables below.

Table 4.3.12 Verification of Bearing Capacity

Bride Axis Direction Unit: kN

Pier No.	Item	Ordinary Condition*1			Earthquake Condition*2		
		Vertical Reaction	Allowable Value	Judgement	Vertical Reaction	Allowable Value	Judgement
P6	Axial compression resistance	1,567<	3,946	OK	1,379<	5,919	OK
	Pulling-out resistance	1,567>	-1,863	OK	1,288>	-3,196	OK
P7	Axial compression resistance	1,554<	3,273	OK	1,412<	4,909	OK
	Pulling-out resistance	1,544>	-1,686	OK	1,306>	-2,855	OK

Bridge Axis Perpendicular Direction Unit: kN

Pier No.	Item	Ordinary Condition*1			Earthquake Condition*2		
		Vertical Reaction	Allowable Value	Judgement	Vertical Reaction	Allowable Value	Judgement
P6	Axial compression resistance	1,567<	3,946	OK	1,388<	5,919	OK
	Pulling-out resistance	1,567>	-1,863	OK	1,279>	-3,196	OK
P7	Axial compression resistance	1,554<	3,273	OK	1,390<	4,909	OK
	Pulling-out resistance	1,544>	-1,686	OK	1,328>	-2,855	OK

Note: *1: ordinary condition at low tide in spring tide w/o local scouring

*2: earthquake condition at 1/2 of maximum local scouring

Source: JICA Study Team

Table 4.3.13 Verification of Displacement

Pier No.	Item	Earthquake Condition*1		
		Displacement*2	Allowable Value	Judgement
P6	Bride Axis Direction	2.2cm <	5.0cm	OK
	Bridge axis perp. direction	1.6cm <	5.0cm	OK
P7	Bride Axis Direction	1.9cm <	5.0cm	OK
	Bridge axis perp. direction	1.8cm <	5.0cm	OK

Note: *1: earthquake condition at 1/2 of maximum local scouring

*2: displacement at design ground level

Source: JICA Study Team

2) Stress of Outer Steel Pipe Sheet Piles

As explained in 7-span bridge part, cofferdam walls shall be verified to be safe against the loads acting during temporary work.

(5) Verification of Structural Members

1) Footing (Top Slab)

a) Design Sections

The same sections of footing as 7-span bridge will be verified in terms of bending moment and shear force.

b) Design Conditions

- Width of footing for design b = 100.0 cm, thickness of footing h = 400.0 cm
- Concrete design strength: 24 N/mm^2
- Applied reinforcement bar: SD345 (underwater member)

c) Rebar Arrangement

P6

Bridge Axis Direction

Upper tension:	cover 150 mm D32@288
Lower tension:	cover 150 mm D32@203
	cover 300 mm D32@208

Bridge Axis Perpendicular Direction

Upper tension:	cover 120 mm D29@189
Lower tension:	cover 118 mm D32@189
	cover 268 mm D32@201

P7

Bridge Axis Direction

Upper tension:	cover 150 mm D29@278
	cover 300 mm D29@286
Lower tension:	cover 290 mm D38@228
	cover 440 mm D38@234

Bridge Axis Perpendicular Direction

Upper tension:	cover 121 mm D29@198
	cover 271 mm D29@410
Lower tension:	cover 236 mm D38@198
	cover 386 mm D38@212

It is noted that shear reinforcement is arranged by D22 at approximately 600mm at chessboard patterns, which quantity is equal to approximately 0.15%, although it is not required in the calculation.

d) Verification of Stress in Footing and Content of Rebar

Design of bending moment is verified by tensile stress and content of rebar in the section as deep beam which has a deeper depth of the footing than 1/2 of design span that is the distance from the edge of pier column to the inside surface of the outer steel sheet pile.

Design of shear force is verified so that average shear stress should be within the allowable shear stress of concrete or allowable shear stress of concrete and shear reinforcement.

2) Connection between SPSP and Footing

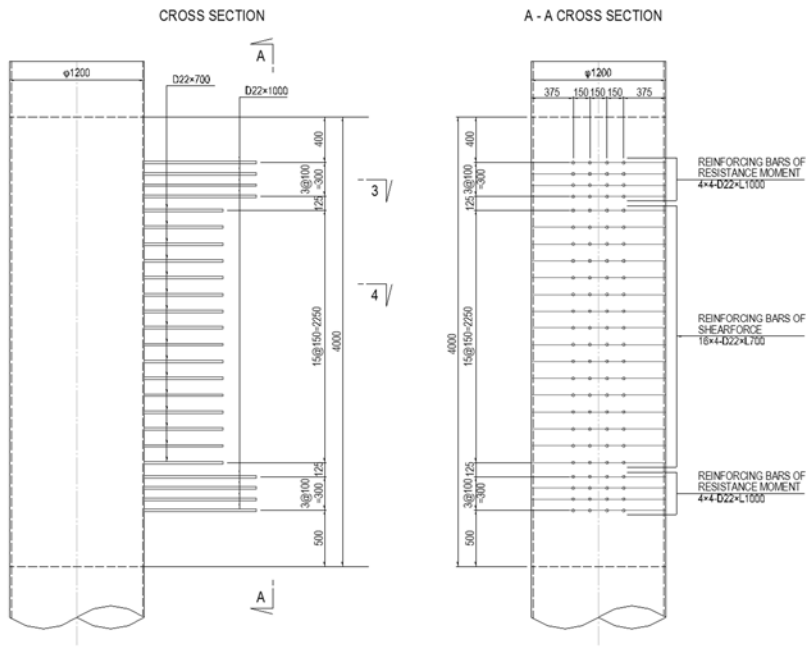
The required number of moment and shear reinforcement for connection between SPSP and footing by Reinforcement Stud Method is calculated as follows:

a) Design Condition

- Applied reinforcement bar: SD345 (underwater member), Diameter 22 mm
- Concrete design strength: 24 N/mm^2
- Material of SPSP: SKY400
- Joint method: Reinforcement Stud Method

b) Required Number of Moment and Shear Reinforcement

The required number of reinforcement is 12 and 13 for moment and 43 and 50 for shear, respectively at P6 and P7. Therefore, 16 studs for moment at both piers, 56 studs for shear at P6 and 64 studs for shear at P7 are arranged as shown in the figure below and it was verified by the allowable stress summarized in the table below.



Source: JICA Study Team

Figure 4.3.22 Layout of Reinforcement Stud (P7)

3) Connection between Footing and Pile Head of Bulkhead Piles

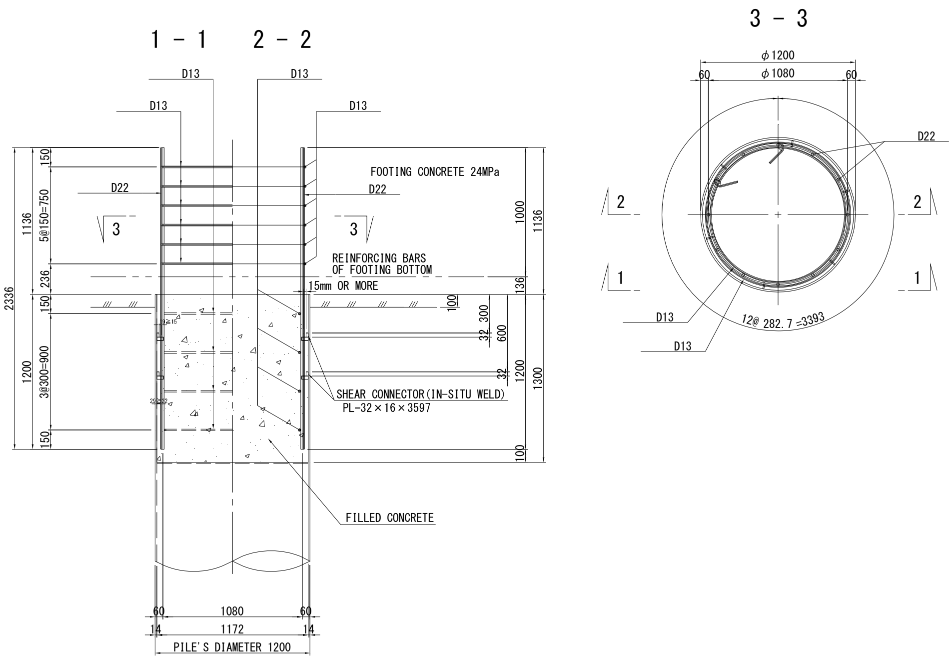
Since P7 has the bulkhead steel sheet pile, the connection of the pile head is verified in terms of stress and content of reinforcement as follows:

a) Design Condition

- Applied reinforcement bar: SD345 (underwater member)
- Concrete design strength: 24 N/mm^2

b) Rebar Arrangement

Steel pile is inserted at 100 mm to the footing and it is fixed by 12 numbers of main reinforcements of φ22 mm and filled concrete as shown in the figure below.



Source: JICA Study Team

Figure 4.3.23 Detail of Pile Head Connection (P7)

4.3.5.2 RC Pier

(1) Verification of RC Pier Column

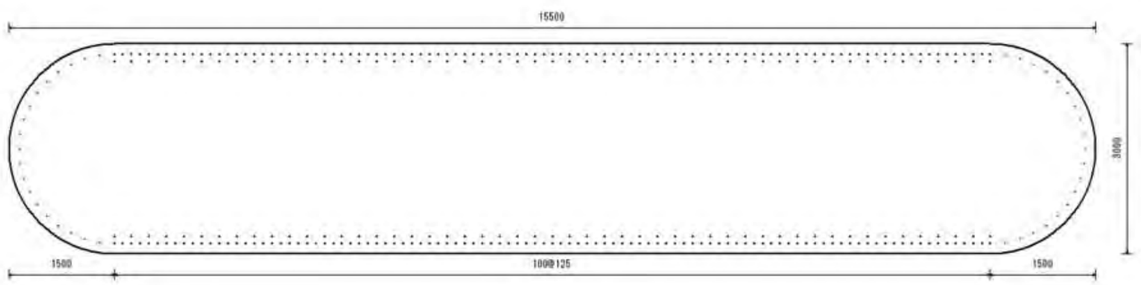
1) Rebar Arrangement

a) Main Reinforcement

- Main reinforcement is arranged as shown in the figure below, and no deduction of the rebar is made through the pier column.

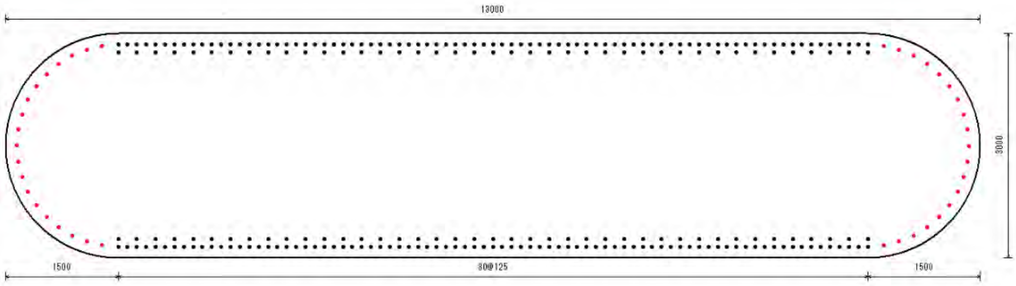
<u>P6</u>

Cover (mm)	Straight Section		Arc Section	
	Diameter	Arrangement	Diameter	Arrangement
150	D32	100@125	D32	2 x nos.19/side
250	D32	50@250	-	-



<u>P7</u>

Cover (mm)	Straight Section		Arc Section	
	Diameter	Arrangement	Diameter	Arrangement
150	D38	80@125	D38	2 x nos.19/side
250	D38	40@250	-	-



Source: JICA Study Team

Figure 4.3.24 Rebar Arrangement (Main Reinforcement)

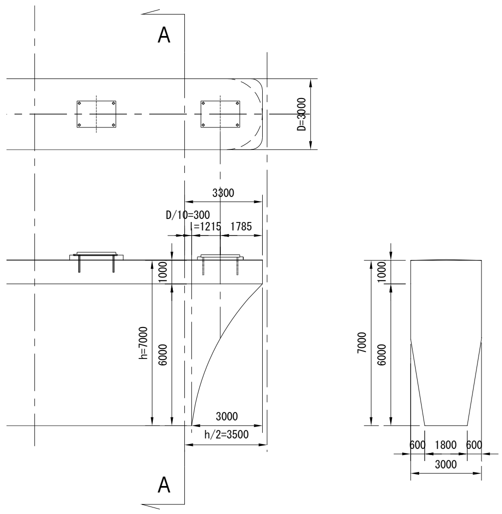
b) Shear Reinforcement

- Lateral tie to avoid the column from buckling due to shear force: D19 (P6) and D22 (P7), 150 mm pitch through the column
- Intermediate hoop to avoid the lateral tie from jutting outside: 15 nos.(P6) and 11 nos.(P7) for bridge axis direction and 2 nos. for bridge axis perpendicular direction per cross section, 150 mm pitch through the column

(2) Verification of Beam at Pier Head

1) Design Section

Since the distance from the front of the column to the loading point (bearing), l, is smaller than the height of beam, h, namely h/l=7000/1215=5.8≥1.0, this kind of beam will be designed as a corbel. And, design section (A-A) is set at 300 mm inside of column because of the oval column shape as shown in the figure below. It will be verified at A-A section in terms of bending moment and shear. The section at h/2 (=3500 mm) from A-A section is outside of the beam, so verification of shear force will be made only at A-A section.



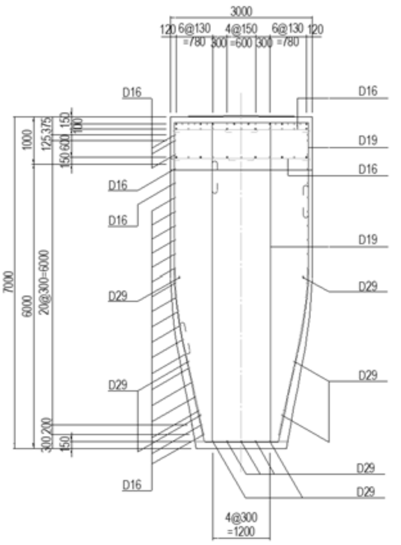
Source: JICA Study Team

Figure 4.3.25 Design Section of Pier Head Beam

2) Rebar Arrangement

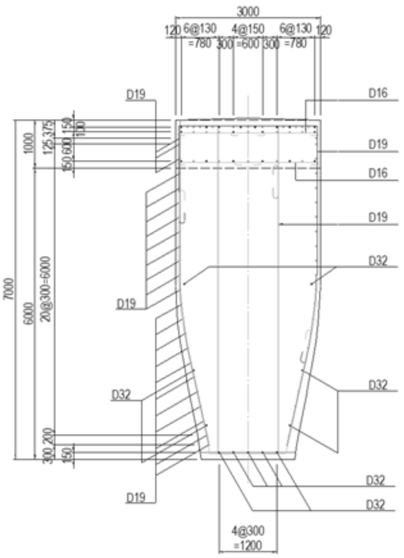
Main reinforcement and stirrup at the design section (A-A) is arranged as shown in the figure below.

P6



Location	Cover (mm)	Diameter	Arrangement
Upper	150	D29	18@153 in average
(main reinforcement)	250	D29	10@276 in average
Lower	150	D29	4@300
Side	97	D16	(125+20@300+200) x 2 sides
Stirrup	-	D19	150mm pitch

P7



Location	Cover (mm)	Diameter	Arrangement
Upper	150	D32	18@153 in average
(main reinforcement)	250	D32	10@276 in average
Lower	150	D32	4@300
Side	103	D19	(125+20@300+200) x 2 sides
Stirrup	-	D19	150mm pitch

Source: JICA Study Team

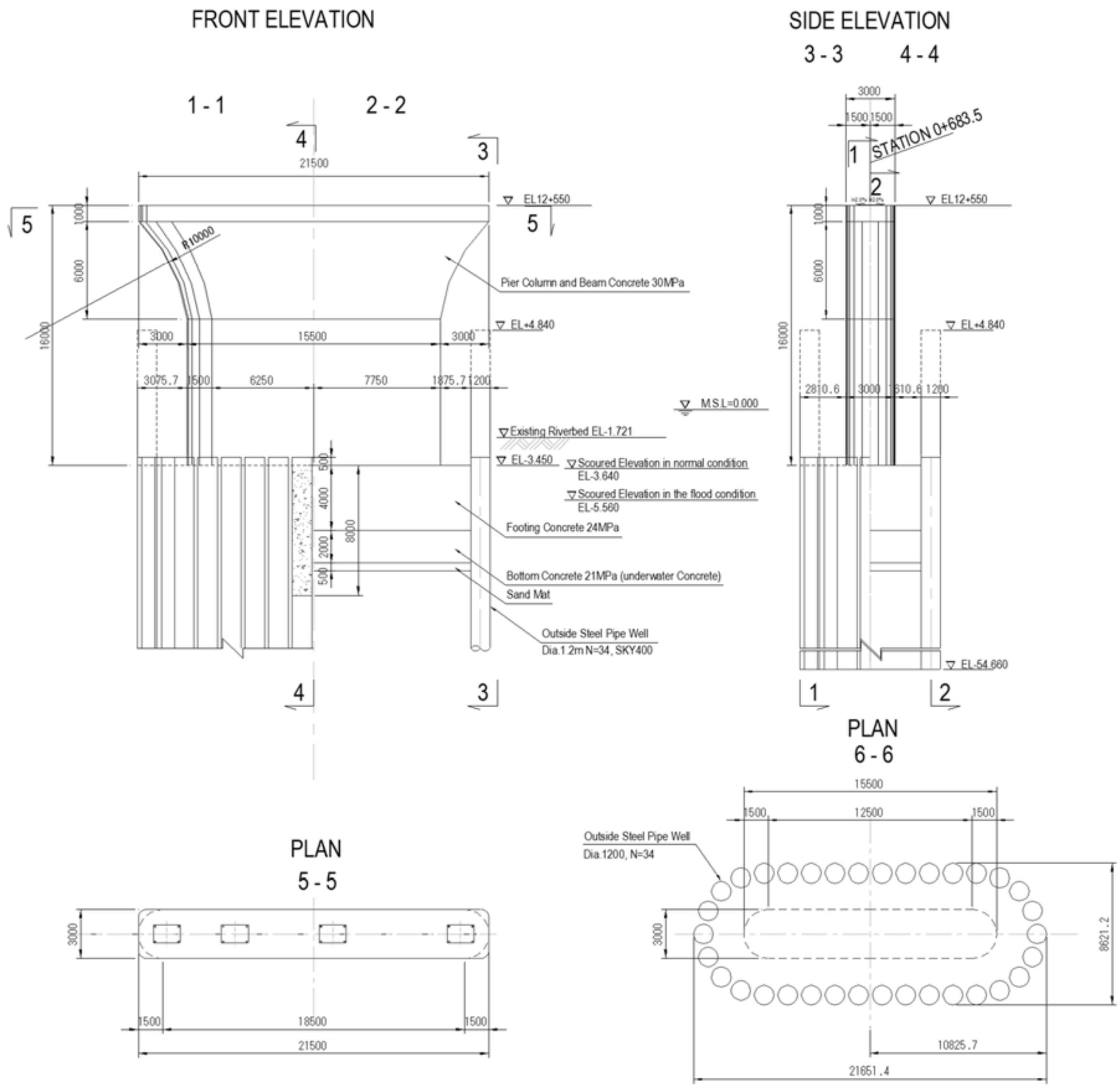
Figure 4.3.26 Rebar Arrangement (Main Reinforcement)

3) Verification of Reinforcement Content (Vertical Bridge Axis Perpendicular Direction)

As for the section against bending moment at vertical bridge axis perpendicular direction, it is verified by the reinforcement content of tension rebar arranged at upper beam and side rebar by corbel design, and the result is summarized in the table below.

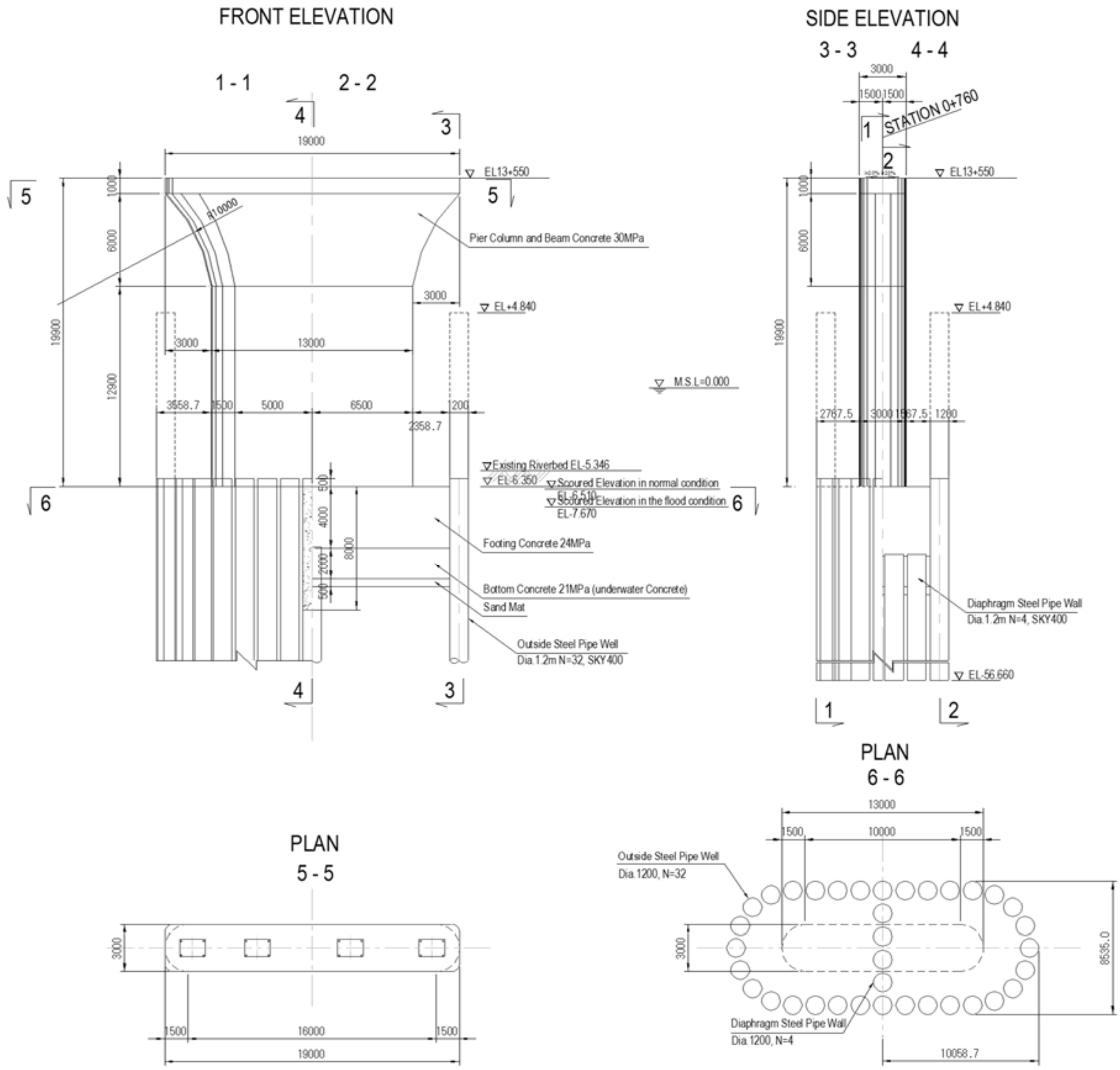
4.3.5.3 Structure Drawing

Through the basic design and the detailed design presented in Sections 4.3.2 and 4.3.6, the substructure of P6-P7 is designed in terms of economical and structural aspects. The structural drawing is shown in the figure below.



Source: JICA Study Team

Figure 4.3.27 Structure Drawing of Substructure of P6



Source: JICA Study Team

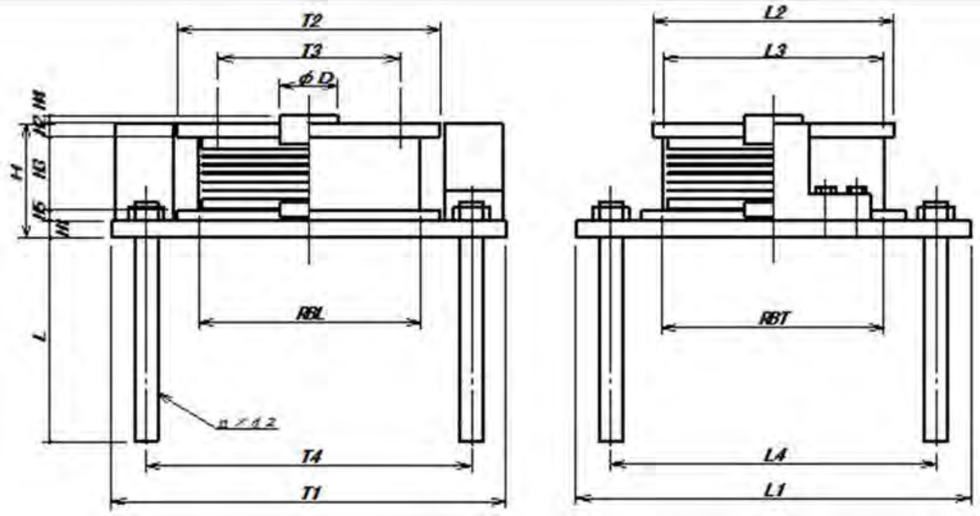
Figure 4.3.28 Structure Drawing of Substructure of P7

4.3.6 Detailed Design of Bridge Accessories

4.3.6.1 Bearings for 7-Span Bridge

(1) Dimension of Bearing

P14-P19 Fix Support (Rubber Bearing Type):



Bridge Axis Direction Bridge Axis Perpendicular Direction

Pier	Base Plate			Upper Plate					Rubber		
No.	L1	T1	H1	L2	T2	H2	D	H4	RBT	RBL	H3
P14	1420	1620	85	970	1100	50	350	35	920	920	184
P15	1370	1590	85	970	1100	40	300	30	920	920	188
P16	1370	1590	85	970	1100	40	300	30	920	920	188
P17	1370	1590	85	970	1100	40	300	30	920	920	188
P18	1370	1590	85	970	1100	40	300	30	920	920	188
P19	1420	1620	85	970	1100	50	350	35	920	920	184

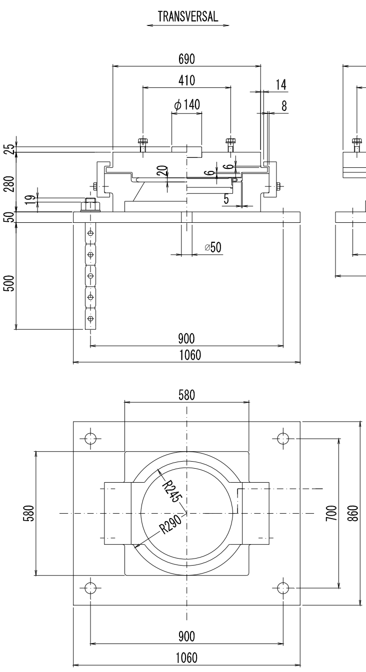
Pier	Hexagon Bolt				Anchor Bolt						
No.	d1	n	L3	T3	d2	L	n	L4	T4	H5	H
P14	M24	8	3×230	1×690	φ100	1000	4	1100	1300	40	359
P15	M24	8	3×230	1×690	φ90	900	4	1080	1300	40	353
P16	M24	8	3×230	1×690	φ90	900	4	1080	1300	40	353
P17	M24	8	3×230	1×690	φ90	900	4	1080	1300	40	353
P18	M24	8	3×230	1×690	φ90	900	4	1080	1300	40	353
P19	M24	8	3×230	1×690	φ100	1000	4	1100	1300	40	359

Source: JICA Study Team

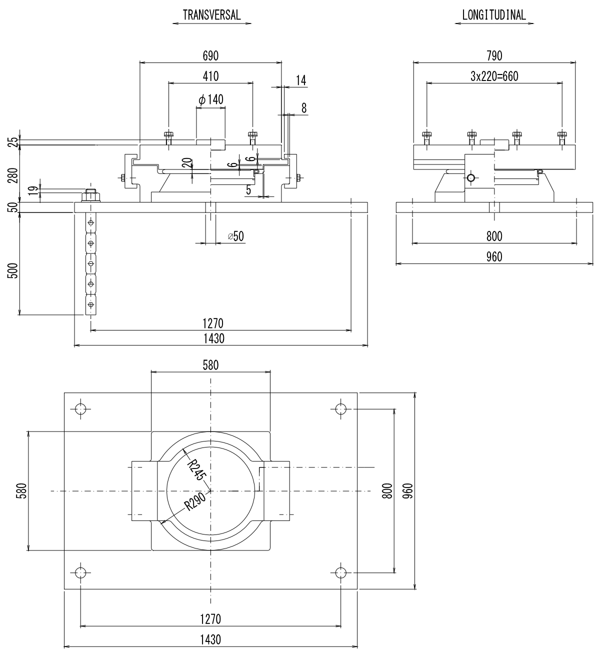
Figure 4.3.29 Dimension of Bearing (Fixed Rubber Bearing Type)

P13 and P20 Movable Support (BPB Type):

P13



P20

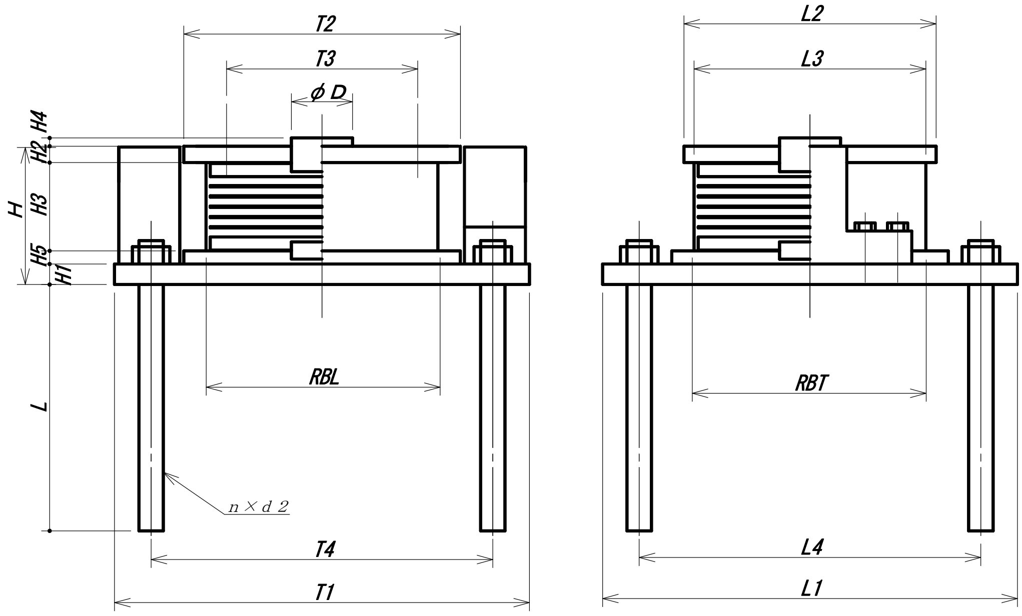


Source: JICA Study Team

Figure 4.3.30 Dimension of Bearing (Movable BPB Type)

4.3.6.2 Bearings for 3-Span Bridge

(1) Dimension of Bearing



Bridge Axis Direction   Bridge Axis Perpendicular Direction

Pier	Base Plate			Upper Plate					Rubber		
No.	L1	T1	H1	L2	T2	H2	D	H4	RBT	RBL	H3
P5	1150	1600	80	1020	1130	60	250	30	970	970	354
P6	1130	1690	110	970	1100	60	250	30	920	920	274
P7	1160	1730	130	970	1100	65	250	35	920	920	274
P10	1100	1800	85	970	1130	60	250	30	920	970	342

Pier	Hexagon Bolt				Anchor Bolt						
No.	d1	n	L3	T3	d2	L	n	L4	T4	H5	H
P5	M36	8	3×240	1×920	φ65	650	4	940	1390	40	534
P6	M36	8	3×220	1×990	φ65	650	4	920	1450	40	484
P7	M39	8	3×220	1×980	φ75	750	4	920	1450	40	509
P10	M36	8	3×220	1×920	φ65	650	4	890	1590	40	527

Pier	Horizontal Spring Stiffness(N/mm)	
No.	Per bearing	Per pier
P5	4,628	18,513
P6	6,395	25,579
P7	6,395	25,579
P10	4,622	18,486

Source: JICA Study Team

Figure 4.3.31 Dimension of Bearing

4.3.6.3 Expansion Joint for 7-Span Bridge

(1) Required Expansion/Contraction Value for the Displacement of Expansion Joint

The displacement due to seismic behavior and temperature expansion/contraction is shown in the table below.

Table 4.3.14 Table of Displacement at Different Factor (P13,P20)

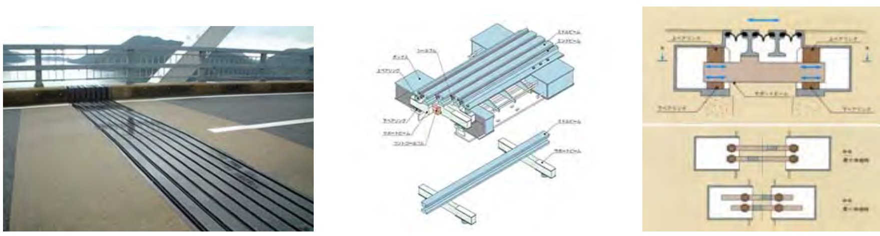
Item		Unit	P13		P20	
			Cable Stay Bridge	Steel Box	Steel Box	PC Box
Seismic Level (L1)	Displacement per one side	mm	±87	±34	±55	±212
	Maximum displacement (1)	mm	±87		±212	
	Coefficient due to different natural period (2)		√2		√2	
	Margin 15mm (3)	mm	±15		±15	
	Displacement (1)x(2)+(3)	mm	±138		±315	
	Design Value for Seismic Behavior (A)	**mm**	**±138**		**±315**	
Normal Condition Elongation/ Shrinkage (25℃± 25℃)	Creep	mm	-	-	-	-
	Shrinkage due to drying	mm	-	-	-	-
	Expanded length of the device	mm	+68	+102	+112	+68
	Contraction length of the device	mm	-68	-102	-112	-30
	Basic Expansion + Contraction (1)	mm	136	204	224	98
	Margin (2)=(1) x20%, min10mm	mm	27	41	45	20
	Expansion + Contraction (3)=(1)+(2)	mm	163 (±82)	245 (±123)	269 (±135)	118 (±59)
	Design Value for Normal Behavior (B)	**mm**	**±204**		**±194**	
Final Design Value for Expansion/Contraction Larger amount (A) or (B)		mm	**±204**		**±315**	
Marginal Gap		mm	400		350	

Source: JICA Study Team

(2) Selection of Expansion Type

Modular type joint was adopted since it is suitable for the large expansion and contraction of more than 200 mm. The typical modular joint is described below.

- Several edge beams with rubber sheets are placed on support beams that were on the sliding bar in anchor box. The capacity of displacement depends on the length of anchor box and its marginal space.



Source: Catalogue from manufacturer

Figure 4.3.32 Sample of Modular Expansion Joint

- The end of steel deck plate must be cut out, since this expansion joint type has about 500 mm height. So, additional deck plate was prepared from the end diaphragm to the girder end.
- After the installation of the expansion joint, the space between the expansion joint to diaphragm shall be filled by casting concrete.

4.3.6.4 Expansion Joint for 3-Span Bridge

(1) Required Expansion/Contraction Value for the Displacement of Expansion Joint

The displacement due to seismic behavior and temperature expansion/contraction is shown in the table below.

Table 4.3.15 Table of Displacement at Different Factor (P5,P10)

Item		Unit	P5 (Main line)		P5(Ramp)	
			PC Box	Steel Box	PC Composite Slab	Steel Box
Seismic Level (L1)	Displacement per one side	mm	±194	±207	±17	±207
	Maximum displacement (1)	mm	±207		±207	
	Coefficient due to different natural period (2)		1.0		1.0	
	Margin 15mm (3)	mm	±15		±15	
	Displacement (1)x(2)+(3)	mm	±222		±222	
	Design Value for Seismic Behavior (A)	**mm**	**±222**		**±222**	
Normal Condition Elongation/ Shrinkage (25°C± 25°C)	Creep	mm	-	-	-	-
	Shrinkage due to drying	mm	-	-	-	-
	Expanded length of the device	mm	+55	+41	+33	+41
	Contraction length of the device	mm	-25	-41	-14	-41
	Basic Expansion + Contraction (1)	mm	80	82	47	82
	Margin (2)=(1) x20%, min10mm	mm	16	16	10	16
	Expansion + Contraction (3)=(1)+(2)	mm	96 (±48)	98 (±49)	57 (±29)	98 (±49)
	Design Value for Normal Behavior (B)	**mm**	**±97**		**±78**	
Final Design Value for Expansion/Contraction Larger amount (A) or (B)		mm	**±222**		**±222**	
Marginal Gap		mm	350		250	

Item		Unit	P10	
			Steel Box	Cable Stay
Seismic Level (L1)	Displacement per one side	mm	±190	±56
	Maximum displacement (1)	mm	±190	
	Coefficient due to different natural period (2)		√2	
	Margin 15mm (3)	mm	±15	
	Displacement (1)x(2)+(3)	mm	±284	
	Design Value for Seismic Behavior (A)	**mm**	**±284**	
Normal Condition Elongation/ Shrinkage (25°C±25°C)	Creep	mm	-	-
	Shrinkage due to drying	mm	-	-
	Expanded length of the device	mm	+44	+62
	Contraction length of the device	mm	-44	-62
	Basic Expansion + Contraction (1)	mm	88	124
	Margin (2)=(1) x20%, min10mm	mm	18	25
	Expansion + Contraction (3)=(1)+(2)	mm	106 (±53)	149 (±75)
	Design Value for Normal Behavior (B)	**mm**	**±128**	
Final Design Value for Expansion/Contraction Larger amount (A) or (B)		mm	**±284**	
Marginal Gap		mm	400	

Source: JICA Study Team

(2) Selection of Expansion Type

- Modular type joint was adopted since it is suitable for the large expansion and contraction of more than 200 mm.
- Several edge beams with rubber sheets are placed on support beams that were on the sliding bar in anchor box. The capacity of displacement depends on the length of anchor box and its marginal space.
- The end of steel deck plate must be cut out, since this expansion joint type has about 500 mm height. So, additional deck plate was prepared from the end diaphragm to the girder end.
- After the installation of the expansion joint, the space between the expansion joint to diaphragm shall be filled by casting concrete.

4.4 STUDY ON PC BOX GIRDER BRIDGE

4.4.1 General

The B/D of the concrete box girder bridge was conducted based on the terms of agreement in the F/S, and the design team performed confirmation and studies of design policy, design conditions, structural types, bridge length and spanning, and other works that were necessary for this Project. The design team conducted the F/S report review work and found some outstanding issues that should be worked out prior to the subsequent detailed design stage.

Thereafter, D/D was conducted in order to ensure rationality of facilities planned at the B/D stage under some updated design conditions such as natural condition survey result (soil investigation, topographic survey, etc.) and the future ground elevation.

The summary of the evolution of the design is shown in Table 4.4.1.

Table 4.4.1 Summary of Design Output Evolution

Item	Feasibility Study	Basic Design	Detailed Design
Bridge Width	22.300 m	20.700 m ~ 27.297 m	**20.700 m**
A1 (Thilawa) Side			
Box Girders Width & Cantilever Slab Length	Box width 7.400 m Cantilever 1.800 m	Box width 6.500 m & 8.500 m Cantilever 1.650 m ~ 3.950 m	Box width **6.500 m** Cantilever **1.650 m**
Bridge Length	407.0m	507.0m	**250.0m**
Number of substructure	8 nos.	10 nos.	**6 nos.**
Foundation Type	SPSP: 4 nos. Cast-In-Situ: 4 nos.	SPSP: 3 nos. Cast-In-Situ: 7 nos.	SPSP: **0 nos**. Cast-In-Situ: **6 nos**.
A2 (Yangon) Side			
Box Girders Width & Cantilever Slab Length	Box width 7.400 m Cantilever 1.800 m	Box width 6.500 m Cantilever 1.650 m	Box width 6.500 m Cantilever 1.650 m
Bridge Length	300.0 m	300.0 m	300.0 m
Number of substructure	7 nos.	7 nos.	7 nos.
Foundation Type	SPSP: 4 nos. Cast-In-Situ: 3 nos.	SPSP: 3 nos. Cast-In-Situ: 4 nos.	SPSP: 3 nos. Cast-In-Situ: 4 nos.

Source: JICA Study Team

The basic design and detailed design of the PC box girder bridge are explained hereinafter.

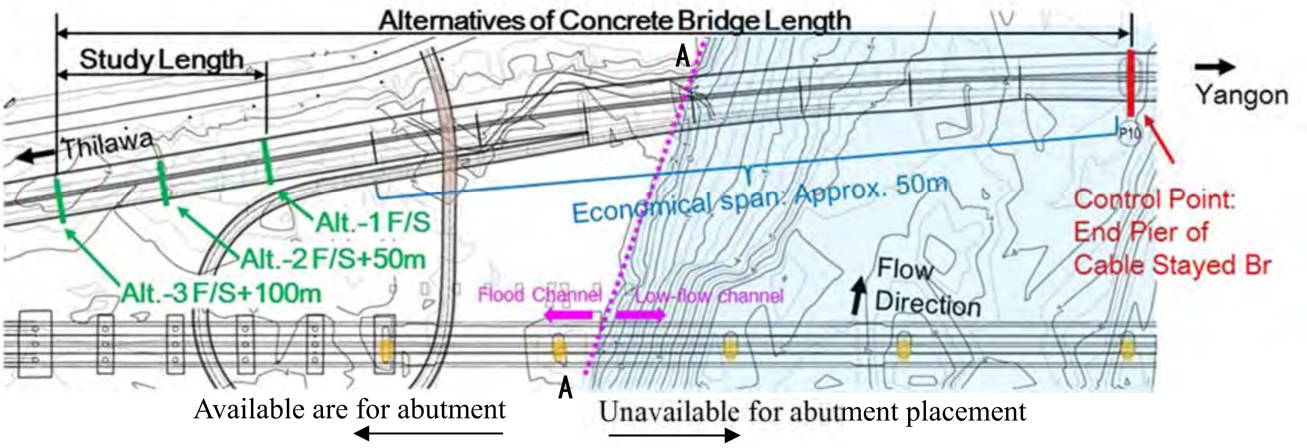
4.4.2 Study on Bridge Length of PC Box Girder Bridge

4.4.2.1 Determination of Bridge Length

(1) A1 (Thilawa) Side

- Available Area for Abutment Placement

On the left bank, a relatively dense grove exists and overall ground elevation is approximately MSL+4.0 m, which is nearly the same height as the normal H.W.L of Bago River. As a result of these natural circumstances, it is thought that water flow at the flood channel of the left bank is stagnated or quite small; consequently, discharge at the flood channel is nearly ignorable. Therefore, placement of abutment on the flood channel is possible without major impact on river discharge capacity. Hence, placement of abutment is possible up to the line A-A in Figure 4.4.1.



