8.4 Design of Approach Bridge

8.4.1 Study on Structure of Approach Bridge

8.4.1.1 Selection of Erection Method for Approach Bridge

(1) Comparative Study

1) Conditions of Study

In this project, as discussed in the report of JICA's Preparatory Survey (SAPROF Study), the construction period is planned as short as 32 months for aiming at opening in 2015 and the construction period of Approach Bridge which is approximately 5km long is critical. Therefore, in this study, two alternatives of erection method, SBS and MSS, which are superior in construction period, are introduced to be compared as summarized below. The other conventional erection methods, such as cast-in-situ cantilever method and cast-in-situ all-staging method, are not included because of their long construction period.

- Alternative-1: Span-by-Span Erection Method with
  25 spans@60.0m x 3 erection girders = 4,500.0m
- Alternative-2: Moving Scaffolding System with
  22 spans@50.0m x 2 erection girders + 23 spans @ 50m x 2 erection girders
  = 4,500.0m

The span length of SBS Method is defined as 60.0m as recommended by SAPROF Study.as recommended in Chapter 8.2.2 and agreed in the Notice No. 107/TB-TCDBVN. The span length of MSS Method is defined as 50.0m, which is the longest in the method

The number of the erection girders for SBS Method is defined as 3, by which 16 months of erection period of 4.5km long bridge can be realized.

The number of the erection girders for MSS Method is defined as 4, by which equivalent erection period can be realized for comparison of the construction cost with SBS method.

2) Results of Study

a) Results of Comparative Study

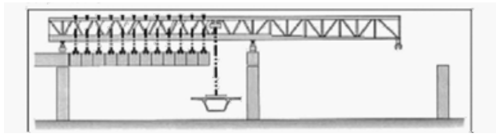
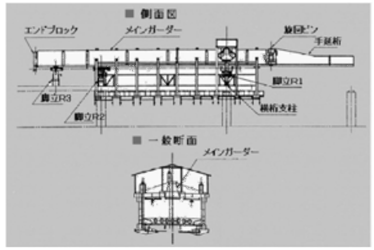
The table on the next page shows the result of the comparative study.

As shown in the table, results of comparison reveal that the SBS method is more preferable for the following reasons;

- Segment preserved at the factory can have well-managed quality, which is favorable for costal condition,
- Construction period is the shorter with 3 erection girders, while 4 erection girders are introduced for comparison in MSS Method.

The detailed construction plan for the erection including SBS method is now under preparation and will be presented later.

Table 8.4.1-1 Comparison on Erection Method for Approach Bridge

Evaluation Items		Alternative-1: Span-By-Span Method A1 ~ P75 (25@60.0m x 3 girders = 4,500.0m)		Alternative-2: Movable Scaffolding System A1 ~ P90 (22@50m x 2 + 23@50m x 2 = 4,500.0m)	
Photos and Schematics		  		 	
Structural Aspect and stability	10	Nominal strength of concrete: σck = 50MPa , better quality for strength and anti-corrosion Longer Span Length (60m) Weight of Erection Girder : 811 ton (refer to Second Tomei Expressway in Japan) Applicable for longitudinal slope Girder concrete volume: Vc = 46,697m3 (4,500m) (Due to using external cable, concrete volume is not so huge compared with Alternative 2.)	10	Nominal strength of concrete: σck = 40MPa Shorter Span Length (Maximum 50m) Weight of Erection Girder: 950 ton (refer to Second Tomei Expressway in Japan) Not applicable for longitudinal slope more than 2% (KM 8+700 - Km 8+950) Girder concrete volume: Vc = 52,853m3 (4,500m) (Due to coupling PC tendons, dimension of web becomes thicker than Alternative 1and concrete volume is huge.)	4
Construction Cost	40	Bearings, Sub-structure and Foundations: A1 ~ P75 : 76 nos. Necessary Number of Erection Girder: 3 Cost and construction period can be saved compared with Alternative 2. Main Girder: VND 16,144,000 x 46,697 = 753,876,368 Fabrication yard: VND 250,000,000 Erection Girder: VND 101,375,000,000 x 3 x 0.9 = 273,712,500 Pier: VND 633,729,000 x 75 = 47,529,675 Foundation: VND 17,519,488,000 x 75 = 1,313,961,600 Total: (1000 VND) 2,639,080,143 1.00	40	Bearings, Sub-structure and Foundation: A1 ~ P90: 91 nos. Necessary Number of Erection Girder: 4 Cost for bearings, sub-structure and foundation is bigger than Alternative 1. Construction period is also longer than Alter Main Girder: VND 16,278,000 x 52,853 = 860,341,134 Temporary Yard: VND 25,000,000 Erection Girder: VND 118,750,000,000 x 4 x 0.9 = 427,500,000 Pier: VND 570,356,000 x 90 = 51,332,040 Foundation: VND 15,767,539,000 x 90 = 1,419,078,528 Total: (1000 VND) 2,783,251,702 1.05	32
Construction Plan and Period	10	Order and fabrication of erection girder requires 1 year. After assembling erection girder, pre-cast segment shall be transported under the erection girder and lifted one by one. Then, after completion of lifting one span whole segments, pre-stressing force is given by external cables. Utilizing precast segments, the quality is well controlled in the segment fabrication, which is preferable for costal condition. The construction period (16.0 months) is shorter than Alternative 2. 60 days for assembling 15 days for 1 span x 25 = 375 days 45 days for removal It requires pre-cast segment yard for fabrication and stock of segment.	10	Order and fabrication of erection girder requires 1 year. After assembling erection girder including steel form work, arrangement of reinforcing bars and PC tendons shall be executed as same as usual cast-in-situ concrete girder. After placing concrete, curing and pre-stressing work shall be executed. The quality control for girder fabrication is similar to cast-in-situ concrete girder. The construction period (18.9 months) is longer than Alternative 1. 60 days for assembling 20 days for 1 span x 23 = 460 days 45 days for removal Placing concrete is executed on the form work hanged by erection girder.	8
Maintenance	15	The number of necessary items for periodical maintenance, such as bearings and expansion joints, is smaller	12	The number of necessary items for periodical maintenance, such as bearings and expansion joints, is larger	9
STEP Clearance	10	56% (Preliminary Estimate)	10	58% (Preliminary Estimate)	10
Aesthetics	5	Slender appearance with long span	4	Ordinary appearance	3
New Technology	5	SBS is New Technology in Vietnam (first time in Vietnam)	5	MSS is New Technology in Vietnam (there is one experience in Thanh Tri Bridge)	5
Environmental Impact	5	No major difference in Environmental Impact	4	No major difference in Environmental Impact	4
Evaluation	100	**Most Recommended.** This alternative is superior in construction period and quality control owing to utilization of precast segments.	95	Less recommended. This alternative is inferior in construction period and quality control comparing to Alternative 1.	75

Source : Study Team

(2) Comparative Study (2)

1) Conditions of Study

In this project, two (2) alternatives of erection method, SBS and MSS, which are superior in construction period, were introduced to be compared as summarized below.

- Alternative-1: Span-by-Span Erection Method with

  25 spans@60.0m x 3 erection girders = 4,500.0m

- Alternative-2: Moving Scaffolding System with

  22 spans@50.0m x 2 erection girders + 23 spans @ 50m x 2 erection girders = 4,500.0m

The span length of SBS Method is defined as 60.0m as recommended by SAPROF Study.as recommended in Chapter 8.2.2 and agreed in the Notice No. 107/TB-TCDBVN. The span length of MSS Method is defined as 50.0m, which is the longest in the method

The number of the erection girders for SBS Method is defined as 3, by which 16 months of erection period of 4.5km long bridge can be realized.

The number of the erection girders for MSS Method is defined as 4, by which equivalent erection period can be realized for comparison of the construction cost with SBS method.

In addition to the above two (2) alternatives comparison, PMU-2 requested JICA Study Team to study other alternatives, which are like cantilever method and something like this. Therefore, the following two (2) alternatives were added for comparison study:

- 60m span length with P&Z
- 60m span length with ordinary cantilever method

2) Results of Comparative Study

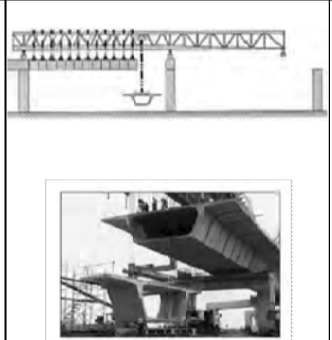
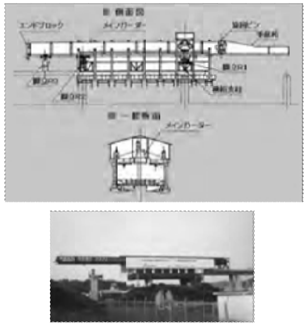
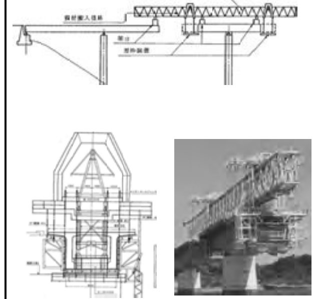
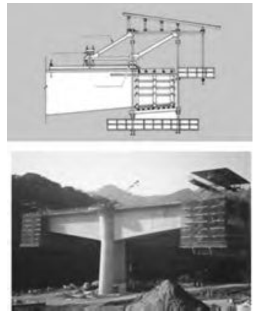
The table on the next page shows the result of the comparative study.

As shown in the table, results of comparison reveal that the SBS method is still more preferable for the following reasons;

- Segment preserved at the factory can have well-managed quality, which is favorable for costal condition,
- Construction period is the shorter with 3 erection girders, while 4 erection girders are introduced for comparison in MSS Method and other two (2) additional alternatives.

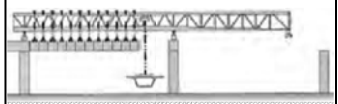
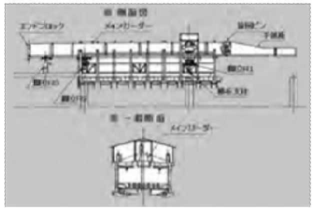
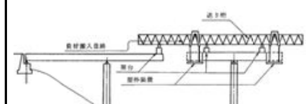
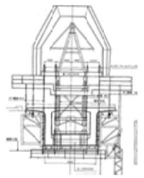
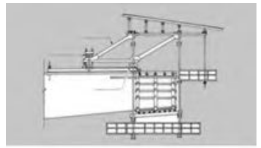
The detailed construction plan for the erection including SBS method is shown in Chapter 8.4.1.1 (3).

Table 8.4.1-2 Comparison on Erection for Approach Bridge (A1-P75)

Evaluation Items		Alternative-1 Span by Span Erection Method		Alternative-2 Moving Scaffolding System		Alternative-3 P&Z Election Method		Alternative-4 Cantilever Consturuction	
Length of Bridge		4,433.7m		4,433.7m		4,433.7m		4,433.7m	
Schematics and Photos									
Structural Aspect and Stability	10	•Pre-cast segment is fabricated at yard, then they are transported to the erection site. Segment is placed lifted and placed by SBS erection girder one span by one span. •There are a lot of achievemnt in Japan and other countries.	10	•Form is supported by MSS erection girder, and one span girder is construted by cast-in-situ construction method. •Thanh Tri Bridge is only one achievement in Vietnam. •Max span is 50m due to scale (weight) of erection girder.	10	•P&Z method is one kind of cantilever erection methods with long segment (max 10m). After construction of pier table, cantilever will start as same as ordinary cantilever construction method. •This was developed in Germany and transferred to Japan in 1995. There are numbers of achievement in Japan. (Max span is 100m.)	10	•After construction of pier table, travelers are fabricated and then balanced cantilever erection is statrted with cast-in-situ construction method. •This is most popular construction method for long span (more than 60m), and there are a lot of achievements all over the world.	10
Construction Cost	40	Superstructure: 1,086,118,000,000 VND Erection girder: 273,712,500,000 VND Fabrication Yard: 250,000,000,000 VND Substructure: 1,361,491,275,000 VND Total: 2,971,321,775,000 VND Ratio: 1.11	32	Superstructure: 1,221,955,000,000 VND Erection girder: 427,500,000,000 VND Fabrication Yard: 25,000,000,000 VND Substructure: 1,470,410,568,000 VND Total: 3,144,865,568,000 VND Ratio: 1.17	24	Superstructure: 1,063,768,000,000 VND Erection girder: 503,100,000,000 VND Fabrication Yard: 25,000,000,000 VND Substructure: 1,361,491,275,000 VND Total: 2,953,359,275,000 VND Ratio: 1.10	32	Superstructure: 1,063,708,000,000 VND Traveler: 234,000,000,000 VND Fabrication Yard: 25,000,000,000 VND Substructure: 1,361,491,275,000 VND Total: 2,684,199,275,000 VND Ratio: 1.00	40
Construction Plan and Period	10	From order of erection girder to installation of girder at the site takes 1 year. •In A1～P75, 3 nos. of SBS girder are necessary to meet the requirement from construction period. It takes 480 days exclusive order & transportation of erection girder (365 days). This required period is shortest among 4 alternatives (845 days)	10	From order of erection girder to installation of girder at the site takes 1 year. •In A1～P75 (4,423.7m), 4 nos. of MSS girder are necessary to meet the requirement from construction period. It takes 565 days exclusive order & transportation of erection girder (365 days). This required period (930 days) is longer than Alternative 1(845 days)	8	From order of erection girder to installation of girder at the site takes 1 year. •In A1～P75 (4,423.7m), 8 nos. of P&Z girder are necessary to meet the requirement from construction period. It takes 506 days for 9 numbers of span exclusive order & transportation of erection girder (365 days). This required period is 29 months (871 days) and longer than Alternative 1 & 4.	8	Due to wide width, large scale traveler is required. In Japan, there are a lot of this type and easy to procure them in short time (5 months). However, sub-structure requires at least 6 months for first group completion. •In A1～P75 (4,423.7m), 32 nos. of traveler are necessary to meet the requirement from construction period. It takes 590 days for 10 numbers of span. This required period is 6 months for sub-structures and 24.7 months for superstructures. (Sub-structure will be constructed in order and cannot be constructed at the same time.) Total period is 30.7 months (921 days). This alternative is not realistic after considering sub-structure schedule.	4
Maintenance	15	•This is similar to Alternative 3 & 4 due to same span arrangement.	15	•Due to shorter span (50m), the numbers of pier & bearing shoes increase.	9	•This is similar to Alternative 1 & 4 due to same span arrangement.	15	•This is similar to Alternative 1 & 3 due to same span arrangement.	15
STEP Clearance	10	56% (Preliminary Estimate)	10	58% (Preliminary Estimate)	10	64% (Preliminary Estimate)	10	44% (Preliminary Estimate)	8
Aesthetics	5	•This is similar to Alternative 3 & 4 due to same span arrangement.	5	•Due to large numbers of piers copared with Alternative 1, 3, & 4, this alternative is not superior to the others.	3	•This is similar to Alternative 1 & 4 due to same span arrangement.	5	•This is similar to Alternative 1 & 3 due to same span arrangement.	5
New Technology	5	•First case in Vietnam.	5	•There is one achievement in Vietnam. (Thanh Tri Bridge)	4	•First case in Vietnam.	5	•There are many achievements in Vietnam.	2
Environmental Impact	5	•There is no difference between Alternative 1,3 & 4.	5	•Due to increased number of pier, noise at expansion joint is louder than the other alternatives.	3	•There is no difference between Alternative 1,3 & 4.	5	•There is no difference between Alternative 1,3 & 4.	5
Evaluation	100	Most Recommended •Construction period is shortest among 4 alternatives. After considering this point and cost, this alternaive is most recommendable.	92	Less Recommended •Construction period is longer than Alternative 1 & 3 and scale of ercton girder is very big and it costs much more than Alternative 1.	71	Less Recommended • From the economic & construction speed view points, this alternative is secondly recommendable among 4 alternatives.	90	Less Recommended • This alternative is not recommendable from the view point of construction schedule.	89

Source : Study Team

Table 8.4.1-3 Comparison on Erection for Approach Bridge (P79-A2)

Evaluation Items		Alternative-1 Span by Span Erection Method		Alternative-2 Moving Scaffolding System		Alternative-3 P&Z Election Method		Alternative-4 Cantilever Consturuction	
Length of Bridge		54.8+3@60+54.8m, 54.8+2@60+54.8m =519.2m		42.3+4@50+42.3m, 42.3+3@50+42.3m =519.2m		54.8+3@60+54.8m, 54.8+2@60+54.8m =519.2m		54.8+3@60+54.8m, 54.8+2@60+54.8m =519.2m	
Schematics and Photos		 		 		  		 	
Structural Aspect and Stability	10	▪ Pre-cast segment is fabricated at yard, then they are transported to the erection site. Segment is placed lifted and placed by SBS erection girder one span by one span. ▪ There are a lot of achievemnt in Japan and other countries.	10	▪ Form is supported by MSS erection girder, and one span girder is construted by cast-in-situ construction method. ▪ Thanh Tri Bridge is only one achievement in Vietnam. ▪ Max span is 50m due to scale (weight) of erection girder.	10	▪ P&Z method is one kind of cantilever erection methods with long segment (max 10m). After construction of pier table, cantilever will start as same as ordinary cantilever construction method. ▪ This was developed in Germany and transferred to Japan in 1995. There are numbers of achievement in Japan. (Max span is 100m.)	10	▪ After construction of pier table, travelers are fabricated and then balanced cantilever erection is statrted with cast-in-situ construction method. ▪ This is most popular construction method for long span (more than 60m), and there are a lot of achievements all over the world.	10
Construction Cost	40	Superstructure: 189,675,750,687 VND (Including erection girder) Substructure: 141,736,360,000 VND Total: 331,412,110,687 VND Ratio: 1.20	24	Superstructure: 209,356,995,996 VND (Including erection girder) Substructure: 177,170,450,000 VND Total: 386,527,445,996 VND Ratio: 1.40	16	Superstructure: 163,560,590,992 VND (Including erection girder) Substructure: 141,736,360,000 VND Total: 305,296,950,992 VND Ratio: 1.11	32	Superstructure: 134,166,608,228 VND (Including traveler) Substructure: 141,736,360,000 VND Total: 275,902,968,228 VND Ratio: 1.00	40
Construction Plan and Period	10	From order of erection girder to installation of girder at the site takes 1 year. ▪ In P79～A2 (519.2m), construction period is shortest among 4 alternatives (270 days) and no influence in the total construction period. ▪ Erection girder weight is approximately 815 ton, and total ▪ It is required to carefully considered in transportation of segment.	10	From order of erection girder to installation of girder at the site takes 1 year. ▪ In P79～A2 (519.2m), construction period is 355 days. It is longer than Alternative 1. ▪ Erection girder weight is approximately 950ton for 50m span length. If roof and other facility are added, construction is available in any weather condition. And total numbers of are 5.	4	From order of erection girder to installation of girder at the site takes 1 year. ▪ In P79～A2 (519.2m), construction period is longest among 4 alternatives. (506 days) ▪ Erection girder weight is approximately 559 ton.	4	Due to wide width, large scale traveler is required. In Japan, there are a lot of this type and easy to procure them in short time (5 months). However, sub-structure requires atleast 6 months for first group completion. ▪ In P79～A2,If 4 numbers of travelers are set, the construction period is 464 days. This period is not critical pass for total construction period. (Critical pass in schedule is in Hai An side.) ▪ Traveler's weight is 117 ton X 32 = 3,744 ton(Hai An) ▪ Traveler's weight is 117 ton X 4 = 468 ton(Cat Hai)	4
Maintenance	15	▪ This is similar to Alternative 3 & 4 due to same span arrangement.	15	▪ Due to shorter span (50m), the numbers of pier & bearing shoes increase.	9	▪ This is similar to Alternative 1 & 4 due to same span arrangement.	15	▪ This is similar to Alternative 1 & 3 due to same span arrangement.	15
STEP Clearance	10	65% (Preliminary Estimate)	10	68% (Preliminary Estimate)	10	60% (Preliminary Estimate)	10	44% (Preliminary Estimate)	8
Aesthetics	5	▪ This is similar to Alternative 3 & 4 due to same span arrangement.	5	▪ Due to large numbers of piers copared with Alternative 1, 3, & 4, this alternative is not superior to the others.	3	▪ This is similar to Alternative 1 & 4 due to same span arrangement.	5	▪ This is similar to Alternative 1 & 3 due to same span arrangement.	5
New Technology	5	▪ First case in Vietnam.	5	▪ There is one achievement in Vietnam. (Thanh Tri Bridge)	4	▪ First case in Vietnam.	5	▪ There are many achievements in Vietnam.	2
Environmental Impact	5	▪ There is no difference between Alternative 1,3 & 4.	5	▪ Due to increased number of pier, noise at expansion joint is louder than the other alternatives.	3	▪ There is no difference between Alternative 1,3 & 4.	5	▪ There is no difference between Alternative 1,3 & 4.	5
Evaluation		Less Recommended ▪ Construction period is shortest among 4 alternatives.	84	Not Recommended ▪ Construction cost is high and period is long.	59	Less Recommended ▪ Construction period is longtest among 4 alternatives.	86	Most Recommended ▪ Cheepest. Construction period is longer than Alternative-1, but the section of P79-A2 is not on the critical path.	89

Source : Study Team

(3) BACK UP DATA

The basic concept was established as follows:

1. The priority first is to make the construction period shorter in order to meet the requirements which has been described in SAPROF Study and the statement of Vice Minister of Ministry of Transport at the kick off meeting in MOT on 23rd March 2011.

2. In order to make the construction period shorter, there are various construction methods can be considered. However, most appropriate way is utilization of innovated construction method in large scale bridge such as this project with same span & repeated construction method. Under this condition, SAPROF Study has selected two alternatives, which were SBS and MSS. In both construction methods, extra size erection girder is required. This extra erection girder and other facilities require numbers of span from economic view point due to a huge amount of initial cost.

   The difference between SBS and MSS is mainly applied span length and fabrication yard. In Japan, SBS is adopted up to 70m, and MSS is adopted in from 25m to 35m span length for PC Hollow slab and PC Double-T girder mainly. However, in some case of MSS, maximum span length is 50m as shown below. ("Nanairo Viaduct" with usage of external tendons in Japan) SBS needs fabrication & stock yard for segments, but construction speed is faster than MSS. MSS required heavier erection girder than SBS because erection girder and steel form are in one unit.

3. Another important aspect for selection of optimized construction method is how to harmonize with site conditions like span arrangement at intersection and etc. If any span arrangement with same span length does not fit to the site conditions, it shall be disqualified. Therefore, the following two 2) numbers of standard span lengths were selected:

   - 60m span length with SBS
   - 50m span length with MSS

4. In addition to the above, PMU-2 requested JICA Study Team to study another possibilities, which are like cantilever method and something like his. Therefore, the following two (2) alternatives were added for comparison study:

   - 60m span length with P&Z
   - 60m span length with ordinary cantilever method

   As the results, the three 3) alternatives technical data were re-edited for reference in the next pages.

1) SBS

a) Results of reviewing technical samples

In this Design Study, the following reference papers were prepared as a part of Discussion Paper No.8:

- Construction record
  - DVD: Second Tomei Expressway Furukawa Viaduct constructed by SBS
  - DVD: Second Tomei Expressway Kiso & Ibi River construction by segmental method & steel pipe sheet pile foundation
  - Project outline of Second Tomei Expressway Kiso & Ibi River construction (super & sub-structure pamphlet)

- Some Construction Project Reports on Segmental Construction Method (Vietnamese)
  - Yatomi Viaduct at Second Tomei Expressway
  - Shigenobu Viaduct at East Matsuyama Expressway
  - Bangkok Second Ring Road Viaduct in Thai Land
  - Kawagoe Viaduct at Second Tomei Expressway

- Erection girder photos
  - Japanese fabricator catalogue
  - Example in Korea: max span is 50m

b) Construction sequence of SBS

- Fabrication of segment by short line match cast method
- Fabrication yard layout
- Detail of form work
- Detail of crane at stock yard
- Construction method statement of Pre-cast Segment

c) Feature of SBS

- High quality control by proper and sufficient utilities like factory
- By using external cables, the thickness of girder member can be decreased.

(Concrete volume is smaller than MSS.)

- Shortening construction period by semi-automatic fabrication and work (labor) saving machine
- Improvement safety by decreasing site works and labor saving
- Economical construction due to repeated usage of facilities in large scale project

Connection between each segment can be secured by shear key and bond with longitudinal prestress (full prestress is considered in the design of main girder). And a lot of achievement shows the safety of this construction method as described in the above

d) Main material quantities

In the calculation of cost estimation, P79 ~A2 part was selected as typical section. The quantity calculation is shown in Chapter 4 of Discussion Paper No.8. And the main material quantity for Hai An side was estimated based on the above said calculation results.

2) MSS

a) Results of reviewing technical samples

In this Design Study, the following reference papers were prepared as a part of Discussion Paper No.8:

- Construction record
    - Thanh Tri Bridge constructed by MSS
    - Construction report on Nanairo Viaduct in Japan
    - Project outline of Second Tomei Expressway Ano Viaduct
- Outline of MSS
- Erection girder photos
- Japanese fabricator catalogue

b) Construction sequence of MSS

- As shown in the above paper (Ano Viaduct)

c) Feature of MSS

- Quality control is not so significant compared with SBS because of cast-in-situ concrete construction method.

- Due to construction joint at the point of 0.2L (L is span length) after passing intermediate support, web thickness becomes bigger than standard section in center of span. (Concrete volume increased compared with SBS.)

- Scaffolding works can be omitted compared with normal scaffolding construction method, but it takes longer time of construction than SBS.

- Erection girder weight is heavier than SBS due to inclusive steel form work for main girder and other devices.

- Site works and labor savings are not so much expected compared with SBS, so safety level at the site is lower than SBS.

- From economical view point, if the project scale is not so big, this construction method is most appropriate because it does not require a huge area for fabrication & segment stock yard. However, in this project, the priority first is to save construction period and furthermore project scale is remarkably big. Therefore, this method is not superior to SBS in total aspects.

d) Main material quantities

In the calculation of cost estimation, P79 ~A2 part was selected as typical section. The quantity calculation is shown in Chapter 4 of Discussion Paper No.8. And the main material quantity for Hai An side was estimated based on the above said calculation results.

3) P&Z

a) Results of reviewing technical samples

In this Design Study, the following reference papers were prepared as a part of Discussion Paper No.8:

- Construction record
    - Construction report on Yoshimine Viaduct in Japan
- Erection girder photos
    - Japanese P&Z Association catalogue

b) Feature of P&Z

- Quality control is not so significant compared with SBS because of cast-in-situ concrete construction method.

- Scaffolding works can be omitted compared with normal scaffolding construction method, but it takes longer time of construction than SBS.

- Erection girder weight is lighter than SBS due to cantilever erection with maximum segment length of 10m.

- Site works and labor savings are not so much expected compared with SBS, so safety level at the site is lower than SBS.

- From economical view point, if the project scale is not so big, this construction method is most appropriate because it does not require a huge area for fabrication & segment stock yard. However, in this project, the priority first is to save construction period and furthermore project scale is remarkably big. Therefore, this method is not superior to SBS in total aspects.

4) Main material quantities

In the calculation of cost estimation, P79 ~A2 part was selected as typical section. And the main material quantity for Hai An side was estimated based on the above said calculation results.

8.4.1.2 Design of Approach Bridge on Hai An Side

(1) Arrangement of Members of Main Girder

Dimension of each member for main girder shall be determined as follows:

As for the basic conditions to determine each member's dimension, the followings are considered:

1) Basic Condition of Arrangement of Members

a) The following PC tendons shall be considered to determine member thickness.

- PC tendon for longitudinal direction
  - *19S15.2mm (arranged in box girder as external cable)
  - *12S15.2mm (arranged in upper & lower slab)
- PC tendon for transversal direction
  - *1S28.6mm (arranged in upper slab)

b) Reinforcing bars to be used are as follows:

- Reinforcing bars in upper slab:
  - *Reinforcing bars to be arranged shall be D12 to D14, because upper deck slab is designed as PC members.
- Reinforcing bars in lower slab:
  - *Reinforcing bars to be arranged shall be D14 to D19, because lower deck slab is designed as RC members.

c) Concrete covering for reinforcing bars is as follows: (for salt damage area)

Table 8.4.1-4 Concrete covering for reinforcing bars (22TCN-272-05)

Region		Upper	Under	Remark
		(Inside)	(Outside)	
*1 Upper Slab	Cantilever	40mm	60mm	Fc=50Mpa
	Inside of Box	40mm	40mm	
*2 Lower Slab		40mm	60mm	
*3 Web		40mm	60mm	

Source : Study Team

d) Approximate dimension of each member is as follows from achievements.

(by "PC Bridge Planning Manual")

Table 8.4.1-5 Approximate dimension of each member

*1 Upper Slab

	Main Girder Shape	Structure	D1(m)	D2(m)	D3(m)	D4(m)
Upper Slab	1Box girder	RC	0.24-0.30	0.30-0.50	0.30-0.50	0.25
		PC	0.25-0.35	0.35-0.65	0.35-0.65	0.25

*2 Lower Slab

	Main Girder Shape	Structure	Span Center T1(m)	Middle Support T2(m)
Lower Slab	1Box girder	RC	0.21-0.30	0.40-1.50

*3 Web

	Main Girder Shape	Structure	Span Center T1(m)	Middle Support T2(m)
Web	1Box girder	RC	0.35-0.50	0.50-0.70

Source : Study Team

2) Thickness of Upper Slab

Determination of thickness of upper slab shall be done by considering the below:

a) Minimum thickness considering the fatigue because live load is on directly.

b) Necessary thickness for arrangement of PC tendon and reinforcing bars.

a) Minimum thickness of upper slab which supports live load directly.

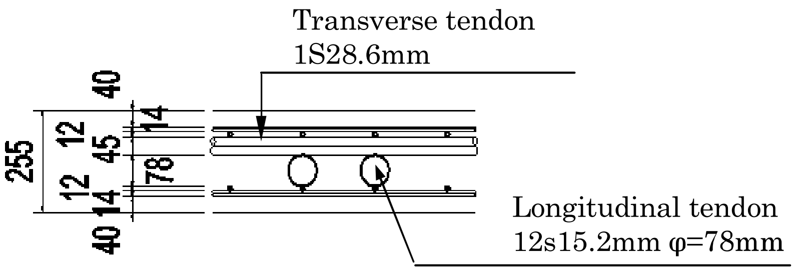
The thickness shall be determined in accordance with JSHB.

*1. Minimum thickness of prestressed concrete slab, t > 160mm

*2. Minimum thickness of tip of the cantilever deck slab, t > 200mm

b) Necessary thickness for arrangement of tendon

Table 8.4.1-6 Necessary thickness for arrangement of tendon



Cover	40mm+40mm	80mm	
Reinforcement Bar	(D12+D14)x2	52mm	
Transverse Tendon	Sheath φ45	45mm	
Longitudinal Tendon	Sheathφ78	78mm	(#1075)
Total		255mm	

Source : Study Team

Therefore, minimum thickness of upper slab shall be 260mm considering any margin.

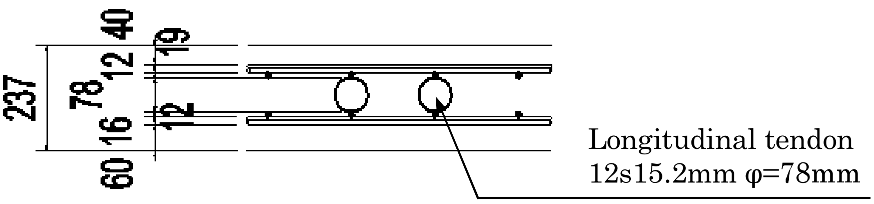
c) Thickness of Lower Slab

Lower slab shall be determined generally thicker around the intermediate support point, considering the negative bending moment comparing to section of span center.

Thicker slab shall be around the place which has negative bending moment.

General thickness of lower slab at span center shall be determined in this clause.

Table 8.4.1-7 Thickness of Lower Slab



Cover	40mm+60mm	100mm	
Reinforcement Bar	D19+D12x2+D16	59mm	
Longitudinal Tendon	Sheathφ78	78mm	(#1075)
Total		237mm	

Source : Study Team

d) Thickness of Web

Thickness of web shall be determined considering the following particulars.

- Necessary thickness against shearing force of the main girder
- Necessary thickness against flexural capacity as transversal members
- Necessary thickness for arrangement of PC tendons

The approach bridge is constructed by span-by-span construction method which doesn't have PC tendon in the web, so that necessary thickness of the member cannot be determined from this view point. Therefore minimum thickness shall be determined from the achievement of actual constructions.

Table 8.4.1-8 Thickness of Web

	Main Girder Shape	Structure	Span Center T1(m)	Middle Support T2(m)
Web	1Box girder	RC	0.35	0.50

Source : Study Team

(2) Design of Longitudinal Direction of Main Girder

1) Outline of Design

This approach bridge shall be constructed with Span by Span method by using precast segment aiming introduction of advance foreign technology and rapidity construction and improvement of qualities

In the design, we do considering the following item so that it may improve the efficiency of the girder production and construction.

*Decentralized arrangement of deviator and standardization of segment shape.

*Simplification of web thickness and minimization of section change.

*Using together of internal & external PC cable for main girder's tendons and making of the external cable a large capacity

*Simplification by straight line arrangement of cable arranged in upper slab and lower slab.

*No arrangement of PC cable in web.

The tensile stress is not caused as full prestress in the service load working state because the reinforced concrete is not arranged in the block seam part and it is integrated by the PC tendon in the precast segment

2) Design condition

a) Dead Load

*Concrete self weight: γc = 24.50kN/m3

*Asphalt pavement (Thickness t=75mm) : γs = 22.50kN/m3

*Curb : Concrete high column weight w = 7.575×2 = 15.15kN/m
: Hand Rail weight wh = 0.600×2 = 1.200kN/m

*water service tube φ400 (Article 2) : W = 2.250×2 = 4.40kN/m

b) Live Load and Impact Load

*Live Load : Vehicular live loading (HL-93) 4 Lanes
*Dynamic Load : Impact factor IM=0.25

c) Temperature

*Uniform Temperature : 40 ˚C (For Bearing and expansion design)
±20 ˚C (For Girder design)

*Temperature Gradient : As shown in following Table.

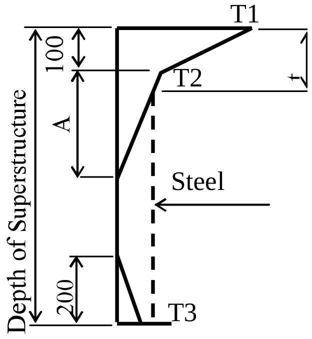


Table 8.4.1-9 Temperature Gradient

	T1	T2	T3
Positive	+23	+6	+3
Negative	-7	-1	0

Source : Study Team

d) Earthquake Load

*Design horizontal seismic intensity : Kh=0.18

e) Load Factor and Load Combination

Table 8.4.1-10 Load Factor and Load Combination

Load Combination / Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS EL	TL	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These At a Time EQ	CT	CV
Strength-I	γ_p	1.75	1.75	1.00	-	-	1.00	0.50/1.20	γTG	γSE	-	-	-
Strength-II	γ_p	-	-	1.00	1.40		1.00	0.50/1.20	γTG	γSE	-	-	-
Strength-III	γ_p	1.35	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γTG	γSE	-	-	-
Extreme	γ_p	0.50	0.50	1.00	-	-	1.00	-	-	-	1.00	1.00	1.00
Service	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00/1.20	γTG	γSE	-	-	-
Fatigue – LL, IM & CE only	-	0.75	0.75	-	-	-	-	-	-	-	-	-	-

Source : Study Team

3) Material

a) Concrete

Table 8.4.1-11 Properties of Concrete (Main Girder)

Item		unit	Girder	Remark
Specified Compression Strength fc		Mpa	50.0	Cylinder
Compressive Strength at Pre-stressing fci		Mpa	36.0	
Modulus of Elasticity Ec		Mpa	33000	
Allowable Stress				
Compressive Stress	Before Losses	Mpa	21.0	
	D.L.W.S	Mpa	17.0	
	Service Limit State	Mpa	17.0	
Tensile Stress	Before Losses	Mpa	1.8	
	D.L.W.S	Mpa	No tension	
	Service Limit State	Mpa	No tension	
Modulus Rupture		Mpa	14348	

Source : Study Team

b) Prestressing Steel

Table 8.4.1-12 Properties of Prestressing Steel (PC Strand Cable)

Item		unit	12S15.2L	19S15.2L	1S28.6
Tensile Strength fpu		Mpa	1850	1850	1800
Yield Strength fpy		Mpa	1600	1600	1500
Modulus of Elasticity Ep		Mpa	200000	200000	200000
Allowable tensile strength	During pre-stressing.	Mpa	1440	1440	1350
	After pre-stressing.	Mpa	1295	1295	1260
	At design load.	Mpa	1110	1110	1080

Source : Study Team

Table 8.4.1-13 Properties of Prestressing Steel (PC Bar - SBPR930/1180)

Item		unit	φ32	Remark
Tensile Strength fpu		Mpa	1180	
Yield Strength fpy		Mpa	930	
Modulus of Elasticity Ep		Mpa	200000	
Allowable tensile strength	During pre-stressing.	Mpa	837	0.9fpy
	After pre-stressing.	Mpa	790	0.85fpy
	At design load.	Mpa	697	0.75fpy

Source : Study Team

Table 8.4.1-14 Reinforcing Bars

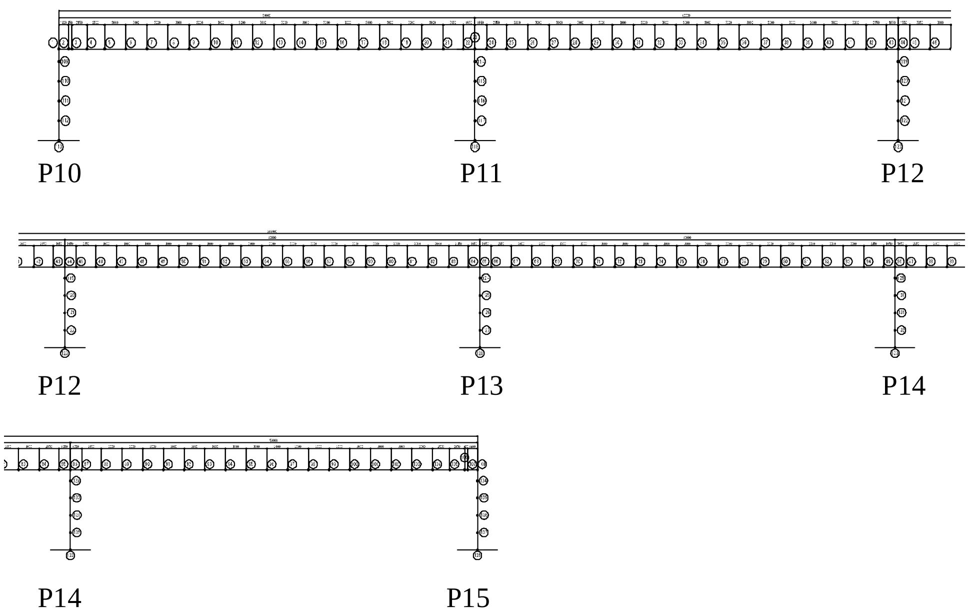
Item	unit	SD345	Remark
Tensile Strength fpu	Mpa	490	
Yield Strength fpy	Mpa	345	
Modulus of Elasticity Ep	Mpa	200000	
Allowable tensile strength at design load.	Mpa	180	

Source : Study Team

4) Model for structural analysis

The analysis as a frame structure by linear theory shall be carried out by modelling the structure in the following way.

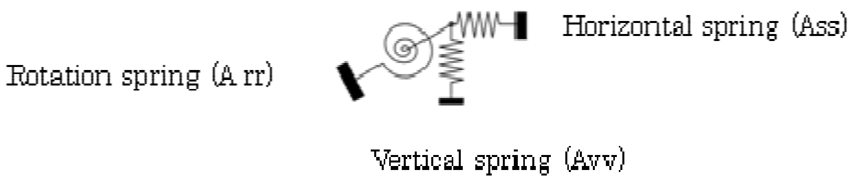
1. The structure model shall consist of the lines connecting the centroid axes of the members, with the axis of the girder made to coincide with the design profile alignment of the completed system.
2. The zones of junction between the towers, girder and piers shall be considered as rigid zones. The rigidity of these rigid zones shall be assumed as 1000 times that of the adjacent members.
3. Supports shall be modelled to correspond with the function of the type of rubber bearing actually used.
4. The foundation structure and ground shall be modelled as elastic springs having equivalent behaviour. Also, the effect of variation shall be considered corresponding with the ground and the foundation



Source :Study Team

Figure 8.4.1-1 Analysis Model

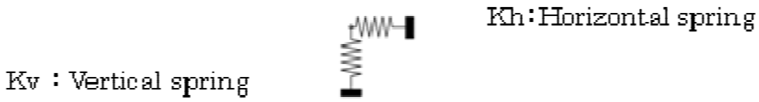
Table 8.4.1-15 Spring Constants of Pile



P10-P15	unit	Service state	Earthquake
Horizontal (Ass)	kN/m	4.35286E+005	2.263068E+006
Compound spring (Asr=Ars)	kN/rad	-1.12780E+006	-3.65098E+006
Rotational angle (Arr)	kNm/rad	4.022810E+007	4.540677E+007
Vertical (Avv)	kN/m	3.59054E+006	3.590544E+006

Source :Study Team

Table 8.4.1-16 Spring Constants of Shoe



	unit	End Support	Middle Support
Vertical direction	kN/m	2858000	8860000
Horizontal direction	kN/m	10314	27000

Source :Study Team

5) Structural analysis

The structural analysis shall be performed with consideration to the following items.

a) The procedure and time schedule for completing the structural system shall be determined, and the structural analysis performed accordingly.

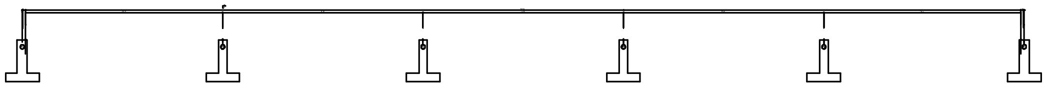
b) The following loads shall be considered as the loads which will work during the construction stage.

- Primary loads
  - Dead load of girder
  - Pre-stressing force (inner tendons and external tendons)
  - Creep and shrinkage of concrete

c) The points of time when superimposed dead load and live load will be applied shall be assumed, and analysis performed for the stress resultants acting at such times.

The process and the schedule of the structure analysis are as following figure.

Step 1.Construction of Pier and Foundation (360 days)

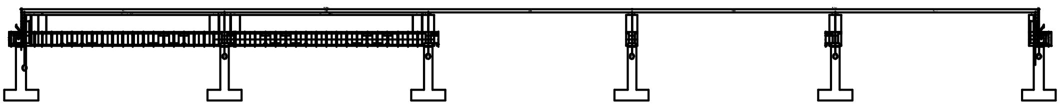


Step 2. Construction of Pier Table（60 days）

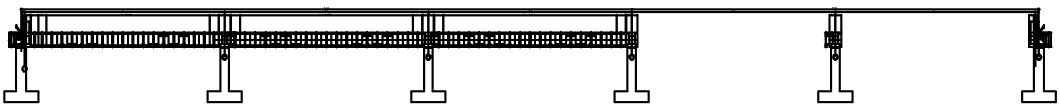
Step3. Construction of First Span (30 days)

（It is assumed that concrete age at erection time is 120 days.）

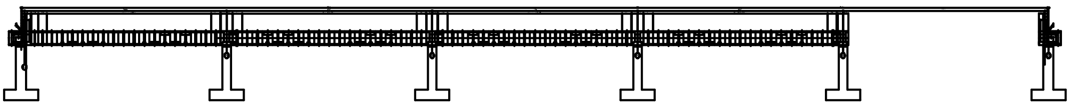
Step4. Construction of Second Span (15 days)



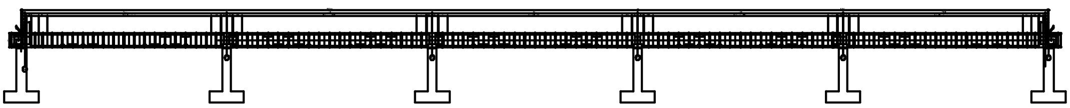
Step5. Construction of third Span (15 days)



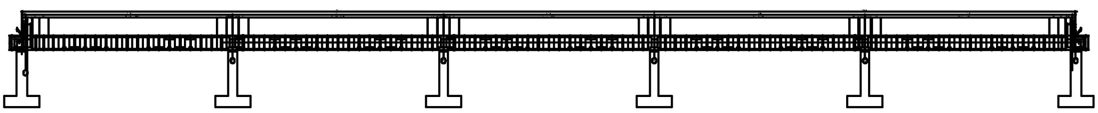
Step6. Construction of 4th Span (15 days)



Step7. Construction of 5th Span (15 days) (Completion of Structure)



Step8. Construction of Surfacing (60days)



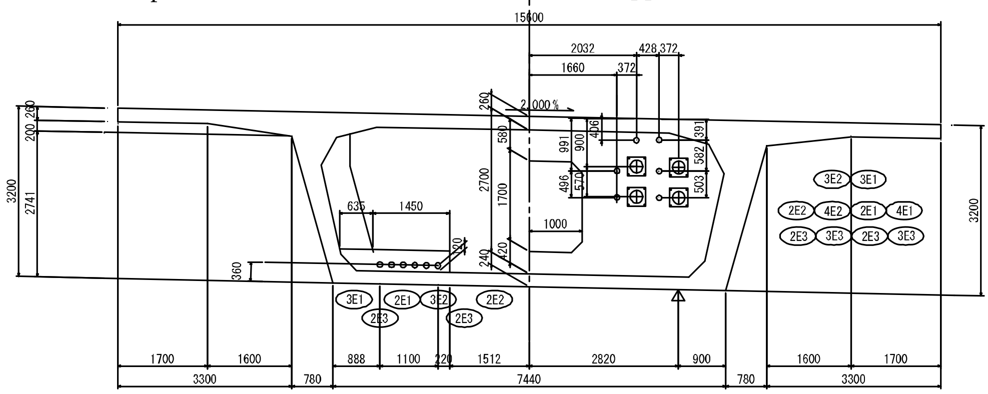
Source : Study Team

Figure 8.4.1-2 Process and the schedule of the structure analysis

6) Arrangement of PC Tendon in main Girder

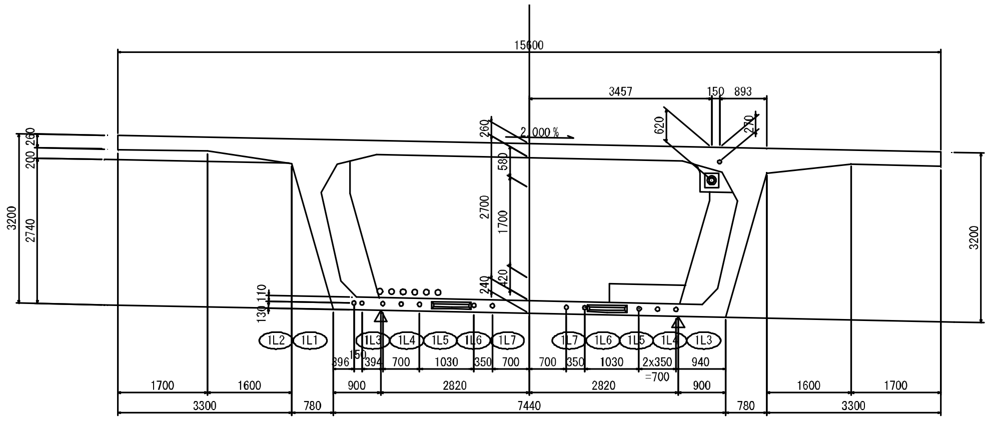
We show the arrangement of the outer cable and inner cable in following Figure.

