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

- Alternative-2: Moving Scaffolding System with

22 spans@50.0m x 2 erection girders + 23 spans @ 50m x 2 erection girders

= 4,500.0 m

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;

- \cdot 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.

Source : Study Team

33 75 4 × 法庭析 Alte xecuted as same as usual cast-in-situ concrete girder. After placing concrete, curing and pre-stressing work shall be arrangement of reinforcing bars and PC tendons shall be han Due to coupling PC tendons, dimension of web becomes thicker than Alternative 1and concrete volume is huge. (ost for bearings, sub-structure and foundation is bigger than Alternative 1. Construction period is also longer Man Grader-WD1 6/25(000×52/83) = 22,000/00 Environment of the March WD1 28/50(00/000×43/0) = 422,000/00 Environment of the March WD1 28/50(00/000×43/0) = 422,000/00 Environment WD1 57/55(00/00×90) = 413,978/58 The number of necessary items for periodical maintenance, such as bearings and expansion joints, is larger This alternative is inferior in construction period and quality control comparing to Alternative 1 AA NG 2 4 500 0m = #100 Alternative-2: Movable Scaffolding System ASS is New Technology in Vietnam (there is one experience in Thanh Tri Bridge) Weight of Erection Girder: 950 ton (refer to Second Tomei Expressway in Japan) The quality control for girder fabrication is similar to cast-in-situ concrete girder Vot applicable for longitudinal slope more than 2% (KM 8+700 - Km 8+950) lacing concrete is executed on the form work hanged by erection girder. CUP Jan construction period (18.9 months) is longer than Alternative 1 P90 (22@50m x 2 After assembling erection girder including steel form work, earings, Sub-structure and Foundation: A1 ~ P90: 91 nos rider and fabrication of erection girder requires 1 year. irder concrete volume: Vc = 52,853m3 (4,500m) Vo major difference in Environmental Impact ominal strength of concrete: $\sigma ck = 40MPa$ ecessary Number of Erection Girder: 4 20 days for 1 span x 23 = 460 daysthorter Span Length (Maximum 50m) 60 days for assembling 45 days for renoval 8% (Preliminary Estimate) ordinary appearance nended. ecuted. reco ess 10 95 10 40 10 12 4 v 4 After assembling erection girder, pre-cast segment shall be transported under the erection girder and lifted one he number of necessary items for periodical maintenance, such as bearings and expansion joints, is smaller segments. one. Then, after completion of lifting one span whole segments, pre-stressing force is given by external tilizing precast segments, the quality is well controlled in the segment fabrication, which is preferable for This alternative is superior in construction period and quality control owing to utilization of precast 753,876,368 250,000,000 273,712,500 47,529,675 1,313,961,600 Due to using external cable, concrete volume is not so huge compared with Alternative 2. VININ MANANAN Veight of Erection Girder : 811 ton (refer to Second Tomei Expressway in Japan) minal strength of concrete: $\sigma ck = 50 MPa$, better quality for strength and anti-cAlternative-1: Span-By-Span Method ost and construction period can be saved compared with Alternative 2. M an Girder: VND 16.144,000 x 46,697 = requires pre-cast segment yard for fabrication and stock of segment The construction period (16.0 months) is shorter than Alternative 2. earings, Sub-structure and Foundations: A1 ~ P75 : 76 nos 3S is New Technology in Vietnam (first time in Vietnam) urder and fabrication of erection girder requires 1 year. h irder concrete volume: Vc = 46,697m3 (4,500m) Fabrication y ard: VND Erection Girder: VND 101,375,000,000 x 3 x 0.9 = Pier: VND 633,729,000 x 75 = Foundation: VND 17,519,488,000 x 75 = A.A.A.A.A. major difference in Environmental Impact ecessary Number of Erection Girder: 3 $15 \text{ days for } 1 \text{ span } \times 25 = 375 \text{ days}$ ender appearance with long span applicable for longitudinal slope 60 days for assembling % (Preliminary Estimate) mger Span Length (60m) 45 days for removal B ost Recommended. costal condition. bles 10 10 100 9 9 15 Evaluation Item Structural Aspect and stability Construction Cost Construction Plan New Technology Environmental Impact STEP Clearance notos and Sch Maintenance Evaluation and Period Aesthetics

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Table 8.4.1-1 Comparison on Erection Method for Approach Bridge

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

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(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@60.0 m x 3 erection girders = 4,500.0 m

- 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;

 \cdot 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).

			10	40	4	15	8	5	2	5	89
Aternative-4	Cantilever Consturuction 4,433.7m		struction of pier beble, travelers are hebricable and then and/ever erection is stathed with casts h-clusterious set popular construction method for long span (more and here are a biol achievements al over the wold.	Cutre 1, 1943, 746, 000, 000 VMD 446 2, 24, 000, 000, 000 VMD 746 25, 000, 000, 000, 000 VMD 1, 361, 441, 275, 000 VMD 1,	work, these weeks on south the south the same are are a and easy by procure them in storthine (5 morks), un-structure explane at leaks (5 morks for left group and structure explane at leaks (5 morks) and (14,2,3,10), 22, no. c fitth each are interestary to quiterment fram construction period, (5 morks) 500 days quiterment fram construction period, (5 morks) 500 days and 24, 7 morks (5 morks) extra (5 morks) extra and 24, in order and cannot be constructed at the same period is 30,7 morths (221 days). This altimative is taken considering sub-structure softended.	milar to Alternative 1 & 3 due to same span ent	liminary Estimate)	milar to Alternative 1 & 3 due to same span ent	e many achievements in Vietnam.	no difference between Alternative 1,3 & 4.	omm ended rnative is notrecommendable from the view point of n schedule.
			After con balanced o method. This is m than 60m),	Superstru Travel Fabrication Substrue Tota Ratio	Due to wide lot of this by However, s compasion of 10 num for 10 num structures be constru firme.) Tota not realistic	 This is si arrangem 	44% (Pre	 This is si arrangem 	 There ar 	 There is 	Less Rec This alle constructio
			10	32	<u>∞</u>	15	10	5	2	2	6
Atemative-3	P&Z Election Method 4,433.7m		ered of some kind formitiever rection methods with long t (rins. trim,) har construction of there table, candhore with anne as ordinary candhorer construction method. usa developed in Germany and transferred b Japan in here are numbers of achievement in Japan. (Max span i here are numbers of achievement in Japan.	CIM-1000 CIR3.7580000 CIR3.7580000 CIR3.7580000 CIR3.758000000 CIR3.758000000000000000000000000000000000000	Also discription grader to insufficient of grader at the sale water 1 year provide the same of the same of the same are necessary to explore the same of the same of the same states of the days of same of same of sales of the same and sales of exection sates of same values of s. 4.	isimlar to Alternative 1 & 4 due to same span ement	Preliminary Estimate)	i simlar to Alternative 1 & 4 due to same span sment.	ase in Vietnam.	is no difference between Alternative 1,3 & 4.	.ecommended the economic & construction speed view points, this we is secondry recommendable among 4 allernatives.
			P 82 n segmen start as tart as 1995 T 1995 T 100m J	Super Erects Fabrication Subs	From on In A1- meet flu for 9 nu glider (and lon	 This is arrange 	64% (F	-This is arrange	-First c	-There	Less F From alternat
			er 10	24	2 . G . S	es 9	10	3	4	3	of 71
Atternative-2	Moving Scaffolding System 4,433.7m		Formis supported by MSS erection glock, and one spin glid s construed by case-in-stu construction method. Theam T it Bridge is only, one achievement in Vietkam. Max span is 50m due b scale (weight) of evection girds.	Superstructure 122:155.00.000.W0D Exceloration 427:00.000.000.WD Submission 427:00.000.000.WD Submission 54:00.000.000.WD Submission 54:00.000.000.WD Submission 44:00.61.000.WD Submission 34:44.065.800.000.WD Fiber 31:44.065.800.000	iom order of exection grinde to treated action of grinder at the alle lates 1 ye and 1 PTS (4, 423, Tm), 4 nos. of MSS grinder are increasing the required from constration for the second grind of the second to a second a stransportation of election grinder (1936 days). This required period (930 days) is bronger than Allermative (1934 days)	Due to shorter span (50m), the numbers of pier & bearing sho norease.	68% (Preliminary Estimate)	Due to large numbers of piers copared with Alternative 1, 3, & 4, fris Merrafive is not superior to the others.	There is one achievementin Vietnam. (Thanh Tri Bridge)	Due to increased number of pier, noise at expansion joint is outer than the other alternatives.	.ess Recommended Construction period is longer fran Alernafive 1 & 3 and scale. Construction grider is very big and it costs much more than Alernafive
			10	32 F	10 10	15	10	5	2	<u>د</u>	92 E
Afternative-1	Span by Span Erection Method 4,433.7m		is segment is failed at year, then they are then pointed for a first segments placed that and placed by SBS grider one span by one span grider one span by one span are a bit of achievennin Japan and other countries,	Inclue 1068115800 000 VMD select 213,712,500 000 VMD TYard 250 000,000 000 VMD sete 251,775,000 VMD 151,775,000 VMD 1,111	e of evolution guider to installetion of guider at the site backes 1 year. PPG5. 3 nose of SBS girleter are mecessary to meet the evolt from construction period. It takes 480 days suddusive fransported among 4 alternatives (945 days).	similar to Alternative 3 & 4 due to same span nent	reliminary Estimate)	similar to Alternative 3 & 4 due to same span nent	se in Vietham.	s no difference between Alternative 1,3 & 4.	escommended teten period is shortestamong 4 allernalives. After ng this point and cost, this allernaive is most natable.
			 Pre-ca: to the ere erection : erection : 	Supers Erection Fabrication Substr Tol Rat	From ords In A1~ requirem order & th period is	This is : arrangen	56% (PI	 This is : arrangen 	 First ca 	 There is 	Most R. Constru consideri recommen
	٩	hotos	10	40	10	15	10	5	5	5	100
Evaluation Item	Length of Bridg.	Schematics and P	Stuctural Aspect and Stab My	Constuction Cost	Constrution Plan and Period	Maintenance	STEP Clearance	Aesthefics	New Technology	Environmental Impact	Evaluation

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

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

—	1 1		T T	_	T			Т						r i	T	1	1	I .
	F			10		6				4			15	~	5	2	5	68
Atemative-4 Cantilever Construction	548+3@60+548m, 548+2@60+548m =5192		construction of pier table. It av ders are fabricated and then sed cantilever erection is statthed with cast-in-situ constructor od	is most popular construction method for long span (more Um), and frere are a lot of adhievements all over the world.	structure 134,166,608,228 VND	(Including traveler) 141.736.360.000 VND	Total 275,902,968,228 VND	Ratio 1.00	wee word, large scale traver is required. In Japan, there are a lis type and easy to procure them in short time (5 months). <i>Let</i> , sub-structure requires at least 6 months for first group	ave. 2 → A2,114 numbers of have bers are set, the construction 1s 464 days. This period is not artical pass for total uction period. (Ortical pass in schedule is in Hai An side.)	der's weight is 117 ten X 32 = 3,744 ton(Hai An)	aler's weight is 117 ton X 4 = 468 ton(C at H ai)	is similar to Alternative 1 & 3 due to same span gement	(Preliminary Estimate)	is similar to Alternative 1 & 3 due to same span gement.	e are many achievements in Vietnam.	e is no difference between Alternative 1,3 & 4.	Recommended esest Construction period is longer fran Alternative-1, but dion of P79-A2 is not on the oritical path.
			-Ater balan mefno	- This than 6	Supers	Substr			lot of th Hower Hower	compare period constr	Trave	- Trave	arran	44%	 This arran 	-The	•The	Most . Che the se
Atemative-3 P&Z Election Method	ᢧ60+54.8m, 54.8+2@60+54.8m =519.2m		s one kind of cantlever erection methods with long 0m). After construction of pier table, cantlever will i ordinary cantlever construction method.	10 Boped in Germany and transferred to Japan in e numbers of achievement in Japan. (Max span is	163,560,590,992 VND	(Induding erection girder) 141.736.360.000 VND 32	305,296,950,992 VND	1.11	ction groter to installation of groter at the site takes 1 year.	619.2m) , construction period is langest among 4 4 6 days)	r weight is approximately 559 ton.		to Alternative 1 & 4 due to same span	nary Estimate) 10	to Alternative 1 & 4 due to same span 5	fetnam. 5	fference between Alternative 1,3 & 4 5	mended seriod is longtest among 4 alternatives.
	54.8+3(P 8.2 method i segment (max start as same a This was dev 1995. There ar 100m.)	Superstructure	anton4sonS	Total	- Ratio	From order of ere	-In P79∼A2 (atternatives. (56	 Erection girde 		This is similar t arrangement	0% (Prelimi	This is similar arrangement	 First case in V 	 There is no di 	Less Recom	
	2m	*1	-a	10		16		_	1	4			es 9	10	3	4	3	59
Alternalive-2 Moving Scaffolding System	42.3+4@50+42.3m, 42.3+3@50+42.3m =519		 Form is supported by MSS erection girder, and one span gird is construted by cash-n-stu construction method. 	-Thanh Tri Bridge is only one advievement in Vietham. • Max span is 50m due to scale (weight) of erection girder.	Superstructure 209, 356, 995, 996 VND	(Inducting erection girder) Substructure 177,170,450,000 VND	Total 386,527,445,996 VND	Ratio 1.40	From order of election grade: 10 installimion of grader at the site lakes 1 y	In P79→A2 (519.2m), constuction period is 355 days. It is longer than Altermative 1.	-Erection girder weight is approximately 950ton for 50m span length. If roof and other facility are added, construction is available in any weather and any second se	condition. And total numbers of are 5	 Due to shorter span (50m), the numbers of pier & bearing sho increase. 	68% (Preliminary Estimate)	. Due to large numbers of piers copared with Alternative 1, 3, 3, 4, this alternative is not superior to the others.	 There is one achievement in Vietnam. (Thanh Tri Bridge) 	 Due to increased number of pier, noise at expansion joint is louder than the other alternatives. 	Not Recommended • Construction cost is high and period is long.
	_			9		24		ľ		9	• =	0	15	10	5	5	5	84
Aternative-1 Span by Span Erection Method	60+54.8m, 54.8+2@60+54.8m =519.2n		nt is fabricated at yard, then they are transported . Segment is placed inted and placed by SBS is span by one span.	of achievermt in Japan and other countries.	189,675,750,687 VND	(Induding erection girder) 141,736,360,000 VND	331,412,110,687 VND	1.20	ion grow to mission of grow at the site takes 1 year.	19.2m), construction period is shortest among 4 days) and no influence in the total construction	weight is approximately 815 ton, and total caref. A considered in transportation of segment		Alternative 3 & 4 due to same span	ary Estimate)	Alternative 3 & 4 due to same span	tham	srence between Alternative 1,3 & 4.	ended sriod is shortest among 4 alternatives.
	54.8+3@I		 Pre-cast segments to the erection site erection girder on 	 There are a lot o 	Superstructure	Substructure	Total	Ratio	From order of erect	In P79~A2(51 alternatives (270 c period	Erection girder v It is required to c		 This is similar to arrangement 	65% (Prelimine	 This is similar to arrangement. 	 First case in Viel 	 There is no diffe 	Less Recomm - Construction pe
sm	lge	Photos	<u> </u>	10		40				10			15	10	5	5	5	
Evaluation Iter	Length of Brid	Schematcs and F		Structural Aspect and Stability		Construction Cost				Construction Plan and Period			Maintenance	STEP Clearance	Aesthetics	New Technology	Erwironmental Impact	Evaluation
							_			-		_	_		-			