Source: JICA Study Team

Figure 4.4.1 Available Area for Abutment Placement and Bridge Length Alternatives

- Alternatives for Bridge Length Comparison

The beginning point of bridge lengths to be utilized for this comparison is "end pier of the cable-stayed bridge" as shown in Figure 4-83. Piers are arranged from this control point to the inland direction at 50 m interval referring to the economical span length of this bridge. Three alternatives for the bridge length comparison are summarized as follows:

Alternative 1: A1 Abutment at STA No. 0+457.0 m, L = 407 m (F/S)

Alternative 2: A1 Abutment at STA No. 0+407.0 m, L = 457 m (F/S + 50 m)

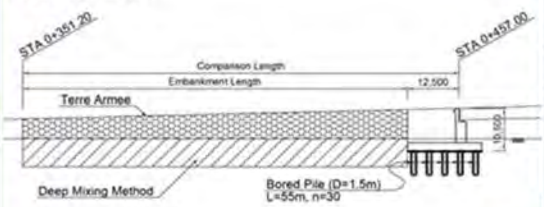
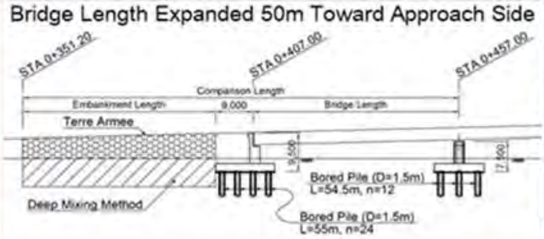
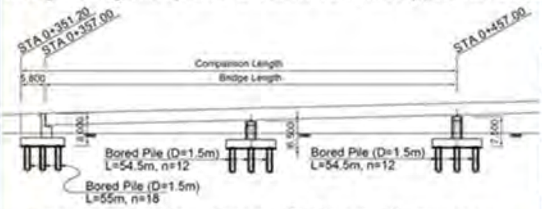
Alternative 3: A1 Abutment at STA No. 0+357.0 m, L = 507 m (F/S + 100 m)

- Comparison Result

As shown in Table 4.4.2, it is confirmed that Alternative 3: "A1 Abutment at STA No. 0+357.0 m, L = 507 m (F/S + 100 m)" is the most recommendable plan in terms of economy, workability, and construction period. Meanwhile, the abutment height for Alternative 3 is the minimum height considering a vertical space in front of the abutment, and any longer bridge length cannot be proposed.

Recommendation Alternative 3: A1 Abutment at STA No. 0+357.0 m, L = 507 m

Table 4.4.2 Comparison of Bridge Length at A1 Side

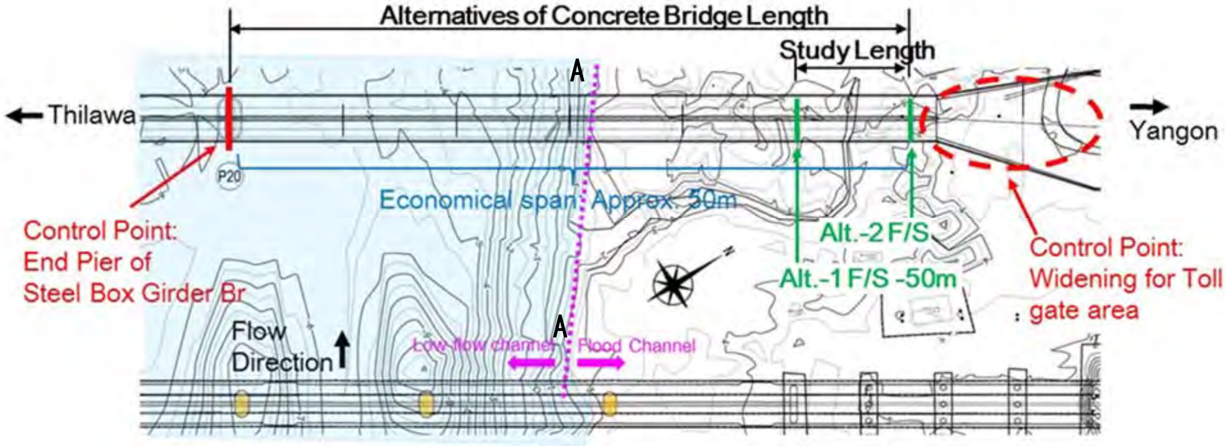
Alt-1	F/S Alternative 	Constructability & Quality Control	-The amount of road works including ground improvement by deep mixing method is largest.	△
		Cost Ratio	1.02	△
		Construction Period	8.7 months	△
		Environmental Aspect	-The amount of road works including ground improvement and embankment is largest.	△
		Evaluation	Less Recommended	
Alt-2	Bridge Length Expanded 50m Toward Approach Side 	Constructability & Quality Control	-The amount of road works including ground improvement by deep mixing method is smaller.	○
		Cost Ratio	1.01	○
		Construction Period	7.5 months	○
		Environmental Aspect	-The amount of road works including ground improvement and embankment is smaller.	○
		Evaluation	Less Recommended	
Alt-3	Bridge Length Expanded 100m Toward Approach Side  Legend : ◎ Very Good, ○ Good, △ Average	Constructability & Quality Control	-The amount of road works including ground improvement by deep mixing method is smallest.	◎
		Cost Ratio	1.00	◎
		Construction Period	6.0 months	◎
		Environmental Aspect	-The amount of road works including ground improvement and embankment is smallest.	◎
		Evaluation	Most Recommended	

Legend: ◎ Very Good, ○ Good, △ Average
Source: JICA Study Team

(2) A2 (Yangon) Side

- Available Area for Abutment Placement

There are JEE and MOC factory buildings and a relatively dense grove on the right bank, and overall ground elevation is approximately MSL+4.0 m or higher, which is nearly the same height as the normal H.W.L of Bago River. As a result of these natural circumstances, it is thought that water flow at the flood channel of left bank is stagnated or quite small; consequently, discharge at the flood channel is nearly ignorable. Therefore, placement of abutment on the flood channel is possible without major impact on river discharge capacity. Hence, placement of abutment is possible up to the line A-A in Figure 4.4.2.



Source: JICA Study Team

Figure 4.4.2 Available Area for Abutment Placement and Bridge Length Alternatives

- Alternatives for Bridge Length Comparison

  The beginning point of bridge lengths to be utilized for this comparison is "end pier of steel box girder bridge" as shown in Figure 4.4.2. Piers are arranged from this control point to the inland direction at 50 m interval referring to the economical span length of this bridge. Two alternatives for the bridge length comparison are summarized as follows:

  Alternative 1: A2 Abutment at STA No. 2+338.0 m, Length = 250 m (F/S-50 m)

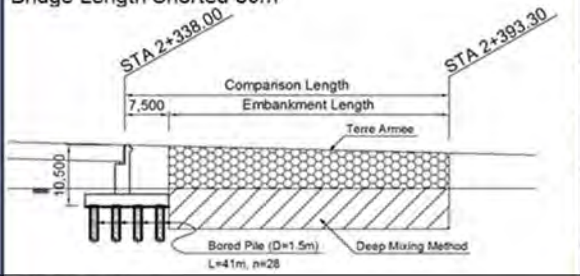
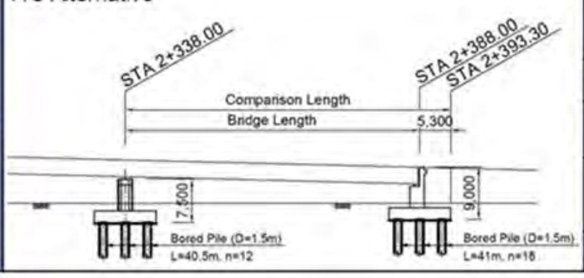
  Alternative 2: A2 Abutment at STA No. 2+388.0 m, Length = 300 m (F/S)

- Comparison Result

  It is confirmed that Alternative 2: "A2 Abutment at STA No. 2+388.0 m, Length = 300 m (F/S)" is the most recommendable plan in terms of economy, workability, and construction period as shown in Table 4.4.3. Meanwhile, no longer bridge length alternative is provided because the toll gate area starts just behind this abutment with a significant road widening.

Recommendation Alternative 2: A2 Abutment at STA No. 2+388.0 m, L = 300 m

Table 4.4.3 Comparison of Bridge Length at A2 Side

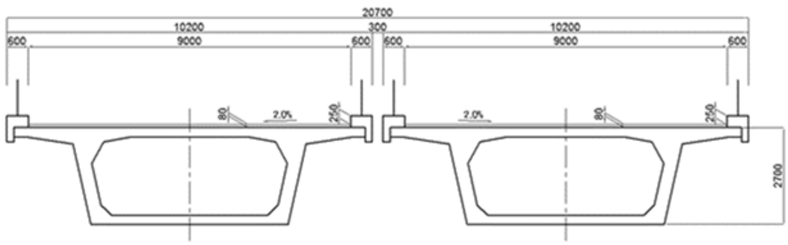
Alt-1	Bridge Length Shorted 50m 	Constructability & Quality Control	-The amount of road works including ground improvement by deep mixing method is largest.	△
		Cost Ratio	1.02	○
		Construction Period	8.3 months	△
		Environmental Aspect	-The amount of road works including ground improvement and embankment is smallest.	△
		Evaluation	Less Recommended	
Alt-2	F/S Alternative 	Constructability & Quality Control	-The amount of road works including ground improvement by deep mixing method is smallest.	◎
		Cost Ratio	1.00	◎
		Construction Period	6.8 months	◎
		Environmental Aspect	-The amount of road works including ground improvement and embankment is largest.	◎
		Evaluation	**Most Recommended**	

Legend: ◎ Very Good, ○ Good, △ Average
Source: JICA Study Team

4.4.3 Study on Span Length

4.4.3.1 Basic Conditions for the Study

Approach bridges (concrete bridge section) are planned as PC box girder bridges with SBS erection. Their roadway composition and cross section are as shown in Figure 4.4.3.



Source: JICA Study Team

Figure 4.4.3 Cross Section of PC Box Girder for the Study (Standard Width)

4.4.3.2 Comparative Study

The PC box girder bridges are planned with a girder height of H = 2.7 m, which is unified with the cable-stayed bridge section and steel box girder bridge section. Comparative study has been carried out on the PC box girder of A2 side section (bridge length = 300 m). Three alternatives have been considered as shown in Figure 4.4.4, for which constant span lengths (advantageous for SBS method) can be applied. Optimum span length has been selected among these three alternatives from the viewpoints of structural aspects, cost, and applicability of the span arrangement to A1 side. These three alternatives are within applicable span lengths, and have no special problems for construction.

Option-1: 60 m,	Option-2: 50 m,	Option-3: 43 m

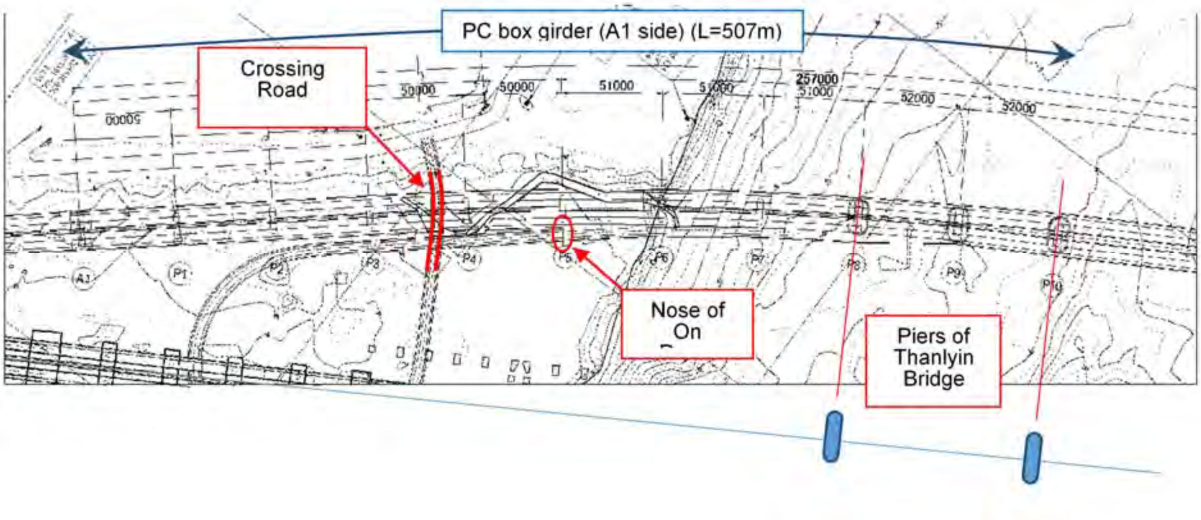
For A1 side, the following shall be taken into account for the determination of the pier locations:

(1) Crossing Road (Embankment section of on-ramp)

(2) Nose of On-ramp (End pier of on-ramp bridge)

(3) Pier Locations of Thanlyin Bridge

In the study of span length, hence, applicability to A1 side is confirmed.



Source: JICA Study Team

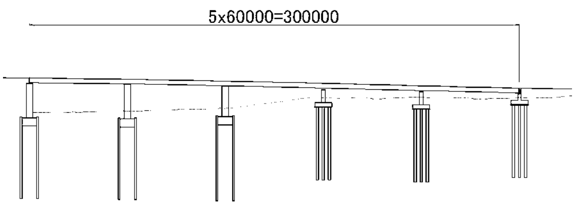
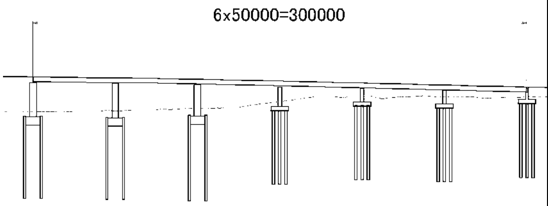
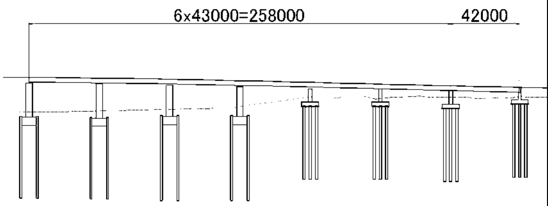
Figure 4.4.4 Restricting Conditions for Span Arrangement of A1 Side

4.4.3.3 Yangon Side (A2 Side)

Result of the comparative study on span length at Yangon side (A2 side) is tabulated in Table 4.4.4.

Span length of 50 m is recommended as the optimum solution, as the girder height is adequate for the span length and reasonable design is possible, and this is the most economical option.

Table 4.4.4 Comparison of Span Arrangement of PC Box Girder (A2 Side)

	Reference Drawing	Comments	Evaluation
60m		Girder height: 2.7 m (Adequate height: 3.2 m) Smaller girder height for span length, and required amount of prestressing tendons is greater. Cost: Ratio = 1.04	
50 m		Girder height: 2.7 m = adequate height Girder height is adequate for span length, and reasonable design is possible. Cost: Ratio = 1.00	**Most Recommended**
43 m		Girder height: 2.7 m (Adequate height: 2.3 m) Greater girder height for span length, and required amount of prestressing tendons is smaller. Cost: Ratio = 1.08	

Source: JICA Study Team

4.4.3.4 Thilawa Side (A1 Side)

Result of the comparative study on span length at Thilawa side (A1 side) is tabulated in Table 4.4.5.

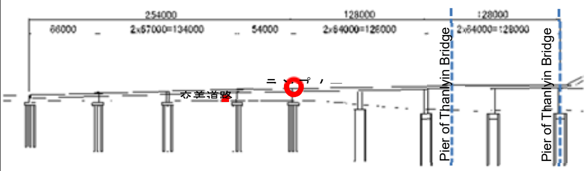
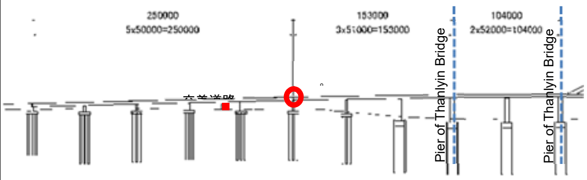
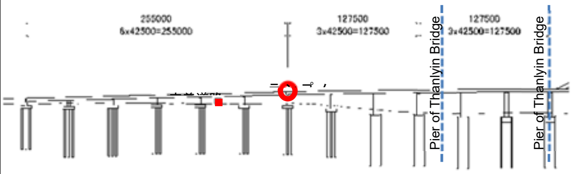
In addition to the advantages shown in the study at Yangon side (A2 side), span length of 50 m has the following advantages:

- Arrangement with same/similar span length is possible, even considering restrictions such as on-ramp nose and crossing road.
- Pier locations fit with Thanlyin bridge.

On the other hand, the other options have disadvantages such as uneven span lengths due to the restrictions, too long maximum span length (approx. 70 m for span length 60 m), or pier locations do not fit with Thanlyin bridge.

Span length 50 m is hence recommended also for Thilawa side (A1 side).

Table 4.4.5 Comparison of Span Arrangement of PC Box Girder (A1 Side)

	Reference Drawing	Comments	Evaluation
60 m		➢ Uneven span lengths (54~67 m) due to control of crossing road and on-ramp nose. Maximum span length exceeds 60 m. ➢ Position of in-river piers cannot accommodate with those of Thanlyin bridge.	
50 m		➢ Almost even span length (50~52 m) is possible, even considering the location of crossing road and on-ramp nose. ➢ Position of in-river piers can accommodate with those of Thanlyin Bridge.	**Most Recommended**
43 m		➢ Even span length (42.5) is possible, even considering the location of crossing road and on-ramp nose. ➢ Position of in-river piers cannot accommodate with those of Thanlyin Bridge.	

Source: JICA Study Team

4.4.3.5 Conclusion

As a result of the study above, 50 m has been selected as the basic span length for the PC box girder bridge section because of adequate girder height to span length, lowest cost, and applicability to A1 side.

4.4.3.6 Change of Length of PC Box Girder Bridge in the D/D Stage

For Thilawa side (A1 side), according to the request for restriction of pier location in river portion from MWIR to MoC, the span arrangement of the section between P5 and P10 has been changed. To respond to this request, the bridge type of P5 ~ P10 section has been changed to steel box girder bridge. The detailed design of PC box girder bridge has therefore been carried out for A1 ~ P5 section in Thilawa side, and for P20 ~ A2 section in Yangon side.

4.4.4 Study on Superstructure of PC Box Girder Bridge

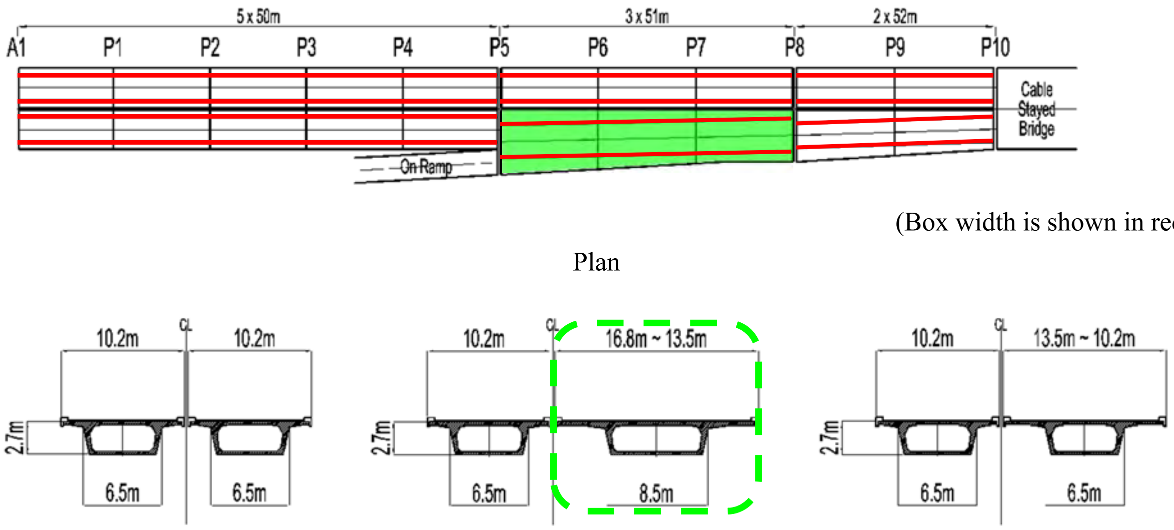
4.4.4.1 Superstructure of PC Box Girder Bridge

(1) Bridge Layout and Variation of Bridge Width

1) A1~P10

In the A1~P10 section, the bridge is divided at P5 and P8, and the bridge layout is 5 x 50 m + 3 x 51 m + 2 x 52 m.

For the box width, 6.5 m is adopted as the standard width, and 8.5 m is adopted for the especially wide section of P5~P8 (upstream).



(Box width is shown in red)

Plan

Cross Section (A1~P5) Cross Section (P5~P8) Cross Section (P8~P10)

Source: JICA Study Team

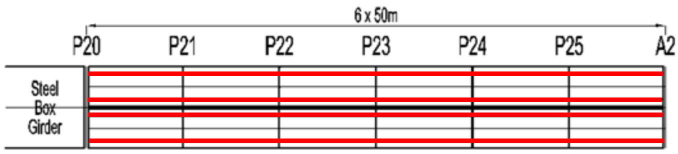
Figure 4.4.5 Bridge Layout and Box Width of the Girder (A1~P10)

- As the on-ramp is merged in the A1~P10 section (PC box girder section), bridge width is discontinuous at the ramp nose, and the width varies at the merging section.
- The superstructure shall be divided adjacent to the on-ramp nose due to the discontinuous bridge width.

- Box width and box shape shall be basically unified for ease of fabrication of precast segments and erection by SBS method. On the other hand, the bridge width of upstream side largely varies due to merging of on-ramp (10.2 m (standard) ~ 16.8 m (at P5) ~ 10.2 m (P10)), and this large variation cannot be accommodated just by the widening of the cantilever slabs while maintaining uniform box width. Two types of box width are hence adopted (6.5 m as standard width, and 8.5 m for especially wide section (P5~P8)).
- Taking the above into account, the superstructure is divided at P5 and P8. The bridge layout between A1~P10 is 5 x 50 m + 3 x 51 m + 2 x 52 m consequently.

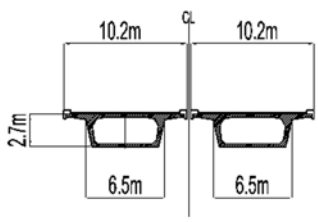
2) P20~A2

For the P20~A2 section, bridge layout is 6 x 50 m. The box width is 6.5 m (same as the standard section in A1~P10), as the bridge width is 10.2 m uniform.



(Box width is shown in red)

Plan



Cross Section (P20~A2)

Source: JICA Study Team

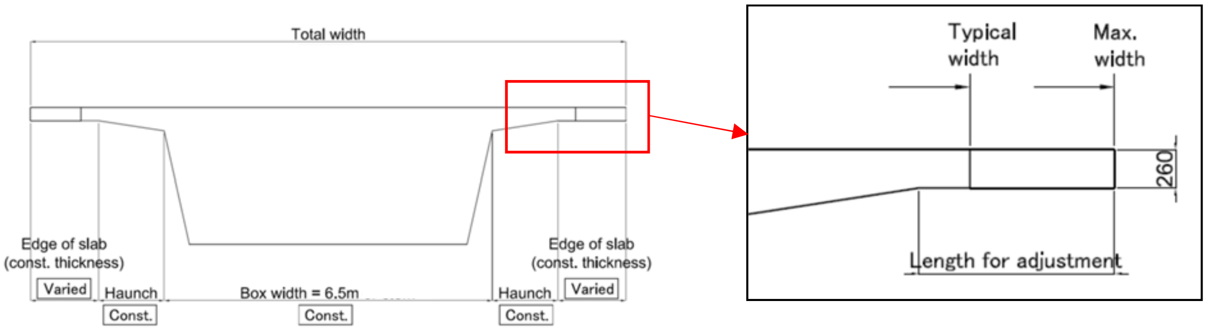
Figure 4.4.6 Bridge Layout and Box Width of the Girder (A1~P10)

3) Change of PC Box Girder Bridge Length in the Detailed Design Stage

In the detailed design stage, bridge type of the spans of P5~P10 has been changed to steel box girder bridge. The detailed design of PC box girder bridges has therefore been carried out for the spans A1~P5 at Thilawa side, and for the spans P20~A2 at Yangon side.

(2) Accommodation to Curvature of Bridge

The approach bridge has a slightly curved alignment (R = 2000 m) in A1 side (Thilawa side). On the other hand, the box element of the girder is planned to be straight between pier tables, considering ease of prefabrication and construction by SBS method. These curvatures are hence accommodated by varying the width of slab tip (const. thickness), while arranging the box element straight between pier tables and maintaining box width and width of tapered section of slab.



Source: JICA Study Team

Figure 4.4.7 Accommodation to Curvature and Widening of Bridge

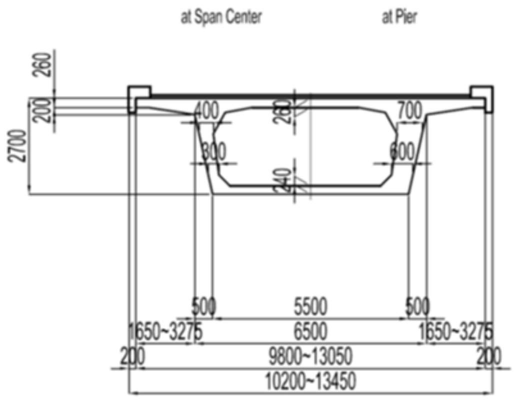
(3) Girder Height

A height of 2.7 m is adopted for the girder height of PC box girder bridges, unified with cable-stayed bridge and steel box girder bridge. The ratio of girder height to span length is 1/18.5~1/19.3 for span length of 50 m~52 m, which is within adequate range (desirable ratio for continuous PC box girder with SBS erection is 1/17~1/20).

(4) Member Thickness

Thicknesses of girder elements are determined based on structural function as longitudinal girder and transverse box frame, and function to place prestressing tendons.

The girder cross sections and thicknesses of members are shown below.



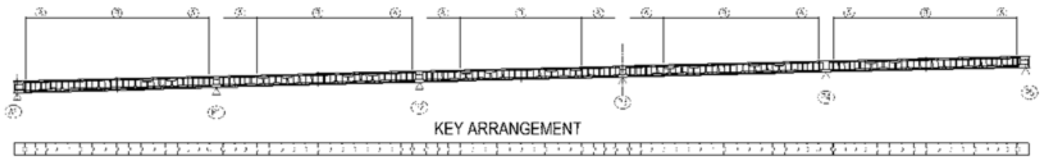
Source: JICA Study Team

Figure 4.4.8 Girder Cross Section (Standard Section and P8~P10 Widened Section)

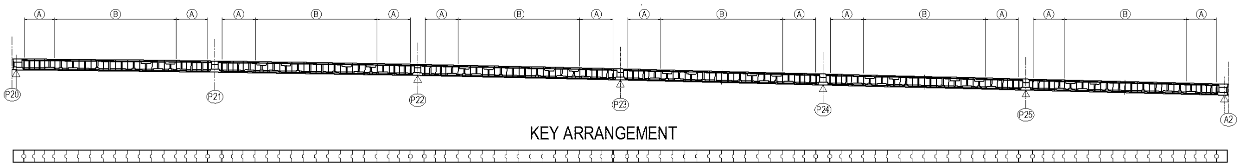
(5) Shear Key Arrangement

As the PC box girder bridges in this Project are constructed with precast segments by SBS method, shear keys were provided at each joint between segments in order to transfer the shear stresses adequately across joints and to avoid harmful displacement at joints. Concrete multiple shear keys were applied as the type of shear key which is reliable and commonly used in PC box girder bridges. The outline of design result is shown in Figure 4.4.9 and Figure 4.4.10

<A1-P5>

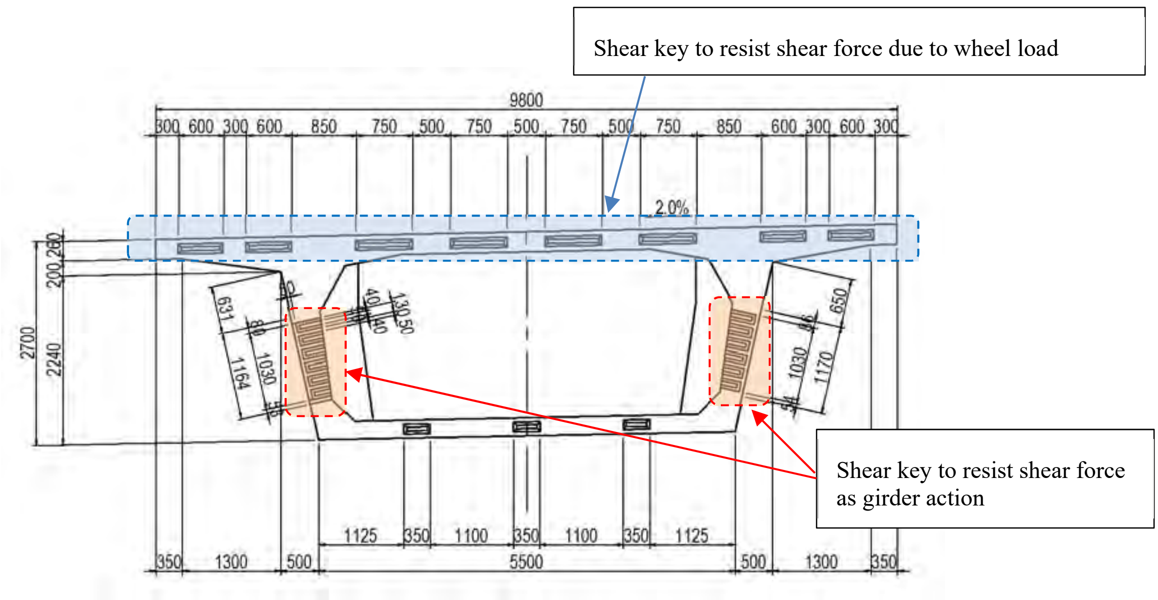


<P20-A2>



Source: JICA Study Team

Figure 4.4.9 Shear Key Arrangement (Side View)



Source: JICA Study Team

Figure 4.4.10 Shear Key Arrangement (Typical Section)

(6) Prestressing Tendons

1) Longitudinal Tendons

a) External Tendons

The 19S15.2 mm has been selected for external tendons, which is reasonable for PC box girders of similar span lengths and has many experiences of application. Considering the possibility of future cable replacement, ECF (Epoxy Coated and Filled Strand) + HDPE sheath has been selected as the type of external tendon, obtaining multiple anti-corrosion function while aiming to improve the workability of cable replacement.

Table 4.4.6 Comparison of External Tendon Type

	Bare Strand	ECF Strand	Semi-Prefabricated Cable
Schematic View			
Protection for Corrosion	• Grouting + HDPE sheath	• Epoxy coating on each strand + HDPE sheath	• Galvanizing or epoxy coating etc. on each strand (+ filler agent) + HDPE sheath/coating
Workability	• Strands are pushed one by one into HDPE sheath. After stressing, the sheath is grouted along all length. • Larger equipment is not required as the strands are installed one by one.	• Strands are pushed one by one into HDPE sheath. After stressing, anchor zone is grouted (sheath is not grouted). • Larger equipment is not required as the strands are installed one by one.	• Larger cranes etc. are required for installation as the strands have been prefabricated in the shape of one unit cable at factory. • Grouting is required only at anchorage.
Maintenance	• Difficulties in cable replacement as the cables are grouted.	• Easier cable replacement as the cables are not grouted except anchorage zone, and each strand can be handled one by one.	• Difficulties in handling at cable replacement as the cables in the shape of unit and installed in the girder.
Evaluation		**MOST RECOMMENDED**	

Source: JICA Study Team

b) Internal Tendons

The 12S15.2 mm has been applied as internal longitudinal tendon, which has many experiences of application to PC box girders of similar span lengths, and whose anchorage can be installed within the length of precast segment. At least two internal tendons have been installed at each section in order to ensure the deformability of the girder.

c) Transverse Tendons for Deck Slab

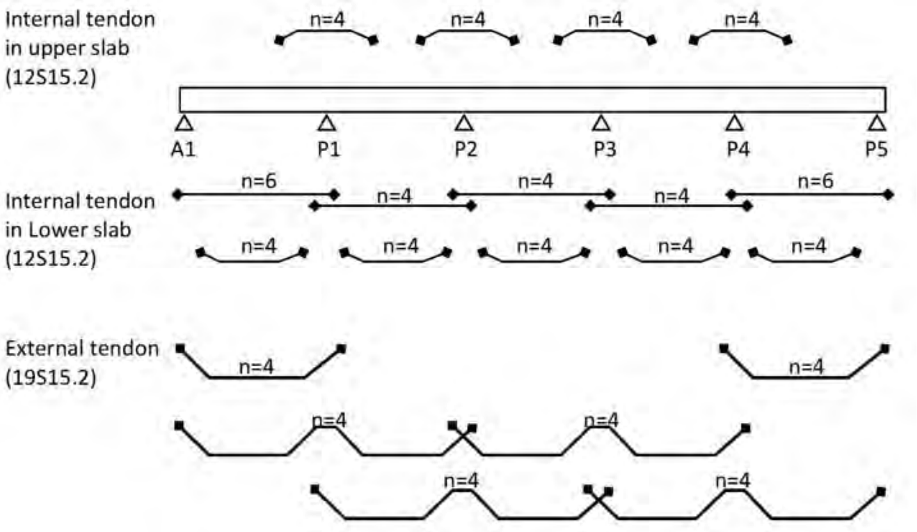
Both pre-tensioning and post-tensioning are applicable to transverse prestressing for deck slabs of precast segmental box girders. In this Project, post-tensioning method has been selected for deck slab prestressing, which is superior in geometry control of precast segments. The 3S12.7 mm has been selected as the type of tendons, as multi-strand is better in terms of procurement in Myanmar compared to large capacity single strands.

d) Tendons for Crossbeam Reinforcement

The crossbeams at pier table have functions to transfer reaction from superstructure to substructure through bearings. In addition, in this bridge, it is also a stress concentrated zone due to anchorage of external tendons. The crossbeams thus need to be reinforced by prestressing. For transverse prestressing, 4S15.2 mm has been used. For vertical prestressing, PC bars of 32 mm diameter have been applied, as the vertical tendon is short and PC bar with threaded anchorage system is advantageous than PC strands with wedge anchorages which have large loss of prestress for short tendons by pull-in of wedges.

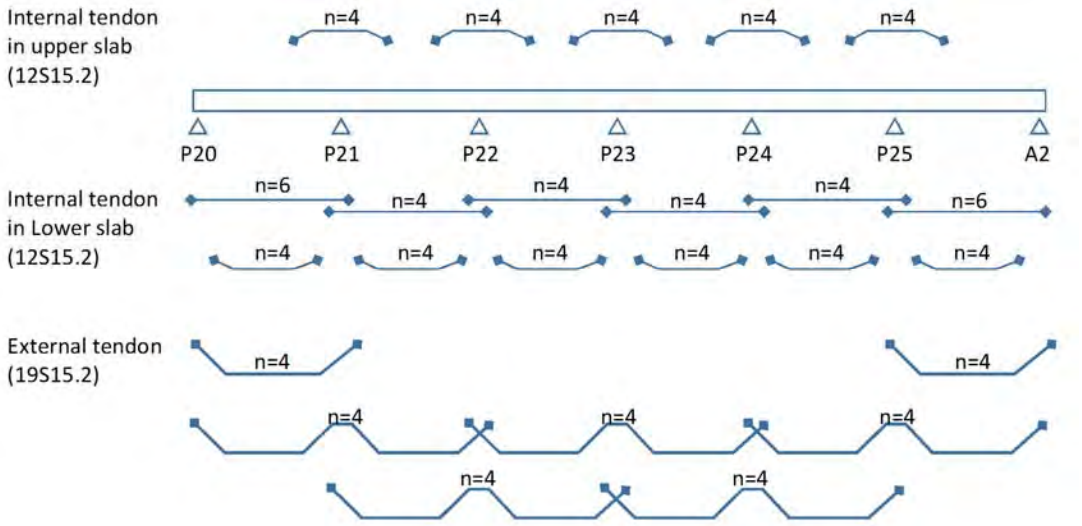
2) Longitudinal Tendon Arrangement (External and Internal Tendon)

a) A1-P5



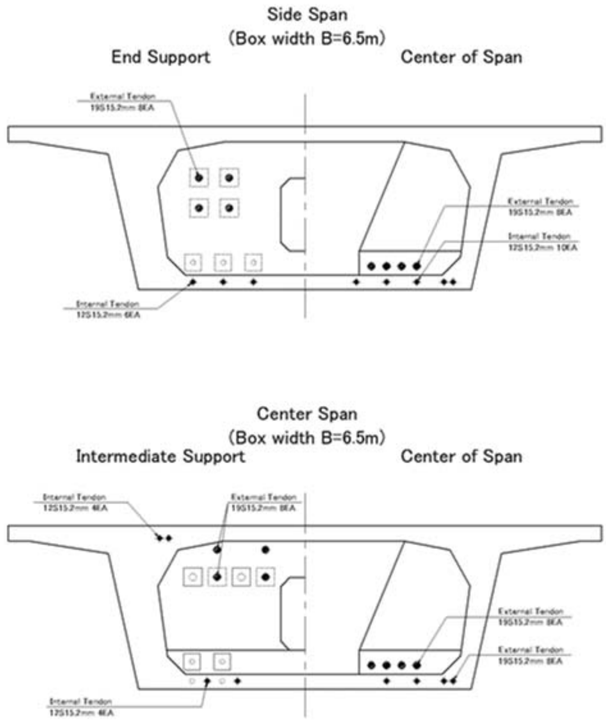
Source: JICA Study Team

b) P20-A2



Source: JICA Study Team

3) Standard Section (Box Width 6.5 m)



Source: JICA Study Team

Figure 4.4.11 Prestressing Tendon Arrangement (Standard Section, Box Width 6.5 m)

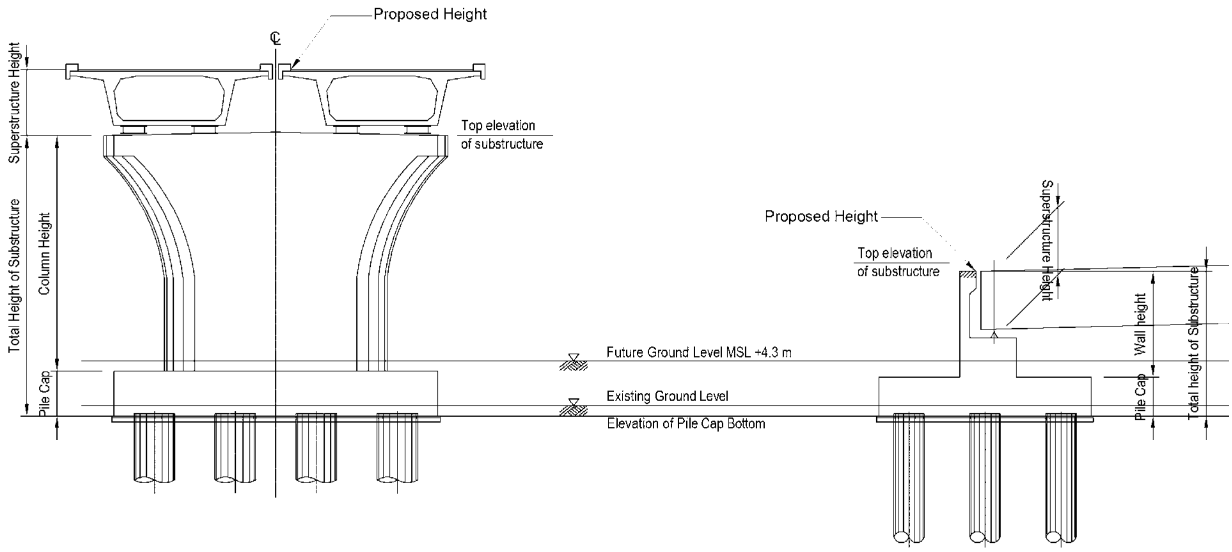
4.4.5 Substructure of PC Box Girder Bridge

4.4.5.1 Study of Substructure Height

(1) General

Substructure height was designed referring to the proposed heights of planned road (PH), ground level (GL), and required heights related to superstructure which include height from pavement structure through bridge bearing. As a result, substructure heights were determined as round numbers by 10 cm.

Reflecting an elevation of reclamation for construction yard preparation (MSL+4.300 m), foundation level of on-land substructures was determined based on MSL+4.300 m.



Source: JICA Study Team

Figure 4.4.12 Explanatory Diagram of Substructure Height

(2) Conclusion of Substructure Heights

Conclusions of the substructure heights are presented in Table 4.4.7 and Table 4.4.8.

