8.3 Study of Main Bridge

8.3.1 Selection of Type of Main Bridge

8.3.1.1 General

(1) Study Objections

In order to select appropriate main bridge by comprehensive evaluation in terms of construction cost, construction planning and aesthetic point of view, comparison study for the types of the main bridge is performed.

(2) Conditions of Study

In this study, the span arrangement of 95m+150m+150m+95m for fulfill the navigation clearance which mentioned in the previous chapter is assumed. The foundation of steel pipe sheet pile type and the erection method of Cantilever with Precast Girder Segments are also assumed, as recommended by JICA's Preparatory Survey (SAPROF Study). The comparison studies concerning erection method and foundation type are discussed in Chapters 8.3.2 and 8.3.5.

(3) Scope of Study

The study was implemented by checking the following items.

1) Selection of Main Bridge Type

In JICA's Preparatory Survey, PC Girder Bridge with V-shaped Piers is recommended and approved by MOT.

In this study, the following two alternatives are studied.

(i) Alternative-1: PC Girder with V-shape piers (Approved by MOT)

(ii) Alternative-2: Extradosed Bridge

Because of the lane arrangement in the future as shown in the figure below, the pylons for the extradosed bridge is to be located at the outside of the girder.

(4) Items to be studied

Items to be studied are economic, technical and other validities as shown in Table 8.1.6-1.

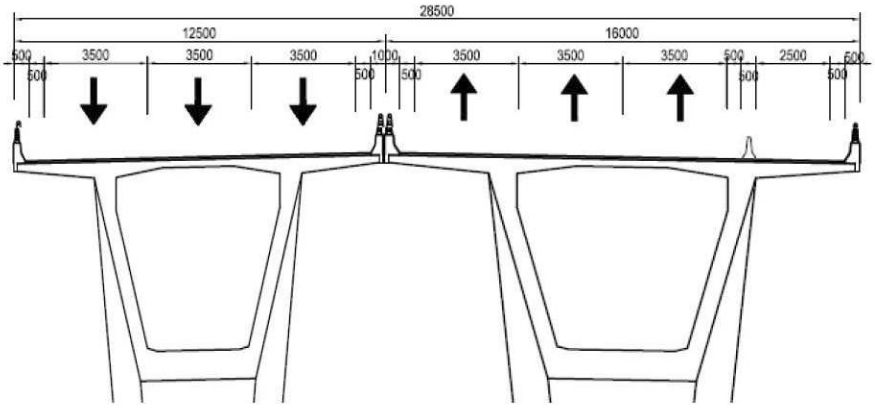
8.3.1.2 Contents of Study

(1) Comparison Study

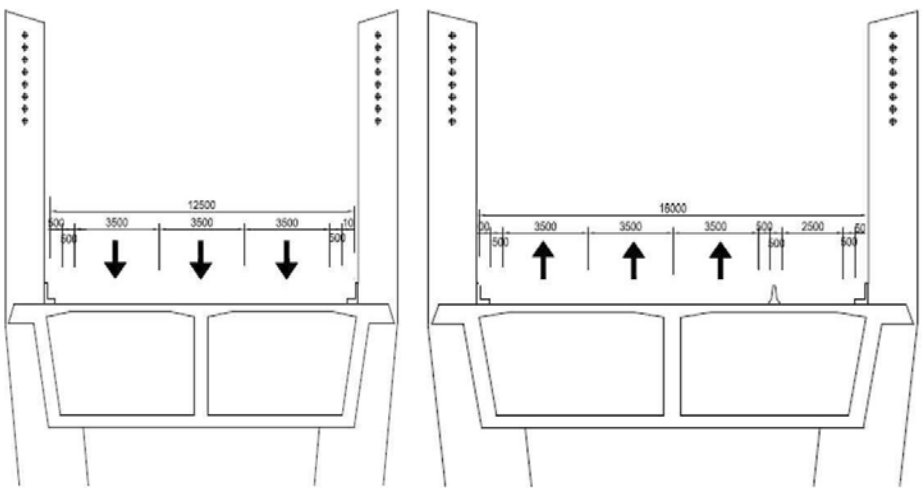
The comparison study is performed as shown in the following table.

1) Study Result

As shown in the following figure, Alternative-1: PC Girder with V-shaped piers is recommended from comprehensive view points, economic, technical and other viabilities.



(a) Alternative -1: PC Girder with V-shaped Piers

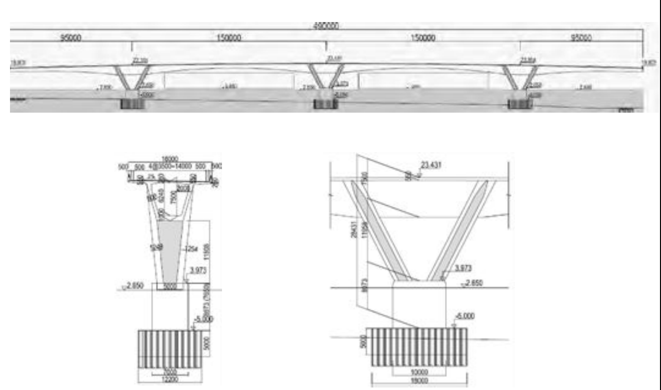
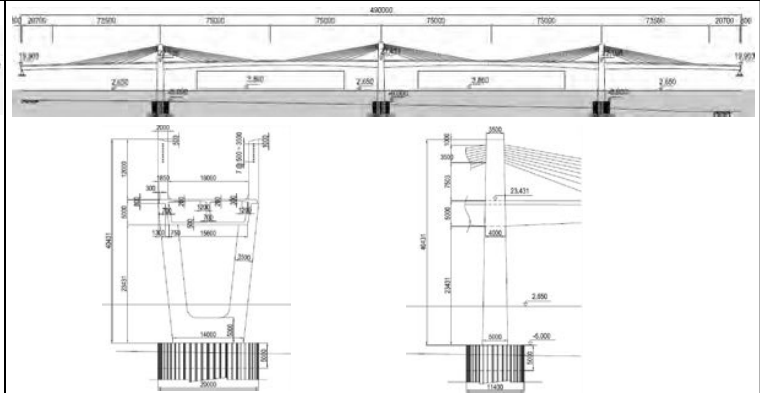


(b) Alternative -2: Extradosed Bridge

Source: Study Team

Figure 8.3.1-1 Cross Section of Main Bridge at Pier (Second Stage)

Table 8.3.1-1 Comparison on Type of Main Bridge

Evaluation Items	Max. Point	Alternative-1 PC Girder with V Shape Pier		Alternative-2 Extradosed Bridge with Twin Pylon	
Schematics		 SAPROF Study, Approved by MOT		 Double Pilar Tower, Double Suspension, Two Cell Box Girder	
Structural Aspect and Stability	10	Structurally Stable Girder is higher than Alternative-2, which resulted in higher vertical alignment There are no pilars nor cables outside of the girder, which resulted in narrower occupied area. Distance of webs is shorter (9m), which results in smaller concrete volume. Owing to V-shaped piers, the net center span length is 120m, which results in reasonable cost. The rigidity of piers is high, which results in large secondary stress induced by shrinkage of the girder.	8	Structurally Stable Girder is lower than Alternative-1, which resulted in lower vertical alignment. Double pilar tower with double suspension cables results in wider area for ocupation. Distance of webs is larger (17.4m), which results in two-cell box girder and larger concrete volume. The net span is 150m which is optimum length for extradosed type bridge. The rigidity of piers is high, which results in large secondary stress induced by shrinkage of the girder. Saddle or anchor box is needed at the top of pylon to hold the extradosed cables, which is expensive.	8
Construction Cost	40	Superstructure 142,818,000,000 VND Substructure 302,655,000,000 VND Total 445,473,000,000 VND Ratio 1.00	40	Superstructure 232,402,000,000 VND Substructure 309,321,000,000 VND Total 541,723,000,000 VND Ratio 1.22	24
Construction Plan and Period	10	V-shape pier and pier head needs large scale scafoldings. Construction Period: 22 months	6	Construction of pylon is an additional phase to Altenrative-1. Construction Period: 22 months	6
Maintenance	15	Any special maintentance for PC cables is not necessary. The number of bearings is the same as Alternative-2.	6	Cable maintenance such as tensile force monitoring will be applied. The number of bearings are the same as Alternative-1.	10
STEP Clearance	10	63% (Preliminary Estimate) Steel pipe sheet piles, PC strands, rebars and cement make contributions.	10	74%(Preliminary Estimate) In addition to Alternative-1, extradosed cables, anchors and saddle make a contribution.	10
Aesthetics	5	Slender appearance V-shape pier make a symbolic showing.	3	Slender appearnace with long span Tower and extradosed cables make a symbolic showing.	4
New Technology	5	Steel pipe sheet pile foundation is new technology in Vietnam.	4	Extradosed Bridge is new technology in Vietnam Steel pipe sheet pile foundation is new technology in Vietnam	5
Traffic Management/ Environmental Aspect	5	There are no pylons which are obstacles for air navigation. The cantilever method for girder erection can avoid obstacles for navigation.	4	There are pylons, but low enough for air navigation. The cantilever method for girder erection can avoid obstacles for navigation.	4
Evaluation	100	Construction cost is lower than Alternative-2 Recommended	81	Construction Cost is higher than Alternative-1 Less Recommended	71

Source : Study Team

8.3.2 Selection of Erection Method for Main Bridge

8.3.2.1 Comparative Study between Cast-in-place and Pre-cast segment

For concrete box girder with long span over 100m, a balanced cantilever construction is the most appropriated and suitable erection method under the constraints such as navigation channel, deep valley and river. The main bridge has 2 spans of 150m for clearance of navigation channel and 90m on both side spans which are constructed with the balanced cantilever construction. The balanced cantilever construction is divided into two methods which are cast-in-place and precast segments in relation with design and construction concepts and economic point. The comparative study for cast-in-place and precast segment methods are shown in the table on the next page.

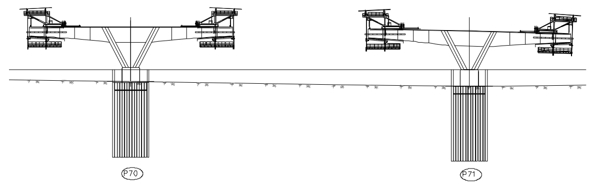
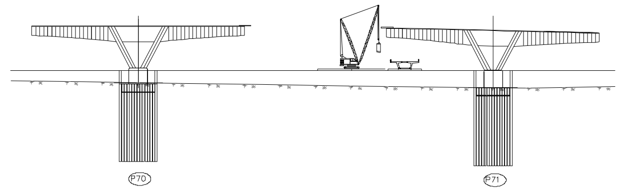
As the results on the comparative study, cast-in place balanced cantilever segment method is selected by the following main points;

-Precast balanced cantilever segment method can be shorten construction period by two months in comparison with cast-in-place balanced cantilever segment method but the construction period of cast-in-place is not in critical pass of total construction schedule.

-Precast segments are erected by crawler cranes mounted on the barges that disturb ship traffic in the navigation channel during construction.

-Wide precast yard and casting equipment are required for fabrication of box segment in the Cut Hi side where is in the limited small island.

Table 8.3.2-1 Comparison between Cast-in-place Cantilever Method and Precast Segment Method of Main Bridge

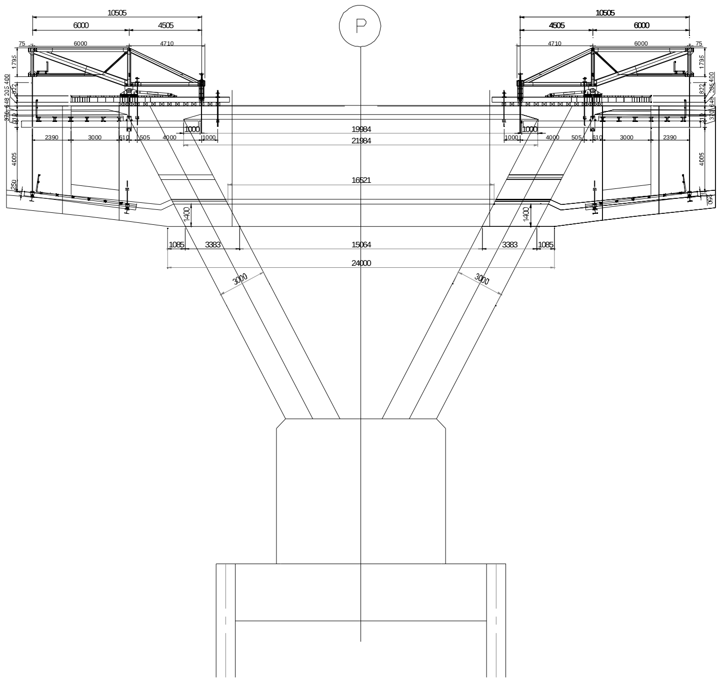
Evaluation Items	Max. Point	Alternative-1		Alternative-2	
Side View					
		Cast-in-place Balanced Cantilever Method		Precast Balanced Cantilever Method	
Structural Aspect and Stability	10	・Cast-in-place technique is applied for long and irregular span length with a varying depth girder like a proposed main bridge. ・Prime application of cast-in-place cantilever bridge is used over navigation channel without any disturbance of sea traffic.	10	・Casting of superstructure segments are started at the beginning of construction at the same time as construction of substructure. ・Segments are produced in the fabrication yard providing consistent rates of production with superior quality control.	10
Construction Cost (Vietnam Million Dong)	40	Superstructure 154,459 Substructure 282,784 Total 430,281 Ratio 1.00	40	Superstructure 177,628 Substructure 282,784 Total 460,412 Ratio 1.22	24
Construction Plan and Period	10	・Number of segment is reduced due to the use of large erection wagon and the rate of construction is improved. ・Construction period: 24 months	8	・Since eliminating the use of falsewaork, the rate of erection girder is increased in comparison with Cast-in-place. . ・Construction period: 22 months	10
Maintenance	15	・All bridge members are rigid connection and maintenance free except 4 bearing shoes. Efficient concrete coverage is applied for sea side area.	12	・All bridge members are rigid connection and maintenance free except 4 bearing shoes. Efficient concrete coverage is applied for sea side area.	12
STEP Clearance	10	・Procurement material: steel pipe, PC cable and anchor, Steel pipe pile ・Procurement equipment: Large erection wagon ・Procurement ratio: Approximately 55%	10	・Procurement material: steel pipe, PC cable and anchor, Steel pile ・Procurement equipment: Erection Lifting, Match casting machine ・Procurement ratio: Approximately 55%	10
Aesthetics	5	・Aesthetically configuration is the same after completion of bridge.	5	・Aesthetically configuration is the same after completion of bridge.	5
New Technology	5	・Common technology but large scale in Vietnam	4	New technology of precasting: Match casting in Vietnam	5
Environmental Impact	5	・Avoid disruption to the existing ship traffic in the navigation.	5	・Temporary yard is required for precasting girder segment and disturbance ship traffic during construction.	3
Evaluation	100	・Cast-in-place balanced cantilever method is recommended because construction of girder can be proceeded without any disturbance of sea traffic and out of critical pass of project. Recommendable	94	・This alternative has advantage of short construction period but disadvantage is disturbance ship traffic due to segment transportation. ・Large fabrication area is required in Cat Hai Island. Less Recommendable	79

Remark: Construction cost is direct cost based on JICA's Preparatory Survey, therfore its amount is revised in detail design.

Source : Study Team

(1) Election of Main Bridge

Erection of the segments in “balanced” cantilever was done with traveling forms supported by the previously completed portions of the superstructure as shown in the figure below. These space frames supported the forms and wet concrete weight of the segments, the largest of which weighed on the order of 4,000 KN and was approximately 16 m width. The form travelers also support platforms required to finish the concrete surfaces and install transverse and longitudinal prestressing tendons. The construction sequence is shown in the Construction plan.



Source: Study Team

Figure 8.3.2-1 Erection of Main Girder with Form traveler

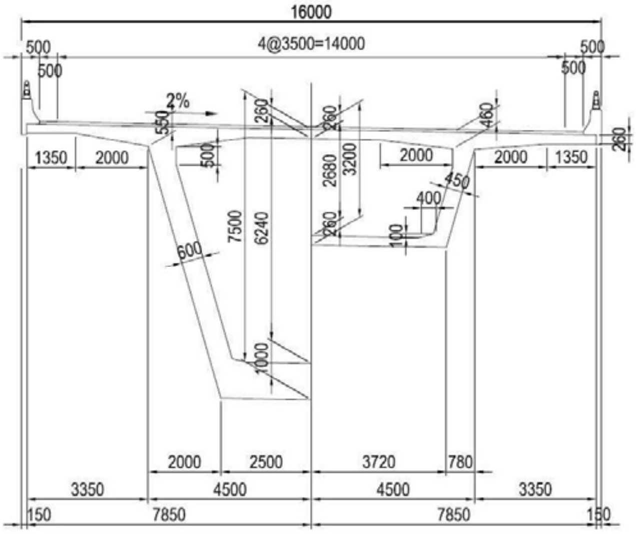
8.3.3 Superstructure of Main Bridge

8.3.3.1 Cross Section of Main Girder

Outside configurations of main and approach bridges applies to the same dimension of Approach Bridge to focus on the continuity of structure. Comparative study of the cross section was presented in the article 3.5 of Preliminary Study Report.

The cross-section of the main bridge is a single-cell box girder, consisting of the upper slab with cantilevering flanges, webs, and the lower slab as shown in the Figure below. The box girder webs have a constant vertical inclination of 28.78 to 100 considering the width of lower slab of 5.0m of main bridge.

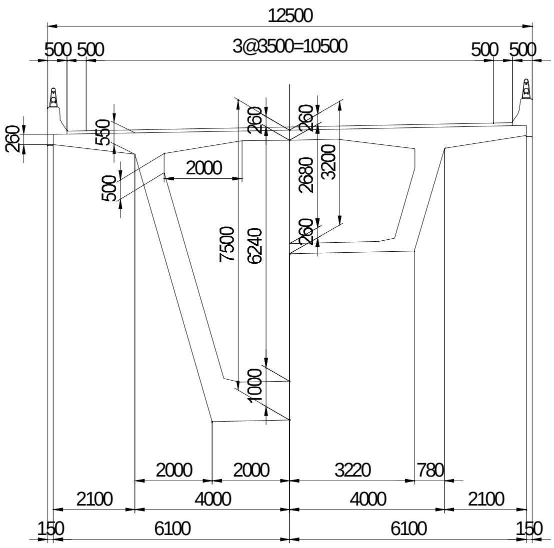
The height of box girder varies from 7.5m (Height/Span=7.5/150=1/20) at top of the piers to 3.5m at the mid-spans. The width of the upper slab is constant at 16.0 m including the cantilevering flanges of about 3.35m width each. The box girder will carry 4 lanes of traffic that are each 3.50m wide. The bridge deck has a transverse slope of 2% for drainage purposes and will carry a concrete railing at its edges. The thickness of the upper slab cantilevers varies transversally from 0.26m to 0.55m outside the box but from 0.26m to 0.46m at the end segments of main bridge to adjust the thickness to the approach bridge. The lower slab has a thickness variable from 1.0m at the top piers to 0.26m at the mid-span. The thickness of webs also varies transversally from 0.6m to 0.45m as shown in the figure below. Internal diaphragms are provided at the top of V-shaped walls piers, on which the deck is restrained, where they are 3.0m thick, as well as at the end pier of main bridge where they are 2.5m thick which are situated over the bridge bearings in the last box girder segment directly adjacent to the approach bridge. Internal diaphragms are provided inside the spans for positioning at 7th segment from the top of the pier.



Source: Study Team

Figure 8.3.3-1 Cross Section for Main Girder of the Initial Stage

The main bridge is planned with the stage construction integrated with pile cap in JICA Preparation Study. Is case that integrated pile cap is applied; cross section of 2nd stage is configured as shown in the following figure to estimate external forces for pile cap design.

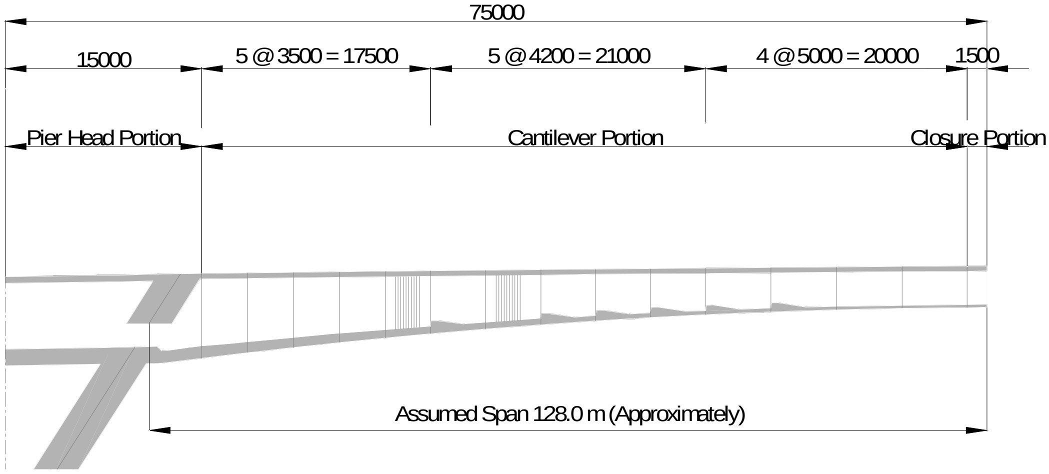


Source: Study Team

Figure 8.3.3-2 Cross Section for Main Girder for the Second Stage

8.3.3.2 Cantilever Segments and Pier Head

A pier table length of 15.0m with an asymmetry of to the pier centerline, and several types of segments, being 5.0m long in the spans and 3.0m at the closure segments but only 3.5m near the pier table as shown in the figure below. Keeping deeper segments shorter was intended to level volumes of concrete placement and capacity of form traveler during construction. To reduce of number of segments, the form traveler with capacity of 4,000kNm is selected for a long span bridge.



Source: Study Team

Figure 8.3.3-3 Cantilever Segments and Pier Head

8.3.3.3 Cast-in-Place Segments on the False work

At the end segments of side span, the long segments out of cantilever portion are built on-site using formwork supported by temporary false work as shown in the figure below.

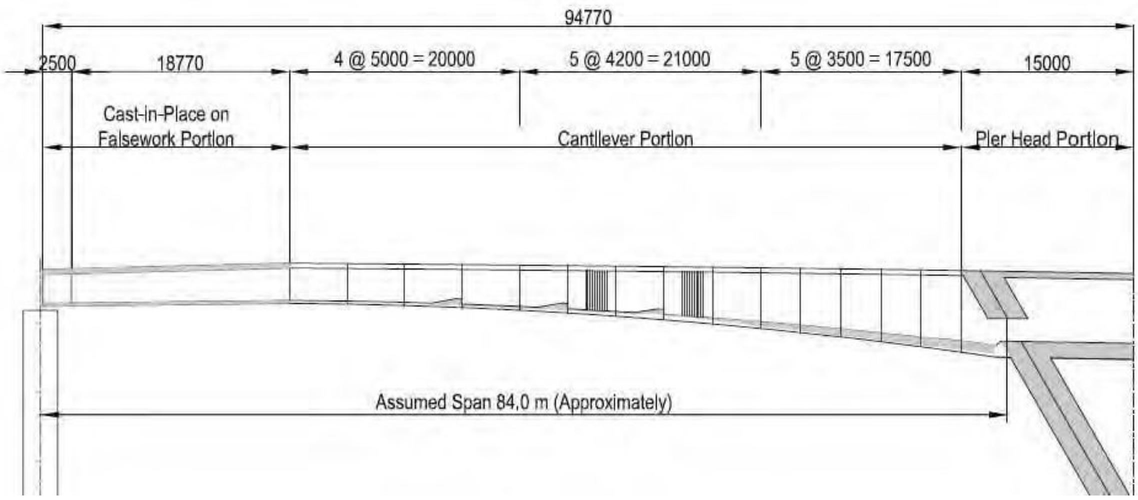
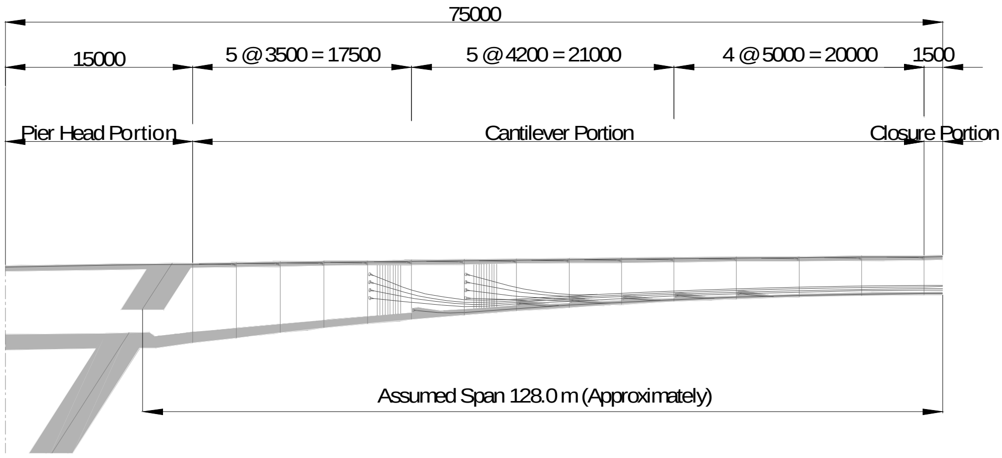


Figure 8.3.3-4 Cast-in-place Segments on False Work

8.3.3.4 Post-Tensioning System

Longitudinal cantilever erection tendons are located in the upper slab of both box and cantilever sections. Naturally, they will be stressed beginning with the shortest tendons that connect only a few segments. Stressing of the tendons will be performed on newly cast segments prior to stripping of the forms in an alternating manner on the end at which the newly cast segment. This sequence is expected to generate a more even distribution of prestressing forces, including losses of prestress, in the superstructure.

The continuity tendon prestrssing is carried out after the completion of the cantilever segment and the connection of them by casting key segment in the mid span. The tendons are divided in 5 groups: two of them are placed on the expansions of the webs and the remaining three are placed on the lower slab. Tendons are anchored surrounded by spiral reinforcement in anchorage blocks, also called anchorage blisters, which are symmetrically located on the inside lower slab edges of the box girder. The length of tendons varies from 30.0 m to 68.0 m and their profile follows the profile of the lower slab. The arrangement of the longitudinal cantilever tendons in the upper slab is shown in the figure below.



Source: Study Team

Figure 8.3.3-5 Arrangement of Longitudinal Cantilever Tendons

8.3.3.5 Materials used for Main Girder

Main construction materials in main bridge are listed below

- Cantilever cable and Continuity cable: 12S15.2 or 19S15.2
- Transversal cable: 1S28.6
- Concrete strength: 40 N/mm2
- Reinforcing bar: SD345

8.3.4 Substructure of Main Bridge

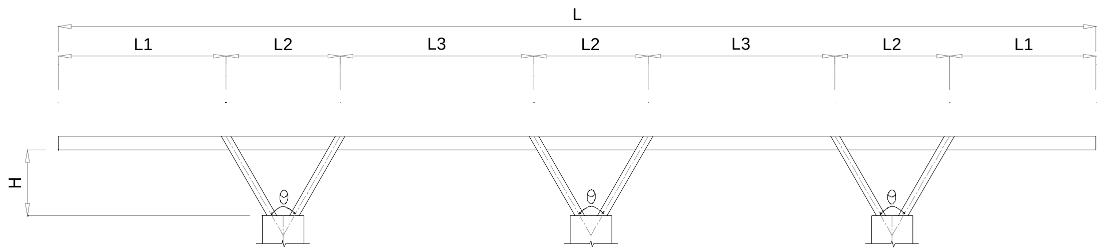
8.3.4.1 General

Main bridge of the Lach Huyen Bridge has 4 spans and is 490m in total length supported 3 V-shaped piers. The V-shaped pier was selected considering the economical point due to reduction of span length and aesthetical view point in the Feasibility Study. In the Preparatory Survey conducted by the JICA in 2010, the V-shaped Pier was also selected by the JICA Study Team and approved by the MOT.

In order to select appropriate structure of the piers in the main bridge, issues on V-shaped pier; Skelton balance, shape in Longitudinal Direction, level on bottom and aesthetic consideration are discussed in this chapter.

8.3.4.2 Skelton Balance of V-shaped Pier

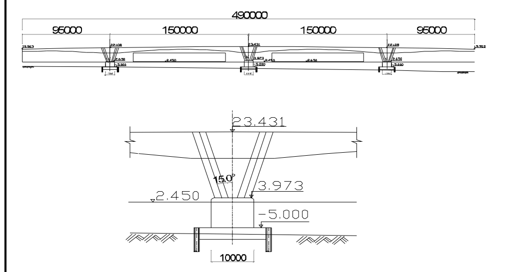
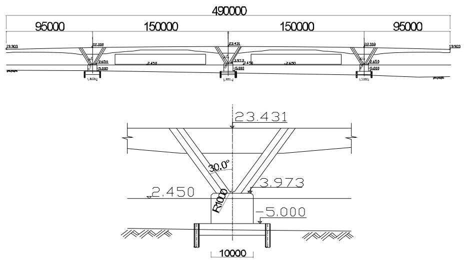
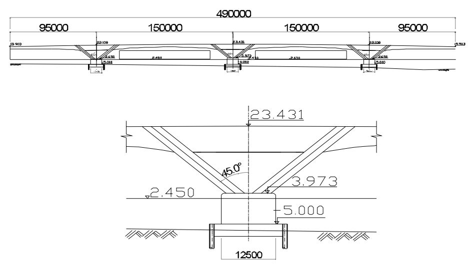
The V-type pier of main Bridge is determined by the balance of 6 elements (skeleton) of θ, L, L1, L2, L3 and H as shown in the figure below considering the natural and structural conditions and aesthetic view point. H (height of wall of V-shape) should be fixed to maintain same shape of triangle and make a simple structure. In case that top of sub-structure is set above high water level of 2.55 m (Design frequency: P=5%), inspection and maintenance of V-shape walls are easily conducted after the construction. In case that the top of sub-structure is set at level of 2.65 m recommended in Sub-section 8.3.4.4, height of 10cm is secured for allowance. θ (Angle at the vertex of triangle is mainly influenced to economical and aesthetical points. The comparative study of V-shape varying θ from 30°to 90°is conducted and shown in the table on the next page.



