4.2 STUDY ON CABLE-STAYED BRIDGE

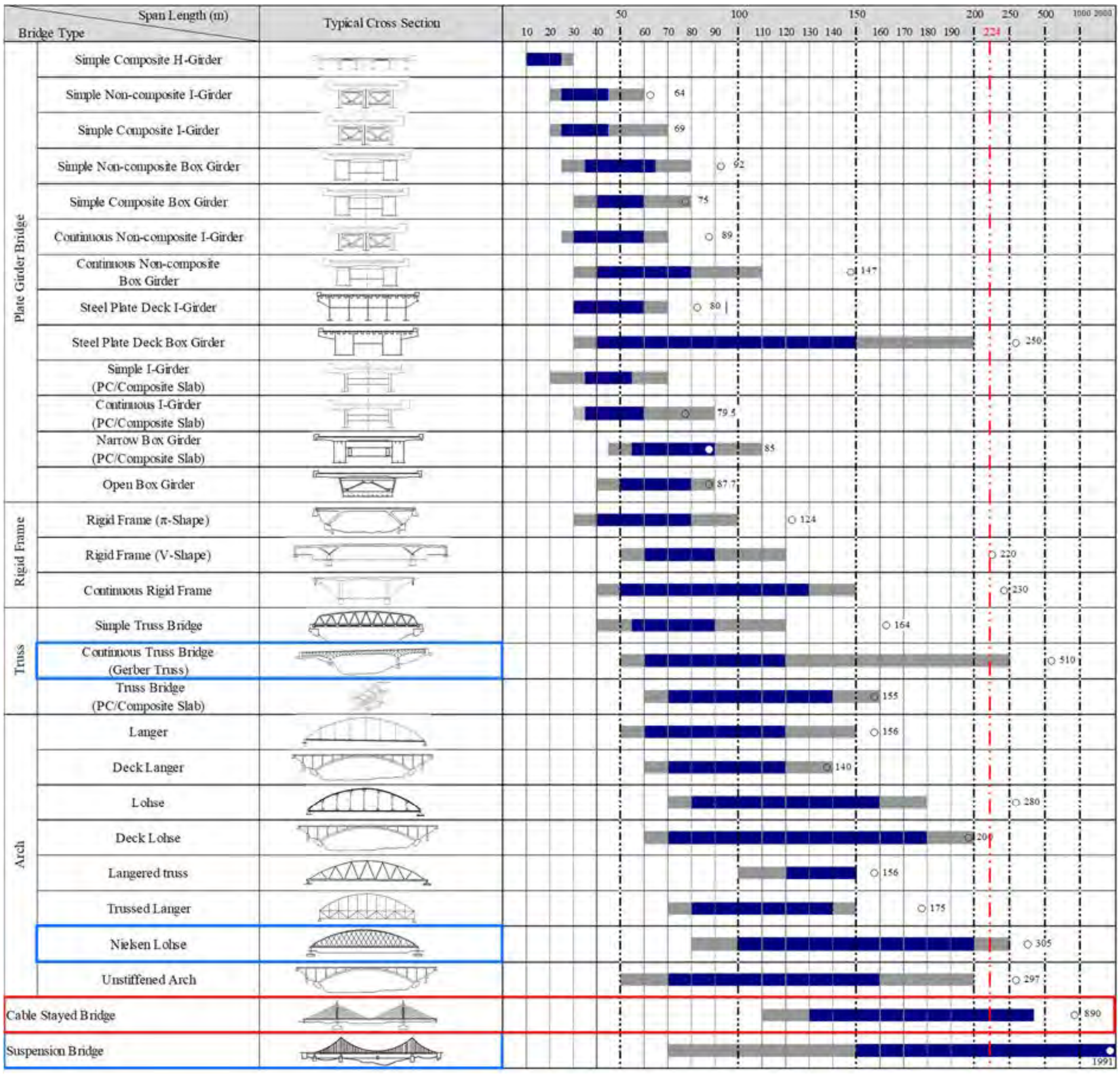
[Basic Design Stage]

4.2.1 Selection of Type of Cable-stayed Bridge

4.2.1.1 Review of the F/S Design

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

Table 4.2.1 Applicable Span of Steel Bridge



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

Source: JICA Study Team

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

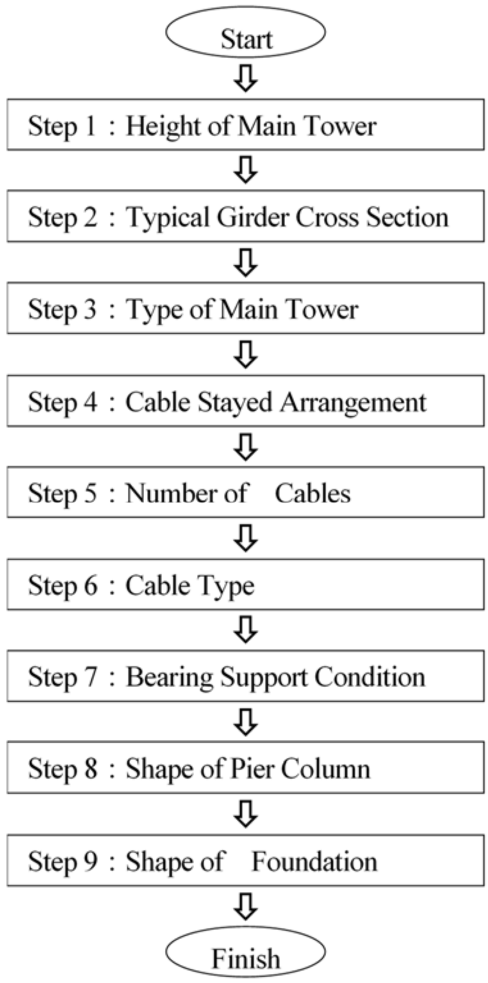
1) Continuous Truss Bridge (Gerber Truss): Usually, continuous truss bridge is applied for around 100 m span length. In case of more than 100 m span, Gerber Truss will be applied, but it is not good for maintenance and construction cost will become expensive.

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

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

4.2.1.2 Flow Chart of Basic Design for Cable-stayed Bridge

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



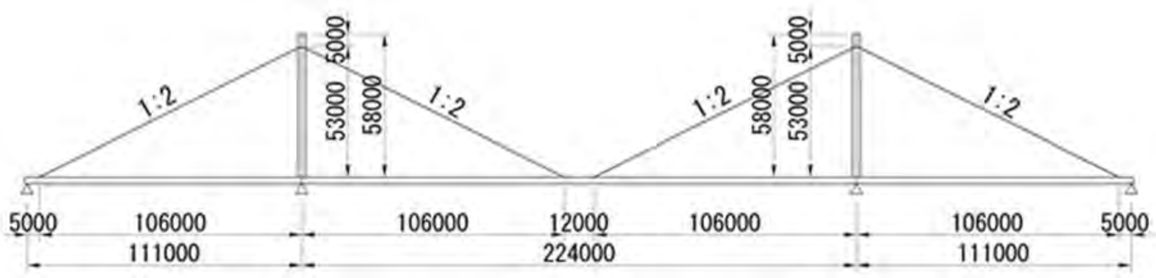
Source: JICA Study Team

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

4.2.2 Superstructure of Cable-stayed Bridge

4.2.2.1 Height of the Main Tower

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



Source: JICA Study Team

Figure 4.2.2 Gradient of Top Cable

4.2.2.2 Typical Girder Cross Section

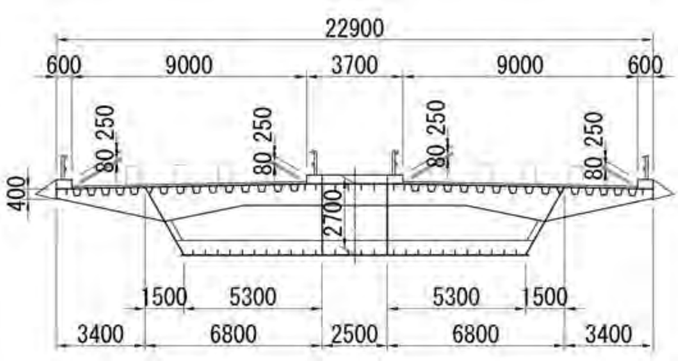
(1) Typical Girder Cross Section

For the typical girder cross section, three types of cross section (Wide Box Cross Section, Conventional Box Cross Section, Narrow Box Cross Section) were compared. Based on the comparison results, <u>"Case-2: Conventional Box Cross Section"</u> was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

The most appropriate girder height for steel cable-stayed bridge is h=2.5 m~2.8 m. In this Project, considering the main girder anchorage for stay cable, working space for cable installation, and economy, the girder height was decided as h=2.7 m. Furthermore, PC box girder and steel box girder have the same girder height and landscape direction in the river bridge.

Case-1 Wide Box Cross Section

Girder type	3-Cell Box Girder
Length of Overhang	3400mm
Girder height	2.7m



Source: JICA Study Team

Figure 4.2.3 Wide Box Cross Section

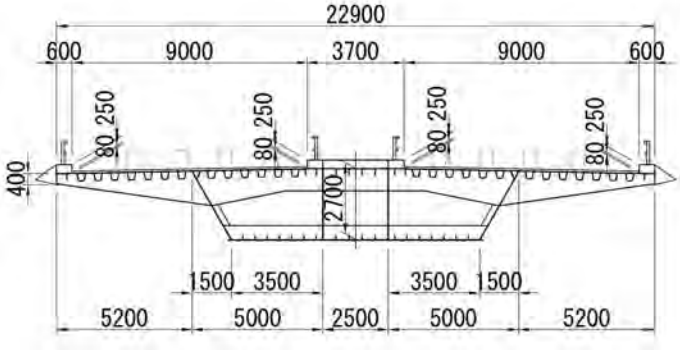
Characteristics:

- Web is not located under the wheel load; therefore, this type is good for fatigue resistance.
- Painting area is smaller than in the other types; therefore, this type is superior in maintenance.

- Width of bottom flange is wide, and width of pier head will become wider.
- Due to the wide box cross section, the number of parts of the girder will be increased, and thus increasing assembly time at the site.

Case-2 Conventional Box Cross Section **< Recommended >**

Girder type	3-Cell Box Girder
Length of Overhang	5200mm
Girder height	2.7m



Source: JICA Study Team

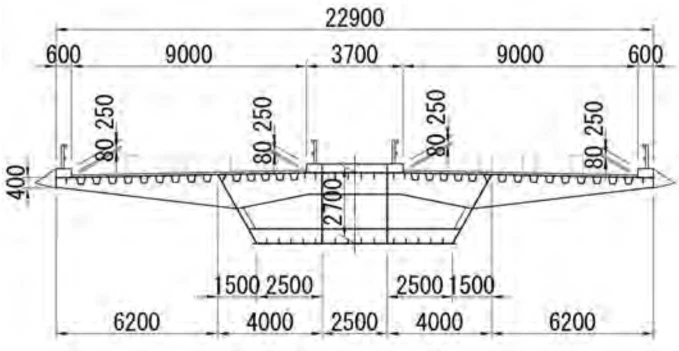
Figure 4.2.4 Conventional Box Cross Section

Characteristics:

- Web is not located under the wheel load; therefore, this type is good for fatigue resistance.
- Overhang length is not so large; therefore, this type has a good balance for fatigue resistance.
- Steel weight is lower than the other types; therefore, this cross section is the most economical.

Case – 3 Narrow Box Cross Section

Girder type	3-Cell Box Girder
Length of Overhang	6200mm
Girder height	2.7m



Source: JICA Study Team

Figure 4.2.5 Narrow Box Cross Section

Characteristics:

- Web is located under the wheel load; therefore, this type is not good for fatigue resistance.
- Overhang length is large; therefore, this type is not good in terms of deflection and fatigue resistance.
- Width of bottom flange is narrow, and width of pier head will become narrower.
- Because of the narrow box section, torsion and bending rigidity are small.

Table 4.2.2 Comparison of Steel Weight and Evaluation Results

Type		CASE - 1 Wide Box Cross Section	CASE - 2 Conventional Box Cross Section	CASE - 3 Narrow Box Cross Section
Steel Weight (t)	Girder	4,660	4,600	4,630
	Tower	680	680	680
	Cable	260	250	260
	Total	5,600	5,530	5,570
Total Cost Ratio		1.01	1.00	1.01
Evaluation			○	

Source: JICA Study Team

(2) Type of Rib for Slab

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

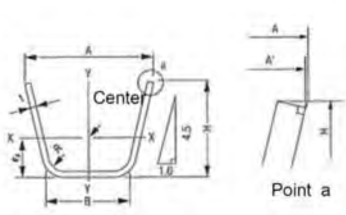
[Study Objective]

The objective is to select the most optimum steel deck longitudinal rib shape for this bridge.

[Study Conditions]

1) The span length of the longitudinal rib is 2.5 m (maximum spacing of transverse ribs is based on the “Fatigue Design Guidelines for Steel Road Bridge”).
2) The ribs to be used in this study are Flat Rib (Open Section Rib) and U Rib (Closed Section Rib).
3) For the longitudinal ribs in the inner cells, Flat Rib will be used (Cables will be placed there so it will become a complex location.)
4) When using closed longitudinal ribs (U Rib), the thickness of the deck plate shall be at least 16 mm under the position where the wheel load of the large cars will be loaded. (Specifications for Highway and Bridges, Part II Steel Bridge (April 2012), from p. 295)
5) Minimum thickness of longitudinal ribs is 8 mm. (Specifications for Highway and Bridges, Part II Steel Bridge, (April 2012), from p. 296)

   (Considering a high humidity climate leading to a corrosion environment, the plate thickness was selected as 8 mm.)



(2) U Shape Rib (JSS Ⅱ 08-1983)
Standard Length: 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0 m

Type	Dimensiones mm						Area cm2	Unit Weight kg/m	Section Performance		Expansion of cross section area/ t mm
	A	A'	B	H	t	R			Center of Gravity ex cm	Moment of Inertia Ix cm4	
320 X 240 X 6-40	320	319.4	213.3	240	6	40	40.26	31.6	8.86	2460	671
320 X 260 X 6-40	320	319.4	204.4	260	6	40	42.19	33.1	9.91	3011	703
320 X 240 X 8-40	324.1	323.3	216.5	242	8	40	53.9	42.3	8.99	3315	674
320 X 260 X 8-40	324.1	323.3	207.7	262	8	40	56.47	44.3	10.03	4055	706

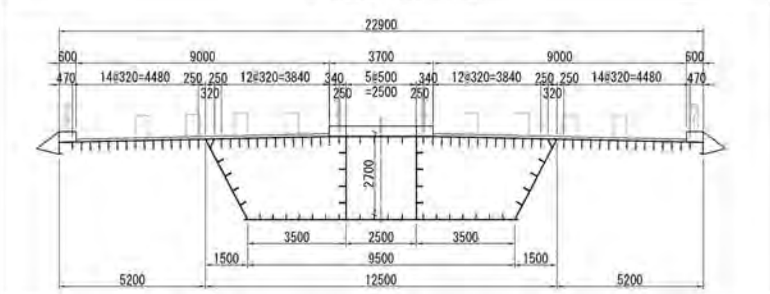
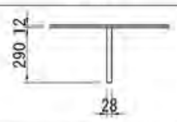
Caution:
1)A' or Section performance , Expansion cross section area referance value are shown.
2)Except for SM400A and SM490YA,materials specification should be asked to the manufacturer.
3) In the U-shaped Ribs used for the steel deck, cold working can be carried if the inner radius is 5 times or more then the plate thickness. (Road bridge specification,1996)

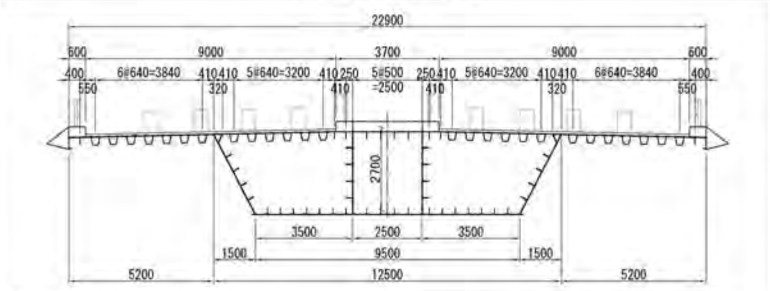
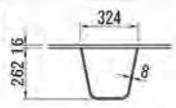
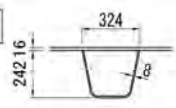
Source: 2016 Design Data Book

Figure 4.2.6 Size of U Rib

[Study Results]

Table 4.2.3 Comparison of Flat Rib and U Rib

Lonfitudinal Rib Shape		Flat Rib (Open Section Rib)
Longitudinal Rib Arrangement		
Longitudinal Rib Cross section		CASE-1 
Longi. Rib Cross section		Flat Rib 220×21
Steel Deck Thickness		t=12mm
Material		SM490Y
Steel Deck Thickness	σc(Com.)	σ=96N/mm² ≦σa=210N/mm² OK
	σt(Ten.)	σ=194N/mm² ≦σa=210N/mm² OK
	τ(Shear)	τ=9N/mm² ≦τa=120N/mm² OK
	Com. stress	κ=0.86 ≦1.2 OK
Steel Deck Weight		Steel Deck 22.900× 0.012× 447.600× 7.850× 1 = 965.6 t Flat Rib① 0.220× 0.021× 447.600× 7.850× 56 = 909.1 t Flat Rib② 0.210× 0.020× 447.600× 7.850× 8 = 118.1 t (Assumption) Total 1992.8 t
Steel Deck Weight Ratio		1.03
Evaluation		

Lonfitudinal Rib Shape		U Rib (Closed Section Rib)	
Longitudinal Rib Arrangement			
Longitudinal Rib Cross section		CASE-2 	CASE-3 
Longi. Rib Cross section		U Rib 320×260×8-40	U Rib 320×240×8-40
Steel Deck Thickness		t=16mm	t=16mm
Material		SM400	SM400
Steel Deck Thickness	σc(Com.)	σ=32N/mm² ≦σa=140N/mm² OK	σ=34N/mm² ≦σa=140N/mm² OK
	σt(Ten.)	σ=74N/mm² ≦σa=140N/mm² OK	σ=78N/mm² ≦σa=140N/mm² OK
	τ(Shear)	τ=12N/mm² ≦τa=80N/mm² OK	τ=13N/mm² ≦τa=80N/mm² OK
	Com. stress	κ=0.30 ≦1.2 OK	κ=0.34 ≦1.2 OK
Steel Deck Weight	Type	・Type 320×260×8-40	・Type 320×240×8-40
	Steel Deck	22.900× 0.016× 447.600× 7.850× 1 =1287.4t	22.900× 0.016× 447.600× 7.850× 1 =1287.4t
	U Rib	44.300× 1000× 447.600× 26 =515.5t	42.300× 1000× 447.600× 26 =492.3t
	Flat Rib	0.210× 0.020× 447.600×7.850×10 =147.6t (Assumption)	0.210× 0.020× 447.600×7.850×10 =147.6t (Assumption)
	Total	=1950.5t	=1927.3t
Steel Deck Weight Ratio		1.01	1.00
Evaluation			○

Source: JICA Study Team

(3) Height of Bracket

The height of the bracket at the cable-stayed bridge was changed from 1.2 m to 1.6 m, and the bracket weight was compared. Based on the comparison results, <u>"Case-2: Bracket Height = 1.3 m"</u> was selected as the best type. Comparison results are shown as follows:

<u>[Study Conditions]</u>

The minimum thickness of the bracket shall be the thickness wherein longitudinal stiffeners will not be required. (Case-1 is determined by horizontal shear stress degree)

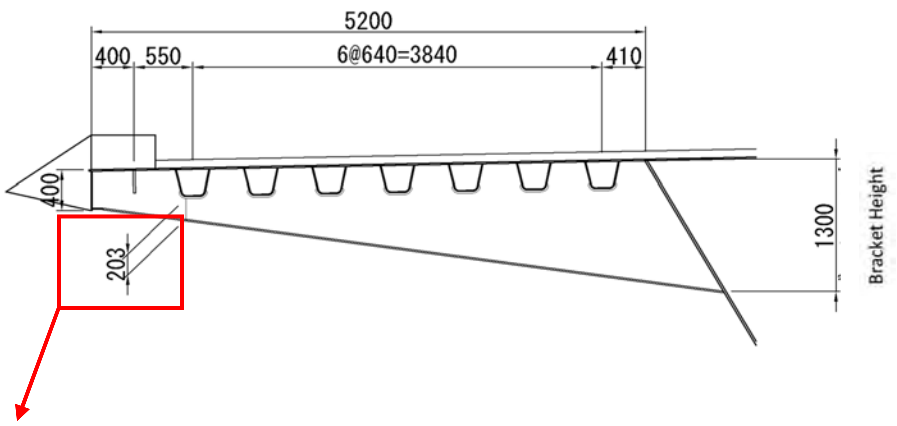
<u>[Study Results]</u>

Table 4.2.4 Comparison of Bracket Height

	Bracket Cross-Section					Stress (N/mm2)					Bracket Weight		Evaluation
	Height	Plate Thicknes	Bracket width	Flange Thicknes	Material Type	σ	σa	τv※	τh※	τa	Weight (t)	Ratio	
CASE-1	1200	12	370	15	SM490Y	142	159	66	105	120	276.7	1.009	
CASE-2	1300	11	370	15	SM490Y	131	159	65	105	120	274.3	1.000	○
CASE-3	1400	12	370	14	SM490Y	118	141	54	90	120	293.5	1.070	
CASE-4	1500	13	370	13	SM490Y	107	122	46	77	120	314.8	1.148	
CASE-5	1600	14	370	12	SM490Y	97	105	40	67	120	337.7	1.231	

※τv (Vertical Shear Stress)、τh (Horizontal Shear Stress)

Source: JICA Study Team



* Check the shear stress in the tip of the U Rib scallop point.

Shear Stress S = 60 kN

Cross Section Area A = 203 × 11 = 2233 mm^2

τ = 60 × 1000/2233 = 26.9 N/mm^2 < τa = 120 N/mm^2 <OK>

Source: JICA Study Team

Figure 4.2.7 Shape of Bracket

(4) Block Width

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

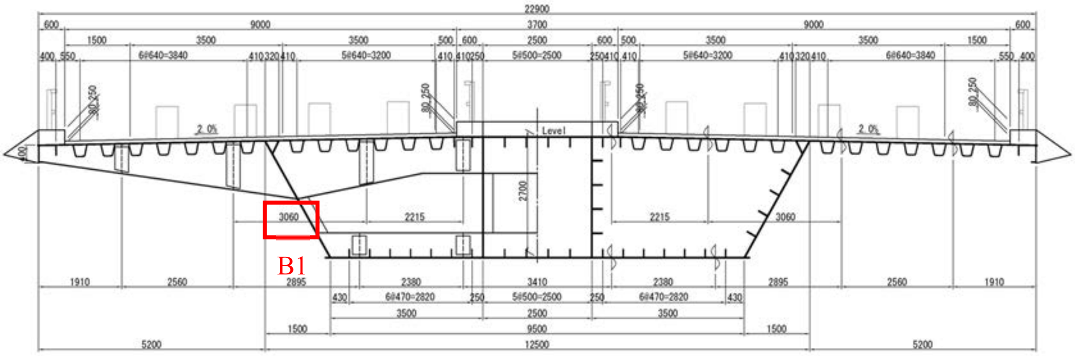
The block width in the transverse direction was studied. Based on the comparison results, "Case-1: Block Maximum Width = 3.06 m" was selected as the best type.

[Study Conditions]

1) Maximum transportable width is 3.5 m.

2) It is preferable that the deck plate's bridge axis direction joint does not fall directly under the wheel loading point. (From "Fatigue Design Guidelines for Steel Road Bridge, Japan Road Association, 2002" p. 46)

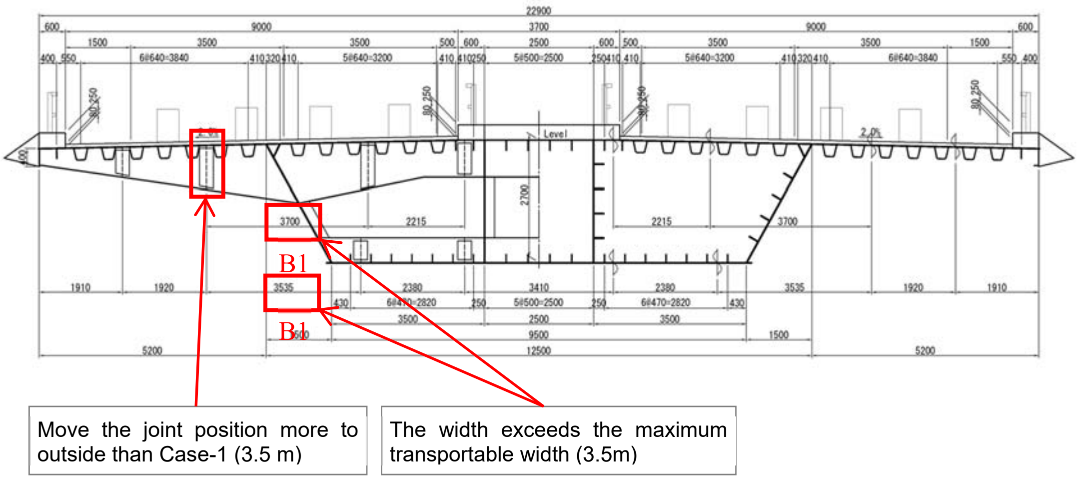
Case - 1 Block Maximum Width = 3.06 m **< Recommended >**



Observations: This case was selected as the most optimum.
Source: JICA Study Team

Figure 4.2.8 Block Maximum Width = 3.06 m

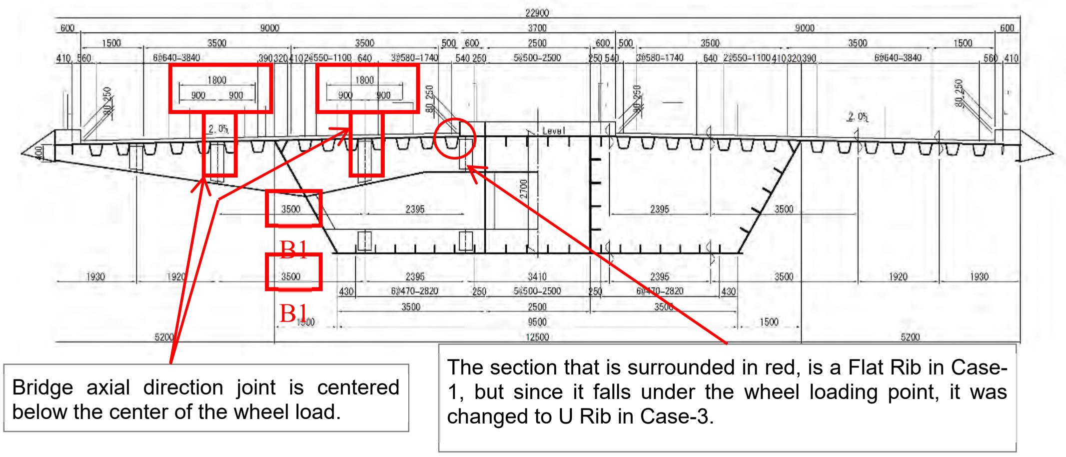
Case - 2 Block Maximum Width = 3.7 m



Observations: The width exceeds the maximum transportable width.
Source: JICA Study Team

Figure 4.2.9 Block Maximum Width = 3.7 m

Case - 3　Block Maximum Width = 3.5 m



Observations: Compared to Case-1, longitudinal U Rib increased by 2 and the longitudinal Flat Rib decreased by 2.

Source: JICA Study Team

Figure 4.2.10　Block Maximum Width = 3.5 m

[Study Results]

Table 4.2.5　Comparison of Block Maximum Width

	Block Maximum Width B1 (m)	No. Longitudinal Rib		Steel Deck Weight (t)	Steel Weight Ratio	Evaluation
		U Rib	Flat Rib			
CASE-1	3.060	26	12	2587	1.000	○
CASE-2	3.700	26	12	2587	1.000	
CASE-3	3.500	28	10	2598	1.004	

Source: JICA Study Team

(5) Diaphragm Plate Thickness

The plate thickness of intermediate diaphragm was studied. As a result, <u>"Diaphragm plate thickness = 9 mm"</u> was enough for the outer cell and inner cell.

<u>[Study Conditions]</u>

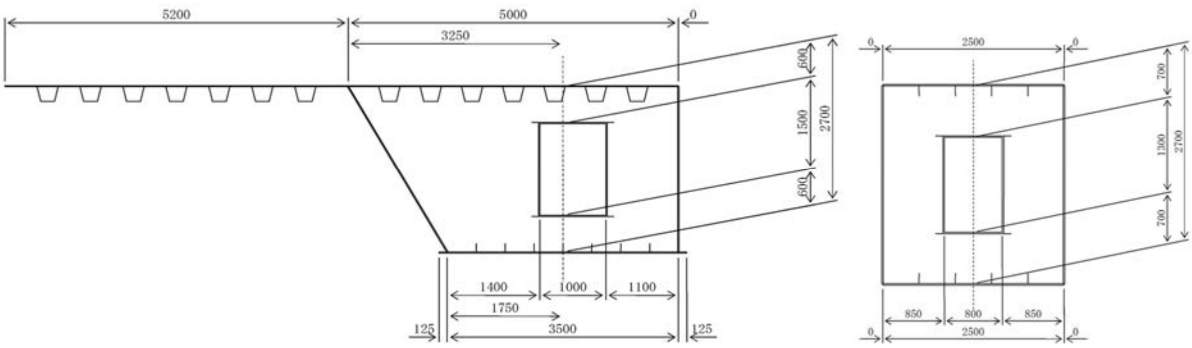
1) Intermediate diaphragm plate thickness was studied, and the method was based on the Steel Highway Bridge Design Handbook (Japan Road Association).
2) Each cell was verified if each maintains its required rigidity

<u>[Study Results]</u>

Table 4.2.6 Results of Diaphragm Plate Thickness

	Diaphragm Thickness	Required Rigidity (N・mm) ①		Diaphragm Rigidity (N・mm) ②	①÷②
Outer Cell	9mm	5.20E+09	＜	3.18E+10	0.16
Inner cell	9mm	1.50E+08	＜	1.87E+10	0.01

Source: JICA Study Team



Verified outer cell cross section Verified inner cell cross section

Source: JICA Study Team

Figure 4.2.11 Cross Section of Diaphragm

4.2.2.3 Types of Main Tower

(1) Comparison of Main Tower Types

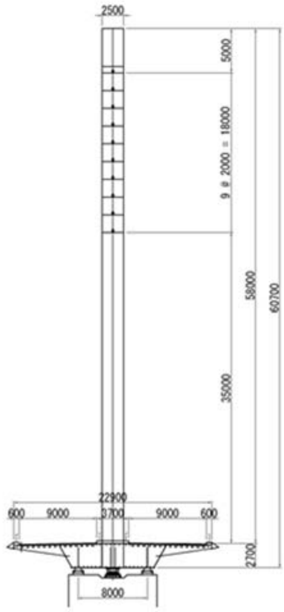
Three types of main tower (Single Tower, A-Shape Tower, Twin Tower) were compared. Based on the comparison results, <u>"Case-1: Single Tower"</u> was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case - 1 <u>Single Tower</u> **< Recommended >**

Girder type	3-Cell Box Girder
Girder Width	22.9m

Characteristics:

- Width of pier is smaller than in the other types.
- Due to the position of the main tower, the median strip is wider.
- This type has one straight pylon, and it is a simpler structure than the others.



Source: JICA Study Team

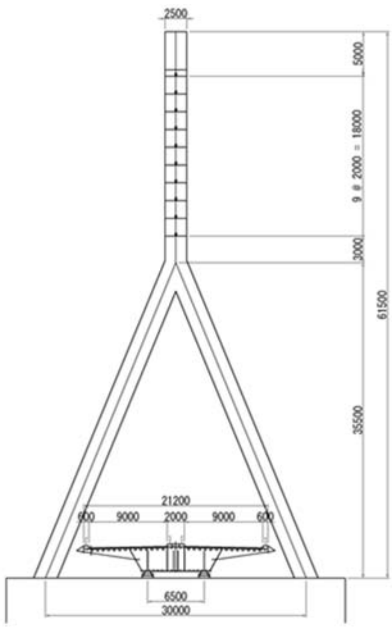
Figure 4.2.12 Single Tower

Case - 2 <u>A-Shape Tower</u>

Girder type	3-Cell Box Girder
Girder Width	21.2m

Characteristics:

- Median strip can be narrowed compared to Case-1.
- Column of main tower will be located at both sides of the girder; therefore, pier width will be wider.



Source: JICA Study Team

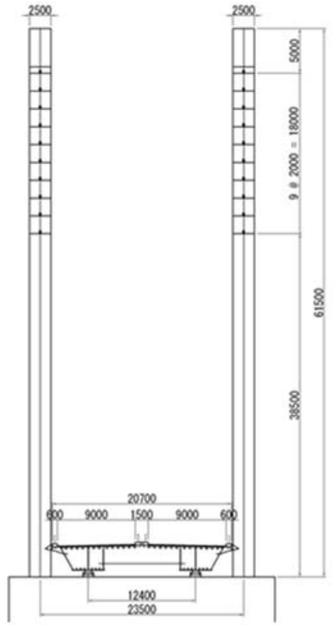
Figure 4.2.13 A-Shape Tower

Case - 3 Twin Tower

Girder type	2 Box Girders
Girder Width	20.7m

Characteristics:

- Cables are stayed at both ends of the girder cross section and median strip can be narrowed; therefore, width of median strip can fit into the next bridges.
- Column of main tower will be located at both sides; therefore, pier width will be wider compared to Case-1.



Source: JICA Study Team

Figure 4.2.14 Twin Tower

Table 4.2.7 Comparison of Tower Type

Type		CASE - 1 Single Tower	CASE - 2 A-Shape Tower	CASE - 3 Twin Tower
Steel Weight (t)	Girder	4,600	4,310	4,450
	Tower	680	1,090	1,060
	Cable	250	250	280
	Total	5,530	5,650	5,790
Total Cost Ratio		1.00	1.23	1.17
Evaluation		○		

** This total cost ratio includes cost of superstructure and substructure.*
Source: JICA Study Team

(2) Pylon Width

The pylon width affects the median width and the main girder width, so it is necessary to study in advance. The basic cross section of the pylon can be changed in the longitudinal direction to some point and can be adjusted by the plate thickness. Therefore, in very few cases, the cross section of the pylon is decided by the section force. In case anchors are placed in the tower, sufficient space for maintenance should be considered in the pylon width.

In case the saddle anchors are being excluded, and in order to compact the anchors, pylon cross section will become a three-cell structure. In that case, a study to evaluate whether there is enough space for maintenance or not must be done.

In case saddle type is selected, pylon cross section will be a one-cell structure. However, depending on the timing of insertion of the saddle, a space (almost the same width as the saddle) is needed at both sides of the saddle, so it is considered that the pylon width will be the same size.

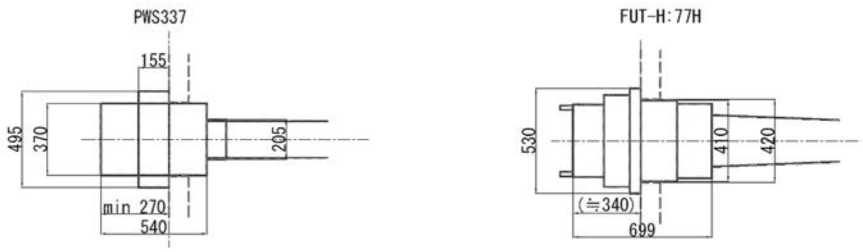
Therefore, the study of pylon width is based on the case of an anchor girder type as a typical anchor structure. The basic cross section to be studied is 2.5 m (width) x 3.0 m.

The cable cross section was estimated through the following size:

In case of PWS : Hiam337 (Tensile strength Tu = 20400 kN)

In case of PC steel wire SEE (FUT-H) : 77H (Tensile strength Tu = 20097 kN)

The following shows the figure of the socket in the case of the tower side anchor being fixed.



Source: JICA Study Team

Figure 4.2.15 Figure of Cable Socket

From the figure, The FUT-H type has a bigger anchor part, so based on the FUT-H:77H, the positioning of the anchor on the pylon section was studied. The figure of the socket being placed in the inner tower is shown in Section 4.2.2.6, (2) Anchor Study.

- The anchor section needs b = 900 mm in the transverse direction.
- Manhole pathway for maintenance is needed in the outer side of the inner cell. The minimum width is as follows:

  b1 = 50 (Vertical plate thickness) + 500 (Manhole ladder width) +

  200 (Longitudinal Rib or Transverse Rib minimum height) = 750 mm
- Minimum width of Pylon, B is

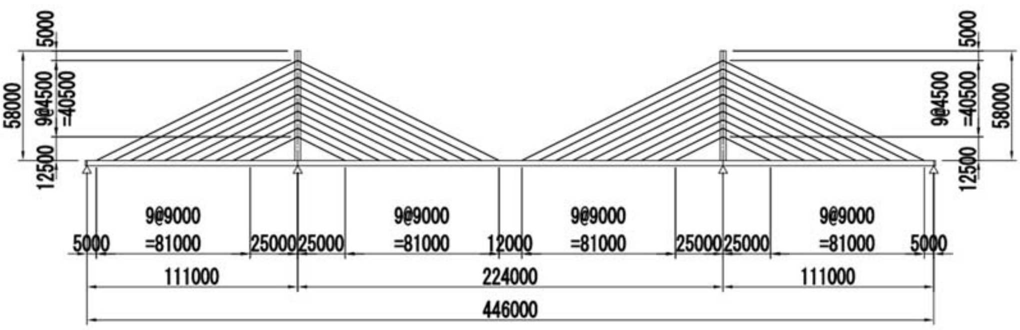
  B = 2 x 750 + 900 = 2400 mm

This minimum width is considered as almost the limit when the anchor structure is selected as the girder anchor type. If possible, it is desirable to have more space; therefore, slight margin on both sides was kept and width of the pylon was set to 2.5 m in the Project.

4.2.2.4 Cable-stayed Arrangement

Three types of cable-stayed arrangement (Harp Arrangement, Fan Arrangement, Semi Fan Arrangement) were compared. Based on the comparison results, "Case-3: Semi Fan Arrangement" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows.:

Case - 1 Harp Arrangement



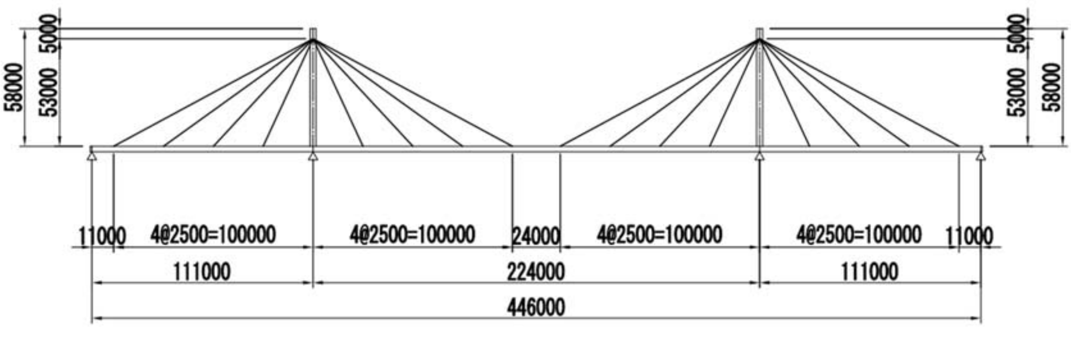
Source: JICA Study Team

Figure 4.2.16 Harp Arrangement

Characteristics:

- Each cable is stayed parallel in the whole part of the pylon.
- From the structural point of view, the lower cables are not so efficient due to the gradient.
- From the aesthetic point of view, it has a good appearance and it is more attractive than the other arrangement types.

Case - 2 Fan Arrangement



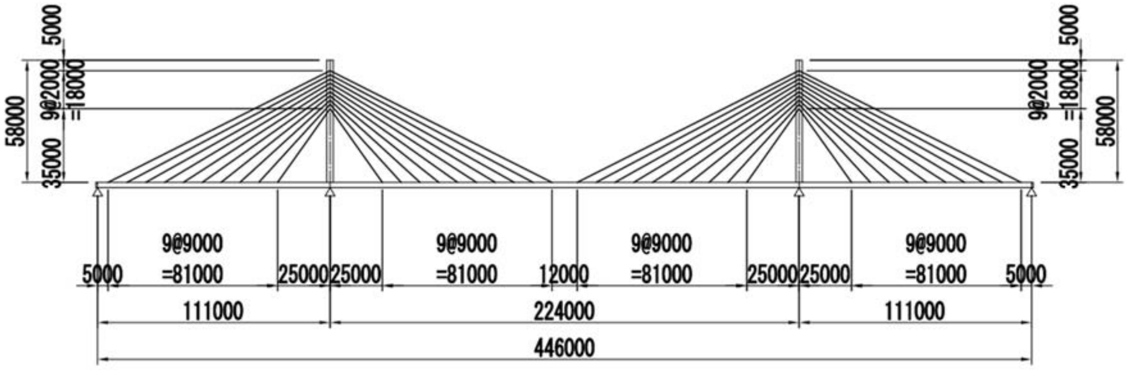
Source: JICA Study Team

Figure 4.2.17 Fan Arrangement

Characteristics:

- All cables are attached to a single point at the top of the pylon, making it difficult to attach the cables to one point; therefore, this type is not applied in long span cable-stayed bridges.
- From the structural point of view, the cables are working efficently due to the high gradient.
- From the aesthetic point of view, since all the cables are attached to the top, they do not give a great apperance if compared to the others.

Case – 3 Semi Fan Arrangement **< Recommended >**



Source: JICA Study Team

Figure 4.2.18 Semi Fan Arrangement

Characteristics:

- The cables are distributed over the upper part of the pylon, which are more steeply inclined close to the pylon.
- From the structural point of view, gradient of the lower cables is bigger than in the Harp Arrangement; therefore, structural efficiency is higher.
- From the aesthetic point of view, cables are arranged in a single plane, giving also a good appearance.

Table 4.2.8 Comparison of Cable Arrangement Types

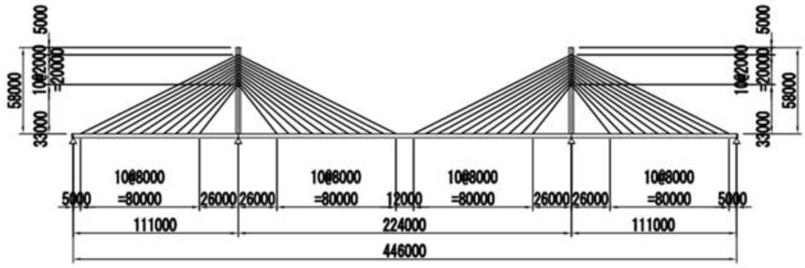
Type		CASE - 1 Harp Arrangement	CASE - 2 Fan Arrangement	CASE - 3 Semi Fan arrangement
Steel Weight (t)	Girder	4,840	4,640	4,600
	Tower	700	700	680
	Cable	260	250	250
	Total	5,800	5,590	5,530
Total Cost Ratio		1.05	1.01	1.00
Evaluation				○

Source: JICA Study Team

4.2.2.5 Number of Cables

Three types of the number of cables at the left (right) side of the pylon (11 Cables, 10 Cables, 9 Cables) were compared. Based on the comparison results, "Case-2: 10 Cables (Total: 40 Cables)" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case – 1 11 Cables



Cable spacing (Tower)	5.0+10@2.0+33.0
Cable spacing (Girder)	(P12)5.0+10*8.0+26.0(P11)+26.0+10*8.0+6.0(CL)

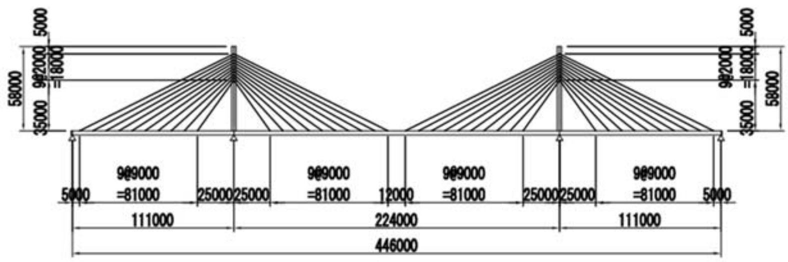
Source: JICA Study Team

Figure 4.2.19 11 Cables (Total: 44 Cables)

Characteristics:

- Increasing the number of cables will also increase significantly the time of erection.
- For each cable installed, the stiffening girder and the pylon need to be strengthened locally in order to be able to receive the stayed forces. Therefore, increasing the cables will also increase the parts that require local strengthening.

Case - 2 10 Cables **< Recommended >**



Cable spacing (Tower)	5.0+9@2.0+35.0
Cable spacing (Girder)	(P12)5.0+9*9.0+25.0(P11)+25.0+9*9.0+6.0(CL)

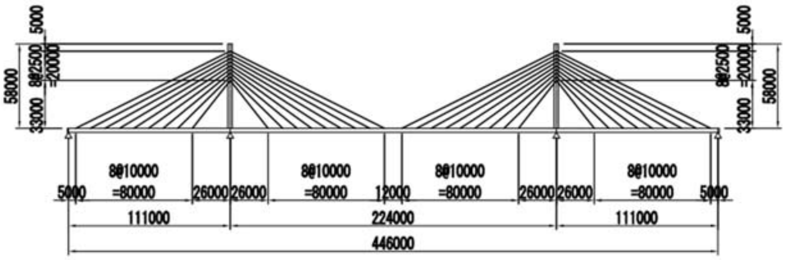
Source: JICA Study Team

Figure 4.2.20 10 Cables (Total: 40 Cables)

Characteristics:

- Its time of erection is faster compared to the erection time of 11 cables.
- This cable arrangement is more economical, due to the lower weight of the total cables compared to the 9 cables arrangement.

Case – 3 9 Cables



Cable spacing (Tower)	5.0+8@2.5+33.0
Cable spacing (Girder)	(P12)5.0+8*10.0+26.0(P11)+26.0+8*10.0+6.0(CL)

Source: JICA Study Team

Figure 4.2.21 9 Cables (Total: 36 Cables)

Characteristics:

- The lower the number of cables, the heavier the girder gets due to the rise of the momentum forces.
- Due to the increase of space between cables, the length and weight of the blocks increase, making it necessary for a bigger crane for the erection work.

Table 4.2.9 Comparison of Number of Cables

Type		CASE - 1 11 Cables	CASE - 2 10 Cables	CASE - 3 9 Cables
Steel Weight (t)	Girder	4,630	4,600	4,670
	Tower	680	680	680
	Cable	260	250	260
	Total	5,570	5,530	5,610
Total Cost Ratio		1.01	1.00	1.02
Evaluation			○	

Source: JICA Study Team

4.2.2.6 Cable Type

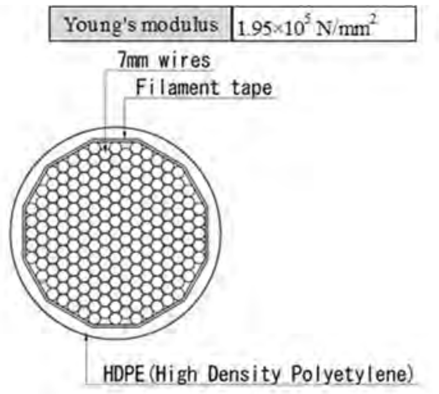
(1) Comparison of Types of Cables

Three cable types (New Parallel Wire Strand Type, FUT-H Strand Type, Locked Coil Type) were compared. Based on the comparison results, "Case-2: FUT-H Strand Cable" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case - 1 New Parallel Wire Strand (NPWS)

Characteristics:

- NPWS cables are prefabricated at the factory, reducing the erection time of the cables on site.
- NPWS cables are coated with polyethylene and zinc plating, making it resistant against corrosion.
- In order to install this type of cable it is necessary to have heavy machinery, such as big cranes and jacks.



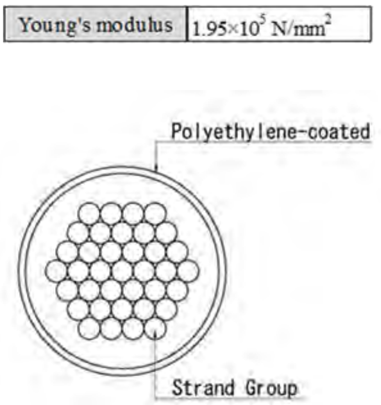
Source: JICA Study Team

Figure 4.2.22 NPWS Cable

Case - 2 FUT-H Strand Cables **< Recommended >**

Characteristics:

- FUT-H cables are formed on site by tensioning each stranded wire one by one, slightly increasing the time of erection.
- FUT-H is covered by three types of materials (zinc plating, grease, polyethylene coating) and additional polyethylene pipe, protecting it from corrosion.
- During installation, it is necessary to have a small crane, jacks, and other small machines, avoiding the use of big cranes and vast loads on the girder during erection.



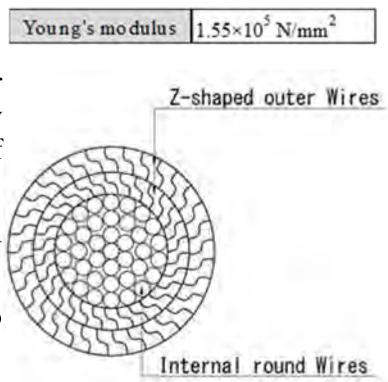
Source: JICA Study Team

Figure 4.2.23 FUT-H Strand Cable

Case - 3 Locked Coil Rope (LCR)

Characteristics:

- LCR cables are prefabricated at the factory, but in order to apply this type of cable in this design, it is necessary to have three cables per section, increasing the time of erection and fabrication.
- LCR cables have poor resistance against corrosion compared to the other types of cables.
- In order to install this type of cable ,it is necessary to have heavy machinery, such as big cranes and jacks.



Source: JICA Study Team

Figure 4.2.24 Locked Coil Rope

Table 4.2.10 Comparison of Types of Cables

Type	CASE - 1 NPWS(New Parallel Wire	CASE - 2 FUT-H strand cables	CASE - 3 Locked Coil Rope
Cable Specifications (Starting from above)	Cables 1～3	Cables 1～5	Cables 1～3
	: ϕ7 x 337 A = 12969mm^2	: ϕ15.6 x 70 A = 10255mm^2	: 3@ϕ92 A = 5850mm^2 (x3)
	Cables 4～6	Cables 6～10	Cables 4～7
	: ϕ7 x 199 A = 7658mm^2	: ϕ15.6 x 44 A = 6446mm^2	: 3@ϕ76 A = 3960mm^2 (x3)
	Cables 7～10		Cables 8～10
	: ϕ7 x 187 A = 7197mm^2		: 3@ϕ64 A = 2840mm^2 (x3)
Cable Weight	250t	250t	380t
Cost Ratio	1.37	1.00	1.05
Evaluation		○	

Source: JICA Study Team

(2) Anchor Study

Anchors on girder and pylon were studied. Pylon cross section and the middle cell of the girder are as follows:

Pylon cross section : Transverse direction width: 2.5 m x Longitudinal direction width: 3.0 m

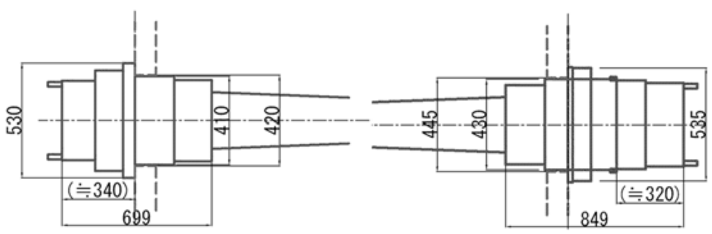
Girder middle cell point : Space between web: 2.5 m, Height of girder: 2.7 m

Cable was selected as FUT-H Strand Cables which is assembled by PC strand at the site. Maximum cross section is estimated as follows:

Cable: SEE, FUT-H-77H Tu≒20100 kN

Cables are fixed to the pylon. The pylon and girder sockets are shown below.

Pylon (Fixed) Girder (Movable)



Source: JICA Study Team

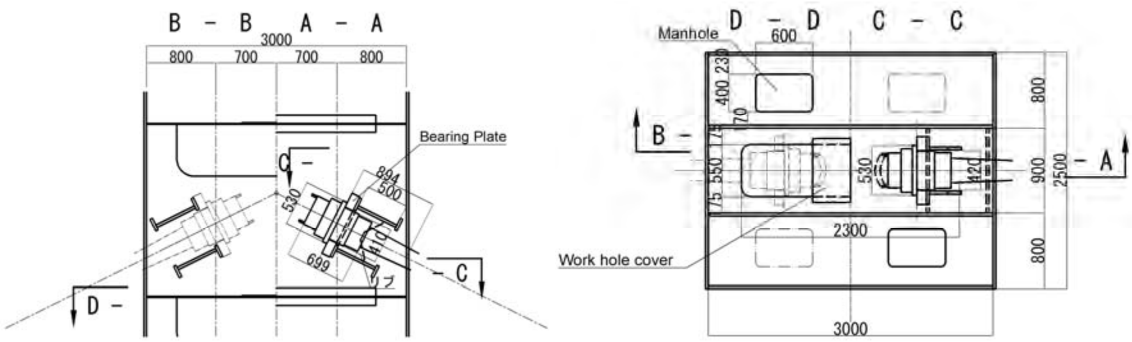
Figure 4.2.25 Cable Socket at Pylon and Girder

For this cross section of girder and pylon, applicable anchor types were proposed. From a structural and workability point of view and considering the economical aspect, the most optimal anchor was selected. Study results are shown in the next page.

1) Pylon Anchor Structure

Three types of pylon anchor structure (Anchor Girder, Anchor Plate, Saddle) were compared. Based on the comparison results, "Case-1: Anchor Girder" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case - 1 Anchor Girder **< Recommended >**



Overview: The cable socket is supported by the bearing plates which cross horizontally the two inner vertical plates of the tower.

Source: JICA Study Team

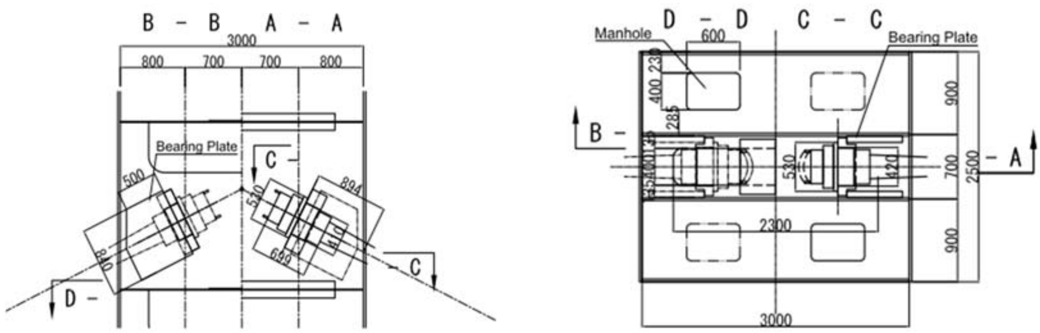
Figure 4.2.26 Anchor Girder

Table 4.2.11 Characteristics of Anchor Girder

Weight (Per unit)	5.2t
Structurability	The tensile forces of the cable are supported by the anchor girders, the differences of horizontal forces and the vertical forces are transmitted to the pylon wall by a diaphragm and inner vertical plates. The bearing plate will become thicker but, it's possible to support the sockets in 2 directions rather than 4 directions.
Erection	It has a narrow section welding, it`s necessary to be cautious with the assembly order and the production time. During erection it`s easy to maintain the work
Evaluation	○

Source: JICA Study Team

Case - 2 Anchor Plate



Overview: The cable socket is supported by the bearing plate, and the bearing plate is suppoted by two bearing plates which are attached to the pylon inner vertical plates.

Source: JICA Study Team

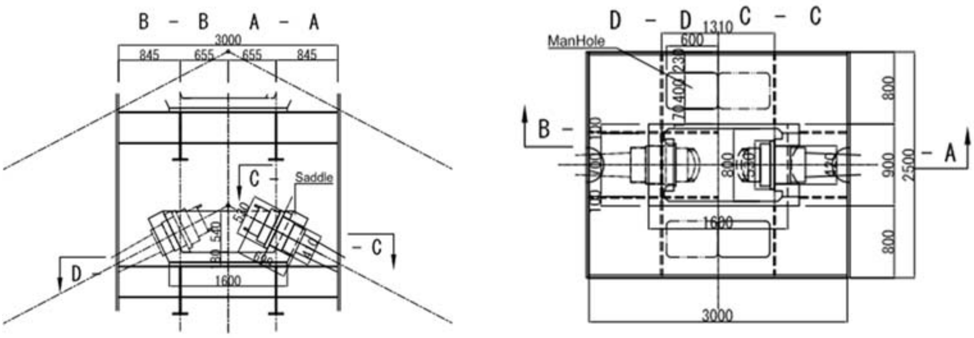
Figure 4.2.27 Anchor Plate

Table 4.2.12 Characteristics of Anchor Plate

Weight (Per unit)	5.6t
Structurability	The vertical plate is in direct contact with the bearing plate making it possible to reduce the width of the pylon. Reducing the width of the pylon would also reduce the working space. The uneven stress in the welding part of the anchor plate should be payed attention in the design.
Erection	When assembling, the angle error of inner virtical plates should be reduced. Welding amount will be increased.
Evaluation	

Source: JICA Study Team

Case - 3 Saddle



Overview: Both sides of the cable socket are attached to the saddle, which is supported by the lower side two-directional beam.
Source: JICA Study Team

Figure 4.2.28 Saddle

Table 4.2.13 Characteristics of Saddle

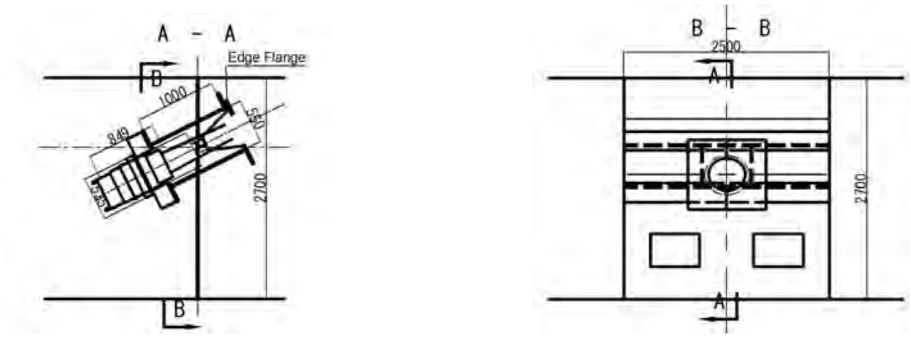
Weight (Per unit)	5.7t
Structurability	The tensile force of the cables is supported by the saddle, the differences of horizontal forces and the vertical forces are transmitted to the pylon wall by a diaphragm and 2 direction beam. The stress condition will become complex in the inner saddle so special attention should be payed.
Erection	Despite the low variety of saddles, every angle of cable attachment needs to be changed. The pylon is easy to built up. When installing, the saddle can either be
Evaluation	

Source: JICA Study Team

2) Girder Anchor Structure

Four types of girder anchor structure (Anchor Girder, Vertical Beam, Pipe Anchor, Vertical Girder) were compared. Based on the comparison results, "Case-1: Anchor Girder" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case - 1 Anchor Girder **< Recommended >**



Overview: The cable socket is supported by bearing plate and the bearing plate is supported by anchor girder at inner cell of the main girder. The anchor girder is fixed horizontally between the two webs of the inner cell.
Source: JICA Study Team

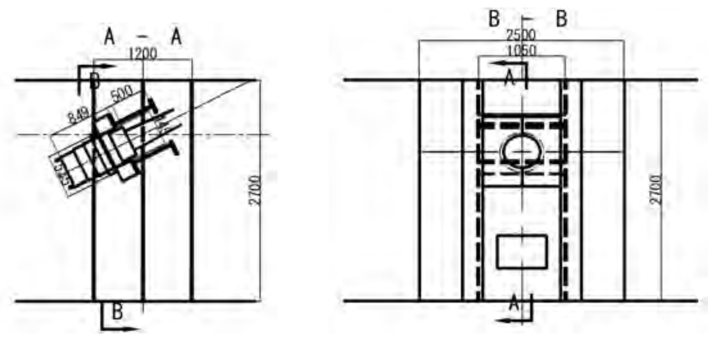
Figure 4.2.29 Anchor Girder

Table 4.2.14 Characteristics of Anchor Girder

Weight (Per unit)	3.6t
Structurability	All the cable tensile forces are passed from the anchor girder into the girder`s web then to the whole Main Girder. The webs thickness tends to become thicker but the mechanical state becomes simple.
Erection	It has narrow section welding, it`s necessary to be cautious with the assembly order and the production time. Since the sockets positioning point is relatively low, therefore when erecting, the space between the jack and the lower flange should be payed attention.
Evaluation	○

Source: JICA Study Team

Case - 2 Vertical Beam



Overview: Between the upper and lower flanges, two vertical beams are installed and bearing plate is attached to the horizontally fixed anchored girder in order to support the cable socket.
Source: JICA Study Team

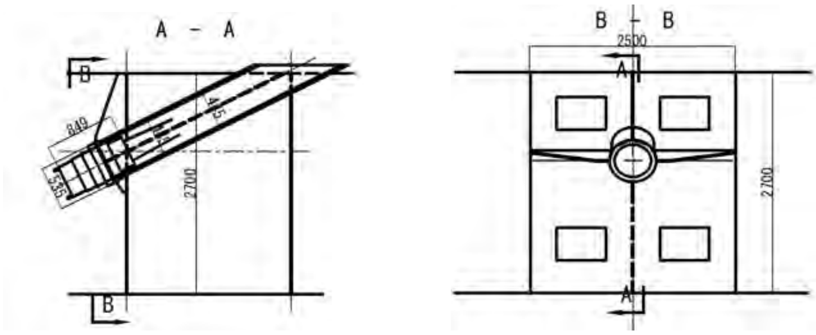
Figure 4.2.30 Vertical Beam

Table 4.2.15 Characteristics of Anchor Girder

Weight (Per unit)	3.9t
Structurability	Cables horizontal forces are transmitted through the vertical beam to the lower and upper flange, the vertical forces are transmitted from the vertical beam into the diaphragm. The socket can also be positioned in the upper part. Since the forces will be concentrated in the lower and upper flange, the flanges width should be thicken specially the upper flange.
Erection	It has narrow section welding, it`s necessary to be cautious with the assembly order and the production time. The position of the jack during the erection should be payed attention, due to the vertical beam and the support plate.
Evaluation	

Source: JICA Study Team

Case - 3 Pipe Anchor



Overview: The cable socket is attached to the pipe placed in the middle cell of the main girder. The force is transmitted to the flange and web throughout the plates which are placed horizontally and vertically between the two diaphragms.

Source: JICA Study Team

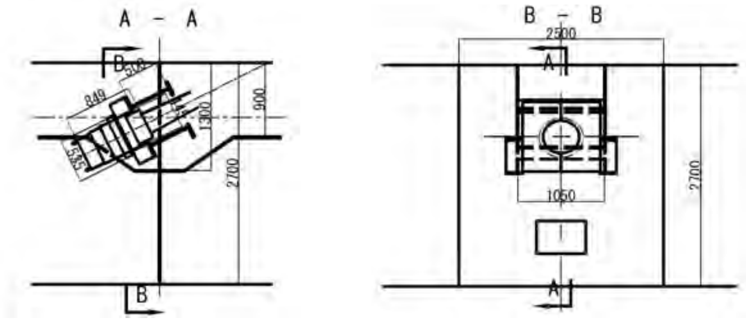
Figure 4.2.31 Pipe Anchor

Table 4.2.16 Characteristics of Pipe Anchor

Weight (Per unit)	3.6t
Structurability	Cables tensile force is transmitted from pipe to the whole main girder throughout 4 direction plate. The proportion of sharing of transmission forces between plates is unclear. Specially the upper plates effectiveness is unclear. It's necessary to pay attention to the momentum that the pipe receives as a part of the plates.
Erection	It has narrow section welding, it`s necessary to be cautious with the assembly order and the production time.The position of the jack during the erection should be payed attention.
Evaluation	

Source: JICA Study Team

Case - 4 Vertical Girder



Overview: Two vertical beams are placed in the inner middle cell, and anchored girder is placed between these two beams. In order to support the cable socket, bearing plate is attached to the anchor girder.

Source: JICA Study Team

Figure 4.2.32 Vertical Girder

Table 4.2.17 Characteristics of Vertical Girder

Weight (Per unit)	5.3t
Structurability	Cables horizontal force is transmitted to the deck plate throughout the vertical girder. The vertical forces are transmitted from the diaphragm to the web. There are unnecessary vertical girders that aren't transmitting cable forces and it is structurally useless.
Erection	It has narrow section welding, it`s necessary to be cautious with the assembly order and the production time. Anchor can be relatively be placed in the upper part, so during erection it's easy to maintain space.
Evaluation	

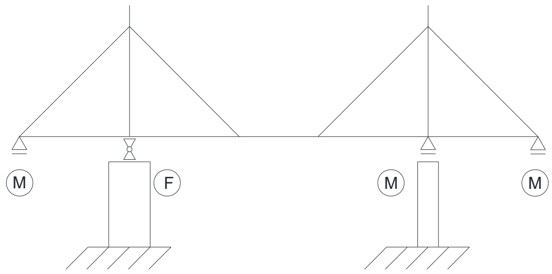
Source: JICA Study Team

4.2.2.7 Support Condition

Three types of support condition (M-F-M-M, M-F-F-M, E-E-E-E) were compared. Based on the comparison results, "Case-2: M-F-F-M Support" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case - 1 M-F-M-M

Type of Bearings	Rocker Bearing - Pin - Rocker Bearing - Rocker Bearing
Displacement of Main Girder	Templature Change : -60mm~+40mm
	Earthquake : 150mm



Source: JICA Study Team

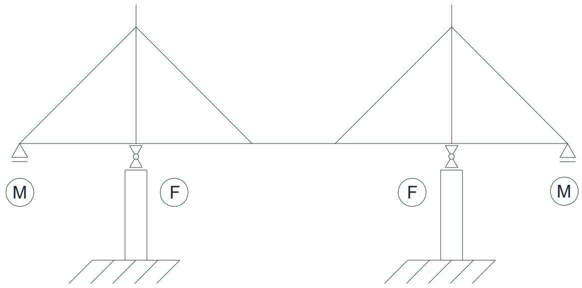
Figure 4.2.33 "M-F-M-M" Support

Characteristics:

- Horizontal displacement is resisted by pin bearing at one main tower, therefore, horizontal displacement of main girder due to temperature change and earthquake becomes slightly large.
- Horizontal displacement is resisted by pin bearing at one main tower and other bearings are movable, therefore, axial force does not act on main girder.
- Collision of girders with the next bridge's girder, and expansion amount at joints should be considered.
- In order to fix horizontal movement of girder, temporary pin bearing supports should be installed at M-supported pier at main tower during erection works.

Case - 2 M-F-F-M **< Recommended >**

Type of Bearings	Rocker Bearing - Pin - Pin - Rocker Bearing
Displacement of Main Girder	Templature Change : -40mm~+30mm
	Earthquake : 70mm



Source: JICA Study Team

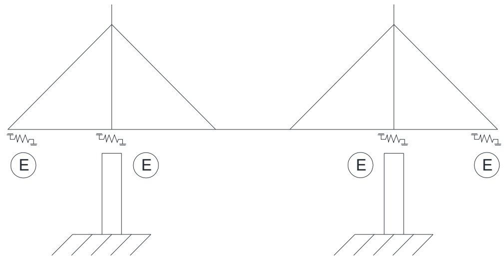
Figure 4.2.34 "M-F-F-M" Support

Characteristics:

- Rigidity of overall structure is high, and horizontal movement is fixed by pin bearing to the main tower.
- Horizontal displacement during earthquake is small; therefore, gap at the girder ends and expansion joints will become compact.
- Because of the influence of temperature change, axial force acts on the main girder at the center span.
- Horizontal movement of the main girder is fixed by pin bearing at the main tower at all times; therefore, this type is a suitable structure even during erection works.

Case – 3 E-E-E-E

Type of Bearings	Rubber - Rubber - Rubber - Rubber
Displacement of Main Girder	Templature Change : -40mm~+30mm
	Earthquake : 650mm



Source: JICA Study Team

Figure 4.2.35 "E-E-E-E" Support

Characteristics:

- Horizontal displacement is resisted by rubber bearing at the two main towers and the two side piers at both ends; therefore, horizontal displacement of main girder due to earthquake becomes large.
- Horizontal force caused by earthquake is dispersed throughout the piers; therefore, this type is efficient for seismic design.
- Vertical reaction force at the main pier is larger and multi-rubber bearing installment becomes necessary. Therefore, space at pier head will be decreased and this type is unfavorable in terms of maintenance.
- In order to fix horizontal movement of girder, temporary pin bearing supports should be installed during erection works.

Table 4.2.18 Comparison of Support Condition

Type		CASE - 1 M-F-M-M	CASE - 2 M-F-F-M	CASE - 3 E-E-E-E
Steel Weight (t)	Girder	4,600	4,600	4,590
	Tower	670	680	700
	Cable	250	250	260
	Total	5,520	5,530	5,550
Total Cost Ratio		1.05	1.01	1.00
Evaluation			○	

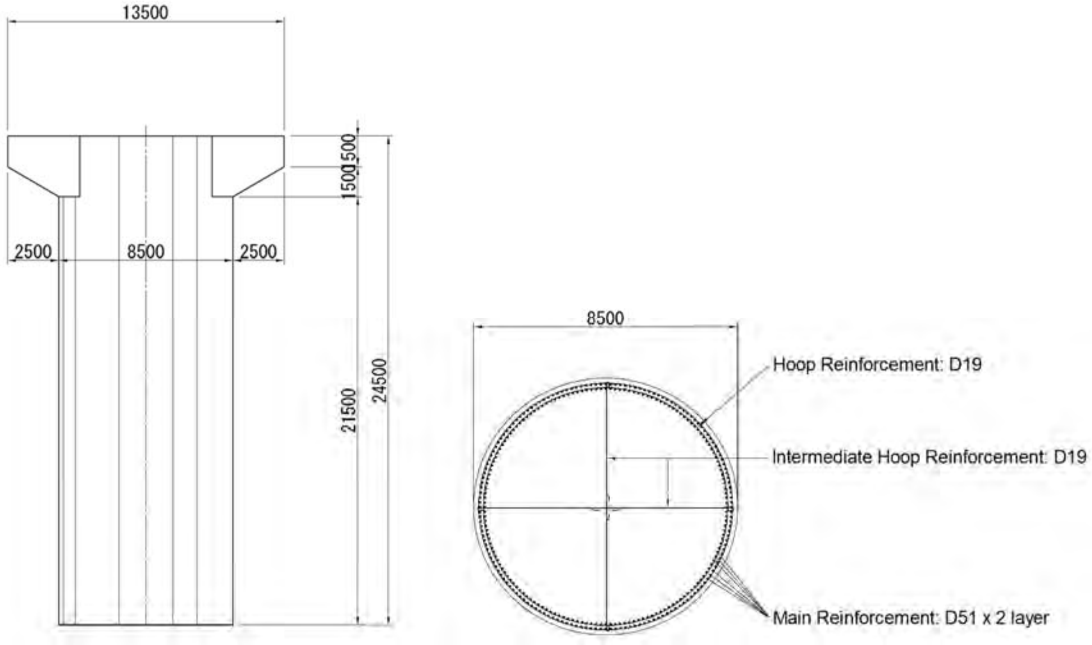
Source: JICA Study Team

4.2.3 Substructure of Cable-stayed Bridge

4.2.3.1 Shape of Pier Column at P11 and P12

Three types of shape of pier column at the intermediate pier of cable-stayed bridge (P11, P12) (Round Shape, Oval Shape, Oval Shape without overhang section) were compared. Based on the comparison results, "Case-2: Oval Shape" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case - 1 Round Shape



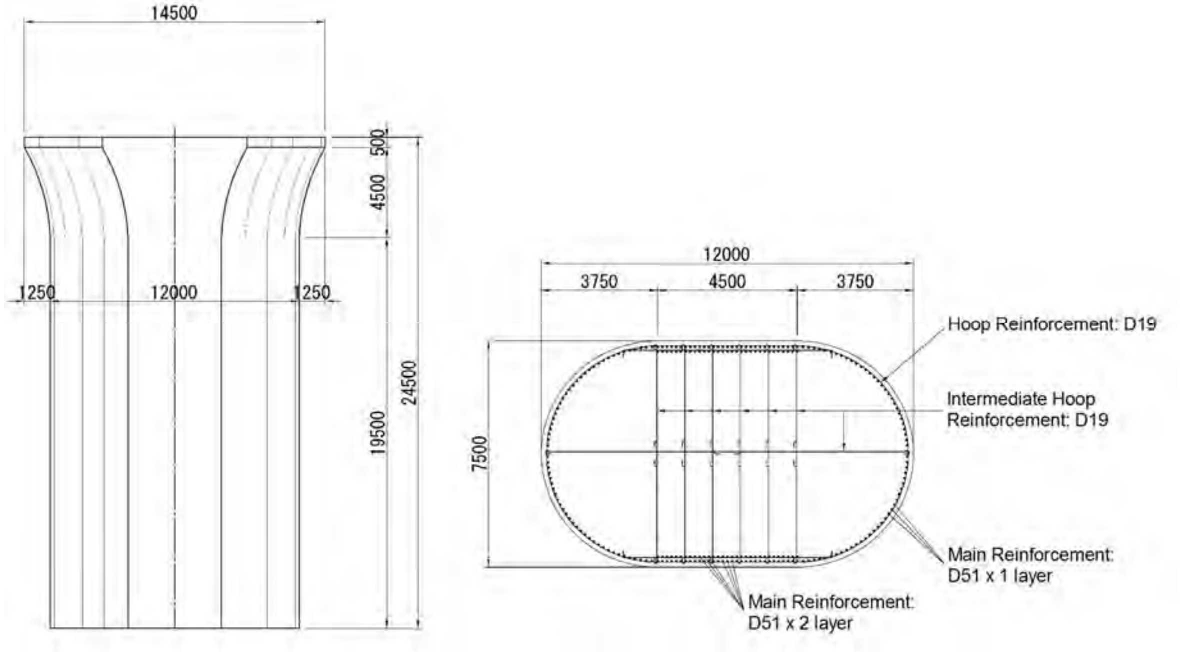
Source: JICA Study Team

Figure 4.2.36 Round Shape Column

Characteristics:

- Width of column in the transverse direction is wider than in other types; therefore, impediment ratio of river flow will become big.
- In order to install bearing support, pier head should be widened.

Case - 2   Oval Shape   **< Recommended >**



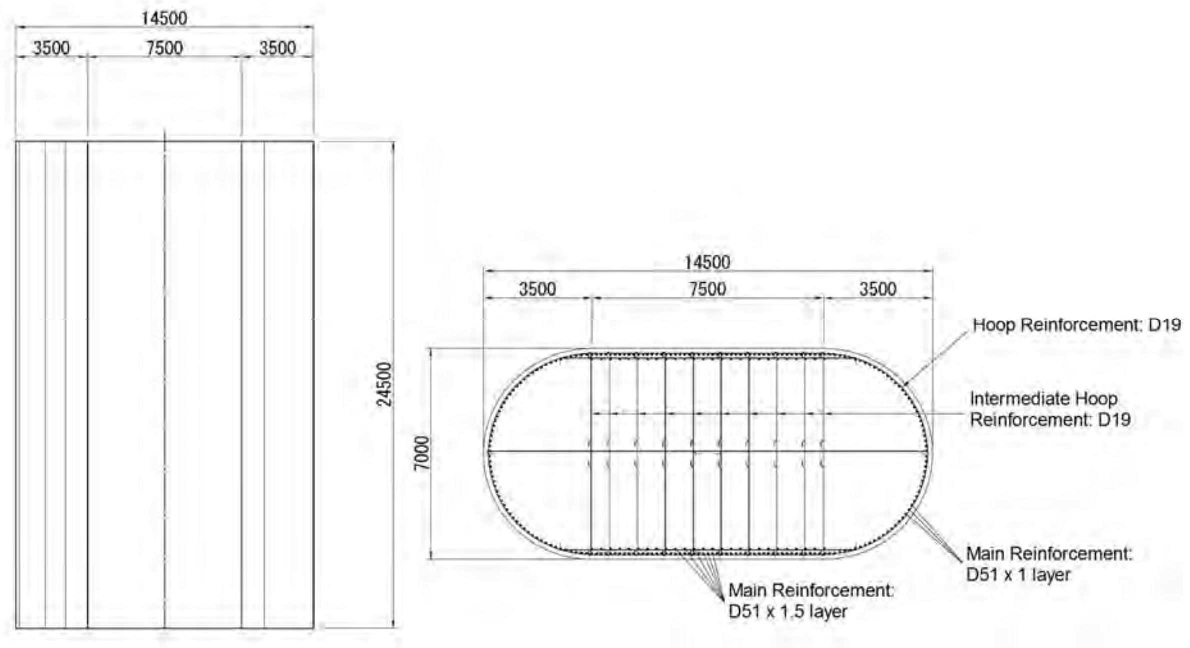
Source: JICA Study Team

Figure 4.2.37 Oval Shape Column

Characteristics:

- Because of the shape of the column in the pier head, construction works will become difficult compared to the other types.
- From the aesthetic point of view, since this type has the same shape with the other spans, it has a good appearance.

Case – 3 Oval Shape (without overhang section)



Source: JICA Study Team

Figure 4.2.38 Oval Shape (without overhang section) Column

Characteristics:

- Width of column in the transverse direction is narrower than in the other types; therefore, impediment ratio of river flow will become small.
- Because of the big cross section, concrete volume will become large.

Table 4.2.19 Comparison of Shape of Pier Column at P11 and P12

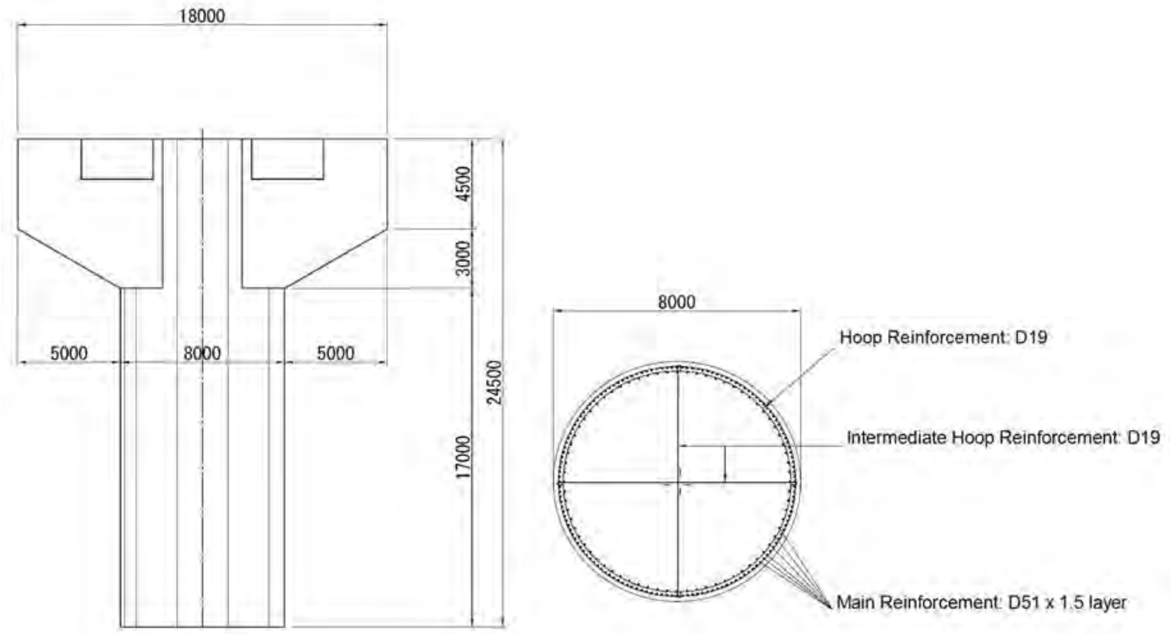
Quantities	Items	CASE - 1 Round Shape	CASE - 2 Oval Shape	CASE - 3 Round Shap (w/o overhang section)
	Concrete (σck=30N/mm²)	1450m³	1960m³	2230m³
	Form Work	700m²	810m²	910m²
	Reinforcement (SD345)	230t	240t	220t
Total Cost Ratio		0.87	1.00	1.01
Evaluation		× (Pier column at river should be oval shape)	○	

Source: JICA Study Team

4.2.3.2 Shape of Pier Column at P10 and P13

Three types of shape of pier column at the side pier of cable-stayed bridge (P10, P13) (Round Shape, Oval Shape, Oval Shape without overhang section) were compared. Based on the comparison results, "Case-2: Oval Shape" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case - 1 Round Shape



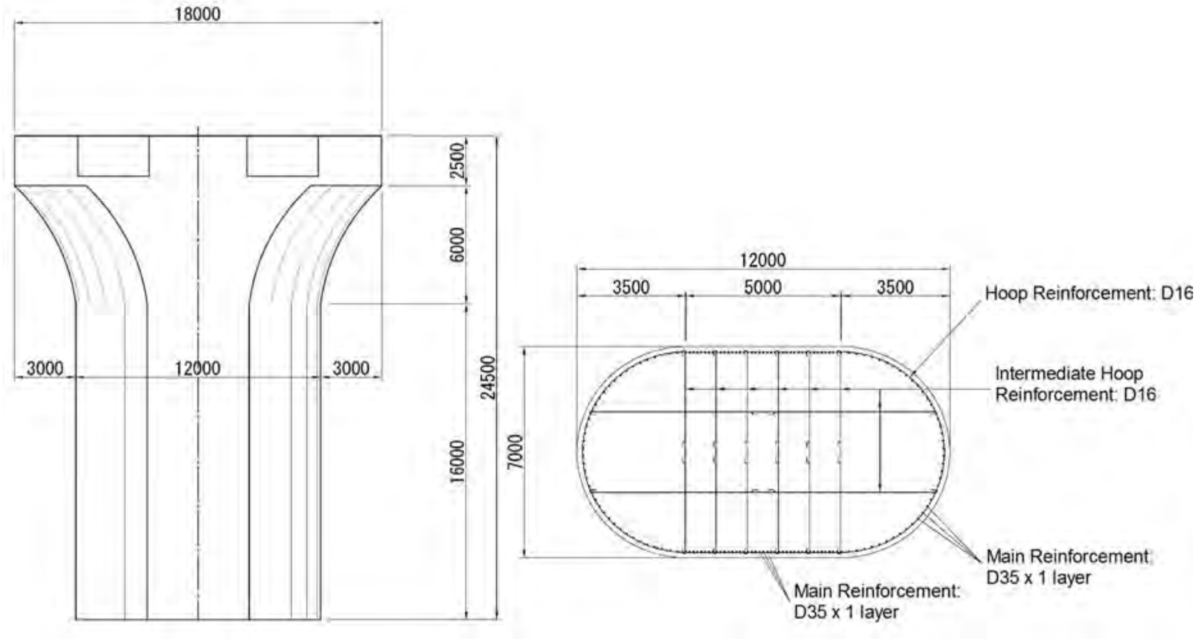
Source: JICA Study Team

Figure 4.2.39 Round Shape Column

Characteristics:

- Width of column in the transverse direction is wider than in the other types; therefore, impediment ratio of river flow will become big.
- In order to install bearing support, pier head should be widened.

Case - 2 Oval Shape **< Recommended >**



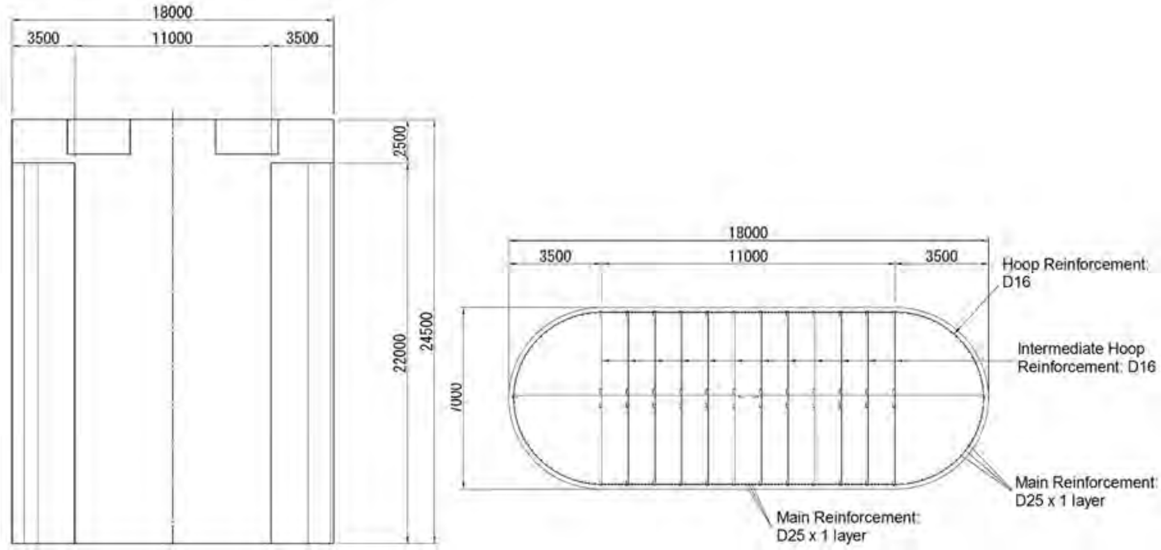
Source: JICA Study Team

Figure 4.2.40 Oval Shape Column

Characteristics:

- Because of the shape of the column in the pier head, construction works will become difficult compared to the other types.
- From the aesthetic point of view, since this type has the same shape with the other spans, it has a good appearance.

Case – 3 Oval Shape (without overhang section)



Source: JICA Study Team

Figure 4.2.41 Oval Shape (without overhang section) Column

Characteristics:

- Width of column in the transverse direction is narrower than in the other types; therefore, impediment ratio of river flow will become small.
- Because of the big cross section, concrete volume will become large.

Table 4.2.20 Comparison of Shape of Pier Column at P10 and P13

Quantities	Items	CASE - 1 Round Shape	CASE - 2 Oval Shape	CASE - 3 Round Shape (w/o overhang section)
	Concrete (σck=30N/mm²)	1650m³	2060m³	2830m³
	Form Work	810m²	860m²	1080m²
	Reinforcement (SD345)	280t	210t	140t
Total Cost Ratio		1.08	1.00	1.01
Evaluation		× (Pier column at river should be oval shape)	○	

Source: JICA Study Team

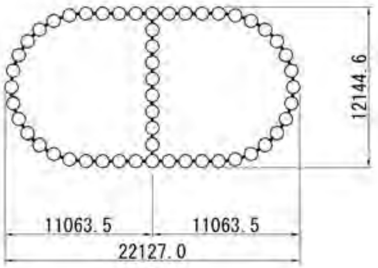
4.2.4 Foundation of Cable-stayed Bridge

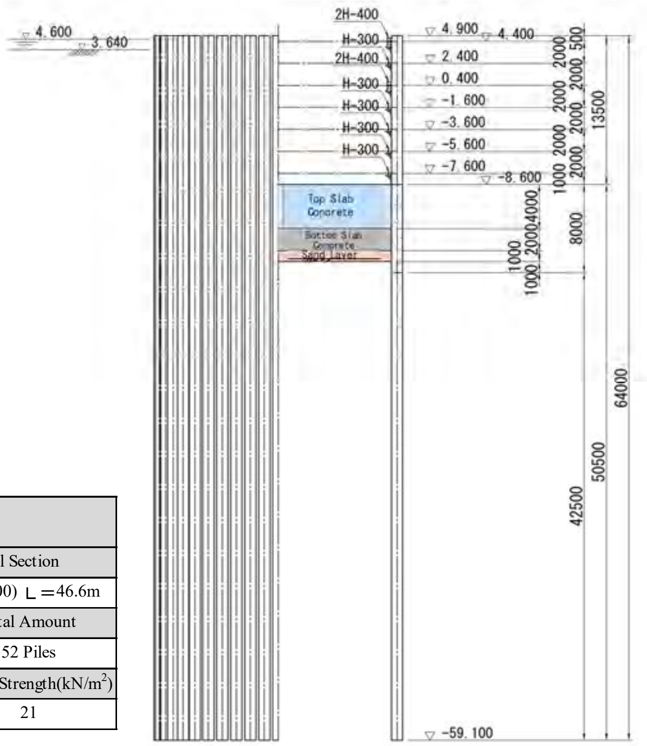
4.2.4.1 Diameter of Steel Pipe Sheet Pile

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

In the F/S stage, the diameter of the steel pipe sheet pile was planned to be 1000 mm. Here, a comparison of the diameters D = 1000 mm, 1200 mm and 1500 mm was conducted. Based on the comparison results, <u>"Case-2: φ1200 mm"</u> was selected as the best type.

$\boxed{\text{Case - 1}}$ <u>RC Oval Shape φ1000 mm</u>



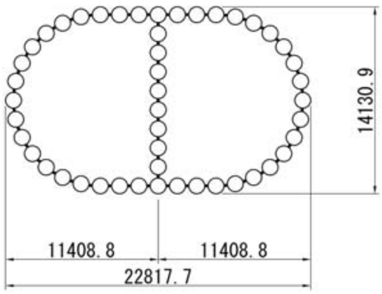


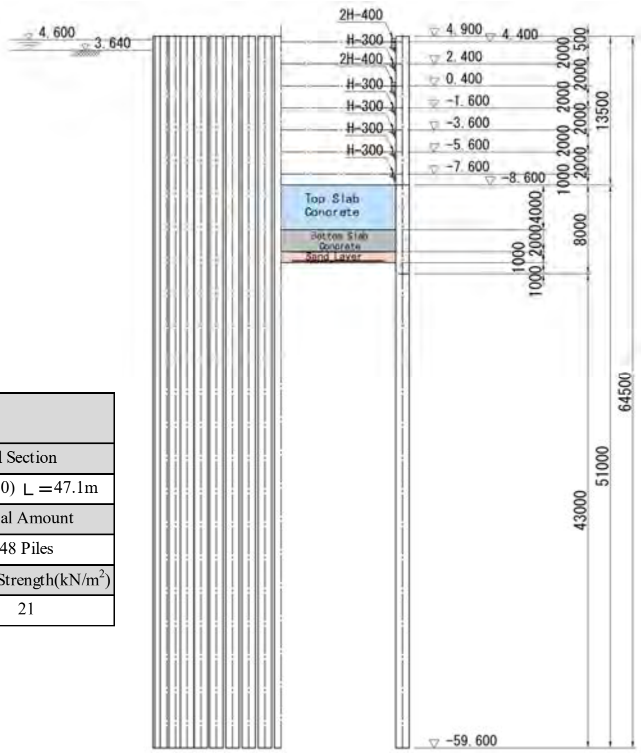
	CASE- 1 RC Oval Shape φ 1000		
Steel Pipe Sheet Pile	Outer Wall Section		Partition Wall Section
	φ1000×t18(SKY490) L =64.0m		φ1000×t11(SKY400) L =46.6m
Steel Pipe Sheet Pile Amount	Outer Wall	Partition Wall	Total Amount
	44 Piles	8 Piles	52 Piles
Pipe Joint	Pipe Joint	Joint Type	Mortar Strength(kN/m^2)
	φ165.2×t11	P-P	21

Source: JICA Study Team

Figure 4.2.42 Study Results for φ1000 mm

Case - 2 RC Oval Shape φ1200 mm **< Recommended >**



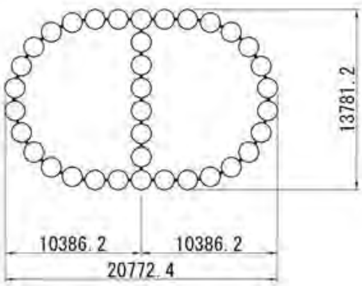


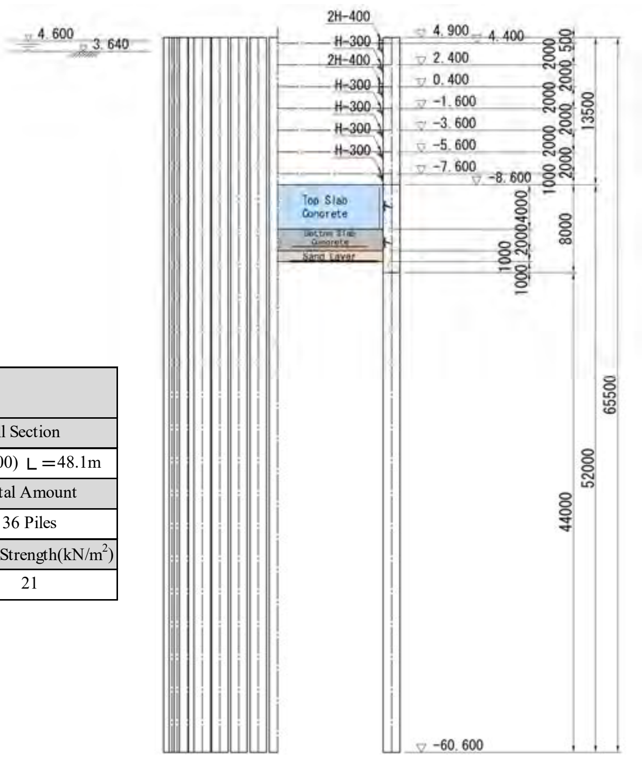
	CASE- 2 RC Oval Shape φ1200		
Steel Pipe Sheet Pile	Outer Wall Section	Partition Wall Section	
	φ1200×t15(SKY490) L =64.5m	φ1200×t14(SKY400) L =47.1m	
Steel Pipe Sheet Pile Amount	Outer Wall	Partition Wall	Total Amount
	40 Piles	8 Piles	48 Piles
Pipe Joint	Pipe Joint	Joint Type	Mortar Strength(kN/m^2)
	φ165.2×t11	P-P	21

Source: JICA Study Team

Figure 4.2.43 Study Results for φ1200 mm

Case – 3 RC Oval Shape φ1500 mm





	CASE-3 RC Oval Shape φ1500		
Steel Pipe Sheet Pile	Outer Wall Section	Partition Wall Section	
	φ1500×t17(SKY490) L =65.5m	φ1500×t17(SKY400) L =48.1m	
Steel Pipe Sheet Pile Amount	Outer Wall	Partition Wall	Total Amount
	30 Piles	6 Piles	36 Piles
Pipe Joint	Pipe Joint	Joint Type	Mortar Strength(kN/m^2)
	φ165.2×t11	P-P	21

Source: JICA Study Team

Figure 4.2.44 Study Results for φ1500 mm

Table 4.2.21 Comparison of Pile Diameters at P11 and P12

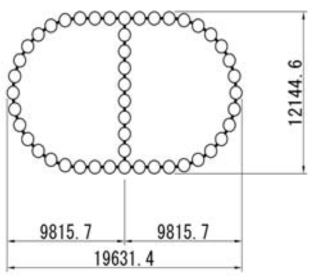
		CASE- 1 RC Oval Shape φ1000						CASE- 2 RC Oval Shape φ1200						CASE-3 RC Oval Shape φ1500					
Shear Rigidity (kN/m2)		Principal Load, Dead Load+Earthquake Load						Principal Load, Dead Load+Earthquake Load						Principal Load, Dead Load+Earthquake Load					
		600						600						600					
Shear Strength (kN/m)		Principal Load			Dead Load + Earthquake Load			Principal Load			Dead Load + Earthquake Load			Principal Load			Dead Load + Earthquake Load		
		100			133			100			133			100			133		
Planar Dimension Determination Factor		Smallest shape was determined from the construction space.						Smallest shape was determined from the construction space.						Smallest shape was determined from the construction space.					
Normal, Earthquake		δ (cm)		Rmax (kN)		Coumpound Stress σ(N/mm²)		δ (cm)		Rmax (kN)		Coumpound Stress σ(N/mm²)		δ (cm)		Rmax (kN)		Coumpound Stress σ(N/mm²)	
	Normal・Earthquake	2.599	Longi. Direction	2856	Longi. Direction	271	Trans. Direction	2.338	Longi. Direction	3023	Longi. Direction	262	Longi. Direction	2.495	Longi. Direction	3939	Longi. Direction	205	Trans. Direction
	Allowable Value	5.000	Earthquake	3800	Principal Load	278	Earthquake	5.000	Earthquake	4400	Earthquake	278	Earthquake	5.000	Earthquake	5300	Principal Load	278	Earthquake
Steel Weight(t)		1550						1520						1610					
Evaluation								○											

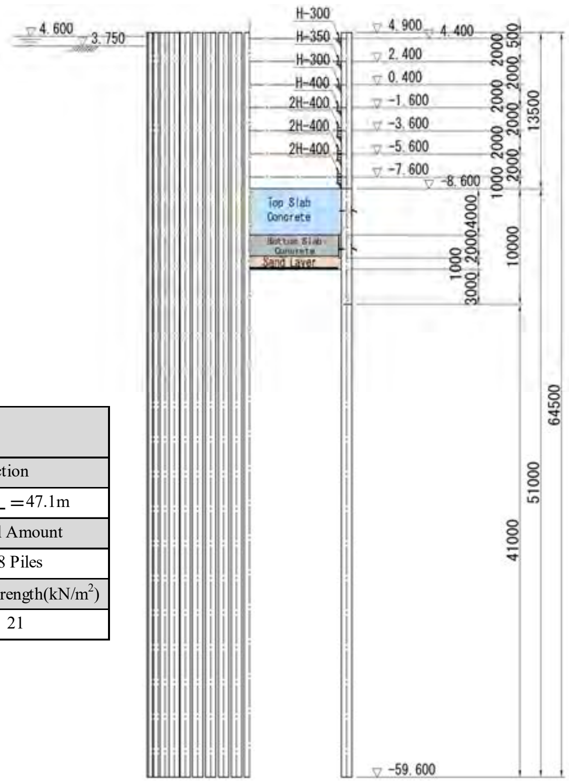
Source: JICA Study Team

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

Same as for P11 and P12, a comparison of the pile diameters at P10 and P13 was conducted. Based on the comparison results, <u>"Case-2: φ1200 mm"</u> was selected as the best type.

Case - 1 RC Oval Shape φ1000 mm



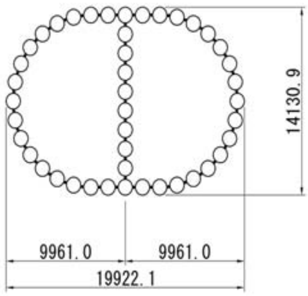


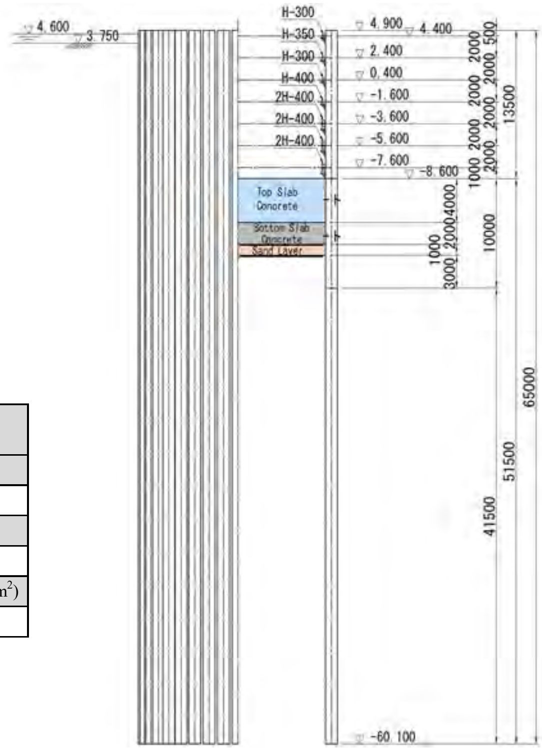
	CASE- 1 RC Oval Shape φ1000		
Steel Pipe Sheet Pile	Outer Wall Section	Partition Wall Section	
	φ1000×t19(SKY400) L=64.5m	φ1000×t11(SKY400) L=47.1m	
Steel Pipe Sheet Pile Amount	Outer Wall	Partition Wall	Total Amount
	40 Piles	8 Piles	48 Piles
Pipe Joint	Pipe Joint	Joint Type	Mortar Strength(kN/m²)
	φ165.2×t11	P-P	21

Source: JICA Study Team

Figure 4.2.45 Study Results for φ1000 mm

Case - 2 RC Oval Shape φ1200 mm **< Recommended >**



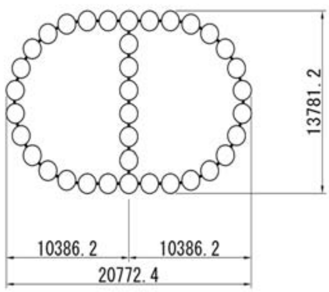


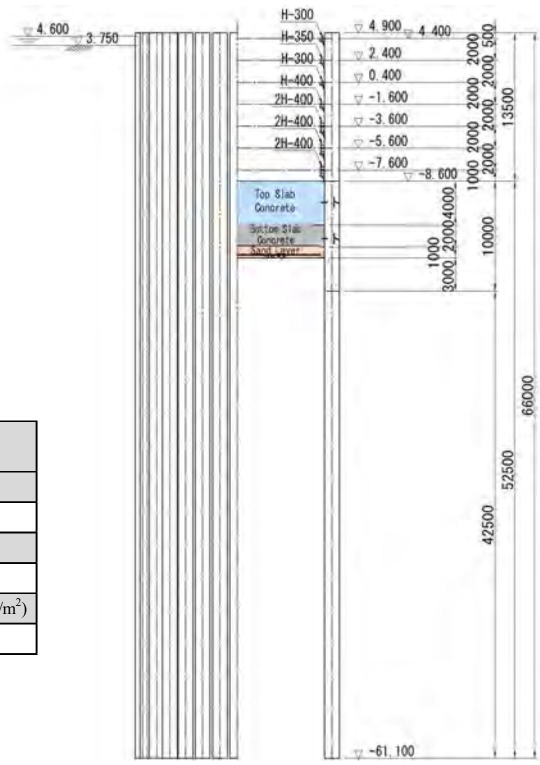
	CASE- 2 RC Oval Shape φ1200		
Steel Pipe Sheet Pile	Outer Wall Section	Partition Wall Section	
	φ1200×t16(SKY400) L =65.0m	φ1200×t14(SKY400) L =47.6m	
Steel Pipe Sheet Pile Amount	Outer Wall	Partition Wall	Total Amount
	36 Piles	8 Piles	44 Piles
Pipe Joint	Pipe Joint	Joint Type	Mortar Strength(kN/m^2)
	φ165.2×t11	P-P	21

Source: JICA Study Team

Figure 4.2.46 Study Results for φ1200 mm

Case - 3 RC Oval Shape φ1500 mm





	CASE-3 RC Oval Shape φ1500		
Steel Pipe Sheet Pile	Outer Wall Section	Partition Wall Section	
	φ1500×t17(SKY400) L =66.0m	φ1500×t17(SKY400) L =48.6m	
Steel Pipe Sheet Pile Amount	Outer Wall	Partition Wall	Total Amount
	30 Piles	6 Piles	36 Piles
Pipe Joint	Pipe Joint	Joint Type	Mortar Strength(kN/m^2)
	φ165.2×t11	P-P	21

Source: JICA Study Team

Figure 4.2.47 Study Results for φ1500 mm

Table 4.2.22 Comparison of Pile Diameters at P10 and P13

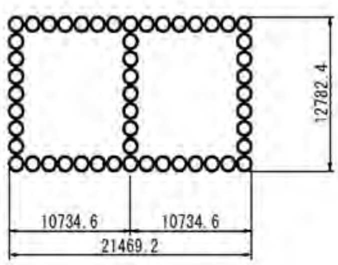
		CASE- 1 RC Oval Shape φ1000						CASE- 2 RC Oval Shape φ1200						CASE-3 RC Oval Shape φ1500					
Shear Rigidity (kN/m²)		Principal Load, Dead Load+Earthquake Load						Principal Load, Dead Load+Earthquake Load						Principal Load, Dead Load+Earthquake Load					
		600						600						600					
Shear Strength (kN/m)		Principal Load			Dead Load + Earthquake Load			Principal Load			Dead Load + Earthquake Load			Principal Load			Dead Load + Earthquake Load		
		100			133			100			133			100			133		
Planar Dimension Determination Factor		Smallest shape was determined from the construction space.						Smallest shape was determined from the construction space.						Smallest shape was determined from the construction space.					
Normal, Earthquake		δ (cm)		Rmax (kN)		Coumpound Stress σ(N/mm²)		δ (cm)		Rmax (kN)		Coumpound Stress σ(N/mm²)		δ (cm)		Rmax (kN)		Coumpound Stress σ(N/mm²)	
	Normal・Earthquake	1.196	Longi. Direction	2103	Longi. Direction	204	Trans. Direction	1.098	Longi. Direction	2285	Longi. Direction	194	Longi. Direction	1.018	Longi. Direction	2830	Longi. Direction	160	Trans. Direction
	Allowable Value	5.000	Earthquake	4100	Principal Load	210	Earthquake	5.000	Earthquake	4800	Earthquake	210	Earthquake	5.000	Earthquake	6100	Principal Load	210	Earthquake
Steel Weight(t)		1250						1160						1620					
Evaluation		○																	

Source: JICA Study Team

4.2.4.2 Shape of Foundation at P11 and P12

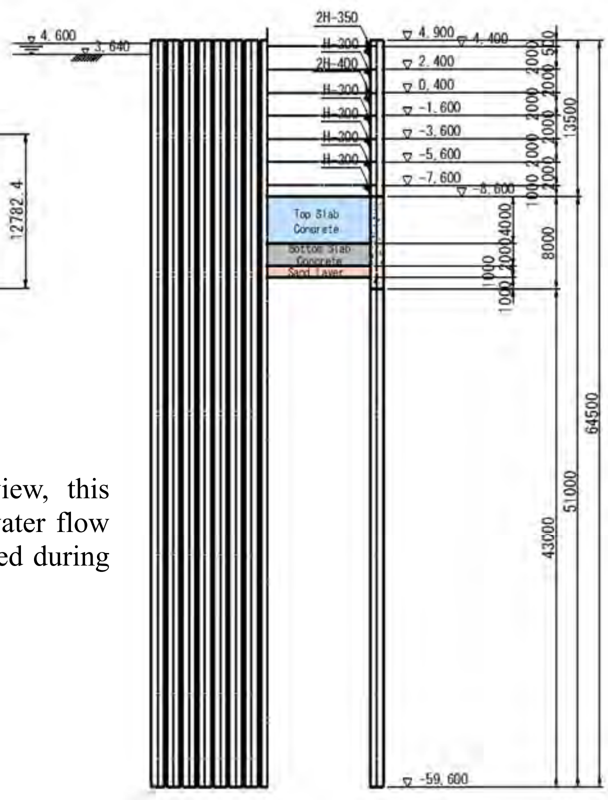
In Section 4.2.4.1, pile diameter of φ1200 mm was selected. The shape of foundation was studied and three types (Rectangular Shape, Round Shape, Oval Shape) were compared. Based on the comparison results, "Case-3: Oval Shape" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case - 1 Rectangle Shape



Characteristics:

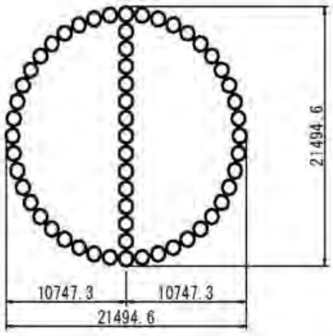
- From the structural point of view, this rectangle shape is unfavorable to water flow and many support should be installed during construction work.

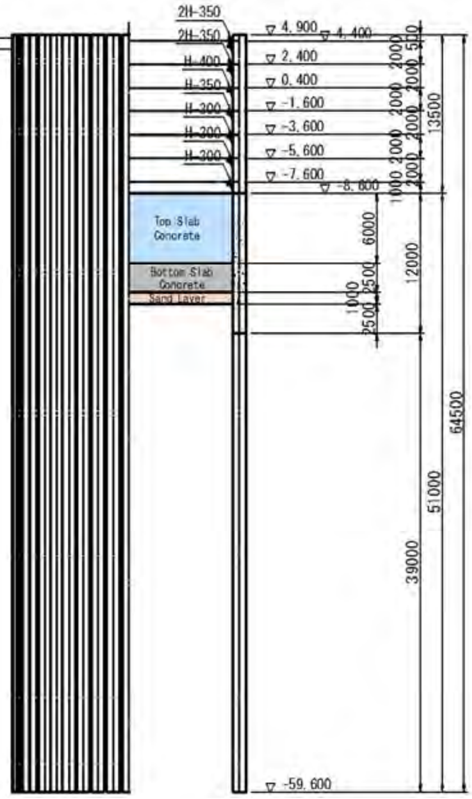


Source: JICA Study Team

Figure 4.2.48 Rectangle Shape

Case - 2 Round Shape





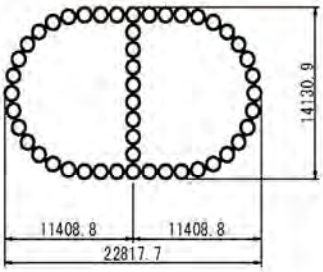
Characteristics:

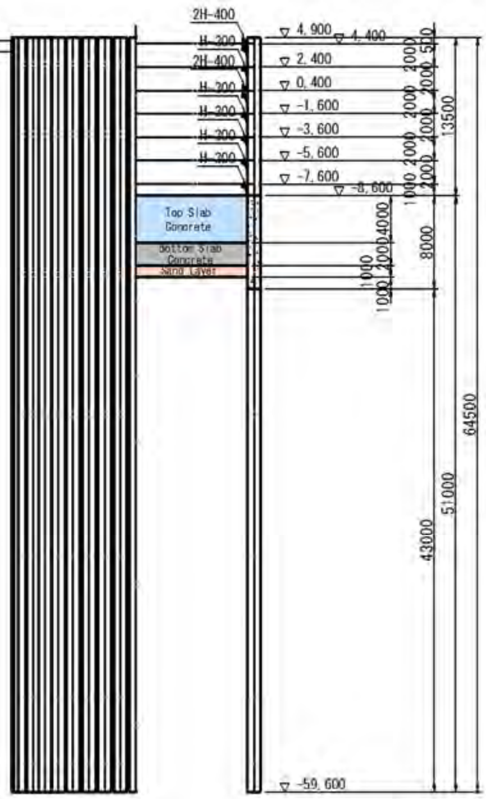
- Pier shape was decided as "Oval Shape"; therefore, this type has too much unnecessary space.

Source: JICA Study Team

Figure 4.2.49 Round Shape

Case - 3 Oval Shape < **Recommended** >





Characteristics:

- Pier shape was decided as "Oval Shape" a foundation has the same shape; therefore, t is the most suitable.

Source: JICA Study Team

Figure 4.2.50 Oval Shape

Table 4.2.23 Comparison of Foundation Shapes at P11 and P12

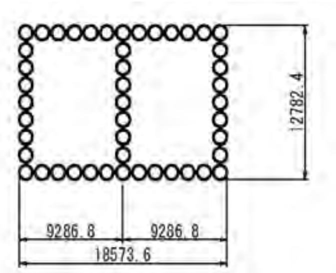
		CASE - 1 Rectangle Shape		CASE - 2 Round Shape		CASE - 3 Oval Shape	
Number and Size of Steel Pipe	Location	Size	Number	Size	Number	Size	Number
	Outer Wall	ϕ1200	44	ϕ1200	44	ϕ1200	40
	Inner Wall	ϕ1200	7	ϕ1200	13	ϕ1200	8
Steel Weight		1700		1800		1600	
Total Cost Ratio		1.06		1.12		1.00	
Evaluation						○	

Source: JICA Study Team

4.2.4.3 Shape of Foundation at P10 and P13

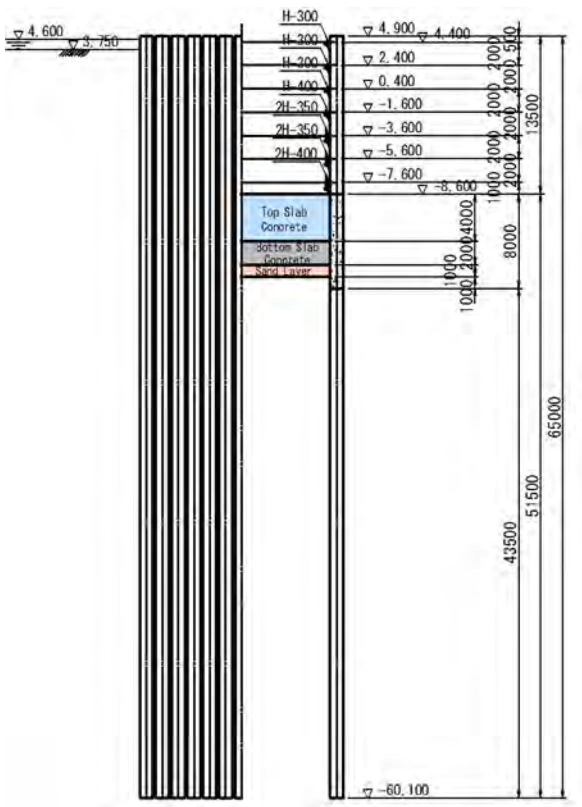
Same as for P11 and P12, comparison of the shape of foundation at P10 and P13 was conducted. Based on the comparison results, "Case-3: Oval Shape" was selected as the best type. Each figure, characteristics, and comparison results are shown as follows:

Case - 1 Rectangle Shape



Characteristics:

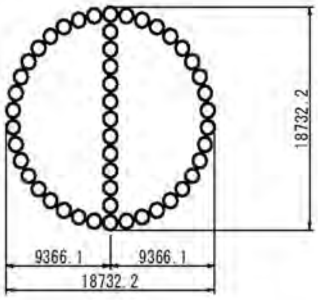
- From the structural point of view, this rectangle shape is unfavorable to water flow and many support should be installed during construction work.



Source: JICA Study Team

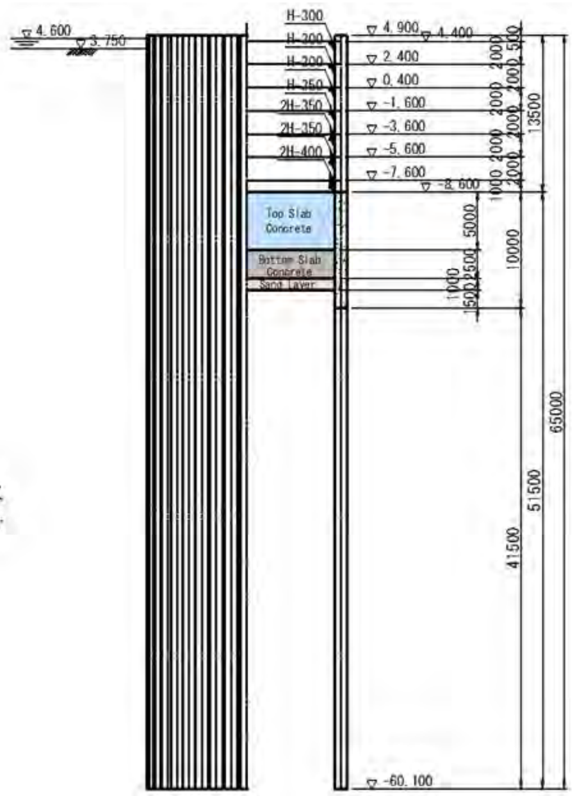
Figure 4.2.51 Rectangle Shape

Case - 2 Round Shape



Characteristics:

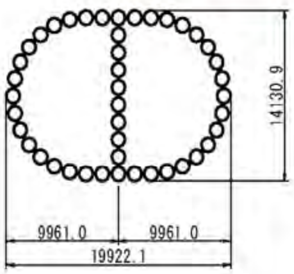
- Pier shape was decided as "Oval Shape"; therefore, this type has too much unnecessary space.



Source: JICA Study Team

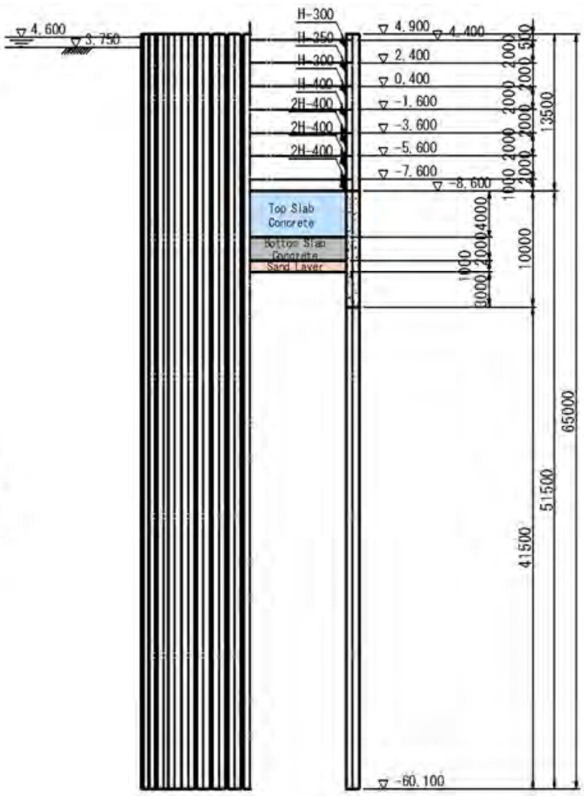
Figure 4.2.52 Round Shape

Case - 3 Oval Shape **< Recommended >**



Characteristics:

- Pier shape was decided as "Oval Shape" and this foundation has the same shape; therefore, this type is the most suitable.



Source: JICA Study Team

Figure 4.2.53 Oval Shape

Table 4.2.24 Comparison of Foundation Shape at P10 and P13

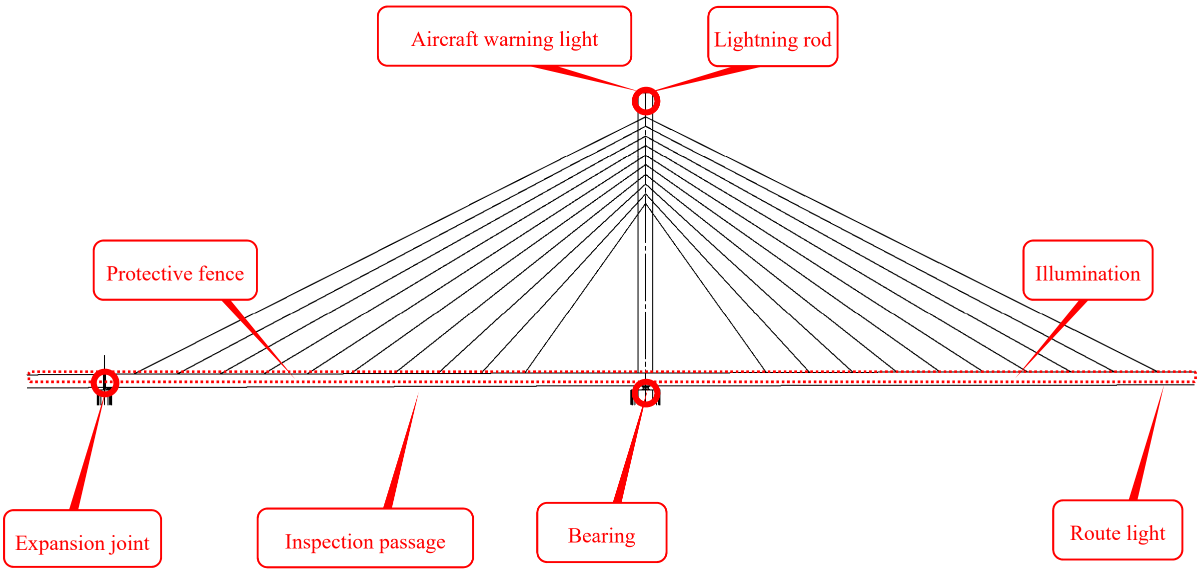
		CASE - 1 Rectangle Shape		CASE - 2 Round Shape		CASE - 3 Oval Shape	
Number and Size of Steel Pipe	Location	Size	Number	Size	Number	Size	Number
	Outer Wall	ϕ1200(t=16mm)	40	ϕ1200(t=16mm)	38	ϕ1200(t=16mm)	36
	Inner Wall	ϕ1200(t=14mm)	7	ϕ1200(t=14mm)	11	ϕ1200(t=14mm)	8
Steel Weight		1700		1800		1600	
Total Cost Ratio		1.07		1.11		1.00	
Evaluation						○	

Source: JICA Study Team

4.2.5 Bridge Accessories

4.2.5.1 Bridge Accessories

The following accessories will be included in the cable-stayed bridge:



Source: JICA Study Team

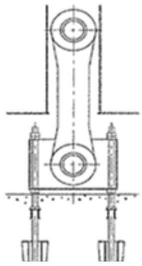
Figure 4.2.54 Bridge Accessories

The structure and location plan of bridge accessories will be studied and decided during the detailed design stage.

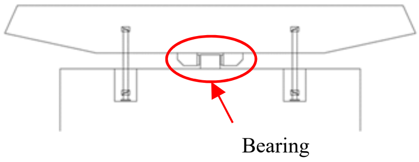
4.2.5.2 Bearing

(1) Edge Support Bearing

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



a) Rocking bearing



b) Horizontal bearing

Source: JICA Study Team

Figure 4.2.55 Edge Support Bearing

(2) Pylon Section Bearings

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

Table 4.2.25 Comparison of Bearing Types

	Fixed Bearing					
	Pivot bearing		Pin bearing		Rubber bearing	
Bearing shape						
Characteristics	The upper section is a concave shape and the bottom section is a convex shape, each of which is spherically finsihed and combined,making a fixed bearing that can rotate in all directions. As a movable bearing, rollers are used in combination.		It's a fixed bearing that can rotate in only one direction in a structure where the upper section and the lower section are connected by cylindrical pins. As a movable bearing, rollers are used in combination.		The vertical load is supported using natural rubber type laminated rubber bearings, horizontal load is supported by the upper section blocks which restrict movement.	
Stability during superstructure construction	Horizontal movement is restricted, therefore, during superstructure erection it's a stable structure.	◎	Horizontal movement is restricted, therefore, during superstructure erection it's a stable structure.	◎	During superstructure erection, horizontal deflection occurs on the rubber bearing making it an unstabel structure. Therefore, tempeorary bearing installation is necessary.	△
Functionality	For the rotation to be free in every direction,it's necessary for lateral rotation to be set free. It is possible to support large reaction force.	◎	It can only rotate in longitudinal direction, it's not suitable for a type of bridge where lateral rotation occurs.	△	Although lateral rotation is possible, it's likely to be difficult to design for a large movement and large rotation, or to be economical.	○
Inspection and repairability	Compared to a pin bearing, it has a simple structure where water is difficult to enter. When abnormal (earthquake etc.)), you can check the main part by removing the ring.	◎	Since the pin can't be protected against corrosion, it's necessary corrosion countermeasures around the pin and cleaning maintenance. It's impossible to check the damage situation of the pin at the time of abnormality (at the time of earthquake etc.).	△	Since it is structured to receive the horizontal force by the rubber body and the side block, inspection after the earthquake is easy (it can be inspected from the appearance).	◎
Evaluation	○					

Source: JICA Study Team

4.2.5.3 Expansion Joint

The expansion and contraction amounts of the expansion joint used in the basic design are shown below.

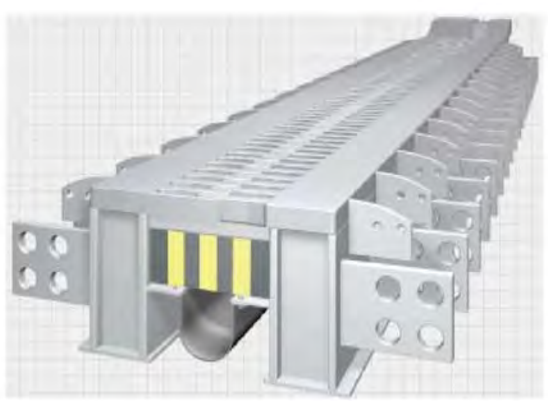
Table 4.2.26　Summary of Expansion Amounts

				P10		P13	
Bridge shape				PC-BOX	Cable Stayed Bridge	Cable Stayed Bridge	Steel Box Girder
Normal	Creep			12	—	—	—
	Drying shrinkage			8	—	—	—
	Temperature variation	Temperature fall(PC Girder+20℃、Steel girder +25℃)		12	63	63	130
		Temperature rise(PC Girder -20℃、Steel girder -25℃)		-12	-63	-63	-130
	Base expansion/contraction amount			44	126	126	260
	Margin amount	Base E/C amount ×20%　Minimum 10mm		10	25	25	52
	Expansion / Contraction amount			54	151	151	312
	Design amount of movement	Expansion/Contraction amount ×1/2	One side amount	±27	±76	±76	±156
Seismic	Movement amount	One side amount		±180	±47	±47	±285
		One side amount×√2		±255	±66	±66	±403
	Margin amount			±15	±15	±15	±15
	Design amount of movement			±270	±81	±81	±418
Design amount of movement used in expansion joint		Maximum value from ordinary and seismic movement		±270	±81	±81	±418
Girder joint gap in expansion joint		Girder joint gap amount		300		500	
Design girder joint gap expansion joint		Girder joint gap amount ± Design amount of movement		30～570		82～918	

Source: JICA Study Team

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

Table 4.2.27　Expansion Joints

Modular Expansion Joint	Steel Finger Joint
	
It's structure combines rubber material and steel material, and can also move in all directions.	The face plate is processed into a comb shape and it's structure is assembled with steel material.
It can also maitain water from filtering by attaching a durable high waterproof rubber into the original structure. It can be used from small bridges to big range bridges. It is used as a countermeasure for very large displacement	Excellent movability. It is used as a countermeasure for large displacements.
○	

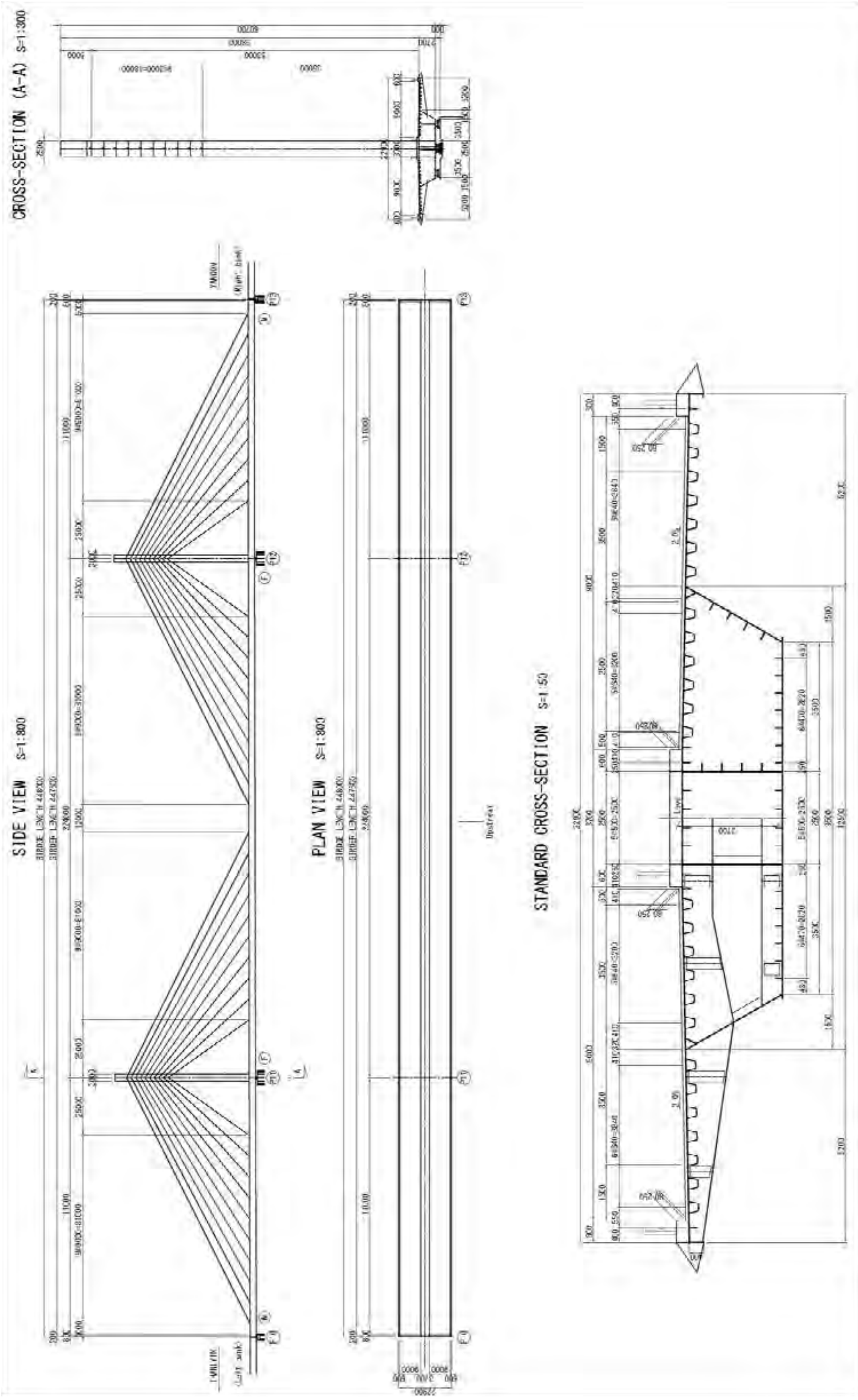
Source: JICA Study Team

In the detailed design stage, the expansion/contraction amount will be calculated again, and the expansion joint will be re-selected based on the D/D results.

