CHAPTER 3 SUBSOIL CONDITIONS

3.1 General

This geotechnical investigation covers three sections of Approach Road Area in Hai An side, Bridge Area and Approach Road Area in Cat Hai side in Lach Huyen Port Infrastructure Construction Project as shown in Figure 3.1-1.

The geotechnical investigation consists of the followings:

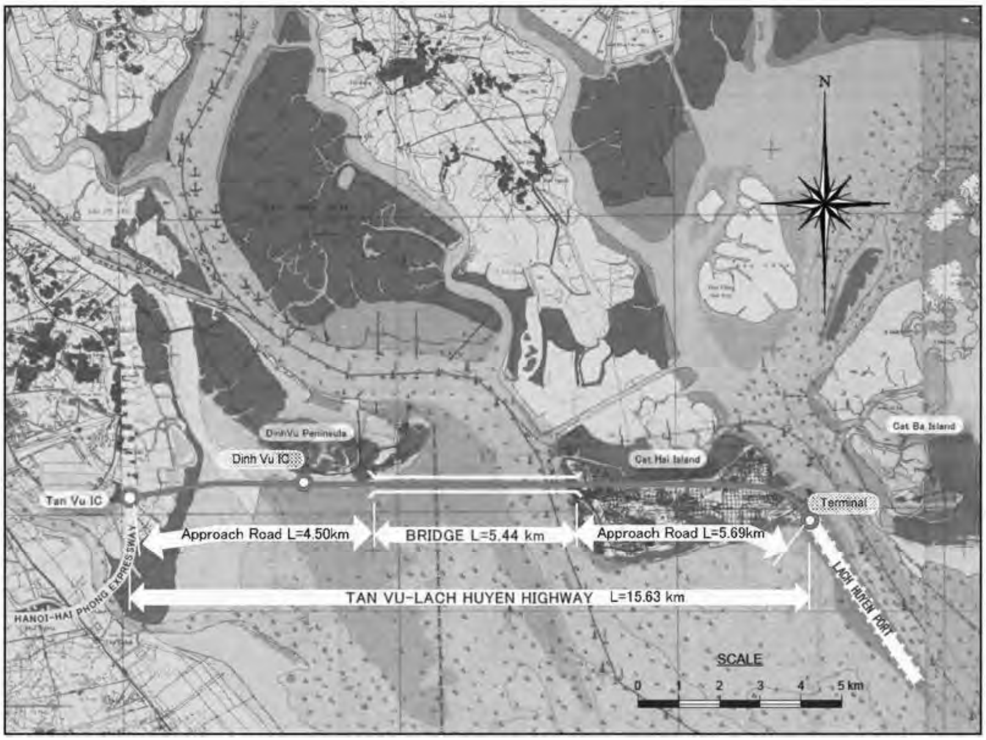
(i) Geotechnical investigation for Approach Road Area in Hai An side (L=4.50 km),

(ii) Geotechnical investigation for Bridge Area (L=5.44 km), and

(iii) Geotechnical investigation for Approach Road Area in Cat Hai side (L=5.69 km).

The borings in Approach Road Area in Hai An side and Cat Hai side were carried out along the centerline of road in 100 ~ 200 m interval. On the other hand, the borings in Bridge Area were conducted at every pier (one borehole/pier for approach bridge and two boreholes/pier for main bridge).

The items and quantities of geotechnical investigation which was carried out in each area are shown in Table 3.1-1.



Source: Preparatory Survey on Lach Huyen Port Infrastructure Construction Project (Road and Bridge Portion)

Figure 3.1-1 Geotechnical Investigation Area

Table 3.1-1 Contents of Geotechnical Investigation

Item	Approach Road Area (Hai An side)	Bridge Area	Approach Road Area (Cat Hai side)	Total
Length	4.50 km	5.44 km	5.69 km	15.63 km
Number of Boring Location	30	92	39	161 locations
Drilling Length	1,178 m	4,173 m	1,201 m	6,552 m
Standard Penetration Test	864	3,693	792	5,349 tests
Undisturbed Sampling	72 samples	29 samples	89 samples	190 samples
Physical Tests of Soil (Specific Gravity, Natural Water Content, Grain Size Analysis, Atterberg Limits)	160	301	199	660 samples
Unit Weight Test of Soil	70	26	85	181 samples
Unconfined Compression Test of Soil	41	14	48	103 samples
Triaxial Compression Test of Soil (UU)	15	12	19	46 samples
Triaxial Compression Test of Soil ($\overline{CU}$)	15	–	18	33 samples
Consolidation Test	70	26	85	181 samples
Number of Field Vane Shear Test Location	30	–	39	69 locations
Accumulated Depth of Field Vane Shear Test	478 m	–	572 m	1,050 m
Specific Gravity, Absorption of Rock	–	18	–	18 samples
Unconfined Compression Test of Rock	–	18	–	18 samples

Source: Study Team

3.2 Stratum Classification

The following main stratum classification is proposed for drawing the soil profile, based on the soil classification and stiffness.

Table 3.2-1 Stratum Classification (Main Layer)

Layer	Soil Discription	Stiffness
D	**Filling soil and agriculture soil**	
1	**Clay** low plasticity	very soft to soft
2	**Sand** with clay, poorly graded	
3	**Clay** low plasticity	very soft to soft
4	**Clay** low plasticity	medium stiff
5	**Sand** with clay, poorly graded	
6	**Clay** low plasticity	stiff to very stiff
7A	**Clay** low plasticity	soft
7B	**Clay** low plasticity	medium stiff
8	**Clay** low plasticity	stiff to very stiff
9	**Clay** low plasticity	medium stiff
10A	**Sand** with clay, poorly graded	
10B	**Sand** with gravel, poorly graded	
11	**Clay** low plasticity	stiff to very stiff
12A	Heavy weathered **silty sand stone**, RQD=0~10%	
12B	Medium weathered **silty sand stone**, RQD=50%	

Source: Study Team

The outline of each main layer is as follows.

(1) Layer-D: Filling soil and agriculture soil

This is dyke filling soil layer, field edge and dike edge established by artificial operation process. Component of layer is clay, sandy clay, medium to stiff state and non-compacted in filling process. This is weak to medium bearing capacity soil layer.

(2) Layer-1: Low plasticity, clay, very soft to soft

Soil layer having brown grey and blackish grey colour, contributes on surface. The baring capacity is low.

(3) Layer-2: Sand with clay, poorly graded, loose to medium dense

This layer having brownish grey, component of layer is sand with silty sand, clay. The bearing capacity is loose to medium.

(4) Layer-3: Low plasticity, clay, very soft to soft

This layer having brownish grey, blackish grey, distributes large in project area. This is weak soil layer and the bearing capacity is low.

(5) Layer-4: Low plasticity, clay, medium stiff

This layer having brownish grey, blackish grey, distributes large in project area, under Layer-3. This is weak soil layer and the bearing capacity is low.

(6) Layer-5: Sand with clay, poorly graded, medium dense

This layer having greenish grey colour, contributes locally in Bridge Area and Approach Road Area in Cat Hai side. The bearing capacity is medium.

(7) Layer-6: High plasticity, clay, stiff to very stiff

This layer having brownish grey, yellowish grey colour, distributes rather large in project area, under Layer-5. This is good soil layer for embankment foundation.

(8) Layer-7A: Low plasticity, clay, soft

This sublayer having grey, brownish grey colour, distributes locally in Hai An side. This is weak soil layer for embankment foundation.

(9) Layer-7B: Low plasticity, clay, medium stiff

This sublayer having brownish grey, blackish grey colour, distributes locally in project area. This is weak soil sublayer for embankment foundation.

(10) Layer-8: Low plasticity, clay, stiff to very stiff

This layer having brownish grey, violetish grey, distributes locally in some sections. This is good soil layer for embankment foundation.

(11) Layer-9: Low plasticity, clay, medium stiff

This layer having brownish grey colour, distributes locally in some sections, under Layer-8. This is weak soil layer for embankment foundation.

(12) Layer-10A: Sand with clay, poorly graded, dense to very dense

This is grayish green colour, distributes rather large in project area. This is good soil layer.

(13) Layer-10B: Sand with clay and gravel, poorly graded, medium dense to very dense

This is grayish green colour, distributes rather large in project area. This is good soil layer.

(14) Layer-11: Low plasticity, clay, stiff to very stiff

The layer has brownish grey, blackish grey colour. Normally this is good soil layer, however it becomes soft layer when it has been saturated by water.

(15) Layer-12A: Strongly to medium weathered silty stone, sandy stone, strongly crack, rather soft to medium hard

The stone layer having brown grey, brown, grey colour. The rock was weathered strongly to medium, the weathered level is not in equal, changes depth on each position and depth. The core recovered mainly fragment, some positions, stone have been completely weathered to become sandy clay with gravel. The total core recover(TCR) is 10 to 50%, Rock Quality Design(RQD) is 0 to 10%. This is good soil layer, N-value is more than 50.

(16) Layer-12B: Medium to light weathered silty stone, sandy stone, lightly crack, very hard. This is very good bearing capacity layer.

3.3 Subsoil Conditions of Approach Road Area in Hai An Side

3.3.1 Stratum Classification

Eleven main strata and six lens strata, as shown in Table 3.3.1-1, compose the subsoil of Approach Road Area in Hai An side. In this area Layer-1(Clay), Layer-2(Sand), Layer-5(Sand), Layer-7A(Clay) and Layer-11(Clay) are not found out.

The thickness of each layer is shown in Table 3.3.1-2. Layer-3 distributes thickest among them in this area, the thickness varies from 7.5 to 30.5 m, 17.3 m in average.

Based on the below stratum classification, the soil profile of Approach Road Area in Hai An side can be drawn in Figures 3.3.1-1 and 3.3.1-2.

Table 3.3.1-1 Stratum Classification (Hai An side)

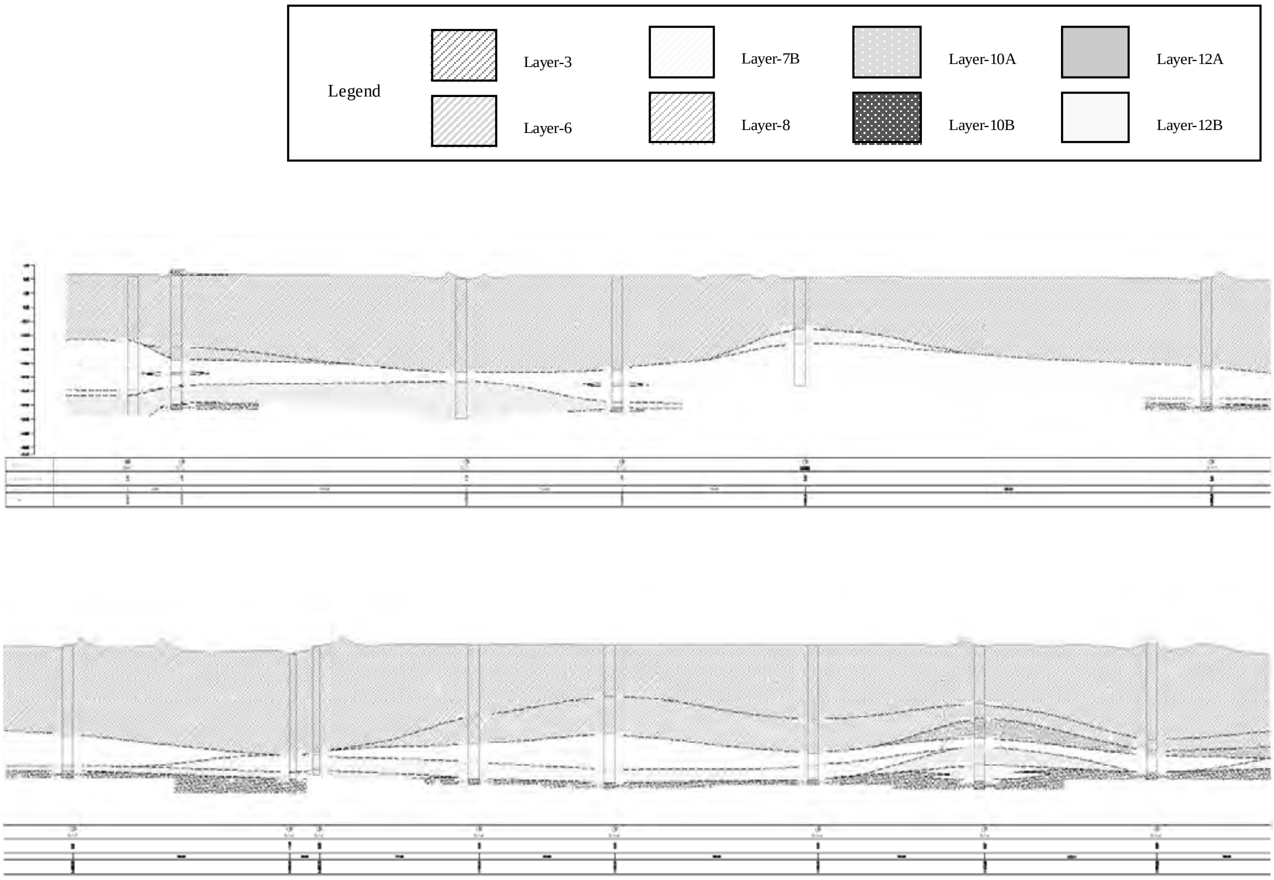
Layer	Soil Discription	Stiffness
Layer-D	**Filling soil and agriculture soil**	
Layer-3	**Clay** low plasticity	very soft to soft
Layer-4	**Clay** low plasticity	medium stiff
Layer-6	**Clay** low plasticity	stiff to very stiff
Layer-7B	**Clay** low plasticity	medium stiff
Layer-8	**Clay** low plasticity	stiff to very stiff
Layer-9	**Clay** low plasticity	medium stiff
Layer-10A	**Sand** with clay, poorly graded	
Layer-10B	**Sand** with gravel, poorly graded	
Layer-12A	Heavy weathered **silty sand stone**, RQD=0~10%	
Layer-12B	Medium weathered **silty sand stone**, RQD=50%	
Layer-L6-1	**Sand** with clay, lens layer in Layer-6	
Layer-L7B-1	**Sand** with clay, lens layer in Layer-7B	
Layer-L7B-2	**Clay**, lens layer in Layer-7B	very stiff
Layer-L7B-3	**Sand** with clay, lens layer in Layer-7B	
Layer-L10B-1	**Clay**, lens layer in Layer-10B	very stiff
Layer-L10B-2	**Clay**, lens layer in Layer-10B	stiff to very stiff

Source: Study Team

Table 3.3.1-2 Thickness of Each Layer (Hai An side)

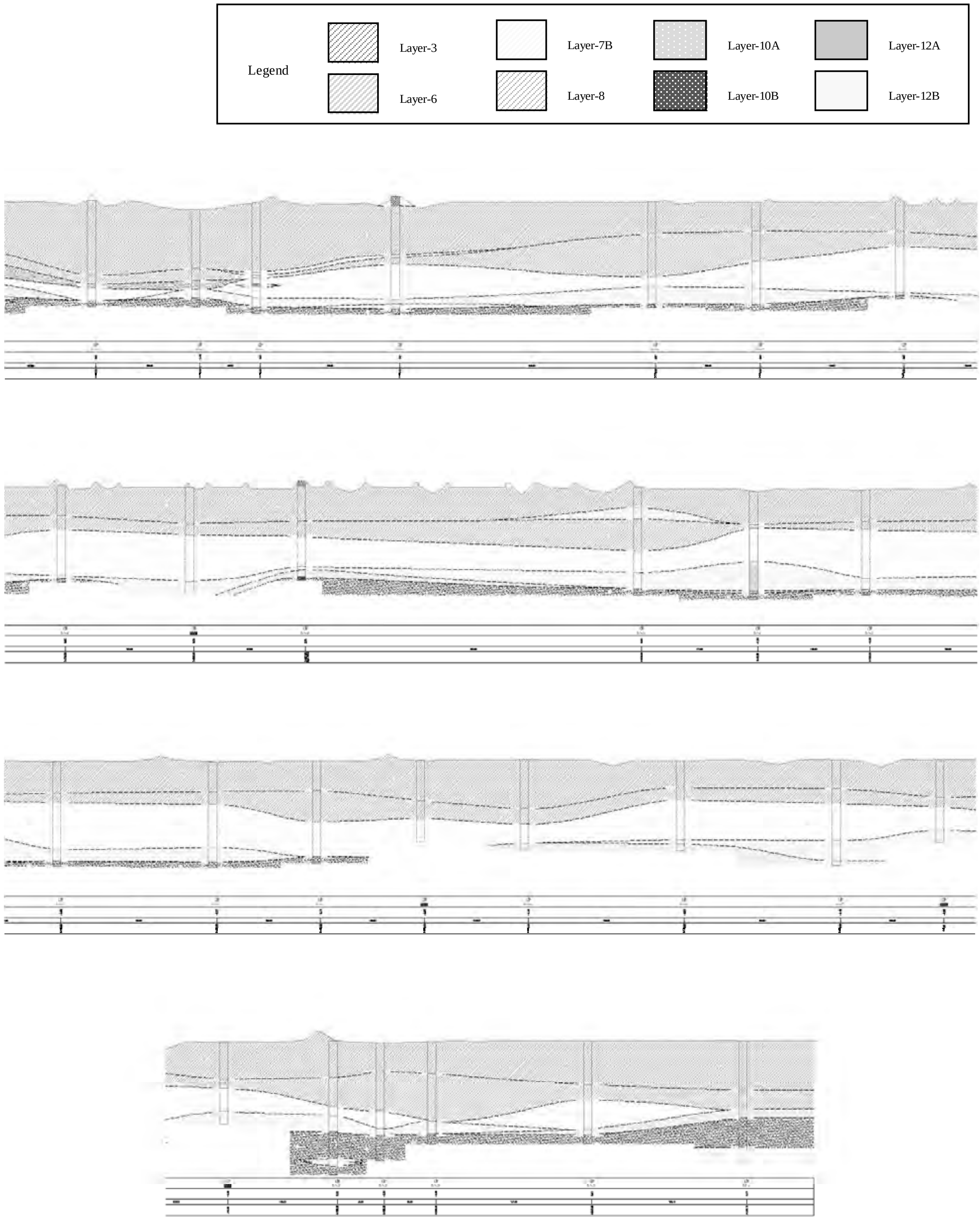
Layer	Soil	Thickness of Layer (m)		
		Minimum	Maximum	Average
D	Filling soil and agriculture soil	1.5	3.5	2.5
3	Clay (very soft to soft)	7.5	30.5	17.3
4	Clay (medium stiff)	1.0	4.3	2.1
6	Clay (stiff to very stiff)	1.2	17.8	7.4
7B	Clay (medium stiff)	0.9	18.0	8.9
8	Clay (stiff to very stiff)	1.0	10.7	4.4
9	Clay (medium stiff)	1.9	1.9	1.9
10A	Sand	1.0	5.5	2.5
10B	Sand	-	-	-
12A	Heavy weathered silty sand stone	-	-	-
12B	Medium weathered silty sand stone	-	-	-

Source: Study Team



Source: Study Team

Figure 3.3.1-1 Soil Profile of Approach Road Area in Hai An side (1/2)



Source: Study Team

Figure 3.3.1-2 Soil Profile of Approach Road Area in Hai An side (2/2)

3.3.2 N-value

N-value of each layer of Approach Road Area in Hai An side is summarized in Table 3.3.2-1.

N-values of Layer-D, 3, 4, 7B and 9 are low within the range of 2 to 7 in average, while N-values of Layer-6 and 8 are high within the range of 13 to 14 in average. On the other hand, Layer-10A and B are the sand layers having N-value of 21 to 49 in average and Layer-12A is weathered silty sand stone having N-value of more than 50.

Therefore, Layer-D, 3, 4, 7B and 9 are weak soil layers for embankment foundation.

Table 3.3.2-1 N-value of Each Layer (Hai An side)

Layer	Soil	Minimum	Maximum	Average
D	Filling soil and agriculture soil	1	2	2
3	Clay (very soft to soft)	1	5	2
4	Clay (medium stiff)	6	7	7
6	Clay (stiff to very stiff)	5	25	13
7B	Clay (medium stiff)	4	15	7
8	Clay (stiff to very stiff)	8	30	14
9	Clay (medium stiff)	5	6	6
10A	Sand	8	62	21
10B	Sand	19	97	49
12A	Heavy weathered silty sand stone	50	72	61
12B	Medium weathered silty sand stone	-	-	Over 50

Source: Study Team

3.3.3 Soil Parameters

3.3.3.1 Soil Characteristics

Based on the results of laboratory tests, the soil characteristics of each layer of Approach Road Area in Hai An side are summarized in Table 3.3.3-1. Figure 3.3.3-1 presents the chart of soil parameters for each layer of Approach Road Area in Hai An side.

Layer-3 has the soil characteristics with natural water content of 51.3%, wet density of 1.70 g/cm^3 and initial void ratio of 1.384 as very soft to soft clay.

Layer-7B has the soil characteristics with natural water content of 41.0%, wet density of 1.76 g/cm^3 and initial void ratio 1.225 as medium stiff clay.

Layer-4 has the soil characteristics with natural water content of 39.6%, while Layer-9 has the soil characteristics with natural water content of 35.3% as medium stiff clay.

On the other hand, Layer-6 has the soil characteristics with natural water content of 32.1%, wet density of 1.86 g/cm^3 and initial void ratio of 0.943 as stiff to very stiff clay.

Layer-8 has the soil characteristics with natural water content of 25.9%, wet density of 1.88 g/cm^3 and initial void ratio of 0.851 as stiff to very stiff clay.

As Pc (Preconsolidation Pressure) of all clay layers are distributed in nearly effective overburden pressure, clay layers of Approach Road Area in Hai An side are evaluated to be normally consolidated clay.

