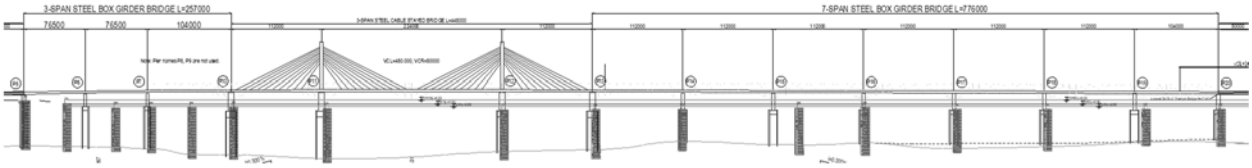
4.3 STUDY ON STEEL BOX GIRDER BRIDGE

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.



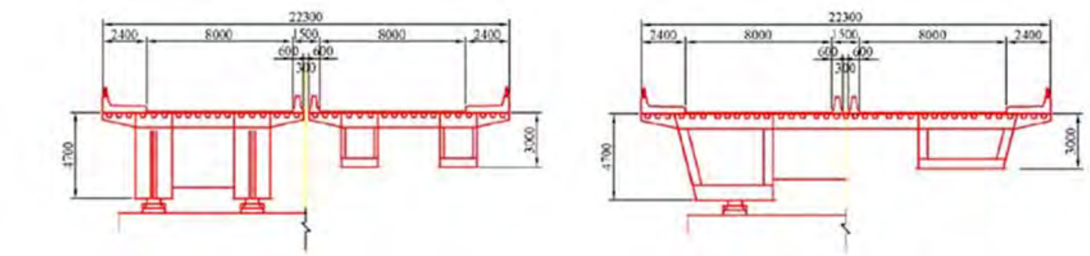
Source: JICA Study Team

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)

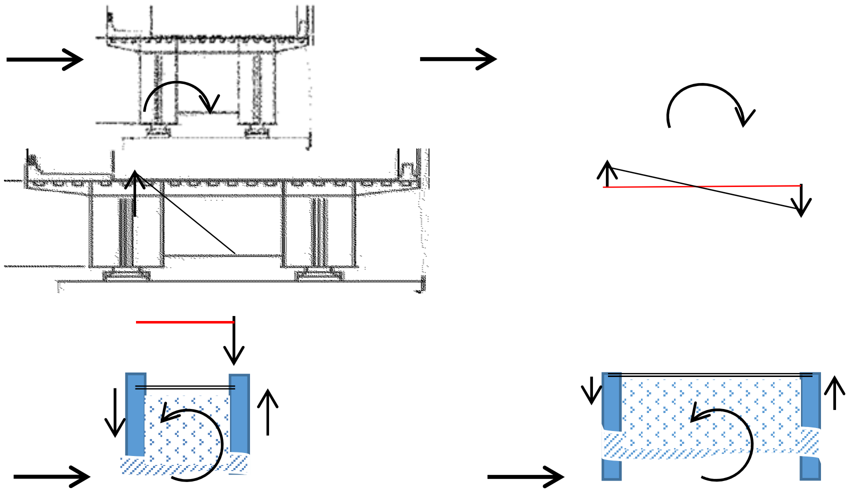
Source: JICA Study Team

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

(1) Structural Stability

1) Reaction of Bearing

If separation structure, especially high height girder structure, is adopted, an up/down-lift force or axial force induced to bearings due to horizontal force like a seismic force or a wind force will be larger than that of the combined structure because of the arm length of the couple force as shown in the figure below.



Source: JICA Study Team

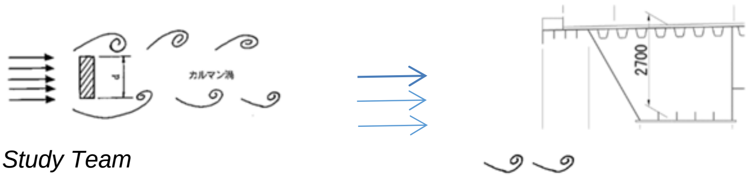
Figure 4.3.3 Image of the Force Induced to the Bearings due to Horizontal Force

Moreover, the pedestrian way in both sides was decided to be unnecessary at the initial stage of B/D, so the total width was reduced by 2.0 m and the distance between both outer girders will also be shortened.

From the reason of the above qualitative consideration, combined structure type seemed to be suitable, and so it was decided to be adopted.

2) Influence of Wind

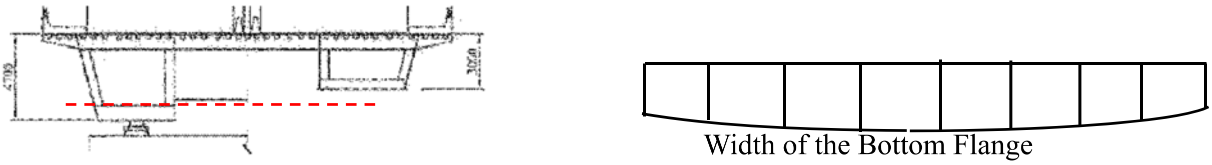
In general, a long span bridge with perpendicular tall web plate will be easy to be oscillated by wind (Karman vortex). One of the ways to avoid this influence is to adopt an inclined web. The inclination angle of approximately 60° of the outer web is common, taking account of fabrication.



Source: JICA Study Team

Figure 4.3.4 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.

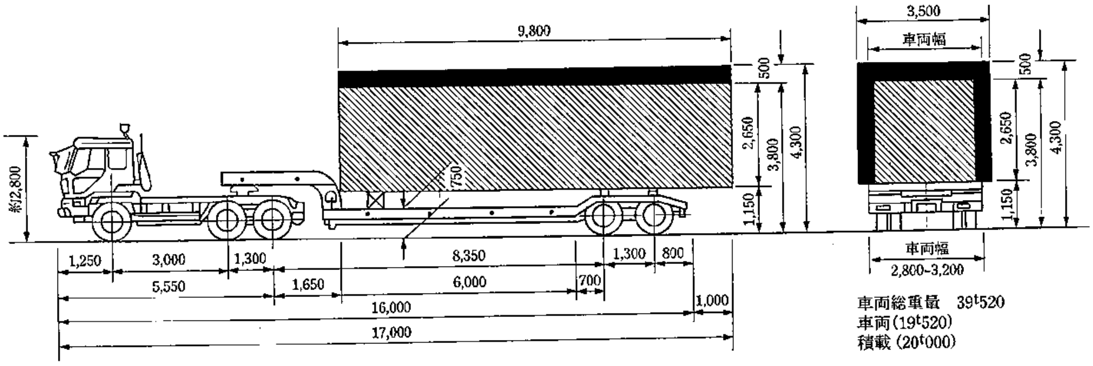


Source: JICA Study Team

Figure 4.3.5 Varying Width of Bottom Flange

3) Web Height

Web height is the most important factor to consider in deciding the suitable stiffness for bending moment induced from dead load and live load. It will be possible to decrease the thickness of the flange in accordance with the increase in the height of the web. This means, eventually there would be a deduction in steel weight. However, if the flange thickness is too thin, then many additional stiffeners will be necessary to keep its minimum stiffness against local buckling. The suitable web height for the continuous steel box girder may be around 1/30 of the 112-m span length from the experiential viewpoint of the suitable thickness of the flange plate. However, there is another requirement that web height shall be harmonized with the adjacent bridges crossing over the Bago River from the viewpoint of uniformity. In addition, the tall segment of over 3-m height will have to be divided into two parts because of transportation possibility. The following figure shows a sample trailer to be used for the transportation of the segment.

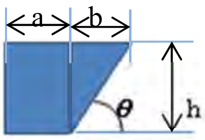


Source: JICA Study Team

Figure 4.3.6 Capability of Segment Dimension Using a Low Deck Trailer

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.

4) Deck Slab Type

A deck slab has two functions, one is to distribute and transmit load to the main girder, and the other is to serve as the primary member as a top flange resisting against bending moment.

i. Steel Deck Type

- Steel deck slab is consisted continuously of steel plate, several longitudinal stiffeners, and crossbeams. Longitudinal stiffeners are supported by crossbeams, and the crossbeams are supported by the web of the main girders, like multi-grid frames.
- Steel deck is fabricated at a shop like the main girder, and is installed at the same time with

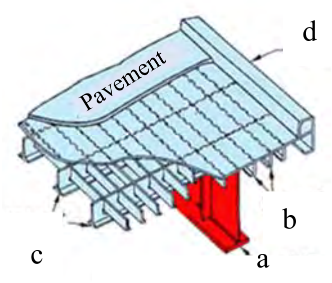
the main girder.

a: Main Girder

b: Longitudinal Stiffener

c: Crossbeam

d: Curb of Concrete



Steel Deck Slab

Source: JICA Study Team

Figure 4.3.7 Steel Deck Slab

ii. Composite Deck Type

- Composite deck with steel and concrete consists of steel plate or shapes, re-bars and concrete.
- Steel part of composite deck will be pre-fabricated at the manufacturer's factory as shown in the figure below.

a: Main Girder

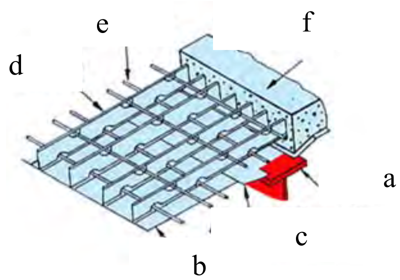
b: Bottom Steel Plate

c: Haunch Steel Plate

d: Crossbeam Stiffened Bottom Plate (I-shape)

e: Longitudinal Re-bar

f: Curb of Concrete



Composite Deck Panel

Source: JICA Study Team

Figure 4.3.8 Composite Deck Panel (Status of Shop Assembly)

Types of deck slab were compared in terms of self-weight, construction period, and cost. The result is summarized in Table 4.3.2 below. Steel deck panel was recommended to be applied because it is superior in all aspects.

Table 4.3.2 Comparison of Deck Slab Type

	Steel Deck Panel	Composite Deck Panel
Depth of Panel	19 mm (maximum) Thickness as a flange plate	260 mm: Required depth in the case of 6 m panel span
Weight	4 kN/m^2 (Flange and U-rib)	8 kN/m^2 (Bottom plate, Crossbeam, Re-bar and Concrete in-situ)
Increase of Bending Moment due to Dead Load	100%	125% The increse of 4 kN/m^2 may be 40% of the total dead load, then at least 25% of B.M will be increse
Construction Period	100%	120% Composite Deck panel installation will be possible after main girder erection
Construction Cost	100%	100% Reduction of steel deck will be mostly cancelled by the increase of B.M
Evaluation	Selected	May not be selected

Source: JICA Study Team

(2) Number of Main Girder and its Position

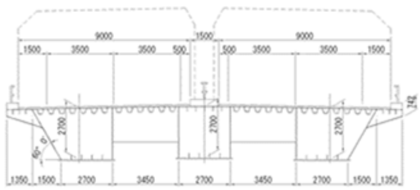
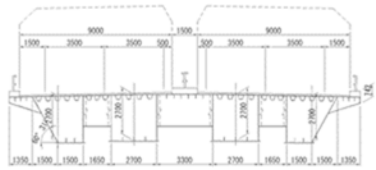
There are two options relating to the number of main girders; one is the 3-girder type and the other is the 4-girder type. Both types were compared from the following viewpoints:

- 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 4.3.3 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.

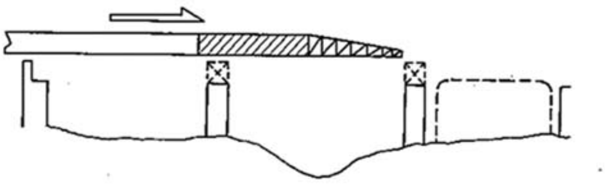
Source: JICA Study Team

4.3.1.2 Selection of Erection Method of Steel Box Girder Bridge

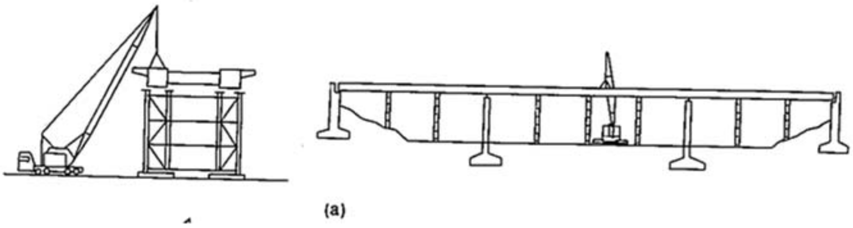
Two erection methods were introduced as follows:

i. Launching Method by Erection Nose Girder
   - The segments of the main girders will be pre-assembled on the pre-stressed concrete girder that was erected beforehand at the backyard.
   - The web plate that is supported on the temporary movable platform shall be stiffened by the additional plates to avoid local buckling.

ii. Bent Erection Method by Crawler Crane on Barge
   - The segments of the main girders will be connected at the site assembling yard and transported by barge.
   - Lifting pieces for maximum lifted weight shall be prepared on the top flange.
   - Several temporary bents shall be prepared at each span, and bolting connection will be done when the erected girders will be installed on the bents, step by step.



i. Launching Method by Erection Nose Girder



ii. Bent Erection Method by Crawler Crane on Barge

Source: JICA Study Team

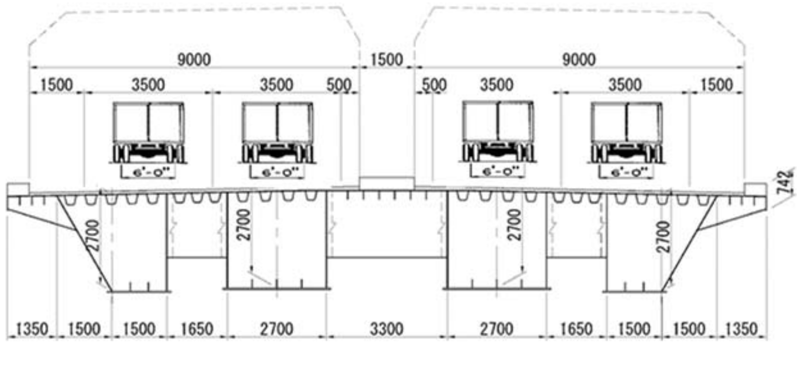
Figure 4.3.9 Erection Methods of Steel Box Girder Bridge

Details of the erection methods will be presented in the Construction Plan part of this report.

4.3.1.3 Superstructure of Steel Box Girder Bridge

(1) Arrangement of Girder Position

The distance between the web and wheel was considered so that the wheel load does not act on the web directly, in relation to the actual lane component and girder position. Eventually, a 450 mm distance at mid lane and 400 mm distance at the outer lane were put on hold.

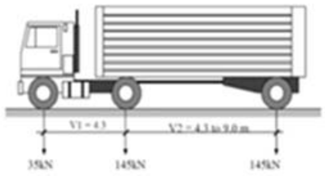


Note: Distance between both wheels is 6 feet (approximately 1.8 meter).
Source: JICA Study Team

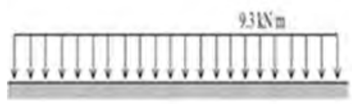
Figure 4.3.10 Position of Wheel Loads and Web

(2) Condition of Live Load

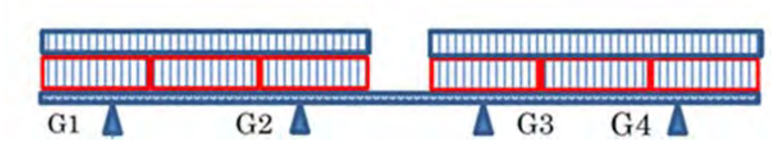
The carriageway was decided to consist of three lanes with 3 m width in each direction, in accordance with the AASHTO LRFD specifications, although actual traffic lane only consists of two lanes per one direction.



Truck load



Lane load



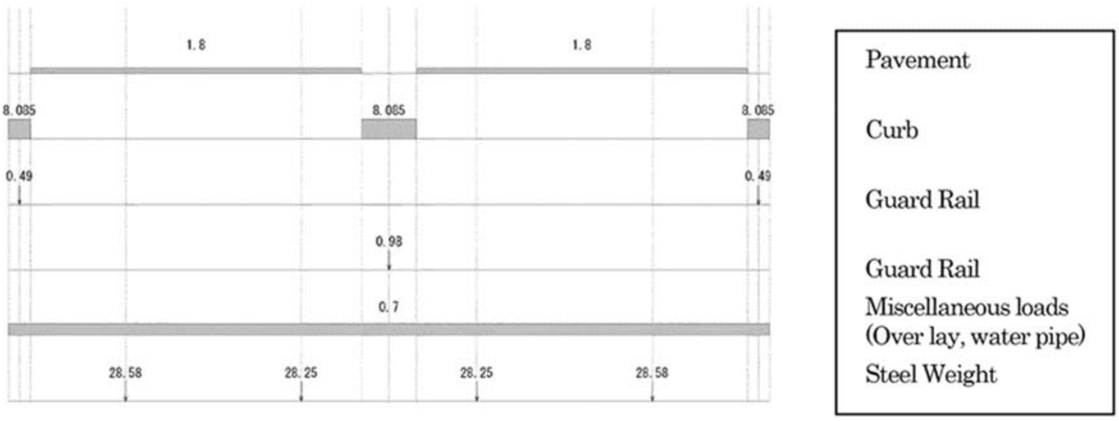
Combination of Truck Load and Lane Loads at the Cross Section

Source: JICA Study Team

Figure 4.3.11 Live Load based on AASHTO LRFD

(3) Condition of Dead Load

Dead loads including steel weight, pavement, curb, guard rail, and future overlay loads will be considered in the design.

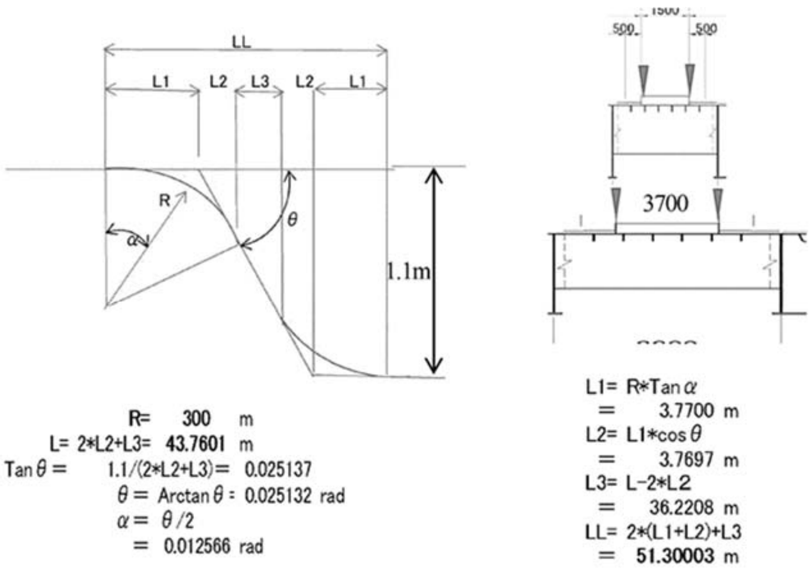


Source: JICA Study Team

Figure 4.3.12 Dead Loads to be Considered

(4) Widening of the Median Curb to the Adjacent Cable-Stayed Bridge

The width of the median curb should be widened from 1.5 m to 3.7 m because the adjacent cable-stayed bridge has a 3.7 m wide median curb of cable anchor zone. Therefore, the width of the median curb shall be widened to keep smooth drivability. The transition curve line of the curb was designed as shown below.

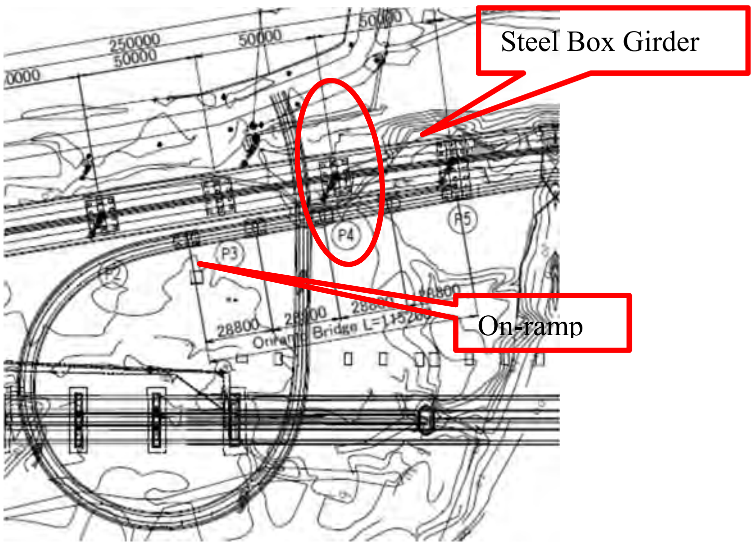


Source: JICA Study Team

Figure 4.3.13 Transition Line Between the Steel Box Girder Bridge and Cable-Stayed Bridge

(5) Adjustment of the Bridge Width to PC Box Girder and On-ramp Bridges

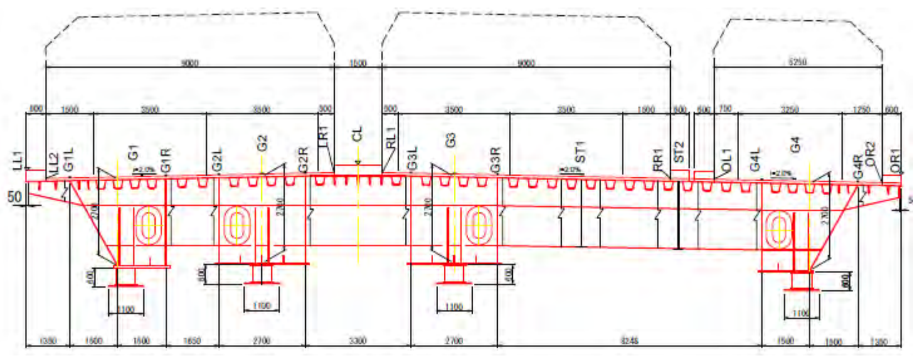
On the Pier 5, PC Box Girder on the main alignment and On-ramp of PC Box Girder are connected (Figure 4.3.14).



Source: JICA Study Team

Figure 4.3.14 On-ramp and Main Alignment at Pier 5

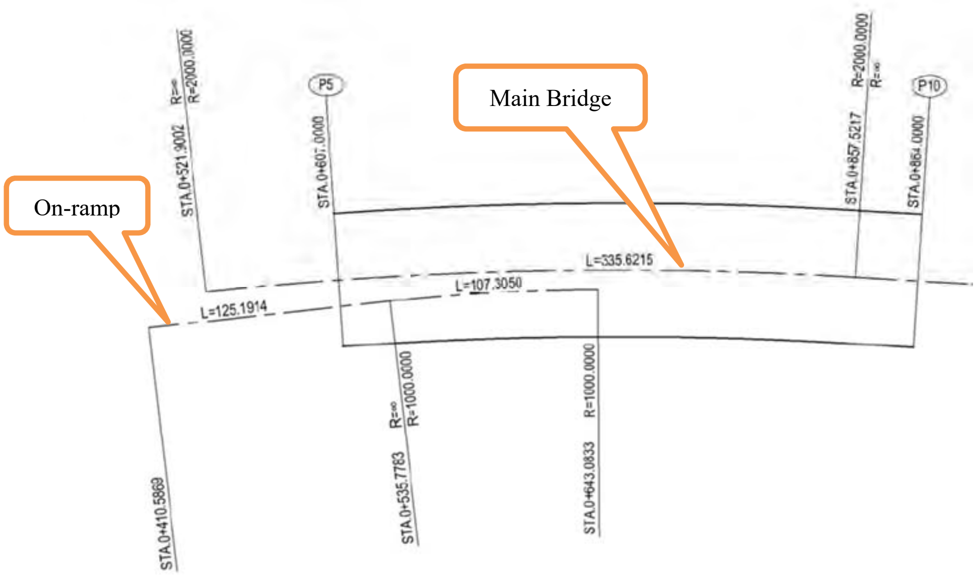
Therefore width of Steel Box Girder is widened as connected to main alignment and on-ramp.



Source: JICA Study Team

Figure 4.3.15 Cross-section of the Steel Box Girder at Pier 5

The alignment of Main bridge and On-ramp are as follows:



Source: JICA Study Team

Figure 4.3.16 Allignment of main bridge and on-ramp

4.3.1.4 Study on the Number of Continuous Span and Supporting Condition

(1) 7-Span Bridge

In the F/S, continuous span was proposed in terms of seismicity. In the B/D, the number of continuous span was comparatively studied between seven and a combination of four and three since the bridge length is 776 m. The general assessment of the number of continuous span is summarized in Table 4.3.4 below.

Table 4.3.4 General Assessment of the Number of Continuous Span

	Case-1: 7-continuous span	Case-2 Combination of 4 and 3 continuous span
Advantage	It can reduce bending moment and the thickness of girder plates.	Relatively smaller displacement due to temperature changes (±25°), approx. 100 mm.
Disadvantage	Relatively larger displacement due to temperature changes (±25°), approx. 140 mm.	It increases the weight of the steel of the girder to approx. 163 tons because of the larger bending moment.

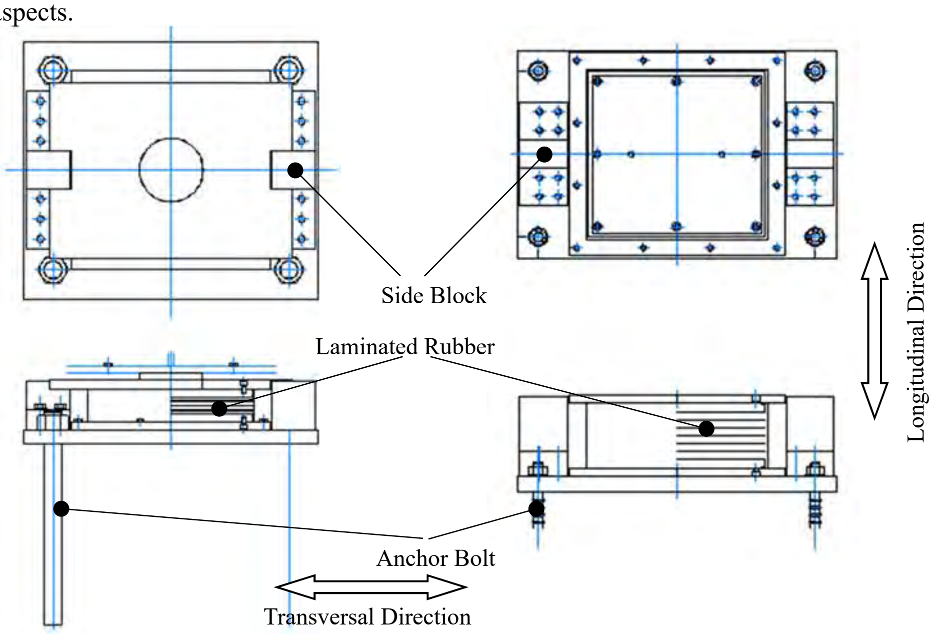
Source: JICA Study Team

As for the support, two types which can distribute inertial force of earthquake into the substructures are comparatively studied, i.e., elastic support type and fixed support type. Basic characteristics of these types are summarized in Table 4.3.5 below.

Table 4.3.5 General Characteristics of Two Types of Support

	Fixed support type	Elastic support type
Characteristic	• Force from the superstructure is directly transferred to the substructure. • If soil layers and geographical feature rise and fall, inertial force cannot be distributed equally to the piers. In such case, it may have a disadvantage in terms of structural and economical aspects.	• Force from the superstructure is distributed to the piers by utilizing shearing rigidity of the rubber bearing. • If soft soil exists around the piers, resonance might occur between the structure and the ground since natural period of oscillation of the bridge is longer.

Applicable bearing type

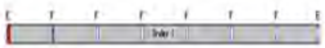
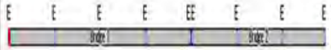


Source: JICA Study Team

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 4.3.6 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

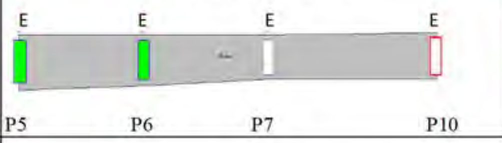
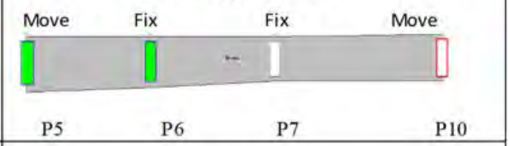
Source: JICA Study Team

(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 4.3.7 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

4.3.2 Basic Design for Substructure of Steel Box Girder Bridge

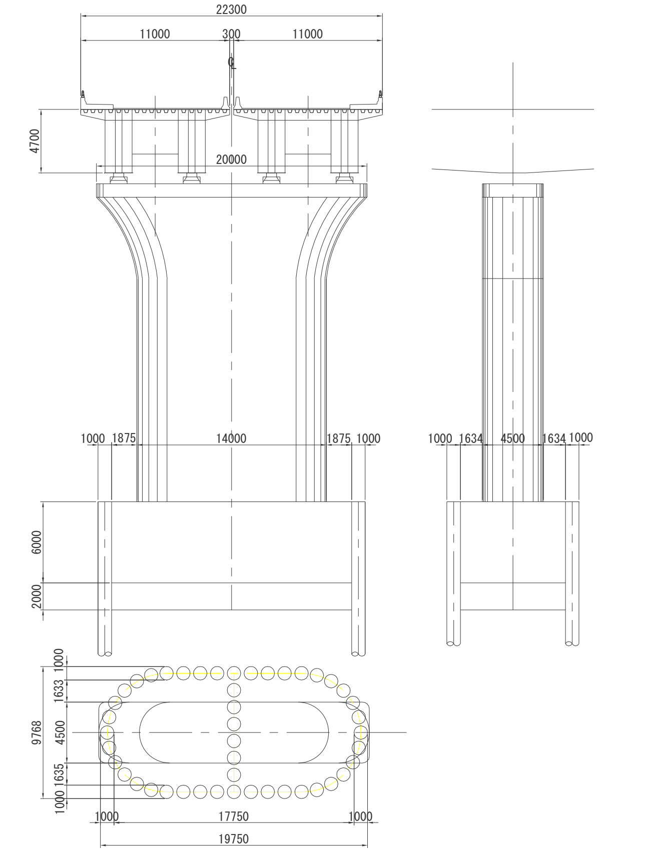
4.3.2.1 Items to be Comparatively Studied in Basic Design

In the F/S, the substructure was preliminary studied and structural outline was planned as summarized in the table below. In B/D, review of the F/S and further comparative studies among the alternatives including the structural analysis were carried out to determine the items listed below, and the optimal option is selected as basis for the detailed design.

- Shape and width, concrete class and grade of re-bars of pier column
- Construction method of SPSP foundation
- Shape and size of SPSP foundation, and material, thickness and length of steel pipe

Table 4.3.8 Structural Outline of Substructure Planned in the F/S

Item	Description
Pier Column	
Shape:	Oval shape with an overhang
Size:	20 m width at top and 14 m at bottom Thickness is 4.5 m
Material:	Reinforced Concrete Class of concrete: N/A Grade of rebar: N/A
Foundation	
Shape:	Oval shape
Size:	Dimension 19.75 m x 9.768 m Thickness of footing 6.0 m Thickness of bottom slab 2.0 m Diameter of steel pipe 1.0 m Thickness of steel pipe: N/A Length of steel pipe: N/A
Material	Grade of steel pipe: N/A
Construction Method:	Foundation & Temporary Cofferdam Method



General View of Substructure Planned in the F/S

Source: JICA Study Team based on the F/S

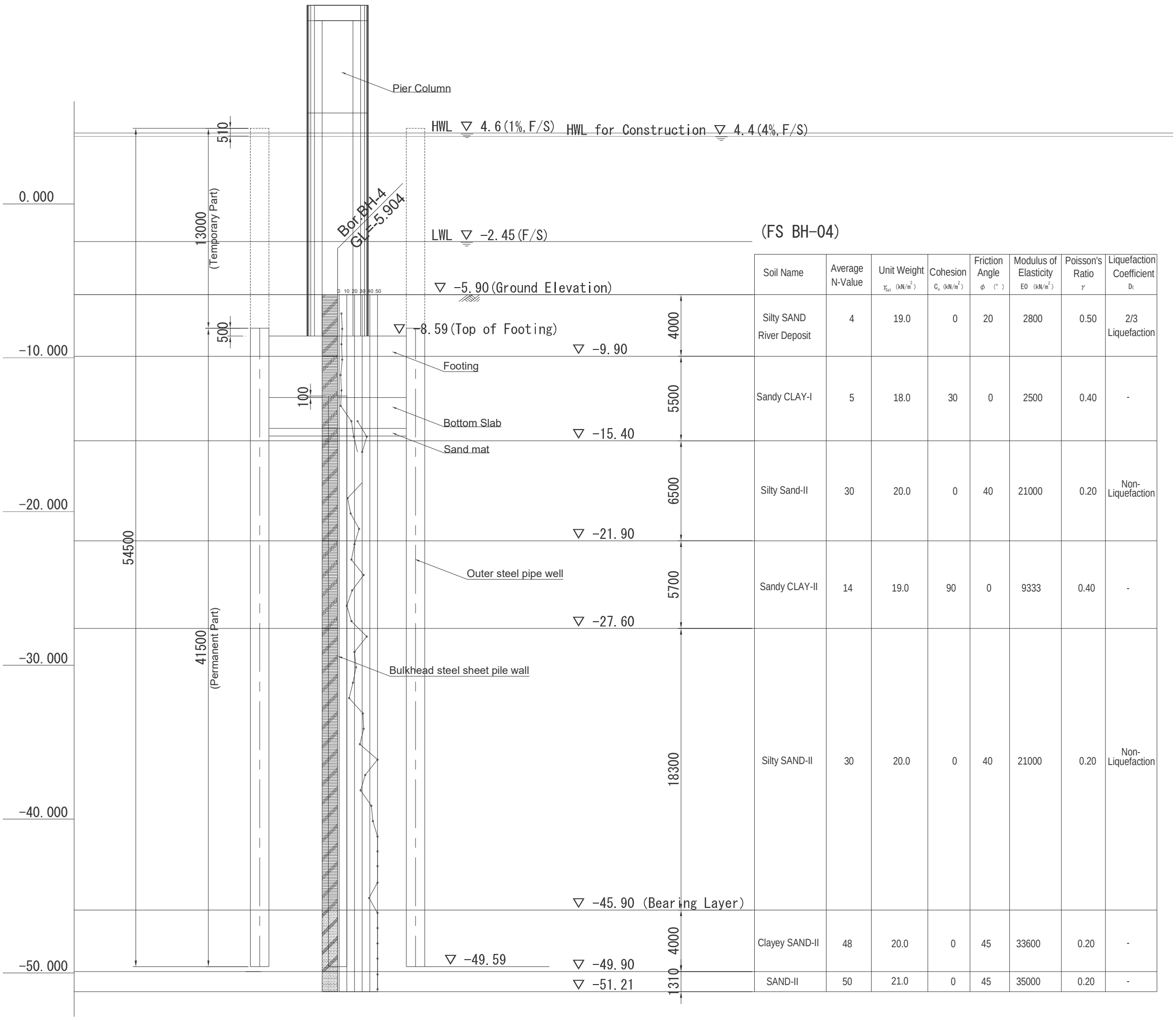
4.3.2.2 Design Conditions for Comparative Study

Substructure at P19 was examined as a representative of the piers from P14 to P19 since boring and laboratory test done in the F/S were at the exact location of P19 and reaction force from superstructure is relatively high among the target piers. The design condition is summarized in nTable 4.3.9, and the soil properties of each layer and the design water levels for the design of P19 are shown in the figure below.

Table 4.3.9 Major Design Conditions for the Design of P19 (Representing the Piers)

Item	Design Conditions
Soil Properties	Use soil properties obtained from BH04 in F/S. Details to be referred to are in Figure 4.3.17 Liquefaction is considered.
Design Water Levels	High Water Level: EL+4.6 m (1%, F/S) High Water Level for Construction: EL+4.4 m (4%, F/S) Low Water Level: EL-2.45 m (Mean Low Water Spring, F/S)
Loads	Reaction Force from Superstructure: due to dead load: 21,884 kN, live load: 5,462 kN Design Horizontal Seismic Coefficient for Inertial Force: 0.3 Water Stream Pressure to Pier and Temporary Cofferdam: 16.8 kN at 0.6 x water depth Seismic Water Pressure to Pier: 1,446 kN for longitudinal direction and 248 kN for transverse direction at 3/7 x water depth
Others	Local scouring is not considered in the basic design

Source: JICA Study Team



Source: JICA Study Team

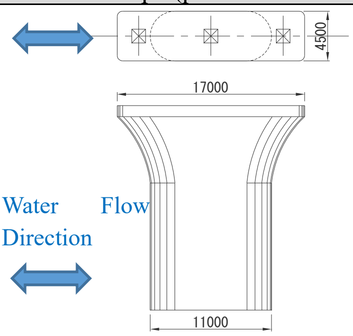
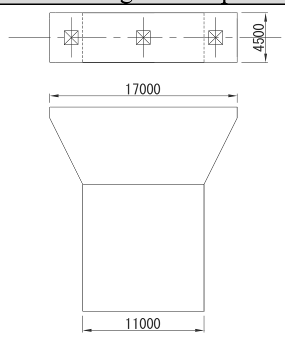
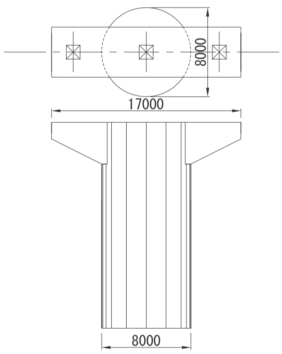
Figure 4.3.17 Soil Properties and Design Water Levels for the Design of P19 (Representing the Piers)

4.3.2.3 Shape of the Cross Section of the Pier Column

The oval shape, rectangular shape, and round shape were comparatively studied and the result is summarized in Table 4.3.10.

The most important consideration for the selection of the shape of the cross section of the pier column in a river bridge is the water flow. At the location of Bago Bridge, water flow always changes toward upstream or downstream due to the tides, but the flow direction is almost uniform. As for the cost aspect, oval shape and round shape are nearly the same. Therefore, oval shape is appropriate from the aspects of the lowest negative influence on the water flow and lower cost of substructure.

Table 4.3.10 Comparison of the Shape of the Pier Column

Comparative Item	Oval Shape (planned in the F/S)	Rectangular Shape
Schematic		
Obstruct the water flow	Lowest because of streamline shape	Moderate
Cost Ratio of Substructure*1/	1.00	1.19
Evaluation	**Most Recommended**	Less Recommended
Comparative Item	**Round Shape**	
Schematic		
Obstruct the water flow	Highest due to wider dimension against water flow. Also, this shape is suitable when the direction of river flow is often changed like at the river junction. However, since there is a uniform flow direction at Bago River Bridge location, it is not necessary to be applied.	
Cost Ratio of Substructure*1/	0.97	
Evaluation	Less Recommended	

*Note: *1/ Construction cost of substructure includes costs of pier column and foundation. Minimum size of foundation depending on the pier column shape is assumed for rough cost estimate.*

Source: JICA Study Team

4.3.2.4 Width of the Pier Column

(1) Width of the Pier Column at the Transverse Direction

The width of the pier column at the transverse direction will be determined by the girder layout of the superstructure and not by structural analysis.

Since the girder cross section is modified from the F/S as explained in Section 4.3.1.1, the width of the pier column can be adjusted to 17 m from 20 m at the top and to 11 m from 14 m at the bottom. A width of 17 m shall meet the requirement of the minimum distance (76 cm) between the anchor bolt of the bearing shoe and the edge of the pier head, and adequate space of jack-up of girders for maintenance is also considered.

(2) Width of the Pier Column at the Longitudinal Direction

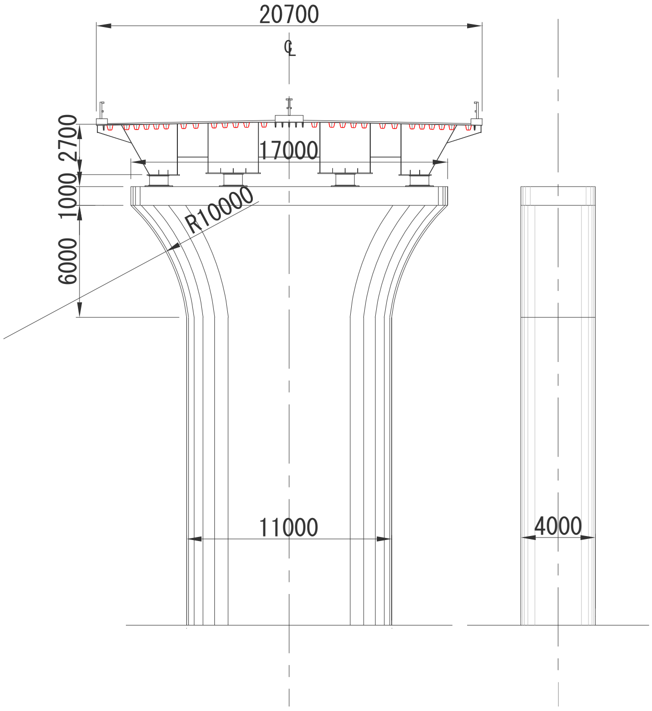
The width of the pier column at the longitudinal direction will be determined by structural analysis. Although width of 3.5 m is possible to be applied as shown in Table 4.3.11, a width of 4.0 m is proposed taking into account the difficulty of the high grade re-bar procurement and workability of triple re-bar arrangement in the cofferdam.

Table 4.3.11 Results of the Structural Analysis of the Pier Column

Case	Material	Structural Analysis
Width 4.5 m (planned in the F/S)	SD345, D38, double re-bar arrangement, σ_{ck} = 24 N/mm^2	Rebar σ s 278 < σ sa 300 N/mm^2 (OK) Concrete σ c 11 < σ ca 12 N/mm^2 (OK)
Width 4.0 m (**Recommended** in the B/D)	SD390, D38, double re-bar arrangement, σ ck = 30 N/mm^2	Rebar σ s 320 < σ sa 345 N/mm^2 (OK) Concrete σ c 12.6 < σ ca 15 N/mm^2 (OK)
Width 3.5 m	SD490, D35, triple re-bar arrangement, σ ck = 30 N/mm^2	Rebar σ s 358 < σ sa 435 N/mm^2 (OK) Concrete σ c 14 < σ ca 15 N/mm^2 (OK)
Width 3.0 m	SD490, D35, triple re-bar arrangement, σ ck = 30 N/mm^2	Rebar σ s 426 < σ sa 435 N/mm^2 (OK) Concrete σ c 17 > σ ca 15 N/mm^2 (NG)

Source: JICA Study Team

As a result of the study above, the width of the pier column is determined as shown in Figure 4.3.18 below.



Source: JICA Study Team

Figure 4.3.18 Width of the Pier Column

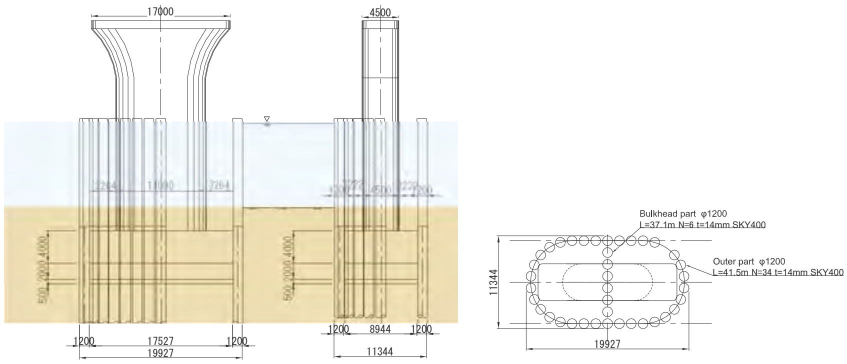
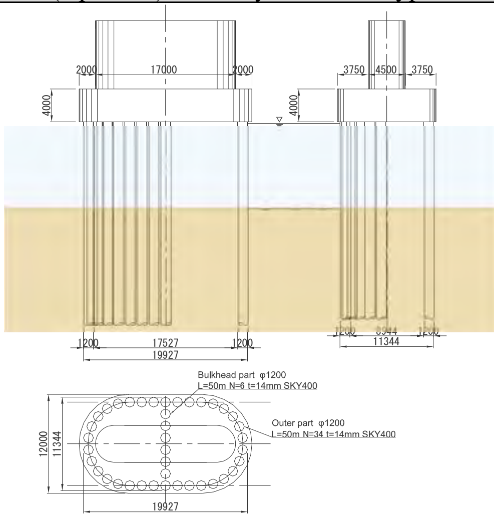
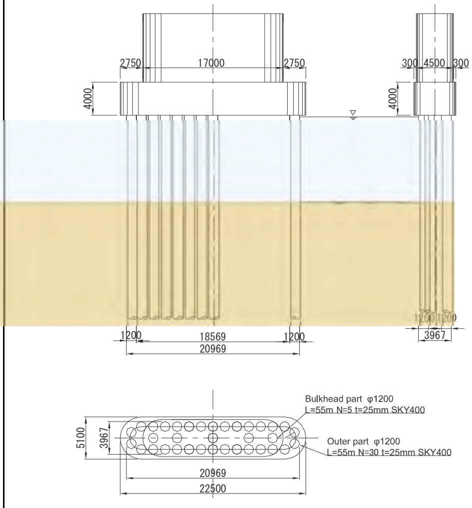
4.3.2.5 Construction Method of SPSP Foundation

There are three construction methods, namely: permanent foundation and temporary cofferdam method (Alt-A), permanent foundation method (Alt-B), and temporary cofferdam method (Alt-C). The third one is the construction of footing and pier inside the steel pipe pile using a temporary cofferdam and it is usually applied in a shallow river. Accordingly, for the foundation of the steel box girder bridge, Alt-A and Alt-B were comparatively studied. Alt-B has two options; one is ordinary structure which has a large dimension so that it can reduce horizontal displacement, and the other is a slender structure which can reduce the influence on water flow but requires reinforcing shear capacity of interlocking.

Option-1 of Alt-B, permanent foundation method, has advantages of moderate construction cost and shorter construction period. However, it is not recommended because of the obstruction of the water flow and passing vessels and faster corrosion of the steel pile that is assumed at 0.1 - 0.3 mm per year. Although, Option-2 of Alt-B can improve influence on the water flow and vessels by slender structure, construction cost is the highest due to longer pile length with high shear capacity of interlocking and thicker steel pipe and same issue about corrosion as in Option-1 shall be unavoidable unless countermeasure against corrosion such as lining is applied.

Finally, Alt-A, permanent foundation and temporary cofferdam method, is selected because of less influence on the water flow and passing vessels and the lowest construction cost. The comparison among the alternatives is referred to in the following table.

Table 4.3.12 Comparison of the Construction Method of SPSP

<table>
<tr><th>Item</th><th colspan="2">Alt-A: Permanent Foundation and Temporary Cofferdam Method</th></tr>
<tr><td>Schematic</td><td colspan="2">
</td></tr>
<tr><td>Outline of Construction Method</td><td colspan="2">SPSP is used in both permanent foundation and temporary cofferdam. First, SPSP is constructed up to the water elevation, filling the material into interlocking that prevents the passage of water. After constructing the footing and pier, the cofferdam will be cut and pulled up by operations.</td></tr>
<tr><td>Steel Pipe Requirement</td><td colspan="2">Outer part: Dia.1.2 m, Length 41.5 m, thickness t = 14 mm, SKY400, nos. 34
Bulkhead part: Dia.1.2 m, Length 37.1 m, thickness t = 14 mm, SKY400, nos. 6</td></tr>
<tr><td>Advantage</td><td colspan="2">・ Less influence on the water flow and passing vessels
・ Lowest construction cost (cost ratio 1.00)
・ Most popular method of SPSP</td></tr>
<tr><td>Disadvantage</td><td colspan="2">・ Longer construction period</td></tr>
<tr><td>Evaluation</td><td colspan="2">Most Recommended</td></tr>
<tr><th>Item</th><th>Alt-B: Permanent Foundation Method
(Option-1) Ordinary Structure Type</th><th>Alt-B: Permanent Foundation Method
(Option-2) Slender Structure Type</th></tr>
<tr><td>Schematic</td><td>
</td><td>
</td></tr>
<tr><td>Outline of Construction Method</td><td colspan="2">SPSP structure is used in permanent foundations only. Footing and pier will be constructed after installing the pile up to the water level. It is usually applied in river areas or sea ports with unrestricted section of flow and clearance for ships crossing.</td></tr>
<tr><td>Steel Pipe Requirement</td><td>Outer part: Dia.1.2 m, Length 50 m, thickness t = 14 mm, SKY400, nos. 34
Bulkhead part: Dia.1.2 m, Length 50 m, thickness t = 14 mm, SKY400, nos. 6</td><td>Outer part: Dia.1.2 m, Length 55 m, thickness t = 25 mm, SKY400, nos. 30
Bulkhead part: Dia.1.2 m, Length 55 m, thickness t = 25 mm, SKY400, nos. 5
・ Analyzed by an imaginary well beam method
・ High shear stiffness and shear capacity of interlocking shall be used.</td></tr>
</table>

Advantage	・ Shorter construction period because no excavation and no footing inside temporary cofferdam ・ Moderate construction cost including countermeasure against corrosion (ratio *1.04*)	・ Less influence on the water flow and passing vessels ・ Shortest construction period
Disadvantage	・ Relatively obstruct the water flow and passing vessels ・ Acceleration of corrosion of the steel part which repeats emerging from water is concerned. Corrosion speed is assumed at 0.1 - 0.3 mm per year. ・ Inferior landscape	・ Highest construction cost including countermeasure against corrosion (ratio *1.26*) ・ Acceleration of corrosion of the steel part which is repeatedly submerged in the water is concerned. Corrosion speed is assumed at 0.1 - 0.3 mm per year. ・ Inferior landscape
Evaluation	Not Recommended	Less Recommended

Source: JICA Study Team

4.3.2.6 Shape and Size of SPSP

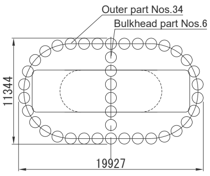
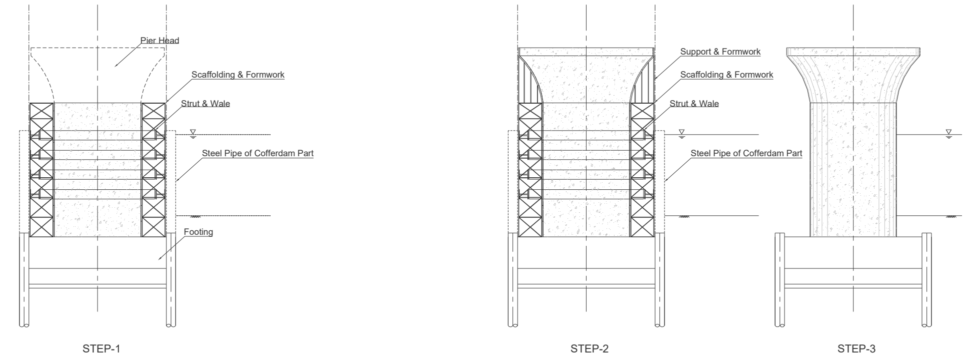
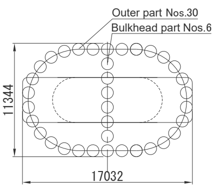
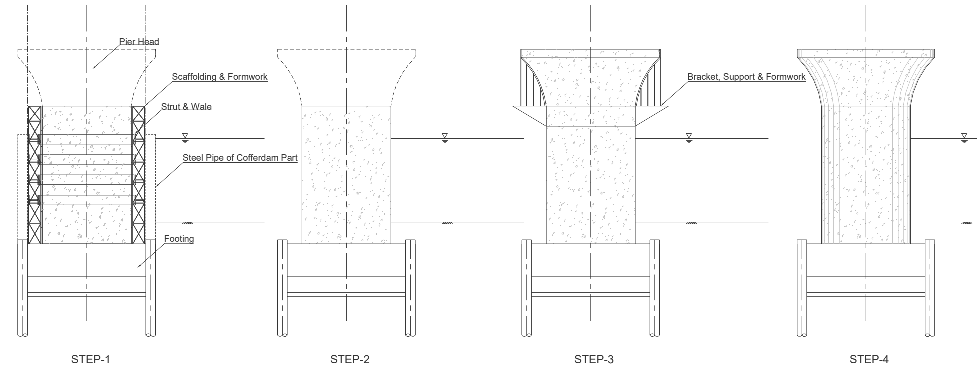
Before studying the shape and size of SPSP, the construction step should be examined first because it will affect the size of the foundation.

(1) Construction Steps

Standard construction step of SPSP in the permanent foundation and temporary cofferdam method, which is proposed in Section 4.3.2.5, is that the cofferdam part will be cut and pulled up by the operations after constructing the footing and pier column (Alt-A). In order to pull up the steel pipes, the steel pipes of the cofferdam shall be arranged outside the pier head. This causes larger dimension of SPSP foundation.

In order to reduce the size of the SPSP foundation, another step, i.e., the steel pipes of cofferdam part will be pulled up after constructing the pier column except the pier head (Alt-B), is comparatively studied and shown in Table 4.3.13 below.

Table 4.3.13 Comparison of the Construction Steps in Removing the Cofferdam Part of the SPSP

<table>
<tr><th>Alternative</th><th colspan="5">Construction Step</th></tr>
<tr><td rowspan="2">Alt-A:
Standard Construction Step

Cross section
(size 19.9 m x 11.3 m, total pipe nos.40)</td><td colspan="5"></td></tr>
<tr><td>Concreting pier column except pier head</td><td colspan="2"></td><td>Concreting pier head, then dismantle scaffolding, strut. After backfill inside cofferdam, pull up steel pipes.</td><td>Complete</td></tr>
<tr><td rowspan="2">Evaluation:
Less Recommended</td><td>Advantage</td><td colspan="4">Pier column is continuously able to allow concreting inside cofferdam.</td></tr>
<tr><td>Disadvantage</td><td colspan="4">Larger dimension of SPSP foundation,
Higher construction cost and longer construction period</td></tr>
<tr><td rowspan="2">Alt-B
Construction Step for reducing size of SPSP

Cross section
(size 17.0 m x 11.3 m, total pipe nos.36)</td><td colspan="5"></td></tr>
<tr><td>Concreting pier column except pier head</td><td colspan="2">Dismantle scaffolding, strut. Then, backfill inside cofferdam and pull up steel pipes.</td><td>Install bracket, support and formwork for concreting pier head</td><td>Complete</td></tr>
<tr><td rowspan="2">Evaluation:
Most Recommended</td><td>Advantage</td><td colspan="4">Can reduce size of SPSP foundation and construction cost
Shorten construction period because of fewer number of pipe installation
Removed pipes can be utilized as temporary bent for superstructure erection earlier than Alt-A.</td></tr>
<tr><td>Disadvantage</td><td colspan="4">None</td></tr>
</table>

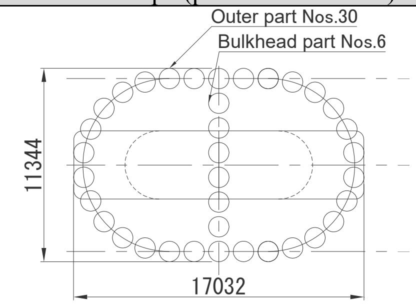
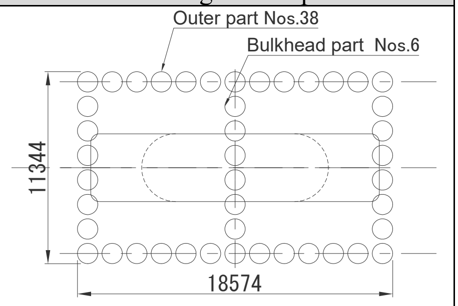
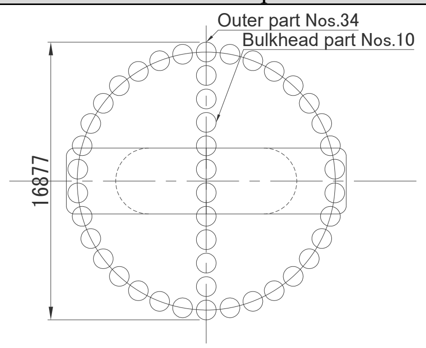
Source: JICA Study Team

Alt-B (construction step for reducing size of SPSP) is recommended because it can reduce the size of the SPSP foundation and is also superior in terms of construction cost and period.

(2) Shape and Size of the SPSP Foundation

The shapes of the SPSP foundation, i.e., oval, rectangular, and round, are comparatively studied as shown in the table below. Dimensions of each shape type are determined as minimum size to keep a temporary work space, more than 1.5 m distance between pier column surface and steel pipe inside surface, and all types have adequate structural capacity. Oval shape is proposed to be applied because it is superior in terms of economic aspect, and has a shorter construction period.

Table 4.3.14 Shape and Dimension of the SPSP Foundation

Comparative Item	Oval Shape (planned in the F/S)	Rectangular Shape
Schematic		
Dimension Outer Pipe Bulkhead Pipe	17.0 m x 11.3 m Nos. 30 x 41.5 m Nos. 6 x 37.1 m	18.5 m x 11.3 m Nos. 38 x 41.5 m Nos. 6 x 37.1 m
Design Result Displacement (cm) Bearing (kN/pile) Stress (N/mm^2)	 3.3 < 5.0 (OK) 1,654 < 6,492 (OK) 175.0 < 210 (OK)	 2.6 < 5.0 (OK) 1,514 < 6,448 (OK) 145.6 < 210 (OK)
Total Weight of Steel Pipes	5,882 kN	7,213 kN
Construction Cost/ Period	Cost ratio 1.00/ Shortest	Cost ratio 1.23/ Longest
Evaluation	**Most Recommended**	Less Recommended
Comparative Item	Round Shape	
Schematic		
Dimension Outer Pipe Bulkhead Pipe	Diameter 16.8 m Nos. 34 x 41.5 m Nos. 10 x 37.1 m	Note: ・ Shape of cross section of pier column is determined as oval type in section 4.3.2.3. ・ The length and the weight of pipes does not include temporary part of steel pipe. ・ The construction cost includes costs of footing, bottom slab, cofferdam and SPSP foundation.
Design Result Displacement (cm) Bearing (kN/pile) Stress (N/mm^2)	 2.3 < 5.0 (OK) 1,404 < 6,468 (OK) 136.4 < 210 (OK)	
Total Weight of Steel Pipes	7,143 kN	
Construction Cost/ Period	Cost ratio 1.13/ Moderate	
Evaluation	Less Recommended	

Source: JICA Study Team

(3) Thickness of Footing

The thickness of the footing is determined based on the bending moment and the shear force. Also, the footing must have enough stiffness to ensure that the connection between the footing and the pier, and between the footing and the pile are rigid. In addition, if the foundation is constructed through the permanent foundation and temporary cofferdam method, then it is required to consider the dimension of the connection with the footing to determine the thickness of the footing. Based on the structural calculation, at least 3.1 m thickness is required taking into account the number of reinforcement stud. Accordingly, the thickness of the footing is proposed at 4.0 m instead of 6.0 m as planned in the F/S.

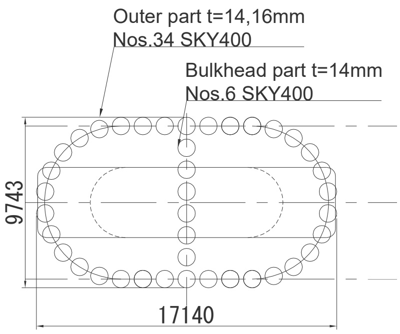
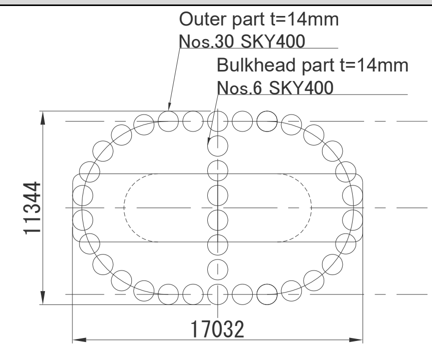
As for the bottom slab by underwater concrete, its thickness in meter can be obtained from the equation of “0.1 x depth (m) $\geqq$1.0” taking account of the temporary cofferdam wall reaction that is nearly in proportionate to the excavation depth, irregularity in underwater excavation, and placement accuracy of underwater concrete, as well as referring to the past construction cases. In this study, the depth is around 20 m. Therefore, the thickness of the bottom slab is proposed at 2.0 m.

When the bottom slab concrete is placed in the weak stratum, a sand layer with a thickness of 0.5-1 m might be provided as enhanced support for the ground. In this study, thickness of the sand mat is assumed at 0.5 m for budgetary purpose. During construction, necessity and thickness of sand mat shall be determined while taking account of the excavated soil condition.

4.3.2.7 Diameter and Thickness of Steel Pipe

For the SPSP foundation with oval shape selected in Section 4.3.2.6, diameter and thickness are comparatively studied between steel pipes with a diameter of 1.0 m and 1.2 m as shown in Table 4.3.15. The optimal one is determined as the steel pipe with a diameter of 1.2 m and thickness of 14 mm because it has the adequate structural capacity, slightly lower construction cost, and shorter construction period.

Table 4.3.15 Comparison of Diameter and Thickness of Steel Pipe

Comparative Item	Dia.1.0 m (planned in the F/S)	Dia.1.2 m
Schematic		
Dimension Outer Pipe*1/ Bulkhead Pipe*1/	17.1 m x 9.7 m Nos. 34 x 29.0 m (t = 16 mm) Nos. 34 x 12.5 m (t = 14 mm) Nos. 6 x 37.1 m (t = 14 mm)	17.0 m x 11.3 m Nos. 30 x 41.5 m (t = 14 mm) Nos. 6 x 37.1 m (t = 14 mm)
Design Result Displacement (cm) Bearing (kN/pile) Stress (N/mm^2)	 3.7 < 5.0 (OK) 1,500 < 5,528 (OK) 187.8 < 210 (OK)	 3.3 < 5.0 (OK) 1,672 < 6,492 (OK) 176.4 < 210 (OK)
Total Weight of Steel Pipes*1/	5,907 kN	5,882 kN
Construction Cost*2/ / Period	Cost ratio *1.01*/ Longer	Cost ratio *1.00*/ Shorter
Evaluation	Less Recommended	**Most Recommended**

*Note: *1/ The length and the weight of pipes do not include the temporary part of the steel pipe.*

**2/ The construction cost includes the costs of footing, bottom slab, cofferdam and SPSP foundation.*

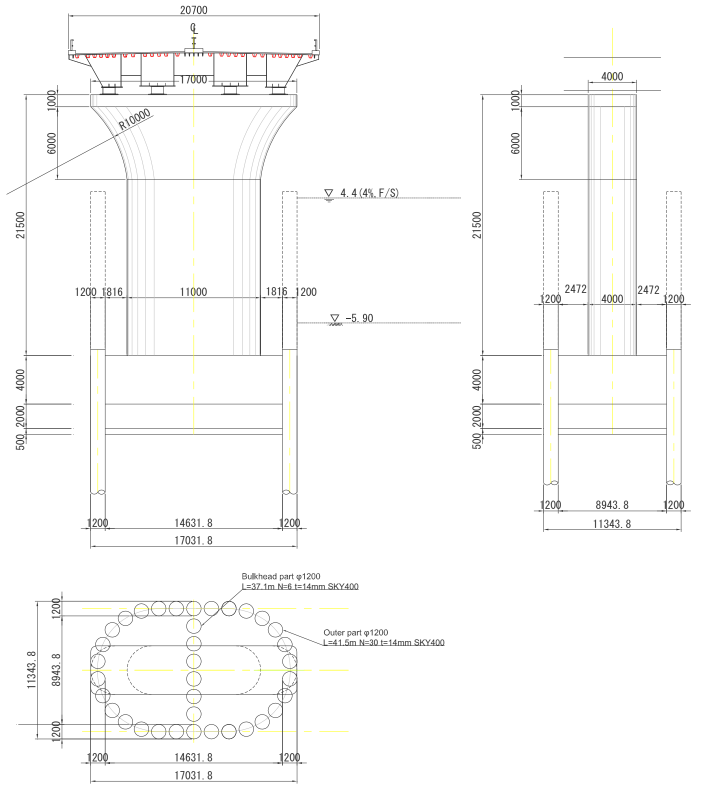
Source: JICA Study Team

4.3.2.8 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 4.3.16 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

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

4.3.3.1 Design Condition

(1) Profile

Span Length:

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

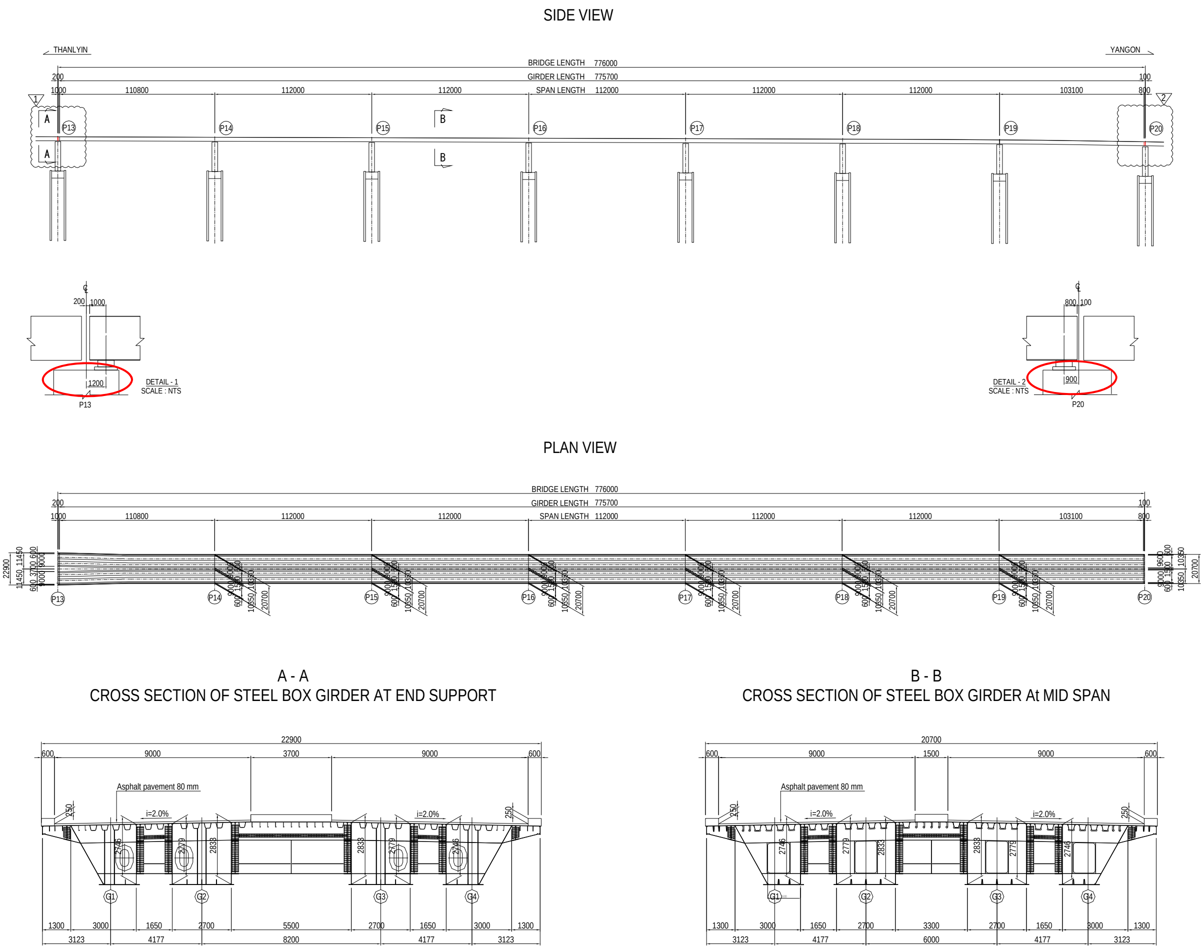
Italicized figures of 1.2 and 0.9 above show the combined length of the clearance and marginal length from the end girder to the bearing position. There has been a slight change for it is longer than the value on the B/D because of the displacement in consideration of the seismic behavior and temperature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



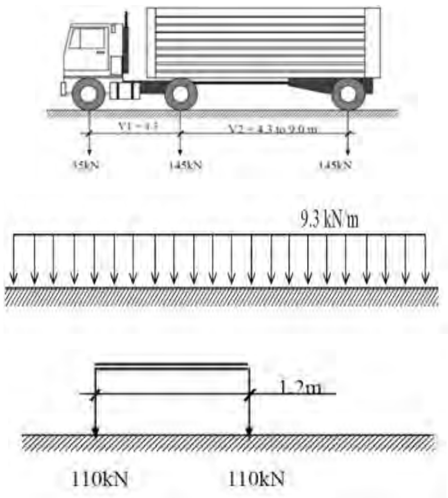
Source: JICA Study Team

Figure 4.3.19 General View

(2) Live Load

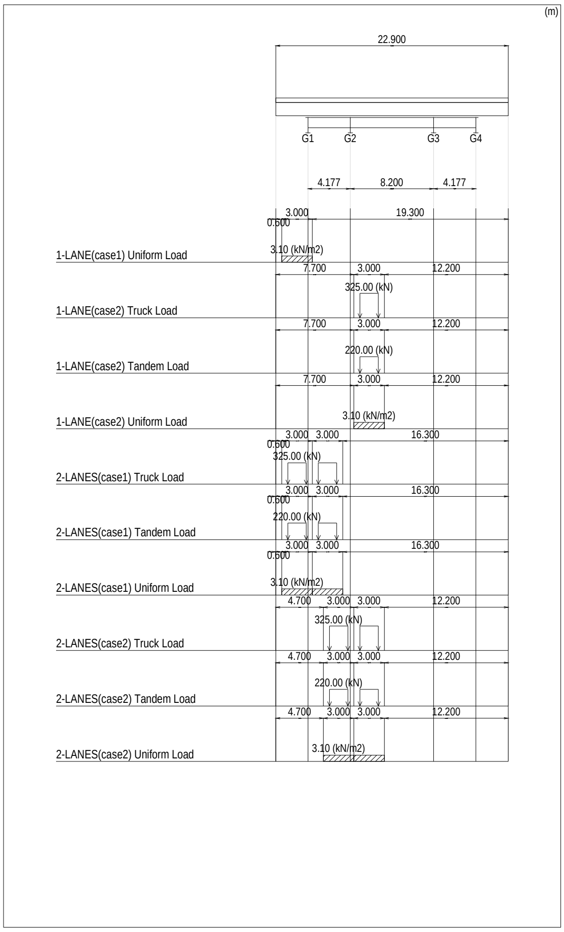
AASHTO load was adopted on the design of the 6-lane carriageways that is different from the actual 4-lane carriageways, and loading lanes were taken in the severest condition.

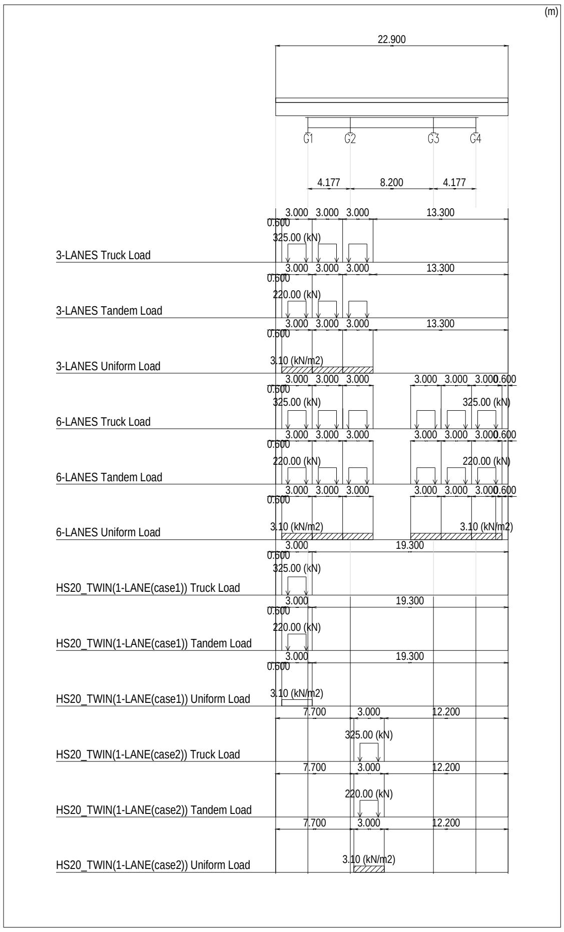
There are three kinds of loading, 1-Truck load, Tandem load, and Uniform Lane load, which are shown in the figure below.



Source: AASHTO specification

Figure 4.3.20 AASHTO Loading





Source: JICA Study Team

Figure 4.3.21 Variations of Loading Position

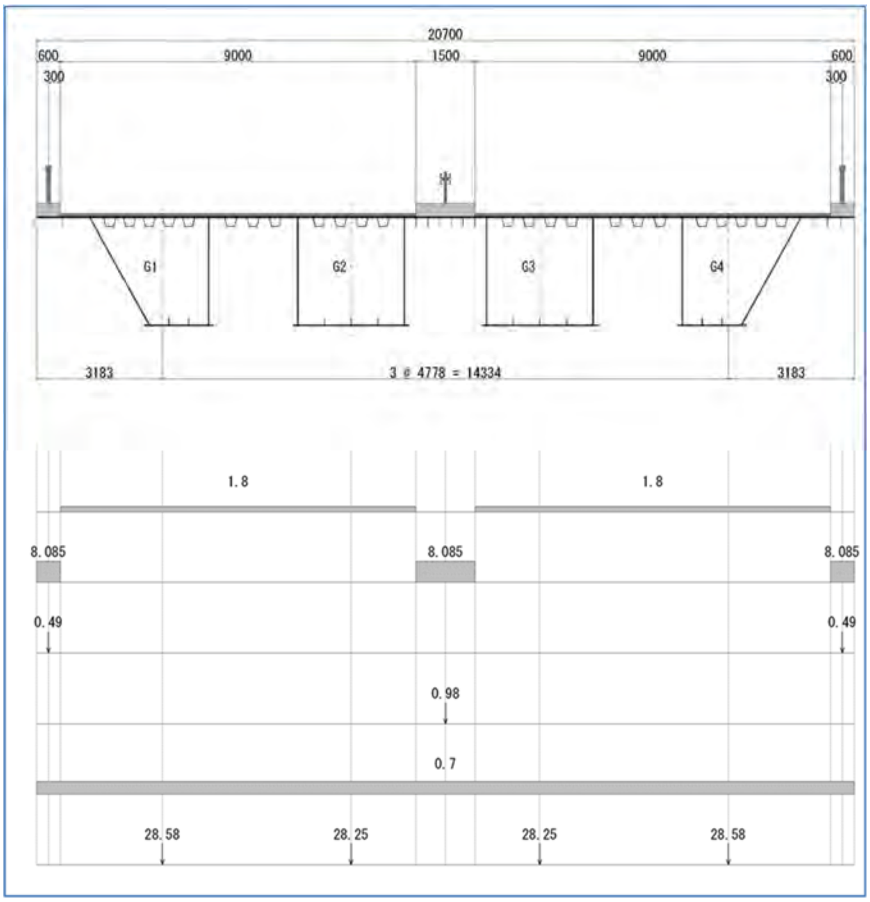
(3) Dead Load

The following items were considered:

- Pavement of asphalt 80 mm thick asphalt laid at whole carriageway
- Coping as wheel guard 330 mm deep concrete casted from steel deck plate
- Railing at side barrier Steel railing weight is assumed.
- Railing at median strip Dual steel railing weight is assumed
- Miscellaneous weight Provisional weight as future overlay load
- Steel weight Assumed in accordance with the girder weight based on B/D

(These weights will be reviewed during the step by step design)

Unit weight of each item is calculated in accordance with its unit volume weight as shown on JSHB.

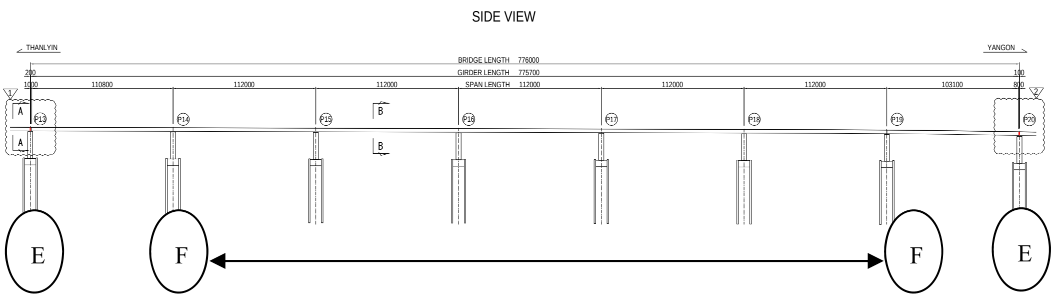


Source: JICA Study Team

Figure 4.3.22 Dead Load Variations

(4) 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 4.3.23 Bearing Support Condition

4.3.3.2 Analysis of the Main Girder

(1) Software for Analysis

- Superstructure was analyzed using the common software named 'APPOLO', which is for Grid Frame Analysis.
- This software consists of 5-steps bridge designing.

1st: Calculating the alignment and coordinates of each line and grid point

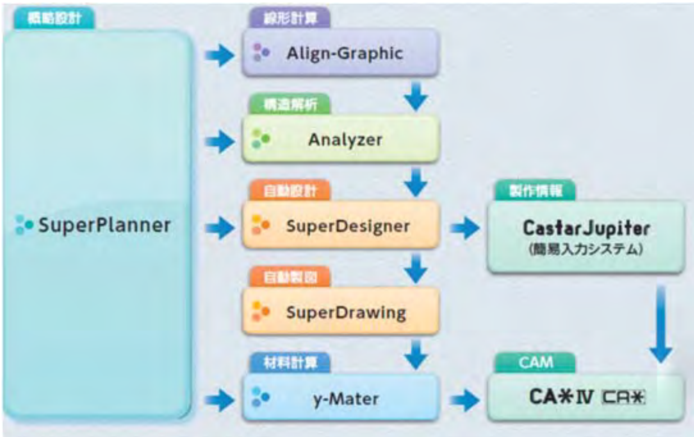
2nd: Analysis of grid frame that is for the purpose of determining design forces of each member.

3rd: Calculating section properties of each member in accordance with JSHB.

4th: Automatically drawing in accordance with determined member section composition.

5th: Quantities calculation

This software system is shown in the following figure.



Source: Catalogue prepared by the software company

Figure 4.3.24 Analysis Flow of the Software

4.3.3.3 Results of the Analysis and the Determined Section Composition

(1) Reaction

Analyzed reaction was reflected into the design of the substructure and the bearing support.

Table 4.3.17 Reaction Components at Each Pier

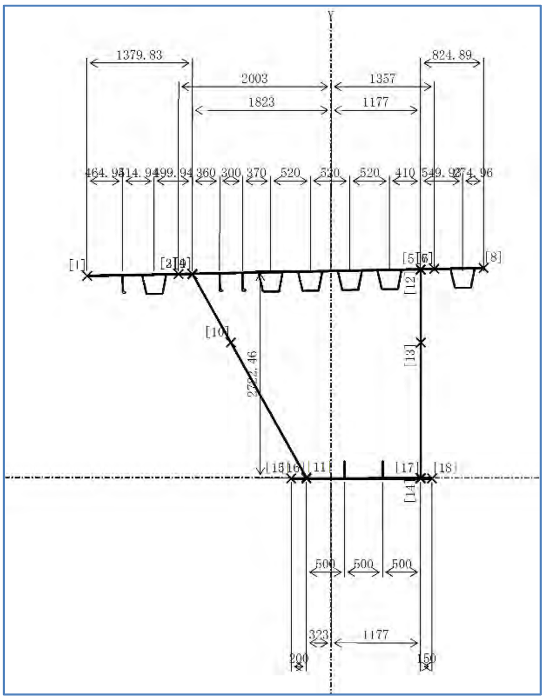
	Reaction Table (unit:Kn)								
		P13	P14	P15	P16	P17	P18	P19	P20
G1 &G4	Pavement	362.1	1,053.7	890.2	953.0	941.5	914.6	1,009.7	337.0
	Side Railing	19.4	52.5	46.3	48.4	47.9	47.0	50.8	18.8
	Side Coping	192.3	519.3	458.4	479.0	474.8	465.4	502.6	185.9
	Steel Girder	1,228.9	3,625.2	3,039.0	3,267.4	3,226.5	3,128.9	3,467.6	1,135.8
	Median coping	116.1	275.9	192.3	223.0	217.0	208.4	237.6	66.7
	Overlay	167.5	478.6	402.5	431.4	426.0	414.0	456.7	152.9
	Median Railing	6.2	20.3	16.1	17.9	17.6	16.8	19.2	5.4
	Total Dead Weight	2,092.5	6,025.6	5,044.7	5,420.0	5,351.3	5,195.2	5,744.3	1,902.5
	Live load with impact (Max)	1,030.3	2,043.6	1,963.6	2,021.3	2,008.9	1,980.2	1,992.3	1,009.9
	Live load with impact (Min)	-338.0	-370.9	-481.5	-466.2	-466.3	-469.4	-367.7	-320.9
	Total Rection(Max)	3,122.8	8,069.2	7,008.3	7,441.3	7,360.2	7,175.3	7,736.6	2,912.4
	Total Rection(Min)	1,754.6	5,654.7	4,563.3	4,953.8	4,884.9	4,725.8	5,376.6	1,581.6
G2&G3	Pavement	334.6	1,007.9	851.4	882.8	876.7	855.9	966.7	299.6
	Side Railing	1.7	9.9	6.4	7.1	7.1	6.5	9.0	0.5
	Side Coping	16.3	98.0	63.2	70.8	69.8	64.8	89.2	4.8
	Steel Girder	1,329.3	3,946.6	3,357.4	3,475.0	3,451.7	3,373.9	3,791.3	1,202.6
	Median coping	330.7	548.0	446.0	468.0	462.5	454.7	502.0	171.6
	Overlay	160.2	447.9	375.2	389.9	387.0	377.8	427.1	131.8
	Median Railing	14.9	42.0	36.6	37.7	37.4	36.7	40.6	13.9
	Total Dead Weight	2,187.6	6,100.3	5,136.1	5,331.3	5,292.2	5,170.3	5,825.9	1,824.8
	Live load with impact (Max)	989.8	1,872.8	1,804.5	1,802.3	1,799.4	1,779.1	1,826.3	909.5
	Live load with impact (Min)	-196.2	-201.6	-319.4	-290.5	-295.9	-299.9	-199.8	-184.3
	Total Rection(Max)	3,177.4	7,973.1	6,940.6	7,133.6	7,091.6	6,949.5	7,652.3	2,734.3
	Total Rection(Min)	1,991.4	5,898.7	4,816.7	5,040.7	4,996.4	4,870.5	5,626.2	1,640.5
Whole Dead Load		8,560.2	24,251.7	20,361.7	21,502.5	21,287.0	20,731.0	23,140.3	7,454.6
Whole Live Load with Impact		4,040.1	7,832.8	7,536.1	7,647.2	7,616.6	7,518.6	7,637.3	3,838.8
Σ	Total	12,600.3	32,084.5	27,897.8	29,149.7	28,903.6	28,249.5	30,777.6	11,293.4
Whole Dead Load		8,560.2	24,251.7	20,361.7	21,502.5	21,287.0	20,731.0	23,140.3	7,454.6
Whole Live Load without Impact		3,593.2	6,971.8	6,707.7	6,806.6	6,779.3	6,692.1	6,797.7	3,395.6
Σ	Total	12,153.3	31,223.5	27,069.4	28,309.1	28,066.4	27,423.1	29,938.1	10,850.2
Whole Dead Load		8,560.2							7,454.6
Whole Live Load without Impact and Truck		3,332.8							3,115.3
Σ	Total	12,153.3							10,850.2

Source: JICA Study Team

(2) Member Force and Section Composition Diagram

The section dimensions and grade of material are determined so that the following criteria are satisfied:

- Each section is designed so that the stress based on bending moment and shearing force shall be within the allowable stress of the adopted material grade.
- The JSHB requires that the deflection due to live load shall be less than 1/500 of span length.
- All block joints are fastened by high strength bolts. Therefore, axial tensile stress at tensile part shall take account of the decreased section area because of the bolt holes. In case that tensile stress would be more than the allowable stress, the thickness of the section should be increased.
- Steel deck plate is stiffened by u-shaped trough ribs, so that torsional stiffness is increased for wheel load.
- Compression stress part of lower flange is stiffened by plate ribs in accordance with thickness of flange. These ribs shall be fastened by high strength bolts at block joint as stress member.
- Web plate is stiffened by horizontal stiffeners at 2-level position, so as decreasing web thickness. These stiffeners only act as stiffeners but not as stress member.
- Required section properties are calculated as follows:



Effective width(mm)		Full width	In-plane
DECK	張出部	1380	1380
	Intermediate	3000	3000
	張出部	825	825
LFLG	Intermediate	1500	1500

Section dimensions	Section area(cm2)	Total	in plane
1-DECK-L	PL 1200 * 16 (SM490Y)	192.0	192.0
1-BULB	PL 230 * 11 (SM490Y)	32.0	32.0
1-U.RIB	320 * 240 * 8 (SM490Y)	53.9	53.9
1-DECK-C	PL 3361 * 16 (SM490Y)	537.7	537.7
2-BULB	PL 230 * 11 (SM490Y)	64.0	64.0
4-U.RIB	320 * 240 * 8 (SM490Y)	215.6	215.6
1-DECK-R	PL 645 * 16 (SM490Y)	103.2	103.2
1-U.RIB	320 * 240 * 8 (SM490Y)	53.9	53.9
1-LWEB	PL 3058 * 12 (SM490Y)	367.0	367.0
1-RWEB	PL 2730 * 12 (SM490Y)	327.6	327.6
1-LFLG	PL 1850 * 20 (SM490Y)	370.0	370.0
2-RIB	PL 220 * 19 (SM490Y)	83.6	83.6

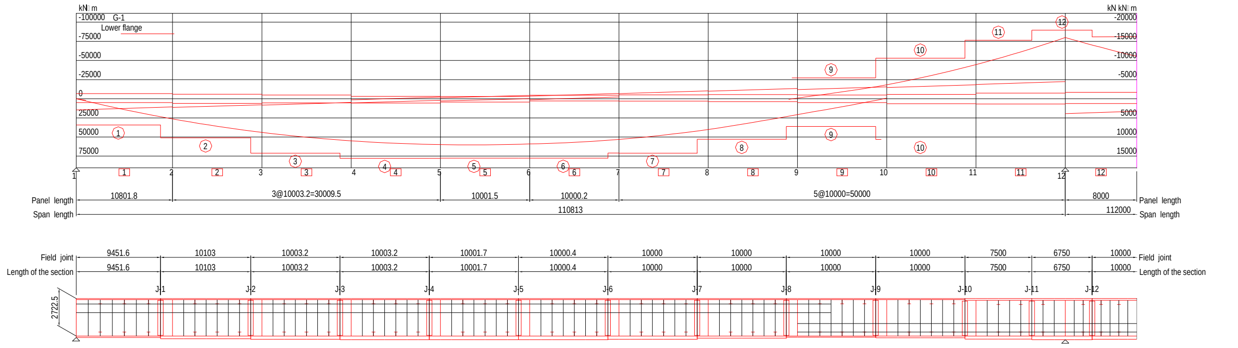
Section property		Total	In-plane
Section area	A (cm2)	2400.5	2400.5
Gravity center	ex (cm)	-22.0	-22.0
	ey (cm)	177.5	177.5
Moment of inertia	Ix (cm4)	29294019	29294019
	Iy (cm4)	40717501	40717501
Torsion Constant	J (cm4)	20180893	

Source: JICA Study Team

Figure 4.3.25 Typical Calculation Sample of the Section

- The thickness and the material grade of all sections have been determined and calculated in accordance with the bending moment.
- The following diagrams show part of the section composition of G1 and G2 girders as example.

STRESS DIAGRAM OF MAIN GIRDER G1 (P13-P20) (1)



Unit: mm N/mm2

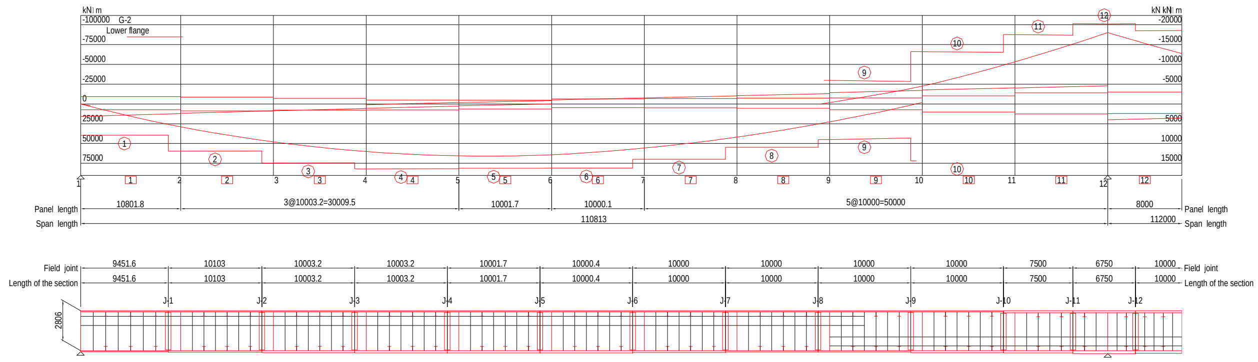
		1		2	3	4	5	6	7	8	9			10			11	12		13		
Section		Sec-1		Sec-2	Sec-3	Sec-4	Sec-5	Sec-6	Sec-7	Sec-8	Sec-9			Sec-10			Sec-11	Sec-12		Sec-13	Section	
Deck Plate	Thickness	16, 16, 16		16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16			22, 22, 22			27, 27, 27	27, 27, 27		27, 27, 27	Thickness	Deck Plate
	Quality	(3),(3),(3)		(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)			(3),(3),(3)			(3),(3),(3)	(3),(3),(3)		(3),(3),(3)	Quality	
Longitu -dinal Rib1	Number	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	3-Bulb	Number	Longitu -dinal Rib1
	Section	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	Section	
Longitu -dinal Rib2	Number	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	Number	Longitu -dinal Rib2
	Section	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	Section	
Left Web	Height	3044.3	3044.2	3044.2	3044.2	3044.2	3044.3	3044.3	3044.3	3044.3	3044.3	3044.3	3044.3	3037.5	3037.5	3037.5	3031.9	3031.9	3031.9	3031.9	Height	Left Web
	Thickness	12(3)		12(3)	12(4)	12(4)	12(4)	12(4)	12(4)	12(4)	12(3)			12(4)			12(4)	14(4)		12(4)	Thickness	
Right Web	Height	2714	2714.1	2714.1	2714.1	2714.1	2714.1	2714	2714	2714	2714	2714	2714	2708	2708	2708	2703	2703	2703	2703	Height	Right Web
	Thickness	12(3)		12(3)	12(4)	12(4)	12(4)	12(4)	12(4)	12(4)	12(3)			12(4)			12(4)	14(4)		12(4)	Thickness	
Lower flange Vertical rib	Number	2		2	2	2	2	2	2	2	5			5			5	5		5	Number	Lower flange Vertical rib
	Width	220		220	220	220	220	220	220	220	220			220			220	220		220	Width	
	Thickness	19(3)		19(3)	19(4)	19(4)	19(4)	19(4)	19(4)	19(4)	19(3)			19(4)			19(4)	19(4)		19(4)	Thickness	
Lflg W=1850 T		20(3)		38(3)	46(8)	52(8)	52(8)	52(8)	46(8)	30(4)	16(3)			24(4)			43(8)	52(8)		47(8)	Lflg W=1850 T	
Deck Plate	σ	0	-80	-132	-165	-180	-181	-178	-165	-136	-79	-7	53	-6	42	111	152	180	180	161	σ	Deck Plate
	σa	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	σa	
	σa-σ	210	130	78	45	30	29	32	45	74	131	203	157	204	168	99	58	30	30	49	σa-σ	
Lower flange	σ	0	145	174	195	197	199	197	196	204	136	12	-91	10	-74	-194	-220	-232	-232	-222	σ	Lower flange
	σa	210	210	210	255	255	255	255	255	255	210	210	158	255	255	255	255	255	255	255	σa	
	σa-σ	210	65	36	60	58	56	58	59	51	74	198	67	245	181	61	35	23	23	33	σa-σ	
Web	τ	60	48	36	25	16	14	19	28	41	53	59	59	59	59	70	81	73	65	73	τ	Web
	τa	120	120	120	145	145	145	145	145	145	120	120	120	145	145	145	145	145	145	145	τa	
	Combined	0.25	0.56	0.72	0.57	0.56	0.57	0.56	0.58	0.67	0.52	0.25	0.32	0.16	0.19	0.71	0.94	0.97	0.93	0.91	Combined	
Calculated points		Left	J-1	J-2	J-3	J-4		J-5	J-6	J-7	J-8	J-9	J-9	Left	Left	J-10	J-11	Max Left	Max Right	J-12	Calculated points	
Stress of Net Area σ																					Stress of Net Area σ	
Lflg σspl																					Lflg σspl	

Grade: (1):SM400
(2):SM490
(3):SM490Y
(4):SM570
(5):SM400-H
(6):SM490-H
(7):SM520-H
(8):SM570-H

Source: JICA Study Team

Figure 4.3.26 Typical Section Composition, G1 (P13-P14)

STRESS DIAGRAM OF MAIN GIRDER G2 (P13-P20) (1)



Unit: mm N/mm2

		1		2	3	4	5	6	7	8	9			10			11	12		13		
Section		Sec-1		Sec-2	Sec-3	Sec-4	Sec-5	Sec-6	Sec-7	Sec-8	Sec-9			Sec-10			Sec-11	Sec-12		Sec-13	Section	
Deck Plate	Thickness	16, 16, 16		16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16			22, 22, 22			27, 27, 27	27, 27, 27		27, 27, 27	Thickness	Deck Plate
	Quality	(3),(3),(3)		(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)	(3),(3),(3)			(3),(3),(3)			(3),(3),(3)	(3),(3),(3)		(3),(3),(3)	Quality	
Longitu -dinal Rib1	Number	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	Number	Longitu -dinal Rib1
	Section	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	Section	
Longitu -dinal Rib2	Number	5-Bulb	5-Bulb	5-Bulb	4-Bulb	4-Bulb	4-Bulb	4-Bulb	4-Bulb	2-Bulb	2-Bulb	2-Bulb	2-Bulb	2-Bulb	2-Bulb	2-Bulb	2-Bulb	2-Bulb	2-Bulb	2-Bulb	Number	Longitu -dinal Rib2
	Section	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	230*11	Section	
Left Web	Height	2747	2746.9	2746.9	2746.9	2746.9	2746.9	2747	2747	2747	2747	2747	2747	2741	2741	2741	2736	2736	2736	2736	Height	Left Web
	Thickness	11(3)		11(4)	11(4)	11(4)	11(4)	11(4)	11(4)	11(4)	11(3)			13(4)			13(4)	13(4)		11(4)	Thickness	
Right Web	Height	2801	2801.1	2801.1	2801.1	2801.1	2801.1	2801	2801	2801	2801	2801	2801	2795	2795	2795	2790	2790	2790	2790	Height	Right Web
	Thickness	11(3)		11(4)	11(4)	11(4)	11(4)	11(4)	11(4)	11(4)	11(3)			13(4)			13(4)	13(4)		11(4)	Thickness	
Lower flange Vertical rib	Number	3		3	3	3	3	3	3	3	3			7			7	7		7	Number	Lower flange Vertical rib
	Width	220		220	220	220	220	220	220	220	220			220			220	240		220	Width	
	Thickness	19(3)		19(4)	19(4)	19(4)	19(4)	19(4)	19(4)	19(4)	19(3)			19(4)			19(4)	19(4)		19(4)	Thickness	
Lflg W=2940 T		14(3)		20(4)	28(4)	32(4)	32(4)	32(4)	26(4)	18(4)	18(3)			27(4)			32(4)	40(4)		35(4)	Lflg W=1850 T	
Deck Plate	σ	0	-73	-128	-165	-185	-186	-185	-167	-140	-81	-5	63	-4	48	123	164	193	193	174	σ	Deck Plate
	σa	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	210	σa	
	σa-σ	210	137	82	45	25	24	25	43	70	129	205	147	206	162	87	46	17	17	36	σa-σ	
Lower flange	σ	0	138	198	203	205	208	205	210	206	118	8	-96	5	-61	-156	-214	-218	-218	-216	σ	Lower flange
	σa	210	210	255	255	255	255	255	255	255	210	210	139	255	255	255	255	255	255	255	σa	
	σa-σ	210	72	57	52	50	47	50	45	49	92	202	43	250	194	99	41	37	37	39	σa-σ	
Web	τ	70	58	44	31	20	17	22	34	44	56	70	70	58	58	71	79	82	75	85	τ	Web
	τa	120	120	145	145	145	145	145	145	145	120	120	120	145	145	145	145	145	145	145	τa	
	Combined	0.34	0.57	0.66	0.65	0.63	0.64	0.63	0.68	0.70	0.45	0.34	0.43	0.16	0.18	0.54	0.91	0.96	0.92	0.98	Combined	
Calculated points		Left	J-1	J-2	J-3	J-4		J-5	J-6	J-7	J-8	J-9	J-9	Left	Left	J-10	J-11	Max Left	Max Right	J-12	Calculated points	
Stress of Net Area σ																					Stress of Net Area σ	
Lflg σspl																					Lflg σspl	

Grade: (1):SM400
(2):SM490
(3):SM490Y
(4):SM570

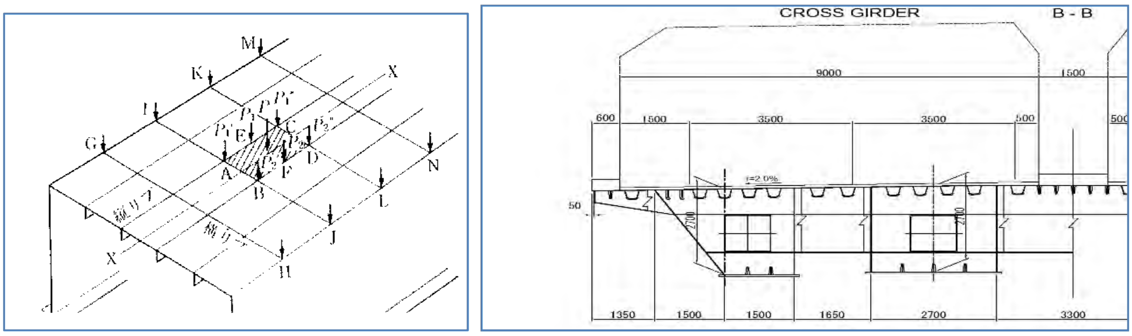
Source: JICA Study Team

Figure 4.3.27 Typical Section Composition, G2 (P13-P14)

4.3.3.4 Analysis of Steel Deck

(1) Design Method of Steel Deck

- The steel deck will be analyzed as equivalent multi-grid frame.
- The model of the grid frame consists of webs, cross girder, crossbeams, and longitudinal stiffeners.



Source: JICA Study Team

Figure 4.3.28 Concept of Wheel Loading on Steel Deck

(2) Stress Evaluation of Steel Deck

- Longitudinal ribs or cross ribs have combined stresses of primary stress as a deck member of the whole main girder and secondary stress as a member of the deck frame.
- The stress of longitudinal rib is of the same direction as the main girder stress. Therefore, this stress shall be combined with the stress of the main girder and shall be within the allowable stress as shown below.

$\sigma 1$: Primary stress as a member of the main girder

$\sigma 2$: Secondary stress as a member of the deck frame

α : Safety factor (1.4 as specified by the JSHB)

σa : Allowable stress of the material deck plate

$$\sigma 1+\sigma 2 < \sigma a \cdot \alpha$$

- If σ2 is smaller than o.4*σa, then the formula above will always be satisfied.
- The stress of the cross rib is in the right angle direction of the stress of the main girder, so the biaxial stress shall be checked.

Biaxial calculation formula:

$$K = (\sigma x / \sigma a)2 - (\sigma x / \sigma a) * (\sigma y / \sigma a) + (\sigma y / \sigma a)2 + (\tau / \tau a)2 \leqq 1.2$$

σ x : Normal stress of the main girder (N/mm^2) τx : Shear stress of the main girder (N/mm^2)

σ y : Normal stress of the crossbeam (N/mm^2) τy : Shear stress of the crossbeam (N/mm^2)

σ a : Allowable tensile stress of the main girder (N/mm^2)

τa : Allowable shear stress of the main girder or crossbeam (N/mm^2)

Where, by checking location:

Flange point (τ / τa) = (τx / τxa)

Web point (τ / τa) = Max((τx/τxa), (τy/Max(τxa,τya)))

(3) Analysis Model of Deck Frame

- Wheel load shall act on the longitudinal stiffener or crossbeam so that the maximum bending moment will occur.
- Vertical ribs are considered as one bar members without any cross sectional deformation. Therefore, the torsional rigidity (using only the simple torsional resistance) is not reduced; it is 100% valid and is calculated by the following formula:

  Torsional rigidity = 4*A2 / {(u/tR) + (a/tP)}

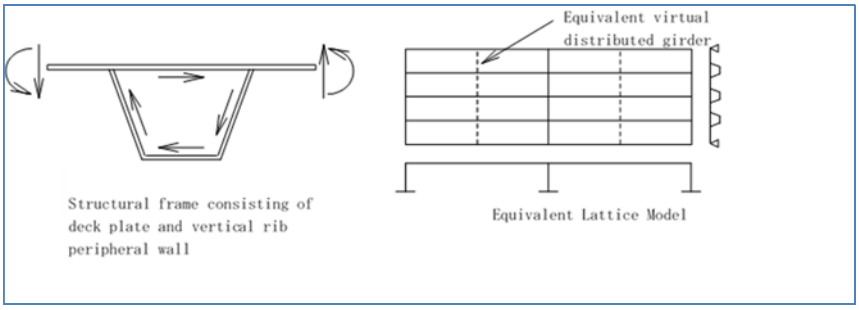
  A : Cross sectional area surrounded by the U-shaped steel

  u : Expanded width of the U-shaped steel

  a : Upper width of the U-shaped steel

  tR: Thickness of the U-shaped steel

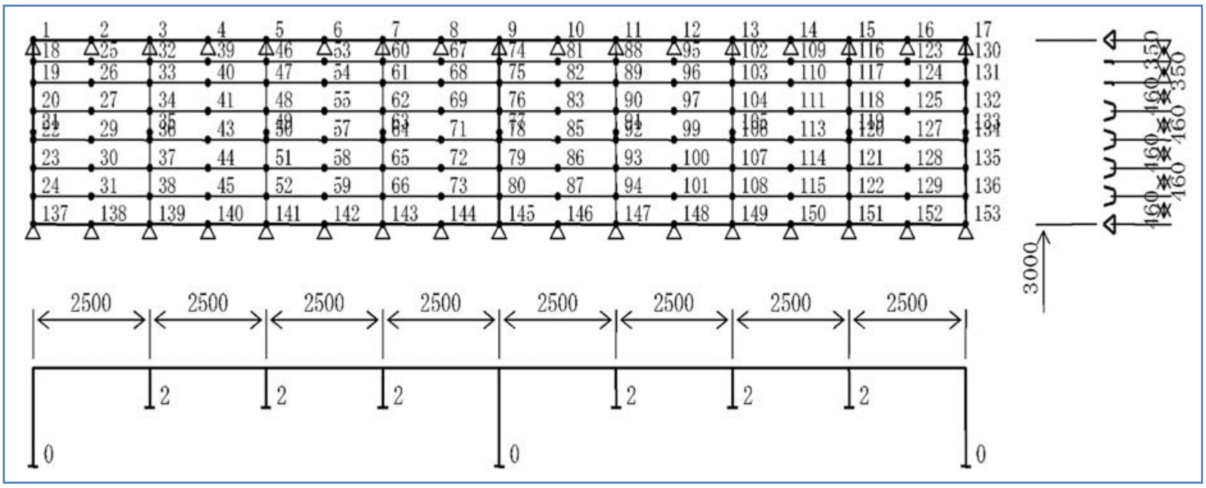
  tP: Thickness of the deck plate
- The virtual distribution girder that performs the load distribution between the longitudinal ribs gives the bending rigidity equivalent to the rigid structure consisting of the deck plate and the longitudinal rib peripheral wall in consideration of the sectional deformation of the longitudinal rib. Since this rigid structure continues in the direction of the bridge axis, the equivalent cross sectional secondary moment per unit length is obtained first, and in the Lattice Model, one distribution girder is provided at the lateral rib intervals to provide bending rigidity.



Source: Analysis manual prepared by the software company

Figure 4.3.29 Concept of Equivalent Virtual Beam of Steel Deck

- There are five deck models, i.e.: side deck, top deck of G1, top deck of G2, median deck, and center deck, that are to be considered.
- The top deck of G2 is shown in the following figure.



Source: JICA Study Team

Figure 4.3.30 Analysis Model of Steel Deck

- Dead load to be considered

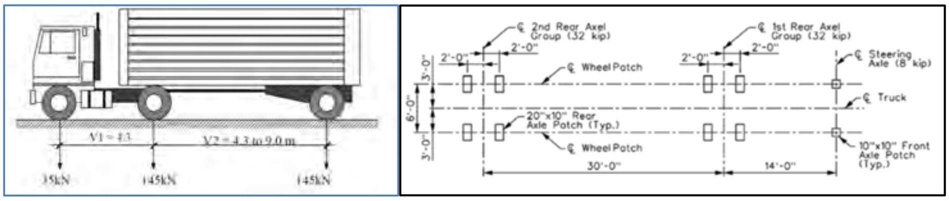
Pavement (road section) 1.80 kN/m^2

Steel weight 2.00 kN/m^2

- Section profile of member

Cross sectional shape		Thickness of deck plate 16 mm	
Longitudinal rib	Sec- 2	U.RIB 320 * 240 * 8	
Lateral rib	Sec- 2	WEB PL 800 * 9	FLG PL 200 * 10
Diaphragm		WEB PL 2100 * 10	FLG PL 220 * 10

- Wheel load shall act on the longitudinal stiffener or crossbeam so the maximum bending moment will occur.
- Distance of the wheel is 1.8 m, and contact area of each wheel is 510 mm wide and 250 mm long.



Source: AASHTO specification

Figure 4.3.31 Wheel Load to be Considered on Steel Deck

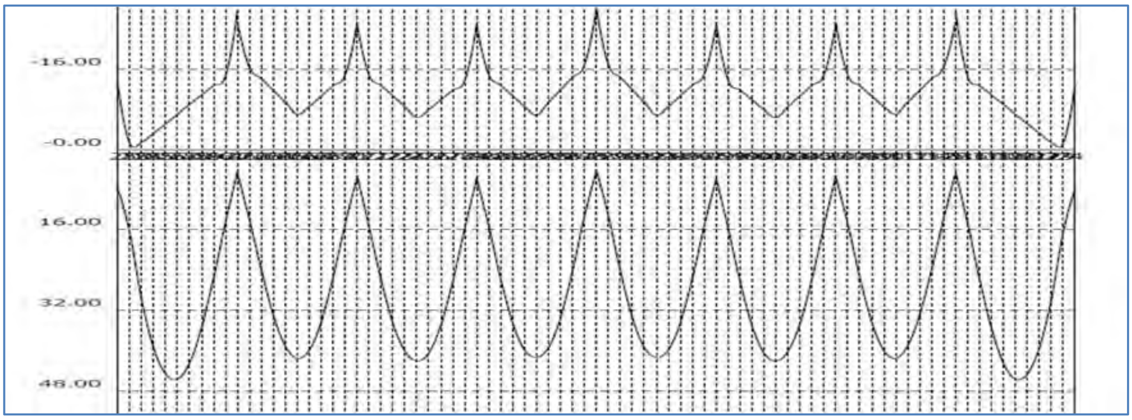
- Impact coefficient is based on the following formula which is based on JSHB:

Longitudinal rib i = 0.4

Lateral rib & bracket i = 20/(50+L) L: Span length of lateral rib and bracket (m)

(4) Diagram of Bending Moment

- The maximum/minimum bending moment of the longitudinal rib was calculated based on influence line and area.
- The critical moment occurred at the mid span.



Source: JICA Study Team

Figure 4.3.32 Bending Moment Diagram

(5) Analysis Results of Each Rib Force

Table 4.3.18 Table of the Member Force

(a) List of Cross-sectional Force

Member	Cross-section	Case of interest	Point of interest	Load type	Bending moment (kN・m)	Shear force (kN)
Longitudinal rib	Sec-2	At Max. bending	45	Dead load	0.73	0.05
				T - Load	43.43	-57.88
				Total	44.16	-57.83
				Additional Total	44.16	-57.83
		At Min. bending	40	Dead load	-0.93	2.61
				T - Load	-23.97	13.43
				Total	-24.90	16.05
				Additional Total	-24.90	16.05
		At Max. shear	67	Dead load	-1.01	2.70
				T - Load	0.18	107.48
				Total	-0.83	110.18
				Additional Total	-0.83	110.18
Lateral rib	Sec-2	At Max. bending	39	Dead load	8.89	-5.52
				T - Load	110.82	0.88
				Total	119.71	-4.64
				Additional Total	95.77	-4.64
		At Min. bending	77	Dead load	7.95	4.92
				T - Load	-14.20	-9.31
				Total	-6.25	-4.39
				Additional Total	-95.77	-4.39
		At Max. shear	3	Dead load	0.00	11.05
				T - Load	0.00	195.64
				Total	0.00	206.68
				Additional Total	0.00	206.68

Source: JICA Study Team

Table 4.3.19 Table of the Deflection due to Live Load

(b) List of Deflection due to Live Load

Member	Cross-section	Point of interest	Load type	Deflection (mm)
Longitudinal rib	Sec-2	44	T - Load	0.63
Lateral rib	Sec-2	38	T - Load	0.15

Source: JICA Study Team

(6) Stress Calculation of the Longitudinal Rib

- A two-type rib, that is a bulb plate rib or a U-shaped rib, is adopted.
- Each rib is calculated as a beam structure, so that stress will be less than allowable stress.
- The typical calculation procedure is shown in the following figure.

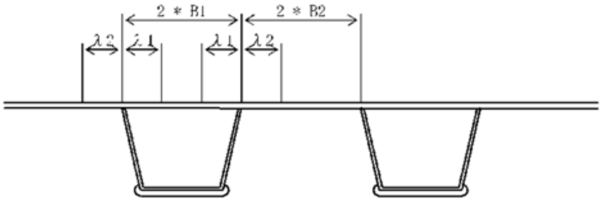
(2) Model-3 Sec-2 (Supporting Part)

Sectional Force and Conditions of the Calculation

For sectional calculation $M = -26.10$ kN·m $S = 110.62$ kN

For spliced (attachment/joint) part $M_j = -0.63$ kN·m $S_j = 110.62$ kN

Calculation of Effective Width



＊Equivalent span length $L = 0.6 * LC = 0.6 * 250.0 = 150.0$ cm

LC : Lateral rib interval (Longitudinal rib span length)

$B1 = 16.2$ cm

$B2 = 11.3$ cm

$B1 / L = 16.2 / 150.0 = 0.11$

$\lambda 1 = \{1.06 - 3.2 * (B1 / L) + 4.5 * (B1 / L)^2\} * B1$

$= \{1.06 - 3.2 * (0.11) + 4.5 * (0.11)^2\} * B1$

$= 12.4$ cm

$B2 / L = 11.3 / 150.0 = 0.08$

$\lambda 2 = \{1.06 - 3.2 * (B2 / L) + 4.5 * (B2 / L)^2\} * B2$

$= \{1.06 - 3.2 * (0.08) + 4.5 * (0.08)^2\} * B2$

$= 9.5$ cm

＊Total effective width

$\lambda = (\lambda 1 + \lambda 2) * 2 = (12.4 + 9.5) * 2$

$= 43.9$ cm

Sectional Quantities

		$A(cm^2)$	$Y(cm)$	$AY(cm^3)$	$I(cm^4)$
1-DECK	PL 439 * 16 (SM400)	70.30	-16.01	-1125.5	18019.3
1-U.RIB	320 * 240 * 8 (SM400)	53.90	0.00	0.0	3315.0
		124.20		-1125.5	21334.3

$e = -1125.5 / 124.20 = -9.06$ cm

$I = 21334.3 - 124.20 * -9.06^2 = 11135$ cm^4

$Y_u = -7.75$ cm, $Y_L = 18.05$ cm

$W_u = -1437$ cm^3, $W_L = 617$ cm^3

Actual Stress

Bending stress

$\sigma_u = -26.10 * 10^6 / (-1437 * 10^3) = 18$ N/mm² $< \sigma_{ta} = 140$ N/mm²

$\sigma_L = -26.10 * 10^6 / (617 * 10^3) = -42$ N/mm² $< \sigma_{ca} = 140$ N/mm²

σ <0.4・ σ a

Shear stress

$\tau = 110.62 * 10^3 / (38.7 * 10^2) = 29$ N/mm² $< \tau_a = 80$ N/mm²

Combined stress

$\kappa = (42 / 140)^2 + (29 / 80)^2 = 0.22 < 1.2$

Source: JICA Study Team

Figure 4.3.33 Sample Calculation of the Longitudinal Rib

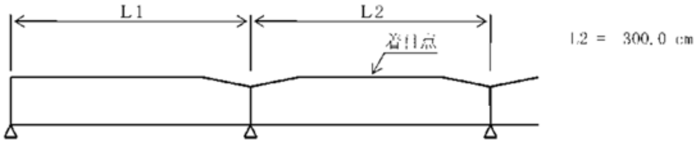
(7) Stress Calculation of the Cross Rib

- Cross ribs are a kind of elastic support member for longitudinal ribs.
- The typical calculation procedure is shown in the following figure.

Sectional Force and Conditions of the Calculation

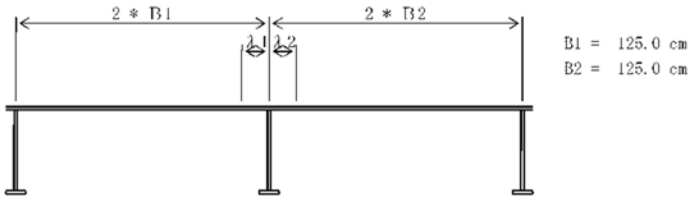
Section force	M =	112.3 kN·m	S =	16.6 kN
Shear force of missing part	S_k =	16.6 kN		
Distance between the fixed points	L =	3.000 m		

Equivalent Span Length



Li = 0.6 * L2 = 0.6 * 300.0 = 180.0 cm

Effective Width



B1/Li – 125.0 / 180.0 – 0.69

$\lambda 1$ = 0.15 * Li = 0.15 * 180.0 = 27.0 cm

B2/Li = 125.0 / 180.0 = 0.69

$\lambda 2$ = 0.15 * Li = 0.15 * 180.0 = 27.0 cm

Total effective width $\lambda = \lambda 1 + \lambda 2$ = 27.0 + 27.0 = 54.0 cm

Source: JICA Study Team

Figure 4.3.34 Sample Calculation of the Effective Width of the Cross Rib

Sectional Area and Moment of Inertia

			A(cm^2)	Y(cm)	AY(cm^3)	I(cm^4)
1-DECK	PL	540 * 16(SM400)	86.40	-40.80	-3525	143825
1-WEB	PL	800 * 9(SM400)	72.00	0.00	0	38400
1-LFLG	PL	200 * 10(SM400)	20.00	40.50	810	32805
			178.40		-2715	215030

E = -2715/ 178.40 = -15.22 cm

I = 215030 - 178.40 * -15.22^2 = 173708 cm^4

Y_u = -26.38 cm , Y_L = 56.22 cm

Bending stress

$\sigma_u = 112.3*10^6 * -263.8 / (173708*10^4) = -17\ N/mm^2 < \sigma_{ca} = 140\ N/mm^2$

$\sigma_L = 112.3*10^6 * 562.2 / (173708*10^4) = 36\ N/mm^2 < \sigma_{ta} = 140\ N/mm^2$

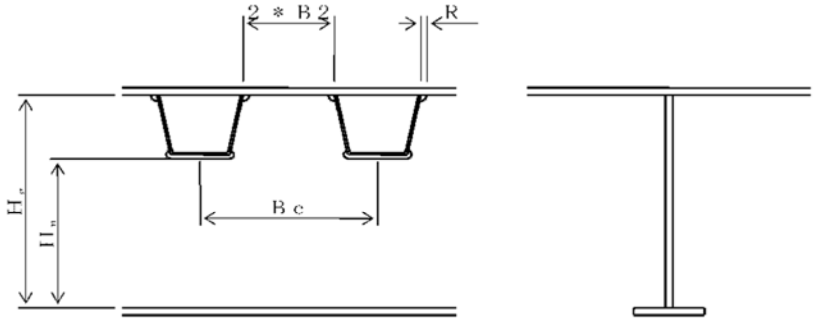
Shear stress

$\tau = 16.6*10^3 / 7200 = 2\ N/mm^2 < \tau_a = 80\ N/mm^2$

Combined stress

$\kappa = (36 / 140)^2 + (2 / 80)^2 = 0.07 < 1.2$

Shear Stress of Vertical Rib-Missing Part



$\tau_k = 16.6*10^3 / 7200 = 2\ N/mm^2 < \tau_a = 80\ N/mm^2$

$\tau_v = \tau_k * H_g / H_n = 2 * 80.0 / 52.3 = 4\ N/mm^2 < \tau_a$

Verification of Deflection

Deflection due to live load $\delta = 0.2$ mm $\leqq \delta_a = L / 500 = 3000 / 500 = 6.0$ mm

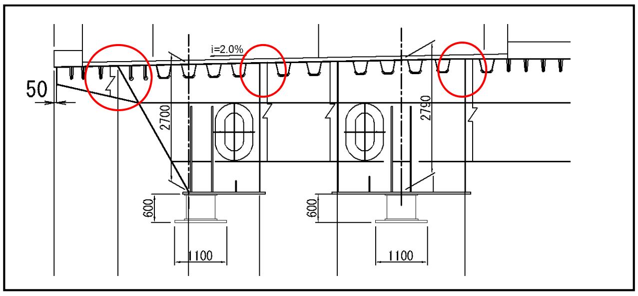
Source: JICA Study Team

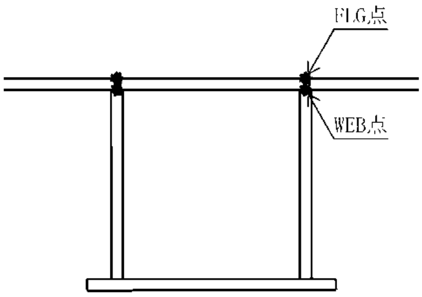
Figure 4.3.35 Sample Calculation of the Cross rib

(8) Biaxial Stress at the Cross Point of Both Webs of the Main Girder and Cross Rib

Calculation procedure of converted stress is as the follows.

- The cross point at both webs is shown in the following figure. This point has biaxial stress, main girder stress, and cross rib stress. These stresses are orthotropic respectively.





- The checking location is shown by the black dot in the sketch above.
- The maximum value is based on the checking of the right and left sides of the node point respectively, and of positive and negative stress, respectively.
- Biaxial calculation formula is shown as follows:

$$K = (\sigma x / \sigma a)2 - (\sigma x / \sigma a) * (\sigma y / \sigma a) + (\sigma y / \sigma a)2 + (\tau / \tau a)2 \leqq 1.2$$

σx : Normal stress of the main girder (N/mm^2)

σy : Normal stress of the crossbeam (N/mm^2)

σa : Allowable tensile stress of the main girder (N/mm^2)

τx : Shear stress of the main girder (N/mm^2)

τy : Shear stress of the crossbeam (N/mm^2)

τa : Allowable shear stress of the main girder or crossbeam (N/mm^2)

Where, by checking location,

FLG point $(\tau / \tau a) = (\tau x / \tau xa)$

WEB point $(\tau / \tau a) = \mathrm{Max}((\tau x/\tau xa), (\tau y/\mathrm{Max}(\tau xa,\tau ya)))$

- The stress of the main girder and cross ribs are calculated at every cross point.
- The calculation result is shown the following table.

Table 4.3.20 Calculation Result of the Biaxial Stress Check

(3) Main girder G-2 Web name : LWEB(UFLG-side)

	Cross section No.	No.	Check P't	σx	σy	σa	τx	τy	τa	K
End. sup.	1	1	FLG	0	-54	210	33	0	120	0.14
			WEB	0	53	210	60	24	120	0.32
	2	2	FLG	-71	22	210	26	0	120	0.21
			WEB	-70	21	210	48	8	145	0.26
	3	3	FLG	-120	22	210	19	0	120	0.42
			WEB	-118	21	210	35	8	145	0.44
	4	4	FLG	-154	26	210	15	0	120	0.66
			WEB	-152	26	210	26	12	145	0.66
	5	5	FLG	-174	26	210	8	0	120	0.81
			WEB	-172	26	210	14	12	145	0.79
	6	6	FLG	174	30	210	12	0	120	0.84
			WEB	-172	30	210	22	14	145	0.83
	7	7	FLG	-158	22	210	16	0	120	0.68
			WEB	-156	21	210	31	8	145	0.69
	8	8	FLG	-123	26	210	22	0	120	0.46
			WEB	-121	26	210	41	12	145	0.50
	9	9	FLG	-66	34	210	28	0	120	0.23
			WEB	65	34	210	54	14	120	0.37
	10	10	FLG	64	-30	210	33	0	120	0.23
			WEB	63	-30	210	54	14	145	0.29
	11	11	FLG	105	-16	210	24	0	120	0.33
			WEB	102	-15	210	45	12	145	0.37
Int. sup.	12	12	FLG	176	-22	210	26	0	120	0.85
			WEB	172	-22	210	49	19	145	0.88
	13	13	FLG	125	16	210	22	0	120	0.44
			WEB	122	-15	210	41	12	145	0.46
	14	14	FLG	119	-26	210	33	0	120	0.48
			WEB	117	-26	210	63	12	145	0.58
	15	15	FLG	56	-30	210	26	0	120	0.18
			WEB	55	-30	210	50	14	120	0.30
	16	16	FLG	-53	34	210	21	0	120	0.16
			WEB	-52	34	210	39	14	120	0.23
	17	17	FLG	-89	34	210	15	0	120	0.29
			WEB	87	34	210	28	14	120	0.32
	18	18	FLG	-103	34	210	9	0	120	0.35
			WEB	-102	34	210	16	14	120	0.36
	19	19	FLG	-98	30	210	12	0	120	0.32
			WEB	-96	30	210	22	14	120	0.33
	20	20	FLG	-75	30	210	17	0	120	0.22
			WEB	-74	30	210	33	14	120	0.27
	21	21	FLG	-33	30	210	24	0	120	0.11
			WEB	-33	30	210	45	14	120	0.21
	22	22	FLG	73	-26	210	29	0	120	0.24
			WEB	72	-26	210	44	12	120	0.31
	23	23	FLG	115	-18	210	28	0	120	0.41
			WEB	113	-18	210	50	8	145	0.46
Int. sup.	24	24	FLG	174	-29	210	31	0	120	0.89
			WEB	171	-29	210	55	16	145	0.94
	25	25	FLG	113	-18	210	29	0	120	0.40
			WEB	112	-18	210	50	8	145	0.46
	26	26	FLG	70	-26	210	31	0	120	0.23
			WEB	69	-26	210	58	12	145	0.32
	27	27	FLG	-48	34	210	26	0	120	0.16
			WEB	-47	34	210	49	14	145	0.23
	28	28	FLG	-97	26	210	20	0	120	0.31
			WEB	-96	26	210	37	8	120	0.37

Source: JICA Study Team

- Index value K is smaller than 1.2 at all of the cross points. This means that the cross ribs have sufficient capacity for the wheel load.

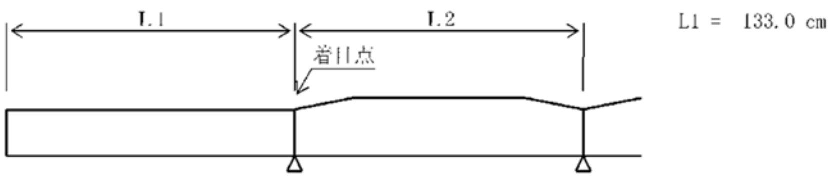
(9) Calculation of Bracket

- Bracket is cantilevered out from the web of outer main girder, and located at every 2.5 m spacing, so the stress is in the transverse direction of the primary stress of the main girder.
- The considered critical condition is the case when the wheel load acts on the point at the forehead of the bracket.
- These brackets are calculated as an I-beam section with the effective width of the deck plate as the top flange.

Sectional Force and Conditions of the Calculation

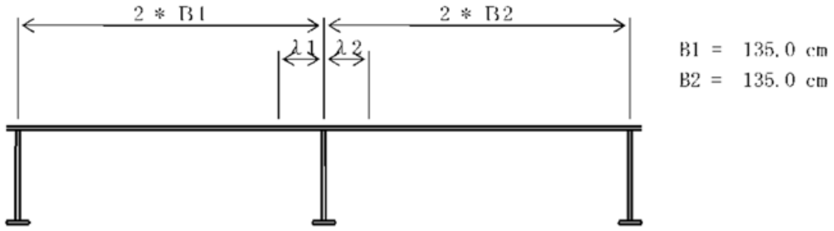
Section force (moment)	$M =$	-94.7 kN·m	S = 167.9 kN
Section force at joint	$M_j =$	-94.7 kN·m	S_j = 167.9 kN
Shear force of missing part	$S_k =$	167.9 kN	
Distance between fixed points	$L =$	1.330 m	

Equivalent Span Length



Li = 2 * L1 = 2 * 133.0 = 266.0 cm

Effective Width



B1/Li	= 135.0 / 266.0 = 0.51
λ1	= 0.15 * Li = 0.15 * 266.0 = 39.9 cm
B2/Li	= 135.0 / 266.0 = 0.51
λ2	= 0.15 * Li = 0.15 * 266.0 = 39.9 cm
Total effective width	λ = λ1 + λ2 = 39.9 + 39.9 = 79.8 cm

Source: JICA Study Team

Figure 4.3.36 Sample Calculation of the Effective Width of the Bracket

Sectional Area and Moment of Inertia

			$A(cm^2)$	$Y(cm)$	$AY(cm^3)$	$I(cm^4)$
1-DECK	PL	798 * 16(SM400)	127.68	-40.80	-5209	212541
1-WEB	PL	800 * 9(SM400)	72.00	0.00	0	38400
1-LFLG	PL	200 * 10(SM400)	20.00	40.50	810	32805
			219.68		-4399	283746

$E = -4399 / 219.68 = -20.03$ cm

$I = 283746 - 219.68 * -20.03^2 = 195644\ cm^4$

$Y_u = -21.57$ cm, $Y_L = 61.03$ cm

Bending stress

$\sigma_u = -94.7*10^6 * -215.7 / (195644*10^4) = 10\ N/mm^2 < \sigma_{ta} = 140\ N/mm^2$

$\sigma_L = -94.7*10^6 * 610.3 / (195644*10^4) = -30\ N/mm^2 < \sigma_{ca} = 133\ N/mm^2$

Shear stress

$\tau = 167.9*10^3 / 7200 = 23\ N/mm^2 < \tau_a = 80\ N/mm^2$

Combined stress;

$\kappa = (-29 / 140)^2 + (23 / 80)^2 = 0.13 < 1.2$

∗ For the Joint (attachment/spliced part)

Bending stress

$\sigma_u = -94.7*10^6 * -215.7 / (195644*10^4) = 10\ N/mm^2 < \sigma_{ta} = 140\ N/mm^2$

$\sigma_L = -94.7*10^6 * 610.3 / (195644*10^4) = -30\ N/mm^2 < \sigma_{ca} = 133\ N/mm^2$

Shear stress

$\tau = 167.9*10^3 / 7200 = 23\ N/mm^2 < \tau_a = 80\ N/mm^2$

Combined stress

$\kappa = (-29 / 140)^2 + (23 / 80)^2 = 0.13 < 1.2$

Source: JICA Study Team

Figure 4.3.37 Sample Calculation of the Bracket

(10) Calculation of the Longitudinal Side Beam

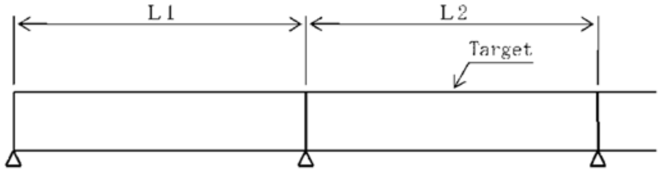
- Longitudinal side beam has the distribution function for the load between two brackets.
- This member is calculated as a beam with the effective width of the deck plate for the top flange.