Table 4.4.7 Summary of Substructure Heights at A1 (Thilawa) Side

Item	Mark	Unit	A1	P1	P2	P3	P4	P5
Station Number	STA	m	357.00	407.00	457.00	507.00	557.00	607.00
Proposed height	PH	m	8.692	9.942	11.192	12.442	13.691	14.830
Top elevation of substructure	KCL	m	8.692	6.424	7.709	8.959	10.173	11.309
Existing Ground EL	GL1	m	3.223	3.254	3.025	3.156	3.260	3.149
Future Ground EL	GL	m	4.300	4.300	4.300	4.300	4.300	4.300
Pile cap thickness	FH	m	1.900	1.900	1.900	1.900	1.900	1.900
Total Substructure height	H	m	6.800	4.600	5.900	7.100	8.300	9.600
EL of Pile cap bottom	FL	m	1.892	1.824	1.809	1.859	1.873	1.709
Foundation Type	-	-	CIP Pile	CIP Pile	CIP Pile	CIP Pile	CIP Pile	CIP Pile

Source: JICA Study Team

Table 4.4.8 Summary of Substructure Heights at A2 (Yangon) Side

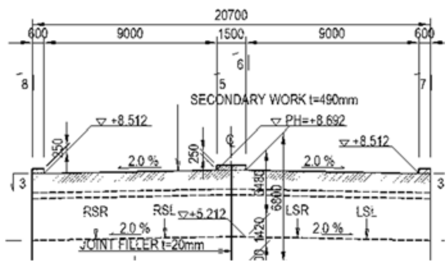
Item	Mark	Unit	P20	P21	P22	P23	P24	P25	A2
Station Number	STA	m	2088.00	2138.00	2188.00	2238.00	2288.00	2338.00	2388.00
Proposed height	PH	m	15.304	14.753	13.926	12.825	11.575	10.325	9.113
Top elevation of substructure	KCL	m	11.868	11.245	10.408	9.342	8.057	6.773	9.113
Existing Ground EL	GL1	m	-6.554	-6.155	-4.610	-0.041	4.116	4.016	4.110
Future Ground EL	GL	m	-7.490	-7.490	-7.490	0.550	4.300	4.300	4.300
Pile cap thickness	FH	m	4.000	4.000	4.000	2.200	1.900	1.900	1.900
Total Substructure height	H	m	23.400	22.800	21.900	14.000	6.200	4.900	7.300
EL of Pile cap bottom	FL	m	-11.532	-11.555	-11.492	-4.658	1.857	1.873	1.813
Foundation Type	-	-	SPSP	SPSP	SPSP	CIP Pile	CIP Pile	CIP Pile	CIP Pile

Source: JICA Study Team

4.4.5.2 Dimensions of Abutment

(1) Width

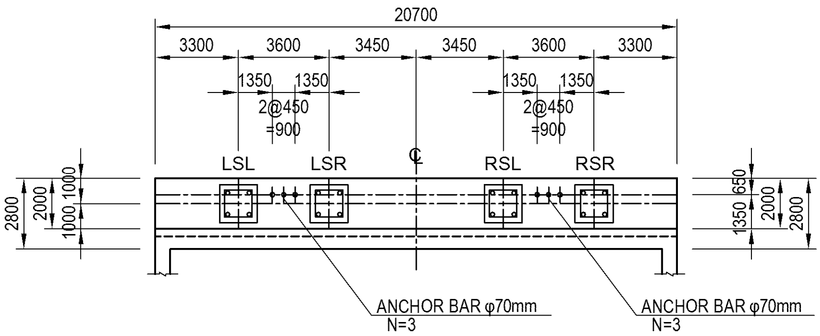
The width at the top surface of the parapet wall shall be the same as the effective cross section of road or wider. The abutments A1 and A2 are located at a straight section of the main bridge. Therefore, constitution of cross section and width can be the same as the typical cross section of the bridge.



Source: JICA Study Team

Figure 4.4.13 Abutment Width

(2) Bridge Seat



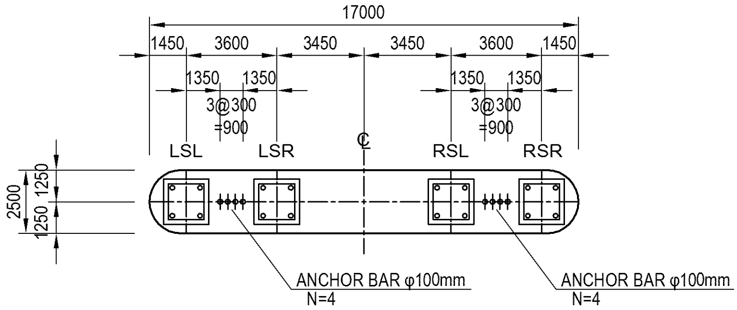
Source: JICA Study Team

Figure 4.4.14 Layout of Bridge Seat for Abutment (A1 and A2)

4.4.5.3 Dimensions of Pier

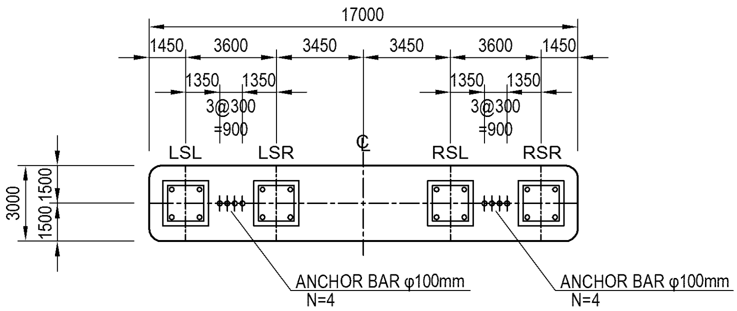
(1) Bridge Seat

Layouts of bridge seat are displayed in Figure 4.4.15 through Figure 4.4.18.



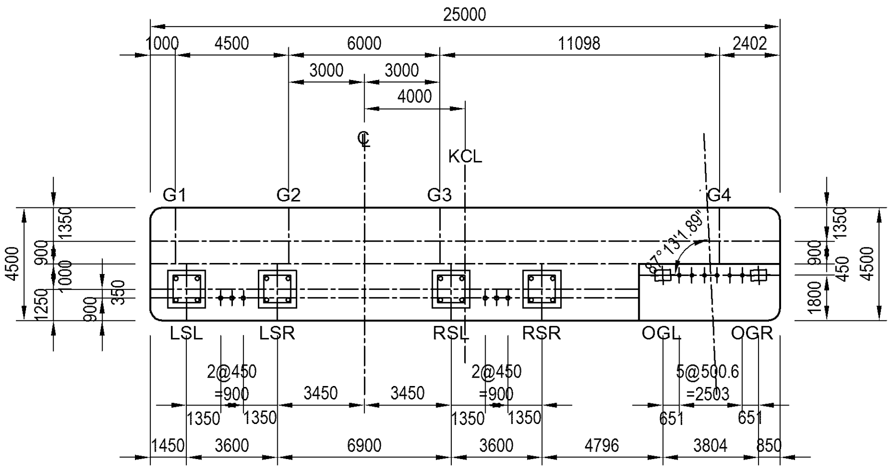
Source: JICA Study Team

Figure 4.4.15 Layout of Bridge Seat for P1~P3, P24 and P25



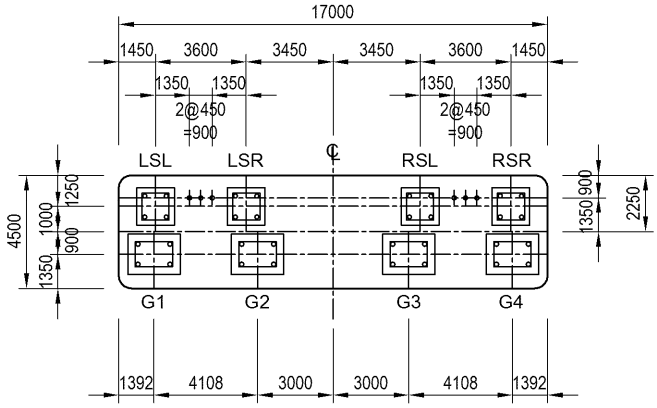
Source: JICA Study Team

Figure 4.4.16 Layout of Bridge Seat for P4, P21~P23



Source: JICA Study Team

Figure 4.4.17 Layout of Bridge Seat for P5



Source: JICA Study Team

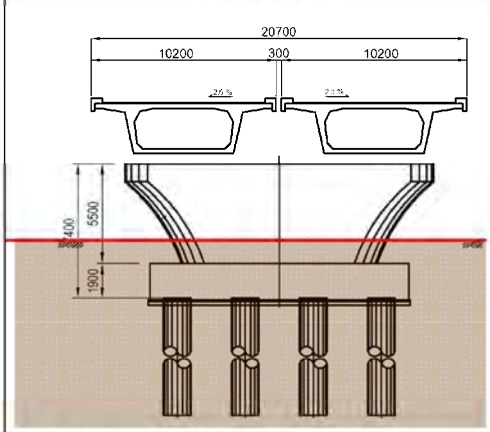
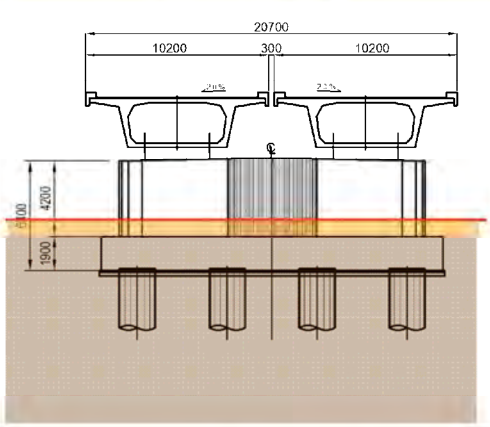
Figure 4.4.18 Layout of Bridge Seat for P20

(2) Dimensions of Pier Column

A wall type column was employed for piers of P1 through P3 at Thilawa side as well as P24 and P25 at Yangon side.

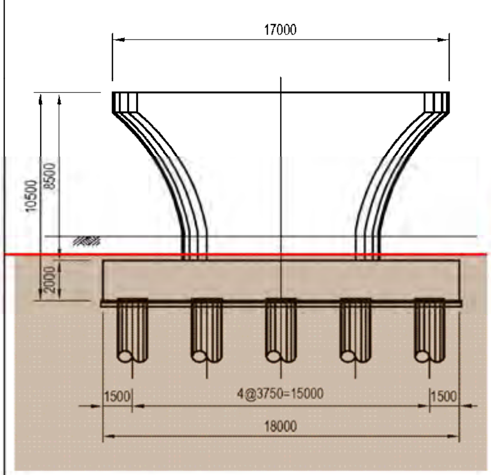
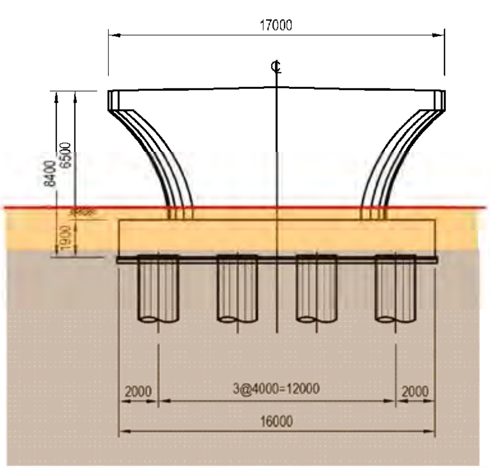
Regarding piers of P4, P5 and P20 through P23, which have reasonable heights for construction of beams on the column, the ginkgo shape pier was employed as selected during B/D. Comparisons are shown in Table 4.4.9 and Table 4.4.10.

Table 4.4.9 General Shapes of Wall Type Piers for P1~P3, P24 and P25 at D/D

Evaluation Item	Basic Design	Detailed Design
Schematic View		
Updates from Basic Design	• **Shortening of pier height:** Due to reclamation soil depth (up to MSL+4.3m) and minimization of design overburden soil depth pier heights are minimized. Piers heights are shortened by 1.6m in average comparing with BD. • **Deign Soil Parameter:** Soil parameters for a foundation design was updated. DD's soil parameters is weaker than BD so that a strengthening of foundation is made by means of change of pile diameter from 1.5m to 2.0m. • **Optimization of Pile Diameter of CIP Pile:** (BD: φ1.5m、3x4=12nos, DD: φ2.0m, 3x4=12nos) Based on the updates of the pier height and the soil parameter, <u>pile diameter of 2.0m is selected.</u> • **Column type:** (BD: T-shape column, DD: Wall type) An adoption of T-shape column is unreasonable in terms of revised column height and optimized pile arrangement, a wall type column is recommended in DD.	

Source: JICA Study Team

Table 4.4.10 General Shapes of Piers with Overhang Beam for P4, P5, P20~P23 at D/D

<table>
<tr><th>Evaluation Item</th><th>Basic Design</th><th>Detailed Design</th></tr>
<tr><td>Schematic View</td><td>
</td><td>
</td></tr>
<tr><td>Updates from Basic Design</td><td colspan="2">

- **Shortening of pier height:** Due to reclamation soil depth (up to MSL+4.3m) and minimization of design overburden soil depth pier heights are minimized. Piers heights are shortened by 1.8m comparing with BD.
- **Deign Soil Parameter:** Soil parameters for a foundation design was updated. DD's soil parameters is weaker than BD so that a strengthening of foundation is made by means of change of pile diameter from 1.5m to 2.0m.
- **Optimization of Pile Diameter of CIP Pile:** (BD: φ1.5m、5x3=15nos, DD: φ2.0m, 4x3=12nos) Based on the updates of the pier height and the soil parameter, pile diameter of 2.0m is selected.

</td></tr>
</table>

Source: JICA Study Team

The cross section of column was determined based on the stress status of column under various load conditions or minimum dimensions of bridge seat. The summary of the basis of determination is explained in Table 4.4.11.

Table 4.4.11 Summary of Basis of Determination of Cross Sectional Dimensions

<table>
<tr><th colspan="2">Pier Number</th><th>Bridge Axis Width</th><th>Transverse Direction Width</th><th>Overhang Length</th></tr>
<tr><td rowspan="3">Overhang beam type</td><td>P4, P21, P22, P23</td><td>3.0 m
Required width for a stress computation</td><td>11.0 m
Required width for a bridge seat arrangement (17.0 m) and an overhang length (3.0 m)</td><td>3.0 m
Landscape preference at F/S stage as well as a stress computation (steel bar arrangement: Diameter 32- 2 layer)</td></tr>
<tr><td>P5</td><td>4.5 m
Required width for a bridge seat arrangement</td><td>25.0 m
Required width for a bridge seat arrangement (17.0 m) and an overhang length (3.0 m)</td><td>3.0 m
Ditto</td></tr>
<tr><td>P20</td><td>4.5 m
Required width for a bridge seat arrangement</td><td>11.0 m
Required width for a bridge seat arrangement (17.0 m) and an overhang length (3.0 m)</td><td>3.0 m
Landscape preference at F/S stage</td></tr>
<tr><td>Wall Type</td><td>P1~P3 P24, P25</td><td>4.5 m
Required width for a bridge seat arrangement</td><td>17.0 m
Required width for a bridge seat arrangement (17.0 m)</td><td>Non
(no overhang beam)</td></tr>
</table>

Source: JICA Study Team

4.4.6 Foundation of PC Box Girder Bridge

4.4.6.1 Selection of Bearing Stratum and Embedment Length of Foundation

(1) Selection of Bearing Stratum

The basement layer in the bridge design for this bridge site is Clayey SAND-II, which is distributed uniformly at the top surface elevation of around MSL-40.0~-60.0 m. Its firmness, represented by N-value of 50, was examined through the standard penetration test (SPT). There are no appropriate soil layers other than the basement layer with sufficient firmness and thickness to support bridge reactions at the left (Thilawa) side flood channel of Bago River. On the other hand, some parts of the Clayey SAND-I layer distributed just above the Clayey SAND-II at the right (Yangon) side flood channel are regarded as the bridge bearing stratum. Soil profile is displayed in Figure 4.4.19 and Figure 4.4.20.

A1 Side (Thilawa): Clayey SAND-II layer, MSL-50.0~-60.0 m

A2 Side (Yangon): Clayey SAND-I and II layers, MSL-30.0~-50.0 m

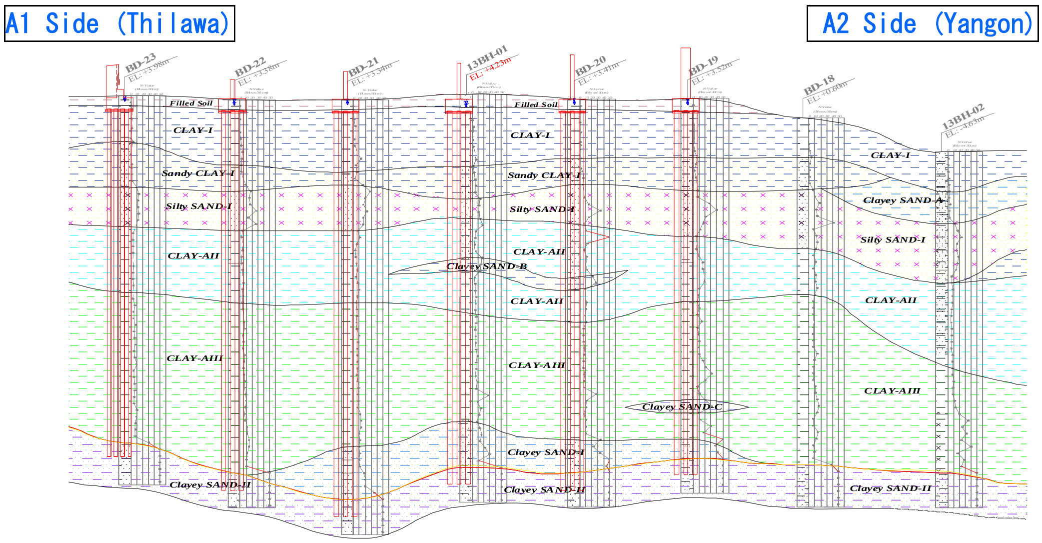
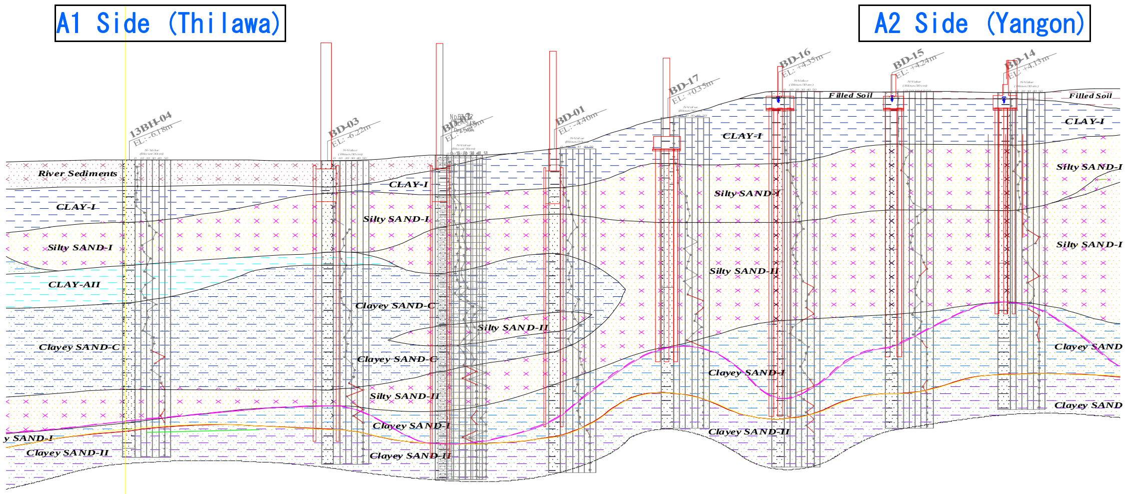


Figure 4.4.19 Prospected Soil Profile and Bearing Stratum (A1 Side)



Source: JICA Study Team

Figure 4.4.20 Prospected Soil Profile and Bearing Stratum (A2 Side)

(2) Embedment Length of Foundation

Embedment length of foundation is complied using a value recommended in the Specifications for Highway Bridges (Japan Road Associations) as follows:

Cast-In-Situ Pile Foundation: Around 1.0 D or more considering unevenness of bearing stratum

Steel Pipe Sheet Pile Foundation: Around 1.0 D or more for obtaining sufficient plunging effect

Note: The "D" represent pile diameter.

Foundation length and bearing stratum elevation determined for each substructure at D/D are summarized in Table 4.4.12 and Table 4.4.13.

Table 4.4.12 Summary of Foundation Length at A1 (Thilawa) Side

Item	Mark	Unit	A1	P1	P2	P3	P4	P5
Station Number	STA	m	357.00	407.00	457.00	507.00	557.00	607.00
EL of Pile cap bottom	FL	m	1.892	1.824	1.809	1.859	1.873	1.709
EL of Bearing layer	S	m	-49.020	-53.620	-57.660	-52.770	-53.590	-51.480
Pile diameter	D	m	1.500	2.000	2.000	2.000	2.000	2.000
Minimum socket length			1.0D	1.0D	1.0D	1.0D	1.0D	1.0D
Foundation Type	-	-	CIP Pile	CIP Pile	CIP Pile	CIP Pile	CIP Pile	CIP Pile
Pile Length	L	m	53.000	58.000	62.000	57.000	58.000	55.500
Reference Boring No.	-	-	*BD23*	*BD22*	*BD21*	*BH-01*	*BD20*	*BD19*
Bearing Stratum	-	-	*CS-II*	*CS-II*	*CS-II*	*CS-II*	*CS-II*	*CS-II*

Source: JICA Study Team

Table 4.4.13 Summary of Foundation Length at A2 (Yangon) Side

Item	Mark	Unit	P20	P21	P22	P23	P24	P25	A2
Station Number	STA	m	2088.00	2138.00	2188.00	2238.00	2288.00	2338.00	2388.00
EL of Pile cap bottom	FL	m	-11.532	-11.555	-11.492	-4.658	1.857	1.873	1.813
EL of Bearing layer	S	m	-47.220	-49.450	-42.400	-34.650	-42.650	-33.760	-27.870
Pile diameter	D	m	1.200	1.200	1.200	2.000	2.000	2.000	1.500
Minimum socket length			1.5D	1.5D	3.0D	1.0D	1.0D	1.0D	1.0D
Foundation Type	-	-	SPSP	SPSP	SPSP	CIP Pile	CIP Pile	CIP Pile	CIP Pile
Pile Length	L	m	41.500	44.000	39.000	32.500	47.000	38.000	31.500
Reference Boring No.	-	-	*BD3*	*BD2*	*BD1*	*BD17*	*BD16*	*BD15*	*BD14*
Bearing Stratum	-	-	*CS-II*	*CS-II*	*CS-I*	*CS-I*	*CS-I*	*CS-I*	*CS-I*

Source: JICA Study Team

4.4.6.2 Selection of Foundation Type

(1) Selection of Foundation Type at B/D

1) On-land (A1～P5, P24～A2)

It was confirmed that "CIP pile foundation (reverse circulation drilling method) D = 2.0 m" was the most economical foundation type for on-land piers. For abutment foundation type, it was confirmed that "CIP pile foundation (reverse circulation drilling method) D = 1.5 m" was the most preferable foundation type as it was selected at the B/D.

Piers: CIP pile foundation (reverse circulation drilling method) D = 2.0 m

Abutments: CIP pile foundation (reverse circulation drilling method) D = 1.5 m

The above review results are shown in Table 4.4.14 and Table 4.4.15.

2) In-river (Low-flow channel) (P20～P22)

The overall size of SPSP foundation is subject to dimensions of column and stability analysis of SPSP foundation. Regarding the dimensions of column for intermediate piers, 3.0 m in the bridge axis direction was sufficient under the updated conditions, whereas 3.5 m was proposed at the B/D. Consequently, it was confirmed that the overall size of SPSP could be minimized.

For the overall size of SPSP for P20 pier, dimensions of column could not be minimized because they were determined as the minimum dimension of a terminal support pier.

The above explanations are summarized in Table 4.4.16.

P20: SPSP foundation cum cofferdam 11.373 m x 17.164 m (Steel pipe diameter 1.2 m)

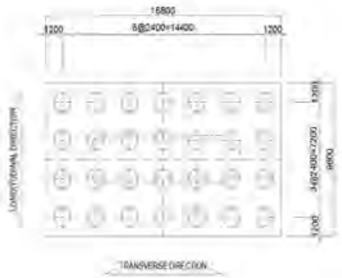
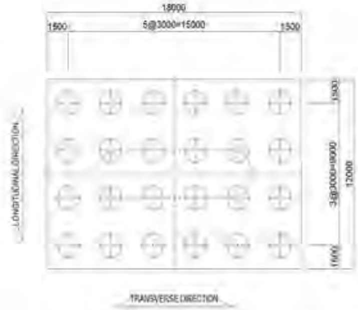
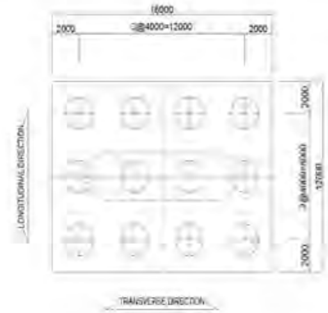
P21~22: SPSP foundation cum cofferdam 8.535 m x 17.222 m (Steel pipe diameter 1.2 m)

3) In-river (Riverfront) (P23)

There were no major changes in terms of construction conditions such as water level and riverbed elevations. Thus, the means of coffering, namely, steel sheet pile, was not changed from the B/D. Regarding the foundation type of P23 pier, CIP pile foundation (reverse circulation drilling method) D = 2.0 m was selected referring to the review result of on-land piers foundation type.

Piers: CIP pile foundation (reverse circulation drilling method) D = 2.0 m

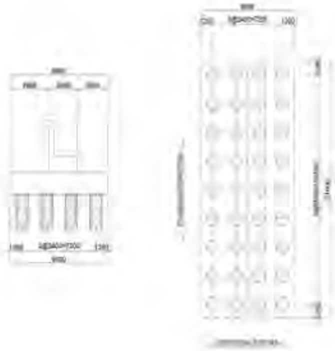
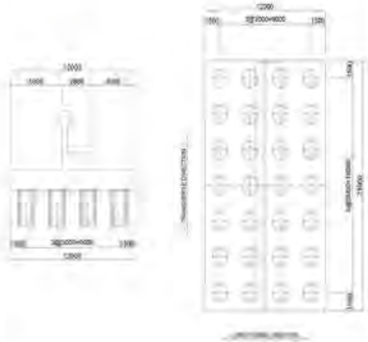
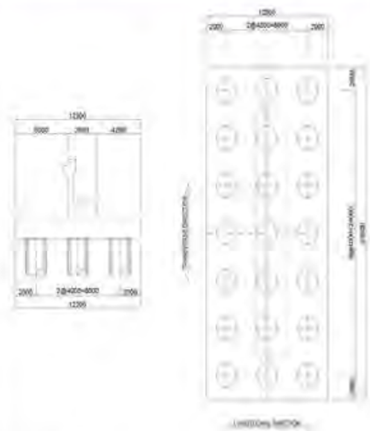
Table 4.4.14 Review of Foundation Type for On-land Piers at D/D

Pile Diameter			Cast in Place RC Piles φ1.2m				Cast in Place RC Piles φ1.5m				Cast in Place RC Piles φ2.0m			
Outline Drawing														
Design Results: item	mark	unit	Longitudinal Direction		Transverse Direction		Longitudinal Direction		Transverse Direction		Longitudinal Direction		Transverse Direction	
			Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation
Maximum Pile Reactions	Pmax	kN	1,608.0	3,089.1	1,608.0	2,978.8	2,003.8	3,540.6	2,003.8	3,582.1	3,895.5	6,853.1	3,895.5	7,390.5
	Ra	kN	3,530.0	5,516.0	3,530.0	5,516.0	4,632.0	7,294.0	4,632.0	7,294.0	6,523.0	10,403.0	6,523.0	10,403.0
	σ/σa	-	0.46	0.56	0.46	0.54	0.43	0.49	0.43	0.49	0.60	0.66	0.60	0.71
Amount of Displacement	σx	mm	0.0	14.4	0.0	14.8	0.0	12.8	0.0	13.4	0.0	17.3	0.0	17.9
	σxa	mm	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	20.0	20.0	20.0	20.0
	R	-	0.00	0.96	0.00	0.98	0.00	0.85	0.00	0.89	0.00	0.86	0.00	0.89
Stress of a Pile	σs	N/mm²	-17.0	211.5	-17.0	267.6	-14.0	184.5	-14.0	231.6	-15.5	180.5	-15.5	242.2
	σsa	N/mm²	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0
	σ/σa	-	0.09	0.71	0.09	0.89	0.07	0.61	0.07	0.77	0.08	0.60	0.08	0.81
Maximum Stress of a Pile			σs= 268 kN/m²<σsa= 300kN/m² (OK)				σs= 232 kN/m2<σsa= 300kN/m2 (OK)				σs= 242 kN/m2<σsa= 300kN/m2 (OK)			
Constructibility			The amount of number of pile is largest and thus this alternative is the most inferior one in terms of constructability.			△	This alternative entails the smallest amount of pile works.			○	This alternative entails smaller amount of pile works.			◎
Construction Period			The amount of pile works including ground excavation is consirably smaller.			◎	The amount of pile works including ground excavation is consirably the smallest.			◎	The amount of pile works including ground excavation is consirably large.			◎
Environmental Aspect			This alternative entails the smallest amount of excavation works.			◎	This alternative entails small amount of excavation works.			△	This alternative entails the largest amount of excavation works.			○
Cost Ratio			1.237				1.235				1.000			◎
Judge											◎			

Note ◎:Good, ○:Fair, △:Not Recommended

Source: JICA Study Team

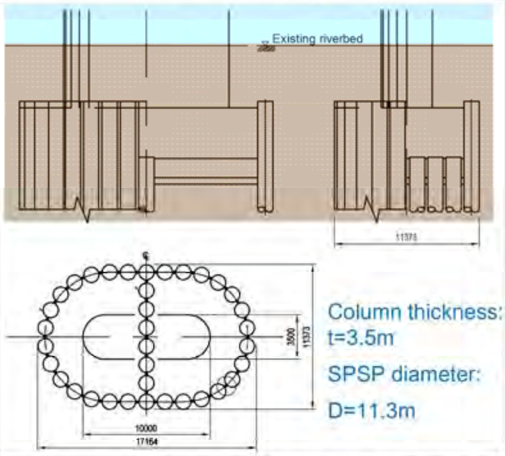
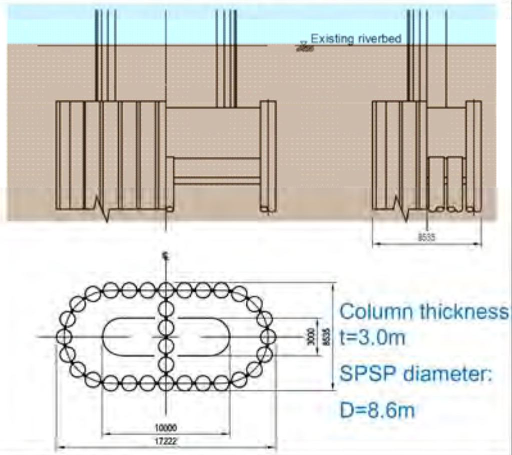
Table 4.4.15 Review of Foundation Type for Abutment at D/D

Pile Diameter				Cast in Place RC Piles φ1.2m		Cast in Place RC Piles φ1.5m		Cast in Place RC Piles φ2.0m	
Outline Drawing									
Design Results	item	mark	unit	Bridge's Longitudinal Direction		Bridge's Longitudinal Direction		Bridge's Longitudinal Direction	
				Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation
	Maximum Pile Reactions	Pmax	kN	1,208.0	1,844.6	1,605.1	2,400.7	2,404.1	3,761.0
		Ra	kN	2,797.0	4,400.0	3,730.0	5,916.0	5,353.0	8,603.0
		σ/σa	-	0.43	0.42	0.43	0.41	0.45	0.44
	Amount of Displacement	σx	mm	5.3	13.6	4.5	13.6	4.3	14.0
		σxa	mm	15.0	15.0	15.0	15.0	15.0	15.0
		R	-	0.35	0.90	0.30	0.91	0.29	0.94
	Stress of a Pile	σs	N/mm^2	38.6	272.7	29.6	262.6	22.1	255.2
		σsa	N/mm^2	160.0	300.0	160.0	300.0	160.0	300.0
		σ/σa	-	0.24	0.91	0.18	0.88	0.14	0.85
	Maximum Stress of a Pile			σs= 273 kN/m^2 <σsa = 300 kN/m^2 (OK)		σs= 263 kN/m^2 <σsa = 300 kN/m^2 (OK)		σs= 255 kN/m^2 <σsa = 300 kN/m^2 (OK)	
Constructability				The amount of number of pile is largest and thus this alternative is the most inferior one in terms of constructability.	△	This alternative entails the smaller amount of pile works.	○	This alternative entails the smallest amount of pile works.	◎
Construction Period				The amount of pile works including ground excavation is consirably smaller.	◎	The amount of pile works including ground excavation is consirably the smallest.	◎	The amount of pile works including ground excavation is consirably large.	◎
Environmental Aspect				This alternative entails the smallest amount of excavation works.	◎	This alternative entails small amount of excavation works.	○	This alternative entails the largest amount of excavation works.	△
Cost Ratio				**1.095**	○	**1.000**	◎	**1.171**	△
Overall Evaluation						◎			

Note ◎:Good, ○:Fair, △:Not Recommended

Source: JICA Study Team

Table 4.4.16 Comparison of Overall Size of SPSP Foundation at D/D

Evaluation Item	Basic Design	Detailed Design
Schematic View		
Updates from Basic Design	• **Optimization of column thickness in a longitudinal direction:** (BD: 3.5m, DD: 3.0m) Through the update of comprehensive bridge analysis, a horizontal force transmitted from superstructure is revised. Then it had applied to the structural design of columns and confirmed that 3.0m in thickness is feasible.. • **SPSP Diameter: (BD: 11.3m DD: 8.6m)** Control point of SPSP diameter is a pier column thickness in our case. In case of a column thickness 3.0m, minimization of SPSP diameter is accomplished. DD's soil parameters is weaker in general though, thanks to a higher rigidity of SPSP its outer diameters can be minimized from 11.3m to 8.6m. • **Benefits of smaller diameter of SPSP:** - Reduction in construction quantities of Steel pipe piles (36⇒32nos), - Reduction in construction quantities of a soil excavation inside the SPSP well (reduction by 20%) - Minimization in quantities of timbering supports within SPSP. (reduction by 25%)	

Source: JICA Study Team

4.4.7 Summary of Detailed Design Results for Substructure and Foundation

4.4.7.1 Computation of Columns of T-shaped Piers

The columns of T-shaped piers can be designed as cantilevers with fixed ends at the section connected to the footings. In the design process, the most adverse combination of axial forces and bending moments shall be applied.

The overhang beams of T-shaped piers can be designed as follows:

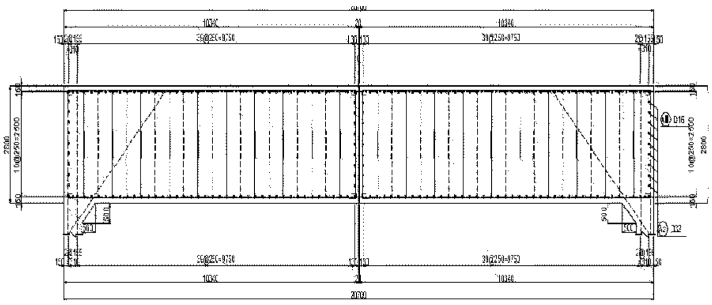
- ➢ The overhang beams are designed as cantilevers.
- ➢ The overhang length of the cantilever is defined as the length from the vertical section at the front surface of the column to the beam in case of rectangular column, and from the position one tenth of the column diameter inward from the front of the column to the beam end in case of an oval section column.

Computation results are shown in Table 4.4.17 through Table 4.4.22.

Table 4.4.17 Calculation Results for Wall and Comume (A1, P1~P3)

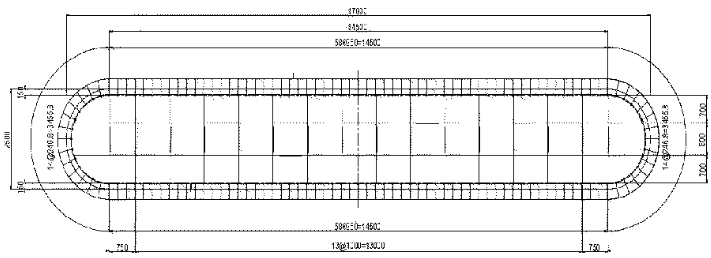
Cross Section of Column

A1



D32@250

P1~P3



D19@250 (P1,P2)
D22@250 (P3)

			A1		P1		P2		P3	
			LL	TT	LL	TT	LL	TT	LL	TT
Material		concrete	24N/mm2		24N/mm2		24N/mm2		24N/mm2	
		reinforcement	SD345		SD345		SD345		SD345	
Check for Bending Moment	Ordinary	σs (N/mm2)	-2.25	-	-9.04	-	-10.32	-	-10.46	-
		σsa(N/mm2)	-200.00	-	-200.00	-	-200.00	-	-200.00	-
		R-ratio	0.01	-	0.05	-	0.05	-	0.05	-
	Seismic	σs (N/mm2)	3.11	-	23.20	-3.94	174.33	-3.57	278.19	-2.86
		σsa(N/mm2)	300.00	-	300.00	-300.00	300.00	-300.00	300.00	-300.00
		R-ratio	0.01	-	0.08	0.01	0.58	0.01	0.93	0.01
Check for Shear	Ordinary	τm(N/mm2)	0.059	-	0.046	-	0.020	-	0.020	-
		τa(N/mm2)	0.145	-	0.129	-	0.129	-	0.137	-
		R-ratio	0.41	-	0.36	-	0.16	-	0.15	-
	Seismic	τm(N/mm2)	0.131	-	0.209	0.201	0.236	0.194	0.244	0.202
		τa(N/mm2)	0.195	-	0.170 (2.550)	0.105 (2.550)	0.170 (2.550)	0.105 (2.550)	0.181 (2.550)	0.111 (2.550)
		R-ratio	0.67	-	1.23 (0.08)	1.91 (0.08)	1.39 (0.09)	1.85 (0.08)	1.35 (0.10)	1.82 (0.08)

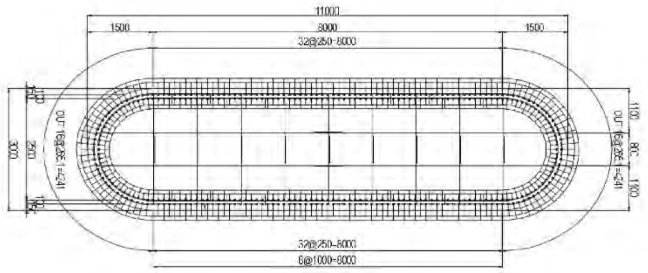
Note : σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.4.18 Calculation Results for Wall and Columns (P4~P5)

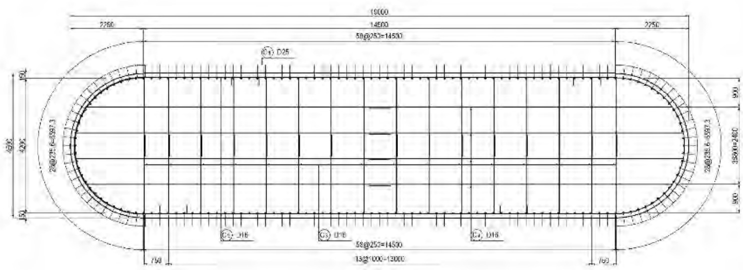
Cross Section of Column

P4



D25@250

P5

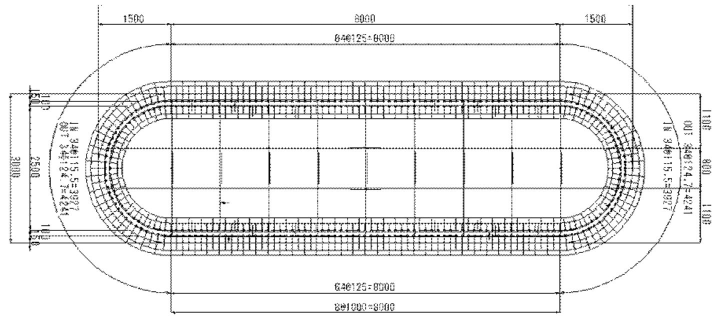
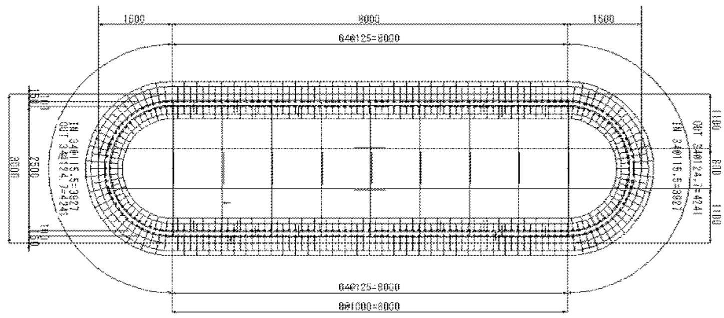


			P4		P5	
			LL	TT	LL	TT
Material		concrete	30N/mm2		30N/mm3	
		reinforcement	SD390		SD390	
Check for Bending Moment	Ordinary	σs (N/mm2)	-10.61	-	-4.63	-5.75
		σsa(N/mm2)	-230.00	-	-230.00	-230.00
		R-ratio	0.05	-	0.02	0.03
	Seismic	σs (N/mm2)	313.80	10.98	166.92	0.03
		σsa(N/mm2)	345.00	345.00	345.00	345.00
		R-ratio	0.91	0.03	0.48	0.00
Check for Shear	Ordinary	τm(N/mm2)	0.061	-	0.014	0.000
		τa(N/mm2)	0.140	-	0.097	0.076
		R-ratio	0.44	-	0.14	0.00
	Seismic	τm(N/mm2)	0.308	0.309	0.182	0.132
		τa(N/mm2)	0.180 (2.850)	0.122 (2.850)	0.144 (2.850)	0.112 (2.850)
		R-ratio	1.71 (0.11)	2.53 (0.11)	1.26 (0.06)	1.18 (0.05)

Note : σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

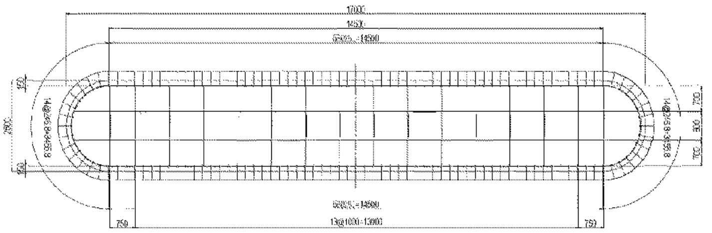
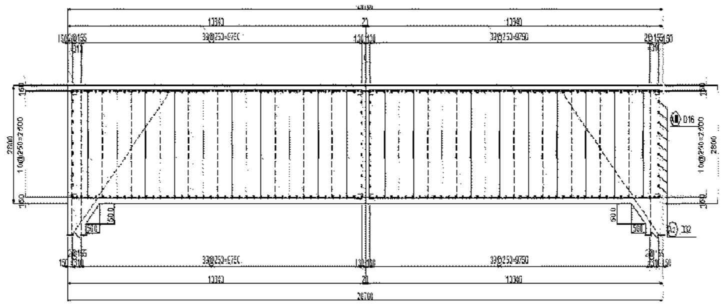
Table 4.4.19 Calculation Results for Wall and Columns (P20~P23)