Table 3.3.3-1 Soil Characteristics of Each Layer (Hai An side)

(Average)

Item					Layer									
					3 (Clay)	4 (Clay)	6 (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)	L10B-1 (Clay)	L10B-2 (Clay)
N-value					2	7	13	7	14	6	21	49	20	-
Natural Water Content			Wn	(%)	51.3	39.6	32.1	41.0	25.9	35.3	21.5	-	27.6	25.1
Specific Gravity			Gs	(g/cm^3)	2.68	2.64	2.70	2.68	2.69	2.72	2.64	2.65	2.66	2.67
Wet Density			γt	(g/cm^3)	1.70	-	1.86	1.76	1.88	-	2.04	-	-	-
Dry Density			γd	(g/cm^3)	1.15	-	1.40	1.23	1.46	-	1.73	-	-	-
Void Ratio			e_0		1.384	-	0.943	1.225	0.851	-	0.535	-	-	-
Saturation			Sr	(%)	97.4	-	93.7	97.3	92.4	-	90.1	-	-	-
Liquid Limit			LL	(%)	43.0	40.0	44.7	44.2	36.2	37.4	22.4	12.2	36.3	31.9
Plastic Limit			PL	(%)	24.4	26.8	24.1	23.9	19.5	22.6	15.2	9.5	21.3	18.5
Plasticity Index			Ip		18.7	13.2	20.6	20.3	16.7	14.8	7.3	2.7	15.0	13.4
Unconfined Compression			qu	(kg/cm^2)	0.32	-	1.27	0.35	1.01	-	1.03	-	-	-
Triaxial (UU)	Cohesion		Cuu	(kg/cm^2)	0.27	-	0.67	0.28	0.63	-	-	-	-	-
	Internal Friction		ϕuu	(degree)	0.0	-	0.0	0.0	0.0	-	-	-	-	-
Triaxial (CU)	Total Stress	Cohesion	Ccu	(kg/cm^2)	0.12	-	0.16	0.24	-	-	-	-	-	-
		Internal Friction	ϕcu	(degree)	14.9	-	15.8	14.1	-	-	-	-	-	-
	Effective Stress	Cohesion	C'	(kg/cm^2)	0.08	-	0.12	0.08	-	-	-	-	-	-
		Internal Friction	ϕ'	(degree)	24.8	-	23.3	23.1	-	-	-	-	-	-
Consolidation	Compression Index		Cc		0.435	-	0.302	0.420	0.262	-	0.103	-	-	-
	Swell Index		Cs		0.050	-	0.039	0.060	0.039	-	0.007	-	-	-
	Preconsolidation Pressure		Pc	(kg/cm^2)	0.74	-	1.70	1.59	1.73	-	1.43	-	-	-

Source: Study Team

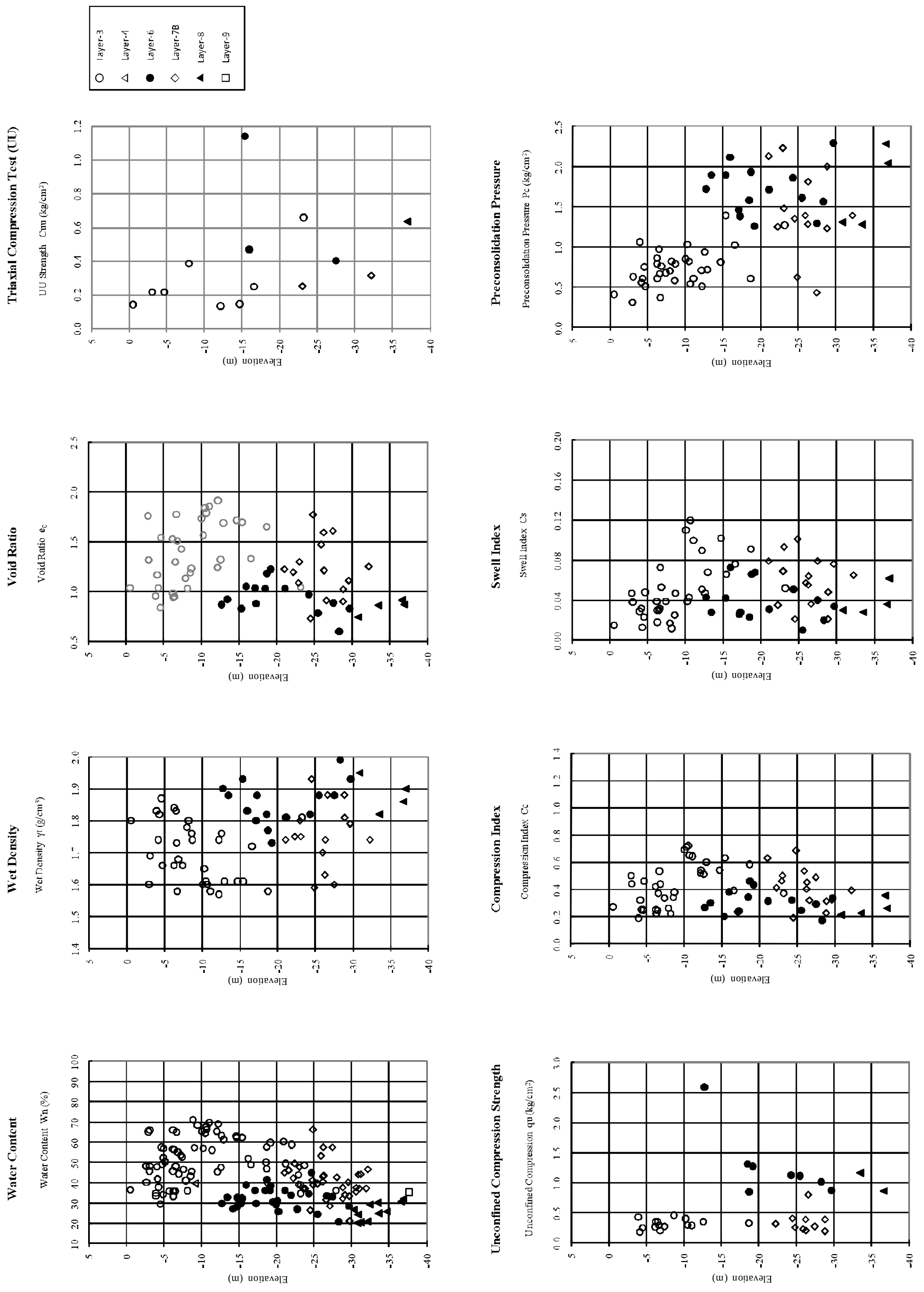


Figure 3.3.3-1 Chart of Soil Parameters (Hai An side)

Source: Study Team

3.3.3.2 Shear Strength

(1) Shear Strength of Clay for Short Term Stability

The shear strength for short term stability of Layer-3, 4, 6, 7B and 8 (clay layers) is determined based on the results of FVST, unconfined compression test, triaxial compression (UU) test and N-value.

However, there were some unreliable results of unconfined compression test due to disturbance of sample during sampling, transportation and laboratory. Therefore, the results of unconfined compression test of which the strain at failure is more than 7% were excluded from data analysis for shear strength.

Figure 3.3.3-2 shows the relationship between Su and elevation using all data of FVST, qu/2, Cuu and assumed strength from N-value. From these results, the shear strength for short term stability of each layer is proposed as follows:

Table 3.3.3-2 Shear Strength of Clay for Short Term Stability (Hai An side)

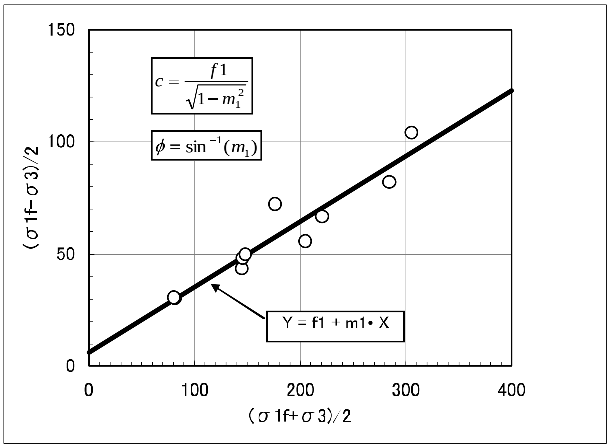
Layer	Shear Strength Su (kg/cm^2)
3	Su = 0.1 kg/cm^2 (Down to El +/- 0m) Su = 0.1 + 0.005 x Z kg/cm^2 (Below EL +/- 0m)
4	Su = 0.25 kg/cm^2
6	Su = 0.5 kg/cm^2
7B	Su = 0.25 kg/cm^2
8	Su = 0.6 kg/cm^2

Source: Study Team

(2) Shear Strength of Clay for Long Term Stability

The shear strength for long term stability of Layer-3, 6 and 7B (clay layers) is determined based on the results of triaxial compression (CU) test.

The shear strength of C and ϕ in total stress and effective stress can be determined using $(\sigma_{1f}+\sigma_3)/2$ and $(\sigma_{1f}-\sigma_3)/2$ of all data of triaxial compression (CU) test as shown in Figure 3.3.3-3.



Source: Basis and Guide of Soil Test (Japanese Geotechnical Society)

Figure 3.3.3-2 Determination Method of C and ϕ using $(\sigma_{1f}+\sigma_3)/2$ and $(\sigma_{1f}-\sigma_3)$

Figure 3.3.3-4 presents the results of analysis of triaxial compression (CU) tests. As a result of examination, the design criteria of shear strength of Layer-3, 6 and 7B in total stress and effective stress is proposed as follows.

Table 3.3.3-3 Shear Strength of Clay for Long Term Stability (Hai An side)

Layer	Total Stress		Effective Stress	
	Ccu (kg/cm^2)	ϕcu (degree)	C' (kg/cm^2)	ϕ' (degree)
3	0.13	14.3	0.06	26.5
6	0.16	15.9	0.12	23.3
7B	0.06	17.1	0.09	22.5

Source: Study Team

(3) Shear Strength of Sand

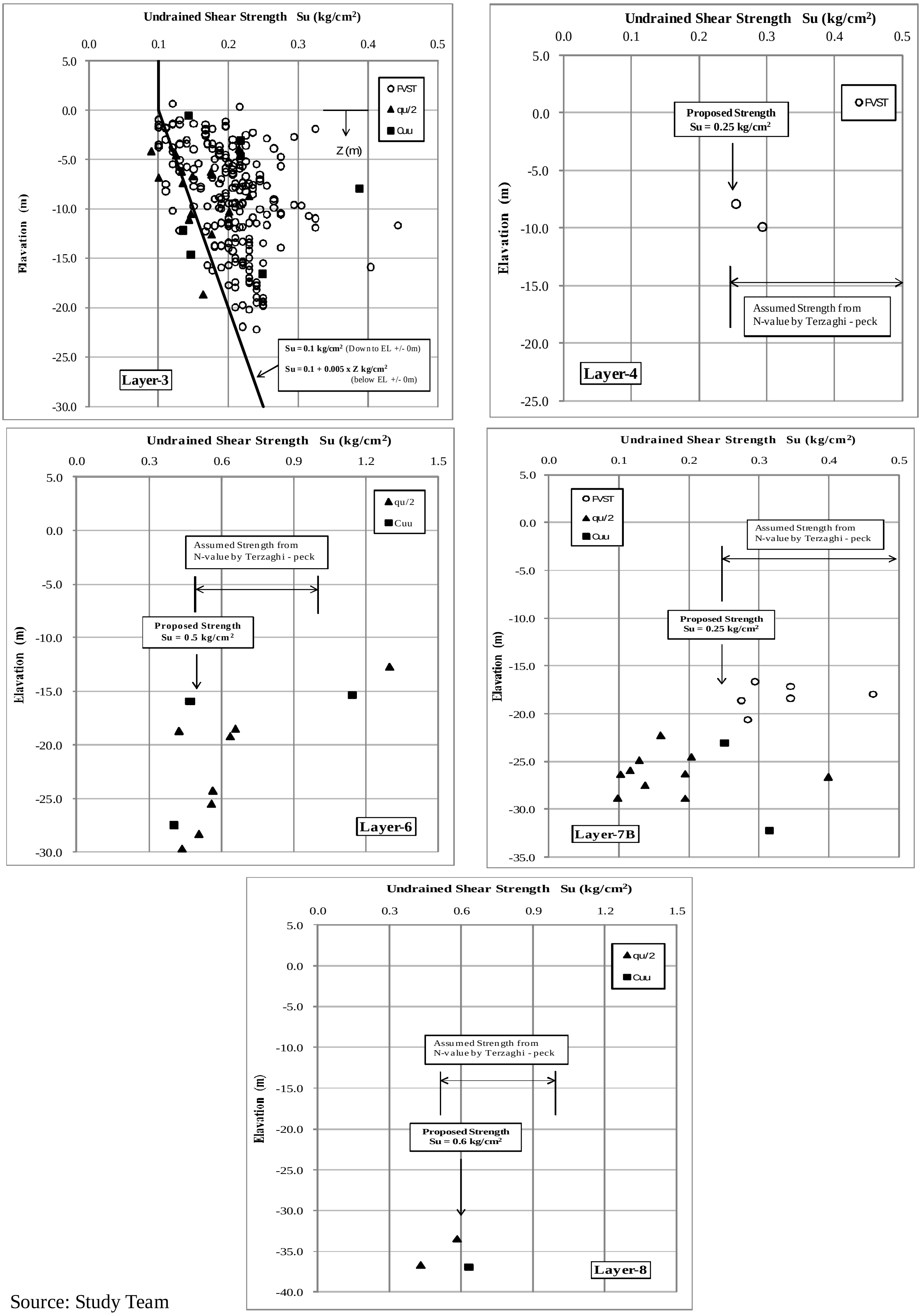
The shear strength of Layer-10A and 10B (sand layers) is determined based on N-value. The shear strength of ϕ of sand layer can be estimated using three formulas by Ohsaki, Peck and Dunham.

As a result of estimation, the shear strength of Layer-10A and 10B (sand layers) is proposed as follows.

Table 3.3.3-4 Shear Strength of Sand (Hai An side)

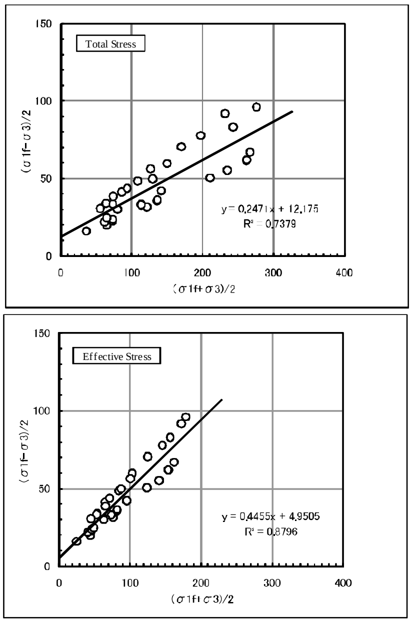
Layer			10A	10B
N-value (Average)			21	49
$\gamma\tau$		(g/cm^3)	(2.00)	(2.05)
ϕ	By Ohsaki $\phi=\sqrt{(20N)}+15$	(degree)	35	46
	By Peck, Dunham(1) $\phi=\sqrt{(12N)}+20$	(degree)	36	44
	By Dunham(2) $\phi=\sqrt{(12N)}+15$	(degree)	31	39
E	(E=28N)	(kg/cm^2)	588	1372
ϕ	**(proposed)**	(degree)	35	40

Note: The value in ( ) is assumed value.
Source: Study Team



Source: Study Team

Figure 3.3.3-3 Shear Strength of Clay for Short Term Stability (Hai An side)



Y=f1+m1・X

$c = \frac{f1}{\sqrt{1-m_1^2}}$

$\phi = \sin^{-1}(m_1)$

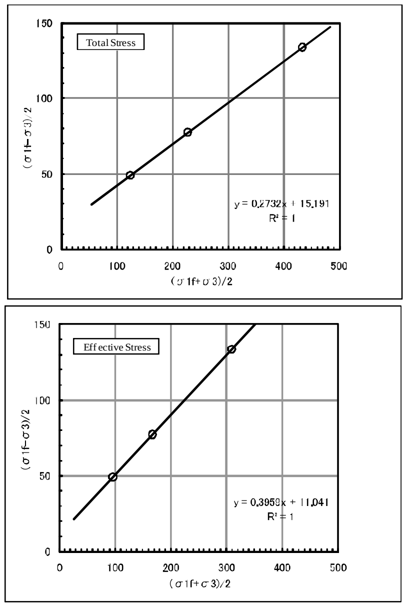
Total Stress:

f1	12.175	
m1	0.2471	
Ccu	12.56	(kN/m²)
φ	0.249686	(rad)
φcu	14.31	(deg)

Effective Stress:

f1	4.9505	
m1	0.4455	
C'	5.53	(kN/m²)
φ	0.461733	(rad)
φ'	26.46	(deg)

Analysis of CU Test (Layer-3)



Y=f1+m1・X

$c = \frac{f1}{\sqrt{1-m_1^2}}$

$\phi = \sin^{-1}(m_1)$

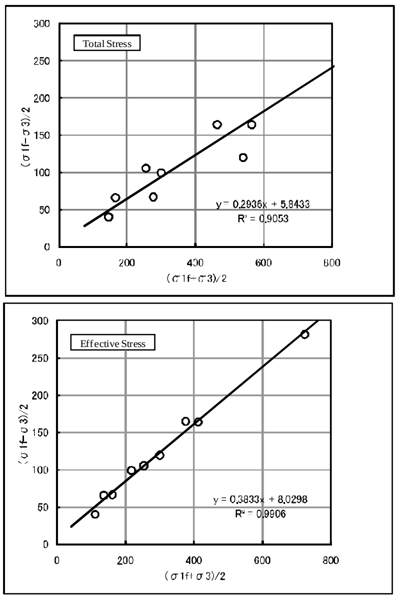
Total Stress:

f1	15.191	
m1	0.2732	
Ccu	15.79	(kN/m²)
φ	0.276718	(rad)
φcu	15.85	(deg)

Effective Stress:

f1	11.041	
m1	0.3959	
C'	12.02	(kN/m²)
φ	0.407048	(rad)
φ'	23.32	(deg)

Analysis of CU Test (Layer-6)



Y=f1+m1・X

$c = \frac{f1}{\sqrt{1-m_1^2}}$

$\phi = \sin^{-1}(m_1)$

Total Stress:

f1	5.8433	
m1	0.2936	
Ccu	6.11	(kN/m²)
φ	0.297991	(rad)
φcu	17.07	(deg)

Effective Stress:

f1	8.0298	
m1	0.3833	
C'	8.69	(kN/m²)
φ	0.393367	(rad)
φ'	22.54	(deg)

Analysis of CU Test (Layer-7B)

Source: Study Team

Figure 3.3.3-4 Shear Strength of Clay for Long Term Stability (Hai An side)

3.3.3.3 Consolidation Parameters

The soil Parameters necessary for settlement analysis are wet density (γt), initial void ratio (eo), compression index (Cc), swell index (Cs), coefficient of consolidation (Cv) and preconsolidation pressure (Pc).

The parameters of γt and eo are determined from results of unit weight test, while those of Cc, Cs, Cv and Pc are determined from the results of consolidation test.

Figure 3.3.3-5 presents e~logP curve and logP~logCv curve for Layer-3, 6, 7B and 8. The parameters of Cc, Cs and Pc are determined from e~logP curve, while the parameter of Cv is determined from logP~logCv curve.

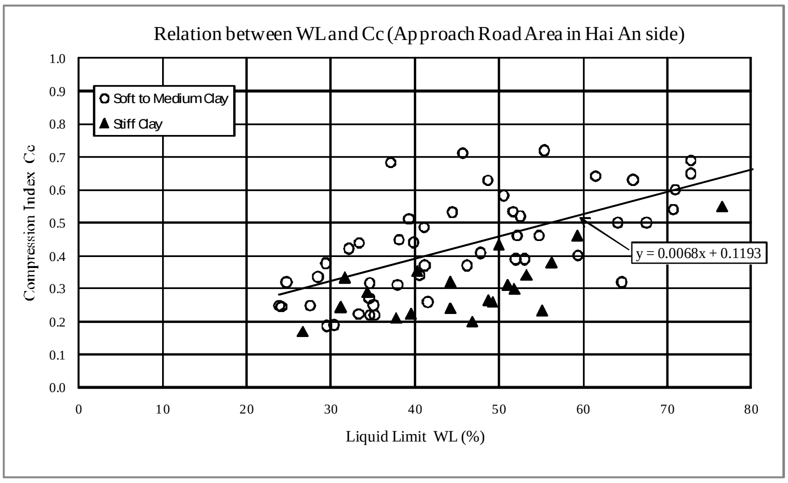
As a result of unit weight test and consolidation test, the soil parameters for settlement analysis are proposed as follows.

Table 3.3.3-5 Soil Parameters for Settlement Analysis (Hai An side)

Item		3	6	7B	8
Unit Weight	γt (g/cm^3)	1.70	1.86	1.76	1.88
Initial Void Ratio	eo	1.384	0.943	1.225	0.851
Compression Index	Cc	0.435	0.302	0.420	0.262
Swell Index	Cs	0.050	0.039	0.060	0.039
Preconsolidation Pressure	Pc (kg/cm^2)	0.74	1.70	1.59	1.73
Coefficient of Consolidation	Cv (cm^2/day)	50	100	70	100

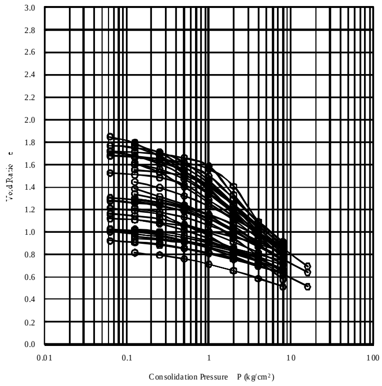
Source: Study Team

The parameter of Cc is very important for estimation of settlemet amount. Figure 3.3.3-6 presents the relationship between Cc and W_L. For both soft to medium clay and stiff clay, Cc increases with W_L. However, Cc of stiff clay is distributed lower than that of soft to medium clay, which means that the estimated amount of settlement of stiff clay is less than that of soft to medium clay.