Sectional Force and Conditions of the Calculation

Sectional force M = 1.6 kN·m S = 0.1 kN

Distance between the fixed points L = 2.500 m

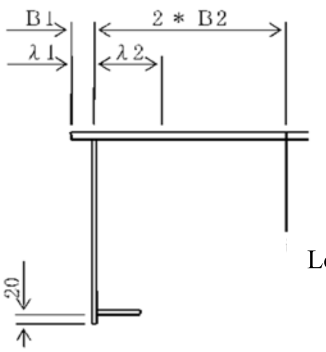
Equivalent Span Length



L1 = 250.0 cm
L2 = 250.0 cm

Li = 0.6 * L2 = 0.6 * 250.0 = 150.0 cm;

Effective Width



B1 = 5.0 cm
B2 = 20.8 cm

B1/Li = 5.0 / 150.0 = 0.03

$\lambda 1 = \{ 1.06 - 3.2 * (B1/Li) + 4.5 * (B1/Li)^2 \} * B1$

$= \{ 1.06 - 3.2 * 0.03 + 4.5 * 0.03^2 \} * 5.0$

$= 4.8$ cm

B2/Li = 20.8 / 150.0 = 0.14

$\lambda 2 = \{ 1.06 - 3.2 * (B2/Li) + 4.5 * (B2/Li)^2 \} * B2$

$= \{ 1.06 - 3.2 * 0.14 + 4.5 * 0.14^2 \} * 20.8$

$= 14.6$ cm

Total effective width $\lambda 1 + \lambda 2 = 4.8 + 14.6 = 19.4$ cm

Source: JICA Study Team

Figure 4.3.38 Calculation of the Effective Width for the Longitudinal Beam

Sectional Area and Moment of Inertia

			$A(cm^2)$	$Y(cm)$	$AY(cm^3)$	$I(cm^4)$
1-DECK	PL	194 * 16(SM400)	31.02	-20.80	-645	13421
1-WEB	PL	400 * 10(SM400)	40.00	0.00	0	5333
1-LFLG	PL	100 * 10(SM400)	10.00	17.50	175	3063
			81.02		-470	21817

$E = -470 / 81.02 = -5.80$ cm

$I = 21817 - 81.02 * -5.80^2 = 19088\ cm^4$

$Y_u = -15.80$ cm , $Y_L = 25.80$ cm

Bending stress

$\sigma_u = 1.6*10^6 * -158.0 / (19088*10^4) = -1\ N/mm^2 < \sigma_{ca} = 140\ N/mm^2$

$\sigma_L = 1.6*10^6 * 258.0 / (19088*10^4) = 2\ N/mm^2 < \sigma_{ta} = 140\ N/mm^2$

Shear stress

$\tau = 0.1*10^3 / 4000 = 0\ N/mm^2 < \tau_a = 80\ N/mm^2$

Combined stress

$\kappa = (2 / 140)^2 + (0 / 80)^2 = 0.00 < 1.2$

Verification of the Deflection

Deflection due to live load $\delta = 0.1$ mm $\leqq \delta_a = L / 500 = 2500 / 500 = 5.0$ mm

Calculation of Stiffener

$b = 40.0$: Abdominal plate height (cm)

$t = 1.0$: Abdominal plate thickness (cm)

$\sigma = 1$: Edge compressive stress intensity of abdominal plate (N/mm^2)

$\tau = 0$: Shear stress intensity of abdominal plate (N/mm^2)

Verification of Abdominal Plate Thickness

$K_h = \sqrt{(\sigma_a / \sigma)} = \sqrt{(140 / 1)} = 10.8 \quad \therefore K_h = 1.2$

$b / (152 * K_h) = 40.0 / (152 * 1.2) = 0.2$ cm $< t = 1.0$ cm

The horizontal stiffener is omitted.

Source: JICA Study Team

Figure 4.3.39 Calculation of the Longitudinal Beam

4.3.3.5 Summary of Steel Weight

(1) Quantity Table of Main Girder

- Steel materials are classified in accordance with each category, grade, and thickness.

Table 4.3.21 Quantity Table of Steel Box Girder

Category	Grade	Thickness	Cross Girder	Deck Plate	Main Girder	Sub-Total
PL	SM400A	9		266.981	254.293	521.274
		10	98,044	105.958	53,128	257,130
		11			1.262	1.262
		12	12,702		3.656	16.358
		13			745	745
		16			1.448	1.448
		19			177.066	177.066
		22			657.060	657.060
		30			4.608	4.608
	SM400A Total		110.746	372.939	1.153.266	1.636.951
	SM490YA	9		244.916	122.426	367.342
		10			10.022	10.022
		11			321.040	321.040
		12			448.150	448.150
		13			47,300	47,300
		14			53.580	53.580
		15			79,414	79,414
		16		1.533.124	159.200	1.692.324
	Total			1.778.040	1.241.132	3.019.172
	SM490YB	17			78,200	78.200
		18		71.859	23.019	94,878
		19		75.872	389.162	465.034
		20		159.696	68.228	227,924
		21			95.766	95.766
		22			20,578	20,578
		23			9.824	9.824
		24		27.636	1.696	29.332
		25		102.653	6.297	108.950
		26		66,870	25,612	92,482
		27			7.254	7.254
		28			22.366	22.366
		36			10.564	10.564
	SM490YB Total			504.586	758.566	1,263.152
	SM570	9			63.848	63.848
		10			2.928	2.928
		11			284,562	284,562
		12			387.587	387.587
		13			20.266	20.266
		14			93.624	93.624
		15			6,468	6,468
		16			36.836	36.836
		17			13.542	13.542
		18			43.868	43.868
		19			169.118	169.118
		20			47.796	47.796
		21			28.210	28.210
		22			30.944	30.944
		23			34,474	34.474
		24			41.018	41,018
		25			29.170	29.170
		26			21.723	21.723
		27			25.987	25.987
		28			47.644	47,644
		29			14.589	14.589
		30			88.146	88.146
		31			938	938
		32			55.038	55,038
		33			33.952	33.952
		34			1.280	1.280
		35			27.168	27.168
		36			10,456	10,456
		37			10.744	10.744
		38			28.698	28.698
		39			12.150	12.150
		40			35,696	35,696
	SM570 Total				1,748.468	1,748.468
	SM570-H	41			23.812	23.812
		42			12.198	12.198
		43			9,372	9,372
		44			12.780	12.780
		45			26.144	26.144
		46			24.048	24.048
		47			13,652	13,652
		51			44.450	44.450
		52			10.196	10.196
		54			10.590	10.590
	SM570-H Total				187.242	187.242
PL Total			110.746	2.655.565	5.088.674	7.854.985

Category	Grade	Thickness	Cross Girder	Deck Plate	Main Girder	Sum
U	SM490YA	320* 240* 8		754.400	32.800	787.200
	SM490YA Total			754.400	32.800	787.200
U Total				754.400	32.800	787.200
BULB	SM490YA	230* 11		202.008	19.471	221.479
	SM490YA Total			202.008	19.471	221.479
BULB Total				202.008	19.471	221.479

Category	Grade	Thickness	Cross Girder	Deck Plate	Main Girder	Sum
TCB	S10T	M 22		11.333	117.329	128.662
	S10T Total			11.333	117.329	128.662
TCB Total				11.333	117.329	128.662

Category	Cross Girder	Deck Plate	Main Girder	Sum
PL Total	110.746	2.655.565	5,088.674	7.854.985
U Total		754.400	32.800	787.200
BULB Total		202.008	19.471	221.479
TCB Total		11.333	117.329	128.662
Total weight	110.746	3.623.306	5.258.274	8.992.326

Source: JICA Study Team

(2) Segment Weight for Erection Block

- The block weight is a very important factor to be considered in the erection procedure.
- The following table shows each segment's weight and assumed pre-assembly weight.

Table 4.3.22 Table of Segment Weight for Erection Block (1)

	G1	G2	G3	G4	2 or 3 Segments Pre-Assembly			
BLK-1	21.701	20.566	20.624	21.701	46.772	42.800	42.858	46.772
BLK-2	25.071	22.234	22.234	25.071				
BLK-3	26.537	24.253	24.253	26.537	54.364	49.680	49.680	54.364
BLK-4	27.827	25.427	25.427	27.827				
BLK-5	27.804	25.424	25.424	27.827	55.217	50.660	50.660	55.256
BLK-6	27.413	25.236	25.236	27.429				
BLK-7	26.052	23.439	23.439	26.052	48.971	44.590	44.590	48.971
BLK-8	22.919	21.151	21.151	22.919				
BLK-9	21.700	21.148	21.148	21.700	45.041	44.772	44.772	45.041
BLK-10	23.341	23.624	23.624	23.341				
BLK-11	21.123	23.214	23.214	21.123	44.111	47.152	47.152	44.111
BLK-12	22.988	23.938	23.938	22.988				
BLK-13	27.762	30.105	30.105	27.762	49.888	51.778	51.778	49.888
BLK-14	22.126	21.673	21.673	22.126				
BLK-15	21.996	21.854	21.854	21.996	65.578	64.853	64.853	65.578
BLK-16	21.706	21.483	21.483	21.706				
BLK-17	21.876	21.516	21.516	21.876				
BLK-18	16.993	16.591	16.591	16.993	61.690	59.009	59.009	61.690
BLK-19	22.548	21.506	21.506	22.548				
BLK-20	22.149	20.912	20.912	22.149				
BLK-21	22.149	20.912	20.912	22.149	42.986	43.040	43.040	42.986
BLK-22	20.837	22.128	22.128	20.837				
BLK-23	25.634	26.501	26.501	25.634	43.720	44.761	44.761	43.720
BLK-24	18.086	18.260	18.260	18.086				
BLK-25	24.799	25.763	25.763	24.799	44.804	46.789	46.789	44.804
BLK-26	20.005	21.026	21.026	20.005				
BLK-27	20.662	22.421	22.421	20.662	64.931	67.800	67.800	64.931
BLK-28	21.522	22.617	22.617	21.522				
BLK-29	22.747	22.762	22.762	22.747				
BLK-30	17.578	17.386	17.386	17.578	62.961	61.089	61.089	62.961
BLK-31	22.794	22.555	22.555	22.794				
BLK-32	22.589	21.148	21.148	22.589				
BLK-33	21.700	21.148	21.148	21.700	42.688	42.498	42.498	42.688
BLK-34	20.988	21.350	21.350	20.988				
BLK-35	26.655	26.002	26.002	26.655	46.181	44.634	44.634	46.181
BLK-36	19.526	18.632	18.632	19.526				
BLK-37	25.412	25.255	25.255	25.412	45.828	46.620	46.620	45.828
BLK-38	20.416	21.365	21.365	20.416				
BLK-39	20.596	20.912	20.912	20.596	66.623	64.056	64.056	66.623
BLK-40	22.202	21.031	21.031	22.202				
BLK-41	23.825	22.113	22.113	23.825				
BLK-42	18.317	16.972	16.972	18.317	64.589	59.846	59.846	64.589
BLK-43	23.683	21.962	21.962	23.683				
BLK-44	22.589	20.912	20.912	22.589				
BLK-45	21.700	20.912	20.912	21.700	42.846	41.726	41.726	42.846
BLK-46	21.146	20.814	20.814	21.146				
BLK-47	26.006	25.612	25.612	26.006	44.994	43.646	43.646	44.994
BLK-48	18.988	18.034	18.034	18.988				

Source: JICA Study Team

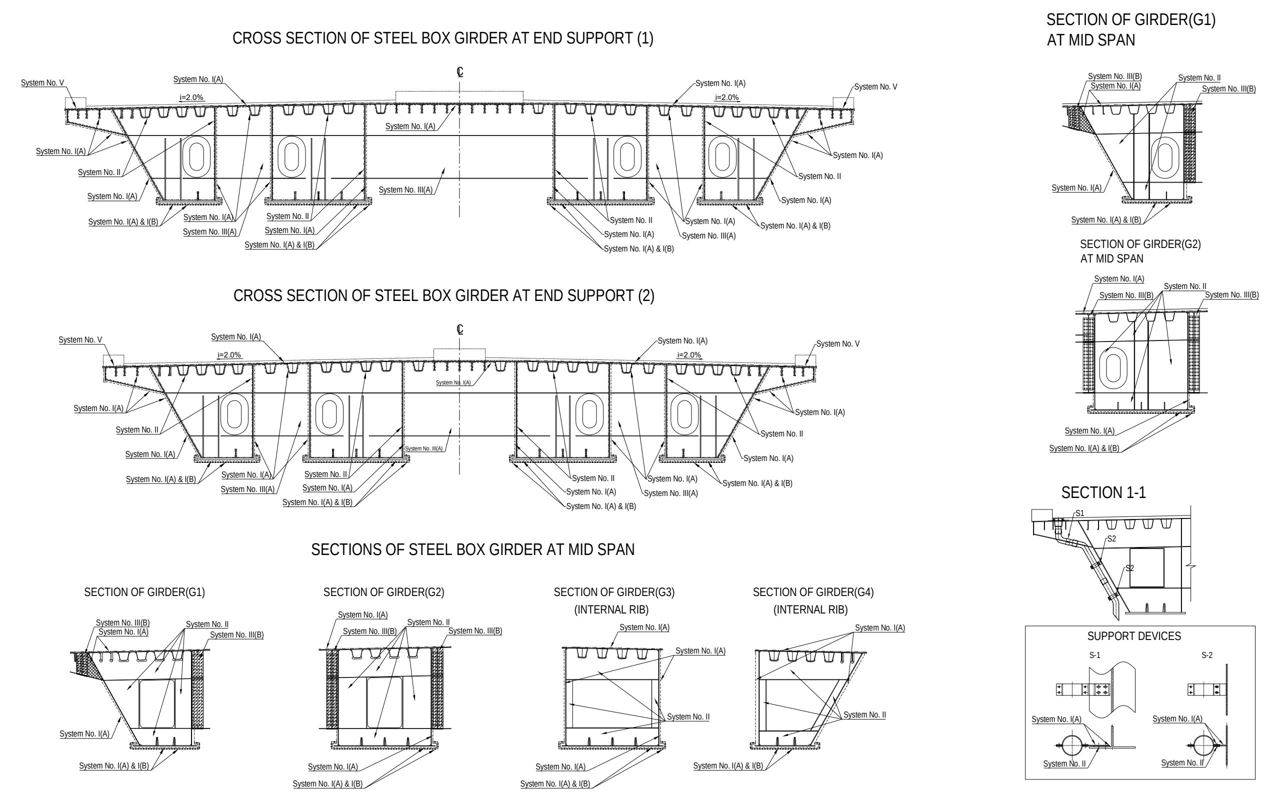
Table 4.3.23 Table of Segment Weight for Erection Block (2)

	G1	G2	G3	G4	2 or 3 Segments Pre-Assembly			
BLK-49	25.306	24.671	24.671	25.306	45.815	45.383	45.383	45.815
BLK-50	20.509	20.712	20.712	20.509				
BLK-51	21.706	20.916	20.916	21.706	68.654	65.289	65.289	68.654
BLK-52	22.794	21.761	21.761	22.794				
BLK-53	24.154	22.612	22.612	24.154				
BLK-54	18.563	18.020	18.020	18.563	65.293	63.735	63.735	65.293
BLK-55	23.987	23.440	23.440	23.987				
BLK-56	22.743	22.275	22.275	22.743				
BLK-57	21.700	21.383	21.383	21.700	42.250	42.406	42.406	42.250
BLK-58	20.550	21.023	21.023	20.550				
BLK-59	25.797	25.332	25.332	25.797	44.000	42.950	42.950	44.000
BLK-60	18.203	17.618	17.618	18.203				
BLK-61	25.333	24.673	24.673	25.333	46.233	45.385	45.385	46.233
BLK-62	20.900	20.712	20.712	20.900				
BLK-63	21.700	20.912	20.912	21.700	66.302	68.373	68.373	66.302
BLK-64	21.808	23.488	23.488	21.808				
BLK-65	22.794	23.973	23.973	22.794				
BLK-66	17.499	18.395	18.395	17.499	61.971	61.279	61.279	61.971
BLK-67	22.624	21.972	21.972	22.624				
BLK-68	21.848	20.912	20.912	21.848				
BLK-69	20.885	21.370	21.370	20.885	41.855	43.178	43.178	41.855
BLK-70	20.970	21.808	21.808	20.970				
BLK-71	26.936	28.273	28.273	26.985	47.849	49.926	49.926	47.898
BLK-72	20.913	21.653	21.653	20.913				
BLK-73	25.180	26.417	26.417	25.180	47.008	49.069	49.069	47.008
BLK-74	21.828	22.652	22.652	21.828				
BLK-75	21.275	21.714	21.714	21.275	44.407	43.675	43.675	44.407
BLK-76	23.132	21.961	21.961	23.132				
BLK-77	25.300	23.367	23.367	25.300	51.258	46.791	46.791	51.258
BLK-78	25.958	23.424	23.424	25.958				
BLK-79	25.715	23.389	23.389	25.715	50.571	45.765	45.765	50.571
BLK-80	24.856	22.376	22.376	24.856				
BLK-81	24.119	24.222	24.222	24.119	41.059	41.060	41.060	41.059
BLK-82	16.940	16.838	16.838	16.940				

Source: JICA Study Team

(3) Painting System

All of superstructure steel shall be painted in accordance with the specifications and drawing as shown in following figure.



Description	General Surface			Joint Connection (Steel Mills and Fabrication Shop: splice plate, filler plate and contact surface of girder) (Construction Site: Splice Plate and Bolts)			Surface in Contact with Concrete and Pavement	
	I. External		II. Internal	III. External		IV. Internal	V. General Surface	VI. Joint Connection
	(A) Normal	(B) Particular		(A) Normal	(B) Particular			
Steel Mills								
1. Preliminary Surface Treatment	Blast Cleaned (ISO Sa2.5)	Blast Cleaned (ISO Sa2.5)	Blast Cleaned (ISO Sa2.5)	Blast Cleaned (ISO Sa2.5)	Blast Cleaned (ISO Sa2.5)	Blast Cleaned (ISO Sa2.5)	SSPC-SP10 Near - white Blast Cleaning	SSPC-SP10 Near - white Blast Cleaning
2. Primer	Inorganic Zinc-Rich Shop Primer DFT : 15μm (160g/m²)	Inorganic Zinc-Rich Shop Primer DFT : 15μm (160g/m²)	Inorganic Zinc-Rich Shop Primer DFT : 15μm (160g/m²)	Inorganic Zinc-Rich Shop Primer DFT : 15μm (160g/m²)	Inorganic Zinc-Rich Shop Primer DFT : 15μm (160g/m²)	Inorganic Zinc-Rich Shop Primer DFT : 15μm (160g/m²)	Inorganic Zinc-Rich Shop Primer DFT : 15μm (200g/m²)	Inorganic Zinc-Rich Shop Primer DFT : 15μm (200g/m²)
Fabrication Shop								
3. Surface Treatment	Blast Cleaned (ISO Sa2.5)	Blast Cleaned (ISO Sa2.5)	Power Tool Cleaned (ISO Sa3)	Blast Cleaned (ISO Sa2.5)	Blast Cleaned (ISO Sa2.5)	Blast Cleaned (ISO Sa2.5)	SSPC-SP10 Near - white Blast Cleaning	SSPC-SP10 Near - white Blast Cleaning
4. 1st Under-Coat	Inorganic Zinc-Rich Paint DFT : 75μm (600g/m²)	Inorganic Zinc-Rich Paint DFT : 75μm (600g/m²)	Formulated Epoxy Resin DFT : 120μm (410g/m²)	Inorganic Zinc-Rich Paint DFT : 75μm (600g/m²)	Inorganic Zinc-Rich Paint DFT : 75μm (600g/m²)	Inorganic Zinc-Rich Paint DFT : 75μm (600g/m²)	High Build Type Inorganic Zinc Rich Paint (Self-Curing Solvent Type) DFT : 30μm (280g/m²)	High Build Type Inorganic Zinc Rich Paint (Self-Curing Solvent Type) DFT : 75μm (700g/m²)
5. 2nd Under-Coat	Epoxy Resin DFT : (160g/m²)	Epoxy Resin DFT : (160g/m²)	Formulated Epoxy Resin DFT : 120μm (410g/m²)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)
6. 3rd Under-Coat	Epoxy Resin DFT : 120μm (540g/m²)	Epoxy Resin DFT : 240μm (1080g/m²)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)
7. 4th Under-Coat	Fluorescent Resin DFT : 30μm (170g/m²)	Fluorescent Resin DFT : 30μm (170g/m²)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)
8. 5th Intermediate Coat	Fluorescent Resin DFT : 25μm (140g/m²)	Fluorescent Resin DFT : 25μm (140g/m²)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)
9. Finish Coat	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.) (N.A.)
Construction Site								
10. Surface Treatment	(N.A.)	(N.A.)	(N.A.)	Power Tool Cleaned (ISO St3)	Power Tool Cleaned (ISO St3)	Power Tool Cleaned (ISO St3)	(N.A.)	(N.A.)
11. 1st Under-Coat	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)
12. 2nd Under-Coat	(N.A.)	(N.A.)	(N.A.)	Formulated Epoxy Resin DFT : 120μm (410g/m²)	Formulated Epoxy Resin DFT : 120μm (410g/m²)	Formulated Epoxy Resin DFT : 160g/m²	(N.A.)	(N.A.)
13. 3rd Under-Coat	(N.A.)	(N.A.)	(N.A.)	Ultra Thick Epoxy Resin DFT : 300μm (1100g/m²)	Ultra Thick Epoxy Resin DFT : 600μm (2200g/m²)	Ultra Thick Epoxy Resin DFT : 300μm (1100g/m²)	(N.A.)	(N.A.)
14. 4th Under-Coat	(N.A.)	(N.A.)	(N.A.)	Fluorescent Resin DFT : 25μm (170g/m²) (140g/m² by brush)	Fluorescent Resin DFT : 25μm (170g/m²) (140g/m² by brush)	(N.A.)	(N.A.)	(N.A.)
15. 5th Under-Coat	(N.A.)	(N.A.)	(N.A.)	Fluorescent Resin DFT : 25μm (140g/m²) (120g/m² by brush)	Fluorescent Resin DFT : 25μm (140g/m²) (120g/m² by brush)	(N.A.)	(N.A.)	(N.A.)
16. Intermediate Coat	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)
17. Finish Coat	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)	(N.A.)
Explanatory Remarks (Line/Hatch)								

Source: JICA Study Team

Figure 4.3.40 Painting System

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

4.3.4.1 Design Condition

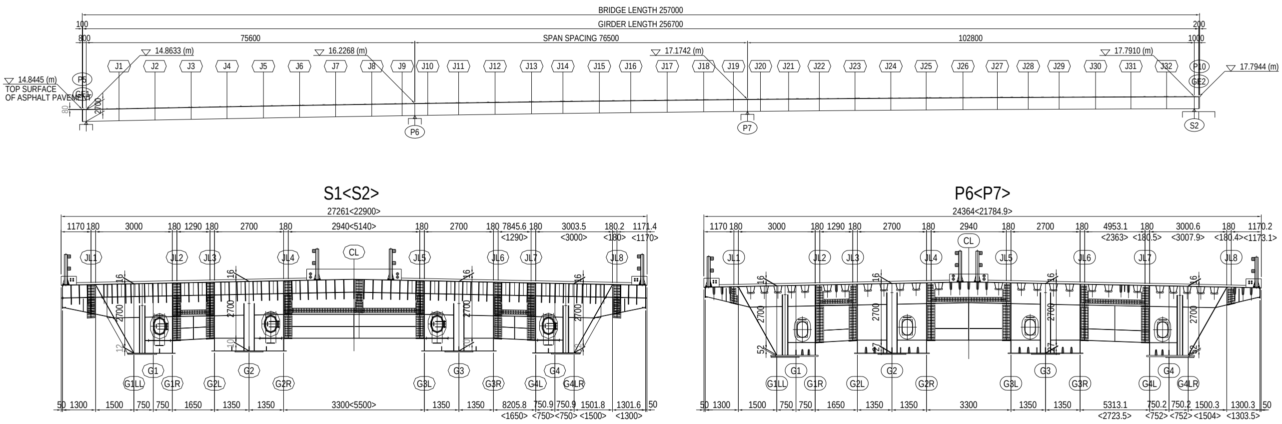
(1) Profile

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 4.3.41 General View

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

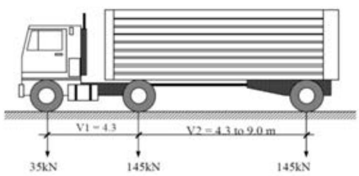
Widened Width (S2) *0.6* + 9.0 + *3.7* + 9.0 + *0.6* = 22.9 m

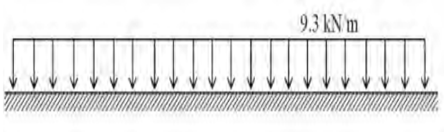
Italicized figures of 0.6, 1.5, and 3.7 above show the side barrier (coping) and median barrier (coping) width.

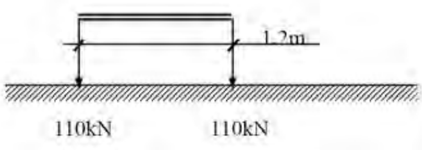
(2) Live Load

AASHTO load was adopted on the design of carriageways, and loading lanes were taken in the severest condition.

There are three kinds of loading, 1-Truck load, Tandem load, and Uniform Lane load, which are shown in the figure below.







Source: AASHTO specification

Figure 4.3.42 AASHTO Loading

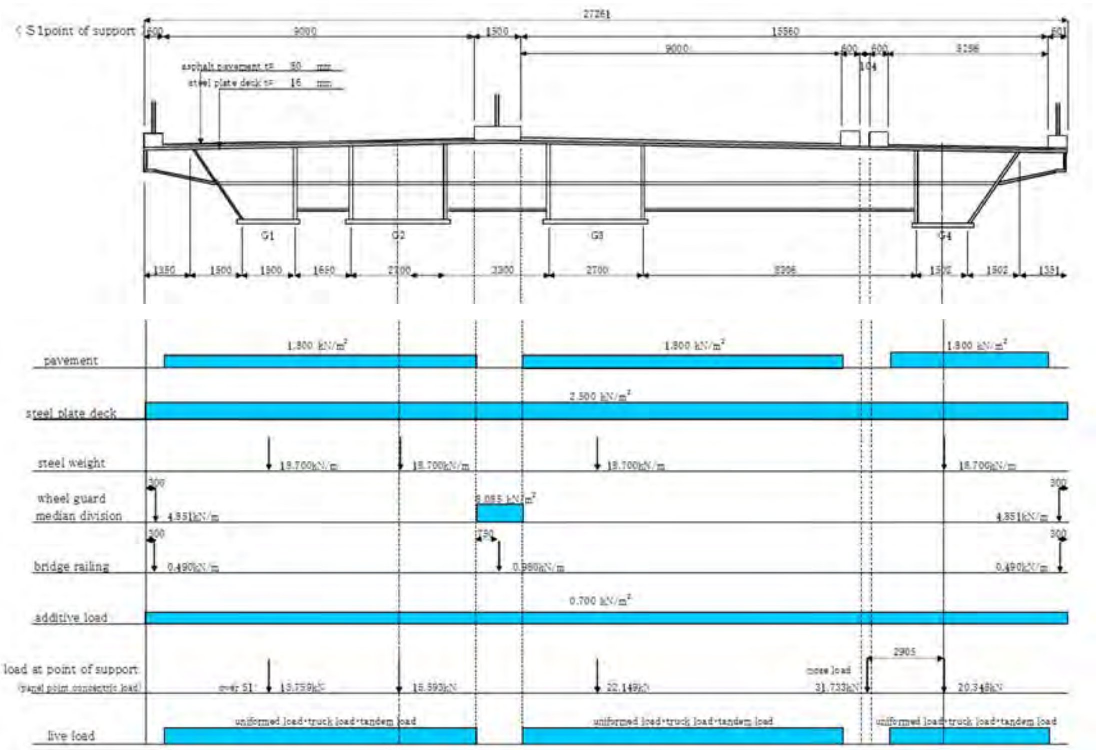
(3) Dead Load

The following items were considered:

- Pavement of asphalt 80 mm thick asphalt laid at whole carriageway
- Coping as wheel guard 330 mm deep concrete casted from steel deck plate
- Railing at side barrier Steel railing weight is assumed.
- Railing at median strip Dual steel railing weight is assumed
- Miscellaneous weight Provisional weight as future overlay load
- Steel weight Assumed in accordance with the girder weight based on B/D

(These weights will be reviewed during the step by step design)

Unit weight of each item is calculated in accordance with its unit volume weight as shown on JSHB.



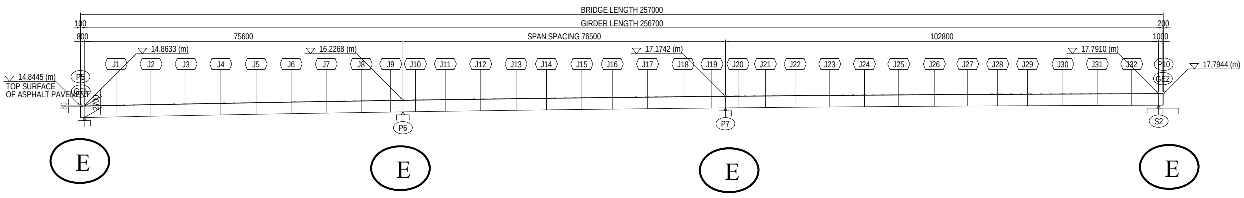
Source: JICA Study Team

Figure 4.3.43 Dead Load and Live Load Variations

(4) 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 4.3.44 Bearing Support Condition

4.3.4.2 Analysis of the Main Girder

(1) Software for Analysis

A Software for 3-span continuous steel bridge is the same software “APPOLO” as Steel Box Girder Bridge (7-Span Bridge).

4.3.4.3 Results of the Analysis and the Determined Section Composition

(1) Reaction

Analyzed reaction was reflected into the design of the substructure and the bearing support.

Table 4.3.24 Reaction Components at Each Pier

					unit: kN
Load	P5				
	Girder				Total
	G1	G2	G3	G4	
PAVEMENT	275.81	272.82	358.50	420.52	1,327.65
RAILING	12.36	2.24	4.03	12.38	31.01
CURB	122.41	22.17	39.89	122.52	306.99
DECK WEIGHT	457.96	413.73	549.89	638.43	2,060.01
GIRDER WEIGH	772.80	454.48	607.65	580.56	2,415.49
MEDIAN STRIP	119.82	90.44	128.52	53.35	392.13
ADITIONAL WEIGHT	128.23	115.85	154.00	178.76	576.84
MEDIAN RAILING	9.47	7.19	10.21	4.13	31.00
SUPPORT WEIGHT	13.76	15.59	22.15	20.35	71.85
NOSE WEIGHT	3.35	-10.96	20.14	19.20	31.73
Total Dead Load	1,915.97	1,383.55	1,894.98	2,050.20	7,244.70
Live Load (without Impact) Girder Max	1,229.65	674.38	812.35	1,184.61	3,900.99
Live Load (without Impact) Girder Min	-436.28	-133.10	-70.19	-113.29	-752.86
Live Load (with Impact) Girder Max	1,320.97	733.18	891.82	1,284.67	4,230.64
Live Load (with Impact) Girder Min	-466.74	-144.11	-75.10	-120.84	-806.79
Total Reaction (D+L:Girder Max(without Impact))	3,145.62	2,057.93	2,707.33	3,234.81	11,145.69
Total Reaction (D+L:Girder Min(without Impact))	1,479.69	1,250.45	1,824.79	1,936.91	6,491.84
Total Reaction (D+L:Girder Max(with Impact))	3,236.94	2,116.73	2,786.80	3,334.87	11,475.34
Total Reaction (D+L:Girder Min(with Impact))	1,449.23	1,239.44	1,819.88	1,929.36	6,437.91

Load	P6				
	Girder				Total
	G1	G2	G3	G4	
PAVEMENT	636.44	693.86	780.46	982.12	3,092.88
RAILING	28.21	7.05	7.58	31.46	74.29
CURB	279.25	69.76	75.02	311.43	735.46
DECK WEIGHT	1,033.01	1,068.40	1,196.89	1,493.87	4,792.17
GIRDER WEIGH	1,473.33	1,363.98	1,419.72	1,417.85	5,674.88
MEDIAN STRIP	202.86	268.85	290.18	108.44	870.34
ADITIONAL WEIGHT	289.24	299.15	335.13	418.28	1,341.81
MEDIAN RAILING	17.32	22.73	24.47	9.90	74.41
SUPPORT WEIGHT	0.00	0.00	0.00	0.00	0.00
NOSE WEIGHT	0.19	-0.03	0.00	-0.16	0.00
Total Dead Load	3,959.84	3,793.74	4,129.45	4,773.19	16,656.24
Live Load (without Impact) Girder Max	1,854.54	1,207.43	1,330.88	2,082.86	6,475.71
Live Load (without Impact) Girder Min	-463.81	-226.89	-210.96	-318.33	-1,219.99
Live Load (with Impact) Girder Max	1,947.85	1,278.07	1,409.40	2,185.38	6,820.70
Live Load (with Impact) Girder Min	-484.27	-239.35	-221.33	-332.39	-1,277.34
Total Reaction (D+L:Girder Max(without Impact))	5,814.38	5,001.17	5,460.33	6,856.05	23,131.95
Total Reaction (D+L:Girder Min(without Impact))	3,496.03	3,566.85	3,918.49	4,454.86	15,436.25
Total Reaction (D+L:Girder Max(with Impact))	5,907.69	5,071.81	5,538.85	6,958.57	23,476.94
Total Reaction (D+L:Girder Min(with Impact))	3,475.57	3,554.39	3,908.12	4,440.80	15,378.90

Load	P7				
	Girder				Total
	G1	G2	G3	G4	
PAVEMENT	904.02	797.38	818.85	1,029.17	3,549.42
RAILING	37.87	12.91	13.43	38.85	103.06
CURB	374.93	127.79	132.92	384.62	1,020.25
DECK WEIGHT	1,473.90	1,269.58	1,303.95	1,642.38	5,689.81
GIRDER WEIGH	2,070.42	1,847.47	1,875.88	2,070.72	7,864.49
MEDIAN STRIP	331.23	396.46	406.12	304.27	1,438.08
ADITIONAL WEIGHT	412.69	355.48	365.11	459.87	1,593.15
MEDIAN RAILING	23.21	29.07	29.85	20.88	103.00
SUPPORT WEIGHT	0.00	0.00	0.00	0.00	0.00
NOSE WEIGHT	-0.02	0.00	0.00	0.02	0.00
Total Dead Load	5,628.25	4,836.14	4,946.10	5,950.77	21,361.26
Live Load (without Impact) Girder Max	2,194.66	1,376.79	1,403.26	2,268.78	7,243.49
Live Load (without Impact) Girder Min	-459.79	-132.85	-124.80	-335.63	-1,053.07
Live Load (with Impact) Girder Max	2,278.57	1,144.41	1,473.67	2,364.77	7,261.42
Live Load (with Impact) Girder Min	-483.95	-142.76	-134.14	-352.79	-1,113.64
Total Reaction (D+L:Girder Max(without Impact))	7,822.91	6,212.93	6,349.36	8,219.55	28,604.75
Total Reaction (D+L:Girder Min(without Impact))	5,168.46	4,703.29	4,821.30	5,615.14	20,308.19
Total Reaction (D+L:Girder Max(with Impact))	7,906.82	5,980.55	6,419.77	8,315.54	28,622.68
Total Reaction (D+L:Girder Min(with Impact))	5,144.30	4,693.38	4,811.96	5,597.98	20,247.62

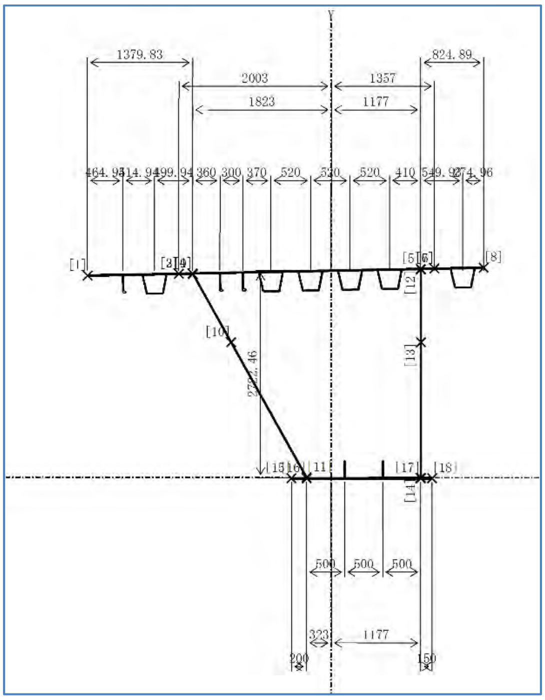
Load	P10				
	Girder				Total
	G1	G2	G3	G4	
PAVEMENT	382.14	307.52	366.59	303.76	1,360.01
RAILING	16.62	4.37	6.38	13.96	41.34
CURB	164.58	43.26	63.18	138.22	409.24
DECK WEIGHT	636.53	532.42	627.67	509.59	2,306.20
GIRDER WEIGH	841.05	757.97	879.39	677.54	3,155.95
MEDIAN STRIP	177.51	297.32	320.12	145.41	940.36
ADITIONAL WEIGHT	178.23	149.08	175.75	142.68	645.74
MEDIAN RAILING	8.61	12.31	13.63	6.80	41.35
SUPPORT WEIGHT	17.20	31.13	31.13	17.20	96.67
NOSE WEIGHT	0.00	0.00	0.00	0.00	0.00
Total Dead Load	2,422.47	2,135.38	2,483.84	1,955.16	8,996.86
Live Load (without Impact) Girder Max	1,521.59	1,046.55	1,114.45	1,452.71	5,135.30
Live Load (without Impact) Girder Min	-573.92	-422.14	-383.40	-644.72	-2,024.18
Live Load (with Impact) Girder Max	1,601.57	1,101.39	1,169.66	1,531.31	5,403.93
Live Load (with Impact) Girder Min	-602.69	-441.99	-402.09	-675.79	-2,122.56
Total Reaction (D+L:Girder Max(without Impact))	3,944.06	3,181.93	3,598.29	3,407.87	14,132.16
Total Reaction (D+L:Girder Min(without Impact))	1,848.55	1,713.24	2,100.44	1,310.44	6,972.68
Total Reaction (D+L:Girder Max(with Impact))	4,024.04	3,236.77	3,653.50	3,486.47	14,400.79
Total Reaction (D+L:Girder Min(with Impact))	1,819.78	1,693.39	2,081.75	1,279.37	6,874.30

Source: JICA Study Team

(2) Member Force and Section Composition Diagram

The section dimensions and grade of material are determined so that the following criteria are satisfied:

- Each section is designed so that the stress based on bending moment and shearing force shall be within the allowable stress of the adopted material grade.
- The JSHB requires that the deflection due to live load shall be less than 1/500 of span length.
- All block joints are fastened by high strength bolts. Therefore, axial tensile stress at tensile part shall take account of the decreased section area because of the bolt holes. In case that tensile stress would be more than the allowable stress, the thickness of the section should be increased.
- Steel deck plate is stiffened by u-shaped trough ribs, so that torsional stiffness is increased for wheel load.
- Compression stress part of lower flange is stiffened by plate ribs in accordance with thickness of flange. These ribs shall be fastened by high strength bolts at block joint as stress member.
- Web plate is stiffened by horizontal stiffeners at 2-level position, so as decreasing web thickness. These stiffeners only act as stiffeners but not as stress member.
- Required section properties are calculated as follows:



Effective width(mm)

		Full width	In-plane
DECK	張出部	1380	1380
	Intermediate	3000	3000
	張出部	825	825
LFLG	Intermediate	1500	1500

Section dimensions	Section area(cm2)	Total	in plane
1-DECK-L PL	1200 * 16 (SM490Y)	192. 0	192. 0
1-BULB PL	230 * 11 (SM490Y)	32. 0	32. 0
1-U. RIB	320 * 240 * 8 (SM490Y)	53. 9	53. 9
1-DECK-C PL	3361 * 16 (SM490Y)	537. 7	537. 7
2-BULB PL	230 * 11 (SM490Y)	64. 0	64. 0
4-U. RIB	320 * 240 * 8 (SM490Y)	215. 6	215. 6
1-DECK-R PL	645 * 16 (SM490Y)	103. 2	103. 2
1-U. RIB	320 * 240 * 8 (SM490Y)	53. 9	53. 9
1-LWEB PL	3058 * 12 (SM490Y)	367. 0	367. 0
1-RWEB PL	2730 * 12 (SM490Y)	327. 6	327. 6
1-LFLG PL	1850 * 20 (SM490Y)	370. 0	370. 0
2-RIB PL	220 * 19 (SM490Y)	83. 6	83. 6

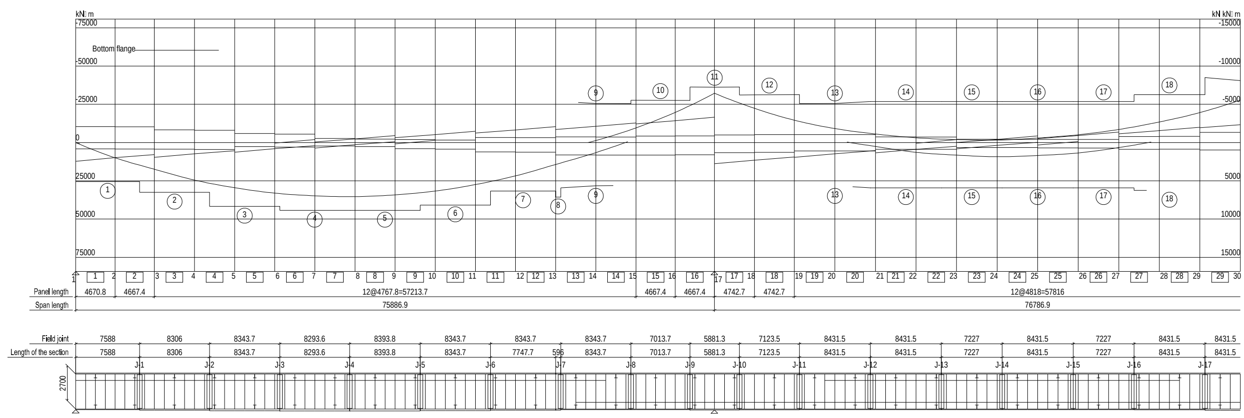
Section property		Total	In-plane
Section area	A (cm2)	2400. 5	2400. 5
Gravity center	ex (cm)	-22. 0	-22. 0
	ey (cm)	177. 5	177. 5
Moment of inertia	Ix (cm4)	29294019	29294019
	Iy (cm4)	40717501	40717501
Torsion Constant	J (cm4)	20180893	

Source: JICA Study Team

Figure 4.3.45 Typical Calculation Sample of the Section

- The thickness and the material grade of all sections have been determined and calculated in accordance with the bending moment.
- The following diagrams show part of the section composition of G1 and G2 girders as example.

G1



Unit: mm N/mm2

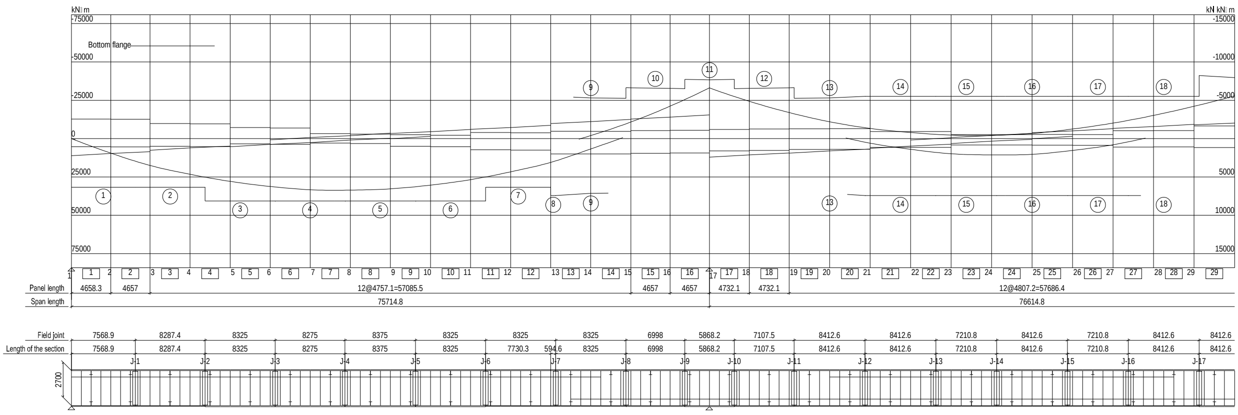
		1		2	3	4	5	6	7	8	9		10	11		12	13			14				15						16			
Section		Sec-1		Sec-2	Sec-3	Sec-4	Sec-5	Sec-6	Sec-7	Sec-7	Sec-8		Sec-9	Sec-10		Sec-11	Sec-12			Sec-13				Sec-14						Sec-15			
Deck Plate	Thickness	16, 16, 16		16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16		16, 16, 16	16, 16, 16		16, 16, 16	16, 16, 16			16, 16, 16				16, 16, 16						16, 16, 16			
	Quality	(1),(1),(1)		(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)		(1),(1),(1)	(3),(3),(3)		(1),(1),(1)	(1),(1),(1)			(1),(1),(1)				(1),(1),(1)						(1),(1),(1)			
Longit Rib 1	Number	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL	3-PL
	Section	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24
Longit Rib 2	Number	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib
	Section	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8
Left Web	Height	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5	3062.5
	Thickness	15(3)		15(3)	15(3)	15(3)	15(3)	15(3)	15(3)	15(3)	15(3)		17(3)	17(3)		17(3)	15(3)			15(3)				15(3)						15(3)			
Right Web	Height	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730	2730
	Thickness	15(3)		15(3)	15(3)	15(3)	15(3)	15(3)	15(3)	15(3)	15(3)		17(3)	17(3)		17(3)	15(3)			15(3)				15(3)						15(3)			
Bottom flange Vertical rib	Number	2		2	2	2	2	2	2	5	5		5	5		5	5			5				5						5			
	Width	170		170	200	200	200	200	170	170	170		170	200		170	170			170				170						170			
	Thickness	17(3)		17(3)	22(3)	22(3)	22(3)	22(3)	17(3)	17(3)	17(3)		17(3)	22(3)		17(3)	17(3)			17(3)				17(3)						17(3)			
Lflg W=1740 T		10(3)		18(3)	27(3)	30(3)	30(3)	26(3)	17(3)	17(3)	10(3)		10(3)	15(3)		12(3)	10(3)			10(3)				10(3)						10(3)			
Deck Plate	σ	0	-52	-87	-103	-108	-108	-102	-84	-46	-45	30	87	110	110	91	49	-7	20	-7	20	-26	8	-26	8	-31	6	-31	6	-31	6	-24	16
	σa	140	140	140	140	140	140	140	140	140	140	140	140	210	210	140	140	140	140	140	140	140	140	140	140	140	140	140	140	140	140	140	140
	σa-σ	140	88	53	37	32	32	38	56	94	95	110	53	100	100	49	91	133	120	133	120	114	132	114	132	109	134	109	134	109	134	116	124
Bottom flange	σ	0	122	172	168	168	168	169	169	84	95	-63	-172	-187	-187	-174	-103	15	-43	15	-43	55	-16	55	-16	66	-12	66	-12	66	-12	50	-34
	σa	210	210	210	210	210	210	210	210	210	210	190	196	210	210	210	190	210	190	210	190	210	190	210	190	210	190	210	190	210	190	210	190
	σa-σ	210	88	38	42	42	42	41	41	126	115	127	24	23	23	36	87	195	147	195	147	155	174	155	174	144	179	144	178	144	178	160	156
Web	τ	48	37	28	17	9	9	17	27	36	34	45	47	49	41	38	33	22	22	22	22	14	14	14	14	10	10	10	10	10	10	17	17
	τa	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120
	Combined	0.16	0.36	0.68	0.63	0.62	0.62	0.64	0.66	0.20	0.23	0.14	0.71	0.85	0.83	0.72	0.27	0.03	0.05	0.03	0.05	0.07	0.01	0.07	0.01	0.10	0.01	0.10	0.01	0.10	0.01	0.06	0.03
Calculated points		Left	J-1	J-2	J-3		J-4	J-5	J-6	Left	J-7	J-8	J-9	Max Left	Max Right	J-10	J-11	J-12	J-12	Left	Left	J-13	J-13	Left	Left			J-14	J-14	Left	Left	J-15	J-15
Deck Plate σspl												45	120			126	70		32				21						19				25
Bottom flange σspl			157	209	205			206	209		124							44				79						97				80	

Grade
(1):SM400
(2):SM490
(3):SM490Y
(4):SM570
(5):SM400-H
(6):SM490-H
(7):SM520-H
(8):SM570-H

Source: JICA Study Team

Figure 4.3.46 Typical Section Composition, G1 (P5-P6)

G2



Unit: mm N/mm2

		1		2	3	4	5	6	7	8	9		10	11		12	13			14				15				16			
Section		Sec-1		Sec-2	Sec-3	Sec-4	Sec-5	Sec-6	Sec-7	Sec-7	Sec-8		Sec-9	Sec-10		Sec-11	Sec-12			Sec-13				Sec-14				Sec-15			
Deck Plate	Thickness	16, 16, 16		16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16	16, 16, 16		16, 16, 16	16, 16, 16		16, 16, 16	16, 16, 16			16, 16, 16				16, 16, 16				16, 16, 16			
	Quality	(1),(1),(1)		(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)	(1),(1),(1)		(1),(1),(1)	(3),(3),(3)		(1),(1),(1)	(1),(1),(1)			(1),(1),(1)				(1),(1),(1)				(1),(1),(1)			
Longit Rib 1	Number	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib	6-U-Rib
	Section	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8	320*240*8
Longit Rib 2	Number	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL	2-PL
	Section	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24	250*24
Left Web	Height	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673	2673
	Thickness	14(3)		14(3)	14(3)	14(3)	14(3)	14(3)	14(3)	14(3)	14(3)		14(3)	14(3)		14(3)	14(3)			14(3)				14(3)				14(3)			
Right Web	Height	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727	2727
	Thickness	14(3)		14(3)	14(3)	14(3)	14(3)	14(3)	14(3)	14(3)	14(3)		14(3)	14(3)		14(3)	14(3)			14(3)				14(3)				14(3)			
Bottom flange Vertical rib	Number	3		3	3	3	3	3	3	7	7		7	7		7	7			7				7				7			
	Width	170		170	170	170	170	170	170	170	170		170	170		170	170			170				170				170			
	Thickness	17(3)		17(3)	17(3)	17(3)	17(3)	17(3)	17(3)	17(3)	17(3)		17(3)	17(3)		17(3)	17(3)			17(3)				17(3)				17(3)			
Lflg W=1740 T		10(3)		10(3)	16(3)	16(3)	16(3)	16(3)	10(3)	10(3)	10(3)		12(3)	14(3)		12(3)	10(3)			10(3)				10(3)				10(3)			
Deck Plate	σ	0	-50	-85	-101	-108	-108	-100	-85	-51	-48	32	86	111	111	92	54	-8	23	-8	23	-30	9	-30	9	-35	7	-35	8	-27	19
	σa	140	140	140	140	140	140	140	140	140	140	140	140	210	210	140	140	140	140	140	140	140	140	140	140	140	140	140	140	140	140
	σa-σ	140	90	55	39	32	32	40	55	89	92	108	54	99	99	48	86	132	117	132	117	110	131	110	131	105	133	105	132	113	121
Bottom flange	σ	0	98	167	163	174	173	162	166	88	82	-57	-142	-173	-173	-153	-96	13	-40	13	-40	51	-16	51	-16	60	-12	60	-13	46	-33
	σa	210	210	210	210	210	210	210	210	210	210	156	182	200	200	182	156	210	156	210	156	210	156	210	156	210	156	210	156	210	156
	σa-σ	210	112	43	47	36	37	48	44	122	128	99	40	27	27	29	60	197	116	197	116	159	140	159	140	150	144	150	143	164	123
Web	τ	47	40	27	18	8	8	17	27	34	40	47	53	56	44	42	35	28	28	28	28	18	18	18	18	11	11	12	12	19	19
	τa	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120	120
	Combined	0.15	0.27	0.65	0.60	0.67	0.67	0.60	0.64	0.22	0.22	0.16	0.56	0.80	0.75	0.59	0.25	0.05	0.07	0.05	0.07	0.07	0.02	0.07	0.02	0.09	0.01	0.09	0.01	0.06	0.04
Calculated points		Left	J-1	J-2	J-3		J-4	J-5	J-6	Left	J-7	J-8	J-9	Max Left	Max Right	J-10	J-11	J-12	J-12	Left	Left	J-13	J-13	Left	Left	J-14	J-14			J-15	J-15
Deck Plate σspl												48	120			128	76		36				23				23				30
Bottom flange σspl			129	200	195		208	195	201		107							39				73				86				71	

Grade
(1):SM400
(2):SM490
(3):SM490Y
(4):SM570

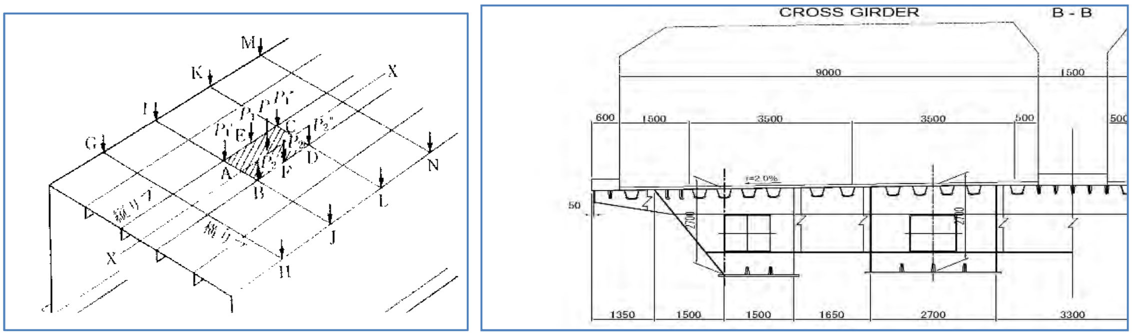
Source: JICA Study Team

Figure 4.3.47 Typical Section Composition, G2 (P5-P6)

4.3.4.4 Analysis of Steel Deck

(1) Design Method of Steel Deck

- The steel deck will be analyzed as equivalent multi-grid frame.
- The model of the grid frame consists of webs, cross girder, crossbeams, and longitudinal stiffeners.



Source: JICA Study Team

Figure 4.3.48 Concept of Wheel Loading on Steel Deck

(2) Stress Evaluation of Steel Deck

- Longitudinal ribs or cross ribs have combined stresses of primary stress as a deck member of the whole main girder and secondary stress as a member of the deck frame.
- The stress of longitudinal rib is of the same direction as the main girder stress. Therefore, this stress shall be combined with the stress of the main girder and shall be within the allowable stress as shown below.

  $\sigma 1$: Primary stress as a member of the main girder

  $\sigma 2$: Secondary stress as a member of the deck frame

  α : Safety factor (1.4 as specified by the JSHB)

  σa : Allowable stress of the material deck plate

  $$\sigma 1+\sigma 2 < \sigma a \cdot \alpha$$

- If $\sigma 2$ is smaller than o.4*σa, then the formula above will always be satisfied.
- The stress of the cross rib is in the right angle direction of the stress of the main girder, so the biaxial stress shall be checked.

Biaxial calculation formula:

$$K = (\sigma x / \sigma a)2 - (\sigma x / \sigma a) * (\sigma y / \sigma a) + (\sigma y / \sigma a)2 + (\tau / \tau a)2 \leqq 1.2$$

σx : Normal stress of the main girder (N/mm^2) τx : Shear stress of the main girder (N/mm^2)

σy : Normal stress of the crossbeam (N/mm^2) τy : Shear stress of the crossbeam (N/mm^2)

σa : Allowable tensile stress of the main girder (N/mm^2)

τa : Allowable shear stress of the main girder or crossbeam (N/mm^2)

Where, by checking location:

Flange point $(\tau / \tau a) = (\tau x / \tau xa)$

Web point $(\tau / \tau a) = Max((\tau x/\tau xa), (\tau y/Max(\tau xa,\tau ya)))$

(3) Analysis Model of Deck Frame

- Wheel load shall act on the longitudinal stiffener or crossbeam so that the maximum bending moment will occur.
- Vertical ribs are considered as one bar members without any cross sectional deformation. Therefore, the torsional rigidity (using only the simple torsional resistance) is not reduced; it is 100% valid and is calculated by the following formula:

Torsional rigidity = 4*A2 / {(u/tR) + (a/tP)}

A : Cross sectional area surrounded by the U-shaped steel

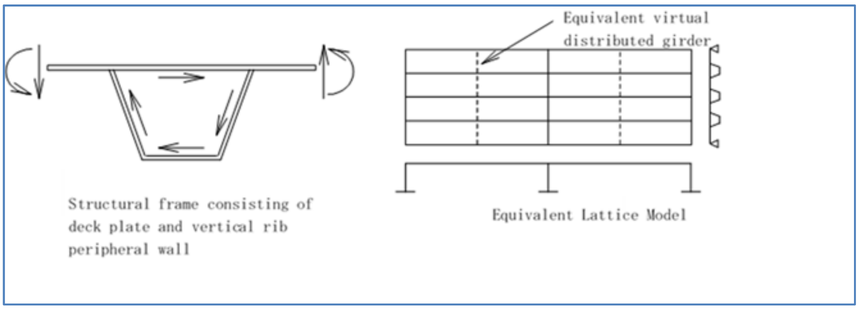
u : Expanded width of the U-shaped steel

a : Upper width of the U-shaped steel

tR: Thickness of the U-shaped steel

tP: Thickness of the deck plate

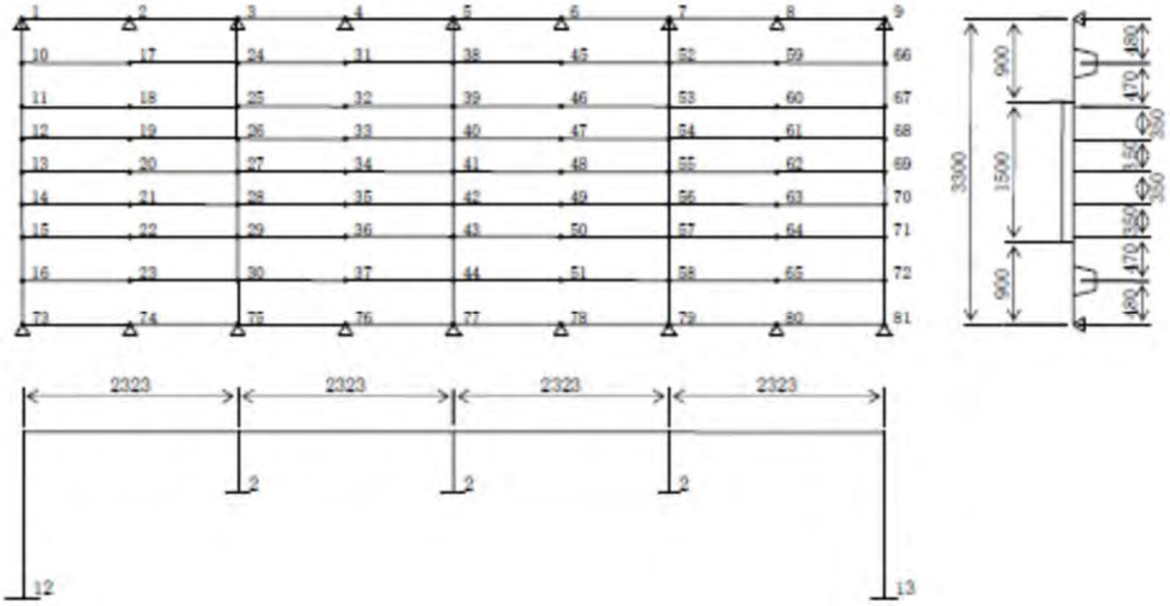
- The virtual distribution girder that performs the load distribution between the longitudinal ribs gives the bending rigidity equivalent to the rigid structure consisting of the deck plate and the longitudinal rib peripheral wall in consideration of the sectional deformation of the longitudinal rib. Since this rigid structure continues in the direction of the bridge axis, the equivalent cross sectional secondary moment per unit length is obtained first, and in the Lattice Model, one distribution girder is provided at the lateral rib intervals to provide bending rigidity.



Source: Analysis manual prepared by the software company

Figure 4.3.49 Concept of Equivalent Virtual Beam of Steel Deck

- There are eight deck models as follows:
  - MODEL-1: S1～C1 / between G2R～G3L : edge・between girder and girder
  - MODEL-2: S1～C1 / between G3R～G4L : edge・between girder and girder
  - MODEL-3: D1～D2 / between G3R～G4L : intermediate・between girder and girder
  - MODEL-4: C9～C10 / between G2R～G3L : intermediate・between girder and girder
  - MODEL-5: C9～C10 / between G3R～G4L : intermediate・between girder and girder
  - MODEL-6: C24～S2 / between LL1～G1UL : edge・intermediate overhang
  - MODEL-7: C24～S2 / between G1UL～G1R : edge・inside the box girder
  - MODEL-8: C24～S2 / between G1R～G2L : edge・between girder and girder
- The MODEL-1 is shown in the following figure.



Source: JICA Study Team

Figure 4.3.50 Analysis Model of Steel Deck

- Dead load to be considered

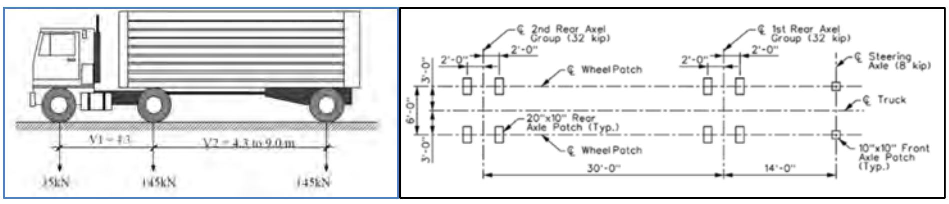
Pavement (road section) 1.80 kN/m^2

Steel weight 2.00 kN/m^2

- Section profile of member

Cross sectional shape	Thickness of deck plate 16 mm		
Longitudinal rib	Sec- 2	U.RIB 320 * 240 * 8	
Lateral rib	Sec- 2	WEB PL 800 * 9	FLG PL 200 * 10
Diaphragm		WEB PL 2100 * 10	FLG PL 220 * 10

- Wheel load shall act on the longitudinal stiffener or crossbeam so the maximum bending moment will occur.
- Distance of the wheel is 1.8 m, and contact area of each wheel is 510 mm wide and 250 mm long.



Source: AASHTO specification

Figure 4.3.51 Wheel Load to be Considered on Steel Deck

- Impact coefficient is based on the following formula which is based on JSHB:

  Longitudinal rib   i = 0.4

  Lateral rib & bracket   i = 20/(50+L)   L: Span length of lateral rib and bracket (m)

(4) Analysis Results of Each Rib Force of MODEL-1

Table 4.3.25 Table of the Member Force of MODEL-1

(a) Cross-sectional force list

member	section	focusing case	panel point	load type	bending moment (kN·m)	shearing force (kN)
longitudinal rib	Sec-1	Max bending moment	18	dead load	1.73	-0.52
				T-LOAD	25.13	-30.92
				total	26.86	-31.44
				premium total	26.86	-31.44
		min bending moment	25	dead load	-1.20	-4.52
				T-LOAD	-11.45	-31.19
				total	-12.66	-35.71
				premium total	-12.66	-35.71
		Max shearing force	25	dead load	-1.20	-4.52
				T-LOAD	1.00	-52.10
				total	-0.21	-56.63
				premium total	-0.21	-56.63
	Sec-2	Max bending moment	17	dead load	1.23	-0.34
				T-LOAD	48.51	-61.44
				total	49.74	-61.77
				premium total	49.74	-61.77
		min bending moment	24	dead load	-0.78	-3.12
				T-LOAD	-23.48	-61.66
				total	-24.27	-64.78
				premium total	-24.27	-64.78
		Max shearing force	24	dead load	-0.78	-3.12
				T-LOAD	0.75	-102.82
				total	-0.03	-105.93
				premium total	-0.03	-105.93
transverse rib	Sec-2	Max bending moment	55	dead load	31.31	-4.88
				T-LOAD	79.58	1.22
				total	110.89	-3.65
				premium total	92.88	-3.57
		min bending moment	5	dead load	0.00	27.36
				T-LOAD	0.00	-13.39
				total	0.00	13.97
				premium total	-92.88	13.09
		Max shearing force	7	dead load	0.00	29.05
				T-LOAD	0.00	149.10
				total	0.00	178.15
				premium total	0.00	187.90
cross beam	Sec-12	Max bending moment	13	dead load	13.34	-2.01
				T-LOAD	89.21	0.01
				total	102.55	-1.99
				premium total	121.46	-3.61
		min bending moment	1	dead load	0.00	11.95
				T-LOAD	0.00	-7.16
				total	0.00	4.78
				premium total	-121.46	11.31
		Max shearing force	1	dead load	0.00	11.95
				T-LOAD	0.00	154.13
				total	0.00	166.07
				premium total	0.00	242.24

Source: JICA Study Team

Table 4.3.26 Table of the Deflection due to Live Load

(b) Live load deflection list				
Member	Section	Panel Point	Load Type	deflection(mm)
Longitudinal Rib	Sec-1	18	T-LOAD	0.54
	Sec-2	17	T-LOAD	0.63
Transverse Rib	Sec-2	55	T-LOAD	0.17
Cross Beam	Sec-12	13	T-LOAD	0.01

Source: JICA Study Team

(5) Stress Calculation of the Longitudinal Rib

- A two-type rib, that is a plate rib or a U-shaped rib, is adopted.
- Each rib is calculated as a beam structure, so that stress will be less than allowable stress.
- The typical calculation procedure is shown in the following figure.

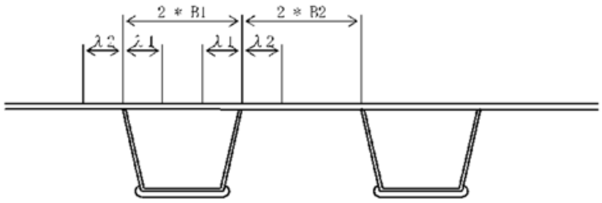
(2) Model-3 Sec-2 (Supporting Part)

Sectional Force and Conditions of the Calculation

For sectional calculation $M = -26.10$ kN·m $S = 110.62$ kN

For spliced (attachment/joint) part $M_j = -0.63$ kN·m $S_j = 110.62$ kN

Calculation of Effective Width



* Equivalent span length $L = 0.6 * LC = 0.6 * 250.0 = 150.0$ cm

LC : Lateral rib interval (Longitudinal rib span length)

B1 = 16.2 cm

B2 = 11.3 cm

$B1 / L = 16.2 / 150.0 = 0.11$

$\lambda 1 = \{1.06 - 3.2 * (B1 / L) + 4.5 * (B1 / L)^2\} * B1$

$= \{1.06 - 3.2 * (0.11) + 4.5 * (0.11)^2\} * B1$

$= 12.4$ cm

$B2 / L = 11.3 / 150.0 = 0.08$

$\lambda 2 = \{1.06 - 3.2 * (B2 / L) + 4.5 * (B2 / L)^2\} * B2$

$= \{1.06 - 3.2 * (0.08) + 4.5 * (0.08)^2\} * B2$

$= 9.5$ cm

* Total effective width

$\lambda = (\lambda 1 + \lambda 2) * 2 = (12.4 + 9.5) * 2$

$= 43.9$ cm

Sectional Quantities

	A(cm²)	Y(cm)	AY(cm³)	I(cm⁴)
1-DECK PL 439 * 16 (SM400)	70.30	-16.01	-1125.5	18019.3
1-U.RIB 320 * 240 * 8 (SM400)	53.90	0.00	0.0	3315.0
	124.20		-1125.5	21334.3

$e = -1125.5 / 124.20 = -9.06$ cm

$I = 21334.3 - 124.20 * -9.06^2 = 11135$ cm^4

$Y_u = -7.75$ cm, $Y_L = 18.05$ cm

$W_u = -1437$ cm^3, $W_L = 617$ cm^3

Actual Stress

Bending stress

$\sigma_u = -26.10 * 10^6 / (-1437 * 10^3) = 18$ N/mm^2 $< \sigma_{ta} = 140$ N/mm^2

$\sigma_L = -26.10 * 10^6 / (617 * 10^3) = -42$ N/mm^2 $< \sigma_{ca} = 140$ N/mm^2

$\sigma < 0.4 \cdot \sigma a$

Shear stress

$\tau = 110.62 * 10^3 / (38.7 * 10^2) = 29$ N/mm^2 $< \tau_a = 80$ N/mm^2

Combined stress

$\kappa = (42 / 140)^2 + (29 / 80)^2 = 0.22 < 1.2$

Source: JICA Study Team

Figure 4.3.52 Sample Calculation of the Longitudinal Rib

(6) Stress Calculation of the Cross Rib

- Cross ribs are a kind of elastic support member for longitudinal ribs.
- The typical calculation procedure is shown in the following figure.

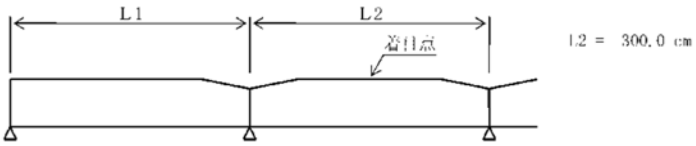
Sectional Force and Conditions of the Calculation

Section force M = 112.3 kN·m S = 16.6 kN

Shear force of missing part S_k = 16.6 kN

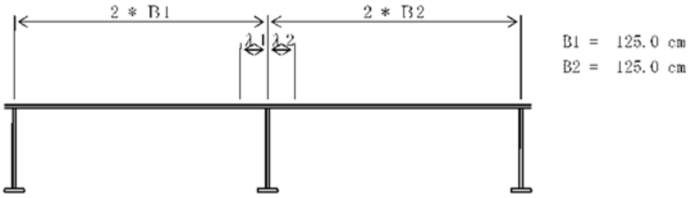
Distance between the fixed points L = 3.000 m

Equivalent Span Length



Li = 0.6 * L2 = 0.6 * 300.0 = 180.0 cm

Effective Width



B1/Li = 125.0 / 180.0 = 0.69

λ1 = 0.15 * Li = 0.15 * 180.0 = 27.0 cm

B2/Li = 125.0 / 180.0 = 0.69

λ2 = 0.15 * Li = 0.15 * 180.0 = 27.0 cm

Total effective width λ = λ1 + λ2 = 27.0 + 27.0 = 54.0 cm

Source: JICA Study Team

Figure 4.3.53 Sample Calculation of the Effective Width of the Cross Rib

Sectional Area and Moment of Inertia

			$A(cm^2)$	Y(cm)	$AY(cm^3)$	$I(cm^4)$
1-DECK	PL	540 * 16(SM400)	86.40	-40.80	-3525	143825
1-WEB	PL	800 * 9(SM400)	72.00	0.00	0	38400
1-LFLG	PL	200 * 10(SM400)	20.00	40.50	810	32805
			178.40		-2715	215030

E = -2715/ 178.40 = -15.22 cm

I = 215030 - 178.40 * -15.22^2 = 173708 cm^4

Y_u = -26.38 cm , Y_L = 56.22 cm

Bending stress

$\sigma_u = 112.3*10^6 * -263.8 / (173708*10^4) = -17 \text{ N/mm}^2 < \sigma_{ca} = 140 \text{ N/mm}^2$

$\sigma_L = 112.3*10^6 * 562.2 / (173708*10^4) = 36 \text{ N/mm}^2 < \sigma_{ta} = 140 \text{ N/mm}^2$

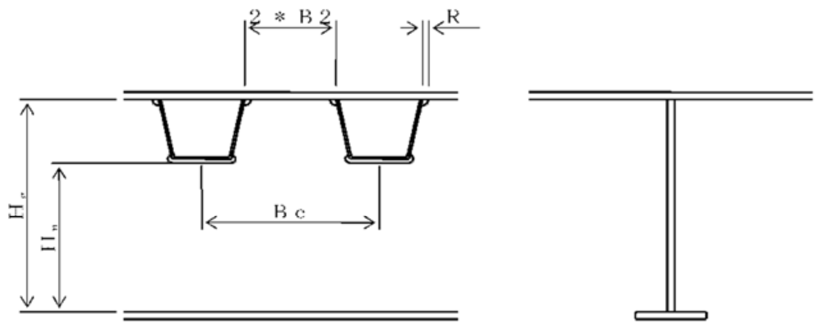
Shear stress

$\tau = 16.6*10^3 / 7200 = 2 \text{ N/mm}^2 < \tau_a = 80 \text{ N/mm}^2$

Combined stress

$\kappa = (36 / 140)^2 + (2 / 80)^2 = 0.07 < 1.2$

Shear Stress of Vertical Rib-Missing Part



$\tau_k = 16.6*10^3 / 7200 = 2 \text{ N/mm}^2 < \tau_a = 80 \text{ N/mm}^2$

$\tau_v = \tau_k * H_g / H_n = 2 * 80.0 / 52.3 = 4 \text{ N/mm}^2 < \tau_a$

Verification of Deflection

Deflection due to live load $\delta = 0.2$ mm $\leqq \delta_a = L / 500 = 3000 / 500 = 6.0$ mm

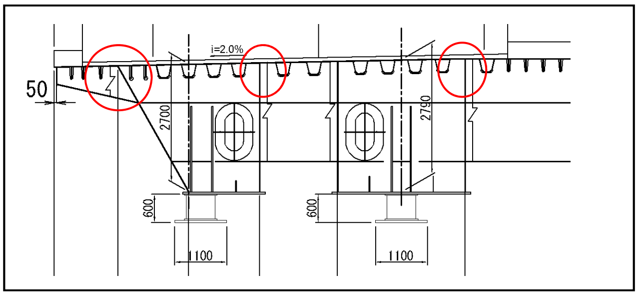
Source: JICA Study Team

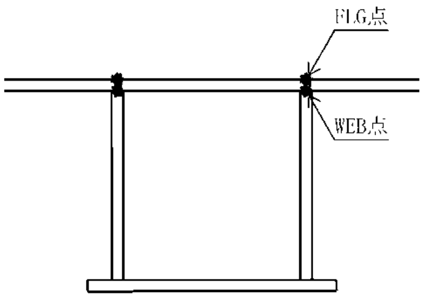
Figure 4.3.54 Sample Calculation of the Cross rib

(7) Biaxial Stress at the Cross Point of Both Webs of the Main Girder and Cross Rib

Calculation procedure of converted stress is as the follows.

- The cross point at both webs is shown in the following figure. This point has biaxial stress, main girder stress, and cross rib stress. These stresses are orthotropic respectively.





- The checking location is shown by the black dot in the sketch above.
- The maximum value is based on the checking of the right and left sides of the node point respectively, and of positive and negative stress, respectively.
- Biaxial calculation formula is shown as follows:

  K = (σx / σa)2 - (σx / σa) * (σy / σa) + (σy / σa)2 + (τ / τa)2 ≦ 1.2

  σ x : Normal stress of the main girder (N/mm^2)

  σ y : Normal stress of the crossbeam (N/mm^2)

  σ a : Allowable tensile stress of the main girder (N/mm^2)

  τx : Shear stress of the main girder (N/mm^2)

  τy : Shear stress of the crossbeam (N/mm^2)

  τa : Allowable shear stress of the main girder or crossbeam (N/mm^2)

Where, by checking location,

FLG point (τ / τa) = (τx / τxa)

WEB point (τ / τa) = Max((τx/τxa), (τy/Max(τxa,τya)))

- The stress of the main girder and cross ribs are calculated at every cross point.
- The calculation result is shown the following table.

Table 4.3.27 Calculation Result of the Biaxial Stress Check

1) Main girder G1 Web name : LWEB(UFLG-side)

	Cross section No.	No.	Check P't	σx	σy	σa	τx	τy	τa	K
End. sup.	1	1	FLG	0	14	140	25	0	80	0.10
	2	1	FLG	-32	11	140	22	0	80	0.15
	3	2	FLG	-52	11	140	19	0	80	0.23
	4	2	FLG	-74	11	140	16	0	80	0.37
	5	3	FLG	-84	11	140	14	0	80	0.44
	6	3	FLG	-95	11	140	10	0	80	0.53
	7	4	FLG	-98	11	140	8	0	80	0.56
	8	5	FLG	-100	11	140	5	0	80	0.57
	9	5	FLG	-97	11	140	7	0	80	0.54
	10	6	FLG	-91	11	140	10	0	80	0.49
	11	6	FLG	-79	11	140	12	0	80	0.39
	12	7	FLG	-66	11	140	15	0	80	0.30
	13	7	FLG	-43	11	140	18	0	80	0.17
	14	9	FLG	-21	11	140	20	0	80	0.10
	15	10	FLG	31	0	140	22	0	80	0.13
	16	10	FLG	65	0	140	24	0	80	0.30
Int. sup.	17	11	FLG	102	0	210	25	0	120	0.28
	18	12	FLG	72	0	140	19	0	80	0.32
	19	12	FLG	47	0	140	17	0	80	0.15
	20	13	FLG	29	0	140	14	0	80	0.07
	21	14	FLG	17	0	140	12	0	80	0.04
	22	14	FLG	-20	11	140	9	0	80	0.05
	23	15	FLG	-25	11	140	8	0	80	0.06
	24	15	FLG	-29	11	140	6	0	80	0.07
	25	16	FLG	-26	11	140	8	0	80	0.07
	26	17	FLG	-21	11	140	9	0	80	0.05
	27	17	FLG	27	0	140	11	0	80	0.06
	28	18	FLG	41	0	140	13	0	80	0.11
	29	18	FLG	60	0	140	17	0	80	0.23
	30	19	FLG	80	0	210	19	0	120	0.17
	31	20	FLG	102	0	210	21	0	120	0.27
	32	20	FLG	138	0	210	24	0	120	0.47
Int. sup.	33	21	FLG	173	0	210	30	0	120	0.74
	34	22	FLG	128	0	210	30	0	120	0.44
	35	22	FLG	84	0	210	28	0	120	0.21
	36	23	FLG	47	0	140	27	0	80	0.23
	37	24	FLG	-14	11	140	26	0	80	0.13
	38	24	FLG	-51	11	140	23	0	80	0.25
	39	26	FLG	-78	11	140	20	0	80	0.42
	40	26	FLG	-103	11	140	17	0	80	0.65
	41	27	FLG	-121	11	210	15	0	120	0.38
	42	28	FLG	-134	11	210	11	0	120	0.45
	43	28	FLG	-145	11	210	12	0	120	0.53
	44	29	FLG	-152	11	210	10	0	120	0.57
	45	29	FLG	-156	11	210	7	0	120	0.60
	46	30	FLG	-157	11	210	6	0	120	0.60
	47	31	FLG	-154	11	210	8	0	120	0.58
	48	31	FLG	-147	11	210	9	0	120	0.53
	49	32	FLG	-138	11	210	12	0	120	0.48
	50	32	FLG	-124	11	210	14	0	120	0.40
	51	33	FLG	-110	11	210	17	0	120	0.33
	52	34	FLG	-94	11	140	19	0	80	0.56
	53	34	FLG	-69	11	140	22	0	80	0.36
	54	35	FLG	-40	11	140	26	0	80	0.22
End. sup.	55	35	FLG	0	14	140	29	0	80	0.14

Source: JICA Study Team

- Index value K is smaller than 1.2 at all of the cross points. This means that the cross ribs have sufficient capacity for the wheel load.

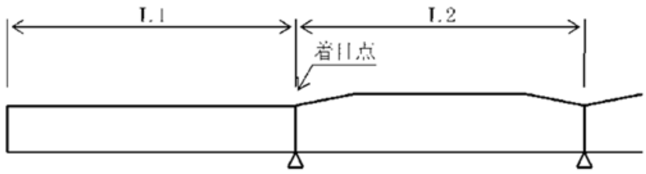
(8) Calculation of Bracket

- Bracket is cantilevered out from the web of outer main girder, and located at every 2.5 m spacing, so the stress is in the transverse direction of the primary stress of the main girder.
- The considered critical condition is the case when the wheel load acts on the point at the forehead of the bracket.
- These brackets are calculated as an I-beam section with the effective width of the deck plate as the top flange.

Sectional Force and Conditions of the Calculation

Section force (moment)	M =	-94.7 kN·m	S = 167.9 kN
Section force at joint	M_j =	-94.7 kN·m	S_j = 167.9 kN
Shear force of missing part	S_k =	167.9 kN	
Distance between fixed points	L =	1.330 m	

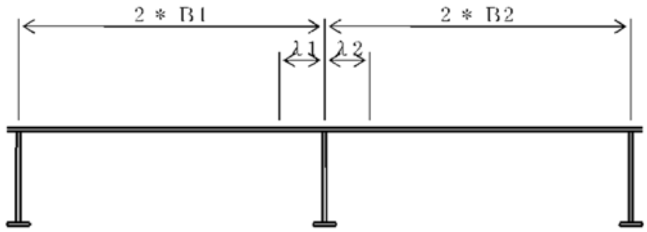
Equivalent Span Length



L1 = 133.0 cm

Li = 2 * L1 = 2 * 133.0 = 266.0 cm

Effective Width



B1 = 135.0 cm
B2 = 135.0 cm

B1/Li	= 135.0 / 266.0 = 0.51
λ1	= 0.15 * Li = 0.15 * 266.0 = 39.9 cm
B2/Li	= 135.0 / 266.0 = 0.51
λ2	= 0.15 * Li = 0.15 * 266.0 = 39.9 cm
Total effective width	λ = λ1 + λ2 = 39.9 + 39.9 = 79.8 cm

Source: JICA Study Team

Figure 4.3.55 Sample Calculation of the Effective Width of the Bracket

Sectional Area and Moment of Inertia

			$A(cm^2)$	$Y(cm)$	$AY(cm^3)$	$I(cm^4)$
1-DECK	PL	798 * 16(SM400)	127.68	-40.80	-5209	212541
1-WEB	PL	800 * 9(SM400)	72.00	0.00	0	38400
1-LFLG	PL	200 * 10(SM400)	20.00	40.50	810	32805
			219.68		-4399	283746

$E = -4399 / 219.68 = -20.03$ cm

$I = 283746 - 219.68 * -20.03^2 = 195644\ cm^4$

$Y_u = -21.57$ cm, $Y_L = 61.03$ cm

Bending stress

$\sigma_u = -94.7*10^6 * -215.7 / (195644*10^4) = 10\ N/mm^2 < \sigma_{ta} = 140\ N/mm^2$

$\sigma_L = -94.7*10^6 * 610.3 / (195644*10^4) = -30\ N/mm^2 < \sigma_{ca} = 133\ N/mm^2$

Shear stress

$\tau = 167.9*10^3 / 7200 = 23\ N/mm^2 < \tau_a = 80\ N/mm^2$

Combined stress;

$\kappa = (-29 / 140)^2 + (23 / 80)^2 = 0.13 < 1.2$

∗ For the Joint (attachment/spliced part)

Bending stress

$\sigma_u = -94.7*10^6 * -215.7 / (195644*10^4) = 10\ N/mm^2 < \sigma_{ta} = 140\ N/mm^2$

$\sigma_L = -94.7*10^6 * 610.3 / (195644*10^4) = -30\ N/mm^2 < \sigma_{ca} = 133\ N/mm^2$

Shear stress

$\tau = 167.9*10^3 / 7200 = 23\ N/mm^2 < \tau_a = 80\ N/mm^2$

Combined stress

$\kappa = (-29 / 140)^2 + (23 / 80)^2 = 0.13 < 1.2$

Source: JICA Study Team

Figure 4.3.56 Sample Calculation of the Bracket

(9) Calculation of the Longitudinal Side Beam

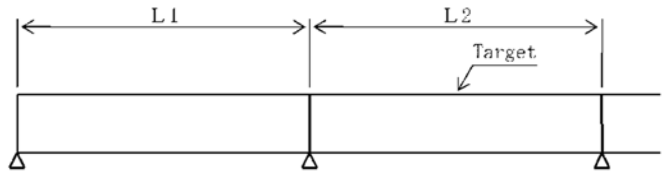
- Longitudinal side beam has the distribution function for the load between two brackets.
- This member is calculated as a beam with the effective width of the deck plate for the top flange.

Sectional Force and Conditions of the Calculation

Sectional force M = 1.6 kN·m S = 0.1 kN

Distance between the fixed points L = 2.500 m

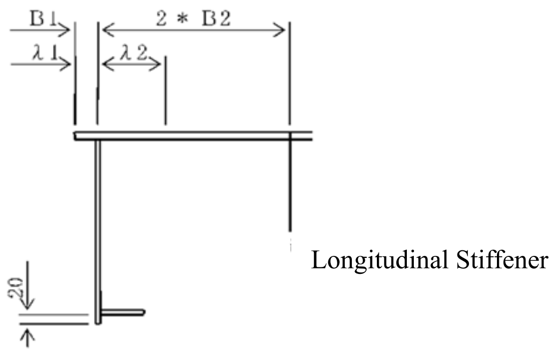
Equivalent Span Length



L1 = 250.0 cm
L2 = 250.0 cm

Li = 0.6 * L2 = 0.6 * 250.0 = 150.0 cm;

Effective Width



B1 = 5.0 cm
B2 = 20.8 cm

B1/Li = 5.0 / 150.0 = 0.03

$\lambda 1 = \{ 1.06 - 3.2 * (B1/Li) + 4.5 * (B1/Li)^2 \} * B1$

$= \{ 1.06 - 3.2 * \quad 0.03 + 4.5 * \quad 0.03^2 \} * 5.0$

$= 4.8$ cm

B2/Li = 20.8 / 150.0 = 0.14

$\lambda 2 = \{ 1.06 - 3.2 * (B2/Li) + 4.5 * (B2/Li)^2 \} * B2$

$= \{ 1.06 - 3.2 * \quad 0.14 + 4.5 * \quad 0.14^2 \} * 20.8$

$= 14.6$ cm

Total effective width $\lambda 1 + \lambda 2 = 4.8 + 14.6 = 19.4$ cm

Source: JICA Study Team

Figure 4.3.57 Calculation of the Effective Width for the Longitudinal Beam

Sectional Area and Moment of Inertia

			$A(cm^2)$	$Y(cm)$	$AY(cm^3)$	$I(cm^4)$
1-DECK	PL	194 * 16(SM400)	31.02	-20.80	-645	13421
1-WEB	PL	400 * 10(SM400)	40.00	0.00	0	5333
1-LFLG	PL	100 * 10(SM400)	10.00	17.50	175	3063
			81.02		-470	21817

$E = -470 / 81.02 = -5.80$ cm

$I = 21817 - 81.02 * -5.80^2 = 19088$ cm^4

$Y_u = -15.80$ cm , $Y_L = 25.80$ cm

Bending stress

$\sigma_u = 1.6*10^6 * -158.0 / (19088*10^4) = -1$ $N/mm^2 < \sigma_{ca} = 140$ N/mm^2

$\sigma_L = 1.6*10^6 * 258.0 / (19088*10^4) = 2$ $N/mm^2 < \sigma_{ta} = 140$ N/mm^2

Shear stress

$\tau = 0.1*10^3 / 4000 = 0$ $N/mm^2 < \tau_a = 80$ N/mm^2

Combined stress

$\kappa = (2 / 140)^2 + (0 / 80)^2 = 0.00 < 1.2$

Verification of the Deflection

Deflection due to live load $\delta = 0.1$ mm $\leqq \delta_a = L / 500 = 2500 / 500 = 5.0$ mm

Calculation of Stiffener

b = 40.0 : Abdominal plate height (cm)

t = 1.0 : Abdominal plate thickness (cm)

σ = 1 : Edge compressive stress intensity of abdominal plate (N/mm^2)

τ = 0 : Shear stress intensity of abdominal plate (N/mm^2)

Verification of Abdominal Plate Thickness

$K_h = \sqrt{(\sigma_a / \sigma)} = \sqrt{(140 / 1)} = 10.8 \quad \therefore K_h = 1.2$

$b / (152 * K_h) = 40.0 / (152 * 1.2) = 0.2$ cm $< t = 1.0$ cm

The horizontal stiffener is omitted.

Source: JICA Study Team

Figure 4.3.58 Calculation of the Longitudinal Beam

4.3.4.5 Summary of Steel Weight

(1) Quantity Table of Main Girder

- Steel materials are classified in accordance with each category, grade, and thickness.

Table 4.3.28 Quantity Table of Steel Box Girder

(Unit:kg)

Type	Material	Thickness	Deck PL	Deck PL Longitudinal Splice	Deck PL Transverse Splice	Girder	Girder Splice	Cross Girder	Cross Girder Splice	Diaphragm	Diaphragm Splice	Crossbeam	Crossbeam Splice	Inspection Walkway	Total
PL	SM570-H	44				13316									13316
		42				12683									12683
		41				4782									4782
	SM570-H Subtotal					30781									30781
	SM570	38				8767									8767
		36				4120									4120
		32				3582									3582
		30					993								993
		29				3223	916								4139
		28					441								441
		27				3010									3010
		26				2890	740								3630
		25					356								356
		24					929								929
		23					1272								1272
		22				7596	1479								9075
		21					935								935
		20					333								333
		19					214								214
		18					468								468
		17				85340	222								85562
		16					209								209
		15					196								196
		11					5220								5220
		10					4740								4740
	SM570 Subtotal					118528	19663								138191
	SM520C-H	57				4897									4897
		52				4400									4400
		50						920							920
		45				14432									14432
		44						808							808
		42				3732									3732
		41				8515									8515
	SM520C-H Subtotal					35976		1728							37704
	SM490YB	40				4544									4544
		38				22847									22847
		36				18398									18398
		35				13652									13652
		34				6574									6574
		32				6310									6310
		31					955								955
		30				6856									6856
		29				3277	440								3717
		28				12441	1137	2216							15794
		27				15061									15061
		26				10828	360								11188
		25				6697	1404	3846							11947
		24	64136			5531	944								70611
		23				2649	2103								4752
		22				45297	1607	6812							53716
		21				7869	4529								12398
		20				13850	1913								15763
		19				4700	4693								9393
		18			5206	12424	624								18254
		17				131122	5908								137030
	SM490YB Subtotal		64136		5206	350927	26617	12874							459760
	SM490YA	16	255218			15364	1422								272004
		15				228309	3277								231586
		14			9089	318352	501								327942
		13				3960	1466								5426
		12				18464	566								19030
		11				3892	5043								8935
		10			2506	53279	41637								97422
		9		23464	8328		55822								87614
	SM490YA Subtotal		255218	23464	19923	641620	109734								1049959
	SM490C	52				1316									1316
		43				1080									1080
		41				1200									1200
	SM490C Subtotal					3596									3596
	SM490B	40				1072									1072
	SM400B	38						418							418
	SM400A	32						1056							1056
		24	150185												150185
		22				2364		1238							3602
		19	486			43838		2580				12852			59756
		17				20690									20690
		16	512702					17305							530007
		15				25121						7236			32357
		13				1978									1978
		12	1840												1840
		11				12370		3178							15548
		10	5876			108		18752		17174		26071			67981
		9	14961			3200		80029		57758		81741			237689
		6	5945												5945
	SM400A Subtotal		691995			109669		124138		74932		127900			1128634
	SS400	21					241								241
		20					105								105
		19	756				80								836
		18			10304										10304
		16			896										896
		15					152								152
		14			19811		307								20118
		13					165								165
		12					196		168						364
		11					207		356						563
		10			5063		291								5354

		9	1090	47382	18459		439		23846		8741		17450		117407
		8					260								260
		6			2221		635								2856
		4.5					270								270
		3.2					196								196
		2.3					739								739
	SS400 Subtotal		1846	47382	56754		4283		24370		8741		17450		160826
PL Subtotal			1013195	70846	81883	1292169	160297	139158	24370	74932	8741	127900	17450		3010941
FB	SS400	65* 6												4752	4752
		50* 6	115			758									873
FB Subtotal			115			758								4752	5625
L	SS400	65* 65* 6												26475	26475
U	SM490YA	320* 240* 8	92871												92871
	SM400A	320* 240* 8	194269												194269
U Subtotal			287140												287140
STK	STK400	165.2* 4.5	486												486
RB	SS400	13 φ			254										254
EXP	XG11	600												8605	8605
TCB	S10T	M 22		23171	21763		44549		10687		4086		8033		112289
HTB	F10T	M 22			10149		1810								11959
BN	SS400	M 12			485									1296	1781
	SUS304	M 16	82												82
BN Subtotal			82		485									1296	1863
Chain	SUS304	5*18*42*250	54												54
Component Weight Total			1301072	94017	114534	1292927	206656	139158	35057	74932	12827	127900	25483	41128	3465691
Total			1301072	94017	114534	1292927	206656	139158	35057	74932	12827	127900	25483	41128	3465691

Source: JICA Study Team

(2) Segment Weight for Erection Block

- The block weight is a very important factor to be considered in the erection procedure.
- The following table shows each segment's weight and assumed pre-assembly weight.

Table 4.3.29 Table of Segment Weight for Erection Block

				unit: kgf
	G1	G2	G3	G4
Block-1	21,281	20,709	20,669	21,898
Block-2	18,733	16,952	17,489	20,162
Block-3	20,428	18,589	19,016	22,330
Block-4	20,360	18,031	18,842	22,465
Block-5	20,860	18,671	19,492	22,854
Block-6	19,886	18,150	18,528	21,737
Block-7	18,686	17,165	17,534	19,915
Block-8	18,157	17,182	17,307	18,460
Block-9	16,627	15,787	15,785	17,298
Block-10	16,820	15,953	16,438	17,746
Block-11	16,690	15,693	15,667	17,328
Block-12	18,349	17,338	17,302	18,201
Block-13	18,887	18,190	18,169	18,768
Block-14	16,277	15,669	15,683	16,148
Block-15	18,563	17,707	17,686	18,446
Block-16	16,280	15,692	15,681	16,612
Block-17	19,111	18,127	18,088	19,025
Block-18	20,688	18,432	18,430	21,074
Block-19	20,980	17,927	18,044	21,485
Block-20	21,773	19,458	19,604	22,466
Block-21	18,636	16,582	16,689	19,039
Block-22	16,979	15,177	15,312	17,215
Block-23	18,617	17,428	17,197	18,398
Block-24	20,712	18,726	18,482	20,164
Block-25	21,829	20,570	20,285	21,264
Block-26	24,244	23,782	23,011	23,459
Block-27	23,179	22,943	22,499	22,793
Block-28	20,879	20,424	20,055	20,615
Block-29	20,782	20,839	20,430	20,544
Block-30	23,125	23,411	23,154	22,838
Block-31	20,329	20,470	20,208	20,157
Block-32	19,685	18,674	18,411	19,479
Block-33	18,639	17,679	17,520	18,253

Source: JICA Study Team

4.3.5 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.

4.3.5.1 Design Conditions

(1) Standard and Design Criteria

- Specifications for Highway Bridges Part-I, IV, V 2012 (Japan Road Association)
- Design and construction handbook for Steel Sheet Pile Foundation Method 1997 (Japan Road Association)
- JIS A5530, Japanese Industrial Standard for Steel Pipe Sheet Pile
- JIS G3444 STK400, Japanese Industrial Standard for Interlocking Pipe

(2) Analytical Software for Design

- UC-1 Foundation Design developed by Forum 8 for SPSP Foundation
- UC-1 Substructure Design developed by Forum 8 for RC Pier

(3) Materials to be Used

- Concrete : σ_{ck} = 30 N/mm^2 [for pier column and beam]
  : σ_{ck} = 24 N/mm^2 [for footing (top slab concrete)]
  : σ_{ck} = 21 N/mm^2 [for bottom slab concrete, concrete filling to steel pipe]
- Reinforcing bars: SD390 [for main reinforcement of pier column]
  : SD345 [for other members]
- Reinforcing stud bars: SD345 [for connection between footing and steel sheet pile]
- Steel sheet pile : SKY400, SKY490
  Estimated corrosion thickness: 2.0 mm/100 years
- Pipe-pipe interlocking joint: STK400 φ165.2 x t11

(4) Design Soil Condition

At the section of the steel box bridge, two boreholes in the F/S and four boreholes in D/D, a total of six boreholes, were performed. Boreholes at P15 and P19 are far from the center of the foundation, which are around 37 m and 28 m away, respectively. Soil profile and geotechnical design parameters were established based on the laboratory soil test results and field test results, and design parameters such as N-value, unit weight, internal friction angle, cohesive strength and deformation modulus are provided as the average value in Bago River section as presented in Chapter 2.1 Soil Investigation.

Table 4.3.30 Borings at Steel Box Bridge Section

Pier No.	Boring No.	Boring Location
P14	No.BD-07 (D/D)	center of the foundation
P15	No.13BH-03 (F/S)	37 m from the center of the foundation
P16	No.BD-06 (D/D)	center of the foundation
P17	No.BD-05 (D/D)	center of the foundation
P18	No.BD-04 (D/D)	center of the foundation
P19	No.13BH-04 (F/S)	28 m from the center of the foundation

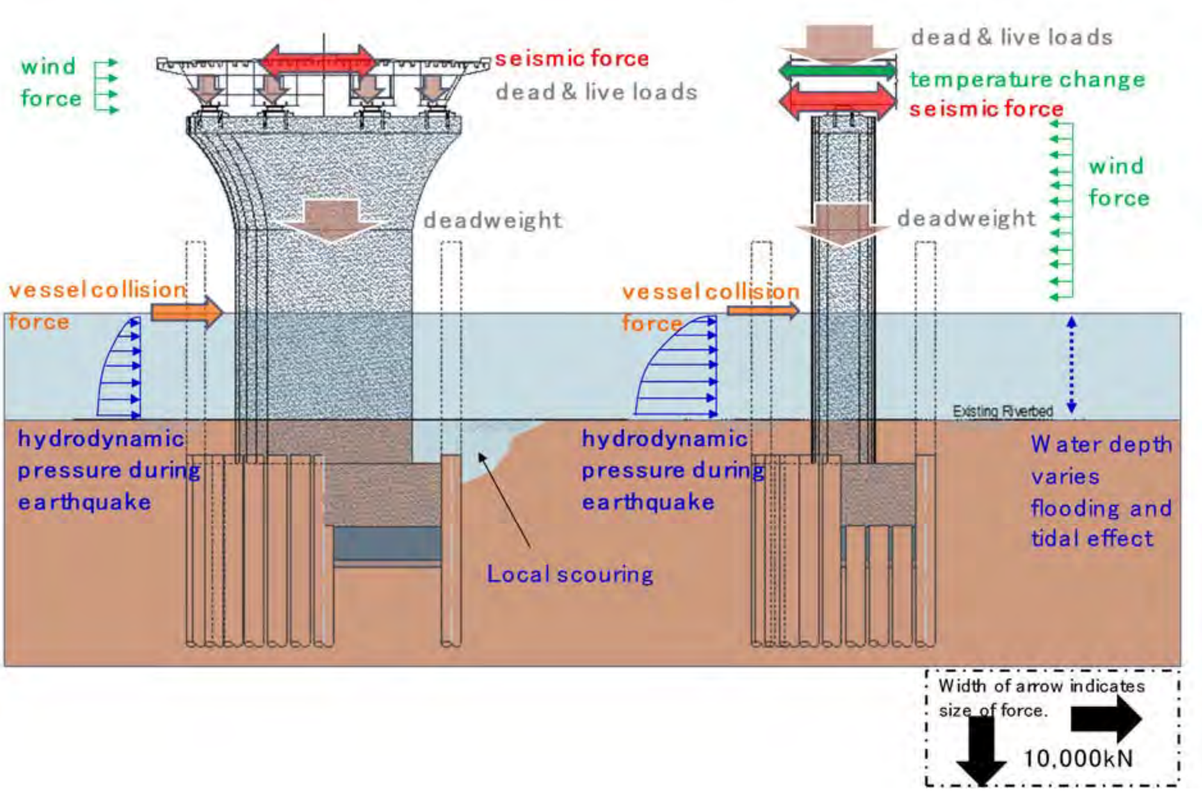
Source: JICA Study Team

Since there is a saturated soil layer having ground water level higher than 10 m below the ground surface and located at a depth less than 20 m below the ground surface, liquefaction potential is evaluated and deduction factor due to liquefaction at the time of earthquake is considered in the foundation design.

(5) Loads and Load Combinations

1) Loads

For substructure design, various forces including earth pressure, water pressure, wind loads, effect of temperature change, collision load of vessel, flowing water pressure and hydrodynamic pressure during earthquake as shown in the figure below were properly considered as critical load combination.



Source: JICA Study Team

Figure 4.3.59 Image of External Forces to be Considered

2) Load Combination

The design cases and corresponding allowable stress for members are shown in the following tables.

Table 4.3.31 Load Combination and Allowable Stress in Pier Column and Foundation Design

Load Combination		Design Water Level (MSL+m)		Water Velocity (m/s) for Flowing Water Pressure	Local Scouring	Increase of Allowable Stress
A.	Ordinary Condition	High tide in spring tide	3.18 m	No consideration	No consideration	1.0
					Maximum	
		Low tide in spring tide	-2.39 m	No consideration	No consideration	
					Maximum	
B.	Ordinary condition with effect of temperature change *only for longitudinal direction	High tide in spring tide	3.18 m	No consideration	No consideration	1.15
					Maximum	
		Low tide in spring tide	-2.39 m	No consideration	No consideration	
					Maximum	
C.	Extreme wind situation with effect of temperature change	HHWL (1%)	4.99 m	No consideration	No consideration	1.35 *1.25 for transversal direction
					1/2 of maximum	
D.	Vessel Collision for P14	High tide in spring tide	3.18 m	No consideration	No consideration	1.5
					1/2 of maximum	
	Vessel Collision for P15-P19	At maximum water flow	2.53 m	1.19 m/s	No consideration	
					1/2 of maximum	
E.	Earthquake Condition (Level-1)	Average	0.29 m	0.6 m/s for dynamic water pressure	No consideration	1.5
					1/2 of maximum	
F.	During Construction	HWL (5%)	4.34 m	0.65 m/s	No consideration	1.5
		Low tide in spring tide	-2.39 m			

Source: JICA Study Team

Table 4.3.32 Load Combination and Allowable Stress in Pier Beam Design

Load Combination		Increase of Allowable Stress
Vertical Direction		
G.	Ordinary Condition *live load with impact	1.0
H.	Earthquake Condition	1.5
Horizontal Direction		
I.	With effect of temperature change	1.15
J.	Earthquake Condition	1.5

Source: JICA Study Team

3) Loads from Superstructure

Dead load and live load with/without impact from superstructure for the substructure design is summarized in the table below.

Table 4.3.33 Dead Load and Live Load with/without Impact for the Substructure Design

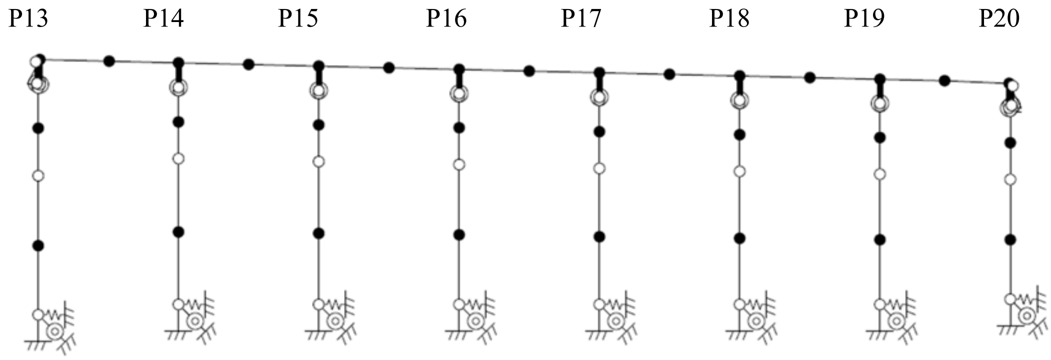
Loads	P13	P14	P15	P16	P17	P18	P19	P20
Dead Load	8,600	24,300	20,400	21,600	21,300	20,800	23,200	7,500
Live Load with Impact	4,100	7,900	7,600	7,700	7,700	7,600	7,700	3,900
ΣD+L+I	**12,700**	**32,200**	**28,000**	**29,300**	**29,000**	**28,400**	**30,900**	**11,400**
Dead Load	8,600	24,300	20,400	21,600	21,300	20,800	23,200	7,500
Live Load w/o Impact	3,600	7,000	6,800	6,900	6,800	6,700	6,800	3,400
ΣD+L	**12,200**	**31,300**	**27,200**	**28,500**	**28,100**	**27,500**	**30,000**	**10,900**
Horizontal Force due to temperature change ±15°	900	5,400	3,100	1,200	900	3,100	5,800	800

Note: - Values of P13 and P20 are just for reference.

- Friction force (dead load x 0.1) is considered for horizontal force at movable supports at P13 and P20.

Source: JICA Study Team

Since the structure which can distribute inertial force of earthquake into several substructures is applied for the 7-continuous spans, a shared weight on each substructure is calculated by Eigenvalue analysis using framed structure model as shown in the figure below.



Source: JICA Study Team

Figure 4.3.60 Framed Structure Model

The result is summarized in the table below.

Table 4.3.34 Shared Weight of Superstructure on Substructures

Item		P13	P14	P15	P16	P17	P18	P19	P20
Bridge Axis Direction									
-	Shared Weight (kN)	900	23,200	23,200	24,200	25,000	25,500	26,400	800
-	Natural period of oscillation of the bridge (second)	0.790(s)							
Bridge Axis Perpendicular Direction									
-	Shared Weight (kN)	8,600	24,300	20,400	21,600	21,300	20,800	23,200	7,500
-	Natural period of oscillation of the bridge (second)	0.52(s)	0.57(s)	0.54(s)	0.53(s)	0.53(s)	0.53(s)	0.54(s)	0.46(s)

Note: - Values of P13 and P20 are just for reference.

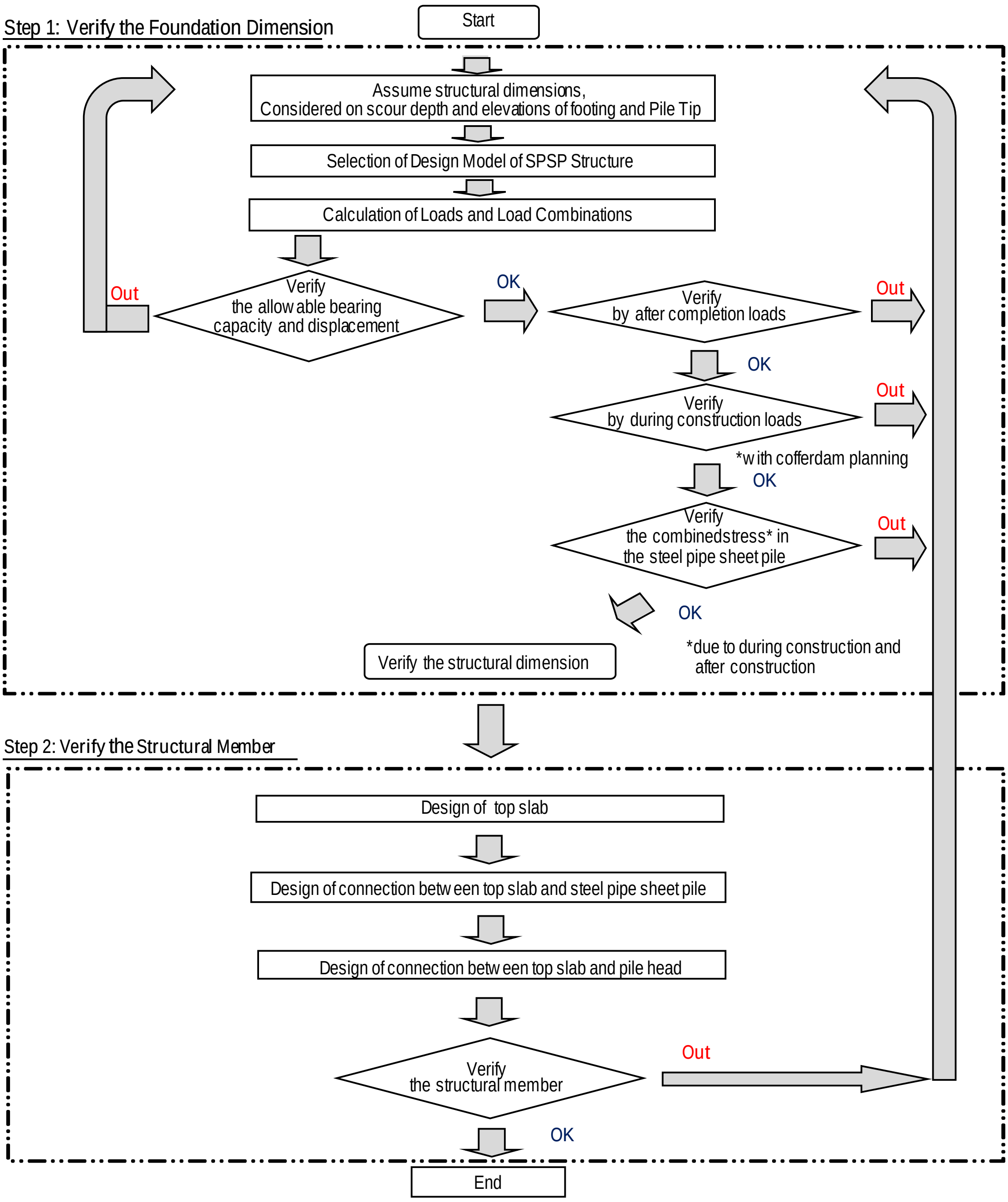
- The amount of the shared weight in bridge axis direction of P13 and P20 are considered as friction force which is calculated by dead load x 0.1.

Source: JICA Study Team

4.3.5.2 SPSP Foundation Design

(1) Design Flow

Detailed design of the SPSP foundation is carried out based on the flow as shown in the figure below.



Source: JICA Study Team

Figure 4.3.61 Design Flow for the Basic Design of the SPSP Foundation

(2) 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.

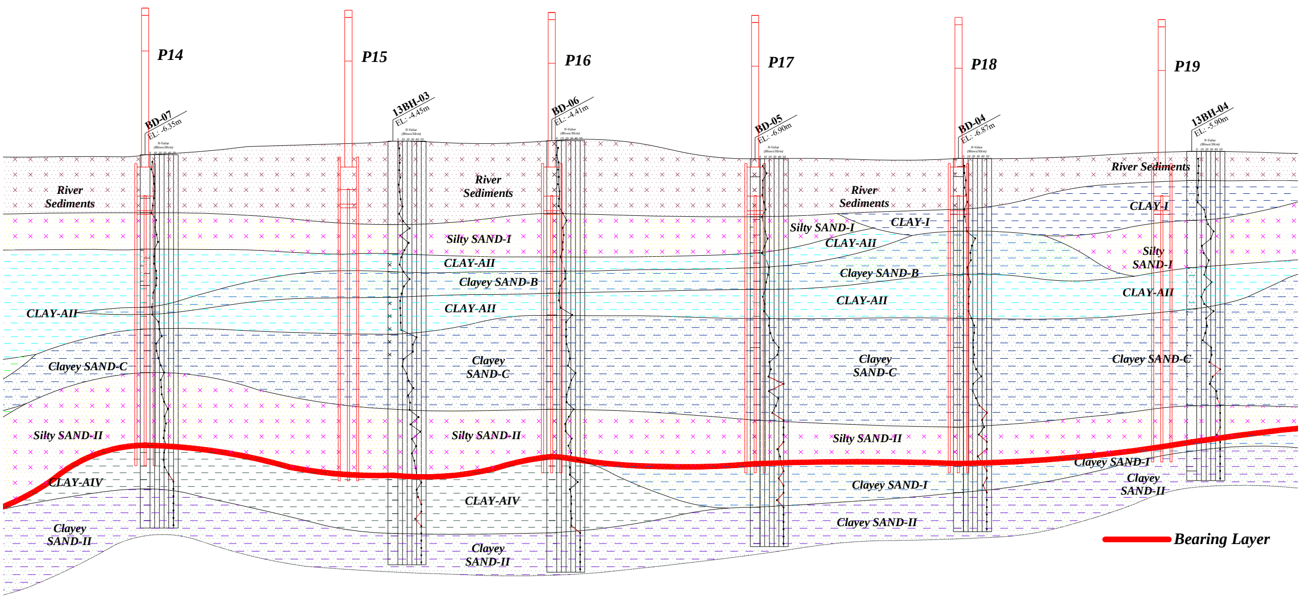
Table 4.3.35 Setting of Footing Top Elevation

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

(3) 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 4.3.62 Soil Profile and Pile Tip Position

(4) Design Model of SPSP Structure

As for the design model of the SPSP, if D < 30 m and L/D > 1 and βLe > 1, then finite-length beam on an elastic ground model is used, and if D > 30 m and L/D ≤ 1 and βLe ≤ 1, analysis by an imaginary well beam that considers shear slippage of the interlocking or three dimension model is applied.

Finite-length beam on an elastic ground is applied for the design model of the SPSP structure for all foundations from P14 to P19 based on the criteria mentioned above.

Table 4.3.36 Selection of the Design Model of the SPSP Structure

Pier No.	P14	P15	P16	P17	P18	P19
D (m)	17.16	17.16	17.16	17.16	17.16	17.16
L/D	2.39	2.57	2.45	2.45	2.45	2.37
βL_e	1.49	1.60	1.57	1.57	1.57	1.52

D (m): width of foundation: larger value among D (m) or B (m)

L (m): length of steel pipe pile

L_e (m): embedded length of foundation underground

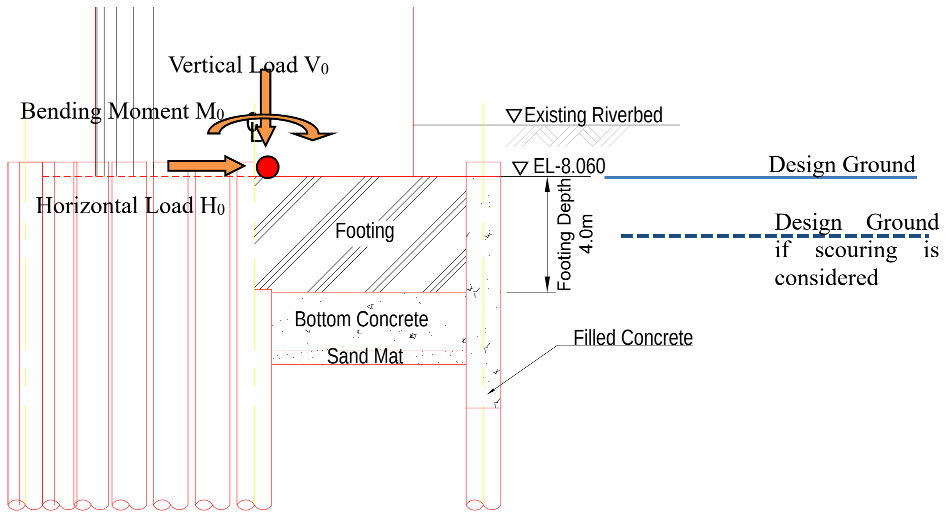
Β (1/m): characteristic factor of foundation

Source: JICA Study Team

(5) Design External Force

Design external force acting as point forces through the axis of the centroid on the center of the footing is considered for the SPSP foundation design as shown in the figure below. The external force (V_0,H_0,M_0) of the top of footing is considered. The vertical load V_0 includes weights of footing, filled concrete inside steel piles, soil on the footing and buoyancy of pier. If the footing projects due to local scouring, inertial forces working on the projected parts will be considered as distributed load in addition to the external force (V_0,H_0,M_0) of the top of footing.

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 4.3.63 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 4.3.37 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

(6) 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 4.3.38 Verification of Bearing Capacity

<u>Bride Axis Direction</u> 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

<u>Bridge Axis Perpendicular Direction</u> 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 4.3.39 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.

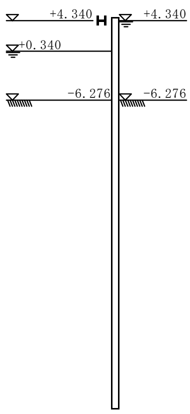
As the top slab concrete is placed with the steel pipe sheet piles in a deformed state, the residual stress (σ1) due to and remaining after work execution and the stress (σ2) occurring due to the design external forces after completion should be added. The sum (σ) shall be equal to the allowable stress (σa) or less.

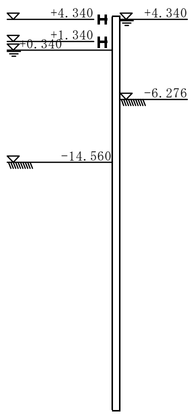
Because the stress occurring in the steel pipe sheet pile during drainage is influenced by the sequence of work execution, it is necessary to fully investigate the work sequence and execute the design according to that work.

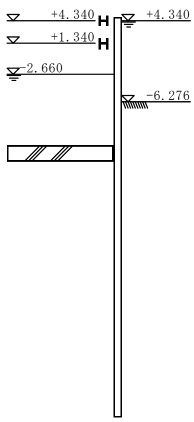
a) Construction Step of Temporary Cofferdam

The underwater/atmospheric excavation method is applied because the stress during drainage and residual stress can be smaller. The construction step of temporary cofferdam for the case of P14 is shown in the figure below, and other cases have similar steps as that of P14.

Schematic

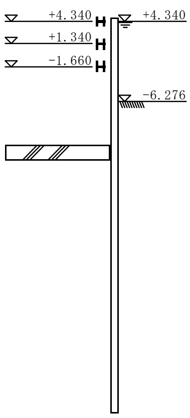


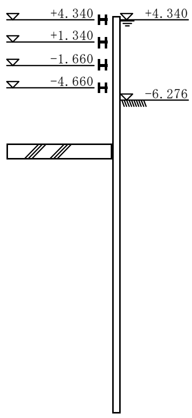


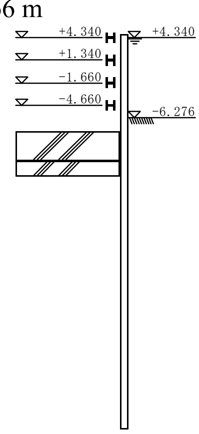


Work activities	Step-1: Install 1st support and drainage up to EL+0.34 m	Step-2: Install 2nd support and excavate up to EL-14.56 m	Step-3: Placement of bottom slab concrete and drainage up to EL-2.66 m

Schematic





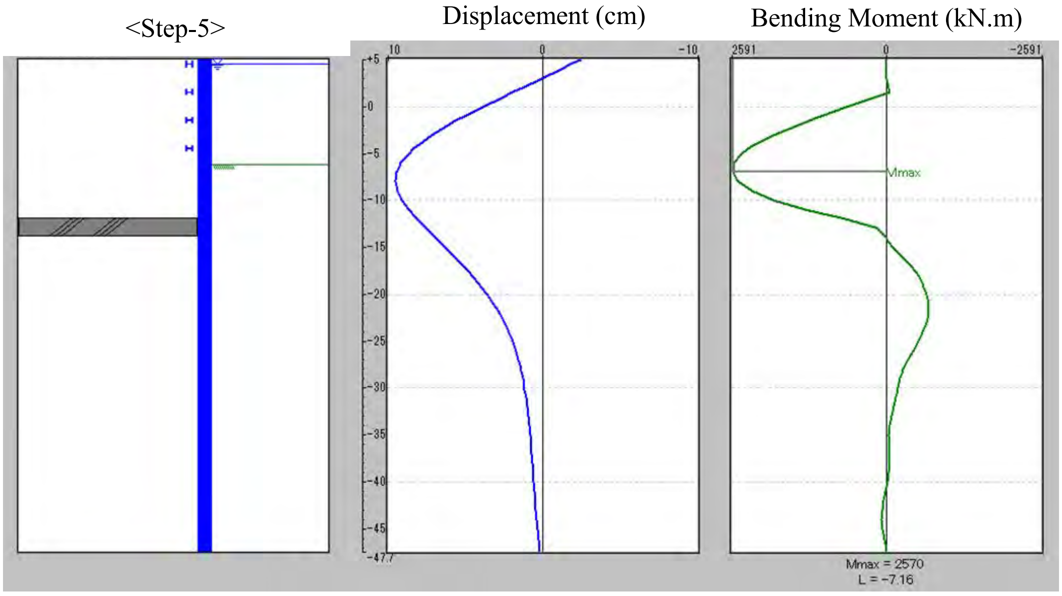


Work activities	Step-4: Install 3rd support and dry up inside well	Step-5: Install 4th support	Step-6: Construction of footing

Source: JICA Study Team

Figure 4.3.64 Construction Step of Temporary Cofferdam by Combined Underwater and Atmospheric Excavation (P14 Case)

As explained above, at the construction step just before construction of footing concrete, namely Step-5, residual stress of the pile will be considered. Diagram of the displacement and bending moment for the case of P14 is shown in the figure below, and the maximum displacement due to moment occurs between the lowest support and bottom slab concrete.

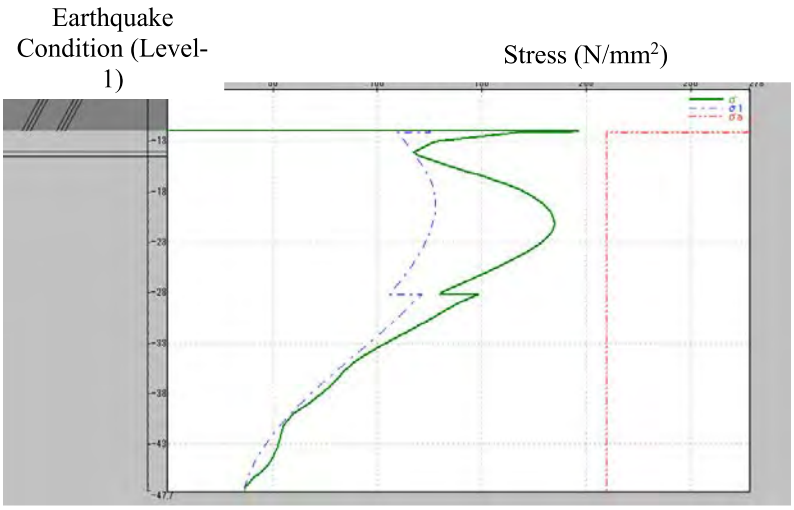


Source: JICA Study Team

Figure 4.3.65 Diagram of Displacement and Bending Moment at the Construction Step when Residual Stress of the Pile is Considered (P14 Case)

b) Combined stress of the pile during construction and due to the design external forces after completion

The following figure shows that combined stress is within the allowable stress under earthquake condition.



where

σ : combined stress (= σ1+σ2), σ1 : stress after completion loads.

σ2 : residual stress during construction, σa : allowable stress in steel pipe sheet pile

Source: JICA Study Team

Figure 4.3.66 Combined Stress for the SPSP of P14 at Earthquake Condition

Table 4.3.40 Verification of SPSP (SKY400 part) Combined Stress at Ordinary Condition

Bridge Axis Direction

Pier	Elevation (m)	σ1*1 (N/mm^2)	σ2 (N/mm^2)	σ_{max} (N/mm^2)	σa (N/mm^2)	Judgement
P14	-22.09	35.86	58.61	94.48 <	140	OK
P15	-22.59	34.06	66.67	100.73 <	140	OK
P16	-12.16	34.52	64.26	98.78 <	140	OK
P17	-22.66	33.35	85.54	118.89 <	140	OK
P18	-20.16	32.71	81.03	113.74 <	140	OK
P19	-33.16	38.63	77.11	115.74 <	140	OK

Bridge Axis Perpendicular Direction

Pier	Elevation (m)	σ1*1 (N/mm^2)	σ2 (N/mm^2)	σ_{max} (N/mm^2)	σa (N/mm^2)	Judgement
P14	-12.16	35.86	60.90	96.76 <	140	OK
P15	-22.59	34.06	68.65	102.71 <	140	OK
P16	-12.16	34.52	68.61	103.14 <	140	OK
P17	-22.66	33.35	87.63	120.98 <	140	OK
P18	-20.16	32.71	83.20	115.91 <	140	OK
P19	-33.16	38.63	77.11	115.74 <	140	OK

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

Source: JICA Study Team

Table 4.3.41 Verification of SPSP (SKY400 part) Combined Stress at Earthquake Condition

Bridge Axis Direction

Pier	Elevation (m)	σ1*1 (N/mm^2)	σ2 (N/mm^2)	σ_{max} (N/mm^2)	σa (N/mm^2)	Judgement
P14	-21.16	126.74	58.58	185.32 <	210	OK
P15	-22.16	119.45	66.14	185.59 <	210	OK
P16	-22.06	113.95	62.61	176.56 <	210	OK
P17	-22.06	111.39	85.41	196.80 <	210	OK
P18	-20.16	125.14	81.03	206.18 <	210	OK
P19	-19.16	120.04	66.40	186.44 <	210	OK

Bridge Axis Perpendicular Direction

Pier	Elevation (m)	σ1*1 (N/mm^2)	σ2 (N/mm^2)	σ_{max} (N/mm^2)	σa (N/mm^2)	Judgement
P14	-21.16	127.23	59.34	186.57 <	210	OK
P15	-22.59	111.90	68.65	180.55 <	210	OK
P16	-22.16	109.21	64.47	173.68 <	210	OK
P17	-22.16	105.36	87.00	193.02 <	210	OK
P18	-20.16	111.46	83.20	194.66 <	210	OK
P19	-19.16	112.04	68.44	180.48 <	210	OK

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

Source: JICA Study Team

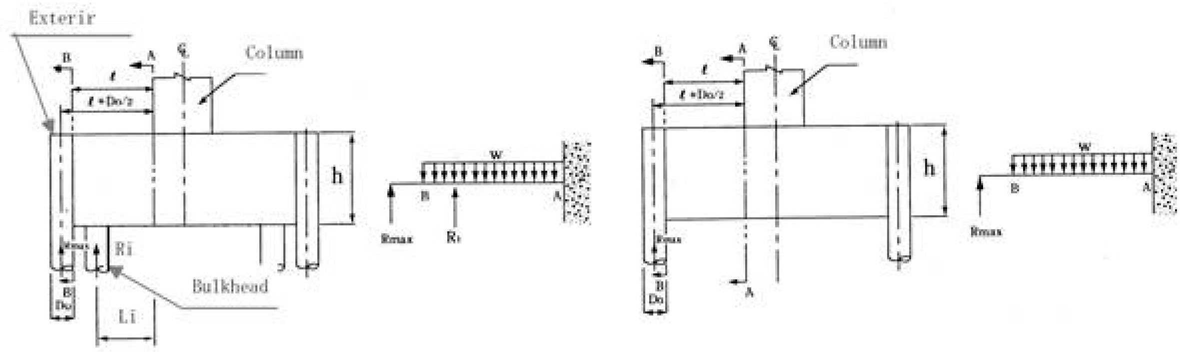
(7) Verification of Structural Members

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 4.3.67 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

Bridge Axis Direction

Upper tension:	cover 150 mm D32@260
	cover 300 mm D32@260
Lower tension:	cover 300 mm D51@183
	cover 500 mm D51@302

Bridge Axis Perpendicular Direction

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

Bridge Axis Direction

Upper tension:	cover 150 mm D32@260
	cover 300 mm D32@260
Lower tension:	cover 300 mm D51@183
	cover 500 mm D51@370

Bridge Axis Perpendicular Direction

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.

Verification of the footing structure is summarized in the table below.