4.2.6 Basic Design Results

4.2.6.1 Superstructure Basic Design Results

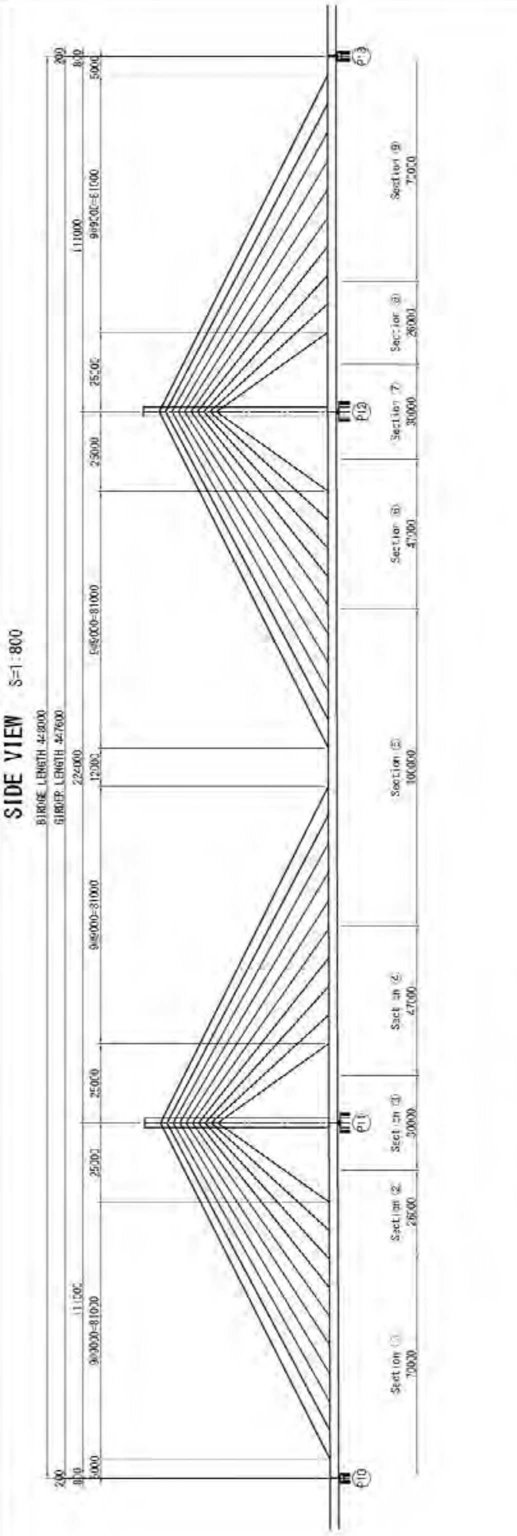
(1) Superstructure



Source: JICA Study Team

Figure 4.2.56 B/D Results for Superstructure

(2) Main Girder Cross Section



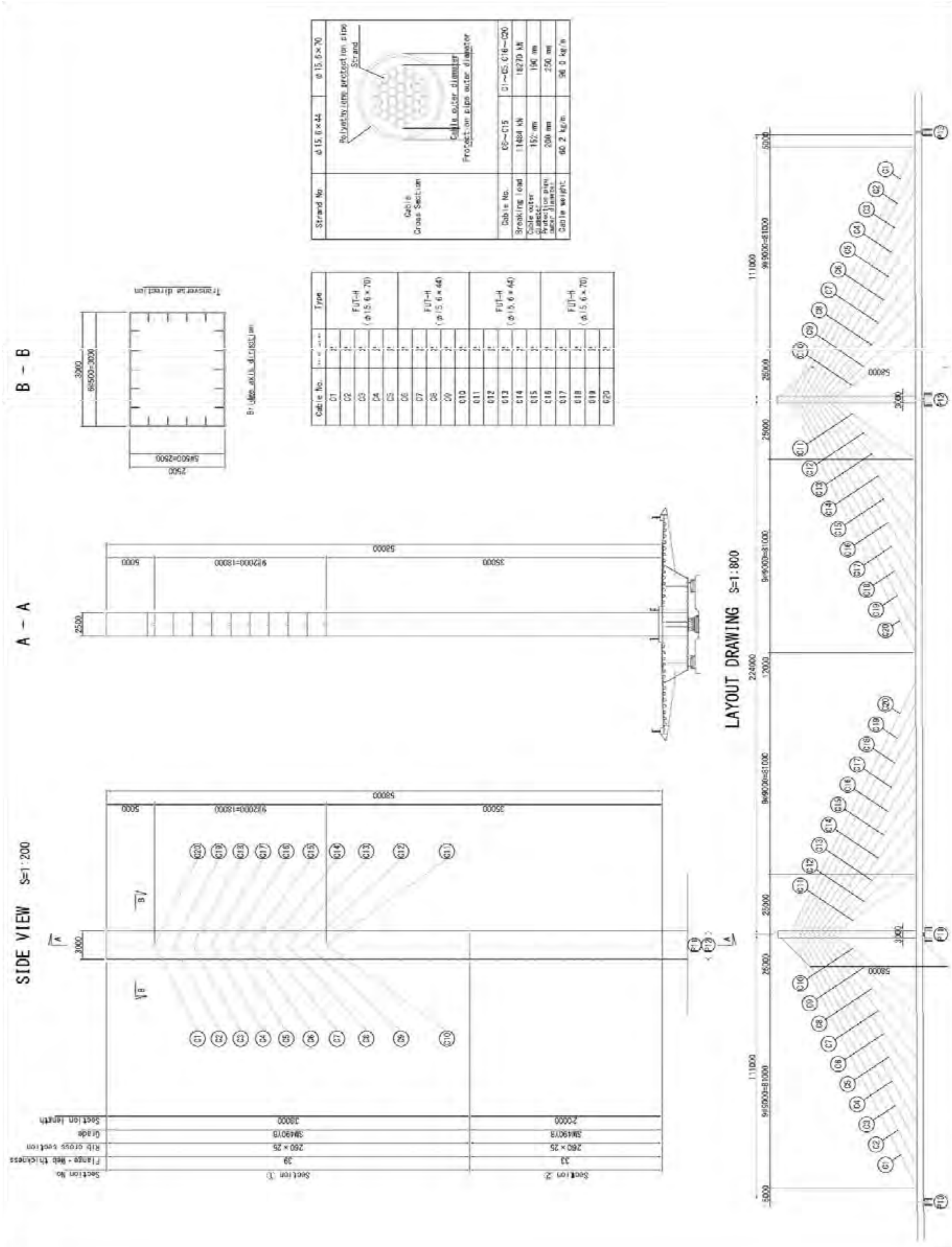
Unit: mm N/mm2

Section No.			①	②	③	④	⑤	⑥	⑦	⑧	⑨
Steel deck	Base steel	Thickness	16	13	16	15	16	16	16	16	16
		Grade	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y
	U Rib	C. Section	320x240x8-40	320x240x8-40	320x240x8-40	320x240x8-40	320x240x8-40	320x240x8-40	320x240x8-40	320x240x8-40	320x240x8-40
	Flat Rib	C. Section	210x20	210x20	210x20	210x20	210x20	210x20	210x20	210x20	210x20
Outer Web	Base steel	Thickness	15	14	17	14	14	14	17	14	15
		Grade	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y
	Longi. Rib	C. Section	160x16	130x13	150x16	160x15	160x16	160x16	190x19	160x16	160x16
Intermediate Web	Base steel	Thickness	15	14	17	14	14	14	17	14	15
		Grade	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y
	Longi. Rib	C. Section	160x16	130x15	150x18	180x15	160x16	160x16	190x19	160x16	160x16
Lower Flange	Base Steel	Thickness	15	12	16	11	11	11	16	12	15
		Grade	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y
	Long. Rib	C. Section	190x19	140x14	150x16	130x13	130x13	130x13	190x19	140x14	190x19
Cross Section Determination Case			Comp. stress	Comp. stress	Comp. stress	Minimum plate thickness	Minimum plate thickness	Minimum plate thickness	Comp. stress	Comp. stress	Comp. stress

Source: JICA Study Team

Figure 4.2.57 B/D Results for Main Girder Cross Section

(3) Pylon and Cable Structure

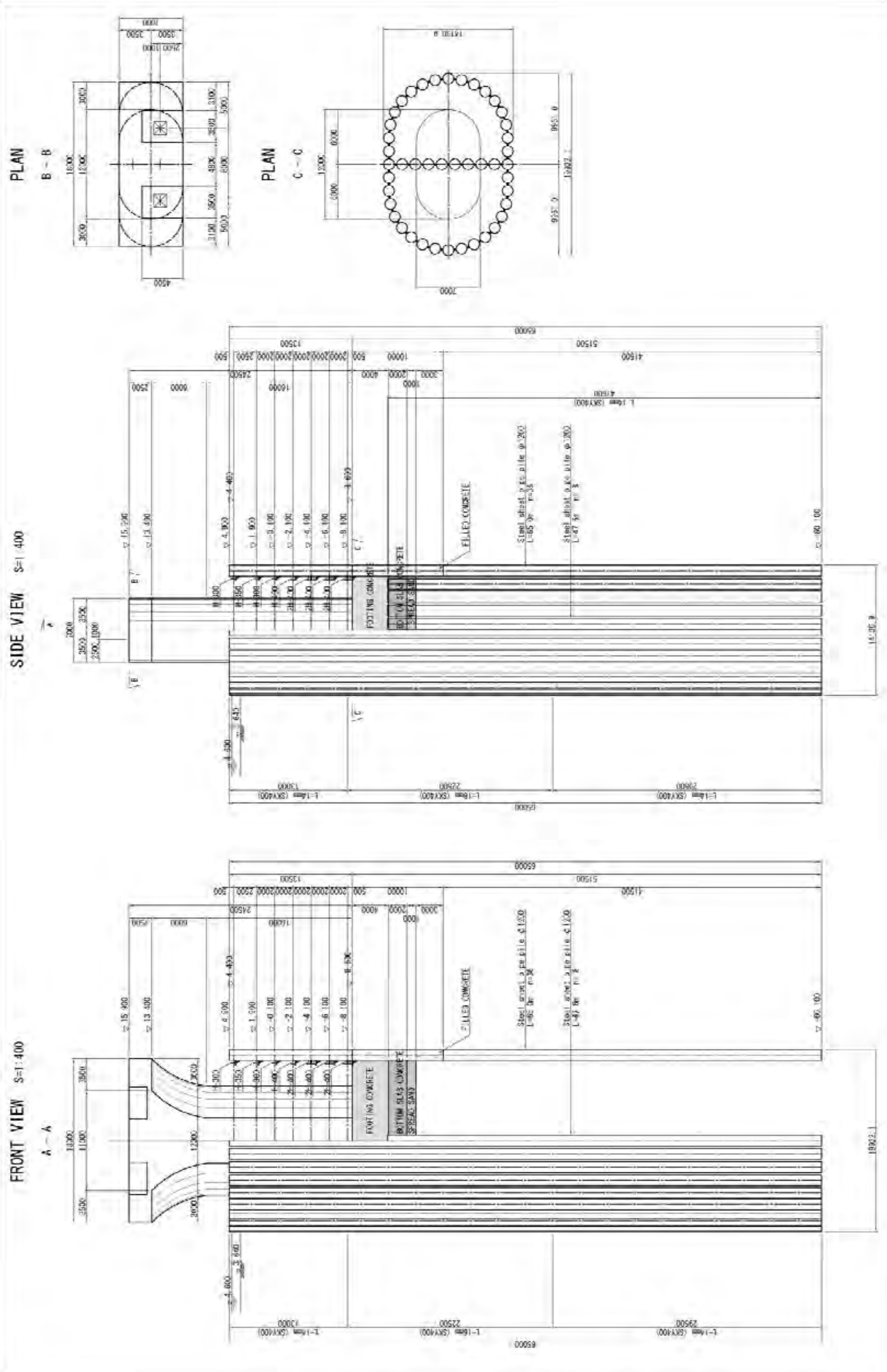


Source: JICA Study Team

Figure 4.2.58 B/D Results for Pylon and Cable Structure

4.2.6.2 Substructure Basic Design Results

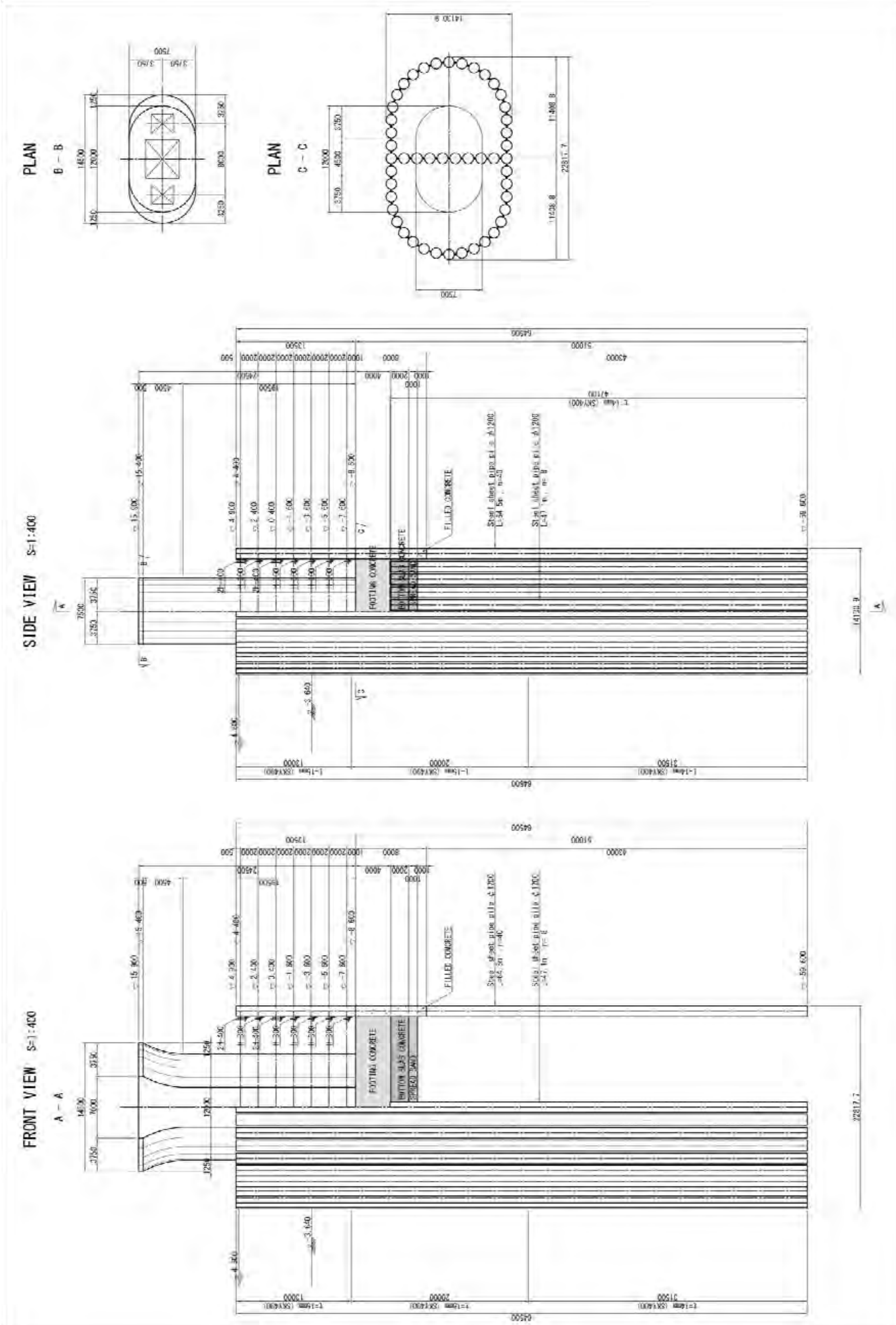
(1) P10 and P13



Source: JICA Study Team

Figure 4.2.59 B/D Results for Substructure of P10 and P13

(2) P11 and P12



Source: JICA Study Team

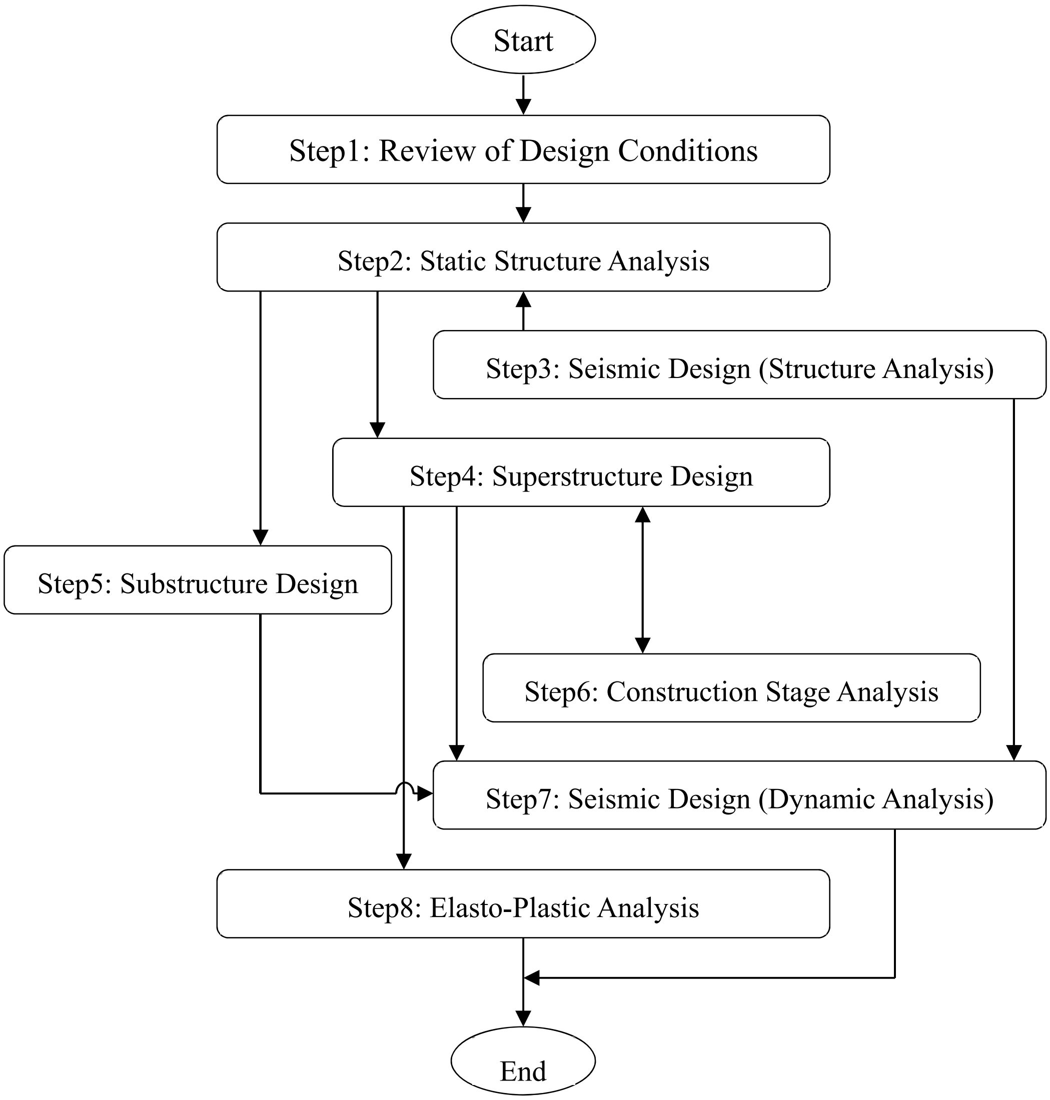
Figure 4.2.60 B/D Results for Substructure of P10 and P13

[Detailed Design Stage]

4.2.7 Summary of Detailed Design

4.2.7.1 Design Flow

The detailed design was carried out through the following steps:

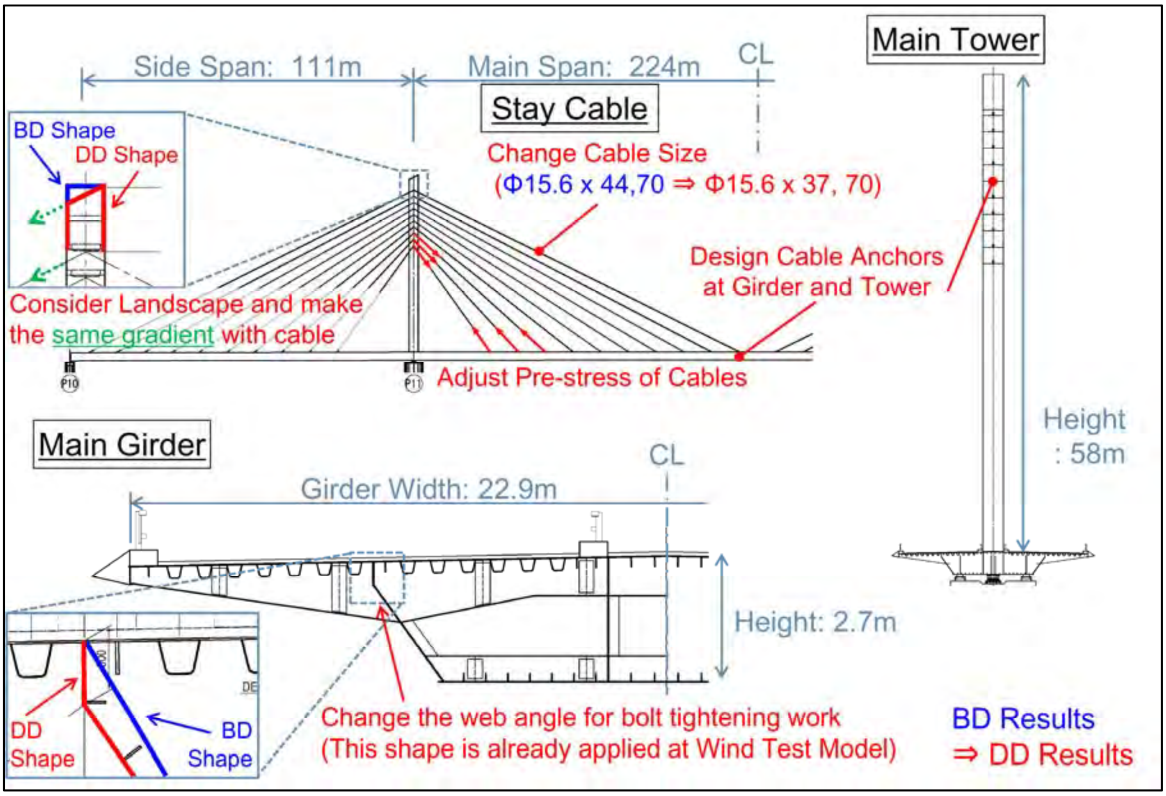


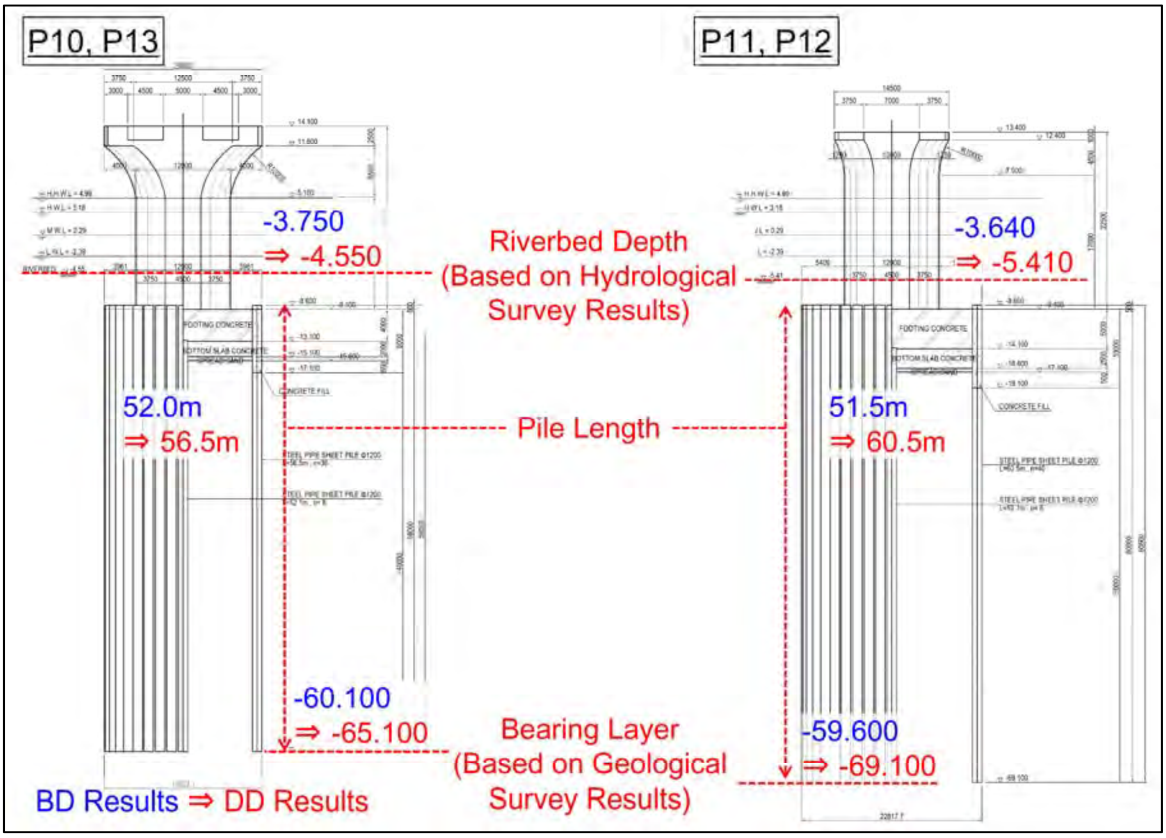
Source: JICA Study Team

Figure 4.2.61 Detailed Design Flow

4.2.7.2 Review of Design Conditions

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



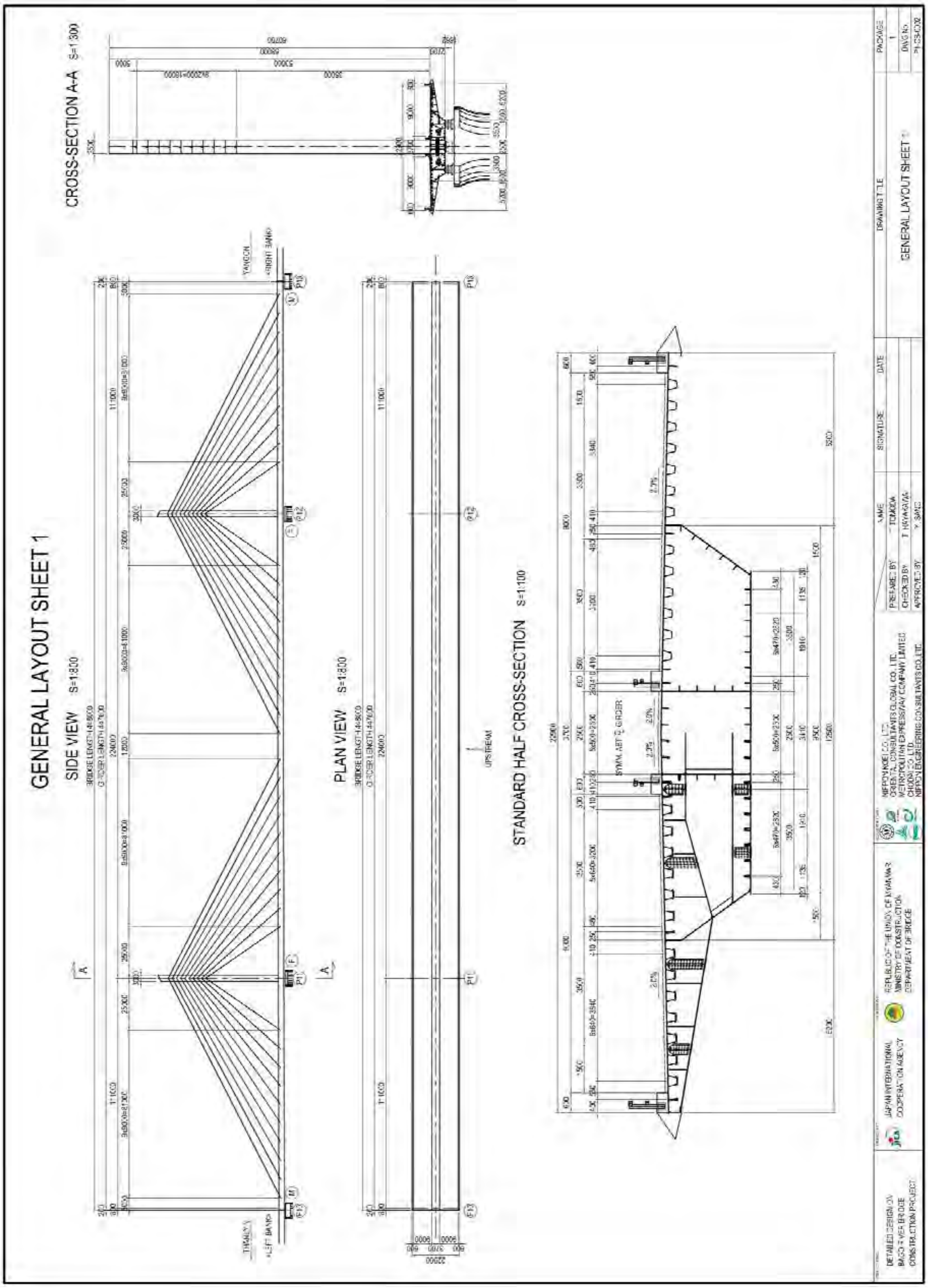


Source: JICA Study Team

Figure 4.2.62 Revised Design Conditions

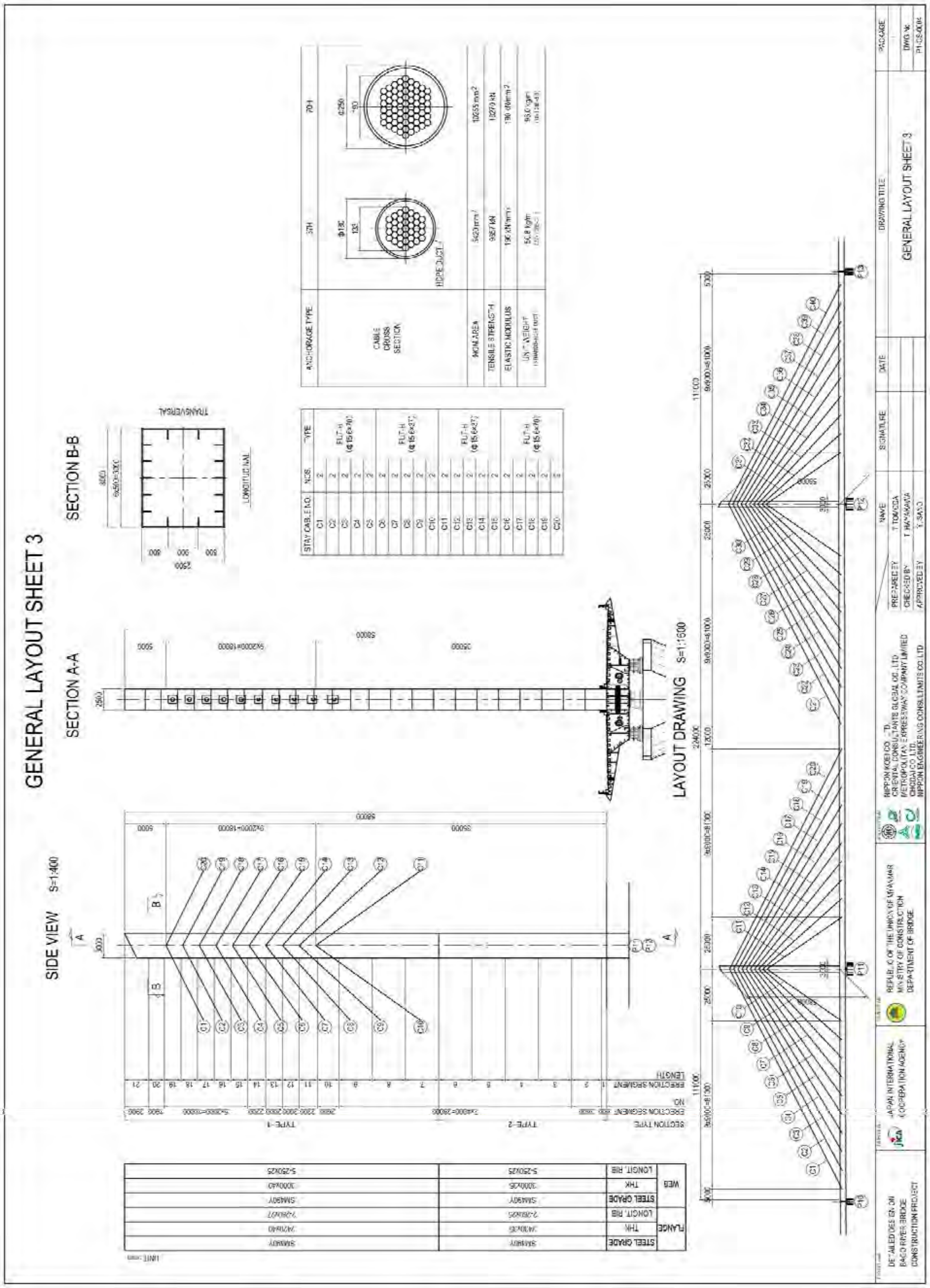
4.2.7.3 Detailed Design Results

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



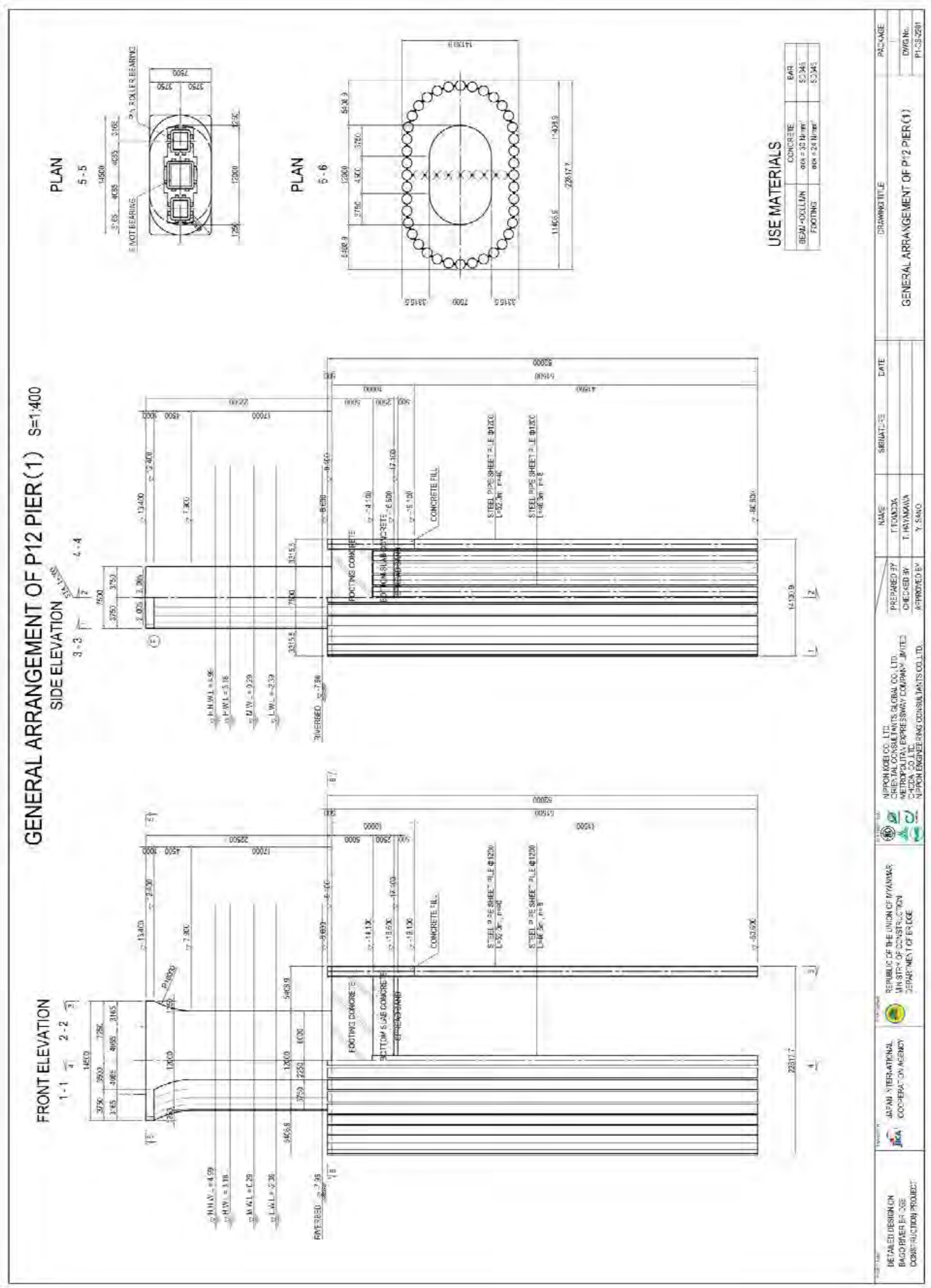
Source: JICA Study Team

Figure 4.2.63 Design Results for Cable-stayed Bridge (Superstructure: Girder)



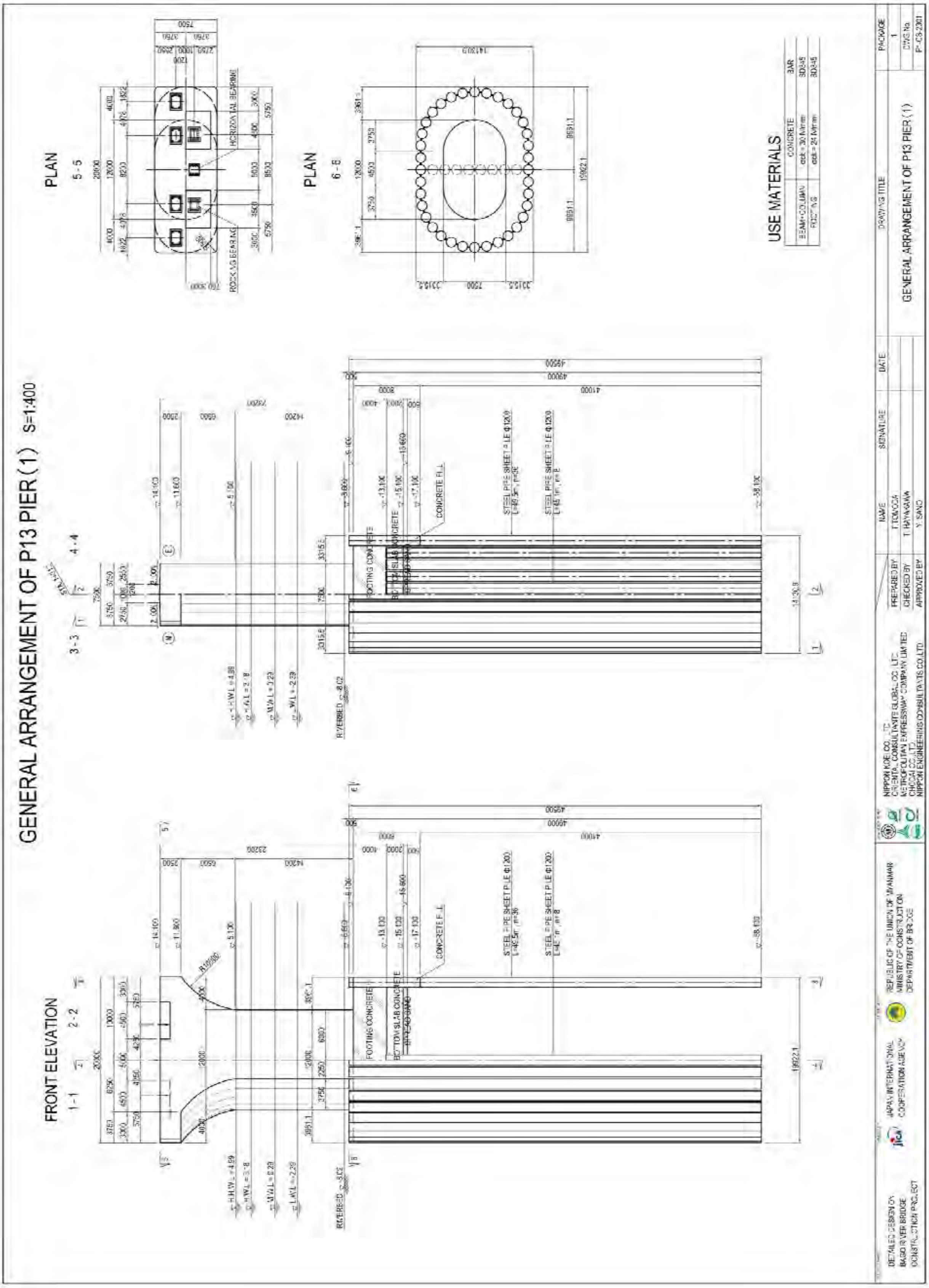
Source: JICA Study Team

Figure 4.2.64 Design Results for Cable-stayed Bridge (Superstructure: Tower and Cable)



Source: JICA Study Team

Figure 4.2.65 Design Results for Cable-stayed Bridge (Substructure: P11, P12)

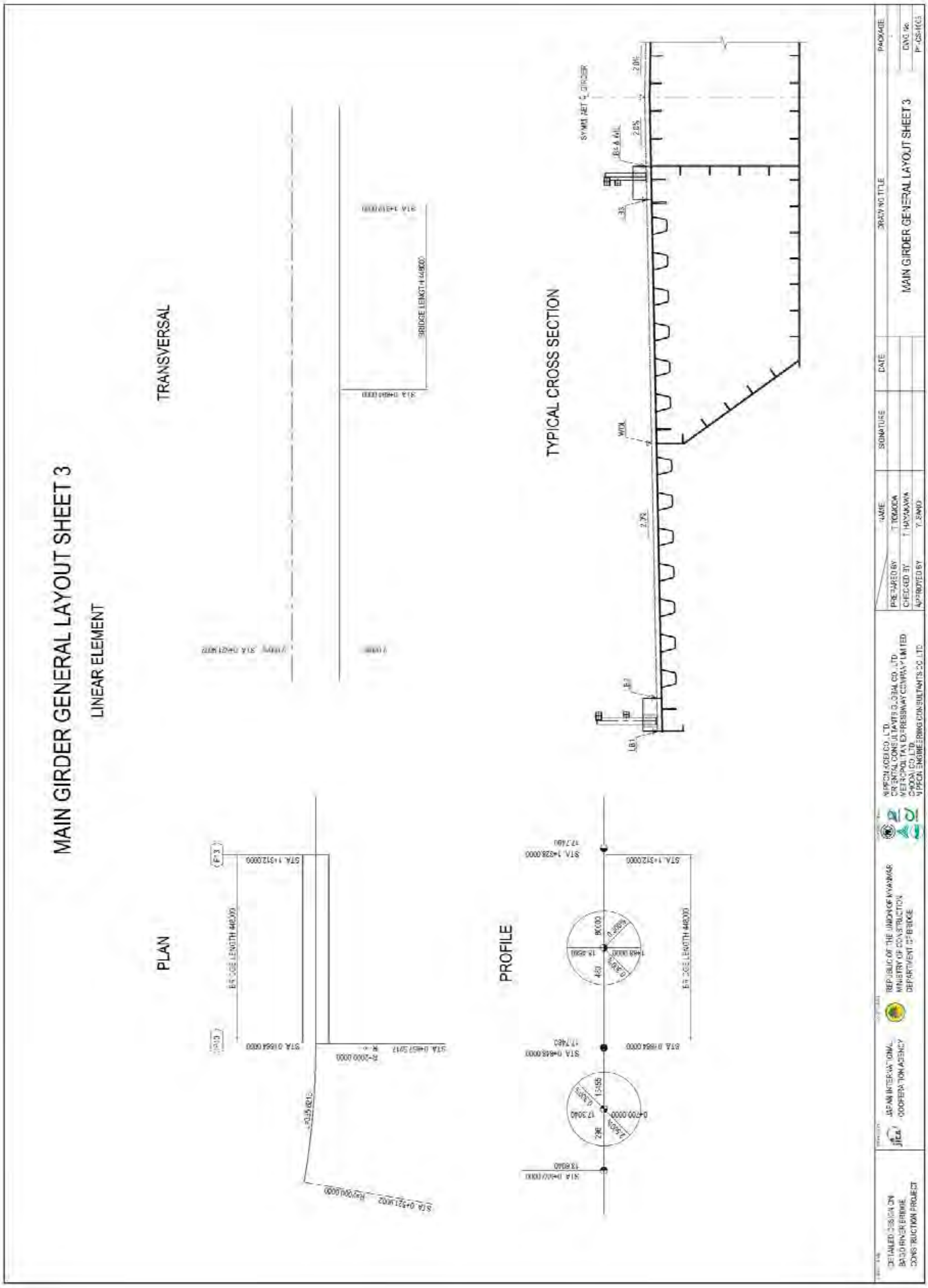


Source: JICA Study Team

Figure 4.2.66 Design Results for Cable-stayed Bridge (Substructure: P10, P13)

4.2.8 Alignment Calculation

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



Source: JICA Study Team

Figure 4.2.67 Alignment Information for Cable-stayed Bridge

4.2.9 Summary of Superstructure Design

4.2.9.1 Design Calculation of Steel Deck

(1) Design Principle

1) Application of Equivalent Lattice Method

The Equivalent Lattice Method was used for the analysis of the steel deck. The Equivalent Lattice Method models the steel deck, stiffened by the attachment of the longitudinal and transverse ribs to the deck plate, as a plane lattice and applies the standard displacement method for analysis.

2) Selected Stiffness for Analysis

a) Bending Stiffness of Material

The bending stiffness of the longitudinal and transverse ribs was obtained from the Specifications for Highway Bridges II Steel Bridges - Table 9.4.2, with consideration of the effective width of the deck plate as a flange. The effective width was calculated by setting the equivalent effective length of the transverse ribs as L at the central section and 2L at the overhanging section.

Furthermore, torsional stiffness shall be taken into consideration for the longitudinal U Ribs. Also, a virtual beam for load distribution shall be created at equivalent intervals of the transverse ribs to incorporate the load distribution created by the deck plate on the longitudinal rib section.

b) Torsional Stiffness of U Ribs

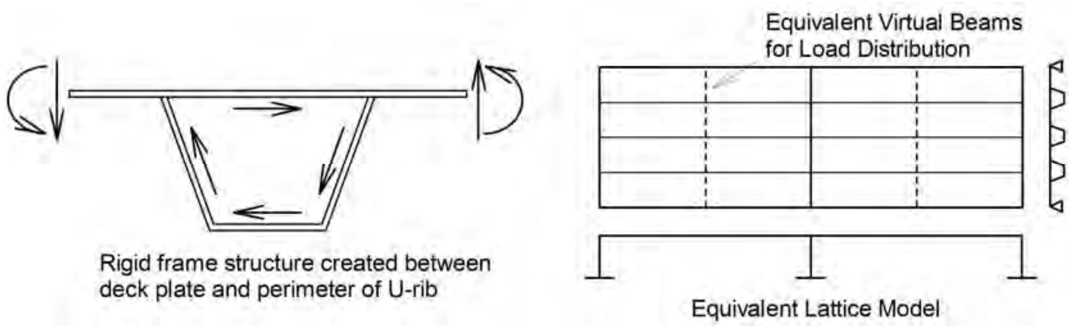
Each longitudinal rib shall be considered as a rod member that does not undergo cross sectional deformation. Hence, the torsional stiffness (with only simple torsion resistance) does not decrease and shall be considered 100% effective as determined by the following equation:

Torsional Stiffness = $4 \cdot A^2 / \{(u/tR) + (a/tP)\}$

A: Enclosed cross sectional area of U Rib
u: Expanded width of U Rib
a: Upside width of U Rib
tR: Thickness of U Rib
tP: Thickness of deck plate

c) Calculation of Equivalent Virtual Beams for Load Distribution

The virtual beam for load distribution, which provides the load distribution to the longitudinal ribs, shall have an equivalent bending stiffness as a rigid frame structure created between the deck plate and the perimeter of the U Rib. Since this rigid frame structure extends along the longitudinal direction, the equivalent second moment of area for unit length is determined first, and in the lattice model, a load distribution beam is created at every interval of the transverse rib where the bending stiffness shall be concentrated.



Source: JICA Study Team

Figure 4.2.68 Virtual Equivalent Load Distribution Beam

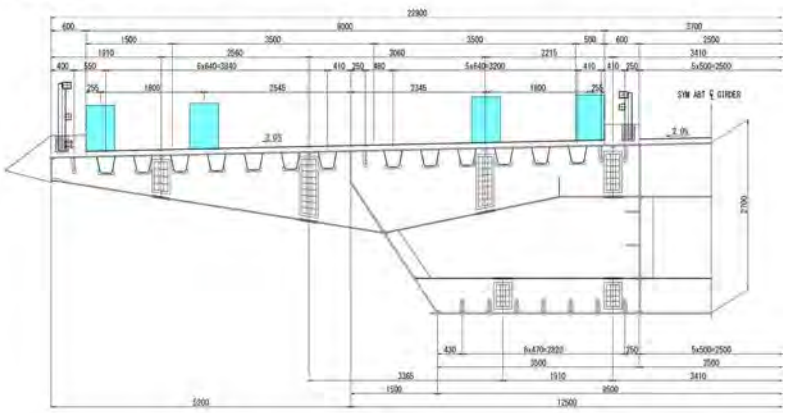
3) Section Force through Analysis of Influence Line

The maximum and minimum section forces for every member of the longitudinal and transverse ribs are calculated by analyzing the effect of the influence line at every point.

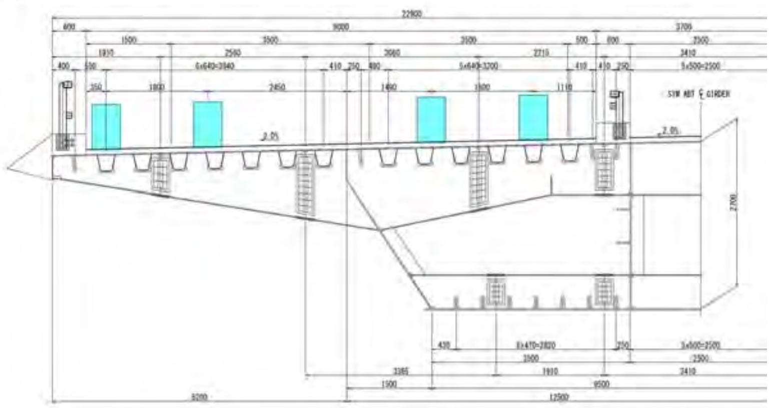
4) AASHTO Configuration of Live Load

The AASHTO Design Live Load was considered as the design load of the steel deck. Based on AASHTO, more severe live load of Design Truck or Design Tandem shall be applied, while tire contact area is 250 mm (length) x 510 mm (width). The design load on the steel deck was set as shown in the figure below.

<Live Load Position-1>



<Live Load Position-2>



Source: JICA Study Team

Figure 4.2.69 AASHTO Configuration of Live Load

(2) Design Results

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

Table 4.2.28 Design Results for Steel Deck (Transverse Rib)

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

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

Source: JICA Study Team

Table 4.2.29 Design Results for Steel Deck (Bracket)

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

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

Source: JICA Study Team

Table 4.2.30 Design Results for Steel Deck (Longitudinal Rib)

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

Source: JICA Study Team

4.2.9.2 Design Calculation for Main Girder

(1) Design Principle

1) Design Section Force

The section force determined by the static structural analysis for Case 1-6 (Refer to Section 4.2.3.7) shall be the design section force. The design force used shall be a factor of the ordinary load conditions.

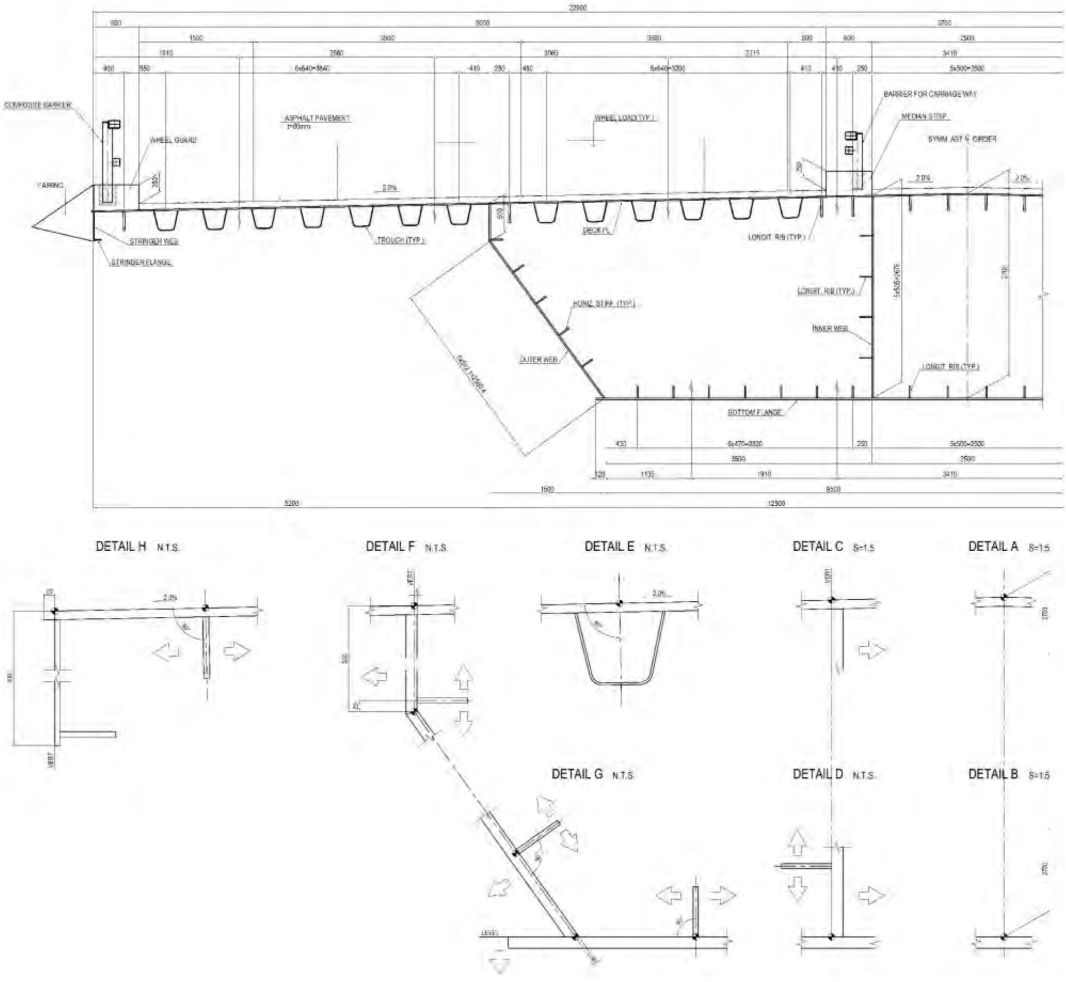
Table 4.2.31 Load Case

Case	Description
Case 1	Dead Load
Case 2	Live Load
Case 3	Influence of Temperature Change
Case 4	Wind Load
Case 5	Earthquake
Case 6	Pre-stress

Source: JICA Study Team

2) Design Cross Section

The main girder cross section and plate joint directions are as shown in the figure below.



Source: JICA Study Team

Figure 4.2.70 Cross Section of Main Girder

(2) Effective Width

The effective width against bending along the horizontal axis for the steel deck and flange is as given below.

Table 4.2.32 Effective Width of Main Girder

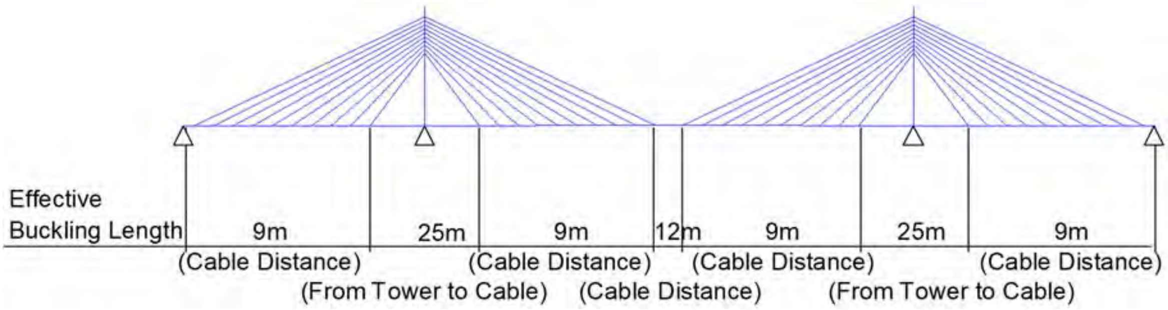
(mm)

Section		L	ℓ	Interval	b	b/ℓ	λ	Eq. in JSHB
Section-1	U-Flg1	92000	92000	5200	5200	0.057	5127	(11.3.1)
	U-Flg2	92000	92000	5000	2500	0.027	2500	(11.3.1)
	U-Flg3	92000	92000	2500	1250	0.014	1250	(11.3.1)
	L-Flg	92000	92000	3500	1750	0.019	1750	(11.3.1)
	Web1	46000	46000	2559	1280	0.028	1280	(11.3.1)
	Web2	46000	46000	2659	1330	0.029	1330	(11.3.1)
Section-2	U-Flg1	92000	92000	5200	5200	0.057	5127	(11.3.1)
	U-Flg2	92000	92000	5000	2500	0.027	2500	(11.3.1)
	U-Flg3	92000	92000	2500	1250	0.014	1250	(11.3.1)
	L-Flg	92000	92000	3500	1750	0.019	1750	(11.3.1)
	Web1	104000	104000	2559	1280	0.012	1280	(11.3.2)
	Web2	104000	104000	2659	1330	0.013	1330	(11.3.2)
Section-3	U-Flg1	32000	32000	5200	5200	0.163	3421	(11.3.2)
	U-Flg2	32000	32000	5000	2500	0.078	2094	(11.3.2)
	U-Flg3	32000	32000	2500	1250	0.039	1178	(11.3.2)
	L-Flg	32000	32000	3500	1750	0.055	1571	(11.3.2)
	Web1	104000	104000	2559	1280	0.012	1280	(11.3.2)
	Web2	104000	104000	2659	1330	0.013	1330	(11.3.2)
Section-4	U-Flg1	115000	115000	5200	5200	0.045	5200	(11.3.1)
	U-Flg2	115000	115000	5000	2500	0.022	2500	(11.3.1)
	U-Flg3	115000	115000	2500	1250	0.011	1250	(11.3.1)
	L-Flg	115000	115000	3500	1750	0.015	1750	(11.3.1)
	Web1	146000	146000	2559	1280	0.009	1280	(11.3.1)
	Web2	146000	146000	2659	1330	0.009	1330	(11.3.1)

Source: JICA Study Team

(3) Effective Buckling Length

The main girder shall not be analyzed for overall buckling except in the vicinity of the tower. The effective buckling length in the horizontal and vertical plane for the main girder near the tower is shown in the figure below.



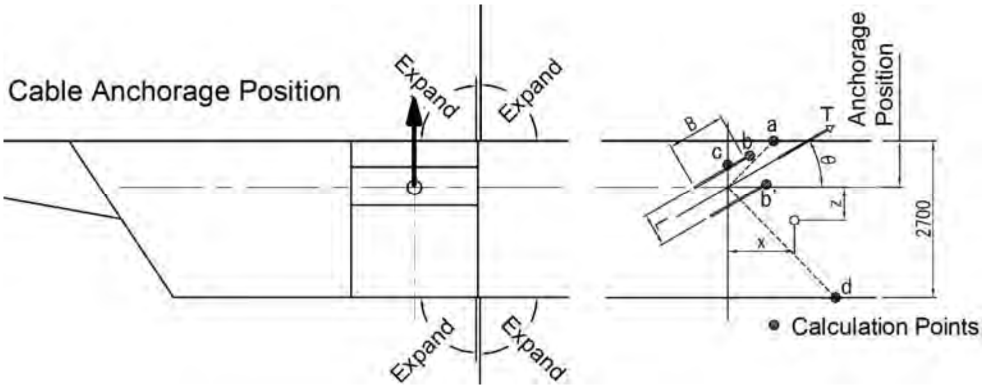
Source: JICA Study Team

Figure 4.2.71 Effective Buckling Length of Main Girder

(4) Additional Stress

1) Additional Stress at Cable Anchorage Member

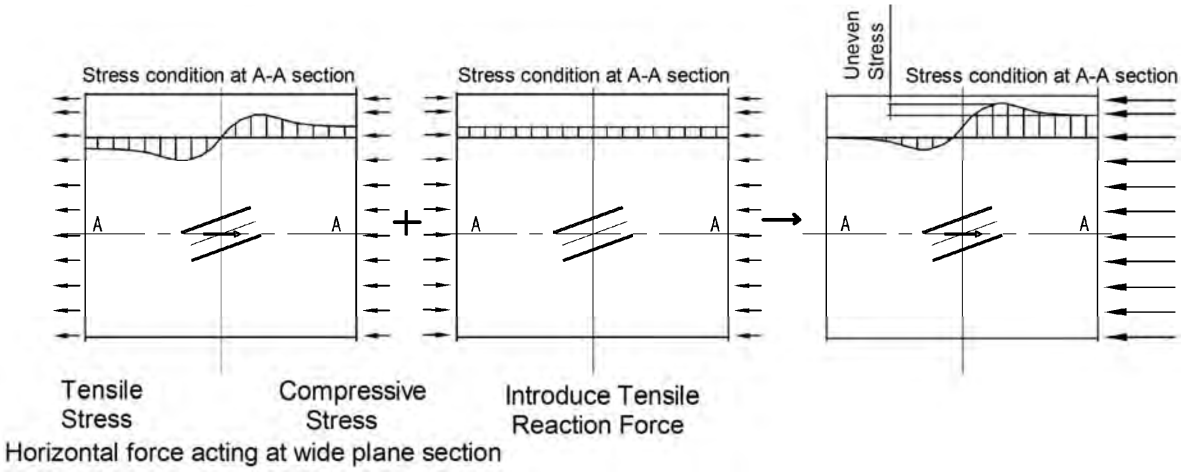
While the cable propagates energy through the cable anchorage girder to the main girder web, it propagates through the cable anchorage location on the web, a comparatively localized point, causing an uneven distribution of stress in the main girder web. Therefore, the uneven distribution of stress at the cable anchorage location is verified with a calculation model which considers the application of reaction force from the cable anchorage girder on the surface which consists of expanded top and bottom flange at the cable anchorage position on the web.



Source: JICA Study Team

Figure 4.2.72 Stress Analysis Model of Cable Anchorage Position

In the horizontal direction, an adjustment to stress as shown below is necessary because the main girder has not been constrained at the opposite of the cable extending direction.



Source: JICA Study Team

Figure 4.2.73 Adjustment of Horizontal Component Force

a) Evaluation of Total Stress with Additional Stress

The total value of the main structure stress and additional stresses was evaluated. In the additional stresses, stress caused by anchorage position between diaphragms and uneven stress at cable anchorage were included.

As shown in the table below, the total stress was less than the allowable value in all sections.

Table 4.2.33 Evaluation of Total Stress (N/mm^2)

σ : Compression +

		σM	σD	σs1	σs2	Σσ	σcal	Σσ<=σcal
C5	U.Flg	63.8	3.6	14.9	-5.8	82.3	131.6	OK
	Web	25.1	-	51.5	51.5	76.6	157.0	OK
	L.Flg	97.6	8.3	4.5	1.0	106.9	146.9	OK
C6	U.Flg	67.9	2.3	8.6	-3.9	78.8	131.6	OK
	Web	28.3	-	51.7	51.7	80.0	158.0	OK
	L.Flg	92.8	5.9	2.5	0.8	99.5	102.1	OK
C9	U.Flg	52.6	3.2	9.4	-5.5	65.2	131.6	OK
	Web	35.5	-	41.2	41.2	76.7	146.0	OK
	L.Flg	70.9	8.1	1.8	1.9	80.9	102.1	OK
C11	U.Flg	53.4	3.7	8.4	-5.7	65.5	131.6	OK
	Web	41.9	-	27.2	27.2	69.1	141.0	OK
	L.Flg	75.6	9.6	1.3	2.3	87.5	102.1	OK
C16	U.Flg	55.5	3.8	15.4	-5.9	74.7	131.6	OK
	Web	29.0	-	53.0	53.0	82.0	153.0	OK
	L.Flg	64.2	9.7	4.6	1.1	75.0	102.1	OK

σM : Compressive stress due to main structure effect.
(At web position: the stress caused by axial force only)

σD : Additional stress caused by anchorage position between diaphragms)

σs1: Maximum uneven stress at anchorage
σs2: Uneven stress at anchorage position (Transverse rib position)

Source: JICA Study Team

2) Analysis of Biaxial Stress Condition

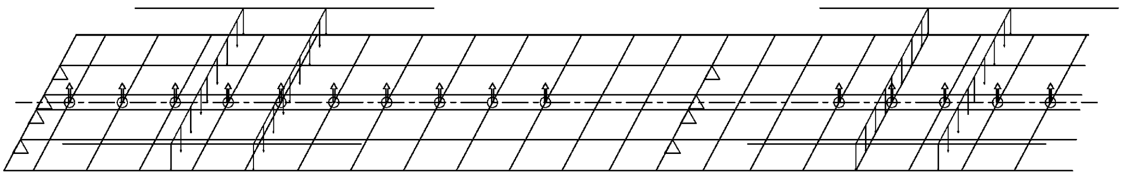
As the main girder is suspended by cable at the center of the cross section, arching upwards along the transverse direction, the bottom flange undergoes compression. For load conditions, which create compressive stress along the bottom flange in the longitudinal direction, a state where the bottom flange is affected by biaxial compressive stress occurs. Buckling is more likely to occur in this state than under uniaxial stress conditions and therefore, another evaluation is required.

The evaluation shall be performed by calculating the stress along the transverse direction and determining the pertinent allowable stress along the longitudinal direction.

a) Analysis along the Transverse Direction

By considering the main girder to be a lattice structure comprised of the vertical girder, the web, the diaphragms and the brackets, stress in the transverse direction is determined through the lattice analysis.

The analysis shall be performed for dead and live load conditions. The effect of the cable shall be considered by applying the vertical component of the tension in the cable as load.



Source: JICA Study Team

Figure 4.2.74 Lattice Analysis Model

b) Result of Lattice Analysis

The results of the lattice analysis are shown in the table below.

Table 4.2.34 Stress at Bottom Flange of Crossbeam

Section		Bending Moment		Bot. Flange Thickness	Bot. Flange Stress	
		M1	M2		σM1	σM2
①	C1	-5632	-3993	14	44.4	31.5
	C2	-8555	-6954	14	63.0	51.2
	C5	-8159	-6472	14	60.0	47.6
②	C8	-7571	-6176	14	55.7	45.4
③	C9	-7888	-6407	11	71.0	57.7
④	C12	-7971	-6438	11	71.8	58.0
⑤	C16	-8334	-6555	11	75.0	59.0
	C20	-5902	-4594	11	53.1	41.4

M1: Bending moment of Cross Beam at center of the middle cell

M2: Bending moment of cross beam at inner web

Source: JICA Study Team

c) Allowable Compressive Stress along the Longitudinal Direction

The bottom flange buckling evaluation under the biaxial compressive stress condition was performed through the evaluation of stress in the transverse direction and allowable stress in the longitudinal direction.

In the bottom flange, there are two values of allowable compressive stress in the longitudinal direction, i.e., in the inner cell section and in the outer cell section. The allowable compressive stress in the inner cell is smaller than that in the outer cell; therefore, the evaluation was performed for the inner cell section.

The calculation result shows that the buckling safety ratio ν is bigger than 1.7, and the safety for biaxial buckling is ensured.

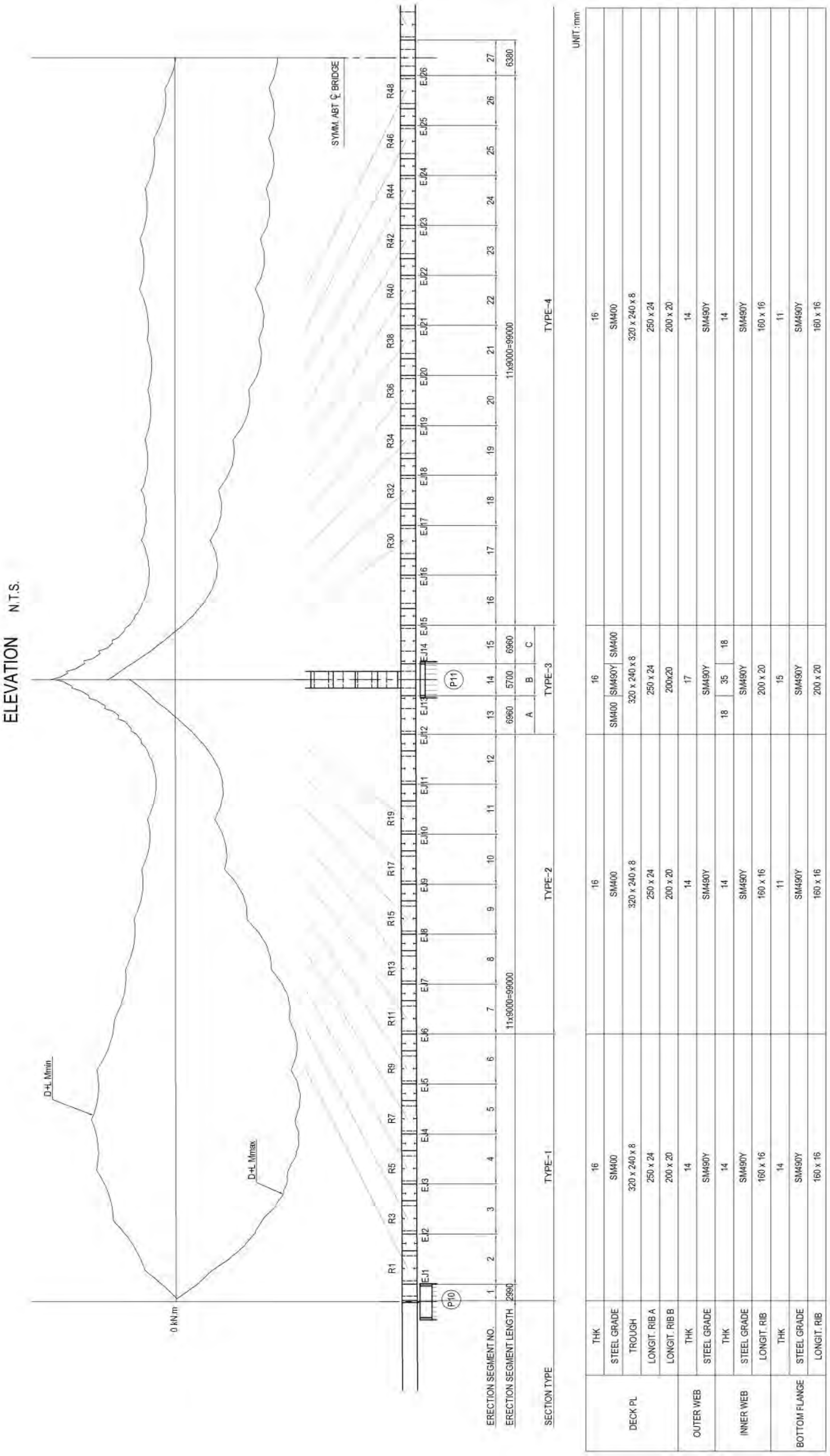
Table 4.2.35 Allowable Compressive Stress in Longitudinal Direction

Thickness t mm	Width B mm	Trans. Rib Distance L mm	Longi. Rib Distance b mm	Longi. Rib Section br * tr	Trans. Direction σy N/mm2	Longi. Allowable Value σcal N/mm2	ν>=1.7 for σx=σcal
14 (C1～C5)	2500	2250	500	160*16	32.3	147	1.76
	3500		470			157	
11 (C6～C10)	2500	2250	500	160*16	36.4	102	2.43
	3500		470			115	
11 (C11～C15)	2500	2250	500	160*16	36.5	102	2.42
	3500		470			115	
11 (C16～C20)	2500	2250	500	160*16	37.2	102	2.41
	3500		470			115	

Source: JICA Study Team

(5) Calculation Results for Cross Section of Main Girder

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



Source: JICA Study Team

Figure 4.2.75 Cross Sectional Diagram of Main Girder

SECTION TYPE		TYPE-1							TYPE-2					
SECTION		S1	EJ1	EJ2	EJ3	EJ4	EJ5	EJ6	EJ7	EJ8	EJ9	EJ10	EJ11	EJ12
DECK	σ	33.6	38.1	48.7	52.3	-57.1	-61.5	-67.3	-65.4	-60.1	-54.2	-50.8	-56.9	-88.9
	σa	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0
	LOAD CASE	D+L+T	D+L+T	D+L+T	D+L+T	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+ETR	D+ETR
BOTTOM	σ	-8.7	-17.7	-47.4	-62.7	-93.5	-26.1	-27.0	-15.6	-19.2	-35.0	-39.5	-21.0	-88.8
FLANGE	σa	146.9	146.9	146.9	146.9	146.9	146.9	102.1	102.2	102.2	102.1	102.1	102.4	102.1
	LOAD CASE	D+L+T	D+L+T	D+L+T	D+L+T	D+L+T	D+ELG	D+ELG	D+WgTR	D+WgTR	D+ELG	D+ELG	D+WgTR	D+L
WEB	τ	42.6	42.3	32.1	25.8	22.4	21.2	19.6	24.3	26.6	26.6	26.1	25.1	45.1
	τa	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0
	COMBINED STRESS	0.13	0.12	0.09	0.12	0.22	0.10	0.11	0.12	0.12	0.11	0.10	0.10	0.27
	LOAD CASE	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L

UNIT : kN・m, kN

TYPE-3				TYPE-4											
EJ13	P11L	P11R	EJ14	EJ15	EJ16	EJ17	EJ18	EJ19	EJ20	EJ21	EJ22	EJ23	EJ24	EJ25	EJ26
-71.8	-74.0	-73.6	-70.2	-62.3	-56.6	-55.2	-55.7	-57.4	-58.4	-56.6	-50.2	-46.4	-46.3	-46.1	-45.6
140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0	140.0
D+ETR	D+ETR	D+ETR	D+ETR	D+ETR	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L+T	D+ETR	D+ETR	D+ETR
-112.9	-130.1	-122.7	-107.6	-86.7	-15.6	-19.4	-18.0	-15.3	-12.6	-12.5	-23.9	-21.5	-17.1	-10.5	-16.7
157.9	157.9	157.9	157.9	102.1	102.4	102.2	102.1	102.2	102.4	102.5	102.1	102.1	102.1	102.1	118.5
D+L	D+L	D+L	D+L+T	D+L+T	D+WgTR	D+WgTR	D+WgTR	D+WgTR+T	D+WgTR+T	D+WgTR+T	D+ELG	D+ELG	D+ELG	D+ELG	D+ETR
49.8	50.3	57.3	56.4	53.5	33.9	34.6	34.6	33.1	29.7	24.0	25.8	26.7	27.6	29.0	31.4
120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0
0.40	0.50	0.50	0.39	0.28	0.14	0.14	0.14	0.14	0.13	0.10	0.08	0.08	0.07	0.17	0.19
D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L

SECTION TYPE		TYPE-1					TYPE-2				
SECTION		R1	R3	R5	R7	R9	R11	R13	R15	R17	R19
BOTTOM	σf	-36.0	-71.0	-96.8	-100.0	-94.9	-29.2	-30.7	-34.9	-72.5	-69.9
FLANGE	σxa	130.5	130.5	130.5	130.5	130.5	100.7	100.7	106.7	106.7	106.7
	LOAD CASE	D+L	D+L	D+L+T	D+L	D+L+T	D+ELG	D+ELG	D+ELG	D+L	D+L
WEB	τw'	46.1	36.0	31.0	27.4	30.5	26.6	30.0	32.3	32.5	32.4
	τxz	70.8	75.1	75.1	75.1	75.1	41.1	41.1	41.1	41.1	41.1
	τw=τw'+τxz	116.9	111.1	106.1	102.5	105.6	67.7	71.1	73.4	73.6	73.5
	τa	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0
	LOAD CASE	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L

UNIT : kN・m, kN

TYPE-3	TYPE-4									
--	R30	R32	R34	R36	R38	R40	R42	R44	R46	R48
--	-23.2	-36.0	-20.0	-16.8	-15.0	-28.6	-26.8	-23.0	-17.4	-9.3
--	98.3	98.3	98.3	98.3	98.3	90.6	90.6	90.6	90.6	90.6
--	D+WgTR	D+ELG	D+WgTR+T	D+WgTR+T	D+WgTR+T	D+ELG	D+ELG	D+ELG	D+ELG	D+ELG
--	40.8	40.7	39.1	35.7	31.7	34.2	32.3	33.1	34.4	36.4
--	40.6	40.6	40.6	40.6	40.6	63.9	63.9	63.9	63.9	63.9
--	81.4	81.3	79.7	76.3	72.3	98.1	96.2	97.0	98.3	100.3
--	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0	120.0
--	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L	D+L

D : DEAD LOAD
L : LIVE LOAD
T : Temperature
WgTR : Wing Transverse
ELG : Earth Quake Longitudinal
ETR : Earth Quake Transverse

Source: JICA Study Team

Figure 4.2.76 Calculation Results for Cross Section of Main Girder

4.2.9.3 Design Calculation for Main Tower

(1) Design Principle

1) Design Section Force

The section force determined by the static structural analysis for Case 1-6 (Refer to 4.2.3.11) shall be the design section force. The design force used shall be a factor of the ordinary load conditions.

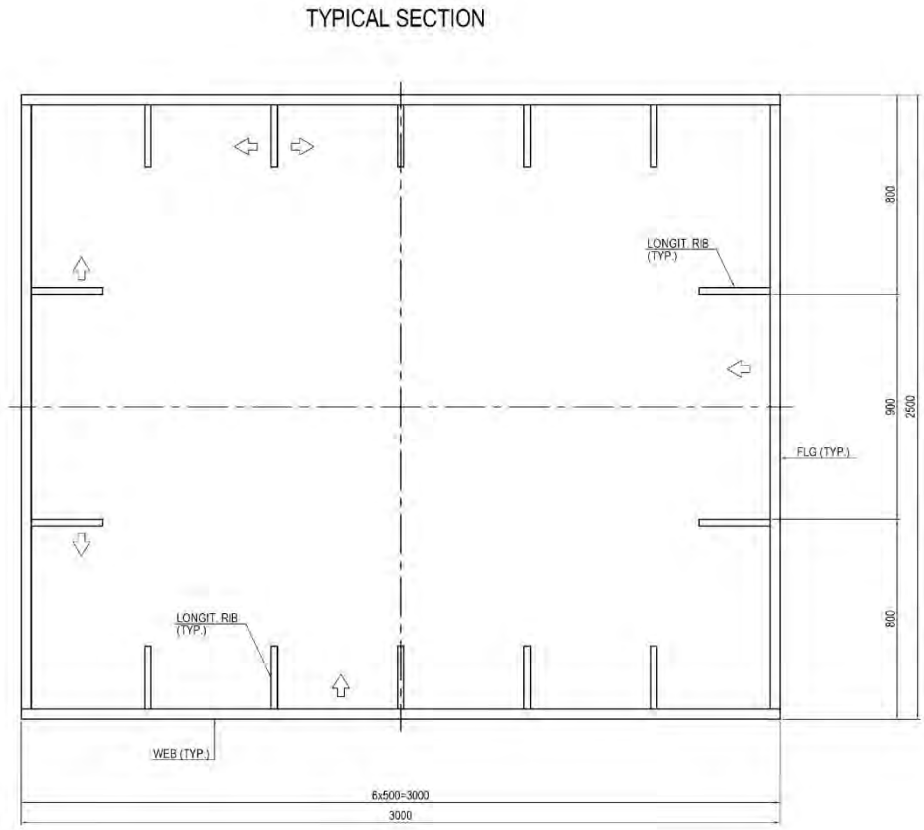
Table 4.2.36 Loading Cases

Case	Description
Case 1	Dead Load
Case 2	Live Load
Case 3	Influence of Temperature Change
Case 4	Wind Load
Case 5	Earthquake
Case 6	Pre-stress

Source: JICA Study Team

2) Design Section

The main tower cross section and plate joint directions are shown in the figure below.



Source: JICA Study Team

Figure 4.2.77 Design Cross Section

(2) Effective Width

The effective width of the flange and web of the main tower is as follows:

Table 4.2.37 Effective Width of Main Tower

(mm)

Section		L	l	Interval	b	b/l	λ	Eq. in JSHB
Tower	Flg	53900	53900	2500	1250	0.023	1250	(10.3.1)
	Web	53900	107800	3000	1500	0.014	1500	(10.3.2)

Source: JICA Study Team

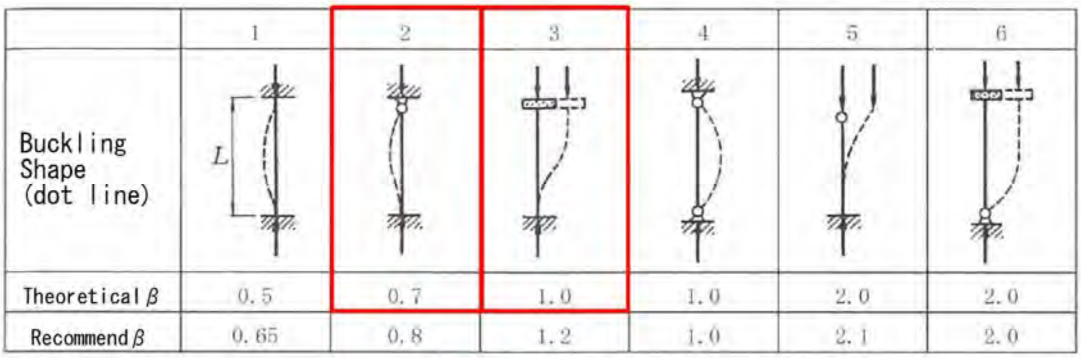
(3) Effective Bucking Length

The effective buckling length of the main tower is as follows:

In-plane direction: 0.7h

Out-of-plane direction: 1.0h

Table 4.2.38 Effective Buckling Length of Column



	1	2	3	4	5	6
Buckling Shape (dot line)						
Theoretical β	0.5	0.7	1.0	1.0	2.0	2.0
Recommend β	0.65	0.8	1.2	1.0	2.1	2.0

Source: JSHB Part II, 2012

Table 4.2.39 Effective Buckling Length of Main Tower

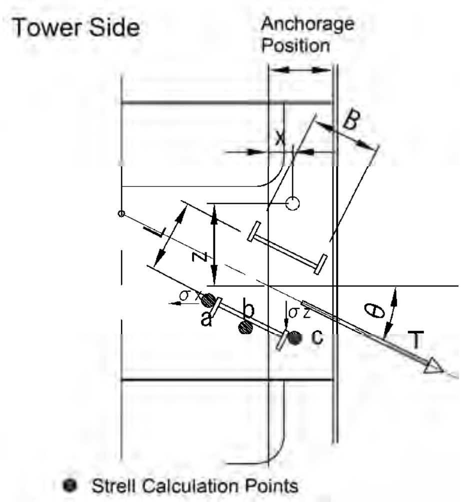
	Structure Length h (m)	Coefficient	Effective Buckling Length (m)
In-plane	53.900	0.700	37.730
Out-of-plane	53.900	1.000	53.900

Source: JICA Study Team

(4) Additional Stress

1) Additional Stress at Cable Anchorage Member

While the cable propagates energy through the cable anchorage girder to the main tower web, it propagates through the cable anchorage location on the web, a comparatively localized point, causing an uneven distribution of stress in the main tower web same as in the main girder. Therefore, additional stress for tower web shall be considered.



Source: JICA Study Team

Figure 4.2.78 Calculation Points of Stress

a) Evaluation of Additional Stress

Uneven stress and evaluation results at the cable anchorage position are shown in the table below.

Table 4.2.40 Uneven Stress Distribution at Cable Anchorage Position

Position		Stress per 10mm		t (mm)	σx1,z1=	Girder Area	Even Stress	Stress at	Even Stress	Uneven Stress (additional)		
		σx0,σz0	τxz0		σx0,z0*10/t	Ag (m2)	(Left, Right)	Anchorage σx	σn	σx'	σz'	τxz
C1,20 (N70)	a	-206.0	96.5	40	-51.5	0.11	-26.4	-77.9	-52.8	-25.1	-	24.1
	b	-139.1	-108.3	40	-34.8			-61.2		-8.4	-	27.1
	c	38.6	-126.4	40	9.7	0.48	3.1	12.8	6.2	-	0.4	31.6
C5,16 (N70)	a	-182.1	115.7	40	-45.5	0.11	-24.9	-70.4	-49.8	-20.6	-	28.9
	b	-158.2	-75.6	40	-39.6			-64.5		-14.7	-	18.9
	c	66.2	-136	40	16.6	0.70	1.3	17.9	2.6	-	12.7	34.0
C6,15 (N37)	a	-97.4	69.5	40	-24.4	0.11	-13.7	-38.1	-27.4	-10.7	-	17.4
	b	-92.7	-33.6	40	-23.2			-36.9		-9.5	-	8.4
	c	44.3	-78.9	40	11.1	0.70	0.8	11.9	1.6	-	8.7	19.7
C10,11 (N37)	a	-45.6	79	40	-11.4	0.11	-9.7	-21.1	-19.4	-1.7	-	19.8
	b	-93.5	37.8	40	-23.4			-33.1		-13.7	-	9.5
	c	96.7	-70.1	40	24.2	0.70	1.1	25.3	2.2	-	20.9	17.5

Source: JICA Study Team

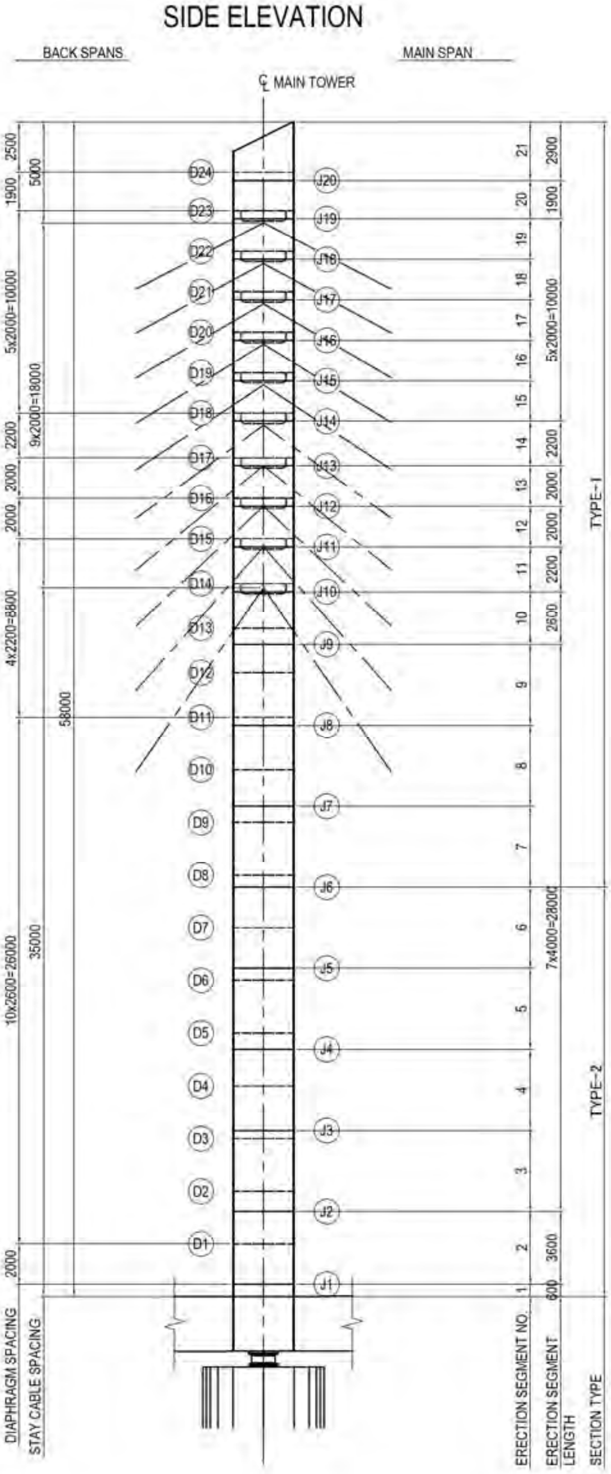
Table 4.2.41 Analysis Result of Uneven Stress Distribution at Cable Anchorage Position

Position		Uneven Stress		Tower Stress		Total				
		σx'	τxz	σf'	τf'	σf	σa	τf	τa	
Section1 J18 (C1,20)	a	25.1	24.1	27.2	7.1	52.3	210	31.2	120	OK
	b	8.4	27.1	27.2	0.7	35.6	210	27.8	120	OK
	c	0.4	31.6	27.2	7.1	27.6	210	38.7	120	OK
Section1 J17-14 (C5,16)	a	20.6	28.9	109.5	6.6	130.1	210	35.5	120	OK
	b	14.7	18.9	109.5	1.7	124.2	210	20.6	120	OK
	c	12.7	34.0	109.5	6.6	122.2	210	40.6	120	OK
Section1 J13 (C6,15)	a	10.7	17.4	125.1	5.3	135.8	210	22.7	120	OK
	b	9.5	8.4	125.1	1.9	134.6	210	10.3	120	OK
	c	8.7	19.7	125.1	5.3	133.8	210	25.0	120	OK
Section1 J12-9 (C10,11)	a	1.7	19.8	154.4	2.6	156.1	210	22.4	120	OK
	b	13.7	9.5	154.4	2.7	168.1	210	12.2	120	OK
	c	20.9	17.5	154.4	2.6	175.3	210	20.1	120	OK

Source: JICA Study Team

(5) Calculation Results for Cross Section of Main Tower

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



UNIT : mm

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

UNIT : kN・m, kN

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

Source: JICA Study Team

Figure 4.2.79 Calculation Results for Cross Section of Main Tower

4.2.9.4 Design Calculation for Cable

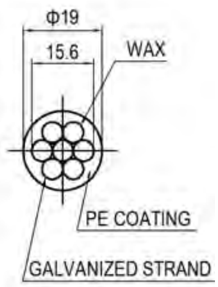
(1) Stay Cable

1) Specifications for Stay Cable

a) Specifications for Strand

Stay cable is composed of strand, which is a set of 7-galvanized strand wire and high-density polyethylene pipe. The specifications for the strand is as follows:

Table 4.2.42 Specifications for Strand

Items	Description
Standard Cross Section	
Nominal Area	146.5 mm^2
Tensile Strength	261 kN
Elastic Modulus	190 kN/mm^2
Unit Weight (Strand + HDPE Coating)	1.288 kg/m

Source: JICA Study Team

b) Cross Section of Stay Cable

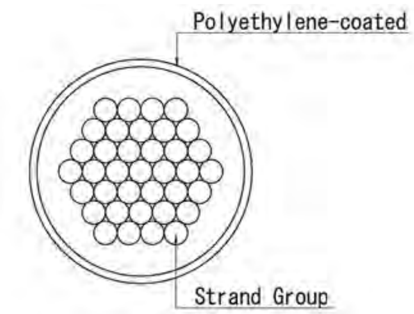
The strands are arranged in a hexagonal pattern in the cross section of the stay cable. The number of strands was decided based on the maximum tension, which is calculated by static analysis.

Table 4.2.43 Characteristics of Stay Cable

Items	Equation
Area (mm^2)	146.5 x N
Unit Weight (kg/m)	1.288 x N + Wp (weight of outer cover pipe)
Yield Point (kN)	222 x N
Tensile Strength (kN)	261 x N
Young's Modulus (kN/mm^2)	190

Note: N: number of strands, Wp: Weight of outer cover pipe (high-density polyethylene pipe)

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.80 Cross Section of Stay Cable

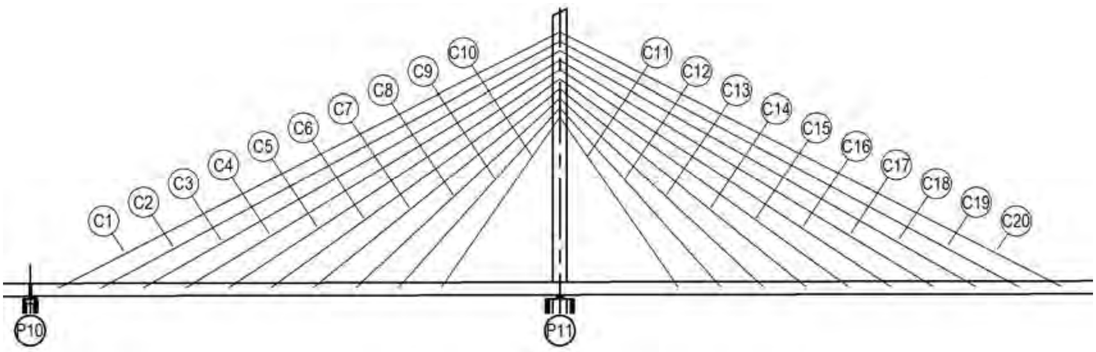
2) Decision of Stay Cable Cross Section

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

Table 4.2.44 Cable Tension and Cross Section

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

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.81 Cable Number

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

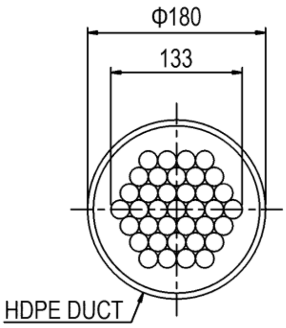
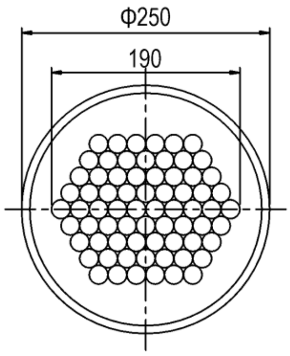
Table 4.2.45 Evaluation of Cable Tension

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

Source: JICA Study Team

The selected cable cross section is as follows:

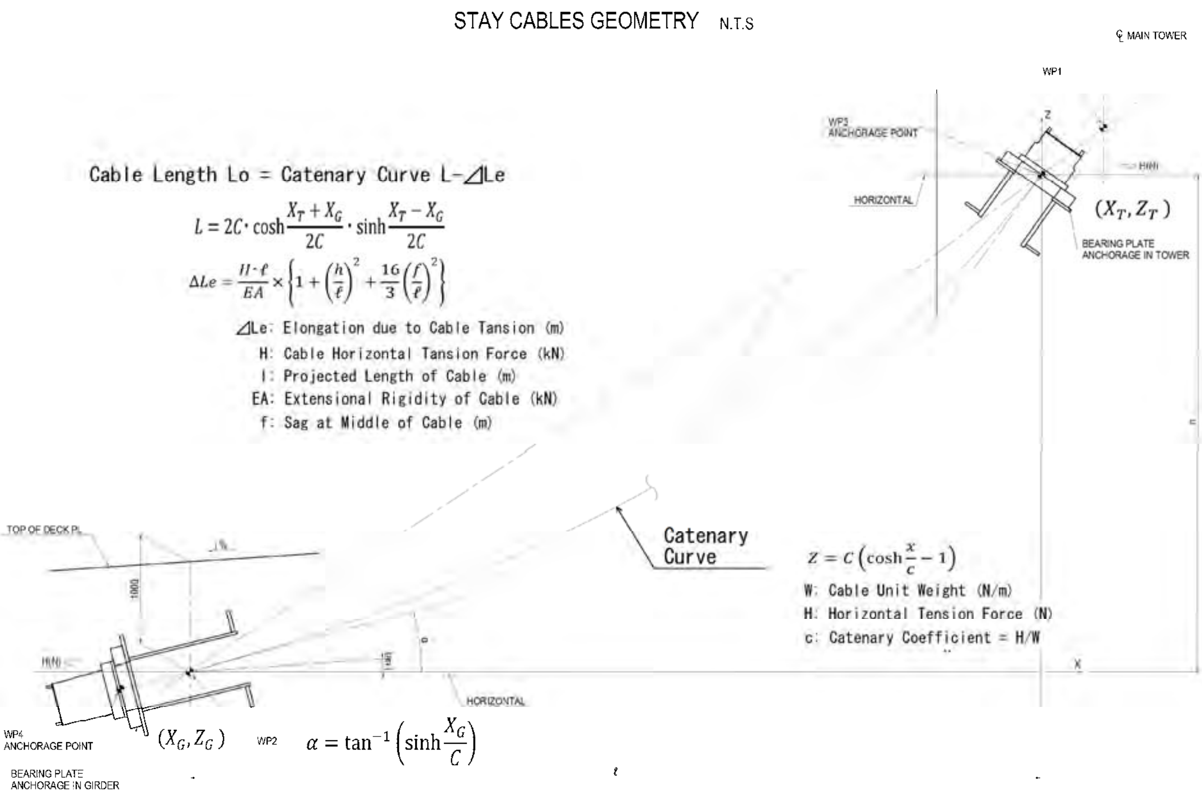
Table 4.2.46 Cross Section of Stay Cable

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

Source: JICA Study Team

(2) Calculation of Stay Cable Length

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



Source: JICA Study Team

Figure 4.2.82 Calculation Method of Catenary Curve

Table 4.2.47 Cable Section and Characteristics

STAY CABLE NO.	X_G (m)	Z_G (m)	X_T (m)	Z_T (m)	ℓ (m)	h (m)	W (N/m)	H (kN)	A (m^2)	f (mm)	α (deg)	⊿Le (m)	L (m)	L_0 (m)
C1	-218.000	-1.297	-112.000	52.922	106.000	54.219	941.400	6093.200	0.010	246.400	26.673	0.410	119.705	119.295
C2	-209.000	-1.273	-112.000	50.922	97.000	52.195	941.400	5346.200	0.010	237.900	27.855	0.336	110.794	110.458
C3	-200.000	-1.250	-112.000	48.922	88.000	50.172	941.400	4678.100	0.010	227.100	29.250	0.275	101.941	101.666
C4	-191.000	-1.228	-112.000	46.922	79.000	48.150	941.400	4317.300	0.010	202.000	30.942	0.236	93.160	92.924
C5	-182.000	-1.207	-112.000	44.922	70.000	46.129	941.400	4405.500	0.010	159.200	33.028	0.223	84.475	84.252
C6	-173.000	-1.187	-112.000	42.922	61.000	44.109	497.900	2534.800	0.005	114.300	35.594	0.224	75.798	75.574
C7	-164.000	-1.168	-112.000	40.922	52.000	42.090	497.900	2652.900	0.005	82.900	38.771	0.218	67.421	67.203
C8	-155.000	-1.150	-112.000	38.922	43.000	40.072	497.900	2655.600	0.005	60.300	42.813	0.204	59.298	59.095
C9	-146.000	-1.133	-112.000	36.922	34.000	38.055	497.900	2453.200	0.005	44.900	48.090	0.180	51.552	51.373
C10	-137.000	-1.117	-112.000	34.922	25.000	36.039	497.900	2024.000	0.005	34.500	55.152	0.149	44.382	44.233
C11	-87.000	-1.047	-112.000	34.922	-25.000	35.969	497.900	2003.300	0.005	34.800	55.098	0.147	44.325	44.178
C12	-78.000	-1.038	-112.000	36.922	-34.000	37.960	497.900	2437.900	0.005	45.100	48.018	0.178	51.481	51.303
C13	-69.000	-1.030	-112.000	38.922	-43.000	39.951	497.900	2672.500	0.005	59.800	42.728	0.204	59.216	59.012
C14	-60.000	-1.023	-112.000	40.922	-52.000	41.944	497.900	2728.100	0.005	80.500	38.680	0.223	67.329	67.106
C15	-51.000	-1.016	-112.000	42.922	-61.000	43.938	497.900	2612.000	0.005	110.800	35.496	0.231	75.698	75.467
C16	-42.000	-1.011	-112.000	44.922	-70.000	45.933	941.400	4591.700	0.010	152.500	32.930	0.232	84.367	84.135
C17	-33.000	-1.007	-112.000	46.922	-79.000	47.928	941.400	4433.800	0.010	196.400	30.836	0.241	93.045	92.804
C18	-24.000	-1.004	-112.000	48.922	-88.000	49.925	941.400	4522.800	0.010	234.600	29.113	0.265	101.819	101.554
C19	-15.000	-1.001	-112.000	50.922	-97.000	51.923	941.400	4800.500	0.010	264.700	27.681	0.301	110.666	110.365
C20	-6.000	-1.000	-112.000	52.922	-106.000	53.922	941.400	5188.300	0.010	289.000	26.472	0.348	119.570	119.222

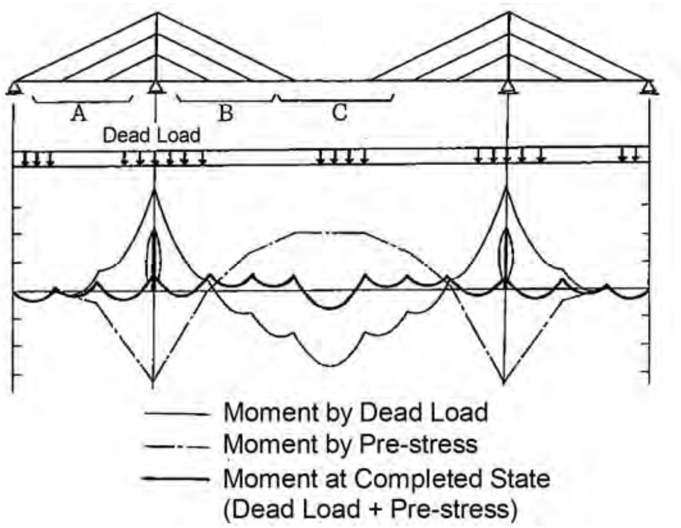
Source: JICA Study Team

4.2.9.5 Study on Cable Pre-stressing Force

(1) Study Overview

For a cable-stayed bridge, a type of bridge where the main girder is supported by diagonally stayed cables from towers, the stress at the main girder and the towers can be adjusted by pre-stressing the cables.

Ordinarily the main girder is subjected to bending moment with a tendency to be subjected to larger amounts around the center and around the main towers when pre-stressing has not been applied to the cable. Therefore, a study on the pre-stressing force in the cables was conducted to optimize the bending moment distribution in the girder and determine the pre-stressing force to be installed in the cable.



Source: JICA Study Team

Figure 4.2.83 Bending Moment for Completed Stage

(2) Design Principle

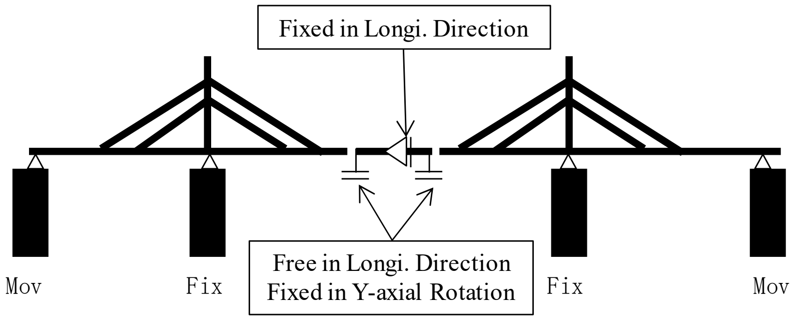
The pre-stressing force in the cables was determined to satisfy the conditions below during the completed stage (D+Ps).

1. The bending moment distribution along the main girder is smoothened.
2. The tower must not be subjected to bending moment during the completed stage.
3. During the final girder closing, the girders do not require any force (closing force, enforcement) → M≒0 at joint

For the purpose of the study, the assumed loading on the structure during the closing state mentioned in item 3 above shall include the loads temporarily created by construction equipment such as cranes.

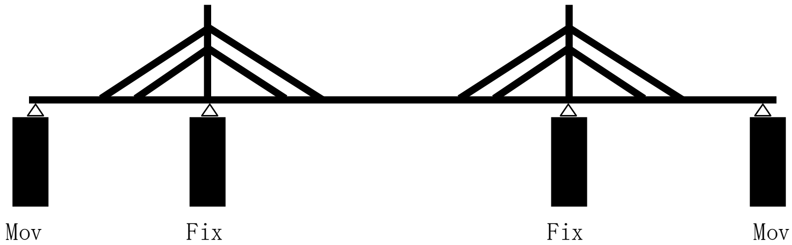
The analysis model for the study is shown below.

< Before Closing Model >



Load Condition: Girder Weight, Tower Weight, Cable Weight, Pier Weight, Fairing, Attached Equipments, Rail for Inspection Car, Rocking Bearing, Pre-stress

< After Closing Model >



Load Condition: All loads (exclude included loads in Before Closing Model)

Source: JICA Study Team

Figure 4.2.84 Overview of Analysis Model

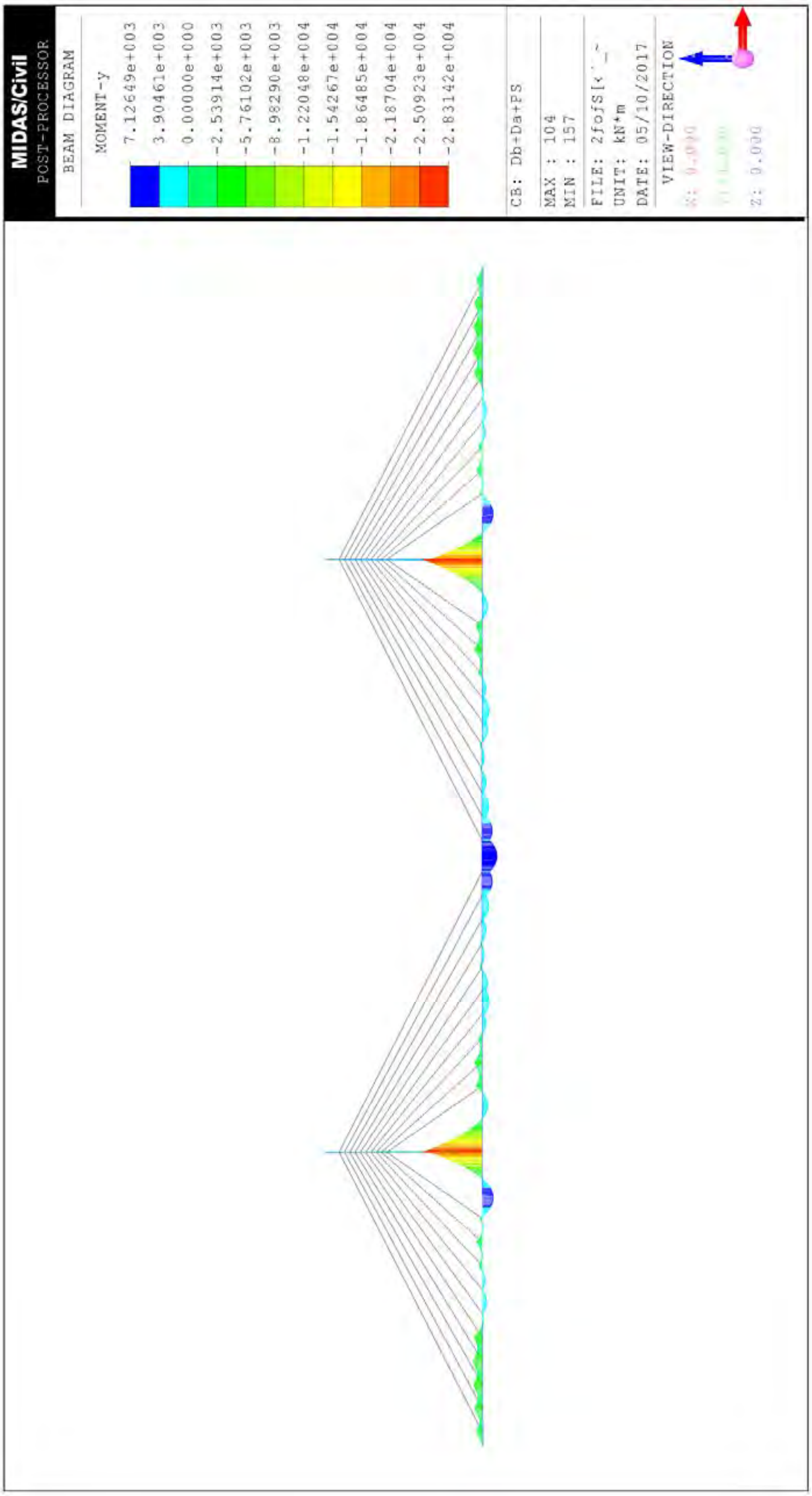
(3) Results of Study

The study results are shown in the table below.

Table 4.2.48 Study Results for Cable Pre-stressing

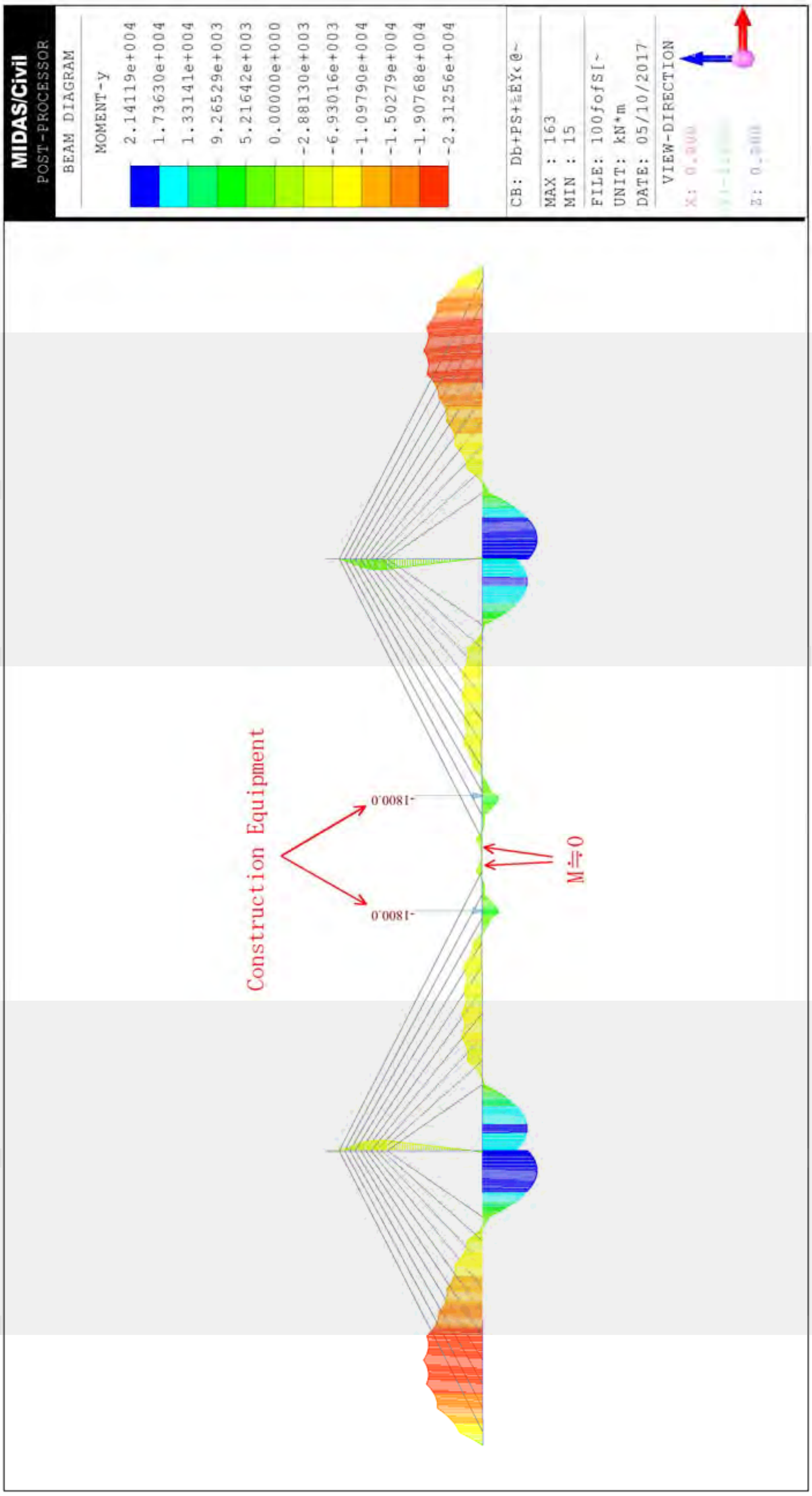
Section		Element	PS(kN)	Section		Element	PS(kN)	Section		Element	PS(kN)	Section		Element	PS(kN)
Side Span_P10	Upper	401	**720**	Main Span_P11	Upper	411	**1420**	Main Span_P12	Upper	421	**1420**	Side Span_P13	Upper	431	**720**
		402	**330**			412	**650**			422	**650**			432	**330**
		403	**0**			413	**20**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

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.85 Bending Moment Diagram for Completed Stage (D+Ps)



Source: JICA Study Team

Figure 4.2.86 Bending Moment Diagram during Closing State (Pre-Closure Dead Load + Ps + Construction Equipment)

4.2.9.6 Study on Cable Anchorage Structure

(1) Study Overview

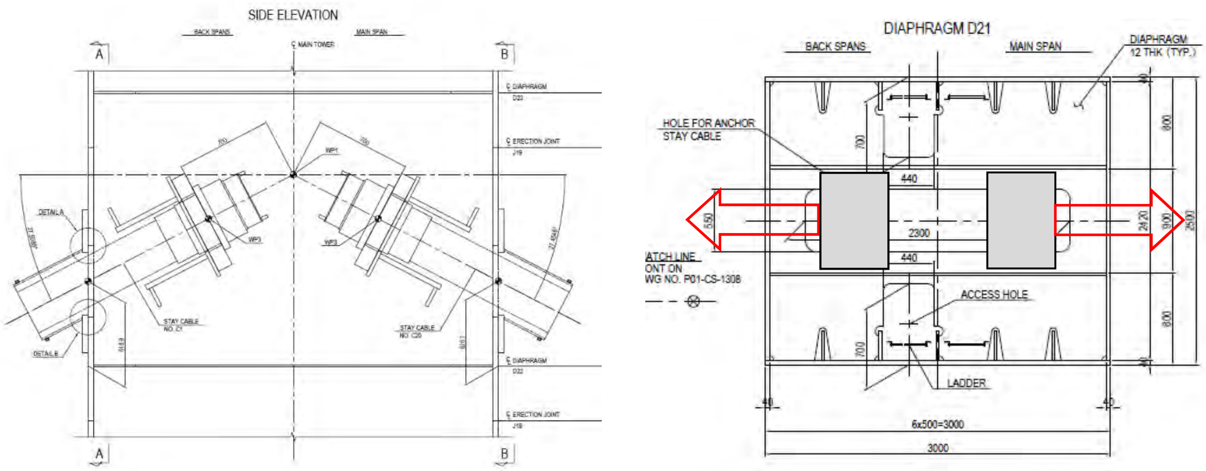
The purpose of this study is to verify suitability of the cross section of each anchor and member which constitute the main tower and main girder close to the anchor. The three-dimensional finite element (3D FE) analysis was conducted on the cable anchor and the members near the anchor to obtain the distribution of local stress induced by cable tension (Maximum: D+L+PS). The study items are the following:

■ Study Items

1. Determining the stress of each member caused by cable tension
2. Evaluation of the additional stress intensity at the web assumed by a simple calculation

(2) Anchor Structure on Main Tower

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



Source: JICA Study Team

Figure 4.2.87 Cable Anchor Structure on Main Tower Side

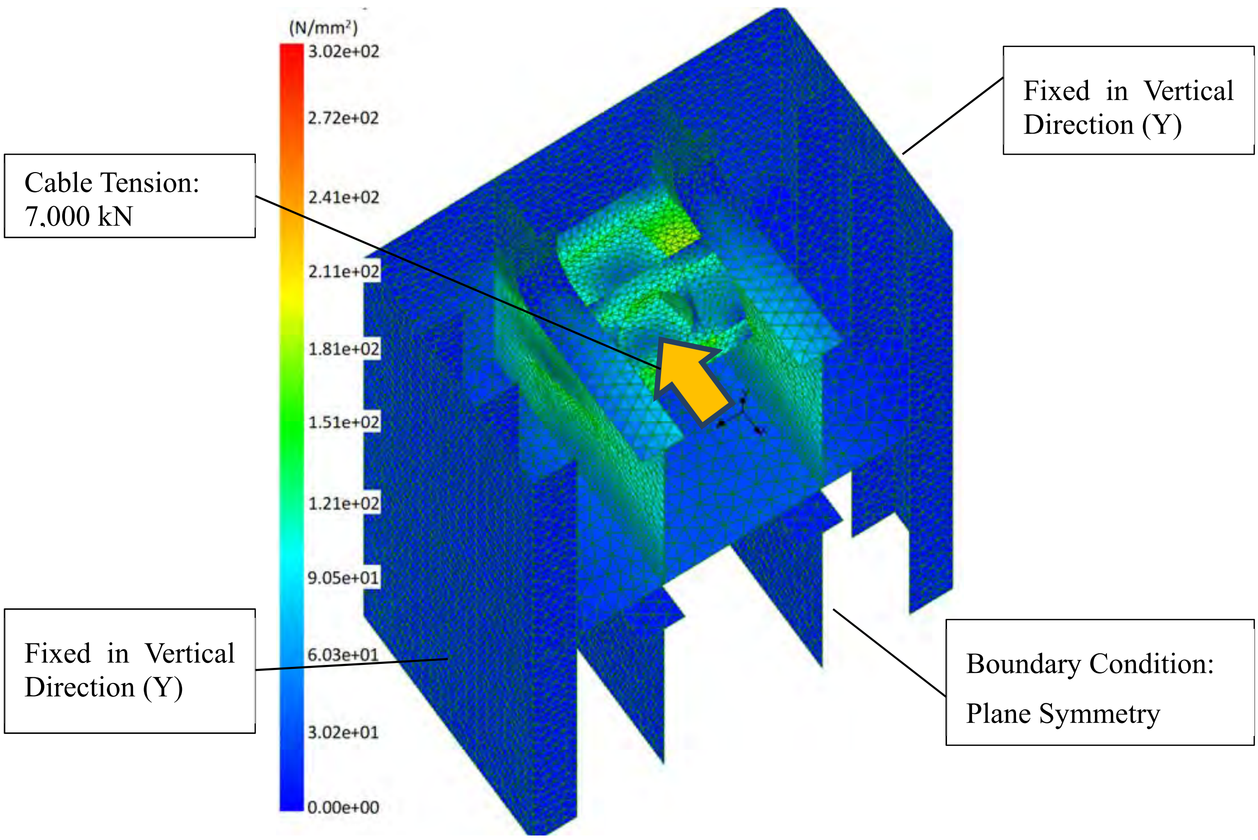
1) Analysis Model of Anchor Structure on Main Tower

The specifications for the analysis model and the model itself are shown in the table and figure below. As an analysis model, C401 (the anchor block (anchor girder and tower members) close to the top of the tower where the cable tension is maximum) was selected. The load was assumed to be P = 7,000 kN by rounding up the maximum load, which is defined as D (Dead Load) + L (Live Load) + PS.

Table 4.2.49 Specifications for Analysis Model

Analysis code	COMP (Nagaoka University of Technology)
Element type	Three-node shell element (17,370 elements)
Material model	Linear elastic model
Boundary conditions	Top and bottom: Fixed in vertical direction Axis of symmetry: Symmetric condition
Working load	Cable tension at the top: 7,000 kN (Rounded up) (Maximum design load: 6,617 kN)

Source: JICA Study Team

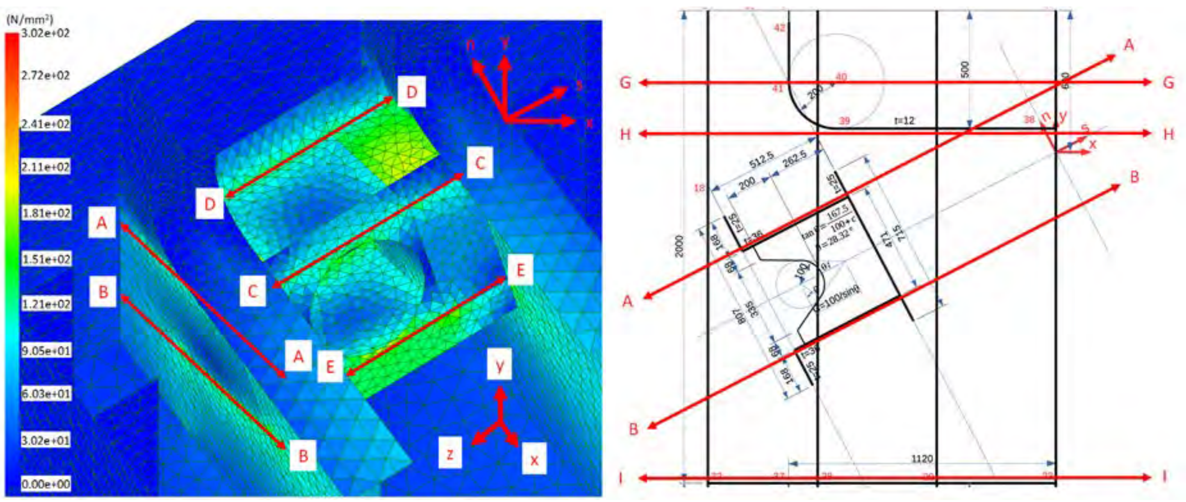


Source: JICA Study Team

Figure 4.2.88 Analysis Model

2) Analysis Results of Anchor Structure on Main Tower

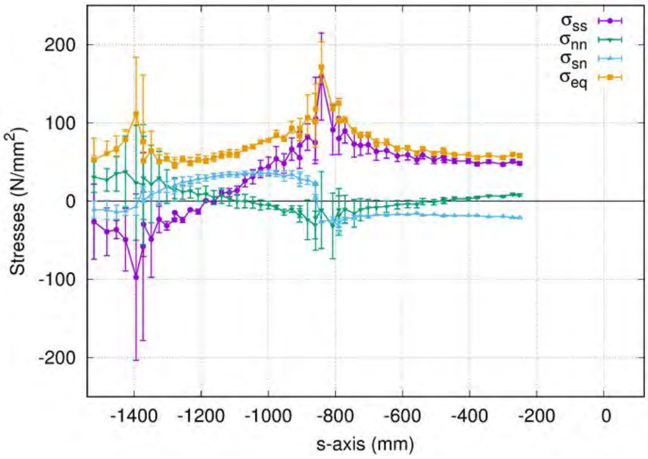
The coordinate system and the stress output lines are shown in the figure below.



Source: JICA Study Team

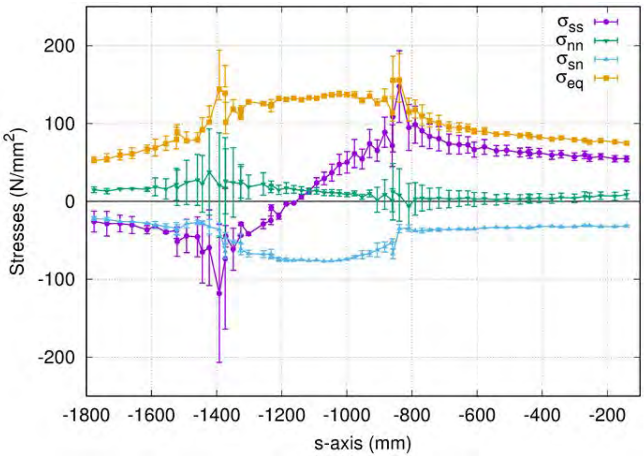
Figure 4.2.89 Coordinate System and Stress Output Lines

Stress distribution of each stress output lines is shown in the figure below.



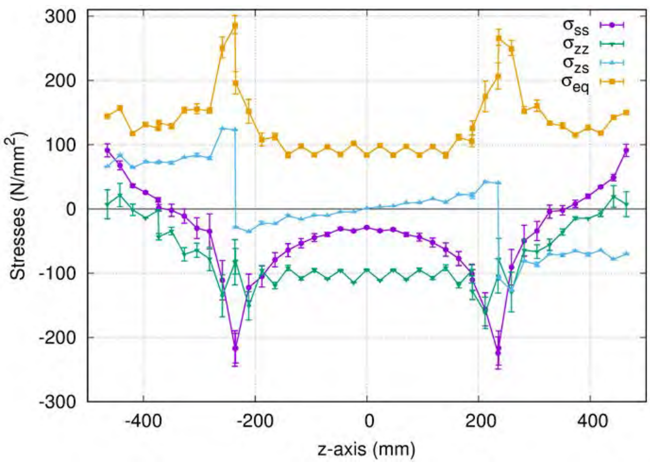
Source: JICA Study Team

Figure 4.2.90 Stress Distribution on A-A (Inner Web)



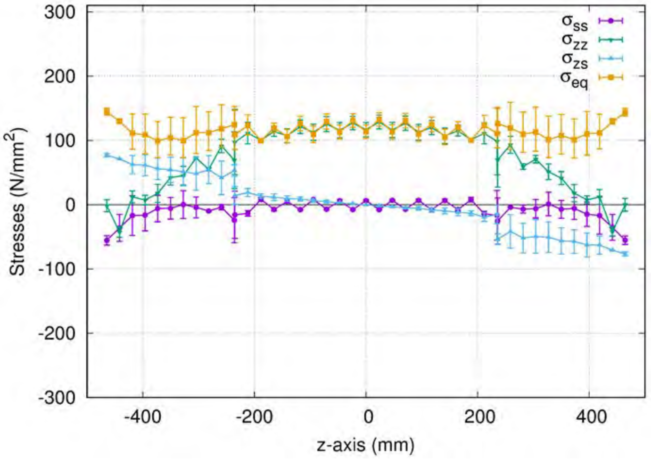
Source: JICA Study Team

Figure 4.2.91 Stress Distribution on B-B (Inner Web)



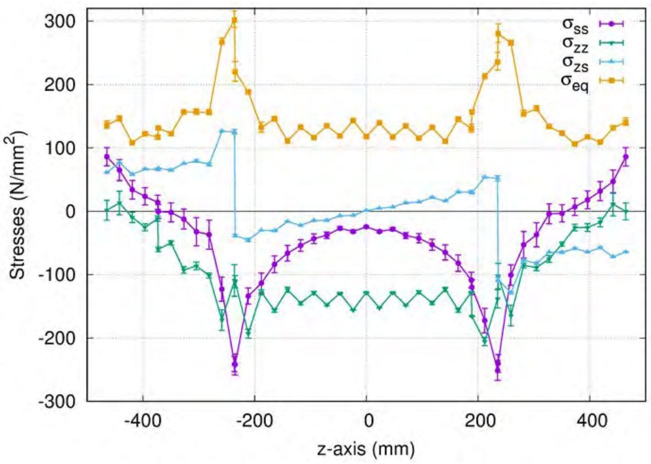
Source: JICA Study Team

Figure 4.2.92 Stress Distribution on C-C (Inner Web)



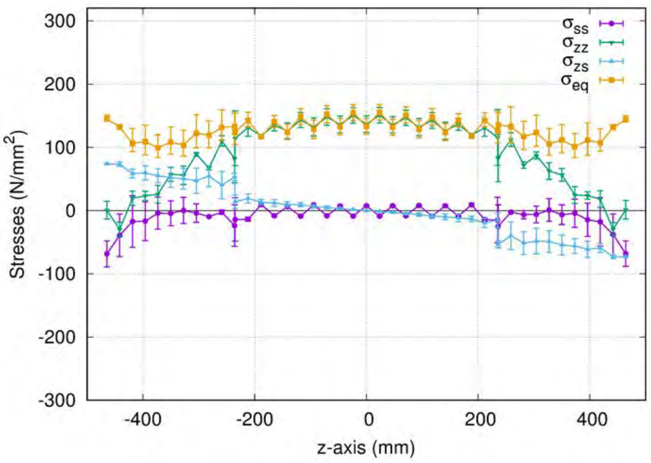
Source: JICA Study Team

Figure 4.2.93 Stress Distribution on D-D (Anchor Girder Web)



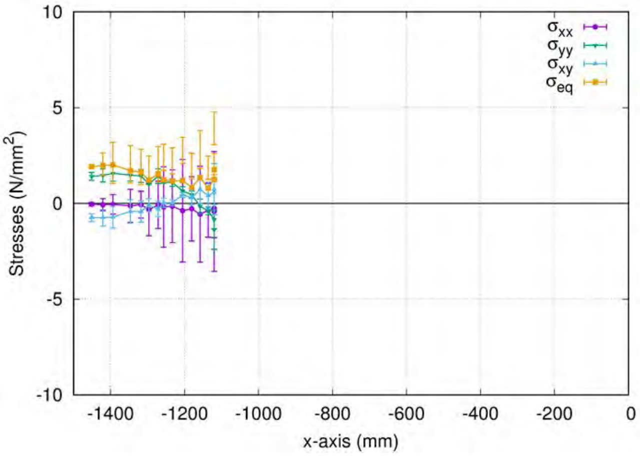
Source: JICA Study Team

Figure 4.2.94 Stress Distribution on E-E (Anchor Girder Web)



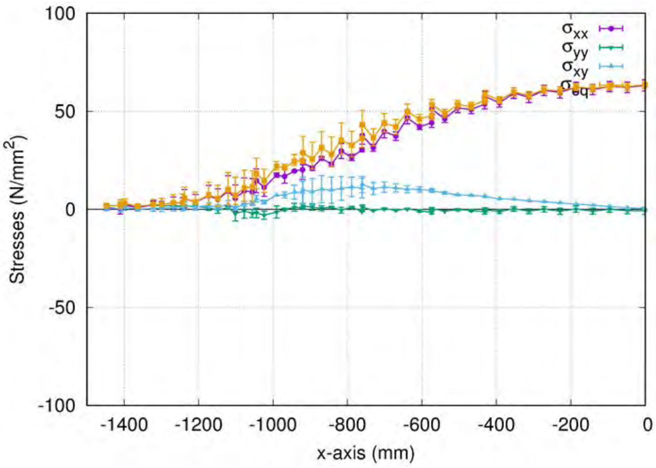
Source: JICA Study Team

Figure 4.2.95 Stress Distribution on F-F (Anchor Girder Web)



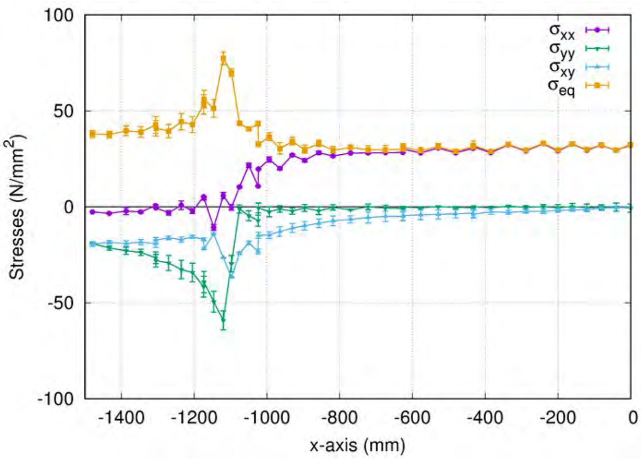
Source: JICA Study Team

Figure 4.2.96 Stress Distribution on G-G (Inner Web)



Source: JICA Study Team

Figure 4.2.97 Stress Distribution on H-H (Center Web)

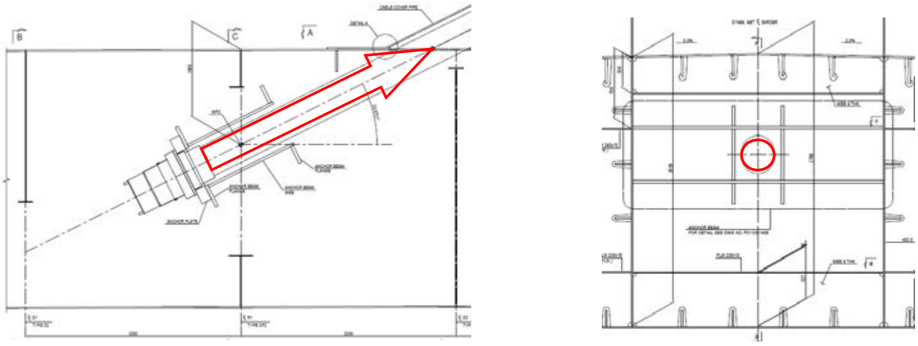


Source: JICA Study Team

Figure 4.2.98 Stress Distribution on I-I (Center Web)

(3) Anchor Structure on Main Girder

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



Source: JICA Study Team

Figure 4.2.99 Cable Anchor Structure on Main Girder

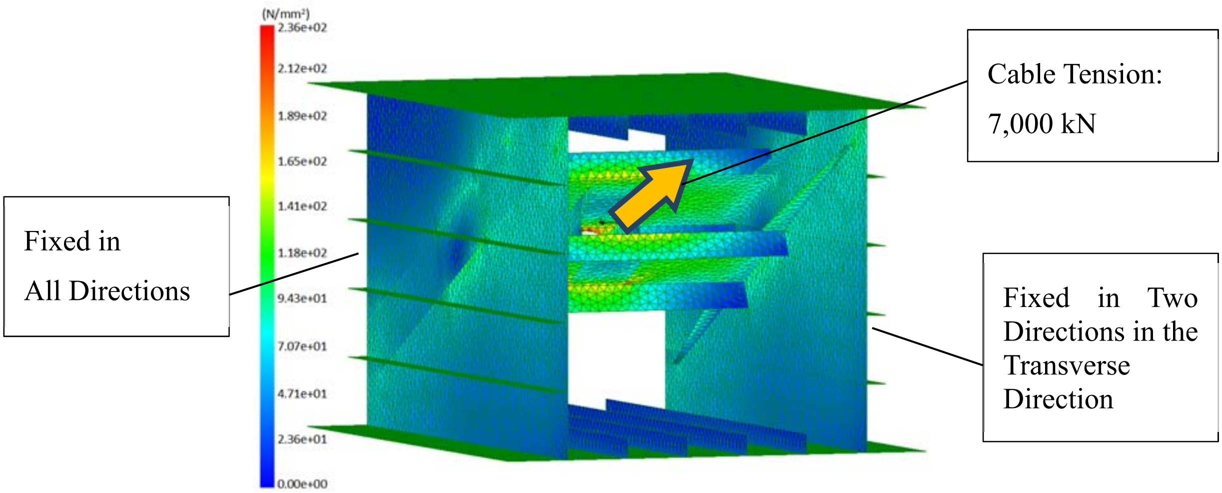
1) Analysis Model of Anchor Structure on Main Girder

The specifications for the analysis model and the model are shown in the table and figure below. The C401 analysis model (the anchor block (anchor girder and tower members) close to the top of the tower where the cable tension is maximum) was selected. The load was assumed to be P = 7,000 kN.

Table 4.2.50 Specifications for Analysis Model

Analysis code	COMP (Nagaoka University of Technology)
Element type	Three-node shell element (33,820 elements)
Material model	Linear elastic model
Boundary conditions	Left edge: Fixed in all directions Right edge: Fixed in two directions in the transverse direction
Working load	Cable tension at the top: 7,000 kN (Rounded up) (Maximum design load: 6,617 kN)

Source: JICA Study Team

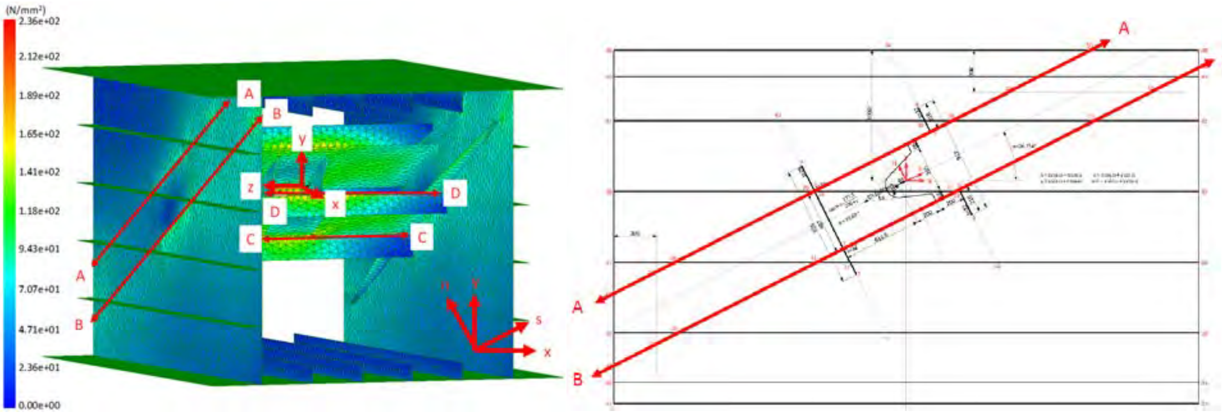


Source: JICA Study Team

Figure 4.2.100 Analysis Model

2) Analysis Results of Anchor Structure on Main Girder

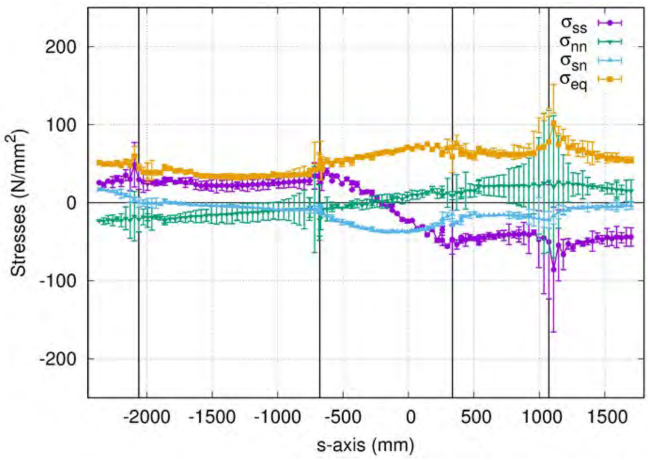
The coordinate system and the stress output lines are shown in the figure below.