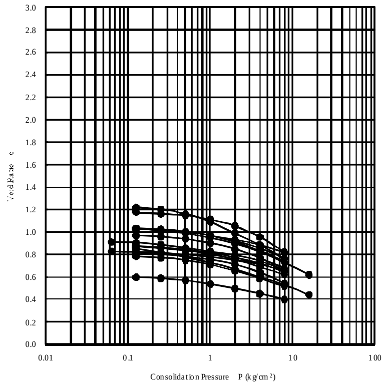


Source: Study Team

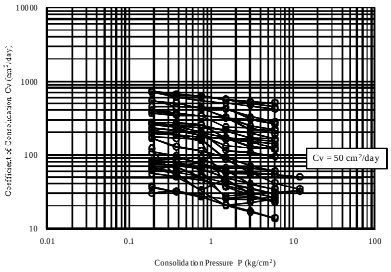
Figure 3.3.3-5 Relationship between Cc and W_L (Hai An side)



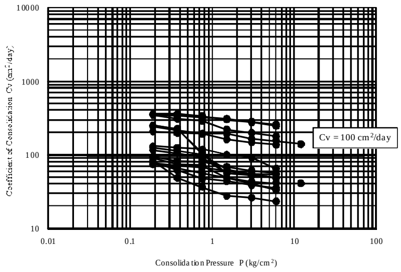
e ~ logP Curve (Hai An area: Layer-3)



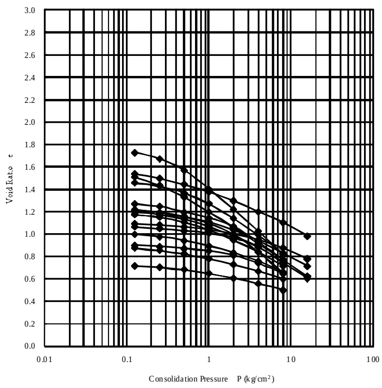
e ~ logP Curve (Hai An area: Layer-6)



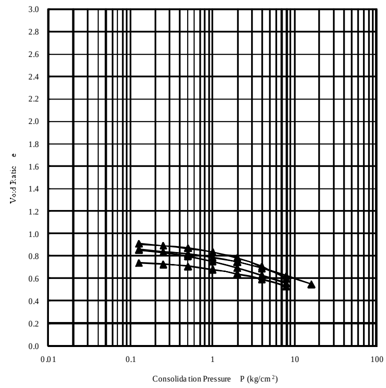
logP ~ logCv Curve (Hai An area: Layer-3)



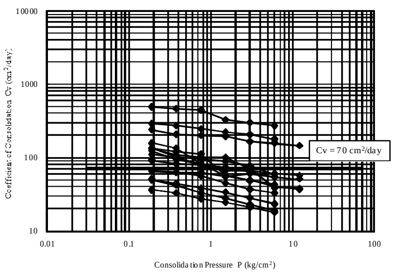
logP ~ logCv Curve (Hai An area: Layer-6)



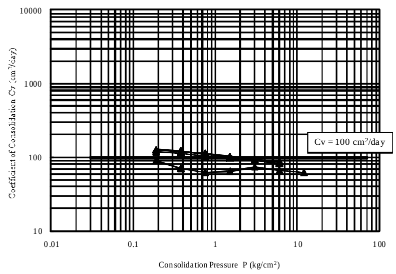
e ~ logP Curve (Hai An area: Layer-7B)



e ~ logP Curve (Hai An area: Layer-8)



logP ~ logCv Curve (Hai An area: Layer-7B)



logP ~ logCv Curve (Hai An area: Layer-8)

Source: Study Team

Figure 3.3.3-6 e~logP Curve and logP~logCv Curve (Hai An side)

3.3.4 Soil Parameters for Design

As a result of laboratory tests, shear strength analysis and consolidation analysis, the soil parameters for design of Approach Road Area in Hai An side are proposed as shown in Table 3.3.4-1.

However, there were no triaxial compression (CU) tests for Layer-4, 8 and 9, and no consolidation tests for Layer-4 and 9. Therefore, the soil parameters (Ccu, ϕcu, C’, ϕ’) of Layer-8 are proposed using those of Layer-6, while the soil parameters (Ccu, ϕcu, C’, ϕ’, eo, Cc, Cs, Pc, Cv) of Layer-4 and 9 are proposed using those of Layer-7B.

Table 3.3.4-1 Soil Parameters for Design (Hai An side)

Item					3 (Clay)	4 (Clay)	6 (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)
N-value					2	7	13	7	14	6	21	49
Unit Weight			γt	(g/cm^3)	1.70	(1.76)	1.86	1.76	1.88	(1.76)	(2.00)	(2.05)
Shear Strength	For Short Term		Su or Cd	(kg/cm^2)	Note (2)	0.25	0.50	0.25	0.60	(0.25)	0.00	0.00
			ϕu or ϕd	(degree)	0.0	0.0	0.0	0.0	0.0	0.0	35.0	40.0
	For Long Term	Total Stress	Ccu	(kg/cm^2)	0.13	0.06	0.16	0.06	0.16	0.06	-	-
			ϕcu	(degree)	14.3	17.1	15.9	17.1	15.9	17.1	-	-
		Effective Stress	C'	(kg/cm^2)	0.06	0.09	0.12	0.09	0.12	0.09	-	-
			ϕ'	(degree)	26.5	22.5	23.3	22.5	23.3	22.5	-	-
	Rate of Strength Increase		m		0.2	0.2	0.2	0.2	0.2	0.2	-	-
Consolidation	Initial Void Ratio		eo		1.384	1.225	0.943	1.225	0.851	1.225	-	-
	Compression Index		Cc		0.435	0.420	0.302	0.420	0.262	0.420	-	-
	Swell Index		Cs		0.050	0.060	0.039	0.060	0.039	0.060	-	-
	Preconsolidation Pressure		Pc	(kg/cm^2)	0.74	1.59	1.70	1.59	1.73	1.59	-	-
	Coefficient of Consolidation		Cv	(cm^2/day)	50	70	100	70	100	70	-	-

Note (1): The value in ( ) is assumed.
Note (2): Su = 0.1 kg/cm^2 (Down to EL 0.0m), Su = 0.1 + 0.005 x Z kg/cm^2 (below EL 0.0m, Zo = EL 0.0m)
Note (3): Adopt the soil parameters (Ccu, ϕcu, C', ϕ') of Layer-6 for Layer-8
Note (4): Adopt the soil parameters (Ccu, ϕcu, C', ϕ', eo, Cc, Cs, Pc, Cv) of Layer-7B for Layer-4 and Layer-9

Source: Study Team

3.4 Subsoil Conditions of Bridge Area

3.4.1 Stratum Classification

Fourteen main strata and eighteen lens strata, as shown in Table 3.4.1-1, compose the subsoil of Bridge Area. In this area Layer-D (Filling Soil and Agriculture Soil) and Layer-1 (Clay) are not found out.

The thickness of each layer is shown in Table 3.4.1-2. Layer-3 distributes thickest among them in this area, the thickness varies from 1.8 to 20.5 m, 11.0 m in average.

Based on the below stratum classification, the soil profile of Bridge Area can be drawn in Figure 3.4.1-1 and Figure 3.4.1-2.

Table 3.4.1-1 Stratum Classification (Bridge Area)

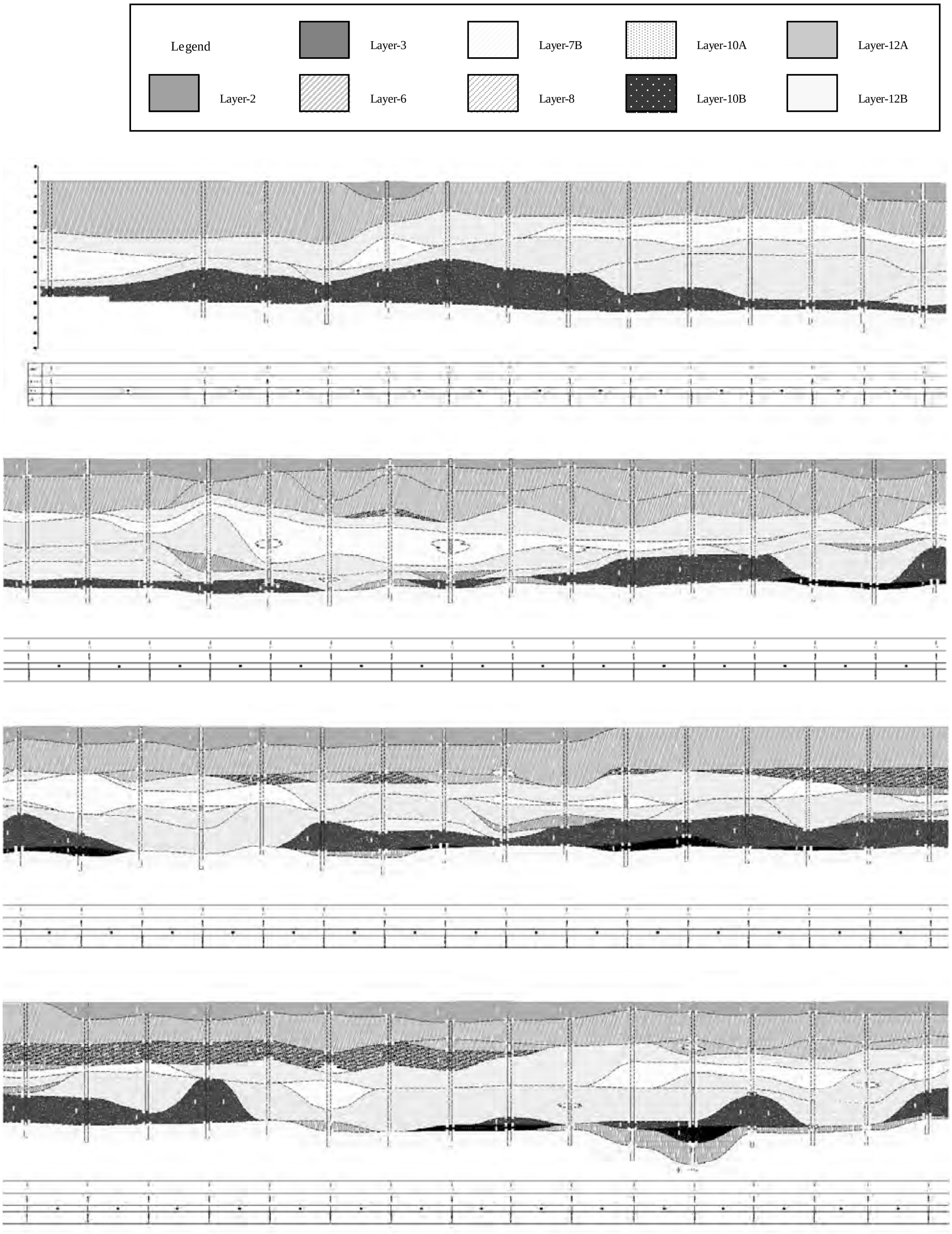
Layer	Soil Discription	Stiffness
Layer-2	**Sand** with clay, poorly graded	
Layer-3	**Clay** low plasticity	very soft to soft
Layer-4	**Clay** low plasticity	medium stiff
Layer-5	**Sand** with clay, poorly graded	
Layer-6	**Clay** low plasticity	stiff to very stiff
Layer-7A	**Clay** low plasticity	soft
Layer-7B	**Clay** low plasticity	medium stiff
Layer-8	**Clay** low plasticity	stiff to very stiff
Layer-9	**Clay** low plasticity	medium stiff
Layer-10A	**Sand** with clay, poorly graded	
Layer-10B	**Sand** with gravel, poorly graded	
Layer-11	**Clay** low plasticity	stiff to very stiff
Layer-12A	Heavy weathered **silty sand stone**, RQD=0~10%	
Layer-12B	Medium weathered **silty sand stone**, RQD=50%	
Layer-L3-1	**Sand** with clay, lens layer in Layer-3	
Layer-L4-1	**Sand** with clay, lens layer in Layer-4	
Layer-L5-1	**Clay**, lens layer in Layer-5	medium stiff
Layer-L6-1	**Sand** with clay, lens layer in Layer-6	
Layer-L6-2	**Sand** with clay, lens layer in Layer-6	
Layer-L6-3	**Clay**, lens layer in Layer-6	medium stiff
Layer-L6-4	**Sand** with clay, lens layer in Layer-6	
Layer-L7B-1	**Sand** with clay, lens layer in Layer-7B	
Layer-L7B-2	**Sand** with clay, lens layer in Layer-7B	
Layer-L7B-3	**Sand** with clay, lens layer in Layer-7B	
Layer-L8-1	**Sand** with clay, lens layer in Layer-8	
Layer-L8-2	**Sand** with clay, lens layer in Layer-8	
Layer-L10A-1	**Clay**, lens layer in Layer-10A	stiff
Layer-L10A-2	**Clay**, lens layer in Layer-10A	stiff
Layer-L10A-3	**Clay**, lens layer in Layer-10A	medium stiff
Layer-L10A-4	**Clay** with sand, lens layer in Layer-10A	very stiff
Layer-L10A-5	**Clay**, lens layer in Layer-10A	very stiff
Layer-L10B-1	**Clay**, lens layer in Layer-10B	stiff

Source: Study Team

Table 3.4.1-2 Thickness of Each Layer (Bridge Area)

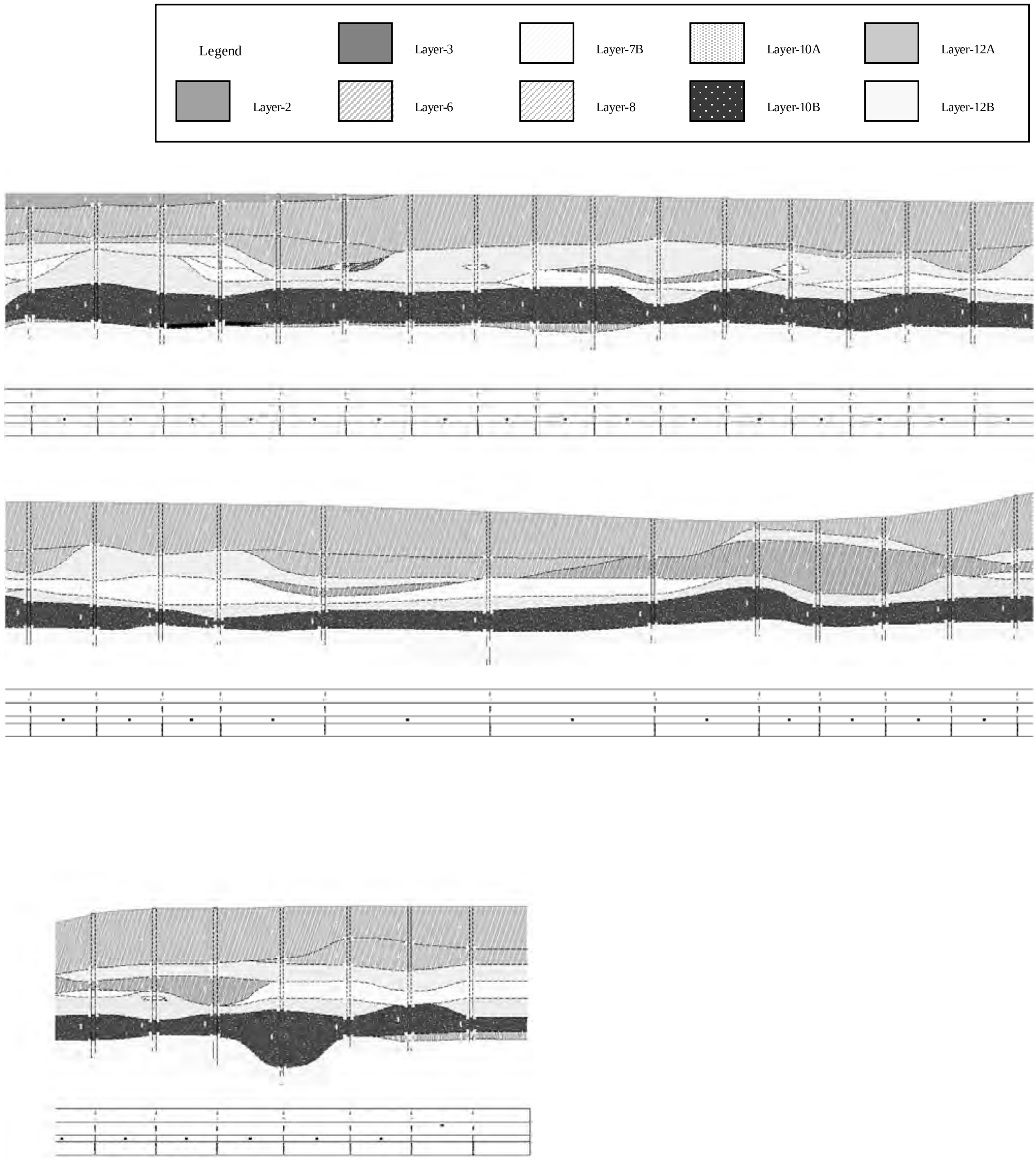
Layer	Soil	Thickness of Layer (m)		
		Minimum	Maximum	Average
2	Sand	1.5	7.7	4.6
3	Clay (very soft to soft)	1.8	20.5	11.0
4	Clay (medium stiff)	1.4	12.1	5.6
5	Sand	1.5	7.3	4.5
6	Clay (stiff to very stiff)	1.9	13.0	5.6
7A	Clay (soft)	1.5	15.4	6.4
7B	Clay (medium stiff)	1.0	13.1	5.1
8	Clay (stiff to very stiff)	1.4	13.8	5.6
9	Clay (medium stiff)	2.0	6.1	3.1
10A	Sand	1.1	15.0	6.2
10B	Sand	1.3	18.5	7.5
11	Clay (stiff to very stiff)	0.8	5.8	2.0
12A	Heavy weathered silty sand stone	0.7	7.2	2.2
12B	Medium weathered silty sand stone	The thickness of more than 5m is confirmed.		

Source: Study Team



Source: Study Team

Figure 3.4.1-1 Soil Profile of Bridge Area (1/2)



Source: Study Team

Figure 3.4.1-2 Soil Profile of Bridge Area (2/2)

3.4.2 N-value

N-value of each layer of Bridge Area is summarized in Table 3.4.2-1.

Regarding clay layers, N-values of Layer-3, 4, 7A, 7B and 9 are low within the range of 2 to 7 in average, while N-values of Layer-6, 8 and 11 are high within the range of 12 to 18 in average. On the other hand, regarding sand layers, N-value of Layer-2 is 3 in average, while N-value of Layer-5 is 11 and those of Layer-10A and 10B are high within the range of 21 to 38 in average. Layer-12A and 12B are weathered silty sand stone having N-value of more than 50.

Therefore, Layer-12A and 12B are evaluated to be the bearing strata for pile foundation. On the other hand, Layer-2, 3, 4, 7A, 7B and 9 are weak soil layers for embankment and reclamation.

Table 3.4.2-1 N-value of Each Layer (Bridge Area)

Layer	Soil	Minimum	Maximum	Average
2	Sand	1	8	3
3	Clay (very soft to soft)	0	6	2
4	Clay (medium stiff)	3	14	6
5	Sand	3	30	11
6	Clay (stiff to very stiff)	5	34	12
7A	Clay (soft)	2	11	4
7B	Clay (medium stiff)	4	14	7
8	Clay (stiff to very stiff)	5	29	15
9	Clay (medium stiff)	3	13	7
10A	Sand	4	47	21
10B	Sand	12	59	38
11	Clay (stiff to very stiff)	9	31	18
12A	Heavy weathered silty sand stone	-	-	Over 50
12B	Medium weathered silty sand stone	-	-	Over 50

Source: Study Team

3.4.3 Soil Parameters

3.4.3.1 Soil Characteristics

Based on the results of laboratory tests, the soil characteristics of each layer of Bridge Area are summarized in Table 3.4.3-1. Figure 3.4.3-1 presents the chart of soil parameters for each layer of Bridge Area.

Layer-3 has the soil characteristics with natural water content of 49.4%, wet density of 1.72 g/cm^3 and initial void ratio of 1.314 as very soft to soft clay.

Layer-4 has the soil characteristics with natural water content of 44.2%, wet density of 1.81 g/cm^3 and initial void ratio of 1.127 as medium stiff clay.

Layer-7B has the soil characteristics with natural water content of 34.5%, wet density of 1.83 g/cm^3 and initial void ratio 1.056 as medium stiff clay.

Layer-7A and 9 has the soil characteristics with natural water content of 38.8% and 46.9% respectively as soft to medium stiff clay, while Layer-11 has the soil characteristics with natural water content of 22.1% as stiff to very stiff clay.

On the other hand, Layer-6 has the soil characteristics with natural water content of 29.3%, wet density of 1.95 g/cm^3 and initial void ratio of 0.777 as stiff to very stiff clay.

Layer-8 has the soil characteristics with natural water content of 28.2%, wet density of 1.92 g/cm^3 and initial void ratio of 0.838 as stiff to very stiff clay.

Regarding sand layers, Layer-2, 5, 10A and 10B has the soil characteristics with natural water content of 31.3%, 26.7%, 21.4% and 17.7% respectively.

As Pc (Preconsolidation Pressure) of all clay layers are distributed in nearly effective overburden pressure, clay layers of Bridge Area are evaluated to be normally consolidated clay.