Table 4.3.42 Verification of Footing Structure

Bridge Axis Direction

Pier No.	Item	Ordinary Condition*1			Earthquake Condition*2		
		Stress/Rebar Content	Allowable Value	Judgement	Stress/Rebar Content	Allowable Value	Judgement
P14	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c: 2.85 <	12	OK
		σ_s: 0.00 <	160	OK	σ_s:178.13<	300	OK
	Lower tensile stress	σ_c: 2.16 <	8	OK	σ_c: 5.68 <	12	OK
		σ_s: 71.81 <	160	OK	σ_s:189.34<	300	OK
	Rebar Content	177.88 >	111.49	OK	177.88 >	157.11	OK
	Shear stress	τ_m: 0.33 <	1.00	OK	τ_m: 0.85<	1.51	OK
P15	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c: 2.89<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:180.20<	300	OK
	Lower tensile stress	σ_c: 2.05 <	8	OK	σ_c: 5.51<	12	OK
		σ_s:70.92 <	160	OK	σ_s:191.13<	300	OK
	Rebar Content	165.55 >	103.74	OK	165.55 >	149.10	OK
	Shear stress	τ_m: 0.31 <	0.98	OK	τ_m: 0.81 <	1.51	OK
P16	Upper tensile stress	σ_c:0.00<	8	OK	σ_c:2.89<	12	OK
		σ_s:0.00<	160	OK	σ_s:180.48<	300	OK
	Lower tensile stress	σ_c:2.08<	8	OK	σ_c:5.59<	12	OK
		σ_s:72.19<	160	OK	σ_s:193.72<	300	OK
	Rebar Content	165.55 >	105.59	OK	165.55 >	151.13	OK
	Shear stress	τ_m:0.31<	0.98	OK	τ_m:0.79<	1.51	OK
P17	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c:2.94<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:183.79<	300	OK
	Lower tensile stress	σ_c:2.04 <	8	OK	σ_c:5.57<	12	OK
		σ_s: 70.67 <	160	OK	σ_s:193.03<	300	OK
	Rebar Content	165.55 >	103.37	OK	165.55 >	150.59	OK
	Shear stress	τ_m: 0.30	0.98	OK	τ_m:0.81<	1.49	OK
P18	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c:3.01<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:188.22<	300	OK
	Lower tensile stress	σ_c:2.00<	8	OK	σ_c:5.56<	12	OK
		σ_s:69.39<	160	OK	σ_s:192.69<	300	OK
	Rebar Content	165.55 >	101.49	OK	165.55 >	150.32	OK
	Shear stress	τ_m:0.29<	0.98	OK	τ_m:0.81<	1.49	OK
P19	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c:2.91<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:181.55<	300	OK
	Lower tensile stress	σ_c:2.04	8	OK	σ_c:5.52<	12	OK
		σ_s:68.02<	160	OK	σ_s:183.90<	300	OK
	Rebar Content	177.88 >	105.83	OK	177.88 >	152.60	OK
	Shear stress	τ_m:0.31<	1.00	OK	τ_m:0.83<	1.53	OK

Note: Unit stress in N/mm², Rebar Content in cm²

Bridge Axis Perpendicular Direction

Pier No.	Item	Ordinary Condition*1 Stress/Rebar Content	Allowable Value	Judgement	Earthquake Condition*2 Stress/Rebar Content	Allowable Value	Judgement
P14	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c: 2.37<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:153.16<	300	OK
	Lower tensile stress	σ_c: 1.77 <	8	OK	σ_c:4.74<	12	OK
		σ_s: 66.50 <	160	OK	σ_s:178.35<	300	OK
	Rebar Content	146.67 >	88.39	OK	146.67 >	126.43	OK
	Shear stress	τ_m: 0.32 <	0.98	OK	τ_m:0.81<	1.49	OK
P15	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c: 1.97<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:127.35<	300	OK
	Lower tensile stress	σ_c: 1.64 <	8	OK	σ_c: 4.19 <	12	OK
		σ_s:61.72 <	160	OK	σ_s:157.74<	300	OK
	Rebar Content	146.67 >	54.36	OK	146.67 >	111.82	OK
	Shear stress	τ_m: 0.30 <	0.98	OK	τ_m: 0.73<	1.49	OK
P16	Upper tensile stress	σ_c:0.00<	8	OK	σ_c:2.01<	12	OK
		σ_s:0.00<	160	OK	σ_s:129.74<	300	OK
	Lower tensile stress	σ_c:1.67<	8	OK	σ_c:4.28<	12	OK
		σ_s:62.83<	160	OK	σ_s:160.91<	300	OK
	Rebar Content	146.67 >	83.51	OK	146.67 >	114.07	OK
	Shear stress	τ_m:0.30<	0.98	OK	τ_m:0.74<	1.49	OK
P17	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c:1.94<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:125.41<	300	OK
	Lower tensile stress	σ_c:1.64<	8	OK	σ_c:4.18<	12	OK
		σ_s:61.54<	160	OK	σ_s:157.33<	300	OK
	Rebar Content	146.67 >	81.80	OK	146.67 >	111.53	OK
	Shear stress	τ_m:0.30<	0.98	OK	τ_m:0.72<	1.49	OK
P18	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c:1.89<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:121.89<	300	OK
	Lower tensile stress	σ_c:1.61<	8	OK	σ_c:4.09<	12	OK
		σ_s:60.42<	160	OK	σ_s:153.99<	300	OK
	Rebar Content	146.67 >	80.13	OK	146.67 >	109.17	OK
	Shear stress	τ_m:0.29<	0.98	OK	τ_m:0.71<	1.49	OK
P19	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c:1.99<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:128.50<	300	OK
	Lower tensile stress	σ_c:1.67<	8	OK	σ_c:4.30<	12	OK
		σ_s:62.99<	160	OK	σ_s:161.65<	300	OK
	Rebar Content	146.67 >	83.73	OK	146.67 >	114.60	OK
	Shear stress	τ_m:0.3<	0.98	OK	τ_m:0.74<	1.49	OK

Note: Unit stress in N/mm^2, Rebar Content in cm^2

**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

2) Connection between SPSP and Footing

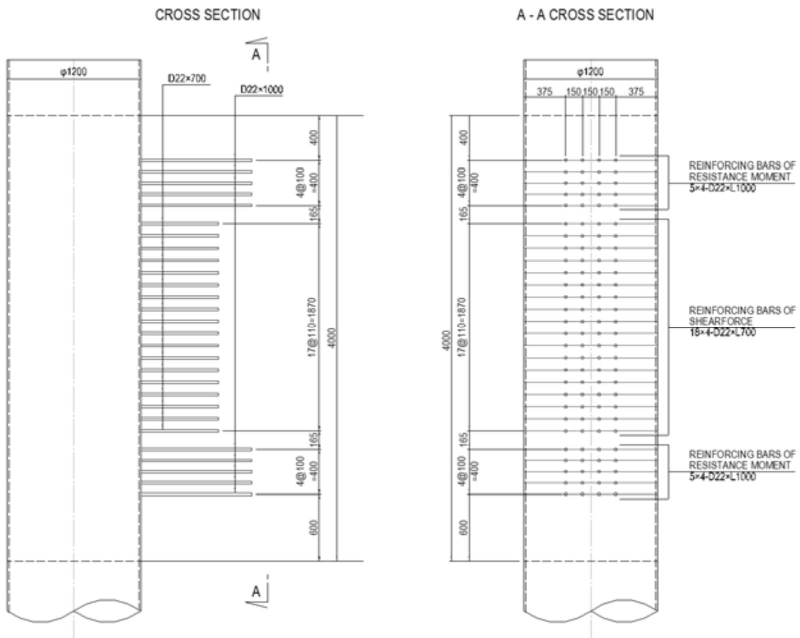
The required number of moment and shear reinforcement for connection between SPSP and footing by Reinforcement Stud Method is calculated as follows:

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 4.3.68 Layout of Reinforcement Stud

Table 4.3.43 Verification of Connection between SPSP and Footing

Bridge Axis Direction

Pier No.	Critical condition	σ_s	σ_{sa}	nb nba	Critical condition	τ_s	τ_{sa}	ns nsa
P14	Wind+ Temperature	174.1<	216.0	20 ≥17	Temperature	103.3<	110.4	76 ≥72
P15	Wind+ Temperature	169.2<	216.0	20 ≥16	Earthquake	151.8<	180.0	72 ≥61
P16	Wind+ Temperature	165.1<	216.0	20 ≥16	Earthquake	153.9<	180.0	72 ≥62
P17	Wind+ Temperature	164.4<	216.0	20 ≥16	Earthquake	153.0<	180.0	72 ≥62
P18	Wind+ Temperature	169.2<	216.0	20 ≥16	Earthquake	152.7<	180.0	72 ≥62
P19	Wind+ Temperature	175.0<	216.0	20 ≥17	Earthquake	154.9<	180.0	72 ≥62

Bridge Axis Perpendicular Direction

Pier No.	Critical condition	σ_s	σ_{sa}	nb nba	Critical condition	τ_s	τ_{sa}	ns nsa
P14	Wind	152.9<	200.0	20 ≥16	Earthquake	147.7<	180.0	76 ≥63
P15	Wind	152.9<	200.0	20 ≥16	Earthquake	139.8<	180.0	72 ≥56
P16	Wind	152.9<	200.0	20 ≥16	Earthquake	142.3<	180.0	72 ≥57
P17	Wind	152.9<	200.0	20 ≥16	Earthquake	137.8<	180.0	72 ≥56
P18	Wind	152.9<	200.0	20 ≥16	Earthquake	134.9<	180.0	72 ≥54
P19	Wind	152.9<	200.0	20 ≥16	Earthquake	141.5<	180.0	72 ≥57

Note:
σs: tensile stress of the moment reinforcing bar caused by moment and horizontal force (N/mm²)
σsa: allowable tensile stress of the reinforcing bar (N/mm²)
nb: number of moment reinforcement nba: required number of moment reinforcement
τs: shear stress of shear reinforcement (N/mm²)
τsa: allowable shear stress (N/mm²)
ns: number of shear reinforcement nsa: required number of shear reinforcement

Source: JICA Study Team

3) Connection between Footing and Pile Head of Bulkhead Piles

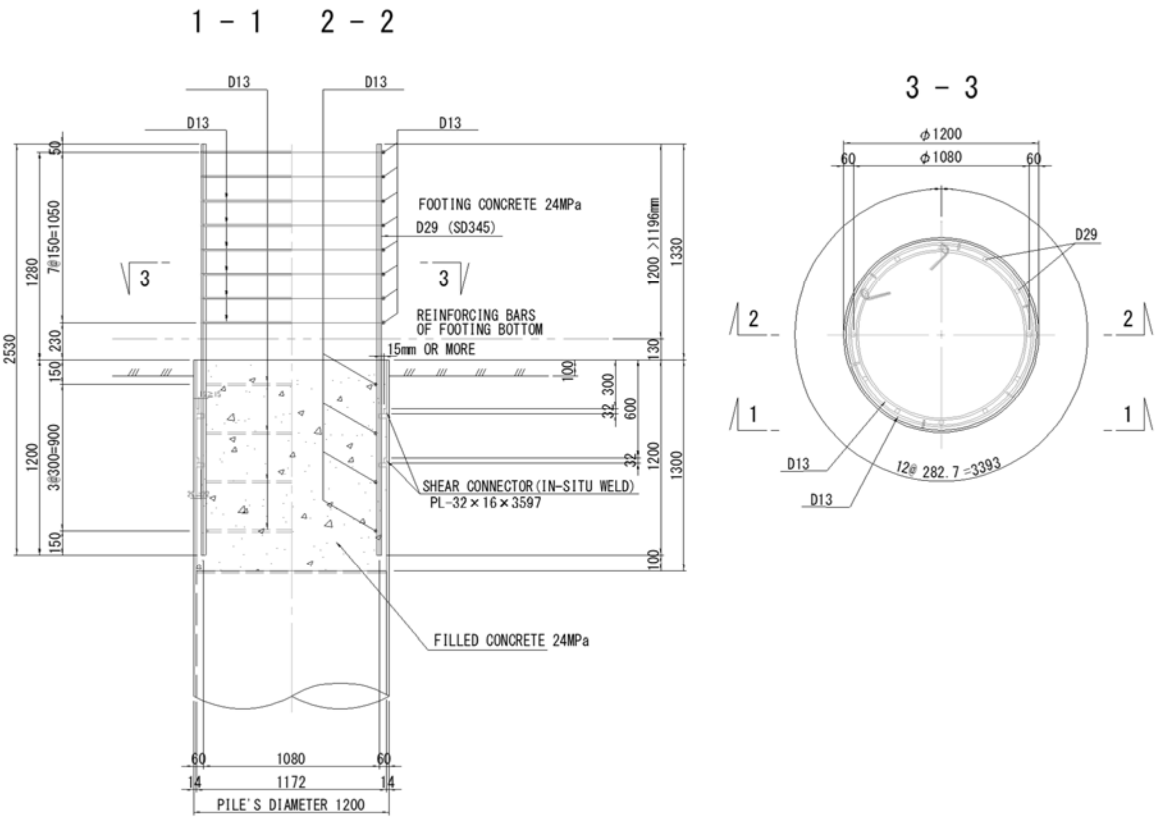
The pile head of the bulkhead part of the SPSP will be inserted and rigidly connected by reinforcing bars with the footing, and it has been verified in terms of stress and content of reinforcement as follows:

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 4.3.69 Detail of Pile Head Connection

c) Verification of Required Content of Reinforcement and Stress

The result of the verification of the connection between footing and bulkhead pile for the case of critical condition is summarized in the table below.

Table 4.3.44 Verification of Connection between Footing and Bulkhead Pile

<u>Bridge Axis Direction</u>

Pier No.	Critical Condition	Load on the Pile Head Moment (kN.m)	Load on the Pile Head Axial Load (kN)	Content of Rebar (cm^2)	Required Content of Rebar (cm^2)	σ_s (N/mm^2)	σ_{sa} (N/mm^2)
P14	Earthquake	348.0	Min-718 Max 3588	77.1 >	56.7	222.8	300.0
P15	Earthquake	335.0	Min-747 Max 3395	77.1 >	57.1	223.7	300.0
P16	Earthquake	327.0	Min-676 Max 3370	77.1 >	53.3	209.5	300.0
P17	Earthquake	320.0	Min-655 Max 3306	77.1 >	51.8	204.0	300.0
P18	Earthquake	327.0	Min-719 Max 3322	77.1 >	55.2	216.8	300.0
P19	Earthquake	327.0	Min-660 Max 3380	77.1 >	52.6	207.0	300.0

Source: JICA Study Team

d) Required Anchorage Length of Reinforcing Bars

Anchorage length of reinforcing bars, L = 1,200 mm, from the main reinforcement of footing must be longer than L0+10 x φ.

$$Lo = \frac{\sigma sa}{4 \cdot \tau oa}\phi = 906 \ (mm)$$

$L \geq Lo + 10 \times \varphi = 1,196$ (mm)

σsa: allowable tensile stress of the reinforcing bar 200.00 (N/mm^2)

τsa: allowable shear stress 1.600 (N/mm^2)

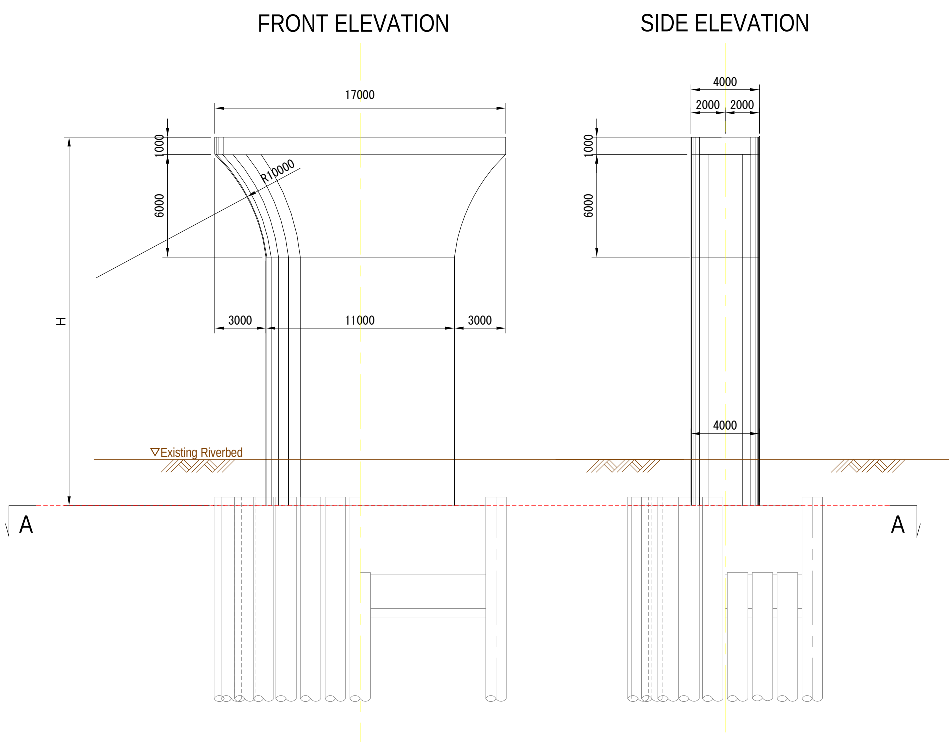
φ: Diameter of Reinforcing bar: φ29 mm

4.3.5.3 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 4.3.70 Design Section of Pier Column

2) Design Condition

- Applied reinforcement bar: SD345 for shear reinforcement, SD390 for main reinforcement (underwater member)
- Concrete design strength: 30 N/mm^2

3) Sectional Forces at the Bottom of the Pier Column

Sectional forces at the bottom of the pier column during earthquake condition as critical load for the design are summarized in the table below. The force due to the hydrodynamic pressure during earthquake is included in the shear force, S, and bending moment, M.

Table 4.3.45 Sectional Force in Earthquake Condition

	Load Direction	V (kN)	S (kN)	M (kN.m)
P14	Bridge axis direction	48,500	15,700	243,600
	Bridge axis perpendicular direction	48,500	14,800	266,700
P15	Bridge axis direction	44,300	15,600	239,300
	Bridge axis perpendicular direction	44,300	13,600	235,300
P16	Bridge axis direction	45,200	15,800	241,500
	Bridge axis perpendicular direction	45,200	13,800	238,300
P17	Bridge axis direction	44,500	15,900	239,900
	Bridge axis perpendicular direction	44,500	13,600	230,900
P18	Bridge axis direction	43,700	16,000	239,700
	Bridge axis perpendicular direction	43,700	13,400	224,400
P19	Bridge axis direction	45,800	16,200	241,500
	Bridge axis perpendicular direction	45,800	14,000	236,300

Source: JICA Study Team

4) 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	-	-

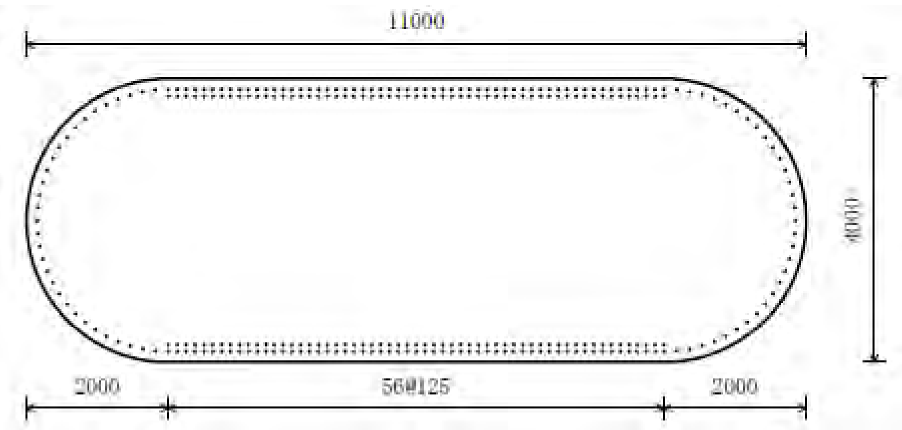


Figure 4.3.71 Rebar Arrangement (Main Reinforcement)

Source: JICA Study Team

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

5) Verification

Pier column structure is verified by compressive stress of concrete, tensile stress of rebar, shear stress and content of shear reinforcement. The result of verification at earthquake condition which is critical condition for pier column design is summarized in the table below. Since average shear stress is over allowable stress that only concrete resists against shear force, shear reinforcement is arranged to meet the requirement.

Table 4.3.46 Verification of Pier Column Stress at Earthquake Condition

Bridge Axis Direction

Pier No.	Compressive Stress (N/mm^2)		Tensile Stress (N/mm^2)		Shear Stress (N/mm^2)		Shear Reinforcement Content (mm^2)		Judgement
	σ_c	σ_{ca}	σ_s	σ_{sa}	τ_m	τ_{a1}, τ_{a2}	Aw	Aw_{Req}	
P14	12.1<	15.0	330.6<	345.0	0.40<	0.28,2.85	3871.0>	747.7	OK
P15	11.9<	15.0	333.5<	345.0	0.40<	0.28,2.85	3871.0>	733.8	OK
P16	12.0<	15.0	335.2<	345.0	0.40<	0.27,2.85	3871.0>	766.5	OK
P17	11.9<	15.0	334.0<	345.0	0.41<	0.28,2.85	3871.0>	779.0	OK
P18	11.9<	15.0	335.8<	345.0	0.41<	0.28,2.85	3871.0>	796.2	OK
P19	12.0<	15.0	333.6<	345.0	0.42<	0.28,2.85	3871.0>	828.8	OK

Bridge Axis Perpendicular Direction

Pier No.	Compressive Stress (N/mm^2)		Tensile Stress (N/mm^2)		Shear Stress (N/mm^2)		Shear Reinforcement Content (mm^2)		Judgement
	σ_c	σ_{ca}	σ_s	σ_{sa}	τ_m	τ_{a1}, τ_{a2}	Aw	Aw_{Req}	
P14	7.1<	15.0	142.8<	345.0	0.37<	0.21,2.85	2322.6>	359.0	OK
P15	6.2<	15.0	121.1<	345.0	0.34<	0.21,2.85	2322.6>	293.5	OK
P16	6.3<	15.0	121.5<	345.0	0.35<	0.21,2.85	2322.6>	305.1	OK
P17	6.1<	15.0	115.5<	345.0	0.34<	0.21,2.85	2322.6>	293.0	OK
P18	5.9<	15.0	110.8<	345.0	0.34<	0.21,2.85	2322.6>	282.6	OK
P19	6.3<	15.0	117.4<	345.0	0.35<	0.21,2.85	2322.6>	316.2	OK

σc: Compressive Stress of Concrete *σca: Allowable Compressive Stress of Concrete*

σs: Tensile Stress of Rebar *σsa: Allowable Tensile Stress of Rebar*

τm: Average Shear Stress *τa1: Allowable Shear Stress if only concrete resists against shear force*

τa2: Allowable Shear Stress if both concrete and shear reinforcement resist against shear force

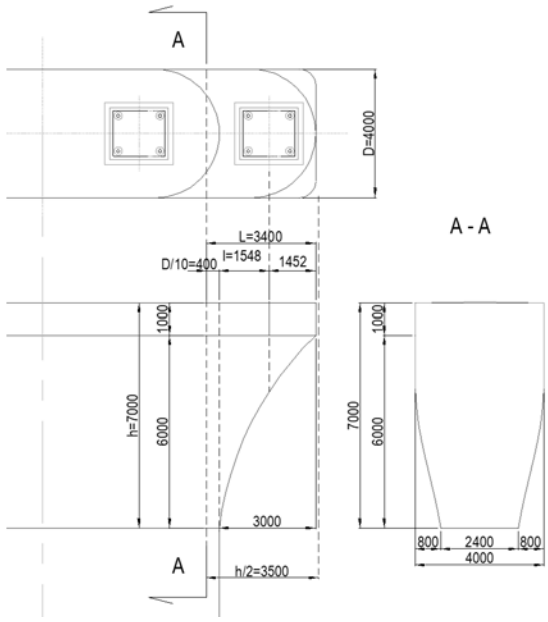
Aw: Shear reinforcement content *Awreq: Required shear reinforcement content in τa1 <τm*

Source: JICA Study Team

(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 4.3.72 Design Section of Pier Head Beam

2) Design Loads

Design loads for verification of the beam structure are summarized in the table below, and the largest values among piers for each condition are used for the verification.

Table 4.3.47 Design Loads for Beam Design

Condition	Load Component	P14	P15	P16	P17	P18	P19
Vertical direction							
Ordinary Condition (Dead + Live Loads)	Dead Load at G1 girder	6,100	5,100	5,500	5,400	5,200	5,800
	Live Load with Impact at G1 girder	2,100	2,000	2,100	2,100	2,000	2,000
	Weight of Beam	1,578	1,578	1,578	1,578	1,578	1,578
	Total	9,778	8,678	9,178	9,078	8,778	9,378
Earthquake Condition (Dead Load + Effect of earthquake)	Dead Load at G1 girder	6,100	5,100	5,500	5,400	5,200	5,800
	Weight of Beam	1,578	1,578	1,578	1,578	1,578	1,578
	Vertical reaction force due to earthquake from Superstructure*1	1,400	1,200	1,200	1,200	1,200	1,300
	Total	9,078	7,878	8,278	8,178	7,978	8,678
Bridge axis perpendicular direction							
Earthquake Condition	Inertia force of superstructure	2,400	2,100	2,200	2,100	2,100	2,300
Bridge axis direction							
Effect of temperature change	Horizontal force due to temperature change	1,400	500	300	300	800	1,500
Earthquake Condition	Inertia force on the beam	473	473	473	473	473	473
	Inertia force on superstructure	1,800	1,800	1,900	1,900	2,000	2,000
	Total	2,273	2,273	2,373	2,373	2,473	2,473

*Note: *1: It is calculated in accordance with Chapter 15.4 of Specifications for Highway Bridges Part-V 2012.*

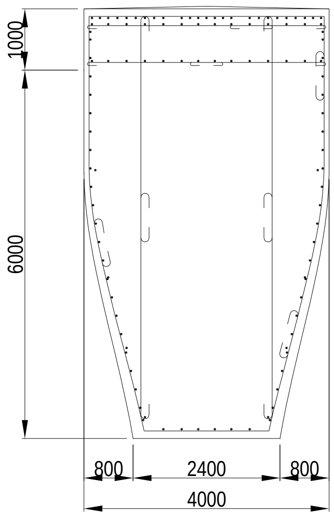
Source: JICA Study Team

3) Design condition

- Applied reinforcement bar: SD345 for main reinforcement, stirrup
- Concrete design strength: 30 N/mm^2

4) 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 4.3.73 Rebar Arrangement (Main Reinforcement)

5) 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.

Table 4.3.48 Verification of Reinforcement Content at Vertical Direction of Beam

Item	Unit	Ordinary Condition (Dead Load)	Ordinary Condition (Dead and Live Loads)	Earthquake Condition
Design Tensile Force (T)	kN	2,454.8	3,161.0	5,325.6
Allowable Tension Stress (σ_{sa})	N/mm^2	100.00	180.00	300.00
Tensile Reinforcement Arranged Content (Asu) Required Content (AsuReq^{*1})	mm^2	Asu ≥ AsuReq OK 30,973.80 24,547.28	Asu ≥ AsuReq OK 30,973.80 17,561.19	Asu ≥ AsuReq OK 30,973.80 17,751.95
Reinforcement at Sides Arranged Content (Ass) Required Content (AssReq^{*2})	mm^2	Ass ≥ AssReq OK 17,806.60 12,389.52	Ass ≥ AssReq OK 17,806.60 12,389.52	Ass ≥ AssReq OK 17,806.60 12,389.52

*Note: *1: AsuReq = 1000 ・T / σsa*

**2: AssReq = 0.4 ・Asu*

Source: JICA Study Team

6) Verification for Bending Moment (Horizontal Bridge Axis Direction)

As for the section against bending moment at horizontal bridge axis direction, it is verified by the compressive and tensile stress occurring at section A-A by the allowable stress method, and the result is summarized in the table below.

Table 4.3.49 Verification of Bending Moment Stress at Horizontal Direction of Beam

Item	Unit	Effect of Temperature Change	Earthquake Condition
Bending Moment (M)	kN.m	2,922 (= 1,500 kN x 1.948 m)	4,597 (= 473 kN x 1.48 m + 2000 kN x 1.948 m)
Distance between edge of compressive side and neutral axis (x)	Mm	538	538
Compressive Stress σ_c Tensile Stress σ_s	N/mm^2 N/mm^2	0.86 $<\sigma_{ca}$ 80.88 $<\sigma_{sa}$	1.36 $<\sigma_{ca}$ 127.23 $<\sigma_{sa}$
Coefficient Increase of Allowable Stress α Allowable Compressive Stress σ_{ca} Allowable Tensile Stress σ_{sa}	- N/mm^2 N/mm^2	1.15 11.50 207.00	1.50 15.00 300.00

Source: JICA Study Team

7) Verification of Shear Force (Horizontal Bridge Axis Direction)

As for the section against shear force at horizontal bridge axis direction, it is verified by the shear stress occurring at section A-A by the allowable stress method, and the result is summarized in the table below. Content of stirrup is 774.2 mm^2, which is equivalent to approximately 0.2% at the minimum, although arrangement is not required.

Table 4.3.50 Verification of Shear Stress at Horizontal Direction of Beam

Item	Unit	Effect of Temperature Change	Earthquake Condition
Sectional Force Shear Force (S) Bending Moment (M)	 kN kN.m	 1,500 2,922	 2,474 4,597
Effective Height (d)	mm	3,645	3,645
Shear Force considering Effective Height (Sh)*1	kN	1,500	2,474
Coefficient Increase of Allowable Stress (α) Ratio of rebar (pt)*2 Coefficient of allowable shear stress related to "d" (ce) Coefficient of allowable shear stress related to "pt"(cpt)	- % - -	1.15 0.050 0.668 0.601	1.50 0.050 0.668 0.601
τ_m τ_{a1} τ_{a2}	N/mm^2 N/mm^2 N/mm^2	0.067 < 0.115 2.185	0.111 < 0.148 2.850
Shear Reinforcement Content Aw AwReq	 mm^2 mm^2	 774.2 0.0	 774.2 0.0

Note: *1: Sh=M-M/d x (tanβ+tanγ), β and γ = zero in this case *2: pt=As/(b x d)

Source: JICA Study Team

8) Verification of Bridge Seat

a) Seating Length at P13 and P20

The seating length, which is defined as the distance between the edge of the girder and the edge of the top of substructure in longitudinal direction, should be long enough to prevent departure and unseating of the superstructure from the top of the substructure. The required value (S_{EM}) can be calculated at the equation below as specified in the JSBH.

S_{EM} = 0.7 + 0.005 x span length (m)

Table 4.3.51 Verification of Seating Length

Pier No.	P13	P20
Span Length	110.8m	103.1m
Required Seating Length (S_{EM})	1.254m	1.216m
Seating Length	3.550m	2.150m
Judge	OK	OK

Source: JICA Study Team

b) Bearing Edge Distance (S)

The bearing edge distance (S), which is defined as the distance between the center of anchor bolt of bearing and the edge of the top of the substructure, shall be equal to or larger than the following value:

S ≥ 0.2+0.005 x span length (m)

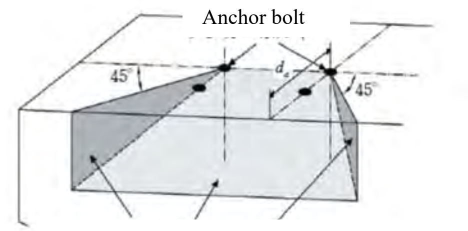
Table 4.3.52 Verification of Bearing Edge Distance

Pier No.	P13	P14	P15	P16	P17	P18	P19	P20
Span Length	110.8m	112m	112m	112m	112m	112m	112m	103.1m
Minimum Edge Distance	0.754m	0.76m	0.76m	0.76m	0.76m	0.76m	0.76m	0.72m
Edge Distance (S)	1.157m	0.802m	0.802m	0.802m	0.802m	0.802m	0.802m	0.757m
Judge	OK	OK	OK	OK	OK	OK	OK	OK

Source: JICA Study Team

c) Bridge Seat Strength

Bridge seats is designed with sufficient strength to withstand the vertical and horizontal forces through bearings. Horizontal force (design horizontal seismic force) transmitted from the bearings is resisted by concrete and reinforcement. The resisting area of concrete is the summation of three planes in directions of sideward and downward with edge angles of 45 degrees as shown in figure below.



Source: JSBH

Figure 4.3.74 Image of Resisting area of Concrete

The bridge seats against design horizontal seismic force is verified as follows:

Evaluation of strength

P_{bs} = Pc + Ps (Pc ≧ Ps), P_{bs} ≧ Design horizontal seismic force (P_h (N))

Where,

P_{bs} : Strength of bridge seat (N)
Note that the strength is determined under the condition that the strength borne by reinforcements does not exceed that borne by concrete.

P_c : Strength borne by concrete (N)
$Pc = (\alpha \cdot 0.32 \cdot \sqrt{\sigma_{ck}} \cdot Ac) / 1000.0$

P_s : Strength borne by reinforcement (N)
$Ps = \Sigma\{\beta \cdot (1 - h_i / da) \cdot \sigma_{sy} \cdot A_{si}\} / 1000.0$

α : Coefficient for determining the strength borne by concrete
σ_n : Bearing stress at bottom of bearing support against vertical force (N/mm^2)
σ_{ck} : Design strength of concrete (N/mm^2)
A_c : Resistance area of concrete (mm^2)
β : Correction factor associated with the strength borne by reinforcement
h_i : Distance from bridge seat surface of ith reinforcement (m)
d_a : Distance from center of anchor bolt in the rear side of bearing support to bridge seat edge
σ_{sy} : Yield point of reinforcement (N/mm^2)
A_{si} : Cross sectional area of ith reinforcement (mm^2)

The calculation results is summarized in table below.

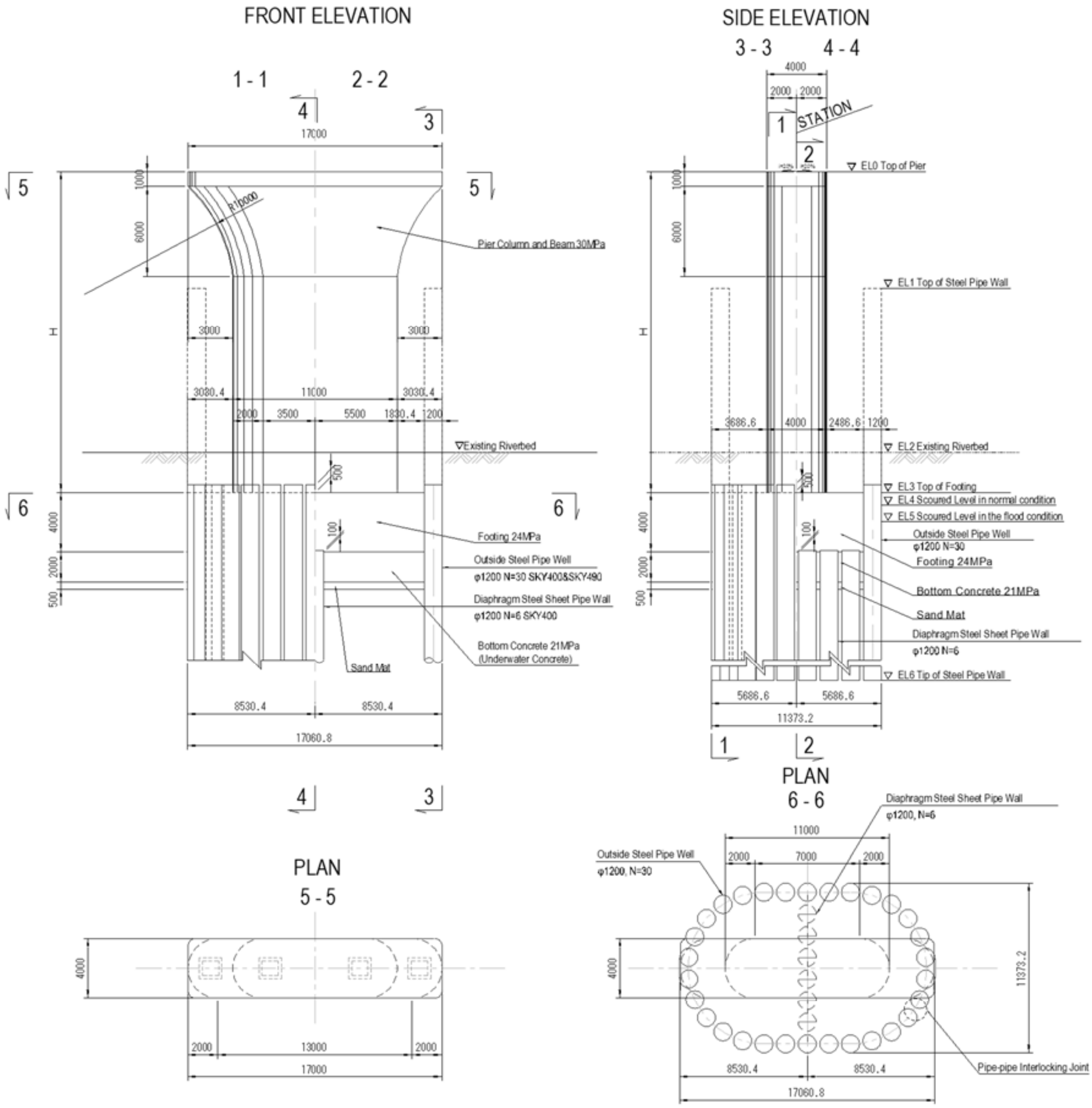
Table 4.3.53 Verification Result of Bridge Seat Strength

Pier No.	P19	
Girder	G1&G4	G2&G3
Design horizontal seismic force Ph (kN) *per bearing	2,000	2,000
Resistance area of concrete Ac (mm^2)	5,901,000	14,771,000
Bearing stress at bottom of bearing support against vertical force σ n (N/mm^2)	2.50	2.53
Coefficient for determining the strength borne by concrete	0.30	0.30
Strength borne by concrete Pc (kN)	3,050	7,700
Strength borne by reinforcement Ps (kN)	2,090	3,140
Strength of bridge seat Pbs (kN)	5,140	10,840
Judge (Pbs≧Ph)	OK	OK

Source: JICA Study Team

4.3.5.4 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 4.3.75 Structure Drawing of Substructure of P14-P19

4.3.5.5 Quantities of Major Items of Substructure

Quantity of the major items of the substructure for the steel box girder bridge is calculated for cost estimation and it is summarized in the following table.

Table 4.3.54 Quantity of the Major Items of the Substructure for the Steel Box Girder Bridge

Item	P14	P15	P16	P17	P18	P19	Total
SPSP (ton)							
Permanent part	629.6 t	673.8 t	644.4 t	644.4 t	644.4 t	622.3 t	3,858.8 t
Temporary part	193.3 t	193.3 t	193.3 t	193.3 t	193.3 t	193.3 t	1,159.8 t
Concrete (m^3)							
24 MPa	493 m^3	493 m^3	493 m^3	493 m^3	493 m^3	493 m^3	2,958 m^3
30 MPa	971 m^3	958 m^3	946 m^3	930 m^3	917 m^3	905 m^3	5,627 m^3
Re-bar (ton)							
D13-D32	95.0 t	95.0 t	95.0 t	95.0 t	95.0 t	95.0 t	570.0 t
D38-D51	99.6 t	97.5 t	96.7 t	95.7 t	94.9 t	95.4 t	579.7 t

Source: JICA Study Team

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

4.3.6.1 Design Conditions

(1) Standard and Design Criteria

The same standard and design criteria as 7-Span Bridge will be applied.

(2) Analytical Software for Design

The same analytical software as 7-Span Bridge will be used.

(3) Materials to be Used

- Concrete : σ_{ck} = 30 N/mm^2 [for pier column and beam]
  : σ_{ck} = 24 N/mm^2 [for footing (top slab concrete)]
  : σ_{ck} = 21 N/mm^2 [for bottom slab concrete, concrete filling to steel pipe]
- Reinforcing bars: SD345 [for all members]
- Reinforcing stud bars: SD345 [for connection between footing and steel sheet pile]
- Steel sheet pile : SKY400, SKY490
  Estimated corrosion thickness: 2.0 mm/100 years
- Pipe-pipe interlocking joint: STK400 φ165.2 x t11

(4) Design Soil Condition

At the section of the 3-span of steel box bridge, two boreholes drilling were performed in D/D. Boreholes for P6 is far from the center of the foundation, which are around 26 m away.

Table 4.3.55 Borings at Steel Box Bridge Section

Pier No.	Boring No.	Boring Location
P6	No.BD-18 (D/D)	26 m from the center of the foundation
P7	No.BD-13 (D/D)	center of the foundation

Source: JICA Study Team

Since there is a saturated soil layer having ground water level higher than 10 m below the ground surface and located at a depth less than 20 m below the ground surface, liquefaction potential is evaluated and deduction factor due to liquefaction at the time of earthquake is considered in the foundation design.

(5) Loads and Load Combinations

1) Loads

The same loads for substructure design as 7-span bridge will be considered.

2) Load Combination

The design cases and corresponding allowable stress for members are same as 7-span bridge except vessel collision case, which is shown in the following table.

Table 4.3.56 Load Combination and Allowable Stress in Pier Column and Foundation Design

<table>
<tr><th colspan="2">Load Combination</th><th colspan="2">Design Water Level (MSL+m)</th><th>Water Velocity (m/s) for Flowing Water Pressure</th><th>Local Scouring</th><th>Increase of Allowable Stress</th></tr>
<tr><td rowspan="2">D.</td><td rowspan="2">Vessel Collision for P6,P7</td><td rowspan="2">High tide in spring tide</td><td rowspan="2">3.18 m</td><td rowspan="2">No consideration</td><td>No consideration</td><td rowspan="2">1.5</td></tr>
<tr><td>1/2 of maximum</td></tr>
</table>

Source: JICA Study Team

3) Loads from Superstructure

Dead load and live load with/without impact from superstructure for the substructure design is summarized in the table below.

Table 4.3.57 Loads from Superstructure for the Substructure Design

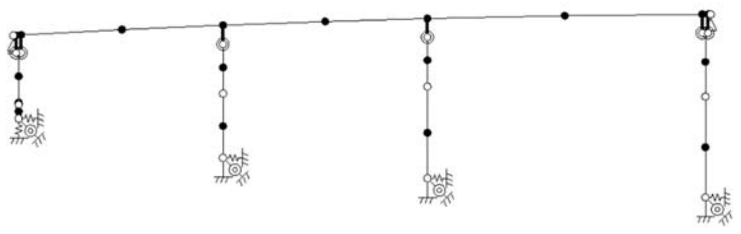
Unit: kN

Loads	P5	P6	P7	P10
Dead Load	7,200	16,700	21,400	9,000
Live Load with Impact	2,700	4,700	5,400	2,900
ΣD+L+I	**9,900**	**21,400**	**26,800**	**11,900**
Dead Load	7,200	16,700	21,400	9,000
Live Load w/o Impact	2,500	4,500	5,200	2,700
ΣD+L	**9,700**	**21,200**	**26,600**	**11,700**
Horizontal Force due to temperature change ±15°	400	200	200	500

Values of P5 and P10 are loads only from 3-span bridge.

Source: JICA Study Team

Since the structure which can distribute inertial force of earthquake into several substructures is applied for the 3-continuous spans, a shared weight on each substructure is calculated by Eigenvalue analysis using framed structure model as shown in the figure below.



Source: JICA Study Team

Figure 4.3.76 Framed Structure Model

The result is summarized in the table below.

Table 4.3.58 Shared Weight of Superstructure on Substructures

Item		P5	P6	P7	P10
Bridge Axis Direction					
-	Shared Weight (kN)	12,800	15,300	14,500	11,800
-	Natural period of oscillation of the bridge (second)	1.60(s)			
Bridge Axis Perpendicular Direction					
-	Shared Weight (kN)	6,700	16,300	23,100	8,400
-	Natural period of oscillation of the bridge (second)	0.28(s)	0.38(s)	0.45(s)	0.51(s)
		0.45(s) in oscillation unit consisting of all substructures			

Values of P5 and P10 are loads only from 3-span bridge.

Source: JICA Study Team

4.3.6.2 SPSP Foundation Design

(1) Design Flow

Design flow of the SPSP foundation is same as that of 7-span bridge.

(2) 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.

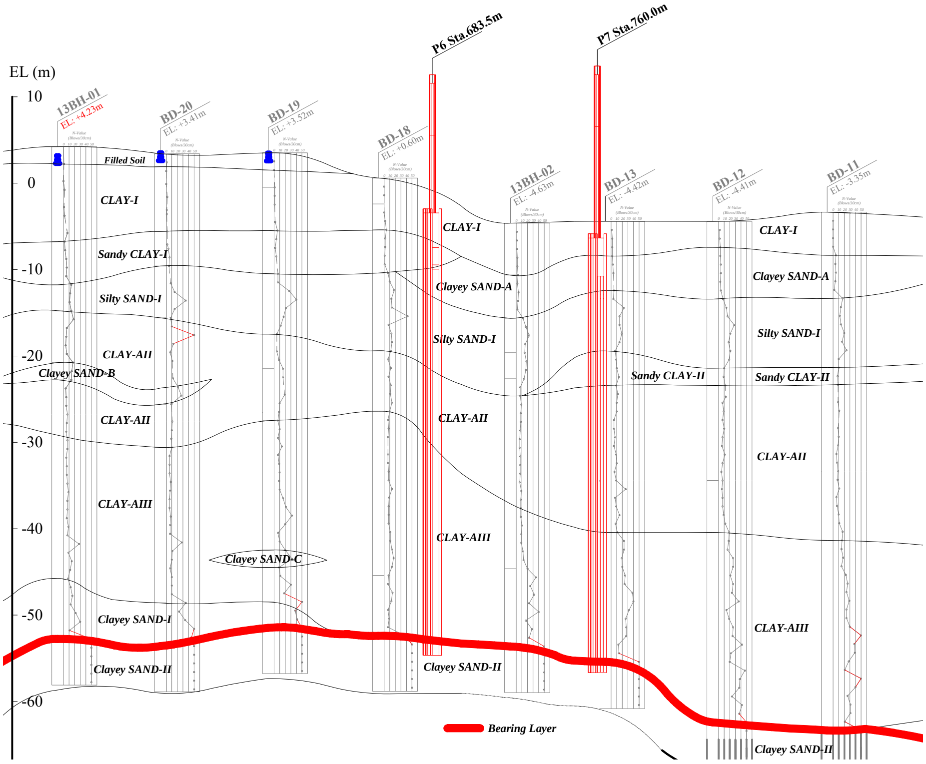
Table 4.3.59 Setting of Footing Top Elevation

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

(3) 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 4.3.77 Soil Profile and Pile Tip Position

(4) Design Model of SPSP Structure

Finite-length beam on an elastic ground is applied for the design model of the SPSP structure for all foundations of P6 and P7 as same as the cases of 7-span bridge.

Table 4.3.60 Selection of the Design Model of the SPSP Structure

Pier No.	P6	P7
D (m)	21.7	20.1
L/D	2.37	2.50
βL_e	1.46	1.45

D (m): width of foundation: larger value among D (m) or B (m)

L (m): length of steel pipe pile

L_e (m): embedded length of foundation underground

Β (1/m): characteristic factor of foundation

Source: JICA Study Team

(5) Design External Force

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

Table 4.3.61 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

(6) 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 4.3.62 Verification of Bearing Capacity

Bride Axis Direction

Unit: kN

Pier No.	Item	Ordinary Condition*1 Vertical Reaction	Ordinary Condition*1 Allowable Value	Ordinary Condition*1 Judgement	Earthquake Condition*2 Vertical Reaction	Earthquake Condition*2 Allowable Value	Earthquake Condition*2 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 Vertical Reaction	Ordinary Condition*1 Allowable Value	Ordinary Condition*1 Judgement	Earthquake Condition*2 Vertical Reaction	Earthquake Condition*2 Allowable Value	Earthquake Condition*2 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 4.3.63 Verification of Displacement

Pier No.	Item	Earthquake Condition*1 Displacement*2	Earthquake Condition*1 Allowable Value	Earthquake Condition*1 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

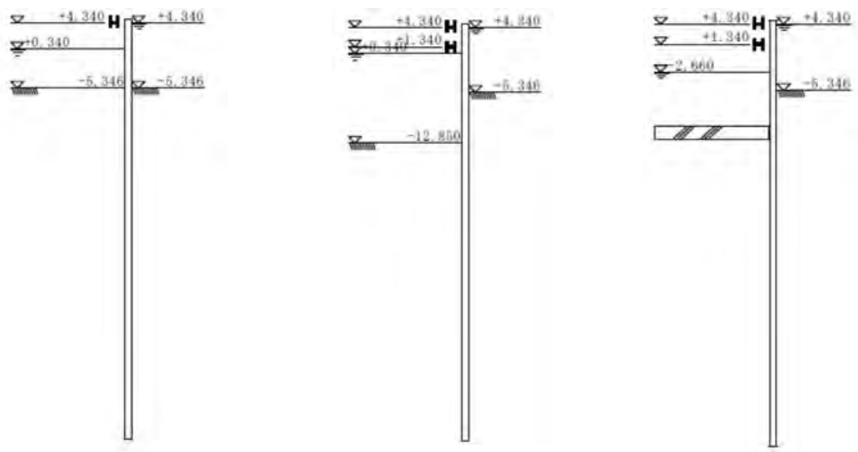
2) Stress of Outer Steel Pipe Sheet Piles

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

a) Construction Step of Temporary Cofferdam

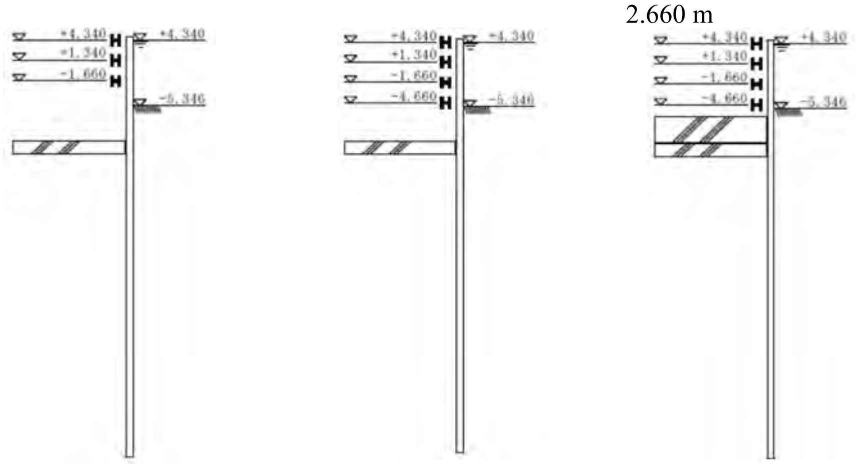
The underwater/atmospheric excavation method is applied because the stress during drainage and residual stress can be smaller. The construction step of temporary cofferdam for the case of P7 is shown in the figure below, and P6 cases have similar steps as that of P7.

Schematic



Work activities	Step-1: Install 1st support and drainage up to EL+0.34 m	Step-2: Install 2nd support and excavate up to EL-12.850 m	Step-3: Placement of bottom slab concrete and drainage up to EL-2.660 m

Schematic

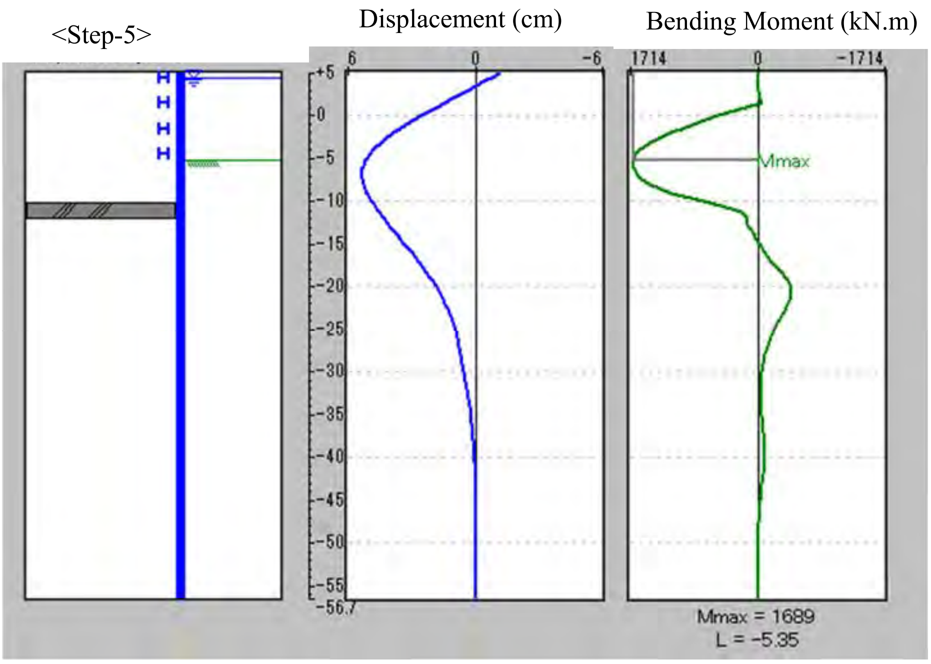


Work activities	Step-4: Install 3rd support and dry up inside well	Step-5: Install 4th support	Step-6: Construction of footing

Source: JICA Study Team

Figure 4.3.78 Construction Step of Temporary Cofferdam by Combined Underwater and Atmospheric Excavation (P7 Case)

At the construction step just before construction of footing concrete, namely Step-5, residual stress of the pile will be considered. Diagram of the displacement and bending moment for the case of P7 is shown in the figure below, and the maximum displacement due to moment occurs between the lowest support and bottom slab concrete.

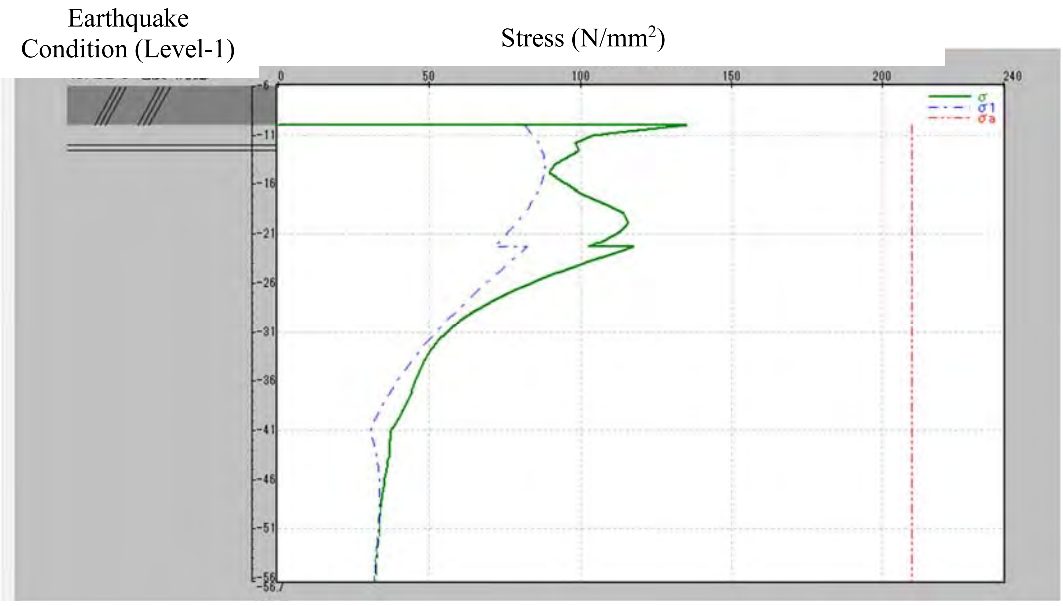


Source: JICA Study Team

Figure 4.3.79 Diagram of Displacement and Bending Moment at the Construction Step when Residual Stress of the Pile is Considered (P7 Case)

b) Combined stress of the pile during construction and due to the design external forces after completion

The following figure shows that combined stress is within the allowable stress under earthquake condition.



where

σ : combined stress (= σ1+σ2), σ1 : stress after completion loads.

σ2 : residual stress during construction, σa : allowable stress in steel pipe sheet pile

Source: JICA Study Team

Figure 4.3.80 Combined Stress for the SPSP of P7 at Earthquake Condition

Table 4.3.64 Verification of SPSP (SKY400 part) Combined Stress at Ordinary Condition

Bridge Axis Direction

Pier	Elevation (m)	$\sigma 1^{*1}$ (N/mm^2)	$\sigma 2$ (N/mm^2)	σ_{max} (N/mm^2)	σa (N/mm^2)	Judgement
P6	-11.16	30.14	66.97	97.12<	140	OK
P7	-10.35	30.37	53.58	83.94<	140	OK

Bridge Axis Perpendicular Direction

Pier	Elevation (m)	$\sigma 1^{*1}$ (N/mm^2)	$\sigma 2$ (N/mm^2)	σ_{max} (N/mm^2)	σa (N/mm^2)	Judgement
P6	-11.16	31.46	69.36	100.82<	140	OK
P7	-10.35	31.02	63.08	94.11<	140	OK

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

Source: JICA Study Team

Table 4.3.65 Verification of SPSP (SKY400 part) Combined Stress at Earthquake Condition

Bridge Axis Direction

Pier	Elevation (m)	$\sigma 1^{*1}$ (N/mm^2)	$\sigma 2$ (N/mm^2)	σ_{max} (N/mm^2)	σa (N/mm^2)	Judgement
P6	-11.45	75.16	66.85	142.00<	210	OK
P7	-10.35	81.65	53.58	135.23<	210	OK

Bridge Axis Perpendicular Direction

Pier	Elevation (m)	$\sigma 1^{*1}$ (N/mm^2)	$\sigma 2$ (N/mm^2)	σ_{max} (N/mm^2)	σa (N/mm^2)	Judgement
P6	-11.16	69.57	69.36	138.93 <	210	OK
P7	-10.35	84.46	63.08	147.54<	210	OK

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

Source: JICA Study Team

(7) 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		P7	
Bridge Axis Direction		Bridge Axis Direction	
Upper tension:	cover 150 mm D32@288	Upper tension:	cover 150 mm D29@278
Lower tension:	cover 150 mm D32@203		cover 300 mm D29@286
	cover 300 mm D32@208	Lower tension:	cover 290 mm D38@228
			cover 440 mm D38@234
Bridge Axis Perpendicular Direction		Bridge Axis Perpendicular Direction	
Upper tension:	cover 120 mm D29@189	Upper tension:	cover 121 mm D29@198
Lower tension:	cover 118 mm D32@189		cover 271 mm D29@410
	cover 268 mm D32@201	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.

Verification of the footing structure is summarized in the table below.

Table 4.3.66 Verification of Footing Structure

Bridge Axis Direction

Pier No.	Item	Ordinary Condition*1 Stress/Rebar Content	Allowable Value	Judgement	Earthquake Condition*2 Stress/Rebar Content	Allowable Value	Judgement
P6	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c: 0.95 <	12	OK
		σ_s: 0.00 <	160	OK	σ_s:90.63<	300	OK
	Lower tensile stress	σ_c: 1.41 <	8	OK	σ_c: 3.06 <	12	OK
		σ_s: 77.49 <	160	OK	σ_s:167.85<	300	OK
	Rebar Content	77.31 >	56.84	OK	77.31 >	65.67	OK
	Shear stress	τ_m: 0.26 <	0.88	OK	τ_m: 0.54<	1.34	OK
P7	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c: 1.31<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:95.73<	300	OK
	Lower tensile stress	σ_c: 1.51 <	8	OK	σ_c: 3.64<	12	OK
		σ_s:70.73 <	160	OK	σ_s:170.30<	300	OK
	Rebar Content	98.72 >	63.03	OK	98.72 >	80.94	OK
	Shear stress	τ_m: 0.28 <	0.96	OK	τ_m: 0.66 <	1.45	OK

Note: Unit stress in N/mm², Rebar Content in cm²

Bridge Axis Perpendicular Direction

Pier No.	Item	Ordinary Condition*1 Stress/Rebar Content	Allowable Value	Judgement	Earthquake Condition*2 Stress/Rebar Content	Allowable Value	Judgement
P6	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c: 0.25<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:21.51<	300	OK
	Lower tensile stress	σ_c: 1.73 <	8	OK	σ_c:3.06<	12	OK
		σ_s: 92.56 <	160	OK	σ_s:163.70	300	OK
	Rebar Content	81.53 >	72.19	OK	81.53 >	68.09	OK
	Shear stress	τ_m: 0.26 <	0.84	OK	τ_m:0.45<	1.28	OK
P7	Upper tensile stress	σ_c: 0.00 <	8	OK	σ_c: 1.66<	12	OK
		σ_s: 0.00 <	160	OK	σ_s:117.85<	300	OK
	Lower tensile stress	σ_c: 1.93 <	8	OK	σ_c: 4.55 <	12	OK
		σ_s:85.27 <	160	OK	σ_s:200.69<	300	OK
	Rebar Content	111.35 >	86.71	OK	111.35 >	108.83	OK
	Shear stress	τ_m: 0.28 <	0.82	OK	τ_m: 0.63<	1.25	OK

Note: Unit stress in N/mm², Rebar Content in cm²

**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

2) Connection between SPSP and Footing

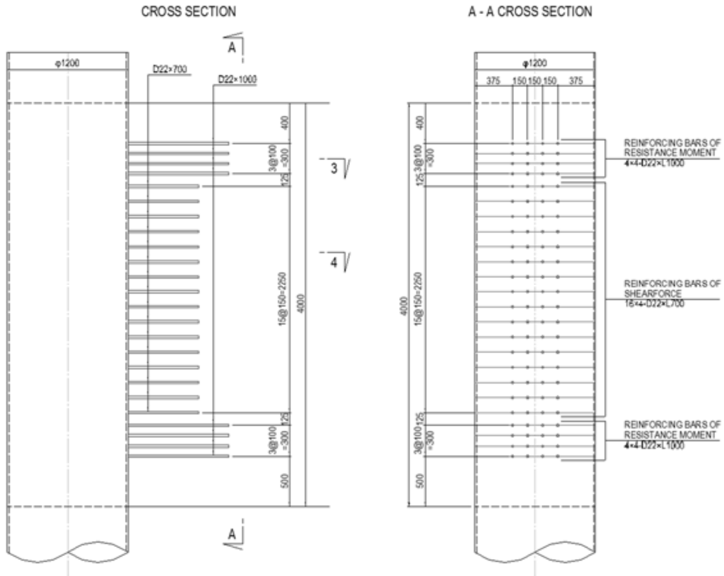
The required number of moment and shear reinforcement for connection between SPSP and footing by Reinforcement Stud Method is calculated as follows:

c) Design Condition

- Applied reinforcement bar: SD345 (underwater member), Diameter 22 mm
- Concrete design strength: 24 N/mm²
- Material of SPSP: SKY400
- Joint method: Reinforcement Stud Method

d) 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 4.3.81 Layout of Reinforcement Stud (P7)

Table 4.3.67 Verification of Connection between SPSP and Footing

Bridge Axis Direction

Pier No.	Critical condition	σ_s	σ_{sa}	nb	nba	Critical condition	τ_s	τ_{sa}	ns	nsa
P6	Wind+ Temperature	158.0<	216.0	16	≥12	Earthquake	136.2<	180.0	56	≥43
P7	Wind+ Temperature	166.2<	216.0	16	≥13	Earthquake	140.5<	180.0	64	≥50

Bridge Axis Perpendicular Direction

Pier No.	Critical condition	σ_s	σ_{sa}	nb	nba	Critical condition	τ_s	τ_{sa}	ns	nsa
P6	Wind	147.4<	200.0	16	≥12	Earthquake	113.9<	180.0	56	≥36
P7	Wind	155.8<	200.0	16	≥13	Earthquake	133.9<	180.0	64	≥48

Note: *σs: tensile stress of the moment reinforcing bar caused by moment and horizontal force (N/mm^2)*

σsa: allowable tensile stress of the reinforcing bar (N/mm^2)

nb: number of moment reinforcement *nba: required number of moment reinforcement*

τs: shear stress of shear reinforcement (N/mm^2)

τsa: allowable shear stress (N/mm^2)

ns: number of shear reinforcement *nsa: required number of shear reinforcement*

Source: JICA Study Team

3) Connection between Footing and Pile Head of Bulkhead Piles

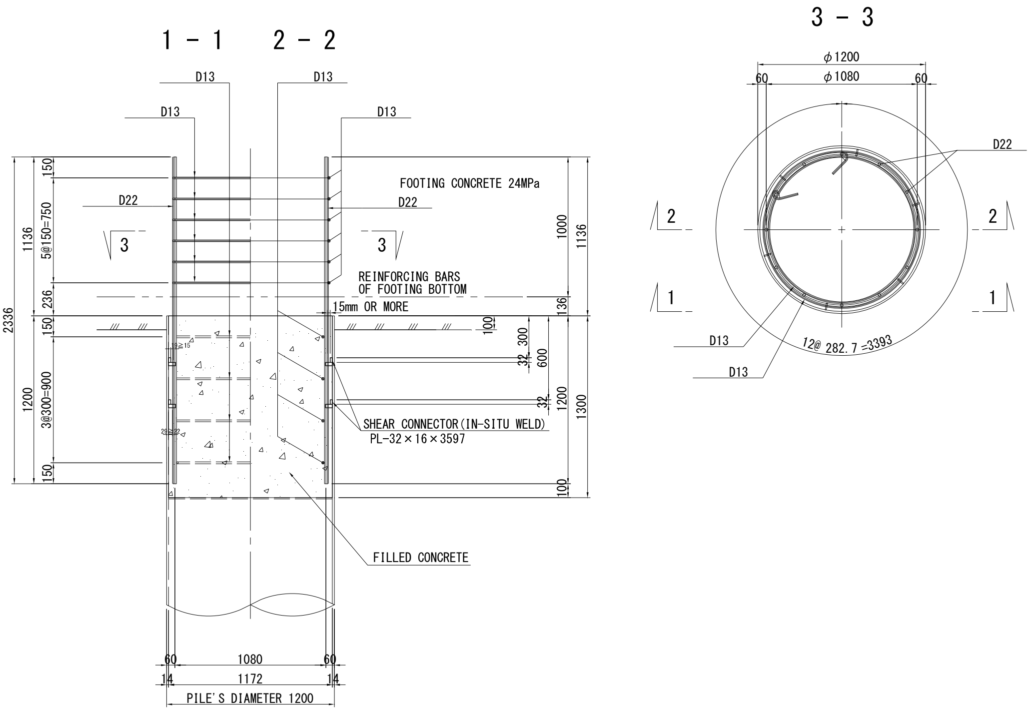
Since P7 has the bulkhead steel sheet pile, the connection of the pile head is verified in terms of stress and content of reinforcement as follows:

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 4.3.82 Detail of Pile Head Connection (P7)

c) Verification of Required Content of Reinforcement and Stress

The result of the verification of the connection between footing and bulkhead pile for the case of critical condition is summarized in the table below.

Table 4.3.68 Verification of Connection between Footing and Bulkhead Pile

Bridge Axis Direction

Pier No.	Critical Condition	Load on the Pile Head		Content of Rebar (cm^2)	Required Content of Rebar(cm^2)	σ_s (N/mm^2)	σ_{sa} (N/mm^2)
		Moment (kN.m)	Axial Load(kN)				
P7	Earthquake	188.0	Min -18.0 Max 2178	46.5 >	13.3	92.95	300.0

Source: JICA Study Team

d) Required Anchorage Length of Reinforcing Bars

Anchorage length of reinforcing bars, L = 1,000 mm, from the main reinforcement of footing must be longer than L0+10 x φ.

$$L_o = \frac{\sigma sa}{4 \cdot \tau oa} \phi = 688\text{mm}$$

L ≥ Lo + 10 x φ = 908 (mm)

σsa: allowable tensile stress of the reinforcing bar 200.00 (N/mm^2)

τsa: allowable shear stress 1.600 (N/mm^2)

φ: Diameter of Reinforcing bar: φ22mm

4.3.6.3 RC Pier

(1) Verification of RC Pier Column

1) Design Section

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

2) Design Condition

- Applied reinforcement bar:

  SD345 for shear reinforcement and main reinforcement (underwater member)

- Concrete design strength: 30 N/mm^2

3) Sectional Forces at the Bottom of the Pier Column

Sectional forces at the bottom of the pier column during earthquake condition as critical load for the design are summarized in the table below. The force due to the hydrodynamic pressure during earthquake is included in the shear force, S, and bending moment, M.

Table 4.3.69 Section Force in Earthquake Condition

	Load Direction	V (kN)	S (kN)	M (kN.m)
P6	Bridge axis direction	36,000	11,000	123,700
	Bridge axis perpendicular direction	36,000	10,800	146,600
P7	Bridge axis direction	41,300	11,600	154,200
	Bridge axis perpendicular direction	41,300	13,100	223,400

Source: JICA Study Team

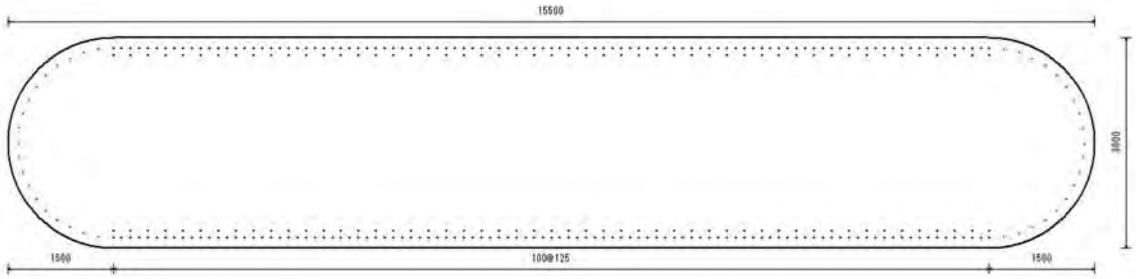
4) Rebar Arrangement

c) Main Reinforcement

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

P6

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



P7

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

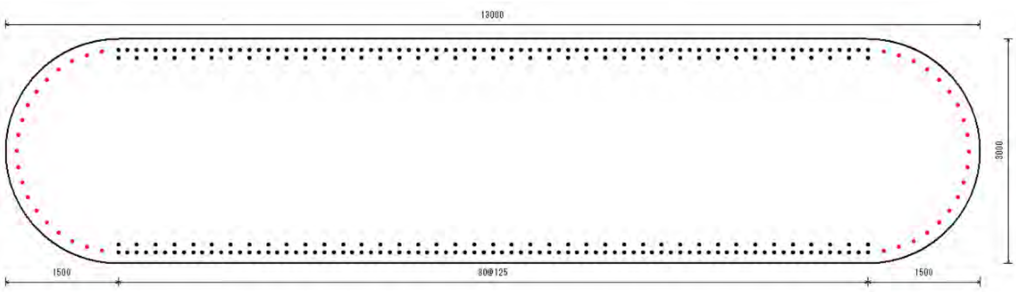


Figure 4.3.83 Rebar Arrangement (Main Reinforcement)

Source: JICA Study Team

d) 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

5) Verification

Pier column structure is verified by compressive stress of concrete, tensile stress of rebar, shear stress and content of shear reinforcement. The result of verification at earthquake condition which is critical condition for pier column design is summarized in the table below. Since average shear stress is over allowable stress that only concrete resists against shear force, shear reinforcement is arranged to meet the requirement.

Table 4.3.70 Verification of Pier Column Stress at Earthquake Condition

Bridge Axis Direction

Pier No.	Compressive Stress (N/mm^2)		Tensile Stress (N/mm^2)		Shear Stress (N/mm^2)		Shear Reinforcement Content (mm^2)		Judgement
	σ_c	σ_{ca}	σ_s	σ_{sa}	τ_m	τ_{a1}, τ_{a2}	Aw	Aw_{Req}	
P6	8.3<	15.0	249.4<	300.0	0.26<	0.28,2.85	4871.0>	0.0	OK
P7	10.7<	15.0	281.3<	300.0	0.33<	0.32,2.85	5032.0>	106.3	OK

Bridge Axis Perpendicular Direction

Pier No.	Compressive Stress (N/mm^2)		Tensile Stress (N/mm^2)		Shear Stress (N/mm^2)		Shear Reinforcement Content (mm^2)		Judgement
	σ_c	σ_{ca}	σ_s	σ_{sa}	τ_m	τ_{a1}, τ_{a2}	Aw	Aw_{Req}	
P6	2.2<	15.0	13.9<	300.0	0.25<	0.19,2.85	1146.0>	95.1	OK
P7	4.9<	15.0	77.4<	300.0	0.36<	0.21,2.85	1548.4>	244.5	OK

σc: Compressive Stress of Concrete σca: Allowable Compressive Stress of Concrete

σs: Tensile Stress of Rebar σsa: Allowable Tensile Stress of Rebar

τm: Average Shear Stress τa1: Allowable Shear Stress if only concrete resists against shear force

τa2: Allowable Shear Stress if both concrete and shear reinforcement resist against shear force

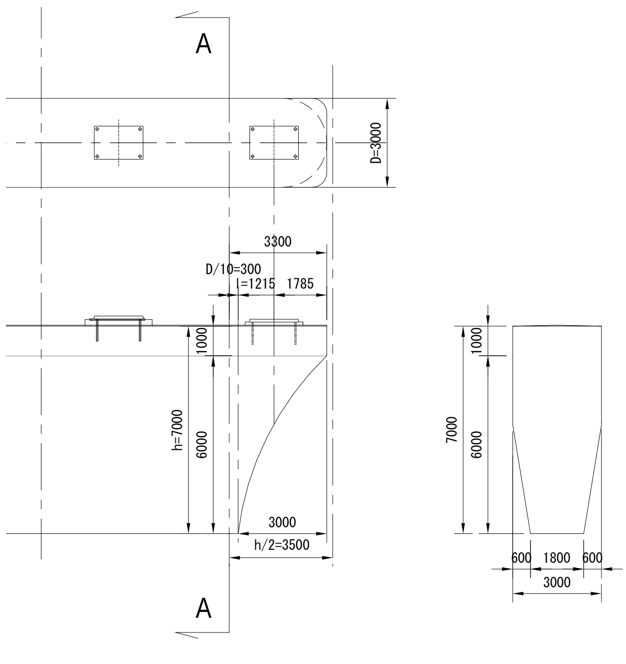
Aw: Shear reinforcement content Awreq: Required shear reinforcement content in τa1 <τm

Source: JICA Study Team

(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 4.3.84 Design Section of Pier Head Beam

2) Design Loads

Design loads for verification of the beam structure are summarized in the table below, and the largest values among piers for each condition are used for the verification.

Table 4.3.71 Design Loads for Beam Design

Condition	Load Component	P6		P7	
		G1	G4	G1	G4
Vertical Section					
Ordinary Condition (Dead + Live Loads)	Dead Load	4,000	4,800	5,700	6,000
	Live Load with Impact	2,000	2,200	2,300	2,400
	Weight of Beam	1,223	1,223	1,223	1,223
	Total	7,223	8,223	9,223	9,623
Earthquake Condition (Dead Load + Effect of earthquake)	Dead Load	4,000	4,800	5,700	6,000
	Weight of Beam	1,223	1,223	1,223	1,223
	Vertical reaction force due to earthquake from Superstructure*1	700	800	900	1,000
	Total	5,923	6,823	7,823	8,223
Additional load in the earthquake condition for corbel design	Inertia force on superstructure	1,300	1,300	1,800	1,800
	Inertia force on the beam	400	400	400	400
	Total	1,700	1,700	2,200	2,200
Horizontal Section					
Effect of temperature change	Horizontal force due to temperature change	100	100	100	100
Earthquake Condition	Inertia force on superstructure	1,200	1,200	1,100	1,100
	Inertia force on the beam	400	400	400	400
	Total	1,600	1,600	1,500	1,500

*Note: *1: It is calculated in accordance with Chapter 15.4 of Specifications for Highway Bridges Part-V 2012 (Japan Road Association)*

Source: JICA Study Team

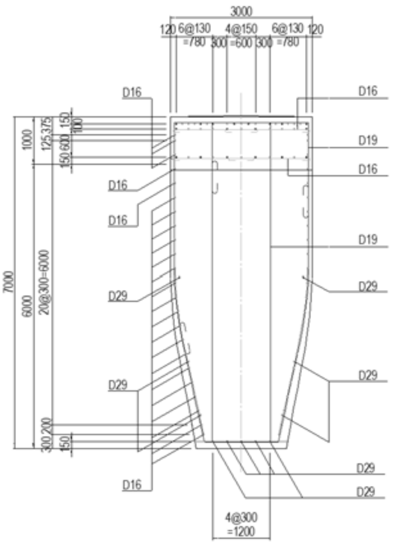
3) Design condition

- Applied reinforcement bar: SD345 for main reinforcement, stirrup
- Concrete design strength: 30 N/mm^2

4) Rebar Arrangement

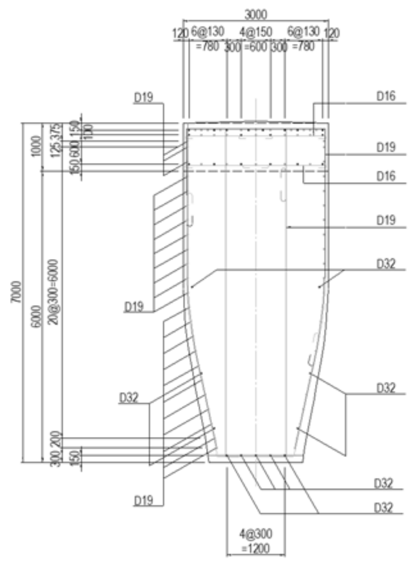
Main reinforcement and stirrup at the design section (A-A) is arranged as shown in the figure below.

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 4.3.85 Rebar Arrangement (Main Reinforcement)

5) 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.

Table 4.3.72 Verification of Reinforcement Content at Vertical Direction of Beam

P6

Item	Unit	Ordinary Condition (Dead Load)	Ordinary Condition (Dead and Live Loads)	Earthquake Condition
Design Tensile Force (T)	kN	1,604.1	2,196.0	3,519.3
Allowable Tension Stress (σ_{sa})	N/mm^2	100.00	180.00	300.00
Tensile Reinforcement Arranged Content (Asu) Required Content (AsuReq*1)	mm^2	Asu ≥ AsuReq OK 19,272.00 16,041.00	Asu ≥ AsuReq OK 19,272.00 12,199.73	Asu ≥ AsuReq OK 19,272.00 11,731.07
Reinforcement at Sides Arranged Content (Ass) Required Content (AssReq*2)	mm^2	Ass ≥ AssReq OK 9,135.00 7,708.80	Ass ≥ AssReq OK 9,135.00 7,708.80	Ass ≥ AssReq OK 9,135.00 7,708.80

P7

Item	Unit	Ordinary Condition (Dead Load)	Ordinary Condition (Dead and Live Loads)	Earthquake Condition
Design Tensile Force (T)	kN	1,882.4	2,510.2	4,344.0
Allowable Tension Stress (σ_{sa})	N/mm^2	100.00	180.00	300.00
Tensile Reinforcement Arranged Content (Asu) Required Content (AsuReq*1)	mm^2	Asu ≥ AsuReq OK 23,826.00 18,823.80	Asu ≥ AsuReq OK 23,826.00 13,945.64	Asu ≥ AsuReq OK 23,826.00 14,479.93
Reinforcement at Sides Arranged Content (Ass) Required Content (AssReq*2)	mm^2	Ass ≥ AssReq OK 13,179.00 9,530.40	Ass ≥ AssReq OK 13,179.00 9,530.40	Ass ≥ AssReq OK 13,179.00 9,530.40

*Note: *1: AsuReq = 1000 ・T / σsa*

**2: AssReq = 0.4 ・Asu*

Source: JICA Study Team

6) Verification for Bending Moment (Horizontal Bridge Axis Direction)

As for the section against bending moment at horizontal bridge axis direction, it is verified by the compressive and tensile stress occurring at section A-A by the allowable stress method, and the result is summarized in the table below.

Table 4.3.73 Verification of Bending Moment Stress at Horizontal Direction of Beam

Item	Unit	P6		P7	
		Effect of Temperature Change	Earthquake Condition	Effect of Temperature Change	Earthquake Condition
Bending Moment (M)	kN.m	155.80	2,413.05	151.50	2,209.95
Distance between edge of compressive side and neutral axis (x)	mm	340	340	391	391
Compressive Stress σ_c Tensile Stress σ_s	N/mm^2 N/mm^2	0.09 <σ_{ca} 9.80 <σ_{sa}	1.34 <σ_{ca} 151.75 <σ_{sa}	0.07 <σ_{ca} 6.95 <σ_{sa}	1.05 <σ_{ca} 101.34 <σ_{sa}
Coefficient Increase of Allowable Stress α Allowable Compressive Stress σ_{ca} Allowable Tensile Stress σ_{sa}	- N/mm^2 N/mm^2	1.15 11.50 207.00	1.50 15.00 300.00	1.15 11.50 207.00	1.50 15.00 300.00

Source: JICA Study Team

7) Verification of Shear Force (Horizontal Bridge Axis Direction)

As for the section against shear force at horizontal bridge axis direction, it is verified by the shear stress occurring at section A-A by the allowable stress method, and the result is summarized in the table below. Content of stirrup is 573.0mm^2, which is equivalent to approximately 0.2% at the minimum, although arrangement is not required.

Table 4.3.74 Verification of Shear Stress at Horizontal Direction of Beam

Item	Unit	P6		P7	
		Effect of Temperature Change	Earthquake Condition	Effect of Temperature Change	Earthquake Condition
Section Force Shear Force (S) Bending Moment (M)	 kN kN.m	 100.00 155.80	 1,566.89 2,413.05	 100.00 151.50	 1,466.89 2,209.95
Effective Height (d)	mm	2,732	2,732	2,735	2,735
Shear Force considering Effective Height (Sh)*1	kN	100.00	1,566.89	100.00	1,466.89
Coefficient Increase of Allowable Stress (α) Ratio of rebar (pt)*2 Coefficient of allowable shear stress related to "d" (ce) Coefficient of allowable shear stress related to "pt"(cpt)	- % - -	1.15 0.038 0.740 0.576	1.50 0.038 0.740 0.576	1.15 0.052 0.740 0.605	1.50 0.052 0.740 0.605
τ_m τ_{a1} τ_{a2}	N/mm^2 N/mm^2 N/mm^2	0.006 < 0.123 2.185	0.092 < 0.158 2.850	0.006 < 0.129 2.185	0.086 < 0.166 2.850
Shear Reinforcement Content Aw AwReq	 mm^2 mm^2	 573.0 0.0	 573.0 0.0	 573.0 0.0	 573.0 0.0

*Note: *1: Sh=M-M/d x (tanβ+tanγ), β and γ = zero in this case *2: pt=As/(b x d)*

Source: JICA Study Team

8) Verification of Bridge Seat

a) Seating Length at P5 and P10

The seating length, which is defined as the distance between the edge of the girder and the edge of the top of substructure in longitudinal direction, should be long enough to prevent departure and unseating of the superstructure from the top of the substructure. The required value (S_{EM}) can be calculated at the equation below as specified in the JSBH.

S_{EM} = 0.7 + 0.005 x span length (m)

Table 4.3.75 Verification of Seating Length

Pier No.	P5	P10
Span Length	75.6m	102.8m
Required Seating Length (S_{EM})	1.078m	1.214m
Seating Length	2.150m	3.550m
Judge	OK	OK

Source: JICA Study Team

b) Bearing Edge Distance (S)

The bearing edge distance (S), which is defined as the distance between the center of anchor bolt of bearing and the edge of the top of the substructure, shall be equal to or larger than the following value:

S ≥ 0.2+0.005 x span length (m)

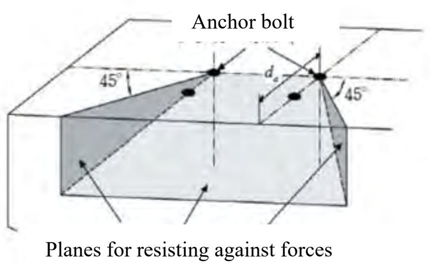
Table 4.3.76 Verification of Bearing Edge Distance

Pier No.	P5	P6	P7	P10
Span Length	75.6m	76.5m	102.8m	102.8m
Minimum Edge Distance	0.578m	0.583m	0.714m	0.714m
Edge Distance (S)	0.620m	1.016m	1.040m	0.918m
Judge	OK	OK	OK	OK

Source: JICA Study Team

c) Bridge Seat Strength

Bridge seats is designed with sufficient strength to withstand the vertical and horizontal forces through bearings. Horizontal force (design horizontal seismic force) transmitted from the bearings is resisted by concrete and reinforcement. The resisting area of concrete is the summation of three planes in directions of sideward and downward with edge angles of 45 degrees as shown in figure below.



Source: JSAB

Figure 4.3.86 Image of Resisting area of Concrete

The calculation results on the bridge seats against design horizontal seismic force is summarized in table below.

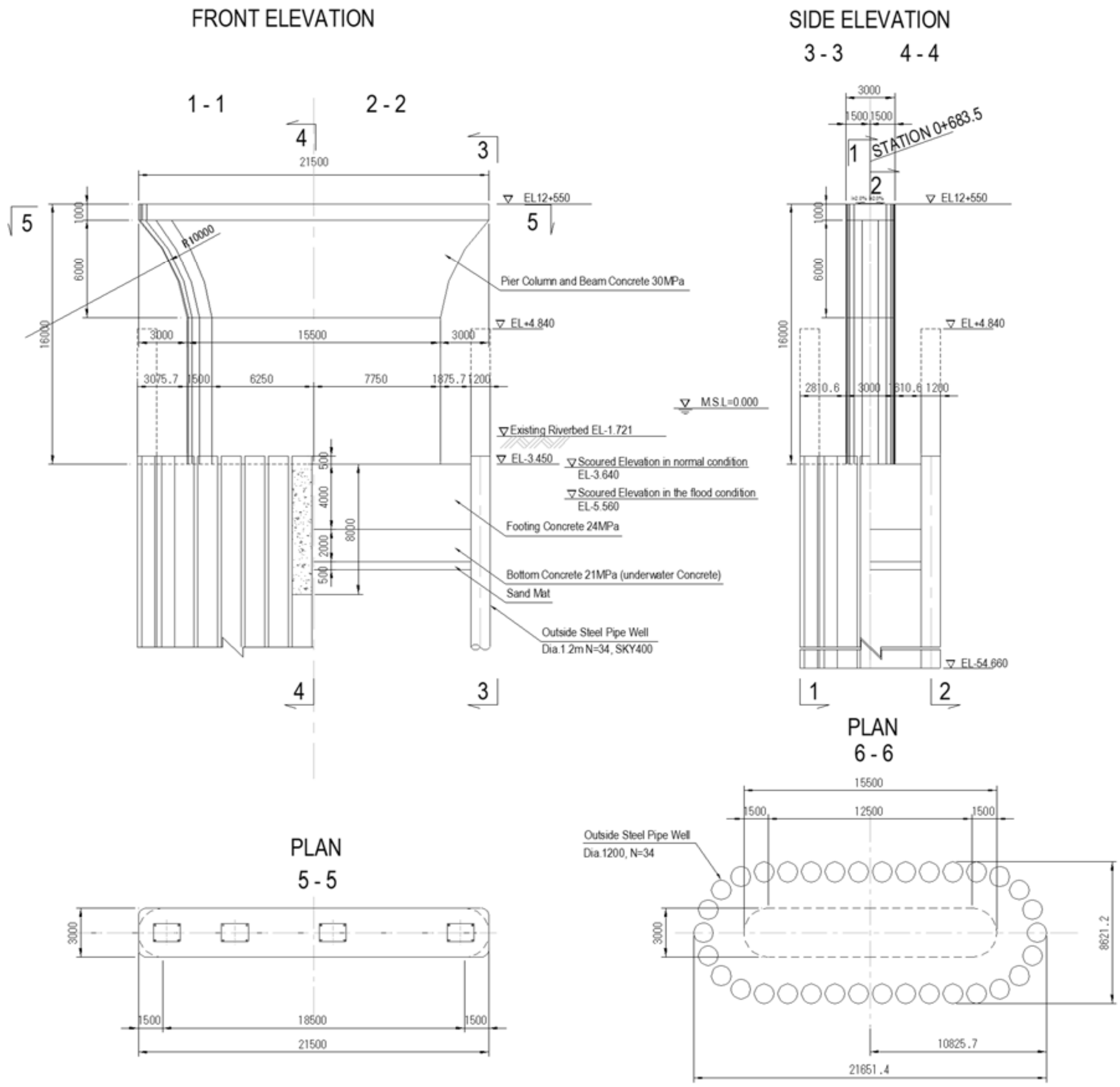
Table 4.3.77 Verification Result of Bridge Seat Strength

Pier No.	P6		P7	
Girder	G1	G4	G1	G4
Design horizontal seismic force Ph (kN) *per bearing	1,200	1,200	1,100	1,100
Resistance area of concrete Ac (mm^2)	6,657,000	8,218,000	6,234,000	8,330,000
Bearing stress at bottom of bearing support against vertical force σ n (N/mm^2)	2.07	2.50	2.80	2.97
Coefficient for determining the strength borne by concrete	0.27	0.30	0.31	0.32
Strength borne by concrete Pc (kN)	3,160	4,260	3,420	4,710
Strength borne by reinforcement Ps (kN)	1,890	1,890	1,790	1,980
Strength of bridge seat Pbs (kN)	5,050	6,150	5,210	6,690
Judge (Pbs≧Ph)	OK	OK	OK	OK

Source: JICA Study Team

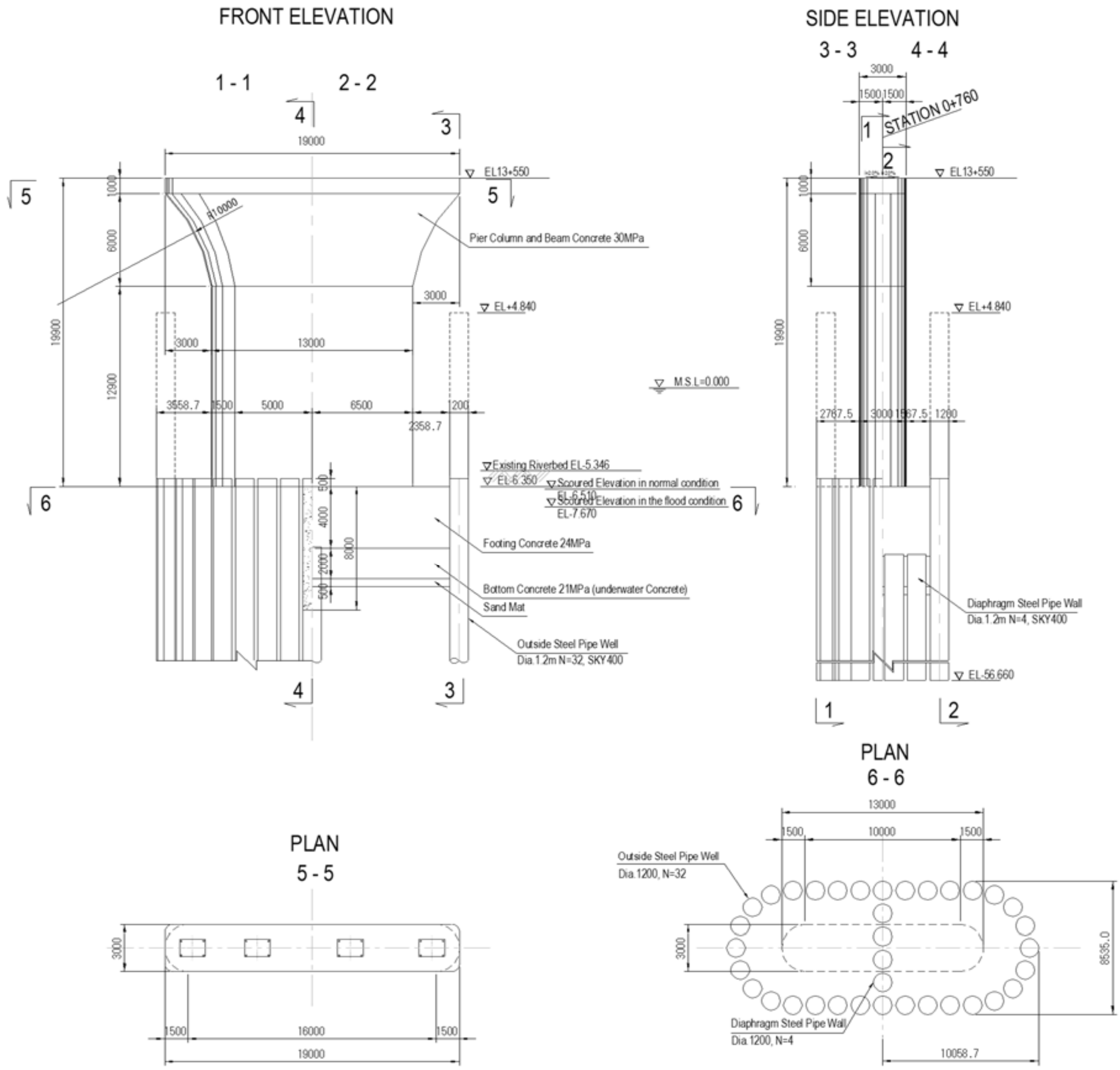
4.3.6.4 Structure Drawing

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 4.3.87 Structure Drawing of Substructure of P6



Source: JICA Study Team

Figure 4.3.88 Structure Drawing of Substructure of P7

4.3.6.5 Quantities of Major Items of Substructure

Quantity of the major items of the substructure for the steel box girder bridge is calculated for cost estimation and it is summarized in the following table.

Table 4.3.78 Quantity of the Major Items of the Substructure for the Steel Box Girder Bridge

Item	P6	P7	Total
SPSP (ton)			
Permanent part	741.2t	772.1t	1,513.3t
Temporary part	109.9t	165.9t	275.8t
Concrete (m^3)			
24 MPa	482.8m^3	436.4m^3	919.2m^3
30 MPa	774.4m^3	797.5m^3	1,571.9m^3
Re-bar (ton)			
D13-D32	119.3t	76.2t	195.5t
D38-D51	0.0t	77.6t	77.6t

Source: JICA Study Team

4.3.7 Detailed Design of Bridge Accessories

4.3.7.1 Bearings for 7-Span Bridge

(1) Design Conditions

a) Applied Design Standard:

- Specifications for Highway Bridges Part-V 2012 (Japan Road Association)
- Manual for Bearing for Highway Bridges 2004 (Japan Road Association)

b) Design Temperature Range: 25℃±25℃, and additional 5℃ is considered as initial deformation at the installation of the bearings.

c) Classification of Ground Condition: Class III

d) Support Condition

Table 4.3.79 Support Condition

Pier No.	Span Length L(m)	Nos. of bearing N	Support Condition	
			Bridge Axis Direction	Bridge Axis Perpendicular Direction
P13	---	4	Movable	Fix
P14	110.8	4	Fix	Fix
P15	112.0	4	Fix	Fix
P16	112.0	4	Fix	Fix
P17	112.0	4	Fix	Fix
P18	112.0	4	Fix	Fix
P19	112.0	4	Fix	Fix
P20	103.1	4	Movable	Fix

Source: JICA Study Team

e) Design Loads from Superstructure for Bearing Design:

Table 4.3.80 Design Loads from Superstructure

Pier No.	Max. Reaction Rmax1 (kN)	For Rotation Rmax2 (kN)	For Stress Amplitude Rmax' (kN)	For Stress Amplitude Rmin' (kN)	Dead Load ΣRd (kN/pier)	Dead Load Rdmax (kN)	Max. Live load Rlmax (kN)
P13	3178	3123	3178	1991	8561	2188	1031
P14	8070	7973	8070	5655	24252	6101	0
P15	7009	6941	7009	4563	20362	5137	0
P16	7442	7134	7442	4954	21503	5332	0
P17	7360	7092	7360	4885	21287	5293	0
P18	7176	6950	7176	4726	20731	5171	0
P19	7737	7653	7737	5377	23141	5826	0
P20	2913	2735	2913	1582	7455	1825	1010

Source: JICA Study Team

f) Design Horizontal Force for Bearing Design:

Table 4.3.81 Horizontal Forces

Pier No.	Bridge Axis Direction		Bridge Axis Perpendicular Direction
	Ordinary Condition (kN/pier)	Earthquake Condition Level-1 (kN/pier)	Earthquake Condition Level-1 (kN/pier)
P13	900	900	2,600
P14	10,800	7,000	7,300
P15	6,200	7,000	6,200
P16	2,400	7,300	6,500
P17	1,800	7,500	6,400
P18	6,200	7,700	6,300
P19	11,500	7,900	7,000
P20	800	800	2,300

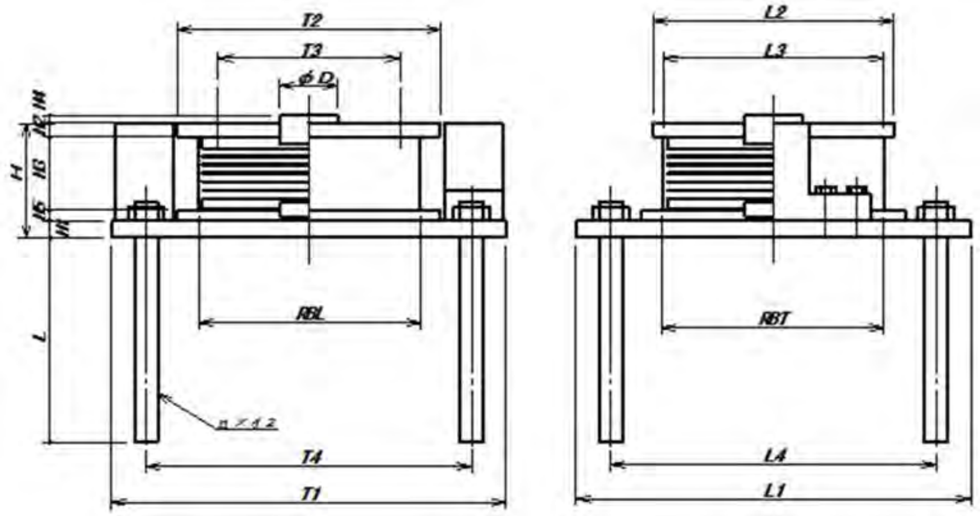
Source: JICA Study Team

g) Displacement for Bearing Design:

130 mm at P13 and 120 mm at P20 due to effect of temperature change (Δ30°C).

(2) 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

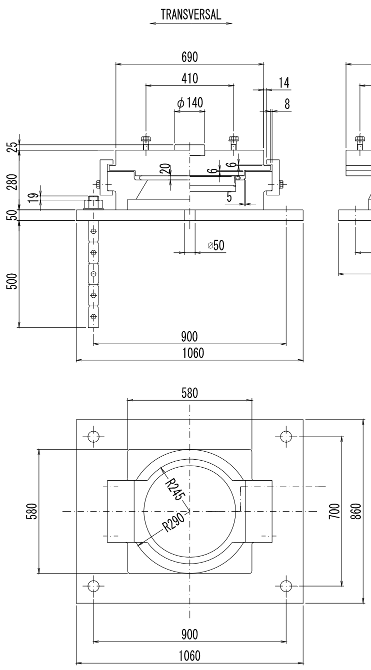
Pier	Hexagon Bolt				Anchor Bolt						
No.	d1	n	L3	T3	d2	L	n	L4	T4	H5	H
P14	M24	8	3×230	1×690	φ100	1000	4	1100	1300	40	359
P15	M24	8	3×230	1×690	φ90	900	4	1080	1300	40	353
P16	M24	8	3×230	1×690	φ90	900	4	1080	1300	40	353
P17	M24	8	3×230	1×690	φ90	900	4	1080	1300	40	353
P18	M24	8	3×230	1×690	φ90	900	4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

Source: JICA Study Team

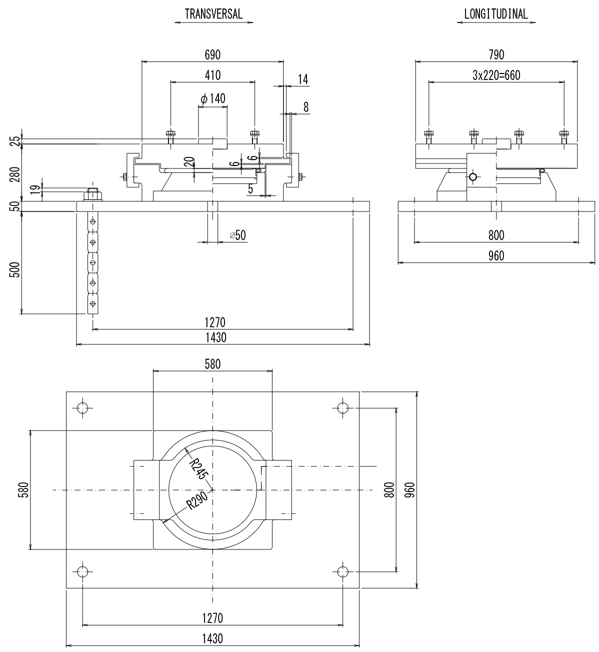
Figure 4.3.89 Dimension of Bearing (Fixed Rubber Bearing Type)

P13 and P20 Movable Support (BPB Type):

P13



P20



Source: JICA Study Team

Figure 4.3.90 Dimension of Bearing (Movable BPB Type)

4.3.7.2 Bearings for 3-Span Bridge

(1) Design Conditions

a) Applied Design Standard: same as 7-span bridge

b) Design Temperature Range: 25℃±25℃

c) Classification of Ground Condition: same as 7-span bridge

d) Design Seismic Coefficient: The bearings of P5 and P10 shall be designed by design seismic coefficient 0.45 (0.3 x 1.5times) to meet the requirement of the seismic performance 2 instead of installation of any other anti-collapse structure.

e) Support Condition

Table 4.3.82 Support Condition

Pier No.	Span Length L(m)	Nos. of bearing N	Support Condition	
			Bridge Axis Direction	Bridge Axis Perpendicular Direction
P5	---	4	Elastic	Fix
P6	75.6	4	Elastic	Fix
P7	76.5	4	Elastic	Fix
P10	102.8	4	Elastic	Fix

Source: JICA Study Team

f) Design Loads from Superstructure for Bearing Design:

Table 4.3.83 Design Loads from Superstructure

Pier No.	Max. Reaction Rmax1 (kN)	For Rotation Rmax2 (kN)	For Stress Amplitude Rmax' (kN)	For Stress Amplitude Rmin' (kN)	Dead Load ΣRd (kN/pier)	Dead Load Rdmax (kN)	Max. Live load Rlmax (kN)
P5	3,400	2,100	3,200	1,300	7,200	2,100	1,400
P6	7,000	5,000	7,000	4,400	16,700	4,800	2,200
P7	8,400	6,200	8,000	5,100	21,400	6,000	2,400
P10	4,100	3,200	3,500	1,200	9,000	2,500	1,700

Source: JICA Study Team

g) Design Horizontal Force for Bearing Design:

To estimate the horizontal force due to effect of temperature change, range Δ50°C is applied in accordance with the conditions for estimation of the displacement.

Horizontal force in bridge axis direction at end bearings is calculated by using 1.5 times of design seismic coefficient

Table 4.3.84 Horizontal Forces

Pier No.	Bridge Axis Direction: Ordinary Condition(Δ50°) (kN/pier)	Bridge Axis Direction: Earthquake Condition Level-1 (kN/pier)	Bridge Axis Direction: Earthquake Condition Level-1 Design seismic coefficient	Bridge Axis Perpendicular Direction: Earthquake Condition Level-1 (kN/pier)	Bridge Axis Perpendicular Direction: Earthquake Condition Level-1 Design seismic coefficient
P5	1,300	5,800	0.3 x 1.5	2,000	0.3
P6	600	4,600	0.3	4,900	0.3
P7	500	4,400	0.3	6,900	0.3
P10	1,400	5,300	0.3 x 1.5	2,500	0.3

Source: JICA Study Team

h) Displacement for Bearing Design:

To estimate the displacement due to deflection of superstructure due to live loads and effects of temperature change, range Δ50°C is applied so that the bearing is able to be set at any temperature without adjustment of displacement (rubber deformation).

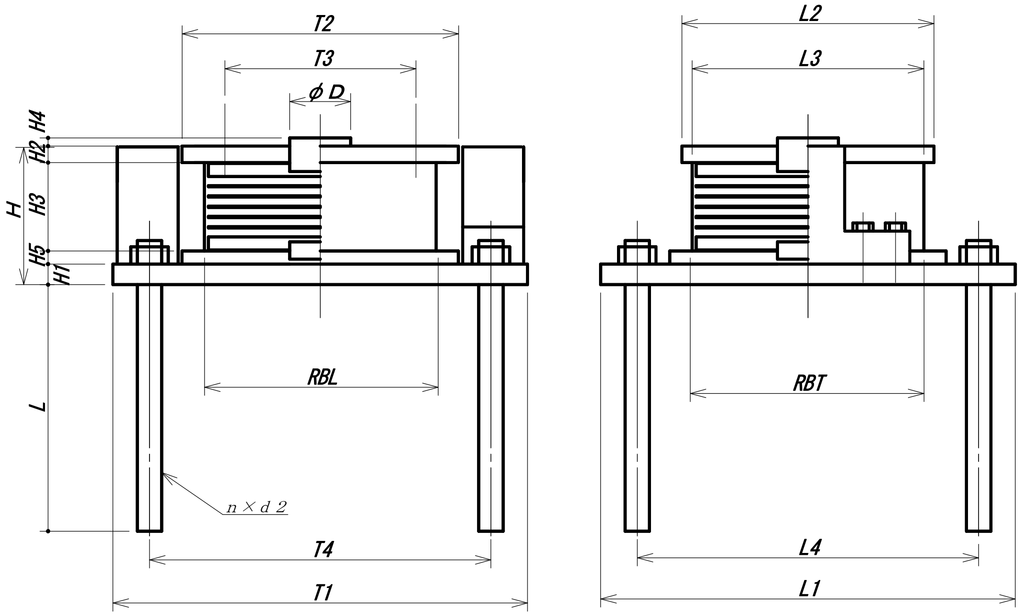
According to the design condition as mentioned above, bearings of P5 and P10 are designed by 1.5 times of design seismic coefficient.

Table 4.3.85 Displacement

Pier No.	Bridge Axis Direction: Ordinary Condition(Δ50°C) Displacement	Bridge Axis Direction: Earthquake Condition Level-1 Displacement	Bridge Axis Direction: Earthquake Condition Level-1 Design seismic coefficient
P5	80mm	310mm	0.3 x 1.5
P6	28mm	180mm	0.3
P7	10mm	170mm	0.3
P10	88mm	286mm	0.3 x 1.5

Source: JICA Study Team

(2) Dimension of Bearing



Bridge Axis Direction Bridge Axis Perpendicular Direction

Pier No.	Base Plate			Upper Plate				Rubber		
	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 No.	Hexagon Bolt				Anchor Bolt						
	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 No.	Horizontal Spring Stiffness(N/mm)	
	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 4.3.91 Dimension of Bearing

4.3.7.3 Expansion Joint for 7-Span Bridge

(1) Design Concept

- The adjacent gap and marginal length of girder end have been determined taking account of the displacement due to seismic movement and temperature elongation between adjacent bridges.
- The modular type is adopted considering the above condition.

(2) 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 4.3.86 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°C± 25°C)	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

(3) 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.

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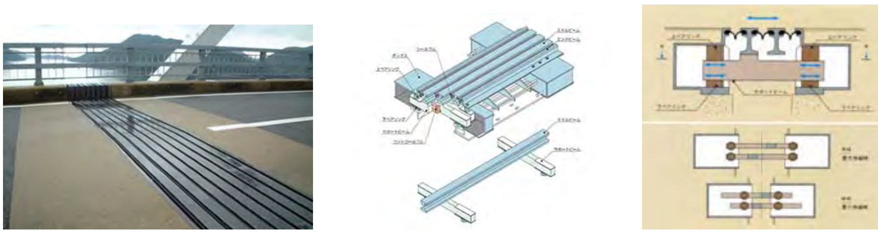
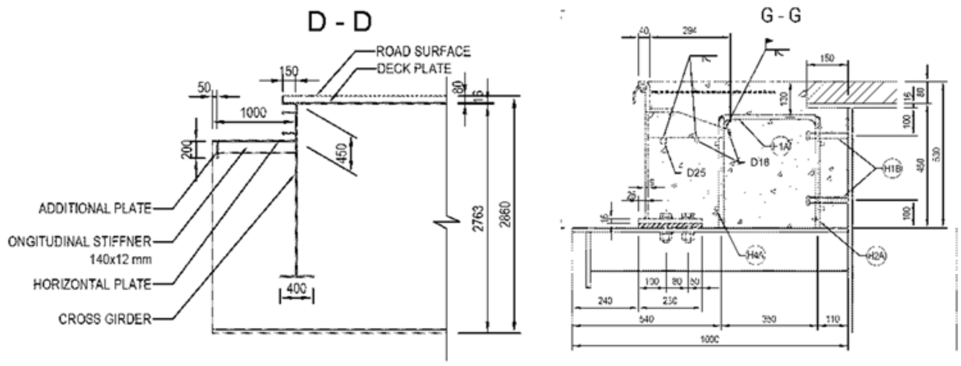


Source: Catalogue from manufacturer

Figure 4.3.92 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.



Source: JICA Study Team

Figure 4.3.93 Modification at End of Steel Deck for the Installation of the Expansion Joint

4.3.7.4 Expansion Joint for 3-Span Bridge

(1) Design Concept

Same design concept as 7-span bridge is considered.

(2) 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 4.3.87 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℃± 25℃)	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	P10 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℃±25℃)	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

(3) 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 Design Principle

The length of the PC box girder bridge and its span arrangement were comprehensively examined considering terrain on site, geological condition, crossing obstacles, construction workability and economic efficiency. At this conjuncture, utilization of technologies of Japanese companies and promotion for their participation shall be taken into account in accordance with the F/S as instructed in the TOR of this study.

4.4.2.2 Study Conditions

(1) Geography and Geology

- A1 (Thilawa) side

The site for this approach bridge consists of a flood channel and a low-flow channel river whose elevations are around MSL+3.00 m~4.00 m and MSL -5.00 m~ 7.00 m, respectively. No future land use plan, including reclamation or river training plan, exists.

The bearing stratum for the bridge is Clayey SAND-II distributed at MSL-40.0 m~ -55.0 m, whose N-value is around 50. There are no appropriate soil layers other than this layer with sufficient firmness and thickness to support bridge reactions.

- A2 (Yangon) side

The site for this approach bridge consists of a flood channel and a low-flow channel river whose elevations are around MSL+2.50 m~4.50 m and MSL-1.50 m~ 6.00 m, respectively. No future land use plan, including a reclamation or river training plan, exists.

The bearing stratum for the bridge is Clayey SAND-I and Clayey SAND-II distributed at MSL-40.0 m~ -45.0 m, whose N-value is around 50. There are no appropriate soil layers other than this layer with sufficient firmness and thickness to support bridge reactions.

(2) Crossing Object Conditions

Crossing objects are investigated by means of site survey and literature survey supported by the counterpart. There are some objects that should be taken into account in the bridge span arrangement planning as control points. These are navigation channel of Bago River, existing in-river piers of Thanlyin Bridge, and the embankment section of on-ramp road (to be constructed under this Project). It is confirmed that other crossing objects are available for relocation. Summary of the crossing objects is shown in Table 4.4.2.

Table 4.4.2 Summary of Crossing Obstacles for Span Arrangement

No.	Crossing Object Name	Chainage	Relocation	Control Point	
				Abut-ment	Pier
1	On-ramp (embankment section)	0+542.5	Possible	No	No
2	Natural levee (boundary of low-flow channel)	0+654.0	No	**Yes**	No
3	Left end of navigation (P10) *Note 1	0+864.0	No	-	**Yes**
4	Right end of navigation (P22)	2+88.0	No	-	**Yes**
5	Low-flow channel	2+238.0	No	**Yes**	No
6	Power cable (high tension)	2+384.0	No	No	No
7	Toll gate area	2+400.0	No	**Yes**	**Yes**

Note: *Control point of bridge span examination for B/D stage. Based on a notice of DWIR made during the D/D stage regarding the navigation channel location of Bago River, the control point of the left end of navigation channel should be around STA No. 0+760. In accordance with this notice, the JICA Study Team reconsidered the span arrangement and bridge type, then decided to substitute 3-span continuous steel box girder bridge for 4-span PC box girder bridge.*

Source: JICA Study Team

(3) Construction Conditions

- Site Conditions

There are no buildings/facilities which require cautious construction adjacent to the bridge in order to avoid any harmful displacement. There are no objects that restrain the construction works.

- River Conditions

Tidal fluctuation is a dominant factor for the river water level variation. Design high water level is MSL+4.990 m, and the design high water level for construction is MSL+ 4.340 m. Tidal range is approximately 5.00 m for spring tide.

- River flow velocity and flow direction:

Maximum flow velocity is 1.0 m/s.

- Economic span for BD

The economic span length is estimated using the following formula which is recommended in the "Bridge Design Standards of NEXCO (East Nippon Expressway Company Limited)":

L= a x {h+1/3(Df)}

Where,

h = Substructure height

Df = Foundation depth

a = coefficient (1 ~ 1.5) depending on construction circumstances of a proposed bridge

The construction circumstance of the target bridge is worse because of existence of in-river piers. Consequently, a longer span length is more economical than one with shorter length in general due to lesser number of in-river piers. Based on this viewpoint, the coefficient "a" should be 1.5. As a result of economic span length estimation, it is determined that the economic span is 50.0 m as shown in Table 4.4.3.

Table 4.4.3 Estimation of Economic Span Length

Item	A1 (Thilawa) Side	A2 (Yangon) Side
Average substructure height: h	18.7 m	19.4 m
Average foundation depth: Df	48.4 m	37.4 m
h + 1/3 x Df	34.8 m	31.9 m
Case 1: a = 1.0	35.0 m	31.9 m
Case 2: a = 1.5	52.3 m	47.8 m
Proposed economic span length	**50.0 m**	**50.0 m**

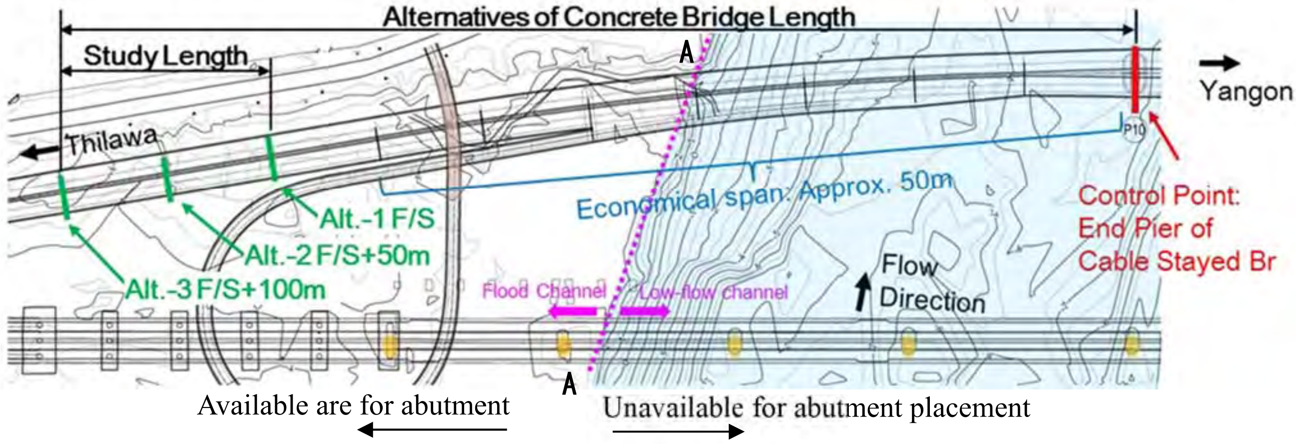
Source: JICA Study Team

4.4.2.3 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)

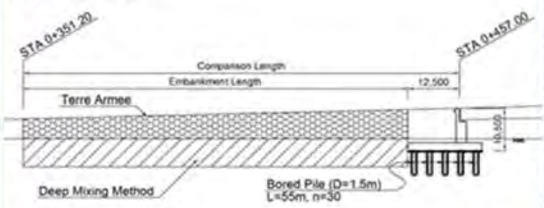
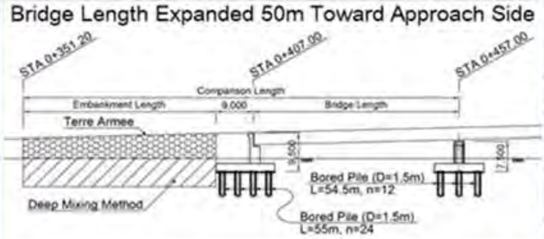
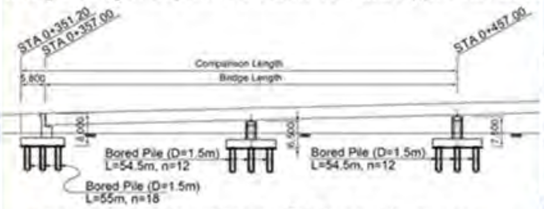
Alternative 3: A1 Abutment at STA No. 0+357.0 m, L = 507 m (F/S + 100 m)

- Comparison Result

As shown in Table 4.4.4, 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 4.4.4 Comparison of Bridge Length at A1 Side

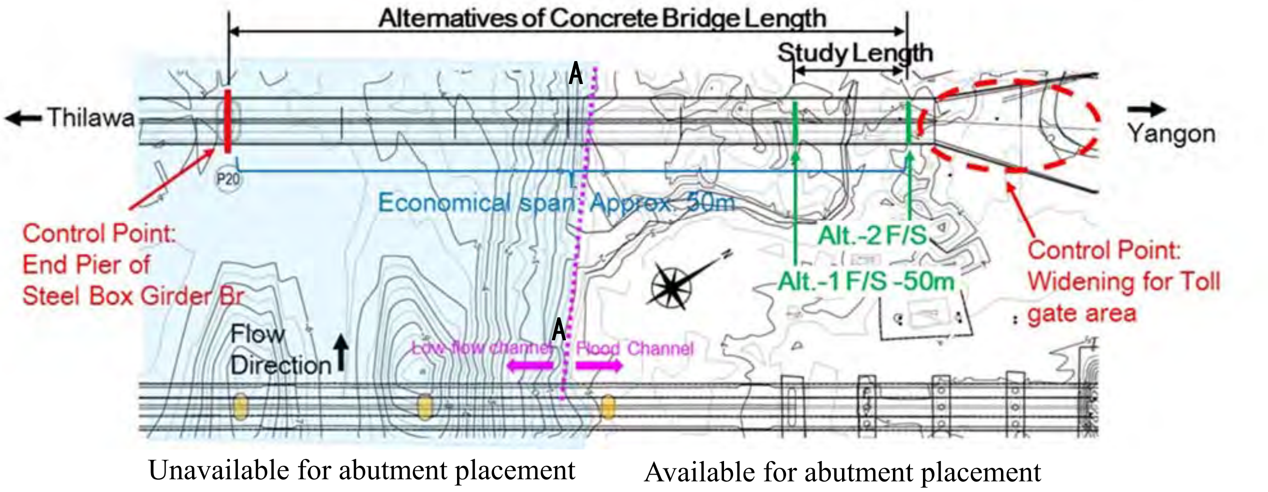
Alt-1	F/S Alternative 	Constructability & Quality Control	-The amount of road works including ground improvement by deep mixing method is largest.	△
		Cost Ratio	1.02	△
		Construction Period	8.7 months	△
		Environmental Aspect	-The amount of road works including ground improvement and embankment is largest.	△
		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)

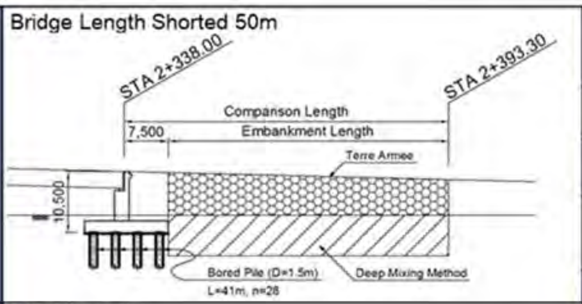
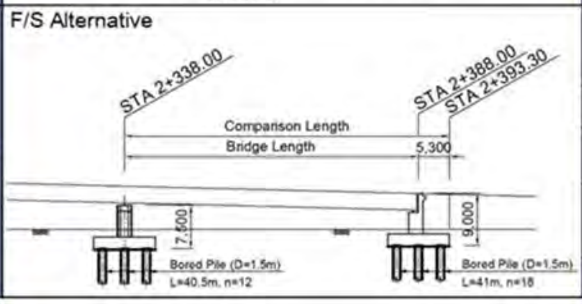
  Alternative 2: A2 Abutment at STA No. 2+388.0 m, Length = 300 m (F/S)

- 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 4.4.5. 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 4.4.5 Comparison of Bridge Length at A2 Side

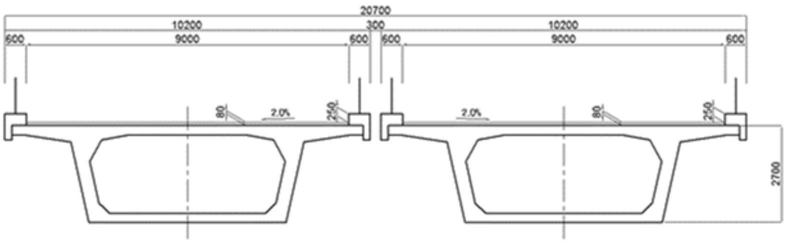
Alt-1	Bridge Length Shorted 50m 	Constructability & Quality Control	-The amount of road works including ground improvement by deep mixing method is largest.	△
		Cost Ratio	1.02	○
		Construction Period	8.3 months	△
		Environmental Aspect	-The amount of road works including ground improvement and embankment 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

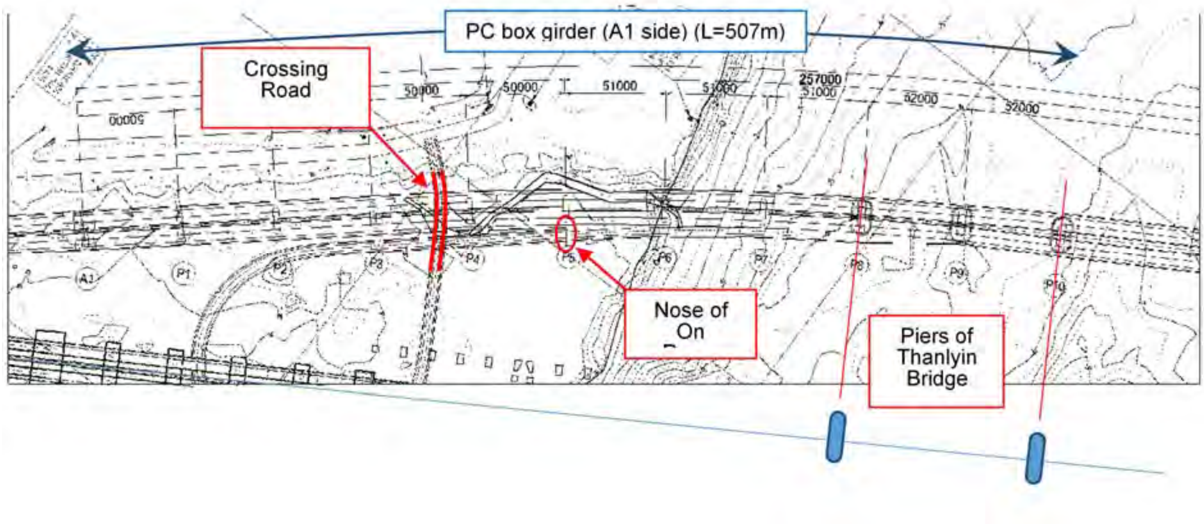
For A1 side, the following shall be taken into account for the determination of the pier locations:

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

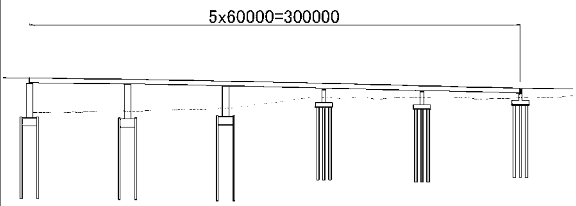
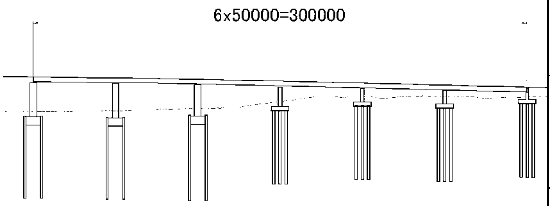
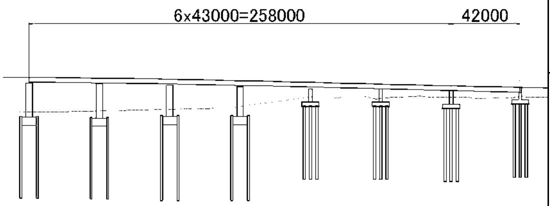
Figure 4.4.4 Restricting Conditions for Span Arrangement of A1 Side

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 4.4.6.

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 4.4.6 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	**Most Recommended**
		Girder height is adequate for span length, and reasonable design is possible.	
		Cost: Ratio = 1.00	
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 4.4.7.

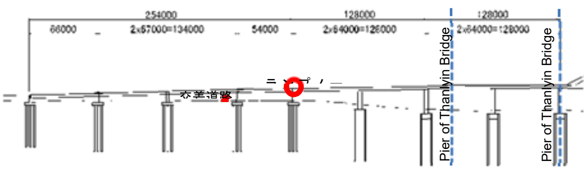
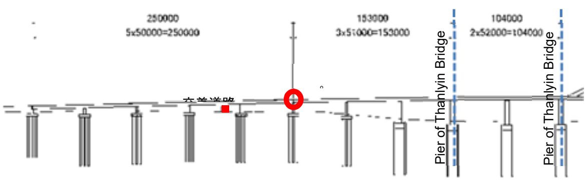
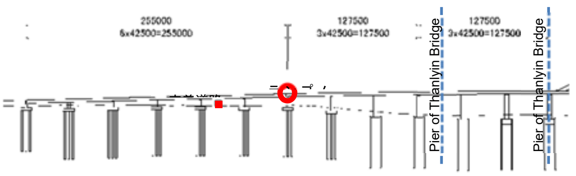
In addition to the advantages shown in the study at Yangon side (A2 side), span length of 50 m has the following advantages:

- 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 4.4.7 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.

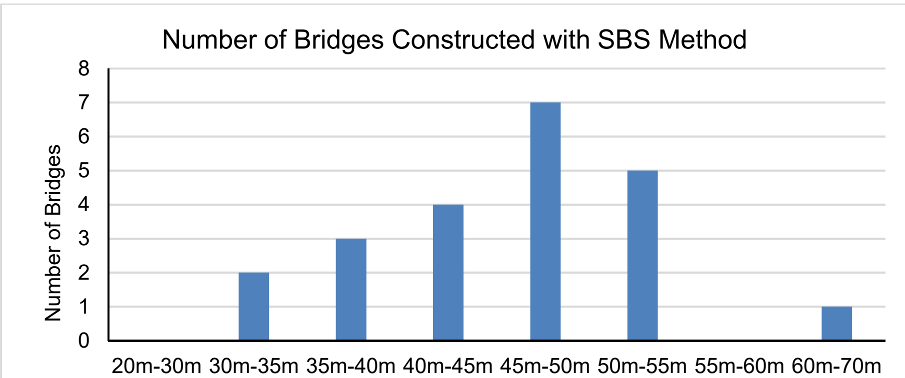
4.4.4 Study on Superstructure of PC Box Girder Bridge

4.4.4.1 Review of Bridge Type and Erection Method Selected in the F/S

In the preceding F/S, PC box girder bridge with precast segments (SBS (span-by-span) erection) has been selected as the bridge type and erection method from the viewpoints of utilization of Japanese bridge technology and request from MoC for introduction of new technology. JICA's ToR of this Detailed Design Study also states to adopt this policy. In this section, consequently, bridge type (PC box girder bridge) and erection method (SBS method with precast elements) are taken as precondition for the study, and applicability of these bridge type and erection method for the given design condition has been reviewed.

(1) PC Box Girder Bridge with Span-by-Span (SBS) Erection

Typical span length applied to SBS method is between 40 m ~ 55 m, but can also reach up to approximately 60 m based on some experiences. Hence, PC box girder with SBS erection is applicable without problem to spans of approximately 50 m.



Note: 22 examples were picked up (16 in Japan, 6 in overseas)
Source: JICA Study Team

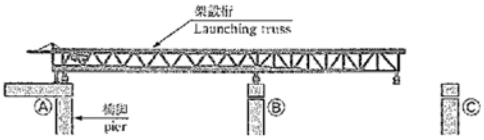
Figure 4.4.5 Typical Span Lengths Erected with SBS Method

Application of SBS method to this Project also has the following advantages:

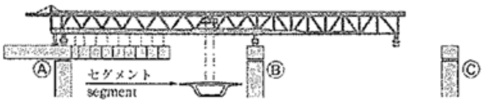
- PC box girder section has long total bridge length (1.6 km in total) – effectiveness in construction cost due to re-use of erection equipment
- Erection of superstructure with minimum use of the space below bridge for in-river section

Typical SBS erection procedure is shown in Figure 4.4.6.

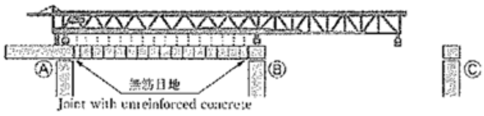
Step 1: Placing the pier segments and installing the overhead truss on them.



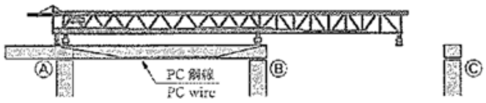
Step:2 : Lifting all segments of one span to be assembled.



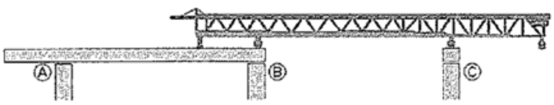
Step 3: Placing and curing concrete at closure joints (unreinforced).



Step 4: Post-tensioning the entire span.



Step 5: Launching the overhead truss forward and repeat new cycle.