Span Center section Middle Support section



Source : Study Team

Figure 8.4.1-3 Arrangement of External PC Tendon



Source : Study Team

Figure 8.4.1-4 Arrangement of Inner PC Tendon

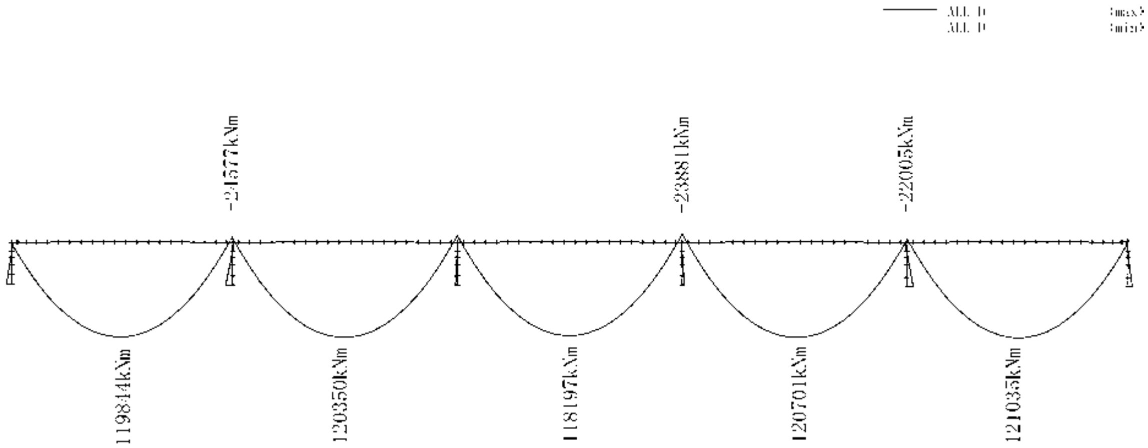
7) Design result of Main Girder

a) Approach Bridge P35-P40 (5@60.0m=300m)

The approach bridge of same to P10-P15(5@60.0m=300m) are the following eight bridges.

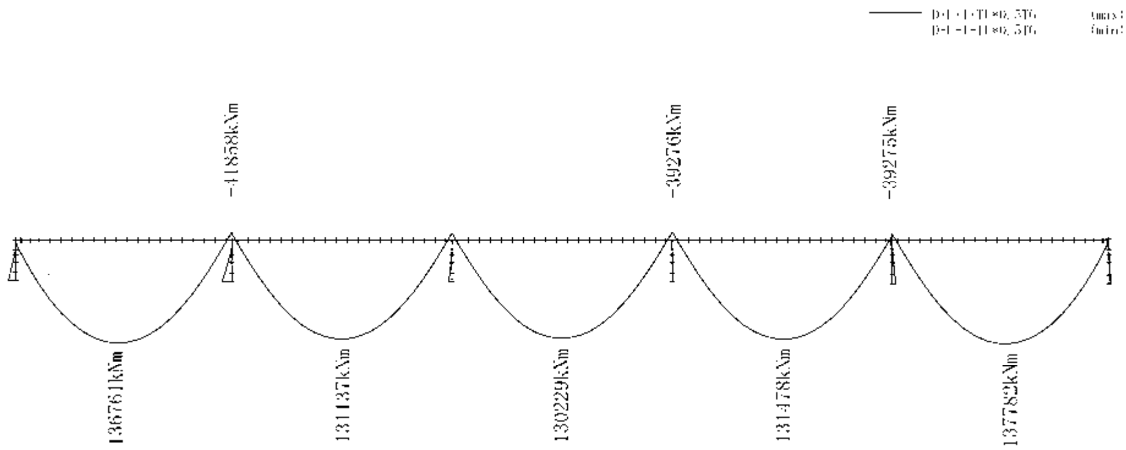
A1-P5, P5-P10, P15-P20, P20-P25, P25-P30, P30-P35, P40-P45, P50-P55

<1>Bending Moment of Main Girder



Source : Study Team

Figure 8.4.1-5 Bending Moment caused by Dead Load

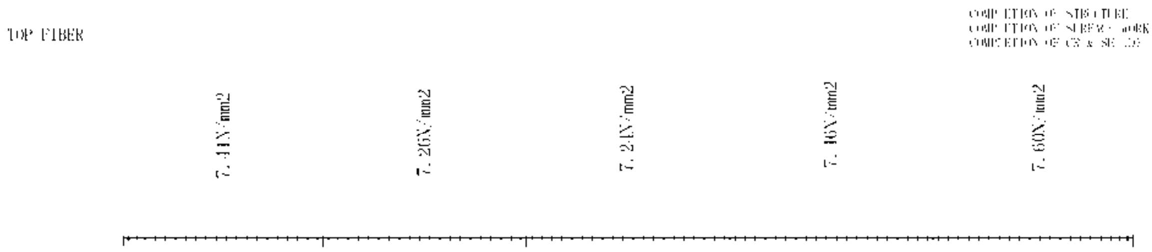


Source : Study Team

Figure 8.4.1-6 Bending Moment in Service state

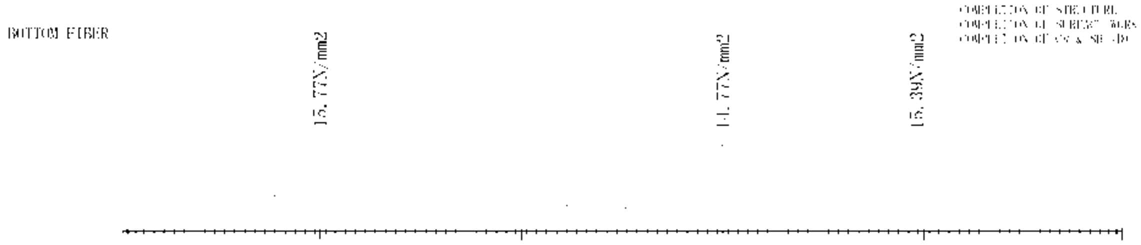
<2>Fiber Stress of Main Girder

<u>< Dead Load State ></u>



Source : Study Team

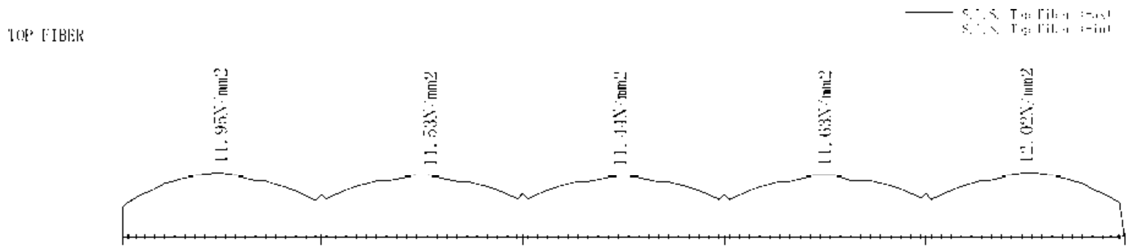
Figure 8.4.1-4 Stress of Top Fiber (Dead Load)



Source : Study Team

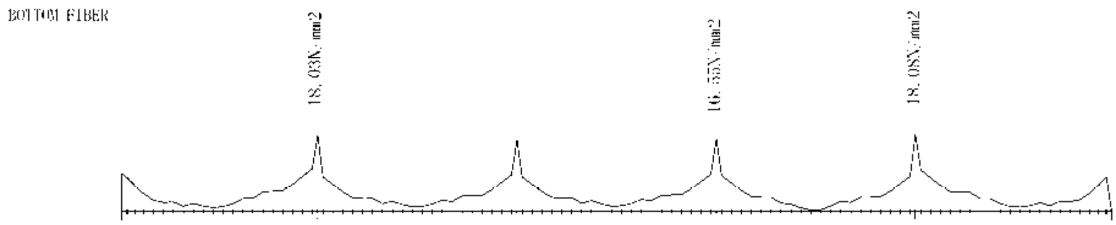
Figure 8.4.1-5 Stress of Bottom Fiber (Dead Load)

<Service Limit State>



Source : Study Team

Figure 8.4.1-6 Stress of Top Fiber (Service Limit State)



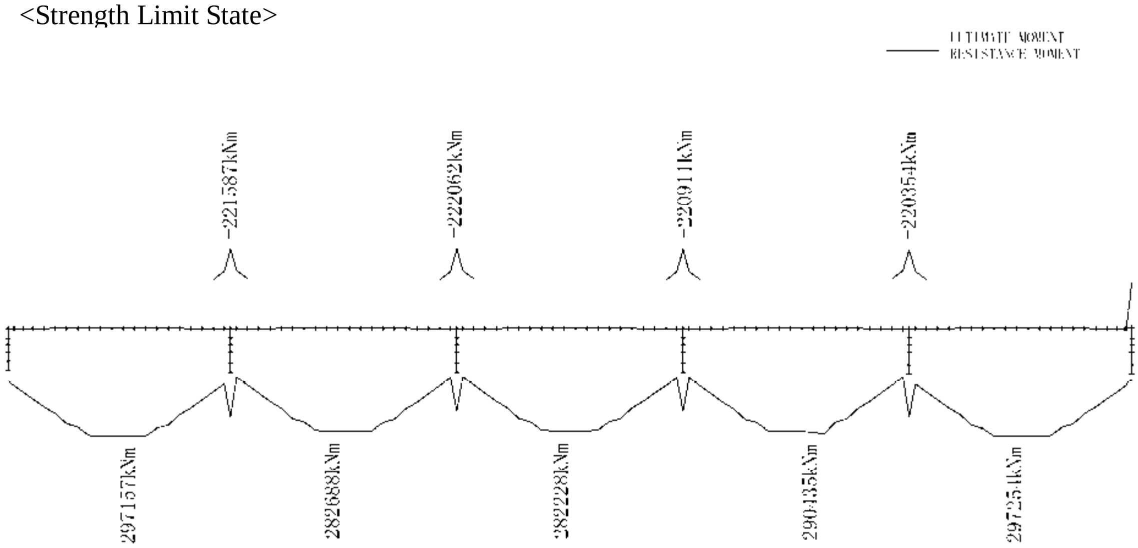
Source : Study Team

Figure 8.4.1-7 Stress of Bottom Fiber (Service Limit State)

The tensile stress doesn't occur in all sections.

<3> Ultimate Bending Moment of Main Girder

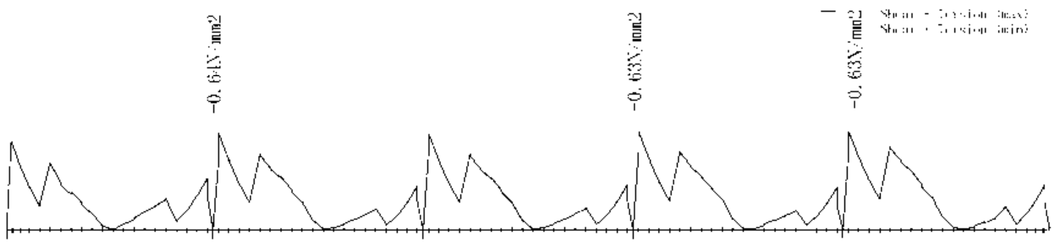
<Strength Limit State>



Source : Study Team

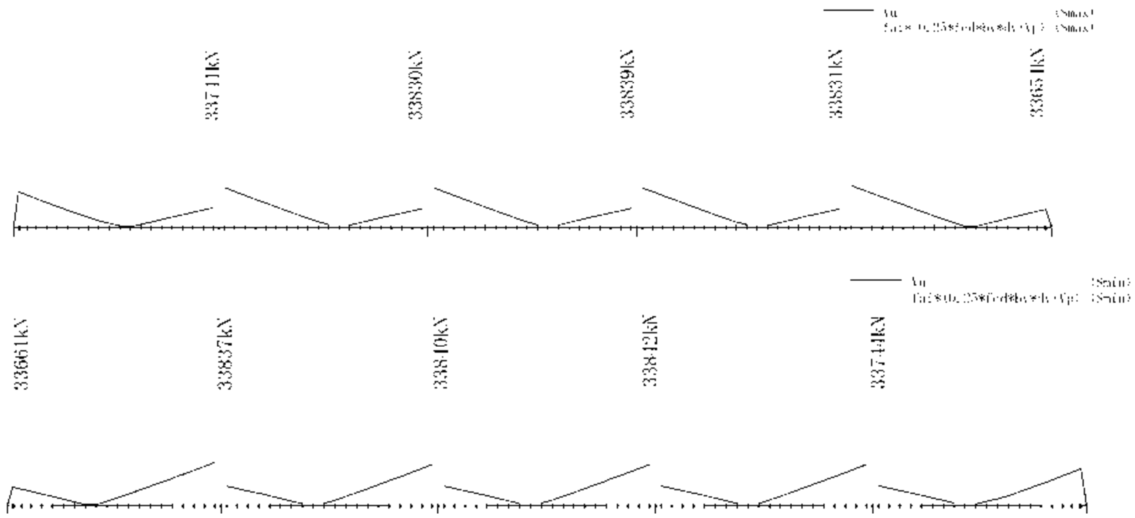
Figure 8.4.1-8 Bending Moment of Main Girder (Strength Limit State)

<4> Shear stress of Main Girder at service limit state



Source : Study Team

Figure 8.4.1-9 Diagonal tensile stress

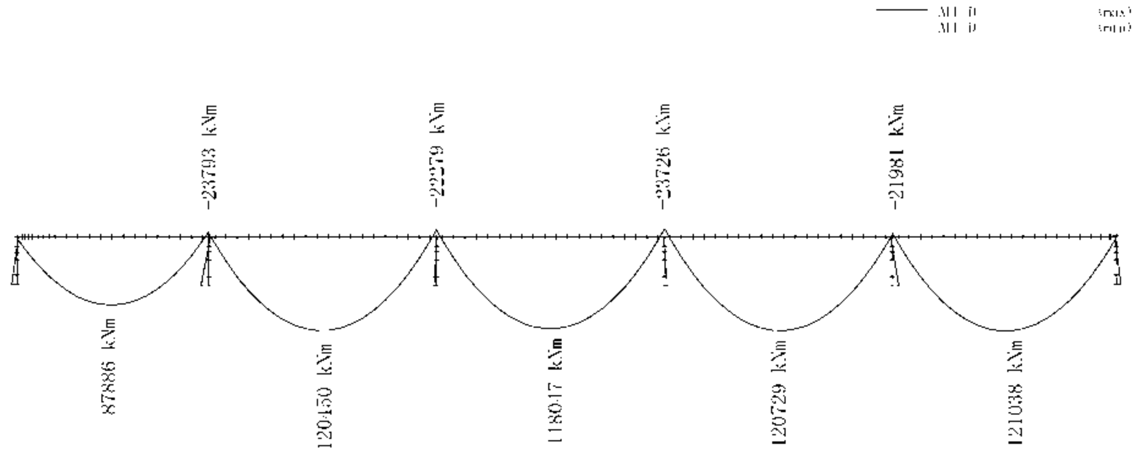


Source : Study Team

Figure 8.4.1-10 Maximum Shear stress

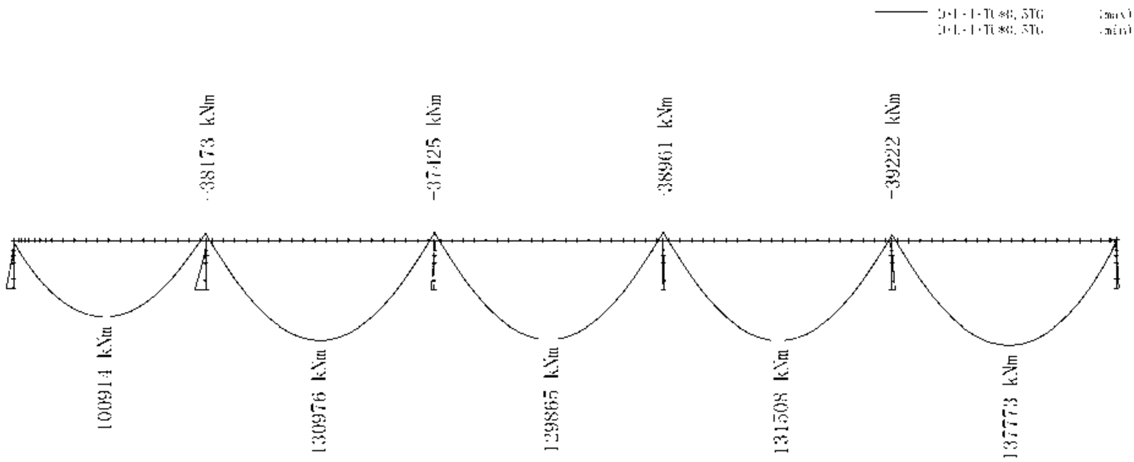
b) Approach Bridge P10-P15 (51.5m+4@60.0m=291.50m)

<1>Bending Moment of Main Girder



Source : Study Team

Figure 8.4.1-5 Bending Moment caused by Dead Load



Source : Study Team

Figure 8.4.1-6 Bending Moment in Service

<2>Fiber Stress of Main Girder

<u>< Dead Load State ></u>

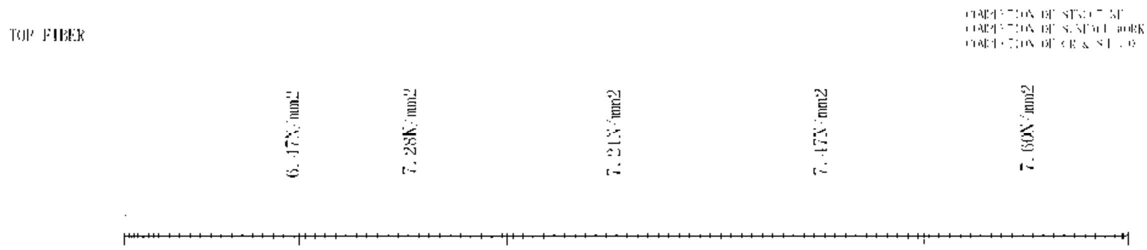
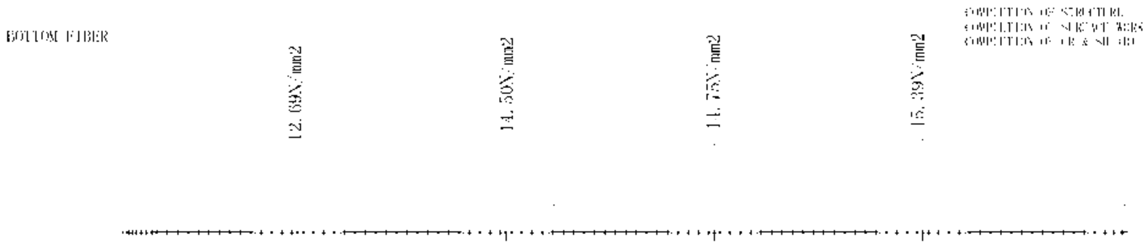


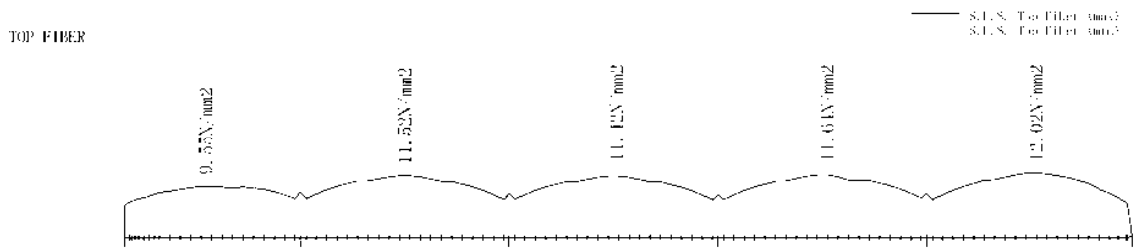
Figure 8.4.1-11 Stress of Top Fiber (Dead Load)



Source : Study Team

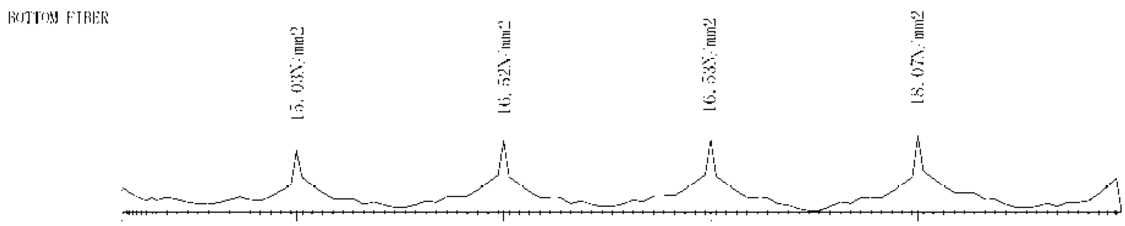
Figure 8.4.1-12 Stress of Bottom Fiber (Dead Load)

<Service Limit State>



Source : Study Team

Figure 8.4.1-13 Stress of Top Fiber (Service Limit State)



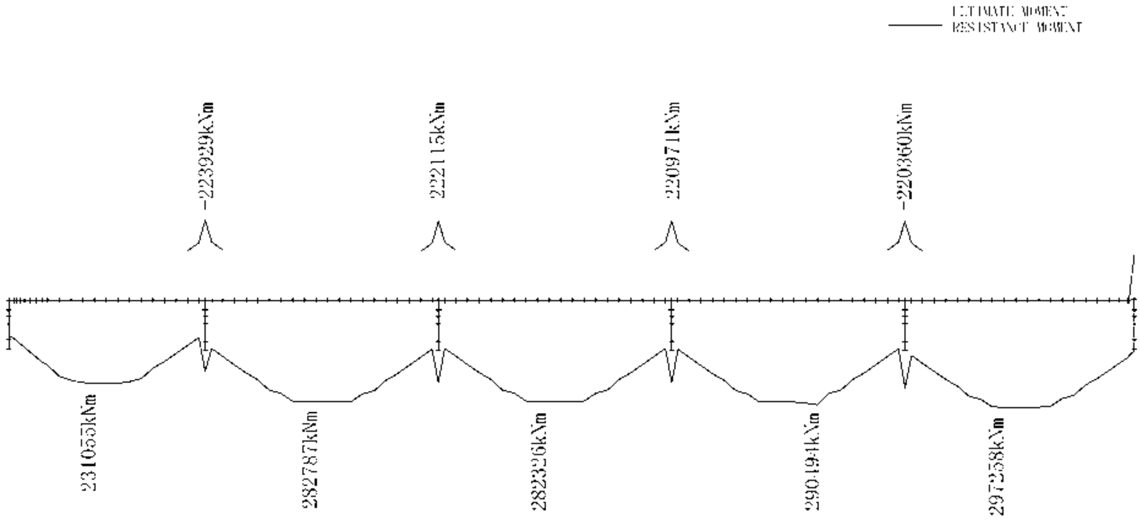
Source : Study Team

Figure 8.4.1-14 Stress of Bottom Fiber (Service Limit State)

The tensile stress doesn't occur in all sections.

<3> Ultimate Bending Moment of Main Girder

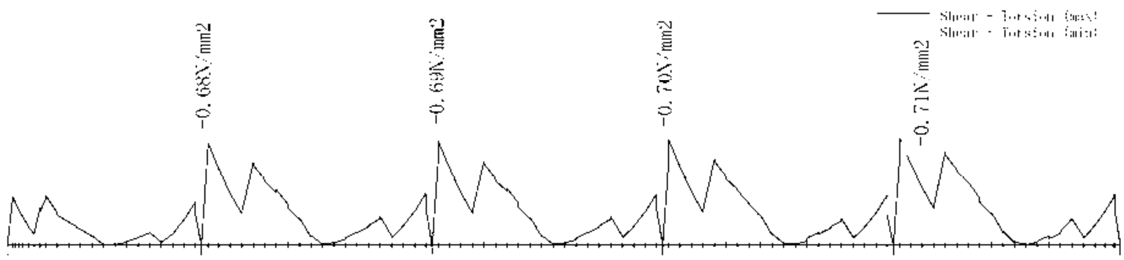
<Strength Limit State>



Source : Study Team

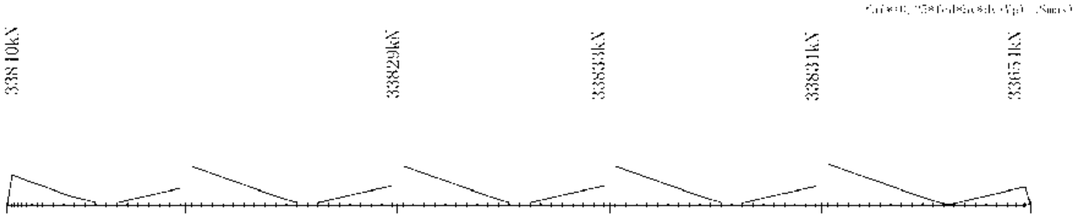
Figure 8.4.1-15 Bending Moment of Main Girder (Strength Limit State)

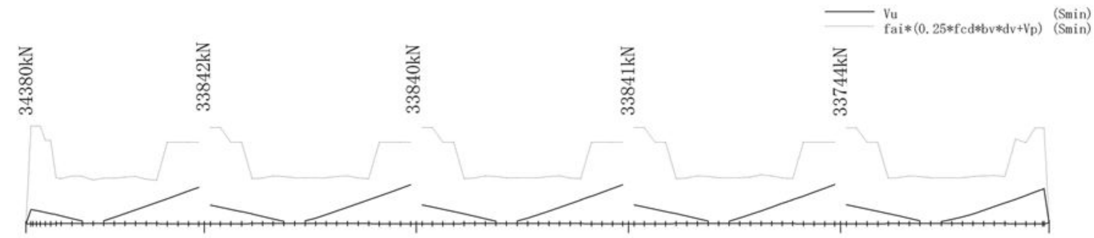
<4> Shear stress of Main Girder at service limit state



Source : Study Team

Figure 8.4.1-16 Diagonal tensile stress





Source : Study Team

Figure 8.4.1-17 Maximum Shear stress

c) Approach Bridge P45-P50 (4@60.0m+58.36m=298.36m)

<1> Bending Moment of Main Girder

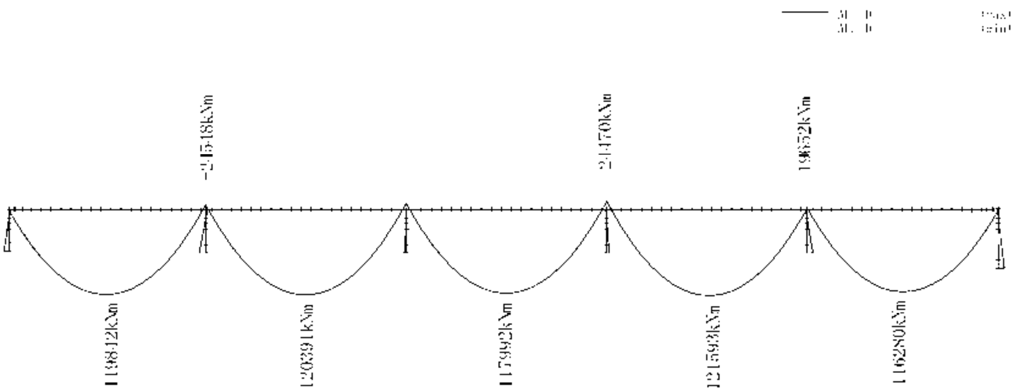
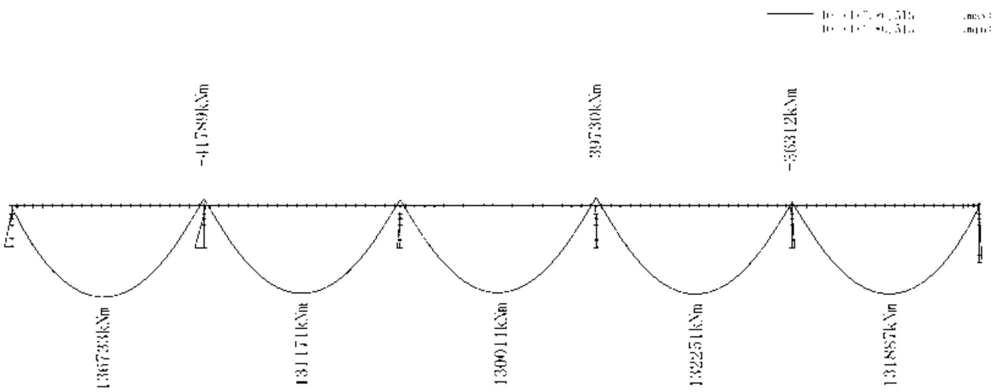


Figure 8.4.1-5 Bending Moment caused by Dead Load



Source : Study Team

Figure 8.4.1-6 Bending Moment in Service

<2> Fiber Stress of Main Girder

<u>< Dead Load State ></u>

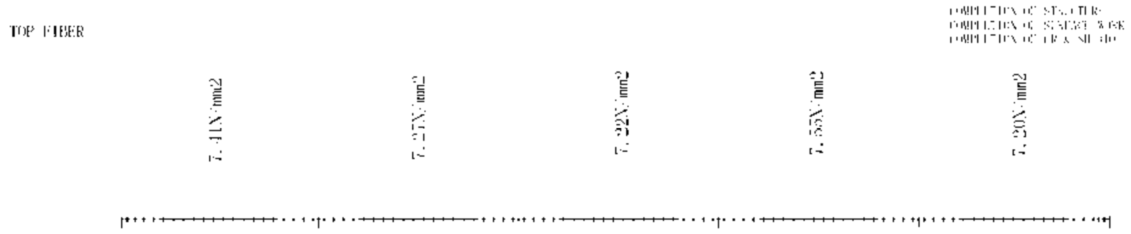
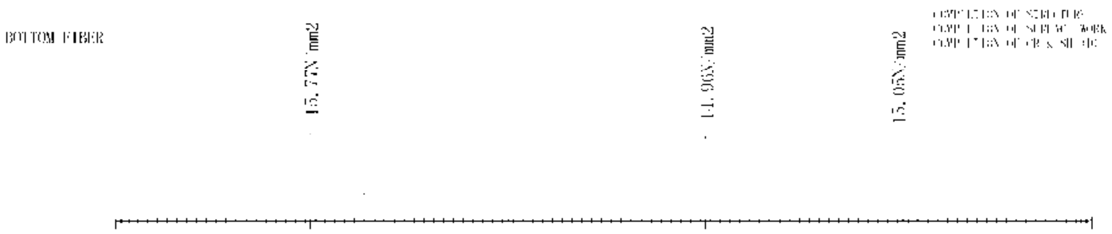


Figure 8.4.1-18 Stress of Top Fiber (Dead Load)



Source : Study Team

Figure 8.4.1-19 Stress of Bottom Fiber (Dead Load)

<Service Limit State>

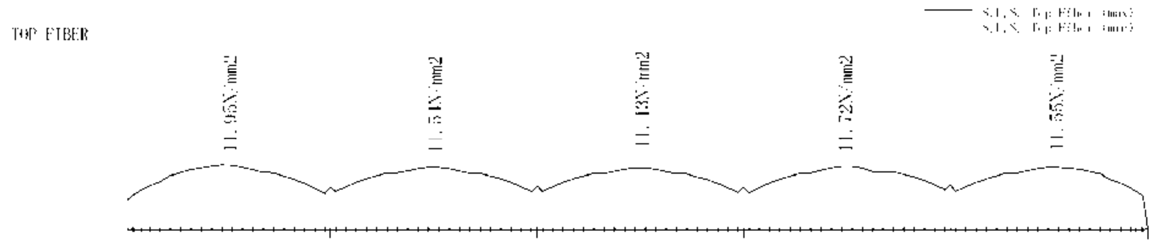
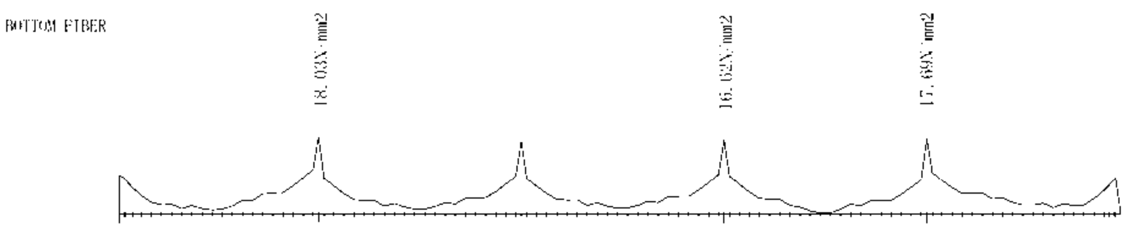


Figure 8.4.1-20 Stress of Top Fiber (Service Limit State)



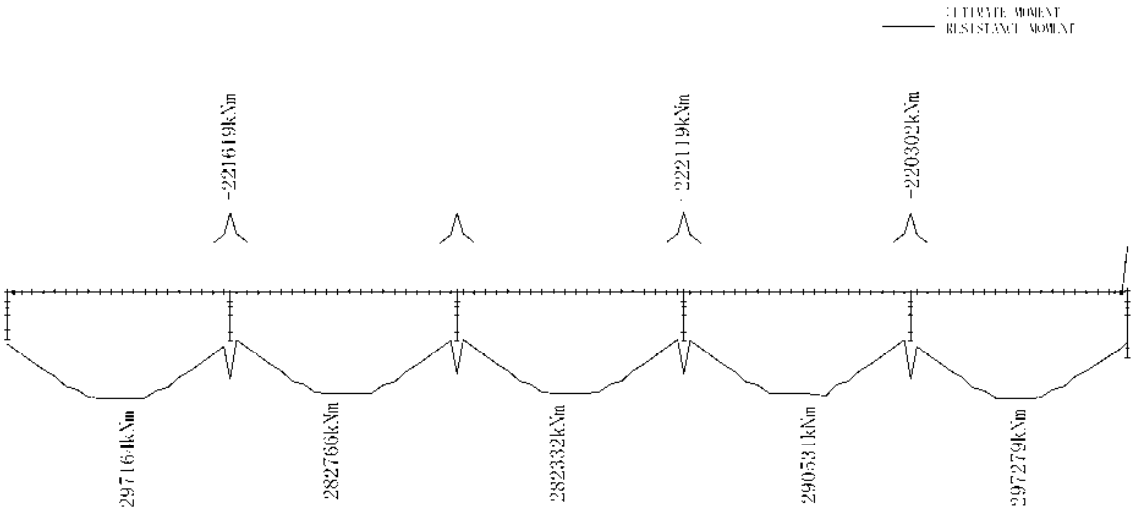
Source : Study Team

Figure 8.4.1-21 Stress of Bottom Fiber (Service Limit State)

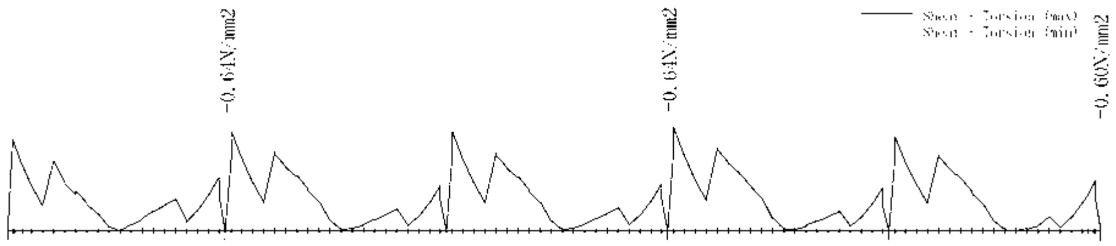
The tensile stress doesn't occur in all sections.

<3> Ultimate Bending Moment of Main Girder

<Strength Limit State>

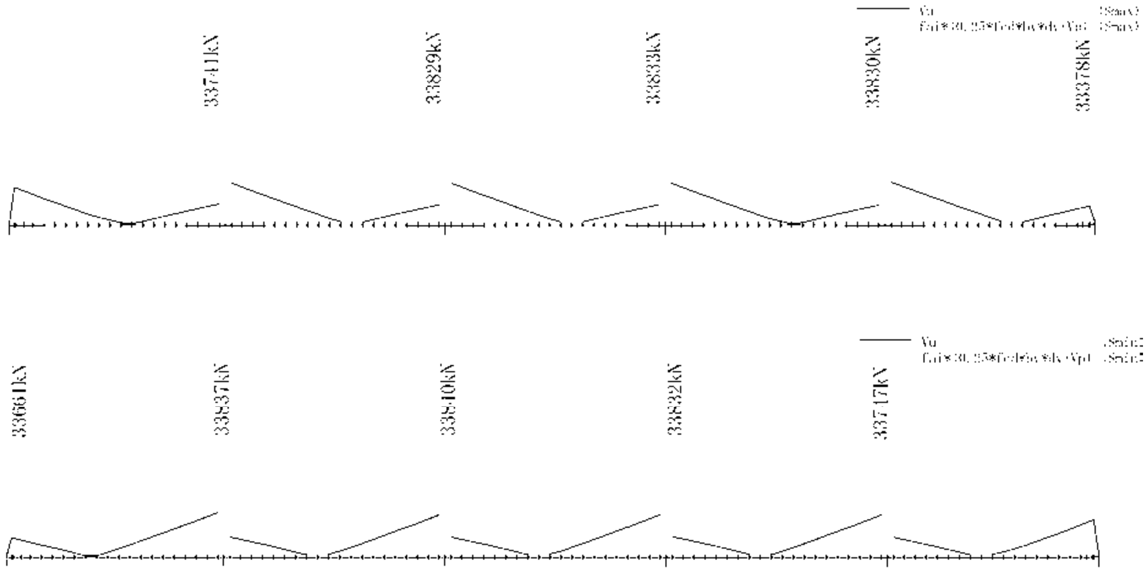


<4> Shear stress of Main Girder at service limit state



Source : Study Team

Figure 8.4.1-23 Diagonal tensile stress



Source : Study Team

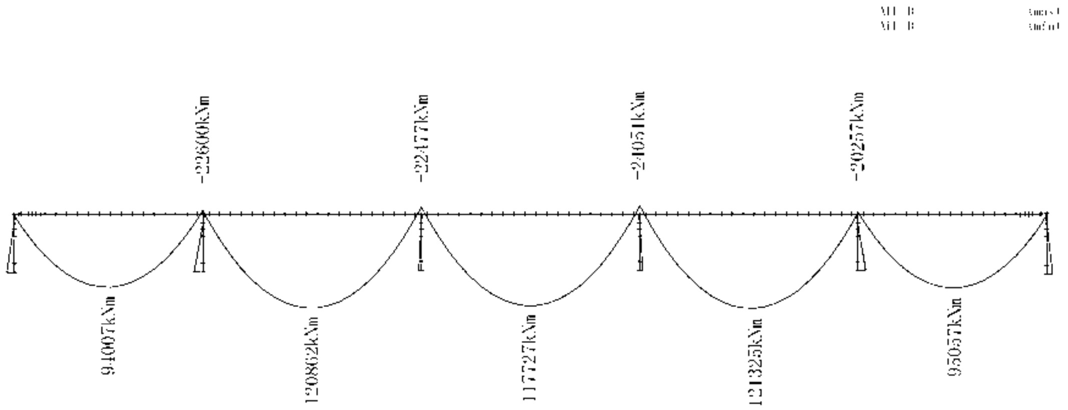
Figure 8.4.1-24 Maximum Shear stress

d) Approach Bridge P60-P65 (52.98m+3@60.0m+52.98m=285.96m)

The approach bridge of same to P60-P65(52.98m+3@60.0m+52.98m=285.96m) are the following three bridges. (P55-P60, P65-P70, P70-P75)

<1> Bending Moment of Main Girder

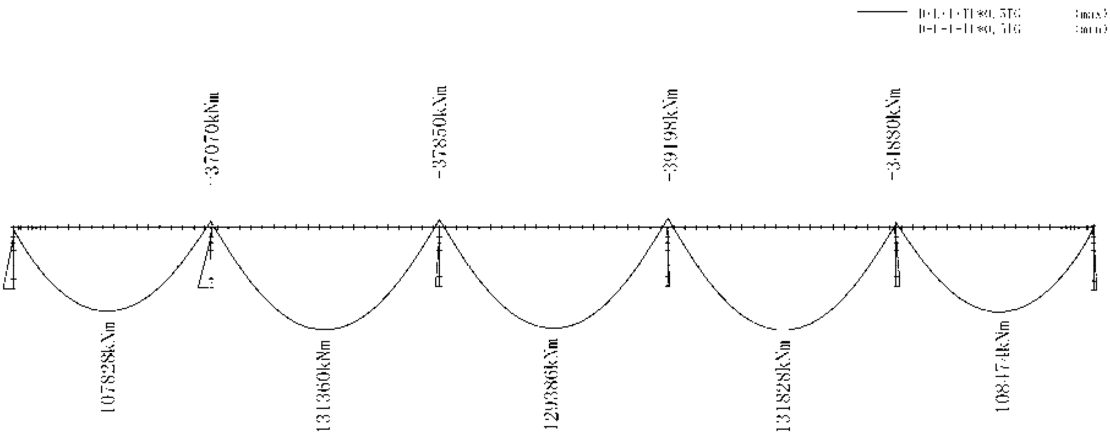
*All dead load state



Source : Study Team

Figure 8.4.1-5 Bending Moment caused by Dead Load

*Service load state

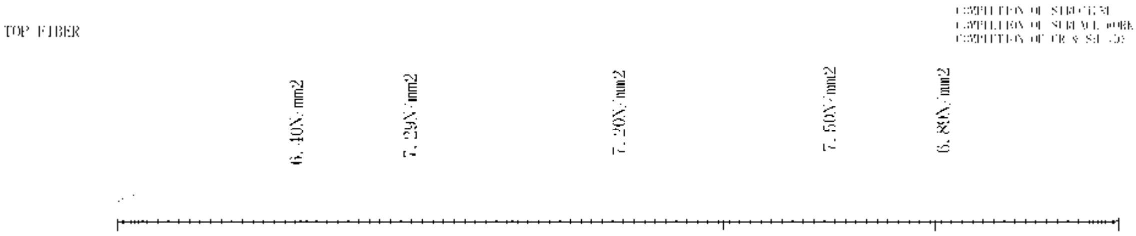


Source : Study Team

Figure 8.4.1-6 Bending Moment in Service

< 2 >Fiber Stress of Main Girder

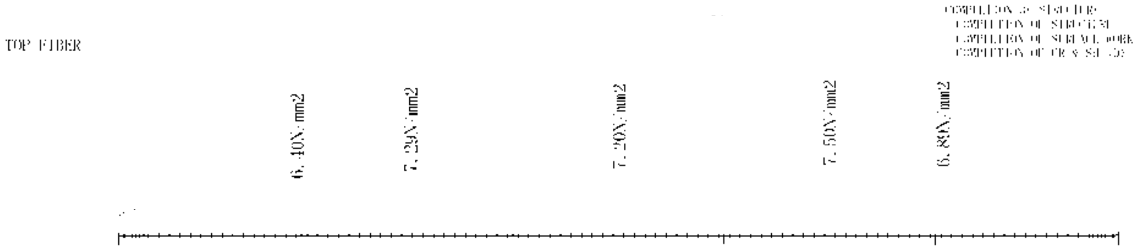
* Dead Load State (Top fiber stress)



Source : Study Team

Figure 8.4.1-25 Stress of Top Fiber (Dead Load)

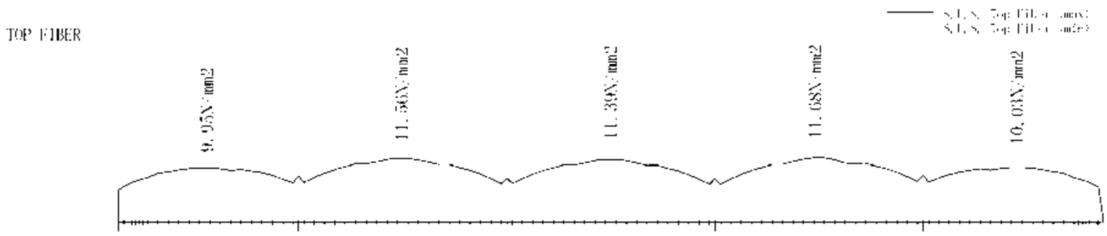
* Dead Load State (Bottom fiber stress)



Source : Study Team

Figure 8.4.1-26 Stress of Bottom Fiber (Dead Load)

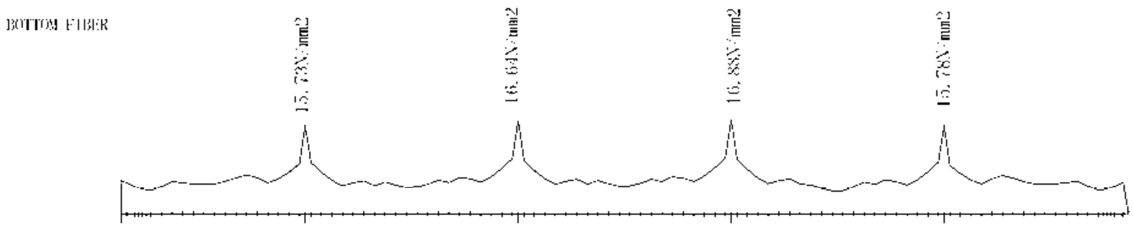
*Service Limit State (Top fiber stress)



Source : Study Team

Figure 8.4.1-27 Stress of Top Fiber (Service Limit State)

*Service Limit State (Bottom fiber stress)



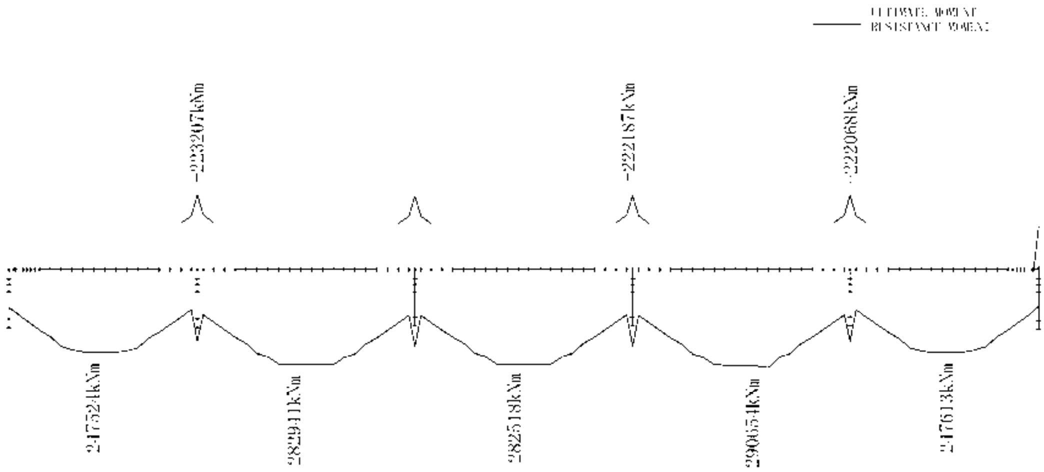
Source : Study Team

Figure 8.4.1-28 Stress of Bottom Fiber (Service Limit State)

The tensile stress doesn't occur in all sections.