			P20		P21		P22		P23	
Cross Section of Column			P20  2-D29@125							
			P21~P23  2-D38@125(P21,P22) 2-D29@125(P23)							
			LL	TT	LL	TT	LL	TT	LL	TT
Material		concrete	30N/mm2		30N/mm3		30N/mm3		30N/mm3	
		reinforcement	SD390		SD390		SD390		SD390	
Check for Bending Moment	Ordinary	σs (N/mm2)	1.82	-15.04	25.38	-17.45	-9.21	-17.41	-14.95	-
		σsa(N/mm2)	184.00	-230.00	184.00	-230.00	-230.00	-230.00	-230.00	-
		R-ratio	0.01	0.07	0.14	0.08	0.04	0.08	0.07	-
	Seismic	σs (N/mm2)	271.55	59.12	326.91	78.05	282.56	55.39	305.67	33.46
		σsa(N/mm2)	345.00	345.00	345.00	345.00	345.00	345.00	345.00	345.00
		R-ratio	0.79	0.17	0.95	0.23	0.82	0.16	0.89	0.10
Check for Shear	Ordinary	τm(N/mm2)	0.039	-	0.082	-	0.043	-	0.049	-
		τa(N/mm2)	0.187	-	0.290	-	0.290	-	0.210	-
		R-ratio	0.21	-	0.28	-	0.15	-	0.23	-
	Seismic	τm(N/mm2)	0.312	0.276	0.460	0.406	0.423	0.366	0.444	0.330
		τa(N/mm2)	0.241 (2.850)	0.186 (2.850)	0.373 (2.850)	0.248 (2.850)	0.373 (2.850)	0.248 (2.850)	0.310 (2.850)	0.206 (2.850)
		R-ratio	1.29 (0.11)	1.48 (0.10)	1.23 (0.16)	1.64 (0.14)	1.13 (0.15)	1.48 (0.13)	1.43 (0.16)	1.60 (0.12)

Note : σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.4.20 Calculation Results for Wall and Columns (P24, P25, and A2)

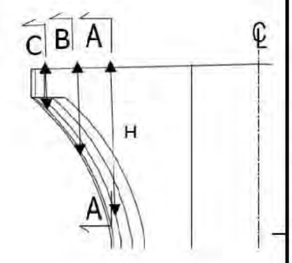
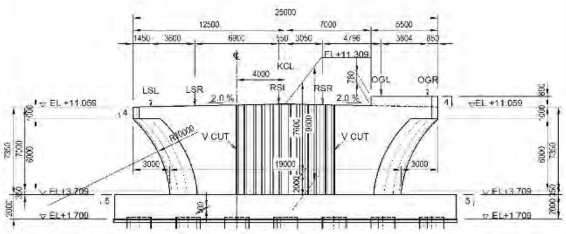
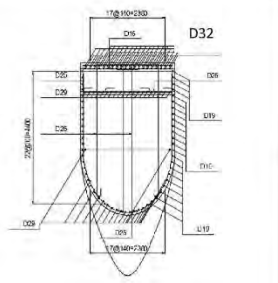
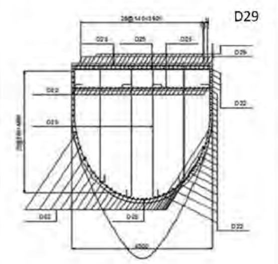
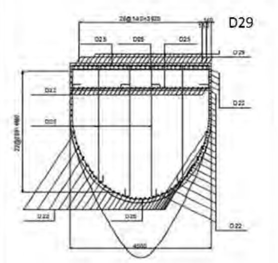
Cross Section of Column (Longitudinal Direction)	P24,P25  D19@250 A2  D32@250

			P24		P25		A2			
			LL	TT	LL	TT	LL	TT		
Material		concrete	24N/mm2		24N/mm2		24N/mm2			
		reinforcement	SD345		SD345		SD345			
Check for Bending Moment	Ordinary	σs (N/mm2)	-9.48	-	-8.59	-	-1.69	-		
		σsa(N/mm2)	-200.00	-	-200.00	-	-200.00	-		
		R-ratio	0.05	-	0.04	-	0.01	-		
	Seismic	σs (N/mm2)	210.82	-3.48	38.86	-3.58	11.14	-		
		σsa(N/mm2)	300.00	-300.00	300.00	-300.00	300.00	-		
		R-ratio	0.70	0.01	0.13	0.01	0.04	-		
Check for Shear	Ordinary	τm(N/mm2)	0.032	-	0.053	-	0.068	-		
		τa(N/mm2)	0.129	-	0.129	-	0.145	-		
		R-ratio	0.25	-	0.41	-	0.47	-		
	Seismic	τm(N/mm2)	0.236	0.195	0.212	0.208	0.150	-		
		τa(N/mm2)	0.170 (2.550)	0.105 (2.550)	0.170 (2.550)	0.105 (2.550)	0.195	-		
		R-ratio	1.39 (0.09)	1.86 (0.08)	1.25 (0.08)	1.98 (0.08)	0.77	-		

Note : σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.4.21 Calculation Results for Overhang Beams (P4 and P5)

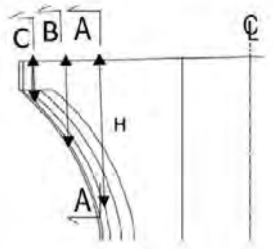
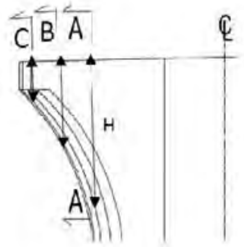
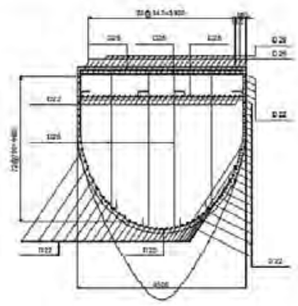
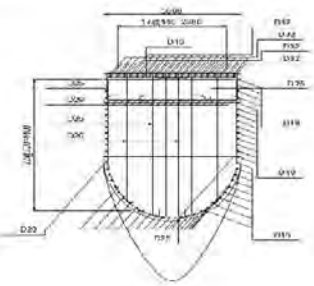
	P4 LEFT&RIGHT	P5 LEFT	P5 RIGHT
Side View of Beam & Cross Section of Beam			
	Cross Section of Beam	Cross Section of Beam	Cross Section of Beam
	 2-D32@140	 2-D29@140	 2-D29@140

		P4 LEFT&RIGHT			P5 LEFT			P5 RIGHT		
		Section	Section	Section	Section	Section	Section	Section	Section	Section
Material	concrete	30N/mm2			30N/mm2			30N/mm2		
	reinforcement	SD345			SD345			SD345		
section position	B	3.000	3.000	3.000	4.500	4.500	4.500	4.500	4.500	4.500
	H	5.000	1.818	2.444	5.000	2.267	2.933	5.000	2.018	4.000
Check for Bending Moment	σs (N/mm2)	79.360		-	72.940	-	-	51.110	-	-
	σsa(N/mm2)	100.000	-	-	100.000	-	-	100.000	-	-
	R-ratio	0.794	-	-	0.729	-	-	0.511	-	-
Check for Shear	τm(N/mm2)	-	0.020	1.250	-	0.020	0.620	-	0.210	0.470
	τa(N/mm2)	-	0.288	1.900	-	0.224	1.900	-	0.245	1.900
	R-ratio	-	0.069	0.658	-	0.089	0.326	-	0.857	0.247
Judgement		OK	OK	OK	OK	OK	OK	OK	OK	OK

Note :
σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.4.22 Calculation Results for Overhang Beams (P20~P23)

	P20 LEFT&RIGHT	P21 ~P23LEFT&RIGHT
Side View of Beam & Cross Section of Beam		
	Cross Section of Beam	Cross Section of Beam
	 2-D29@140	 2-D32@140

		P20 LEFT&RIGHT			P21 ~P23LEFT&RIGHT		
		Section A	Section B	Section C	Section A	Section B	Section C
Material	concrete	24N/mm2			24N/mm2		
	reinforcement	SD345			SD345		
section position	B	4.500	4.500	4.500	3.000	3.000	3.000
	H	5.000	2.016	2.744	5.000	1.800	2.750
Check for Bending Moment	σs (N/mm2)	68.110	-	-	80.660	-	-
	σsa(N/mm2)	100.000	-	-	100.000	-	-
	R-ratio	0.681	-	-	0.807	-	-
Check for Shear	τm(N/mm2)	-	0.020	0.680	-	0.020	1.110
	τa(N/mm2)	-	0.249	1.900	-	0.290	1.900
	R-ratio	-	0.080	0.358	-	0.069	0.584
Judgement		OK	OK	OK	OK	OK	OK

Note : σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

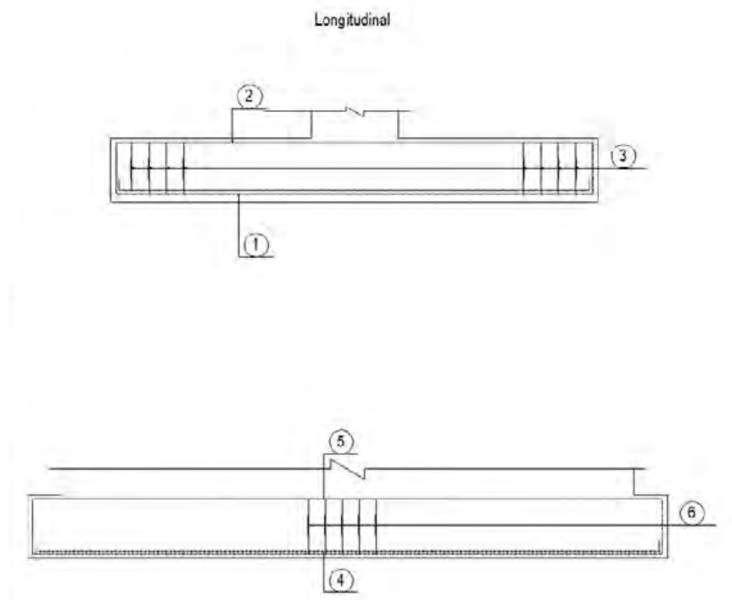
4.4.7.2 Computation of Footings

Footing shall be designed in consideration of the most adverse load combinations among self-weights, overburden load such as soils, presence of buoyancy, subgrade reaction, and reaction from foundations. Footings may be designed as beam members and as cantilevers.

The footings shall retain thickness necessary to serve as structural members. Also, the footings shall have sufficient thickness to be regarded as rigid bodies, when they are assumed as rigid bodies in the stability analysis of the foundation.

Computation results are shown in Table 4.4.23 through Table 4.4.26.

Table 4.4.23 Calculation Results for Footing of Piers (P1~P3)

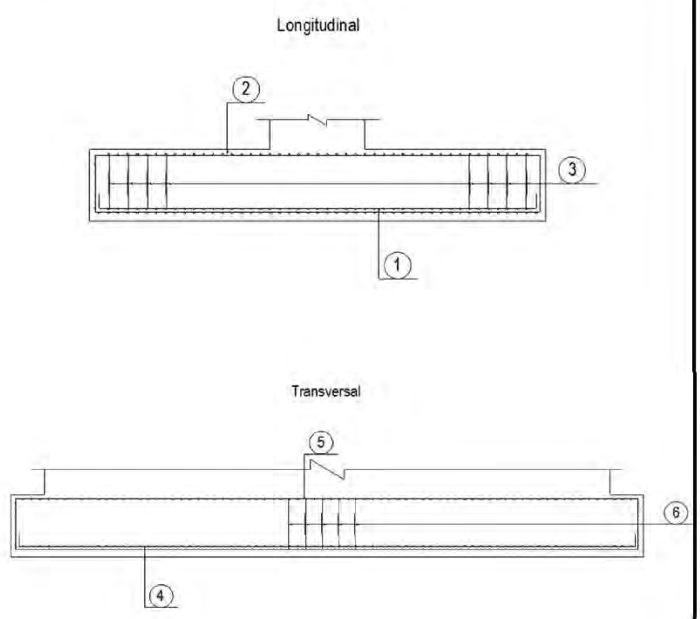
	PIER
Cross Section of Pile Cap (Longitudinal & Transversal Direction) SD345	

			P1		P2		P3	
			LL	TT	LL	TT	LL	TT
Arangement of reinforcement	①	④	2-D29@125	2-D25@250	2-D29@125	2-D25@250	2-D32@125	2-D29@250
	②	⑤	D25@125	D22@250	D25@125	D22@250	D29@125	D25@250
	③	⑥	D16@500	D16@500	D16@500	D16@500	D22@500	D22@500
Check for Bending Stress	Ordinary	σs (N/mm²)	120.29	47.39	117.83	47.39	142.02	42.43
		σsa(N/mm²)	160.00	160.00	160.00	160.00	160.00	160.00
		R-ratio	0.75	0.30	0.74	0.30	0.89	0.27
	Seismic	σs (N/mm²)	205.79	47.39	235.02	47.39	262.49	42.43
		σsa(N/mm²)	300.00	300.00	300.00	300.00	300.00	300.00
		R-ratio	0.69	0.16	0.78	0.16	0.87	0.14
Check for Shear Stress	Ordinary	τm(N/mm²)	0.450	—	0.443	—	0.482	—
		τa (N/mm²)	0.566	—	0.566	—	0.341 (1.700)	—
		R-ratio	0.80	—	0.78	—	1.41 (0.28)	—
	Seismic	τm(N/mm²)	0.717	—	0.809	—	0.820	—
		τa (N/mm²)	0.861	—	0.861	—	0.519 (2.550)	—
		R-ratio	0.83	—	0.94	—	1.58 (0.32)	—

σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.4.24 Calculation Results for Footing of Piers (P4 and P5)

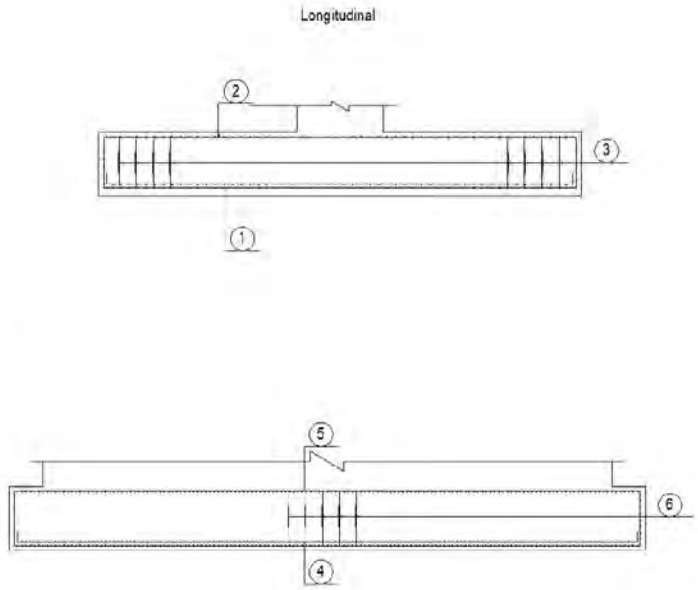
	PIER
Cross Section of Pile Cap (Longitudinal & Transversal Direction) SD345	

			P4		P5			
			LL	TT	LL	TT	LL	TT
Arangement of reinforcement	①	④	2-D32@125	2-D32@250	1.5-D29@125	2-D32@125		
	②	⑤	D29@125	D29@250	D29@250	D29@125		
	③	⑥	D16@500	D16@500	D16@500	D16@500		
Check for Bending Stress	Ordinary	σs (N/mm^2)	142.48	113.89	120.81	148.30		
		σsa(N/mm^2)	160.00	160.00	160.00	160.00		
		R-ratio	0.89	0.71	0.76	0.93		
	Seismic	σs (N/mm^2)	268.55	259.49	268.95	235.88		
		σsa(N/mm^2)	300.00	300.00	300.000	300.000		
		R-ratio	0.90	0.86	0.90	0.79		
Check for Shear Stress	Ordinary	τm(N/mm^2)	0.585	—	0.429	—		
		τa (N/mm^2)	0.686	—	0.930	—		
		R-ratio	0.85	—	0.46	—		
	Seismic	τm (N/mm^2)	1.038	—	0.870	—		
		τa (N/mm^2)	1.044 (2.550)	—	1.415(2.550)	—		
		R-ratio	0.99 (0.41)	—	0.61(0.34)	—		

σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.4.25 Calculation Results for Footing of Piers (P23, P24 and P25)

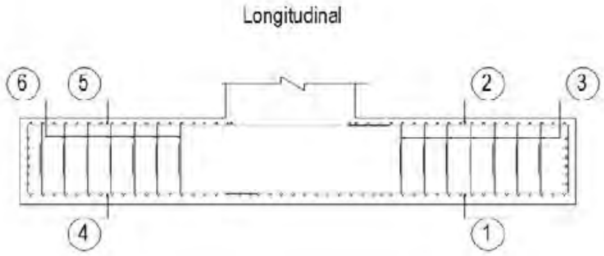
	PIER
Cross Section of Pile Cap (Longitudinal & Transversal Direction) SD345	

			P23		P24		P25	
			LL	TT	LL	TT	LL	TT
Arangement of reinforcement	①	④	2-D35@125	2-D32@250	1.5-D32@125	2-D25@250	D32@125	D29@250
	②	⑤	D29@125	D29@250	D32@250	D22@250	D29@250	D19@250
	③	⑥	D16@500	D16@500	D16@500	D16@500	D16@500	D16@500
Check for Bending Stress	Ordinary	σs (N/mm^2)	115.19	119.20	128.54	47.28	124.04	47.67
		σsa(N/mm^2)	160.00	160.00	160.00	160.00	160.00	160.00
		R-ratio	0.72	0.75	0.80	0.30	0.78	0.30
	Seismic	σs (N/mm^2)	255.46	247.37	244.38	47.28	188.66	47.67
		σsa(N/mm^2)	300.00	300.00	300.00	300.00	300.00	300.00
		R-ratio	0.85	0.82	0.81	0.16	0.63	0.16
Check for Shear Stress	Ordinary	τm (N/mm^2)	0.556	—	0.456	—	0.642	—
		τa (N/mm^2)	0.838	—	0.559	—	1.160	—
		R-ratio	0.66	—	0.82	—	0.55	—
	Seismic	τm (N/mm^2)	1.191	—	0.800	—	0.937	—
		τa (N/mm^2)	1.275	—	0.851	—	1.766	—
		R-ratio	0.93	—	0.94	—	0.53	—

σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.4.26 Calculation Results for Footing of Abutments (A1, A2, and AO1)

			ABUTMENT					
Cross Section of Pile Cap (Longitudinal & Transversal Direction) SD345								
			A1		A2		AO1	
			FRONT	BACK	FRONT	BACK	FRONT	BACK
Arangement of reinforcement	①	④	D29@125	D29@250	D29@125	D29@250	D29@250	D25@250
	②	⑤	D29@250	D25@125	D29@250	D25@250	D22@250	D32@250
	③	⑥	D16@500	D16@500	D16@500	D16@500	D16@500	D16@500
Check for Bending Stress	Ordinary	σs (N/mm²)	117.42	27.92	152.83	38.34	135.89	60.23
		σsa(N/mm²)	160.00	184.00	160.00	160.00	160.00	160.00
		R-ratio	0.73	0.15	0.96	0.24	0.85	0.38
	Seismic	σs (N/mm²)	224.96	204.16	283.10	273.60	264.21	235.89
		σsa(N/mm²)	300.00	300.00	300.00	300.00	300.00	300.00
		R-ratio	0.75	0.68	0.94	0.91	0.88	0.79
Check for Shear Stress	Ordinary	τm (N/mm²)	0.254	0.154	0.335	0.153	0.260	0.143
		τa (N/mm²)	0.473	0.190	0.519	0.259	0.747	0.282
		R-ratio	0.54	0.81	0.65	0.59	0.35	0.51
	Seismic	τm (N/mm²)	0.439	0.284	0.579	0.204	0.466	0.264
		τa (N/mm²)	0.720	0.289	0.790	0.348	1.137	0.429
		R-ratio	0.61	0.98	0.73	0.59	0.41	0.62

σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

4.4.7.3 Design of Foundation

Pile foundation and SPSP foundation shall conform to the following requirements under ordinary, earthquake, and vessel collision conditions:

- The axial reaction at each pile head shall not exceed the allowable pile bearing capacity. The axial allowable bearing capacity can be estimated by dividing the ultimate bearing capacity of the pile determined from related factors such as ground conditions and construction methods by the factor of safety.
- The displacements at each pile head shall not exceed the allowable displacements in order not to leave a large residual displacement and to keep within the limit of possibility of evaluation of elastic behavior. The allowable horizontal displacement is principally determined to be 1% based on the results of many loading tests.

  For a large elastic foundation with a width of 5 m or more such as SPSP foundation, the allowable displacement is determined to be 50 mm because few loading tests data are available.

  For a pile foundation with a pile diameter of 1.5 m or less, the allowable displacement is 15 mm. For a pile foundation with a pile diameter of 2.0 m, the allowable displacement is 20 mm.

  The allowable displacement of abutment foundation is 15 mm regardless of the foundation width because the displacement may increase with time due to the effects of creep and backfill settlement.
- The axial reaction at each pile head shall not exceed the allowable pile bearing capacity.

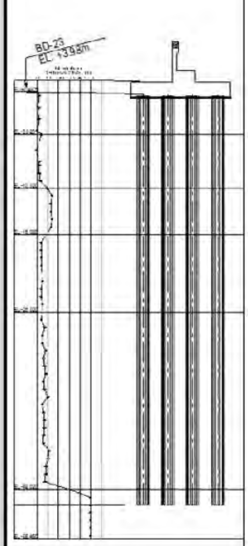
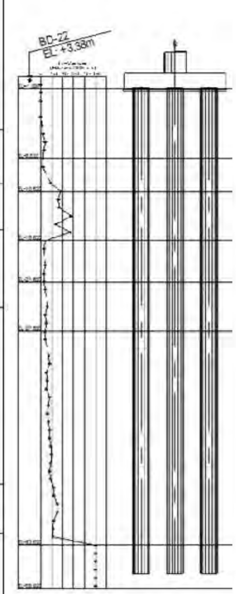
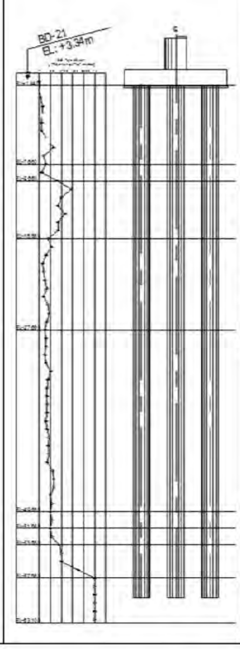
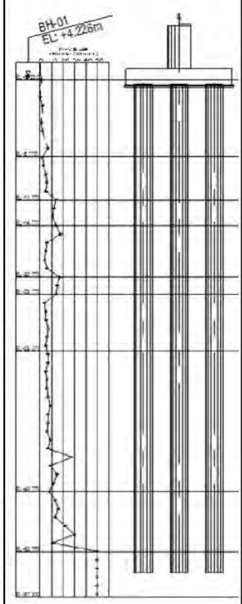
➢ The stresses generated in members of pile foundations shall not exceed the allowable stresses specified in the relevant section of this report.

Computation results of CIP pile foundation stability are shown in Table 4.4.27 through Table 4.4.29.

The calculation results of cross sectional stress of CIP piles are shown in Table 4.4.30 through Table 4.4.32.

Also, calculation results of SPSP are summarized in through Table 4.4.33.

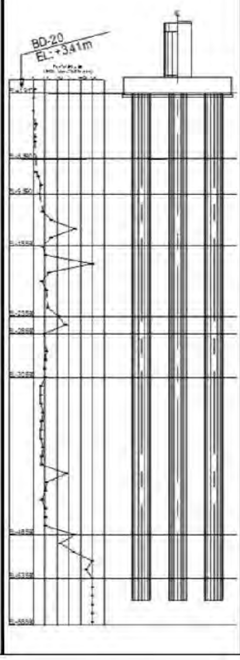
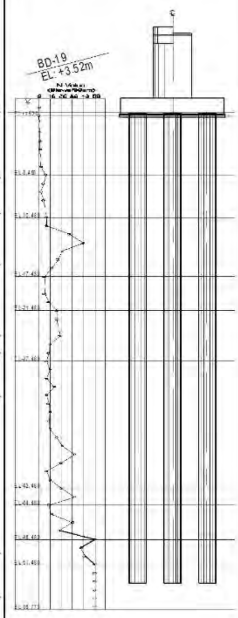
Table 4.4.27 Calculation Results of CIP Pile Foundation Stability (A1~P3)

	A1	P1	P2	P3
Boling Log & Pile Length (m)				
Pile Information				
Diameter of Pile (mm)	1,500	2,000	2,000	2,000
Number of Piles (Nos.)	28	12	12	12
Pile Length (m)	52.9	57.9	61.9	56.9
Bearing Resistance of Ordinary				
Pile Head Reaction (kN)	1,602	4,196	4,019	4,307
Bearing Capacity (kN)	3,730	5,531	7,140	6,320
R-Ratio	0.430	0.759	0.563	0.681
Horizontal Movement of Ordinary				
Horizontal Movement (mm)	3.7	4.1	1.8	1.7
Capacity (mm)	15.0	20.0	20.0	20.0
R-Ratio	0.246	0.205	0.090	0.083
Bearing Resistance of Seismic				
Pile Head Reaction (kN)	2,384	5,982	6,677	6,762
Bearing Capacity (kN)	5,916	8,920	11,372	10,087
R-Ratio	0.403	0.671	0.587	0.670
Horizontal Movement of Seismic				
Horizontal Movement (mm)	13.5	17.5	19.3	18.2
Capacity (mm)	15.0	20.0	20.0	20.0
R-Ratio	0.902	0.876	0.964	0.909
Bearing Capacity of Group Piles of Ordinary				
Axial Compression Fordes (kN	42,816	43,606	44,962	—
Bearing Capacity (kN)	1,053,211	949,265	1,026,902	—
R-Ratio	0.041	0.046	0.044	—
Judgement of Lateral Movement				
Identifying Index	1.830	—	—	—
Capacity	1.200	—	—	—

* We also conducted a study on the negative skin friction force, but another case became severe in all foundations, so the value of bearing resistance is ordinary case.

Source: JICA Study Team

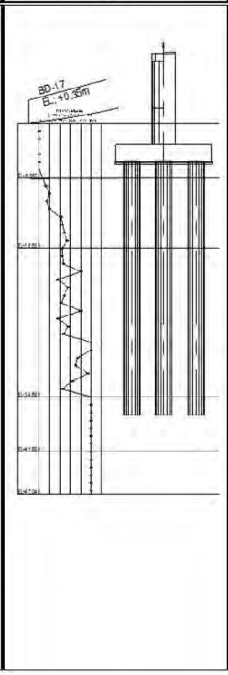
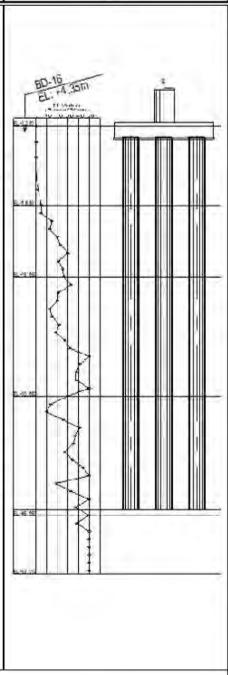
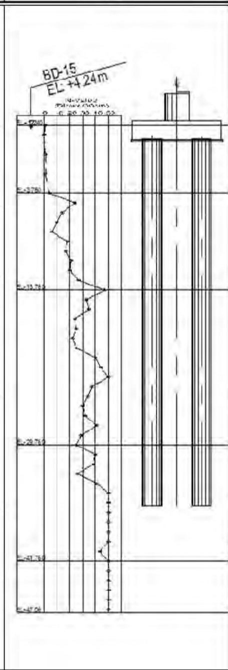
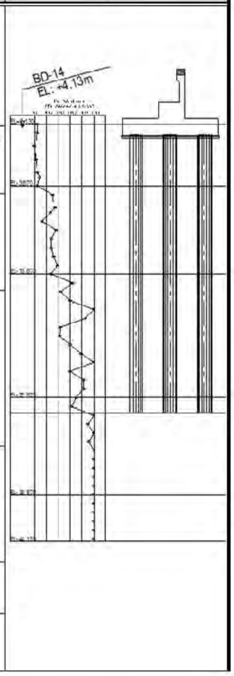
Table 4.4.28 Calculation Results of CIP Pile Foundation Stability (P4 and P5)

	P4	P5
Boling Log & Pile Length (m)		
Pile Information		
Diameter of Pile (mm)	2,000	2,000
Number of Piles (Nos.)	12	21
Pile Length (m)	57.9	55.4
Bearing Resistance of Ordinary		
Pile Head Reaction (kN)	4,506	3,415
Bearing Capacity (kN)	6,511	6,127
R-Ratio	0.692	0.557
Horizontal Movement of Ordinary		
Horizontal Movement (mm)	4.2	1.8
Capacity (mm)	20.0	20.0
R-Ratio	0.209	0.088
Bearing Resistance of Seismic		
Pile Head Reaction (kN)	7,090	6,443
Bearing Capacity (kN)	10,386	9,783
R-Ratio	0.683	0.659
Horizontal Movement of Seismic		
Horizontal Movement (mm)	19.1	18.0
Capacity (mm)	20.0	20.0
R-Ratio	0.955	0.901
Bearing Capacity of Group Piles of Ordinary		
Axial Compression Fordes (kN)	45,210	63,530
Bearing Capacity (kN)	791,906	1,413,665
R-Ratio	0.057	0.045

* We also conducted a study on the negative skin friction force, but another case became severe in all foundations, so the valu bearing resistance is ordinary case.

Source: JICA Study Team

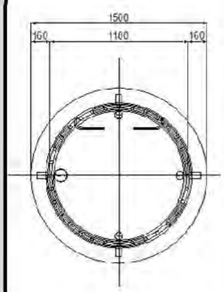
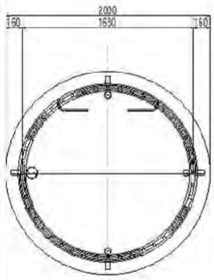
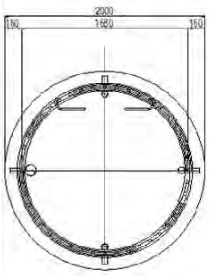
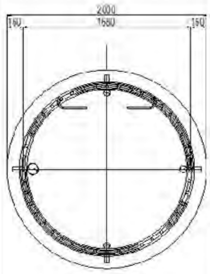
Table 4.4.29 Calculation Results of CIP Pile Foundation Stability (P23~P25 and A2)

	P23	P24	P25	A2
Boling Log & Pile Length (m)				
Pile Information				
Diameter of Pile (mm)	2,000	2,000	2,000	1,500
Number of Piles (Nos.)	12	12	8	18
Pile Length (m)	32.4	46.9	37.9	31.4
Bearing Resistance of Ordinary				
Pile Head Reaction (kN)	5,554	4,223	5,922	2,299
Bearing Capacity (kN)	8,559	11,527	9,177	5,085
R-Ratio	0.649	0.366	0.645	0.452
Horizontal Movement of Ordinary				
Horizontal Movement (mm)	0.3	2.5	5.0	4.6
Capacity (mm)	20.0	20.0	20.0	15.0
R-Ratio	0.017	0.123	0.251	0.303
Bearing Resistance of Seismic				
Pile Head Reaction (kN)	10352.4	6,676	7,879	3,537
Bearing Capacity (kN)	12959.0	17,731	14,137	7,807
R-Ratio	0.799	0.376	0.557	0.453
Horizontal Movement of Seismic				
Horizontal Movement (mm)	13.9	16.4	17.8	14.6
Capacity (mm)	20.0	20.0	20.0	15.0
R-Ratio	0.695	0.819	0.892	0.971
Bearing Capacity of Group Piles of Ordinary				
Axial Compression Fordes (kN)	65,741	45,475	—	39,934
Bearing Capacity (kN)	460,353	812,083	—	534,902
R-Ratio	0.143	0.056	—	0.075
Judgement of Lateral Movement				
Identifying Index	—	—	—	0.509
Capacity	—	—	—	1.200

* We also conducted a study on the negative skin friction force, but another case became severe in all foundations, so the value of bearing resistance is ordinary case.

Source: JICA Study Team

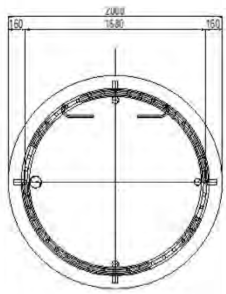
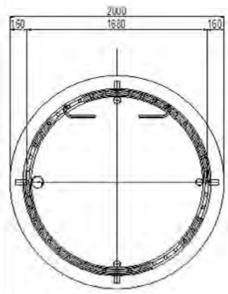
Table 4.4.30 Calculation Results of Cross Section of CIP Pile Foundation (A1~P3)

	A1	P1	P2	P3
Cross Section of Pile SD345	 32-D29@116 AS=205.568cm^2	 44-D32@120 AS=349.448cm^2	 44-D32@120 AS=349.448cm^2	 44-D32@120 AS=349.448cm^2
Check for Bending Stress				
Ordinary				
σs (N/mm^2)	37.98	2.05	—	—
σsa (N/mm^2)	184.00	184.00	—	—
R-ratio	0.21	0.01	—	—
Seismic				
σs (N/mm^2)	261.33	231.75	261.79	272.44
σsa (N/mm^2)	300.00	300.00	300.00	300.00
R-ratio	0.87	0.77	0.87	0.91
Check for Shear Stress				
Ordinary				
τm (N/mm^2)	0.095	0.052	0.022	0.022
τa (N/mm^2)	0.446	0.505	0.601	0.601
R-ratio	0.21	0.10	0.04	0.04
Seismic				
τ m(N/mm^2)	0.335	0.324	0.354	0.378
τa (N/mm^2)	0.445	0.399	0.399	0.399
R-ratio	0.75	0.81	0.89	0.95

σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force

Source: JICA Study Team

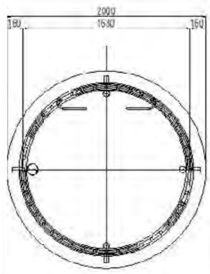
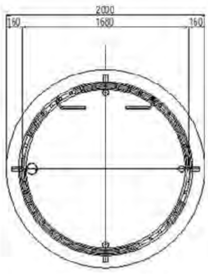
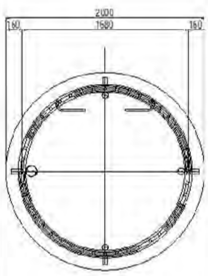
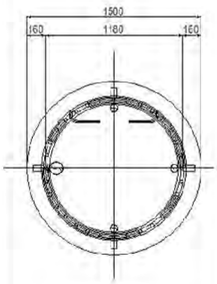
Table 4.4.31 Calculation Results of Cross Section of CIP Pile Foundation (P4 and P5)

	P4	P5
Cross Section of Pile SD345	 44-D35@120 AS=420.904cm^2	 44-D32@120 AS=349.448cm^2
Check for Bending Stress		
Ordinary		
σs (N/mm^2)	1.24	—
σsa (N/mm^2)	184.00	—
R-ratio	0.01	—
Seismic		
σs (N/mm^2)	211.74	248.61
σsa (N/mm^2)	300.00	300.00
R-ratio	0.71	0.83
Check for Shear Stress		
Ordinary		
τm (N/mm2)	0.052	0.018
τa (N/mm2)	0.562	0.524
R-ratio	0.09	0.03
Seismic		
τ m(N/mm2)	0.338	0.315
τa (N/mm2)	0.422	0.399
R-ratio	0.80	0.79

σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force

Source: JICA Study Team

Table 4.4.32 Calculation Results of Cross Section of CIP Pile Foundation (P23~P25 and A2)

	P23	P24	P25	A2
Cross Section of Pile SD345	 44-D35@120 AS=420.904cm^2	 44-D32@120 AS=349.448cm^2	 44-D35@120 AS=420.904cm^2	 32-D35@116 AS=306.112cm^2
Check for Bending Stress				
Ordinary				
σs (N/mm2)	—	—	5.88	54.92
σsa (N/mm2)	—	—	184.00	184.00
R-ratio	—	—	0.03	0.30
Seismic				
σs (N/mm2)	288.62	271.91	260.33	269.11
σsa (N/mm2)	300.00	300.00	300.00	300.00
R-ratio	0.96	0.91	0.87	0.90
Check for Shear Stress				
Ordinary				
τm (N/mm2)	0.035	0.036	0.088	0.164
τa (N/mm2)	0.636	0.601	0.616	0.474
R-ratio	0.06	0.06	0.14	0.35
Seismic				
τ m(N/mm2)	0.457	0.355	0.457	0.529
τa (N/mm2)	0.422 (2.550)	0.399	0.437 (2.550)	0.508 (2.550)
R-ratio	1.08 (0.18)	0.89	1.05 (0.18)	1.04 (0.21)

σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force

Source: JICA Study Team

Table 4.4.33 Calculation Results for Connection Stud of SPSP Foundation

Design condition

- Type of stad bars : SD345 (underwater)
- Design strength of concrete : σck = 24 (N/mm²)
- Material of sheet pile : SKY490 (P20,P21), SKY400(P22)
- Diameter of sheet pile : D = 1200.0 (mm)
- Section modulus of sheet pile : Z = 13081.0(P20,P21), 15184.5(P22) (cm³)
- Connection method : reinforcement stud welding

Table 8.3.56-5 Design Results of connection between Top Slab and Steel Pipe Sheet Pile

	Load case	σs1 (N/mm²)	σs2 (N/mm²)	σs (N/mm²)	σsa (N/mm²)	nb nba (nos/layer)	τs (N/mm²)	τsa (N/mm²)	ns nsa (nos)
P20	Ordinary	153.93	4.60	158.53	185.00	16 ≧ 14	66.15	111.00	76 ≧ 46
P20	Seismic	200.46	38.51	238.97	300.00	16 ≧ 13	124.64	180.00	76 ≧ 53
P21	Ordinary	153.93	6.86	160.79	185.00	16 ≧ 14	75.05	111.00	76 ≧ 52
P21	Seismic	200.46	38.67	239.13	300.00	16 ≧13	155.78	180.00	76 ≧66
P22	Ordinary	116.35	2.02	118.37	160.00	16 ≧ 12	59.42	96.00	76 ≧ 48
P22	Seismic	174.52	35.52	210.04	300.00	16 ≧ 12	146.40	180.00	76 ≧ 62

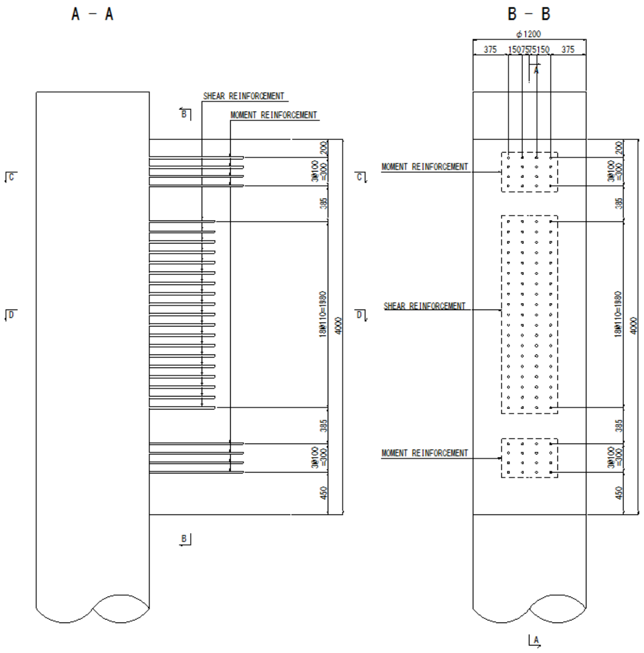


Figure : Detail for Connection between Top Slab and Steel Pipe Sheet Pile

Source: JICA Study Team

4.4.8 Bridge Accessories

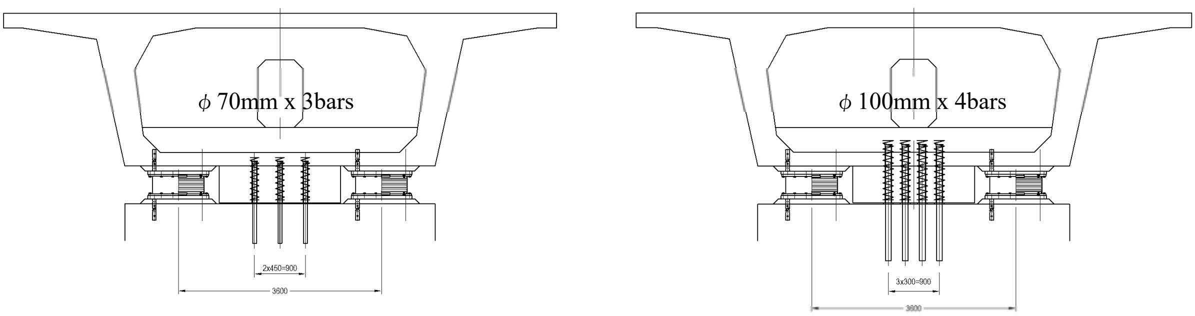
4.4.8.1 Bearings

The lengths of PC box girder bridges are L = 250 m in Thilawa side (A1 side) and L = 300 m in Yangon side (A2 side), and the effect of restraint forces is significant due to the shortening of the girder induced by creep and shrinkage as well as due to the shortening/expansion induced by temperature change. On the other hand, horizontal forces from the superstructure during earthquake must be adequately distributed to each substructure. For the support condition of the PC box girder bridges, therefore, the superstructure is planned to be elastically supported in the longitudinal direction, and elastomeric rubber bearings are adopted. The superstructure is transversally fixed, considering the connection with the on-ramp bridge

Table 4.4.34 Comparison of Support Condition and Bearing Type

	Elastic Support	Fixed + Moveable Support
Applicable type of bearings	Elastomeric Rubber Bearing 	Pot Bearing 
Effect of restraint forces	• Effect of restraint force to substructures is smaller, as the superstructure is elastically supported in the longitudinal direction.	• Effect of restraint forces to substructures is larger, as the superstructure is fixed at most of the superstructures.
Transfer of seismic horizontal force	• In the longitudinal direction, horizontal forces are elastically distributed to each substructure. • In the transverse direction, horizontal forces are transferred from superstructure to substructures by anchor bars.	• Horizontal forces are transferred to the substructures through steel components of bearings. Substructures with movable supports do not contribute in resisting seismic forces.
Evaluation	**RECOMMENDED**	

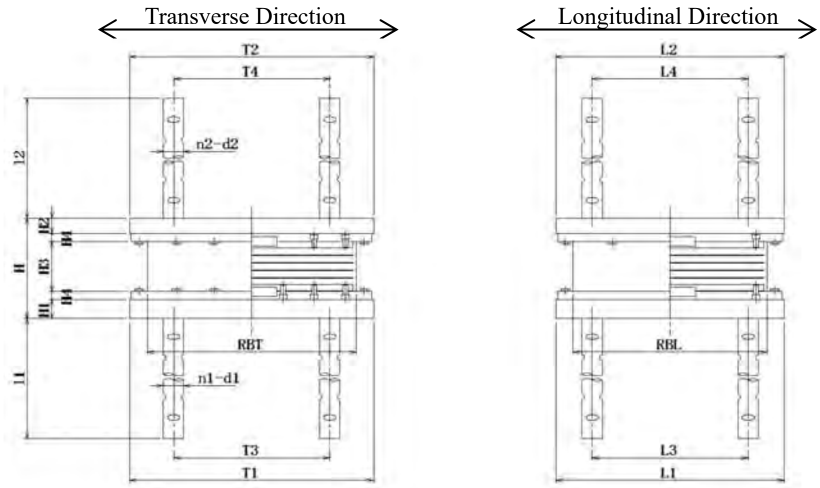
Source: JICA Study Team



At End Support At Intermediate Support

Source: JICA Study Team

Figure 4.4.21 Arrangement of Bearing and Anchor Bar



	Base Plate			Anchor Bolt					Rubber Bearing		
	L1	T1	H1	d1	l1	n1	L3	T3	RBL	RBT	H3
A1	1080	1080	60	φ65	650	4	850	850	920	920	309
P1	1420	1420	75	φ85	850	4	1100	1100	1220	1220	293
P2	1420	1420	75	φ85	850	4	1100	1100	1220	1220	258
P3	1420	1420	75	φ85	850	4	1100	1100	1220	1220	258
P4	1420	1420	75	φ85	850	4	1100	1100	1220	1220	293
P5	1080	1080	60	φ65	650	4	850	850	920	920	275

	Sole Plate			Anchor Bar					PL	Height
	L2	T2	H2	d2	l2	n2	L4	T4	H4	H
A1	1080	1080	60	φ65	650	4	850	850	32	493
P1	1420	1420	75	φ85	850	4	1000	1000	40	523
P2	1420	1420	75	φ85	850	4	1000	1000	40	488
P3	1420	1420	75	φ85	850	4	1000	1000	40	488
P4	1420	1420	75	φ85	850	4	1000	1000	40	523
P5	1080	1080	60	φ65	650	4	850	850	32	459

	Base Plate			Anchor Bolt					Rubber Bearing		
	L1	T1	H1	d1	l1	n1	L3	T3	RBL	RBT	H3
P20	1080	1080	60	φ65	650	4	850	850	920	920	309
P21	1420	1420	75	φ85	850	4	1100	1100	1220	1220	293
P22	1420	1420	75	φ85	850	4	1100	1100	1220	1220	293
P23	1420	1420	75	φ85	850	4	1100	1100	1220	1220	258
P24	1420	1420	75	φ85	850	4	1100	1100	1220	1220	293
P25	1420	1420	75	φ85	850	4	1100	1100	1220	1220	327
A2	1080	1080	60	φ65	650	4	850	850	920	920	323

	Sole Plate			Anchor Bar					PL	Height
	L2	T2	H2	d2	l2	n2	L4	T4	H4	H
P20	1080	1080	60	φ65	650	4	850	850	32	493
P21	1420	1420	75	φ85	850	4	1000	1000	40	523
P22	1420	1420	75	φ85	850	4	1000	1000	40	523
P23	1420	1420	75	φ85	850	4	1000	1000	40	488
P24	1420	1420	75	φ85	850	4	1000	1000	40	523
P25	1420	1420	75	φ85	850	4	1000	1000	40	557
A2	1080	1080	60	φ65	650	4	850	850	32	507

Source: JICA Study Team

Figure 4.4.22 Elastomeric Rubber Bearing

4.4.8.2 Expansion Joints

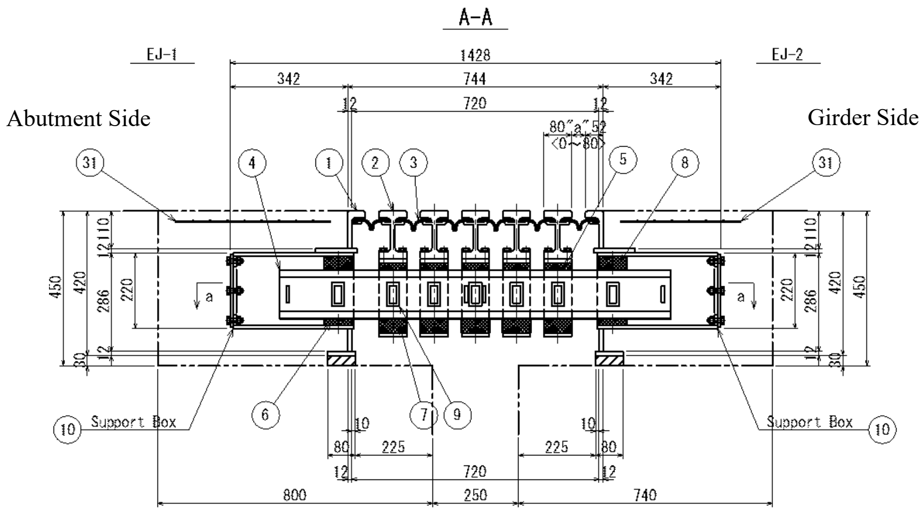
As the horizontal movement of PC box girder bridge during earthquake is large due to the relatively high design seismic coefficient (kh = 0.3), its expansion joints need to accommodate large displacement. As a result of the following comparative study, “modular expansion joint” has been selected, considering various aspects such as waterproofing, driving comfort, and maintenance as well as accommodation of large displacement.