Source: JICA Study Team, based on the "Manual for Planning of Prestressed Concrete Highway Bridges" by Japan Prestressed Concrete Contractors Association, 2007

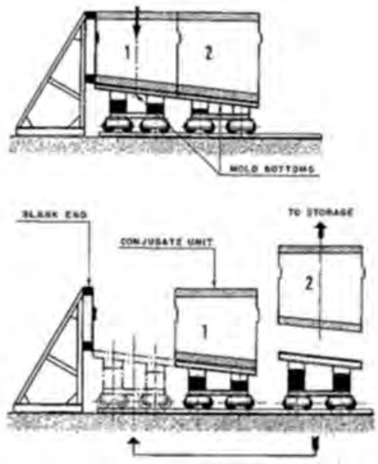
Figure 4.4.6 Erection Procedure of Span-by-Span Method (with Overhead Truss)

(2) Fabrication of Precast Elements

Fabrication of precast segments is categorized into two basic categories: long-line casting method and short-line casting method. Short-line method is applied in this Project. Advantages of short-line method compared to long-line method are as follows:

- Smaller yards for segment prefabrication.
- Applicable to curved girders using three-dimensional adjustment mechanism.
- Smaller area of soil improvement in case of soft ground yards.
- Concentrated quality control as the concrete is cast at the same place

Short-line method is thus considered to be suitable for this approach bridge as it has curved section and limited space for construction yards with soft ground.



Source : "Construction and Design of Prestressed Concrete Segmental Bridges" by Walter Podolny Jr., Jean M. Muller, 1982

Figure 4.4.7 Typical Short-Line Precasting Operation

4.4.4.2 Superstructure of PC Box Girder Bridge

(1) Basic Conditions for the Study of the Superstructure

Basic conditions for the study/review of superstructure are as follows:

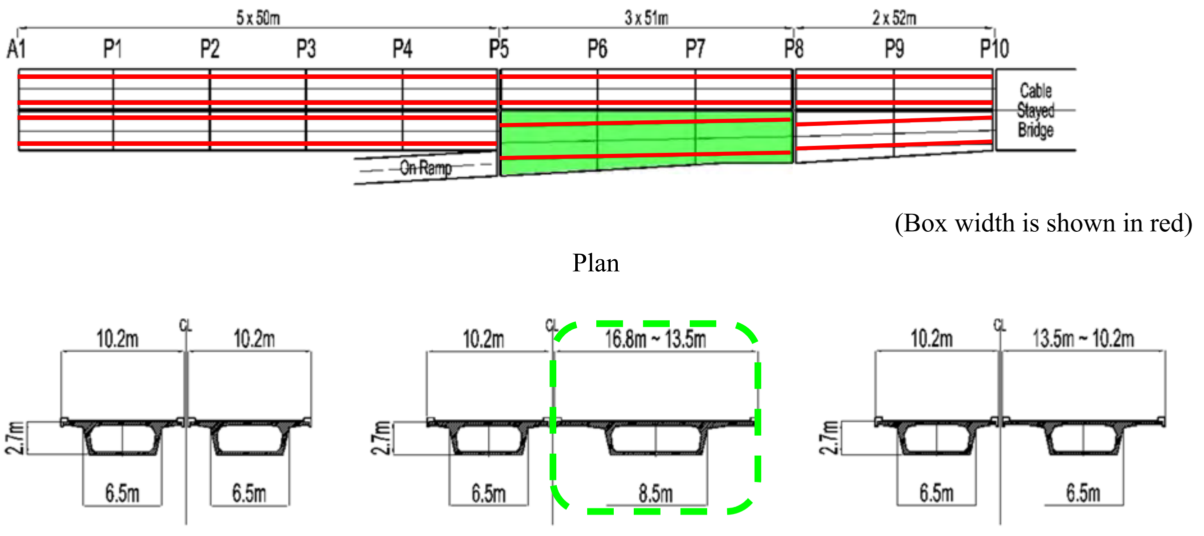
- Span length : approx. 50 m (from the study result on span length)
- Girder height : 2.7 m (unified with cable-stayed bridge and steel box girder bridge)
- Bridge type : PC box girder bridge (adopted in the F/S)
- Erection method : SBS erection with precast segments (adopted in the F/S)
- Road width : Widening due to merging of on-ramp shall be taken into account.
- Location of on-ramp nose, end of cable-stayed bridge shall also be considered.

(2) 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).



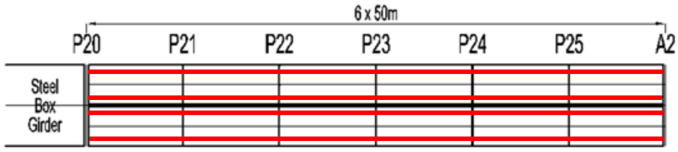
Source: JICA Study Team

Figure 4.4.8 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.

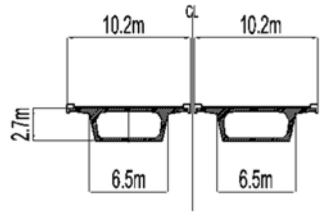
2) P20~A2

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

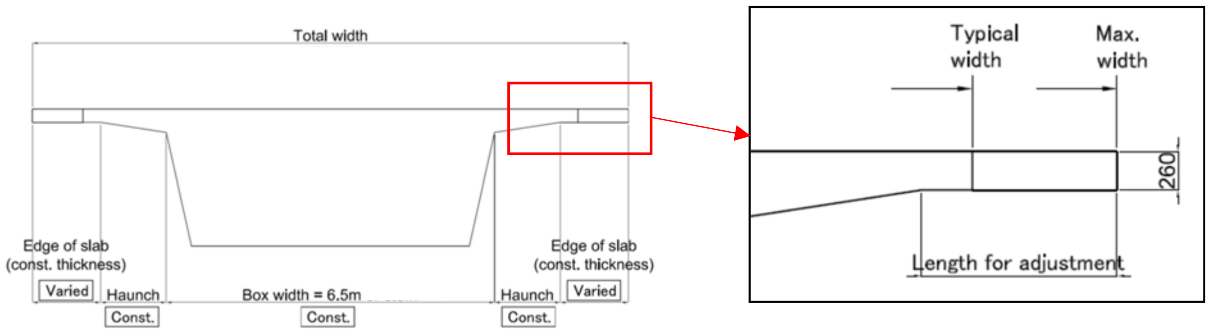
Figure 4.4.9 Bridge Layout and Box Width of the Girder (A1~P10)

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.

(3) 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 4.4.10 Accommodation to Curvature and Widening of Bridge

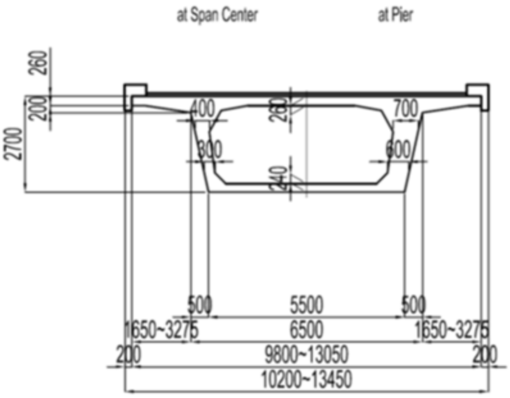
(4) Girder Height

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).

(5) 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 4.4.11 Girder Cross Section (Standard Section and P8~P10 Widened Section)

Table 4.4.8 Thickness of Girder Members (Standard Section and P8~P10 Widened Section)

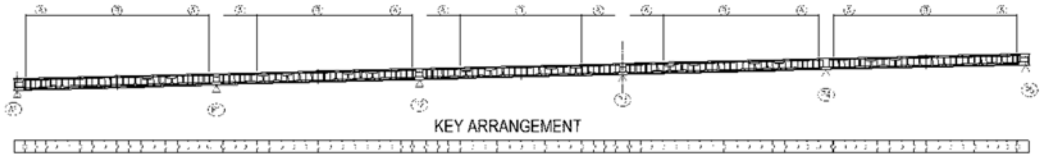
	Member	Thickness [mm]		Function
At span center	Top slab	At center	260	- Structural function: transverse deck slab to support wheel load, compression flange to resist bending of the girder - Arrangement of transverse tendons
	Cantilever slab	At web	460	- Structural function: transverse cantilever slab to support wheel load, compression flange to resist bending of the girder - Arrangement of transverse tendons
		At edge	260	- Structural function: transverse cantilever slab to support wheel load, compression flange to resist bending of the girder - Arrangement of transverse tendon anchors
	Bottom slab	At center	240	- Structural function: transverse box frame member to resist deformation - Arrangement of longitudinal tendons
	Web	Bottom	300	- Structural function: transverse box frame member to resist bending from top/cantilever slab, girder member to resist shear
		Top	400	
At intermediate support	Top slab	At center	260	- Structural function: transverse deck slab to support wheel load - Arrangement of longitudinal/transverse tendons
	Cantilever slab	At web	460	- Structural function: transverse cantilever slab to support wheel load - Arrangement of transverse tendons
		At edge	260	- Structural function: transverse cantilever slab to support wheel load - Arrangement of transverse tendon anchors
	Bottom slab	At center	240	- Structural function: transverse box frame member to resist deformation, compression flange to resist bending of the girder - Arrangement of longitudinal tendons
	Web	Bottom	450	- Structural function: transverse box frame member to resist bending from top/cantilever slab, girder member to resist shear
		Top	550	

Source: JICA Study Team

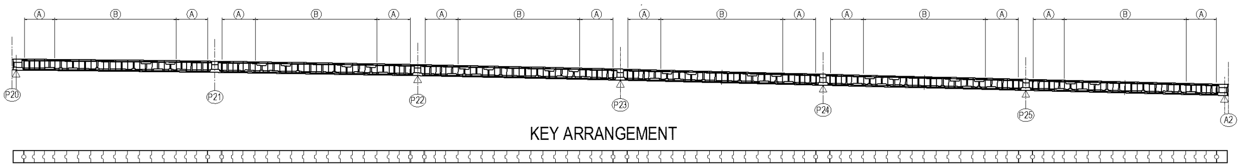
(6) Shear Key Arrangement

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 4.4.12 and Figure 4.4.13

<A1-P5>

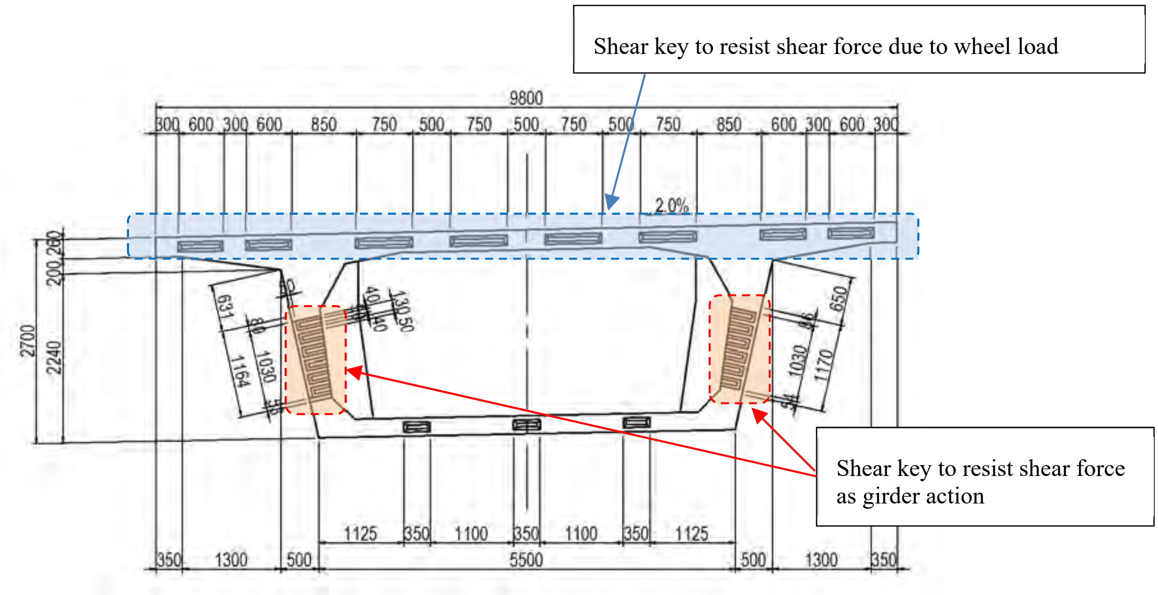


<P20-A2>



Source: JICA Study Team

Figure 4.4.12 Shear Key Arrangement (Side View)



Source: JICA Study Team

Figure 4.4.13 Shear Key Arrangement (Typical Section)

(7) Prestressing Tendons

1) Longitudinal Tendons

As the PC box girder bridge will be constructed with precast elements using the SBS method, the weight of superstructure shall be trimmed from construction points of view and for seismic aspects, On the other hand, it is desirable to place some internal tendons, which are integrated with and behave together with concrete section, to obtain adequate deformability of the girder. For the longitudinal prestressing of PC box girder bridges, therefore, internal tendons are applied in combination with external tendons, to obtain deformability of the girder while minimizing increase of member thickness due to arrangement of internal 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 4.4.9 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

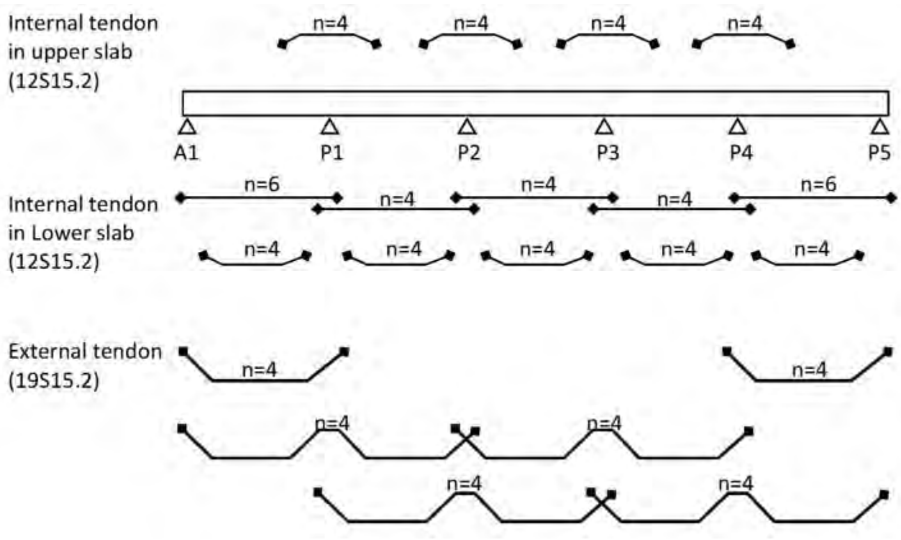
Both pre-tensioning and post-tensioning are applicable to transverse prestressing for deck slabs of precast segmental box girders. In this Project, post-tensioning method has been selected for deck slab prestressing, which is superior in geometry control of precast segments. The 3S12.7 mm has been selected 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.

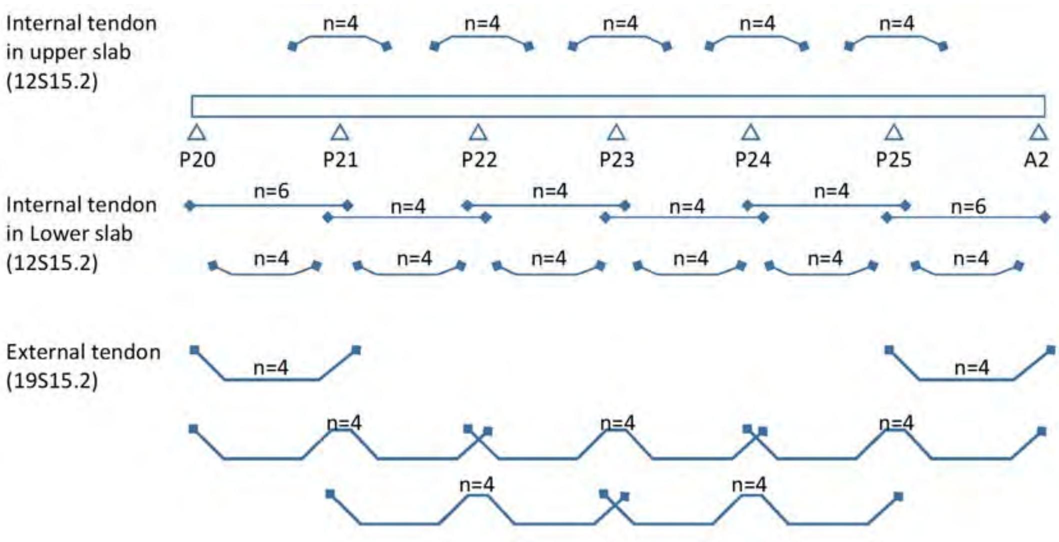
2) Longitudinal Tendon Arrangement (External and Internal Tendon)

a) A1-P5



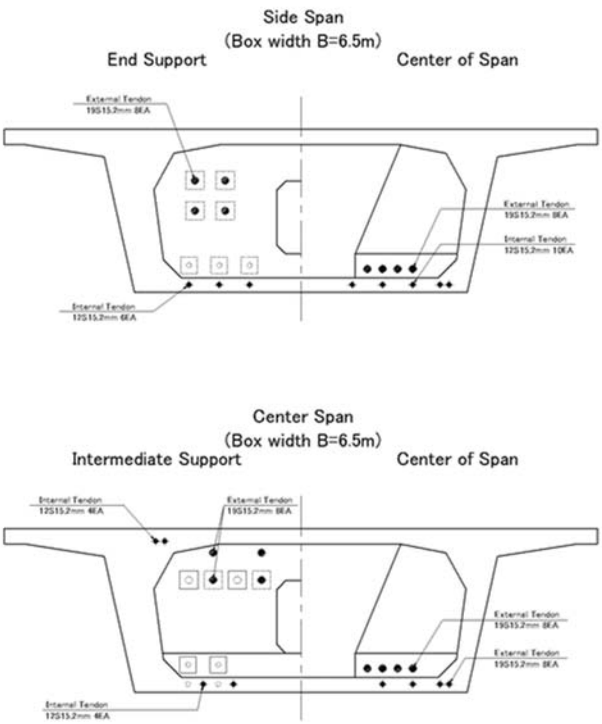
Source: JICA Study Team

b) P20-A2



Source: JICA Study Team

3) Standard Section (Box Width 6.5 m)



Source: JICA Study Team

Figure 4.4.14 Prestressing Tendon Arrangement (Standard Section, Box Width 6.5 m)

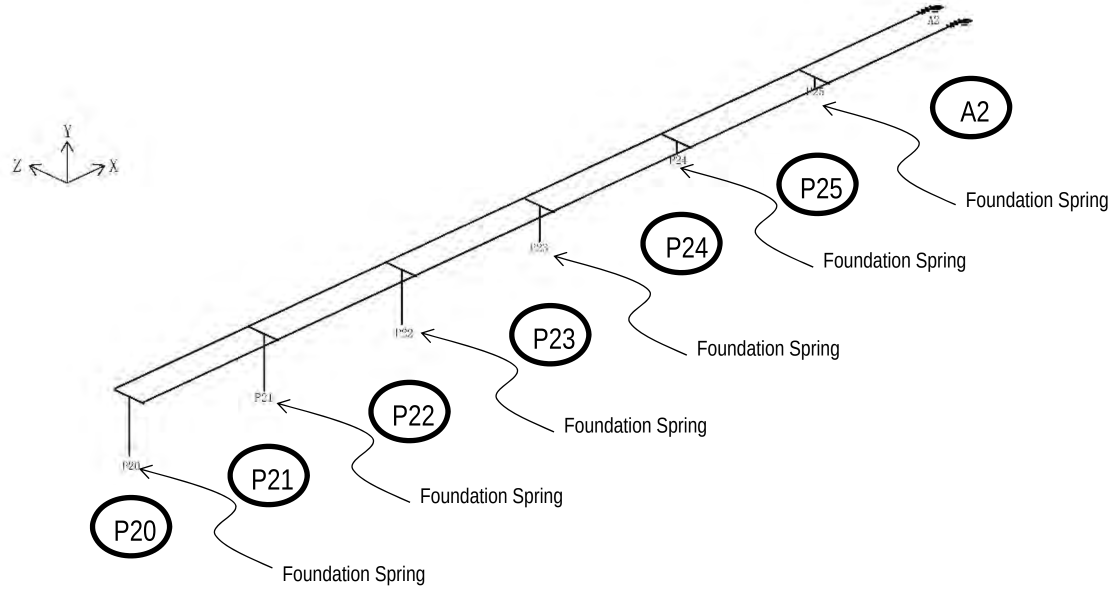
4.4.4.3 Global Analysis

(1) Analysis Models

In the global analysis of PC box girder bridges, two different analysis models were used for normal loads and seismic loads, respectively. For normal loads, sectional forces were calculated using plane frame models, and superstructure and substructures were analyzed separately. In this analysis, sectional forces were calculated considering construction steps of superstructure (span-by-span construction). For seismic actions, the analysis was performed using three-dimensional frame models, and distribution of seismic horizontal forces from superstructure acting on each substructure was calculated by the models in which superstructures and substructures are incorporated together.

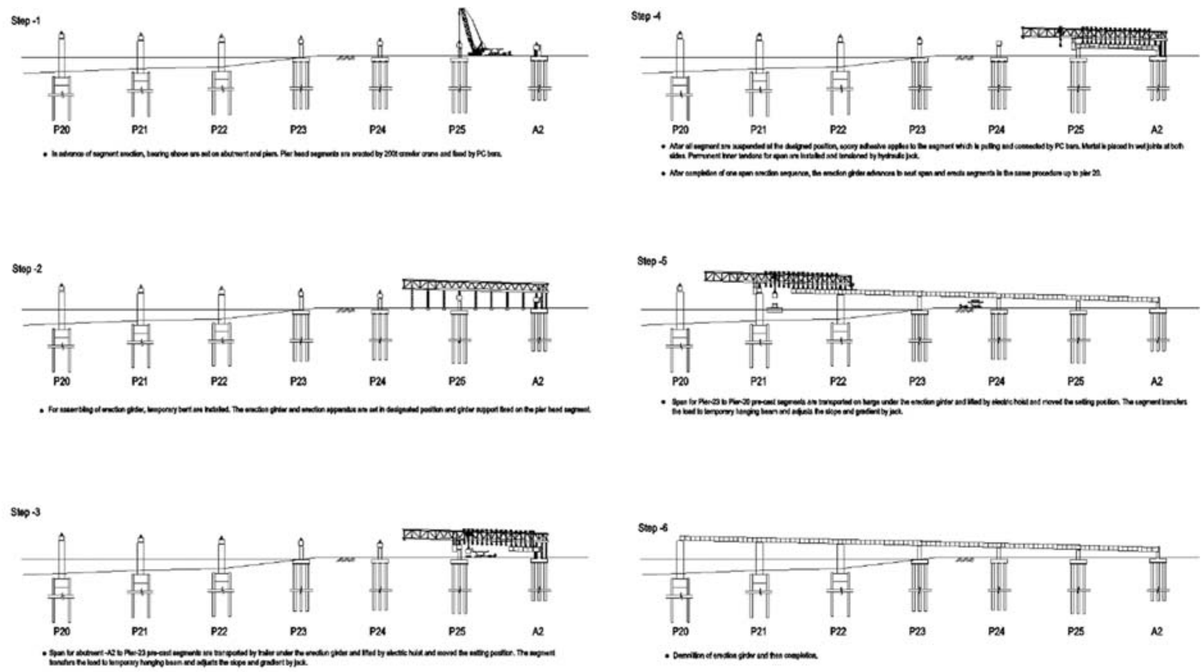
Source: JICA Study Team

Figure 4.4.15 Analysis Model for Normal Loads (A1~P5)



Source: JICA Study Team

Figure 4.4.16 Analysis Model for Seismic Action (P20~A2)



- The girders were assumed to be erected from abutment toward river, both at A1-P5 and P20-A2 section.
- Longitudinal tendons anchored at girder end at abutment were assumed to be tensioned on one side in the girder, and not tensioned at the abutment side (the other longitudinal tendons were assumed to be tensioned at both sides).
- Internal and external tendons in longitudinal direction were assumed to be tensioned at erection of each span.

Source: JICA Study Team

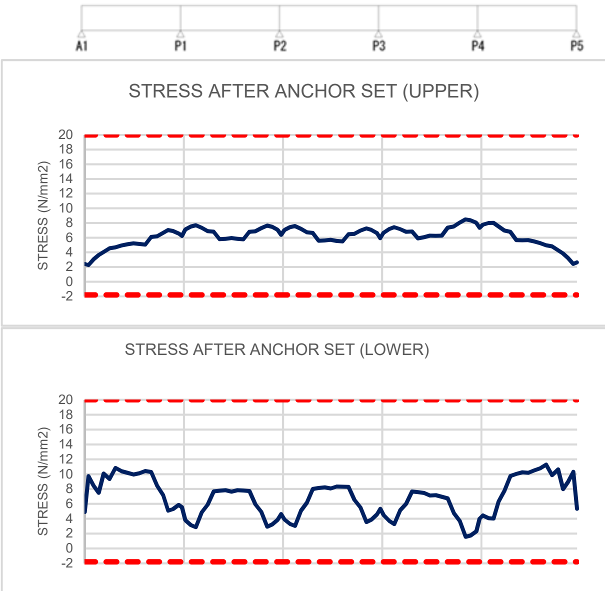
Figure 4.4.17 Assumption of Construction Sequence (P20~A2)

4.4.5 Summary of the Detailed Design Result for Superstructure

(1) A1~P5

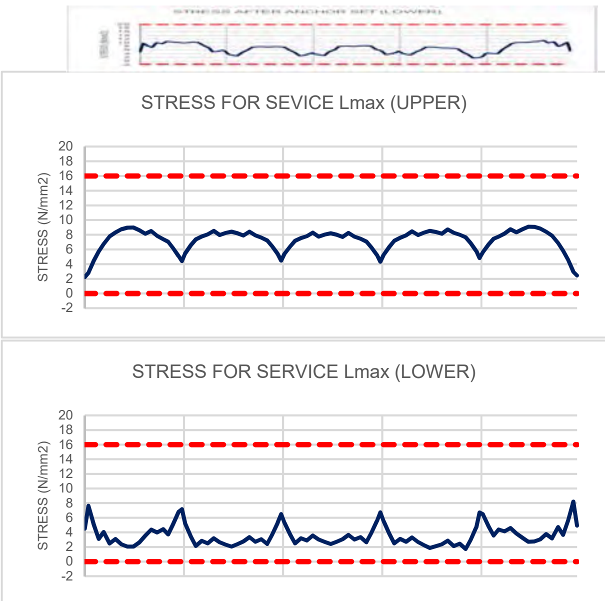
1) For Bending

Stress immediately after Anchor Set



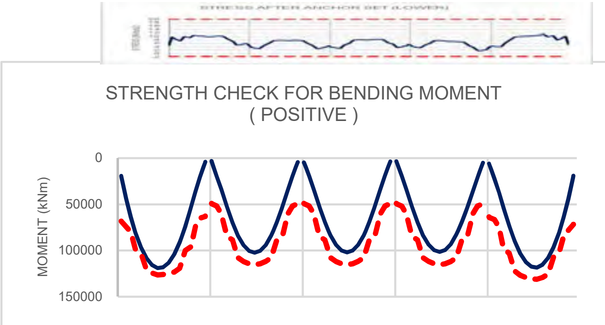
Stress for Permanent Load

Stress for Service Load (Lmax)

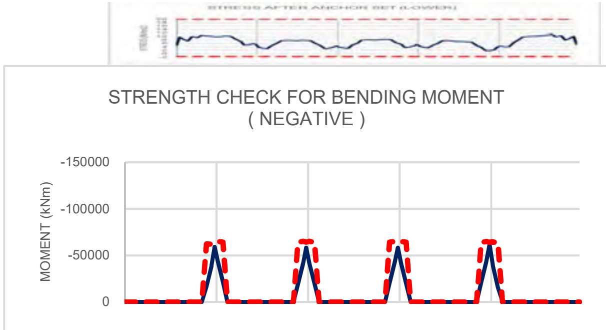


Stress for Service Load (Lmin)

Check for Ultimate Moment (Positive)



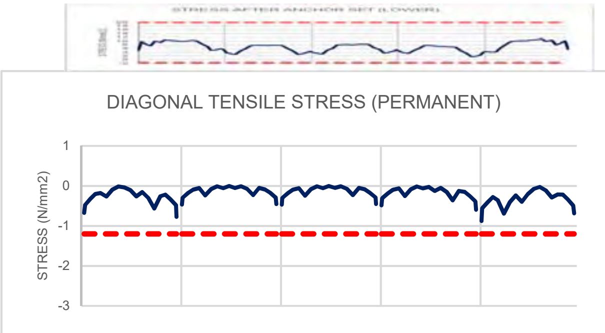
Check for Ultimate Moment (Negative)



Source: JICA Study Team

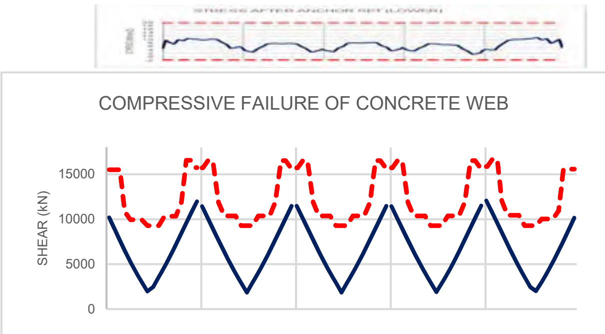
1) For Shear

Diagonal Tensile Stress (Permanent Load)

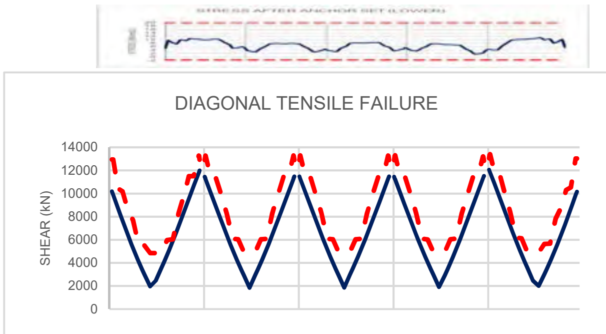


Diagonal Tensile Stress (Service Load)

Check for Compressive Failure of Concrete Web



Check for Diagonal Tensile Failure

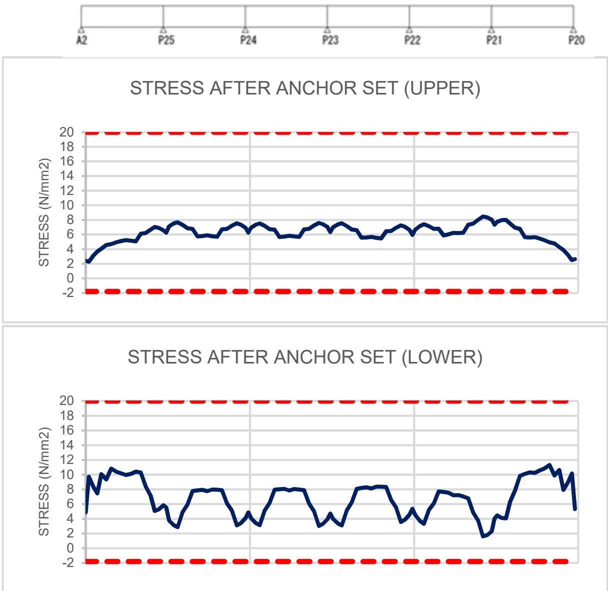


Source: JICA Study Team

(2) P20~A2

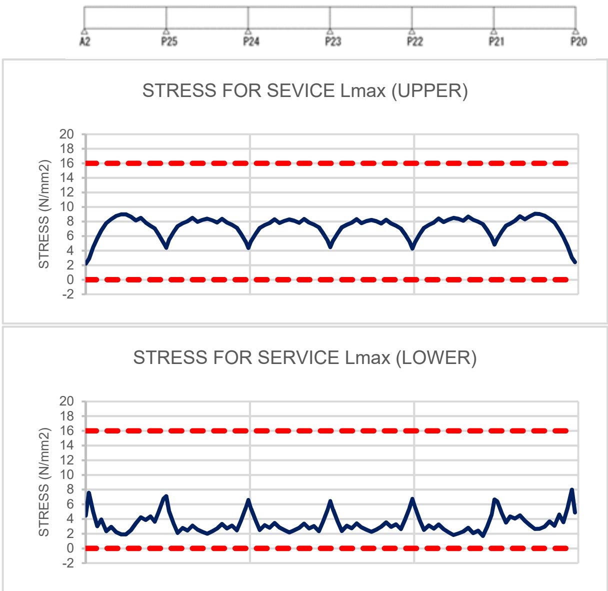
1) For Bending

Stress immediately after Anchor Set



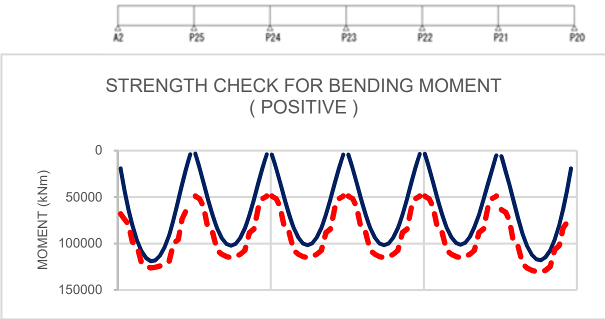
Stress for Permanent Load

Stress for Service Load (Lmax)

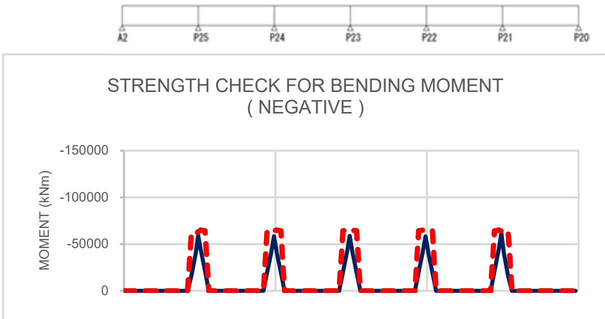


Stress for Service Load (Lmin)

Check for Ultimate Moment (Positive)



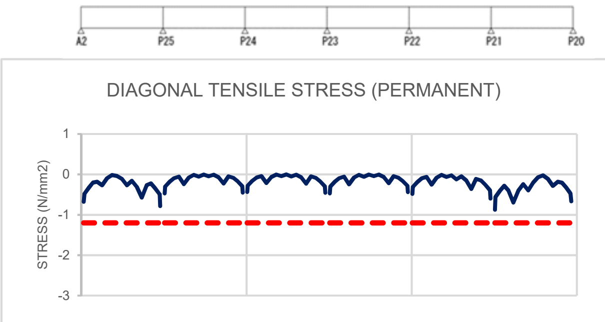
Check for Ultimate Moment (Negative)



Source: JICA Study Team

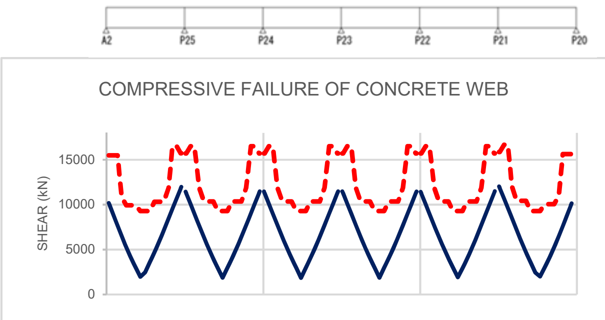
2) For Shear

Diagonal Tensile Stress (Permanent Load)

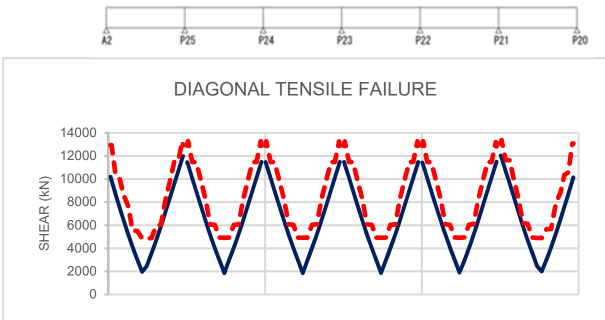


Diagonal Tensile Stress (Service Load)

Check for Compressive Failure of Concrete Web



Check for Diagonal Tensile Failure



Source: JICA Study Team

4.4.6 Substructure of PC Box Girder Bridge

4.4.6.1 Design Principle

Review of appropriateness of design outputs of the F/S was performed at the B/D for optimization of contents of facilities in relation to the bridge substructure with respect to structural types, dimensions, and number. Such optimizations were carried out with sufficient structural calculations, comparative studies in terms of economy, workability, constructability, and construction period.

Detailed structural analysis was performed at D/D for updating design conditions and upgrading analytical accuracy based on the B/D results. Major updates at D/D were natural conditions obtained from geographic survey and topographic survey, and proposed future ground elevation. Reaction forces of superstructure for design of substructure were also updated.

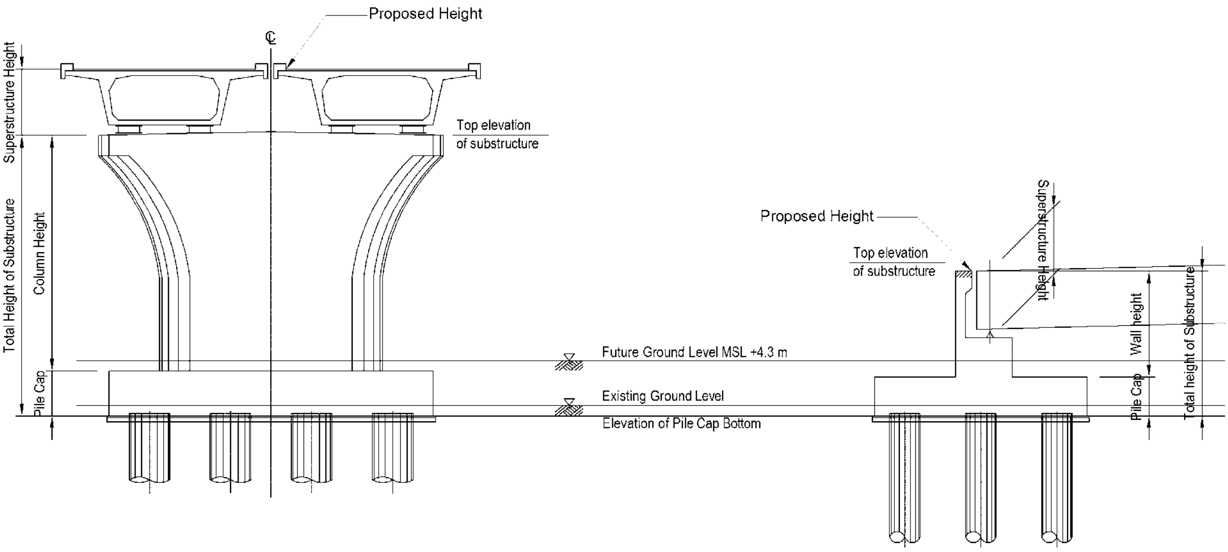
Also note that a notice was issued by DWIR during D/D stage regarding the navigation channel location of Bago River. In complying with the notice, the JICA Study Team reconsidered the span arrangement and bridge type, then decided to substitute a 3-span continuous steel box girder bridge for a 4-span PC box girder bridge. Due to this change, design of piers of P6 through P9 was omitted from the design scope of the PC concrete bridge.

4.4.6.2 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 4.4.18 Explanatory Diagram of Substructure Height

(2) Planned Depth of Overburden above Pile Cap and SPSP Top Slab

- On Land Substructures

The depth of overburden above the pile cap shall be secured sufficiently with regard to the planned future ground level. Amount of the overburden was around 0.5 m. The amount of overburden shall be altered at respective substructure locations in case of necessity such as an arrangement of buried conduit, and surface roads.

In the B/D, the amount of overburden was set to 1.0 m as a default plan subject to the update of the topography survey result, buried utility survey, and determination of future ground level at the D/D stage.

In D/D, it was confirmed that there would be no buried utilities around the substructures both now and in the future other than drainage lines to be constructed under this Project for discharge of rainwater on the bridge. Also, it was determined that future ground level would be MSL+4.300 m as mentioned in the foregoing. Thus, the amount of overburden was set to 0.5 m.

- In-river Substructures (P20~P22)

  The overburden depth of top slab of SPSP to riverbed shall be secured sufficiently for assurance of workability of grout injection work at permanent segment of SPSP connection pipes. Such elevation is determined as 1.0 m from the existing deepest riverbed around the target bridge section.

  In the B/D, elevation of top slab upper surface for P7~P9 and P20~P22 was set at MSL-8.4 m as default plan subject to the update of the bathymetric survey result and riverbed analysis result at D/D stage.

  In this D/D, the elevation of top slab upper surface for P20-P22 was set at MSL-7.900 m referring to the aforementioned bathymetric survey result. Note that piers of P7~P9 were omitted from the design scope of PC bridge based on the notice of DWIR

- In-river Substructures at Riverfront (P23)

  Riverbed levels at the riverfront, where P23 is planned, are much shallower than that of low-flow channel section. The elevation of top surface of pile cap should be set at a level which secures 0.5 m of overburden depth from the existing riverbed (MSL-0.50 m) or the Low Water of Ordinary Spring Tide (L.W.O.S.T = MSL-2.06 m) whichever is lower for assurance of aesthetic.

  Note that the P6 pier was omitted from the design scope of the PC bridge based on the notice of DWIR.

(3) Conclusion of Substructure Heights

Conclusions of the substructure heights are presented in Table 4.4.10 and Table 4.4.11.

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Table 4.4.10 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 4.4.11 Summary of Substructure Heights at A2 (Yangon) Side

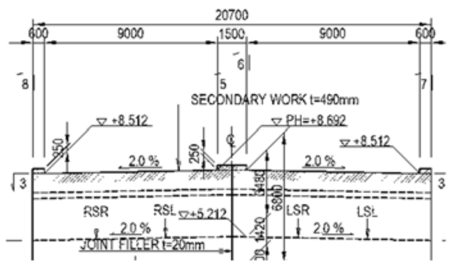
Item	Mark	Unit	P20	P21	P22	P23	P24	P25	A2
Station Number	STA	m	2088.00	2138.00	2188.00	2238.00	2288.00	2338.00	2388.00
Proposed height	PH	m	15.304	14.753	13.926	12.825	11.575	10.325	9.113
Top elevation of substructure	KCL	m	11.868	11.245	10.408	9.342	8.057	6.773	9.113
Existing Ground EL	GL1	m	-6.554	-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

4.4.6.3 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 4.4.19 Abutment Width

(2) Bridge Seat

The bridge seat shall have sufficient space for arrangement of bridge bearings that support the superstructure. The bridge shall also have a space that works to prevent the unseating of bridge in case of occurrence of unexpected seismic force, displacement or deformation occurring in a bridge caused by unpredicted earthquake ground motion in the design, destruction of the surrounding ground, or unexpectedly complicated vibration in the structural members.

As for unseating prevention in the bridge axis, the seating length is to be provided at the terminal supports and the halving joints. As for the unseating prevention in the transverse direction to the bridge axis, anchor bars are installed as displacement constraint structures.

- Determination of Seating Length (S_{EM})

The seating length should be long enough to prevent departure and unseating of the superstructure from the top of the substructure. Value of the seating was obtained from the equation below as specified in the JSBH.

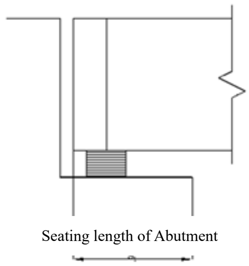
SEM = 0.7 + 0.005ℓ

Where,

ℓ: Length of the effective span (m). When two superstructures with different span length are supported on one bridge pier, the longer of the two shall be used. ℓ = 50.0 m for A1 and A2.

S_{EM} = 0.7 + 0.005 x 50.000

= 0.950 (m)



Source: JICA Study Team

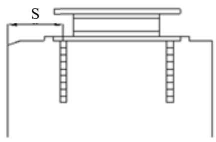
Figure 4.4.20 Unseating Length

- Determination of Bearing Edge Distance (S)

The bearing edge distance, which is defined as the distance between the edge of bearing and the edge of top of the substructure (or bearing support edge distance) shall be equal to or larger than the following value:

S ≧ 0.2 + 0.005ℓ

= 0.450 (m)



Source: JICA Study Team

Figure 4.4.21 Bearing Edge Distance

Check results of the bridge seat dimensions are summarized in Table 4.4.12 and

Table 4.4.13. The layout of the bridge seat is presented in Figure 4.4.22.

Table 4.4.12 Check Results of Bridge Seat Width (A1 Side)

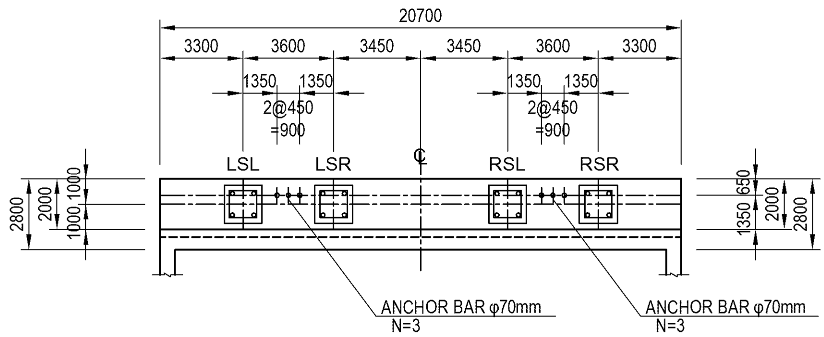
Unit : mm		A1	P1	P2	P3	P4	P5	
Span Length		50,000	50,000	50,000	50,000	50,000	50,000	76,500
Width of Bridge Seat		2,000	2,500	2,500	2,500	3,000	2,250	2,250
		20,700	17,000	17,000	17,000	17,000	25,000	25,000
Anchor Bolt (Fixing bolts of bearings)	φ	65	85	85	85	85	65	
	nos.	4	4	4	4	4	4	
	c.t.c.	850	1,100	1,100	1,100	1,100	850	
Anchor Bar (Displacement Constraint Structure)	φ	70	100	100	100	100	70	-
	nos.	3	4	4	4	4	3	-
	c.t.c.	450	300	300	300	300	450	-
Edge Distance from "Anchor Bolt"	LL	575	700	700	700	950	825	
	TT	2,875	900	900	900	900	1,025	
	Skew	813	598	598	598	1,102	1,117	
Minimum Edge Distance		**450**	**450**	**450**	**450**	**450**	**450**	**583**
Seating Length of Girder		1,750	-	-	-	-	2,050	
Minimum Seating Length of Girder		**950**	**950**	**950**	**950**	**950**	**950**	**1,083**

Source: JICA Study Team

Table 4.4.13 Check Results of Bridge Seat Width (A2 Side)

Unit : mm		P20		P21	P22	P23	P24	P25	A2
Span Length		104,000	50,000	50,000	50,000	50,000	50,000	50,000	50,000
Width of Bridge Seat		2,250	2,250	3,000	3,000	3,000	2,500	2,500	2,000
		17,000	17,000	17,000	17,000	17,000	17,000	17,000	20,700
Anchor Bolt (Fixing bolts of bearings)	φ	75	65	85	85	85	85	85	65
	nos.	4	4	4	4	4	4	4	4
	c.t.c.	1270x800	850	1,100	1,100	1,100	1,100	1,100	850
Anchor Bar (Displacement Constraint Structure)	φ	-	70	100	100	100	100	100	70
	nos.	-	3	4	4	4	4	4	3
	c.t.c.	-	450	300	300	300	300	300	450
Edge Distance from "Anchor Bolt"	LL	950	825	950	950	950	700	700	575
	TT	757	1,025	900	900	900	900	900	2,875
	Skew	1,018	1,117	1,102	1,102	1,102	598	598	813
Minimum Edge Distance		**720**	**450**	**450**	**450**	**450**	**450**	**450**	**450**
Seating Length of Girder		1,950	2,000	-	-	-	-	-	1,750
Minimum Seating Length of Girder		**1,220**	**950**	**950**	**950**	**950**	**950**	**950**	**950**

Source: JICA Study Team



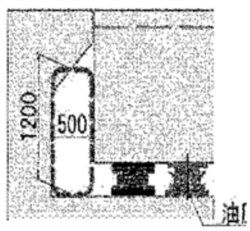
Source: JICA Study Team

Figure 4.4.22 Layout of Bridge Seat for Abutment (A1 and A2)

(3) Maintenance Space at Bridge Seat of Abutment

As for the structural details, which may contribute to prolongation of the bridge life span, bridge seats shall be graded by around 2% in order to avoid puddle on the bridge seat.

Moreover, a space for ventilation should be provided at the terminal support. For this purpose, a distance of 500 mm was secured. This space will also be utilized for inspections of bearings, and entrance path of PC box girder. Schematic illustration is shown in Figure 4.4.23.



Source: JSHB

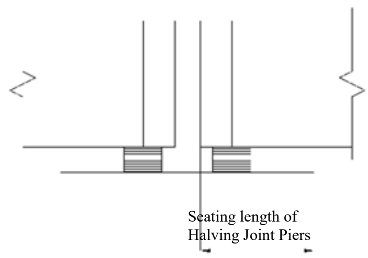
Figure 4.4.23 Maintenance Space at Bridge of Abutment

4.4.6.4 Dimensions of Pier

(1) Bridge Seat

The bridge seat shall have sufficient space for arrangement of bridge bearings that support superstructures. The bridge shall also have a space that works to prevent the unseating of bridge in case of occurrence of unexpected seismic force, displacement or deformation in a bridge caused by unpredicted earthquake ground motion in the design, destruction of the surrounding ground, or unexpectedly complicated vibration in the structural members.

As for unseating prevention in the bridge axis, the seating length is to be provided at the terminal supports and the halving joints. As for the unseating prevention in the transverse direction to the bridge axis, anchor bars are installed as displacement constraint structures.



Source:
JICA Study Team

Figure 4.4.24 Unseating Length

The seating length was estimated for piers of P5 and P20 that were terminal support piers. For other intermediate piers, such seating length is unnecessary because PC concrete bridge is continuous bridge and there is no unseating situation in the bridge axis.

- - Determination of Seating Length (S_{EM}) [P5 and P20]

The seating length should be long enough to prevent departure and unseating of the superstructure from the top of the substructure. The value of the seating was obtained from the equation below as specified in the JSBH.

SEM = 0.7 + 0.005ℓ

Where,

ℓ: Length of the effective span (m). When two superstructures with different span length are supported on one bridge pier, the longer of the two shall be used. The ℓ for P5 and P20 are 74.0 m and 104.0 m, respectively.

S_{EM} = 0.7 + 0.005 x [74(m) or 104 (m)]

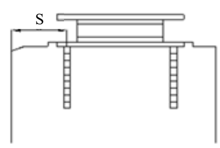
= 1.070 (m) [P5 Pier]

= 1.220 (m) [P20 Pier]

- Determination of Bearing Edge Distance (S)

The bearing edge distance, which is defined as the distance between the edge of the bearing and the edge of the top of the substructure (or bearing support edge distance) shall be equal to or larger than the following value:

S ≧ 0.2 + 0.005ℓ

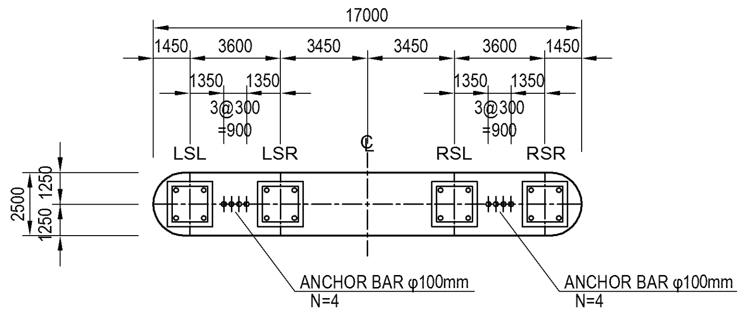


Source:
JICA Study Team

Figure 4.4.25 Bearing Edge Distance

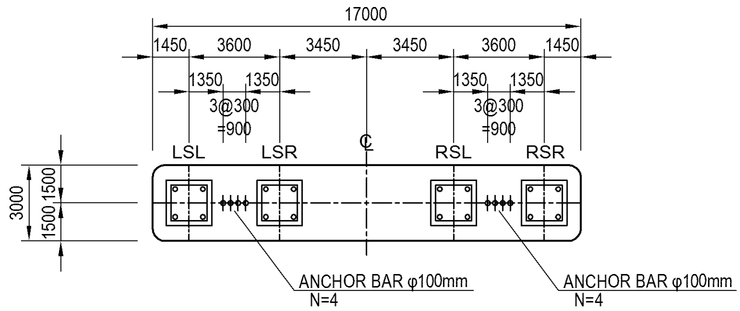
Check results of bridge seat dimensions are summarized in Table 4.4.12 and

Table 4.4.13. Layouts of bridge seat are displayed in Figure 4.4.26 through Figure 4.4.29.



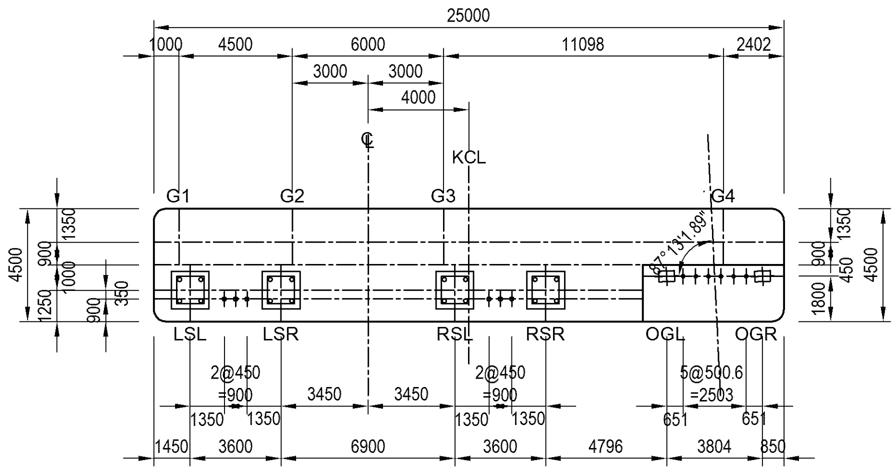
Source: JICA Study Team

Figure 4.4.26 Layout of Bridge Seat for P1~P3, P24 and P25



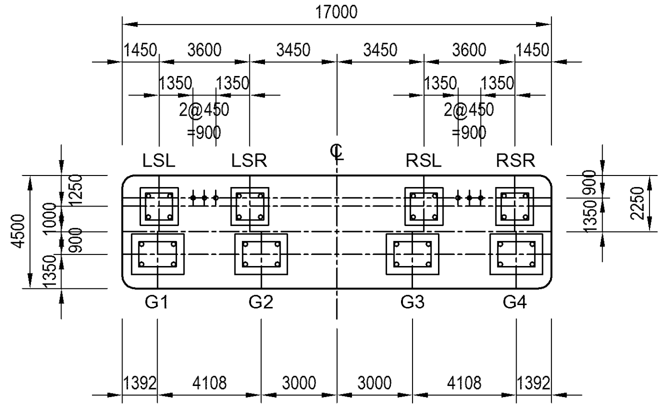
Source: JICA Study Team

Figure 4.4.27 Layout of Bridge Seat for P4, P21~P23



Source: JICA Study Team

Figure 4.4.28 Layout of Bridge Seat for P5



Source: JICA Study Team

Figure 4.4.29 Layout of Bridge Seat for P20

(2) Dimensions of Pier Column

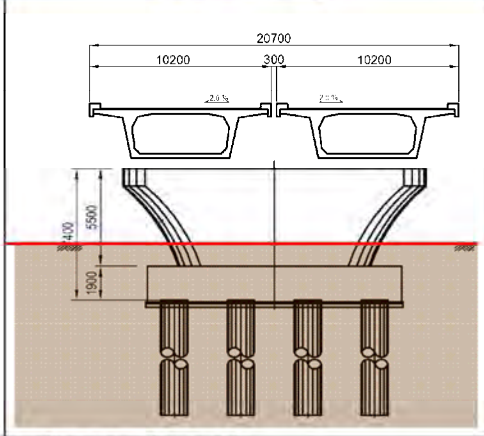
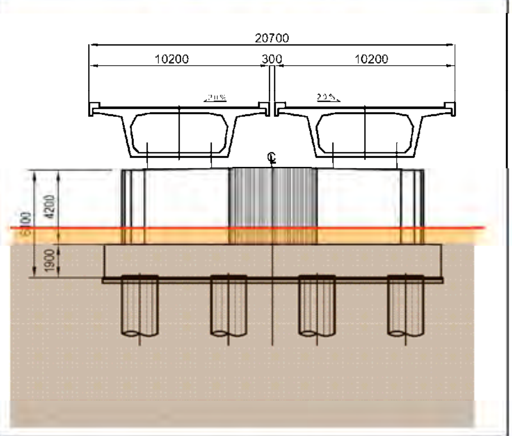
For the general concept of dimensions for piers adopted from the F/S and applied to all piers in the B/D, a ginkgo shape pier (*see Figure 4.4.30 Conceptual Diagram of "Ginkgo Shape"*) was employed. However, after due review of structural heights under the latest configurations during the D/D, it was confirmed that it is not rationale to adopt the gingko shape pier for some piers with relatively low height. In other words, those low height piers should not have overhang beams because of insufficient column height and such beams would just be buried on the ground despite using more reinforcement bars and timbering supports for their construction compared with a wall type column. Due to this, a wall type column was employed for piers of P1 through P3 at Thilawa side as well as P24 and P25 at Yangon side.


Source: Logo of Tokyo Metropolitan Government (Left) and Wikipedia (Right)

Figure 4.4.30 Conceptual Diagram of "Ginkgo Shape"

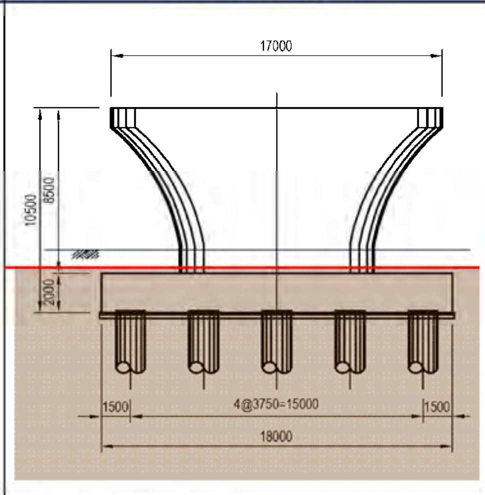
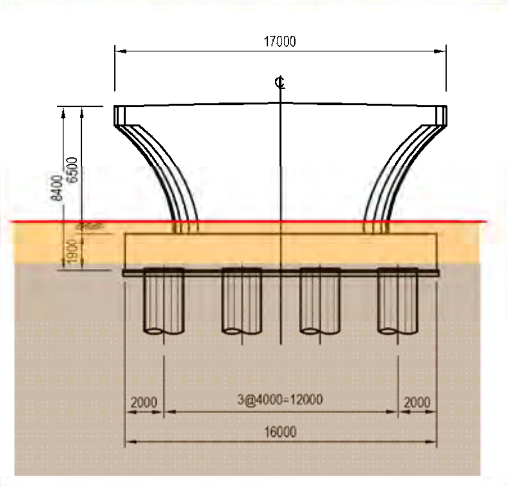
Regarding piers of P4, P5 and P20 through P23, which have reasonable heights for construction of beams on the column, the ginkgo shape pier was employed as selected during B/D. Comparisons are shown in Table 4.4.14 and Table 4.4.15.

Table 4.4.14 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 4.4.15 General Shapes of Piers with Overhang Beam for P4, P5, P20~P23 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 1.8m 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, <u>pile diameter of 2.0m is selected.</u>	

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 4.4.16.

Table 4.4.16 Summary of Basis of Determination of Cross Sectional Dimensions

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

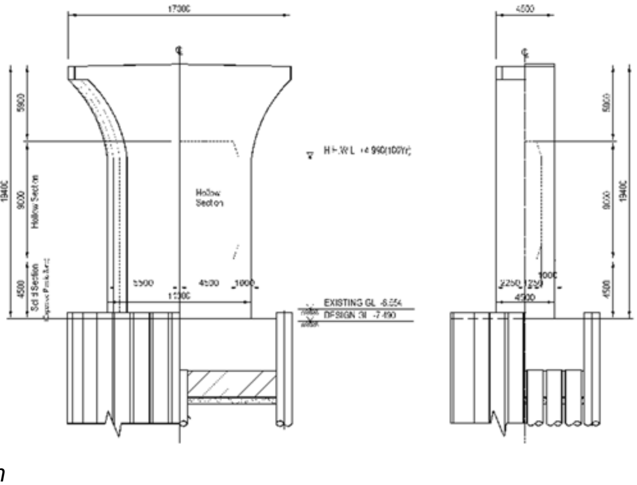
Source: JICA Study Team

(3) Study on Applicability of Hollow Section Column

When a high pier with a pier column having a large cross sectional area is required, a hollow section column may be suitable instead of a solid section column due to reduction of inertial force rooted in the mass of the pier column which may facilitate seismic design of substructure and foundation. Also, reduction of concrete volume to be used may provide cost benefits.

Looking at the substructures of PC box girder bridges, the maximum column height is 19.4 m at P20 pier, which is not classified as a high pier in general. However, its cross sectional dimension in bridge axis is 4.5 m that is sufficient for construction of a hollow section column (an assumed minimum inner dimension for construction is 2.0 m). Therefore, a study on applicability of a hollow section column was performed at the D/D.

A model used in this study is shown in Figure 4.4.31. Dimensions of the hollow section are (H)9.0 m x (B)2.5 m x (W)9.0 m. The bottom elevation of the hollow section was determined based on the height of an assumed plastic zone, where the column shall have solid section as stipulated in the JSBH. The upper elevation of the hollow section was determined with respect to rebar arrangement of overhang beam.

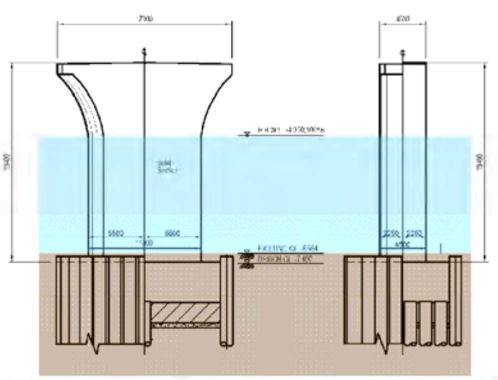
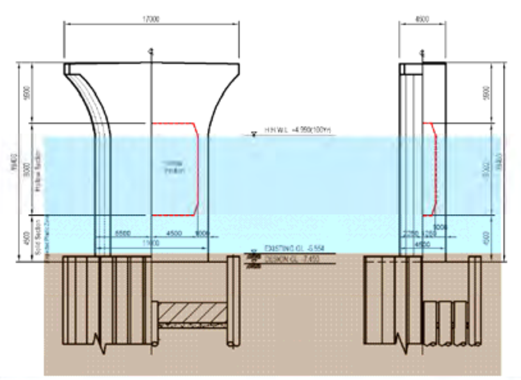


Source: JICA Study Team

Figure 4.4.31 Schematic Diagram of Hollow Section Column Pier (P20)

This study revealed that there were little benefits in terms of cost (cost ratio of 0.99 against the solid column type) whereas there were disadvantages in terms of workability for construction and maintenance. Specifically, for the cost aspect, concrete volume could be reduced but quantity of formwork and falsework for inner space construction and axial rebar of column will be increased. It is also possible that a narrow space construction has various difficulties and may cause a longer construction duration. Also, such narrow space may toughen maintenance and rehabilitation of column in the future. Overall, it was concluded that a hollow sectional column was not applicable to the PC box girder bridge section. Comparison result is summarized in Table 4.4.17

Table 4.4.17 Comparison of Solid Section Column and Hollow Section Column

Evaluation Item	Solid Cross Section Column	Hollow Cross Section Column
Schematic View		
Evaluation	• **Construction Cost:** Concrete volume of a hollow column can be reduced by around 200m3 comparing with a solid column. However, quantities of formwork and falsework for inner space are additionally required for the hollow column. Moreover, larger diameter of axil rebar of column is necessary for the hollow column. Thus, cost benefit of the hollow column is minor. (99% comparing with the solid column type) • **Construction Workability:** Narrow space construction is required for the hollow column thus various difficulties cause a longer construction duration. • **Maintenance Workability:** Maintenance of inner surface is difficult. Moreover, a rehabilitation of column, if required, is also difficult. • **Conclusion:** Solid Type Pier Column is recommended.	

Source: JICA Study Team

4.4.7 Foundation of PC Box Girder Bridge

4.4.7.1 Design Principle

Review of appropriateness of design outputs in the F/S was performed in the B/D for optimization of contents of facilities in relation to the bridge foundation with respect to structural types, dimensions, and number. Such optimizations were carried out with sufficient structural calculations and comparative studies in terms of economy, workability, constructability, and construction period.

Detailed structural analysis was performed at the D/D for updating design conditions and upgrading analytical accuracy based on the B/D results. Major updates at the D/D were natural conditions obtained from geographic survey and topographic survey, and proposed future ground elevation. Reaction forces of superstructure for design of substructure were also updated.

4.4.7.2 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 4.4.32 and Figure 4.4.33.

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

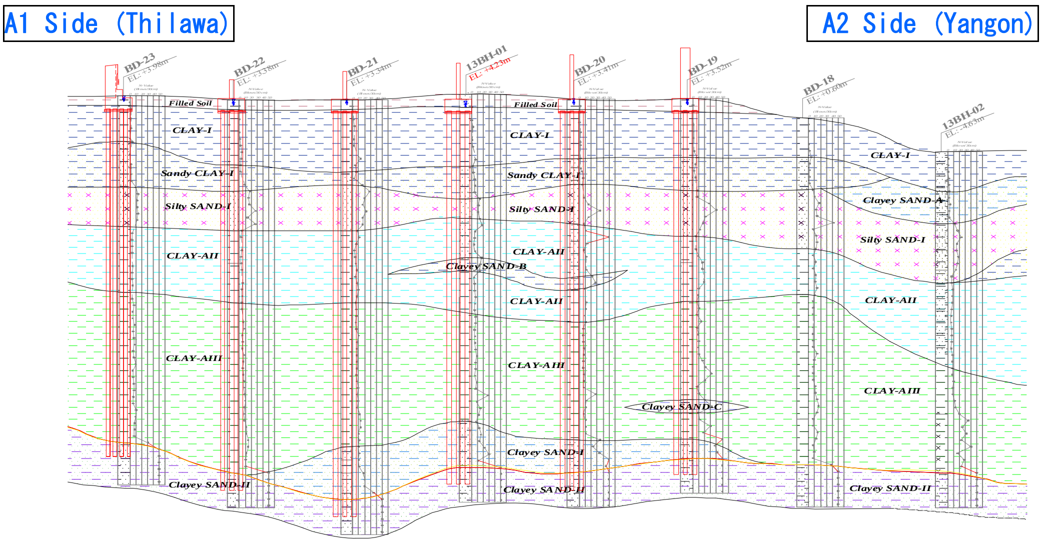
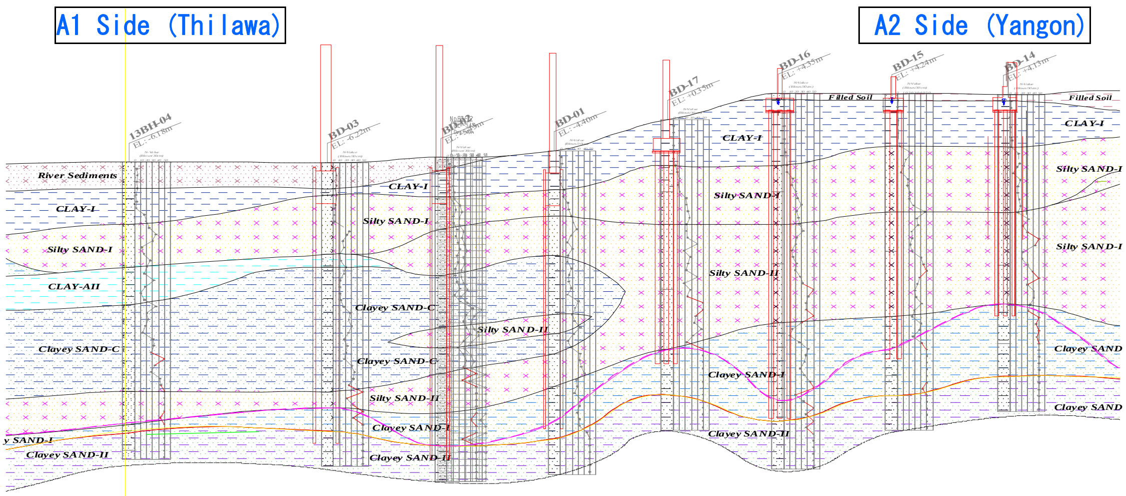


Figure 4.4.32 Prospected Soil Profile and Bearing Stratum (A1 Side)



Source: JICA Study Team

Figure 4.4.33 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 4.4.18 and Table 4.4.19.

Table 4.4.18 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 4.4.19 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

4.4.7.3 Evaluation of Geotechnical Parameters for Design

Evaluation of geotechnical investigation results was performed by the geological specialists of the Project with unified viewpoint and described in detail at the relevant section of this report. With regard to evaluation of geotechnical parameters for bridge design, bridge designers should take into account the specific features of each bridge locations in deference to outputs of the geological specialists. In this sub-section, modulus of deformation of soils and reduction factor (D_E) for geotechnical parameters due to liquefaction, that have a profound effect on design of bridge foundation, are reported.

Generally, the displacement of foundations largely depends on the behavior of the weak sections of the bearing ground. In addition, the horizontal displacement of foundations with respect to the loads acting on the top of the foundations varies with the properties of the surface layers. According to the geotechnical investigation results that had been conducted during B/D, the modulus of deformation of soils shall be reduced as an overall tendency comparing with ones obtained in F/S and used in B/D. It is remarked that the modulus of surface layers should be decreased approximately by 50% based on the results. Additionally, it was discovered that the effects of liquefaction should be considered more seriously than that considered in the F/S and the B/D. Since a single soil layer at A2 (Yangon) side was considered as a liquefaction layer in the F/S-B/D, number of soil layers and scale of reduction for geotechnical parameters due to liquefaction should be increased in the D/D. The modulus of deformation of soils and reduction factor (D_E) are shown in Table 4.4.20 and Table 4.4.21.

Above all, it was confirmed that the properties of the surface layers for deformation were weaker than that of the F/S-B/D, and the fact brought increments of pile number and/or pile diameters for assurance of structural stability.

Table 4.4.20 Comparison of Modulus of Deformation of Soil for A1 (Thilawa) Side (B/D vs D/D)

Str. No.	A1			P1			P2			P3			P4			P5		
Br No.	BD-23			BD-22			BD-21			No13BH-01			BD-20			BD-19		
Depth	Soil type	BD	DD	Soil type	BD	DD	Soil type	BD	DD	Soil type	BD	DD	Soil type	BD	DD	Soil type	BD	DD
1	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A
2																		
3	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A
4																		
5																		
6																		
7																		D_E=2/3
8	Sandy CLAY-I	N=5 E=2500 D_E=N/A	N=3 E=2000 D_E=1/3			D_E=2/3			D_E=2/3				Sandy CLAY-I	N=5 E=2500 DE=N/A	N=3 E=2000 D_E=1/3			
9																Sandy CLAY-I	N=5 E=2500 D_E=N/A	N=3 E=2000 D_E=2/3
10				Sandy CLAY-I	N=5 E=2500 D_E=N/A	N=3 E=2000 D_E=N/A			D_E=1						D_E=2/3			
11			D_E=2/3				Sandy CLAY-I	N=5 E=2500 D_E=N/A	N=3 E=2000 D_E=1	SandyCLAY-I	N=5 E=2500 D_E=N/A	N=3 E=2000 D_E=N/A			D_E=1			D_E=1
12																		
13							Silty SAND-I	N=14 E=9800 D_E=1	N=15 E=6000 DE=1				Silty SAND-I	N=14 E=9800 D_E=1	N=15 E=6000 D_E=1	Silty SAND-I	N=14 E=9800 D_E=1	N=15 E=6000 DE=1
14	Silty SAND-I	N=14 E=9800 D_E=1	N=15 E=6000 D_E=1	Silty SAND-I	N=14 E=9800 D_E=1	N=15 E=6000 D_E=1												
15																		
16																		
17										Silty SAND-1	N=14 E=9800 D_E=1	N=15 E=6000 D_E=1						
18																		
19																		
20										SaCL-II	D_E=N/A	D_E=N/A	CL-AII	D_E=N/A	D_E=N/A			

Source: JICA Study Team

Table 4.4.21 Comparison of Modulus of Deformation of Soil for A2 (Yangon) Side (B/D vs D/D)

<table>
<tr><th>Str. No.</th><th colspan="3">P20</th><th colspan="3">P21</th><th colspan="3">P22</th><th colspan="3">P23</th><th colspan="3">P24</th><th colspan="3">P25</th><th colspan="3">A2</th></tr>
<tr><th>Br No.</th><th colspan="3">BD-3</th><th colspan="3">BD-2</th><th colspan="3">BD-1</th><th colspan="3">BD-17</th><th colspan="3">BD-16</th><th colspan="3">BD-15</th><th colspan="3">BD-14</th></tr>
<tr><th>Depth</th><th>Soil type</th><th>BD</th><th>DD</th><th>Soil type</th><th>BD</th><th>DD</th><th>Soil type</th><th>BD</th><th>DD</th><th>Soil type</th><th>BD</th><th>DD</th><th>Soil type</th><th>BD</th><th>DD</th><th>Soil type</th><th>BD</th><th>DD</th><th>Soil type</th><th>BD</th><th>DD</th></tr>
<tr><td>1</td><td rowspan="4">River sediments</td><td rowspan="4">N=4
E=2800
D_E=N/A</td><td rowspan="4">N=3
E=**1200**
D_E=**1/3**</td><td>R.S.</td><td>E=2800, D_E=N/A</td><td>E=**1200**, D_E=**1/3**</td><td rowspan="5">CLAY-I</td><td rowspan="5">N=4
E=2800
D_E=N/A</td><td rowspan="5">N=1
E=**900**
D_E=**1/3**</td><td rowspan="6">CLAY-I</td><td rowspan="6">N=1
E=1800
D_E=N/A</td><td rowspan="6">N=1
E=**900**
D_E=**1/3**</td><td>F.S</td><td>E=2000, D_E=N/A</td><td>E=2100, D_E=N/A</td><td>F.S</td><td>E=2000, D_E=N/A</td><td>E=2100, D_E=N/A</td><td>F.S</td><td>E=2000, D_E=N/A</td><td>E=2100, D_E=N/A</td></tr>
<tr><td>2</td><td rowspan="4">CLAY-I</td><td rowspan="4">N=4
E=2800
D_E=N/A</td><td rowspan="4">N=1
E=**900**
D_E=2/3</td><td rowspan="9">CLAY-I</td><td rowspan="9">N=1
E=1800
D_E=N/A</td><td rowspan="9">N=1
E=**900**
D_E=N/A</td><td rowspan="5">CLAY-I</td><td rowspan="5">N=1
E=1800
D_E=N/A</td><td rowspan="5">N=1
E=**900**
D_E=N/A</td><td rowspan="7">CLAY-I</td><td rowspan="7">N=1
E=1800
D_E=N/A</td><td rowspan="5">N=1
E=**900**
D_E=N/A</td></tr>
<tr><td>3</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td><td>CL-I</td><td>E=2800, D_E=N/A</td><td>E=**900**, D_E=N/A</td></tr>
<tr><td>6</td><td rowspan="8">SiltySAND-I</td><td rowspan="8">N=5
E=2500
D_E=2/3</td><td rowspan="5">N=13
E=**5200**
D_E=**1/3**</td><td rowspan="12">SiltySAND-I</td><td rowspan="12">N=5
E=2500
D_E=2/3</td><td rowspan="5">N=13
E=5200
D_E=2/3</td><td rowspan="4">Silty SAND-I</td><td rowspan="4">N=5
E=2500
D_E=2/3</td><td rowspan="4">N=13
E=5200
D_E=2/3</td></tr>
<tr><td>7</td><td rowspan="9">SiltySAND-I</td><td rowspan="9">N=5
E=2500
D_E=2/3</td><td rowspan="4">N=13
E=6500
D_E=2/3</td><td rowspan="11">SiltySAND-I</td><td rowspan="11">N=5
E=2500
DE=2/3</td><td rowspan="11">N=13
E=6500
D_E=1</td><td>D_E=1</td></tr>
<tr><td>8</td><td>D_E=**2/3**</td></tr>
<tr><td>9</td><td rowspan="9">SiltySAND-I</td><td rowspan="9">N=5
E=2500
D_E=2/3</td><td rowspan="9">N=13
E=6500
D_E=1</td></tr>
<tr><td>10</td><td rowspan="6">SiltySAND-II</td><td rowspan="6">N=5
E=2500
D_E=2/3</td><td rowspan="6">N=30
E=17500
D_E=1</td></tr>
<tr><td>11</td><td rowspan="3">D_E=2/3</td><td rowspan="7">D_E=1</td><td rowspan="5">D_E=1</td><td rowspan="10">SiltySAND-I</td><td rowspan="10">N=5
E=2500
D_E=2/3</td><td rowspan="10">N=13
E=6500
D_E=1</td></tr>
<tr><td>12</td></tr>
<tr><td>13</td></tr>
<tr><td>14</td><td rowspan="2">CL-AII</td><td rowspan="2">N=5
E=2500
D_E=2/3</td><td rowspan="2">N=7
E=4900
D_E=N/A</td></tr>
<tr><td>15</td></tr>
<tr><td>16</td><td rowspan="5">ClayeySAND-C</td><td rowspan="5">N=30
E=21000
D_E=1</td><td rowspan="5">N=20
E=**14000**
D_E=1</td><td rowspan="5">ClayeySAND-C</td><td rowspan="5">N=30
E=21000
D_E=1</td><td rowspan="5">N=20
E=**14000**
D_E=1</td><td rowspan="5">SiltySAND-II</td><td rowspan="5">N=30
E=21000
D_E=1</td><td rowspan="5">N=25
E=**17500**
D_E=1</td></tr>
<tr><td>17</td></tr>
<tr><td>18</td><td rowspan="3">Clayey SAND-C</td><td rowspan="3">N=30
E=21000
D_E=1</td><td rowspan="3">N=20
E=**14000**
D_E=1</td><td rowspan="3">Silty SAND-II</td><td rowspan="3">N=30
E=21000
D_E=1</td><td rowspan="3">N=25
E=**17500**
D_E=1</td><td rowspan="3">Silty SAND-II</td><td rowspan="3">N=30
E=21000
D_E=1</td><td rowspan="3">N=25
E=**17500**
D_E=1</td></tr>
<tr><td>19</td></tr>
<tr><td>20</td></tr>
</table>

Source: JICA Study Team

4.4.7.4 Estimation of Down Drag Zone

Occurrence of down drag due to reclamation for preparation of construction yard whose finished elevation is MSL+4.300 m was anticipated. Accordingly, depth of the down drag zone was analyzed using laboratory test results of soil samples obtained at D/D. Analysis results shown in Table 4.4.22 were utilized for the design of foundation.

Table 4.4.22 Assessment Result of Down Drag

1. A1 (Thilawa) Side: PC-Box Girder Bridge

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
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**
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**
Downdrag Zone	EL	m	**-10.0**	**-10.6**	**-9.7**	**-11.8**	**-9.6**	**-10.5**
Reference Boring No.	-	-	*BD23*	*BD22*	*BD21*	*BH-01*	*BD20*	*BD19*
Bearing Stratum	-	-	*CS-II*	*CS-II*	*CS-II*	*CS-II*	*CS-II*	*CS-II*

2. A2 (Yangon) Side: PC-Box Girder Bridge

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
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**
Foundation Type	-	-	SPSP	SPSP	SPSP	CIP Pile	CIP Pile	CIP Pile	CIP Pile
Pile Length	L	m				**32.500**	**47.000**	**38.000**	**31.500**
Downdrag Zone	EL	m				**-6.7**	**-6.7**	**-3.8**	**-3.9**
Reference Boring No.	-	-	*BD3*	*BD2*	*BD1*	*BD17*	*BD16*	*BD15*	*BD14*
Bearing Stratum	-	-	*CS-I*	*CS-I*	*CS-I*	*CS-I*	*CS-I*	*CS-I*	*CS-I*

Source: JICA Study Team

4.4.7.5 Selection of Foundation Type

(1) Design Policy

Bridge foundation type of Bago Bridge should be selected considering the temporary structures to be required at the respective substructure locations due to site conditions. Thus, comparative studies for the selection of foundation type should be conducted for on-land and in-river sections separately. Additionally, a study for the in-river section should consist of a low-flow channel and a waterfront section, where riverbed level is much shallower than that of the low-flow channel.

As the result of comparative study at B/D, the steel pile sheet pile (SPSP) foundation cum cofferdam was recommended as foundation type of in-river substructures, and a combination of cast-in-place (CIP) pile foundation (pile diameter = 1.500 m) and steel sheet pile cofferdam was recommended for on-land and waterfront substructures.

Updating of design conditions and upgrading of analytical accuracy were made to the above B/D results. Then, when necessary, reconsideration of pile arrangement including selection of pile diameter was performed at D/D for optimization.

(2) Design Conditions

Design conditions applied for the B/D are summarized as follows:

- Bearing stratum is Clayey SAND-II at around MSL-50.0 m
- Design reaction from superstructure is moderate to large due to superstructure type (concrete box girder) and span length (50.0 m)

- Groundwater level is high (cross to ground surface)
- Representative substructure height of pier employed for this study is 8 m and 20 m for on-land pier and in-river pier, respectively
- Representative substructure height of abutment employed for this study is 8 m
- Unit costs of bridgework employed for this study are reconfigured based on those used in the F/S.

(3) Selection of Foundation Type at B/D

1) B/D Result: On-land (A1～P5, P24～A2)

On-land foundations are constructed after ground preparation that includes reclamation.

Regarding prefabricated concrete pipe pile types, such as PHC (Pretensioned Spun High Strength Concrete) pile, diameters of 600 mm or smaller can be procured in Myanmar. However, these diameters are too small against bridge scale and seismic force. Moreover, only percussion method is procurable in Myanmar whereas inner excavation method is demanded in terms of the required driving depth and penetration against a relatively firm intermediate sandy soil layer. In order to overcome these situations, an offshore procurement of large diameter PHC piles with inner excavation drilling machines is one of the options, but less economical than adoption of CIP pile that is locally procurable. As mentioned above, prefabricated concrete pipe pile types were excluded from this comparative study of foundation type.

Regarding steel pipe pile foundation, those with diameters of 600 mm or smaller are procurable in Myanmar. However, these diameters are also too small against bridge scale and seismic force. Moreover, only percussion method is procurable in Myanmar whereas inner excavation method is demanded in terms of the required driving depth and penetration against a relatively firm intermediate sandy soil layer. In order to overcome these situations, an offshore procurement of large diameter steel piles with inner excavation drilling machines is one of the options similar to the PHC pile case, but less economical than adoption of CIP pile that is locally procurable. As mentioned above, steel pipe pile foundation was excluded from the comparative study of foundation type at B/D.

About the CIP pile foundation, reverse circulation drilling method is suitable for the required borehole depth. Based on bridge scale and seismic force, pile diameters for comparative study were 1.2 m, 1.5 m, and 2.0 m. Procurement and construction plan should be referred to relevant chapters of this report. It was confirmed as the result of study that “CIP pile foundation (reverse circulation drilling method) D = 1.5 m” was the most advantageous foundation type among the alternatives in terms of economy, workability, and construction period. This foundation type was adopted for abutments and piers (on-land). Summary of these comparative studies is shown in Table 4.4.23 and Table 4.4.24.

2) B/D Result: In-river (Low-flow channel) (P5～P9, P20～P22)

Exposure to water surface of bridge structure other than pier column has not been permitted by a relevant authority. Consequently, members of foundation such as steel pipe sheet piles and top slab shall be constructed below a certain riverbed elevation, and a temporary cofferdam is necessary for their construction. Because of a very high design water head for cofferdam that is around 15 m, SPSP cofferdam is the best structural type that will enable to resist such high water pressure. The steel sheet pile method, which is a conventional cofferdam type, is not suitable for the water head.

When the pile type foundation is required due to depth of bearing stratum and when SPSP cofferdam is selected as an economical and feasible cofferdam type, SPSP foundation cum cofferdam is commonly adopted.

Based on above considerations, “SPSP Foundation cum cofferdam D = 1.2 m” was the most advantageous foundation type among the alternatives in terms of economy, workability, and construction period. This foundation type was adopted for in-river piers (low-flow channel) at B/D. Summary of the comparative studies is shown in Table 4.4.25.

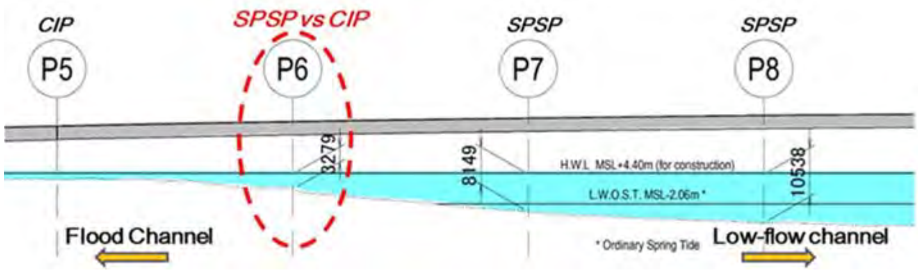
3) B/D Result: In-river (Riverfront) (P6 and P23)

Exposure to water surface of bridge structure other than pier column has not been permitted by a relevant authority. Consequently, members of foundation such as steel pipe sheet piles and top slab or pile cap for CIP pile foundation shall be constructed below a certain riverbed elevation, and a temporary cofferdam is necessary for their construction.

Because of the relatively shallower riverbed level at riverfront pier locations, the design water head for cofferdam is set at around 10 m which is applicable for a steel sheet pile cofferdam. In this study, alternatives were as follows:

Alternative 1: SPSP foundation cum cofferdam D = 1.2 m

Alternative 2: CIP pile foundation (reverse circulation drilling method) D = 1.5 m



Source: JICA Study Team

Figure 4.4.34 Comparison of Riverbed Depth

It was confirmed that Alternative 2: “CIP pile foundation (reverse circulation drilling method) D = 1.5 m” was the most advantageous foundation type for the waterfront pier in terms of economy, workability, and construction period. This foundation type was adopted for waterfront piers (P6 and P23). Summary of the comparative studies is shown in Table 4.4.26.

Note: Based on a notice of DWIR made during the D/D stage regarding the navigation channel location of Bago River, location of piers P6 through P9 had been rearranged or omitted, and design of the in-river piers at Thilawa side was deleted from the scope of PC concrete bridge after due consideration of the JICA Study Team. However, for the purpose of design activity log above, illustration is still used. In addition, the result of the comparative study on foundation type of riverfront piers is still available to explain the adequateness of P23 foundation type, this illustration is used in D/D report.

(4) Review Policy of Foundation Type at the D/D

Update of design conditions and upgrade of analytical accuracy were conducted as mentioned in the design principle.

Soil parameter	To apply updated soil parameters that was obtained after the B/D. It was revealed that deformation coefficients of soils were smaller than the ones used in the B/D. Additionally, the scale of liquefaction effect for foundation design is larger than the expected at the B/D.
Ground elevation and structural height	The amount of the overburden depth is 0.5 m at the D/D, whereas it was 1.0 m at the B/D. The foundation level of on-land substructures was determined, taking into account the ground level for construction (MSL+4.300 m). Due to those changes, structural heights were shortened by 1.0~1.5 m
Analytical accuracy	Implementation of global analysis under the updated conditions and improvement of analytical accuracy adequately as D/D.

(5) Review Results

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 4.4.27 and Table 4.4.28.

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 4.4.29.

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

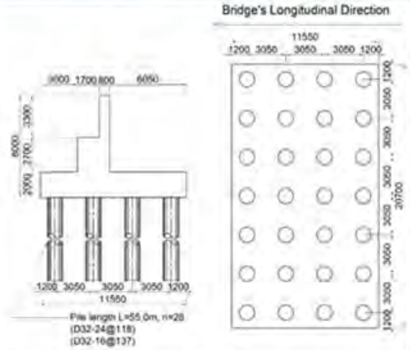
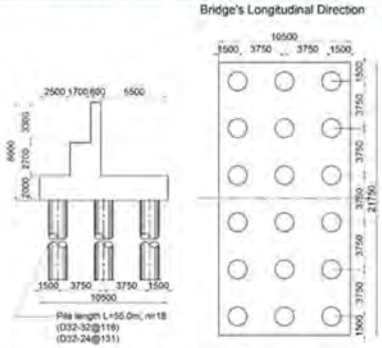
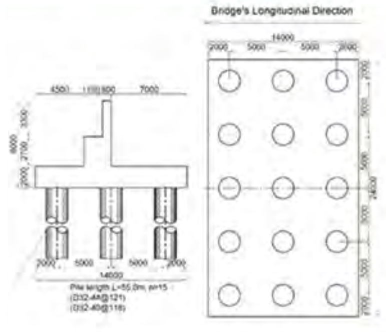
P21~22: SPSP foundation cum cofferdam 8.535 m x 17.222 m (Steel pipe diameter 1.2 m)

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

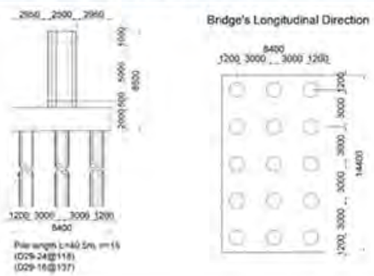
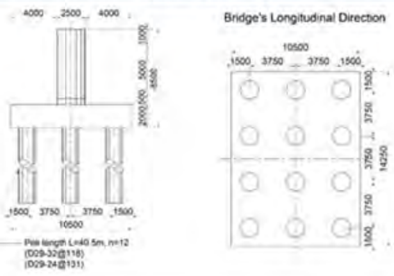
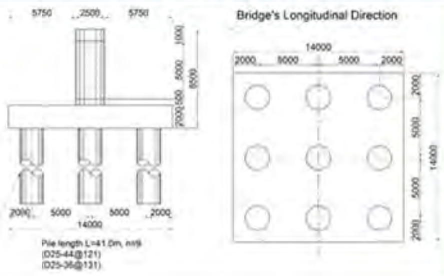
Table 4.4.23 Comparison of Foundation Type for Abutment at B/D

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	2,070.2	3,124.4	2,778.1	4,468.1	4,399.9	6,267.3
		Ra	kN	6,156.0	9,413.0	7,784.0	11,956.0	10,600.0	16,397.0
		σ / σa	–	0.3	0.3	0.4	0.4	0.4	0.4
	Amount of Displacement	σx	mm	3.8	12.0	4.2	14.1	3.2	13.6
		σxa	mm	15.0	15.0	15.0	15.0	15.0	15.0
		R	–	0.3	0.8	0.3	0.9	0.2	0.9
	Stress of a Pile	σs	N/mm^2	10.4	210.3	11.2	193.3	5.3	170.0
		σsa	N/mm^2	160.0	300.0	160.0	300.0	160.0	300.0
		σ / σa	–	0.1	0.7	0.1	0.6	0.0	0.6
	Maximum Stress of a Pile			σs= 210.26 kN/m^2 <σsa = 300 kN/m^2 (OK)		σs= 193.26 kN/m^2 <σsa = 300 kN/m^2 (OK)		σs= 170.00 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 large. (3.7Months)	△	The amount of pile works including ground excavation is consirably the smallest. (3.1Months)	◎	The amount of pile works including ground excavation is consirably smaller. (3.2Months)	○
Environmental Aspect				This alternative entails the largest amount of excavation works.	△	This alternative entails the smallest amount of excavation works.	◎	This alternative entails small amount of excavation works.	○
Cost Ratio				1.279	△	1.000	◎	1.260	○
Overall Evaluation				△		◎		○	

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

Source: JICA Study Team

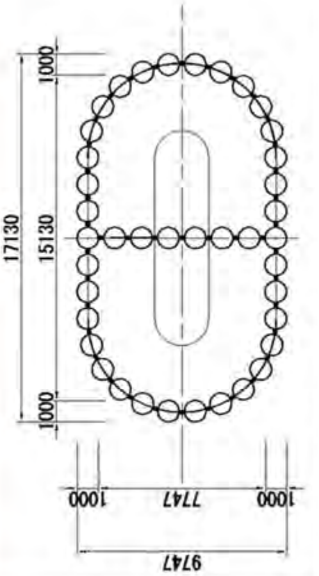
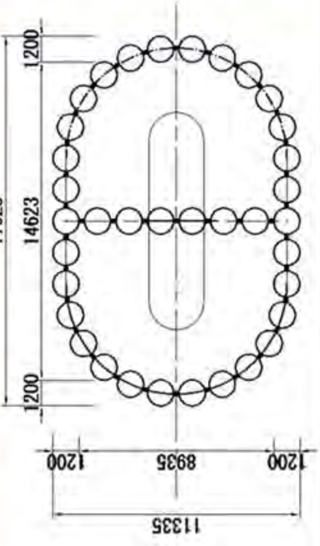
Table 4.4.24 Comparison of Foundation Type for On-land Piers at B/D

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	2,892.3	5,480.1	2,892.3	5,107.0	3,796.7	6,531.1	3,796.7	6,576.1	5,453.5	8,420.9	5,453.5	9,044.6
		Ra	kN	6,995.0	10,611.0	6,995.0	10,611.0	8,883.0	13,510.0	8,883.0	13,510.0	12,351.0	18,860.0	12,351.0	18,860.0
		σ/σa	-	0.4	0.5	0.4	0.5	0.4	0.5	0.4	0.5	0.4	0.4	0.4	0.5
	Amount of Displacement	σx	mm	0.0	13.5	0.0	11.2	0.0	12.3	0.0	11.2	0.0	10.5	0.0	11.1
		σxa	mm	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
		R	-	0.0	0.9	0.0	0.7	0.0	0.8	0.0	0.7	0.0	0.7	0.0	0.7
	Stress of a Pile	σs	N/mm^2	-28.6	229.7	-28.6	261.4	-24.7	178.0	-24.7	205.8	-21.8	185.4	-21.8	202.9
		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	-	0.1	0.8	0.1	0.9	0.1	0.6	0.1	0.7	0.1	0.6	0.1	0.7
	Maximum Stress of a Pile			$σs= 274.69 kN/m^2<σsa= 300kN/m^2$ (OK)				σs= 220.05 kN/m2<σsa= 300kN/m2 (OK)				σs= 225.66 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 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 large. (2.18Months)			○	The amount of pile works including ground excavation is consirably the smallest. (2.14Months)			◎	The amount of pile works including ground excavation is consirably smaller. (2.20Months)			△
Environmental Aspect				This alternative entails the largest amount of excavation works.			△	This alternative entails the smallest amount of excavation works.			◎	This alternative entails small amount of excavation works.			○
Cost Ratio				1.037			○	1.000			◎	1.155			△
Judge				○				◎				△			

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

Source: JICA Study Team

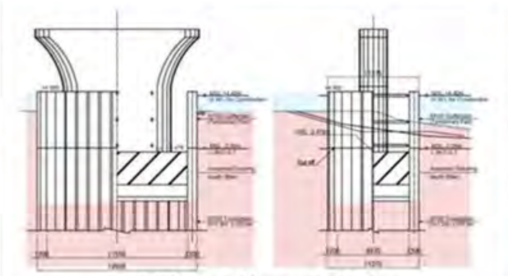
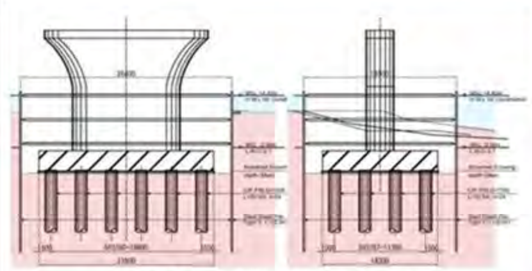
Table 4.4.25 Comparison of Pipe Diameter of SPSP Foundation at B/D (Low-flow Channel Piers P7~P9, P20~P22)

Pile Diameter	Alternative 1. SPSP φ1.0m (t=14~18, n= 40)		Alternative 2. SPSP φ1.2m(t=14~16, n= 36)	
Outline Drawing				
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 smaller amount of pile works.	○
Construction Period	The amount of pile works including ground excavation is consirably large. (5.4Months)	○	The amount of pile works including ground excavation is consirably the smallest. (4.8Months)	◎
Environmental Aspect	This alternative entails the largest amount of excavation works.	△	This alternative entails the smallest amount of excavation works.	◎
Cost Ratio	1.050	○	1.000	◎
Judge	○		◎	

Legend : ◎ Very Good, ○ Good, △ Average

Source: JICA Study Team

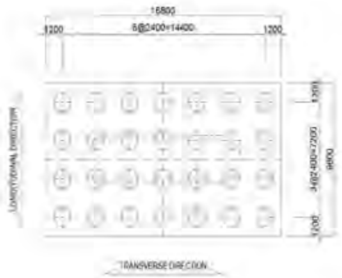
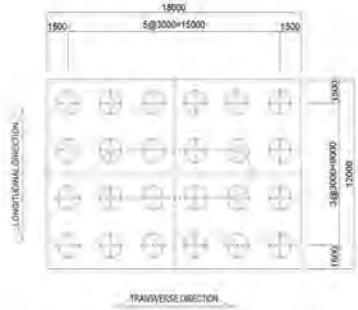
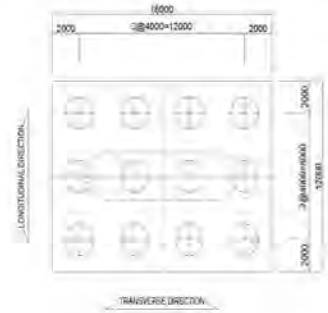
Table 4.4.26 Comparison of Foundation Type for Riverfront Piers at B/D (P6 and P23)

Evaluation Item	Alt-1 : SPSP Foundation (D=1.2m) [selected inFS]		Alt-2 : CIP Pile Foundation (D=1.5m)	
Schematic View	 D=1200mm x 44 nos. (L=55.0m)		 D=1500mm x 24 nos. (L=50.5m)	
Workability & Quality Control	- Sufficient watertightness to a planned waterhead - Changes of pile length during construction is available - Facile quality control due to use of prefabricated steel pipe piles - Careful adjustment is necessary for driving of deep steel pipes	◎	- Sufficient watertightness to a planned waterhead - Flexible to changes of pile length during construction - Careful quality control is necessary for in-situ concrete casting - Careful quality control is necessary for construction of deep borehole	○
Structural Aspect	- Sufficient to support a superstrucure reaction	○	- Sufficient to support a superstrucure reaction	○
Cost Ratio	1.87	△	1.00	◎
Construction Period	4.4 Months (for Pier P6)	○	4.1 Months (for Pier P6)	◎
Environmental Aspect	- Louder noise and larger vibration than CIP pile construction of foundation - Smaller amount of disposal of excavated soil	○	- Lower noise and vibration than SPSP foundation construction - Larger amount of disposal of excavated soil	○
Evaluation	Less Recommended		**Recommended**	

Legend : ◎ Very Good, ○ Good, △ Average

Source: JICA Study Team

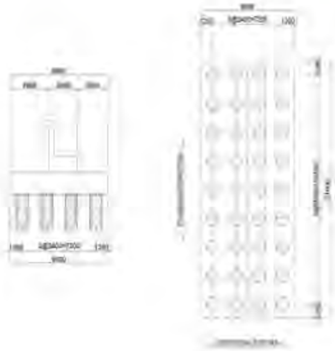
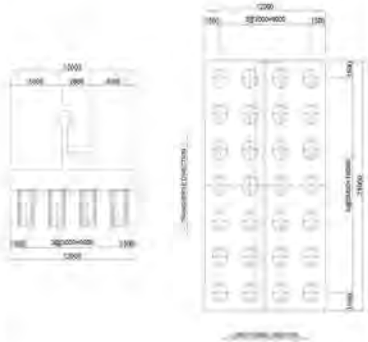
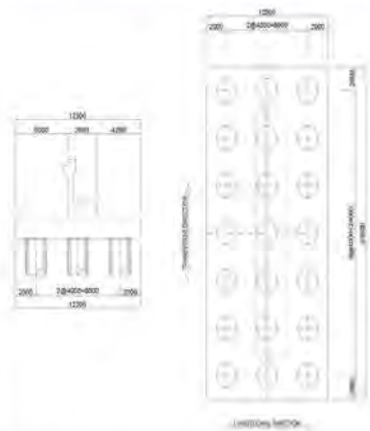
Table 4.4.27 Review of Foundation Type for On-land Piers at D/D

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															
	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
Design Results	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

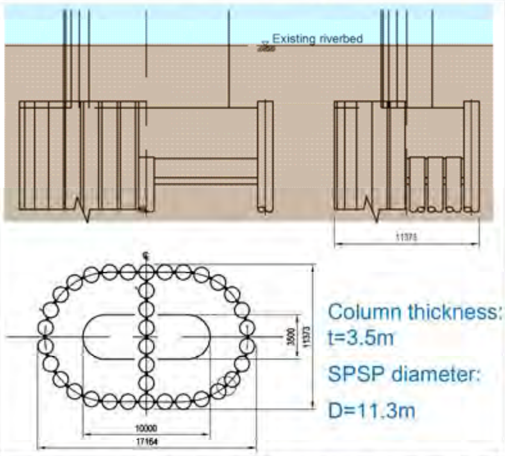
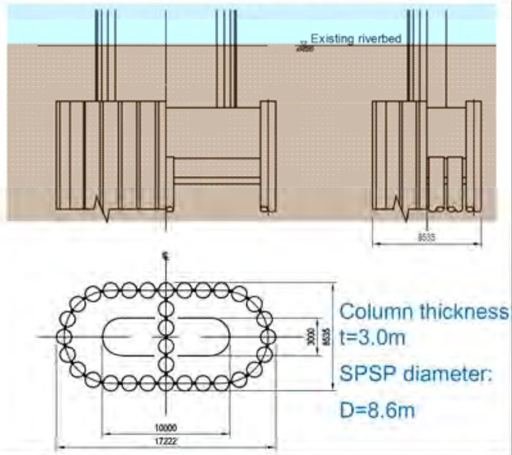
Table 4.4.28 Review of Foundation Type for Abutment at D/D

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 4.4.29 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

4.4.8 Summary of Detailed Design Results for Substructure and Foundation

4.4.8.1 Load Combinations

Load combinations for design of substructures and foundations are shown in Table 4.4.30 which shall comply with the specifications in the JSHB. It is remarked that load situation relating to “extreme wind situation” was not applied to the PC box girder bridge because the amount of wind load on the concrete structures was quite small compared with seismic inertia force.

Table 4.4.30 Load Combinations for Design of PC Box Girder Bridge

Load Combinations	Load Situations	(1) Dead Loads (D)	(2) Live Loads (L)	(3) Impact (I)	(4) Prestress Force (PS)	(5) Influence of Creep of Concrete (CR)	(6) Influence of D dry Shrinkage of Concrete (SH)	(7) Earth Pressure (E)	(8) Water Pressure (HP)	(9) Buoyancy (U)	(10) Wind Loads (W)	(11) Effect of Temperature Change (T)	(12) Seismic Effects (EQ)	(13) Collision of Vessel (CO)	Increase Coefficients
1. Principal loads (P) + particular loads corresponding to principal loads (PP)	Ordinary condition	○	○	○	○	○	○	○	○	○					1.00
		○	○	○	○	○	○	○	○	○					1.00
		○	○	○	○	○	○	○	○	○					1.00
		○	○	○	○	○	○	○	○	○					1.00
2. Principal loads (P) + particular loads corresponding to principal loads (PP) + effects of temperature change (T)		○	○	○	○	○	○	○	○	○		○			1.15
		○	○	○	○	○	○	○	○	○		○			1.15
		○	○	○	○	○	○	○	○	○		○			1.15
		○	○	○	○	○	○	○	○	○		○			1.15
3. Principal loads (P) + particular loads corresponding to principal loads (PP) + wind loads (W)	Extreme Wind	○	○	○	○	○	○	○	○	○	○				1.25
		○	○	○	○	○	○	○	○	○	○				1.25
4. Principal loads (P) + particular loads corresponding to principal loads (PP) + effects of temperature change (T) +wind loads (W)		○	○	○	○	○	○	○	○	○	○	○			1.35
		○	○	○	○	○	○	○	○	○	○	○			1.35
5. Principal loads (P) + particular loads corresponding to principal loads (PP) + vessel collision loads (CO)	Vessel Collision	○	○	○	○	○	○	○	○	○				○	1.50
		○	○	○	○	○	○	○	○	○				○	1.50
6. Principal loads except live loads and impacts + seismic effects (EQ)	Earth-quake	○			○	○	○	○	○	○			○		1.50
		○			○	○	○	○	○	○			○		1.50

Source: JICA Study Team

4.4.8.2 Reaction Forces of Superstructure for Design of Substructures

The values of reaction force transmitted from superstructure to substructure are summarized in Table 4.4.31 and Table 4.4.32.

Table 4.4.31 Reaction Forces of Superstructure for A1 Side

Descriptions					Package-1 : PC-Box								
					A1	P1	P2	P3	P4	P5			
										P4side	P6side	PO4	Total
Bearing Conditions (**M**: Movable, **F**: Fixed, **E**: Elastic support)					E	E	E	E	E	E	E	M	
Working Height Above Bridge Seat	For Bridge Axis direction			m	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	For Transverse Direction			m	-	2.500	2.500	2.500	2.500	2.500	2.500	2.500	2.500
V	Dead Loads		①	kN	11,600	22,800	22,800	23,200	22,800	11,800	13,700	2,000	27,500
	Live Loads	Max	②	kN	2,800	5,600	5,600	5,600	5,600	2,800	3,500	600	6,900
		Min	③		-400	-600	-1,000	-1,000	-600	-400		-100	-500
H	Influence of dry shrinkage of concrete		④	kN	300	390	160	-160	-390	-350	0	-110	-460
	Influence of creep of concrete		⑤	kN	530	640	270	-270	-640	-620	0	-50	-670
	Effect of temperature change (+)		⑥	kN	-620	-770	-350	350	770	650	750	100	1,500
	Effect of temperature change (-)		⑥	kN	620	770	350	-350	-770	-650	-750	-100	-1,500
	Seismic effects	Longitudinal	⑦	kN	3,050	6,250	7,500	7,450	6,200	3,500	3,900	300	7,700
		Transversal	⑧	kN	2,650	7,400	6,700	6,700	7,600	2,650	3,350	750	6,750
M	Eccentric moment due to Dead Load	Longitudinal	⑨	kN.m	0	0	0	0	0				-400
		Transversal	⑩	kN.m	0	0	0	0	0				-60,900

Source: JICA Study Team

Table 4.4.32 Reaction Forces of Superstructure for A2 Side

Descriptions					Package-2 : PC-Box								
					P20			P21	P22	P23	P24	P25	A2
					P19side	P21side	Total						
Bearing Conditions (**M**: Movable, **F**: Fixed, **E**: Elastic support)					E	E		E	E	E	E	E	E
Working Height Above Bridge Seat	For Bridge Axis direction			m	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	For Transverse Direction			m	2.500	2.500	2.500	2.450	2.450	2.400	2.450	2.500	-
V	Dead Loads		①	kN	7,650	11,800	19,450	22,600	23,200	22,800	23,000	22,800	11,600
	Live Loads	Max	②	kN	3,400	2,800	6,200	5,600	5,600	5,600	5,600	5,600	2,800
		Min	③		-900	-400	-1,300	-600	-1,000	-1,000	-1,000	-600	-400
H	Influence of dry shrinkage of concrete		④	kN	0	360	360	480	260	0	-260	-420	-340
	Influence of creep of concrete		⑤	kN	0	620	620	870	450	0	-450	-750	-580
	Effect of temperature change (+)		⑥	kN	-110	-620	-730	-1,030	-550	0	550	880	700
	Effect of temperature change (-)		⑥	kN	110	620	730	1,030	550	0	-550	-880	-700
	Seismic effects	Longitudinal	⑦	kN	1,150	3,300	4,450	6,400	6,500	8,050	7,150	6,150	3,250
		Transversal	⑧	kN	2,250	2,700	4,950	7,600	6,600	6,950	6,650	7,600	2,650
M	Eccentric moment due to Dead Load	Longitudinal	⑨	kN.m			5,000	0	0	0	0	0	0
		Transversal	⑩	kN.m			0	0	0	0	0	0	0

Source: JICA Study Team

4.4.8.3 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.

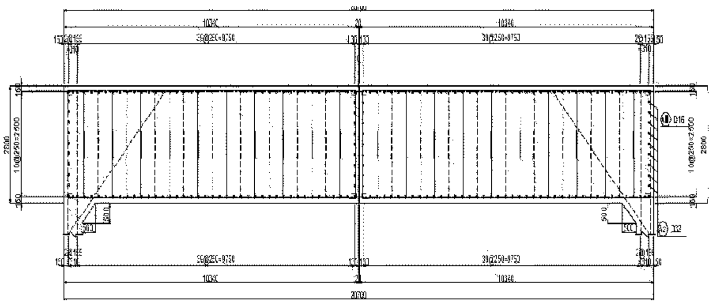
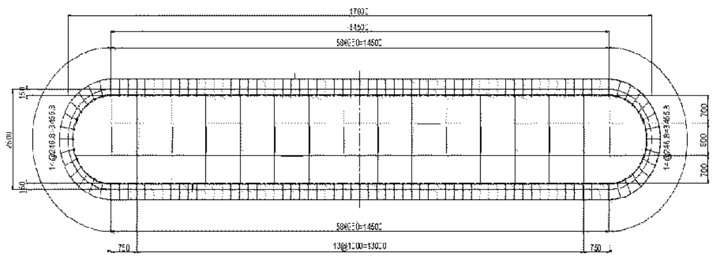
The overhang beams of T-shaped piers can be designed as follows:

- 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 4.4.33 through Table 4.4.38.

Table 4.4.33 Calculation Results for Wall and Comume (A1, P1~P3)

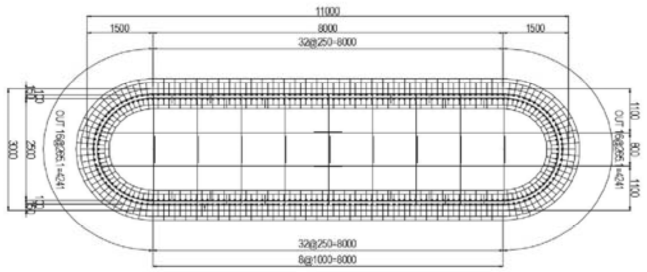
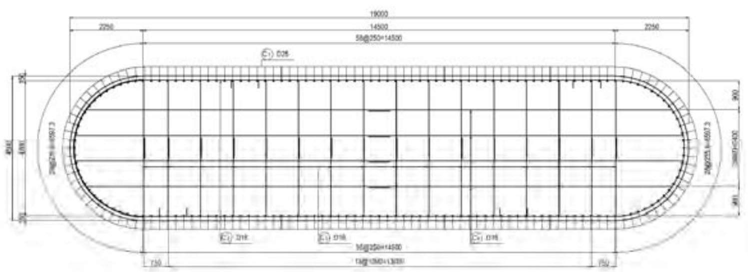
Cross Section of Column	A1  D32@250 P1~P3  D19@250 (P1,P2) D22@250 (P3)

			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	σ_s (N/mm2)	-2.25	-	-9.04	-	-10.32	-	-10.46	-
		σ_{sa}(N/mm2)	-200.00	-	-200.00	-	-200.00	-	-200.00	-
		R-ratio	0.01	-	0.05	-	0.05	-	0.05	-
	Seismic	σ_s (N/mm2)	3.11	-	23.20	-3.94	174.33	-3.57	278.19	-2.86
		σ_{sa}(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	τ_m(N/mm2)	0.059	-	0.046	-	0.020	-	0.020	-
		τ_a(N/mm2)	0.145	-	0.129	-	0.129	-	0.137	-
		R-ratio	0.41	-	0.36	-	0.16	-	0.15	-
	Seismic	τ_m(N/mm2)	0.131	-	0.209	0.201	0.236	0.194	0.244	0.202
		τ_a(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 : σ_s ; Bending Unit Stress
σ_{sa} ; Allowable Unit Stress
τ_m ; Unit Share Force
τ_a ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.4.34 Calculation Results for Wall and Columns (P4~P5)

			P4		P5					
Cross Section of Column			P4  D25@250							
			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)				

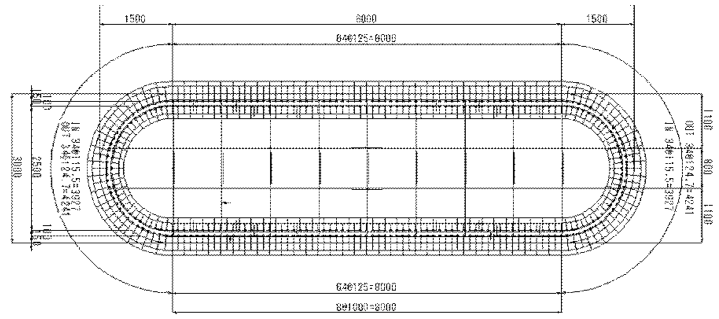
Note : σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

Table 4.4.35 Calculation Results for Wall and Columns (P20~P23)

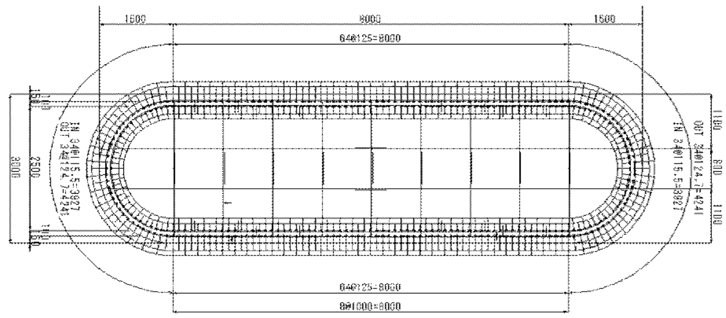
Cross Section of Column

P20



2-D29@125

P21~P23



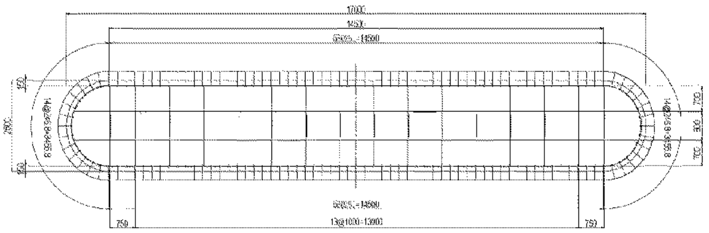
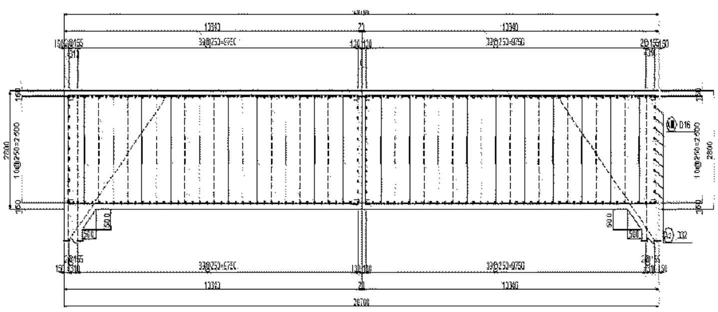
2-D38@125(P21,P22)
2-D29@125(P23)

			P20		P21		P22		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 4.4.36 Calculation Results for Wall and Columns (P24, P25, and A2)

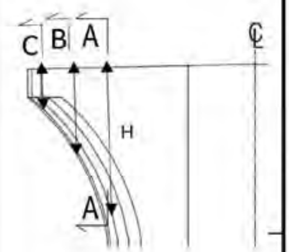
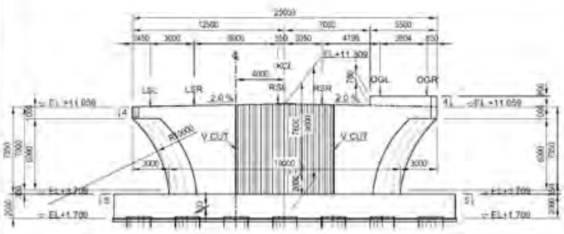
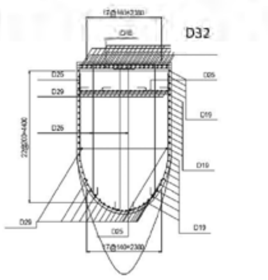
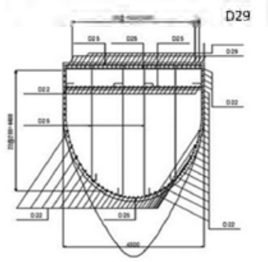
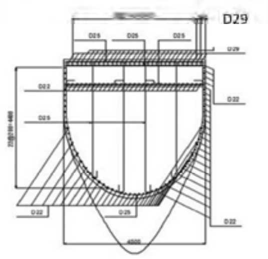
Cross Section of Column (Longitudinal Direction)	P24,P25  D19@250 A2  D32@250

			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 4.4.37 Calculation Results for Overhang Beams (P4 and P5)

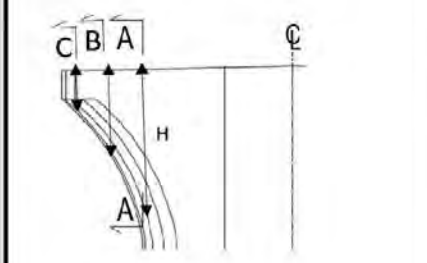
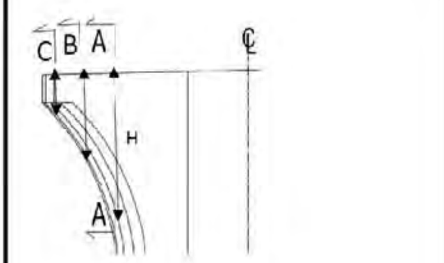
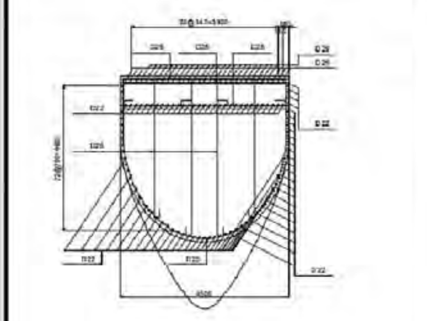
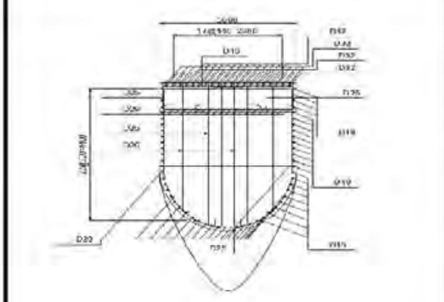
	P4 LEFT&RIGHT	P5 LEFT	P5 RIGHT
Side View of Beam & Cross Section of Beam			
	Cross Section of Beam	Cross Section of Beam	Cross Section of Beam
	 2-D32@140	 2-D29@140	 2-D29@140

		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 4.4.38 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
		

		P20 LEFT&RIGHT Section A	P20 LEFT&RIGHT Section B	P20 LEFT&RIGHT Section C	P21 ~P23LEFT&RIGHT Section A	P21 ~P23LEFT&RIGHT Section B	P21 ~P23LEFT&RIGHT 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

4.4.8.4 Computation of Reverse T-shaped Abutment

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

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

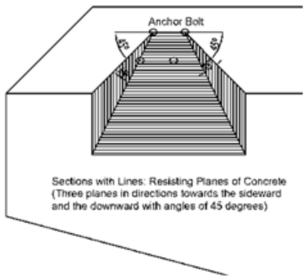
The wing wall shall be designed as slabs to receive superimposed loads due to live loads and earth pressure. The slab in this case shall be cantilevers fixed on two sides to a wall and footing.

Computation results are shown in Table 4.4.33 and Table 4.4.36.

4.4.8.5 Design of Bridge Seats

Bridge seats shall be designed with sufficient strength to withstand the vertical and horizontal forces from bearings. Bridge seats should be designed so that corrosion of bearing and girders can be minimized.

Horizontal forces transmitted from bearings are carried by concrete and reinforcement. The resisting area of concrete is the summation of three planes in directions of sideward and downward with edge angles of 45 degrees as shown in Figure 4.4.35. The calculation results of the required reinforcement bar are shown in Table 4.4.39.



Source: JICA Study Team

Figure 4.4.35 Calculation of Bridge Seat

4.4.8.6 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 4.4.40 through Table 4.4.43.

Table 4.4.39 Calculation Results of Reinforcement Bar for Bridge Seats

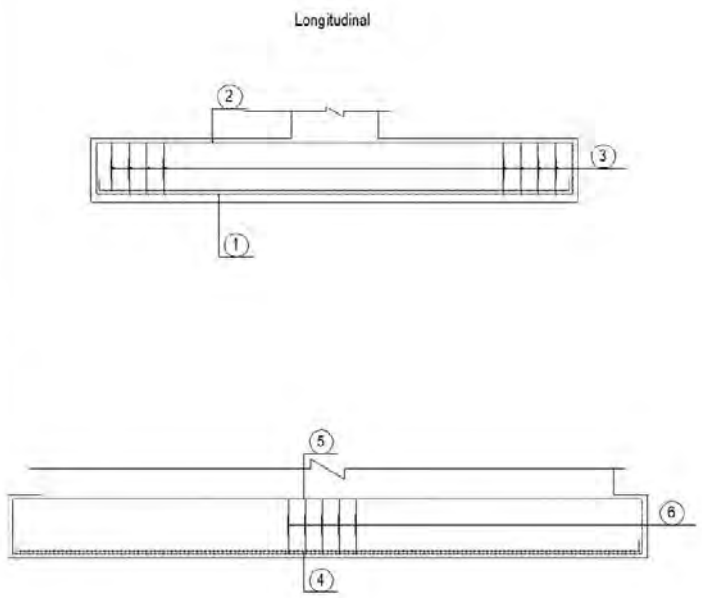
		Unit	A1	P1	P2	P3	P4	P5		P20		P21	P22	P23	P24	P25	A2	AO1	PO1	PO2	PO3
								P4 Side	P6 Side	P19 Side	P21 Side										
Breadth of Anchor Bars	B	mm	900	900	900	900	900	900		1270	900	900	900	900	900	900	900	2500	2500	2500	2500
Distance from the Center of the Anchor Bar to the Edge of Bridge Seat	da	mm	650	1250	1250	1250	1500	900		1750	900	1500	1500	1500	1250	1250	650	650	750	750	750
Resisting Area in Concrete	Ac	mm2	2,022,325	6,010,408	6,010,408	6,010,408	8,273,149	3,436,539	0	########	3,436,539	8,273,149	8,273,149	8,273,149	6,010,408	6,010,408	2,022,325	3,493,107	4,242,641	4,242,641	4,242,641
Design Strength of Concrete	σck	N/mm2	24.0	24.0	24.0	24.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	24.0	24.0	24.0	24.0	24.0	24.0	24.0
Coefficient to Calculate Pc	α	-	0.31	0.33	0.33	0.34	0.31	0.30	#DIV/0!	0.23	0.30	0.31	0.32	0.31	0.34	0.33	0.31	0.40	0.40	0.41	0.41
Load Carried by Concrete; $Pc = 0.32\alpha\sqrt{\sigma ck} * Ac$	Pc	kN	**990**	**3,151**	**3,151**	**3,181**	**4,566**	**1,792**	**#DIV/0!**	**4,674**	**1,792**	**4,546**	**4,608**	**4,566**	**3,166**	**3,151**	**990**	**2,193**	**2,664**	**2,747**	**2,747**
Yield Point of Reinforcement	σsy	N/mm2	345	345	345	345	345	345	345	345	345	345	345	345	345	345	345	345	345	345	345
Modification Coefficient for Load Carried by Reinforcement	β	-	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Total Cross Sectional Area of Reinforcing Bar *1	ΣAsi	mm2	6111	16198	12760	12760	11083	9696	#DIV/0!	14138	9696	11083	11083	11083	12760	16198	6111	4384	13572	13572	13572
Load Carried by Reinforcement; $Ps = \Sigma\{\beta\,(1-hi/da)\,\sigma sy * Asi\}$	Ps	kN	**1,054**	**2,794**	**2,201**	**2,201**	**1,912**	**1,673**	**#DIV/0!**	**2,439**	**1,673**	**1,912**	**1,912**	**1,912**	**2,201**	**2,794**	**1,054**	**756**	**2,341**	**2,341**	**2,341**
Strength of the Bridge Seat; $Pbs = Pc + Ps$	Pbs	kN	2,044	5,945	5,352	5,382	6,478	3,465	#DIV/0!	7,113	3,465	6,457	6,520	6,478	5,367	5,945	2,044	2,949	5,005	5,088	5,088
Design Horizontal Force	H	kN	2,025	5,850	5,100	5,100	5,850	2,025		431	2,025	5,775	5,100	5,325	5,100	5,850	2,025	810	3,825	1,845	1,350
R-ratio	R	-	**0.99**	**0.98**	**0.95**	**0.95**	**0.90**	**0.58**	**#DIV/0!**	**0.06**	**0.58**	**0.89**	**0.78**	**0.82**	**0.95**	**0.98**	**0.99**	**0.27**	**0.76**	**0.36**	**0.27**

Calculation of Reinforcing Bar Area (ΣAsi)

		Unit	A1	P1	P2	P3	P4	P5		P20		P21	P22	P23	P24	P25	A2	AO1	PO1	PO2	PO3
								P4 Side	P6 Side	P19 Side	P21 Side										
Diameter of Reinforcing Bar	φ	mm	D16	D25	D25	D25	D25	D25	D25	D25	D25	D25	D25	D25	D25	D25	D16	D16	D25	D25	D25
Number of Reinforcing Bar	n	nos.	20	18	16	16	19	18		24	18	19	19	19	16	18	20	21	23	23	23
Cross Sectional Area of Reinforcing Bar	As1	mm2	3972	9120.6	8107.2	8107.2	9627.3	9120.6	0	12160.8	9120.6	9627.3	9627.3	9627.3	8107.2	9120.6	3972	4170.6	11654.1	11654.1	11654.1
Depth of Reinforcing Bar from the Surface of the Bridge Seat	h1	mm	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
(1-h1/da)	-	-	0.846	0.920	0.920	0.920	0.933	0.889	#DIV/0!	0.943	0.889	0.933	0.933	0.933	0.920	0.920	0.846	0.846	0.867	0.867	0.867
Diameter of Reinforcing Bar	φ	mm	D16	D25	D22	D22	D16	D16	D16	D16	D16	D16	D16	D16	D22	D25	D16	D16	D16	D16	D16
Number of Reinforcing Bar	n	nos.	20	18	16	16	12	10		15	10	12	12	12	16	18	20	7	23	23	23
Cross Sectional Area of Reinforcing Bar	As2	mm2	3972	9120.6	6193.6	6193.6	2383.2	1986	0	2979	1986	2383.2	2383.2	2383.2	6193.6	9120.6	3972	1390.2	4567.8	4567.8	4567.8
Depth of Reinforcing Bar from the Surface of the Bridge Seat	h2	mm	200	180	180	180	180	180	180	180	180	180	180	180	180	180	200	250	180	180	180
(1-h2/da)	-	-	0.692	0.856	0.856	0.856	0.880	0.800	#DIV/0!	0.897	0.800	0.880	0.880	0.880	0.856	0.856	0.692	0.615	0.760	0.760	0.760
Total Cross Sectional Area of Reinforcing Bar; $\Sigma Asi = \Sigma\{Asi\,(1-hi/da)\}$	ΣAsi	mm2	6111	16198	12760	12760	11083	9696	#DIV/0!	14138	9696	11083	11083	11083	12760	16198	6111	4384	13572	13572	13572

Source: JICA Study Team

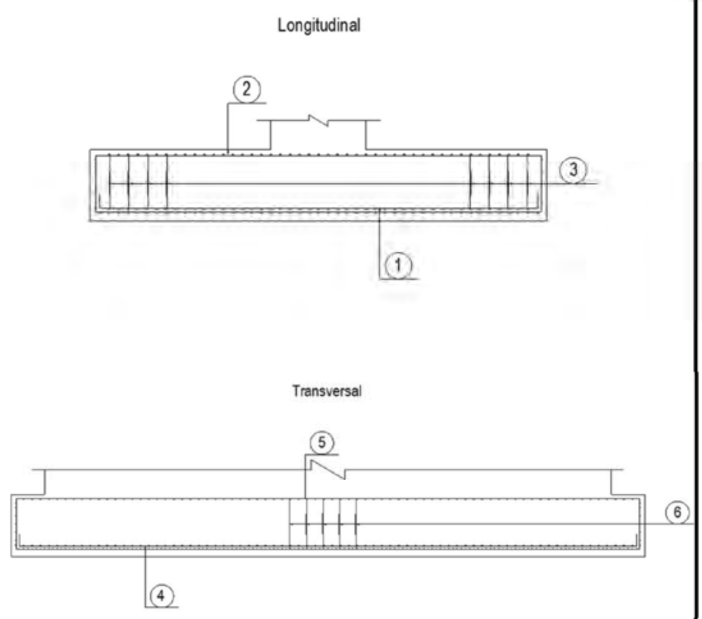
Table 4.4.40 Calculation Results for Footing of Piers (P1~P3)

			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 4.4.41 Calculation Results for Footing of Piers (P4 and P5)

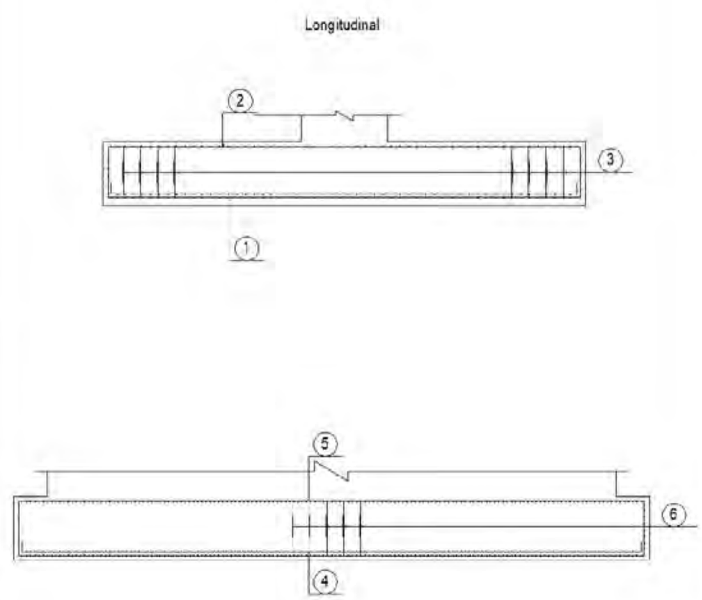
	PIER
Cross Section of Pile Cap (Longitudinal & Transversal Direction) SD345	

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

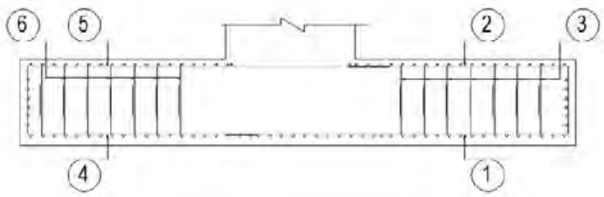
	PIER
Cross Section of Pile Cap (Longitudinal & Transversal Direction) SD345	

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

			ABUTMENT					
Cross Section of Pile Cap (Longitudinal & Transversal Direction) SD345			Longitudinal 					
			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/mm^2)	117.42	27.92	152.83	38.34	135.89	60.23
		σsa(N/mm^2)	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/mm^2)	224.96	204.16	283.10	273.60	264.21	235.89
		σsa(N/mm^2)	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/mm^2)	0.254	0.154	0.335	0.153	0.260	0.143
		τa (N/mm^2)	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/mm^2)	0.439	0.284	0.579	0.204	0.466	0.264
		τa (N/mm^2)	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

4.4.8.7 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.