Source: JICA Study Team

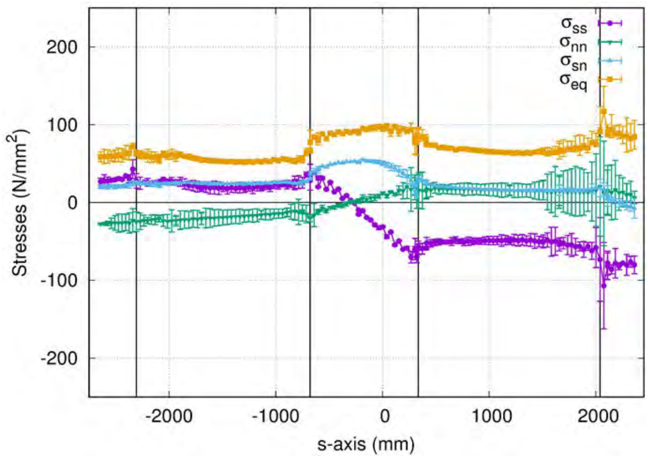
Figure 4.2.101 Coordinate System and Stress Output Lines

Stress distribution of each stress output lines is shown in the figure below.



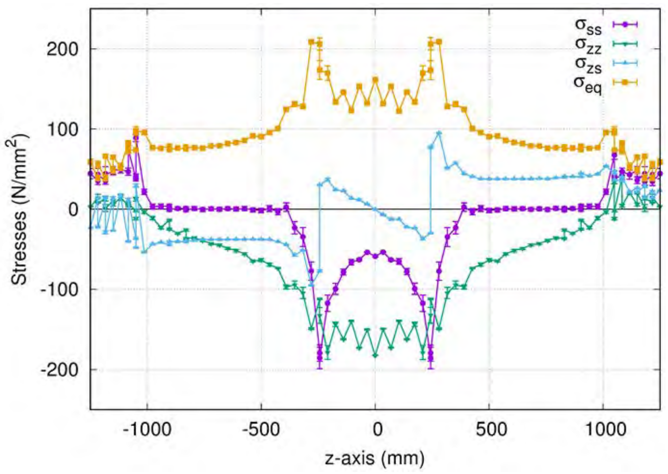
Source: JICA Study Team

Figure 4.2.102 Stress Distribution on A-A (Inner Web)



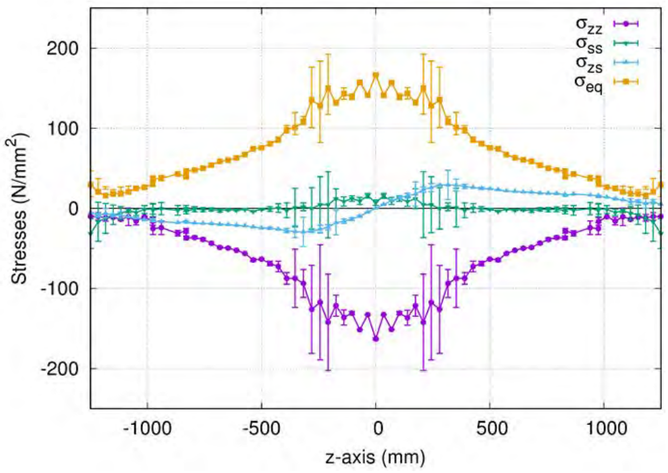
Source: JICA Study Team

Figure 4.2.103 Stress Distribution on B-B (Inner Web)



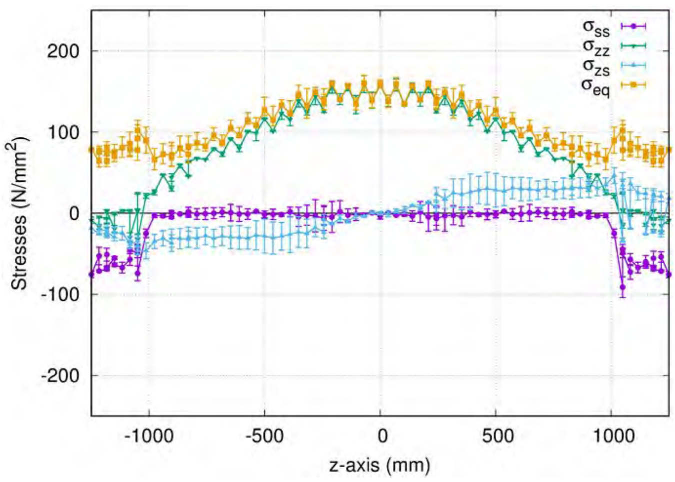
Source: JICA Study Team

Figure 4.2.104 Stress Distribution on C-C (Anchor Girder Web)



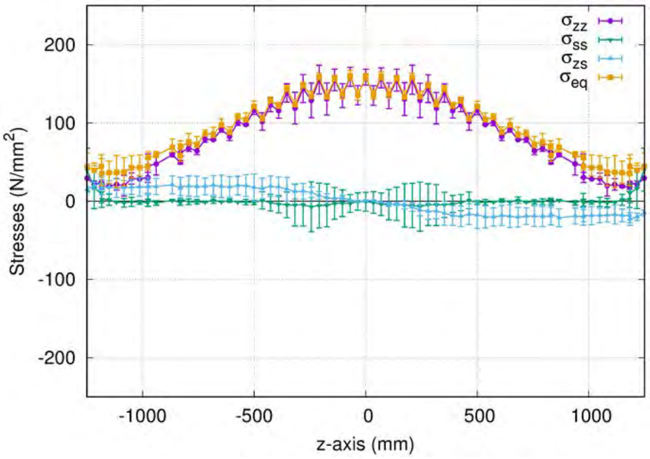
Source: JICA Study Team

Figure 4.2.105 Stress Distribution on C-C (Anchor Girder Flange)



Source: JICA Study Team

Figure 4.2.106 Stress Distribution on D-D (Anchor Girder Web)



Source: JICA Study Team

Figure 4.2.107 Stress Distribution on D-D (Anchor Girder Flange)

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

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

Table 4.2.51 Stress at Inner Web

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

Source: JICA Study Team

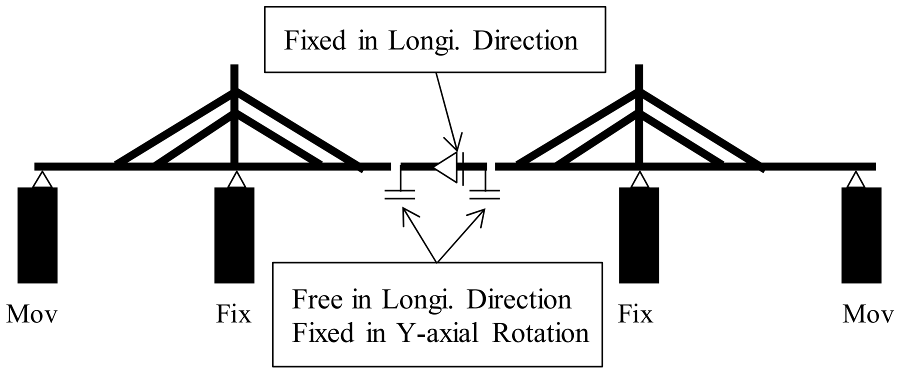
In the design calculation, cross section of the web was decided by considering additional stress which was estimated by a simple calculation around the web. Furthermore, the safety performance of the web was confirmed through the FE analysis.

4.2.9.7 Static Structure Analysis

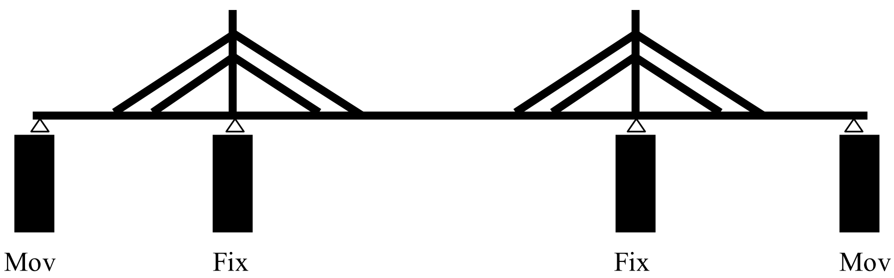
(1) Analysis Principle

- The superstructure and pier were modeled and the 3D frame analysis was performed.
- Midas Civil (developed by MIDAS IT Co.,) was employed as the analysis software.
- Considering the bridge construction steps, two analysis models were utilized, i.e., before girder closing and after girder closing.

< Before Closing Model >



< After Closing Model >



Source: JICA Study Team

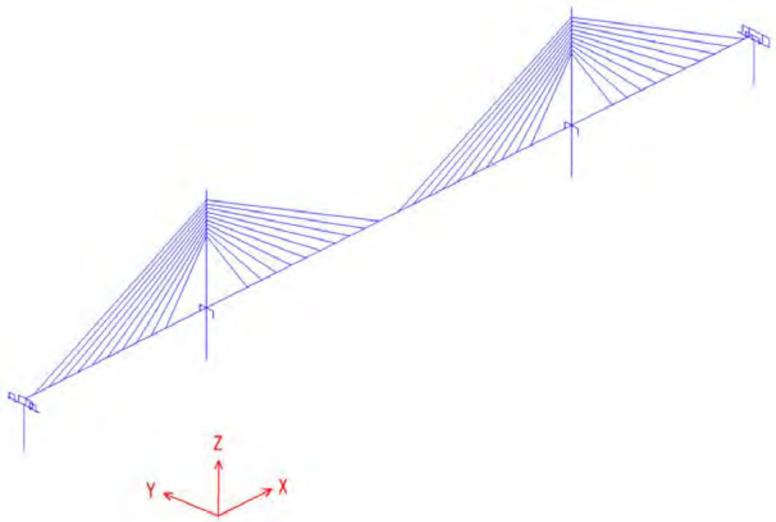
Figure 4.2.108 Analysis Models

For cables, the equivalent modulus of elasticity (EFFF) calculated by the Ernest Equation was employed to take into consideration the effect of sag. It should be noted that the tension force caused by the dead load at the completed stage (D+PS) was employed to calculate the stress σ.

$$EFFF = E0 / \{1 + \gamma^2 \cdot l^2 \cdot E0 / (12 \cdot \sigma^3)\}$$

Where,
- EFFF : Modulus of elasticity of cable with sag (Equivalent modulus of elasticity)
- E0 : Modulus of elasticity for straight cable
- γ : Weight of cable per unit length
- l : Horizontal projected length of cable
- σ : Tensile stress of cable (Dead load + Pre-stress)

The analysis model is shown in the figure below.



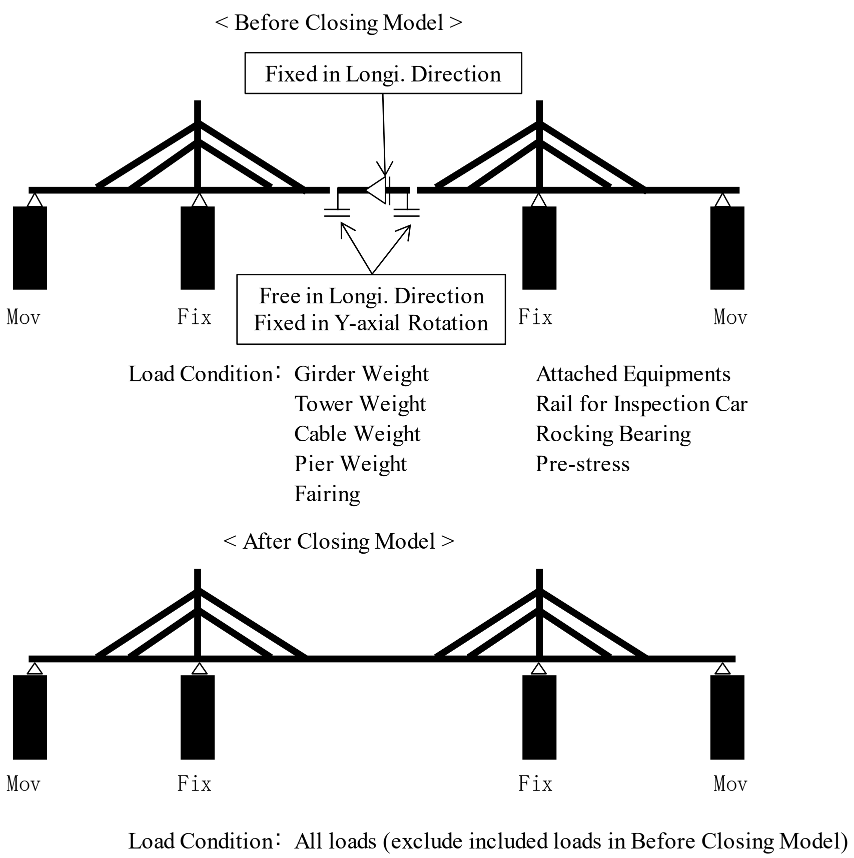
Source: JICA Study Team

Figure 4.2.109 Frame Analysis Model

(2) Loading Condition

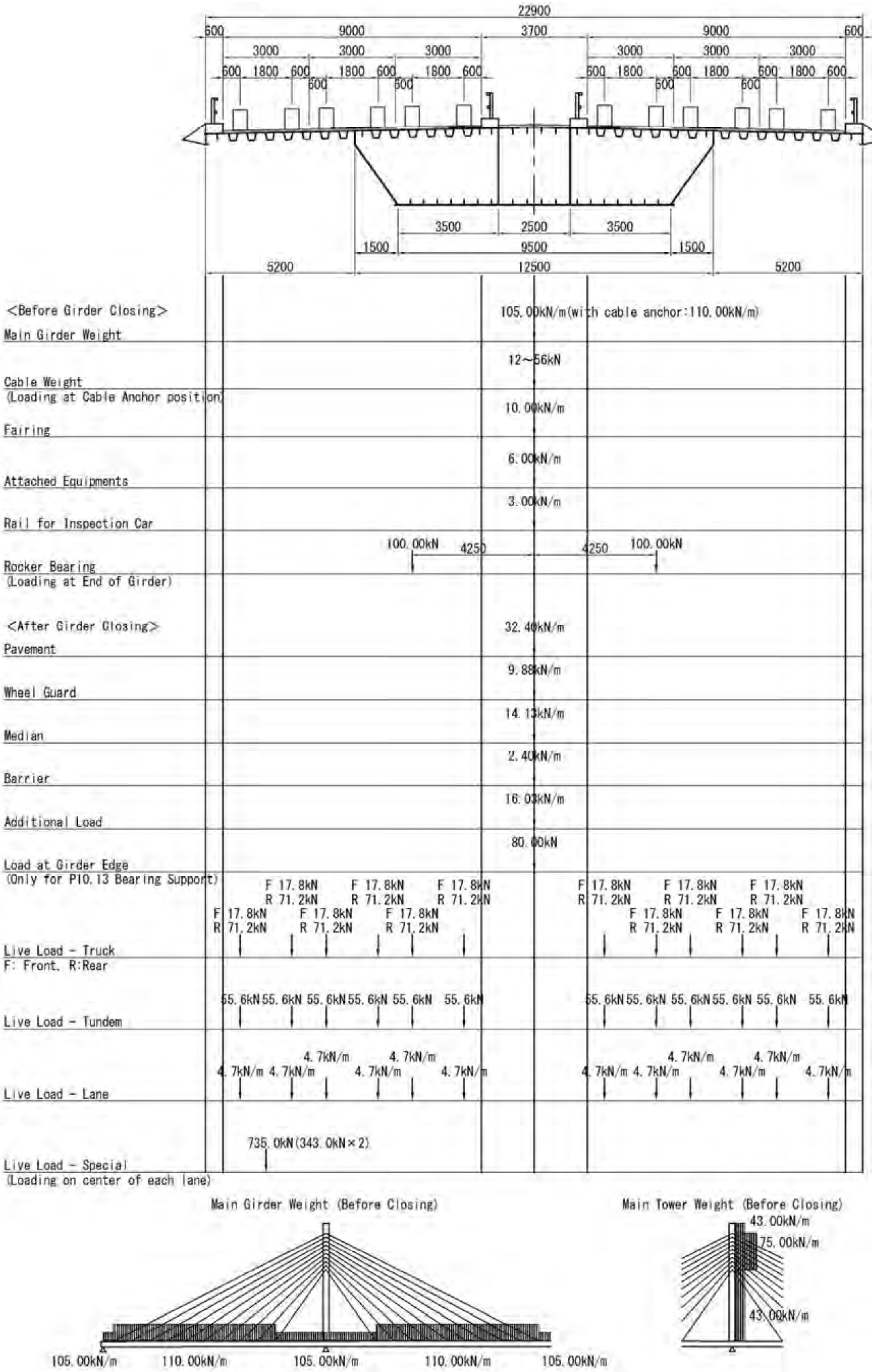
1) Load Strength

Considering the bridge construction steps, design loads were separated and loaded into two analysis models, i.e., before girder closing and after girder closing.



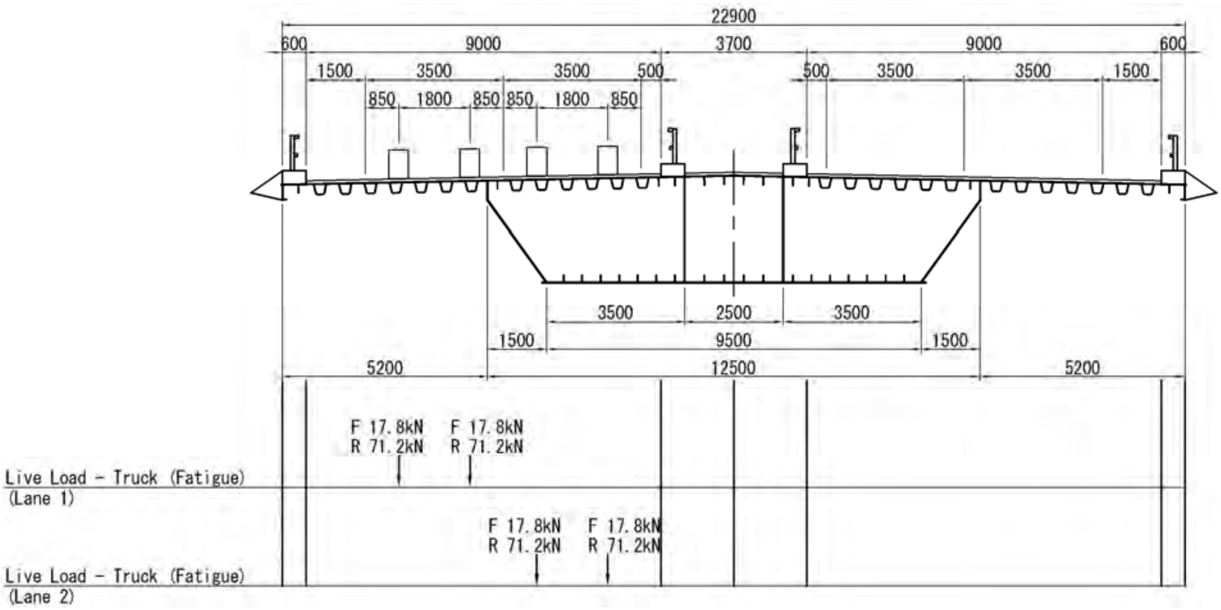
Source: JICA Study Team

Figure 4.2.110 Analysis Models During Loading



Source: JICA Study Team

Figure 4.2.111 Loading State-1



Source: JICA Study Team

Figure 4.2.112 Loading State-2

2) Loading Combinations

a) Design Section Force of Superstructure

- The names in ( ) in the load combinations of wind and wind + temperature show load cases for main tower design.
- The stress resultants are equivalent values: the section force over the increase coefficient.

Table 4.2.52 Loading Combination (Design Stress Resultants for Superstructure)

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

Source: JICA Study Team

b) Section Force for Bearing Supports

- The names in ( ) in the load combinations of wind and wind + temperature show load cases for main tower design.
- The stress resultants are raw values.
- The stress resultants at seismic performance level 2 are calculated for the bearing support design. Meanwhile, the bearing support was not designed based on the stress resultants for safety investigation of substructure.

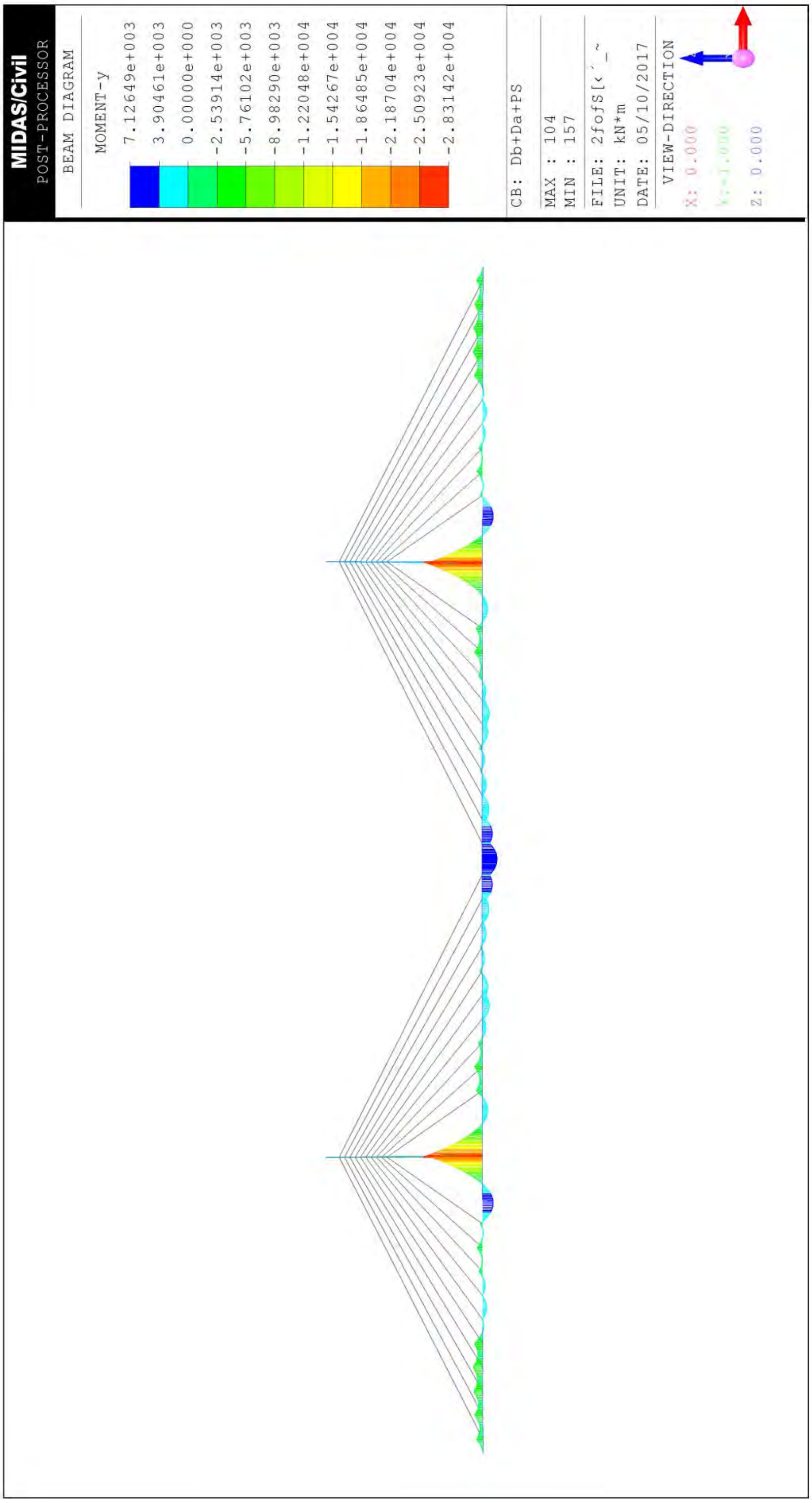
Table 4.2.53 Loading Combination (Design Stress Resultants for Bearing Support)

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

Source: JICA Study Team

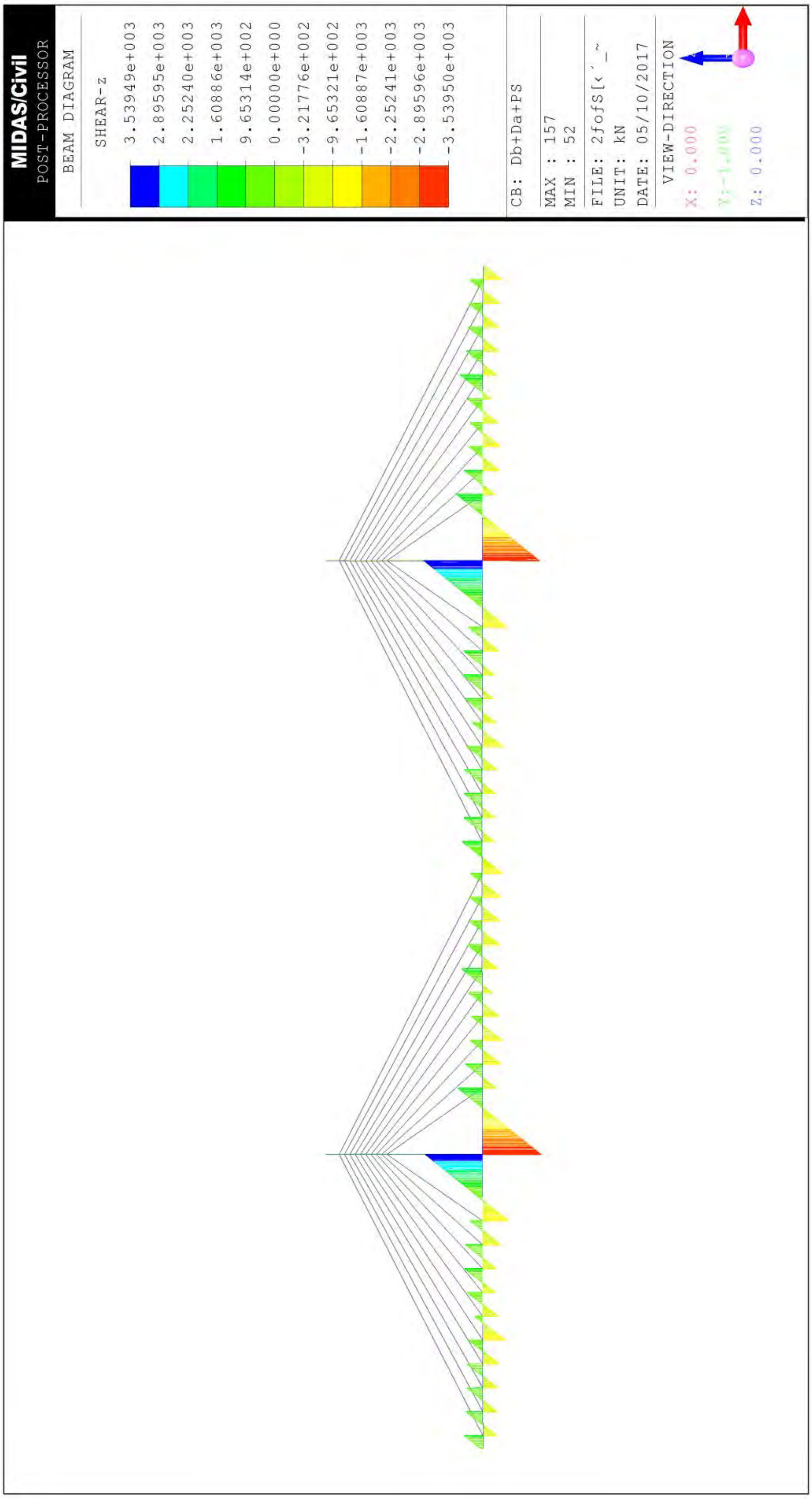
(3) Analysis Results

The analysis results are as follows:



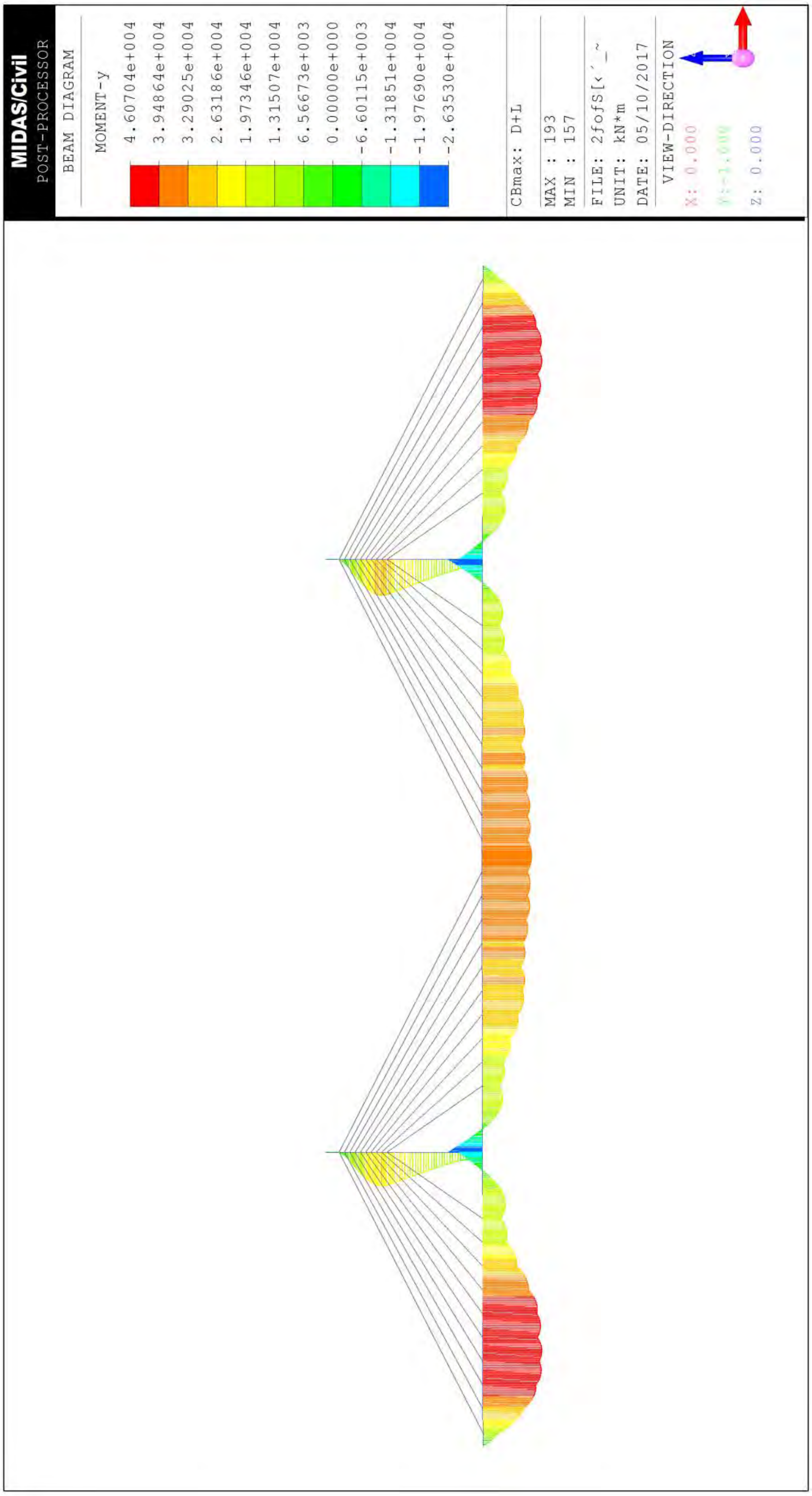
Source: JICA Study Team

Figure 4.2.113 Load at Completed Stage (D+Ps) - My



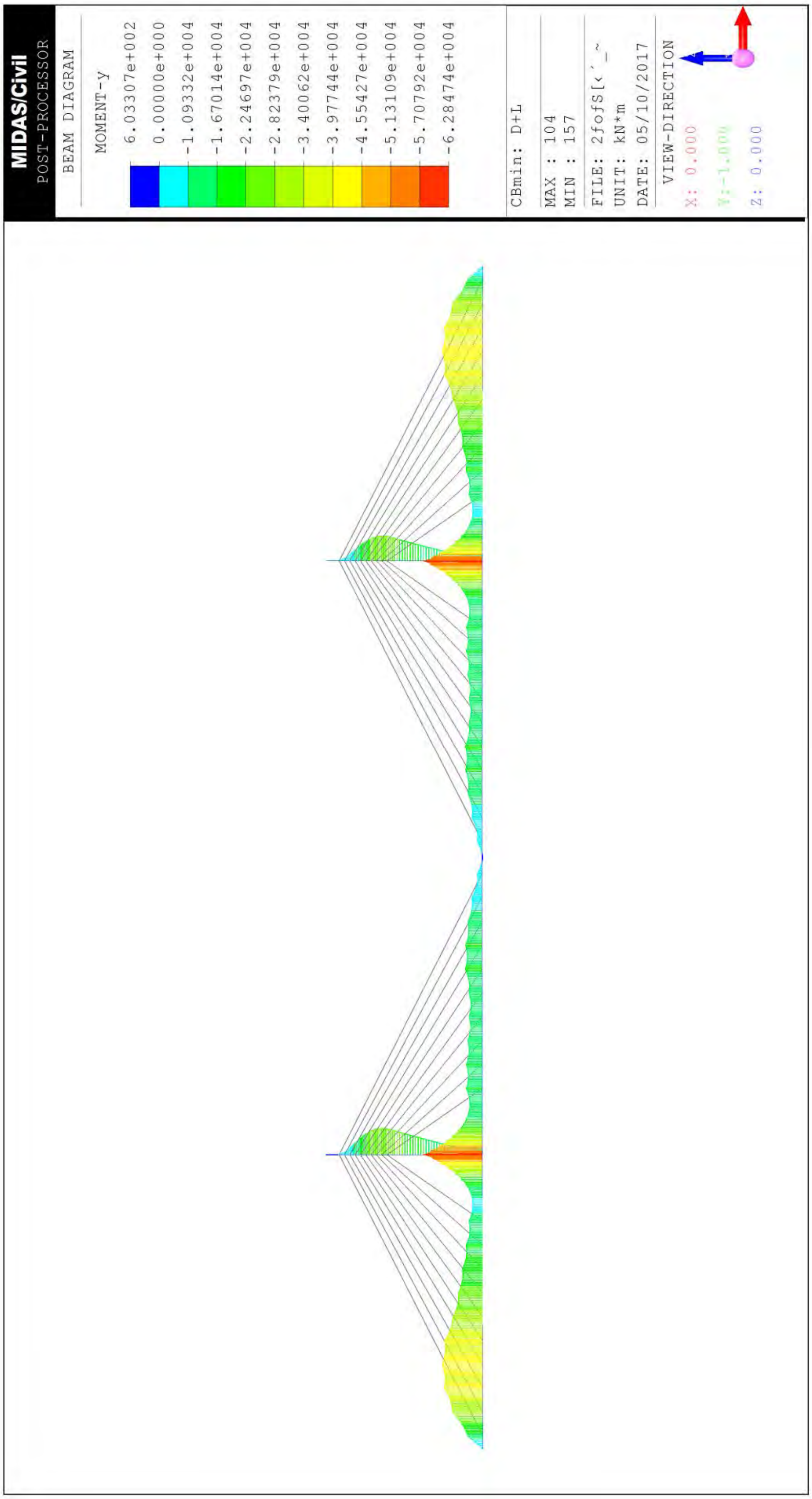
Source: JICA Study Team

Figure 4.2.114 Load at Completed Stage (D+Ps) - Sz



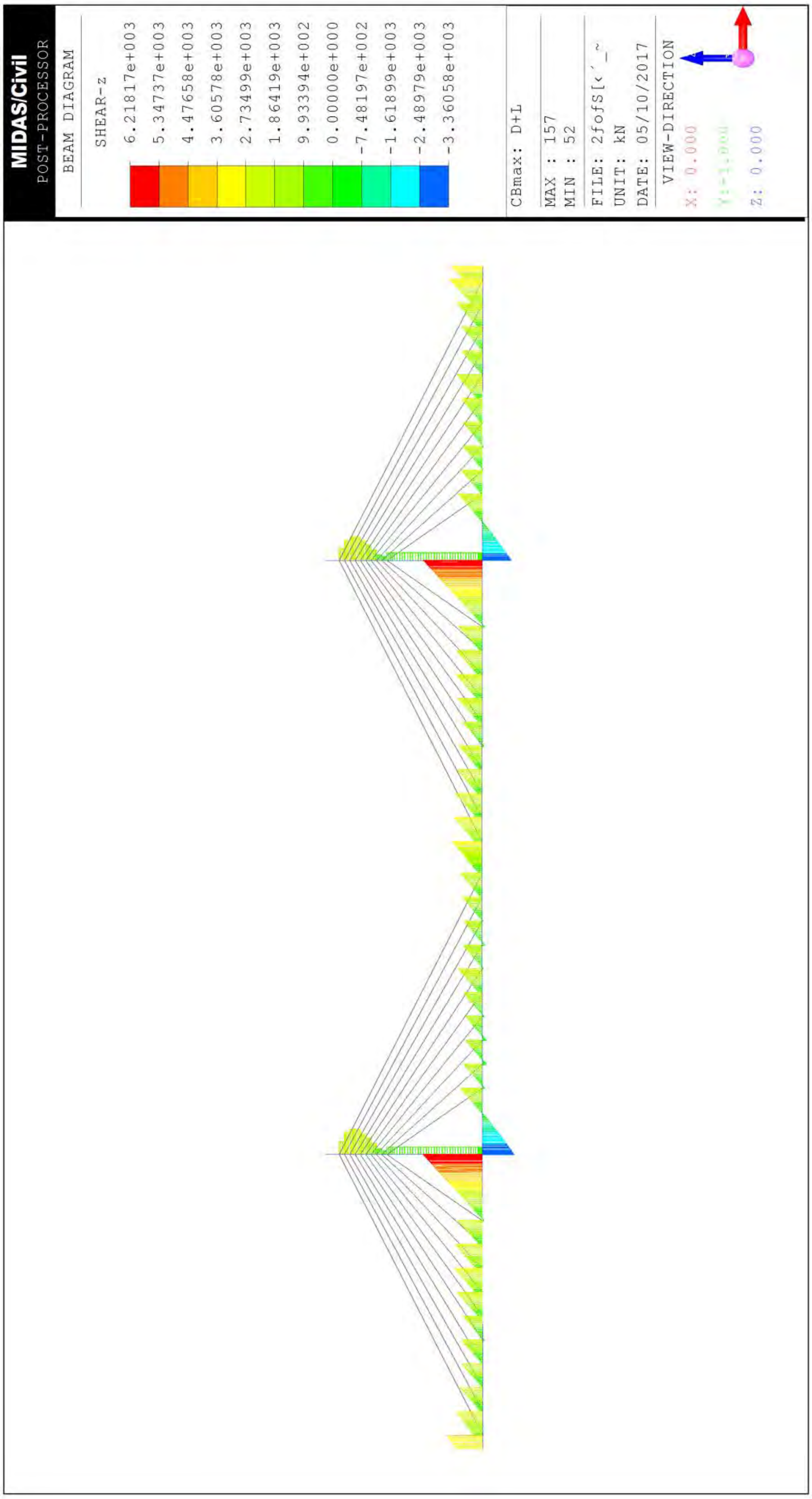
Source: JICA Study Team

Figure 4.2.115 Load at Normal State - Mmax



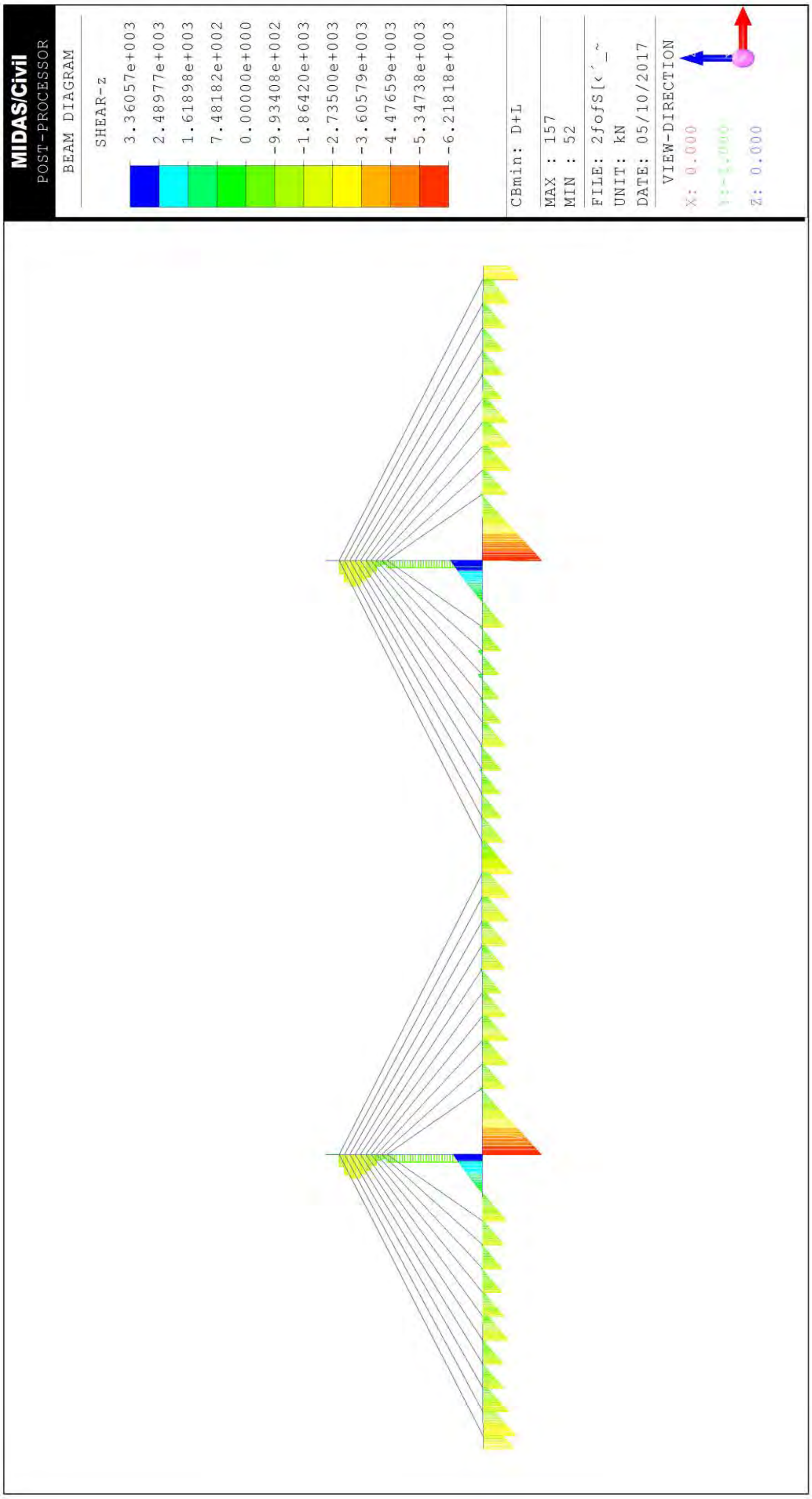
Source: JICA Study Team

Figure 4.2.116 Load at Normal State - Mmin



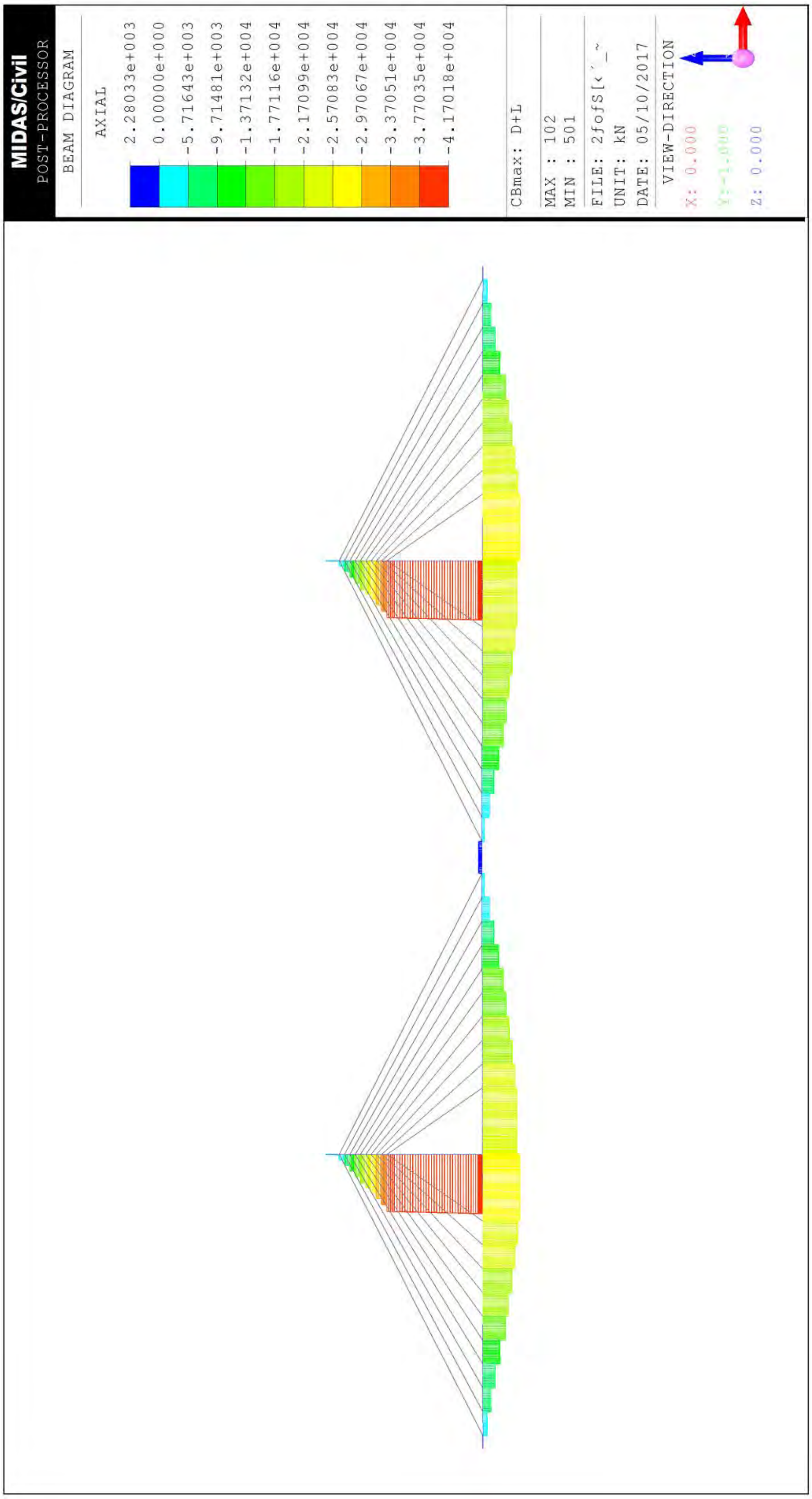
Source: JICA Study Team

Figure 4.2.117 Load at Normal State - Szmax



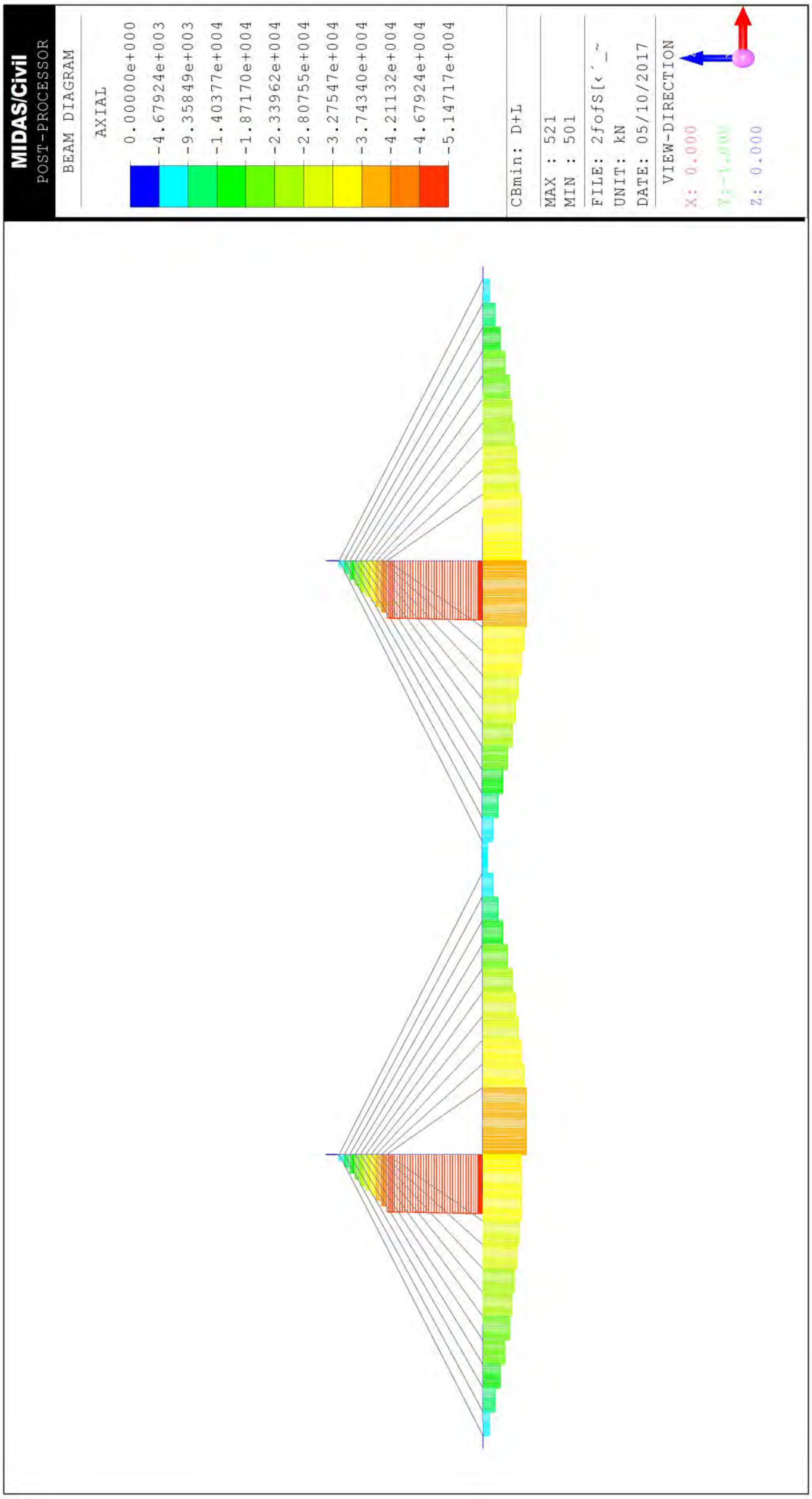
Source: JICA Study Team

Figure 4.2.118 Load at Normal State - Szmin



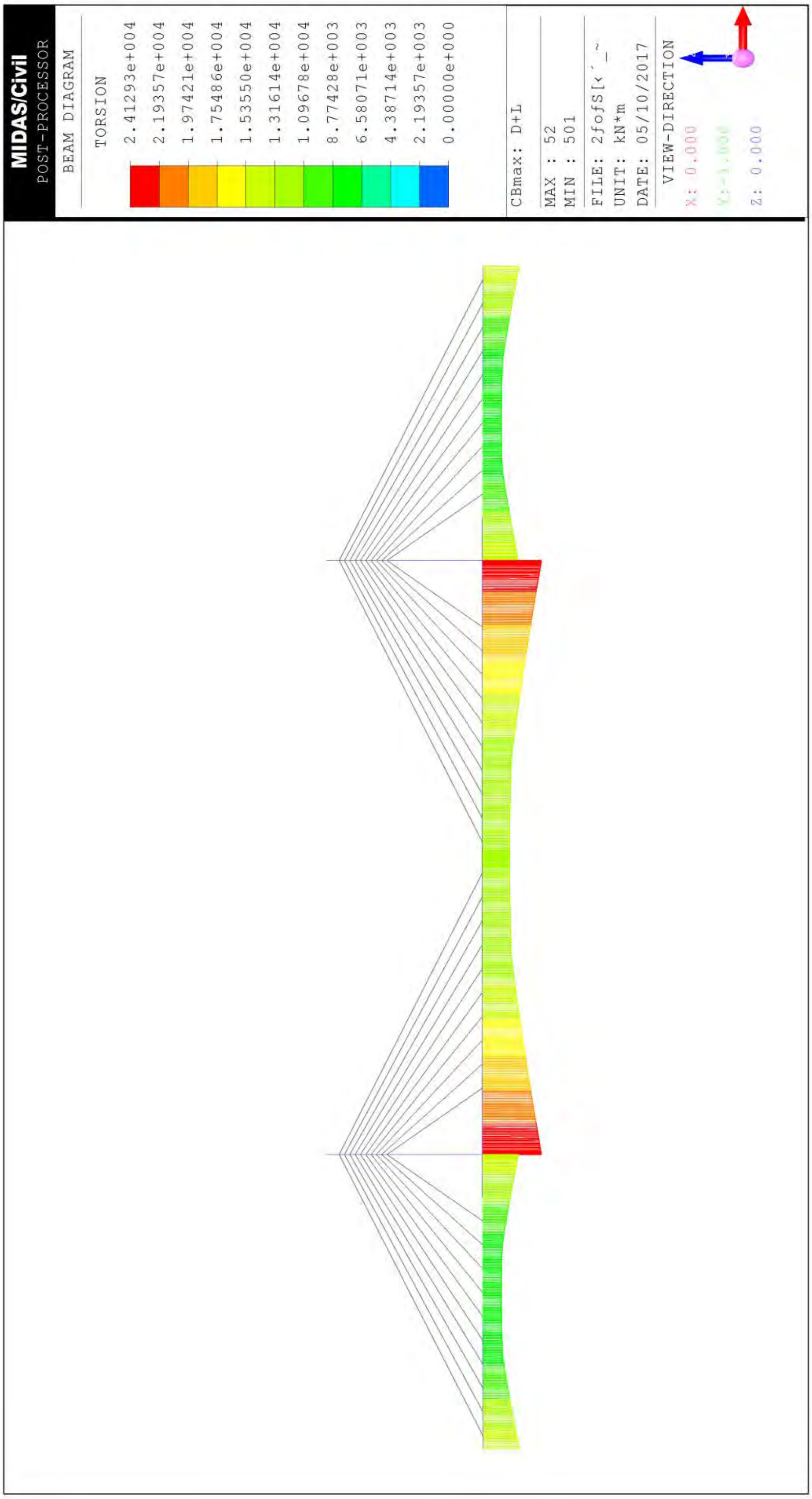
Source: JICA Study Team

Figure 4.2.119 Load at Normal State - AXmax



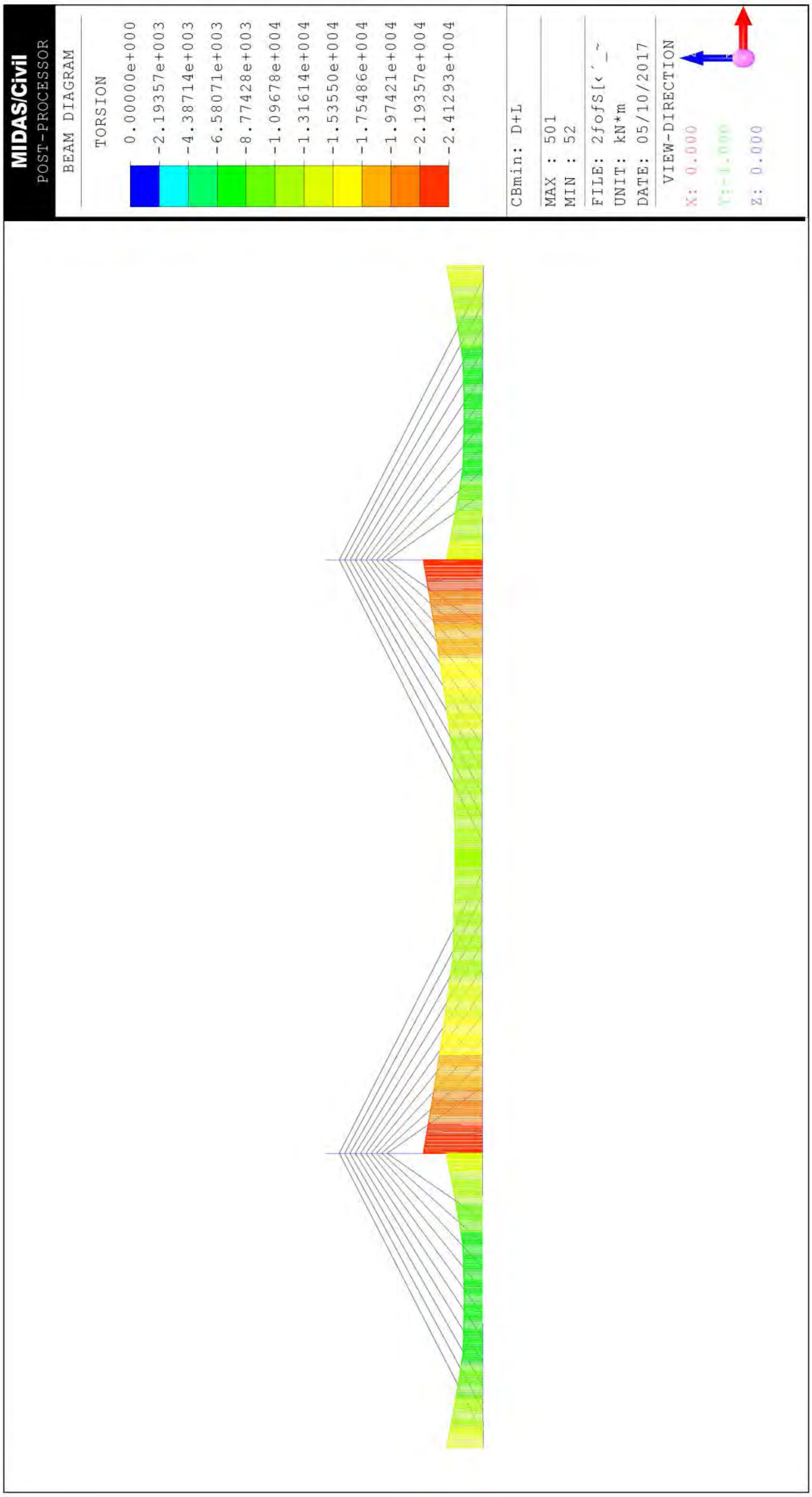
Source: JICA Study Team

Figure 4.2.120 Load at Normal State - AXmin



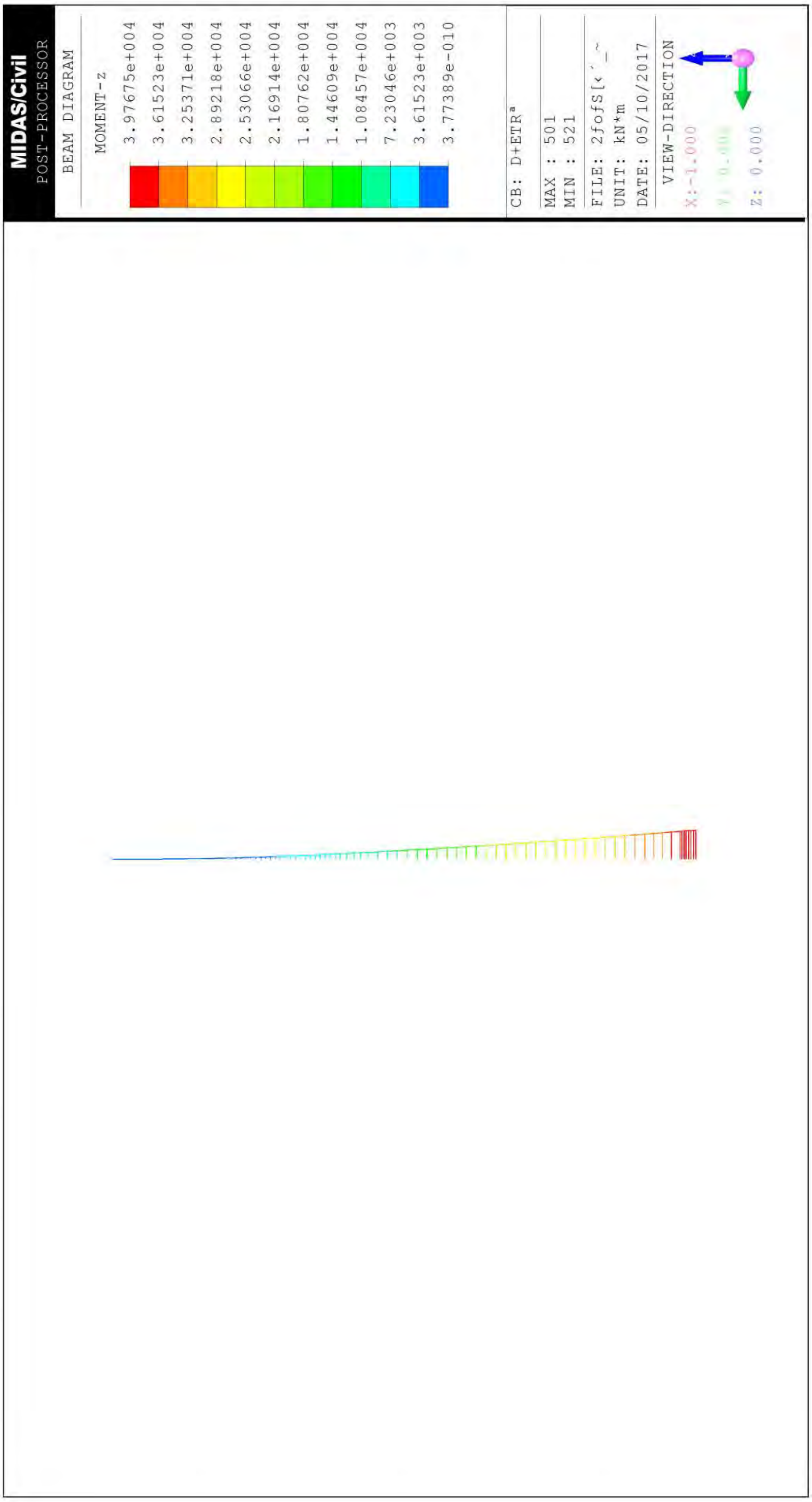
Source: JICA Study Team

Figure 4.2.121 Load at Normal State - Mxmax



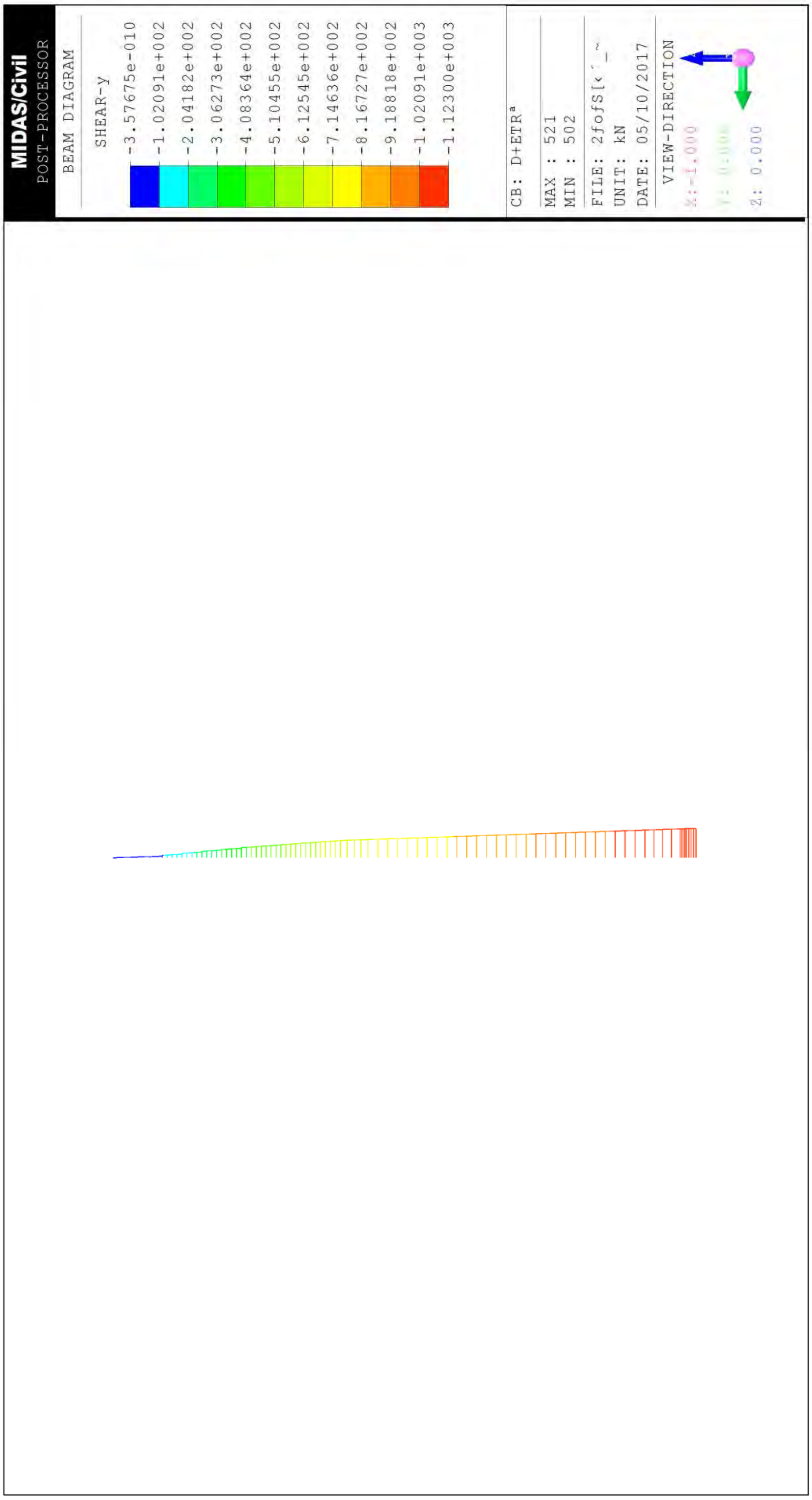
Source: JICA Study Team

Figure 4.2.122 Load at Normal State - Mxmin



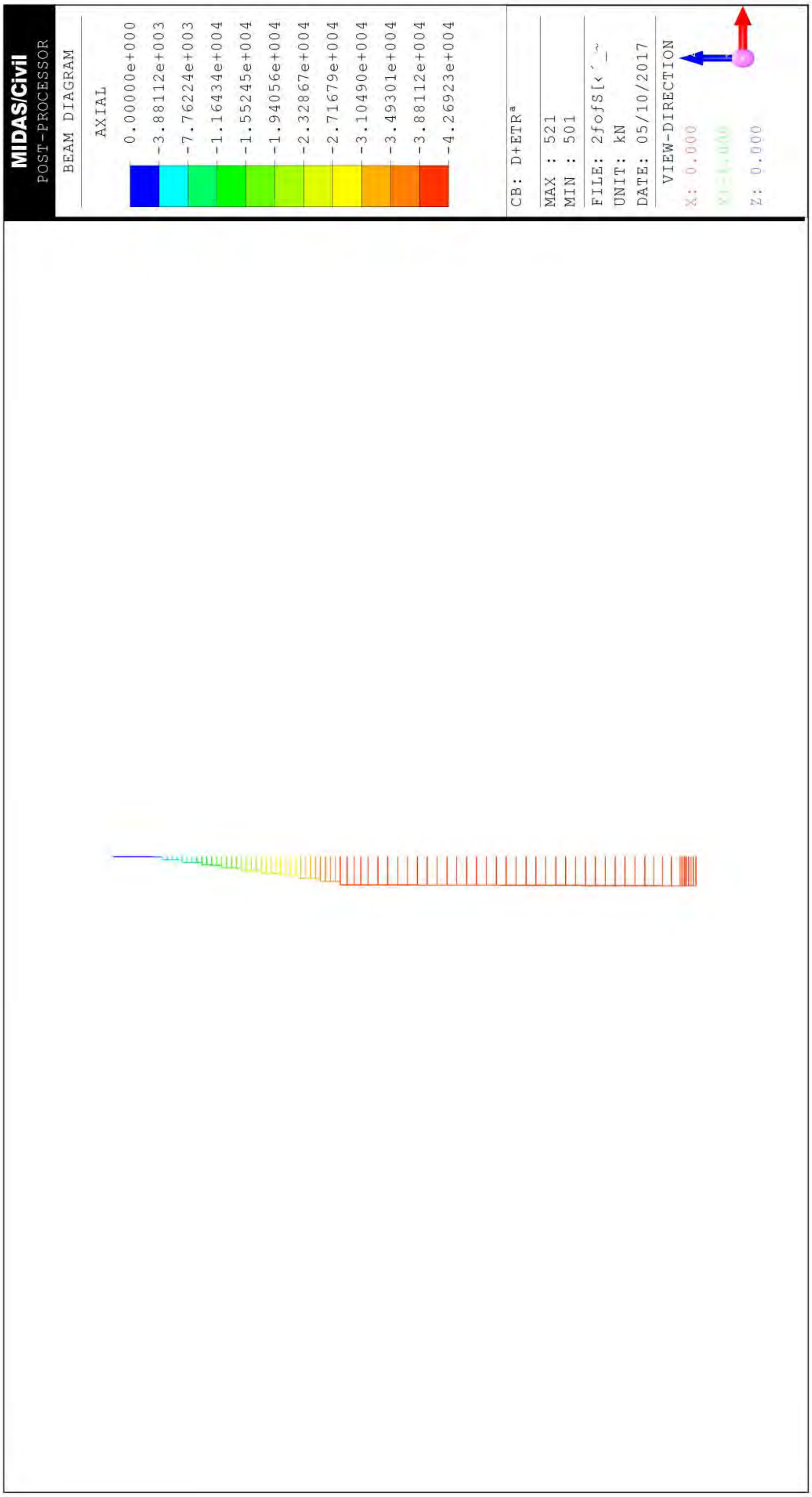
Source: JICA Study Team

Figure 4.2.123 Perpendicular to the Main Tower at Seismic State - Mz



Source: JICA Study Team

Figure 4.2.124 Perpendicular to the Main Tower at Seismic State - Sy



Source: JICA Study Team

Figure 4.2.125 Perpendicular to the Main Tower at Seismic State – AX

Table 4.2.54 Section Force of Cables

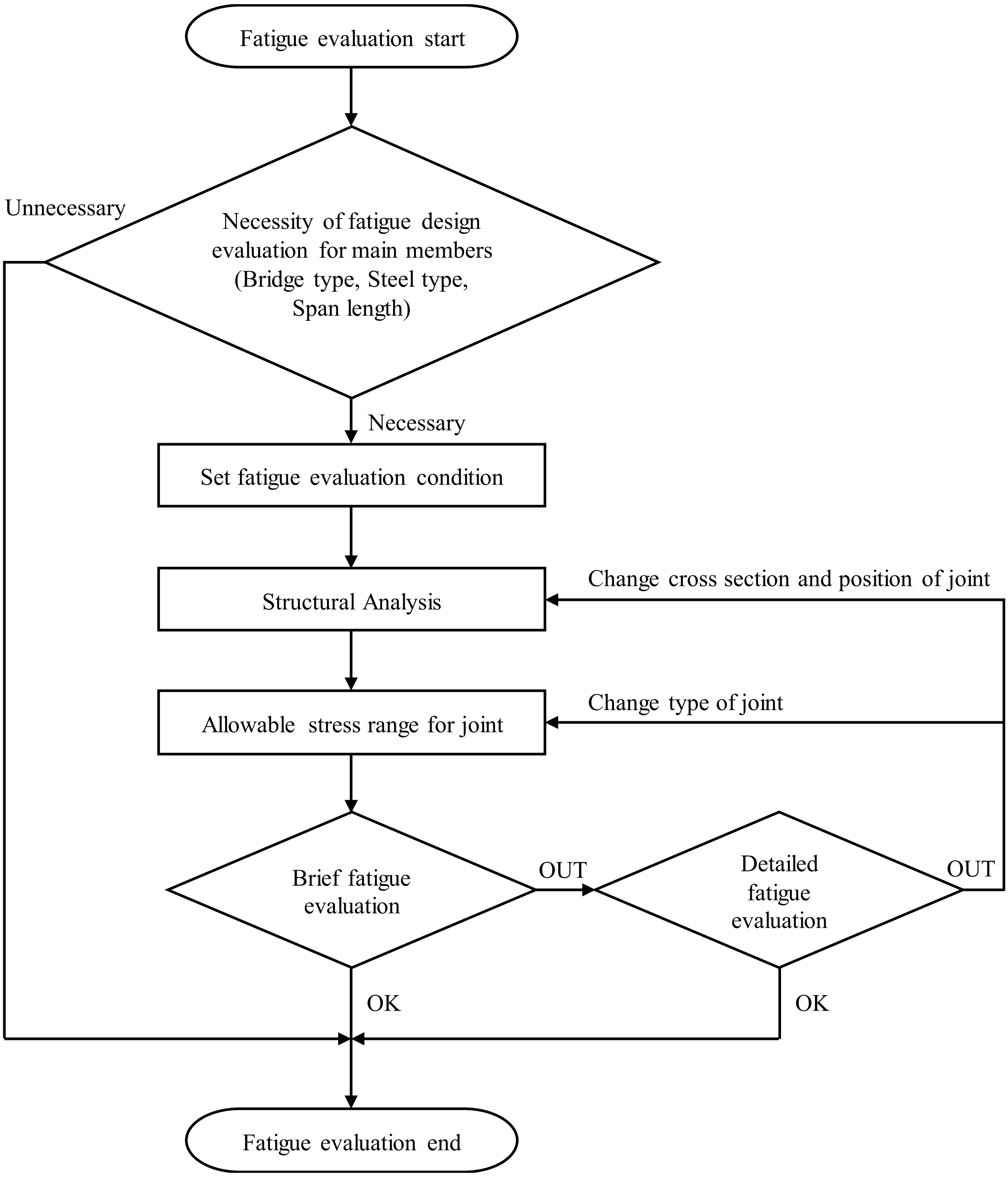
Elem	Load	Force (kN)	Elem	Load	Force (kN)
401	D+L(max)	6616.58	421	D+L(max)	5622.46
402	D+L(max)	5935.07	422	D+L(max)	5335.81
403	D+L(max)	5322.17	423	D+L(max)	5149.99
404	D+L(max)	5033.07	424	D+L(max)	5177.10
405	D+L(max)	5291.65	425	D+L(max)	5488.03
406	D+L(max)	3144.18	426	D+L(max)	3227.47
407	D+L(max)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

Source: JICA Study Team

4.2.9.8 Fatigue Design

(1) Flowchart for Fatigue Evaluation

Fatigue evaluation is conducted through the following flowchart:



Source: JICA Study Team

Figure 4.2.126 Flowchart for Fatigue Evaluation

(2) Conditions for Fatigue Evaluation

1) Design Working Life and Loading

- Design working life: 100 years
- Traffic volume of large-sized car: $ADTT_{SLi}$ = 1672 (Design traffic volume of large-sized car per day per lane in one direction)
- Load for fatigue design = (T-load) * (1 + i_f)

T load: 200 kN

i_f: Impact coefficient i_f = 10 / (50 + L)

L: Span length for calculating the impact coefficient (m)

- Correction coefficient for live load

Correction coefficient for live load $\gamma_T = \gamma_{T1} * \gamma_{T2}$ (Coefficient is multiplied when calculating stress range)

γ_{T1}: Correction coefficient for T-load

γ_{T1} = Log L_{B1} +1.50 (Here, $2.00 \leq \gamma_{T1} \leq 3.00$.)

L_{B1}: Baseline length employed for calculating the correction coefficient for T-load (m)

(γ_{T1} is rounded to three decimal places)

γ_{T2}: Simultaneous loading coefficient

Table 4.2.55 Simultaneous Loading Coefficient γ_{T2}

$ADTT_{SLi}$	$L_{B2} \leqq$ 50 m	50 m < L_{B2}
≦2000	1.0	1.0
2000<	1.0	1.1

(In case the influence line of a member does not alternate in positive and negative)
Source: Fatigue Design Recommendations for Steel Structure, JRA 2002

L: Baseline length for calculating the simultaneous loading coefficient (m)

$ADTT_{SLi}$: Design traffic volume of large-sized car per day per lane in one direction

(Car / (Day · Lane))

2) Calculation Method for Stress (General Equation)

$$\sigma = \frac{Rc}{Ri} * \left\{ \frac{N}{A} + \frac{Mx * (y*Iy+x*Ixy) + My * (x*Ix+y*Ixy)}{Ix*Iy-Ixy^2} \right\} * \gamma a$$

Where,

σ: Stress

Rc: Radius of curvature to neutral axis

Ri: Radius of curvature to evaluation position

N: Axial force

Mx: In-plane bending moment

My: Out-plane bending moment

A: Cross sectional area

Ix: Second moment of area with respect to x axis

Iy: Second moment of area with respect to y axis

Ixy: Product of inertia

x: Distance in x axis from neutral axis to evaluation position

y: Distance in y axis from neutral axis to evaluation position

γa: Structural analysis coefficient

γa = 0.8 for RC slab plate girder and box girder (except few main girder bridge)

γa = 1.0 for other types of bridge

3) Stress Range

Basic equation: $\triangle\sigma_{i,j} = | \sigma_{i,k1} - \sigma_{i,k2} | * \gamma_{T(i)}$

Where,

$\triangle\sigma_{i,j}$: Stress range	(i is lane number and j is stress range number)
$\sigma_{i,k1}$: Maximum stress	(i is lane number and k1 is transversal load number)
$\sigma_{i,k2}$: Minimum stress	(i is lane number and k2 is transversal load number)
$\gamma_{T(i)}$: Correction coefficient for live load	(i is lane number)

4) Evaluation Procedure

a) Evaluation of Cutoff Limit of Stress Range against Constant Stress Amplitude (Simple Fatigue Evaluation)

The safety of the joint against fatigue is ensured if $\Delta\sigma_{ce}$ (the cutoff limit of the stress range against constant stress amplitude) and $\Delta\sigma_{max}$ (maximum stress range calculated from the previous chapter) satisfy the conditions below.

$$\Delta\sigma_{max} \leqq \Delta\sigma_{ce} \cdot C_R \cdot C_t$$

Where, $\Delta\sigma_{max}$: Maximum stress range calculated for target joint members from previous chapter

$\Delta\sigma_{ce}$: Cutoff limit of stress range for constant stress amplitude

C_R : Correction factor for average stress

C_t : Correction factor for plate thickness

b) Evaluation of Cumulative Fatigue Damage (Detailed Fatigue Evaluation)

In case where the evaluation mentioned above is not satisfied, the safety of the joint against fatigue is ensured if the evaluation equation stated below is satisfied.

$$D \leqq 1.00$$

Where, D : Cumulative fatigue damage, $D = \Sigma D_i$

D_i : Cumulative fatigue damage caused by moving load of design fatigue load of lane i.

$$D_i = \Sigma\,(nt_i / N_{i,j})$$

nt_i : Loading frequency of design fatigue load

$N_{i,j}$: Fatigue life corresponding to $\sigma_{i,j}$ determined from design fatigue curve

$$N_{i,j} = C_0 \cdot (C_R \cdot C_t) / \sigma_{i,j}{}^m$$

$\Delta\sigma_{i,j}$: jth stress range determined by moving load of design fatigue load set of lane i

C_0 : $2\times10^6 \cdot \Delta\sigma_f{}^m$

C_R : Correction factor for average stress

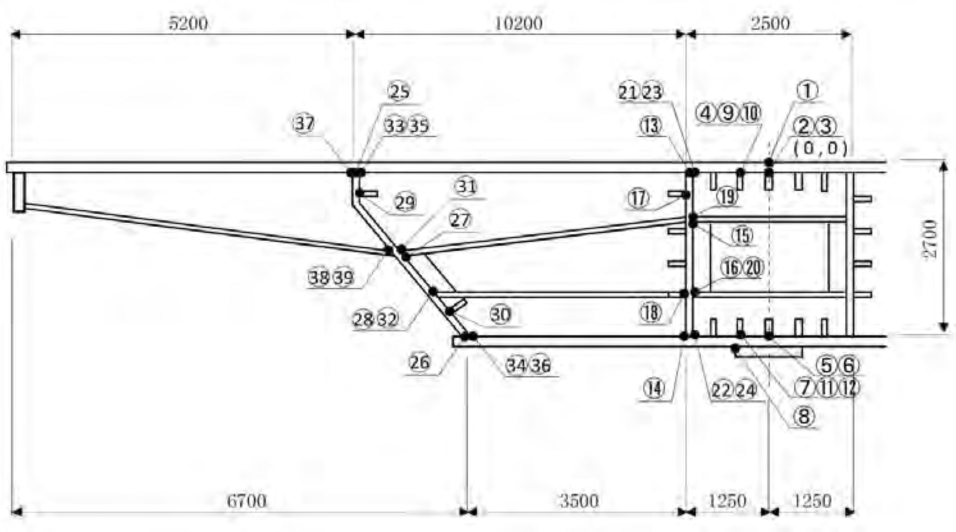
C_t : Correction factor for plate thickness

m : Coefficient to describe slope of design fatigue curve

(3) Fatigue Evaluation of the Main Girder

1) Fatigue Evaluation Point

The fatigue evaluation points are shown below.



Source: JICA Study Team

Figure 4.2.127 Fatigue Evaluation Points

Table 4.2.56 Fatigue Evaluation Points

①	Deck plate joint	㉑	Main girder inner web and transverse rib web (upper)
②	Deck and horizontal rib web	㉒	Main girder inner web and transverse rib web (lower)
③	Deck and diaphragm	㉓	Main girder inner web and diaphragm (upper)
④	Deck and vertical rib	㉔	Main girder inner web and diaphragm (lower)
⑤	Bottom flange and horizontal rib web	㉕	Main girder outer web and deck
⑥	Bottom flange and diaphragm	㉖	Main girder outer web and bottom flange
⑦	Bottom flange and vertical rib	㉗	Main girder outer web (upper) and vertical stiffener
⑧	Sole plate (longitudinal)	㉘	Main girder outer web (lower) and vertical stiffner
⑨	Longitudinal rib of deck and transverse rib	㉙	Main girder outer web (upper) and horizontal stiffener
⑩	Longitudinal rib of deck and diaphragm	㉚	Main girder outer web (lower) and horizontal stiffner
⑪	Longitudinal rib of bottom flange and transverse rib	㉛	Main girder outer web and transverse rib flange (upper)
⑫	Longitudinal rib of bottom flange and diaphragm	㉜	Main girder outer web and transverse rib flange (lower)
⑬	Main girder inner web and deck	㉝	Main girder outer web and transverse rib web (upper)
⑭	Main girder inner web and bottom flange	㉞	Main girder outer web and transverse rib web (lower)
⑮	Main girder inner web (upper) and vertical stiffener	㉟	Main girder outer web and diaphragm (upper)
⑯	Main girder inner web (lower) and vertical stiffner	㊱	Main girder outer web and diaphragm (lower)
⑰	Main girder inner web rib (upper)	㊲	Main girder web and bracket web upper edge
⑱	Main girder inner web rib(lower)	㊳	Main girder web and bracket web lower edge
⑲	Main girder inner web and transverse rib flange (upper)	㊴	Main girder web and bracket bottom flange
⑳	Main girder inner web and transverse rib flange (lower)		

Source: JICA Study Team

2) Result of Fatigue Evaluation

The result of the fatigue evaluation is shown below.

Table 4.2.57 Results of Fatigue Evaluation (1)

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

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

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

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

Source: JICA Study Team

Table 4.2.58 Results of Fatigue Evaluation (2)

Position	No.	Point-13		Point-14		Point-15		Point-16	
		Grade D		Grade D		Grade E		Grade E	
		a)	b)	a)	b)	a)	b)	a)	b)
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j
Near P10	2	-----	-------	10<92	-------	*****	*******	*****	*******
	3	1<90	-------	18<101	-------	5<74	-------	14<75	-------
	4	1<109	-------	24<96	-------	*****	*******	*****	*******
Middle od Side Span	24	2<109	-------	50<93	-------	*****	*******	*****	*******
	25	2<109	-------	49<91	-------	14<81	-------	39<72	-------
	26	2<109	-------	47<94	-------	*****	*******	*****	*******
Near P11	49	1<109	-------	15<109	-------	5<81	-------	12<81	-------
	50	1<109	-------	16<109	-------	*****	*******	*****	*******
	52	2<109	-------	19<109	-------	*****	*******	*****	*******
Middle of Center	104	1<84	-------	33<84	-------	*****	*******	*****	*******
	105	1<84	-------	32<84	-------	10<62	-------	26<62	-------
	106	1<84	-------	33<84	-------	*****	*******	*****	*******

Position	No.	Point-17		Point-18		Point-19		Point-20	
		Grade G		Grade G		Grade G		Grade G	
		a)	b)	a)	b)	a)	b)	a)	b)
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j
Near P10	2	2<35	-------	8<35	-------	*****	*******	*****	*******
	3	4<38	-------	14<38	-------	5<38	-------	14<38	-------
	4	5<42	-------	19<37	-------	*****	*******	*****	*******
Middle od Side Span	24	11<42	-------	41>37	0.62	*****	*******	*****	*******
	25	11<42	-------	39>37	0.58	14<42	-------	39>37	0.57
	26	11<42	-------	38<38	-------	*****	*******	*****	*******
Near P11	49	4<42	-------	12<42	-------	5<42	-------	12<42	-------
	50	4<38	-------	13<38	-------	*****	*******	*****	*******
	52	5<38	-------	16<38	-------	*****	*******	*****	*******
Middle of Center	104	7<32	-------	26<32	-------	*****	*******	*****	*******
	105	7<32	-------	26<32	-------	9<32	-------	26<32	-------
	106	7<32	-------	26<32	-------	*****	*******	*****	*******

Position	No.	Point-21		Point-22		Point-23		Point-24	
		Grade E		Grade E		Grade E		Grade E	
		a)	b)	a)	b)	a)	b)	a)	b)
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j
Near P10	2	*****	*******	*****	*******	-----	-------	10<68	-------
	3	1<67	-------	18<75	-------	*****	*******	*****	*******
	4	*****	*******	*****	*******	1<81	-------	24<71	-------
Middle od Side Span	24	*****	*******	*****	*******	2<81	-------	50<68	-------
	25	2<81	-------	49<68	-------	*****	*******	*****	*******
	26	*****	*******	*****	*******	2<81	-------	47<69	-------
Near P11	49	1<81	-------	15<81	-------	*****	*******	*****	*******
	50	*****	*******	*****	*******	1<81	-------	16<81	-------
	52	*****	*******	*****	*******	2<81	-------	19<81	-------
Middle of Center	104	*****	*******	*****	*******	1<62	-------	33<62	-------
	105	1<62	-------	32<62	-------	*****	*******	*****	*******
	106	*****	*******	*****	*******	1<62	-------	33<62	-------

Note: a) is the simple fatigue evaluation, b) is the detailed fatigue evaluation
Source: JICA Study Team

Table 4.2.59 Results of Fatigue Evaluation (3)

Position	No.	Point-25		Point-26		Point-27		Point-28	
		Grade D		Grade D		Grade E		Grade E	
		a)	b)	a)	b)	a)	b)	a)	b)
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j
Near P10	2	-----	-------	10<92	-------	*****	*******	*****	*******
	3	1<95	-------	18<101	-------	11<74	-------	14<75	-------
	4	2<109	-------	24<96	-------	*****	*******	*****	*******
Middle od Side Span	24	4<109	-------	50<93	-------	*****	*******	*****	*******
	25	4<109	-------	49<91	-------	29<81	-------	39<72	-------
	26	3<109	-------	47<94	-------	*****	*******	*****	*******
Near P11	49	1<109	-------	15<109	-------	9<81	-------	12<81	-------
	50	1<109	-------	16<109	-------	*****	*******	*****	*******
	52	3<109	-------	19<109	-------	*****	*******	*****	*******
Middle of Center	104	2<84	-------	33<84	-------	*****	*******	*****	*******
	105	2<84	-------	32<84	-------	19<62	-------	26<62	-------
	106	2<84	-------	33<84	-------	*****	*******	*****	*******

Position	No.	Point-29		Point-30		Point-31		Point-32	
		Grade G		Grade G		Grade G		Grade G	
		a)	b)	a)	b)	a)	b)	a)	b)
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j
Near P10	2	2<35	-------	8<35	-------	*****	*******	*****	*******
	3	4<38	-------	15<38	-------	10<38	-------	14<38	-------
	4	6<42	-------	20<37	-------	*****	*******	*****	*******
Middle od Side Span	24	12<42	-------	43>37	0.75	*****	*******	*****	*******
	25	12<42	-------	41>36	0.7	28<42	-------	39>37	0.57
	26	11<42	-------	40>37	0.58	*****	*******	*****	*******
Near P11	49	4<42	-------	13<42	-------	9<42	-------	12<42	-------
	50	4<42	-------	14<42	-------	*****	*******	*****	*******
	52	6<42	-------	16<42	-------	*****	*******	*****	*******
Middle of Center	104	8<32	-------	28<32	-------	*****	*******	*****	*******
	105	8<32	-------	27<32	-------	19<32	-------	26<32	-------
	106	8<32	-------	28<32	-------	*****	*******	*****	*******

Position	No.	Point-33		Point-34		Point-35		Point-36	
		Grade E		Grade E		Grade E		Grade E	
		a)	b)	a)	b)	a)	b)	a)	b)
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j
Near P10	2	*****	*******	*****	*******	-----	-------	10<68	-------
	3	1<70	-------	18<75	-------	*****	*******	*****	*******
	4	*****	*******	*****	*******	2<81	-------	24<71	-------
Middle od Side Span	24	*****	*******	*****	*******	4<81	-------	50<68	-------
	25	4<81	-------	49<68	-------	*****	*******	*****	*******
	26	*****	*******	*****	*******	3<81	-------	47<69	-------
Near P11	49	1<81	-------	15<81	-------	*****	*******	*****	*******
	50	*****	*******	*****	*******	1<81	-------	16<81	-------
	52	*****	*******	*****	*******	3<81	-------	19<81	-------
Middle of Center	104	*****	*******	*****	*******	2<62	-------	33<62	-------
	105	2<62	-------	32<62	-------	*****	*******	*****	*******
	106	*****	*******	*****	*******	2<62	-------	33<62	-------

Position	No.	Point-37		Point-38		Point-39			
		Grade E		Grade E		Grade G			
		a)	b)	a)	b)	a)	b)		
		Judge	D = Σ Di,j	Judge	D = Σ Di,j	Judge	D = Σ Di,j		
Near P10	2	-----	-------	5<68	-------	5<35	-------		
	3	1<70	-------	10<74	-------	10<38	-------		
	4	2<81	-------	13<77	-------	13<40	-------		
Middle od Side Span	24	4<81	-------	27<81	-------	27<42	-------		
	25	4<81	-------	26<81	-------	26<42	-------		
	26	3<81	-------	25<81	-------	25<42	-------		
Near P11	49	1<81	-------	8<81	-------	8<42	-------		
	50	1<81	-------	9<81	-------	9<42	-------		
	52	3<81	-------	11<81	-------	11<42	-------		
Middle of Center	104	2<62	-------	18<62	-------	18<32	-------		
	105	2<62	-------	18<62	-------	18<32	-------		
	106	2<62	-------	18<62	-------	18<32	-------		

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

Source: JICA Study Team

(4) Fatigue Evaluation of the Cable Anchorage Member

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

1) Cable Number C6~C15: Cable Cross Section φ 15.6×37

Fatigue evaluation equation

$$\Delta\sigma max \leqq \Delta\sigma ce \cdot C_R \cdot Ct$$

$\Delta\sigma max$: Maximum stress range

$\Delta\sigma ce$: Constant stress amplitude

Maximum difference between maximum and minimum tension caused by fatigue load per cable

Δ = 89.3 kN (contains impact coefficient for fatigue evaluation)

Maximum difference between maximum and minimum stress of member caused by fatigue load on each cable anchorage member

				Minimum Yield Stress of Cable		Allowable Stress
Δ	=	89.3	/	3870	×	210

= 4.8 N/mm^2 (contains impact coefficient for fatigue evaluation)

Calculation of maximum stress range Δσmax for entire bridge

$\Delta\sigma max$ = Stress range coefficient × Maximum difference between maximum and minimum stress

= 3.0 × 4.8

= 14.5 N/mm^2 $\leqq$ $\Delta\sigma ce \cdot C_R \cdot Ct$

$\Delta\sigma ce \cdot C_R \cdot Ct$ = 32.0 × 1.0 × 0.71

= 22.7 N/mm^2

Here $\Delta\sigma ce$ = 32.0 N/mm^2 (Application of weld joint of G-grade or higher)

C_R = 1.00

Ct = 0.71

From the result of $\Delta\sigma max \leqq \Delta\sigma ce \cdot CR \cdot Ct$, it can be judged that the safety for fatigue was ensured at the welding connection for cable anchorage.

2) Cable Number C1~C5・C16~C20: Cable Cross Section φ 15.6×37

Fatigue evaluation equation

$$\angle\sigma max \leqq \angle\sigma ce \cdot C_R \cdot Ct$$

$\angle\sigma max$: Maximum stress range
$\angle\sigma ce$: Constant stress amplitude

Maximum difference between maximum and minimum tension caused by fatigue load per cable

Δ = 209.2 kN (contains impact coefficient for fatigue evaluation)

Maximum difference between maximum and minimum stress of member caused by fatigue load on each cable anchorage member

				Minimum Yield Stress of Cable		Allowable Stress
Δ	=	209.2	/	7310	×	210
	=	6.0	N/mm^2			

(contains impact coefficient for fatigue evaluation)

Calculation of maximum stress range Δσmax for entire bridge

$\angle\sigma max$ = Stress range coefficient × Maximum difference between maximum and minimum stress
= 3.0 × 6.0
= 18.0 N/mm^2 $\leqq \angle\sigma ce \cdot C_R \cdot Ct$

$\angle\sigma ce \cdot C_R \cdot Ct$ = 32.0 × 1.0 × 0.71
= 22.7 N/mm^2

Here $\angle\sigma ce$ = 32.0 N/mm^2 (Application of weld joint of G-grade or higher)
C_R = 1.00
Ct = 0.71

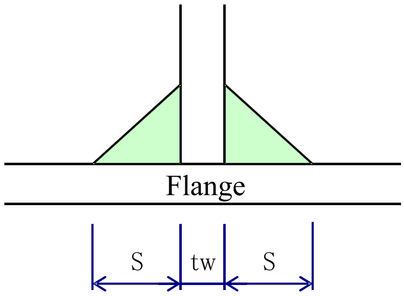
From the result of $\angle\sigma max \leqq \angle\sigma ce$・CR・Ct, it can be judged that the safety for fatigue was ensured at the welding connection for cable anchorage.

4.2.9.9 Welding Design

(1) Calculation for Main Girder Welds

1) Calculation Principle

The welding of the main girder flange and the web shall use the largest weld size determined through the comparison of weld size based on shear stress, composite stress, and plate thickness.



Source: JICA Study Team

Figure 4.2.128 Welding of Flange and Web

a) Weld Size Based on Shear Stress

$S1 = \tau \cdot tw / (\tau a \cdot 0.707 \cdot 2)$

Where,

τ : Shear Stress of Upper and Lower Component of Web (N/mm^2)

τa : Allowable Shear Stress (N/mm^2)

tw : Main Girder Web Thickness (mm)

tu : Main Girder Upper Flange Thickness (mm)

tl : Main Girder Bottom Flange Thickness (mm)

b) Weld Size Based on Composite Stress

$S2 = \tau \cdot tw / (\tau a \cdot 0.707 \cdot 2 \cdot \sqrt{1.2-(\sigma/\sigma a)^2}))$

Where,

σ : Vertical Stress due to Bending Moment from Upper and Lower Component of Web (N/mm^2)

σa : Allowable Vertical Stress (N/mm^2)

c) Weld Size Based on Plate Thickness

$t1 > St \geqq \sqrt{(2 \cdot t2)}$

Where,

t1 : Thickness of thinner base metal (mm)

t2 : Thickness of thicker base metal (mm)

d) Required Size of Fillet Weld

$Sreq = Max\{S1, S2, St\}$

Where, $6 \leqq S \leqq 12$

2) Calculation Results for Welds

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

Table 4.2.60 Calculation Results for Fillet Welds (Outer Web)

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

Source: JICA Study Team

Table 4.2.61 Calculation Results for Fillet Welds (Inner Web)

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

Source: JICA Study Team

(2) Calculation for Main Tower Welds

1) Calculation Principle

The welding of the flange and web that bear the shear stress of the corner component shall be conducted using partial penetration welding. Further, the required throat thickness shall be the largest weld size determined through the comparison of weld size based on shear stress, composite stress, and plate thickness.

a) Required Throat Thickness Calculation

- Required Throat Thickness based on Shear Stress

$a1 = \tau \cdot (tu \text{ or } tl) / \tau a$

Where, τ : Shear Stress of Top・Bott (N/mm^2)

τa : Allowable Shear Stress (N/mm^2)

tw : Web Thickness (mm)

tu : Top Thickness (mm)

tl : Bott Thickness (mm)

- Required Throat Thickness based on Composite Stress

$a2 = \tau \cdot (tu \text{ or } tl) / (\tau a \cdot \sqrt{(1.2-(\sigma/\sigma a)^2)})$

Where, σ : Vertical Stress due to Bending Moment from Upper and Lower Edge of Web (N/mm^2)

σa : Allowable Stress (N/mm^2)

- Required Throat Thickness

$areq = 1.5 \cdot Max(a1, a2)$

b) Required Partial Penetration Weld Size

1. Design of Throat Thickness for ≦ 25 : $a = S1 + 0.707 \cdot S2$

Analysis 1 : $S1 \geqq tw/2$

Analysis 2 : -

Analysis 3 : $S1 \geqq 2*\sqrt{t} \geqq 6$ mm

Analysis 4 : $T1 > S \geqq \sqrt{(2*T2)} \geqq 6$ mm

2. Design of Throat Thickness for ＞ 25 : $a = S1' + 0.707 \cdot (S1''+S2)$

Analysis 1 : $S1'+S1'' \geqq tw/2$

Analysis 2 : $S2 \geqq S1'' \cdot (SEC-1)$

Analysis 3 : $S1', S1'' \geqq 2*\sqrt{t} \geqq 6$ mm

Analysis 4 : $T1 > S \geqq \sqrt{(2*T2)} \geqq 6$ mm

Where, t1 : Thickness of Thinner Base Metal (mm)

t2 : Thickness of Thicker Base Metal (mm)

2) Calculation Results for Welds

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

Table 4.2.62 Calculation Results for Fillet Welds (Inner Web)

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

Source: JICA Study Team

4.2.9.10 Evaluation of Ultimate Strength (1.7 x Design Loads)

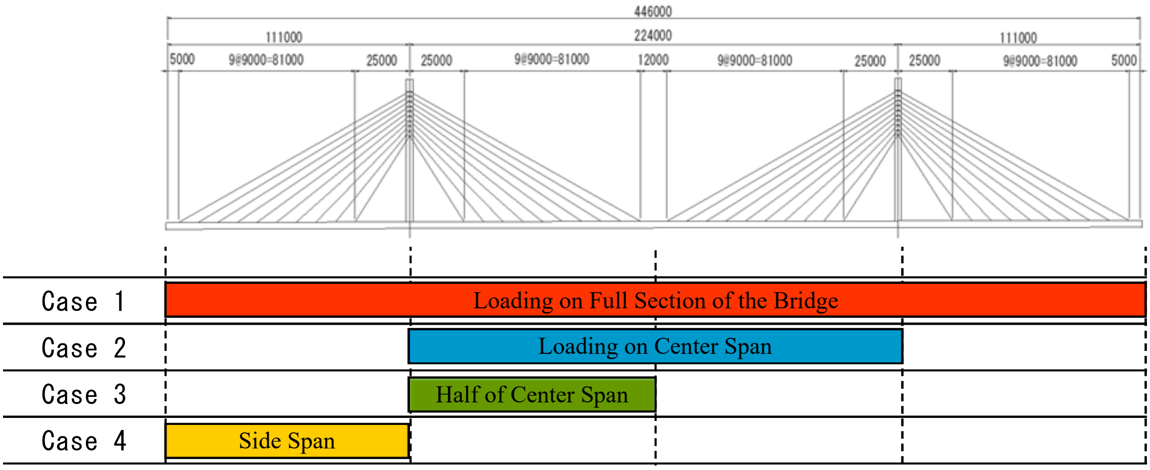
(1) Examination Overview

For cable-stayed bridges, the analysis of design loads only does not ensure the designated safety factor (safety factor for steel bridges: 1.7) because the section force does not increase linearly with the increase of load. Therefore, the induced stress for at least 1.7 times the design load was verified to be lower than the yield stress.

Analysis Method: Finite Elastic Displacement Method

Evaluation Procedure: For the 1.7 x (D + Li) + PS condition, members must not reach yield stress.

(D: Dead Load, L: Live Load, PS: Pre-Stressing Force, i: Impact Coefficient)

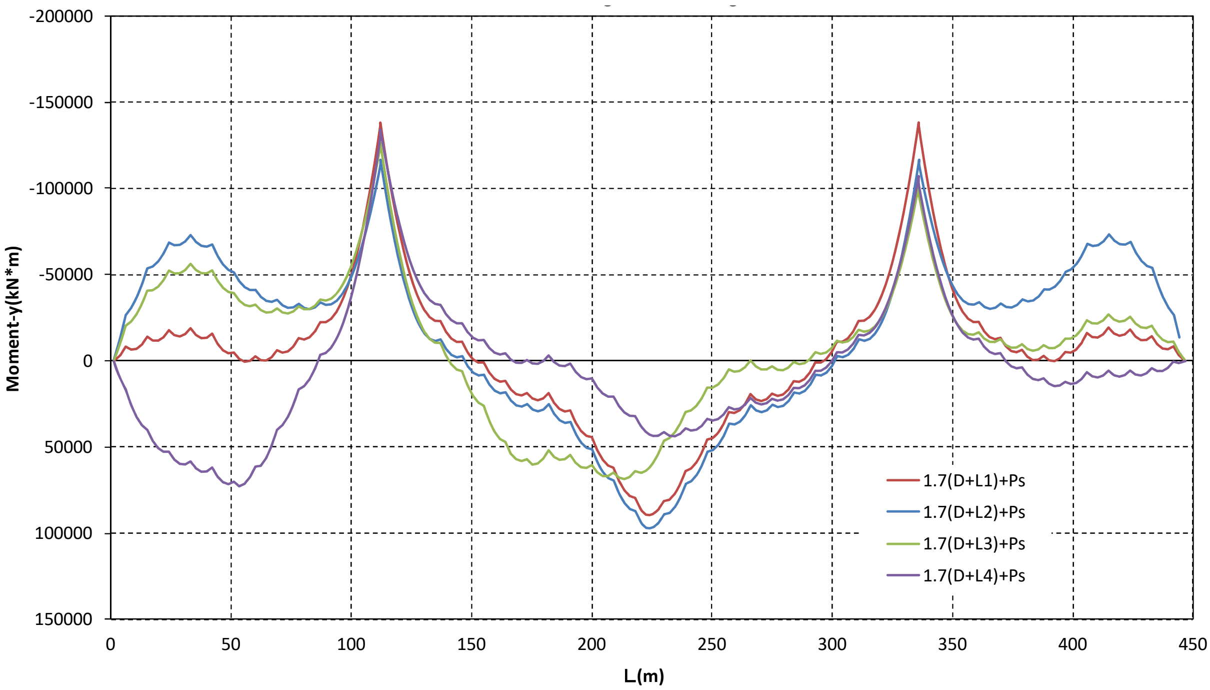


Source: JICA Study Team

Figure 4.2.129 Load Cases for Live Loads

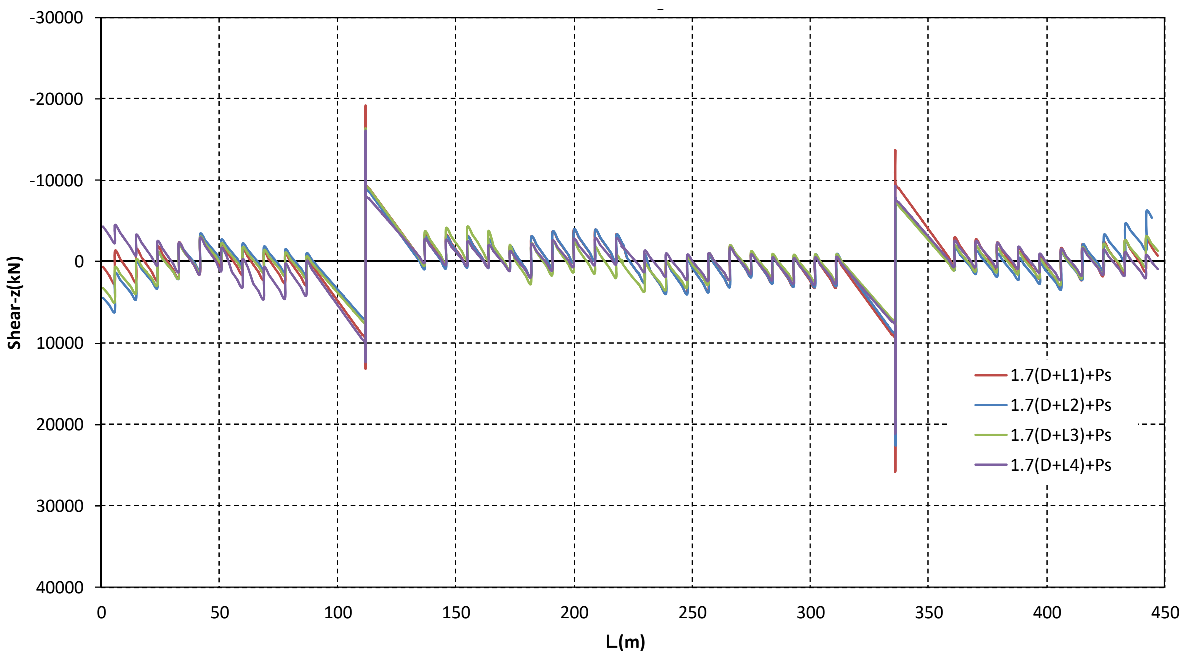
(2) Analysis Result

The stress resultants in the main girder for all load cases are shown below. As a result, stress in each member was less than the allowable value.



Source: JICA Study Team

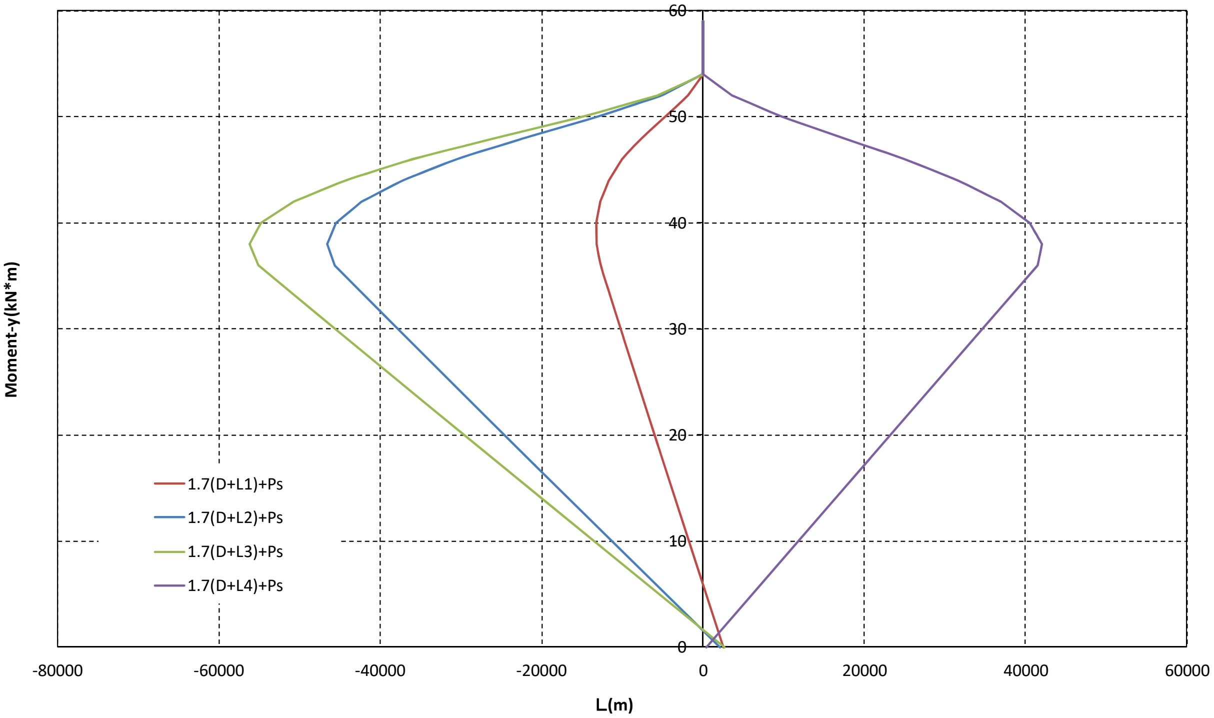
Figure 4.2.130 Main Girder - Bending Moment Diagram



Source: JICA Study Team

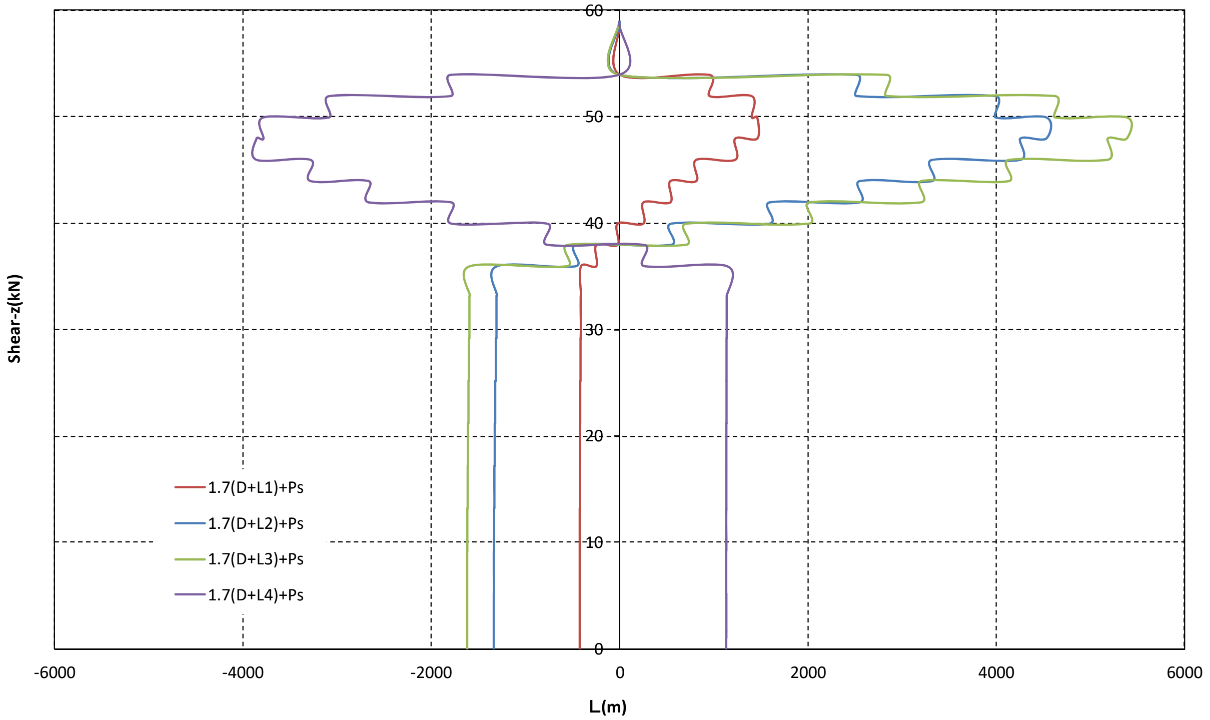
Figure 4.2.131 Main Girder - Shear Force Diagram

The stress resultants in the main tower for all load cases are shown below. As a result, stress in each member was less than the allowable value.



Source: JICA Study Team

Figure 4.2.132 Main Tower - Bending Moment Diagram



Source: JICA Study Team

Figure 4.2.133 Main Tower - Shear Force Diagram

(3) Evaluation Results for Girder

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

Table 4.2.63 Evaluation Results for Main Girder

Sider Span-Section 1

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

Side Span-Section 2

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

Intermediate Pier(at Tower)-Section 3

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

Main Span-Section4

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

Source: JICA Study Team

(4) Evaluation Results for Tower

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

Table 4.2.64 Evaluation Results for Tower

Upper Cable Section

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

Lower Cable Section

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

Bottom of Tower

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

Source: JICA Study Team

4.2.9.11 Structural Analysis Considering Plasticity of Superstructure

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

1) Purpose of Structural Analysis

Different from a general girder bridge, a cable-stayed bridge has a complicated structure that is composed of cables and axial-force members. It is more difficult to accurately specify the ultimate load and the destruction mode for a cable-stayed bridge from past construction reports or research papers compared to a general girder bridge. The elasto-plastic and finite displacement analysis, which can track plastic buckling of main girder or main tower and plastic deformation of cable elements in a proper manner, was implemented to check the safety when the designed bridge reaches its ultimate state.

2) Contents of Analysis

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

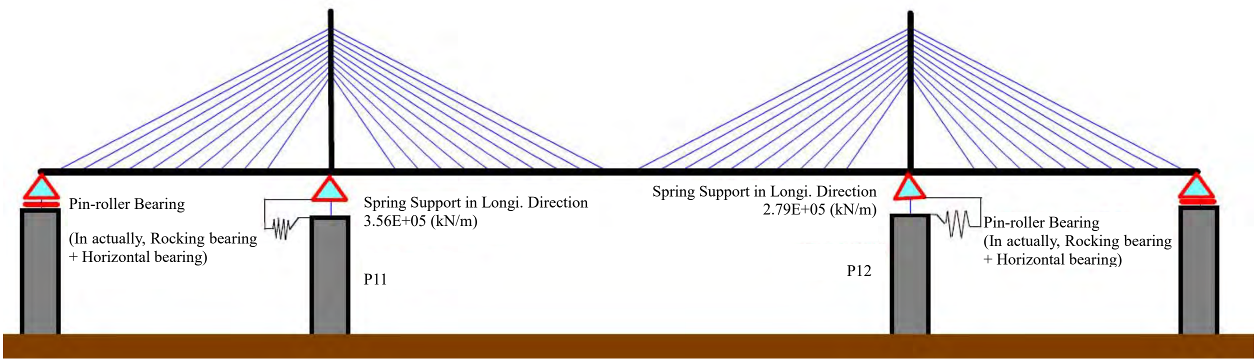
In order to evaluate the effect of loading range to the ultimate state, four cases of different loading conditions were assumed and employed in the analysis model shown in the figure below.

Table 4.2.65 Load Combination and Loading Range of Live Load

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

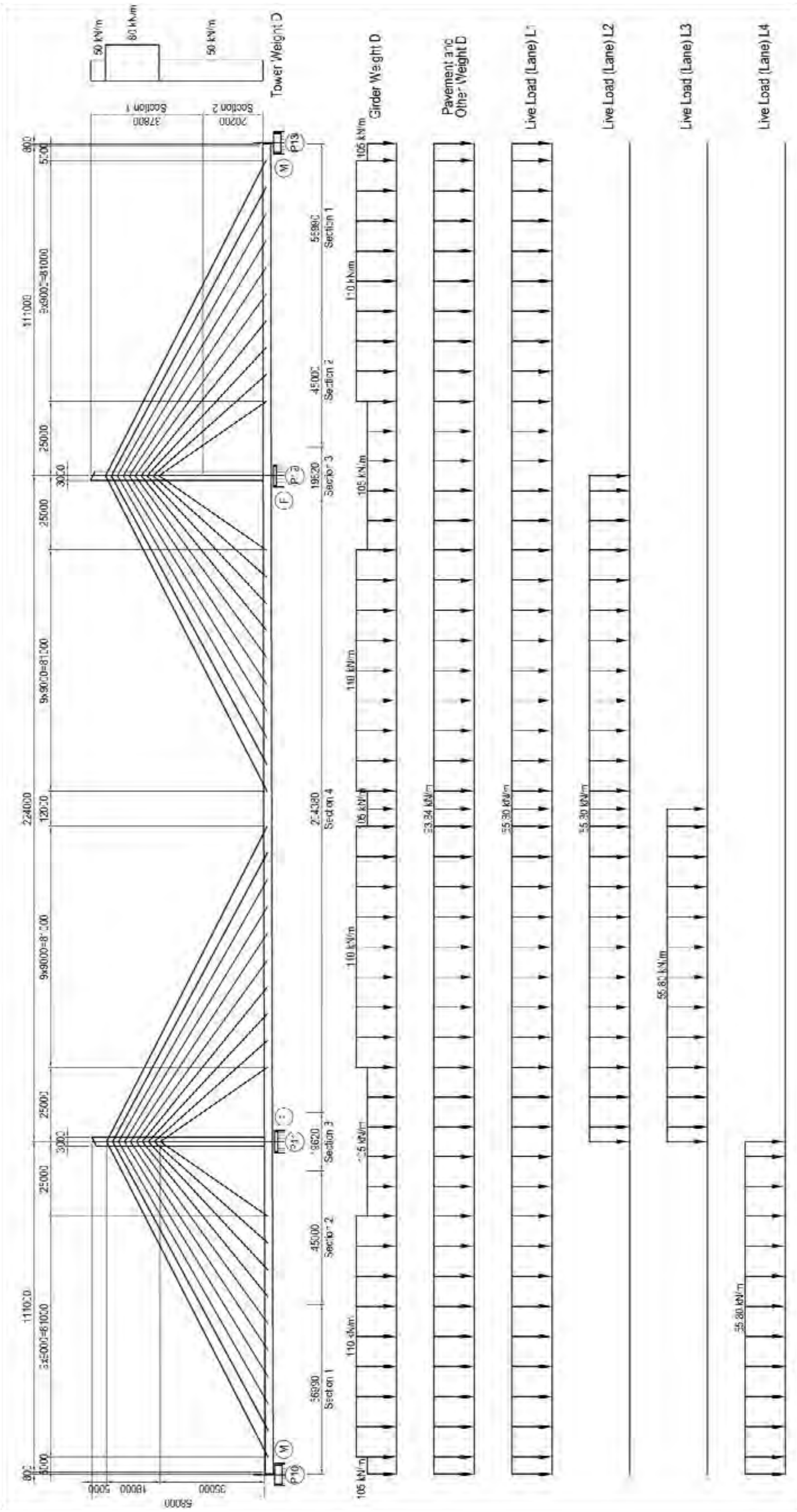
Note: α: Load scale factor, D: Dead load, L: Live load, PS: Pre-stress

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.134 Analysis Model of Cable-stayed Bridge



Source: JICA Study Team

Figure 4.2.135 Loading Range

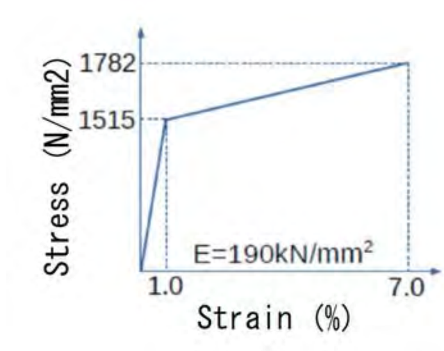
3) Analysis Model

Table 4.2.66 depicts the fundamental information of the analysis model and Figure 4.2.136 shows the material model of the cable element.

Table 4.2.66 Specifications for Analysis Model

Analysis code	COMP (Nagaoka University of Technology)
Analysis method	2D elasto-plastic and finite displacement analysis
Element type	Main tower and main girder: Fiber element based elasto-plastic frame element (566 elements) Cable: Elastic cable element considering a sag (40 elements)
Material model	Main tower and main girder: Perfect elasto-plasticity Cable: Bilinear model (refer to the following figure)
Boundary conditions	Under the main tower: Pin support + spring in longitudinal direction (P11) 3.56E+05 kN/m (P12) 2.79E+05 kN/m Both edge of the girder: Pin-roller support

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.136 Bilinear Model for Cable

4) Analysis Result

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

Table 4.2.67 Load Scale Factor α

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

Source: JICA Study Team

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

Table 4.2.68 Processes to Ultimate State

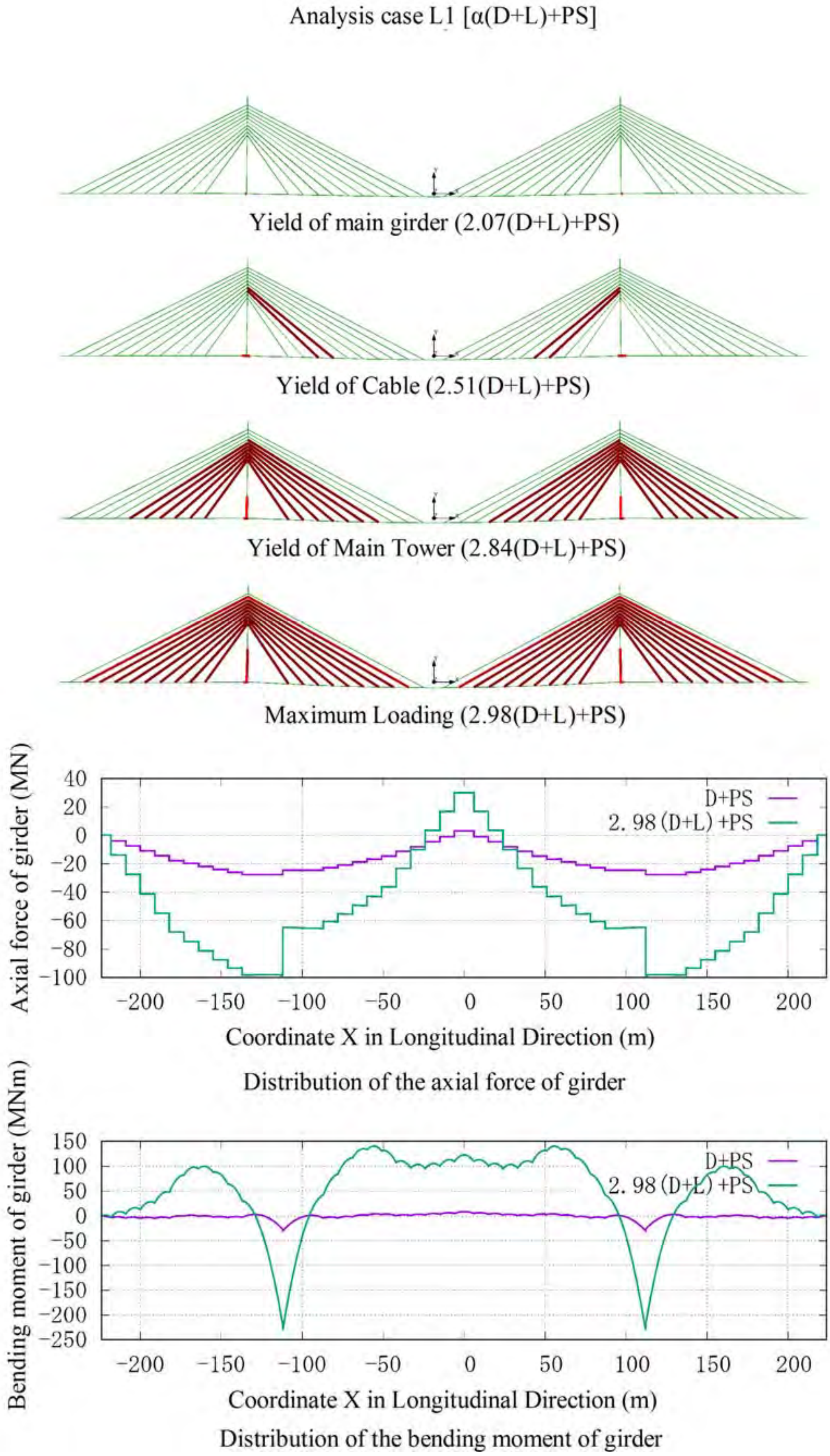
Loading Range for Live Load	Process to Ultimate State
L1: loading on the entire span	Main girder (near the main tower) → Cable (center) → Main tower (base) → [Ultimate state]
L2: loading on the center span	Main tower (middle) → Main girder (near the main tower) → Cable (middle) → [Ultimate state]
L3: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

Source: JICA Study Team

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

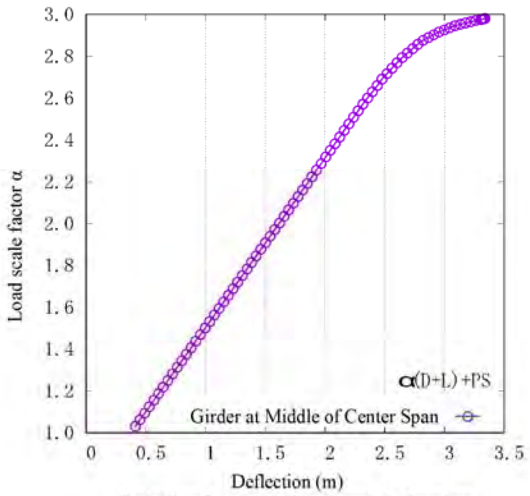
- The maximum load at ultimate state is about 2.7 times larger than D+L (dead load + live load). It means that the loading capacity of the designed bridge is high enough for the design load (D+L+PS).
- The designed bridge has sufficient loading capacity until the ultimate state. The relation between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

The deformation mode and displacement for each loading case are shown in the figure below.