Source: Study Team

Figure 8.3.4-1 Skelton of V-shaped Piers

Table 8.3.4-1 Comparison Study on Angle of V-shaped Pier

Evaluation Items	Max. Point	Alternative-1		Alternative-2		Alternative-3	
Side View							
		Approx. θ =15°		Approx. θ =30°		Approx. θ =45°	
Structural Aspect and Stability	10	・Center span is reduced from 150m to 140m but deduction of girder depth (7.8m) is almost same as single pier (8.0m). ・Bottom of V-shaped pier is a rigid connection to substructure considering earthquake-resistance.	8	・Center span is reduced from 150m to 125m and depth of girder also from 8.0m to 7.5m economically. ・Bottom of V-shaped pier is a rigid connection to substructure considering earthquake-resistance.	10	・Center span is reduced from 150m to 110m and depth of girder from 8.0m to 7.2m. But Pier top structure has long span and large tension force may occur on the lower slab. ・Bottom of V-shaped pier is a rigid connection to substructure considering earthquake-resistance.	6
Construction Cost (Vietnam Million Dong)	40	Superstructure 171,621 Substructure 274,548 Total 446,169 Ratio 1.02	32	Superstructure 154,459 Substructure 282,784 Total 437,243 Ratio 1.00	40	Superstructure 145,878 Substructure 329,458 Total 475,335 Ratio 1.09	32
Construction Plan and Period	10	・Substructure is almost same as Alternative-2 but 15 cantilever segments are required. ・Construction period: 25 months	8	・Construction period can be minimized by the deduction cantilever segments from 15 to 14 segments ・Construction period: 24 months	10	・Erection period of 13 segments can be minimized but construction period of substructure be extended by large large foundation. Construction period: 26 months	6
Maintenance	15	・All bridge members are rigid connection and maintenance free except 4 bearing shoes. Efficient concrete coverage is applied for sea side area.	12	・All bridge members are rigid connection and maintenance free except 4 bearing shoes. Efficient concrete coverage is applied for sea side area.	12	・All bridge members are rigid connection and maintenance free except 4 bearing shoes. Efficient concrete coverage is applied for sea side area.	12
STEP Clearance	10	・Procurement material: steel pipe, PC cable and anchor ・Procurement equipment: Large erection wagon ・Procurement ratio: Approximately 35%	8	・Procurement material: steel pipe, PC cable and anchor ・Procurement equipment: Large erection wagon ・Procurement ratio: Approximately 35%	8	・Procurement material: steel pipe, PC cable and anchor ・Procurement equipment: Large erection wagon ・Procurement ratio: Approximately 37%	8
Aesthetics	5	・V-shaped pier is sharp angle and elegant as a single pier, but does not balance with center span (150m) aesthetically.	4	・V-shaped pier is simply an equilateral triangle and aesthetically stable and matching with horizontal sea line	5	・Wide V-shaped pier is unstable image in low navigation clearance.	3
New Technology	5	New shaped pier using new scaffolding system in Vietnam	5	New shaped pier using new scaffolding system in Vietnam	5	New shaped pier using new scaffolding system in Vietnam	5
Environmental Impact	5	No major environmental impact	5	No major environmental impact	5	No major environmental impact	5
Evaluation	100	・Aesthetically recommendable but narrow V-shaped pier has less effect on structural and economical aspects than Alternative-2 Less Recommendable	82	・This alternative is superior to other cases in economic and aesthetic points. Most Recommendable	95	・Angle of V-shape is too much opened so that structurally and aesthetically unfavorable with the structure on the sea. Not Recommendable	77

Remark: Construction cost is direct cost based on JICA's Preparatory Survey, therefore its amount is revised in detail design.

Source : Study Team

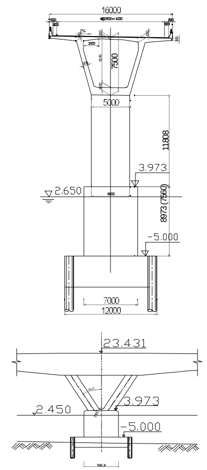
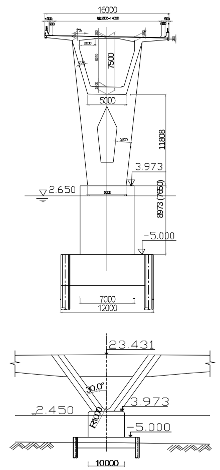
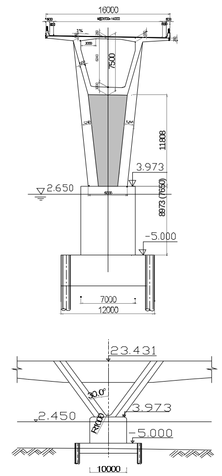
8.3.4.3 Comparative Study on Pier Appearance in Longitudinal Direction

Angle at the vertex of triangle (θ) of approximate 60°is selected for V-shaped pier from structural and economical viewpoints. The V-shaped pier in longitudinal direction has 3 alternative configurations planned considering structural stability and aesthetic point.

a) Alternative-1: Simple solid rectangular type
b) Alternative-2: Reversed trapezoid with void
c) Alternative-3: Reversed trapezoid with slit

V-shaped wall extended to upper slab is recommended to make a large triangle shape as a symbolic appearance and aesthetic structure from side view. Comparative study of sub-structure with V-shaped pier in longitudinal direction is shown in the table on the next page. Comparisons are mainly evaluated with structural and aesthetical points. Difference of construction cost is minimal so that an economical evaluation is considered to be almost same. Alternative-3 is recommended in aesthetical point.

Table 8.3.4-2 Comparison Study on Pier Appearance in Longitudinal Direction

Evaluation Items	Max. Point	Alternative-1		Alternative-2		Alternative-3	
Side View							
Structural Aspect and Stability	10	•Loads (Morment) from superstructure is concentrated at part connecting superstructure and V-shaped wall.	6	•Loads from superstructure flow smoothly to substructure through webs of box girder and V-shaped wall.	10	•Loads from superstructure flow smoothly to substructure through webs of box girder and separated V-shaped wall.	10
Construction Cost (Vietnam Million Dong)	40	Superstructure 154,459 Substructure 282,784 Total 437,243 Ratio 1.00	40	Superstructure 154,459 Substructure 283,632 Total 438,091 Ratio 1.002	40	Superstructure 154,459 Substructure 284,481 Total 438,940 Ratio 1.004	40
Construction Plan and Period	10	24 months	10	24 months	10	24 months	10
Maintenance	15	•All bridge members are rigid connection and maintenance free except 4 bearing shoes. Efficient concrete coverage is applied for sea side area.	12	•All bridge members are rigid connection and maintenance free except 4 bearing shoes. Efficient concrete coverage is applied for sea side area.	12	•All bridge members are rigid connection and maintenance free except 4 bearing shoes. Efficient concrete coverage is applied for sea side area.	12
STEP Clearance	10	•Procurement material: steel pipe, PC cable and anchor •Procurement equipment: Large erection wagon •Procurement ratio: Approximately 55%	10	•Procurement material: steel pipe, PC cable and anchor •Procurement equipment: Large erection wagon •Procurement ratio: Approximately 55%	10	•Procurement material: steel pipe, PC cable and anchor •Procurement equipment: Large erection wagon •Procurement ratio: Approximately 55%	10
Aesthetics	5	•Small triangle shaped with V-shaped wall is an oppressive feeling in low clearance of bridge. •V-shaped wall, which is very simple structure, is need some more aesthetic accents.	3	•A large triangle shaped with V-leg wall extended to upper slab is a symbolic appearance and aesthetic structure.	4	•A large slit shaped traprzoid extended to the bottom of lower slab is a symbolic appearance and aesthetic view. •The slit can be painted to make contrast for aesthetics.	5
New Technology	5	•New shaped pier using new scaffolding system in Vietnam	5	•New shaped pier using new scaffolding system in Vietnam	5	•New shaped pier using new scaffolding system in Vietnam	5
Environmental Impact	5	•No major environmental impact	5	•No major environmental impact	5	•No major environmental impact	5
Evaluation	100	•Structurally and aesthetically not recommended in comparison with other Alternatives. Not Recommendable	91	•Aesthetically not recommendable in comparison with Alternative-3 Less Recommendable	96	•This alternative is superior to other Alternatives in structural and aesthetic points. Most Recommendable	97

Remark: Construction cost is direct cost based on JICA's Preparatory Survey. Therefore, its amount is to be revised in detail design.

Source : Study Team

8.3.4.4 evel on Bottom of V-shaped Wall (Level of Pier Top)

Level of bottom of V-shape wall should be determined by the following points;

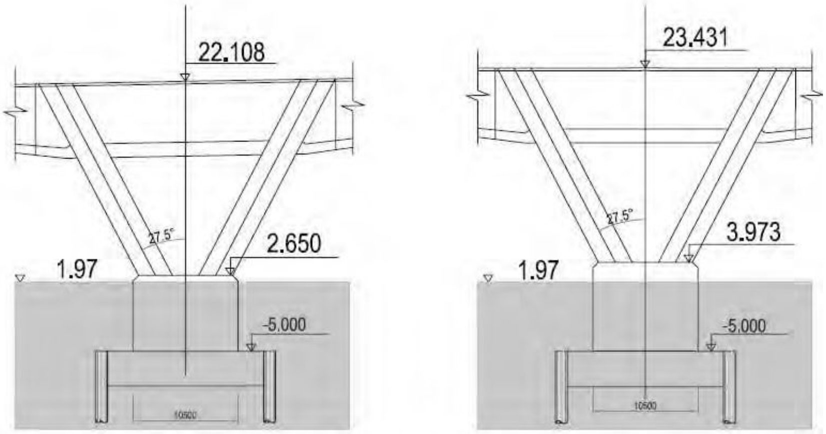
a) Sea water level
b) Size of triangle of V-shape wall
c) Planning height of bridge

Main bridge is a symmetric vertical alignment at the center of bridge length. 3 V-shaped piers of main bridge are basically same size triangle in structural and aesthetical considerations. Level on bottom of V-shaped wall at center pier is higher than adjacent both side piers. For future bridge inspection and maintenance (if necessary), the level of all top piers should maintain above sea water level. Sea water levels in the main bridge site are set for bridge design as follow;

- Design High Water Level (P=1%): 2.720m
- High Water Level (P=5%) : 2.550m
- Mean High Water Level: 1.970m
- Mean Water Level: 0.150m
- Mean Low Water Level: -1.67m

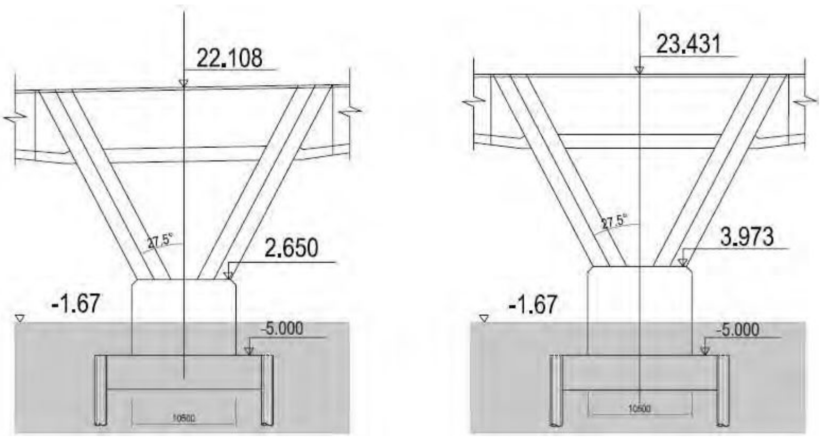
In case that the bottom of V-shaped wall of both side piers is above high water level (P=5%) for determination of navigation clearance, the V-shaped wall is always seen above water level. However, mass-concrete of sub-structure comes in sights above water level at mean low water level and may spoil aesthetic view of V-shaped pier as shown in following Figure 8.3.4-2. On the other hand, in case that the bottom of V-shaped wall of both side piers is set above mean low water level as shown in Figure 8.3.4-3, the V-shaped wall always sinks under water and not only spoils aesthetic view due to submergence of triangle but also requires periodical inspection and maintenance.

Consequently, the level of pier top is designed both by comparing with the water level of 2.55 m (High Water Level) for easy maintenance and by comparing with the water level of 1.97m (Mean High Water Level) for aesthetical view. The pier top of sub-structure should be set at level of 2.65 m considering allowance of 10 cm.



Side Piers Center Pier

(a) Mean High Water Level

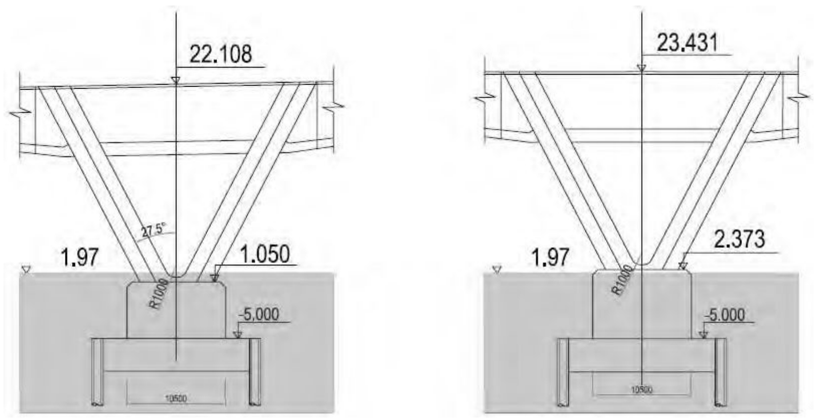


Side Piers Center Pier

(b) Mean Low Water Level

Source: Study Team

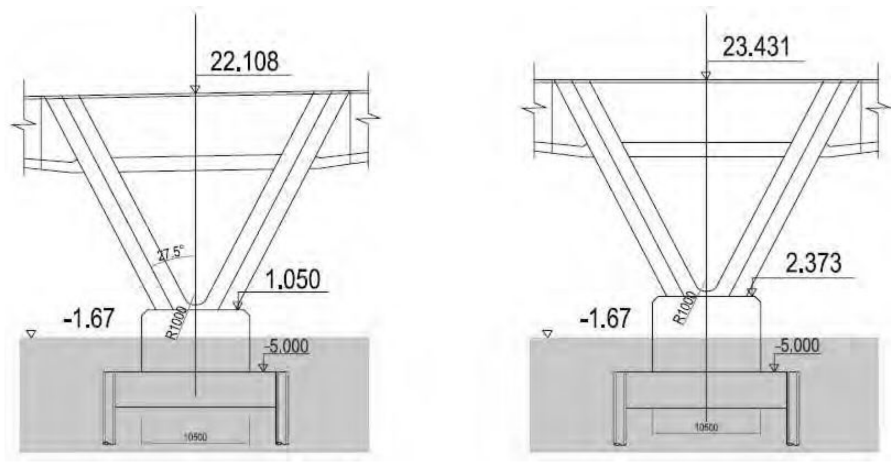
Figure 8.3.4-2 Projected Pier (2.650m) above Mean High Water Level and Mean Low Water Level



Side Piers

Center Pier

(a) Mean High Water Level



Side Piers

Center Piers

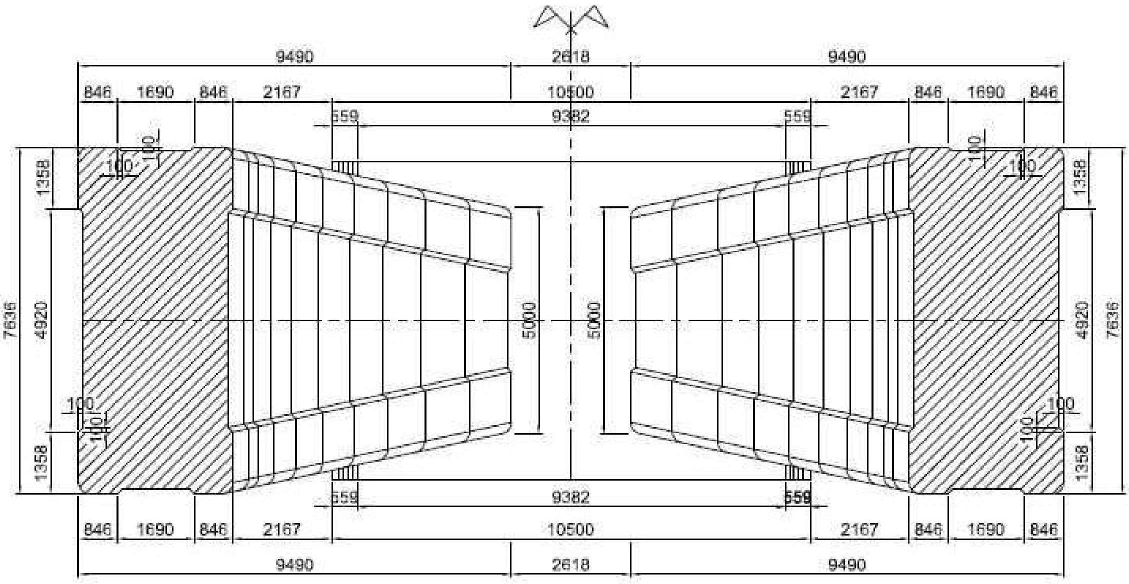
(b) Mean Low Water Level

Source: Study Team

Figure 8.3.4-3 Projected Pier (1.050m) above High Water Level and Mean Low Water Level

8.3.4.5 Aesthetic Considerations of V-shaped Pier

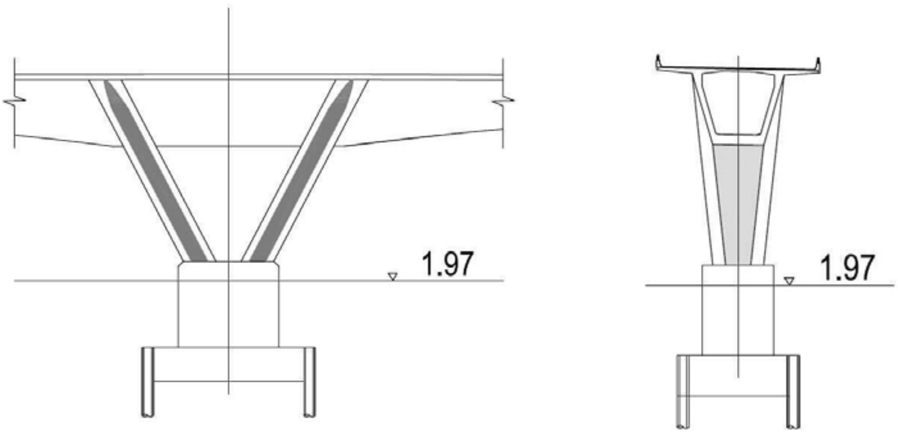
One of creativity on structure is aesthetics of pier. We create smooth and soft appearance by creating curved line. Walls of V-shaped pier make a slim on concrete wall with moderate curve and straight lines. At the corner of column, curves of 250mm radius are provided to make a soft line corners as shown in the figure below.



Source: Study Team

Figure 8.3.4-4 Corner Arrangement of V-shaped Wall Pier

Vertical slits are also provided to show slender V-shaped wall pier and to enjoy contrast between light and shade as shown in the figure below. Furthermore, to show stronger impact, the slits are painted to make contrast with white concrete and blue sea water.



Source: Study Team

Figure 8.3.4-5 Vertical Slit for Aesthetical Aspect

8.3.4.6 Effect of Creep and Shrinkage on V-shaped Pier

(1) Effect of Creep and Shrinkage in Ridged Frame Bridge

1) General

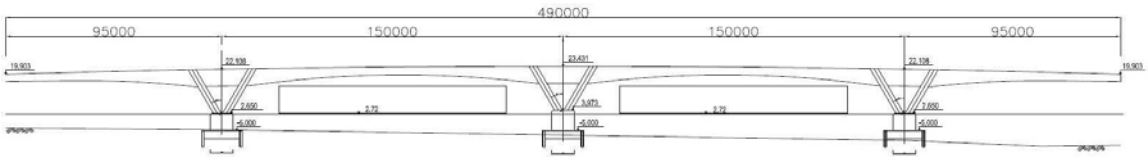
In general, piers of a concrete ridged frame bridge are vulnerable to the effect of creep and shrinkage of the girder, especially; in case of a long span ridged bridge with short pier.

Pre-stressed concrete girder has tendency to shorten due to creep and shrinkage through its life and the deformation becomes large in proportion to span length.

2) Effect of Creep and Shrinkage on Pier of Main Bridge

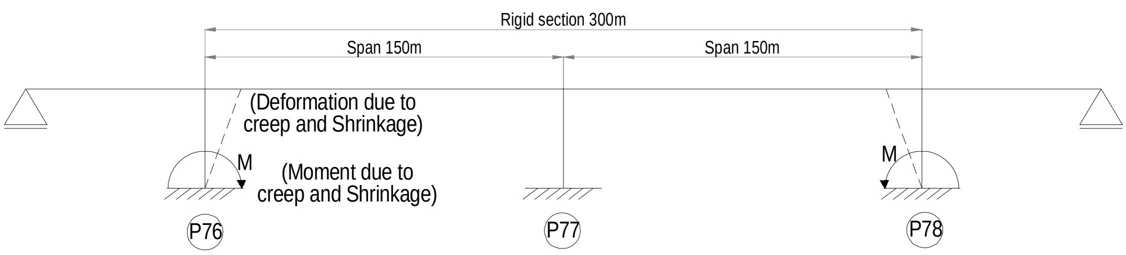
Main bridge consists a pre-stressed concrete 4-span continuous rigid-frame bridge with 2 spans of 150m in the center. The rigid section is 300m in longitudinal direction and piers are short (approx. 20m high from top of the pile cap), rigidly connected to box girder with triangle-shape as shown in the figure below. The main bridge, which is constructed by cantilever method, is not flexible enough to accommodate longitudinal movement due to creep and shrinkage through its long time behavior.

After the main bridge is connected in the mid-span, the deformation is generated toward inside. The constraint effects of the longitudinal movement of the box girders can introduce large stresses and consequently large moments in short and V-shaped piers. Time dependent effects of creep and shrinkage are particularly significant on long-term deformation.



Source: Study Team

Figure 8.3.4-6 Span Arrangement and Profile of Main Bridge



Source: Study Team

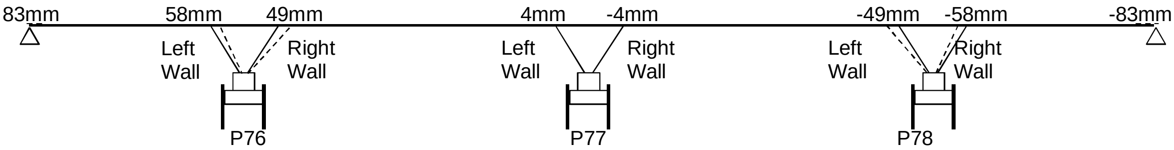
Figure 8.3.4-7 Deformation and Moment due to Creep and Shrinkage

(2) Result of Preliminary Analysis

To indicate the effects of the measure, results from preliminary structural analysis, displacement, section forces and stress, are introduced, in this section. The conditions of analysis are presented in the section 8.3.7.2. The values of displacement, sectional forces and stress presented in this section are tentative based on the preliminary analysis.

1) Displacement

The displacements 100 years after completion of construction are summarized in the figure below.

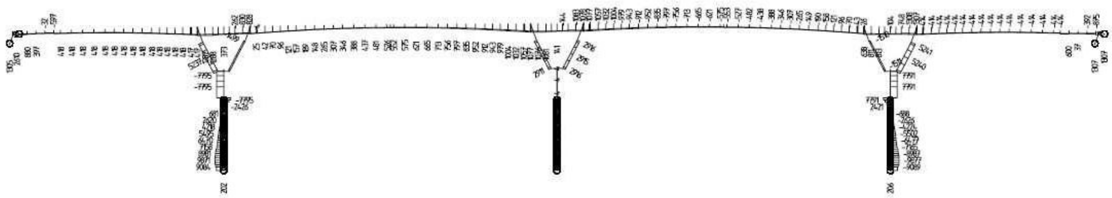


Source: Study Team

Figure 8.3.4-8 Longitudinal Displacement* due to Creep and Shrinkage

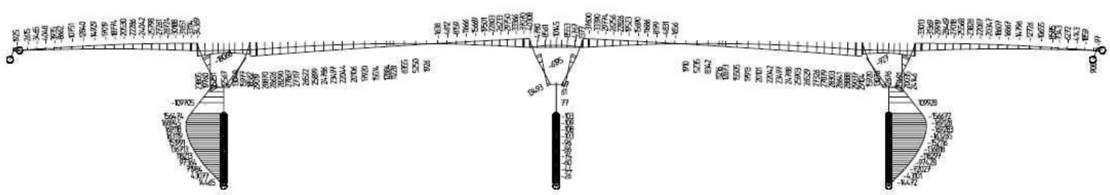
2) Section Forces

The section forces 100 years after completion of construction due to creep and shrinkage are shown in the figure and the table below.



Source: Study Team

Figure 8.3.4-9 Shear Force* due to Creep and Shrinkage



Source: Study Team

Figure 8.3.4-10 Bending Moment* due to Creep and Shrinkage

Table 8.3.4-3 Summary of Section Forces* of Pier due to Creep and Shrinkage

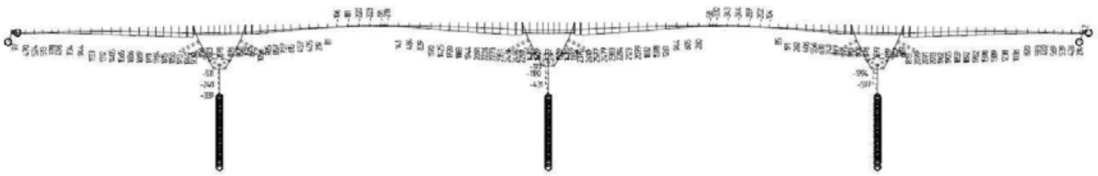
Pier No.	Wall	Position	Axial Force* (kN)	Shear Force* (kN)	Bending Moment* (kN・m)
P76	Left Wall	Top of Wall	-2,136	5,380	-34,349
		Bottom of Wall	-2,136	5,358	-35,857
	Right Wall	Top of Wall	2,488	-1,324	2,542
		Bottom of Wall	2,488	-1,324	-15,214
P77	Left Wall	Top of Wall	-1,187	3,111	-24,448
		Bottom of Wall	-1,187	3,111	14,496

3) *the values are based on the results from the preliminary structure analysis

Source: Study Team

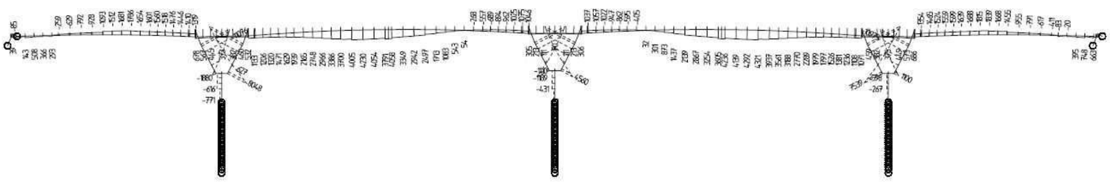
4) Stress

The stress due to creep and shrinkage 100 years after completion of construction are summarized in the figure and the table below. Creep and shrinkage of the girder induce additional tensile stress up to 5MPa which causes damages in the left wall of P76 and at the right wall of P78.



Source: Study Team

Figure 8.3.4-11 Stress* of Top Fiber due to Creep and Shrinkage



Source: Study Team

Figure 8.3.4-12 Stress* of Bottom Fiber due to Creep and Shrinkage

Table 8.3.4-4 Summary of Stress* of Pier due to Creep and Shrinkage

Pier No.	Wall	Position	Tensile Stress* (MPa)		
			Dead Load (DC, DW, PS)	Creep, Shrinkage (CR,SH)	DC+DW+PS+CR+SH
P76 (Left Pier)	Left Wall	Top of Wall	5.258	3.428	8.686
		Bottom of Wall	7.079	4.933	12.012
	Right Wall	Top of Wall	4.582	-0.143	4.439
		Bottom of Wall	6.521	-1.988	4.533
P77 (Center Pier)	Left Wall	Top of Wall	2.977	2.456	5.433
		Bottom of Wall	5.778	1.972	7.751

5) *the values are based on the results from the preliminary structure analysis

Source: Study Team

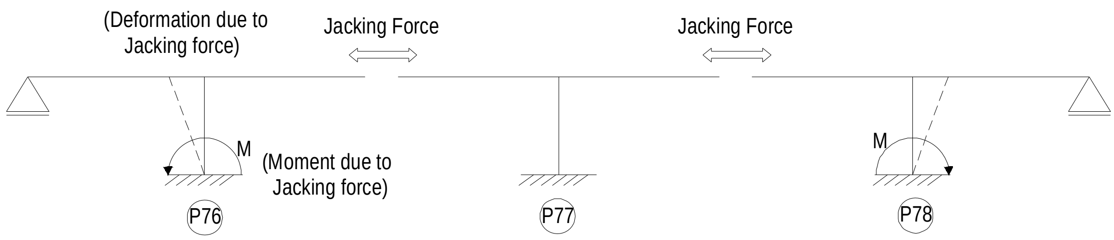
(3) Subsidiary Measure

1) Pressurization before Closing

In order to reduce large bending moment induced by creep and shrinkage, which causes damages on the V-shaped piers, a subsidiary measure is required.

The recommended measure is pressurization immediately after the segments are constructed by cantilever method, which was applied to rigid frame bridges in Thanh Chi Bridge Construction Project and other projects.

In the subsidiary measure, jacking forces in the longitudinal direction are loaded just before closing segments are casted at center of span. The bending moment in the V-shaped piers due to the jacking forces can cancels out the part of bending moment due to creep and shrinkage.



Source: Study Team

Figure 8.3.4-13 Deformation and Moment due to Pressurization (Jacking Force)

2) Construction Step and Control of Pressurization

The recommended method involves the following steps as shown in the figure on the next page.

Step 1: Erect box girders successively with pre-stressing cantilever tendons

Step 2: Install blisters, brackets and struts for jacking at both top and lower slabs of box girder. 4 jacking equipment (2,000 kN) set at the blisters and struts and pushes the end of box girder outward to introduce moment into the V-shaped pier.

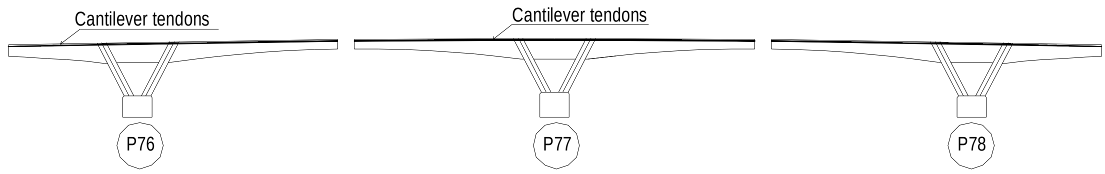
Step 3: Placing concrete cure at closure segment of the center span. The jacking equipment maintain until concrete reaches required strength.

Step 4: Releasing jacking force and then continuity cables are tensioned.

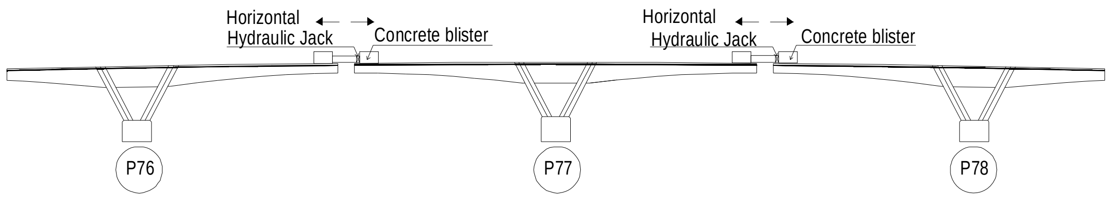
Pressurization by jacks in Step-2 is operated simultaneously at the both mid-spans and all hydraulic jacks are required to synchronize, control and monitor jacking force, jack ram and displacement of box girder.

Since the construction of closure segments in Step-3 is carried out in narrow and intricate space due to pressurization equipment, formwork, reinforcement and PC sheath of closure concrete are required to set in the position before pressurization equipment is provided.

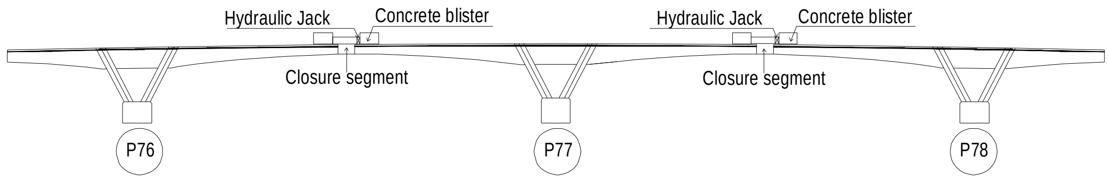
It is confirmed that continuity cables at the both mid-spans should be tensioned and then continuity cables in the both side-spans be tensioned later because deformation and moment due to creep and shrinkage are controlled properly.