<3> Ultimate Bending Moment of Main Girder

* Strength Limit State



Source : Study Team

Figure 8.4.1-29 Bending Moment of Main Girder (Strength Limit State)

<4> Shear stress of Main Girder at service limit state

* Diagonal tensile stress

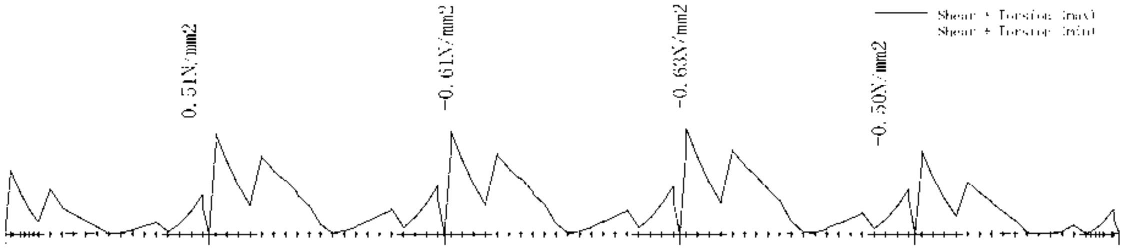
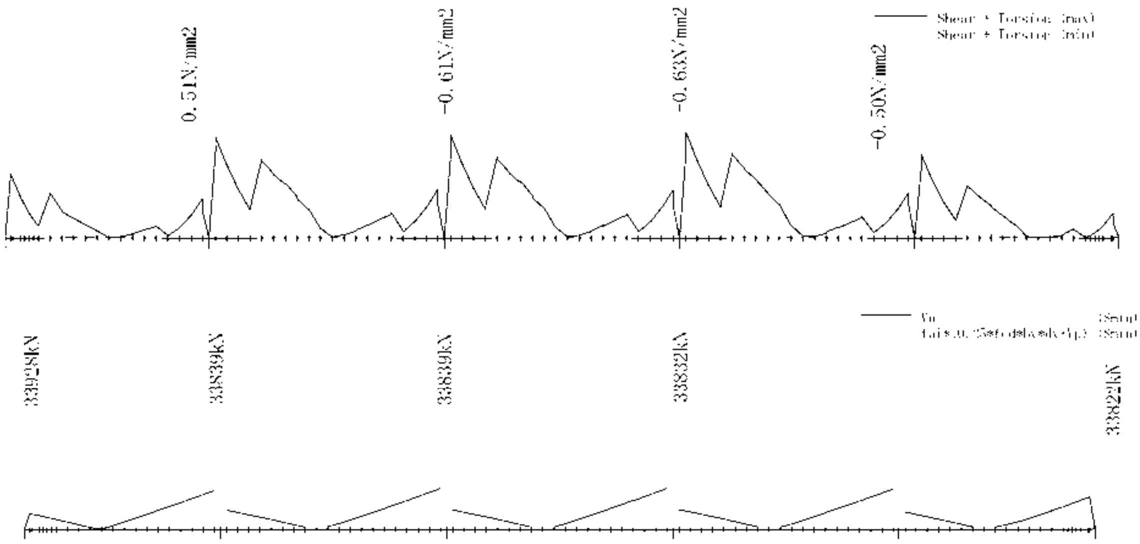


Figure 8.4.1-30 Diagonal tensile stress

* Maximum shear stress



Source : Study Team

Figure 8.4.1-31 Maximum Shear stress

(3) Design of Transversal Direction of Main Girder

1) General

This is a design report for transversal direction of main girder of approach bridge at Hai An side which leads to the main bridge planned in connecting road to Lach Huyen Port. Superstructure of the approach bridge is PC structure with transversal pre-stressing tendon of 1S28.6mm (SWPR19L). Each member (upper deck, web, and lower deck) of the main girder shall be shown in following Table. Construction shall be carried out by precast segment construction method. PC transversal tendon is arranged at a stage of segment production and after that, pre-stressing shall be executed at casting yard, and the segment shall be transferred and erected.

Table 8.4.1-17 Structure clarification of Main girder

Member	Structure clarification	Remark
Upper slab	PC structure	1S28.6mm
Web	RC structure	
Lower slab	RC structure	

Source : Study Team

As for design bending moment, safety for transversal direction shall be confirmed by formula of bending moment of live load (T load) instructed in JSHB (Japanese Specifications for Highway Bridges III 7.4.2) and bending moment calculated by FEM analysis of slab structure which is directly loaded wheel load P (=145/2*1.25=91kN≒100 k N).

a) Upper slab

As for design live load, flexural stress by bending moment of each for formula instructed in JSHB (Japanese Specifications for Highway Bridges) and direct wheel load (100kN) shall be set as follows:

Table 8.4.1-18 Direct wheel load on Upper slab

	JSHB	FEM analysis
Under Dead load working state	Full pre-stressed	
Under service loads(Live load)	Limit state for cracking	Full pre-stressed

Source : Study Team

b) Web and Lower slab

Web and Lower slab shall be designed as follows:

- For JSHB : stress of reinforcing bar when use shall be less than σsa = 180N/mm2
- For FEM : stress of reinforcing bar when use shall be less than σsa = 140N/mm2
- For JSHB : stress of reinforcing bar under service load working state shall be less than σsa = 180N/mm2
- For FEM : stress of reinforcing bar under service load working state shall be less than σsa = 140N/mm2

Comparison on bending moment due to live load

c) Standard Cross Section [Upper slab] - [A]

Table 8.4.1-19 Bending moment (Live load) [A]

Designed point	JSHB M (kN・m)	FEM Analysis (P=100kN) M(kN・m)	JSHB/FEM
*1 Connected part for cantilever slab	-150.150	-125.500	1.196
*2 Connected part in box girder	-165.713	-134.174	1.235
*3 Middle of box girder	103.002	43.001	2.395

Source : Study Team

Table 8.4.1-20 Designed Bending moment (D+LL+IM+Ps+Cr+SH) [A]

Designed point	JSHB M (kN・m)	FEM Analysis (P=100kN) M (kN・m)	JSHB/FEM
*1 Connected part for cantilever slab	-223.388	-198.738	1.124
*2 Connected part in box girder	-240.909	-209.370	1.151
*3 Middle of box girder	91.044	31.043	2.933

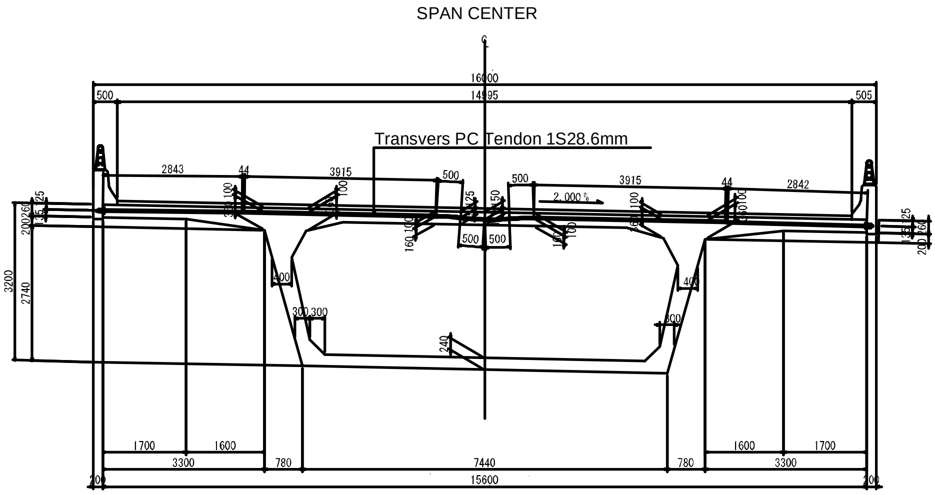
Source : Study Team

Table 8.4.1-21 Composite bending fiber stress [A]

		JSHB		FEM Analysis		
		σco (N/mm2)	σcu (N/mm2)	σco (N/mm2)	σcu (N/mm2)	
Dead Load	*1	3.84	0.55	3.84	0.55	
	*2	3.88	-0.11	3.88	-0.11	
	*3	1.12	6.98	1.12	6.980	
Service Load	*1	-0.42	4.81	0.28	4.11	
	*2	0.44	3.33	1.10	2.67	
	*3	10.26	-2.16	4.94	3.17	
			> -3.0	> 0.0		

Source : Study Team

Transversal PC tendon shall be 1S28.6mm and arranged with interval of ctc500mm shown in following Figure.



Source : Study Team

Figure 8.4.1-32 General Section for Arrangement of Transversal PC Tendon (ctc500)

Table 8.4.1-22 Safety factor for ultimate load working state [A]

Mu＝1.3*D+2.5*(LL+IM)+Ps+Cr+SH [JSHB-16]

Mu＝1.25*DC+1.5*DW+1.75*(LL+IM)+1.2 or 0.5*(CR+SH) [P=100kN]

Designed point	JSHB (kN・m)	FEM Analysis（P=100kN）(kN・m)	Mr (kN・m)
*1 Connected part for cantilever slab	452.710	320.288	< 598.847
*2 Connected part in box girder	415.729	330.379	< 736.402
*3 Middle of box girder	194.238	79.445	< 220.590

Here, Mr is a bending fracture resistance.

Source : Study Team

Standard Cross Section [Web and Lower slab] - [B]

Table 8.4.1-23 Bending moment (live load) [B]

Designed cross section	JSHB (kN・m)	FEM Analysis （P=100kN） (kN・m)	JSHB/FEM
*6 Upper end of web	107.277	23.912	4.486
*7 Lower end of web	48.798	7.581	6.437
*10 Connected area of lower slab	45.852	6.508	7.045
*11 Middle of lower slab	4.638	0.454	10.216

Source : Study Team

Table 8.4.1-24 Designed Bending moment (D+LL+IM+Ps+Cr+SH) [B]

Designed point	JSHB	FEM Analysis （P=100kN）	JSHB/FEM
*6 Upper end of web	110.654	55.221	2.004
*7 Lower end of web	65.291	39.825	1.639
*10 Connected area of lower slab	63.739	34.157	1.866
*11 Middle of lower slab	18.291	18.019	1.015

Source : Study Team

Arrangement of reinforcing bars at web and lower slab shall be as follows:

Table 8.4.1-25 Arrangement of reinforced bars

		Arrangement of reinforcing bars	Quantity (mm2/m)
Web	Upper end	D19ctc125	2292.0
	Lower end	D19ctc125	2292.0
Lower Slab	Connected area	D19ctc125	2292.0
	middle	D13ctc125	1013.6

Source : Study Team

Table 8.4.1-26 Stress of reinforcing bars

		JSHB		FEM Analysis		Restricted value
		σc (N/mm2)	σs (N/mm2)	σc (N/mm2)	σs (N/mm2)	
Dead Load	*6	0.2	4.6	0.2	4.6	σs < 100
	*7	1.5	34.4	1.5	34.4	σs < 100
	*10	2.5	47.0	2.5	47.0	σs < 100
	*11	2.6	79.6	2.6	79.6	σs < 100
Service Load	*6	5.4	150.4	1.3	37.1	σs < 180
	*7	5.4	123.7	2.1	48.8	σs < 180
	*10	9.3	172.0	3.5	64.5	σs < 180
	*11	3.5	106.6	2.7	82.2	σs < 180

Source : Study Team

Table 8.4.1-27 Safety factor under ultimate load working state [B]

Mu＝1.3*D+2.5*(LL+IM)+Ps+Cr+SH [JSHB-16]

Mu＝1.25*DC+1.5*DW+1.75*(LL+IM)+1.2or0.5*(Cr+SH) [P=100kN]

Designed cross section		JSHB Mu (kN・m)	FEM Analysis (P=100kN) Mu(kN・m)	Mr (kN・m)
Web	*6	232.306	55.221	< 280.116
	*7	130.308	39.825	< 202.573
Lower Slab	*10	122.012	34.157	< 142.964
	*11	30.211	18.019	< 65.044

Here, Mr is a bending fracture resistance.

Source : Study Team

d) Cross section of Intermediate support section [Upper Slab] - [C]

Table 8.4.1-28 Bending moment (live load) [C]

Designed point	JSHB (kN・m)	FEM Analysis (P=100kN) (kN・m)	JSHB/FEM
*1 Connected part for cantilever slab	-150.150	-122.736	1.223
*2 Connected part in box	-153.666	-137.552	1.117
*3 Middle of box girder	95.338	39.243	2.429

Source : Study Team

Table 8.4.1-29 Designed bending moment (D+LL+IM+Ps+Cr+SH) [C]

Designed point	JSHB (kN・m)	FEM Analysis （P=100kN） (kN・m)	JSHB/FEM
*1 Connected part for cantilever slab	-223.388	-195.974	1.140
*2 Connected part in box	-212.814	-196.700	1.082
*3 Middle of box girder	91.689	35.594	2.576

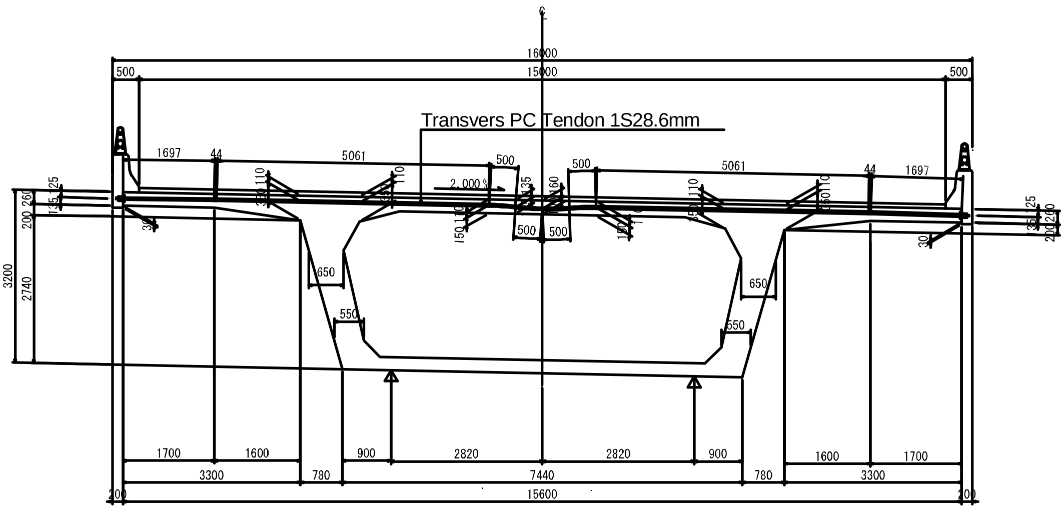
Source : Study Team

Table 8.4.1-30 Composite bending stress [C]

		JSHB		FEM Analysis		Remarks
		σco (N/mm2)	σcu (N/mm2)	σco (N/mm2)	σcu (N/mm2)	
Dead Load	*1	3.57	0.83	3.57	0.83	
	*2	4.00	-0.23	4.00	-0.23	
	*3	0.92	7.15	0.92	7.15	
Service Load	*1	-0.68	5.09	0.10	4.31	
	*2	0.82	2.95	1.15	2.62	
	*3	9.38	-1.32	4.40	3.66	
			> -3.0	> 0.0	> 0.0	

Source : Study Team

Transversal PC tendon shall be 1S28.6mm and arrange with interval of ctc500mm shown in following Figure.



Source : Study Team

Figure 8.4.1-33 Arrangement of Transversal PC Tendon
(intermediate support cross section) ctc500mm

Table 8.4.1-31 Safety factor under ultimate load working state [C]

Mu＝1.3*D+2.5*(LL+IM)+Ps+Cr+SH [JSHB-16]

Mu＝1.25*DC+1.5*DW+1.75*(LL+IM)+1.2or0.5*(Cr+SH) [P=100kN]

Designed point	JSHB (kN･m)	FEM Analysis （P=100kN） (kN･m)	Mr (kN･m)
*1 Connected part for cantilever slab	452.710	315.451	< 581.201
*2 Connected part in box girder	378.664	316.161	< 718.755
*3 Middle of box girder	190.122	76.445	< 239.144

Here, Mr is a bending fracture resistance.

Source : Study Team

e) Cross section of Intermediate support section [Web and Lower deck slab] - [D]

Table 8.4.1-32 Bending moment (live load) [D]

Designed point	JSHB (kN･m)	FEM Analysis （P=100kN） (kN･m)	JSHB/FEM
*6 Upper end of web	104.673	35.027	2.988
*7 Lower end of web	57.292	13.132	4.363
*10 Connected part of lower slab	51.612	3.844	13.427
*11 Middle of lower slab	1.057	0.157	6.732

Source : Study Team

Table 8.4.1-33 Designed bending moment (D+LL+IM+Ps+Cr+SH) [D]

Designed point	JSHB (kN･m)	FEM Analysis （P=100kN） (kN･m)	JSHB/FEM
*6 Upper end of web	109.496	29.952	3.656
*7 Lower end of web	76.602	32.913	2.327
*10 Connected part of lower slab	66.694	18.925	3.524
*11 Middle of lower slab	12.388	11.488	1.078

Source : Study Team

Arrangement of reinforcing bars at web and lower slab shall be as follows:

Table 8.4.1-34 Arrangement of reinforced bars

		Arrangement of reinforcing bars	Quantity(mm2/m)
Web	Upper end	D19ctc125	2292.0
	Lower end	D19ctc125	2292.0
Lower Slab	Connected	D19ctc125	2292.0
	Middle	D13ctc125	1013.6

Source : Study Team

Table 8.4.1-35 Stress of reinforcing bars

		JSHB		FEM Analysis		Restricted value
		σc (N/mm2)	σs (N/mm2)	σc (N/mm2)	σs (N/mm2)	
Dead Load	*6	0.1	5.6	0.1	5.6	σs < 100
	*7	0.5	18.5	0.5	18.5	σs < 100
	*10	2.2	40.7	2.2	40.7	σs < 100
	*11	2.1	66.1	2.1	66.1	σs < 100
Service Load	*6	2.2	85.6	0.6	23.4	σs < 180
	*7	2.1	71.7	0.9	31.1	σs < 180
	*10	9.7	180.0	2.8	51.1	σs < 180
	*11	2.4	72.2	2.2	67.0	σs < 180

Source : Study Team

Table 8.4.1-36 Safety factor under ultimate load working state [D]

Mu＝1.3*D+2.5*(LL+IM)+Ps+Cr+SH [JSHB-16]

Mu＝1.25*DC+1.5*DW+1.75*(LL+IM)+1.2or0.5*(Cr+SH) [P=100kN]

Designed point		JSHB (kN・m)	FEM Analysis (P=100kN) (kN・m)	Mr (kN・m)
Web	*6	256.314	63.292	< 480.253
	*7	154.896	51.320	< 402.764
Lower Slab	*10	132.654	26.828	< 142.964
	*11	19.908	14.407	< 65.044

Here, Mr is a bending fracture resistance.

Source : Study Team

8.4.1.3 Design Result of Approach Bridge on Cat Hai Island Side

(1) Arrangement of Members of Main Girder

The study shall be carried out to determine minimum thickness of each member of main girder.
As for the basic conditions to determine each member's dimension, the followings are considered:

1) Basic Condition of Arrangement of Members

a) The following PC tendons shall be considered to determine member thickness.

- PC tendon for longitudinal direction
  *12S12.7mm (arranged in upper & lower slab and web)
- PC tendon for transversal direction
  *1S28.6mm (arranged in upper slab)

b) Reinforcing bars to be used are as follows:

- Reinforcing bars in upper slab:
  *Reinforcing bars to be arranged shall be D12 to D14, because upper slab is designed as PC members.
- Reinforcing bars in lower slab:
  *Reinforcing bars to be arranged shall be D14 to D19, because lower slab is designed as RC members.

c) Concrete covering for reinforcing bars is as follows: (for salt damage area)

Table 8.4.1-37 Concrete covering for reinforcing bars (TCXDVN327: 2004)

Region		Upper (Inside)	Under (Outside)	Remark
*1 Upper Slab	Cantilever	40mm	60mm	Fc=40Mpa
	Inside of Box	40mm	40mm	
*2 Lower Slab		40mm	60mm	
*3 Web		40mm	60mm	

Source : Study Team

d) Approximate dimension of each member is as follows from achievements.

(by "PC Bridge Planning Manual")

Table 8.4.1-38 Approximate dimension of each member

*1 Upper Slab

	Main Girder Shape	Structure	D1(m)	D2(m)	D3(m)	D4(m)
Upper Slab	1Box girder	RC	0.24-0.30	0.30-0.50	0.30-0.50	0.25
		PC	0.25-0.35	0.35-0.65	0.35-0.65	0.25

*2 Lower Slab

	Main Girder Shape	Structure	Span Center T1(m)	Middle Support T2(m)
Lower Slab	1Box girder	RC	0.21-0.30	0.40-1.50

*3 Web

	Main Girder Shape	Structure	Span Center T1(m)	Middle Support T2(m)
Web	1Box girder	RC	0.35-0.50	0.50-0.70

Source : Study Team

2) Thickness of Upper Slab

Determination of thickness of upper slab shall be done by considering the below:

a) Minimum thickness considering the fatigue because live load is on directly.

b) Necessary thickness for arrangement of PC tendon and reinforcing bars.

a) Minimum thickness of upper slab which supports live load directly.

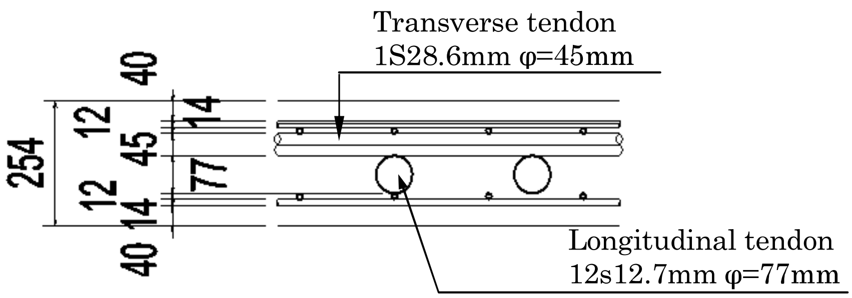
The thickness shall be determined in accordance with JSHB.

*1. Minimum thickness of pre-stressed concrete slab, t>160mm

*2. Minimum thickness of tip of the cantilever deck slab, t>200mm

b) Necessary thickness for arrangement of tendon

Table 8.4.1-39 Necessary thickness for arrangement of tendon



Cover	40mm+40mm	80mm	
Reinforcement Bar	(D12+D14)x2	52mm	
Transvers Tendon	Sheath φ45	45mm	
Longitudinal Tendon	Sheathφ77	77mm	(#1075)
Total		254mm	

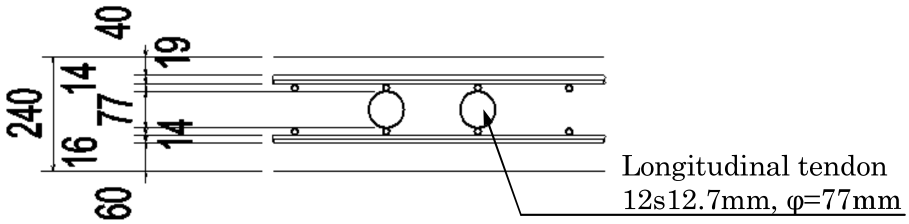
Source : Study Team

Therefore, minimum thickness of upper slab shall be 260mm considering any margin.

c) Thickness of Lower Slab

Thickness of lower slab is normally determined even thicker around intermediate support point comparing to span center, due to its compression force by negative bending moment in longitudinal direction and its serviceability of the road.

Table 8.4.1-40 Thickness of Lower Slab



Cover	40mm+60mm	100mm	
Reinforcement Bar	D19+D14x2+D16	63mm	
Longitudinal Tendon	Sheathφ77	77mm	(#1075)
Total		240mm	

Source : Study Team

d) Thickness of Web

Thickness of web shall be determined considering the following particulars.

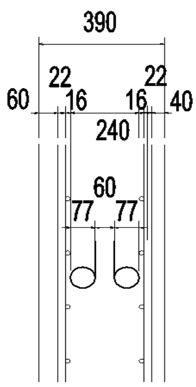
- Necessary thickness against shearing force of the main girder
- Necessary thickness against flexural capacity as transversal members
- Necessary thickness for arrangement of PC tendons

Necessary thickness against shear capacity of main girder shall be adjusted at the detailed design stage, and necessary thickness for arrangement of tendon shall be determined.

- Necessary thickness for arrangement of tendon

PC tendon for longitudinal direction shall be arranged for 2 line in the web

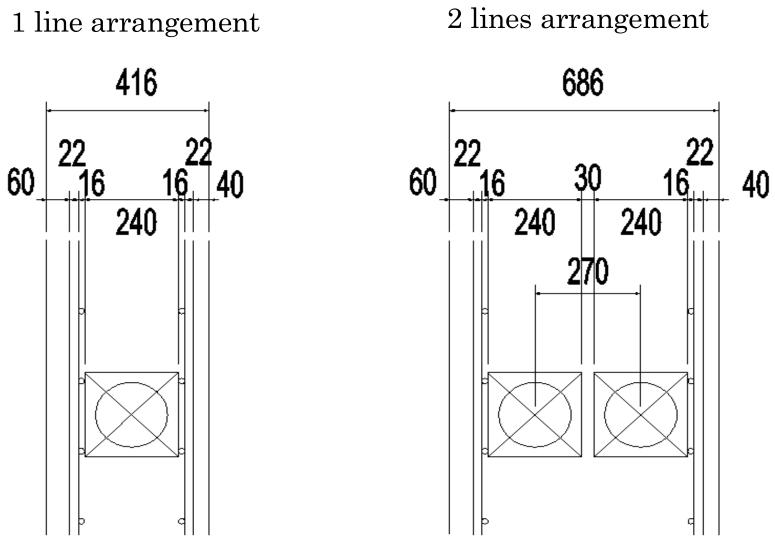
Table 8.4.1-41 Necessary thickness for arrangement of cable



Cover	60mm+40mm	100mm
Reinforcement Bar	(D22+D16)x2	76mm
Longitudinal Tendon	Sheath φ77	77mm
Longitudinal Tendon	Sheath φ77	77mm
		60mm
Total		390mm

Source : Study Team

- Necessary thickness for anchorage of PC Tendon



416mm > 2x180mm=360mm　　686mm > 2x180mm+270mm=630mm

Source : Study Team

Figure 8.4.1-34 Necessary thickness for anchorage of PC Tendon

(2) Main girder segment arrangement

Approach bridges at Cat Hai side is constructed by free cantilever method, it is necessary to decided segment arrangement.

a) Traveler

In selection of traveler, main aspect is span length and bridge width.

The standard classification of traveler is as follows:

Table 8.4.1-42 Standard classification of traveler

	unit	Common Traveler			Large Traveler
Number of girder	nos	2	3	4	2
Width	m	14	17	20	14
Capacity	KNm	2000	3000	4000	3500
Maximum Length	m	4	4	4	5
Total Weight of Traveler	kN	850	1050	1300	1200

Source : Study Team

After considering span length (60m) and space of pier table, bridge width and limited schedule, Large size traveler with adjustment of its width was selected.

b) Pier table

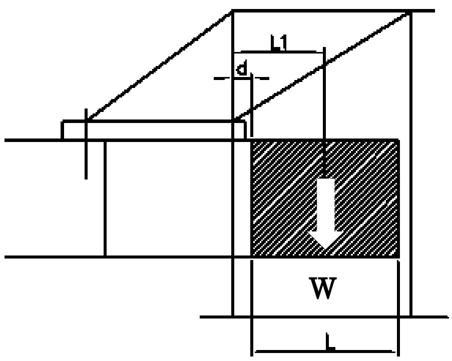
Length of pier table is decided depending on traveler's size. The following table shows standard required pier table length for each size of traveler..

Table 8.4.1-43 Length of Pier Table

	unit	Common Traveler	Large Traveler
Length of Pier Table	m	12.00	15.00

Source : Study Team

c) Segment length in cantilever erection



Here,

M(kNm) = W* L1 < 3500kNm (Capacity)

L1(m) = (L/2)+d

W(kN) = V*24.5

d(m) = 0.50m

L(m) = Block Length <5.00m

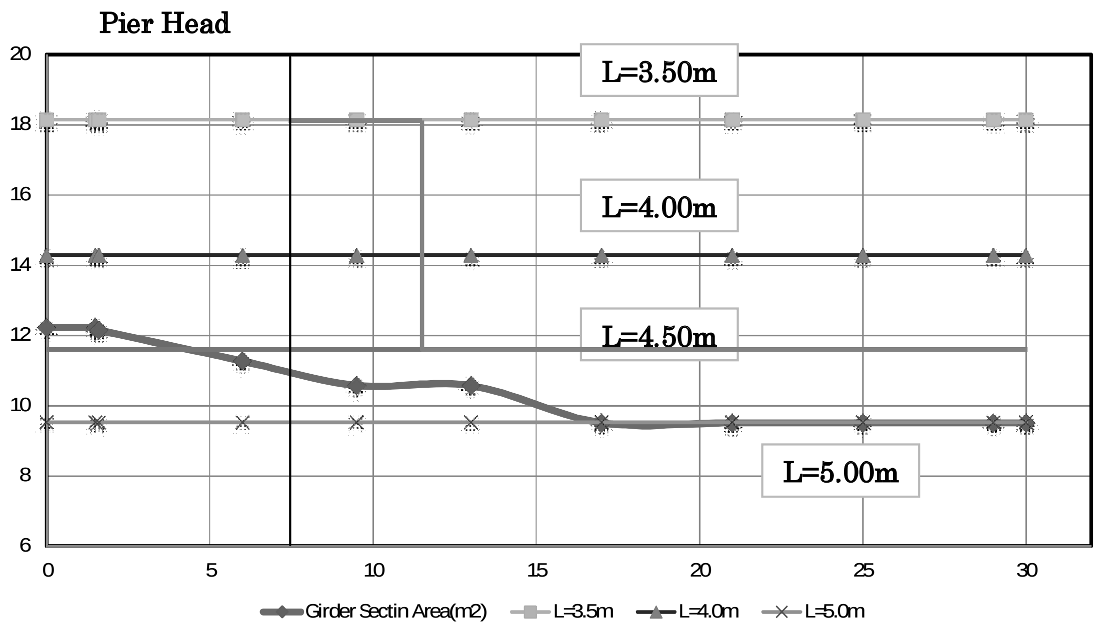
If capacity of traveler is 3500kNm, segment length in this design can be determined as follows:

Table 8.4.1-44 Relation between segment length and area of main girder

Segment length L (m)	Reach L1(m)	Capacity Moment M(kNm)	Maximum section area A(m2)
3.00	2.00	3500	23.810
3.50	2.250		18.141
4.00	2.50		14.286
4.50	2.75		11.544
5.00	3.00		9.524

Source : Study Team

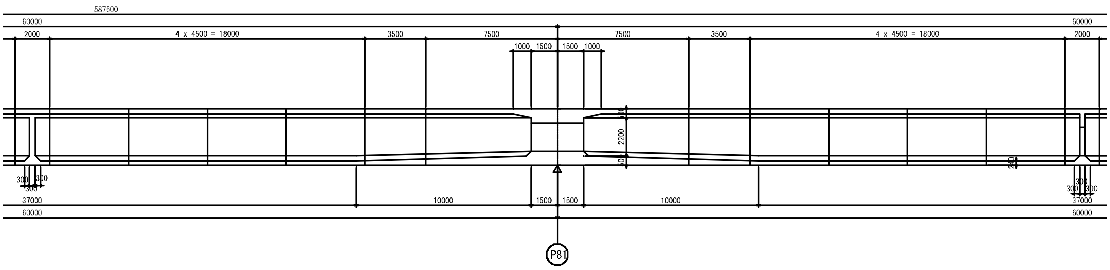
Based on the above said table, it is necessary to check the segment length as follows:



Source : Study Team

Figure 8.4.1-35 Segment length-section area

As conclusion, selection of segment length (3.5 & 4.5m in this design) is proper.



Source : Study Team

Figure 8.4.1-36 Segment arrangement

(3) Design of Longitudinal Direction of Main Girder

1) Outline of Design

It was decided to design as continuous girder by free cantilever erection method based on the comparison study of construction methods at Cat Hai Island side. This construction method is popular in Vietnam. In this design, it was assumed that large size traveler should be used dut to wide bridge width (16m). Pier table segment and other segment's length was decided by this traveler's capacity (M = 3500kNm).

2) Design condition

a) Dead Load

*Concrete self weight: γc = 24.50kN/m3

*Asphalt pavement (Thickness t=75mm) : γs = 22.50kN/m3

*Curb : Concrete high column weight w = 7.575×2 = 15.15kN/m
: Hand Rail weight wh = 0.600×2 = 1.200kN/m

*water service tube φ400 (Article 2) : W = 2.250×2 = 4.500kN/m

b) Live Load and Impact Load

*Live Load : Vehicular live loading (HL-93) 4 Lanes
*Dynamic Load : Impact factor IM=0.25

c) Temperature

*Uniform Temperature: 40 ˚C (For Bearing and Expansion design)

±20˚C (For Girder Design)

*Temperature Gradient: As shown in following Table.

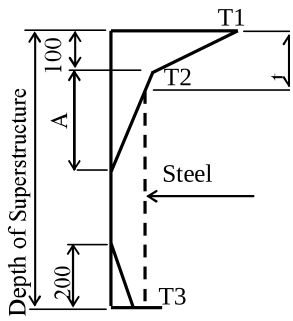


Table 8.4.1-45Temperature Gradient

	T1	T2	T3
Positive	+23	+6	+3
Negative	-7	-1	0

Source : Study Team

d) Earthquake Load

*Design horizontal seismic intensity : Kh=0.18

e) Load Factor and Load Combination

Table 8.4.1-46 Load Factor and Load Combination

Load Combination / Limit State	DC DD D W EH EV ES	LL IM CE BR PL LS EL	TL	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These At a Time: EQ	CT	CV
Strength-I	γ_P	1.75	1.75	1.00	-	-	1.00	0.50/1.20	γTG	γSE	-	-	-
Strength-II	γ_P	-	-	1.00	1.40		1.00	0.50/1.20	γTG	γSE	-	-	-
Strength-III	γ_P	1.35	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γTG	γSE	-	-	-
Extreme	γ_P	0.50	0.50	1.00	-	-	1.00	-	-	-	1.00	1.00	1.00
Service	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00/1.20	γTG	γSE	-	-	-
Fatigue – LL, IM & CE only	-	0.75	0.75	-	-	-	-	-	-	-	-	-	-

Source : Study Team

3) Material

a) Concrete

Table 8.4.1-47 Properties of Concrete (Main Girder)

Item		unit	Girder	Remark
Specified Compression Strength fc		Mpa	40	
Compressive Strength at Pre-stressing fci		Mpa	33	
Modulus of Elasticity Ec		Mpa	31000	
Allowable Stress				
Compressive Stress	Before Losses	Mpa	19.0	
	D.L.W.S	Mpa	15.0	
	Service Limit State	Mpa	15.0	
Tensile Stress	Before Losses	Mpa	1.50	
	D.L.W.S	Mpa	0.00	
	Service Limit State	Mpa	1.50	
Modulus of Rupture		Mpa	13478	

Source : Study Team

b) Pre-stressing Steel

Table 8.4.1-48 Properties of Pre stressing Steel (PC Strand Cable)

Item		unit	12S12.7L	1S28.6
Tensile Strength fpu		Mpa	1850	1800
Yield Strength fpy		Mpa	1600	1500
Modulus of Elasticity Ep		Mpa	200000	200000
Allowable tensile strength	During pre-stressing	Mpa	1440	1350
	After pre-stressing	Mpa	1295	1260
	At design load	Mpa	1110	1080

Source : Study Team

Table 8.4.1-49 Properties of Pre-stressing Steel (PC Bar - SBPR930/1180)

Item		unit	φ32	Remark
Tensile Strength fpu		Mpa	1180	
Yield Strength fpy		Mpa	930	0.9fpu
Modulus of Elasticity Ep		Mpa	200000	
Allowable tensile strength	During pre-stressing	Mpa	837	0.9fpy
	After pre-stressing	Mpa	790	0.85fpy
	At design load	Mpa	697	0.75fpy

Source : Study Team

Table 8.4.1-50 Properties of Pre-stressing Steel (Reinforcement Bar)

Item	unit	SD345	Remark
Tensile Strength fpu	Mpa	490	
Yield Strength fpy	Mpa	345	
Modulus of Elasticity Ep	Mpa	200000	
Allowable tensile strength at design load	Mpa	180	

Source : Study Team

4) Model for structural analysis

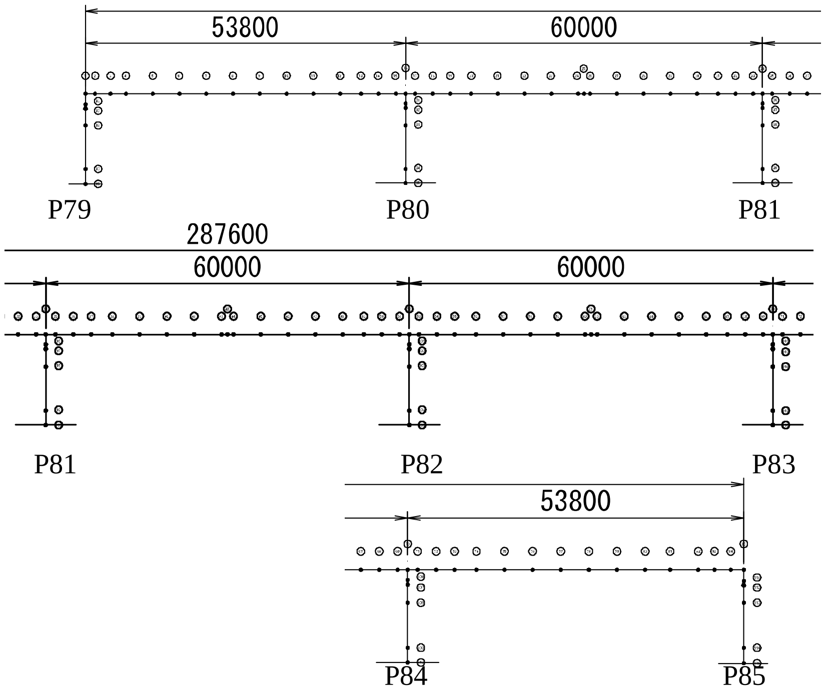
The analysis as a frame structure by linear theory shall be carried out by modelling the structure in the following way.

1. The structure model shall consist of the lines connecting the centroid axes of the members, with the axis of the girder made to coincide with the design profile alignment of the completed system.

2. The zones of junction between the towers, girder and piers shall be considered as rigid zones. The rigidity of these rigid zones shall be assumed as 1000 times that of the adjacent members.

3. Supports shall be modelled to correspond with the function of the type of bearing actually used.

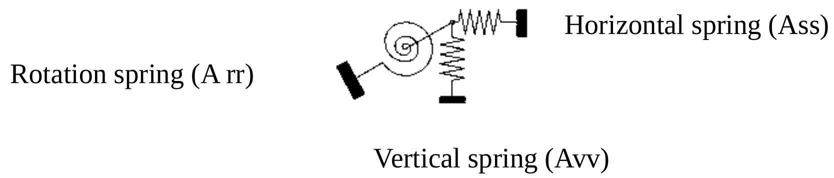
4. The foundation structure and ground shall be modelled as elastic springs having equivalent behaviour. Also, the effect of variation shall be considered corresponding with the ground and the foundation



Source : Study Team

Figure 8.4.1-37 Analysis Model

Table 8.4.1-51 Spring Constants of Pile



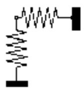
P79-P84	unit	Service state	Earthquake
Horizontal (Ass)	kN/m	3.169339E+05	1.043346E+05
Compound spring (Asr=Ars)	kN/rad	-2.013796E+06	-4.520726E+06
Rotational angle (Arr)	kNm/rad	1.736875E+08	1.831579E+08
Vertical (Avv)	kN/m	8.807076+006	8.807076E+06

Source : Study Team

Table 8.4.1-52 Spring Constants of Shoe

Kh：Horizontal spring

Kv：Vertical spring



P79-P84	unit	End Support	Middle Support
Vertical direction	kN/m	2518000	15018000
Horizontal direction	kN/m	10838	42250

Source : Study Team

5) Structural analysis

The structural analysis shall be performed with consideration to the following items.

a) The procedure and time schedule for completing the structural system shall be determined, and the structural analysis performed accordingly.

b) The following loads shall be considered as the loads which will work during the construction stage.

c) Primary loads

- Dead load of girder
- Pre-stressing force (internal cables)
- Creep and shrinkage of concrete

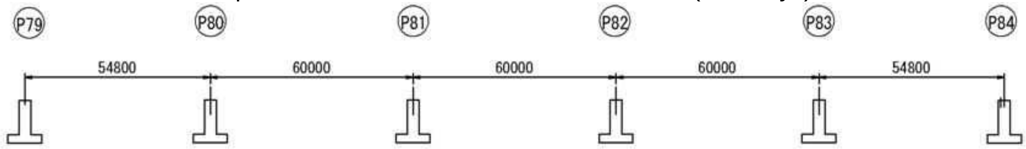
d) Construction loads

- Dead load of the travellers for cantilever construction (including formwork)
- Dead load of scaffolding for stay cable installation (when necessary)
- Dead load of false works for closing segments (including formwork)

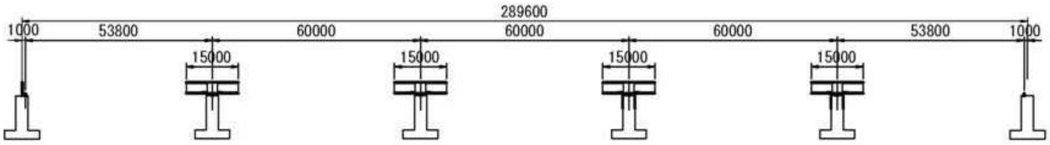
e) The points of time when superimposed dead load and live load will be applied shall be assumed, and analysis performed for the stress resultants acting at such times.

The process and the schedule of the structure analysis are as following Figures.

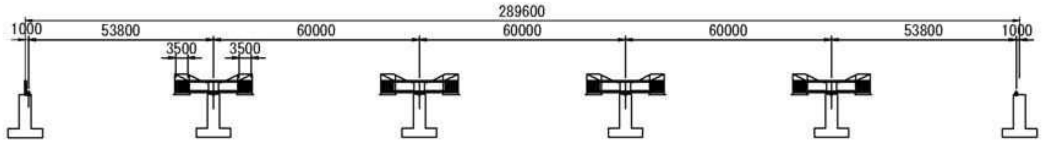
Step 1.Construction of Pier and Foundation (360 days)



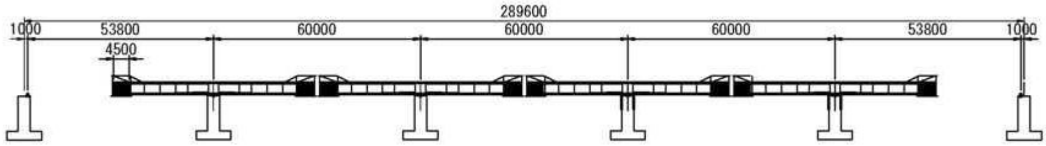
Step 2. Construction of Pier Table（60 days）



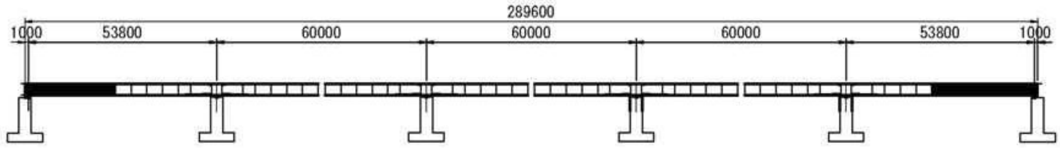
Step3. Construction of First Segment (12 days)



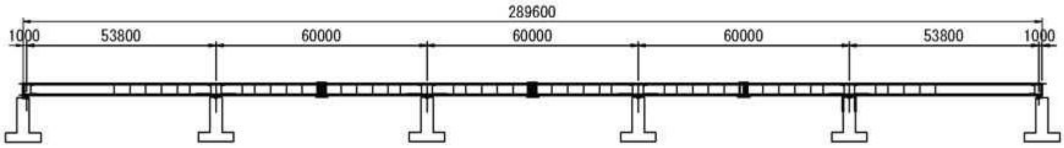
Step4.～Step7 Construction of Second Segment from 5th Segment (cycle time12 days)



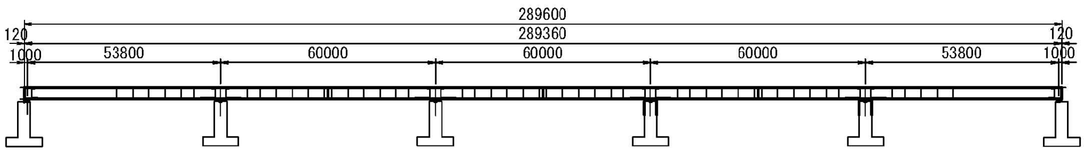
Step8. Construction of Side span　(60 days)



Step9. Construction of center closer (20 days) (Completion of Structure)



Step10. Construction of Surfacing (60days)



Source : Study Team

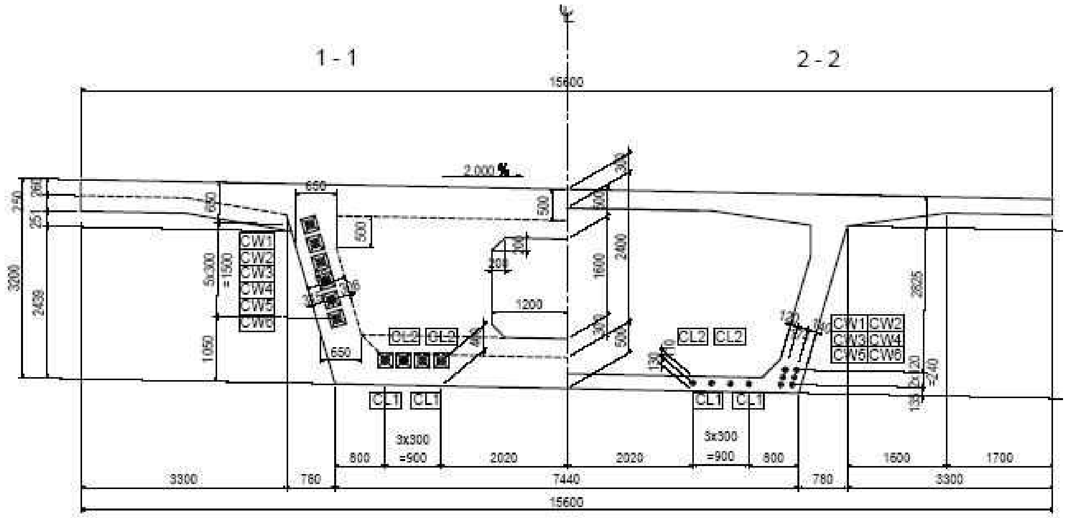
Figure 8.4.1-38 Process and the schedule of the structure analysis

6) Arrangement of PC Tendon in main Girder

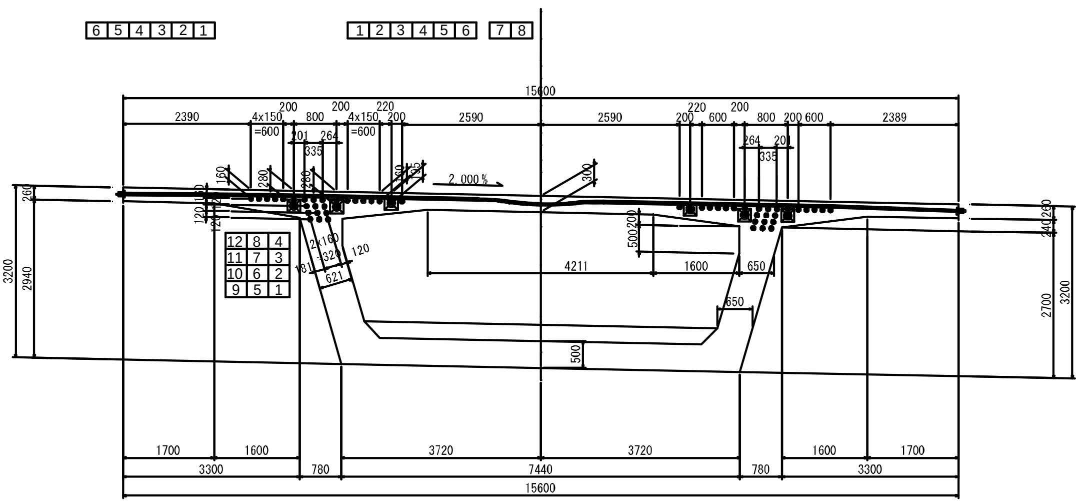
We show the arrangement of the outer cable and inner cable in following Figures.