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

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

(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

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

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)

Dari	o n	Upper	Under	Domortz		
Kegi	OII	(Inside)	(Inside) (Outside)			
*1 Upper Sleb	Cantilever	40mm	60mm	Fc=50Mpa		
· I Opper Stab	Inside of Box	40mm	40mm			
*2 Lower Slab		40mm	60mm			
*3 Web		40mm	60mm			

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

Source : Study Team

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

(by "PC Bridge Planning Manual")

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

Table 8.4.1-5 Approximate dimension of each member

*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

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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 <i>q</i> 45	45mm	
Longitudinal Tendon	Sheathø78	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.





Cover	40mm+60mm	100mm	
Reinforcement Bar	D19+D12x2+D16	59mm	
Longitudinal Tendon	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.

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

Table 8.4.1-8 Thickness of Web

- (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:

*Asphalt pavement (Thickness t=75mm) :

*Curb : Concrete high column weight : Hand Rail weight

*water service tube $\phi400$ (Article 2) :

 $\gamma c = 24.50 \text{kN/m3}$ $\gamma s = 22.50 \text{kN/m3}$ $w = 7.575 \times 2 = 15.15 \text{kN/m}$ $wh = 0.600 \times 2 = 1.200 \text{kN/m}$ $W = 2.250 \times 2 = 4.40 \text{kN/m}$

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)

 \pm 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

Load Combination	DC DD D	LL IM CE						TU			Use Thes At a	One e Time	e of
Limit State	W EH EV ES	BR PL LS EL	TL	WA	WS	WL	FR	CR SH	TG	SE	EQ	СТ	CV
Strength-I	γ_p	1.75	1.75	1.00	-	-	1.00	0.50/1.20	γTG	γSE	-	-	-
Strength-II	$\gamma_{\rm p}$	-	-	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	n Strength fc	Мра	50.0	Cylinder
Compressive Strength	at Pre-stressing fci	Мра	36.0	
Modulus of Elasticity l	Ec	Mpa	33000	
Allowable Stress				
Compressive Stress	Before Losses	Mpa	21.0	
	D.L.W.S	Мра	17.0	
	Service Limit State	Мра	17.0	
Tensile Stress	Before Losses	Мра	1.8	
	D.L.W.S	Мра	No tension	
	Service Limit State	Мра	No tension	
Modulus Rupture		Мра	14348	

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b) Prestressing Steel

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

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

Source : Study Team

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

Item		unit	φ32	Remark
Tensile Strength fpu		Мра	1180	
Yield Strength fpy		Мра	930	
Modulus of Elasticity Ep		Мра	200000	
Allowable	During pre-stressing.	Мра	837	0.9fpy
tensile	After pre-stressing.	Мра	790	0.85fpy
strength	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	Мра	345	
Modulus of Elasticity Ep	Мра	200000	
Allowable tensile strength at design load.	Мра	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



Table 8.4.1-15 Spring Constants of Pile

Rotation spring (A rr)



Horizontal spring (Ass)

Vertical spring (Avv)

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

t₩₩-**I**

Kh:Horizontal spring

Kv : Vertical spring

	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)

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

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6) Arrangement of PC Tendon in main Girder

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



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





Source : Study Team





THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT

The tensile stress doesn't occur in all sections.

THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT



Source : Study Team















Figure 8.4.1-5 Bending Moment caused by Dead Load



Source : Study Team





THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT

Source : Study Team

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

The tensile stress doesn't occur in all sections.

THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT



Source : Study Team



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



Source : Study Team

Figure 8.4.1-16 Diagonal tensile stress





Source : Study Team



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



<1> Bending Moment of Main Girder





THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT

The tensile stress doesn't occur in all sections.