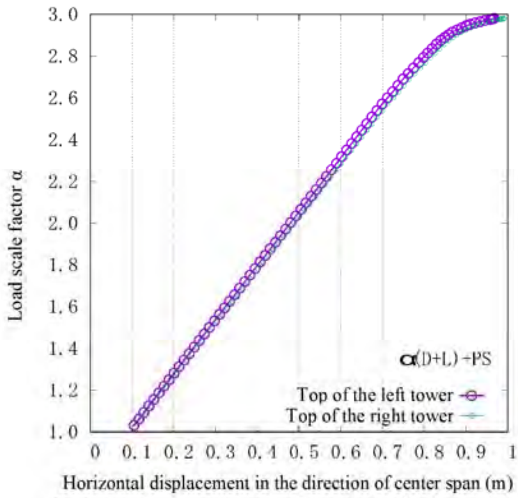


Source: JICA Study Team

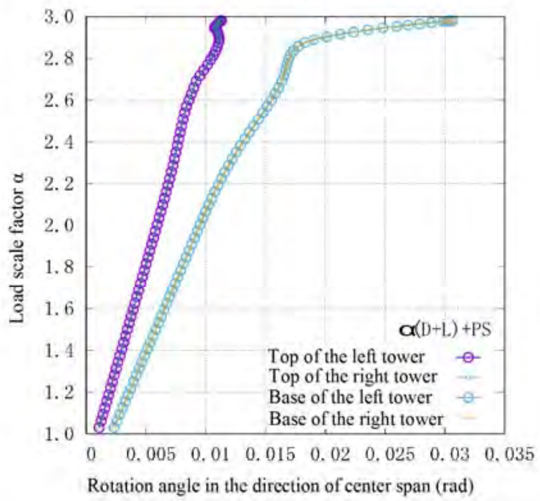
Figure 4.2.137 Deformation Mode and Stress Resultants of Girder (L1)



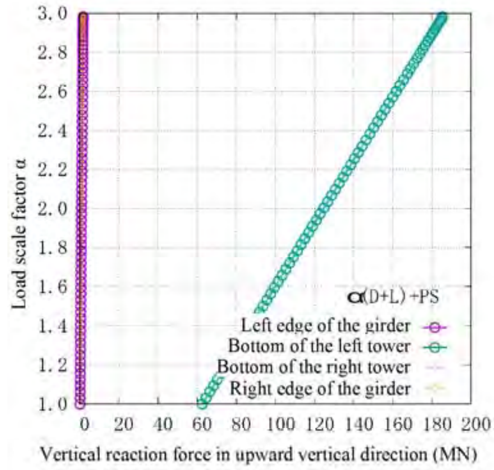
Relation between load and deflection at the center of the girder



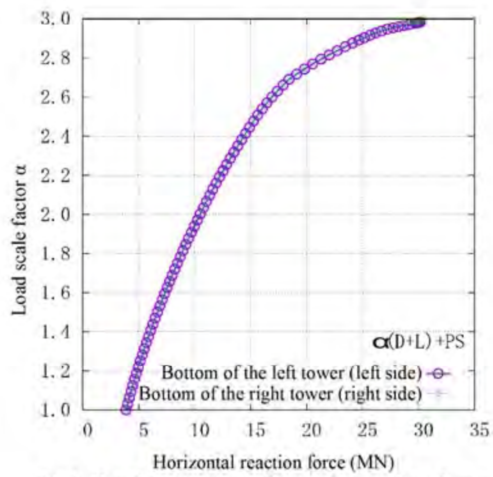
Relation between load and horizontal displacement at the top and bottom of the tower



Relation between load and rotation angle at the top and bottom of the tower



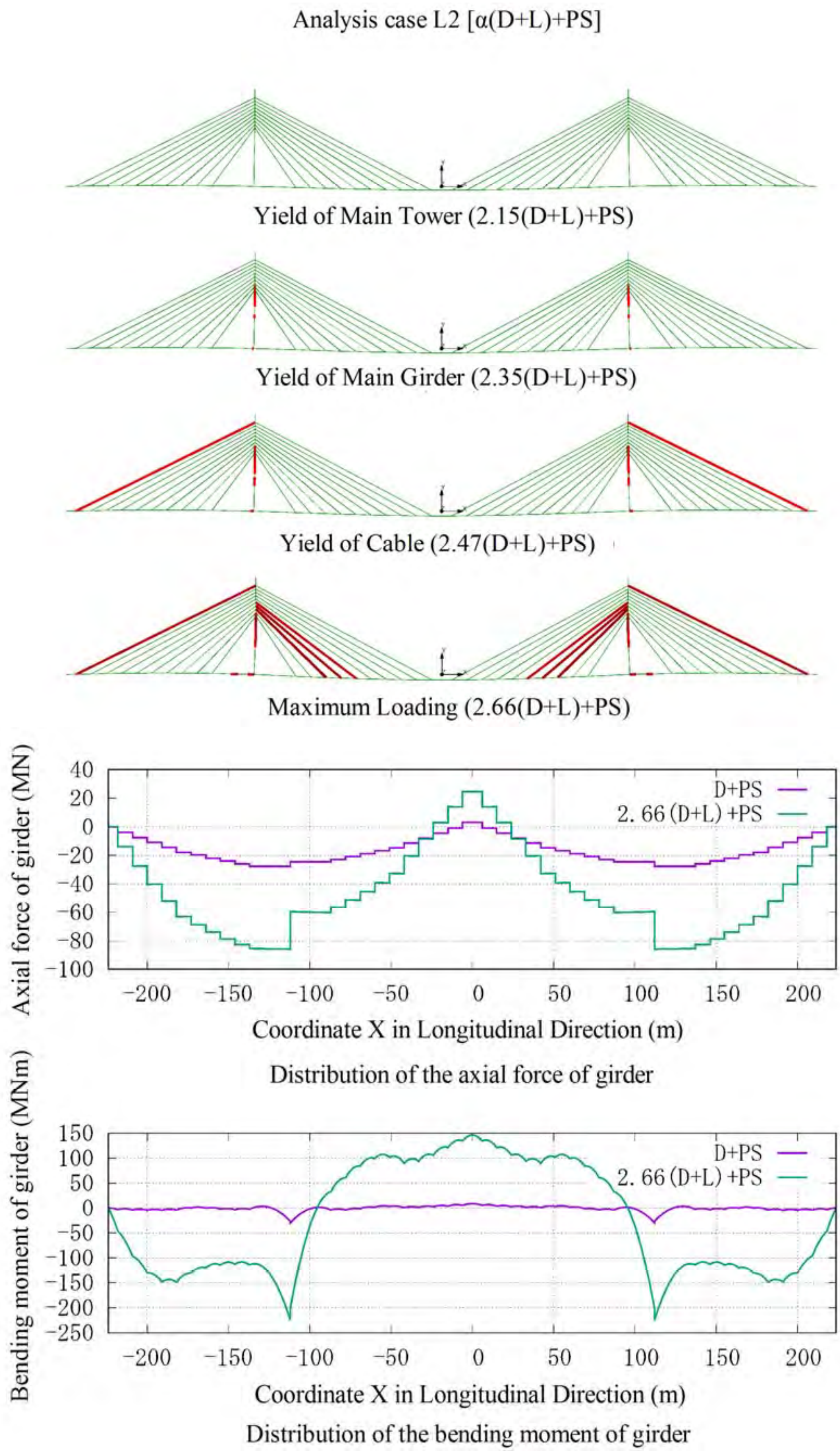
Relation between vertical reaction force and load regarding the main girder



Relation between horizontal reaction force and load regarding the main girder

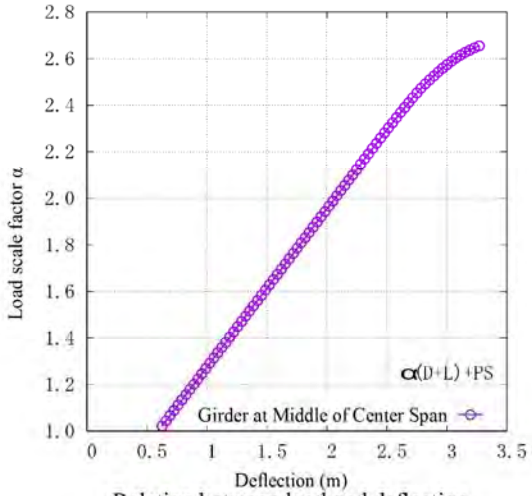
Source: JICA Study Team

Figure 4.2.138 Deformation Figure (L1)

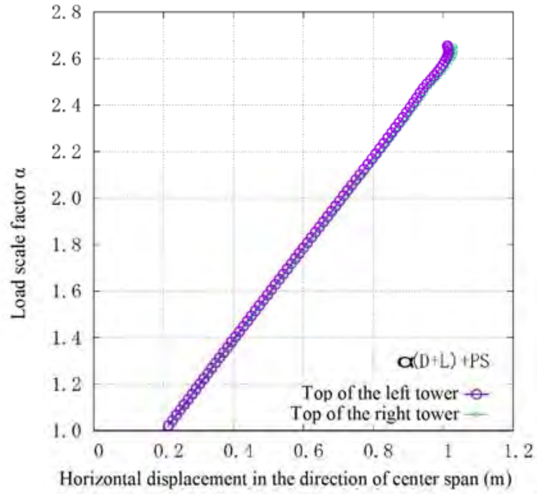


Source: JICA Study Team

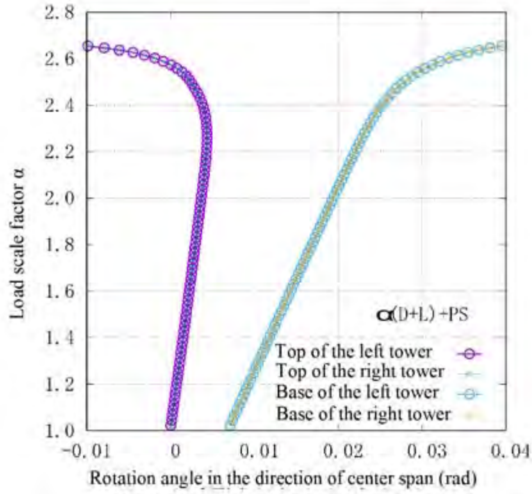
Figure 4.2.139 Deformation Mode and Stress Resultants of Girder (L2)



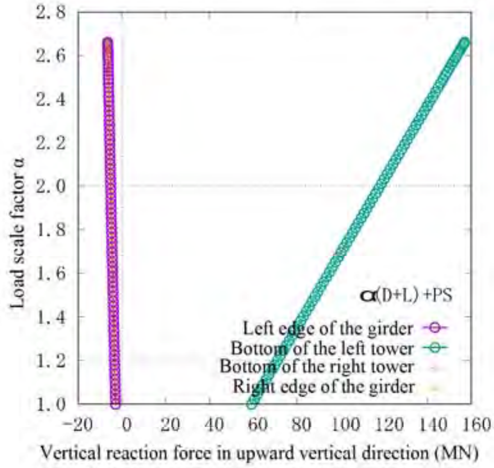
Relation between load and deflection
at the center of the girder



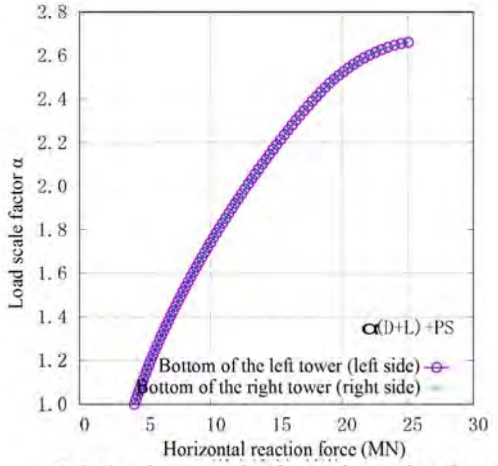
Relation between load and horizontal displacement
at the top and bottom of the tower



Relation between load and rotation angle
at the top and bottom of the tower



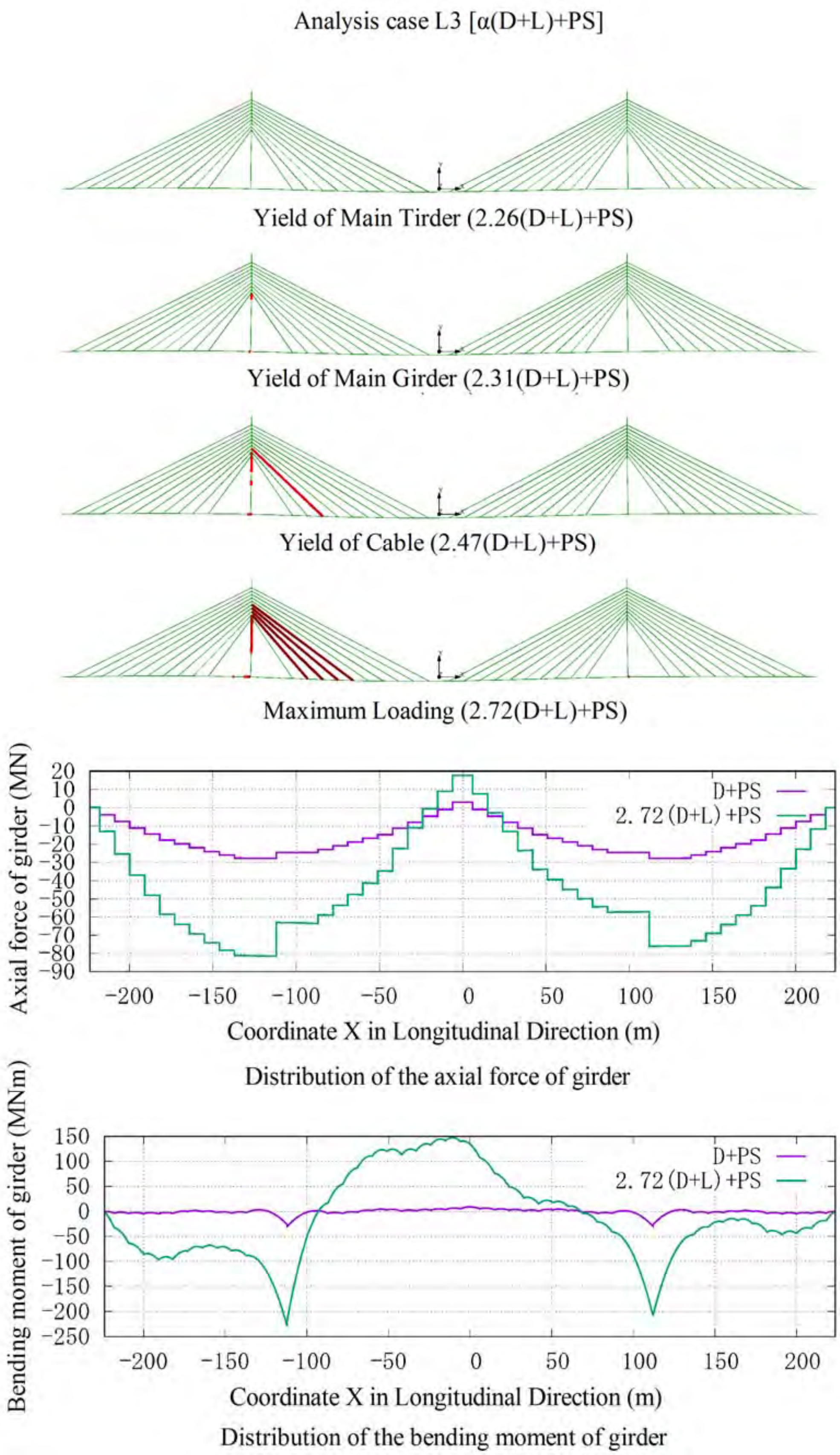
Relation between vertical reaction force
and load regarding the main girder



Relation between horizontal reaction force
and load regarding the main girder

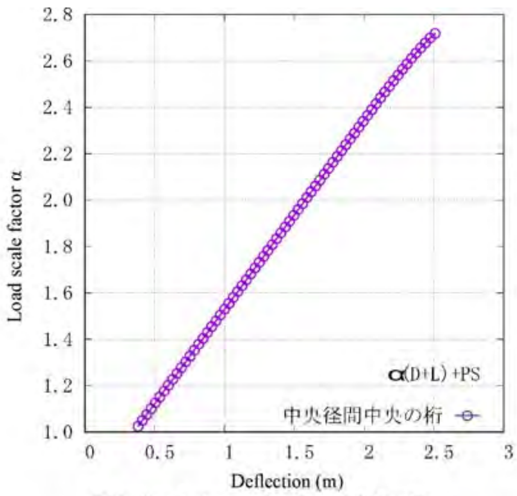
Source: JICA Study Team

Figure 4.2.140 Deformation Figure (L2)

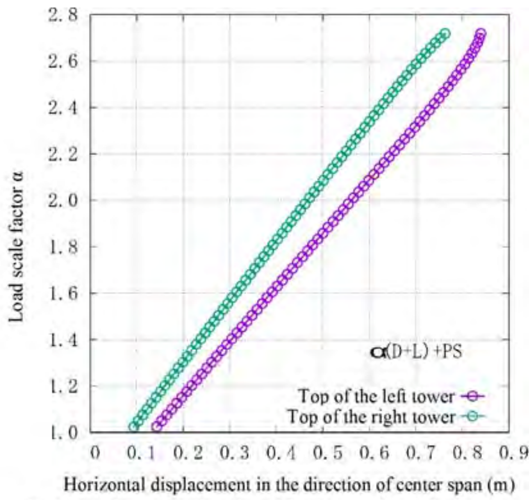


Source: JICA Study Team

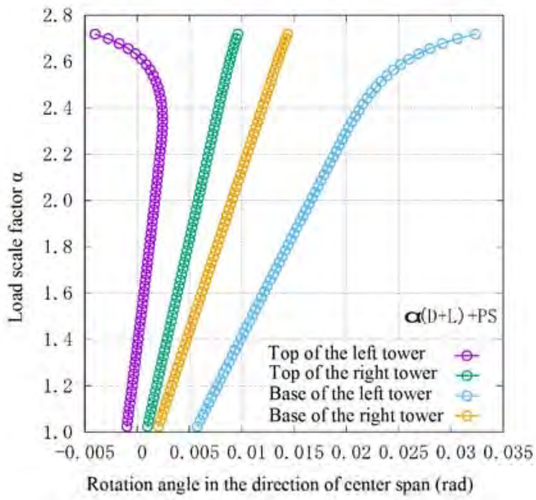
Figure 4.2.141 Deformation Mode and Stress Resultants of Girder (L3)



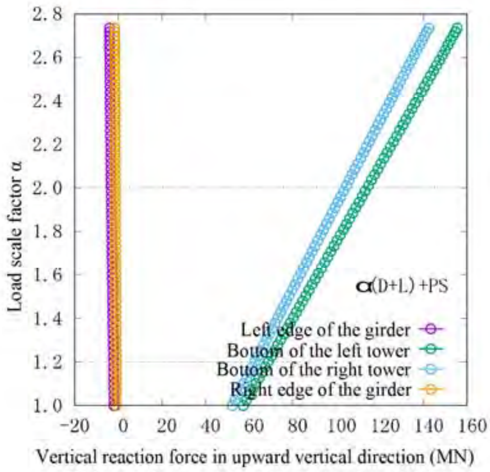
Relation between load and deflection at the center of the girder



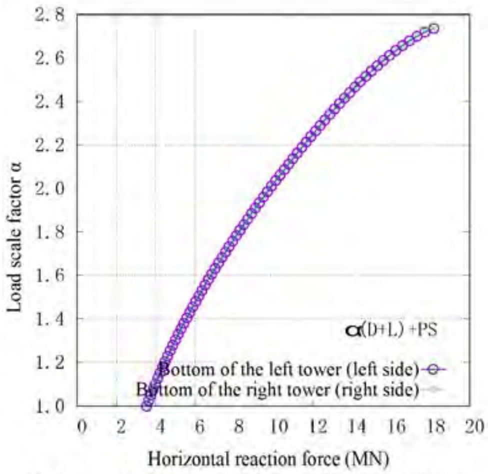
Relation between load and horizontal displacement at the top and bottom of the tower



Relation between load and rotation angle at the top and bottom of the tower



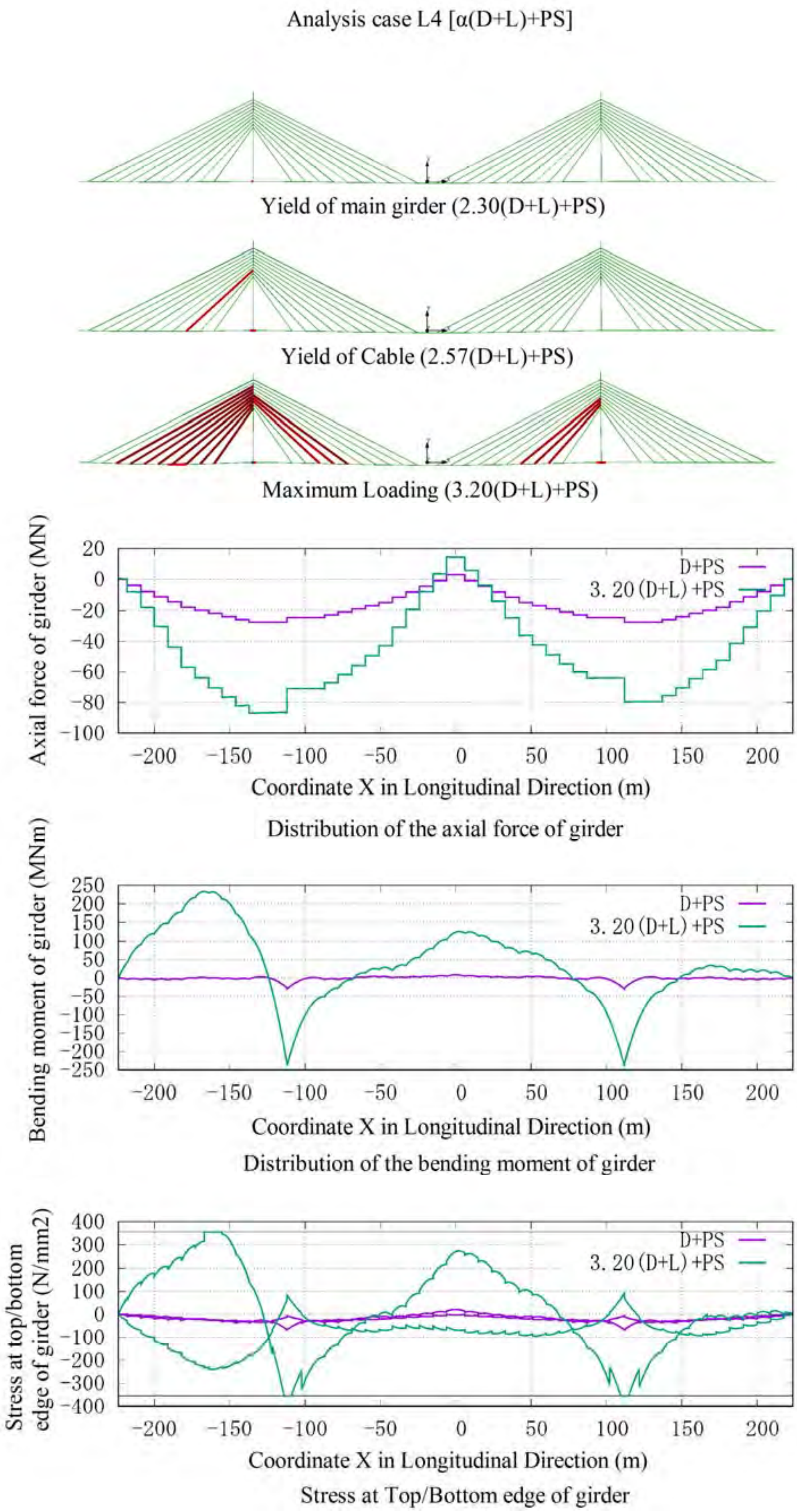
Relation between vertical reaction force and load regarding the main girder



Relation between horizontal reaction force and load regarding the main girder

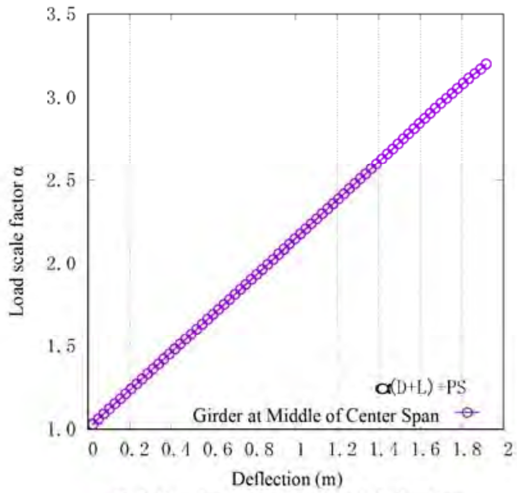
Source: JICA Study Team

Figure 4.2.142 Deformation Figure (L3)

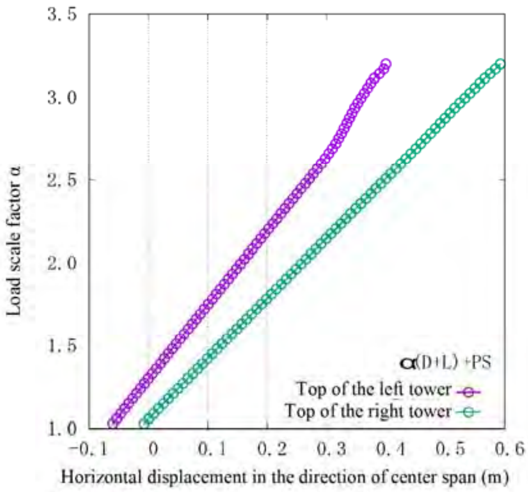


Source: JICA Study Team

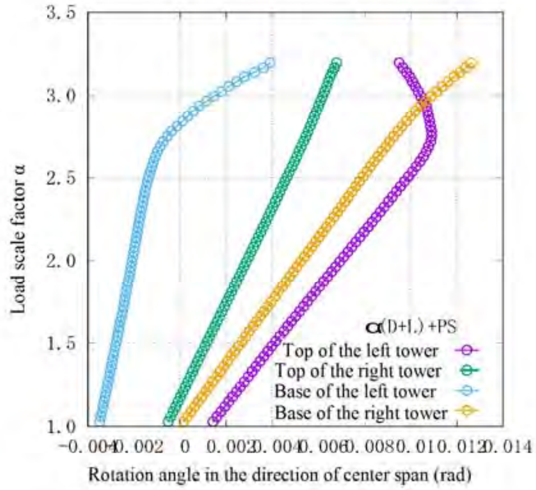
Figure 4.2.143 Deformation Mode and Stress Resultants of Girder (L4)



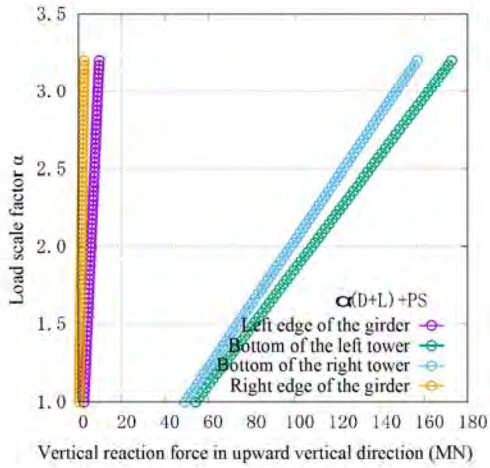
Relation between load and deflection at the center of the girder



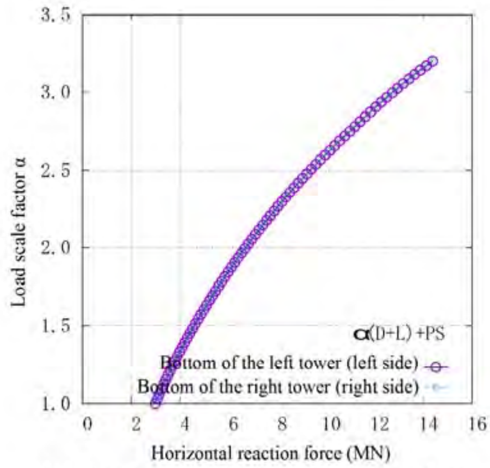
Relation between load and horizontal displacement at the top and bottom of the tower



Relation between load and rotation angle at the top and bottom of the tower



Relation between vertical reaction force and load regarding the main girder



Relation between horizontal reaction force and load regarding the main girder

Source: JICA Study Team

Figure 4.2.144 Deformation Figure (L4)

4.2.10 Summary of Substructure Design

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

(1) Design Conditions

1) Load Case

Table 4.2.69 Load Case

Scenario	Basic Load						Overdesign factor
	Dead load	Live Load	Temperature Load	Wind Load	Impact Load	Seismic Load	
Regular	○	○*1					1.00
Temperature Flux	○	○*1	○				1.15
Wind	○			○			1.25
Marine Vessel Impact	○				○		1.50
Seismic	○					○	1.50

*1 Depending on combination with design water level, cases with and without is verified

Source: JICA Study Team

a) Reaction Force for Substructure Design

Table 4.2.70 Reaction Force for Substructure Design

Scenario		P11			P12		
		Rv(KN)	RH(KN)	RM(KNm)	Rv(KN)	RH(KN)	RM(KNm)
Longi. Direction	Reguler HWL	51300	4700	0	51300	2200	0
	Reguler LWL	62800	-2200	0	62800	-4700	0
	Temperature HWL	51000	9300	0	51000	6800	0
	Temperature LWL	62900	-6800	0	62900	-9300	0
	Wind	52100	1100	0	52100	-1100	0
	Vessel Impact	51300	4700	0	62800	-4700	0
	Seismic	52000	18500	0	52100	-15400	0
Trans. Direction	Reguler HWL	51300	100	32000	51300	100	-32000
	Reguler LWL	62800	100	32000	62800	-100	32000
	Wind	52100	2200	33400	52100	2200	33300
	Vessel Impact	51300	100	32000	51300	100	-32000
	Seismic	52100	14300	93900	52100	14200	93100

Source: JICA Study Team

Reaction forces at P11 which has critical force (horizontal force and bending moment at seismic scenario) were selected as the design force for substructure of P11 and P12.

2) Design Lateral Seismic Factor

Seismic performance 1 kh = 0.30

Seismic performance 2 kh = 0.45 (used for evaluation of strength in the bridge seat member)

a) Design Water Level

Table 4.2.71 Design Water Level

Scenario	Water Level (MSL+m)	Flow rate (m/s)
Regular (Temperature Flux)	+3.18	---
	-2.39	---
Wind	+4.99	---
Marine Vessel Impact	+3.18	---
Seismic	+0.29	0.60

Source: JICA Study Team

3) Impact Load of Marine Vessel

Longitudinal direction: 4850 kN

Transverse direction: 9700 kN (impact height +3.98)

4) Utilized Material

a) Unit Weight

Reinforced Concrete γc = 24.5 kN/m^3

Filling Sand γd = 18.0 kN/m^3

Water γw = 10.0 kN/m^3

b) Utilized Material and Allowable Stress

Table 4.2.72 Utilized Material and Allowable Stress (Concrete)

(N/mm2)

		Pier	Pile Cap
Design strength σck		30.0	24.0
Compressive stress	Against bending	10.00	8.00
	Against axial force	8.50	6.50
Shearing stress	Borne by concrete only	0.25	0.23
	Bourne together with diagonal tension bars	1.90	1.70
	Punching shear stress (τa^3)	1.00	0.90
Bond stress	Deformed steel bars	1.80	1.60

Source: JICA Study Team

Table 4.2.73 Utilized Material and Allowable Stress (Steel)

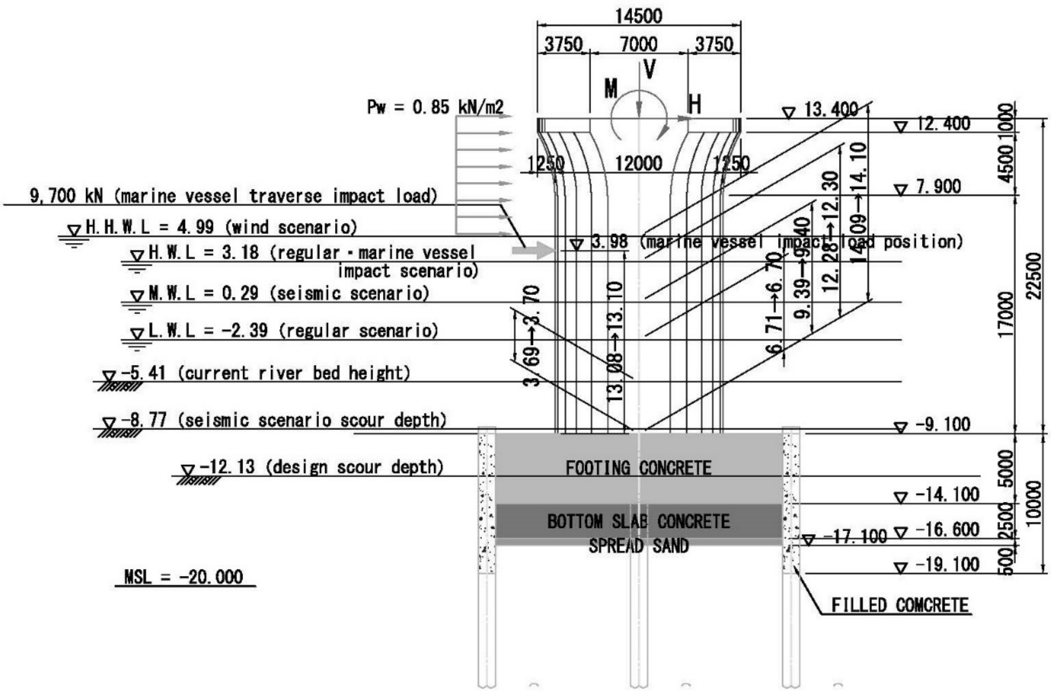
(N/mm2)

<table>
<tr><th colspan="4"></th><th>Pier</th><th>Pile Cap</th></tr>
<tr><td colspan="4">Type of steel member</td><td>SD345</td><td>SD345</td></tr>
<tr><td rowspan="6">Tensile stress</td><td colspan="3">Principal load exluding live load and impact load are in effect</td><td>100.0</td><td>100.0</td></tr>
<tr><td rowspan="2">Load combination does not include effect of impact and seismic event</td><td colspan="2">Regular members</td><td>180.0</td><td>180.0</td></tr>
<tr><td colspan="2">Members underwater or underneath ground water level</td><td>160.0</td><td>160.0</td></tr>
<tr><td colspan="2" rowspan="2">Load combination includes effect of impact and seismic event</td><td>Axial reinforcement</td><td>200.0</td><td>200.0</td></tr>
<tr><td>Other than the above</td><td>200.0</td><td>200.0</td></tr>
<tr><td colspan="3">Calculation of rebar lap joint and embedment length</td><td>200.0</td><td>200.0</td></tr>
<tr><td colspan="4">Compressive stress</td><td>200.0</td><td>200.0</td></tr>
</table>

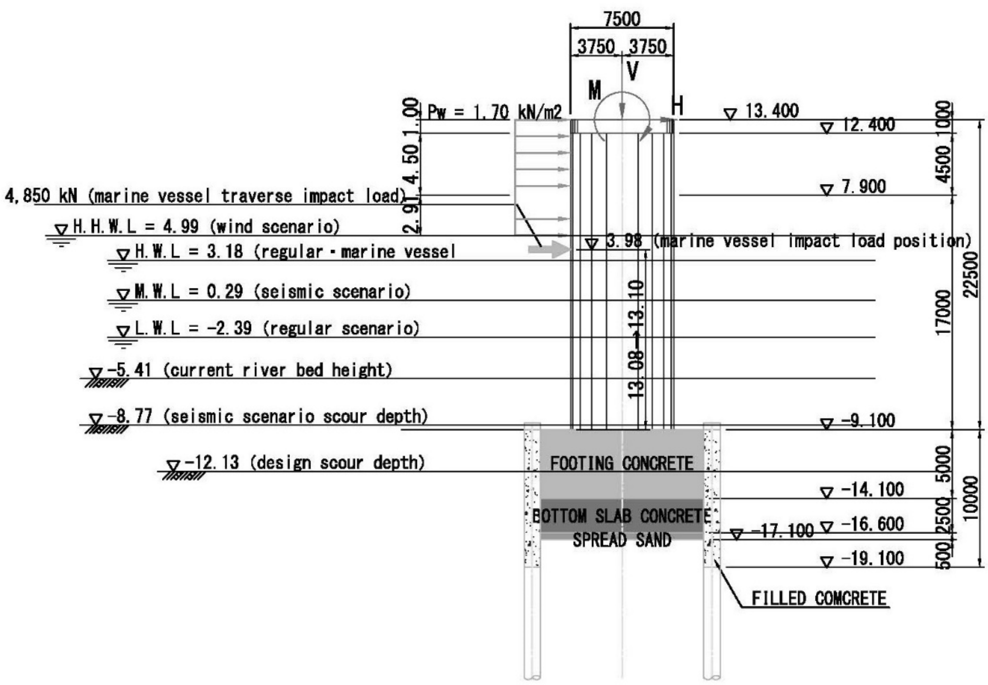
Source: JICA Study Team

5) Figure of Design Condition

Front View



Side View



Source: JICA Study Team

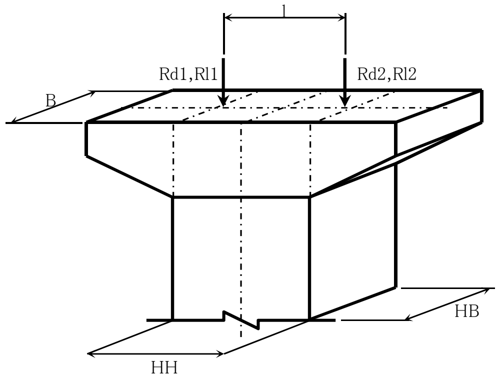
Figure 4.2.145 Design Condition

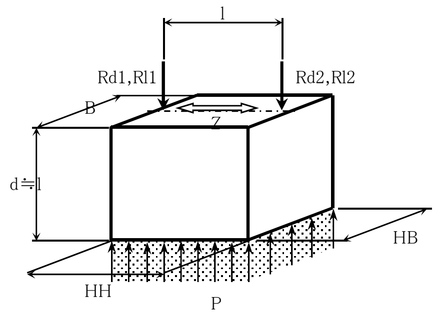
(2) Design of Pier

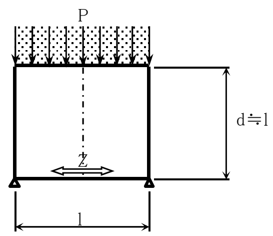
1) Design of Beam

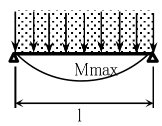
Table 4.2.74 Evaluation Result for Beam

Item	Sign	Unit	Value
Dead load reaction force	Rd1	kN	12500.0
	Rd2	kN	12500.0
Live load reaction force	Rl1	kN	8400.0
	Rl2	kN	8400.0
Total reaction force	ΣRd+ΣRl	kN	41800.0
Pier width	HH	m	10.390
Pier thickness	HB	m	7.500
Bridge seat width	B	m	7.500
Distance between bearing	l	m	8.170
Induced load	P	kN/m2	536.388
Splitting tensile force	Z*	kN	6902.108
Allowable stress	σsa	N/mm2	180.000
Required amount of steel reinforcement	Asr	cm2	383.450
Used amount of steel reinforcement	As	cm^2	D32 52 (bars) 412.984
Judgement	-	-	As≧Aw OK









* Calculation of Splitting tensile force Z

Can be determined by the assumption of tensile chord occuring in deep beam
The force of the tensile chord can be determined by the Mmax from beam theory

$$Z = \frac{M\ \max}{a}$$

Considering difference between ordinary beams, inner arm length is determined through the following

$$a = 0.15 \cdot d \cdot (3 + l/d) \qquad (2 > l/d > l)$$

$$a = 0.6 \cdot l \qquad (l/d \leq l)$$

Due to stress transmission d≒l

$$a = 0.6 \cdot l$$

$$M_{\max} = \frac{P \cdot l^2}{8}$$

$$\therefore Z = \frac{\frac{P \cdot l^2}{8}}{0.6 \cdot l} = 0.21 \cdot P \cdot l$$

$$P = \frac{\sum Rd + \sum Rl}{HH \cdot HB}$$

$$Z = 0.21 \cdot P \cdot l \cdot B$$

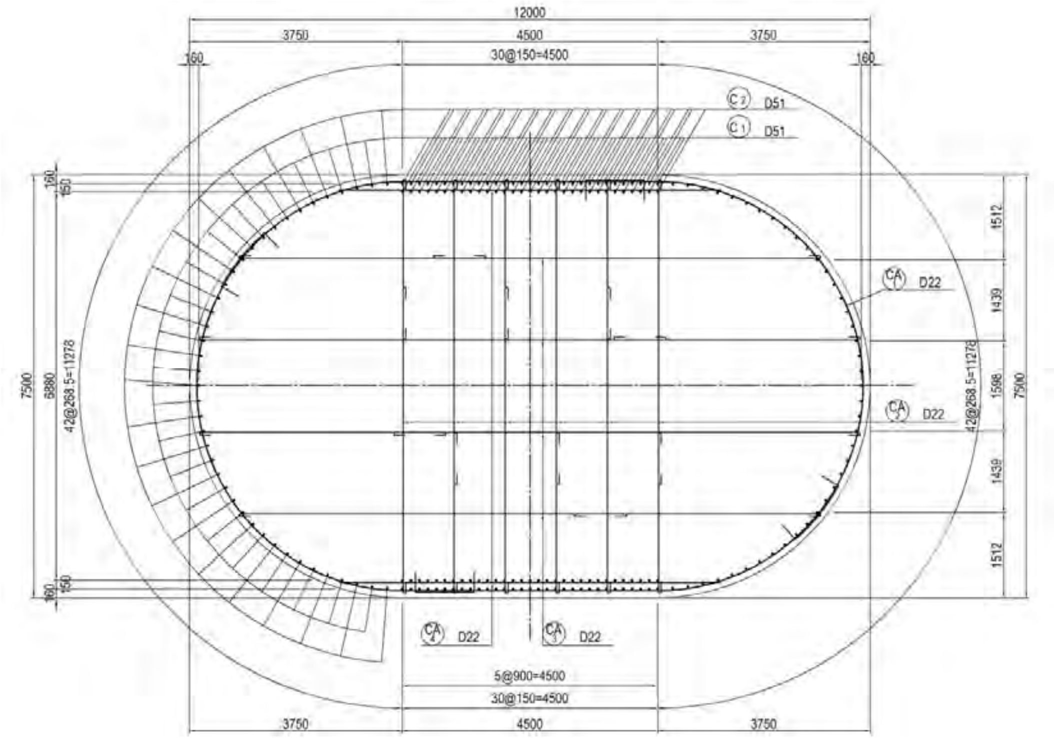
$$Asr = \frac{Z \cdot 10}{\sigma sa}$$

Source: JICA Study Team

2) Design of Column

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

a) Cross Section and Rebar Configuration



Source: JICA Study Team

Figure 4.2.146 Design Condition

[Overview of Calculation Result]

The following table shows the calculation results for beam.

Table 4.2.75 Calculation Result for Beam

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

Source: JICA Study Team

b) Cross Section Evaluation Results

The evaluation results for the column cross section are shown below.

Table 4.2.76 Examination of Bending Moment (Longitudinal)

Category	Unit	Regular Scenario HWL Water Level Considered	Regular LWL Scenario Water Level Considered	Temperature Flux HWL Scenario Water Level Considered	Temperature Flux LWL Scenario Water Level Considered
Load Condition	―――	Dead load	Regular load	Dead + temp load	Dead + live + temp load
Axial Force N	kN	96841.24	108341.24	96541.24	108441.24
Bending Moment M	kN.m	105750	49500	209250	153000
Compression Edge～Neutral Axis x	mm	7838	13519	5148	6855
Compressive Stress σc	N/mm²	2.21	1.78	3.51	2.83
Tensile Stress σs	N/mm²	-2.1	-12.21	22.41	3
Overdesign Factor α	―――	1	1	1.15	1.15
Allowable Compressive Stress σca	N/mm²	10	10	11.5	11.5
Allowable Tensile Stress σsa	N/mm²	-200	-200	184	184
Cracking Moment Mc	kN.m	289550.53	301888.19	289228.67	301995.47
Yielding Moment My0	kN.m	690877.8	720620.98	690097.76	720877.34
Ultimate Bending Moment Mu	kN.m	822644.51	857066.21	821736.94	857362.01
Minimum Reinforcement for Bending Element	―――	1.7M≦Mc	1.7M≦Mc	Mc≦Mu	1.7M≦Mc
Minimum Reinforcement for Axial Element	mm²	76705.9	85814.8	66494.2	74690.5
Axial Force Nu	kN	97641.24	97641.24	97641.24	97641.24
0.008A1' (Axial Force Na=N)	mm²	76705.9	85814.8	66494.2	74690.5
0.008A2' (Axial Force Nu)	mm²	27640.8	27640.8	27640.8	27640.8
Total Reinforcement Content As ≧ Asmin	―――	OK	OK	OK	OK
Maximum Reinforcement Content Evaluation (My0≦Mu)	―――	OK	OK	OK	OK

Category	Unit	Wind Scenario Water Level Considered	Marine Vessel Impact Scenario Water Level Considered	Sesimic Scenario Water Level Considered
Load Condition	―――	Wind load	Impact load	Lv1 Seismic Load
Axial Force N	kN	97641.24	96841.24	97541.24
Bending Moment M	kN.m	28173.51	169285	585015.11
Compression Edge～Neutral Axis x	mm	19219	6007	2670
Compressive Stress σc	N/mm²	1.44	2.93	10.46
Tensile Stress σs	N/mm²	-13.36	9.73	274.38
Overdesign Factor α	―――	1.25	1.5	1.5
Allowable Compressive Stress σca	N/mm²	12.5	15	15
Allowable Tensile Stress σsa	N/mm²	-250	300	300
Cracking Moment Mc	kN.m	290408.8	289550.53	290301.51
Yielding Moment My0	kN.m	692959.57	690877.8	692701.74
Ultimate Bending Moment Mu	kN.m	825061.65	822644.51	824762.94
Minimum Reinforcement for Bending Element	―――	1.7M≦Mc	1.7M≦Mc	Mc≦Mu
Minimum Reinforcement for Axial Element	mm²	61871.7	51137.3	51506.9
Axial Force Nu	kN	97641.24	97641.24	97641.24
0.008A1' (Axial Force Na=N)	mm²	61871.7	51137.3	51506.9
0.008A2' (Axial Force Nu)	mm²	27640.8	27640.8	27640.8
Total Reinforcement Content As ≧ Asmin	―――	OK	OK	OK
Maximum Reinforcement Content Evaluation (My0≦Mu)	―――	OK	OK	OK

Source: JICA Study Team

Table 4.2.77 Examination of Bending Moment (Transverse)

Category	Unit	Regular Scenario HWL Water Level Considered	Regular LWL Scenario Water Level Considered	Wind Scenario Water Level Considered
Load Condition	―――	Dead load	Regular load	Wind load
Axial Force N	kN	96841.24	108341.24	97641.24
Bending Moment M	kN.m	34250	34250	83879.97
Compression Edge～ Neutral Axis x	mm	33352	36600	17261
Compressive Stress σc	N/mm^2	1.4	1.54	1.78
Tensile Stress σs	N/mm^2	-13.57	-15.62	-8.37
Overdesign Factor α	―――	1	1	1.25
Allowable Compressive Stress σca	N/mm^2	10	10	12.5
Allowable Tensile Stress σsa	N/mm^2	-200	-200	-250
Cracking Moment Mc	kN.m	434259.12	452762.77	435546.33
Yielding Moment My0	kN.m	804648.26	847252.37	807625.48
Ultimate Bending Moment Mu	kN.m	1195519.89	1242889.66	1198842.86
Minimum Reinforcement for Bending Element	―――	1.7M≦Mc	1.7M≦Mc	1.7M≦Mc
Minimum Reinforcement for Axial Element	mm^2	76705.9	85814.8	61871.7
Axial Force Nu	kN	97641.24	97641.24	97641.24
0.008A1' (Axial Force Na=N)	mm^2	76705.9	85814.8	61871.7
0.008A2' (Axial Force Nu)	mm^2	27640.8	27640.8	27640.8
Total Reinforcement Content As ≧ Asmin	―――	OK	OK	OK
Maximum Reinforcement Content Evaluation (My0≦Mu)	―――	OK	OK	OK

Category	Unit	Marine Vessel Impact Scenario Water Level Considered	Sesimic Scenario Water Level Considered
Load Condition	―――	Impact load	Lv1 Seismic Load
Axial Force N	kN	96841.24	97641.24
Bending Moment M	kN.m	161320	577239.17
Compression Edge～ Neutral Axis x	mm	11806	4720
Compressive Stress σc	N/mm^2	2.34	8.85
Tensile Stress σs	N/mm^2	0.1	200.15
Overdesign Factor α	―――	1.5	1.5
Allowable Compressive Stress σca	N/mm^2	15	15
Allowable Tensile Stress σsa	N/mm^2	300	300
Cracking Moment Mc	kN.m	434259.12	435546.33
Yielding Moment My0	kN.m	804648.26	807625.48
Ultimate Bending Moment Mu	kN.m	1195519.89	1198842.86
Minimum Reinforcement for Bending Element	―――	1.7M≦Mc	Mc≦Mu
Minimum Reinforcement for Axial Element	mm^2	51137.3	51559.7
Axial Force Nu	kN	97641.24	97641.24
0.008A1' (Axial Force Na=N)	mm^2	51137.3	51559.7
0.008A2' (Axial Force Nu)	mm^2	27640.8	27640.8
Total Reinforcement Content As ≧ Asmin	―――	OK	OK
Maximum Reinforcement Content Evaluation (My0≦Mu)	―――	OK	OK

Source: JICA Study Team

Table 4.2.78 Examination of Shear Force (Longitudinal)

Category	Unit	Regular Scenario HWL Water Level Considered	Regular LWL Scenario Water Level Considered	Temperature Flux HWL Scenario Water Level Considered	Temperature Flux LWL Scenario Water Level Considered	Wind Scenario Water Level Considered
Load Condition	―――	Dead load	Regular load	Dead + temp load	Dead + live + temp load	Wind load
b	mm	11147	11147	11147	11147	11147
d	mm	6937	6937	6937	6937	6937
S	kN	4700	2200	9300	6800	1285.38
N	kN	96841.24	108341.24	96541.24	108441.24	97641.24
M	kN.m	105750	49500	209250	153000	28173.51
α	―――	1	1	1.15	1.15	1.25
pt	%	0.27	0.27	0.27	0.27	0.27
ce	―――	0.561	0.561	0.561	0.561	0.561
cpt	―――	0.97	0.97	0.97	0.97	0.97
CN	―――	1	1	1	1	1
τm	N/mm^2	0.061	0.028	0.12	0.088	0.017
$τa_1$	N/mm^2	0.136	0.136	0.157	0.157	0.17
$τa_2$	N/mm^2	1.9	1.9	2.185	2.185	2.375
σsa	N/mm^2	―――	―――	―――	―――	―――
s	mm	―――	―――	―――	―――	―――
Sca	kN	―――	―――	―――	―――	―――
Sh'	kN	―――	―――	―――	―――	―――
AwReq	mm^2	―――	―――	―――	―――	―――
Aw	mm^2	―――	―――	―――	―――	―――

Category	Unit	Marine Vessel Impact Scenario Water Level Considered	Sesimic Scenario Water Level Considered
Load Condition	―――	Impact load	Lv1 Seismic Load
b	mm	11147	11147
d	mm	6937	6937
S	kN	9550	33957.85
N	kN	96841.24	97541.24
M	kN.m	169285	585015.11
α	―――	1.5	1.5
pt	%	0.27	0.27
ce	―――	0.561	0.561
cpt	―――	0.97	0.97
CN	―――	1	1
τm	N/mm^2	0.123	0.439
$τa_1$	N/mm^2	0.201	0.201
$τa_2$	N/mm^2	2.85	2.85
σsa	N/mm^2	―――	300
s	mm	―――	150
Sca	kN	―――	15576.54
Sh'	kN	―――	18381.31
AwReq	mm^2	―――	1523.51
Aw	mm^2	―――	3096.8

Here
- S : Shear Force
- N : Axial Load
- M : Bending Moment
- b : Sectional Width of Element
- d : Effective Height
- α : Overdesign factor for allowable stress
- pt : Primary tension bar ratio
- ce : Correction factor of allowable shear force for effective height d
- cpt : Correction factor of allowable shear force for tension bar ratio
- CN : Correction factor due to longitudinal compressive load
- τm : Average shear force
- τa1 : Allowable shear force when only concrete bears shear force
- τa2 : Allowable shear force when shear reinforcement rebar and concrete bears shear force
- σsa : Allowable tensile stress of rebar
- s : Spacing of shear reinforcement rebar
- Sca : Shear force borne by concrete
- Sh' : Shear force borne by reinforcement rebar
- Awreq : Necessary shear reinforcement content to meet condition τa1 < τm
- Aw : Shear reinforcement content

Source: JICA Study Team

Table 4.2.79 Examination of Shear Force (Transverse)

Category	Unit	Regular Scenario HWL Water Level Considered	Regular LWL Scenario Water Level Considered	Wind Scenario Water Level Considered	Marine Vessel Impact Scenario Water Level Considered
Load Condition	———	Dead load	Regular load	Wind load	Impact load
b	mm	6991	6991	6991	6991
d	mm	11056	11056	11056	11056
S	kN	100	100	2253.55	9800
N	kN	96841.24	108341.24	97641.24	96841.24
M	kN.m	34250	34250	83879.97	161320
α	———	1	1	1.25	1.5
pt	%	0.27	0.27	0.27	0.27
ce	———	0.5	0.5	0.5	0.5
cpt	———	0.97	0.97	0.97	0.97
CN	———	1	1	1	1
τm	N/mm^2	0.001	0.001	0.029	0.127
τa_1	N/mm^2	0.121	0.121	0.152	0.179
τa_2	N/mm^2	1.9	1.9	2.375	2.85
σsa	N/mm^2	———	———	———	———
s	mm	———	———	———	———
Sca	kN	———	———	———	———
Sh’	kN	———	———	———	———
AwReq	mm^2	———	———	———	———
Aw	mm^2	———	———	———	———

Category	Unit	Wind Scenario Water Level Considered
Load Condition	———	Wind load
b	mm	6991
d	mm	11056
S	kN	27972.52
N	kN	97641.24
M	kN.m	577239.17
α	———	1.5
pt	%	0.27
ce	———	0.5
cpt	———	0.97
CN	———	1
τm	N/mm^2	0.362
τa_1	N/mm^2	0.179
τa_2	N/mm^2	2.85
σsa	N/mm^2	300
s	mm	150
Sca	kN	13872.31
Sh’	kN	14100.21
AwReq	mm^2	733.31
Aw	mm^2	2322.6

Here
- S : Shear Force
- N : Axial Load
- M : Bending Moment
- b : Sectional Width of Element
- d : Effective Height
- α : Overdesign factor for allowable stress
- pt : Primary tension bar ratio
- ce : Correction factor of allowable shear force for effective height d
- cpt : Correction factor of allowable shear force for tension bar ratio
- CN : Correction factor due to longitudinal compressive load
- τm : Average shear force
- τa1 : Allowable shear force when only concrete bears shear force
- τa2 : Allowable shear force when shear reinforcement rebar and concrete bears shear force
- σsa : Allowable tensile stress of rebar
- s : Spacing of shear reinforcement rebar
- Sca : Shear force borne by concrete
- Sh’ : Shear force borne by reinforcement rebar
- Awreq : Necessary shear reinforcement content to meet condition τa1 < τm
- Aw : Shear reinforcement content

Source: JICA Study Team

3) Bridge Seat Design

a) Dimension of Bridge Seat Width

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

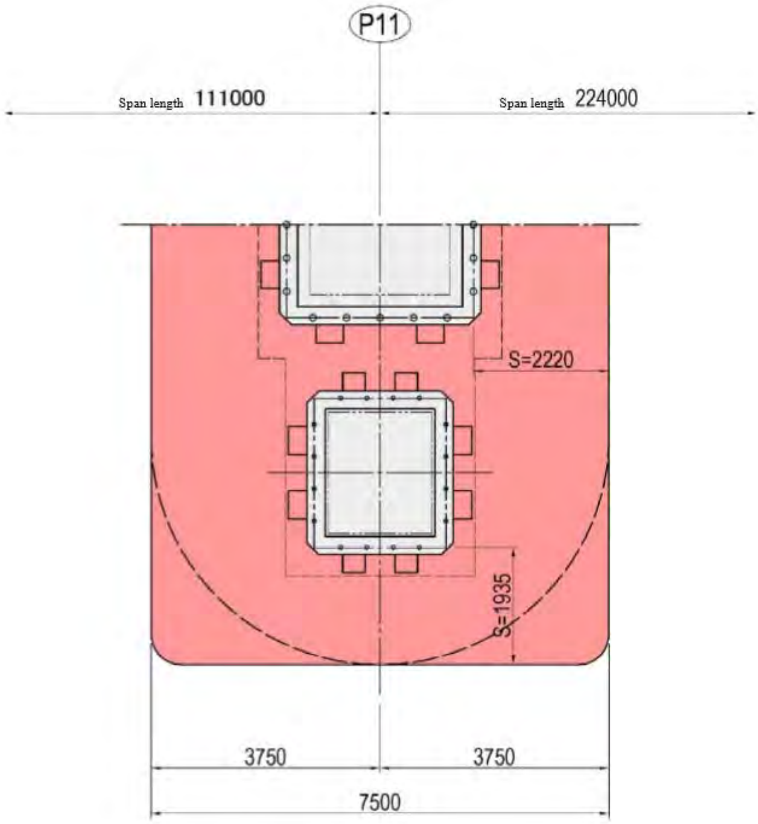


Figure 4.2.147 Bridge Seat Width

Source: JICA Study Team

- Evaluation of edge distance for bearing support

The edge distance of bearing support was set through the following equation:

$S = 0.2 + 0.0051$

$= 0.2 + 0.005 \times 224.000 = 1.320$ m

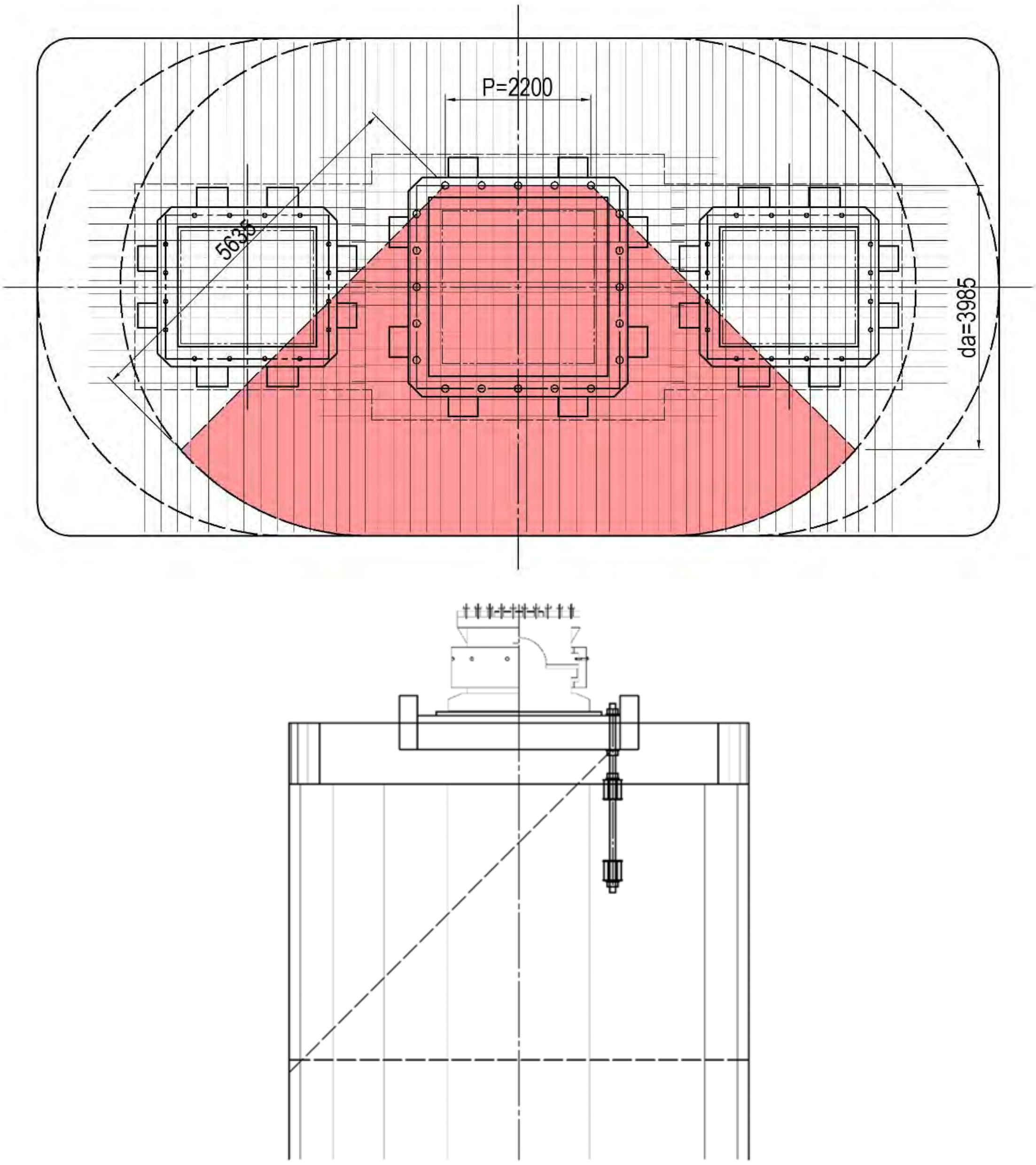
Hence, the edge distance of bearing support can be set as:

$S = 1.320 \text{ m} < 1.935$ ・・・OK

b) Evaluation of Bridge Seat Strength

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

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



Source: JICA Study Team

Figure 4.2.148 Resistance Area of Concrete

- Evaluation of strength

$Pbs = Pc + Ps \quad (Pc \geqq Ps)$

$Pbs = 2.0 \times Pc \quad (Pc < Ps)$

Here,

Pbs : Strength of bridge seat (kN)

Note that the strength is determined under the condition that the strength borne by reinforcements does not exceed that borne by concrete.

Pc : Strength borne by concrete (kN)

$Pc = (\alpha \cdot 0.32 \cdot \sqrt{\sigma ck} \cdot Ac) / 1000.0$

Ps : Strength borne by reinforcement (kN)

$Ps = \Sigma\{\beta \cdot (1 - hi / da) \cdot \sigma sy \cdot Asi\} / 1000.0$

α : Coefficient for determining the strength borne by concrete

σn : Bearing stress at bottom of bearing support against vertical force

σck : Design strength of concrete (kN/mm^2)

Ac : Resistance area of concrete (mm^2)

B : Correction factor associated with the strength borne by reinforcement

Hi : Distance from bridge seat surface of ith reinforcement (m)

Da : Distance from center of anchor bolt in the rear side of bearing support to bridge seat edge

Σsy : Yield point of reinforcement (N/mm^2)

Asi : Cross sectional area of ith reinforcement

Table 4.2.80 Result of Bridge Seat Evaluation

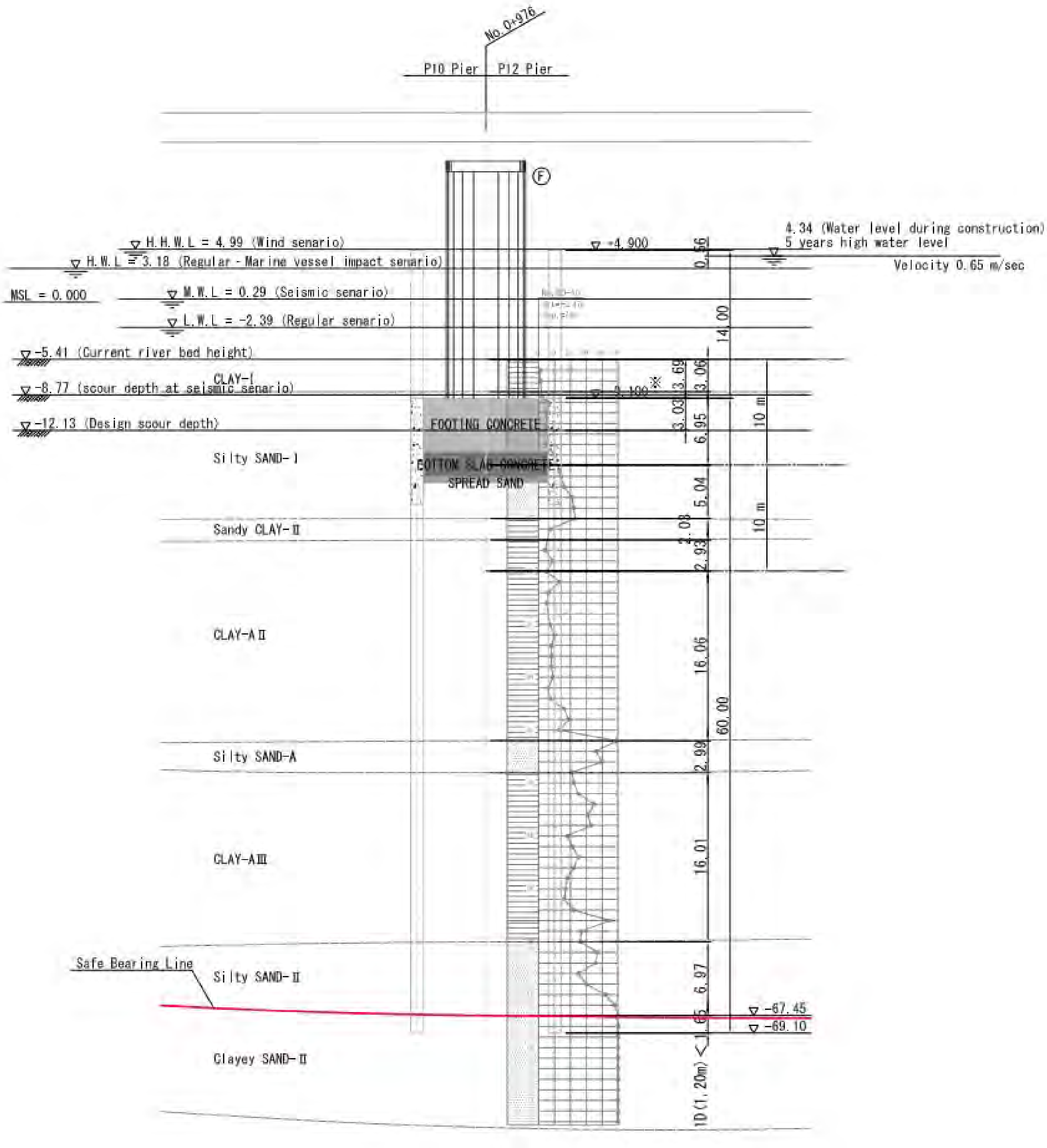
Items	Results
Resistance area of concrete Ac (mm^2)	72756068
Bearing stress σn (N/mm^2)	5.6
Coefficient for determining the strength borne by concrete α	0.477
Strength borne by concrete Pc (kN)	60790.489
Strength borne by reinforcement Ps (kN)	1246.459
Design horizontal seismic force Ph (kN)	25900
Strength of bridge seat Pbs (kN)	62036.948
Judge (Ph≦Pbs)	OK

Source: JICA Study Team

(3) Foundation Design

1) Ground Conditions

The following figure shows the ground condition:

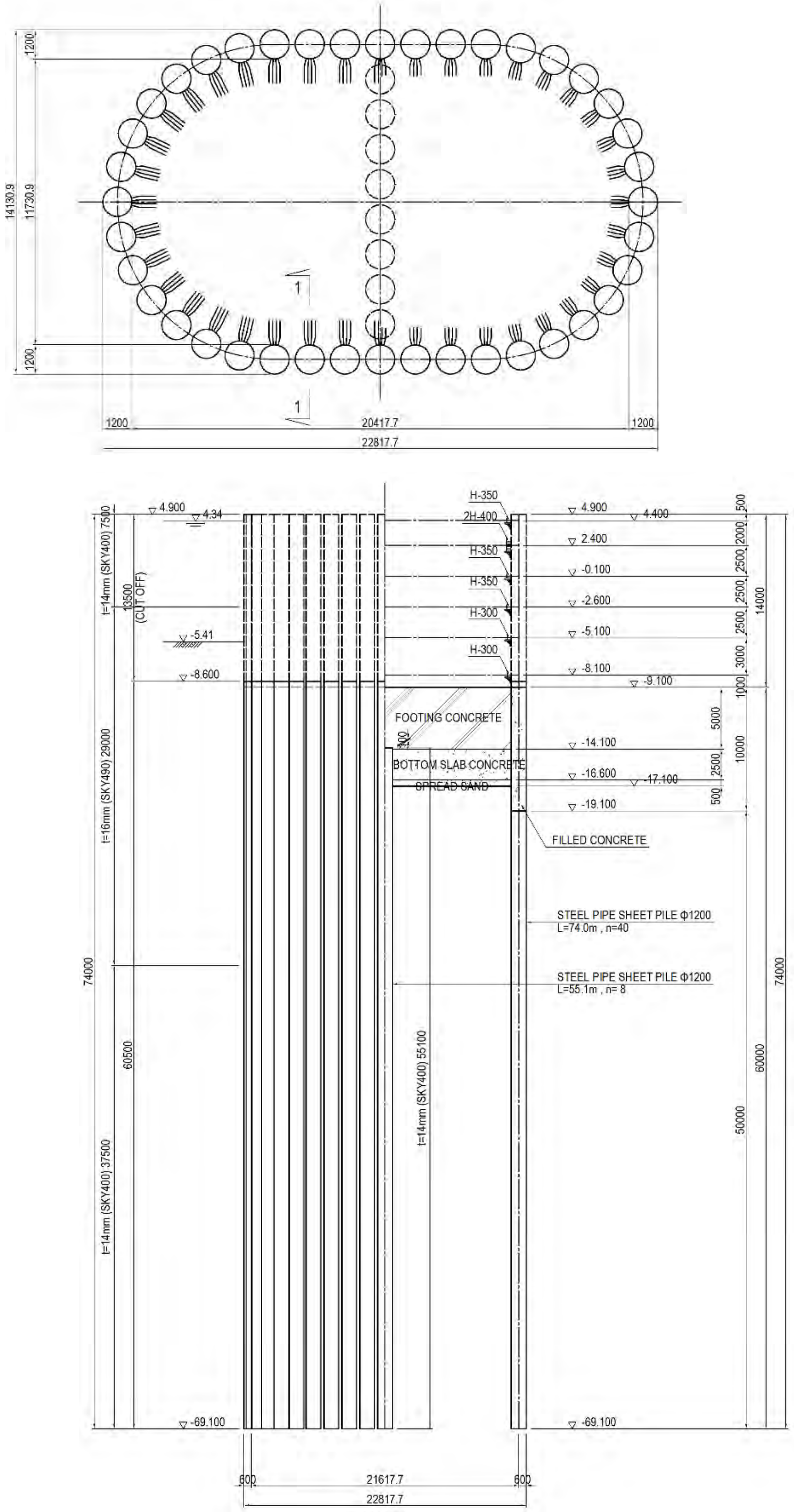


Source: JICA Study Team

Figure 4.2.149 Ground Condition

2) Foundation Shape (Steel Pile Sheet Pile Foundation)

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



Source: JICA Study Team

Figure 4.2.150 Dimensional Drawing of Foundation Shape

[Calculation Result Table]

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

Table 4.2.81 Calculation Results for Foundation

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

Source: JICA Study Team

3) Evaluation Results (Current Riverbed)

The steel pipe sheet pile foundation was designed by satisfying the following conditions:

- Reaction force in longitudinal direction from steel pipe sheet pile ≦ Allowable bearing capacity, Displacement ≦ Allowable displacement
- Stress of steel pile sheet pile ≦ Allowable stress

The evaluation results are shown in the next page.

Table 4.2.82 Evaluation Results for Foundation (Longitudinal Direction) - 1

Items		Unit		HWL[W] at regular senario	LWL[W] at regular senario	HWL[W] temperature flux senario
Acting force		Vo	kN	115432.2	131272.9	115132.2
		Ho	kN	4700	2200	9300
		Mo	kN.m	105750	49500	209250
Level crown of fundation	Displacement	δ1	cm	0.409	0.191	0.809
	Deflection angle	θ1	mrad	-0.319	-0.149	-0.63
Design ground surface	Displacement	δ2	cm	0.409	0.191	0.809
	Deflection angle	θ2	mrad	-0.319	-0.149	-0.63
Max bending moment of opening caisson		Mmax	kN.m	-118087	-55275	-233661
Location of Mmax		Lm	m	-15.1	-15.1	-15.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	66.36	67.11	78.44
		Lm	m	-31.6	-31.6	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	66.15	62.66	84.4
		Lm	m	-15.1	-15.1	-15.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	64.56	66.27	79.06
		Lm	m	-31.6	-31.6	-15.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―	―	―
		Lm	m	―	―	―
	Pile (SKK400)	σmax	N/mm2	―	―	―
	Pile (SKK490)	σmax	N/mm2	―	―	―
Max bending moment of opening caisson at bottom		MB	kN.m	2860	1339	5659
Vertical reaction force	Maximum	Rmax	kN/pile	2421	2742	2430
	Minimum	Rmin	kN/pile	2389	2727	2367
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3535	3535	3535
	Pulling-out bearing capasity	Pa	kN/pile	-1865	-1865	-1865
	Stress (SKY400)	σa	N/mm2	140	140	161
	Stress (SKY490)	σa	N/mm2	185	185	212.75

Source: JICA Study Team

Table 4.2.83 Evaluation Results for Foundation (Longitudinal Direction) - 2

Items		Unit		LWL[W] at temperature senario	Wind senario [W]	Marine vessle impact senario [W]
Acting force		Vo	kN	131372.9	114821.7	115432.2
		Ho	kN	6800	1285.4	9550
		Mo	kN.m	153000	28173.5	169285
Level crown of fundation	Displacement	δ1	cm	0.591	0.11	0.736
	Deflection angle	θ1	mrad	-0.461	-0.086	-0.552
Design ground surface	Displacement	δ2	cm	0.591	0.11	0.736
	Deflection angle	θ2	mrad	-0.461	-0.086	-0.552
Max bending moment of opening caisson		Mmax	kN.m	-170849	-31605	-198003
Location of Mmax		Lm	m	-15.1	-15.1	-15.42
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	79.38	56.95	75.87
		Lm	m	-31.6	-31.6	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	81.08	52.15	78.85
		Lm	m	-15.1	-15.1	-15.42
	Partitioned sheet pile (SKY400)	σmax	N/mm2	77.18	56.47	74.33
		Lm	m	-15.1	-31.6	-15.42
	Partitioned sheet pile (SKY490)	σmax	N/mm2	———	———	———
		Lm	m	———	———	———
	Pile (SKK400)	σmax	N/mm2	———	———	———
	Pile (SKK490)	σmax	N/mm2	———	———	———
Max bending moment of opening caisson at bottom		MB	kN.m	4138	767	4892
Vertical reaction force	Maximum	Rmax	kN/pile	2760	2396	2432
	Minimum	Rmin	kN/pile	2714	2388	2378
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3535	5267	3535
	Pulling-out bearing capasity	Pa	kN/pile	-1865	-3092	-1865
	Stress (SKY400)	σa	N/mm2	161	175	210
	Stress (SKY490)	σa	N/mm2	212.75	231.25	277.5

Source: JICA Study Team

Table 4.2.84 Evaluation Results for Foundation (Longitudinal Direction) - 3

Items		Unit		Seismic senario [W]	Dynamic analysis Smax	Dynamic analysis Mmax
Acting force		Vo	kN	118384.4	116430.6	116099.6
		Ho	kN	33957.8	30839.6	28637.3
		Mo	kN.m	585015.1	564659	590183
Level crown of fundation	Displacement	δ1	cm	2.68	2.435	2.386
	Deflection angle	θ1	mrad	-1.986	-1.847	-1.848
Design ground surface	Displacement	δ2	cm	2.68	2.435	2.386
	Deflection angle	θ2	mrad	-1.986	-1.847	-1.848
Max bending moment of opening caisson		Mmax	kN.m	-710854	-673577	-684630
Location of Mmax		Lm	m	-17.1	-16.6	-16.3
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	135.89	128.66	127.77
		Lm	m	-31.6	-31.6	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	161.6	154.87	156.5
		Lm	m	-17.1	-16.6	-16.3
	Partitioned sheet pile (SKY400)	σmax	N/mm2	145.36	139.49	140.85
		Lm	m	-17.1	-16.6	-16.3
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――	―――	―――
		Lm	m	―――	―――	―――
	Pile (SKK400)	σmax	N/mm2	―――	―――	―――
	Pile (SKK490)	σmax	N/mm2	―――	―――	―――
Max bending moment of opening caisson at bottom		MB	kN.m	25513	22998	22801
Vertical reaction force	Maximum	Rmax	kN/pile	2607	2552	2544
	Minimum	Rmin	kN/pile	2326	2299	2293
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	5267	5267	5267
	Pulling-out bearing capasity	Pa	kN/pile	-3092	-3092	-3092
	Stress (SKY400)	σa	N/mm2	210	210	210
	Stress (SKY490)	σa	N/mm2	277.5	277.5	277.5

Source: JICA Study Team

Table 4.2.85 Evaluation Results for Foundation (Transverse Direction) - 1

Items		Unit		HWL[W] at regular senario	LWL[W] at regular senario	Wind senario [W]
Acting force		Vo	kN	115432.2	131272.9	114821.7
		Ho	kN	100	100	2253.6
		Mo	kN.m	34250	34250	83880
Level crown of fundation	Displacement	δ1	cm	0.068	0.068	0.259
	Deflection angle	θ1	mrad	-0.051	-0.051	-0.161
Design ground surface	Displacement	δ2	cm	0.068	0.068	0.259
	Deflection angle	θ2	mrad	-0.051	-0.051	-0.161
Max bending moment of opening caisson		Mmax	kN.m	-34288	-34288	-90274
Location of Mmax		Lm	m	-10.3	-10.3	-15.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	57.24	64.63	64.46
		Lm	m	-31.6	-31.6	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	52.43	58.93	60.42
		Lm	m	-10.3	-10.3	-15.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	55.14	62.53	57.67
		Lm	m	-31.6	-31.6	-31.6
	Partitioned sheet pile (SKY490)	σmax	N/mm2	—	—	—
		Lm	m	—	—	—
	Pile (SKK400)	σmax	N/mm2	—	—	—
	Pile (SKK490)	σmax	N/mm2	—	—	—
Max bending moment of opening caisson at bottom		MB	kN.m	1143	1143	2387
Vertical reaction force	Maximum	Rmax	kN/pile	2410	2740	2404
	Minimum	Rmin	kN/pile	2399	2729	2380
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3535	3535	5267
	Pulling-out bearing capasity	Pa	kN/pile	-1865	-1865	-3092
	Stress (SKY400)	σa	N/mm2	140	140	175
	Stress (SKY490)	σa	N/mm2	185	185	231.25

Source: JICA Study Team

Table 4.2.86 Evaluation Results for Foundation (Transverse Direction) - 2

<table>
<tr><th colspan="2">Items</th><th colspan="2">Unit</th><th>Marine vessle impact senario [W]</th><th>Seismic senario [W]</th><th>Dynamic analysis Smax</th></tr>
<tr><td colspan="2" rowspan="3">Acting force</td><td>Vo</td><td>kN</td><td>115432.2</td><td>118484.4</td><td>115142</td></tr>
<tr><td>Ho</td><td>kN</td><td>9800</td><td>27972.5</td><td>-20268.8</td></tr>
<tr><td>Mo</td><td>kN.m</td><td>161320</td><td>577239.2</td><td>-439530</td></tr>
<tr><td rowspan="2">Level crown of fundation</td><td>Displacement</td><td>δ1</td><td>cm</td><td>0.753</td><td>2.262</td><td>-1.471</td></tr>
<tr><td>Deflection angle</td><td>θ1</td><td>mrad</td><td>-0.411</td><td>-1.325</td><td>0.923</td></tr>
<tr><td rowspan="2">Design ground surface</td><td>Displacement</td><td>δ2</td><td>cm</td><td>0.753</td><td>2.262</td><td>-1.471</td></tr>
<tr><td>Deflection angle</td><td>θ2</td><td>mrad</td><td>-0.411</td><td>-1.325</td><td>0.923</td></tr>
<tr><td colspan="2">Max bending moment of opening caisson</td><td>Mmax</td><td>kN.m</td><td>-205259</td><td>-708517</td><td>525808</td></tr>
<tr><td colspan="2">Location of Mmax</td><td>Lm</td><td>m</td><td>-19.1</td><td>-18.7</td><td>-17.5</td></tr>
<tr><td rowspan="10">Stress</td><td rowspan="2">Outer peripheral sheet pile (SKY400)</td><td>σmax</td><td>N/mm2</td><td>81.96</td><td>148.25</td><td>118.98</td></tr>
<tr><td>Lm</td><td>m</td><td>-31.6</td><td>-31.6</td><td>-31.6</td></tr>
<tr><td rowspan="2">Outer peripheral sheet pile (SKY490)</td><td>σmax</td><td>N/mm2</td><td>77.61</td><td>153</td><td>124.72</td></tr>
<tr><td>Lm</td><td>m</td><td>-19.1</td><td>-18.7</td><td>-17.5</td></tr>
<tr><td rowspan="2">Partitioned sheet pile (SKY400)</td><td>σmax</td><td>N/mm2</td><td>64.41</td><td>90.16</td><td>78.21</td></tr>
<tr><td>Lm</td><td>m</td><td>-31.6</td><td>-31.6</td><td>-31.6</td></tr>
<tr><td rowspan="2">Partitioned sheet pile (SKY490)</td><td>σmax</td><td>N/mm2</td><td>———</td><td>———</td><td>———</td></tr>
<tr><td>Lm</td><td>m</td><td>———</td><td>———</td><td>———</td></tr>
<tr><td>Pile (SKK400)</td><td>σmax</td><td>N/mm2</td><td>———</td><td>———</td><td>———</td></tr>
<tr><td>Pile (SKK490)</td><td>σmax</td><td>N/mm2</td><td>———</td><td>———</td><td>———</td></tr>
<tr><td colspan="2">Max bending moment of opening caisson at bottom</td><td>MB</td><td>kN.m</td><td>3465</td><td>31306</td><td>-24613</td></tr>
<tr><td rowspan="2">Vertical reaction force</td><td>Maximum</td><td>Rmax</td><td>kN/pile</td><td>2422</td><td>2623</td><td>2520</td></tr>
<tr><td>Minimum</td><td>Rmin</td><td>kN/pile</td><td>2388</td><td>2314</td><td>2277</td></tr>
<tr><td rowspan="5">Allowable value</td><td>Displacement</td><td>δa</td><td>cm</td><td>5</td><td>5</td><td>5</td></tr>
<tr><td>Pushing bearing capacity</td><td>Ra</td><td>kN/pile</td><td>3535</td><td>5267</td><td>5267</td></tr>
<tr><td>Pulling-out bearing capasity</td><td>Pa</td><td>kN/pile</td><td>-1865</td><td>-3092</td><td>-3092</td></tr>
<tr><td>Stress (SKY400)</td><td>σa</td><td>N/mm2</td><td>210</td><td>210</td><td>210</td></tr>
<tr><td>Stress (SKY490)</td><td>σa</td><td>N/mm2</td><td>277.5</td><td>277.5</td><td>277.5</td></tr>
</table>

Source: JICA Study Team

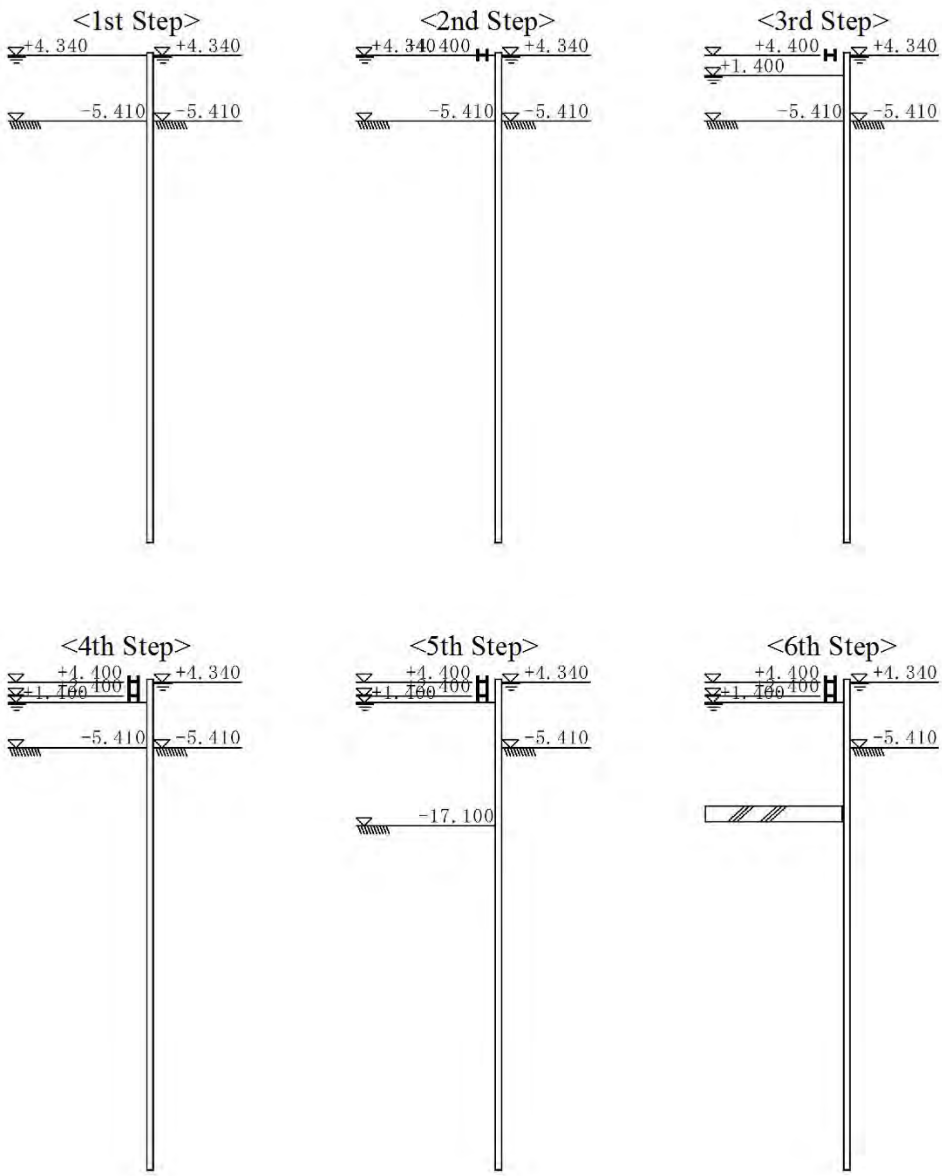
Table 4.2.87 Evaluation Results for Foundation (Transverse Direction) - 3

Items		Unit		Dynamic analysis Mmax
Acting force		Vo	kN	115142
		Ho	kN	-20217.6
		Mo	kN.m	-440242
Level crown of fundation	Displacement	δ1	cm	-1.47
	Deflection angle	θ1	mrad	0.923
Design ground surface	Displacement	δ2	cm	-1.47
	Deflection angle	θ2	mrad	0.923
Max bending moment of opening caisson		Mmax	kN.m	526131
Location of Mmax		Lm	m	-17.5
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	118.97
		Lm	m	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	124.76
		Lm	m	-17.5
	Partitioned sheet pile (SKY400)	σmax	N/mm2	78.21
		Lm	m	-31.6
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――
		Lm	m	―――
	Pile (SKK400)	σmax	N/mm2	―――
	Pile (SKK490)	σmax	N/mm2	―――
Max bending moment of opening caisson at bottom		MB	kN.m	-24638
Vertical reaction force	Maximum	Rmax	kN/pile	2520
	Minimum	Rmin	kN/pile	2277
Allowable value	Displacement	δa	cm	5
	Pushing bearing capacity	Ra	kN/pile	5267
	Pulling-out bearing capasity	Pa	kN/pile	-3092
	Stress (SKY400)	σa	N/mm2	210
	Stress (SKY490)	σa	N/mm2	277.5

Source: JICA Study Team

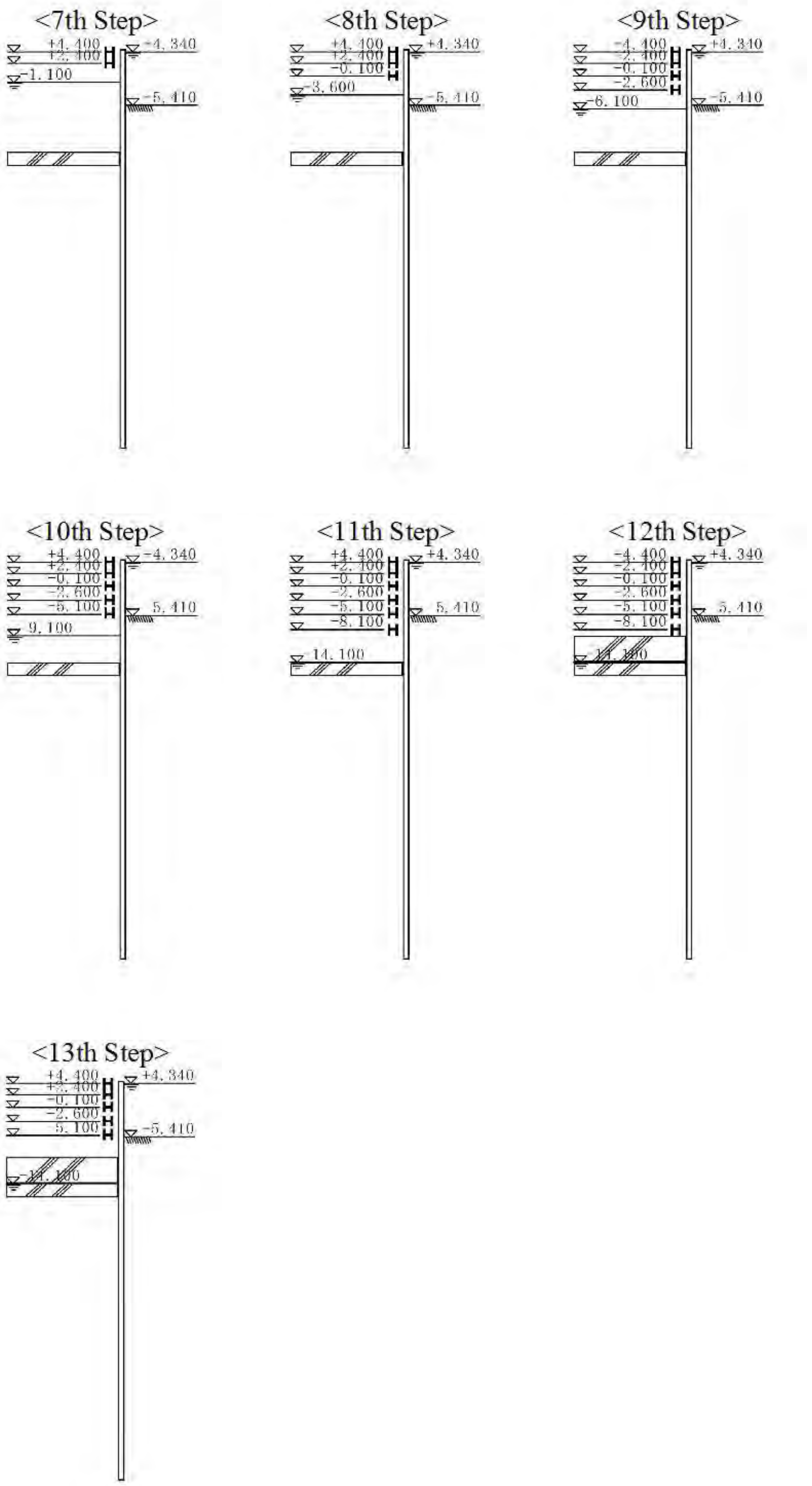
4) Temporary Coffering Calculation (Current Riverbed)

During the temporary coffering, as the stress the steel pipe sheet pile is subjected to is affected by the construction sequence, the stress is calculated for each construction stage.



Source: JICA Study Team

Figure 4.2.151 Construction Stage (1st – 6th Stage)



Source: JICA Study Team

Figure 4.2.152 Construction Stage (7^{th} – 13^{th} Stage)

Table 4.2.88 Temporary Coffering Calculation Results (Longitudinal Direction)

・Longitudinal Direction

Item		Unit	1st Step	2nd Step	3rd Step	4th Step	5th Step	6th Step	7th Step
Maximum Displacement		cm	0.154	0.154	1.575	1.575	7.961	5.566	9.291
Cofferdam Section	SKY400	N/mm2	0.51	0.51	58.59	58.59	102.75	88.69	159.11
	SKY490	N/mm2	1.13	1.13	51.41	51.41	133.59	105.15	182.58
Well Section	SKY400	N/mm2	0.04	0.04	0.66	0.66	31.30	12.71	22.32
	SKY490	N/mm2	1.13	1.13	15.50	15.50	134.79	105.24	182.29
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	210.00
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	280.00
Item		単位	8th Step	9th Step	10th Step	11th Step	12th Step	13th Step	
Maximum Displacement		cm	9.841	10.108	10.238	10.242	10.238	10.238	
Cofferdam Section	SKY400	N/mm2	168.08	154.52	147.96	147.85	147.70	147.69	
	SKY490	N/mm2	210.72	229.42	240.12	239.38	243.74	243.77	
Well Section	SKY400	N/mm2	20.78	16.08	12.06	15.15	15.14	15.14	
	SKY490	N/mm2	207.99	226.52	238.78	238.14	243.03	242.98	
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	

Source: JICA Study Team

Table 4.2.89 Temporary Coffering Calculation Results (Transverse Direction)

・Traverse Direction

Item		Unit	1st Step	2nd Step	3rd Step	4th Step	5th Step	6th Step	7th Step
Maximum Displacement		cm	0.154	0.154	1.591	1.591	7.990	5.590	9.335
Cofferdam Section	SKY400	N/mm2	0.51	0.51	58.38	58.38	102.69	88.61	159.10
	SKY490	N/mm2	1.13	1.13	51.22	51.22	133.47	104.99	182.57
Well Section	SKY400	N/mm2	0.04	0.04	0.66	0.66	31.33	12.71	22.37
	SKY490	N/mm2	1.13	1.13	15.64	15.64	134.66	105.07	182.28
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	210.00
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	280.00
Item		単位	8th Step	9th Step	10th Step	11th Step	12th Step	13th Step	
Maximum Displacement		cm	9.973	10.290	10.456	10.476	10.472	10.473	
Cofferdam Section	SKY400	N/mm2	171.03	158.35	150.91	150.19	150.18	150.14	
	SKY490	N/mm2	212.33	231.89	243.13	241.07	245.37	245.51	
Well Section	SKY400	N/mm2	20.98	16.47	12.67	15.08	15.08	15.08	
	SKY490	N/mm2	209.41	228.85	241.79	240.19	245.02	244.66	
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	

Source: JICA Study Team

5) Total Stress Calculation (Current Riverbed)

The stress the steel pipe sheet pile is subjected to is evaluated as the total of the leftover stress from the construction stage and the design external force after completion.

Leftover stress + Design external force after completion ≦ Allowable stress of steel pipe sheet pile

Table 4.2.90 Temporary Coffering Calculation Results (Longitudinal Direction)

1) Material：SKY400

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-31.6	66.36	7.03	73.39	140
2	Regular scenario LWL[W]	-47.9	62.29	15.15	77.43	140
3	Temperature flux scenario HWL[W]	-31.6	78.44	7.03	85.47	161
4	Temperature flux scenario LWL[W]	-31.6	79.38	7.03	86.41	161
5	Wind scenario [W]	-47.9	54.18	15.15	69.33	175
6	Marine vessel impact scenario [W]	-31.6	75.87	7.03	82.9	210
7	Seismic scenario [W]	-31.6	135.89	7.03	142.92	210
8	Seismic scenario[Smax]	-31.6	128.66	7.03	135.69	210
9	Seismic scenario [Mmax]	-31.6	127.77	7.03	134.8	210

2) Material：SKY490

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-21.5	64.39	88.43	152.82	185
2	Regular scenario LWL[W]	-21.5	61.84	88.43	150.27	185
3	Temperature flux scenario HWL[W]	-21.5	80.92	88.43	169.35	212.75
4	Temperature flux scenario LWL[W]	-21.5	78.54	88.43	166.97	212.75
5	Wind scenario [W]	-21.5	51.69	88.43	140.12	231.25
6	Marine vessel impact scenario [W]	-21.5	76.58	88.43	165.01	277.5
7	Seismic scenario [W]	-21.5	155.64	88.43	244.07	277.5
8	Seismic scenario[Smax]	-21.5	148.36	88.43	236.79	277.5
9	Seismic scenario [Mmax]	-21.5	148.81	88.43	237.24	277.5

Source: JICA Study Team

Table 4.2.91 Temporary Coffering Calculation Results (Transverse Direction)

1) Material：SKY400

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-47.9	54.88	15.08	69.96	140
2	Regular scenario LWL[W]	-47.9	62.27	15.08	77.36	140
3	Wind scenario [W]	-47.9	57.65	15.08	72.74	175
4	Marine vessel impact scenario [W]	-31.6	81.96	8.18	90.15	210
5	Seismic scenario [W]	-31.6	148.25	8.18	156.43	210
6	Seismic scenario[Smax]	-31.6	118.98	8.18	127.17	210
7	Seismic scenario [Mmax]	-31.6	118.97	8.18	127.16	210

2) Material：SKY490

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-21.5	51.63	90.66	142.3	185
2	Regular scenario LWL[W]	-21.5	58.13	90.66	148.8	185
3	Wind scenario [W]	-21.5	59.68	90.66	150.35	231.25
4	Marine vessel impact scenario [W]	-21.5	77.32	90.66	167.99	277.5
5	Seismic scenario [W]	-21.5	151.45	90.66	242.11	277.5
6	Seismic scenario[Smax]	-21.5	122.74	90.66	213.4	277.5

Source: JICA Study Team

6) Evaluation Results (Considering Scour)

Evaluation of steel pipe sheet pile foundation was also done considering the effects of scour.

Table 4.2.92 Evaluation Results for Foundation (Longitudinal Direction) - 1

Items		Unit		HWL[W] at regular senario	LWL[W] at regular senario	HWL[W] temperature flux senario
Acting force		Vo	kN	109177.4	125018.1	108877.4
		Ho	kN	4700	2200	9300
		Mo	kN.m	105750	49500	209250
Level crown of fundation	Displacement	δ1	cm	0.551	0.258	1.09
	Deflection angle	θ1	mrad	-0.386	-0.181	-0.763
Design ground surface	Displacement	δ2	cm	0.44	0.206	0.87
	Deflection angle	θ2	mrad	-0.348	-0.163	-0.689
Max bending moment of opening caisson		Mmax	kN.m	-131370	-61492	-259945
Location of Mmax		Lm	m	-17.5	-17.5	-17.5
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	66.8	65.76	82.16
		Lm	m	-31.6	-31.6	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	65.69	61.09	86.01
		Lm	m	-17.5	-17.5	-17.5
	Partitioned sheet pile (SKY400)	σmax	N/mm2	64.51	64.69	80.07
		Lm	m	-31.6	-31.6	-17.5
	Partitioned sheet pile (SKY490)	σmax	N/mm2	――	――	――
		Lm	m	――	――	――
	Pile (SKK400)	σmax	N/mm2	――	――	――
	Pile (SKK490)	σmax	N/mm2	――	――	――
Max bending moment of opening caisson at bottom		MB	kN.m	3337	1562	6603
Vertical reaction force	Maximum	Rmax	kN/pile	2293	2613	2305
	Minimum	Rmin	kN/pile	2256	2596	2232
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3501	3501	3501
	Pulling-out bearing capasity	Pa	kN/pile	-1848	-1848	-1848
	Stress (SKY400)	σa	N/mm2	140	140	161
	Stress (SKY490)	σa	N/mm2	185	185	212.75

Source: JICA Study Team

Table 4.2.93 Evaluation Results for Foundation (Longitudinal Direction) - 2

Items		Unit		LWL[W] at temperature senario	Wind senario [W]	Marine vessle impact senario [W]
Acting force		Vo	kN	125118.1	108566.8	109177.4
		Ho	kN	6800	1285.4	9550
		Mo	kN.m	153000	28173.5	169285
Level crown of fundation	Displacement	δ1	cm	0.797	0.149	0.994
	Deflection angle	θ1	mrad	-0.558	-0.104	-0.673
Design ground surface	Displacement	δ2	cm	0.636	0.119	0.799
	Deflection angle	θ2	mrad	-0.504	-0.094	-0.613
Max bending moment of opening caisson		Mmax	kN.m	-190067	-35227	-224282
Location of Mmax		Lm	m	-17.5	-17.5	-18.7
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	81.31	54.94	79.03
		Lm	m	-31.6	-31.6	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	81.57	50.16	80.47
		Lm	m	-17.5	-17.5	-18.7
	Partitioned sheet pile (SKY400)	σmax	N/mm2	78	54.32	75.34
		Lm	m	-31.6	-31.6	-18.7
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――	―――	―――
		Lm	m	―――	―――	―――
	Pile (SKK400)	σmax	N/mm2	―――	―――	―――
	Pile (SKK490)	σmax	N/mm2	―――	―――	―――
Max bending moment of opening caisson at bottom		MB	kN.m	4828	896	5756
Vertical reaction force	Maximum	Rmax	kN/pile	2633	2267	2306
	Minimum	Rmin	kN/pile	2580	2257	2243
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3501	5267	3501
	Pulling-out bearing capasity	Pa	kN/pile	-1848	-3092	-1848
	Stress (SKY400)	σa	N/mm2	161	175	210
	Stress (SKY490)	σa	N/mm2	212.75	231.25	277.5

Source: JICA Study Team

Table 4.2.94 Evaluation Results for Foundation (Longitudinal Direction) - 3

Items		Unit		Seismic senario [W]	Dynamic analysis Smax	Dynamic analysis Mmax
Acting force		Vo	kN	112688.9	110735.1	110404.1
		Ho	kN	33957.8	30839.6	28637.3
		Mo	kN.m	585015.1	564659	590183
Level crown of fundation	Displacement	δ1	cm	2.68	2.435	2.386
	Deflection angle	θ1	mrad	-1.986	-1.847	-1.848
Design ground surface	Displacement	δ2	cm	2.68	2.435	2.386
	Deflection angle	θ2	mrad	-1.986	-1.847	-1.848
Max bending moment of opening caisson		Mmax	kN.m	-710854	-673577	-684630
Location of Mmax		Lm	m	-17.1	-16.6	-16.3
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	133.23	126	125.12
		Lm	m	-31.6	-31.6	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	159.27	152.54	154.16
		Lm	m	-17.1	-16.6	-16.3
	Partitioned sheet pile (SKY400)	σmax	N/mm2	143.03	137.15	138.52
		Lm	m	-17.1	-16.6	-16.3
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――	―――	―――
		Lm	m	―――	―――	―――
	Pile (SKK400)	σmax	N/mm2	―――	―――	―――
	Pile (SKK490)	σmax	N/mm2	―――	―――	―――
Max bending moment of opening caisson at bottom		MB	kN.m	25513	22998	22801
Vertical reaction force	Maximum	Rmax	kN/pile	2488	2434	2426
	Minimum	Rmin	kN/pile	2207	2180	2174
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	5267	5267	5267
	Pulling-out bearing capasity	Pa	kN/pile	-3092	-3092	-3092
	Stress (SKY400)	σa	N/mm2	210	210	210
	Stress (SKY490)	σa	N/mm2	277.5	277.5	277.5

Source: JICA Study Team

Table 4.2.95 Evaluation Results for Foundation (Transverse Direction) - 1

Items		Unit		HWL[W] at regular senario	LWL[W] at regular senario	Wind senario [W]
Acting force		Vo	kN	109177.4	125018.1	108566.8
		Ho	kN	100	100	2253.6
		Mo	kN.m	34250	34250	83880
Level crown of fundation	Displacement	δ1	cm	0.085	0.085	0.327
	Deflection angle	θ1	mrad	-0.057	-0.057	-0.187
Design ground surface	Displacement	δ2	cm	0.069	0.069	0.272
	Deflection angle	θ2	mrad	-0.051	-0.051	-0.171
Max bending moment of opening caisson		Mmax	kN.m	-34591	-34591	-96685
Location of Mmax		Lm	m	-12.7	-12.7	-17.5
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	54.79	62.18	63.38
		Lm	m	-31.6	-31.6	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	49.9	56.4	58.8
		Lm	m	-12.7	-12.7	-17.5
	Partitioned sheet pile (SKY400)	σmax	N/mm2	52.39	59.79	55.44
		Lm	m	-31.6	-31.6	-31.6
	Partitioned sheet pile (SKY490)	σmax	N/mm2	———	———	———
		Lm	m	———	———	———
	Pile (SKK400)	σmax	N/mm2	———	———	———
	Pile (SKK490)	σmax	N/mm2	———	———	———
Max bending moment of opening caisson at bottom		MB	kN.m	1043	1043	2009
Vertical reaction force	Maximum	Rmax	kN/pile	2280	2610	2272
	Minimum	Rmin	kN/pile	2269	2599	2252
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3501	3501	5267
	Pulling-out bearing capasity	Pa	kN/pile	-1848	-1848	-3092
	Stress (SKY400)	σa	N/mm2	140	140	175
	Stress (SKY490)	σa	N/mm2	185	185	231.25

Source: JICA Study Team

Table 4.2.96 Evaluation Results for Foundation (Transverse Direction) - 2

Items		Unit		Marine vessle impact senario [W]	Seismic senario [W]	Dynamic analysis Smax
Acting force		Vo	kN	109177.4	112788.9	109446.5
		Ho	kN	9800	27972.5	-20268.8
		Mo	kN.m	161320	577239.2	-439530
Level crown of fundation	Displacement	δ1	cm	0.95	2.262	-1.471
	Deflection angle	θ1	mrad	-0.486	-1.325	0.923
Design ground surface	Displacement	δ2	cm	0.808	2.262	-1.471
	Deflection angle	θ2	mrad	-0.455	-1.325	0.923
Max bending moment of opening caisson		Mmax	kN.m	-231109	-708517	525808
Location of Mmax		Lm	m	-21.1	-18.7	-17.5
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	84.43	145.59	116.33
		Lm	m	-31.6	-31.6	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	78.85	150.67	122.38
		Lm	m	-21.1	-18.7	-17.5
	Partitioned sheet pile (SKY400)	σmax	N/mm2	63.51	87.5	75.55
		Lm	m	-31.6	-31.6	-31.6
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――	―――	―――
		Lm	m	―――	―――	―――
	Pile (SKK400)	σmax	N/mm2	―――	―――	―――
	Pile (SKK490)	σmax	N/mm2	―――	―――	―――
Max bending moment of opening caisson at bottom		MB	kN.m	2380	31306	-24613
Vertical reaction force	Maximum	Rmax	kN/pile	2286	2504	2402
	Minimum	Rmin	kN/pile	2263	2195	2159
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3501	5267	5267
	Pulling-out bearing capasity	Pa	kN/pile	-1848	-3092	-3092
	Stress (SKY400)	σa	N/mm2	210	210	210
	Stress (SKY490)	σa	N/mm2	277.5	277.5	277.5

Source: JICA Study Team

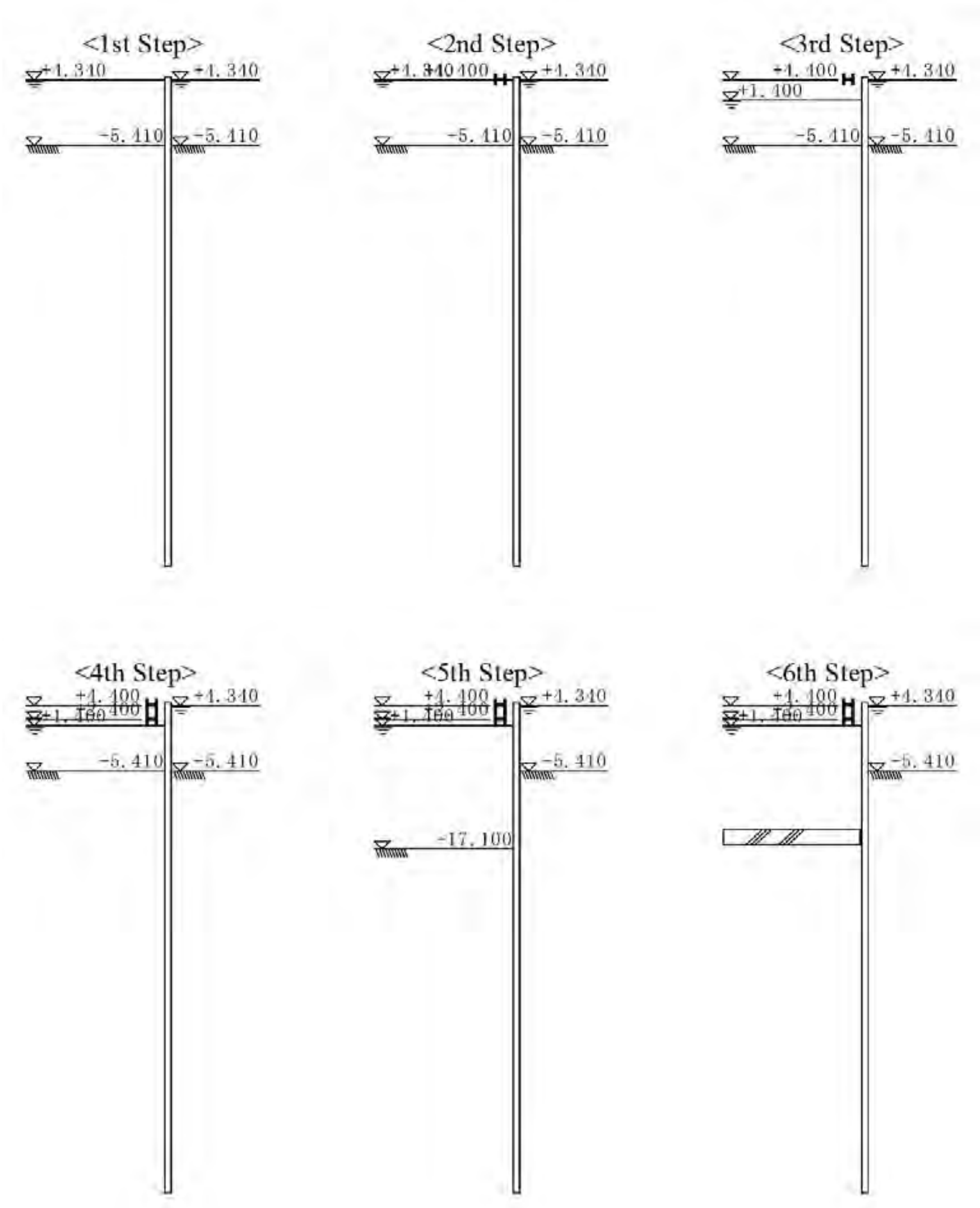
Table 4.2.97 Evaluation Results for Foundation (Transverse Direction) - 3

Items		Unit		Dynamic analysis Mmax
Acting force		Vo	kN	109177.4
		Ho	kN	9800
		Mo	kN.m	161320
Level crown of fundation	Displacement	δ1	cm	0.95
	Deflection angle	θ1	mrad	-0.486
Design ground surface	Displacement	δ2	cm	0.808
	Deflection angle	θ2	mrad	-0.455
Max bending moment of opening caisson		Mmax	kN.m	-231109
Location of Mmax		Lm	m	-21.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	84.43
		Lm	m	-31.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	78.85
		Lm	m	-21.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	63.51
		Lm	m	-31.6
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――
		Lm	m	―――
	Pile (SKK400)	σmax	N/mm2	―――
	Pile (SKK490)	σmax	N/mm2	―――
Max bending moment of opening caisson at bottom		MB	kN.m	2380
Vertical reaction force	Maximum	Rmax	kN/pile	2286
	Minimum	Rmin	kN/pile	2263
Allowable value	Displacement	δa	cm	5
	Pushing bearing capacity	Ra	kN/pile	3501
	Pulling-out bearing capasity	Pa	kN/pile	-1848
	Stress (SKY400)	σa	N/mm2	210
	Stress (SKY490)	σa	N/mm2	277.5

Source: JICA Study Team

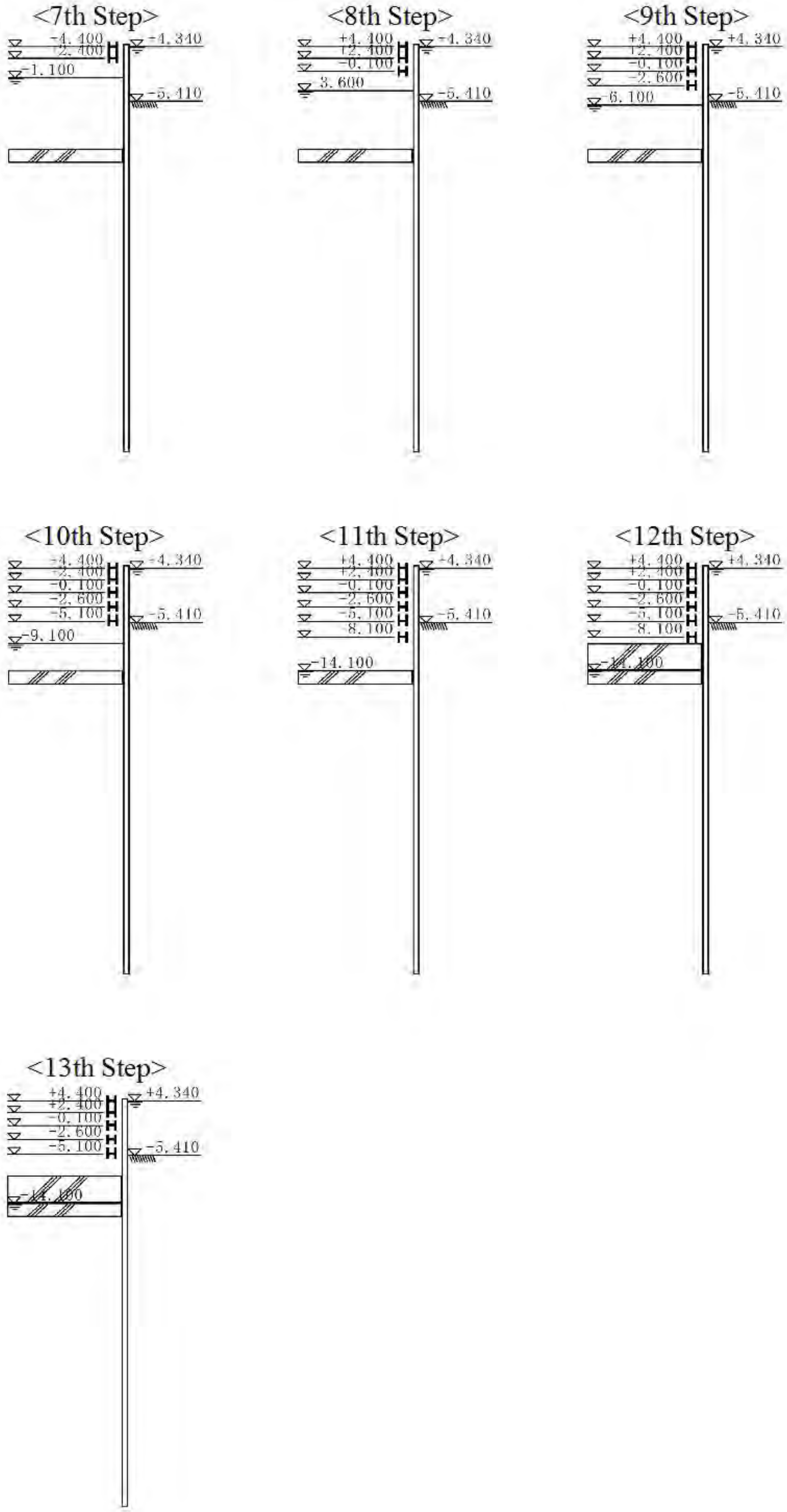
7) Temporary Coffering Calculation (Considering Scour)

During the temporary coffering, as the stress the steel pipe sheet pile is subjected to is affected by the construction sequence, the stress is calculated for each construction stage.



Source: JICA Study Team

Figure 4.2.153 Construction Stage (1st – 6th Stage)



Source: JICA Study Team

Figure 4.2.154 Construction Stage (7th – 13th Stage)

Table 4.2.98 Temporary Coffering Calculation Results (Longitudinal Direction)

・Longitudinal Direction

Item		Unit	1st Step	2nd Step	3rd Step	4th Step	5th Step	6th Step	7th Step
Maximum Displacement		cm	0.154	0.154	1.575	1.575	7.961	5.566	9.291
Cofferdam Section	SKY400	N/mm2	0.51	0.51	58.59	58.59	102.75	88.69	159.11
	SKY490	N/mm2	1.13	1.13	51.41	51.41	133.59	105.15	182.58
Well Section	SKY400	N/mm2	0.04	0.04	0.66	0.66	31.30	12.71	22.32
	SKY490	N/mm2	1.13	1.13	15.50	15.50	134.79	105.24	182.29
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	210.00
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	280.00
Item		単位	8th Step	9th Step	10th Step	11th Step	12th Step	13th Step	
Maximum Displacement		cm	9.841	10.108	10.238	10.242	10.238	10.238	
Cofferdam Section	SKY400	N/mm2	168.08	154.52	147.96	147.85	147.70	147.69	
	SKY490	N/mm2	210.72	229.42	240.12	239.38	243.74	243.77	
Well Section	SKY400	N/mm2	20.78	16.08	12.06	15.15	15.14	15.14	
	SKY490	N/mm2	207.99	226.52	238.78	238.14	243.03	242.98	
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	

Source: JICA Study Team

Table 4.2.99 Temporary Coffering Calculation Results (Transverse Direction)

・Traverse Direction

Item		Unit	1st Step	2nd Step	3rd Step	4th Step	5th Step	6th Step	7th Step
Maximum Displacement		cm	0.154	0.154	1.591	1.591	7.990	5.590	9.335
Cofferdam Section	SKY400	N/mm2	0.51	0.51	58.38	58.38	102.69	88.61	159.10
	SKY490	N/mm2	1.13	1.13	51.22	51.22	133.47	104.99	182.57
Well Section	SKY400	N/mm2	0.04	0.04	0.66	0.66	31.33	12.71	22.37
	SKY490	N/mm2	1.13	1.13	15.64	15.64	134.66	105.07	182.28
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	210.00
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	280.00
Item		単位	8th Step	9th Step	10th Step	11th Step	12th Step	13th Step	
Maximum Displacement		cm	9.973	10.290	10.456	10.476	10.472	10.473	
Cofferdam Section	SKY400	N/mm2	171.03	158.35	150.91	150.19	150.18	150.14	
	SKY490	N/mm2	212.33	231.89	243.13	241.07	245.37	245.51	
Well Section	SKY400	N/mm2	20.98	16.47	12.67	15.08	15.08	15.08	
	SKY490	N/mm2	209.41	228.85	241.79	240.19	245.02	244.66	
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	

Source: JICA Study Team

8) Total Stress Calculation (Considering Scour)

The stress the steel pipe sheet pile is subjected to is evaluated as the total of the leftover stress from the construction stage and the design external force after completion.

Leftover stress + Design external force after completion ≦ Allowable stress of steel pipe sheet pile

Table 4.2.100 Temporary Coffering Calculation Results (Longitudinal Direction)

1) Material：SKY400

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-31.6	66.8	7.03	73.83	140
2	Regular scenario LWL[W]	-47.9	60.01	15.15	75.15	140
3	Temperature flux scenario HWL[W]	-31.6	82.16	7.03	89.19	161
4	Temperature flux scenario LWL[W]	-31.6	81.31	7.03	88.34	161
5	Wind scenario [W]	-47.9	51.63	15.15	66.78	175
6	Marine vessel impact scenario [W]	-31.6	79.03	7.03	86.06	210
7	Seismic scenario [W]	-31.6	133.23	7.03	140.26	210
8	Seismic scenario[Smax]	-31.6	126	7.03	133.03	210
9	Seismic scenario [Mmax]	-31.6	125.12	7.03	132.15	210

2) Material：SKY490

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-21.5	64.85	88.43	153.28	185
2	Regular scenario LWL[W]	-21.5	60.69	88.43	149.12	185
3	Temperature flux scenario HWL[W]	-21.5	84.35	88.43	172.78	212.75
4	Temperature flux scenario LWL[W]	-21.5	80.35	88.43	168.78	212.75
5	Wind scenario [W]	-21.5	49.94	88.43	138.37	231.25
6	Marine vessel impact scenario [W]	-21.5	79.5	88.43	167.93	277.5
7	Seismic scenario [W]	-21.5	153.3	88.43	241.73	277.5
8	Seismic scenario[Smax]	-21.5	146.02	88.43	234.45	277.5
9	Seismic scenario [Mmax]	-21.5	146.47	88.43	234.9	277.5

Source: JICA Study Team

Table 4.2.101 Temporary Coffering Calculation Results (Transverse Direction)

1) Material：SKY400

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-47.9	54.88	15.08	69.96	140
2	Regular scenario LWL[W]	-47.9	62.27	15.08	77.36	140
3	Wind scenario [W]	-47.9	57.65	15.08	72.74	175
4	Marine vessel impact scenario [W]	-31.6	81.96	8.18	90.15	210
5	Seismic scenario [W]	-31.6	148.25	8.18	156.43	210
6	Seismic scenario[Smax]	-31.6	118.98	8.18	127.17	210
7	Seismic scenario [Mmax]	-31.6	118.97	8.18	127.16	210

2) Material：SKY490

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-21.5	49.42	90.66	140.09	185
2	Regular scenario LWL[W]	-21.5	55.92	90.66	146.59	185
3	Wind scenario [W]	-21.5	58.5	90.66	149.17	231.25
4	Marine vessel impact scenario [W]	-21.5	78.82	90.66	169.48	277.5
5	Seismic scenario [W]	-21.5	149.11	90.66	239.78	277.5
6	Seismic scenario[Smax]	-21.5	120.4	90.66	211.06	277.5
7	Seismic scenario [Mmax]	-21.5	120.42	90.66	211.09	277.5

Source: JICA Study Team

4.2.10.2 Calculation for Side Pier (P10 and P13)

(1) Design Conditions

1) Load Case

Table 4.2.102 Load Case

Scenario	Basic Load						Overdesign factor
	Dead load	Live Load	Temperature Load	Wind Load	Impact Load	Seismic Load	
Regular	○	○*1					1.00
Temperature Flux	○	○*1	○				1.15
Wind	○			○			1.25
Marine Vessel Impact	○				○		1.50
Seismic	○					○	1.50

*1 Depending on combination with design water level, cases with and without is verified

Source: JICA Study Team

a) Reaction Force for Substructure Design

Table 4.2.103 Reaction Force for Substructure Design

Scenario		P10 Cable Stayed Bridge＋PC Girder			P13 Cable Stayed Bridge＋Steel Box Girder		
		Rv(KN)	RH(KN)	RM(KNm)	Rv(KN)	RH(KN)	RM(KNm)
Longi. Direction	Reguler HWL	10200	450	12400	6200	0	10120
	Reguler LWL	19000	450	12400	23900	0	10120
	Temperature HWL	10100	750	12400	6100	900	10120
	Temperature LWL	19300	750	12400	24200	900	10120
	Wind	12800	0	12400	8800	0	10120
	Vessel Impact	10200	450	12400	6200	0	10120
	Seismic	12200	4350	12400	8300	900	10120
Trans. Direction	Reguler HWL	10200	100	16800	6200	100	14200
	Reguler LWL	19000	100	16800	23900	-100	14200
	Wind	12800	600	4620	8800	900	2860
	Vessel Impact	10200	100	16800	6200	100	14200
	Seismic	12800	4300	16010	8800	4300	14160

Source: JICA Study Team

Reaction forces at P10 which has critical force (horizontal force and bending moment at seismic scenario) were selected as the design force for substructure of P10 and P13.

2) Design Lateral Seismic Factor

Seismic performance 1 kh = 0.30

Seismic performance 2 kh = 0.45 (used for evaluation of strength in the bridge seat member)

a) Design Water Level

Table 4.2.104 Design Water Level

Scenario	Water Level (MSL+m)	Flow rate (m/s)
Regular (Temperature Flux)	+3.18	---
	-2.39	---
Wind	+4.99	---
Marine Vessel Impact	+3.18	---
Seismic	+0.29	0.60

Source: JICA Study Team

3) Impact Load of Marine Vessel

Longitudinal direction: 4850 kN

Transverse direction: 9700 kN (impact height +3.98)

4) Utilized Material

a) Unit Weight

Reinforced Concrete γc = 24.5 kN/m^3

Filling Sand γd = 18.0 kN/m^3

Water γw = 10.0 kN/m^3

b) Utilized Material and Allowable Stress

Table 4.2.105 Utilized Material and Allowable Stress (Concrete)

(N/mm2)

		Pier	Pile Cap
Design strength σck		30.0	24.0
Compressive stress	Against bending	10.00	8.00
	Against axial force	8.50	6.50
Shearing stress	Borne by concrete only	0.25	0.23
	Bourne together with diagonal tension bars	1.90	1.70
	Punching shear stress (τa^3)	1.00	0.90
Bond stress	Deformed steel bars	1.80	1.60

Source: JICA Study Team

Table 4.2.106 Utilized Material and Allowable Stress (Steel)

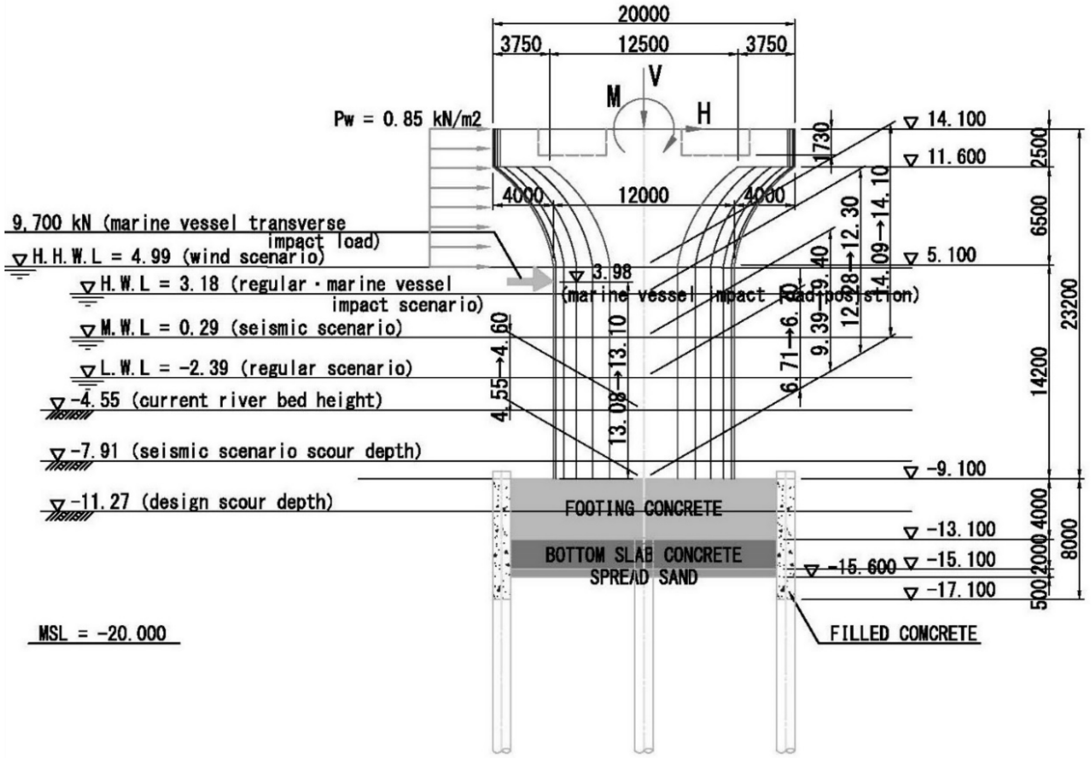
(N/mm2)

<table>
<tr><th colspan="4"></th><th>Pier</th><th>Pile Cap</th></tr>
<tr><td colspan="4">Type of steel member</td><td>SD345</td><td>SD345</td></tr>
<tr><td rowspan="6">Tensile stress</td><td colspan="3">Principal load exluding live load and impact load are in effect</td><td>100.0</td><td>100.0</td></tr>
<tr><td rowspan="2">Load combination does not include effect of impact and seismic event</td><td colspan="2">Regular members</td><td>180.0</td><td>180.0</td></tr>
<tr><td colspan="2">Members underwater or underneath ground water level</td><td>160.0</td><td>160.0</td></tr>
<tr><td colspan="2" rowspan="2">Load combination includes effect of impact and seismic event</td><td>Axial reinforcement</td><td>200.0</td><td>200.0</td></tr>
<tr><td>Other than the above</td><td>200.0</td><td>200.0</td></tr>
<tr><td colspan="3">Calculation of rebar lap joint and embedment length</td><td>200.0</td><td>200.0</td></tr>
<tr><td colspan="4">Compressive stress</td><td>200.0</td><td>200.0</td></tr>
</table>

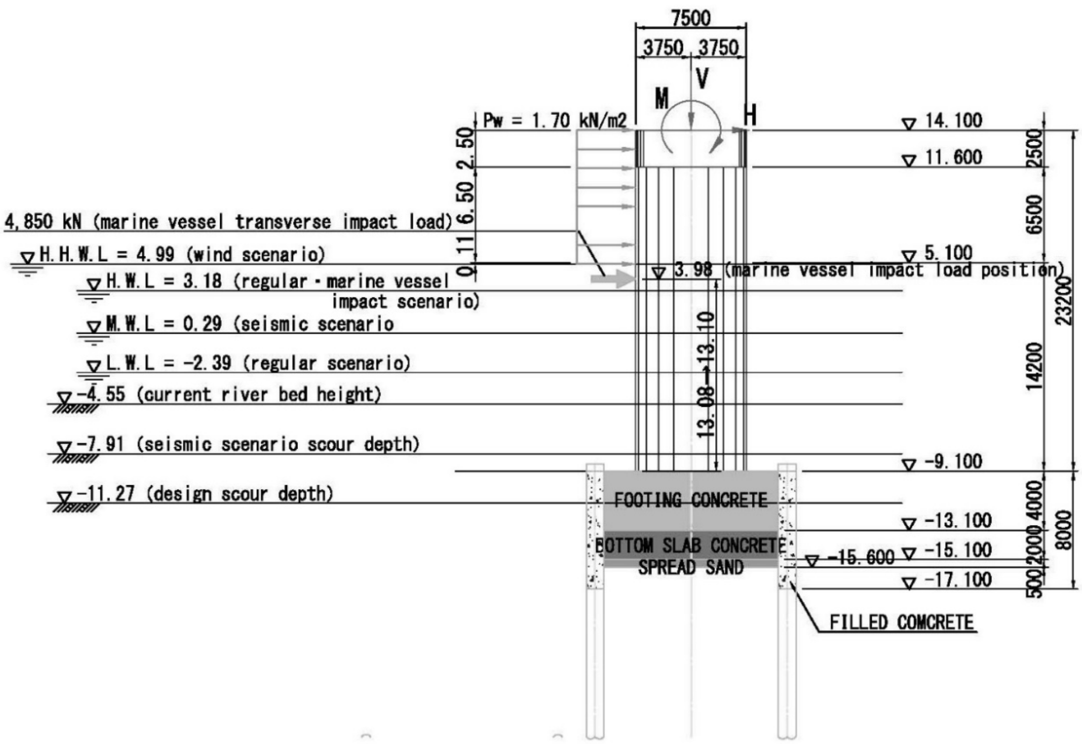
Source: JICA Study Team

5) Design Condition

Front View



Side View



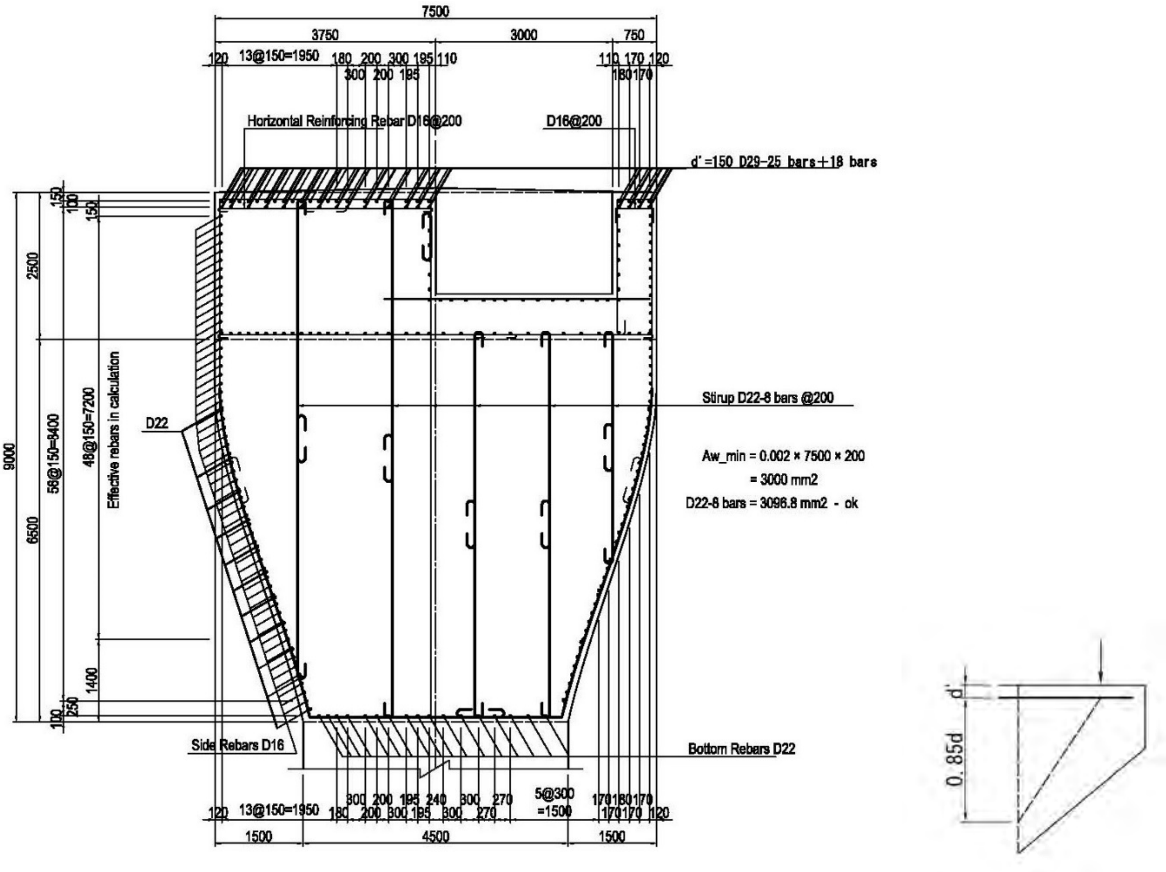
Source: JICA Study Team

Figure 4.2.155 Design Condition

(2) Pier Design

1) Beam Design

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



Note: Side reinforcement is effective in the range of 0.85 times the effective height.
Concrete cover for main steel reinforcement d' = 192 mm
Effective height d = 8808 mm
0.85d = 7487 mm
Effective range for side steel reinforcement = 192 + 7487 = 7679 mm

Source: JICA Study Team

Figure 4.2.156 Cross Section of Beam

[Overview of Calculation Result]

The following table shows the calculation results for the beam.

Table 4.2.107 Calculation Results for Beam

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

Source: JICA Study Team

a) Cross Section Design in Vertical Direction (as a Corbel)

The design tension force needs to be verified because the ratio of the beam height to the distance between root and loading point is more than 1.0.

Table 4.2.108 Evaluation of Amount of Steel Reinforcement

Item	Unit	Dead Load	Dead and Live Load
Load Condition	―――	Dead Load	Regular Load
Design Tensile Force T	kN	2551.18	3035.36
Allowable Tensile Stressσsa	N/mm^2	100	180
Upper Surface Tension Steel Reinforcement Used Amount Asu Required Amount AsuReq	mm^2	Asu ≧ AsuReq OK 27623.2 25511.79	Asu ≧ AsuReq OK 27623.2 16863.09
Additional reinforcement steel for side surface Used Amount Ass Required Amount AssReq	mm^2	Ass ≧ AssReq OK 19462.8 11049.28	Ass ≧ AssReq OK 19462.8 11049.28

※ AsuReq = 1000・T / σsa

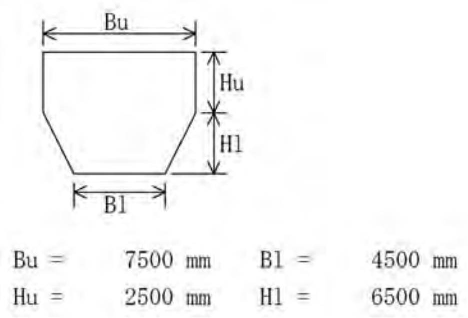
※ AssReq = 0.4・Asu

Source: JICA Study Team

b) Cross Section Design in Vertical Direction (Allowable Stress Method)

- Evaluation for Bending Moment

The evaluation for the bending moment was performed at the root of the beam.



Source: JICA Study Team

Figure 4.2.157 Cross Sectional Shape

Table 4.2.109 Main Steel Reinforcement Used for Cross Section Calculation (Vertical Direction)

No.	Position(mm)	Size	Number	Amount (mm^2)
1	150	D29	25	16060
2	250	D29	18	11563.2
Sum ΣAs = 27623.2				

Note: Minimum amount of steel reinforcement
[Total steel reinforcement amount (27623.2 mm^2)
≧ 500 mm^2 of steel reinforcement amount per m (3750.0 mm^2)] OK
Maximum amount of steel reinforcement
[Tension steel reinforcement amount (27623.2 mm^2)
≦ Balanced reinforcement amount Asb (2028980.4 mm^2)] OK
Source: JICA Study Team

Table 4.2.110 Evaluation Results for Cross Section

Item	Unit	Dead Load	Dead and Live Load
Load Consition	———	Dead load	Regular load
Bending Moment M	kN.m	19100.47	22725.47
Compression Edge～Neutral Axis x	mm	1163	1163
Compressive Stress σc	N/mm^2	0.83	0.99
Tensile Stress σs	N/mm^2	82.64	98.32
Overdesign Factor α	———	1	1
Allowable Compressive Stress σca	N/mm^2	10	10
Allowable Tensile Stress σsa	N/mm^2	100	180
Minimum Reinforcement Amount as Bending Element	———	1.7M≦Mc	1.7M≦Mc

Note: Cracking Bending Moment Mc = 200477.52 kNm, Ultimate Bending Moment Mu = 83534.65 kNm
Source: JICA Study Team

- Evaluation for Shear Force

The evaluation for the shear force was performed at the H/2 point from the beam root and bearing support position outside the H/2 point.

Table 4.2.111 Verified Cross Section

Cross Section	Verified point x(m)	Width (Equivalent Width) b(m)	Beam Height H(m)	$\tan\beta + \tan\gamma$
1	4.5	7.487	2.906	1.625

Source: JICA Study Team

Table 4.2.112 Evaluation Result for Cross Section

Cross Section[1] b = 7487mm h = 2906mm

Item	Unit	Dead Load	Dead and Live Load
State	————	Dead Load	Regular Load
S	kN	124.1	124.1
M	kN.m	15.13	15.13
d	mm	2714	2714
Sh	kN	115.04	115.04
α	————	1	1
pt	%	0.136	0.136
ce	————	0.743	0.743
cpt	————	0.772	0.772
τm	N/mm^2	0.006	0.006
τa_1	N/mm^2	0.143	0.143
τa_2	N/mm^2	1.9	1.9

※ $Sh = S - M / d \cdot (\tan\beta + \tan\gamma)$

$\tau m = Sh / bd$

Here

S : Shear Force

M : Bending Moment

d : Effective Height

$\tan\beta + \tan\gamma$: Effective Height Change

Sh : Shear Force in Accordance with Effective Height Change

pt : Primary tension bar ratio

ce : Correction factor of allowable shear force for effective height d

cpt : Correction factor of allowable shear force for tension bar ratio

τm : Average shear force

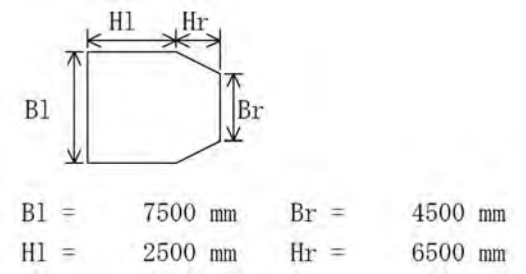
τa1 : Allowable shear force when only concrete bears shear force

τa2 : Allowable shear force when shear reinforcement rebar and concrete bears shear force

Source: JICA Study Team

c) Cross Section Design in Horizontal Direction (Allowable Stress Method)

The evaluation for the bending moment was performed at the root of the beam.