Table 3.4.3-1 Soil Characteristics of Each Layer (Bridge Area)

(Average)

Item				Layer												
				2 (Sand)	3 (Clay)	4 (Clay)	5 (Sand)	6 (Clay)	7A (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)	11 (Clay)	12A (Weathered Siltstone)
N-value				3	2	6	11	12	4	7	15	7	21	38	18	Over 50
Natural Water Content			Wn (%)	31.3	49.4	44.2	26.7	29.3	38.8	34.5	28.2	46.9	21.4	17.7	22.1	18.8
Specific Gravity			Gs (g/cm^3)	2.69	2.69	2.70	2.69	2.71	2.71	2.69	2.71	2.71	2.68	2.67	2.72	2.69
Wet Density			γt (g/cm^3)	-	1.72	1.81	2.08	1.95	-	1.83	1.92	-	-	-	-	-
Dry Density			γd (g/cm^3)	-	1.18	1.29	1.75	1.53	-	1.34	1.48	-	-	-	-	-
Void Ratio			e_0	-	1.314	1.127	0.537	0.777	-	1.056	0.838	-	-	-	-	-
Saturation			Sr (%)	-	96.3	99.1	93.6	96.4	-	97.8	97.8	-	-	-	-	-
Liquid Limit			LL (%)	29.2	51.1	52.8	28.0	43.6	45.1	40.5	40.1	57.6	22.4	18.5	33.9	32.2
Plastic Limit			PL (%)	19.0	26.5	25.8	14.8	21.6	21.3	21.4	20.4	28.1	15.5	14.2	18.6	17.7
Plasticity Index			Ip	10.2	24.5	27.0	13.2	22.0	23.9	19.2	19.7	29.5	6.9	4.3	15.3	14.5
Unconfined Compression			qu (kg/cm^2)	-	0.82	-	-	1.90	-	0.48	-	-	-	-	-	-
Triaxial (UU)	Cohesion		Cuu (kg/cm^2)	-	0.17	0.85	0.62	0.73	-	0.45	0.58	-	-	-	-	-
	Internal Friction		ϕuu (degree)	-	0.0	0.0	0.0	0.0	-	0.0	0.0	-	-	-	-	-
Triaxial (CU)	Total Stress	Cohesion	Ccu (kg/cm^2)	-	-	-	-	-	-	-	-	-	-	-	-	-
		Internal Friction	ϕcu (degree)	-	-	-	-	-	-	-	-	-	-	-	-	-
	Effective Stress	Cohesion	C' (kg/cm^2)	-	-	-	-	-	-	-	-	-	-	-	-	-
		Internal Friction	ϕ' (degree)	-	-	-	-	-	-	-	-	-	-	-	-	-
Consolidation	Compression Index		Cc	-	0.390	0.385	0.110	0.234	-	0.345	0.200	-	-	-	-	-
	Swell Index		Cs	-	0.042	0.076	0.018	0.052	-	0.057	0.058	-	-	-	-	-
	Preconsolidation Pressure		Pc (kg/cm^2)	-	0.70	2.10	1.78	2.67	-	1.64	1.98	-	-	-	-	-

Source: Study Team

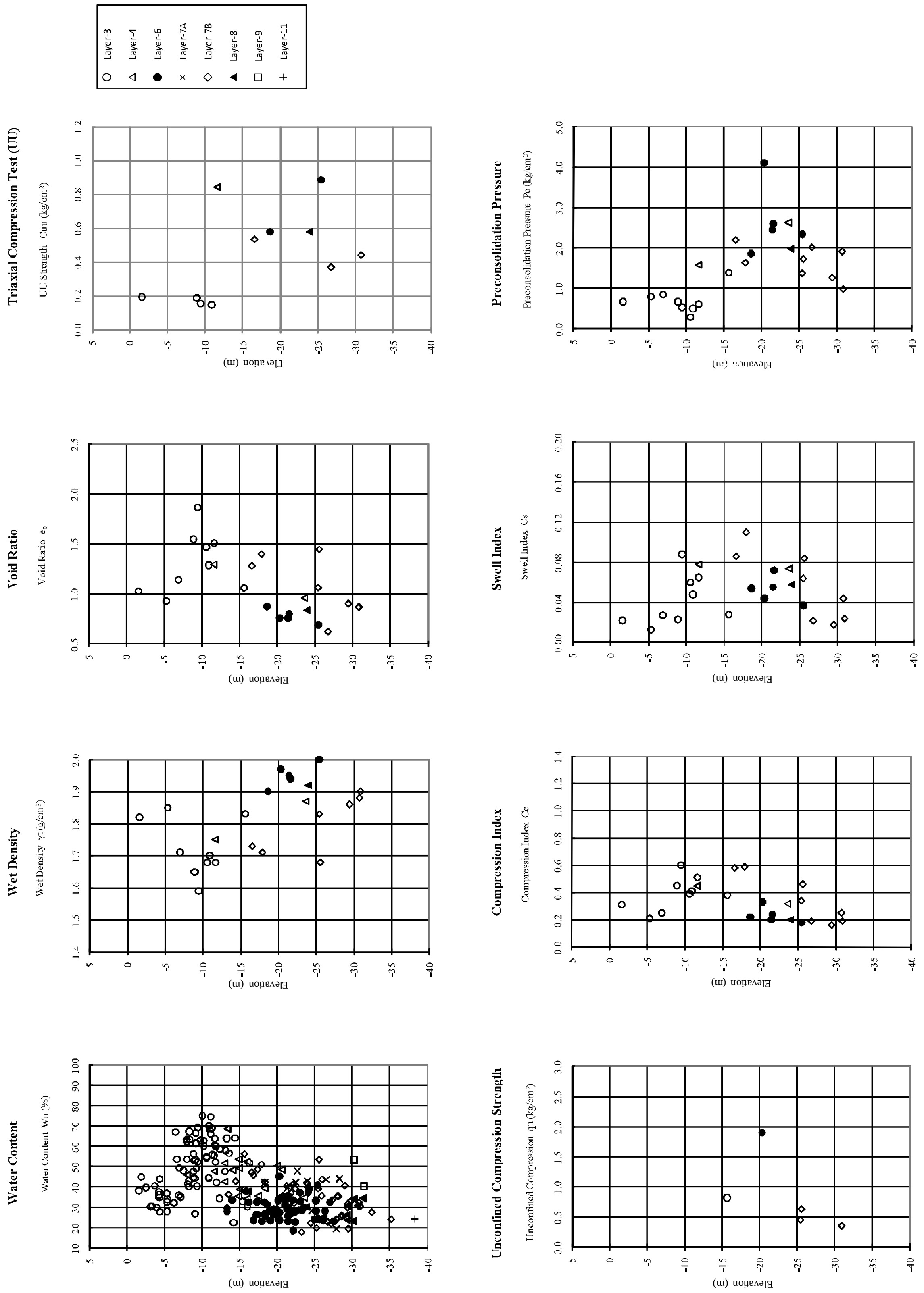


Figure 3.4.3-1 Chart of Soil Parameters (Bridge Area)

Source: Study Team

3.4.3.2 Shear Strength

(1) Shear Strength of Clay for Short Term Stability

The shear strength for short term stability of Layer-3, 4, 6, 7B and 8 (clay layers) is determined based on the results of unconfined compression test, triaxial compression (UU) test and N-value.

However, there were some unreliable results of unconfined compression test due to disturbance of sample during sampling, transportation and laboratory. Therefore, the results of unconfined compression test of which the strain at failure is more than 7% were excluded from data analysis for shear strength.

Figure 3.4.3-2 shows the relationship between Su and elevation using all data of qu/2, Cuu and assumed strength from N-value. From these results, the shear strength for short term stability of each layer is proposed as follows:

Table 3.4.3-2 Shear Strength of Clay for Short Term Stability (Bridge Area)

Layer	Shear Strength Su (kg/cm^2)
3	Su = 0.15 kg/cm^2
4	Su = 0.25 kg/cm^2
6	Su = 0.6 kg/cm^2
7B	Su = 0.3 kg/cm^2
8	Su = 0.6 kg/cm^2

Source: Study Team

(2) Shear Strength of Sand

The shear strength of Layer-2, 5, 10A and 10B (sand layers) is determined based on N-value. The shear strength of ϕ of sand layer can be estimated using three formulas by Ohsaki, Peck and Dunham.

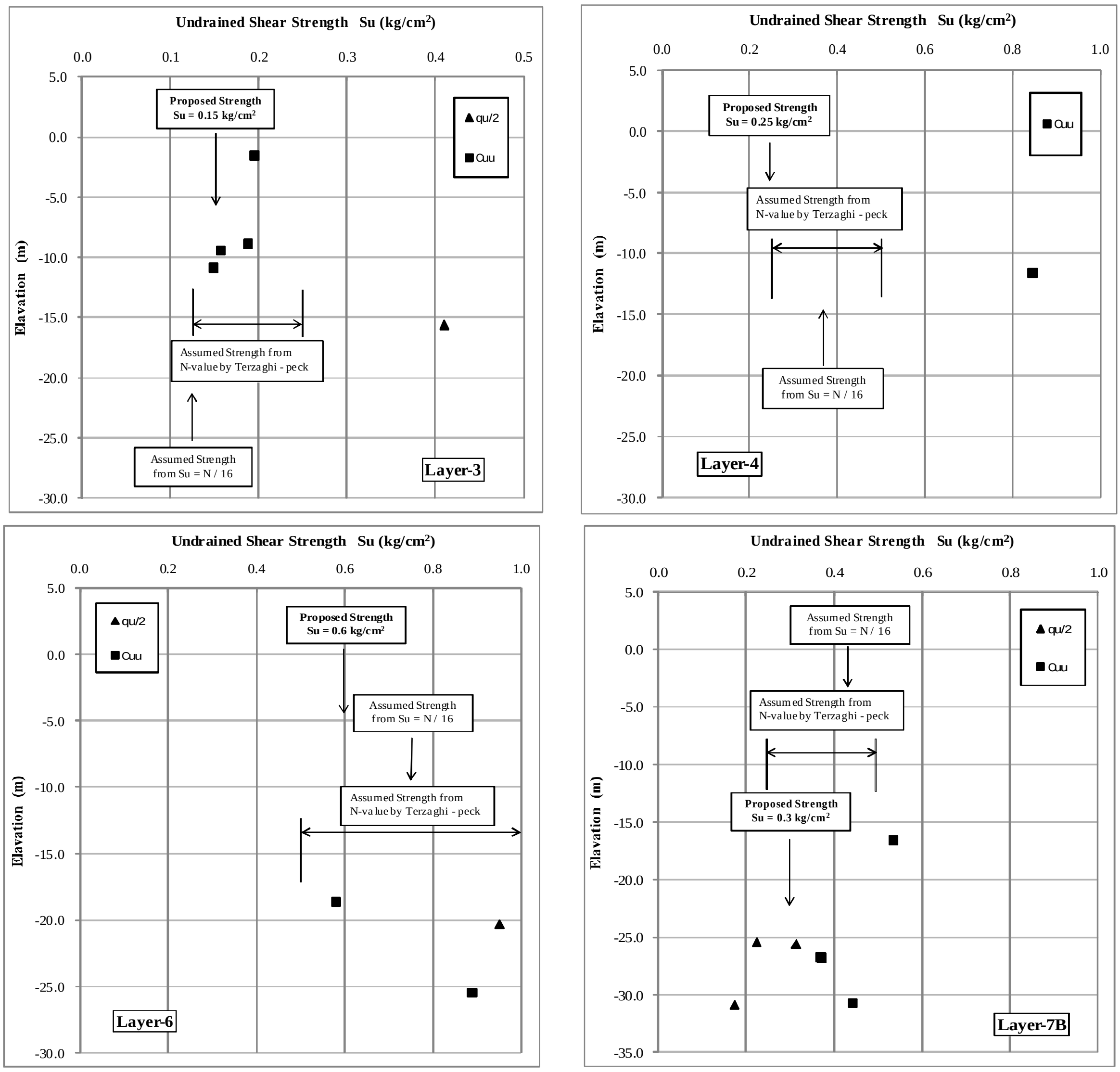
As a result of estimation, the shear strength of Layer-2, 5, 10A and 10B (sand layers) is proposed as follows.

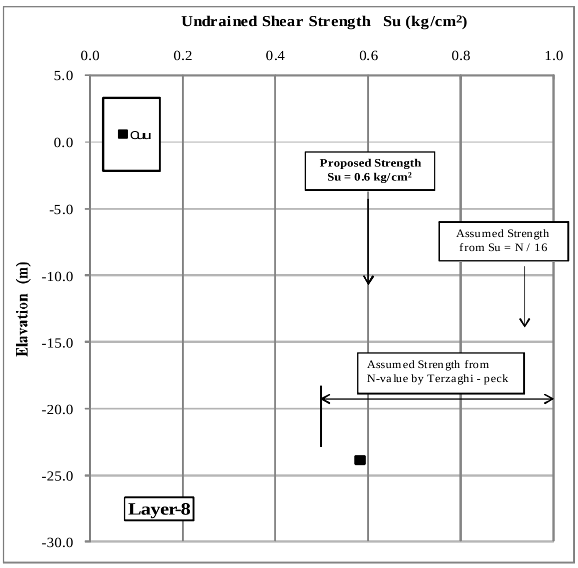
Table 3.4.3-3 Shear Strength of Sand (Bridge Area)

Layer			2	5	10A	10B
N-value (Average)			3	11	21	38
γτ		(g/cm^3)	(1.90)	(1.95)	(2.00)	(2.05)
ϕ	By Ohsaki $\phi=\sqrt{(20N)}+15$	(degree)	23	30	35	43
	By Peck, Dunham(1) $\phi=\sqrt{(12N)}+20$	(degree)	26	31	36	41
	By Dunham(2) $\phi=\sqrt{(12N)}+15$	(degree)	21	26	31	36
E	(E=28N)	(kg/cm^2)	84	308	588	1064
ϕ	**(proposed)**	(degree)	21	25	35	40

Note: The value in ( ) is assumed value.

Source: Study Team





Source: Study Team

Figure 3.4.3-2 Shear Strength of Clay for Short Term Stability (Bridge Area)

3.4.3.3 Consolidation Parameters

The soil Parameters necessary for settlement analysis are wet density (γt), initial void ratio (eo), compression index (Cc), swell index (Cs), coefficient of consolidation (Cv) and preconsolidation pressure (Pc).

The parameters of γt and eo are determined from results of unit weight test, while those of Cc, Cs, Cv and Pc are determined from the results of consolidation test.

Figures 3.4.3-3 and 3.4.3-4 present e~logP curve and logP~logCv curve for Layer-3, 4, 6, 7B and 8. The parameters of Cc, Cs and Pc are determined from e~logP curve, while the parameter of Cv is determined from logP~logCv curve.

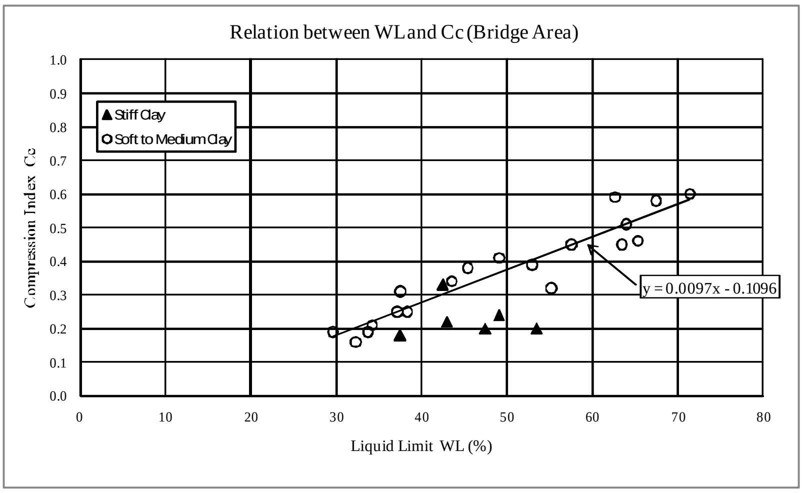
As a result of unit weight test and consolidation test, the soil parameters for settlement analysis are proposed as follows.

Table 3.4.3-4 Soil Parameters for Settlement Analysis (Bridge Area)

Item		3	4	6	7B	8
Unit Weight	γt (g/cm^3)	1.72	1.75	1.95	1.83	1.92
Initial Void Ratio	eo	1.314	1.297	0.777	1.056	0.838
Compression Index	Cc	0.390	0.385	0.234	0.345	0.200
Swell Index	Cs	0.042	0.076	0.052	0.057	0.058
Preconsolidation Pressure	Pc (kg/cm^2)	0.70	1.58	2.67	1.64	1.98
Coefficient of Consolidation	Cv (cm^2/day)	60	70	80	70	80

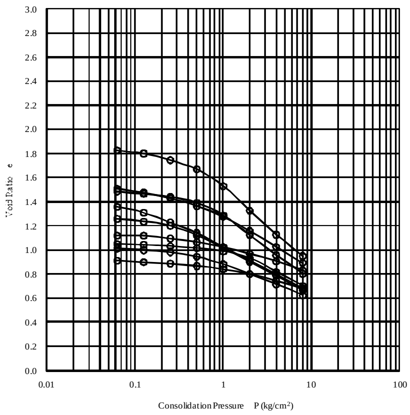
Source: Study Team

The parameter of Cc is very important for estimation of settlemet amount. Figure 3.4.3-5 presents the relationship between Cc and W_L. For both soft to medium clay and stiff clay, Cc increases with W_L. However, Cc of stiff clay is distributed lower than that of soft to medium clay, which means that the estimated amount of settlement of stiff clay is less than that of soft to medium clay.

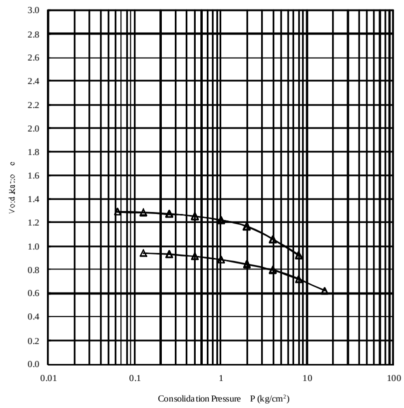


Source: Study Team

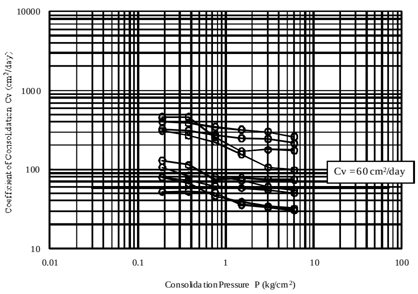
Figure 3.4.3-3 Relationship between Cc and W_L (Bridge Area)



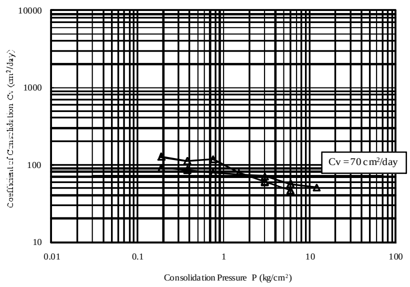
e ~ logP Curve (Bridge area: Layer-3)



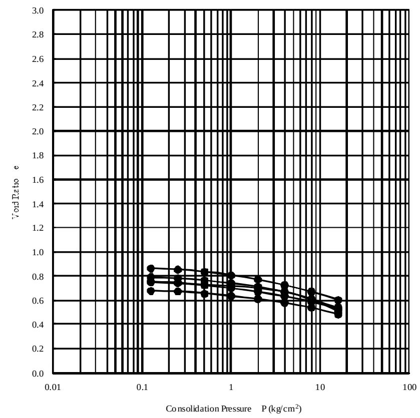
e ~ logP Curve (Bridge area: Layer-4)



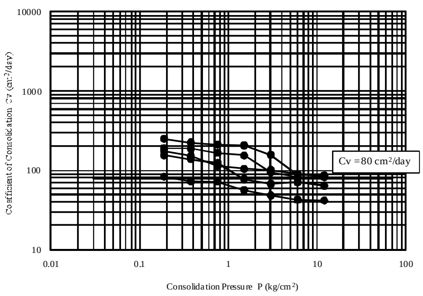
logP ~ logCv Curve (Bridge area: Layer-3)



logP ~ logCv Curve (Bridge area: Layer-4)



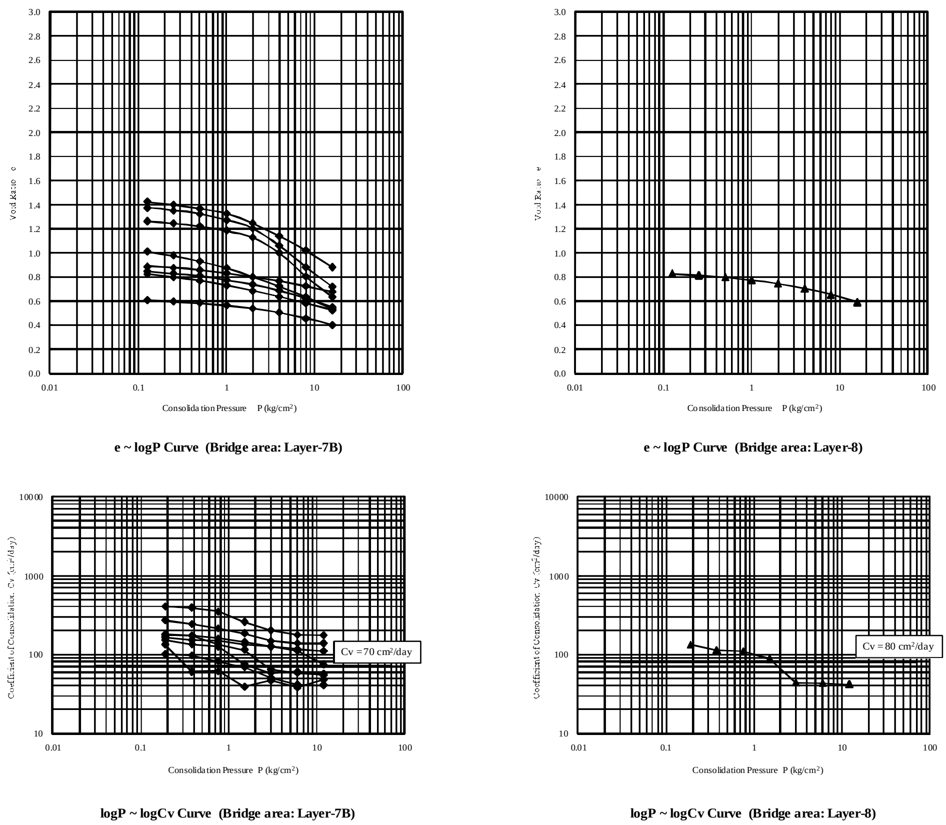
e ~ logP Curve (Bridge area: Layer-6)



logP ~ logCv Curve (Bridge area: Layer-6)

Source: Study Team

Figure 3.4.3-4 e~logP Curve and logP~logCv Curve (Bridge Area)-1/2



Source: Study Team

Figure 3.4.3-5 e~logP Curve and logP~logCv Curve (Bridge Area)-2/2

3.4.4 Rock Parameters

Based on the results of rock tests, the rock characteristics of Layer-12A and 12B of Bridge Area are summarized in Table 3.4.4-1. Figure 3.4.4-1 presents the relationship between bulk density and unconfined strength and Figure 3.4.4-2 shows the relationship between absorption and unconfined strength.

As Layer-12B is medium weathered silty sand stone, the bulk density is high and the absorption is low in general. However, there are some results of Layer-12B of which the bulk density is low and the absorption is high. This is due to highly weathered silty sand stone of Layer-12B.

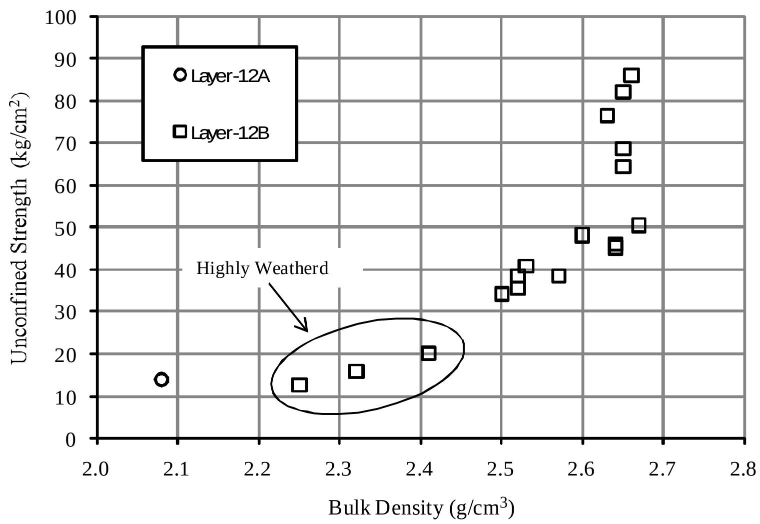
Therefore, the rock parameters of Layer-12B were determined using the rock test results without these highly weathered samples of Layer-12B, as shown in Table 3.4.4-2.