Source : Study Team



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



*Service load state



Figure 8.4.1-6 Bending Moment in Service

< 2 >Fiber Stres	s of Mair	ı Girder			
* Dead Load Sta	te (Top fi	ber stress)			
TOP FIBER					COMPLETENCOL SERVICEM COMPLETENCOL SERVICEM COMPLETENCION (R. 9. SEL-5)
	겯	싙	2	DML2	57 10
	E NO		UN07	50V	
	é		14	1÷	2
			••••••••••		•••••
ource : Study Team					
	Figu	re 8.4.1-25 Stre	ess of Top Fiber (D	ead Load)	
* Dead Load St	ate (Botto	om fiber stress)		and the second second
OP FIBER					CONTRACTOR AS A STRUCTURE CONTRACTOR OF STRUCTURE CONTRACTOR STRUCTURE CONTRACTOR STRUCTURE
	21	얻	칱	겯	2
	, mm . N	EU. NG	0N. m	00N ⁻ⁱⁿ	а. (V. а.
	6, 40	01 (-		1-	2°.9
10 ⁻¹					
*Service Limit S	tate (Top	fiber stress)		2mm N89.1	S.L.N. Sop Files courses S.L.N. Sop Files courses S.L.N. Sop Files courses S.L.N. Sop Files S.L.N. Sop Files
			~~~~	л ң	
ource : Study Team	Eiguro 9	1 1 27 Strace	of Ton Eibor (Son in	o Limit State)	
ource : Study Team	Figure 8	.4.1-27 Stress	of Top Fiber (Servio	ce Limit State)	
ource : Study Team *Service Limit	Figure 8 t State ( B	.4.1-27 Stress ottom fiber stre	of Top Fiber (Servio ess)	ce Limit State) ্রু	41
ource : Study Team *Service Limit BOTTOM FILMER	Figure 8 t State ( B	.4.1-27 Stress ottom fiber stre	of Top Fiber (Servic	ce Limit State)	41 min - N3
ource : Study Team *Service Limit BOTTOM FIBER	Figure 8 t State ( B	.4.1-27 Stress ottom fiber stre	of Top Fiber (Servic ess )	ce Limit State)	15. 7887. mult
OURCE : Study Team *Service Limit	Figure 8 t State ( B	.4.1-27 Stress ottom fiber stre	of Top Fiber (Servic ess )		
Source : Study Team *Service Limit BOTTOM FIBER	Figure 8 t State ( B	.4.1-27 Stress ottom fiber stre	of Top Fiber (Servic	ce Limit State)	15, 76X munt
Source : Study Team *Service Limit BOTTOM FIBER	Figure 8 t State ( B	.4.1-27 Stress ottom fiber stre	of Top Fiber (Servic ess )	ce Limit State)	Church 78N france
Source : Study Team *Service Limit BOTTOM FIBER	Figure 8 t State ( B	.4.1-27 Stress ottom fiber stre	of Top Fiber (Servic ess )	ce Limit State)	1.5. 7kN inne2

#### THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT

The tensile stress doesn't occur in all sections.





# (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.

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

Table 8.4.1-17 Structure clarification of Main girder

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  $\Rightarrow 100$  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:

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

Table 8.4.1-18 Direct wheel load on Upper slab

b) Web and Lower sla	ab	
Web and Lower slab	shall be designed as follows:	
- For JSHB	: stress of reinforcing bar when use shall be	

	less than $\sigma sa = 180 \text{N/mm2}$
- For FEM	: stress of reinforcing bar when use shall be
	less than $\sigma sa = 140 \text{N/mm2}$
- For JSHB	: stress of reinforcing bar under service load
	working state shall be less than $\sigma sa = 180 N/mm2$
- For FEM	: stress of reinforcing bar under service load
	working state shall be less than $\sigma sa = 140 N/mm2$
	Comparison on bending moment due to live load

c) Standard Cross Section [Upper slab] - [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

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

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

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		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		

Table 8.4.1-21 Composite bending fiber stress [A]

Source : Study Team

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



Source : Study Team



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

Mu=1.3*D+2.5*(LL	[JSHB-16]		
Mu=1.25*DC+1.5*I	[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]

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

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

Source : Study Team

Table 8.4.1-24 Designed Bendin	a moment (		IB1
Table 0.4. 1-24 Designed Denuing	y moment (	D+LL+IIVI+FS+OI+SH)	[D]

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

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

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

Table 8.4.1	I-25 Arrangei	ment of r	reinforced	bars

Source : Study Team

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

Table 8.4.1-26 Stress of reinforcing bars

Source : Study Team

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

Mu=1.3*	[JSHB-16]			
Mu=1.25	H) [P=100kN]			
Designed sectio	$\begin{array}{c} \text{Designed cross}\\ \text{section} \end{array} \begin{array}{c} \text{JSHB} \\ \text{Mu (kN \cdot m)} \end{array} \begin{array}{c} \text{FEM Analysis}\\ (P=100kN)\\ \text{Mu(kN \cdot m)} \end{array}$		Mr (kN∙m)	
Wah	*6	232.306	55.221	< 280.116
web *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.

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

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

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

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

Table 8.4.1-30 Composite bending stress [C]

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	[JSHB-16]		
Mu=1.25*DC+1.5*I	DW+1.75*(LL+IM)+	1.2or0.5*(Cr+SH)	[P=100kN]
Designed point	JSHB (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]

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

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

Source : Study Team

Table 8.4.1-33 Designed bending	$m_{O}m_{O}m_{O}m_{O}m_{O}m_{O}m_{O}m_{O}$	[D1
Table 0.4. 1-55 Designed bending		LD1

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

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

Table 8.4.1-34 Arrangement of reinforced bars

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

Source : Study Team

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

	Table 8.4.1	-35 Stress	of I	reinforcina	bars
--	-------------	------------	------	-------------	------

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=1						
Designed point JSHB (kN·m) (kN·m) (kN·m)				Mr (kN∙m)		
Wah	*6	256.314	63.292	< 480.253		
web	*7	154.896	51.320	< 402.764		
Lower	*10	132.654	26.828	< 142.964		
Slab	*11	19.908	14.407	< 65.044		

14.407< 65.044</th>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 bar	s (TCXDVN327: 2004)

Region		Upper	Under	Remark
		(Inside)	(Outside)	
*1 Upper Sleb	Cantilever	40mm	60mm	Fc=40Mpa
*1 Opper Slab	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 1Poy gird	1 Doy girdor	RC	0.24-0.30	0.30-0.50	0.30-0.50	0.25
Slab	i box giidei	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

Thickness of Upper Slab 2)

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 <i>q</i> 45	45mm	
Longitudinal Tendon	Sheathq77	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	Sheath \$\overline{77}\$	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	
390			

Cover	60mm+40mm	100mm
Reinforcement Bar	(D22+D16)x2	76mm
Longitudinal Tendon	Sheath $\phi77$	77mm
Longitudinal Tendon	Sheath <b>q</b> 77	77mm
		60mm
Tota	1	390mm

#### Source : Study Team

• Necessary thickness for anchorage of PC Tendon



416mm > 2x180mm=360mm

686mm > 2x180mm+270mm=630mm



(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:

	unit	Cor	nmon Travel	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	

Table 8.4.1-42 Standard classification of traveler

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

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

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

Source : Study Team

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



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



# (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:	$\gamma c = 24.50 kN/m3$			
*Asphalt j	pavement (Thickness t=75mm) :	$\gamma s = 22.50 kN/m3$			
*Curb	: Concrete high column weight : Hand Rail weight	$  w = 7.575 \times 2 = 15.15 kN/m \\ wh = 0.600 \times 2 = 1.200 kN/m $			
*water ser	vice tube φ400 (Article 2):	$W = 2.250 \times 2 = 4.500 \text{kN/m}$			

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)  $\pm 20$  °C (For Girder Design)

*Temperature Gradient: As shown in following Table.

Table 8.4.1-45Temperature Gradient

	T1	T2	Т3
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

Load Combination	DC DD D	LL IM CE						TU			Use Thes At a '	One e Time	e of
Limit State	W EH EV ES	BR PL LS EL	TL	WA	WS	WL	FR	CR SH	TG	SE	EQ	СТ	CV
Strength-I	$\gamma_p$	1.75	1.75	1.00	-	-	1.00	0.50/1.20	γTG	γSE	-	-	-
Strength-II	$\gamma_{\rm p}$	-	-	1.00	1.40		1.00	0.50/1.20	γTG	γSE	-	-	-
Strength-III	$\gamma_{\rm p}$	1.35	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γTG	γSE	-	-	-
Extreme	$\gamma_{\rm p}$	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	-	-	-	-	-	-	-	-	-	-

# Table 8.4.1-46 Load Factor and Load Combination

Source : Study Team

- 3) Material
- a) Concrete

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

#### Table 8.4.1-47 Properties of Concrete (Main Girder)

Source : Study Team

# b) Pre-stressing Steel

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

Item		unit	12S12.7L	1\$28.6
Tensile Stren	gth fpu	Mpa	1850	1800
Yield Strengt	h fpy	Mpa	1600	1500
Modulus of E	Elasticity Ep	Mpa	200000	200000
	During pre-stressing	Mpa	1440	1350
Allowable tensile strength	After pre-stressing	Mpa	1295	1260
	At design load	Мра	1110	1080

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

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

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

Item	unit	SD345	Remark
Tensile Strength fpu	Мра	490	
Yield Strength fpy	Мра	345	
Modulus of Elasticity Ep	Мра	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

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Figure 8.4.1-37 Analysis Model

Table 8.4.1-51 Spring Constants of Pile

Rotation spring (A rr)



Horizontal spring (Ass)

Vertical spring (Avv)

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

₩₩٠-MW-

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.





# 6) Arrangement of PC Tendon in main Girder

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







Source : Study Team



- 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



Figure 8.4.1-41 Bending Moment in Service load state

#### <2> Fiber Stress of Main Girder

*Dead Load State (Top fib	er stress)		CORDITION OF STRUCTURE
TOP FIBER	3. 38Х/тт.2 5. 79М/тт.2	ā, 6 <u>1</u> V/mn2	

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

*Dead Load State (Bottom fiber stress )

BUTTOM FIBER	9, 88N//mm2	8. 87N/mm2	9, 86N/mm2	COMPLETION OF STATUTER COMPLETION OF SERVICE WORK COMPLETION OF EXAMINED
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### Figure 8.4.1-44 Stress of Bottom Fiber (Dead Load)

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Source : Study Team
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Figure 8.4.1-46 Stress of Bottom Fiber (Service Limit State)



*Strength Limit State





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



Source : Study Team



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









# <2> Fiber Stress of Main Girder



















Source : Study Team



# (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.

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

Table 8.4.1-53 Structure	clarification	of Main	girder
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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  $\doteq 100$  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:

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

Table 8.4.1-54 Direct wheel load on Upper deck slab

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 $\sigma sa = 180 \text{N/mm2}$
- For FEM	: stress of reinforcing bar under service load
	working state shall be less than $\sigma sa = 140 \text{N/mm2}$

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]

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

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

Source : Study Team

Table 8 4 1-56 Designed Bending moment (D+LL+IM+Ps+Cr+S	-1)	IA1
	•/	L, 7

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

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	JSHB FEM Analysis		nalysis			
		σco (N/mm2)	σcu (N/mm2)	σco (N/mm2)	σcu (N/mm2)	
	*1	4.01	-0.05	4.01	-0.05	
Dead Load	*2	3.36	0.65	3.36	0.65	
2000	*3	0.27	6.73	0.27	6.73	
	*1	0.42	3.55	0.98	2.97	
Service	*2	-0.44	4.43	0.41	3.60	
Load	*3	6.81	0.19	2.99	4.00	
			> -3.0	> 0.0	> 0.0	

Table 8.4.1-57 Composite bending stress [A]

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)

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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*I	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 lo	bad) [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

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

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

Table 8.4.1-61 Arrangement of reinforced bars
-----------------------------------------------

Source : Study Team

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

Table 8.4.1-62 Stress of reinforcing bars

Table 8.4.1-63 Safety factor under ultimate load working state [B]						
Mu=1.3*	[JSHB-16]					
Mu=1.25*DC+1.5*DW+1.75*(LL+IM)+1.2or0.5*(Cr+SH) [P=100kN]						
Designed	l point	JSHB (kN∙m)	Mr (kN∙m)			
Wah	*6	231.571	95.305	< 276.952		
web	*7	114.165	35.199	< 276.952		
Lower	*10	106.848	29.340	< 141.089		
Slab	*11	19.773	13.230	< 64.666		

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

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

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

Table 8.4.1-66 Composite bending stress [C]

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*(LI	[JSHB-16]		
Mu=1.25*DC+1.5*	[P=100kN]		
Designed point	JSHB (kN·m) (kN·m) (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]

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

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

Source : Study Team

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			

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

Source : Study Team

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

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