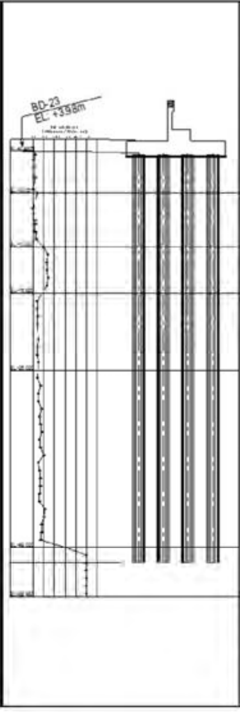
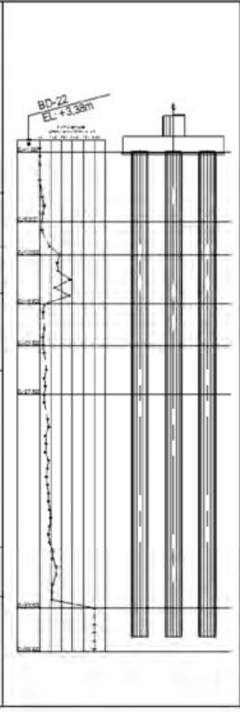
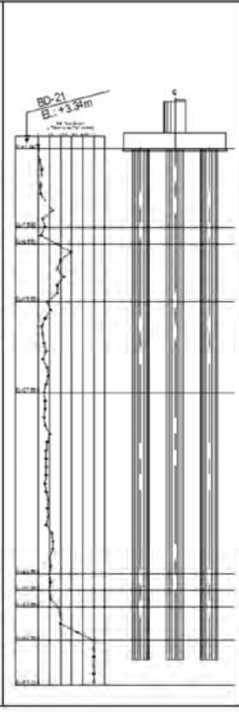
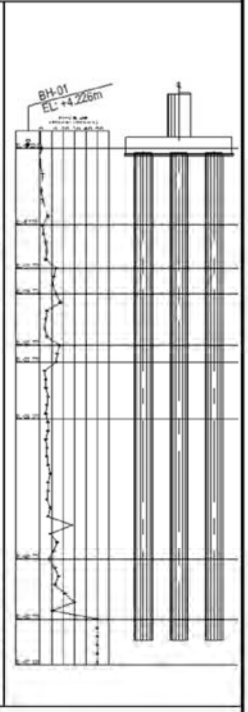
  The allowable displacement of abutment foundation is 15 mm regardless of the foundation width because the displacement may increase with time due to the effects of creep and backfill settlement.
- The axial reaction at each pile head shall not exceed the allowable pile bearing capacity.
- The stresses 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 4.4.44 through Table 4.4.46.

The calculation results of cross sectional stress of CIP piles are shown in Table 4.4.47 through Table 4.4.49.

Also, calculation results of SPSP are summarized in Table 4.4.50 through Table 4.4.54.

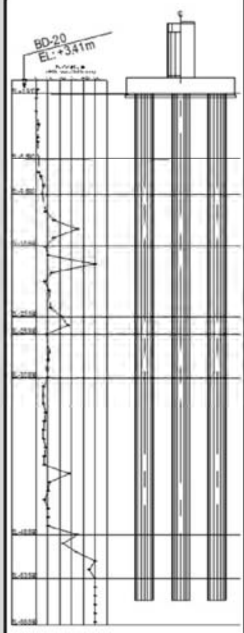
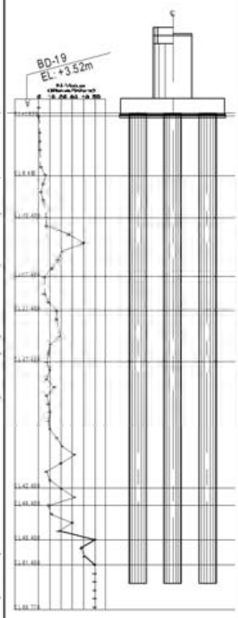
Table 4.4.44 Calculation Results of CIP Pile Foundation Stability (A1~P3)

	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

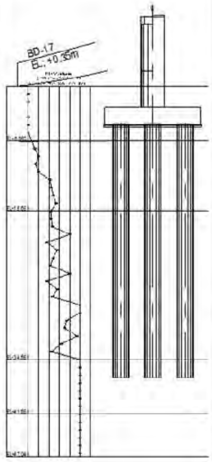
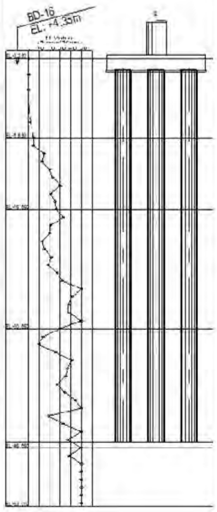
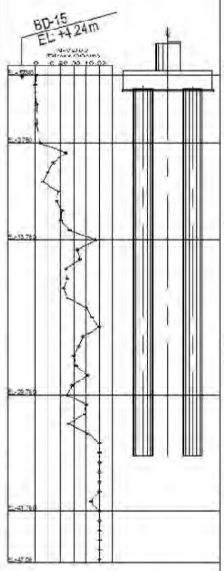
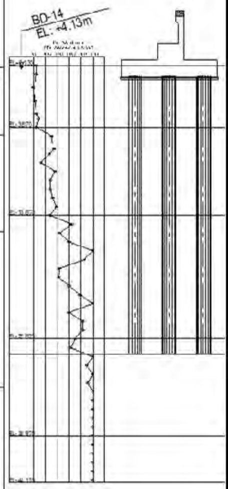
Table 4.4.45 Calculation Results of CIP Pile Foundation Stability (P4 and P5)

	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

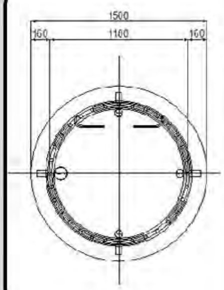
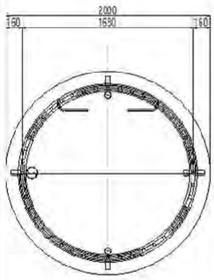
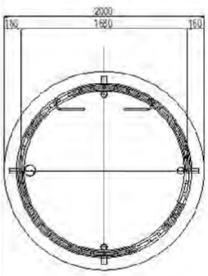
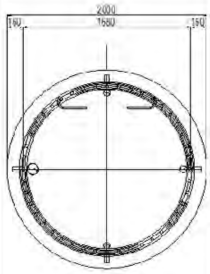
Table 4.4.46 Calculation Results of CIP Pile Foundation Stability (P23~P25 and A2)

	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

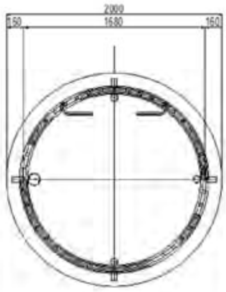
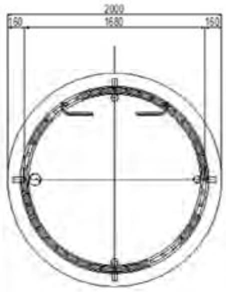
Table 4.4.47 Calculation Results of Cross Section of CIP Pile Foundation (A1~P3)

	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

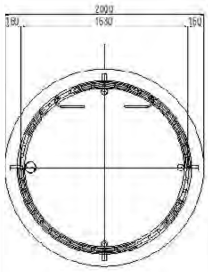
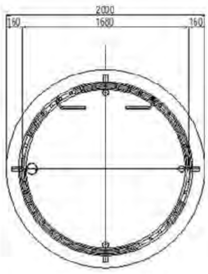
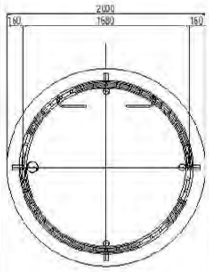
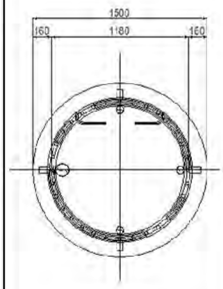
Table 4.4.48 Calculation Results of Cross Section of CIP Pile Foundation (P4 and P5)

	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 4.4.49 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

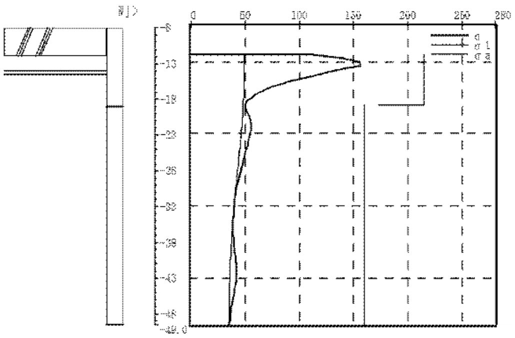
Source: JICA Study Team

Table 4.4.50 Calculation Results of SPSP Foundation Stability and Stress (Longitudinal)

Item			Unit	P20		P21		P22	
				Ordinary	Seismic	Ordinary	Seismic	Ordinary	Seismic
Forces*2		Vo	kN	58,871.1	51,485.3	52,152.5	45,736.5	53,296.9	47,057.5
		Ho	kN	1,710.0	13,531.0	2,380.0	13,400.6	1,260.0	12,319.8
		Mo	kN.m	38,174.0	188,271.1	44,744.0	197,122.1	22,554.0	175,437.3
Displacement*2									
At Top of Top Slab	Displacement	$\delta 1$	cm	0.281	1.971	0.453	2.076	0.185	1.413
	Allowable	δa	cm	5.000	5.000	5.000	5.000	5.000	5.000
Pile Bearing*2 (P20:L=47.5m, P21:L=56.5m, P22:L=51.5m)									
Vertical Reaction	Max	Rmax	kN/pile	1,646	1,538	1,639	1,478	1,670	1,516
	Min	Rmin	kN/pile	1,625	1,322	1,621	1,381	1,661	1,425
	Bearing	Ra	kN/pile	3,760	5,440	4,231	6,286	4,483	6,664
	Pull-out	Pa	kN/pile	-1,672	-2,778	-1,940	-3,375	-1,602	-2,757
Pile Stresses									
Exterior P20,21:SKY 490 P22:SKY4 00	Thickness	t	mm	14		14		14	
	After Construction	σ_1*2	N/mm²	44.85	106.40	48.29	127.76	37.71	104.43
	During construction	σ_2	N/mm²	107.16	107.16	95.35	95.35	83.11	83.11
	Combined	σ_{max}	N/mm²	152.01	213.56	143.64	223.11	120.82	187.54
	Allowable	σa	N/mm²	185.00	280.00	185.00	280.00	140.00	210.00
Bulkhead*2 (SKY400) t=14mm	After Construction	σ_1	N/mm²	43.40	112.73	48.50	132.77	—	—
	Allowable	σa	N/mm²	140.00	210.00	140.00	210.00	—	—

*1:Designed by Well Model according

*2:due to after construction loads



Longitudinal Direction – Ordinary Condition

Figure:Stress Diagram of Steel Pipe Sheet Pile for P22

Source: JICA Study Team

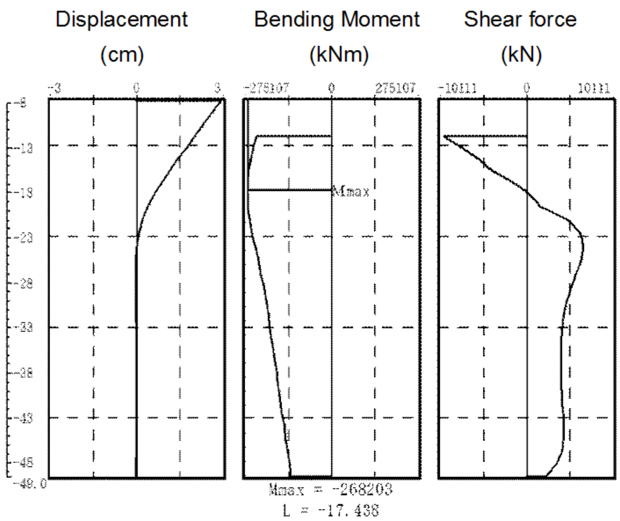
Table 4.4.51 Calculation Results of SPSP Foundation Stability and Stress (Transverse)

Item			Unit	P20		P21		P22	
				Ordinary	Seismic	Ordinary	Seismic	Ordinary	Seismic
Forces*2		Vo	kN	58,871.1	51,485.3	52,152.5	45,736.5	53,296.9	46,880.9
		Ho	kN	2.7	12,290.8	1.6	12,430.5	1.0	11,196.3
		Mo	kN.m	9.6	183,347.3	6.0	209,890.0	4.3	178,441.8
Displacement*2									
At Top of Top Slab	Displacement	δ1	cm	0.000	1.521	0.000	1.308	0.000	0.903
	Allowable	δa	cm	5.000	5.000	5.000	5.000	5.000	5.000
Pile Bearing*2 (P20:L=47.5m, P21:L=56.5m, P22:L=51.5m)									
Vertical Reaction	Max	Rmax	kN/pile	1,635	1,486	1,630	1,557	1,666	1,528
	Min	Rmin	kN/pile	1,635	1,374	1,630	1,301	1,666	1,402
	Bearing	Ra	kN/pile	3,760	5,440	4,231	6,286	4,483	6,664
	Pull-out	Pa	kN/pile	-1,672	-2,778	-1,940	-3,375	-1,602	-2,757
Pile Stresses									
Exterior (SKY490) (P22:SKY400)	Thickness	t	mm	14		16		14	
	After Construction	σ_1*2	N/mm²	36.64	99.65	36.52	115.76	32.62	91.13
	During construction	σ_2	N/mm²	102.80	102.80	80.18	79.95	70.37	70.37
	Combined	σ_{max}	N/mm²	139.45	202.46	116.70	195.70	102.98	161.50
	Allowable	σa	N/mm²	185.00	280.00	185.00	280.00	140.00	210.00
Bulkhead*2 (SKY400) t=14mm	After Construction	σ_1	N/mm²	36.65	109.72	36.52	122.44	—	—
	Allowable	σa	N/mm²	140.00	210.00	140.00	210.00	—	—

*1:Designed by Well Model according

*2:due to after construction loads

*:Designed by Well Model according



Transversal Direction – Seismic Condition

Figure:calculation results of Steel Pipe Sheet Pile for P22

Source: JICA Study Team

Table 4.4.52 Calculation Results for SPSP Foundation Top Slab (Longitudinal)

				P20		P21		P22	
			Unit	Ordinary	Seismic	Ordinary	Seismic	Ordinary	Seismic
Lower tensile	Bending moment	MA	kN.m	3807.0	7869.0	3494.0	7243.0	2719.0	6729.0
	Necessary reinforcement	Asr	cm^2	72.627	80.397	56.271	72.589	50.456	67.240
	Neutral axis	x	cm	108.5	108.5	100.8	100.8	100.8	100.8
	Stresses	σc	N/mm^2	2.13	4.41	2.07	4.30	1.61	4.00
		σs	N/mm^2	78.49	162.25	84.62	175.40	65.84	162.95
	Resultant tensile force	T	kN	1691.9	3497.5	1552.9	3219.0	1208.4	2990.6
	Reinforcement requirements	As	cm^2	105.742	116.582	83.943	107.301	75.524	99.685
Upper tensile	Bending moment	MA'	kN.m	2644.0	-2340.0	1242.0	-3165.0	2075.0	-2578.0
	Necessary reinforcement	Asr	cm^2	0.000	21.104	0.000	28.733	0.000	23.303
	Neutral axis	x	cm	9.3	69.7	9.3	69.7	9.3	69.7
	Stresses	σc	N/mm^2	0.00	1.85	0.00	2.51	0.00	2.04
		σs	N/mm^2	0.00	125.86	0.00	170.27	0.00	138.70
	Resultant tensile force	T	kN	0.0	1039.8	0.0	1406.7	0.0	1145.9
	Reinforcement requirements	As	cm^2	0.000	34.662	0.000	46.891	0.000	38.197
Allowable stresses		σca	N/mm^2	8.00	12.00	9.20	12.00	8.00	12.00
		σsa	N/mm^2	160.00	300.00	185.00	300.00	160.00	300.00
Average shearing force		QB	kN	1230.0	2533.0	1525.0	3165.0	1208.0	2975.0
		τm	N/mm^2	0.34	0.69	0.41	0.86	0.33	0.81
		τa1'	N/mm^2	1.01	1.53	1.16	1.56	1.03	1.56
Average shearing force		S	kN	1214.0	2517.0	1525.0	3165.0	1208.0	2975.0
		τm	N/mm^2	0.33	0.69	0.41	0.86	0.33	0.81
		τa1'	N/mm^2	1.01	1.53	1.16	1.56	1.03	1.56
Shearing force carried by concrete		Sca	kN	3675.0	5592.0	4261.0	5736.0	3769.0	5736.0
Diagonal tension reinforcement	Shearing force	Sh'	kN	0.0	0.0	0.0	0.0	0.0	0.0
	Longitudinal spacing	s	cm	100.0	100.0	100.0	100.0	100.0	100.0
	Reduction coefficient	Cds	—	0.245	0.245	0.171	0.171	0.171	0.171
	Allowable tensile stresses	σsa	N/mm^2	160.00	200.00	160.00	200.00	160.00	200.00
	Used reinforcement	Aw	cm^2	1.986	1.986	1.986	1.986	1.986	1.986
	Necessary reinforcement	Awreq	cm^2	0.000	0.000	0.000	0.000	0.000	0.000

Source: JICA Study Team

Table 4.4.53 Calculation Results for SPSP Foundation Top Slab (Transverse)

			Unit	P20		P21		P22	
				Ordinary+W	Seismic	Ordinary	Seismic	Ordinary	Seismic
Lower tensile	Bending moment	MA	kN.m	2879.0	6672.0	2765.0	7418.0	2825.0	6763.0
	Necessary reinforcement	Asr	cm^2	53.017	66.002	50.862	73.694	51.986	66.953
	Neutral axis	x	cm	101.3	101.3	101.3	101.3	101.3	101.3
	Stresses	σc	N/mm^2	1.68	3.90	1.62	4.34	1.65	3.96
		σs	N/mm^2	69.07	160.06	66.35	178.00	67.78	162.28
	Resultant tensile force	T	kN	1279.7	2965.4	1229.1	3297.0	1255.4	3005.9
	Reinforcement requirements	As	cm^2	79.983	98.847	76.817	109.902	78.463	100.196
Upper tensile	Bending moment	MA'	kN.m	2879.0	-1745.0	2765.0	-2676.0	2824.0	-1916.0
	Necessary reinforcement	Asr	cm^2	0.000	15.535	0.000	24.014	0.000	17.084
	Neutral axis	x	cm	8.0	70.0	8.0	70.0	8.0	70.0
	Stresses	σc	N/mm^2	0.00	1.37	0.00	2.10	0.00	1.50
		σs	N/mm^2	0.00	93.16	0.00	142.83	0.00	102.28
	Resultant tensile force	T	kN	0.0	775.6	0.0	1189.2	0.0	851.6
	Reinforcement requirements	As	cm^2	0.000	25.854	0.000	39.640	0.000	28.387
Allowable stresses		σca	N/mm^2	8.00	12.00	8.00	12.00	8.00	12.00
		σsa	N/mm^2	160.00	300.00	160.00	300.00	160.00	300.00
Average shearing force		QB	kN	1045.0	2338.0	1046.0	2701.0	1079.0	2480.0
		τm	N/mm^2	0.28	0.63	0.28	0.73	0.29	0.67
		τa1'	N/mm^2	0.92	1.40	0.95	1.44	0.95	1.44
Average shearing force		S	kN	1022.0	2316.0	1031.0	2686.0	1061.0	2462.0
		τm	N/mm^2	0.28	0.62	0.28	0.72	0.29	0.66
		τa1'	N/mm^2	0.92	1.40	0.95	1.44	0.95	1.44
Shearing force carried by concrete		Sca	kN	3417.0	5199.0	3508.0	5339.0	3508.0	5339.0
Diagonal tension reinforcement	Shearing force	Sh'	kN	0.0	0.0	0.0	0.0	0.0	0.0
	Longitudinal spacing	s	cm	100.0	100.0	100.0	100.0	100.0	100.0
	Reduction coefficient	Cds	—	0.252	0.252	0.239	0.239	0.239	0.239
	Allowable tensile stresses	σsa	N/mm^2	160.00	200.00	160.00	200.00	160.00	200.00
	Used reinforcement	Aw	cm^2	1.986	1.986	1.986	1.986	1.986	1.986
	Necessary reinforcement	Awreq	cm^2	0.000	0.000	0.000	0.000	0.000	0.000

Source: JICA Study Team

Table 4.4.54 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

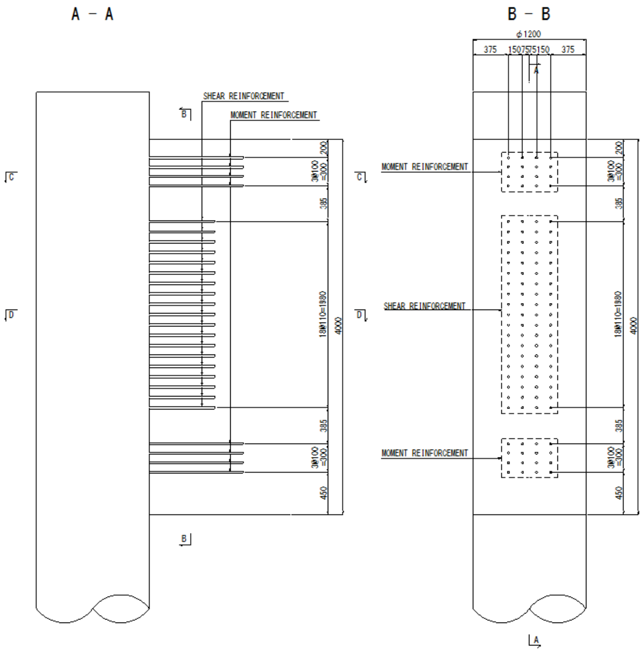


Figure : Detail for Connection between Top Slab and Steel Pipe Sheet Pile

Source: JICA Study Team

4.4.9 Bridge Accessories

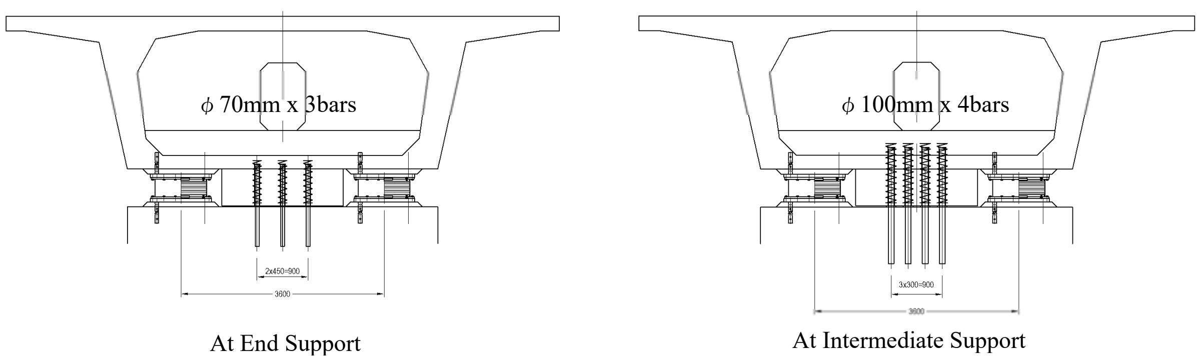
4.4.9.1 Bearings

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 4.4.55 Comparison of Support Condition and Bearing Type

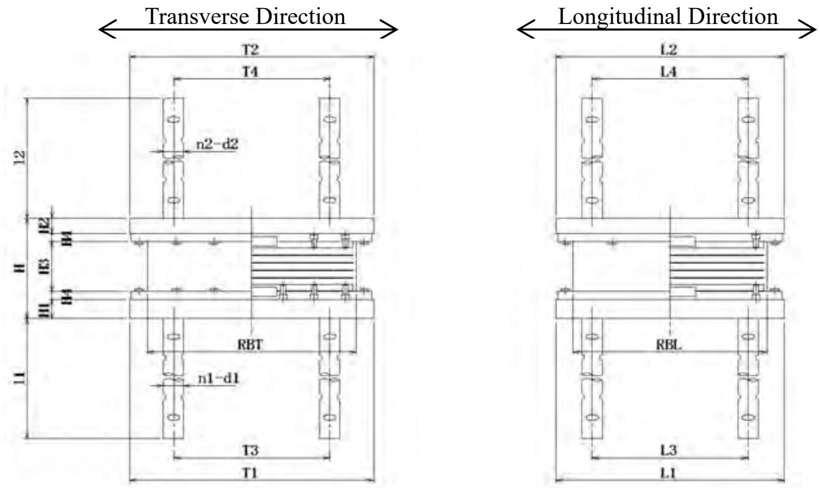
	Elastic Support	Fixed + Moveable Support
Applicable type of bearings	Elastomeric Rubber Bearing 	Pot Bearing 
Effect of restraint forces	• Effect of restraint force to substructures is smaller, as the superstructure is elastically supported in the longitudinal direction.	• Effect of restraint forces 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



Source: JICA Study Team

Figure 4.4.36 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

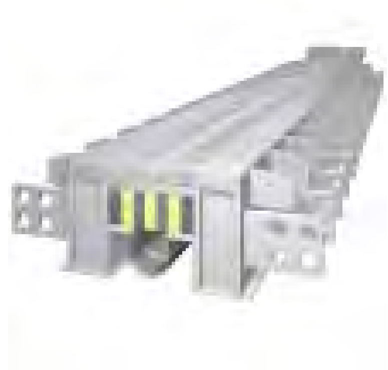
Source: JICA Study Team

Figure 4.4.37 Elastomeric Rubber Bearing

4.4.9.2 Expansion Joints

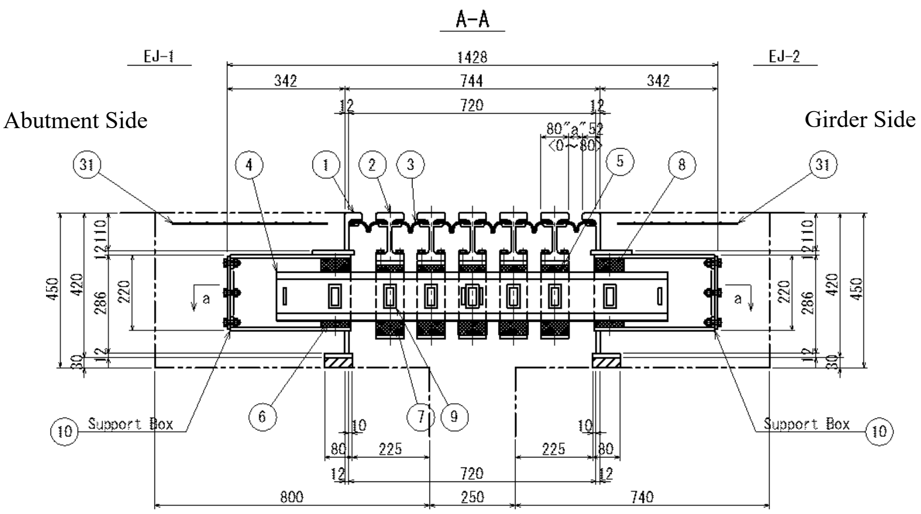
As the horizontal movement of PC box girder bridge during earthquake is large due to the relatively high design seismic coefficient (kh = 0.3), its expansion joints need to accommodate large displacement. As a result of the following comparative study, “modular expansion joint” has been selected, considering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

Table 4.4.56 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

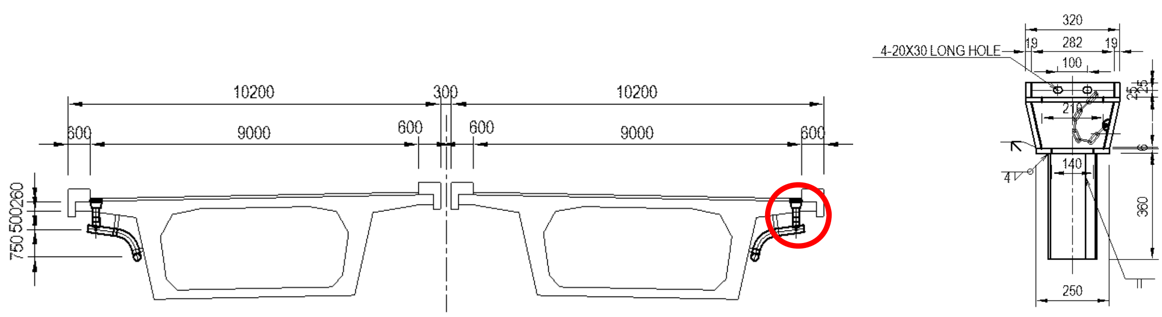
Figure 4.4.38 Expansion Joint at A1 and A2

4.4.9.3 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.

4.4.9.4 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 4.4.39 Catch Pits Arrangement and Detail (PC Box Girder Bridge)

4.5 STUDY ON ON-RAMP BRIDGE

The B/D of the on-ramp bridge was conducted based on the terms of agreement in the F/S, and the design team confirmed and studied the design policy, design conditions, structural types, bridge length and spanning, and other works that are necessary for the Project. The design team reviewed the F/S report and found out that some outstanding issues should be worked out prior to the subsequent detailed design stage.

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

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 Design Principle

The length of on-ramp bridge and its span arrangement was comprehensively examined considering the terrain on site, geological conditions, crossing obstacles, construction workability, and economic efficiency.

4.5.1.2 Study Conditions

(1) Geography and Geology

The site for this approach bridge consists of a flood channel and a low-flow channel river whose elevations are around MSL+3.00 m~4.00 m and MSL -5.00 m~7.00 m, respectively. Neither future land use plan including reclamation nor river training plan exists.

The bearing stratum for the bridge is distributed at MSL-40.0 m~ -55.0 m, whose N-value is around 50. There are no appropriate soil layers other than this layer with sufficient firmness and thickness to support bridge reactions.

(2) Crossing Objects

Crossing objects are investigated by means of site survey and literature survey supported by the counterpart. There are objects that should be taken into account for the bridge span arrangement planning as control points such as embankment section of on-ramp road (to be constructed under this Project). It is confirmed that relocation of other crossing objects is possible. Summary of the crossing objects is shown in Table 4.5.2.

Table 4.5.2 Summary of Crossing Obstacles for Span Arrangement

No.	Crossing Object Name	Chainage	Relocation	Control Point	
				Abutment	Pier
1	On-ramp (embankment section)	0+542.5	No	Yes	Yes
2	Location of approach end (nose)	0+654.0	No	No	Yes
3	Beginning point of on-ramp curved section	0+542.5	No	Yes	No

Source: JICA Study Team

(3) Construction Conditions

- Site conditions

There are no buildings/facilities which require cautious construction adjacent to the bridge in order to avoid any harmful displacement. There are no objects that will restrain the construction works.

Pile cap should be outside of the on-ramp road in the plan.

- Ground water level

Temporary cofferdam shall be used due to relatively high ground water level.

- Superstructure erection

Fixed staging support is not to be used because of the existence of soft surface soil that requires ground improvement work for installation of the fixed staging support.

The bridge section of the on-ramp should not be in a curved section.

- Economic span for BD

The economic span length is estimated using the following formula that is recommended in the "Bridge Design Standards of NEXCO (East Nippon Expressway Company Limited)":

$$L = a \times \{h+1/3(Df)\}$$

Where,

h = Substructure height

Df = Foundation depth

a = coefficient (1~1.5) depending on the construction circumstances of the proposed bridge

The construction circumstance of the proposed bridge is worse because of the existence of in-river piers. Consequently, a longer span length is more economical than one with a shorter length due to lesser number of in-river piers. Based on this viewpoint, the coefficient "a" should be 1.5. As a result of the economic span length estimation, it is determined that the economic span is 50.0 m.

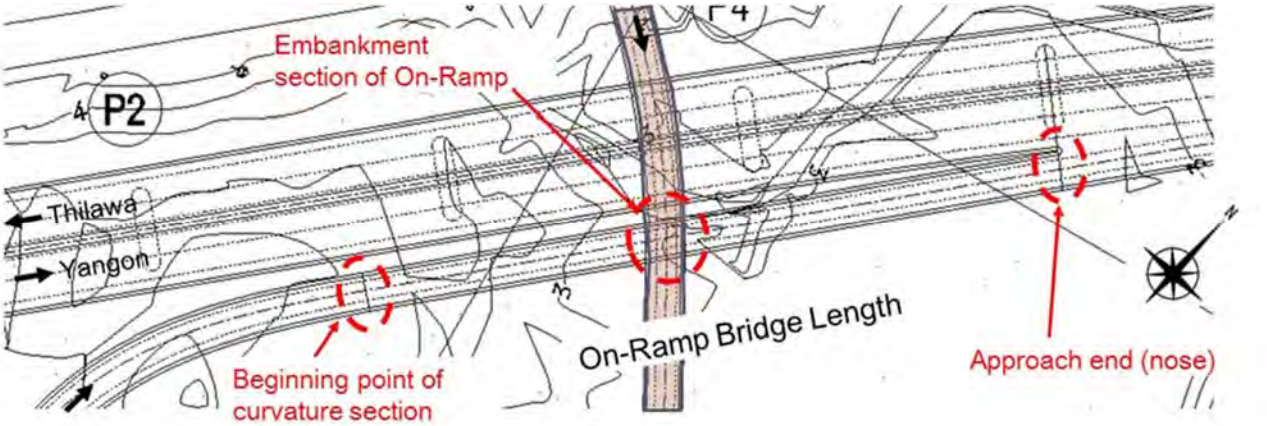
Table 4.5.3 Estimation of Economic Span Length

Item	On-ramp Bridge
Average substructure height: h	10.4 m
Average foundation depth: Df	53.1 m
h + 1/3 x Df	28.1 m
Case 1: a = 1.0	28.1 m
Case 2: a = 1.5	42.2 m
Proposed economic span length	**30.0 m**

Source: JICA Study Team

(4) 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

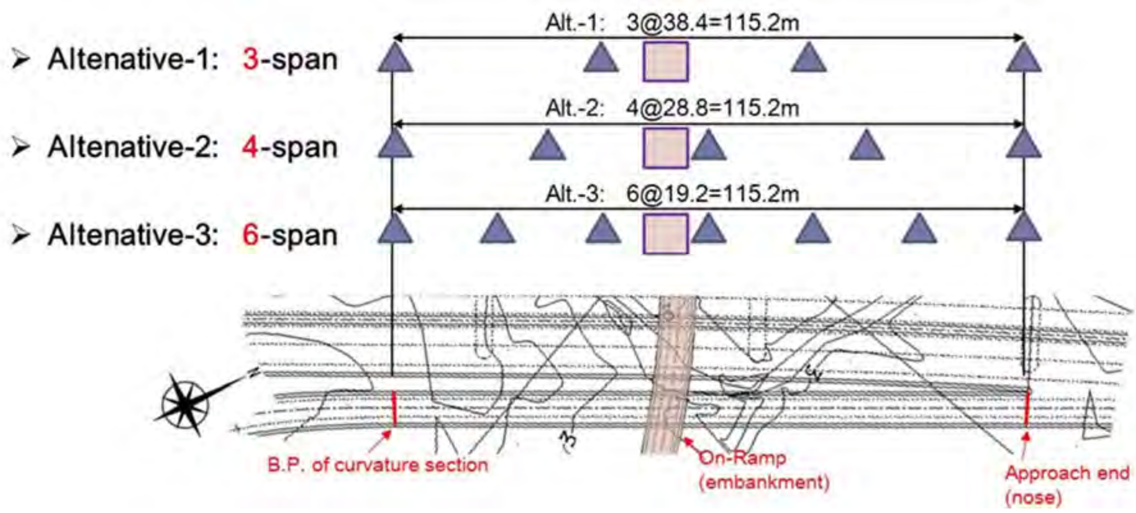
Beginning Point (Abutment) : STA No.0+410.000 (approximate station number)

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

4.5.1.3 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:

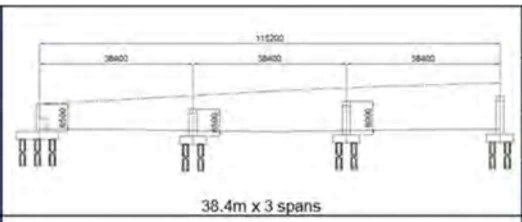
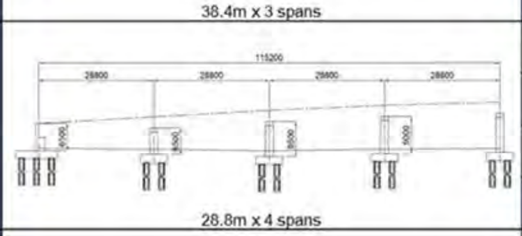
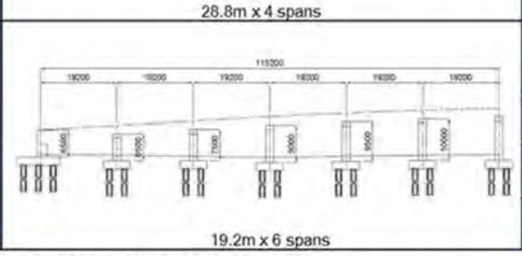


Source: JICA Study Team

Figure 4.5.2 Alternatives for Span Arrangement for On-ramp Bridge

- Comparison Result

Table 4.5.4 Comparison of Span Arrangement (On-ramp Bridge)

Alt-1		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		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		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

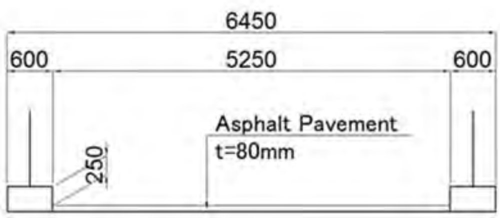
Source: JICA Study Team

4.5.2 Study on Superstructure of On-ramp Bridge

4.5.2.1 Selection of Type of On-ramp Bridge

(1) Basic Conditions

The on-ramp bridge is planned to be a 4-span continuous bridge with straight alignment and length of 115.2 m (4 spans x 28.8 m). Its width composition is shown below.



Source: JICA Study Team

Figure 4.5.3 Width Composition of On-ramp Bridge

The project site has soft grounds with insufficient bearing capacity, and soft soil treatment might be necessary in case conventional falsework is required to support heavy structure.

(2) 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

<u>Option-1: PC Hollow Slab</u>

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.

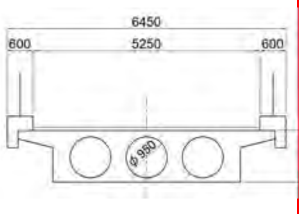
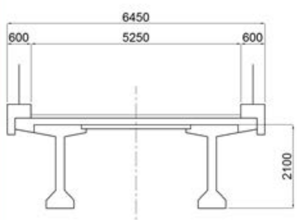
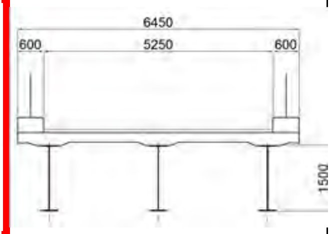
<u>Option-2: PC I Girder</u>

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.

<u>Option-3: Steel I Girder</u>

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

Table 4.5.5 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

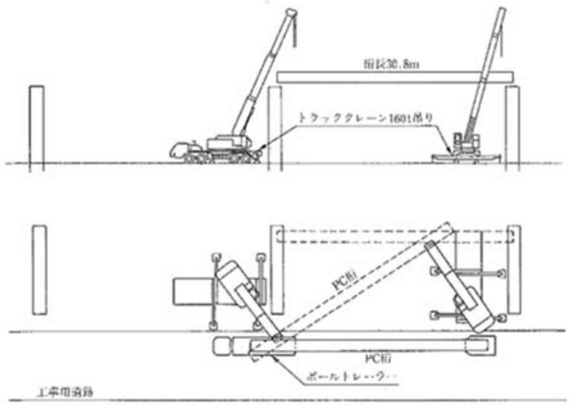
As a result of the study, PC I girder has been selected as the bridge type of the on-ramp bridge because of lowest cost, without need of conventional falsework, and superior in quality control. Girder fabrication yard can be prepared adjacent to the bridge.

4.5.2.2 Selection of Erection Method of On-ramp Bridge

PC I girder will be constructed based on the following procedure:

(1) Fabrication of girders → (2) Erection of girders → (3) Installation of PC panels → (4) Construction of crossbeams → (5) Construction of CIP slabs → (6) Longitudinal connections

The girders are planned to be erected by cranes as their number is small (8 girders). The girder weight is approximately 75 t per girder.



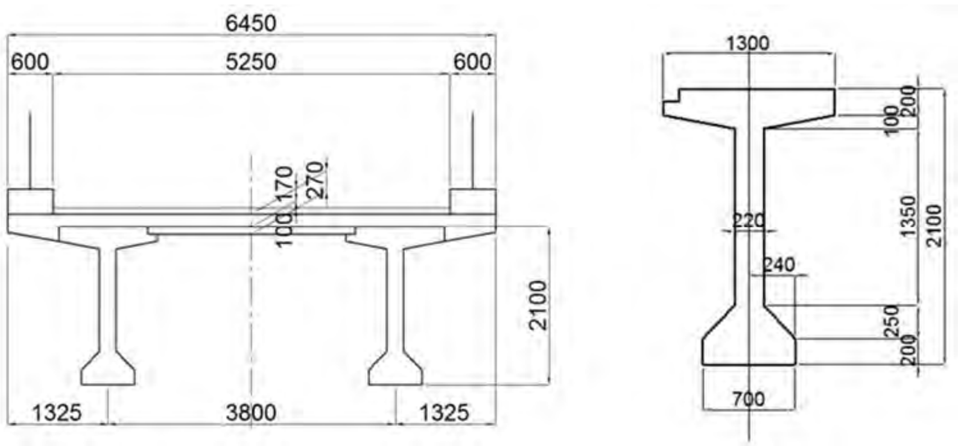
Source: "Guidebook for Preparation of Method Statement (for Simple Beams and Segmental Beams)" by Japan Prestressed Concrete Contractors Association

Figure 4.5.4 Girder Erection by Cranes (for Reference)

4.5.2.3 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.

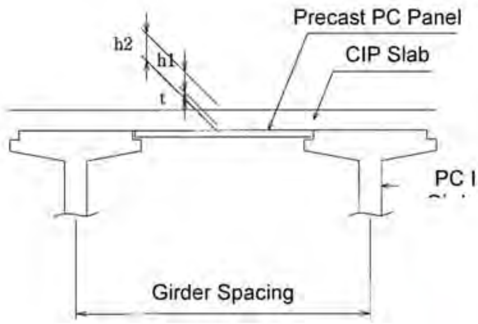


Source: JICA Study Team

Figure 4.5.5 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 4.5.6 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.

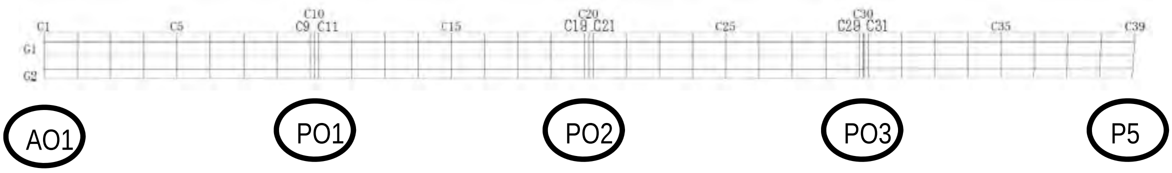
3) Tendons for Crossbeam Reinforcement

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.

4.5.2.4 Global Analysis

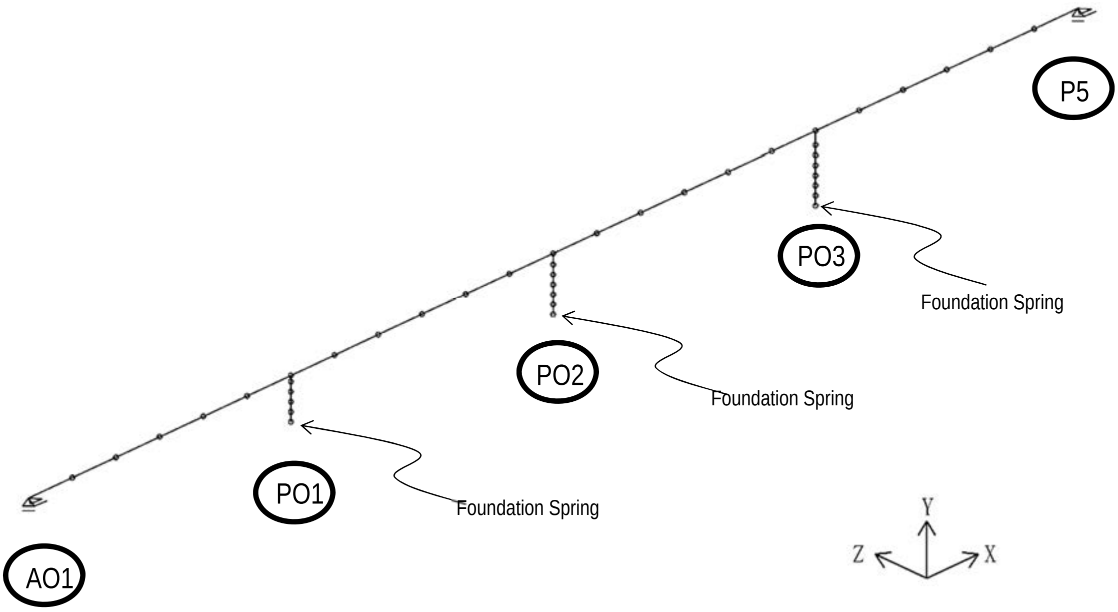
(1) Analysis models

In the global analysis of PC-I girder bridges, three different analysis models were used for normal loads and seismic loads, respectively. For normal loads, sectional forces due to self-weight were calculated using beam model, while sectional forces due to superimposed dead load and live load were calculated using plane grid models to take the load distribution to each girder into account, and superstructure and substructures were analyzed separately. In this analysis, sectional forces were calculated considering construction steps of superstructure (erection of I girders, construction of deck slab and crossbeams). For seismic actions, the analysis was performed using three-dimensional frame models, and distribution of seismic horizontal forces from superstructure acting on each substructure was calculated by the models in which superstructures and substructures are incorporated together.



Source: JICA Study Team

Figure 4.5.7 Analysis Model for Normal Loads (On-ramp Bridge)



Source: JICA Study Team

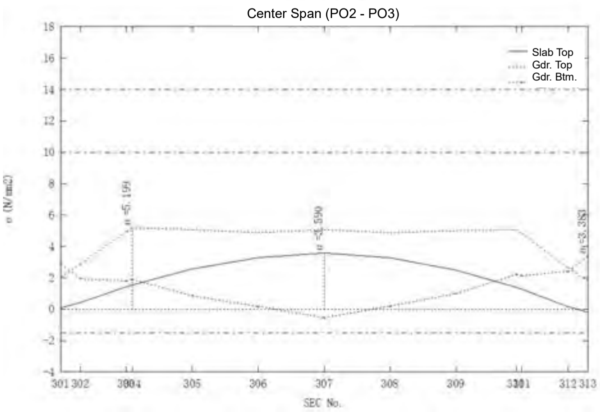
Figure 4.5.8 Analysis Model for Seismic Action (On-ramp Bridge)

4.5.2.5 Summary of the Detailed Design Results for Superstructure

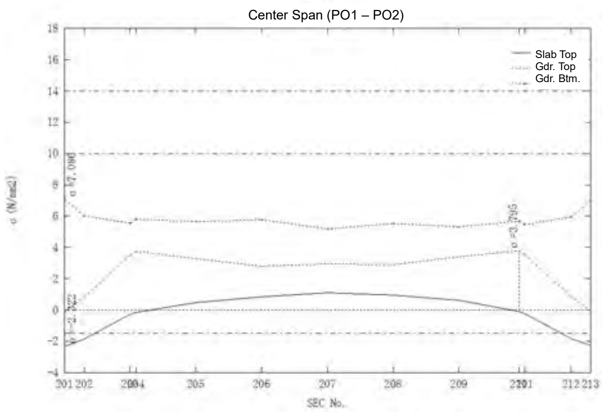
(1) AO1~P5

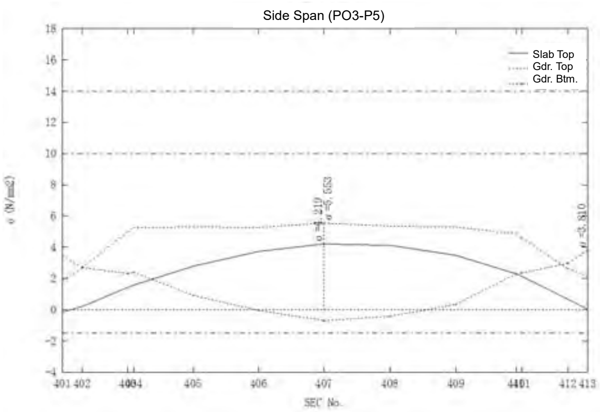
1) For Bending

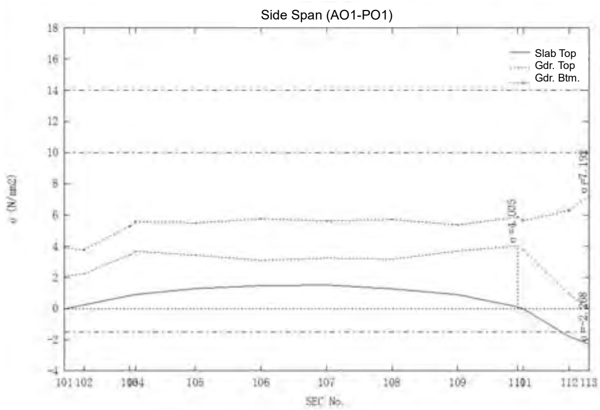
Stress for Service Load (Lmax)



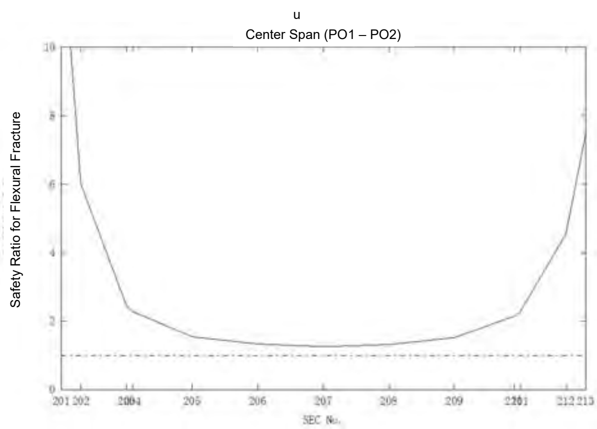
Stress for Service Load (Lmin)



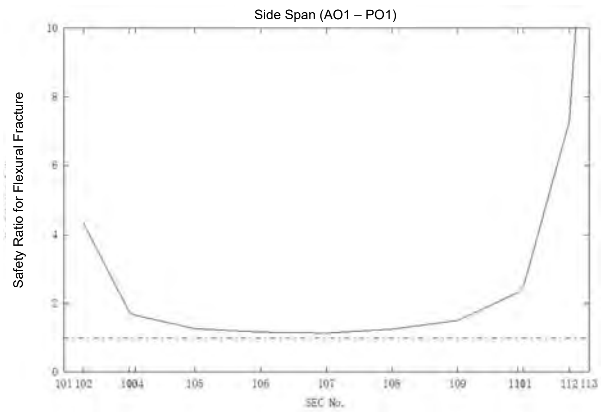




Safety Ratio for Flexural Fracture at Ultimate State (Center Span)



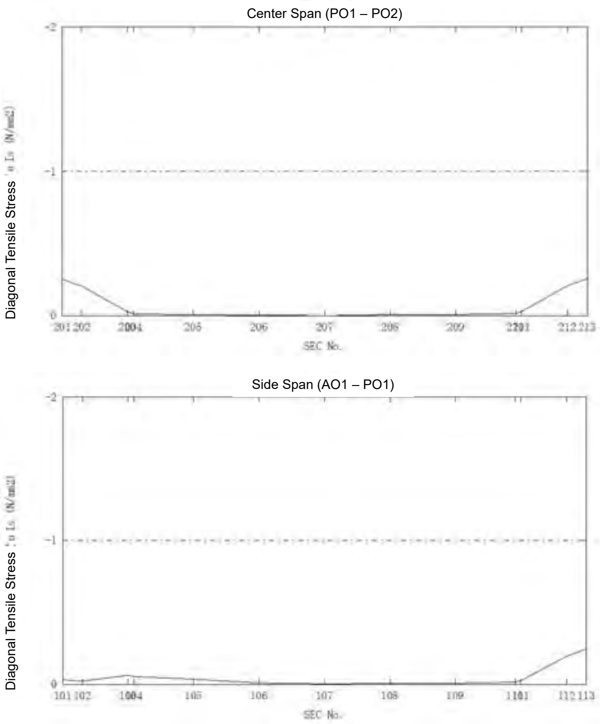
Safety Ratio for Flexural Fracture at Ultimate State (Side Span)



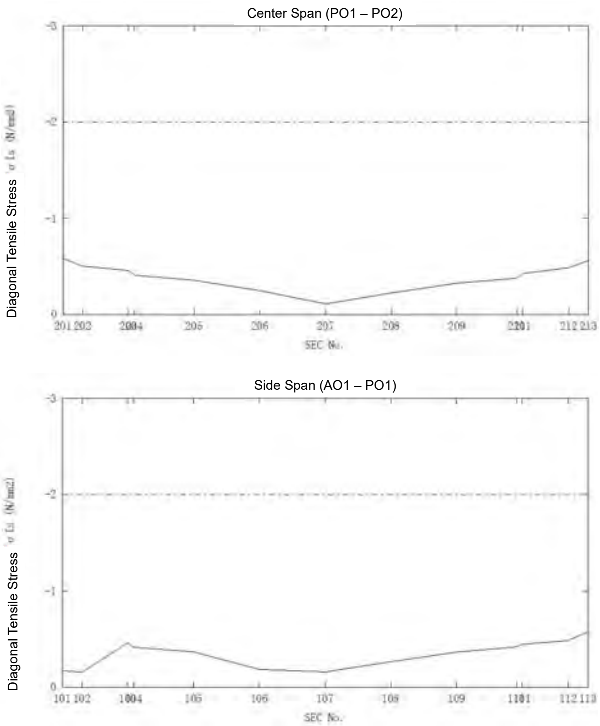
Source: JICA Study Team

2) For Shear

Diagonal Tensile Stress (Permanent Load)



Diagonal Tensile Stress (Service Load)



Source: JICA Study Team

4.5.3 Substructure of On-ramp Bridge

4.5.3.1 Design Principle

The review of the appropriateness of design outputs in the F/S was performed at the B/D stage for optimization of contents of facilities in relation to the bridge substructure with respect to structural types, dimensions, and number. Such optimizations were carried out with sufficient structural calculations and comparative studies in terms of economy, workability, constructability, and construction period.

Detailed structural analysis was performed at the D/D stage for updating design conditions and upgrading analytical accuracy based on the B/D results. Major updates in the D/D include natural conditions obtained from geographic survey and topographic survey and a proposed future ground elevation. Reaction forces of superstructure for design of substructure were also updated.

4.5.3.2 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 4.5.9.

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.

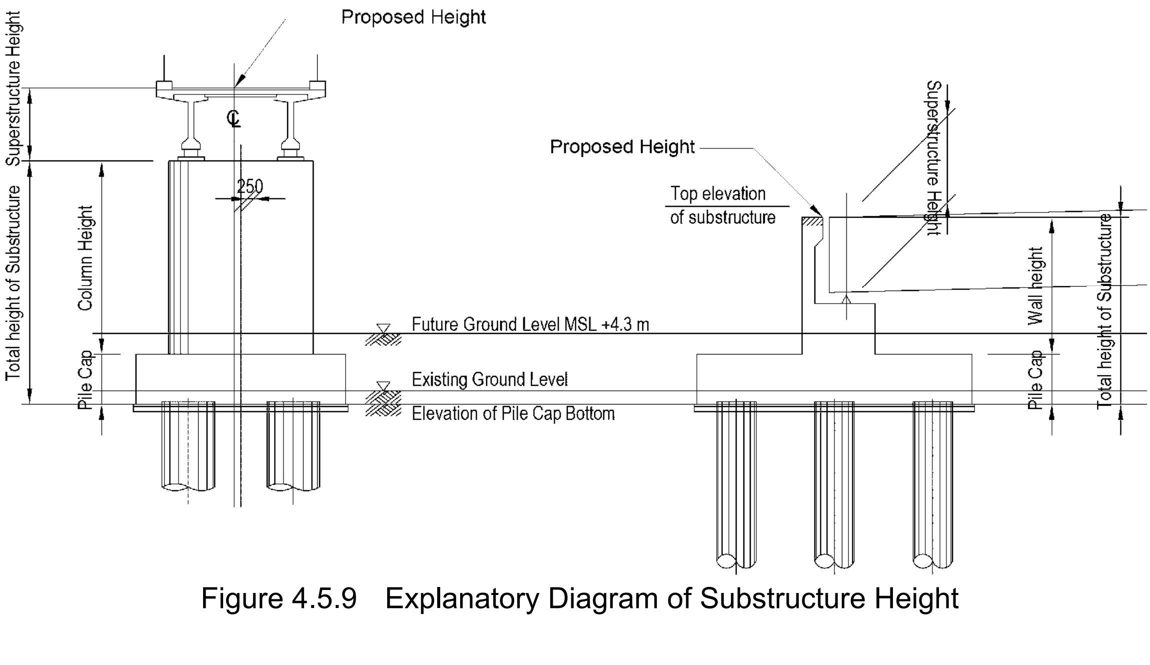


Figure 4.5.9 Explanatory Diagram of Substructure Height

(2) Planned Depth of Overburden above Pile Cap

The depth of overburden above pile cap shall be secured sufficiently with reference to the planned future ground level. Amount of the overburden was around 0.5 m. The amount of overburden shall be altered at respective substructure locations in case of necessity such as the arrangement of buried conduit, surface roads, etc.

In the B/D, the amount of overburden was set at 1.0 m as the default plan subject to updating based on the topography survey result, buried utility survey, and determination of the future ground level in the D/D stage.

In the D/D, it was confirmed that there would be no buried utilities around the substructures both at present and in the future, other than the drainage lines that are to be constructed under this Project for discharge of rainwater on the bridge. Also, it was determined that future ground level would be MSL+4.300 m as mentioned in the foregoing. Thus, the amount of overburden was set at 0.5 m.

(3) Conclusion of Substructure Heights

The conclusions on substructure heights are presented in Table 4.5.6.

Table 4.5.6 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

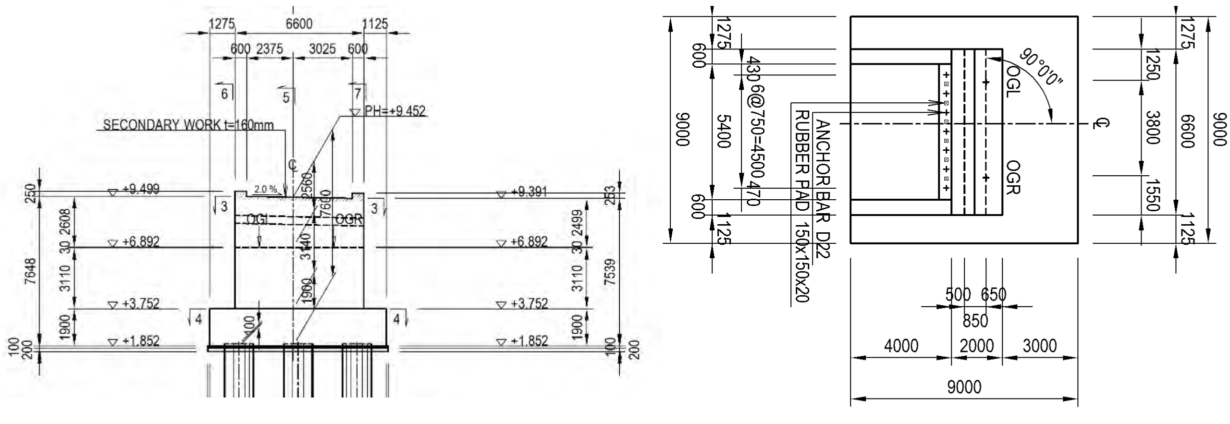
Source: JICA Study Team

4.5.3.3 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 4.5.10 Abutment Width

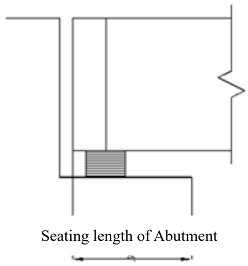
(2) Bridge Seat

The bridge seat shall have sufficient space for arrangement of bridge bearings that support the superstructures. The bridge shall also have a space that works to prevent the unseating of the bridge in case of the occurrence of unexpected seismic force, displacement or deformation occurring in the bridge caused by unpredicted earthquake ground motion, destruction of the surrounding ground, or unexpectedly complicated vibration in the structural members.

As for unseating prevention in the bridge axis, the seating length is to be provided at the terminal supports and the halving joints. For intermediate piers, the seating length is not required because the bridge is a continuous bridge that has continuous girders. As for the unseating prevention in the transverse direction to the bridge axis, anchor bars are installed as displacement constraint structures.

- Determination of Seating Length (S_{EM})

The seating length should be long enough to prevent departure and unseating of the superstructure from the top of the substructure. Value of the seating was obtained from the equation below as specified in the JSBH.



Source: JICA Study Team

Figure 4.5.11 Unseating Length

$SEM = 0.7 + 0.005\ell$

Where,

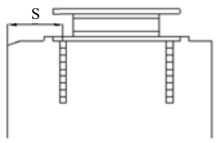
ℓ: Length of the effective span (m). When two superstructures with different span lengths are supported on one bridge pier, the longer of the two shall be used. ℓ = 28.8 m for AO1.

$S_{EM} = 0.7 + 0.005 \times 28.800$

$= 0.844$ (m)

- Determination of Bearing Edge Distance (S)

The bearing edge distance, which is defined as the distance between the edge of the bearing and the edge of the top of the substructure (or bearing support edge distance) shall be equal to or larger than the following value:



Source: JICA Study Team

Figure 4.5.12 Bearing Edge Distance

$S \geqq 0.2 + 0.005\ell$

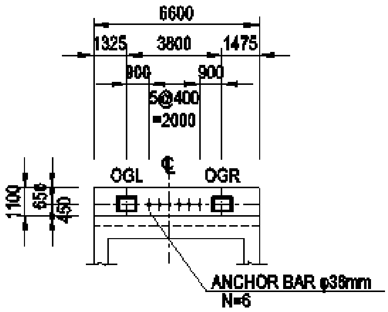
$= 0.344$ (m)

The results of the checking of bridge seat dimensions are summarized in Table 4.5.7.

Table 4.5.7 Results of Bridge Seat Width (On-ramp)

Unit : mm		AO1	PO1	PO2	PO3	P5
Span Length		28,800	28,800	28,800	28,800	28,800
Width of Bridge Seat		1,100	1,500	1,500	1,500	2,250
		6,450	5,500	5,500	5,500	25,000
Anchor Bolt (Fixing bolts of bearings)	B_{LL}	420	420	420	420	420
	B_{TT}	620	620	620	620	620
	c.t.c.	3,800	3,800	3,800	3,800	3,800
Anchor Bar (Displacement Constraint Structure)	φ	36	75	50	46	36
	nos.	6	6	6	6	6
	c.t.c.	500	500	500	500	500
Edge Distance from "Anchor Bar"	LL	650	750	750	750	450
	TT	1,975	1,500	1,500	1,500	1,501
	Skew	919	1,061	1,061	1,061	636
Minimum Edge Distance		**344**	**344**	**344**	**344**	**344**
Seating Length of Girder		1,000	-	-	-	1,000
Minimum Seating Length of Girder		**844**	**844**	**844**	**844**	**844**

Source: JICA Study Team



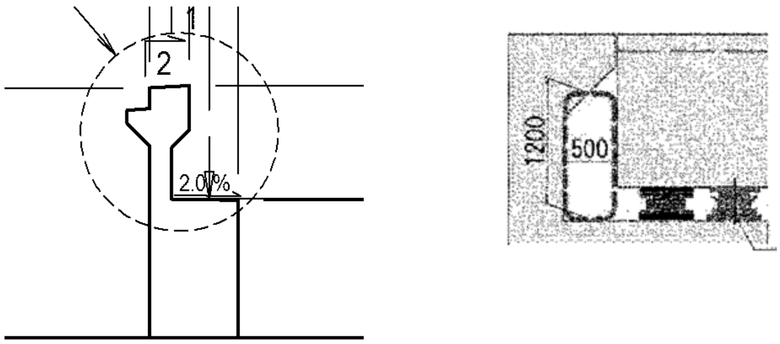
Source: JICA Study Team

Figure 4.5.13 Layout of Bridge Seat for Abutment (A1 and A2)

(3) Space for Maintenance

As for structural details which may contribute to the prolongation of the bridge lifespan, bridge seats shall be graded by around 2% in order to avoid puddle on the bridge seat.

Moreover, a space for ventilation should be provided at the terminal support. For this purpose, a distance of 500 mm was secured. This space will also be utilized for inspections of bearings. Schematic illustration is shown below.



Source: JICA Study Team (based on "Manual for Design and Construction – Bridges" by Tohoku Regional Bureau, MLIT, Japan)

Figure 4.5.14 Space for Maintenance

4.5.3.4 Dimensions of Pier

(1) Bridge Seat

The bridge seat shall have sufficient space for arrangement of bridge bearings that support the superstructures. The bridge shall also have a space that works to prevent the unseating of the bridge in case of the occurrence of unexpected seismic force, displacement or deformation occurring in a bridge caused by unpredicted earthquake ground motion, destruction of the surrounding ground, or unexpectedly complicated vibration in the structural members.

As for the unseating prevention in the bridge axis, the seating length is to be provided at the terminal supports and the halving joints. As for the unseating prevention in the transverse direction to the bridge axis, anchor bars are installed as displacement constraint structures.

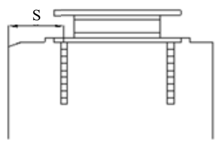
The examination of the seating length for P5 pier, which is the other side of the terminal support, should be referred to the relevant section of PC concrete bridge. For other intermediate piers, such seating length is unnecessary because the on-ramp bridge is a continuous bridge and there is no possibility of unseating in the bridge axis.

- Determination of Seating Length (S_{EM})

Not required.

- Determination of Bearing Edge Distance (S)

The bearing edge distance, which is defined as the distance between the edge of the bearing and the edge of the top of the substructure (or bearing support edge distance) shall be equal to or larger than the following value:



Source: JICA Study Team

Figure 4.5.15 Bearing Edge Distance

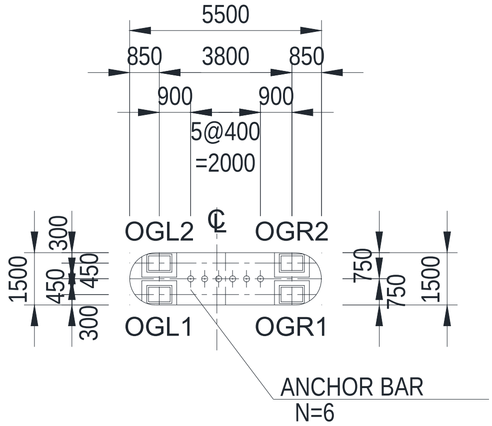
$$S \geqq 0.2 + 0.005\ell$$
$$= 0.344 \text{ (m)}$$

Where,

ℓ: Length of the effective span (m). When two superstructures with different span lengths are supported on one bridge pier, the longer of the two shall be used. The ℓ for PO1 through PO3 is 28.8 m.

The bridge seat dimensions, considering the above, are summarized in Table 4.5.7.

The layout of the bridge seat of Piers PO1 through PO3 is shown in Figure 4.5.16.



Source: JICA Study Team

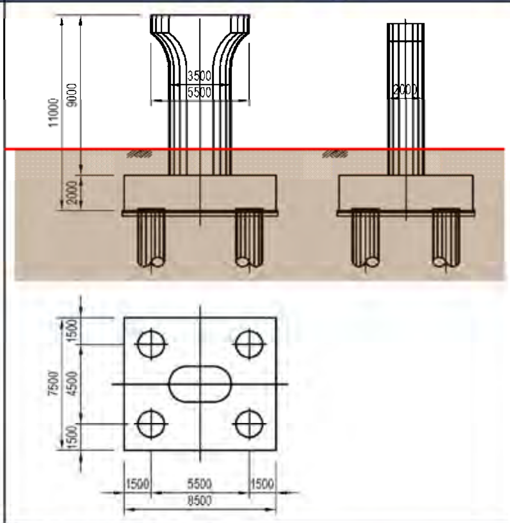
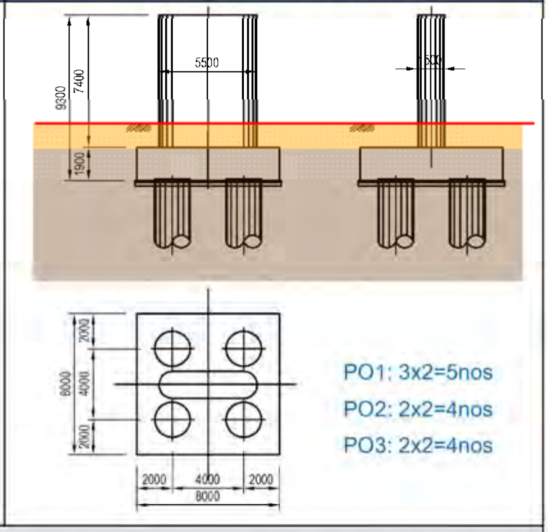
Figure 4.5.16 Layout of Bridge Seat for Piers P01 through P03

(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 4.5.8.

Table 4.5.8 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 4.5.9.

Table 4.5.9 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 Design Principle

The review of the appropriateness of the design outputs in the F/S was performed in the B/D stage for optimization of contents of facilities in relation to the bridge foundation with respect to the structural types, dimensions, and number. Such optimizations were carried out with sufficient structural calculations and comparative studies in terms of economy, workability, constructability, and construction period.

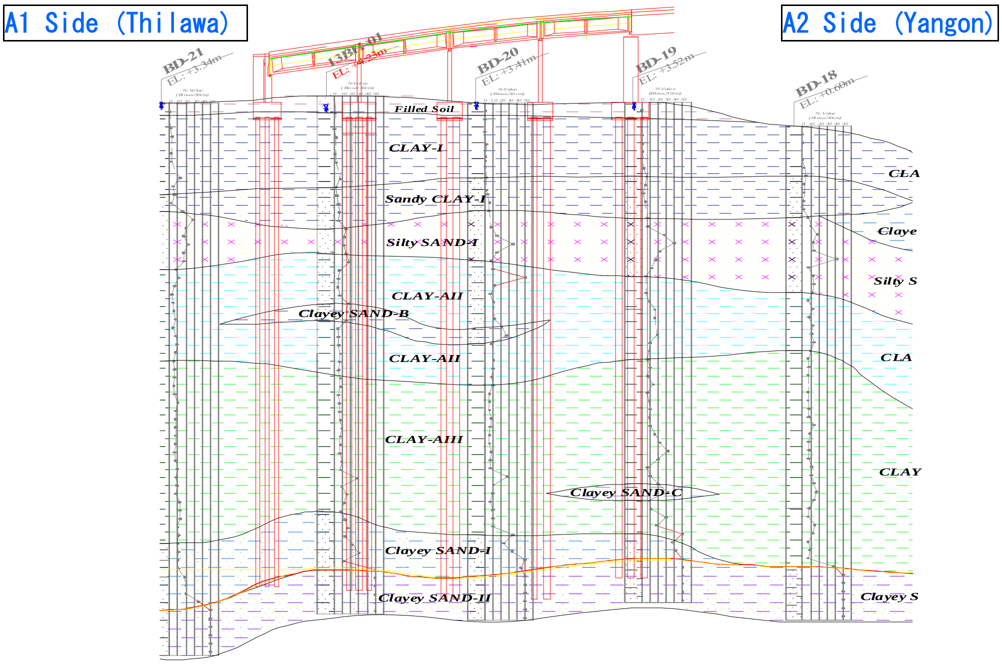
Detailed structural analysis was performed in the D/D stage for updating the design conditions and upgrading analytical accuracy based on the B/D results. Major updates in the D/D include natural conditions obtained from the geographic survey and topographic survey, and the proposed future ground elevation. Reaction forces of superstructure for design of substructure were also updated.

4.5.4.2 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 4.5.17 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 4.5.10 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

4.5.4.3 Evaluation of Geotechnical Parameters for Design

Evaluation of geotechnical investigation results was performed by the geological specialists of the Project with unified viewpoint and was described in detail at the relevant section of this report. With regard to the evaluation of the geotechnical parameters for bridge design, bridge designers should take into account the specific features of each bridge location in deference to outputs of the geological specialists. In this sub-section, the modulus of deformation of soils and reduction factor (D_E) for geotechnical parameters due to liquefaction, that have a profound effect on the design of bridge foundation, are reported.

Generally, the displacement of foundations largely depends on the behavior of the weak sections of the bearing ground. In addition, the horizontal displacement of foundations with respect to the loads acting on the top of the foundations vary with the properties of the surface layers. According to the geotechnical investigation results that had been conducted during the B/D, the modulus of deformation of soils shall be reduced generally compared with the ones obtained in the F/S and used in the B/D. It is remarked that the modulus of surface layers should be decreased approximately by 50% based on the results. Additionally, it was discovered that the effects of liquefaction should be considered more seriously in the D/D than in the F/S and the B/D. Since a single soil layer at A2 (Yangon) side was considered as a liquefaction layer in the F/S-B/D, the number of soil layers and scale of reduction for geotechnical parameters due to liquefaction should be increased in the D/D. The summary of the modulus of deformation of soils and reduction factor (D_E) is displayed in Table 4.5.11.

Overall, it was confirmed that the properties of the surface layers against deformation were weaker than those in the F/S-B/D, and the fact brought increments of pile number and/or pile diameters for assurance of structural stability.

Table 4.5.11 Comparison of Modulus of Deformation of Soil for On-ramp Bridge (B/D vs D/D)

Str. No.	AO1			PO1			PO2			PO3		
Br No.	No13BH-01			No13BH-01			BD-20			BD-20		
Depth	Soil type	BD	DD	Soil type	BD	DD	Soil type	BD	DD	Soil type	BD	DD
1	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A	Filled Soil	N=2 E=1400 D_E=N/A	N=1 E=700 D_E=N/A
2												
3	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A	CLAY-I	N=1 E=1800 D_E=N/A	N=1 E=900 D_E=N/A
4												
5												
6												
7												
8							Sandy CLAY-I	N=5 E=2500 DE=N/A	N=3 E=2000 D_E=1/3	Sandy CLAY-I	N=5 E=2500 DE=N/A	N=3 E=2000 D_E=1/3
9												
10									D_E=2/3			D_E=2/3
11	SandyCLAY-I	N=5 E=2500 D_E=N/A	N=3 E=2000 D_E=N/A	SandyCLAY-I	N=5 E=2500 D_E=N/A	N=3 E=2000 D_E=N/A			D_E=1			D_E=1
12												
13							Silty SAND-I	N=14 E=9800 D_E=1	N=15 E=6000 D_E=1	Silty SAND-I	N=14 E=9800 D_E=1	N=15 E=6000 D_E=1
14												
15												
16												
17	Silty SAND-1	N=14 E=9800 D_E=1	N=15 E=6000 D_E=1	Silty SAND-1	N=14 E=9800 D_E=1	N=15 E=6000 D_E=1						
18												
19												
20	SaCL-II	D_E=N/A	D_E=N/A	SaCL-II	D_E=N/A	D_E=N/A	CL-AII	D_E=N/A	D_E=N/A	CL-AII	D_E=N/A	D_E=N/A

Source: JICA Study Team

4.5.4.4 Estimation of Down Drag Zone

Occurrence of down drag due to reclamation for preparation of construction yard whose finished elevation is MSL+4.300 m was anticipated. Accordingly, the depth of the down drag zone was analyzed using laboratory test results of soil samples obtained in the D/D. Analysis results shown in Table 4.5.12 were utilized for the design of the foundation.

Table 4.5.12 Assessment Result of Down Drag

3. ON Ramp Bridge: PC-T Girder Bridge							
Item	Mark	Unit	AO1	PO1	PO2	PO3	PO4 (P5)
Station Number	STA	m	0+411.009	0+439.809	0+468.609	0+497.409	0+526.209
Existing Ground EL	GL1	m	3.281	2.936	2.959	3.076	3.149
Future Ground EL	GL	m	4.300	4.300	4.300	4.300	4.300
Foundation Type	−	−	CIP Pile	CIP Pile	CIP Pile	CIP Pile	CIP Pile
Pile Length	L	m	56.500	57.000	57.500	58.000	55.500
Downdrag Zone	EL	m	−11.8	−11.8	−9.6	−9.6	−10.5
Reference Boring No.	−	−	*BH-01*	*BH-01*	*BD20*	*BD20*	*BD19*
Bearing Stratum	−	−	*CS-II*	*CS-II*	*CS-II*	*CS-II*	*CS-II*

Source: JICA Study Team

4.5.4.5 Selection of Foundation Type

(1) Design Policy

Bridge foundation type of the on-ramp bridge should be selected through a comprehensive comparative study, in which economy, workability, constructability, and construction period are examined.

As a result of the comparative study in the B/D, cast-in-place (CIP) pile foundation (pile diameter = 1.500 m) was recommended as the foundation type of the on-ramp bridge.

Updating of design conditions and upgrading of analytical accuracy were made to the above B/D results. Then, when necessary, reconsideration of pile arrangement including the selection of pile diameter was performed in the D/D for optimization.

(2) Design Conditions

Design conditions applied for B/D are summarized as follows:

- Bearing stratum is Clayey Sand-II at around MSL-50.0 m
- Design reaction from superstructure is moderate due to superstructure type (concrete I girder) and span length (30.0 m)
- Groundwater level is high (close to ground surface)
- Representative substructure height of pier employed for this study is 8 m for on-land pier
- Representative substructure height of abutment employed for this study is 9 m
- Unit costs of bridgework employed for this study are reconfigured based on the ones used in the F/S.

(3) Selection of Foundation Type at B/D

On-land foundations are constructed after ground preparation that includes reclamation.

Prefabricated concrete pipe pile types, such as PHC (Pretensioned Spun High Strength Concrete) pile, with diameter of 600 mm or smaller are procurable in Myanmar. However, these diameters are too small against bridge scale and seismic force. Moreover, only the percussion method is procurable in Myanmar, whereas, inner excavation method is necessary in terms of the required driving depth and penetration against a relatively firm intermediate sandy soil layer. In order to overcome these situations, offshore procurement of large-diameter PHC piles with inner excavation drilling machines is one of the options, but less economical than adoption of CIP pile that is locally procurable. As mentioned above, prefabricated concrete pipe pile types were excluded from this comparative study of foundation type.

Regarding steel pipe pile foundation, piles with a diameter of 600 mm or smaller are procurable in Myanmar. However, these diameters are also too small against bridge scale and seismic force. Moreover, only the percussion method is procurable in Myanmar, whereas, inner excavation method is necessary in terms of the required driving depth and penetration against a relatively firm intermediate sandy soil layer. In order to overcome these situations, the offshore procurement of large-diameter steel piles with inner excavation drilling machines is one of the options similar to the PHC pile case, but less economical than the adoption of the CIP pile that is locally procurable. As mentioned above, steel pipe pile foundation was excluded from this comparative study of foundation type in the B/D.

About the CIP pile foundation, reverse circulation drilling method is suitable for the required borehole depth. Based on the bridge scale and seismic force, pile diameters for comparative study were 1.2 m, 1.5 m, and 2.0 m. Procurement and construction plan should be referred to the relevant chapters of this report. It was confirmed as a result of the study that "CIP pile foundation (reverse circulation drilling method) D = 1.5 m" was the most advantageous foundation type among the alternatives in terms of economy, workability, and construction period. This foundation type was adopted for abutments and piers (on-land). The summary of these comparative studies is shown in Table 4.5.14 and Table 4.5.15.

(4) Review Policy of Foundation Type in D/D

Updating of design conditions and upgrading of analytical accuracy were conducted as mentioned in the design principle. Additionally, the overlap of pile cap and embankment section of the on-ramp road in the plan (*see Figure 4.5.18*) should be confirmed.

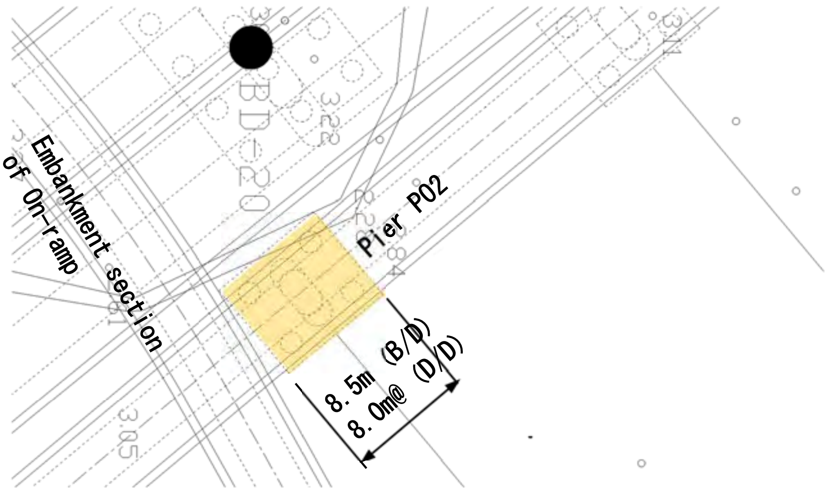
Soil parameter	To apply updated soil parameters that were obtained after the B/D. It is revealed that deformation coefficients of soils were smaller than the ones used in the B/D. Additionally, the scale of liquefaction effect for foundation design is larger than that expected in the B/D.
Ground elevation and structural height	Amount of the overburden depth is 0.5 m in the D/D, whereas it was 1.0 m in the B/D. The foundation level of on-land substructures was determined while taking into account the ground level for construction (MSL+4.300 m). Due to these changes, structural heights were shortened by 1.5~2.0 m
Analytical accuracy	Implementation of global analysis under the updated conditions and improvement of analytical accuracy adequately as D/D.
Overlap in plan position	For securing the flatness of road surface of embankment section of on-ramp road and flexibility in the construction schedule, the pile cap of PO2 pier should not overlap with the embankment road.

(5) Review Results

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 4.5.13. Result of this review is shown in Table 4.5.16 and Table 4.5.17.

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 4.5.18 Crossing Point of Embankment Road of On-ramp and PO2 Pier

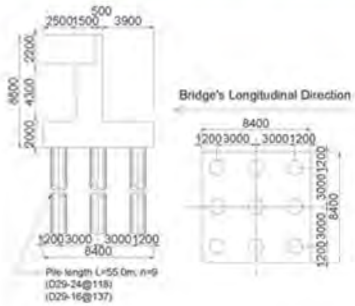
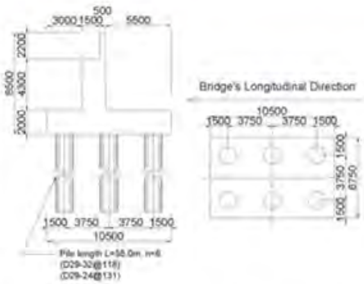
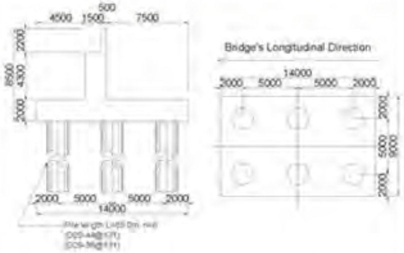
Table 4.5.13 Available Pile Cap Size and Costs of On-ramp Pier

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

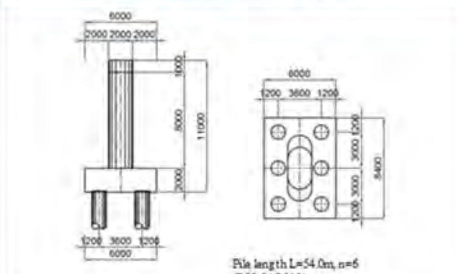
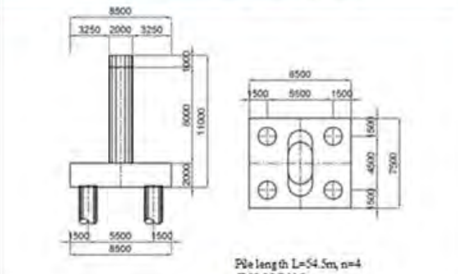
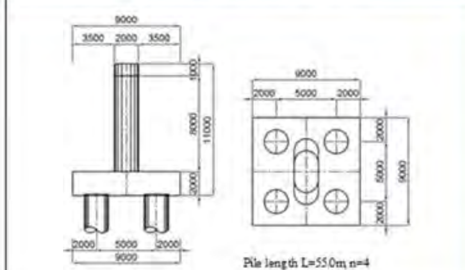
Table 4.5.14 Comparison of Foundation Type for On-ramp Bridge Abutment

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	1,574.9	2,622.0	2,369.2	3,645.4	4,173.2	4,947.2
		Ra	kN	6,161.0	9,421.0	7,790.0	11,966.0	10,606.0	16,406.0
		σ/σa	-	0.3	0.3	0.3	0.3	0.4	0.3
	Amount of Displacement	σx	mm	4.9	11.4	4.4	11.7	2.9	11.3
		σxa	mm	15.0	15.0	15.0	15.0	15.0	15.0
		R	-	0.3	0.8	0.3	0.8	0.2	0.8
	Stress of a Pile	σs	N/mm^2	35.7	218.0	21.3	193.6	20.7	183.1
		σsa	N/mm^2	160.0	300.0	160.0	300.0	160.0	300.0
		σ/σa	-	0.2	0.7	0.1	0.6	0.1	0.6
	Maximum Stress of a Pile			σs= 217.96 kN/m^2 <σsa = 300 kN/m^2 (OK)		σs= 193.63 kN/m^2 <σsa = 300 kN/m^2 (OK)		σs= 183.11 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	△	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 large. (1.5Months)	○	The amount of pile works including ground excavation is consirably the smallest. (1.3Months)	◎	The amount of pile works including ground excavation is consirably smaller. (1.6Months)	△
Environmental Aspect				This alternative entails the largest amount of excavation works.	△	This alternative entails the smallest amount of excavation works.	◎	This alternative entails small amount of excavation works.	○
Cost Ratio				1.185	○	1.000	◎	1.396	△
Evaluation				○		◎		△	

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

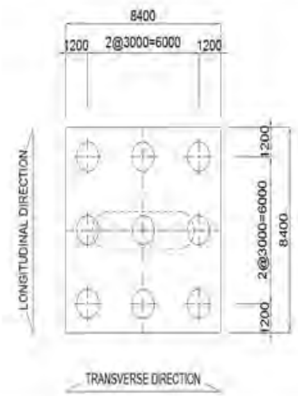
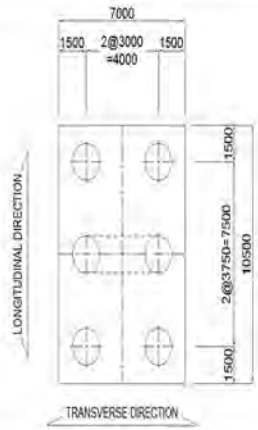
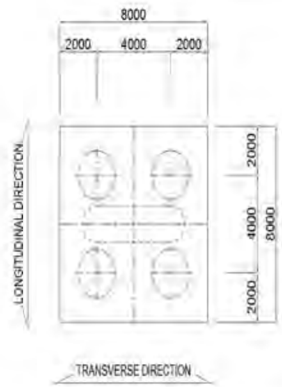
Source: JICA Study Team

Table 4.5.15 Comparison of Foundation Type for On-ramp Bridge Piers

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				 Pile length L=54.0m, n=6 (D32-24@115)				 Pile length L=54.5m, n=4 (D32-32@116)				 Pile length L=55.0m n=4 (D29-44@120)			
	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
Design Results	Maximum Pile Reactions	Pmax	kN	1,734.9	3,810.8	1,734.9	3,464.3	2,826.0	5,084.3	2,826.0	5,079.2	3,114.9	5,573.9	3,114.9	5,210.4
		Ra	kN	6,041.0	9,243.0	6,041.0	9,243.0	7,737.0	11,844.0	7,737.0	11,844.0	10,781.0	16,684.0	10,781.0	16,684.0
		σ/σa	-	0.3	0.4	0.3	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.3	0.3
	Amount of Displacement	σx	mm	0.0	12.9	0.0	9.1	0.0	13.1	0.0	12.6	0.0	11.4	0.0	9.8
		σxa	mm	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
		R	-	0.0	0.9	0.0	0.6	0.0	0.9	0.0	0.8	0.0	0.8	0.0	0.7
	Stress of a Pile	σs	N/mm^2	-18.4	236.0	-18.4	186.7	-19.7	222.2	-19.7	192.5	-13.1	165.3	-13.1	205.5
		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	-	0.1	0.8	0.1	0.6	0.1	0.7	0.1	0.6	0.1	0.6	0.1	0.7
	Maximum Stress of a Pile			σs= 235.95 kN/m^2<σsa= 300kN/m^2 (OK)				σs= 222.22 kN/m2<σsa= 300kN/m2 (OK)				σs= 205.53 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				1.140			○	1.000			◎	1.348			△
Judge				○				◎				△			

Source: JICA Study Team

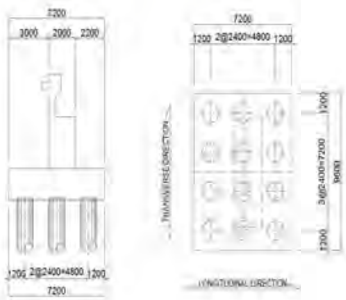
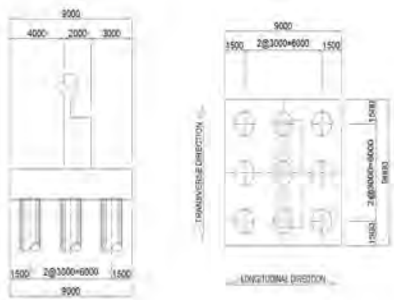
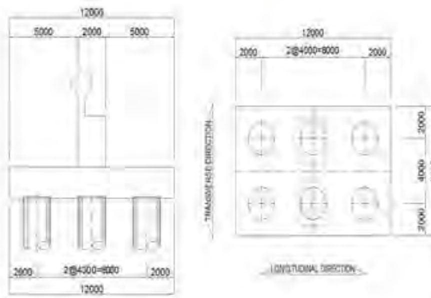
Table 4.5.16 Review of Foundation Type for On-ramp Piers in the D/D

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 4.5.17 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									
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	1,211.9	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/mm^2	70.3	260.4	54.3	255.9	48.2	264.0
		σsa	N/mm^2	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/m^2 <σsa = 300 kN/m^2 (OK)		σs= 256 kN/m^2 <σsa = 300 kN/m^2 (OK)		σs= 264 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 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 Load Combinations

Load combinations for design of substructures and foundations are displayed in Table 4.5.18 which shall comply with the specifications in the JSHB. It is remarked that load situation relating to "extreme wind situation" was not applied to the PC box girder bridge because the amount of wind load on the concrete structures was quite small in comparison with the seismic inertia force.

Table 4.5.18 Load Combinations for Design of On-ramp Bridge

Load Combinations	Load Situations	(1) Dead Loads (D)	(2) Live Loads (L)	(3) Impact (I)	(4) Prestress Force (PS)	(5) Influence of Creep of Concrete (CR)	(6) Influence of D dry Shrinkage of Concrete (SH)	(7) Earth Pressure (E)	(8) Water Pressure (HP)	(9) Buoyancy (U)	(10) Wind Loads (W)	(11) Effect of Temperature Change (T)	(12) Seismic Effects (EQ)	(13) Collision of Vessel (CO)	Increase Coefficients
1. Principal loads (P) + particular loads corresponding to principal loads (PP)	Ordinary condition	○	○	○	○	○	○	○	○	○					1.00
		○	○	○	○	○	○	○	○	○					1.00
		○	○	○	○	○	○	○	○	○					1.00
		○	○	○	○	○	○	○	○	○					1.00
2. Principal loads (P) + particular loads corresponding to principal loads (PP) + effects of temperature change (T)		○	○	○	○	○	○	○	○	○		○			1.15
		○	○	○	○	○	○	○	○	○		○			1.15
		○	○	○	○	○	○	○	○	○		○			1.15
		○	○	○	○	○	○	○	○	○		○			1.15
3. Principal loads (P) + particular loads corresponding to principal loads (PP) + wind loads (W)	Extreme Wind	○	○	○	○	○	○	○	○	○	○				1.25
		○	○	○	○	○	○	○	○	○	○				1.25
4. Principal loads (P) + particular loads corresponding to principal loads (PP) + effects of temperature change (T) +wind loads (W)		○	○	○	○	○	○	○	○	○	○	○			1.35
		○	○	○	○	○	○	○	○	○	○	○			1.35
5. Principal loads (P) + particular loads corresponding to principal loads (PP) + vessel collision loads (CO)	Vessel Collision	○	○	○	○	○	○	○	○	○				○	1.50
		○	○	○	○	○	○	○	○	○				○	1.50
6. Principal loads except live loads and impacts + seismic effects (EQ)	Earth-quake	○			○	○	○	○	○	○			○		1.50
		○			○	○	○	○	○	○			○		1.50

Source: JICA Study Team

4.5.5.2 Reaction Forces of Superstructure for Design of Substructures

The values of reaction force transmitted from the superstructure to the substructure are summarized in Table 4.5.19.

Table 4.5.19 Reaction Forces of Superstructures (On-ramp Bridge)

Descriptions					Package-1 : On-Ramp				
					AO1	PO1	PO2	PO3	PO4
Bearing Conditions (**M**: Movable, **F**: Fixed, **E**: Elastic support)					M	F	F	F	M
Working Height Above Bridge Seat	For Bridge Axis direction			m	0.000	0.000	0.000	0.000	0.000
	For Transverse Direction			m	1.900	1.900	1.900	1.900	2.500
V	Dead Loads		①	kN	2000	4000	4200	4200	2000
	Live Loads	Max	②	kN	600	1200	1100	1200	600
		Min	③		-100	-100	-200	-100	-100
H	Influence of dry shrinkage of concrete		④	kN	110	480	-100	-390	-110
	Influence of creep of concrete		⑤	kN	50	190	-40	-150	-50
	Effect of temperature change (+)		⑥	kN	-100	-440	90	350	100
	Effect of temperature change (-)		⑥	kN	100	440	-90	-350	-100
	Seismic effects	Longitudinal	⑦	kN	300	2650	1250	900	300
		Transversal	⑧	kN	550	1300	1300	950	750
M	Eccentric moment due to Dead Load	Longitudinal	⑨	kN.m	0	0	0	0	0
		Transversal	⑩	kN.m	0	0	0	0	0

Source: JICA Study Team

4.5.5.3 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 4.5.20.

4.5.5.4 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.

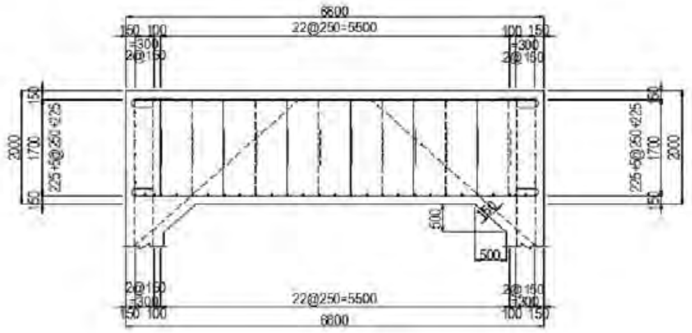
The wing wall shall be designed as slabs to receive superimposed loads due to live loads and earth pressure. The slab in this case shall be cantilever fixed on two sides to a wall and footing.

The calculation results for the wall of the reverse T-shaped abutment are shown in Table 4.5.20.

Table 4.5.20 Calculation Results for Wall and Columns (AO1, PO1~PO3)

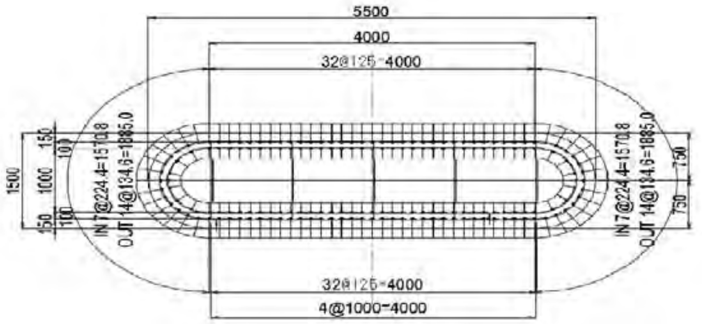
Cross Section of Column (Longitudinal Direction)

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

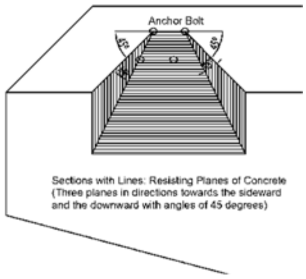
Note :
σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force
R-ratio ; Design result / Capacity

Source: JICA Study Team

4.5.5.5 Design of Bridge Seats

Bridge seats shall be designed with sufficient strength to withstand the vertical and horizontal forces from bearings. Bridge seats should be designed so that corrosion of bearing and girders can be minimized.

Horizontal forces transmitted from bearings are carried by concrete and reinforcement. The resisting area of concrete is the summation of the three planes in the sideward and downward directions with edge angles of 45 degrees as shown in Figure 4.5.19. The calculation results of the required reinforcement bar are shown in Table 4.4.39.



Source: JICA Study Team

Figure 4.5.19 Calculation of Bridge Seat

4.5.5.6 Computation of Footings

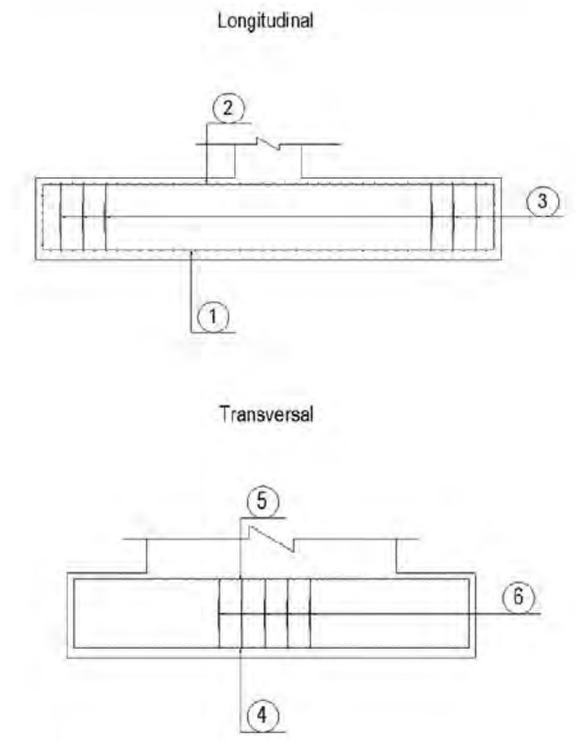
Footing shall be designed in consideration of the most adverse load combination of self-weight, overburden load such as soils, presence of buoyancy, subgrade reaction, and reaction from foundations. Footings may be designed as beam members and as cantilevers.

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 4.5.21.

Table 4.5.21 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

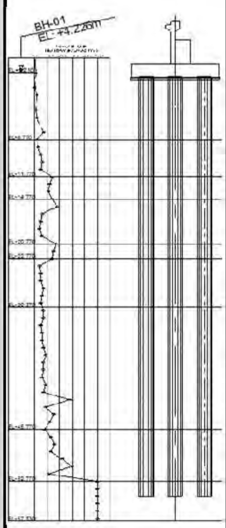
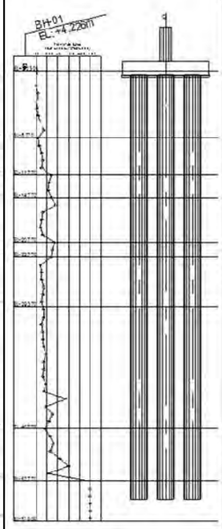
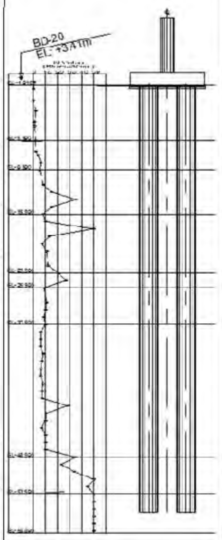
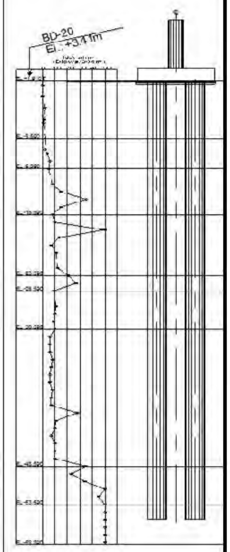
4.5.5.7 Design of Foundation

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 4.5.22 and Table 4.5.23, respectively.

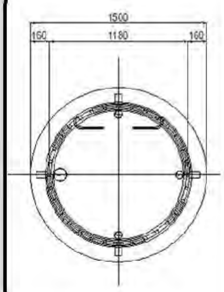
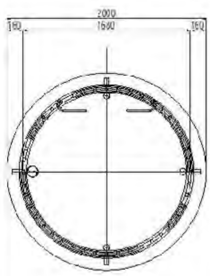
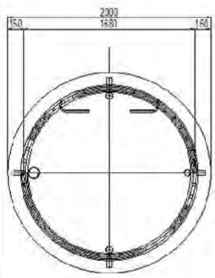
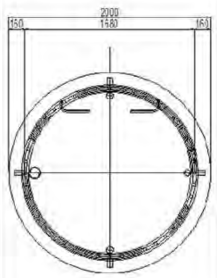
Table 4.5.22 Calculation Results of CIP Pile Foundation Stability(AO1~PO3)

	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 4.5.23 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

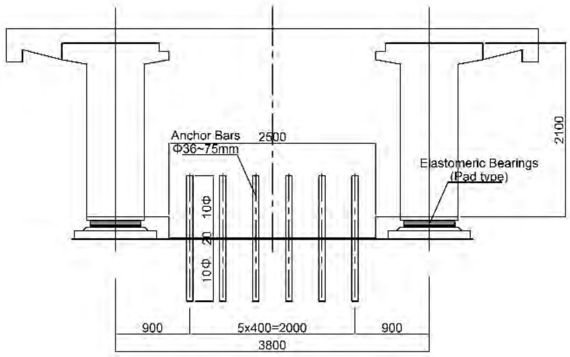
σs ; Bending Unit Stress
σsa ; Allowable Unit Stress
τm ; Unit Share Force
τa ; Allowable Unit Share Force

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.



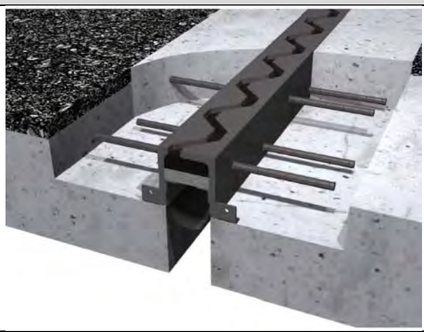
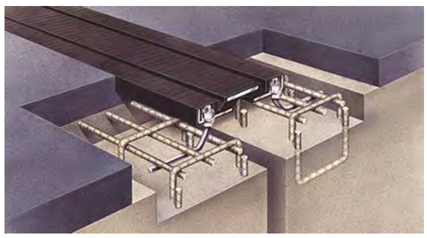
Source: JICA Study Team

Figure 4.5.20 Arrangement of Bearings and Anchor Bars of On-ramp Bridge

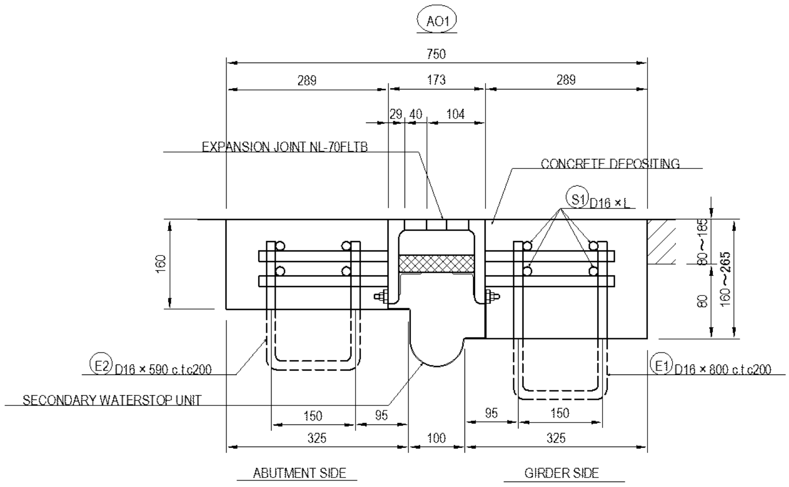
4.5.6.2 Expansion Joints

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

Table 4.5.24 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

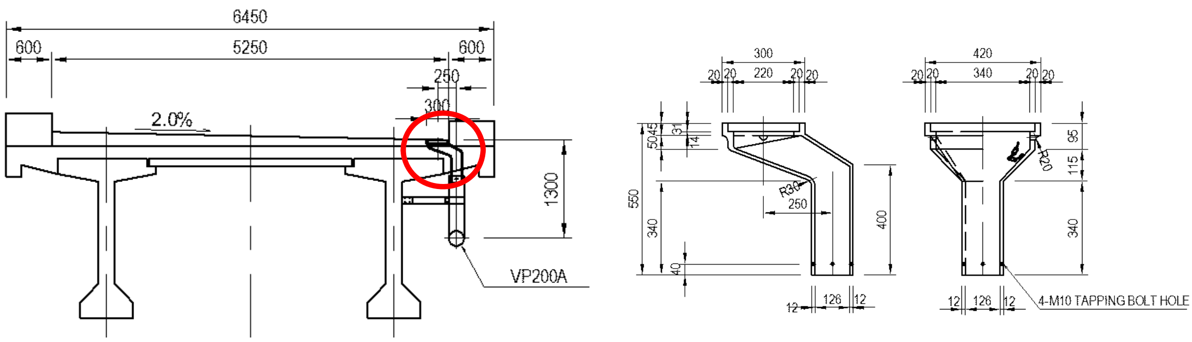
Figure 4.5.21 Expansion Joint of On-ramp Bridge (at AO1)

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 4.5.22 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.

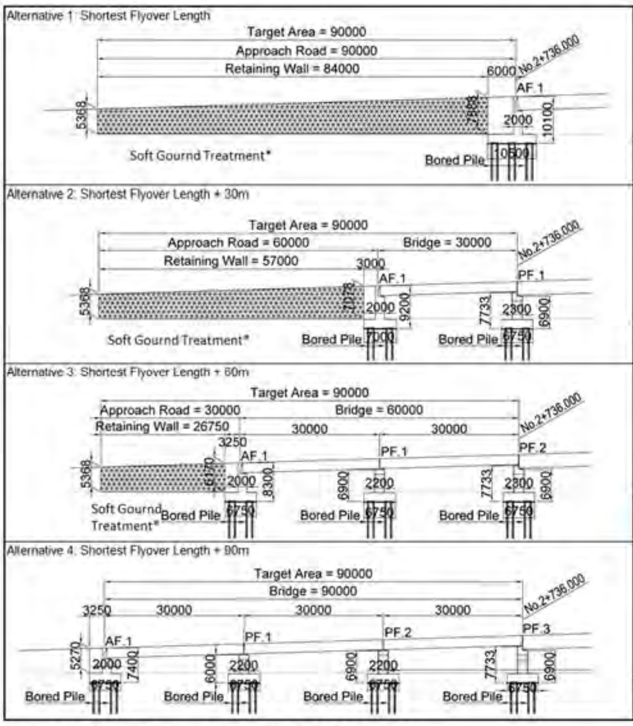
- Alternative-1 : Shortest Flyover Length / L = 542 m (Nearly the original flyover length of 542 m in the F/S)
- Alternative-2 : Shortest Flyover Length + 30 m
- Alternative-3 : Shortest Flyover Length + 60 m
- Alternative-4 : Shortest Flyover Length + 90 m

(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

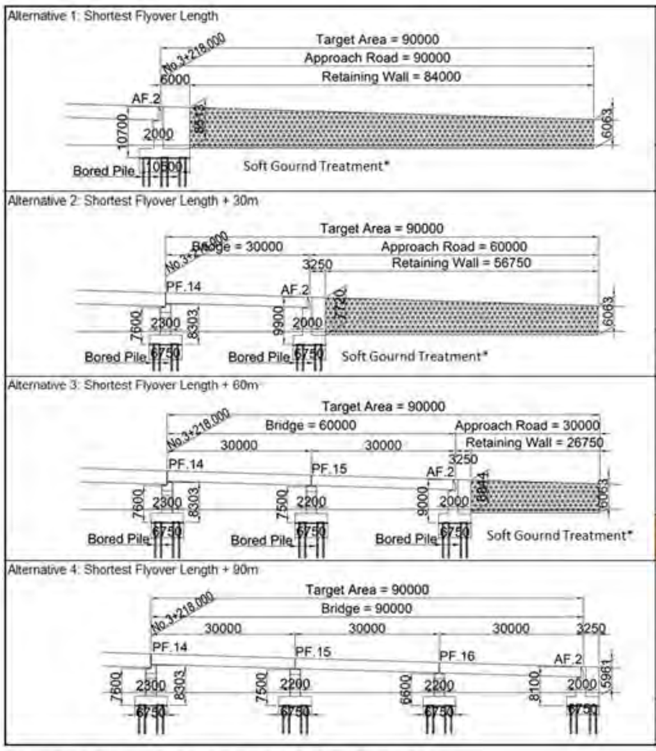
Abutment 1



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

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

Abutment 2



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

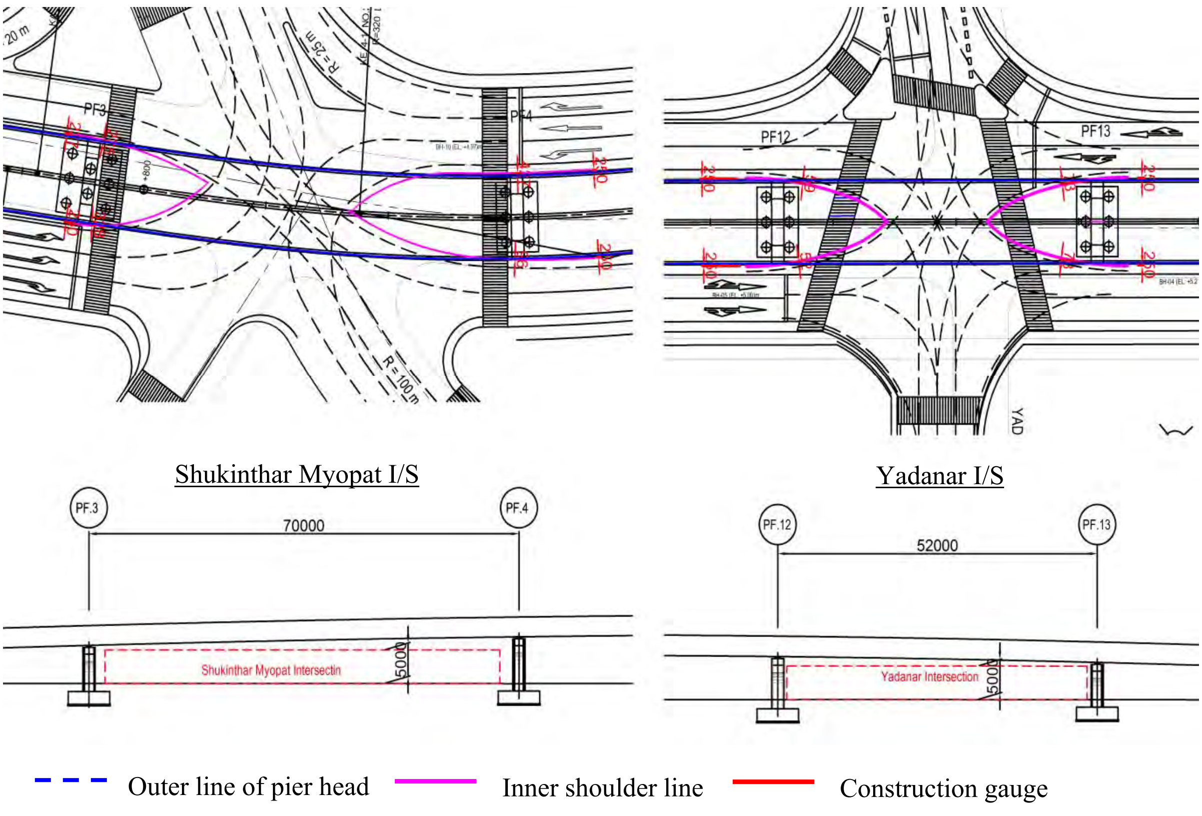
a) Required Minimum Span Length for Special Section at the Intersections

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



Source: JICA Study Team

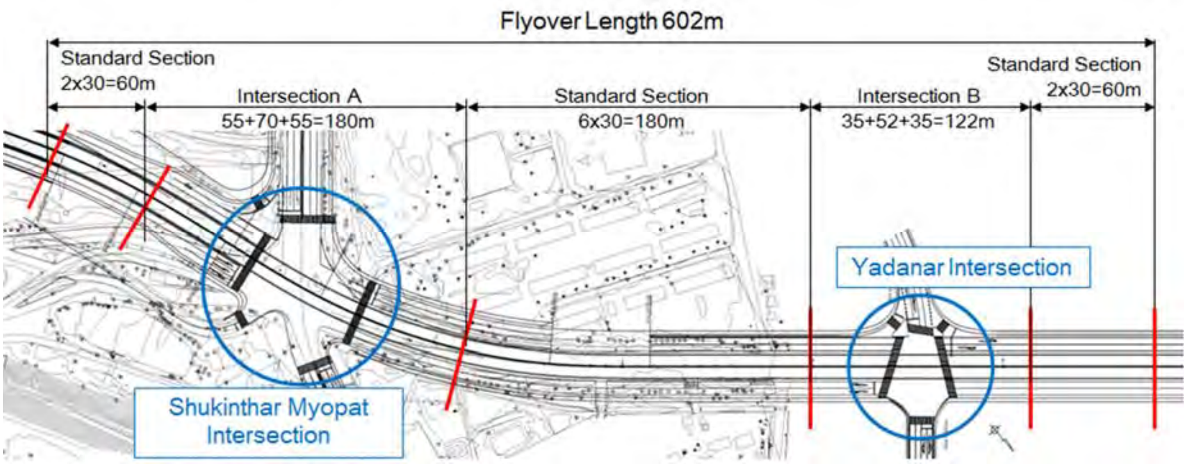
Figure 4.6.1 Required Minimum Span Length at Shukinthar Myopat I/S and Yadanar I/S

b) 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

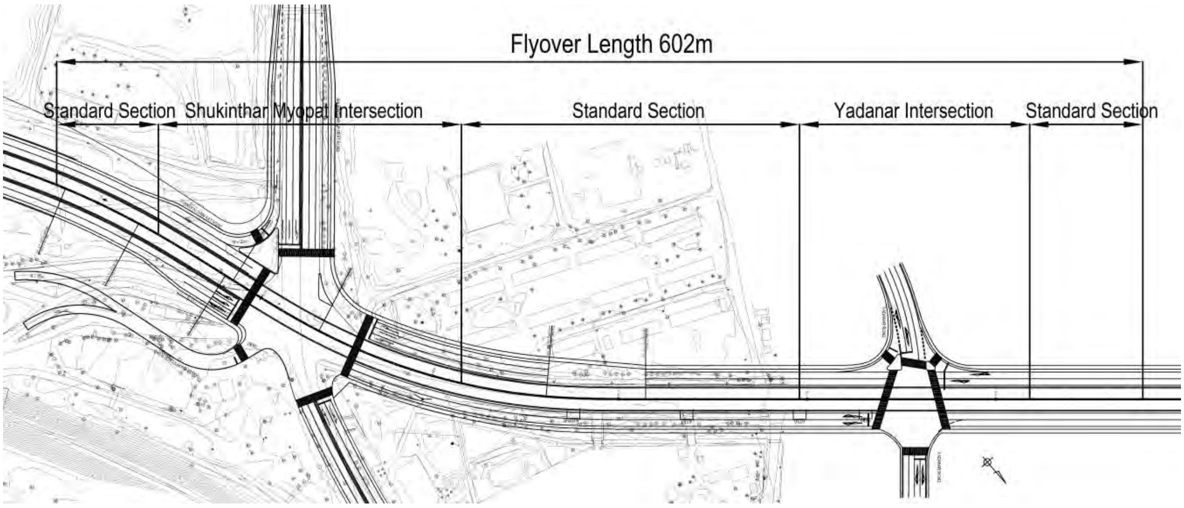
Figure 4.6.2 Span Arrangement of Flyover

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

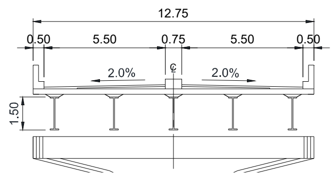
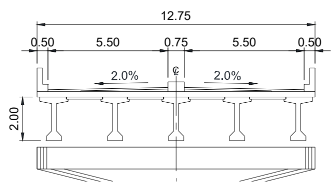
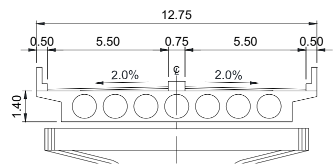
(2) Superstructure Type for Standard Section

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

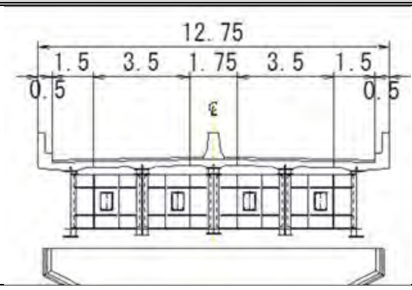
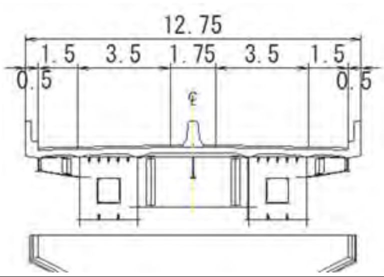
(3) Superstructure Type for Special Section at Shukinthar Myopat I/S

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).

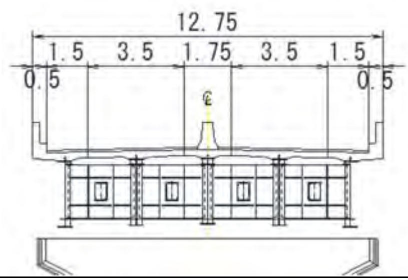
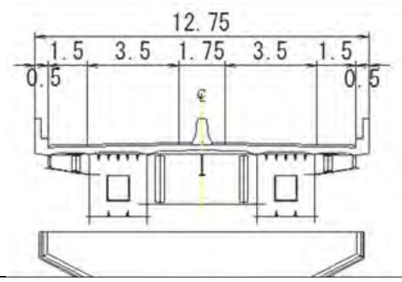
(4) Superstructure Type for Special Section at Yadanar I/S

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

[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".

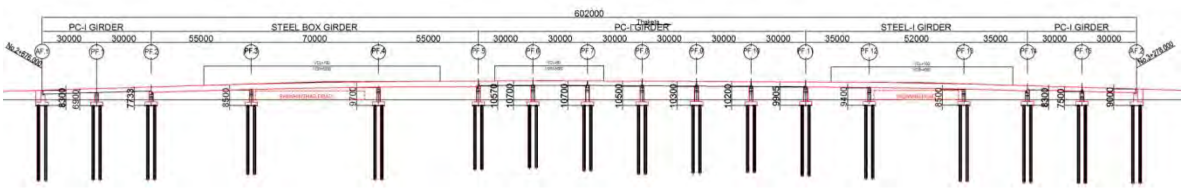
- Alternative-1 Precast PC Pile
- Alternative-2 Cast-in-place RC Pile (Original plan in the Supplemental F/S)
- Alternative-3 Steel Pipe Pile

Table 4.6.7 Possible Foundation Type for Flyover

Criteria			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



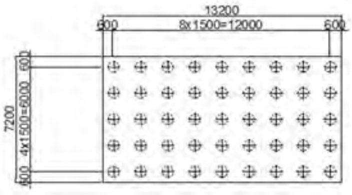
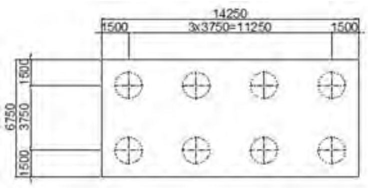
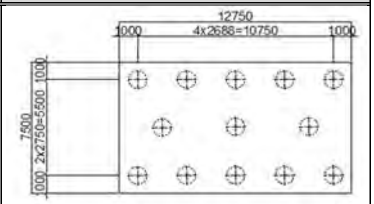
Source: JICA Study Team

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.

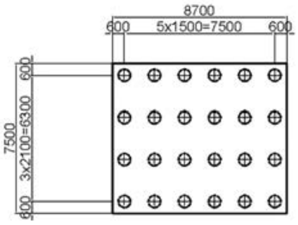
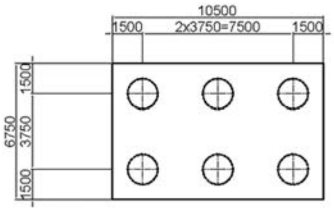
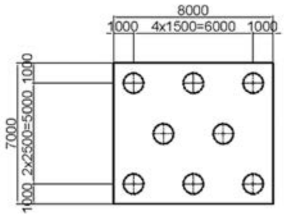
Table 4.6.8 Comparative Study of Foundation Type for Special Section (AF1)

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

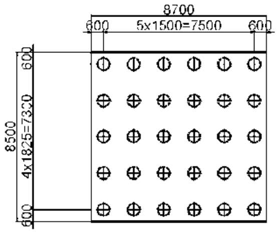
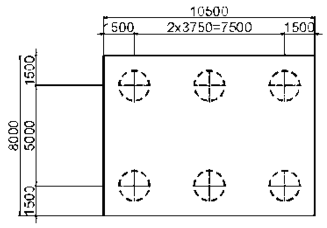
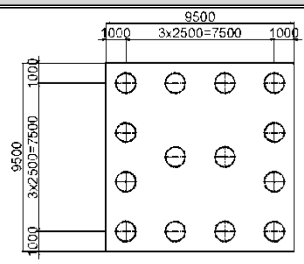
Table 4.6.9 Comparative Study of Foundation Type for Special Section (PF6)

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**

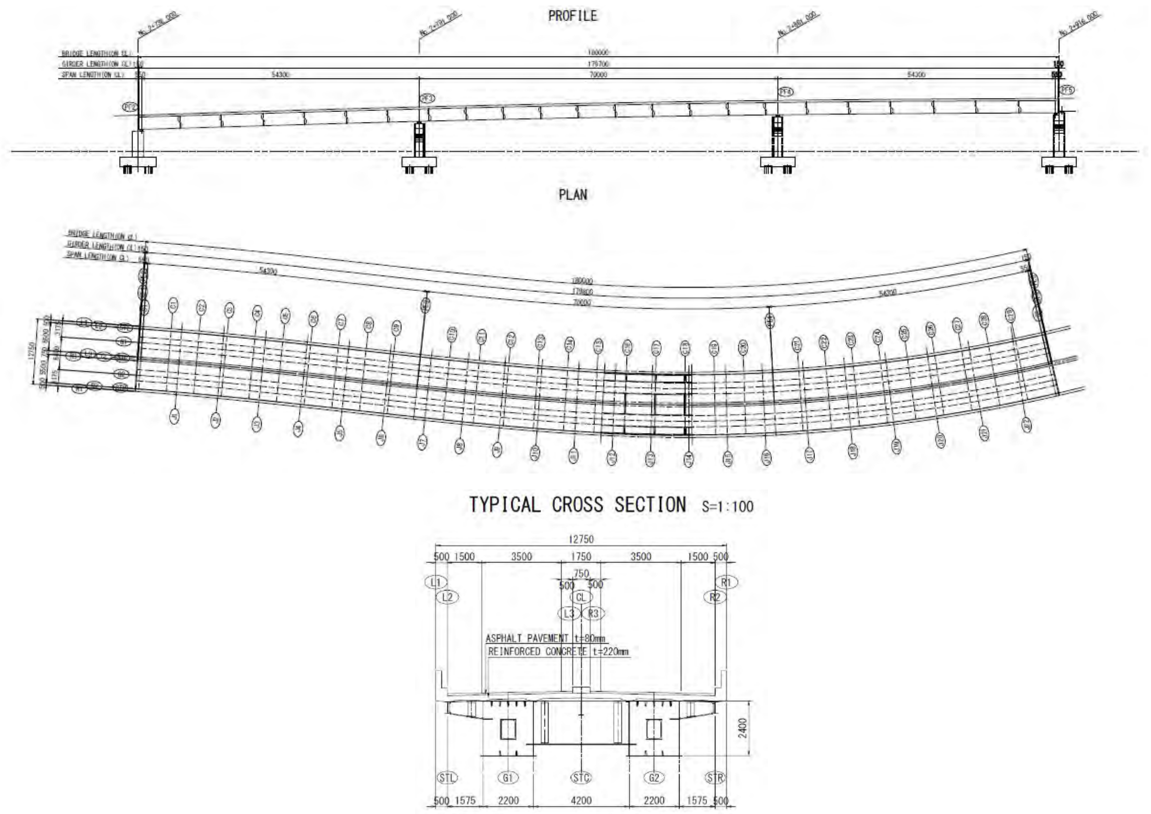
Source: JICA Study Team

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.