Table 3.4.4-1 Results of Rock Test (Bridge Area)

Layer	BoringNo.	Sample Depth (m)	Bulk Density (g/cm^3)	Unconfined Strength (kg/cm^2)		Softening Ratio	Absorption (%)
				Dry	Saturated		
12A	BP-21	41.3 - 41.5	2.08	14.0	9.0	0.64	-
	Average		2.08	14.0	9.0	0.64	-
12B	BP-1	44.2 - 44.5	2.63	76.5	-	-	2.7
	BP-3	45.8 - 46.0	(2.32)	(16.0)	-	-	(10.9)
	BP-6	46.3 - 46.5	2.65	82.0	-	-	1.9
	BP-16	47.0 - 47.8	2.66	86.0	40.0	0.47	-
	BP-30	43.0 - 43.2	2.65	68.7	-	-	1.9
	BP-36	44.0 - 44.2	(2.41)	(20.2)	-	-	(8.0)
	BP-42	43.5 - 43.7	2.57	38.5	-	-	3.5
	BP-47	43.5 - 43.8	2.52	35.6	-	-	4.8
	BP-55	47.8 - 48.0	2.53	40.8	-	-	4.9
	BP-62	45.0 - 45.3	2.64	45.2	-	-	1.9
	BP-68	41.9 - 42.0	2.67	50.4	-	-	1.3
	BP-73	42.3 - 42.5	2.65	64.3	-	-	2.3
	BP-77	41.4 - 41.5	2.64	46.1	-	-	2.0
	BP-80	41.0 - 41.1	(2.25)	(12.6)	-	-	(13.0)
	BP-81	33.5 - 33.6	2.52	38.6	-	-	5.5
	BP-86	37.0 - 37.2	2.50	34.2	-	-	5.5
	BP-92	43.2 - 43.5	2.60	48.2	-	-	3.0
	Minimum		2.50	34.2			1.3
	Maximum		2.67	86.0			5.5
	Average		2.60	53.9	40.0	0.47	3.2

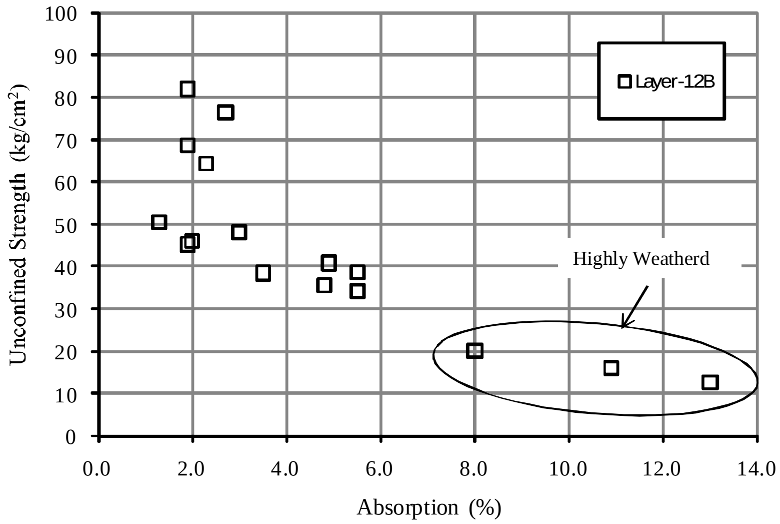
Note: Data in ( ) is excluded from data processing for minimum, maximum and average

Source: Study Team



Source: Study Team

Figure 3.4.4-1 Relationship between Unconfined Strength and Bulk Density (Bridge Area)



Source: Study Team

Figure 3.4.4-2 Relationship between Unconfined Strength and Absorption (Bridge Area)

Table 3.4.4-2 Rock Parameters for Design (Bridge Area)

Item		Layer-12A	Layer-12B
Unit Weight	(g/cm^3)	2.08	2.60
Absorption	(%)	-	3.2
Unconfined Strength	On dry condition (kg/cm^2)	14.0	53.9
	On saturated condition (kg/cm^2)	9.0	25.3
	Softening	0.64	0.47

Source: Study Team

3.4.5 Soil and Rock Parameters for Design

As a result of laboratory tests of soil and rock, shear strength analysis and consolidation analysis, the soil and rock parameters for design of Bridge Area are proposed as shown in Table 3.4.5-1.

However, there were no undisturbed samples of Layer-7A and 9 for unconfined compression test, triaxial compression (UU) test and consolidation test. Therefore, the soil parameters (γt, Su, eo, Cc, Cs, Pc, Cv) of Layer-7A are proposed using those of Layer-4, while the soil parameters (γt, Su, eo, Cc, Cs, Pc, Cv) of Layer-9 are proposed using those of Layer-7B.

Table 3.4.5-1 Soil and Rock Parameters for Design (Bridge Area)

Item				2 (Sand)	3 (Clay)	4 (Clay)	5 (Sand)	6 (Clay)	7A (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)	11 (Clay)	12A (Highly Weathered Siltstone)	12B (Moderately Weathered Siltstone)
N-value				3	2	6	11	12	4	7	15	7	21	38	18	Over 50	Over 50
Unit Weight		γt	(g/cm^3)	(1.90)	1.72	1.75	(1.95)	1.95	1.75	1.83	1.92	1.82	(2.00)	(2.05)	(1.95)	2.08	2.60
Absorption			(%)	-	-	-	-	-	-	-	-	-	-	-	-	-	3.20
Shear Strength	For Short Term	Su or Cd	(kg/cm^2)	0.00	0.15	0.25	0.00	0.60	0.25	0.30	0.60	0.30	0.00	0.00	(1.20)	-	-
		ϕu or ϕd	(degree)	21.0	0.0	0.0	25.0	0.0	0.0	0.0	0.0	0.0	35.0	40.0	0.0	-	-
	Rate of Strength Increase	m		-	0.2	0.2	-	0.2	0.2	0.2	0.2	0.2	-	-	-	-	-
Unconfined Strength of Rock	On Dry Condition		(kg/cm^2)	-	-	-	-	-	-	-	-	-	-	-	-	14.0	53.9
	On Saturated Condition		(kg/cm^2)	-	-	-	-	-	-	-	-	-	-	-	-	9.0	(25.3)
	Softening Ratio			-	-	-	-	-	-	-	-	-	-	-	-	0.64	0.47
Consolidation	Initial Void Ratio	eo		-	1.314	1.297	-	0.777	1.297	1.056	0.838	1.082	-	-	-	-	-
	Compression Index	Cc		-	0.390	0.385	-	0.234	0.385	0.345	0.200	0.367	-	-	-	-	-
	Swell Index	Cs		-	0.042	0.076	-	0.052	0.076	0.057	0.058	0.061	-	-	-	-	-
	Preconsolidation Pressure	Pc	(kg/cm^2)	-	0.70	1.58	-	2.67	1.58	1.64	1.98	1.74	-	-	-	-	-
	Coefficient of Consolidation	Cv	(cm^2/day)	-	60	70	-	80	70	70	80	70	-	-	-	-	-

Note (1): The value in ( ) is assumed.
Note (2): Adopt the soil parameters (γ t, Su, eo, Cc, Cs, Pc, Cv) of Layer-4 for Layer-7A
Note (3): Adopt the soil parameters (γ t, Su, eo, Cc, Cs, Pc, Cv) of Layer-7B for Layer-9

Source: Study Team

3.5 Subsoil Conditions of Approach Road Area in Cat Hai side

3.5.1 Stratum Classification

Sixteen main strata and five lens strata, as shown in Table 3.5.1-1, compose the subsoil of Approach Road Area in Cat Hai side. In this area, all main layers are found out.

The thickness of every layer varies from 0.9 to 6.6 m in average, as shown in Table 3.5.1-2. Layer-3 distributes thickest among them in this area, the thickness varies from 2.0 to 14.3 m, 6.6 m in average.

Based on the below stratum classification, the soil profile of Bridge Area can be drawn in Figures 3.5.1-1 and 3.5.1-2.

Table 3.5.1-1 Stratum Classification (Cat Hai side)

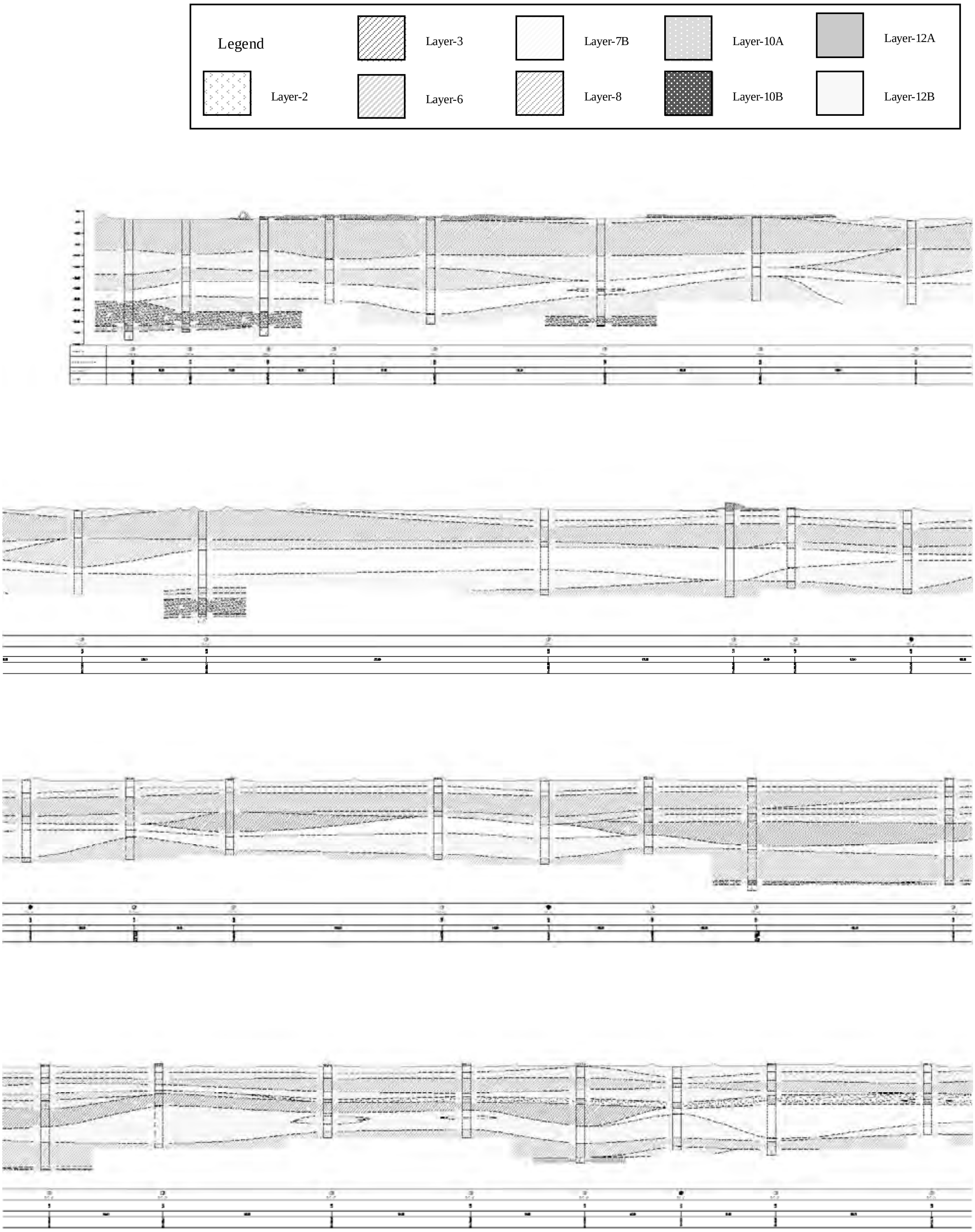
Layer	Soil Discription	Stiffness
Layer-D	**Filling soil and agriculture soil**	
Layer-1	**Clay** low plasticity	very soft to soft
Layer-2	**Sand** with clay, poorly graded	
Layer-3	**Clay** low plasticity	very soft to soft
Layer-4	**Clay** low plasticity	medium stiff
Layer-5	**Sand** with clay, poorly graded	
Layer-6	**Clay** low plasticity	stiff to very stiff
Layer-7A	**Clay** low plasticity	soft
Layer-7B	**Clay** low plasticity	medium stiff
Layer-8	**Clay** low plasticity	stiff to very stiff
Layer-9	**Clay** low plasticity	medium stiff
Layer-10A	**Sand** with clay, poorly graded	
Layer-10B	**Sand** with gravel, poorly graded	
Layer-11	**Clay** low plasticity	stiff to very stiff
Layer-12A	Heavy weathered **silty sand stone**, RQD=0~10%	
Layer-12B	Medium weathered **silty sand stone**, RQD=50%	
Layer-L5-1	**Sand** with clay, lens layer in Layer-5	
Layer-L6-1	**Sand** with clay, lens layer in Layer-6	
Layer-L7B-1	**Sand** with clay, lens layer in Layer-7B	
Layer-L8-1	**Clay**, lens layer in Layer-8	soft
Layer-L8-2	**Sand** with clay, lens layer in Layer-8	

Source: Study Team

Table 3.5.1-2 Thickness of Each Layer (Cat Hai side)

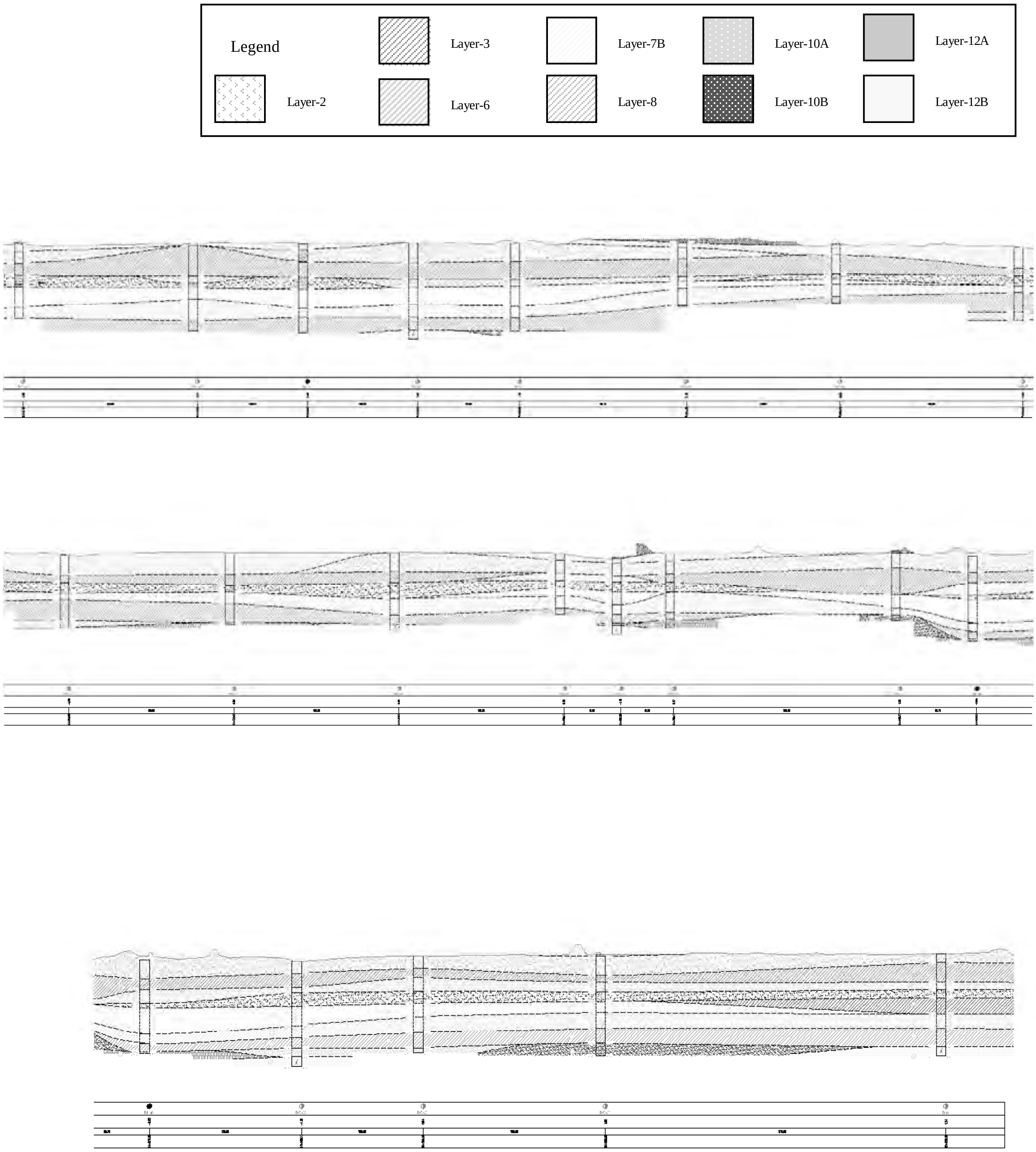
Layer	Soil	Thickness of Layer (m)		
		Minimum	Maximum	Average
D	Filling soil and agriculture soil	0.4	3.0	0.9
1	Clay (very soft to soft)	0.5	7.5	2.8
2	Sand	1.7	6.5	3.5
3	Clay (very soft to soft)	2.0	14.3	6.6
4	Clay (medium stiff)	1.0	8.5	3.6
5	Sand	1.0	5.4	2.8
L5-1	Sand (dense to very dense)	1.0	2.6	2.0
6	Clay (stiff to very stiff)	1.5	10.5	4.0
7A	Clay (soft)	3.0	9.4	5.9
7B	Clay (medium stiff)	1.5	12.5	5.5
8	Clay (stiff to very stiff)	1.0	13.9	4.9
9	Clay (medium stiff)	1.8	11.6	4.8
10A	Sand	0.7	7.8	3.5
10B	Sand	1.4	5.5	4.0
11	Clay (stiff to very stiff)	1.0	1.0	1.0
12A	Heavy weathered silty sand stone	1.0	3.0	2.0
12B	Medium weathered silty sand stone	-	-	-

Source: Study Team



Source: Study Team

Figure 3.5.1-1 Soil Profile of Approach Road Area in Cat Hai side (1/2)



Source: Study Team

Figure 3.5.1-2 Soil Profile of Approach Road Area in Cat Hai side (2/2)

3.5.2 N-value

N-value of each layer of Approach Road Area in Cat Hai side is summarized in Table 3.5.2-1.

Regarding clay layers, N-values of Layer-D, 1, 3, 4, 7A, 7B and 9 are low within the range of 2 to 6 in average, while N-values of Layer-6, 8 and 11 are high within the range of 11 to 21 in average. On the other hand, regarding sand layers, N-value of Layer-2 is 5 in average, while N-value of Layer-5 is 10 and that of Layer-10A is 14. Furthermore, N-values of Layer-10B and Layer-L5-1 (lens layer in Layer-5) are very high within the range of 42 to 57 in average. Layer-12A and 12B are weathered silty sand stone having N-value of more than 50.

Therefore, Layer-D, 1, 2, 3, 4, 7A, 7B and 9 are weak soil layers for embankment foundation.

Figure 3.5.2-1 N-value of Each Layer (Cat Hai side)

Layer	Soil	Minimum	Maximum	Average
D	Filling soil and agriculture soil	2	8	5
1	Clay (very soft to soft)	0	7	2
2	Sand	0	13	5
3	Clay (very soft to soft)	0	6	3
4	Clay (medium stiff)	3	9	5
5	Sand	3	23	10
6	Clay (stiff to very stiff)	6	14	11
7A	Clay (soft)	2	7	4
7B	Clay (medium stiff)	3	9	5
8	Clay (stiff to very stiff)	6	33	14
9	Clay (medium stiff)	2	11	6
10A	Sand	5	28	14
10B	Sand	30	57	42
11	Clay (stiff to very stiff)	21	21	21
12A	Heavy weathered silty sand stone	-	-	Over 50
12B	Medium weathered silty sand stone	-	-	Over 50
L5-1	Sand (dense to very dense)	30	76	57

Source: Study Team

3.5.3 Soil Parameters

3.5.3.1 Soil Characteristics

Based on the results of laboratory tests, the soil characteristics of each layer of Approach Road Area in Cat Hai side are summarized in Table 3.5.3-1. Figure 3.5.3-1 presents the chart of soil parameters for each layer of Approach Road Area in Cat Hai side.

Layer-1 has the soil characteristics with natural water content of 52.1%, wet density of 1.65 g/cm^3 and initial void ratio of 1.595 as very soft to soft clay.

Layer-3 has the soil characteristics with natural water content of 45.5%, wet density of 1.73 g/cm^3 and initial void ratio of 1.294 as very soft to soft clay.

Layer-4 has the soil characteristics with natural water content of 49.3%, wet density of 1.73 g/cm^3 and initial void ratio of 1.317 as medium stiff clay.

Layer-7A has the soil characteristics with natural water content of 38.0%, wet density of 1.75 g/cm^3 and initial void ratio 1.185 as soft clay.

Layer-7B has the soil characteristics with natural water content of 37.4%, wet density of 1.81 g/cm^3 and initial void ratio 1.078 as medium stiff clay.

Layer-9 has the soil characteristics with natural water content of 42.1%, wet density of 1.76 g/cm^3 and initial void ratio 1.200 as medium stiff clay.

On the other hand, Layer-6 has the soil characteristics with natural water content of 32.5%, wet density of 1.86 g/cm^3 and initial void ratio of 0.962 as stiff to very stiff clay.

Layer-8 has the soil characteristics with natural water content of 31.7%, wet density of 1.93 g/cm^3 and initial void ratio of 0.825 as stiff to very stiff clay.

Regarding sand layers, Layer-L5-1, L6-1, 10A and 10B has the soil characteristics with natural water content of 23.8%, 27.8%, 24.4% and 23.8% respectively.

Layer-2 has the soil characteristics with natural water content of 32.3%, wet density of 1.90 g/cm^3 and initial void ratio of 0.862 as loose sand.

Layer-5 has the soil characteristics with natural water content of 27.1%, wet density of 1.93 g/cm^3 and initial void ratio of 0.805 as medium dense sand.

As Pc (Preconsolidation Pressure) of all clay layers are distributed in nearly effective overburden pressure, clay layers of Approach Road Area in Cat Hai side are evaluated to be normally consolidated clay.