Table 4.4.35 Comparison of Expansion Joint Type for PC Box Girder Bridge

	Modular Expansion Joint	Steel Finger Joint
Schematic View		
Accommodation of large displacement	• Can accommodate wide range of movement, and applicable especially to large movement.	• Can accommodate wide range of movement.
Waterproofing	• Excellent cut-off performance against water.	• Moderate cut-off performance against water.
Driving comfort	• Good driving comfort	• Good driving comfort
Maintenance	• High durability of steel components • The components can be replaced relatively easily.	• Relatively difficult to replace the components.
Evaluation	**RECOMMENDED**	

Source: JICA Study Team

Design Result (A1 and A2)



Source: JICA Study Team

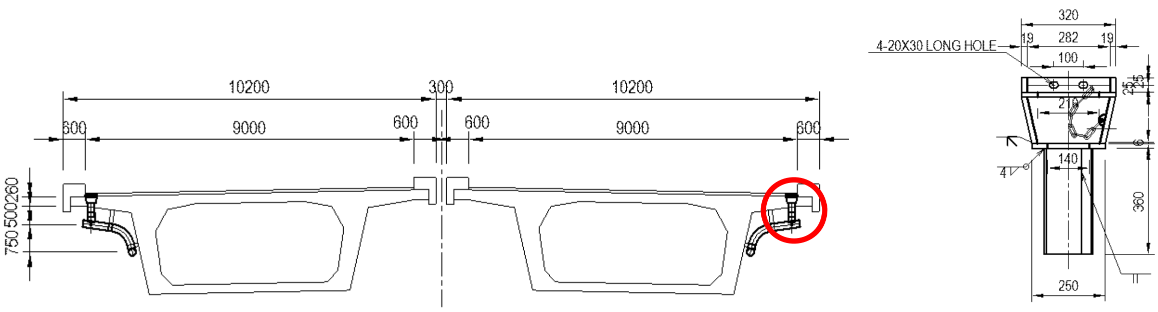
Figure 4.4.23 Expansion Joint at A1 and A2

4.4.8.3 Bridge Railing

Steel railings have been adopted as the bridge railings, uniformly with the main bridge (with cable-stayed bridge and steel box girder bridge). Class of railing is Class A in "Specifications for Highway Railings" by Japan Road Association. The heights are 0.9 m at the median side and 1.1 m at the roadside considering fall prevention of pedestrians in case of emergency.

4.4.8.4 Drainage System

Rainwater on the bridge surface is drained by catch pits installed at the shoulder of the bridge deck. As the bridge is located on land for the A1~P5 section, the rainwater from the catch pits is horizontally led to the substructures, and then vertically drained to the catch basin on the ground, which is connected to the side ditch. For the A2 side, the rainwater from the catch pits between P20~P23 (in-river section) is led under the girder by vertical drain pipes and discharged on to the river, while rainwater from those between P23~A2 (on-land section) is treated in the same manner as in the A1~P5 section.



Source: JICA Study Team

Figure 4.4.24 Catch Pits Arrangement and Detail (PC Box Girder Bridge)

4.5 STUDY ON ON-RAMP BRIDGE

A summary of the evolution of design output is shown in Table 4.5.1.

Table 4.5.1 Summary of Design Outputs Evolution

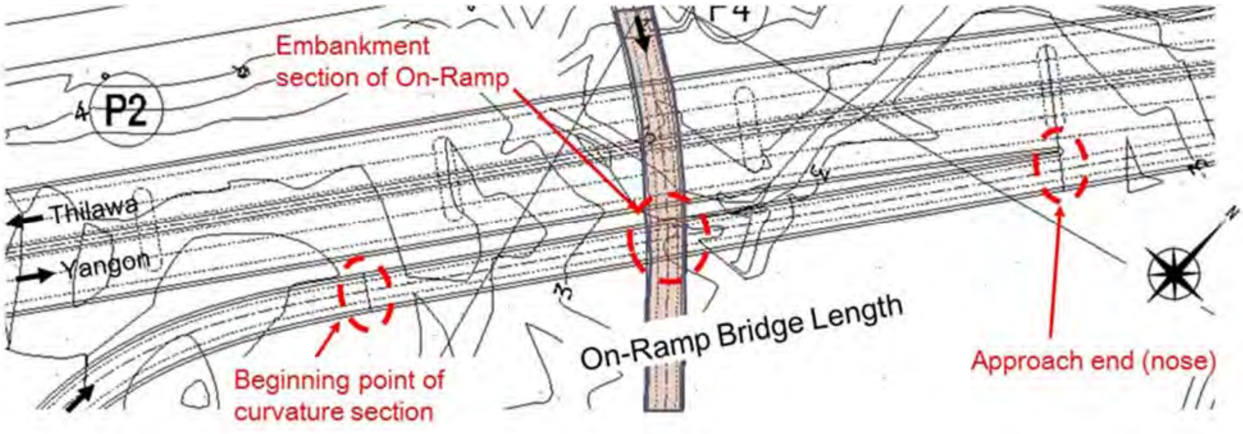
Item	Feasibility Study	Basic Design	Detailed Design
Bridge Width	5.750 m	6.450 m	6.450 m
Superstructure	PC-I Girder 3 girders	PC-I Girder 2 girder	PC-I Girder 2 girder
Bridge Length	187.8 m	115.2 m	115.2 m
Number of Substructure	7 nos.	5 nos.	5 nos.
Foundation Type	Cast-In-Situ: 7 nos. Diameter: 1.0 m	Cast-In-Situ: 5 nos. Diameter: 1.5 m	Cast-In-Situ: 5 nos. Diameter: 1.5 m

Source: JICA Study Team

4.5.1 Study on Bridge Length of On-ramp Bridge

4.5.1.1 Determination of Bridge Length and Span Arrangement

The previously mentioned study conditions are illustrated as follows:



Source: JICA Study Team

Figure 4.5.1 Control Points for Bridge Length and Span Arrangement

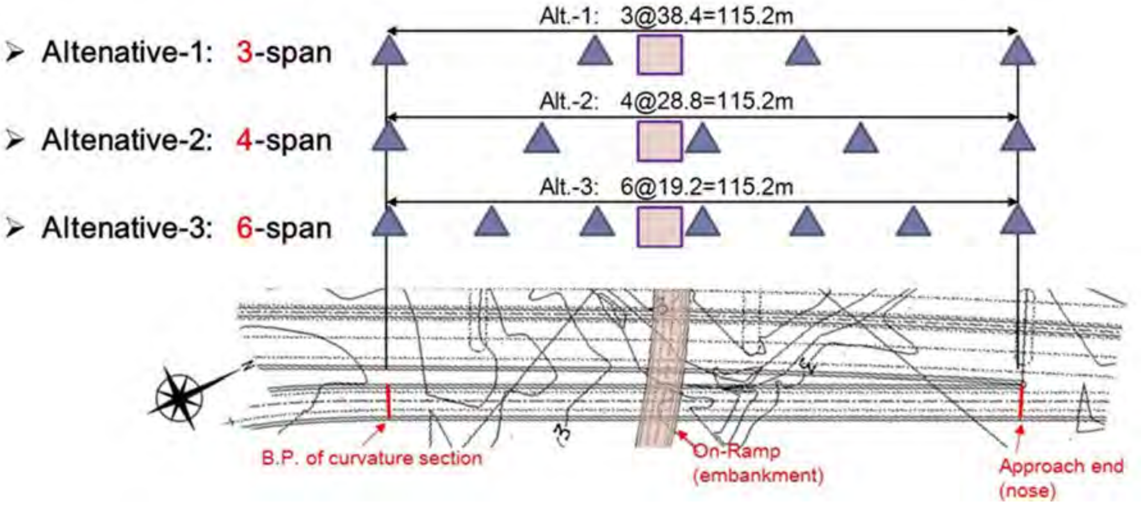
Beginning Point (Abutment) : STA No.0+410.000 (approximate station number)

End Point (Pier) : STA No.0+526.000

4.5.1.2 Study on Span Arrangement

- Alternatives

There are two restrictions that control the bridge length. These are the abutment location as the beginning point of the on-ramp bridge at STA No.0+410.000 and the approach end (nose) as the end point of the on-ramp bridge at STA No.0+526.000, as displayed in Figure 4.5.1. Piers are arranged between these control points with careful attention to the embankment section of the on-ramp road as the crossing object. Span length should be close to 30 m referring to the economical span length of this bridge. After due consideration of the span arrangement, three alternatives were proposed as follows:

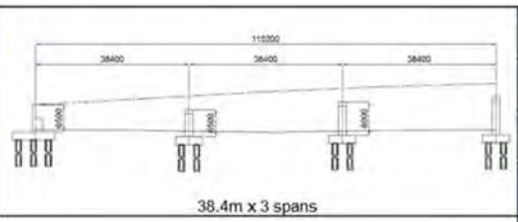
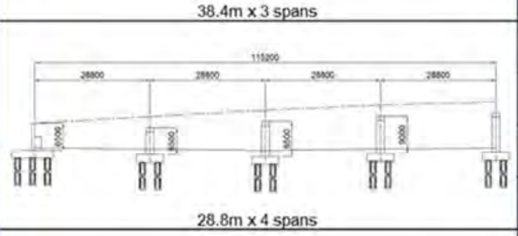
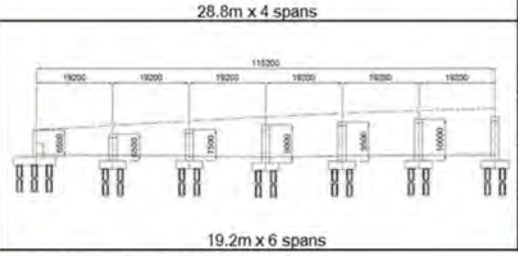


Source: JICA Study Team

Figure 4.5.2 Alternatives for Span Arrangement for On-ramp Bridge

- Comparison Result

Table 4.5.2 Comparison of Span Arrangement (On-ramp Bridge)

Alt-1	 38.4m x 3 spans	Constructability & Quality Control	- This alternative entails smaller amount of pier.	○
		Cost Ratio	1.02	○
		Environmental Aspect	-The amount of excavated soil is the smallest.	◎
		Evaluation	Less Recommended	
Alt-2	 28.8m x 4 spans	Constructability & Quality Control	- This alternative entails smaller amount of pier.	○
		Cost Ratio	1.00	◎
		Environmental Aspect	-The amount of excavated soil is smaller.	○
		Evaluation	Most Recommended	
Alt-3	 19.2m x 6 spans	Constructability & Quality Control	- The amount of number of pier is the largest and thus this alternative is the most inferior one in terms of constructability.	△
		Cost Ratio	1.21	△
		Environmental Aspect	-The amount of excavated soil is the largest.	△
		Evaluation	Less Recommended	

Legend : ◎ Very Good, ○ Good, △ Average

Source: JICA Study Team

4.5.2 Study on Superstructure of On-ramp Bridge

4.5.2.1 Selection of Type of On-ramp Bridge

(1) Comparative Study

The study is carried out for the following three alternatives, and the optimum option is selected based on the study on workability (quality control), structural aspects, cost, and maintenance.

Option-1: PC Hollow Slab	Option-2: PC I Girder	Option-3: Steel I Girder

Option-1: PC Hollow Slab

PC hollow slab has been widely used in ramp bridges due to its applicability to curved alignment (cast-in-place) and low girder height. In this case, however, soil improvement might be necessary to support conventional falsework required for construction of superstructure.

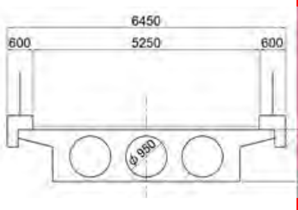
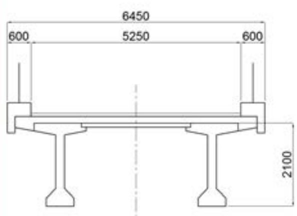
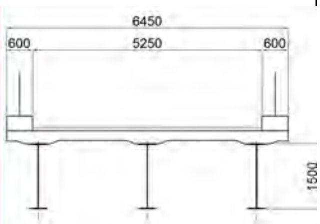
Option-2: PC I Girder

PC I girder is one of the most economical options, and can be applied to this on-ramp bridge without problem as it is planned as a straight bridge. Fabrication yard for precast girders is required.

Option-3: Steel I Girder

A steel girder with RC slab. Periodical re-painting for steel member is required.

Table 4.5.3 Comparison of Bridge Types for On-ramp Bridge

	PC Hollow	PC I Girder (Plan at F/S)	Steel I Girder
Reference drawing			
Election Method	All Staging Method	Crane Erection Method	Crane Erection Method
Workability and Quality Control	• Inferior in quality control as the girder is cast-in-situ. • Soil improvement might be necessary in order to support falseworks.	• Superior in quality control as the girders are pre-cast. • No scaffolding below the girder is required. • Girder fabrication yard is required.	• Superior in quality control as the girders are pre-fabricated in factory. • No special problem on erection, although the scaffolding below the girder is required.
Structural Aspect	• Applicable span length: 20-30 m • Heavy weight.	• Applicable span length: 25-40 m • Moderate weight.	• Applicable span length: 25-60 m • Light weight.
Cost	Ratio = 1.04	Ratio = 1.00	Ratio = 1.05
Maintenance Aspect	• Replacement of bearings and expansion joints is required.	• Replacement of bearings and expansion joints is required.	• Re-painting is required in addition to replacement of bearings and expansion joints.
Evaluation		**Most Recommended**	

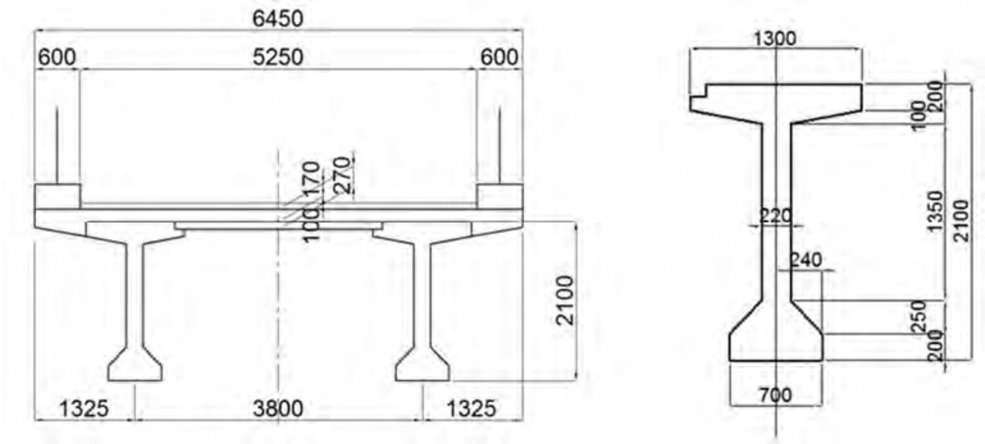
Source: JICA Study Team

As a result of the study, PC I girder has been selected as the bridge type of the on-ramp bridge because of lowest cost, without need of conventional falsework, and superior in quality control. Girder fabrication yard can be prepared adjacent to the bridge.

4.5.2.2 Superstructure of On-ramp Bridge

(1) Girder Arrangement

The girder arrangement is planned based on the policy of reducing the weight of superstructure as much as possible, in order to reduce the seismic load to substructure. As the bridge width is 6.45 m, two girders with 3.8 m spacing has been adopted.

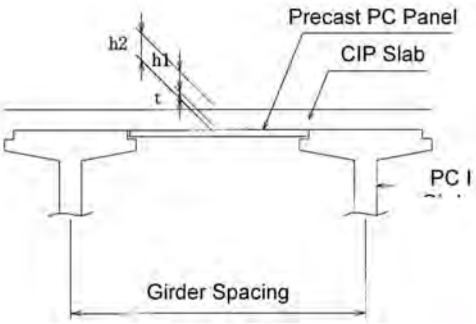


Source: JICA Study Team

Figure 4.5.3 Arrangement and Cross Section of the Girder

(2) Slab Thickness

Slab thickness is planned as the standard thickness related to the girder spacing. As the girder spacing is 3.8 m, total slab thickness (PC panel + CIP slab) is 270 mm.



Girder	PC Panel t (mm)	CIP Slab h1 (mm)	Composite h2* (mm)
2 600mm	70	160	230
2 900mm	70	170	240
3 200mm	80	170	250
3 500mm	90	170	260
3 800mm	100	170	270

Source: "Guidebook for design and construction of PC-I girder bridges with precast PC panel" by Japan Prestressed Concrete Contractors Association

Figure 4.5.4 Slab Thickness

(3) Prestressing Tendon

1) Longitudinal Tendons

The 12S12.7 mm is applied as longitudinal tendons, referring to "Guidebook for design and construction of PC-I girder bridges with precast PC panel" by Japan Prestressed Concrete Contractors Association.

2) Transverse Tendons for Precast PC Panel of Deck Slab

PC tendons for precast PC panels of deck slab are planned to be pre-tensioned. The 1S9.3 mm is applied as transverse tendons for the precast PC panels, referring to "Guidebook for design and construction of PC-I girder bridges with precast PC panel" by Japan Prestressed Concrete Contractors Association.

3) Tendons for Crossbeam Reinforcement

For transversal prestressing, PC bars with diameter of 32 mm have been applied, as the transversal tendon is short and PC bar with threaded anchorage system is advantageous than PC strands with wedge anchorages which have large loss of prestress for short tendons by pull-in of wedges.

4.5.3 Substructure of On-ramp Bridge

4.5.3.1 Study of Substructure Height

(1) General

Substructure heights were designed referring to the proposed heights of the planned road (PH), ground level (GL) and required heights related to superstructure which include the height from pavement structure through bridge bearing. As a result, substructure heights were determined and rounded to the nearest 10 cm. Refer to the schematic diagram shown in Figure 4.5.5.

Considering the elevation of reclamation for construction yard preparation (MSL+4.300 m), foundation level of on-land substructures was determined based on MSL+4.300 m.

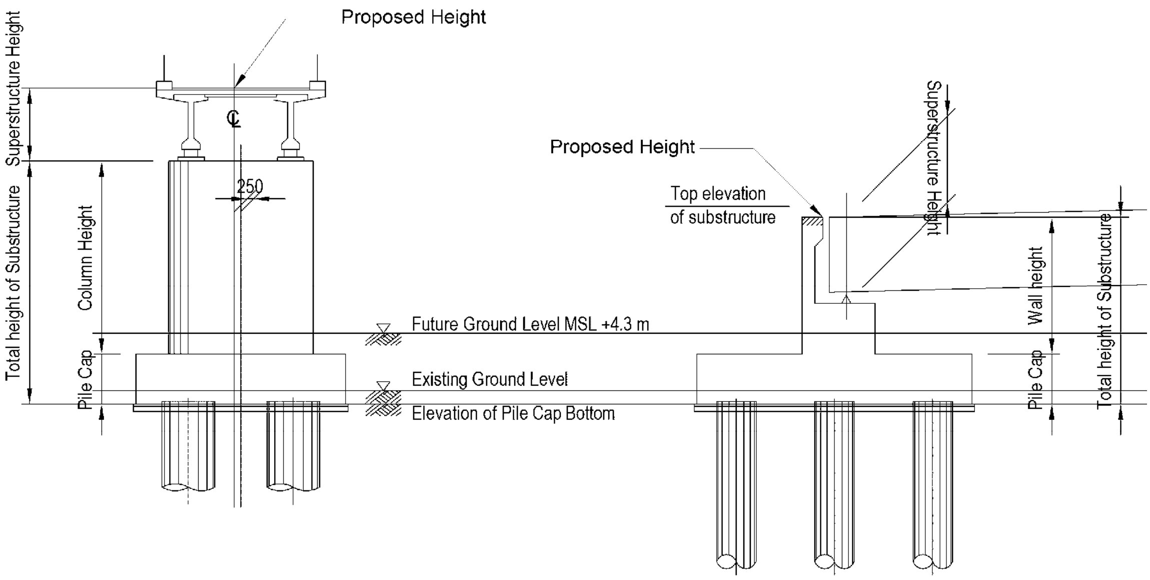


Figure 4.5.5 Explanatory Diagram of Substructure Height

(2) Conclusion of Substructure Heights

The conclusions on substructure heights are presented in Table 4.5.4.

Table 4.5.4 Summary of Substructure Heights of On-ramp Bridge

Item	Mark	Unit	AO1	PO1	PO2	PO3
Station Number	STA	m	0+411.009	0+439.809	0+468.609	0+497.409
Proposed height	PH	m	9.452	11.030	12.587	13.803
Top elevation of substructure	KCL	m	9.452	8.332	9.891	11.111
Existing Ground EL	GL1	m	3.281	2.936	2.959	3.076
Future Ground EL	GL	m	4.300	4.300	4.300	4.300
Pile cap thickness	FH	m	1.900	1.900	1.900	1.900
Total Substructure height	**H**	**m**	**7.600**	**6.500**	**8.100**	**9.300**
EL of Pile cap bottom	FL	m	1.852	1.832	1.791	1.811
Foundation Type	-	-	CIP Pile	CIP Pile	CIP Pile	CIP Pile

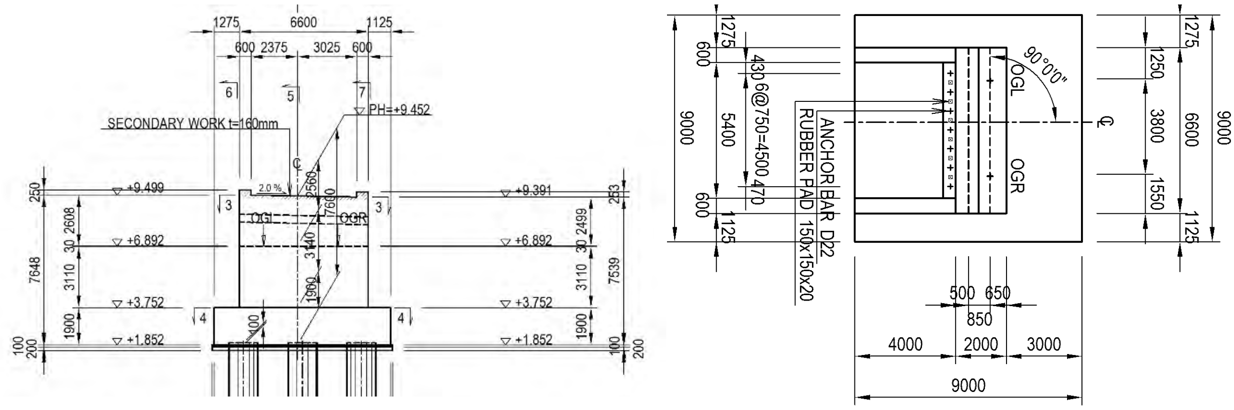
Source: JICA Study Team

4.5.3.2 Dimensions of Abutment

(1) Width

The width at the top surface of the parapet wall shall be the same as the effective cross section of the road or wider. The abutment AO1 is located between a straight section and an easement (clothoid) curve section of on-ramp bridge. Thus, a certain amount of road widening is necessary for securing the prescribed effective road width.

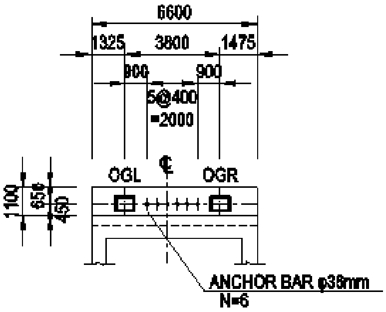
Concretely, the intersection point of the front edge of the parapet wall and the inside surface of the left side curb concrete line should be a control point for the left side width. In the same way, the intersection point of the right side curb concrete line and the end edge of approach slab should be the control point for the right side width. As a result of the above consideration, the required distance of the widening is 150 mm.



Source: JICA Study Team

Figure 4.5.6 Abutment Width

(2) Bridge Seat



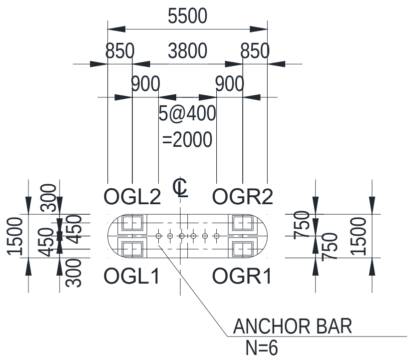
Source: JICA Study Team

Figure 4.5.7 Layout of Bridge Seat for Abutment (A1 and A2)

4.5.3.3 Dimensions of Pier

(1) Bridge Seat

The layout of the bridge seat of Piers PO1 through PO3 is shown in Figure 4.5.8.



Source: JICA Study Team

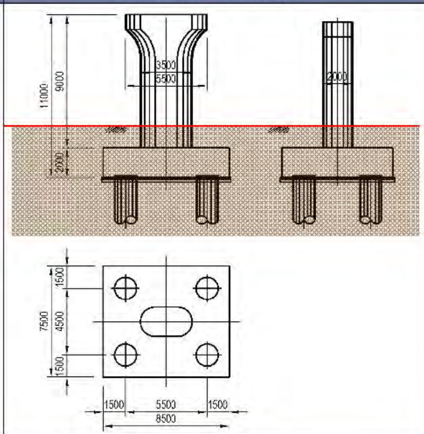
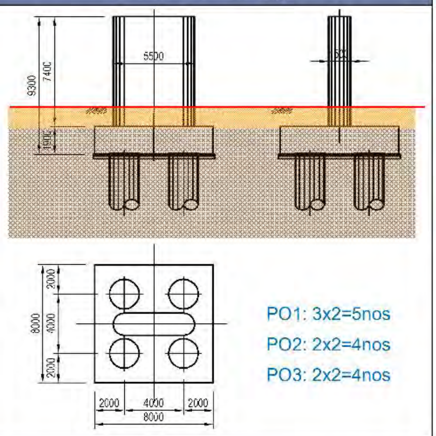
Figure 4.5.8 Layout of Bridge Seat for Piers P01 through P03

(2) Dimensions of Pier Column

The exterior view of the on-ramp substructures was based on the concept of the main bridge substructures. However, the exterior view of the main bridge substructures was revised after the review of the D/D due to the shorter height of the substructures. Consequently, the exterior view of the on-ramp substructures was demanded to be revised from an overhang beam type to a wall type referring to adjacent on-land piers.

These changes resulted in a slight increment of pier column concrete volume, whereas, quantities of reinforcement bar and timber support for the overhang beam became unnecessary. The comparison of the abovementioned general shapes of piers is summarized in Table 4.5.5.

Table 4.5.5 General Shapes of Piers in D/D

Evaluation Item	Basic Design	Detailed Design
Schematic View		
Updates from Basic Design	• **Shortening of pier height:** Due to reclamation soil depth (up to MSL+4.3m) and minimization of design overburden soil depth pier heights are minimized. Piers heights are shortened by 1.7m comparing with BD. • **Deign Soil Parameter:** Soil parameters for a foundation design was updated. DD's soil parameters is weaker than BD so that a strengthening of foundation is made by means of change of pile diameter from 1.5m to 2.0m. • **Optimization of Pile Diameter of CIP Pile:** (BD:φ1.5m, 2x2=4nos, DD: PO1 φ2.0m, 3x2=6nos, PO2&3, 3x2=4nos) Based on the updates of the pier height and the soil parameter, pile diameter of 2.0m is selected.	

Source: JICA Study Team

The cross section of the column was determined based on the stress status of column under various load conditions or minimum dimensions of bridge seat, as summarized in Table 4.5.6.

Table 4.5.6 Summary of Basis of Determination of Cross Sectional Dimensions

Pier Number		Bridge Axis Width	Transverse Direction Width	Overhang Length
Wall Type	PO1 PO2 PO3	**1.5 m** Required width for bridge seat arrangement	**5.5 m** Required width for bridge seat arrangement (5.50 m)	**None** (no overhang beam)

Source: JICA Study Team

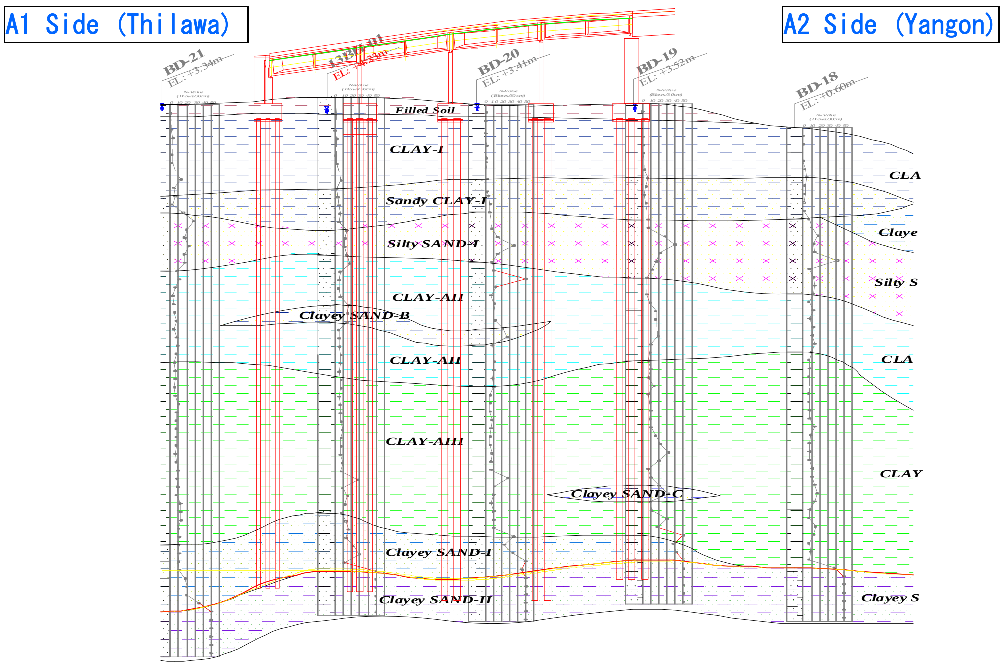
4.5.4 Foundation of On-ramp Bridge

4.5.4.1 Selection of Bearing Stratum and Embedment Length of Foundation

(1) Selection of Bearing Stratum

The basement layer in the bridge design for this bridge site is Clayey Sand-II, which is distributed uniformly at the top surface elevation of around MSL-40.0 ~ -60.0 m. Its firmness is represented by an N-value of 50, which was examined by SPT. There are no appropriate soil layers other than the basement layer with sufficient firmness and thickness to support bridge reactions.

On-ramp Bridge: Clayey Sand-II layer, MSL-50.0~-55.0 m



Source: JICA Study Team

Figure 4.5.9 Prospected Soil Profile and Bearing Stratum

(2) Embedment Length of Foundation

Embedment length of foundation is based on the value recommended in the Specifications for Highway Bridges (Japan Road Associations) as follows:

Cast-In-Situ Pile Foundation: Around 1D or more considering unevenness of bearing stratum

Note: The "D" represent pile diameter.

Table 4.5.7 Summary of Foundation Length (On-ramp)

Item	Mark	Unit	AO1	PO1	PO2	PO3
Station Number	STA	m	0+411.009	0+439.809	0+468.609	0+497.409
EL of Pile cap bottom	FL	m	1.852	1.832	1.791	1.811
EL of Bearing layer	S	m	-52.770	-52.770	-53.590	-53.590
Pile diameter	D	m	1.500	2.000	2.000	2.000
Minimum socket length			1.0D	1.0D	1.0D	1.0D
Foundation Type	-	-	CIP Pile	CIP Pile	CIP Pile	CIP Pile
Pile Length	L	m	56.500	57.000	57.500	58.000
Reference Boring No.	-	-	*BH-01*	*BH-01*	*BD20*	*BD20*
Bearing Stratum	-	-	*CS-II*	*CS-II*	*CS-II*	*CS-II*

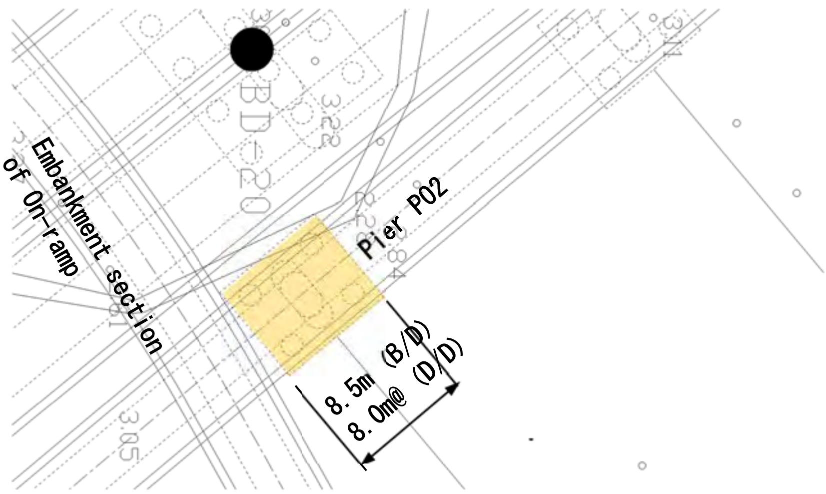
Source: JICA Study Team

4.5.4.2 Selection of Foundation Type

It was confirmed that the "CIP pile foundation (reverse circulation drilling method) D = 2.0 m" was the most economical foundation type for on-land piers. For abutment foundation type, it was confirmed that the "CIP pile foundation (reverse circulation drilling method) D = 1.5 m" was the most preferable foundation type as it was selected in the B/D. It should be remarked that the comparison was made taking into account the available sizes of pile cap as summarized in Table 4.5.8. Result of this review is shown in Table 4.5.9 and Table 4.5.10.