## Table 8.4.1-70 Arrangement of reinforced bars

Source : Study Team

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

|--|

Source : Study Team

Safety factor under ultimate load working state [D]

Designed	l 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

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# 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



#### 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



## [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]

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#### [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.

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

Table 8.4.2-1	Stress of extern	al cable and	reinforcing bar
---------------	------------------	--------------	-----------------

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]
	8.99	235	140	32900	295771
	6.95	250	140	35000	243250
σzz	6.37	175	140	24500	156065
	6.37	175	140	24500	156065
		835			851151



#### < viewpoint-2 Vertical reinforcing bar >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
	5.56	220	140	30800	171248
σzz	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



#### 2) Calculation of horizontal reinforcement bar

< viewpoint-3 Horizontal reinforcing bar >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]		
	3.93	175	140	24500	96285		
	4.13	175	140	24500	101185		
	4.13	95	140	13300	54929		
	4.06	200	140	28000	113680		
σуу	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 = ∴ Areq =	180 T / σa =	N/mm2 3843	mm2			
	Steel bar : interval =	D22 200 mm	layer =	2	numb	er of bar =	12.5
	.'. ΣA =	4752	mm2	> Areq		0	k

3.13 3.53 3.71 3.71 3.64 3.57 3.09 2.07 1.65 2.24 2.77 2:10. -0.52 0.44 0.89 1.33 1.76 2.36 2.97 3.39 3.76 76 3.37 2.83 2.29 viewpoint-3 -0.83 -0.73 1.11 1.85 2.17 2.52 3.06 3.54 111 881 3.46 3.09 2.38 -1.19 -1.03 1.11 3.13 3.13 1.98 4.65 2.93 3.08 354 309 2.46 354 1.78 2.93 3.08 4.08 1.59 3.14 2.14 2.2 3.46 326 364 199 40 59 3.14 2.59 2.18 2.82 2.83 1.43 2.48 3.48 0.85 SB 3.13 2.61 2.18 3.0 1.11 3.38 2.18 2.12 2.77 56 3.06 2.61 0.74 1.69 2.51 3.09 3.49 1.17 3.41 3.06 2.60 2.05 3.21 3.04 2.40 2.84 3.22 3.38 0.59 1.50 2.53 1.86 3 0.85 1.74 2.26 2.75 3.11 3.28 3.28 3.26 3.10 2.84 2.49 1.76 2.22 2.77 2.53 2.26 1,47 2.29 2.36 2.61 2.85 2.96 2.96 2.93 2.75 2.63 2.43 2.49 2.58 2.60 2.60 2.53 2.37 2.14 1.92 1.19 275 2.36 2.35 2.31 2.20 2.02 1.77 1.55 1.14 0.59 0.96 1.66 2.66 2.75 2.75 2.63 2.43 2.37 1.07 1.24 1.44 1.78 2.26 2.40 2.35 2.26 2.21 2.15 2.07 1.94 1.74 1.46 1.29 1.14 2.40 1.42 1.48 1.55 1.71 1.58 1.42 1.29 1.64 1.91 2.12 2.14 2.14 2.09 2.02 1.92 1.82

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]
	7.91	165	100	16500	130515
σzz	6.60	123	100	12300	81180
		288			211695



#### < A1-SIDE Vertical reinforcing bar viewpoint-2 >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]		
	6.16	197	100	19700	121352		
	5.71	175	100	17500	99925		
	5.40	175	100	17500	94500		
	5.18	195	100	19500	101010		
077	4.93	155	100	15500	76415		
022	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		
	$\sigma_{a} = 180 \text{ N/mm}^{2}$						

ou		100			
. Are	a= T	/ σa =	4268	mm2	

ΣA =	5862 (	mm2	> Areq		ok
Steel bar : interval =	D22 200 mm	layer =	= 2	number of bar =	15.4

[A1 side of P3 cross beam]



#### 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		
077	1.49	155	200	31000	46190		
022	1.84	100	200	20000	36800		
	2.65	175	200	35000	92750		
	2.65	175	200	35000	92750		
		1150			503120		
	σa = · Area –	180 T / ca =	Wmm2	mm2			
	Aleq –	170a -	2195	111112			
	Steel bar : interval =	D22 200 mm	layer =	2	numb	er of bar =	11.5
	.:. ΣA =	4372	mm2	> Areq		0	k

#### [P2 side of P3 cross beam]

viewpoint-1



STEP1 prestressing one side

STEP2 prestressing both sides

- 2) Calculation of horizontal reinforcement bar
- a) A1 side

< A1-SIDE Horizontal reinforcing bar viewpoint-3 >

						i i
	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]	
	3.41	175	100	17500	59675	
	3.41	175	100	17500	59675	
$\sigma_{\rm MM}$	2.84	220	100	22000	62480	
буу	3.04	175	100	17500	53200	
	3.04	175	100	17500	53200	
		920			288230	
	σa =	180	N/mm2			
	∴ Areq =	Τ / σa =	1601	mm2		
	Steel bar :	D22				
	interval =	250 mm	layer =	2	numb	er of bar

> Areq

2798 mm2

#### [A1 side of P3 cross beam]

. ΣA =



STEP1 prestressing one side

STEP2 prestressing both sides

7.4

ok

b) P2 side

< P2-SIDE Horizontal reinforcing bar viewpoint-2 >

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]		
	3.67	175	100	17500	64225		
<b>7</b> 10 (	3.67	175	100	17500	64225		
	3.49	95	100	9500	33155		
буу	3.68	200	100	20000	73600		
	3.68	200	100	20000	73600		
		845			308805		
	σa = ∴ Areq =	180 T / σa =	N/mm2 1716	mm2			
	Steel bar : interval =	D22 250 mm	layer =	2	numb	er of bar =	6.8
	.:. ΣA =	2570	mm2	> Areq		ol	<b>K</b>

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

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.

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Source: Study Team



Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

# 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.

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



# 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.



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# 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



#### Source: Study Team





[Type1]

Source: Study Team



Results of examination about Reinforcing bar is shown following,

a) T	vpe1:	Two external	cables are	deviated at	L2 and L5	(R2 and R5	) at the same time
------	-------	--------------	------------	-------------	-----------	------------	--------------------

case2	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]				
	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				
Deviator	2.92	150	120	18000	52560				
R2+R5	1.22	150	60	9000	10980				
(L2+L5)	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		1	353280				

< Type1 : Two external cables are daviated >

#### 180 N/mm2 σa =

$$\therefore$$
 Areq = T /  $\sigma a$  = 1963 mm2

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

ok

ok

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

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]	1
	0.00	150	120	18000	0	1
	0.00	150	120	18000	0	1
	0.00	100	120	12000	0	1
	0.00	100	120	12000	0	
	0.00	150	120	18000	0	1
Deviator	0.00	150	120	18000	0	1
R1	0.00	150	60	9000	0	1
(L1)	0.00	150	60	9000	0	
	0.00	100	60	6000	0	
	0.00	100	60	6000	0	1
	0.00	150	60	9000	0	1
	0.00	150	60	9000	0	1
		800			0	1
	2.53	150	120	18000	45540	1
	2.65	150	120	18000	47700	ĺ
	2.68	100	120	12000	32160	ĺ
	2.68	100	120	12000	32160	1
	2.65	150	120	18000	47700	
Deviator	2.53	150	120	18000	45540	1
R6	1.20	150	60	9000	10800	1
(L6)	1.31	150	60	9000	11790	
	1.33	100	60	6000	7980	
	1.33	100	60	6000	7980	1
	1.31	150	60	9000	11790	1
	1.20	150	60	9000	10800	
		800			311940	1
	Tmax = ∴ Areq =	311940 Tmax	N σa =</td <td>σa = 1733 mm2</td> <td>180</td> <td>N/mm2</td>	σa = 1733 mm2	180	N/mm2
	Steel bar :	D19	interval =	125 mm	number =	6.4
	ΣA =	1815	mm2	> Areq		

< Type2 : One external cable is daviated >

* 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.



*Considering stress distribution,

area that tensile stress is generated in is effective in calculation.

Deviator L1 ~ L6(R1~R6)

#### Source: Study Team







Source: Study Team



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

case2	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]		
	3.97	150	100	15000	59550		
	3.81	150	100	15000	57150		
Deviator	3.52	100	100	10000	35200		
L2+L5	3.52	100	100	10000	35200		
(R2+R5)	3.81	150	100	15000	57150		
	3.97	150	100	15000	59550		
		800			303800		
	σa = ∴ Areq =	180 T / σa =	N/mm2 1688 mm2				
	Steel bar : .:. ΣA =	D19 1815	interval = mm2	125 mm > <b>Areq</b>	number =	6.4	ok

< Type1 : Two external cables are daviated >

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

	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
	1.17	150	100	15000	17550
	1.11	150	100	15000	16650
Deviator	1.04	100	100	10000	10400
L1	1.04	100	100	10000	10400
(R1)	1.11	150	100	15000	16650
	1.17	150	100	15000	17550
		800			89200
	4.09	150	100	15000	61350
	3.98	150	100	15000	59700
Deviator	3.71	100	100	10000	37100
L6	3.71	100	100	10000	37100
(R6)	3.98	150	100	15000	59700
	4.09	150	100	15000	61350
		800			316300
	Tmax =	316300	N	σa =	180
	: Areq =	Tmax	(/ σa =	1757 mm2	

•	rypez . One	external car	16 12	uavialeu	~
		σ [N/mm2]	B	[mm]	

Tmax = ∴ Areq =	316300 N Tmax/	l ′σa =	σa = 1757 mm2	180 N/	mm2
Steel bar : ∴ ΣA =	D22 2433 n	interval = nm2	125 mm > Areq	number =	6.4

ok

< Type2 : One external cable is daviated >

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

σa = ∴ Areq =	180 N/mm2 T / σa = 1118 mm2					
Steel bar :	D16	interval =	125 mm	number =	6.4	
ΣΑ=	1287	mm2 >	> Areq			ok

Upper side of Deviator 3)



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

#### Source: Study Team





Source: Study Team



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

σ[N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]	
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 = .∹ Areq =	180 T / σa =	N/mm2 1265 mm2			
Steel bar : ΣA =	D16 1407	mm2	> Areq	number =	7.0
	2.27         2.32         2.34         2.34         2.32         2.34         2.32         2.34         2.32         2.34         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90         0.90	C [1011112]       D [1111]         2.27       150         2.32       150         2.34       100         2.32       150         2.34       100         2.32       150         2.32       150         2.32       150         2.32       150         0.90       150         0.90       150         0.90       100         0.90       150         0.90       150         0.90       150         0.90       150         0.90       150         0.90       150         0.90       150         0.90       150         0.90       150         0.90       150         0.90       150          Req =       T / σa =         Steel bar :       D16          ΣA =       1407	0 [101111]       D [1111]       D [1111]         2.27       150       100         2.32       150       100         2.34       100       100         2.34       100       100         2.34       100       100         2.32       150       100         2.32       150       100         2.32       150       60         0.90       150       60         0.90       150       60         0.90       150       60         0.90       150       60         0.90       150       60         0.90       150       60         0.90       150       60         0.90       150       60         0.90       150       60         0.90       150       60          800          σa =       180       N/mm2         ∴ Areq =       T / σa =       1265 mm2         Steel bar :       D16       ∴ ΣA =          1407 mm2       1407 mm2	0 [101112]       D [1111]       D [1111]       D [1111]         2.27       150       100       15000         2.32       150       100       15000         2.34       100       100       10000         2.34       100       100       10000         2.32       150       100       15000         2.32       150       100       15000         2.32       150       100       15000         2.32       150       100       15000         2.32       150       100       15000         0.90       150       60       9000         0.90       150       60       9000         0.90       150       60       9000         0.90       150       60       9000         0.90       150       60       9000         0.90       150       60       9000         0.90       150       60       9000          800           σa =       180       N/mm2       -          Areq =       T / σa =       1265 mm2	0 [1011112]       D [11112]       T [11112]       T [1112]         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         2.32       150       100       15000       34800         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       150       60       9000       8100         0.90       150       60       9000       8100         0.90       150       60       9000       8100         0.90       150       60       9000       8100         0.90       150       60       9000       8100         0.90       150       60       9000       8100

< Type1 : Two external cables are daviated >

ype2 : O	ne external c	able is daviat	ed >		
	σ [N/mm2]	B [mm]	H [mm]	A [mm2]	T [N]
, , ,	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
Deviator	2.39	150	100	15000	35850
L1	1.13	150	60	9000	10170
(R1)	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
	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
Deviator	1.08	150	100	15000	16200
L6	0.38	150	60	9000	3420
(R6)	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	<u>60</u>	<u>90</u> 00	3420
		800			103900
	Tmax =	240680	N	σa =	180

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

	Z.44	100	100	10000	24400
	2.40	150	100	15000	36000
Deviator	2.39	150	100	15000	35850
L1	1.13	150	60	9000	10170
(R1)	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
	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
Deviator	1.08	150	100	15000	16200
L6	0.38	150	60	9000	3420
(R6)	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
	Imax =	240680	N	σa =	180
	∴ Areq =	Imax	( / σa =	1337 mm2	
	Steel bar :	D16			number =
	. ΣA =	1407	mm2	> Areq	

< 1

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.

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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.

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

# 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



# (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	orrdinar	soft ground	
Height of abutment	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

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd.,

PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

# 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.

			9	32	9	6	10	ŝ	3	3	72