Table 3.5.3-1 Soil Characteristics of Each Layer (Cat Hai side)

Item				Layer											
				1 (Clay)	2 (Sand)	3 (Clay)	4 (Clay)	5 (Sand)	6 (Clay)	7A (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)
N-value				2	5	3	5	10	11	4	5	14	6	14	42
Natural Water Content			Wn (%)	52.1	32.3	45.5	49.3	27.1	32.5	38.0	37.4	31.7	42.1	24.4	23.8
Specific Gravity			Gs (g/cm^3)	2.69	2.68	2.70	2.70	2.67	2.71	2.69	2.70	2.71	2.71	2.67	2.66
Wet Density			γt (g/cm^3)	1.65	1.90	1.73	1.73	1.93	1.86	1.75	1.81	1.93	1.76	-	-
Dry Density			γd (g/cm^3)	1.05	1.44	1.19	1.18	1.49	1.39	1.25	1.33	1.49	1.23	-	-
Void Ratio			e_0	1.595	0.862	1.294	1.317	0.805	0.962	1.185	1.078	0.825	1.200	-	-
Saturation			Sr (%)	98.0	98.9	97.1	96.2	99.3	97.0	93.1	95.0	97.3	97.5	-	-
Liquid Limit			LL (%)	45.1	26.9	43.7	48.7	24.5	47.5	39.1	43.9	44.5	46.8	24.9	24.3
Plastic Limit			PL (%)	24.7	19.5	24.0	26.3	18.6	24.6	22.7	23.3	23.4	24.6	18.2	20.6
Plasticity Index			Ip	20.5	7.4	19.7	22.5	5.8	22.9	16.5	20.6	21.1	22.2	6.8	3.7
Unconfined Compression			qu (kg/cm^2)	0.18	-	0.30	0.27	-	1.74	-	0.51	2.15	0.32	-	-
Triaxial (UU)	Cohesion		Cuu (kg/cm^2)	0.10	0.23	0.15	-	-	-	-	0.21	0.49	0.34	-	-
	Internal Friction		ϕuu (degree)	0.4	1.9	0.7	-	-	-	-	0.4	1.1	0.3	-	-
Triaxial (CU)	Total Stress	Cohesion	Ccu (kg/cm^2)	0.14	-	0.12	0.15	-	0.23	-	0.31	0.08	0.20	-	-
		Internal Friction	ϕcu (degree)	12.4	-	16.5	14.3	-	17.4	-	19.5	20.0	13.8	-	-
	Effective Stress	Cohesion	C' (kg/cm^2)	0.12	-	0.08	0.11	-	0.13	-	0.12	0.02	0.10	-	-
		Internal Friction	ϕ' (degree)	23.5	-	26.2	23.4	-	25.6	-	26.3	25.4	24.6	-	-
Consolidation	Compression Index		Cc	0.527	0.075	0.397	0.386	0.170	0.220	0.430	0.351	0.193	0.457	-	-
	Swell Index		Cs	0.070	0.005	0.054	0.054	0.014	0.042	0.059	0.054	0.031	0.067	-	-
	Preconsolidation Pressure		Pc (kg/cm^2)	0.34	0.43	0.66	0.83	1.10	1.46	1.15	1.13	1.44	1.51	-	-

Source: Study Team

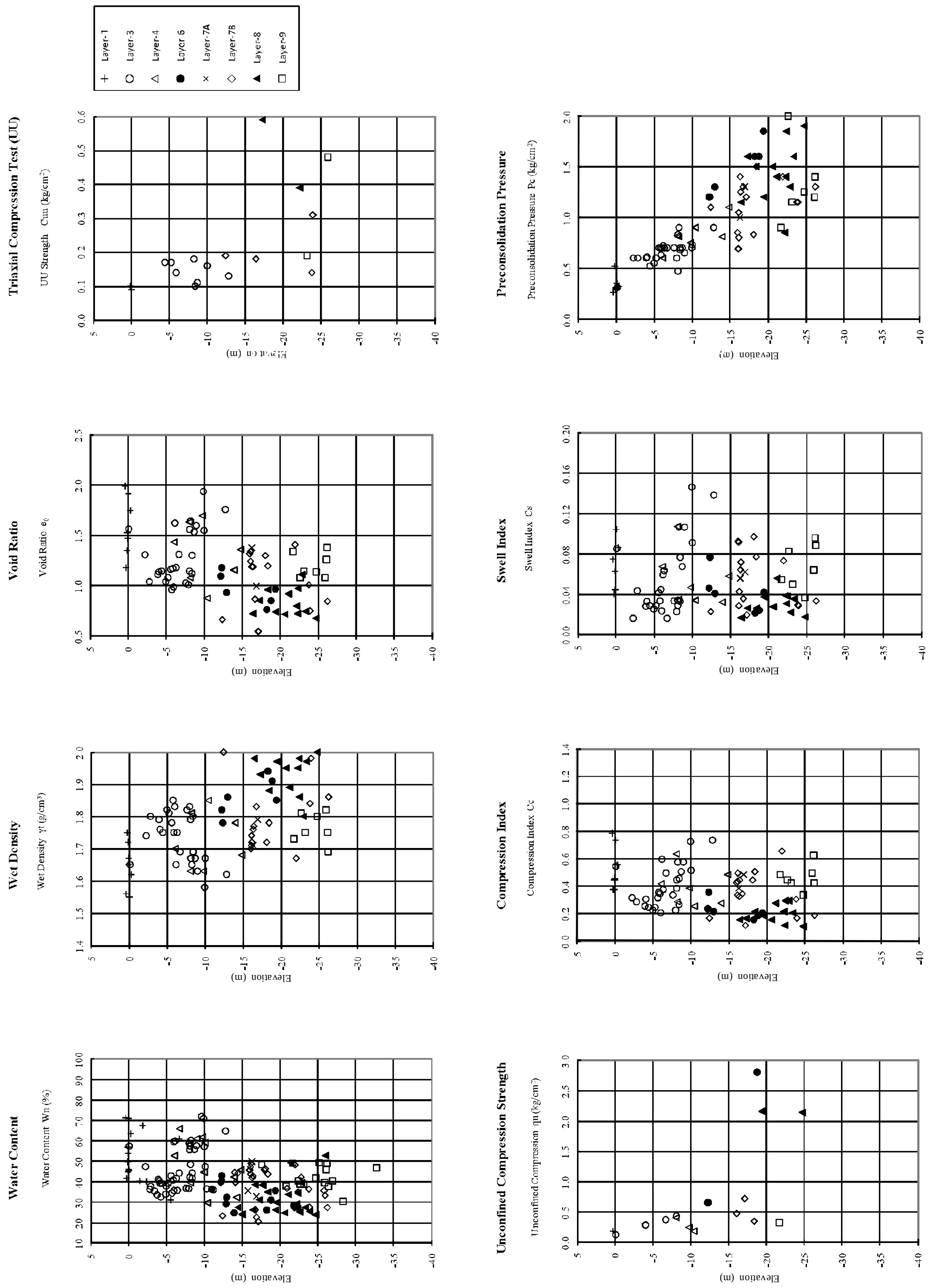


Figure 3.5.3-1 Chart of Soil Parameters (Cat Hai side)

Source: Study Team

3.5.3.2 Shear Strength

(1) Shear Strength of Clay for Short Term Stability

The shear strength for short term stability of Layer-1, 3, 4, 6, 7A, 7B, 8 and 9 (clay layers) is determined based on the results of FVST, unconfined compression test, triaxial compression (UU) test and N-value.

However, there were some unreliable results of unconfined compression test due to disturbance of sample during sampling, transportation and laboratory. Therefore, the results of unconfined compression test of which the strain at failure is more than 7% were excluded from data analysis for shear strength.

Figures 3.5.3-2 and 3.5.3-3 show the relationship between Su and elevation using all data of FVST, qu/2, Cuu and assumed strength from N-value. From these results, the shear strength for short term stability of each layer is proposed as follows:

Table 3.5.3-2 Shear Strength of Clay for Short Term Stability (Cat Hai side)

Layer	Shear Strength Su (kg/cm^2)
1	Su = 0.1 kg/cm^2 (Down to El +/- 0m) Su = 0.1 + 0.02 x Z kg/cm^2 (Below EL +/- 0m)
3	Su = 0.15 kg/cm^2
4	Su = 0.1 + 0.02 x Z kg/cm^2 (Below EL -5m)
6	Su = 0.6 kg/cm^2
7A	Su = 0.2 kg/cm^2
7B	Su = 0.25 kg/cm^2
8	Su = 0.8 kg/cm^2
9	Su = 0.25 kg/cm^2

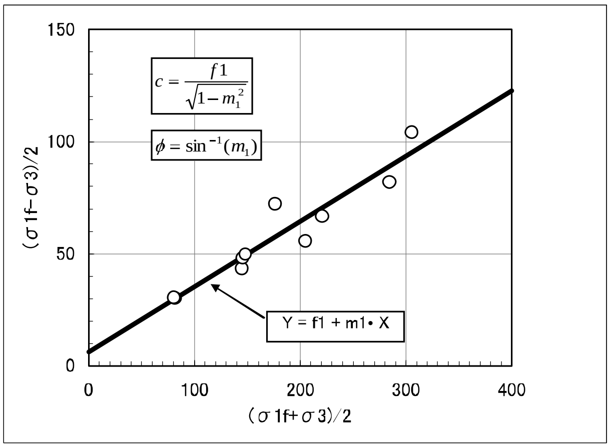
Source: Study Team

(2) Shear Strength of Clay for Long Term Stability

The shear strength for long term stability of Layer-1, 3, 4, 6, 7B, 8 and 9 (clay layers) is determined based on the results of triaxial compression (CU) test.

The shear strength of C and ϕ in total stress and effective stress can be determined using $(\sigma_{1f}+\sigma_3)/2$ and $(\sigma_{1f}-\sigma_3)/2$ of all data of triaxial compression (CU) test as shown in Figure 3.5.3-4.

Figures 3.5.3-5 and 3.5.3-6 present the results of analysis of triaxial compression (CU) tests. As a result of examination, the design criteria of shear strength of Layer-1, 3, 4, 6, 7B, 8 and 9 in total stress and effective stress is proposed as follows.



Source: Basis and Guide of Soil Test (Japanese Geotechnical Society)

Figure 3.5.3-2 Determination Method of C and ϕ using $(\sigma_{1f}+\sigma_3)/2$ and $(\sigma_{1f}-\sigma_3)$

Table 3.5.3-3 Shear Strength of Clay for Long Term Stability (Cat Hai side)

Layer	Total Stress		Effective Stress	
	Ccu (kg/cm^2)	ϕcu (degree)	C' (kg/cm^2)	ϕ' (degree)
1	0.11	13.5	0.10	25.2
3	0.01	21.2	0.02	30.5
4	0.19	9.7	0.14	21.5
6	0.22	17.6	0.16	24.9
7B	0.31	19.5	0.12	26.3
8	0.08	20.0	0.03	25.2
9	0.20	13.8	0.11	24.5

Source: Study Team

(3) Shear Strength of Sand

The shear strength of Layer-2, 5, L5-1, L6-1, 10A and 10B (sand layers) is determined based on N-value. The shear strength of ϕ of sand layer can be estimated using three formulas by Ohsaki, Peck and Dunham.

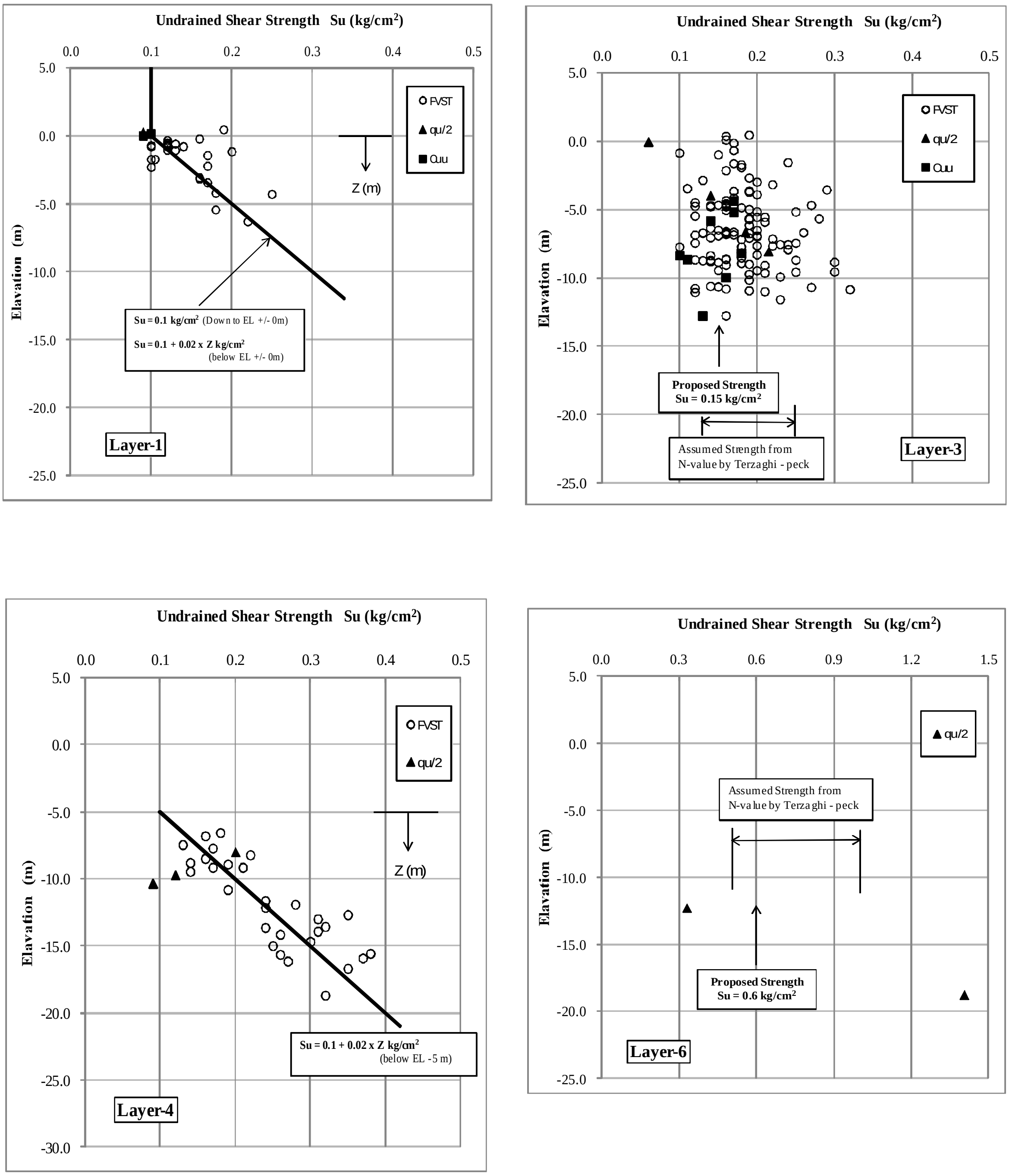
As a result of estimation, the shear strength of Layer-2, 5, L5-1, L6-1, 10A and 10B (sand layers) is proposed as follows.

Table 3.5.3-4 Shear Strength of Sand (Cat Hai side)

Layer		2	5	L5-1	L6-1	10A	10B
N-value (Average)		5	10	57	9	14	42
$\gamma\tau$ (g/cm^3)		1.90	1.93	(2.10)	(1.93)	(1.95)	(2.05)
ϕ	By Ohsaki (degree) $\phi=\sqrt{(20N)}+15$	25	29	49	28	32	44
	By Peck, Dunham(1) (degree) $\phi=\sqrt{(12N)}+20$	28	31	46	30	33	42
	By Dunham(2) (degree) $\phi=\sqrt{(12N)}+15$	23	26	41	25	28	37
E	(E=28N) (kg/cm^2)	140	280	1596	252	392	1176
ϕ	**(proposed)** (degree)	23	25	45	25	30	40

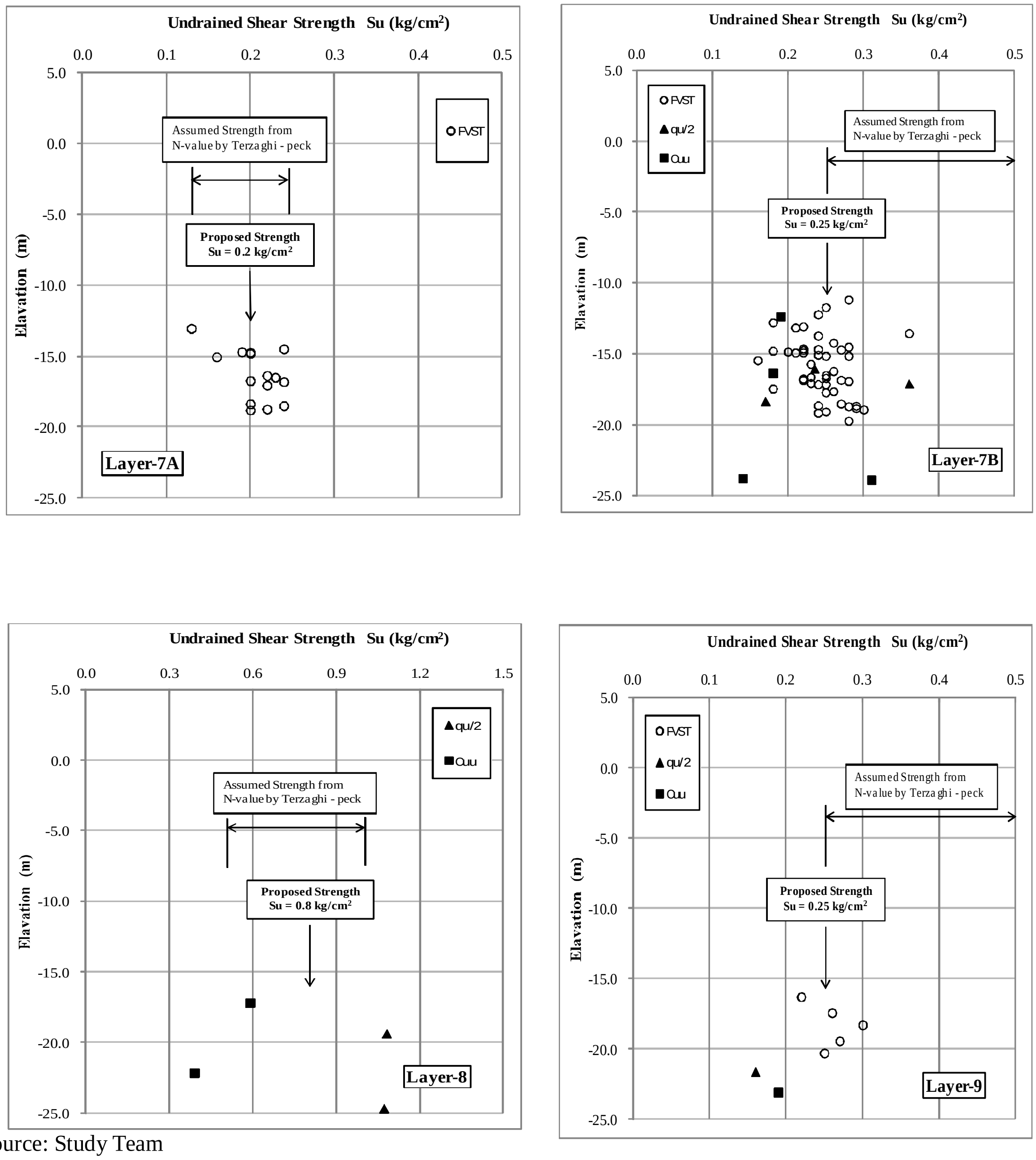
Note: The value in ( ) is assumed value.

Source: Study Team



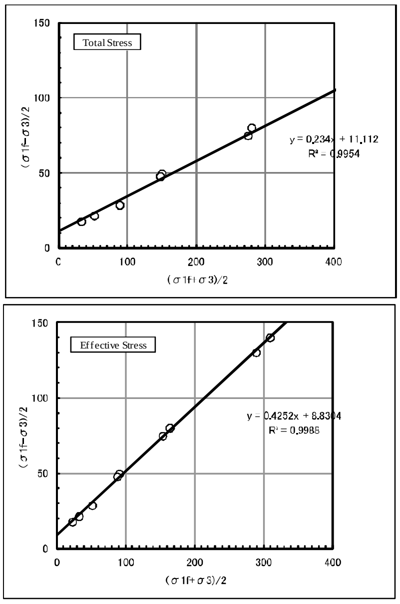
Source: Study Team

Figure 3.5.3-3 Shear Strength of Clay for Short Term Stability (Cat Hai side)-1/2



Source: Study Team

Figure 3.5.3-4 Shear Strength of Clay for Short Term Stability (Cat Hai side)-2/2



Y=f1+m1・X

$c = \frac{f1}{\sqrt{1-m_1^2}}$

$\phi = \sin^{-1}(m_1)$

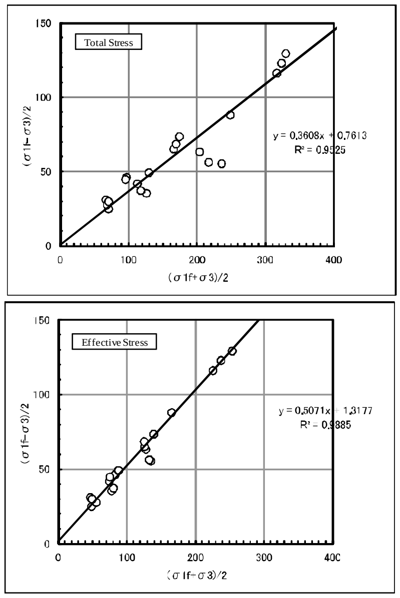
Total Stress:

f1	11.112	
m1	0.234	
Ccu	11.43	(kN/m²)
φ	0.23619	(rad)
φcu	13.53	(deg)

Effective Stress:

f1	8.8304	
m1	0.4252	
C'	9.76	(kN/m²)
φ	0.439183	(rad)
φ'	25.16	(deg)

Analysis of CU Test (Layer-1)



Y=f1+m1・X

$c = \frac{f1}{\sqrt{1-m_1^2}}$

$\phi = \sin^{-1}(m_1)$

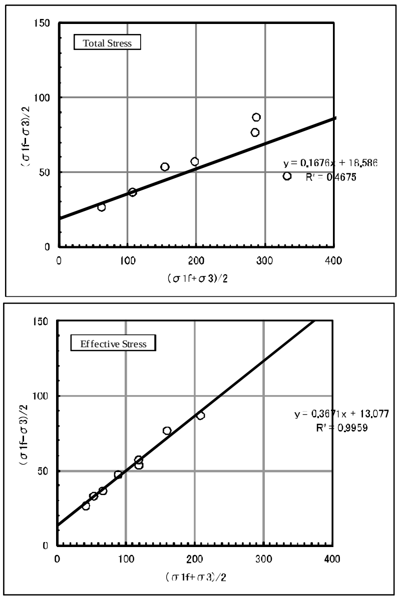
Total Stress:

f1	0.7613	
m1	0.3608	
Ccu	0.82	(kN/m²)
φ	0.369126	(rad)
φcu	21.15	(deg)

Effective Stress:

f1	1.3177	
m1	0.5071	
C'	1.53	(kN/m²)
φ	0.531817	(rad)
φ'	30.47	(deg)

Analysis of CU Test (Layer-3)



Y=f1+m1・X

$c = \frac{f1}{\sqrt{1-m_1^2}}$

$\phi = \sin^{-1}(m_1)$

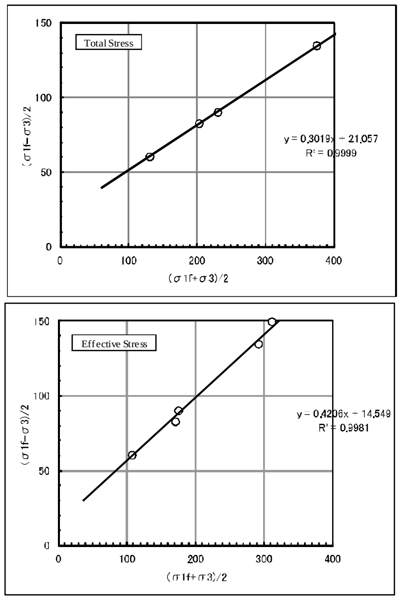
Total Stress:

f1	18.586	
m1	0.1676	
Ccu	18.85	(kN/m²)
φ	0.168395	(rad)
φcu	9.65	(deg)