Source: JICA Study Team

Figure 4.2.158 Cross Sectional Shape

Table 4.2.113 Main Steel Reinforcement Used for Cross Section Calculation (Horizontal Direction)

No.	Position(mm)	Size	Number	Amount (mm^2)
1	98	D16	14	2780.4
2	689	D16	35	6951
Sum ΣAs = 9731.4				

Note: Minimum amount of steel reinforcement

[Total steel reinforcement amount (9731.4 mm^2)

≧ 500 mm^2 of steel reinforcement amount per m (4585.4 mm^2)] OK

Maximum amount of steel reinforcement

[Tension steel reinforcement amount (9731.4 mm^2)

≦ Balanced reinforcement amount Asb (2792298.9 mm^2)] OK

Source: JICA Study Team

Table 4.2.114 Evaluation Result for Cross Section

Item	Unit	Temp Flux Scenario	Seismic Scenario
Load Consition	―――	Dead + Temp load	Lv1 Seismic Load
Bending Moment M	kN.m	580	6092.64
Compression Edge～Neutral Axis x	mm	719	719
Compressive Stress σc	N/mm^2	0.07	0.71
Tensile Stress σs	N/mm^2	9.47	99.51
Overdesign Factor α	―――	1.15	1.5
Allowable Compressive Stress σca	N/mm^2	11.5	15
Allowable Tensile Stress σsa	N/mm^2	207	300
Minimum Reinforcement Amount as Bending Element	―――	1.7M≦Mc	1.7M≦Mc

Note: Cracking Bending Moment Mc = 142980.53 kNm, Ultimate Bending Moment Mu = 23344.64 kNm

Source: JICA Study Team

- Evaluation for Shear Force

The evaluation for the shear force was performed at the beam root and bearing support position.

Table 4.2.115 Verified Cross Section

Cross Section	Verified point x(m)	Width (Equivalent Width) b(m)	Beam Height H(m)	$\tan\beta+\tan\gamma$
1	0	7.7	7.5	0
2	2.9	5.228	7.5	0

Source: JICA Study Team

Table 4.2.116 Evaluation Result for Cross Section

Cross Section[1] b = 7700mm h = 7500mm

Item	Unit	Temp Flux Scenario	Seismic Scenario
State	————	Dead + Temp load	Lvl Seismic Load
S	kN	200	2523.89
M	kN.m	580	6092.64
d	mm	6980	6980
Sh	kN	200	2523.89
α	————	1.15	1.5
pt	%	0.018	0.018
ce	————	0.56	0.56
cpt	————	0.536	0.536
τm	N/mm^2	0.004	0.047
τa_1	N/mm^2	0.086	0.111
τa_2	N/mm^2	2.185	2.85

Here

S : Shear Force

M : Bending Moment

d : Effective Height

$\tan\beta+\tan\gamma$: Effective Height Change

Sh : Shear Force in Accordance with Effective Height Change

pt : Primary tension bar ratio

ce : Correction factor of allowable shear force for effective height d

cpt : Correction factor of allowable shear force for tension bar ratio

τm : Average shear force

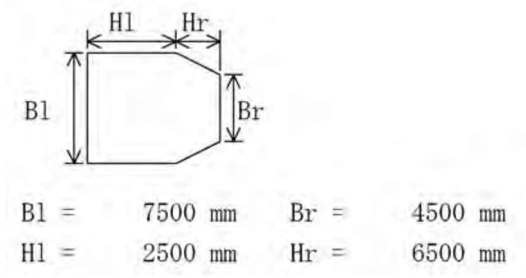
τa1 : Allowable shear force when only concrete bears shear force

τa2 : Allowable shear force when shear reinforcement rebar and concrete bears shear force

Source: JICA Study Team

d) Cross Section Design in Horizontal Direction (Evaluation for Seismic Performance 2)

The evaluation for the bending moment was performed at the root of the beam.



Source: JICA Study Team

Figure 4.2.159 Cross Sectional Shape

Table 4.2.117 Main Steel Reinforcement Used for Cross Section Calculation (Horizontal Direction)

Main Steel Reinforcement (Position means the distance from the side surface of the beam)

No.	Position(mm)	Size	Number	Amount (mm^2)
1	98	D16	14	2780.4
2	689	D16	35	6951
Sum ΣAs =	9731.4			

Note: Total steel reinforcement amount 9731.4 mm^2 satisfies [500 mm^2 of steel reinforcement amount per m (4585.4 mm^2)]

Source: JICA Study Team

Table 4.2.118 Evaluation Result for Cross Section

Item	Unit	Seismic Performance 2
Load Condition	——	Type 2
Bending Moment M	kN.m	8703.96
Yielding Bending Moment My	kN.m	21370.63

Note; Cracking Bending Moment Mc = 142980.53 kNm, Ultimate Bending Moment Mu = 23344.64 kNm

Source: JICA Study Team

- Evaluation for Shear Force

The evaluation for the shear force was performed at the beam root and bearing support position.

Table 4.2.119 Sectional Force

$Sh = S - M / d \cdot (\tan\beta + \tan\gamma)$

No.	Verified point x(m)	Effective Height d(m)	$\tan\beta+\tan\gamma$	Shear Force S (kN)	Bending Moment M (kN.m)	Sh (kN)
1	0	6.98	0	3635.84	8703.96	3635.84
2	2.9	7.207	0	2098.19	488.98	2098.19

Source: JICA Study Team

Table 4.2.120 Evaluation of Shear Strength

$$Ps = Sc + Ss$$

$$Sc = Ce \cdot Cpt \cdot \tau c \cdot b \cdot d$$

$$Ss = \frac{Aw \cdot \sigma sy \cdot d(\sin\theta + \cos\theta)}{1.15 \cdot s}$$

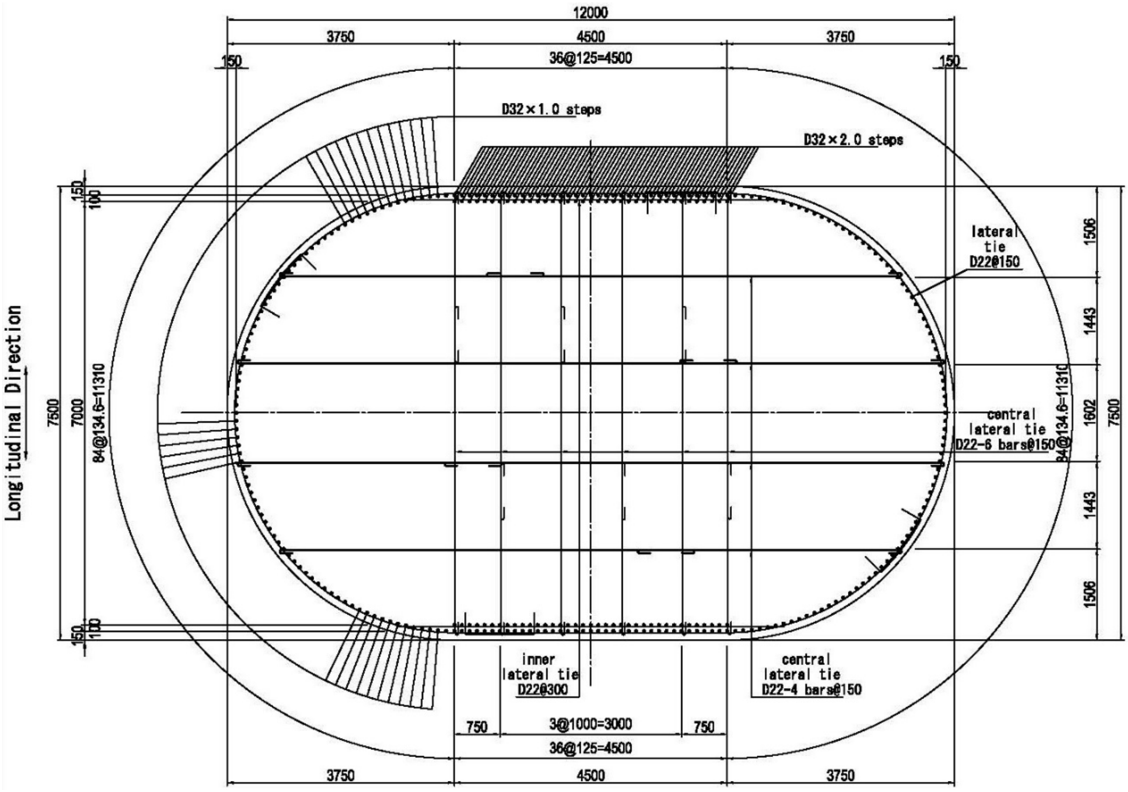
No.	Verified Point x(m)	Sc (kN)	Ss (kN)	Ps (kN)		Sh (kN)
1	0	5975.44	10184.77	16160.21	≧	3635.84
2	2.9	4152.44	10516.36	14668.79	≧	2098.19

Source: JICA Study Team

2) Design of Column

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

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



Source: JICA Study Team

Figure 4.2.160 Cross Section of Column

[Overview of Calculation Result]

The following table shows the calculation results for the column.

Table 4.2.121 Calculation Result for Column

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

Note: ※ was decided by dynamic analysis

Source: JICA Study Team

a) Cross Section Evaluation Results

The evaluation results for the column cross section are shown below.

Table 4.2.122 Examination of Bending Moment (Longitudinal)

Category	Unit	Regular Scenario HWL Water Level Considered	Regular LWL Scenario Water Level Considered	Temperature Flux HWL Scenario Water Level Considered	Temperature Flux LWL Scenario Water Level Considered
Load Condition	―――	Dead load	Regular load	Dead + temp load	Dead + live + temp load
Axial Force N	kN	62727.94	71527.94	62627.94	71827.94
Bending Moment M	kN.m	24556.6	24556.6	31516.6	31516.6
Compression Edge～ Neutral Axis x	mm	14647	16176	12227	13472
Compressive Stress σc	N/mm^2	1.03	1.14	1.11	1.22
Tensile Stress σs	N/mm^2	-7.71	-9.33	-6.62	-8.31
Overdesign Factor α	―――	1	1	1.15	1.15
Allowable Compressive Stress σca	N/mm^2	10	10	11.5	11.5
Allowable Tensile Stress σsa	N/mm^2	-200	-200	-230	-230
Cracking Moment Mc	kN.m	252952.42	262393.41	252845.13	262715.26
Yielding Moment My0	kN.m	430262.88	454422.32	429986.86	455241.75
Ultimate Bending Moment Mu	kN.m	518103.16	546140.58	517783.4	547091.07
Minimum Reinforcement for Bending Element	―――	1.7M≦Mc	1.7M≦Mc	1.7M≦Mc	1.7M≦Mc
Minimum Reinforcement for Axial Element	mm^2	49685.5	56655.8	43135.9	49472.5
Axial Force Nu	kN	65327.94	65327.94	65327.94	65327.94
0.008A1' (Axial Force Na=N)	mm^2	49685.5	56655.8	43135.9	49472.5
0.008A2' (Axial Force Nu)	mm^2	18493.4	18493.4	18493.4	18493.4
Total Reinforcement Content As ≧ Asmin	―――	OK	OK	OK	OK
Maximum Reinforcement Content Evaluation (My0≦Mu)	―――	OK	OK	OK	OK

Category	Unit	Wind Scenario Water Level Considered	Marine Vessel Impact Scenario Water Level Considered	Sesimic Scenario Water Level Considered
Load Condition	―――	Wind load	Impact load	Lv1 Seismic Load
Axial Force N	kN	65327.94	62727.94	64727.94
Bending Moment M	kN.m	19147.13	88091.6	325677.33
Compression Edge～ Neutral Axis x	mm	18305	6689	2471
Compressive Stress σc	N/mm^2	1.01	1.73	7.29
Tensile Stress σs	N/mm^2	-9.03	2.56	216
Overdesign Factor α	―――	1.25	1.5	1.5
Allowable Compressive Stress σca	N/mm^2	12.5	15	15
Allowable Tensile Stress σsa	N/mm^2	-250	300	300
Cracking Moment Mc	kN.m	255741.8	252952.42	255098.1
Yielding Moment My0	kN.m	437432.56	430262.88	435779.14
Ultimate Bending Moment Mu	kN.m	526418.57	518103.16	524502.03
Minimum Reinforcement for Bending Element	―――	1.7M≦Mc	1.7M≦Mc	Mc≦Mu
Minimum Reinforcement for Axial Element	mm^2	41395.9	33123.7	34179.8
Axial Force Nu	kN	65327.94	65327.94	65327.94
0.008A1' (Axial Force Na=N)	mm^2	41395.9	33123.7	34179.8
0.008A2' (Axial Force Nu)	mm^2	18493.4	18493.4	18493.4
Total Reinforcement Content As ≧ Asmin	―――	OK	OK	OK
Maximum Reinforcement Content Evaluation (My0≦Mu)	―――	OK	OK	OK

Source: JICA Study Team

Table 4.2.123 Examination of Bending Moment (Transverse)

Category	Unit	Regular Scenario HWL Water Level Considered	Regular LWL Scenario Water Level Considered	Wind Scenario Water Level Considered
Load Condition	―――	Dead load	Regular load	Wind load
Axial Force N	kN	62727.94	71527.94	65327.94
Bending Moment M	kN.m	19120	19120	19621.93
Compression Edge～Neutral Axis x	mm	38100	42604	38576
Compressive Stress σc	N/mm^2	0.91	1.02	0.95
Tensile Stress σs	N/mm^2	-9.42	-11.04	-9.84
Overdesign Factor α	―――	1	1	1.25
Allowable Compressive Stress σca	N/mm^2	10	10	12.5
Allowable Tensile Stress σsa	N/mm^2	-200	-200	-250
Cracking Moment Mc	kN.m	379370.38	393529.7	383553.82
Yielding Moment My0	kN.m	557398.67	593291.93	568049.29
Ultimate Bending Moment Mu	kN.m	782135.06	823147.76	794314.99
Minimum Reinforcement for Bending Element	―――	1.7M≦Mc	1.7M≦Mc	1.7M≦Mc
Minimum Reinforcement for Axial Element	mm^2	49685.5	56655.8	41395.9
Axial Force Nu	kN	65327.94	65327.94	65327.94
0.008A1' (Axial Force Na=N)	mm^2	49685.5	56655.8	41395.9
0.008A2' (Axial Force Nu)	mm^2	18493.4	18493.4	18493.4
Total Reinforcement Content As ≧ Asmin	―――	OK	OK	OK
Maximum Reinforcement Content Evaluation (My0≦Mu)	―――	OK	OK	OK

Category	Unit	Marine Vessel Impact Scenario Water Level Considered	Sesimic Scenario Water Level Considered
Load Condition	―――	Impact load	Lv1 Seismic Load
Axial Force N	kN	62727.94	65327.94
Bending Moment M	kN.m	146190	319234.78
Compression Edge～Neutral Axis x	mm	9828	5045
Compressive Stress σc	N/mm^2	1.91	4.96
Tensile Stress σs	N/mm^2	5.9	100.32
Overdesign Factor α	―――	1.5	1.5
Allowable Compressive Stress σca	N/mm^2	15	15
Allowable Tensile Stress σsa	N/mm^2	300	300
Cracking Moment Mc	kN.m	379370.38	383553.82
Yielding Moment My0	kN.m	557398.67	568049.29
Ultimate Bending Moment Mu	kN.m	782135.06	794314.99
Minimum Reinforcement for Bending Element	―――	1.7M≦Mc	Mc≦Mu
Minimum Reinforcement for Axial Element	mm^2	33123.7	34496.6
Axial Force Nu	kN	65327.94	65327.94
0.008A1' (Axial Force Na=N)	mm^2	33123.7	34496.6
0.008A2' (Axial Force Nu)	mm^2	18493.4	18493.4
Total Reinforcement Content As ≧ Asmin	―――	OK	OK
Maximum Reinforcement Content Evaluation (My0≦Mu)	―――	OK	OK

Source: JICA Study Team

Table 4.2.124 Examination of Shear Force (Longitudinal)

Category	Unit	Regular Scenario HWL Water Level Considered	Regular LWL Scenario Water Level Considered	Temperature Flux HWL Scenario Water Level Considered	Temperature Flux LWL Scenario Water Level Considered	Wind Scenario Water Level Considered
Load Condition	―――	Dead load	Regular load	Dead + temp load	Dead + live + temp load	Wind load
b	mm	11147	11147	11147	11147	11147
d	mm	6932	6932	6932	6932	6932
S	kN	450	450	750	750	264.04
N	kN	62727.94	71527.94	62627.94	71827.94	65327.94
M	kN.m	24556.6	24556.6	31516.6	31516.6	19147.13
α	―――	1	1	1.15	1.15	1.25
pt	%	0.161	0.161	0.161	0.161	0.161
ce	―――	0.561	0.561	0.561	0.561	0.561
cpt	―――	0.823	0.823	0.823	0.823	0.823
CN	―――	1	1	1	1	1
τm	N/mm^2	0.006	0.006	0.01	0.01	0.003
$τa_1$	N/mm^2	0.115	0.115	0.133	0.133	0.144
$τa_2$	N/mm^2	1.9	1.9	2.185	2.185	2.375
σsa	N/mm^2	―――	―――	―――	―――	―――
s	mm	―――	―――	―――	―――	―――
Sca	kN	―――	―――	―――	―――	―――
Sh’	kN	―――	―――	―――	―――	―――
AwReq	mm^2	―――	―――	―――	―――	―――
Aw	mm^2	―――	―――	―――	―――	―――

Category	Unit	Marine Vessel Impact Scenario Water Level Considered	Sesimic Scenario Water Level Considered
Load Condition	―――	Impact load	Lv1 Seismic Load
b	mm	11147	11147
d	mm	6932	6932
S	kN	5300	21903.86
N	kN	62727.94	64727.94
M	kN.m	88091.6	325677.33
α	―――	1.5	1.5
pt	%	0.161	0.161
ce	―――	0.561	0.561
cpt	―――	0.823	0.823
CN	―――	1	1
τm	N/mm^2	0.069	0.283
$τa_1$	N/mm^2	0.171	0.171
$τa_2$	N/mm^2	2.85	2.85
σsa	N/mm^2	―――	300
s	mm	―――	150
Sca	kN	―――	13204.33
Sh’	kN	―――	8699.53
AwReq	mm^2	―――	721.59
Aw	mm^2	―――	3096.8

Here
- S : Shear Force
- N : Axial Load
- M : Bending Moment
- b : Sectional Width of Element
- d : Effective Height
- α : Overdesign factor for allowable stress
- pt : Primary tension bar ratio
- ce : Correction factor of allowable shear force for effective height d
- cpt : Correction factor of allowable shear force for tension bar ratio
- CN : Correction factor due to longitudinal compressive load
- τm : Average shear force
- τa1 : Allowable shear force when only concrete bears shear force
- τa2 : Allowable shear force when shear reinforcement rebar and concrete bears shear force
- σsa : Allowable tensile stress of rebar
- s : Spacing of shear reinforcement rebar
- Sca : Shear force borne by concrete
- Sh’ : Shear force borne by reinforcement rebar
- Awreq : Necessary shear reinforcement content to meet condition τa1 < τm
- Aw : Shear reinforcement content

Source: JICA Study Team

Table 4.2.125 Examination of Shear Force (Transverse)

Category	Unit	Regular Scenario HWL Water Level Considered	Regular LWL Scenario Water Level Considered	Wind Scenario Water Level Considered	Marine Vessel Impact Scenario Water Level Considered
Load Condition	―――	Dead load	Regular load	Wind load	Impact load
b	mm	6991	6991	6991	6991
d	mm	11064	11064	11064	11064
S	kN	100	100	658.01	9800
N	kN	62727.94	71527.94	65327.94	62727.94
M	kN.m	19120	19120	19621.93	146190
α	―――	1	1	1.25	1.5
pt	%	0.161	0.161	0.161	0.161
ce	―――	0.5	0.5	0.5	0.5
cpt	―――	0.822	0.822	0.822	0.822
CN	―――	1	1	1	1
τm	N/mm^2	0.001	0.001	0.009	0.127
$τa_1$	N/mm^2	0.103	0.103	0.128	0.152
$τa_2$	N/mm^2	1.9	1.9	2.375	2.85
σsa	N/mm^2	―――	―――	―――	―――
s	mm	―――	―――	―――	―――
Sca	kN	―――	―――	―――	―――
Sh’	kN	―――	―――	―――	―――
AwReq	mm^2	―――	―――	―――	―――
Aw	mm^2	―――	―――	―――	―――

Category	Unit	Wind Scenario Water Level Considered
Load Condition	―――	Wind load
b	mm	6991
d	mm	11064
S	kN	20068.53
N	kN	65327.94
M	kN.m	319234.78
α	―――	1.5
pt	%	0.161
ce	―――	0.5
cpt	―――	0.822
CN	―――	1
τm	N/mm^2	0.259
$τa_1$	N/mm^2	0.152
$τa_2$	N/mm^2	2.85
σsa	N/mm^2	300
s	mm	150
Sca	kN	11768.5
Sh’	kN	8300.04
AwReq	mm^2	431.35
Aw	mm^2	2322.6

Here
- S : Shear Force
- N : Axial Load
- M : Bending Moment
- b : Sectional Width of Element
- d : Effective Height
- α : Overdesign factor for allowable stress
- pt : Primary tension bar ratio
- ce : Correction factor of allowable shear force for effective height d
- cpt : Correction factor of allowable shear force for tension bar ratio
- CN : Correction factor due to longitudinal compressive load
- τm : Average shear force
- τa1 : Allowable shear force when only concrete bears shear force
- τa2 : Allowable shear force when shear reinforcement rebar and concrete bears shear force
- σsa : Allowable tensile stress of rebar
- s : Spacing of shear reinforcement rebar
- Sca : Shear force borne by concrete
- Sh’ : Shear force borne by reinforcement rebar
- Awreq : Necessary shear reinforcement content to meet condition τa1 < τm
- Aw : Shear reinforcement content

Source: JICA Study Team

b) Evaluation of Cross Section through Dynamic Analysis

Steel reinforcements in the column-axial direction were decided based on the dynamic analysis evaluation. The following table shows the results of the dynamic analysis.

Table 4.2.126 Dynamic Analysis Results for P10

P10Pier Result of Cross Section Calculation in Longitudinal Direction

	Element no.	Bending Moment(kN·m)		Axial Force(kN)		Bending Moment	Axial Force	Bending Stress (N/mm2)		Judgement	Shear Force(kN)			Shear Stress		Rebar for Shear		Judgement
								σc	σs					τ	τa	Required Rebar Amount	Arranged Rebar Amount	
		Max	Min	Max	Min	(kN·m)	(kN)	σca=15	σsa=300		Max	Min	Adopted	(N/mm2)	(N/mm2)	(cm2)	(cm2)	
Upper End of Column	6010	51135.9	-26954.4	-26178.8	-41699.1	51135.9	41699.1				10496.7	-10553.4	10553.4					
Curved Section	6011	71089.1	-46660.2	-30821.2	-24066.4	71089.1	30821.2				12266.2	-12292.3	12292.3					
	6012	93152.3	-68759.3	-34612.6	-28276.2	93152.3	34612.6				13599.4	-13593.7	13599.4					
	6013	116815.0	-92524.7	-37790.8	-31755.4	116815.0	37790.8				14641.5	-14592.3	14641.5					
P10 Pier	6014	144279.0	-120182.0	-41037.8	-34963.1	144279.0	41037.8	3.0	59.9	○	15630.3	-15537.2	15630.3	0.202	0.223		30.968	○
	6015	173250.0	-149437.0	-44428.2	-38277.1	173250.0	44428.2	3.7	84.5	○	16565.0	-16456.3	16565.0	0.214	0.218		30.968	○
	6016	203597.0	-180218.0	-47818.2	-41663.0	203597.0	47818.2	4.4	111.8	○	17493.9	-17315.4	17493.9	0.226	0.214	0.796	30.968	○
	6017	235347.0	-212487.0	-51208.2	-45049.3	235347.0	51208.2	5.2	141.5	○	18526.4	-18279.2	18526.4	0.240	0.211	1.855	30.968	○
	6018	268612.0	-246312.0	-54598.0	-48436.1	268612.0	54598.0	6.0	173.7	○	19595.3	-19238.8	19595.3	0.254	0.208	2.910	30.968	○
	6019	301556.0	-280462.0	-57987.4	-51823.4	301556.0	57987.4	6.8	205.8	○	20621.2	-19908.7	20621.2	0.267	0.206	3.890	30.968	○
	6020	337526.0	-317815.0	-61376.8	-55211.1	337526.0	61376.8	7.6	242.2	○	21463.6	-20705.8	21463.6	0.278	0.204	4.711	30.968	○
	6021	344978.0	-325483.0	-63410.2	-57921.6	344978.0	63410.2	7.8	245.3	○	22136.3	-21285.5	22136.3	0.286	0.205	5.245	30.968	○
	6022	352344.0	-333235.0	-64088.2	-59277.0	352344.0	64088.2	8.0	252.8	○	22314.9	-21419.3	22314.9	0.289	0.204	5.416	30.968	○
	6023	359803.0	-341074.0	-64766.0	-59954.7	359803.0	64766.0	8.2	260.4	○	22490.9	-21550.1	22490.9	0.291	0.204	5.584	30.968	○
	6024	367302.0	-348967.0	-65443.8	-60632.4	367302.0	65443.8	8.3	268.1	○	22664.5	-21678.1	22664.5	0.293	0.204	5.750	30.968	○
Lower End of Column	6025	374855.0	-356903.0	-66121.6	-61310.1	374855.0	66121.6	8.5	275.9	○	22835.7	-21803.5	22835.7	0.295	0.203	5.913	30.968	○

Source: JICA Study Team

Table 4.2.127 Dynamic Analysis Results for P13

P13Pier Result of Cross Section Calculation in Longitudinal Direction

	Element no.	Bending Moment(kN·m)		Axial Force(kN)		Bending Moment	Axial Force	Bending Stress (N/mm2)		Judgement	Shear Force(kN)			Shear Stress		Rebar for Shear		Judgement
								σc	σs					τ	τa	Required Rebar Amount	Arranged Rebar Amount	
		Max	Min	Max	Min	(kN·m)	(kN)	σca=15	σsa=300		Max	Min	Adopted	(N/mm2)	(N/mm2)	(cm2)	(cm2)	
Upper End of Column	6310	14460.3	-36151.7	-22319.4	-18153.3	36151.7	22319.4				8256.8	-7907.2	8256.8					
Curved Section	6311	30931.4	-53261.4	-26960.8	-22792.2	53261.4	26960.8				10537.8	-10148.2	10537.8					
	6312	50144.1	-73105.3	-30751.8	-26581.0	73105.3	30751.8				12230.5	-11834.2	12230.5					
	6313	71457.3	-95025.1	-33929.4	-29757.0	95025.1	33929.4				13511.6	-13130.4	13511.6					
P13 Pier	6314	96863.2	-121035.0	-37176.0	-33002.2	121035.0	37176.0	2.5	43.8	○	14689.8	-14343.8	14689.8	0.190	0.227		30.968	○
	6315	124267.0	-148916.0	-40565.6	-36390.6	148916.0	40565.6	3.1	66.3	○	15796.3	-15498.4	15796.3	0.204	0.221		30.968	○
	6316	153460.0	-178472.0	-43955.2	-39779.0	178472.0	43955.2	3.8	92.1	○	16835.8	-16557.5	16835.8	0.218	0.216	0.116	30.968	○
	6317	184452.0	-209637.0	-47344.6	-43167.4	209637.0	47344.6	4.6	120.8	○	17949.6	-17652.8	17949.6	0.232	0.212	1.280	30.968	○
	6318	217267.0	-242456.0	-50734.0	-46556.0	242456.0	50734.0	5.4	152.2	○	19308.3	-18846.2	19308.3	0.250	0.209	2.602	30.968	○
	6319	251838.0	-274776.0	-54123.2	-49944.6	274776.0	54123.2	6.2	183.3	○	20683.7	-19993.3	20683.7	0.268	0.207	3.886	30.968	○
	6320	285258.0	-309642.0	-57512.4	-53333.4	309642.0	57512.4	7.0	218.1	○	21763.5	-20856.8	21763.5	0.282	0.205	4.914	30.968	○
	6321	291535.0	-317174.0	-59545.8	-55366.8	317174.0	59545.8	7.1	221.3	○	22850.3	-21589.0	22850.3	0.296	0.205	5.791	30.968	○
	6322	298892.0	-324758.0	-60223.6	-56044.6	324758.0	60223.6	7.3	229.1	○	23123.8	-21770.9	23123.8	0.299	0.205	6.045	30.968	○
	6323	306338.0	-332424.0	-60901.4	-56722.2	332424.0	60901.4	7.5	237.0	○	23394.1	-21948.3	23394.1	0.303	0.204	6.296	30.968	○
	6324	313851.0	-340197.0	-61579.2	-57400.0	340197.0	61579.2	7.7	245.1	○	23661.1	-22121.6	23661.1	0.306	0.204	6.543	30.968	○
Lower End of Column	6325	321410.0	-348034.0	-62257.0	-58077.8	348034.0	62257.0	7.9	253.2	○	23925.3	-22291.0	23925.3	0.310	0.204	6.787	30.968	○

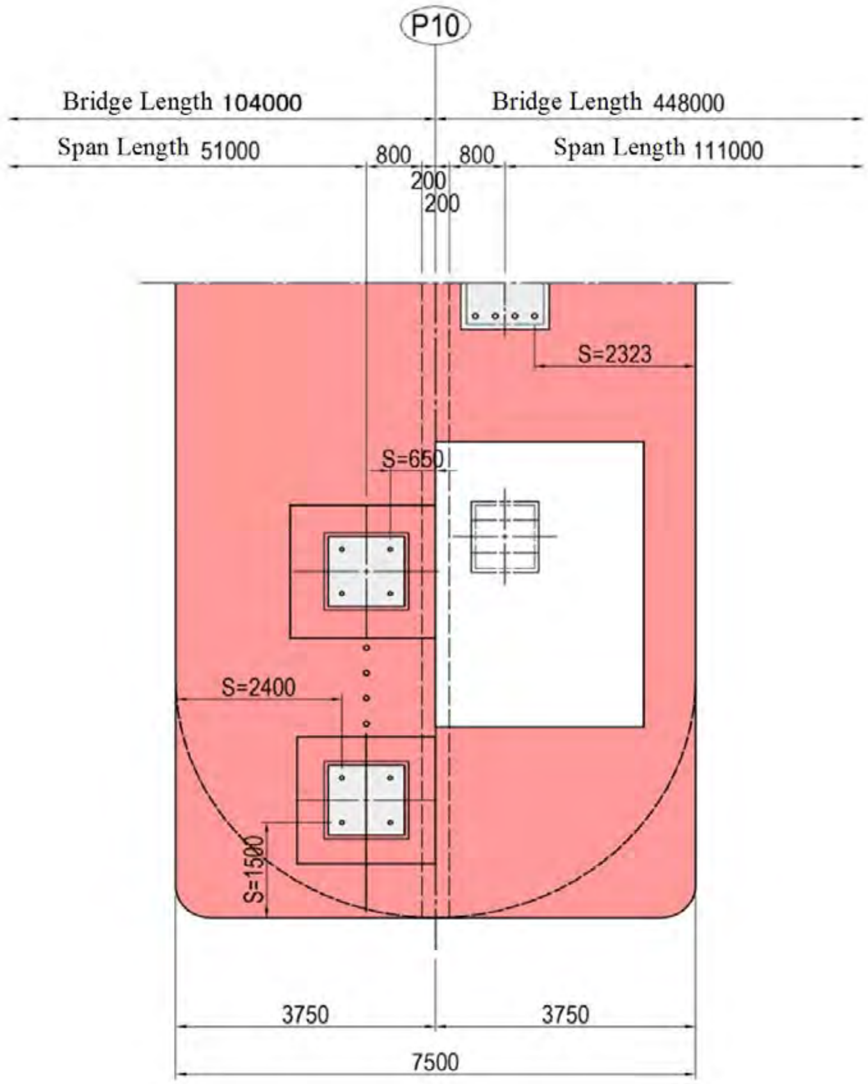
Source: JICA Study Team

3) Bridge Seat Design

a) Dimension of Bridge Seat Width

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

[P10 Pier]



Source: JICA Study Team

Figure 4.2.161 Bridge Seat Width

- Evaluation of edge distance for bearing support

The edge distance of bearing support was set through the following equation:

S1 = 0.2 + 0.0051

= 0.2 + 0.005 × 51.000 = 0.455 m

Hence, the edge distance of bearing support can be set as:

$S1 = 0.455\ \mathrm{m} < 0.650$ ・・・OK

Similarly, the edge distance of the other bearing support was set through the following equation:

S2 = 0.2 + 0.0051

= 0.2 + 0.005 × 111.000 = 0.755 m

Hence, the edge distance of bearing support can be set as:

$S2 = 0.755\ \mathrm{m} < 2.323$ ・・・OK

- Evaluation of length of beam placement on column

The beam placement length is configured to satisfy the following equation:

SEM = 0.7 + 0.0051

= 0.7 + 0.005 × 111.000 = 1.255 m

SE = UR + UG

= 0.560 + 0.555 = 1.115 m

UR = 0.560 m (0.5 times longitudinal bearing width (Specifications of Highway Bridges (p. 306))

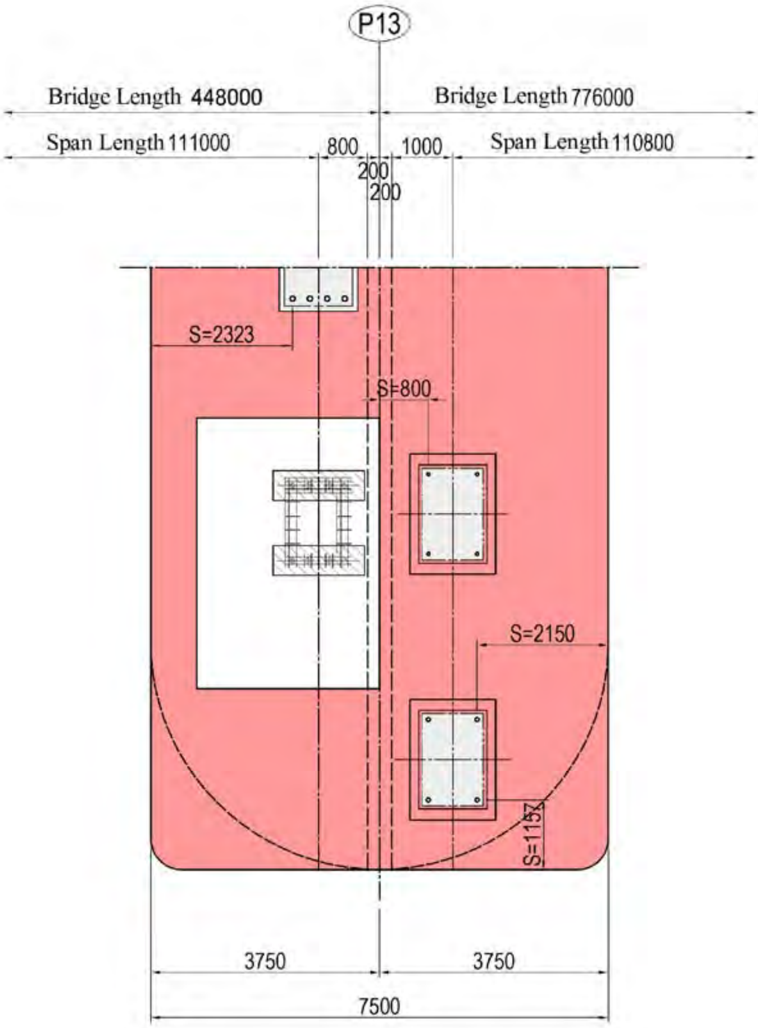
UG = $\varepsilon g \cdot L$ (Type III Ground)

= 0.00500 × 111.000 = 0.555m

Therefore, the length of beam placement on column is as follows:

SE = 1.255 m < 3.550 m ・・・OK

[P13 Pier]



Source: JICA Study Team

Figure 4.2.162 Bridge Seat Width

- Evaluation of edge distance for bearing support

The edge distance of bearing support was set through the following equation:

$S1 = 0.2 + 0.0051$

$= 0.2 + 0.005 \times 111.000 = 0.755$ m

Hence, the edge distance of bearing support can be set as:

$S1 = 0.755\ \mathrm{m} < 2.323\mathrm{m}$ ・・・OK

Similarly, the edge distance of the other bearing support was set through the following equation:

$S2 = 0.2 + 0.0051$

$= 0.2 + 0.005 \times 110.800 = 0.754$ m

Hence, the edge distance of bearing support can be set as:

$S2 = 0.754\ \mathrm{m} < 0.800\mathrm{m}$ ・・・OK

- Evaluation of length of beam placement on column

The beam placement length is configured to satisfy the following equation:

$SEM = 0.7 + 0.0051$

$= 0.7 + 0.005 \times 110.000 = 1.255$ m

$SE = UR + UG$

$= 0.560 + 0.555 = 1.115$ m

$UR = 0.560$ m (0.5 times longitudinal bearing width (Specifications of Highway Bridges (p. 306))

$UG = \varepsilon g \cdot L$ (Type III Ground)

$= 0.00500 \times 111.000 = 0.555$

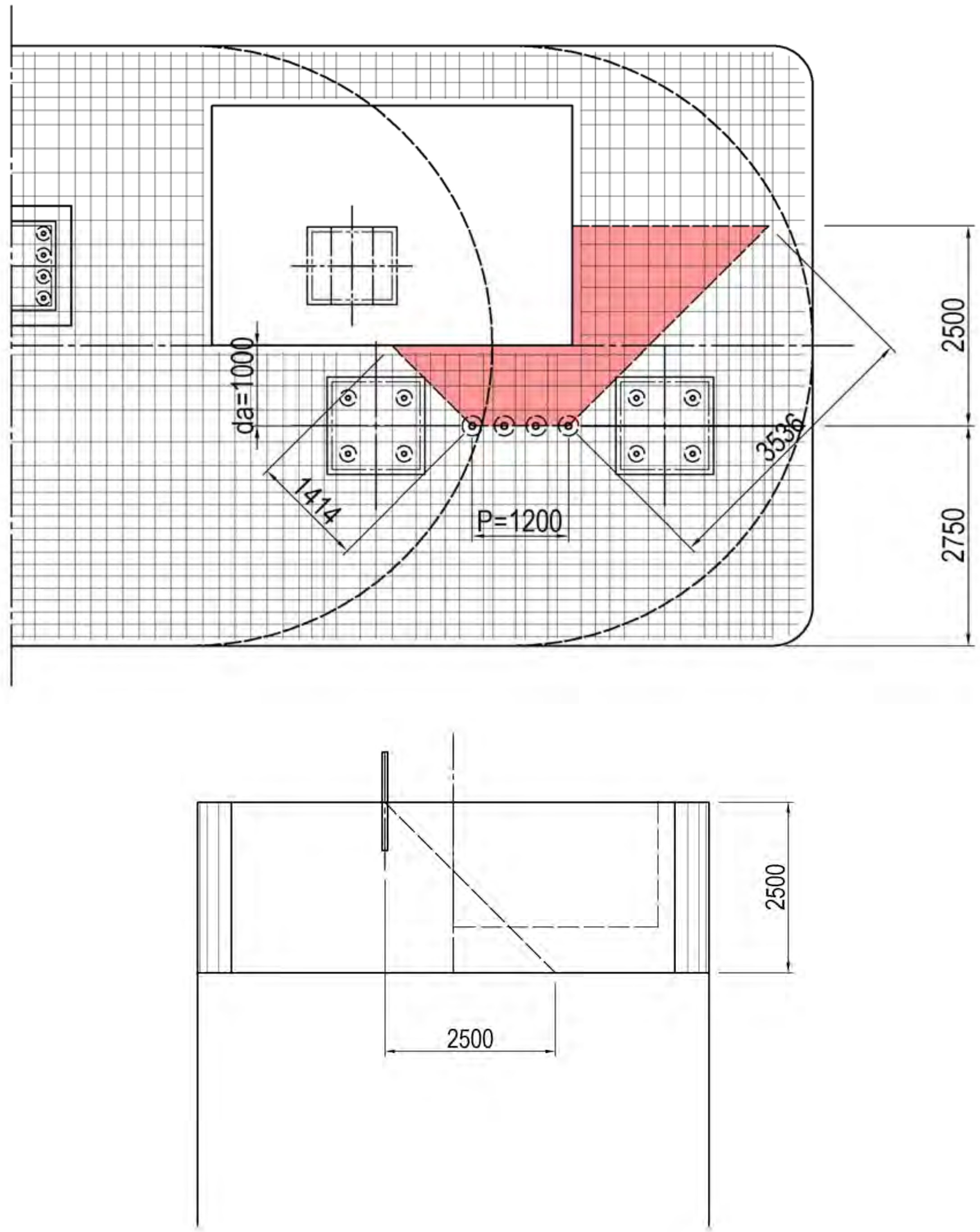
Therefore, the length of beam placement on column is as follows:

$SE = 1.255\ \mathrm{m} < 3.550\ \mathrm{m}$ ・・・OK

b) Evaluation of Bridge Seat Strength

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

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



Source: JICA Study Team

Figure 4.2.163 Resistance Area of Concrete

- Evaluation of strength

$Pbs = Pc + Ps \quad (Pc \geqq Ps)$

$Pbs = 2.0 \times Pc \quad (Pc < Ps)$

Where,

Pbs : Strength of bridge seat (kN)

Note that the strength is determined under the condition that the strength borne by reinforcements does not exceed that borne by concrete.

Pc : Strength borne by concrete (kN)

$Pc = (\alpha \cdot 0.32 \cdot \sqrt{\sigma ck} \cdot Ac) / 1000.0$

Ps : Strength borne by reinforcement (kN)

$Ps = \Sigma\{\beta \cdot (1 - hi / da) \cdot \sigma sy \cdot Asi\} / 1000.0$

α : Coefficient for determining the strength borne by concrete

σn : Bearing stress at bottom of bearing support against vertical force

σck : Design strength of concrete (kN/mm^2)

Ac : Resistance area of concrete (mm^2)

β : Correction factor associated with the strength borne by reinforcement

hi : Distance from bridge seat surface of ith reinforcement (m)

da : Distance from center of anchor bolt in the rear side of bearing support to bridge seat edge

σsy : Yield point of reinforcement (N/mm^2)

Asi : Cross sectional area of ith reinforcement (mm^2)

Table 4.2.128 Result of Bridge Seat Evaluation

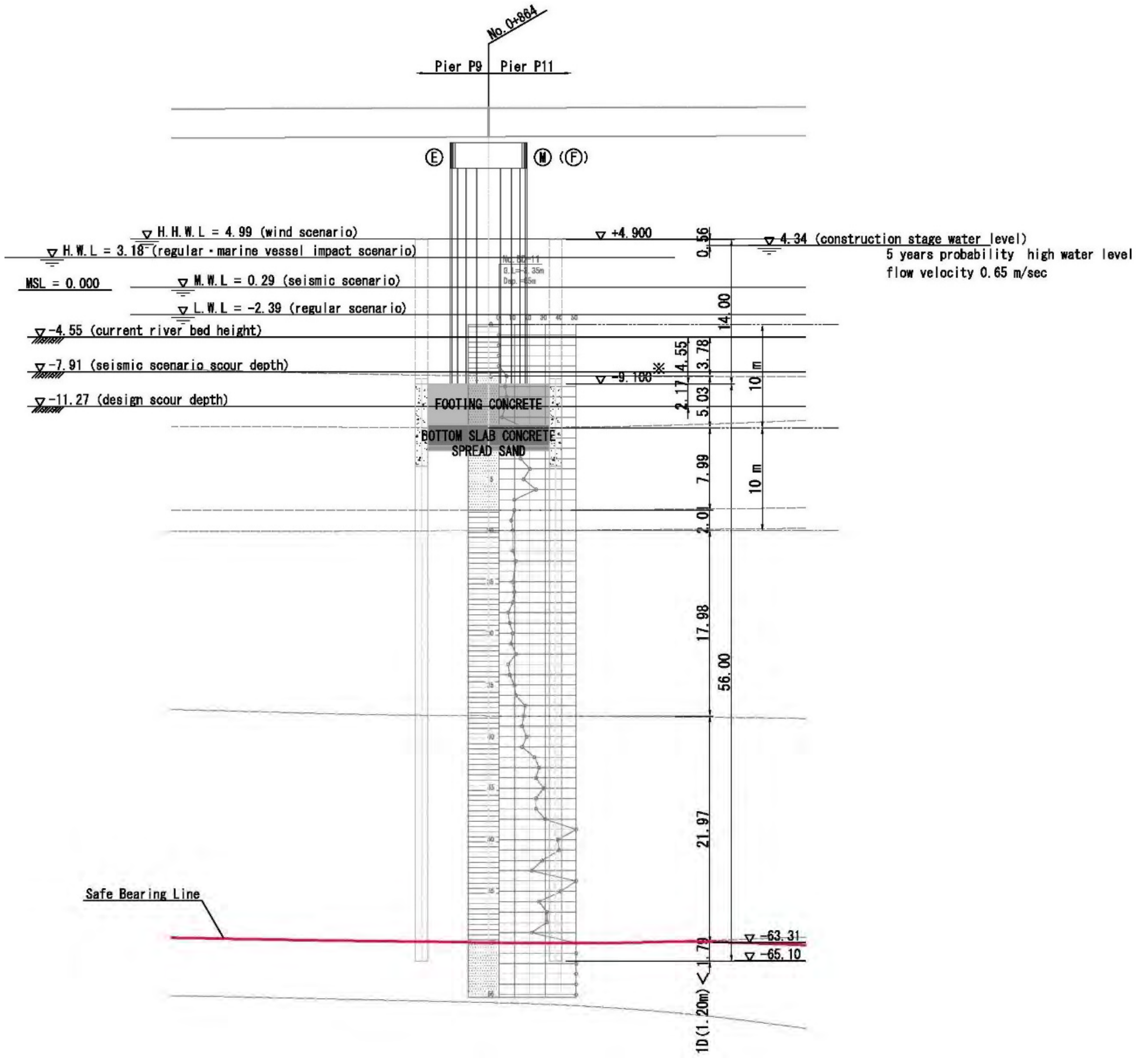
Items	Results
Resistance area of concrete Ac (mm^2)	11844514
Bearing stress σn (N/mm^2)	0
Coefficient for determining the strength borne by concrete α	0.15
Strength borne by concrete Pc (kN)	3114.004
Strength borne by reinforcement Ps (kN)	1896.405
Design horizontal seismic force Ph (kN)	3000
Strength of bridge seat Pbs (kN)	5010.409
Judge (Ph≦Pbs)	OK

Source: JICA Study Team

(3) Foundation Design

1) Ground Conditions

The following figure shows the ground condition:

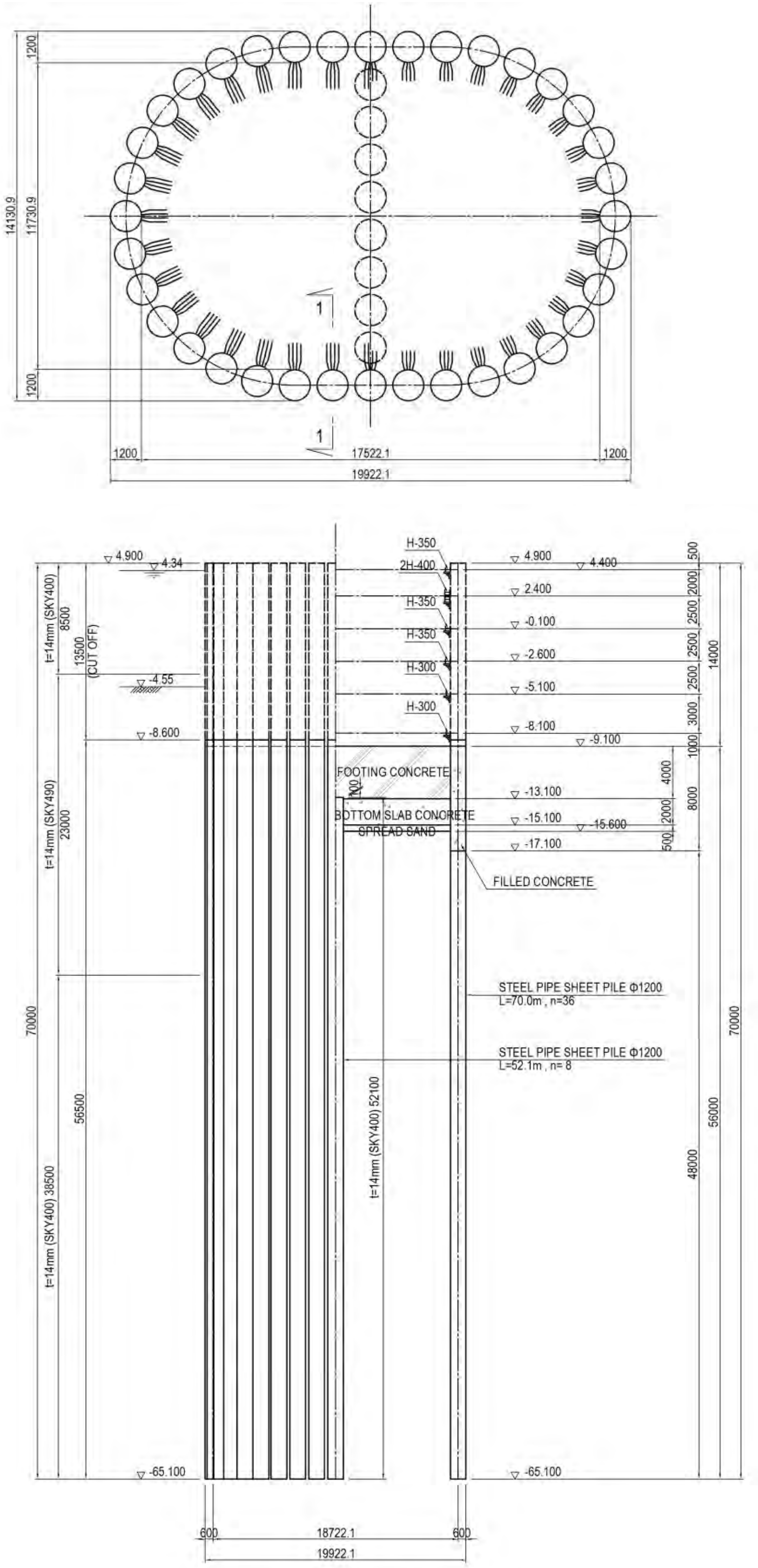


Source: JICA Study Team

Figure 4.2.164 Ground Condition

2) Foundation Shape (Steel Pile Sheet Pile Foundation)

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



Source: JICA Study Team

Figure 4.2.165 Dimensional Drawing of Foundation Shape

[Calculation Result Table]

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

Table 4.2.129 Calculation Results for Foundation

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

Source: JICA Study Team

3) Evaluation Results (Current Riverbed)

The steel pipe sheet pile foundation was designed by satisfying the following conditions:

- Reaction force in longitudinal direction from steel pipe sheet pile ≦ Allowable bearing capacity, Displacement ≦ Allowable displacement
- Stress of steel pile sheet pile ≦ Allowable stress

The evaluation results are shown in the next page.

Table 4.2.130 Evaluation Results for Foundation (Longitudinal Direction) - 1

Items		Unit		HWL[W] at regular senario	LWL[W] at regular senario	HWL[W] temperature flux senario
Acting force		Vo	kN	74258.6	87399.3	74158.6
		Ho	kN	450	450	750
		Mo	kN.m	24556.6	24556.6	31516.6
Level crown of fundation	Displacement	δ1	cm	0.11	0.11	0.152
	Deflection angle	θ1	mrad	-0.09	-0.09	-0.122
Design ground surface	Displacement	δ2	cm	0.11	0.11	0.152
	Deflection angle	θ2	mrad	-0.09	-0.09	-0.122
Max bending moment of opening caisson		Mmax	kN.m	-26162	-26162	-34502
Location of Mmax		Lm	m	-15.1	-15.1	-15.6
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	41.83	48.52	43.21
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	43.24	49.93	44.92
		Lm	m	-15.1	-15.1	-15.6
	Partitioned sheet pile (SKY400)	σmax	N/mm2	42.44	49.13	43.87
		Lm	m	-15.1	-15.1	-15.6
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――	―――	―――
		Lm	m	―――	―――	―――
	Pile (SKK400)	σmax	N/mm2	―――	―――	―――
	Pile (SKK490)	σmax	N/mm2	―――	―――	―――
Max bending moment of opening caisson at bottom		MB	kN.m	739	739	976
Vertical reaction force	Maximum	Rmax	kN/pile	1692	1991	1692
	Minimum	Rmin	kN/pile	1683	1982	1679
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3893	3893	3893
	Pulling-out bearing capasity	Pa	kN/pile	-1959	-1959	-1959
	Stress (SKY400)	σa	N/mm2	140	140	161
	Stress (SKY490)	σa	N/mm2	185	185	212.75

Source: JICA Study Team

Table 4.2.131 Evaluation Results for Foundation (Longitudinal Direction) - 2

Items		Unit		LWL[W] at temperature senario	Wind senario [W]	Marine vessle impact senario [W]
Acting force		Vo	kN	87699.3	75448.1	74258.6
		Ho	kN	750	264	5300
		Mo	kN.m	31516.6	19147.1	88091.6
Level crown of fundation	Displacement	δ1	cm	0.152	0.08	0.627
	Deflection angle	θ1	mrad	-0.122	-0.068	-0.446
Design ground surface	Displacement	δ2	cm	0.152	0.08	0.627
	Deflection angle	θ2	mrad	-0.122	-0.068	-0.446
Max bending moment of opening caisson		Mmax	kN.m	-34502	-19972	-116848
Location of Mmax		Lm	m	-15.6	-14.1	-18.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	50.1	41.41	58.28
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	51.82	42.56	62.08
		Lm	m	-15.6	-14.1	-18.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	50.76	41.95	58.49
		Lm	m	-15.6	-14.1	-18.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	———	———	———
		Lm	m	———	———	———
	Pile (SKK400)	σmax	N/mm2	———	———	———
	Pile (SKK490)	σmax	N/mm2	———	———	———
Max bending moment of opening caisson at bottom		MB	kN.m	976	563	3234
Vertical reaction force	Maximum	Rmax	kN/pile	1999	1718	1708
	Minimum	Rmin	kN/pile	1987	1711	1667
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3893	5839	3893
	Pulling-out bearing capasity	Pa	kN/pile	-1959	-3344	-1959
	Stress (SKY400)	σa	N/mm2	161	175	210
	Stress (SKY490)	σa	N/mm2	212.75	231.25	277.5

Source: JICA Study Team

Table 4.2.132 Evaluation Results for Foundation (Longitudinal Direction) - 3

Items		Unit		Seismic senario [W]	Dynamic analysis Smax	Dynamic analysis Mmax
Acting force		Vo	kN	78510.8	78177.1	78493.9
		Ho	kN	21903.9	22835.7	-20836.6
		Mo	kN.m	325677.3	356633	-374855
Level crown of fundation	Displacement	δ1	cm	2.29	2.506	-2.426
	Deflection angle	θ1	mrad	-1.681	-1.829	1.812
Design ground surface	Displacement	δ2	cm	2.29	2.506	-2.426
	Deflection angle	θ2	mrad	-1.681	-1.829	1.812
Max bending moment of opening caisson		Mmax	kN.m	-460266	-496750	498649
Location of Mmax		Lm	m	-18.1	-18.1	-18.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	119.68	126.05	125.01
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	135.56	142.97	143.52
		Lm	m	-18.1	-18.1	-18.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	121.43	127.72	128.21
		Lm	m	-18.1	-18.1	-18.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	———	———	———
		Lm	m	———	———	———
	Pile (SKK400)	σmax	N/mm2	———	———	———
	Pile (SKK490)	σmax	N/mm2	———	———	———
Max bending moment of opening caisson at bottom		MB	kN.m	19595	21627	-21449
Vertical reaction force	Maximum	Rmax	kN/pile	1910	1916	1922
	Minimum	Rmin	kN/pile	1658	1638	1646
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	5839	5839	5839
	Pulling-out bearing capasity	Pa	kN/pile	-3344	-3344	-3344
	Stress (SKY400)	σa	N/mm2	210	210	210
	Stress (SKY490)	σa	N/mm2	277.5	277.5	277.5

Source: JICA Study Team

Table 4.2.133 Evaluation Results for Foundation (Transverse Direction) - 1

Items		Unit		HWL[W] at regular senario	LWL[W] at regular senario	Wind senario [W]
Acting force		Vo	kN	74258.6	87399.3	75448.1
		Ho	kN	100	100	658
		Mo	kN.m	19120	19120	19621.9
Level crown of fundation	Displacement	δ1	cm	0.064	0.064	0.103
	Deflection angle	θ1	mrad	-0.048	-0.048	-0.066
Design ground surface	Displacement	δ2	cm	0.064	0.064	0.103
	Deflection angle	θ2	mrad	-0.048	-0.048	-0.066
Max bending moment of opening caisson		Mmax	kN.m	-19333	-19333	-23096
Location of Mmax		Lm	m	-13.36	-13.36	-18.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	40.79	47.48	42.5
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	41.73	48.42	43.1
		Lm	m	-13.36	-13.36	-18.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	39.15	45.84	40.02
		Lm	m	-13.36	-13.36	-18.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	———	———	———
		Lm	m	———	———	———
	Pile (SKK400)	σmax	N/mm2	———	———	———
	Pile (SKK490)	σmax	N/mm2	———	———	———
Max bending moment of opening caisson at bottom		MB	kN.m	594	594	502
Vertical reaction force	Maximum	Rmax	kN/pile	1691	1990	1718
	Minimum	Rmin	kN/pile	1684	1983	1712
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3893	3893	5839
	Pulling-out bearing capasity	Pa	kN/pile	-1959	-1959	-3344
	Stress (SKY400)	σa	N/mm2	140	140	175
	Stress (SKY490)	σa	N/mm2	185	185	231.25

Source: JICA Study Team

Table 4.2.134 Evaluation Results for Foundation (Transverse Direction) - 2

Items		Unit		Marine vessle impact senario [W]	Seismic senario [W]	Dynamic analysis Smax
Acting force		Vo	kN	74258.6	79110.8	77726.3
		Ho	kN	9800	20068.5	26332.8
		Mo	kN.m	146190	319234.8	453519
Level crown of fundation	Displacement	δ1	cm	1.145	2.026	3.104
	Deflection angle	θ1	mrad	-0.653	-1.279	-1.871
Design ground surface	Displacement	δ2	cm	1.145	2.026	3.104
	Deflection angle	θ2	mrad	-0.653	-1.279	-1.871
Max bending moment of opening caisson		Mmax	kN.m	-210778	-454812	-635677
Location of Mmax		Lm	m	-20.1	-20.1	-20.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	77.68	123.45	157.69
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	80.57	132.54	168.52
		Lm	m	-20.1	-20.1	-20.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	52.46	71.89	83.75
		Lm	m	-20.1	-20.1	-20.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――	―――	―――
		Lm	m	―――	―――	―――
	Pile (SKK400)	σmax	N/mm2	―――	―――	―――
	Pile (SKK490)	σmax	N/mm2	―――	―――	―――
Max bending moment of opening caisson at bottom		MB	kN.m	2103	19547	24002
Vertical reaction force	Maximum	Rmax	kN/pile	1701	1919	1915
	Minimum	Rmin	kN/pile	1675	1677	1618
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3893	5839	5839
	Pulling-out bearing capasity	Pa	kN/pile	-1959	-3344	-3344
	Stress (SKY400)	σa	N/mm2	210	210	210
	Stress (SKY490)	σa	N/mm2	277.5	277.5	277.5

Source: JICA Study Team

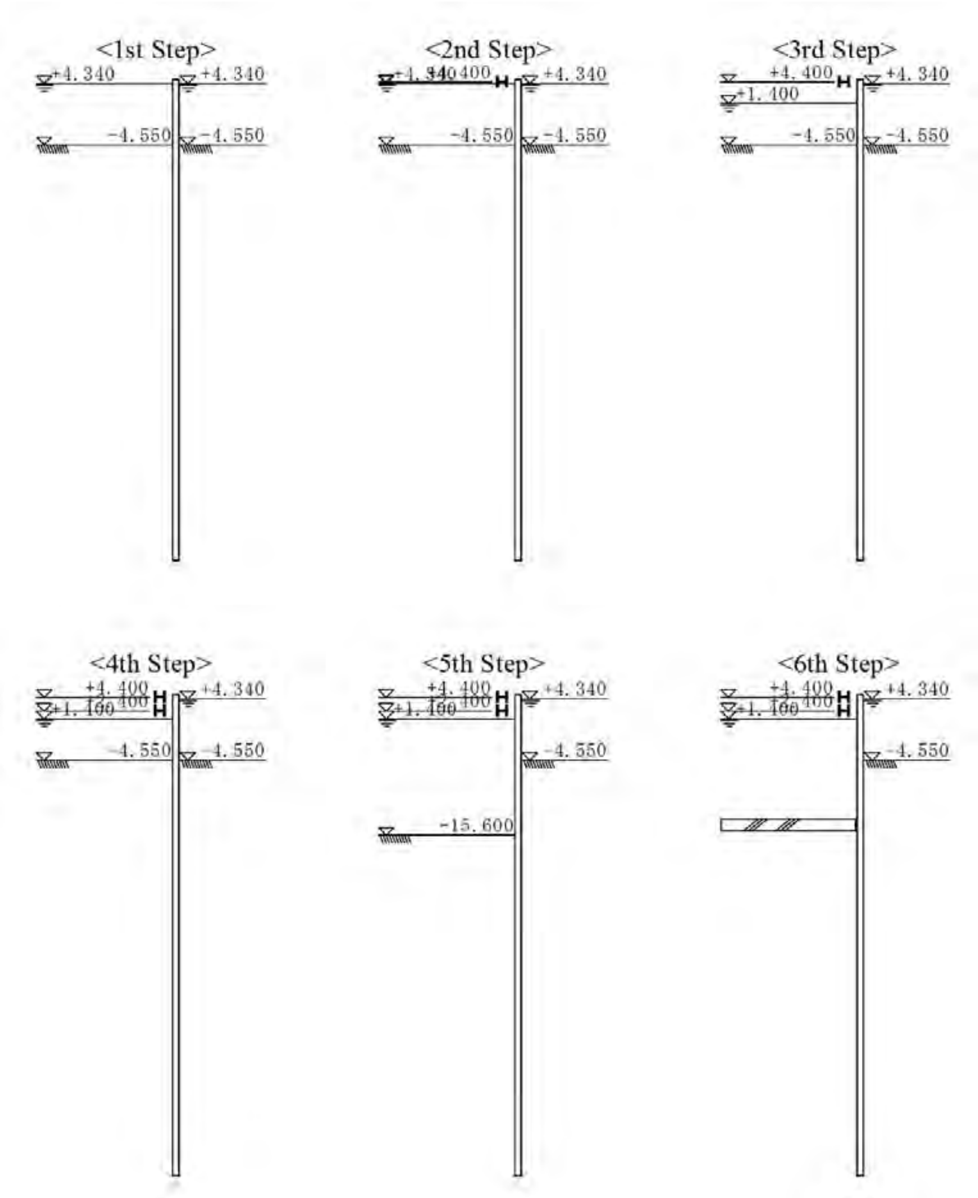
Table 4.2.135 Evaluation Results for Foundation (Transverse Direction) - 3

Items		Unit		Dynamic analysis Mmax
Acting force		Vo	kN	77703.9
		Ho	kN	23647.4
		Mo	kN.m	470199
Level crown of fundation	Displacement	δ1	cm	2.93
	Deflection angle	θ1	mrad	-1.815
Design ground surface	Displacement	δ2	cm	2.93
	Deflection angle	θ2	mrad	-1.815
Max bending moment of opening caisson		Mmax	kN.m	-627628
Location of Mmax		Lm	m	-19.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	154.5
		Lm	m	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	166.88
		Lm	m	-19.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	83.18
		Lm	m	-19.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――
		Lm	m	―――
	Pile (SKK400)	σmax	N/mm2	―――
	Pile (SKK490)	σmax	N/mm2	―――
Max bending moment of opening caisson at bottom		MB	kN.m	25507
Vertical reaction force	Maximum	Rmax	kN/pile	1924
	Minimum	Rmin	kN/pile	1608
Allowable value	Displacement	δa	cm	5
	Pushing bearing capacity	Ra	kN/pile	5839
	Pulling-out bearing capasity	Pa	kN/pile	-3344
	Stress (SKY400)	σa	N/mm2	210
	Stress (SKY490)	σa	N/mm2	277.5

Source: JICA Study Team

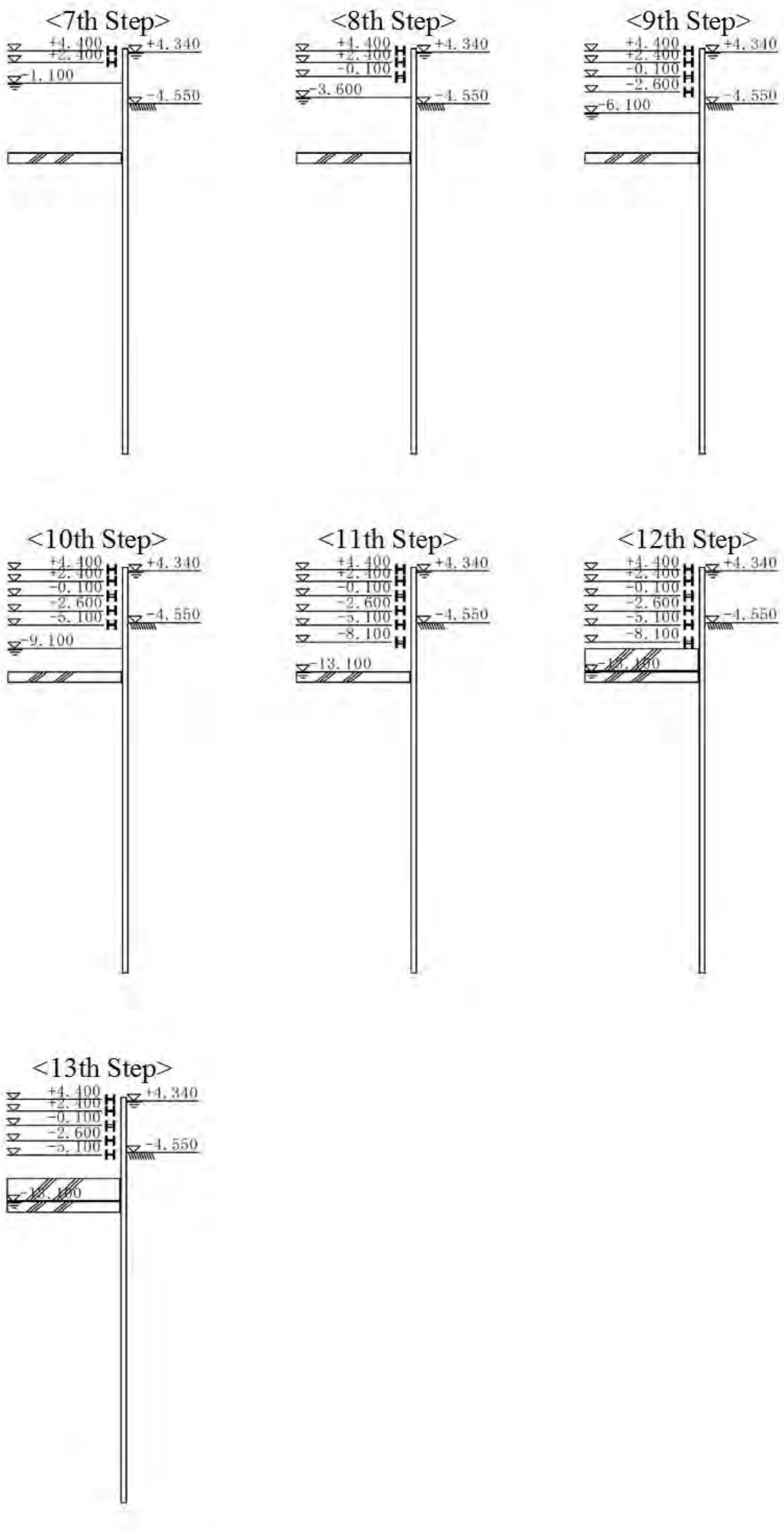
4) Temporary Coffering Calculation (Current Riverbed)

During the temporary coffering, as the stress the steel pipe sheet pile is subjected to is affected by the construction sequence, the stress is calculated for each construction stage.



Source: JICA Study Team

Figure 4.2.166 Construction Stage (1st – 6th Stage)



Source: JICA Study Team

Figure 4.2.167 Construction Stage (7th – 13th Stage)

Table 4.2.136 Temporary Coffering Calculation Results (Longitudinal Direction)

・Longitudinal Direction

Item		Unit	1st Step	2nd Step	3rd Step	4th Step	5th Step	6th Step	7th Step
Maximum Displacement		cm	0.165	0.165	2.203	2.203	7.505	4.920	8.615
Cofferdam Section	SKY400	N/mm2	0.63	0.63	62.99	62.99	102.15	83.19	158.40
	SKY490	N/mm2	1.02	1.02	61.41	61.41	136.42	100.07	182.23
Well Section	SKY400	N/mm2	0.04	0.04	5.83	5.83	58.00	31.02	51.56
	SKY490	N/mm2	1.02	1.02	26.91	26.91	137.65	100.07	180.59
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	210.00
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	280.00
Item		単位	8th Step	9th Step	10th Step	11th Step	12th Step	13th Step	
Maximum Displacement		cm	9.041	9.238	9.321	9.306	9.294	9.294	
Cofferdam Section	SKY400	N/mm2	175.13	172.06	168.21	168.82	169.43	169.43	
	SKY490	N/mm2	207.30	222.73	230.20	227.46	226.57	226.57	
Well Section	SKY400	N/mm2	45.19	41.44	38.31	34.98	35.05	35.05	
	SKY490	N/mm2	200.19	214.58	223.22	219.46	218.07	218.70	
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	

Source: JICA Study Team

Table 4.2.137 Temporary Coffering Calculation Results (Transverse Direction)

・Traverse Direction

Item		Unit	1st Step	2nd Step	3rd Step	4th Step	5th Step	6th Step	7th Step
Maximum Displacement		cm	0.165	0.165	2.218	2.218	7.533	4.945	8.657
Cofferdam Section	SKY400	N/mm2	0.63	0.63	62.79	62.79	102.09	83.08	158.37
	SKY490	N/mm2	1.02	1.02	61.18	61.18	136.30	99.88	182.18
Well Section	SKY400	N/mm2	0.04	0.04	5.82	5.82	58.12	31.06	51.71
	SKY490	N/mm2	1.02	1.02	26.62	26.62	137.51	99.88	180.54
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	210.00
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	280.00
Item		単位	8th Step	9th Step	10th Step	11th Step	12th Step	13th Step	
Maximum Displacement		cm	9.152	9.387	9.496	9.497	9.484	9.484	
Cofferdam Section	SKY400	N/mm2	177.40	175.03	170.72	170.40	170.94	170.94	
	SKY490	N/mm2	208.83	225.02	232.71	229.69	228.89	228.89	
Well Section	SKY400	N/mm2	45.68	42.26	39.52	36.64	36.71	36.71	
	SKY490	N/mm2	201.30	216.48	225.71	222.05	220.75	220.75	
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	

Source: JICA Study Team

5) Total Stress Calculation (Current Riverbed)

The stress the steel pipe sheet pile is subjected to is evaluated as the total of the leftover stress from the construction stage and the design external force after completion.

Leftover stress + Design external force after completion ≦ Allowable stress of steel pipe sheet pile

Table 4.2.138 Temporary Coffering Calculation Results (Longitudinal Direction)

1) Material：SKY400

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-26.6	41.83	34.98	76.82	140
2	Regular scenario LWL[W]	-26.6	48.52	34.98	83.51	140
3	Temperature flux scenario HWL[W]	-26.6	43.21	34.98	78.19	161
4	Temperature flux scenario LWL[W]	-26.6	50.1	34.98	85.08	161
5	Wind scenario [W]	-26.6	41.41	34.98	76.39	175
6	Marine vessel impact scenario [W]	-26.6	58.28	34.98	93.26	210
7	Seismic scenario [W]	-26.6	119.68	34.98	154.66	210
8	Seismic scenario[Smax]	-26.6	126.05	34.98	161.04	210
9	Seismic scenario [Mmax]	-26.6	125.01	34.98	159.99	210

2) Material：SKY490

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-22.1	42.61	69.58	112.19	185
2	Regular scenario LWL[W]	-22.1	49.31	69.58	118.88	185
3	Temperature flux scenario HWL[W]	-22.1	44.21	69.58	113.79	212.75
4	Temperature flux scenario LWL[W]	-22.1	51.1	69.58	120.68	212.75
5	Wind scenario [W]	-22.1	42.02	69.58	111.6	231.25
6	Marine vessel impact scenario [W]	-22.1	61.12	69.58	130.7	277.5
7	Seismic scenario [W]	-22.1	131.81	69.58	201.39	277.5
8	Seismic scenario[Smax]	-22.1	138.95	69.58	208.53	277.5
9	Seismic scenario [Mmax]	-22.1	138.55	69.58	208.13	277.5

Source: JICA Study Team

Table 4.2.139 Temporary Coffering Calculation Results (Transverse Direction)

1) Material：SKY400

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-26.6	40.79	36.64	77.44	140
2	Regular scenario LWL[W]	-26.6	47.48	36.64	84.13	140
3	Wind scenario [W]	-26.6	42.5	36.64	79.14	175
4	Marine vessel impact scenario [W]	-26.6	77.68	36.64	114.32	210
5	Seismic scenario [W]	-26.6	123.45	36.64	160.1	210
6	Seismic scenario[Smax]	-26.6	157.69	36.64	194.34	210
7	Seismic scenario [Mmax]	-26.6	154.5	36.64	191.15	210

2) Material：SKY490

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-22.1	41.27	72.26	113.53	185
2	Regular scenario LWL[W]	-22.1	47.96	72.26	120.22	185
3	Wind scenario [W]	-22.1	42.94	72.26	115.2	231.25
4	Marine vessel impact scenario [W]	-22.1	80.37	72.26	152.63	277.5
5	Seismic scenario [W]	-22.1	131.33	72.26	203.59	277.5
6	Seismic scenario[Smax]	-22.1	167.3	72.26	239.56	277.5
7	Seismic scenario [Mmax]	-22.1	164.96	72.26	237.23	277.5

Source: JICA Study Team

6) Evaluation Results (Considering Scour)

Evaluation of steel pipe sheet pile foundation was also done considering the effects of scour.

The steel pipe sheet pile foundation was designed by satisfying the following conditions:

- Reaction force in longitudinal direction from steel pipe sheet pile ≦ Allowable bearing capacity, Displacement ≦ Allowable displacement
- Stress of steel pile sheet pile ≦ Allowable stress

Table 4.2.140 Evaluation Results for Foundation (Longitudinal Direction) - 1

Items		Unit		HWL[W] at regular scenario	LWL[W] at regular scenario	HWL[W] temperature flux scenario
Acting force		Vo	kN	68171.9	81312.6	68071.9
		Ho	kN	450	450	750
		Mo	kN.m	24556.6	24556.6	31516.6
Level crown of fundation	Displacement	δ1	cm	0.117	0.117	0.162
	Deflection angle	θ1	mrad	-0.094	-0.094	-0.127
Design ground surface	Displacement	δ2	cm	0.097	0.097	0.135
	Deflection angle	θ2	mrad	-0.086	-0.086	-0.116
Max bending moment of opening caisson		Mmax	kN.m	-26706	-26706	-35322
Location of Mmax		Lm	m	-15.6	-15.6	-16.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	38.92	45.61	40.36
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	40.26	46.95	42
		Lm	m	-15.6	-15.6	-16.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	39.44	46.13	40.91
		Lm	m	-15.6	-15.6	-16.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――	―――	―――
		Lm	m	―――	―――	―――
	Pile (SKK400)	σmax	N/mm2	―――	―――	―――
	Pile (SKK490)	σmax	N/mm2	―――	―――	―――
Max bending moment of opening caisson at bottom		MB	kN.m	754	754	996
Vertical reaction force	Maximum	Rmax	kN/pile	1554	1853	1553
	Minimum	Rmin	kN/pile	1545	1843	1541
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3893	3893	3893
	Pulling-out bearing capasity	Pa	kN/pile	-1959	-1959	-1959
	Stress (SKY400)	σa	N/mm2	140	140	161
	Stress (SKY490)	σa	N/mm2	185	185	212.75

Source: JICA Study Team

Table 4.2.141 Evaluation Results for Foundation (Longitudinal Direction) - 2

Items		Unit		LWL[W] at temperature scenario	Wind scenario [W]	Marine vessel impact scenario [W]
Acting force		Vo	kN	81612.6	69361.4	68171.9
		Ho	kN	750	264	5300
		Mo	kN.m	31516.6	19147.1	88091.6
Level crown of fundation	Displacement	δ1	cm	0.162	0.085	0.668
	Deflection angle	θ1	mrad	-0.127	-0.07	-0.467
Design ground surface	Displacement	δ2	cm	0.135	0.071	0.57
	Deflection angle	θ2	mrad	-0.116	-0.064	-0.437
Max bending moment of opening caisson		Mmax	kN.m	-35322	-20330	-121160
Location of Mmax		Lm	m	-16.1	-15.1	-18.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	47.26	38.45	56.25
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	48.89	39.54	59.87
		Lm	m	-16.1	-15.1	-18.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	47.81	38.91	56.15
		Lm	m	-16.1	-15.1	-18.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――	―――	―――
		Lm	m	―――	―――	―――
	Pile (SKK400)	σmax	N/mm2	―――	―――	―――
	Pile (SKK490)	σmax	N/mm2	―――	―――	―――
Max bending moment of opening caisson at bottom		MB	kN.m	996	573	3325
Vertical reaction force	Maximum	Rmax	kN/pile	1861	1580	1571
	Minimum	Rmin	kN/pile	1848	1573	1528
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3893	5839	3893
	Pulling-out bearing capasity	Pa	kN/pile	-1959	-3344	-1959
	Stress (SKY400)	σa	N/mm2	161	175	210
	Stress (SKY490)	σa	N/mm2	212.75	231.25	277.5

Source: JICA Study Team

Table 4.2.142 Evaluation Results for Foundation (Longitudinal Direction) - 3

Items		Unit		Seismic scenario [W]	Dynamic analysis Smax	Dynamic analysis Mmax
Acting force		Vo	kN	74016	73682.3	73999.1
		Ho	kN	21903.9	22835.7	-20836.6
		Mo	kN.m	325677.3	356633	-374855
Level crown of fundation	Displacement	δ1	cm	2.29	2.506	-2.426
	Deflection angle	θ1	mrad	-1.681	-1.829	1.812
Design ground surface	Displacement	δ2	cm	2.29	2.506	-2.426
	Deflection angle	θ2	mrad	-1.681	-1.829	1.812
Max bending moment of opening caisson		Mmax	kN.m	-460266	-496750	498649
Location of Mmax		Lm	m	-18.1	-18.1	-18.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	117.39	123.77	122.72
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	133.27	140.68	141.24
		Lm	m	-18.1	-18.1	-18.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	119.14	125.43	125.92
		Lm	m	-18.1	-18.1	-18.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	—	—	—
		Lm	m	—	—	—
	Pile (SKK400)	σmax	N/mm2	—	—	—
	Pile (SKK490)	σmax	N/mm2	—	—	—
Max bending moment of opening caisson at bottom		MB	kN.m	19595	21627	-21449
Vertical reaction force	Maximum	Rmax	kN/pile	1808	1814	1820
	Minimum	Rmin	kN/pile	1556	1536	1544
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	5839	5839	5839
	Pulling-out bearing capasity	Pa	kN/pile	-3344	-3344	-3344
	Stress (SKY400)	σa	N/mm2	210	210	210
	Stress (SKY490)	σa	N/mm2	277.5	277.5	277.5

Source: JICA Study Team

Table 4.2.143 Evaluation Results for Foundation (Transverse Direction) - 1

Items		Unit		HWL[W] at regular scenario	LWL[W] at regular scenario	Wind scenario [W]
Acting force		Vo	kN	68171.9	81312.6	69361.4
		Ho	kN	100	100	658
		Mo	kN.m	19120	19120	19621.9
Level crown of fundation	Displacement	δ1	cm	0.067	0.067	0.108
	Deflection angle	θ1	mrad	-0.05	-0.05	-0.068
Design ground surface	Displacement	δ2	cm	0.057	0.057	0.094
	Deflection angle	θ2	mrad	-0.046	-0.046	-0.064
Max bending moment of opening caisson		Mmax	kN.m	-19515	-19515	-23654
Location of Mmax		Lm	m	-14.1	-14.1	-18.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	37.78	44.47	39.54
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	38.67	45.36	40.12
		Lm	m	-14.1	-14.1	-18.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	36.07	42.76	36.96
		Lm	m	-14.1	-14.1	-18.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	———	———	———
		Lm	m	———	———	———
	Pile (SKK400)	σmax	N/mm2	———	———	———
	Pile (SKK490)	σmax	N/mm2	———	———	———
Max bending moment of opening caisson at bottom		MB	kN.m	584	584	490
Vertical reaction force	Maximum	Rmax	kN/pile	1553	1852	1579
	Minimum	Rmin	kN/pile	1546	1844	1573
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3893	3893	5839
	Pulling-out bearing capasity	Pa	kN/pile	-1959	-1959	-3344
	Stress (SKY400)	σa	N/mm2	140	140	175
	Stress (SKY490)	σa	N/mm2	185	185	231.25

Source: JICA Study Team

Table 4.2.144 Evaluation Results for Foundation (Transverse Direction) - 2

Items		Unit		Marine vessel impact scenario [W]	Seismic scenario [W]	Dynamic analysis Smax
Acting force		Vo	kN	68171.9	74616	73231.5
		Ho	kN	9800	20068.5	26332.8
		Mo	kN.m	146190	319234.8	453519
Level crown of fundation	Displacement	δ1	cm	1.158	2.026	3.104
	Deflection angle	θ1	mrad	-0.664	-1.279	-1.871
Design ground surface	Displacement	δ2	cm	1.017	2.026	3.104
	Deflection angle	θ2	mrad	-0.631	-1.279	-1.871
Max bending moment of opening caisson		Mmax	kN.m	-216793	-454812	-635677
Location of Mmax		Lm	m	-21.1	-20.1	-20.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	75.75	121.17	155.41
		Lm	m	-26.6	-26.6	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	78.69	130.25	166.23
		Lm	m	-21.1	-20.1	-20.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	49.78	69.6	81.47
		Lm	m	-21.1	-20.1	-20.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	―――	―――	―――
		Lm	m	―――	―――	―――
	Pile (SKK400)	σmax	N/mm2	―――	―――	―――
	Pile (SKK490)	σmax	N/mm2	―――	―――	―――
Max bending moment of opening caisson at bottom		MB	kN.m	2587	19547	24002
Vertical reaction force	Maximum	Rmax	kN/pile	1565	1817	1813
	Minimum	Rmin	kN/pile	1533	1575	1516
Allowable value	Displacement	δa	cm	5	5	5
	Pushing bearing capacity	Ra	kN/pile	3893	5839	5839
	Pulling-out bearing capasity	Pa	kN/pile	-1959	-3344	-3344
	Stress (SKY400)	σa	N/mm2	210	210	210
	Stress (SKY490)	σa	N/mm2	277.5	277.5	277.5

Source: JICA Study Team

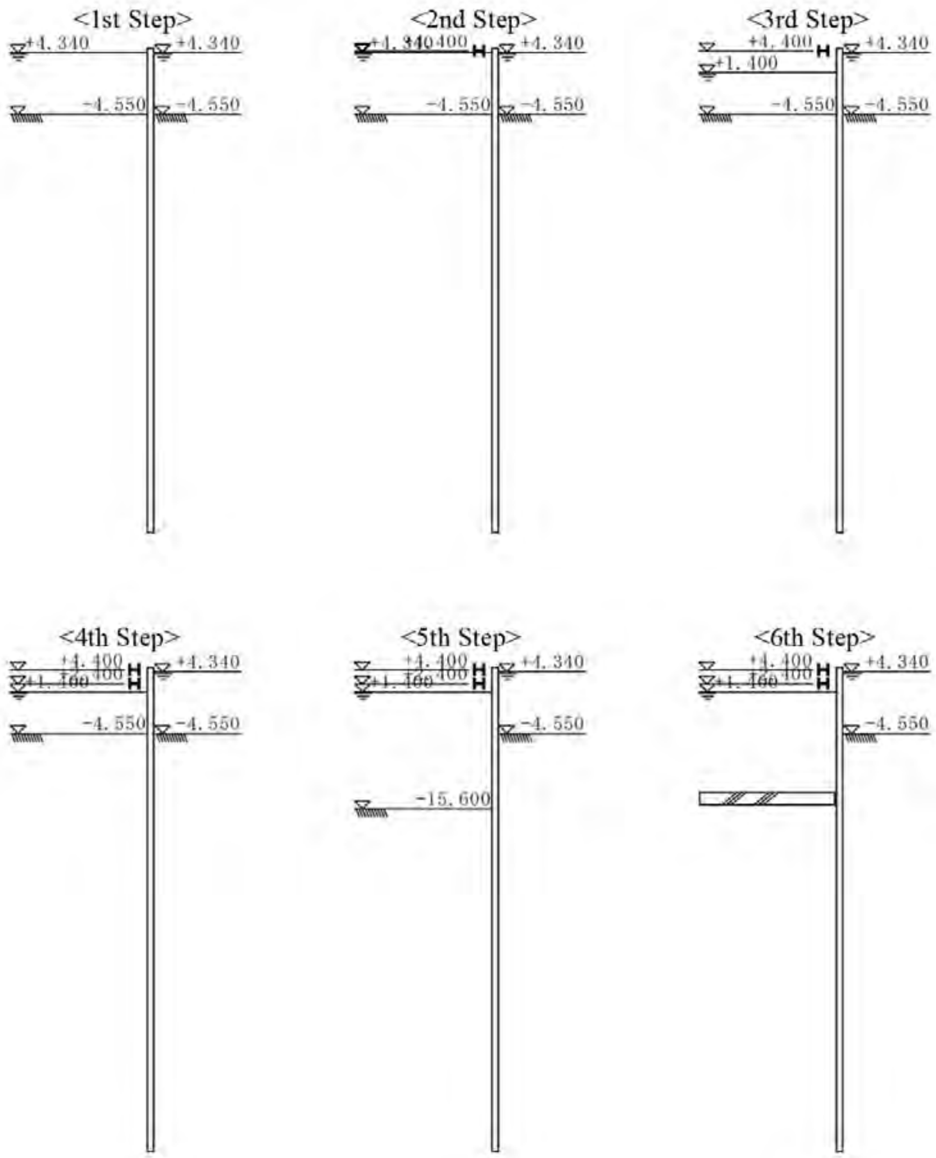
Table 4.2.145 Evaluation Results for Foundation (Transverse Direction) - 3

Items		Unit		Dynamic analysis Mmax
Acting force		Vo	kN	73209.1
		Ho	kN	23647.4
		Mo	kN.m	470199
Level crown of fundation	Displacement	δ1	cm	2.93
	Deflection angle	θ1	mrad	-1.815
Design ground surface	Displacement	δ2	cm	2.93
	Deflection angle	θ2	mrad	-1.815
Max bending moment of opening caisson		Mmax	kN.m	-627628
Location of Mmax		Lm	m	-19.1
Stress	Outer peripheral sheet pile (SKY400)	σmax	N/mm2	152.21
		Lm	m	-26.6
	Outer peripheral sheet pile (SKY490)	σmax	N/mm2	164.59
		Lm	m	-19.1
	Partitioned sheet pile (SKY400)	σmax	N/mm2	80.89
		Lm	m	-19.1
	Partitioned sheet pile (SKY490)	σmax	N/mm2	———
		Lm	m	———
	Pile (SKK400)	σmax	N/mm2	———
	Pile (SKK490)	σmax	N/mm2	———
Max bending moment of opening caisson at bottom		MB	kN.m	25507
Vertical reaction force	Maximum	Rmax	kN/pile	1822
	Minimum	Rmin	kN/pile	1506
Allowable value	Displacement	δa	cm	5
	Pushing bearing capacity	Ra	kN/pile	5839
	Pulling-out bearing capasity	Pa	kN/pile	-3344
	Stress (SKY400)	σa	N/mm2	210
	Stress (SKY490)	σa	N/mm2	277.5

Source: JICA Study Team

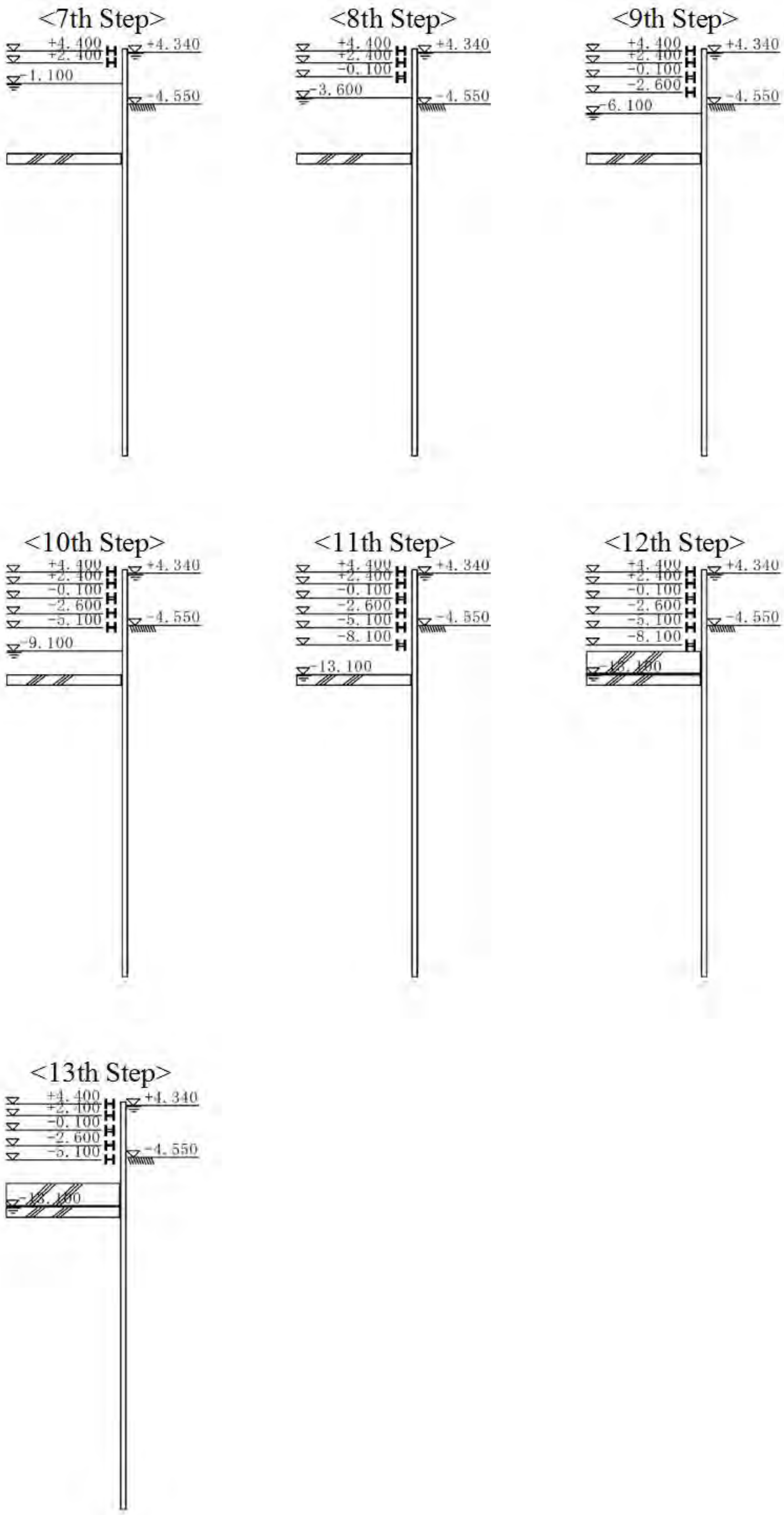
7) Temporary Coffering Calculation (Considering Scour)

During the temporary coffering, as the stress the steel pipe sheet pile is subjected to is affected by the construction sequence, the stress is calculated for each construction stage.



Source: JICA Study Team

Figure 4.2.168 Construction Stage (1st – 6th Stage)



Source: JICA Study Team

Figure 4.2.169 Construction Stage (7^{th} – 13^{th} Stage)

Table 4.2.146 Temporary Coffering Calculation Results (Longitudinal Direction)

・Longitudinal Direction

Item		Unit	1st Step	2nd Step	3rd Step	4th Step	5th Step	6th Step	7th Step
Maximum Displacement		cm	0.165	0.165	2.203	2.203	7.505	4.920	8.615
Cofferdam Section	SKY400	N/mm2	0.63	0.63	62.99	62.99	102.15	83.19	158.40
	SKY490	N/mm2	1.02	1.02	61.41	61.41	136.42	100.07	182.23
Well Section	SKY400	N/mm2	0.04	0.04	5.83	5.83	58.00	31.02	51.56
	SKY490	N/mm2	1.02	1.02	26.91	26.91	137.65	100.07	180.59
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	210.00
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	280.00
Item		単位	8th Step	9th Step	10th Step	11th Step	12th Step	13th Step	
Maximum Displacement		cm	9.041	9.238	9.321	9.306	9.294	9.294	
Cofferdam Section	SKY400	N/mm2	175.13	172.06	168.21	168.82	169.43	169.43	
	SKY490	N/mm2	207.30	222.73	230.20	227.46	226.57	226.57	
Well Section	SKY400	N/mm2	45.19	41.44	38.31	34.98	35.05	35.05	
	SKY490	N/mm2	200.19	214.58	223.22	219.46	218.07	218.07	
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	

Source: JICA Study Team

Table 4.2.147 Temporary Coffering Calculation Results (Transverse Direction)

・Traverse Direction

Item		Unit	1st Step	2nd Step	3rd Step	4th Step	5th Step	6th Step	7th Step
Maximum Displacement		cm	0.165	0.165	2.218	2.218	7.533	4.945	8.657
Cofferdam Section	SKY400	N/mm2	0.63	0.63	62.79	62.79	102.09	83.08	158.37
	SKY490	N/mm2	1.02	1.02	61.18	61.18	136.30	99.88	182.18
Well Section	SKY400	N/mm2	0.04	0.04	5.82	5.82	58.12	31.06	51.71
	SKY490	N/mm2	1.02	1.02	26.62	26.62	137.51	99.88	180.54
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	210.00
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	280.00
Item		単位	8th Step	9th Step	10th Step	11th Step	12th Step	13th Step	
Maximum Displacement		cm	9.152	9.387	9.496	9.497	9.484	9.484	
Cofferdam Section	SKY400	N/mm2	177.40	175.03	170.72	170.40	170.94	170.94	
	SKY490	N/mm2	208.83	225.02	232.71	229.69	228.89	228.89	
Well Section	SKY400	N/mm2	45.68	42.26	39.52	36.64	36.71	36.71	
	SKY490	N/mm2	201.30	216.48	225.71	222.05	220.75	220.75	
Allowable Stress	SKY400	N/mm2	210.00	210.00	210.00	210.00	210.00	210.00	
	SKY490	N/mm2	280.00	280.00	280.00	280.00	280.00	280.00	

Source: JICA Study Team

8) Total Stress Calculation (Considering Scour)

The stress the steel pipe sheet pile is subjected to is to be evaluated as the total of the leftover stress from the construction stage and the design external force after completion.

Leftover stress + Design external force after completion ≦ Allowable stress of steel pipe sheet pile

Table 4.2.148 Temporary Coffering Calculation Results (Longitudinal Direction)

1) Material：SKY400

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-26.6	38.92	34.98	73.9	140
2	Regular scenario LWL[W]	-26.6	45.61	34.98	80.59	140
3	Temperature flux scenario HWL[W]	-26.6	40.36	34.98	75.34	161
4	Temperature flux scenario LWL[W]	-26.6	47.26	34.98	82.24	161
5	Wind scenario [W]	-26.6	38.45	34.98	73.43	175
6	Marine vessel impact scenario [W]	-26.6	56.25	34.98	91.23	210
7	Seismic scenario [W]	-26.6	117.39	34.98	152.37	210
8	Seismic scenario[Smax]	-26.6	123.77	34.98	158.75	210
9	Seismic scenario [Mmax]	-26.6	122.72	34.98	157.71	210

2) Material：SKY490

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-22.1	39.7	69.58	109.28	185
2	Regular scenario LWL[W]	-22.1	46.39	69.58	115.97	185
3	Temperature flux scenario HWL[W]	-22.1	41.36	69.58	110.94	212.75
4	Temperature flux scenario LWL[W]	-22.1	48.26	69.58	117.84	212.75
5	Wind scenario [W]	-22.1	39.05	69.58	108.63	231.25
6	Marine vessel impact scenario [W]	-22.1	59.08	69.58	128.66	277.5
7	Seismic scenario [W]	-22.1	129.52	69.58	199.1	277.5
8	Seismic scenario[Smax]	-22.1	136.66	69.58	206.24	277.5
9	Seismic scenario [Mmax]	-22.1	136.26	69.58	205.84	277.5

Source: JICA Study Team

Table 4.2.149 Temporary Coffering Calculation Results (Transverse Direction)

1) Material：SKY400

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-26.6	37.78	36.64	74.43	140
2	Regular scenario LWL[W]	-26.6	44.47	36.64	81.12	140
3	Wind scenario [W]	-26.6	39.54	36.64	76.19	175
4	Marine vessel impact scenario [W]	-26.6	75.75	36.64	112.4	210
5	Seismic scenario [W]	-26.6	121.17	36.64	157.81	210
6	Seismic scenario[Smax]	-26.6	155.41	36.64	192.05	210
7	Seismic scenario [Mmax]	-26.6	152.21	36.64	188.86	210

2) Material：SKY490

Case	Load Case	Occuring Position	σ1(N/mm^2)	σ2(N/mm^2)	σmax(N/mm^2)	σa(N/mm^2)
1	Regular scenario HWL[W]	-22.1	38.25	72.26	110.51	185
2	Regular scenario LWL[W]	-22.1	44.94	72.26	117.2	185
3	Wind scenario [W]	-22.1	39.98	72.26	112.24	231.25
4	Marine vessel impact scenario [W]	-22.1	78.52	72.26	150.78	277.5
5	Seismic scenario [W]	-22.1	129.04	72.26	201.3	277.5
6	Seismic scenario[Smax]	-22.1	165.01	72.26	237.27	277.5
7	Seismic scenario [Mmax]	-22.1	162.68	72.26	234.94	277.5

Source: JICA Study Team

4.2.11 Summary of Bridge Accessories Design

4.2.11.1 Design Calculation of Rocking Bearing and Bearing Support

(1) Design Conditions

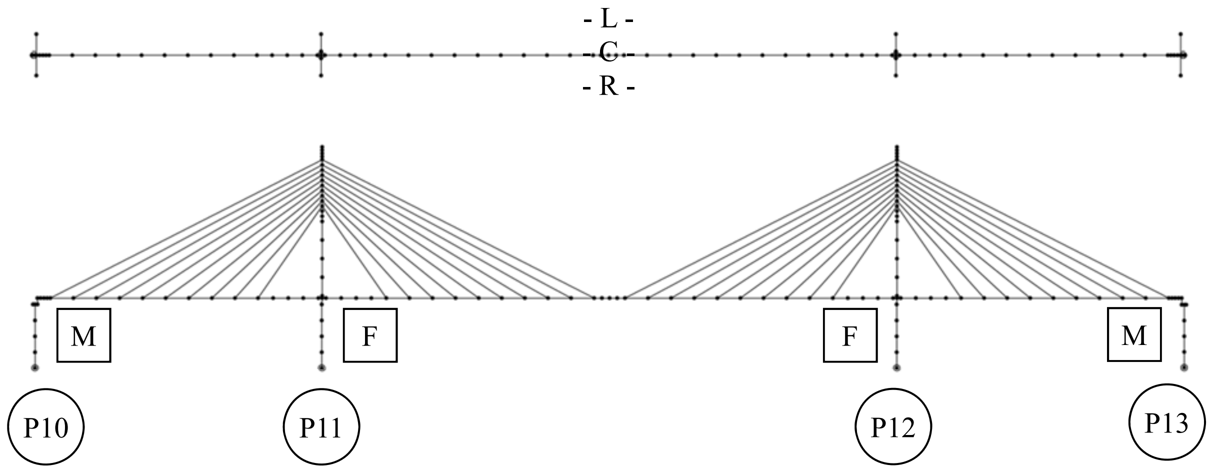
1) Support Conditions

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

Table 4.2.150 Condition of Support

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

Source: JICA Study Team



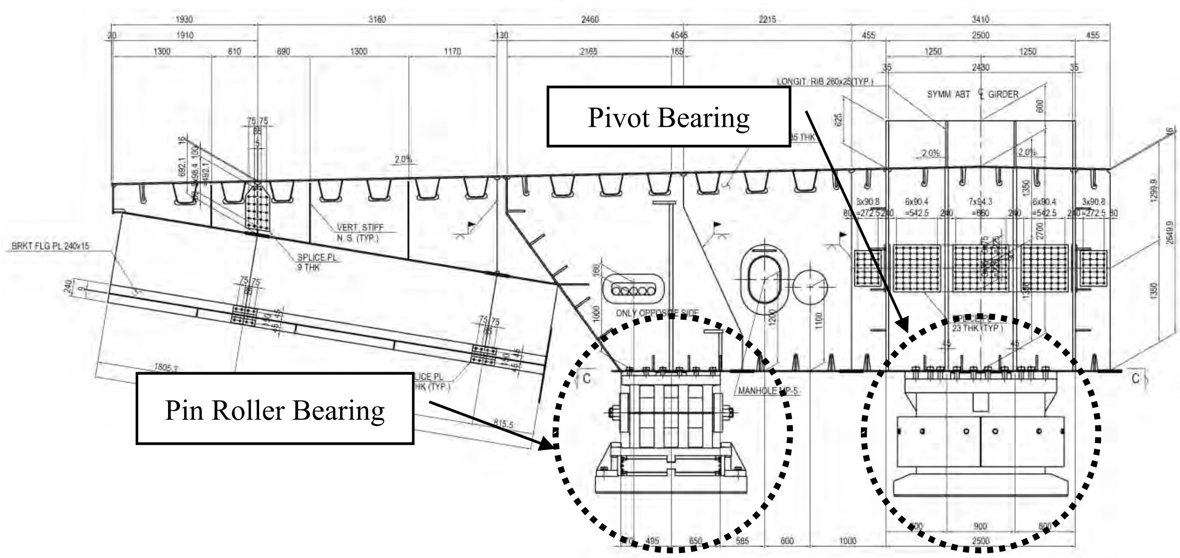
Source: JICA Study Team

Figure 4.2.170 Condition of Support

2) Structure of Bearings

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

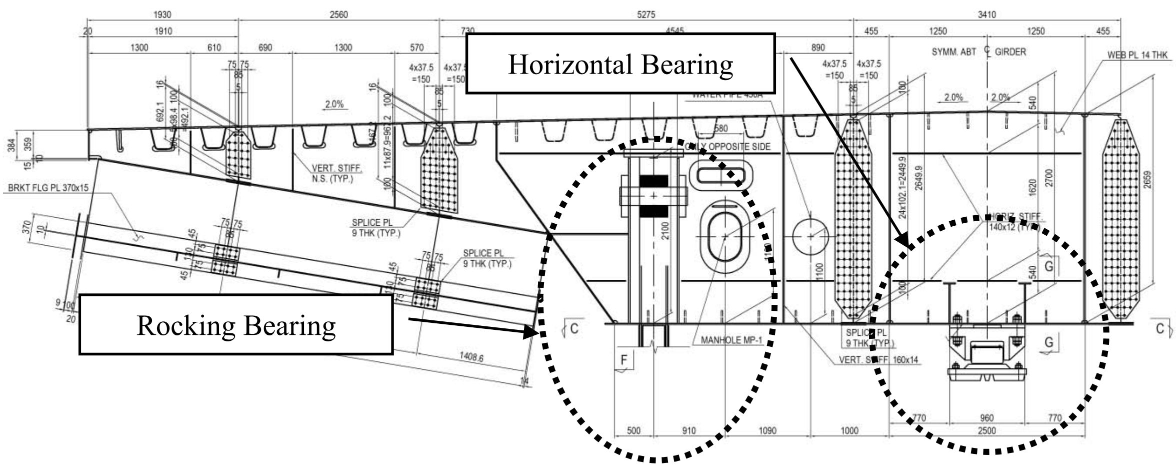
a) Support Section Underneath Main Tower



Source: JICA Study Team

Figure 4.2.171 Bearing Support under the Main Tower

b) Bearing at Ends



Source: JICA Study Team

Figure 4.2.172 Bearing Support at Girder End

3) Design Reaction Force of Bearing Support Section

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

Table 4.2.151 Reaction Forces at Support

				Cable-Stayed Bridge						Remarks
				End Supports (P10･P13)			Center Supports (P11･P12)			
				Rocking		Horizontal Bearing	Pin-Roller		Pivot	
				L	R	C	L	R	C	
Bearing Restriction Condition	Longitudinal			Movable		Movable	Movable		Fixed	
	Transverse			Movable		Fixed	Movable		Fixed	
	Vertical			Fixed		Movable	Fixed		Fixed	
Dead Load Scenario Reaction Force	Longitudinal		kN	—	—	—	—	—	1000	Per Bearing
	Transverse		kN	—	—	0	0	0	0	100KN Round Up
	Vertical		kN	100	100	—	12400	12400	46200	
Regular Scenario Reaction Force	Longitudinal	|max|	kN	—	—	—	—	—	4800	Per Bearing
	Transverse	|max|	kN	—	—	100	0	0	100	100KN Round Up
	Vertical	max	kN	3100	3100	—	20800	20800	57700	
		min	kN	-1800	-1800	—	7300	7300	44900	
Temperature Flux Scenario Reaction Force	Longitudinal	|max|	kN	—	—	—	—	—	9200	Per Bearing
	Transverse	|max|	kN	—	—	100	0	0	100	100KN Round Up
	Vertical	max	kN	3200	3200	—	20900	20900	58000	
		min	kN	-1900	-1900	—	7100	7100	44200	
Wind Scenario Reaction Force Transverse Direction	Longitudinal	|max|	kN	—	—	—	—	—	4800	Per Bearing
	Transverse	|max|	kN	—	—	200	0	0	2100	100KN Round Up
	Vertical	max	kN	3100	3100	—	22300	22300	57700	
		min	kN	-1900	-1900	—	5800	5800	44900	
Wind+Temperature Fux Reaction Force Transverse Direction	Longitudinal	|max|	kN	—	—	—	—	—	9200	Per Bearing
	Transverse	|max|	kN	—	—	200	0	0	2100	100KN Round Up
	Vertical	max	kN	3300	3300	—	22400	22400	58000	
		min	kN	-1900	-1900	—	5600	5600	44200	
Seismic Performance 1 Seismic Reaction Force Transverse Direction	Longitudinal	|max|	kN	—	—	—	—	—	1000	Per Bearing
	Transverse	|max|	kN	—	—	1600	3800	3800	13700	100KN Round Up
	Vertical	max	kN	700	700	—	17500	17500	46200	kh=0.30
		min	kN	-500	-500	—	7400	7400	46200	
Seismic Performance 2 Seismic Reaction Force Longitudinal Direction	Longitudinal	|max|	kN	—	—	—	5600	5600	25900	Per Bearing
	Transverse	|max|	kN	—	—	0	0	0	0	100KN Round Up
	Vertical	max	kN	500	500	—	12500	12500	46400	kh=0.45
		min	kN	-300	-300	—	12400	12400	46000	
Seismic Performance 2 Seismic Reaction Force Transverse Direction	Longitudinal	|max|	kN	—	—	—	—	—	1000	Per Bearing
	Transverse	|max|	kN	—	—	6700	5600	5600	20500	100KN Round Up
	Vertical	max	kN	1000	1000	—	20000	20000	46200	kh=0.45
		min	kN	-800	-800	—	4900	4900	46200	
Movement Amount	Temperature Flux Sceneario		mm	68.0	68.0	68.0	—	—	—	About 25°C
	Wind Scenario Longitudinal Direction		mm	—	—	—	—	—	—	
	Seismic Performance 1 Longitudinal Direction		mm	55.8	55.8	55.8	—	—	—	
	Seismic Performance 2 Longitudinal Direction		mm	83.8	83.8	83.8	—	—	—	
Beam Rotation			rad	1/140	1/140	—	1/230	1/230	1/230	1/10 Round Down
Negative Reaction Force Evaluation	Dead Load	Rd	kN	100	100	—	12400	12400	46200	Per Bearing
	Live Load	Rl(min)	kN	-1900	-1900	—	-5100	-5100	-1300	100KN Round Up
	Negative Reaction Force	Rd+2×Rl	kN	-3700	-3700	—	2200	2200	43600	
	Decision: Negative Reaction Force Countermeasure			Necessary	Necessary	—	Not Necessary	Not Necessary	Not Necessary	

Source: JICA Study Team

(2) Design of Pivot Bearing

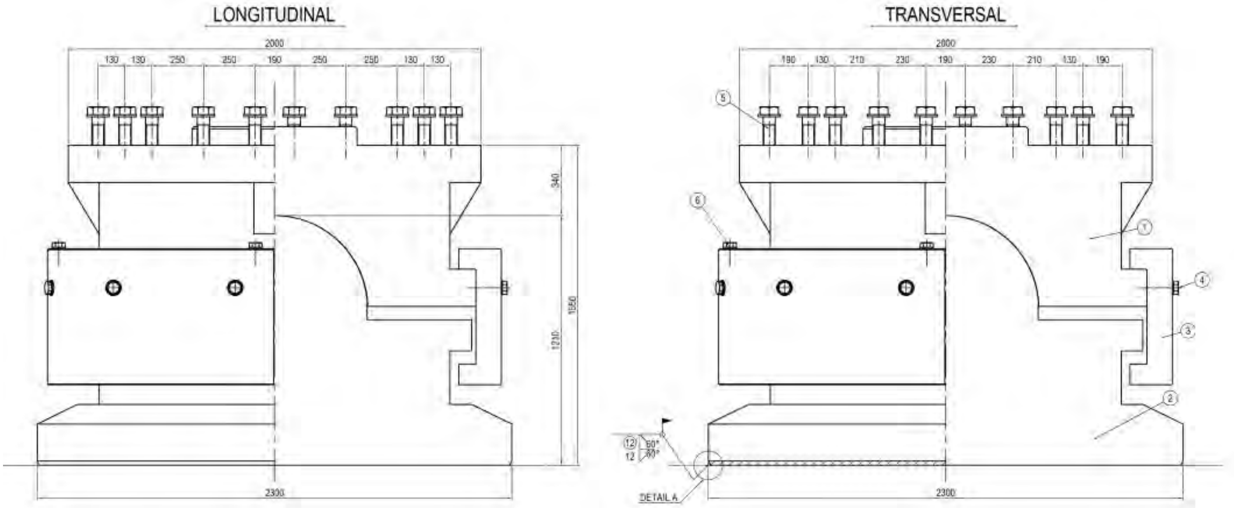
The results of the pivot bearing design are listed below.

Table 4.2.152 Design Calculation Results

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

※Refer to the next page for cross-section position

Source: JICA Study Team



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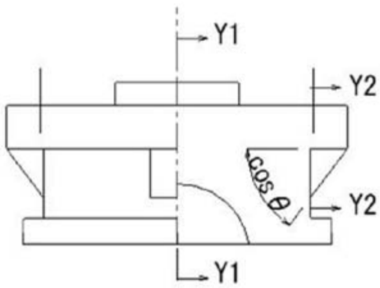

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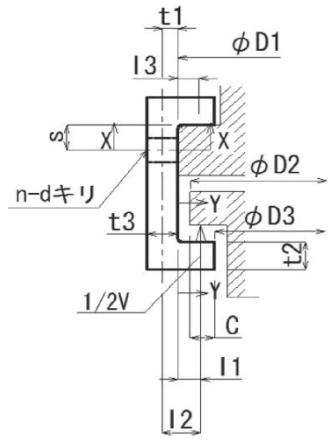


Source: JICA Study Team

Figure 4.2.173 Pivot Bearing Overview and Cross Section Location

(3) Design of Pin Roller Bearing

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

Table 4.2.153 Design Calculation Results - 1

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

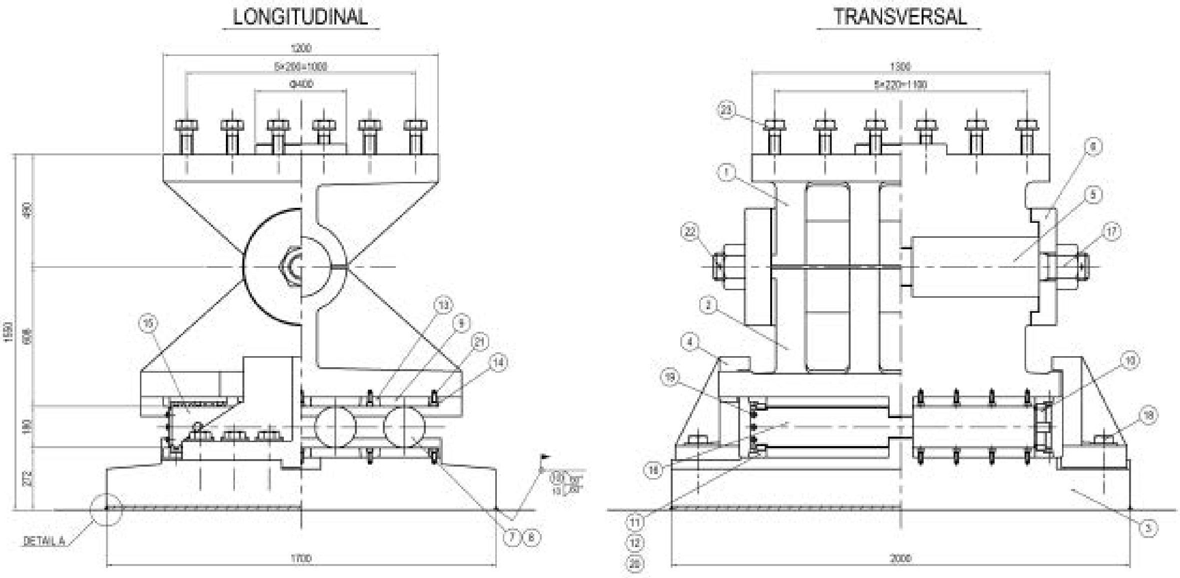
Source: JICA Study Team

Table 4.2.154 Design Calculation Results - 2

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

※See the next page for the cross-section position

Source: JICA Study Team



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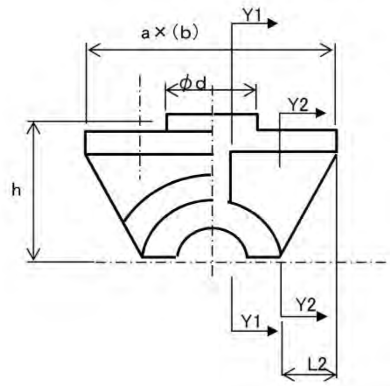

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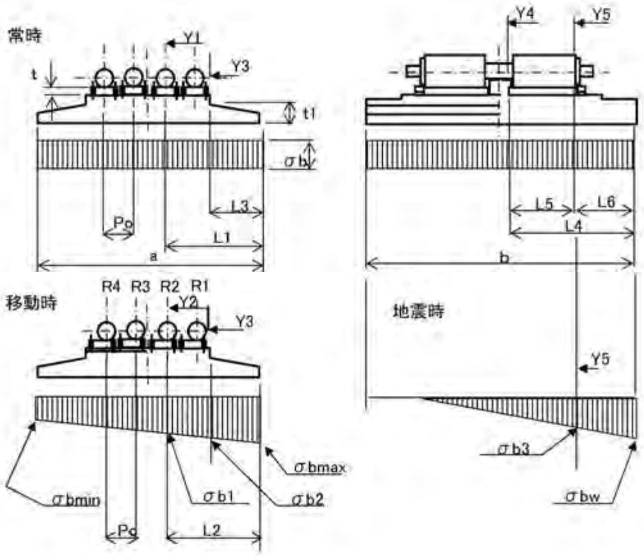

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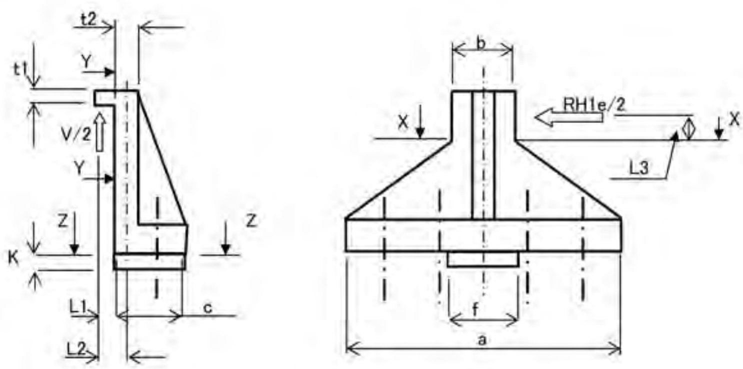

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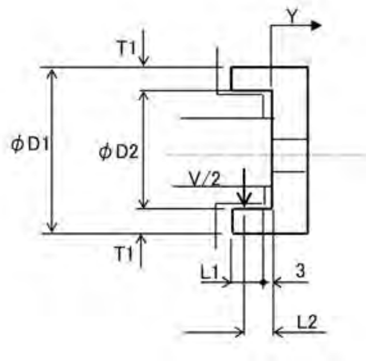


Source: JICA Study Team

Figure 4.2.174 Pin Roller Bearing Overview and Cross Section Location

(4) Design of Horizontal Bearing

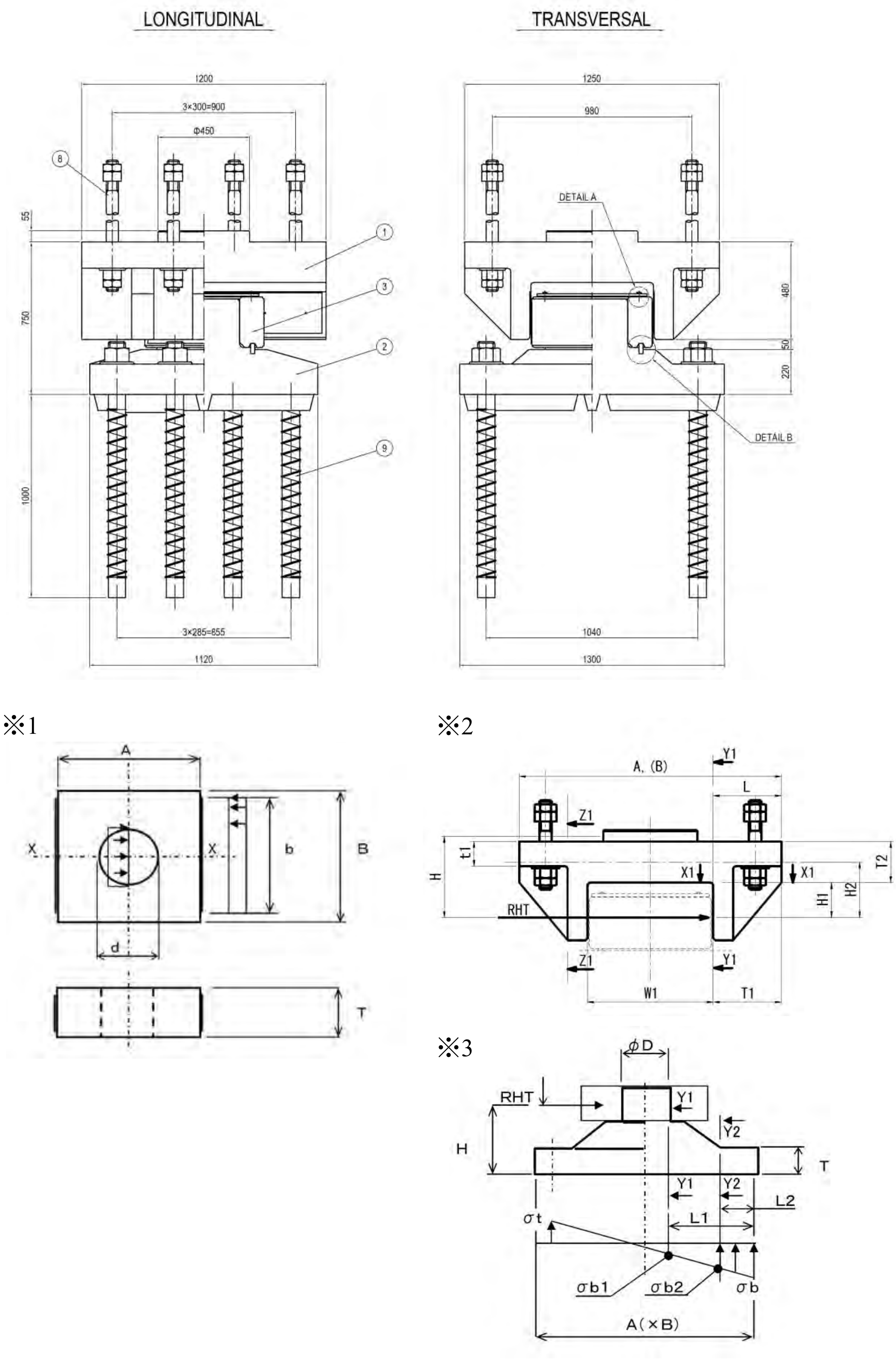
The results of the horizontal bearing design are listed below.

Table 4.2.155 Design Calculation Results

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

※See the next page for the cross-section position

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.175 Horizontal Bearing Overview and Cross Section Location

(5) Design of Rocking Bearing

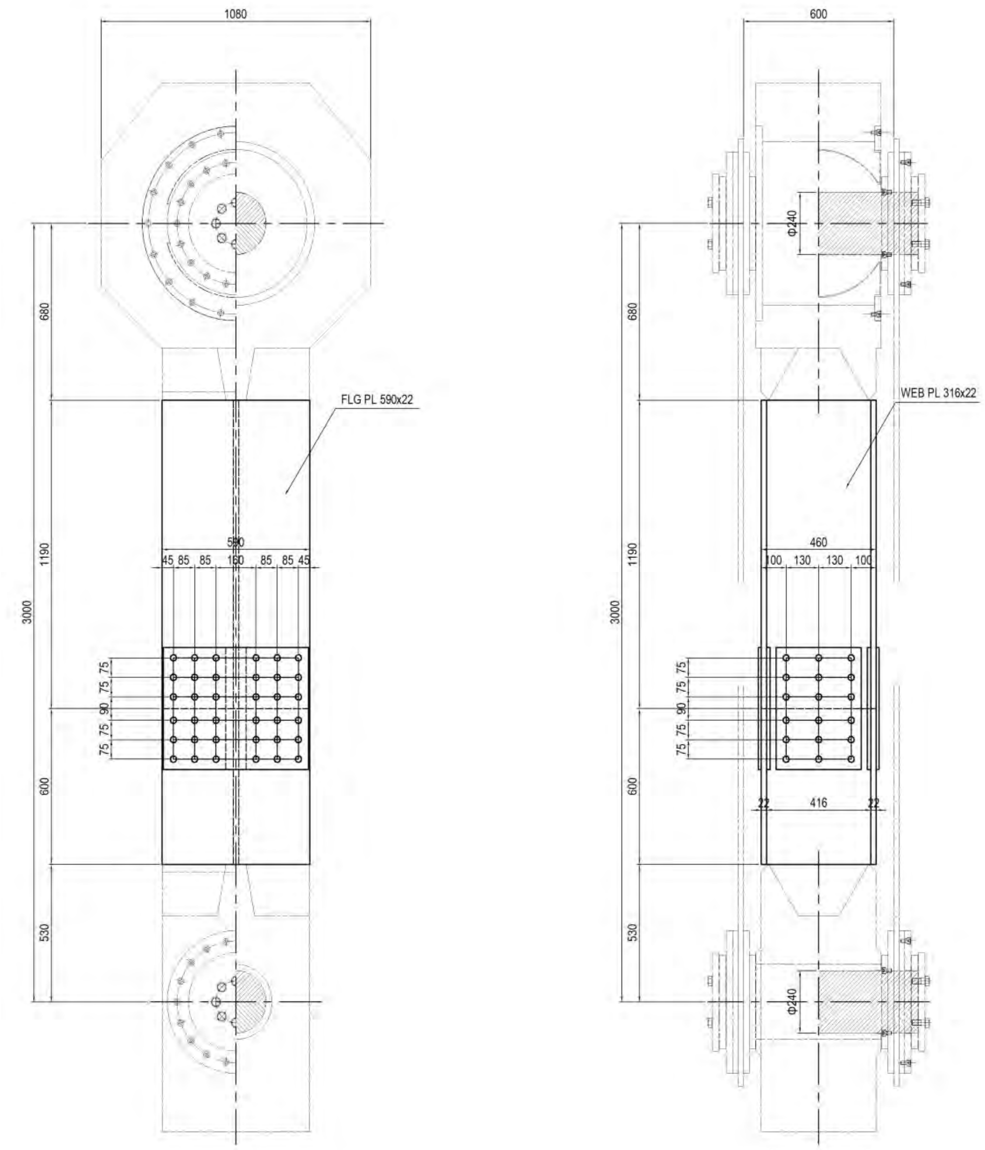
The results of the rocking bearing design are listed below.

Table 4.2.156 Design Calculation Results

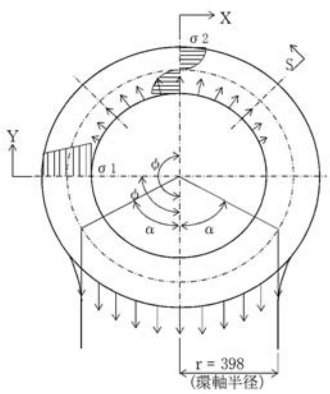
Category				Units	Value		Allowable Value
Endlink	Spherical Surface Bearing	Bearing Pressure（Internal Diameter）		N/mm2	41	<	50
	Spherical Bush Bearing	Maximum Bearing Stress at Center Cross Section		N/mm2	23	<	50
		Tensile Stress		N/mm2	55	<	290
	Pin	Bending Stress		N/mm2	262	<	290
		Shear Stress		N/mm2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

※See the next page for the cross-section position

Source: JICA Study Team



※1



Source: JICA Study Team

Figure 4.2.176 Rocking Bearing Overview and Cross Section Location

4.2.11.2 Study on Cable Damping Device

(1) Design Overview

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

1) Vortex induced vibration

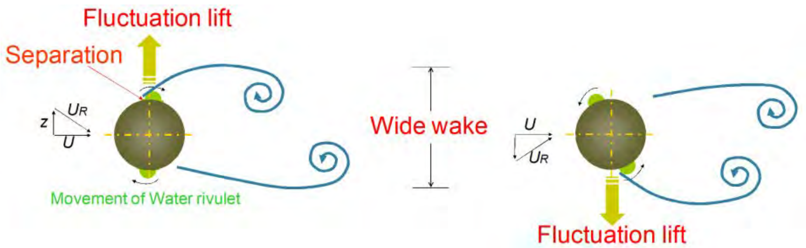
With the exposure to constant winds, a Karman vortex occurs behind the cables which vibrates at the same natural frequency as the cable. The vibration is highly unlikely to be vibrating at a natural frequency in the primary mode, but generally found to be in a much higher mode. Because the vibrational energy is comparatively low, a logarithmic damping coefficient of about 0.01 (about 60 when expressed as a Scruton number) is said to be able to mitigate the vibration.

Source: JICA Study Team

Figure 4.2.177 Karman Vortex Schematics

2) Rain-wind induced vibration

With the exposure to winds during rain, the cable becomes hydrodynamically unstable due to the formation of a waterway on the cable causing vibration to be more easily generated. The vibration, occurring at a relatively low mode, has a larger swing compared to the vortex induced vibration. The vibrational energy is larger than that created by the vortex induced vibration, and therefore, a logarithmic damping coefficient of more than 0.02~0.03 (about 120~200 when expressed as a Scruton number) is needed to mitigate most of the vibration.



Source: JICA Study Team

Figure 4.2.178 Rain-Wind Induced Vibration Schematics

(2) Review for the Necessity of Vibration Countermeasure

In this design, the necessity for vibration countermeasure was determined by focusing the attention on the rain-wind induced vibration which needs a high additional damping factor. As a condition to mitigate the rain-wind induced vibration, the two points below must be satisfied. Therefore, a review of the necessity of vibration countermeasures shall be based on natural frequency and Scruton number.

<Conditions for mitigating rain-wind induced vibration>

The natural frequency of the cable must be 3 Hz or higher.

$$f_n = \frac{n}{2L}\sqrt{\frac{T}{m}} \qquad \geqq 3$$

The Scruton number (a dimensionless quantity expressing the ease to vibrate) must be higher than 120~200.

$$S_C = \frac{2m\delta}{\rho D^2} \qquad \geqq 120$$

The results of the review of the necessity of vibration countermeasures are shown in the table below. As the natural frequency and Scruton number do not satisfy the condition for mitigating rain-wind induced vibration for all cables, the installation of dampers (strong damping rubber damper) is necessary.

Table 4.2.157 Results for the Necessity of Vibration Countermeasure

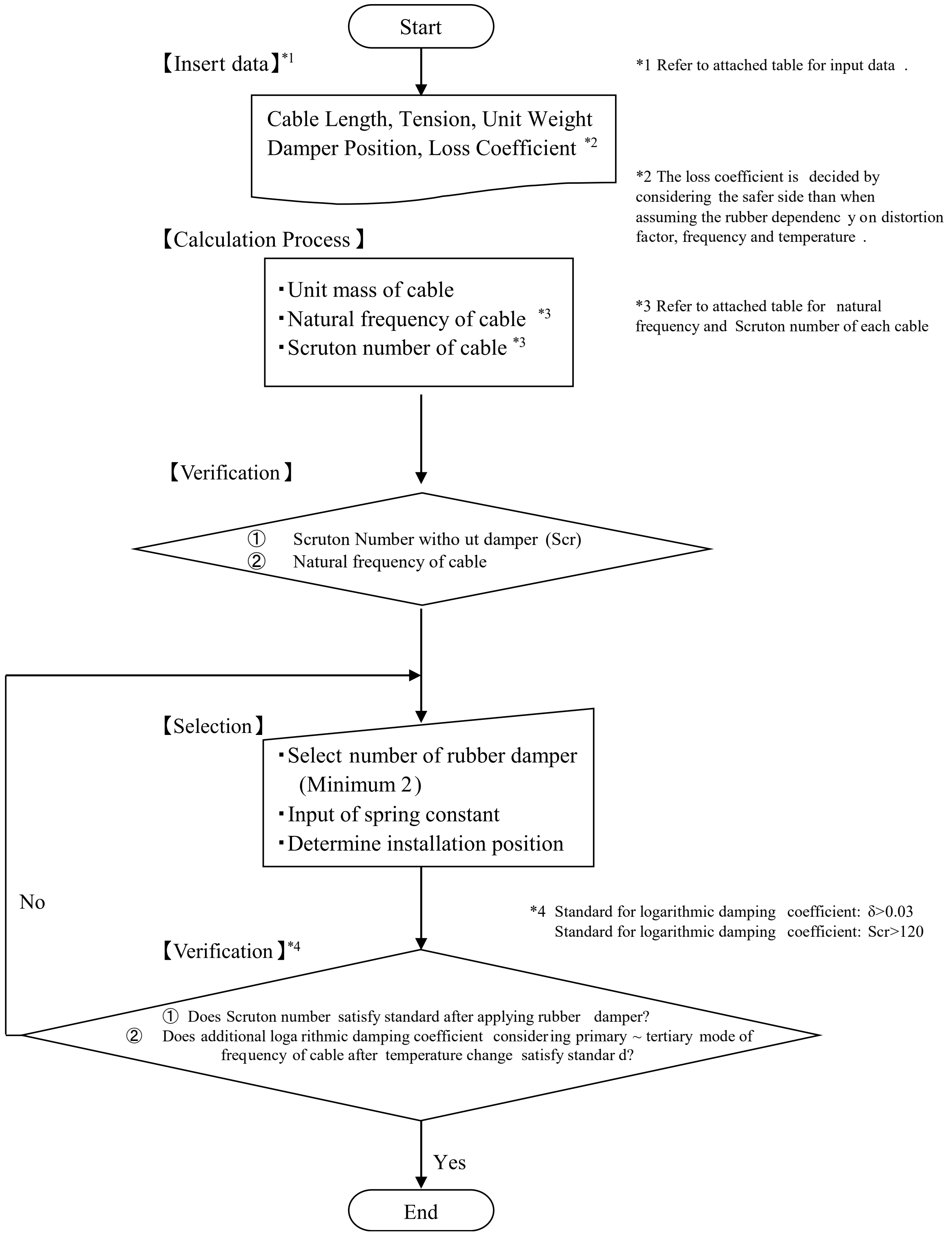
	Cable number	Cable tension T (kN)	Mass per unit length m	Cable length	Structural damping δ	Air density ρ (kg/m3)	Cable Diameter D (m)	Natural frequency of cable f			Scruton number Sc
								Primary f1(Hz)	Secondary f2(Hz)	Tertiary f3(Hz)	
Side Span Side (Top→Bottom)	C1	4400	90.2	118.917	0.005	1.293	0.190	0.929	1.857	2.786	19.324
	C2	4100	90.2	110.012	0.005	1.293	0.190	0.969	1.938	2.907	19.324
	C3	3900	90.2	101.163	0.005	1.293	0.190	1.028	2.055	3.083	19.324
	C4	3900	90.2	92.387	0.005	1.293	0.190	1.125	2.251	3.376	19.324
	C5	4100	90.2	83.707	0.005	1.293	0.190	1.273	2.547	3.820	19.324
	C6	2500	47.7	75.155	0.005	1.293	0.133	1.523	3.046	4.569	20.855
	C7	2700	47.7	66.780	0.005	1.293	0.133	1.781	3.563	5.344	20.855
	C8	2900	47.7	58.660	0.005	1.293	0.133	2.102	4.203	6.305	20.855
	C9	2900	47.7	50.916	0.005	1.293	0.133	2.421	4.843	7.264	20.855
	C10	2900	47.7	43.747	0.005	1.293	0.133	2.818	5.636	8.454	20.855
Center Span Side (Bottom→Top)	C11	2900	47.7	43.747	0.005	1.293	0.133	2.818	5.636	8.454	20.855
	C12	2900	47.7	50.916	0.005	1.293	0.133	2.421	4.843	7.264	20.855
	C13	2900	47.7	58.660	0.005	1.293	0.133	2.102	4.203	6.305	20.855
	C14	2700	47.7	66.780	0.005	1.293	0.133	1.781	3.563	5.344	20.855
	C15	2500	47.7	75.155	0.005	1.293	0.133	1.523	3.046	4.569	20.855
	C16	4100	90.2	83.707	0.005	1.293	0.190	1.273	2.547	3.820	19.324
	C17	3900	90.2	92.387	0.005	1.293	0.190	1.125	2.251	3.376	19.324
	C18	3900	90.2	101.163	0.005	1.293	0.190	1.028	2.055	3.083	19.324
	C19	4100	90.2	110.012	0.005	1.293	0.190	0.969	1.938	2.907	19.324
	C20	4400	90.2	118.917	0.005	1.293	0.190	0.929	1.857	2.786	19.324

Source: JICA Study Team

(3) Design Method

1) Study Flow

The flow for the study of the cable mitigation apparatus is shown below.