Alternative-4	Alternative-4 Round shape column		<ul> <li>The arrangement of bar at the connection between column, beam and pile cap will be conficated. So this shape is inferior in constructbility.</li> </ul>	Quantity         Unit Cost         Total           Quantity         QNDD         QUOVND)           Concrete         119m3         5877.864           Concrete         119m3         5677.864           Concrete         119m3         698.803	column and beam 13 days	u nou cr naintenance.	- 32% (Preliminary Estimate)	Round shape of column nakes soft inpression.	-Nothing new technology is used	No major emvironmental impact.	<ul> <li>This alternative is most inferior in constructulity.</li> <li>Not Recommended</li> </ul>
			s 4	38 88	∞	0	10	61	ŝ	3	63
narpe of Approach Bridge	Alternative-3 Oval shape column		<ul> <li>Beaning weight for the foundation will increase because of adding it own weight.</li> <li>No necessity to establish the timbering.</li> </ul>	Quantity         Unit Cost         Total           Quantity         Unit Cost         Total           Concrete         137m3         5.867.861         895.66           Concrete         137m3         7.867.861         895.66           Concrete         137m3         7.867.861         895.66           Ratio         17na         8.846         10.66	column and beam 14 0 days	vo necessity to maintenance.	-32% (Preliminary Estimate)	-Wide width of column makes a sense of oppression.	-Nothing new(technology is used.	-No najor emvironmental impact.	-This alternative is most infeavran economic and in aesthetic points Nor B economication
ler		7	8 8	6 6 6	~	6	10	5	ŝ	3	8
2-2 Comparative Study on Pie Alemative-2 Rectangle solution and	Alternative-2 Rectangle shape cohunn smoothing angle between cohunn and Pier head		-Bearing weight for the foundation will reduce because of deduction is own weight.	Quantity         Unit Cost         Total           Quantity         QNND         QOWND         QOWND           Concrete         108m3         S87_366         637,7           Concrete         108m3         S87_366         637,7           Fraid         108m3         S87_366         637,7	column and beam 11 days	-No necessity to insinitenance.	- 32% (Prelininary Estimate)	-Smoothing the angle between pier head and column makes soft impression .	-Nothing new technology is used.	-No major emvironmental impact.	<ul> <li>This alternule is superior to obtact cases in aesthetic points. Construction Cost is the second lowest in the alternatives. but almost same as alternative-1. More Recommended</li> </ul>
8.			∞	40	~	6	10	61	ŝ	3	83
Table	Alternative-1 (SAPROF) Rectangle shape column		Beaning weight for the foundation will reduce because of deduction of its own weight.	Cuantity         Unit Cast         Total           Quantity         Quantity         Quantity         Quantity           Concrete         107m3         5.867.366         073.361           Concrete         107m3         5.867.366         0.273.461           Concrete         107m3         5.867.366         0.273.461	column and beam 11 days	No necessity to maintenance.	- 3.2% (Prelininary Estinate)	-Sharp corner nokes inorganic and hard inpressions.	Nothing newtechnology is used	-No major emvironmental impact.	This alternative is superior in acconomic, but inferior in aethetic points compared to alternative 2. Less. Recommended
			10	40	10	15	10	5	Ś	5	100
	Evaluation Items	Side View	Structural Aspect and Stability	Construction Cost (for Foundation)	Construction Plan and Period	Maintenance	STEP Clearance	Aesthetics	New Technology	Environmental Aspect	Evaluation

THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

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



b) Arrangement of reinforcement



Source : Study Team



# (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

Source : Study Team

Table 8.4.3-5 Refer from JSHB



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



# 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.

Type	Pier No	Pier Height*	Column Height	Water depth
rype	1101110.	(m)	(m)	(m)
	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
Iyi	P23	9.0	6.5	3.37
L	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

Table 8 4 4-1	Pier Height and	Seawater Depth
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Tuna	Dior No.	Pier Height*	Column Height	Water depth
rype	Pier No.	(m)	(m)	(m)
	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
e-	P52	15.5	13.0	7.51
Γyŗ	P53	15.5	13.0	7.51
r .	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
	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
5	P67	13.0	10.5	7.51
vpe	P68	13.0	10.5	7.51
É.	P69	13.0	10.5	7.51
	<b>P</b> 70	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
-3	P76	23.5	21.0	6.94
ype	P77	27.0	24.5	8.67
É	P78	28.0	25.5	10.80
4	P79	20.0	17.5	11.53
-e-	P80	19.0	16.5	11.13
Tyj	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.

Study Type	Type-1 ₋₁	Tupe-2	Туре-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.L5.0 (Top of Pile Cap)	Variation 4	Variation 1

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

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

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

Source: Study Team



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

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.
## (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.



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.

# 

Source : Study Team

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

## 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 st	udy : 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".

#### THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT

			10	IDIE 0.4.4	+-5 Companse	on for pir	e ulameter ul	steet hit	e plie al Fiel		
	Alterr	native		SA	PROF study	AI	trenative-1	AI	trenative-2	Al	trenative-3
	Pile 7	Туре		Steel p	ile pipe D=0.8m	Steel p	ile pipe D=0.8m	Steel p	ile pipe D=1.1m	Steel p	ile pipe D=1.4m
	Plan of F	Pile C	ap	1000 - 2200 1000 - 7200 0 0 0 - 7200 0 0 0	22400=7200 100 0 ⊕ ⊕ ⊕ 0 ⊕ ⊕ 0 ⊕ ⊕ 0 ⊕ ⊕	$1 \frac{10000}{1000} = 8000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 10000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1000 - 1$	$ \begin{array}{c} 10000\\ 1000\\ \hline 1000\\ \hline 0\\ \hline 0\\ \hline 0\\ \hline 0$	$1375 \xrightarrow{11000} 1375 \xrightarrow{1100} 375 $	$\begin{array}{c} 11000 \\ \hline \\ \hline \\ \hline \\ \hline \\ \\ \hline \\ \\ \\ \\ \\ \\ \\ $	$\begin{array}{c c} & & & & \\ \hline & & & & \\ \hline & & & & \\ \hline & & & &$	
Dianlage		-		L=46.0m	n=14nos	L=46.0m	n=25nos	L=46.0m	n=16nos	L=46.0m	n=12nos
Displace	ement		mm	ox=1.8	≥oa=15 (UK)	0x=1.0	≥oa=15 (UK)	0x=1.7	≥oa=15 (UK)	0x=1.4	≥oa=15 (UK)
Pile Rea	action		KN	P _{nmax} =312	0 > Ra=2050 (NG)	P _{nmax} = 1	850 < Ra=2050	P _{nmax} =3	000 < Ra=3160	P _{nmax} =4	-370 < Ka=4530
	thickness		mm	t=11	mm(SKK400)	t=11	mm(SKK400)	t=11	mm(SKK400)	t=11i	mm(SKK400)
	Mu		KIN.M		483.2		280.1		5/5.2		762.2
Pile body	fe		KIN.III		2 /7		5.56		4 37		5 28
	13	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	10	1.0 < 140.0	59	.0 < 140.0	80	.0 < 140.0	83	.0 < 140.0
	Cost Es	stimat	e	Motorial	Cont	Meterial	Cont	Motorial	Cont	Motorial	Cont
Iten	n	unit	Unit cost (VND)	Quantities	(VND)	Quantities	(VND)	Quantities	(VND)	Quantities	(VND)
	Pile	cap	( = /		Altrenative-2           ile pipe D=0.8m         Steel pile pipe D=0.8m         Steel pile pipe D=0.8m           9200         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         1000         10000         1000         1000 <th< td=""><td></td><td></td></th<>						
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 Co	ncrete	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
Excava	ation	m ³	318.066			113.0	35.941.458	128.0	40,712,448	147.0	46,755,702
Coffer	dam	ton	24,798,638			87.0	2,157,481,506	93.0	2,306,273,334	101.0	2,504,662,438
	Sub	total			$\mathbf{X}$		3,691,547,074		4,159,404,374		4,753,899,325
	Found	dation			X						
Steel Pile	0.8m	m	0		$\angle$		15,063,501,667				
(Diameter)	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
	10	tal		/			18,755,048,741		17,082,677,374		19,734,417,325
	rat	UU		/	<u>_</u>		1.098	Moot	1.000		1.105
	Evall	auon		v	\ \	1		IVIOST I	recommended		

## Table 8.4.4-5 Comparison for nile diameter of steel nine nile at Pier

Source: Study Team

	Altern	ative		Altre	enative-1	Altı	enative-2	Altı	enative-3					
	Pile 7	уре		Steel pile	pipe D=0.8m	Steel pil	e pipe D=1.1m	Steel pil	e pipe D=1.4m					
	Plan of F	Pile Ca	p		2000 00 4	1000-100-100-100-100-100-100-100-100-10	4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4 4							
			_	L=43.5m	n=44nos	L=43.5m	n=40nos	L=43.5m	n=36nos					
Displace	ment	100-11	mm	δx=7 8 ≦	δa=15 (OK)	δx=5.5 ≦	έδa=15 (OK)	δx=5.7 😫	έδa=15 (OK)					
Lateral res	teral resistance kN Pile Reaction kN		kN	Qr =553	0 < N=5977	Qr =553	30 < N=6116	Qr =5975 < N=6039						
Pile Rea	Pile Reaction kN thickness mm			Pnmax =6610	09 < Ra=83512	Pemax =822	79 < Ra=120116	Prmay =110930 < Ra=160088						
Pile Reaction kN thickness mm Mu kN.m			mm	t=12mi	m(SKK400)	t=12m	im(SKK400)	t=12mm(SKK400)						
	Plan of Pile Cap  Displacement mm  tetral resistance k/N  Pile Reaction k/N  Pile Reaction k/N  thickness mm  Mu k/N m  Mn k/N m  Latrema ft N/m  Cost Estimate  tetrema unit Unit (/N  Pile cap  oncrete 28/MPa m ³ 5.9  Lean Concrete m ² 1.7;  Bilinding stone m ³ 66  Excavation m ³ 3  Cofferdam ton 24.7;  Sub total  Excendence			1	732.8		1208.6	1858 7						
Displacement mm ,ateral resistance kM Pile Reaction kN thickness mm Mu kNr Mn kNr fs - Extreme ft NV Cost Estimate tem unit Uni		kN.m	1	657.7		3328 3	3	5190.1						
Plie body	tile Reaction kN tile Reaction kN Mu kN.m Mu kN.m Mn kN.m Extreme ft N/m Cost Estimate tem unit Unit c		-		2.26		2.75		2.79					
	Plan of Pile Cap Plan of Pile Of Plan Plan of Plan of Plan of Plan Plan of Pile Of Plan Plan of Plan of Plan of Plan Plan of Plan of Plan of Plan Plan of Pla	N/mm ²	5.6	< 140.0	1.4	< 140.0	21.4 < 140.0							
	Landonio	ft	N/mm ²	98.0	< 140.0	84.0	< 140.0	84.0 < 140.0						
İten	Cost Es	unit	Unit cost	Material Quantities	Cost (VND)	Material Quantities	Cost (VND)	Material Quantities	Cost (VND)					
UD D D D D D D D D D D D D D D D D D D	Pile	can	(VIAL)											
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 Co	icrete	m3	1,723,811	23.5	40 568 168	32.1	55 4 10 18 1	45.0	77 595 628					
Blinding	stone	m ²	696 000	47.1	32 759 328	64.3	44 744 448	90.0	62 659 488					
Excava	tion	m ³	318 066	372.4	118 435 056	468.7	149 064 812	617.8	196 488 452					
Coffere	lam	ton	24,798,638	78.6	1.955 124 620	126.4	3 133 555 898	142.6	3 535 293 833					
	Cofferdam ton 24,798 Sub total				6,737,463,876		9,227,607,972	1.12	11.587.251.686					
Sub total Foundation					(d)									
Steel Pile	0.8m	m	0.	1,914.0	26,511,762,933									
Diameter)	1.1m	m	0			1,740.0	32 308 182,500							
or solves by state	1.4m	m	0					1,566.0	54,928,566,000					
_	Sub	total			26.511.762.933	1	32,308,182,500	-	54,928,566,000					
	fot	al	1		33,249,226,809		41,535,790,472		66.515.817.686					
	rat	10		1	11000		244		2 001					

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

#### Source: Study Team

	Alter	native			Altrenative-1		Altrenative-2		Altrenative-3
	Pile	Type		Cast-ii	n-place pile D=1.2m	Cast-i	n-place pile D=1.5m	Cast-i	n-place pile D=2.0m
	Plan of	Pile C	`ap	ישריין איז				900	. 22110000 .271 0 0 0 0 0 0
D: 1				L=46.5m	n=12nos	L=46.5m	n=9nos	L=46.5m	n=6nos
Displace Dila Par	ement		him kN	0X-1	$1.5 \ge 0.1 - 1.5$ (UK) $1.3 \ge 0.1 - 1.5$ (UK) $1.3 \ge 0.1 - 1.5$ (UK)	OX-1	$-2 \ge 0a - 13$ (OK) $-42005 \le P_{0} - 44005$	0X-1	$-3 \ge 6a - 13$ (OK) $-42002 \le P_0 - 42460$
The Rea			nos	D25	-24nos(minimum)	D28	$=24 \operatorname{nos}(\operatorname{minimum})$	1 nmax -	$D_{32-32nos}$
	Mu		kN.m	023	1050.2	D20	1587.1		3320.4
	Mn		kN.m		2124.6		3464.2		8904.2
Pile body	fs		-		2.02		2.18		2.68
	<b>F</b>	fc	N/mm ²		2.6 < 11.2		2.3 < 11.2		1.9 < 11.2
	Extreme	fs	N/mm ²		30 < 202		26 < 202		21 < 182
	Cost E	estimat	te	Material	Cost	Material	Cost	Material	Cost
Iten	n	unit	Unit cost (VND)	Quantities	(VND)	Quantities	(VND)	Quantities	(VND)
	Pile	e 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 Co	ncrete	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
Excava	ation	m ³	318,066	110.0	34,987,260	120.0	38,167,920	133.0	42,302,778
Coffer	dam	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
	Foun	dation							
Bord Pile	1.2m	m	14,553,000	576.0	8,382,528,000	100 0			
(Diameter)	1.5m	m	17,423,000			432.0	7,526,736,000	200.0	7 074 704 000
	2.0m Sub	m total	27,545,000		8 382 528 000		7 526 726 000	288.0	7,874,784,000
	T	otal			11 985 426 880		11 448 238 579		12 210 741 367
	ra	tio			1.047		1.000		1.067
	Excavation         m ³ 3           Cofferdam         ton         24,7!           Sub total         Sub total           Bord Pile         1.2m         m           1.5m         m         17,4'.2           2.0m         m         27,3'           Sub total         Total           ratio         Evaliation					Mo	st Recommended		

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

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	d combination and load factor	
ad combination and Load factor for Abutment		

Load Combina	tion			Pe	erman	ent					Tran	sient			14/ 4	MC	14/1	ED				то	CE.	FO	ст	CV
Limit State		DC	DD	DW	EH	EV	ES	EL	LL	IM	CE	BR	PL	LS	WA	vv 3	VVL	FK	TU	CR	SH	IG	35	EQ	U	0
	max	1.25	1.80	1.50	1.50	1.35	1.50	1.75	-			-	-	-	-	-	-	-	-				-	-	-	-
STRENGTH-T	min	0.90	0.45	0.65	0.9	0.90	0.75	1.75	-			-	-	-	-	-		-	-					-		-
	max	1.25	1.80	1.50	1.50	1.35	1.50	-	•	•	•	•	-	-	-	•	•	-	-	•	•	•	•	-	•	-
STRENGTH-I	min	0.90	0.45	0.65	0.90	0.90	0.75	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		
	max	1.25	1.80	1.50	1.50	1.35	1.50	1.35	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		
STRENGTH-III	min	0.90	0.45	0.65	0.9	0.90	0.75	1.35	-			-	-	-	-	-		-	-					-		-
EVEDEME	max	1.25	-	1.50	1.5	1.35	1.50	0.50	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
EXIRENE	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 combinat	ion a	nd Lo	ad fac	tor fo	r Pier																					
Load Combina	tion			Pe	rmane	ent					Tran	sient			10/0	we	\\/1	ED				TG	QE	E0	СТ	CV
Limit State		DC	DD	DW	EH	EV	ES	EL	LL	IM	CE	BR	PL	LS	WA	W3	VVL	FK	TU	CR	SH	10	3	Ľ	01	0.
STRENGTH I	max	1.25	1.80	1.50	1	-	-	1.75	-	-	•	-	-	-	•	-	-	ł	-	-	•	•	•	-	-	-
STRENGTI-1	min	0.90	0.45	0.65	-	-	-	1.75	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	max	1.25	1.80	1.50	1	-	-	-	-	-		-	-	-	-	-	-	I.	-	-	-	-	-	-	-	-
STRENGTIFE	min	0.90	0.45	0.65	ŀ	-	-	-	-	-		-	-	-	-	-	•	I.	-	-	-	-	-	-		