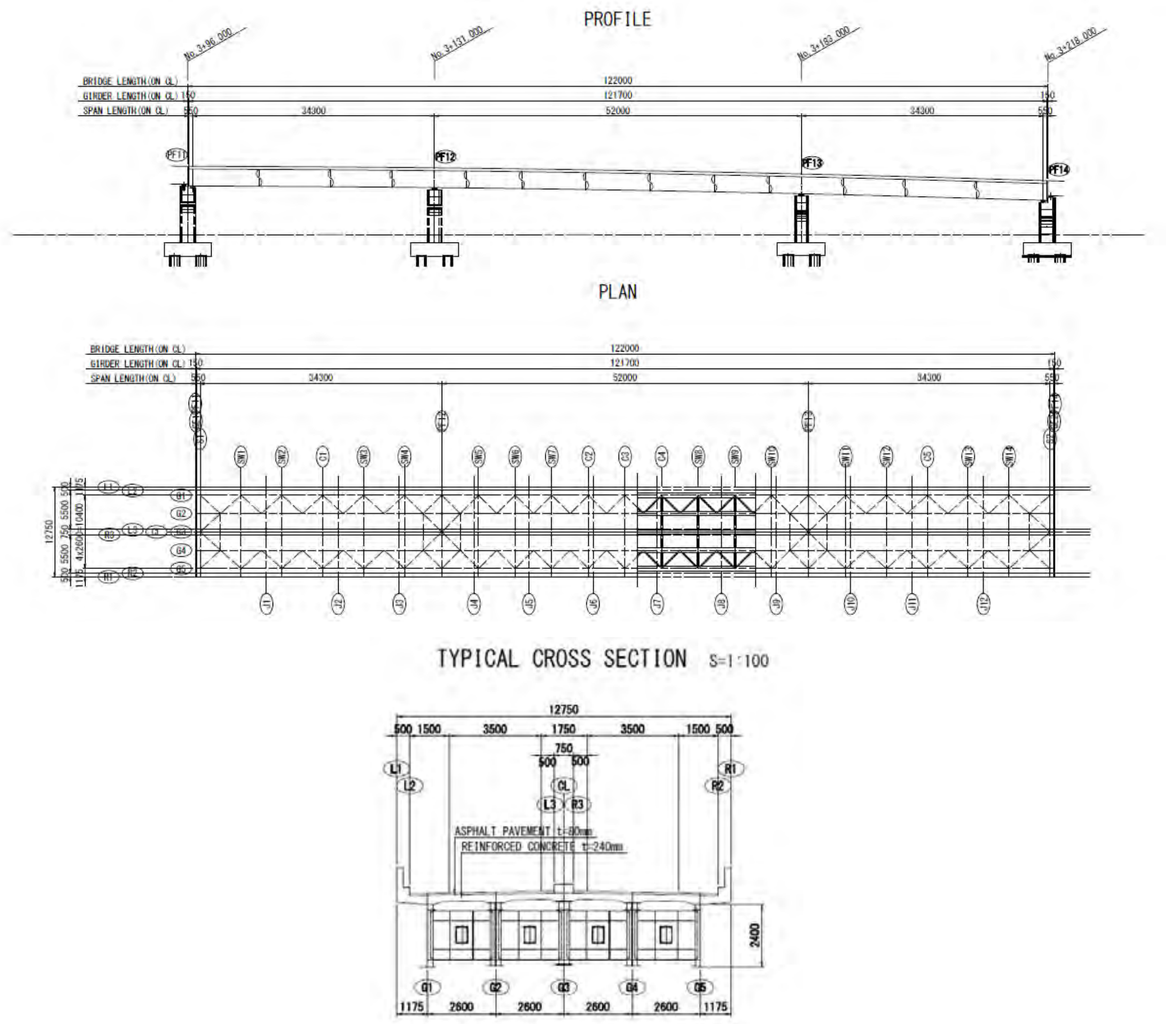


Source: JICA Study Team

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.

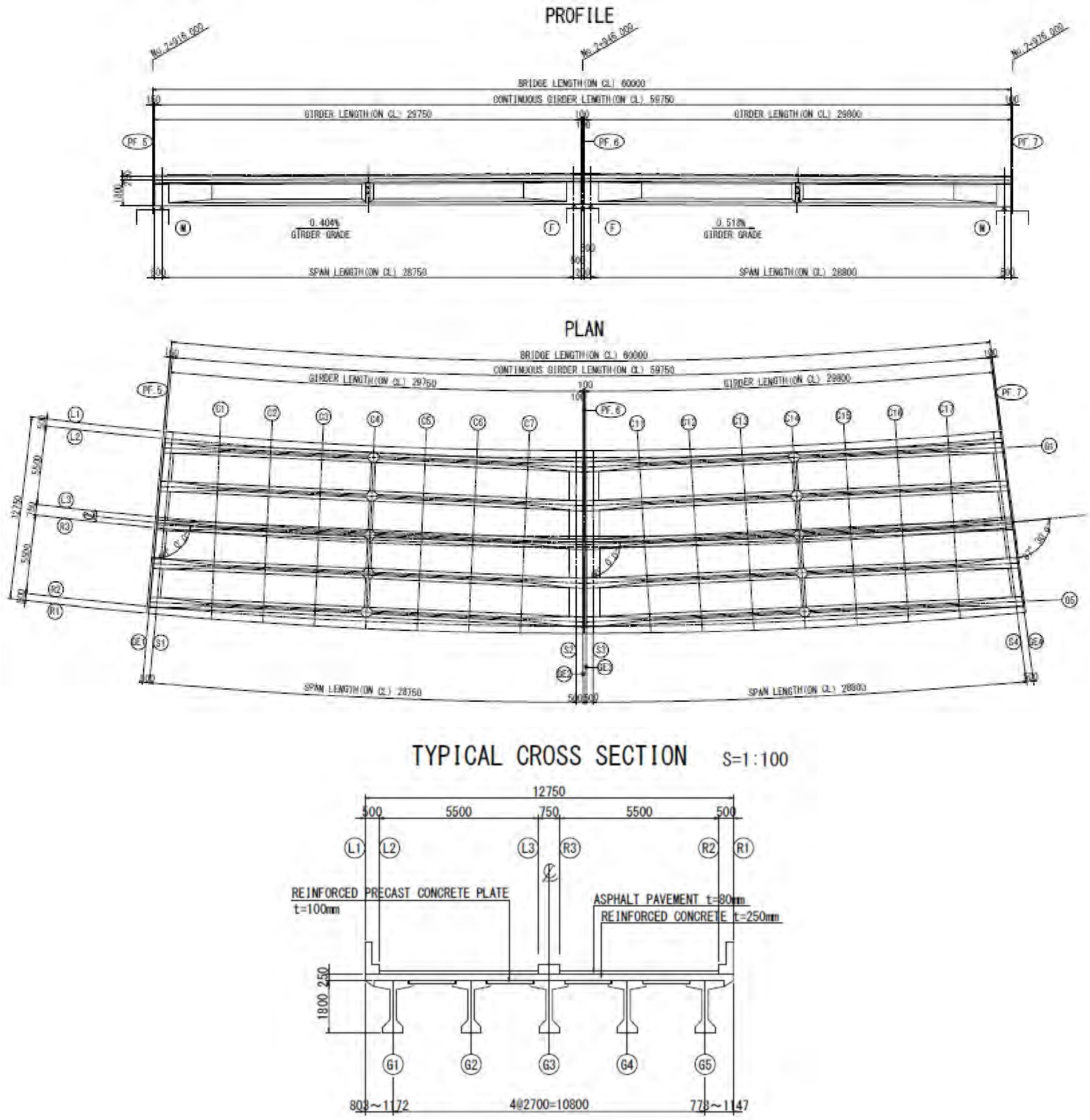


Source: JICA Study Team

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

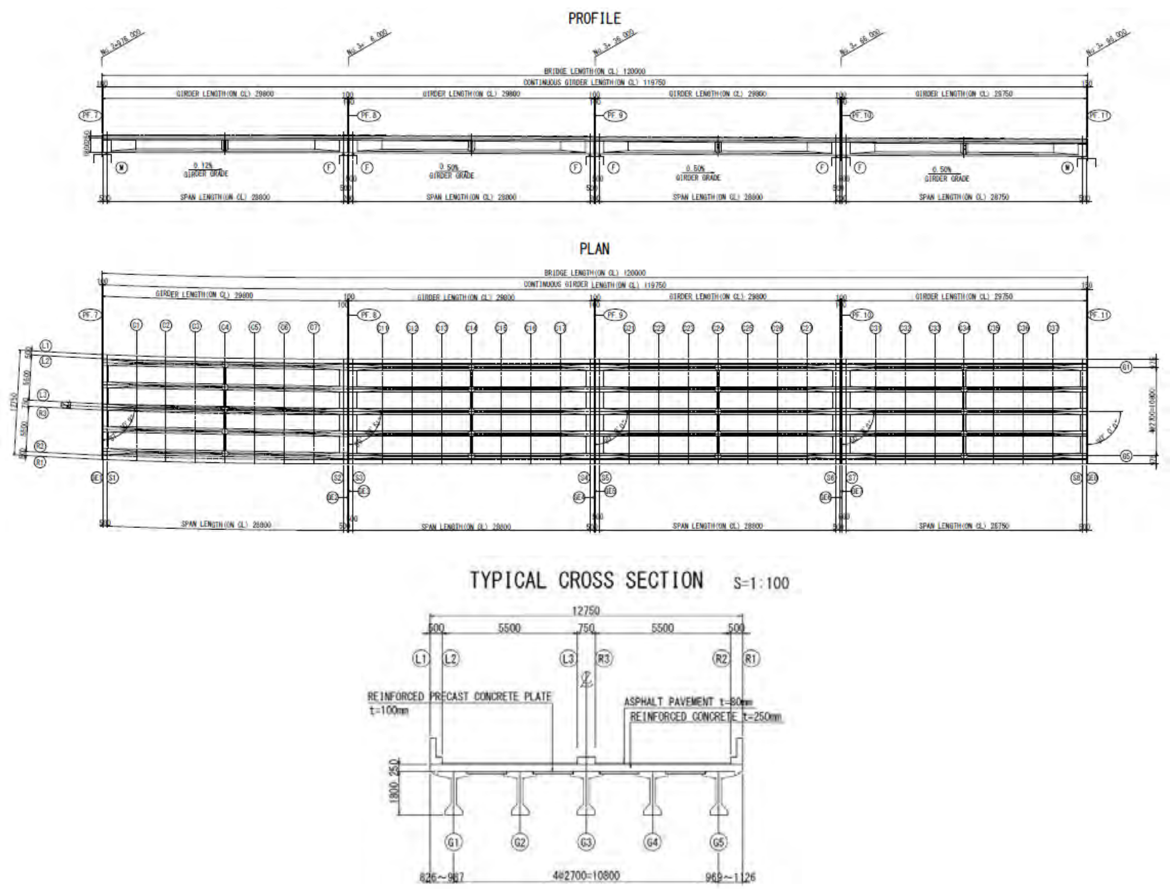
4.6.2.2 PC-I Girder Bridge

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)

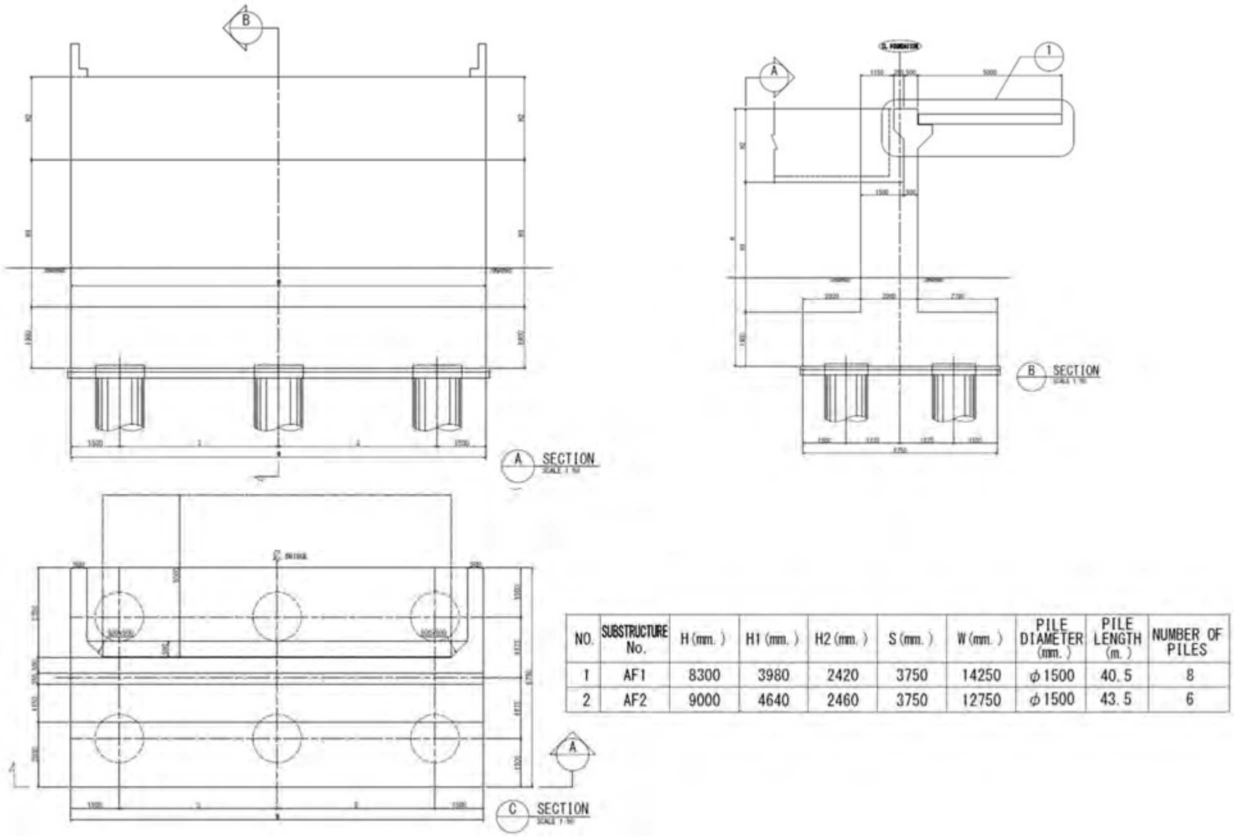


Source: JICA Study Team

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

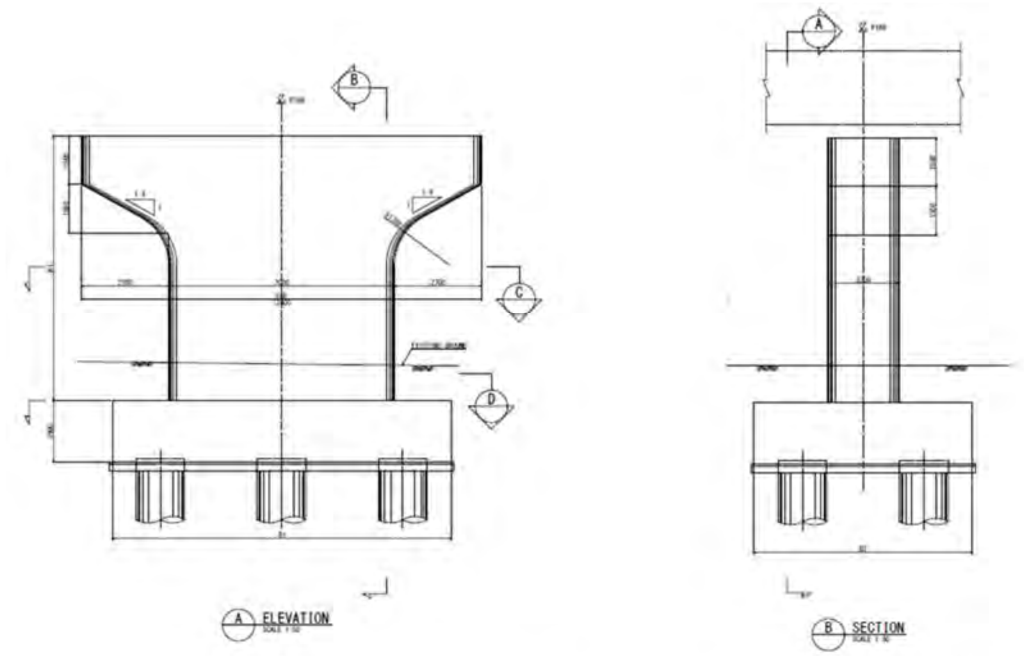
The general view of the abutment and pier in the B/D is shown in the following figures.

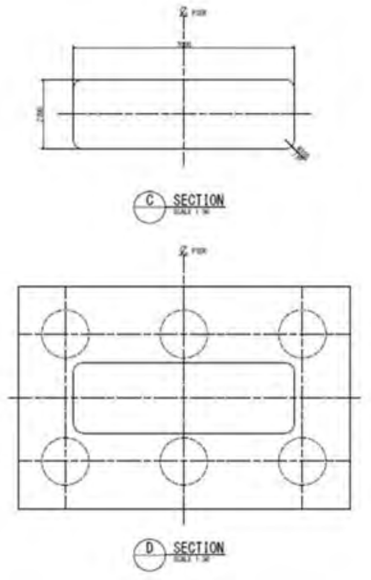


NO.	SUBSTRUCTURE No.	H (mm.)	H1 (mm.)	H2 (mm.)	S (mm.)	W (mm.)	PILE DIAMETER (mm.)	PILE LENGTH (m.)	NUMBER OF PILES
1	AF1	8300	3980	2420	3750	14250	φ1500	40.5	8
2	AF2	9000	4640	2460	3750	12750	φ1500	43.5	6

Source: JICA Study Team

Figure 4.6.9 General View of Abutment in the B/D

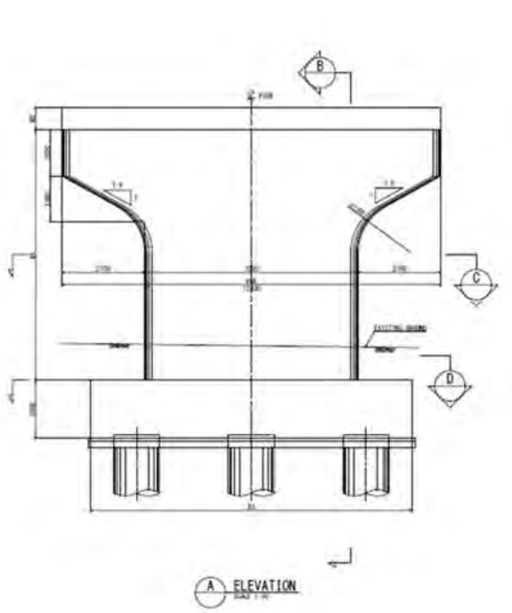


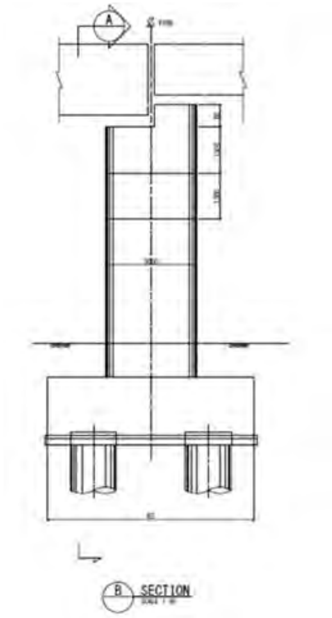


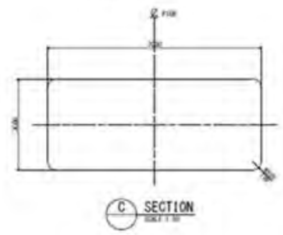
NO.	SUBSTRUCTURE No.	H1 (mm.)	B1 (mm.)	B2 (mm.)	PILE DIAMETER (mm.)	PILE LENGTH (m.)	NUMBER OF PILES
1	PF1	4800	10500	6750	φ1500	41.5	6
2	PF3	6600	10500	8000	φ1500	40.0	6
3	PF4	7800	10500	7500	φ1500	41.0	6
4	PF6	8800	10500	6750	φ1500	37.5	6
5	PF7	8800	10500	6750	φ1500	39.0	6
6	PF8	8600	10500	7500	φ1500	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

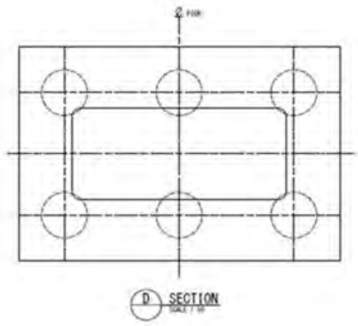
Figure 4.6.10 General View of Pier (Type A) in the B/D







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

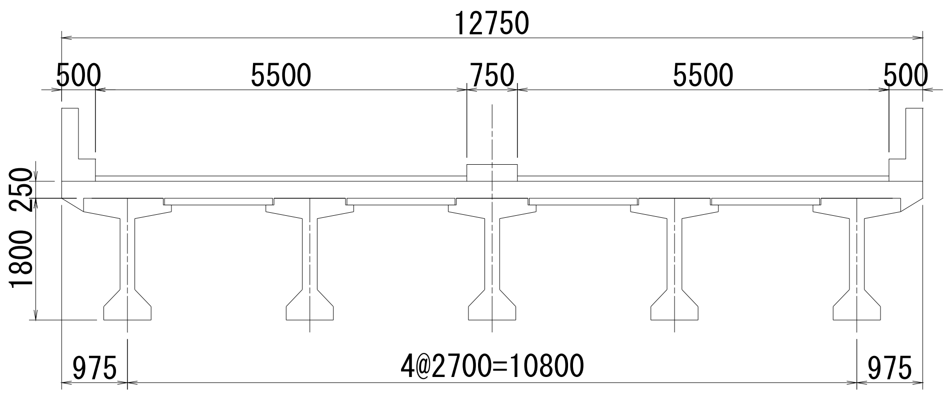
The updates on PC-I girder bridge are shown in the following table:

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.

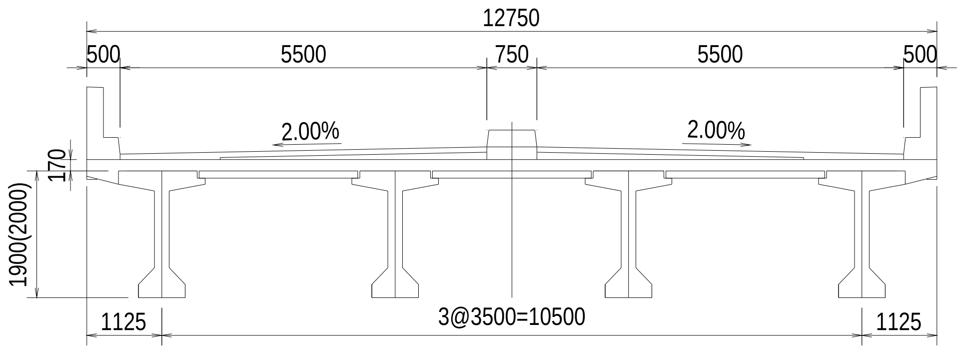


Source: JICA Study Team

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**		-	

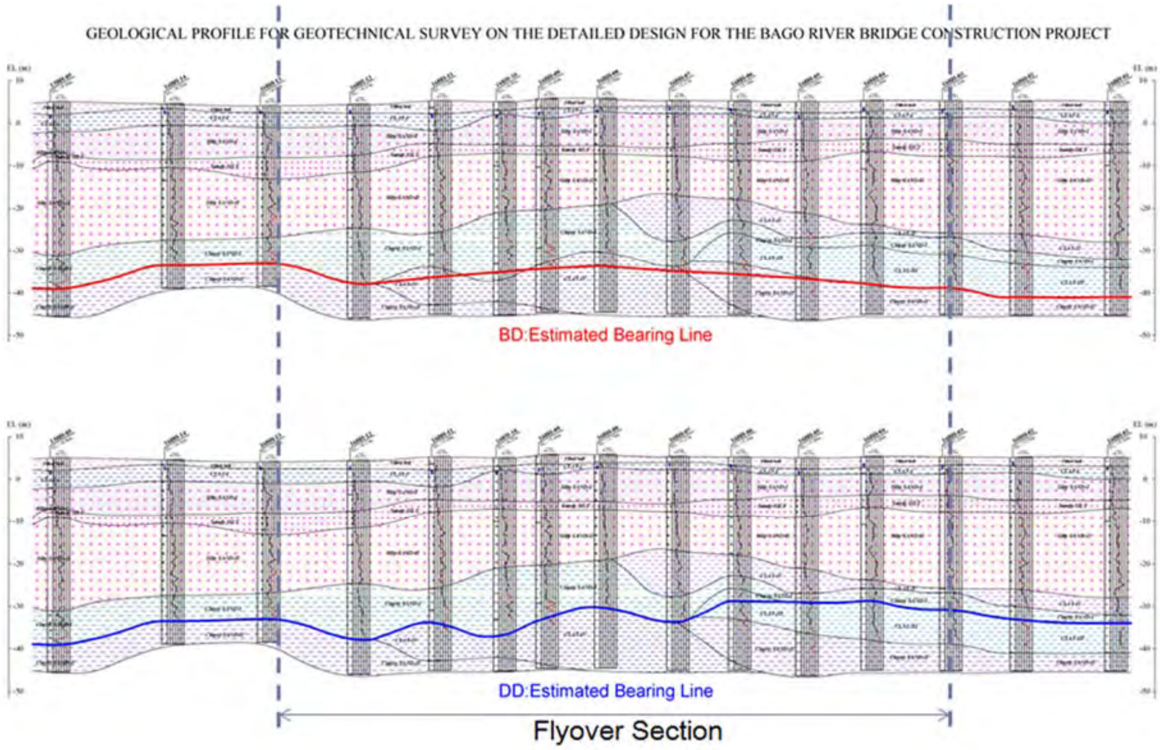
(b) 10<x≦20

	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).

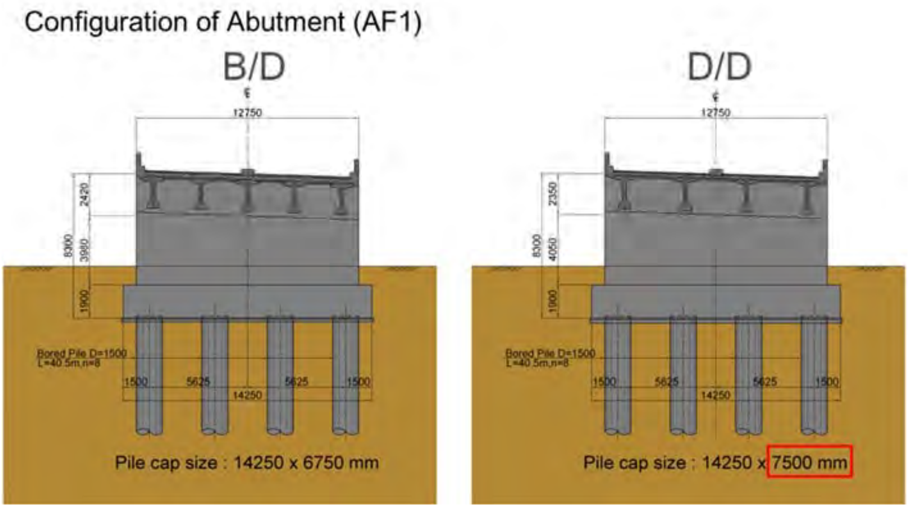


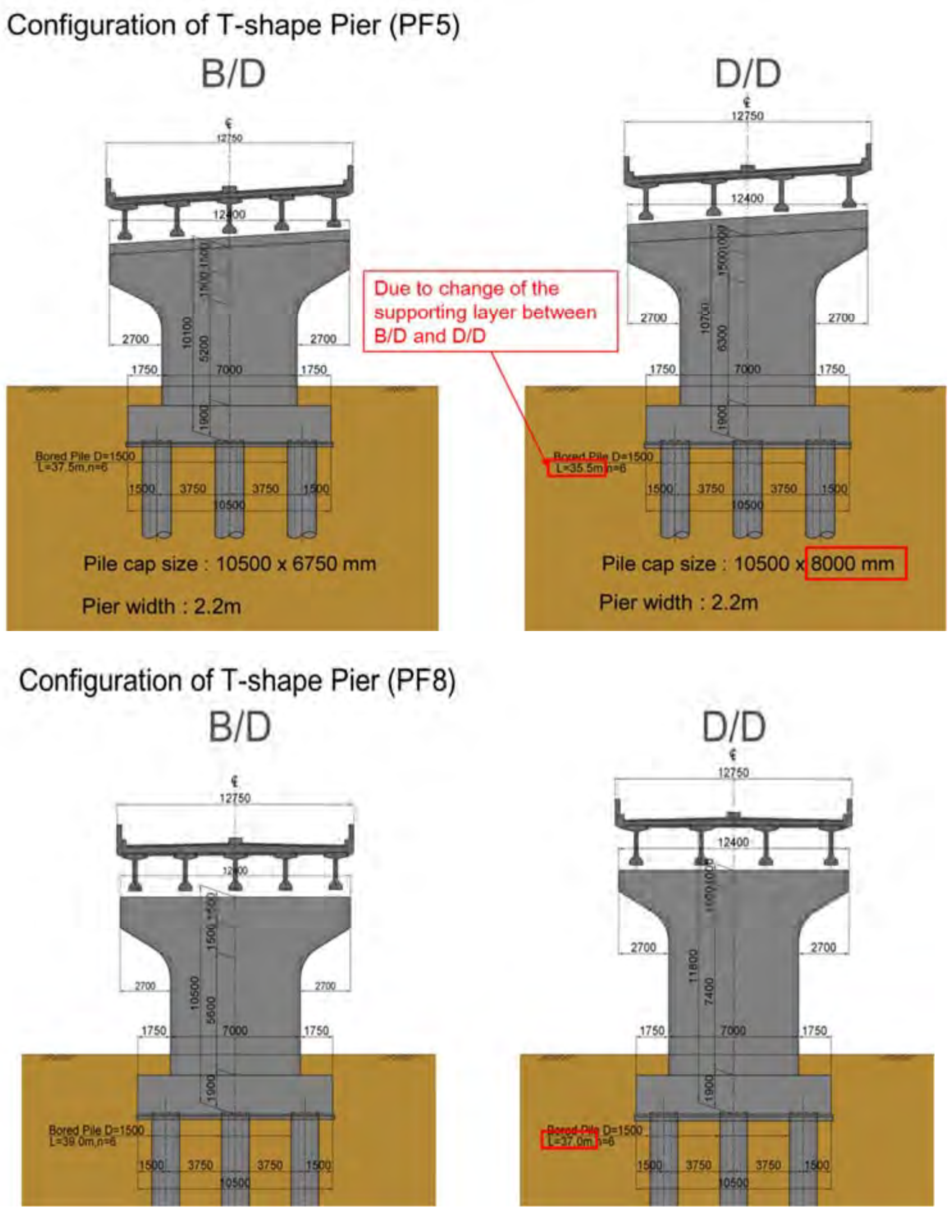
Source: JICA Study Team

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 Detailed Design Results

4.6.4.1 Detailed Design of Steel Girder Bridge

(1) Design Condition

1) General Design Condition

The general design conditions for steel box girder and steel-I girder are shown in Table 4.6.16 and Table 4.6.17, respectively.

Table 4.6.16 General Design Condition of Superstructure in Steel Box Girder Bridge

Item	Conditions
General	
Bridge Type	Continuous 3-span steel box girder bridge (Sta. 2+736.000 to Sta. 2+916.000)
Bridge Length	180 m
Girder Length	179.4 m
Span Arrangement	55.0 m + 70.0 m + 55.0 m
Total Width	12.75 m
Effective Width	11.00 m
Structure Type	
Main Girder	Steel Structure
Crossbeam	Steel Structure
Deck	Reinforced Concrete Structure (RC Structure)
Materials	
Concrete	RC Structure: 24 N/mm^2 (Deck)
Reinforcement Bar	SD345
Steel	SM400, SM490Y, SS400, S10T

Source: JICA Study Team

Table 4.6.17 General Design Condition of Superstructure in Steel-I Girder Bridge

Item	Conditions
General	
Bridge Type	Continuous 3-span I-section steel plate girder (Sta. 3+96.000 to Sta. 3+218.000)
Bridge Length	122 m
Girder Length	121.4 m
Span Arrangement	35.0 m + 52.0 m + 35.0 m
Total Width	12.75 m
Effective Width	11.00 m
Structure Type	
Main Girder	Steel Structure
Crossbeam	Steel Structure
Deck	Reinforced Concrete Structure (RC Structure)
Materials	
Concrete	RC Structure: 24 N/mm^2 (Deck)
Reinforcement Bar	SD345
Steel	SM400, SM490Y, SS400, S10T

Source: JICA Study Team

2) Design Case for Superstructure and Load Combinations

The design load case, the load combinations, and the load factors are shown in Table 4.6.18 and Table 4.6.15.

Table 4.6.18 Design Case, Load Combinations and Multiplication Factors

Design Load Case	Load Combinations	Load Factors
Under Dead Load	Self-Weight, Weight of Bridge Surface	1.00
Under Design Load	Dead Load + Live Load + Impact	1.00
Under Wind Load	Dead Load + Wind Load	1.25
	Design Load + Wind Load	1.25
Under Earthquake Load	Dead Load + Earthquake Load	1.50
Under Collision Load	Design Load + Collision Load	1.50

Source: JSHB

Table 4.6.19 Member Design Case

	Design Case
Main Girder	Under Dead Load + Live Load + Impact
Crossbeam, Stringer Bracket	Under Dead Load + Live Load + Impact
Sway Bracing	Wind or Earthquake, Slenderness Ratio
Lateral Bracing	Wind or Earthquake, Slenderness Ratio

Source: JSHB

3) Design Parameter of Materials

a) Structural Steel and Reinforcement Bar

The design parameters of the structural steel and the reinforcement bar are shown in Table 4.6.20.

Table 4.6.20 Design Parameters of Structural Steel and Reinforcement Bar

Material	Type	Young's Modulus (N/mm^2)	Yield Stress (N/mm^2)		Tensile Strength (N/mm^2)
			Steel Thickness 16 mm to 40 mm	Steel Thickness 40 mm to 75 mm	
Structural Steel	SS400/SM400	2.0×10^5	235	215	400 to 510
	SM490Y		355	335	490 to 610
Reinforcement Bar	SD345	2.0×10^5	345 to 440		490 or over

Source: JSHB

b) Concrete

The design parameter of concrete is shown in .

Table 4.6.21.

Table 4.6.21 Design Parameter of Concrete

Material	Design Strength	Young's Modulus
Concrete	24 N/mm^2	2.0×10^5 N/mm^2

Source: JSHB

4) Allowable Stress of Materials

a) Structural Steel

- Basic Allowable Stress

The basic allowable stress of the structural steel is defined based on the yield stress. The basic allowable stress and the yield stress of the structural steel are shown in Table 4.6.22.

Table 4.6.22 Basic Allowable Stress and Yield Stress of Structural Steel

Stress Type	Unit	SM400		SM490Y	
		Steel Thickness Up to 40 mm	Steel Thickness 40 mm to100 mm	Steel Thickness Up to 40 mm	Steel Thickness 40 mm to 75 mm
Basic Allowable Stress	N/mm^2	140	125	210	195
Yield Stress	N/mm^2	235	215	355	335
Safety Factor	---	1.68	1.72	1.69	1.72

Source: JSHB

- Allowable Axial Compressive Stress of Structural Steel

The allowable axial compressive stress is calculated based on the allowable axial compressive stress in the case of not considering the local buckling, the upper limit of the allowable axial compressive stress, and the allowable stress in consideration of the local buckling.

The allowable axial compressive stress without local buckling is determined based on the strength obtained considering its variations in quality, as specified in the JSHB II 3.2.

The allowable compressive stress considering local buckling is shown in the item “e” below. The upper limit of the allowable axial compression stress is given in Table 4.6.22.

The allowable axial tensile stress and allowable bending tensile stress are the values considering the safety factor against the basic tensile yield stress as shown in Table 4.6.22.

- Allowable Bending Compressive Stress of Structural Steel

The allowable bending compressive stress is decided by the fixing conditions of the compression flange and section type, as specified in the JSHB II 3.2.

The allowable bending compressive stress is based on the values shown in Table 4.6.22 and is defined as the value taking into consideration the influence of the lateral buckling strength of the girder.

- Allowable Stress for Local Buckling of Structural Steel

The allowable stress for local buckling is decided by considering the influence of residual stress due to initial irregularities such as the support condition, initial deformation, welding and other initial irregularities, based on the values in Table 4.6.22. It is calculated by the equation shown in the JSHB II 4.2.

- Allowable Shear Stress of Structural Steel

The allowable shear stress applies the tensile yield condition of Von Mises against the basic tensile yield stress, $\tau y = \sigma y/\sqrt{3}$, and is provided in consideration of the safety factor of 1.7. The value is shown in Table 4.6.23.

Table 4.6.23 Allowable Shear Stress of Structural Steel

Stress Type	Unit	SM400		SM490Y	
		Steel Thickness Up to 40 mm	Steel Thickness 40 mm to 75 mm	Steel Thickness Up to 40 mm	Steel Thickness 40 mm to 75 mm
Allowable Shear Stress	N/mm^2	80	75	120	115

Source: JSHB

- Allowable Stress of Reinforcement Bar for Deck Slab

The allowable stress of reinforcement bars in the deck is shown in Table 4.6.24.

Table 4.6.24 Allowable Stress of Reinforcement Bar

Stress Type	SD345
Allowable Tensile Stress	140 N/mm^2
Allowable Compressive Stress	200 N/mm^2

Source: JSHB

b) Concrete

The allowable compressive stress of concrete is shown in Table 4.6.25.

Table 4.6.25 Allowable Compressive Stress of Concrete

Design Strength	Allowable Compressive Stress
24 N/mm^2	8.0 N/mm^2

Source: JSHB

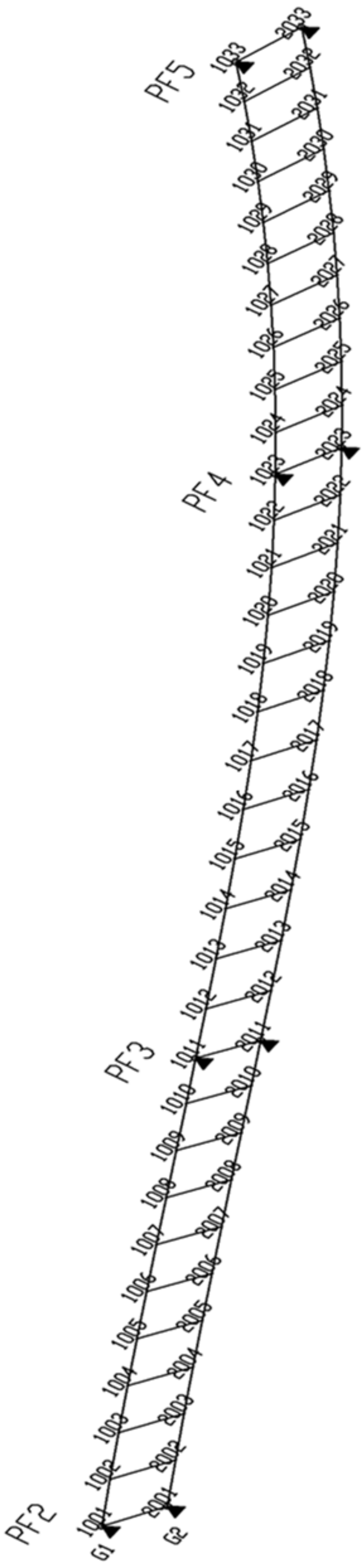
(2) Detailed Design of Steel Box Girder Bridge

1) Analytical Modelling

a) Grillage Analysis

The steel box girder bridge, composed of beam structures, main box girders, and crossbeams, is modelled with linear beam elements in two dimension as shown in Figure 4.6.16. The geometrical moment of inertia and physical property shall be inputted as linear element, and dead load and live load are put on the grillage model. The load is statically distributed to the longitudinal members by the crossbeams. The grillage model is analyzed by the displacement method; the sectional force, the displacement at an arbitrary point, and the reaction force at the supports are calculated by using the line of influence.

b) Grid Model



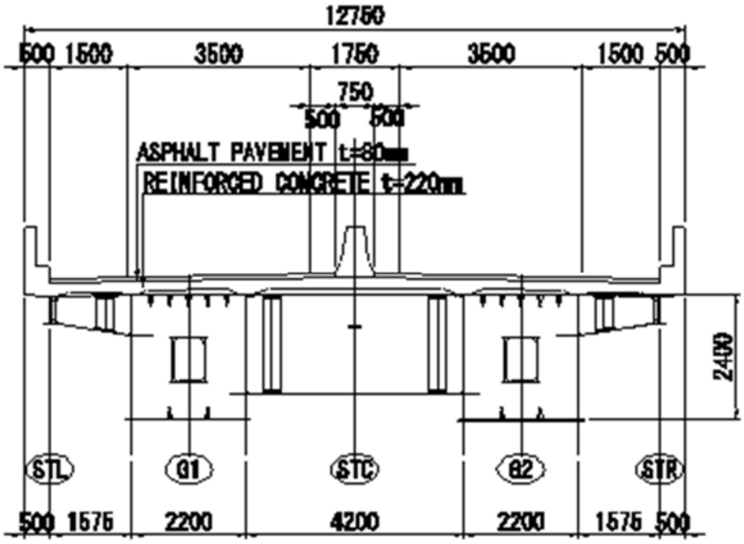
Source: JICA Study Team

Figure 4.6.16 Grid Model

2) Design of Main Girder

a) Main Girder Arrangement

The arrangement of the main girder is determined by the supporting span of the slab. The desired overhang length of the deck is 1.2 m or less. The spacing of the main girder is determined in such a way that the influence of additional bending moment due to the difference in rigidity between the stringer and the girder is not large. As a result, the arrangement of the main girders is shown in Figure 4.6.17.



Source: JICA Study Team

Figure 4.6.17 Arrangement of Main Girders

b) Stiffener of Box Girder

For the purpose of stiffening, the upper and lower flanges are placed as two rib plates in the area of positive bending moment, and five rib plates in the area of negative moment.

The thickness of the flange of the main girder is designed to satisfy the allowable stress considering the local buckling.

c) Connection of Members

The joining method of the main girder is friction grip connection with high strength bolts. The number of bolt rows should be eight or less. When designing a connection, fillers shall be used to adjust the physical gaps between the base metals.

The base material of the splice plate and bolt connecting part shall be coated with inorganic zinc paint.

d) Checking Item

The girders are designed based on the sectional forces calculated by the grillage analysis, and each member is designed so that its stress is less than the allowable unit stress under each load combination. The checking items are shown in Table 4.6.26.

Table 4.6.26 Checking Items for Main Girder Design

Members	Checking Item
Flange	- Compressive stress - Tensile stress
Web	- Shear stress
Flange and Web	- Verification of resultant stress
Girder	- Deflection

Source: JICA Study Team

e) Stress of Main Girder

The flange is a stiffened plate, and the thickness of the main girder flange is determined in such a way that the sectional stress is less than the allowable stress.

At the center of the span, the positive bending moment is maximum, and the shear force is small. Also, the lower flange is in tension state and the upper flange is in a compression state. In Section (4), (17), (30), which are in the positive moment area, the upper flange is stiffened with five rib plates, and was checked if its stress is smaller than the allowable stress in consideration of local buckling. The lower flange is stiffened with two rib plates, and was checked if its stress is smaller than the allowable tensile stress.

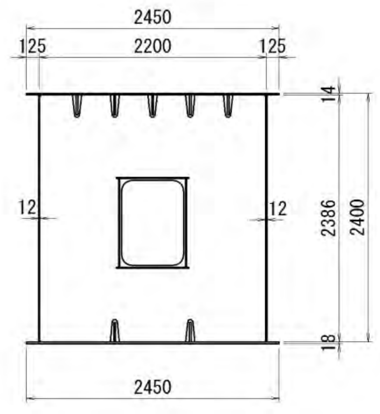
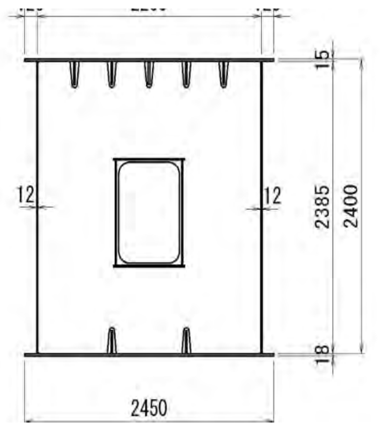
In the continuous girder at the intermediate support, the negative bending moment is maximum, and shearing force is also large. The lower flange is in a compression state and the upper flange is in a tension state.

In Section (10), (24), which are in the negative moment range, the upper flange is stiffened with two rib plates, and was checked if its stress is smaller than the allowable tensile stress. The lower flange is stiffened with five rib plates and was checked if its stress is smaller than the allowable stress considering local buckling.

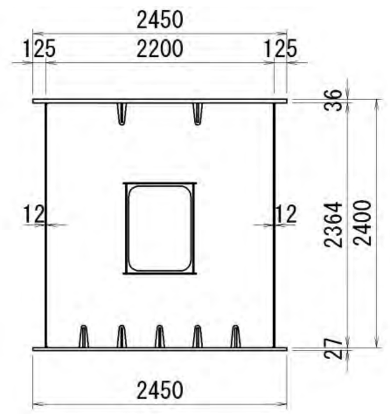
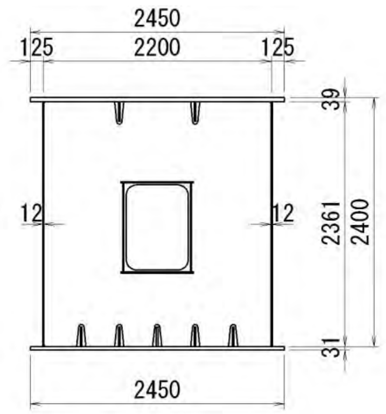
The thickness of web plate is decided by the minimum thickness in which local buckling is not expected when the horizontal stiffener is installed in one stage.

The calculation result of stress is shown in Table 4.6.27.

Table 4.6.27 Stress Check of the Main Girder

G1	G2
(4)	(30)
	

Shape		Stress (N/mm²)		Shape		Stress (N/mm²)	
Upper flange	2450x14	**-173 < 191**	OK	Upper flange	2450x15	**-177 < 199**	OK
Lower flange	2450x18	171 < 210	OK	Lower flange	2450x22	174 < 210	OK
Web	2386x12	13 < 120	OK	Web	2385x12	13 < 120	OK

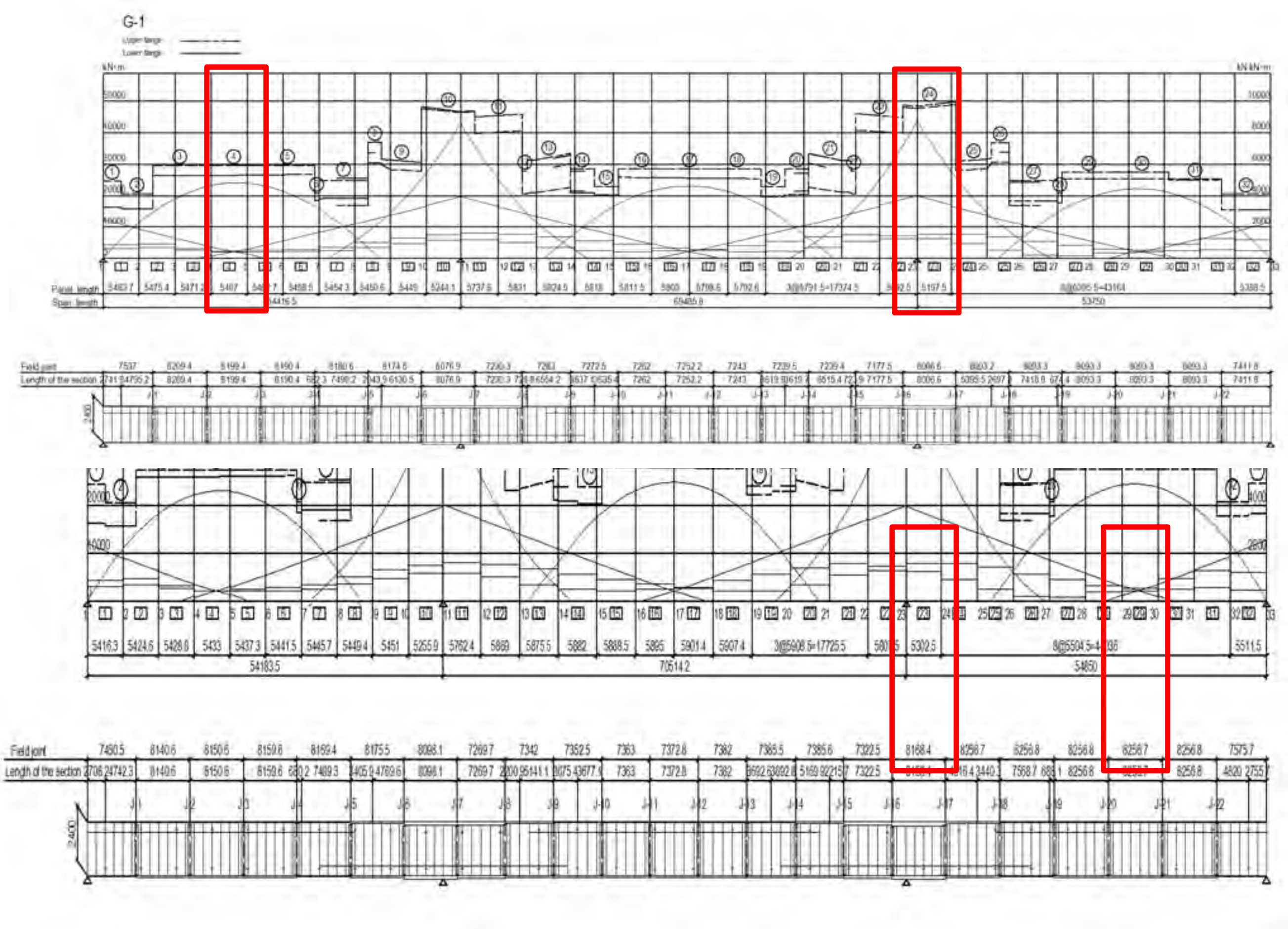
(24)	(24)
	

Shape		Stress (N/mm²)		Shape		Stress (N/mm²)	
Upper flange	2450x36	190 < 210	OK	Upper flange	2450x39	**188 < 210**	OK
Lower flange	2450x27	**-192 < 210**	OK	Lower flange	2450x31	-186 < 210	OK
Web	2364x12	66 < 120	OK	Web	2361x12	66 < 120	OK

Source: JICA Study Team

f) Moment Diagram

The moment diagram of the main girder is shown in Figure 4.6.18.



Source: JICA Study Team

Figure 4.6.18 Moment Diagram

g) Verification of Safety for Loads

The deflection was checked to ensure the stability of the structure and its safety. The allowable value of deflection is defined by the equation below.

L / 500 where, L: span length (m).

Table 4.6.28 Verification of Deflection

Check Position	Girder Name	Span Length (m)	Deflection Value (mm)	Allowable Value (mm)	Judgement
First Span	G1	55	66	109	OK
	G2	55	65	108	OK
Second Span	G1	70	96	139	OK
	G2	70	101	141	OK
Third Span	G1	55	63	108	OK
	G2	55	68	110	OK

Source: JICA Study Team

3) Design of RC Deck Slab

a) Thickness of Deck

The thickness of the deck was decided from the following equation:

$$D = 1.25 \times (30L + 110) \quad = 220 \text{ mm} \Rightarrow 220 \text{ mm}$$

1.25: coefficient related to traffic volume of large vehicles.

b) Design for Bending Moment of Deck

This slab was designed according to the JSHB II. The reason is that the weight of the design wheel of JSHB T-Load is 200 kN while the weight of the axle and the wheel of the design car of AASHTO HL 93 is 145 kN, so it is better for the RC slab to be designed with a larger force. Therefore, the bending moment of the deck is calculated by the unit width (1 m) of the T-Load specified in the JSHBII 8.2.4.

c) Checking Item

The depth of the deck and arrangement of reinforcement bars are determined in such a way that the sectional stress calculated by the sectional force is less than the allowable unit stress.

Table 4.6.29 Checking Items for RC Deck Slab Design

Materials	Checking Item
Concrete	- Compressive stress
Reinforcement Bar	- Tensile stress

Source: JICA Study Team

d) Stress of RC Slab

The calculation result of stress is shown in Table 4.6.30.

Table 4.6.30 Stress of Deck

	Main Reinforcement Direction			Transverse Reinforcement Direction		
	Stress (N/mm^2)			Stress (N/mm^2)		
Center of Span	D19@150			D16@125		
	Concrete	5.0<8.0	OK	Concrete	5.4<8.0	OK
	Reinforcement Bar	112<140	OK	Reinforcement Bar	118<140	OK
Support	D19@150					
	Concrete	3.4<8.0	OK	---	---	---
	Reinforcement Bar	84<140	OK	---	---	---

Source: JICA Study Team

4) Design of Crossbeam

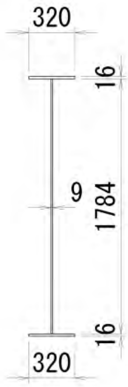
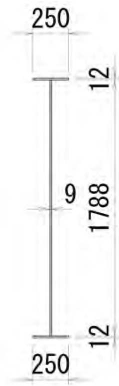
a) Design Case for Crossbeam

・Load Case: Dead Load + Live load + Impact

b) Shape of Crossbeam and Stress Check

The calculation result of stress is shown in Table 4.6.31.

Table 4.6.31 Stress of Crossbeam

<table>
<tr><th colspan="2">End Crossbeam</th><th colspan="2">Intermediate Crossbeam</th></tr>
<tr><td colspan="2">
</td><td colspan="2">
</td></tr>
<tr><td></td><td>Stress (N/mm^2)</td><td></td><td>Stress (N/mm^2)</td></tr>
<tr><td>Upper Flange</td><td>76 < 140 OK</td><td>Upper Flange</td><td>48 < 140</td></tr>
<tr><td>Web</td><td>24 < 80 OK</td><td>Web</td><td>24 < 80</td></tr>
<tr><td>Lower Flange</td><td>77 < 117 OK</td><td>Lower Flange</td><td>48<103</td></tr>
</table>

Source: JICA Study Team

5) Design of Stringer

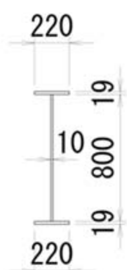
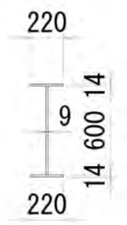
a) Design Case for Stringer

・Load Case: Dead Load + Live load + Impact

b) Shape of Stringer and Stress Check

The calculation result of stress is shown in Table 4.6.32.

Table 4.6.32 Stress of Stringer

<table>
<tr><th colspan="2">Inside Stringer</th><th colspan="2">Outside Stringer</th></tr>
<tr><td colspan="2">
</td><td colspan="2">
</td></tr>
<tr><td></td><td>Stress (N/mm^2)</td><td></td><td>Stress (N/mm^2)</td></tr>
<tr><td>Flange</td><td>101 < 136 OK</td><td>Flange</td><td>116 < 140 OK</td></tr>
<tr><td>Web</td><td>40 < 80 OK</td><td>Web</td><td>30 < 80 OK</td></tr>
</table>

Source: JICA Study Team

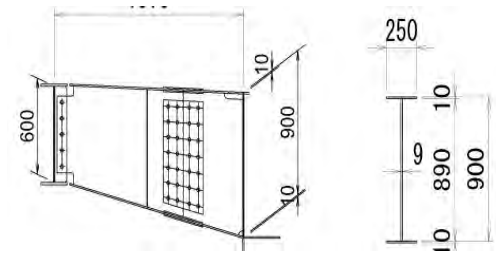
6) Design of Bracket

− Design Case for Bracket

・Load Case: Dead Load + Live load + Impact

The calculation result of stress is shown in Table 4.6.31.

Table 4.6.33 Stress of Bracket



Flange	250x10	104.7 < 135 OK
Web	890x9	28.3 < 80 OK

Source: JICA Study Team

7) Checking of Welding by Fatigue Accumulated Stress-Range Cycles

The additional finish of the welding is not necessary as a result of fatigue check. The welding positions to be checked and the results are shown in Table 4.6.34.

Table 4.6.34 Results of Fatigue Check

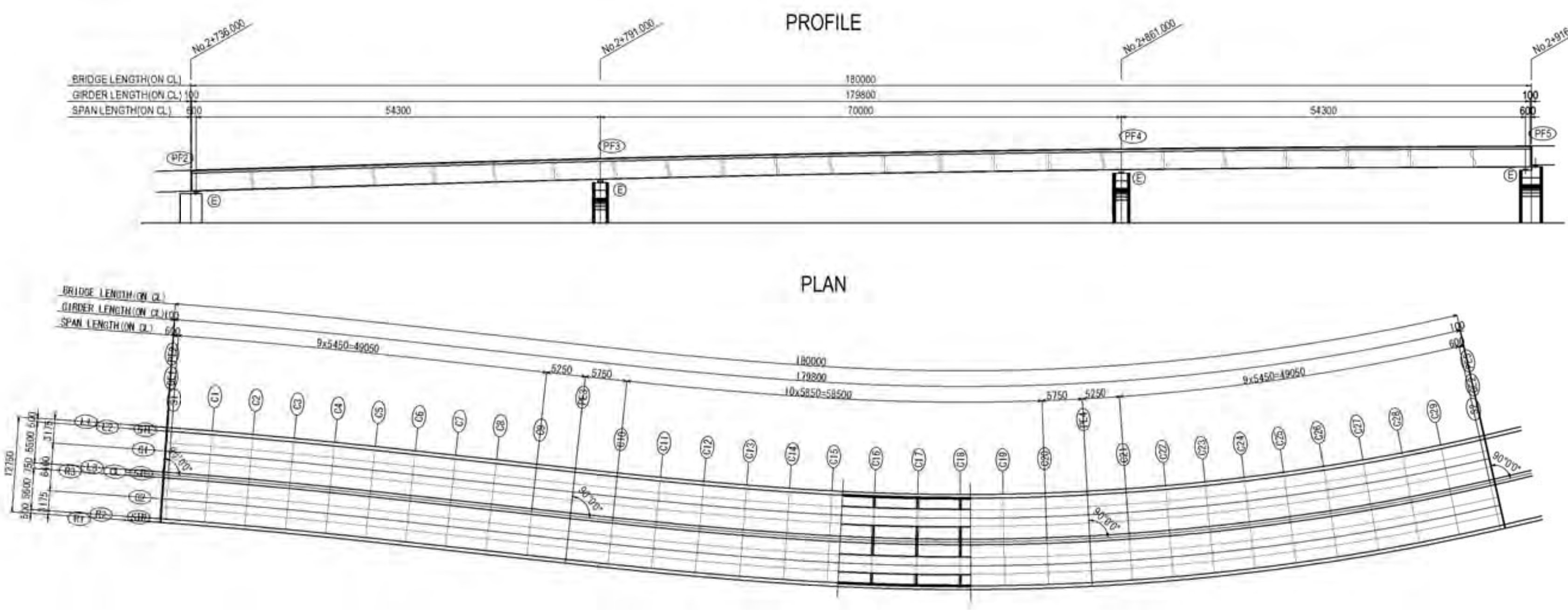
	Check Point Maximum Stress Range	Accumulated Stress-range Cycles	
Flange – Web	43 N/mm^2 < 46 N/mm^2	---	OK
Web – Stiffener	21 N/mm^2 < 42 N/mm^2	---	OK
Web－Gusset	27 N/mm^2 < 32 N/mm^2	---	OK

Source: JICA Study Team

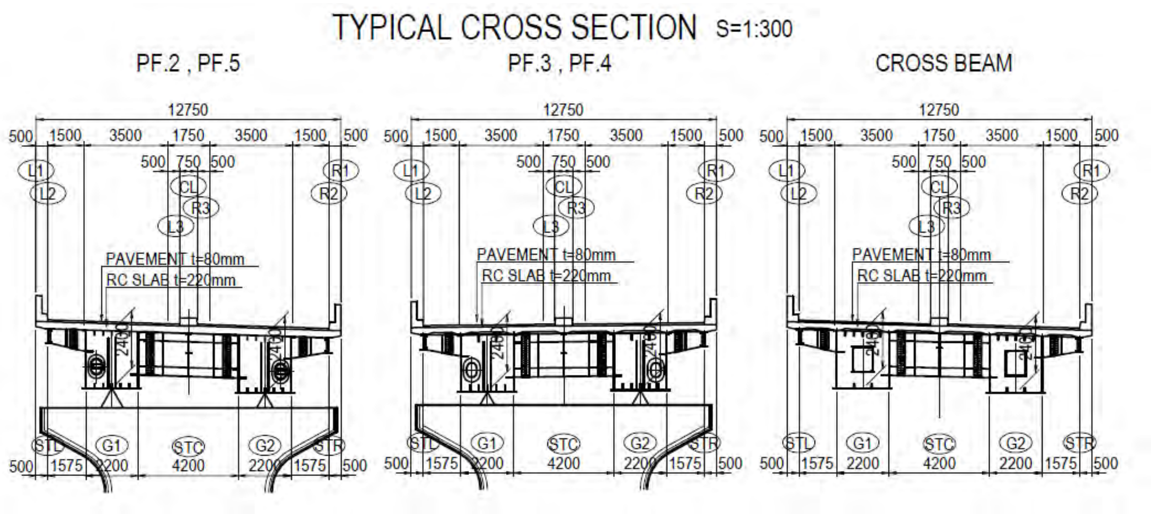
8) General View of Steel Box Girder Bridge

The following figures show the profile, plan, and typical cross section of the steel box girder bridge.

Figure 4.6.19 Plan and Profile of Steel Box Girder



Source: JICA Study Team



Source: JICA Study Team

Figure 4.6.20 Typical Cross Section of Steel Box Girder

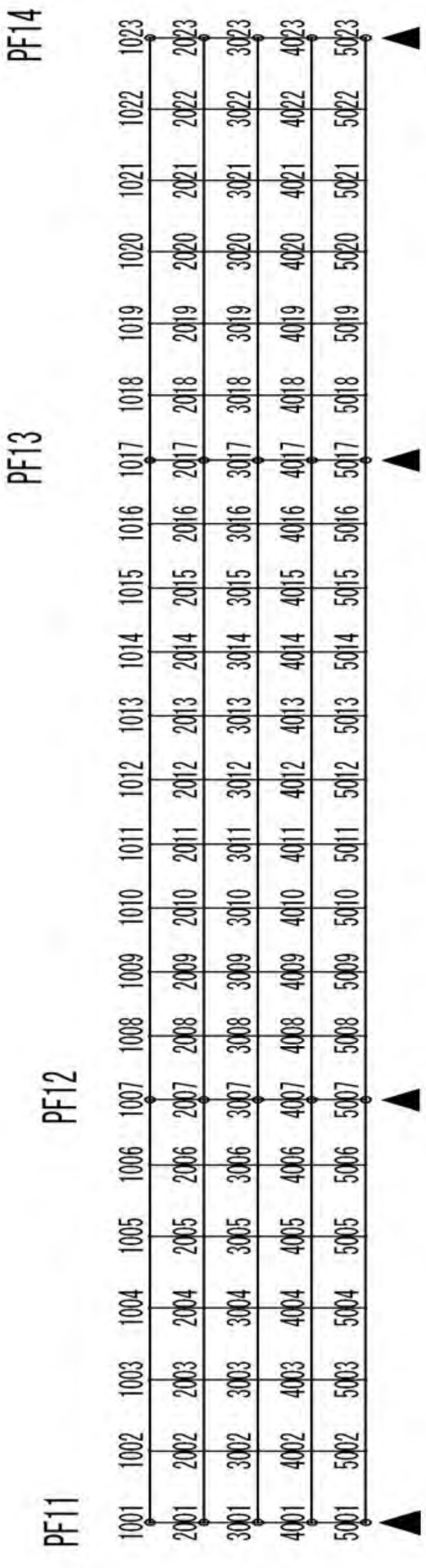
(3) Detailed Design of Steel-I Girder Bridge

1) Analytical Modelling

a) Grillage Analysis

This bridge is made of beam structures, and both girders and crossbeams are modelled as linear beam elements with two dimensions. The geometrical moment of inertia and physical property shall be inputted as linear elements, and dead load and live load are put on the grillage model. The load is statically distributed by the crossbeams between the longitudinal members. The grillage model is analyzed by the displacement method; and the sectional force and displacement at an arbitrary point and the support reaction are calculated using the line of influence.

b) Grillage Analysis



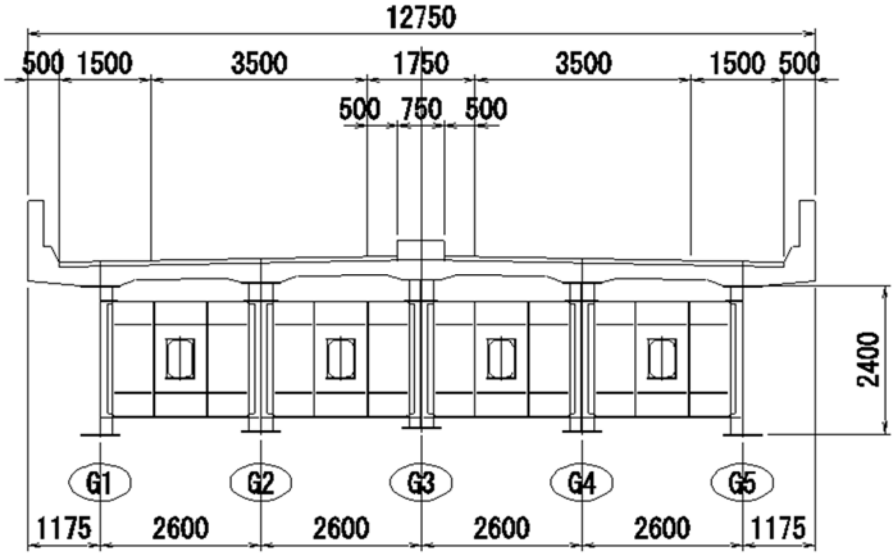
Source: JICA Study Team

Figure 4.6.21 Grid Model

2) Design of Main Girder

a) Arrangement of Main Girder

The arrangement of the main girder is decided by the supporting span of the deck. The desired spacing of the main girder with RC deck is 3 m or less, and the desired overhang length of the deck is 1.2 m or less. As a result, the arrangement of the main girder is as shown in Figure 4.6.22.



Source: JICA Study Team

Figure 4.6.22 Arrangement of Main Girder

b) Allowable Stress of Flange

If the compression flange is directly fixed to a concrete deck, then almost no lateral buckling occurs, therefore, the lateral buckling can be neglected in the design. However, the lower flange is designed by the allowable bending compressive stress.

c) Shape of Flange

The thickness of a projecting plate subjected to compressive stress shall be 1/16 of the projecting width.

The flange width is adjusted to be about 1/4 of the main girder height.

d) Connection of Members

The joining method of the main girder is friction grip connection with high strength bolts. The number of bolt rows should be eight or less. When designing a connection, fillers shall be used to adjust the physical gaps between the base metals.

The base material of the splice plate and bolt connecting part shall be coated with inorganic zinc paint.

e) Checking Item

The girders are designed based on the sectional forces calculated by the grillage analysis, and each member is designed so that its stress is less than the allowable unit stress under each load combination. The checking items are shown in Table 4.6.35

Table 4.6.35 Checking Items for Main Girder Design

Members	Checking Item
Flange	- Compressive stress - Tensile stress
Web	- Shear stress
Flange and Web	- Verification of resultant stress
Girder	- Deflection

Source: JICA Study Team

f) Stress of the Main Girder

The width of the flange should be smaller than 1/3 of the girder height and as for the thickness, 40 mm or less is better.

In the continuous girder, as for the intermediate support, the negative bending moment is maximum, and the shearing force is also large. The lower flange is in compression state and the upper flange is in tension state.

In Section (4), which is the negative moment area, the upper flange is set in such a way that its stress is smaller than the allowable tensile stress. Also, the thickness of the lower flange is decided in such a way that its stress is smaller than the allowable stress in consideration of local buckling and lateral buckling.

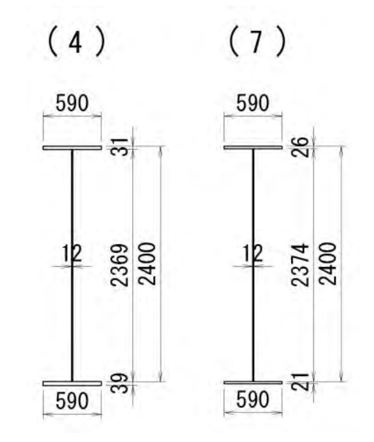
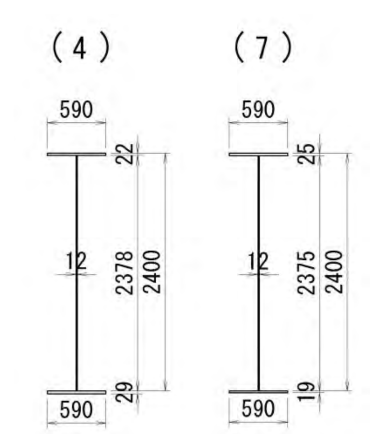
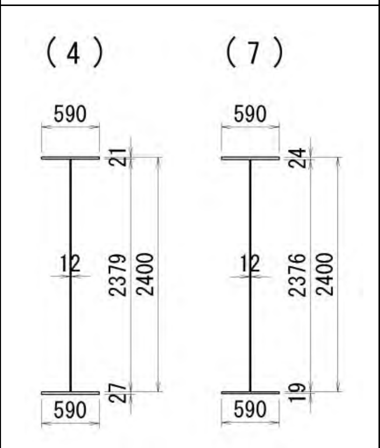
At the center of the span, the positive bending moment is maximum and the shear force is small. The lower flange is in tension state and the upper flange is in compression state. The thickness of the main girder flange is determined in such a way that the stress is less than the allowable stress.

Section (7) is in the positive moment range and the lower flange is set in such a way that its stress is smaller than the allowable tensile stress. Furthermore, the thickness of the upper flange is decided in such a way that its stress is smaller than the allowable stress considering local buckling.

The thickness of web plate is decided by the minimum thickness in which local buckling is not expected when the horizontal stiffener is installed in one stage.

The calculation result of stress is shown in Table 4.6.36.

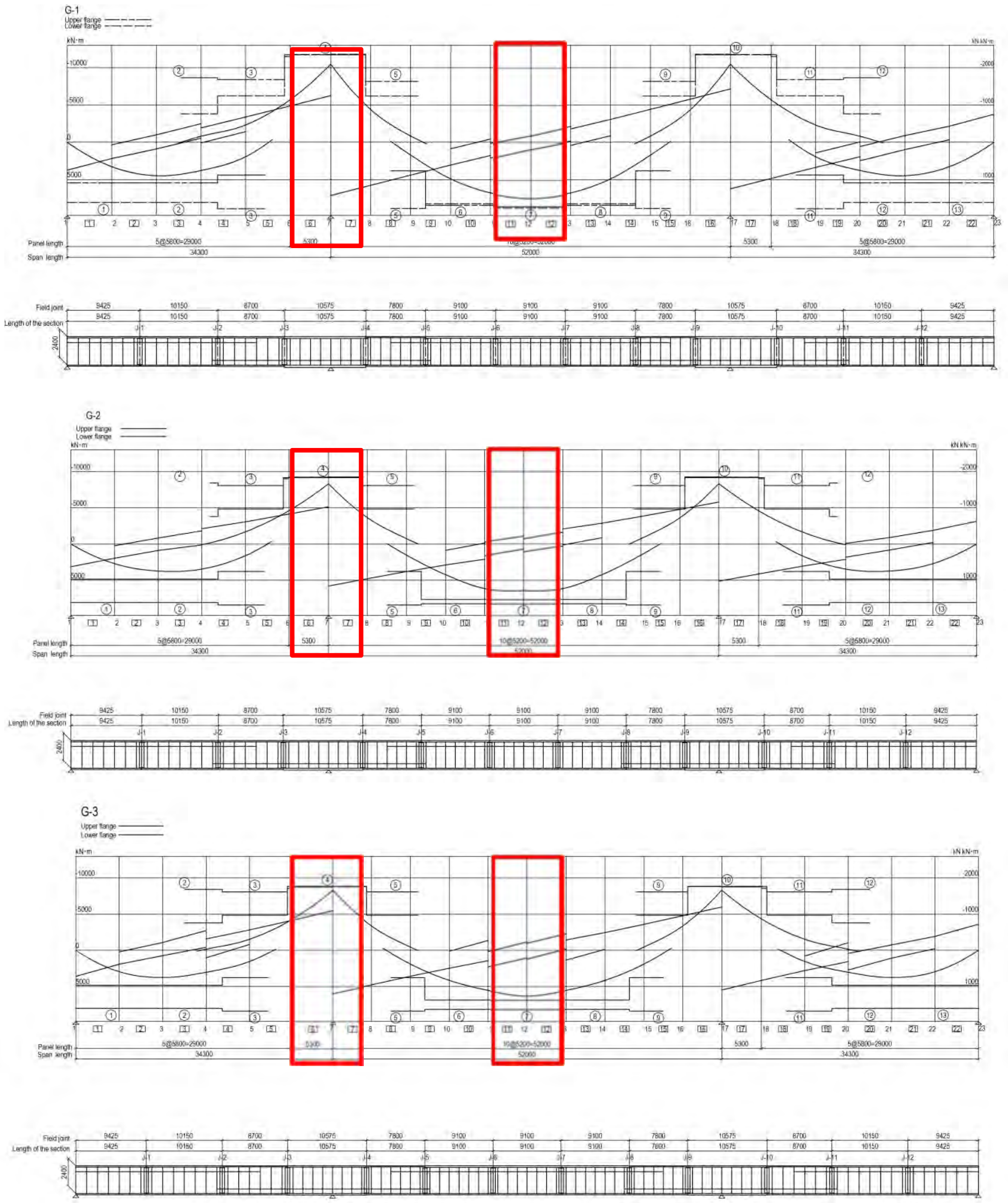
Table 4.6.36 Stress Check of Main Girder

Girder	Section	Shape			Stress (N/mm^2)	Judgement
G1,G5		(4)	Upper flange	590x32	**188<210**	OK
			Lower flange	590x39	-165<185	OK
			Web	2369x12	50<120.0	OK
			Resultant Stress	---	0.94<1.2	OK
		(7)	Upper flange	590x26	-162<186	OK
			Lower flange	590x21	**179<210**	OK
			Web	2374x12	6 <120.0	OK
			Resultant Stress	---	0.71<1.2	OK
G2,G4		(4)	Upper flange	590x22	**191<210**	OK
			Lower flange	590x29	-167<185	OK
			Web	2378x12	41<120.0	OK
			Resultant Stress	---	0.92<1.2	OK
		(7)	Upper flange	590x25	**-144<172**	OK
			Lower flange	590x19	164<210	OK
			Web	2374x12	8<120	OK
			Resultant Stress	---	0.59<1.2	OK
G3		(4)	Upper flange	590x21	**198<210**	OK
			Lower flange	590x27	-175<185	OK
			Web	2379x12	42<120.0	OK
			Resultant Stress	---	0.98<1.2	OK
		(7)	Upper flange	590x24	**-146<159**	OK
			Lower flange	590x19	162<210	OK
			Web	2376x12	8 <120.0	OK
			Resultant Stress	---	0.58<1.2	OK

Source: JICA Study Team

g) Moment Diagram

The moment diagram of the main girders is shown in Figure 4.6.23.



Source: JICA Study Team

Figure 4.6.23 Moment Diagram

h) Verification of Safety for Loads

The deflection is checked to ensure the stability of the structure and safety. The allowable value of deflection is defined by the equation below.

L/(20000/L) (10< L<40)

L/500 (40<L)

where, L: span length (m)

Table 4.6.37 Verification of Deflection

Check Position	Girder Number	Span Length (m)	Deflection Value (mm)	Allowable Value (mm)	Judgement
First Span	G1,G5	35	23	59	OK
	G2,G4	52	18	59	OK
	G3	35	18	59	OK
Second Span	G1,G5	52	54	104	OK
	G2,G4	52	46	104	OK
	G3	52	43	104	OK
Third Span	G1,G5	35	23	59	OK
	G2,G4	35	18	59	OK
	G3	35	18	59	OK

Source: JICA Study Team

3) Design of RC Deck Slab

a) Thickness of Deck

The thickness of the deck is decided from the following equation:

$d = 1.25 \times (30L+110) = 235 \text{ mm} \Rightarrow 240 \text{ mm}$

1.25: coefficient related to traffic volume of large vehicles

b) Design Bending Moment of Deck

This slab is designed according to the JSHB II. The reason is that the weight of the design wheel of JSHB T-Load is 200 kN while the weight of the axle and the wheel of the design car of AASHTO HL 93 is 145 kN, so it is better for the RC slab to be designed with a larger force. Therefore, the bending moment of the deck is calculated as the unit width (1 m) of the T-Load specified in the JSHBII 8.2.4.

c) Arrangement of Reinforcement Bars

The reinforcement bars of the deck are arranged with deformed rebar with diameters of 13, 16 or 19. The concrete cover is 30 mm or more. The center-to-center distance of the main reinforcement is a minimum of 100 mm or a maximum of 300 mm.

d) Checking Item

The depth of deck and arrangement of reinforcement bars are determined in such a way that the sectional stress calculated by the sectional force is less than the allowable unit stress. If the influence of the differential settlement is not considered, it is desirable that the allowable stress of reinforcement bar has a margin of approximately 20 N/mm^2 with respect to the allowable stress of 140 N/mm^2.

Table 4.6.38 Checking Items for RC Deck Slab Design

Materials	Checking Item
Concrete	- Compressive stress
Reinforcement Bar	- Tensile stress

Source: JICA Study Team

e) Stress of RC Slab

The calculation result of stress is shown in Table 4.6.39.

Table 4.6.39 Stress of RC Slab

Bending Moment	Main Reinforcement Direction Stress (N/mm^2)			Transverse Reinforcement Direction Stress (N/mm^2)		
Cantilever Slab Girder End	D19@150			---		
	Concrete	5.7<8.0	OK	---	---	---
	Reinforcement Bar	**110<140**	OK	---	---	---
Cantilever Slab Standard	D19@150			D16@150		
	Concrete	4.1<8.0	OK	Concrete	2.1<8.0	OK
	Reinforcement Bar	**120<140**	OK	Reinforcement Bar	65<140	OK
Center of Span	D19@150			D16@150		
	Concrete	4.4<8.0	OK	Concrete	-4.4<8.0	OK
	Reinforcement Bar	**105<140**	OK	Reinforcement Bar	**114<140**	OK
Support	D19@150			---		
	Concrete	-2.8<8.0	OK	---	---	---
	Reinforcement Bar	47<140	OK	---	---	---

Source: JICA Study Team

f) Design of Crossbeam

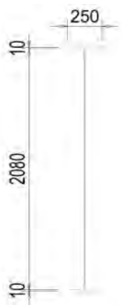
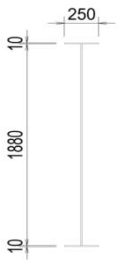
− Design Case for Crossbeam

・Load Case: Dead Load + Live load + Impact

− Shape of Crossbeam and Stress Check

The calculation result of stress is shown in Table 4.6.40.

Table 4.6.40 Stress of Crossbeam

End Crossbeam		Intermediate Crossbeam	
			
	Stress (N/mm^2)		Stress (N/mm^2)
Flange	16 < 140 OK	flange	74 < 119
Web	19 < 80 OK	Web	12 < 80

Source: JICA Study Team

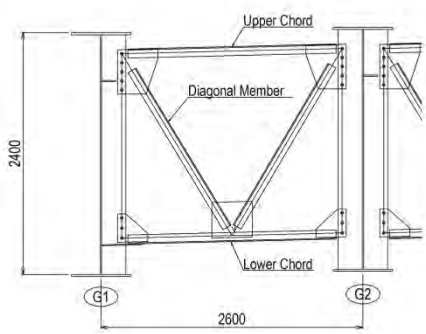
4) Design of Sway Bracing

− Design Case for Sway Bracing

・Load Case: Wind, Earthquake

The calculation result of stress is shown in Table 4.6.41.

Table 4.6.41 Stress of Sway Bracing



Upper & Lower Chord L-90*90*10*10	Slenderness ratio	149 < 150 OK
	Stress (N/mm^2)	16 < 41 OK
Diagonal member L-90*90*10*10	Slenderness ratio	129 < 150 OK
	Stress (N/mm^2)	24 < 51 OK

Source: JICA Study Team

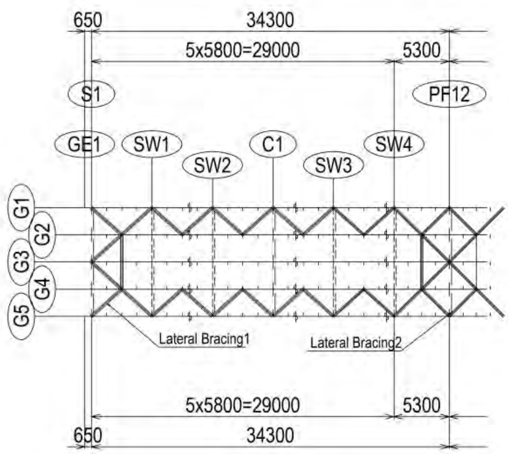
5) Design of Lateral Bracing

− Design Case for Lateral Bracing

・Load Case: Wind, Earthquake

The calculation result of stress is shown in Table 4.6.42.

Table 4.6.42 Result of Lateral Bracing



Lateral Bracing 1 CT-118*178*10*8	Slenderness Ratio	109< 150	OK
	Stress (N/mm^2)	24 < 39	OK
Lateral Bracing 2 BT-200*1334*16*16	Slenderness Ratio	83 < 150	OK
	Stress (N/mm^2)	21<50	OK

Source: JICA Study Team

(4) Checking of Welding by Fatigue Accumulated Stress-Range Cycles

The additional finish of the welding is not necessary as a result of fatigue check. The welding positions to be checked and the results are shown in Table 4.6.43.

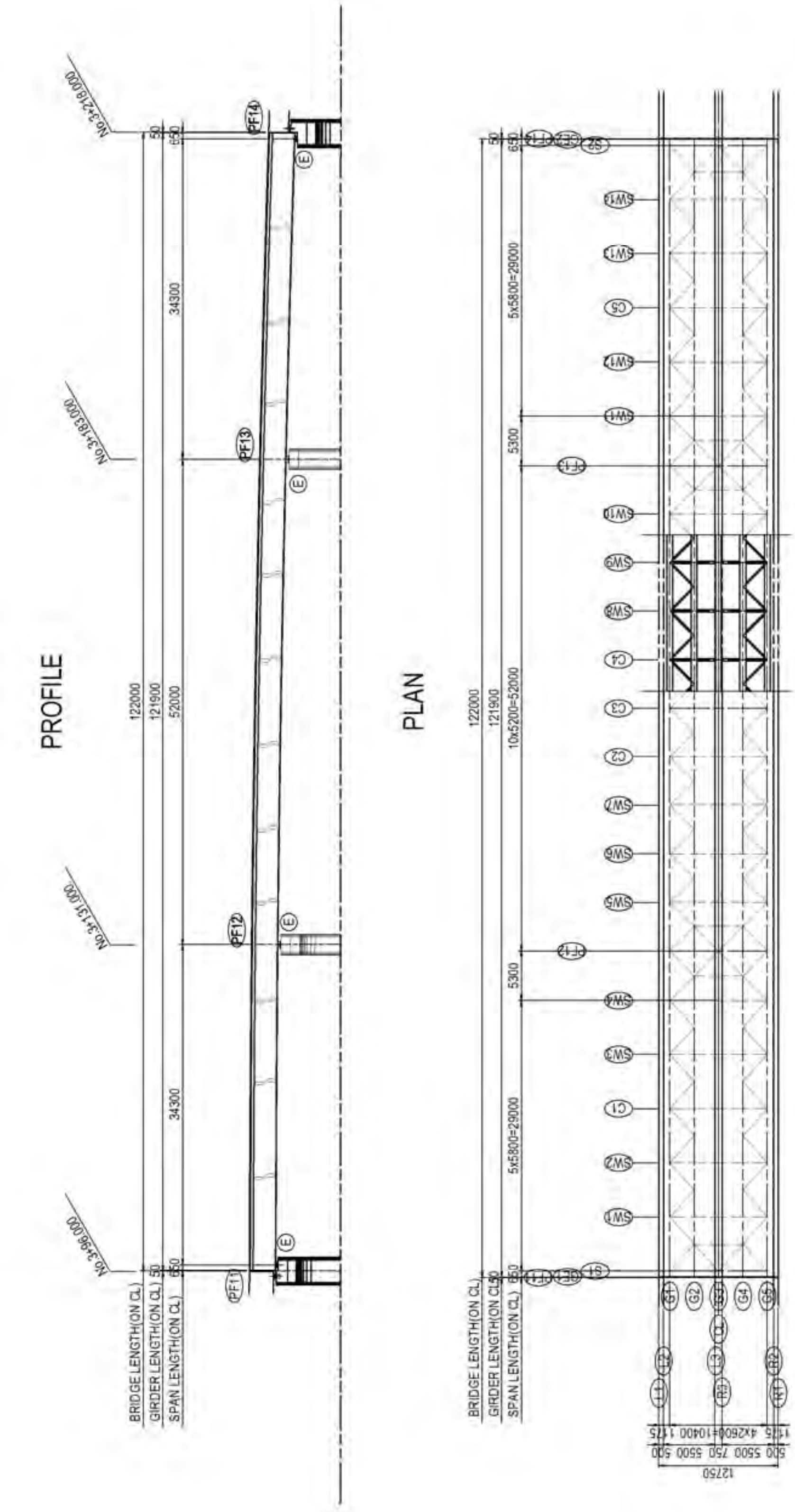
Table 4.6.43 Results of Fatigue Check

Check Point	Maximum Stress Range	Accumulated Stress-range Cycles	
Flange – Web	52 N/mm^2> 47 N/mm^2	0.54 < 1.0	OK
Web – Stiffener	56 N/mm^2> 62 N/mm^2	--	OK
Web－Gusset	40 N/mm^2> 32 N/mm^2	0.57 < 1.0	OK

Source: JICA Study Team

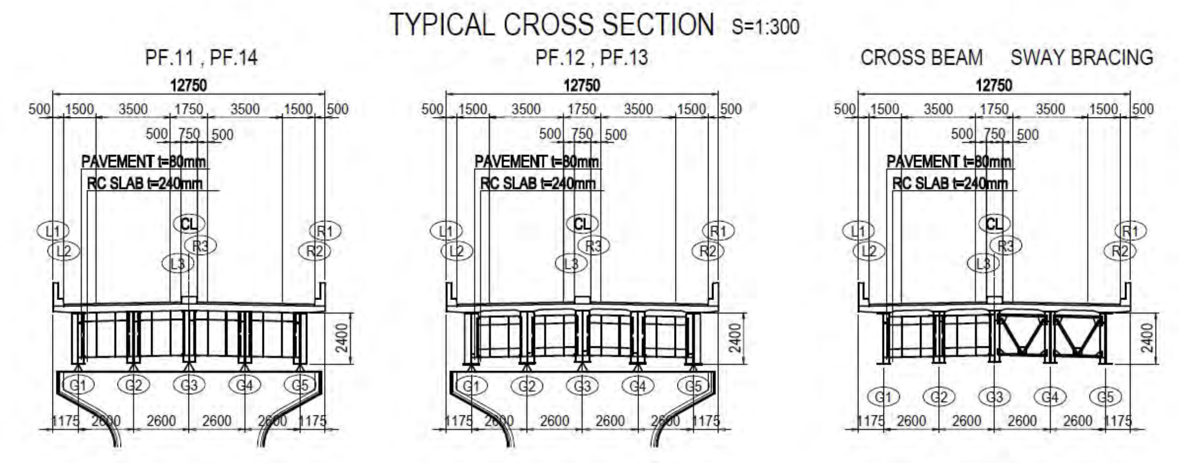
1) General View of Steel-I Girder

The following figures show the profile, plan, and typical cross section of the steel-I girder bridge.



Source: JICA Study Team

Figure 4.6.24 Plan and Profile of Steel-I Girder



Source: JICA Study Team

Figure 4.6.25 Typical Cross Section of Steel-I Girder

4.6.4.2 Detailed Design of PC-I Girder Bridge

The design of the PC-I Girder Bridge is described for the representative section between PF14 and PF15.

(1) Design Condition

1) General Design Condition

The general design conditions of the superstructure in PC-I Girder Bridge are shown in Table 4.6.44.

Table 4.6.44 General Design Conditions of Superstructure in PC-I Girder Bridge

Item	Conditions
General	
Bridge Type	(1) 2 spans continuous PC-I girder bridge with composite deck (Sta. 2+676.000 to Sta. 2+736.000) (2) 2 spans continuous PC-I girder bridge with composite deck (Sta. 2+916.000 to Sta. 3+96.000) (3) 2 spans continuous PC-I girder bridge with composite deck (Sta. 3+218.000 to Sta. 3+278.000)
Bridge Length	(1) 60.00 m, (2) 180.00 m, (3) 60.00 m
Girder Length	(1) 59.85 m, (2) 59.90 m, 119.80 m, (3) 59.85 m
Span Arrangement	(1) 28.80 m + 28.85 m, (2) 28.85 m+28.85 m, 4@28.80 m, (3) 28.85 m + 28.80 m
Total Width	12.75 m
Effective Width	11.00 m
Structure Type	
Deck Slab	Composite Structure (Prestressed Concrete and Reinforced Concrete)
Main Girder	Prestressed Concrete Structure (PC Structure)
Crossbeam	Prestressed Concrete Structure (PC Structure)
Connecting Part	Reinforced Concrete Structure (RC Structure)
Materials	
Concrete	Main Girder, PC Plate: 40 N/mm^2
	RC Deck, Crossbeam, Connecting Part: 30 N/mm^2
Re-bar	SD345
PC Strand	Main Girder: SWPR7BL, 7S15.2
	Crossbeam: SWPR7BL, 4S15.2
	PC Plate: SWPR7AL, 1S9.3

Source: JICA Study Team

2) Design of Superstructure

a) Design Case and Load Combination

PC strands are used for PC plate, main girder, and crossbeam. For the PC strand, it is necessary to check the tensile stress for the "Under Design Load" case. The design case and its load combination are shown in Table 4.6.45.

Table 4.6.45 Design Case and Load Combination for PC Strand for "Under Design Load" Case

	Design Case	Load Combination
PC Strand Design	During Prestressing	-
	Immediately after Prestressing	-
	Under Effective Prestressing Force	-
	During Deck Slab Construction (for Main Girder Design)	Self-Weight + Weight of Deck Slab + Effective Prestress Force + Secondary Bending Moment of Effective Prestress Force + Influence of Creep of Concrete + Influence of Drying Shrinkage of Concrete
	Under Dead Load (for Main Girder Design)	Self-Weight + Weight of Deck Slab and Bridge Surface + Effective Prestress Force + Secondary Bending Moment of Effective Prestress Force + Influence of Creep of Concrete + Influence of Drying Shrinkage of Concrete
	Under Live Load (for Main Girder Design)	Under Dead Load + Live Load + Impact

Source: Prepared by the JICA Study Team based on JSHB

It is needed to check the concrete stress in the PC structure and the concrete and re-bar stress in the RC structure for the "Under Design Load" case. For checking of PC-I girder bridge, the design case and its load combination are shown in Table 4.6.46.

Table 4.6.46 Design Case and Load Combination for Concrete and Re-bar for "Under Design Load" Case

Design Case	Load Combination
Immediately after Prestressing	Self-Weight + Prestress Force Immediately after Prestressing + Secondary Bending Moment of Prestress Force Immediately after Prestressing
Before Deck Slab Construction	Self-Weight + Effective Prestress Force + Secondary Bending Moment of Effective Prestress Force + Influence of Creep of Concrete + Influence of Drying Shrinkage of Concrete
During Deck Slab Construction	Self-Weight + Weight of Deck Slab + Effective Prestress Force + Secondary Bending Moment of Effective Prestress Force + Influence of Creep of Concrete + Influence of Drying Shrinkage of Concrete
Under Dead Load (for Deck Slab Design)	Self-Weight + Weight of Bridge Surface + Effective Prestress Force + Secondary Bending Moment of Effective Prestress Force + Influence of Creep of Concrete + Influence of Drying Shrinkage of Concrete
Under Dead Load (for Main Girder, Crossbeam and Connection Part Design)	Self-Weight + Weight of Deck Slab and Bridge Surface + Effective Prestress Force + Secondary Bending Moment of Effective Prestress Force + Influence of Creep of Concrete + Influence of Drying Shrinkage of Concrete
Under Live Load	Under Dead Load + Live Load + Impact
Under Collision Load	Under Dead Load + Live Load + Impact + Collision Load
Under Wind Load (with Live Load)	Under Dead Load + Wind Load + Live Load + Impact
Under Wind Load (without Live Load)	Under Dead Load + Wind Load
Under Temperature Load	Under Dead Load + Live Load + Impact + Effective Temperature Change

Source: Prepared by the JICA Study Team based on JSHB

The main girder, crossbeam, and connecting parts are needed to be checked for the "Under Ultimate Load" case. The "Under Ultimate Load" case is further divided into three cases. The details of the design cases are shown in Table 4.6.47.

Table 4.6.47 Design Case and Load Combination for the "Under Ultimate Load" Case

Design Case	Load Combination
Case 1	1.3 x (Under Dead Load) + 2.5 x (Live Load + Impact) +1.0 x (Statically Indeterminate Force)
Case 2	1.0 x (Under Dead Load) + 2.5 x (Live Load + Impact) +1.0 x (Statically Indeterminate Force)
Case 3	1.7 x (Under Dead Load) + 1.7 x (Live Load + Impact) +1.0 x (Statically Indeterminate Force)

Source: Prepared by the JICA Study Team based on JSHB

b) Checking Item for Deck Slab Design

For the design of deck slab, the weights of axles and wheels for the design truck under the AASHTO HL93 is 145 kN. On the other hand, the weights of wheels for the design under the Specifications for Highway Bridges is 200 kN. The stress of slab is better to be designed with large force. Therefore, the bending moment of the deck is calculated by the unit width (1 m) due to the T-load as specified in the Japanese Standard (Specifications for Highway Bridges 2012 II 8.2.4).

PC Plate Design

The checking items for the PC plate design are shown in Table 4.6.48.

Table 4.6.48 Checking Items for PC Plate Design

Checking Item	Load Condition
Tensile Stress of PC Strand	• During Prestressing • Immediately after Prestressing • Under Effective Prestressing Force
Combined Flexural Stress	• Immediately after Prestressing • During Deck Slab Construction

Source: Prepared by the JICA Study Team based on JSHB

Design of Composite Deck Slab and RC Deck Slab

The checking items for design of the composite deck slab and RC deck slab are shown in Table 4.6.49.

Table 4.6.49 Checking Items for Design of Composite Deck Slab and RC Deck Slab

<table>
<tr><th>Location</th><th>Direction</th><th>Type of Deck Slab</th><th>Checking Item</th><th>Load Condition</th></tr>
<tr><td rowspan="3">Between Main Girders (Continuous Structure)</td><td rowspan="2">Transverse Direction</td><td>Composite Deck Slab</td><td>- Combined Flexural Stress (Volume of Tension Re-bar*1)</td><td>• Under Live Load</td></tr>
<tr><td>RC Deck Slab</td><td>- Compressive Stress of Concrete
- Tensile Stress of Re-bar</td><td>• Under Live Load</td></tr>
<tr><td>Longitudinal Direction</td><td>RC Deck Slab</td><td>- Compressive Stress of Concrete
- Tensile Stress of Re-bar</td><td>• Under Live Load</td></tr>
<tr><td rowspan="2">Outside of Girder (Cantilever Structure)</td><td>Transverse Direction</td><td rowspan="2">RC Deck Slab</td><td rowspan="2">- Compressive Stress of Concrete
- Tensile Stress of Re-bar</td><td>• Under Dead Load
• Under Live Load
• Under Collision Load
• Under Wind Load (with/without Live Load)</td></tr>
<tr><td>Longitudinal Direction</td><td>• Under Live Load</td></tr>
</table>

Source: Prepared by the JICA Study Team based on JSHB

*1 For the checking of the volume of tension reinforcement bar, if the tensile stress occurs due to the combined flexural stress, the minimum tension reinforcement bar volume written in the JSHB is placed in the RC deck slab. If the tensile stress does not occur due to the combined flexural stress, the minimum tension reinforcement bar volume and required tension reinforcement bar volume are calculated. The larger one is placed in the RC deck slab.

c) Checking Item for Main Girder Design

The checking items for the main girder design are the bending moment and the shear force. These forces are checked for the "Under Design Load" and "Under Ultimate Load" cases and the details of the checking items are shown in Table 4.6.50.

Table 4.6.50 Checking Items for Main Girder Design

Type of Force	Design Case	Checking Item	Load Condition
Bending Moment	Under Design Load	- Tensile Stress of PC Strand	• During Prestressing • Immediately after Prestressing • During Deck Slab Construction • Under Live Load
		- Combined Flexural Stress (Volume of Tension Re-bar[*1])	• Immediately after Prestressing • During Deck Slab Construction • Under Dead Load • Under Live Load • Under Temperature Load
	Under Ultimate Load	- Bending Moment to Failure	• Case 1 • Case 2 • Case 3
Shear Force	Under Design Load	- Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar[*2])	• Under Live Load • Under Temperature Load
		- Diagonal Tensile Stress of Concrete	• Under Dead Load • Under Live Load • Under Temperature Load
	Under Ultimate Load	- Web Concrete against Compressive Strength to Failure	• Case 1 • Case 2 • Case 3
		- Members against Diagonal Tensile Strength to Failure[*3]	

Source: Prepared by the JICA Study Team based on JSHB

*1 For the checking of the volume of tension reinforcement bar, if the tensile stress occurs due to the combined flexural stress, the minimum tension reinforcement bar volume written in the JSHB is placed in the main girder. If the tensile stress does not occur due to the combined flexural stress, the minimum tension reinforcement bar volume and required tension reinforcement bar volume are calculated. The larger one is placed in the main girder.

*2 For the checking of the mean shear stress, if the checking is satisfied, the minimum diagonal tension reinforcement bar volume written in the JSHB is placed in the main girder. If the checking is not satisfied, the minimum diagonal tension reinforcement bar volume and required diagonal tension reinforcement bar volume are calculated. The larger one is placed in the main girder.

*3 For the checking of members against diagonal tensile strength failure, if the checking for mean shear stress of concrete is not satisfied, the checking of members against diagonal tensile stress failure is calculated.

d) Checking Item for Crossbeam Design

Crossbeam Design at the End of Main Girder

The checking items for the crossbeam design at the end of the main girder are bending moment and shear force. These forces are checked for the “Under Design load” and “Under Ultimate load” cases and the details of the checking items are shown in Table 4.6.51.

Table 4.6.51 Checking Items for Crossbeam Design at the End of Main Girder

<table>
<tr><th>Type of Force</th><th>Design Case</th><th>Checking Item</th><th>Load Condition</th></tr>
<tr><td rowspan="2">Bending Moment</td><td rowspan="2">Under Design Load</td><td>- Tensile Stress of PC Strand</td><td>• During Prestressing
• Immediately after Prestressing
• Under Effective Prestressing Force</td></tr>
<tr><td>- Combined Flexural Stress (Volume of Tension Re-bar*1)</td><td>• Under Dead Load
• Under Live Load</td></tr>
<tr><td rowspan="4">Shear Force</td><td rowspan="2">Under Design Load</td><td>- Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar*2)</td><td>• Under Live Load</td></tr>
<tr><td>- Diagonal Tensile Stress of Concrete</td><td>• Under Dead Load
• Under Live Load</td></tr>
<tr><td rowspan="2">Under Ultimate Load</td><td>- Web Concrete against Compressive Strength to Failure</td><td rowspan="2">• Case 1
• Case 2
• Case 3</td></tr>
<tr><td>- Members against Diagonal Tensile Strength to Failure*3</td></tr>
</table>

Source: Prepared by the JICA Study Team based on JSHB

*1 For the checking of the volume of the tension reinforcement bar, if the tensile stress occurs due to the combined flexural stress, the minimum tension reinforcement bar volume written in JSHB is placed. If the tensile stress does not occur due to the combined flexural stress, the minimum tension reinforcement bar volume and required tension reinforcement bar volume are calculated. The larger one is placed.

*2 For the checking of the mean shear stress, if the checking is satisfied, the minimum diagonal tension reinforcement bar volume written in JSHB is placed. If the checking is not satisfied, the minimum diagonal tension reinforcement bar volume and required diagonal tension reinforcement bar volume are calculated. The larger one is placed.

*3 For the checking of members against diagonal tensile strength failure, if the checking for mean shear stress of concrete is not satisfied, the checking of members against diagonal tensile stress failure is calculated.

Intermediate Crossbeam Design

The checking items for the intermediate crossbeam design are bending moment and shear force. These forces are checked for the “Under Design load” and “Under Ultimate load” cases and the details of the checking items are shown in Table 4.6.52.

Table 4.6.52 Checking Items for Intermediate Crossbeam Design

Type of Force	Design Case	Checking Item	Load Condition
Bending Moment	Under Design Load	- Tensile Stress of PC Strand	• During Prestressing • Immediately after Prestressing • Under Effective Prestressing Force
		- Combined Flexural Stress (Volume of Tension Re-bar*1)	• Under Dead Load • Under Live Load
	Under Ultimate Load	- Bending Moment to Failure	• Case 1 • Case 2 • Case 3
Shear Force	Under Design Load	- Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar*2)	• Under Live Load
		- Diagonal Tensile Stress of Concrete	• Under Dead Load • Under Live Load
	Under Ultimate Load	- Compressive Strength to Failure - Members against Diagonal Tensile Strength to Failure	• Case 1 • Case 2 • Case 3

Source: Prepared by the JICA Study Team based on JSHB

*1 For the checking for volume of the tension reinforcement bar, if the tensile stress occurs by the combined flexural stress, the minimum tension reinforcement bar volume written in the JSHB is placed. If the tensile stress does not occur by the combined flexural stress, the minimum tension reinforcement bar volume and required tension reinforcement bar volume are calculated. The larger one is placed.

*2 For checking for the mean shear stress, if the checking is satisfied, the minimum diagonal tension reinforcement bar volume written in the JSHB is placed in the crossbeam. If the checking is not satisfied, the minimum diagonal tension reinforcement bar volume and required diagonal tension reinforcement bar volume are calculated. The larger one is placed in the crossbeam.

Crossbeam Design at Connecting Part

The checking items for the crossbeam design at connecting part are shown in Table 4.6.53.

Table 4.6.53 Checking Items for Crossbeam Design at Connecting Part

Checking Item	Load Condition
- Tensile Stress of PC Strand	• During Prestressing • Immediately after Prestressing • Under Effective Prestressing Force
- Mean Compressive Stress of Concrete	

Source: Prepared by JICA Study Team based on JSHB

e) Checking Items for Coupling Concrete Design at Connecting Part

The checking items for the coupling concrete design at connecting part are bending moment. These forces are checked under the "Under Design load" and "Under Ultimate load" and the detail of checking items is shown in Table 4.6.54.

Table 4.6.54 Checking Items for Coupling Concrete Design at Connecting Part

Type of Force	Design Case	Checking Item	Load Condition
Bending Moment	Under Design Load	- Compressive Stress of Concrete - Tensile Stress of Re-bar	• Under Dead Load • Under Live Load • Under Temperature Load
	Under Ultimate Load	- Bending Moment to Failure	• Case 1 • Case 2 • Case 3

Source: Prepared by JICA Study Team based on JSHB

3) Design Strength and Allowable Stress of Materials

a) Concrete

Concrete is applied in the deck slab, the main girder, the crossbeam and the coupling concrete at the connection part. Design Strength of 40 N/mm^2 is applied for the PC plate in the deck slab and main girder. A 30 N/mm^2 is applied for the RC slab in the deck slab, crossbeam and coupling concrete at the connecting part. The design strength and the allowable stress of these concrete are shown in Table 4.6.55.

Table 4.6.55 Design Strength and Allowable Stress of Concrete

Item			Unit	Main Girder	Crossbeam	Deck Slab	Coupling Concrete
Design Strength			N/mm^2	40.00	30.0	30.0	30.00
Compressive Strength during Prestressing			N/mm^2	34.00	25.00		25.00
Allowable Flexural Compressive Stress		Immediately after Prestressing	N/mm^2	19.00	14.00		
		Under Live Load	N/mm^2	14.00	11.00	10.00	10.00
		Under Temperature Load	N/mm^2	16.10			
Allowable Flexural Tensile Stress		Immediately after Prestressing	N/mm^2	-1.50	0.00		
		Under Dead Load	N/mm^2	0.00	0.00		
		Under Live Load	N/mm^2	-1.50	0.00		
		Under Temperature Load	N/mm^2	-2.00			
Mean Shear Stress		Under Design Load	N/mm^2	0.55	0.45		
		Under Ultimate Load	N/mm^2	5.30	4.00		
		Under Ultimate Load (Shear Force & Torsional Moment)	N/mm^2	6.10	4.80		
Allowable Diagonal Tensile Stress	Under Dead Load	Shear Force or Torsional Moment	N/mm^2	-1.00	-0.80		
		Shear Force & Torsional Moment	N/mm^2	-1.30	-1.10		
	Under Live Load	Shear Force or Torsional Moment	N/mm^2	-2.00	-1.70		
		Shear Force & Torsional Moment	N/mm^2	-2.50	-2.20		

Source: Prepared by JICA Study Team based on JSHB

b) PC Strand

PC strands are applied in the PC plate, the main girder and the crossbeam. The SWPR7BL is applied for the main girder and the crossbeam. The SWPR7AL is applied PC plates. The design strength and allowable stress of PC strand is shown in Table 4.6.56.

Table 4.6.56 Design Strength and Allowable Stress of PC Strand

Item		Unit	Main Girder Crossbeam	PC Plate
Material of PC Strand			SWPR7BL	SWPR7AL
Tensile Strength		N/mm^2	1850.0	1700.0
Yield Stress		N/mm^2	1600.0	1450.0
Allowable Tensile Stress	During Prestressing	N/mm^2	1440.0	1305.0
	Immediately after Prestressing	N/mm^2	1295.0	1190.0
	During Deck Slab Construction	N/mm^2	1100.0	1020.0
	Under Live Load	N/mm^2	1100.0	1020.0

Source: Prepared by JICA Study Team based on JSHB

c) Reinforcement Bar

Reinforcement bars are applied in the deck slab, main girder, crossbeam and coupling concrete at the connection part. The allowable stress of the reinforcement bar is shown in Table 4.6.57.

Table 4.6.57 Yield Strength and Allowable Stress of Reinforcement Bar

Item		Unit	Main Girder, Crossbeam, Coupling Concrete	Deck Slab
Yield Strength		N/mm^2	345	345
Allowable Tensile Stress	Under Dead Load	N/mm^2	100	100
	Under Live Load	N/mm^2	180	140
	Under Impact	N/mm^2	200	200

Source: Prepared by JICA Study Team based on JSHB

4) Design Parameters of Materials

a) Concrete

The design parameters of concrete are shown in Table 4.6.58.

Table 4.6.58 Design Parameters of Concrete

Item		Unit	Main Girder	Crossbeam, Deck Slab Connection Part
Design Strength		N/mm^2	40.00	30.00
Compressive Strength during Prestressing		N/mm^2	34.00	25.00
Young's Modulus	Under Live Load	N/mm^2	3.10×10^4	2.80×10^4
	Immediately after Prestressing	N/mm^2	2.92×10^4	2.58×10^4
Creep Coefficient		N/mm^2	2.60	2.60
Drying Shrinkage Strain		N/mm^2	20.0×10^{-5}	20.0×10^{-5}

Source: Prepared by JICA Study Team based on JSHB

b) PC Strand

The design parameters of PC strands are shown in Table 4.6.59.

Table 4.6.59 Design Parameters of PC Strands

Item		Unit	Main Girder, Crossbeam	PC Plate
Type of PC Strand		-	15.2 mm	9.3 mm
Material of PC Strand		-	SWPR7BL	SWPR7AL
Cross Sectional Area		mm^2	138.7	51.61
Young's Modulus		N/mm^2	2.00×10^5	2.00×10^5
Relaxation Rate		%	1.5	1.5
Amount of PC Strand Set		Mm	6.0	-
Friction Coefficient	λ	1/m	0.004	-
	μ	1/rad	0.300	-
	μ	1/rad	0.300	

Source: Prepared by JICA Study Team based on JSHB

c) Reinforcement Bar

The design parameters of reinforcement bar are shown in Table 4.6.60.

Table 4.6.60 Design Parameters of Reinforcement Bar

Item	Unit	Reinforcement Bar
Type of Reinforcement Bars	-	SD345
Young's Modulus	N/mm^2	2.00×10^5

Source: Prepared by JICA Study Team based on JSHB

5) Load Combination and Multiplication Factor

The load combination and the multiplication factor "Under Design Load" are shown in Table 4.6.61.

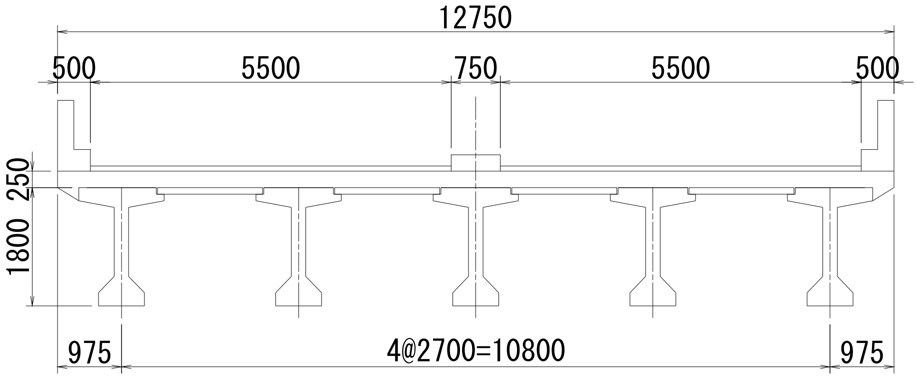
Table 4.6.61 Load Combination and Multiplication Factor "Under Design Load"

Load Combination	Multiplication Factor
Under Dead Load/ Under Live Load	1.00
Under Temperature Load	1.15
Under Wind Load	1.25
Under Collision Load	1.50
Under Earthquake	1.50
Under Erection Load	1.25

Source: Prepared by JICA Study Team based on JSHB

(2) Consideration of Superstructure Design

In the B/D, the reinforced concrete deck slab had been applied in 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 girder to the end of deck slab. The girder height is assumed to be 1.8 m by a conventional ratio, which is 1/17, to the average span length. Hence, the main girder on the cross section is shown in Figure 4.6.26.

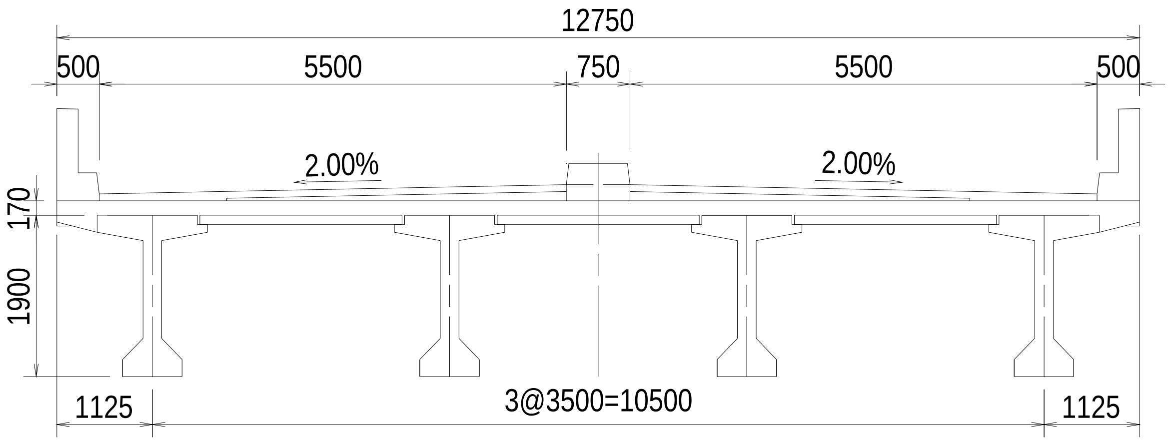


Source: JICA Study Team

Figure 4.6.26 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) is considered to be applied to the superstructure between the main girders to reduce the number of main girders. The span length of 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 is reduced. Finally, the main girder height is 1900 mm and number of main girders is four.

On the other hand, the structure type of overhang is the same as 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 4.6.27.

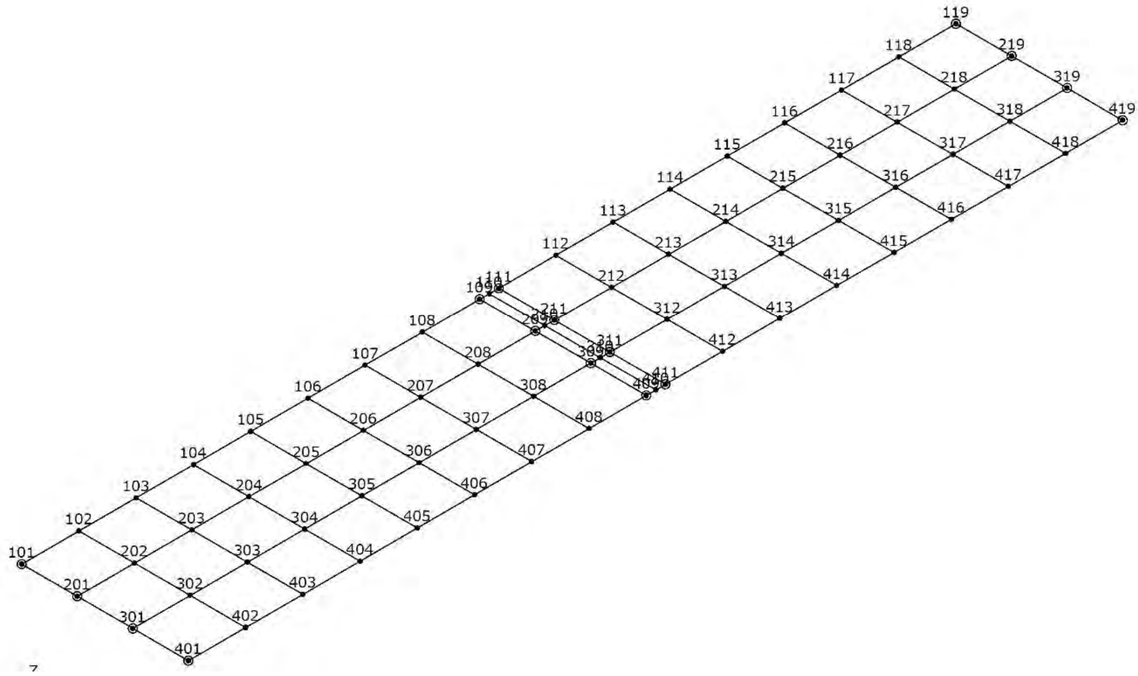


Source: JICA Study Team

Figure 4.6.27 Cross Section of Superstructure for PC-I Girder Bridge in the D/D

(3) Grid Model Analysis

The PC-I girder bridge has four main girders as shown in Figure 4.6.27. The main girder design is carried out after one main girder is selected from the four main girders by the grid model analysis. The plan of the grid model analysis is shown in Figure 4.6.28.



Source: JICA Study Team

Figure 4.6.28 Plan of Grid Model Analysis from PF14 to PF15

1) Result of Grid Model Analysis

The grid model analysis is carried out using the dead load and live load in the grid model as shown in Figure 4.6.28. From the result of the grid model analysis, the G2 has the largest sectional force, thus, the G2 main girder was selected for the design of the main girder. The sectional forces of G1 to G4 are shown in Table 4.6.62.

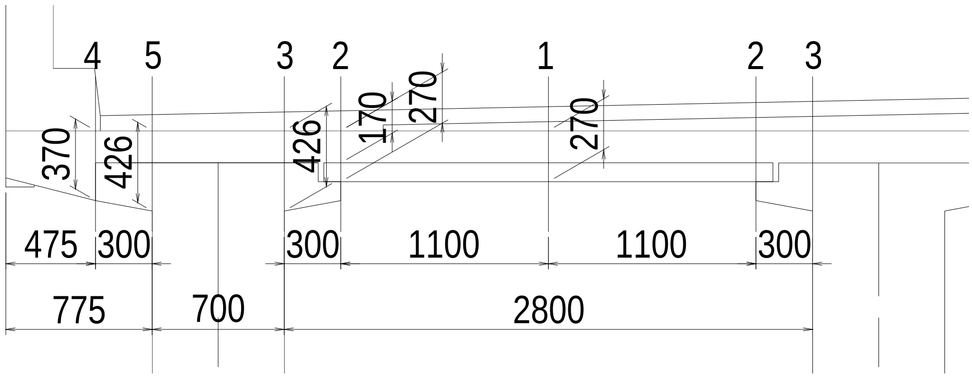
Table 4.6.62 Sectional Forces in Main Girder

Type of Force	Design Case	Unit	G1	**G2**	G3	G4
Bending Moment	Under Design Load	kNm	8047	**8317**	8317	8047
Shear Force	Under Dead Load	kN	-1110	**-1150**	-1150	-1110

Source: JICA Study Team

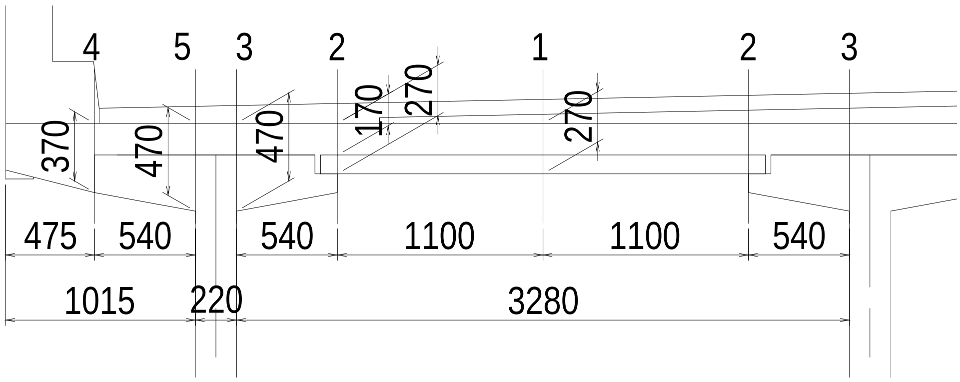
(4) Design of Deck Slab

The deck slab is designed for both directions, namely, transverse and longitudinal directions. For the design of the deck slab for the transverse direction, the checking points are “1” to “5” as shown in Figure 4.6.29 which is at the end of the girder and in Figure 4.6.30 which is at the center of the span. The “1” section is checked as composite concrete structure, while “2” to “5” sections are checked as reinforced concrete structure. For the design of the deck slab in the longitudinal direction, the deck slab is checked as reinforced concrete structure.



Source: JICA Study Team

Figure 4.6.29 Cross Section of Deck Slab at the End of Girder



Source: JICA Study Team

Figure 4.6.30 Cross Section of Deck Slab at the Center of Span

1) Design of PC Plate

The results of the design of PC plate are shown in Table 4.6.63.

Table 4.6.63 Result of Design of PC Plate

Unit: N/mm²

Load Condition	Checking Position	Combined Flexural Stress		Tensile Stress of PC Strand	
		Allowable Value	Result	Allowable Value	Result
During Prestressing		-	-	1305.0	1225.0
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190.0	1131.1
	Bottom of PC Plate		8.34		
During Deck Slab Construction (Under Effective Prestressing Force)	Top of PC Plate	0.0~15.0	10.58	1020.0	921.9
	Bottom of PC Plate		3.69		

Source: JICA Study Team

2) Design of Deck Slab between Main Girders

<u>**Deck Slab Design at End of Main Girder**</u>

The results of the design of deck slab between main girders at the end of the main girder are shown in Table 4.6.64 to Table 4.6.66.

Transverse Direction

Table 4.6.64 Result of Design of Deck Slab between Main Girders in Transverse Direction at “1” Section at End of Main Girder

Unit: N/mm²

Section	Load Condition	Checking Position	Combined Flexural Stress	
			Allowable Value	Result
1	Under Live Load	Top of RC Deck Slab	11.0	3.53
		Bottom of RC Deck Slab		-0.81
		Top of PC Plate	0.0 ~15.0	8.35
		Bottom of PC Plate		1.31

Source: JICA Study Team

Table 4.6.65 Result of Design of Deck Slab between Main Girders in Transverse Direction at “2” and “3” Sections at End of Main Girder

Unit: N/mm²

Section	Load Condition	Type of Force	Compressive Strength of Concrete		Tensile Stress of Re-bar	
			Allowable Value	Result	Allowable Value	Result
2	Under Live Load	Positive Moment	10.0	3.90	140.0	60.6
		Negative Moment		3.79		79.3
3	Under Live Load	Negative Moment		2.58		74.4

Source: JICA Study Team

Longitudinal Direction

Table 4.6.66 Result of Design of Deck Slab between Main Girders in Longitudinal Direction at End of Main Girder

Unit: N/mm²

Load Condition	Compressive Strength of Concrete		Tensile Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	8.66	140.0	100.2

Source: JICA Study Team

Deck Slab Design at Center of Span

The results of the design of deck slab between the main girders at the center of the span are shown in Table 4.6.67 to Table 4.6.69.

<u>Transverse Direction</u>

Table 4.6.67 Result of Design of Deck Slab between Main Girders in Transverse Direction at "1" Section at Center of Span

Unit: N/mm^2

Section	Load Condition	Checking Position	Combined Flexural Stress	
			Allowable Value	Result
1	Under Live Load	Top of RC Deck Slab	11.0	4.23
		Bottom of RC Deck Slab		-0.97
		Top of PC Plate	0.0~15.0	8.17
		Bottom of PC Plate		0.58

Source: JICA Study Team

Table 4.6.68 Result of Design of Deck Slab between Main Girders in Transverse Direction at "2" and "3" Sections at Center of Span

Unit: N/mm^2

Section	Load Condition	Type of Force	Compressive Strength of Concrete		Tensile Stress of Re-bar	
			Allowable Value	Result	Allowable Value	Result
2	Under Live Load	Positive Moment	10.0	6.71	140.0	104.3
		Negative Moment		3.27		68.4
3	Under Live Load	Negative Moment		2.56		78.6

Source: JICA Study Team

<u>Longitudinal Direction</u>

Table 4.6.69 Result of Design of Deck Slab between Main Girders in Longitudinal Direction at Center of Span

Unit: N/mm^2

Load Condition	Compressive Strength of Concrete		Tensile Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	9.96	140.0	115.25

Source: JICA Study Team

3) Design of Deck Slab outside the Main Girder

<u>Deck Slab Design at End of Main Girder</u>

The results of the design of the deck slab outside of the main girder at the end of the main girder are shown in Table 4.6.70 and Table 4.6.71.

<u>Transverse Direction</u>

Table 4.6.70 Result of Design of Deck Slab Outside the Main Girder in Transverse Direction at “4” and “5” Sections at End of Main Girder

Unit: N/mm²

Section	Load Condition	Compressive Strength of Concrete		Tensile Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result
4	Under Dead Load	-	-	100.0	5.39
	Under Live Load	10.0	0.21	140.0	5.39
	Under Collision Load	15.0	2.04	300.0	53.54
	Under Wind Load (with Live Load)	12.5	0.35	175.0	9.19
	Under Wind Load (without Live Load)	12.5	0.50	175.0	12.99
5	Under Dead Load	-	-	100.0	10.21
	Under Live Load	10.0	1.14	140.0	32.82
	Under Collision Load	15.0	2.59	300.0	74.65
	Under Wind Load (with Live Load)	12.5	1.26	175.0	36.16
	Under Wind Load (without Live Load)	12.5	0.59	175.0	16.90

Source: JICA Study Team

<u>Longitudinal Direction</u>

Table 4.6.71 Result of Design of Deck Slab Outside the Main Girder in Longitudinal Direction at the End of Main Girder

Unit: N/mm²

Load Condition	Compressive Strength of Concrete		Tensile Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	1.62	140.0	58.50

Source: JICA Study Team

<u>Deck Slab Design at Center of Span</u>

The results of the design of the deck slab outside the main girder at the center of the span are shown in Table 4.6.72 to Table 4.6.73.

Transverse Direction

Table 4.6.72 Result of Design of Deck Slab Outside the Main Girder for Transverse Direction at "4" and "5" Sections at Center of Span

Unit: N/mm²

Section	Load Condition	Compressive Strength of Concrete		Tensile Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result
4	Under Dead Load	-	-	100.0	5.39
	Under Live Load	10.0	0.21	140.0	5.39
	Under Collision Load	15.0	2.04	300.0	53.54
	Under Wind Load (with Live Load)	12.5	0.35	175.0	9.19
	Under Wind Load (without Live Load)	12.5	0.50	175.0	12.99
5	Under Dead Load	-	-	100.0	13.78
	Under Live Load	10.0	2.10	140.0	64.55
	Under Collision Load	15.0	3.33	300.0	102.54
	Under Wind Load (with Live Load)	12.5	2.20	175.0	67.62
	Under Wind Load (without Live Load)	12.5	0.65	175.0	19.90

Source: JICA Study Team

Longitudinal Direction

Table 4.6.73 Result of Design of Deck Slab Outside the Main Girder in Longitudinal Direction at Center of Span

Unit: N/mm²

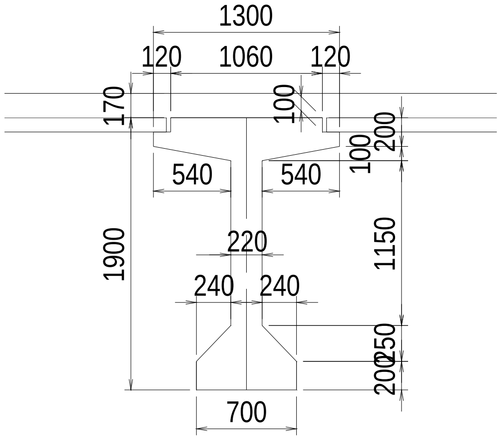
Load Condition	Compressive Strength of Concrete		Tensile Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	2.05	140.0	74.25

Source: JICA Study Team

(5) Main Girder Design

1) Cross Section of Main Girder

For the design of the main girder, the cross section of the main girder is assumed as shown in Figure 4.6.31.

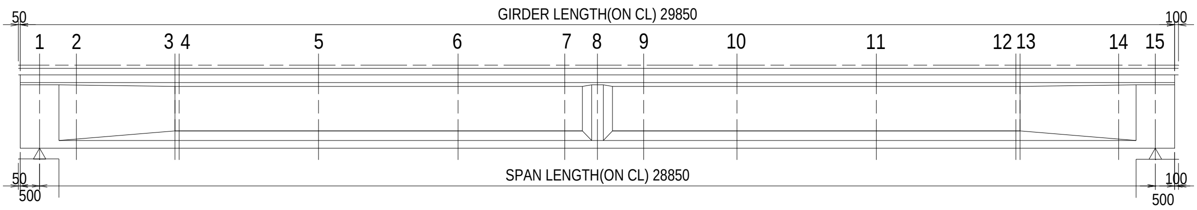


Source: JICA Study Team

Figure 4.6.31 Cross Section of Main Girder

2) Checking Position of Main Girder

The checking position for the bending moment and shear force is shown in Figure 4.6.32 and Table 4.6.74.