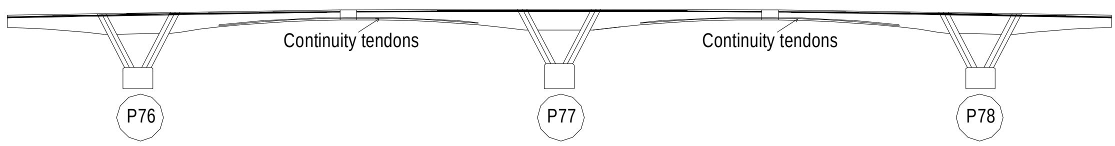
Step-1 Completion of Cantilever Erection Segments



Step-2 Outward Pressurization by Jacking Force



Step-3 Construction of Closure Segment



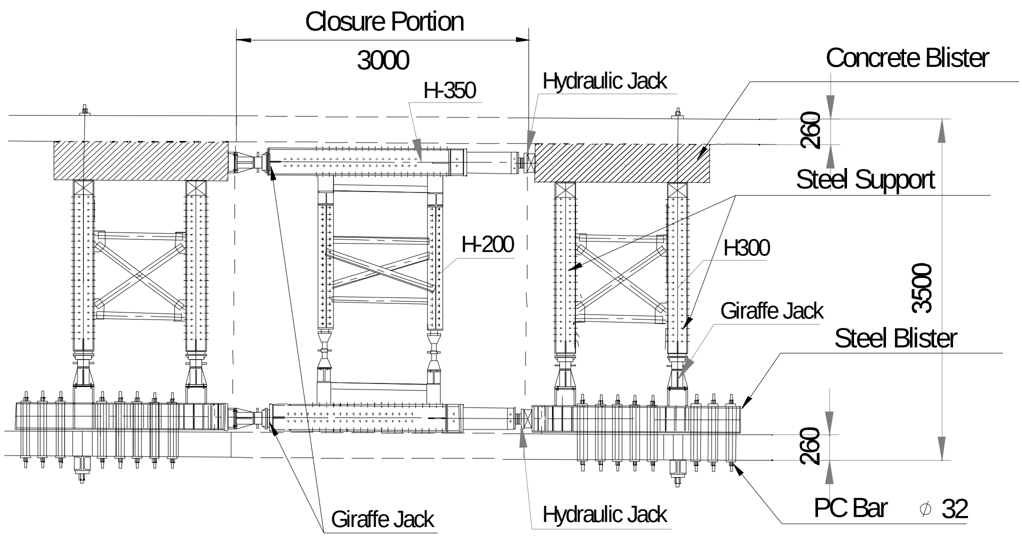
Step-4 Tension of Continuity PC Cable

Source: Study Team

Figure 8.3.4-14 Procedures of Pressurization Method by Jacking Force

3) Structure of Pressurization By Jacks

In order to reduce secondary moment of upper and lower slabs and webs due to pressurization, 4 blisters for jack base are provided at upper and lower slabs and pressure is carried out by 4 number of jacks simultaneously at two center spans as shown in the figure below. Total jacking force each jacking place is estimated at 2,000kN introduced into the box girder and its displacement is approximately 40mm which are approximately calculated on half as much as time dependent displacement due to creep and shrinkage over the entire life of bridge (100 years). Material properties of PC concrete are assumed that compressive strength of 28 days is 400Mpa, ultimate creep factor 2.35 and ultimate shrinkage strain 0.0008 respectively.



Source: Study Team

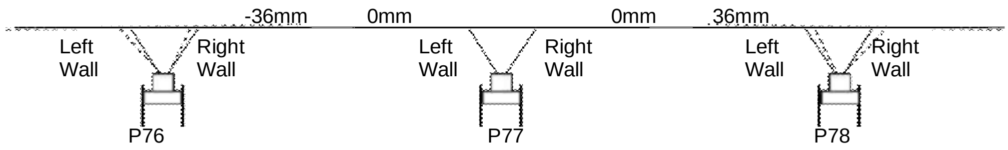
Figure 8.3.4-15 Structure of Pressurization by Jacks

(4) Effects of Pressurization on V-shaped Piers

To indicate the effects of the subsidiary measure, results from structural analysis, displacement, bending moment and stress, are introduced.

1) Displacement

Displacement due to the pressurization is shown in the figure below.



Source: Study Team

Figure 8.3.4-16 Longitudinal Displacement due to Pressurization

2) Bending Moment

Bending moment on V-shaped piers due to the pressurization is summarized in the table below.

Table 8.3.4-5 Summary of Bending Moment of Pier due to Pressurization

Pier No.	Wall	Position	Bending Moment (kN・m)	
			Creep and Shrinkage	Pressurization
P76	Left Wall	Top of Wall	-34,349	15,228
		Bottom of Wall	35,857	-29,014
	Right Wall	Top of Wall	2,542	-13,269
		Bottom of Wall	-15,214	27,850
P77	Left Wall	Top of Wall	-24,448	955
		Bottom of Wall	14,496	-194

Source: Study Team

3) Stress

Stress in V-shaped piers due to the pressurization is summarized in the table below.

Table 8.3.4-6 Summary of Stress of Pier due to Pressurization

Pier No.	Wall	Position	Tensile Stress (MPa)		
			Creep, Shrinkage (CR,SH)	Pressurization	DC+DW+PS+CR+SH
P76 (Left Pier)	Left Wall	Top of Wall	3.428	-1.649	7.037
		Bottom of Wall	4.933	-4.224	7.788
	Right Wall	Top of Wall	-0.143	1.456	5.895
		Bottom of Wall	-1.988	4.071	8.604
P77 (Center Pier)	Left Wall	Top of Wall	2.456	-0.096	3.227
		Bottom of Wall	1.972	-0.024	7.726

Source: Study Team

(5) Measurement during Loading Pressurization

The effectiveness of pressurization is verified by the following measurements;

1) Measurement of displacement at each pier top on the upper slab and space length between the tips of both cantilever box girders in pressurization section

2) Instrumentation and measurement of rotation at the end of pier during loading

3) Crack width at the bottom of V-shaped walls and the connection of V-shape walls and box girder

(6) Cost Estimate

The preliminarily estimated cost for one (1) set of pressurization work is summarized in the table below. It is noted that two (2) sets are necessary for the center spans.

Table 8.3.4-7 Summary of Preliminary Cost Estimate of Pressurization Work (One Set)

Items	Description	Unit: JPY
A. Direct Cost	H beam	150,220
	Fixing Materials	405,898
	Hydraulic Jacks	1,351,800
	Workers, Crane Operation and Miscellaneous	266,090
	Other Direct Cost	43,480
	Total Direct Cost	2,217,488
B. Common Cost	(5.5% x A)	121,962
C. Taxable Income	(6% x (A + B))	140,367
Pre-tax Cost	(A+B+C)	2,479,817

Source: Study Team

8.3.5 Study on Main Bridge Foundation

8.3.5.1 Selection of Foundation Type for Main Bridge

(1) Introduction

1) General

In Preparatory Survey of JICA, it is recommended to install steel pipe sheet pile (SPSP) foundation for Main Bridge which is located in deep water. In this section, a comparative study is presented for selection of type of foundation for Main Bridge. The study consists of 2 sub studies, 1) comparative study between steel pipe sheet pile foundation and cast in place pile foundation, 2) comparative study for steel pipe sheet pile foundation between Integrated foundation and separate foundation of main bridge. The point of this study is construction period and safety construction at the deep water need to focus on detail.

2) Site Condition

The site conditions indicate as following.

Table 8.3.5-1 Site Conditions for Study

Study Type	Type-3
Bridge Type	Main Br.
Station	Km +561.3 ~8+77.12
Pier No.	P76 ~ P78
Reclamation Plan	No
Bridge Span length (m)	150.0
Estimated Corrosion Thicknesof Steel Pile(mm)	7
Water depth (m)	7.0~11.0
E.L. of Pile Cap	E.L.-9.0 (Top of Pile Cap)

Source: Study Team

(2) Selection of Foundation Type for Main Bridge

1) General

In order to select appropriate types of foundation for main bridge, the comparative study was conducted.

2) Comparative Study

a) Foundation Type of Comparative

In this comparative study, the following three alternatives are studied.

Altenative-1: Steel pipe pile foundation (recommended by SAPROF Study)

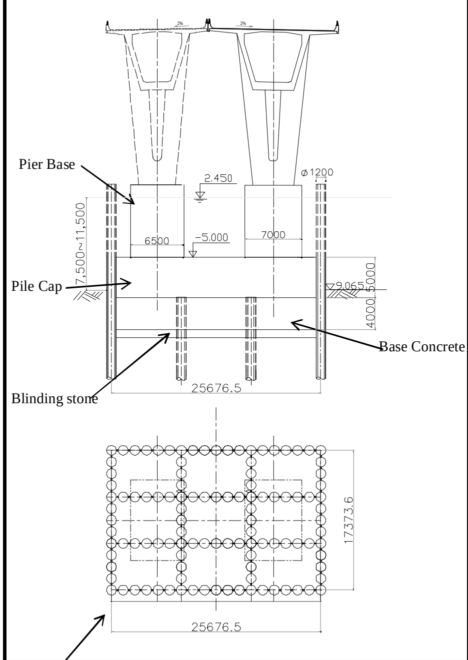
Altenative-2: Cast in place pile foundation

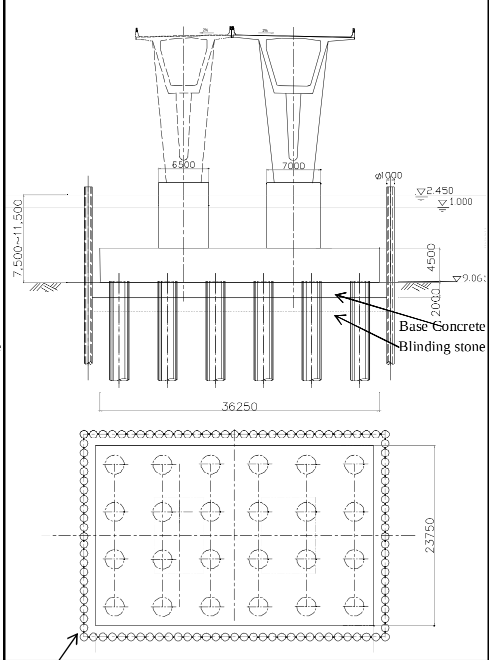
b) Result of Comparative Study

The result of comparative study is shown in the table 8.3.5-2 following Table. As the table indicates, Altenative-1, Steel pile sheet pile foundation, is most recommendable foundation type of main bridge. Since Altenative-1 is advantages in construction cost and workability in the sea.

Table 8.3.5-2 Comparison on Foundation for Main Bridge

<table>
<tr><th>Evaluation Items</th><th>Max. Point</th><th colspan="2">Alternative-1
SPSP Foundation Type</th><th colspan="2">Alternative-2
C.I.P. Pile Foundation with SPSP cofferdam Type</th></tr>
<tr><td>Side View</td><td></td><td colspan="2">SAPROF Study
Diameter of pile : 1200 mm
Total number of pile : 9
Total length of pile : 47.0 m

Future Construction Interim construction

Steel Pipe Sheet Pile (SPSP) Foundation (D=1.2m,t=16,SKY490)</td><td colspan="2">Diameter of pile : 2500 mm
Total number of pile : 24
Total length of pile : 50.0 m

Future Construction Interim construction

Steel Pipe Sheet Pile (SPSP) Foundation (D=1.0m,t=12,SKY490)</td></tr>
<tr><td>Structural Aspect and Stability</td><td>10</td><td>- Small number of Pile Cap concrete</td><td>8</td><td>- Large number of Pile Cap concrete and C.I.P. Piles due to Large statically indeterminate force of rigid-frame pier.
- Furthermore, the design of C.I.P. Foundation Separate type is inferior due to large statically indeterminate force and construction clearance</td><td>4</td></tr>
<tr><td>Construction Cost
(due to main items of SPSP Foundation)</td><td>40</td><td><table>
<tr><th colspan="2">Quantity
(for 1 foundation)</th><th>Unit Cost
(VND)</th><th>Total
(1,000VND)</th></tr>
<tr><td>Pile Cap + Pier Base Co</td><td>3,419m3</td><td>6,859,258</td><td>23,451,118</td></tr>
<tr><td>Steel Pipe Sheet Pile</td><td>2,507 t</td><td>44,003,858</td><td>110,318,244</td></tr>
<tr><td>Base Concrete (h=4m)</td><td>1,933m3</td><td>1,723,811</td><td>3,331,506</td></tr>
<tr><td>Blinding stone (h=1m)</td><td>483m3</td><td>737,139</td><td>356,156</td></tr>
<tr><td>Excavation</td><td>4,832m3</td><td>318,066</td><td>1,536,768</td></tr>
<tr><td>C.I.P. Pile</td><td>0m</td><td>49,217,400</td><td>0</td></tr>
<tr><td></td><td></td><td>Total</td><td>138,993,792</td></tr>
<tr><td></td><td></td><td>Ratio</td><td>1.000</td></tr>
</table></td><td>40</td><td><table>
<tr><th colspan="2">Quantity
(for 1 foundation)</th><th>Unit Cost
(VND)</th><th>Total
(1,000VND)</th></tr>
<tr><td>Pile Cap + Pier Base Co</td><td>5,035m3</td><td>6,859,258</td><td>34,537,866</td></tr>
<tr><td>Steel Pipe Sheet Pile</td><td>1,139 t</td><td>44,003,858</td><td>50,111,593</td></tr>
<tr><td>Base Concrete (h=2m)</td><td>1,970m3</td><td>1,723,811</td><td>3,395,692</td></tr>
<tr><td>Blinding stone (h=1m)</td><td>985m3</td><td>737,139</td><td>726,036</td></tr>
<tr><td>Excavation</td><td>2,955m3</td><td>318,066</td><td>939,825</td></tr>
<tr><td>C.I.P. Pile (D=2.5m)</td><td>1200m</td><td>49,217,400</td><td>59,060,880</td></tr>
<tr><td></td><td></td><td>Total</td><td>148,771,893</td></tr>
<tr><td></td><td></td><td>Ratio</td><td>1.070</td></tr>
</table></td><td>32</td></tr>
<tr><td>Construction Plan and Period</td><td>10</td><td>- Workability is superior with small number of foundation work.</td><td>4</td><td>- Workability is inferior due to large number of Cast in Place Pile work to rock.</td><td>4</td></tr>
<tr><td>Maintenance</td><td>15</td><td>- Inferior in Maintenance due to large column base of future construction on the sea.
- Superior in Maintenance with small number of pile cap in the sea.</td><td>10</td><td>- Inferior in Maintenance due to large column base of future construction on the sea.
- Inferior in Maintenance with rarge number of pile cap in the sea.</td><td>6</td></tr>
<tr><td>STEP Clearance</td><td>10</td><td>- 85% (Preliminary Estimate)
- Large number of steel pipe pile acceptance a contribution.</td><td>10</td><td>- 32% (Preliminary Estimate)
- Small number of steel pipe pile acceptance a contribution</td><td>8</td></tr>
<tr><td>Aesthetics</td><td>5</td><td>- Slender appearance of Pier
- Column base of future construction to be exposed above water level.</td><td>3</td><td>- Slender appearance of Pier
- Column base of future construction to be exposed above water level.</td><td>3</td></tr>
<tr><td>New Technology</td><td>5</td><td>- Steel Pipe Pile Foundation is new technology in Vietnam</td><td>5</td><td>- Steel Pipe Pile Foundation is new technology in Vietnam</td><td>2</td></tr>
<tr><td>Environmental Aspect</td><td>5</td><td>- Superior in Environmental aspect with small number of excavated soil.</td><td>5</td><td>- Inferior in Environmental aspect due to large number of excavated</td><td>3</td></tr>
<tr><td>Evaluation</td><td>100</td><td>- Construction cost is highest with long construction period.
- Workability is superior with small number of foundation work.

Most Recommended</td><td>85</td><td>- The design of C.I.P. Foundation Separate type is inferior due to large statically indeterminate force and construction clearance
- Workability is inferior due to large number of Cast in Place Pile work t

Less Recommended</td><td>62</td></tr>
</table>

*Not consider for scour

Source: Study Team (consideration in B/D)

(3) Selection of Foundation Style for Main Bridge

1) General

In order to select appropriate style of foundation for main bridge, the comparative study was conducted.

2) Comparative Study

a) Foundation Type of Comparative

In this comparative study, the following three alternatives are studied.

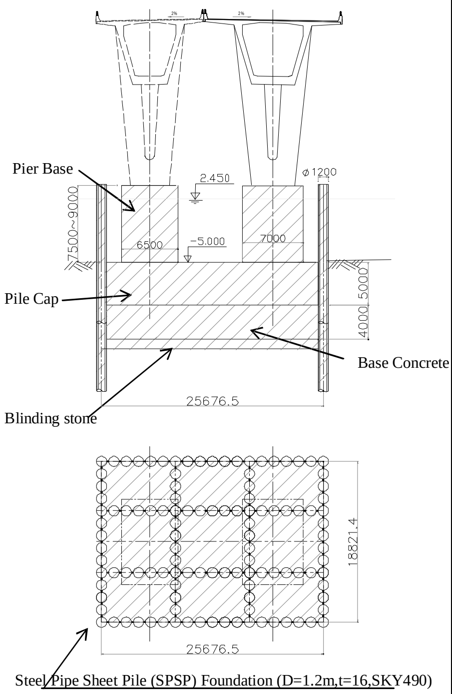
Altenative-1: Integrated Type of Steel pipe pile foundation
(recommended by SAPROF Study)
Altenative-2: Separate Type of Steel pipe pile foundation

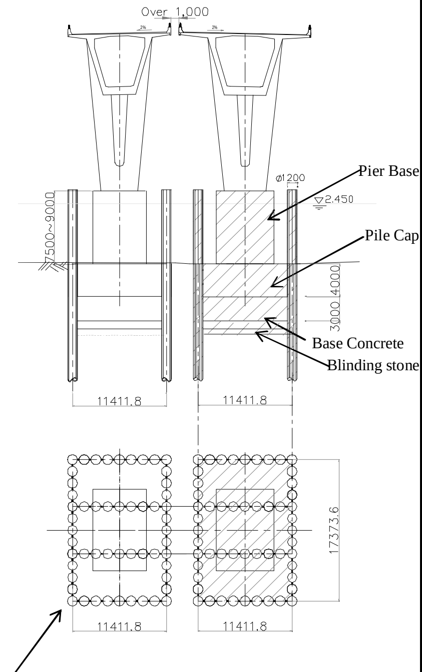
b) Result of Comparative Study

The result of comparative study is shown in following Table. As the table indicates, Altenative-2, Separate type of Steel pile sheet pile foundation, is most recommendable foundation type of main bridge. Since Altenative-2 is advantages in construction period and cost. Especially, in the Integrated foundation type could not have been in time construction period.

Table 8.3.5-3 Comparison on Foundation Style for Main Bridge

<table>
<tr><th>Evaluation Items</th><th>Max. Point</th><th colspan="2">Alternative-1
Integrated Foundation Type</th><th colspan="2">Alternative-2
Separate Foundation Type</th></tr>
<tr><td>Side View

▨ ：for Interim construction</td><td></td><td colspan="2">SAPROF Study


Steel Pipe Sheet Pile (SPSP) Foundation (D=1.2m,t=16,SKY490)</td><td colspan="2">Future Construction　Interim construction


Steel Pipe Sheet Pile (SPSP) Foundation (D=1.2m,t=19,SKY490)</td></tr>
<tr><td>Structural Aspect and Stability</td><td>10</td><td>- Eccentric load by pier dead load always act on foundation.
- Large number of Pile Cap concrete
- Large number of steel pipe sheet piles</td><td>6</td><td>- Consideration of construction clearance (over 1m) for future foundation construction is required.
- Small number of
- Small number of steel pipe sheet piles</td><td>8</td></tr>
<tr><td>Construction Cost (due to main items of SPSP Foundation)</td><td>40</td><td><table>
<tr><th colspan="2">Quantity (for 1 foundation)</th><th>Unit Cost (VND)</th><th>Total (1,000VND)</th></tr>
<tr><td>Pile Cap + Pier Base Co</td><td>3,419m3</td><td>6,859,258</td><td>23,451,118</td></tr>
<tr><td>Steel Pipe Sheet Pile</td><td>2,507 t</td><td>44,003,858</td><td>110,318,244</td></tr>
<tr><td>Base Concrete (h=4m</td><td>1,933m3</td><td>1,723,811</td><td>3,331,506</td></tr>
<tr><td>Blinding stone (h=1m</td><td>483m3</td><td>737,139</td><td>356,156</td></tr>
<tr><td>Excavation</td><td>4,832m3</td><td>318,066</td><td>1,536,768</td></tr>
<tr><td></td><td></td><td>Total</td><td>138,993,792</td></tr>
<tr><td></td><td></td><td>Ratio</td><td>1.000</td></tr>
</table></td><td>40</td><td><table>
<tr><th colspan="2">Quantity (for 1 foundation)</th><th>Unit Cost (VND)</th><th>Total (1,000VND)</th></tr>
<tr><td>Pile Cap + Pier Base Co</td><td>1,318m3</td><td>6,859,258</td><td>9,043,521</td></tr>
<tr><td>Steel Pipe Sheet Pile</td><td>1,373 t</td><td>44,003,858</td><td>60,431,818</td></tr>
<tr><td>Base Concrete (h=3m)</td><td>595m3</td><td>1,723,811</td><td>1,025,805</td></tr>
<tr><td>Blinding stone (h=1m)</td><td>198m3</td><td>737,139</td><td>146,219</td></tr>
<tr><td>Excavation</td><td>1,587m3</td><td>318,066</td><td>504,733</td></tr>
<tr><td></td><td></td><td>Total</td><td>72,318,790</td></tr>
<tr><td></td><td></td><td>Ratio</td><td>0.520</td></tr>
</table></td><td>32</td></tr>
<tr><td>Construction Plan and Period</td><td>10</td><td>- Workability is inferior due to large number of foundation work in the sea.</td><td>4</td><td>- Workability is superior with small number of foundation work.</td><td>8</td></tr>
<tr><td>Maintenance</td><td>15</td><td>- Inferior in Maintenance due to large column base of future construction on the sea.</td><td>6</td><td>- Superior in Maintenance with small number of Maintenance points.</td><td>8</td></tr>
<tr><td>STEP Clearance</td><td>10</td><td>- 85% (Preliminary Estimate)
- Large number of steel pipe pile acceptance a contribution.</td><td>10</td><td>- 32% (Preliminary Estimate)
- Small number of steel pipe pile acceptance a contribution</td><td>8</td></tr>
<tr><td>Aesthetics</td><td>5</td><td>- Slender appearance of Pier
- Column base of future construction to be exposed above water level.</td><td>3</td><td>- Slender appearance of Pier
- Column base of future construction to not be exposed above water level.</td><td>5</td></tr>
<tr><td>New Technology</td><td>5</td><td>- Steel Pipe Pile Foundation is new technology in Vietnam</td><td>5</td><td>- Steel Pipe Pile Foundation is new technology in Vietnam</td><td>5</td></tr>
<tr><td>Environmental Aspect</td><td>5</td><td>- Inferior in Environmental aspect due to large number of excavated soil.</td><td>3</td><td>- Superior in Environmental aspect with small number of</td><td>5</td></tr>
<tr><td>Evaluation</td><td>100</td><td>- Construction cost is highest with long construction period.
- Eccentric load by pier dead load always act on foundation.
- Inferior in Aesthetics due to column base of future construction to be exposed above water level.

Less Recommended</td><td>77</td><td>- Minimum Construction cost with construction period.
- Superior in Aesthetics due to column base of future construction to not be exposed above water level.

Most Recommended</td><td>79</td></tr>
</table>

*Not consider for scour

Source: Study Team (consideration in B/D)

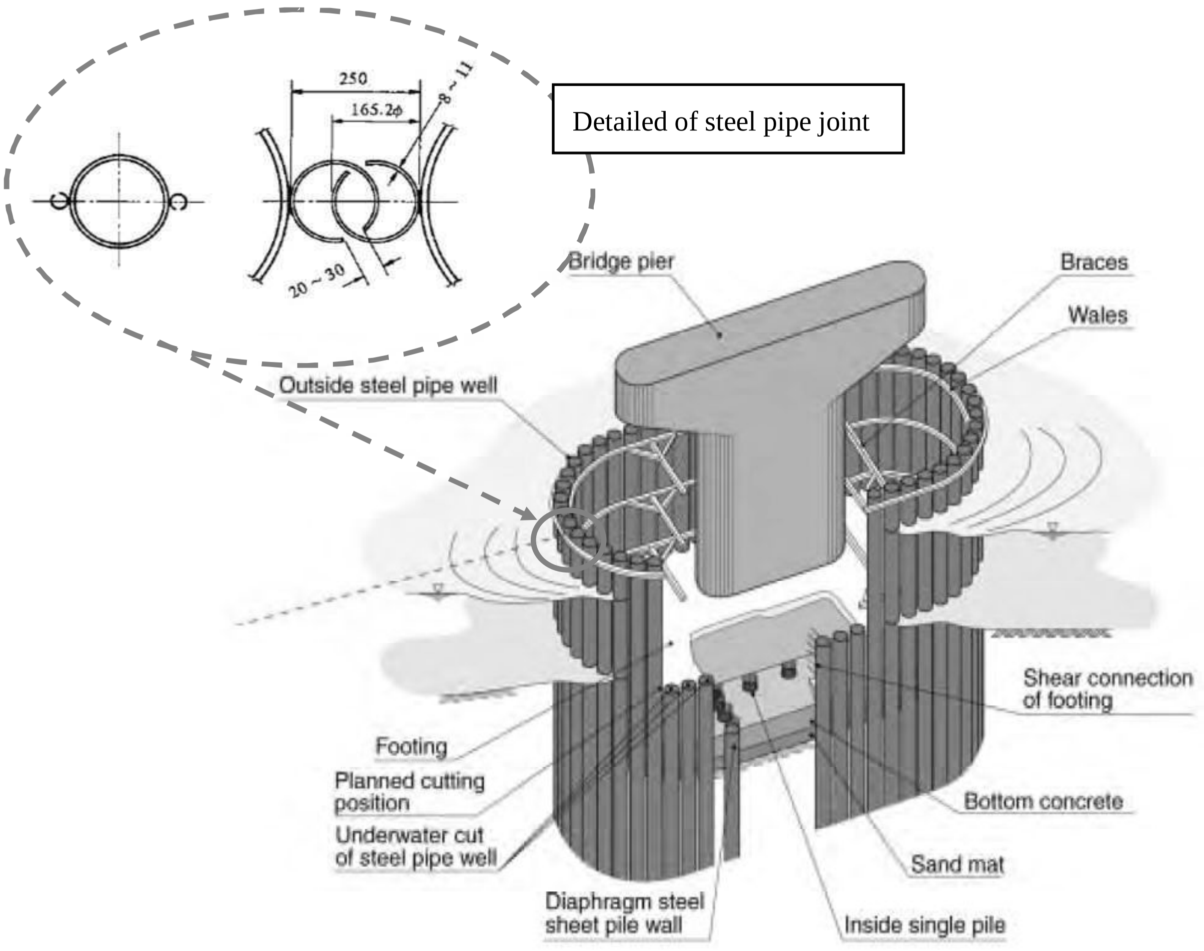
8.3.5.2 Study on Design Condition for Steel Pipe Sheet Pile

(1) General

A Steel pipe sheet pile consists of a steel pipe pile as the main component member, to with the joints illustrated in detail of following figure are attached. Compared to an ordinary steel sheet pile, it has an advantage of considerable rigidity; witch lends itself highly useful for wall structures such as earth retaining walls for deep excavation and deep water foundations.

Steel pipe sheet pile foundation is consisted of outside steel pipe sheet pile well and diaphragm steel pile sheet pile well. Open end steel pipe sheet piles are driven to the designated depth, loads from superstructure are transmitted to the upper slab and then to the sheet piles and finally to soil by friction and tip bearing.

Steel pipe sheet pile foundation lets the Outside sheet pipe well itself get up over the water surface, its joints being filled with cut off materials to serve as temporary cofferdam with temporary braces and wales. The inside of well is dried, and after a pile cap and a pier are erected there, the pipe pile temporary cofferdam planning cutting passion around above the top end of the footing is underwater cut and removed.

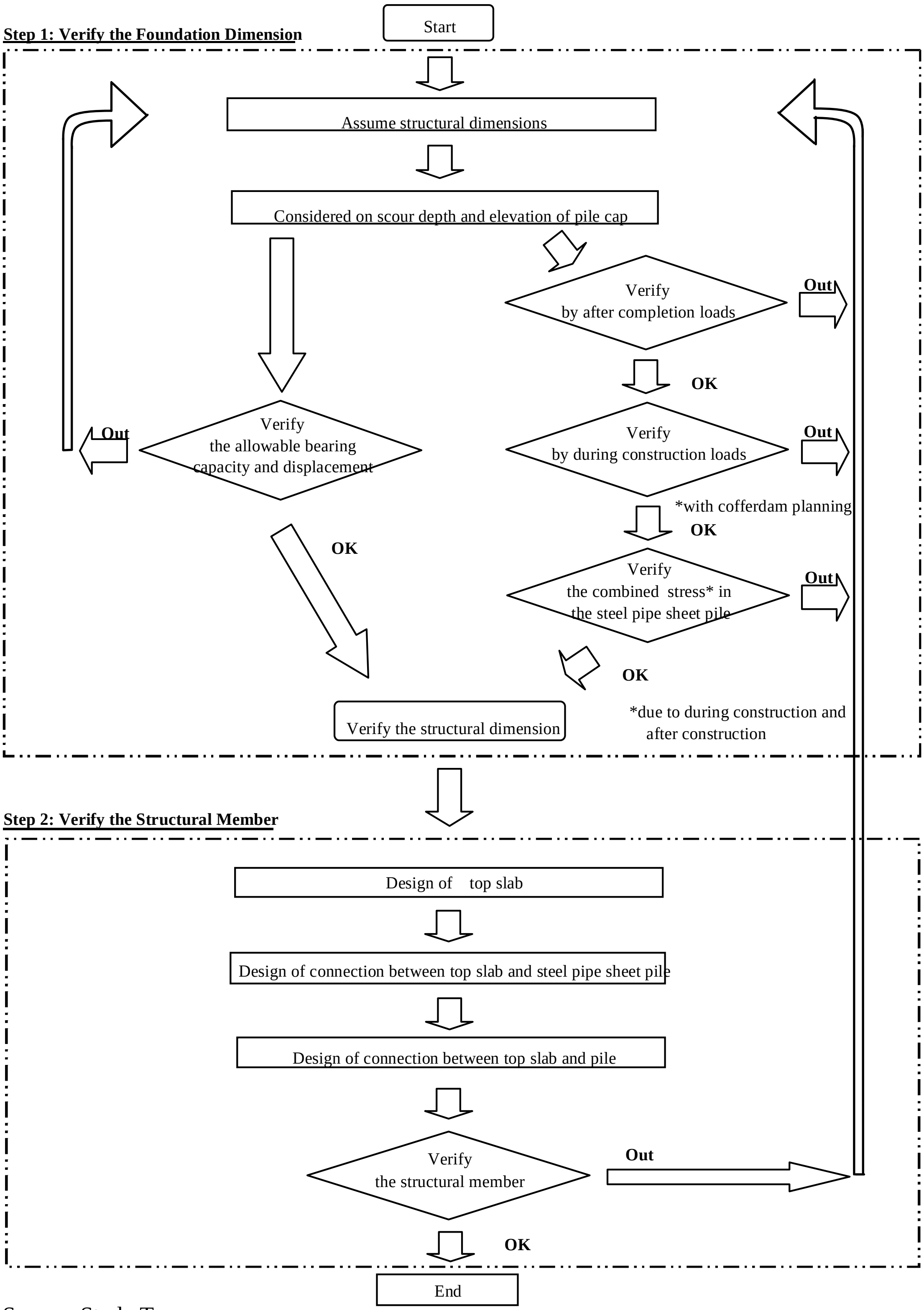


Source: Study Team

Figure 8.3.5-1 Conceptual View of Steel Sheet Pile Foundation

(2) Design Flow

Detail Design of steel pipe sheet pile foundation based on following flow.



Source: Study Team

Figure 8.3.5-2 Design Flow for basic design of steel pipe sheet pile foundation

(3) Construction Step

The procedure for construction method of steel pipe sheet pile foundations and points of construction at each stage are shown as below.