	max	1.25	1.80	1.50	1	-	-	1.35	-	-	•	-	-	-	-	-	-	ı.	-	-	•	•	•	-		
3TKENGTI-III	min	0.90	0.45	0.65	ŀ	-	-	1.35	-	-		-	-	-	-	-	•	I.	-	-	•	-	-	-	-	-
EVTREME	max	1.25	-	1.50	-	-	-	0.50	-	-		-	-	-	-	-	-	1	-	-				-	-	-
EATREIME	mini	0.90	•	0.65	1	-	-	0.50	-	-	•	-	-	-	-	-	-	•	-	-	•	•	•	-	-	-
SERVICE		1.00	1.00	1.00	-	-	-	1.00	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

BR : Vehicular braking force

CE: Vehichlar centrifugal forth

CT: Vehicular collision force

DC: Component and Attachment DD : Downdrag

- DW : Wearing Surfaces and Utilities
- EH : Horizontal Earth Pressure EL : Locked-in Erection Stress Vertical Earth Pressure

Earth surcharge load

CV: Vessel collision force

CR : Creep

- EQ : Earthquake FR: Friction
- IM: Vehicular dynamic load allowance LL: Vehicular live load
- LS : Live load surcharge TU: Uniform temperature
- PL: Pedestrian live load SE : Settlement
  - WL: Wind on live load
- SH : Shrinkage TG : Temperature gradient
- WA: Water load and stream pressure
- WS : Wind load on structure

#### Source: Study Team

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

EV:

ES :

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:



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).

Study Type	Type-1 ₋₁	Tupe-2	Туре-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.L5.0 (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 ( <b>Separate Type</b> )	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

Table 8.4.4-9 The results of Study of Foundation Type

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





#### 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.

Site Condition 2)

> 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.

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
Study Team	Variation2	

	Table 8.4.5-1	Site (	Condition	for	Study	of v	Type-1
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Source:

Figure 8.4.5-1 Pile Cap Elevation of Variations 2

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

## 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.

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cutantity     Unit Cost       arres     Quantity       arres     17.104       arres     17.104       arres     17.104       arres     17.125.811       arres     17.104       arres     2.135.61       arres     2.135.61       arres     2.1408       arres     2.1408       arres     2.1408       arres     2.1408       arres     2.1408       arres     2.1408       arres</td> <td>g Ratio (Pile Reaction/Pile Berning) is 0.96       y colfer dam work for foundation construction is necessary.       ber of Pile Reaction.       neare to negative friction.       ente to negative friction.       can to negative friction.       name to negative friction.       control of the foundation construction is necessary.       control of the foundation.       control of the foundation.</td> <td>g Ratio (Pile Reaction/Pile Benning) is 0.96       y colfer dam work for foundation construction is necessary.       ber of Pile Bould magnity from I flones to 25 nos for       same to negative friction.       name to 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                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                              |

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.

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

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

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

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

## 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.

Stel Vew Pik arrangener			STATION STA.8+77~STA.9+944, Pier number : P61~P75					
State View Price anagement         Description of price in the prior of price in the Standard with Collection Construction Club of the Collection Total number of price in the prior of price in the Standard with Collection Total number of price in the prior price in the prior of price in the prior of price in the prior	Evaluation Items		Alternative-1		Alternative-2			
Ské Vew Pik arangeneer     Total kegén of pik     : 43.5 m.       Ské Vew Pik arangeneer     Image of pik     : 10.0 mm       Ské Vew Pik arangeneer     Image of pik     : 20.m       Ské Vew Pik arangeneer     Image of pik     : 20.m       Ské Vew Pik arangeneer     Image of pik     : 20.m       Sweismal Aspect and Stability     Image of pik     : 20.m       Sweismal Aspect and Stability     Image of pik     : 20.m       Sweismal Aspect and Stability     Image of set State of the Restore Pik Bestrage 10.0%.     Image of the Restore Pik Bestrage 10.0%.       Construction Cost (for Foundation     Image of set State of the Restore Pik Bestrage 10.0%.     Image of the Restore Pik Bestrage 10.0%.       Construction Cost (for Foundation     Image of set State of the Restore Pik Bestrage 10.0%.     Image of the Restore Pik Bestrage 10.0%.       Construction Cost (for Foundation     Image of set State of the Restore Pik Bestrage 10.0%.     Image of the Restore Pik Bestrage 10.0%.       Construction Cost (for Foundation     Image of set State of the Restore Pik Bestrage 10.0%.     Image of the Restore Pik Bestrage Bestrage 10.0%.       Construction Pik and Pixel     Image of set State of the Restore Pik Bestrage Bestrage 10.0%.     Image of set State of the Restore Pik Bestrage Bestrage 10.0%.       Construction Cost (for Foundation     Image of set State of the Restore Pik Bestrage 10.0%.     Image of set State of the Restore Pik Bestrage Bestrage 10.0%.     Image o			Steel Pipe Pile Foundation with Sheet Pile Cofferdam Diameter of pile : 1100 mm Total number of pile : 16		Cast In Place Pile Foundation with Cofferdam Diameter of pile : 1500 mm Total number of pile : 9			
Sike Vew Fik arrangement     Image: Sign of the besing bits (Procession Plane and Point)       Sike Vew Fik arrangement     Image: Sign of the besing bits (Procession Plane and Point)       Sike Vew Fik arrangement     Image: Sign of the besing bits (Procession Plane and Point)       Sinceural Aspect and Stability     Image: Sign of the besing bits (Procession Plane and Point)       Senceural Aspect and Stability     Image: Sign of the besing bits (Procession Plane and Point)       Senceural Aspect and Stability     Image: Sign of the besing bits (Procession Plane and Point)       Senceural Aspect and Stability     Image: Sign of the besing bits (Procession Plane and Point)       Construction Cost (for Fromdate)     Image: Sign of the basing bits (Procession Plane and Point)       Construction Plane and Point     Image: Sign of the basing bits (Procession Plane and Point)       Construction Cost (for Fromdate)     Image: Sign of the basing bits (Procession Plane and Point)       Construction Plane and Point     Image: Sign of the basing bits (Procession Plane and Point)       Construction Plane and Point     Image: Sign of the basing bits (Procession Plane and Point)       Construction Plane and Point     Image: Sign of the basing bits (Procession Plane and Point)       Construction Plane and Point     Image: Sign of the basing bits (Procession Plane and Point)       Construction Plane and Point     Image: Sign of the basing bits (Procession Plane and Point)       Construction Plane and Point     Image: Sign of the basing bits (P			Total length of pile : 43.5 m Thickness : 19.0 mm		Total length of pile : 42.0 m			
Image: Structural Aspect and Stability     I	Side View Pile arrangement							
- Large number of Steel Sheet Piles and Steel pipe pales         - small number of Steel Sheet Piles and CLP. piles.           - Small number of Steel Sheet Piles and CLP. piles.         - small number of Steel Sheet Piles and CLP. piles.         - small number of Steel Sheet Piles and CLP. piles.           Construction Cost (for Foundation)         40         Quantity         Unit Cost (VND)         Total (1,000VND)         Pile Cap Concrete         308a12         5.367.864         1.075.029 (1,000VND)         Pile Cap Concrete         276m3         5.867.864         1.67.03 (1,000VND)           Pile Cap Concrete         12m3         1,172.811         20.686         40.712 (2007 rdmm)         Pile Cap Concrete         11m3         1.723.811         6.602.00         15.312 (2007 rdmm)         16         Binding stone         22.337         6.602.00         15.312 (2007 rdmm)         12m3         318.066         40.712 (2007 rdmm)         Executation         120m3         318.066         38.16 (2007 rdmm)         9000         24.798.638         2.2.31.877 (22.371         24.398.638         2.2.31.877 (22.371         070m3         070m3         070m3         072.22.37         42.398.688         2.006.773         0.007m3         070m3         07.22.237         42.398.68         2.006.773         0.07m3         070m3         07.22.237         42.398.68         0.017m3         0.007m3         0.0	tructural Aspect and Stability	10	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.91. - Temporary cofferdam work for foundation construction is necessary.	6	Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.91. - Prine Dearing State (Pile Reaction/Pile Bearing) is 0.91. - Temporary cofferdam work for foundation construction is necessary.	6		
Construction Cost (for Foundation)         Image: Construction Plan and Period         Image: Con	- Large number of Steel Sheet Piles and steel pipe piles			- Small number of Steel Sheet Piles and C.I.P. piles.				
Provide bility is inferior due to large temporary cofferdam work in the sea.         -Workability is inferior due to large temporary cofferdam work in the sea.           Construction Plan and Period         10         Pile work         9 days           Pile work         13 days         8         Pile work ² 20 days           Pile Cap         29 days         Pile Cap         29 days           Coltram & Column & Column Beam         23 days         Colume & Colume Beam         23 days           Total         74 days         Total         81 days	truction Cost (for Foundation)	40	Quantity         Unit Cost (VND)         Total (VND)           Pile Cap Concrete         303rd         5,867.864         1,775.029           Pile         696m         15,728.273         10.946.878           Izara Concrete         12m3         1,723.811         20.686           Binding stone         24m3         696.000         16.704           Exaa valion         12m3         318.066         40.712           Cofferdam         93ron         2.2377         436.810           Driving         702m         622.237         436.810           Total         1.543.993         Ratio         1.418	16	Quantity         Unit Cost (VND)         Total (LOONND)           Pile Cap Concrete         276m3         5.867.864         1.617.183           Pile         378m         17.514.043         6.620.308           Leas Concrete         11m3         1.722.811         18.902           Blinding stone         22m3         666.000         15.312           Exa avaion         120m3         318.066         38.166           Cofferdam         90ton         24.798.635         2.231.977           Driving         678m         622.237         421.977           Cotal         10.903.086         Ratio         1.0093	40		
Construction Plan and Period         I0         Cofferdam Work         9 days         Cofferdam Work         9 days           Construction Plan and Period         10         Pile work         13 days         8         Pile work ² 20 days           Pile Work         13 days         9 days         Pile work ² 20 days           Colum & Column Beam         23 days         Pile work         23 days           Total         74 days         Total         81 days			- Workability is inferior due to large temporary cofferdam work in the sea.		- Workability is inferior due to large temporary cofferdam work in the sea.			
	Construction Plan and Period	10	Cofferdam Work     9 days       Pile work     13 days       Pile Cap     29 days       Column & Column Beam     23 days       Total     74 daws	8	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.	Maintenance	15	- Superior in Maintenance with small number of maintenance points.	9	- Superior in Maintenance with small number of maintenance points.	9		
STEP Clearance     89%(preliminary Estimate)       10     - Large number of steel pipe pile acceptance a contribution         0   - 27% (Preliminary Estimate)   - 27% (Preliminary Estimate) - small number of Cast in place pile acceptance a contribution	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			
Aesthetics     5     -Slender appearance of Pier       5     -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.	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.			
New Technology     S     Steel Pipe Pile Foundation is new technology in Vietnam.     S     Cast in pile (D=1.5m) is no special technology in Vietnam.	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.			
Environmental Aspect 5 - Superior in Environmental aspect with small number of excavated soil & bentonite values for surplus soil and discharging water is necessary.	Environmental Aspect	5	- Superior in Environmental aspect with small number of excavated soil & bentonite water.	5	<ul> <li>Environmental measures for surplus soil and discharging water is necessary.</li> </ul>	2		
Evaluation 100 - Superior in Environmental aspect with small number of excavated soil & bentonite water. - Minimum Construction period with efficient workability. 62 - Construction cost is lowest in area to take no account of negative friction.	Evaluation	.00	<ul> <li>Superior in Environmental aspect with small number of excavated soil &amp; bentonite water.</li> <li>MinimumConstruction period with efficient workability.</li> </ul>	62	Environmental measures for surplus soil and discharging water is necessary.     Construction cost is lowest in area to take no account of negative friction.			
Not Recommended Most Recommended			Not Recommended		Most Recommended			

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

Source : Study Team

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

## (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.

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				