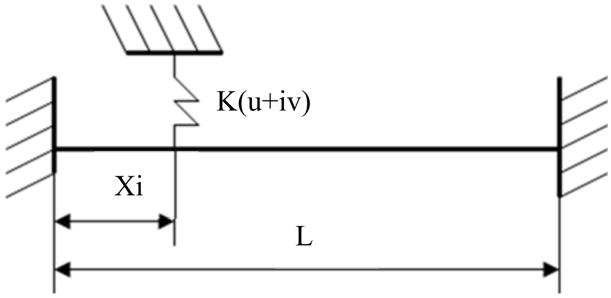


Source: JICA Study Team

Figure 4.2.179 Flow for the Study of Cable Mitigation Apparatus

2) Analysis Model

The analysis model is shown below.



Source: JICA Study Team

Figure 4.2.180 Analysis Model of Cable Mitigation Apparatus

a) Input Conditions

- Cable length : L[m]
- Tension : T[kgf]
- Unit weight : W[kgf/m]
- Installation position : Xi[m]
- Loss coefficient : γ [= tan δ]

b) Calculations

- Unit mass of cable $\mu = \frac{W}{g} \quad [kg/m]$

- Reduced mass of cable : $M = \frac{1}{2} \times \mu \times L \quad [kg]$

- nth angular frequency $\omega_n = \frac{n\pi}{L} \times \sqrt{\frac{T}{\mu}} \quad [rad/\sec]$

- Stiffness of shear modulus : $K = \frac{G \times A}{t} \quad [kg/m]$

Where, G : Shear modulus of internal layer of rubber [kg/m^2]

A : Cross sectional area of internal layer of rubber [m^2]

t : Height of internal layer of rubber [m]

- Mode function of mitigation apparatus position: $\phi_i(X_i) = \sin(n\pi \frac{X_i}{L})$

- Imaginary component of complex stiffness : $v = \frac{\gamma}{\sqrt{1+\gamma^2}}$

- nth natural frequency of cable $f_n = \frac{\omega_n}{2\pi}$

- Logarithmic damping coefficient : $\delta = \frac{2\pi[\phi_1(X_i)]^2}{2M\omega_n} \frac{Kv}{\omega_n} e^{-0.72 \frac{\omega_n x_i (L-x_i)}{TL} \frac{Kv}{\omega_n} \frac{2}{\gamma}}$

c) Design Constants

- Loss coefficient of rubber

Considering the design value to be on the safer side by assuming the rubber dependency on distortion factor, frequency, and temperature, it shall be set as follows:

$\tan\delta = 0.63$ (20 ℃, 40 ℃)

$\tan\delta = 0.76$ (0 ℃)

- Logarithmic damping coefficient of cable

$\delta C = 0.005$ is set as the design value.

(4) Design of Mitigation Apparatus

1) Study of Mitigation Effect

By calculating the natural frequency of the cable, the Scruton number, and logarithmic damping coefficient with and without the mitigation apparatus, the effect of the mitigation is verified. The number of rubber dampers in the mitigation apparatus is examined to satisfy the condition below to consider the effect of temperature change on the elastic spring constant.

1) The logarithmic damping coefficient $\delta_{(C+D)}$ of low mode of frequency satisfies **$\boldsymbol{\delta_{(C+D)} > 0.03}$**

2) The addition of rubber damper satisfies **Scruton Number > 120**

2) Study Result

1) When the mitigation apparatus is installed, if the logarithmic damping coefficient for the primary ~ tertiary mode of vibration is higher than 0.03 for all cables, the countermeasure for vortex induced vibration and rain-wind induced vibration is valid.

2) When the mitigation apparatus is installed, if the Scruton number is higher than 120 for all cables, the countermeasure for vortex induced vibration and rain-wind induced vibration is valid.

The calculation results for the natural frequency, logarithmic damping coefficient for all temperatures, and Scruton number for each cable condition are shown below.

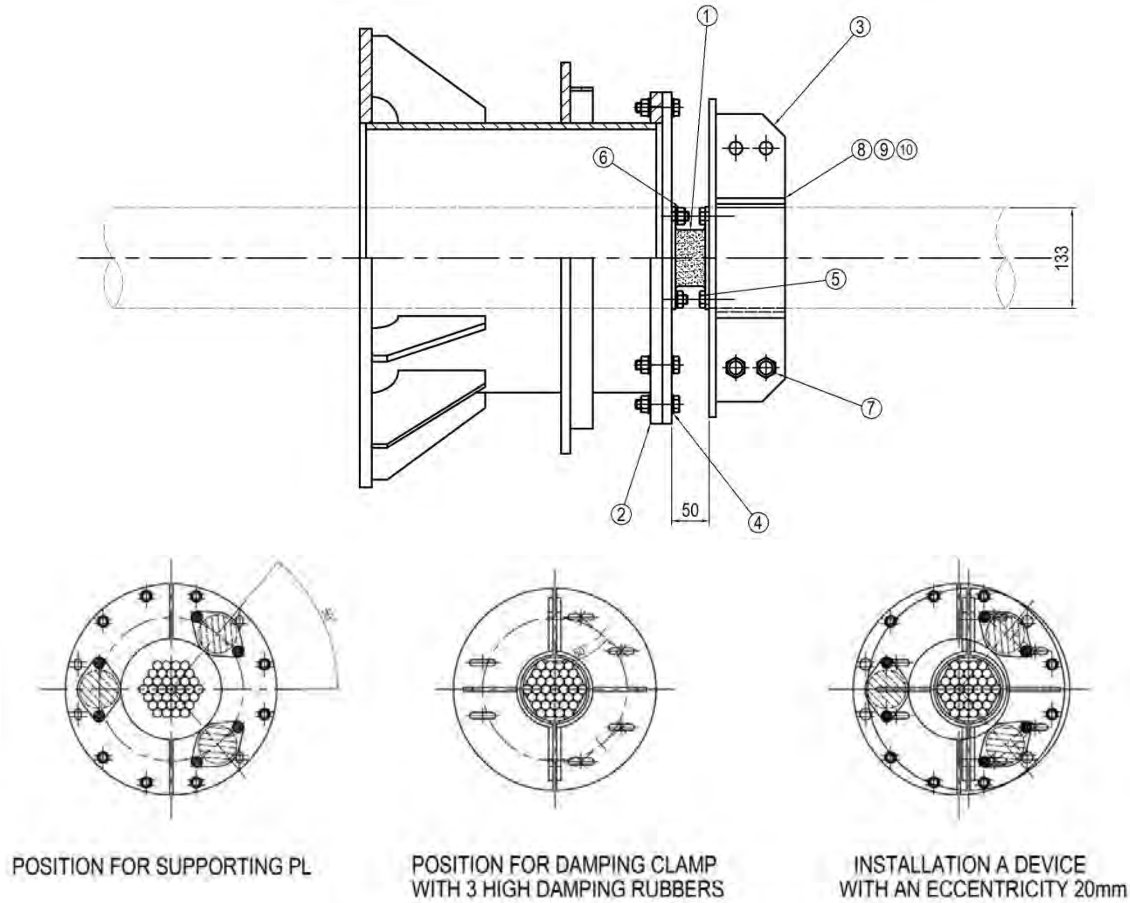
Table 4.2.158 Study Results for Mitigation Apparatus

			Cable Dimensions: Length L (m)	Tension T (kN)	Outer Perimeter Dc (mm)	Unit weight w (kg/m)	Damper Installation Position Xi (m)	Logarithmic Damping Coefficient (20℃) $\delta_{(C+D)}$ Primary	Secondary	Tertiary	Scruton Number Sc (C+D)	Logarithmic Damping Coefficient (40℃) $\delta_{(C+D)}$ Primary	Secondary	Tertiary	Scruton Number Sc (C+D)	Logarithmic Damping Coefficient (0℃) $\delta_{(C+D)}$ Primary	Secondary	Tertiary	Scruton Number Sc (C+D)
P11	P11 Main column side span side (Top→Bottom)	C1	119.294	4400	190	90.2	4.44	0.0392	0.0387	0.0379	151	0.0336	0.0333	0.0326	130	0.0518	0.0512	0.0501	200
		C2	110.457	4100	190	90.2	4.31	0.0414	0.0409	0.0400	160	0.0357	0.0353	0.0345	138	0.0542	0.0535	0.0523	210
		C3	101.666	3900	190	90.2	4.17	0.0435	0.0429	0.0419	168	0.0376	0.0371	0.0362	145	0.0568	0.0560	0.0546	220
		C4	92.924	3900	190	90.2	4.01	0.0503	0.0495	0.0482	195	0.0481	0.0473	0.0460	186	0.0528	0.0520	0.0505	204
		C5	84.251	4100	190	90.2	3.84	0.0528	0.0518	0.0502	204	0.0493	0.0484	0.0469	190	0.0583	0.0572	0.0554	225
		C6	75.573	2500	133	47.7	3.55	0.0516	0.0506	0.0490	215	0.0545	0.0534	0.0517	227	0.0442	0.0434	0.0420	184
		C7	67.203	2700	133	47.7	3.36	0.0568	0.0555	0.0535	237	0.0573	0.0560	0.0539	239	0.0534	0.0522	0.0503	223
		C8	59.094	2900	133	47.7	3.17	0.0617	0.0601	0.0575	257	0.0597	0.0582	0.0557	249	0.0630	0.0614	0.0587	263
		C9	51.372	2900	133	47.7	2.98	0.0665	0.0645	0.0612	277	0.0632	0.0613	0.0582	263	0.0706	0.0684	0.0650	294
		C10	44.233	2900	133	47.7	2.78	0.0718	0.0692	0.0651	300	0.0670	0.0646	0.0608	279	0.0792	0.0764	0.0718	331
	P1 Main column center span side (Bottom→Top)	C11	44.177	2900	133	47.7	2.79	0.0719	0.0694	0.0652	300	0.0671	0.0647	0.0608	280	0.0794	0.0765	0.0719	331
		C12	51.303	2900	133	47.7	2.98	0.0666	0.0646	0.0613	278	0.0633	0.0614	0.0583	264	0.0707	0.0685	0.0651	295
		C13	59.011	2900	133	47.7	3.17	0.0619	0.0602	0.0576	258	0.0599	0.0583	0.0558	250	0.0631	0.0615	0.0588	263
		C14	67.105	2700	133	47.7	3.37	0.0570	0.0557	0.0536	238	0.0574	0.0561	0.0540	240	0.0535	0.0523	0.0503	223
		C15	75.467	2500	133	47.7	3.56	0.0517	0.0507	0.0491	216	0.0546	0.0535	0.0518	228	0.0443	0.0434	0.0420	185
		C16	84.135	4100	190	90.2	3.85	0.0530	0.0520	0.0504	205	0.0494	0.0485	0.0470	191	0.0584	0.0573	0.0555	226
		C17	92.803	3900	190	90.2	4.02	0.0505	0.0497	0.0483	195	0.0483	0.0475	0.0462	187	0.0529	0.0521	0.0506	205
		C18	101.554	3900	190	90.2	4.18	0.0438	0.0431	0.0421	169	0.0378	0.0372	0.0364	146	0.0570	0.0562	0.0548	220
		C19	110.364	4100	190	90.2	4.33	0.0417	0.0411	0.0402	161	0.0360	0.0355	0.0347	139	0.0545	0.0537	0.0525	211
		C20	119.222	4400	190	90.2	4.47	0.0395	0.0390	0.0382	153	0.0339	0.0335	0.0329	131	0.0521	0.0515	0.0504	201
P12	P12 Main column side span side (Top→Bottom)	C1	119.294	4400	190	90.2	4.44	0.0392	0.0387	0.0379	151	0.0336	0.0333	0.0326	130	0.0518	0.0512	0.0501	200
		C2	110.457	4100	190	90.2	4.31	0.0414	0.0409	0.0400	160	0.0357	0.0353	0.0345	138	0.0542	0.0535	0.0523	210
		C3	101.666	3900	190	90.2	4.17	0.0435	0.0429	0.0419	168	0.0376	0.0371	0.0362	145	0.0568	0.0560	0.0546	220
		C4	92.924	3900	190	90.2	4.01	0.0503	0.0495	0.0482	195	0.0481	0.0473	0.0460	186	0.0528	0.0520	0.0505	204
		C5	84.251	4100	190	90.2	3.84	0.0528	0.0518	0.0502	204	0.0493	0.0484	0.0469	190	0.0583	0.0572	0.0554	225
		C6	75.573	2500	133	47.7	3.55	0.0516	0.0506	0.0490	215	0.0545	0.0534	0.0517	227	0.0442	0.0434	0.0420	184
		C7	67.203	2700	133	47.7	3.36	0.0568	0.0555	0.0535	237	0.0573	0.0560	0.0539	239	0.0534	0.0522	0.0503	223
		C8	59.094	2900	133	47.7	3.17	0.0617	0.0601	0.0575	257	0.0597	0.0582	0.0557	249	0.0630	0.0614	0.0587	263
		C9	51.372	2900	133	47.7	2.98	0.0665	0.0645	0.0612	277	0.0632	0.0613	0.0582	263	0.0706	0.0684	0.0650	294
		C10	44.233	2900	133	47.7	2.78	0.0718	0.0692	0.0651	300	0.0670	0.0646	0.0608	279	0.0792	0.0764	0.0718	331
	P12 Main column center span side (Bottom→Top)	C11	44.177	2900	133	47.7	2.79	0.0719	0.0694	0.0652	300	0.0671	0.0647	0.0608	280	0.0794	0.0765	0.0719	331
		C12	51.303	2900	133	47.7	2.98	0.0666	0.0646	0.0613	278	0.0633	0.0614	0.0583	264	0.0707	0.0685	0.0651	295
		C13	59.011	2900	133	47.7	3.17	0.0619	0.0602	0.0576	258	0.0599	0.0583	0.0558	250	0.0631	0.0615	0.0588	263
		C14	67.105	2700	133	47.7	3.37	0.0570	0.0557	0.0536	238	0.0574	0.0561	0.0540	240	0.0535	0.0523	0.0503	223
		C15	75.467	2500	133	47.7	3.56	0.0517	0.0507	0.0491	216	0.0546	0.0535	0.0518	228	0.0443	0.0434	0.0420	185
		C16	84.135	4100	190	90.2	3.85	0.0530	0.0520	0.0504	205	0.0494	0.0485	0.0470	191	0.0584	0.0573	0.0555	226
		C17	92.803	3900	190	90.2	4.02	0.0505	0.0497	0.0483	195	0.0483	0.0475	0.0462	187	0.0529	0.0521	0.0506	205
		C18	101.554	3900	190	90.2	4.18	0.0438	0.0431	0.0421	169	0.0378	0.0372	0.0364	146	0.0570	0.0562	0.0548	220
		C19	110.364	4100	190	90.2	4.33	0.0417	0.0411	0.0402	161	0.0360	0.0355	0.0347	139	0.0545	0.0537	0.0525	211
		C20	119.222	4400	190	90.2	4.47	0.0395	0.0390	0.0382	153	0.0339	0.0335	0.0329	131	0.0521	0.0515	0.0504	201

Source: JICA Study Team

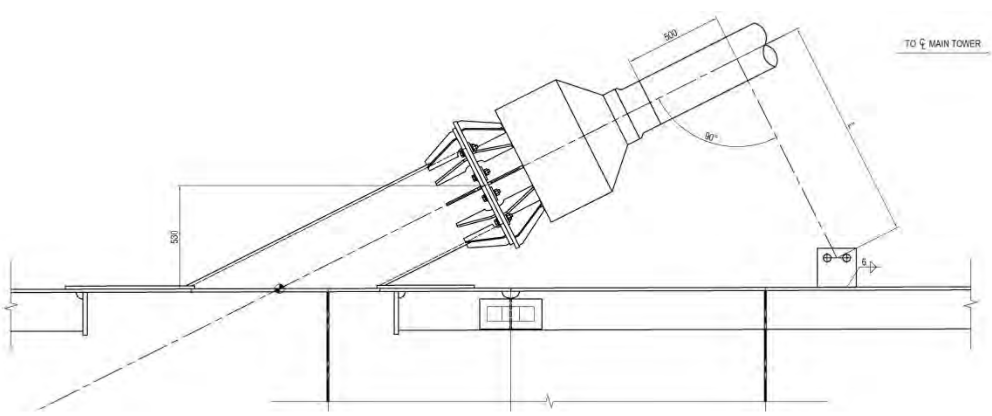
3) Installation of Mitigation Apparatus

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



Source: JICA Study Team

Figure 4.2.181 Installed Mitigation Apparatus



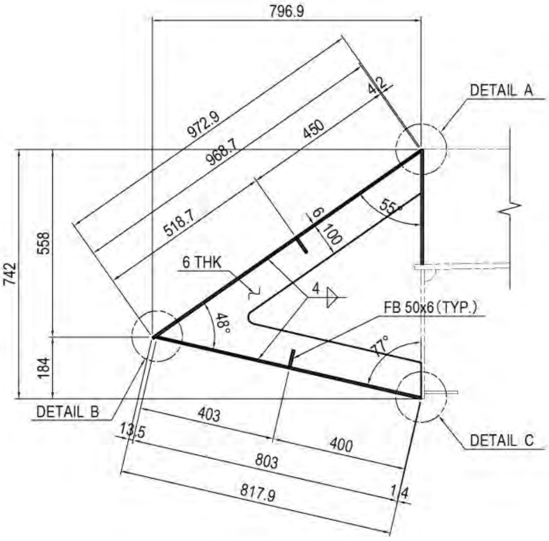
Source: JICA Study Team

Figure 4.2.182 Fitting Metal for Rod-type Mitigation Apparatus

4.2.11.3 Main Body Design of Fairing

(1) Fairing Shape

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



Source: JICA Study Team

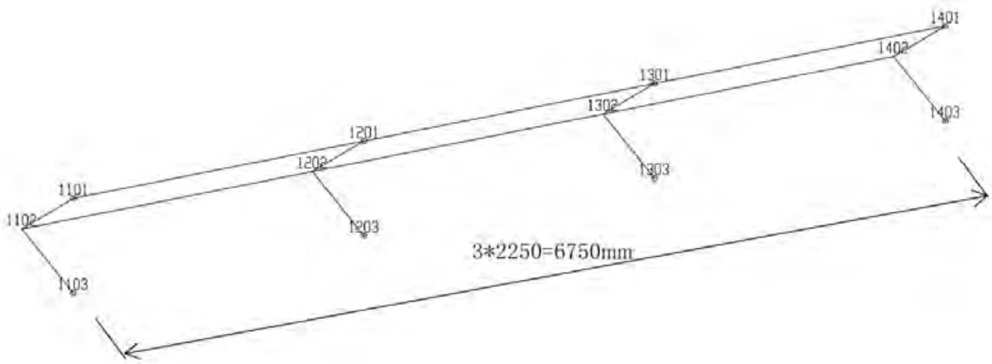
Figure 4.2.183 Fairing Shape

(2) Design Method

Design calculation is performed by applying wind and dead load.

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

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



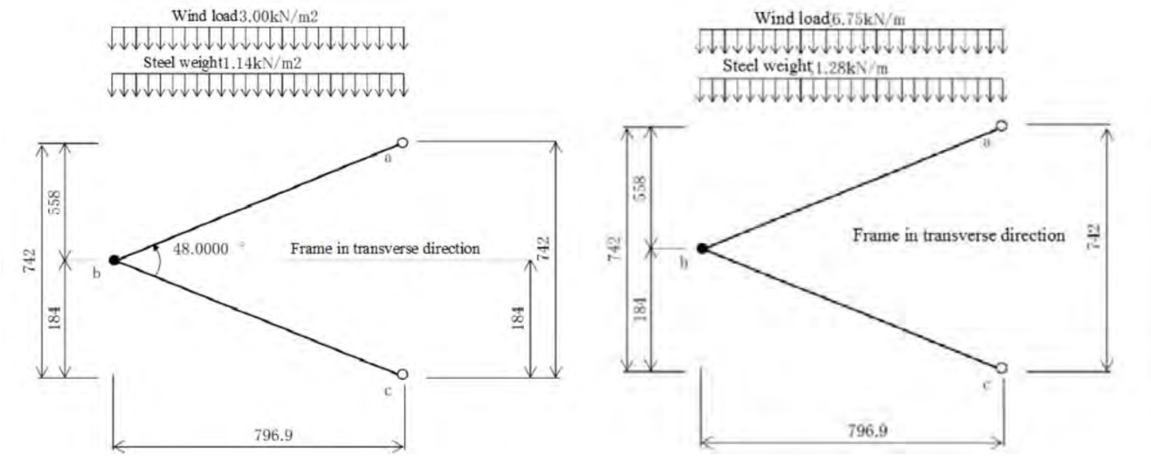
Note: The interval of frame panel is 2250 mm of the maximum transverse rib interval
Source: JICA Study Team

Figure 4.2.184 Space Frame Model

(3) Design Load

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

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



Source: JICA Study Team

Figure 4.2.185 Space Frame Model

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

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

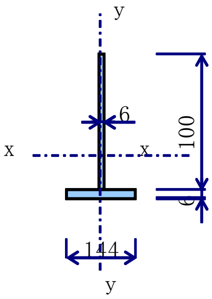
(4) Evaluation of Cross Section

<member ab,bc>

Member ab which has the largest cross-sectional force was employed for this verification.

Cross-sectional force From solid frame analysis

Member no.	1201			
Bending moment	M =	0.243	kN·m	(Equivalent value with regular scenario)
Shear stress	S =	2.347	kN	(Equivalent value with regular scenario)
Axial force	N =	5.184	kN	(Equivalent value with regular scenario)



Effective width of flange on one side is 12t

Surface outside $Iy = 1494792$ (mm^4)

(WEB Height 100mm)					A (mm^2)	y (mm)	Ay (mm^3)	Ay^2 (mm^4)	I o(mm^4)
1 − FLG PL	SM400	144	×	6	864	-53	-45792	2426976	2592
1 − WEB PL	SM400	100	×	6	600	0	0	0	500000
					1464		-45792		2929568
e = -31.3 mm								−	1432374
								Ix =	1497194

yl = -25 mm ∴ Zl = Ix / yl = $-59888\ mm^3$

yu = 81 mm ∴ Zu = Ix / yu = $18416\ mm^3$

Aw = $600\ mm^2$

Verification of allowable stress

σl = N/A + M / Zl
= 3.5 + -4
= $-1\ N/mm^2$ < 1.00 × 140 = $140\ N/mm^2$

σu = N/A + M / Zu
= 3.5 + 13.2
= $17\ N/mm^2$ < 1.00 × 140 = $140\ N/mm^2$

τ max = S / Aw
= $3.9\ N/mm^2$ < 1.00 × 80 = $80\ N/mm^2$

$$F = \left(\frac{\sigma}{\sigma a}\right)^2 + \left(\frac{\tau}{\tau a}\right)^2 = 0.02 \leqq 1.2$$

4.2.11.4 Design of Expansion Joint

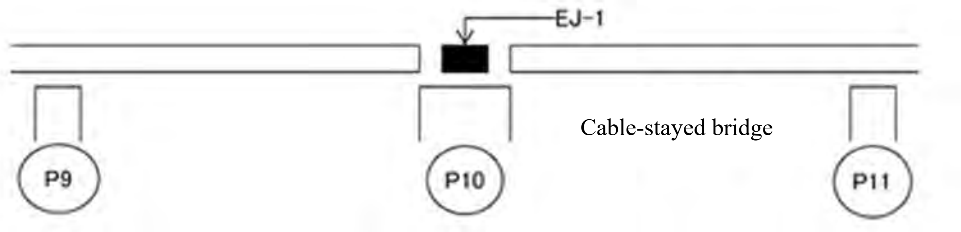
(1) Design Conditions of EJ-1 (P10)

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

Table 4.2.159 Design Conditions

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

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.186 Location of Expansion Joint

(2) Expansion Amount

The design expansion amount of the expansion joint shall consider the regular and seismic conditions.

1) Regular Condition

Table 4.2.160 Expansion Amount at Regular Condition

	Left Girder（P9 side）	Right Girder（P11 side）
Elongation amount by temp. change ΔLt	88 mm	136 mm
Elongation tolerance ΔLy（General elongation tolerance × 20%）	18 mm	27 mm
Sum（Regular scenario） ΔLj	269 mm	

Source: JICA Study Team

2) Seismic Condition

The design expansion amount at seismic condition is as follows:

$\angle Lq = \sqrt{2} \times \pm 190 + \pm 15 = 568$ mm

(3) Selection of Expansion Joint Type

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

ΔLj：269 mm ＜ ΔLq：568 mm

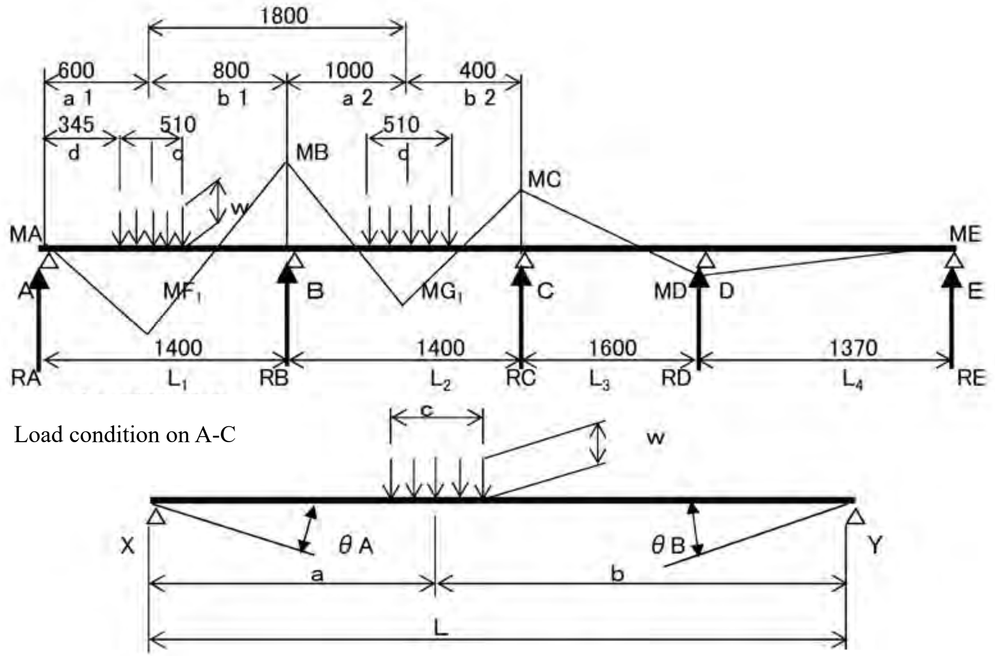
Due to the design expansion amount, the modular type joint (maximum design movement of 640 mm) was selected.

(4) Evaluation of Cross Section

1) Evaluation of Middle Beam

- Calculation of Bending Moment

The middle beam was considered as a four-span continuous beam and the bending moment was calculated with the wheel loading condition as shown in the figure below.

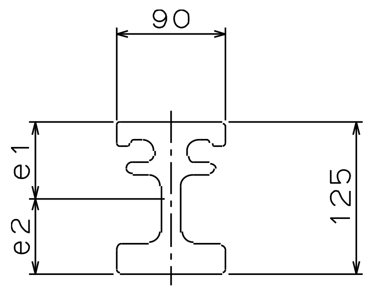


Source: JICA Study Team

Figure 4.2.187 Bending Moment of Middle Beam

The maximum bending moment, calculated as shown above, shall be Mmax = 15240 kN・mm.

- Stress Evaluation



a = 90 mm
b = 125 mm
A = 5904 mm2
e1 = 63 mm
e2 = 62 mm
I = 11552000 mm4

A2 = 1875 mm2 (Cross sectional area of web)

Source: JICA Study Team

Figure 4.2.188 Cross Section of Middle Beam

Impact Coefficient: i

i = 0.4

Maximum bending moment M of A-D

Mmax = 15240 [KN・mm]

Bending Stress: $\sigma 1$

σ_1 = Mmax x (1+i) x e_1 x 1000 / I

= 15240 x (1 + 0.4) x 63 x 1000 / 11552000

= 116.4 N/mm^2 < σba = 210 N/mm^2 (S355J2 + N)

σba = σy / 1.7
= 355 / 1.7
≒ 210 N/mm^2

Shear Stress: $\tau 1$

τ_1 = Rmax x (1 + i) x 1000 / A2

= 72.5 x (1 + 0.4) x 1000 / 1875

= 54.1 N/mm^2 < τa = 120 N/mm^2 (S355J2 + N)

τa = σy/$\sqrt{3}$/1.7
= 355/$\sqrt{3}$/1.7
≒ 120 N/mm^2

Total Stress: U

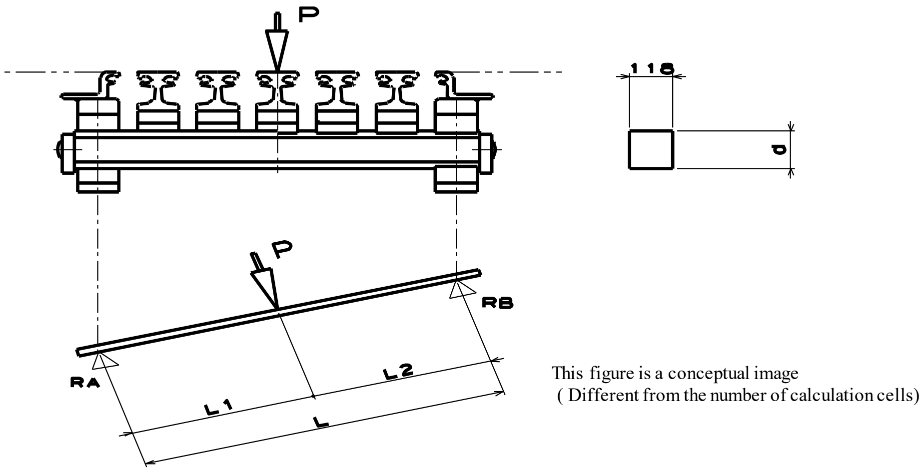
U = (σ1 / σba)^2 + (τ1 / τa)^2

= (116.4 / 210)^2 + (54.1 / 120)^2

= 0.51 < 1.2

2) Evaluation of Support Beam

The support beam shall be evaluated as a simple beam with the support located at the position of the bearing during maximum expansion.



Source: JICA Study Team

Figure 4.2.189 Cross Section of Support Beam

Max load acting on support beam	P = 72.5 kN
Max fulcrum interval	L = 1435 mm
Loading position	L1 = 717.5 mm
Loading position	L2 = 717.5 mm
Support beam height	d = 145mm

Impact coeficient I

I = 0.4

Bending moment M1

M 1 = P x L1 x L2 / L

= 72.5 x 717.5 x 717.5 / 1435 = 26100 KN・mm

Cross-section coefficient Z1

Z 1 = 1/6 x d^2 x 118

= 1 / 6 x 145^2 x 118 = 413500 mm3

Bending stress σ1

σ1 = M 1 x (1 + I) x 1000 / Z 1

= 26100 x (1 + 0.4) x 1000 / 413500

= 88.4 N /mm2 < 167 N /mm2

Shear stress τ1

τ1 = P x (1 + I) x 1000 / (d x H)

= 72.5 x (1 + 0.4) x 1000 / (145 x 118)

= 5.9 N /mm2 < 98 N /mm2

Total stress U

U= (σ1 / σba)^2 + (τ1 / τa)^2

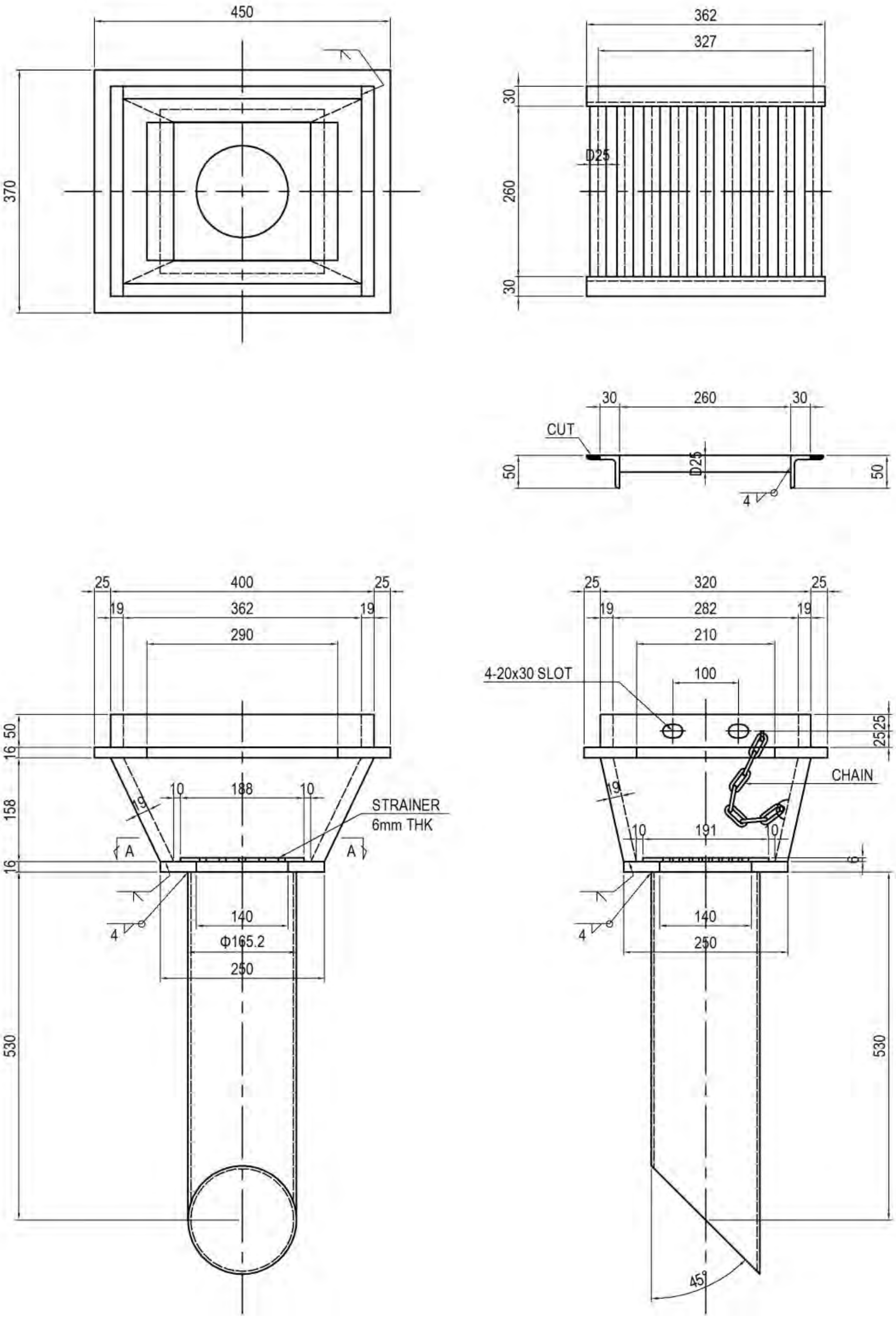
= (88.4 / 167) ^2 + (5.9 / 98) ^2

= 0.284 < 1.2

4.2.11.5 Drainage Device

(1) Catch Basin Shape

The catch basin shape is shown in the figure below.



Source: JICA Study Team

Figure 4.2.190 Catch Basin Shape

(2) Configuration of Catch Basin Interval

The design conditions for the catch basin are as follows:

Rain intensity: 149 mm/h, Runoff coefficient: 0.9, Road drainage width: 11.450 m,

Gauckler-Manning coefficient: 0.013, Safety factor of flow: 0.8, Proportion of falling flow: 0.9

1) Calculation of Water Discharge

The water discharge shall be calculated from the following rational runoff formula to determine the size of the drainage structure:

$$q=\frac{1}{3600}\times C\cdot I\cdot W$$

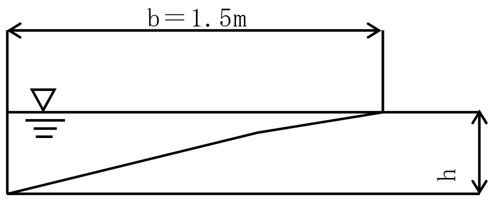
q : Discharge per unit road length (l/sec/m)
C : Rational method runoff coefficient
I : Rainfall intensity (mm/h)
W : Road drainage width (m)

2) Calculation of Flow Rate

The average flow rate within the conduit shall be determined in principle using the following Manning's formula:

$$V=\frac{1}{n}\cdot R^{2/3}\cdot i^{1/2}$$

V : Average velocity (m/sec)
R : Hydraulic radius (m), R = A / S
i : Slope of energy grade line
n : Gauckler-Manning coefficient
S : Wetted perimeter (m) $s=h+\sqrt{(b^2+h^2)}$
A : Cross-sectional area of flow (m2) A=1/2xhxb



Source: JICA Study Team

Figure 4.2.191 Calculated Cross Section

3) Calculation of Flow Rate

The flow volume within the drainage ditch and drainage pipe shall be determined by the average flow rate and flow area.

$$Q=V\cdot A\cdot \alpha$$

Q : Allowable flow volume (m3/sec)
A : Cross-sectional area of flow
α : Safety factor of flow (＝0.8)

4) Calculation of Maximum Interval of Catch Basin

$$Ls=\frac{\gamma\cdot Q}{q}$$

Ls : Maximum interval of catch baisn (m) (Ls≦20m)
γ : Proportion of flow falling into catch ba γ = 0.9
Q : Allowable flow volume (m3/sec)
q : Discharge per unit road length (m3/sec/m)

5) Configuration of Catch Basin Interval

The calculation results of the maximum interval of catch basin are shown below.

Table 4.2.161 Calculation Results of Maximum Interval of Catch Basin

Section		Distance	C.L Design height	Longitudinal slope	Transverse slope	Flow width B	Shoulder depth h	Area of flow A	Wetted perimeter P	Hydraulic radiusR
Edge i	Edge j	(m)	(m)	(%)	(%)	(m)	(m)	(m2)	(m)	(m)
0+860	0+880	20	17.746	-	2	1.5	0.03	0.0225	1.5303	0.014703
0+880	0+900	20	17.801	0.275	2	1.5	0.03	0.0225	1.5303	0.014703
0+900	0+920	20	17.85	0.245	2	1.5	0.03	0.0225	1.5303	0.014703
0+920	0+940	20	17.895	0.225	2	1.5	0.03	0.0225	1.5303	0.014703
0+940	0+960	20	17.934	0.195	2	1.5	0.03	0.0225	1.5303	0.014703
0+960	0+980	20	17.969	0.175	2	1.5	0.03	0.0225	1.5303	0.014703
0+980	0+1000	20	17.998	0.145	2	1.5	0.03	0.0225	1.5303	0.014703
1+0	1+20	20	18.023	0.125	2	1.5	0.03	0.0225	1.5303	0.014703
1+20	1+40	20	18.042	0.095	2	1.5	0.03	0.0225	1.5303	0.014703
1+40	1+60	20	18.057	0.075	2	1.5	0.03	0.0225	1.5303	0.014703
1+60	1+80	20	18.066	0.045	2	1.5	0.03	0.0225	1.5303	0.014703
1+80	1+88	8	18.071	0.025	2	1.5	0.03	0.0225	1.5303	0.014703
1+88	1+100	12	18.071	0	2	1.5	0.03	0.0225	1.5303	0.014703
1+100	1+120	20	18.07	-0.00833	2	1.5	0.03	0.0225	1.5303	0.014703
1+120	1+140	20	18.065	-0.025	2	1.5	0.03	0.0225	1.5303	0.014703
1+140	1+160	20	18.054	-0.055	2	1.5	0.03	0.0225	1.5303	0.014703
1+160	1+180	20	18.039	-0.075	2	1.5	0.03	0.0225	1.5303	0.014703
1+180	1+200	20	18.018	-0.105	2	1.5	0.03	0.0225	1.5303	0.014703
1+200	1+220	20	17.993	-0.125	2	1.5	0.03	0.0225	1.5303	0.014703
1+220	1+240	20	17.962	-0.155	2	1.5	0.03	0.0225	1.5303	0.014703
1+240	1+260	20	17.927	-0.175	2	1.5	0.03	0.0225	1.5303	0.014703
1+260	1+280	20	17.886	-0.205	2	1.5	0.03	0.0225	1.5303	0.014703
1+280	1+300	20	17.841	-0.225	2	1.5	0.03	0.0225	1.5303	0.014703
1+300	1+320	20	17.79	-0.255	2	1.5	0.03	0.0225	1.5303	0.014703

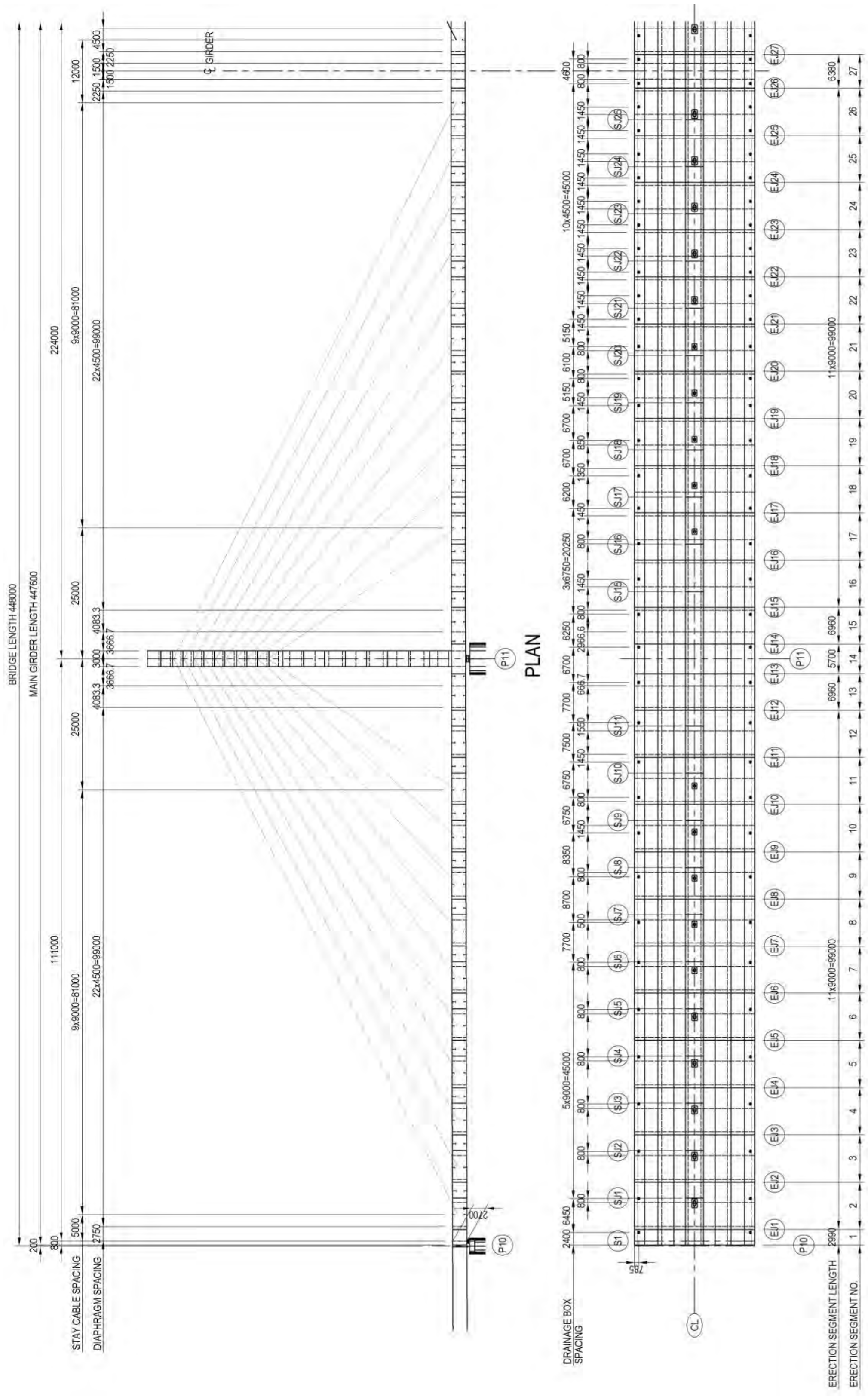
Section		Safety factor	Allowable flow volume Q	Road drainage width	Runoff coefficient	Rain intensity	per unit road length	Propotion of falling flow	Maximum interval of catch basin	Interval of catch basin
Edge i	Edge j	α	(l/sec)	(m)	C	I[mm/h]	q(l/sec/m)	γ	Ls (m)	
0+860	0+880	-	-	-			-	-	-	
0+880	0+900	0.8	5.447235	11.45	0.9	149	0.426513	0.9	9.195532	9
0+900	0+920	0.8	5.141536	11.45	0.9	149	0.426513	0.9	8.679477	8
0+920	0+940	0.8	4.92721	11.45	0.9	149	0.426513	0.9	8.317672	8
0+940	0+960	0.8	4.586983	11.45	0.9	149	0.426513	0.9	7.743331	7
0+960	0+980	0.8	4.345391	11.45	0.9	149	0.426513	0.9	7.335497	7
0+980	0+1000	0.8	3.955431	11.45	0.9	149	0.426513	0.9	6.677202	6
1+0	1+20	0.8	3.672525	11.45	0.9	149	0.426513	0.9	6.199627	6
1+20	1+40	0.8	3.201633	11.45	0.9	149	0.426513	0.9	5.404709	5
1+40	1+60	0.8	2.844726	11.45	0.9	149	0.426513	0.9	4.80221	5
1+60	1+80	0.8	2.203515	11.45	0.9	149	0.426513	0.9	3.719776	5
1+80	1+88	0.8	1.642403	11.45	0.9	149	0.426513	0.9	2.772557	5
1+88	1+100	0.8	0	11.45	0.9	149	0.426513	0.9	0	5
1+100	1+120	0.8	0.948242	11.45	0.9	149	0.426513	0.9	1.600737	5
1+120	1+140	0.8	1.642403	11.45	0.9	149	0.426513	0.9	2.772557	3
1+140	1+160	0.8	2.436078	11.45	0.9	149	0.426513	0.9	4.112367	5
1+160	1+180	0.8	2.844726	11.45	0.9	149	0.426513	0.9	4.80221	5
1+180	1+200	0.8	3.365925	11.45	0.9	149	0.426513	0.9	5.682052	5
1+200	1+220	0.8	3.672525	11.45	0.9	149	0.426513	0.9	6.199627	6
1+220	1+240	0.8	4.089551	11.45	0.9	149	0.426513	0.9	6.903612	6
1+240	1+260	0.8	4.345391	11.45	0.9	149	0.426513	0.9	7.335497	7
1+260	1+280	0.8	4.703127	11.45	0.9	149	0.426513	0.9	7.939396	7
1+280	1+300	0.8	4.92721	11.45	0.9	149	0.426513	0.9	8.317672	8
1+300	1+320	0.8	5.245415	11.45	0.9	149	0.426513	0.9	8.854838	8

Note: Where the calculated value for maximum interval between catch basin is less than 5 m, the catch basin interval is set to more than 5 m

Source: JICA Study Team

(3) Catch Basin Arrangement

The position of the catch basin is shown below:



Source: JICA Study Team

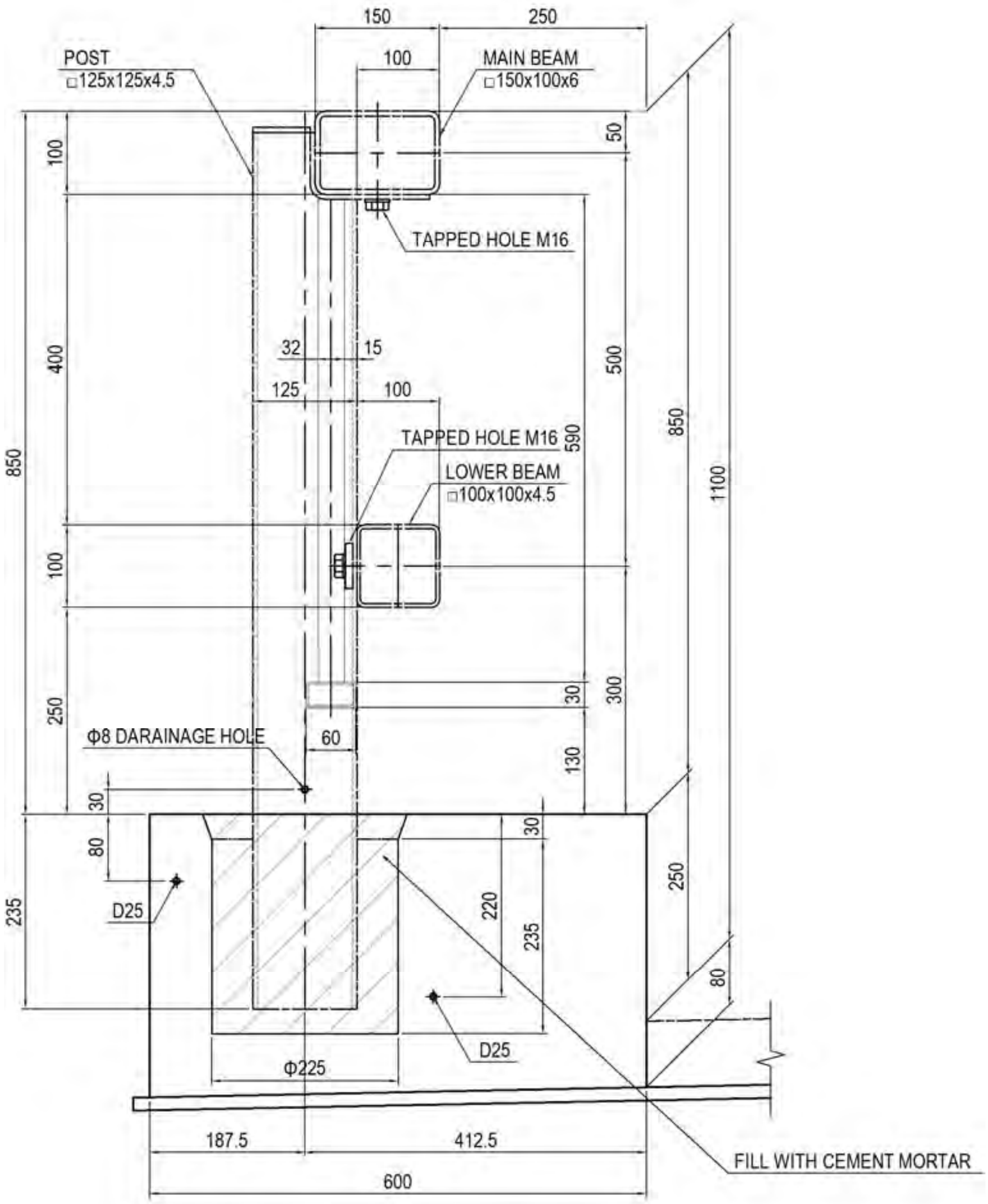
Figure 4.2.192 Catch Basin Location

4.2.11.6 Guardrail

(1) Specifications of Guardrail

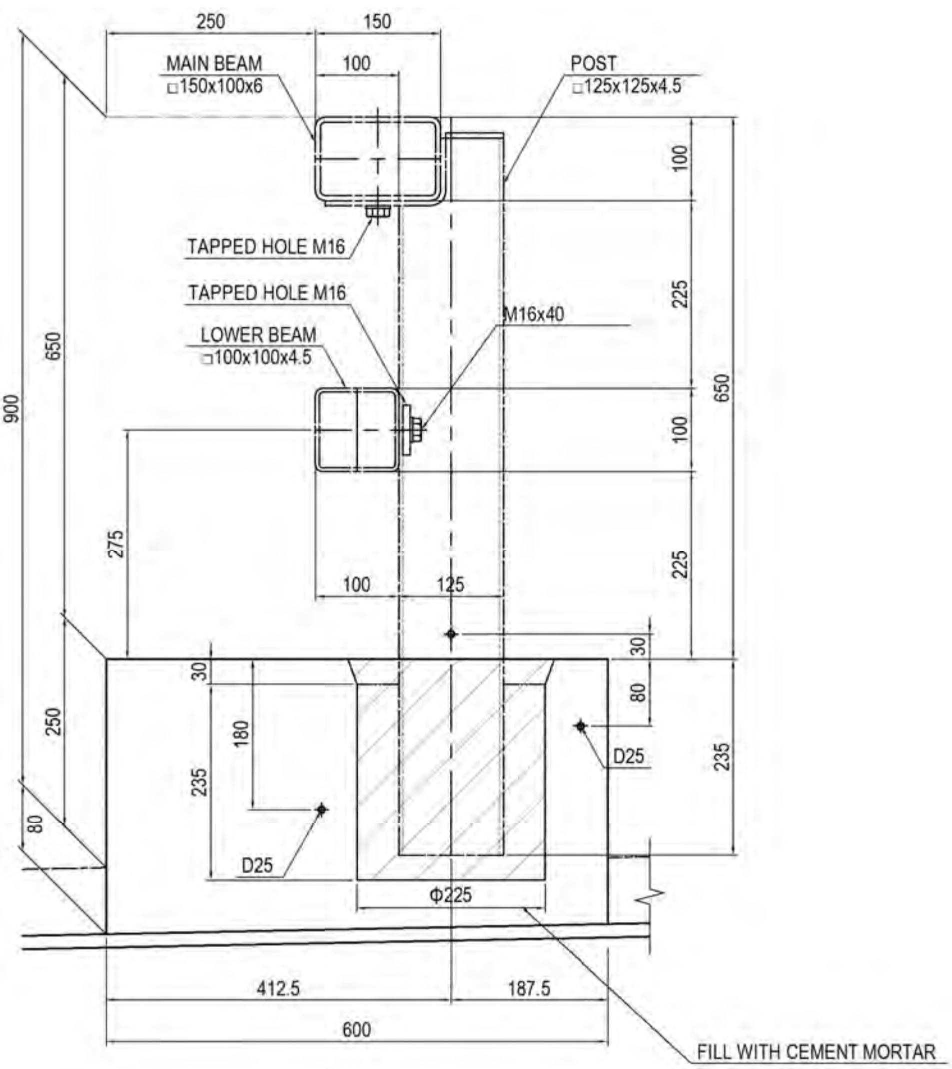
The Type-A combination railing (steel) which is shown in the standard drawings for Ministry of Land, Infrastructure, Transport and Tourism Hokuriku Regional Development Bureau was selected. The specifications of guardrail are as follows:

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



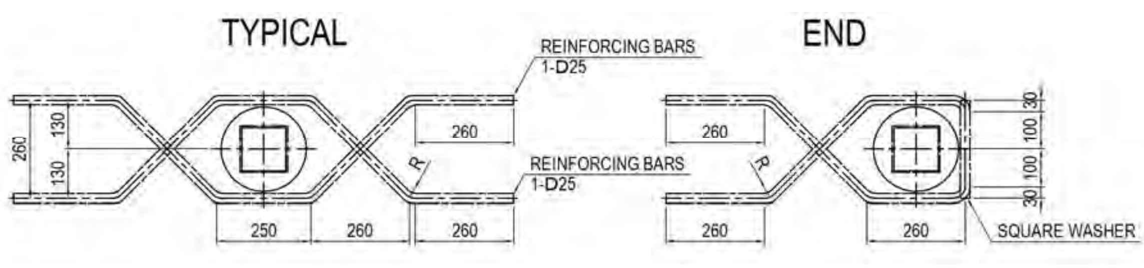
Source: JICA Study Team

Figure 4.2.193 Detailed Plan of Guardrail (Outer Side)



Source: JICA Study Team

Figure 4.2.194 Detailed Plan of Guardrail (Median Side)



Source: JICA Study Team

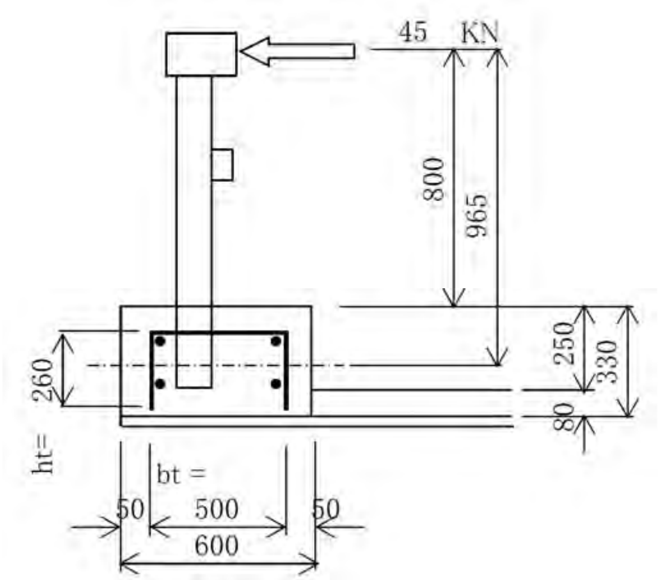
Figure 4.2.195 Reinforcing Steel

(2) Design of Barrier Curb Footing

The fixing of the guardrail shall be designed. A continuous footing curb able to withstand the impact of a vehicle shall be installed, and the guardrail post shall be fixed on top of the curb.

1) Design Condition

- Design strength of concrete : $\sigma ck = 24\ N/mm^2$
- Force applied per post : Pmax = 45.0 kN*



Source: JICA Study Team

Figure 4.2.196 Schematics of Continuous Footing

* Maximum Resistance Force of Railing Post: Pmax

Outline of the Railing Post

Post	: □-125x125x4.5
Cross Section Area	: A = 21.17 cm2
Second Moment of Inertia	: I = 506 cm4
Section Modulus	: Z = 80.09 cm3
Plastic Section Modulus	: Zp = 94.8 cm3

All Plastic Bending Moment

Mp = σv x Zp = 235 / 94,800

= 22,278,000 N・mm

Ultimate Resistance Force of Railing Post

Pw = Mp / H = 22,278,000 / 600

= 37130 N = 37.13kN

Maximum Resistance Force of Railing Post

Pmax = 37 x 1.2

= 44.556 kN ≒ 45.0 kN

The ratio of Pw and Pmax: 1.2 was assumed from experimental results of other railings.

2) Design of Torsion Reinforcement

- Horizontal reinforcement against torque

If the cross sectional area of one bar for horizontal reinforcement against torque, arranged at interval a, is Awt (mm^2), then:

$$Awt = \frac{Mt \cdot a}{1.6 \cdot bt \cdot ht \cdot \sigma y}$$

Here a : Interval of horizontal reinforcement bar (mm)
Mt : Torsion acting on cross section of member N・mm
σy : Yield point of reinforcement bars (N/mm^2)
bt,ht : Width and height specified in the above figure (mm)

$$Mt = P \cdot L = 45000 \times 965 = 43425000 \ N \cdot mm$$

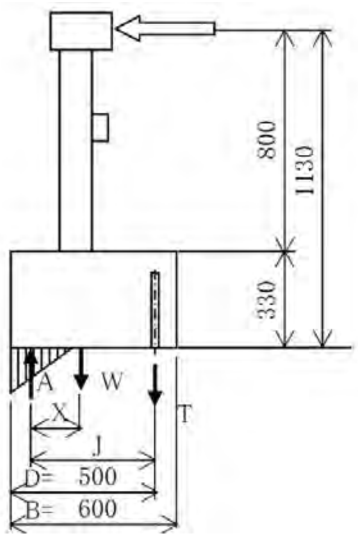
bt = 500 mm
ht = 260 mm
a = 300 mm
If SD345(σy = 345 N/mm^2)is used

$$Awt = \frac{43425000 \times 300}{1.6 \times 500 \times 260 \times 345} = 181.5 \ mm^2 < 198.6 \ mm^2$$

Therefore, SD345-D16 (198.6 mm^2) is utilized.

3) Anchorage Reinforcement against Overturning

The floor deck of the continuous footing shall be fixed using post-installed anchor. The post-installed fixing anchor per effective width, discussed in the next chapter, is designed below.



Source: JICA Study Team

Figure 4.2.197 Anchorage of Floor Slab

Tensile force of the anchor is T, and self-weight per effective width is W. Considering equilibrium of forces about the point A, then:

$$T = \frac{Mt - W \cdot X}{J}$$

Effective Width

$W = \quad 24.5 \times 0.60 \times 0.33 \times 1.050 = 5093.55 \text{ N}$

Assuming D = 500 mm

$J = \quad 7/8 \times D = \quad 7 / 8 \times 500 = 438 \text{ mm}$

$X = \quad B/2 - D/8 = \quad 600 / 2 - 500 / 8 = 238 \text{ mm}$

$$T = \frac{43425000 - 5093.55 \times 238}{438} = 96376 \text{ N}$$

Assuming per effective width of 1050 mm, four bars are needed, the tensile strength T1 per bar of anchor is:

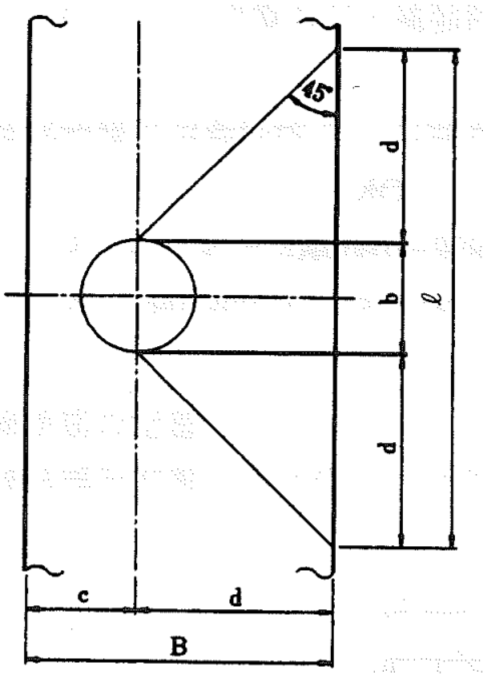
$$T1 = \frac{96376}{4} = 24094 \text{ N}$$

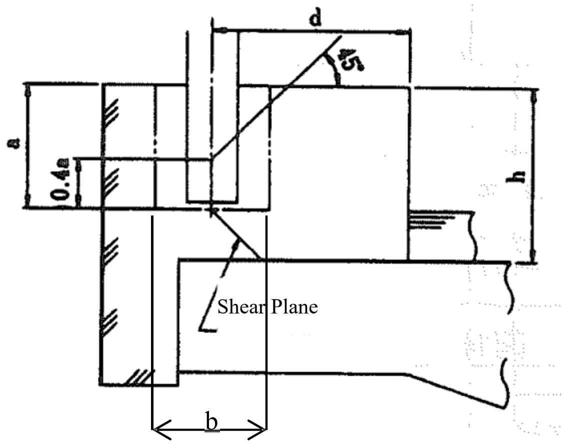
Therefore, the required cross sectional area As per bar is:

$$As = \frac{T1}{\sigma y} = \frac{24094}{345} = 69.8 \text{ mm}^2 < 198.6 \text{ mm}^2$$

Therefore, the guardrail is fixed by SD345-D16-4 post-installed fixing anchors per effective width of 1050 mm.

< Calculation of effective width (l)>





$b = 225$ mm

$d = 412.5$ mm

$l = b + 2d$
$= 1050$ mm

Source: JICA Study Team

Figure 4.2.198 Calculation of Effective Width

- Evaluation of weld between rebar and steel deck

Allowable stress of studs	σsa	= 140 N/mm^2
Increase coefficient at impact		= 1.5
Maximum tensile stress	σs	= T1 / Aw (D16)
		= 24094 / 198.6 = 121.3 N/mm^2 < σca
Allowable tensile stress	σca	= 0.9 x 0.9 x 140 x 1.5 = 170 N/mm^2

- Evaluation of fixation length of reinforcement

Fixation length	L	= T1 / (π x ϕ x nc x τoa)
		= 24094 / (π x 15.9 x 1.5 x 1.60)
		= more than 201 mm
Increase coefficient	nc	= 1.5
Allowable fixation stress	τoa	= 1.60 N/mm^2
Nominal diameter (D16)	ϕ	= 15.9 mm

4.2.11.7 Design of Base for Miscellaneous Items

(1) Base for Road Lighting Pole

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

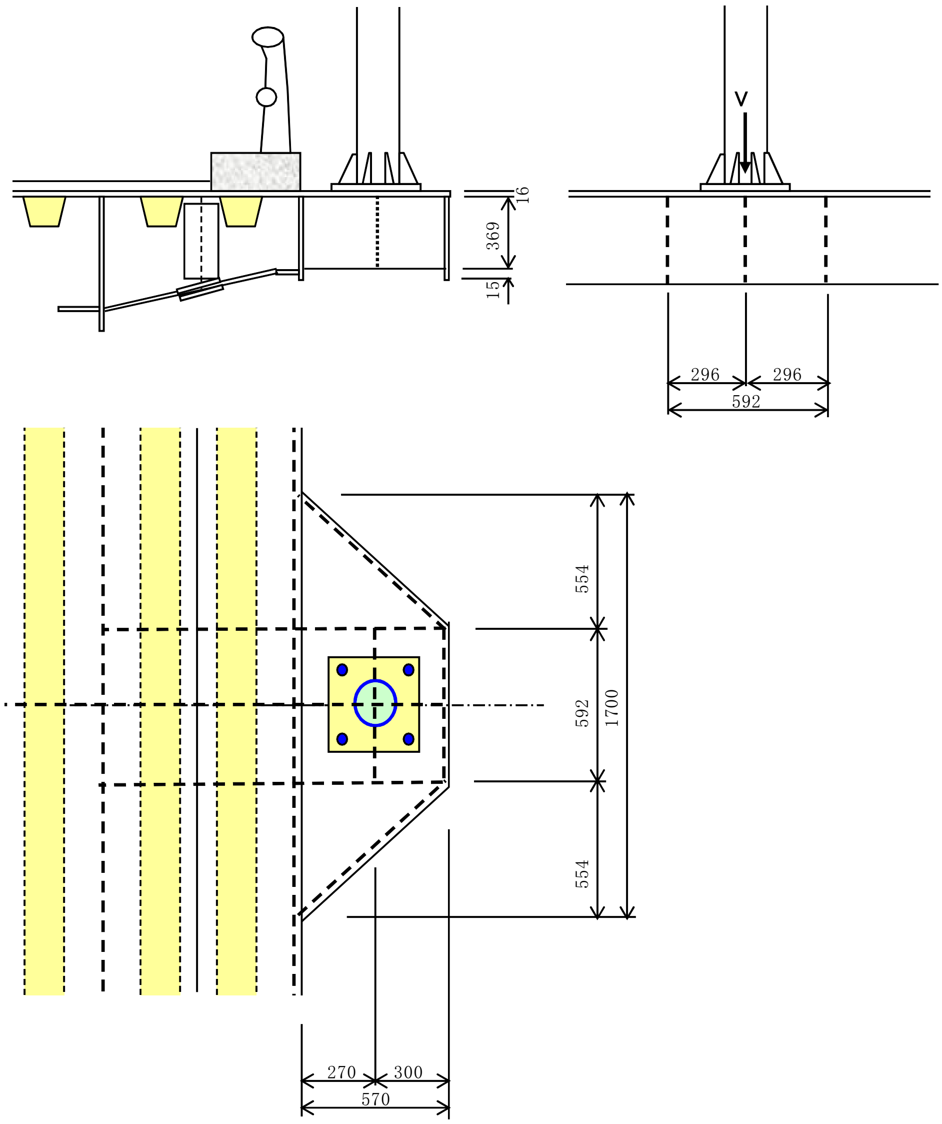
1) Design Load

The assumed weight of the road lighting pole is:

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

2) Design of Base

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



Source: JICA Study Team

Figure 4.2.199 Base for Road Lighting Pole

a) Cross Section Design

(SM400)			A	y	Ay	$Ay^2 + I$
1 - PL	976	× #	156.2	-19.3	-3015	58190
3 - PL	369	× 9	99.6	0	0	11305
			255.8 cm^2		-3015 cm^3	69495 cm^4
						-35536
					I =	33959 cm^4

$$\delta = \frac{Ay}{A} = -11.8 \text{ cm}$$

$$I' = 255.8 \times -11.8^2 = -35536$$

yu = - 18.5 - 1.6 - -11.8 = -8.3 cm
yl = + 18.5 + - -11.8 = 30.2 cm

wu = -4091 cm^3
wl = 1124 cm^3

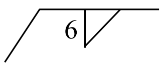
$\sigma u = M / wu = -0.1 \text{ N/mm}^2 < \sigma a = 140 \text{ N/mm}^2$
$\sigma l = M / wl = 0.5 \text{ N/mm}^2 < \sigma a = 140 \text{ N/mm}^2$
$\tau = S / Aw = 0.2 \text{ N/mm}^2 < \tau a = 80 \text{ N/mm}^2$

Composite Stress
$(0.5 / 140)^2 + (0.2 / 80)^2 = 0.00 < 1.2$ *OK*

b) Welding Design

Upper Flange: Full Penetration Welding

Web:



Throat Thickness
$a = 6 / \sqrt{2}$
$= 4.2$ mm

(SM400)			A	y	Ay	$Ay^2 + I$
1 - PL	976	× 16	156.2	-19.3	-3015	58190
6 - PL	369	× 4.2	93.9	0	0	10658
			250.1 cm^2		-3015 cm^3	68848 cm^4
						-36346
					I =	32502 cm^4

$$\delta = \frac{Ay}{A} = -12.1 \text{ cm}$$

$$I' = 250.1 \times -12.1^2 = -36346$$

yu = - 18.5 - 1.6 - -12.1 = -8.0 cm
yl = + 18.5 + - -12.1 = 30.5 cm

wu = -4063 cm^3
wl = 1066 cm^3

$\sigma u = M / wu = -0.1 \text{ N/mm}^2 < \tau a = 80 \text{ N/mm}^2$
$\sigma l = M / wl = 0.5 \text{ N/mm}^2 < \tau a = 80 \text{ N/mm}^2$
$\tau = S / Aw = 0.2 \text{ N/mm}^2 < \tau a = 80 \text{ N/mm}^2$

Composite Stress
$(0.5 / 80)^2 + (0.2 / 80)^2 = 0.00 < 1$ *OK*

(2) Base for Navigation Sign

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

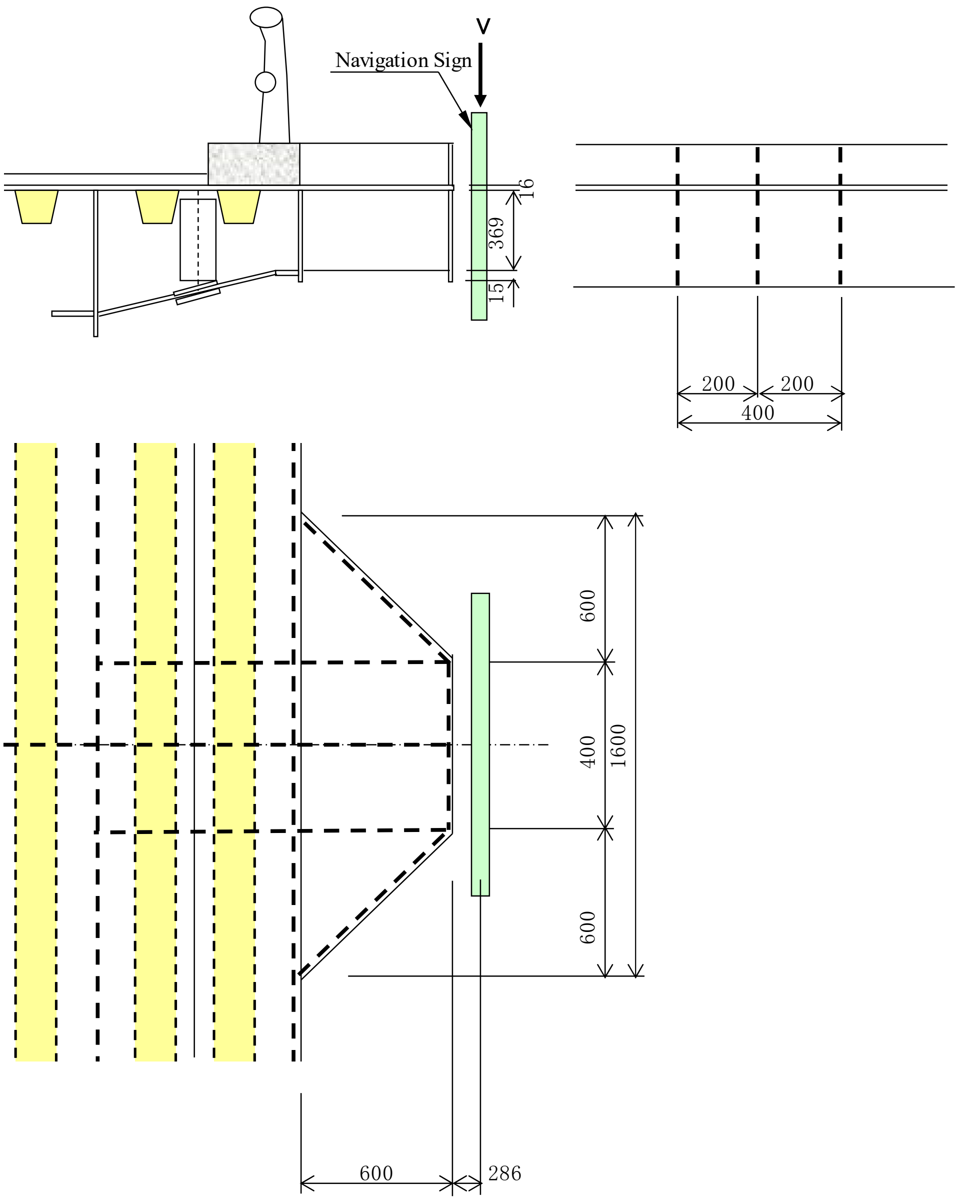
1) Design Load

The assumed weight of the navigation sign is:

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

2) Design of Base

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



Source: JICA Study Team

Figure 4.2.200 Base for Navigation Sign

a) Cross Section Design

(SM400)			A	y	Ay	$Ay^2 + I$
1 - PL	784 ×	#	125.4	-19.3	-2420	46706
3 - PL	369 ×	9	99.6	0	0	11305
			225.0 cm^2		-2420 cm^3	58011 cm^4

$\delta = \dfrac{Ay}{A} = -10.8$ cm

-26028

I = 31983 cm^4

$I' = 225.0 \times -10.8^2 = -26028$

yu = - 18.5 - 1.6 - -10.8 = -9.3 cm

yl = + 18.5 + - -10.8 = 29.2 cm

wu = -3439 cm^3

wl = 1095 cm^3

σu = M／wu = -0.3 N/mm^2 < σa = 140 N/mm^2

σl = M／wl = 0.8 N/mm^2 < σa = 140 N/mm^2

τ = S / Aw = 0.1 N/mm^2 < τa = 80 N/mm^2

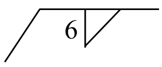
Composite Stress

$(\ 0.8\ /\ 140\ )^2 + (\ 0.1\ /\ 80\ )^2 = 0.00 < 1.2$ *OK*

b) Welding Design

Upper Flange: Full Penetration Welding

Web:



Throat Thickness

a = 6 / $\sqrt{2}$

= 4.2 mm

(SM400)			A	y	Ay	$Ay^2 + I$
1 - PL	784 ×	16	125.4	-19.3	-2420	46706
6 - PL	369 ×	4.2	93.9	0	0	10658
			219.3 cm^2		-2420 cm^3	57364 cm^4

$\delta = \dfrac{Ay}{A} = -11.0$ cm

-26705

I = 30659 cm^4

$I' = 219.3 \times -11.0^2 = -26705$

yu = - 18.5 - 1.6 - -11.0 = -9.0 cm

yl = + 18.5 + - -11.0 = 29.5 cm

wu = -3407 cm^3

wl = 1039 cm^3

σu = M／wu = -0.3 N/mm^2 < τa = 80 N/mm^2

σl = M／wl = 0.9 N/mm^2 < τa = 80 N/mm^2

τ = S / Aw = 0.1 N/mm^2 < τa = 80 N/mm^2

Composite Stress

$(\ 0.9\ /\ 80\ )^2 + (\ 0.1\ /\ 80\ )^2 = 0.00 < 1$ *OK*

(3) Base for Aircraft Warning Light

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

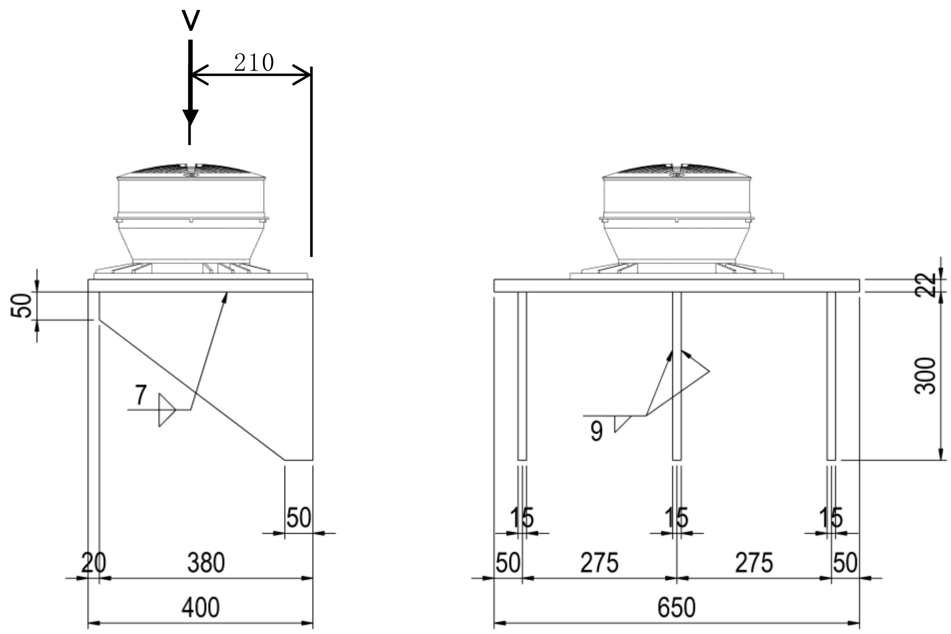
1) Design Load

The assumed weight of the aircraft warning light is:

Aircraft warning light (assumed weight) V = 0.200 kN (about 20 kg)

2) Design of Base

$$M = 0.200 \times 0.210$$
$$= 0.042 \text{ kNm}$$
$$S = 0.200 \text{ kN}$$



Source: JICA Study Team

Figure 4.2.201 Base for Aircraft Warning Light

a) Cross Section Design

	(SM400)	A	y	Ay	$Ay^2 + I$
1 - PL	650 × 22	143.0	-16.1	-2302	37062
3 - PL	300 × 15	135.0	0	0	10125
		278.0 cm^2		-2302 cm^3	47187 cm^4

$\delta = \dfrac{Ay}{A} = -8.3$ cm

-19062

I = 28125 cm^4

$I' = 278.0 \times -8.3^2 = -19062$

yu = - 15 - 2.2 - -8.3 = -8.9 cm

yl = + 15 + - -8.3 = 23.3 cm

wu = -3160 cm^3

wl = 1207 cm^3

σu = M／wu = 0.0 N/mm^2 < σa = 140 N/mm^2

σl = M／wl = 0.0 N/mm^2 < σa = 140 N/mm^2

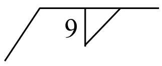
τ = S / Aw = 0.0 N/mm^2 < τa = 80 N/mm^2

Composite Stress

$(0.0 / 140)^2 + (0.0 / 80)^2 = 0.00 < 1.2$ *OK*

b) Welding Design

Upper Flange:

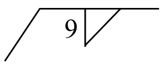


Throat Thickness

$a = 9 / \sqrt{2}$

$= 6.4$ mm

Web;



Throat Thickness

$a = 9 / \sqrt{2}$

$= 6.4$ mm

	(SM400)	A	y	Ay	$Ay^2 + I$
2 - PL	650 × 6.4	82.7	-15.3	-1265	19355
6 - PL	300 × 6.4	114.6	0	0	8591
		197.3 cm^2		-1265 cm^3	27946 cm^4

$\delta = \dfrac{Ay}{A} = -6.4$ cm

-8111

I = 19835 cm^4

$I' = 197.3 \times -6.4^2 = -8111$

yu = - 15 - 0.6 - -6.4 = -9.2 cm

yl = + 15 + - -6.4 = 21.4 cm

wu = -2156 cm^3

wl = 927 cm^3

σu = M／wu = 0.0 N/mm^2 < τa = 80 N/mm^2

σl = M／wl = 0.0 N/mm^2 < τa = 80 N/mm^2

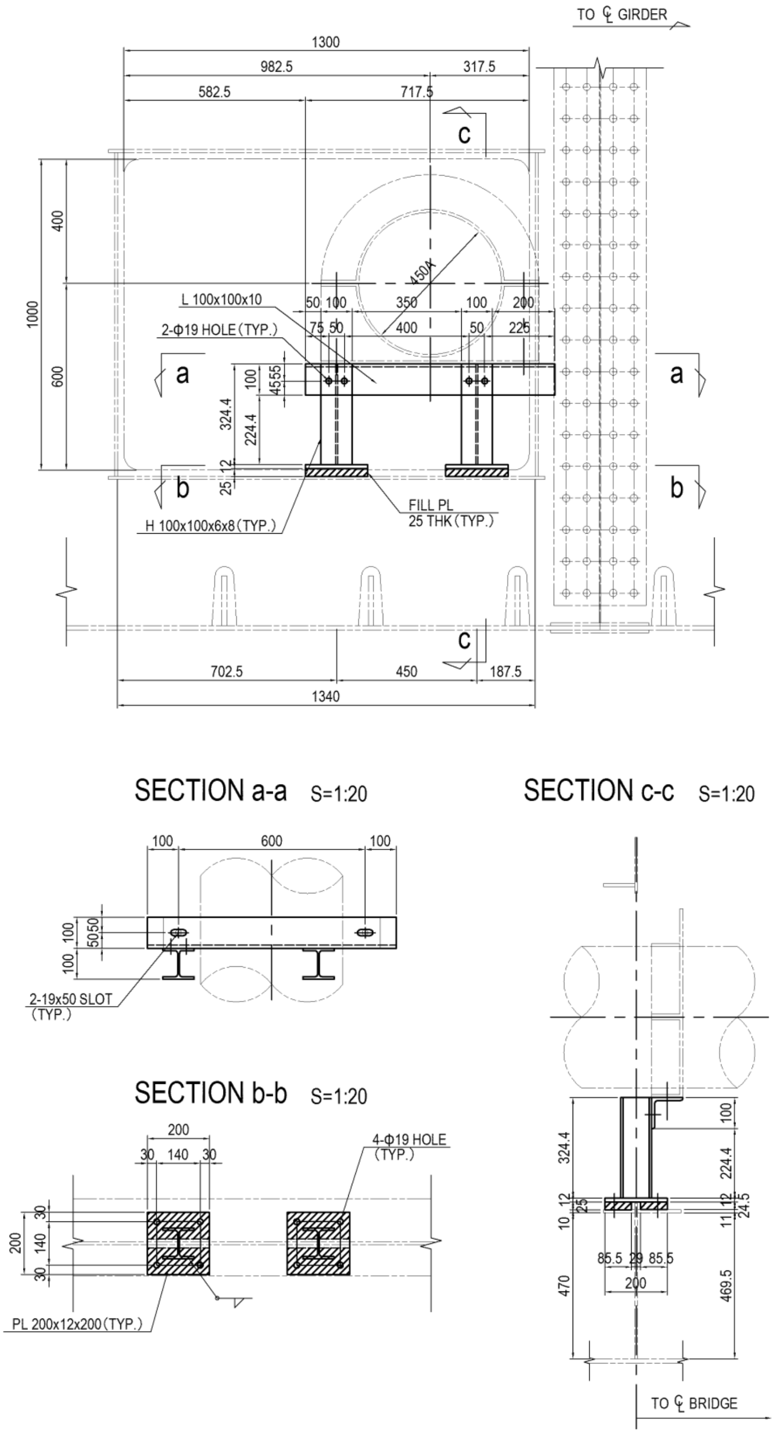
τ = S / Aw = 0.0 N/mm^2 < τa = 80 N/mm^2

Composite Stress

$(0.0 / 80)^2 + (0.0 / 80)^2 = 0.00 < 1$ *OK*

(4) Support for Water Pipe

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



Source: JICA Study Team

Figure 4.2.202 Water Pipe Support

- Design Load

Water Pipe (Full Water)				=	6.00	kN/m		
Supporting Metals	0.015	×	9.81	=	0.15	kN/m		
			W	=	6.15	kN/m	→	6.20 kN/m

- Install Distance L= 2.25 m (less than 2.25m)

-Force at Each Supporting Position P= W･L= 6.20 × 2.25 = 13.95 kN

- Stress Resultants

M= 13.95 × 0.450 / 4 = 1.57 kN･m

S= 13.95 / 2 = 6.98 kN

- Cross Section Design of Supporting Metals

1-L 100 x 100 x 10（SS400)

Z= 24.4 cm3

Aw= 9.0 cm2

σ= M/Z= 1.57 × 10^6 / 24.4 × 10^3 = 64 N/mm2

< 140 N/mm2

τ= S/Aw= 6.98 × 10^3 / 9.0 × 10^2 = 8 N/mm2

< 80 N/mm2

- Evaluation of Bolts

Bolt 2 - M16 (equivalent to SS400)

Calculate as the 2-bolts will work effectively on shear force

A = 13.835^2 × π × 1 / 4 × 2 nos = 301 mm^2

- Shear Stress of Bolts

Shear stress τ calculated from the shear force S

τ = 6.98 × 10^3 / 301 = 23 N/mm^2 < σa= 80 N/mm^2

- Bearing Stress of Bolts: σ

Area A = 14.5 × 6.0 = 87 mm^2

σ= 13.835^2 × π × 1 / 4 × 23 / 87 = 40 N/mm^2

< σa = 210 N/mm^2

- Evaluation of Diaphragm and Transverse Rib

The web cross section directly under the supporting metal was evaluated:

Force at 1 supporting metal P = 13.95 kN

Effective Width Thickness

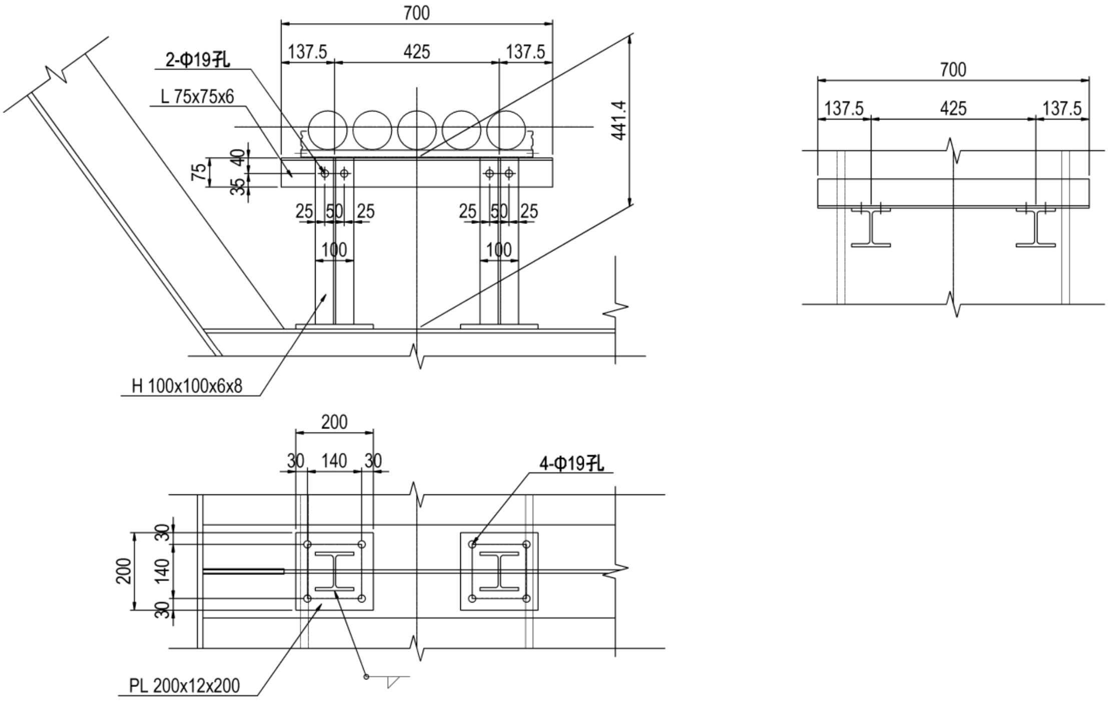
Web cross section Aw = 100 × 9 = 900 mm^2

(Width of shape beam)

σ= P/Aw= 13.95 × 10^3 / 900 = 16 N/mm^2 < 140 N/mm^2

(5) Support for Electrical Cables

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



Source: JICA Study Team

Figure 4.2.203 Electrical Cable Support

- Design Load

Electrical Cable	0.300	×	5	=	1.50	kN/m			
Supporting Metals	0.007	×	9.81	=	0.07	kN/m			
			W	=	1.57	kN/m	→	1.60	kN/m

- Install Distance L= 2.25 m (less than 2.25m)

-Force at Each Supporting Position P= W・L = 1.60 × 2.25 = 3.60 kN

- Stress Resultants

M= 3.60 × 0.425 / 4 = 0.38 kN・m

S= 3.60 / 2 = 1.80 kN

- Cross Section Design of Supporting Metals

1-L 75 x 75 x 6 (SS400)

Z= 8.47 cm3

Aw= 4.1 cm2

σ= M/Z= 0.38 × 10^6 / 8.47 × 10^3 = 45 N/mm2 < 140 N/mm2

τ= S/Aw= 1.80 × 10^3 / 4.1 × 10^2 = 4 N/mm2 < 80 N/mm2

- Evaluation of Bolts

Bolt 2 - M16 (equivalent to SS400)

Calculate as the 2-bolts will work effectively on shear force

A = 13.835^2 × π × 1 / 4 × 2 nos = 301 mm^2

- Shear Stress of Bolts

Shear stress τ calculated from the shear force S

τ = 1.80 × 10^3 / 301 = 6 N/mm^2 < σa= 80 N/mm^2

- Bearing Stress of Bolts: σ

Area A= 14.5 × 6.0 = 87 mm^2

σ= 13.835^2 × π × 1 / 4 × 6 / 87 = 10 N/mm^2 < σa = 210 N/mm^2

- Evaluation of Diaphragm and Transverse Rib

The web cross section directly under the supporting metal was evaluated:

Force at 1 supporting metal P = 3.60 kN

		Effective Width		Thickness			
Web cross section	Aw =	100	×	9	=	900	mm^2
		(Width of shape beam)					

σ= P/Aw= 3.60 × 10^3 / 900 = 4 N/mm^2 < 140 N/mm^2

4.2.11.8 Maintenance Equipment

(1) Inspection Facility Plan

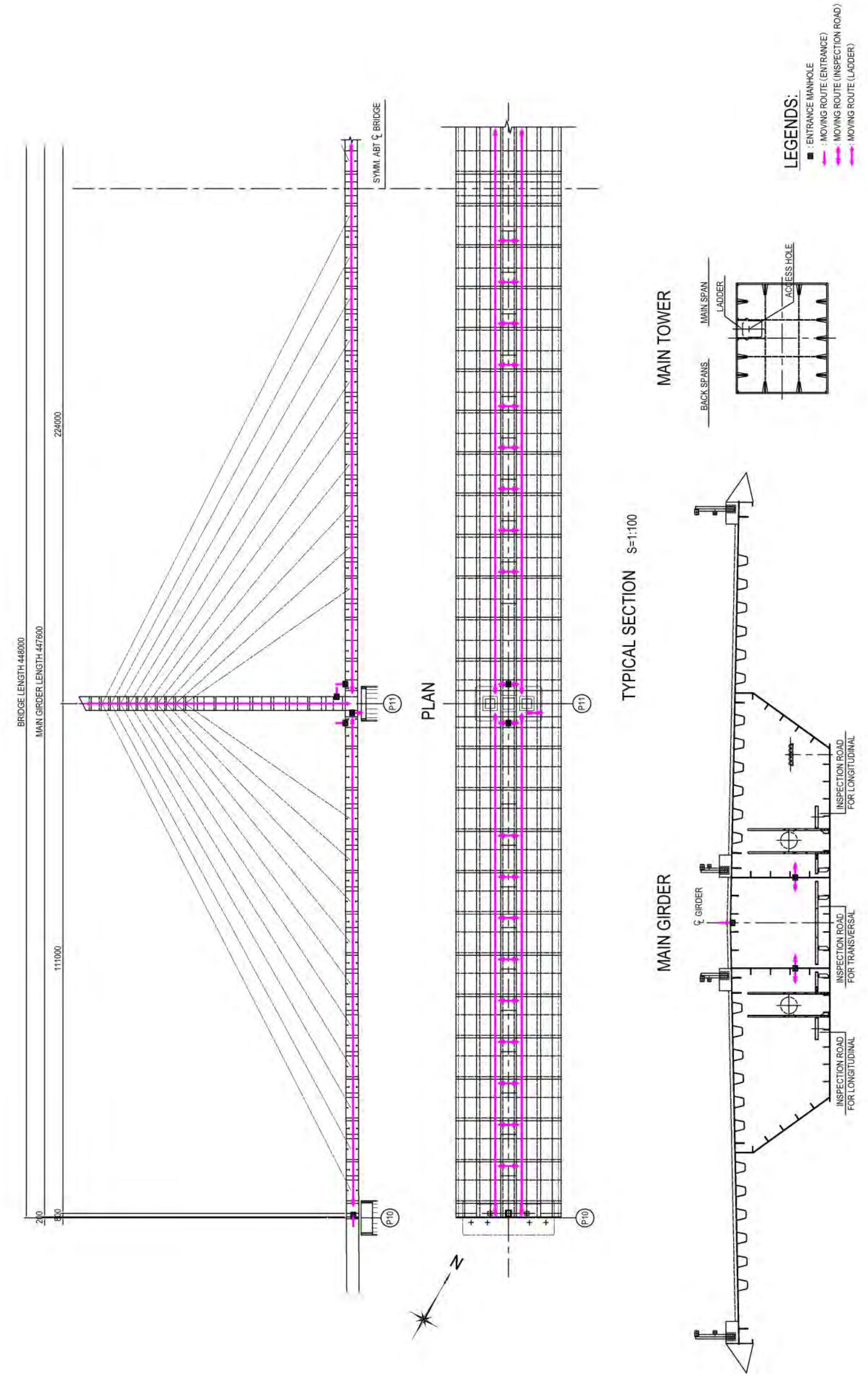
Based on the inspection facility arrangement plan listed below, the installation of the inspection facility at the necessary positions are examined.

Table 4.2.162 Inspection Facility Arrangement Plan

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

Source: JICA Study Team

The maintenance route is shown in the figure below.



Source: JICA Study Team

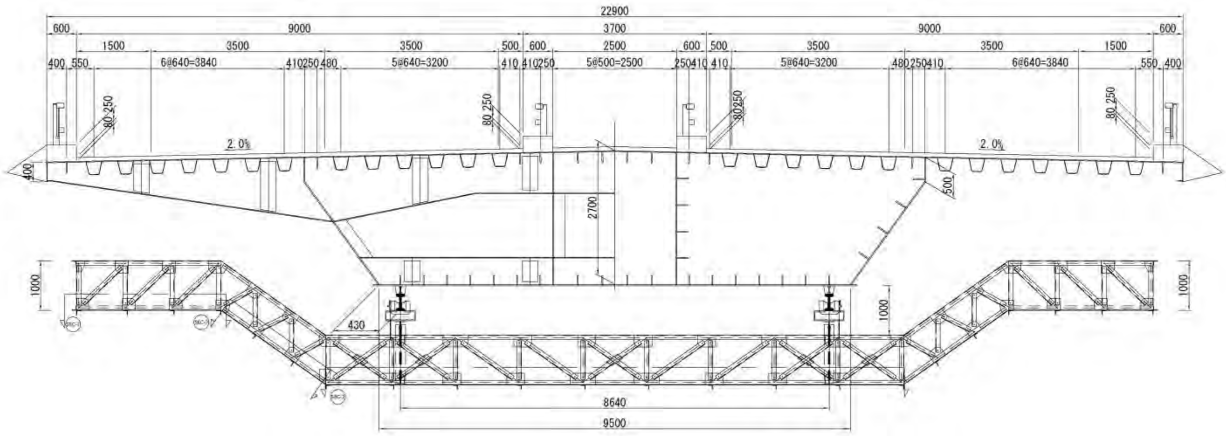
Figure 4.2.204 Maintenance Route

(2) Examination of Inspection Car Rail (Reference)

1) Examination Overview

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

The outline for the assumed inspection car is shown below.

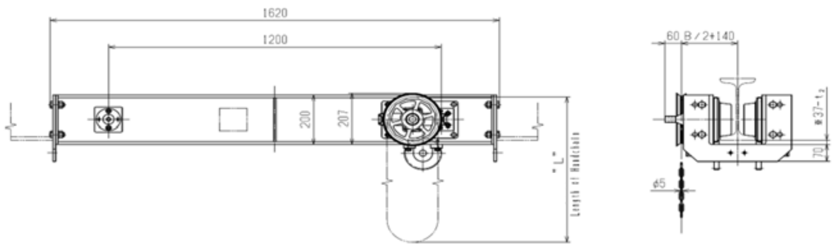


Source: JICA Study Team

Figure 4.2.205 Girder Undersurface Inspection Car

2) Design Conditions

- ➢ The self-weight of the inspection car was set by referring to the inspection car at the Thanlyin Bridge.
- ➢ The assumptions made for the saddle of the inspection car is as listed below.
  - ・ Number of suspension points : 4 Points
  - ・ Number of wheels per suspension point : 2 wheels
  - ・ Suspension point interval Long. Direction : 1.2m
    Trans. Direction : 8.64m
  - ・ Maximum load of one suspension point : 34.3kN (3.5t)
  - ・ Sample image of saddle



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

Source: JICA Study Team

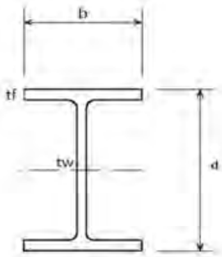
Figure 4.2.206 Sample Image of Saddle

The scaffolding loads during construction has not been accounted for in this calculation.

The calculation results (reference) is shown below.

(1)Specification

Inspection Car			
Longitudinal length	1.2	m	
Transverse width	22.9	m	
Self weight, W	100	kN	
Live load, Pl	15	kN	
Hanging Points of Inspection Car	4	points	
Longitudinal distance of points	1.2	m	
Transverse distance of points	8.5	m	
Wheels at 1 point	2	wheels	
Wheels Transverse distance, Lb1	100	mm	
Wheels Longitudinal distance, Lb2	0	mm	



Rail for Inspection Car

Section Properties of I-Beam (Rail)	d(mm)	b(mm)	tw(mm)	tf(mm)	I(cm4)	Z(cm3)
	300	150	11.5	22	14700	978

Span of Rail	2.5	m	
Young's Modulus of Steel	2.00E+05	N/mm2	

(2) Load at one contact point

Increase Coefficient				
Impact factor for live load, i	0.2			
Load at one hanging point: P	34.0	kN	P=(W/4+Pl*(1+i)/2)	OK
	3.47	ton		

(3)Verification of stress and deflection

(3.1) As a beam

Span length of rail, L	2.5	m		
Max Bending Moment, M	21.3	kNm	M=PL/4	
Max Shear Force, S	51.0	kN	S=1.5P	
Moment of Inertia, I	14700	cm4		
Section Modulus, Z	978	cm3		
Web Area, Aw	2944	mm2	Aw=(h-2*tf)*tw	
Bending Stress, σ	21.7	N/mm2	σ=M/Z	OK
Shear Stress, τ	17.3	N/mm2	τ=S/Aw	OK
Allowable Bending Stress, σa	138.5	N/mm2	σa=1.25*(140-2.4*(L*1000/bf))	
Allowable Shear Stress, τa	100.0	N/mm2	τa=1.25*80	
Deflection, δ	0.3765	mm	δ=PL3/48EI	
Ratio of Deflection/Length: L/δ	6641			

(3.2) As a flange plate carrying wheel

Extended length of flange, Lt	44.25	mm	Lt=(Lb1-tw)/2	
BendingMoment, Mt	0.75	kNm	M=PLt/2	
ShearForce, St	25.5	kN	S=P/2	

Stress spread by 45 degree. Contact width is not considered.

Flange thickness at Loading Point, tf'	26.9	mm		
Effective width, b	88.5	mm	b=2*Lt	
Section Modulus, Zt	10673	mm3		
Area, A	2381	mm2	A=b*tf'	
Bending Stress, σ	70.5	N/mm2		OK
Shear Stress, τ	16.1	N/mm2		OK
Allowable Bending Stress, σa	175.0	N/mm2	σa=1.25*(140)	
Allowable Shear Stress, τa	100.0	N/mm2	τa=1.25*80	

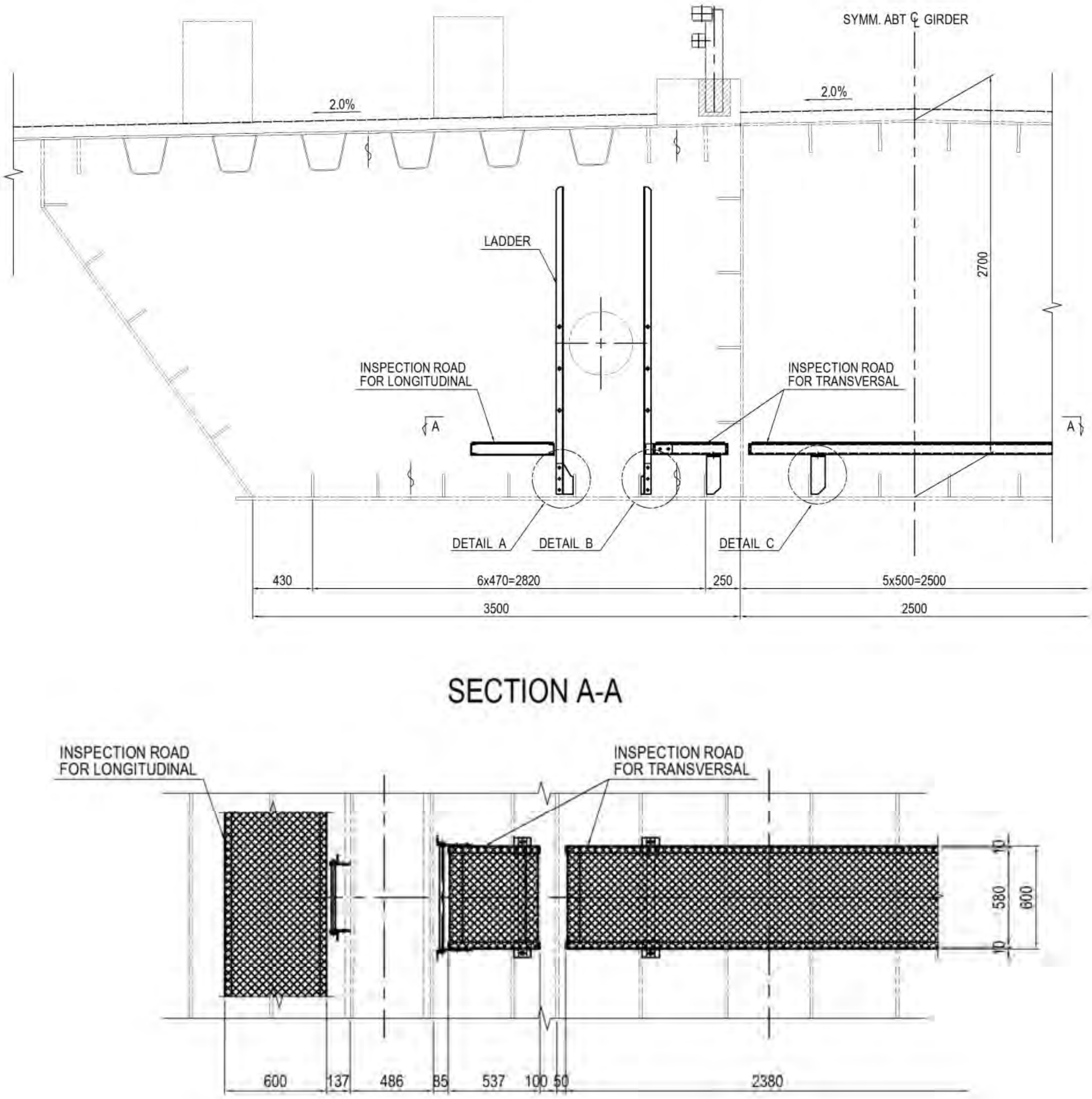
length of welding

a*Lreq=P/τa

welding size,s	7	mm	
a	6.1	mm	
Lreq	55.7377	mm	Lreq=P/(a*τa)

(3) Inspection Route inside Girder

The inspection route inside the girder is shown below.



Source: JICA Study Team

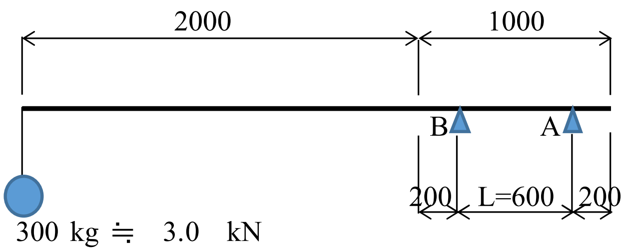
Figure 4.2.207 Inspection Route inside Girder

(4) Examination of Supporting Member of Gondola for Tower Outer Surface Inspection

1) Gondola Supporting Member

The weight of the gondola was assumed as 300kg. Accordingly, the cross-section of the supporting members and stiffeners of the supports were decided as shown below.

･Section Force



Source: JICA Study Team

Figure 4.2.208 Design Condition for Supporting Member

Reaction Force

R_A = -P × a / L = -3.0 × 2.200 / 0.600
= -11.0 kN

R_B = P × a / L = 3.0 × (0.600 + 2.200) / 0.600
= 14.0 kN

Bending Moment, Shear

M = -P × a = -3.0 × 2.2
= -6.6 kN･m

S = R_A = -11.0 kN

･Examination of applied cross-section

Applied cross-section

H - 250 × 250 × 9 × 14 … Z = 860 cm^3

Verification of bending stress

σ = M / Z
= -6.6 × 10 ^6 / 860 × 10 ^3
= -7.7 N/mm^2 < σ_a = 140 N/mm^2

Verification of shear stress

Aw = 9 × (250 - 2 × 14) = 1998 mm^2

τ = S / Aw = 11.0 × 10 ^3 / 1998
= 5.5 N/mm^2 < τ_a = 80 N/mm^2

･Stiffners for supports

Applied cross-section

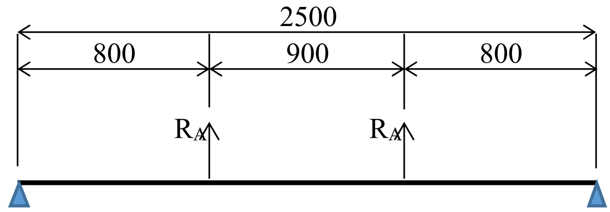
V-Stiff PL 2- 100 × 9

Verification of stress

σ_a = R_B / A_b = 14.0 × 10 ^3 / (2 × 100 × 9)
= 7.8 N/mm^2

2) Reinforcement at Tower Side

・Section Force



$M = R_A \times 0.800$
$= -8.8$ kN・m
$S = R_A = -11.0$ kN

Source: JICA Study Team

Figure 4.2.209 Design Condition for Reinforcement at Tower Side

・Necessary section modulus at reinforced section

Section force shall equal that of gondola suspension member

Shall be adjusted to equal H - 250 × 250 × 9 × 14 … Z = 860 cm^3

・Stiffners for supports

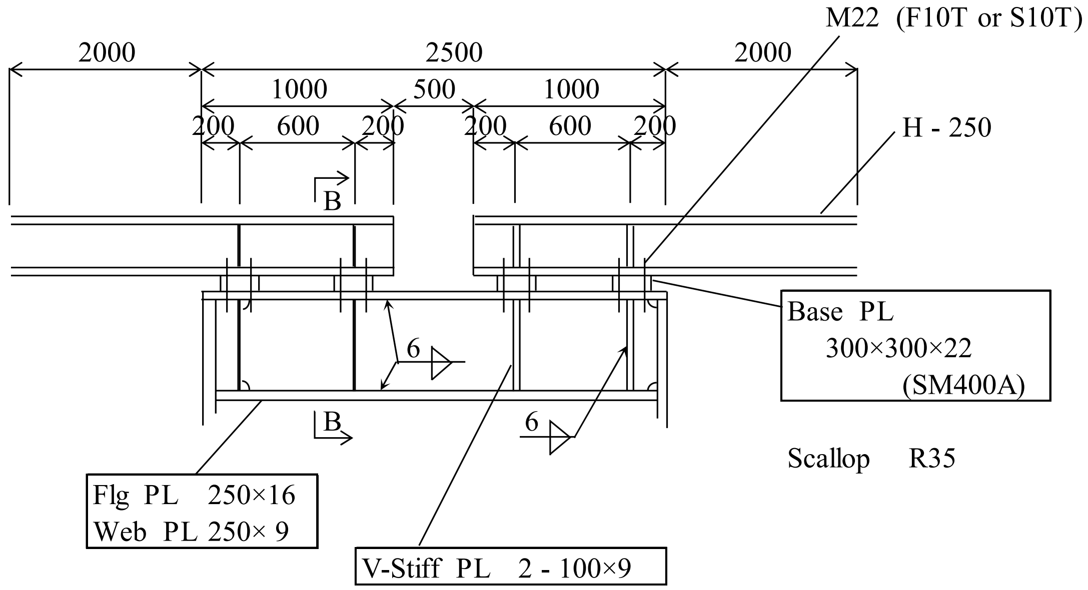
Applied cross-section

V-Stiff PL 2- 100 × 9

Verification of stress

$\sigma_a = R_B / A_b =$ 14.0 × 10 ^3 / (2 × 100 × 9)
$=$ 7.8 N/mm^2

・Configuration

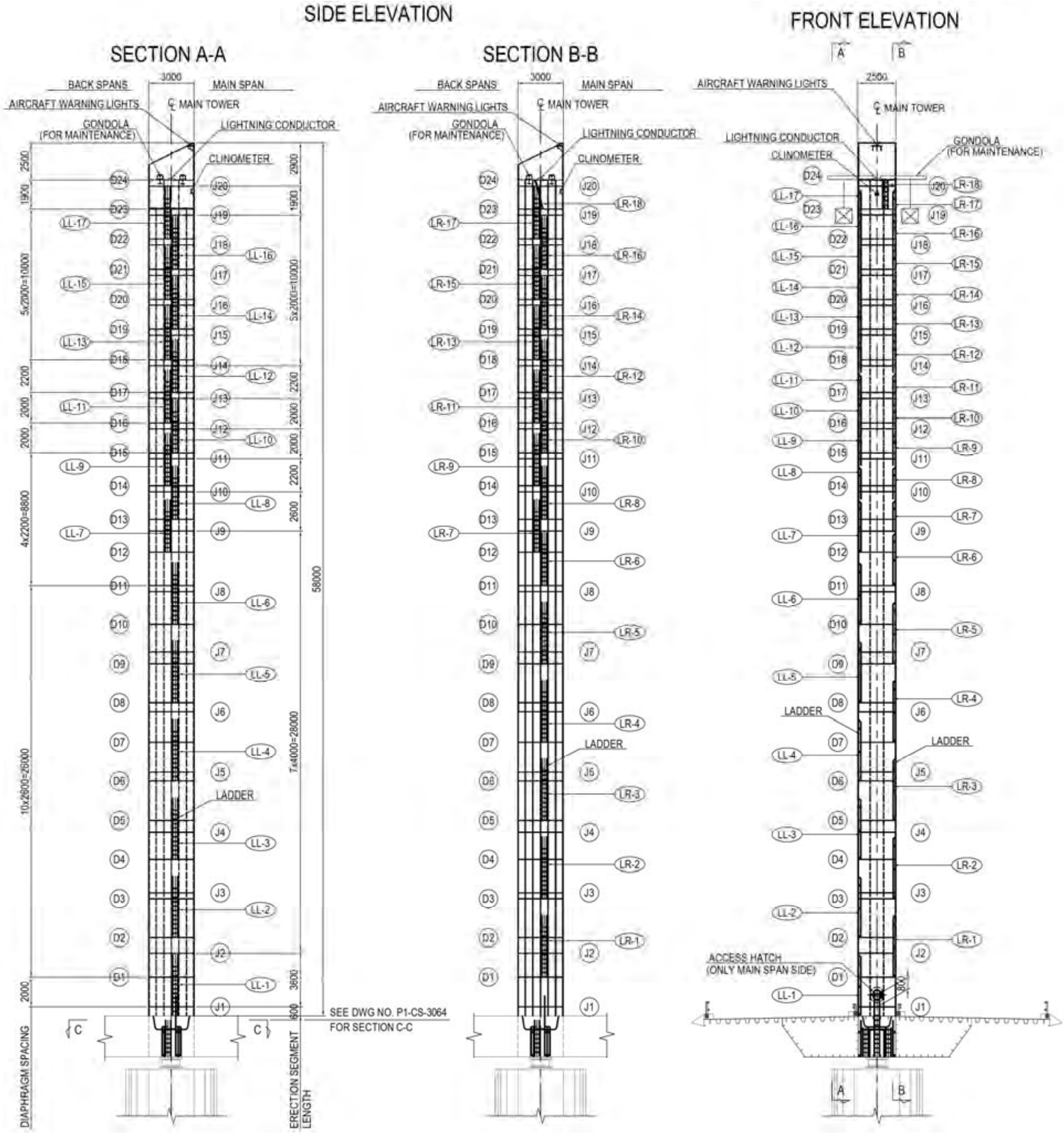


Source: JICA Study Team

Figure 4.2.210 Tower Side Reinforcement Plan

(5) Inspection Route inside Tower

The shaft ladders inside the tower is as shown below.

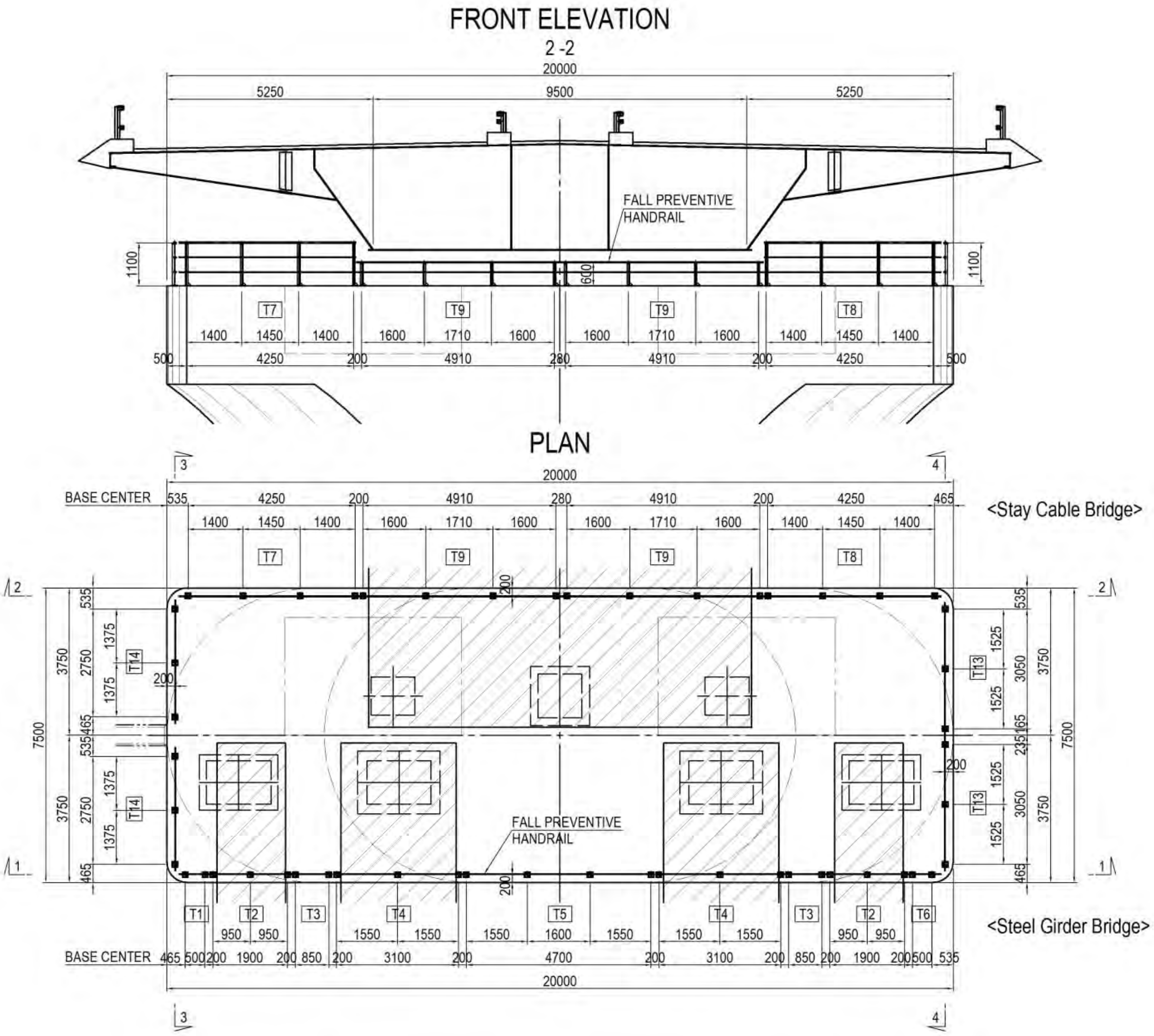


Source: JICA Study Team

Figure 4.2.211 Shaft Ladders inside Towers

(6) Fall Preventive Handrail at Pier Top

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



Source: JICA Study Team

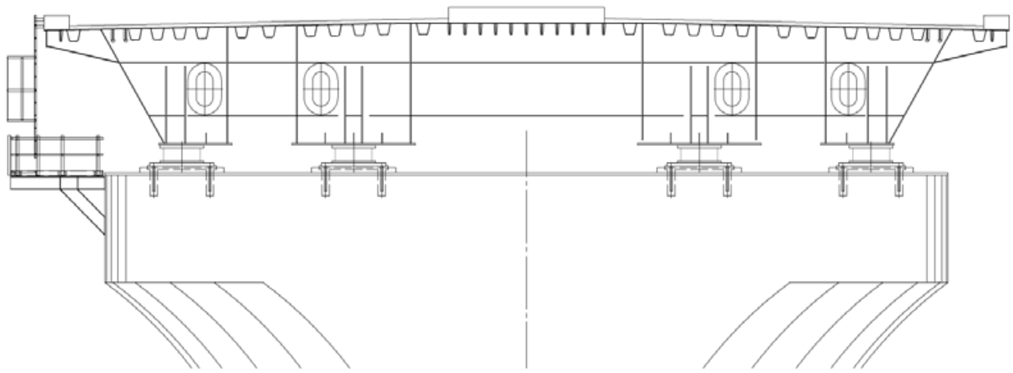
Figure 4.2.212 Fall Preventive Handrail at Pier Top

(7) Shaft Ladder

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

1) Side Pier

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

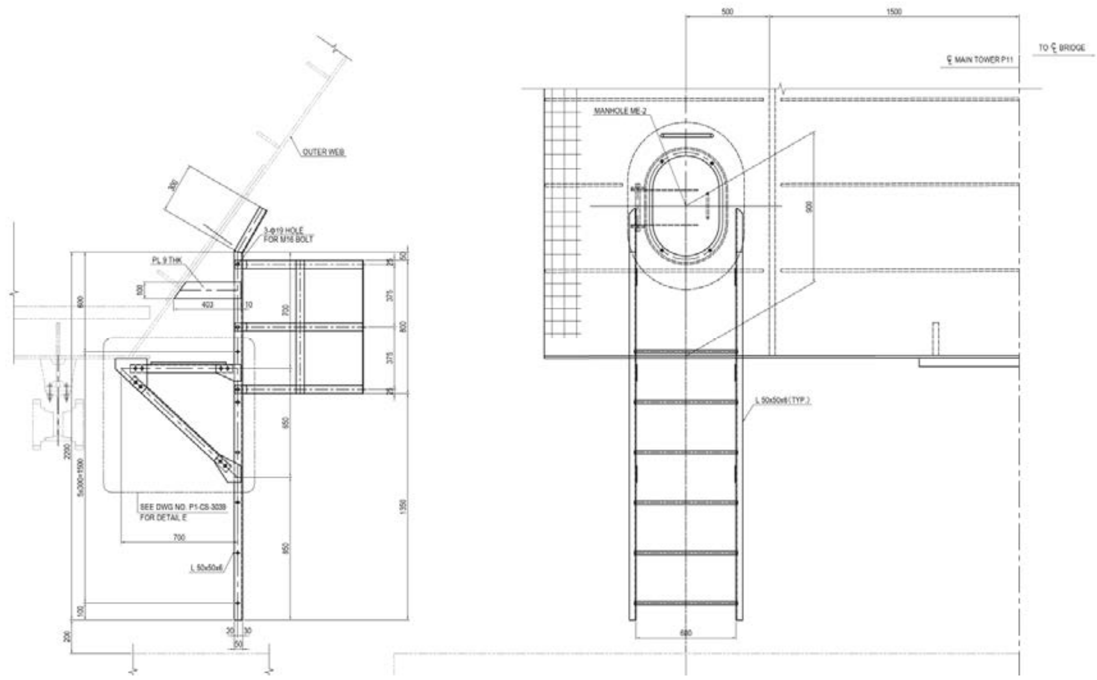


Source: JICA Study Team

Figure 4.2.213 Shaft Ladder (Side Pier)

2) Tower Pier

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



Source: JICA Study Team

Figure 4.2.214 Shaft Ladder (Tower Pier)

4.2.12 Summary of Seismic Analysis

4.2.12.1 Dynamic Analysis of Overall Structure

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

(1) Non-Linear Dynamic Analysis Conditions

1) Outline of Structure

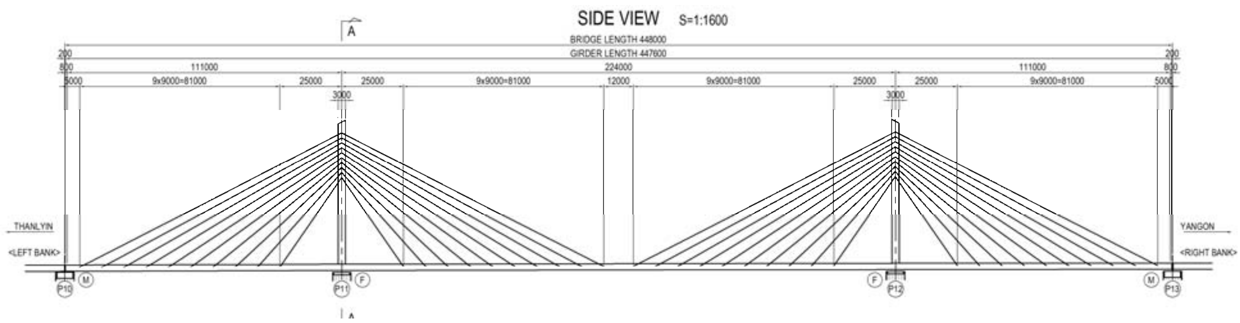
a) Structure Type

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

b) Bearing Support Condition

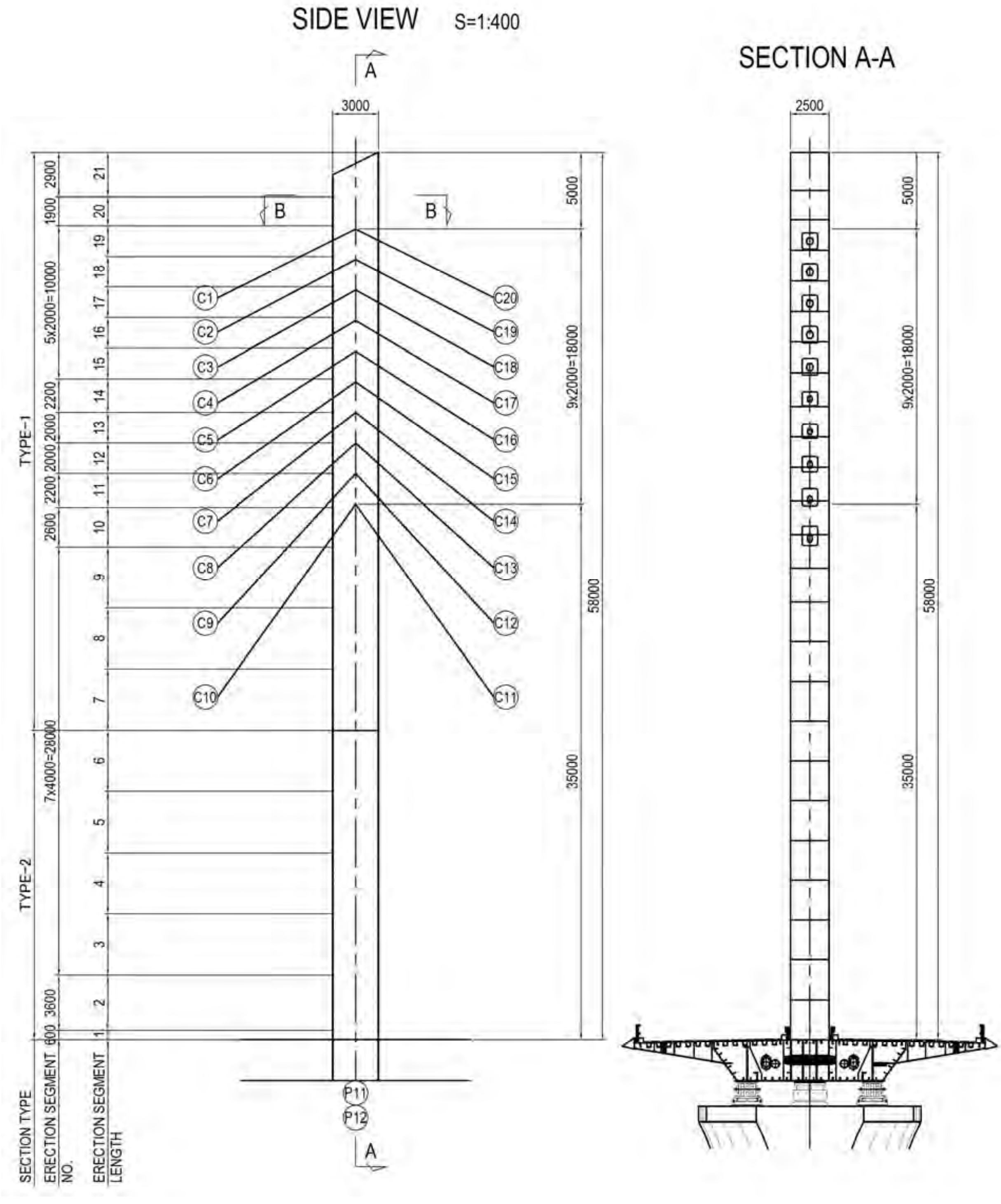
- P10 Pier: Movable	(Fixed on transverse direction) Rocking Bearing
- P11 Pier: Fixed	(Fixed on transverse direction)
- P12 Pier: Fixed	(Fixed on transverse direction)
- P13 Pier: Movable	(Fixed on transverse direction) Rocking Bearing

c) Structural Plan



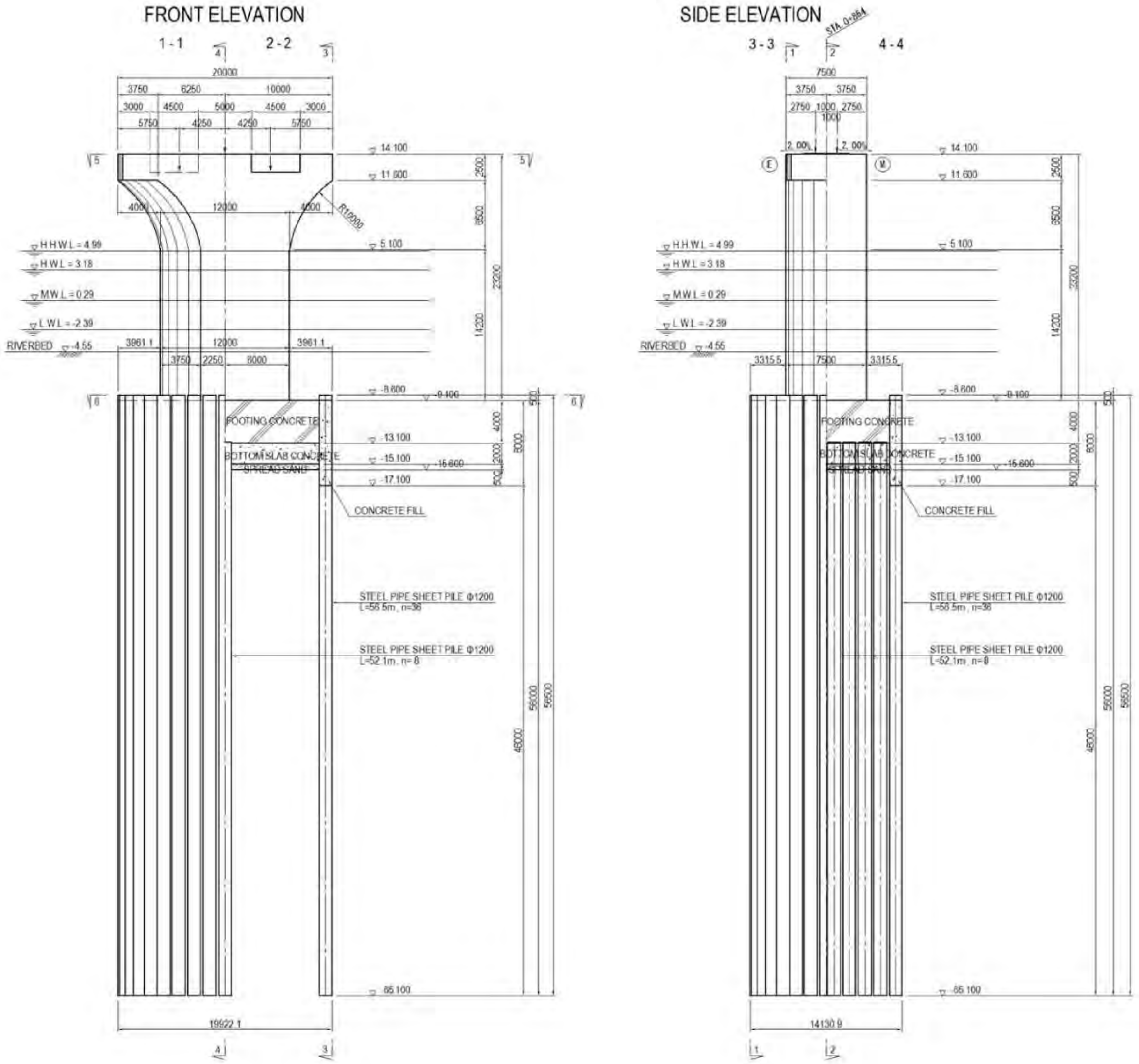
Source: JICA Study Team

Figure 4.2.215 Structural Plan of Superstructure



Source: JICA Study Team

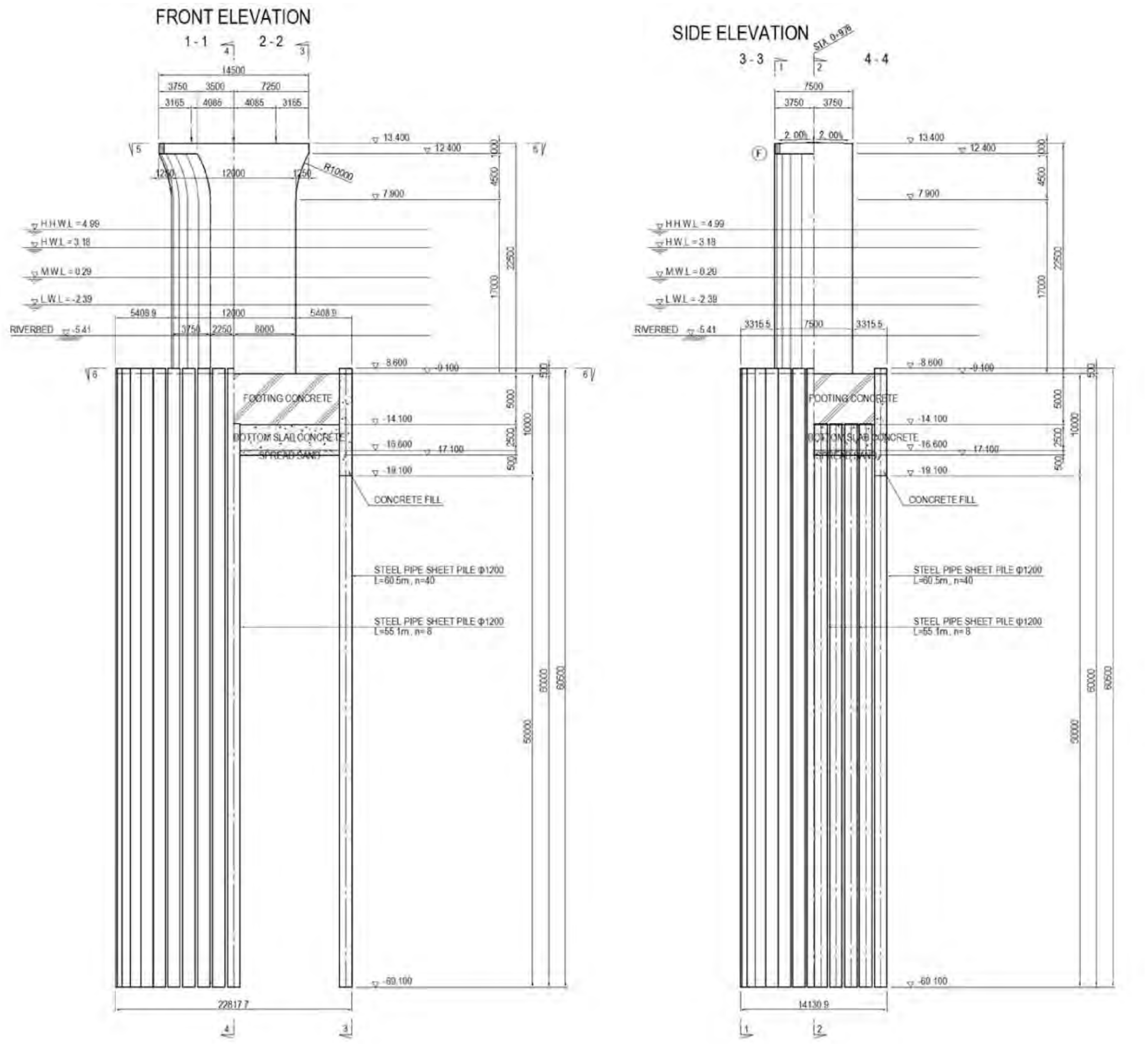
Figure 4.2.216 Structural Plan of Main Tower



Source: JICA Study Team

Figure 4.2.217 Structural Plan of P10 Pier

The figure above shows P10 pier. P13 pier has the same column dimension except for the embedded footing length, which was changed from 56.5 m to 49.5 m.