Span Center Section

Girder End Side span



Middle Support Section



Source : Study Team

Figure 8.4.1-39 Arrangement of PC Tendon

7) Design result of Main girder

a) Approach Bridge P79-P84 (52.98m+3@60.0m+52.98m=285.96m)

<1> Bending Moment of Main Girder

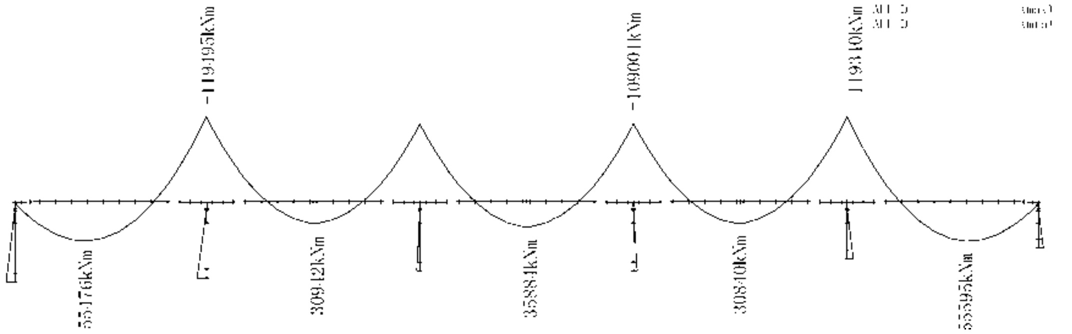
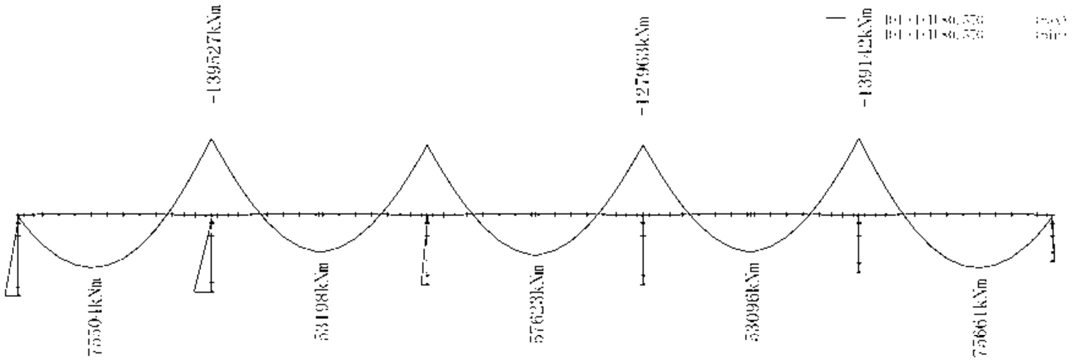


Figure 8.4.1-40 Bending Moment in all Dead load state



Source : Study Team

Figure 8.4.1-41 Bending Moment in Service load state

<2> Fiber Stress of Main Girder

*Dead Load State (Top fiber stress)

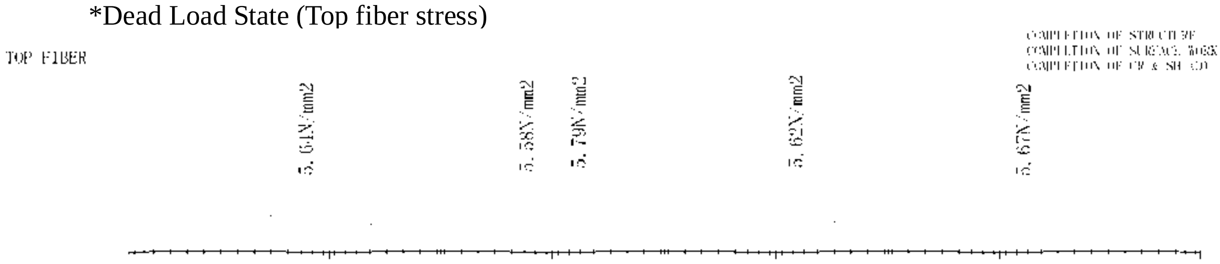


Figure 8.4.1-43 Stress of Top Fiber (Dead Load)

*Dead Load State (Bottom fiber stress)

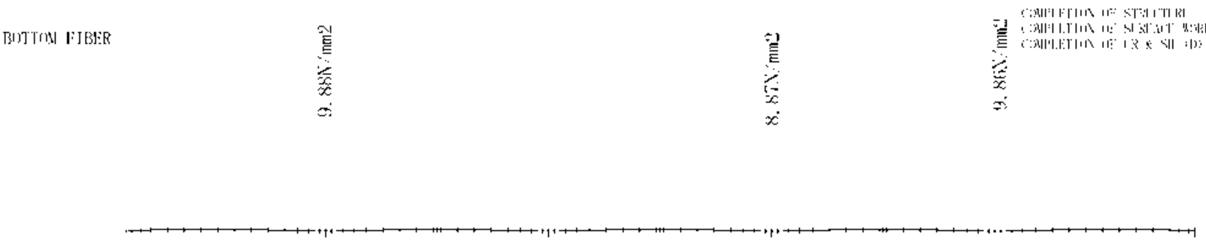


Figure 8.4.1-44 Stress of Bottom Fiber (Dead Load)

Source : Study Team

*Service Limit State (Top fiber stress)

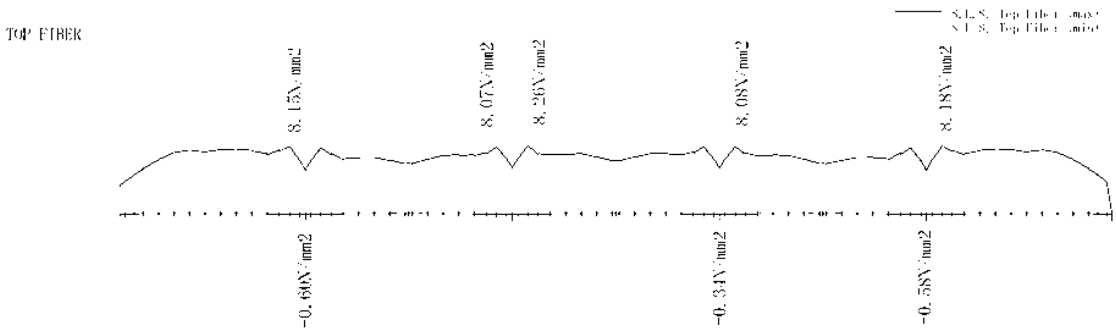
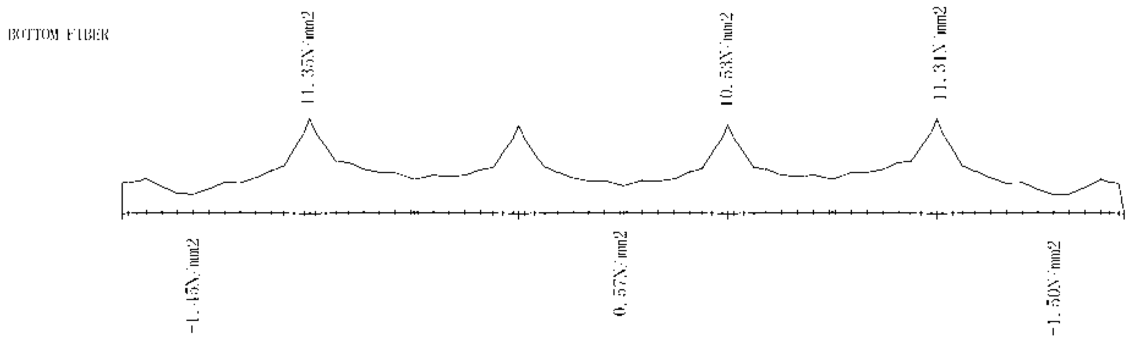


Figure 8.4.1-45 Stress of Top Fiber (Service Limit State)

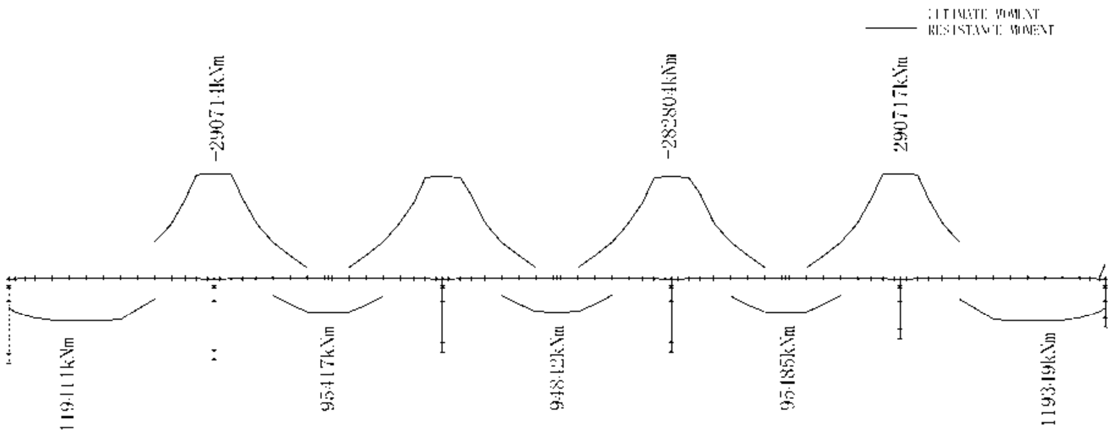


Source : Study Team

Figure 8.4.1-46 Stress of Bottom Fiber (Service Limit State)

<3> Ultimate Bending Moment of Main Girder

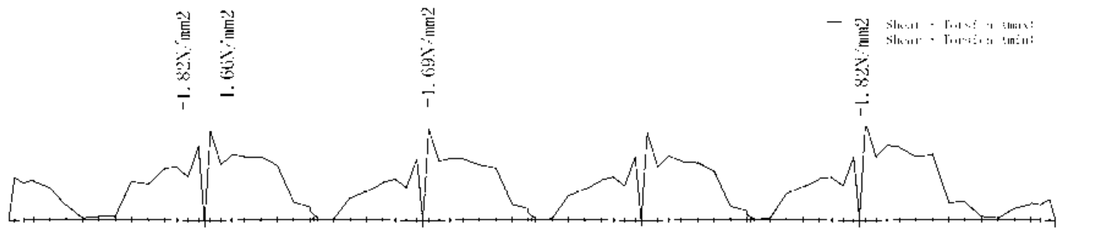
*Strength Limit State



Source : Study Team

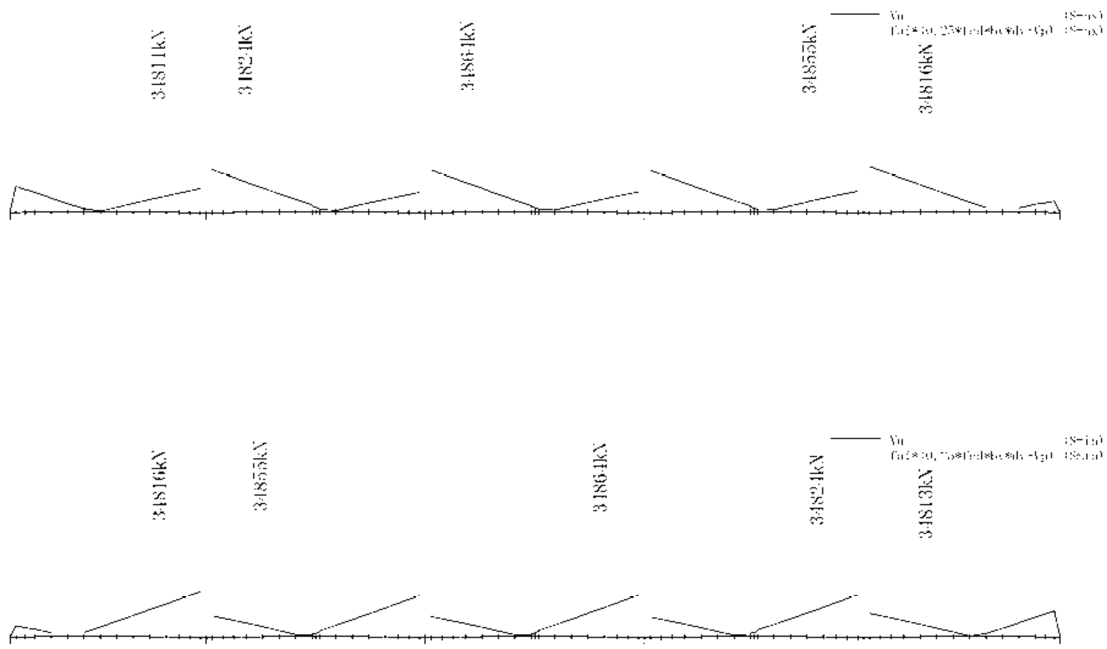
Figure 8.4.1-47 Bending Moment of Main Girder (Strength Limit State)

<4> Shear Stress



Source : Study Team

Figure 8.4.1-48 Diagonal tensile stress



Source : Study Team

Figure 8.4.1-49 Maximum Shear stress

b) Approach Bridge P84-A2 (52.98m+2@60.0m+52.98m=285.96m)

<1> Bending Moment of Main Girder

*Dead load state

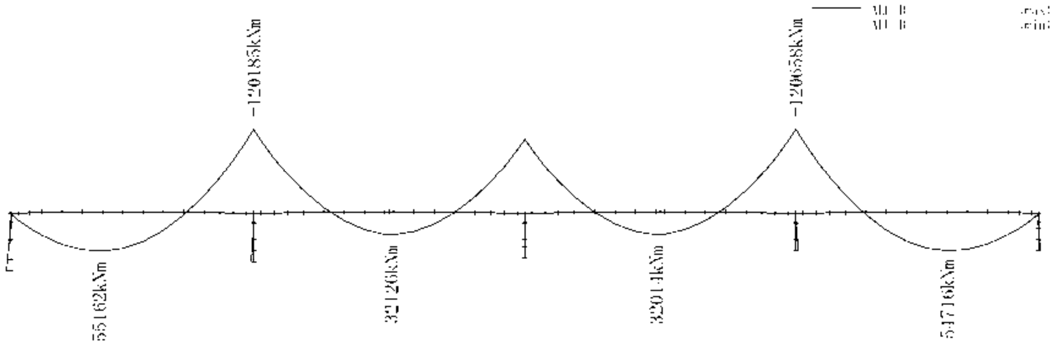


Figure 8.4.1-50 Bending Moment caused by Dead Load

*Service load state

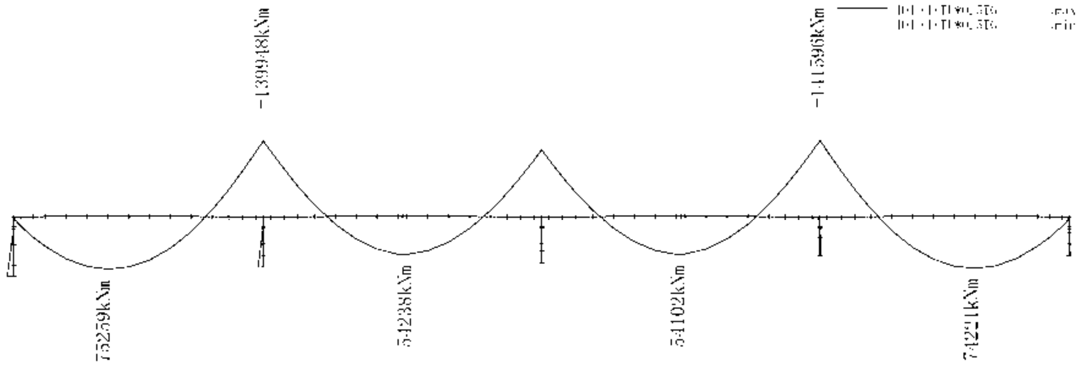
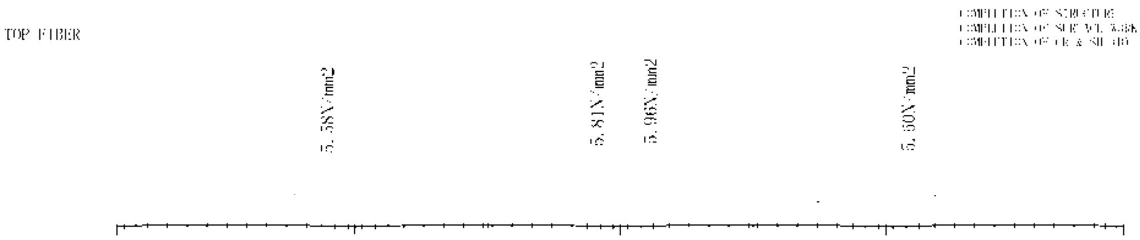


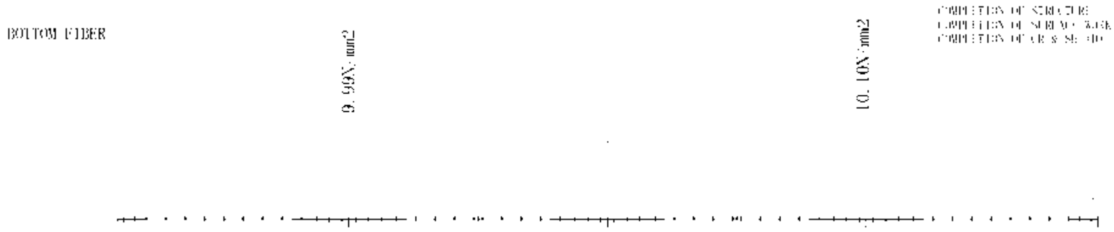
Figure 8.4.1-51 Bending Moment in Service

<2> Fiber Stress of Main Girder

*Dead Load State (Top fiber)



*Dead Load State (Bottom fiber)



Source : Study Team

Figure 8.4.1-53 Stress of Bottom Fiber (Dead Load)

*Service Limit State (Top fiber)

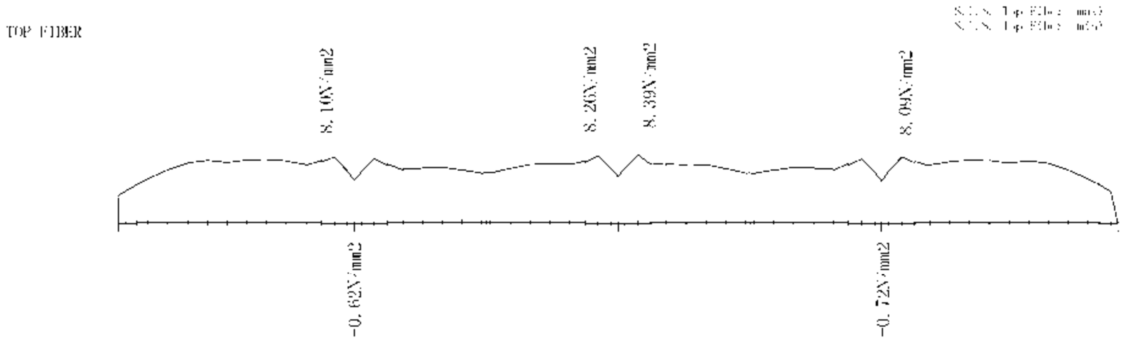
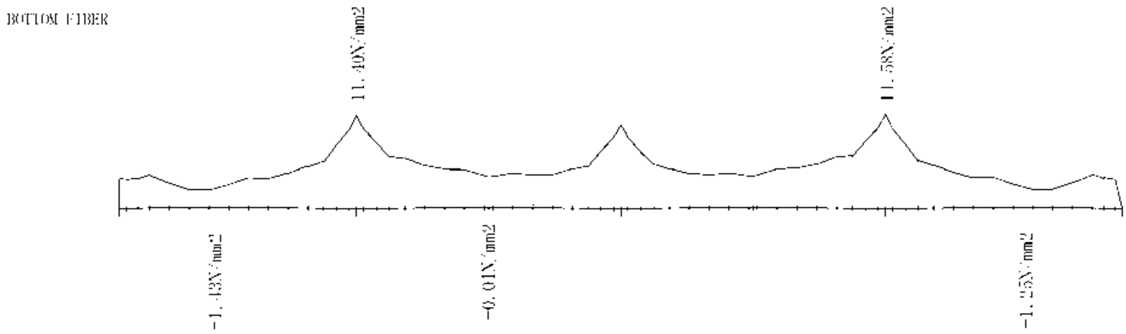


Figure 8.4.1-54 Stress of Top Fiber (Service Limit State)

*Service Limit State (Bottom fiber)

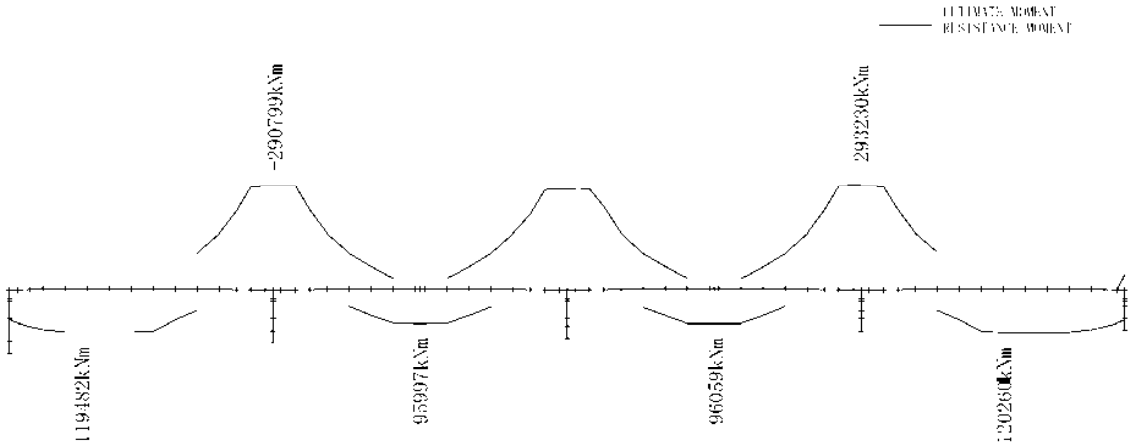


Source : Study Team

Figure 8.4.1-55 Stress of Bottom Fiber (Service Limit State)

<3> Ultimate Bending Moment of Main Girder

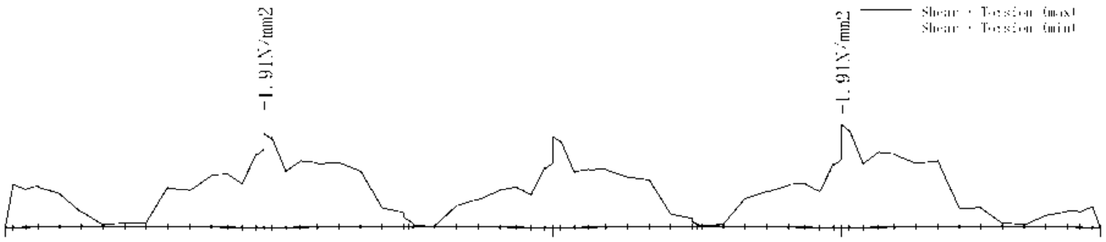
*Strength Limit State



Source : Study Team

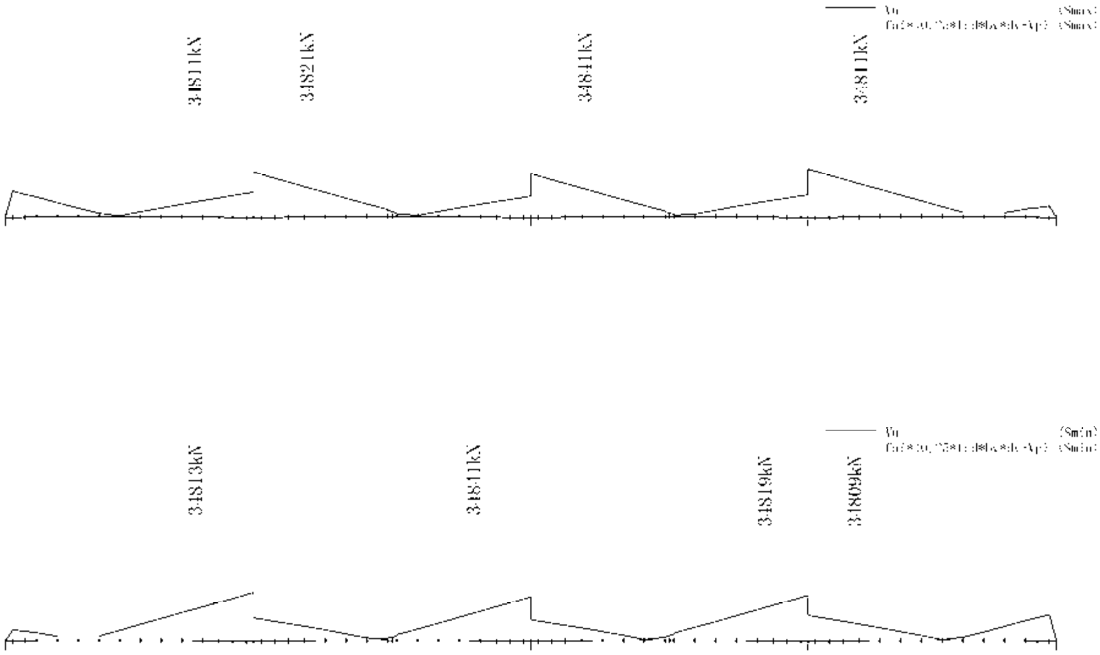
Figure 8.4.1-56 Bending Moment of Main Girder (Strength Limit State)

<4> Shear Stress



Source : Study Team

Figure 8.4.1-57 Diagonal tensile stress



Source : Study Team

Figure 8.4.1-58 Maximum Shear stress

(4) Design of Transversal Direction of Main Girder

1) General

This is a design report for transversal direction of approach bridge at Cat Hai side planned in connecting road to the Lach Huyen Port. Upper slab deck of the approach bridge is PC structure with transversal prestressing tendon of 1S28.6mm (SWPR19L) as the same as the bridge to Tan Vu side approach bridges. Structural features of each member (upper slab , web and lower slab) for the main girder is shown in following Table. Cantilevering construction method with cast-in-situ concrete shall be adopted for this bridge.

Table 8.4.1-53 Structure clarification of Main girder

Member	Structure Clarification	Remarks
Upper slab	PC structure	1S28.6mm
Web	RC structure	
Lower slab	RC structure	

Source : Study Team

As for design bending moment, safety for transversal direction shall be confirmed by formula of bending moment of live load (T load) instructed in JSHB (Japanese Specifications for Highway Bridges III 7.4.2) and bending moment calculated by FEM analysis of slab structure which is directly loaded wheel load P (=145/2*1.25=91kN≒100 k N).

a) Upper deck slab

As for design live load, flexural stress by bending moment of each for formula instructed in JSHB (Japanese Specifications for Highway Bridges) and direct wheel load (100kN) shall be set as follows:

Table 8.4.1-54 Direct wheel load on Upper deck slab

	JSHB	FEM analysis
Under Dead load working state	Full prestressed	
Under service loads(Live load)	Limit state for cracking	Full prestressed

Source : Study Team

b) Web and Bottom slab

Web and Lower slab shall be designed as follows:

- For JSHB : stress of reinforcing bar under service load working state shall be less than σsa = 180N/mm2
- For FEM : stress of reinforcing bar under service load working state shall be less than σsa = 140N/mm2

2) Comparison on bending moment due to live load

Bending moment for live load by each formula (JSHB III.7.4.2 and FEM analysis: (P=100kN)) shall be compared.

a) Standard Cross Section [Upper slab] - [A]

Table 8.4.1-55 Bending moment (Live load) [A]

Designed point	JSHB (kN･m)	FEM Analysis （P=100kN） (kN･m)	JSHB/FEM
*1 Connected part for cantilever slab	-150.150	-126.076	1.191
*2 Connected part in box girder	-158.000	-122.827	1.286
*3 Middle of box girder	98.095	40.811	2.404

Source : Study Team

Table 8.4.1-56 Designed Bending moment (D+LL+IM+Ps+Cr+SH) [A]

Designed point	JSHB (kN･m)	FEM Analysis （P=100kN） (kN･m)	JSHB/FEM
*1 Connected part for cantilever slab	-223.806	-199.732	1.122
*2 Connected part in box girder	-261.957	-226.784	1.155
*3 Middle of box girder	60.170	2.886	20.849

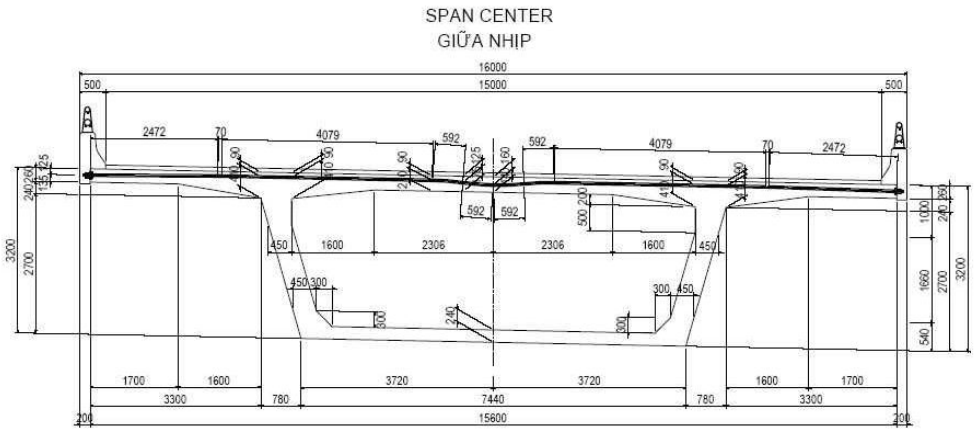
Source : Study Team

Table 8.4.1-57 Composite bending stress [A]

		JSHB		FEM Analysis		
		σco (N/mm2)	σcu (N/mm2)	σco (N/mm2)	σcu (N/mm2)	
Dead Load	*1	4.01	-0.05	4.01	-0.05	
	*2	3.36	0.65	3.36	0.65	
	*3	0.27	6.73	0.27	6.73	
Service Load	*1	0.42	3.55	0.98	2.97	
	*2	-0.44	4.43	0.41	3.60	
	*3	6.81	0.19	2.99	4.00	
			> -3.0	> 0.0	> 0.0	

Source : Study Team

Transversal PC tendon shall be 1S28.6mm and arranged with interval of less than ctc500mm as shown in following Figure.



Source : Study Team

Figure 8.4.1-59 General Section for Arrangement of Transversal PC Tendon (ctc550)

Table 8.4.1-58 Safety factor for ultimate load working state [A]

Mu＝1.3*D+2.5*(LL+IM)+Ps+Cr+SH [JSHB-16]

Mu＝1.25*DC+1.5*DW+1.75*(LL+IM)+1.2 or 0.5*(CR+SH) [P=100kN]

Designed point	JSHB (kN・m)	FEM Analysis (P=100kN) (kN・m)	Mr (kN・m)
*1 Connected part for cantilever direction	453.253kN・m	321.819 kN・m	< 677.837
*2 Connected part in box girder	432.934 kN・m	344.841 kN・m	<677.837
*3 Middle of box girder	160.828 kN・m	63.396 kN・m	< 225.587

Source : Study Team

b) Standard Cross Section [Web and Lower slab] - [B]

Table 8.4.1-59 Bending moment (live load) [B]

Designed cross section	JSHB (kN・m)	FEM Analysis (P=100kN) (kN・m)	JSHB/FEM
*6 Upper end of web	104.742	30.379	3.448
*7 Bottom end of web	42.832	6.921	6.189
*10 Connected area of deck slab	39.806	5.205	7.648
*11 Middle of deck slab	1.483	0.215	6.898

Source : Study Team

Table 8.4.1-60 Designed Bending moment (D+LL+IM+Ps+Cr+SH) [B]

Designed point	JSHB (kN・m)	FEM Analysis (P=100kN) (kN・m)	JSHB/FEM
*6 Upper end of web	131.410	57.046	2.304
*7 Bottom end of web	54.656	19.765	2.765
*10 Connected area of lower slab	55.359	20.741	2.669
*11 Middle of lower slab	12.074	10.805	1.117

Source : Study Team

Arrangement of reinforcing bars at web and lower slab shall be as follows:

Table 8.4.1-61 Arrangement of reinforced bars

		Arrangement of reinforcing bars	Quantity (mm2/m)
Web	*6 Upper end	D16ctc125	1588.8
	*7 Lower end	D16ctc125	1588.8
Lower Slab	*10 Connected area	D19ctc125	2292.0
	*11 middle	D13ctc125	1013.6

Source : Study Team

Table 8.4.1-62 Stress of reinforcing bars

		JSHB		FEM Analysis		Restricted value
		σc (N/mm2)	σs (N/mm2)	σc (N/mm2)	σs (N/mm2)	
Dead Load	*6	0.8	35.9	0.8	35.9	σs < 100
	*7	0.4	17.3	0.4	17.3	σs < 100
	*10	2.3	42.0	2.3	42.0	σs < 100
	*11	2.0	61.7	2.0	61.7	σs < 100
Service Load	*6	4.2	176.8	1.8	76.7	σs < 180
	*7	1.7	73.5	0.6	26.6	σs < 180
	*10	8.1	149.4	3.0	56.0	σs < 180
	*11	2.3	70.4	2.0	63.0	σs < 180

Source : Study Team

Table 8.4.1-63 Safety factor under ultimate load working state [B]

Mu＝1.3*D+2.5*(LL+IM)+Ps+Cr+SH [JSHB-16]

Mu＝1.25*DC+1.5*DW+1.75*(LL+IM)+1.2or0.5*(Cr+SH) [P=100kN]

Designed point		JSHB (kN・m)	FEM Analysis（P=100kN）(kN・m)	Mr (kN・m)
Web	*6	231.571	95.305	< 276.952
	*7	114.165	35.199	< 276.952
Lower Slab	*10	106.848	29.340	< 141.089
	*11	19.773	13.230	< 64.666

Source : Study Team

c) Cross section of Intermediate support section [Upper slab] - [C]

Table 8.4.1-64 Bending moment (live load) [C]

	JSHB (kN・m)	FEM Analysis (P=100kN) (kN・m)	JSHB/FEM
*1 Connected part for cantilever slab	-150.150	-123.194	1.219
*2 Connected part in box	-148.601	-125.844	1.181
*3 Middle of box girder	92.116	38.113	2.417

Source : Study Team

Table 8.4.1-65 Designed bending moment (D+LL+IM+Ps+Cr+SH) [C]

Designed point	JSHB (kN・m)	FEM Analysis（P=100kN）(kN・m)	JSHB/FEM
*1 Connected part for cantilever slab	-223.806	-196.850	1.139
*2 Connected part in box	-248.012	-225.255	1.101
*3 Middle of box girder	52.117	-1.886	-

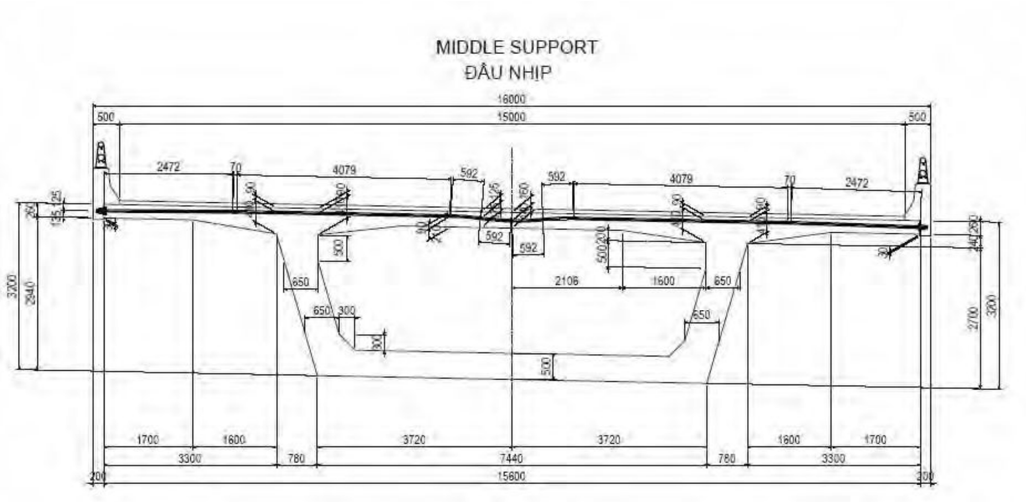
Source : Study Team

Table 8.4.1-66 Composite bending stress [C]

		JSHB		FEM Analysis		
		σco (N/mm2)	σcu (N/mm2)	σco (N/mm2)	σcu (N/mm2)	
Dead Load	*1	4.00	-0.05	4.01	-0.05	
	*2	3.46	0.54	3.46	0.54	
	*3	0.13	6.86	0.13	6.86	
Service Load	*1	0.40	3.55	1.05	2.91	
	*2	-0.10	4.11	0.44	3.56	
	*3	6.27	0.72	2.67	4.32	
			> -3.0	> 0.0	> 0.0	

Source : Study Team

Transversal PC tendon shall be 1S28.6mm and arranged with interval of less than ctc500mm as shown in following Figure.



Source : Study Team

Figure 8.4.1-60 Arrangement of Transversal PC Tendon (intermediate support cross section) ctc500mm

Table 8.4.1-67 Safety factor under ultimate load working state [C]

Mu＝1.3*D+2.5*(LL+IM)+Ps+Cr+SH [JSHB-16]

Mu＝1.25*DC+1.5*DW+1.75*(LL+IM)+1.2or0.5*(Cr+SH) [P=100kN]

Designed point	JSHB (kN・m)	FEM Analysis （P=100kN） (kN・m)	Mr (kN・m)
*1 Connected part for cantilever slab	453.253	316.775	< 677.837
*2 Connected part in box	411.865	344.058	< 677.837
*3 Middle of box girder	148.692	56.524	< 225.587

Source : Study Team

d) Cross section of Intermediate support section [Web and Lower slab] - [D]

Table 8.4.1-68 Bending moment (live load) [D]

Designed point	JSHB (kN・m)	FEM Analysis (P=100kN) (kN・m)	JSHB/FEM
*6 Upper end of web	100.135	36.884	2.715
*7 Lower end of web	100.501	25.012	4.018
*10 Connected part of lower deck slab	86.331	9.228	9.355
*11 Middle of lower deck slab	4.688	0.555	8.447

Source : Study Team

Table 8.4.1-69 Designed bending moment (D+LL+IM+Ps+Cr+SH) [D]

Designed point	JSHB (kN・m)	FEM Analysis (P=100kN) (kN・m)	JSHB/FEM
*6 Upper end of web	118.092	51.862	2.198
*7 Lower end of web	120.982	47.225	2.598
*10 Connected part of lower slab	104.206	28.000	3.753
*11 Middle of lower slab	36.762	31.769	1.129

Source : Study Team

Arrangement of reinforcing bars at web and lower slab shall be as follows:

Table 8.4.1-70 Arrangement of reinforced bars

		Arrangement of reinforcing bars	Quantity(mm2/m)
Web	Upper end	D16ctc125	1588.8
	Lower end	D16ctc125	1588.8
Lower Slab	Connected part	D16ctc125	1588.8
	Middle	D13ctc125	1013.6

Source : Study Team

Table 8.4.1-71 Stress of reinforcing bars

		JSHB		FEM Analysis		Restricted value
		σc (N/mm2)	σs (N/mm2)	σc (N/mm2)	σs (N/mm2)	
Dead Load	*6	0.4	18.4	0.3	14.5	σs < 100
	*7	0.4	19.8	0.4	21.5	σs < 100
	*10	0.7	27.6	0.7	28.9	σs < 100
	*11	1.5	76.1	1.5	74.1	σs < 100
Service Load	*6	2.3	114.2	1.0	50.1	σs < 180
	*7	2.3	116.9	0.9	45.7	σs < 180
	*10	4.1	160.6	1.1	43.1	σs < 180
	*11	1.7	87.2	1.5	75.4	σs < 180

Source : Study Team

Safety factor under ultimate load working state [D]

Designed point		JSHB (kN･m)	FEM Analysis (P=100kN) (kN･m)	Mr (kN･m)
Web	*6	225.558	81.515	< 382.529
	*7	257.642	84.856	< 382.529
Lower Slab	*10	251.088	47.071	< 242.548
	*11	54.564	40.713	< 155.848

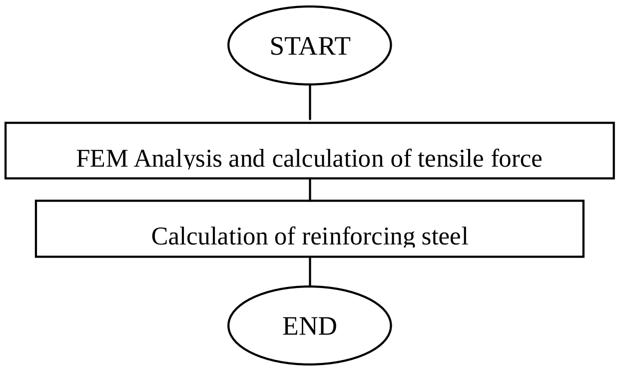
8.4.2 FEM analysis of Cross Beam

8.4.2.1 Outline of Design

(1) Design Principle

Because external cables are going through cross beam, it generates complicated local stress on the transverse direction and vertical direction. As for the local stress, it is needed to reinforce in a suitable manner so that all their structure can fulfill the function.

When this bridge is designed, the local stress is calculated by 3D FEM analysis that can simulate almost truly, accordingly the stress, the amount of reinforcing bar shall be calculated.



Source: Study Team

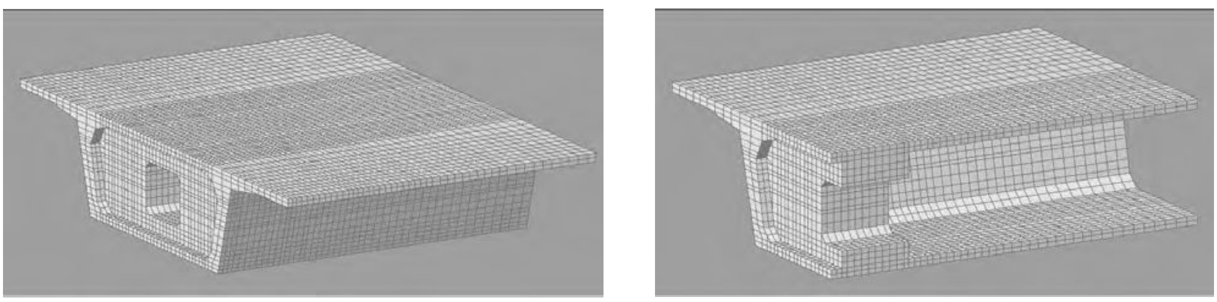
Figure 8.4.2-1 Flowchart of Designing Cross beam

8.4.2.2 Analysis model and condition for FEM

(1) End cross beam (A1)

1) Analysis model

In this analysis, the girder model is produced in the range that some impact of the tensile force could make small, and local stress is calculated by the adding tensile force.

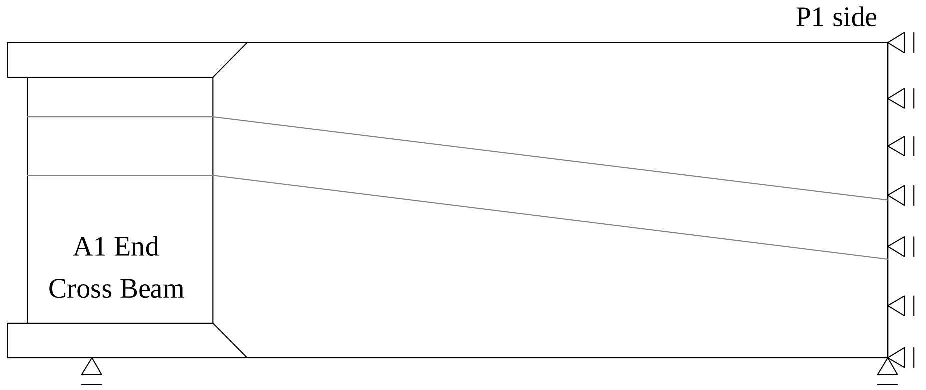


Source: Study Team

Figure 8.4.2-2 End cross beam A1 in FEM mesh

2) Restraint Condition

Because box girder between A1 and P1 has been completed before prestressing force is given, displacement of longitudinal direction is fixed on end of P1 side.



Source: Study Team

Figure 8.4.2-3 Restraint Condition of End cross beam A1

3) Loading Condition

The component force of external cable that enters the FEM is working on surface of cross beam with distribution load as same size as anchorage and nodal load in cross beam.

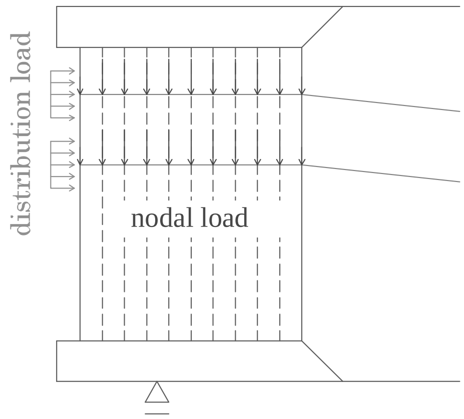


Figure 8.4.2-4 Loading Condition in FEM [A1]

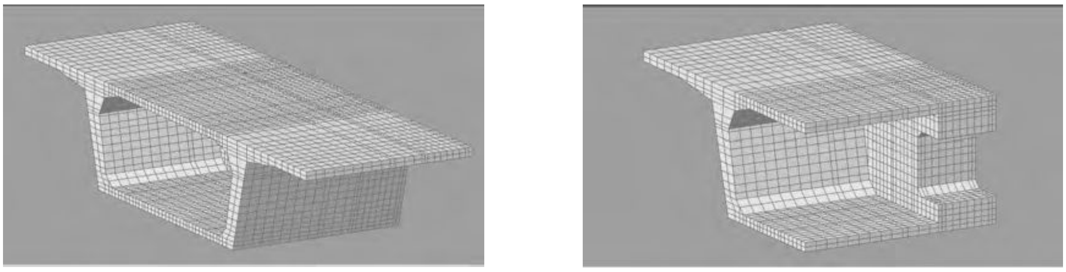
(2) Intermediate support cross beam (P1)

Cross beam (P1) has two construction steps to be completed building it. Therefore, 3D FEM analysis has to be run two times with different type of models.

1) Analysis model

a) Model -1

Model 1 is modeled on “Construction Step-1”. Construction Step-1 means that girder between A1 and P1 and P1 cross beam has been completed and external cables between A1 and P1 are given prestressed force.



Source: Study Team

Figure 8.4.2-5 Cross beam P1 [Model-1] in FEM mesh

[Restraint Condition]

Because girder of P2 side has not constructed yet, end of P2 side on FEM model is free on restraint condition.