Source: JICA Study Team

Figure 4.6.32 Checking Position for Bending Moment and Shear Force

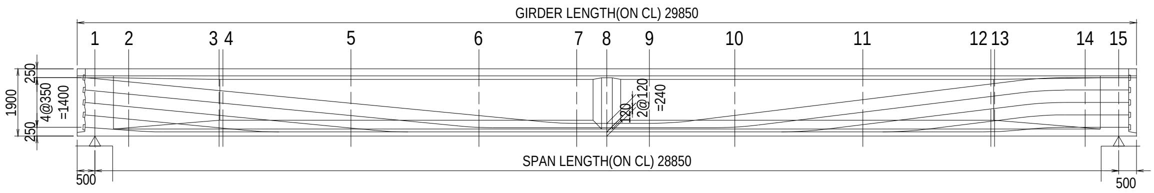
Table 4.6.74 Checking Position for Bending Moment and Shear Force

No.	Distance from Beginning of Girder	Description	No.	Distance from Beginning of Girder	Description
1	0.500 m	Supporting Point	9	16.121 m	Bend Point of PC Strand
2	1.450 m	Checking of Shear Force	10	18.531 m	Five-eighths of Span
3	4.000 m	Cross Section Changes	11	22.138 m	Six-eighths of Span
4	4.106 m	One-eighth of Span	12	25.744 m	Seven-eighths of Span
5	7.713 m	Two-eighths of Span	13	25.850 m	Cross Section Changes
6	11.319 m	Three-eighths of Span	14	28.400 m	Checking of Shear Force
7	14.078 m	Bend Point of PC Strand	15	29.350 m	Supporting Point
8	14.925 m	Center of Span			

Source: JICA Study Team

3) Arrangement of PC Strand

Five PC strands with specifications for 7S15.2 of SWPR7BL are placed in the main girder. The arrangement of the five PC strands is shown in Figure 4.6.33. The stress of the PC strand during prestressing is 1,300 N/mm^2.



Source: JICA Study Team

Figure 4.6.33 Arrangement of PC Strand in Main Girder

4) Design for Bending Moment

The design for bending moment under design load involves "Tensile Stress of PC Strand" and "Combined Flexural Stress (Volume of Tension Re-bar)". The "Bending Moment to Failure" will be checked for the bending moment under ultimate load.

Design for Tensile Stress of PC Strand

The result of the design for tensile stress for PC strands is shown in Table 4.6.75. The checking case is indicated for "During Prestressing", "Immediately after Prestressing", "During Deck Construction", and "Under Live Load".

Table 4.6.75 Result of Design for Tensile Stress of PC Strand

Unit: N/mm^2

Load Condition	Checking Position	Allowable Value	Result
During Prestressing	-	Under 1440	1300.00
Immediately after Prestressing	Cross Section "6"	Under 1295	1178.26
During Deck Construction	Cross Section "2"	Under 1100	1085.92
Under Live Load	Cross Section "1"	Under 1100	1013.90

Source: JICA Study Team

Design for Combined Flexural Stress (Checking for Volume of Tension Re-bar)

The result of the design for the combined flexural stress is shown in Table 4.6.76. The checking case is indicated for "Immediately after Prestressing", "During Deck Construction", "Under Dead Load", "Under Live Load", and "Under Temperature Load".

Table 4.6.76 Result of Design for Combined Flexural Stress

Unit: N/mm²

<table>
<tr><th>Load Condition</th><th>Section</th><th>Checking Position</th><th>Allowable Value</th><th colspan="2">Result</th></tr>
<tr><td rowspan="2">Immediately after Prestressing</td><td rowspan="18">Cross Section “8” (Center of Span)</td><td>Top of Girder</td><td rowspan="2">-1.5<σc<19.0</td><td colspan="2">0.84</td></tr>
<tr><td>Bottom of Girder</td><td colspan="2">14.70</td></tr>
<tr><td rowspan="2">During Deck Construction</td><td>Top of Girder</td><td rowspan="2">-1.5<σc<14.0</td><td colspan="2">6.36</td></tr>
<tr><td>Bottom of Girder</td><td colspan="2">6.74</td></tr>
<tr><td rowspan="3">Under Dead Load</td><td>Top of Deck</td><td>(Deck<10.0)</td><td colspan="2">2.78</td></tr>
<tr><td>Top of Girder</td><td rowspan="2">0.0<σc<14.0</td><td colspan="2">5.29</td></tr>
<tr><td>Bottom of Girder</td><td colspan="2">2.89</td></tr>
<tr><td rowspan="6">Under Live Load</td><td rowspan="2">Top of Deck</td><td rowspan="2">(Deck<10.0)</td><td>Max</td><td>4.26</td></tr>
<tr><td>Min</td><td>2.40</td></tr>
<tr><td rowspan="2">Top of Girder</td><td rowspan="4">-1.5<σc<14.0</td><td>Max</td><td>6.51</td></tr>
<tr><td>Min</td><td>4.98</td></tr>
<tr><td rowspan="2">Bottom of Girder</td><td>Max</td><td>-0.54</td></tr>
<tr><td>Min</td><td>3.77</td></tr>
<tr><td rowspan="6">Under Temperature Load</td><td rowspan="2">Top of Deck</td><td rowspan="2">(Deck<11.5)</td><td>Max</td><td>4.99</td></tr>
<tr><td>Min</td><td>3.13</td></tr>
<tr><td rowspan="2">Top of Girder</td><td rowspan="4">-2.0<σc<16.10</td><td>Max</td><td>5.80</td></tr>
<tr><td>Min</td><td>4.27</td></tr>
<tr><td rowspan="2">Bottom of Girder</td><td>Max</td><td>-0.85</td></tr>
<tr><td>Min</td><td>3.46</td></tr>
</table>

Source: JICA Study Team

The result of the checking for volume of tension reinforcement bars is shown in Table 4.6.77. In Table 4.6.76, the main girder has tensile stress “Under Live Load” and “Under Temperature Load”. The stress for “Under Temperature Load” case is larger than that for the “Under Live Load” case. Hence, the checking for volume of tension reinforcement bar is carried out for the “Under Temperature Load” case.

Table 4.6.77 Result of Volume of Tension Re-bar

Unit: cm²

Load Condition	Checking Position	Minimum Tension Re-bar Volume	Required Tension Re-bar Volume
Under Temperature Load	Cross Section “8” (Center of Span)	8.529	3.518

Source: JICA Study Team

From Table 4.6.77, the minimum volume of the tension reinforcement bar is placed in the main girder.

<u>Checking for Bending Moment to Failure (Under Ultimate Load)</u>

The result of the checking for the bending moment to failure is shown in Table 4.6.78. The critical case, “case 3”, is shown in Table 4.6.47.

Table 4.6.78 Result of Checking for Bending Moment to Failure

Checking Position	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
Cross Section “8” (Center of Span)	15295	12868	1.189

Source: JICA Study Team

5) Design for Shear Force

The design for shear force under design load involves "Mean Shear Stress (Volume of Diagonal Tension Reinforcement Bar)" and "Diagonal Tensile Stress of Concrete". For the "Under Ultimate Load" case, the "Web Concrete against Compressive Strength to Failure" and "Members against Diagonal Tensile Strength to Failure" will be checked.

Design for Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

The result of the design for the mean shear stress is shown in Table 4.6.79.

Table 4.6.79 Result of Design for Mean Shear Stress of Concrete

Unit: N/mm²

Checking Position	Allowable Value	Result
Cross Section "3"	τ_m<=0.55	1.276

Source: JICA Study Team

In Table 4.6.79, the calculated value is over the allowable value on the mean shear stress. Hence, the required volume of diagonal tension reinforcement bar is calculated as given in Table 4.6.80.

Table 4.6.80 Result of Volume of Diagonal Tension Re-bar

Unit: cm²

Checking Position	Minimum Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
Cross Section "3"	4.40	10.77

Source: JICA Study Team

From Table 4.6.80, the required volume of diagonal tension reinforcement bar is placed in the main girder.

Design for Diagonal Tensile Stress of Concrete

The result of the design for diagonal tensile stress of concrete is shown in Table 4.6.81.

Table 4.6.81 Result of Design for Diagonal Tensile Stress of Concrete

Unit: N/mm²

Load Condition	Section	Checking Position	Allowable Value	Result	
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-1.0	-0.11	
		Neutral Axis before Composition		-0.11	
		Neutral Axis after Composition		-0.12	
		Base of Lower Flange		-0.06	
Under Live Load	Cross Section "3"	Base of Upper Flange	σI=>-2.0	Max	-0.46
				Min	-0.09
		Neutral Axis before Composition		Max	-0.48
				Min	-0.09
		Neutral Axis after Composition		Max	-0.49
				Min	-0.09
		Base of Lower Flange		Max	-0.34
				Min	-0.05

Source: JICA Study Team

Checking for Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

The result of the checking for web concrete against compressive strength to failure is shown in Table 4.6.82.

Table 4.6.82 Result of Checking for Web Concrete against Compressive Strength to Failure

Checking Position	a) Compressive Strength to Failure (kN)	b) Acting Shear Force (kN)	Safety Degree a) / b)
Cross Section "3"	2709.7	1482.1	1.83

Source: JICA Study Team

Checking for Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

The acting shear force is shown in Table 4.6.83. If D13 is used as the diagonal tension re-bar, the interval of re-bar should be less than 235 mm. If D16 is used, the interval of re-bar should be less than 368 mm. After all, re-bar of D16 and 125 mm interval are applied.

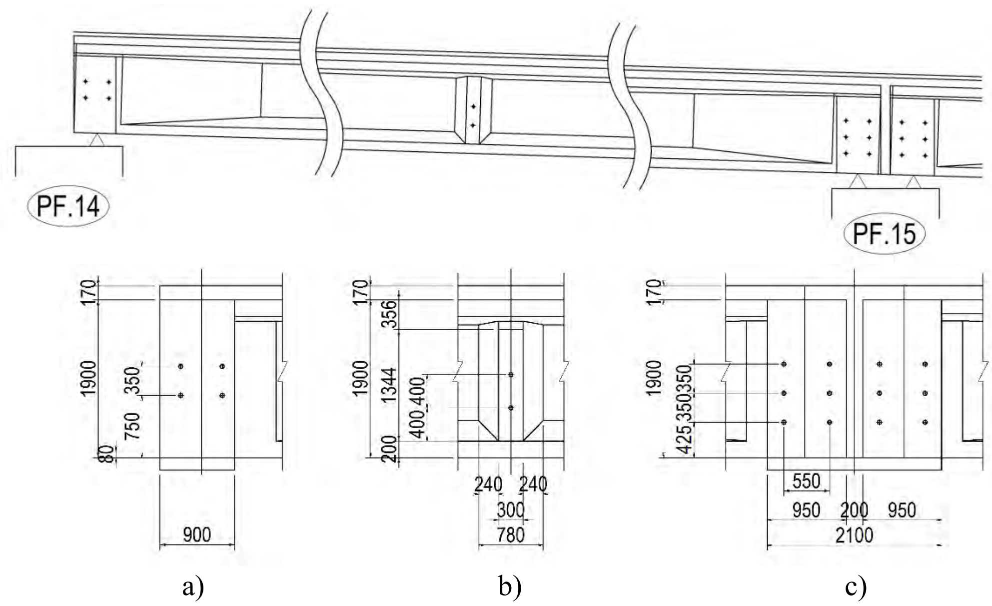
Table 4.6.83 Result of Acting Shear Force

Checking Position	Acting Shear Force (kN)
Cross Section "3"	1482.1

Source: JICA Study Team

(6) Design of Crossbeam

For the design of the crossbeam, it is divided into the following three parts: a) Crossbeam at the End of Main Girder, b) Intermediate Crossbeam, and c) Crossbeam at Connecting Part. Figure 4.6.34 shows the dimension of the crossbeam for the three parts.



Source: JICA Study Team

Figure 4.6.34 Dimension of Crossbeam

1) Crossbeam at the End of Main Girder

a) Design for Bending Moment

The design for the bending moment under design load includes "Tensile Stress of PC Strand" and "Combined Flexural Stress (Volume of Tension Re-bar)".

Design for Tensile Stress of PC Strand

The result of the design for tensile stress for PC strands is shown in Table 4.6.84. The checking case is indicated for "During Prestressing", "Immediately after Prestressing", and "Under Effective Prestressing Force".

Table 4.6.84 Result of Design for Tensile Stress of PC Strand

Unit: N/mm²

Load Condition	Allowable Value	Result
During Prestressing	Under 1440	1250.00
Immediately after Prestressing	Under 1295	1150.60
Under Effective Prestressing Force	Under 1110	1056.00

Source: JICA Study Team

Design for Combined Flexural Stress (Checking for Volume of Tension Re-bar)

The result of the design for combined flexural stress is shown in Table 4.6.85. The checking case is indicated for the "Under Dead Load" and "Under Live Load" cases.

Table 4.6.85 Result of Design for Combined Flexural Stress

Unit: N/mm²

Load Condition	Checking Position	Allowable Value	Result	
Under Dead Load	Top of Crossbeam	0.0 ~ 12.0		1.17
	Bottom of Crossbeam			1.58
Under Live Load	Top of Crossbeam	0.0 ~ 12.0	Max	1.61
			Min	0.89
	Bottom of Crossbeam		Max	1.11
			Min	1.88

Source: JICA Study Team

b) Design for Shear Force

The design for shear force under design load involves "Mean Shear Stress of Concrete (Volume of Diagonal Tension Reinforcement Bar)" and "Diagonal Tensile Stress of Concrete". The checking for the "Under Ultimate Load" case involve "Web Concrete against Compressive Strength to Failure" and "Members against Diagonal Tensile Strength to Failure".

Design for Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

The result of the design for the mean shear stress is shown in Table 4.6.86.

Table 4.6.86 Result of Design for Mean Shear Stress of Concrete

Unit: N/mm²

Allowable Value	Result
τ_m<=0.45	0.08

Source: JICA Study Team

Design for Diagonal Tensile Stress of Concrete

The result of the design for diagonal tensile stress of concrete is shown in Table 4.6.87.

Table 4.6.87 Result of Design for Diagonal Tensile Stress of Concrete

Unit: N/mm²

Load Condition	Allowable Value	Result
Under Dead Load	σI=>-0.80	0.000
Under Live Load	σI=>-1.70	-0.005

Source: JICA Study Team

Checking for Concrete against Compressive Strength to Failure (Under Ultimate Load)

The result of the checking for web concrete against compressive strength to failure is shown in Table 4.6.88.

Table 4.6.88 Result of Checking for Concrete against Compressive Strength to Failure

a) Compressive Strength to Failure (kN)	b) Acting Shear Force (kN)	Safety Degree a) / b)
5688	253	22.5

Source: JICA Study Team

Checking for Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

The result of the checking for members against diagonal tensile strength to failure is shown in Table 4.6.89.

Table 4.6.89 Result of Checking for Members against Diagonal Tensile Strength to Failure

a) Diagonal Tensile Strength to Failure (kN)	b) Acting Shear Force (kN)	Safety Degree a) / b)
2133	253	8.43

Source: JICA Study Team

2) Intermediate Crossbeam

a) Design for Bending Moment

The design for the bending moment under design load involes “Tensile Stress of PC Strand” and “Combined Flexural Stress (Volume of Tension Re-bar)”. The checking for the bending moment under ultimate load involves “Bending Moment to Failure”.

Design for Tensile Stress of PC Strand

The result of the design for tensile stress for PC strands is shown in Table 4.6.90. The checking case is indicated for “During Prestressing”, “Immediately after Prestressing”, and “Under Effective Prestressing Force”.

Table 4.6.90 Result of Design for Tensile Stress of PC Strand

Unit: N/mm²

Load Condition	Allowable Value	Result
During Prestressing	Under 1440	1250.0
Immediately after Prestressing	Under 1295	1149.1
Under Effective Prestressing Force	Under 1110	1026.0

Source: JICA Study Team

Design for Combined Flexural Stress (Checking for Volume of Tension Re-bar)

The result of design for combined flexural stress is shown in Table 4.6.91. The checking case is indicated for the “Under Dead Load” and “Under Live Load” cases.

Table 4.6.91 Result of Design for Combined Flexural Stress

Unit: N/mm²

Load Condition	Checking Position	Allowable Value	Result	
Under Dead Load	Top of Deck Slab	<=10.0		-0.32
	Bottom of Deck Slab			-0.21
	Top of Crossbeam	0.0 ~ 11.0		1.92
	Bottom of Crossbeam			4.57
Under Live Load	Top of Deck Slab	<=10.0	Max	0.55
			Min	-0.89
	Bottom of Deck Slab		Max	0.36
			Min	-0.60
	Top of Crossbeam	0.0 ~ 11.0	Max	1.90
			Min	1.94
	Bottom of Crossbeam		Max	2.29
			Min	6.10

Source: JICA Study Team

Checking for Bending Moment to Failure (Under Ultimate Load)

The result of the checking for the bending moment to failure is shown in Table 4.6.92. The critical case, "case 2", is shown in Table 4.6.53.

Table 4.6.92 Result of Checking for Bending Moment to Failure

a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
2383.7	1325.8	1.80

Source: JICA Study Team

b) Design for Shear Force

The design for shear force under design load involves "Mean Shear Stress of Concrete (Volume of Diagonal Tension Reinforcement Bar)" and "Diagonal Tensile Stress of Concrete". The checking for the "Under Ultimate Load" case involves "Compressive Strength to Failure" and "Members against Diagonal Tensile Strength to Failure".

Design for Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

The result of the design for the mean shear stress is shown in Table 4.6.93.

Table 4.6.93 Result of Design for Mean Shear Stress of Concrete

Unit: N/mm²

Allowable Value	Result
τ_m<=0.45	0.65

Source: JICA Study Team

In Table 4.6.93, the calculated value is over the allowable value of the mean shear stress. Hence, the required volume of diagonal tension reinforcement bar is calculated as given in Table 4.6.94.

Table 4.6.94 Result of Volume of Diagonal Tension Re-bar

Unit: cm^2

Minimum Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
6.00	9.66

Source: JICA Study Team

From Table 4.6.94, the required volume of diagonal tension reinforcement bar is placed in the crossbeam.

Design for Diagonal Tensile Stress of Concrete

The result of the design for diagonal tensile stress of concrete is shown in Table 4.6.95.

Table 4.6.95 Result of Design for Diagonal Tensile Stress of Concrete

Unit: N/mm^2

Load Condition	Allowable Value	Result	
Under Dead Load	σI=>-0.80	Max	-0.014
		Min	-0.014
Under Live Load	σI=>-1.70	Max	-0.18
		Min	-0.18

Source: JICA Study Team

Checking for Compressive Strength to Failure (Under Ultimate Load)

The result of the checking for compressive strength to failure is shown in Table 4.6.96.

Table 4.6.96 Result of Checking for Compressive Strength to Failure

a) Compressive Strength to Failure (kN)	b) Acting Shear Force (kN)	Safety Degree a) / b)
1494.6	529.3	2.82

Source: JICA Study Team

Checking for Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

The acting shear force is shown in Table 4.6.97. If D13 is used as the diagonal tension re-bar, the interval of re-bar should be less than 262 mm. If D16 is used, the interval of re-bar should be less than 400 mm. After all, re-bar of D13 and 250 mm interval are applied.

Table 4.6.97 Result of Acting Shear Force

Acting Shear Force (kN)
529.3

Source: JICA Study Team

3) Crossbeam at Connection Part

The checking items are "Tensile Stress of PC Strand" and "Mean Compressive Strength of Concrete".

Design for Tensile Stress of PC Strand

The result of design for tensile stress for PC strands is shown in Table 4.6.98. The checking cases include "During Prestressing", "Immediately after Prestressing", and "Under Effective Prestressing Force".

Table 4.6.98 Result of Design for Tensile Stress of PC Strand and Mean Compressive Stress of Concrete

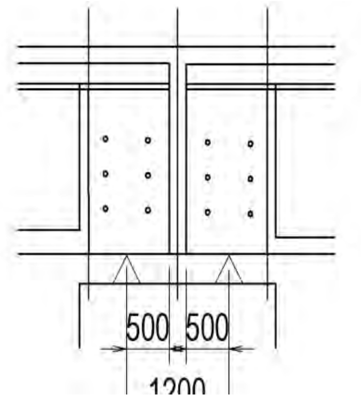
Unit: N/mm^2

Load Condition	Tensile Stress of PC Strand		Mean Compressive Stress of Concrete	
	Allowable Value	Result	Allowable Value	Result
During Prestressing	Under 1440	1250.0	more than 1.50	1.61
Immediately after Prestressing	Under 1295	1151.2		
Under Effective Prestressing Force	Under 1110	1051.5		

Source: JICA Study Team

(7) Design of Coupling Concrete (Connecting Part)

For the design of coupling concrete, the checking position for the bending moment is shown in Figure 4.6.35.



Source: JICA Study Team

Figure 4.6.35 Checking Position of Coupling Concrete

The design for the bending moment under design load involves “Compressive Stress of Concrete” and “Tensile Stress of Re-bar”. The checking for bending moment under ultimate load involves “Bending Moment to Failure”.

<u>Design for Compressive Stress of Concrete and Tensile Stress of Re-bar</u>

The result of the design for compressive stress of concrete is shown in Table 4.6.99. The checking cases include “Under Dead Load”, “Under Live Load”, and “Under Temperature Load”.

Table 4.6.99 Result of Design for Compressive Stress of Concrete and Tensile Stress of Re-bar

Unit: N/mm²

Section	Load Condition	Type of Force	Compressive Strength of Concrete		Tensile Stress of Re-bar	
			Allowable Value	Result	Allowable Value	Result
1	Under Dead Load	Negative Moment	-	-	100	0.00
2		Positive Moment	-	-		27.25
		Negative Moment	-	-		0.00
3		Negative Moment	-	-		0.00
1	Under Live Load	Negative Moment	10.0	3.53	160.0	95.18
2		Positive Moment		0.74		43.54
		Negative Moment		3.53		95.18
3		Negative Moment		3.53		95.18
1	Under Temperature Load	Negative Moment	11.5	3.65	184.0	98.28
2		Positive Moment		1.62		95.51
		Negative Moment		3.53		95.18
3		Negative Moment		3.65		98.02

Source: JICA Study Team

<u>Checking for Bending Moment to Failure (Under Ultimate Load)</u>

The result of checking for bending moment to failure is shown in Table 4.6.100. The critical case, "Case 1 and 2", is shown in Table 4.6.54.

Table 4.6.100 Result of Checking for Bending Moment to Failure

Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-5969.78	-5031.28	1.19
2	Positive Moment	4561.52	847.72	5.38
	Negative Moment	-5969.78	-4783.31	1.25
3	Negative Moment		-5022.72	1.20

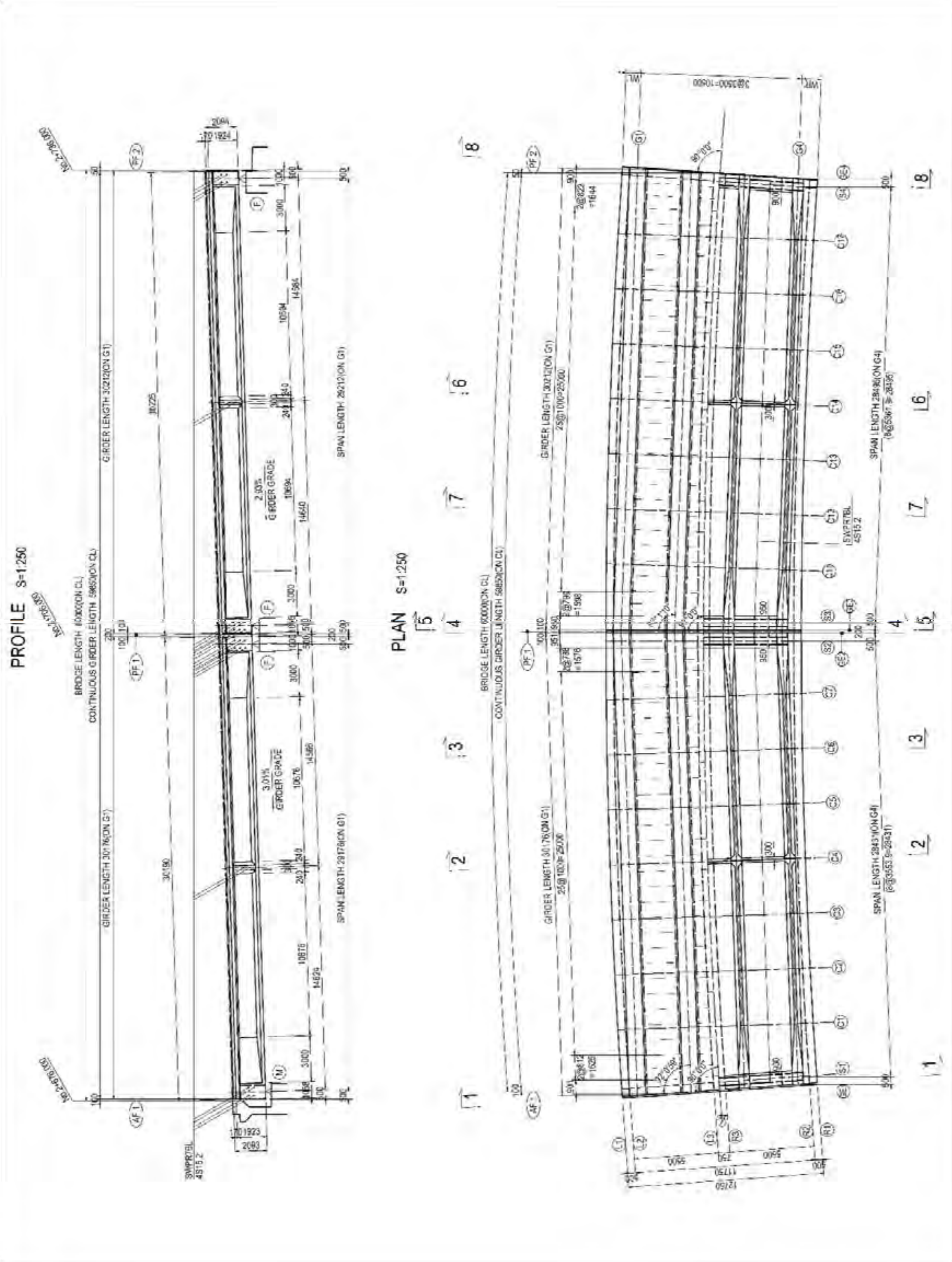
Source: JICA Study Team

(8) Result of Design for Other Sections

Since the above result was for the representative section of PF14-PF15, the following figures show the major profile, cross section, and calculation results of the other sections.

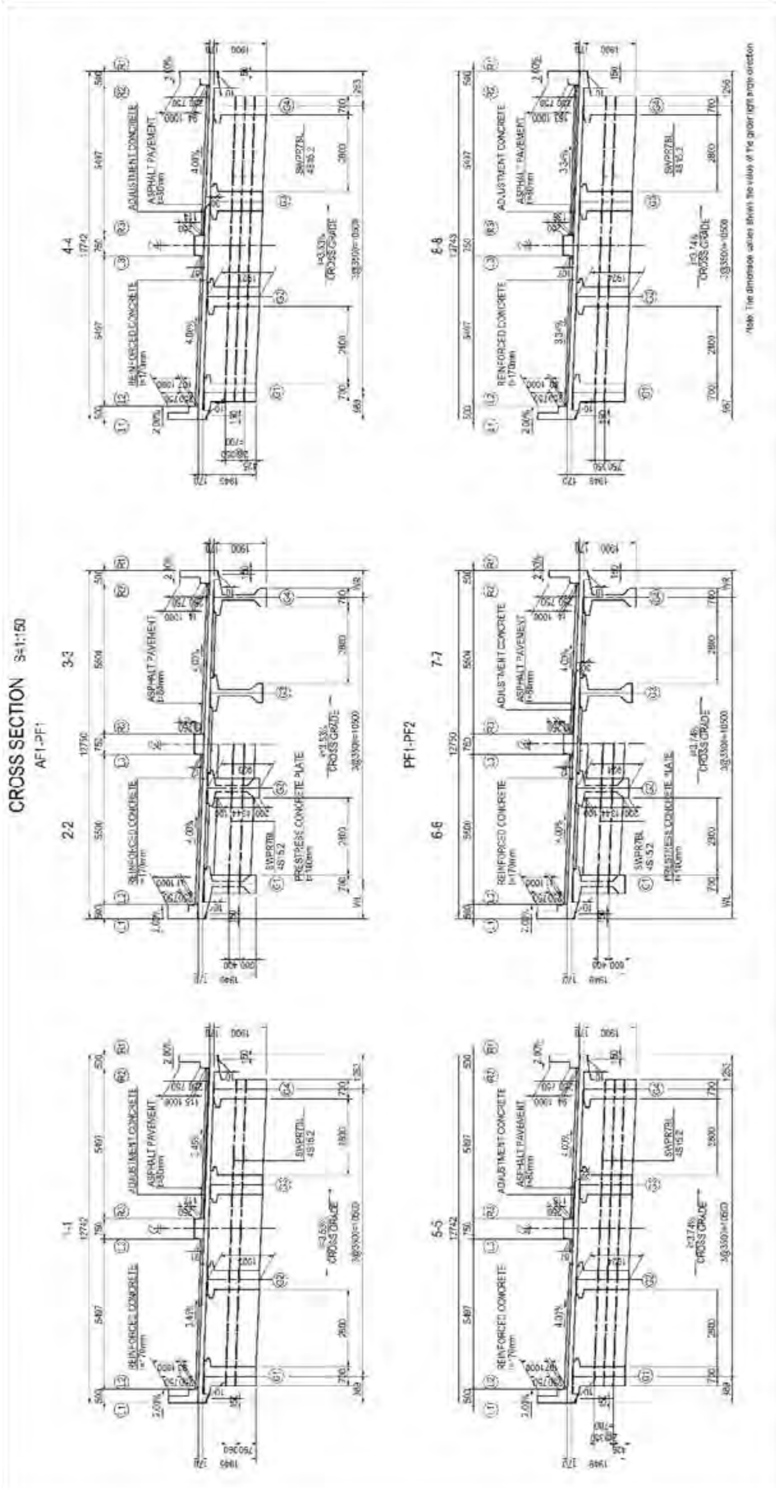
1) AF1-PF2

Figure 4.6.36 and Figure 4.6.37 show the profile, plan, and cross section of AF1-PF2.



Source: JICA Study Team

Figure 4.6.36 Profile and Plan (AF1-PF2)



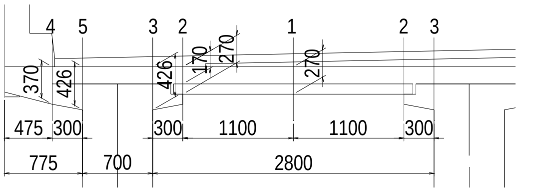
Source: JICA Study Team

Figure 4.6.37 Cross Section (AF1-PF2)

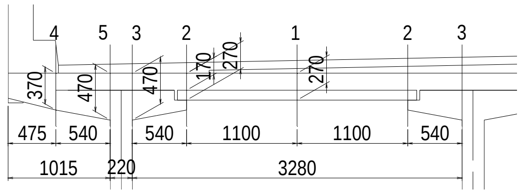
a) AF1-PF1

The following tables show the calculation results for AF1-PF1.

Table 4.6.101 Calculation Results for the Deck (AF1-PF1)



Deck at End of Girder



Deck at Center Span

Result of Design of PC Plate (Unit: N/mm2)

Deck at End of Girder

Load Condition	Checking Position	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
During Prestressing	-	-	-	1305	1225
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131
	Bottom of PC Plate		8.34		
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.58	1020	921
	Bottom of PC Plate		3.69		

Deck at Center Span

Load Condition	Checking Position	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
During Prestressing	-	-	-	1305	1225
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131
	Bottom of PC Plate		8.34		
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.58	1020	921
	Bottom of PC Plate		3.69		

Result of Design of Deck (Unit: N/mm2)

Deck between Main Girders

Deck at End of Girder (Tranverse Direction)

Section	Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result
1	Under Live Load	Top of RC Deck	11.0	3.48
		Bottom of RC Deck		-0.80
		Top pf PC Plate	0.0 ~15.0	8.35
		Bottom of PC Plate		1.36

Section	Load Condition	Type of Force	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
2	Under Live Load	Positive Moment	10.0	3.84	140.0	59.7
		Negative Moment		3.74		78.4
3		Negative Moment		2.56		73.6

Deck at Center Span (Tranverse Direction)

Section	Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result
1	Under Live Load	Top of RC Deck	11.0	4.15
		Bottom of RC Deck		-0.95
		Top pf PC Plate	0.0~15.0	8.19
		Bottom of PC Plate		0.65

Section	Load Condition	Type of Force	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
2	Under Live Load	Positive Moment	10.0	6.60	140.0	102.5
		Negative Moment		3.23		67.5
3		Negative Moment		2.52		77.6

Direction	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
Deck at End of Girder (Longitudinal Direction)	Under Live Load	10.0	8.66	140.0	100.2
Deck at Center Span (Longitudinal Direction)	Under Live Load	10.0	9.96	140.0	115.3

Deck outside of Main Girder

Deck at End of Girder (Tranverse Direction)

Section	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
4	Under Dead Load	-	-	100.0	8.2
	Under Live Load	10.0	0.31	140.0	8.2
	Under Collison Load	15.0	2.15	300.0	56.4
	Under Wind Load (with Live Load)	12.5	0.46	175.0	12.0
	Under Wind Load (w/o Live Load)	12.5	0.60	175.0	15.8
5	Under Dead Load	-	-	100.0	13.2
	Under Live Load	10.0	3.53	140.0	101.5
	Under Collison Load	15.0	4.98	300.0	143.4
	Under Wind Load (with Live Load)	12.5	3.64	175.0	104.9
	Under Wind Load (w/o Live Load)	12.5	0.69	175.0	19.9

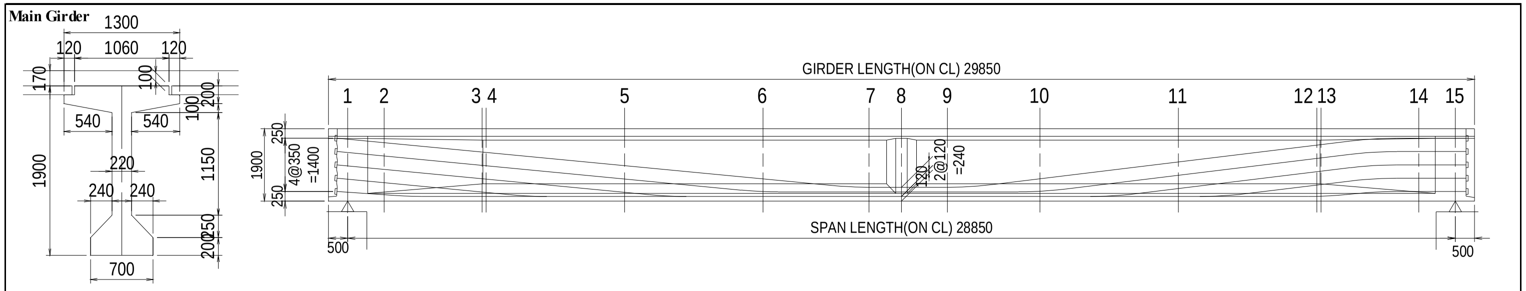
Deck at Center Span (Tranverse Direction)

Section	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
4	Under Dead Load	-	-	100.0	8.2
	Under Live Load	10.0	0.31	140.0	8.2
	Under Collison Load	15.0	2.15	300.0	56.4
	Under Wind Load (with Live Load)	12.5	0.46	175.0	12.0
	Under Wind Load (w/o Live Load)	12.5	0.60	175.0	15.8
5	Under Dead Load	-	-	100.0	16.7
	Under Live Load	10.0	2.46	140.0	75.7
	Under Collison Load	15.0	3.70	300.0	113.7
	Under Wind Load (with Live Load)	12.5	2.56	175.0	78.8
	Under Wind Load (w/o Live Load)	12.5	0.74	175.0	22.8

Direction	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
Deck at End of Girder (Longitudinal Direction)	Under Live Load	10.0	1.86	140.0	67.2
Deck at Center Span (Longitudinal Direction)	Under Live Load	10.0	2.29	140.0	82.9

Source: JICA Study Team

Table 4.6.102 Calculation Results for the Main Girder (AF1-PF1)



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2)	
		Allowable Value	Result
During Prestressing	-	1440	1320
Immediately after Prestressing	Cross Section "8"	1295	1172
During Deck Construction	Cross Section "8"	1100	1068
Under Live Load	Cross Section "8"	1100	985

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2)		
			Allowable Value	Result	
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.80
		Bottom of Girder			14.99
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		6.36
		Bottom of Girder			6.96
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		3.04
		Top of Girder	0.0<σc<14.0		5.51
		Bottom of Girder			2.45
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	4.54
				Min	2.65
		Top of Girder	-1.5<σc<14.0	Max	6.75
				Min	5.19
		Bottom of Girder		Max	-1.05
				Min	3.35
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	5.27
				Min	3.38
		Top of Girder	-2.0<σc<16.10	Max	6.05
				Min	4.49
		Bottom of Girder		Max	-1.36
				Min	3.04
		Tension Re-bar Volume (cm2)			
		Minimum 12.23	Required 8.05		

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	15295 kNm	13176 kNm	1.16

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2)		Volume of Diagonal Tension Re-bar (cm2)	
	Allowable Value	Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
Cross Section "3"	τm<=0.55	1.29	4.40	10.54

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2)		
			Allowable Value	Result	
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-1.0		-0.11
		Neutral Axis before Composition			-0.10
		Neutral Axis after Composition			-0.12
		Base of Lower Flange			-0.06
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-2.0	Max	-0.46
				Min	-0.09
		Neutral Axis before Composition		Max	-0.47
				Min	-0.09
		Neutral Axis after Composition		Max	-0.49
				Min	-0.10
		Base of Lower Flange		Max	-0.33
				Min	-0.05

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section "3"	2715 kN	1483 kN	1.83

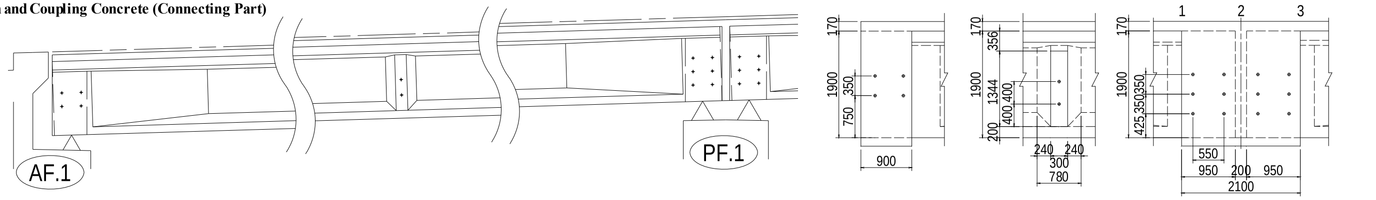
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section "3"	1483 kN	D13@125mm

Source: JICA Study Team

Table 4.6.103 Calculation Results for the Crossbeam and Coupling Concrete (AF1-PF1)

Cross Beam and Coupling Concrete (Connecting Part)



Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1151
During Deck Construction	-	-	-	1110	1056
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0	1.14	-	-
	Bottom of Cross Beam		1.60	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max 1.58	-	-
			Min 1.13	-	-
	Bottom of Cross Beam		Max 0.92	-	-
			Min 1.83	-	-

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Diagonal Tensile Stress of Concrete: Allowable Value	Diagonal Tensile Stress of Concrete: Result
Under Dead Load	τ_m<=0.45	0.08	σI=>-0.80	0.00
Under Live Load			σI=>-1.70	-0.01

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
5682 kN	260 kN	21.85

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
2131 kN	260 kN	8.20

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2): Allowable Value	Result
During Prestressing	1440	1250
Immediately after Prestressing	1295	1151
Under Effective Prestressing Force	1110	1051

Load Condition	Mean Compressive Stress of Concrete (N/mm2): Allowable Value	Result
Under Effective Prestressing Force	1.50	1.61

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1149
During Deck Construction	-	-	-	1110	1024
Under Dead Load	Top of Deck	<=10.0	-0.36	-	-
	Bottom of Deck		-0.24	-	-
	Top of Cross Beam	0.0 ~ 11.0	2.28	-	-
	Bottom of Cross Beam		4.55	-	-
Under Live Load	Top of Deck	<=10.0	Max 0.52	-	-
			Min -0.91	-	-
	Bottom of Deck		Max 0.35	-	-
			Min -0.61	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max 2.18	-	-
			Min 2.34	-	-
	Bottom of Cross Beam		Max 2.21	-	-
			Min 6.01	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2384 kNm	1329 kNm	1.79

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.69	6.00 cm2	10.28 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result
Under Dead Load	σI=>-0.80	Max -0.02
		Min -0.02
Under Live Load	σI=>-1.70	Max -0.17
		Min -0.18

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1495 kN	552 kN	2.71

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
552 kN	D13@250mm

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete: Allowable Value	Compressive Strength of Concrete: Result	Tensile Stress of Re-bar: Allowable Value	Tensile Stress of Re-bar: Result
1	Under Dead Load	Negative Moment	-	-	100.0	0.0
2		Positive Moment		-		95.8
		Negative Moment		-		0.0
3		Negative Moment		-		0.0
1	Under Live Load	Negative Moment	10.00	1.53	160.0	41.4
2		Positive Moment		2.18		109.0
		Negative Moment		1.53		41.4
3		Negative Moment		1.53		41.4
1	Under Temperature Load	Negative Moment	11.50	1.90	184.0	51.2
2		Positive Moment		2.99		149.8
		Negative Moment		1.53		41.4
3		Negative Moment		1.90		51.2

Bending Moment to Failure (Under Ultimate Load)

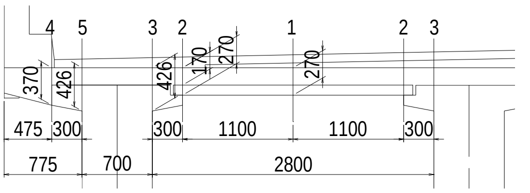
Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-5970	-4343	1.37
2	Positive Moment	6058	2123	2.85
	Negative Moment	-5970	-3844	1.55
3	Negative Moment		-4346	1.37

Source: JICA Study Team

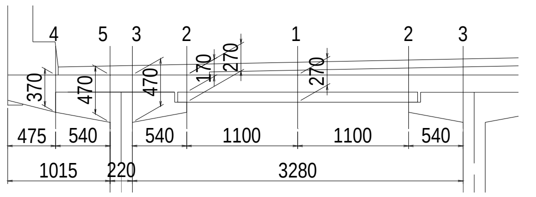
b) PF1-PF2

The following tables show the calculation results for PF1-PF2.

Table 4.6.104 Calculation Results for the Deck (PF1-PF2)



Deck at End of Girder



Deck at Center Span

Result of Design of PC Plate (Unit: N/mm2)

Deck at End of Girder

Load Condition	Checking Position	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
During Prestressing	-	-	-	1305	1225
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131
	Bottom of PC Plate		8.34		
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.58	1020	922
	Bottom of PC Plate		3.69		

Deck at Center Span

Load Condition	Checking Position	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
During Prestressing	-	-	-	1305	1225
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131
	Bottom of PC Plate		8.34		
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.59	1020	922
	Bottom of PC Plate		3.69		

Result of Design of Deck (Unit: N/mm2)

Deck between Main Girders

Deck at End of Girder (Tranverse Direction)

Section	Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result
1	Under Live Load	Top of RC Deck	11.0	3.50
		Bottom of RC Deck		-0.81
		Top pf PC Plate	0.0 ~15.0	8.35
		Bottom of PC Plate		1.33

Section	Load Condition	Type of Force	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
2	Under Live Load	Positive Moment	10.0	3.87	140.0	60.1
		Negative Moment		3.76		78.8
3		Negative Moment		2.57		74.0

Deck at Center Span (Tranverse Direction)

Section	Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result
1	Under Live Load	Top of RC Deck	11.0	4.19
		Bottom of RC Deck		-0.96
		Top pf PC Plate	0.0~15.0	8.18
		Bottom of PC Plate		0.62

Section	Load Condition	Type of Force	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
2	Under Live Load	Positive Moment	10.0	6.65	140.0	103.3
		Negative Moment		3.25		67.9
3		Negative Moment		2.54		78.1

Deck at End of Girder (Longitudinal Direction) / Deck at Center Span (Longitudinal Direction)

Deck	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
Deck at End of Girder (Longitudinal Direction)	Under Live Load	10.0	8.66	140.0	100.2
Deck at Center Span (Longitudinal Direction)	Under Live Load	10.0	9.96	140.0	115.3

Deck outside of Main Girder

Deck at End of Girder (Tranverse Direction)

Section	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
4	Under Dead Load	-	-	100.0	8.2
	Under Live Load	10.0	0.31	140.0	8.2
	Under Collison Load	15.0	2.15	300.0	56.3
	Under Wind Load (with Live Load)	12.5	0.46	175.0	12.0
	Under Wind Load (w/o Live Load)	12.5	0.60	175.0	15.8
5	Under Dead Load	-	-	100.0	13.1
	Under Live Load	10.0	3.52	140.0	101.4
	Under Collison Load	15.0	4.97	300.0	143.3
	Under Wind Load (with Live Load)	12.5	3.64	175.0	104.8
	Under Wind Load (w/o Live Load)	12.5	0.69	175.0	19.8

Deck at Center Span (Tranverse Direction)

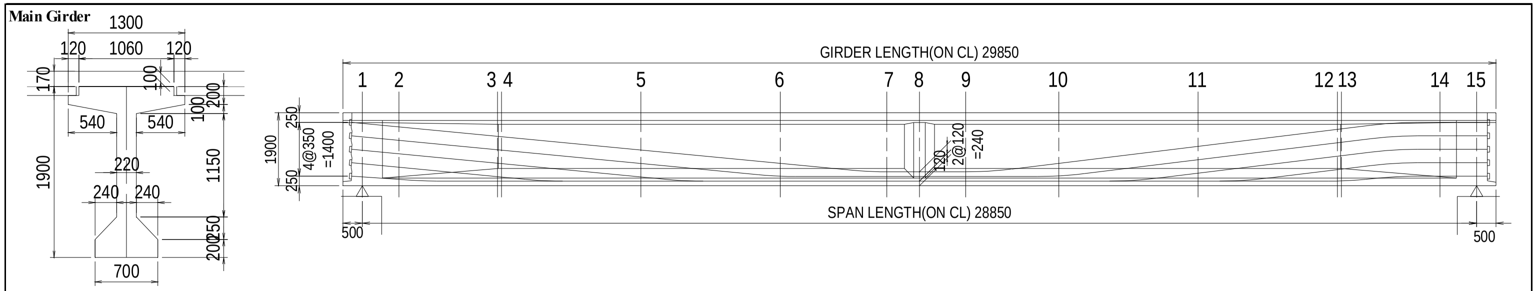
Section	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
4	Under Dead Load	-	-	100.0	8.2
	Under Live Load	10.0	0.31	140.0	8.2
	Under Collison Load	15.0	2.15	300.0	56.3
	Under Wind Load (with Live Load)	12.5	0.46	175.0	12.0
	Under Wind Load (w/o Live Load)	12.5	0.60	175.0	15.8
5	Under Dead Load	-	-	100.0	16.6
	Under Live Load	10.0	2.46	140.0	75.6
	Under Collison Load	15.0	3.69	300.0	113.6
	Under Wind Load (with Live Load)	12.5	2.56	175.0	78.7
	Under Wind Load (w/o Live Load)	12.5	0.74	175.0	22.7

Deck at End of Girder (Longitudinal Direction) / Deck at Center Span (Longitudinal Direction)

Deck	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
Deck at End of Girder (Longitudinal Direction)	Under Live Load	10.0	1.86	140.0	67.2
Deck at Center Span (Longitudinal Direction)	Under Live Load	10.0	2.29	140.0	82.9

Source: JICA Study Team

Table 4.6.105 Calculation Results for the Main Girder (PF1-PF2)



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2)	
		Allowable Value	Result
During Prestressing	-	1440	1320
Immediately after Prestressing	Cross Section "8"	1295	1172
During Deck Construction	Cross Section "8"	1100	1068
Under Live Load	Cross Section "8"	1100	986

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2)		
			Allowable Value	Result	
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.82
		Bottom of Girder			14.97
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		6.39
		Bottom of Girder			6.92
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		3.08
		Top of Girder	0.0<σc<14.0		5.55
		Bottom of Girder			2.36
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	4.58
				Min	2.69
		Top of Girder	-1.5<σc<14.0	Max	6.79
				Min	5.23
		Bottom of Girder		Max	-1.14
				Min	3.26
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	5.31
				Min	3.42
		Top of Girder	-2.0<σc<16.10	Max	6.09
				Min	4.53
		Bottom of Girder		Max	-1.46
				Min	2.94
		Tension Re-bar Volume (cm2)			
		Minimum 12.83	Required 9.02		

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	15295 kNm	13264 kNm	1.15

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2)		Volume of Diagonal Tension Re-bar (cm2)	
	Allowable Value	Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
Cross Section "13"	τm<=0.55	1.28	4.40	10.31

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2)		
			Allowable Value	Result	
Under Dead Load	Cross Section "13"	Base of Upper Flange	σI=>-1.0		-0.12
		Neutral Axis before Composition			-0.11
		Neutral Axis after Composition			-0.12
		Base of Lower Flange			-0.06
Under Dead Load	Cross Section "13"	Base of Upper Flange	σI=>-2.0	Max	-0.10
				Min	-0.47
		Neutral Axis before Composition		Max	-0.09
				Min	-0.46
		Neutral Axis after Composition		Max	-0.10
				Min	-0.49
		Base of Lower Flange		Max	-0.05
				Min	-0.32

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section "13"	2715 kN	1476 kN	1.84

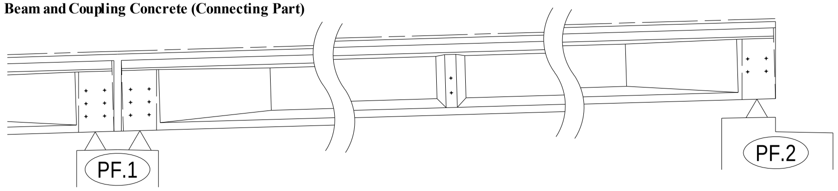
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

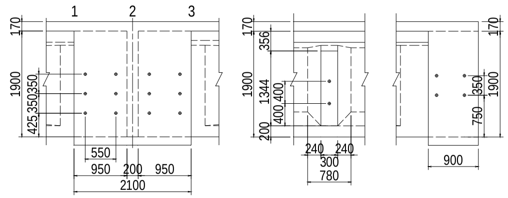
Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section "13"	1476 kN	D13@125mm

Source: JICA Study Team

Table 4.6.106 Calculation Results for the Crossbeam and Coupling Concrete (PF1-PF2)

Cross Beam and Coupling Concrete (Connecting Part)





Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1151
During Deck Construction	-	-	-	1110	1056
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0	1.14	-	-
	Bottom of Cross Beam		1.60	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max 1.58	-	-
			Min 1.13	-	-
	Bottom of Cross Beam		Max 0.92	-	-
			Min 1.83	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2384 kNm	1334 kNm	1.79

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Diagonal Tensile Stress of Concrete: Allowable Value	Diagonal Tensile Stress of Concrete: Result
Under Dead Load	τ_m<=0.45	0.08	σI=>-0.80	0.00
Under Live Load			σI=>-1.70	-0.01

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
5682 kN	260 kN	21.85

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
2131 kN	260 kN	8.20

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1149
During Deck Construction	-	-	-	1110	1024
Under Dead Load	Top of Deck	<=10.0	-0.36	-	-
	Bottom of Deck		-0.24	-	-
	Top of Cross Beam	0.0 ~ 11.0	2.28	-	-
	Bottom of Cross Beam		4.53	-	-
Under Live Load	Top of Deck	<=10.0	Max 0.53	-	-
			Min -0.92	-	-
	Bottom of Deck		Max 0.35	-	-
			Min -0.61	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max 2.18	-	-
			Min 2.35	-	-
	Bottom of Cross Beam		Max 2.19	-	-
			Min 6.01	-	-

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.68	6.00 cm2	10.23 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result
Under Dead Load	σI=>-0.80	Max -0.02
		Min -0.02
Under Live Load	σI=>-1.70	Max -0.17
		Min -0.18

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1495 kN	550 kN	2.72

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
550 kN	D13@250mm

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2): Allowable Value	Result
During Prestressing	1440	-
Immediately after Prestressing	1295	-
Under Effective Prestressing Force	1110	-

Load Condition	Mean Compressive Stress of Concrete (N/mm2): Allowable Value	Result
Under Effective Prestressing Force	1.50	-

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete: Allowable Value	Compressive Strength of Concrete: Result	Tensile Stress of Re-bar: Allowable Value	Tensile Stress of Re-bar: Result
1	Under Dead Load	Negative Moment	-	-	100.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-
1	Under Live Load	Negative Moment	10.00	-	160.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-
1	Under Temperature Load	Negative Moment	11.50	-	184.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-

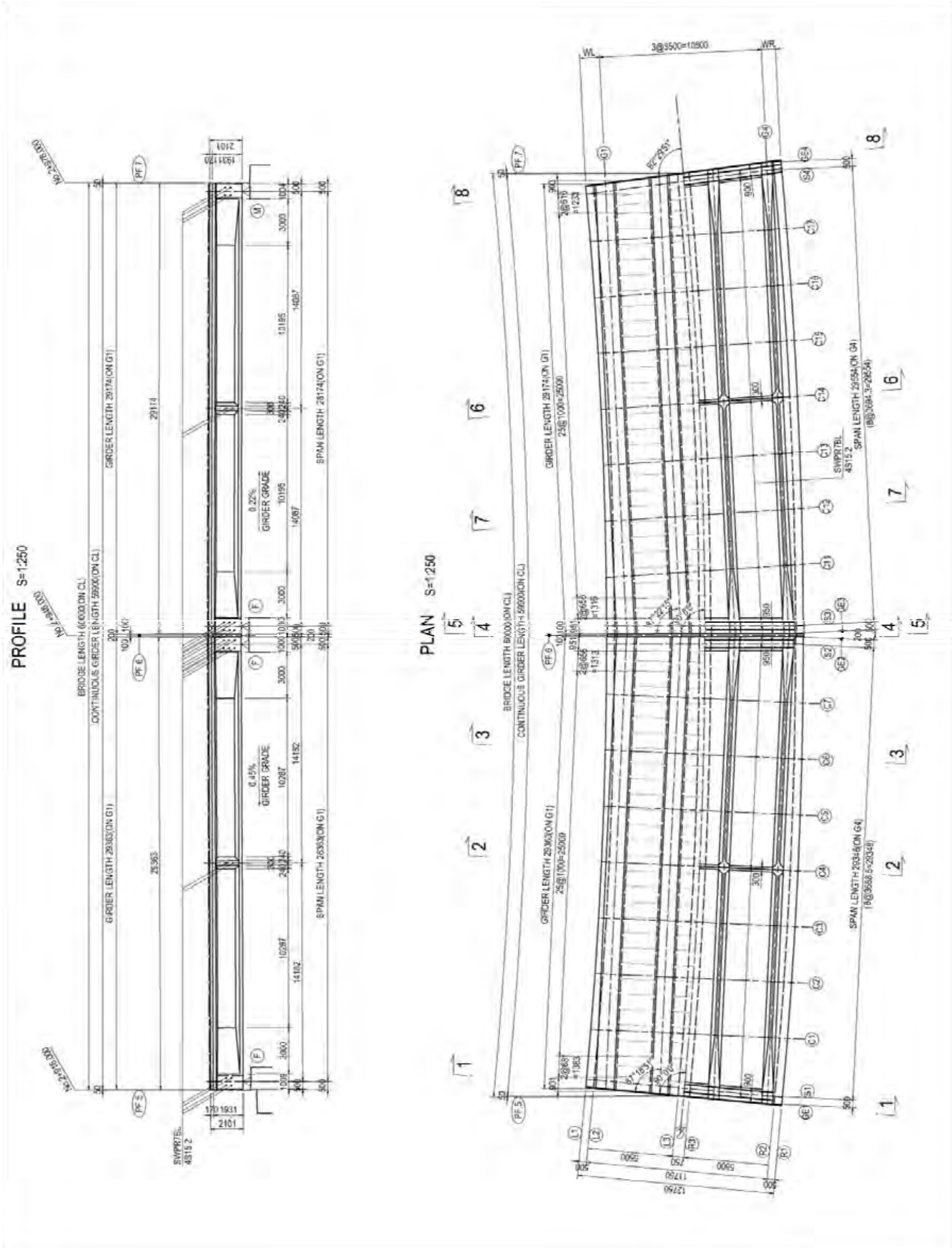
Bending Moment to Failure (Under Ultimate Load)

Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-	-	-
2	Positive Moment	-	-	-
	Negative Moment	-	-	-
3	Negative Moment		-	-

Source: JICA Study Team

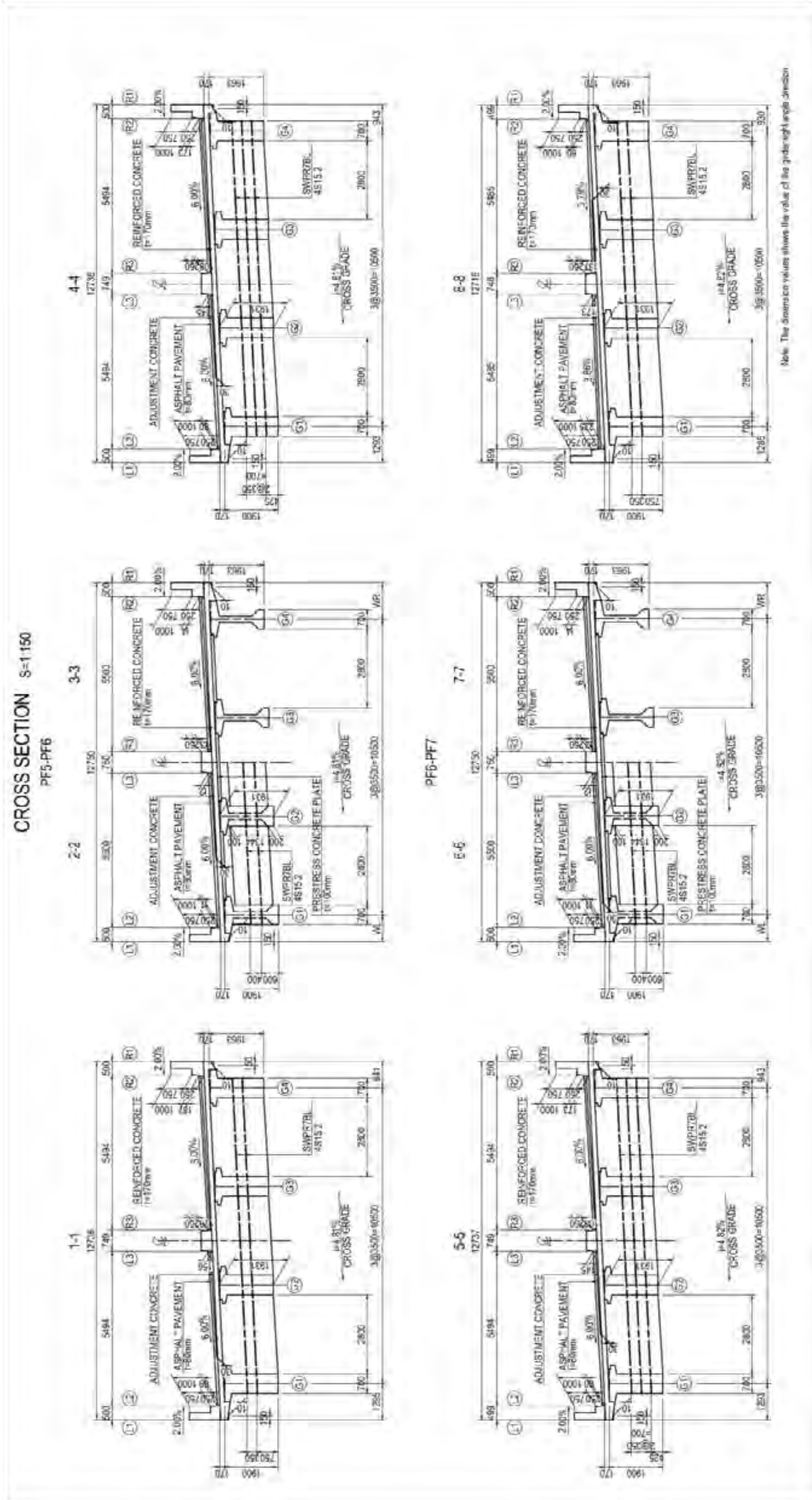
2) PF5-PF7

Figure 4.6.38 and Figure 4.6.39 show the profile, plan, and cross section of PF5-PF7.



Source: JICA Study Team

Figure 4.6.38 Profile and Plan (PF5-PF7)



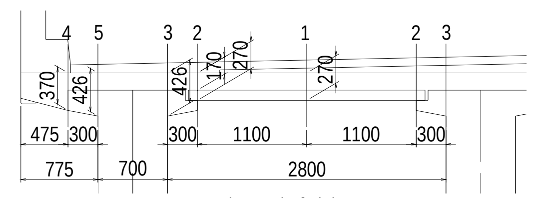
Source: JICA Study Team

Figure 4.6.39 Cross Section (PF5-PF7)

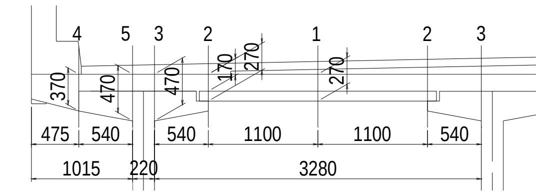
a) PF5-PF6

The following tables show the calculation results for PF5-PF6.

Table 4.6.107 Calculation Results for the Deck (PF5-PF6)



Deck at End of Girder



Deck at Center Span

Result of Design of PC Plate (Unit: N/mm2)

Deck at End of Girder						Deck at Center Span					
Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar		Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result			Allowable Value	Result	Allowable Value	Result
During Prestressing	-	-	-	1305	1225	During Prestressing	-	-	-	1305	1225
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131	Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131
	Bottom of PC Plate		8.34				Bottom of PC Plate		8.34		
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.59	1020	922	During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.59	1020	923
	Bottom of PC Plate		3.70				Bottom of PC Plate		3.70		

Result of Design of Deck (Unit: N/mm2)

Deck between Main Girders

Deck at End of Girder (Tranverse Direction)					Deck at Center Span (Tranverse Direction)				
Section	Load Condition	Checking Position	Combined Flexural Stress		Section	Load Condition	Checking Position	Combined Flexural Stress	
			Allowable Value	Result				Allowable Value	Result
1	Under Live Load	Top of RC Deck	11.0	3.55	1	Under Live Load	Top of RC Deck	11.0	4.25
		Bottom of RC Deck		-0.81			Bottom of RC Deck		-0.98
		Top pf PC Plate	0.0~15.0	8.34			Top pf PC Plate	0.0~15.0	8.17
		Bottom of PC Plate		1.29			Bottom of PC Plate		0.56

Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar	
			Allowable Value	Result	Allowable Value	Result				Allowable Value	Result	Allowable Value	Result
2	Under Live Load	Positive Moment	10.0	3.92	140.0	60.9	2	Under Live Load	Positive Moment	10.0	6.74	140.0	104.8
		Negative Moment		3.80		79.5			Negative Moment		3.28		68.7
3		Negative Moment		2.59		74.7	3		Negative Moment		2.57		78.9

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	8.66	140.0	100.2	Under Live Load	10.0	9.96	140.0	115.3

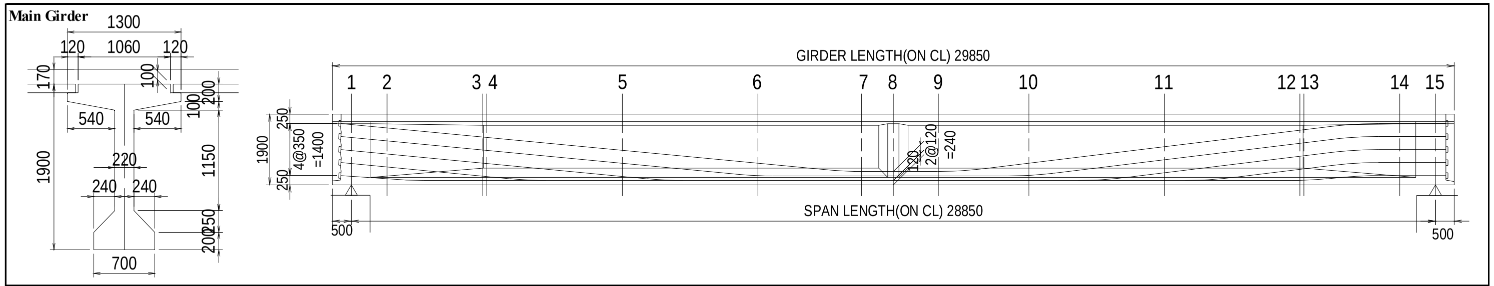
Deck outside of Main Girder

Deck at End of Girder (Tranverse Direction)						Deck at Center Span (Tranverse Direction)					
Section	Load Condition	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Strength of Concrete		Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result			Allowable Value	Result	Allowable Value	Result
4	Under Dead Load	-	-	100.0	8.9	4	Under Dead Load	-	-	100.0	8.9
	Under Live Load	10.0	0.34	140.0	8.9		Under Live Load	10.0	0.34	140.0	8.9
	Under Collison Load	15.0	2.18	300.0	57.0		Under Collison Load	15.0	2.18	300.0	57.0
	Under Wind Load (with Live Load)	12.5	0.49	175.0	12.7		Under Wind Load (with Live Load)	12.5	0.48	175.0	12.7
	Under Wind Load (w/o Live Load)	12.5	0.63	175.0	16.5		Under Wind Load (w/o Live Load)	12.5	0.63	175.0	16.5
5	Under Dead Load	-	-	100.0	13.9	5	Under Dead Load	-	-	100.0	17.2
	Under Live Load	10.0	3.94	140.0	113.6		Under Live Load	10.0	2.54	140.0	78.1
	Under Collison Load	15.0	5.40	300.0	155.4		Under Collison Load	15.0	3.78	300.0	116.1
	Under Wind Load (with Live Load)	12.5	4.06	175.0	116.9		Under Wind Load (with Live Load)	12.5	2.64	175.0	81.2
	Under Wind Load (w/o Live Load)	12.5	0.71	175.0	20.5		Under Wind Load (w/o Live Load)	12.5	0.76	175.0	23.4

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	1.93	140.0	69.9	Under Live Load	10.0	2.37	140.0	85.6

Source: JICA Study Team

Table 4.6.108 Calculation Results for the Main Girder (PF5-PF6)



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2)	
		Allowable Value	Result
During Prestressing	-	1440	1320
Immediately after Prestressing	Cross Section "8"	1295	1172
During Deck Construction	Cross Section "8"	1100	1068
Under Live Load	Cross Section "8"	1100	988

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2)		
			Allowable Value	Result	
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.83
		Bottom of Girder			14.95
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		6.42
		Bottom of Girder			6.88
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		2.86
		Top of Girder	0.0<σc<14.0		5.38
		Bottom of Girder			2.88
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	4.38
				Min	2.47
		Top of Girder	-1.5<σc<14.0	Max	6.63
				Min	5.06
		Bottom of Girder		Max	-0.65
				Min	3.80
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	5.11
				Min	3.20
		Top of Girder	-2.0<σc<16.10	Max	5.93
				Min	4.35
		Bottom of Girder		Max	-0.96
				Min	3.48
		Tension Re-bar Volume (cm2)			
		Minimum 9.29	Required 4.32		

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	15295 kNm	13154 kNm	1.16

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2)		Volume of Diagonal Tension Re-bar (cm2)	
	Allowable Value	Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
Cross Section “3”	τm<=0.55	1.28	4.40	10.90

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2)		
			Allowable Value	Result	
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-1.0		-0.11
		Neutral Axis before Composition			-0.10
		Neutral Axis after Composition			-0.11
		Base of Lower Flange			-0.06
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-2.0	Max	-0.46
				Min	-0.09
		Neutral Axis before Composition		Max	-0.47
				Min	-0.09
		Neutral Axis after Composition		Max	-0.49
				Min	-0.09
		Base of Lower Flange		Max	-0.33
				Min	-0.05

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section “3”	2716 kN	1498 kN	1.81

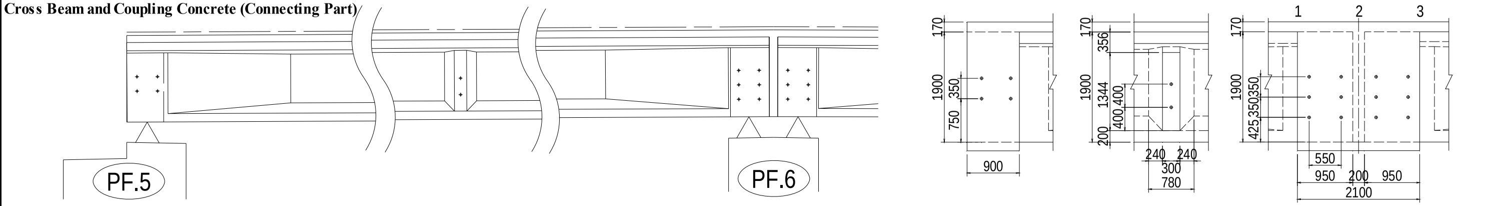
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section “3”	1498 kN	D13@125mm

Source: JICA Study Team

Table 4.6.109 Calculation Results for the Main Girder (PF5-PF6)

Cross Beam and Coupling Concrete (Connecting Part)



Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1151
During Deck Construction	-	-	-	1110	1056
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0	1.15	-	-
	Bottom of Cross Beam		1.60	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max 1.59	-	-
			Min 1.13	-	-
	Bottom of Cross Beam		Max 0.93	-	-
			Min 1.84	-	-

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Diagonal Tensile Stress of Concrete: Allowable Value	Diagonal Tensile Stress of Concrete: Result
Under Dead Load	τ_m<=0.45	0.08	σI=>-0.80	0.00
Under Live Load			σI=>-1.70	-0.01

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
5682 kN	259 kN	21.94

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
2131 kN	259 kN	8.23

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1149
During Deck Construction	-	-	-	1110	1024
Under Dead Load	Top of Deck	<=10.0	-0.38	-	-
	Bottom of Deck		-0.25	-	-
	Top of Cross Beam	0.0 ~ 11.0	2.28	-	-
	Bottom of Cross Beam		4.58	-	-
Under Live Load	Top of Deck	<=10.0	Max 0.45	-	-
			Min -0.85	-	-
	Bottom of Deck		Max 0.30	-	-
			Min -0.57	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max 2.19	-	-
			Min 2.34	-	-
	Bottom of Cross Beam		Max 2.40	-	-
			Min 5.83	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2384 kNm	1216 kNm	1.96

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.68	6.00 cm2	9.74 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result
Under Dead Load	σI=>-0.80	Max -0.02
		Min -0.02
Under Live Load	σI=>-1.70	Max -0.17
		Min -0.16

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1495 kN	532 kN	2.81

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
532 kN	D13@250mm

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2): Allowable Value	Tensile Stress of PC Strand (N/mm2): Result
During Prestressing	1440	1250
Immediately after Prestressing	1295	1151
Under Effective Prestressing Force	1110	1051

Load Condition	Mean Compressive Stress of Concrete (N/mm2): Allowable Value	Mean Compressive Stress of Concrete (N/mm2): Result
Under Effective Prestressing Force	1.50	1.61

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete: Allowable Value	Compressive Strength of Concrete: Result	Tensile Stress of Re-bar: Allowable Value	Tensile Stress of Re-bar: Result
1	Under Dead Load	Negative Moment	-	-	100.0	0.0
2		Positive Moment		-		49.4
		Negative Moment		-		5.4
3		Negative Moment		-		0.0
1	Under Live Load	Negative Moment	10.00	4.54	160.0	122.3
2		Positive Moment		1.27		63.4
		Negative Moment		4.54		122.3
3		Negative Moment		4.54		122.3
1	Under Temperature Load	Negative Moment	11.50	4.79	184.0	129.1
2		Positive Moment		2.09		104.8
		Negative Moment		4.54		122.3
3		Negative Moment		4.82		129.9

Bending Moment to Failure (Under Ultimate Load)

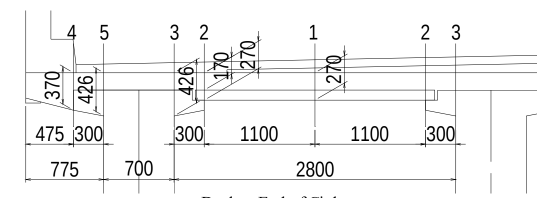
Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-5970	-5756	1.04
2	Positive Moment	6058	1391	4.36
	Negative Moment	-5970	-5312	1.12
3	Negative Moment		-5785	1.03

Source: JICA Study Team

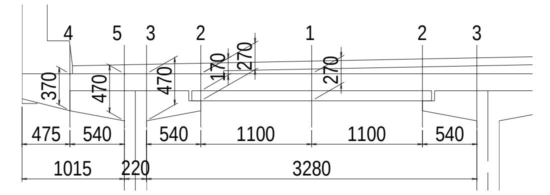
b) PF6-PF7

The following tables show the calculation results for PF6-PF7.

Table 4.6.110 Calculation Results for the Deck (PF6-PF7)



Deck at End of Girder



Deck at Center Span

Result of Design of PC Plate (Unit: N/mm2)

Deck at End of Girder						Deck at Center Span					
Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar		Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result			Allowable Value	Result	Allowable Value	Result
During Prestressing	-	-	-	1305	1225	During Prestressing	-	-	-	1305	1225
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131	Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131
	Bottom of PC Plate		8.34				Bottom of PC Plate		8.34		
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.59	1020	922	During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.59	1020	923
	Bottom of PC Plate		3.70				Bottom of PC Plate		3.70		

Result of Design of Deck (Unit: N/mm2)

Deck between Main Girders

Deck at End of Girder (Tranverse Direction)					Deck at Center Span (Tranverse Direction)				
Section	Load Condition	Checking Position	Combined Flexural Stress		Section	Load Condition	Checking Position	Combined Flexural Stress	
			Allowable Value	Result				Allowable Value	Result
1	Under Live Load	Top of RC Deck	11.0	3.55	1	Under Live Load	Top of RC Deck	11.0	4.25
		Bottom of RC Deck		-0.81			Bottom of RC Deck		-0.98
		Top pf PC Plate	0.0 ~15.0	8.34			Top pf PC Plate	0.0~15.0	8.17
		Bottom of PC Plate		1.29			Bottom of PC Plate		0.56

Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar	
			Allowable Value	Result	Allowable Value	Result				Allowable Value	Result	Allowable Value	Result
2	Under Live Load	Positive Moment	10.0	3.92	140.0	60.9	2	Under Live Load	Positive Moment	10.0	6.74	140.0	104.8
		Negative Moment		3.80		79.5			Negative Moment		3.28		68.7
3		Negative Moment		2.59		74.7	3		Negative Moment		2.57		78.9

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	8.66	140.0	100.2	Under Live Load	10.0	9.96	140.0	115.3

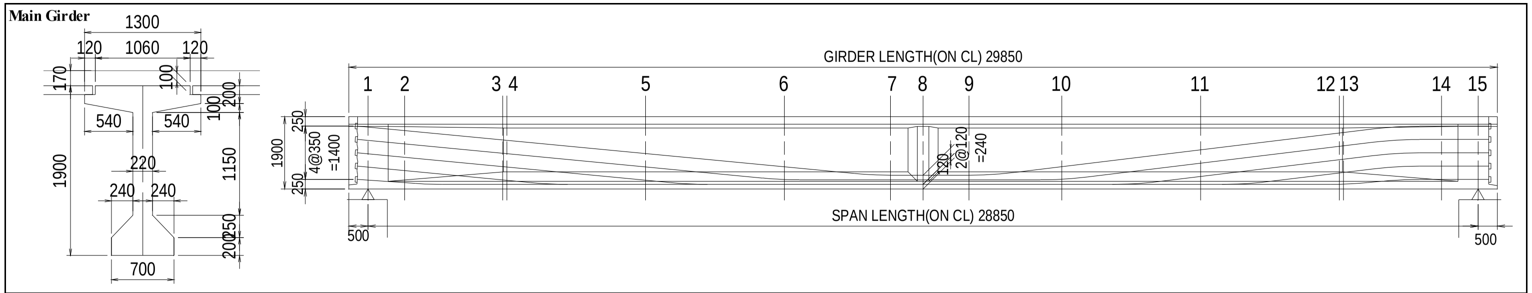
Deck outside of Main Girder

Deck at End of Girder (Tranverse Direction)						Deck at Center Span (Tranverse Direction)					
Section	Load Condition	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Strength of Concrete		Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result			Allowable Value	Result	Allowable Value	Result
4	Under Dead Load	-	-	100.0	9.3	4	Under Dead Load	-	-	100.0	9.3
	Under Live Load	10.0	0.36	140.0	9.3		Under Live Load	10.0	0.35	140.0	9.3
	Under Collison Load	15.0	2.19	300.0	57.4		Under Collison Load	15.0	2.19	300.0	57.4
	Under Wind Load (with Live Load)	12.5	0.50	175.0	13.1		Under Wind Load (with Live Load)	12.5	0.50	175.0	13.1
	Under Wind Load (w/o Live Load)	12.5	0.65	175.0	16.9		Under Wind Load (w/o Live Load)	12.5	0.64	175.0	16.9
5	Under Dead Load	-	-	100.0	14.5	5	Under Dead Load	-	-	100.0	17.9
	Under Live Load	10.0	3.96	140.0	114.2		Under Live Load	10.0	2.56	140.0	78.8
	Under Collison Load	15.0	5.42	300.0	156.0		Under Collison Load	15.0	3.80	300.0	116.8
	Under Wind Load (with Live Load)	12.5	4.08	175.0	117.5		Under Wind Load (with Live Load)	12.5	2.66	175.0	81.9
	Under Wind Load (w/o Live Load)	12.5	0.74	175.0	21.2		Under Wind Load (w/o Live Load)	12.5	0.78	175.0	24.0

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	1.93	140.0	69.9	Under Live Load	10.0	2.37	140.0	85.6

Source: JICA Study Team

Table 4.6.111 Calculation Results of the Main Girder (PF6-PF7)



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2) Allowable Value	Combined Flexural Stress (N/mm2) Result
During Prestressing	-	1440	1320
Immediately after Prestressing	Cross Section "8"	1295	1172
During Deck Construction	Cross Section "8"	1100	1068
Under Live Load	Cross Section "8"	1100	990

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2) Allowable Value		Result
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.86
		Bottom of Girder			14.91
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		6.47
		Bottom of Girder			6.82
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		2.94
		Top of Girder	0.0<σc<14.0		5.46
		Bottom of Girder			2.71
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	4.46
				Min	2.55
		Top of Girder	-1.5<σc<14.0	Max	6.71
				Min	5.14
		Bottom of Girder		Max	-0.83
				Min	3.62
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	5.19
				Min	3.28
		Top of Girder	-2.0<σc<16.10	Max	6.01
				Min	4.43
		Bottom of Girder		Max	-1.15
				Min	3.30
		Tension Re-bar Volume (cm2)			
		Minimum 10.67		Required 5.92	

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	15295 kNm	13330 kNm	1.15

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2) Allowable Value	Mean Shear Stress of Concrete (N/mm2) Result	Volume of Diagonal Tension Re-bar (cm2) Min. Diagonal Tension Re-bar Volume	Volume of Diagonal Tension Re-bar (cm2) Required Diagonal Tension Re-bar Volume
Cross Section "13"	τm<=0.55	1.30	4.40	11.00

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2) Allowable Value		Result
Under Dead Load	Cross Section "13"	Base of Upper Flange	σI=>-1.0		-0.12
		Neutral Axis before Composition			-0.11
		Neutral Axis after Composition			-0.12
		Base of Lower Flange			-0.07
Under Dead Load	Cross Section "13"	Base of Upper Flange	σI=>-2.0	Max	-0.10
				Min	-0.48
		Neutral Axis before Composition		Max	-0.09
				Min	-0.48
		Neutral Axis after Composition		Max	-0.10
				Min	-0.50
		Base of Lower Flange		Max	-0.05
				Min	-0.33

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section "13"	2716 kN	1513 kN	1.80

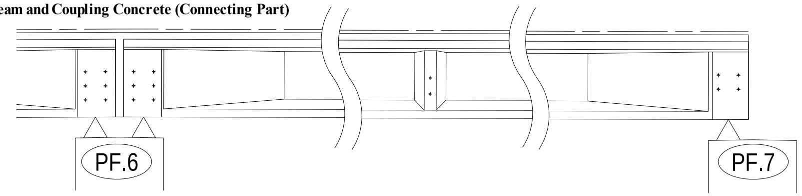
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

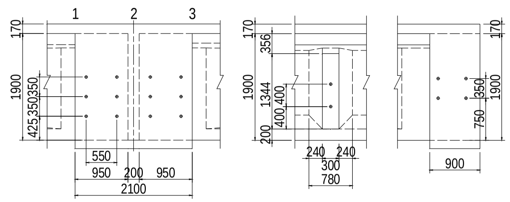
Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section "13"	1513 kN	D13@125mm

Source: JICA Study Team

Table 4.6.112 Calculation Results for the Crossbeam and Coupling Concrete (PF6-PF7)

Cross Beam and Coupling Concrete (Connecting Part)





Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1151
During Deck Construction	-	-	-	1110	1056
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0	1.15	-	-
	Bottom of Cross Beam		1.60	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max 1.59	-	-
			Min 1.13	-	-
	Bottom of Cross Beam		Max 0.93	-	-
			Min 1.84	-	-

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Diagonal Tensile Stress of Concrete: Allowable Value	Diagonal Tensile Stress of Concrete: Result
Under Dead Load	τ_m<=0.45	0.08	σI=>-0.80	0.00
Under Live Load			σI=>-1.70	-0.01

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
5682 kN	259 kN	21.94

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
2131 kN	259 kN	8.23

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1149
During Deck Construction	-	-	-	1110	1024
Under Dead Load	Top of Deck	<=10.0	-0.36	-	-
	Bottom of Deck		-0.24	-	-
	Top of Cross Beam	0.0 ~ 11.0	2.28	-	-
	Bottom of Cross Beam		4.54	-	-
Under Live Load	Top of Deck	<=10.0	Max 0.47	-	-
			Min -0.85	-	-
	Bottom of Deck		Max 0.31	-	-
			Min -0.57	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max 2.19	-	-
			Min 2.34	-	-
	Bottom of Cross Beam		Max 2.34	-	-
			Min 5.84	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2384 kNm	1234 kNm	1.93

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.68	6.00 cm2	9.75 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result	
Under Dead Load	σI=>-0.80	Max	-0.02
		Min	-0.02
Under Live Load	σI=>-1.70	Max	-0.17
		Min	-0.16

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1494 kN	533 kN	2.80

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
533 kN	D13@250mm

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2): Allowable Value	Tensile Stress of PC Strand (N/mm2): Result
During Prestressing	1440	-
Immediately after Prestressing	1295	-
Under Effective Prestressing Force	1110	-

Load Condition	Mean Compressive Stress of Concrete (N/mm2): Allowable Value	Mean Compressive Stress of Concrete (N/mm2): Result
Under Effective Prestressing Force	1.50	-

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete: Allowable Value	Compressive Strength of Concrete: Result	Tensile Stress of Re-bar: Allowable Value	Tensile Stress of Re-bar: Result
1	Under Dead Load	Negative Moment	-	-	100.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-
1	Under Live Load	Negative Moment	10.00	-	160.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-
1	Under Temperature Load	Negative Moment	11.50	-	184.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-

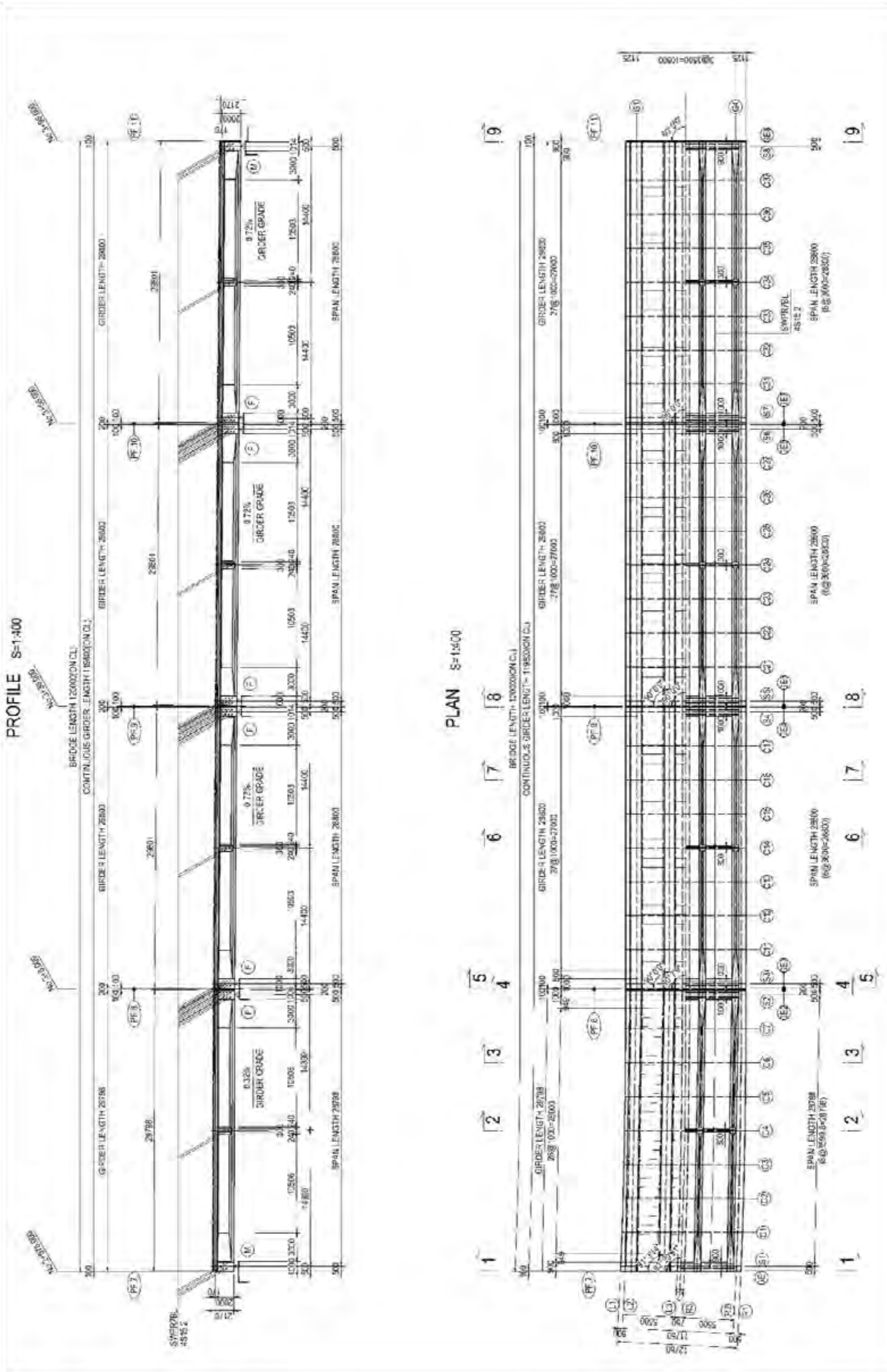
Bending Moment to Failure (Under Ultimate Load)

Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-	-	-
2	Positive Moment	-	-	-
	Negative Moment	-	-	-
3	Negative Moment		-	-

Source: JICA Study Team

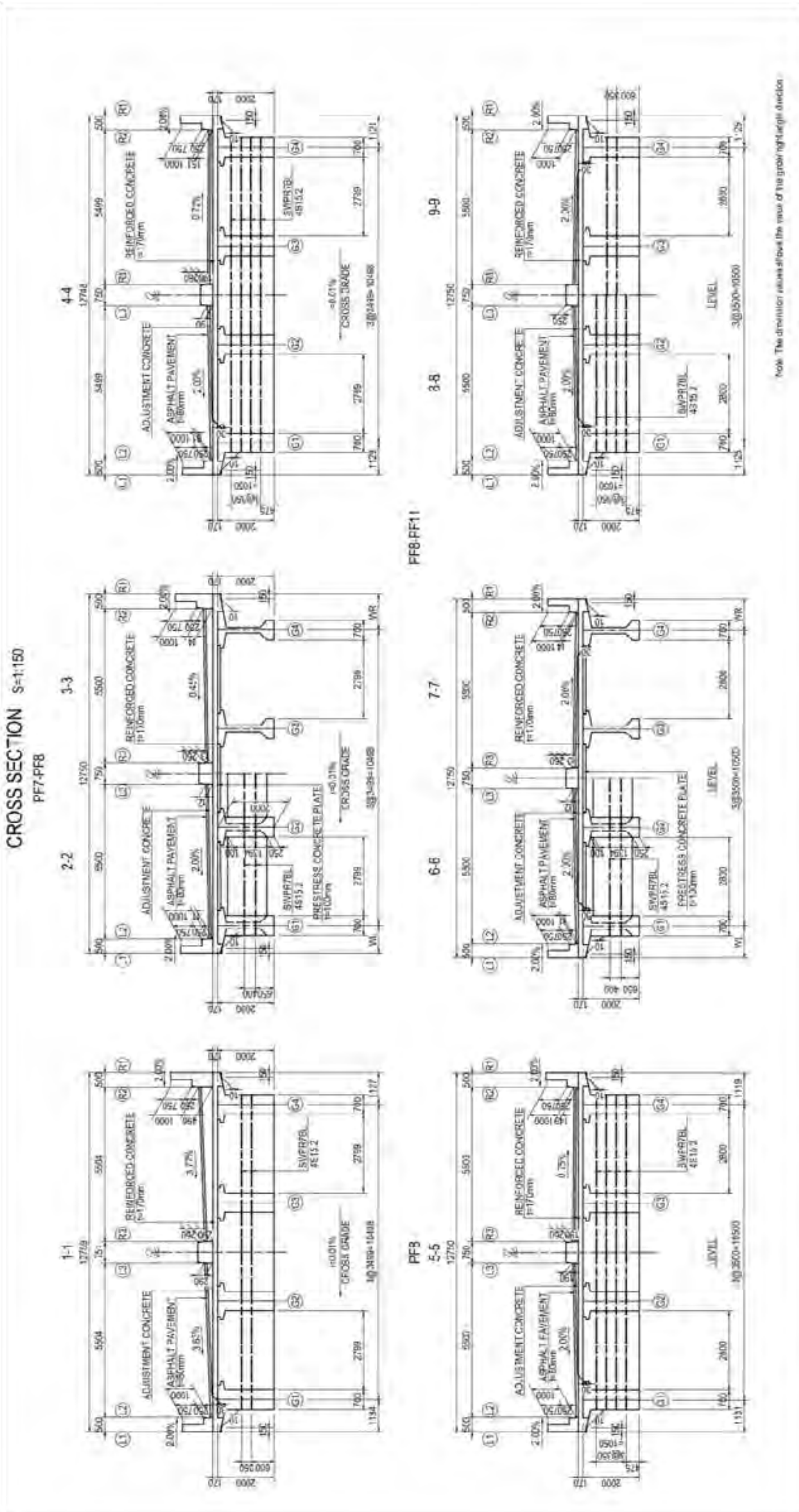
3) PF7-PF11

Figure 4.6.40 and Figure 4.6.41 shows the profile, plan, and cross section of PF7-PF11.



Source: JICA Study Team

Figure 4.6.40 Profile and Plan (PF7-PF11)



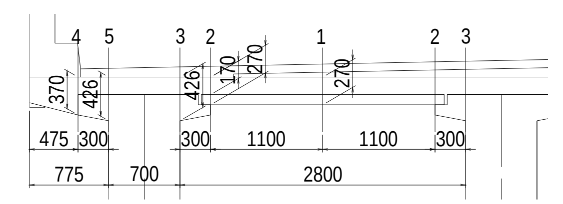
Source: JICA Study Team

Figure 4.6.41 Cross Section (PF7-PF11)

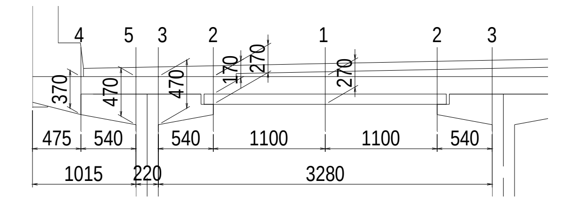
a) PF7-PF8

The following tables show the calculation results for PF7-PF8.

Table 4.6.113 Calculation Results for the Deck (PF7-PF8)



Deck at End of Girder



Deck at Center Span

Result of Design of PC Plate (Unit: N/mm2)

Deck at End of Girder: Load Condition	Checking Position	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result	Deck at Center Span: Load Condition	Checking Position	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
During Prestressing	-	-	-	1305	1225	During Prestressing	-	-	-	1305	1225
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131	Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131
	Bottom of PC Plate		8.34				Bottom of PC Plate		8.34		
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.60	1020	924	During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.62	1020	926
	Bottom of PC Plate		3.71				Bottom of PC Plate		3.73		

Result of Design of Deck (Unit: N/mm2)

Deck between Main Girders

Deck at End of Girder (Tranverse Direction): Section	Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Deck at Center Span (Tranverse Direction): Section	Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result
1	Under Live Load	Top of RC Deck	11.0	3.75	1	Under Live Load	Top of RC Deck	11.0	4.53
		Bottom of RC Deck		-0.86			Bottom of RC Deck		-1.04
		Top pf PC Plate	0.0 ~15.0	8.31			Top pf PC Plate	0.0~15.0	8.12
		Bottom of PC Plate		1.09			Bottom of PC Plate		0.29

Section	Load Condition	Type of Force	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result	Section	Load Condition	Type of Force	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
2	Under Live Load	Positive Moment	10.0	4.15	140.0	64.4	2	Under Live Load	Positive Moment	10.0	7.20	140.0	111.8
		Negative Moment		3.97		83.0			Negative Moment		3.45		72.2
3		Negative Moment		2.71		78.0	3		Negative Moment		2.70		83.0

Deck at End of Girder (Longitudinal Direction): Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result	Deck at Center Span (Longitudinal Direction): Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
Under Live Load	10.0	8.66	140.0	100.2	Under Live Load	10.0	9.96	140.0	115.3

Deck outside of Main Girder

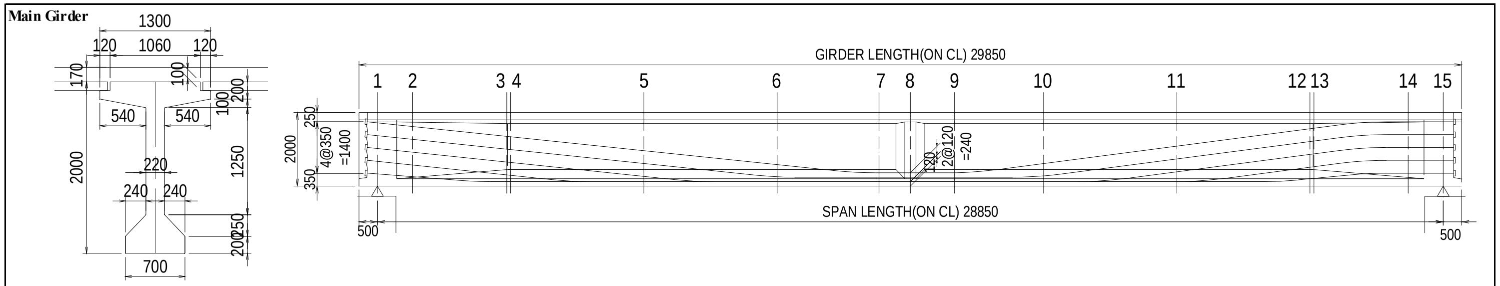
Deck at End of Girder (Tranverse Direction): Section	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result	Deck at Center Span (Tranverse Direction): Section	Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
4	Under Dead Load	-	-	100.0	5.8	4	Under Dead Load	-	-	100.0	5.8
	Under Live Load	10.0	0.22	140.0	5.8		Under Live Load	10.0	0.22	140.0	5.8
	Under Collison Load	15.0	2.06	300.0	53.9		Under Collison Load	15.0	2.06	300.0	53.9
	Under Wind Load (with Live Load)	12.5	0.37	175.0	9.6		Under Wind Load (with Live Load)	12.5	0.37	175.0	9.6
	Under Wind Load (w/o Live Load)	12.5	0.51	175.0	13.4		Under Wind Load (w/o Live Load)	12.5	0.51	175.0	13.4
5	Under Dead Load	-	-	100.0	10.8	5	Under Dead Load	-	-	100.0	14.4
	Under Live Load	10.0	1.40	140.0	40.3		Under Live Load	10.0	2.14	140.0	65.9
	Under Collison Load	15.0	2.85	300.0	82.2		Under Collison Load	15.0	3.38	300.0	103.9
	Under Wind Load (with Live Load)	12.5	1.52	175.0	43.7		Under Wind Load (with Live Load)	12.5	2.24	175.0	69.0
	Under Wind Load (w/o Live Load)	12.5	0.61	175.0	17.5		Under Wind Load (w/o Live Load)	12.5	0.67	175.0	20.6

Deck at End of Girder (Longitudinal Direction): Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result	Deck at Center Span (Longitudinal Direction): Load Condition	Strength of Concrete: Allowable Value	Strength of Concrete: Result	Stress of Re-bar: Allowable Value	Stress of Re-bar: Result
Under Live Load	10.0	1.63	140.0	59.1	Under Live Load	10.0	2.07	140.0	74.8

Source: JICA Study Team

Table 4.6.114 Calculation Results for the Main Girder (PF7-PF8)

Main Girder



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2) Allowable Value	Combined Flexural Stress (N/mm2) Result
During Prestressing	-	1440	1320
Immediately after Prestressing	Cross Section "8"	1295	1172
During Deck Construction	Cross Section "8"	1100	1071
Under Live Load	Cross Section "8"	1100	996

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2) Allowable Value		Result
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.68
		Bottom of Girder			13.88
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		4.77
		Bottom of Girder			7.86
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		3.21
		Top of Girder	0.0<σc<14.0		5.19
		Bottom of Girder			2.44
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	4.94
				Min	2.69
		Top of Girder	-1.5<σc<14.0	Max	6.67
				Min	4.75
		Bottom of Girder		Max	-1.27
				Min	3.56
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	5.67
				Min	3.42
		Top of Girder	-2.0<σc<16.10	Max	5.97
				Min	4.05
		Bottom of Girder		Max	-1.46
				Min	3.36
		Tension Re-bar Volume (cm2)			
		Minimum 13.75		Required 9.70	

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	16030 kNm	14462 kNm	1.11

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2) Allowable Value	Mean Shear Stress of Concrete (N/mm2) Result	Volume of Diagonal Tension Re-bar (cm2) Min. Diagonal Tension Re-bar Volume	Volume of Diagonal Tension Re-bar (cm2) Required Diagonal Tension Re-bar Volume
Cross Section "3"	τm<=0.55	1.34	4.40	12.44

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2) Allowable Value		Result
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-1.0		-0.16
		Neutral Axis before Composition			-0.15
		Neutral Axis after Composition			-0.17
		Base of Lower Flange			-0.10
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-2.0	Max	-0.51
				Min	-0.13
		Neutral Axis before Composition		Max	-0.55
				Min	-0.12
		Neutral Axis after Composition		Max	-0.56
				Min	-0.14
		Base of Lower Flange		Max	-0.44
				Min	-0.08

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section "3"	2873 kN	1667 kN	1.72

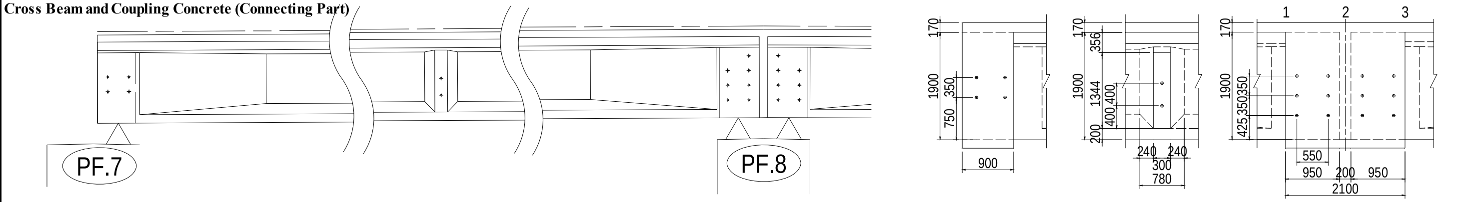
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section "3"	1667 kN	D13@125mm

Source: JICA Study Team

Table 4.6.115 Calculation Results for the Crossbeam and Coupling Concrete (PF7-PF8)

Cross Beam and Coupling Concrete (Connecting Part)



Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result		Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-		-	1440	1250
Immediately after Prestressing	-	-		-	1295	1151
During Deck Construction	-	-		-	1110	1058
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0		1.12	-	-
	Bottom of Cross Beam			1.49	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max	1.53	-	-
			Min	1.05	-	-
	Bottom of Cross Beam	0.0 ~ 12.0	Max	0.86	-	-
			Min	1.77	-	-

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Diagonal Tensile Stress of Concrete: Allowable Value	Diagonal Tensile Stress of Concrete: Result
Under Dead Load	τ_m<=0.45	0.08	σI=>-0.80	0.00
Under Live Load			σI=>-1.70	-0.01

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
5682 kN	261 kN	21.77

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
2131 kN	261 kN	8.16

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2): Allowable Value	Tensile Stress of PC Strand (N/mm2): Result
During Prestressing	1440	1250
Immediately after Prestressing	1295	1151
Under Effective Prestressing Force	1110	1050

Load Condition	Mean Compressive Stress of Concrete (N/mm2): Allowable Value	Mean Compressive Stress of Concrete (N/mm2): Result
Under Effective Prestressing Force	1.50	1.71

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result		Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-		-	1440	1250
Immediately after Prestressing	-	-		-	1295	1149
During Deck Construction	-	-		-	1110	1028
Under Dead Load	Top of Deck	<=10.0		-0.33	-	-
	Bottom of Deck			-0.23	-	-
	Top of Cross Beam	0.0 ~ 11.0		1.56	-	-
	Bottom of Cross Beam			4.73	-	-
Under Live Load	Top of Deck	<=10.0	Max	0.50	-	-
			Min	-0.81	-	-
	Bottom of Deck		Max	0.34	-	-
			Min	-0.56	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max	1.54	-	-
			Min	1.58	-	-
	Bottom of Cross Beam		Max	2.60	-	-
			Min	5.97	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2575 kNm	1348 kNm	1.91

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.61	6.00 cm2	8.49 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result	
Under Dead Load	σI=>-0.80	Max	-0.02
		Min	-0.02
Under Live Load	σI=>-1.70	Max	-0.19
		Min	-0.19

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1614 kN	524 kN	3.08

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
524 kN	D13@250mm

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete: Allowable Value	Compressive Strength of Concrete: Result	Tensile Stress of Re-bar: Allowable Value	Tensile Stress of Re-bar: Result
1	Under Dead Load	Negative Moment	-	-	100.0	16.0
2		Positive Moment		-		18.8
		Negative Moment		-		28.0
3		Negative Moment		-		17.0
1	Under Live Load	Negative Moment	10.00	4.91	160.0	136.5
2		Positive Moment		0.77		46.3
		Negative Moment		4.91		136.5
3		Negative Moment		4.91		136.5
1	Under Temperature Load	Negative Moment	11.50	4.95	184.0	137.6
2		Positive Moment		1.51		91.2
		Negative Moment		4.91		136.5
3		Negative Moment		4.77		132.6

Bending Moment to Failure (Under Ultimate Load)

Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-6286	-5883	1.07
2	Positive Moment	4802	1220	3.94
	Negative Moment	-6286	-5588	1.13
3	Negative Moment		-5641	1.11

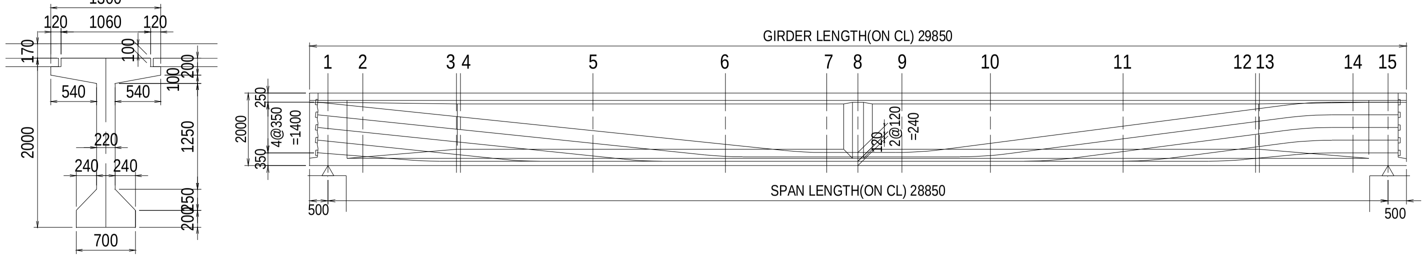
Source: JICA Study Team

b) PF8-PF9

The following tables show the calculation results for PF8-PF9.

Table 4.6.116 Calculation Results for the Main Girder (PF8-PF9)

Main Girder



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2) Allowable Value	Combined Flexural Stress (N/mm2) Result
During Prestressing	-	1440	1320
Immediately after Prestressing	Cross Section "8"	1295	1121
During Deck Construction	Cross Section "8"	1100	1025
Under Live Load	Cross Section "8"	1100	944

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2) Allowable Value		Result
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.83
		Bottom of Girder			13.09
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		5.93
		Bottom of Girder			6.10
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		2.49
		Top of Girder	0.0<σc<14.0		4.93
		Bottom of Girder			2.83
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	3.63
				Min	2.16
		Top of Girder	-1.5<σc<14.0	Max	5.89
				Min	4.64
		Bottom of Girder		Max	0.30
				Min	3.58
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	4.50
				Min	3.02
		Top of Girder	-2.0<σc<16.10	Max	5.31
				Min	4.06
		Bottom of Girder		Max	-0.28
				Min	3.00
		Tension Re-bar Volume (cm2)			
		Minimum 3.45	Required 0.46		

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	16130 kNm	12061 kNm	1.34

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2) Allowable Value	Mean Shear Stress of Concrete (N/mm2) Result	Volume of Diagonal Tension Re-bar (cm2) Min. Diagonal Tension Re-bar Volume	Volume of Diagonal Tension Re-bar (cm2) Required Diagonal Tension Re-bar Volume
Cross Section “3”	τm<=0.55	0.79	4.40	8.84

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2) Allowable Value		Result
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-1.0		0.00
		Neutral Axis before Composition			0.00
		Neutral Axis after Composition			0.00
		Base of Lower Flange			0.00
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-2.0	Max	-0.21
				Min	0.00
		Neutral Axis before Composition		Max	-0.16
				Min	0.00
		Neutral Axis after Composition		Max	-0.20
				Min	0.00
		Base of Lower Flange		Max	-0.10
				Min	0.00

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section “3”	3054 kN	1662 kN	1.84

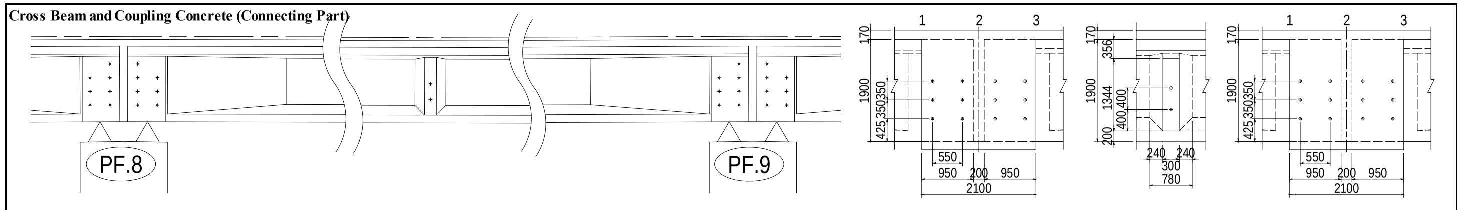
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section “3”	1662 kN	D13@125mm

Source: JICA Study Team

Table 4.6.117 Calculation Results for the Crossbeam and Coupling Concrete (PF8-PF9)

Cross Beam and Coupling Concrete (Connecting Part)



Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	-
Immediately after Prestressing	-	-	-	1295	-
During Deck Construction	-	-	-	1110	-
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0	-	-	-
	Bottom of Cross Beam		-	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max	-	-
			Min	-	-
	Bottom of Cross Beam		Max	-	-
			Min	-	-

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Diagonal Tensile Stress of Concrete: Allowable Value	Diagonal Tensile Stress of Concrete: Result
Under Dead Load	τ_m<=0.45	-	σI=>-0.80	-
Under Live Load			σI=>-1.70	-

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
- kN	- kN	-

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
- kN	- kN	-

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2): Allowable Value	Result
During Prestressing	1440	1250
Immediately after Prestressing	1295	1151
Under Effective Prestressing Force	1110	1050

Load Condition	Mean Compressive Stress of Concrete (N/mm2): Allowable Value	Result
Under Effective Prestressing Force	1.50	1.71

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1149
During Deck Construction	-	-	-	1110	1028
Under Dead Load	Top of Deck	<=10.0	-0.25	-	-
	Bottom of Deck		-0.17	-	-
	Top of Cross Beam	0.0 ~ 11.0	1.56	-	-
	Bottom of Cross Beam		4.51	-	-
Under Live Load	Top of Deck	<=10.0	Max 0.52	-	-
			Min -0.78	-	-
	Bottom of Deck		Max 0.36	-	-
			Min -0.54	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max 1.54	-	-
			Min 1.58	-	-
	Bottom of Cross Beam		Max 2.54	-	-
			Min 5.88	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2575 kNm	1292 kNm	1.99

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.54	6.00 cm2	7.45 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result	
Under Dead Load	σI=>-0.80	Max	-0.01
		Min	-0.01
Under Live Load	σI=>-1.70	Max	-0.16
		Min	-0.16

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1615 kN	483 kN	3.34

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
483 kN	D13@250mm

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete: Allowable Value	Compressive Strength of Concrete: Result	Tensile Stress of Re-bar: Allowable Value	Tensile Stress of Re-bar: Result
1	Under Dead Load	Negative Moment	-	-	100.0	0.0
2		Positive Moment		-		19.4
		Negative Moment		-		0.0
3		Negative Moment		-		0.0
1	Under Live Load	Negative Moment	10.00	3.34	160.0	92.7
2		Positive Moment		0.94		57.0
		Negative Moment		3.34		92.7
3		Negative Moment		3.34		92.7
1	Under Temperature Load	Negative Moment	11.50	3.21	184.0	89.1
2		Positive Moment		1.46		88.1
		Negative Moment		3.34		92.7
3		Negative Moment		3.21		89.2

Bending Moment to Failure (Under Ultimate Load)

Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-6286	-4306	1.46
2	Positive Moment	4802	1540	3.12
	Negative Moment	-6286	-4289	1.47
3	Negative Moment		-4309	1.46

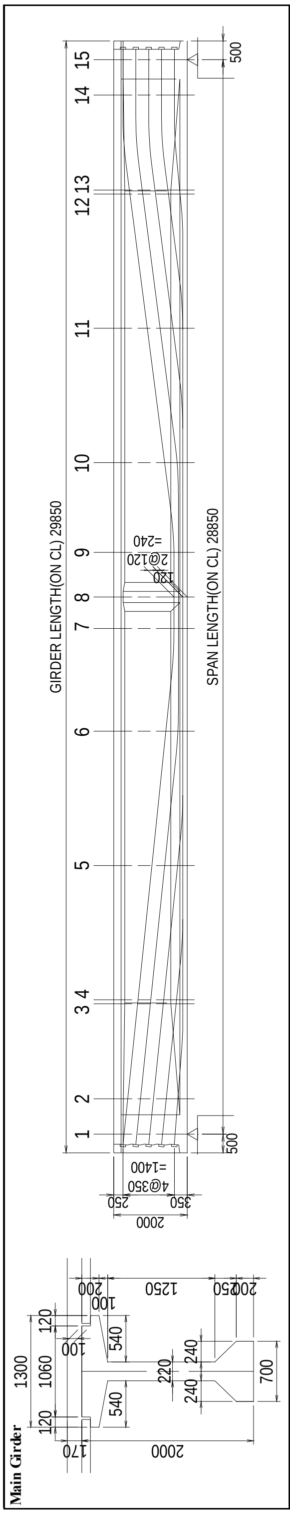
Source: JICA Study Team

c) PF9-PF10

The following tables show the calculation results for PF9-PF10.

Table 4.6.118 Calculation Results for the Main Girder (PF9-PF10)

Main Girder



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2) Allowable Value	Combined Flexural Stress (N/mm2) Result
During Prestressing	-	1440	1320
Immediately after Prestressing	Cross Section "8"	1295	1121
During Deck Construction	Cross Section "8"	1100	1025
Under Live Load	Cross Section "8"	1100	946

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2) Allowable Value		Result
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.83
		Bottom of Girder			13.09
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		5.93
		Bottom of Girder			6.10
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		2.60
		Top of Girder	0.0<σc<14.0		5.01
		Bottom of Girder			2.62
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	3.74
				Min	2.25
		Top of Girder	-1.5<σc<14.0	Max	5.98
				Min	4.72
		Bottom of Girder		Max	0.09
				Min	3.40
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	4.61
				Min	3.12
		Top of Girder	-2.0<σc<16.10	Max	5.39
				Min	4.13
		Bottom of Girder		Max	-0.48
				Min	2.83
		Tension Re-bar Volume (cm2)	Minimum 5.74		Required 1.33

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	16130 kNm	12287 kNm	1.31

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2) Allowable Value	Mean Shear Stress of Concrete (N/mm2) Result	Volume of Diagonal Tension Re-bar (cm2) Min. Diagonal Tension Re-bar Volume	Volume of Diagonal Tension Re-bar (cm2) Required Diagonal Tension Re-bar Volume
Cross Section "3"	τm<=0.55	0.75	4.40	8.25

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2) Allowable Value		Result
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-1.0		0.00
		Neutral Axis before Composition			0.00
		Neutral Axis after Composition			0.00
		Base of Lower Flange			0.00
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-2.0	Max	-0.17
				Min	0.00
		Neutral Axis before Composition		Max	-0.18
				Min	0.00
		Neutral Axis after Composition		Max	-0.18
				Min	0.00
		Base of Lower Flange		Max	-0.15
				Min	0.00

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section "3"	3057 kN	1618 kN	1.89

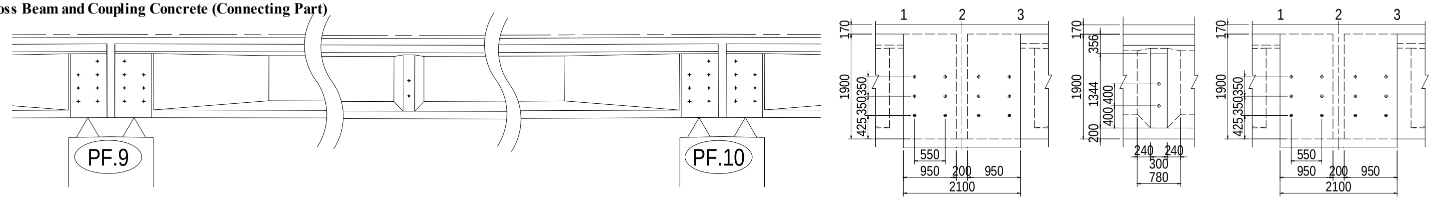
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section "3"	1618 kN	D13@125mm

Source: JICA Study Team

Table 4.6.119 Calculation Results for the Crossbeam and Coupling Concrete (PF9-PF10)

Cross Beam and Coupling Concrete (Connecting Part)



Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	-
Immediately after Prestressing	-	-	-	1295	-
During Deck Construction	-	-	-	1110	-
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0	-	-	-
	Bottom of Cross Beam		-	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max	-	-
			Min	-	-
	Bottom of Cross Beam		Max	-	-
			Min	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2575 kNm	1291 kNm	1.99

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Diagonal Tensile Stress of Concrete: Allowable Value	Diagonal Tensile Stress of Concrete: Result
Under Dead Load	τ_m<=0.45	-	σI=>-0.80	-
Under Live Load			σI=>-1.70	-

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
- kN	- kN	-

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
- kN	- kN	-

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result		Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-		-	1440	1250
Immediately after Prestressing	-	-		-	1295	1149
During Deck Construction	-	-		-	1110	1028
Under Dead Load	Top of Deck	<=10.0		-0.25	-	-
	Bottom of Deck			-0.17	-	-
	Top of Cross Beam	0.0 ~ 11.0		1.56	-	-
	Bottom of Cross Beam			4.51	-	-
Under Live Load	Top of Deck	<=10.0	Max	0.52	-	-
			Min	-0.78	-	-
	Bottom of Deck		Max	0.36	-	-
			Min	-0.54	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max	1.54	-	-
			Min	1.58	-	-
	Bottom of Cross Beam		Max	2.55	-	-
			Min	5.88	-	-

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.54	6.00 cm2	7.46 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result	
Under Dead Load	σI=>-0.80	Max	-0.01
		Min	-0.01
Under Live Load	σI=>-1.70	Max	-0.16
		Min	-0.16

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1615 kN	483 kN	3.34

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
483 kN	D13@250mm

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2): Allowable Value	Result
During Prestressing	1440	1250
Immediately after Prestressing	1295	1151
Under Effective Prestressing Force	1110	1050

Load Condition	Mean Compressive Stress of Concrete (N/mm2): Allowable Value	Result
Under Effective Prestressing Force	1.50	1.71

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete: Allowable Value	Compressive Strength of Concrete: Result	Tensile Stress of Re-bar: Allowable Value	Tensile Stress of Re-bar: Result
1	Under Dead Load	Negative Moment	-	-	100.0	0.0
2		Positive Moment		-		22.6
		Negative Moment		-		0.0
3		Negative Moment		-		0.0
1	Under Live Load	Negative Moment	10.00	3.30	160.0	91.7
2		Positive Moment		0.82		49.8
		Negative Moment		3.30		91.7
3		Negative Moment		3.30		91.7
1	Under Temperature Load	Negative Moment	11.50	3.26	184.0	90.7
2		Positive Moment		1.57		94.7
		Negative Moment		3.30		91.7
3		Negative Moment		3.41		94.7

Bending Moment to Failure (Under Ultimate Load)

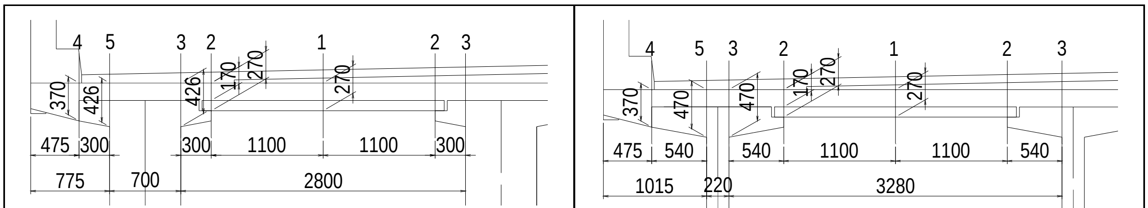
Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-6286	-4683	1.34
2	Positive Moment	4802	1264	3.80
	Negative Moment	-6286	-4600	1.37
3	Negative Moment		-4907	1.28

Source: JICA Study Team

d) PF10-PF11

The following tables show the calculation results for PF10-PF11.

Table 4.6.120 Calculation Results for the Deck (PF10-PF11)



Deck at End of Girder | Deck at Center Span

Result of Design of PC Plate (Unit: N/mm2)

Deck at End of Girder							Deck at Center Span						
Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar			Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar		
		Allowable Value	Result	Allowable Value	Result				Allowable Value	Result	Allowable Value	Result	
During Prestressing	-	-	-	1305	1225		During Prestressing	-	-	-	1305	1225	
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131		Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131	
	Bottom of PC Plate		8.34					Bottom of PC Plate		8.34			
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.58	1020	922		During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.59	1020	923	
	Bottom of PC Plate		3.69					Bottom of PC Plate		3.70			

Result of Design of Deck (Unit: N/mm2)

Deck between Main Girders

Deck at End of Girder (Tranverse Direction)							Deck at Center Span (Tranverse Direction)						
Section	Load Condition	Checking Position	Combined Flexural Stress				Section	Load Condition	Checking Position	Combined Flexural Stress			
			Allowable Value		Result					Allowable Value		Result	
1	Under Live Load	Top of RC Deck	11.0		3.53		1	Under Live Load	Top of RC Deck	11.0		4.23	
		Bottom of RC Deck			-0.81				Bottom of RC Deck			-0.97	
		Top pf PC Plate	0.0 ~15.0		8.35				Top pf PC Plate	0.0~15.0		8.17	
		Bottom of PC Plate			1.31				Bottom of PC Plate			0.58	
Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar	
			Allowable Value	Result	Allowable Value	Result				Allowable Value	Result	Allowable Value	Result
2	Under Live Load	Positive Moment	10.0	3.90	140.0	60.6	2	Under Live Load	Positive Moment	10.0	6.71	140.0	104.3
		Negative Moment		3.79		79.3			Negative Moment		3.27		68.4
3		Negative Moment		2.58		74.4	3		Negative Moment		2.56		78.6

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	8.66	140.0	100.2	Under Live Load	10.0	9.96	140.0	115.3

Deck outside of Main Girder

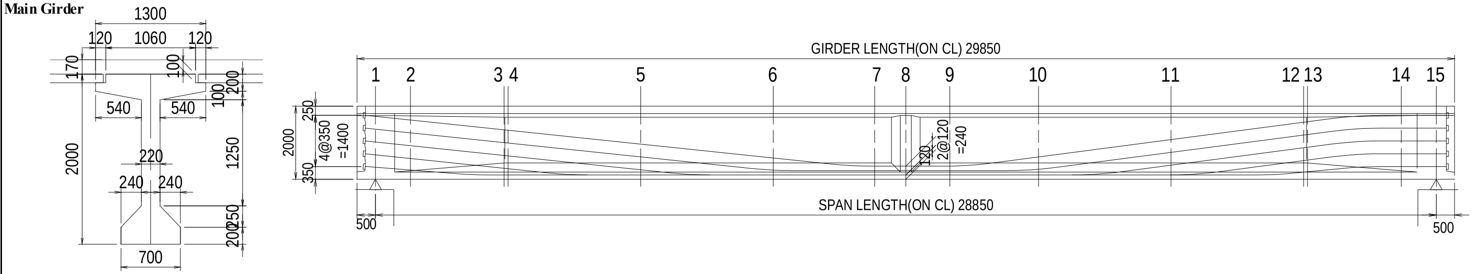
Deck at End of Girder (Tranverse Direction)						Deck at Center Span (Tranverse Direction)					
Section	Load Condition	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Strength of Concrete		Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result			Allowable Value	Result	Allowable Value	Result
4	Under Dead Load	-	-	100.0	5.4	4	Under Dead Load	-	-	100.0	5.4
	Under Live Load	10.0	0.21	140.0	5.4		Under Live Load	10.0	0.21	140.0	5.4
	Under Collison Load	15.0	2.04	300.0	53.5		Under Collison Load	15.0	2.04	300.0	53.5
	Under Wind Load (with Live Load)	12.5	0.35	175.0	9.2		Under Wind Load (with Live Load)	12.5	0.35	175.0	9.2
	Under Wind Load (w/o Live Load)	12.5	0.50	175.0	13.0		Under Wind Load (w/o Live Load)	12.5	0.50	175.0	13.0
5	Under Dead Load	-	-	100.0	10.2	5	Under Dead Load	-	-	100.0	13.8
	Under Live Load	10.0	1.14	140.0	32.8		Under Live Load	10.0	2.10	140.0	64.6
	Under Collison Load	15.0	2.59	300.0	74.7		Under Collison Load	15.0	3.33	300.0	102.5
	Under Wind Load (with Live Load)	12.5	1.26	175.0	36.2		Under Wind Load (with Live Load)	12.5	2.20	175.0	67.6
	Under Wind Load (w/o Live Load)	12.5	0.59	175.0	16.9		Under Wind Load (w/o Live Load)	12.5	0.65	175.0	19.9

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	1.62	140.0	58.5	Under Live Load	10.0	2.05	140.0	74.2

Source: JICA Study Team

Table 4.6.121 Calculation Results for the Main Girder (PF10-PF11)

Main Girder



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2)	
		Allowable Value	Result
During Prestressing	-	1440	1320
Immediately after Prestressing	Cross Section "8"	1295	1172
During Deck Construction	Cross Section "8"	1100	1071
Under Live Load	Cross Section "8"	1100	990

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2)		
			Allowable Value	Result	
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.65
		Bottom of Girder			13.94
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		5.76
		Bottom of Girder			6.88
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		2.54
		Top of Girder	0.0<σc<14.0		4.90
		Bottom of Girder			3.39
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	3.92
				Min	2.26
		Top of Girder	-1.5<σc<14.0	Max	6.07
				Min	4.66
		Bottom of Girder		Max	0.32
				Min	4.02
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	4.63
				Min	2.96
		Top of Girder	-2.0<σc<16.10	Max	5.34
				Min	3.93
		Bottom of Girder		Max	0.11
				Min	3.81
		Tension Re-bar Volume (cm2)			
		Minimum -		Required -	

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	16130 kNm	13040 kNm	1.24

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2)		Volume of Diagonal Tension Re-bar (cm2)	
	Allowable Value	Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
Cross Section "13"	τm<=0.55	1.12	4.40	9.07

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2)		
			Allowable Value	Result	
Under Dead Load	Cross Section "13"	Base of Upper Flange	σI=>-1.0		-0.07
		Neutral Axis before Composition			-0.07
		Neutral Axis after Composition			-0.08
		Base of Lower Flange			-0.04
Under Dead Load	Cross Section "13"	Base of Upper Flange	σI=>-2.0	Max	-0.06
				Min	-0.37
		Neutral Axis before Composition		Max	-0.06
				Min	-0.37
		Neutral Axis after Composition		Max	-0.06
				Min	-0.39
		Base of Lower Flange		Max	-0.04
				Min	-0.27

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section "13"	2870 kN	1492 kN	1.92

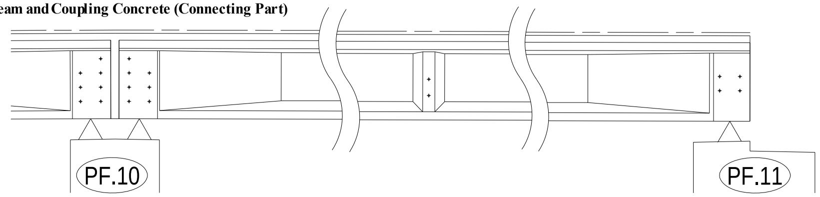
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

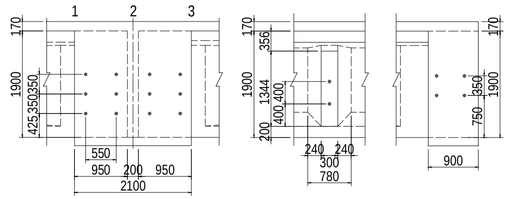
Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section "13"	1492 kN	D13@125mm

Source: JICA Study Team

Table 4.6.122 Calculation Results for the Crossbeam and Coupling Concrete (PF10-PF11)

Cross Beam and Coupling Concrete (Connecting Part)





Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1151
During Deck Construction	-	-	-	1110	1058
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0	1.12	-	-
	Bottom of Cross Beam		1.49	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max 1.53	-	-
			Min 1.05	-	-
	Bottom of Cross Beam		Max 0.86	-	-
			Min 1.77	-	-

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Diagonal Tensile Stress of Concrete: Allowable Value	Diagonal Tensile Stress of Concrete: Result
Under Dead Load	τ_m<=0.45	0.08	σI=>-0.80	0.00
Under Live Load			σI=>-1.70	-0.01

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
5682 kN	261 kN	21.77

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
2131 kN	261 kN	8.16

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2): Allowable Value	Tensile Stress of PC Strand (N/mm2): Result
During Prestressing	1440	-
Immediately after Prestressing	1295	-
Under Effective Prestressing Force	1110	-

Load Condition	Mean Compressive Stress of Concrete (N/mm2): Allowable Value	Mean Compressive Stress of Concrete (N/mm2): Result
Under Effective Prestressing Force	1.50	-

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1149
During Deck Construction	-	-	-	1110	1028
Under Dead Load	Top of Deck	<=10.0	-0.29	-	-
	Bottom of Deck		-0.20	-	-
	Top of Cross Beam	0.0 ~ 11.0	1.56	-	-
	Bottom of Cross Beam		4.62	-	-
Under Live Load	Top of Deck	<=10.0	Max 0.52	-	-
			Min -0.83	-	-
	Bottom of Deck		Max 0.36	-	-
			Min -0.57	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max 1.54	-	-
			Min 1.58	-	-
	Bottom of Cross Beam		Max 2.55	-	-
			Min 6.00	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2575 kNm	1333 kNm	1.93

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.57	6.00 cm2	7.84 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result	
Under Dead Load	σI=>-0.80	Max	-0.02
		Min	-0.02
Under Live Load	σI=>-1.70	Max	-0.17
		Min	-0.17

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1615 kN	498 kN	3.24

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
498 kN	D13@250mm

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete: Allowable Value	Compressive Strength of Concrete: Result	Tensile Stress of Re-bar: Allowable Value	Tensile Stress of Re-bar: Result
1	Under Dead Load	Negative Moment	-	-	100.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-
1	Under Live Load	Negative Moment	10.00	-	160.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-
1	Under Temperature Load	Negative Moment	11.50	-	184.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-

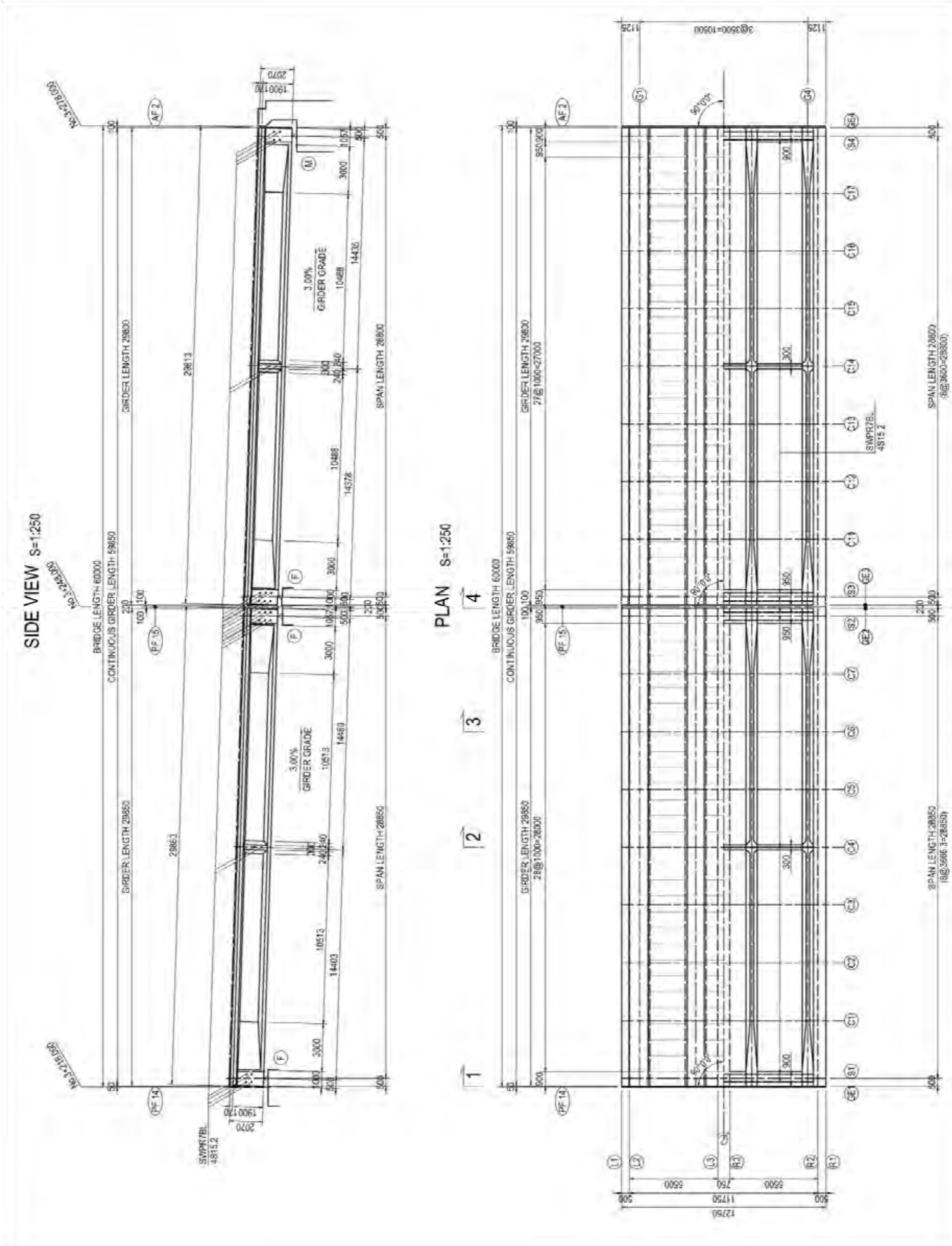
Bending Moment to Failure (Under Ultimate Load)

Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-	-	-
2	Positive Moment	-	-	-
	Negative Moment	-	-	-
3	Negative Moment		-	-

Source: JICA Study Team

4) PF14-AF2

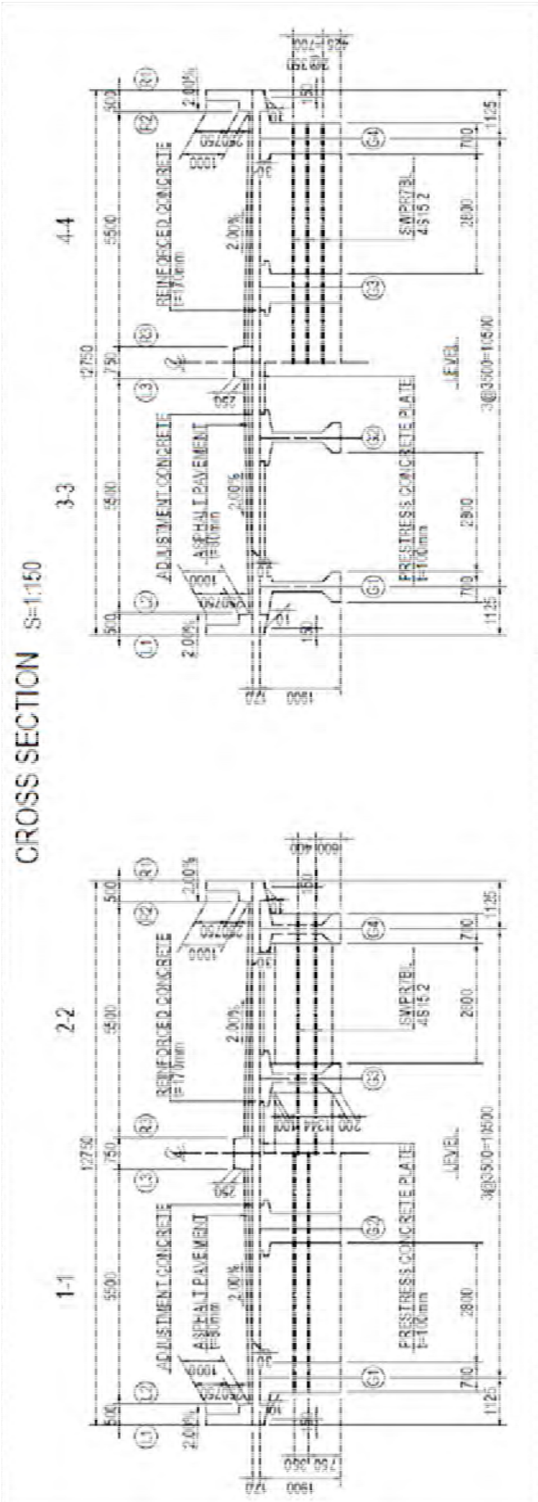
Figure 4.6.42 and Figure 4.6.43 show the profile, plan, and cross section of PF14-AF2.