1) Steel pipe sheet pile setting and driving

In setting and driving steel pipe sheet piles, to prevent the rotating and tilting of pipe pile, guide frames are installed inside and outside the circumference of the well are attached. Pile setting and driving work is performed by the pile driver on the boat or on the scaffold at site. Piles are set, one by one, by the Vibro hammer, at positions determined by the guides.

2) Joint work and bottom slab concrete casting

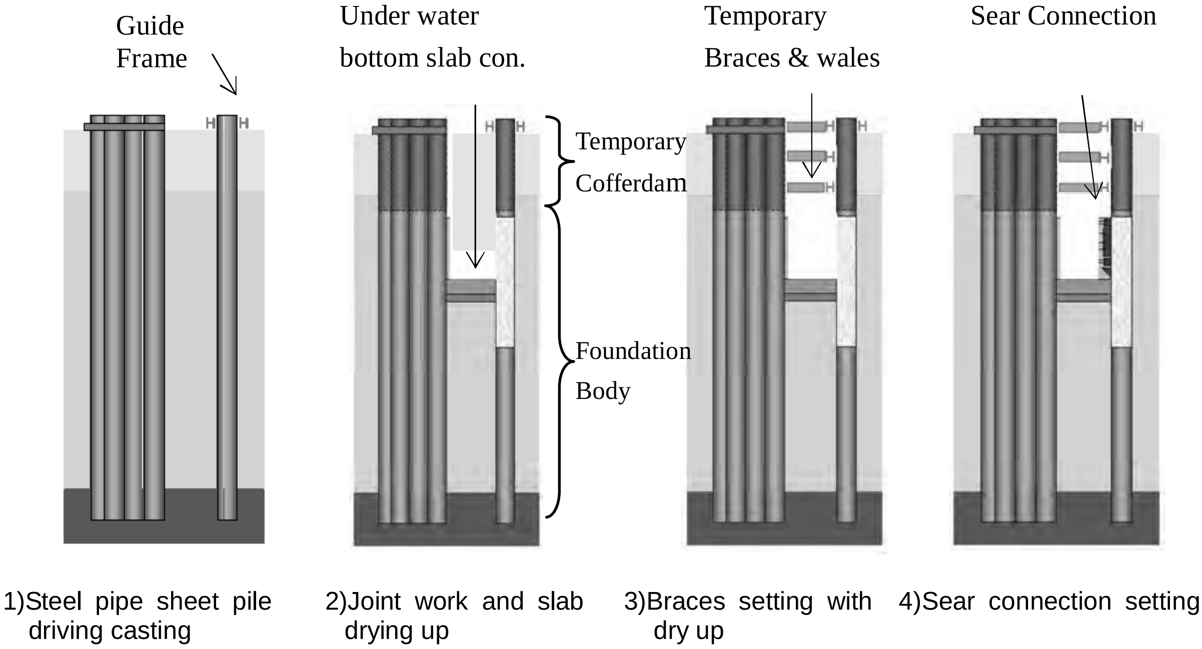
When the pipe pile driving is completed to the designed depth, earth and sand in the joint are removed and filling mortar into the joints to below the pile cap. Next comes pouring of cut off material into the joint of temporary cofferdam section by Nylon bags. Then the internal excavation to the prescribed depth by a clamshell and water pump is performed. Upon completion of the excavation, the ground surface is evened with sand gravel, and under water slab concrete is cast.

3) Braces setting with drying up

After curing of underwater bottom slab concrete, drying up of the inside of the temporary cofferdam is started. The water level is lowered to the lower level of the stage braces and a wales is set up, one by one.

4) Shear connection setting

In order to make the pile cap and the internal wall of steel pipe sheet pile foundation in one body, a shear connection is welded to the pipe pile.



Source: Study Team

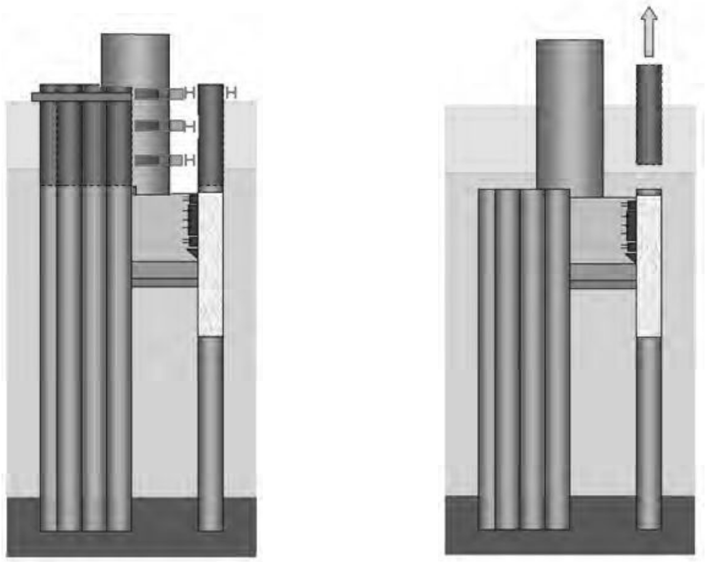
Figure 8.3.5-3 The procedure for construction method of steel pipe sheet pile foundations (1)

5) Pile cap and column construction

The arrangement on pile cap reinforcement as well as concrete casting follows. Then, a column is erected.

6) Temporary Braces & wales removal and underwater cutting of sheet pile

While water is poured inside the temporary cofferdam, braces and wales are removed, one by one. After external and internal pressure are balanced, steel pipe sheet pile are cut under the water at the top of the pile cap.



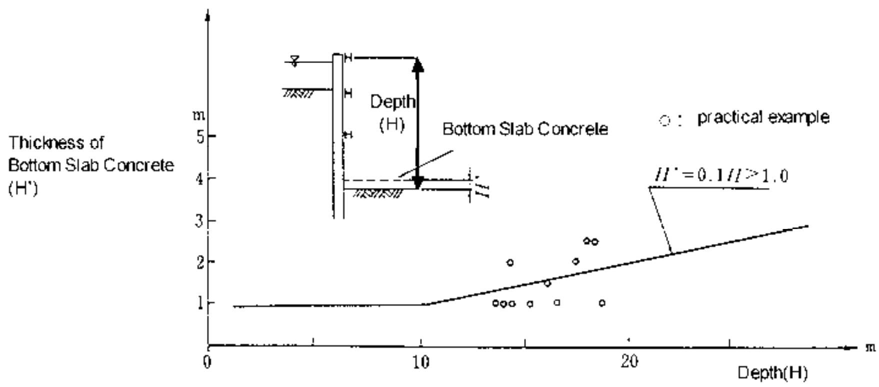
5) Pile cap and column construction

6) Temporary Braces & wales removal and underwater cutting of sheet pile

Source: Study Team

Figure 8.3.5-4 The procedure for construction method of steel pipe sheet pile foundations (2)

Thickness of Bottom Slab concrete is determined by the following figure.



Source: Study Team (refer from "SPSP Foundation", Japanese Association for Steel Pipe Piles, 2002)

Figure 8.3.5-5 The procedure for construction method of steel pipe sheet pile foundation

8.3.5.3 Design Considerations

(1) Design Principal

1) Design Specifications

Basically, the bridges and structures in this project shall be designed with the Vietnamese Design Standard (22 TCN 272-05) and AASHTO-LRFD (Load and Resistance Factor Design, 3rd Edition 2004). However, the design of steel pipe sheet pile not fit for these standards, therefore it design shall be determined referring to Japanese Standard of “Specifications for Highway Bridges -Part IV” (SHB-2002).

2) Design soil condition

<u>Boring Data</u>

A total of 6 boreholes, two boreholes for each main bridge foundation were performed. The soil is consisted clay and silt, layers up to approximately top 30 m are clay and N-SPT values are low. Soil constants and the elevation of bearing stratum for foundation design of main bridge are decided by comparing the 2 boring logs of each foundation, the smaller N-SPT and deeper elevation were applied as follows.

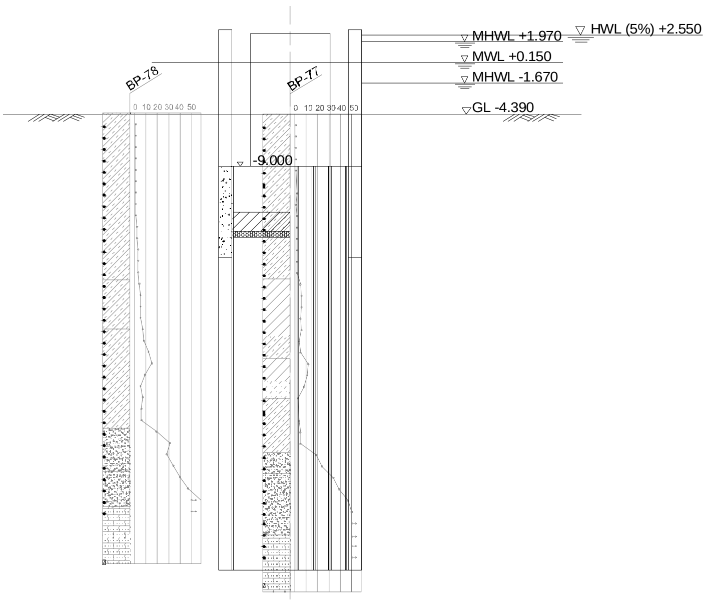
Table 8.3.5-4 Determined design boring No.

Pier No	Boring No	Boring Location
P76	BP-77	center of foundation
P77	BP-80	14m from center of foundation
P78	BP-81	center of foundation

Source: Study Team

<u>Liquefaction</u>

Liquefaction potential at earthquake is excluded in foundation design of main bridge due to there is no saturated soil layer having ground water level higher than 10m below the ground surface and located at a depth less than 20 m below the ground surface.



Source: Study Team

Figure 8.3.5-6 P76 Foundation with Boring No BP- 77 & BP-78

3) Loading Combination and Safety factors

The design cases and corresponding safety factor for stability and allowable stress for members is shown in the following table.

Table 8.3.5-5 Safety Factor for Bearing Capacity and Allowable Stress in Steel Pipe

Loading Combinations	Safety factor (*n*) for stability	Increase of allowable stress
1.Ordinary Condition : (DC+DW)+EV+CR+SH+EL+LL+WA	3.0	1.00
2.Temperature Condition : (DC+DW)+EV+CR+SH+EL+LL+WA + TG+TU	3.0	1.15
3.Wind Condition: (DC+DW)+EV+CR+SH+EL+LL +WA +WS	2.0	1.25
4.Seismic Condition: (DC+DW)+EV+CR+SH+EL+WA' +EQ	2.0	1.50
5.Vessel Collision force (DC+DW)+EV+CR+SH+EL+LL+WA' + CV	2.0	1.70

DC	=	Dead load of structural components and non-structural attachments
DW	=	Dead load of wearing surfaces and utilities
EV	=	Vertical pressure from dead load of earth fill
CR	=	Creep
SH	=	Shrinkage
EL	=	Accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning
LL	=	Vehicular live load
TG	=	Temperature gradient
TU	=	Uniform temperature
WA,WA'	=	Water load (WA': due to MWL)
WS	=	Wind load on structure
EQ	=	Earthquake, Includes effect of liquefaction due to earthquake
CV	=	Vessel collision force

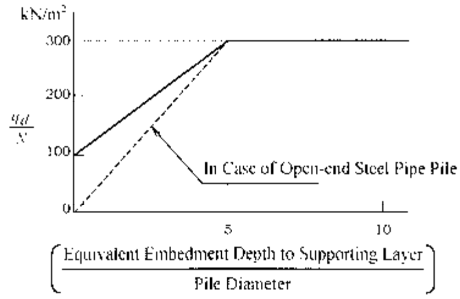
Source: Article 4 of Part IV, SHB-2002 & Article 4 of 22 TCN 272-05

4) Vertical Bearing Capacity

Vertical bearing capacity(Ra) and safety factor(n, refer to table8.3.5-5) of Steel Pipe Sheet Pile foundation shall be calculated as follows.

$$R_a = \frac{1}{n} R_u$$

$$R_u = q_d \cdot A_1 + \frac{1}{n_1 + n_2 + n_3} \left\{ U_1 \sum L_i f_i + U_2 \sum L_j f_j \right\}$$



where

A_1 :Tip closed section of sheet pile (m^2)

q_d :Tip resistance per unit area (kN/m^2)

$$q_d / N = 300 \qquad \overline{N = 40}$$

n_1 :number of sheet piles in exterior wall

n_2 :number of sheet piles in bulkhead

U_1 :circumference envelop length of exterior wall (m)

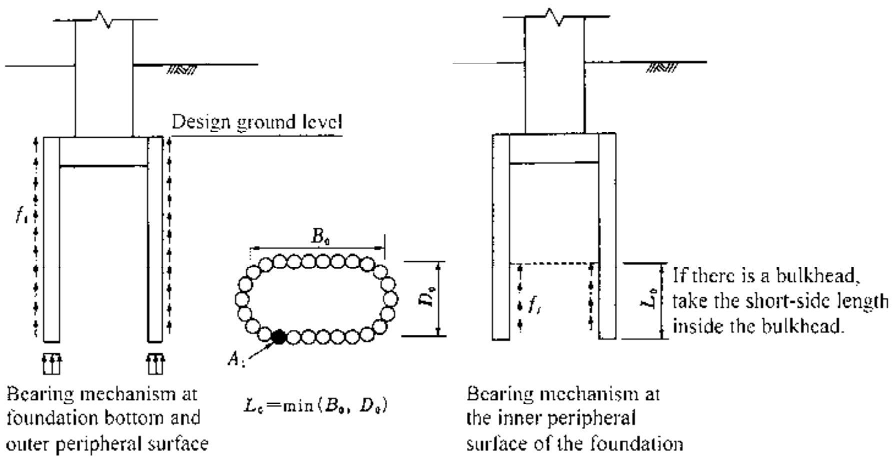
U_2 :circumference envelop length of interior wall and bulkhead (m)

L_i :length of each layer considering side friction for exterior wall (m)

f_i :maximum friction coefficient for exterior wall (kN/m^2)

L_j :length of each layer considering side friction for exterior wall (m)

f_j :maximum friction coefficient for interior wall (kN/m^2)



Source: Article 13 of Prt IV, SHB-2002

Figure 8.3.5-7 Region where the skin friction force at the inter peripheral surface of the well portion of the foundation should be taken into account

5) Material

Steel Pipe Sheet Pile

Two types of steel pipe of steel pipe sheet pile Grade SKY400 and Grade SKY 490 based on the Japanese Standard JIS 5530 or equivalent international standard shall be used. The properties and strength are as follows.

Table 8.3.5-6 Properties and Stress Limit of Steel Pipe for Steel Pipe Sheet Pile

Type	Yield Strength f y (MPa)	Tensile Strength f u (MPa)	Modulus of Elasticity (Mpa)
Grade SKY 400	235	400	200,000
Grade SKY 490	315	490	200,000

Source: JIS 5530

Estimated Corrosion Thickness of Steel Pile

The estimated corrosion thickness of steel pipe pile and steel pipe sheet pile based on the Report No PMU2/110422-1 shall be used. The design of corrosion thickness is as follows.

Table 8.3.5-7 Design of Estimated Corrosion Thicknesses

	Cast against earth	Direct exposure to selt water
Estimated Corrosion Thickness	2mm	7mm

Source: Study Team

(2) Design Model

1) General

The steel pipe sheet pile foundation has a very wide range of βLe, which indicates the applicable scope of the design method, and generally belongs to elastic foundations of finite length. Judging from βLe, some are regarded as an elastic foundation with a value less than 1, however, the steel pipe sheet pile foundation is a structure consisting of steel pipe sheet pile mutually joined by joint pipes of smaller rigidity than the steel pipe body and with mortar filled in the joint pipes, and a shear slippage deformation easily occurs in it. Therefore, verification of the slippage at the foundation bottom may be omitted. That is, stability should be verified on vertical bearing capacity and horizontal displacement.

An outline of the stability calculation model used in verification for ordinary, storm and seismic condition is shown in Table 8.3.5-8.

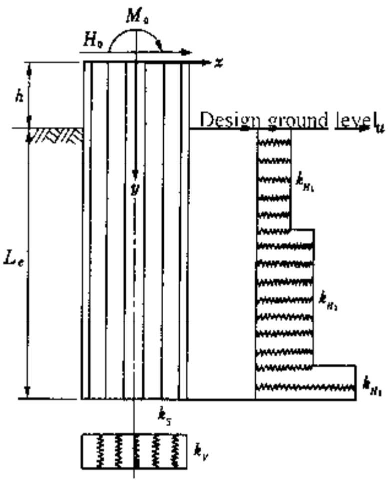
Table 8.3.5-8 Stability Calculation model

		Verification for ordinary conditions, storm and Level 1 earthquake conditions: B≦30m , L/B＞1 and βL_e＞1	Verification for ordinary conditions, storm and Level 1 earthquake conditions: B＞30m , L/B≦1 or βL_e≦1
Design model		Finite-length beam on an elastic floor (Beam Model)	Analysis by an imaginary well beam that considering shear slippage of the joints (Well Model)
Foundation body	Steel pipe sheet pile	Linear	Linear
Foundation body	Shear resistance of joint	Evaluation by composite efficiency and moment distribution factor	Bilinear
Ground hesitance element	Horizontal ground resistance at the foundation front face	Linear considering strain dependency	Linear considering strain dependency
Ground hesitance element	Horizontal shear ground resistance at the foundation peripheral faces	Included in the horizontal resistance of the front ground	Included in the horizontal resistance of the front ground
Ground hesitance element	Vertical shear ground resistance at the foundation outer and inner peripheral faces	Included in the bearing capacity of the steel pipe sheet pile	Included in the bearing capacity of the steel pipe sheet pile
Ground hesitance element	Vertical ground resistance at the foundation bottom face	Linear	Linear
Ground hesitance element	Horizontal shear ground resistance at the foundation bottom faces	Linear	Linear

Source: Article 13 of Prt IV, SHB-2002

2) Finite-length beam Model

The sectional forces, displacement and unit ground reaction force of a well-type steel pipe sheet pile foundation may be derived by regarding the steel pipe sheet pile foundation as a finite-length beam on an elastic model, as shown in following model.



Source: Article 13 of Prt IV, SHB-2002

Figure 8.3.5-8 Calculation Model of Steel Pipe Sheet Pile Foundation

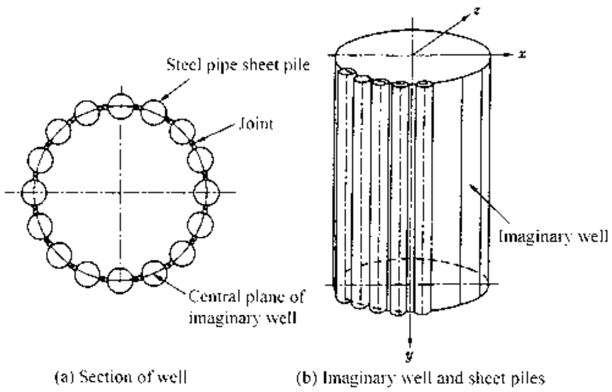
3) Imaginary well Method

This analysis method is based on a beam theory that considers shear deformation. That is, an imaginary well is assumed by tying up steel pipe sheet piles at their central axes as shown in Figure 8.3.5-8. Assuming that the central axis of each steel pipe sheet pile is fixed to the central plane of the imaginary well wall, establish the following assumptions in order to render the well into one dimension as a beam theory.

a) The imaginary well obeys a beam theory that considers shear deformation.

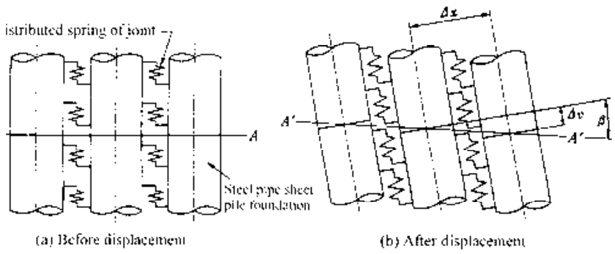
b) Each individual deformation steel pipe sheet pile obeys a beam theory that neglects shear deformation.

Assumption a. assumes the law of plain conservation for the imaginary well. As shown in Figure 8.3.5-9, this law asserts that a section AA in the imaginary well that was horizontal before displacement keeps to be flat after displacement and displaces to a section A'A'.



Source: Article 13 of Art IV, JRA-2002

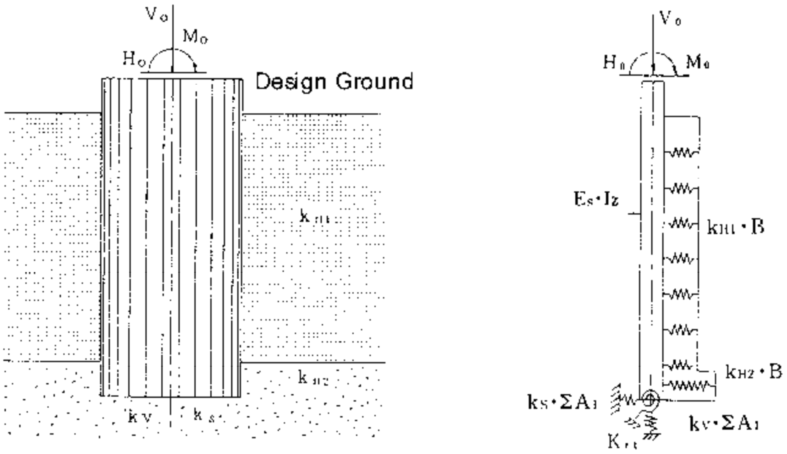
Figure 8.3.5-10 Steel pile sheet pile Foundation



Source: Article 13 of Art IV, JRA-2002

Figure 8.3.5-9 Slippage displacement δv of Joint Accompanying Shear Deformation β of Imaginary Well

The sectional forces, displacement and unit ground reaction force of a well-type steel pipe sheet pile foundation may be derived by regarding the steel pipe sheet pile foundation as a imaginary well method, as shown in following model.



Source: Study Team (refer from "SPSP Foundation", Japanese Association for Steel Pipe Piles, 2002)

Figure 8.3.5-11 Calculation Model of Imaginary well

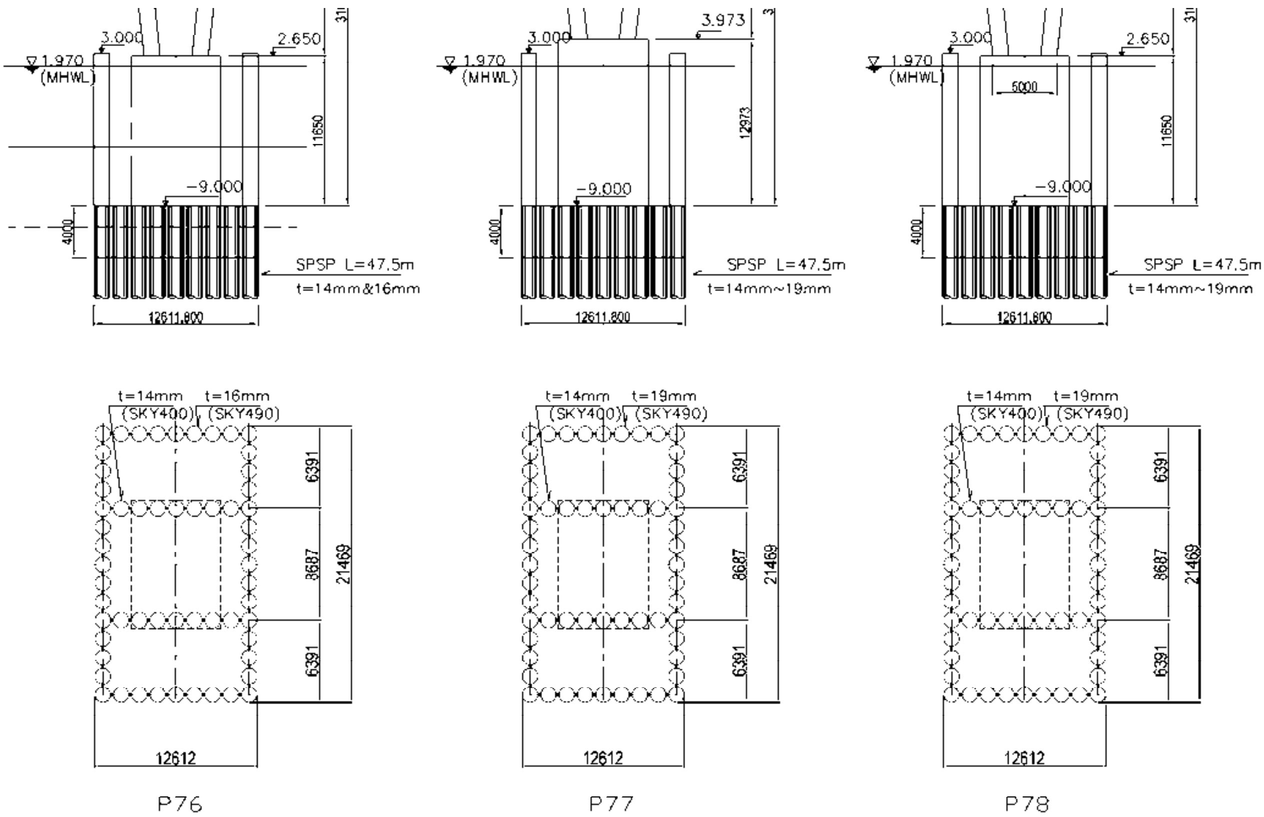
4) Determined of Design Model

Design model for Steel Pile Sheet Pile foundation design of main bridge are decided by table 8.3.5-9 as follows.

Table 8.3.5-9 Determined design model

			D(m)	L(m)	L/D	β(m^{-1})	Le(m)	βLe	Design Model*	
P76	Normal	LL	21.469 <30m	35.50	1.6535 >1	0.0340	33.69	1.145 >1	Beam	Beam
		TT	12.782 <30m	35.50	2.7773 >1	0.0394	33.69	1.327 >1	Beam	
	Seismic	LL	21.469 <30m	35.50	1.6535 >1	0.0328	36.81	1.207 >1	Beam	Beam
		TT	12.782 <30m	35.50	2.7773 >1	0.0378	36.81	1.391 >1	Beam	
P77	Normal	LL	21.469 <30m	35.50	1.6535 >1	0.0345	30.19	1.042 >1	Beam	Beam
		TT	12.782 <30m	35.50	2.7773 >1	0.0429	30.19	1.295 >1	Beam	
	Seismic	LL	21.469 <30m	35.50	1.6535 >1	0.0324	33.92	1.099 >1	Beam	Beam
		TT	12.782 <30m	35.50	2.7773 >1	0.0392	33.92	1.330 >1	Beam	
P78	Normal	LL	21.469 <30m	35.50	1.6535 >1	0.0426	23.49	**1.001≒1**	**Well**	**Well**
		TT	12.782 <30m	35.50	2.7773 >1	0.0499	23.49	1.172 >1	Beam	
	Seismic	LL	21.469 <30m	35.50	1.6535 >1	0.0354	29.09	1.030 >1	Beam	Beam
		TT	12.782 <30m	35.50	2.7773 >1	0.0443	29.09	1.289 >1	Beam	

Note) Beam : Finite-length beam on an elastic floor (Beam Model)
Well : Imaginary well beam that considering shear slippage of the joints (Well model)



Source: Study Team

Figure 8.3.5-12 Shape of SPSP Foundation

(3) Spring Model for Global Analysis

1) Lateral soil reaction coefficient

$$k_H = \frac{1}{0.3}\alpha E_o\left(\frac{B_H}{0.3}\right)^{-3/4}$$

where

kH : lateral soil reaction coefficient (kN/m3)

BH : reduced loading width orthogonal to loading direction (m)

$$\beta = \sqrt[4]{\frac{k_H \cdot D}{4EI}}$$

D:loading width orthogonal to loading direction (m)

De:effective width orthogonal to loading direction

1/β:soil depth relating to lateral resistance, and is less than foundation length (m)

β:characteristic value of foundation (1/m)

E:Young's modulus of foundation = 2.00E8 (kN/m2)

I:moment of inertia of foundation (m4)

Le:effective embedment depth of foundation (m)

$$k_{H1} = (1+\alpha_H)\cdot k_H \cdot \left(\frac{y}{y_o}\right)^{-1/2}$$

where

kH1 : basis of lateral reaction coefficient considering strain dependency (kN/m3)

αH : Increment factor considering contribution from lateral shear reaction
and resistance of soil inside well (=1.00)

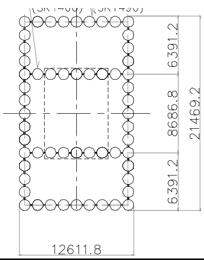
y : horizontal displacement of foundation at design stratum (m)

yo : basis of displacement (m)

2) Spring Model of Steel Pipe Sheet Pile Foundation

Spring Model of Steel Pipe Sheet Pile foundation for Global Analysis are shown as follows.