Source: JICA Study Team

Figure 4.2.218 Structural Plan of P11 Pier

The figure above shows P11 pier. P12 pier has the same column dimension except for the embedded footing length, which was changed from 60.5 m to 52.0 m.

2) Basic Policy of Models

a) Analysis Model

The one mass point spring SR model shall be used.

b) Excitation Method

Acceleration for the excitation of the foundation is inputted.

c) Response Calculation

1) Integral time : 0.02 seconds

2) Integration method : Newmark β Method (β = 0.25)

d) Effect of Gravity

The section force and the cable pre-stress induced by gravity are set as the initial stage section force and included in the first step of the time response analysis.

e) Internal Damping · Radiational Damping

Rayleigh's damping shall be applied. For the configuration of Rayleigh's damping, the coefficients for the vibrational modes shall be specified as stated in the Specification of Highway Bridges V and shall be determined by the following equation:

$$h_i = \frac{\sum_{j=1}^{n} h_j \{\phi_{ij}\}^T [K_j] \{\phi_{ij}\}}{\{\phi_i\}^T [K] \{\phi_i\}}$$

$\{\phi_{ij}\}$; Mode Vector of element j at ith mode

h_j ; Dumping coefficient of element j

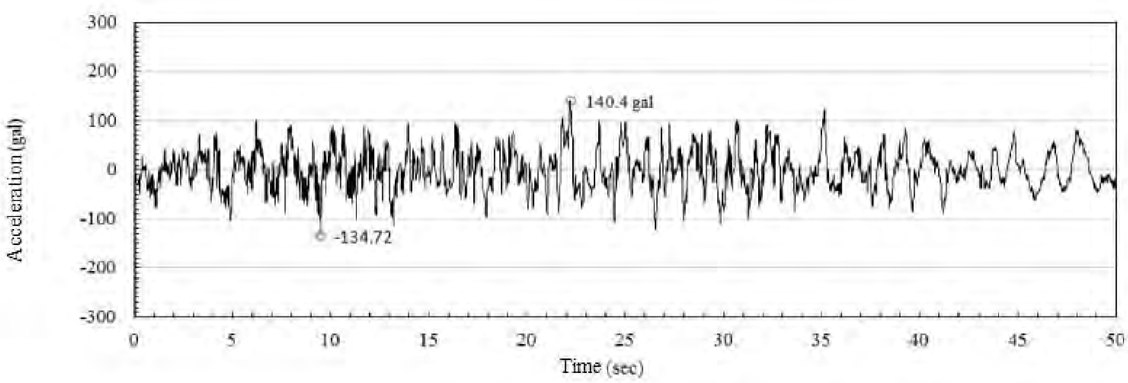
$[K_j]$; Stiffness matrix of element j

$\{\phi_i\}$; Mode Vector of whole structure j at ith mode

$[K]$; Stiffness matrix of whole structure

3) Design Seismic Wave

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

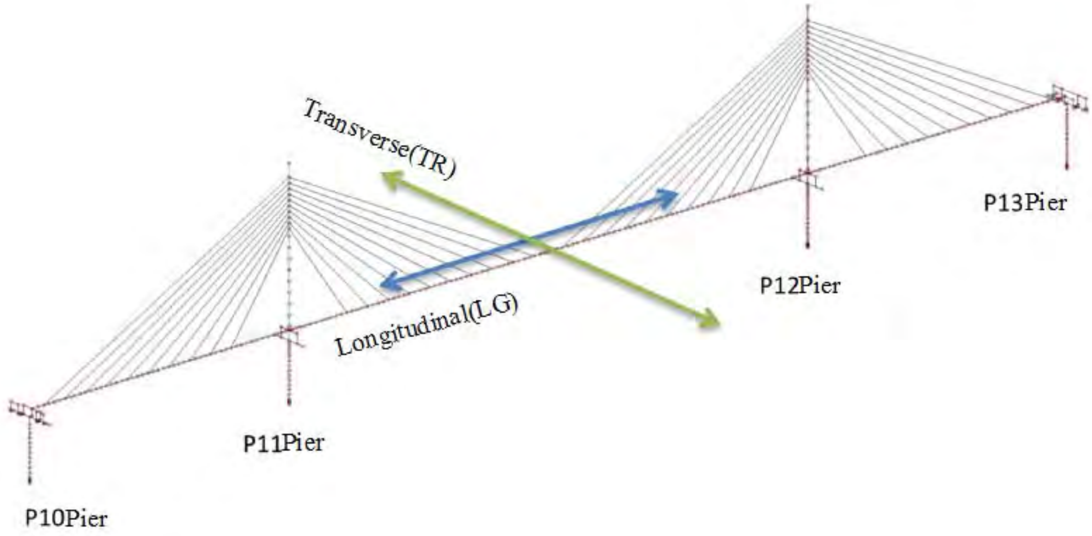


Source: JICA Study Team

Figure 4.2.219 Design Seismic Wave

4) Analysis Direction

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



Source: JICA Study Team

Figure 4.2.220 Analysis Direction for Dynamic Analysis

5) Evaluation Method for the Dynamic Analysis Results

a) Evaluation of Superstructure

The main girder and main column are verified to not undergo plasticization due to the seismic response section force. Furthermore, the response at the joint gap at the girder end and bearing support is verified to be below the allowable value.

b) Evaluation of Cable Member

The tension in the cable due the seismic response is verified to be below the allowable value. Furthermore, it is verified that no compression acts on the cable.

c) Evaluation of Pier

- Flexural capacity

The bending stress on the reinforced concrete member generated by the bending moment due to the seismic response is verified to be below the allowable bending stress.

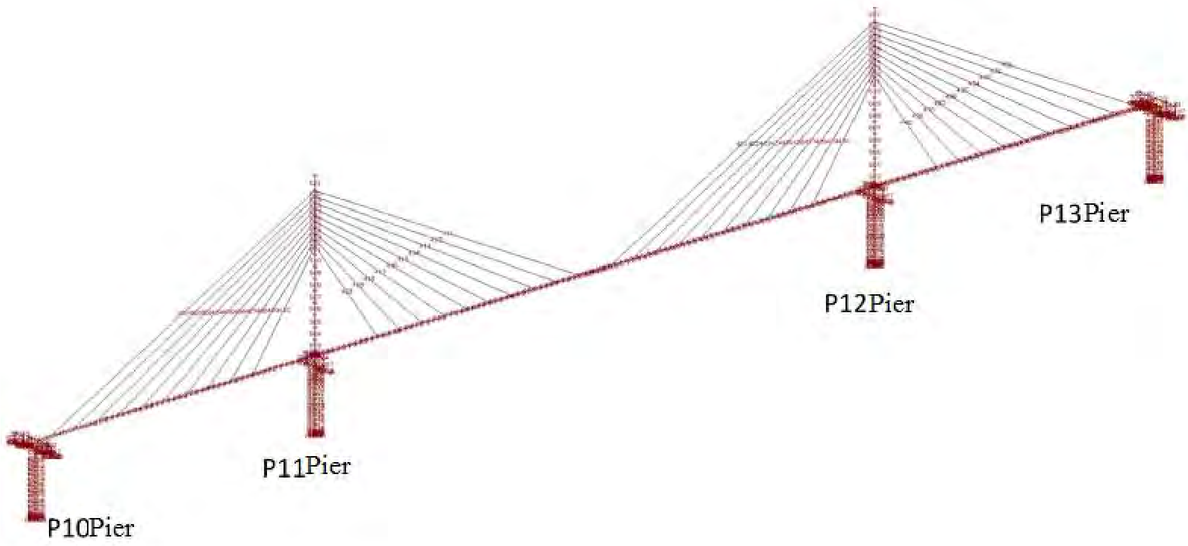
- Shear capacity

The shearing stress on the concrete generated by the shear stress due to the seismic response is verified to be below the allowable shearing stress.

(2) Analysis Model

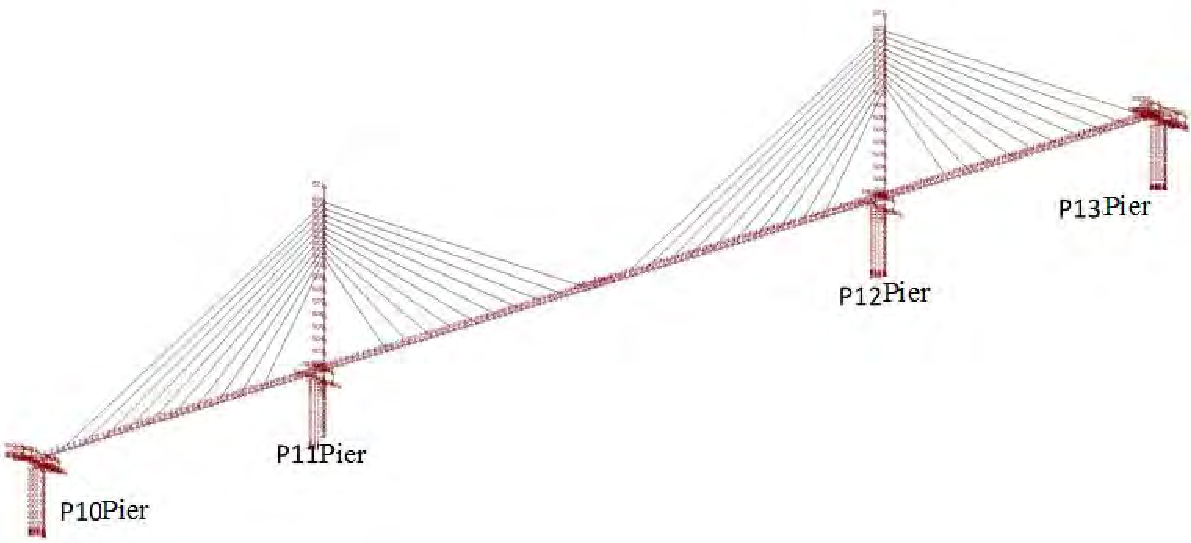
1) Analysis Model

The analysis model is a 3D model (6 degrees of freedom) of the entire bridge system.



Source: JICA Study Team

Figure 4.2.221 Node Numbers



Source: JICA Study Team

Figure 4.2.222 Element Numbers

2) Models of Members

a) Superstructure

The superstructure shall be modelled as a linear beam member. An axis and a mass point shall be established at the centroid position of the superstructure, regardless of the analysis direction.

b) Bearing Support

The condition of the bearing support shall be as listed below.

Table 4.2.163 Bearing Support Condition

Pier Number	Longitudinal	Transverse
Pier P10	Movable	Fixed
Pier P11	Fixed	Fixed
Pier P12	Fixed	Fixed
Pier P13	Movable	Fixed

Source: JICA Study Team

Table 4.2.164 Models of Bearing Support

Bearing Support Condition	Longitudinal	Transverse	Vertical	Longi.-axis Rotation	Trans.-axis Rotation	Vertical-axis Rotation
Movable Support	Free	Restricted	Restricted	Restricted	Free	Free
Fixed Support	Restricted	Restricted	Restricted	Restricted	Free	Free

Source: JICA Study Team

c) RC Pier

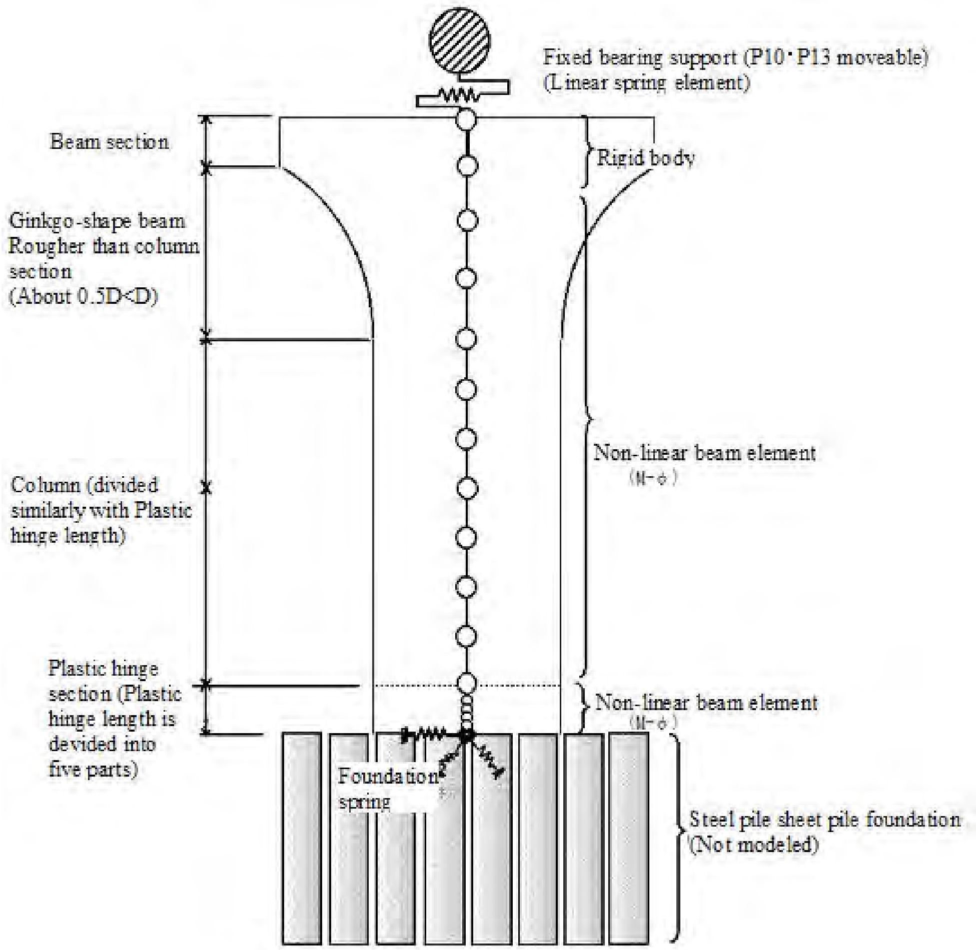
Plastic hinge member : The plastic hinge section shall be considered as a non-linear beam element and the length shall be divided into five equal parts.

Ordinary member : Non-linear beam element

d) Foundation

The effect of the ground/foundation shall be replaced by a linear concentrated spring. The concentrated spring shall consider rotation and horizontal couple.

The overall model of the pier and foundation is shown below.



Source: JICA Study Team

Figure 4.2.223 Model of Pier and Foundation

3) Damping

a) Hysteresis Damping

Hysteresis damping is considered automatically in the dynamic analysis.

b) Damping Coefficient of Structural Elements

Table 4.2.165 Damping Coefficients of Structural Elements

Structural Member			Dumping Coef	Remarks
Superstructure	Steel		2%	Steel Structure Linear Member is 2%
	Cable		1%	Cable is 1%
Substructure	RC Pier	Linear	5%	Linear Member is 5%
		Non-linear	1%	Non-linear Member is 5%
Foundation	Ground Type Ⅲ		20%	Steel Pipe Sheet Pile Foundation
Bearing support	Fixed Bearing Support		0%	Fixed Bearing Support is 0%
	Movable Bearing Support		0%	Movable Bearing Support is 0%

Source: JICA Study Team

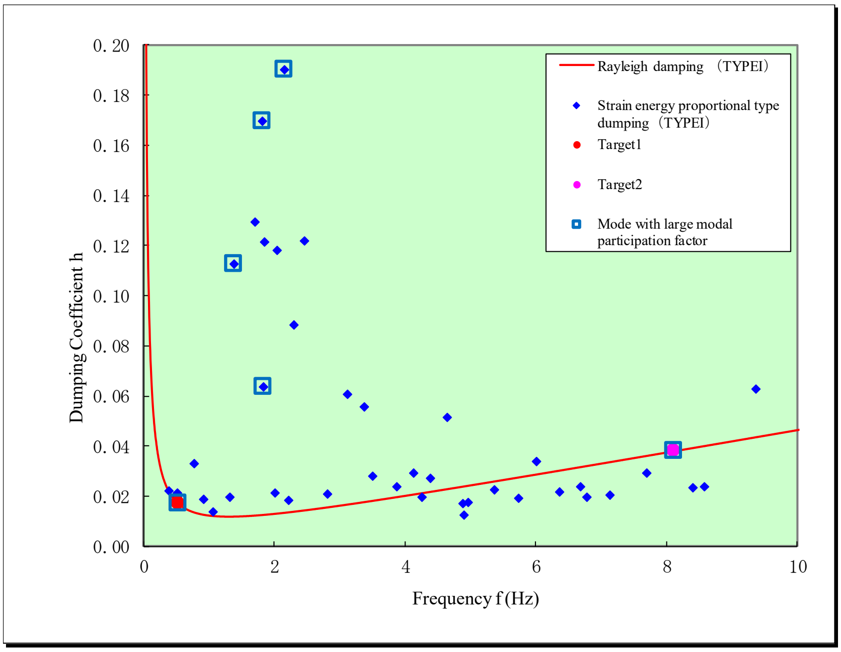
c) Internal Damping and Radiational Damping

Internal damping and radiational damping shall apply Rayleigh's damping and the damping coefficient for each member shall be obtained from the Specifications of Highway Bridges, Graph 7.3.1.

(3) Analysis Result

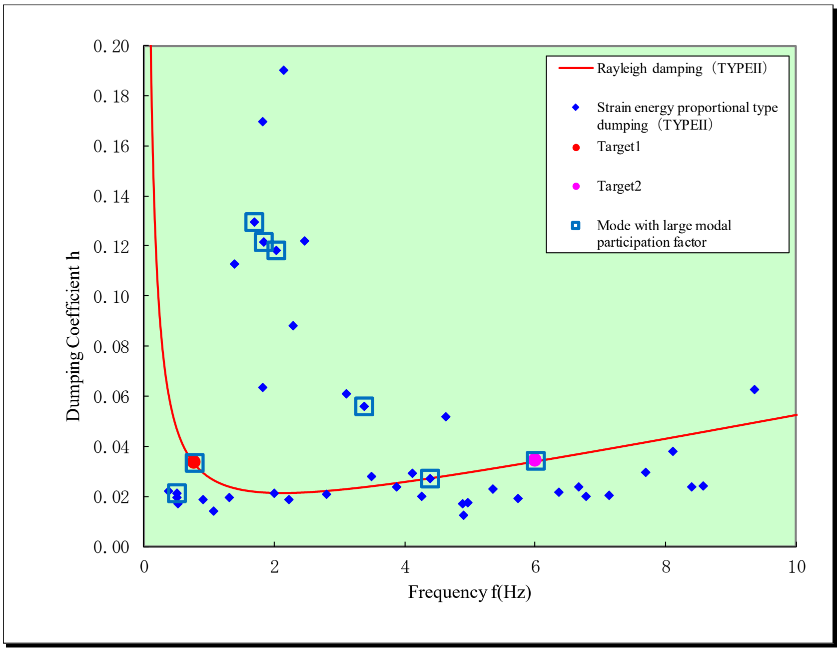
1) Natural Value Analysis

Rayleigh's damping was configured with two selected natural vibration modes which showed a clear distinction between the natural frequency and damping coefficient of the natural vibration mode in the focused direction.



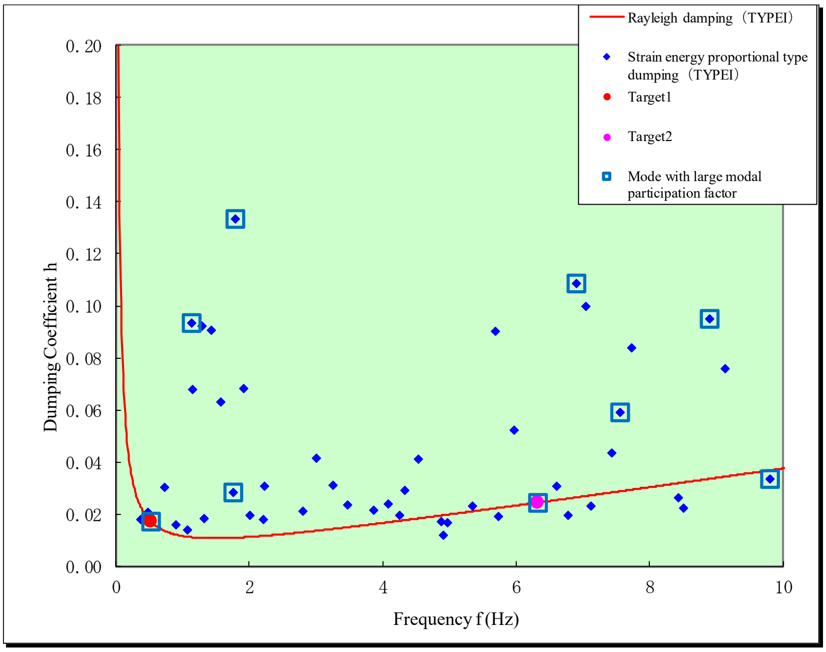
Source: JICA Study Team

Figure 4.2.224 Rayleigh Damping (Whole Cross Section Stiffness – Longitudinal)



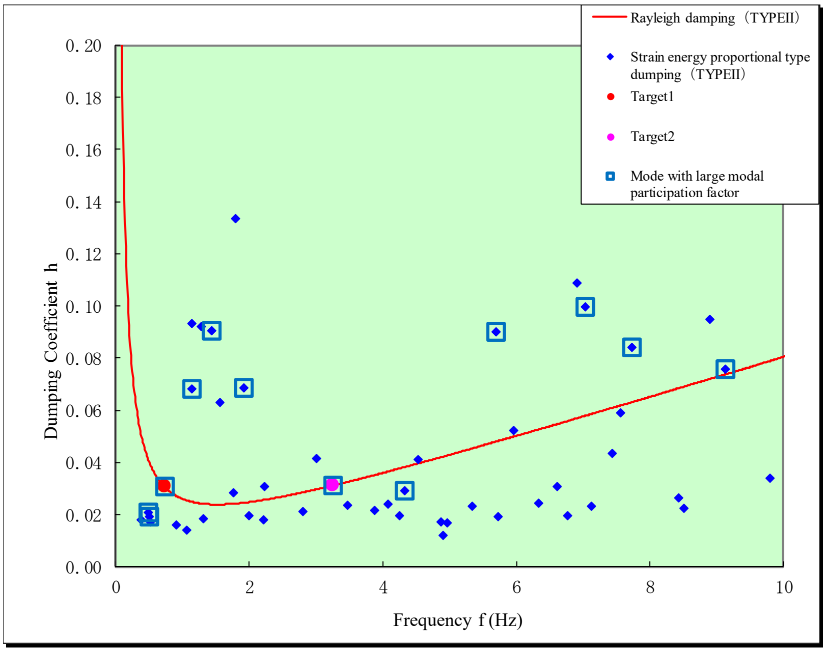
Source: JICA Study Team

Figure 4.2.225 Pier and Foundation Model (Whole Cross Section Stiffness – Transverse)



Source: JICA Study Team

Figure 4.2.226 Rayleigh Damping (Yield Stiffness – Longitudinal)

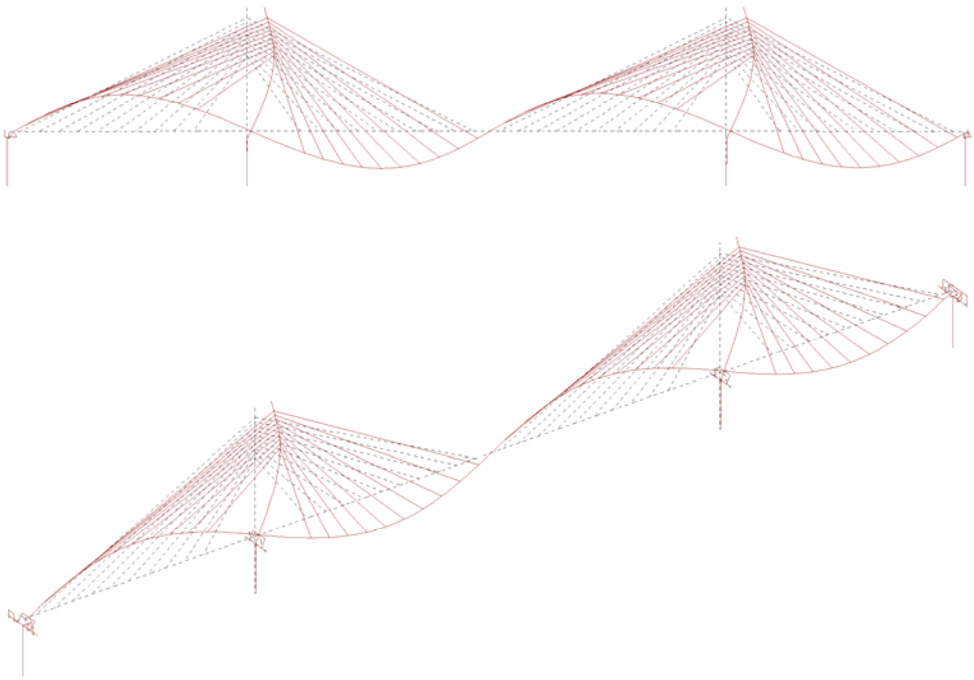


Source: JICA Study Team

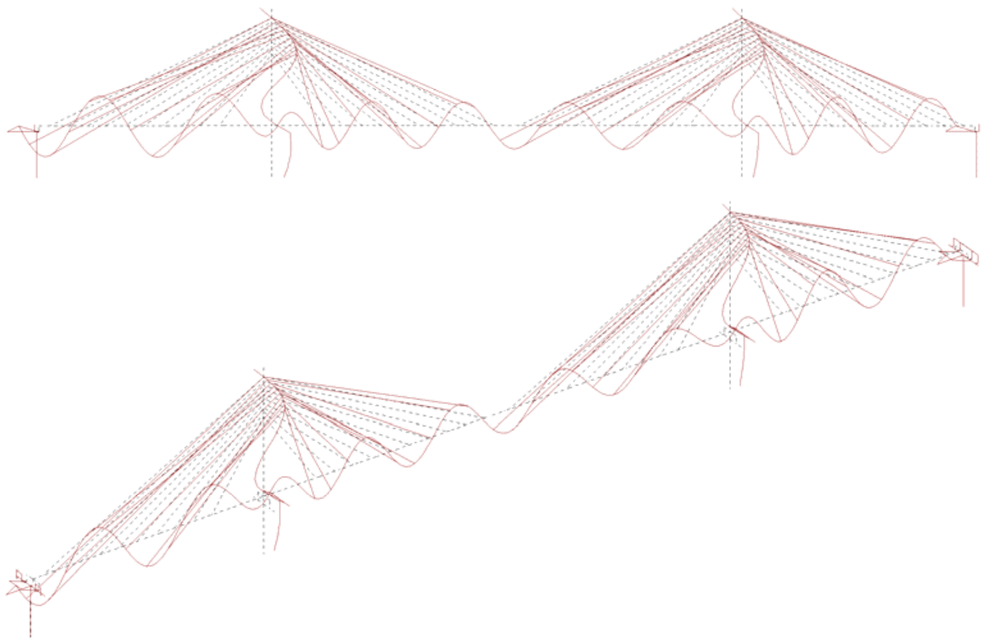
Figure 4.2.227 Rayleigh Damping (Yield Surface Stiffness – Transverse)

The fundamental natural frequency mode for each analysis model is shown below.

4^{th} Mode F = 0.527 [Hz]



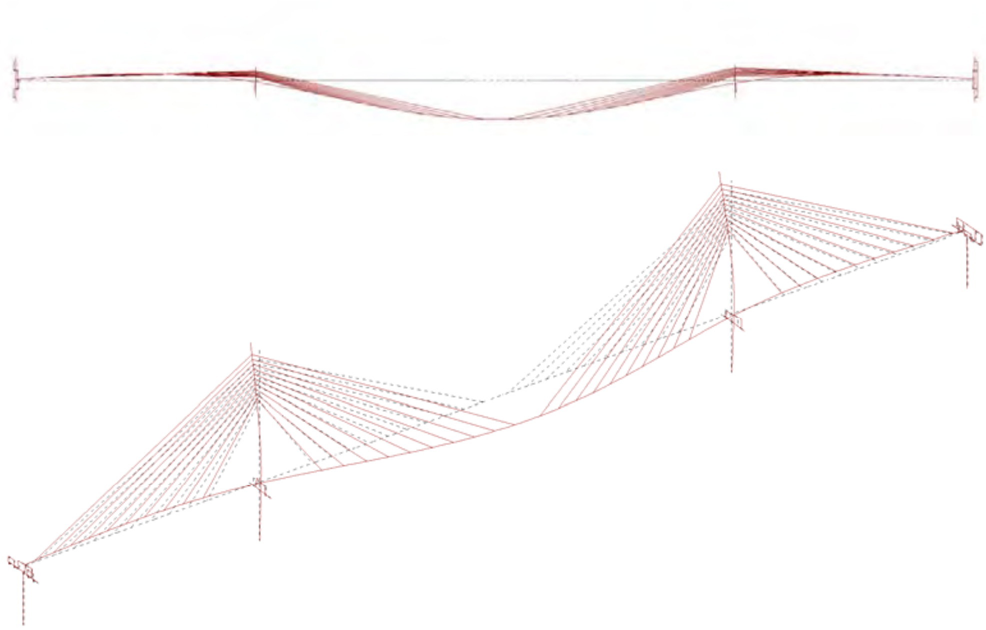
40^{th} Mode F = 8.118 [Hz]



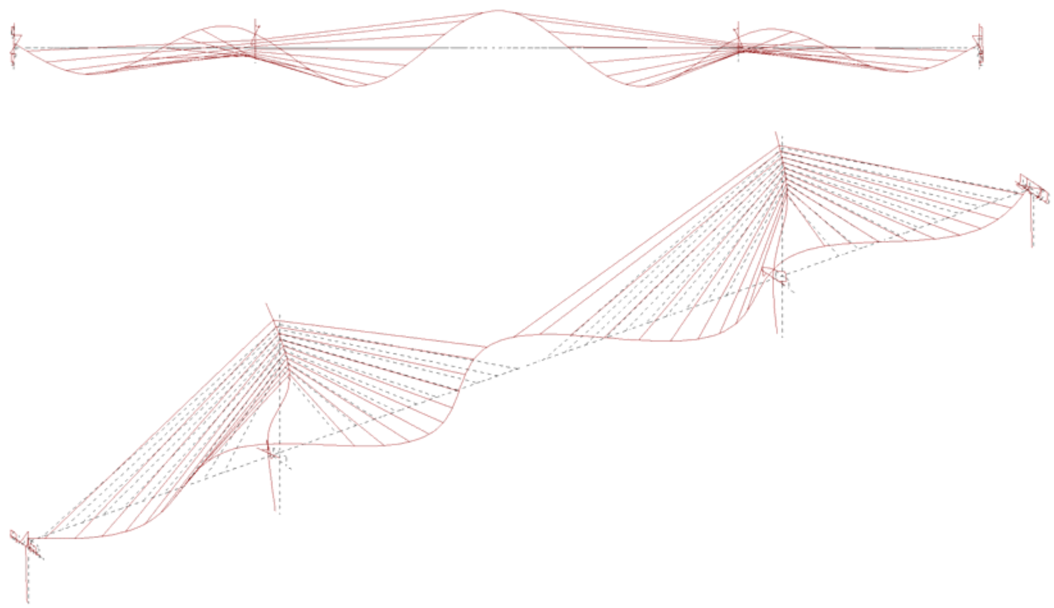
Source: JICA Study Team

Figure 4.2.228 Fundamental Vibration Mode (Whole Cross Section Stiffness – Transverse)

5th Mode F = 0.777 [Hz]



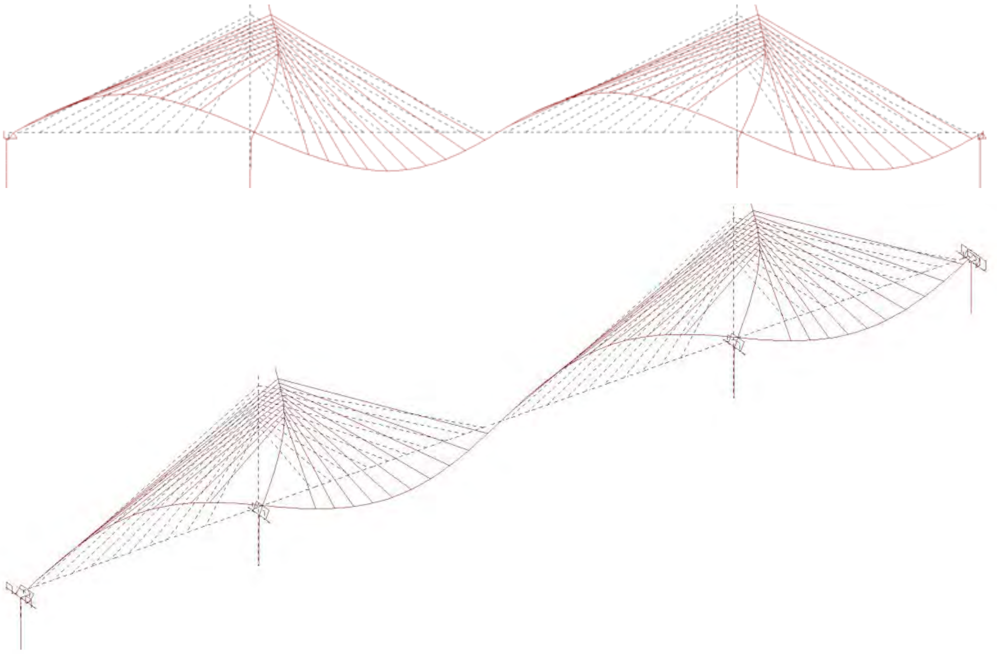
34th Mode F = 6.011 [Hz]



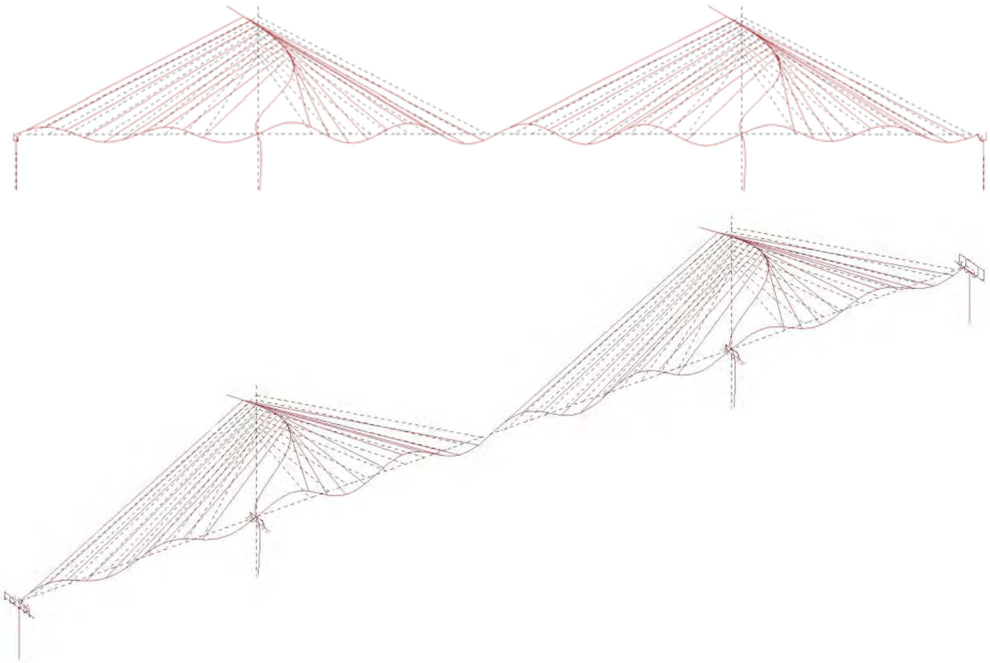
Source: JICA Study Team

Figure 4.2.229 Fundamental Vibration Mode (Whole Cross Section Stiffness – Longitudinal)

4th Mode F = 0.526 [Hz]



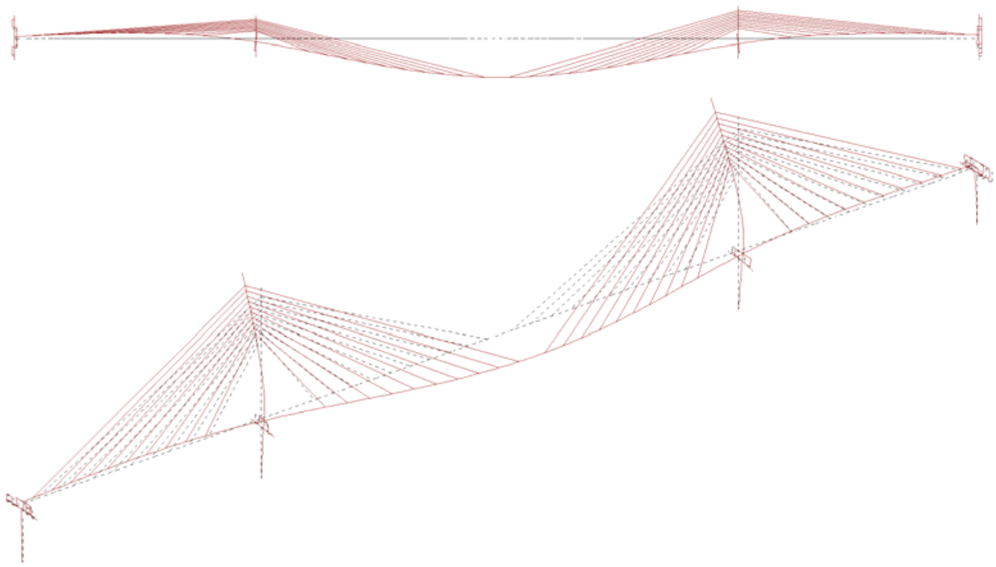
36th Mode F = 6.334 [Hz]



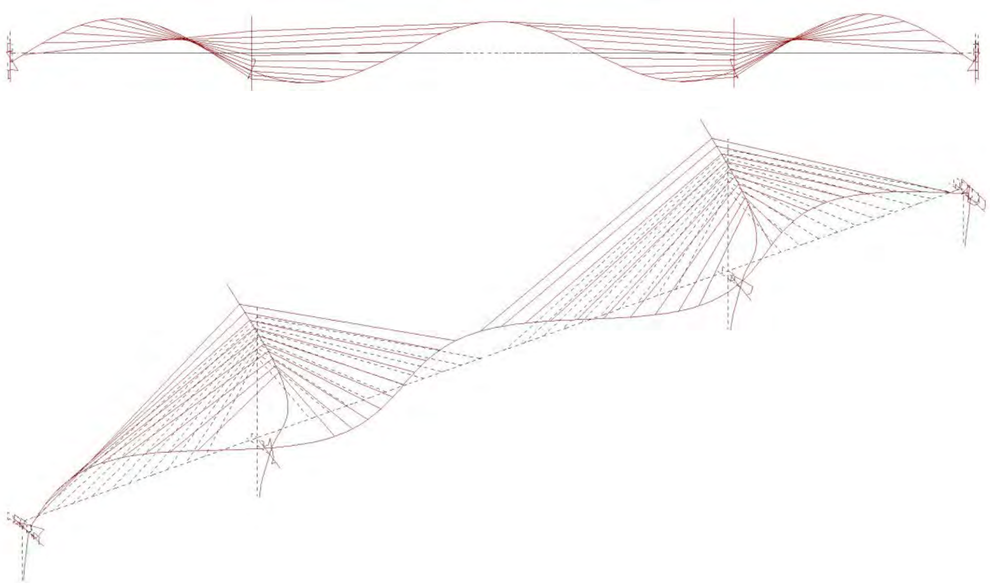
Source: JICA Study Team

Figure 4.2.230 Fundamental Vibration Mode (Yield Surface Stiffness – Longitudinal)

5th Mode F = 0.735 [Hz]



22th Mode F = 3.251 [Hz]



Source: JICA Study Team

Figure 4.2.231 Fundamental Vibration Mode (Yield Surface Stiffness – Transverse)

2) Evaluation of Members

a) Main Girder Cross Section Evaluation

The seismic response value was verified to be below the allowable value, as shown below.

Table 4.2.166 Main Girder Cross Section Evaluation Result 1-2

Main Girder Cross-Section 1

Stress of Members										
Member Number	Position	σ		σa	τ		τa	F		Fa
DECK 7 (EFT)	5097	-66.2	<	108.8	0.2	<	80.0	0.22	<	1.2
DECK 6 (W.R)	0	-33.2	<	108.8	10.5	<	80.0	0.07	<	1.2
DECK 7 (EFT)	5097	-66.2	<	108.8	0.2	<	80.0	0.22	<	1.2
BOTM 12 (W.L)	3500	-78.5	<	156.8	8.1	<	120.0	0.14	<	1.2
BOTM 12 (W.L)	3500	-78.5	<	156.8	8.1	<	120.0	0.14	<	1.2
BOTM 12 (W.L)	3500	-78.5	<	156.8	8.1	<	120.0	0.14	<	1.2
WEB 17 (W.L)	2962	-78.2	<	116.3	8.3	<	120.0	0.14	<	1.2
WEB 17 (W.C)	1481	-46.8	<	116.3	9.1	<	120.0	0.06	<	1.2
WEB 17 (W.L)	2962	-78.2	<	116.3	8.3	<	120.0	0.14	<	1.2
Safety Evaluation of Member										
Member Number	Position	K		Ka	σ		σcal			
DECK 7 (EFT)	5097	0.50	<	1.0	66.2	<	108.8			
BOTM 12 (W.L)	3500	0.39	<	1.0	78.5	<	156.8			
WEB 17 (W.L)	2962	0.42	<	1.0	78.2	<	116.3			
Vertical Rib Bottom Curb Stress										
Member Number	Rib Number	σ		σa						
DECK 2 (U.RIB)	2	36.6	<	140.0						
DECK 2 (RIB)	1	38.1	<	140.0						
DECK 6 (U.RIB)	29	-42.4	<	140.0						
DECK 6 (RIB)	30	-43.9	<	140.0						
DECK 7 (U.RIB)	37	-58.7	<	140.0						
DECK 7 (RIB)	38	-60.3	<	140.0						

Main Girder Cross-Section 2

Stress of Members										
Member Number	Position	σ		σa	τ		τa	F		Fa
DECK 2 (EFT)	-5066	-82.2	<	140.0	0.3	<	80.0	0.34	<	1.2
DECK 3 (W.L)	5001	-38.9	<	140.0	11.5	<	80.0	0.10	<	1.2
DECK 2 (EFT)	-5066	-82.2	<	140.0	0.3	<	80.0	0.34	<	1.2
BOTM 10 (W.R)	0	-67.6	<	114.7	11.4	<	120.0	0.11	<	1.2
BOTM 10 (W.R)	0	-67.6	<	114.7	11.4	<	120.0	0.11	<	1.2
BOTM 10 (W.R)	0	-67.6	<	114.7	11.4	<	120.0	0.11	<	1.2
WEB 14 (W.L)	2962	-67.5	<	116.3	9.1	<	120.0	0.11	<	1.2
WEB 14 (W.C)	1481	-53.0	<	116.3	10.6	<	120.0	0.07	<	1.2
WEB 14 (W.L)	2962	-67.5	<	116.3	9.1	<	120.0	0.11	<	1.2
Safety Evaluation of Member										
Member Number	Position	K		Ka	σ		σcal			
DECK 2 (EFT)	-5066	0.59	<	1.0	82.2	<	140.0			
BOTM 10 (W.R)	0	0.40	<	1.0	67.6	<	114.7			
WEB 14 (W.L)	2962	0.39	<	1.0	67.5	<	116.3			
Vertical Rib Bottom Curb Stress										
Member Number	Rib Number	σ		σa						
DECK 2 (U.RIB)	2	-74.2	<	140.0						
DECK 2 (RIB)	1	-76.5	<	140.0						
DECK 3 (U.RIB)	10	-52.8	<	140.0						
DECK 3 (RIB)	9	-54.7	<	140.0						
DECK 7 (U.RIB)	37	34.8	<	140.0						
DECK 7 (RIB)	38	37.2	<	140.0						

Source: JICA Study Team

Table 4.2.167 Main Girder Cross Section Evaluation Result 3-4

Main Girder Cross-Section 3

Stress of Members												
Member Number			Position	σ		σa	τ		τa	F		Fa
DECK	7	(EFT)	3422	-88.5	<	140.0	9.5	<	80.0	0.41	<	1.2
DECK	6	(W.R)	0	-40.0	<	140.0	27.8	<	80.0	0.20	<	1.2
DECK	7	(EFT)	3422	-88.5	<	140.0	9.5	<	80.0	0.41	<	1.2
BOTM	12	(W.L)	3500	-114.5	<	167.1	23.3	<	120.0	0.33	<	1.2
BOTM	12	(W.L)	3500	-114.5	<	167.1	23.3	<	120.0	0.33	<	1.2
BOTM	12	(W.L)	3500	-114.5	<	167.1	23.3	<	120.0	0.33	<	1.2
WEB	17	(W.L)	2962	-113.9	<	150.7	21.0	<	120.0	0.32	<	1.2
WEB	17	(W.C)	1481	-70.1	<	150.7	23.7	<	120.0	0.15	<	1.2
WEB	17	(W.L)	2962	-113.9	<	150.7	21.0	<	120.0	0.32	<	1.2
Safety Evaluation of Member												
Member Number			Position	K		Ka	σ		σcal			
DECK	7	(EFT)	3422	0.63	<	1.0	88.5	<	140.0			
BOTM	12	(W.L)	3500	0.58	<	1.0	114.6	<	167.1			
WEB	17	(W.L)	2962	0.59	<	1.0	114.0	<	150.7			
Vertical Rib Bottom Curb Stress												
Member Number			Rib Number	σ		σa						
DECK	2	(U.RIB)	2	60.1	<	140.0						
DECK	2	(RIB)	1	62.6	<	140.0						
DECK	6	(U.RIB)	29	-62.8	<	140.0						
DECK	6	(RIB)	30	-65.6	<	140.0						
DECK	7	(U.RIB)	37	-91.5	<	140.0						
DECK	7	(RIB)	38	-94.6	<	140.0						

Main Girder Cross-Section 4

Stress of Members												
Member Number			Rib Number	σ		σa	τ		τa	F		Fa
DECK	7	(EFT)	3422	-87.7	<	140.0	6.7	<	80.0	0.40	<	1.2
DECK	6	(W.R)	0	-49.5	<	140.0	19.7	<	80.0	0.19	<	1.2
DECK	7	(EFT)	3422	-87.7	<	140.0	6.7	<	80.0	0.40	<	1.2
BOTM	12	(W.L)	3500	-88.9	<	114.7	18.1	<	120.0	0.20	<	1.2
BOTM	12	(W.L)	3500	-88.9	<	114.7	18.1	<	120.0	0.20	<	1.2
BOTM	12	(W.L)	3500	-88.9	<	114.7	18.1	<	120.0	0.20	<	1.2
WEB	17	(W.L)	2962	-88.7	<	116.3	14.6	<	120.0	0.19	<	1.2
WEB	17	(W.C)	1481	-64.9	<	116.3	17.0	<	120.0	0.12	<	1.2
WEB	17	(W.L)	2962	-88.7	<	116.3	14.6	<	120.0	0.19	<	1.2
Safety Evaluation of Member												
Member Number			Position	K		Ka	σ		σcal			
DECK	7	(EFT)	3422	0.63	<	1.0	87.7	<	140.0			
BOTM	12	(W.L)	3500	0.53	<	1.0	89.0	<	114.7			
WEB	17	(W.L)	2962	0.53	<	1.0	88.8	<	116.3			
Vertical Rib Bottom Curb Stress												
Member Number			Rib Number	σ		σa						
DECK	2	(U.RIB)	8	-37.2	<	140.0						
DECK	2	(RIB)	1	36.0	<	140.0						
DECK	6	(U.RIB)	29	-64.6	<	140.0						
DECK	6	(RIB)	30	-66.6	<	140.0						
DECK	7	(U.RIB)	37	-87.2	<	140.0						
DECK	7	(RIB)	38	-89.5	<	140.0						

Source: JICA Study Team

b) Main Tower Cross Section Evaluation

The seismic response value was verified to be below the allowable value, as shown below.

Table 4.2.168 Main Tower Cross Section Evaluation

Main Tower Cross Section 1

Stress of Members												
Member Number			Position	σ		σa	τ		τa	F		Fa
U.FLG	1	(F.R)	2420	-113.8	<	210.0	3.3	<	120.0	0.29	<	1.2
U.FLG	1	(F.R)	2420	-113.8	<	210.0	3.3	<	120.0	0.29	<	1.2
U.FLG	1	(F.R)	2420	-113.8	<	210.0	3.3	<	120.0	0.29	<	1.2
L.FLG	2	(F.R)	2420	-123.7	<	210.0	3.3	<	120.0	0.35	<	1.2
L.FLG	2	(F.R)	2420	-123.7	<	210.0	3.3	<	120.0	0.35	<	1.2
L.FLG	2	(F.R)	2420	-123.7	<	210.0	3.3	<	120.0	0.35	<	1.2
WEB	4	(W.L)	3000	-124.4	<	210.0	3.3	<	120.0	0.35	<	1.2
WEB	4	(W.L)	3000	-124.4	<	210.0	3.3	<	120.0	0.35	<	1.2
WEB	4	(W.L)	3000	-124.4	<	210.0	3.3	<	120.0	0.35	<	1.2
Safety Evaluation of Member												
Member Number			Position	K		Ka	σ		σcal			
U.FLG	1	(F.R)	2420	0.635	<	1.0	117.6	<	210.0			
L.FLG	2	(F.R)	2420	0.684	<	1.0	127.9	<	210.0			
WEB	4	(W.L)	3000	0.687	<	1.0	128.7	<	210.0			

Main Tower Cross Section 2 Main Tower Foundation

Stress of Members												
Member Number			Position	σ		σa	τ		τa	F		Fa
U.FLG	1	(F.L)	0	-169.6	<	206.6	4.5	<	120.0	0.65	<	1.2
U.FLG	1	(F.L)	0	-169.6	<	206.6	4.5	<	120.0	0.65	<	1.2
U.FLG	1	(F.L)	0	-169.6	<	206.6	4.5	<	120.0	0.65	<	1.2
L.FLG	2	(F.L)	0	-173.3	<	206.1	4.5	<	120.0	0.68	<	1.2
L.FLG	2	(F.L)	0	-173.3	<	206.1	4.5	<	120.0	0.68	<	1.2
L.FLG	2	(F.L)	0	-173.3	<	206.1	4.5	<	120.0	0.68	<	1.2
WEB	3	(W.L)	3000	-175.7	<	210.0	4.5	<	120.0	0.70	<	1.2
WEB	3	(W.L)	3000	-175.7	<	210.0	4.5	<	120.0	0.70	<	1.2
WEB	3	(W.L)	3000	-175.7	<	210.0	4.5	<	120.0	0.70	<	1.2
Safety Evaluation of Member												
Member Number			Position	K		Ka	σ		σcal			
U.FLG	1	(F.L)	0	0.955	<	1.0	181.1	<	206.6			
L.FLG	2	(F.L)	0	0.974	<	1.0	185.0	<	206.1			
WEB	3	(W.L)	3000	0.980	<	1.0	187.7	<	210.0			

Main Tower Cross Section 2 Cross Sectional Change

Stress of Members												
Member Number			Position	σ		σa	τ		τa	F		Fa
U.FLG	1	(F.R)	2430	-147.3	<	201.2	3.9	<	120.0	0.49	<	1.2
U.FLG	1	(F.R)	2430	-147.3	<	201.2	3.9	<	120.0	0.49	<	1.2
U.FLG	1	(F.R)	2430	-147.3	<	201.2	3.9	<	120.0	0.49	<	1.2
L.FLG	2	(F.R)	2430	-153.9	<	200.7	3.9	<	120.0	0.54	<	1.2
L.FLG	2	(F.R)	2430	-153.9	<	200.7	3.9	<	120.0	0.54	<	1.2
L.FLG	2	(F.R)	2430	-153.9	<	200.7	3.9	<	120.0	0.54	<	1.2
WEB	4	(W.L)	3000	-155.4	<	210.0	3.9	<	120.0	0.55	<	1.2
WEB	4	(W.L)	3000	-155.4	<	210.0	3.9	<	120.0	0.55	<	1.2
WEB	4	(W.L)	3000	-155.4	<	210.0	3.9	<	120.0	0.55	<	1.2
Safety Evaluation of Member												
Member Number			Position	K		Ka	σ		σcal			
U.FLG	1	(F.R)	2430	0.848	<	1.0	156.9	<	201.2			
L.FLG	2	(F.R)	2430	0.884	<	1.0	164.2	<	200.7			
WEB	4	(W.L)	3000	0.874	<	1.0	165.8	<	210.0			

Source: JICA Study Team

c) Cable Evaluation

The tension in the cable due to the seismic response is verified to be below the allowable value. Furthermore, it is verified that no compression acts on the cable.

Table 4.2.169 Cable Evaluation Results

■ Evaluation of Cable

	Element Number	Response Axial Force of Cable（kN）				Evaluation of Maximum Tension			Evaluation of Minimum Tension		
		Longitudinal Analysis		Transverse Analysis		Max Tension	Allowable Tension	Verdict	Max Tension	Allowable Compression	Verdict
		Max	Min	Max	Min	(kN)	(kN)		(kN)	(kN)	
P11 Main tower side span side (Top→Bottom)	401	5103.3	3758.2	4390.8	4390.8	5103.3	7308.0	○	3758.2	0.0	○
	402	4588.0	3689.3	4101.8	4101.8	4588.0	7308.0	○	3689.3	0.0	○
	403	4262.0	3560.0	3881.0	3881.0	4262.0	7308.0	○	3560.0	0.0	○
	404	4340.8	3383.2	3831.0	3831.0	4340.8	7308.0	○	3383.2	0.0	○
	405	4737.3	3535.0	4074.6	4074.6	4737.3	7308.0	○	3535.0	0.0	○
	406	2804.4	2067.9	2438.2	2438.2	2804.4	3862.8	○	2067.9	0.0	○
	407	3097.0	2277.8	2692.2	2692.1	3097.0	3862.8	○	2277.8	0.0	○
	408	3273.5	2380.1	2816.2	2816.1	3273.5	3862.8	○	2380.1	0.0	○
	409	3381.1	2274.7	2860.8	2860.7	3381.1	3862.8	○	2274.7	0.0	○
	410	3344.3	2117.7	2807.7	2807.6	3344.3	3862.8	○	2117.7	0.0	○
P11 Main tower center span side (Top→Bottom)	411	4965.0	3754.4	4371.0	4371.0	4965.0	7308.0	○	3754.4	0.0	○
	412	4519.4	3657.8	4086.1	4086.1	4519.4	7308.0	○	3657.8	0.0	○
	413	4161.3	3529.2	3867.7	3867.7	4161.3	7308.0	○	3529.2	0.0	○
	414	4202.1	3510.6	3832.4	3832.4	4202.1	7308.0	○	3510.6	0.0	○
	415	4526.1	3644.5	4069.3	4069.3	4526.1	7308.0	○	3644.5	0.0	○
	416	2760.6	2159.0	2438.1	2438.1	2760.6	3862.8	○	2159.0	0.0	○
	417	3047.4	2362.4	2688.0	2688.0	3047.4	3862.8	○	2362.4	0.0	○
	418	3294.4	2431.8	2821.5	2821.5	3294.4	3862.8	○	2431.8	0.0	○
	419	3505.0	2300.6	2864.8	2864.8	3505.0	3862.8	○	2300.6	0.0	○
	420	3561.4	2123.6	2814.6	2814.5	3561.4	3862.8	○	2123.6	0.0	○
P12 Main tower center span side (Top→Bottom)	421	5050.7	3737.0	4370.9	4370.9	5050.7	7308.0	○	3737.0	0.0	○
	422	4501.8	3651.2	4086.1	4086.0	4501.8	7308.0	○	3651.2	0.0	○
	423	4219.2	3600.9	3867.7	3867.7	4219.2	7308.0	○	3600.9	0.0	○
	424	4232.9	3422.0	3832.4	3832.4	4232.9	7308.0	○	3422.0	0.0	○
	425	4561.3	3590.4	4069.3	4069.2	4561.3	7308.0	○	3590.4	0.0	○
	426	2768.7	2112.4	2438.1	2438.1	2768.7	3862.8	○	2112.4	0.0	○
	427	3072.2	2315.4	2688.0	2688.0	3072.2	3862.8	○	2315.4	0.0	○
	428	3240.1	2361.3	2821.5	2821.5	3240.1	3862.8	○	2361.3	0.0	○
	429	3408.2	2256.3	2864.8	2864.8	3408.2	3862.8	○	2256.3	0.0	○
	430	3481.9	2117.8	2814.6	2814.5	3481.9	3862.8	○	2117.8	0.0	○
P12 Main tower side span side (Top→Bottom)	431	5011.1	3725.9	4390.8	4390.8	5011.1	7308.0	○	3725.9	0.0	○
	432	4510.7	3656.0	4101.7	4101.7	4510.7	7308.0	○	3656.0	0.0	○
	433	4176.4	3530.8	3880.9	3880.9	4176.4	7308.0	○	3530.8	0.0	○
	434	4178.4	3461.2	3830.9	3830.9	4178.4	7308.0	○	3461.2	0.0	○
	435	4563.9	3583.0	4074.6	4074.6	4563.9	7308.0	○	3583.0	0.0	○
	436	2759.3	2167.0	2438.2	2438.2	2759.3	3862.8	○	2167.0	0.0	○
	437	3063.1	2332.3	2692.1	2692.1	3063.1	3862.8	○	2332.3	0.0	○
	438	3211.6	2393.1	2816.2	2816.2	3211.6	3862.8	○	2393.1	0.0	○
	439	3370.8	2349.0	2860.8	2860.7	3370.8	3862.8	○	2349.0	0.0	○
	440	3421.8	2238.5	2807.7	2807.7	3421.8	3862.8	○	2238.5	0.0	○

Source: JICA Study Team

d) Pier Evaluation

The bending stress on the reinforced concrete member generated by the bending moment due to the seismic response is verified to be below the allowable bending stress as shown below. Furthermore, the shearing stress on the concrete generated by the shear stress due to the seismic response is verified to be below the allowable shearing stress.

Table 4.2.170 P10 Pier and P11 Pier Evaluation Results (Longitudinal)

Pier P10 Longitudinal Cross-Section Calculation Result

	Element Number	Bending Moment (kN·m)		Axial Force(kN)		Bending Moment	Axial Force	Bending Stress (N/mm^2)		Verdict	Shear Force (kN)			Shearing Stress		Shear Reinforcement		Verdict
								σc	σs		Max Val	Min Val	Adopted Value	τ	τa	Necessary Reinforcement Content	Reinforcement Content	
		Max	Min	Max	Min	(kN·m)	(kN)	σca=15	σsa=300					(N/mm^2)	(N/mm^2)	(cm^2)	(cm^2)	
(Column Top End)	6010	51135.9	-26954.4	-26178.8	-41699.1	51135.9	41699.1				10496.7	-10553.4	10553.4					
(Ginko Type Section)	6011	71089.1	-46660.2	-30821.2	-24066.4	71089.1	30821.2				12266.2	-12292.3	12292.3					
	6012	93152.3	-68759.3	-34612.6	-28276.2	93152.3	34612.6				13599.4	-13593.7	13599.4					
	6013	116815.0	-92524.7	-37790.8	-31755.4	116815.0	37790.8				14641.5	-14592.3	14641.5					
Pier P10	6014	144279.0	-120182.0	-41037.8	-34963.1	144279.0	41037.8	3.0	59.9	○	15630.3	-15537.2	15630.3	0.202	0.223		30.968	○
	6015	173250.0	-149437.0	-44428.2	-38277.1	173250.0	44428.2	3.7	84.5	○	16565.0	-16456.3	16565.0	0.214	0.218		30.968	○
	6016	203597.0	-180218.0	-47818.2	-41663.0	203597.0	47818.2	4.4	111.8	○	17493.9	-17315.4	17493.9	0.226	0.214	0.796	30.968	○
	6017	235347.0	-212487.0	-51208.2	-45049.3	235347.0	51208.2	5.2	141.5	○	18526.4	-18279.2	18526.4	0.240	0.211	1.855	30.968	○
	6018	268612.0	-246312.0	-54598.0	-48436.1	268612.0	54598.0	6.0	173.7	○	19595.3	-19238.8	19595.3	0.254	0.208	2.910	30.968	○
	6019	301556.0	-280462.0	-57987.4	-51823.4	301556.0	57987.4	6.8	205.8	○	20621.2	-19908.7	20621.2	0.267	0.206	3.890	30.968	○
	6020	337526.0	-317815.0	-61376.8	-55211.1	337526.0	61376.8	7.6	242.2	○	21463.6	-20705.8	21463.6	0.278	0.204	4.711	30.968	○
	6021	344978.0	-325483.0	-63410.2	-57921.6	344978.0	63410.2	7.8	245.3	○	22136.3	-21285.5	22136.3	0.286	0.205	5.245	30.968	○
	6022	352344.0	-333235.0	-64088.2	-59277.0	352344.0	64088.2	8.0	252.8	○	22314.9	-21419.3	22314.9	0.289	0.204	5.416	30.968	○
	6023	359803.0	-341074.0	-64766.0	-59954.7	359803.0	64766.0	8.2	260.4	○	22490.9	-21550.1	22490.9	0.291	0.204	5.584	30.968	○
	6024	367302.0	-348967.0	-65443.8	-60632.4	367302.0	65443.8	8.3	268.1	○	22664.5	-21678.1	22664.5	0.293	0.204	5.750	30.968	○
(Column Bottom End)	6025	374855.0	-356903.0	-66121.6	-61310.1	374855.0	66121.6	8.5	275.9	○	22835.7	-21803.5	22835.7	0.295	0.203	5.913	30.968	○

Pier P11 Longitudinal Cross-Section Calculation Result

	Element Number	Bending Moment (kN·m)		Axial Force(kN)		Bending Moment	Axial Force	Bending Stress (N/mm^2)		Verdict	Shear Force (kN)			Shearing Stress		Shear Reinforcement		Verdict
								σc	σs		Max Val	Min Val	Adopted Value	τ	τa	Necessary	Reinforcement	
		Max	Min	Max	Min	(kN·m)	(kN)	σca=15	σsa=300					(N/mm^2)	(N/mm^2)	(cm^2)	(cm^2)	
(Column Top End)	6106	44364.0	-45939.5	-56319.0	-52324.2	45939.5	56319.0				22055.3	-21153.9	22055.3					
(Ginko Type Section)	6107	68787.0	-71722.7	-58798.8	-54800.8	71722.7	58798.8				22926.4	-21709.3	22926.4					
	6108	93755.9	-98351.9	-61092.4	-57091.6	98351.9	61092.4				23677.7	-22194.6	23677.7					
	6109	119210.0	-125718.0	-63255.8	-59252.6	125718.0	63255.8				24334.6	-22626.0	24334.6					
Pier P11	6110	158534.0	-168351.0	-65954.0	-61948.0	168351.0	65954.0	2.8	26.1	○	25086.0	-23198.5	25086.0	0.324	0.286	2.451	30.968	○
	6111	198796.0	-212307.0	-69201.2	-65192.2	212307.0	69201.2	3.6	46.6	○	25869.8	-23943.0	25869.8	0.334	0.272	4.012	30.968	○
	6112	239904.0	-257396.0	-72448.4	-68436.6	257396.0	72448.4	4.4	71.0	○	26532.6	-24609.2	26532.6	0.343	0.262	5.177	30.968	○
	6113	281802.0	-303396.0	-75695.4	-71681.0	303396.0	75695.4	5.3	97.9	○	27080.6	-25192.2	27080.6	0.350	0.255	6.074	30.968	○
	6114	325171.0	-348946.0	-78942.2	-74925.6	348946.0	78942.2	6.1	125.7	○	27498.9	-25696.0	27498.9	0.355	0.250	6.742	30.968	○
	6115	369526.0	-396304.0	-82188.6	-78170.6	396304.0	82188.6	7.0	155.5	○	27926.2	-26178.7	27926.2	0.361	0.246	7.357	30.968	○
	6116	414686.0	-444111.0	-85435.2	-81415.6	444111.0	85435.2	7.9	186.1	○	28398.4	-26620.5	28398.4	0.367	0.243	7.956	30.968	○
	6117	460532.0	-492394.0	-88681.4	-84660.8	492394.0	88681.4	8.8	217.5	○	29013.9	-27027.3	29013.9	0.375	0.240	8.636	30.968	○
	6118	507005.0	-541112.0	-91927.4	-87906.2	541112.0	91927.4	9.6	249.4	○	29733.7	-27395.4	29733.7	0.384	0.238	9.374	30.968	○
	6119	516323.0	-550936.0	-93875.0	-89853.4	550936.0	93875.0	9.8	253.4	○	30231.3	-27632.5	30231.3	0.391	0.238	9.779	30.968	○
	6120	525692.0	-560722.0	-94524.2	-90502.6	560722.0	94524.2	10.0	259.9	○	30383.1	-27698.5	30383.1	0.393	0.238	9.930	30.968	○
	6121	535080.0	-570520.0	-95173.4	-91151.6	570520.0	95173.4	10.2	266.3	○	30534.2	-27764.5	30534.2	0.395	0.237	10.080	30.968	○
	6122	544481.0	-580342.0	-95822.4	-91800.8	580342.0	95822.4	10.3	272.8	○	30687.6	-27828.8	30687.6	0.397	0.237	10.230	30.968	○
(Column Bottom End)	6123	553906.0	-590183.0	-96471.6	-92450.0	590183.0	96471.6	10.5	279.3	○	30839.6	-27891.5	30839.6	0.399	0.237	10.379	30.968	○

Source: JICA Study Team

Table 4.2.171 P12 Pier and P13 Pier Evaluation Results (Longitudinal)

Pier P12 Longitudinal Cross-Section Calculation Result

	Element Number	Bending Moment (kN·m)		Axial Force(kN)		Bending Moment	Axial Force	Bending Stress (N/mm^2)		Verdict	Shear Force (kN)			Shearing Stress		Shear Reinforcement		Verdict
								σc	σs		Max Val	Min Val	Adopted Value	τ (N/mm^2)	τa (N/mm^2)	Necessary (cm^2)	Reinforcement (cm^2)	
		Max	Min	Max	Min	(kN·m)	(kN)	σca=15	σsa=300									
(Column Top End)	6206	38560.8	-44448.9	-55489.6	-52740.0	44448.9	55489.6				21347.0	-18458.3	21347.0					
(Ginko Type Section)	6207	60029.6	-69419.2	-57968.6	-55217.8	69419.2	57968.6				22206.6	-19083.4	22206.6					
	6208	82111.4	-95218.7	-60261.4	-57509.6	95218.7	60261.4				22959.7	-19628.3	22959.7					
	6209	104737.0	-121768.0	-62424.2	-59671.4	121768.0	62424.2				23631.2	-20111.2	23631.2					
Pier P12	6210	139884.0	-163184.0	-65121.8	-62367.8	163184.0	65121.8	2.7	24.3	○	24418.0	-20699.1	24418.0	0.316	0.288	1.796	30.968	○
	6211	176073.0	-205999.0	-68368.2	-65613.0	205999.0	68368.2	3.5	43.8	○	25257.7	-21402.4	25257.7	0.326	0.273	3.422	30.968	○
	6212	213203.0	-250040.0	-71614.6	-68858.2	250040.0	71614.6	4.3	67.3	○	25993.0	-21994.5	25993.0	0.336	0.263	4.661	30.968	○
	6213	251154.0	-295119.0	-74860.8	-72103.6	295119.0	74860.8	5.1	93.3	○	26624.5	-22541.6	26624.5	0.344	0.256	5.638	30.968	○
	6214	289841.0	-339917.0	-78107.0	-75349.0	339917.0	78107.0	6.0	120.4	○	27144.6	-23106.5	27144.6	0.351	0.251	6.400	30.968	○
	6215	328923.0	-386629.0	-81353.2	-78594.4	386629.0	81353.2	6.8	149.6	○	27718.1	-23684.4	27718.1	0.358	0.247	7.143	30.968	○
	6216	369438.0	-433995.0	-84599.2	-81839.8	433995.0	84599.2	7.7	179.9	○	28363.5	-24240.9	28363.5	0.367	0.244	7.892	30.968	○
	6217	410869.0	-482029.0	-87845.2	-85085.4	482029.0	87845.2	8.6	211.0	○	29123.1	-24736.0	29123.1	0.376	0.241	8.697	30.968	○
	6218	453107.0	-530784.0	-91091.0	-88331.0	530784.0	91091.0	9.5	242.9	○	29980.2	-25187.0	29980.2	0.388	0.238	9.554	30.968	○
	6219	461591.0	-540664.0	-93038.6	-90278.4	540664.0	93038.6	9.6	246.9	○	30551.6	-25448.3	30551.6	0.395	0.239	10.021	30.968	○
	6220	470140.0	-550502.0	-93687.6	-90927.6	550502.0	93687.6	9.8	253.4	○	30725.6	-25538.1	30725.6	0.397	0.238	10.192	30.968	○
	6221	478711.0	-560385.0	-94336.8	-91576.8	560385.0	94336.8	10.0	259.9	○	30897.6	-25626.6	30897.6	0.399	0.238	10.360	30.968	○
	6222	487320.0	-570280.0	-94986.0	-92225.8	570280.0	94986.0	10.2	266.5	○	31069.7	-25716.2	31069.7	0.402	0.237	10.527	30.968	○
(Column Bottom End)	6223	495947.0	-580222.0	-95635.2	-92875.0	580222.0	95635.2	10.3	273.1	○	31242.6	-25840.4	31242.6	0.404	0.237	10.694	30.968	○

Pier P13 Longitudinal Cross-Section Calculation Result

	Element Number	Bending Moment (kN·m)		Axial Force(kN)		Bending Moment	Axial Force	Bending Stress (N/mm^2)		Verdict	Shear Force (kN)			Shearing Stress		Shear Reinforcement		Verdict
								σc	σs		Max Val	Min Val	Adopted Value	τ (N/mm^2)	τa (N/mm^2)	Necessary (cm^2)	Reinforcement (cm^2)	
		Max	Min	Max	Min	(kN·m)	(kN)	σca=15	σsa=300									
(Column Top End)	6310	14460.3	-36151.7	-22319.4	-18153.3	36151.7	22319.4				8256.8	-7907.2	8256.8					
(Ginko Type Section)	6311	30931.4	-53261.4	-26960.8	-22792.2	53261.4	26960.8				10537.8	-10148.2	10537.8					
	6312	50144.1	-73105.3	-30751.8	-26581.0	73105.3	30751.8				12230.5	-11834.2	12230.5					
	6313	71457.3	-95025.1	-33929.4	-29757.0	95025.1	33929.4				13511.6	-13130.4	13511.6					
Pier P13	6314	96863.2	-121035.0	-37176.0	-33002.2	121035.0	37176.0	2.5	43.8	○	14689.8	-14343.8	14689.8	0.190	0.227		30.968	○
	6315	124267.0	-148916.0	-40565.6	-36390.6	148916.0	40565.6	3.1	66.3	○	15796.3	-15498.4	15796.3	0.204	0.221		30.968	○
	6316	153460.0	-178472.0	-43955.2	-39779.0	178472.0	43955.2	3.8	92.1	○	16835.8	-16557.5	16835.8	0.218	0.216	0.116	30.968	○
	6317	184452.0	-209637.0	-47344.6	-43167.4	209637.0	47344.6	4.6	120.8	○	17949.6	-17652.8	17949.6	0.232	0.212	1.280	30.968	○
	6318	217267.0	-242456.0	-50734.0	-46556.0	242456.0	50734.0	5.4	152.2	○	19308.3	-18846.2	19308.3	0.250	0.209	2.602	30.968	○
	6319	251838.0	-274776.0	-54123.2	-49944.6	274776.0	54123.2	6.2	183.3	○	20683.7	-19993.3	20683.7	0.268	0.207	3.886	30.968	○
	6320	285258.0	-309642.0	-57512.4	-53333.4	309642.0	57512.4	7.0	218.1	○	21763.5	-20856.8	21763.5	0.282	0.205	4.914	30.968	○
	6321	291535.0	-317174.0	-59545.8	-55366.8	317174.0	59545.8	7.1	221.3	○	22850.3	-21589.0	22850.3	0.296	0.205	5.791	30.968	○
	6322	298892.0	-324758.0	-60223.6	-56044.6	324758.0	60223.6	7.3	229.1	○	23123.8	-21770.9	23123.8	0.299	0.205	6.045	30.968	○
	6323	306338.0	-332424.0	-60901.4	-56722.2	332424.0	60901.4	7.5	237.0	○	23394.1	-21948.3	23394.1	0.303	0.204	6.296	30.968	○
	6324	313851.0	-340197.0	-61579.2	-57400.0	340197.0	61579.2	7.7	245.1	○	23661.1	-22121.6	23661.1	0.306	0.204	6.543	30.968	○
(Column Bottom End)	6325	321410.0	-348034.0	-62257.0	-58077.8	348034.0	62257.0	7.9	253.2	○	23925.3	-22291.0	23925.3	0.310	0.204	6.787	30.968	○

Source: JICA Study Team

Table 4.2.172 P10 Pier and P11 Pier Evaluation Results (Transverse)

Pier P10 Traverse Cross-Section Calculation Result

	Element Number	Bending Moment (kN·m) Max	Bending Moment (kN·m) Min	Axial Force(kN) Max	Axial Force(kN) Min	Bending Moment (kN·m)	Axial Force (kN)	Bending Stress (N/mm²) σc σca=15	Bending Stress (N/mm²) σs σsa=300	Verdict	Shear Force (kN) Max Val	Shear Force (kN) Min Val	Shear Force (kN) Adopted Value	Shearing Stress τ (N/mm²)	Shearing Stress τa (N/mm²)	Shear Reinforcement Necessary (cm²)	Shear Reinforcement Reinforcement (cm²)	Verdict
(Column Top End)	16010	95762.7	-89533.8	-24083.6	-23981.8	95762.7	24083.6				14516.4	-14586.6	14586.6					
(Ginko Type Section)	16011	121789.0	-115829.0	-28724.0	-28621.6	121789.0	28724.0				16245.0	-16081.2	16245.0					
	16012	149606.0	-144279.0	-32514.0	-32411.0	149606.0	32514.0				17560.7	-17218.9	17560.7					
	16013	178836.0	-174348.0	-35691.0	-35587.6	178836.0	35691.0				18586.7	-18102.4	18586.7					
Pier P10	16014	212200.0	-208915.0	-38936.8	-38833.0	212200.0	38936.8	3.5	82.2	○	19565.9	-18935.3	19565.9	0.253	0.197	2.246	23.226	○
	16015	246965.0	-245065.0	-42326.0	-42221.8	246965.0	42326.0	4.2	106.8	○	20522.9	-19725.0	20522.9	0.265	0.194	2.863	23.226	○
	16016	283021.0	-282717.0	-45715.0	-45610.6	283021.0	45715.0	4.9	133.4	○	21445.3	-20507.0	21445.3	0.277	0.192	3.439	23.226	○
	16017	320327.0	-321768.0	-49104.2	-48999.4	321768.0	49104.2	5.7	163.5	○	22393.1	-21512.9	22393.1	0.289	0.189	4.020	23.226	○
	16018	358778.0	-362237.0	-52493.2	-52388.2	362237.0	52493.2	6.5	195.9	○	23360.9	-22515.7	23360.9	0.302	0.188	4.598	23.226	○
	16019	397772.0	-400379.0	-55882.2	-55777.0	400379.0	55882.2	7.3	225.8	○	24305.9	-23482.2	24305.9	0.314	0.186	5.142	23.226	○
	16020	427466.0	-430603.0	-59271.2	-59165.8	430603.0	59271.2	7.9	246.5	○	24479.9	-24389.9	24479.9	0.316	0.186	5.251	23.226	○
	16021	433856.0	-437549.0	-61304.4	-61199.2	437549.0	61304.4	7.9	245.7	○	25680.8	-24923.5	25680.8	0.332	0.186	5.851	23.226	○
	16022	441611.0	-445623.0	-61982.2	-61877.0	445623.0	61982.2	8.1	252.2	○	25846.5	-25096.3	25846.5	0.334	0.186	5.947	23.226	○
	16023	449669.0	-453764.0	-62660.0	-62554.8	453764.0	62660.0	8.3	258.8	○	26011.6	-25267.2	26011.6	0.336	0.186	6.043	23.226	○
	16024	457710.0	-461950.0	-63337.8	-63232.4	461950.0	63337.8	8.4	265.5	○	26173.8	-25434.4	26173.8	0.338	0.186	6.137	23.226	○
(Column Bottom End)	16025	465796.0	-470199.0	-64015.6	-63910.2	470199.0	64015.6	8.6	272.3	○	26332.8	-25602.3	26332.8	0.340	0.185	6.229	23.226	○

Pier P11 Traverse Cross-Section Calculation Result

	Element Number	Bending Moment (kN·m) Max	Bending Moment (kN·m) Min	Axial Force(kN) Max	Axial Force(kN) Min	Bending Moment (kN·m)	Axial Force (kN)	Bending Stress (N/mm²) σc σca=15	Bending Stress (N/mm²) σs σsa=300	Verdict	Shear Force (kN) Max Val	Shear Force (kN) Min Val	Shear Force (kN) Adopted Value	Shearing Stress τ (N/mm²)	Shearing Stress τa (N/mm²)	Shear Reinforcement Necessary (cm²)	Shear Reinforcement Reinforcement (cm²)	Verdict
(Column Top End)	16106	96437.0	-88251.4	-54159.6	-54159.2	96437.0	54159.6				12215.8	-13147.1	13147.1					
(Ginko Type Section)	16107	111739.0	-100879.0	-56637.8	-56637.4	111739.0	56637.8				12530.0	-13623.7	13623.7					
	16108	127530.0	-113851.0	-58930.0	-58929.8	127530.0	58930.0				12807.3	-14055.9	14055.9					
	16109	143778.0	-127120.0	-61092.4	-61092.0	143778.0	61092.4				13056.8	-14456.0	14456.0					
Pier P11	16110	169159.0	-147712.0	-63789.2	-63788.8	169159.0	63789.2	2.1	10.5	○	13351.6	-14946.1	14946.1	0.193	0.288		23.226	○
	16111	195512.0	-168883.0	-67034.8	-67034.4	195512.0	67034.8	2.5	16.0	○	13674.8	-15520.4	15520.4	0.201	0.278		23.226	○
	16112	222816.0	-190605.0	-70280.6	-70280.2	222816.0	70280.6	2.8	22.8	○	13969.1	-16080.3	16080.3	0.208	0.271		23.226	○
	16113	251034.0	-214491.0	-73526.4	-73526.0	251034.0	73526.4	3.2	31.1	○	14232.0	-16627.0	16627.0	0.215	0.264		23.226	○
	16114	280162.0	-238936.0	-76772.0	-76771.6	280162.0	76772.0	3.7	40.9	○	14469.0	-17164.5	17164.5	0.222	0.259		23.226	○
	16115	310234.0	-263734.0	-80017.8	-80017.4	310234.0	80017.8	4.1	52.1	○	14696.3	-17727.6	17727.6	0.229	0.254		23.226	○
	16116	341271.0	-288883.0	-83263.6	-83263.2	341271.0	83263.6	4.6	64.8	○	14914.2	-18307.5	18307.5	0.237	0.250		23.226	○
	16117	373293.0	-314342.0	-86509.2	-86508.8	373293.0	86509.2	5.2	78.8	○	15126.5	-18888.8	18888.8	0.244	0.246		23.226	○
	16118	406287.0	-340049.0	-89755.0	-89754.6	406287.0	89755.0	5.7	94.1	○	15345.1	-19467.5	19467.5	0.252	0.243	0.346	23.226	○
	16119	413002.0	-345217.0	-91702.4	-91702.0	413002.0	91702.4	5.8	94.8	○	15479.0	-19811.2	19811.2	0.256	0.244	0.512	23.226	○
	16120	419753.0	-350393.0	-92351.6	-92351.2	419753.0	92351.6	5.9	98.0	○	15525.8	-19925.9	19925.9	0.258	0.243	0.595	23.226	○
	16121	426542.0	-355577.0	-93000.6	-93000.4	426542.0	93000.6	6.0	101.3	○	15571.8	-20040.3	20040.3	0.259	0.242	0.677	23.226	○
	16122	433368.0	-360769.0	-93649.8	-93649.4	433368.0	93649.8	6.2	104.6	○	15620.9	-20154.4	20154.4	0.261	0.242	0.759	23.226	○
(Column Bottom End)	16123	440242.0	-365969.0	-94299.0	-94298.6	440242.0	94299.0	6.3	108.0	○	15691.4	-20268.8	20268.8	0.262	0.241	0.841	23.226	○

Source: JICA Study Team

Table 4.2.173 P12 Pier and P13 Pier Evaluation Results (Transverse)

Pier P12 Traverse Cross-Section Calculation Result

	Element Number	Bending Moment (kN·m) Max	Bending Moment (kN·m) Min	Axial Force(kN) Max	Axial Force(kN) Min	Bending Moment (kN·m)	Axial Force (kN)	Bending Stress (N/mm^2) σc σca=15	Bending Stress (N/mm^2) σs σsa=300	Verdict	Shear Force (kN) Max Val	Shear Force (kN) Min Val	Shear Force (kN) Adopted Value	Shearing Stress τ (N/mm^2)	Shearing Stress τa (N/mm^2)	Shear Reinforcement Necessary (cm^2)	Shear Reinforcement Reinforcement (cm^2)	Verdict
(Column Top End)	16206	104123.0	-95467.3	-54159.2	-54159.2	104123.0	54159.2				14558.9	-14919.1	14919.1					
(Ginko Type Section)	16207	121548.0	-110769.0	-56637.6	-56637.4	121548.0	56637.6				15020.7	-15513.8	15513.8					
	16208	139583.0	-126615.0	-58929.8	-58929.6	139583.0	58929.8				15430.2	-16049.5	16049.5					
	16209	158171.0	-142941.0	-61092.0	-61091.8	158171.0	61092.0				15800.0	-16540.6	16540.6					
Pier P12	16210	187273.0	-168942.0	-63788.8	-63788.6	187273.0	63788.8	2.4	15.6	○	16241.1	-17136.6	17136.6	0.222	0.278		23.226	○
	16211	217543.0	-197384.0	-67034.6	-67034.4	217543.0	67034.6	2.8	23.7	○	16772.0	-17824.6	17824.6	0.231	0.268		23.226	○
	16212	248920.0	-226641.0	-70280.2	-70280.2	248920.0	70280.2	3.2	33.8	○	17330.8	-18486.4	18486.4	0.239	0.261		23.226	○
	16213	281376.0	-256664.0	-73526.0	-73525.8	281376.0	73526.0	3.7	45.9	○	17862.2	-19124.9	19124.9	0.247	0.255		23.226	○
	16214	314854.0	-287415.0	-76771.8	-76771.6	314854.0	76771.8	4.3	59.8	○	18374.3	-19746.3	19746.3	0.255	0.250	0.225	23.226	○
	16215	349396.0	-318928.0	-80017.4	-80017.4	349396.0	80017.4	4.9	75.5	○	18903.8	-20392.4	20392.4	0.264	0.246	0.733	23.226	○
	16216	385020.0	-351748.0	-83263.2	-83263.0	385020.0	83263.2	5.5	92.8	○	19440.3	-21056.7	21056.7	0.272	0.242	1.227	23.226	○
	16217	421710.0	-385467.0	-86509.0	-86508.8	421710.0	86509.0	6.1	111.5	○	19965.7	-21722.0	21722.0	0.281	0.239	1.702	23.226	○
	16218	456654.0	-420096.0	-89754.6	-89754.6	456654.0	89754.6	6.7	129.5	○	20478.4	-22347.0	22347.0	0.289	0.236	2.126	23.226	○
	16219	461745.0	-427123.0	-91702.0	-91702.0	461745.0	91702.0	6.8	128.8	○	20771.7	-22778.0	22778.0	0.295	0.237	2.327	23.226	○
	16220	468920.0	-434193.0	-92351.2	-92351.0	468920.0	92351.2	6.9	132.6	○	20869.1	-22909.1	22909.1	0.296	0.236	2.414	23.226	○
	16221	476207.0	-441300.0	-93000.4	-93000.2	476207.0	93000.4	7.0	136.4	○	20967.0	-23039.8	23039.8	0.298	0.236	2.501	23.226	○
	16222	483586.0	-446483.0	-93649.6	-93649.4	483586.0	93649.6	7.1	140.3	○	21056.2	-23171.0	23171.0	0.300	0.235	2.589	23.226	○
(Column Bottom End)	16223	491047.0	-452398.0	-94298.6	-94298.6	491047.0	94298.6	7.3	144.4	○	21151.0	-23301.6	23301.6	0.301	0.235	2.675	23.226	○

Pier P13 Traverse Cross-Section Calculation Result

	Element Number	Bending Moment (kN·m) Max	Bending Moment (kN·m) Min	Axial Force(kN) Max	Axial Force(kN) Min	Bending Moment (kN·m)	Axial Force (kN)	Bending Stress (N/mm^2) σc σca=15	Bending Stress (N/mm^2) σs σsa=300	Verdict	Shear Force (kN) Max Val	Shear Force (kN) Min Val	Shear Force (kN) Adopted Value	Shearing Stress τ (N/mm^2)	Shearing Stress τa (N/mm^2)	Shear Reinforcement Necessary (cm^2)	Shear Reinforcement Reinforcement (cm^2)	Verdict
(Column Top End)	16310	87471.1	-81812.0	-20031.4	-20030.8	87471.1	20031.4				13308.3	-14150.0	14150.0					
(Ginko Type Section)	16311	113031.0	-105398.0	-24671.4	-24670.8	113031.0	24671.4				14831.5	-15803.7	15803.7					
	16312	140607.0	-130846.0	-28461.0	-28460.6	140607.0	28461.0				16007.0	-17070.9	17070.9					
	16313	169770.0	-158356.0	-31637.8	-31637.4	169770.0	31637.8				16935.5	-18070.0	18070.0					
Pier P13	16314	203278.0	-189996.0	-34883.6	-34883.0	203278.0	34883.6	3.4	87.7	○	17828.5	-19036.7	19036.7	0.246	0.194	2.088	23.226	○
	16315	238367.0	-223176.0	-38272.6	-38272.0	238367.0	38272.6	4.1	113.3	○	18696.5	-20000.8	20000.8	0.259	0.191	2.698	23.226	○
	16316	274941.0	-257784.0	-41661.4	-41661.0	274941.0	41661.4	4.9	140.9	○	19524.9	-20945.2	20945.2	0.271	0.189	3.278	23.226	○
	16317	313007.0	-293791.0	-45050.4	-45049.8	313007.0	45050.4	5.6	170.6	○	20361.5	-21921.0	21921.0	0.283	0.187	3.860	23.226	○
	16318	352586.0	-331160.0	-48439.2	-48438.8	352586.0	48439.2	6.4	202.2	○	21179.5	-22936.7	22936.7	0.297	0.186	4.452	23.226	○
	16319	392174.0	-369806.0	-51828.2	-51827.6	392174.0	51828.2	7.2	234.1	○	21948.6	-23950.4	23950.4	0.310	0.184	5.030	23.226	○
	16320	422167.0	-403935.0	-55217.2	-55216.6	422167.0	55217.2	7.8	254.5	○	22321.7	-24407.8	24407.8	0.316	0.184	5.281	23.226	○
	16321	429053.0	-410682.0	-57250.4	-57250.0	429053.0	57250.4	7.9	253.6	○	23037.8	-25489.2	25489.2	0.330	0.185	5.817	23.226	○
	16322	437000.0	-418383.0	-57928.2	-57927.8	437000.0	57928.2	8.1	260.0	○	23166.6	-25679.3	25679.3	0.332	0.185	5.925	23.226	○
	16323	445104.0	-426168.0	-58606.0	-58605.6	445104.0	58606.0	8.2	266.6	○	23293.2	-25867.8	25867.8	0.334	0.184	6.031	23.226	○
	16324	453319.0	-433986.0	-59283.8	-59283.4	453319.0	59283.8	8.4	273.4	○	23417.4	-26054.2	26054.2	0.337	0.184	6.137	23.226	○
(Column Bottom End)	16325	461586.0	-441787.0	-59961.6	-59961.2	461586.0	59961.6	8.6	280.2	○	23539.6	-26238.6	26238.6	0.339	0.184	6.241	23.226	○

Source: JICA Study Team

e) Response Value of Bearing Support Section

The response value calculated by the dynamic analysis is shown in the table below.

Table 4.2.174 Bearing Support Reaction Force

Longitudinal Analysis

			Vertical Force		Longitudinal Horizontal Force		Traverse Horizontal Force	
			Max (kN)	Min (kN)	Max (kN)	Min (kN)	Max (kN)	Min (kN)
Pier P10	L	Rocking Bearing	1097.5	-1009.9	0.0	0.0	0.0	0.0
	CL	Horizontal Bearing	0.0	0.0	0.0	0.0	4.1	-4.2
	R	Rocking Bearing	1097.4	-1010.3	0.0	0.0	0.0	0.0
Pier P11	L	Pin Roller Bearing	12402.7	12396.8	0.0	0.0	0.0	0.0
	CL	Pivot Bearing	48297.4	44317.5	20067.1	-20419.4	4.7	-4.6
	R	Pin Roller Bearing	12402.7	12396.9	0.0	0.0	0.0	0.0
Pier P12	L	Pin Roller Bearing	12402.3	12397.4	0.0	0.0	0.0	0.0
	CL	Pivot Bearing	47471.8	44730.7	19234.0	-17744.9	4.1	-4.0
	R	Pin Roller Bearing	12402.1	12397.4	0.0	0.0	0.0	0.0
Pier P13	L	Rocking Bearing	1212.7	-821.5	0.0	0.0	0.0	0.0
	CL	Horizontal Bearing	0.0	0.0	0.0	0.0	2.1	-2.1
	R	Rocking Bearing	1212.0	-821.5	0.0	0.0	0.0	0.0

Traverse Analysis

			Vertical Force		Longitudinal Horizontal Force		Traverse Horizontal Force	
			Max (kN)	Min (kN)	Max (kN)	Min (kN)	Max (kN)	Min (kN)
Pier P10	L	Rocking Bearing	2060.4	-1907.7	0.0	0.0	0.0	0.0
	CL	Horizontal Bearing	0.0	0.0	0.0	0.0	6696.1	-6385.0
	R	Rocking Bearing	2082.6	-1885.5	0.0	0.0	0.0	0.0
Pier P11	L	Pin Roller Bearing	20422.1	3951.1	0.0	0.0	0.0	0.0
	CL	Pivot Bearing	46145.9	46145.5	976.6	976.2	12274.5	-11636.1
	R	Pin Roller Bearing	20848.4	4377.5	0.0	0.0	0.0	0.0
Pier P12	L	Pin Roller Bearing	20764.2	3452.3	0.0	0.0	0.0	0.0
	CL	Pivot Bearing	46145.8	46145.6	976.5	976.3	13820.4	-13692.4
	R	Pin Roller Bearing	21347.2	4035.3	0.0	0.0	0.0	0.0
Pier P13	L	Rocking Bearing	2045.2	-1813.8	0.0	0.0	0.0	0.0
	CL	Horizontal Bearing	0.0	0.0	0.0	0.0	6218.9	-6255.5
	R	Rocking Bearing	1988.6	-1870.3	0.0	0.0	0.0	0.0

Min Max Summary

			Vertical Force		Longitudinal Horizontal Force		Traverse Horizontal Force	
			Max (kN)	Min (kN)	Max (kN)	Min (kN)	Max (kN)	Min (kN)
Pier P10	L	Rocking Bearing	2060.4	-1907.7	0.0	0.0	0.0	0.0
	CL	Horizontal Bearing	0.0	0.0	0.0	0.0	6696.1	-6385.0
	R	Rocking Bearing	2082.6	-1885.5	0.0	0.0	0.0	0.0
Pier P11	L	Pin Roller Bearing	20422.1	3951.1	0.0	0.0	0.0	0.0
	CL	Pivot Bearing	48297.4	44317.5	20067.1	-20419.4	12274.5	-11636.1
	R	Pin Roller Bearing	20848.4	4377.5	0.0	0.0	0.0	0.0
Pier P12	L	Pin Roller Bearing	20764.2	3452.3	0.0	0.0	0.0	0.0
	CL	Pivot Bearing	47471.8	44730.7	19234.0	-17744.9	13820.4	-13692.4
	R	Pin Roller Bearing	21347.2	4035.3	0.0	0.0	0.0	0.0
Pier P13	L	Rocking Bearing	2045.2	-1813.8	0.0	0.0	0.0	0.0
	CL	Horizontal Bearing	0.0	0.0	0.0	0.0	6218.9	-6255.5
	R	Rocking Bearing	1988.6	-1870.3	0.0	0.0	0.0	0.0

Source: JICA Study Team

The displacement determined by the dynamic analysis is shown in the table below.

Table 4.2.175 Bearing Support Displacement (Relative Displacement to Upper and Lower Member)

Longitudinal Analysis

			Longitudinal Diplacement		Transversal Displacement	
			Max	Min	Max	Min
			(m)	(m)	(m)	(m)
Pier P10	CL	Horizontal	0.067	-0.085	0.000	0.000
Pier P13	CL	Horizontal	0.087	-0.075	0.000	0.000

Transverse Analysis

			Longitudinal Diplacement		Transversal Displacement	
			Max	Min	Max	Min
			(kN)	(kN)	(kN)	(kN)
Pier P10	CL	Horizontal	-0.011	-0.011	0.000	0.000
Pier P13	CL	Horizontal	0.010	0.010	0.000	0.000

Source: JICA Study Team

The bearing support member was designed to have an allowable value that satisfies the design reaction force of the static analysis and the response value of the dynamic analysis.

4.2.13 Superstructure Construction Stage Analysis

4.2.13.1 Construction Stage Analysis Overview

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

4.2.13.2 Analysis Condition

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

Table 4.2.176 Analysis Condition

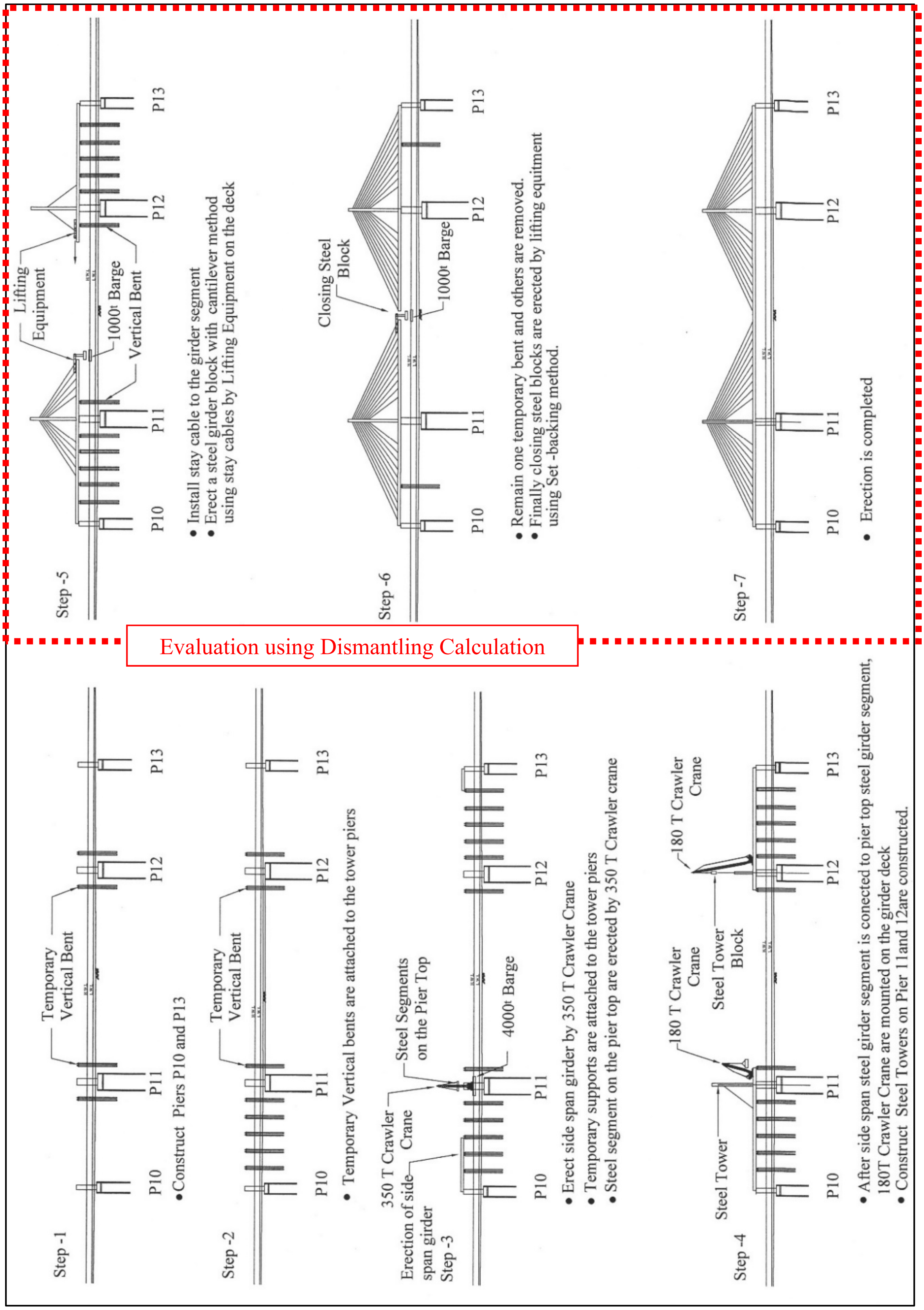
Item	Content
Analysis Theory	Linear structural analysis
Analysis Model	3d structure model
Considered Temporary Load	180 t considered Erection Machine W = 160 t Movement Protection Scaffolding W = 20 t
Analysis Stage	All 24 Stages （CS0～CS23）

Source: JICA Study Team

4.2.13.3 Construction Stage

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

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



Source: JICA Study Team

Figure 4.2.232 Construction Steps for Cable-stayed Bridge

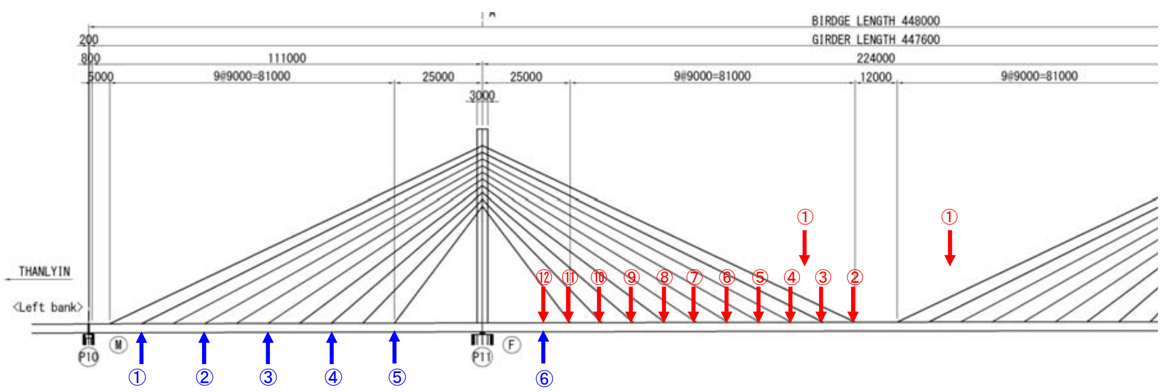
For the construction stage analysis, reversed order for the dismantling of temporary structure is configured as shown below.

Table 4.2.177 Dismantling Stages

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

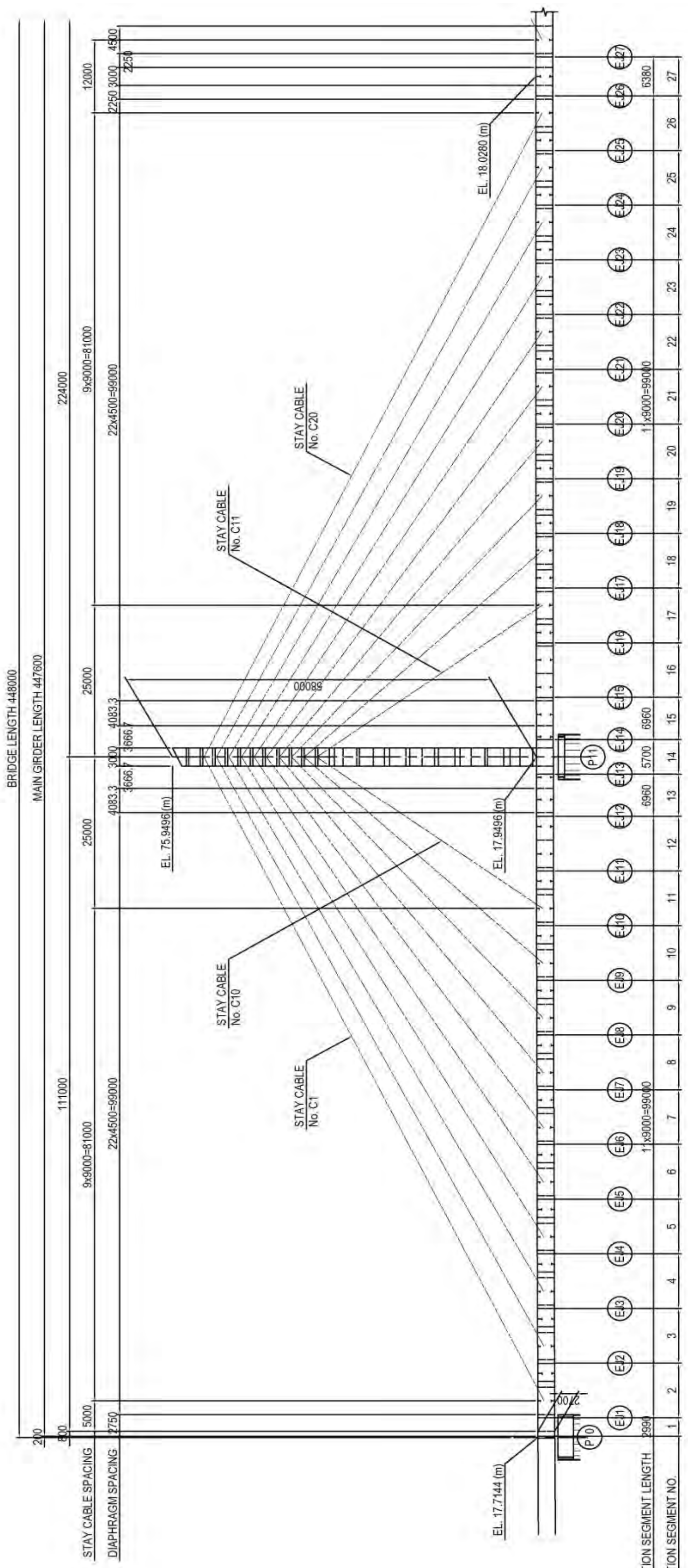
※Refer to the next page for cable number and main grider numbers

Source: JICA Study Team



Source: JICA Study Team

Figure 4.2.233 Bent Position and Crane Set Position



Source: JICA Study Team

Figure 4.2.234 Overview of Superstructure

4.2.13.4 Dismantling Calculation Results

(1) Cross Section Evaluation

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

Table 4.2.178 Main Girder Section Force Summary Table

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

Source: JICA Study Team

Table 4.2.179 Main Tower Section Force Summary Table

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

Source: JICA Study Team

Table 4.2.180 Maximum Cable Tension

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

Source: JICA Study Team

(2) Bent Reaction Force

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

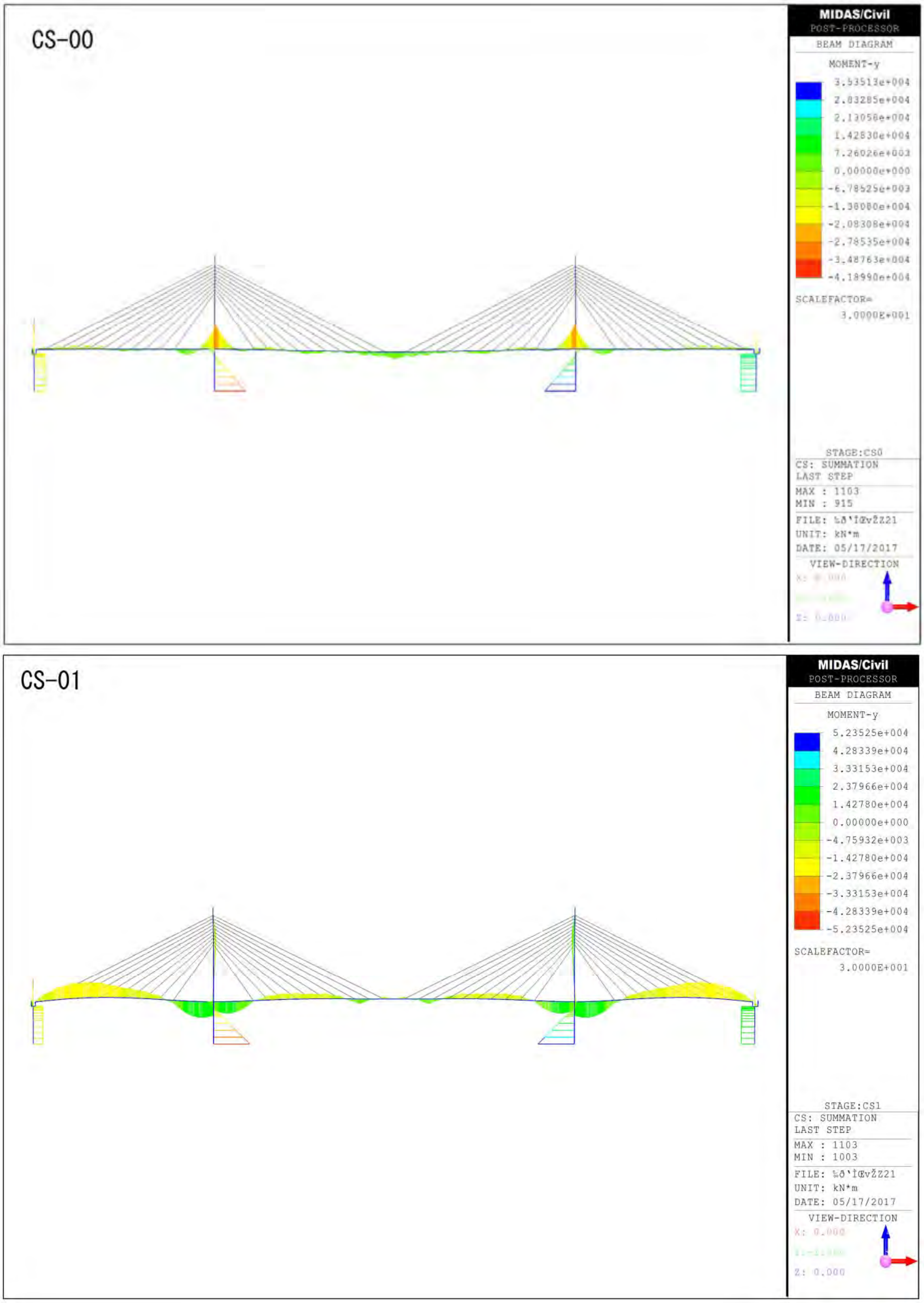
Table 4.2.181 Bent Reaction Table

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

Source: JICA Study Team

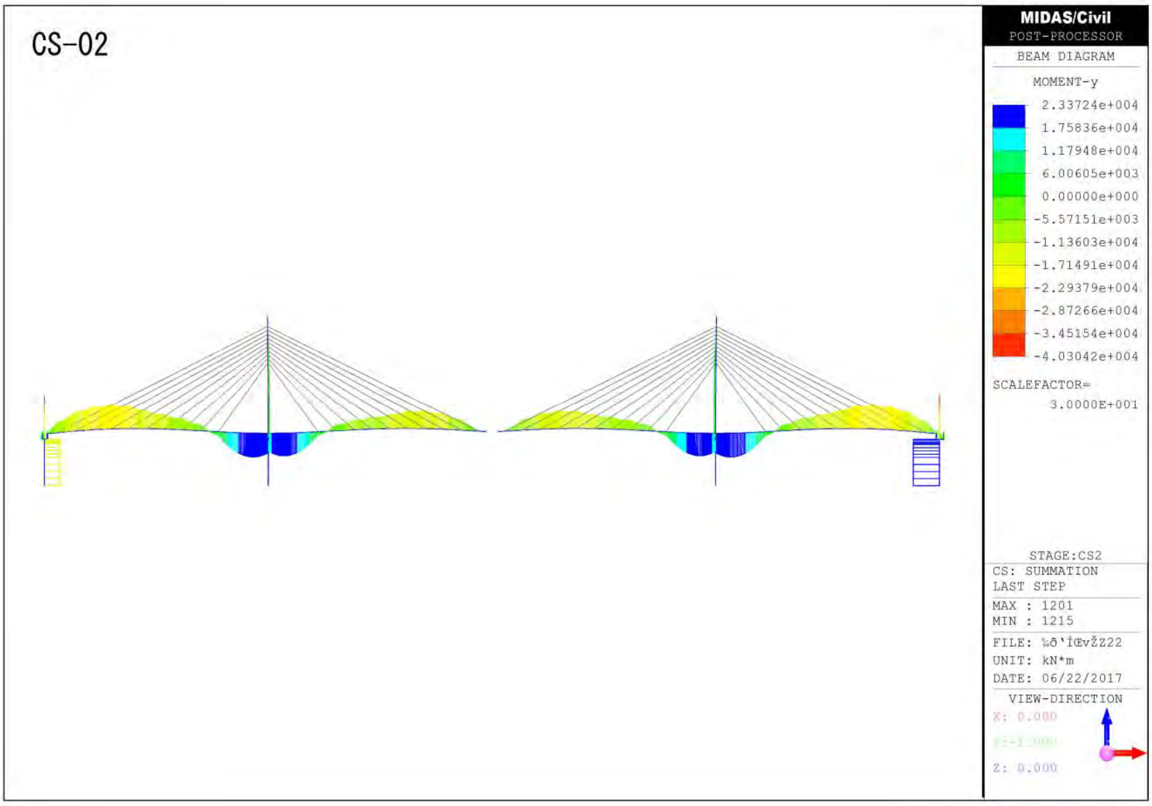
(3) Bending Moment Diagram

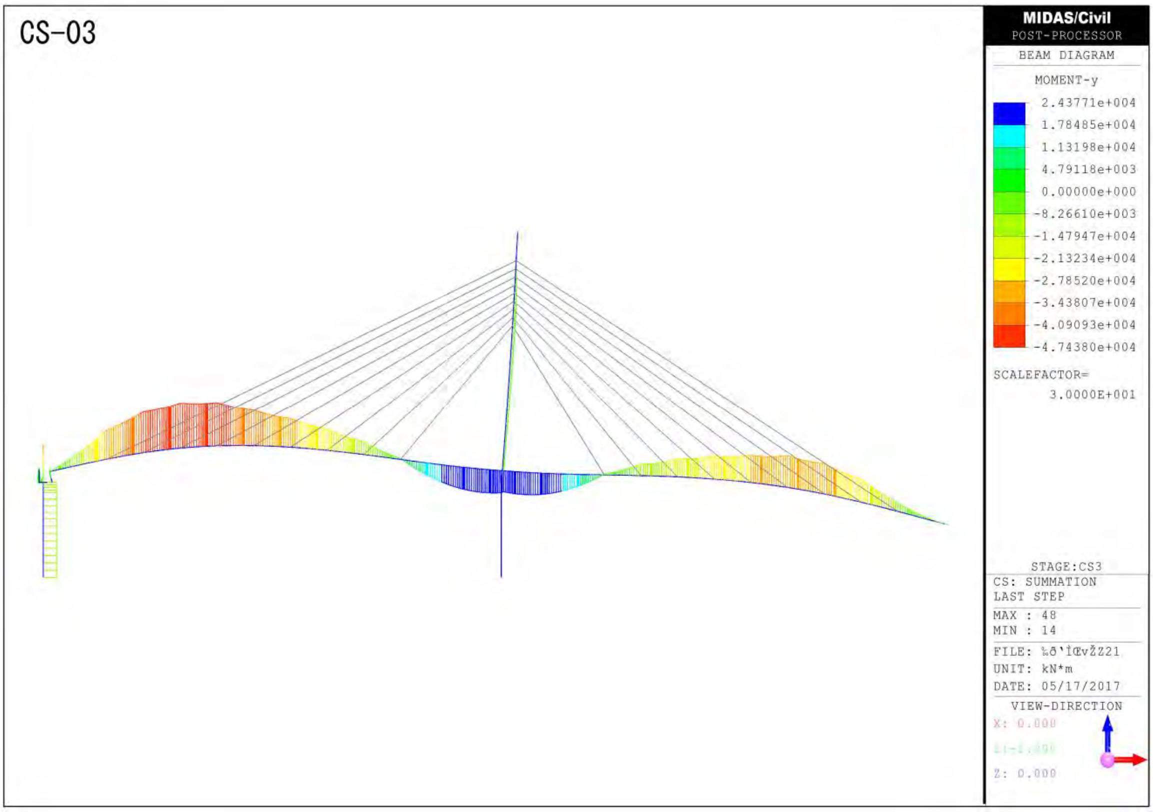
The bending moment diagrams for each analysis stage are shown below.