Source: JICA Study Team

Source: JICA Study Team

Figure 4.6.42 Profile and Plan (PF14-AF2)



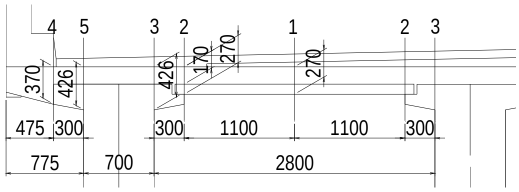
Source: JICA Study Team

Figure 4.6.43 Cross Section (PF14-AF2)

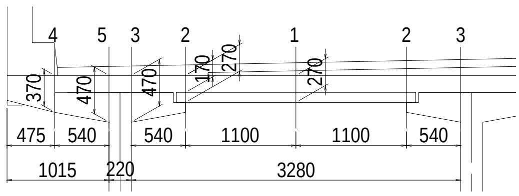
a) PF14-PF15

The following tables show the calculation results for PF14-PF15.

Table 4.6.123 Calculation Results for the Deck (PF14-PF15)



Deck at End of Girder



Deck at Center Span

Result of Design of PC Plate (Unit: N/mm2)

Deck at End of Girder						Deck at Center Span					
Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar		Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result			Allowable Value	Result	Allowable Value	Result
During Prestressing	-	-	-	1305	1225	During Prestressing	-	-	-	1305	1225
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131	Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131
	Bottom of PC Plate		8.34				Bottom of PC Plate		8.34		
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.58	1020	922	During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.59	1020	923
	Bottom of PC Plate		3.69				Bottom of PC Plate		3.70		

Result of Design of Deck (Unit: N/mm2)

Deck between Main Girders

Deck at End of Girder (Tranverse Direction)					Deck at Center Span (Tranverse Direction)				
Section	Load Condition	Checking Position	Combined Flexural Stress		Section	Load Condition	Checking Position	Combined Flexural Stress	
			Allowable Value	Result				Allowable Value	Result
1	Under Live Load	Top of RC Deck	11.0	3.53	1	Under Live Load	Top of RC Deck	11.0	4.23
		Bottom of RC Deck		-0.81			Bottom of RC Deck		-0.97
		Top pf PC Plate	0.0 ~15.0	8.35			Top pf PC Plate	0.0~15.0	8.17
		Bottom of PC Plate		1.31			Bottom of PC Plate		0.58

Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar	
			Allowable Value	Result	Allowable Value	Result				Allowable Value	Result	Allowable Value	Result
2	Under Live Load	Positive Moment	10.0	3.90	140.0	60.6	2	Under Live Load	Positive Moment	10.0	6.71	140.0	104.3
		Negative Moment		3.79		79.3			Negative Moment		3.27		68.4
3		Negative Moment		2.58		74.4	3		Negative Moment		2.56		78.6

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	8.66	140.0	100.2	Under Live Load	10.0	9.96	140.0	115.3

Deck outside of Main Girder

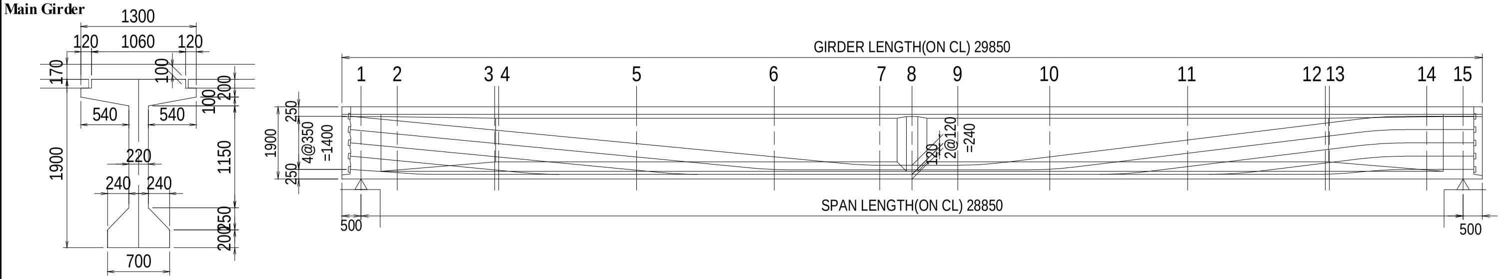
Deck at End of Girder (Tranverse Direction)						Deck at Center Span (Tranverse Direction)					
Section	Load Condition	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Strength of Concrete		Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result			Allowable Value	Result	Allowable Value	Result
4	Under Dead Load	-	-	100.0	5.4	4	Under Dead Load	-	-	100.0	5.4
	Under Live Load	10.0	0.21	140.0	5.4		Under Live Load	10.0	0.21	140.0	5.4
	Under Collison Load	15.0	2.04	300.0	53.5		Under Collison Load	15.0	2.04	300.0	53.5
	Under Wind Load (with Live Load)	12.5	0.35	175.0	9.2		Under Wind Load (with Live Load)	12.5	0.35	175.0	9.2
	Under Wind Load (w/o Live Load)	12.5	0.50	175.0	13.0		Under Wind Load (w/o Live Load)	12.5	0.50	175.0	13.0
5	Under Dead Load	-	-	100.0	10.2	5	Under Dead Load	-	-	100.0	13.8
	Under Live Load	10.0	1.14	140.0	32.8		Under Live Load	10.0	2.10	140.0	64.6
	Under Collison Load	15.0	2.59	300.0	74.7		Under Collison Load	15.0	3.33	300.0	102.5
	Under Wind Load (with Live Load)	12.5	1.26	175.0	36.2		Under Wind Load (with Live Load)	12.5	2.20	175.0	67.6
	Under Wind Load (w/o Live Load)	12.5	0.59	175.0	16.9		Under Wind Load (w/o Live Load)	12.5	0.65	175.0	19.9

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	1.62	140.0	58.5	Under Live Load	10.0	2.05	140.0	74.2

Source: JICA Study Team

Table 4.6.124 Calculation Results for the Main Girder (PF14-PF15)

Main Girder



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2) Allowable Value	Result
During Prestressing	-	1440	1300
Immediately after Prestressing	Cross Section "8"	1295	1155
During Deck Construction	Cross Section "8"	1100	1052
Under Live Load	Cross Section "8"	1100	973

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2) Allowable Value		Result
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.84
		Bottom of Girder			14.70
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		6.36
		Bottom of Girder			6.74
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		2.78
		Top of Girder	0.0<σc<14.0		5.29
		Bottom of Girder			2.89
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	4.26
				Min	2.40
		Top of Girder	-1.5<σc<14.0	Max	6.51
				Min	4.98
		Bottom of Girder		Max	-0.54
				Min	3.77
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	4.99
				Min	3.13
		Top of Girder	-2.0<σc<16.10	Max	5.80
				Min	4.27
		Bottom of Girder		Max	-0.85
				Min	3.46
		Tension Re-bar Volume (cm2)			
		Minimum 8.53		Required 3.52	

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	15295 kNm	12868 kNm	1.19

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2) Allowable Value	Result	Volume of Diagonal Tension Re-bar (cm2) Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
Cross Section "3"	τm<=0.55	1.28	4.40	10.77

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2) Allowable Value		Result
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-1.0		-0.11
		Neutral Axis before Composition			-0.11
		Neutral Axis after Composition			-0.12
		Base of Lower Flange			-0.06
Under Dead Load	Cross Section "3"	Base of Upper Flange	σI=>-2.0	Max	-0.46
				Min	-0.09
		Neutral Axis before Composition		Max	-0.48
				Min	-0.09
		Neutral Axis after Composition		Max	-0.49
				Min	-0.09
		Base of Lower Flange		Max	-0.34
				Min	-0.05

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section "3"	2710 kN	1482 kN	1.83

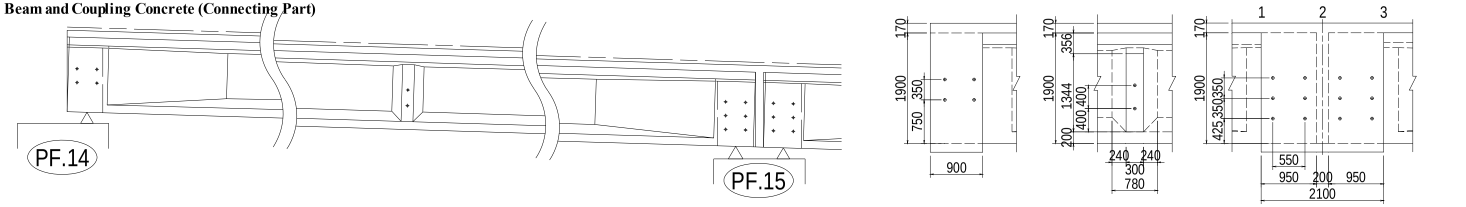
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section "3"	1482 kN	D13@125mm

Source: JICA Study Team

Table 4.6.125 Calculation Results for the Crossbeam and Coupling Concrete (PF14-PF15)

Cross Beam and Coupling Concrete (Connecting Part)



Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1151
During Deck Construction	-	-	-	1110	1056
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0	1.17	-	-
	Bottom of Cross Beam		1.58	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max 1.61	-	-
			Min 1.11	-	-
	Bottom of Cross Beam		Max 0.89	-	-
			Min 1.88	-	-

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Diagonal Tensile Stress of Concrete: Allowable Value	Diagonal Tensile Stress of Concrete: Result
Under Dead Load	τ_m<=0.45	0.08	σI=>-0.80	0.00
Under Live Load			σI=>-1.70	-0.01

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
5688 kN	253 kN	22.48

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
2133 kN	253 kN	8.43

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2): Allowable Value	Result
During Prestressing	1440	1250
Immediately after Prestressing	1295	1151
Under Effective Prestressing Force	1110	1051

Load Condition	Mean Compressive Stress of Concrete (N/mm2): Allowable Value	Result
Under Effective Prestressing Force	1.50	1.61

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress: Allowable Value	Combined Flexural Stress: Result	Stress of PC Strand: Allowable Value	Stress of PC Strand: Result
During Prestressing	-	-	-	1440	1250
Immediately after Prestressing	-	-	-	1295	1149
During Deck Construction	-	-	-	1110	1026
Under Dead Load	Top of Deck	<=10.0	-0.32	-	-
	Bottom of Deck		-0.21	-	-
	Top of Cross Beam	0.0 ~ 11.0	1.92	-	-
	Bottom of Cross Beam		4.57	-	-
Under Live Load	Top of Deck	<=10.0	Max 0.55	-	-
			Min -0.89	-	-
	Bottom of Deck		Max 0.36	-	-
			Min -0.60	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max 1.90	-	-
			Min 1.94	-	-
	Bottom of Cross Beam		Max 2.29	-	-
			Min 6.10	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2384 kNm	1326 kNm	1.80

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete: Allowable Value	Mean Shear Stress of Concrete: Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.65	6.00 cm2	9.66 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result
Under Dead Load	σI=>-0.80	Max -0.01
		Min -0.01
Under Live Load	σI=>-1.70	Max -0.18
		Min -0.18

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1495 kN	529 kN	2.83

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
529 kN	D13@250mm

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete: Allowable Value	Compressive Strength of Concrete: Result	Tensile Stress of Re-bar: Allowable Value	Tensile Stress of Re-bar: Result
1	Under Dead Load	Negative Moment	-	-	100.0	0.0
2		Positive Moment		-		27.3
		Negative Moment		-		0.0
3		Negative Moment		-		0.0
1	Under Live Load	Negative Moment	10.00	3.53	160.0	95.2
2		Positive Moment		0.74		43.5
		Negative Moment		3.53		95.2
3		Negative Moment		3.53		95.2
1	Under Temperature Load	Negative Moment	11.50	3.65	184.0	98.3
2		Positive Moment		1.62		95.5
		Negative Moment		3.53		95.2
3		Negative Moment		3.64		98.0

Bending Moment to Failure (Under Ultimate Load)

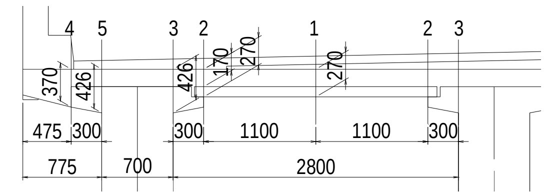
Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-5970	-5031	1.19
2	Positive Moment	4562	848	5.38
	Negative Moment	-5970	-4783	1.25
3	Negative Moment		-5023	1.19

Source: JICA Study Team

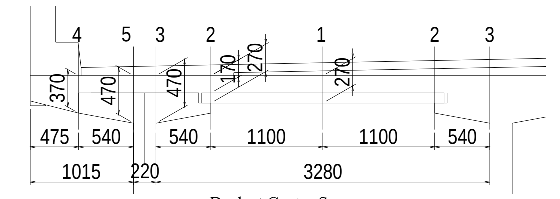
b) PF15-AF2

The following tables show the calculation results for PF15-AF2.

Table 4.6.126 Calculation Results for the Deck (PF15-AF2)



Deck at End of Girder



Deck at Center Span

Result of Design of PC Plate (Unit: N/mm2)

Deck at End of Girder						Deck at Center Span					
Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar		Load Condition	Checking Position	Strength of Concrete		Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result			Allowable Value	Result	Allowable Value	Result
During Prestressing	-	-	-	1305	1225	During Prestressing	-	-	-	1305	1225
Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131	Immediately after Prestressing	Top of PC Plate	-1.5~19.0	9.17	1190	1131
	Bottom of PC Plate		8.34				Bottom of PC Plate		8.34		
During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.58	1020	922	During Deck Construction (Under Effective Prestressing)	Top of PC Plate	0.0~15.0	10.59	1020	923
	Bottom of PC Plate		3.69				Bottom of PC Plate		3.70		

Result of Design of Deck (Unit: N/mm2)

Deck between Main Girders

Deck at End of Girder (Tranverse Direction)					Deck at Center Span (Tranverse Direction)				
Section	Load Condition	Checking Position	Combined Flexural Stress		Section	Load Condition	Checking Position	Combined Flexural Stress	
			Allowable Value	Result				Allowable Value	Result
1	Under Live Load	Top of RC Deck	11.0	3.53	1	Under Live Load	Top of RC Deck	11.0	4.23
		Bottom of RC Deck		-0.81			Bottom of RC Deck		-0.97
		Top pf PC Plate	0.0 ~15.0	8.35			Top pf PC Plate	0.0~15.0	8.17
		Bottom of PC Plate		1.31			Bottom of PC Plate		0.58

Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Type of Force	Strength of Concrete		Stress of Re-bar	
			Allowable Value	Result	Allowable Value	Result				Allowable Value	Result	Allowable Value	Result
2	Under Live Load	Positive Moment	10.0	3.90	140.0	60.6	2	Under Live Load	Positive Moment	10.0	6.71	140.0	104.3
		Negative Moment		3.79		79.3			Negative Moment		3.27		68.4
3		Negative Moment		2.58		74.4	3		Negative Moment		2.56		78.6

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	8.66	140.0	100.2	Under Live Load	10.0	9.96	140.0	115.3

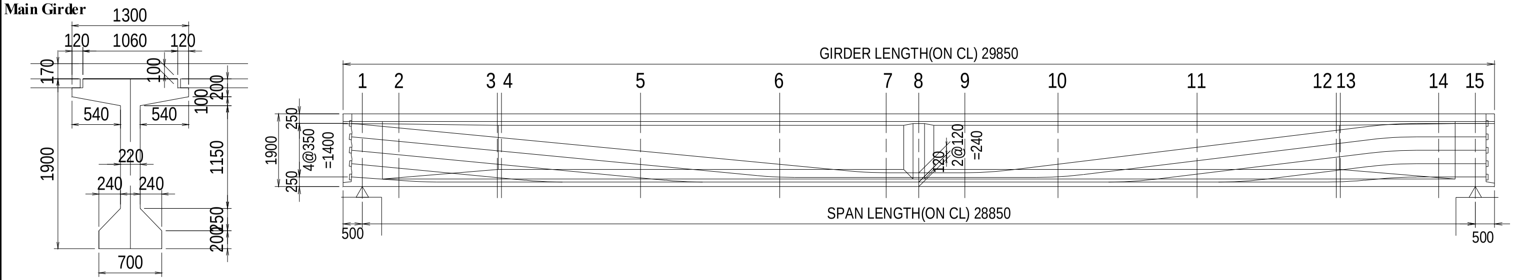
Deck outside of Main Girder

Deck at End of Girder (Tranverse Direction)						Deck at Center Span (Tranverse Direction)					
Section	Load Condition	Strength of Concrete		Stress of Re-bar		Section	Load Condition	Strength of Concrete		Stress of Re-bar	
		Allowable Value	Result	Allowable Value	Result			Allowable Value	Result	Allowable Value	Result
4	Under Dead Load	-	-	100.0	5.4	4	Under Dead Load	-	-	100.0	5.4
	Under Live Load	10.0	0.21	140.0	5.4		Under Live Load	10.0	0.21	140.0	5.4
	Under Collison Load	15.0	2.04	300.0	53.5		Under Collison Load	15.0	2.04	300.0	53.5
	Under Wind Load (with Live Load)	12.5	0.35	175.0	9.2		Under Wind Load (with Live Load)	12.5	0.35	175.0	9.2
	Under Wind Load (w/o Live Load)	12.5	0.50	175.0	13.0		Under Wind Load (w/o Live Load)	12.5	0.50	175.0	13.0
5	Under Dead Load	-	-	100.0	10.2	5	Under Dead Load	-	-	100.0	13.8
	Under Live Load	10.0	1.14	140.0	32.8		Under Live Load	10.0	2.10	140.0	64.6
	Under Collison Load	15.0	2.59	300.0	74.7		Under Collison Load	15.0	3.33	300.0	102.5
	Under Wind Load (with Live Load)	12.5	1.26	175.0	36.2		Under Wind Load (with Live Load)	12.5	2.20	175.0	67.6
	Under Wind Load (w/o Live Load)	12.5	0.59	175.0	16.9		Under Wind Load (w/o Live Load)	12.5	0.65	175.0	19.9

Deck at End of Girder (Longitudinal Direction)					Deck at Center Span (Longitudinal Direction)				
Load Condition	Strength of Concrete		Stress of Re-bar		Load Condition	Strength of Concrete		Stress of Re-bar	
	Allowable Value	Result	Allowable Value	Result		Allowable Value	Result	Allowable Value	Result
Under Live Load	10.0	1.62	140.0	58.5	Under Live Load	10.0	2.05	140.0	74.2

Source: JICA Study Team

Table 4.6.127 Calculation Results for the Main Girder (PF15-AF2)



Result of Design of Main Girder of Bending Moment

Tensile Stress of PC Strand

Load Condition	Checking Position	Combined Flexural Stress (N/mm2) Allowable Value	Combined Flexural Stress (N/mm2) Result
During Prestressing	-	1440	1300
Immediately after Prestressing	Cross Section "8"	1295	1155
During Deck Construction	Cross Section "8"	1100	1052
Under Live Load	Cross Section "8"	1100	972

Combined Flexural Stress (Checking for Volume of Tension Re-bar)

Load Condition	Section	Checking Position	Tensile Stress of Re-bar (N/mm2) Allowable Value		Result
Immediately after Prestressing	Cross Section "8"	Top of Girder	-1.5<σc<19		0.82
		Bottom of Girder			14.73
During Deck Construction	Cross Section "8"	Top of Girder	-1.5<σc<14.0		6.33
		Bottom of Girder			6.78
Under Dead Load	Cross Section "8"	Top of Deck	(Deck<10.0)		2.77
		Top of Girder	0.0<σc<14.0		5.27
		Bottom of Girder			2.93
Under Live Load	Cross Section "8"	Top of Deck	(Deck<10.0)	Max	4.24
				Min	2.39
		Top of Girder	-1.5<σc<14.0	Max	6.48
				Min	4.96
		Bottom of Girder		Max	-0.49
				Min	3.82
Under Temperature Load	Cross Section "8"	Top of Deck	(Deck<11.5)	Max	4.97
				Min	3.12
		Top of Girder	-2.0<σc<16.10	Max	5.78
				Min	4.25
		Bottom of Girder		Max	-0.81
				Min	3.50
		Tension Re-bar Volume (cm2)			
		Minimum 8.16		Required 3.19	

Bending Moment to Failure (Under Ultimate Load)

Checking Position	a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
Cross Section "8"	15295 kNm	12828 kNm	1.19

Result of Design of Main Girder of Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Checking Position	Mean Shear Stress of Concrete (N/mm2) Allowable Value	Mean Shear Stress of Concrete (N/mm2) Result	Volume of Diagonal Tension Re-bar (cm2) Min. Diagonal Tension Re-bar Volume	Volume of Diagonal Tension Re-bar (cm2) Required Diagonal Tension Re-bar Volume
Cross Section "13"	τm<=0.55	1.27	4.40	10.54

Diagonal Tensile Stress of Concrete

Load Condition	Section	Checking Position	Diagonal Tensile Stress of Concrete (N/mm2) Allowable Value		Result
Under Dead Load	Cross Section "13"	Base of Upper Flange	σI=>-1.0		-0.11
		Neutral Axis before Composition			-0.10
		Neutral Axis after Composition			-0.11
		Base of Lower Flange			-0.06
Under Dead Load	Cross Section "13"	Base of Upper Flange	σI=>-2.0	Max	-0.09
				Min	-0.46
		Neutral Axis before Composition		Max	-0.09
				Min	-0.47
		Neutral Axis after Composition		Max	-0.09
				Min	-0.49
		Base of Lower Flange		Max	-0.05
				Min	-0.32

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

Checking Position	a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
Cross Section "13"	2710 kN	1479 kN	1.83

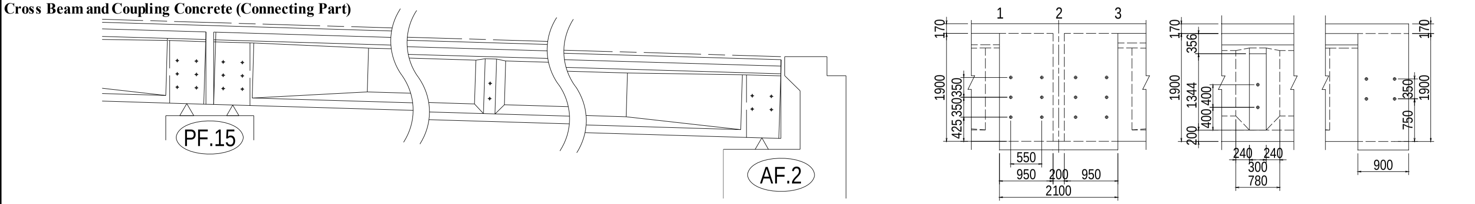
Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Checking Position	Acting Shear Force	Arrangement of Re-bar
Cross Section "13"	1479 kN	D13@125mm

Source: JICA Study Team

Table 4.6.128 Calculation Results for the Crossbeam and Coupling Concrete (PF15-AF2)

Cross Beam and Coupling Concrete (Connecting Part)



Result of Design of Cross Beam

Cross Beam at End of Main Girder

Design for Bending Moment (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress Allowable Value	Combined Flexural Stress Result		Stress of PC Strand Allowable Value	Stress of PC Strand Result
During Prestressing	-	-		-	1440	1250
Immediately after Prestressing	-	-		-	1295	1151
During Deck Construction	-	-		-	1110	1056
Under Dead Load	Top of Cross Beam	0.0 ~ 12.0		1.17	-	-
	Bottom of Cross Beam			1.58	-	-
Under Live Load	Top of Cross Beam	0.0 ~ 12.0	Max	1.61	-	-
			Min	1.11	-	-
	Bottom of Cross Beam		Max	0.89	-	-
			Min	1.88	-	-

Design for Shear Force

Mean Shear Stress of Concrete (N/mm2)

Checking Position	Mean Shear Stress of Concrete Allowable Value	Mean Shear Stress of Concrete Result	Diagonal Tensile Stress of Concrete Allowable Value	Diagonal Tensile Stress of Concrete Result
Under Dead Load	τ_m<=0.45	0.08	σI=>-0.80	0.00
Under Live Load			σI=>-1.70	-0.01

Web Concrete against Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
5688 kN	253 kN	22.48

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

a) Diagonal Tensile Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
2133 kN	253 kN	8.43

Cross Beam at Connection Part

Load Condition	Tensile Stress of PC Strand (N/mm2) Allowable Value	Result
During Prestressing	1440	-
Immediately after Prestressing	1295	-
Under Effective Prestressing Force	1110	-

Load Condition	Mean Compressive Stress of Concrete (N/mm2) Allowable Value	Result
Under Effective Prestressing Force	1.50	-

Intermediate Cross Beam

Design for Bending Moment

Combined Flexural Stress and Tensile Stress of PC Strand (N/mm2)

Load Condition	Checking Position	Combined Flexural Stress Allowable Value	Combined Flexural Stress Result		Stress of PC Strand Allowable Value	Stress of PC Strand Result
During Prestressing	-	-		-	1440	1250
Immediately after Prestressing	-	-		-	1295	1149
During Deck Construction	-	-		-	1110	1026
Under Dead Load	Top of Deck	<=10.0		-0.32	-	-
	Bottom of Deck			-0.21	-	-
	Top of Cross Beam	0.0 ~ 11.0		1.92	-	-
	Bottom of Cross Beam			4.57	-	-
Under Live Load	Top of Deck	<=10.0	Max	0.55	-	-
			Min	-0.89	-	-
	Bottom of Deck		Max	0.36	-	-
			Min	-0.60	-	-
	Top of Cross Beam	0.0 ~ 11.0	Max	1.90	-	-
			Min	1.94	-	-
	Bottom of Cross Beam		Max	2.29	-	-
			Min	6.10	-	-

Bending Moment to Failure (Under Ultimate Load)

a) Resisting Bending Moment to Failure	b) Acting Bending Moment to Failure	Safety Degree a) / b)
2384 kNm	1325 kNm	1.80

Design for Shear Force

Mean Shear Stress of Concrete (Volume of Diagonal Tension Re-bar)

Mean Shear Stress of Concrete Allowable Value	Result	Min. Diagonal Tension Re-bar Volume	Required Diagonal Tension Re-bar Volume
τ_m<=0.45	0.65	6.00 cm2	9.65 cm2

Diagonal Tensile Stress of Concrete (N/mm2)

Load Condition	Allowable Value	Result	
Under Dead Load	σI=>-0.80	Max	-0.01
		Min	-0.01
Under Live Load	σI=>-1.70	Max	-0.18
		Min	-0.18

Compressive Strength to Failure (Under Ultimate Load)

a) Compressive Strength to Failure	b) Acting Shear Force	Safety Degree a) / b)
1495 kN	529 kN	2.83

Members against Diagonal Tensile Strength to Failure (Under Ultimate Load)

Acting Shear Force	Arrangement of Re-bar
529 kN	D13@250mm

Result of Design of Coupling Concrete (Connecting Part)

Compressive Stress of Concrete and Tensile Stress of Re-bar (N/mm2)

Section	Load Condition	Type of Force	Compressive Strength of Concrete Allowable Value	Result	Tensile Stress of Re-bar Allowable Value	Result
1	Under Dead Load	Negative Moment	-	-	100.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-
1	Under Live Load	Negative Moment	10.00	-	160.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-
1	Under Temperature Load	Negative Moment	11.50	-	184.0	-
2		Positive Moment		-		-
		Negative Moment		-		-
3		Negative Moment		-		-

Bending Moment to Failure (Under Ultimate Load)

Section	Type of Force	a) Resisting Bending Moment to Failure (kNm)	b) Acting Bending Moment to Failure (kNm)	Safety Degree a) / b)
1	Negative Moment	-	-	-
2	Positive Moment	-	-	-
	Negative Moment	-	-	-
3	Negative Moment		-	-

Source: JICA Study Team

4.6.4.3 Detailed Design of Substructures and Foundations

(1) Soil Conditions

Soil conditions in the flyover section had been reviewed in the B/D based on the results of the soil investigation surveys conducted in the Supplemental F/S. In the D/D, one additional result, which is shown in Figure 4.6.44 and marked with yellow, was added to determine the geotechnical design parameters. The location and coordinates of boreholes are shown in Figure 4.6.44 and Table 4.6.129, respectively.

Source: JICA Study Team

Figure 4.6.44 Location of Boring Points in Flyover Section

Table 4.6.129 Coordinates of Boring Points

BH No.	Easting (E)	Northing (N)	Elevation MSL: (m)
BH-01	203871.632	1860013.429	+5.02
BH-02	203939.419	1859955.273	+5.05
BH-03	203988.555	1859910.930	+5.21
BH-04	204044.248	1859862.131	+5.26
BH-05	204091.678	1859823.064	+5.00
BH-06	204138.122	1859780.059	+5.18
BH-07	204182.001	1859742.035	+5.27
BH-08	204231.206	1859651.127	+5.76
BH-09	204264.719	1859651.489	+5.66
BH-10	204261.084	1859612.551	+4.97
BH-11	204288.053	1859558.128	+5.20
BH-12	204312.961	1859485.491	+4.37
BH-13	204341.023	1859405.546	+4.01
BH-14	204384.785	1859326.929	+4.52
13BH-05	204429.640	1859229.371	+4.96

Source: JICA Study Team

Regarding the information for the determination of geotechnical parameters, the FR of the Supplemental F/S had been used in the B/D. In the D/D, the geotechnical parameters determined in the B/D were reviewed and modified because additional boring was considered. The parameters established in the B/D and D/D are shown in the following tables.

Table 4.6.130 Geotechnical Design Parameters for Flyover Design in the B/D

Layer	N Average [*1]	Unit Weight "γ" (kN/m^3)	Cohesion "c" (kN/m^2)	Friction Angle "φ" [*5] (°)	Modulus of Deformation "E" (kN/m^2)
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]

Source: JICA Study Team

*1 Maximum N value is 50
*2 Average values obtained by each test
*3 Referenced by Japanese Standard (NEXCO)
*4 Calculated by C = 6 N (referenced by Japanese Standard (NEXCO)). The value of sandy soil is 0.
*5 Calculated with N value using effective overburden pressure
*6 Test value obtained by unconfined compression test
*7 E = 700 N according to the worth value obtained by borehole lateral load test

Table 4.6.131 Geotechnical Design Parameters for Flyover Design in the D/D

Layer	N Average [*1]	Unit Weight "γ" (kN/m^3)	Cohesion "c" (kN/m$_2$)	Friction Angle "φ" [*5] (°)	Modulus of Deformation "E" (kN/m^2)
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

*1 Maximum N value is 50
*2 Average values obtained by each test
*3 Referenced by Japanese Standard (NEXCO)
*4 Calculated by C = 6 N (referenced by Japanese Standard (NEXCO)). The value of sandy soil is 0.
*5 Calculated with N value using effective overburden pressure
*6 Test value obtained by unconfined compression test
*7 E = 700 N according to the worth value obtained by borehole lateral load test
*8 E = 500 N according to the worth value obtained by borehole lateral load test
Note: Red parts show the changes from the B/D to D/D.

The modulus of deformation “E” had been calculated to be 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 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, as shown in Figure 4.6.45 and Figure 4.6.46.

(2) Assessment of Soil Liquefaction

According to the JSHB V, the conditions which require liquefaction assessment are as follows:

i. The saturated soil layer which exists in the depth of less than 20 m from the existing ground level, and the groundwater level is less than 10 m from the existing ground level.

ii. The soil layer whose fine fraction content “FC” is 35% or less, or whose plasticity index “Ip” is 15 or more even if FC is over 35%.

iii. The soil layer whose mean particle diameter D50 is 10 mm or less, and whose 10% particle size D10 is 1 mm or less.

The requirements for liquefaction assessment of the ground in the flyover section are as follows:

- Saturated soil layer: Alluvium exists between GL 0 m and 20 m (Corresponding to i)
- Groundwater level: Between GL 1.5 m and 3.6 m (Corresponding to i)
- Fine fraction content “FC”: The value of FC is distributed from 8.7% (Corresponding to ii)
- Mean particle diameter D50: Maximum value is 0.73 mm up to GL -20 m (Corresponding to iii)

Considering the above, the liquefaction assessment should be conducted and the reduction coefficient “DE” of geotechnical parameters was determined according to Table 4.6.132 as specified in the JSHB V. DE was determined by using the mean value of the range of resistivity against liquefaction “FL” and dynamic shear strength ratio “R” calculated with respect to each layer for the related boreholes.

Table 4.6.132 Reduction Coefficient of Geotechnical Parameters

Range of Resistivity against Liquefaction “FL”	Depth from Existing Ground Surface “x” (m)	Dynamic Shear Strength Ratio “R”	
		R≤0.3	0.3<R
FL<1/3	0<x<10	0	1/6
	10<x<20	1/3	1/3
1/3<FL<2/3	0<x<10	1/3	2/3
	10<x<20	2/3	2/3
2/3<FL<1	0<x<10	2/3	1
	10<x<20	1	1

Source: JSHB

Although DE had been determined by using the mean value of the range of resistivity against liquefaction “FL” and dynamic shear strength ratio “R” calculated with respect to each layer for the related 14 boreholes in the B/D, another result of 13BH-05 was added to be considered in the D/D. The results of liquefaction assessment conducted in the B/D and D/D are shown in the following tables. As shown in Table 4.6.133 (a) and Table 4.6.134 (a), geotechnical parameters are reduced only in the layer of the Sandy Silt up to 10 m in depth. In the other layers, on the other hand, geotechnical parameters are not necessary to be reduced. This result of the assessment was not changed from the B/D to the D/D.

Figure 4.6.45 and Figure 4.6.46 show the soil profile including the results of liquefaction assessment in the B/D and in D/D, respectively.

Table 4.6.133 Results of Liquefaction Assessment in the 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**	

Source: JICA Study Team

Table 4.6.134 Results of Liquefaction Assessment in the D/D

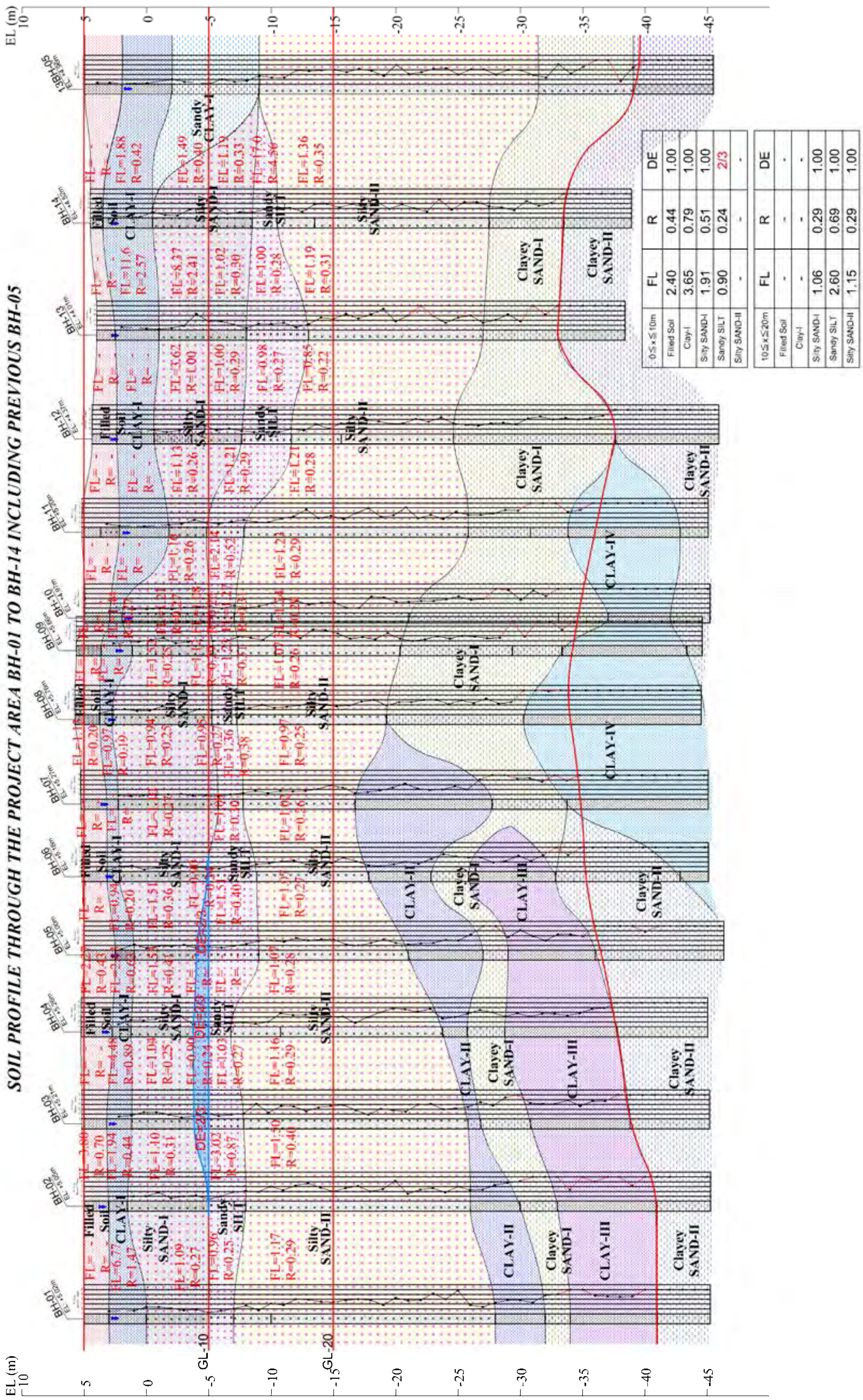
(a) 0≦x≦10

	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

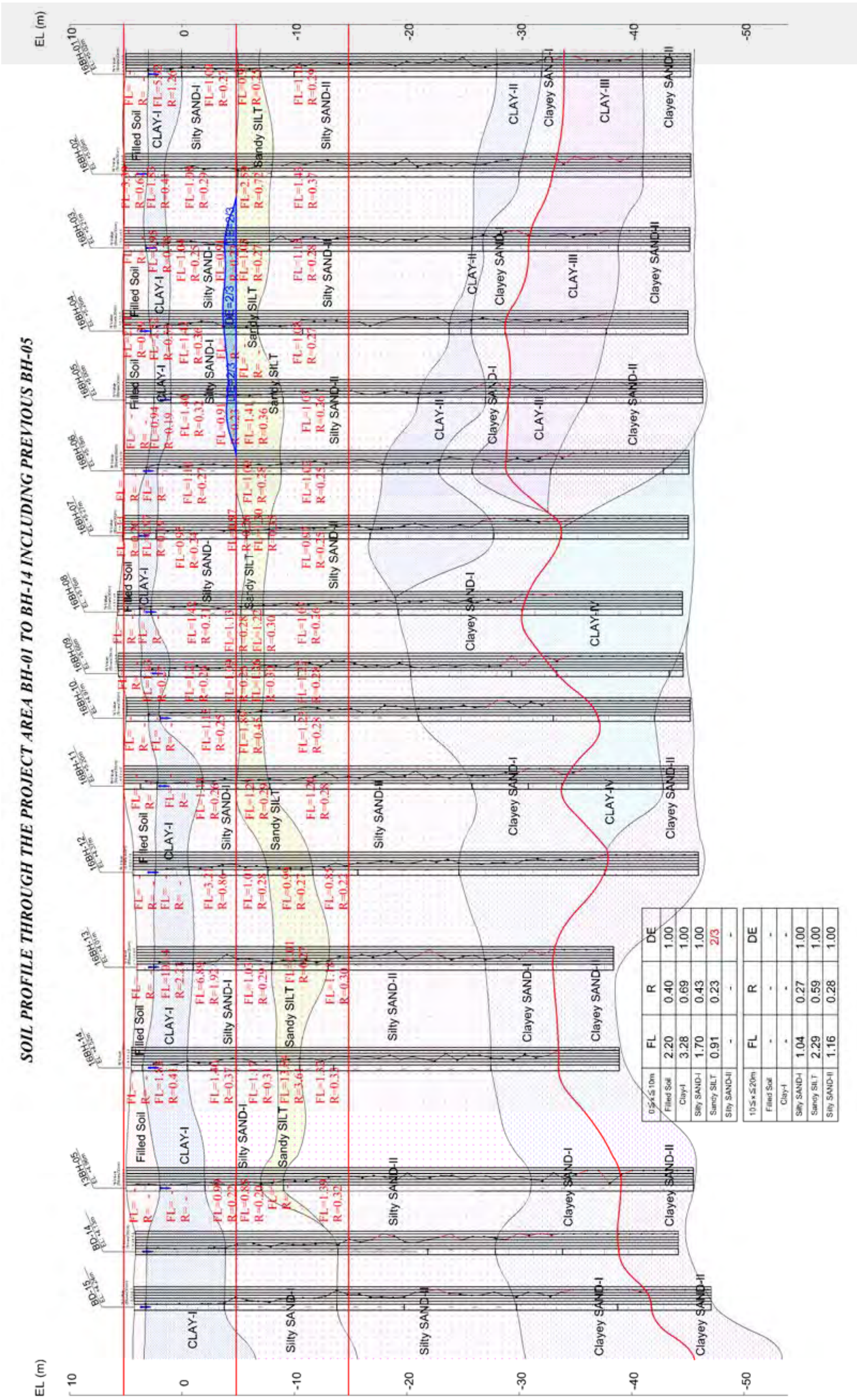
	FL	R	FL	R	FL	R	FL	R	FL	R
	FILLED SOIL		CLAY-I		SILTY SAND-I		SANDY SILT		SILTY SAND-II	
BH-01							0.975	0.250	1.163	0.286
BH-02							2.588	0.717	1.434	0.374
BH-03							1.034	0.266	1.147	0.283
BH-04									1.076	0.272
BH-05							1.409	0.362	1.071	0.263
BH-06							1.089	0.285	1.021	0.255
BH-07					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



Source: JICA Study Team

Figure 4.6.45 Soil Profile Including Results of Liquefaction Assessment in the B/D



Source: JICA Study Team

Figure 4.6.46 Soil Profile Including Results of Liquefaction Assessment in the D/D

(3) Selection of Supporting Layer

Bridge foundations should be supported by a good and hard supporting layer. The supporting layer can be determined by the following conditions as specified in the JSHB:

- Clay: N value more than 20
- Sandy Soil: N value more than 30

The Clay-III and Clay-IV layer with N value of 20 or more are identified as the supporting layer. Since the unconfined compressive strength of the layers was not confirmed, 60 N is expected to be secured as the ultimate bearing capacity of the pile end according to the Standard Design Specifications Vol. 2 Bridge Construction published by NEXCO (Japan Road Association), so that the ultimate bearing capacity of 2100 kN/m^2 and 3000 kN/m^2 can be applied for the Clay-III and Clay-IV layer respectively.

Based on the soil investigation results, the supporting layer had been defined in the B/D as shown in Figure 4.6.45. Most of the supporting layers were the upper end of Clayey Sand-II. However, some were the upper end of Clayey Sand-I and Clay-IV layers whose N value is 50 or more. The supporting layer, on the other hand, was modified in the D/D as shown in Figure 4.6.46 because the results of the soil investigation surveys were updated in the D/D. Most of the supporting layer are the upper end of the Clayey Sand-II and Clay-IV layers. However, some are the upper end of Clay-III whose N value is 20 or more.

(4) Design of Substructures and Foundation

Table 4.6.135 shows the general design conditions of the substructure, and Table 4.6.136 shows the support conditions between the superstructure and substructure of each abutment and pier. In the D/D, the condition determined in the B/D was reviewed and modified to optimize the lateral force distribution on each substructure considering the height of substructures and the soil conditions. It was modified for only two supports between PF5 and PF6, and between PF14 and PF15.

Table 4.6.135 General Design Conditions of Substructure

Item	Conditions	
General		
Structure Type	Abutment	Inverted T-type abutment
	Pier	T-type pier
	Foundation	Cast-in-place RC pile
Materials		
Concrete	Abutment and Pier	σ_{ck} = 24 N/mm^2
	Cast-in-place RC pile	σ_{ck} = 30 N/mm^2
Reinforcement Bar	SD345	
Backfilled Material	γ = 19 kN/m^3, C = 0, φ= 30	
Overburden Material	γ = 18 kN/m^3	
Foundation		
Diameter	φ = 1500 mm	
Soil Design Parameters	Referenced by 4.6.4.3(1)	
Liquefaction	Referenced by 4.6.4.3(2)	

Source: JICA Study Team

Table 4.6.136 Support Conditions between Superstructure and Substructure

Substructures	Superstructures	Support Condition in B/D	Support Condition in D/D
AF1	2@PC-I	M (Moveable Condition)	Same as on the left
PF1		F (Fixed Condition)	Same as on the left
PF2		F	Same as on the left
	3@Steel box	E (Elastic Condition)	Same as on the left
PF3		E	Same as on the left
PF4		E	Same as on the left
PF5		E	Same as on the left
	2@PC-I	M	F
PF6		F	Same as on the left
PF7		M	Same as on the left
	4@PC-I	M	Same as on the left
PF8		F	Same as on the left
PF9		F	Same as on the left
PF10		F	Same as on the left
PF11		M	Same as on the left
	3@Steel-I	E	Same as on the left
PF12		E	Same as on the left
PF13		E	Same as on the left
PF14		E	Same as on the left
	2@PC-I	M	F
PF15		F	Same as on the left
AF2		M	Same as on the left

Source: JICA Study Team

1) Strength and Allowable Stress of Materials

a) Concrete

Concrete is used for the abutments, piers, and foundations. The strength and the allowable stress of concrete are shown in Table 4.6.137.

Table 4.6.137 Strength and Allowable Stress of Concrete

Item		Unit	Abutment, Pier	Foundation
Design Strength		N/mm^2	24.00	30.00
Allowable Compressive Stress	Flexural Compressive Stress	N/mm^2	8.00	8.00
	Axial Compressive Stress	N/mm^2	6.50	6.50
Shear Stress	Resisted by Only Concrete	N/mm^2	0.23	0.23
	Resisted by Concrete and Stirrup	N/mm^2	1.70	1.70
	Punching Shear Stress	N/mm^2	0.90	-
Bond Stress		N/mm^2	1.6	1.2

Source: JICA Study Team

b) Reinforcement Bar

Reinforcement bar is used for the abutments, piers, and foundations. The strength and allowable stress of reinforcement bar to be used are shown in Table 4.6.138.

Table 4.6.138 Strength and Allowable Stress of Reinforcement Bar

<table>
<tr><th colspan="3">Item</th><th>Unit</th><th>Abutment, Pier</th><th>Foundation</th></tr>
<tr><td colspan="3">Yield Stress</td><td>N/mm^2</td><td>345</td><td>345</td></tr>
<tr><td rowspan="4">Allowable Tensile Stress</td><td colspan="2">Under Dead Load</td><td>N/mm^2</td><td>100</td><td>100</td></tr>
<tr><td rowspan="2">Under Live Load</td><td>Normal Member</td><td>N/mm^2</td><td>180</td><td>180</td></tr>
<tr><td>Underwater Member</td><td>N/mm^2</td><td>160</td><td>160</td></tr>
<tr><td colspan="2">Under Impact</td><td>N/mm^2</td><td>200</td><td>200</td></tr>
</table>

Source: Prepared by JICA Study Team based on JSHB

2) Design Parameters

a) Concrete

The design parameters of concrete are shown in Table 4.6.139.

Table 4.6.139 Design Parameters of Concrete

Item	Unit	Abutment, Pier	Foundation
Design Strength	N/mm^2	24.00	30.00
Young's Modulus	N/mm^2	2.5×10^4	2.5×10^4
Creep Coefficient	N/mm^2	2.60	2.60
Drying Shrinkage Strain	-	20.0×10^{-5}	20.0×10^{-5}

Source: Prepared by the JICA Study Team based on JSHB

b) Reinforcement Bar

The design parameters of the reinforcement bar are shown in Table 4.6.140.

Table 4.6.140 Design Parameters of Reinforcement Bar

	Unit	Reinforcement Bar
Type of Reinforcement Bar	-	SD345
Young's Modulus	N/mm^2	2.00×10^5

Source: Prepared by the JICA Study Team based on JSHB

c) Load Conditions

Table 4.6.141 shows the vertical reaction force applied by two types of superstructures, namely, PC-I girder bridge and steel box girder bridge, in the B/D. Since the design of the superstructure was updated in the D/D, the vertical reaction force was also updated as shown in Table 4.6.142

Table 4.6.141 Load Conditions in the B/D

<table>
<tr><th rowspan="2">Type of Reaction Force</th><th rowspan="2">Unit</th><th colspan="8">Substructure Number</th></tr>
<tr><th>AF1</th><th>PF1</th><th colspan="2">PF2</th><th>PF3</th><th>PF4</th><th colspan="2">PF5</th></tr>
<tr><td>Dead Load “Rd”</td><td>kN</td><td>4300</td><td>1000
0</td><td>4300</td><td>3100</td><td>11100</td><td>11100</td><td>3100</td><td>4300</td></tr>
<tr><td>Live Load “Rl”</td><td>kN</td><td>1200</td><td>2100</td><td>1200</td><td>1600</td><td>3200</td><td>3200</td><td>1600</td><td>2100</td></tr>
<tr><td>Lateral Force in Bridge Axis Direction</td><td>kN</td><td>650</td><td>1000</td><td colspan="2">1300</td><td>3200</td><td>3200</td><td colspan="2">1800</td></tr>
<tr><td>Lateral Force in Perpendicular Direction to Bridge Axis</td><td>kN</td><td>-</td><td>3000</td><td colspan="2">2300</td><td>3400</td><td>3400</td><td colspan="2">2300</td></tr>
</table>

Type of Reaction Force	Unit	Substructure Number							
		PF6	PF7		PF8	PF9	PF10	PF11	
Dead Load "Rd"	kN	10000	4300	4300	9700	9300	9700	4300	1900
Live Load "Rl"	kN	2100	1200	1200	2100	2000	2100	1200	1300
Lateral Force in Bridge Axis Direction	kN	2900	3500		3800	3800	3800	1500	
Lateral Force in Perpendicular Direction to Bridge Axis	kN	3000	2600		2800	2800	3000	1900	

Type of Reaction Force	Unit	Substructure Number					
		PF12	PF13	PF14		PF15	AF2
Dead Load "Rd"	kN	8000	8000	1900	4300	10000	4300
Live Load "Rl"	kN	2500	2500	1300	1200	2100	1200
Lateral Force in Bridge Axis Direction	kN	2100	2100	1000		1000	650
Lateral Force in Direction of Perpendicular to Bridge Axis	kN	2500	2500	1900		3000	-

Source: JICA Study Team

Table 4.6.142 Load Conditions in the D/D

Type of Reaction Force	Unit	Substructure Number							
		AF1	PF1	PF2		PF3	PF4	PF5	
Dead Load "Rd"	kN	3800	7000	3900	3200	11100	11200	3200	3800
Live Load "Rl"	kN	1100	2000	1100	1600	3100	3200	1600	1100
Lateral Force in Bridge Axis Direction	kN	600	3200	1300	1700	3200	2600	1400	2600
Lateral Force in Direction of Perpendicular to Bridge Axis	kN	-	2000	2300		3500	3100	2100	

Type of Reaction Force	Unit	Substructure Number							
		PF6	PF7		PF8	PF9	PF10	PF11	
Dead Load "Rd"	kN	7700	3900	4300	8400	7900	7800	3900	1900
Live Load "Rl"	kN	2000	1100	1100	1900	1800	1900	1100	1200
Lateral Force in Bridge Axis Direction	kN	2100	600	700	3200	2800	3800	600	1100
Lateral Force in Perpendicular Direction to Bridge Axis	kN	3000	1600		2900	2600	3000	1500	

Type of Reaction Force	Unit	Substructure Number					
		PF12	PF13	PF14		PF15	AF2
Dead Load “Rd”	kN	7800	7800	1900	3800	7700	3800
Live Load “Rl”	kN	2500	2500	1200	1100	2000	1100
Lateral Force in Bridge Axis Direction	kN	1700	1900	1300	1200	3300	600
Lateral Force in Perpendicular Direction to Bridge Axis	kN	2200	2300	1700		2800	-

Source: JICA Study Team

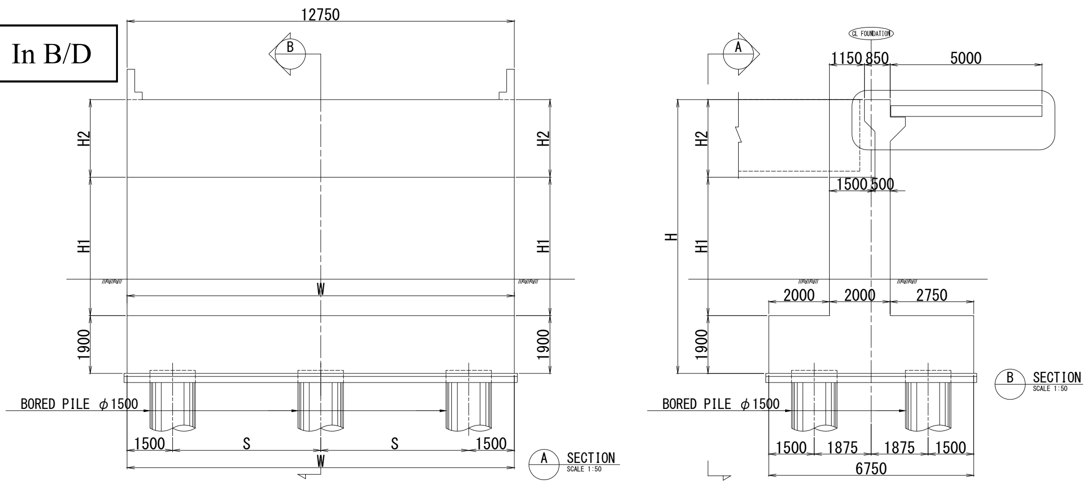
In the B/D, the structural calculation had been conducted at the representative substructures, such as AF1, PF3, PF5, PF8 and PF12, in consideration of the above design reaction force. The configuration of other substructures had been assumed based on the calculations for representative substructures. In the D/D, on the other hand, the calculation was conducted for all substructures.

(5) Design of Reversed T-type Abutment Including Foundation

The configuration of the reversed T-type abutment in the B/D is shown in Figure 4.6.47. The design had been carried out taking into consideration the load applied on the abutment wall by the bearings supporting the bridge structures. The lateral loads had also been considered, which are applied on the abutment walls due to the earth pressure generated from the retained side including surcharge loads and loads from the approach slab. In addition to the lateral earth pressure due to backfill materials, a live loading surcharge equal to 11.6 kN/m^2 had also been considered for the design.

The configuration was also modified in the D/D as shown in Figure 4.6.48 because the geotechnical design parameters and supporting layer were updated as mentioned above. The design was carried out as with the B/D.

In B/D

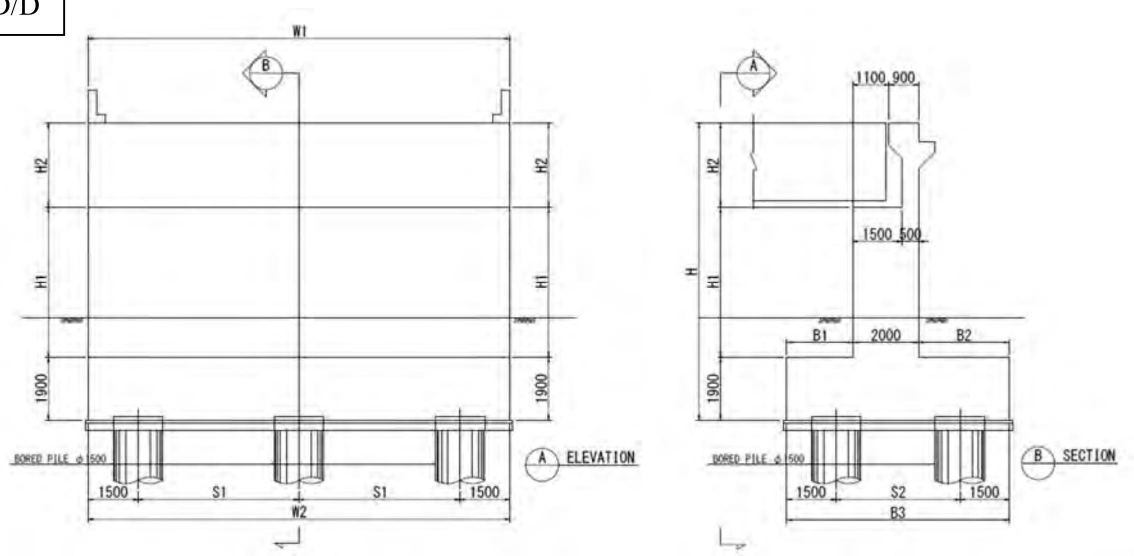


Item	Dimension		Remark
	AF1	AF2	
H (mm)	8300	9000	Total height of abutment
H1 (mm)	3980	4640	Wall height
H2 (mm)	2420	2460	Parapet height
S (mm)	3750	3750	Spacing of piles : 2.5 x Pile diameter (= 1500 mm) or more
W (mm)	14250	12750	Width of pile cap in perpendicular direction to bridge axis
Pile Length (m)	40.5	43.5	
No. of Pile	8	6	Determined by the displacement at pile head under earthquake load

Source: JICA Study Team

Figure 4.6.47 Configuration of Reversed T-type Abutment in the B/D

In D/D

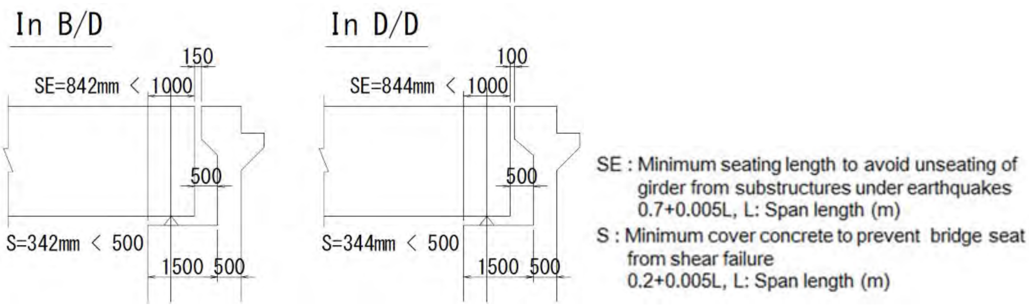


Item	Dimension		Remark
	AF1	AF2	
H (mm)	8300	9600	Total height of abutment
H1 (mm)	4050	5290	Wall height
H2 (mm)	2350	2410	Parapet height
S1 (mm)	3750	4875	Spacing of piles in perpendicular direction to bridge axis : 2.5 x Pile diameter (=1500 mm) or more
S2 (mm)	4500	4000	Spacing of piles in : 2.5 x Pile diameter (= 1500 mm) or more
W1(mm)	12770	12750	Width of abutment in perpendicular direction to bridge axis
W2 (mm)	14250	12750	Width of pile cap in perpendicular direction to bridge axis
B1 (mm)	2500	2000	Width of heel in bridge axis direction
B2 (mm)	3000	3000	Width of toe in bridge axis direction
B3 (mm)	7500	7000	Width of pile cap in bridge axis direction
Pile Length (m)	40.5	34.0	
No. of Pile	8	6	Determined by the displacement at pile head under earthquake load

Source: JICA Study Team

Figure 4.6.48 Configuration of Reversed T-type Abutment in the D/D

The width of the abutment wall had been determined to be 2.0 m considering the reinforcement arrangement, the minimum seating length to avoid unseating of girder from substructures under earthquakes (SE), and the minimum cover concrete to prevent bridge seat from shear failure (S). In the D/D, the expansion joint gap was changed, and SE and S were also modified because the span length was revised. Figure 4.6.49 shows the width of the abutment wall with SE and S in the B/D and D/D.



Source: JICA Study Team

Figure 4.6.49 Determination of Abutment Wall Width

The results of the structural calculation conducted in the D/D for AF1 and AF2 are shown in the following tables.

Table 4.6.143 Calculation Results of Abutments

AF1

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Calculated Value	Allowable Value
Under Design Load	2972＜ 7610		2288＞0		5.3□15.0	
During Earthquake	4511＜ 11592		255＞-7931		**13.9□15.0**	

Sectional Calculation Results

Member		Parapet		Wall	Footing		Pile
Location		Front	Back	Back	Under Toe	Above Heel	--
Load Case		Design Load	Earthquake	Earthquake	Design Load	Earthquake	Earthquake
Sectional Force	M (kN/m)	68.16	59.36	1012.41	688.79	772.17	2881.8
	N (kN)	——	——	540.37	——	——	254.8
	S (kN)	——	50.39	314.07	1196.39	312.77	1019.6
Reinforcement Bar Volume (mm^2)		Front D22@250 Back D22@250		Front D16@250 Back D22@250	D32@250	D25@250	D32×36
Stress (N/mm^2)	Σc	4.07	3.55	3.99	2.55	3.15	10.91
	σca	8.00	12.00	12.00	8.00	12.00	12.00
	σs	**139.95**	121.87	**206.52**	**141.47**	**230.75**	**263.79**
	σsa	**160.00**	300.00	**300.00**	**160.00**	**300.00**	**300.00**
	τ	——	0.144	0.170	0.725	0.179	0.641
	τa	——	0.548	0.204	1.647	0.405	0.499
Stirrup	Awreq, Aw	——	——	——	——	——	1.091 < 5.730

AF2

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Calculated Value	Allowable Value
Under Design Load	4206＜6169		2937＞0		6.20□15.0	
During Earthquake	6236＜9331		219＞-6595		**14.3□15.0**	

Sectional Calculation Results

Member		Parapet		Wall	Footing		Pile
Location		Front	Back	Back	Under Toe	Above Heel	--
Load Case		Design Load	Earthquake	Earthquake	Design Load	Earthquake	Earthquake
Sectional Force	M (kN/m)	68.16	62.43	1382.95	405.22	1022.44	2673.2
	N (kN)	——	——	602.33	——	——	-59.3
	S (kN)	——	52.00	419.52	——	468.95	1590.4
Reinforcement Bar Volume (mm^2)		Front D22@250 Back D22@250		Front D25@250 Back D25@250	D25@250	D29@250	D32×36
Stress (N/mm^2)	σc	4.07	3.73	4.85	1.81	3.77	10.09
	σca	8.00	12.00	12.00	8.00	12.00	12.00
	σs	**139.95**	128.18	**249.62**	**128.65**	**242.66**	**253.36**
	σsa	**160.00**	300.00	**300.00**	**160.00**	**300.00**	**300.00**
	τ	——	0.148	0.227	——	0.268	1.000
	τa	——	0.548	0.220	——	0.669	0.489
Stirrup	Awreq, Aw	——	——	0.165< 1.986	——	——	3.909 < 5.730

Source: JICA Study Team

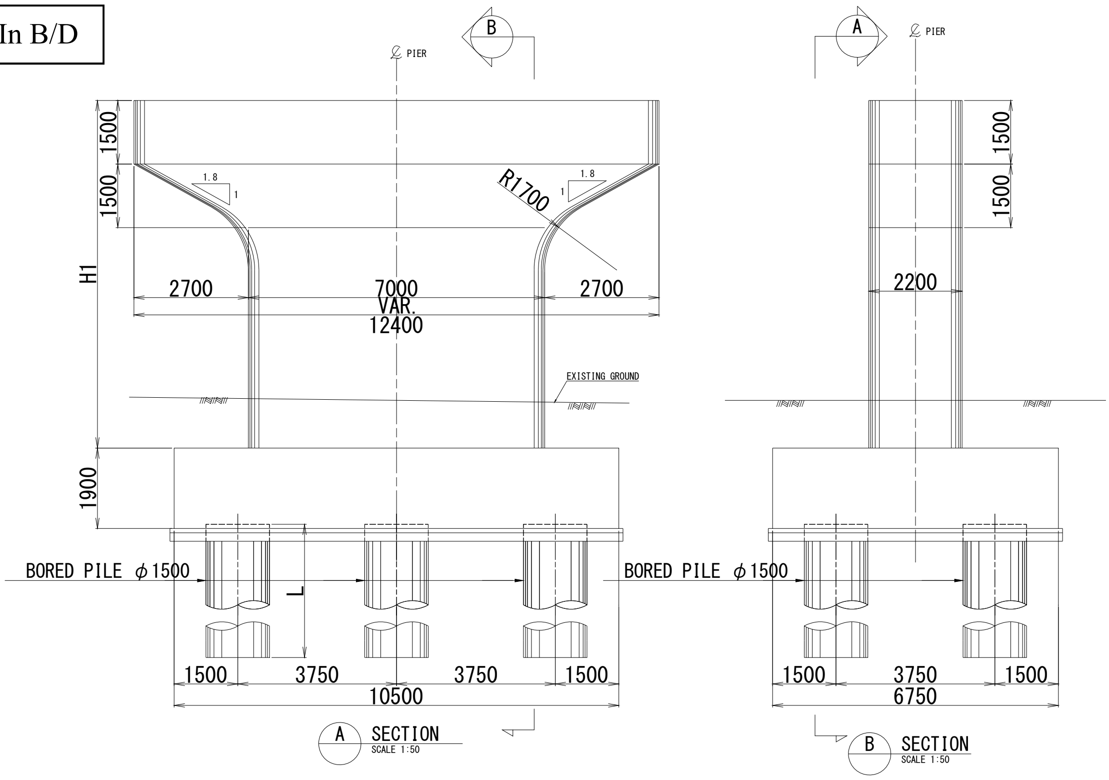
Where, M is bending moment (kN・m)
N is axial force (kN)
S is shear force (kN)
σc is compressive stress (N/mm^2)
σca is allowable compressive stress (N/mm^2)
σs is tensile stress (N/mm^2)
σsa is allowable tensile stress (N/mm^2)
τ is shear stress (N/mm^2)
τa is allowable shear stress (N/mm^2)

(6) Design of T-type Pier Including Foundation

The configurations of the T-type pier in the B/D are shown in Figure 4.6.50 and Figure 4.6.53. The circle haunch is applied to the shape of the pier head considering the landscape. The piers can be divided into two types; one is the normal type pier and the other is the end pier. Similar to the width of abutment wall, the column width is determined considering the reinforcement arrangement, the minimum seating length to avoid unseating of girders from the substructures because of earthquakes (SE), and the minimum cover concrete to prevent bridge seat from shear failure (S). In the B/D, the column width had been determined to be 2.2 m at PF8 normal piers and 3.0 m at PF5 end piers.

The configurations of the T-type pier in the D/D are shown in Figure 4.6.51 and Figure 4.6.54. The dimension was modified based on the calculation in the D/D. Same with the B/D, the column width was determined to be 2.2 m for the normal pier and 3.0 m for the end pier.

In B/D

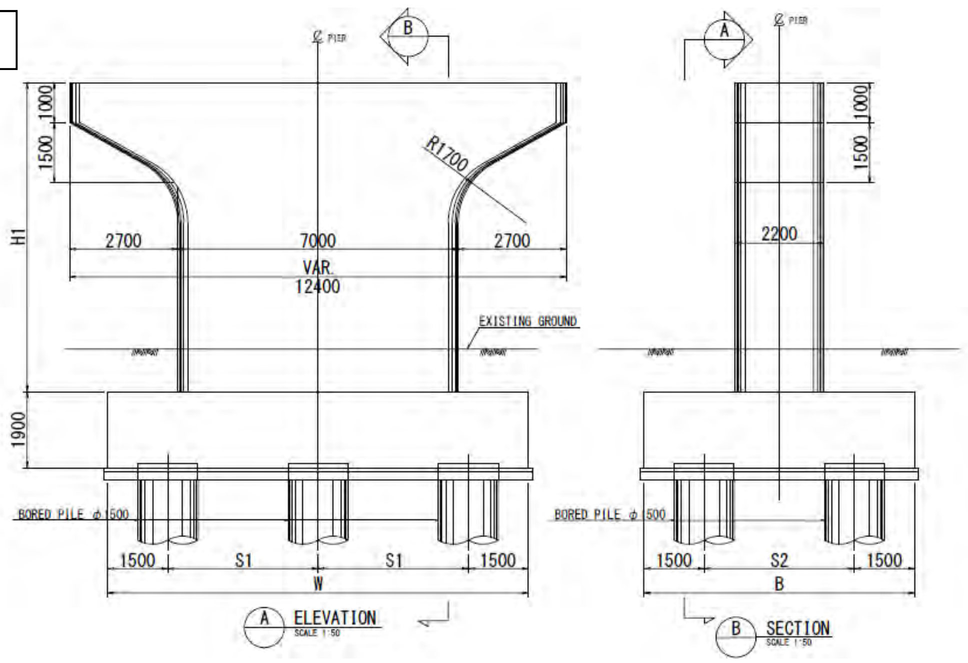


Item	Dimension											Remark
	PF1	PF3	PF4	PF6	PF7	PF8	PF9	PF10	PF12	PF13	PF15	
H1 (mm)	4800	6600	7800	8800	8800	8600	8400	8300	7500	6400	5600	Pier height
L (m)	41.5	40.0	41.0	37.5	39.0	39.0	39.0	39.0	40.0	42.0	43.0	Pile length
No. of Pile	6	6	6	6	6	6	6	6	6	6	6	Determined by the displacement at pile head under earthquake load

Source: JICA Study Team

Figure 4.6.50 Configuration of T-type Pier (Normal Pier) in the B/D

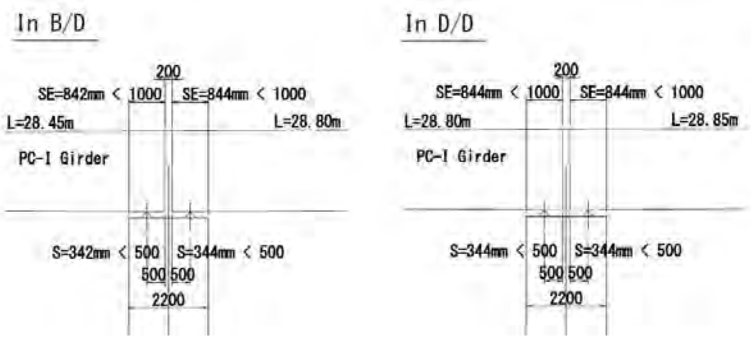
In D/D



<table>
<tr><th rowspan="2">Item</th><th colspan="11">Dimension</th><th rowspan="2">Remark</th></tr>
<tr><th>PF1</th><th>PF3</th><th>PF4</th><th>PF6</th><th>PF7</th><th>PF8</th><th>PF9</th><th>PF10</th><th>PF12</th><th>PF13</th><th>PF15</th></tr>
<tr><td>H1 (mm)</td><td>5000</td><td>6400</td><td>7900</td><td>10000</td><td>9900</td><td>9900</td><td>9500</td><td>9100</td><td>7300</td><td>6400</td><td>5900</td><td>Pier height</td></tr>
<tr><td>S1 (mm)</td><td>3750</td><td>3750</td><td>3750</td><td>3750</td><td>5000</td><td>3750</td><td>3750</td><td>3750</td><td>5000</td><td>5000</td><td>3750</td><td>Spacing of pile in perpendicular direction to bridge axis</td></tr>
<tr><td>S2 (mm)</td><td>5500</td><td>3750</td><td>3750</td><td>3750</td><td>3750</td><td>4500</td><td>3750</td><td>4500</td><td>3750</td><td>3750</td><td>4500</td><td>Spacing of pile in bridge axis direction</td></tr>
<tr><td>W (mm)</td><td>10500</td><td>10500</td><td>10500</td><td>10500</td><td>8000</td><td>10500</td><td>10500</td><td>10500</td><td>8000</td><td>8000</td><td>10500</td><td>Width of pile cap in perpendicular direction to bridge axis</td></tr>
<tr><td>B (mm)</td><td>8500</td><td>10500</td><td>6750</td><td>6750</td><td>6750</td><td>7500</td><td>6750</td><td>7500</td><td>6750</td><td>6750</td><td>7500</td><td>Width of pile cap in bridge axis direction</td></tr>
<tr><td>L (m)</td><td>41.5</td><td>38.0</td><td>40.5</td><td>33.5</td><td>37.0</td><td>37.0</td><td>35.5</td><td>32.5</td><td>33.0</td><td>32.5</td><td>34.0</td><td>Pile length</td></tr>
<tr><td>No. of Pile</td><td>6</td><td>9</td><td>6</td><td>6</td><td>4</td><td>6</td><td>6</td><td>6</td><td>4</td><td>4</td><td>6</td><td>Determined by the displacement at pile head under earthquake load</td></tr>
</table>

Source: JICA Study Team

Figure 4.6.51 Configuration of T-type Pier (Normal Pier) in the D/D



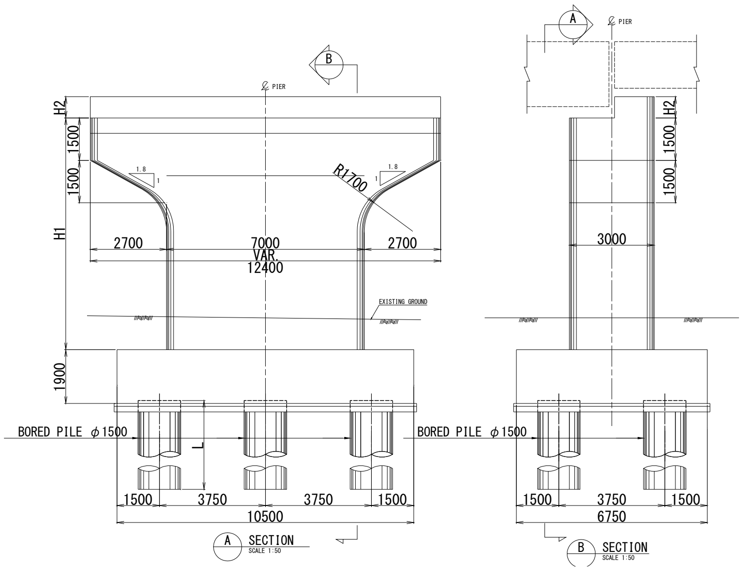
SE : Minimum seating length to avoid unseating of girder from substructures under earthquakes
0.7+0.005L, L: Span length (m)

S : Minimum cover concrete to prevent bridge seat from shear failure
0.2+0.005L, L: Span length (m)

Source: JICA Study Team

Figure 4.6.52 Determination of Column Width for Normal Pier

In B/D

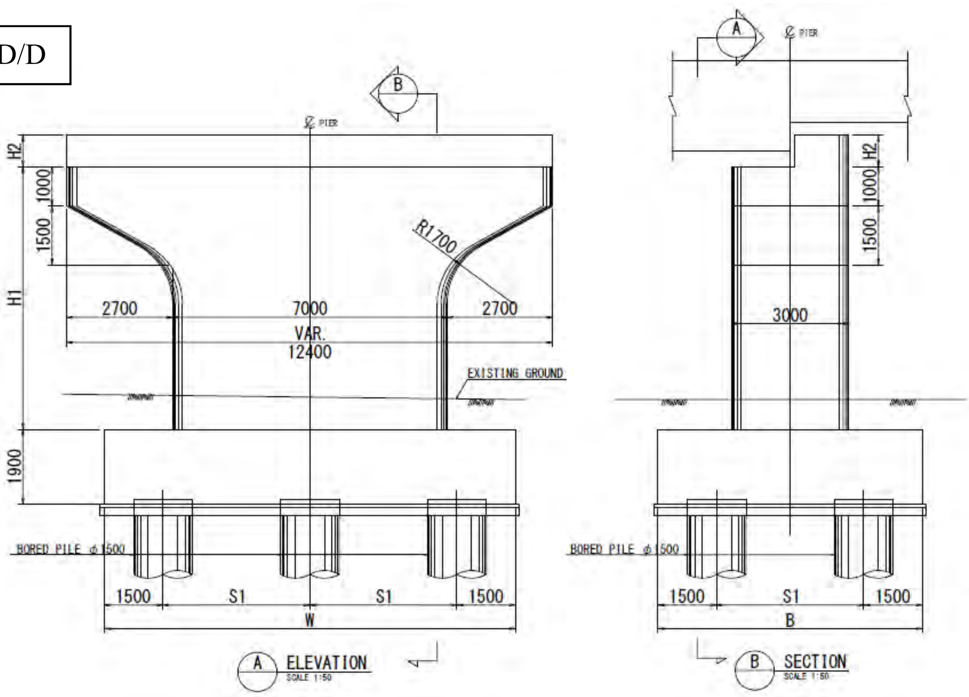


Item	Dimension				Remark
	PF2	PF5	PF11	PF14	
H1 (mm)	5000	8200	7600	5600	Pier height
H2 (mm)	830	520	580	790	Differential height of pier head
L (m)	41.5	37.5	39.5	42.5	Pile length
No. of Pile	6	6	6	6	Determined by stress of pile

Source: JICA Study Team

Figure 4.6.53 Configuration of T-type Pier (End Pier) in the B/D

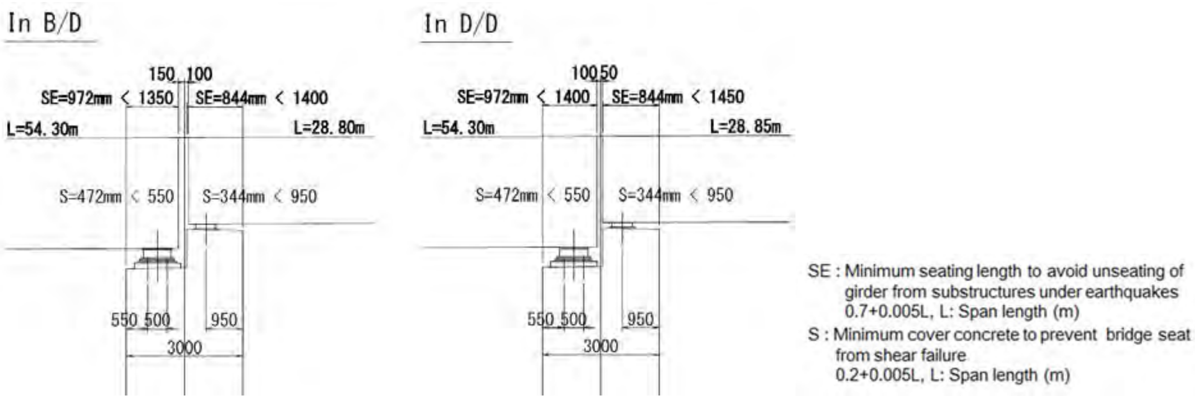
In D/D



Item	Dimension				Remark
	PF2	PF5	PF11	PF14	
H1 (mm)	5000	8800	7900	5700	Pier height
H2 (mm)	960	950	790	790	Differential height of pier head
S1 (mm)	3750	3750	5000	5000	Spacing of pile in perpendicular direction to bridge axis
S2 (mm)	5500	5000	3750	5000	Spacing of pile in bridge axis direction
W(mm)	10500	10500	8000	8000	Width of pile cap in perpendicular direction to bridge axis
B(mm)	8500	8000	6750	8000	Width of pile cap in bridge axis direction
L (m)	41.5	35.5	32.5	33.5	Pile length
No. of Pile	6	6	4	6	Determined by stress of pile

Source: JICA Study Team

Figure 4.6.54 Configuration of T-type Pier (End Pier) in the D/D



Source: JICA Study Team

Figure 4.6.55 Determination of Column Width for End Pier

The structural calculation results at all piers are shown in Table 4.6.144.

Table 4.6.144 Calculation Results for Piers

PF1 (PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	5598＜12253		-505＞-8407		**13.5＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	4680＜12253		413>-8407		**8.4＜15.0**	

Sectional Calculation Results

In Bridge Axis Direction

Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead Load	Earthquake	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	3376.90	1881.69	29017.02	-6552.70	24065.22	2652.0
	N (kN)	——	——	9395.86	——	——	-787.6
	S (kN)	2785.21	976.56	4518.76	-3539.71	15691.70	960.9
Reinforcement Bar Volume (mm²)		Top:D32×14 D32×14 Side:D22×13		D32@125	D29@250	D32@125	D32×36
Stress (N/mm^2)	σc	2.13	2.31	7.82	2.76	6.60	9.90
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**75.69**	187.82	**273.19**	177.74	**244.74**	**283.53**
	σsa	**100.00**	300.00	**300.00**	300.00	**300.00**	**300.00**
	τ	1.000	0.318	0.315	0.193	0.906	0.604
	τa	0.305	0.190	0.301	0.590	1.365	0.489
Stirrup	Awreq, Aw	1464.4 <2322.6	107.4<972.8	55.7<397.2	——	——	0.882<5.730

In Perpendicular Direction to Bridge Axis

Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	16237.02	-703.86	1535.38	1824.02
	N (kN)	——	——	9395.86	——	——	130.53
	S (kN)	——	——	2718.76	-475.66	-475.66	660.86
Reinforcement Bar Volume (mm²)		—— ——		D32@250	D19@250	D29@250	D32×36
Stress (N/mm^2)	σc	——	——	1.56	0.93	1.07	-6.90
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	6.06	96.38	66.16	168.17
	σsa	——	——	300.00	300.00	300.00	300.00
	τ	——	——	0.180	0.032	0.033	0.416
	τa	——	——	0.198	0.783	1.077	0.497
Stirrup	Awreq, Aw	——	——	——	——	——	——

Source: JICA Study Team

PF2 (Steel Box Girder Bridge and PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	6109＜12380		-592＞-8460		**14.0＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	5434＜12380		83.7>-8460		**9.8＜15.0**	

Sectional Calculation Results

In Bridge Axis Direction							
Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead Load	Earthquake	Earthquake	Earthquak e	Design Load	Earthquake
Sectional Force	M (kN/m)	2400.05	1062.33	33640.10	-5301.56	11239.22	2736.6
	N (kN)	——	——	10801.08	——	——	-874.3
	S (kN)	1333.98	1049.66	4810.32	-3585.90	17226.07	1009.5
Reinforcement Bar Volume (mm^2)		Top D32×17 Side D16×14		D29@125	D25@250	D29@125	D32×36
Stress (N/mm^2)	σc	1.47	1.03	5.92	2.47	3.34	10.20
	σca	8.00	12.00	12.00	12.00	8.00	12.00
	σs	**81.41**	136.37	**235.57**	181.06	**140.03**	**295.07**
	σsa	**100.00**	300.00	**300.00**	300.00	**160.00**	**300.00**
	τ	0.335	0.152	0.241	0.195	0.994	0.635
	τa	0.229	0.145	0.227	0.543	1.644	0.489
Stirrup	Awreq, Aw	406.5<1548.4	11.5 <972.8	55.2<397.2	——	——	1.116<5.730
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquak e	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	22058.75	-868.60	1912.48	2104.05
	N (kN)	——	——	10801.08	——	——	-198.94
	S (kN)	——	——	3410.32	-475.66	-475.66	776.12
Reinforcement Bar Volume (mm^2)		—— ——		D29@250	D16@250	D25@250	D32×36
Stress (N/mm^2)	σc	——	——	1.57	1.10	1.32	7.92
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	11.83	134.97	95.49	207.84
	σsa	——	——	300.00	300.00	300.00	300.00
	τ	——	——	0.166	0.032	0.033	0.488
	τa	——	——	0.175	0.734	0.990	0.493
Stirrup	Awreq, Aw	——	——	——	——	——	——

Source: JICA Study Team

PF3 (Steel Box Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	4884＜11021		-134＞-7338		**9.6＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	5063＜11021		-313＞-7338		**10.1＜15.0**	

Sectional Calculation Results

In Bridge Axis Direction							
Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead Load	Earthquake	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	852.7	248.41	34194.20	-7518.66	32500.48	1814.4
	N (kN)	——	——	14024.08	——	——	-366.3
	S (kN)	109.63	868.50	4077.22	-2810.73	13549.32	624.1
Reinforcement Bar Volume (mm^2)		Top D22×15 Side D16×14		D32@125	D32@250	D32@125 D32@125	D32×28
Stress (N/mm^2)	σc	0.89	0.40	9.26	2.90	7.96	7.81
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**66.18**	44.29	**290.94**	166.07	**234.12**	235.20
	σsa	**100.00**	300.00	**300.00**	300.00	**300.00**	300.00
	τ	0.023	0.173	0.284	0.153	0.799	0.393
	τa	0.170	0.179	0.301	0.387	0.895	0.448
Stirrup	Awreq, Aw	——	——	——	——	——	——
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	37104.20	-1359.45	2803.65	1911.3
	N (kN)	——	——	14024.08	——	——	-313.0
	S (kN)	——	——	4377.22	-587.58	-587.58	657.4
Reinforcement Bar Volume (mm^2)		—— ——		D32@250	D25@250	D29@250 D29@250	D32×28
Stress (N/mm^2)	σc	——	——	4.10	1.39	1.63	8.23
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	73.07	106.43	72.57	**243.95**
	σsa	——	——	300.00	300.00	300.00	**300.00**
	τ	——	——	0.290	0.031	0.034	0.414
	τa	——	——	0.198	0.907	1.363	0.448
Stirrup	Awreq, Aw	——	——	116.8<397.2	——	——	——

Source: JICA Study Team

PF4 (Steel Box Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	6457＜13382		-28＞-9003		**10.2＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	6699＜13382		-270>-9003		**7.9＜15.0**	

Sectional Calculation Results

In Bridge Axis Direction							
Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead Load	Earthquake	Earthquake	Earthquak e	Earthquake	Earthquake
Sectional Force	M (kN/m)	858.70	218.41	33717.38	-1971.48	13112.28	1893.7
	N (kN)	——	——	14690.03	——	——	-252.5
	S (kN)	109.63	718.50	3647.01	-973.18	-973.18	772.8
Reinforcement Bar Volume (mm^2)		Top D22×15 Side D16×14		D32@125	D19@250	D32@250	D32×28
Stress (N/mm^2)	σc	0.89	0.35	9.13	1.18	4.64	8.16
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**66.65**	38.94	**278.03**	117.47	**250.84**	**238.69**
	σsa	**100.00**	300.00	**300.00**	300.00	**300.00**	**300.00**
	τ	0.023	0.143	0.254	0.053	0.056	0.486
	τa	0.170	0.179	0.301	1.139	1.188	0.448
Stirrup	Awreq, Aw	——	——	——	——	——	0.289<5.73 0
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquak e	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	33917.38	-857.94	2710.67	1434.20
	N (kN)	——	——	14690.03	——	——	-494.33
	S (kN)	——	——	4147.01	-377.73	-377.73	856.13
Reinforcement Bar Volume (mm^2)		—— ——		D32@250	D16@250	D19@250	D32×28
Stress (N/mm^2)	σc	——	——	4.38	1.35	2.69	6.60
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	81.01	169.00	257.22	196.70
	σsa	——	——	300.00	300.00	300.00	300.00
	τ	——	——	0.275	0.032	0.033	0.539
	τa	——	——	0.198	0.731	0.843	0.448
Stirrup	Awreq, Aw	——	——	97.4<397.2	——	——	0.690<5.73 0

Source: JICA Study Team

PF5 (Steel-I Girder Bridge and PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	7866 ＜11610		-1857 ＞-7638		**13.6 ＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	6104 ＜11610		-95＞-7638		7.5 ＜15.0	

Sectional Calculation Results

In Bridge Axis Direction							
Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead Load	Earthquake	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	2480.09	1660.24	61262.59	-8040.90	21304.05	2140.7
	N (kN)	——	——	12643.78	——	——	-2123.0
	S (kN)	1347.67	779.30	6093.13	-7198.31	22497.57	1211.0
Reinforcement Bar Volume (mm^2)		Top D32×17 Side D16×14		D32@125 D32@250	D29@250	D32@125	D32×36
Stress (N/mm^2)	σc	1.52	1.61	8.27	3.39	5.84	7.58
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**84.13**	213.13	**293.35**	218.10	216.00	**289.96**
	σsa	**100.00**	300.00	**300.00**	300.00	300.00	**300.00**
	τ	0.344	0.187	0.309	0.392	1.299	0.762
	τa	0.231	0.142	0.270	0.675	2.011	0.489
Stirrup	Awreq, Aw	435.3<1548.4	49.6<972.8	159.2<397.2	——	——	2.085<5.730
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	33719.64	-904.62	2294.93	1386.73
	N (kN)	——	——	12643.78	——	——	-360.74
	S (kN)	——	——	3793.13	-447.68	-447.68	827.70
Reinforcement Bar Volume (mm^2)		—— ——		D32@250	D19@250	D29@250	D32×36
Stress (N/mm^2)	σc	——	——	2.70	0.97	1.43	5.18
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	47.68	97.87	90.83	146.19
	σsa	——	——	300.00	300.00	300.00	300.00
	τ	——	——	0.185	0.032	0.033	0.521
	τa	——	——	0.206	0.784	1.078	0.489
Stirrup	Awreq, Aw	——	——	——	——	——	0.242<5.730

Source: JICA Study Team

PF6 (PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	6107＜10984		-580＞-7146		**10.7＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	6485＜10984		-958＞-7146		8.4＜15.0	

Sectional Calculation Results

In Bridge Axis Direction							
Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead Load	Earthquake	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	3824.50	1233.21	36085.67	-3255.48	12297.83	2069.9
	N (kN)	——	——	11982.35	——	——	-804.8
	S (kN)	3002.08	615.62	3684.71	-973.18	-973.18	779.1
Reinforcement Bar Volume (mm^2)		Top D32×14 D32×14 Side D22×13		D29@125 D29@125	D19@250	D32@250	D32×36
Stress (N/mm^2)	σc	2.41	1.51	8.22	1.94	4.35	7.69
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**85.72**	123.09	**229.45**	193.97	**235.26**	**229.01**
	σsa	**100.00**	300.00	**300.00**	300.00	**300.00**	**300.00**
	τ	1.097	0.204	0.263	0.053	0.056	0.490
	τa	0.308	0.191	0.357	1.139	1.188	0.489
Stirrup	Awreq, Aw	1664.1<2322.6	10.4<972.8	——	——	——	0.008<5.730
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	43345.67	-1202.17	2603.62	1473.0
	N (kN)	——	——	11982.35	——	——	-1182.8
	S (kN)	——	——	4284.71	-377.73	-377.73	879.1
Reinforcement Bar Volume (mm^2)		—— ——		D29@250	D16@250	D19@250	D32×36
Stress (N/mm^2)	σc	——	——	6.69	1.89	2.58	6.29
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	291.89	**236.81**	**247.06**	187.99
	σsa	——	——	300.00	**300.00**	**300.00**	300.00
	τ	——	——	0.284	0.032	0.033	0.553
	τa	——	——	0.229	0.731	0.843	0.489
Stirrup	Awreq, Aw	——	——	69.4<397.2	——	——	0.489<5.730

Source: JICA Study Team

PF7 (PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN) Calculated Value	Pushing Force (kN) Allowable Value	Tensile Force (kN) Calculated Value	Tensile Force (kN) Allowable Value	Displacement (mm) Allowable Value	Displacement (mm) Calculated Value
During Earthquake in Bridge Axis Direction	7372＜12276		596＞-8139		**11.0＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	7307＜12276		661＞-8139		9.7＜15.0	

Sectional Calculation Results

In Bridge Axis Direction							
Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead Load	Earthquake	Earthquake	Earthquake	Design Load	Earthquake
Sectional Force	M (kN/m)	4030.06	698.48	23270.62	-530.96	5638.51	2017.8
	N (kN)	——	——	12519.03	——	——	301.9
	S (kN)	2758.92	350.68	2595.71	-741.47	-741.47	837.5
Reinforcement Bar Volume (mm^2)		Top D32×14 D32×14 Side D22×13		D25@125	D19@250	D32@250	D32×24
Stress (N/mm^2)	σc	2.54	0.86	7.56	0.34	2.58	9.42
	σca	8.00	12.00	12.00	12.00	8.00	12.00
	σs	**90.32**	69.72	**252.84**	33.58	**140.52**	**257.85**
	σsa	**100.00**	300.00	**300.00**	300.00	**160.00**	**300.00**
	τ	0.922	0.110	0.181	0.053	0.056	0.527
	τa	0.298	0.196	0.258	1.141	1.192	0.456
Stirrup	Awreq, Aw	1316.3<2322.6	——	——	——	——	0.545<5.730
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Design Load	——	Earthquake
Sectional Force	M (kN/m)	——	——	26498.74	-59.02	——	1528.64
	N (kN)	——	——	12519.03	——	——	367.27
	S (kN)	——	——	2895.71	——	——	912.46
Reinforcement Bar Volume (mm^2)		—— ——		D25@125	D16@250	D19@250	D32×24
Stress (N/mm^2)	σc	——	——	2.64	0.09	——	7.12
	σca	——	——	12.00	8.00	——	12.00
	σs	——	——	23.45	11.63	——	187.07
	σsa	——	——	300.00	160.00	——	300.00
	τ	——	——	0.192	——	——	0.574
	τa	——	——	0.175	——	——	0.628
Stirrup	Awreq, Aw	——	——	21.2<397.2	——	——	——

Source: JICA Study Team

PF8 (PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN) Calculated Value	Pushing Force (kN) Allowable Value	Tensile Force (kN) Calculated Value	Tensile Force (kN) Allowable Value	Displacement (mm) Allowable Value	Displacement (mm) Calculated Value
During Earthquake in Bridge Axis Direction	7084＜12076		-1153＞-8009		**12.3＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	6603＜12076		-673＞-8009		8.2＜15.0	

Member Calculation Results

In Bridge Axis Direction

Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead load	Earthquake	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	3969.10	1838.91	46517.77	-6741.36	21860.04	1998.5
	N (kN)	——	——	12644.63	——	——	-1438.4
	S (kN)	3095.13	1019.54	5373.39	-5176.45	20149.56	1062.2
Reinforcement Bar Volume (mm^2)		Top D32×14 D32×14 Side D22×13		D32@125 D32@125	D29@250	D32@125	D32×36
Stress (N/mm^2)	σc	2.50	2.26	9.51	2.84	5.99	7.26
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**88.96**	183.55	**257.46**	182.85	**222.31**	**248.19**
	σsa	**100.00**	300.00	**300.00**	300.00	**300.00**	**300.00**
	τ	1.051	0.317	0.377	0.282	1.163	0.668
	τa	0.296	0.195	0.378	0.782	1.862	0.489
Stirrup	Awreq, Aw	1595.3 <2322.6	107.2<972.8	——	——	——	1.369<5.730

In Perpendicular Direction to Bridge Axis

Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	41717.77	-1139.58	2604.81	1472.28
	N (kN)	——	——	12644.63	——	——	-958.08
	S (kN)	——	——	4173.39	-419.7	-419.70	878.86
Reinforcement Bar Volume (mm^2)		—— ——		D32@250	D19@250	D29@250	D32×36
Stress (N/mm^2)	σc	——	——	5.60	1.51	1.82	5.38
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	183.86	156.05	112.24	178.66
	σsa	——	——	300.00	300.00	300.00	300.00
	τ	——	——	0.277	0.032	0.033	0.553
	τa	——	——	0.245	0.784	1.079	0.489
Stirrup	Awreq, Aw	——	——	40.1<573.0	——	——	0.488<5.730

Source: JICA Study Team

PF9 (PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	6121＜11459		-591＞-7657		**11.1＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	5998＜11459		-467＞-7657		7.5＜15.0	

Member Calculation Results

In Bridge Axis Direction							
Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead load	Earthquake	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	3567.20	1383.41	33967.47	-3279.97	12330.26	2011.1
	N (kN)	——	——	11993.71	——	——	-825.8
	S (kN)	2794.72	759.42	4128.11	-973.18	-973.18	853.0
Reinforcement Bar Volume (mm^2)		Top D32×14 D32×14 Side D22×13		D29@125 D29@250	D19@250	D32@250	D32×32
Stress (N/mm^2)	σc	2.25	1.70	8.62	1.96	4.37	7.98
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**79.95**	138.09	**265.45**	195.43	**235.88**	**250.04**
	σsa	**100.00**	300.00	**300.00**	300.00	**300.00**	**300.00**
	τ	0.953	0.237	0.293	0.053	0.056	0.537
	τa	0.297	0.195	0.323	1.139	1.188	0.469
Stirrup	Awreq, Aw	1382.9 <2322.6	36.2<972.8	——	——	——	0.519<5.730
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	36577.47	-956.62	2363.82	1345.94
	N (kN)	——	——	11993.71	——	——	-702.57
	S (kN)	——	——	3828.11	-377.73	-377.73	802.98
Reinforcement Bar Volume (mm^2)		—— ——		D29@250	D16@250	D19@250	D32×32
Stress (N/mm^2)	σc	——	——	4.78	1.50	2.35	5.31
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	138.10	188.44	224.31	174.27
	σsa	——	——	300.00	300.00	300.00	300.00
	τ	——	——	0.254	0.032	0.033	0.505
	τa	——	——	0.209	0.731	0.843	0.469
Stirrup	Awreq, Aw	——	——	56.5<397.2	——	——	0.279<5.730

Source: JICA Study Team

PF10 (PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	7172＜9365		-1542＞-6567		**13.4＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	6313＜9365		-683＞-6567		8.2＜15.0	

Member Calculation Results

In Bridge Axis Direction

Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead load	Earthquake	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	3514.70	2153.41	48705.28	-8059.90	22166.83	2183.1
	N (kN)	——	——	11742.78	——	——	-1791.7
	S (kN)	2834.72	1199.42	5882.84	-6284.65	20416.34	1163.8
Reinforcement Bar Volume (mm^2)		Top D32×14 D32×14 Side D22×13		D32@125 D32@125	D29@250	D32@125	D32×36
Stress (N/mm^2)	σc	2.21	2.64	9.90	3.40	6.08	8.51
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**78.77**	214.94	**277.81**	218.62	**225.43**	**280.23**
	σsa	**100.00**	300.00	**300.00**	300.00	**300.00**	**300.00**
	τ	0.966	0.374	0.420	0.342	1.178	0.732
	τa	0.297	0.195	0.378	0.782	1.862	0.489
Stirrup	Awreq, Aw	1411.7 <2322.6	156.7<972.8	169.4<573.0	——	——	1.858<5.730

In Perpendicular Direction to Bridge Axis

Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	39835.28	-1145.02	2447.03	1475.33
	N (kN)	——	——	11742.78	——	——	-932.74
	S (kN)	——	——	4182.84	-419.70	-419.70	880.43
Reinforcement Bar Volume (mm^2)		—— ——		D32@250	D19@250	D29@250	D32×36
Stress (N/mm^2)	σc	——	——	5.40	1.51	1.71	5.40
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	185.62	156.79	105.44	178.01
	σsa	——	——	300.00	300.00	300.00	300.00
	τ	——	——	0.278	0.032	0.033	0.554
	τa	——	——	0.245	0.784	1.079	0.489
Stirrup	Awreq, Aw	——	——	40.9<573.0	——	——	0.495<5.730

Source: JICA Study Team

PF11 (PC-I Girder Bridge and Steel-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	7271＜9281		-175＞-6510		**13.5＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	6739＜9281		357＞-6510		10.5＜15.0	

Member Calculation Results

In Bridge Axis Direction							
Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead load	Earthquake	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	2895.92	787.68	23169.77	-1115.55	4545.11	2240.6
	N (kN)	——	——	10906.33	——	——	-432.3
	S (kN)	2171.92	406.57	3231.90	-517.63	-517.63	996.5
Reinforcement Bar Volume (mm^2)		Top D32×15 D32×8 Side D19×13		D25@250	D16@250	D22@250	D32×28
Stress (N/mm^2)	σc	1.60	0.67	5.57	0.85	2.81	9.64
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**75.99**	75.88	**258.67**	101.02	**227.52**	**289.39**
	σsa	**100.00**	300.00	**300.00**	300.00	**300.00**	**300.00**
	τ	0.516	0.090	0.162	0.037	0.039	0.627
	τa	0.249	0.147	0.175	0.864	0.921	0.448
Stirrup	Awreq, Aw	1026.3 <1548.4	——	——	——	——	1.365<5.730
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Dead Load	——	Earthquake
Sectional Force	M (kN/m)	——	——	23442.01	-59.02	——	1777.70
	N (kN)	——	——	10906.33	——	——	99.3
	S (kN)	——	——	3031.90	——	——	946.50
Reinforcement Bar Volume (mm^2)		—— ——		D25@250	D16@250	D16@250	D32×28
Stress (N/mm^2)	σc	——	——	1.71	0.08	——	7.67
	σca	——	——	12.00	8.00	——	12.00
	σs	——	——	15.87	9.19	——	206.64
	σsa	——	——	300.00	160.00	——	300.00
	τ	——	——	0.148	——	——	0.595
	τa	——	——	0.135	——	——	0.465
Stirrup	Awreq, Aw	——	——	20.8<397.2	——	——	0.997<5.730

Source: JICA Study Team

PF12 (Steel-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	6603＜8980		637＞-6329		12.0＜15.0	
During Earthquake in Perpendicular Direction to Bridge Axis	**7234＜8980**		6.3＞-6329		**12.2＜15.0**	

Member Calculation Results

In Bridge Axis Direction

Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead load	Earthquake	Earthquake	Earthquake	Design Load	Earthquake
Sectional Force	M (kN/m)	3405.70	666.41	16399.54	-467.78	5131.84	1798.67
	N (kN)	——	——	11063.65	——	——	380.51
	S (kN)	2568.87	360.66	2679.09	-741.47	-741.47	858.30
Reinforcement Bar Volume (mm^2)		Top D32×14 D32×14 Side D19×13		D25@250	D19@250	D32@250	D32×28
Stress (N/mm^2)	σc	2.15	0.93	6.60	0.30	2.35	7.76
	σca	8.00	12.00	12.00	8.00	8.00	12.00
	σs	**76.33**	89.23	**249.48**	29.58	**127.89**	194.78
	σsa	100.00	300.00	**300.00**	300.00	**160.00**	300.00
	τ	0.857	0.110	0.187	0.053	0.056	0.540
	τa	0.293	0.182	0.218	1.141	1.192	0.897
Stirrup	Awreq, Aw	1188.3<1548.4	——	——	——	——	——

In Perpendicular Direction to Bridge Axis

Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Dead Load	——	Earthquake
Sectional Force	M (kN/m)	——	——	26869.54	-59.02	——	2069.6
	N (kN)	——	——	11063.65	——	——	-252.2
	S (kN)	——	——	3179.09	——	——	983.3
Reinforcement Bar Volume (mm^2)		—— ——		D25@250	D16@250	D19@250	D32×28
Stress (N/mm^2)	σc	——	——	2.89	0.09	——	8.92
	σca	——	——	12.00	8.00	——	12.00
	σs	——	——	41.86	11.63	——	**259.63**
	σsa	——	——	300.00	160.00	——	**300.00**
	τ	——	——	0.211	——	——	0.619
	τa	——	——	0.144	——	——	0.448
Stirrup	Awreq, Aw	——	——	85.2<397.2	——	——	1.301<5.730

Source: JICA Study Team

PF13 (Steel-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	6410＜8808		661＞-6190		**11.6＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	**6956＜8808**		115＞-6190		11.5＜15.0	

Member Calculation Results

In Bridge Axis Direction

Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead load	Earthquake	Earthquake	Earthquake	Design Load	Earthquake
Sectional Force	M (kN/m)	3405.70	734.41	15314.20	-431.23	5000.26	1772.69
	N (kN)	——	——	10724.08	——	——	402.30
	S (kN)	2568.87	400.66	2777.22	-741.47	-741.47	882.83
Reinforcement Bar Volume (mm2)		Top D32×14 D32×14 Side D19×13		D25@250	D19@250	D32@250	D32×28
Stress (N/mm^2)	σc	2.15	1.03	6.11	0.28	2.29	7.65
	σca	8.00	12.00	12.00	12.00	8.00	12.00
	σs	**76.33**	98.34	**221.78**	27.27	**124.61**	190.58
	σsa	**100.00**	300.00	**300.00**	300.00	**160.00**	300.00
	τ	0.857	0.123	0.194	0.053	0.056	0.555
	τa	0.293	0.182	0.218	1.141	1.192	0.897
Stirrup	Awreq, Aw	1188.3<1548.4	——	——	——	——	——
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Dead Load	——	Earthquake
Sectional Force	M (kN/m)	——	——	25004.20	-59.02	——	1973.5
	N (kN)	——	——	10724.08	——	——	-143.7
	S (kN)	——	——	3177.22	——	——	982.8
Reinforcement Bar Volume (mm^2)		—— ——		D25@250	D16@250	D19@250	D32×28
Stress (N/mm^2)	σc	——	——	2.61	0.09	——	8.51
	σca	——	——	12.00	8.00	——	12.00
	σs	——	——	32.79	11.63	——	**242.52**
	σsa	——	——	300.00	160.00	——	**300.00**
	τ	——	——	0.211	——	——	0.618
	τa	——	——	0.144	——	——	0.448
Stirrup	Awreq, Aw	——	——	85.1<397.2	——	——	1.299<5.730

Source: JICA Study Team

PF14 (Steel-I Girder Bridge and PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	7275＜9151		-420＞-6456		**14.2＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	6304＜9151		551＞-6456		10.3＜15.0	

Member Calculation Results

In Bridge Axis Direction							
Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead load	Earthquake	Earthquake	Earthquake	Design Load	Earthquake
Sectional Force	M (kN/m)	2893.00	1437.55	26367.70	-2122.68	9221.99	2632.8
	N (kN)	——	——	9724.03	——	——	-727.3
	S (kN)	2165.92	777.78	4407.21	-727.48	13716.63	1325.2
Reinforcement Bar Volume (mm^2)		Top D32×15 D32×8 Side D19×13		D32@250	D29@250	D32@125	D32×36
Stress (N/mm^2)	σc	1.60	1.22	5.50	0.97	3.28	9.83
	σca	8.00	12.00	12.00	12.00	8.00	12.00
	σs	**75.91**	138.48	**249.11**	61.12	**123.44**	**279.27**
	σsa	**100.00**	300.00	**300.00**	300.00	**160.00**	**300.00**
	τ	0.515	0.172	0.221	0.052	1.039	0.834
	τa	0.249	0.147	0.203	1.169	1.760	0.489
Stirrup	Awreq, Aw	1020.1 <1548.4	29.9 <972.8	72.3<397.2	——	——	2.634<5.730
In Perpendicular Direction to Bridge Axis							
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Dead Load	——	Earthquake
Sectional Force	M (kN/m)	——	——	18970.18	-69.95	——	1887.81
	N (kN)	——	——	9724.03	——	——	244.58
	S (kN)	——	——	2907.21	——	——	950.24
Reinforcement Bar Volume (mm^2)		—— ——		D32@250	D19@250	D29@250	D32×36
Stress (N/mm^2)	σc	——	——	1.33	0.08	——	7.15
	σca	——	——	12.00	8.00	——	12.00
	σs	——	——	8.26	7.57	——	169.76
	σsa	——	——	300.00	160.00	——	300.00
	τ	——	——	0.141	——	——	0.598
	τa	——	——	0.156	——	——	0.512
Stirrup	Awreq, Aw	——	——	——	——	——	0.632<5.730

Source: JICA Study Team

PF15 (PC-I Girder Bridge)

Stability Calculation Results

Load Case	Pushing Force (kN)		Tensile Force (kN)		Displacement (mm)	
	Calculated Value	Allowable Value	Calculated Value	Allowable Value	Allowable Value	Calculated Value
During Earthquake in Bridge Axis Direction	5410＜9245		-215＞-6537		**10.7＜15.0**	
During Earthquake in Perpendicular Direction to Bridge Axis	5216＜9245		-21＞-6537		7.3＜15.0	

Member Calculation Results

In Bridge Axis Direction

Member		Beam		Column	Footing		Pile
Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		Dead load	Earthquake	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	3462.20	1908.41	26919.74	-2470.30	21400.44	1903.2
	N (kN)	——	——	10435.42	——	——	-467.0
	S (kN)	2974.72	1059.42	4920.63	-973.18	15134.94	1903.4
Reinforcement Bar Volume (mm^2)		Top D32×14 D32×14 Side D22×13		D32@125	D25@250	D29@125	D32×28
Stress (N/mm^2)	σc	2.18	2.34	7.28	1.15	6.36	8.18
	σca	8.00	12.00	12.00	12.00	12.00	12.00
	σs	**77.60**	190.49	**235.32**	84.37	**266.62**	**251.06**
	σsa	**100.00**	300.00	**300.00**	300.00	**300.00**	**300.00**
	τ	1.014	0.330	0.343	0.053	0.874	0.631
	τa	0.297	0.195	0.301	1.324	1.389	0.448
Stirrup	Awreq, Aw	1512.8 <2322.6	118.3<972.8	168.4<397.2	——	——	1.398<5.730

In Perpendicular Direction to Bridge Axis

Location		Vertical	Horizontal	Bottom	Top Surface	Bottom Surface	Under Ground
Load Case		——	——	Earthquake	Earthquake	Earthquake	Earthquake
Sectional Force	M (kN/m)	——	——	25129.74	-814.84	1898.96	1492.20
	N (kN)	——	——	10435.42	——	——	-272.39
	S (kN)	——	——	3620.63	-419.70	-419.70	786.73
Reinforcement Bar Volume (mm^2)		—— ——		D32@250	D16@250	D25@250	D32×28
Stress (N/mm^2)	σc	——	——	2.60	1.24	1.50	6.42
	σca	——	——	12.00	12.00	12.00	12.00
	σs	——	——	34.20	150.38	107.78	191.92
	σsa	——	——	300.00	300.00	300.00	300.00
	τ	——	——	0.240	0.032	0.033	0.495
	τa	——	——	0.198	0.734	0.992	0.448
Stirrup	Awreq, Aw	——	——	53.8<397.2	——	——	0.356<5.730

Source: JICA Study Team

4.6.4.4 Bridge Accessories

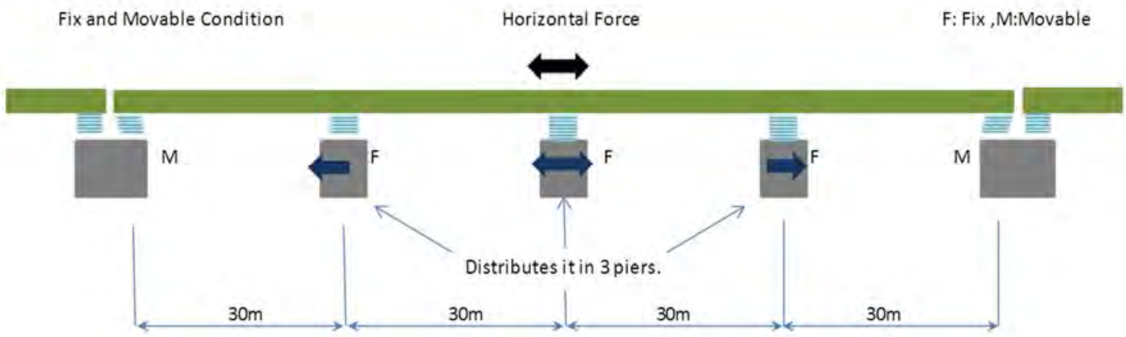
(1) Bearing Condition and Bearing

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

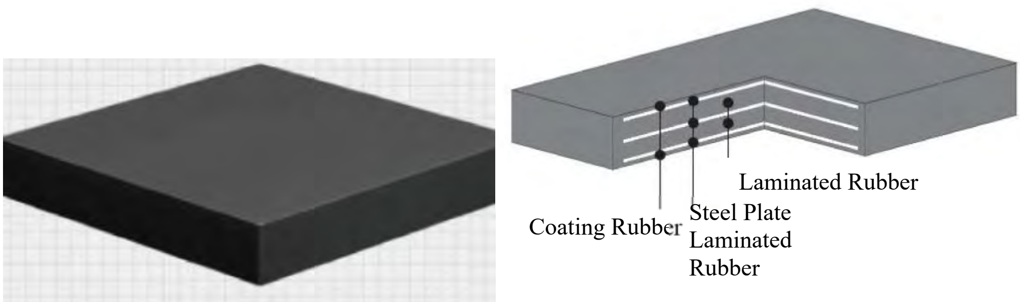
The lateral force under earthquake and/or temperature load may not be large even if the "Fixed and Moveable" support condition is applied to a three span PC-I girder bridge since PC-I girder bridge with span of 30 m is a small-scale bridge. Hence, the "Fixed and Moveable" support condition shall be applied to the PC-I girder bridges in the flyover section using an economical pad type rubber bearing as shown in Figure 4.6.56 and Figure 4.6.57.

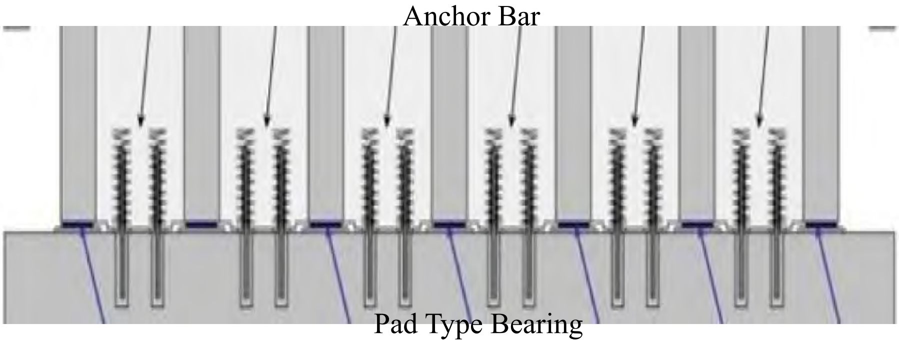


Source: JICA Study Team

Figure 4.6.56 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 4.6.57.



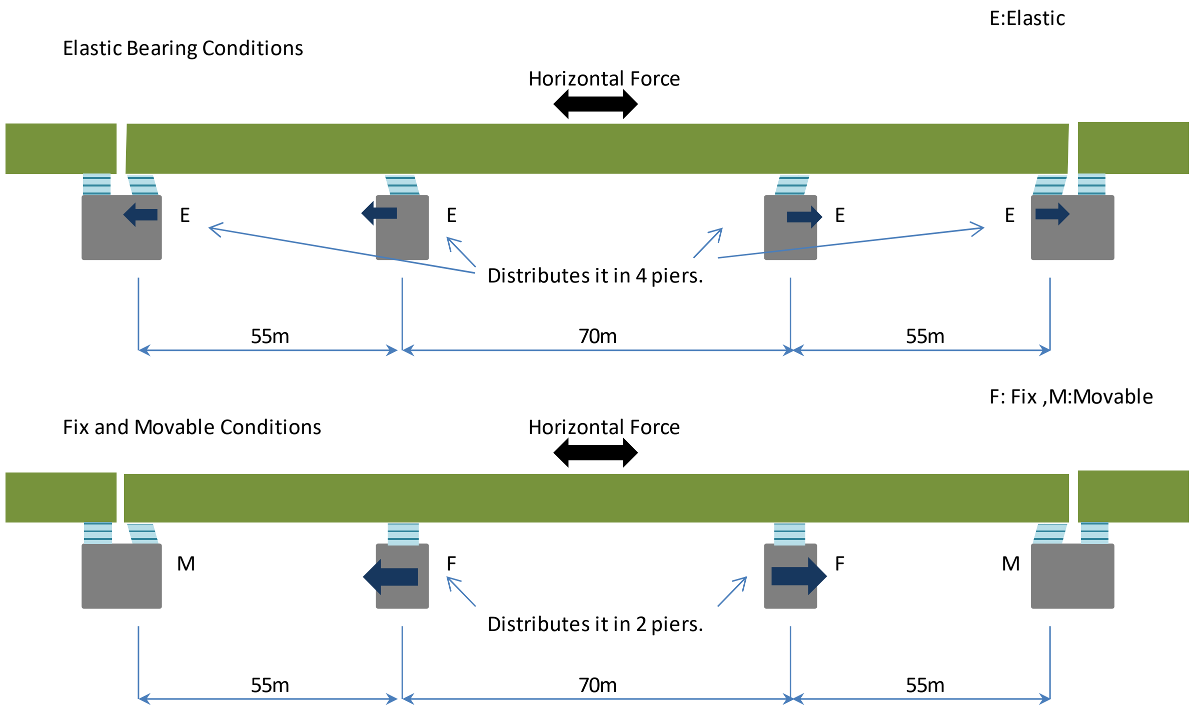


Source: JICA Study Team

Figure 4.6.57 Arrangement of Anchor Bars

2) Steel Bridge

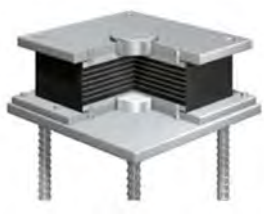
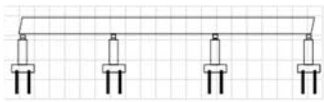
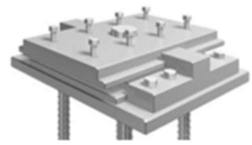
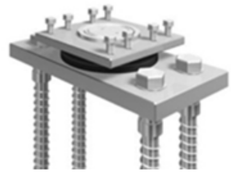
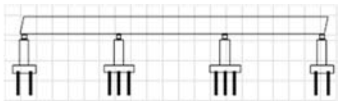
The length of the steel girder bridge (180 m) is relatively long and the maximum span is 70 m. As shown in Figure 4.6.58, in case of elastic bearing condition, the lateral force is shared by four piers. On the other hand, in case of the "Fixed and Moveable" bearing condition, the lateral force is shared 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 4.6.58 Distribution of Horizontal Force

Table 4.6.145 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

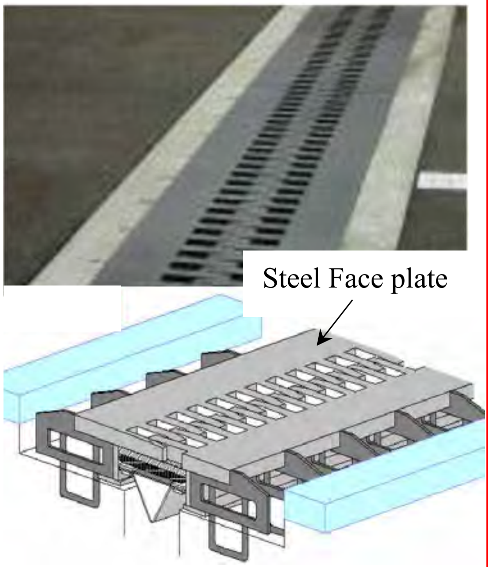
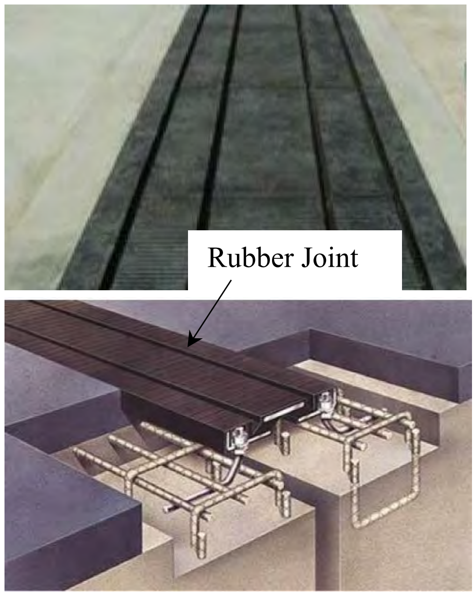
(2) Expansion Joint

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 4.6.146 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.

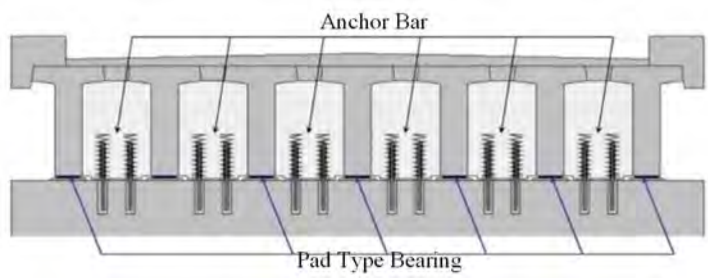
The possibility of the unseating of the superstructure from substructures during an earthquake is quite low if the superstructure is supported by four or more substructures as specified in the JSHB. On the other hand, an unseating prevention system should be installed since the possibility of unseating 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 4.6.147.

As shown in Figure 4.6.59, the unseating prevention system by anchor bars shall be applied to the two span PC-I girder bridges in flyover sections.

Table 4.6.147 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



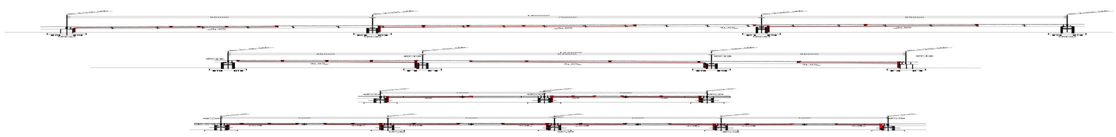
Source: JICA Study Team

Figure 4.6.59 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 4.6.60 to Figure 4.6.62.

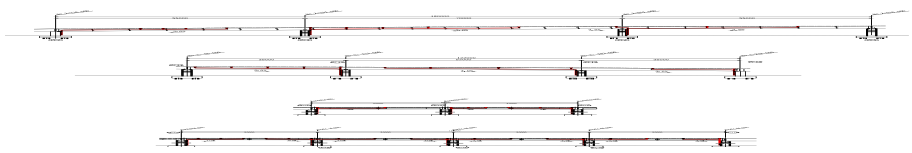
- Steel Box Girder Bridge



Source: JICA Study Team

Figure 4.6.60 Drainage Distribution Diagram of Steel Box Girder Bridge

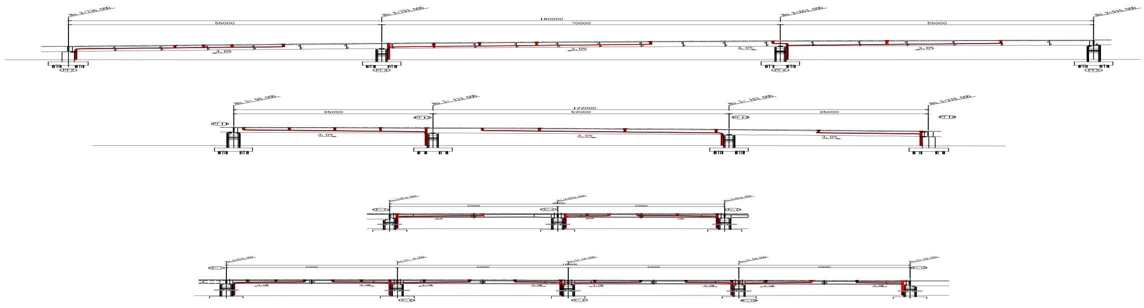
- Steel I-section Girder Bridge



Source: JICA Study Team

Figure 4.6.61 Drainage Distribution Diagram of Steel-I Girder Bridge

- PC-I Girder Bridge



Source: JICA Study Team

Figure 4.6.62 Drainage Distribution Diagram of PC-I Girder Bridge