P76



Stiffless of SPSP Fouldatiol

− Stiffless

E = 2.0E+08 kI/m2 A = 3.3487 m^2

− Geometrical Momelt of Ilertia (t=14&16)

Lolgitudilal Directiol = 127.498 (m^4) B_{LL}= 12.611 m

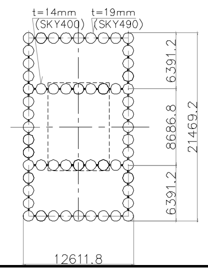
Tralsverse Directiol = 55.329 (m^4) B_{TT}= 21.469 m

Horizoltal Sprilg (BP−77)

Layer	Iormal					Seismic				
	Depth (m)	Lolgitudilal Directiol (LL)		Tralsverse Directiol (TT)		Depth (m)	Lolgitudilal Directiol (LL)		Tralsverse Directiol (TT)	
		kh1 (kI/m3)	Sprilg(kI/m)	kh1 (kI/m3)	Sprilg(kI/m)		kh1 (kI/m3)	Sprilg(kI/m)	kh1 (kI/m3)	Sprilg(kI/m)
overhaIg	0					0				
1	9.89	1,542	33,105	1,354	0	9.89	3,084	38,892	2,707	58,117
2	7.00	3,855	82,763	3,384	0	7.00	7,709	97,218	6,769	145,324
3	3.50	6,939	148,973	6,092	0	3.50	13,877	175,003	12,183	261,557
4	4.80	3,855	82,763	3,384	0	4.80	7,709	97,218	6,769	145,324
5	1.80	17,732	380,688	15,568	0	1.80	35,464	447,237	31,135	668,437
6	5.40	34,693	744,824	60,917	0	5.40	69,385	875,014	60,917	1,307,827
7	3.11	38,547	827,566	67,666	1,452,721	3.11	77,095	972,245	67,686	853,588
Total	35.50					35.50				

Pile Tip Sprilg		Iormal		Seismic		
Vertical Sprilg	=	1.0823E+07 kI/m		2.1646E+07 kI/m		
Horizoltal Sprilg	=	3.2469E+06 kI/m		6.4937E+06 kI/m		
RotatioI Sprilg	=	5.3664E+08 kIm/rad	(LL)	1.0733E+09 kIm/rad	(LL)	(for Lolgitudilal Directiol)
		2.3155E+08 kIm/rad	(TT)	4.6311E+08 kIm/rad	(TT)	(for Tralsverse Directiol)

P77



Stiffless of SPSP Fouldatiol

− Stiffless

E = 2.0E+08 kI/m2 A = 3.8318 m^2

− Geometrical Momelt of Ilertia (t=14&19)

Lolgitudilal Directiol = 149.088 (m^4) B_{LL}= 12.611 m

Tralsverse Directiol = 64.663 (m^4) B_{TT}= 21.469 m

Horizoltal Sprilg (BP−80)

Layer	Iormal					Seismic				
	Depth (m)	Lolgitudilal Directiol (LL)		Tralsverse Directiol (TT)		Depth (m)	Lolgitudilal Directiol (LL)		Tralsverse Directiol (TT)	
		kh1 (kI/m3)	Sprilg(kI/m)	kh1 (kI/m3)	Sprilg(kI/m)		kh1 (kI/m3)	Sprilg(kI/m)	kh1 (kI/m3)	Sprilg(kI/m)
overhaIg	0					1.58				
1	7.92	722	9,105	684	14,685	6.34	1,557	19,635	712	15,286
2	3.90	4,629	58,376	4,102	88,066	3.90	9,341	117,799	4,271	91,694
3	9.00	3,086	38,918	2,735	58,718	9.00	6,227	78,529	2,847	61,122
4	4.00	16,974	214,059	15,042	322,937	4.00	34,250	431,927	15,661	336,226
5	8.00	33,947	428,106	30,083	645,852	8.00	68,499	863,841	31,321	672,431
6	2.68	38,577	486,495	34,185	733,918	2.68	77,840	981,640	35,592	764,125
Total	35.50					35.50				

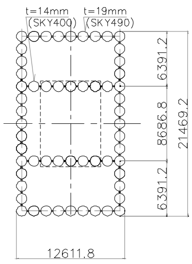
Pile Tip Sprilg		Iormal	Seismic		
Vertical Sprilg	=	1.0823E+07 kI/m	2.1646E+07 kI/m		
Horizoltal Sprilg	=	3.2469E+06 kI/m	6.4937E+06 kI/m		
RotatioI Sprilg	=	5.3664E+08 kIm/rad	1.0733E+09 kIm/rad	(LL)	(for Lolgitudilal Directiol)
		2.3155E+08 kIm/rad	4.6311E+08 kIm/rad	(TT)	(for Tralsverse Directiol)

P78

Stiffless of SPSP Fouldatiol

– Stiffless

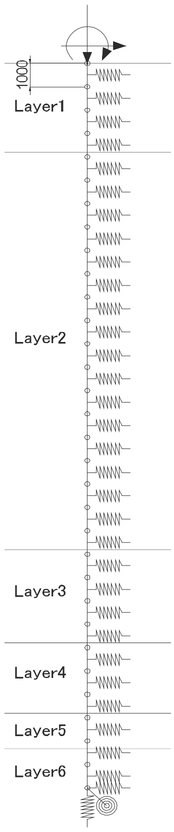
E =	2.0E+08 kI/m2	A =	3.8318 m^2
– Geometrical Momelt of Ilertia			(t=14&19)
Lolgitudilal Directiol =	149.088 (m^4)	B_{LL}=	12.611 m
Tralsverse Directiol =	64.663 (m^4)	B_{TT}=	21.469 m



Horizoltal Sprilg (BP–81)

Layer	Iormal					Seismic				
	Depth (m)	Lolgitudilal Directiol (LL)		Tralsverse Directiol (TT)		Depth (m)	Lolgitudilal Directiol (LL)		Tralsverse Directiol (TT)	
		kh1 (kI/m3)	Sprilg(kI/m)	kh1 (kI/m3)	Sprilg(kI/m)		kh1 (kI/m3)	Sprilg(kI/m)	kh1 (kI/m3)	Sprilg(kI/m)
overhaIg	0					6.41				
1	2.35	0	0	0	0	0	0	0	0	0
2	5.70	784	9,887	698	14,985	1.64	1,645	20,745	1,440	30,915
3	3.80	2,351	29,648	2,095	44,978	3.80	4,934	62,223	4,320	92,746
4	10.10	4,701	59,284	4,189	89,934	10.10	9,867	124,433	8,639	185,471
5	3.10	23,506	296,434	20,947	449,711	3.10	49,337	622,189	43,197	927,396
6	8.10	36,043	454,538	32,118	689,541	8.10	75,651	954,035	66,236	1,422,021
7	2.35	39,177	494,061	34,911	749,504	2.35	82,229	1,036,990	71,995	907,929
Total	35.50					35.50				

Pile Tip Sprilg		Iormal	Seismic		
Vertical Sprilg	=	1.0823E+07 kI/m	2.1646E+07 kI/m		
Horizoltal Sprilg	=	3.2469E+06 kI/m	6.4937E+06 kI/m		
Rotatiol Sprilg	=	5.3664E+08 kIm/rad	1.0733E+09 kIm/rad	(LL)	(for Lolgitudilal Directiol)
		2.3155E+08 kIm/rad	4.6311E+08 kIm/rad	(TT)	(for Tralsverse Directiol)



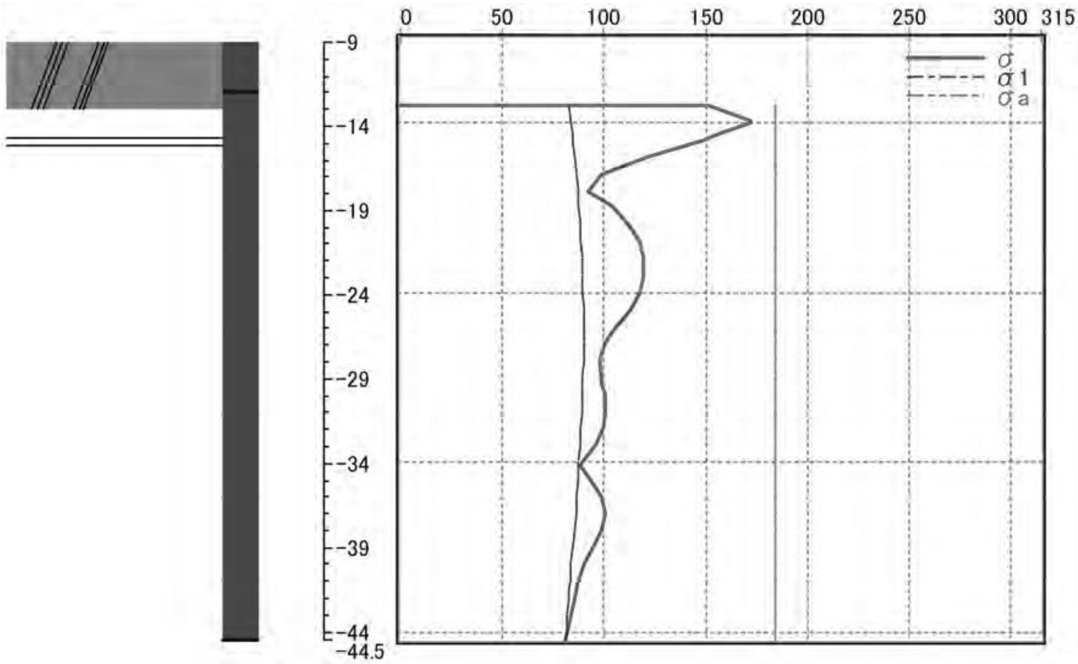
Source: Study Team

(4) Pile Stress

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

As the top slab concrete is placed with the steel pipe sheet piles in a deformed state, the residual stress (σ_1) due to and remaining after work execution and the stress (σ_2) occurring due to the design external forces after completion should be added. The sum (σ) shall be equal to the allowable stress (σ_a) or less.

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



where

σ : combined stress (= σ1 + σ2)

σ1 : stress due to after completion loads.

σ2 : residual stress due to during construction

σa : allowable stress in steel pipe sheet pile

Source: Study Team

Figure 8.3.5-13 Combined stress for steel pipe sheet pile foundation

(5) Top Slab

The top slab of a steel pipe sheet pile foundation generally has a large rigidity and is rigidly connected to the steel pipe sheet piles, it may be calculated as a cantilever with the fixed end at the outer edge of the lower end of the body. However, the soil in the well should share no load.

1) Pile Reaction Force

The vertical pile reaction force (Ri) of the steel pipe sheet pile used for the design of a top slab should be calculated using following formula.

$$Ri = \frac{Vo \cdot Aoi}{\Sigma(ni \cdot Aoi)} + \frac{Mo \cdot Aoi}{\Sigma(IBi \cdot Aoi)} \cdot Xi$$

where,

Ri : vertical reaction force of ith sheet pile and inner single piles (kN/pile)

Vo : vertical load acting on bottom face of top slab (kN)

Mo : moment acting on bottom face of top slab (kN.m)

n1 : number of sheet piles comprising outer periphery of well part

n2 : number of sheet piles comprising bulkhead

n3 : number of inner single piles

Ao1 : net cross-sectional area of sheet piles comprising outer periphery of well part

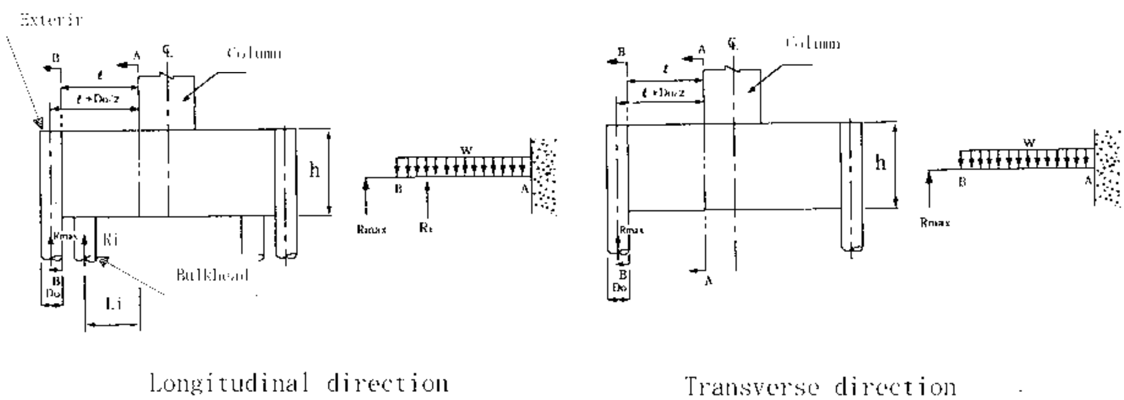
Ao2 : net cross-sectional area of sheet piles comprising bulkhead

Ao3 : net cross-sectional area of inner single piles

IBi : sum of distance from center of steel pipe sheet piles and inner single piles to the neutral axis of well section (m^2)

2) Bending Moment

In a section of top slab, investigate the per unit width the stress per unit width at the position of the steel pipe sheet pile that produces the maximum vertical reaction force



Source: Study Team

Figure 8.3.5-14 Section Calculation Model of Top Slab

$$MA = \frac{Rmax}{Do'} \cdot \left(L + \frac{Do}{2}\right) + \Sigma \left(Ri \cdot \frac{Li}{ai}\right) - \frac{w \cdot L^2}{2} \quad (kN.m/m)$$

$$MA' = \frac{Rmin}{Do'} \cdot \left(L + \frac{Do}{2}\right) + \Sigma \left(Ri \cdot \frac{Li}{ai}\right) - \frac{w \cdot L^2}{2} \quad (kN.m/m)$$

where, MA,MA' : bending moment per meter at the column bottom of a fixed edge (kN.m/m)

Rmaxi : maximum vertical reaction force of exterior pile (kN/pile)

Rmini : minimum vertical reaction force of exterior pile (kN/pile)

Ri : vertical reaction force of bulkhead pile (kN/pile)

L : distance between the column bottom and the inner surface of pile

Li : distance from face of column to center of bulkhead pile (m)

w : dead weight of top slab and overburden load (kN/m^2)

Do : outer diameter of sheet pile and inner pile (m) :

Do' : center-to-center interval of exterior sheet pile (m)

d : effective depth of top slab (m)

(6) Connection between Top Slab and Steel Pipe Pile

Connection between top slab and steel pipe pile for "Reinforcement Stud Method" should be calculated as followings.

1) Design bending moment

$Me = Rp \cdot e$

$MFix = \sigma sa \cdot Zo$

where, Me : moment due to eccentricity of reaction force (kN.m)

MFix : restraining moment (kN.m)

Rp : vertical reaction force of a sheet pile (kN)

e : eccentricity (m)

σsa : allowable stress of sheet pile (kN/m^2)

Zo : section modulus of sheet pile body (m^3)

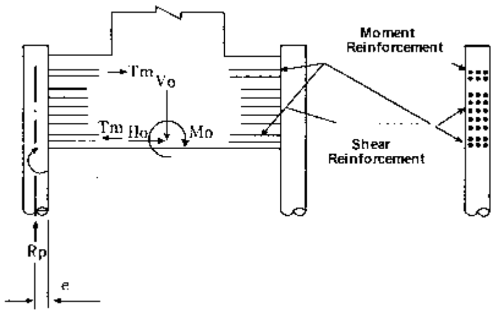
Use the maximum between Me and MFix.

2) Design of moment reinforcement

- Tensile stress due to moment

$$T1 = \frac{M}{h}$$

$$\sigma s1 = \frac{T1}{nb \cdot Ab}$$



where, $T1$: tensile force acting on moment reinforcement (N)

M : design moment (N.mm)

h : center-to-center interval of moment reinforcement (mm)

$\sigma s1$: tensile stress of moment reinforcement (N/mm^2)

nb : number of moment reinforcement (piles/layer)

Ab : cross-sectional area of one moment reinforcement (mm^2)

- Tensile stress due to horizontal force

$$T2 = \frac{Ho}{n1}$$

$$\sigma s2 = \frac{T2}{2 \cdot nb \cdot Ab}$$

where, $T2$: horizontal force acting on moment reinforcement (N)

Ho : horizontal force acting on the bottom of top slab (N)

$n1$: number of exterior sheet piles

$\sigma s2$: tensile stress of moment reinforcement (N/mm^2)

-Resultant stress

$\sigma s = \sigma s1 + \sigma s2 \leq \sigma sa$

where, σsa : allowable tensile stress of moment reinforcement (N/mm^2)

- Required number of piles

$$nba \geqq \frac{2 \cdot T1 + T2}{2 \cdot \sigma sa \cdot Ab}$$

where, nba : necessary number of moment reinforcement (piles/layer)

3) Design of shear reinforcement

- Shear stress

$$\tau s = \frac{Rp}{ns \cdot As} \leqq \tau sa$$

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

Rp : vertical reaction force of a sheet pile (N)

ns : number of shear reinforcement

As : cross-sectional area of shear reinforcement (mm^2)

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

- Required number of piles

$$nsa \geqq \frac{Rp}{\tau sa \cdot As}$$

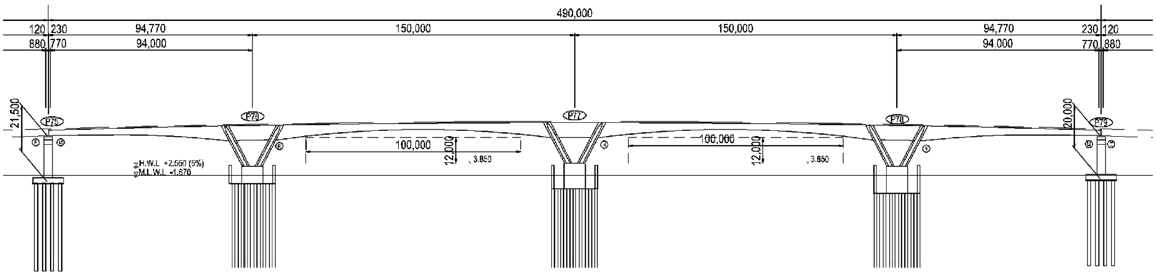
where, nsa : necessary number of shear reinforcement (piles)

8.3.6 Detailed Design of Main Bridge

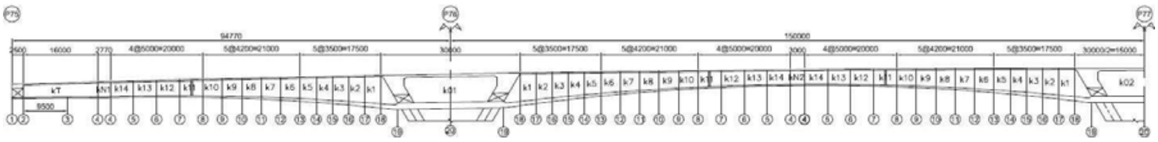
8.3.6.1 Design Condition

(1) Profile of bridge

The profile of Main Bridge is as shown in the figure below.



(a) Whole Profile of Main Bridge



(b) Segments and Sections of Superstructure

Source: Study Team

Figure 8.3.6-1 Profile of Main Bridge

(2) Conditions of Foundation

The conditions of the foundation of Main Bridge are presented in Section 8.3.5.3.

(3) Condition of Superstructure and Substructure

1) Geometry

Geometric Conditions of the pier are summarized in the table below.

Table 8.3.6-1 Dimensions of Structural Members of Substructure

Items		Dimension(m)	Area (m^2)	Inertia -1 (m^4)	Inertia -2 (m^4)
V-shaped Wall	Top	3.0 x 5.0	141	28.9	10.0
	Bottom	3.0 x 7.0	211	98.0	14.6
Lower Pier Column		7.0 x 10.5	73.5	300	675

Source: Study Team

Table 8.3.6-2 Dimensions of Structural Members of Superstructure

Items		Dimension (m)	Note
Girder Height	Cantilever Segments	Variation from 3.50 to 7.50	
	False Work Segments	Variation from 3.20 to 3.50	
Girder Width	Upper Slab	15.7	
	Lower Slab	Variation from 5.00 to 7.475	
Member Thickness	Web	Variation from 0.400 to 0.600	
	Upper Slab	Variation from 0.260 to 0.550	
	Lower Slab	Variation from 0.260 to 1.40	

Source: Study Team

2) Material

a) Concrete

Table 8.3.6-3 Properties of Concrete

<table>
<tr><th colspan="2">Items</th><th>Superstructure</th><th>Substructure</th></tr>
<tr><td colspan="2">Compressive Strength, f'c (MPa)*</td><td>40</td><td>40</td></tr>
<tr><td colspan="2">Compressive Strength at Pre-stressing, fci (MPa)*</td><td>32.5</td><td>32.5</td></tr>
<tr><td colspan="2">Specific Weight (kN/m^3)</td><td>24.5</td><td>24.5</td></tr>
<tr><td colspan="2">Young's Modulus, Ec (MPa)</td><td>33900</td><td>33900</td></tr>
<tr><td colspan="2">Allowable Stress (MPa)</td><td colspan="2"></td></tr>
<tr><td rowspan="2">Compressive</td><td>Before Losses</td><td>19.5</td><td>19.5</td></tr>
<tr><td>After Losses, Service Limit</td><td>16</td><td>16</td></tr>
<tr><td rowspan="4">Tensile</td><td rowspan="2">Before Losses</td><td>Longitudinal: 1.42</td><td rowspan="2">2.85</td></tr>
<tr><td>Transversal: 2.85</td></tr>
<tr><td rowspan="2">After Losses, Service Limit</td><td>Longitudinal: 1.58</td><td rowspan="2">3.16</td></tr>
<tr><td>Transversal: 3.16</td></tr>
</table>

*Based on Cylinder Samples

Source: Study Team

b) Steel

Table 8.3.6-4 Properties of Prestressing Steel

Items	12S15.2	19S15.2	1S28.6
Tensile Strength, fpu (MPa)	1850	1850	1800
Yield Strength, fpy (MPa)	1600	1600	1500
Modulus of Elasticity, Ep (MPa)	195000	195000	195000
Allowable Tensile Stress (MPa)	1295	1295	1260

Source: Study Team

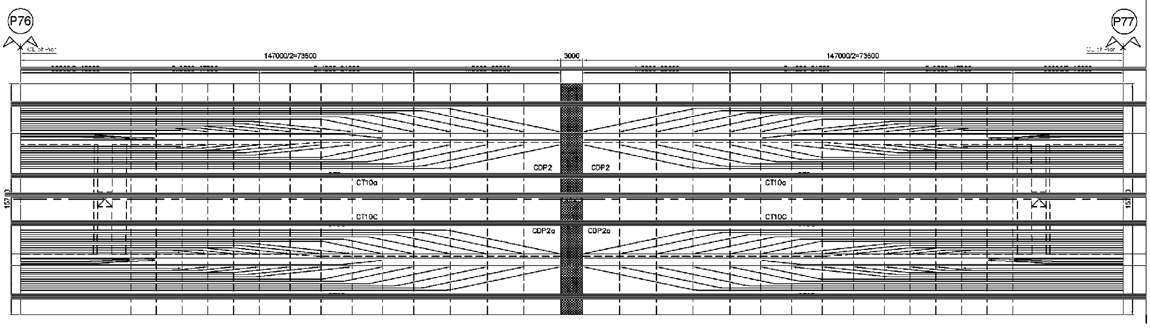
Table 8.3.6-5 Properties of Reinforcing Steel

Items	SD345	Note
Tensile Strength, fpu	490	
Yield Strength, fpy	345	
Modulus of Elasticity, Ep	200000	

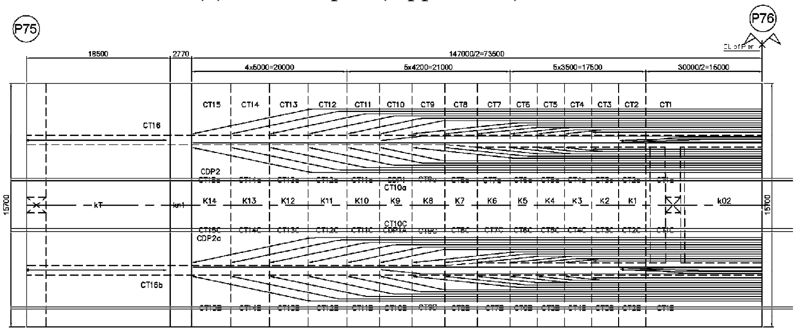
Source: Study Team

3) Arrangement of PC Tendons

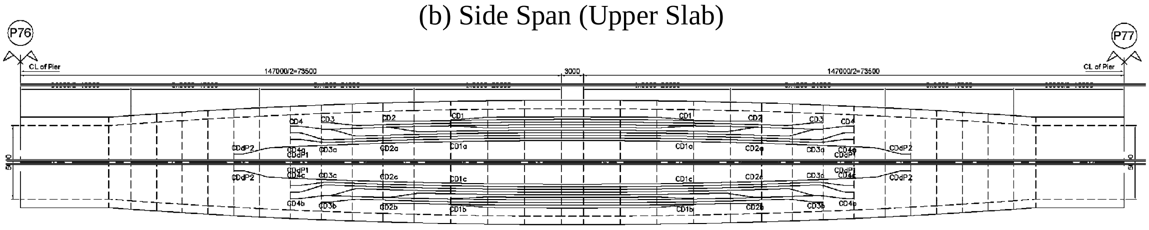
The arrangement of PC tendons is shown in the figure below.



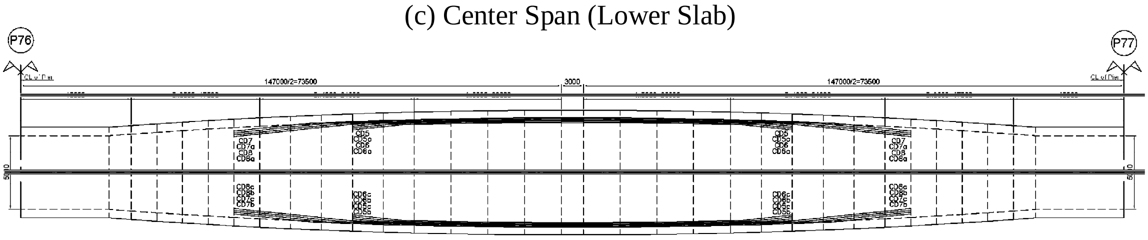
(a) Center Span (Upper Slab)



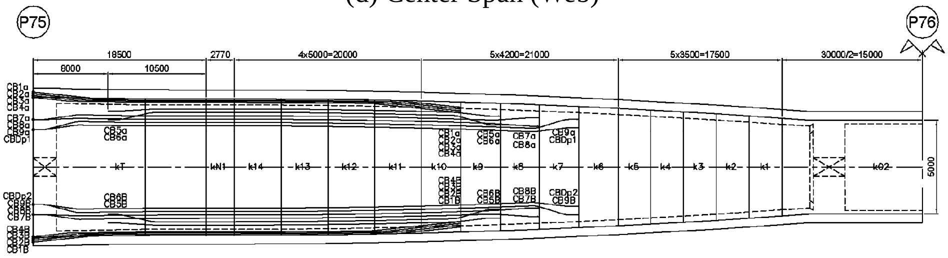
(b) Side Span (Upper Slab)



(c) Center Span (Lower Slab)



(d) Center Span (Web)



(e) Side Span (Lower Slab and Web)

Source: Study Team

Figure 8.3.6-2 Arrangement of PC Tendons

8.3.6.2 Design of Superstructure for Longitudinal Direction

(1) Model for Analysis

The conditions of the model of analysis is as follows,

- The model includes the members of superstructure, substructure and foundation.

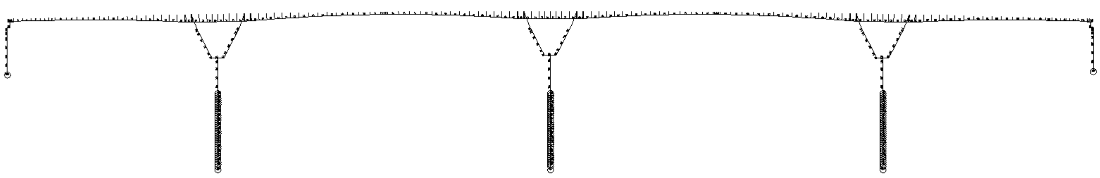
- The members of the structure are modeled into frame elements at the positions of the centroid.

- The rigid zones are considered at the connections between the girder and the V-shaped walls, and, the V-shaped walls and the lower pier column.

- The foundations are modeled as presented in Section 8.3.5.2

- The both ends at the side span are fixed in vertical direction, movable in longitudinal direction, and supported by a spring in transversal direction. The spring constants are also presented in Section 8.3.5.2.

The structural model for the analysis is shown in the figure below.



Source: Study Team

Figure 8.3.6-3 Model for Structural Analysis

(2) Loading Conditions

The loading conditions for Limit States are based on Chapter 3 in Specification for Bridge Design, 22TCN272-05, summarized in the article 8.1.3 of this report.

The loading conditions during construction stage is based on Section 5.14.2.3 in Specification for Bridge Design, 22TCN272-05.

The weight and capacity of the form traveler are assumed as 1,000 kN and 4,000 kN・m, respectively.

(3) Simulation of Construction Sequence

The construction sequence of the bridge is simulated in the analysis as shown in the table on the next page.

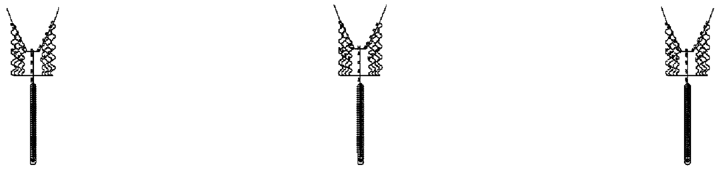
The subsidiary measure, pressurization by jacking force just before casting the closure segments, is also considered, as presented in Section 8.3.4. The jacking force applied in the final analysis is 2,000kN.

The analysis models corresponding to the construction stages are shown in the following figures.

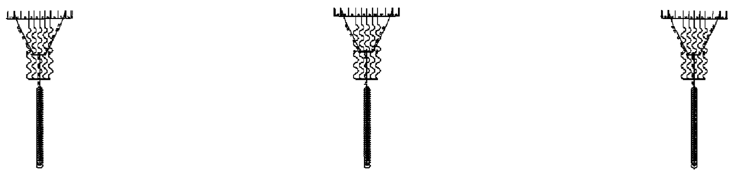
Table 8.3.6-6 Construction Sequence simulated in Structural Analysis

Segment	Loading	Duration	Age of Concrete at Prestressing	Segment																
		(day)	(day)	Pier	K0	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10	K11	K12	K13	K14	KN
KT1	Self weight, moving traveler, PT, CS	10	3																K14	
KN1	arrange traveler, Wet concrete Self weight, Remove Wet Concrete, Remove Support, Remove traveler, PT, CS	10	3																	KN1
K14	Self weight, moving traveler, PT, CS	10	3																K14	
K13	Self weight, moving traveler, PT, CS	10	3															K13		
K12	Self weight, moving traveler, PT, CS	10	3														K12			
K11	Self weight, moving traveler, PT, CS	10	3													K11				
K10	Self weight, moving traveler, PT, CS	10	3												K10					
K9	Self weight, moving traveler, PT, CS	10	3											K9						
K8	Self weight, moving traveler, PT, CS	10	3										K8							
K7	Self weight, moving traveler, PT, CS	10	3									K7								
K6	Self weight, moving traveler, PT, CS	10	3								K6									
K5	Self weight, moving traveler, PT, CS	10	3							K5										
K4	Self weight, moving traveler, PT, CS	10	3						K4											
K3	Self weight, moving traveler, PT, CS	10	3					K3												
K2	Self weight, moving traveler, PT, CS	10	3				K2													
K1	Self weight, moving traveler, PT, CS	10	3			K1														
K0	Self weight, instal traveler, PT, CS	30	20		K0															
Pier	Self weight	60	-	Pier	K0															
K0	Self weight, instal traveler, PT, CS	30	20		K0															
K1	Self weight, moving traveler, PT, CS	10	3			K1														
K2	Self weight, moving traveler, PT, CS	10	3				K2													
K3	Self weight, moving traveler, PT, CS	10	3					K3												
K4	Self weight, moving traveler, PT, CS	10	3						K4											
K5	Self weight, moving traveler, PT, CS	10	3							K5										
K6	Self weight, moving traveler, PT, CS	10	3								K6									
K7	Self weight, moving traveler, PT, CS	10	3									K7								
K8	Self weight, moving traveler, PT, CS	10	3										K8							
K9	Self weight, moving traveler, PT, CS	10	3											K9						
K10	Self weight, moving traveler, PT, CS	10	3												K10					
K11	Self weight, moving traveler, PT, CS	10	3													K11				
K12	Self weight, moving traveler, PT, CS	10	3														K12			
K13	Self weight, moving traveler, PT, CS	10	3															K13		
K14	Self weight, moving traveler, PT, CS	10	3																K14	
KN2	arrange traveler, Jacking force, Wet concrete Self weight, Remove Wet Concrete, Remove traveler, Remove Jacking force, PT, CS	10	3																	KN2
K14	Self weight, moving traveler, PT, CS	10	3																K14	
K13	Self weight, moving traveler, PT, CS	10	3															K13		
K12	Self weight, moving traveler, PT, CS	10	3														K12			
K11	Self weight, moving traveler, PT, CS	10	3													K11				
K10	Self weight, moving traveler, PT, CS	10	3												K10					
K9	Self weight, moving traveler, PT, CS	10	3											K9						
K8	Self weight, moving traveler, PT, CS	10	3										K8							
K7	Self weight, moving traveler, PT, CS	10	3									K7								
K6	Self weight, moving traveler, PT, CS	10	3								K6									
K5	Self weight, moving traveler, PT, CS	10	3							K5										
K4	Self weight, moving traveler, PT, CS	10	3						K4											
K3	Self weight, moving traveler, PT, CS	10	3					K3												
K2	Self weight, moving traveler, PT, CS	10	3				K2													
K1	Self weight, moving traveler, PT, CS	10	3			K1														
K0	Self weight, instal traveler, PT, CS	30	20		K0															
Pier	Self weight	60	-	Pier	K0															
K0	Self weight, instal traveler, PT, CS	30	20		K0															
K1	Self weight, moving traveler, PT, CS	10	3			K1														
K2	Self weight, moving traveler, PT, CS	10	3				K2													
K3	Self weight, moving traveler, PT, CS	10	3					K3												
K4	Self weight, moving traveler, PT, CS	10	3						K4											
K5	Self weight, moving traveler, PT, CS	10	3							K5										
K6	Self weight, moving traveler, PT, CS	10	3								K6									
K7	Self weight, moving traveler, PT, CS	10	3									K7								
K8	Self weight, moving traveler, PT, CS	10	3										K8							
K9	Self weight, moving traveler, PT, CS	10	3											K9						
K10	Self weight, moving traveler, PT, CS	10	3												K10					
K11	Self weight, moving traveler, PT, CS	10	3													K11				
K12	Self weight, moving traveler, PT, CS	10	3														K12			
K13	Self weight, moving traveler, PT, CS	10	3															K13		
K14	Self weight, moving traveler, PT, CS	10	3																K14	
KN3	arrange traveler, Jacking force, Wet concrete Self weight, Remove Wet Concrete, Remove traveler, Remove Jacking force, PT, CS	10	3																	KN3
K14	Self weight, moving traveler, PT, CS	10	3																K14	
K13	Self weight, moving traveler, PT, CS	10	3															K13		
K12	Self weight, moving traveler, PT, CS	10	3														K12			
K11	Self weight, moving traveler, PT, CS	10	3													K11				
K10	Self weight, moving traveler, PT, CS	10	3												K10					
K9	Self weight, moving traveler, PT, CS	10	3											K9						
K8	Self weight, moving traveler, PT, CS	10	3										K8							
K7	Self weight, moving traveler, PT, CS	10	3									K7								
K6	Self weight, moving traveler, PT, CS	10	3								K6									
K5	Self weight, moving traveler, PT, CS	10	3							K5										
K4	Self weight, moving traveler, PT, CS	10	3						K4											
K3	Self weight, moving traveler, PT, CS	10	3					K3												
K2	Self weight, moving traveler, PT, CS	10	3				K2													
K1	Self weight, moving traveler, PT, CS	10	3			K1														
K0	Self weight, instal traveler, PT, CS	30	20		K0															
Pier	Self weight	60	-	Pier	K0															
K0	Self weight, instal traveler, PT, CS	30	20		K0															
K1	Self weight, moving traveler, PT, CS	10	3			K1														
K2	Self weight, moving traveler, PT, CS	10	3				K2													
K3	Self weight, moving traveler, PT, CS	10	3					K3												
K4	Self weight, moving traveler, PT, CS	10	3						K4											
K5	Self weight, moving traveler, PT, CS	10	3							K5										
K6	Self weight, moving traveler, PT, CS	10	3								K6									
K7	Self weight, moving traveler, PT, CS	10	3									K7								
K8	Self weight, moving traveler, PT, CS	10	3										K8							
K9	Self weight, moving traveler, PT, CS	10	3											K9						
K10	Self weight, moving traveler, PT, CS	10	3												K10					
K11	Self weight, moving traveler, PT, CS	10	3													K11				
K12	Self weight, moving traveler, PT, CS	10	3														K12			
K13	Self weight, moving traveler, PT, CS	10	3															K13		
K14	Self weight, moving traveler, PT, CS	10	3																K14	
KN4	arrange traveler, Wet concrete Self weight, Remove Wet Concrete, Remove Support, Remove traveler, PT, CS	10	3																	KN3
KT4	Self weight, moving traveler, PT, CS	30	10																K14	