Piers: CIP pile foundation (reverse circulation drilling method) D =2.0 m

Abutments: CIP pile foundation (reverse circulation drilling method) D =1.5 m



Source: JICA Study Team

Figure 4.5.10 Crossing Point of Embankment Road of On-ramp and PO2 Pier

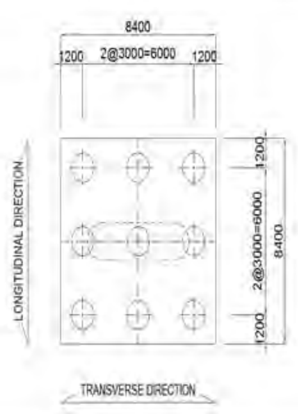
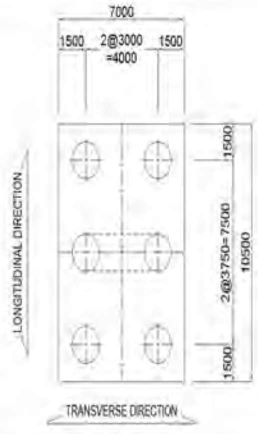
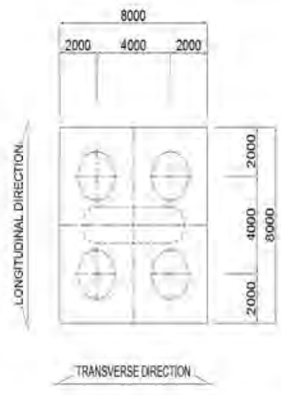
Table 4.5.8 Available Pile Cap Size and Costs of On-ramp Pier

Pile Diameter	Pile Cap Size	
	Pile spanning 2.5D	Pile spanning 2.0D
φ1.0	*9.5 m* x 7.0 m (overlap)	8.0 m x 7.0 m (lack of stability)
φ1.2	**8.4 m x 8.4 m** **(1.264)**	7.2 m x 7.2 m (lack of stability)
φ1.5	*10.5 m* x 7.0 m (overlap)	*9.0 m* x 7.0 m (lack of stability)
φ2.0	*9.0 m x 9.0 m* (overlap)	**8.0 m x 8.0 m** **(1.000)**

Top Row: Size of pile cap Bottom Row: Cost Ratio

Source: JICA Study Team

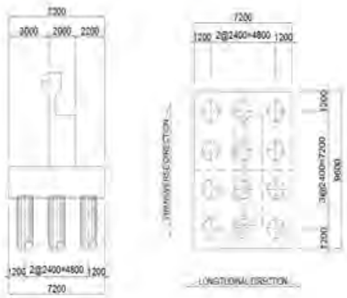
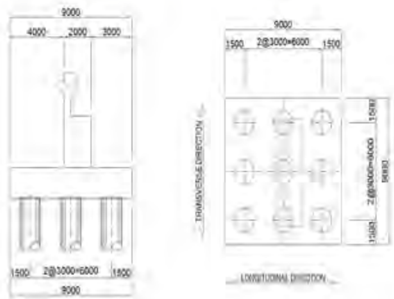
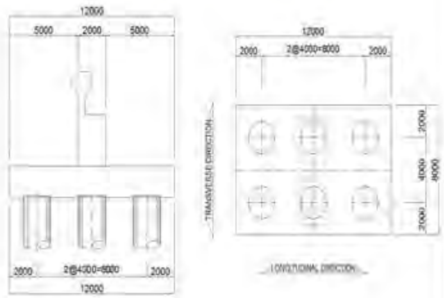
Table 4.5.9 Review of Foundation Type for On-ramp Piers in the D/D

Pile Diameter				Cast in Place RC Piles φ1.2m				Cast in Place RC Piles φ1.5m				Cast in Place RC Piles φ2.0m			
Outline Drawing															
Design Results	item	mark	unit	Longitudinal Direction		Transverse Direction		Longitudinal Direction		Transverse Direction		Longitudinal Direction		Transverse Direction	
				Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation
	Maximum Pile Reactions	Pmax	kN	928.8	2,507.3	928.8	2,300.7	1,494.0	3,221.7	1,494.0	3,177.3	2,174.6	4,935.9	2,174.6	4,544.2
		Ra	kN	3,562.0	5,563.0	3,562.0	5,563.0	4,521.0	7,126.0	4,521.0	7,126.0	6,253.0	9,992.0	6,253.0	9,992.0
		σ/σa	-	0.26	0.45	0.26	0.41	0.33	0.45	0.33	0.45	0.35	0.49	0.35	0.45
	Amount of Displacement	σx	mm	0.0	14.0	0.0	11.8	0.0	14.0	0.0	14.6	0.0	19.5	0.0	16.7
		σxa	mm	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	20.0	20.0	20.0	20.0
		R	-	0.00	0.93	0.00	0.79	0.00	0.93	0.00	0.97	0.00	0.98	0.00	0.83
	Stress of a Pile	σs	N/mm^2	-10.6	297.7	-10.6	242.0	-10.7	247.4	-10.7	186.1	-9.2	265.8	-9.2	214.6
		σsa	N/mm^2	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0
		σ/σa	-	0.05	0.99	0.05	0.81	0.05	0.82	0.05	0.62	0.05	0.89	0.05	0.72
	Maximum Stress of a Pile			σs= 298 kN/m^2 <σsa= 300kN/m^2 (OK)				σs= 247 kN/m2<σsa= 300kN/m2 (OK)				σs= 266 kN/m2<σsa= 300kN/m2 (OK)			
Constructibility				The amount of number of pile is largest and thus this alternative is the most inferior one in terms of constructability.			△	This alternative entails the smallest amount of pile works.			○	This alternative entails smaller amount of pile works.			◎
Construction Period				The amount of pile works including ground excavation is consirably smaller.			◎	The amount of pile works including ground excavation is consirably the smallest.			◎	The amount of pile works including ground excavation is consirably large.			◎
Environmental Aspect				This alternative entails the smallest amount of excavation works.			◎	This alternative entails small amount of excavation works.			◎	This alternative entails the largest amount of excavation works.			○
Cost Ratio				1.216			△	1.001			○	1.000			◎
Judge								Unselect (Overlap with embankment section of On-ramp)				◎			

Note ◎:Good, ○:Fair, △:Not Recommended

Source: JICA Study Team

Table 4.5.10 Review of Foundation Type for On-ramp Abutment in the D/D

Pile Diameter				Cast in Place RC Piles φ1.2m		Cast in Place RC Piles φ1.5m		Cast in Place RC Piles φ2.0m	
Outline Drawing									
	item	mark	unit	Bridge's Longitudinal Direction		Bridge's Longitudinal Direction		Bridge's Longitudinal Direction	
				Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation
Design Results	Maximum Pile Reactions	Pmax	kN	1,211.9	1,859.7	1,613.2	2,502.0	2,512.6	3,604.1
		Ra	kN	3,403.0	5,320.0	4,476.0	7,054.0	6,323.0	10,091.0
		σ/σa	-	0.36	0.35	0.36	0.35	0.40	0.36
	Amount of Displacement	σx	mm	7.5	14.1	6.7	14.4	5.4	14.2
		σxa	mm	15.0	15.0	15.0	15.0	15.0	15.0
		R	-	0.50	0.94	0.45	0.96	0.36	0.95
	Stress of a Pile	σs	N/mm²	70.3	260.4	54.3	255.9	48.2	264.0
		σsa	N/mm²	160.0	300.0	160.0	300.0	160.0	300.0
		σ/σa	-	0.44	0.87	0.34	0.85	0.30	0.88
	Maximum Stress of a Pile			σs= 260 kN/m² <σsa = 300 kN/m² (OK)		σs= 256 kN/m² <σsa = 300 kN/m² (OK)		σs= 264 kN/m² <σsa = 300 kN/m² (OK)	
Constructability				The amount of number of pile is largest and thus this alternative is the most inferior one in terms of constructability.	△	This alternative entails the smallest amount of pile works.	○	This alternative entails smallest amount of pile works.	◎
Construction Period				The amount of pile works including ground excavation is almost same as other options	◎	The amount of pile works including ground excavation is almost same as other options	◎	The amount of pile works including ground excavation is almost same as other options	◎
Environmental Aspect				This alternative entails the smallest amount of excavation works.	◎	This alternative entails small amount of excavation works.	○	This alternative entails the largest amount of excavation works.	△
Cost Ratio				1.117	△	1.000	◎	1.048	○
Overall Evaluation						◎			

Note ◎:Good, ○:Fair, △:Not Recommended

Source: JICA Study Team

4.5.5 Summary of Detailed Design Results for Substructure and Foundations

4.5.5.1 Computation of Columns of T-shaped Piers

The columns of T-shaped piers can be designed as cantilevers with fixed ends at the section connected to the footings. In the design process, the most adverse combination of axial forces and bending moments shall be applied.

The overhang beams of T-shaped piers can be designed as follows:

- The overhang beams are designed as cantilevers.
- The overhang length of the cantilever is defined as the length from the vertical section at the front surface of the column to the beam in case of rectangular column, and from the position one tenth of the column diameter inward from the front of the column to the beam end in case of an oval section column.

The calculation results for columns of T-shaped piers are shown in Table 4.5.11.

4.5.5.2 Computation of Reverse T-shaped Abutment

The wall of reverse T-shaped abutment can be designed as cantilever with fixed ends at the section connected to the footings.

The parapet shall be designed to carry earth pressure as well as vehicle load (T-loads) and the loads from the approach slab.

The wing wall shall be designed as slabs to receive superimposed loads due to live loads and earth pressure. The slab in this case shall be cantilever fixed on two sides to a wall and footing.

The calculation results for the wall of the reverse T-shaped abutment are shown in Table 4.5.11.

4.5.5.3 Computation of Footings

Footing shall be designed in consideration of the most adverse load combination of self-weight, overburden load such as soils, presence of buoyancy, subgrade reaction, and reaction from foundations. Footings may be designed as beam members and as cantilevers.

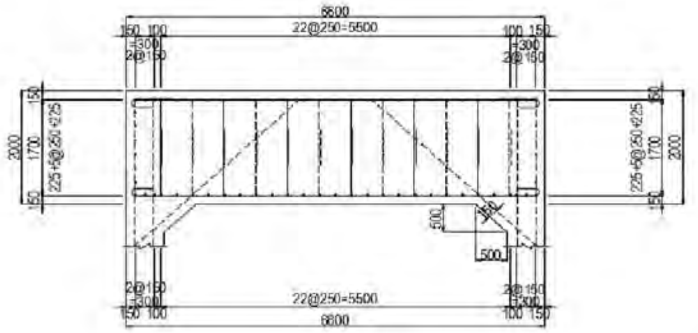
The footings shall retain a thickness necessary to serve as structural members. Also, the footings shall have sufficient thickness to be regarded as rigid bodies, when they are assumed as rigid bodies in the stability analysis of the foundation.

The calculation results for the footing of piers are shown in Table 4.5.12.

Table 4.5.11 Calculation Results for Wall and Columns (AO1, PO1~PO3)

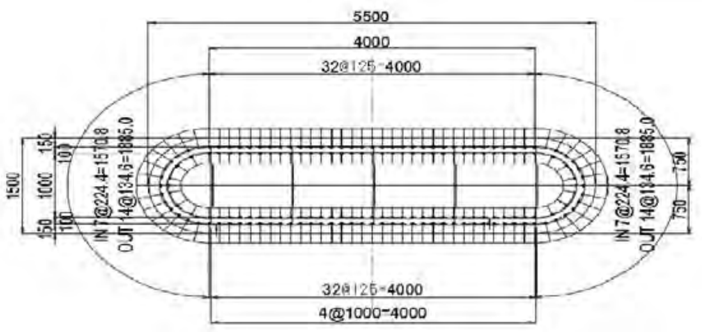
Cross Section of Column (Longitudinal Direction)

AO1



D22@250

PO1~PO3



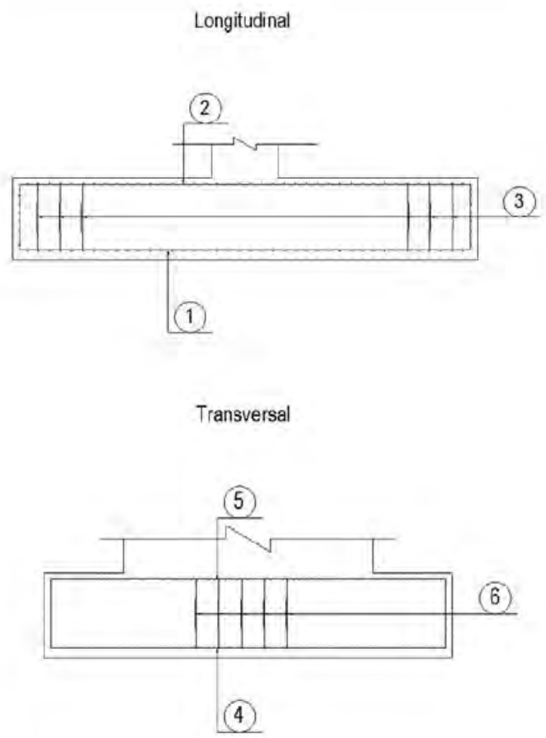
1.5-D32@125 (PO1)
D25@125 (PO2)
D32@125 (PO3)

			AO1		PO1		PO2		PO3	
			LL	TT	LL	TT	LL	TT	LL	TT
Material		concrete	24N/mm2		24N/mm2		24N/mm2		24N/mm2	
		reinforcement	SD345		SD345		SD345		SD345	
Check for Bending Moment	Ordinary	σs (N/mm2)	13.34	-	51.66	-	-6.56	-	94.87	-
		σsa(N/mm2)	184.00	-	184.00	-	-200.00	-	184.00	-
		R-ratio	0.07	-	0.28	-	0.03	-	0.52	-
	Seismic	σs (N/mm2)	91.47	-	244.12	16.79	260.62	44.67	232.05	23.49
		σsa(N/mm2)	300.00	-	300.00	300.00	300.00	300.00	300.00	300.00
		R-ratio	0.30	-	0.81	0.06	0.87	0.15	0.77	0.08
Check for Shear	Ordinary	τm(N/mm2)	0.075	-	0.162	-	0.033	-	0.127	-
		τa(N/mm2)	0.134	-	0.349	-	0.260	-	0.305	-
		R-ratio	0.56	-	0.46	-	0.13	-	0.42	-
	Seismic	τm(N/mm2)	0.154	-	0.522	0.207	0.248	0.218	0.265	0.181
		τa(N/mm2)	0.204	-	0.462 (2.550)	0.279	0.344	0.210 (2.550)	0.404	0.246
		R-ratio	0.75	-	1.13 (0.20)	0.74	0.72	1.04 (0.09)	0.66	0.74

Note :
σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.5.12 Calculation Results for Footing of Piers(PO1~PO3)

	PIER
Cross Section of Pile Cap (Longitudinal & Transversal Direction) SD345	

			PO1		PO2		PO3	
			LL	TT	LL	TT	LL	TT
Arangement of reinforcement	①	④	D32@125	D29@250	D29@250	D19@250	D25@125	D22@250
	②	⑤	D25@125	D22@250	D22@250	D19@250	D25@250	D19@250
	③	⑥	D16@500	D16@500	D16@500	D16@500	D16@500	D16@500
Check for Bending Stress	Ordinary	σs (N/mm2)	131.97	124.07	132.88	69.67	150.20	69.58
		σsa(N/mm2)	184.00	160.00	160.00	160.00	184.00	160.00
		R-ratio	0.72	0.78	0.83	0.44	0.82	0.43
	Seismic	σs (N/mm2)	258.47	124.07	289.50	69.67	214.81	69.58
		σsa(N/mm2)	300.00	300.00	300.00	300.00	300.00	300.00
		R-ratio	0.86	0.41	0.97	0.23	0.72	0.23
Check for Shear Stress	Ordinary	τm (N/mm2)	0.366	—	0.339	—	0.441	—
		τa (N/mm2)	0.656	—	0.880	—	1.009	—
		R-ratio	0.56	—	0.39	—	0.44	—
	Seismic	τm (N/mm2)	0.650	—	0.649	—	0.703	—
		τa (N/mm2)	0.868	—	1.339	—	1.536	—
		R-ratio	0.75	—	0.48	—	0.46	—

σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

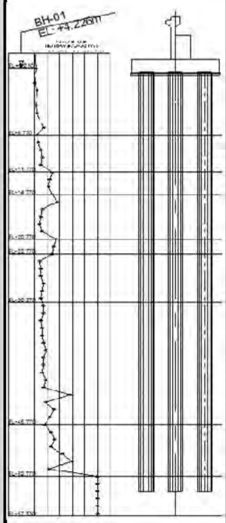
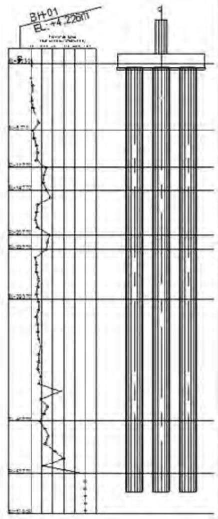
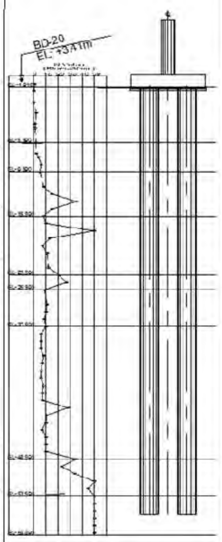
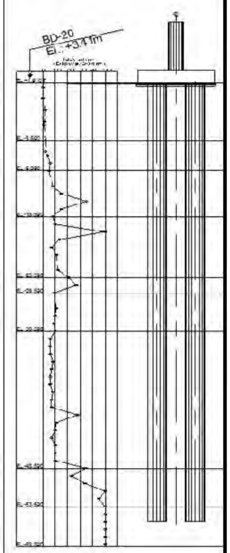
4.5.5.4 Design of Foundation

The pile foundation and the SPSP foundation shall conform to the following requirements under ordinary, earthquake, and vessel collision conditions.

- The axial reaction at each pile head shall not exceed the allowable pile bearing capacity. The axial allowable bearing capacity can be estimated by dividing the ultimate bearing capacity of the pile, determined from related factors such as ground conditions and construction methods, by the factor of safety.
- The displacements at each pile head shall not exceed the allowable displacements in order not to leave a large residual displacement and to keep within the limit of possibility of evaluation of elastic behavior. The allowable horizontal displacement is principally determined to be 1% based on the results of many loading tests.
- For a large elastic foundation with a width of 5 m or more such as SPSP foundation, the allowable displacement is determined to be 50 mm because few loading tests data are available.
- For a pile foundation with a pile diameter of 1.5 m or less, the allowable displacement is 15 mm. For a pile foundation with a pile diameter of 2.0 m, the allowable displacement is 20 mm.
- The allowable displacement of abutment foundation is 15 mm regardless of the foundation width because the displacement may increase with time due to the effects of creep and backfill settlement.
- The axial reaction at each pile head shall not exceed the allowable pile bearing capacity.
- The stresses generated in the members of pile foundations shall not exceed the allowable stresses specified in the relevant section of this report.

The calculation results of the CIP pile foundation stability and cross sectional stress are shown in Table 4.5.13 and Table 4.5.14, respectively.

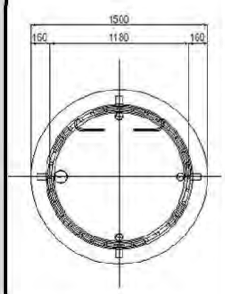
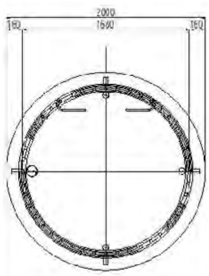
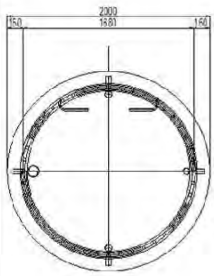
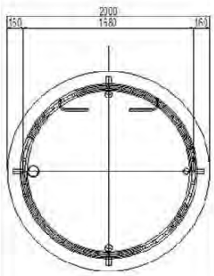
Table 4.5.13 Calculation Results of CIP Pile Foundation Stability(AO1~PO3)

	AO1	PO1	PO2	PO3
Boling Log & Pile Length (m)				
Pile Information				
Diameter of Pile (mm)	1,500	2,000	2,000	2,000
Number of Piles (Nos.)	9	5	4	4
Pile Length (m)	56.4	57.0	57.5	58.0
Bearing Resistance of Ordinary				
Pile Head Reaction (kN)	1,546	3,220	2,805	3,864
Bearing Capacity (kN)	4,476	6,361	6,385	6,550
R-Ratio	0.345	0.506	0.439	0.590
Horizontal Movement of Ordinary				
Horizontal Movement (mm)	6.4	6.9	2.1	8.6
Capacity (mm)	15.0	20.0	20.0	20.0
R-Ratio	0.427	0.347	0.107	0.432
Bearing Resistance of Seismic				
Pile Head Reaction (kN)	2,512	5,330	4,733	5,088
Bearing Capacity (kN)	7,054	10,149	10,193	10,445
R-Ratio	0.356	0.525	0.464	0.487
Horizontal Movement of Seismic				
Horizontal Movement (mm)	14.2	19.8	16.4	17.7
Capacity (mm)	15.0	20.0	20.0	20.0
R-Ratio	0.949	0.992	0.818	0.887
Bearing Capacity of Group Piles of Ordinary				
Axial Compression Fordes (kN)	12,144	11,248	9,993	10,322
Bearing Capacity (kN)	310,198	330,692	205,404	207,765
R-Ratio	0.039	0.034	0.049	0.050
Judgement of Lateral Movement				
Identifying Index	3.569	—	—	—
Capacity	1.200	—	—	—

* We also conducted a study on the negative skin friction force, but another case became severe in all foundations, so the value of bearing resistance is ordinary case.

Source: JICA Study Team

Table 4.5.14 Calculation Results of Cross Section of CIP Pile Foundation(AO1~PO3)

	AO1	PO1	PO2	PO3
Cross Section of Pile SD345	 32-D29@115.84 AS=205.568cm^2	 44-D32@120 AS=349.448cm^2	 44-D29@124 AS=282.656cm^2	 44-D29@124 AS=282.656cm^2
Check for Bending Stress				
Ordinary				
σs (N/mm2)	57.60	39.39	—	50.64
σsa (N/mm2)	184.00	184.00	—	184.00
R-ratio	0.31	0.21	—	0.28
Seismic				
σs (N/mm2)	251.96	268.69	203.16	227.68
σsa (N/mm2)	300.00	300.00	300.00	300.00
R-ratio	0.84	0.90	0.68	0.76
Check for Shear Stress				
Ordinary				
τm (N/mm2)	0.105	0.076	0.020	0.076
τa (N/mm2)	0.412	0.349	0.566	0.379
R-ratio	0.25	0.22	0.04	0.20
Seismic				
τm(N/mm2)	0.332	0.336	0.227	0.238
τa (N/mm2)	0.438	0.399	0.375	0.375
R-ratio	0.76	0.84	0.61	0.63

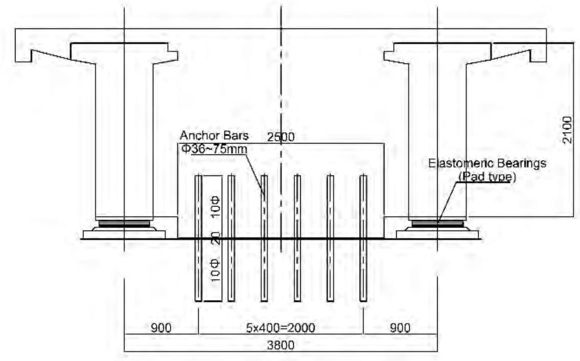
σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force

Source: JICA Study Team

4.5.6 Bridge Accessories

4.5.6.1 Bearings

In the on-ramp bridge, the superstructure is planned to be longitudinally fixed at intermediate supports, and movable at the end supports. The superstructure is transversally fixed, considering the connection with the approach bridge. Rubber pads are adopted for bridge bearings, and anchor bars are planned to be installed on top of the substructures for fixing.



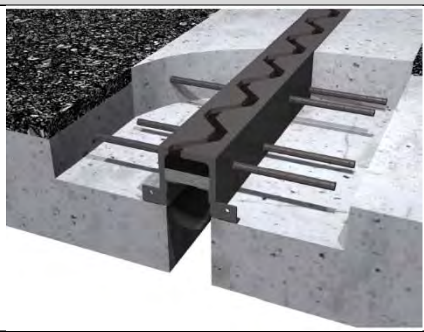
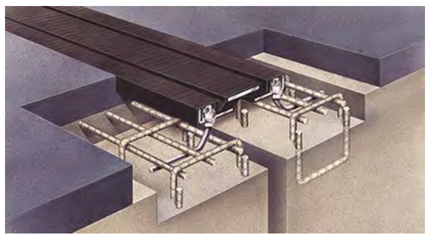
Source: JICA Study Team

Figure 4.5.11 Arrangement of Bearings and Anchor Bars of On-ramp Bridge

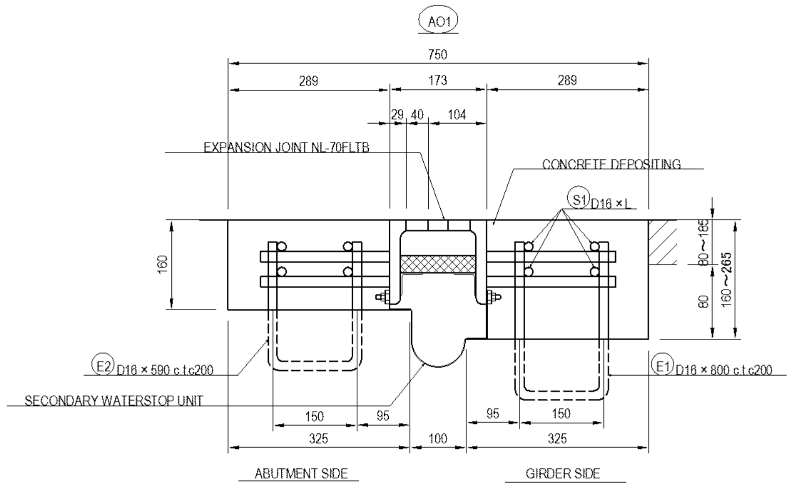
4.5.6.2 Expansion Joints

Steel joint has been selected as the type of expansion joint for the on-ramp bridge, considering better durability and maintenance.

Table 4.5.15 Comparison of Expansion Joint Type for On-ramp Bridge

	Steel Joint	Rubber Joint
Schematic View		
Functional performance	• High stiffness of steel component • High durability of steel components. • Moderate driving comfort	• Larger deflection due to rubber components • Deterioration by UV rays. • Better driving comfort due to rubber surface
Construction	• Easy installation • Light weight	• Easy installation • Light weight
Maintenance	• The components can be partially replaced • Long service life	• Relatively difficult to replace the components. • Slightly shorter service life
Evaluation	**RECOMMENDED**	

Source: JICA Study Team



Source: JICA Study Team

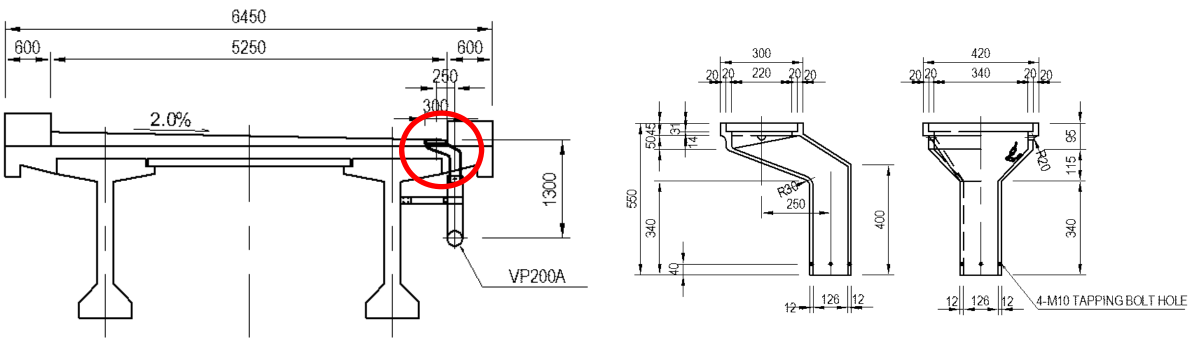
Figure 4.5.12 Expansion Joint of On-ramp Bridge (at AO1)

4.5.6.3 Bridge Railing

Steel railings have been adopted as the bridge railings, uniformly with the main bridge. Class of railing is Class A in "Specifications for Highway Railings" by Japan Road Association. The height of railing is 1.1 m, considering the conformity with the main bridge.

4.5.6.4 Drainage System

Rainwater on the bridge surface is drained by catch pits installed at the shoulder of the bridge deck. As the on-ramp bridge is located on land, the rainwater from the catch pits is horizontally led to the substructures, and then vertically drained to the catch basin on the ground, which is connected to the side ditch.



Source: JICA Study Team

Figure 4.5.13 Catch Pits Arrangement and Detail (On-ramp Bridge)

4.6 STUDY ON FLYOVER BRIDGE

4.6.1 Study on Flyover Bridge

4.6.1.1 Decision of Length of North Approach Road and Flyover Bridge

The original bridge plan in the Supplemental F/S had been reviewed prior to the commencement of the B/D, based on the updated design condition and soil investigation survey.

The summary of the review results is given in Table 4.6.1 and each review result is explained in the following sections.

Table 4.6.1 Summary of Review Result

Review Item	Original Plan in Supplemental F/S	Revised Plan in D/D	Reference
Flyover Length	L = 547 m	L = 602 m	4.6.1.1
Span Arrangement	34 + (40 + 60 + 33) + (7@30 m) + (33 + 64 + 40) + 33	2@30 m + (55 + 70 + 55) + 6@30 m+ 35 + 52 + 35 +2@30	4.6.1.2
Superstructure Type	1) Standard Section PC-I Girder (Max. span length = 34 m) 2) Special Sec. at Shukinthar Myopat I/S Steel-I Girder (Max. span length = 60 m) 3) Special Sec. at Yadanar I/S Steel-I Girder (Max. span length = 64 m)	1) Standard Section PC-I Girder (Max. span length = 30 m) 2) Special Sec. at Shukinthar Myopat I/S Steel Box Girder (Max. span length = 70 m) 3) Special Sec. at Yadanar I/S Steel-I Girder (Max. span length = 52 m)	4.6.1.3
Foundation Type	Cast-in-place RC Pile (D = 1200)	Cast-in-place RC Pile (D = 1500)	4.6.1.4

Source: JICA Study Team

4.6.1.2 Flyover Length

(1) Introduction

In the Supplemental F/S, the flyover length was determined by the generally applicable abutment height on soft soil ground without the technical comparative study since the available existing information was limited. Therefore, in this study, the optimum flyover length was reviewed/re-examined in terms of economical aspect through the following comparative study, taking into account the additional soil investigation and the updated design condition. The alternatives are given below.

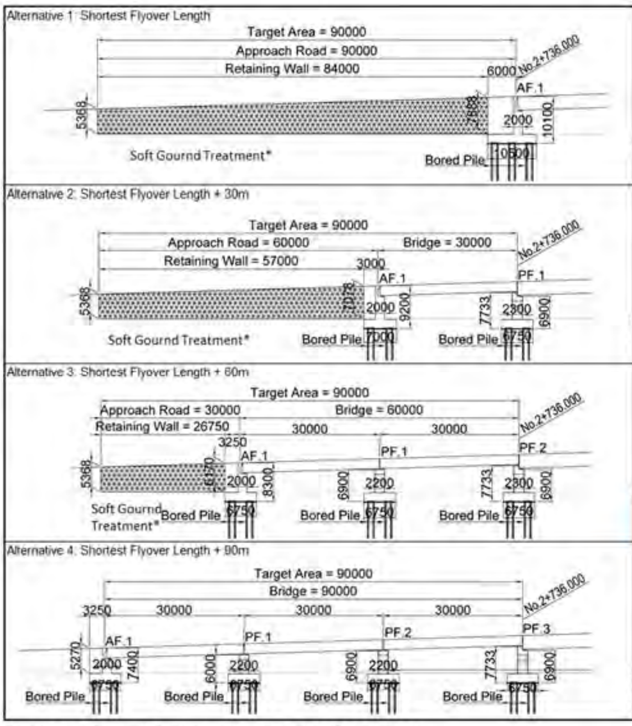
- Alternative-1 : Shortest Flyover Length / L = 542 m (Nearly the original flyover length of 542 m in the F/S)
- Alternative-2 : Shortest Flyover Length + 30 m
- Alternative-3 : Shortest Flyover Length + 60 m
- Alternative-4 : Shortest Flyover Length + 90 m

(2) Review Result

As a result of the comparative study given in Table 4.6.2, “Alternative-3: Shortest Flyover Length + 60 m” was revealed to be the most economical option. The flyover length is 602 m.

Table 4.6.2 Comparative Study on Location of Abutment 1 and Abutment 2

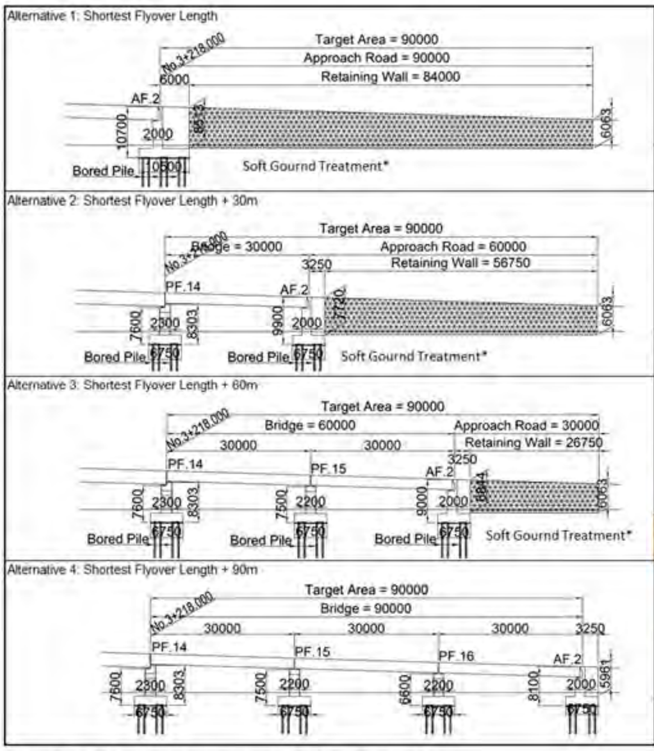
<u>Abutment 1</u>



Alternative			
Alternative 1	Quantities	Superstructure	0m
		Substructure Foundation	1 Number
		Approach Road	90m
	Construction Cost	Ratio	1.06
Alternative 2	Quantities	Superstructure	30m
		Substructure Foundation	2 Numbers
		Approach Road	60m
	Construction Cost	Ratio	1.02
Alternative 3	Quantities	Superstructure	60m
		Substructure Foundation	3 Numbers
		Approach Road	30m
	Construction Cost	Ratio	1.00
Alternative 4	Quantities	Superstructure	90m
		Substructure Foundation	4 Numbers
		Approach Road	0m
	Construction Cost	Ratio	1.03

*Depth for Soft Gournd Treatment is assumed to be 16.5m from G.L.

<u>Abutment 2</u>



Alternative			
Alternative 1	Quantities	Superstructure	0m
		Substructure Foundation	1 Number
		Approach Road	90m
	Construction Cost	Ratio	1.11
Alternative 2	Quantities	Superstructure	30m
		Substructure Foundation	2 Numbers
		Approach Road	60m
	Construction Cost	Ratio	1.05
Alternative 3	Quantities	Superstructure	60m
		Substructure Foundation	3 Numbers
		Approach Road	30m
	Construction Cost	Ratio	1.00
Alternative 4	Quantities	Superstructure	90m
		Substructure Foundation	4 Numbers
		Approach Road	0m
	Construction Cost	Ratio	1.02

*Depth for Soft Gournd Treatment is assumed to be 13.0m from G.L.

Source: JICA Study Team

4.6.1.3 Span Arrangement for Flyover

(1) Introduction

A flyover can be divided into two sections; one is the special section at intersections and the other is the standard section between/outside of the intersections as shown in Figure 4.6.1. In accordance with the revised flyover length, span arrangement was re-examined in consideration of the following points:

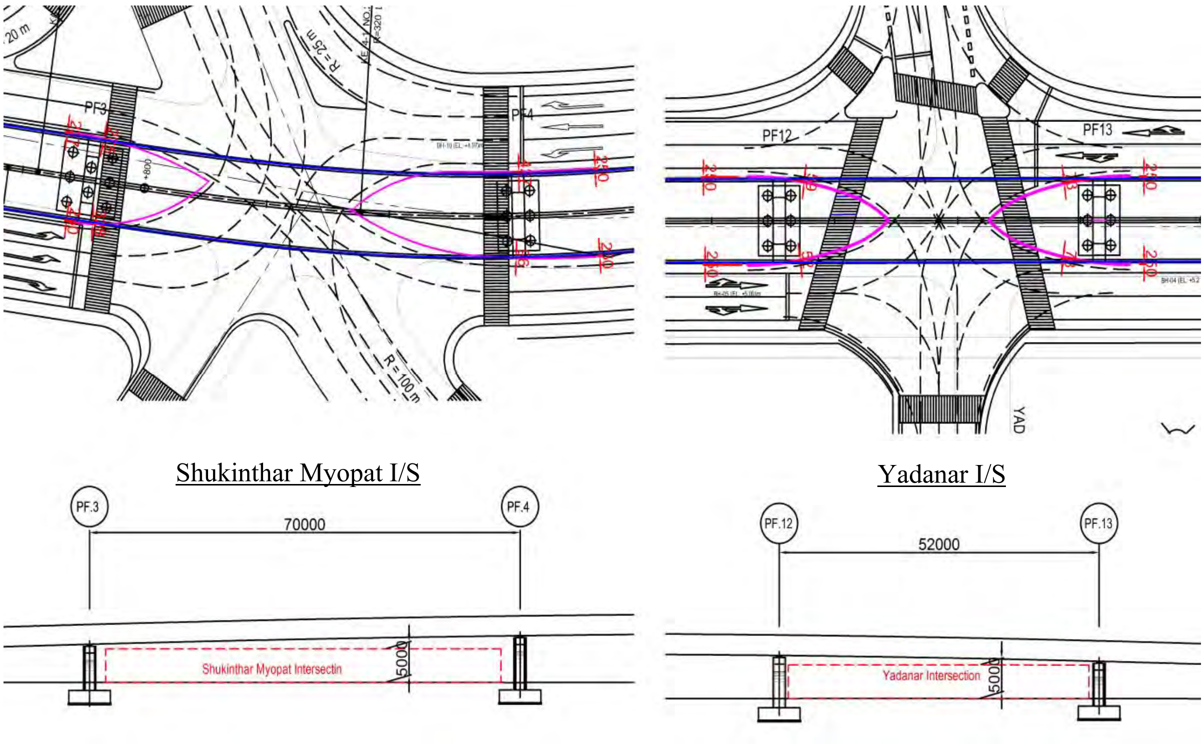
1) Required Minimum Span Length for Special Section at the Intersections

Construction gauge (5.0 m) should be secured under the flyover, and the pier should be located outside of the intersection (crosswalk) for road safety in order that pedestrians can be recognized by drivers in the intersection. Accordingly, pier location/minimum span length is controlled by the construction gauge (5.0 m) and/or location of the crosswalk. The required minimum span length for each intersection is shown in Table 4.6.3.

Table 4.6.3 Required Minimum Span Length at Intersection

Location	Required Min. Span Length	Remark
Shukinthar I/S	70 m	Pier location is controlled by crosswalk as shown in Figure 4.6.1
Yadanar I/S	52 m	Pier location is controlled by construction gauge as shown in Figure 4.6.1

Source: JICA Study Team



Outer line of pier head Inner shoulder line Construction gauge

Source: JICA Study Team

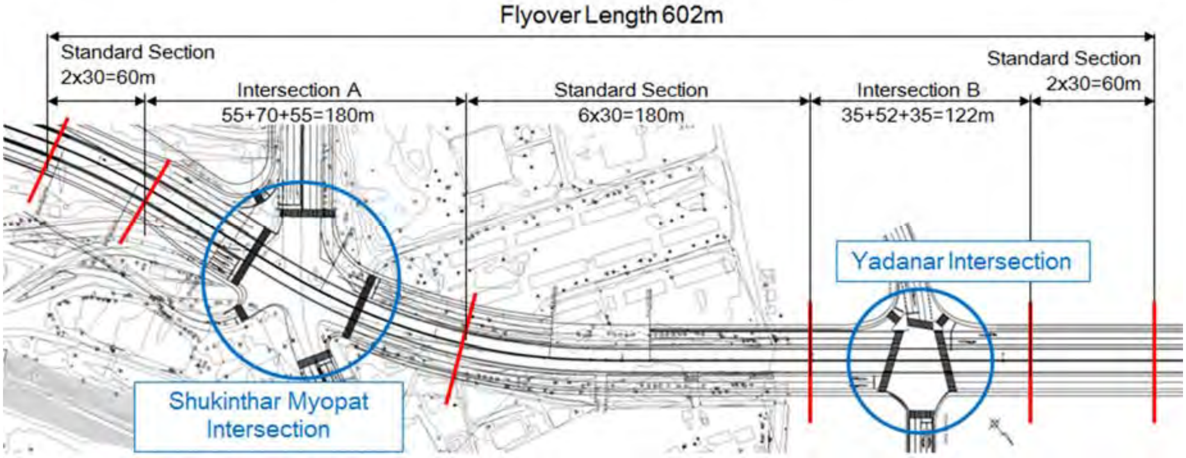
Figure 4.6.1 Required Minimum Span Length at Shukinthar Myopat I/S and Yadanar I/S

2) Economical Span Arrangement

For the special section at both intersections, the side span length can be determined by the economical span ratio between the side span length and the center span length (0.7 to 0.8:1.0). The standard section is basically divided into a 30 m span.

(2) Span Arrangement of Flyover

As a result of the review, the below span arrangement is applied to the flyover.



Source: JICA Study Team

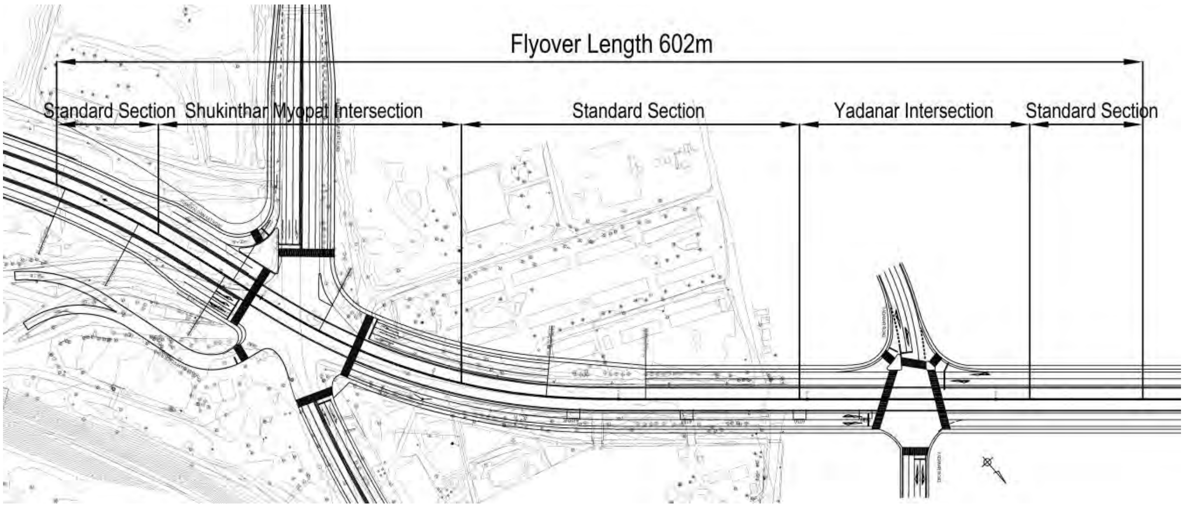
Figure 4.6.2 Span Arrangement of Flyover

4.6.1.4 Superstructure Type for Flyover

(1) Introduction

In accordance with the revised span arrangement in the previous section, the original superstructure type was reviewed through a comparative study. The comparative study was conducted for 1) Standard section, 2) Special section at Shukinthar Myopat I/S, and 3) Special section at Yadanar I/S. The items below are taken into account for the evaluation.

- Workability and Quality Control at the Site
- Structural Aspect
- Construction Cost
- Construction Period
- Maintenance Aspect



Source: JICA Study Team

Figure 4.6.3 Location of Shukinthar Myopat I/S, Yadanar I/S and Standard Section in Flyover

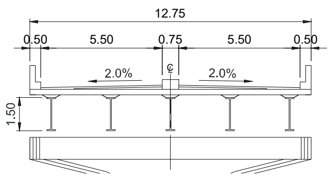
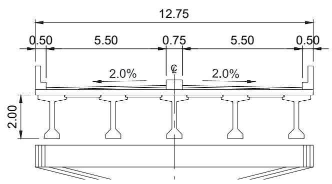
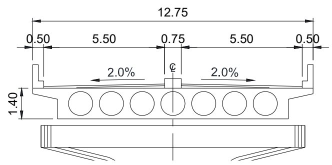
(2) Superstructure Type for Standard Section

In consideration of the applied maximum span length (30 m) in this section, the alternatives below are extracted for the comparison.