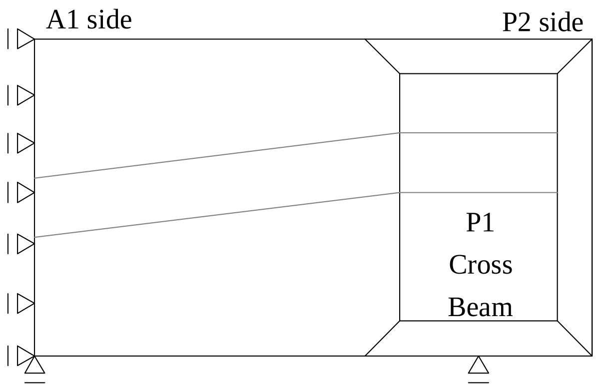


Figure 8.4.2-6 Restraint Condition of P1 [Model-1]

[Load Condition]

The component force of external cable that enters the FEM is working on surface of cross beam with distribution load as same size as anchorage and nodal load in cross beam.

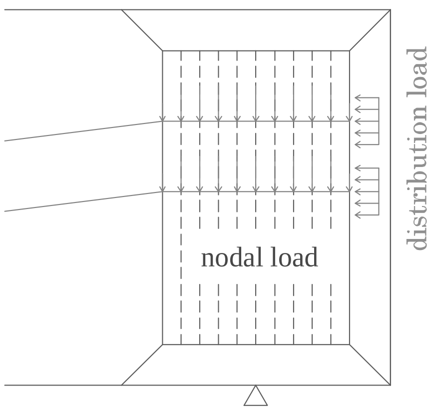
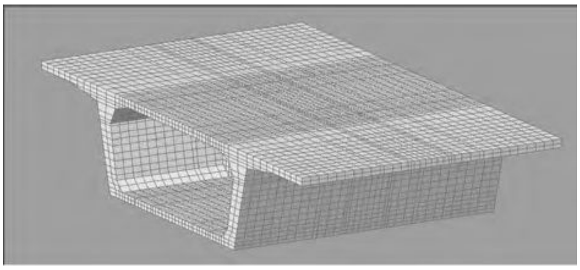
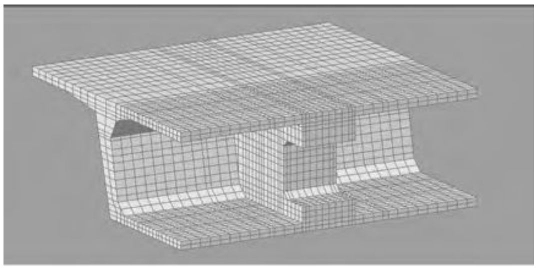


Figure 8.4.2-7 Load Condition in P1 [Model-1]

b) Model 2

Model 2 is modeled on "Construction Step-2". Construction Step-2 means that after "Step-1". Girder between P1 and P2 has been completed. Two types of external cable have added. One is external cable between P1 and P2, the other is continuous external cable, and all cables are given prestressed force in this condition.





Source: Study Team

Figure 8.4.2-8 Cross beam P1 [Model-2] in FEM mesh

[Restraint Condition]

Because girder between P1 and P2 has constructed, end of P2 side on FEM model is also fixed.

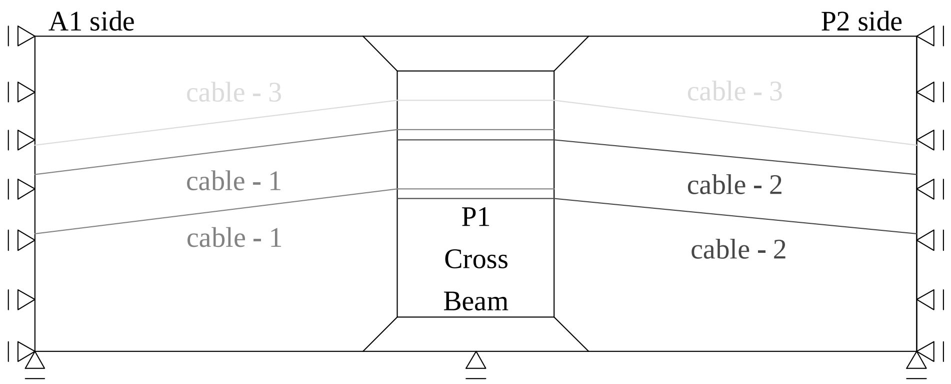


Figure 8.4.2-9 Restraint Condition of P1 [Model-2]

[Load Condition]

Cable-1 cable-2

The component force of external cable that enters the FEM is working on surface of cross beam with distribution load as same size as anchorage and nodal load in cross beam.

Cable-3

The component force of external cable that enters the FEM is working with nodal load in cross beam.

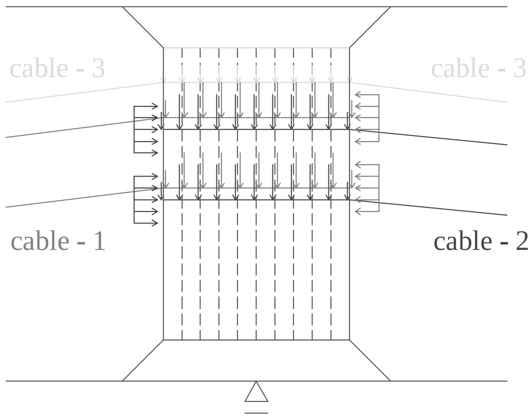


Figure 8.4.2-10 Load Condition in FEM [A1]

(3) Tensile Force of External Cable

As for the cross beam examination, it is need to study in Pre-stressing stage, Design stage and Ultimate load stage.

The relationship between each stage is the following.

Table 8.4.2-1 Stress of external cable and reinforcing bar

Stage	Stress of external cable σp (N/mm2)	Allowable stress of reinforcing bar σs (N/mm2)	Stress Ratio σp / σs
Pre-stressing	1440	180	8.0
Design load	1110	180	6.2
Ultimate load	1600	345	4.6

Source: Study Team

Above table shows that the pre-stressing stage is the highest risk condition and the condition would be need to study.

1) Calculation of Tensile Force

The way to calculate for the tensile force is shown as below.

At first, the stress in each nodal point is calculated based on FEM and the stress multiplied by the area, and then sum all the values on studied cross section.

8.4.2.3 Result of study

Following study has to examine about cross beams surface.

The tensile stress is caused on opposite surface to the PC anchor side. The stress distribution on an Opposite side is shown in the figure below.

(1) Reinforcing bar of A1 end cross beam

1) Calculation of vertical reinforcement bar

< viewpoint-1 Vertical reinforcing bar >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
σzz	8.99	235	140	32900	295771
	6.95	250	140	35000	243250
	6.37	175	140	24500	156065
	6.37	175	140	24500	156065
	--	835	--	--	851151

σa = 180 N/mm2
∴ Areq = T / σa = 4729 mm2

Steel bar : D22
interval = 125 mm layer = 2 number of bar = 13.4

∴ ΣA = 5079 mm2 > Areq ok

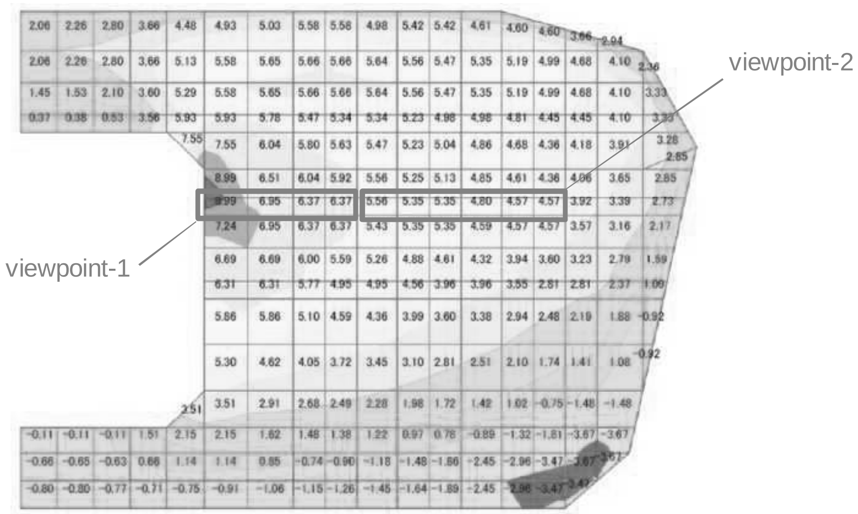
< viewpoint-2 Vertical reinforcing bar >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
σzz	5.56	220	140	30800	171248
	5.35	175	140	24500	131075
	5.35	175	140	24500	131075
	4.80	220	140	30800	147840
	4.57	175	140	24500	111965
	4.57	175	140	24500	111965
	--	1140	--	--	805168

σa = 180 N/mm2
∴ Areq = T / σa = 4473 mm2

Steel bar : D22
interval = 125 mm layer = 2 number of bar = 18.2

∴ ΣA = 6934 mm2 > Areq ok



2) Calculation of horizontal reinforcement bar

< viewpoint-3 Horizontal reinforcing bar >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
σyy	3.93	175	140	24500	96285
	4.13	175	140	24500	101185
	4.13	95	140	13300	54929
	4.06	200	140	28000	113680
	4.03	160	140	22400	90272
	3.95	95	140	13300	52535
	3.79	175	140	24500	92855
	3.67	175	140	24500	89915
	--	1250	--	--	691656

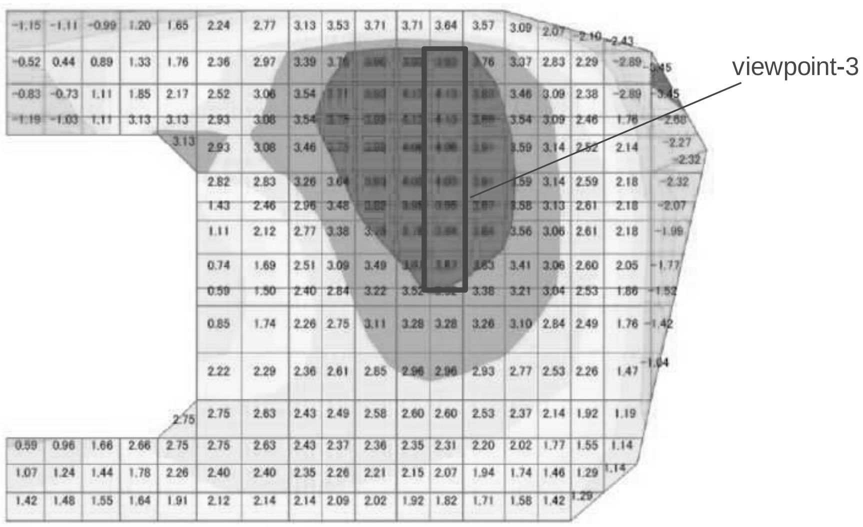
σa = 180 N/mm2

∴ Areq = T / σa = 3843 mm2

Steel bar : D22

interval = 200 mm layer = 2 number of bar = 12.5

∴ ΣA = 4752 mm2 > Areq ok



3) Reinforce Arrangement on surface of end cross beam

The reinforcement bar arrangement by FEM analysis is as follows.

<Vertical reinforcement bar>

D22 reinforcement bar is arranged on the opposite surface of cross beam in two rows at intervals of 125 mm.

<Horizontal reinforcement bar>

D22 reinforcement bar is arranged on the opposite surface of cross beam in two rows at intervals of 200 mm.

(2) Reinforcing bar of P1 cross beam

1) Calculation of vertical reinforcement bar

a) A1 side

< A1-SIDE Vertical reinforcing bar viewpoint-1>

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
σzz	7.91	165	100	16500	130515
	6.60	123	100	12300	81180
	--	288	--	--	211695

σa = 180 N/mm2
∴ Areq = T / σa = 1176 mm2

Steel bar : D22
interval = 150 mm layer = 2 number of bar = 3.8

∴ ΣA = 1460 mm2 > Areq ok

< A1-SIDE Vertical reinforcing bar viewpoint-2 >

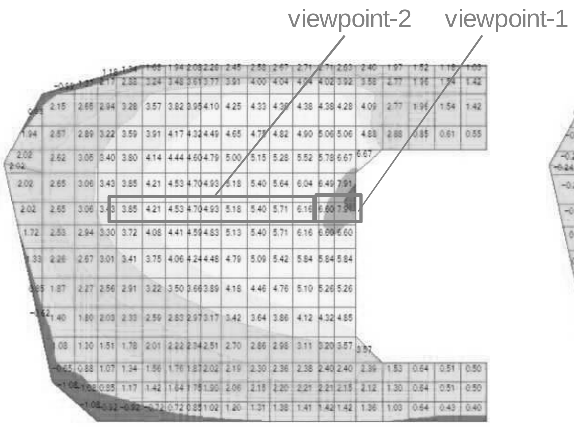
	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
σzz	6.16	197	100	19700	121352
	5.71	175	100	17500	99925
	5.40	175	100	17500	94500
	5.18	195	100	19500	101010
	4.93	155	100	15500	76415
	4.70	100	100	10000	47000
	4.53	175	100	17500	79275
	4.21	175	100	17500	73675
	3.85	195	100	19500	75075
	--	1542	--	--	768227

σa = 180 N/mm2
∴ Areq = T / σa = 4268 mm2

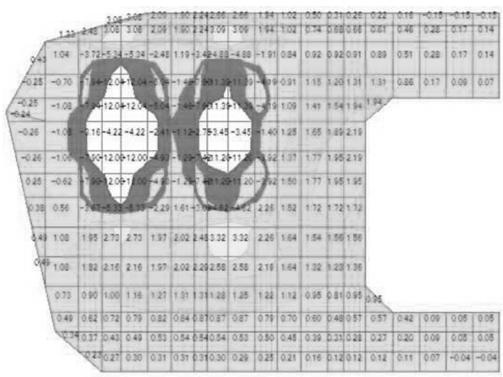
Steel bar : D22
interval = 200 mm layer = 2 number of bar = 15.4

∴ ΣA = 5862 mm2 > Areq ok

[A1 side of P3 cross beam]



STEP1 prestressing one side



STEP2 prestressing both sides

b) P2 side

< P2-SIDE Vertical reinforcing bar viewpoint-1 >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
	2.31	175	200	35000	80850
	2.31	175	200	35000	80850
	1.87	195	200	39000	72930
σzz	1.49	155	200	31000	46190
	1.84	100	200	20000	36800
	2.65	175	200	35000	92750
	2.65	175	200	35000	92750
	--	1150	--	--	503120

σa = 180 N/mm2

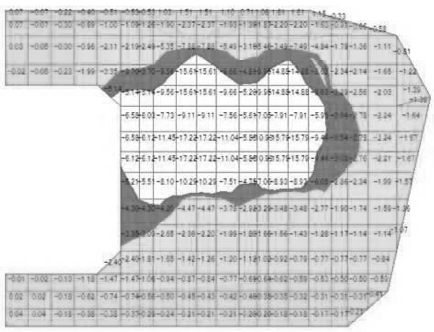
∴ Areq = T / σa = 2795 mm2

Steel bar : D22

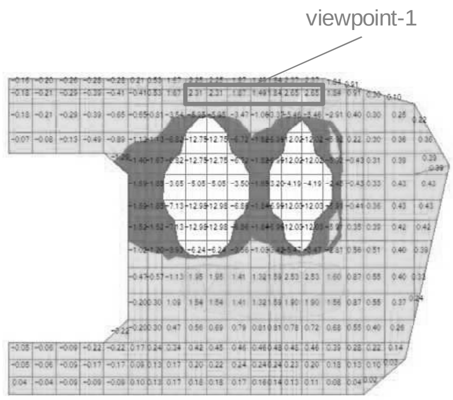
interval = 200 mm layer = 2 number of bar = 11.5

∴ ΣA = 4372 mm2 > Areq ok

[P2 side of P3 cross beam]



STEP1 prestressing one side



STEP2 prestressing both sides

2) Calculation of horizontal reinforcement bar

a) A1 side

< A1-SIDE Horizontal reinforcing bar viewpoint-3 >

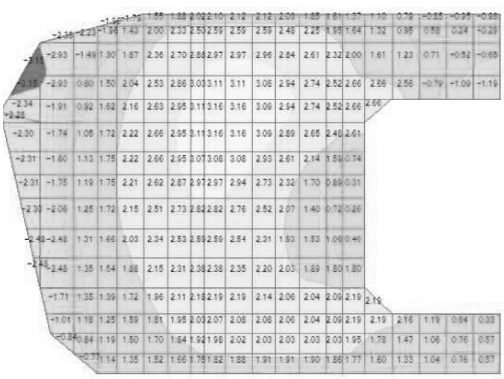
	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
σyy	3.41	175	100	17500	59675
	3.41	175	100	17500	59675
	2.84	220	100	22000	62480
	3.04	175	100	17500	53200
	3.04	175	100	17500	53200
	--	920	--	--	288230

σa = 180 N/mm2
∴ Areq = T / σa = 1601 mm2

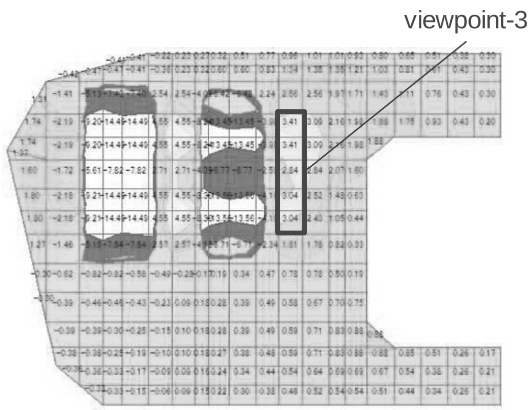
Steel bar : D22
interval = 250 mm layer = 2 number of bar = 7.4

∴ ΣA = 2798 mm2 > Areq ok

[A1 side of P3 cross beam]



STEP1 prestressing one side



STEP2 prestressing both sides

b) P2 side

< P2-SIDE Horizontal reinforcing bar viewpoint-2 >

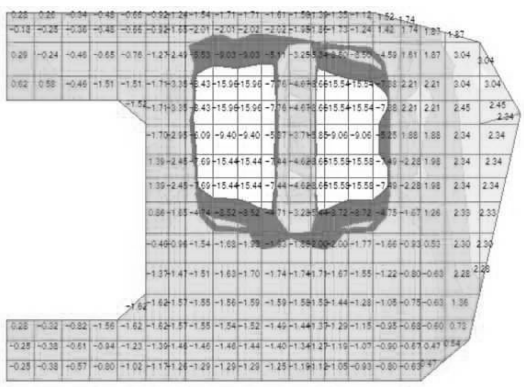
	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
σyy	3.67	175	100	17500	64225
	3.67	175	100	17500	64225
	3.49	95	100	9500	33155
	3.68	200	100	20000	73600
	3.68	200	100	20000	73600
	--	845	--	--	308805

σa = 180 N/mm2
∴ Areq = T / σa = 1716 mm2

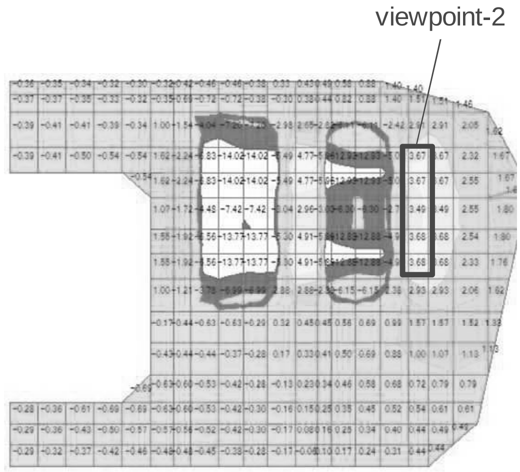
Steel bar : D22
interval = 250 mm layer = 2 number of bar = 6.8

∴ ΣA = 2570 mm2 > Areq ok

[P2 side of P3 cross beam]



STEP1 prestressing one side



STEP2 prestressing both sides

(3) Reinforce Arrangement on surface of intermediate support cross beam (P2, P3, P4)

The reinforcement bar arrangement by FEM analysis is as follows.

1) Vertical reinforcement bar

D22 reinforcement bar is arranged on the opposite surface of cross beam in two rows at intervals of 200 mm.

However, it is arranged at intervals of 150mm in manhole neighborhood.

2) Horizontal reinforcement bar

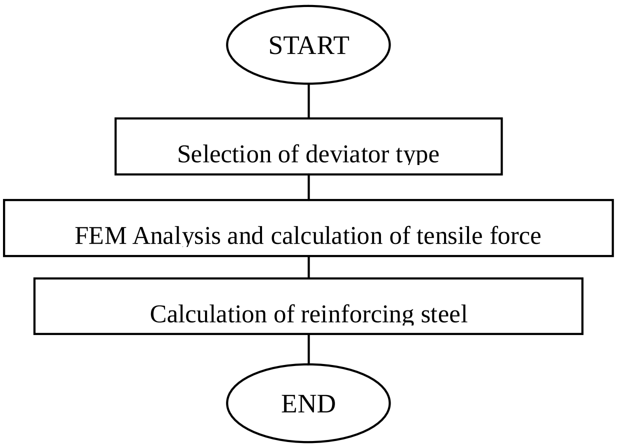
D22 reinforcement bar is arranged on the opposite surface of cross beam in two rows at intervals of 250 mm.

8.4.3 Design of Deviator

8.4.3.1 Outline of Design

Though deviator is the specific device for external cable, it generates complicated local stress on the longitudinal direction, transverse direction and vertical direction. As for the local stress, it is needed to reinforce in a suitable manner so that all their structure can fulfill the function.

When this bridge is designed, the local stress is calculated by 3D FEM analysis that can simulate almost truly, accordingly the stress, the amount of reinforcing bar shall be calculated.

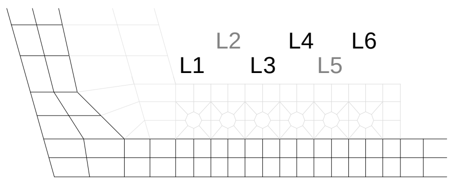


Source: Study Team

Figure 8.4.3-1 Flowchart of Designing Deviator

8.4.3.2 Selection of Deviator Type

Because the type of the deviator shall be a projection type that is easy to use in construction, basically one external cable is arranged per one deviation on one side (L1/L3/L4/L6). But in this model, there is a type deviating 2 external cables at L2 and L5 at the same time.



Source: Study Team

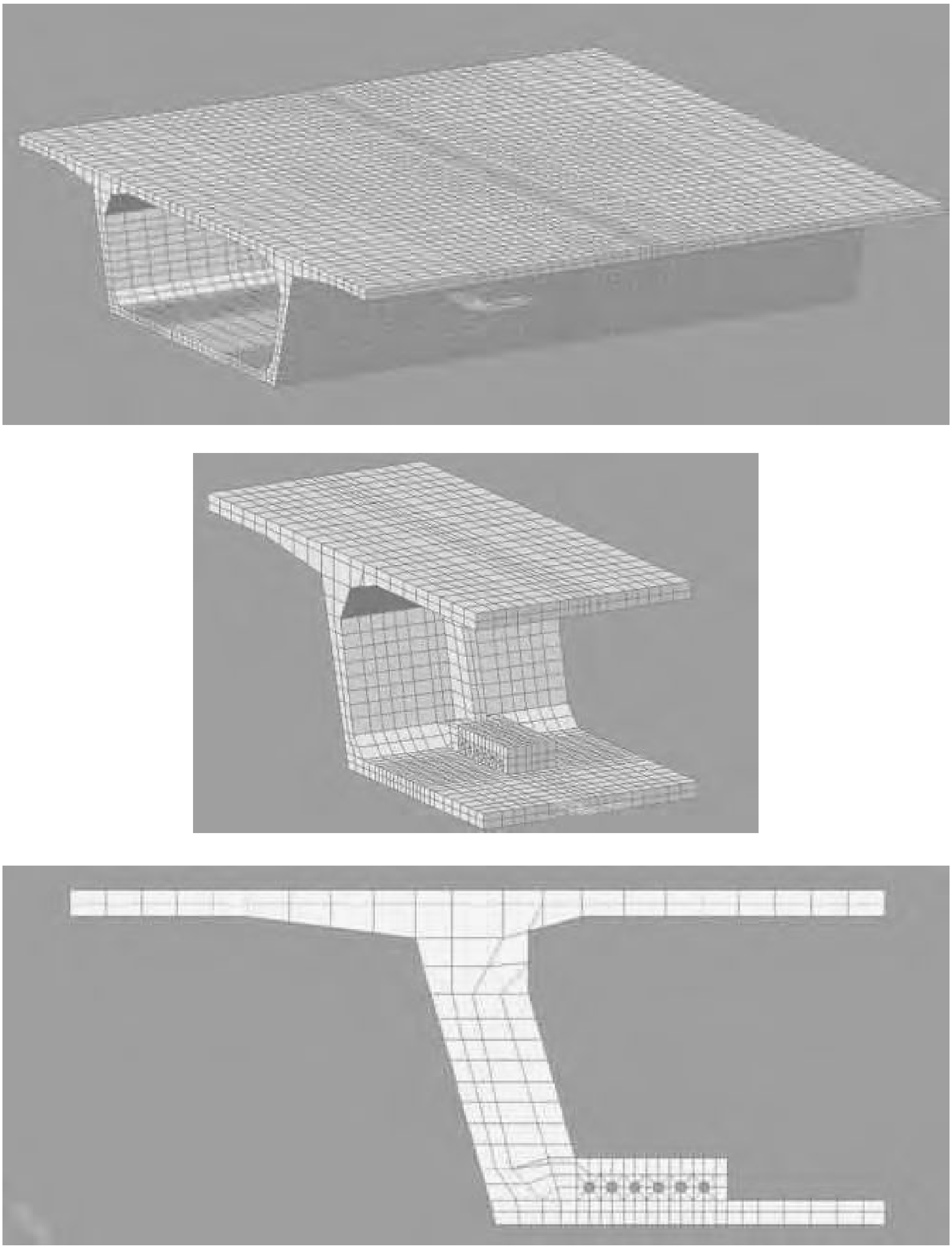
Figure 8.4.3-2 Number of deviators

8.4.3.3 Calculation of Tensile Force by FEM

(1) Analysis model

In this analysis, the girder model is produced in the range that some impact of the tensile force could make small, and local stress is calculated by the adding tensile force.

Analysis model is shown as next page.



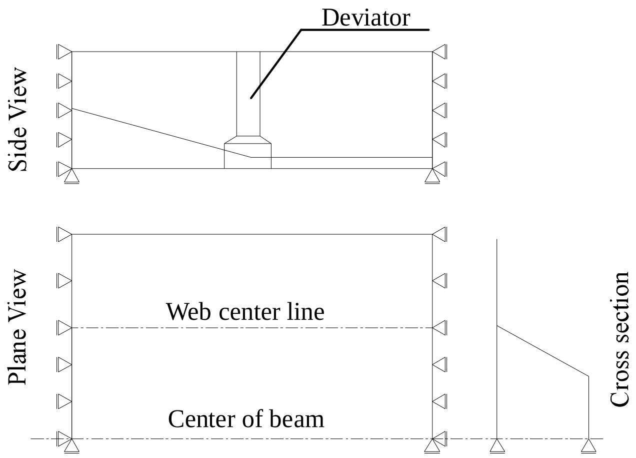
Source: Study Team

Figure 8.4.3-3 Deviator Examination in FEM mesh

8.4.3.4 Restraint Condition

*Stress on longitudinal and transverse direction

According to the analysis model above mentioned, both edges of displacements are restrained.



Source: Study Team

Figure 8.4.3-4 Analysis model for calculation of Stress on longitudinal and transverse direction

8.4.3.5 Tensile Force of External Cable

As for the deviator examination, it is need to study in Pre-stressing stage, Design stage and Ultimate load stage.

(1)Ultimate load

The relationship between each stage is the following.

Table 8.4.3-1 Stress of external cable and reinforcing bar

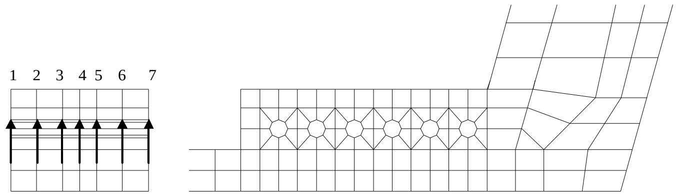
Stage	Stress of external cable σp (N/mm2)	Allowable stress of reinforcing bar σs (N/mm2)	Stress Ratio σp / σs
Pre-stressing	1440	180	8.0
Design load	1110	180	6.2
Ultimate load	1600	345	4.6

Source:

Above table shows that the pre-stressing stage is the highest risk condition and the condition would be need to study.

8.4.3.6 Load Condition

The component force of external cable that enters the FEM is divided horizontal direction and vertical direction, and then loaded as the nodal points as shown below.



Source: Study Team

Figure 8.4.3-5 The component force of external cable in FEM analysis

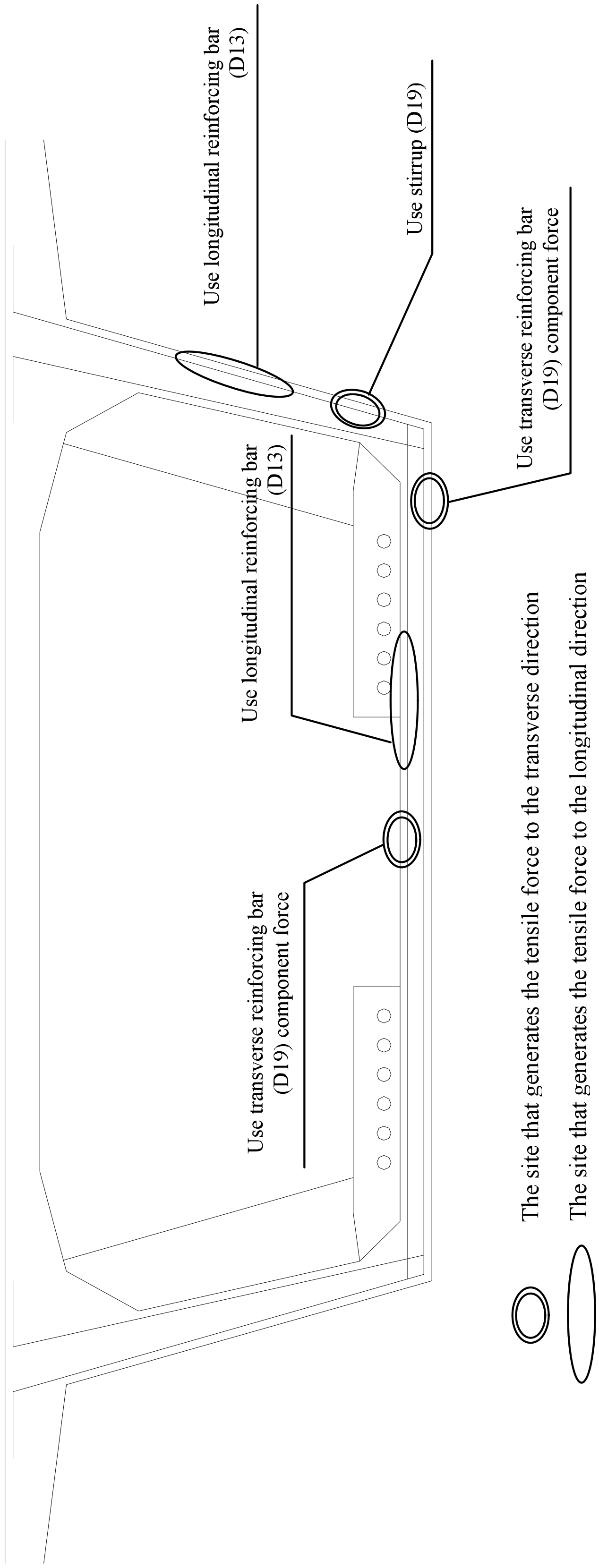
8.4.3.7 Calculation of Tensile Force

The way to calculate for the tensile force is shown as below.

At first, the stress in each nodal point is calculated based on FEM and the stress multiplied by the area, and then sum all the values on studied cross section.

(1) Reinforcing Method

The result of FEM analysis and reinforcing method is shown to next page.



Source : Study Team

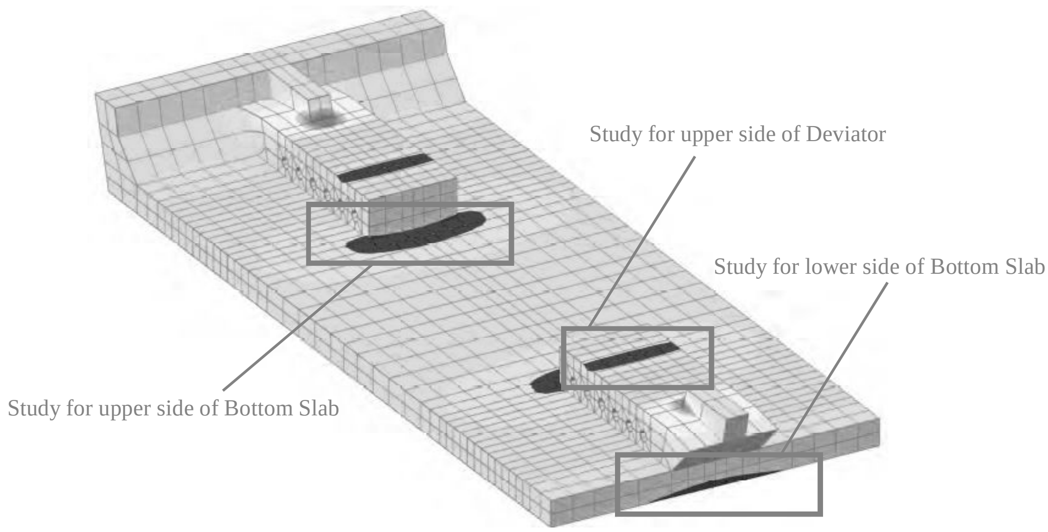
Figure 8.4.3-6 The study result and the amount of reinforcing bar of deviator component force

8.4.3.8 Result of study

As stated above, one external cable is arranged per a deviation on one side. But, at some deviators, two external cables are arranged per a deviator on one side. In the case, stronger tensile force is added to the deviator than others arranged one external cable.

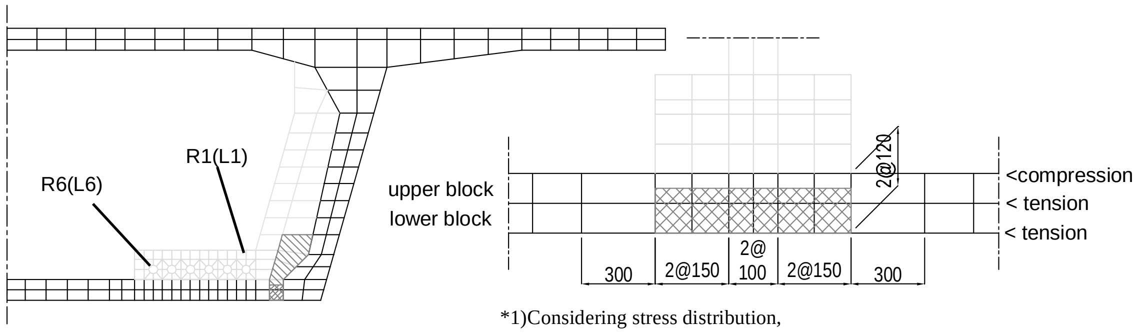
Therefore, following study has to examine about deviator that has one or two external cables deviated on one side. Deviators with one external cable are deviator R1 (L1), R6 (L6) and R4 (L4). .

Cases :
- Type1: Two external cables are deviated at L2 and L5 (R2 and R5) at the same time
- Type2: One external cable is deviated at R1 (L1), R6 (L6) and R4 (L4).



(1) Reinforcing bar of bottom slab around deviator

1) Lower side of bottom slab

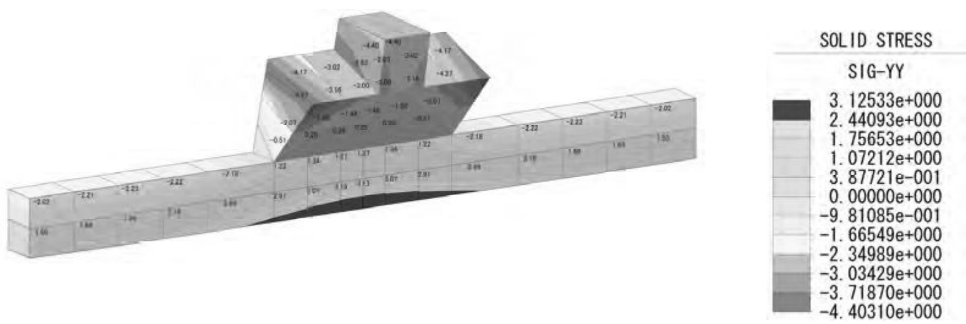


*1)Considering stress distribution,
area tensile stress is generated in is effective in calculation.
2)Because there have been reinforcing bars near upper side of bottom slab, upper half of upper blocks are excluded from area of 1)

Deviator R6 ~ R1(L6~L1)

Source: Study Team

Figure 8.4.3-7 FEM mesh for examination of bottom slab



[Type1]

Source: Study Team

Figure 8.4.3-8 Result of FEM analysis in Lower side of bottom slab

Results of examination about Reinforcing bar is shown following,

a) Type1: Two external cables are deviated at L2 and L5 (R2 and R5) at the same time

< Type1 : Two external cables are daviated >

case2	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
Deviator R2+R5 (L2+L5)	2.92	150	120	18000	52560
	3.07	150	120	18000	55260
	3.13	100	120	12000	37560
	3.13	100	120	12000	37560
	3.07	150	120	18000	55260
	2.92	150	120	18000	52560
	1.22	150	60	9000	10980
	1.34	150	60	9000	12060
	1.37	100	60	6000	8220
	1.37	100	60	6000	8220
	1.34	150	60	9000	12060
	1.22	150	60	9000	10980
	--	800	--	--	353280

σa = 180 N/mm2
∴ Areq = T / σa = 1963 mm2

Steel bar : D22 interval = 125 mm number = 6.4
∴ ΣA = 2433 mm2 > Areq ok

b) Type2: One external cable is deviated at R1 (L1), R6 (L6) and R4 (L4).

< Type2 : One external cable is daviated >

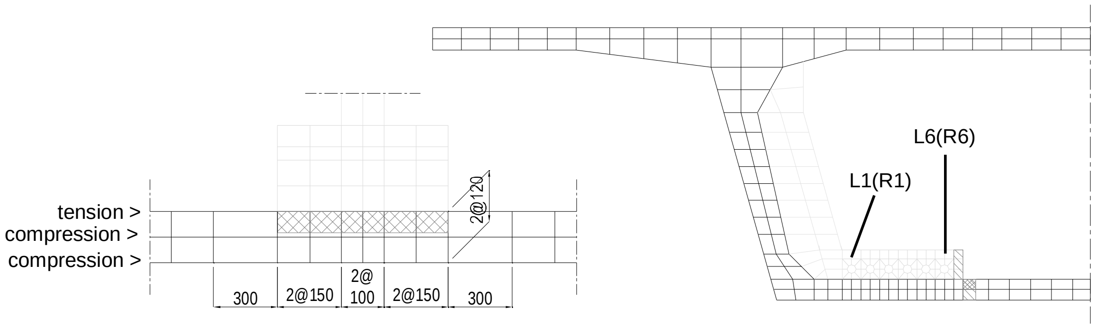
	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
Deviator R1 (L1)	0.00	150	120	18000	0
	0.00	150	120	18000	0
	0.00	100	120	12000	0
	0.00	100	120	12000	0
	0.00	150	120	18000	0
	0.00	150	120	18000	0
	0.00	150	60	9000	0
	0.00	150	60	9000	0
	0.00	100	60	6000	0
	0.00	100	60	6000	0
	0.00	150	60	9000	0
	0.00	150	60	9000	0
	--	800	--	--	0
Deviator R6 (L6)	2.53	150	120	18000	45540
	2.65	150	120	18000	47700
	2.68	100	120	12000	32160
	2.68	100	120	12000	32160
	2.65	150	120	18000	47700
	2.53	150	120	18000	45540
	1.20	150	60	9000	10800
	1.31	150	60	9000	11790
	1.33	100	60	6000	7980
	1.33	100	60	6000	7980
	1.31	150	60	9000	11790
	1.20	150	60	9000	10800
	.	800	--	--	311940

Tmax = 311940 N σa = 180 N/mm2
∴ Areq = Tmax / σa = 1733 mm2

Steel bar : D19 interval = 125 mm number = 6.4
∴ ΣA = 1815 mm2 > Areq ok

* In the results of Type2 (External cable is going through deviator R1 (L1)), tensile force doesn't generate. Therefore R1 (L1) is excluded from study of reinforcing steel bars.

2) Upper side of bottom slab

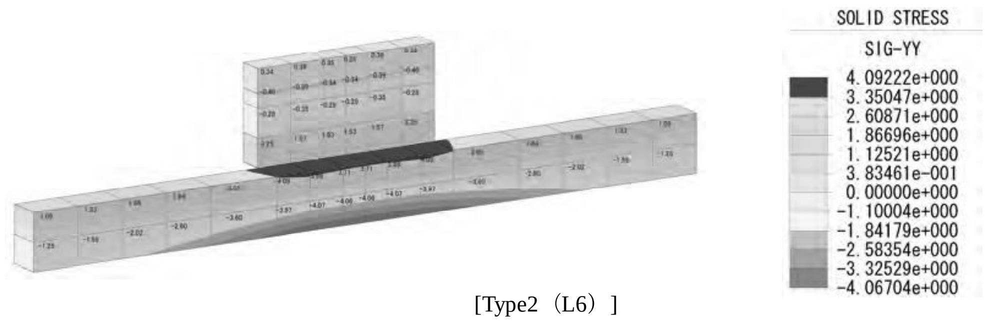


*Considering stress distribution,
area that tensile stress is generated in is effective in calculation.

Deviator L1 ~ L6(R1~R6)

Source: Study Team

Figure 8.4.3-9 FEM mesh for examination of bottom slab



[Type2（L6）]

Source: Study Team

Figure 8.4.3-10 Result of FEM analysis in Upper side of bottom slab

Results of examination about Reinforcing bar is shown following tables,

a) Type1: Two external cables are deviated at L2 and L5 (R2 and R5) at the same time

< Type1 : Two external cables are daviated >

case2	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
Deviator L2+L5 (R2+R5)	3.97	150	100	15000	59550
	3.81	150	100	15000	57150
	3.52	100	100	10000	35200
	3.52	100	100	10000	35200
	3.81	150	100	15000	57150
	3.97	150	100	15000	59550
	--	800	--	--	303800

σa = 180 N/mm2
∴ Areq = T / σa = 1688 mm2

Steel bar : D19 interval = 125 mm number = 6.4
∴ ΣA = 1815 mm2 > Areq ok

b) Type2: One external cable is deviated at R1 (L1), R6 (L6) and R4 (L4).

< Type2 : One external cable is daviated >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
Deviator L1 (R1)	1.17	150	100	15000	17550
	1.11	150	100	15000	16650
	1.04	100	100	10000	10400
	1.04	100	100	10000	10400
	1.11	150	100	15000	16650
	1.17	150	100	15000	17550
	--	800	--	--	89200
Deviator L6 (R6)	4.09	150	100	15000	61350
	3.98	150	100	15000	59700
	3.71	100	100	10000	37100
	3.71	100	100	10000	37100
	3.98	150	100	15000	59700
	4.09	150	100	15000	61350
	--	800	--	--	316300

Tmax = 316300 N σa = 180 N/mm2
∴ Areq = Tmax / σa = 1757 mm2

Steel bar : D22 interval = 125 mm number = 6.4
∴ ΣA = 2433 mm2 > Areq ok

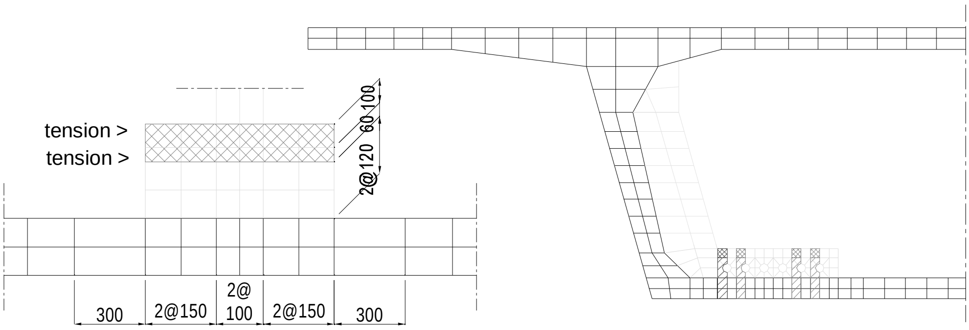
< Type2 : One external cable is daviated >

case2	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
Deviator R4 (L4)	2.62	150	100	15000	39300
	2.52	150	100	15000	37800
	2.35	100	100	10000	23500
	2.35	100	100	10000	23500
	2.52	150	100	15000	37800
	2.62	150	100	15000	39300
	--	800	--	--	201200

σa = 180 N/mm2
∴ Areq = T / σa = 1118 mm2

Steel bar : D16 interval = 125 mm number = 6.4
∴ ΣA = 1287 mm2 > Areq ok

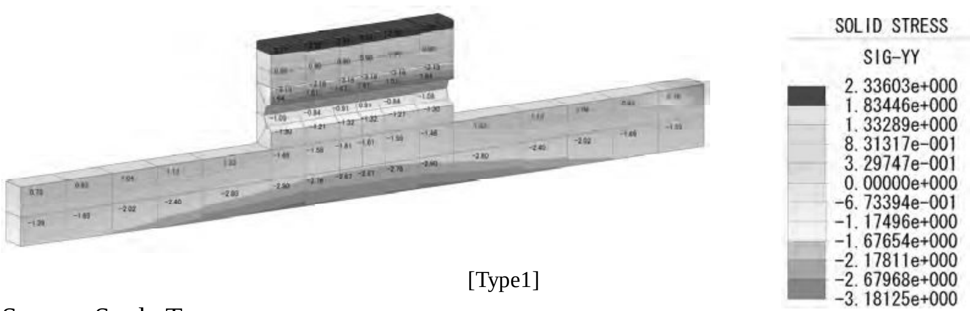
3) Upper side of Deviator



*Considering stress distribution,
area that tensile stress is generated is effective in calculation.

Source: Study Team

Figure 8.4.3-11 FEM mesh for examination of deviator



Source: Study Team

Figure 8.4.3-12 Result of FEM analysis in Upper side of deviator

a) Type1: Two external cables are deviated at L2 and L5 (R2 and R5) at the same time

< Type1 : Two external cables are daviated >

case2	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
Deviator L2+L5 (R2+R5)	2.27	150	100	15000	34050
	2.32	150	100	15000	34800
	2.34	100	100	10000	23400
	2.34	100	100	10000	23400
	2.32	150	100	15000	34800
	2.27	150	100	15000	34050
	0.90	150	60	9000	8100
	0.90	150	60	9000	8100
	0.90	100	60	6000	5400
	0.90	100	60	6000	5400
	0.90	150	60	9000	8100
	0.90	150	60	9000	8100
	--	800	--	--	227700

σa = 180 N/mm2
∴ Areq = T / σa = 1265 mm2

Steel bar : D16 number = 7.0
∴ ΣA = 1407 mm2 > Areq ok

b) Type2: One external cable is deviated at R1 (L1), R6 (L6) and R4 (L4).