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

Source: Study Team



Source: Study Team



Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

## 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



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.

	····	10	8	00	10	9	~	4	ŝ	18
Allemative-4 CLP. Pile Muhi Cohum Foundation with Steel Sheet Pile Cofferdan	Diameter of pile :: 1500 mm Teral muther of pile :: 12 Teral insert of pile :: 292.5 m	Pible Bearing Existic Ofick Reaction Pile Bearing ) is 0.89, State and a state of the Characterization of the Characterization - State and a state of the Characterization and a state of the Characterization - Transportery Present Characterization and a state of the Characterization is necessary. - Vortradificre can be reduced date to bury mary CPT and Characterization.	Quartify         Card Oracle         Tand Oracle           Piller         Quartify         QND         Q	-Workshifty is superior with present form and steed pipe pile     Present Form Work 18 days     Pile York? 29 days     Pile York? 29 days     Pile Cop     Pile Cop     Total     Total     Total	<ul> <li>Inferior in Maintenance due to large precast form and pile cap on the sea.</li> </ul>	11% (preliminary Estimate) - snall number of Cast in place pile acceptance a contrbution	- Pile cap not to be exposed above water level.	- Multi Cohum Foundation with Cast in pile (D=1.5n) is no special technology in Vietnam	-Environmental measures for stuplus soil and discharging water is necessary.	- Mäimmu Coastaction cost with coastaction persol. Mainum Coastaction cost with coastantedia
		4	0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	10	10 10	9	0	6	т	76
Alternative-3 Cast In Place Multi Column Foundation	Diameter of rple :: 1500 mm Total truther of rple :: 22.0 m Trad truther of rple :: 22.0 m	<ul> <li>Pile Beering Earlo (Pile Reaction/Pile Beering) is 0.92.</li> <li>Steel chains in the Street of Control Cospection chalmkon.</li> <li>Temporty Precent Control Pile cap, construction is necessary.</li> <li>Protection of Ship allision for pile is necessary.</li> </ul>	Quantify (Partic)         Quantify (NSD)         Total (100002)           Perceptionnese         23mm         2323.25         99.28           Cohman Connette         23mm         232.35.25         99.28           Cohman Connette         23mm         23.23.25         99.28           Cohman Connette         23mm         23.23.25         99.28           Discretize         0.0000.26         0.0000.26         0.0000.26           Discretize         0.0000.26         0.0000.26         0.0000.26           Discretize         0.000         17.51.06         1.0000.26           Discretize         0.000         10.0000.26         1.0000.26           Discretize         0.000         10.2000.66         1.0000.76           Discretize         0.000         10.2000.66         1.0000.76           Discretize         0.0000         10.2000.66         1.0000.76           Discretize         0.0000         0.0000.76         1.0000.76           Discretize         0.0000         10.0000.66         1.0000.76           Discretize         0.0000         0.0000.76         1.0000.76           Discretize         0.00000         0.0000.76         1.0000.76           Discretize         0.00	-Werkshilty is superior due to no edificultur work. Present From W onk 15 days Present From W onk 22 days Plac work ² 29 days Plac work 20 20 days Cohum Ream 16 days Total 70 days	- Inferior in Maintenance due 10 large precast form and pile cap on th sea.	11% (pre liminary Estimate) - small number of Cast in place pile acceptance a contribution	-Large pile cap to be exposed above water level is large.	<ul> <li>- Muthi Column Foundation with Cast in pile (D=1.5m) is no special technology in Vietnam</li> </ul>	-Environmental measures for surplus soil and discharging water is necessary.	-Mäännun Construction cost with construction period. Net Recommended
m		00	85 11 10 10 10 10 10 10 10 10 10 10 10 10	9	10	4	5	m	e	55
Alemative-2 Cast in Place Pile Foundation with Steel Pipe Pile Cofferds	Diameter of pile :: 100 mm Total muther of pile :: 12 Tread length of pile :: 40.0 m 	<ul> <li>Die Beumig Rake (Pie Reaction Pie Rearing) is 0.22.</li> <li>Temporey Offer dam verk for foundation construction is necessary 1args number of - Small number of CLP, piles</li> </ul>	Cutrify         Und Cot (1000)         Total           Photocree         2014         (1000050)         9313           Photocree         20143         9313         9313           Photocree         20143         9314         9314           Photocree         20143         9314         9314           Photocree         20143         9314         9314           Photocree         20143         20143         9405           Photocree         20143         20143         9405           Control Concree         1140         172441         215           Control Concree         11306         03200         031           Exerction         11204         11306         0314           Conflictum         11306         21243         120105         0314           Definition         11306         03149         120105         0314           Definition         11600         03149         120105         120105	-Workshilly is infraved due to large temporary cofferdium work in the set for the set of	-Superior in Maintenance with small number of maintenance points.	- 32% (Prelimin ary Estimate) - small number of Cast in place pile acceptance a contribution	-Slender appearance of Pier - Pile cap not to be exposed above water level	- Cast in pile (D=1.5m) is no special technology in Vietnam	-Euvironmental measures for surphis soil and discharging water is	- Vordabability is inférier due to temporary codfar dam work and CLP wor in the seaso- - Environmental messaures for surphus soil and discharging worter is excernented soil & bernoniet work mentalised
	1 <del>4</del>	00	7 7 0 0 7 7 7 0 0 U	4	10	10	5	en	5	<del>1</del> 6
Alternative-1 Steel Pipe Pile Foundation with Steel Pipe Pile Cofferdam	Damker of pile :100 mm Total mutter of pile :20 Total mutter of pile :20 Thickness :120 mm Thickness :120 mm	<ul> <li>Pile Bearing Batto (Pile Reaction Pile Bearing) is 0.31.</li> <li>Temporary Citer dans vork for foundation construction is necessary.</li> <li>Large number of</li> <li>Large number of</li> <li>Large number of</li> </ul>	Plant         Quantity         Under Under Version         Quantity         Under Version         Quantity         Under Version         Quantity         Under Version         Quantity         Quantity </th <th>-Wockedulty is infervite there to large temporary coefficientian work in the set Coefficientian Work 2014 and 2014</th> <th>–Superior in Maintenance with small number of maintenance points.</th> <th>- 85% (Perliminary Estruate) - Large number of steel pipe pile acceptance a contribution</th> <th>– Slender appearance of Pier – Pile cap not to be exposed above water level</th> <th>-Steel Pipe Pile Foundation is new technology in Vietnam</th> <th>- Superior in Environmental aspect with small number of excavated soil &amp; bentonite water</th> <th><ul> <li>- Construction cost is lightest</li> <li>- Workability is inferior due to temporary coffer dam work in the sea.</li> <li>- More and the sea is a sea in the sea is a sea in the sea is a sea in the sea in the sea is a sea in the sea is a sea in the sea in the sea is a sea in the sea in th</li></ul></th>	-Wockedulty is infervite there to large temporary coefficientian work in the set Coefficientian Work 2014 and 2014	–Superior in Maintenance with small number of maintenance points.	- 85% (Perliminary Estruate) - Large number of steel pipe pile acceptance a contribution	– Slender appearance of Pier – Pile cap not to be exposed above water level	-Steel Pipe Pile Foundation is new technology in Vietnam	- Superior in Environmental aspect with small number of excavated soil & bentonite water	<ul> <li>- Construction cost is lightest</li> <li>- Workability is inferior due to temporary coffer dam work in the sea.</li> <li>- More and the sea is a sea in the sea is a sea in the sea is a sea in the sea in the sea is a sea in the sea is a sea in the sea in the sea is a sea in the sea in th</li></ul>
Max. Point		10	40	10	15	10	5	2	2	100
Evaluation Items	Side View	Structural Aspect and Stability	Construction Cost (for Foundation)	Construction Plan and Period	Maintenance	STEP Clearance	Aesthetics	NewTechnology	Environmental Aspect	Evaluation

*2.Including for Pile top treatmen

Note) *1. Including for Pile top treatment

Source : Study Team

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

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT

#### 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  $\sigma ck$ : 28N/mm2

- b) Reinforcement
  - Reinforcement : SD345
- c) Back filling material

Density	:	19kN/m3
Internal friction angle	:	$30^{\circ}$

2) Reclamation plan

There is reclamation plan from A1 to P60 and P83 to A2.

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

	Elavation of	Elavation of	Reclamation thickness		Elavation of	Elavation of	Reclamation thickness
	Design Level	bottom of pilecap	from bottom of pile cap		Design Level	bottom of pilecap	from bottom of pile cap
	(m)	(m)	(m)		(m)	(m)	(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	P/1		-5.64	-
P29	3.10	-2.17	5.3	P/2		-5.48	-
P30	3.10	-2.40	5.5	P73		-5.80	
P31	3.10	-2.30	5.4	P/4		-5.62	
P32	3.10	-2.26	5.4	P/5		-5./8	
P33	3.10	-2.15	5.3	P/9	-	-4.18	-
P34	3.10	-1.98	5.1	P80	-	-4.63	-
P35	3.10	-1.93	5.0	P81		-4.31	-
P30	3.10	-2.12	5.2	P82	- 2.40	-5.99	-
P3/	3.10	-1.94	5.0	P83	3.10	-4.1/	1.3
P38	3.10	-1./0	4.9	P84	3.10	-2.70	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	-2.0/	5.0
P41	3.10	-1.72	4.6	P87	3.10	-2.13	5.2
P42	3.10	-1.54	4.6	AZ	3.10	-0.61	3.1

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

Source: Study Team

3) Blockout at parapet

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

2-1. Detail of blockout



## 2-2. Affixed articles

a) Electrical cable

## (Technical parameter from THE NORTHERN ELECTRIC CORPORATION)

- External diameter:93mmCable weight:16.690kg/mNumber of cable:2nos
- b) Optical cable

c)

## (Technical parameter from THE NORTHERN ELECTRIC CORPORATION)

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

 $Concrete \qquad : \ \sigma ck{=}28N/mm2$ 

- b) Reinforcement Reinforcement :SD345
- 2) Reclamation plan

Refer to Section 8.1.4.1

## 3) Dimension of Substructure

			Column	Pile		Thislansson	Thickness of
		<b>-</b>				I hickness of	consolidation
Span length of	Number of	I otal height	Dimensions of			reclamation	layer
Superstructure	Substructure	of Pier	column	I ype of pile	Diameter	(From bottom of	(From bottom of
						plie cap)	pile cap)
	A1	8.0m	28.5m × 2.5m		0.8m	3.8m	24.7m
	P1	6.0m	7.8m × 2.5m			5.1m	23.8m
5@60.0	P2	7.5m	4.5m × 2.5m			5.9m	30.7m
=300.0m	P3	8.5m	4.5m × 2.5m			6.2m	25.9m
	P4	8.5m	4.5m × 2.5m			5.4m	23.4m
	P5	8.5m	4.5m × 4.0m			4.8m	26.8m
	P6	8.5m	4.5m × 2.5m			4.0m	29.7m
5@60.0	P7	9.0m	4.5m × 2.5m			3.9m	26.1m
=300.0m	P8	9.5m	4.5m × 2.5m			3.9m	23.1m
	P9	10.0m	4.5m × 2.5m			4.0m	23.1m
	P10	10.0m	4.5m × 4.0m			3.9m	23.9m
	P11	10.5m	4.5m × 2.5m			4.2m	23.9m
51.5+4@60.0	P12	10.5m	4.5m × 2.5m			4.2m	28.9m
=291.5m	P13	10.5m	4.5m × 2.5m			4.4m	28.0m
	P14	10.5m	4.5m × 2.5m			4.6m	26.2m
	P15	10.0m	4.5m × 4.0m			4.4m	30.8m
-	P16	10.0m	4.5m × 2.5m			4.4m	35.7m
5@60.0	P17	10.0m	4.5m × 2.5m			4.6m	37.6m
=300.0m	P18	10.0m	4.5m × 2.5m			4.8m	36.0m
	P19	9.5m	4.5m × 2.5m			4.5m	36.9m
	P20	9.5m	4.5m × 4.0m			4.8m	35.8m
5@60.0 =300.0m	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	Ctacl size sile		4./m	27.6m
	P24	9.0m	4.5m × 2.5m			4.9m	26.7m
	P25	8.5m	4.5m × 4.0m	Steel pipe pile	1.1m	4./m	27.0m
F@ 60 0	P20	8.5m	4.5m x 2.5m			4.7m	27.4m
-200.0m	P27	8.5m	4.5m x 2.5m			4.9m	23.2m
=300.011	F20	0.0111	4.5m x 2.5m			5.111	27.011
	P29 P20	8.5m	4.5m x 4.0m			5.3m	28.7m
	P31	8.5m	4.5m x 2.5m			5.0m	23.4m
5@60.0	P32	8.5m	4.5m x 2.5m			5.4m	25.7m
-300 0m	P33	8.5m	4.5m × 2.5m			5.4m	20.0m
-000.011	P34	8.5m	4.5m × 2.5m			5.0m	20.3m
	P35	8.5m	4.5m × 4.0m	u -		5.0m	34 1m
	P36	9.0m	4.5m × 2.5m			5.2m	27.8m
_	P37	9.0m	4.5m x 2.5m			5.0m	29.1m
5@60.0=300.0m	P38	9.0m	4.5m × 2.5m			4.9m	24 1m
	P39	9.5m	4.5m × 2.5m			5.2m	22.6m
	P40	9.5m	4.5m × 4.0m			5.1m	33.0m
	P41	9.5m	4.5m × 2.5m			4.8m	28.0m
5@60.0	P42	9.5m	4.5m × 2.5m			4.6m	29.0m
=300.0m	P43	10.0m	4.5m × 2.5m			5.0m	22.1m
	P44	10.0m	4.5m × 2.5m			4.8m	21.8m
	P45	10.0m	4.5m × 4.0m			4.7m	23.3m
	P46	10.5m	4.5m × 2.5m			4.9m	25.6m
4@60.0+58.36	P47	10.5m	4.5m × 2.5m			4.7m	34.8m
=298.36m	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

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

Source: Study Team

				Pile		Thickness of	Thickness of	
Span of Superstructure	Number of Substructure	Column	Dimensions of column	Type of pile	Diameter	reclamation (From bottom of pile cap)	consolidation layer (From bottom of pile cap)	
	P50	15.0m	4.5m × 4.0m			8.8m	22.6m	
5@60.0 =300.0m	P51	15.0m	4.5m × 2.5m			8.6m	22.9m	
	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	P57	15.0m	4.5m × 2.5m			9.0m	20.5m	
+52.98=285.96m	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	P62	14.0m	4.5m × 2.5m					
+52.98=285.96m	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	P67	13.0m	4.5m × 2.5m				1	
+52.98=285.96m	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	P72	16.5m	4.5m × 2.5m					
+52.98=285.96m	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	P81	17.0m	4.5m × 3.5m	Dored File	1.0111			
+54.8=289.6m	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	
54.0.0000	P85	8.5m	4.5m × 3.5m	Steel nine nile	1.1m	5.3m	5.3m	
54.8+2@60.0	P86	7.5m	4.5m × 3.5m	ereer hihe hile		5.8m	5.8m	
+54.8=229.6m	P87	6.0m	4.5m × 3.5m			5.2m	5.2m	
	A2	8.0m	4.5m × 4.0m		0.8m	3.7m	3.7m	

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

Source: Study Team

## 8.4.6.2 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		P1	P1
Type2	6.0m	7.8×3.5	ns of 5 5 5 5 5 5 5 5 5 5 5 5 5	P87	P87
Туре3	10.5m			P11 P12 P13 P14 P46 P47 P48 P49	P14
Type4	7.5m			P2	P2
Туре5	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	4.5×2.5		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		Steel pipe pile	P52.P53	P52