Source: JICA Study Team

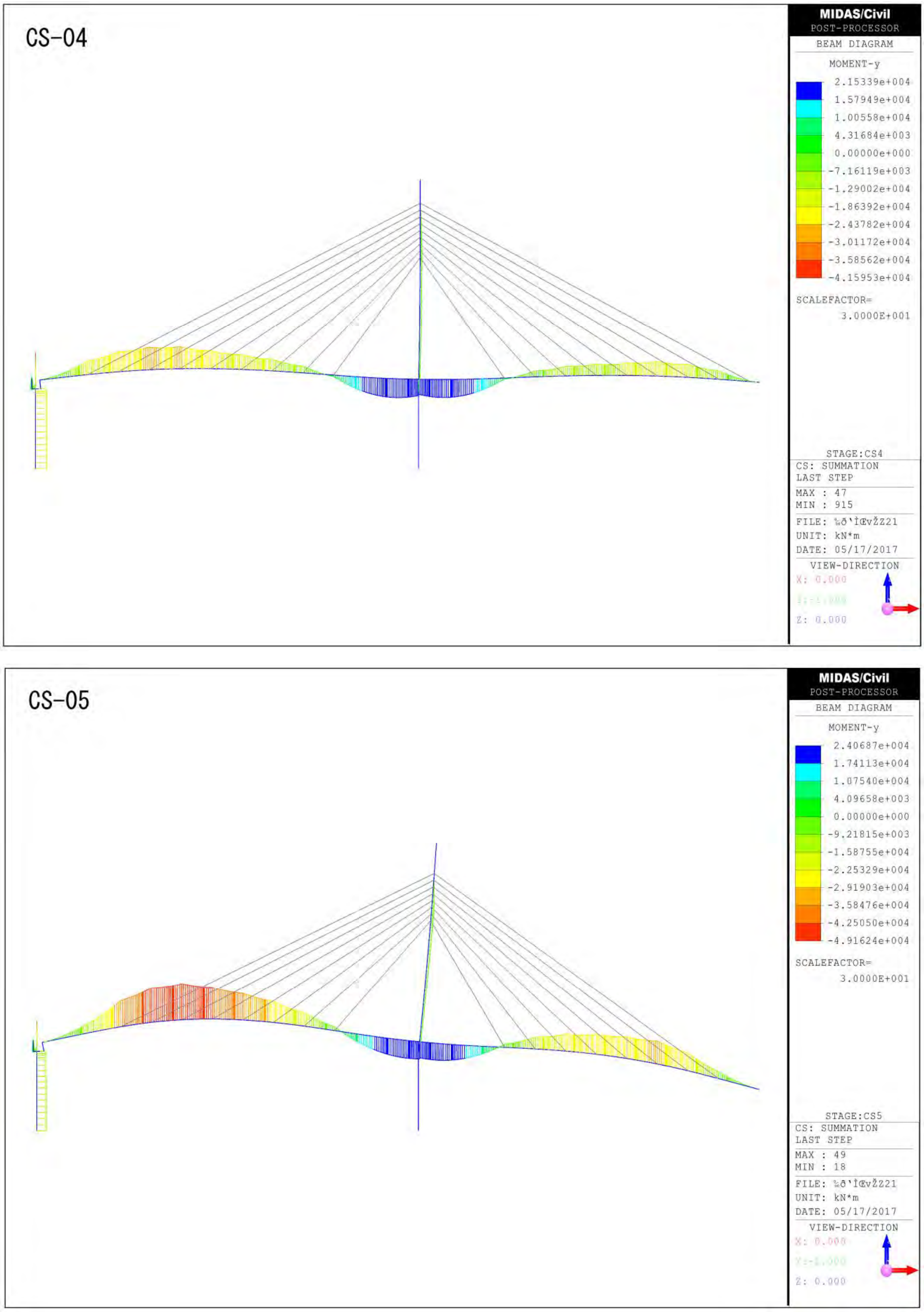
Figure 4.2.235 Bending Moment Diagram (Top: CS0, Bottom: CS1)





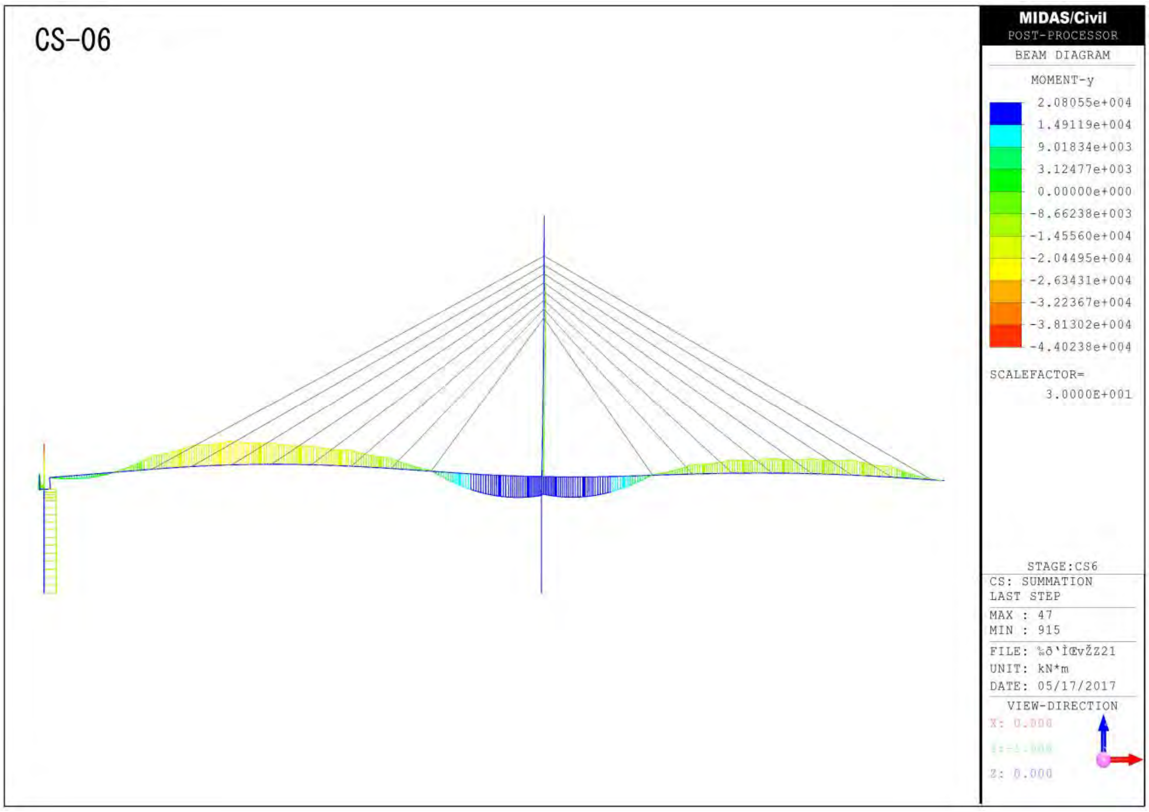
Source: JICA Study Team

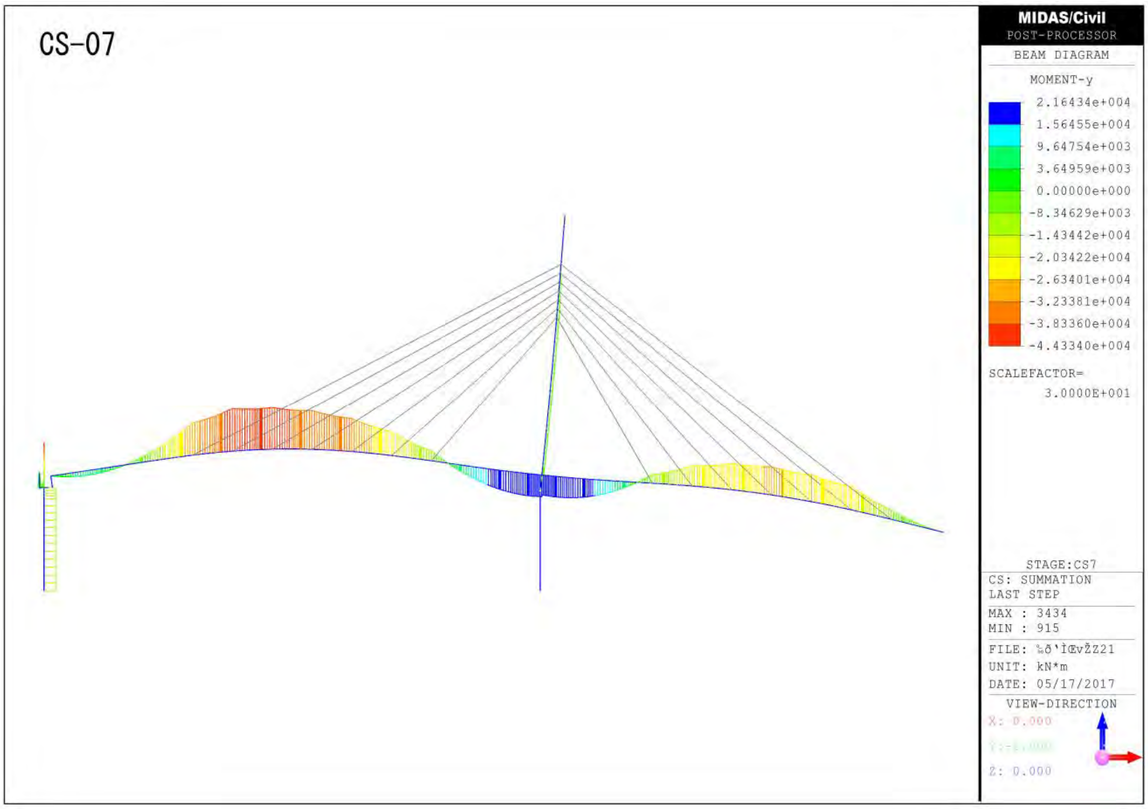
Figure 4.2.236 Bending Moment Diagram (Top: CS2, Bottom: CS3)



Source: JICA Study Team

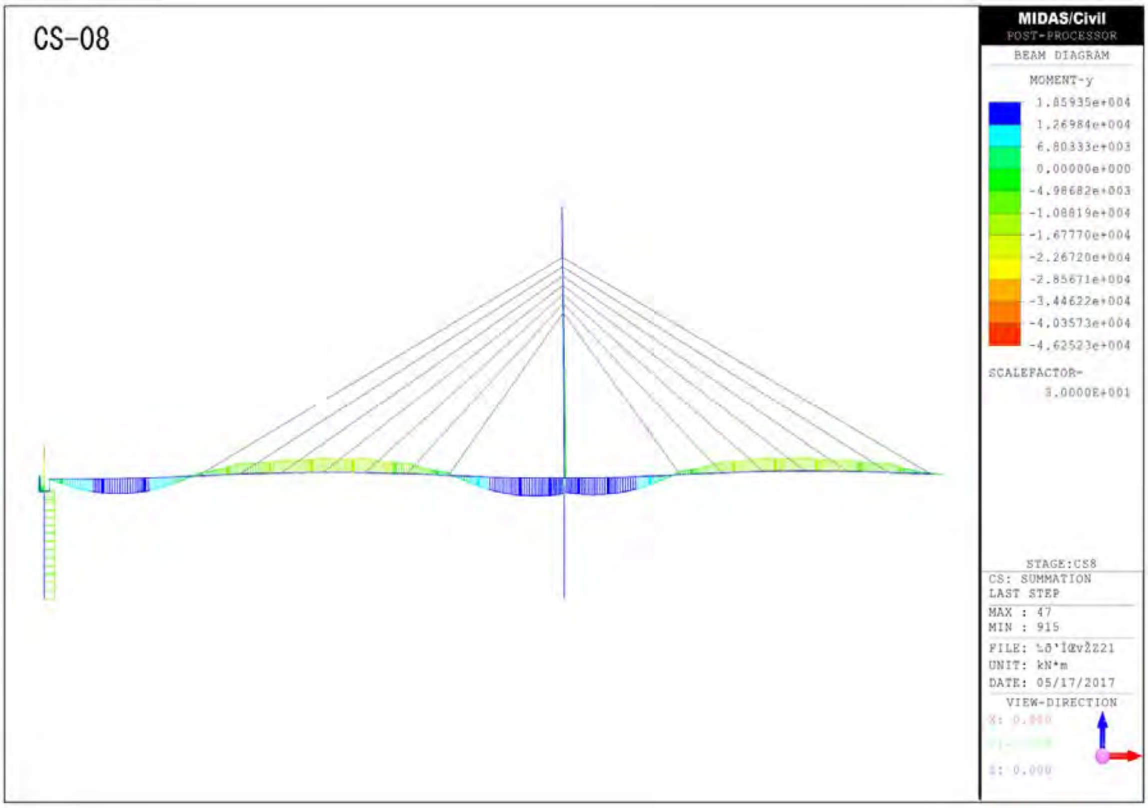
Figure 4.2.237 Bending Moment Diagram (Top: CS4, Bottom: CS5)

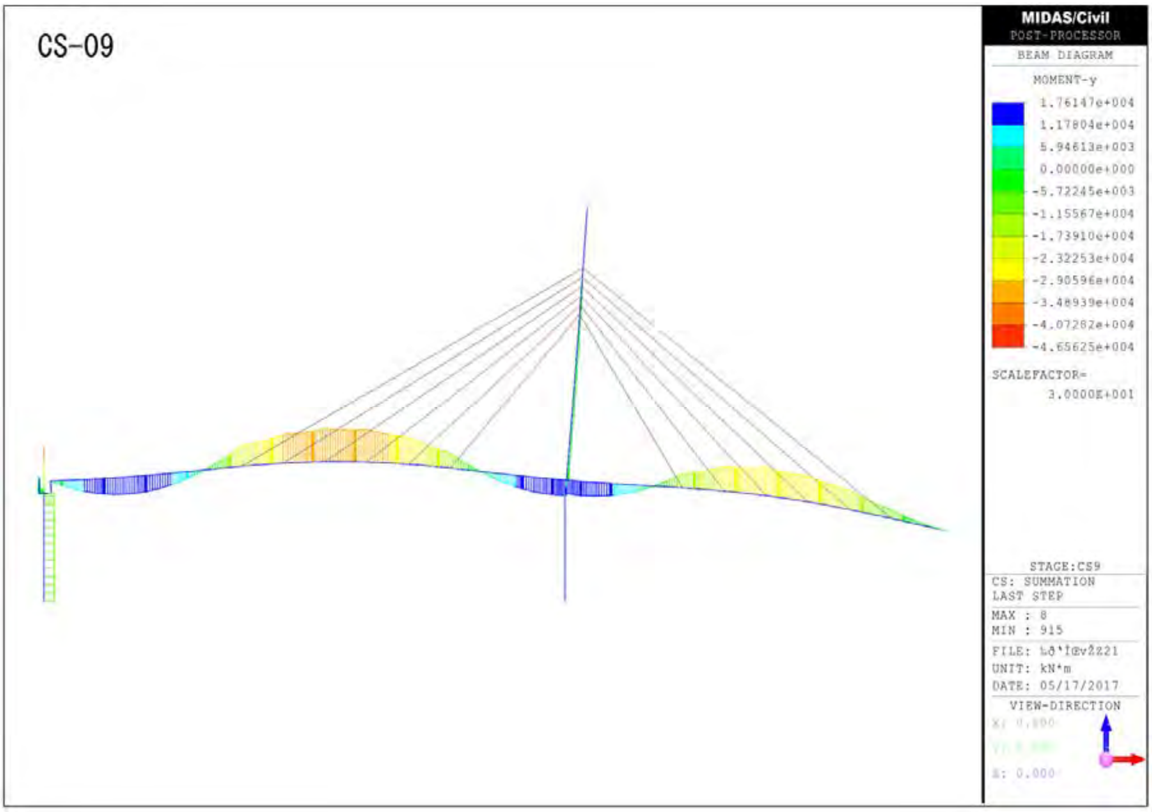




Source: JICA Study Team

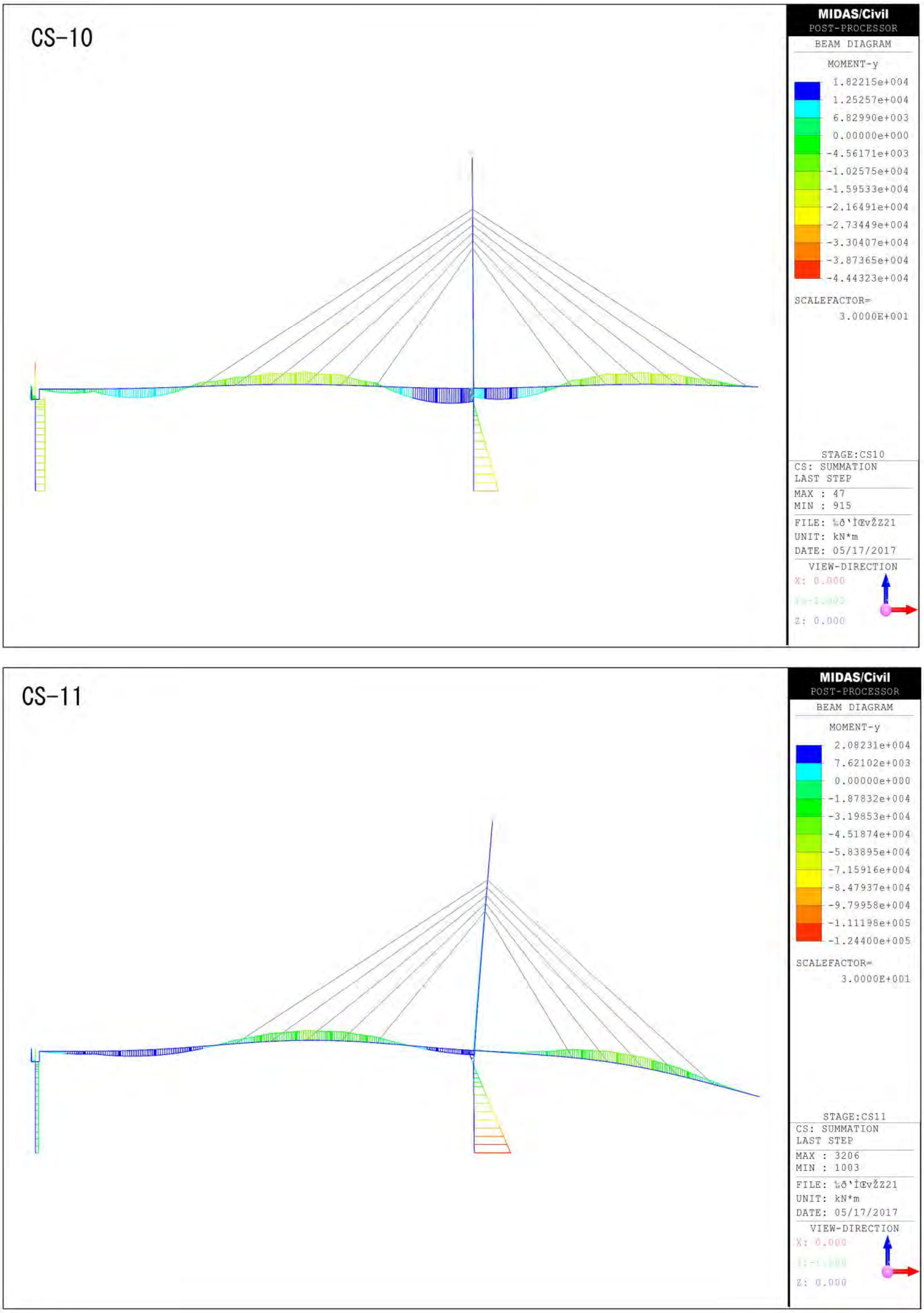
Figure 4.2.238 Bending Moment Diagram (Top: CS6, Bottom: CS7)





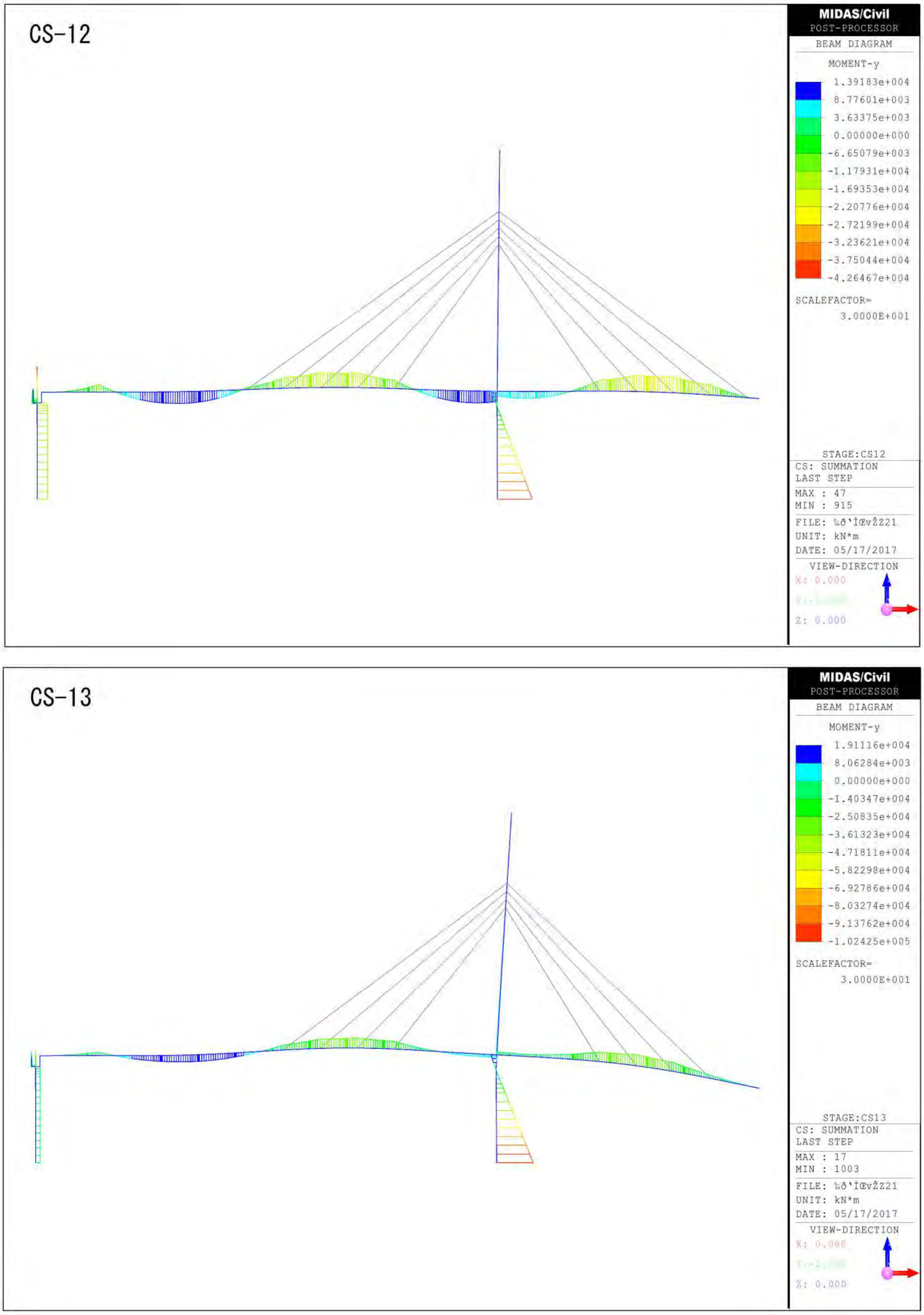
Source: JICA Study Team

Figure 4.2.239 Bending Moment Diagram (Top: CS8, Bottom: CS9)



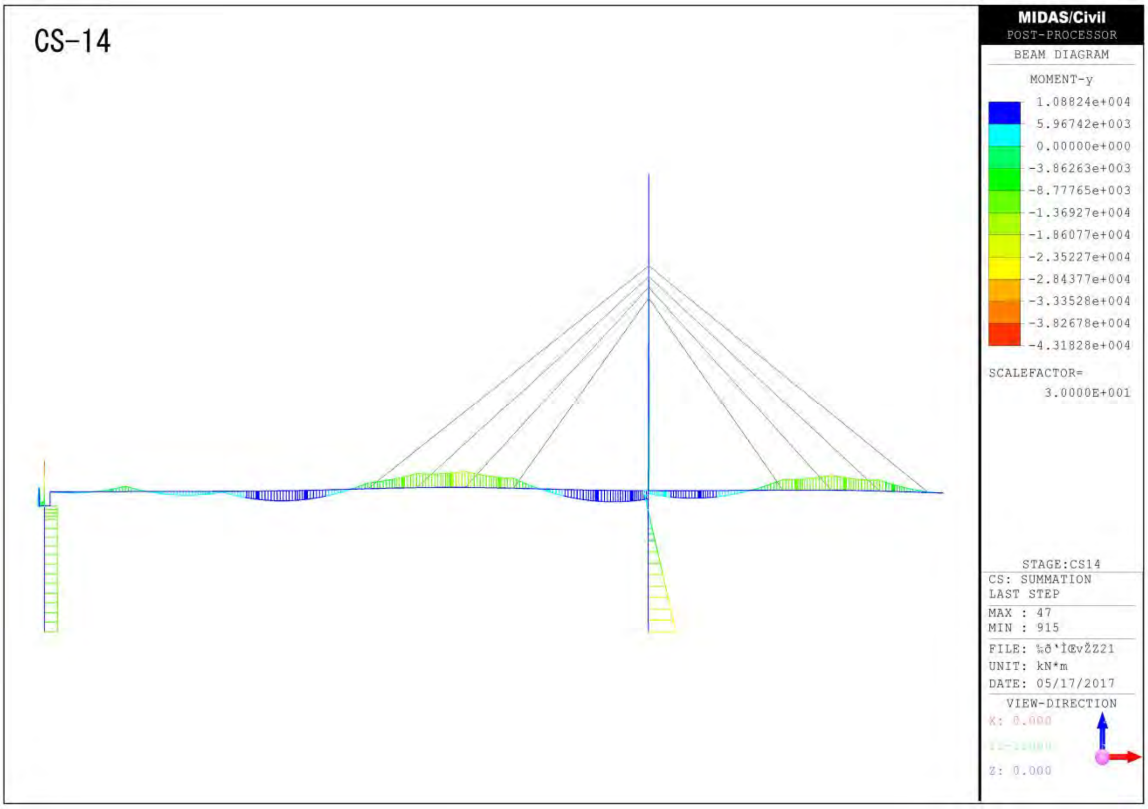
Source: JICA Study Team

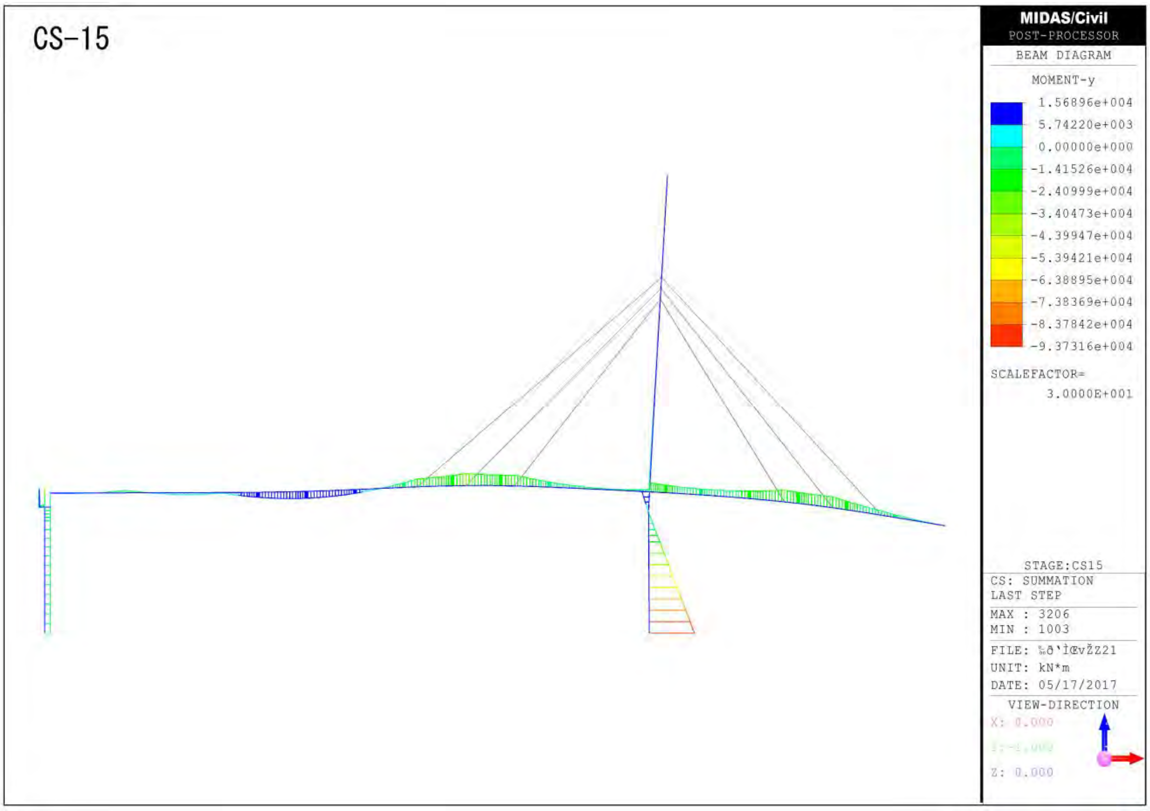
Figure 4.2.240 Bending Moment Diagram (Top: CS10, Bottom: CS11)



Source: JICA Study Team

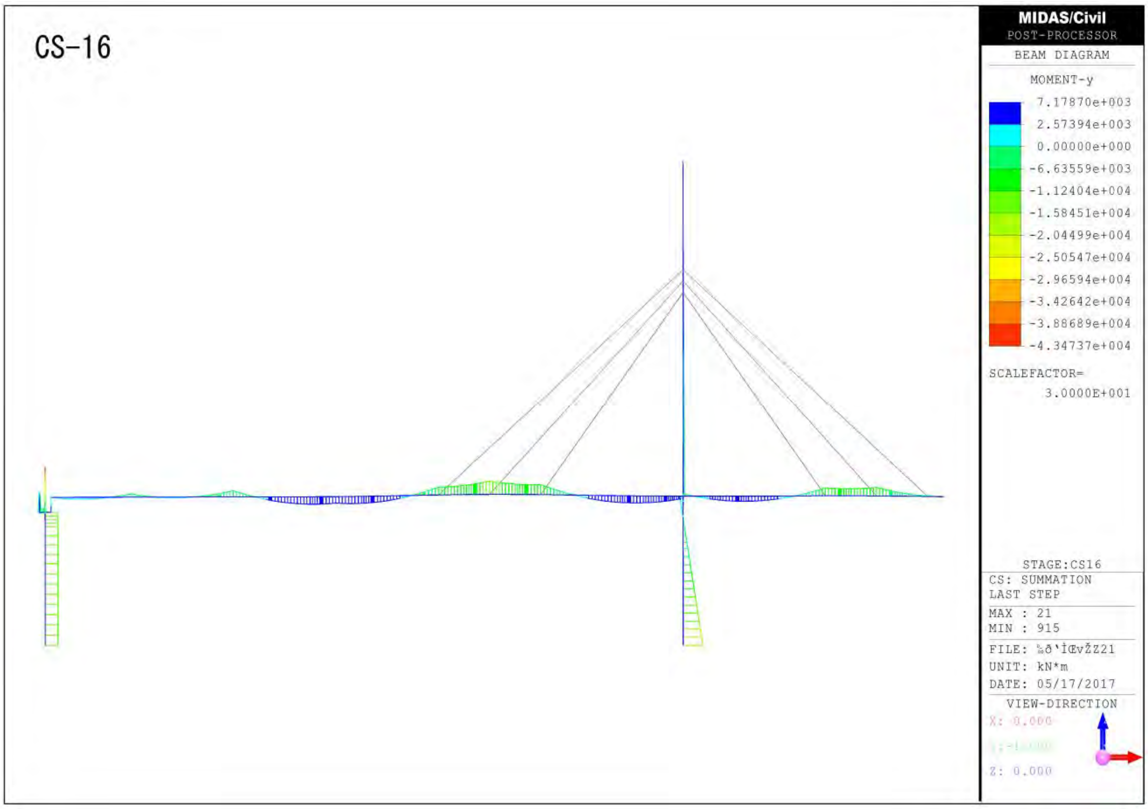
Figure 4.2.241 Bending Moment Diagram (Top: CS12, Bottom: CS13)

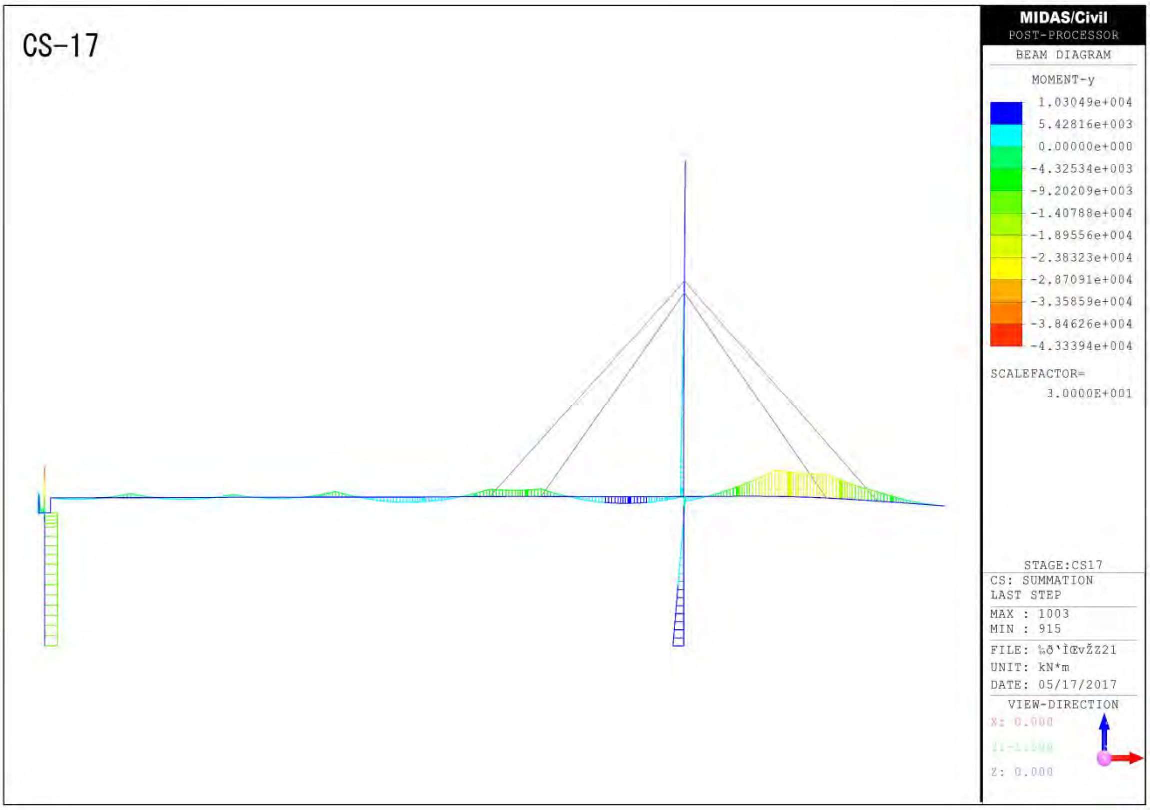




Source: JICA Study Team

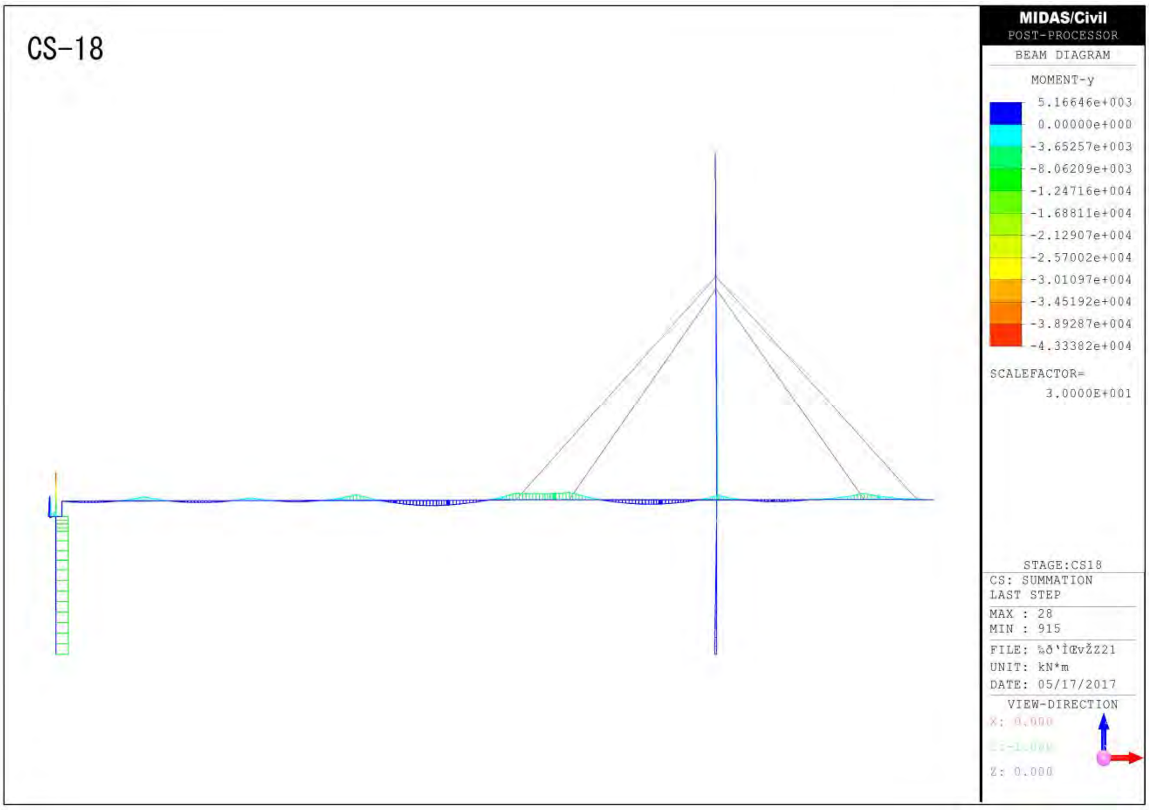
Figure 4.2.242 Bending Moment Diagram (Top: CS14, Bottom: CS15)

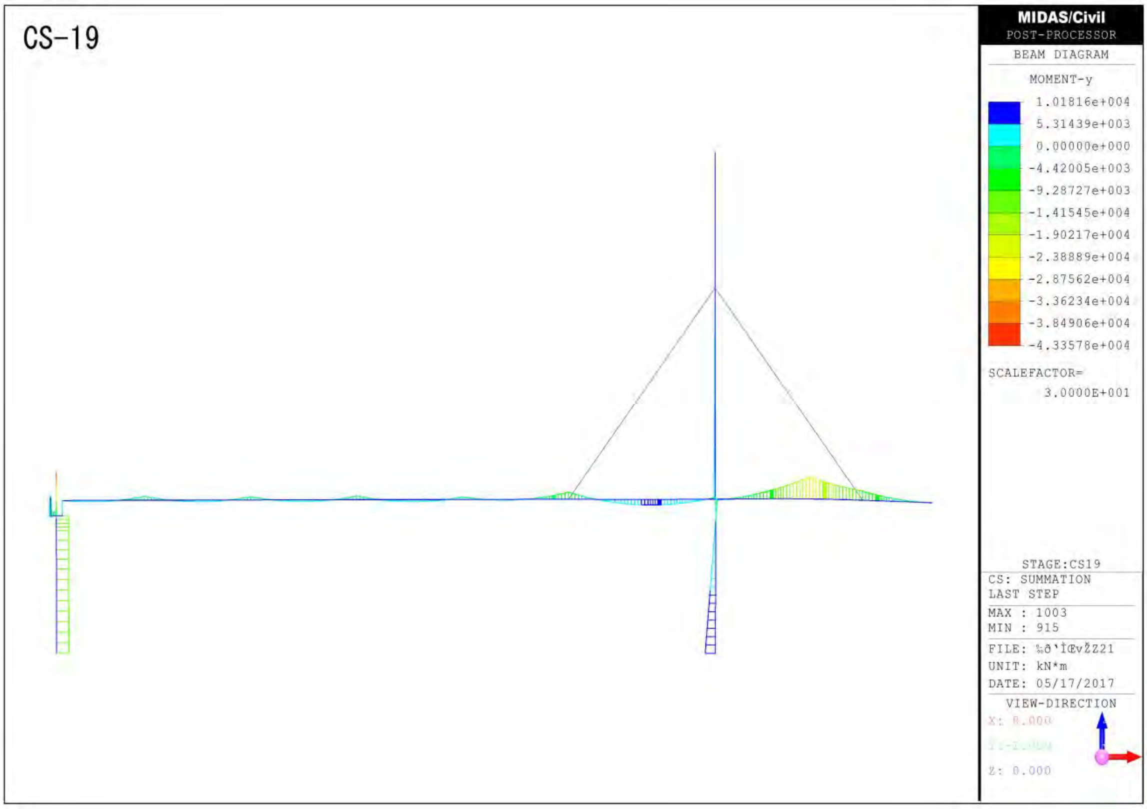




Source: JICA Study Team

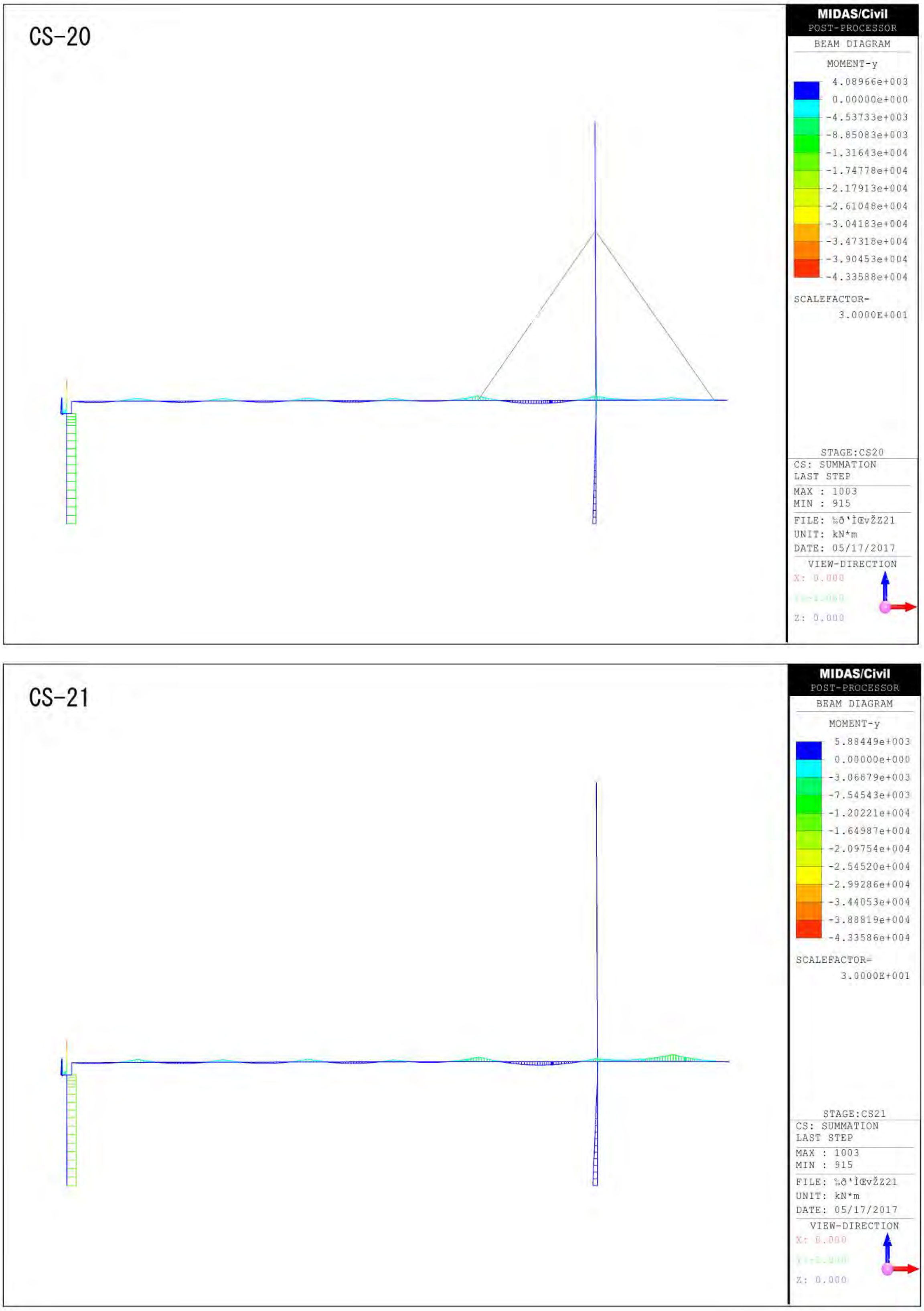
Figure 4.2.243 Bending Moment Diagram (Top: CS16, Bottom: CS17)





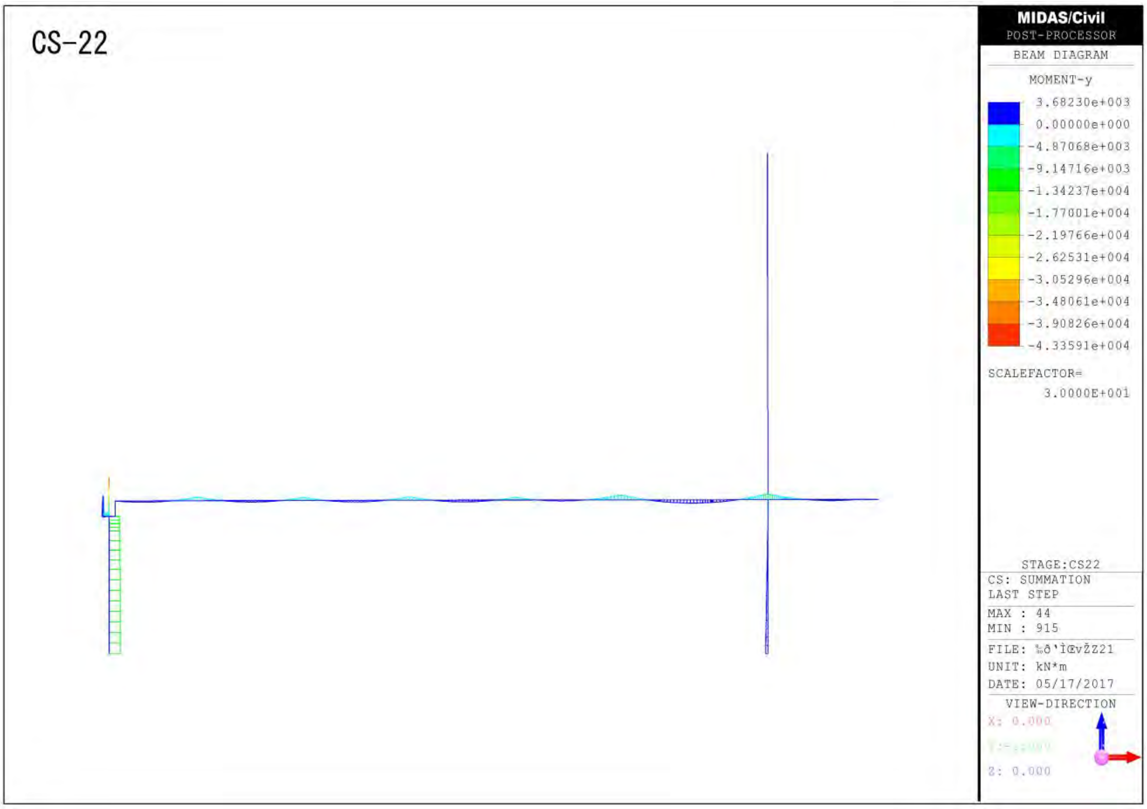
Source: JICA Study Team

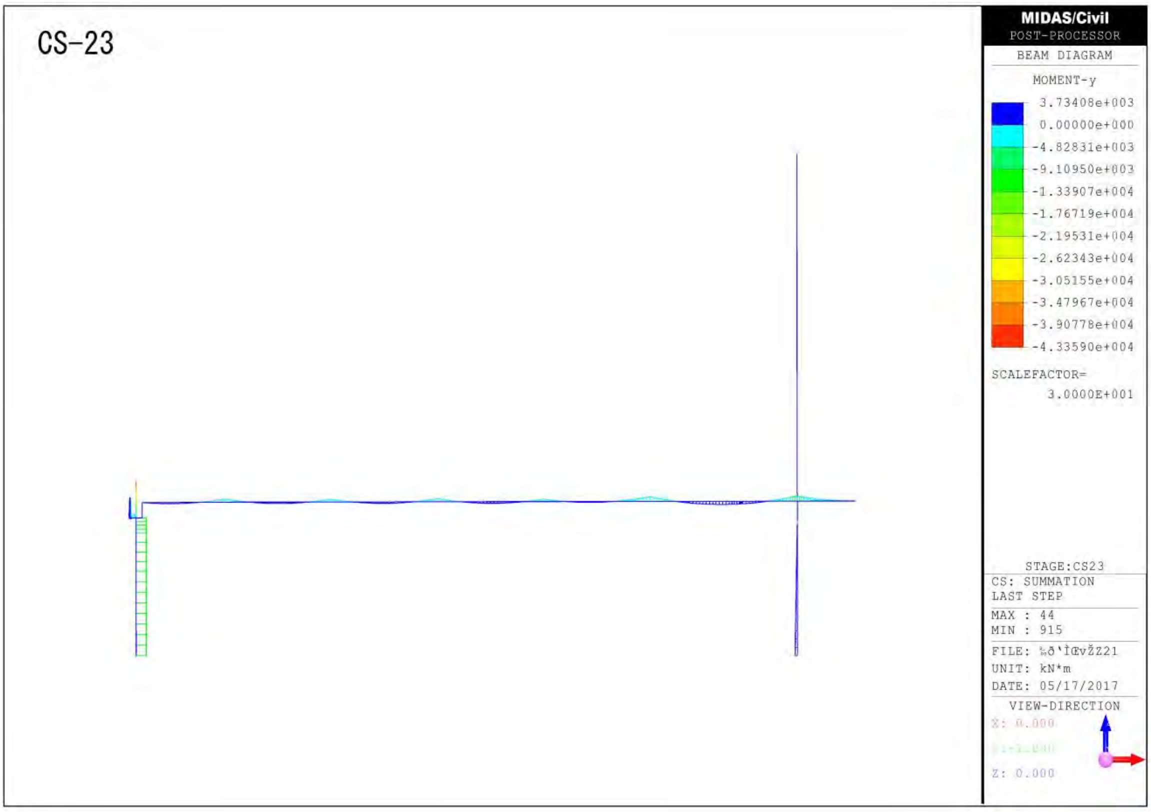
Figure 4.2.244 Bending Moment Diagram (Top: CS18, Bottom: CS19)



Source: JICA Study Team

Figure 4.2.245 Bending Moment Diagram (Top: CS20, Bottom: CS21)





Source: JICA Study Team

Figure 4.2.246 Bending Moment Diagram (Top: CS22, Bottom: CS23)

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

4.2.14.1 Pier Design

(1) Design Conditions

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

1) Reaction Force for Substructure Design

Design reaction force (before and after) are shown in the table below.

Reaction force at P10 is larger than P13, therefore following reaction force is used as the design force for substructure of P10 and P13.

Table 4.2.182 Reaction Force for Substructure Design

		P10 (Before)			P10 (After)		
		Cable Stayed Bridge＋PC Box			Cable Stayed Bridge＋3-span Steel Box		
		Rv(KN)	RH(KN)	RM(KNm)	Rv(KN)	RH(KN)	RM(KNm)
Longi. Direction	Reguler HWL	10200	450	12400	6600	0	10600
	Reguler LWL	19000	450	12400	15200	0	10600
	Temp. HWL	10100	750	12400	6500	900	10600
	Temp. LWL	19300	750	12400	15500	900	10600
	Wind	12800	0	12400	9200	0	10600
	Vessel Impact	10200	450	12400	6600	0	10600
	Seismic	12200	4350	12400	8600	3900	24250
Trans. Direction	Reguler HWL	10200	100	16800	6600	100	16700
	Reguler LWL	19000	100	16800	15200	100	16700
	Wind	12800	600	4620	9200	800	5300
	Vessel Impact	10200	100	16800	6600	100	16700
	Seismic	12800	4300	16010	9200	4100	15750

Source: JICA Study Team

As shown in the section 4.2.10.2, substructure design with PC box girder bridge version has already conducted. Therefore, substructure re-design based on the adobe reaction force was conducted, and some reinforcement was reduced in the revised design.

2) Effect to the Dynamic Static Analysis

As shown in the section 4.2.10.2, column axis reinforcement was decided by the dynamic analysis in transverse direction. The shared weight of the adjacent bridge is shown in the table below.

Table 4.2.183 Shared Weight of Adjacent Bridge

	P10 (Before)	P10 (After)
	PC Box Girder Shared Weight	3-span Steel Box Girder Shared Weight
Longi. Direction	13000	11760
Trans. Direction	9000	8328

Source: JICA Study Team

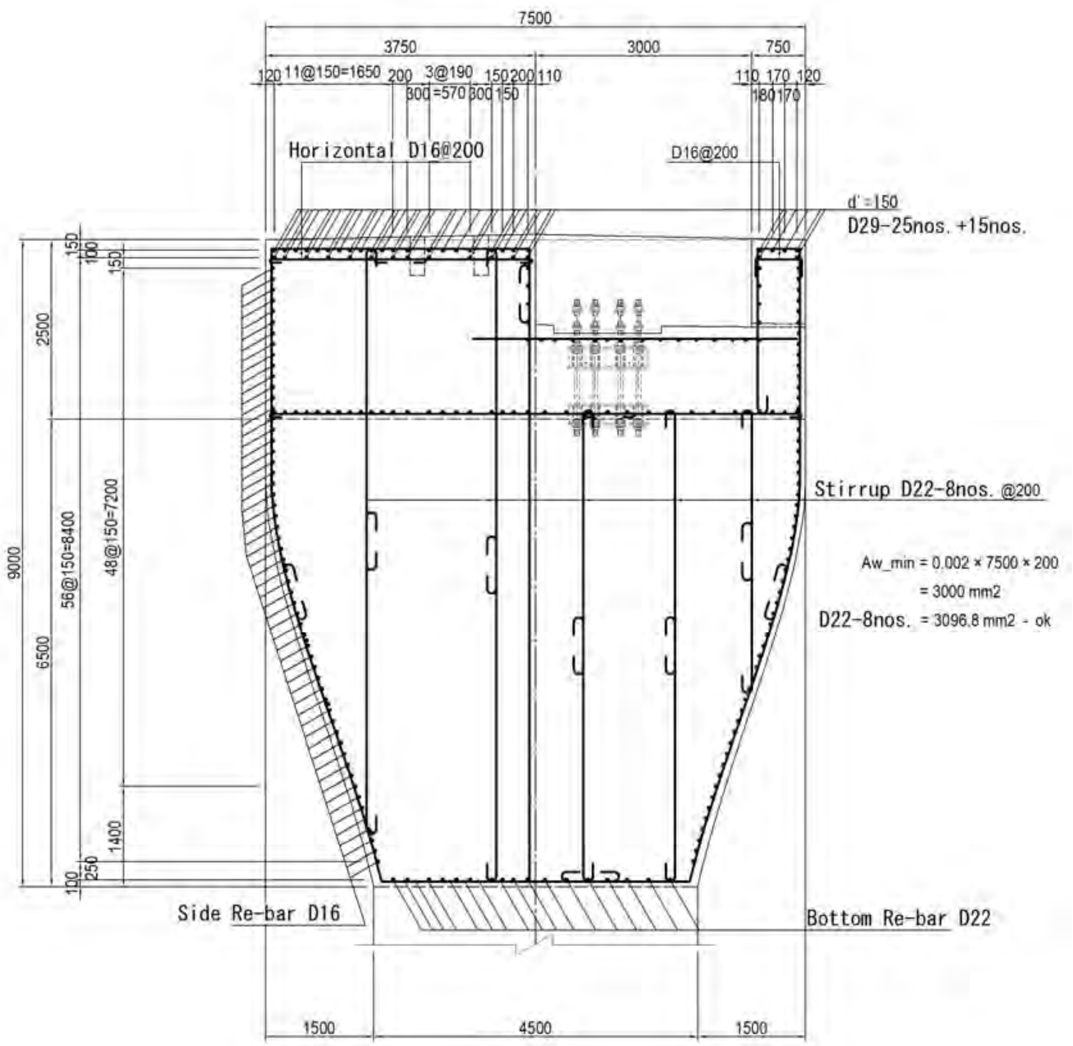
From the table, the shared weight difference in transverse direction is less than 8%. And effect to the dynamic analysis caused by the difference can be considered as a little or nothing. Therefore, the dynamic analysis was not conducted again in the revised design.

Furthermore, the column dimension and axial reinforcement were not changed and pier rigidity become same with the previous design. Therefore, it can be considered that re-calculation of the static structure analysis is not necessary.

(2) Pier Design

1) Beam Design

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



Source: JICA Study Team

Figure 4.2.247 Cross Section of Beam

[Overview of Calculation Result]

The following table shows the calculation results for the beam.

Table 4.2.184 Calculation Results for Beam

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

Source: JICA Study Team

a) Cross Section Design in Vertical Direction (as a Corbel)

The design tension force needs to be verified because the ratio of the beam height to the distance between root and loading point is more than 1.0.

Table 4.2.185 Evaluation of Amount of Steel Reinforcement

Item	Unit	Dead Load	Dead + Live Load
Load Condition	——	Dead Load	Regular Load
Design Tensile Force T	kN	2108.00	2754.57
Allowable Tensile Force σsa	N/mm²	100.00	180.00
Upper surface tension Re-bar Used Amount Asu Required Amount AsuReq	mm²	Asu ≧ AsuReq OK 25696.00 21080.03	Asu ≧ AsuReq OK 25696.00 15303.17
Additional reinforcement for side surface Used Amount Ass Required Amount AssReq	mm²	Ass ≧ AssReq OK 19462.80 10278.40	Ass ≧ AssReq OK 19462.80 10278.40

※ AsuReq = 1000・T / σsa

※ AssReq = 0.4・Asu

Source: JICA Study Team

b) Cross Section Design in Vertical Direction (Allowable Stress Method)

- Evaluation for Bending Moment

The evaluation for the bending moment was performed at the root of the beam.

Table 4.2.186 Evaluation Results for Cross Section

Item	Unit	Dead Load	Dead + Live Load
Load Consition	——	Dead Load	Regular Load
Bending Moment M	kN.m	15790.26	20633.46
Compressive Edge-Nutral Axis x	mm	1126	1126
Compressive Stress σc Tensile Stress σs	N/mm^2 N/mm^2	0.71 73.25	0.93 95.71
Increase Factor α Allowable Compressive Stress σca Allowable Tensile Stress σsa	—— N/mm^2 N/mm^2	1.00 10.00 100.00	1.00 10.00 180.00
Minimum Reinforcement Amount as Bending Element	——	1.7M≦Mc	1.7M≦Mc

Note: Cracking Bending Moment Mc = 200477.52 kNm, Ultimate Bending Moment Mu = 77771.30 kNm

Source: JICA Study Team

- Evaluation for Shear Force

The evaluation for the shear force was performed at the H/2 point from the beam root and bearing support position outside the H/2 point.

Table 4.2.187 Evaluation Result for Cross Section

Item	Unit	Dead Load	Dead + Live Load
State	——	Dead Load	Regular Load
S M	kN kN.m	124.10 15.13	124.10 15.13
d	mm	2719	2719
Sh	kN	115.05	115.05
α pt ce cpt	—— % —— ——	1.00 0.126 0.742 0.752	1.00 0.126 0.742 0.752
τm $τa_1$ $τa_2$	N/mm^2 N/mm^2 N/mm^2	0.006 0.140 1.900	0.006 0.140 1.900

※ $Sh = S - M / d \cdot (\tan\beta + \tan\gamma)$

$\tau m = Sh / bd$

Source: JICA Study Team

c) Cross Section Design in Horizontal Direction (Allowable Stress Method)

The evaluation for the bending moment was performed at the root of the beam.

Table 4.2.188 Evaluation Result for Cross Section

Item	Unit	Temp. Flux	Seismic
Load Consition	——	Dead + Temp.	Lv1 Seismic
Bending Moment M	kN.m	756.75	5947.88
Compressive Edge-Nutral Axis x	mm	719	719
Compressive Stress σc Tensile Stress σs	N/mm² N/mm²	0.09 12.36	0.70 97.14
Increase Factor α Allowable Compressive Stress σca Allowable Tensile Stress σsa	—— N/mm² N/mm²	1.15 11.50 207.00	1.50 15.00 300.00
Minimum Reinforcement Amount as Bending Element	——	1.7M≦Mc	1.7M≦Mc

Note: Cracking Bending Moment Mc = 142980.53 kNm, Ultimate Bending Moment Mu = 23344.64 kNm
Source: JICA Study Team

- Evaluation for Shear Force

The evaluation for the shear force was performed at the beam root and bearing support position.

Table 4.2.189 Evaluation Result for Cross Section

Item	Unit	Temp.	Seismic
State	——	Dead + Temp.	Lv1 Seismic
S M	kN kN.m	250.00 756.75	2444.73 5947.88
d	mm	6980	6980
Sh	kN	250.00	2444.73
α pt ce cpt	—— % —— ——	1.15 0.018 0.560 0.536	1.50 0.018 0.560 0.536
τm τa_1 τa_2	N/mm² N/mm² N/mm²	0.005 0.086 2.185	0.045 0.111 2.850

Source: JICA Study Team

d) Cross Section Design in Horizontal Direction (Evaluation for Seismic Performance 2)

The evaluation for the bending moment was performed at the root of the beam.

Table 4.2.190 Evaluation Result for Cross Section

Item	Unit	Seismic Performance 2
Load Condition	—	Type 2
Bending Moment M	kN.m	7559.67
Yieleding Bengind Moment My	kN.m	21370.63

Note; Cracking Bending Moment Mc = 142980.53 kNm, Ultimate Bending Moment Mu = 23344.64 kNm
Source: JICA Study Team

- Evaluation for Shear Force

The evaluation for the shear force was performed at the beam root and bearing support position.

Table 4.2.191 Evaluation of Shear Strength

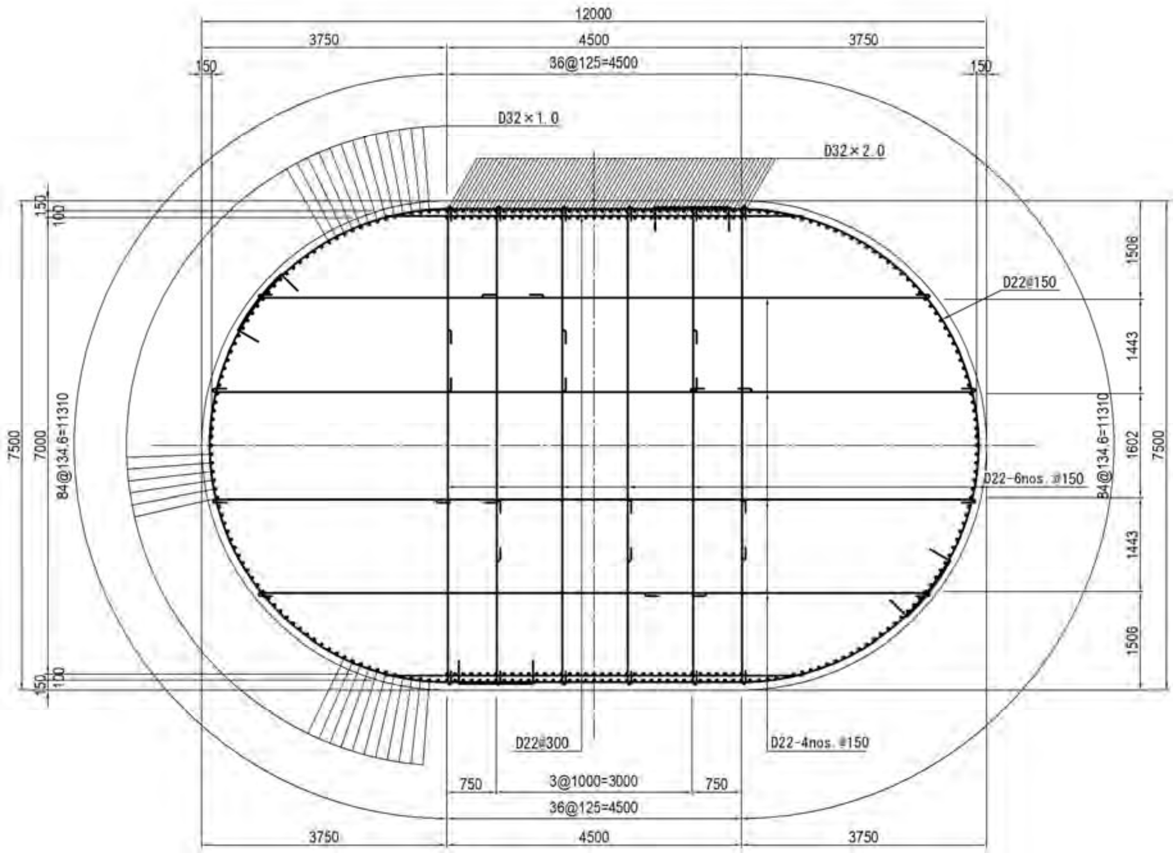
No	Verified Point x(m)	Sc (kN)	Ss (kN)	Ps (kN)	Sh (kN)
1	0.000	5975.44	10184.77	16160.21≧	3217.10
2	3.027	4021.03	10557.22	14578.25≧	1594.17

Source: JICA Study Team

2) Design of Column

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

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



Source: JICA Study Team

Figure 4.2.248 Cross Section of Column

[Overview of Calculation Result]

The following table shows the calculation results for the column.

Table 4.2.192 Calculation Result for Column

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

Note: ※ was decided by dynamic analysis

Source: JICA Study Team

a) Cross Section Evaluation Results

The evaluation results for the column cross section are shown below.

Table 4.2.193 Examination of Bending Moment (Longitudinal)

Category	Unit	Regular Sceanrio HWL Water Level Considered	Regular Sceanrio LWL Water Level Considered	Temp. HWL Water Level Considered	Temp. LWL Water Level Considered
Load Condition	——	Dead Load	Regular Load	Dead + Temp.	Dead+Live+Temp.
Axial Force N	kN	59485.14	68085.14	59385.14	68385.14
Bending Moment M	kN.m	11780.8	11780.8	32660.8	32660.8
Compression Edge ～Neutral Axis x	mm	25291	28405	11507	12682
Compressive Stress σc	N/mm^2	0.86	0.96	1.08	1.19
Tensile Stress σs	N/mm^2	-9.1	-10.68	-5.84	-7.5
Increase Coefficient α	——	1	1	1.15	1.15
Allowable Comp. Stress σca	N/mm^2	10	10	11.5	11.5
Allowable Tens. Stress σsa	N/mm^2	-200	-200	-230	-230
Cracking Moment Mc	kN.m	249473.41	258699.84	249366.13	259021.69
Yield Moment My0	kN.m	421286.12	445005.6	421007.66	445826.88
Ultimate Bending Moment Tu	kN.m	507688.27	535206.45	507368.7	536163.01
Min. Re-bar for Bending Elem.	——	1.7M≦Mc	1.7M≦Mc	1.7M≦Mc	1.7M≦Mc
Min. Re-bar for Axial Elem.	mm^2	47116.9	53928.8	40902.4	47101.3
Axial Force Nu	kN	62085.14	62085.14	62085.14	62085.14
0.008A1' (Axial Force Na=N)	mm^2	47116.9	53928.8	40902.4	47101.3
0.008A2' (Axial Force Nu)	mm^2	17575.4	17575.4	17575.4	17575.4
Total Re-bar As ≧ Asmin	——	OK	OK	OK	OK
Max. Re-bar Check (My0≦Mu)	——	OK	OK	OK	OK

Category	Unit	Wind Water Level Considered	Vessel Impact Water Level Considered	Seismic Water Level Considered
Load Condition	——	Wind	Impact	Lv1 Seismic
Axial Force N	kN	62085.14	59485.14	61485.14
Bending Moment M	kN.m	16811.33	75315.8	328881.18
Compression Edge ～Neutral Axis x	mm	19505	7095	2392
Compressive Stress σc	N/mm^2	0.94	1.54	7.43
Tensile Stress σs	N/mm^2	-8.8	0.83	230.98
Increase Coefficient α	—	1.25	1.5	1.5
Allowable Comp. Stress σca	N/mm^2	12.5	15	15
Allowable Tens. Stress σsa	N/mm^2	-250	300	300
Cracking Moment Mc	kN.m	252262.79	249473.41	251619.09
Yield Moment My0	kN.m	428485.97	421286.12	426826.7
Ultimate Bending Moment Tu	kN.m	516041.59	507688.27	514118.48
Min. Re-bar for Bending Elem.	———	1.7M≦Mc	1.7M≦Mc	Mc≦Mu
Min. Re-bar for Axial Elem.	mm^2	39341.1	31411.3	32467.4
Axial Force Nu	kN	62085.14	62085.14	62085.14
0.008A1' (Axial Force Na=N)	mm^2	39341.1	31411.3	32467.4
0.008A2' (Axial Force Nu)	mm^2	17575.4	17575.4	17575.4
Total Re-bar As ≧ Asmin	———	OK	OK	OK
Max. Re-bar Check (My0≦Mu)	———	OK	OK	OK

Source: JICA Study Team

Table 4.2.194 Examination of Bending Moment (Transverse)

Category	Unit	Regular Sceanrio HWL Water Level Considered	Regular Sceanrio LWL Water Level Considered	Wind Water Level Considered
Load Condition	——	Dead Load	Regular Load	Wind
Axial Force N	kN	59485.14	68085.14	62085.14
Bending Moment M	kN.m	19020	19020	24941.93
Compression Edge ～Neutral Axis x	mm	36601	41025	30355
Compressive Stress σc	N/mm²	0.87	0.98	0.95
Tensile Stress σs	N/mm²	-8.84	-10.42	-8.66
Increase Coefficient α	——	1	1	1.25
Allowable Comp. Stress σca	N/mm²	10	10	12.5
Allowable Tens. Stress σsa	N/mm²	-200	-200	-250
Cracking Moment Mc	kN.m	374152.67	387990.19	378336.11
Yield Moment My0	kN.m	544064.18	579301.5	554760.74
Ultimate Bending Moment Tu	kN.m	766885.28	807164.16	779117.38
Min. Re-bar for Bending Elem.	——	1.7M≦Mc	1.7M≦Mc	1.7M≦Mc
Min. Re-bar for Axial Elem.	mm²	47116.9	53928.8	39341.1
Axial Force Nu	kN	62085.14	62085.14	62085.14
0.008A1' (Axial Force Na=N)	mm²	47116.9	53928.8	39341.1
0.008A2' (Axial Force Nu)	mm²	17575.4	17575.4	17575.4
Total Re-bar As ≧ Asmin	——	OK	OK	OK
Max. Re-bar Check (My0≦Mu)	——	OK	OK	OK

Category	Unit	Vessel Impact Water Level Considered	Seismic Water Level Considered
Load Condition	——	Impact	Lv1 Seismic
Axial Force N	kN	59485.14	62085.14
Bending Moment M	kN.m	146090	316664.43
Compression Edge ～Neutral Axis x	mm	9483	4863
Compressive Stress σc	N/mm²	1.89	5.02
Tensile Stress σs	N/mm²	7.09	108.22
Increase Coefficient α	——	1.5	1.5
Allowable Comp. Stress σca	N/mm²	15	15
Allowable Tens. Stress σsa	N/mm²	300	300
Cracking Moment Mc	kN.m	374152.67	378336.11
Yield Moment My0	kN.m	544064.18	554760.74
Ultimate Bending Moment Tu	kN.m	766885.28	779117.38
Min. Re-bar for Bending Elem.	——	1.7M≦Mc	Mc≦Mu
Min. Re-bar for Axial Elem.	mm²	31411.3	32784.2
Axial Force Nu	kN	62085.14	62085.14
0.008A1' (Axial Force Na=N)	mm²	31411.3	32784.2
0.008A2' (Axial Force Nu)	mm²	17575.4	17575.4
Total Re-bar As ≧ Asmin	——	OK	OK
Max. Re-bar Check (My0≦Mu)	——	OK	OK

Source: JICA Study Team

Table 4.2.195 Examination of Shear Force (Longitudinal)

Category	Unit	Regular Sceanrio HWL Water Level Considered	Regular Sceanrio LWL Water Level Considered	Temp. HWL Water Level Considered	Temp. LWL Water Level Considered	Wind Water Level Considered
Load Conditio	——	Dead Load	Regular Load	Dead + Temp.	Dead+Live+Temp.	Wind
b	mm	11147	11147	11147	11147	11147
d	mm	6932	6932	6932	6932	6932
S	kN	0	0	900	900	264.04
N	kN	59485.14	68085.14	59385.14	68385.14	62085.14
M	kN.m	11780.8	11780.8	32660.8	32660.8	16811.33
α	——	1.000	1.000	1.150	1.150	1.25
pt	%	0.161	0.161	0.161	0.161	0.161
ce	——	0.561	0.561	0.561	0.561	0.561
cpt	——	0.823	0.823	0.823	0.823	0.823
CN	——	1.000	1.000	1.000	1.000	1
τm	N/mm^2	0.000	0.000	0.012	0.012	0.003
τa_1	N/mm^2	0.115	0.115	0.133	0.133	0.144
τa_2	N/mm^2	1.900	1.900	2.185	2.185	2.375
σsa	N/mm^2	————	————	————	————	————
s	mm	————	————	————	————	————
Sca	kN	————	————	————	————	————
Sh’	kN	————	————	————	————	————
AwReq	mm^2	————	————	————	————	————
Aw	mm^2	————	————	————	————	————

Category	Unit	Vessel Impact Water Level Considered	Seismic Water Level Considered
Load Conditio	——	Impact	Lv1 Seismic
b	mm	11147	11147
d	mm	6932	6932
S	kN	4850	21561.02
N	kN	59485.14	61485.14
M	kN.m	75315.8	328881.18
α	——	1.5	1.5
pt	%	0.161	0.161
ce	——	0.561	0.561
cpt	——	0.823	0.823
CN	——	1	1
τm	N/mm^2	0.063	0.279
τa_1	N/mm^2	0.171	0.171
τa_2	N/mm^2	2.85	2.85
σsa	N/mm^2	————	300
s	mm	————	150
Sca	kN	————	13204.33
Sh’	kN	————	8356.69
AwReq	mm^2	————	693.15
Aw	mm^2	————	3096.8

Here
- S : Shear Force
- N : Axial Load
- M : Bending Moment
- b : Sectional Width of Element
- d : Effective Height
- α : Overdesign factor for allowable stress
- pt : Primary tension bar ratio
- ce : Correction factor of allowable shear force for effective height d
- cpt : Correction factor of allowable shear force for tension bar ratio
- CN : Correction factor due to longitudinal compressive load
- τm : Average shear force
- τa1 : Allowable shear force when only concrete bears shear force
- τa2 : Allowable shear force when shear reinforcement rebar and concrete bears shear force
- σsa : Allowable tensile stress of rebar
- s : Spacing of shear reinforcement rebar
- Sca : Shear force borne by concrete
- Sh’ : Shear force borne by reinforcement rebar
- Awreq : Necessary shear reinforcement content to meet condition τa1 < τm
- Aw : Shear reinforcement content

Source: JICA Study Team

Table 4.2.196 Examination of Shear Force (Transverse)

Category	Unit	Regular Sceanrio HWL Water Level Considered	Regular Sceanrio LWL Water Level Considered	Wind Water Level Considered	Vessel Impact Water Level Considered
Load Conditio	——	Dead Load	Regular Load	Wind	Impact
b	mm	6991	6991	6991	6991
d	mm	11064	11064	11064	11064
S	kN	100	100	858.01	9800
N	kN	59485.14	68085.14	62085.14	59485.14
M	kN.m	19020	19020	24941.93	146090
α	——	1	1	1.25	1.5
pt	%	0.161	0.161	0.161	0.161
ce	——	0.5	0.5	0.5	0.5
cpt	——	0.822	0.822	0.822	0.822
CN	——	1	1	1	1
τm	N/mm^2	0.001	0.001	0.011	0.127
$τa_1$	N/mm^2	0.103	0.103	0.128	0.152
$τa_2$	N/mm^2	1.9	1.9	2.375	2.85
σsa	N/mm^2	——————	——————	——————	——————
s	mm	——————	——————	——————	——————
Sca	kN	——————	——————	——————	——————
Sh’	kN	——————	——————	——————	——————
AwReq	mm^2	——————	——————	——————	——————
Aw	mm^2	——————	——————	——————	——————

Category	Unit	Seismic Water Level Considered
Load Conditio	——	Lv1 Seismic
b	mm	6991
d	mm	11064
S	kN	19975.69
N	kN	62085.14
M	kN.m	316664.43
α	——	1.5
pt	%	0.161
ce	——	0.5
cpt	——	0.822
CN	——	1
τm	N/mm^2	0.258
$τa_1$	N/mm^2	0.152
$τa_2$	N/mm^2	2.85
σsa	N/mm^2	300
s	mm	150
Sca	kN	11768.5
Sh’	kN	8207.2
AwReq	mm^2	426.53
Aw	mm^2	2322.6

Here
S : Shear Force
N : Axial Load
M : Bending Moment
b : Sectional Width of Element
d : Effective Height
α : Overdesign factor for allowable stress
pt : Primary tension bar ratio
ce : Correction factor of allowable shear force for effective height d
cpt : Correction factor of allowable shear force for tension bar ratio
CN : Correction factor due to longitudinal compressive load
τm : Average shear force
τa1 : Allowable shear force when only concrete bears shear force
τa2 : Allowable shear force when shear reinforcement rebar and concrete bears shear force
σsa : Allowable tensile stress of rebar
s : Spacing of shear reinforcement rebar
Sca : Shear force borne by concrete
Sh’ : Shear force borne by reinforcement rebar
Awreq : Necessary shear reinforcement content to meet condition τa1 < τm
Aw : Shear reinforcement content

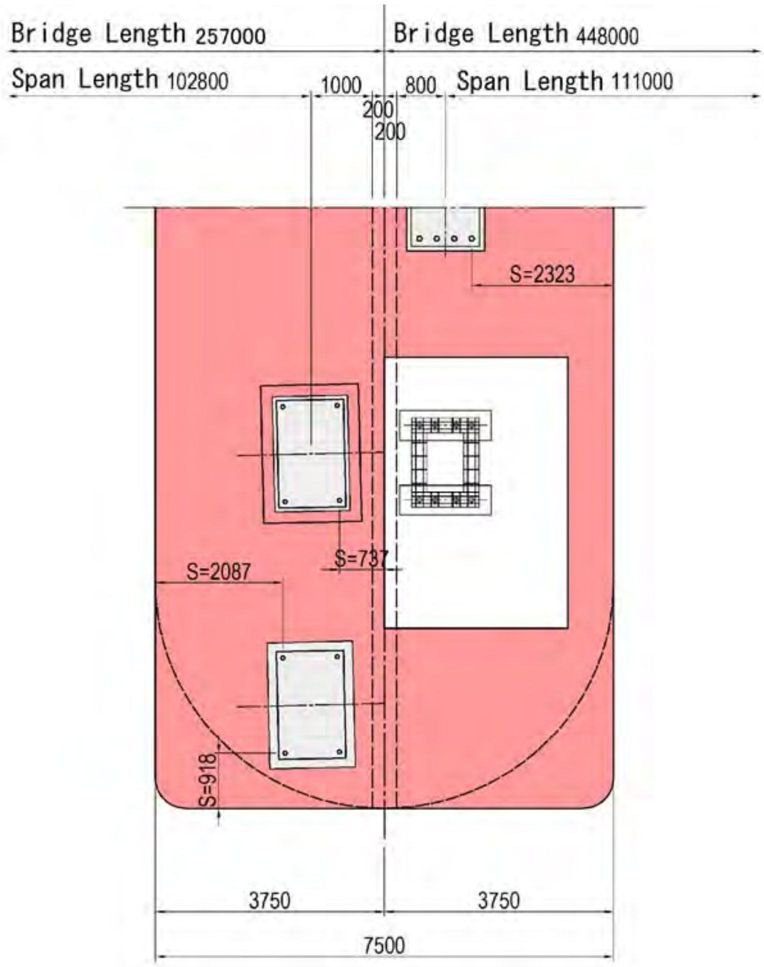
Source: JICA Study Team

3) Bridge Seat Design

a) Dimension of Bridge Seat Width

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

[P10 Pier]



Source: JICA Study Team

Figure 4.2.249 Bridge Seat Width

- Evaluation of edge distance for bearing support

The edge distance of bearing support was set through the following equation:

S1 = 0.2 + 0.0051

= 0.2 + 0.005 × 102.800 = 0.714 m

Hence, the edge distance of bearing support can be set as:

$S1 = 0.714 \text{ m} < 0.737$ ・・・OK

Similarly, the edge distance of the other bearing support was set through the following equation:

S2 = 0.2 + 0.0051

= 0.2 + 0.005 × 111.000 = 0.755 m

Hence, the edge distance of bearing support can be set as:

$S2 = 0.755\ \mathrm{m} < 2.323$ ・・・OK

- Evaluation of length of beam placement on column

The beam placement length is configured to satisfy the following equation:

SEM = 0.7 + 0.0051

= 0.7 + 0.005 × 111.000 = 1.255 m

SE = UR + UG

= 0.560 + 0.555 = 1.115 m

UR = 0.560 m (0.5 times longitudinal bearing width (Specifications of Highway Bridges (p. 306))

UG = εg·L (Type III Ground)

= 0.00500 × 111.000 = 0.555m

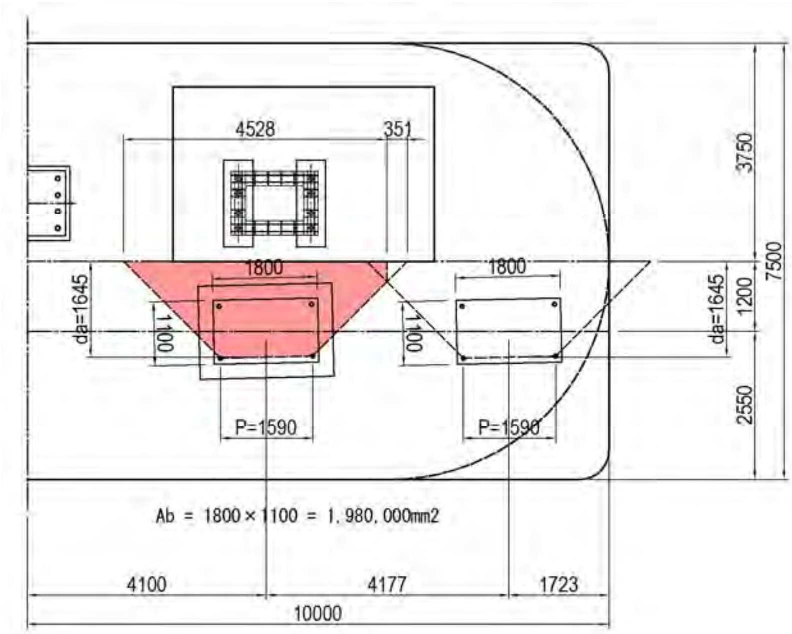
Therefore, the length of beam placement on column is as follows:

SE = 1.255 m < 3.550 m ・・・OK

b) Evaluation of Bridge Seat Strength

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

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



Source: JICA Study Team

Figure 4.2.250 Resistance Area of Concrete

Table 4.2.197 Result of Bridge Seat Evaluation

Item	G2(G3)
Resistance area of concrete Ac (mm^2)	10536181.0
Bearing Stress σn (N/mm^2)	1.061
Coefficient for determining the strength borne by concrete α	0.212
Strength borne by concrete Pc (kN)	3912.714
Strength borne by reinforcement Ps (kN)	0.000
Design horizontal seismic force Ph (kN)	1050.000
Strength of bridge seat Pbs (kN)	3912.714
Judge (Ph≦Pbs)	OK

Source: JICA Study Team

4.2.14.2 Foundation Desing

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

Table 4.2.198 Suammary of Foundation Desing

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

Source: JICA Study Team

4.2.15 Summary of Wind Tunnel Test

4.2.15.1 Introduction

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

The Bridge is a 3-span steel cable-stayed bridge (112.0m+224.0m+112.0m) to cross Bago River in Yangon city. The pylon with a rectangular cross section of 3.0m (along bridge axis) x 2.5m has 58.0m high above the upper deck level. 10 stay cables are installed in single plane at the center of the girder. The elevation of the main girder being taken from the average water level (M.W.L.) to the upper surface level of the main girder is 14.963+2.70=17.663m.

The cross section of the main girder has 22.9m in width (B) and the fairing with 0.80m in horizontal width is installed at both ends. The fairing is partially installed in under-construction stage. Therefore, the overall width of girder is defined as 0.8+22.9+0.8=24.7m. The height of the girder (D) is chosen as 2.70m which is the distance between the lower surface of bottom flange of box girder and the upper deck surface at the center of the main girder.

4.2.15.2 Basic Condition to Evaluate Aerodynamic Stability

The wind tunnel test is to be conducted based on the following conditions:

- Elevation of main girder 17.663 m
- Girder width (B) 22.9 m
- Girder depth (D) 2.70 m (B/D = 8.48)
- Category of surface roughness: II
- Power exponent of vertical profile of wind speed 0.16
- (Longitudinal) intensity of turbulence: 17 %

  At the elevation of the main girder

<u>For after-completion stage</u>

- Basic wind speed (U_{10}) 30 m/s

  10 minute mean wind speed at 10m elevation
- Design wind speed (U_d) 32.7 m/s

  At the girder elevation, the design wind speed U_d is

  $U_d = U_{10} \times E_1 = 30 \times 1.09 = 32.7$

  (E_1: a factor based on surface roughness and elevation)
- Reference wind speed for flutter (U_{rf}) 45.1 m/s

  $U_{rf} = 1.2 \times E_{r1} \times U_d = 1.2 \times 1.15 \times 32.7 = 45.1$

  (E_{r1}: a factor based on the variation of wind speed and natural period of the target bridge)

  The safety for flutter requires that the critical wind speed for flutter measured in wind tunnel test in smooth flow is higher than U_{rf}.
- Reference wind speed for VIV (U_{rv}) 32.7 m/s

$U_{rv} = U_d$ for heaving and torsional

The safety for VIV requires that the onset wind speed at which the maximum amplitude is measured in wind tunnel test in smooth flow is higher than U_{rv}.

- Allowable amplitude for VIV (h_{rvh}, $h_{rv\theta}$) 0.09 m for heaving
  0.39 deg. for torsional

$h_{rvh} = h_\alpha$ (for heaving), $h_{rv\theta} = \theta_a$ (for torsional)

If the onset wind speed is lower than U_{rv}, the safety for VIV requires that the maximum amplitude being measured in wind tunnel test in smooth flow is lower than the allowable amplitude h_α, θ_a. The allowable amplitude h_α, θ_a are evaluated by the following formulae:

$h_a = 0.04/f_h$ (for heaving), $\theta_a = 2.28/(b \cdot f\theta)$

where, ha: allowable heaving amplitude (m),

θ_α: allowable torsional amplitude (deg.),

f_h: heaving natural frequency (Hz),

f_θ: torsional natural frequency (Hz),

b: distance between the center of most outer road traffic or pedestrian lane to the girder center.

$h_a = 0.04/0.446 = 0.09$ m (for heaving), $\theta_a = 2.28/(6.5\times0.895) = 0.39$ deg.

For under-construction stage

- Basic wind speed (U_{10E}) 22.1 m/s
  Considering limited period of construction.
- Design wind speed (U_{dE}) 24.1 m/s
  At the girder elevation ($22.1\times E_1 = 24.1$).
- Reference wind speed for flutter (U_{rfE}) 33.3 m/s
  $U_{rfE} = 1.2\times E_{r1}\times U_{dE} = 1.2\times1.15\times24.1 = 33.3$
- Reference wind speed for VIV (U_{rfE}) 24.1 m/s

$U_{rfE} = U_{dE}$ for heaving and torsional

- Allowable amplitude for VIV (h_{rvhE}, $h_{rv\theta E}$) 0.01 m for heaving
  0.14 deg. for torsional

$ha = 0.04/3.977 = 0.01$ m (for heaving), $\theta_a = 2.28/(6.5\times2.594) = 0.14$ deg.

4.2.15.3 Aerodynamic phenomena to be examined

By taking the geometry of the girder into account, the dynamic stability of the following phenomena should be checked mainly in smooth and in turbulent flow:

- Vortex-induced vibration (VIV)
- Flutter
- Buffeting

Galloping is considered as one of the most destructive aerodynamic phenomena. The occurrence of galloping is not so much expected since the side ratio B/D (B: width, D: height) of the girder cross section is relatively large. Heaving response should be also measured carefully.

The above aerodynamic phenomena are to be measured by the free vibration test in wind tunnel, mainly. Stability for flutter is to be also examined by the forced vibration test.

Target modes for heaving/torsional DOF will be determined by mode shape and equivalent mass for two phases in under-construction stage and for after-completion stage.

Wind tunnel test was conducted for under-construction stage in which and for after-completion stage.

4.2.15.4 Wind Tunnel

The wind tunnel facility used for the test is the Eiffel type wind tunnel in Department of Civil and Earth Resources Engineering, Kyoto University, Japan (see Figure 4.2.251 and Figure 4.2.252). Width and

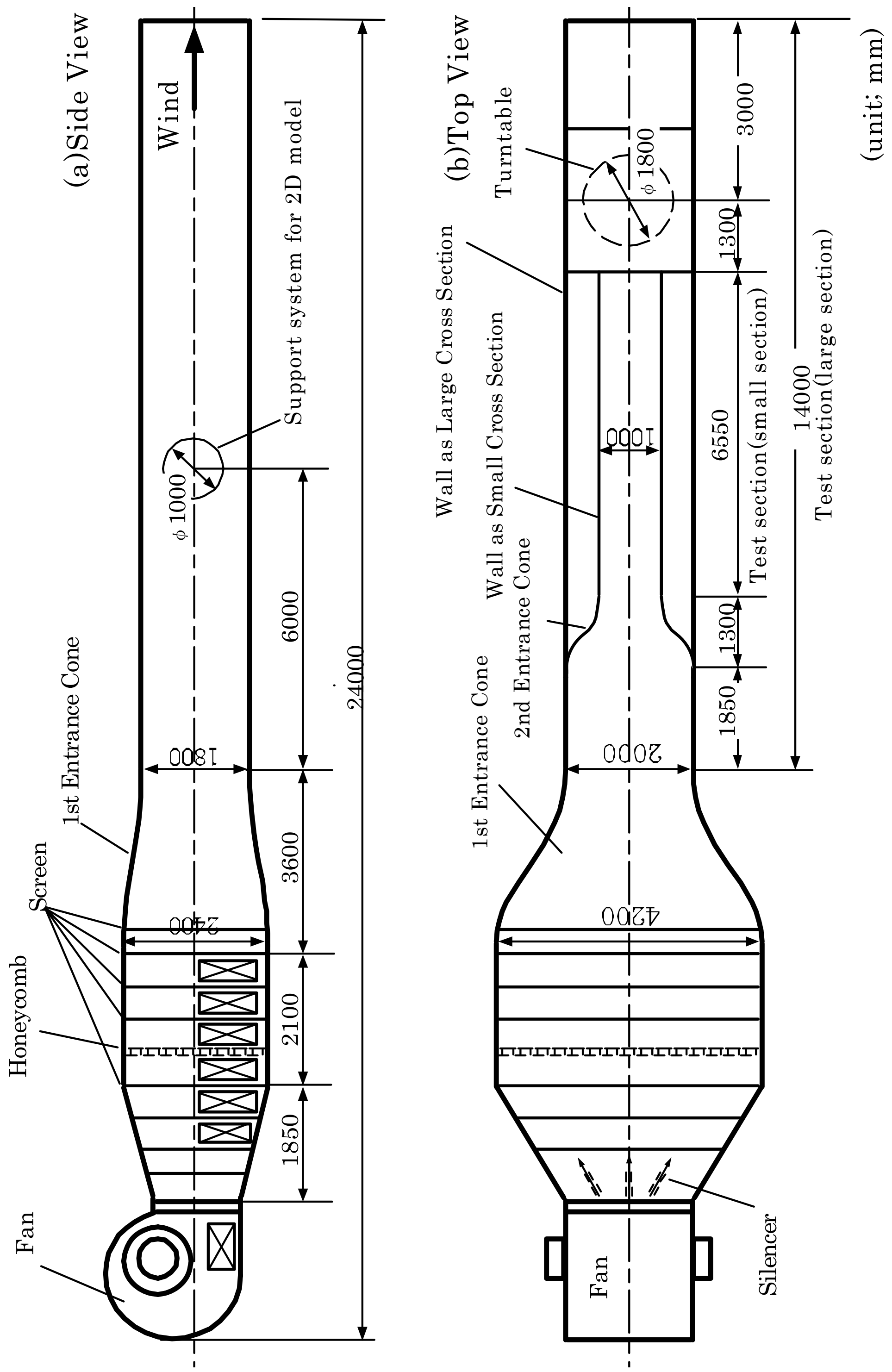
height of working section is 1.0m and 1.8m for section model test. Wind velocity in the working

section can be adjusted up to about 25m/s. Turbulent intensity in the empty working section is

less than 0.5(%).

Source: Kyoto University

Figure 4.2.251 Wind Tunnel in Department of Civil and Earth Resources Engineering,Kyoto University



Source: Kyoto University

Figure 4.2.252 Wind Tunnel

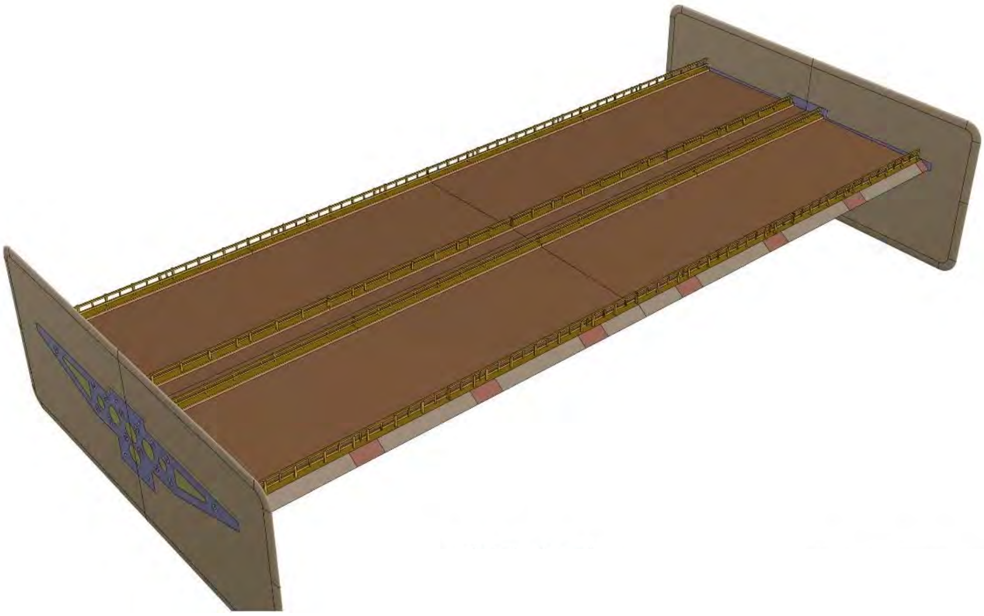
4.2.15.5 Models for Wind Tunnel Test

(1) Section model of main girder

The cross section of the model realizes the representative outer configuration of the main girder. Between the under construction stage and the after completion, main difference of the model is:

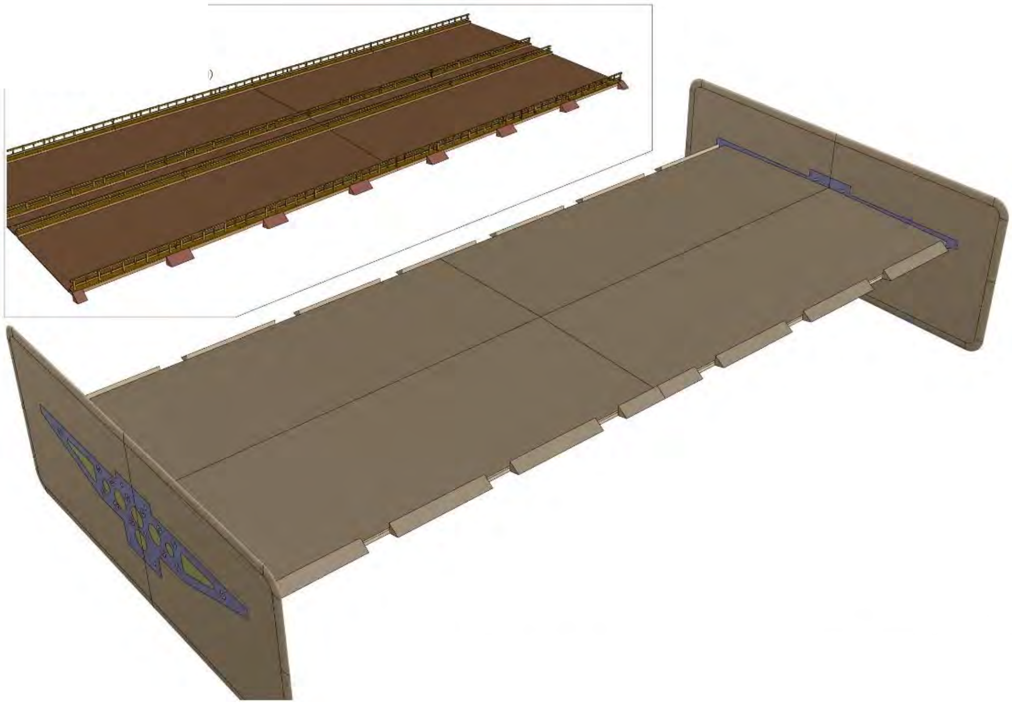
- Fairing: Installed in discrete manner for the under construction stage and installed continuously for after completion stage.
- Handrails and pavement layer: Installed only in after completion stage.

Scale ratio of the model was determined as 1/70 by taking the wind tunnel facility condition into account. The detail of the section model is shown in Figure 4.2.253 to Figure 4.2.256.



Source: JICA Study Team

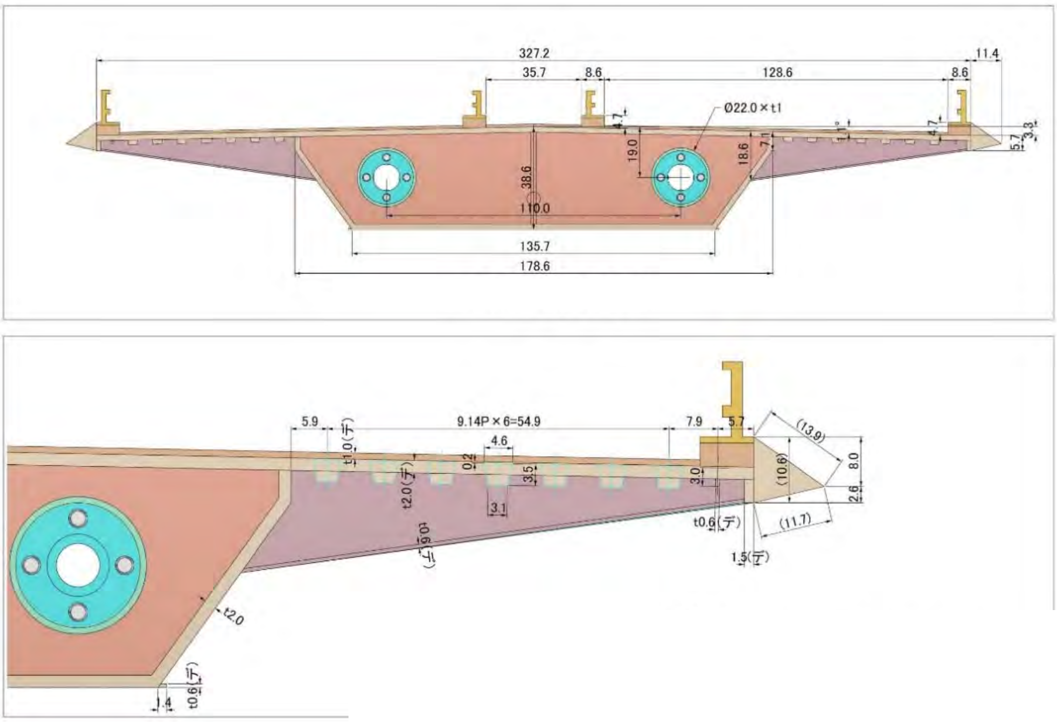
Figure 4.2.253 3-D image of section model (for after-completion stage)



Source: JICA Study Team

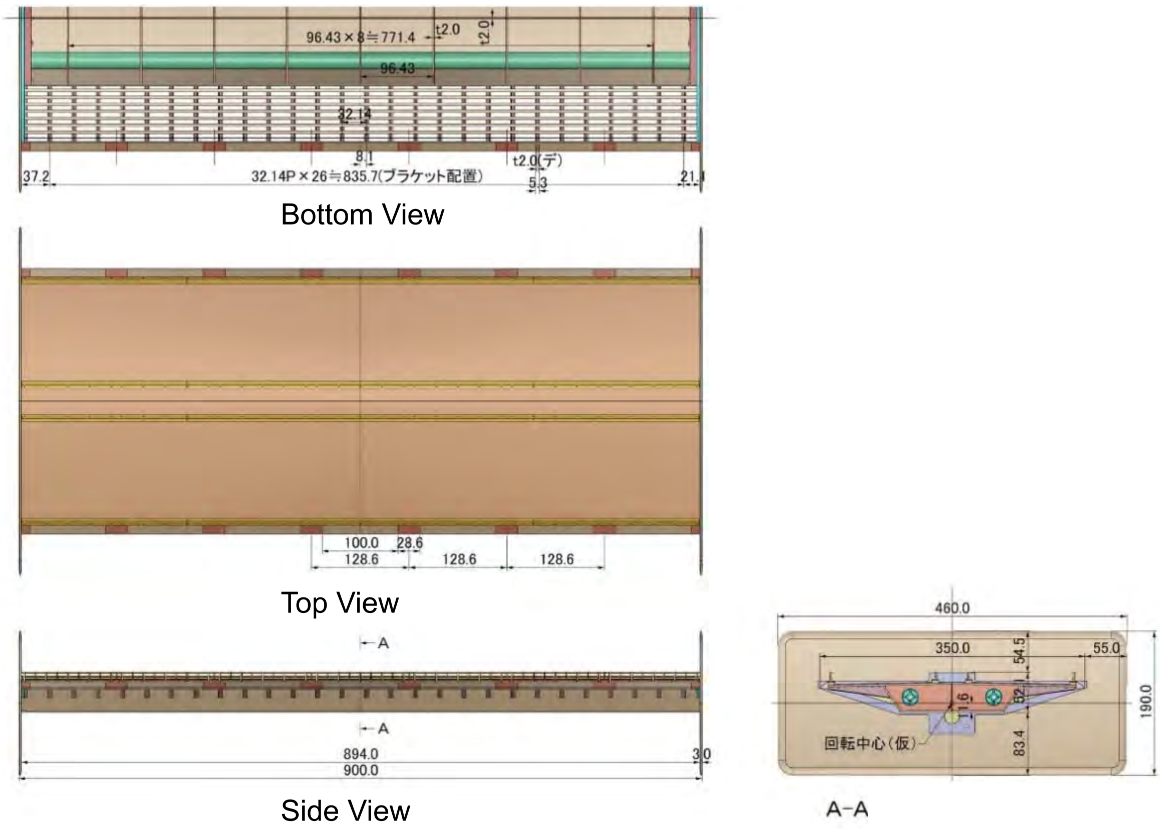
Figure 4.2.254 3-D image of section model (for under-completion stage)

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



Source: JICA Study Team

Figure 4.2.255 Cross section of the section model (unit in mm)



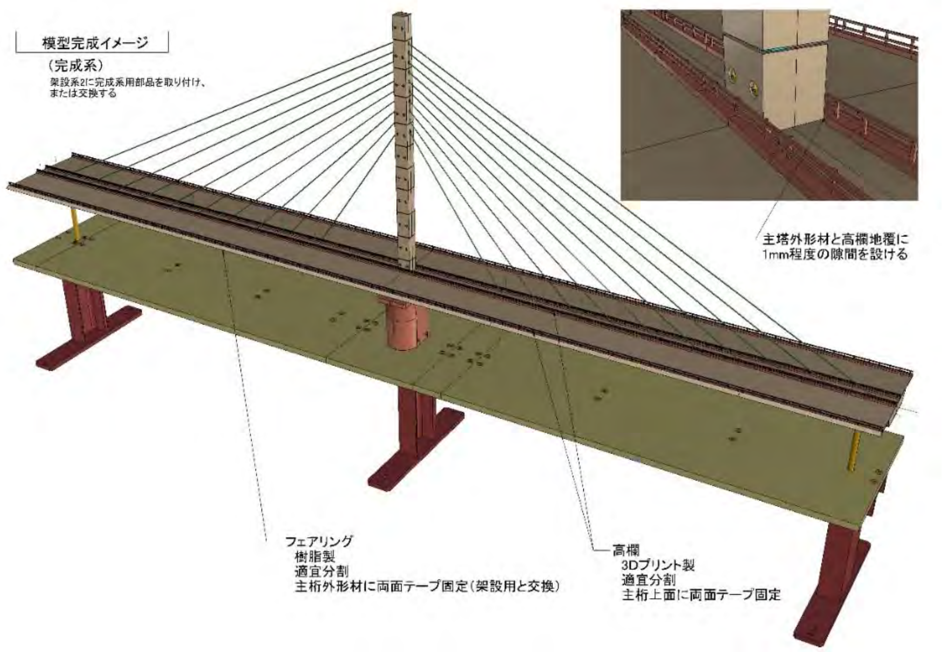
Source: JICA Study Team

Figure 4.2.256 General view of the section model (unit in mm)

(2) 3-D elastic model of tower

The 3-D elastic model is to be manufactured in order to measure the aerodynamic response of the tower. This model consists of the fully elastic tower part and the rigid girder part. The bent during the early period of the under-construction stage is also realized. Girder length is changeable and the cable can be installed when necessary. The cable is realized by steel wires which the diameter is determined so as to simulate the drag force. Tensile force in each cable is given by using a weight before fixing.

Scale ratio of the model was determined as 1/120. The detail of the section model is shown in Figure 4.2.257 to Figure 4.2.260.



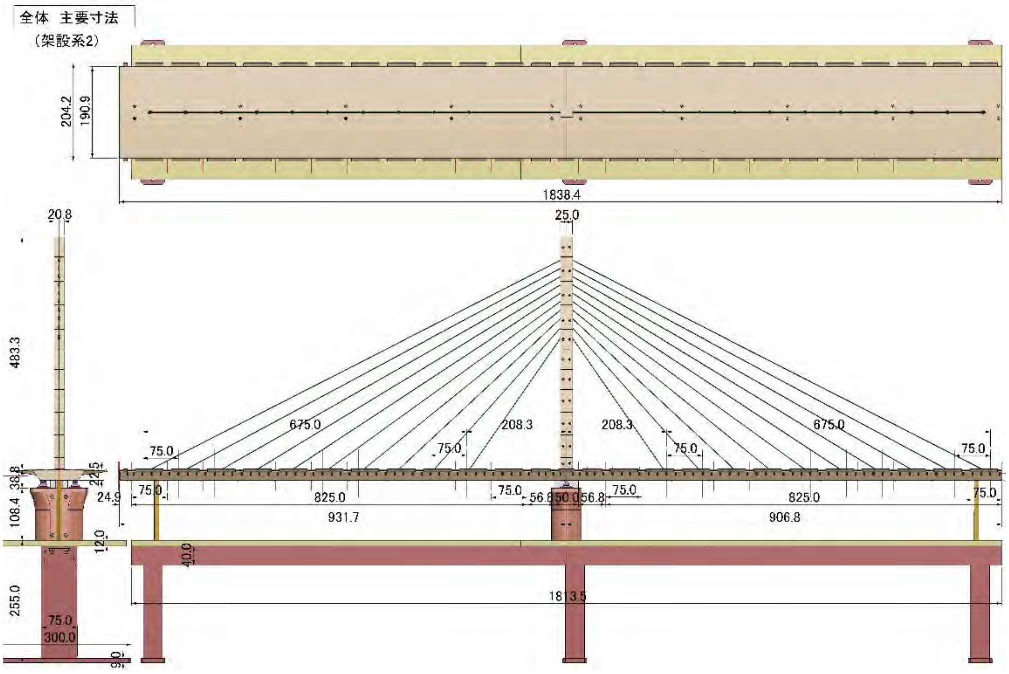
Source: JICA Study Team

Figure 4.2.257 3-D image of the elastic model (for after-completion stage)

(The min girder part is a rigid model.)

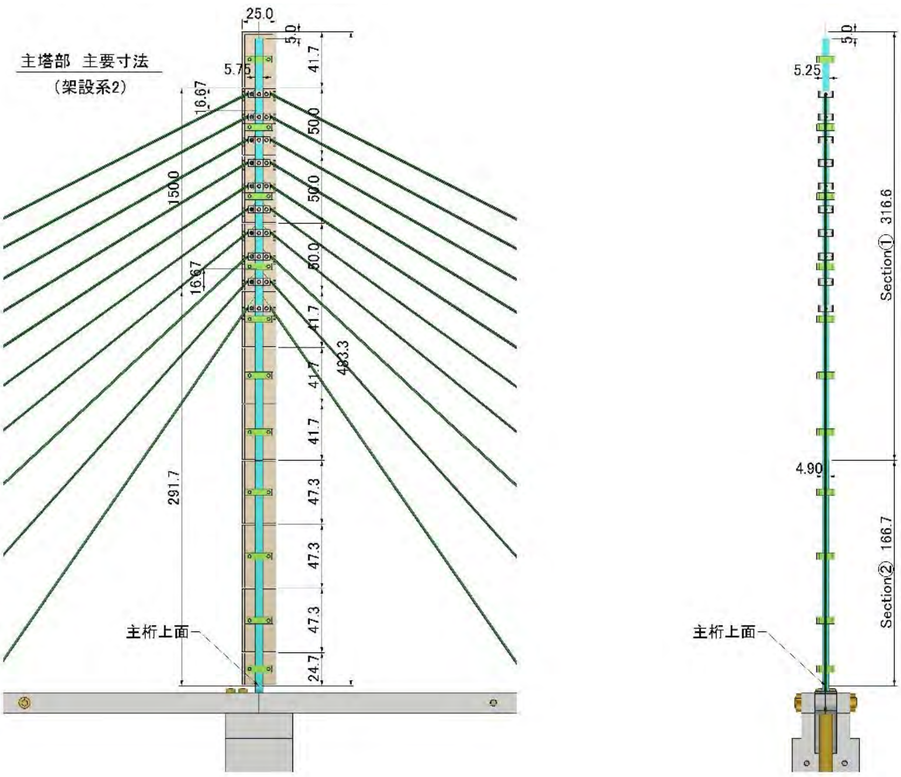
Source: JICA Study Team

Figure 4.2.258 3-D image of elastic bar for the tower and rigid bar for the main girder (The supports at both ends of the model are to keep the girder as rigid.)



Source: JICA Study Team

Figure 4.2.259 The elastic model (for after-completion stage, unit in mm)



Source: JICA Study Team

Figure 4.2.260 The tower and elastic bar with additional mass arrangement (unit in mm)

4.2.15.6 Free vibration test of main girder (scale ratio 1/70)

The test was conducted in or der to measure the aerodynamic response of the main girder during under-construction stage and after-completion stage.

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

- Before the lowest cable being installed and just after the first segment of the main girder was installed. (Heaving 1 degree of freedom (DOF); This condition was abbreviated as UC1, hereafter.)
- Just before the last segment of the main girder in the main span is installed. (Heaving and torsional 2 DOF, UC2)

For after-completion stage,

- the modes combination, heaving and torsional: 2 DOF (AC) was set to the model

In the following tables, 'o' means the corresponding response was not observed, and '×' means the corresponding response occurred. The corresponding prototype wind speed interval Up [m/s] is shown for the vortex-induced vibration (VIV), while the prototype onset wind speed is shown for flutter and galloping.

(1) Under-construction (UC1: Before the lowest cable being installed)

In the case of UC1, the free vibration test was conducted under 3 incidence angles of wind (0, +3, and -3 [deg]) in smooth and in turbulent flow, respectively. Displacement of the model was allowed only 1 DOF along heaving (across-wind) direction, and an initial heaving vibration (disturbance) was applied to the model at several wind speed conditions in the test. The response was recorded after the response amplitude became stable.

For all of the cases in UC1, neither vortex-induced vibration (VIV) nor flutter was observed. The results are summarized in Table 4.2.199.

Table 4.2.199 Aerodynamic response of the main girder in UC1 (Heaving 1 DOF)

Flow condition	Vertical incidence angle of wind [deg]	Vortex-induced vibration	Flutter
Smooth	0	o	o
	+3	o	o
	-3	o	o
Turbulent	0	o	o
	+3	o	o
	-3	o	o

"o" : The corresponding response was not observed.
Source: JICA Study Team

(2) Under-construction (UC2: Just before the last segment of the main girder in the main span is installed)

In the case of UC2, the free vibration test was conducted under 3 incidence angles of wind (0, +3, and -3 [deg]) in smooth and in turbulent flow, respectively. Displacements being allowed in the model was 2 DOF along heaving and torsional direction. An initial heaving or torsional disturbance was applied to the model separately at several wind speed conditions in the test. The response was recorded after the response amplitude became stable.

For all of the tests, neither VIV nor flutter was observed. The results are summarized in Table 4.2.200.

Table 4.2.200 Aerodynamic response of the main girder in UC2 (Heaving/torsional 2 DOF)

Flow condition	Vertical incidence angle of wind [deg]	Vortex-induced vibration	Flutter
Smooth	0	o	o
	+3	o	o
	-3	o	o
Turbulent	0	o	o
	+3	o	o
	-3	o	o

"o" : The corresponding response was not observed.
Source: JICA Study Team

(3) After-completion (AC, Heaving and torsional 2 DOF)

In the case of AC, the free vibration test was conducted under 3 incidence angles of wind (0, +3, and -3 [deg]) in smooth and in turbulent flow, respectively. Displacement being allowed in the model was 2 DOF along heaving and torsional direction. An initial heaving or torsional disturbance was applied to the model separately at several wind speed conditions in the test. The response was recorded after the response amplitude became stable. The results are summarized in Table 4.2.201.

In smooth flow condition, the torsional VIV was observed for all of the three incidence angles (0, +3, and -3 [deg]), while no heaving VIV was observed. The prototype wind speed Up of torsional VIV was at around 21.6 [m/s] for the incidence angle of 0 [deg], 15.4 - 17.9 [m/s] and 22.8 - 25.2 [m/s] for +3 [deg], and 15.4 - 17.9 [m/s] and 20.3 - 24.0 [m/s] for -3 [deg].

In turbulent flow, no VIV was observed for both heaving and torsional direction.

For all of the tests conducted in smooth flow and in turbulent flow, no flutter was observed.

Table 4.2.201 Aerodynamic response of the main girder in AC (Heaving/torsional 2 DOF)

Flow condition	Vertical incidence angle of wind [deg]	Vortex-induced vibration	Flutter
Smooth	0	× Torsional (at around 21.6 [m/s])	o
	+3	× Torsional (15.4 - 17.9 [m/s]) (22.8 - 25.2 [m/s])	o
	-3	× Torsional (15.4 - 17.9 [m/s]) (20.3 - 24.0 [m/s])	o
Turbulent	0	o	o
	+3	o	o
	-3	o	o

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.
Source: JICA Study Team

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

Aerodynamic response of the tower was tested in wind tunnel using fully elastic tower model of 1/120 scale ratio.

The target stage for wind tunnel test was chosen to be the same as those for the main girder test:

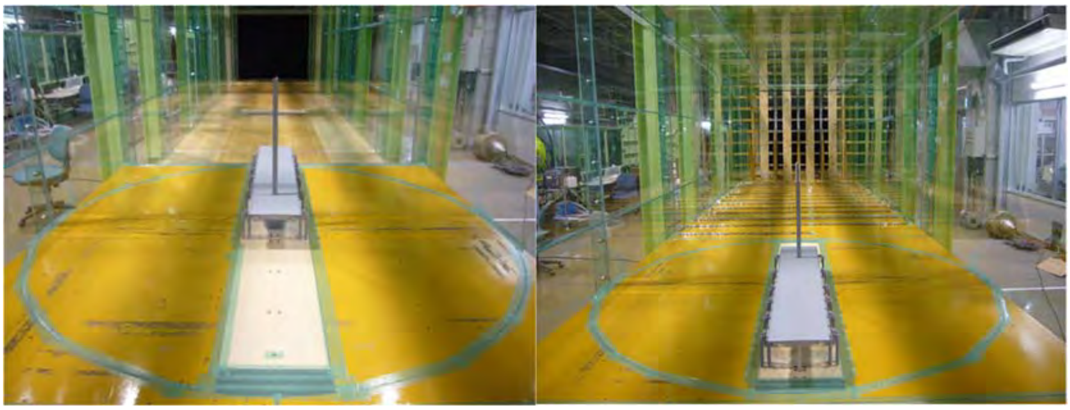
For the under-construction stage, the following 2 stages were focused:

- Before the lowest cable being installed and just after the first segment of the main girder was installed. (Heaving 1 DOF; This condition was abbreviated as UC1, hereafter.)
- Just before the last segment of the main girder in the main span is installed. (Heaving and torsional 2 DOF, UC2)

For after-completion stage, the following modes combination was set to the model.

- Heaving and torsional (2 DOF, AC)

In the following tables, the wind direction along cable plane (i.e. along bridge axis) is denoted as x-direction, while the wind direction normal to cable plane is denoted as y-direction. In addition, yawing angle 0º refers to the angle when the wind is along the bridge axis, and 90º refers to the angle when the wind blows along the lateral direction of the bridge. For the cases of 'Under construction 1 (UC1)', the yawing angle 0º and 180º is defined as shown in Fig. 4.2.253 (a) and (b), respectively.



(a) Yawing angle 0º (b) Yawing angle 180º

Source: JICA Study Team

Figure 4.2.261 Definition of yawing angle for cases of under construction 1 (UC1) (Wind comes from the back to the model side.)

(1) Under construction 1 (UC1, Before the lowest cable being installed)

(Original tower configuration)

During the under-construction stage of UC1, the tower stands alone without any cables. In this situation, the aerodynamic sensitivity of the tower will be higher than those in the other 2 stages.

In the condition of smooth flow, y-direction (normal to cable plane) VIV was observed for yawing angle 0º and 5º (wind comes along bridge axis), while x-direction (along cable plane) VIV was observed for yawing angles 80º, 85º and 90º (wind comes normal to bridge axis). The y-direction galloping occurred for yawing angle 5º, while the x-direction galloping was observed for yawing angles 80º and 90º. The results are summarized in Table 4.2.202.

Table 4.2.202 Under construction 1 (UC1, Original tower configuration (without aerodynamic device))

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping
Smooth	0	× in y-direction (16.5m/s~18.6m/s)	o
	5	× in y-direction (16.5m/s~18.6m/s)	× in y-direction (60.0m/s~)
	22.5	o	o
	45	o	o
	67.5	o	o
	80	× in x-direction (14.2m/s~22.3m/s)	× in x-direction (58.7m/s~)
	85	× in x-direction (14.2m/s~20.3m/s)	o
	90	× in x-direction (16.2m/s~22.3m/s)	× in x-direction (43.6m/s~)

"o" : The corresponding response was not observed.

"×" : The corresponding response occurred.

Source: JICA Study Team

(2) Under construction 1 (UC1, Before the lowest cable being installed)

(With L-shaped aerodynamic device, length: 91.7mm)

For UC1, countermeasure to stabilize the aerodynamic vibration, VIV and Galloping should be discussed, since these phenomena were observed in smooth flow condition as described in the previous section. The L-shaped aerodynamic device as shown in Fig.4.2.254 was proposed and its stabilizing effect was tested in wind tunnel by attaching the device near the edge of the tower cross section on the front and rear surface near the top of the tower model.

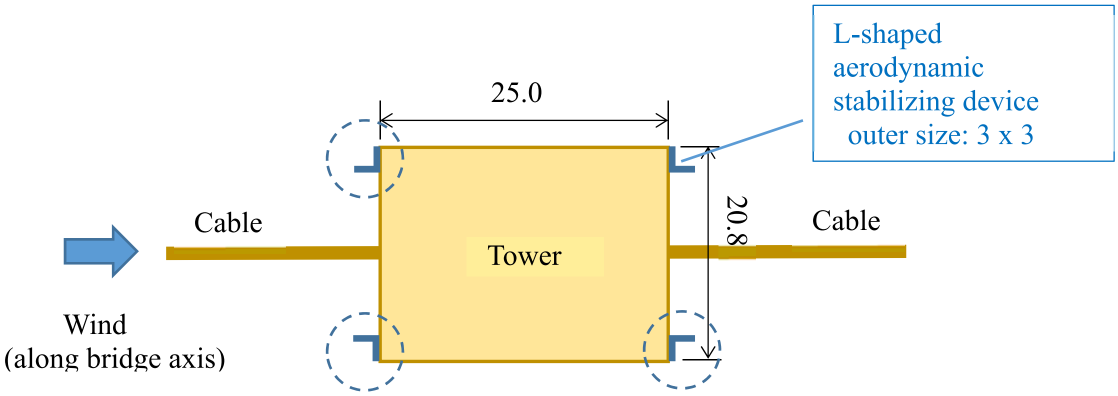
In the condition of smooth flow, y-direction VIV was observed for yawing angle 0º, and x-direction VIV was observed for yawing angle 90º. In the condition of turbulence, no VIV and galloping were observed. The results are summarized in Table 4.2.203.

Table 4.2.203 Under construction 1 (UC1, With L-shaped aerodynamic device)

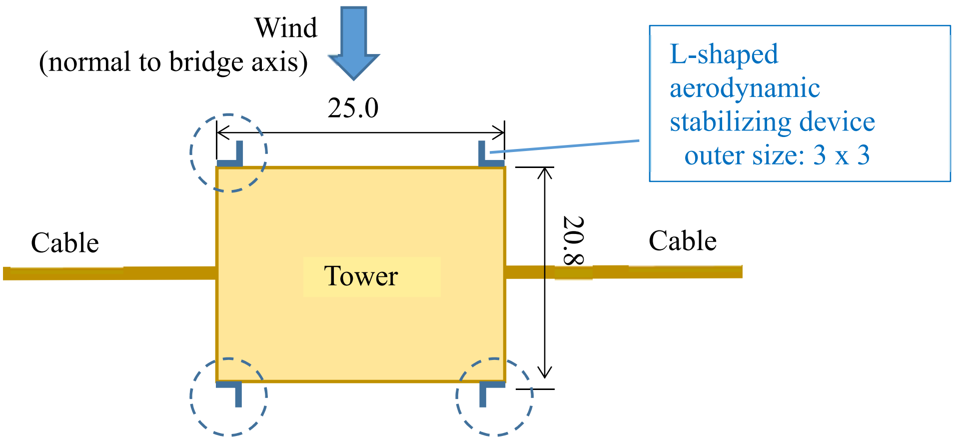
(Length of aerodynamic device: 91.7mm (= 11.0m for real bridge) from the top of the tower))

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping
Smooth	0	× in y-direction (12.6m/s~14.7m/s)	o
	90	× in x-direction (18.5m/s~20.6m/s)	o
Turbulent	80	o	o
	85	o	o
	90	o	o
	180	o	o

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.
Source: JICA Study Team



(a) For wind along bridge axis



(b) For wind normal to bridge axis

Source: JICA Study Team

Figure 4.2.262 L-shaped aerodynamic device (Model scale ratio: 1/120, unit: in mm)

(3) Under construction 2 (UC2, Before the last girder segment being installed)

(With L-shaped aerodynamic device, length: 91.7mm)

Aerodynamic vibration response of the tower was tested for the under-construction stage UC2, in which the aerodynamic device was attached with the length of 91.7mm (= 11.0m for real bridge) expecting its stabilizing effect to the wind along bridge axis (see Fig. 4.2.254).

While the y-direction VIV was observed in smooth flow for yawing angle 0º, no vibration was observed in turbulent flow condition for yawing angle 0º and5 º. The results are summarized in Table 4.2.204.

Table 4.2.204 Under construction 2 (UC2, With L-shaped aerodynamic device)

(Length of aerodynamic device: 91.7mm (= 11.0m for real bridge) from the top of the tower))

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping
Smooth	0	× in y-direction (11.1m/s~29.5m/s) (34.2m/s~34.2m/s)	o
	5	o	o
Turbulent	0	o	o
	5	o	o

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.
Source: JICA Study Team

(4) After-completion (AC) (Original tower configuration)

In the condition of smooth flow, y-direction VIV was observed for yawing angles 0º and 5º. Besides, y-direction galloping occurred for yawing angel 5º. In the condition of turbulence, both of y-direction VIV and y-direction galloping were observed for yawing angle 0º. The results are summarized in Table 4.2.205.

Table 4.2.205 After conpletion (AC, without L-shaped aerodynamic device)

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping
Smooth	0	× in y-direction (10.7m/s~30.4m/s)	o
	5	× in y-direction (10.7m/s~19.7m/s)	× in y-direction (28.6m/s~)
	10	o	o
	22.5	o	o
	45	o	o
	67.5	o	o
	90	o	o
Turbulent	0	× in y-direction (17.9m/s~19.7m/s)	× in y-direction (23.3m/s~)
	5	o	o
	10	o	o
	22.5	o	o
	45	o	o
	67.5	o	o
	90	o	o

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.
Source: JICA Study Team

(5) After-completion (AC, with L-shaped aerodynamic device)

(With L-shaped aerodynamic device, length: 91.7mm)

The response was tested with the aerodynamic device attached (with the length of 91.7mm). The y-direction VIV was observed in smooth flow with yawing angle 0º. No galloping occurred in both of the smooth flow and turbulence. The results are summarized in Table 4.2.206.

Table 4.2.206 After-completion (AC, With L-shaped aerodynamic device)

(Length of aerodynamic device: 91.7mm (= 11.0m for real bridge) from the top of the tower))

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping
Smooth	0	× in y-direction (22.1m/s~25.8m/s)	o
	5	o	o
Turbulent	0	o	o
	5	o	o

"o" : The corresponding response was not observed.

"×" : The corresponding response occurred.

Source: JICA Study Team

(6) Determination of the length of aerodynamic device

The length of the aerodynamic device was determined based on the tower response with yawing angles 0º and 5º, in both of smooth and turbulent flow conditions. As shown in Table 4.2.205, galloping occurred in the case of smooth flow with yawing angle 5º as well as the case of turbulence with yawing angle 0º. The aerodynamic device with the length of 91.7mm could suppress the galloping for both cases.

On the other hand, the length of the aerodynamic device should be as small as possible and have enough stabilizing performance. Therefore, aerodynamic devices with different length of 41.7mm (= 5.0m for real bridge), 141.7mm (17.0m) , 191.7mm (23.0m) and 233.4mm (= 28.0m)) were attached to the top of the tower.

From these results, it indicates that the length of 3 pieces is an optimal choice by taking the fact that VIV was measured in 141.7mm of the installed length in smooth flow and 0[deg] yawing angle. And this response was stabilized in turbulent flow condition. With the aerodynamic device applied, the galloping was enough suppressed as shown in Table 4.2.207.

Table 4.2.207 After-completion (AC, With L-shaped aerodynamic device)

Flow condition	Yawing angle [deg]	Length of aerodynamic device [mm]	Vortex-induced vibration	Galloping
Smooth	0	141.7	× in y-direction (15.0m/s~40.0m/s)	o
	5	141.7	o	o
Turbulent	0	41.7	o	o
		141.7	o	o
		191.7	o	o
		233.4	o	o

	5	41.7	o	o
		141.7	o	o
		191.7	o	o
		233.4	o	o

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.
Source: JICA Study Team

4.2.15.8 Conclusions on aerodynamic response of main girder and tower

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

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

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

For after-completion stage,

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

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

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

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

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

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

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

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

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

For UC1 in smooth flow, the response characteristics were examined by installing the device along 11.0 [m] from the top of the tower. VIV was still observed at 12 to 15 [m/s] for 0 [deg.] and 18 to 21 [m/s] for 90 [deg.], although no galloping was measured for both measurement cases. The response in turbulent flow condition was examined for 80, 85, 90 and 180 [deg.]. The tower was stable for all of these wind direction conditions. (There is no big difference in the response for 0 [deg.] and 180 [deg.], since the flow around the top of tower can be almost identical and no significant influence by the upstream elongation length of the main girder. (see Fig.6.3.1))

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

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

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

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

4.2.16 Summary of Quantities

4.2.16.1 Quantities for Superstructure

The quantities for the superstructure are shown in the table below.

Table 4.2.208 Quantities for Superstructure (Tower)

Items		Description	Spec	Unit	Q'ty
					Total
Tower Fabrication	Material	Steel plate		ton	609.1
		Shapes		ton	5.1
		Torque share high tension bolt		ton	6.2
	Fabrication	"A1":No. of major piece		nos	160
				ton	372.2
		"A2":No. of minor piece		nos	2702
				ton	226.2
		"B1":Length of welding form major piece (converted to 6mm fillet welding)		m	0
		"B2":Length of T-joint welding form major piece		m	906.6
		"C":Total No. of pieces *Pieces connected meanwhile being erection girder		nos	40
		"X":Total fabricated steel weight		ton	598.4
	Paint in factory	Blasting area (pre-processing before paint)		m2	5053.2
		Outside GEN		m2	1380.5
		Inside GEN		m2	3158.2
Tower erection	Pre-assembling	Welding		m	906.6
	Welding in site	Length of welding		m	17468.2
	Dead-bolting in site	Tower interior_Torque share high tension bolt		nos	10360
	Paint in site	Outside Welding		m2	44
		Inside SPL		m2	139.2
		Inside Bolt Head		m2	52.4
		Inside Welding		m2	52.9
		Connection		m2	278.3

Source: JICA Study Team

Table 4.2.209 Quantities for Superstructure (Girder)

Items		Description	Spec	Unit	Qty Total
Girder Fabrication	Material	Steel plate		ton	4439.8
		Shapes		ton	540.4
		Torque share high tension bolt		ton	104.2
	Fabrication	"A1"No. of major piece		nos	1238
				ton	2285.5
		"A2"No. of minor piece		nos	53950
				ton	2440.3
		"B1"Length of welding for major piece(converted to 6mm fillet welding)		m	9611.5
		"B2"Length of T-joint welding for major piece		m	7171.6
		"W0"Percentage of weight of material equivalent to 570 within total fabricated steel weight		%	0.1
		"C":Total No. of pieces *Pieces connected meanwhile being erection girder		nos	515
		"X":Total fabricated steel weight		ton	4980.3
		Rail For Inspection Car		ton	3.8
		PL For Connect To Rocking Bearing		ton	2.7
	Paint in factory	Blasting area (pre-processing before paint)		m2	86047.8
		Outside GEN		m2	22135.4
		Overglaze Outside GEN		m2	641.1
		Inside GEN		m2	42466.7
		Surface Of Deck PL GEN		m2	9614.8
Girder erection_Main Span	Pre-assembling	"Welding for main girder"Length of welding for major piece(converted to 6mm fillet welding)		m	3914
		"Welding for main girder"Length of T-joint welding for major piece		m	2982.1
		Welding for metal deck		m	2024.1
		U-rib welding		m	0
		Welding for Fairing PL		m	0
	Erection	Welding for metal deck		m	458
		Welding for Fairing PL		m	302.1
	Dead-bolting in site	Torque share high tension bolt		nos	41312
	Paint in site	Outside SPL		m2	144.6
		Outside Bolt Head		m2	36.6
		Outside Welding		m2	352.4
		Overglaze Outside Welding		m2	0
		Inside SPL		m2	426.4
		Inside Bolt Head		m2	174.2
		Inside Welding		m2	375.8
		Surface Of Deck PL Welding		m2	404.8
		Connection		m2	1140.1
Girder erection_Back Spans	Pre-assembling	"Welding for main girder"Length of welding for major piece(converted to 6mm fillet welding)		m	5697.5
		"Welding for main girder"Length of T-joint welding for major piece		m	4189.5
		Welding for metal deck		m	2885.6
		U-rib welding		m	0
		Welding for Fairing PL		m	0
	Erection	Welding for metal deck		m	2885.6
		Welding for Fairing PL		m	483.4
	Dead-bolting in site	Torque share high tension bolt		nos	152168
	Paint in site	Outside SPL		m2	817.5
		Outside Bolt Head		m2	189.1
		Outside Welding		m2	270.8
		Overglaze Outside Welding		m2	4.8
		Inside SPL		m2	1529.5
		Inside Bolt Head		m2	577.1
		Inside Welding		m2	301.6
		Surface Of Deck PL Welding		m2	577.1
		Connection		m2	4809.5

Source: JICA Study Team

Table 4.2.210 Quantities for Superstructure (Cable)

Items		Description	Spec	Unit	Q'ty Total
Cable stay material	PC strands(7S15.6)	SWPR7BL		ton	224.4
		Loss(2.0%)			0.02
	HDPE duct	ϕ180		m	952.8
		ϕ250		m	1785.4
		Loss(3.0%)			0.03
	Adjustmentable Anchorage	37H (Girder side)		nos	20
		70H (Girder side)		nos	20
	Fixed Anchorage	37H (Tower side)		nos	20
		70H (Tower side)		nos	20
	Sliding Tube	37H		nos	20
		70H		nos	20
	HDPE Joint Tube	37H		nos	20
		70H		nos	20
	Support Ring	37H		nos	20
		70H		nos	20
	Positioning Tube	37H		nos	20
		70H		nos	20
	Protection Tube	37H		nos	20
		70H		nos	20
	Buffer Device	37H (Girder side)		nos	20
		70H (Girder side)		nos	20
		37H (Tower side)		nos	20
		70H (Tower side)		nos	20
	Vibration Control Device	37H		set	20
		70H		set	20

Source: JICA Study Team

Table 4.2.211 Quantities for Superstructure (Accessories)

Items		Description	Spec	Unit	Qty
					Total
Bearing	Bearing For Horizontal Force	6700kN		nos	2
	Rocking Bearing	3700kN		nos	4
	Pivot Bearing	57700kN		nos	2
	Pin Roller Bearing	20800kN		nos	4
	Anchor Bolt			ton	20.7
	Anchor Frame			ton	24.2
	Pedestal Frame			ton	65.4
Accessory & Miscellaneous work	Bridge Surface Work	Asphalt Pavement		m2	8028
		Concrete for Median		m3	89.2
		Stud for Median	SD345	kg	334
		Welded Wire Mesh for Median		m2	1115
		Water-Resistant Coating for Road Way		m2	8028
		Water-Resistant Coating for Median		m2	1115
	Wheel Guard & Median Strip	Concrete		m3	354.5
		Form		m2	877.1
		Reinforcing Bar	SD345	kg	21815
		Stud	SD345	kg	5237
		Drainage Pipe	STKR400	kg	4068
	Composite Barrier & Barrier For Carriage Way	Composite Barrier		m	895
		Reinforcing Bar for Composite Barrier	SD345	kg	5567
		Mortar for Composite Barrier		m3	4.76
		Barrier For Carriage Way		m	895
		Reinforcing Bar for Barrier For Carriage Way	SD345	kg	5567
		Mortar for Barrier For Carriage Way		m3	4.76
	Expansion Joint	Module Type		m	45.8
		Reinforcing Bar	SD345	kg	800
		Stud	SD345	kg	142
	Ligting	Load Lighting		nos	22
		Light-up System for Tower		nos	4
		Light-up System for Pier		nos	4
	Navigation Sign & Light	Safe Water		nos	2
		Port Hand		nos	6
		Starboard Hand		nos	6
	Aircraft Warning Light			nos	2
	Lightning Conductor			nos	2
	Manhole	Girder_Polychloroprene	480x9x680	nos	6
		Tower_Polychloroprene	510x10x710	nos	4
	Cable Rack(Reference)	Cable Rack Length	W=0.6m	m	448
		Cable Rack	W=0.6m L=3.0m	nos	149
		Cable Rack	L=0.5m	nos	2
		Joint		nos	148
		End Cap		nos	2
		Steady Piece		nos	203
	Drainage	Drainage Box		nos	140
		Bridge Surface Drainage		nos	102
	Ladder In Main Tower			ton	3
	Inspection Road			ton	38.1
	Water Pipe	Support		ton	16.6
	Fairing			ton	96.2
	Aerodynamic Device			ton	12.8

Source: JICA Study Team

4.2.16.2 Quantities for Substructure

The quantities for the substructure are shown in the table below.

Table 4.2.212 Quantities for Substructure (RC Pier)

Type of Works	Item		Specification	Classification	Unit	Quantities: P10 Pier (Origin Side)	Quantities: P10Pier	Quantities: P11Pier	Quantities: P12Pier	Quantities: P13Pier	Sum	Note
Concrete	Pier	Primary Construction	σck=30N/mm2		m3	—	2,097.4	1,801.1	1,801.1	2,099.4	7,799.0	
Formwork	Pier	Primary Construction	Normal Form	H≦30m	m2	—	411.6	268.3	268.3	416.6	1,364.8	
			Circular Formwork		〃	—	526.9	514.8	514.8	526.9	2,083.4	
	Drainage Steel Pipe			φ= 50mm	m	—	3.0	—	—	3.0	6.0	
Bearing Support	Bearing Mortar		Mortar		m3	—	—	12.535	12.535	—	25.070	Superstructure Construction
			Non-shrinkable Mortar		m3	1.394	0.886	2.815	2.815	0.886	8.796	Superstructure Construction
	Form for void for Anchor Bolt		Cylindrical Form	φ= 150mm	m	—	—	—	—	8.6	8.6	
				φ= 200mm	〃	10.6	—	—	—	—	10.6	
				φ= 250mm	〃	—	7.9	—	—	7.9	15.8	
	Box-out Formwork		Box-out From		m2	11.4	2.3	21.8	21.8	7.3	64.6	
Steel Reinforcement	Weight		SD345	D 13	kg	—	—	—	—	—	—	
				D16 ～ D25	〃	—	88,526	77,588	77,588	88,457	332,159	
				D29 ～ D32	〃	—	66,425	6,604	6,604	66,444	146,077	
				D 35	〃	—	—	—	—	—	—	
				D 38	〃	—	—	—	—	—	—	
				D 51	〃	—	—	88,337	88,337	—	176,674	
				Total	〃	—	154,951	172,529	172,529	154,901	654,910	
	Large Diameter Re-bar Ratio				—	—	—	—	—	—	—	
	Mechanical Splice		SD345	D 35	Point	—	—	—	—	—	—	
				D 38	〃	—	—	—	—	—	—	
				D 51	〃	—	—	412	412	—	824	
				Total	〃	—	—	412	412	—	824	
Fall Preventive Handrail	Steel Weight	PIPE	STK400	φ34.0×2.3	kg	—	93	72	72	93	330	
				φ21.7×1.9	〃	—	75	78	78	75	306	
		L	SS400	65×65×6	〃	—	271	226	226	271	994	
		PL	SM400A	t=9mm	〃	—	82	56	56	82	276	
				t=6mm	〃	—	39	26	26	39	130	
		Total			〃	—	560	458	458	560	2,036	
	Number of Bolts	U.Bolt	SS400	Nominal_25C	Nos.	—	52	36	36	52	176	
				Nominal_15C	〃	—	84	72	72	84	312	
		ANC	SS400	M16×125	〃	—	208	144	144	208	704	
	Mass of Plating	HDZ55			kg	—	392	308	308	392	1,400	
		HDZ35			〃	—	271	216	216	271	974	

Source: JICA Study Team

Table 4.2.213 Quantities for Substructure (SPSP Foundation - 1)

Type of Works	Item	Classification				Unit	Quantities P10Pier	P11Pier	P12Pier	P13Pier	Sum	Note
Steel Pipe Sheet Pile Foundation	Steel Pipe Sheet Pile	Length of Pile (Pile Diameter φ1200mm)				m/Nos.	70.0	74.0	65.5	63.0	—	
		Pile Number				Nos.	36	40	40	36	152	Outside Steel Pipe Well
						〃	8	8	8	8	32	Diaphragm Steel Sheet Pipe Wall
		Total				〃	44	48	48	44	184	
		Total Pile Length				m	3,080.0	3,552.0	3,144.0	2,772.0	12,548.0	
		Embedded Depth				m	60.6	63.7	52.6	50.1	227.0	Soil Coefficient=1.00
						〃	—	—	—	—	—	Soil Coefficient=1.07
		Per pipe (TypeA・C・E)	Weight of Steel Pipe Section	φ1200	t=14mm	t	19.224	18.406	14.929	16.361	68.920	SKY400
					t=14mm	〃	9.407	—	—	9.407	18.814	SKY490
					t=16mm	〃	—	13.543	13.543	—	27.086	SKY490
				φ165.2	t=11mm	〃	5.676	6.018	5.308	5.108	22.110	STK400
			Weight of Attachments	Tip Reinforcing Band	PL t= 9mm	t	0.080	0.080	0.080	0.080	0.320	SS400
				Member for Site Circumference Welding (Backing Ring・Stopper)	PL t=14mm	〃	0.012	0.012	0.012	0.012	0.048	SS400
					PL t=16mm	〃	—	—	—	—	—	SS400
				Sling	PL t=22mm	〃	0.052	0.052	0.052	0.052	0.208	SM490A
				Interlocking Toe	PL t=12mm	Piece	2	2	2	2	8	SS400
				Combined Splice Pipe		Point	2	2	2	2	8	STK400
				Precut		〃	2	2	2	2	8	
		Per pipe (TypeB・D)	Weight of Steel Pipe Section	φ1200	t=14mm	t	19.224	18.406	14.929	16.361	68.920	SKY400
					t=14mm	〃	9.407	—	—	9.407	18.814	SKY490
					t=16mm	〃	—	13.543	13.543	—	27.086	SKY490
				φ165.2	t=11mm	〃	5.676	6.020	5.308	5.108	22.112	STK400
			Weight of Attachments	Tip Reinforcing Band	PL t= 9mm	t	0.080	0.080	0.080	0.080	0.320	SS400
				Member for Site Circumference Welding (Backing Ring・Stopper)	PL t=14mm	〃	0.012	0.012	0.012	0.012	0.048	SS400
					PL t=16mm	〃	—	—	—	—	—	SS400
				Sling	PL t=22mm	〃	0.052	0.052	0.052	0.052	0.208	SM490A
				Interlocking Toe	PL t=12mm	Piece	2	2	2	2	8	SS400
				Combined Splice Pipe		Point	2	2	2	2	8	STK400
				Precut		〃	2	2	2	2	8	
		Per pipe (TypeF)	Weight of Steel Pipe Section	φ1200	t=14mm	t	19.224	18.406	14.929	16.361	68.920	SKY400
					t=14mm	〃	9.407	—	—	9.407	18.814	SKY490
					t=16mm	〃	—	13.543	13.543	—	27.086	SKY490
				φ165.2	t=11mm	〃	8.514	9.030	7.962	7.662	33.168	STK400
			Weight of Attachments	Tip Reinforcing Band	PL t= 9mm	t	0.080	0.080	0.080	0.080	0.320	SS400
				Member for Site Circumference Welding (Backing Ring・Stopper)	PL t=14mm	〃	0.012	0.012	0.012	0.012	0.048	SS400
					PL t=16mm	〃	—	—	—	—	—	SS400
				Sling	PL t=22mm	〃	0.052	0.052	0.052	0.052	0.208	SM490A
				Interlocking Toe	PL t=12mm	Piece	3	3	3	3	12	SS400
				Combined Splice Pipe		Point	3	3	3	3	12	STK400
				Precut		〃	3	3	3	3	12	

Source: JICA Study Team

Table 4.2.214 Quantities for Substructure (SPSP Foundation - 2)

Type of Works	Item	Classification				Unit	Quantities P10Pier	Quantities P11Pier	Quantities P12Pier	Quantities P13Pier	Sum	Note
Steel Pipe Sheet Pile Foundation	Steel Pipe Sheet Pile	Per pipe (TypeG)	Weight of Steel Pipe Section	φ1200	t=14mm	t	28.631	30.267	26.790	25.768	111.456	SKY400
					t=14mm	〃	—	—	—	—	—	SKY490
				φ165.2	t=11mm	〃	5.676	6.020	5.308	5.108	22.112	STK400
			Weight of Attachments	Tip Reinforcing Band	PL t= 9mm	t	0.080	0.080	0.080	0.080	0.320	SS400
				Member for Site Circumference Welding (Backing Ring・Stopper)	PL t=14mm	〃	0.012	0.012	0.012	0.012	0.048	SS400
					PL t=16mm	〃	—	—	—	—	—	SS400
				Sling	PL t=22mm	〃	0.052	0.052	0.052	0.052	0.208	SM490A
				Interlocking Toe	PL t=12mm	Piece	2	2	2	2	8	SS400
				Combined Splice Pipe		Point	2	2	2	2	8	STK400
				Precut		〃	2	2	2	2	8	
		Per pipe (TypeH)	Weight of Steel Pipe Section	φ1200	t=14mm	t	28.631	30.267	26.790	25.768	111.456	SKY400
					t=14mm	〃	—	—	—	—	—	SKY490
				φ165.2	t=11mm	〃	5.678	6.020	5.310	5.108	22.116	STK400
			Weight of Attachments	Tip Reinforcing Band	PL t= 9mm	t	0.080	0.080	0.080	0.080	0.320	SS400
				Member for Site Circumference Welding (Backing Ring・Stopper)	PL t=14mm	〃	0.012	0.012	0.012	0.012	0.048	SS400
					PL t=16mm	〃	—	—	—	—	—	SS400
				Sling	PL t=22mm	〃	0.052	0.052	0.052	0.052	0.208	SM490A
				Interlocking Toe	PL t=12mm	Piece	2	2	2	2	8	SS400
				Combined Splice Pipe		Point	2	2	2	2	8	STK400
				Precut		〃	2	2	2	2	8	
		Total Number	Weight of Steel Pipe Section	φ1200	t=14mm	t	921.112	978.376	811.480	795.140	3,506.108	SKY400
					t=14mm	〃	338.652	—	—	338.652	677.304	SKY490
					t=16mm	〃	—	541.720	541.720	—	1,083.440	SKY490
				φ165.2	t=11mm	〃	255.428	294.940	260.100	229.860	1,040.328	STK400
			Weight of Attachments	Tip Reinforcing Band	PL t= 9mm	t	3.520	3.840	3.840	3.520	14.720	SS400
				Member for Site Circumference Welding (Backing Ring・Stopper)	PL t=14mm	〃	0.528	0.576	0.576	0.528	2.208	SS400
					PL t=16mm	〃	—	—	—	—	—	SS400
				Sling	PL t=22mm	〃	2.288	2.496	2.496	2.288	9.568	SM490A
				Interlocking Toe	PL t=12mm	Piece	90	98	98	90	376	SS400
				Combined Splice Pipe		Point	90	98	98	90	376	STK400
				Precut		〃	90	98	98	90	376	

Source: JICA Study Team

Table 4.2.215 Quantities for Substructure (SPSP Foundation - 3)

Type of Works	Item	Classification			Unit	Quantities P10Pier	Quantities P11Pier	Quantities P12Pier	Quantities P13Pier	Sum	Note
Steel Pipe Sheet Pile Foundation	Excavation inside				m3	511.0	619.3	504.0	369.7	2,004.0	
		Pile Head			m3	90.0	92.2	69.1	58.6	309.9	
	Concrete Filling	Infilling Concrete	σck=18N/mm2		m3	338.7	470.5	470.5	338.7	1,618.4	Correction factor = 0.04
		Pile Head			m3	12.7	13.6	13.6	12.7	52.6	
	Cleaning inside Joint Pipe				m	2,643.8	3,037.5	2,496.1	2,181.6	10,359.0	
	Mortar filling inside	σck=21N/mm2	Injected Length of Splice Mortar		m	2,439.0	2,856.7	2,440.2	2,133.0	9,868.9	Used Mortar = 2.5m3/100m
			Used Amount of Splice Mortar		m3	64.1	75.0	64.1	56.0	259.2	Correction factor = 0.05
	Sealing inside Joint Pipe	σck=0.2N/mm2	ength of Splice Water Stop Materia		m	493.2	548.0	548.0	493.2	2,082.4	
			d Amount of Splice Water Stop Mater		m3	14.0	15.6	15.6	14.0	59.2	
			Water Stop Bag		m	986.4	1,096.0	1,096.0	986.4	4,164.8	
	Excavation inside Well				m3	2,031.7	2,556.5	1,992.9	1,386.6	7,967.7	
	Backfill Inside Well				m3	731.6	744.3	229.9	173.7	1,879.5	
	Surplus Soil				m3	1,300.1	1,812.2	1,763.0	1,212.9	6,088.2	
	Footing Concrete	σck=24N/mm2			m3	743.6	1,105.0	1,105.0	743.6	3,697.2	Correction factor = 0.09
	Bottom Slab Concrete	σck=21N/mm2			m3	385.6	577.7	577.7	385.6	1,926.6	
	Spread Sand				m3	88.5	106.0	106.0	88.5	389.0	
	Pile Head Combination	Shear Connector	PL-32×16×3597		kg	231	231	231	231	924	
		Stopper	PL-25×9×50		〃	4	4	4	4	16	
	Pile Head Re-bar	Weight	SD345	D 13	kg	475	467	467	475	1,884	
				D16 ～ D25	kg	1,393	—	—	1,393	2,786	
				D29 ～ D32	〃	—	1,860	1,860	—	3,720	
				Total	〃	1,868	2,327	2,327	1,868	8,390	
	Re-bar for Top Slab	Weight	SD345	D 13	kg	—	—	—	—	—	
				D16 ～ D25	〃	8,533	11,096	11,096	8,533	39,258	
				D29 ～ D32	〃	8,672	—	—	8,672	17,344	
				D 35	〃	14,264	—	—	14,264	28,528	
				D 38	〃	—	31,452	31,452	—	62,904	
				D 51	〃	53,833	84,529	84,529	53,833	276,724	
				Total	〃	85,302	127,077	127,077	85,302	424,758	
		Mechanical Splice	SD345	D 35	Point	110	—	—	110	220	
				D 38	〃	—	174	174	—	348	
				D 51	〃	214	322	322	214	1,072	
				Total	〃	324	496	496	324	1,640	
	Connector (Welding of Dowel)	Block Number of Welding of Dowel			Stage	1,080	1,520	1,520	1,080	5,200	
		Mass of Welding of Dowel			kg	10,512	14,406	14,406	10,512	49,836	
	SPSP Cut	ϕ1200			Nos.	44	48	48	44	184	

Source: JICA Study Team