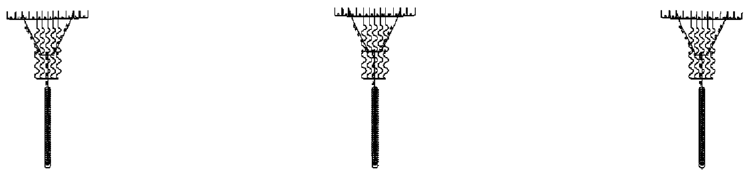
Source: Study Team



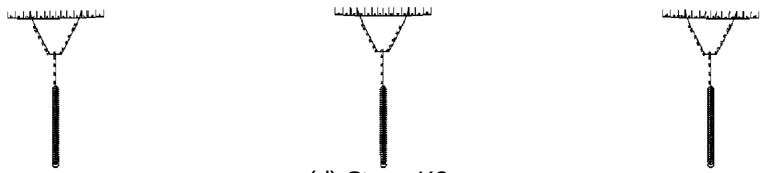
(a) Stage Pier



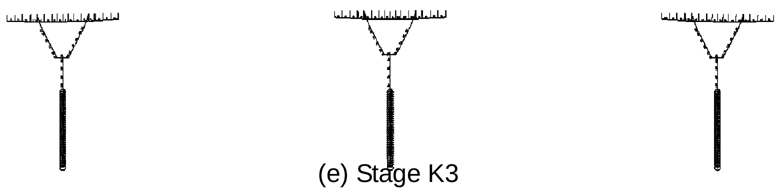
(b) Stage K0



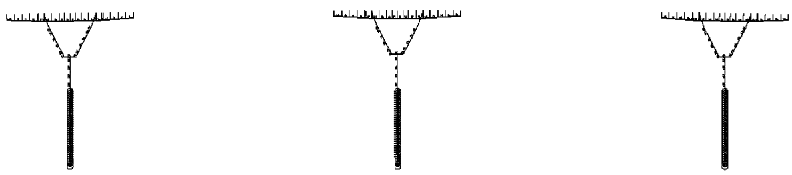
(c) Stage K1



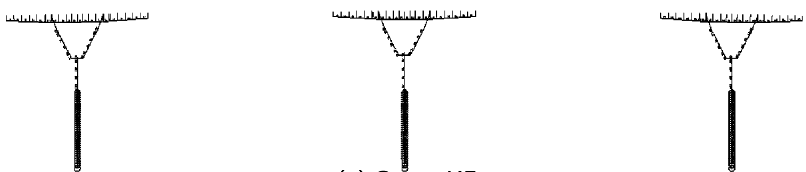
(d) Stage K2



(e) Stage K3



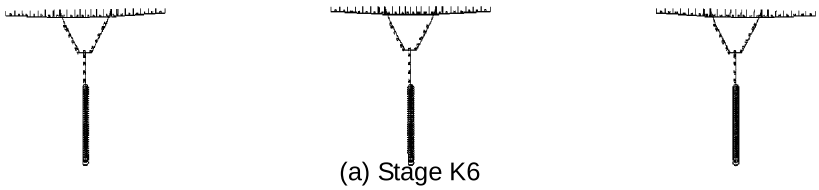
(f) Stage K4



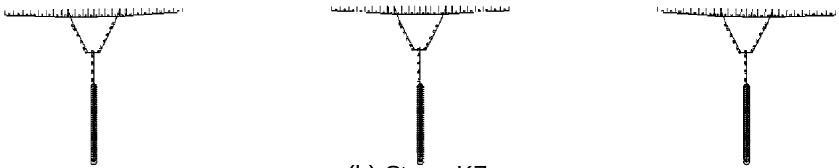
(g) Stage K5

Source: Study Team

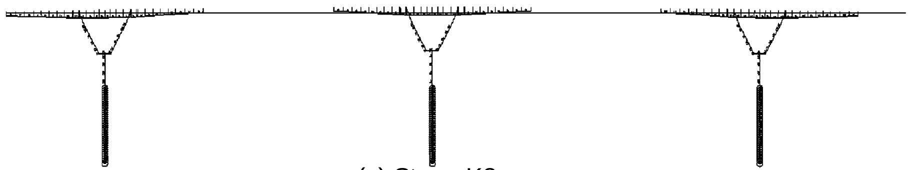
Figure 8.3.6-4 Models corresponding to Construction Sequence (1)



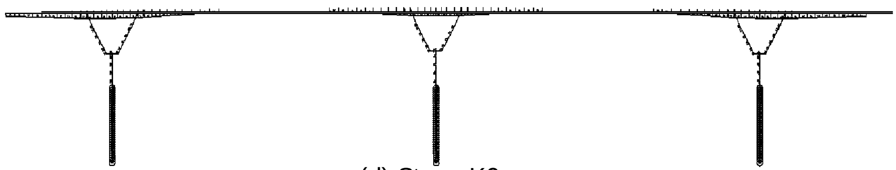
(a) Stage K6



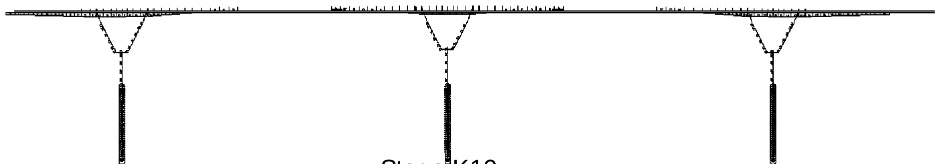
(b) Stage K7



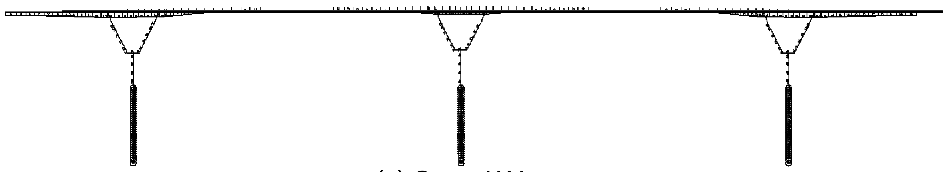
(c) Stage K8



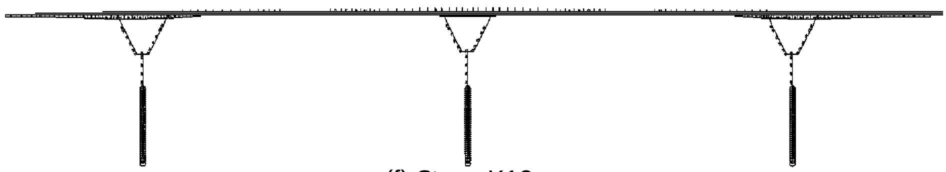
(d) Stage K9



Stage K10



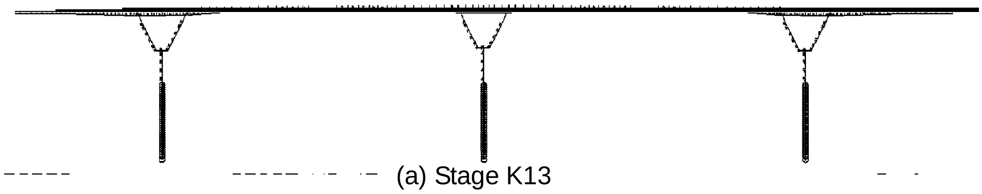
(e) Stage K11



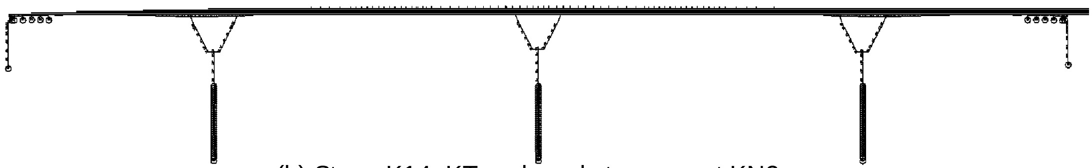
(f) Stage K12

Source: Study Team

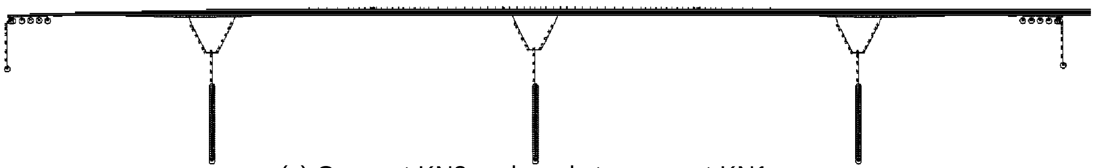
Figure 8.3.6-5 Models corresponding to Construction Sequence (2)



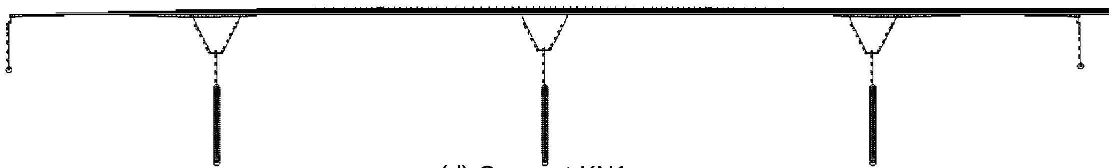
(a) Stage K13



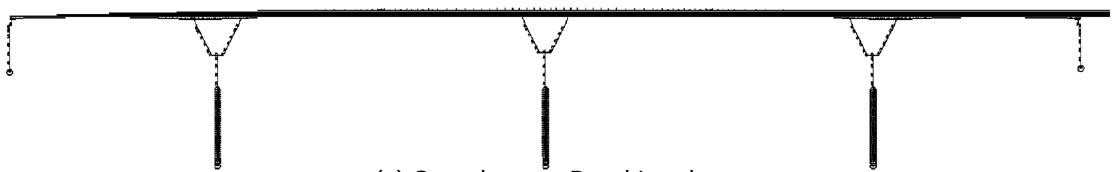
(b) Stage K14, KT and ready to connect KN2



(c) Connect KN2 and ready to connect KN1



(d) Connect KN1



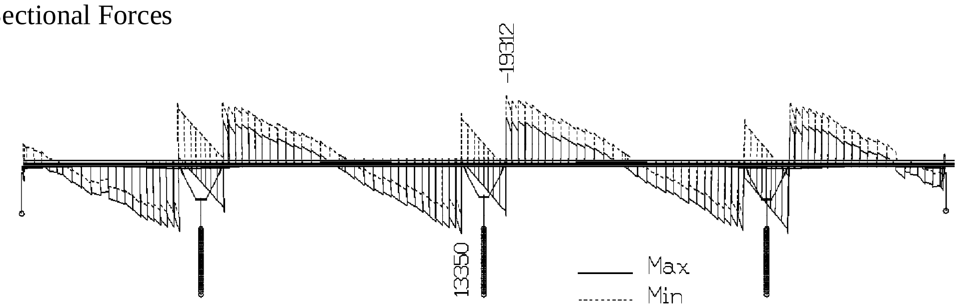
(e) Superimpose Dead Load

Source: Study Team

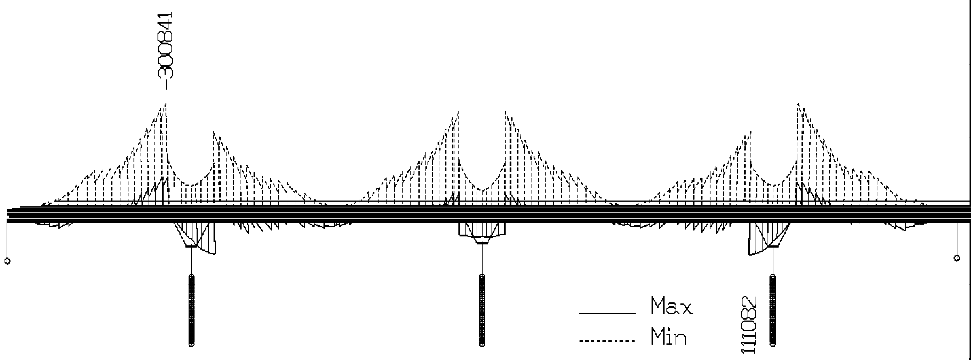
Figure 8.3.6-6 Models corresponding to Construction Sequence (3)

(4) Result of Analysis

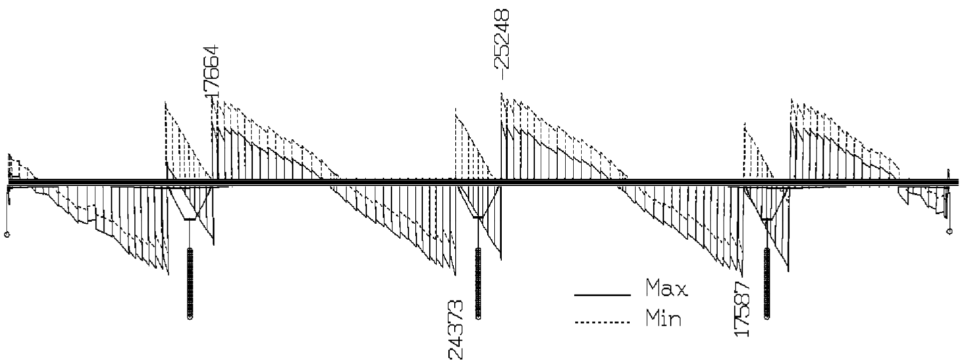
1) Sectional Forces



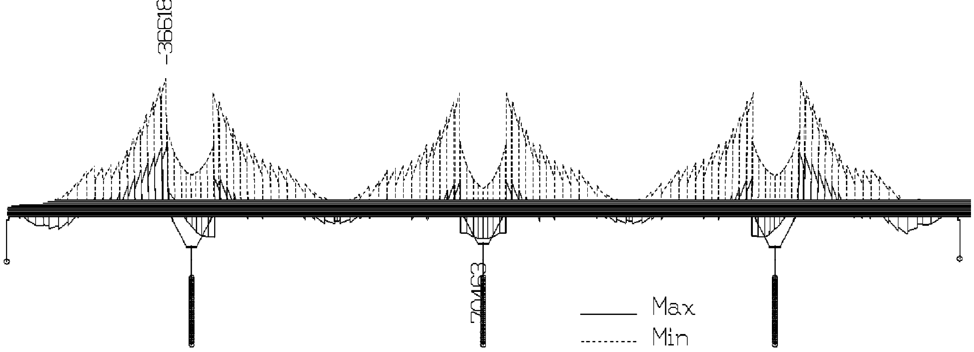
(a) Shear Force (Service Limit State)



(b) Bending Moment (Service Limit State)



(c) Shear Force (Strength Limit State)

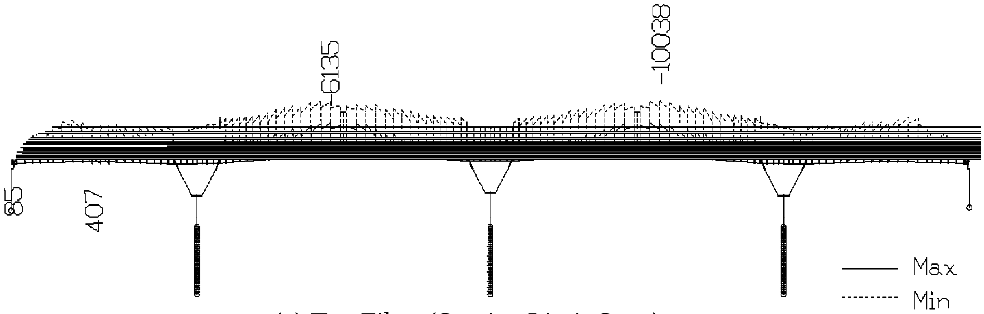


(d) Bending Moment (Strength Limit State)

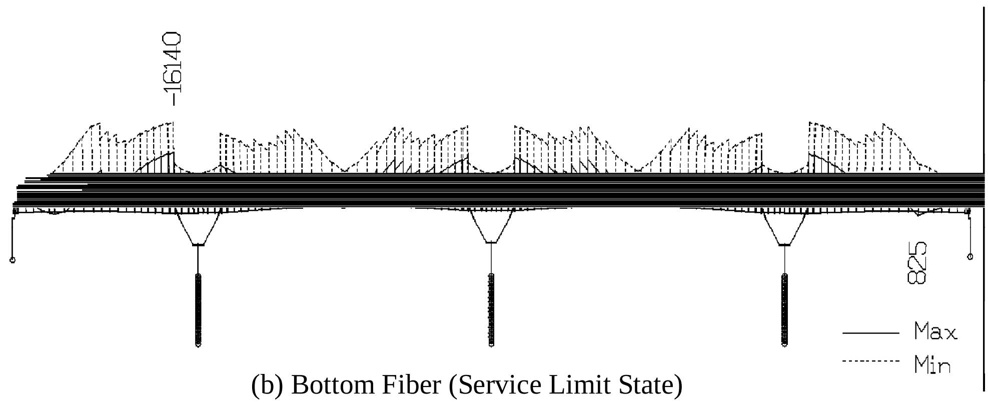
Source: Study Team

Figure 8.3.6-7 Sectional Forces

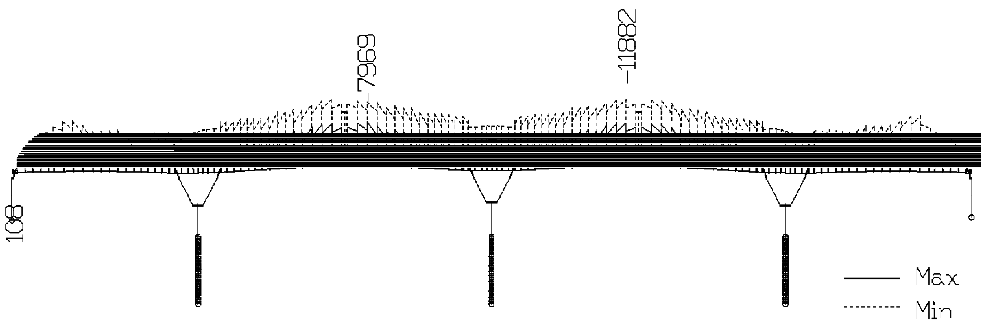
2) Fiber Stress



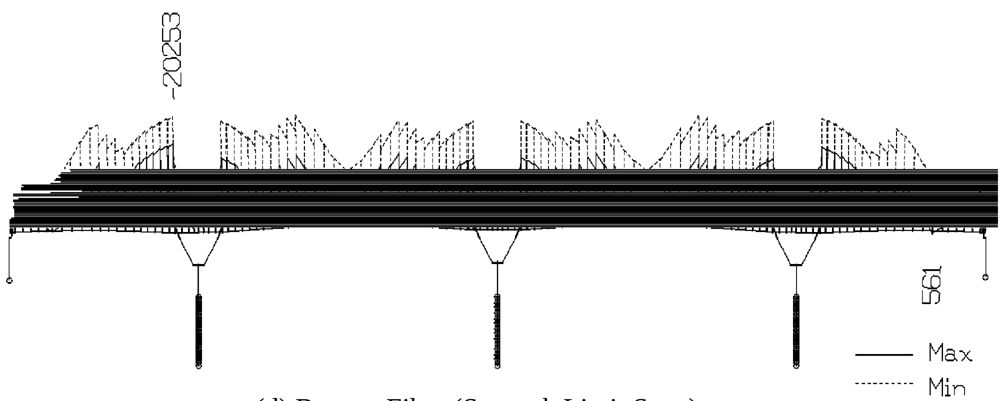
(a) Top Fiber (Service Limit State)



(b) Bottom Fiber (Service Limit State)



(c) Top Fiber (Strength Limit State)



(d) Bottom Fiber (Strength Limit State)

Source: Study Team

Figure 8.3.6-8 Fiber Stress

(5) Reinforcement

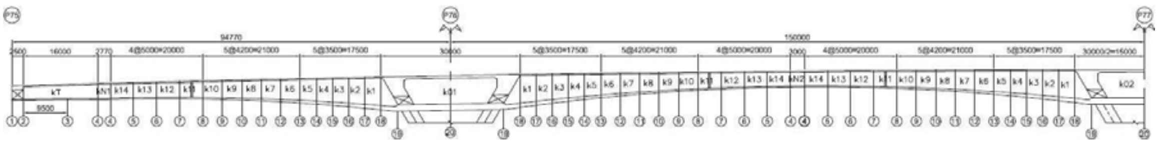
1) Reinforcement for Bending Moment in Longitudinal Direction

The nominal flexural resistance of the girder according to the formula 5.7.3.2.2-1 of the specification for bridge design 22TCN272-05 with minimum reinforcement is always larger than the 1.33 times the factored moment required by applicable strength load combinations specified in Section 8.1.3. Therefore, minimum reinforcement, approximately 0.3% of gross area in accordance with the formula 5.7.3.3.2-1 of 22TCN272-05, is applied for the main girder.

2) Reinforcement for Shear Force in Longitudinal Direction

The required amount of reinforcement is summarized in the table below.

Table 8.3.6-7 Required Reinforcement for Shear Force in Longitudinal Direction



Section	Factored Shear Force Vu min (kN)	Factored Shear Force Vu max (kN)	Component of Prestressing Force Vp (kN)	Nominal Shear Resistance of Concrete Vc (KN)	Required Shear Resistance by Reinforcement Vs (kN)	Required Area of Reinforcement As (cm^2)*
S1	-7170	-12541	2469	2797	8669	12.6
S2	-5671	-10600	1695	2871	7213	11.1
S3	1213	-2663	-368	2583	744	1.8
S4	2767	-1265	-1126	2425	0	1.8
S5	5136	1091	-989	2350	2368	3.4
S6	7358	3180	-1228	2645	4303	4.6
S7	9621	5168	-1211	3231	6247	4.5
S8	11935	7096	-2904	3531	6826	5.0
S9	14050	8808	-2666	3678	9266	8.3
S10	16193	10523	-3638	3821	10534	8.9
S11	18390	12270	-4954	4035	11445	9.3
S12	21028	14444	-5217	5954	12193	9.7
S13	23439	16440	-5853	6317	13873	10.7
S14	25512	18168	-6983	6736	14628	10.5
S15	27652	19977	-7873	7164	15688	10.4
S16	29859	21867	-9177	7589	16411	10.1
S17	32133	23841	-11979	8046	15679	8.9
S18	34479	25933	-14151	8537	15622	7.3
S21	34798	26196	-13729	8606	16330	7.8

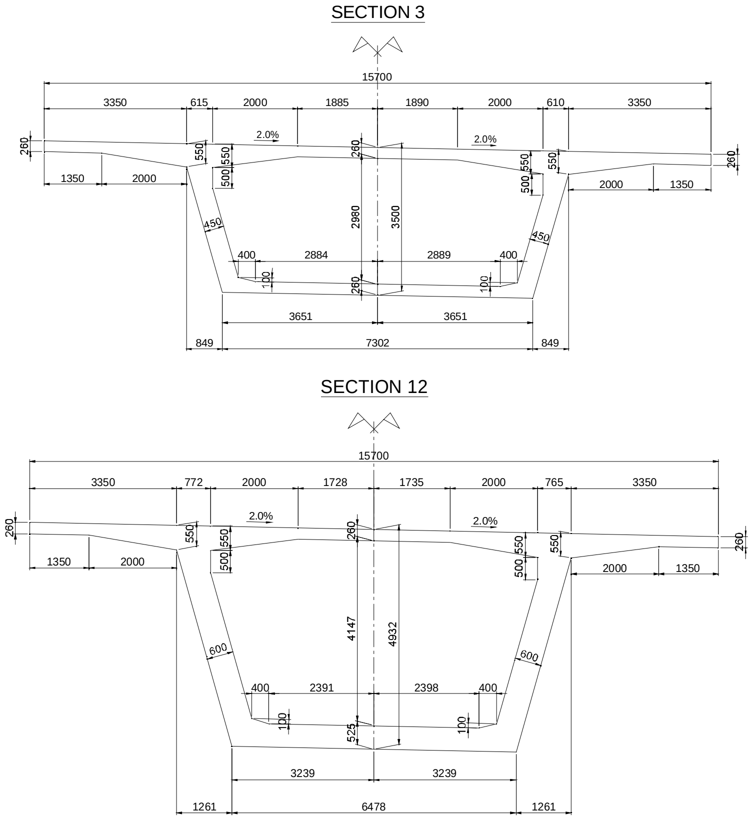
* Spacing of Stirrup : 0.15m, number of stirrup in a section: 4

Source: Study Team

8.3.6.3 Design of Superstructure in Transversal Direction

(1) Sections for Analysis

The design calculation was performed for the two unfavorable sections, Section 3 with 450mm web at the center of the span and Section 12 with 600mm, shown in the following figure.



Source: Study Team

Figure 8.3.6-9 Sections for Transversal Analysis

(2) Model of Analysis

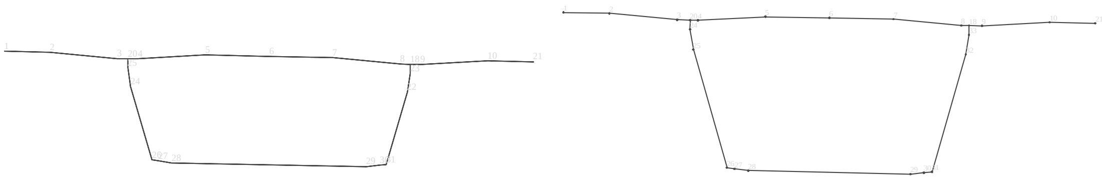
The conditions of the model of analysis is as follows,

- The members of the structure are modeled into frame elements at the positions of the centroid.

- The rigid zones are considered at the connections between the upper slab and the webs, and, lower slab and webs.

- The both ends of lower slab are fixed in vertical direction, movable in horizontal direction.

The structural model for the analysis is shown in the figure below.



(a) Section 3 (b) Section 12

Source: Study Team

Figure 8.3.6-10 Model for Transversal Analysis

(3) Loading Conditions

The loading conditions for Limit States are based on Chapter 3 in Specification for Bridge Design, 22TCN272-05, summarized in the article 8.1.3 in this report.

1) Dead Load:

Weight of components: DC

Unit weight of reinforced concrete is assumed as 24.5 kN/m^3.

Weight of Wearing Course: DW

Depth of wearing course is : 75.00 mm

Unit weight of wearing course is : 22.5 kN/m^3

Uniform load for wearing course is : 1.688 kN/m

Curb Concrete and Handrail: 8.54kN at each end

Water Pipe: 2.25kN at 3050mm from each end

2) Live Load:

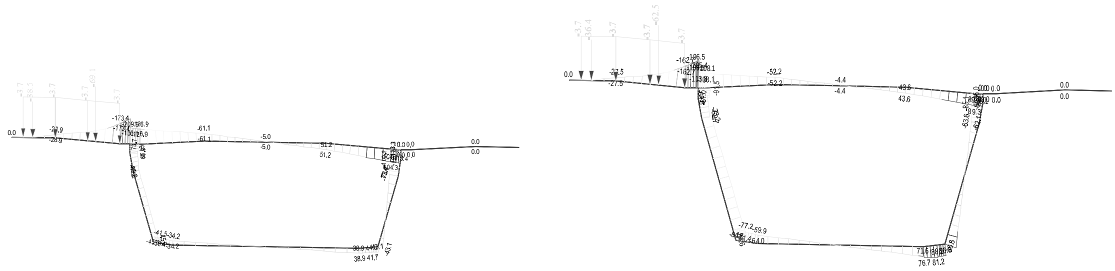
Live Load: LL

As specified in the article 3.6.1.2 of 22TCN272-05, Design Truck and Lane Load were simultaneously loaded. Equivalent Interior Strips based on the article 4.6.2.1 of 22 TCN272-05 were assumed for loading Design Truck in transversal direction.

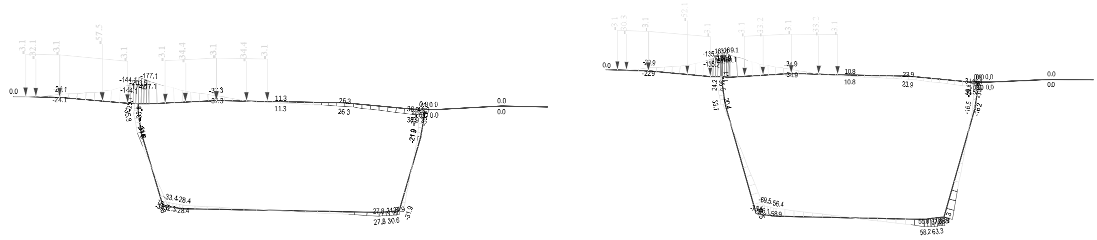
Dynamic Load Allowance: IM

Dynamic load allowance is 25% based on Table 3.6.2.1-1 of 22TCN272-05.

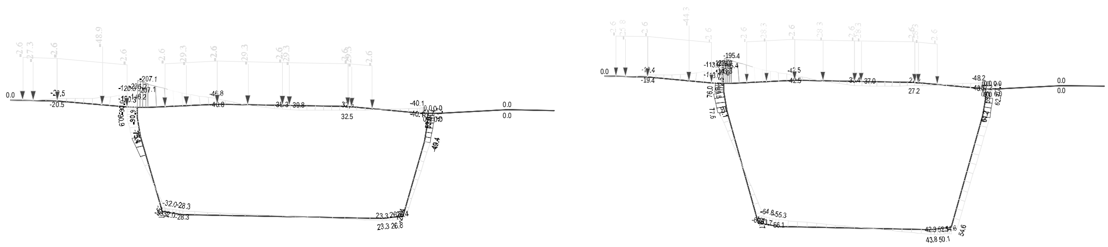
The conditions for LL including IM for the transversal analysis are illustrated in the figures on the next page.