< Type2 : One external cable is daviated >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
Deviator L1 (R1)	2.39	150	100	15000	35850
	2.40	150	100	15000	36000
	2.44	100	100	10000	24400
	2.44	100	100	10000	24400
	2.40	150	100	15000	36000
	2.39	150	100	15000	35850
	1.13	150	60	9000	10170
	0.96	150	60	9000	8640
	0.88	100	60	6000	5280
	0.88	100	60	6000	5280
	0.96	150	60	9000	8640
	1.13	150	60	9000	10170
	--	800	--	--	240680
Deviator L6 (R6)	1.08	150	100	15000	16200
	1.08	150	100	15000	16200
	1.10	100	100	10000	11000
	1.10	100	100	10000	11000
	1.08	150	100	15000	16200
	1.08	150	100	15000	16200
	0.38	150	60	9000	3420
	0.35	150	60	9000	3150
	0.33	100	60	6000	1980
	0.33	100	60	6000	1980
	0.35	150	60	9000	3150
	0.38	150	60	9000	3420
	--	800	--	--	103900

Tmax = 240680 N σa = 180 N/mm2

∴ Areq = Tmax / σa = 1337 mm2

Steel bar : D16 number = 7.0

∴ ΣA = 1407 mm2 > Areq ok

(2) Reinforcement arrangement around Deviator

1) Lower side reinforcement bar of bottom slab

a) When two external tendons are deviated, D22 is arranged as lower side reinforcement bar at intervals of 125mm.

b) When one external tendon is deviated, D19 is arranged as lower side reinforcement bar at intervals of 125mm.

2) Upper side reinforcement bar of bottom slab

a) When two external tendons are deviated, D19 is arranged as upper side reinforcement bar at intervals of 125mm.

b) When one external tendon is deviated at the position of L6, D22 is arranged as upper side reinforcement bar at intervals of 125mm.

c) D19 is arranged as upper side reinforcement bar at intervals of 125mm in other cases.

3) Upper side reinforcement bar of deviator

a) When two external tendons are deviated, 7-D19 is arranged in the upper side of deviator.

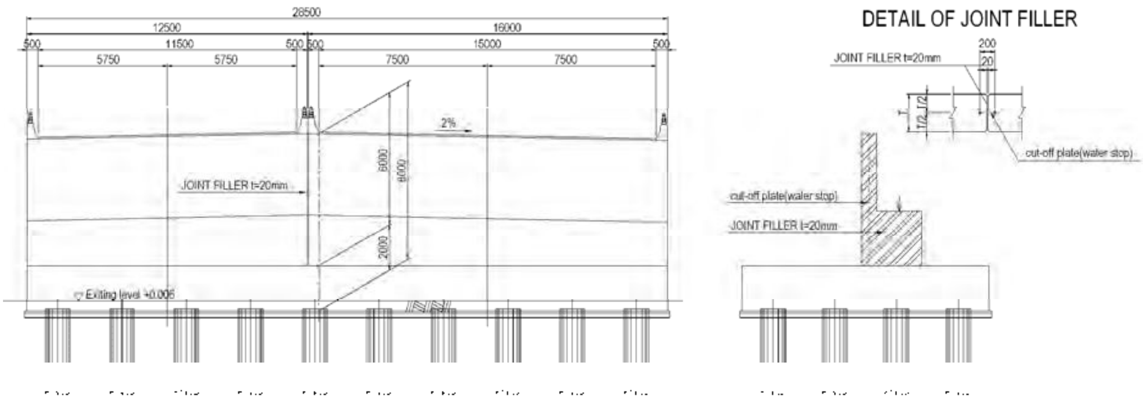
b) When one external tendon is deviated, 7-D16 is arranged in the upper side of deviator.

8.4.3.9 Substructure of Approach Bridge

8.4.3.10 Abutment

(1) Abutment with large width

Abutment with large width shall be designed in consideration of temperature changes, vertical cracks due to drying shrinkage and vertical loads, and uneven settlement in the lateral direction. For body width exceeding about 15m, it is good to install vertical joints having a V-groove in the body surface or the expansion joint. In this project adopt the expansion joint.



Source : Study Team

Figure 8.4.3-13 Joint Filler

(2) Rear side of abutment

The backfill behind an abutment may be deformed not only by settlement due to consolidation or the like but also by settlement accompanying abutment vibration or liquefaction during earthquakes. Therefore, it is desirable to install an approach cushion in order to ensure smooth road traffic after the earthquake in addition consideration for not imparting shocks to running vehicles or the abutment.

The length of approach cushion slab adopt 8.0m based on JSHB.

Table 8.4.3-2 Length of approach cushion slab

ground condition / back filling material / Height of abutment	orrdinary ground		soft ground
	unscreened gravel hard rock	except for left column material	All kind of material
H < 6.0m	-	5.0m	8.0m
6.0m ≦ H < 15.0m	5.0m	5.0m	8.0m
15.0m < H	8.0m	5.0m	8.0m

Source : Study Team

8.4.3.11 Pier

(1) Study on Shape of Pier

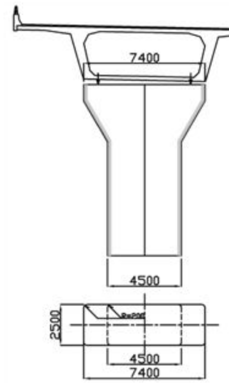
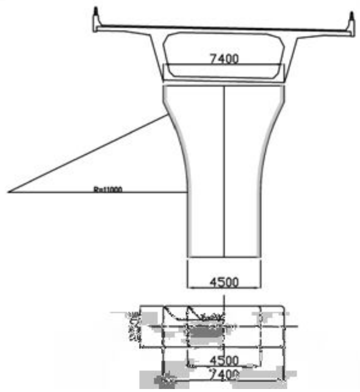
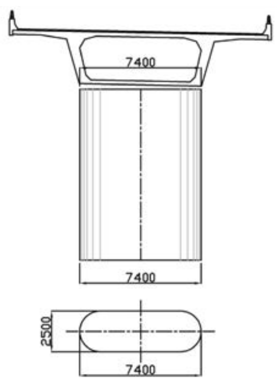
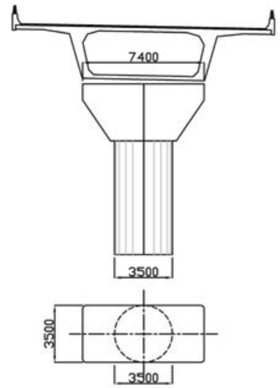
The study was implemented by comparing the following alternatives:

- ●(1) Alternative-1:Rectangle shape column(SAPROF)
- ●(2) Alternative-2:Rectangle shape column smoothing angle between column and Pier head.
- ●(3) Alternative-3:Oval shape column
- ●(4) Alternative-4:Round shape column

8.4.3.12 Results of Comparative Study

As shown in the Table 13.5.2-4, Alternative-2 Rectangle shape column smoothing angle between column and Pier head is recommended because of its advantage in aesthetics and construction cost.

Table 8.4.2-2 Comparative Study on Pier Sharpe of Approach Bridge

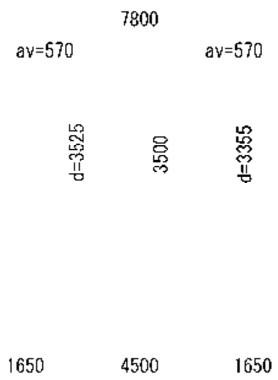
Evaluation Items		Alternative-1 (SAPROF) Rectangle shape column		Alternative-2 Rectangle shape column smoothing angle between column and Pier head		Alternative-3 Oval shape column		Alternative-4 Round shape column	
Side View									
Structural Aspect and Stability	10	-Bearing weight for the foundation will reduce because of deduction of its own weight.	8	-Bearing weight for the foundation will reduce because of deduction of its own weight.	8	-Bearing weight for the foundation will increase because of adding its own weight. -No necessity to establish the timbering.	4	-The arrangement of bar at the connection between column,beam and pile cap will be comlicated. So this shape is inferior in constructbility.	6
Construction Cost (for Foundation)	40	Quantity / Unit Cost (VND) / Total (1,000VND) Concrete 107m3 / 5,867,864 / 627,861 Total 627,861 Ratio **1.000**	40	Quantity / Unit Cost (VND) / Total (1,000VND) Concrete 108m3 / 5,867,864 / 633,729 Total 633,729 Ratio **1.009**	40	Quantity / Unit Cost (VND) / Total (1,000VND) Concrete 137m3 / 5,867,864 / 805,658 Total 805,658 Ratio **1.283**	24	Quantity / Unit Cost (VND) / Total (1,000VND) Concrete 119m3 / 5,867,864 / 698,863 Total 698,863 Ratio **1.113**	32
Construction Plan and Period	10	column and beam 11 days Total **11 days**	8	column and beam 11 days Total **11 days**	8	column and beam 14 0 days Total **14 days**	8	column and beam 13 days Total **13 days**	6
Maintenance	15	-No necessity to maintenance.	9	-No necessity to maintenance.	9	-No necessity to maintenance.	9	-No necessity to maintenance.	9
STEP Clearance	10	- 32% (Preliminary Estimate)	10	- 32% (Preliminary Estimate)	10	- 32% (Preliminary Estimate)	10	- 32% (Preliminary Estimate)	10
Aesthetics	5	-Sharp corner makes inorganic and hard impressions.	2	-Smoothing the angle between pier head and column makes soft impression .	5	-Wide width of column makes a sense of oppression.	2	-Round shape of column makes soft impression.	3
New Technology	5	-Nothing new technology is used.	3	-Nothing new technology is used.	3	-Nothing new technology is used.	3	-Nothing new technology is used.	3
Environmental Aspect	5	-No major environmental impact.	3	-No major environmental impact.	3	-No major environmental impact.	3	-No major environmental impact.	3
Evaluation	100	-This alternative is superior in economic ,but inferior in aesthetic points compared to alternative.2. Less Recommended	**83**	-This alternative is superior to other cases in aesthetic points. -Construction Cost is the second lowest in the alternatives, but almost same as alternative-1. Most Recommended	**86**	-This alternative is most inferior in economic and in aesthetic points. Not Recommended	**63**	-This alternative is most inferior in constructbility. **Not Recommended**	**72**

Source : Study Team

(1) Design of Beam

a) Analysis

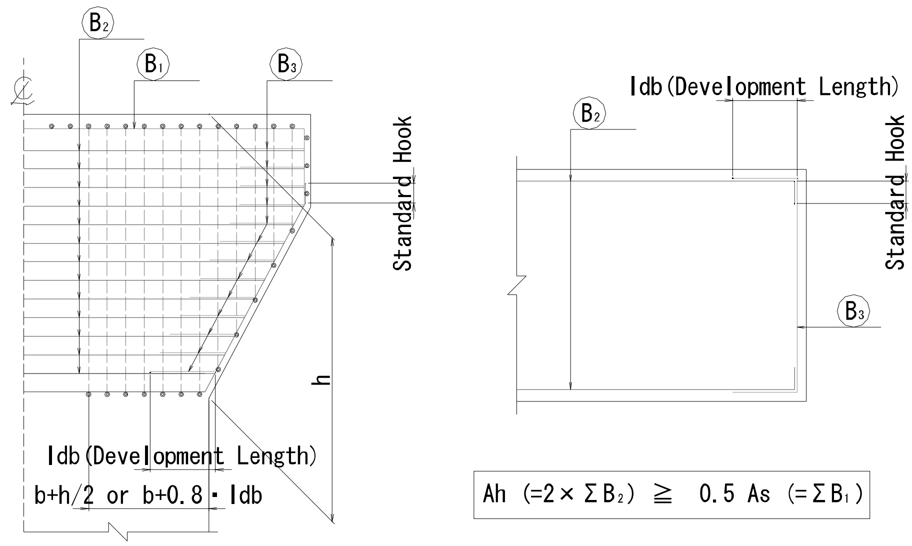
Components in which “av”, as shown in following Figure, is less than “d” shall be considered to be corbels. Strut-and-tie models to analysis the corbels.



Source : Study Team

Figure 8.4.3-14 Sectional view of Beam

b) Arrangement of reinforcement



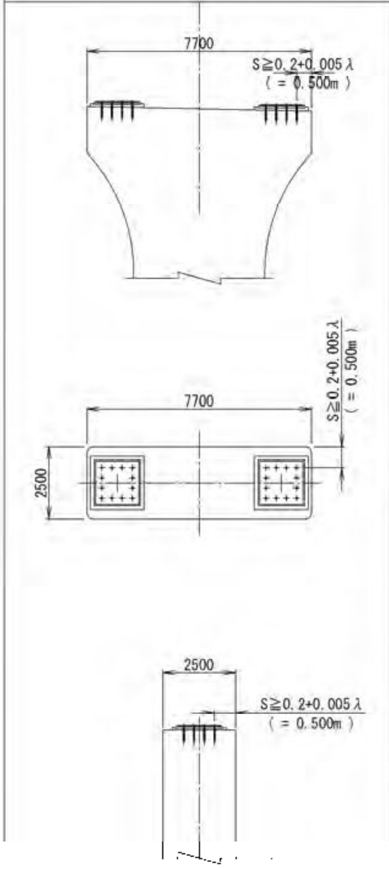
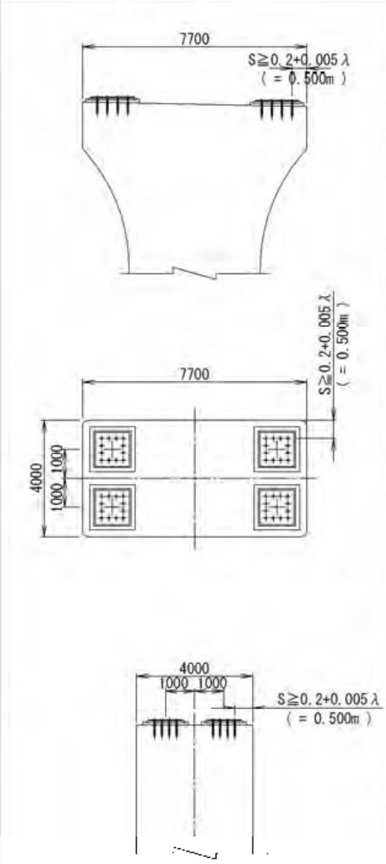
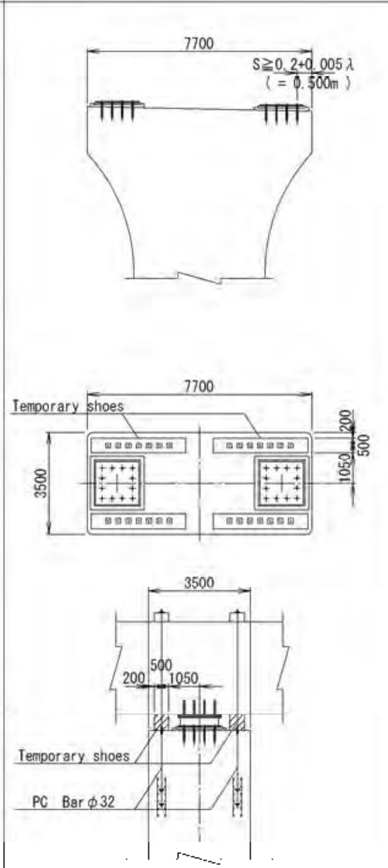
Source : Study Team

Figure 8.4.3-15 Arrangement of reinforcement at beam

(2) Width of bridge seats

The width of bridge seats is decided by bearings edge distance based on Japanese Standard of Highway Bridge or the arrangement of temporary shoes which is used to erect the superstructure by cantilever construction.

Table 8.4.3-4 Width of bridge seats

Erection method of superstructure Span by span Erection Location of Pier Intermediate Pier	Erection method of superstructure Span by span Erection Cantilever Construction Location of Pier End Pier	Erection method of superstructure Cantilever Construction Location of Pier Intermediate Pier
		

Source : Study Team

Table 8.4.3-5 Refer from JSHB

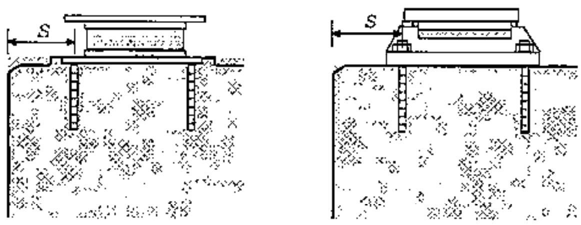
2) The distance, S(m), between the edge of bearings and the edge of the top of the substructure (or bearing support edge distance) shall be equal to or larger than the following value:

$$S = 0.2 + 0.005\ \lambda \qquad (8.6.1)$$

where,

S : bearings edge distance (m)

λ : span length (m)



(a) Rubber Bearing (b) Steel Bearing

Fig. C.8.6.4 Bearing Support Edge Distance S

Source : Study Team

8.4.4 Study on Foundation

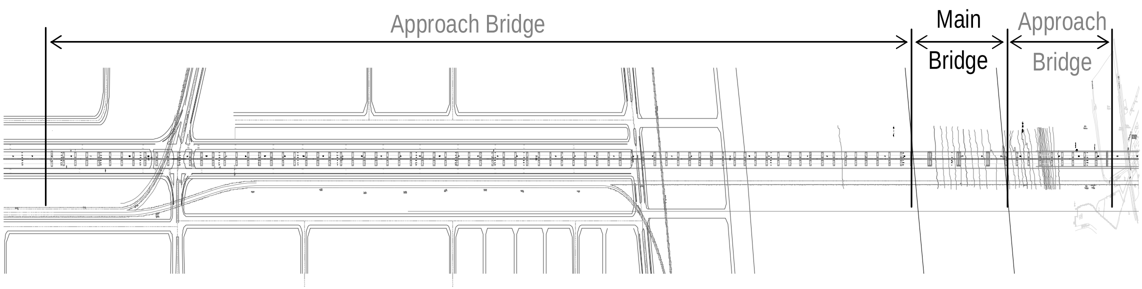
8.4.4.1 General

(1) Objectives of Condition

In the SAPROF Study, steel pipe pile foundation was selected for approach bridge and steel pipe sheet pile for main bridge substructures in terms of its rapid construction speed comparing with Cast in place pile foundation. This study aimed to carry out verifies the foundation type of SAPROF Study by comprehensive evaluation in terms of structural stability, construction cost, construction planning, and aesthetic point of view including re-evaluation of construction conditions for foundation of approach bridge and main bridge.

(2) Scope of Study

This study consists of two (2) sub-studies; the study on steel pipe sheet pile foundation, and the study on selection of bridge foundation. In the study on steel pipe sheet pile foundation, design principal and design elevation of pile cap is the key discussion. In the study on selection of bridge foundation, study of site conditions for selecting an appropriate foundation type is a key discussion.



Source: Study Team

Figure 8.4.4-1 Plan Layout for Approach Bridge and Main Bridge

8.4.4.2 Condition of Study

(1) Soil Condition

Refer to Chapter 3 “Subsoil Conditions”.

(2) Structure Height and Seawater Depth

Following Table is summary for the pier height and seawater depth with classification of study type for approach bridge and main bridge.

Table 8.4.4-1 Pier Height and Seawater Depth

Type	Pier No.	Pier Height* (m)	Column Height (m)	Water depth (m)
Type-1	P1	6.0	3.5	2.54
	P2	7.5	5.0	2.67
	P3	8.5	6.0	2.65
	P4	8.5	6.0	2.70
	P5	8.5	6.0	2.58
	P6	8.5	6.0	2.60
	P7	9.0	6.5	2.66
	P8	9.5	7.0	2.69
	P9	10.0	7.5	2.71
	P10	10.0	7.5	2.81
	P11	10.5	8.0	2.92
	P12	10.5	8.0	3.18
	P13	10.5	8.0	3.28
	P14	10.5	8.0	3.25
	P15	10.0	7.5	3.15
	P16	10.0	7.5	3.16
	P17	10.0	7.5	3.19
	P18	10.0	7.5	3.25
	P19	9.5	7.0	3.27
	P20	9.5	7.0	3.27
	P21	9.5	7.0	3.29
	P22	9.0	6.5	3.32
	P23	9.0	6.5	3.37
	P24	9.0	6.5	3.39
	P25	8.5	6.0	3.46
	P26	8.5	6.0	3.50
	P27	8.5	6.0	3.46
	P28	8.5	6.0	3.48
	P29	8.5	6.0	3.44
	P30	8.5	6.0	3.61
	P31	8.5	6.0	3.64
	P32	8.5	6.0	3.71
	P33	8.5	6.0	3.78
	P34	8.5	6.0	3.79
	P35	8.5	6.0	3.77
	P36	9.0	6.5	3.82
	P37	9.0	6.5	3.71
	P38	9.0	6.5	3.65
	P39	9.5	7.0	3.65
	P40	9.5	7.0	3.64
	P41	9.5	7.0	3.62
	P42	9.5	7.0	3.58
	P43	10.0	7.5	3.51
	P44	10.0	7.5	3.50
Type-1	*P45*	10.0	7.5	3.48
	P46	10.5	8.0	3.50
	P47	10.5	8.0	3.51
	P48	10.5	8.0	3.42
	P49	10.5	8.0	3.31
	P50	15.0	12.5	7.51
	P51	15.0	12.5	7.51
	P52	15.5	13.0	7.51
	P53	15.5	13.0	7.51
	P54	15.0	12.5	7.51
	P55	15.0	12.5	7.51
	P56	15.0	12.5	7.51
	P57	15.0	12.5	7.51
	P58	14.5	12.0	7.51
	P59	14.5	12.0	2.55
	P60	14.0	11.5	2.55
Type-2	P61	14.0	11.5	7.51
	P62	14.0	11.5	7.51
	P63	14.0	11.5	7.51
	P64	13.5	11.0	7.51
	P65	13.5	11.0	7.51
	P66	13.5	11.0	7.51
	P67	13.0	10.5	7.51
	P68	13.0	10.5	7.51
	P69	13.0	10.5	7.51
	P70	14.0	11.5	7.51
	P71	15.0	12.5	7.51
	P72	16.5	14.0	7.51
	P73	18.5	16.0	7.51
	P74	20.0	17.5	7.51
	P75	21.5	**19.0**	7.51
Type-3	P76	23.5	21.0	6.94
	P77	27.0	24.5	8.67
	P78	28.0	25.5	10.80
Type-4	*P79*	20.0	**17.5**	**11.53**
	P80	19.0	**16.5**	**11.13**
	P81	17.0	**14.5**	**9.98**
	P82	17.0	14.5	**7.87**
Type-1	P83	13.5	11.0	3.75
	P84	10.5	8.0	2.42
	P85	8.5	6.0	2.11
	P86	7.5	5.0	1.84
	P87	6.0	3.5	1.46

*pier Height : Column + Pile Cap Height

Source: Study Team

8.4.4.3 Principle of Study

(1) Classification of Foundation based on Study Conditions

As the first step, the foundations to be studied are classified into four (4) main types as shown in below table based on study conditions. In the study of Type-1, the downdrag effect for piles due to consolidation of clay layer needs to be examined in detail. In the study of Type-2, there is less number of piers; the point of this study is that the construction costs are needed to be focused on the detail. In the study of Type-3, there is planning to steel pipe sheet pile (SPSP) foundation of critical for construction period at deep water. The point to be focused in detail for this type is construction period and safe construction at the deep water. In the study of Type-4, there is planning closed the navigation channel with Cat Hai Island at deep water. The point of this study is the structural aesthetics which shall be harmonized with scenery of Cat Hai Island and safe construction at the deep water.

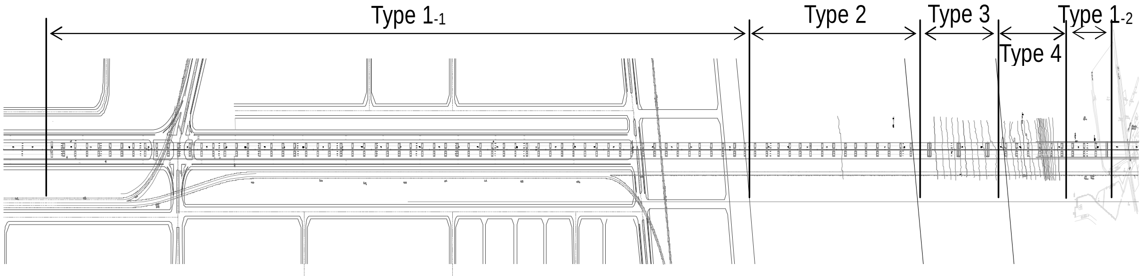
Table 8.4.4-2Foundation Study Type for Approach Bridge and Main Bridge

Study Type	Type-1$_{-1}$	Tupe-2	Type-3	Type-4	Type-1$_{-2}$
Bridge Type	Approach	Approach	Main Br.	Approach	Approach
Station	Km +561.3 ~8+77.12	Km +561.3 ~8+77.12	Km +561.3 ~8+77.12	Km +561.3 ~8+77.12	Km +561.3 ~8+77.12
Pier No.	A1 ~ P60	P61 ~ P75	P76 ~ P78	P70 ~ P82	P84 ~ P87
Reclamation Plan	in operation	No	No	No	planning for the future
Bridge Span length (m)	60.0	60.0	150.0	60.0	60.0
Estimated Corrosion Thickness of Steel Pile (mm)	2[*1]	7[*2]	7[*2]	7[*2]	2[*1]
Water depth (m)	2.5~3.8	3.2~6.5	7.1~11.0	8.3~11.5	1.63~3.8
E.L. of Pile Cap	Variation 2	Variation 2 or 3	E.L.-5.0 (Top of Pile Cap)	Variation 4	Variation 1

Note, *1; protected due to reclamation by filling up

*2; according to report of Refer No PMU2/110422-1

Source: Study Team

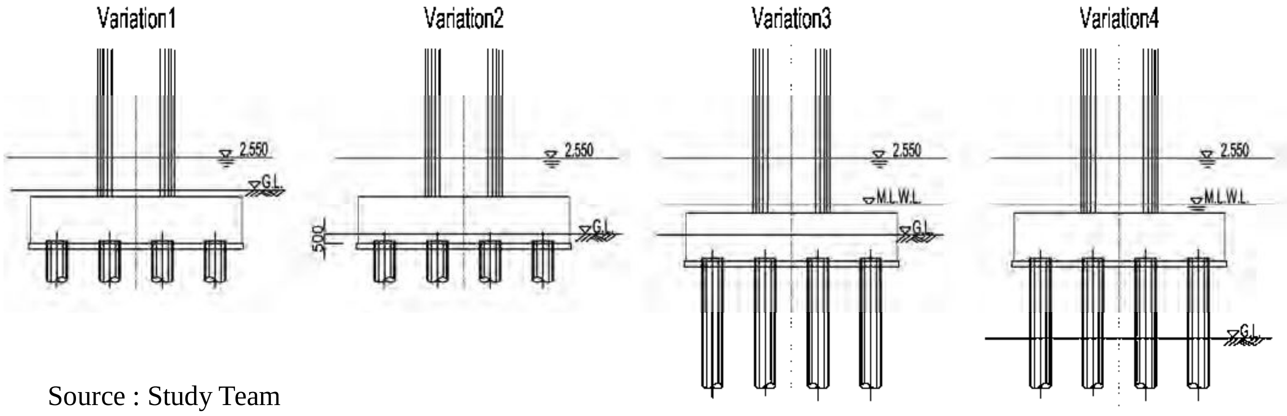


Source : Study Team

Figure 8.4.4-2 Plan Layout for Approach Bridge and Main Bridge

(2) Classification on Elevation of Pile Cap

After due consideration of site conditions, pile caps for approach bride are classified into four (4) alternatives (refer to below figures). In the Variation 1, top of pile cap is seated below the seabed when the sea is shallow and pier height is low. Variation 2, bottom of pile cap is seated on the sea bed of deepwater. The construction of pile cap at below the sea bed is difficult in terms of structure of cofferdam. Furthermore, the pile cap is backfilled by reclamation in the future. In the Variation 3, top of pile cap shall be set below Mean Low Water Level (EL-1.670) in order to comply with the regulation which prohibits exposure of pile cap body above Mean Low Water Level. In the Variation 4, top of pile cap is seated below the Mean Low Water Level (EL-1.670) as same as Variation 3. In addition to the condition, bottom of pile cap does not contact onto seabed due the deepwater; construction of pile cap at seabed is difficult in terms of structure of cofferdam.



Source : Study Team

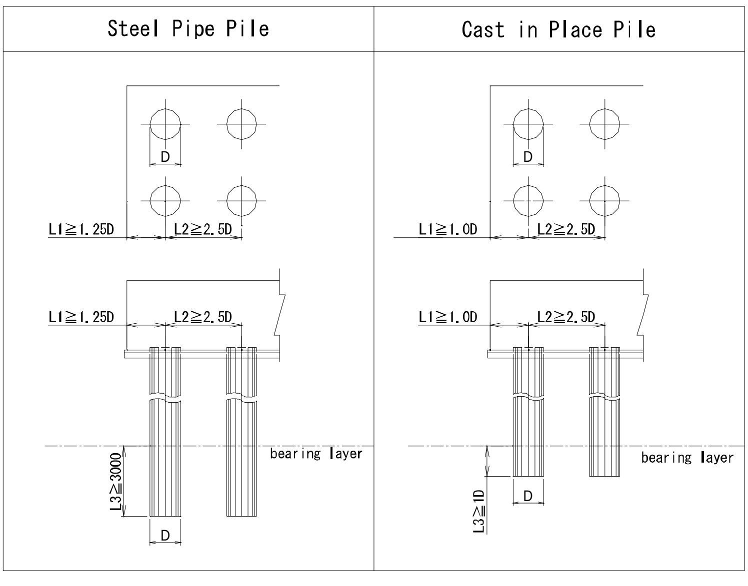
Figure 8.4.4-3 Variations for Pile Cap Elevation

(3) Pile arrangement and type of pile connection

1) Pile arrangement of Steel pipe pile and cast in place pile

Pile arrangement of steel pipe pile and cast in place based on following table.

Table 8.4.4-3Pile arrangement

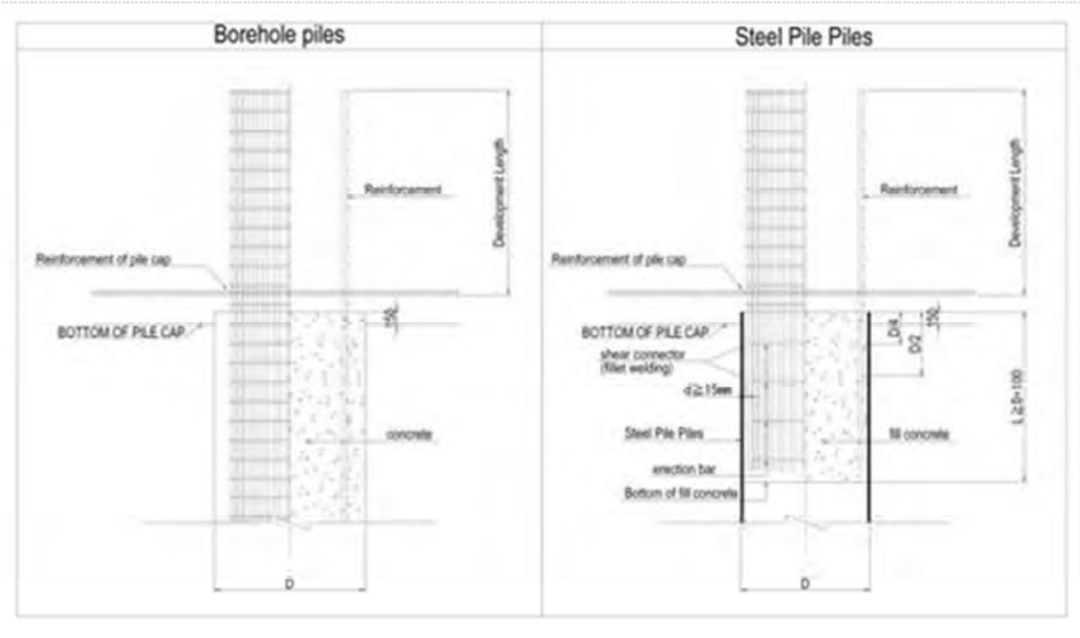


Source : Study Team

2) Connections between pile and pile cap

Connections between pile and pile cap shall be designed as rigid connections, and shall be verified against all forces acting at the pile head including push-in forces, pull-out forces, lateral forces, and bending moments. When reinforcing bars are used to reinforce pile heads, the stresses in the concrete and reinforcing bars in the footing is reviewed by assuming a virtual RC pile section in the pile cap.

Table 8.4.4-4Connections between pile and pile cap



Source : Study Team

3) Pile diameter for comparison

The results of review of SAPROF study by B/D design condition are shown in Table.13.5.3-7. In this comparative study, the following alternatives are studied.

Steel pipe pile(Pier)

- ●SAPROF study : D=0.8m, 4x4-2=14nos
- ●Altanative-1 : D=0.8m, 5x5 =25nos
- ●Altanative-2 : D=1.1m, 4x4 =16nos
- ●Altanative-3 : D=1.4m, 3X4 =12nos

Steel pipe pile(Abutment)

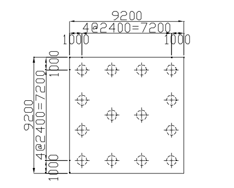
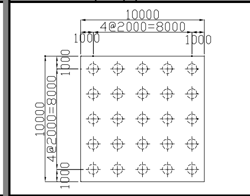
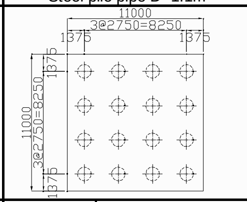
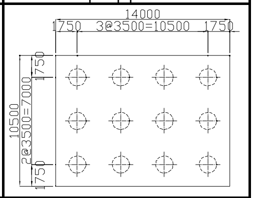
- ●Altanative-1 : D=0.8m, 4x11 =44nos
- ●Altanative-2 : D=1.1m, 4x10 =40nos
- ●Altanative-3 : D=1.4m, 4X 9 =36nos

Cast in place pile;

- ●Altanative-1 : D=1.2m, 3x4 =12nos
- ●Altanative-2 : D=1.5m, 3x3 = 9nos
- ●Altanative-3 : D=2.0m, 2X3 = 6nos

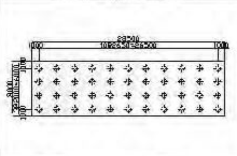
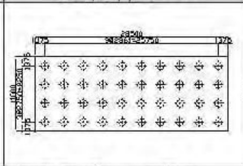
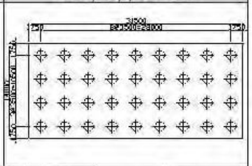
As theses table indicates, the changing condition design of SAPROF study, D=0.8m for 14nos steel pipe pile is not enough bearing capacity, Altenative-2, D=1.1m for 16nos steel pipe pile is the most recommendable for approach bridge because of its advantages in lowest construction cost. Therefore, the B/D study is applied D=1.1m for 16nos steel pipe pile as modified “SAPROF study”.

Table 8.4.4-5 Comparison for pile diameter of steel pipe pile at Pier

Alternative				SAPROF study		Altrenative-1		Altrenative-2		Altrenative-3	
Pile Type				Steel pile pipe D=0.8m		Steel pile pipe D=0.8m		Steel pile pipe D=1.1m		Steel pile pipe D=1.4m	
Plan of Pile Cap											
				L=46.0m	n=14nos	L=46.0m	n=25nos	L=46.0m	n=16nos	L=46.0m	n=12nos
Displacement		mm		δx=1.8≦δa=15 (OK)		δx=1.0≦δa=15 (OK)		δx=1.7≦δa=15 (OK)		δx=1.4≦δa=15 (OK)	
Pile Reaction		kN		P_{nmax}=3120 > Ra=2050 (NG)		P_{nmax}=1850 < Ra=2050		P_{nmax}=3000 < Ra=3160		P_{nmax}=4370 < Ra=4530	
Pile body	thickness	mm		t=11mm(SKK400)		t=11mm(SKK400)		t=11mm(SKK400)		t=11mm(SKK400)	
	Mu	kN.m		483.2		280.1		575.2		762.2	
	Mn	kN.m		1193.8		1556.9		2512.5		4024.8	
	fs	-		2.47		5.56		4.37		5.28	
	Extreme ft	N/mm^2		86.7 < 140.0		50.9 < 140.0		72.1 < 140.0		76.1 < 140.0	
	Extreme ft	N/mm^2		101.0 < 140.0		59.0 < 140.0		80.0 < 140.0		83.0 < 140.0	
Cost Estimate											
Item		unit	Unit cost (VND)	Material Quantities	Cost (VND)	Material Quantities	Cost (VND)	Material Quantities	Cost (VND)	Material Quantities	Cost (VND)
Pile cap											
Concrete 28MPa		m^3	5,867,864			250.0	1,466,966,000	302.5	1,775,028,860	367.5	2,156,440,020
Lean Concrete		m^3	1,723,811			10.0	17,238,110	12.0	20,685,732	15.0	25,857,165
Blinding stone		m^3	696,000			20.0	13,920,000	24.0	16,704,000	29.0	20,184,000
Excavation		m^3	318,066			113.0	35,941,458	128.0	40,712,448	147.0	46,755,702
Cofferdam		ton	24,798,638			87.0	2,157,481,506	93.0	2,306,273,334	101.0	2,504,662,438
Sub total							3,691,547,074		4,159,404,374		4,753,899,325
Foundation											
Steel Pile (Diameter)	0.8m	m	0				15,063,501,667				
	1.1m	m	0						12,923,273,000		
	1.4m	m	0								14,980,518,000
Sub total							15,063,501,667		12,923,273,000		14,980,518,000
Total							18,755,048,741		17,082,677,374		19,734,417,325
ratio						1.098		1.000		1.155	
Evaliation								Most Recommended			

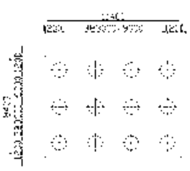
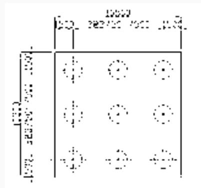
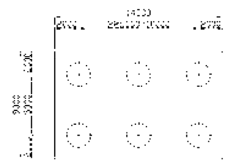
Source: Study Team

Table 8.4.4-6 Comparison for pile diameter of steel pipe pile at abutment

Alternative				Altrenative-1		Altrenative-2		Altrenative-3	
Pile Type				Steel pile pipe D=0.8m		Steel pile pipe D=1.1m		Steel pile pipe D=1.4m	
Plan of Pile Cap									
				L=43.5m	n=44nos	L=43.5m	n=40nos	L=43.5m	n=36nos
Displacement		mm		δx=7.8 ≦ δa=15 (OK)		δx=5.5 ≦ δa=15 (OK)		δx=5.7 ≦ δa=15 (OK)	
Lateral resistance		kN		Qr =5530 < N=5977		Qr =5530 < N=6116		Qr =5975 < N=6039	
Pile Reaction		kN		P_{nmax} =66109 < Ra=83512		P_{nmax} =82279 < Ra=120116		P_{nmax} =110930 < Ra=160088	
Pile body	thickness	mm		t=12mm(SKK400)		t=12mm(SKK400)		t=12mm(SKK400)	
	Mu	kN.m		732.8		1208.6		1858.7	
	Mn	kN.m		1657.7		3328.3		5190.1	
	fs	-		2.26		2.75		2.79	
	Extreme ft	N/mm^2		5.6 < 140.0		1.4 < 140.0		21.4 < 140.0	
	Extreme ft	N/mm^2		98.0 < 140.0		84.0 < 140.0		84.0 < 140.0	
Cost Estimate									
Item		unit	Unit cost (VND)	Material Quantities	Cost (VND)	Material Quantities	Cost (VND)	Material Quantities	Cost (VND)
Pile cap									
Concrete 28MPa		m^3	5,867,864	782.3	4,590,576,704	996.1	5,844,832,634	1,314.8	7,715,214,284
Lean Concrete		m^3	1,723,811	23.5	40,568,168	32.1	55,410,181	45.0	77,595,628
Blinding stone		m^3	696,000	47.1	32,759,328	64.3	44,744,448	90.0	62,659,488
Excavation		m^3	318,066	372.4	118,435,056	468.7	149,064,812	617.8	196,488,452
Cofferdam		ton	24,798,638	78.8	1,955,124,620	126.4	3,133,555,898	142.6	3,535,293,833
Sub total					6,737,463,876		9,227,607,972		11,587,251,686
Foundation									
Steel Pile (Diameter)	0.8m	m	0	1,914.0	26,511,762,933				
	1.1m	m	0			1,740.0	32,308,182,500		
	1.4m	m	0					1,566.0	54,928,566,000
Sub total					26,511,762,933		32,308,182,500		54,928,566,000
Total					33,249,226,809		41,535,790,472		66,515,817,686
ratio				1.000		1.249		2.001	
Evaliation				Most Recommended					

Source: Study Team

Table 8.4.4-7 Comparison for pile diameter of cast in place pile

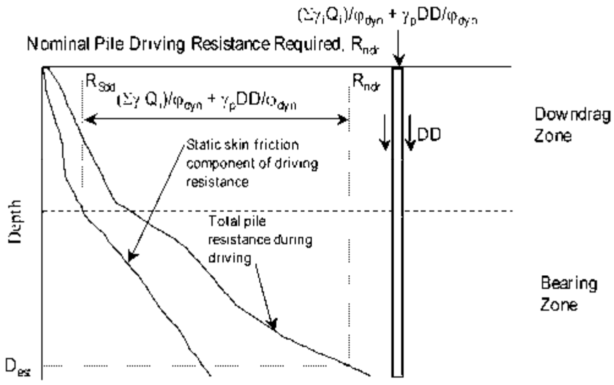
Alternative				Altrenative-1		Altrenative-2		Altrenative-3	
Pile Type				Cast-in-place pile D=1.2m		Cast-in-place pile D=1.5m		Cast-in-place pile D=2.0m	
Plan of Pile Cap									
				L=46.5m	n=12nos	L=46.5m	n=9nos	L=46.5m	n=6nos
Displacement		mm		$\delta x=1.3 \leqq \delta a=15$ (OK)		$\delta x=1.2 \leqq \delta a=15$ (OK)		$\delta x=1.5 \leqq \delta a=15$ (OK)	
Pile Reaction		kN		P_{nmax} =43003 ≦ Ra=43812		P_{nmax} =43005 ≦ Ra=44005		P_{nmax} =43003 ≦ Ra=43469	
Pile body	As	nos		D25-24nos(minimum)		D28-24nos(minimum)		D32-32nos	
	Mu	kN.m		1050.2		1587.1		3320.4	
	Mn	kN.m		2124.6		3464.2		8904.2	
	fs	-		2.02		2.18		2.68	
	Extreme	fc	N/mm^2	$2.6 < 11.2$		$2.3 < 11.2$		$1.9 < 11.2$	
		fs	N/mm^2	$30 < 202$		$26 < 202$		$21 < 182$	
Cost Estimate									
Item		unit	Unit cost (VND)	Material Quantities	Cost (VND)	Material Quantities	Cost (VND)	Material Quantities	Cost (VND)
Pile cap									
Concrete 28MPa		m^3	5,867,864	239.4	1,404,766,642	275.6	1,617,183,318	315.0	1,848,377,160
Lean Concrete		m^3	1,723,811	10.0	17,238,110	11.0	18,961,921	13.0	22,409,543
Blinding stone		m^3	696,000	19.0	13,224,000	22.0	15,312,000	25.0	17,400,000
Excavation		m^3	318,066	110.0	34,987,260	120.0	38,167,920	133.0	42,302,778
Cofferdam		ton	24,798,638	86.0	2,132,682,868	90.0	2,231,877,420	97.0	2,405,467,886
Sub total					3,602,898,880		3,921,502,579		4,335,957,367
Foundation									
Bord Pile (Diameter)	1.2m	m	14,553,000	576.0	8,382,528,000				
	1.5m	m	17,423,000			432.0	7,526,736,000		
	2.0m	m	27,343,000					288.0	7,874,784,000
Sub total					8,382,528,000		7,526,736,000		7,874,784,000
Total					11,985,426,880		11,448,238,579		12,210,741,367
ratio				1.047		1.000		1.067	
Evaliation						Most Recommended			

Source: Study Team

(4) Downdrag load

1) Analysis of downdrag by consolidation

For clay, the drainage time after the application of a load is long because the permeability of clay is small thus consolidation occurs over a long period of time. In contrast, sand and gravel have large permeability's and after the application of a load, the water drains rapidly. Consolidation occurs quickly because water in sand or gravel moves easily through the pores. Also, the amount of compression is small in sand and gravel. For this reason, consolidation is usually used for fine grained soil, such as clay and silt. In the case of a pile driving into the ground where consolidation will occur by reclamation, the foundation should be designed so that the available factored geotechnical resistance is greater than the factored loads applied to the pile, including the downdrag (DD), as following drawing.



Source: Study Team

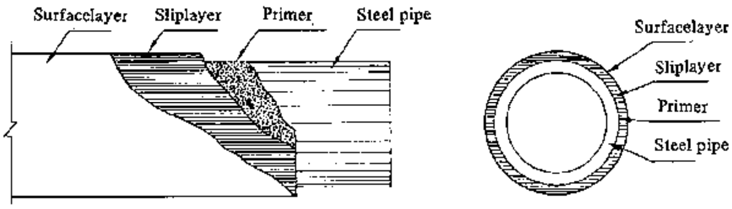
Figure 8.4.4-4Design of pile foundations for downdrag

2) Countermeasure to downdrag

Conventional measures against negative friction, such friction was reduced by increasing the strength of piles or forming a pile group new techniques have recently been utilized , where reduction in negative friction is achieved by use of piles covered with the special asphalt called “Slip layer compound”-SL piles. In this project, countermeasure to downdrag adopts the SL piles due to its economical efficiency.

3) Principles of Reducing Negative Friction on SL pile and standard section

The slip layer material, that is, special asphalt is one of the typical viscoelastic materials, of which physical properties depend on the velocity of shearing. When instantaneous load acts on a pile, especially at the time of pile driving, the velocity of shearing developed on the pile surface increases and thus, asphalt applied on the pile surface present an elastic property. In this case, a great shear resistance attributable to the elastic property enables the pile to be driven without any slippage of the slip layer. On the other hand, where a pile is subject to a slow ground movement such as land subsidence, the velocity of shearing developed on the pile surface is very low; asphalt applied on the pile surface presents a viscous property. In this case, slippage occurring in the slip layer due to the subsidence serves to prevent shearing force from being transmitted to the pile, thus permitting negative friction to be reduced.



Source: Study Team

Figure 8.4.4-5Standard Sections of a SL piles

4) Load combination take account of downdrag

Downdrag is not combined with transient loads because transient loads caused downdrag movement of the pile or pier relative to the ground, causing temporary reduction or elimination of downdrag loads. Therefore, only permanent loads need be included with the drag loads as follows:

Load combination and load factor.