- Alternative-1 Steel-I Girder
- Alternative-2 PC-I Girder (Original plan in the Supplemental F/S)
- Alternative-3 PC Hollow Slab

As a result of the comparative study, in terms of the economical aspect, "Alternative-2 PC-I Girder" is the optimal superstructure type in the standard section as shown in Table 4.6.4.

Table 4.6.4 Comparative Study of Superstructure Type for Standard Section

Evaluation Item	Alt-1 Steel-I Girder		Alt-2 PC-I Girder (Plan at F/S)		Alt-3 PC Hollow Slab	
Schematic View						
Erection Method	Crane Erection Method		Crane Erection Method		All Staging Method	
Workability & Quality Control	- Girder blocks are prefabricated in factory so that quality control can be easier. - Field work can be simplified.	◎	- Girders are pre-casted at the construction yard so that quality control can be easier. - Field work can be simplified	◎	- Cast-in-situ method is inferior in quality control of girders. - Field work is not simple.	△
Structural Aspect	- Applicable span length : 30-60 m - Light weight	◎	- Applicable span length : 20-40 m - Moderate weight	○	- Applicable span length : 20-30 m - Heavy weight	△
Construction Cost	Ratio = 1.18	△	Ratio = 1.00	◎	Ratio = 1.05	○
Construction Period	5 months	◎	7 months	○	11 months	△
Maintenance Aspect	- Re-painting is necessary in addition to replacement of bearing and expansion joints.	△	- Replacement of bearings and expansion joints is necessary.	◎	- Replacement of bearings and expansion joints is necessary.	◎
Evaluation	Less Recommended		**Most Recommended**		Less Recommended	

Legend : ◎ Very Good, ○ Good, △ Moderate × Not Good

Source: JICA Study Team

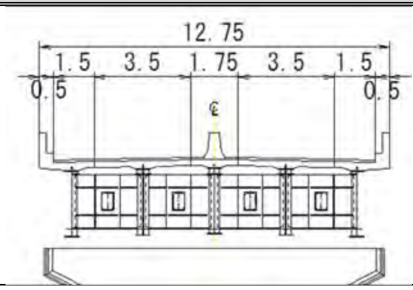
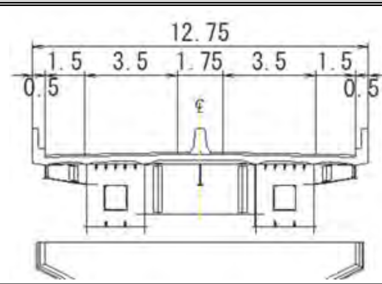
(3) Superstructure Type for Special Section at Shukinthar Myopat I/S

In consideration of the applied maximum span length (70 m) in this section, the alternatives below are extracted for comparison[1].

- Alternative-1 Steel-I Girder (Original plan in the Supplemental F/S)
- Alternative-2 Steel Box Girder

As a result of the comparative study, in terms of the economical aspect, construction schedule and structural aspect, "Alternative-2 Steel Box Girder" is the optimal superstructure type in the special section at Shukinthar MyoPat Intersection as shown in Table 4.6.5.

Table 4.6.5 Comparative Study of Superstructure Type for Special Section at Shukinthar Myopat I/S

Evaluation Item	Alt-1 Steel-I Girder (Plan at F/S)		Alt-2 Steel Box Girder	
Schematic View				
Erection Method	Crane Erection Method		Crane Erection Method	
Workability & Quality Control	- Girder blocks are fabricated in a factory so that quality control can be easier. - Field work can be simplified.	◎	- Girder blocks are fabricated in a factory so that quality control can be easier. - Field work can be simplified.	◎
Structural Aspect	- Applicable span length : 30-60 m - Torsional stiffness is secured by additional lateral bracing for small radius curve section - Heavy weight (956 t)	△	- Applicable span length : 40-80 m - Appropriate bridge type for the section where small curve radius is applied - Light weight (707 t)	◎
Construction Cost	Ratio = 1.16	△	Ratio = 1.00	◎
Construction Period	17 months	○	15 months	◎
Maintenance Aspect	- Re-painting is necessary in addition to replacement of bearing and expansion joints.	○	- Re-painting is necessary in addition to replacement of bearing and expansion joints.	○
Evaluation	Less Recommended		**Most Recommended**	

Legend : ◎ Very Good, ○ Good, △ Moderate × Not Good

Source: JICA Study Team

[1] PC (box) girder is excluded from the above alternatives since its heavy weight is a disadvantage in the erection at the intersection and economical aspect (pile numbers will be increased due to heavy weight).

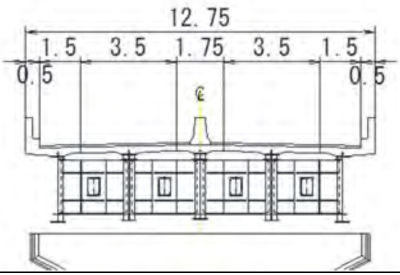
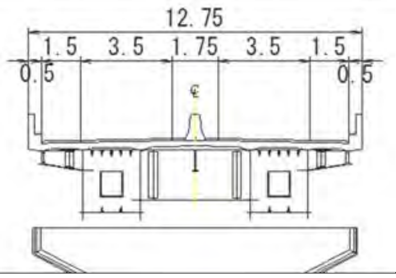
(4) Superstructure Type for Special Section at Yadanar I/S

In consideration of the applied maximum span length (52 m) in this section, the alternatives below are extracted for comparison[2].

- Alternative-1 Steel-I Girder (Original plan at Supplemental F/S)
- Alternative-2 Steel Box Girder

As a result of the comparative study, in terms of economical aspect, construction schedule, and structural aspect, "Alternative-1 Steel-I Girder" is the optimal superstructure type in the special section at Yadanar Intersection as shown in Table 4.6.6.

Table 4.6.6 Comparative Study of Superstructure Type for Special Section at Yadanar I/S

Evaluation Item	Alt-1 Steel-I Girder (Plan at F/S)		Alt-2 Steel Box Girder	
Schematic View				
Erection Method	Crane Erection Method		Crane Erection Method	
Workability & Quality Control	- Girder blocks are fabricated in a factory so that quality control can be easier. - Field work can be simplified.	◎	- Girder blocks are fabricated in a factory so that quality control can be easier. - Field work can be simplified.	◎
Structural Aspect	- Applicable span length : 30-60 m - Light weight (339 t)	◎	- Applicable span length : 40-80 m - Heavy weight (364 t)	○
Construction Cost	Ratio = 1.00	◎	Ratio = 1.19	△
Construction Period	9 months	◎	9 months	◎
Maintenance Aspect	- Re-painting is necessary in addition to replacement of bearing and expansion joints.	○	- Re-painting is necessary in addition to replacement of bearing and expansion joints.	○
Evaluation	**Most Recommended**		Less Recommended	

Legend : ◎ Very Good, ○ Good, △ Moderate × Not Good

Source: JICA Study Team

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[2] PC (box) girder is excluded from the above alternatives since its heavy weight is a disadvantage in the erection at the intersection and economical aspect (pile numbers will be increased due to heavy weight).

4.6.1.5 Foundation Type for Flyover

(1) Introduction

The following site conditions are taken into account for the extraction of alternatives:

- Loading Level : Normal (PC-I Girder/Max. span 30 m)
  Large (Steel-I Girder/Max. span 52 m, Steel Box Girder/Max. span 70 m)
- Construction Yard : Construction yard is limited/narrow in the residential area
- Vibration and Noise : Low possibility of vibration and noise is desirable for construction in the residential area
- Harmful Gas : Low influence of harmful gas due to the construction is desirable for construction in the residential area
- Soil Condition/Depth of Supporting Layer : G.L -40 m to 45 m
- Soil Condition/Soil Type of Supporting Layer : Clay-IV (PF2 – PF8)
  Clayey Sand II (AF1, PF1, PF9- AF2)

According to Table 4.6.7, Cast-in-place RC pile, PHC/SC Pile, Steel Pipe Pile, Diaphragm Wall Foundation and Concrete Caisson can be applied as the foundation type of flyover. However, Diaphragm Wall Foundation and Concrete Caisson are excluded from the alternatives since these foundation types are not economical if the loading level is not so large.

Hence, the three alternatives below are nominated for the comparative study of foundation type. The comparative study was conducted for 1) Standard section represented by “AF1” and”PF6”, and 2) Special section represented by “PF3”.

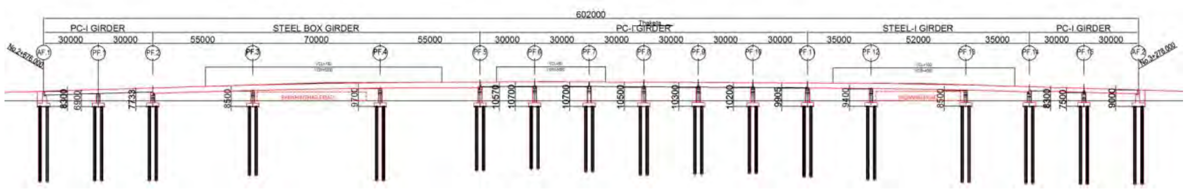
- ➢ Alternative-1 Precast PC Pile
- ➢ Alternative-2 Cast-in-place RC Pile (Original plan in the Supplemental F/S)
- ➢ Alternative-3 Steel Pipe Pile

Table 4.6.7 Possible Foundation Type for Flyover

Criteria \ Applicable Foundation Type			Cast- in-place RC Pile	PHC / SC PIle	Steel Pipe Pile	Diaphragm wall	Steel pipe sheet pile	Concrete Caisson
Condition of Construction	Construction on River/Sea	Water Depth < 5 m	×	△	△	×	○	△
		Water Depth > 5 m	×	△	△	×	○	△
	Construction Yard	Narrow/Limited	△	△	△	△	×	△
	Environment	Vibration, Noise	○	△	△	○	×	○
		Impact on Adjacent Structure	○	△	△	○	△	△
		Harmful Gas	○	○	○	○	○	○
	Loading Level	Small (Span < 20 m)	○	○	○	×	×	○
		Normal (20 m ≦ Span < 50 m)	○	○	○	○	○	○
		Large (50 m < Span)	○	△	○	○	○	○
		Vertical Load > Sway Load	○	○	○	△	△	△
		Vertical Load < Sway Load	○	○	○	○	○	○
Ground Condition	Depth of Supporting Layer from Ground Level	< 5 m	△	×	×	×	×	×
		5 ~ 15 m	○	○	○	△	△	○
		15 ~ 25 m	○	○	○	○	○	○
		25 ~ 40 m	○	○	○	○	○	○
		40 ~ 60 m	○	△	○	○	○	○
		≥ 60 m	△	×	×	△	△	△
	Water Level on Land	W.L is nearly G.L	△	○	○	△	○	○
	Liquefaction		○	○	○	○	○	○
	Soil Type of Supporting Layer	Clay (20 ≤ N)	○	○	○	○	○	○
		Sand/Gravel (30 ≤ N)	○	○	○	○	○	△
		Soft Rock/Hard soil	○	○	○	○	○	○
		Hard Rock	△	×	×	△	×	×

Legend :○ Highly applicable □ Applicable × Inapplicable

Source: Prepared by the JICA Study Team based on JSHB



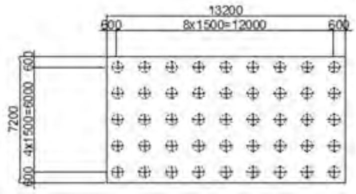
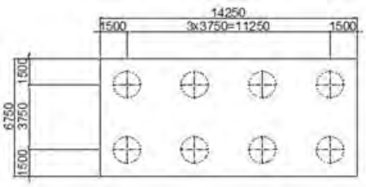
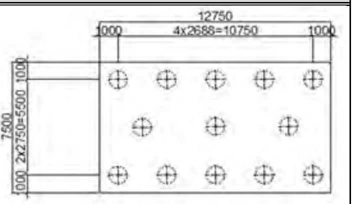
Source: JICA Study Team

Figure 4.6.4 Representative Substructure for the Comparative Study of Foundation Type

(2) Foundation Type for Flyover

As given in Table 4.6.8 to Table 4.6.10, in terms of economical aspect, "Alternative-2 Cast-in-place RC Pile" is the optimal foundation type for the flyover section.

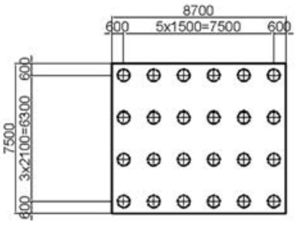
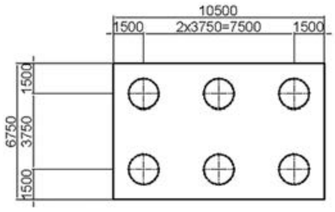
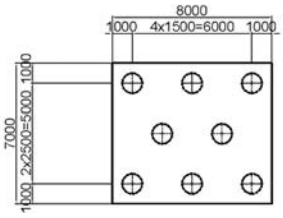
Table 4.6.8 Comparative Study of Foundation Type for Special Section (AF1)

Evaluation Item	Alt-1 Precast PC Pile		Alt-2 Cast-in-place RC Pile (Plan at F/S)		Alt-3 Steel Pipe Pile	
Schematic View	 D = 600 mm x 45 Nos (L = 41.5 m)		 D = 1500 mm x 8 Nos (L = 41.5 m)		 D = 1000 mm x 13 Nos (L= 41.5 m)	
Workability & Quality Control	- Inflexible to changes of pile length during construction - Precast PC pile is superior in quality control	○	- Flexible to changes of pile length during construction - Careful quality control is necessary for cast-in-place pile	○	- Inflexible to changes of pile length during construction - Pre-fabricated steel pile is superior in quality control	○
Structural Aspect	- Bearing capacity/pile: Low - Applicable length : 5 m – 40 m	△	- Bearing capacity/pile: High - Applicable length : 5 m – 60 m	◎	- Bearing capacity/pile: Medium - Applicable length : 5 m – 60 m	○
Construction Cost	Ratio = 1.56	△	Ratio = 1.00	◎	Ratio = 1.34	○
Construction Period	32 days / Foundation	△	23 days / Foundation	○	14 days / Foundation	◎
Environmental Aspect	- Larger noise and vibration - Disposal of excavated soil is necessary	△	- Low noise and vibration - Disposal of excavated soil is necessary	○	- Larger noise and vibration - Disposal of excavated soil is necessary	○
Evaluation	Less Recommended		**Most Recommended**		Less Recommended	

Legend : ◎ Very Good, ○ Good, △ Moderate × Not Good

Source: JICA Study Team

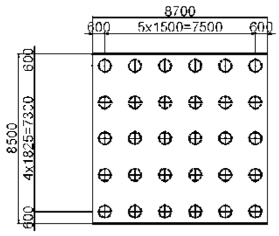
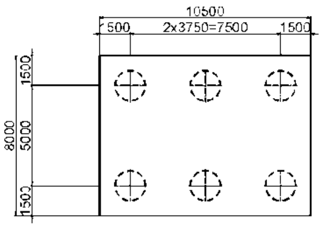
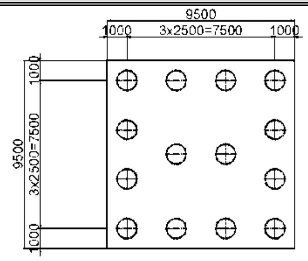
Table 4.6.9 Comparative Study of Foundation Type for Special Section (PF6)

Evaluation Item	Alt-1 Precast PC Pile		Alt-2 Cast-in-place RC Pile (Plan at F/S)		Alt-3 Steel Pipe Pile	
Schematic View	 D = 600 mm x 24 Nos (L = 37.5 m)		 D = 1500 mm x 6 Nos (L = 37.5 m)		 D = 1000 mm x 8 Nos (L = 37.5 m)	
Workability & Quality Control	- Inflexible to changes of pile length during construction - Precast PC pile is superior in quality control	○	- Flexible to changes of pile length during construction - Careful quality control is necessary for cast-in-place pile	○	- Inflexible to changes of pile length during construction - Pre-fabricated steel pile is superior in quality control	○
Structural Aspect	- Bearing capacity/pile: Low - Applicable length : 5 m – 40 m	△	- Bearing capacity/pile: High - Applicable length : 5 m – 60 m	◎	- Bearing capacity/pile: Medium - Applicable length : 5 m – 60 m	○
Construction Cost	Ratio = 1.15	△	Ratio = 1.00	◎	Ratio = 1.09	○
Construction Period	15 days / Foundation	△	14 days / Foundation	○	9 days / Foundation	◎
Environmental Aspect	- Larger noise and vibration - Disposal of excavated soil is necessary	△	- Low noise and vibration - Disposal of excavated soil is necessary	○	- Larger noise and vibration - Disposal of excavated soil is necessary	○
Evaluation	Less Recommended		**Most Recommended**		Less Recommended	

Legend : ◎ Very Good, ○ Good, △ Moderate × Not Good

Source: JICA Study Team

Table 4.6.10 Comparative Study of Foundation Type for Special Section (PF3)

Evaluation Item	Alt-1 Precast PC Pile		Alt-2 Cast-in-place RC Pile (Plan at F/S)		Alt-3 Steel Pipe Pile	
Schematic View	 D = 600 mm x 30 Nos (L = 40.0 m)		 D = 1500 x 6 Nos (L = 40.0 m)		 D = 1000 mm x 14 Nos (L = 40.0 m)	
Workability & Quality Control	- Inflexible to changes of pile length during construction - Precast PC pile is superior in quality control	○	- Flexible to changes of pile length during construction - Careful quality control is necessary for cast-in-place pile	○	- Inflexible to changes of pile length during construction - Pre-fabricated steel pile is superior in quality control	○
Structural Aspect	- Bearing capacity/pile: Low - Applicable length : 5 m – 40 m	△	- Bearing capacity/pile: High - Applicable length : 5 m – 60 m	◎	- Bearing capacity/pile: Medium - Applicable length : 5 m – 60 m	○
Construction Cost	Ratio = 1.37	△	Ratio = 1.00	◎	Ratio = 1.85	○
Construction Period	20 days / Foundation	△	18 days / Foundation	○	15 days / Foundation	◎
Environmental Aspect	- Larger noise and vibration - Disposal of excavated soil is necessary	△	- Low noise and vibration - Disposal of excavated soil is necessary	○	- Larger noise and vibration - Disposal of excavated soil is necessary	○
Evaluation	Less Recommended		**Most Recommended**		Less Recommended	

Legend : ◎ Very Good, ○ Good, △ Moderate × Not Good

Source: JICA Study Team

(3) Optimum Diameter of Foundation Pile

In addition to the above study, the comparative studies were conducted to justify the optimum diameter of cast-in-place RC pile. As shown in Table 4.6.11, "Alternative-3 D = 1500 mm" is the most economical option.

Table 4.6.11 Comparative Study of Foundation Diameter

Item		Alt-1 D = 1000 mm	Alt-2 D = 1200 mm	Alt-3 D = 1500 mm
Construction Cost	AF1	Ratio = 1.05 (18 Nos / L = 41.5 m)	Ratio = 1.21 (12 Nos / L = 41.5 m)	Ratio = 1.00 (8 Nos / L = 41.5 m)
	PF6	Ratio = 1.17 (15 Nos / L = 37.5 m)	Ratio = 1.07 (8 Nos / L = 37.5 m)	Ratio = 1.00 (6 Nos / L = 37.5 m)
	PF3	Ratio = 1.16 (15 Nos / L = 40.0 m)	Ratio = 1.39 (12 Nos / L = 40.0 m)	Ratio = 1.00 (6 Nos / L = 40.0 m)
Evaluation		Less Recommended	Less Recommended	**Most Recommended**

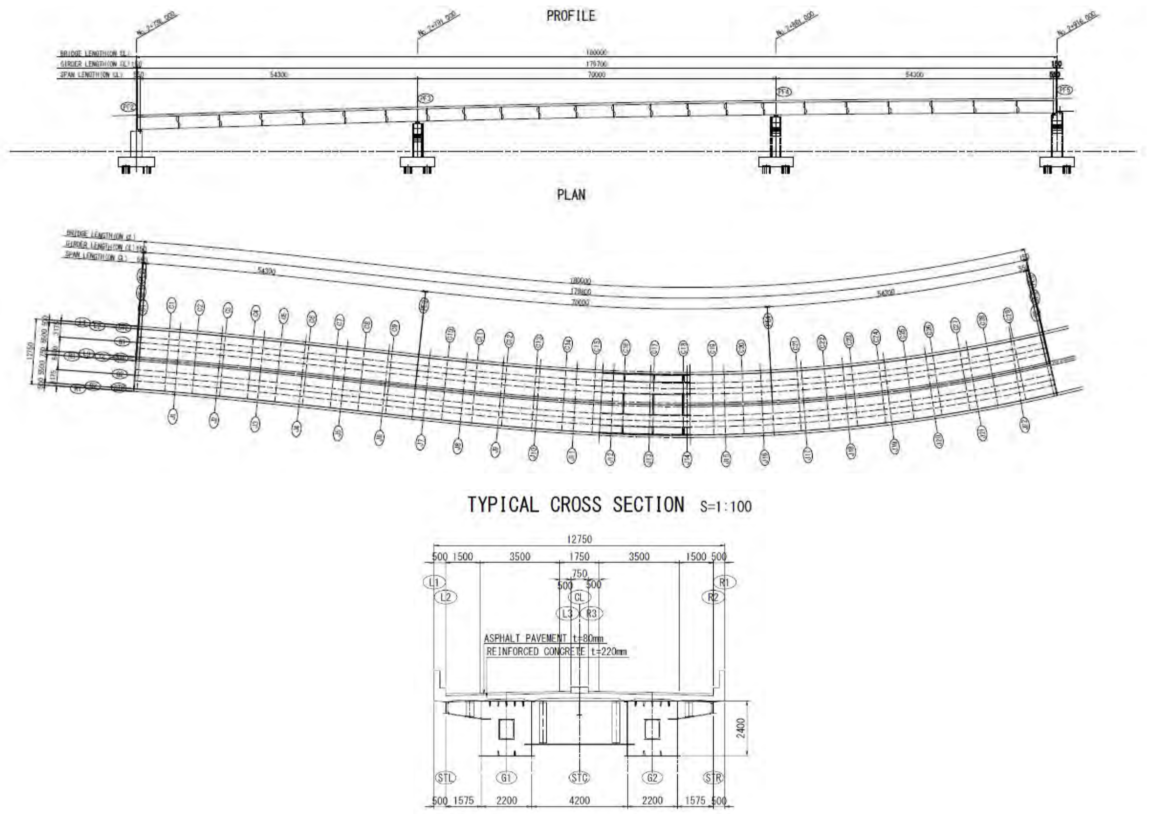
Source: JICA Study Team

4.6.2 Basic Design Results

4.6.2.1 Steel Girder Bridge

(1) Steel Box Girder Bridge

The profile, plan, and typical cross section of the steel box girder bridge in the B/D are shown in the following figure.

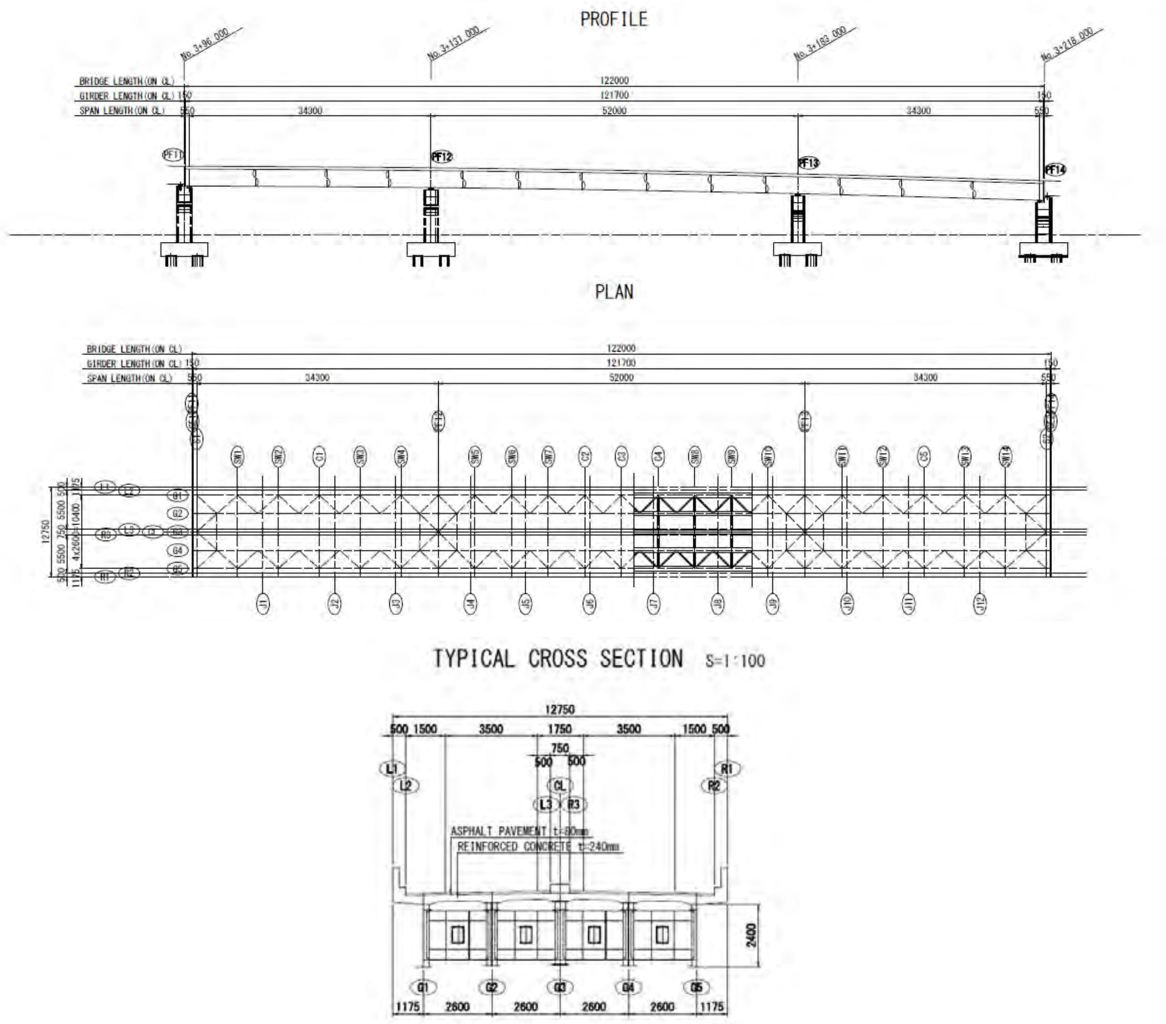


Source: JICA Study Team

Figure 4.6.5 Plan, Profile and Typical Cross Section of Steel Box Girder Bridge in the B/D

(2) Steel-I Girder Bridge

The profile, plan, and typical cross section of the steel-I girder bridge in the B/D are shown in the following figure.

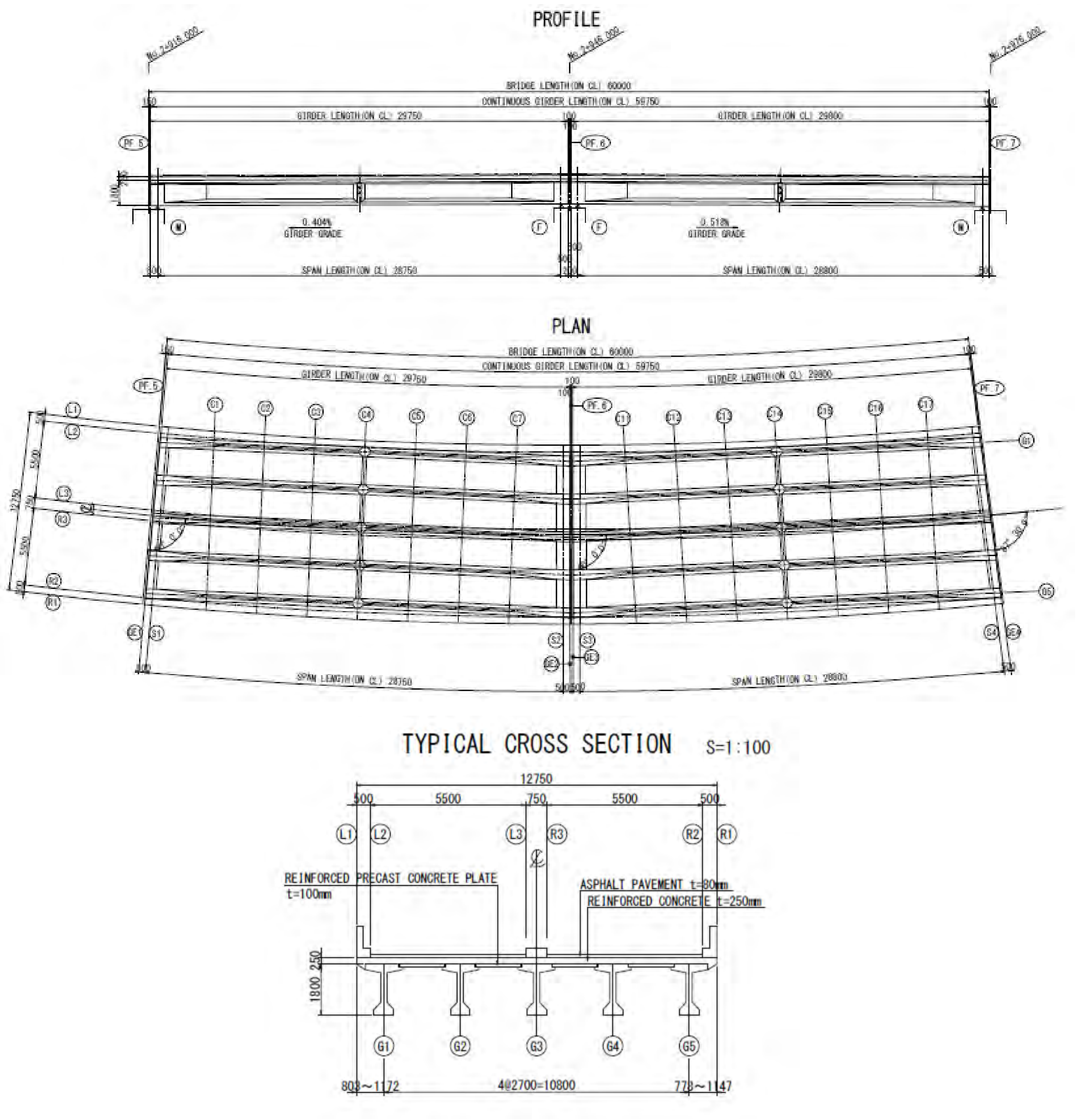


Source: JICA Study Team

Figure 4.6.6 Plan, Profile and Typical Cross Section of Steel-I Girder Bridge in the B/D

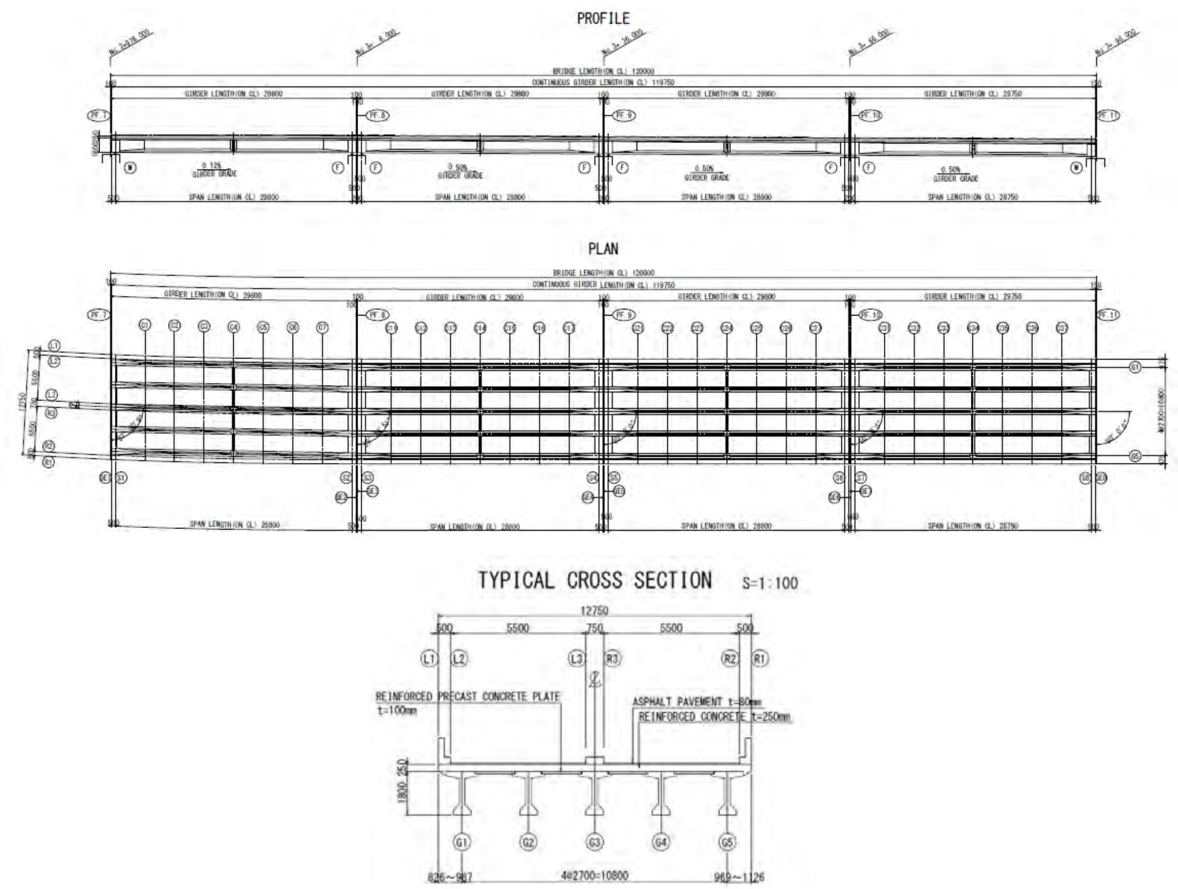
4.6.2.2 PC-I Girder Bridge

The profile, plan, and typical cross section of the PC-I girder bridge in the B/D are shown in the following figures.



Source: JICA Study Team

Figure 4.6.7 Plan, Profile and Typical Cross Section of PC-I Girder Bridge in the B/D (PF5-PF7)

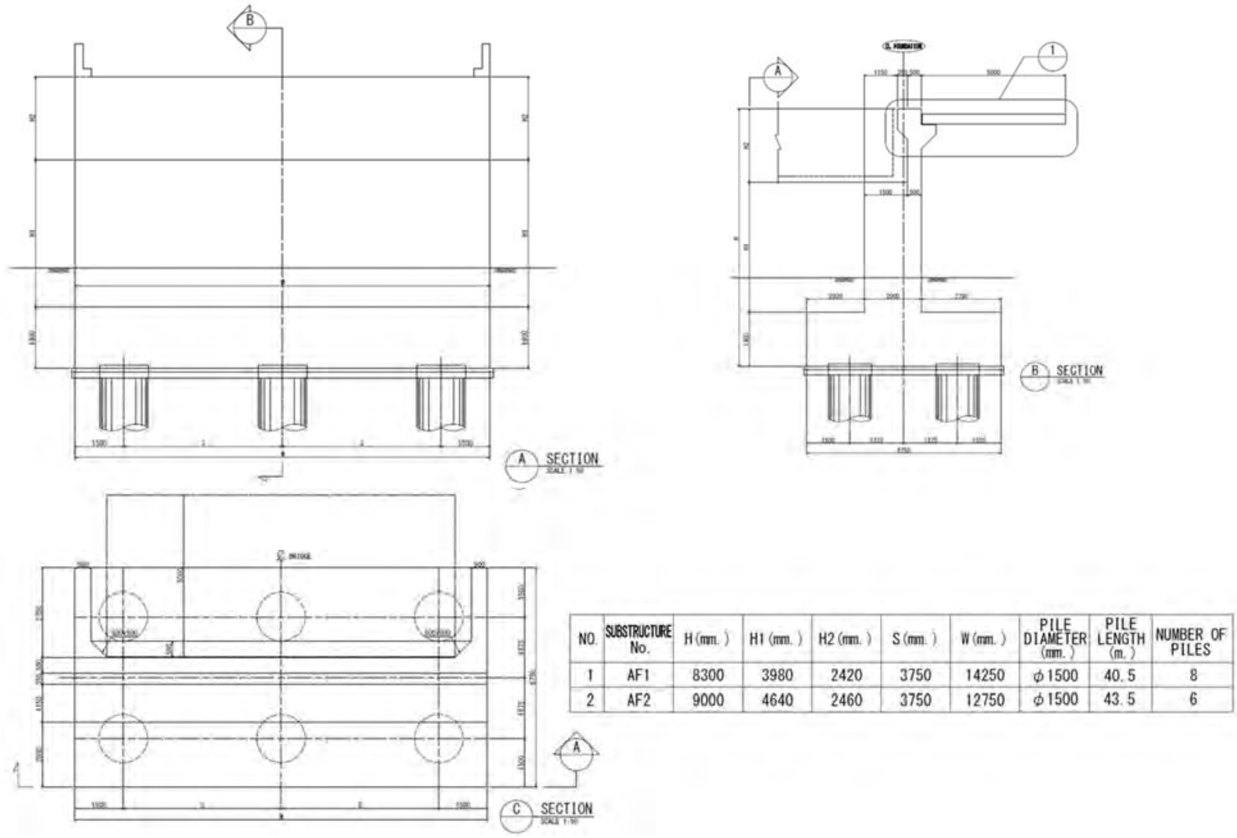


Source: JICA Study Team

Figure 4.6.8 Plan, Profile and Typical Cross Section of PC-I Girder Bridge in the B/D (PF7-PF11)

4.6.2.3 Substructures and Foundations

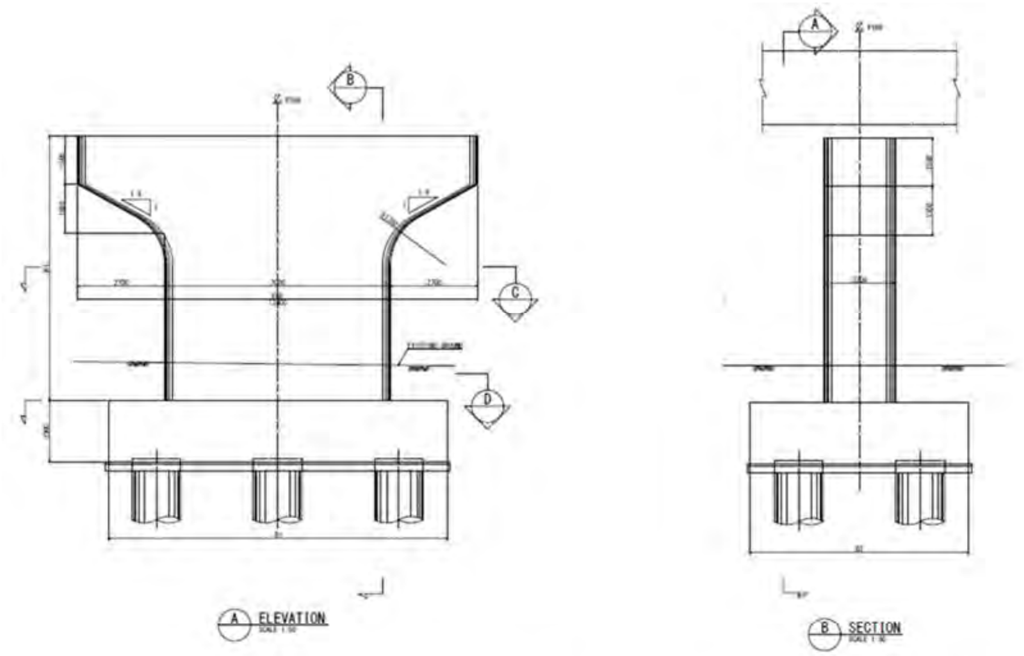
The general view of the abutment and pier in the B/D is shown in the following figures.