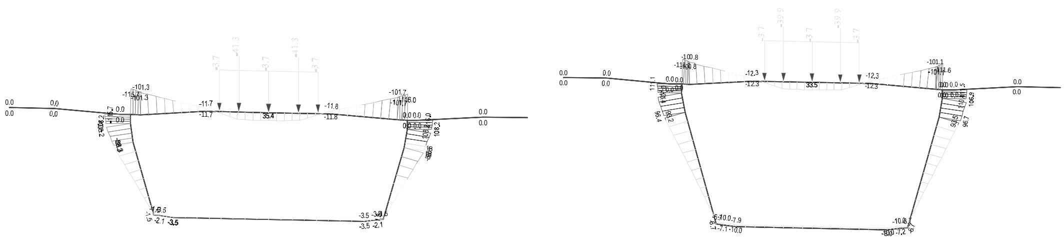
(a) One Lane on Left Side (LL1L)



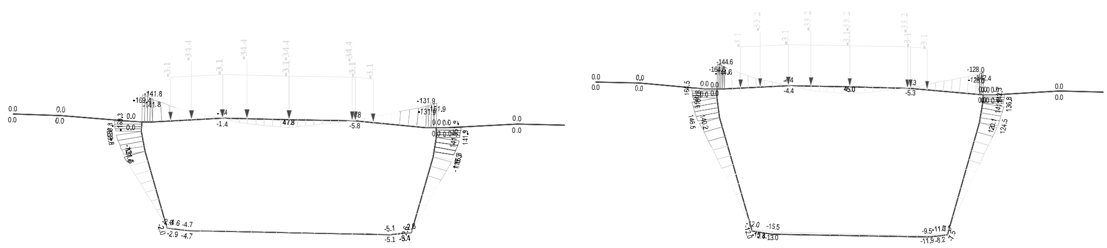
(b) Two Lanes on Left Side (LL2L)



(c) Three Lanes on Left Side (LL3L)



(d) One Lane at Center (LL1C)



(e) Two Lanes at Center (LL2C)

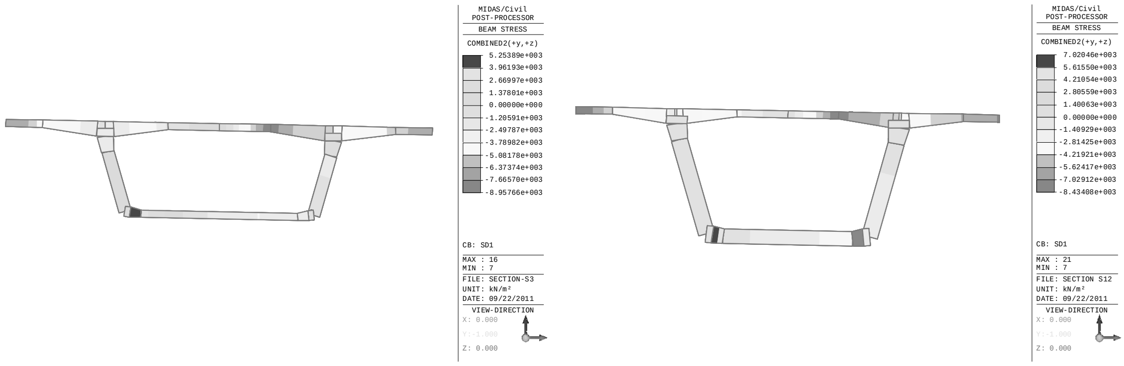
Source: Study Team (Left: Section 3, Right: Section 12)

Figure 8.3.6-11 Design Truck and Lane Loading for Transversal Analysis

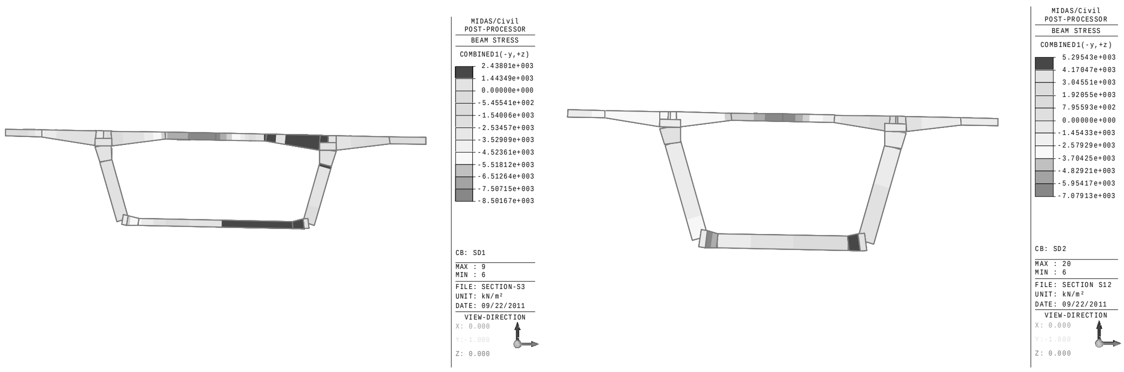
(4) Result of Analysis

1) Service Limit State

All the resultant stresses in the upper slab in Service Limit State are less than the limit stress of full prestressing, 3.2 MPa. The following figure shows the results of fiber stress in one of the unfavorable combinations in the Service Limit.



(a) Top Fiber

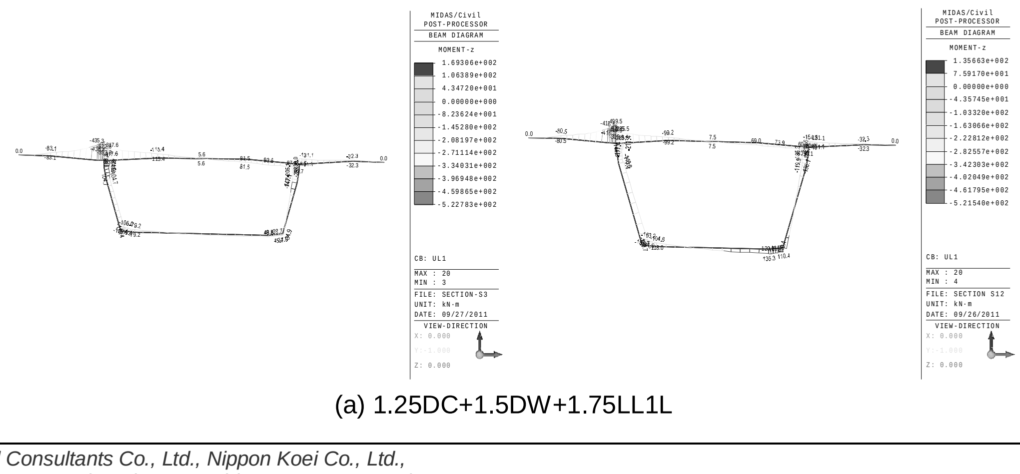


(b) Bottom Fiber

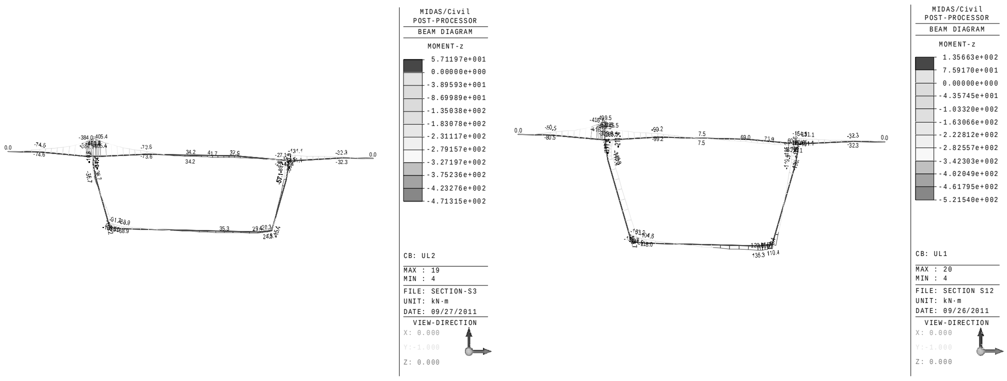
Source: Study Team (Left: Section 3, Right, Section 12)

Figure 8.3.6-12 Stress in Service Limit State (LL1L)

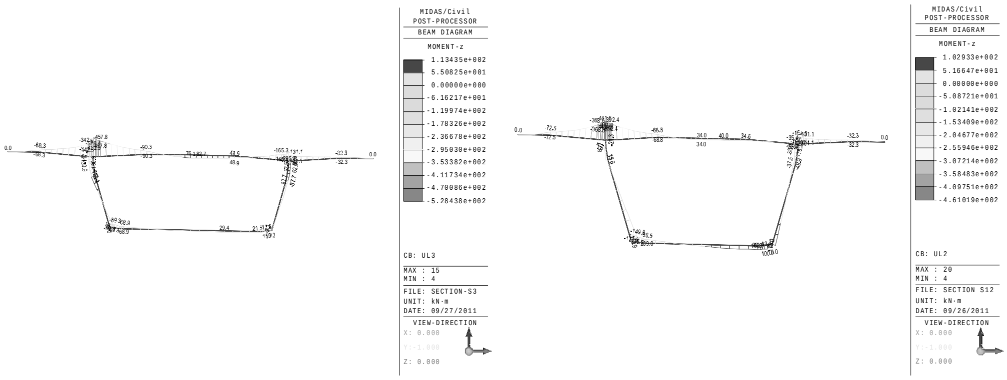
2) Strength Limit



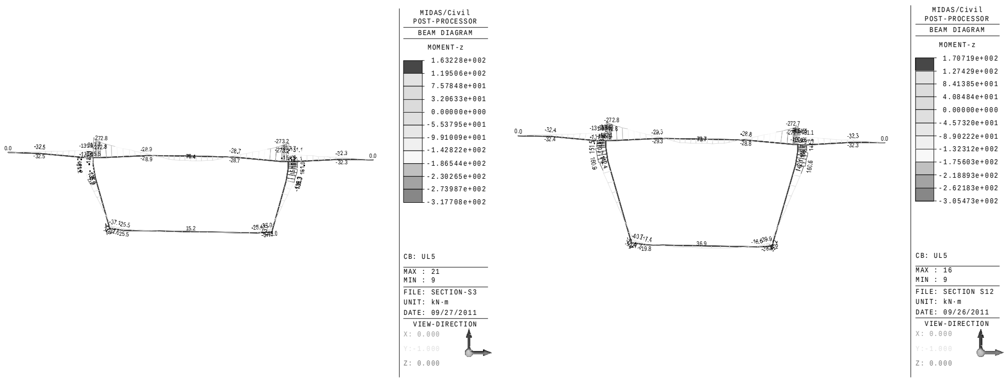
(a) 1.25DC+1.5DW+1.75LL1L



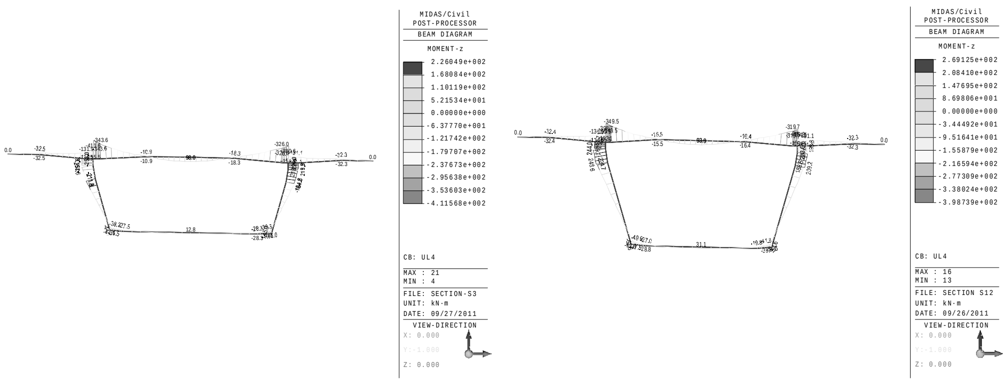
(b) 1.25DC+1.5DW+1.75LL2L



(c) 1.25DC+1.5DW+1.75LL3L



(d) 1.25DC+1.5DW+1.75LL1C



(e) 1.25DC+1.5DW+1.75LL2C

Source: Study Team (Left: Section 3, Right: Section 12)

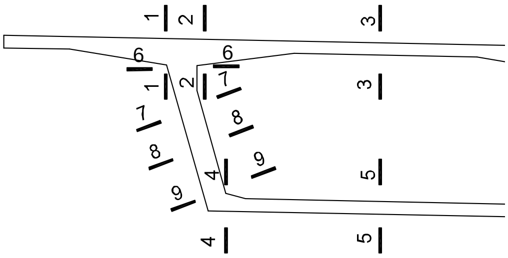
Figure 8.3.6-13 Bending Moment Diagram in Strength Limit State

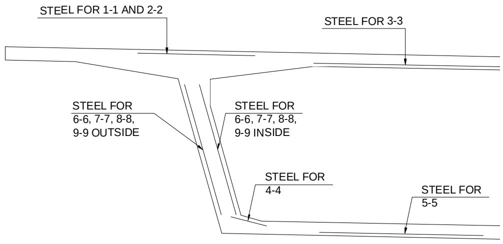
(5) Reinforcement

1) Reinforcement for Bending Moment in Transversal Direction

The required amount of reinforcement is summarized in the table below.

Table 8.3.6-8 Required Reinforcement in Transversal Direction





Location	Required Amount of reinforcement		Adopted Amount of Reinforcement	
	Section 3	Section 12	Section 3	Section 12
1-1	-	-	D14@150mm = 10cm^2	D14@150mm = 10cm^2
2-2	-	-	D14@150mm = 10cm^2	D14@150mm = 10cm^2
3-3	-	-	D14@150mm = 10cm^2	D14@150mm = 10cm^2
4-4	16cm^2	12cm^2	D18@150mm = 17cm^2	D16@150mm = 13cm^2
5-5	5cm^2	5cm^2	D14@150mm = 10cm^2	D14@150mm = 10cm^2
6-6 inside	11cm^2	9cm^2	D16@150mm = 13cm^2	D16@150mm = 13cm^2
7-7 inside	10cm^2	8cm^2	D16@150mm = 13cm^2	D16@150mm = 13cm^2
8-8 inside	11cm^2	10cm^2	D16@150mm = 13cm^2	D16@150mm = 13cm^2
9-9 inside	12cm^2	13cm^2	D16@150mm = 13cm^2	D16@150mm = 13cm^2
6-6 outside	21cm^2	17cm^2	D22@150mm = 25cm^2	D18@150mm = 17cm^2
7-7 outside	22cm^2	17cm^2	D22@150mm = 25cm^2	D18@150mm = 17cm^2
8-8 outside	20cm^2	7cm^2	D22@150mm = 25cm^2	D18@150mm = 17cm^2
9-9 outside	5cm^2	5cm^2	D22@150mm = 25cm^2	D18@150mm = 17cm^2

Source: Study Team

2) Reinforcement for Shear Force in Transversal Direction

It was verified that the capacities of the sections are beyond the maximum shear forces.

8.3.6.4 Design of Substructure of Main Bridge

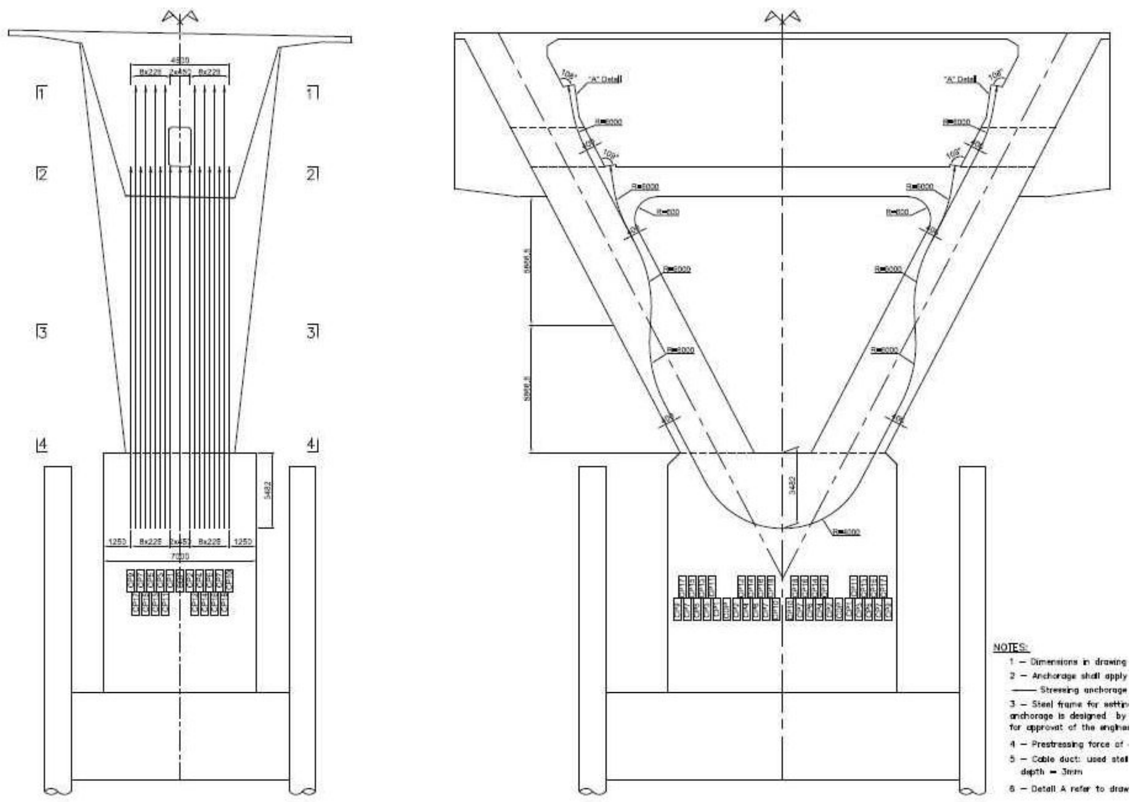
(1) Structural Analysis

The results of structural analysis for longitudinal direction presented in Section 8.3.6.2 are alSourced for designing the substructure.

(2) Conditions of Analysis

1) Cross Section of Piers

The geometric parameters are as presented in the previous section. The cross sections of piers are shown in the figure below. As shown in the figure, 18 PC tendons are installed in the V-shaped walls for avoiding cracking in Service Limit State.



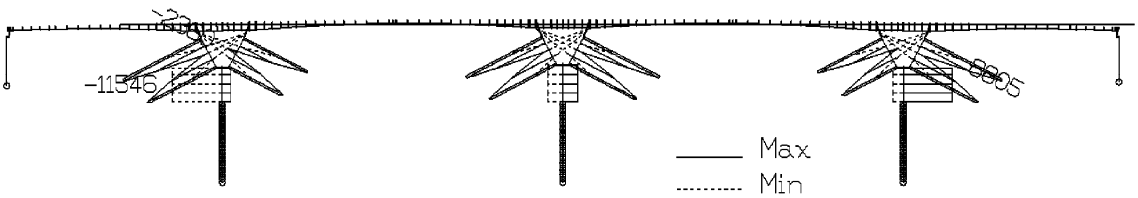
(a) Transversal Section of Pier (b) Longitudinal Section of Pier

Source: Study Team

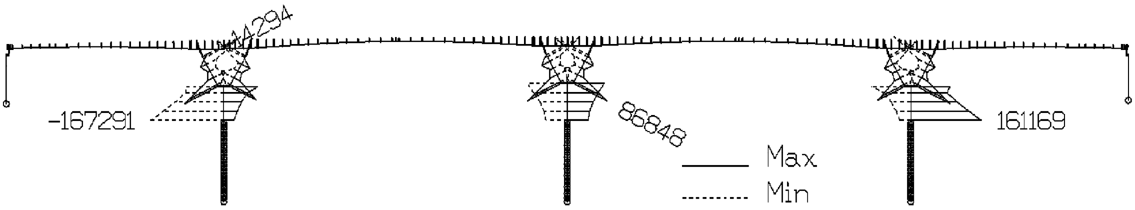
Figure 8.3.6-14 Arrangement of PC Tendons

(3) Result of Analysis

1) Sectional Forces

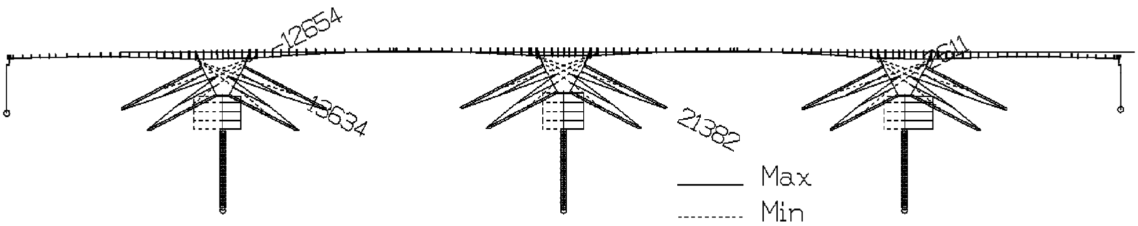


(a) Shear Force

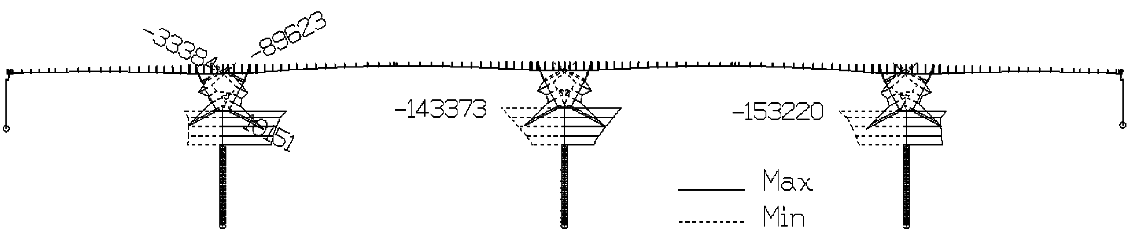


(b) Bending Moment

Figure 8.3.6-15 Sectional Forces in Service Limit State



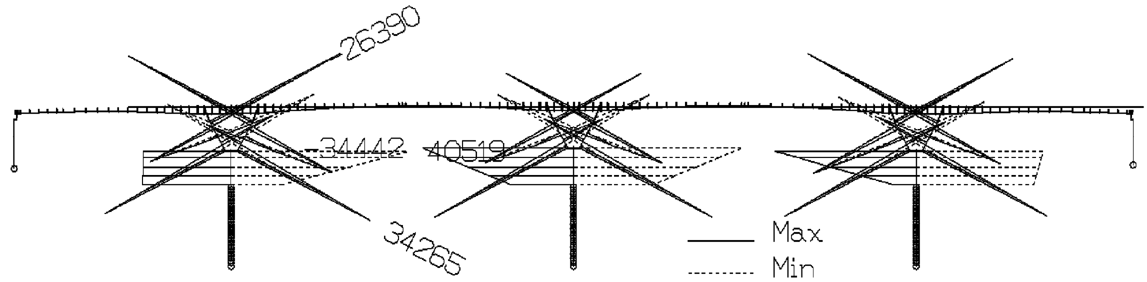
(a) Shear Force



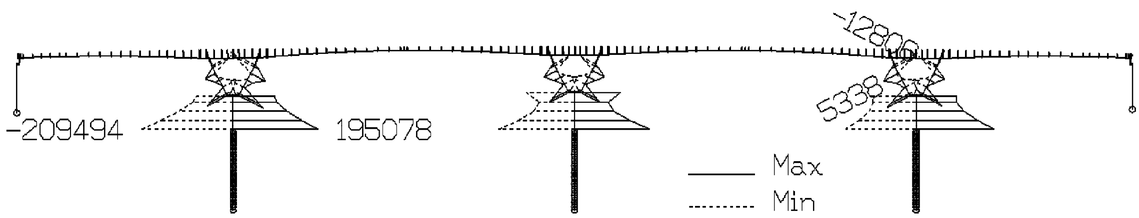
(b) Bending Moment

Source: Study Team

Figure 8.3.6-16 Sectional Forces in Strength Limit State



(a) Shear Force

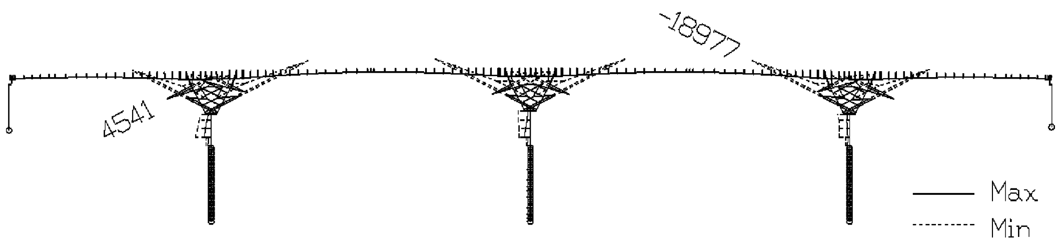


(b) Bending Moment

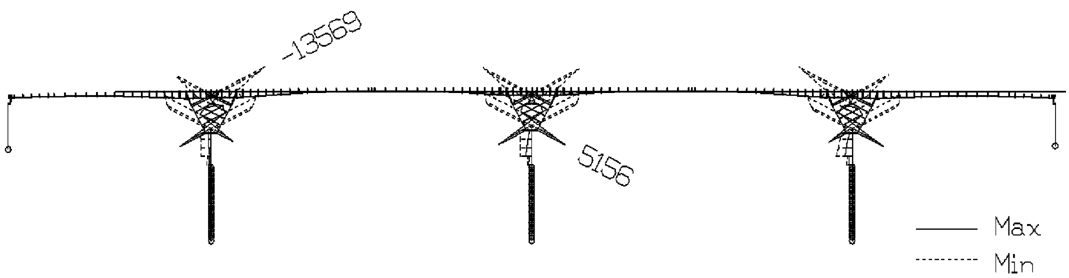
Source: Study Team

Figure 8.3.6-17 Sectional Forces in Extreme Event Limit State

2) Fiber Stress



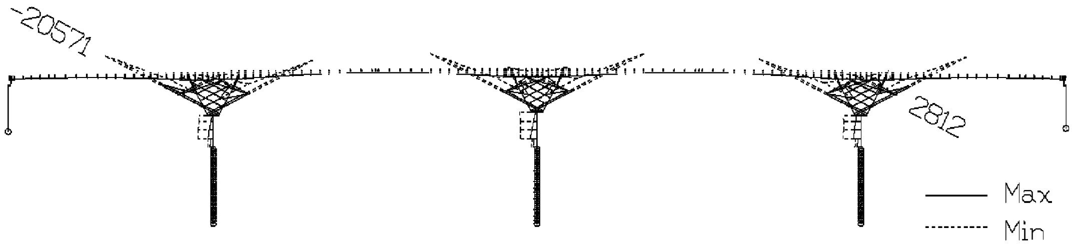
(a) Top Fiber



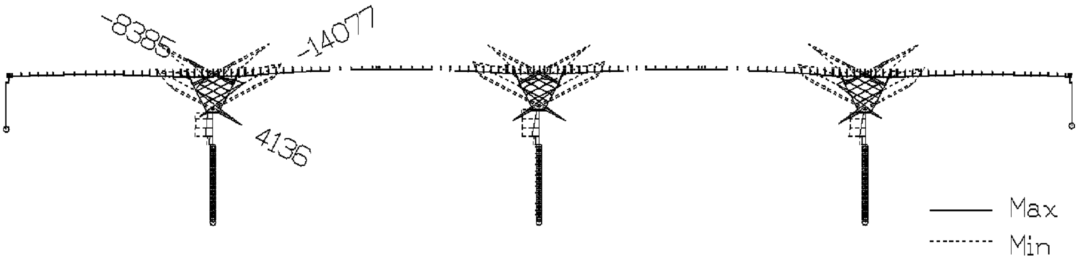
(b) Bottom Fiber

Source: Study Team

Figure 8.3.6-18 Fiber Stress (Service Limit State)



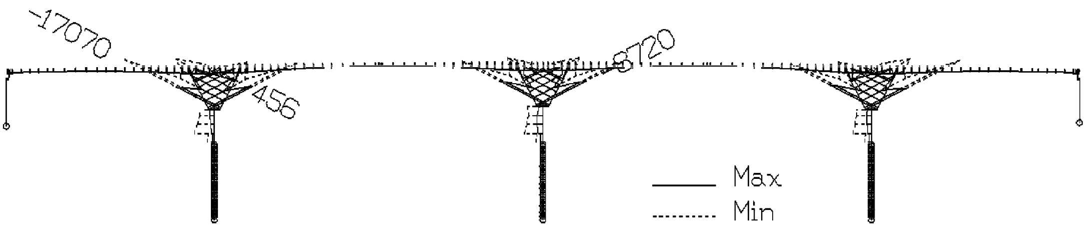
(a) Top Fiber



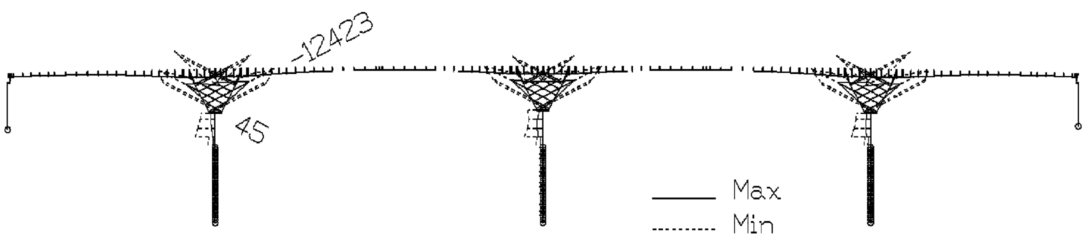
(b) Bottom Fiber

Source: Study Team

Figure 8.3.6-19 Fiber Stress (Strength Limit State)



(a) Top Fiber



(b) Bottom Fiber

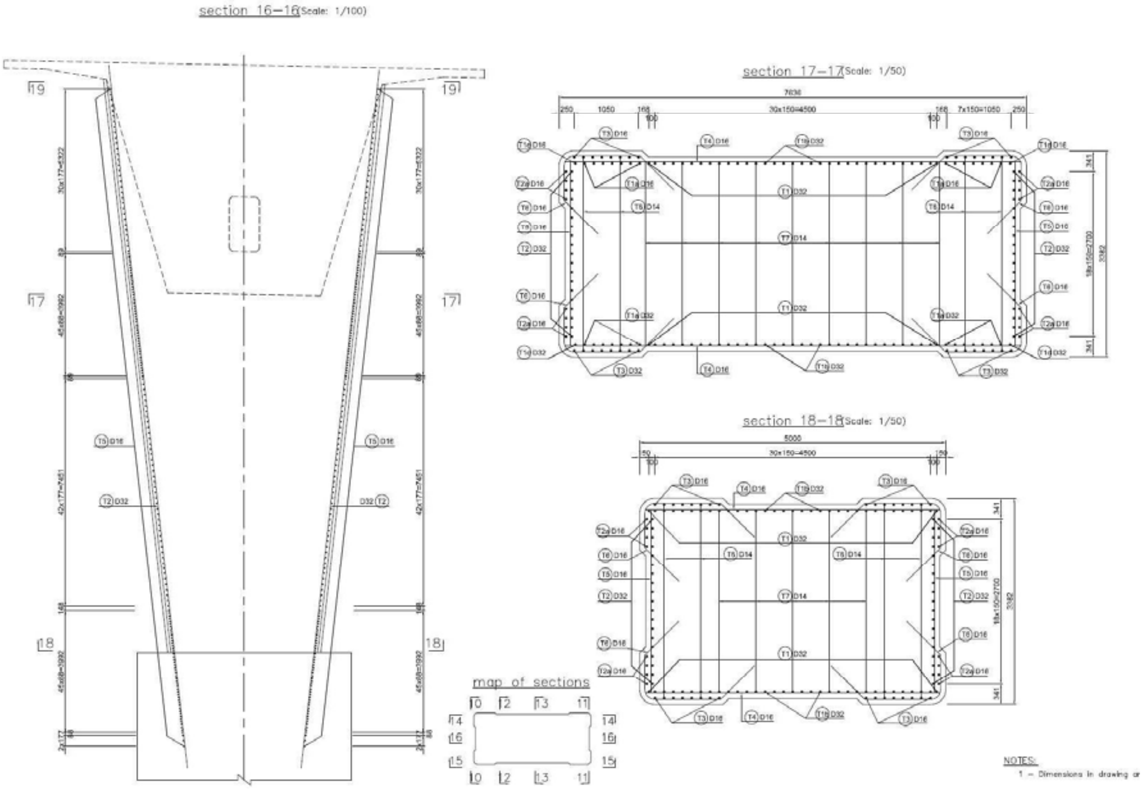
Source: Study Team

Figure 8.3.6-20 Fiber Stress (Extreme Event Limit State)

(4) Reinforcement of Piers

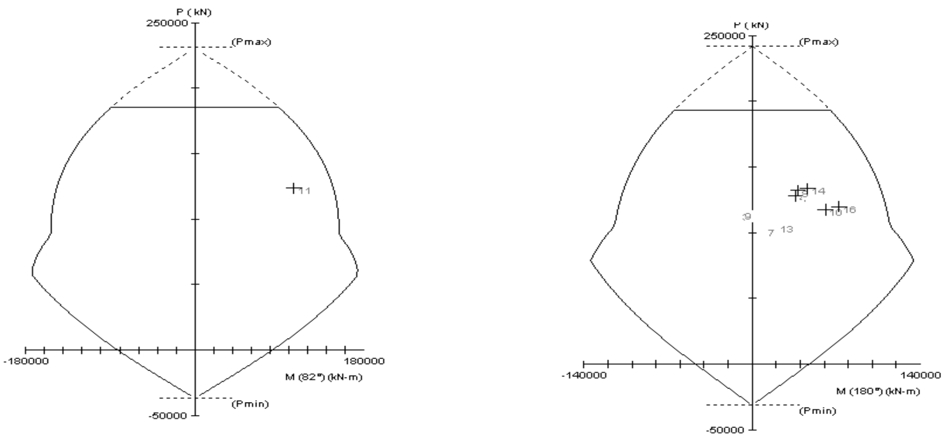
1) Reinforcement of V-shaped Wall

The reinforcement in the V-shaped wall is shown in the figure below. The relationships between the resistance capacity and sectional forces by unfavorable loading combinations in Strength Limit State are shown in the following figure.