Table 8.4.4-8Load combination and load factor

Load combination and Load factor for Abutment

Load Combination Limit State		Permanent							Transient						WA	WS	WL	FR				TG	SE	EQ	CT	CV
		DC	DD	DW	EH	EV	ES	EL	LL	IM	CE	BR	PL	LS					TU	CR	SH					
STRENGTH-Ⅰ	max	1.25	1.80	1.50	1.50	1.35	1.50	1.75	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	min	0.90	0.45	0.65	0.9	0.90	0.75	1.75	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
STRENGTH-Ⅱ	max	1.25	1.80	1.50	1.50	1.35	1.50	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	min	0.90	0.45	0.65	0.90	0.90	0.75	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		
STRENGTH-Ⅲ	max	1.25	1.80	1.50	1.50	1.35	1.50	1.35	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		
	min	0.90	0.45	0.65	0.9	0.90	0.75	1.35	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
EXTREME	max	1.25	-	1.50	1.5	1.35	1.50	0.50	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	mini	0.90	-	0.65	0.9	0.90	0.75	0.50	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
SERVICE		1.00	1.00	1.00	1.00	1.00	1.00	1.00	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Load combination and Load factor for Pier

Load Combination Limit State		Permanent							Transient						WA	WS	WL	FR				TG	SE	EQ	CT	CV
		DC	DD	DW	EH	EV	ES	EL	LL	IM	CE	BR	PL	LS					TU	CR	SH					
STRENGTH-Ⅰ	max	1.25	1.80	1.50	-	-	-	1.75	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	min	0.90	0.45	0.65	-	-	-	1.75	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
STRENGTH-Ⅱ	max	1.25	1.80	1.50	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	min	0.90	0.45	0.65	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		
STRENGTH-Ⅲ	max	1.25	1.80	1.50	-	-	-	1.35	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		
	min	0.90	0.45	0.65	-	-	-	1.35	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
EXTREME	max	1.25	-	1.50	-	-	-	0.50	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	mini	0.90	-	0.65	-	-	-	0.50	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
SERVICE		1.00	1.00	1.00	-	-	-	1.00	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

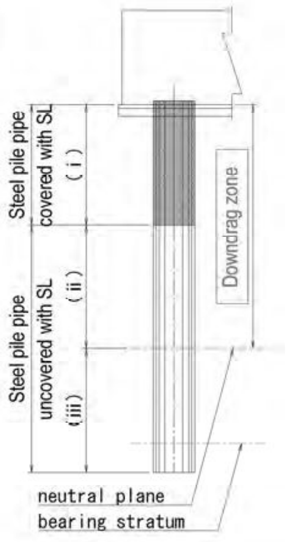
DC : Component and Attachment
DD : Downdrag
DW : Wearing Surfaces and Utilities
EH : Horizontal Earth Pressure
EL : Locked-in Erection Stress
EV : Vertical Earth Pressure
ES : Earth surcharge load
BR : Vehicular braking force
CE : Vehichlar centrifugal forth
CR : Creep
CT : Vehicular collision force
CV : Vessel collision force
EQ : Earthquake
FR : Friction
IM : Vehicular dynamic load allowance
LL : Vehicular live load
LS : Live load surcharge
PL : Pedestrian live load
SE : Settlement
SH : Shrinkage
TG : Temperature gradient
TU : Uniform temperature
WA : Water load and stream pressure
WL : Wind on live load
WS : Wind load on structure

Source: Study Team

5) Downdrag load and the range of SL pile

SL piles should be used as middle pile at a level above neutral point on which a greater negative friction acts. Downdrag loads can be estimated in a similar way to calculation of positive shaft resistance. Pile shaft resistance to calculate the downdrag loads shows as follows:

	Load Combination	
	Combined with only permanent load	Combined with transient loads
(i)	Negative shaft resistance Downdrag loads : estimate calculation of shaft resistance qs : -2 kN/m^2	Take no account of shaft resistance Downdrag loads : — qs : 0 kN/m^2
(ii)	Negative shaft resistance Downdrag loads : estimate calculation of shaft resistance qs : Using α or λ method.	Positive shaft resistance Downdrag loads : — qs : Using α or λ method.
(iii)	Positive shaft resistance Downdrag loads : — qs : Using α or λ method.	Positive shaft resistance Downdrag loads : — qs : Using α or λ method.



Source: Study Team

Figure 8.4.4-6 Downdrag load and the range of SL pile

8.4.4.4 Conclusions of Study

(1) Results of Study for Foundation Type

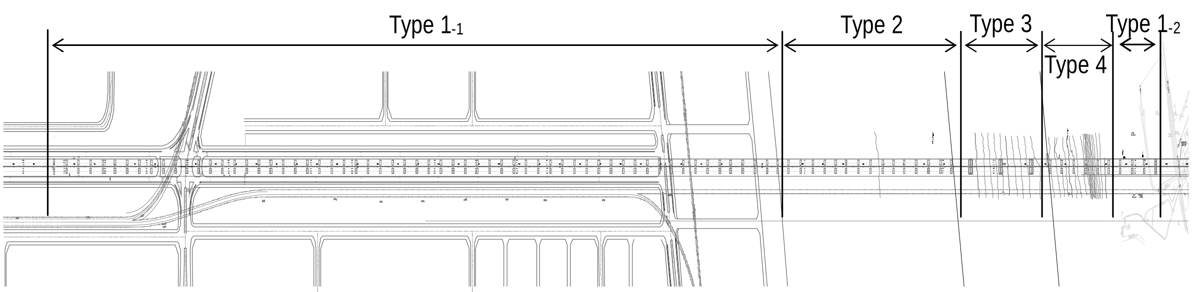
The results of study of foundation type for Approach bridge and main bride are shown in the following table. The underlined parts indicate the changes from the results in JICA's Preparatory Study (SAPROF Study).

Table 8.4.4-9 The results of Study of Foundation Type

Study Type	Type-1_{-1}	Tupe-2	Type-3	Type-4	Type-1_{-2}
Bridge Type	Approach	Approach	Main Br.	Approach	Approach
Station	STA.4+501.3~ STA.8+77.12	STA.8+130.1~ STA.8+935.0	STA.9+30.0~ STA.9+330.0	STA.9+425.0~ STA.9+599.8	STA.9+659.8~ STA.9+944.2
Pier No.	A1 ~ P60	P61 ~ P75	P76 ~ P78	P79 ~ P82	P83~ A2
Reclamation Plan	in operation	No	No	No	planning for the future
Bridge Span length (m)	60.0	60.0	150.0	60.0	60.0
Estimated Corrosion Thickness of Steel Pile (mm)	2	7	7	7	2
Water depth (m)	2.5~3.8	3.2~6.5	7.1~11.0	8.3~11.5	1.63~3.8
E.L. of Pile Cap*1	Variation 2	Variation 2 or 3	E.L.-5.0 (Top of Pile Cap)	Variation 4	Variation 2
Temp. Cofferdam	Sheet Pile	Sheet Pile	Pipe Pile	Sheet Pile	Sheet Pile
Type of Foundation	Pile Foundation	Pile Foundation	SPSP*1 Foundation **(Separate Type)**	**Multi Column Pile (under water)**	Pile Foundation
Type of Pile	Steel Pipe pile **with surface treating*2**	**Cast in Place Pile**	Steel Pipe Sheet Pile	**Cast in Place Pile**	Steel Pipe pile **with surface treating*3**
Determining Factor	Countermeasure to Downdrag	Construction Cost	Construction Period	Constructability and Aesthetics	Countermeasure to Downdrag
SAPROF Study for Type of Pile	Steel Pipe Pile	Steel Pipe Pile	SPSP Integrated Type	Steel Pipe Pile	Steel Pipe Pile

Note, *1: Steel Pipe Sheet Pile,*2: consider countermeasure to Downdrag.

Source: Study Team



Source: Study Team

Figure 8.4.4-7 Grouping for Foundation Study

8.4.5 Study on Type of Bridge Foundation

8.4.5.1 General

This study consists of two (2) sub-studies; the study on selection of approach bridge foundation, and the study on selection of main bridge foundation. In the study on selection of approach bridge foundation, study of site conditions for selecting an appropriate foundation type is a key discussion. In the study on selection of main bridge foundation, study of an appropriate foundation type and comparison of structure (integrated type and separate type) are the key discussions.

8.4.5.2 Study on Approach Bridge Foundation

(1) Selection of Foundation Type for Approach Bridge (Type-1)

1) General

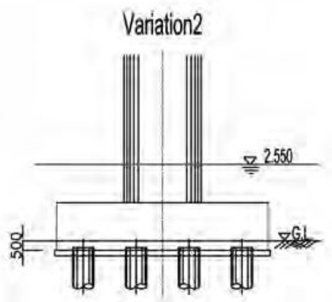
In the study of Type-1, there is reclamation planning with large number of pier. Therefore, the downdrag effect for pile by the definition of consolidation of clay layer and construction period needs to be examined in detail.

2) Site Condition

The site conditions are shown in below table. In the Variation 1, top of pile cap is seated below the seabed when the sea is shallow and pier height is low. Variation 2, bottom of pile cap is seated on the sea bed of deepwater. The construction of pile cap at below the sea bed is difficult in terms of structure of cofferdam. Furthermore, the pile cap is backfilled by reclamation in the future.

Table 8.4.5-1 Site Condition for Study of Type-1

Study Type	Type-1_{-1}	Type-1_{-2}
Bridge Type	Approach	Approach
Station	STA.4+561.3 ~8+77.12	STA.9+561.3~9+944.2
Pier No.	A1 ~ P60	P84 ~ A2
Reclamation Plan	Have a project	Have a plan
Bridge Span length (m)	60.0	60.0
Estimated Corrosion Thickness of Steel Pile (mm)	2	2
Water depth (m)	2.5~3.8	1.63~3.8
E.L. of Pile Cap	Variation 2	Variation 2



Source: Study Team

Figure 8.4.5-1 Pile Cap Elevation of Variations 2

3) Comparative Study

a) Foundation Types for Comparison

In this comparative study, the following three (3) alternatives are studied.

Altenative-1: Steel pipe pile foundation
(Without countermeasures against downdrag)

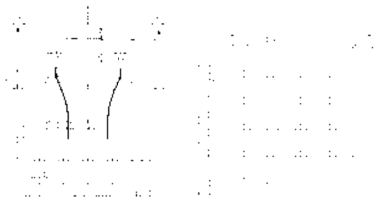
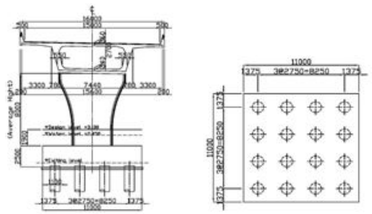
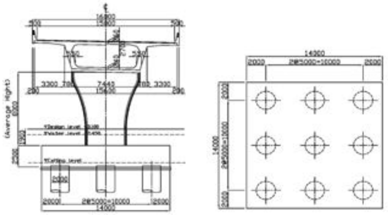
Altenative-2: Steel pipe pile foundation
(With countermeasures against downdrag by pile surface treatment)

Altenative-3: Cast in place pile foundation
(Countermeasure against downdrag is increasing the number of piles. Surface treatment can't be applied to this pile type)

b) Result of Comparative Study

The result of comparative study is shown in the Table 13.5.5-2. As this table indicates, Altenative-2, Steel pipe pile foundation with treatment of pile surface for downdrag, is the most recommendable foundation type for Type-1 of approach bridge because of its advantages in low construction cost and shortest construction period on account of consider countermeasure to downdrag.

Table 8.4.4-2 Comparison on Foundation Type-1 for Approach Bridge

EvaGuation Items		Area to take account of negative friction STATION : STA.4+500～STA.8+77, Pier number : P1 ～P61					
		Alternative-1 Steel Pipe Pile Foundation with Sheet Pile Cofferdam		Alternative-2 Steel Pipe Pile covered with thin coat of bitumen Foundation with Sheet Pile Cofferdam		Alternative-3 Cast In Place Pile Foundation with Cofferdam	
Side View PiGe arrangement		Diameter of pile : 1100 mm Total number of pile : 25 Total lenlth of pile : 46.0 m Thickness : 12.0 mm 		Diameter of pile : 1100 mm Total number of pile : 16 Total lenlth of pile : 46.0 m Thickness : 12.0 mm 		Diameter of pile : 2000 mm Total number of pile : 9 Total lenlth of pile : 48.0 m 	
Structural Aspect and Stability	10	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.96 - Temporary coffer dam work for foundation construction is necessary. - Total number of Pile should magnify from 16nos to 25nos for countermeasure to negative friction.	8	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.96 - Temporary coffer dam work for foundation construction is necessary. - Large number of Steel Sheet Piles and steel pipe. piles - Use steel Pipe Pile covered with thin coat of bitumen for countermeasure to negative friction.	8	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.94 - Temporary coffer dam work for foundation construction is necessary. - Large number of Steel Sheet Piles and C.I.P. piles. - Diameter of Pile should magnify from 1500mm to 2000mm for countermeasure to negative friction.	4
Construction Cost (for Foundation)	40	Item / Quantity / Unit Cost (VND) / Total (1,000VND) Pile Cap Concrete / 473m3 / 5,867,864 / 2,773,739 Pile / 1213m / 14,106,802 / 17,104,497 Lean Concrete / 19m3 / 1,723,811 / 32,752 Blinding stone / 38m3 / 696,000 / 26,448 Excavation / 176m3 / 318,066 / 55,980 Cofferdam / 111ton / 24,798,638 / 2,752,649 Driving *1 / 834m / 622,237 / 518,946 Total 23,265,011 Ratio **1.441**	16	Item / Quantity / Unit Cost (VND) / Total (1,000VND) Pile Cap Concrete / 303m3 / 5,867,864 / 1,775,029 Pile / 736m / 17,558,795 / 12,923,273 Lean Concrete / 12m3 / 1,723,811 / 20,686 Blinding stone / 24m3 / 696,000 / 16,704 Excavation / 128m3 / 318,066 / 40,712 Cofferdam / 93ton / 24,798,638 / 2,306,273 Driving *1 / 702m / 622,237 / 436,810 Total 17,519,488 Ratio **1.085**	32	Item / Quantity / Unit Cost (VND) / Total (1,000VND) Pile Cap Concrete / 490m3 / 5,867,864 / 2,875,253 Pile / 432m / 22,743,000 / 9,824,976 Lean Concrete / 20m3 / 1,723,811 / 34,476 Blinding stone / 39m3 / 696,000 / 27,144 Excavation / 181m3 / 318,066 / 57,570 Cofferdam / 113ton / 24,798,638 / 2,802,246 Driving *1 / 846m / 622,237 / 526,413 Total 16,148,078 Ratio **1.000**	40
Construction Plan and Period	10	- Workability is inferior due to large temporary cofferdam work in the sea. Cofferdam Work 14 days Pile work*2 (1.5pile/day) 21 days Pile Cap 48 days Total **83 days**	4	- Workability is inferior due to large temporary cofferdam work in the sea Cofferdam Work 9 days Pile work*2 (1.5pile/day) 13 days Pile Cap 29 days Total **51 days**	6	- Workability is inferior due to large temporary cofferdam work in the sea Cofferdam Work 14 days Pile work*2 (0.5pile/day) 22 days Pile Cap 50 days Total **86 days**	4
Maintenance	15	- Superior in Maintenance with small number of maintenance points.	9	- Superior in Maintenance with small number of maintenance points.	9	- Superior in Maintenance with small number of maintenance points.	9
STEP Clearance	10	- 90% (Preliminary Estimate) - Large number of steel pipe pile acceptance a contribution	10	- 90% (Preliminary Estimate) - Large number of steel pipe pile acceptance a contribution	10	- 21% (Preliminary Estimate) - small number of Cast in place pile acceptance a contribution	8
Aesthetics	5	- Slender appearance of Pier - Pile cap not to be exposed above water level.	3	- Slender appearance of Pier - Pile cap not to be exposed above water level.	3	- Slender appearance of Pier - Pile cap not to be exposed above water level.	3
New Technology	5	- Steel Pipe Pile Foundation is new technology in Vietnam	4	- Steel Pipe Pile Foundation is new technology in Vietnam - Steel Pipe Pile covered with thin coat of bitumen for countermeasure to negative friction is new technology in Vietnam.	5	- Cast in pile (D=2.0m) is no special technology in Vietnam.	3
Environmental Aspect	5	- Superior in Environmental aspect with small number of excavated soil & bentonite water.	5	- Superior in Environmental aspect with small number of excavated soil & bentonite water.	5	- Environmental measures for surplus soil and discharging water is necessary.	2
Evaluation	100	- Superior in Environmental aspect with small number of excavated soil & bentonite water. Not Recommended	**59**	- Construction cost is lowest in area to take account of negative friction. - Minimum Construction period with efficient workability. - Superior in Environmental aspect with small number of excavated soil & bentonite water. Most Recommended	**78**	- Environmental measures for surplus soil and discharging water is necessary. Not Recommended	**73**

*1. Including for Puling of steel sheet pile.　　*2. Including for Pile top treatment

Source : Study Team

(2) Selection of Foundation Type for Approach Bridge (Type-2)

1) General

In the study of Type-2, there is less number of piers; the point of this study is the construction costs need to focus on the detail.

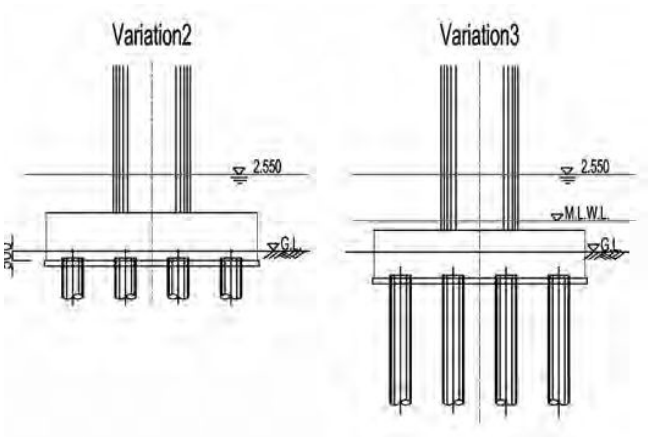
2) Site Condition

The site conditions indicate as following. In the Alternative 3, top of pile cap shall be set below Mean Low Water Level (EL.0.000) in order to comply with the regulation which prohibits exposure of pile cap body above Mean Low Water Level.

Table 8.4.5-3 Site Condition for Study of Type-2

Study Type	Tupe-2
Bridge Type	Approach
Station	STA.4+561.3 ~8+77.12
Pier No.	P61 ~ P75
Reclamation Plan	No
Bridge Span length (m)	60.0
Estimated Corrosion Thickness of Steel Pile (mm)	7
Water depth (m)	3.2~6.5
E.L. of Pile Cap	Variation 2 or 3

Source: Study Team



Source: Study Team

Figure 8.4.5-2 Pile Cap Elevation of Variation 2 or 3

3) Comparative Study

a) Foundation Types for Comparison

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

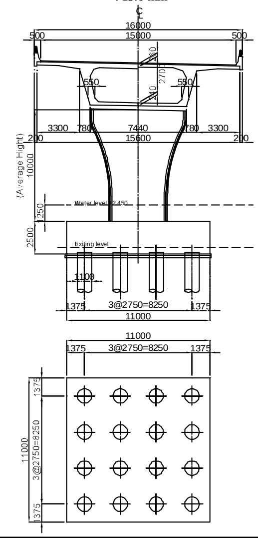
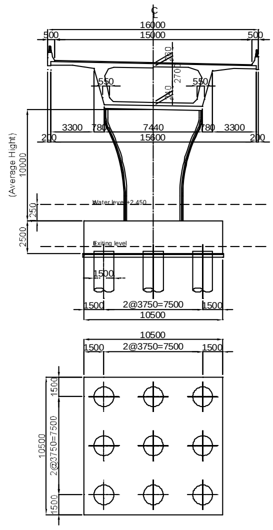
Altenative-1: Steel Pipe Pile Foundation

Altenative-2: Cast in Place Pile Foundation

b) Result of Comparative Study

The result of comparative study is shown in following Table. As this table indicates, Altenative-2, cast in place pile foundation, is the most recommendable foundation type for Type-2 of approach bridge because of its advantages in lowest construction cost.

Table 8.4.5-4 Comparison on Foundation Type-2 for Approach Bridge

Evaluation Items		Area to take no account of negative friction STATION :STA.8+77～STA.9+944, Pier number : P61～P75 Alternative-1 Steel Pipe Pile Foundation with Sheet Pile Cofferdam		Alternative-2 Cast In Place Pile Foundation with Cofferdam	
Side View Pile arrangement		Diameter of pile : 1100 mm Total number of pile : 16 Total length of pile : 43.5 m Thickness : 19.0 mm 		Diameter of pile : 1500 mm Total number of pile : 9 Total length of pile : 42.0 m 	
Structural Aspect and Stability	10	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.91. - Temporary cofferdam work for foundation construction is necessary. - Large number of Steel Sheet Piles and steel pipe piles	6	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.91. - Temporary cofferdam work for foundation construction is necessary. - Small number of Steel Sheet Piles and C.I.P. piles.	6

Construction Cost (for Foundation) — weight 40; Alternative-1 score 16; Alternative-2 score 40

Alternative-1	Quantity	Unit Cost (VND)	Total (1,000VND)	Alternative-2	Quantity	Unit Cost (VND)	Total (1,000VND)
Pile Cap Concrete	303m3	5,867,864	1,775,029	Pile Cap Concrete	276m3	5,867,864	1,617,183
Pile	696m	15,728,273	10,946,878	Pile	378m	17,514,043	6,620,308
Lean Concrete	12m3	1,723,811	20,686	Lean Concrete	11m3	1,723,811	18,962
Blinding stone	24m3	696,000	16,704	Blinding stone	22m3	696,000	15,312
Excavation	128m3	318,066	40,712	Excavation	120m3	318,066	38,168
Cofferdam	93ton	24,798,638	2,306,273	Cofferdam	90ton	24,798,638	2,231,877
Driving *1	702m	622,237	436,810	Driving *1	678m	622,237	421,877
		Total	15,543,093			Total	10,963,688
		Ratio	**1.418**			Ratio	**1.000**

Evaluation Items		Alternative-1		Alternative-2	
Construction Plan and Period	10	- Workability is inferior due to large temporary cofferdam work in the sea. Cofferdam Work 9 days Pile work 13 days Pile Cap 29 days Column & Column Beam 23 days Total **74 days**	8	- Workability is inferior due to large temporary cofferdam work in the sea. Cofferdam Work 9 days Pile work *2 20 days Pile Cap 29 days Column & Column Beam 23 days Total **81 days**	6
Maintenance	15	- Superior in Maintenance with small number of maintenance points.	9	- Superior in Maintenance with small number of maintenance points.	9
STEP Clearance	10	89%(preliminary Estimate) - Large number of steel pipe pile acceptance a contribution	10	- 27% (Preliminary Estimate) - small number of Cast in place pile acceptance a contribution	8
Aesthetics	5	- Slender appearance of Pier - Pile cap not to be exposed above water level.	3	- Slender appearance of Pier - Pile cap not to be exposed most of time above water level.	3
New Technology	5	- Steel Pipe Pile Foundation is new technology in Vietnam.	5	・Cast in pile (D=1.5m) is no special technology in Vietnam.	3
Environmental Aspect	5	- Superior in Environmental aspect with small number of excavated soil & bentonite water.	5	- Environmental measures for surplus soil and discharging water is necessary.	2
Evaluation	100	- Superior in Environmental aspect with small number of excavated soil & bentonite water. - Minimum Construction period with efficient workability. Not Recommended	62	- Environmental measures for surplus soil and discharging water is necessary. - Construction cost is lowest in area to take no account of negative friction. Most Recommended	77

Note) *1. Including for Pile top treatment *2.Including for Pile top treatment

Source : Study Team

(3) Selection of Foundation Type for Approach Bridge (Type-4)

1) General

In the study of Type-4, there is planning closed the navigation channel with Cat Hai Island at deep water. The point of this study is the structural aesthetics which shall be harmonize with scenery of Cut Hai Island and safe construction at the deep water.

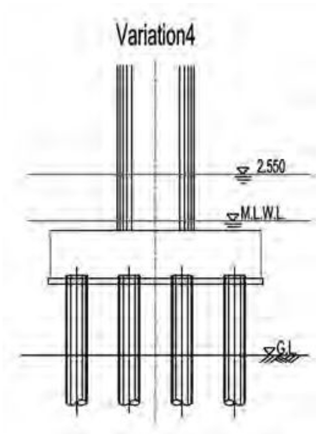
2) Site Condition

The site conditions indicate as following. In the Alternative 4, top of pile cap is seated below the Mean Low Water Level (EL-1.670) as same as Alternative 3. In addition to the condition, bottom of pile cap is not contact onto seabed due the deepwater; construction of pile cap at riverbed is difficult in terms of structure of cofferdam.

Table 8.4.5-5 Site Conditions for Study of Type-4

Study Type	Type-4
Bridge Type	Approach
Station	STA.9+425.0 ~9+599.8
Pier No.	P79 ~ P82
Reclamation Plan	No
Bridge Span length (m)	60.0
Estimated Corrosion Thickness of Steel Pile (mm)	7
Water depth (m)	8.3~11.5
E.L. of Pile Cap*	Variation 4

Source: Study Team



Source: Study Team

Figure 8.4.5-3Pile Cap Elevation of Variation 4

3) Comparative Study

a) Foundation Types for Comparison

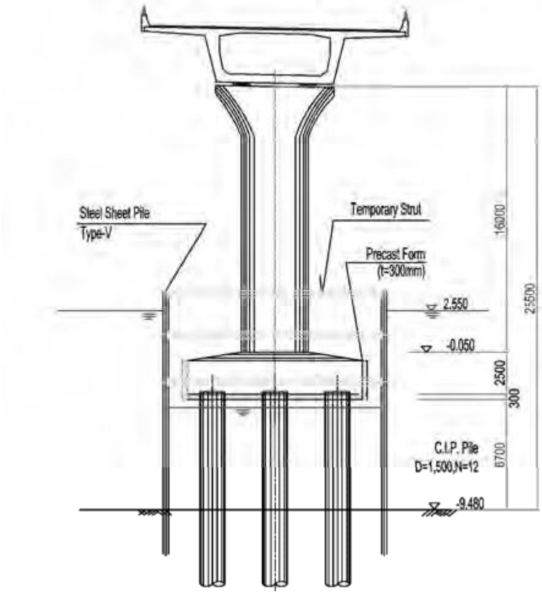
In this comparative study, the following three (3) alternatives are studied. In the Alternative-1 and 2, due to the deepwater, cofferdam work by steel sheet pile is risky work. Therefore, steel sheet pipe pile is selected for the cofferdam work for Alternative-1 and 2. For this reason, in the Alternative-3 is selected multi column foundation by cast in place pile. However, Alternative-3, the portion of pile cap to be exposed above water level is large. The Alternative-4 can be cootch the pile cap to below the mean low water level (refer to following construction drawing).

Alternative-1: Steel Pipe Pile Foundation with Steel Pipe Sheet Pile Cofferdam

Alternative-2: Cast in Place Pile Foundation with Steel Pipe Sheet Pile Cofferdam

Alternative-3: Multi Column Foundation by Cast in Place Pile

Alternative-4: Multi Column Foundation by Cast in Place Pile with Steel Sheet Pile Cofferdam



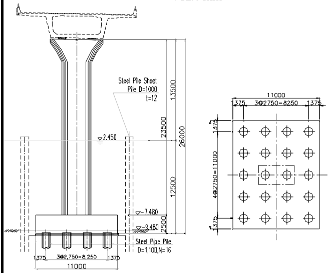
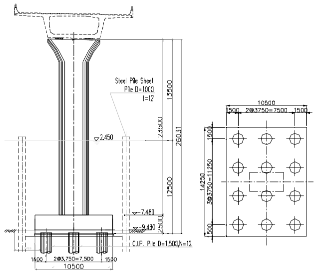
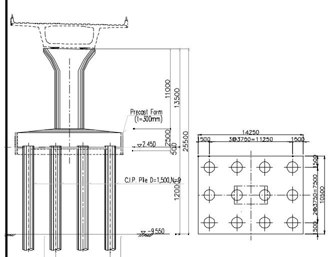
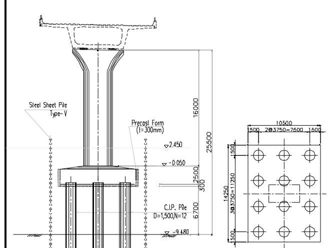
Source: Study Team

Figure 8.4.5-4 Construction Plan of Alternative-4

b) Result of Comparative Study

The result of comparative study is shown in following Table. As the table indicates, Altenative-4, Multi Column Foundation by Cast in Place pile with Steel Sheet Pile Cofferdam, is the most recommendable foundation type for Type-4 of approach bridge because of its advantages in construction cost, construction period and aesthetics. .

Table 8.4.4-6 Comparison on Foundation type-4 for Approach Bridge

Evaluation Items	Max. Point	Alternative-1 Steel Pipe Pile Foundation with Steel Pipe Pile Cofferdam		Alternative-2 Cast in Place Pile Foundation with Steel Pipe Pile Cofferdam		Alternative-3 Cast In Place Multi Column Foundation		Alternative-4 C.I.P. Pile Multi Column Foundation with Steel Sheet Pile Cofferdam	
Side View		Diameter of pile : 1100 mm Total number of pile : 20 Total length of pile 40m Thickness : 12.0 mm 		Diameter of pile : 1100 mm Total number of pile : 12 Total length of pile : 40.0 m 		Diameter of pile : 1500 mm Total number of pile : 12 Total length of pile : 52.0 m 		Diameter of pile : 1500 mm Total number of pile : 12 Total length of pile : 49.5 m 	
Structural Aspect and Stability	10	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.91. - Temporary coffer dam work for foundation construction is necessary. - Large number of - Large number of steel pipe piles	8	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.92. - Temporary coffer dam work for foundation construction is necessary. - Large number of - Small number of C.I.P. piles	8	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.92. - Steel casing is neglected from Cross-section calculation. - Temporary Precast form for pile cap construction is necessary. - Protection of Ship allision for pile is necessary.	4	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.89. - Steel casing is neglected from Cross-section calculation. - Temporary Precast form for pile cap construction is necessary. - Temporary coffer dam work for foundation construction is necessary. - Vertical force can be reduced due to buoyancy of Pile Cap.	10

Construction Cost (for Foundation) — Max. Point 40

Alternative-1 (Point: 0)

	Quantity	Unit Cost (VND)	Total (1,000VND)
Pile Cap Concrete	378m3	2,652,255	1,002,884
Column Concrete	264m3	2,652,255	1,002,884
Pile	258ton	43,982,414	11,329,870
Antiwaashout water Concrete	15m3	1,723,811	26,073
Precast Form	30m3	696,000	21,054
Excavation	102m3	318,066	32,562
Cofferdam	317ton	43,982,414	13,933,629
Driving *1	2340m	622,237	1,456,035
		Total	28,804,990
		Ratio	**1.997**

Alternative-2 (Point: 16)

	Quantity	Unit Cost (VND)	Total (1,000VND)
Pile Cap Concrete	374m3	2,652,255	992,109
Column Concrete	264m3	2,652,255	992,109
Pile	480m	17,514,043	8,406,741
Antiwaashout water Concrete	15m3	1,723,811	25,793
Precast Form	30m3	696,000	20,828
Excavation	102ton	318,066	32,304
Cofferdam	317ton	43,982,414	13,933,629
Driving *1	2160m	622,237	1,344,032
		Total	25,747,544
		Ratio	**1.785**

Alternative-3 (Point: 40)

	Quantity	Unit Cost (VND)	Total (1,000VND)
Pile Cap Concrete	337m3	2,652,255	892,898
Column Concrete	124m3	2,652,255	892,898
Pile	624ton	17,514,043	10,928,763
Antiwaashout water Concrete	67m3	5,304,510	357,159
Precast Form	170m3	7,956,765	1,349,766
Excavation	ton	318,066	0
Cofferdam	ton	43,982,414	0
Driving *1	m	622,237	0
		Total	14,421,484
		Ratio	**1.000**

Alternative-4 (Point: 32)

	Quantity	Unit Cost (VND)	Total (1,000VND)
Pile Cap Concrete	337m3	2,652,255	892,898
Column Concrete	180m3	2,652,255	892,898
Pile	594ton	17,514,043	10,403,342
Antiwaashout water Concrete	m3	5,304,510	0
Precast Form	170m3	7,956,765	1,349,766
Excavation	ton	318,066	0
Cofferdam	73ton	24,798,638	1,812,284
Driving *1	m	622,237	0
		Total	15,351,188
		Ratio	**1.064**

Evaluation Items	Max. Point	Alternative-1		Alternative-2		Alternative-3		Alternative-4	
Construction Plan and Period	10	- Workability is inferior due to large temporary cofferdam work in the sea Cofferdam Work 39 days Pile work 16 days Pile Cap 40 days Column & Column Beam 33 days Total **128 days**	4	- Workability is inferior due to large temporary cofferdam work in the sea Cofferdam Work 38 days Pile work *2 27 days Pile Cap 39 days Column & Column Beam 33 days Total **137 days**	6	- Workability is superior due to no cofferdam work. Precast Form Work 15 days Pile work *2 29 days Pile Cap 39 days Column & Column Beam 16 days Total **99 days**	10	- Workability is superior with precast form and steel pipe pile Precast Form Work 18 days Pile work *2 29 days Pile Cap 39 days Column & Column Beam 23 days Total **109 days**	8
Maintenance	15	- Superior in Maintenance with small number of maintenance points.	10	- Superior in Maintenance with small number of maintenance points.	10	- Inferior in Maintenance due to large precast form and pile cap on the sea.	10	- Inferior in Maintenance due to large precast form and pile cap on the sea.	10
STEP Clearance	10	- 85% (Preliminary Estimate) - Large number of steel pipe pile acceptance a contribution	10	- 32% (Preliminary Estimate) - small number of Cast in place pile acceptance a contribution	4	11%(preliminary Estimate) - small number of Cast in place pile acceptance a contribution	6	11%(preliminary Estimate) - small number of Cast in place pile acceptance a contribution	6
Aesthetics	5	- Slender appearance of Pier - Pile cap not to be exposed above water level.	5	- Slender appearance of Pier - Pile cap not to be exposed above water level.	5	- Large pile cap to be exposed above water level is large.	0	- Pile cap not to be exposed above water level.	5
New Technology	5	- Steel Pipe Pile Foundation is new technology in Vietnam	3	- Cast in pile (D=1.5m) is no special technology in Vietnam	3	- Multi Column Foundation with Cast in pile (D=1.5m) is no special technology in Vietnam	3	- Multi Column Foundation with Cast in pile (D=1.5m) is no special technology in Vietnam	4
Environmental Aspect	5	- Superior in Environmental aspect with small number of excavated soil & bentonite water	5	- Environmental measures for surplus soil and discharging water is	3	- Environmental measures for surplus soil and discharging water is necessary.	3	- Environmental measures for surplus soil and discharging water is necessary.	3
Evaluation	100	- Construction cost is highest - Workability is inferior due to temporary coffer dam work in the sea. Less Recommended	45	- Workability is inferior due to temporary coffer dam work and C.I.P work in the sea. - Environmental measures for surplus soil and discharging water is excavated soil & bentonite water Not Recommended	55	- Minimum Construction cost with construction period. Not Recommended	76	- Minimum Construction cost with construction period. **Most Recommended**	78

Note) *1. Including for Pile top treatment *2.Including for Pile top treatment

Source : Study Team

8.4.6 Detailed Design of Approach Bridge

8.4.6.1 Design of Substructure of Approach Bridge

(1) Abutment

1) Material to be used

a) Concrete

Concrete σck : 28N/mm2

b) Reinforcement

Reinforcement : SD345

c) Back filling material

Density : 19kN/m3

Internal friction angle : 30°

2) Reclamation plan

There is reclamation plan from A1 to P60 and P83 toA2.

In case of analyze the stability and sectional force to take account of the cover soil weight due to reclamation plan as follow;

Table 8.4.6-1 Elevation and Reclamation thickness from bottom of pile cap

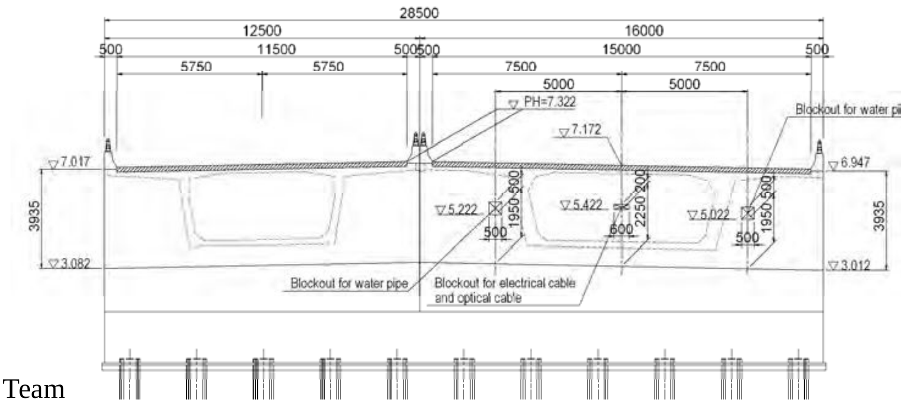
	Elavation of Design Level (m)	Elavation of bottom of pilecap (m)	Reclamation thickness from bottom of pile cap (m)		Elavation of Design Level (m)	Elavation of bottom of pilecap (m)	Reclamation thickness from bottom of pile cap (m)
A1	3.10	-0.68	3.8	P43	3.10	-1.86	5.0
P1	3.10	-1.99	5.1	P44	3.10	-1.68	4.8
P2	3.10	-2.77	5.9	P45	3.10	-1.63	4.7
P3	3.10	-3.05	6.2	P46	3.10	-1.82	4.9
P4	3.10	-2.33	5.4	P47	3.10	-1.64	4.7
P5	3.10	-1.75	4.8	P48	3.10	-1.46	4.6
P6	3.10	-0.89	4.0	P49	3.10	-1.28	4.4
P7	3.10	-0.77	3.9	P50	3.10	-5.74	8.8
P8	3.10	-0.78	3.9	P51	3.10	-5.50	8.6
P9	3.10	-0.92	4.0	P52	3.10	-5.94	9.0
P10	3.10	-0.82	3.9	P53	3.10	-5.93	9.0
P11	3.10	-1.11	4.2	P54	3.10	-5.48	8.6
P12	3.10	-1.14	4.2	P55	3.10	-5.70	8.8
P13	3.10	-1.30	4.4	P56	3.10	-5.71	8.8
P14	3.10	-1.48	4.6	P57	3.10	-5.89	9.0
P15	3.10	-1.28	4.4	P58	3.10	-5.57	8.7
P16	3.10	-1.34	4.4	P59	3.10	-5.75	8.9
P17	3.10	-1.52	4.6	P60	3.10	-5.54	8.6
P18	3.10	-1.70	4.8	P61	-	-5.57	-
P19	3.10	-1.38	4.5	P62	-	-5.75	-
P20	3.10	-1.68	4.8	P63	-	-5.93	-
P21	3.10	-1.74	4.8	P64	-	-5.61	-
P22	3.10	-1.42	4.5	P65	-	-5.90	-
P23	3.10	-1.60	4.7	P66	-	-5.93	-
P24	3.10	-1.78	4.9	P67	-	-5.61	-
P25	3.10	-1.58	4.7	P68	-	-5.76	-
P26	3.10	-1.64	4.7	P69	-	-5.48	-
P27	3.10	-1.82	4.9	P70	-	-5.92	-
P28	3.10	-2.00	5.1	P71	-	-5.64	-
P29	3.10	-2.17	5.3	P72	-	-5.48	-
P30	3.10	-2.40	5.5	P73	-	-5.80	-
P31	3.10	-2.30	5.4	P74	-	-5.62	-
P32	3.10	-2.26	5.4	P75	-	-5.78	-
P33	3.10	-2.15	5.3	P79	-	-4.18	-
P34	3.10	-1.98	5.1	P80	-	-4.63	-
P35	3.10	-1.93	5.0	P81	-	-4.31	-
P36	3.10	-2.12	5.2	P82	-	-5.99	-
P37	3.10	-1.94	5.0	P83	3.10	-4.17	7.3
P38	3.10	-1.76	4.9	P84	3.10	-2.78	5.9
P39	3.10	-2.08	5.2	P85	3.10	-2.24	5.3
P40	3.10	-2.03	5.1	P86	3.10	-2.67	5.8
P41	3.10	-1.72	4.8	P87	3.10	-2.13	5.2
P42	3.10	-1.54	4.6	A2	3.10	-0.61	3.7

Source: Study Team

3) Blockout at parapet

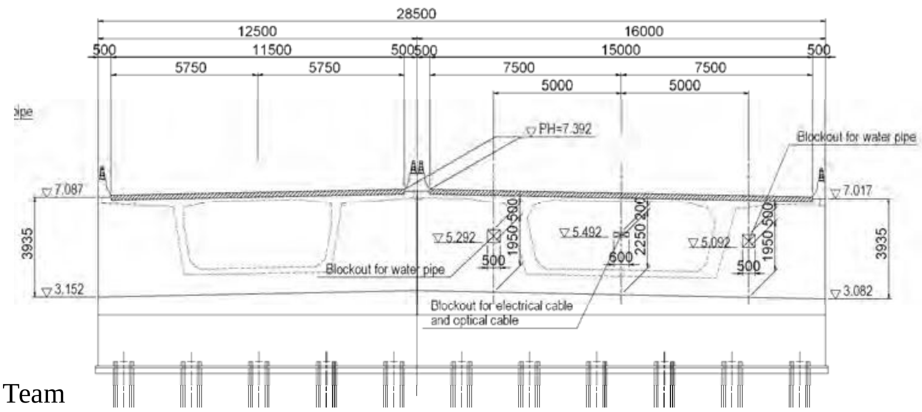
Detail of blockout at parapet for the Affixed articles.are as below

2-1. Detail of blockout



Source: Study Team

Figure 8.4.6-1 Blockout at A1 abutment



Source: Study Team

Figure 8.4.6-2 Blockout at A2 abutment

2-2. Affixed articles

a) Electrical cable

(Technical parameter from THE NORTHERN ELECTRIC CORPORATION)

External diameter	:	93mm
Cable weight	:	16.690kg/m
Number of cable	:	2nos

b) Optical cable

(Technical parameter from THE NORTHERN ELECTRIC CORPORATION)

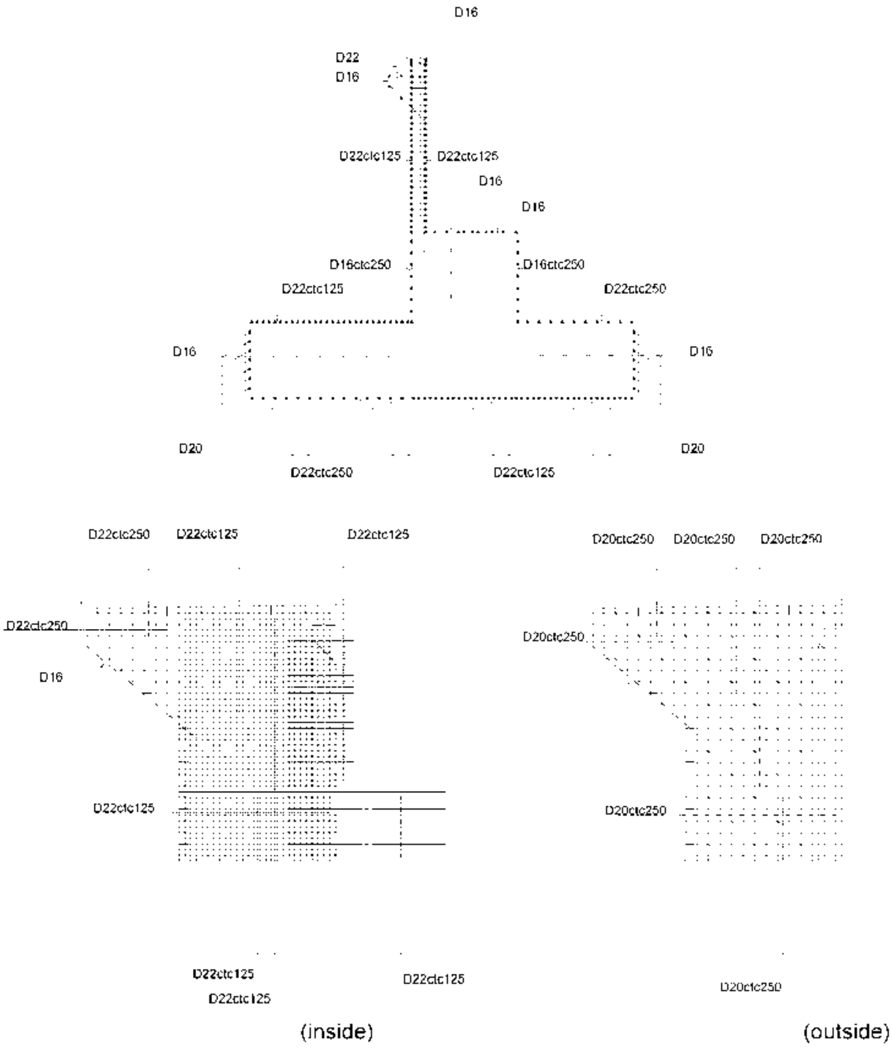
External diameter	:	13-14.2mm
Cable weight	:	125-145kg/m
Number of cable	:	1nos

c) Water pipe

Diameter	:	400mm
Number of cable	:	2nos

4) Arrangement of reinforcement

Arrangement of reinforcement at A1 abutment shows as below.(A2 abutment is same as A1)



Source: Study Team

Figure 8.4.6-3 Arrangement of Reinforcement at Abutment

(2) Pier

1) Material to be used

a) Concrete

Concrete : σck=28N/mm2

b) Reinforcement

Reinforcement :SD345

2) Reclamation plan

Refer to Section 8.1.4.1

3) Dimension of Substructure

Table 8.4.6-2 Dimension list of Substructure(1/2)

Span length of Superstructure	Number of Substructure	Total height of Pier	Column	Pile		Thickness of reclamation (From bottom of pile cap)	Thickness of consolidation layer (From bottom of pile cap)
			Dimensions of column	Type of pile	Diameter		
5@60.0 =300.0m	A1	8.0m	28.5m × 2.5m	Steel pipe pile	0.8m	3.8m	24.7m
	P1	6.0m	7.8m × 2.5m		1.1m	5.1m	23.8m
	P2	7.5m	4.5m × 2.5m			5.9m	30.7m
	P3	8.5m	4.5m × 2.5m			6.2m	25.9m
	P4	8.5m	4.5m × 2.5m			5.4m	23.4m
5@60.0 =300.0m	P5	8.5m	4.5m × 4.0m			4.8m	26.8m
	P6	8.5m	4.5m × 2.5m			4.0m	29.7m
	P7	9.0m	4.5m × 2.5m			3.9m	26.1m
	P8	9.5m	4.5m × 2.5m			3.9m	23.1m
	P9	10.0m	4.5m × 2.5m			4.0m	23.1m
51.5+4@60.0 =291.5m	P10	10.0m	4.5m × 4.0m			3.9m	23.9m
	P11	10.5m	4.5m × 2.5m			4.2m	23.9m
	P12	10.5m	4.5m × 2.5m			4.2m	28.9m
	P13	10.5m	4.5m × 2.5m			4.4m	28.0m
	P14	10.5m	4.5m × 2.5m			4.6m	26.2m
5@60.0 =300.0m	P15	10.0m	4.5m × 4.0m			4.4m	30.8m
	P16	10.0m	4.5m × 2.5m			4.4m	35.7m
	P17	10.0m	4.5m × 2.5m			4.6m	37.6m
	P18	10.0m	4.5m × 2.5m			4.8m	36.0m
	P19	9.5m	4.5m × 2.5m			4.5m	36.9m
5@60.0 =300.0m	P20	9.5m	4.5m × 4.0m			4.8m	35.8m
	P21	9.5m	4.5m × 2.5m			4.8m	34.4m
	P22	9.0m	4.5m × 2.5m			4.5m	29.9m
	P23	9.0m	4.5m × 2.5m			4.7m	27.6m
	P24	9.0m	4.5m × 2.5m			4.9m	26.7m
5@60.0 =300.0m	P25	8.5m	4.5m × 4.0m			4.7m	27.0m
	P26	8.5m	4.5m × 2.5m			4.7m	27.4m
	P27	8.5m	4.5m × 2.5m			4.9m	25.2m
	P28	8.5m	4.5m × 2.5m			5.1m	27.8m
	P29	8.5m	4.5m × 2.5m			5.3m	28.7m
5@60.0 =300.0m	P30	8.5m	4.5m × 4.0m			5.5m	32.4m
	P31	8.5m	4.5m × 2.5m			5.4m	23.7m
	P32	8.5m	4.5m × 2.5m			5.4m	25.7m
	P33	8.5m	4.5m × 2.5m			5.3m	20.9m
	P34	8.5m	4.5m × 2.5m			5.1m	22.8m
5@60.0=300.0m	P35	8.5m	4.5m × 4.0m			5.0m	34.1m
	P36	9.0m	4.5m × 2.5m			5.2m	27.8m
	P37	9.0m	4.5m × 2.5m			5.0m	29.1m
	P38	9.0m	4.5m × 2.5m			4.9m	24.1m
	P39	9.5m	4.5m × 2.5m			5.2m	22.6m
5@60.0 =300.0m	P40	9.5m	4.5m × 4.0m			5.1m	33.0m
	P41	9.5m	4.5m × 2.5m			4.8m	28.0m
	P42	9.5m	4.5m × 2.5m			4.6m	29.0m
	P43	10.0m	4.5m × 2.5m			5.0m	22.1m
	P44	10.0m	4.5m × 2.5m			4.8m	21.8m
4@60.0+58.36 =298.36m	P45	10.0m	4.5m × 4.0m			4.7m	23.3m
	P46	10.5m	4.5m × 2.5m			4.9m	25.6m
	P47	10.5m	4.5m × 2.5m			4.7m	34.8m
	P48	10.5m	4.5m × 2.5m			4.6m	28.0m
	P49	10.5m	4.5m × 2.5m			4.4m	28.1m
	P50	15.0m	4.5m × 2.5m			8.8m	22.6m