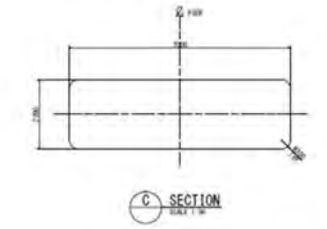


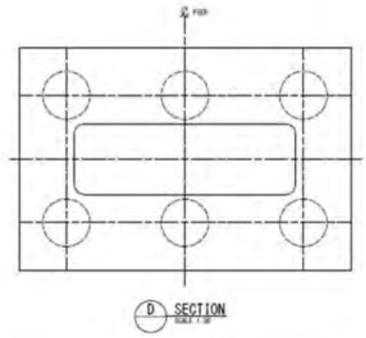
NO.	SUBSTRUCTURE No.	H (mm.)	H1 (mm.)	H2 (mm.)	S (mm.)	W (mm.)	PILE DIAMETER (mm.)	PILE LENGTH (m.)	NUMBER OF PILES
1	AF1	8300	3980	2420	3750	14250	ϕ1500	40.5	8
2	AF2	9000	4640	2460	3750	12750	ϕ1500	43.5	6

Source: JICA Study Team

Figure 4.6.9 General View of Abutment in the B/D



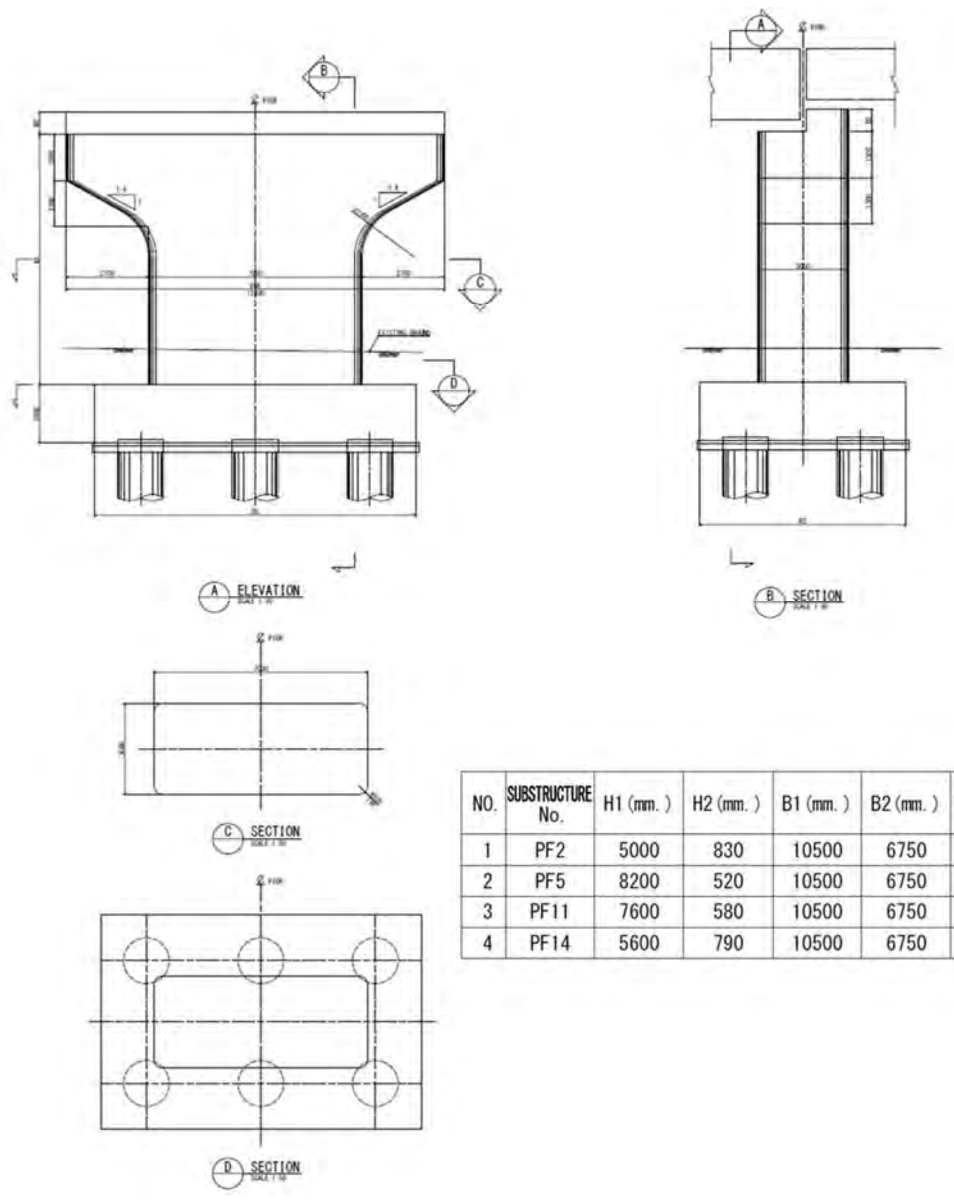




NO.	SUBSTRUCTURE No.	H1 (mm.)	B1 (mm.)	B2 (mm.)	PILE DIAMETER (mm.)	PILE LENGTH (m.)	NUMBER OF PILES
1	PF1	4800	10500	6750	φ1500	41.5	6
2	PF3	6600	10500	8000	φ1500	40.0	6
3	PF4	7800	10500	7500	φ1500	41.0	6
4	PF6	8800	10500	6750	φ1500	37.5	6
5	PF7	8800	10500	6750	φ1500	39.0	6
6	PF8	8600	10500	7500	φ1500	39.0	6
7	PF9	8400	10500	7500	φ1500	39.0	6
8	PF10	8300	10500	7500	φ1500	39.0	6
9	PF12	7500	10500	6750	φ1500	40.0	6
10	PF13	6400	10500	6750	φ1500	42.0	6
11	PF15	5600	10500	6750	φ1500	43.0	6

Source: JICA Study Team

Figure 4.6.10 General View of Pier (Type A) in the B/D



NO.	SUBSTRUCTURE No.	H1 (mm.)	H2 (mm.)	B1 (mm.)	B2 (mm.)	PILE DIAMETER (mm.)	PILE LENGTH (m.)	NUMBER OF PILES
1	PF2	5000	830	10500	6750	φ1500	41.5	6
2	PF5	8200	520	10500	6750	φ1500	37.5	6
3	PF11	7600	580	10500	6750	φ1500	39.5	6
4	PF14	5600	790	10500	6750	φ1500	42.5	6

Source: JICA Study Team

Figure 4.6.11 General View of Pier (Type B) in the B/D

4.6.3 Major Updates in the Detailed Design from the Basic Design

4.6.3.1 Major Updates on Steel Girder Bridge

(1) Steel Box Girder Bridge

Nothing was updated from the B/D.

(2) Steel-I Girder Bridge

In the D/D, the flange width was optimized for cost reduction as shown in the table below.

Table 4.6.12 Comparison of Configuration of Steel-I Girder

Item		B/D	D/D
Girder	Height	2400 mm	2400 mm
	Flange Width	620 mm	590 mm
RC Deck Thickness		240 mm	240 mm

Source: JICA Study Team

4.6.3.2 Major Updates on PC-I Girder Bridge

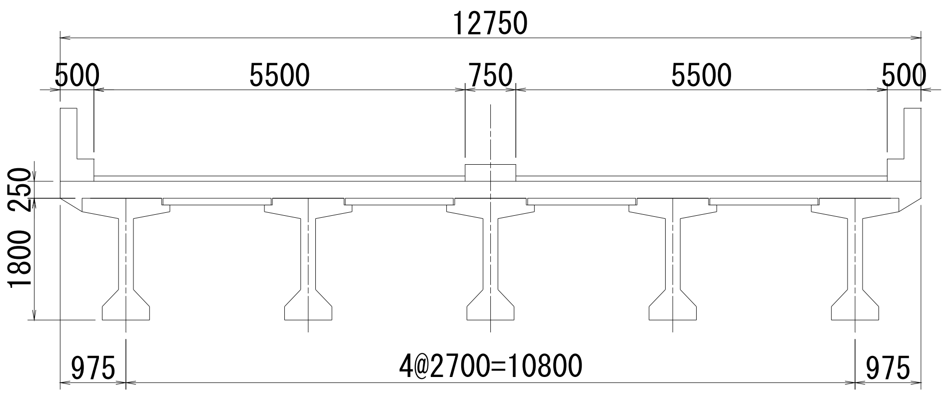
The updates on PC-I girder bridge are shown in the following table:

Table 4.6.13 Comparison of PC-I Girder

Item	B/D	D/D
Number of Girders	5 nos.	4 nos.
Girder Height	1800 mm	1900 mm
Deck Thickness	250 mm	170 mm

Source: JICA Study Team

In the B/D, the reinforced concrete deck slab had been applied in the superstructure. If the reinforced concrete deck slab is applied, the superstructure needs five main girders because the span length of the reinforced concrete deck slab is generally about 3 m between the main girders, and the overhang length of the deck slab is generally about 1.5 m from the center of the girder to the end of the deck slab. In addition, the girder height had been assumed to be 1800 mm based on conventional ratio, which is 1/17, to the average span length. The main girder on the cross section in the B/D is shown in Figure 4.6.12.

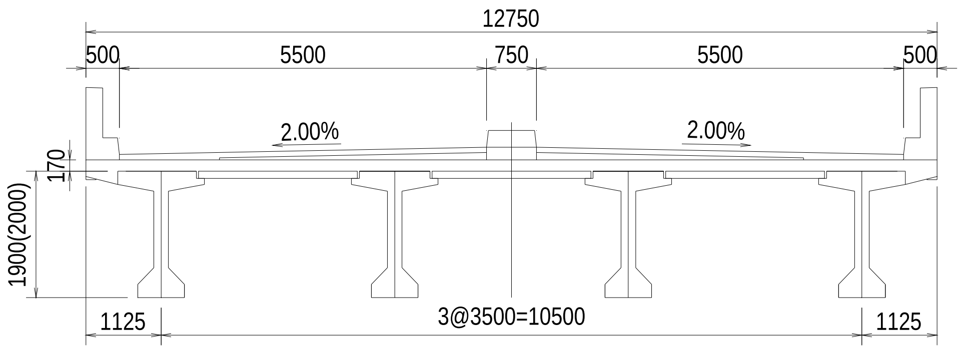


Source: JICA Study Team

Figure 4.6.12 Cross Section of Superstructure for PC-I Girder Bridge in the B/D

In the D/D, the composite concrete deck slab (reinforced concrete deck slab and prestressed concrete plate) was considered to be applied to the superstructure to reduce the number of main girders. The span length of the composite concrete deck slab which is located between the main girders is generally 2.6 m to 3.8 m. Hence, the main girder height increased by 10 mm from the B/D but the number of main girders was reduced. Finally, the main girder height is 1900 mm to 2000 mm and the number of main girders is four.

On the other hand, the structure type of the overhang is same as in the B/D. The overhang length of the reinforced concrete deck slab is 1.125 m. The cross section of the superstructure in the D/D is shown in Figure 4.6.13



( ) : PF7 to PF11

Source: JICA Study Team

Figure 4.6.13 Cross Section of Superstructure for PC-I Girder Bridge in the D/D

4.6.3.3 Major Updates on the Substructures and Foundations

The updates on substructures and foundations are as follows:

- Geotechnical design parameters

The geotechnical design parameters determined in the B/D were reviewed and modified in the D/D, because the number of boring results used to determine the parameters was increased in the D/D. For more details of the location and coordinate of boreholes, refer to Section 4.6.4.4(1). The modulus of deformation "E" had been calculated as E = 700 N for all layers according to the worth value obtained by borehole lateral load test in the B/D. In the D/D, on the other hand, E was calculated to be E = 500 N for only "Silty Sand I" because the results of the additional tests conducted in the D/D were also considered. Additionally, the layer distribution was reviewed and updated before the commencement of the D/D, based on the soil investigation surveys conducted in the D/D. For more details, refer to Section 4.6.4.4(1).

Table 4.6.14 Comparison of Design Soil Parameters between the B/D and D/D

<Design Soil Parameter in B/D>

Layer	N Average *1	Unit Weight "γ" (kN/m3)	Cohesion "c" (kN/m2)	Friction Angle "φ" *5 (°)	Modulus of Deformation "E" (kN/m2)
FILLED SOIL	4	16 *3	24 *4	0	1300 *6
CLAY-I	4	18 *2	24 *1	0	1300 *6
SANDY CLAY-I	6	17 *2	25 *1	0	4200 *7
SILTY SAND-I	10	17 *2	0 *4	32	7000 *7
SANDY SILT	9	18 *3	54 *4	0	6300 *7
SILTY SAND-II	23	19 *3	0 *4	33	16100 *7
CLAY-II	22	18 *3	132 *4	0	15400 *7
CLAYEY SAND-I	41	19 *3	0 *4	33	28700 *7
CLAY-III	35	18 *3	210 *4	0	24500 *7
CLAYEY SAND-II	50	19 *3	0 *4	37	35000 *7
CLAY-IV	50	18 *3	300 *4	0	35000 *7

<Design Soil Parameter in D/D>

Layer	N Average *1	Unit Weight "γ" (kN/m3)	Cohesion "c" (kN/m2)	Friction Angle "φ" *5 (°)	Modulus of Deformation "E" (kN/m2)
FILLED SOIL	4	18 *3	24 *4	0	1300 *6
CLAY-I	4	18 *2	24 *1	0	1300 *6
SILTY SAND-I	10	18 *2	0 *4	32	5000 *8
SANDY SILT	8	17 *3	48 *4	0	5600 *7
SILTY SAND-II	22	19 *3	0 *4	33	15400 *7
CLAY-II	21	18 *3	126 *4	0	14700 *7
CLAYEY SAND-I	35	19 *3	0 *4	33	24500 *7
CLAY-III	35	18 *3	210 *4	0	24500 *7
CLAYEY SAND-II	50	19 *3	0 *4	37	35000 *7
CLAY-IV	50	18 *3	300 *4	0	35000 *7

Source: JICA Study Team

- Assessment result of soil liquefaction

The assessment of soil liquefaction was reviewed in the D/D, because the number of boring sites considered was increased. However, the result of liquefaction assessment was not changed from the B/D to the D/D; the geotechnical parameters are reduced only for the layer of the Sandy Silt up to 10 m in depth. On the other hand, it was not necessary to reduce the geotechnical parameters for the other layers. For more details of the liquefaction assessment, refer to Section 4.6.4.4(2).

Table 4.6.15 Comparison of Assessment Results of Soil Liquefaction between the B/D and D/D

In B/D

(a) 0≦x≦10

	FL	R	FL	R	FL	R	FL	R	FL	R
	FILLED SOIL		CLAY-I		SILTY SAND-I		SANDY SILT		SILTY SAND-II	
BH-01			6.766	1.465	1.086	0.274				
BH-02	3.771	0.689	1.910	0.433	1.093	0.308				
BH-03			4.483	0.894	1.039	0.253	0.898	0.236		
BH-04	2.281	0.424	2.807	0.612	2.146	0.566				
BH-05			0.943	0.189	1.501	0.357	0.896	0.237		
BH-06					1.132	0.272				
BH-07	1.130	0.200	0.979	0.189	1.203	0.305				
BH-08					1.360	0.295				
BH-09			1.441	0.272	1.280	0.278				
BH-10					1.189	0.252				
BH-11			0.922	0.192	1.138	0.261				
BH-12					3.551	0.953				
BH-13			11.587	2.565	7.754	2.149				
BH-14			2.213	0.464	1.453	0.377				
Average	**2.394**	**0.438**	**3.405**	**0.728**	**1.923**	**0.493**	**0.897**	**0.237**		
DE	**1**		**1**		**1**		**2/3**		-	

(b) 10<x≦20

	FL	R	FL	R	FL	R	FL	R	FL	R
	FILLED SOIL		CLAY-I		SILTY SAND-I		SANDY SILT		SILTY SAND-II	
BH-01							0.963	0.254	1.168	0.292
BH-02							2.964	0.847	1.488	0.396
BH-03							1.167	0.307	1.149	0.288
BH-04									1.106	0.284
BH-05							7.336	1.923	1.068	0.266
BH-06							1.131	0.289	1.013	0.251
BH-07					1.884	0.494	0.994	0.259	0.962	0.234
BH-08					1.300	0.321	1.270	0.307	1.073	0.256
BH-09					1.121	0.259	1.677	0.390	1.254	0.291
BH-10							2.044	0.472	1.221	0.285
BH-11							1.232	0.290	1.254	0.294
BH-12					1.040	0.280	1.025	0.269	0.869	0.218
BH-13					0.972	0.265	1.033	0.272	1.200	0.301
BH-14					1.248	0.324	14.509	3.683	1.346	0.333
Average					**1.261**	**0.324**	**2.873**	**0.736**	**1.155**	**0.285**
DE	-		-		**1**		**1**		**1**	

In D/D

(a) 0≦x≦10

	FL	R	FL	R	FL	R	FL	R	FL	R
	FILLED SOIL		CLAY-I		SILTY SAND-I		SANDY SILT		SILTY SAND-II	
BH-01			5.922	1.263	1.093	0.269				
BH-02	3.393	0.617	1.827	0.407	1.078	0.293				
BH-03			3.953	0.780	1.044	0.247	0.910	0.231		
BH-04	2.111	0.395	2.517	0.548	1.432	0.365				
BH-05			0.942	0.186	1.396	0.324	0.912	0.232		
BH-06					1.103	0.267				
BH-07	1.109	0.197	0.968	0.186	0.953	0.242				
BH-08					1.425	0.315				
BH-09			1.433	0.269	1.207	0.264				
BH-10					1.155	0.248				
BH-11					1.130	0.257				
BH-12					3.210	0.859				
BH-13			10.138	2.207	6.886	1.920				
BH-14			1.832	0.407	1.400	0.366				
BH-5(13)					0.991	0.225				
ave	**2.204**	**0.403**	**3.281**	**0.695**	**1.700**	**0.431**	**0.911**	**0.232**		
DE	**1**		**1**		**1**		**2/3**		-	

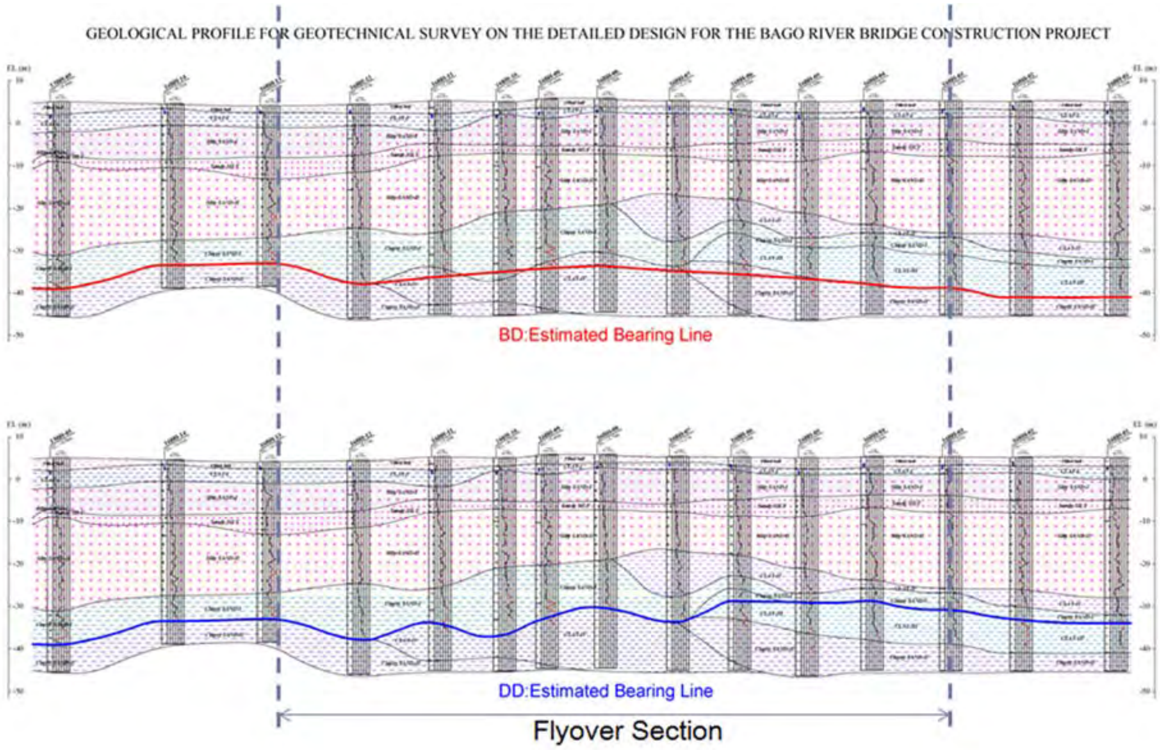
(b) 10<x≦20

	FL	R	FL	R	FL	R	FL	R	FL	R
	FILLED SOIL		CLAY-I		SILTY SAND-I		SANDY SILT		SILTY SAND-II	
BH-01							0.975	0.250	1.163	0.286
BH-02							2.588	0.717	1.434	0.374
BH-03							1.034	0.266	1.147	0.283
BH-04									1.076	0.272
BH-05							1.409	0.362	1.071	0.263
BH-06							1.089	0.285	1.021	0.255
BH-07					0.970	0.261	1.301	0.348	0.974	0.247
BH-08					1.128	0.276	1.221	0.301	1.075	0.256
BH-09					1.089	0.256	1.263	0.301	1.224	0.283
BH-10							1.888	0.447	1.228	0.285
BH-11							1.214	0.287	1.200	0.277
BH-12					1.010	0.278	0.995	0.268	0.854	0.218
BH-13					1.031	0.286	1.007	0.272	1.182	0.302
BH-14					1.168	0.310	13.839	3.613	1.319	0.330
BH-5(13)					0.851	0.201			1.386	0.320
ave					**1.035**	**0.267**	**2.294**	**0.594**	**1.157**	**0.283**
DE	-		-		**1**		**1**		**1**	

Source: JICA Study Team

- Supporting layer

The supporting layer was also reviewed in the D/D because the results of the soil investigation surveys were updated in the D/D as shown in Figure 4.6.14. For more details, refer to Section 4.6.4.4(3).

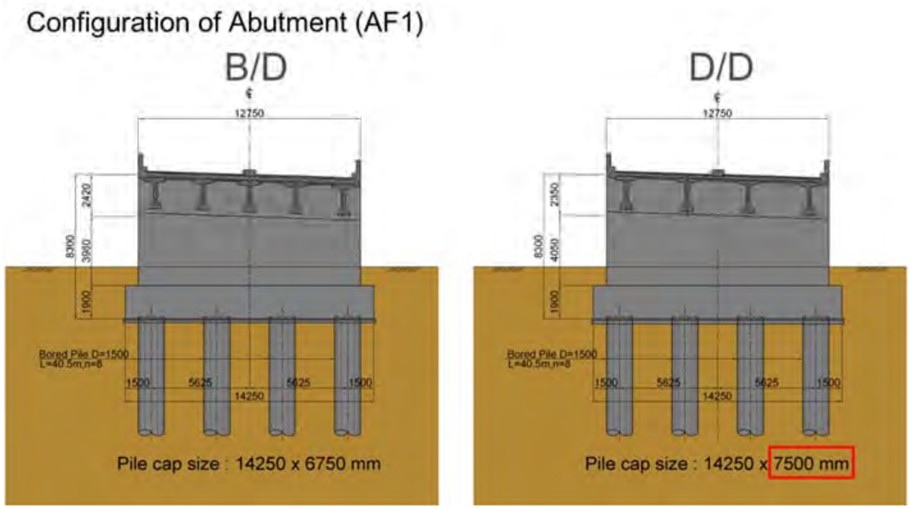


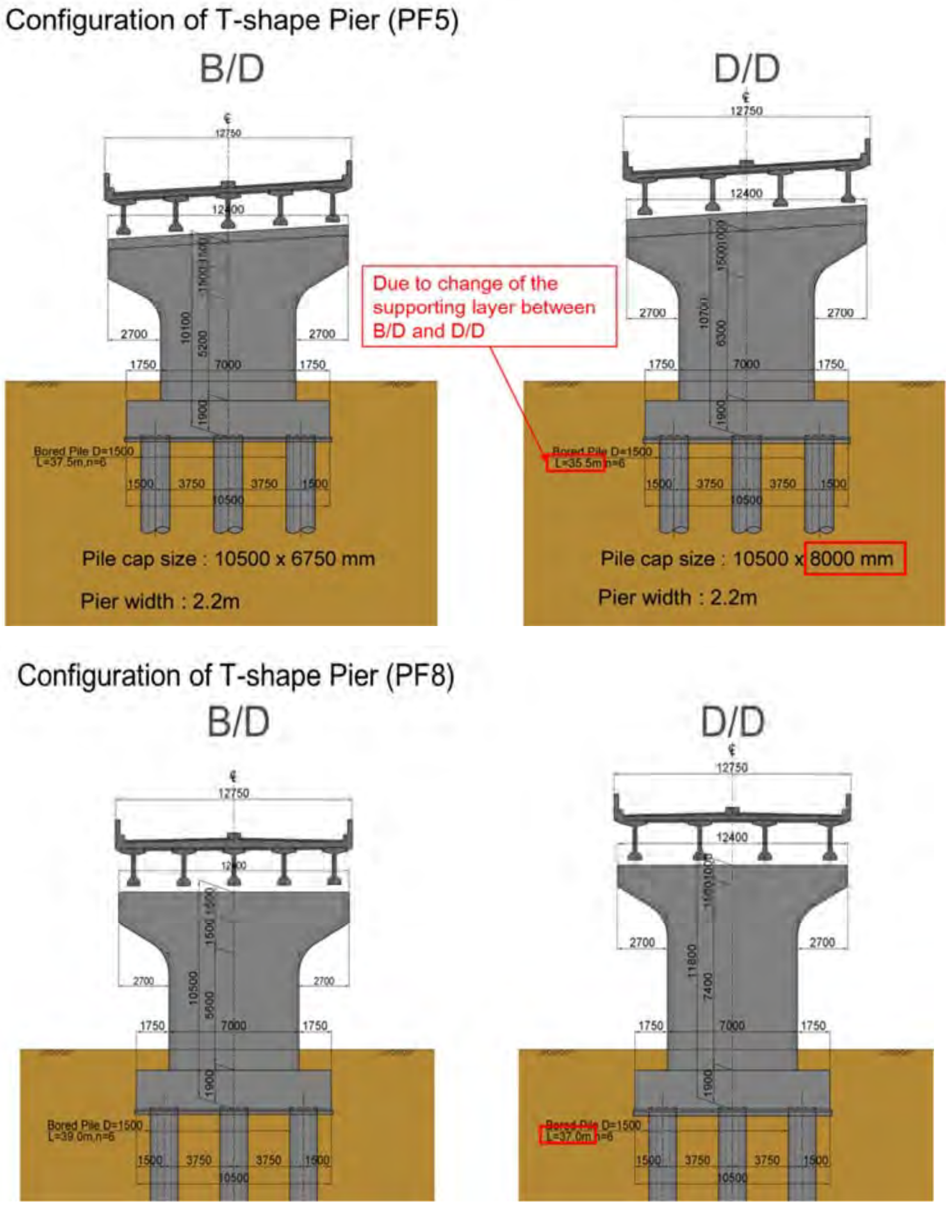
Source: JICA Study Team

Figure 4.6.14 Update on Bearing Layer between the B/D and D/D

- Configuration of abutments and piers

The configuration of abutments and piers was also modified in the D/D due to the abovementioned updates. The following figure shows the configuration of the representative abutment and piers in the B/D and D/D.





Source: JICA Study Team

Figure 4.6.15 Update on Configuration of Representative Substructures between the B/D and D/D

4.6.4 Bridge Accessories

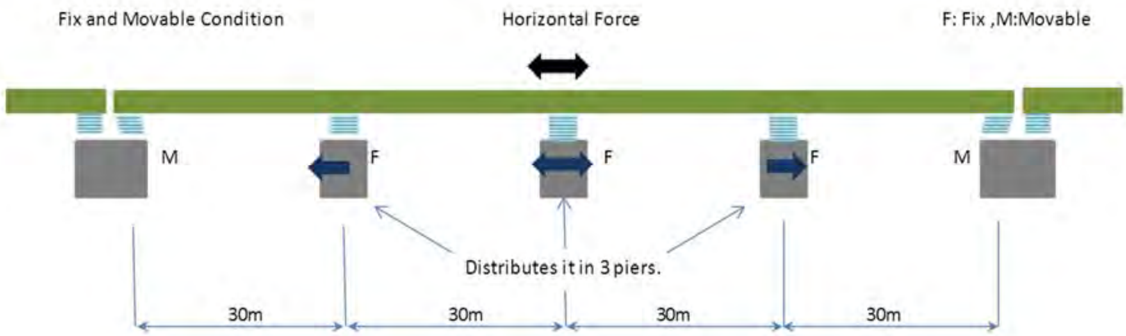
(1) Bearing Condition and Bearing

There are two types of bearing conditions; one is the "Fixed and Moveable Support" and the other is the "Elastic Support". If the "Fixed and Moveable" support conditions are applied to large-scale bridges, the horizontal force during earthquake and/or temperature load tends to be concentrated on the fixed piers, although the displacement at the girder end can be relatively small, then the size of the substructure and foundation would be too large. The "Fixed and Moveable" support condition is widely used in small-scale bridges.

Therefore, when determining the support condition and bearing type, it is necessary to consider the structural effect of the bearing conditions, such as lateral load distribution, displacement, etc.

1) PC-I Girder Bridge

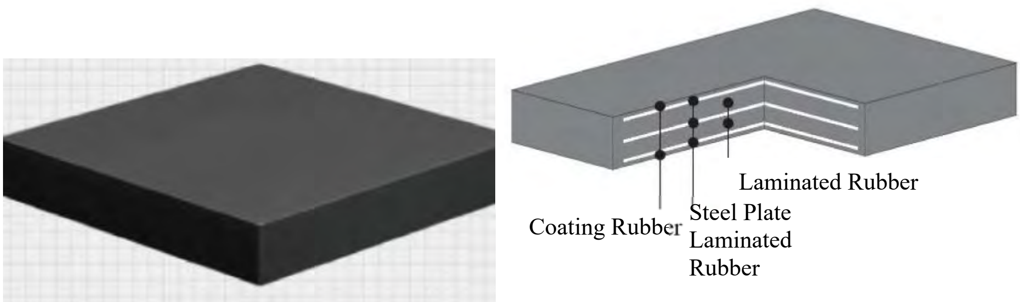
The lateral force under earthquake and/or temperature load may not be large even if the "Fixed and Moveable" support condition is applied to a three span PC-I girder bridge since PC-I girder bridge with span of 30 m is a small-scale bridge. Hence, the "Fixed and Moveable" support condition shall be applied to the PC-I girder bridges in the flyover section using an economical pad type rubber bearing as shown in Figure 4.6.16 and Figure 4.6.17.

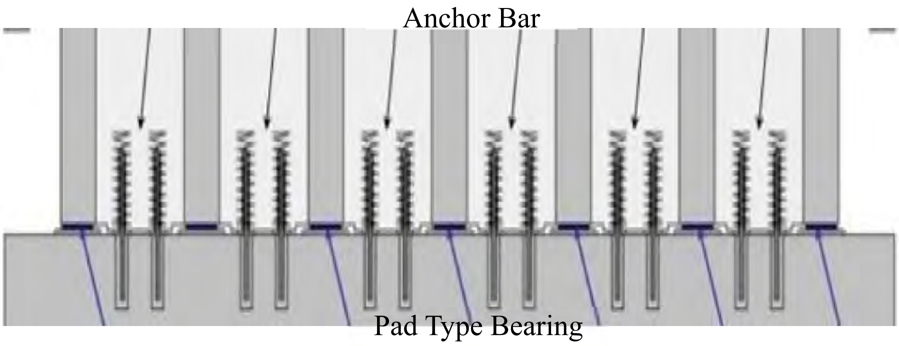


Source: JICA Study Team

Figure 4.6.16 Distribution of Horizontal Force

The lateral force and vertical force of a superstructure can be smoothly transmitted to substructures through the pad type rubber bearing which can follow the displacement of girders caused by temperature change, drying shrinkage, creep, and earthquake. In addition, the bearing is reinforced with thin steel plates to control the swelling of rubber by the compressive force. Fixed bearing condition shall be secured by anchor bars between girders as shown in Figure 4.6.17.



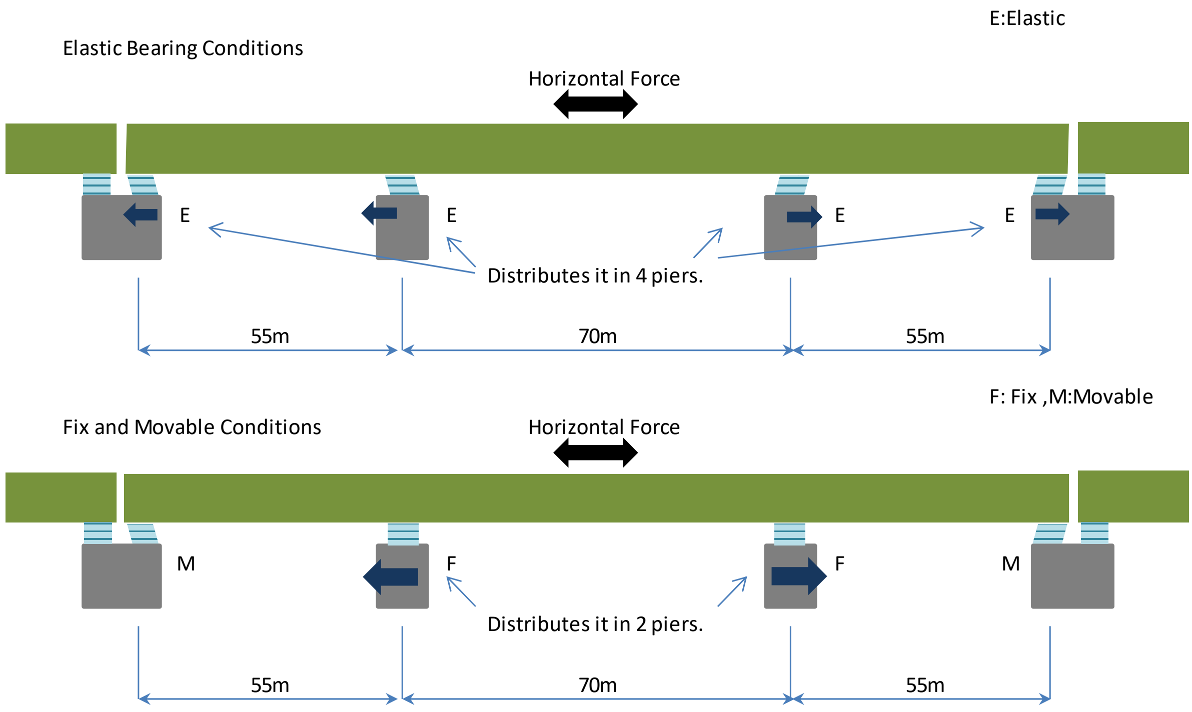


Source: JICA Study Team

Figure 4.6.17 Arrangement of Anchor Bars

2) Steel Bridge

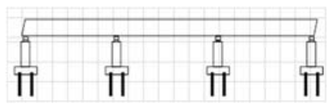
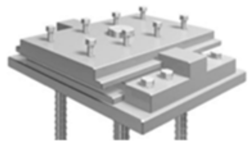
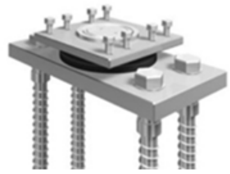
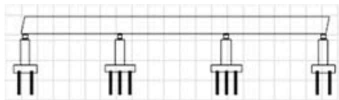
The length of the steel girder bridge (180 m) is relatively long and the maximum span is 70 m. As shown in Figure 4.6.18, in case of elastic bearing condition, the lateral force is shared by four piers. On the other hand, in case of the "Fixed and Moveable" bearing condition, the lateral force is shared only by two piers. The difference in the distributed lateral load to the substructure and displacement at the girder ends due to the bearing conditions may affect the economic viability of the overall structure. Hence, a comparative study was carried out to identify the optimum bearing conditions for steel girder bridges in the flyover section. As a result of the comparative study, in terms of economic aspect, the "Alt-1 Rubber Bearing" condition is the optimum option for the bearing condition for steel girder bridges.



Source: JICA Study Team

Figure 4.6.18 Distribution of Horizontal Force

Table 4.6.16 Bearing of Steel Bridges Condition

	Alt-1 Rubber Bearing	*Alt-2 Fixed and Moveable*
Schematic Picture	 	*Fix:*  *Move*  
Structural Characteristics	➢ *Lateral earthquake load can be distributed to all the piers.* ➢ *Displacement can be small.*	➢ *Lateral earthquake load is concentrated on fixed piers and size of foundation would be larger.* ➢ *Displacement is smallest.*
Displacement at Girder End	*60 mm*	*10 mm*
Horizontal Force at Intermediate Piers	*3,300 kN*	*4,300 kN*
*Cost**	*Ratio = 1.00*	*Ratio = 1.02*
Evaluation	*Most Recommended*	*Less Recommended*

Note: Total cost including substructures, foundations, expansion joints and bearings
Source: JICA Study Team

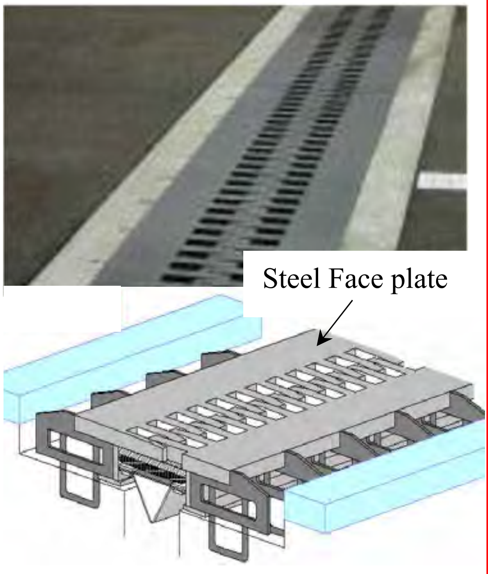
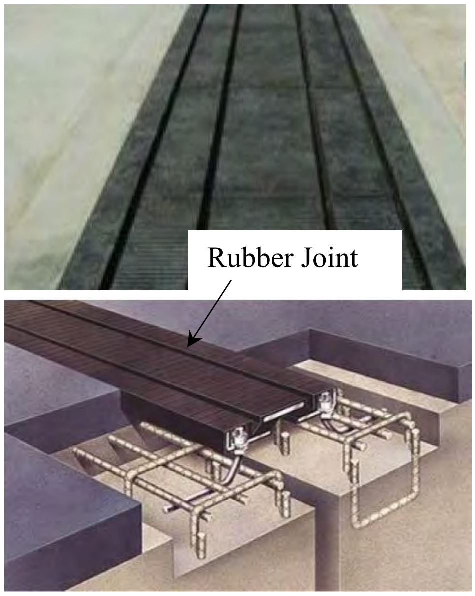
(2) Expansion Joint

The functions required for the expansion joint are the following:

- To ensure good driving conditions, even if the girder is deformed by girder temperature variations, concrete creep, concrete drying shrinkage, and loads.
- To ensure waterproofing against rainwater penetration.
- To ensure durability against vehicular traffic.
- Low noise and vibration caused by traffic.
- Easy maintenance and repair.

Expansion joints are mainly classified into rubber type and steel type. As a result of comparative study, in terms of durability and ease of maintenance, "Alt-1 Steel Type Joint" shall be applied in the flyover section.

Table 4.6.17 Comparison of Expansion Joint

	Alt-1 Steel Type Joint	Alt-2 Rubber Type Joint
Schematic Picture		
Functional Performance	➢ Durability is good. ➢ Light weight. ➢ Construction is easy.	➢ The deflection of the product increases as the gap increases. ➢ It deteriorates due to ultraviolet rays.
Maintenance	➢ Partial replacement is possible ➢ Service life is long	➢ Partial replacement is not possible ➢ Service life is slightly short

Source: JICA Study Team

(3) Unseating Prevention System

The unseating prevention system consists of the seating length of the girder at the support and a structure to prevent the superstructure from unseating during an earthquake. These components are appropriately selected in accordance with the bridge type, type of bearing supports, and ground conditions.

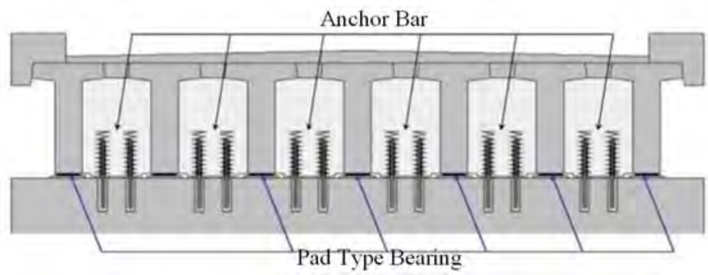
The possibility of the unseating of the superstructure from substructures during an earthquake is quite low if the superstructure is supported by four or more substructures as specified in the JSHB. On the other hand, an unseating prevention system should be installed since the possibility of unseating may be relatively high if the superstructure is supported by less than four substructures. Considering the above, the necessity of the unseating prevention system is evaluated as shown in Table 4.6.18.

As shown in Figure 4.6.19, the unseating prevention system by anchor bars shall be applied to the two span PC-I girder bridges in flyover sections.

Table 4.6.18 Necessity of Unseating Prevention System

Bridge Type	No. of Span	No. of Substructure	Unseating Prevention System
PC-I Girder	2	3 (AF1 – PF2)	Necessary (by anchor bars)
Steel Box Girder	3	4 (PF2-PF5)	Not necessary
PC-I Girder	2	3 (PF5-PF7)	Necessary (by anchor bars)
PC-I Girder	4	5 (PF7-PF11)	Not necessary
Steel-I Girder	3	4 (PF11-PF14)	Not necessary
PC-I Girder	2	3 (PF14- AF2)	Necessary (by anchor bars)

Source: JICA Study Team



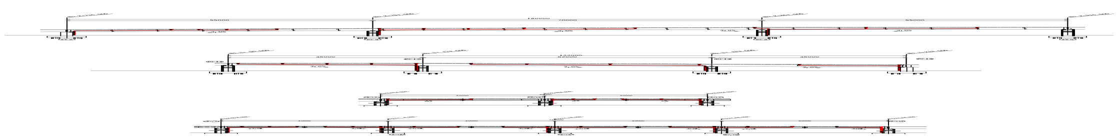
Source: JICA Study Team

Figure 4.6.19 Schematic Picture of Unseating Prevention System

(4) Drainage System

Rainwater on the bridge surface is drained by catch pits placed at an appropriate distance on the shoulder, in order to secure traffic safety. The drain pipe for each pier leads the rainwater to the catch basin, and the rainwater goes to the side ditch. The distribution diagram of the drain is shown in Figure 4.6.20 to Figure 4.6.22.

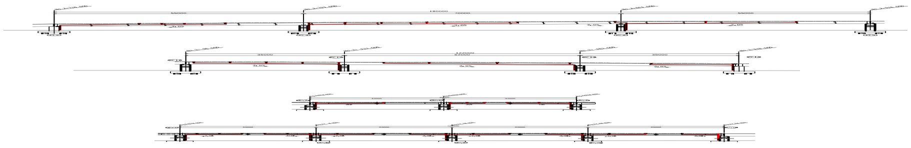
- Steel Box Girder Bridge



Source: JICA Study Team

Figure 4.6.20 Drainage Distribution Diagram of Steel Box Girder Bridge

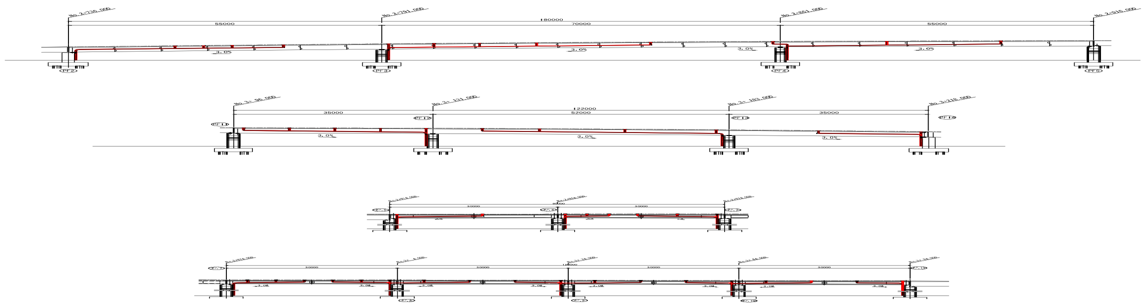
- Steel I-section Girder Bridge



Source: JICA Study Team

Figure 4.6.21 Drainage Distribution Diagram of Steel-I Girder Bridge

- PC-I Girder Bridge



Source: JICA Study Team

Figure 4.6.22 Drainage Distribution Diagram of PC-I Girder Bridge