Effective Stress:

f1	13.077	
m1	0.3671	
C'	14.06	(kN/m²)
φ	0.375889	(rad)
φ'	21.51	(deg)

Analysis of CU Test (Layer-4)



Y=f1+m1・X

$c = \frac{f1}{\sqrt{1-m_1^2}}$

$\phi = \sin^{-1}(m_1)$

Total Stress:

f1	21.057	
m1	0.3019	
Ccu	22.09	(kN/m²)
φ	0.306685	(rad)
φcu	17.57	(deg)

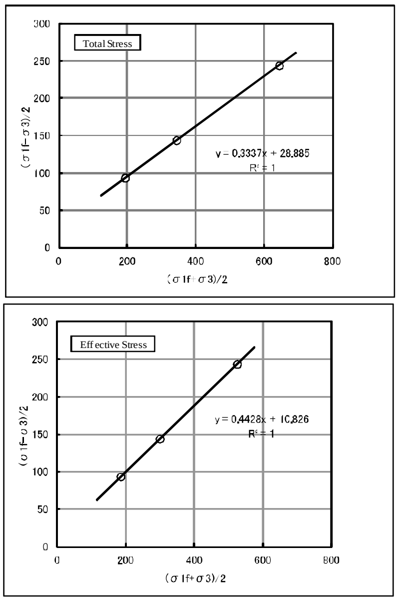
Effective Stress:

f1	14.549	
m1	0.4206	
C'	16.04	(kN/m²)
φ	0.434107	(rad)
φ'	24.87	(deg)

Analysis of CU Test (Layer-6)

Source: Study Team

Figure 3.5.3-5 Shear Strength of Clay for Long Term Stability (Cat Hai side)-1/2



Y=f1+m1・X

$c=\frac{f1}{\sqrt{1-m_1^2}}$

$\phi=\sin^{-1}(m_1)$

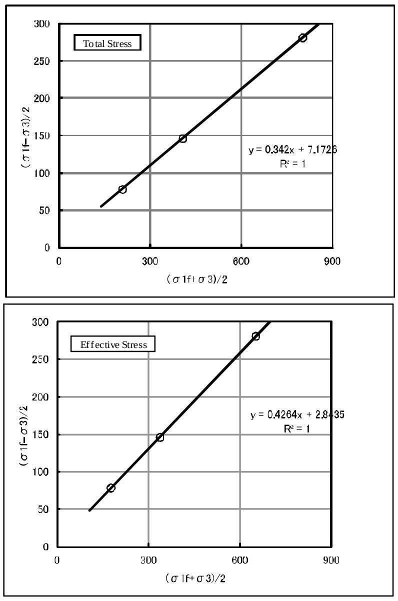
Total Stress:

f1	28.885	
m1	0.3337	
Ccu	30.64	(kN/m²)
ϕ	0.340226	(rad)
ϕcu	19.49	(deg)

Effective Stress:

f1	10.826	
m1	0.4428	
C'	12.07	(kN/m²)
ϕ	0.458719	(rad)
ϕ'	26.28	(deg)

Analysis of CU Test (Layer-7B)



Y=f1+m1・X

$c=\frac{f1}{\sqrt{1-m_1^2}}$

$\phi=\sin^{-1}(m_1)$

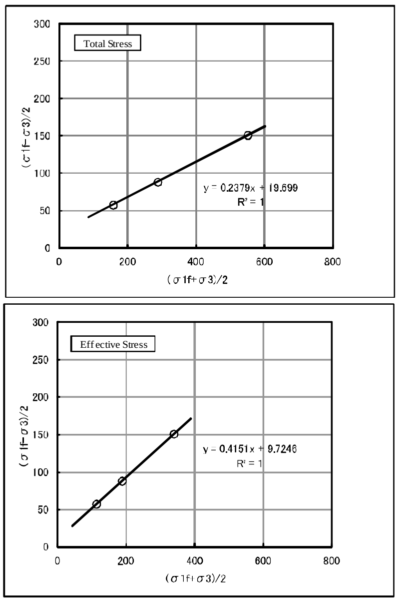
Total Stress:

f1	7.1726	
m1	0.342	
Ccu	7.63	(kN/m²)
ϕ	0.349044	(rad)
ϕcu	20.00	(deg)

Effective Stress:

f1	2.8435	
m1	0.4264	
C'	3.14	(kN/m²)
ϕ	0.440509	(rad)
ϕ'	25.24	(deg)

Analysis of CU Test (Layer-8)



Y=f1+m1・X

$c=\frac{f1}{\sqrt{1-m_1^2}}$

$\phi=\sin^{-1}(m_1)$

Total Stress:

f1	19.699	
m1	0.2379	
Ccu	20.28	(kN/m²)
ϕ	0.240203	(rad)
ϕcu	13.76	(deg)

Effective Stress:

f1	9.7246	
m1	0.4151	
C'	10.69	(kN/m²)
ϕ	0.428053	(rad)
ϕ'	24.53	(deg)

Analysis of CU Test (Layer-9)

Source: Study Team

Figure 3.5.3-6 Shear Strength of Clay for Long Term Stability (Cat Hai side)-2/2

3.5.3.3 Consolidation Parameters

The soil Parameters necessary for settlement analysis are wet density (γt), initial void ratio (eo), compression index (Cc), swell index (Cs), coefficient of consolidation (Cv) and preconsolidation pressure (Pc).

The parameters of γt and eo are determined from results of unit weight test, while those of Cc, Cs, Cv and Pc are determined from the results of consolidation test.

Figures 3.5.3-7 and 3.5.3-8 present e~logP curve and logP~logCv curve for Layer-1, 3, 4, 6, 7A, 7B, 8 and 9. The parameters of Cc, Cs and Pc are determined from e~logP curve, while the parameter of Cv is determined from logP~logCv curve.

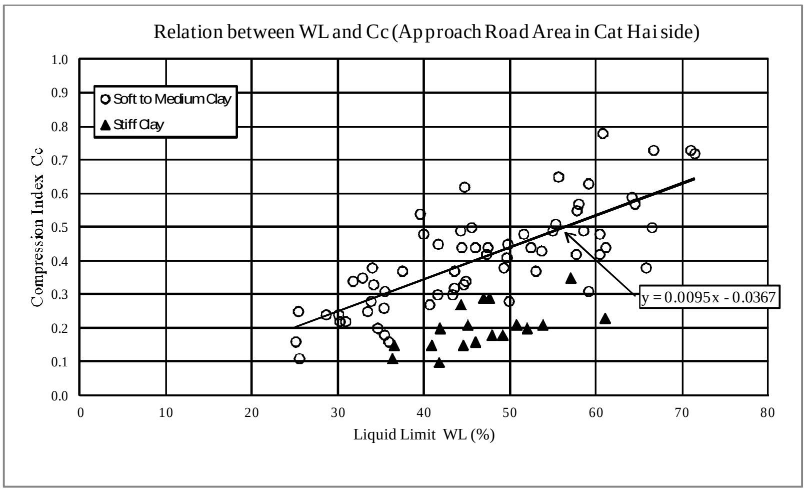
As a result of unit weight test and consolidation test, the soil parameters for settlement analysis are proposed as follows.

Table 3.5.3-5 Soil Parameters for Settlement Analysis (Cat Hai side)

Item		1	3	4	6	7A	7B	8	9
Unit Weight	γt (g/cm^3)	1.65	1.73	1.73	1.86	1.75	1.81	1.93	1.76
Initial Void Ratio	eo	1.595	1.294	1.317	0.962	1.185	1.078	0.825	1.200
Compression Index	Cc	0.527	0.397	0.386	0.220	0.430	0.351	0.193	0.457
Swell Index	Cs	0.070	0.054	0.054	0.042	0.059	0.054	0.031	0.067
Preconsolidation Pressure	Pc (kg/cm^2)	0.34	0.66	0.83	1.46	1.15	1.13	1.44	1.51
Coefficient of Consolidation	Cv (cm^2/day)	40	60	70	150	80	80	150	80

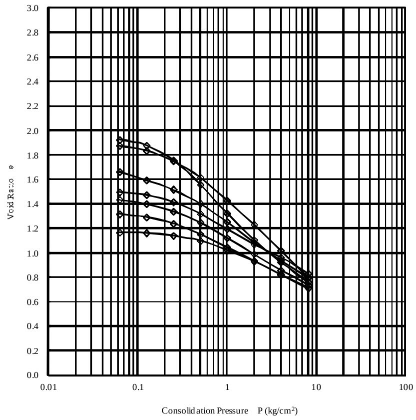
Source: Study Team

The parameter of Cc is very important for estimation of settlemet amount. Figure 3.5.3-9 presents the relationship between Cc and W_L. For both soft to medium clay and stiff clay, Cc increases with W_L. However, Cc of stiff clay is distributed lower than that of soft to medium clay, which means that the estimated amount of settlement of stiff clay is less than that of soft to medium clay.

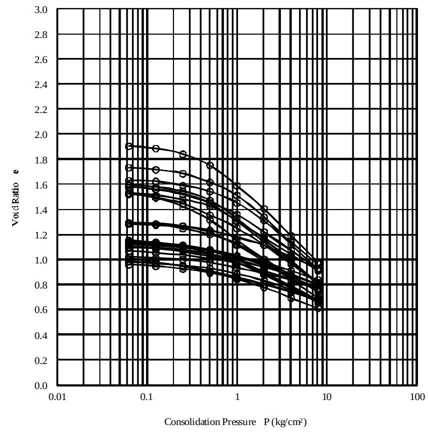


Source: Study Team

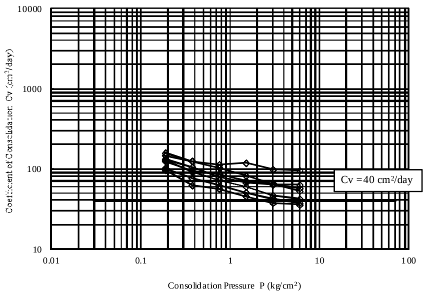
Figure 3.5.3-7 Relationship between Cc and W_L (Cat Hai side)



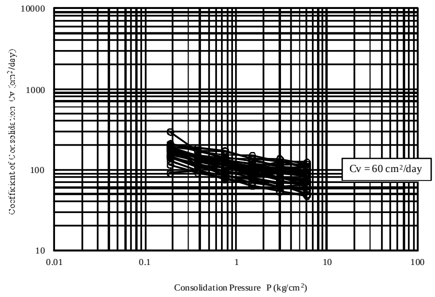
e ~ logP Curve (Cat Hai area: Layer-1)



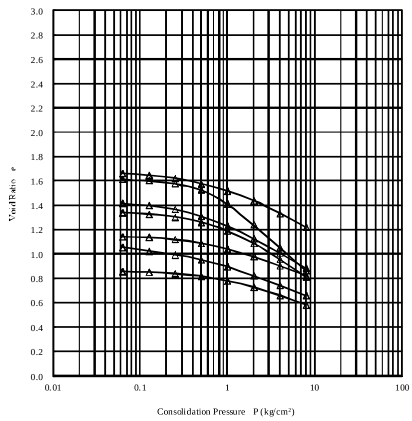
e ~ logP Curve (Cat Hai area: Layer-3)



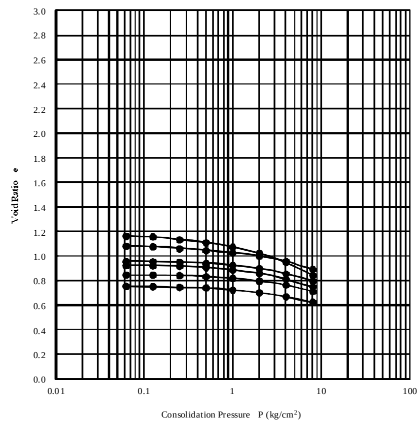
logP ~ logCv Curve (Cat Hai area: Layer-1)



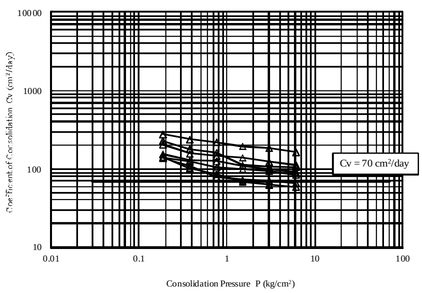
logP ~ logCv Curve (Cat Hai area: Layer-3)



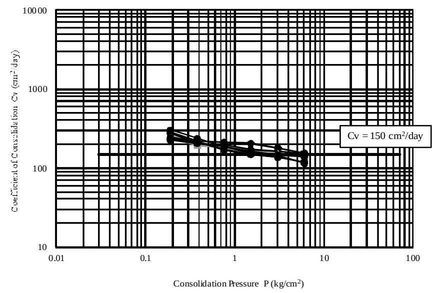
e ~ logP Curve (Cat Hai area: Layer-4)



e ~ logP Curve (Cat Hai area: Layer-6)



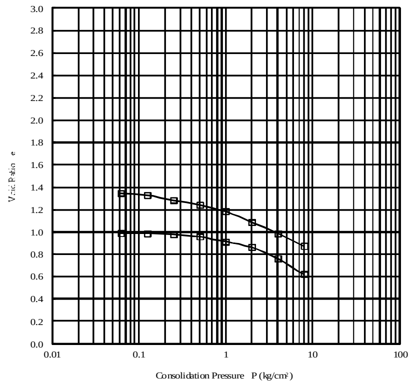
logP ~ logCv Curve (Cat Hai area: Layer-4)



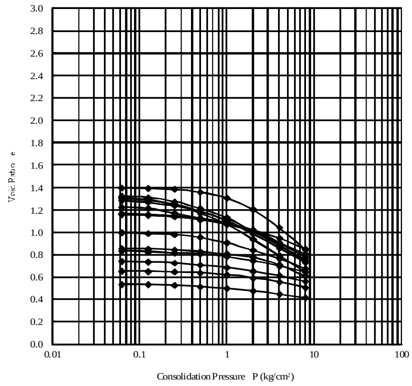
logP ~ logCv Curve (Cat Hai area: Layer-6)

Source: Study Team

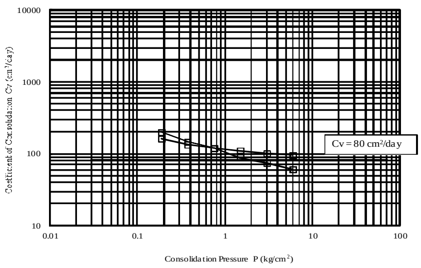
Figure 3.5.3-8 e~logP Curve and logP~logCv Curve (Cat Hai side)-1/2



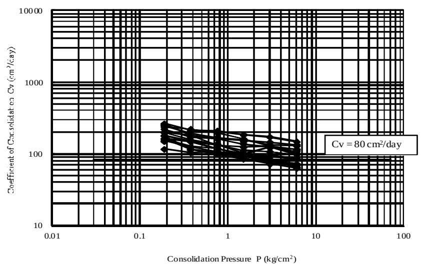
e ~ logP Curve (Cat Hai area: Layer-7A)



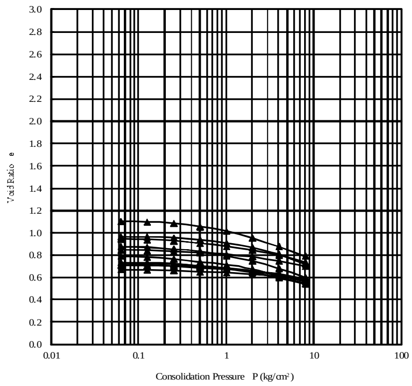
e ~ logP Curve (Cat Hai area: Layer-7B)



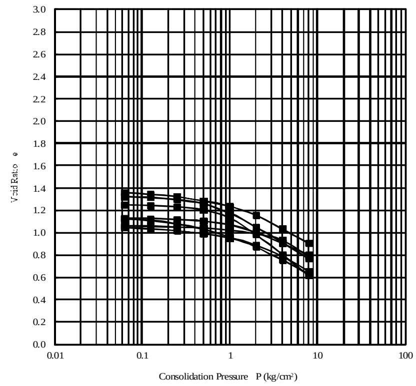
logP ~ logCv Curve (Cat Hai area: Layer-7A)



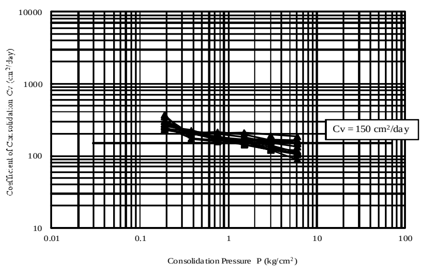
logP ~ logCv Curve (Cat Hai area: Layer-7B)



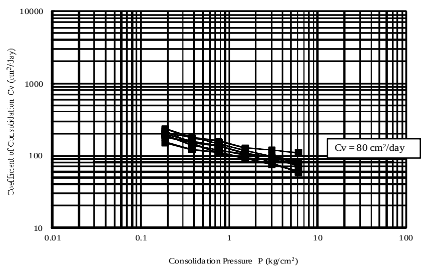
e ~ logP Curve (Cat Hai area: Layer-8)



e ~ logP Curve (Cat Hai area: Layer-9)



logP ~ logCv Curve (Cat Hai area: Layer-8)



logP ~ logCv Curve (Cat Hai area: Layer-9)

Source: Study Team

Figure 3.5.3-9 e~logP Curve and logP~logCv Curve (Cat Hai side)-2/2

3.5.4 Soil Parameters for Design

As a result of laboratory tests, shear strength analysis and consolidation analysis, the soil parameters for design of Approach Road Area in Cat Hai side are proposed as shown in Table 3.5.4-1.

However, there were no triaxial compression (CU) tests for Layer-7A. Therefore, the soil parameters (Ccu, ϕcu, C', ϕ') of Layer-7A are proposed using those of Layer-7B.

Table 3.5.4-1 Soil Parameters for Design (Cat Hai side)

Item					Layer													
					1 (Clay)	2 (Sand)	3 (Clay)	4 (Clay)	5 (Sand)	L5-1 (Sand)	6 (Clay)	L6-1 (Sand)	7A (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)
N-value					2	5	3	5	10	57	11	9	4	5	14	6	14	42
Unit Weight			γt	(g/cm^3)	1.65	1.90	1.73	1.73	1.93	(2.10)	1.86	(1.93)	1.75	1.81	1.93	1.76	(1.95)	(2.05)
Shear Strength	For Short Term		Su or Cd	(kg/cm^2)	Note (2)	0.00	0.15	Note (3)	0.00	0.00	0.60	0.00	0.20	0.25	0.80	0.25	0.00	0.00
			ϕu or ϕd	(degree)	0.0	23.0	0.0	0.0	25.0	45.0	0.0	25.0	0.0	0.0	0.0	0.0	30.0	40.0
	For Long Term	Total Stress	Ccu	(kg/cm^2)	0.11	-	0.01	0.19	-	-	0.22	-	0.31	0.31	0.08	0.20	-	-
			ϕ cu	(degree)	13.5	-	21.2	9.7	-	-	17.6	-	19.5	19.5	20.0	13.8	-	-
		Effective Stress	C'	(kg/cm^2)	0.10	-	0.02	0.14	-	-	0.16	-	0.12	0.12	0.03	0.11	-	-
			ϕ '	(degree)	25.2	-	30.5	21.5	-	-	24.9	-	26.3	26.3	25.2	24.5	-	-
	Rate of Strength Increase		m		0.2	-	0.2	0.2	-	-	-	-	0.2	0.2	-	-	-	-
Consolidation	Initial Void Ratio		eo		1.595	-	1.294	1.317	-	-	0.962	-	1.185	1.078	0.825	1.200	-	-
	Compression Index		Cc		0.527	-	0.397	0.386	-	-	0.220	-	0.430	0.351	0.193	0.457	-	-
	Swell Index		Cs		0.070	-	0.054	0.054	-	-	0.042	-	0.059	0.054	0.031	0.067	-	-
	Preconsolidation Pressure		Pc	(kg/cm^2)	0.34	-	0.66	0.83	-	-	1.46	-	1.15	1.13	1.44	1.51	-	-
	Coefficient of Consolidation		Cv	(cm^2/day)	40	-	60	70	-	-	150	-	80	80	150	80	-	-

Note (1): The value in ( ) is assumed.
Note (2): Su = 0.1 kg/cm^2 (Down to EL 0.0m), Su = 0.1 + 0.02 x Z kg/cm^2 (below EL 0.0m, Zo = EL 0.0m)
Note (3): Su = 0.1 + 0.02 x Z kg/cm^2 (below EL -5m, Zo = EL -5.0m)
Note (4): Adopt the soil parameters (Ccu, ϕ cu, C', ϕ ') of Layer-7B for Layer-7A

Source: Study Team

3.6 Conclusions and Recommendations

3.6.1 Conclusions

As a result of geotechnical investigation in the project area, the following conclusions were grasped.

- The subsoil of project area is mainly composed of ten clay layers, four sand layers and two weathered silty sand stone as shown in Table 3.6.1-1.
- In Approach Road Area of Hai An side, Layer-1, 2, 5, 7A and 11 are not found out. Layer-3 distributes thickest with the thickness of 7.5 to 30.5 m, 17.3 m in average.
- In Bridge Area, Layer-D and 1 are not found out. Layer-12A and Layer-12B are evaluated to be the bearing strata for pile foundation.
- In Approach Road Area of Cat Hai side, all main layers are found out. Layer-3 distributes thickest with the thickness of 2.0 to 14.3 m, 6.6 m in average. Furthermore, Layer-L5-1 of lens layer in Layer-5 is found out having N-value of 57 in average.

Table 3.6.1-1 Stratum Classification (Main Layer)

Layer	Soil Discription	Stiffness
D	**Filling soil and agriculture soil**	
1	**Clay** low plasticity	very soft to soft
2	**Sand** with clay, poorly graded	
3	**Clay** low plasticity	very soft to soft
4	**Clay** low plasticity	medium stiff
5	**Sand** with clay, poorly graded	
6	**Clay** low plasticity	stiff to very stiff
7A	**Clay** low plasticity	soft
7B	**Clay** low plasticity	medium stiff
8	**Clay** low plasticity	stiff to very stiff
9	**Clay** low plasticity	medium stiff
10A	**Sand** with clay, poorly graded	
10B	**Sand** with gravel, poorly graded	
11	**Clay** low plasticity	stiff to very stiff
12A	Heavy weathered **silty sand stone**, RQD=0~10%	
12B	Medium weathered **silty sand stone**, RQD=50%	

Source: Study Team

- As a result of laboratory tests, shear strength analysis and consolidation analysis, the soil parameters for design of Approach Road Area in Hai An side are proposed as shown in Table 3.6.1-2.