Source: Study Team

Table 8.4.6-3 Dimension list of Substructure(2/2)

Span of Superstructure	Number of Substructure	Column	Dimensions of column	Pile		Thickness of reclamation (From bottom of pile cap)	Thickness of consolidation layer (From bottom of pile cap)
				Type of pile	Diameter		
	P50	15.0m	4.5m × 4.0m			8.8m	22.6m
	P51	15.0m	4.5m × 2.5m			8.6m	22.9m
5@60.0 =300.0m	P52	15.5m	4.5m × 2.5m			9.0m	23.4m
	P53	15.5m	4.5m × 2.5m			9.0m	23.4m
	P54	15.0m	4.5m × 2.5m			8.6m	23.8m
	P55	15.0m	4.5m × 4.0m	Steel pipe pile	1.1m	8.8m	27.5m
	P56	15.0m	4.5m × 2.5m			8.8m	28.6m
52.98+3@60.0 +52.98=285.96m	P57	15.0m	4.5m × 2.5m			9.0m	20.5m
	P58	14.5m	4.5m × 2.5m			8.7m	11.9m
	P59	14.5m	4.5m × 2.5m			8.9m	13.6m
	P60	14.0m	4.5m × 4.0m			8.6m	22.0m
	P61	14.0m	4.5m × 2.5m				
52.98+3@60.0 +52.98=285.96m	P62	14.0m	4.5m × 2.5m				
	P63	14.0m	4.5m × 2.5m				
	P64	13.5m	4.5m × 2.5m				
	P65	13.5m	4.5m × 4.0m				
	P66	13.5m	4.5m × 2.5m				
52.98+3@60.0 +52.98=285.96m	P67	13.0m	4.5m × 2.5m				
	P68	13.0m	4.5m × 2.5m	Bored Pile	1.5m	-	-
	P69	13.0m	4.5m × 2.5m				
	P70	14.0m	4.5m × 4.0m				
	P71	15.0m	4.5m × 2.5m				
52.98+3@60.0 +52.98=285.96m	P72	16.5m	4.5m × 2.5m				
	P73	18.5m	4.5m × 3.5m				
	P74	20.0m	4.5m × 3.5m				
	P75	21.5m	4.5m × 4.0m				
	P76						
Main Bridge	P77						
	P78						
	P79	20.0m	4.5m × 4.0m				
	P80	19.0m	4.5m × 3.5m	Bored Pile	1.5m	-	-
54.8+3@60.0 +54.8=289.6m	P81	17.0m	4.5m × 3.5m				
	P82	17.0m	4.5m × 3.5m				
	P83	13.5m	4.5m × 3.5m			7.3m	7.3m
	P84	10.5m	4.5m × 4.0m			5.9m	5.9m
	P85	8.5m	4.5m × 3.5m	Steel pipe pile	1.1m	5.3m	5.3m
54.8+2@60.0 +54.8=229.6m	P86	7.5m	4.5m × 3.5m			5.8m	5.8m
	P87	6.0m	4.5m × 3.5m			5.2m	5.2m
	A2	8.0m	4.5m × 4.0m		0.8m	3.7m	3.7m

Source: Study Team

8.4.6.2 Grouping of Pier

Table 8.4.6-4 Grouping of Pier

Type No.	Total height of column	Dimensions of column	Type of pile	Pier number	Representative calculation Pier
Type1	6.0m	7.8×2.5	Steel pipe pile	P1	P1
Type2	6.0m	7.8×3.5		P87	P87
Type3	10.5m	4.5×2.5		P11.P12.P13.P14.P46.P47.P48.P49	P14
Type4	7.5m			P2	P2
Type5	8.5m			P3.P4.P6.P26.P27.P28.P29.P31.P32.P33.P34	P4,P29,P31
Type6	9.0m			P7.P22.P23.P24.P36.P37.P38	P36
Type7	9.5m			P8.P19.P39.P41.P42	P41
Type8	10.0m			P9.P16.P17.P18.P21.P43.P44	P9,P16,P21
Type9	14.5m			P58.P59	P59
Type10	15.0m			P51.P54.P56.P57	P54,P56
Type11	15.5m			P52.P53	P52
Type12	7.5m	4.5×3.5		P86	P86
Type13	8.5m			P85	P85
Type14	13.5m			P83	P83
Type15	8.5m	4.5×4.0		P5.P25.P30.P35	P5,P25,P30,P35
Type16	9.5m			P20.P40	P20,P40
Type17	10.0m			P15.P10.P45	P10,P15,P45
Type18	10.5m			P84	P84
Type19	14.0m			P60	P60
Type20	15.0m			P50.P55	P50,P55
Type21	13.0m	4.5×2.5	Bored Pile	P67.P68.P69	P69
Type22	13.5m			P64.P66	P66
Type23	14.0m			P61.P62.P63	P61
Type24	15.0m			P71	P71
Type25	16.5m	4.5×3.5		P72	P72
Type26	17.0m			P81.P82	P81,P82
Type27	18.5m			P73	P73
Type28	19.0m			P80	P80
Type29	20.0m			P74	P74
Type30	13.5m	4.5×4.0		P65	P65
Type31	14.0m			P70	P70
Type32	20.0m			P79	P79
Type33	21.5m			P75	P75

Source: Study Team

8.4.6.3 Reinforcement arrangement for each type of pier

Table 8.4.6-5 List of reinforcement for each type of Pier(1/2)

				Type1	Type2	Type3	Type4	Type5	Type6	Type7
Beam	Dimension	Thickness at joint		-	-	3.5	3.5	3.5	3.5	3.5
		Length of overhanging		-	-	1.65	1.65	1.65	1.65	1.65
		width		-	-	2.5	2.5	2.5	2.5	2.5
	Upper side			-	-	D32-12nos	D32-12nos	D32-12nos	D32-12nos	D32-12nos
	Lower side			-	-	D20-12nos	D20-12nos	D20-12nos	D20-12nos	D20-12nos
	Side			-	-	D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos
	Reinforcement for shear			-	-	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200
Column	Dimension	Plane		7.8×2.5	7.8×3.5	4.5×2.5	4.5×2.5	4.5×2.5	4.5×2.5	4.5×2.5
		Height		3.5	3.5	8.0	5.0	6.0	6.5	7.0
	Reinforcement for bending	Longitudinal		D16 ctc250	D16 ctc250	D22 ctc125	D16 ctc125	D16 ctc125	D16 ctc125	D20 ctc125
		Transverse		D16 ctc250	D16 ctc250	D22 ctc250	D16 ctc250	D16 ctc250	D16 ctc250	D20 ctc250
	Reinforcement for shear	Longitudinal		D16-10nos	D16-10nos	D16-10nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos
		Transverse		D16-4nos	D16-5nos	D16-4nos	D16-4nos	D16-4nos	D16-4nos	D16-4nos
Pile cap	Dimension	Plane		11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0
		Thickness		2.5	2.5	2.5	2.5	2.5	2.5	2.5
	Longitudinal	Upper side	1	D25ctc250	D25ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250
			2	-	-	-	-	-	-	-
		Lower side	1	D30ctc125	D30ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125
			2	-	-	-	-	-	-	-
		Reinforcement for shear		D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500
	Transverse	Upper side	1	D16ctc250	D16ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250
			2	-	-	-	-	-	-	-
		Lower side	1	D20ctc125	D20ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125
			2	-	-	-	-	-	-	-
		Reinforcement for shear		D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500

				Type8	Type9	Type10	Type11	Type12	Type13	Type14
Beam	Dimension	Thickness at joint		3.5	3.5	3.5	3.5	3.5	3.5	3.5
		Length of overhanging		1.65	1.65	1.65	1.65	1.65	1.65	1.65
		width		2.5	2.5	2.5	2.5	3.5	3.5	3.5
	Upper side			D32-12nos	D32-12nos	D32-12nos	D32-12nos	D32-18nos	D32-18nos	D32-18nos
	Lower side			D20-12nos	D20-12nos	D20-12nos	D20-12nos	D20-18nos	D20-18nos	D20-18nos
	Side			D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos
	Reinforcement for shear			D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200
Column	Dimension	Plane		4.5×2.5	4.5×2.5	4.5×2.5	4.5×2.5	4.5×3.5	4.5×3.5	4.5×3.5
		Height		7.5	12.0	12.5	13.0	5.0	6.0	11.0
	Reinforcement for bending	Longitudinal		D20 ctc125	D35 ctc125	D35 ctc125	D35 ctc125	D16 ctc125	D16 ctc125	D16 ctc125
		Transverse		D20 ctc250	D35 ctc250	D35 ctc250	D35 ctc250	D16 ctc250	D16 ctc250	D16 ctc250
	Reinforcement for shear	Longitudinal		D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos
		Transverse		D16-4nos	D16-4nos	D16-4nos	D16-4nos	D16-5nos	D16-5nos	D16-5nos
Pile cap	Dimension	Plane		11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0
		Thickness		2.5	2.5	2.5	2.5	2.5	2.5	2.5
	Longitudinal	Upper side	1	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250
			2	-	-	-	-	-	-	-
		Lower side	1	D35ctc125	D38ctc125	D38ctc125	D38ctc125	D35ctc125	D35ctc125	D38ctc125
			2	-	-	-	-	-	-	-
		Reinforcement for shear		D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500
	Transverse	Upper side	1	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250
			2	-	-	-	-	-	-	-
		Lower side	1	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125
			2	-	-	-	-	-	-	-
		Reinforcement for shear		D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500

				Type15	Type16	Type17	Type18	Type19	Type20	Type21
Beam	Dimension	Thickness at joint		3.5	3.5	3.5	3.5	3.5	3.5	3.5
		Length of overhanging		1.65	1.65	1.65	1.65	1.65	1.65	1.65
		width		4	4	4	4	4	4	2.5
	Upper side			D32-19nos	D32-19nos	D32-19nos	D32-19nos	D32-19nos	D32-19nos	D32-12nos
	Lower side			D20-19nos	D20-19nos	D20-19nos	D20-19nos	D20-19nos	D20-19nos	D20-12nos
	Side			D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos
	Reinforcement for shear			D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200
Column	Dimension	Plane		4.5×4.0	4.5×4.0	4.5×4.0	4.5×4.0	4.5×4.0	4.5×4.0	4.5×2.5
		Height		6.0	7.0	7.5	8.0	11.5	12.5	10.5
	Reinforcement for bending	Longitudinal		D16 ctc125	D16 ctc125	D16 ctc125	D16 ctc125	D16 ctc125	D16 ctc125	D32 ctc125
		Transverse		D16 ctc250	D16 ctc250	D16 ctc250	D16 ctc250	D16 ctc250	D16 ctc250	D32 ctc250
	Reinforcement for shear	Longitudinal		D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos
		Transverse		D16-5nos	D16-5nos	D16-5nos	D16-5nos	D16-5nos	D16-5nos	D16-4nos
Pile cap	Dimension	Plane		11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0	11.0×11.0	10.5×10.5
		Thickness		2.5	2.5	2.5	2.5	2.5	2.5	2.5
	Longitudinal	Upper side	1	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D28ctc125
			2	-	-	-	-	-	-	-
		Lower side	1	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D38ctc125	D38ctc125	D32ctc125
			2	-	-	-	-	-	-	D32ctc125
		Reinforcement for shear		D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D25-21nos-ctc500
	Transverse	Upper side	1	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D25ctc125
			2	-	-	-	-	-	-	-
		Lower side	1	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D32ctc125
			2	-	-	-	-	-	-	D32ctc250
		Reinforcement for shear		D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D25-21nos-ctc500

Source: Study Team

Table 8.4.6-6 List of reinforcement for each type of Pier(1/2)

				Type22	Type23	Type24	Type25	Type26	Type27	Type28
Beam	Dimension	Thickness at joint		3.5	3.5	3.5	3.5	3.5	3.5	3.5
		Length of overhanging		1.65	1.65	1.65	1.65	1.65	1.65	1.65
		width		2.5	2.5	2.5	3.5	3.5	3.5	3.5
	Upper side			D32-12nos	D32-12nos	D32-12nos	D32-18nos	D32-18nos	D32-18nos	D32-19nos
	Lower side			D20-12nos	D20-12nos	D20-12nos	D20-18nos	D20-18nos	D20-18nos	D20-19nos
	Side			D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos
	Reinforcement for shear			D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200
Column	Dimension	Plane		4.5×2.5	4.5×2.5	4.5×2.5	4.5×3.5	4.5×3.5	4.5×3.5	4.5×3.5
		Height		11.0	11.5	12.5	14.0	14.5	16.0	16.5
	Reinforcement for bending	Longitudinal		D32 ctc125	D35 ctc125	D35 ctc125	D25 ctc125	D25 ctc125	D32 ctc125	D32 ctc125
		Transverse		D32 ctc250	D35 ctc250	D35 ctc250	D25 ctc250	D25 ctc250	D32 ctc125	D32 ctc125
	Reinforcement for shear	Longitudinal		D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos
		Transverse		D16-4nos	D16-4nos	D16-4nos	D16-5nos	D16-5nos	D16-5nos	D16-5nos
Pile cap	Dimension	Plane		10.5×10.5	10.5×10.5	10.5×10.5	10.5×14.25	10.5×14.25	10.5×14.25	10.5×14.25
		Thickness		2.5	2.5	2.5	2.5	2.5	2.5	2.5
	Longitudinal	Upper side	1	D28ctc125	D28ctc125	D28ctc125	D32ctc125	D32ctc125	D32ctc125	D32ctc125
			2	-	-	-	-	-	-	-
		Lower side	1	D32ctc125	D32ctc125	D32ctc125	D38ctc125	D38ctc125	D38ctc125	D38ctc125
			2	D32ctc125	D32ctc125	D32ctc125	D38ctc125	D38ctc125	D38ctc125	D38ctc125
		Reinforcement for shear		D25-21nos-ctc500	D25-21nos-ctc500	D25-21nos-ctc500	D25-10nos-ctc250	D25-10nos-ctc250	D25-10nos-ctc250	D25-10nos-ctc250
	Transverse	Upper side	1	D25ctc125	D25ctc125	D25ctc125	D32ctc125	D32ctc125	D32ctc125	D32ctc125
			2	-	-	-	-	-	-	-
		Lower side	1	D32ctc125	D32ctc125	D32ctc125	D32ctc125	D32ctc125	D32ctc125	D32ctc125
			2	D32ctc250	D32ctc250	D32ctc250	D32ctc125	D32ctc125	D32ctc125	D32ctc125
		Reinforcement for shear		D25-21nos-ctc500	D25-21nos-ctc500	D25-21nos-ctc500	D25-14nos-ctc500	D25-14nos-ctc500	D25-14nos-ctc500	D25-14nos-ctc500

				Type29	Type30	Type31	Type32	Type33
Beam	Dimension	Thickness at joint		3.5	3.5	3.5	3.5	3.5
		Length of overhanging		1.65	1.65	1.65	1.65	1.65
		width		3.5	4	4	4	4
	Upper side			D32-19nos	D32-19nos	D32-19nos	D32-19nos	D32-19nos
	Lower side			D20-19nos	D20-19nos	D20-19nos	D20-19nos	D20-19nos
	Side			D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos
	Reinforcement for shear			D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200
Column	Dimension	Plane		4.5×3.5	4.5×4.0	4.5×4.0	4.5×4.0	4.5×4.0
		Height		17.5	11.0	11.5	17.5	19.0
	Reinforcement for bending	Longitudinal		D32 ctc125	D16 ctc125	D16 ctc125	D32 ctc125	D32 ctc125
		Transverse		D32 ctc125	D16 ctc125	D16 ctc125	D32 ctc125	D32 ctc125
	Reinforcement for shear	Longitudinal		D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos
		Transverse		D16-5nos	D16-5nos	D16-5nos	D16-5nos	D16-5nos
Pile cap	Dimension	Plane		10.5×14.25	10.5×10.5	10.5×10.5	10.5×14.25	10.5×14.25
		Thickness		2.5	2.5	2.5	2.5	2.5
	Longitudinal	Upper side	1	D32ctc125	D28ctc125	D28ctc125	D32ctc125	D32ctc125
			2	-	-	-	-	-
		Lower side	1	D38ctc125	D32ctc125	D32ctc125	D38ctc125	D38ctc125
			2	D38ctc125	D32ctc125	D32ctc125	D38ctc125	D38ctc125
		Reinforcement for shear		D25-10nos-ctc250	D25-21nos-ctc500	D25-21nos-ctc500	D25-10nos-ctc250	D25-10nos-ctc250
	Transverse	Upper side	1	D32ctc125	D25ctc125	D25ctc125	D32ctc125	D32ctc125
			2	-	-	-	-	-
		Lower side	1	D32ctc125	D32ctc125	D32ctc125	D32ctc125	D32ctc125
			2	D32ctc125	D32ctc250	D32ctc250	D32ctc125	D32ctc125
		Reinforcement for shear		D25-14nos-ctc500	D25-21nos-ctc500	D25-21nos-ctc500	D25-14nos-ctc500	D25-14nos-ctc500

Source: Study Team

8.4.6.4 Design of Foundation of Approach Bridge

(1) Steel pipe pile (A1～P60,P83～A2)

1) Material to be used

1-1. Steel Pipe Pile

a) Properties and Stress Limit of Steel Pipe

Table 8.4.6-7 Properties and Stress Limit and used Steel Pipe

Type	Yield Strength fy (Mpa)	Tensile Strength fu (Mpa)	Modulus of Elasticity (Mpa)	used
Grade SKK400	235	400	200000	○
Grade SKK490	315	490	200000	

Source: Study Team

b) Thickness of Steel pipe pile

Table 8.4.6-8 Range of thickness and used thickness

Diameter(mm)	thickness(mm)	Used thickness(mm)
400	9～12	
500	9～14	
600～800	9～16	12
900～1100	12～19	12
1200～1400	14～22	
1500～1600	16～25	
1800～2000	19～25	

Source: Study Team

c) Design of Estimated Corrosion Thicknesses

Table 8.4.6-9 Design of Estimated Corrosion Thicknesses

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

Source: Study Team

1-2. Fill Concrete for pile head

Concrete : σ ck=28N/mm2

1-3. Reinforcement

Reinforcement : SD345

2) Site condition

2-1. Soil Condition

Refer to Section 8.1.4.1

2-2. Layer to take account of downdrag

3) Result of Steel pipe pile

a) Type of Steel pipe

Steel pipe piles classified as follows;

Table 8.4.6-10 Type of Steel pipe pile

TYPE	Pier	Diameter (m)	Length of pile (m)	Range of SLC coating (from top of the pile) (m)	thickness (mm)
Type1-1	P58	1100	37.0	12.0	12.0
Type1-2	P57			21.0	
Type1-3	P3			26.0	
Type2	P5	1100	38.0	27.0	12.0
Type3-1	P59	1100	39.0	14.0	12.0
Type3-2	P84			22.0	
Type3-3	P4			23.0	
Type3-4	P85			27.0	
Type3-5	P6			30.0	
Type3-6	P2			31.0	
Type4-1	P60	1100	40.0	22.0	12.0
	P83			22.0	
Type4-2	P87			25.0	
Type6-5	P50	1100	42.0	23.0	12.0
	P51			23.0	
	P52			23.0	
Type6-1	P31			24.0	
	P1			24.0	
	P54			24.0	
	P38			24.0	
Type6-2	P46			26.0	
Type6-3	P36			28.0	
Type6-4	P35			34.0	
Type7-1	P44	1100	43.0	22.0	12.0
	P43			22.0	
Type7-2	P39			23.0	
	P34			23.0	
	P45			23.0	
Type7-3	P24			27.0	
Type7-4	P55			28.0	
	P41			28.0	
Type7-5	P56			29.0	
	P42			29.0	
Type7-6	P40			33.0	
Type7-7	P20			36.0	
Type8-1	P11	1100	44.0	24.0	12.0
Type8-2	P27			25.0	
Type8-3	P23			28.0	
Type8-4	P29			29.0	
	P37			29.0	
Type8-6	P21			34.0	
Type8-7	P18			36.0	
Type9-1	P9	1100	45.0	23.0	12.0
	P8			23.0	
	P53			23.0	
Type9-2	P10			24.0	
Type9-3	P32			26.0	
	P14			26.0	
Type9-4	P25			27.0	
Type9-5	P28			28.0	
Type9-7	P47			35.0	
Type10-1	P33	1100	46.0	21.0	12.0
Type10-2	P7			26.0	
Type10-3	P26			27.0	
Type10-4	P13			28.0	
	P48			28.0	
	P49			28.0	
Type10-5	P12			29.0	
Type10-8	P22			30.0	
Type10-9	P30			32.0	
Type10-6	P19			37.0	
Type10-7	P17			38.0	
Type11-1	P15	1100	47.0	31.0	12.0
Type11-2	P16			36.0	
Type12-1	P86	1100	49.0	24.0	12.0
Type13-1	A1	800	36.0	25.0	12.0
Type14-1	A2	800	41.0	25.0	12.0

Source: Study Team

b) Length of Steel pipe pile

Table 8.4.6-11 List of Steel pipe pile(1/2)

Number of substructure	A1	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12	P13	P14	P15	P16	P17	P18
Diameter of pile (m)	0.8	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Thikness of pile (mm)	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12
Length of pile (m)	36.0	42.0	40.0	37.0	39.0	38.0	39.0	46.0	45.0	45.0	45.0	44.0	46.0	46.0	45.0	47.0	47.0	46.0	44.0
Thickness of consolidation layer (From bottom of pile cap)	24.7	23.8	30.7	25.9	23.4	26.8	29.7	26.1	23.1	23.1	23.9	23.9	28.9	28.0	26.2	30.8	35.7	37.6	36.0
Range of SL pile (m)	25.0	24.0	31.0	26.0	23.0	27.0	30.0	26.0	23.0	23.0	24.0	24.0	29.0	28.0	26.0	31.0	36.0	38.0	36.0
Number of pile (nos)	44	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16
Top of the pile	-0.53	-1.84	-2.62	-2.90	-2.18	-1.60	-0.74	-0.62	-0.63	-0.77	-0.67	-0.96	-0.99	-1.15	-1.33	-1.13	-1.19	-1.37	-1.55
Bottom of pilecap	-0.68	-1.99	-2.77	-3.05	-2.33	-1.75	-0.89	-0.77	-0.78	-0.92	-0.82	-1.11	-1.14	-1.30	-1.48	-1.28	-1.34	-1.52	-1.70
Bottom of pile	-36.53	-43.84	-42.62	-39.90	-41.18	-39.60	-39.74	-46.62	-45.63	-45.77	-45.67	-44.96	-46.99	-47.15	-46.33	-48.13	-48.19	-47.37	-45.55
Bor.No	BP-1	BP-2	BP-3	BP-4	BP-5	BP-6	BP-7	BP-8	BP-9	BP-10	BP-11	BP-12	BP-13	BP-14	BP-15	BP-16	BP-17	BP-18	BP-19
Name of bearing layer	10B	12B	10B	10B	10B	10B	10B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B
Depth of bearing layer (m)	-29.0	-40.0	-34.0	-29.0	-30.0	-34.0	-31.0	-38.0	-37.0	-41.8	-42.0	-41.9	-43.2	-43.3	-43.2	-45.1	-44.7	-44.1	-42.4
Embeded length into bearing layer (m)	7.5	3.8	8.6	10.9	11.2	5.6	8.7	8.6	8.6	4.0	3.7	3.1	3.8	3.8	3.1	3.0	3.5	3.2	3.1
Determination factor of pile length and number (1)	b,c,d	a,b	b	b	b	b	b	b	b	a,b	a,b	a,b	a,b	a,b	a,b	a,b	a,b	a,b	a,b

Number of substructure	P19	P20	P21	P22	P23	P24	P25	P26	P27	P28	P29	P30	P31	P32	P33	P34	P35	P36	P37
Diameter of pile (m)	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Thikness of pile (mm)	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12
Length of pile (m)	46.0	43.0	44.0	46.0	44.0	43.0	45.0	46.0	44.0	45.0	44.0	46.0	42.0	45.0	46.0	43.0	42.0	42.0	44.0
Thickness of consolidation layer (From bottom of pile cap)	36.9	35.8	34.4	29.9	27.6	26.7	27.0	27.4	25.2	27.8	28.7	32.4	23.7	25.7	20.9	22.8	34.1	27.8	29.1
Range of SL pile (m)	37.0	36.0	34.0	30.0	28.0	27.0	27.0	27.0	25.0	28.0	29.0	32.0	24.0	26.0	21.0	23.0	34.0	28.0	29.0
Number of pile (nos)	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16
Top of the pile	-1.23	-1.53	-1.59	-1.27	-1.45	-1.63	-1.43	-1.49	-1.67	-1.85	-2.02	-2.25	-2.15	-2.11	-2.00	-1.83	-1.78	-1.97	-1.79
Bottom of pilecap	-1.38	-1.68	-1.74	-1.42	-1.60	-1.78	-1.58	-1.64	-1.82	-2.00	-2.17	-2.40	-2.30	-2.26	-2.15	-1.98	-1.93	-2.12	-1.94
Bottom of pile	-47.23	-44.53	-45.59	-47.27	-45.45	-44.63	-46.43	-47.49	-45.67	-46.85	-46.02	-48.25	-44.15	-47.11	-48.00	-44.83	-43.78	-43.97	-45.79
Bor.No	BP-20	BP-21	BP-22	BP-23	BP-24	BP-25	BP-26	BP-27	BP-28	BP-29	BP-30	BP-31	BP-32	BP-33	BP-34	BP-35	BP-36	BP-37	BP-38
Name of bearing layer	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B
Depth of bearing layer (m)	-43.3	-41.5	-42.2	-43.0	-41.9	-40.9	-42.5	-43.6	-42.1	-43.6	-42.1	-42.8	-40.4	-43.4	-44.2	-40.9	-40.3	-40.3	-41.9
Embeded length into bearing layer (m)	3.9	3.0	3.4	4.3	3.5	3.7	4.0	3.9	3.6	3.3	3.9	5.4	3.8	3.7	3.8	3.9	3.4	3.6	3.9
Determination factor of pile length and number (1)	a,b	a,b	a,b	b	a,b	a,b	b	a,b	a,b	a,b	a,b	b	a,b	a,b	a,b	a,b	a,b	a,b	a,b

Source: Study Team

Table 8.4.6-12 List of Steel pipe pile(2/2)

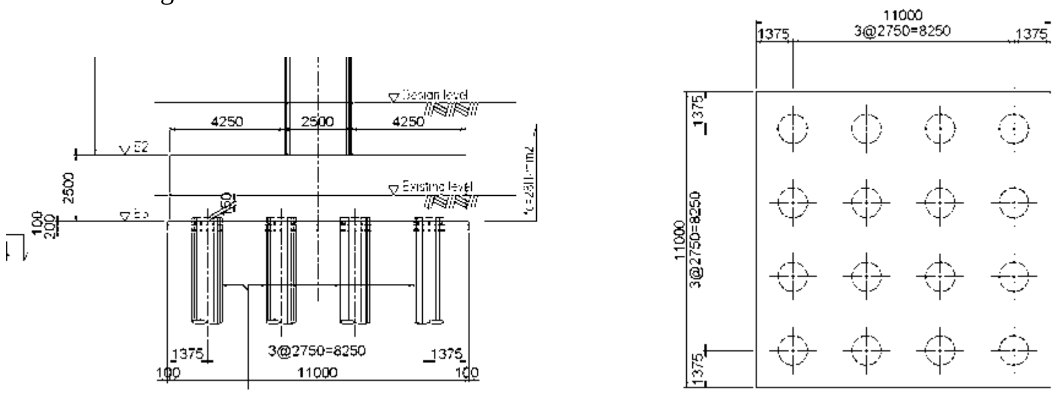
Number of substructure	P38	P39	P40	P41	P42	P43	P44	P45	P46	P47	P48	P49	P50	P51	P52	P53	P54	P55	P56
Diameter of pile (m)	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Thikness of pile (mm)	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12
Length of pile (m)	42.0	43.0	43.0	43.0	43.0	43.0	43.0	43.0	42.0	45.0	46.0	46.0	42.0	42.0	42.0	45.0	42.0	43.0	43.0
Thickness of consolidation layer (From bottom of pile cap)	24.1	22.6	33.0	28.0	29.0	22.1	21.8	23.3	25.6	34.8	28.0	28.1	22.6	22.9	23.4	23.4	23.8	27.5	28.6
Range of SL pile (m)	24.0	23.0	33.0	28.0	29.0	22.0	22.0	23.0	26.0	35.0	28.0	28.0	23.0	23.0	23.0	23.0	24.0	28.0	29.0
Number of pile (nos)	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16	16
Top of the pile	-1.61	-1.93	-1.88	-1.57	-1.39	-1.71	-1.53	-1.48	-1.67	-1.49	-1.31	-1.13	-5.59	-5.35	-5.79	-5.78	-5.33	-5.55	-5.56
Bottom of pilecap	-1.76	-2.08	-2.03	-1.72	-1.54	-1.86	-1.68	-1.63	-1.82	-1.64	-1.46	-1.28	-5.74	-5.50	-5.94	-5.93	-5.48	-5.70	-5.71
Bottom of pile	-43.61	-44.93	-44.88	-44.57	-44.39	-44.71	-44.53	-44.48	-43.67	-46.49	-47.31	-47.13	-47.59	-47.35	-47.79	-50.78	-47.33	-48.55	-48.56
Bor.No	BP-39	BP-40	BP-41	BP-42	BP-43	BP-44	BP-45	BP-46	BP-47	BP-48	BP-49	BP-50	BP-51	BP-52	BP-53	BP-54	BP-55	BP-56	BP-57
Name of bearing layer	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12A	12A	12B	12B	12B
Depth of bearing layer (m)	-40.5	-41.1	-41.8	-41.3	-40.4	-41.0	-41.0	-40.8	-39.7	-43.5	-43.7	-43.4	-43.3	-43.0	-42.7	-47.0	-43.8	-43.9	-42.9
Embeded length into bearing layer (m)	3.1	3.9	3.1	3.3	4.0	3.7	3.5	3.7	3.9	3.0	3.6	3.8	4.3	4.4	5.1	3.8	3.5	4.6	5.7
Determination factor of pile length and number (1)	a,b	a,b	a,b	a,b	b	a,b	a,b	a,b	a,b	a,b	a,b	a,b	b	b	b	a,b	a,b	a,b	a,b

Number of substructure	P57	P58	P59	P60	P83	P84	P85	P86	P87	A2
Diameter of pile (m)	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	0.8
Thikness of pile (mm)	12	12	12	12	12	12	12	12	12	12
Length of pile (m)	37.0	37.0	39.0	40.0	40.0	39.0	39.0	49.0	40.0	41.0
Thickness of consolidation layer (From bottom of pile cap)	20.5	11.9	13.6	22.0	22.4	22.1	26.8	24.2	25.0	28.3
Range of SL pile (m)	21.0	12.0	14.0	22.0	22.0	22.0	27.0	24.0	25.0	28.0
Number of pile (nos)	16	16	16	16	16	16	16	16	16	44
Top of the pile	-5.74	-5.42	-5.60	-5.39	-4.02	-2.63	-2.09	-2.52	-1.98	-0.46
Bottom of pilecap	-5.89	-5.57	-5.75	-5.54	-4.17	-2.78	-2.24	-2.67	-2.13	-0.61
Bottom of pile	-42.74	-42.42	-44.60	-45.39	-44.02	-41.63	-41.09	-51.52	-41.98	-41.46
Bor.No	BP-58	BP-59	BP-60	BP-61	BP-87	BP-88	BP-89	BP-90	BP-91	BP-92
Name of bearing layer	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B
Depth of bearing layer (m)	-39.4	-39.4	-41.2	-40.5	-39.5	-37.7	-38.0	-47.8	-38.3	-37.6
Embeded length into bearing layer (m)	3.3	3.0	3.4	4.9	4.5	3.9	3.1	3.7	3.7	3.9
Determination factor of pile length and number (1)	a,b	a,b	a,b	b	b	a,b	a,b	a,b	a,b	b,c,d

Source: Study Team

c) Pile arrangement

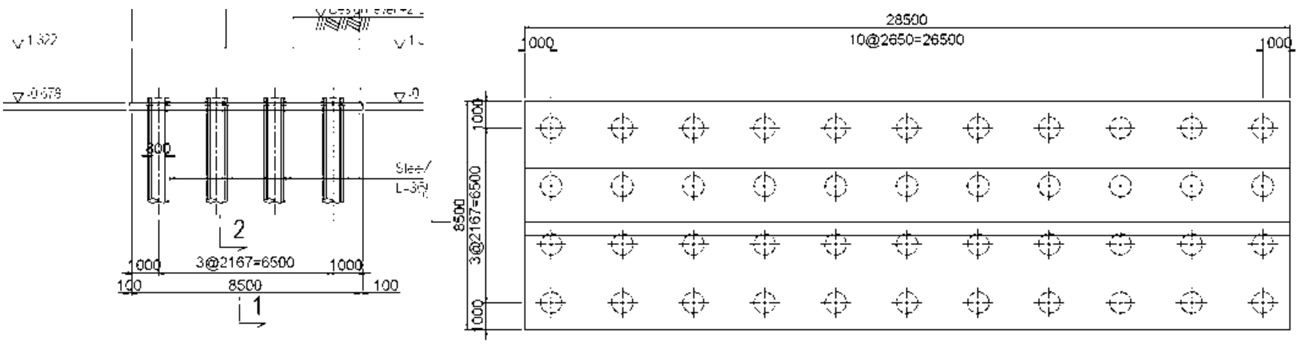
1. Pile arrangement for Pier



Source: Study Team

Figure 8.4.6-4 Pile arrangement for Pier

2. Pile arrangement for Abutment

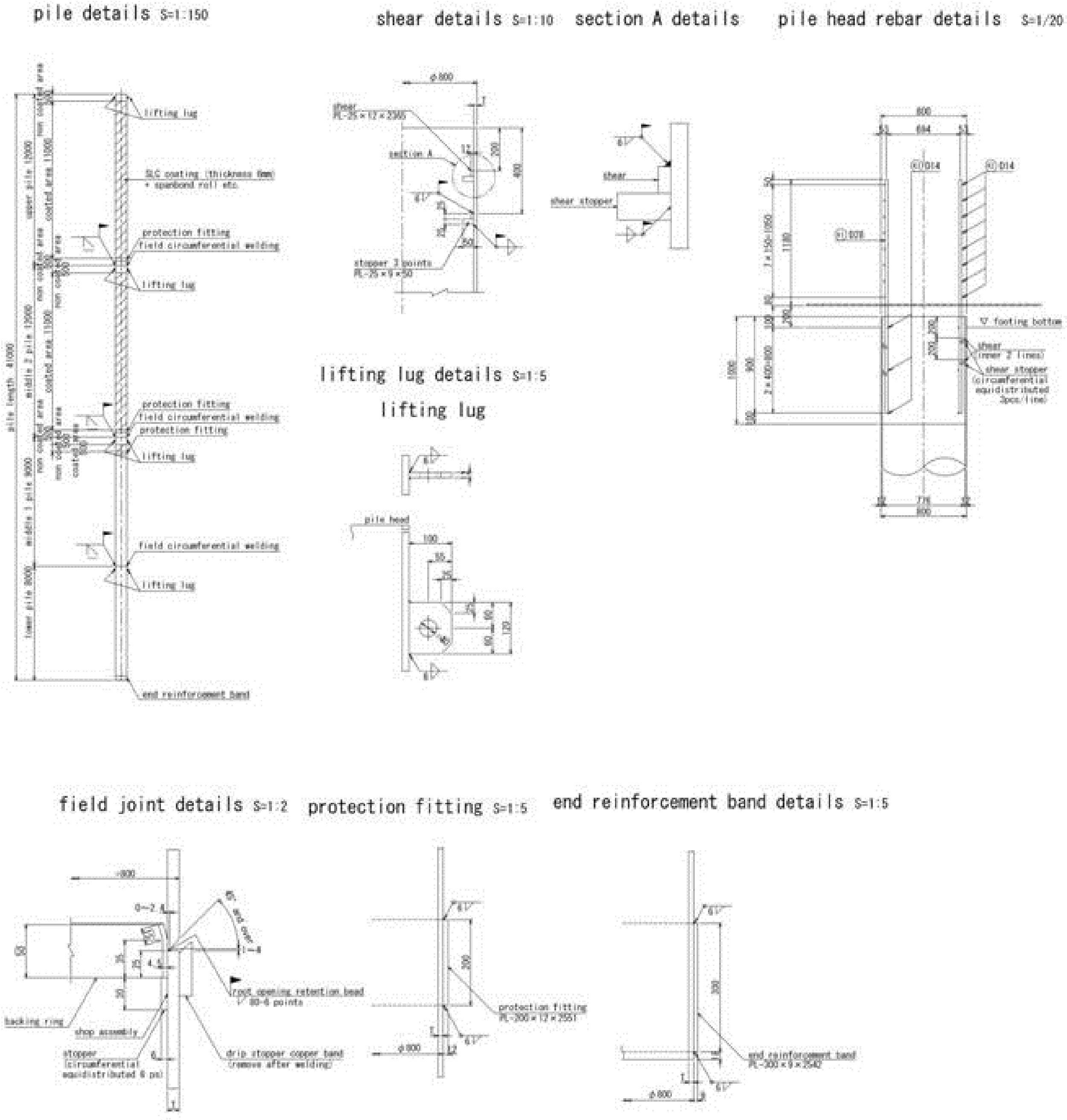


Source: Study Team

Figure 8.4.6-5 Pile arrangement for Abutment

d) Detail of Steel pipe pile

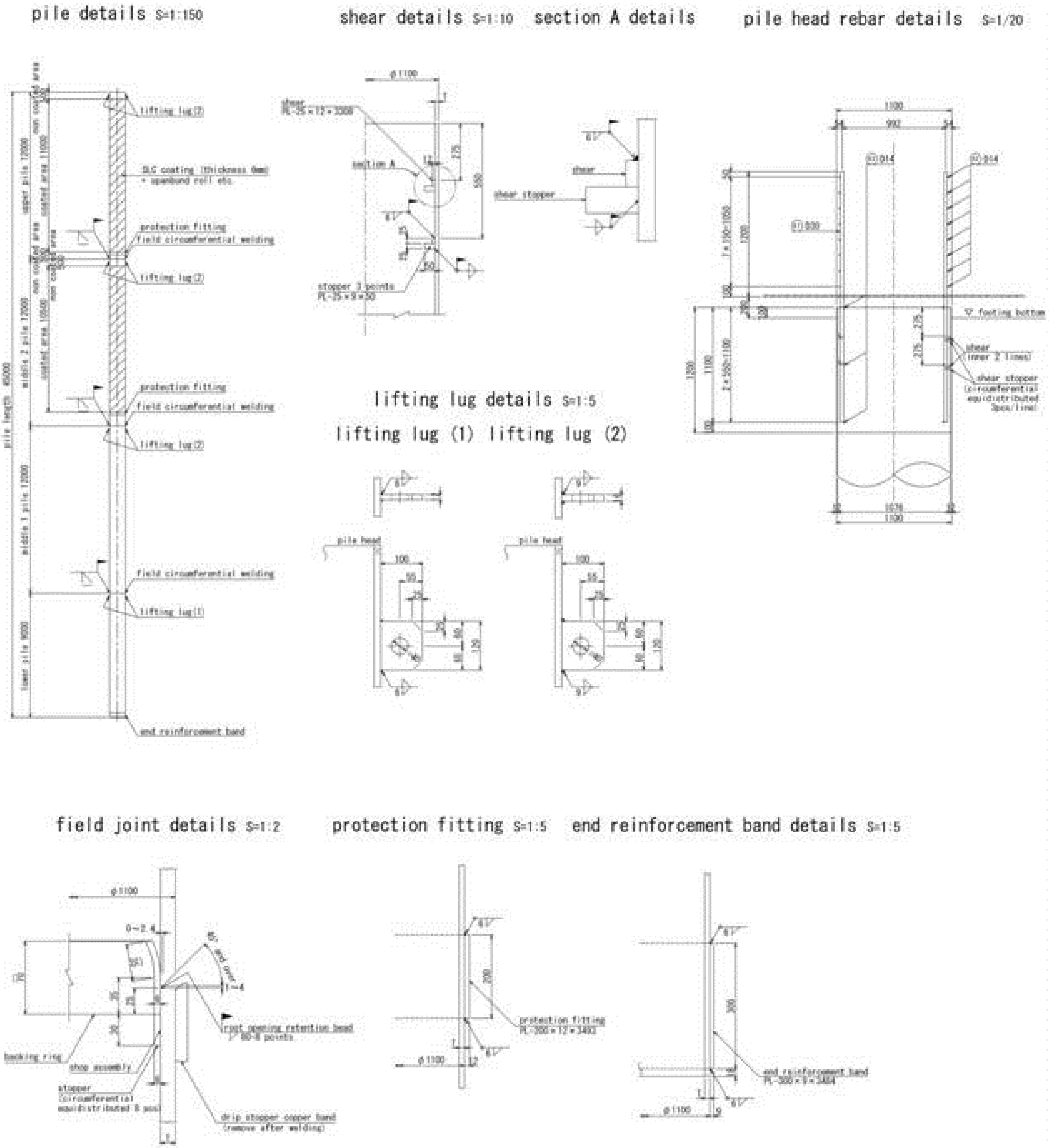
1. Steel pipe pile D=800mm (A2 abutment)



Source: Study Team

Figure 8.4.6-6 Steel Pipe Pile D=800mm (A2 abutment)

2. Steel pipe pile D=1100mm (P8,P9,P53)



Source: Study Team

Figure 8.4.6-7 Steel Pipe Pile D=1100mm (P8,P9,P53)

(2) Bored Pile (P61～P75,P79～P82)

1) Material to be used

1-1. Concrete

Concrete : σ ck=30N/mm2

1-2. Reinforcement

Reinforcement : SD345

2) Soil Condition

Refer to Section 8.1.4.1

3) Scour Depth

Refer to Section 8.1.5.3

4) Result of Bored pile

a) Type of Bored pile

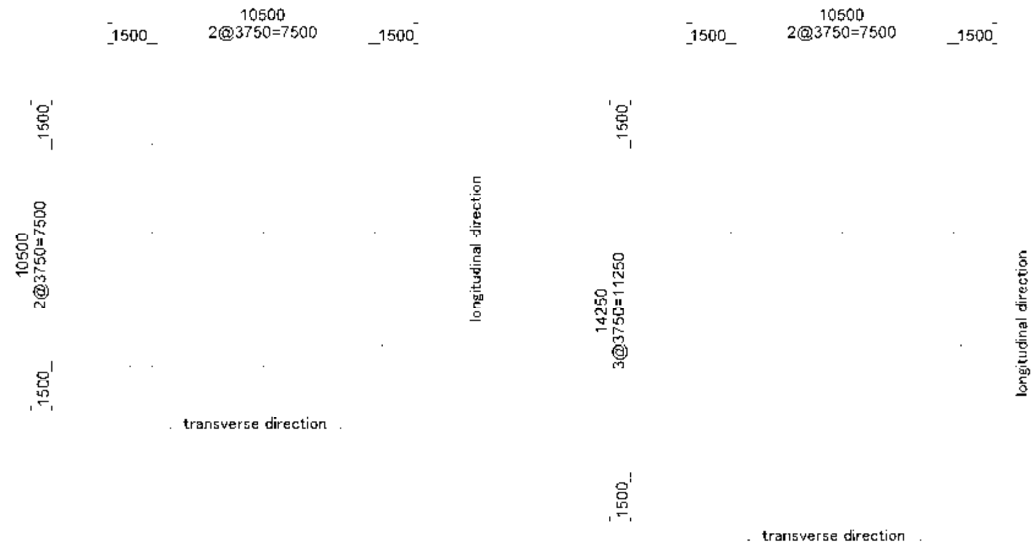
Bored piles classified as follows;

Table 8.4.6-13 Type of Bored pile

TYPE	Pier	Length of pile (m)	Number of pile (nos)
Type1	P63,P67,P68,P69	38.0	9
Type2	P61,P62,P64,P65,P66,P71	39.0	9
Type3	P70	40.0	9
Type4	P82	38.0	12
Type5	P72,P73,P74,P79,P80,P81	39.0	12
Type6	P75	40.0	12

Source: Study Team

b) Pile arrangement



Pile arrangement for Type1,2,3 Pile arrangement for Type4,5,6

Source: Study Team

Figure 8.4.6-8 Pile Arrangement

c) Length of Bored pile

Table 8.4.6-14 List of Bored pile

Number of substructure	P61	P62	P63	P64	P65	P66	P67	P68	P69	P70	P71	P72	P73	P74	P75	P79	P80	P81	P82
Diameter of pile (m)	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Length of pile (m)	39.0	39.0	38.0	39.0	39.0	39.0	38.0	38.0	38.0	40.0	39.0	39.0	39.0	39.0	40.0	39.0	39.0	39.0	38.0
Number of pile (nos)	9	9	9	9	9	9	9	9	9	9	9	12	12	12	12	12	12	12	12
Top of the pile	-5.42	-5.60	-5.78	-5.46	-5.75	-5.78	-5.46	-5.61	-5.33	-5.77	-5.49	-5.33	-5.65	-5.47	-5.63	-4.03	-4.48	-4.16	-5.84
Bottom of pilecap	-5.57	-5.75	-5.93	-5.61	-5.90	-5.93	-5.61	-5.76	-5.48	-5.92	-5.64	-5.48	-5.80	-5.62	-5.78	-4.18	-4.63	-4.31	-5.99
Bottom of pile	-44.42	-44.60	-43.78	-44.46	-44.75	-44.78	-43.46	-43.61	-43.33	-45.77	-44.49	-44.33	-44.65	-44.47	-45.63	-43.03	-43.48	-43.16	-43.84
Bor.No	BP-62	BP-63	BP-64	BP-65	BP-66	BP-67	BP-68	BP-69	BP-70	BP-71	BP-72	BP-73	BP-74	BP-75	BP-76	BP-83	BP-84	BP-85	BP-86
Name of bearing layer	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B	12B
Depth of bearing layer (m)	-41.0	-40.4	-40.0	-40.4	-42.2	-42.9	-40.0	-39.6	-40.3	-42.2	-40.7	-41.1	-41.5	-40.3	-41.0	-38.6	-40.5	-40.2	-39.5
Embeded length into bearing layer (m)	3.4	4.2	3.7	4.0	2.5	1.9	3.5	4.0	3.0	3.5	3.8	3.3	3.2	4.1	4.6	4.4	3.0	3.0	4.4
Determination factor of pile length and number (1)	b	b	b	b	b	b	b	b	b	b	b	b,c	b,c	b,c	b,c	b,c	b,c	b,c	b,c,d

※Note

(1)Determination factor

a : Minimum penetration length into bearing layer（Steel pipe pile:3.0m,Bored pile:1.5m)

b : Bearing resistance

c : Horizontal displacement

d : Lateral movement identifying index

Source: Study Team