Source: Study Team

Figure 8.3.6-21 Reinforcement of V-shaped Wall



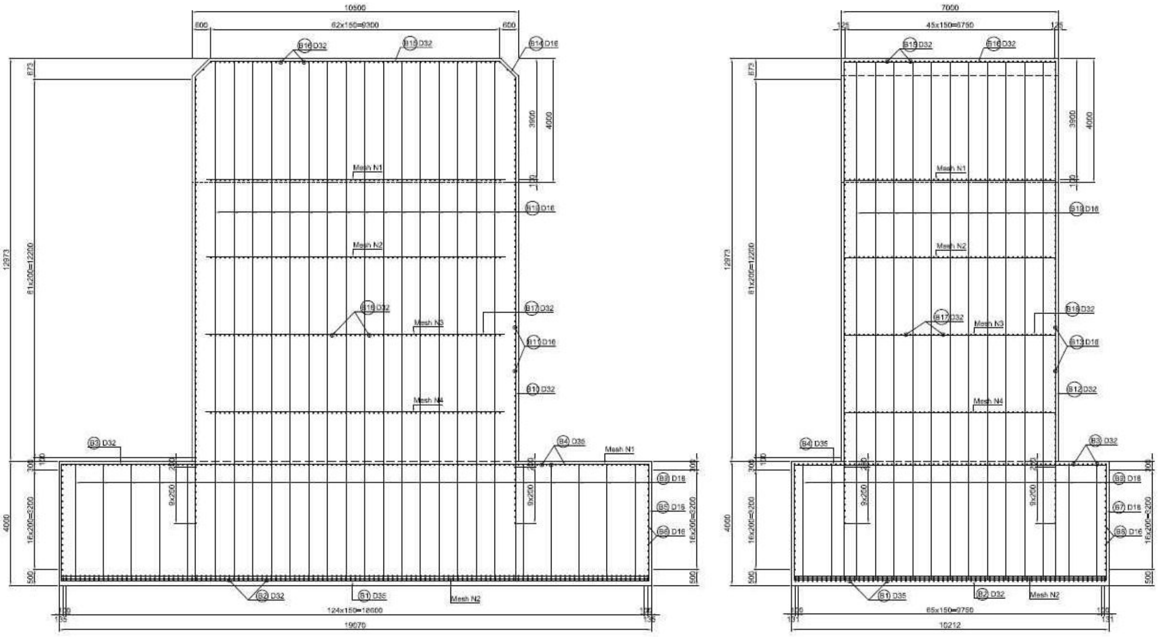
(a) Bottom Section of V-shaped Wall (b) Top Section of V-shaped Wall

Source: Study Team

Figure 8.3.6-22 Relationships between resistance capacity and sectional force in Pier Members

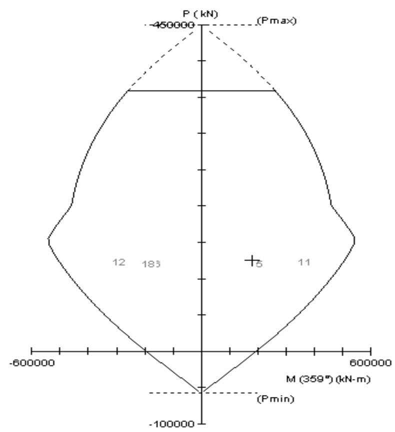
2) Reinforcement of Lower Pier Column

The reinforcement in the lower pier column is shown in the figure below. The relationships between the resistance capacity and sectional forces by unfavorable loading combinations in Strength Limit State are shown in the following figure.

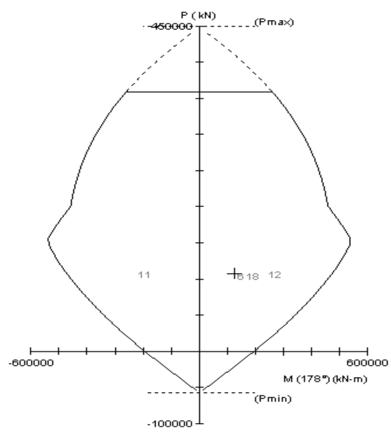


Source: Study Team

Figure 8.3.6-23 Reinforcement of Lower Pier Column and Pile Cap



(a) Bottom Section of Lower Pier Column



(b) Top Section of Lower Pier Column

Source: Study Team

Figure 8.3.6-24 Relationships between resistance capacity and sectional force in Pier Members

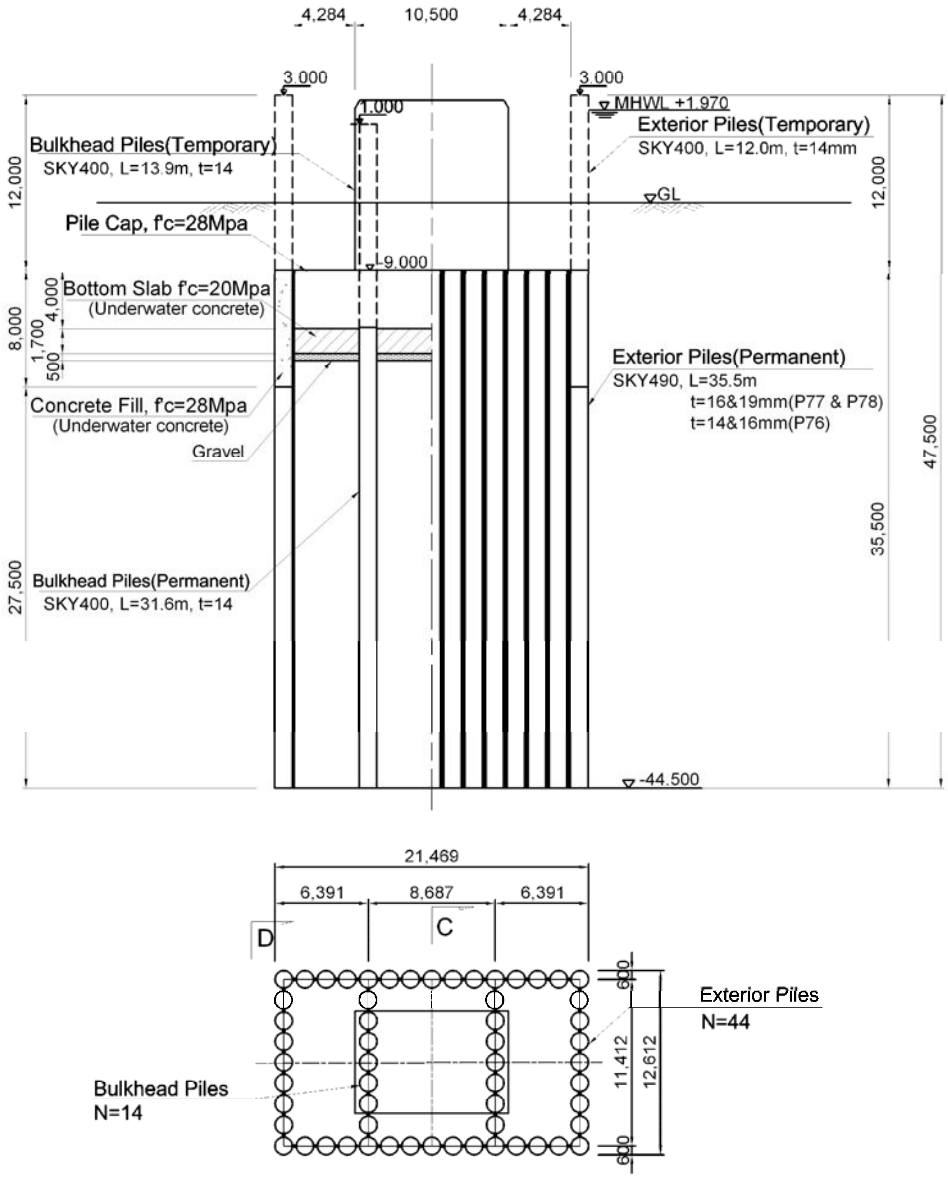
8.3.6.5 Design of Foundation of Main Bridge

(1) General

1) Design Results

Figure 8.3.6-1 shows major dimensions and materials of Steel sheet pipe pile foundations for P76~78 as design results. Dimensions and materials of steel pipe sheet pile for all foundation are same, only thickness of exterior pile at permanent part is different (t=19 for P77 & P78, t=16 for P76). Due to change of corrosion speed, the thickness of steel pipe pile will be changed at under the design scour depth (t=19 to 16 for P77 & P78, t=16 to 14 for P76).

Material of steel pipe sheet piles is SKY400 except exterior sheet pile at permanent part is SKY490.



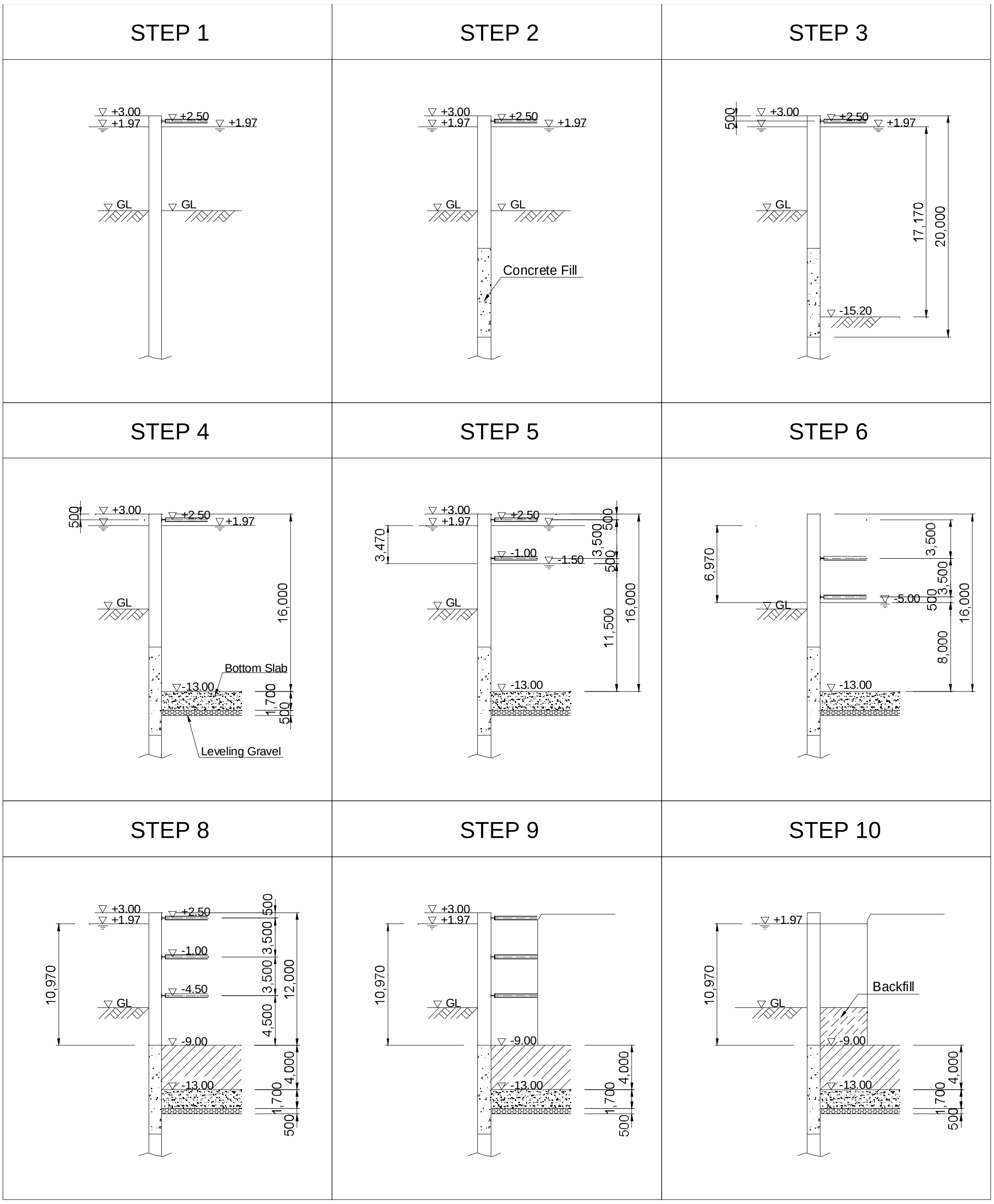
Source: Study Team

Figure 8.3.6-25 Major Dimension and Materials used for SPSP Foundation for P76~P78

2) Construction Step

The section design of external sheet pile, it was required to combine pile stress at during construction and at after construction. Therefore, in order to reduce residual stress in external sheet piles due to hydrostatic with soil pressure during construction, construction step was planed to cast the bottom slab concrete by keeping water inside cofferdam at +1.97m (assumed construction water level) at before the draining inside cofferdam.

In the scour depth at during construction, it was neglected due to safety design.



Source: Study Team

Figure 8.3.6-26 Plan of Construction Step

(2) Design Results

1) Design steel pipe sheet pile

A summary of calculation results for steel pipe sheet pile foundation is shown as following tables. And the stress diagrams of exterior sheet piles is shown as following figures.

<u>Longitudinal direction</u>

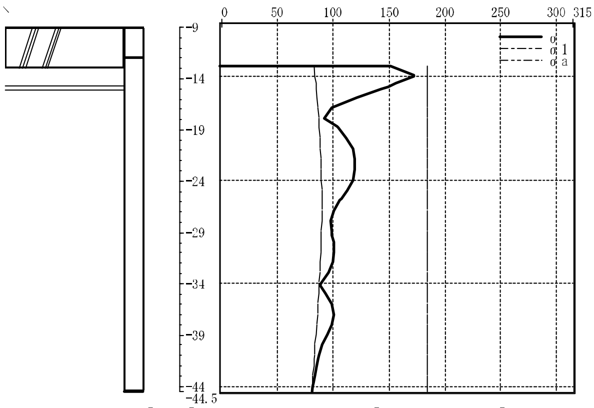
Table 8.3.6-9 Design Results of SPSP for Longitudinal Direction

Item		Symbol	Unit	P76 Ordinary	P76 Seismic	P77 Ordinary	P77 Seismic	P78 Ordinary	P78 Seismic
Forces*2		Vo	kN	127,238.5	126,156.5	126,828.0	122,009.0	120,699.7	121,049.7
		Ho	kN	5,311.0	22,663.0	1,798.0	19,890.0	7,759.0	21,844.0
		Mo	kN.m	52,921.0	252,181.0	7,783.0	162,160.0	49,478.0	220,995.0
Displacement*2									
At Top of Top Slab	Displacement	δ1	cm	0.762	2.524	0.207	1.785	1.160	2.510
	Allowable	δa	cm	5.000	5.000	5.000	5.000	5.000	5.000
Pile Bearing*2 (L=47.5m)									
Vertical Reaction	Max	Rmax	kN/pile	2,547	3,319	2,504	3,199	2,731	3,549
	Min	Rmin	kN/pile	2,088	1,277	2,355	1,476	1,893	1,089
	Bearing	Ra	kN/pile	3,446	5,169	3,046	4,569	3,520	5,353
	Pull-out	Pa	kN/pile	-926	-1,417	-1,193	-1,728	-939	-1512
Pile Stresses									
Exterior (SKY490)	Thickness	t	mm	16		19			
	After Construction	σ_1 *2	MPa	85.33	137.74	57.65	93.61	67.88	107.91
	During construction	σ_2	MPa	87.55	87.55	90.04	90.04	97.41	97.41
	Combined	σ_{max}	MPa	172.87	225.29	147.68	183.65	165.29	205.32
	Allowable	σa	MPa	185.00	280.00	185.00	280.00	185.00	280.00
Bulkhead*2 (SKY400) t=14mm	After Construction	σ_1	MPa	96.64	137.16	58.79	97.20	76.69	119.36
	Allowable	σa	MPa	140.00	210.00	140.00	210.00	140.00	210.00

*1:Designed by Well Model according

*2:due to after construction loads

Source: Study Team



Longitudinal Direction – Ordinary Condition

Figure 8.3.6-27 Stress Diagram of SPSP for P76

Transversal direction

Table 8.3.6-10 Design Results of SPSP for Transversal Direction

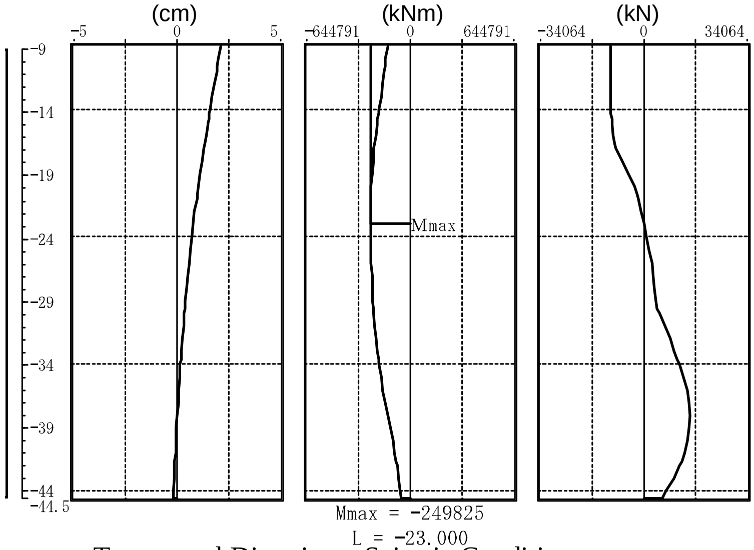
Item			Unit	P76		P77		P78	
				Ordinary	Seismic	Ordinary	Seismic	Ordinary	Seismic
Forces*2		Vo	kN	127,238.5	126,156.5	119914.0	122009.0	113,881.7	121,049.7
		Ho	kN	1,495.0	11,356.0	3027.0	20317.0	5,424.0	15,920.0
		Mo	kN.m	2,613.0	263,971.0	3742.0	484289.0	-7,314.0	331,268.0
Displacement*2									
At Top of Top Slab	Displacement	δ1	cm	0.174	1.819	0.344	4.080	0.753	3.274
	Allowable	δa	cm	5.000	5.000	5.000	5.000	5.000	5.000
Pile Bearing*2 (L=47.5m)									
Vertical Reaction	Max	Rmax	kN/pile	2,338	2,431	2347	2991	2,300	2,807
	Min	Rmin	kN/pile	2,296	2,164	2247	1684	2,064	1,831
	Bearing	Ra	kN/pile	3,446	5,169	3,046	4,569	3,520	5,353
	Pull-out	Pa	kN/pile	-870	-1,417	-1,193	-1,728	-939	-1512
Pile Stresses									
Exterior (SKY490)	Thickness	t	mm	16		19			
	After Construction	σ_1*2	MPa	71.67	132.92	55.01	148.57	52.69	120.78
	During construction	σ_2	MPa	92.39	92.39	95.69	95.69	103.08	103.08
	Combined	σ_{max}	MPa	164.06	225.31	150.70	244.26	155.77	223.86
	Allowable	σa	MPa	185.00	280.00	185.00	280.00	185.00	280.00
Bulkhead*2 (SKY400) t=14mm	After Construction	σ_1	MPa	87.96	136.93	57.23	142.13	61.00	124.35
	Allowable	σa	MPa	140.00	210.00	140.00	210.00	140.00	210.00

*1:Designed by Well Model according

*2:due to after construction loads

*:Designed by Well Model according

Source: Study Team



Transversal Direction – Seismic Condition

Figure 8.3.6-28 Calculation results of Steel Pipe Sheet Pile for P7

2) Design of top slab

A summary of calculation results for Top Slab is shown as following tables.

<u>Longitudinal direction</u>

b = 100.0 (cm), h = 400.0 (cm)

The lower tensile(As = 228.000 (cm^2))

3layers reinforcement cover, 130 (mm) D38 @ 150

The upper tensile(As = 76.000 (cm^2))

1layer reinforcement cover, 100 (mm) D38 @ 150

Table 8.3.6-11 Design Results of Top Slab for Longitudinal Direction

				P76		P77		P78	
			Unit	Ordinary	Seismic	Ordinary	Seismic	Ordinary	Seismic
Lower tensile	Bending moment	MA	kN.m	6934.0	10072.0	6857.0	9339.0	7529.0	10284.0
	Necessary reinforcement	Asr	cm^2	115.856	00.327	129.561	92.717	142.920	102.531
	Neutral axis	x	cm	130.0	30.0	130.0	130.0	130.0	130.0
	Stresses	σc σs	N/mm^2 N/mm^2	3.20 94.74	4.64 137.61	3.16 93.69	4.30 127.61	3.47 102.86	4.74 140.51
	Resultant tensile force Reinforcement requirements	T As	kN cm^2	3081.7 171.203	476.3 149.209	3047.6 190.474	4150.8 138.361	3346.0 209.126	4570.5 152.349
Upper tensile	Bending moment	MA’	kN.m	5166.0	1901.0	6492.0	3406.0	5565.0	2854.0
	Necessary reinforcement	Asr	cm^2	0.000	0.000	0.000	0.000	0.000	0.000
	Neutral axis	x	cm	7.5	7.5	7.5	7.5	7.5	7.5
	Stresses	σc σs	N/mm^2 N/mm^2	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.00
	Resultant tensile force Reinforcement requirements	T As	kN cm^2	0.0 0.000	0.0 0.000	0.0 0.000	0.0 0.000	0.0 0.000	0.0 0.000
Allowable stresses		σca σsa	N/mm^2 N/mm^2	9.00 180.00	14.00 300.00	9.00 160.00	14.00 300.00	9.00 160.00	14.00 300.00
Average shearing force		QB τm τa1’	kN N/mm^2 N/mm^2	1676.0 0.44 0.72	2330.0 0.62 1.08	1614.0 0.43 0.72	2128.0 0.56 1.08	1715.0 0.46 0.72	2303.0 0.61 1.08
Average shearing force		S τm τa1’	kN N/mm^2 N/mm^2	1801.0 0.48 0.72	517.0 0.67 1.08	1692.0 0.45 0.72	2237.0 0.59 1.08	1838.0 0.49 0.72	2469.0 0.65 1.08
Shearing force carried by concrete		Sca	kN	2714.0	4071.0	2714.0	4071.0	2714.0	4071.0
Diagonal tension reinforce ment	Shearing force	Sh’	kN	0.0	0.0	0.0	0.0	0.0	0.0
	Longitudinal spacing	s	cm	90.0	90.0	90.0	90.0	90.0	90.0
	Reduction coefficient	Cds	—	0.455	0.455	0.455	0.455	0.455	0.455
	Allowable tensile stresses	σsa	N/mm^2	180.00	300.00	160.00	300.00	160.00	300.00
	Used reinforcement	Aw	cm^2	2.207	2.207	2.207	2.207	2.207	2.207
	Necessary reinforcement	Awreq	cm^2	0.000	0.000	0.000	0.000	0.000	0.000

Source: Study Team

Transversal direction

b = 100.0 (cm) , h = 400.0 (cm)

The lower tensile(As = 158.840 (cm^2))

layer 1~3 reinforcement cover, 168 (mm), D32 @ 150

The upper tensile(As = 52.947 (cm^2))

layer 1 reinforcement cover, 100 (mm), D32 @ 150

Table 8.3.6-12 Design Results of Top Slab for Transverse Direction

				P76		P77		P78	
			Unit	Ordinary+W	Seismic	Ordinary	Seismic	Ordinary	Seismic
Lower tensile	Bending moment	MA	kN.m	4610.0	6008.0	3624.0	8130.0	3485.0	6800.0
	Necessary reinforcement	Asr	cm^2	60.707	59.282	67.278	81.083	64.616	67.349
	Neutral axis	x	cm	111.7	111.7	111.7	111.7	111.7	111.7
	Stresses	σc	N/mm^2	2.46	3.20	1.93	4.32	1.85	3.62
		σs	N/mm^2	89.61	116.77	70.38	157.92	67.69	132.09
	Resultant tensile force	T	kN	2049.0	2670.1	1610.5	3613.3	1548.8	3022.3
	Reinforcement requirements	As	cm^2	91.068	89.002	100.653	120.443	96.797	100.744
Upper tensile	Bending moment	MA’	kN.m	2510.0	1045.0	3368.0	-1003.0	3253.0	400.0
	Necessary reinforcement	Asr	cm^2	0.000	0.000	0.000	8.804	0.000	0.000
	Neutral axis	x	cm	7.0	7.0	7.0	71.1	7.0	7.0
	Stresses	σc	N/mm^2	0.00	0.00	0.00	0.77	0.00	0.00
		σs	N/mm^2	0.00	0.00	0.00	51.72	0.00	0.00
	Resultant tensile force	T	kN	0.0	0.0	0.0	445.8	0.0	0.0
	Reinforcement requirements	As	cm^2	0.000	0.000	0.000	14.860	0.000	0.000
Allowable stresses		σca	N/mm^2	10.00	14.00	9.00	14.00	9.00	14.00
		σsa	N/mm^2	205.00	300.00	160.00	300.00	160.00	300.00
Average shearing force		QB	kN	1923.0	2496.0	1538.0	3416.0	1470.0	2850.0
		τm	N/mm^2	0.52	0.67	0.41	0.91	0.39	0.76
		τa1’	N/mm^2	1.43	1.72	1.15	1.72	1.15	1.72
Average shearing force		S	kN	1923.0	2496.0	1538.0	3416.0	1470.0	2850.0
		τm	N/mm^2	0.52	0.67	0.41	0.91	0.39	0.76
		τa1’	N/mm^2	1.43	1.72	1.15	1.72	1.15	1.72
Shearing force carried by concrete		Sca	kN	4281.0	6421.0	4282.0	6423.0	4282.0	6423.0
Diagonal tension reinforcement	Shearing force	Sh’	kN	0.0	0.0	0.0	0.0	0.0	0.0
	Longitudinal spacing	s	cm	90.0	90.0	90.0	90.0	90.0	90.0
	Reduction coefficient	Cds	—	0.181	0.181	0.181	0.181	0.181	0.181
	Allowable tensile stresses	σsa	N/mm^2	225.00	300.00	160.00	300.00	160.00	300.00
	Used reinforcement	Aw	cm^2	2.207	2.207	2.207	2.207	2.207	2.207
	Necessary reinforcement	Awreq	cm^2	0.000	0.000	0.000	0.000	0.000	0.000

Source: Study Team

1) Design of connection between Top Slab and Steel Pipe Sheet Pile

A summary of calculation results for connection between Top Slab and Steel Pipe Sheet Pile is shown as following tables.

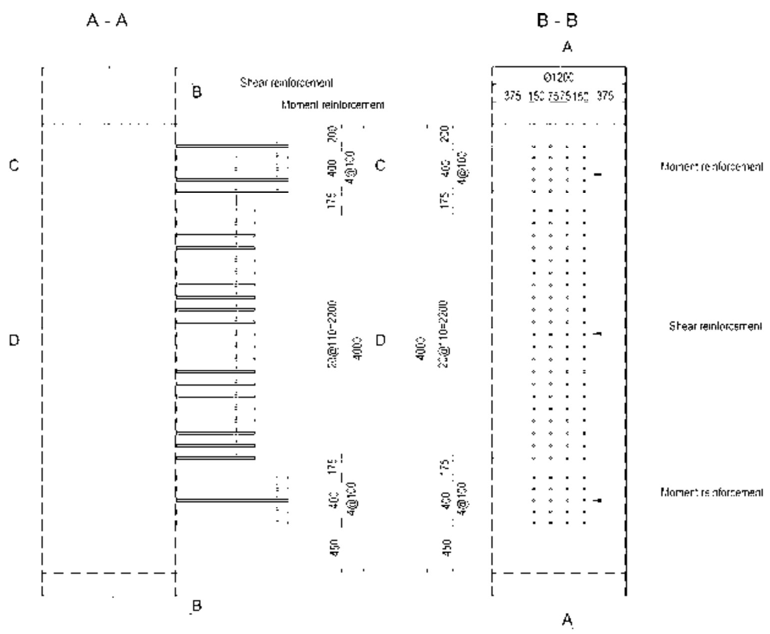
Design condition

- Type of steel : SS400,SM400
- Type of reinforcing bars : SD345（underwater）
- Design strength of concrete : $\sigma ck = 27\ (N/mm^2)$ *Treated as equivalent to an C28
- Material of sheet pile : SKY490
- Diameter of sheet pile : D = 1200.0 (mm)
- Section modulus of sheet pile : $Z = 12859.9\ (cm^3)$
- Connection method : reinforcement stud welding

Table 8.3.6-13 Design Results connection between Top Slab and SPSP

	Load case	σs1 (N/mm^2)	σs2 (N/mm^2)	σs (N/mm^2)	σsa (N/mm^2)	nb nba (nos/layer)	τs (N/mm^2)	τsa (N/mm^2)	ns nsa (nos)
P76	Ordinary	78.72	7.81	86.54	160.00	20 ≧ 11	74.61	96.00	84 ≧ 65
P77	Wind	160.65	15.82	176.48	200.00	20 ≧ 18	93.89	120.00	84 ≧ 66
P78	Ordinary	104.17	11.37	115.53	160.00	20 ≧ 15	76.39	96.00	84 ≧ 67

Source: Study Team



Source: Study Team

Figure 8.3.6-29 Detail of connection between Top Slab and SPSP