Type12	7.5m			P86	P86
Type13	8.5m 4.5×3.5			P85	P85
Type14	13.5m			P83	P83
Type15	8.5m			P5 P25 P30 P35	P5,P25,P30,P35
Type16	9.5m			P20.P40	P20,P40
Type17	10.0m	15.10		P15.P10.P45	P10,P15,P45
Typc18	10.5m	4.5×4.0		P84	P84
Type19	14.0m			P60	P60
Type20	15.0m			P50.P55	P50,P55
Type21	13.0m			P67 P68.P69	P69
Type22	13.5m	15005		P64.P66	P66
Type23	14.0m	4.5*2.5		P61 P62 P63	P61
Type24	15.0m			P71	P71
Type25	16.5m			P72	P72
Type26	17.0m	1		P81 P82	P81 P82
Type27	18.5m	4.5×3.5	Bored Pile	P73 -	P73
Type28	19.0m			P80	P80
Type29	20.0m			P74	P74
Type30	13.5m			P65	P65
Type31	14.0m	45.40		P70	P70
Type32	20.0m	4.9×4.0		P79	P79
Type33	21.5m			P75	P75

## Table 8.4.6-4 Grouping of Pier

Source: Study Team

				0								
				Type1	Type2	Type3	Type4	Type5	Type6	Type7		
		Thickness	at joint	-	-	3.5	3.5	3.5	3.5	3.5		
	Dimension	Length of ove	rhanging	-	-	1.65	1.65	1.65	1.65	1.65		
		width	l I	-	-	2.5	2.5	2.5	2.5	2.5		
Beam	Up	per side		-	-	D32-12nos	D32-12nos	D32-12nos	D32-12nos	D32-12nos		
	Lo	wer 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		
	Dimension	Plan	e	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		
	Dimension	Heigh	nt	3.5	3.5	8.0	5.0	6.0	6.5	7.0		
Column	Reinforcement	Longitudinal		D16 ctc250	D16 ctc250	D22 ctc125	D16 ctc125	D16 ctc125	D16 ctc125	D20 ctc125		
	for bending	Transverse		D16 ctc250	D16 ctc250	D22 ctc250	D16 ctc250	D16 ctc250	D16 ctc250	D20 ctc250		
	Reinforcement	Longitud	linal	D16-10nos	D16-10nos	D16-10nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos		
	for shear	Transve	erse	D16-4nos	D16-5nos	D16-4nos	D16-4nos	D16-4nos	D16-4nos	D16-4nos		
	Dimension	Plane Thickness		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		
	Dimension			2.5	2.5	2.5	2.5	2.5	2.5	2.5		
		Linner eide	1	D25ctc250	D25ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250		
		Opper side	2	-	-	-	-	-	-	-		
	Longitudinal	Longitudinal	Longitudinal	Lower eide	1	D30ctc125	D30ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125
Pile cap		Lower side	2	-	-	-	-	-	-	-		
		Reinforcement	for shear	D16-10nos-ctc500								
			1	D16ctc250	D16ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250		
		Upper side	2	-	-	-	-	-	-	-		
	Transverse	Laura a sida	1	D20ctc125	D20ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125		
		Lower side	2	-	-	-	-	-	-	-		
		Reinforcement	for shear	D16-10nos-ctc500								

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

				Type8	Type9	Type10	Type11	Type12	Type13	Type14
		Thickness	at joint	3.5	3.5	3.5	3.5	3.5	3.5	3.5
	Dimension	Length of ove	rhanging	1.65	1.65	1.65	1.65	1.65	1.65	1.65
		width	1	2.5	2.5	2.5	2.5	3.5	3.5	3.5
Beam	Up	per side		D32-12nos	D32-12nos	D32-12nos	D32-12nos	D32-18nos	D32-18nos	D32-18nos
	Lo	wer side		D20-12nos	D20-12nos	D20-12nos	D20-12nos	D20-18nos	D20-18nos	D20-18nos
	Side			D22-13nos						
	Reinforce	ment for she	ar	D20-4nos-ctc200						
	Dimension	Plane	9	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
	Dimension	Height		7.5	12.0	12.5	13.0	5.0	6.0	11.0
Column	Reinforcement	Longitud	linal	D20 ctc125	D35 ctc125	D35 ctc125	D35 ctc125	D16 ctc125	D16 ctc125	D16 ctc125
Column	for bending Transverse		erse	D20 ctc250	D35 ctc250	D35 ctc250	D35 ctc250	D16 ctc250	D16 ctc250	D16 ctc250
	Reinforcement	Longitud	linal	D16-7nos						
	for shear	Transve	erse	D16-4nos	D16-4nos	D16-4nos	D16-4nos	D16-5nos	D16-5nos	D16-5nos
	Dimonsion Pla		9	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
	Dimension	Thickness		2.5	2.5	2.5	2.5	2.5	2.5	2.5
		Lippor sido	1	D30ctc250						
		Opper side	2	-	-	-	-	-	-	-
	Longitudinal	Lower side	1	D35ctc125	D38ctc125	D38ctc125	D38ctc125	D35ctc125	D35ctc125	D38ctc125
Pilo con		Lower side	2	-	-	-	-	-	-	-
r lie cap		Reinforcement	for shear	D16-10nos-ctc500						
		Lippor sido	1	D30ctc250						
		Opper side	2	-	-	-	-	-	-	-
	Transverse	Lower side	1	D35ctc125						
		Lower side	2	-	-	-	-	-	-	-
		Reinforcement	for shear	D16-10nos-ctc500						

				Type15	Type16	Type17	Type18	Type19	Type20	Type21
		Thickness	at joint	3.5	3.5	3.5	3.5	3.5	3.5	3.5
	Dimension	Length of over	rhanging	1.65	1.65	1.65	1.65	1.65	1.65	1.65
		width	1	4	4	4	4	4	4	2.5
Beam	Up	per side		D32-19nos	D32-19nos	D32-19nos	D32-19nos	D32-19nos	D32-19nos	D32-12nos
	Lo	wer 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
	Reinforce	ment for she	ar	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200
	Dimension	Plane	9	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
	Dimension	Heigh	ıt	6.0	7.0	7.5	8.0	11.5	12.5	10.5
Column	Reinforcement	Longitud	linal	D16 ctc125	D16 ctc125	D16 ctc125	D16 ctc125	D16 ctc125	D16 ctc125	D32 ctc125
Column	for bending Transv		erse	D16 ctc250	D16 ctc250	c250 D16 ctc250 D16 ctc250 D16 ctc250		D16 ctc250	D16 ctc250	D32 ctc250
	Reinforcement	Longitud	linal	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos
	for shear	Transve	erse	D16-5nos	D16-5nos	D16-5nos	D16-5nos	D16-5nos	D16-5nos	D16-4nos
	Dimension	Plane	9	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
	Dimension	Thickness		2.5	2.5	2.5	2.5	2.5	2.5	2.5
		Lippor side	1	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D28ctc125
		Opper side	2	-	-	-	-	-	-	-
	Longitudinal	Lower aide	1	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D38ctc125	D38ctc125	D32ctc125
Dilo con		Lower side	2	-	-	-	-	-	-	D32ctc125
File cap		Reinforcement	for shear	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D16-10nos-ctc500	D25-21nos-ctc500
		Lippor side	1	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D30ctc250	D25ctc125
		Opper side	2	-	-	-	-	-	-	-
	Transverse	Lower aide	1	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D35ctc125	D32ctc125
		Lower side	2	-	-	-	-	-	-	D32ctc250
	F	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

### THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT

			lable	e 8.4.6-6 Lis	st of reinfor	cement for	each type c	of Pier(1/2)		
				Type22	Type23	Type24	Type25	Type26	Type27	Type28
		Thickness	at joint	3.5	3.5	3.5	3.5	3.5	3.5	3.5
	Dimension	Length of ove	rhanging	1.65	1.65	1.65	1.65	1.65	1.65	1.65
		width	ı	2.5	2.5	2.5	3.5	3.5	3.5	3.5
Beam	Up	per side		D32-12nos	D32-12nos	D32-12nos	D32-18nos	D32-18nos	D32-18nos	D32-19nos
	Lo	wer side		D20-12nos	D20-12nos	D20-12nos	D20-18nos	D20-18nos	D20-18nos	D20-19nos
		Side		D22-13nos						
	Reinforce	ement for she	ar	D20-4nos-ctc200						
	Dimension	Plane	e	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
	Dimension	Heigh	nt	11.0	11.5	12.5	14.0	14.5	16.0	16.5
Column	Reinforcement	Longitud	dinal	D32 ctc125	D35 ctc125	D35 ctc125	D25 ctc125	D25 ctc125	D32 ctc125	D32 ctc125
Column	for bending	Transverse		D32 ctc250	D35 ctc250	D35 ctc250	D25 ctc250	D25 ctc250	D32 ctc125	D32 ctc125
	Reinforcement	Longitud	dinal	D16-7nos						
	for shear	Transverse		D16-4nos	D16-4nos	D16-4nos	D16-5nos	D16-5nos	D16-5nos	D16-5nos
	Dimension	Plane	e	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
	Dimension	Thickne	ess	2.5	2.5	2.5	2.5	2.5	2.5	2.5
		Ling og såde	1	D28ctc125	D28ctc125	D28ctc125	D32ctc125	D32ctc125	D32ctc125	D32ctc125
		Opper side	2	-	-	-	-	-	-	-
	Longitudinal	Laura alda	1	D32ctc125	D32ctc125	D32ctc125	D38ctc125	D38ctc125	D38ctc125	D38ctc125
Dila ana		Lower side	2	D32ctc125	D32ctc125	D32ctc125	D38ctc125	D38ctc125	D38ctc125	D38ctc125
Plie cap		Reinforcement	for shear	D25-21nos-ctc500	D25-21nos-ctc500	D25-21nos-ctc500	D25-10nos-ctc250	D25-10nos-ctc250	D25-10nos-ctc250	D25-10nos-ctc250
		Ling og såde	1	D25ctc125	D25ctc125	D25ctc125	D32ctc125	D32ctc125	D32ctc125	D32ctc125
		Opper side	2	-	-	-	-	-	-	-
	Transverse	Lower eide	1	D32ctc125						
		Lower side	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		
		Thickness	at joint	3.5	3.5	3.5	3.5	3.5		
	Dimension	Length of ove	rhanging	1.65	1.65	1.65	1.65	1.65		
		width	ı	3.5	4	4	4	4		
Beam	Up	per side		D32-19nos	D32-19nos	D32-19nos	D32-19nos	D32-19nos		
	Lo	wer side		D20-19nos	D20-19nos	D20-19nos	D20-19nos	D20-19nos		
		Side		D22-13nos	D22-13nos	D22-13nos	D22-13nos	D22-13nos		
	Reinforce	ement for she	ar	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200	D20-4nos-ctc200		
	Dimension	Plane	e	4.5×3.5	4.5×4.0	4.5×4.0	4.5×4.0	4.5×4.0		
	Dimension	Heigh	nt	17.5	11.0	11.5	17.5	19.0		
Caluma	Reinforcement	Longitud	dinal	D32 ctc125	D16 ctc125	D16 ctc125	D32 ctc125	D32 ctc125		
Column	for bending	Transve	erse	D32 ctc125	D16 ctc125	D16 ctc125	D32 ctc125	D32 ctc125		
	Reinforcement	Longitud	linal	D16-7nos	D16-7nos	D16-7nos	D16-7nos	D16-7nos		
	for shear	Transve	erse	D16-5nos	D16-5nos	D16-5nos	D16-5nos	D16-5nos	]	
	Dimonsist	Plane	e	10.5 × 14.25	10.5 × 10.5	10.5 × 10.5	10.5 × 14.25	10.5×14.25		
	Dimension	Thickne	ess	2.5	2.5	2.5	2.5	2.5		
		Line or side	1	D32ctc125	D28ctc125	D28ctc125	D32ctc125	D32ctc125		
1	Upper		2						1	

D32ctc125

D32ctc250

D38ctc12

D38ctc12

-10nos-cto

D32ctc125

D32ctc125

25-14nos-ctc5

D38ctc12

D32ctc12

D32ctc12

D25-14nos

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

Source: Study Team

Longitudina

Transverse

Pile cap

Lower side

Upper side

Lower side

D38ctc12

D38ctc12

10nos

D32ctc125

D32ctc12

tc12

D32ctc125

D32ctc25

## 8.4.6.4 Design of Foundation of Approach Bridge

## (1) Steel pipe pile (A1 $\sim$ P60,P83 $\sim$ 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

Туре	Yield Strength fy (Mpa)	Tensile Strength fu (Mpa)	Modulus of Elasticity (Mpa)	used
Grade SKK400	235	400	200000	0
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 :  $\sigma$  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;

	100	0 01 110			
		Diamotor	Longth of pilo	Range of SLC coating	thicknose
TYPE	Pier	Diameter	Length of pile	(from top of the pile)	( )
		(m)	(m)	(m)	(mm)
Turne 1 4	DEO			12.0	
Type I-1	F 30	4400	27.0	12.0	40.0
Type1-2	P57	1100	37.0	21.0	12.0
Type1-3	P3			26.0	
Type2	P5	1100	38.0	27.0	12.0
Type3-1	P59			14.0	
Type2.2	D94			22.0	
Type3-2	P04			22.0	
Type3-3	P4	1100	39.0	23.0	12.0
Type3-4	P85	1100	55.0	27.0	12.0
Type3-5	P6	1		30.0	
Type3-6	P2			31.0	
Types-o	D C O			22.0	
Type4-1	F 60	4400	40.0	22.0	40.0
21	P83	1100	40.0	22.0	12.0
Type4-2	P87			25.0	
	P50			23.0	
Type6-5	P51	1		23.0	
.,,	D62			22.0	
	F 32			23.0	
	P31			24.0	
Type6 1	P1	1100	12.0	24.0	12.0
Typeo-T	P54	1100	+∠.U	24.0	12.0
	P38	1		24.0	
Type6-2	P/6			26.0	
Tup-0.2	D20			20.0	
турев-з	P36			20.0	
Туре6-4	P35			34.0	
Tupe7.4	P44			22.0	
Type/-T	P43	1		22.0	
	D30			23.0	
T	F 33			23.0	
Typer-2	P34			23.0	
	P45			23.0	
Type7-3	P24	1100	42.0	27.0	10.0
	P55	1100	43.0	28.0	12.0
Type7-4	D41			20.0	
	F41			20.0	
Type7-5	P56			29.0	
Typer o	P42			29.0	
Type7-6	P40			33.0	
Type7-7	P20			36.0	
Type/-/	D11			24.0	
Typeo-1	PTT			24.0	
Type8-2	P27			25.0	
Type8-3	P23			28.0	
	P29	1100	44.0	29.0	12.0
Type8-4	D37			29.0	
T	D04			23.0	
Турев-в	P21			34.0	
Type8-7	P18			36.0	
	P9			23.0	
Type9-1	P8			23.0	
	P53			23.0	
Turne 0	D10			24.0	
туреэ-2	P10	1100	45.0	24.0	10.0
Type9-3	P32	1100	45.0	26.0	12.0
1960-0	P14			26.0	
Type9-4	P25			27.0	
Type9-5	P28			28.0	
Tupe0.7	D47			25.0	
Type9-7	P4/			35.0	
Type10-1	P33			21.0	
Type10-2	P7			26.0	
Type10-3	P26			27.0	
.,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	P13			28.0	
Tune 10.4	D 10			20.0	
Type10-4	P48			28.0	40.5
	P49	1100	46.0	28.0	12.0
Type10-5	P12			29.0	
Type10.8	P22	1		30.0	
Tupe 10-0	D20			20.0	
Type10-9	P30			32.0	
Type10-6	P19			37.0	
Type10-7	P17			38.0	
Type11-1	P15	4400	17.0	31.0	40.0
Type11-2	P16	1100	47.0	36.0	12.0
Tune 40.4	000	1100	40.0	04.0	10.0
Type12-1	P 86	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

## Table 8.4.6-10 Type of Steel pipe pile

### Source: Study Team

## b) Length of Steel pipe pile

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											
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									
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																		
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						

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

Source: Study Team

#### THE DETAILED DESIGN STUDY FOR LACH HUYEN PORT INFRASTRUCTURE CONSTRUCTION PROJET IN VIET NAM FINAL REPORT

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	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	b	b	b	a,b	a,b	a,b	a,b						
Number of substructure	DCZ	DCO	DEO	DCO	D02	D04	DOG	DOC	D07	40									
	P37	P 30	P39	P00	FOJ	F04	F00	F00	F0/	A2									
	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	0.8									
Thickness 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									
(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																		
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)



field joint details s=1:2 protection fitting s=1:5 end reinforcement band details s=1:5





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

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

pile details s=1:150 shear details s=1:10 section A details pile head rebar details s=1/20 01100 Plan tifting log(2) 前男×12+308 15 upper pills 12000 8 (1) \$14 Roan 111 B.C costing Ithicks aparbund roll etc. 4 (1) 000 110 ction fitting 100 tifting log(2) 100 stopper 3 pi PL-25 x 9 x 50 widdle 2 pile 12000 control area 10000 V footing bottom 液 inear Villenti pile legth 8000 protection fitting lifting lug details s=1:5 field circumferential welding lifting lug (1) lifting lug (2) Lifting log(2) midtle 1 pille 12000 aile. pile field circumferential welding tifting log(1) liser sile 9000 end reinforcoment band field joint details s=1:2 protection fitting s=1:5 end reinforcement band details s=1:5 0-12 protection fitting PL-200 + 12 + 3400 t opening extention lease 80-8 points backing rise ЦĿ end rainforcement hand PL-300×9×3454 Щ 16

Source: Study Team

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

@ 1100 9

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

after walding

- (2) Bored Pile (P61~P75,P79~P82)
  - 1) Material to be used
    - 1-1. Concrete

Concrete :  $\sigma$  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
Туре6	P75	40.0	12

Source: Study Team



Figure 8.4.6-8 Pile Arrangement

Oriental Consultants Co., Ltd., Nippon Koei Co., Ltd., PADECO Co., Ltd. and Japan Bridge & Structure Institute Inc.

## c) Length 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																		
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,d						

#### Table 8.4.6-14 List of Bored pile

(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