Table 3.6.1-2 Soil Parameters for Design (Hai An side)

Item					3 (Clay)	4 (Clay)	6 (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)
N-value					2	7	13	7	14	6	21	49
Unit Weight			γt	(g/cm^3)	1.70	(1.76)	1.86	1.76	1.88	(1.76)	(2.00)	(2.05)
Shear Strength	For Short Term		Su or Cd	(kg/cm^2)	Note (2)	0.25	0.50	0.25	0.60	(0.25)	0.00	0.00
			φu or φd	(degree)	0.0	0.0	0.0	0.0	0.0	0.0	35.0	40.0
	For Long Term	Total Stress	Ccu	(kg/cm^2)	0.13	0.06	0.16	0.06	0.16	0.06	-	-
			φcu	(degree)	14.3	17.1	15.9	17.1	15.9	17.1	-	-
		Effective Stress	C'	(kg/cm^2)	0.06	0.09	0.12	0.09	0.12	0.09	-	-
			φ'	(degree)	26.5	22.5	23.3	22.5	23.3	22.5	-	-
	Rate of Strength Increase		m		0.2	0.2	0.2	0.2	0.2	0.2	-	-
Consolidation	Initial Void Ratio		eo		1.384	1.225	0.943	1.225	0.851	1.225	-	-
	Compression Index		Cc		0.435	0.420	0.302	0.420	0.262	0.420	-	-
	Swell Index		Cs		0.050	0.060	0.039	0.060	0.039	0.060	-	-
	Preconsolidation Pressure		Pc	(kg/cm^2)	0.74	1.59	1.70	1.59	1.73	1.59	-	-
	Coefficient of Consolidation		Cv	(cm^2/day)	50	70	100	70	100	70	-	-

Note (1): The value in ( ) is assumed.
Note (2): Su = 0.1 kg/cm^2 (Down to EL 0.0m), Su = 0.1 + 0.005 x Z kg/cm^2 (below EL 0.0m, Zo = EL 0.0m)
Note (3): Adopt the soil parameters (Ccu, φcu, C', φ') of Layer-6 for Layer-8
Note (4): Adopt the soil parameters (Ccu, φcu, C', φ', eo, Cc, Cs, Pc, Cv) of Layer-7B for Layer-4 and Layer-9

Source: Study Team

➢ As a result of laboratory tests of soil and rock, shear strength analysis and consolidation analysis, the soil and rock parameters for design of Bridge Area are proposed as shown in Table 3.6.1-3.

Table 3.6.1-3 Soil and Rock Parameters for Design (Bridge Area)

Item				2 (Sand)	3 (Clay)	4 (Clay)	5 (Sand)	6 (Clay)	7A (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)	11 (Clay)	12A (Highly Weathered Siltstone)	12B (Moderately Weathered Siltstone)
N-value				3	2	6	11	12	4	7	15	7	21	38	18	Over 50	Over 50
Unit Weight		γt	(g/cm^3)	(1.90)	1.72	1.75	(1.95)	1.95	1.75	1.83	1.92	1.82	(2.00)	(2.05)	(1.95)	2.08	2.60
Absorption			(%)	-	-	-	-	-	-	-	-	-	-	-	-	-	3.20
Shear Strength	For Short Term	Su or Cd	(kg/cm^2)	0.00	0.15	0.25	0.00	0.60	0.25	0.30	0.60	0.30	0.00	0.00	(1.20)	-	-
		φu or φd	(degree)	21.0	0.0	0.0	25.0	0.0	0.0	0.0	0.0	0.0	35.0	40.0	0.0	-	-
	Rate of Strength Increase	m		-	0.2	0.2	-	0.2	0.2	0.2	0.2	0.2	-	-	-	-	-
Unconfined Strength of Rock	On Dry Condition		(kg/cm^2)	-	-	-	-	-	-	-	-	-	-	-	-	14.0	53.9
	On Saturated Condition		(kg/cm^2)	-	-	-	-	-	-	-	-	-	-	-	-	9.0	(25.3)
	Softening Ratio			-	-	-	-	-	-	-	-	-	-	-	-	0.64	0.47
Consolidation	Initial Void Ratio	eo		-	1.314	1.297	-	0.777	1.297	1.056	0.838	1.082	-	-	-	-	-
	Compression Index	Cc		-	0.390	0.385	-	0.234	0.385	0.345	0.200	0.367	-	-	-	-	-
	Swell Index	Cs		-	0.042	0.076	-	0.052	0.076	0.057	0.058	0.061	-	-	-	-	-
	Preconsolidation Pressure	Pc	(kg/cm^2)	-	0.70	1.58	-	2.67	1.58	1.64	1.98	1.74	-	-	-	-	-
	Coefficient of Consolidation	Cv	(cm^2/day)	-	60	70	-	80	70	70	80	70	-	-	-	-	-

Note (1): The value in ( ) is assumed.
Note (2): Adopt the soil parameters (γt, Su, eo, Cc, Cs, Pc, Cv) of Layer-4 for Layer-7A
Note (3): Adopt the soil parameters (γt, Su, eo, Cc, Cs, Pc, Cv) of Layer-7B for Layer-9

Source: Study Team

➢As a result of laboratory tests, shear strength analysis and consolidation analysis, the soil parameters for design of Approach Road Area in Cat Hai side are proposed as shown in Table 3.6.1-4.

Table 3.6.1-4 Soil Parameters for Design (Cat Hai side)

Item					Layer													
					1 (Clay)	2 (Sand)	3 (Clay)	4 (Clay)	5 (Sand)	L5-1 (Sand)	6 (Clay)	L6-1 (Sand)	7A (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)
N-value					2	5	3	5	10	57	11	9	4	5	14	6	14	42
Unit Weight			γt	(g/cm^3)	1.65	1.90	1.73	1.73	1.93	(2.10)	1.86	(1.93)	1.75	1.81	1.93	1.76	(1.95)	(2.05)
Shear Strength	For Short Term		Su or Cd	(kg/cm^2)	Note (2)	0.00	0.15	Note (3)	0.00	0.00	0.60	0.00	0.20	0.25	0.80	0.25	0.00	0.00
			ϕu or ϕd	(degree)	0.0	23.0	0.0	0.0	25.0	45.0	0.0	25.0	0.0	0.0	0.0	0.0	30.0	40.0
	For Long Term	Total Stress	Ccu	(kg/cm^2)	0.11	-	0.01	0.19	-	-	0.22	-	0.31	0.31	0.08	0.20	-	-
			ϕ cu	(degree)	13.5	-	21.2	9.7	-	-	17.6	-	19.5	19.5	20.0	13.8	-	-
		Effective Stress	C'	(kg/cm^2)	0.10	-	0.02	0.14	-	-	0.16	-	0.12	0.12	0.03	0.11	-	-
			ϕ '	(degree)	25.2	-	30.5	21.5	-	-	24.9	-	26.3	26.3	25.2	24.5	-	-
	Rate of Strength Increase		m		0.2	-	0.2	0.2	-	-	-	-	0.2	0.2	-	-	-	-
Consolidation	Initial Void Ratio		eo		1.595	-	1.294	1.317	-	-	0.962	-	1.185	1.078	0.825	1.200	-	-
	Compression Index		Cc		0.527	-	0.397	0.386	-	-	0.220	-	0.430	0.351	0.193	0.457	-	-
	Swell Index		Cs		0.070	-	0.054	0.054	-	-	0.042	-	0.059	0.054	0.031	0.067	-	-
	Preconsolidation Pressure		Pc	(kg/cm^2)	0.34	-	0.66	0.83	-	-	1.46	-	1.15	1.13	1.44	1.51	-	-
	Coefficient of Consolidation		Cv	(cm^2/day)	40	-	60	70	-	-	150	-	80	80	150	80	-	-

Note (1): The value in ( ) is assumed.
Note (2): Su = 0.1 kg/cm^2 (Down to EL 0.0m), Su = 0.1 + 0.02 x Z kg/cm^2 (below EL 0.0m, Zo = EL 0.0m)
Note (3): Su = 0.1 + 0.02 x Z kg/cm^2 (below EL -5m, Zo = EL -5.0m)
Note (4): Adopt the soil parameters (Ccu, ϕ cu, C', ϕ ') of Layer-7B for Layer-7A

Source: Study Team

3.6.2 Recommendations

The borings in Bridge Area were conducted at every pier (one borehole/pier for approach bridge and two boreholes/pier for main bridge). On the other hand, in Approach Road Area of Hai An side and Cat Hai side, there are some places where sufficient borings were not conducted due to the permission problem from land owner of shrimp pond.

Therefore, the additional geotechnical investigation shown in Table 3.6.2-1 is recommended to be carried out in construction stage.

Table 3.6.2-1 Additional Geotechnical Investigation in Construction Stage

	Hai An side		Cat Hai side		Total
	Center Line	Cross Section	Center Line	Cross Section	
Number of Boring Location	16	2	18	2	38 locations
Drilling Length	40m x 16 = 640m	40m x 2 = 80m	30m x 18 = 540m	30m x 2 = 60m	1,320 m
Standard Penetration Test 1)	640 – 32 – 10 x 16 = 448	80 – 10 x 2 = 60	540 – 36 – 10 x 18 = 324	60 – 10 x 2 = 40	872 tests
Undisturbed Sampling	2depths x 16locations = 32samples	–	2depths x 18locations = 36samples	–	68 samples
Physical Tests of Soil 2) (Specific Gravity, Natural Water Content, Grain Size Analysis, Atterberg Limits)	3 x 16(disturbed) + 32(undisturbed) = 80	–	3 x 18(disturbed) + 36(undisturbed) = 90	–	170 samples
Unit Weight Test of Soil 3)	32	–	36	–	68 samples
Unconfined Compression Test of Soil 3)	32	–	36	–	68 samples
Triaxial Compression Test of Soil (UU) 3)	32	–	36	–	68 samples
Consolidation Test 3)	32	–	36	–	68 samples
Number of Field Vane Shear Test Location 4)	16	2	18	2	38 locations
Accumulated Depth of Field Vane Shear Test 4)	20m x 16 = 320m	20m x 2 = 40m	20m x 18 = 360m	20m x 2 = 40m	760 m

Note: 1) It shall be performed at every 1.0m intervals except for the depth of undisturbed sampling and FVST.
2) (disturbed) means the SPT samples, while (undisturbed) means the undisturbed samples by undisturbed sampling.
3) Unit Weight Test, Unconfined Compression Test, Triaxial Compression Test (UU) and Consolidation Test shall be carried out using the undisturbed samples.
4) Field Vane Shear Test will be carried out up to the depth of 20 m at every 2.0m intervals.

Source: Study Team

CHAPTER 4 MATERIAL SURVEY

4.1 Introduction

4.1.1 General

Material Survey was conducted in order to confirm current condition of material sources and material itself for this Project, based on the survey results of quarries which can provide materials for the project conducted in FS. In case that there are some material source which are no longer available, other material sources were surveyed for the replacement with necessary information (Location, Capacity, Capability Supply, Transportation Distance, etc). In addition, all the necessary laboratory tests for the design/construction were executed for confirmation of current material condition.

4.1.2 Objectives of Survey

Main objectives of the Survey are as following.

-To find material sources to provide for the Project.

-To determine the suitability for quality requirements

-To determine volume, possibility of supply, potential capacity, transport distance and means of transport, required material quantity.

-To provide necessary information for planning cost estimation

4.2 Material Quantities required for the Project

The material quantities required for the Project are shown in the tables below.

Table 4.1.2-1 Quantities of soil, sand and macadam required for the project

Unit : m^3

Item		Material type	Hai An side	Cat Hai side	Tan Vu Interchange	Temporary road	Total
Pavement	Base	Macadam	15,200	28,400	6,400		50,000
	Sub – base	Macadam	43,400	81,400	18,600		143,400
Embankment	Subgrade	Soil or sand	63,700	94,700	21,400		179,000
	Under subgrade	Soil or sand	357,200				357,000
		Soil or sand		509,000	195,000		704,000
	Slope protection	Soil or sand	46,700	82,100	42,800		171,000
	Temporary Road	Macadam				500,000	500,000
Soft soil treatment	Sand drain	Sand	225,900	157,600	95,200		478,000
	Replacement	Soil or sand		107,100			107,100
	Sand blanket	Sand	266,300	188,700	146,000		601,000
	Settlement	Soil or sand	251,700	173,000	132,800		557,500
	Surcharge	Macadam	179,500	171,000	76,800		427,300
					Soil or sand		2,077,200
					Sand		1,079,700
					Macadam		1,120,700

Source: Study Team

Table 4.1.2-2 Quantities of aggregate for concrete

Unit : m^3

Item		Road in Hai An	Bridge	Road in Cat Hai	Tan Vu IC	Total	Coarse aggregate	Fine aggregate
Pavement	Surface course	72,421	53,862	135,847	42,935	305,065	149,481.9	122,026.0
	Binder course	72,421	53,862	135,847	42,935	305,065	167,785.8	106,772.8
Bridge	Substructure (Hai An)		20,917			20,917	10,458.5	7,948.5
	Substructure (Main bridge)		12,771			12,771	6,385.5	4,853
	Substructure (Cat Hai)		2,822			2,822	1,411	1,072.4
	Superstructure (Hai An site)		43,730			43,730	21,865	16,617.4
	Superstructure (Main bridge)		5,856			5,856	2,928	2,225.3
	Superstructure (Cat Hai)		5,104			5,104	2,552	1,939.5
Retaining wall		23,281				32,277	16,138.5	12,265.3
Box culvert		3,390		2,441.6		5,831.6	2,915.8	2,216
					Total		381,921.9	277,936
					Round of number		382,000	278,000

Source: Study Team

4.3 Conditions of Survey

4.3.1 Standards to be applied

The standards to be applied for the survey are as follows.

-Procedure of Highway survey 22 TCN 262 – 2000;

-Construction soil – Method of taking, packing, transporting and storing sample : TCVN 2683 – 1991;

-Laboratory testing were carried out according to ASTM and TCVN:

□Grain analysis	ASTM D-442
□Unit weight	ASTM D-424
□Water content	ASTM D-2216
□Specific gravity	ASTM D-854
□Liquid limit	ASTM D-423
□Plastic limit	ASTM D-424
□Compaction test	ASTM D-1557
□CBR bearing test	ASTM D-1883
□Direct shear test (in nature and saturated)	TCVN 4199-1995
□Size modulus	TCVN 342-86
□Compression strength of rock	TCVN 7275-2006
□Abrasion of Aggregate LA	ASTM C-535

- Specifications for soil and aggregate are AASHTO, TCN and TCVN:

Table 4.3.1-1 Specification for Soil and Aggregate

Required Materials	The standard applied	Requirement in Specification
Soil for embankment	AASHTO M145 Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	The materials used for embankment is to be conformed to requirements in AASHTO M145. CBR after compaction: greater than 8% Density: 95% (AASHTO T180) Soil material with less than 8% CBR may be used upon the Engineer's consent and testing on trial sections.
Soil for Subgrade	AASHTO M145 Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	The materials used for subgrade layer ($K \geq 0.98$) is to be conformed to requirements in AASHTO M145. Material used for sub-grade (thickness of the top layer to be 50cm) belong to A-1 or A-3 soil group. Group A-2 may be used upon the Engineer's. Liquid limit $W_L \leq 40\%$ (refer to other project) Plasticity index Ip ≤ 17 (refer to other project) CBR: $\geq 10\%$ (compaction at 98%, soaked 4 days) Density: 98%

Required Materials	The standard applied	Requirement in Specification
Coarse aggregate for Sub-base	AASHTO M147-65 Materials for Aggregate and Sub-base, Base and Surfaces Courses	The materials used for base course is to be conformed to requirements in AASHTO M147-65. Abrasion of Aggregate: 50% max. Plasticity Index passing the 0.425 mm: 6 max. Liquid Limit passing the 0.425: 35 max. Soaked CBR: 30% min
Coarse aggregate for Base	AASHTO M147-65 Materials for Aggregate and Soil-Aggregate Sub-base, Base and Surfaces Courses	The materials used for base course is to be conformed to requirements in AASHTO M147-65. Abrasion of Aggregate: 45% max. Plasticity Index passing the 0.425 mm: 6 max. Liquid Limit passing the 0.425 : 25 max. Soaked CBR: 80% min
Coarse aggregate for AC	AASHTO M147-65 Materials for Aggregate and Sub-base, Base and Surfaces Courses 22TCN 249 -98	The materials used for coarse aggregate for AC is to be conformed to requirements in AASHTO M147-65. Dirty or dusty coarse aggregate having more than 2% fines passing the No.200 sieve not to be used. Abrasion of Aggregate: 40% max Bituminous agglutinate degree not less than 95 %. Compression strength of natural rock $\geq$ 800kG/cm^2 LA attrition value : 25% max Bituminous agglutinate degree $\geq$ 95 %.
Fine aggregate for AC	AASHTO M147-65 Materials for Aggregate and Soil-Aggregate Sub-base, Base and Surfaces Courses 22TCN 249-98	The materials used for fine aggregate for AC is to be conformed to requirements in AASHTO M147-65. Dirty or dusty natural sands having less than 8% fines passing the No.200 sieve. ize modulus Mk $\geq$ 2 Grain content from 5-0.14mm >14% Dirty or dusty content less than 3%. Clayey grain content < 0.5%
Coarse aggregate for concrete	AASHTO M80 Coarse aggregate for Porland Cement Concrete TCVN 7570-2006	AASHTO M80 Abrasion of aggregate: 50% max (AASHTO T96). Compression strength of natural rock $\geq$ 1.5 time the compression strength of concrete. Dirty or dusty content <=1%. LA attrition value <= 50%
Fine aggregate for concrete	AASHTO M6 Fine concrete for cement concrete TCVN 7570 - 2006	AASHTO M6. According to AASHTO M6 Size modulus Mk $\geq$ 2 Grain content 0.14mm $\leq$ 10% Dirty or dusty content less than 1.5%. Clayey grain content < 0.5%

Source: Study Team

4.3.2 Procedures of Survey

The procedures of the survey are as follows.

-Collected the report on material investigation in FS stage.

-Gather documents, which relating to conditions of exploitation and transportation, quality and reserves of the quarries.

-Surveying, classification of soil, sand, stone (by visual check), elevation of quality, potential reserves of the quarries.

-Laboratory test determine mechano-physical properties of soil, sand, rock samples by requirement standard.

4.3.3 Contents of Survey

The contents of the survey are as follows.

-Survey on the location, exploited capacity, quality, initial reserve of quarries by quarries management unit has to offer.

-The products of the mine.

-Type of road, transport distance, transport means from quarries to the works.

-Investigate the existence of regional environmental conservation or irrigation canals around the material mines.

-Taking samples for testing laboratory.

-Investigation of mixing concrete plant

4.3.4 Quantities of Survey

The quantities of the survey are shown in the table below.

Table 4.3.4-1 Quantities of Construction Material Source Survey

No	**Work items**	**Unit**	**Quantity**	**Remark**
1	**Mobilization and Demobilization**	LS	1	
2	**Borrow pit survey**	day	4	
2.1	Investigation of Borrow Pit	No	12	
2.2	Laboratory test for physical properties	No	52	
2.3	Standard compaction test	No	39	
2.4	CBR test	No	13	
3	**Sand pit survey**	No	6	
3.1	Investigation of Sand Pit for Filling	day	6	
3.2	Laboratory test for physical properties	No	9	
3.3	Standard compaction test	No	9	
3.4	CBR test	No	9	
4	**Sand Pit Survey for Soft Soil Treatment**	No	6	
4.1	Investigation of Sand Pit for Soft Soil Treatment	day	6	
4.2	Laboratory test for physical properties	No	6	
5	**Rock Quarries**	No	3	
5.1	Investigation of Rock Quarries	day	9	
5.2	Laboratory test for mechanical-physical properties	No	9	
5.3	Abrasion of Aggregate LA test	No	9	
6	**Investigation of Concrete Mixing Plant**	day	3	
7	**Established material report**	LS	1	

Source: Study Team

4.4 Results of Survey

4.4.1 List of Material Sources

The list of material sources is as follows.

Table 4.4.1-1 List of Material Sources

Pit/ Plant	Name		Location	Remarks
Borrow Pit (Soil)	1-1	Thien Hoi	An Tien Commune – An Lao – Hai Phong	
	1-2	Thien Dong Soil Pit	Dong Son Commune – Thuy Nguyen – Hai Phong	Not available
	1-3	Lien Khe Soil Pit	Lien Khe Commune – Thuy Nguyen – Hai Phong	Not available
	1-4	Minh Duc Soil Pit	Minh Duc Town – Thuy Nguyen – Hai Phong	
	1-5	**Doc Do Soil Pit**	**Uong Bi District – Quang Ninh**	**Replaced**
	1-6	**Dia Moi Soil Pit**	**Dia Moi – An Sinh – Dong Trieu – Quang Ninh**	**Replaced**
Sand Pit	2-1	TL353 Sand Pit	Yards along provincial road No.353	
	2-2	Rao Sand Pit	Yards near Rao Bridge	
	2-3	Niem Sand Pit	Yards near Niem Bridge	
	2-4	Dong Hai Sand Pit	Yards along the Cua Cam River, Dong Hai Ward	
	2-5	**Tram Bac Sand Pit**	**Yards in the vicinity of Van Uc River tributaries**	**Replaced**
	2-6	Tien Cuu Sand Pit	Yards near Tien Cuu Bridge	Not available
	2-7	Quy Cao Sand Pit	Yards near Quy Cao Bridge	
Quarry Pit	3-1	Lien Khe Quarry	Lien Khe – Thuy Nguyen – Hai Phong	
	3-2	Phuong Mai Quarry	Phuong Nam – Uong Bi – Quang Ninh	
	3-3	Minh Duc Quarry	Minh Duc Town – Thuy Nguyen – Hai Phong	Not available
	3-4	Thong Nhat Quarry	Phu Thu Town – Kinh Mon - Hai Duong	
Concrete Mixing Plant	**4-1**	**Niem Bridge**	**Vinh Niem – Le Chan – Hai Phong**	**Newly Surveyed**
	4-2	**Hoang Truong -TL353**	**Anh Dung – Duong Kinh – Hai Phong**	**Newly Surveyed**
Asphalt Mixing Plant	5-1	Hoang Truong - TL353	Anh Dung – Duong Kinh – Hai Phong	
	5-2	Rao Bridge	Anh Dung Ward – Duong Kinh – Hai Phong	

Source: Study Team

4.4.2 Borrow Pits

4.4.2.1 Location of Borrow Pits

The borrow pits, Minh Duc, Thien Hoi (Hai Phong city), Doc Do and Diem Moi (Quang Ninh province), were investigated for the purpose of using for embankment, subgrade, protected slope. The borrow pits have been exploited to provide material for several projects in the area of Hai Phong city and Quang Ninh province.

Location of the borrow pits presented in the figure on the next page.

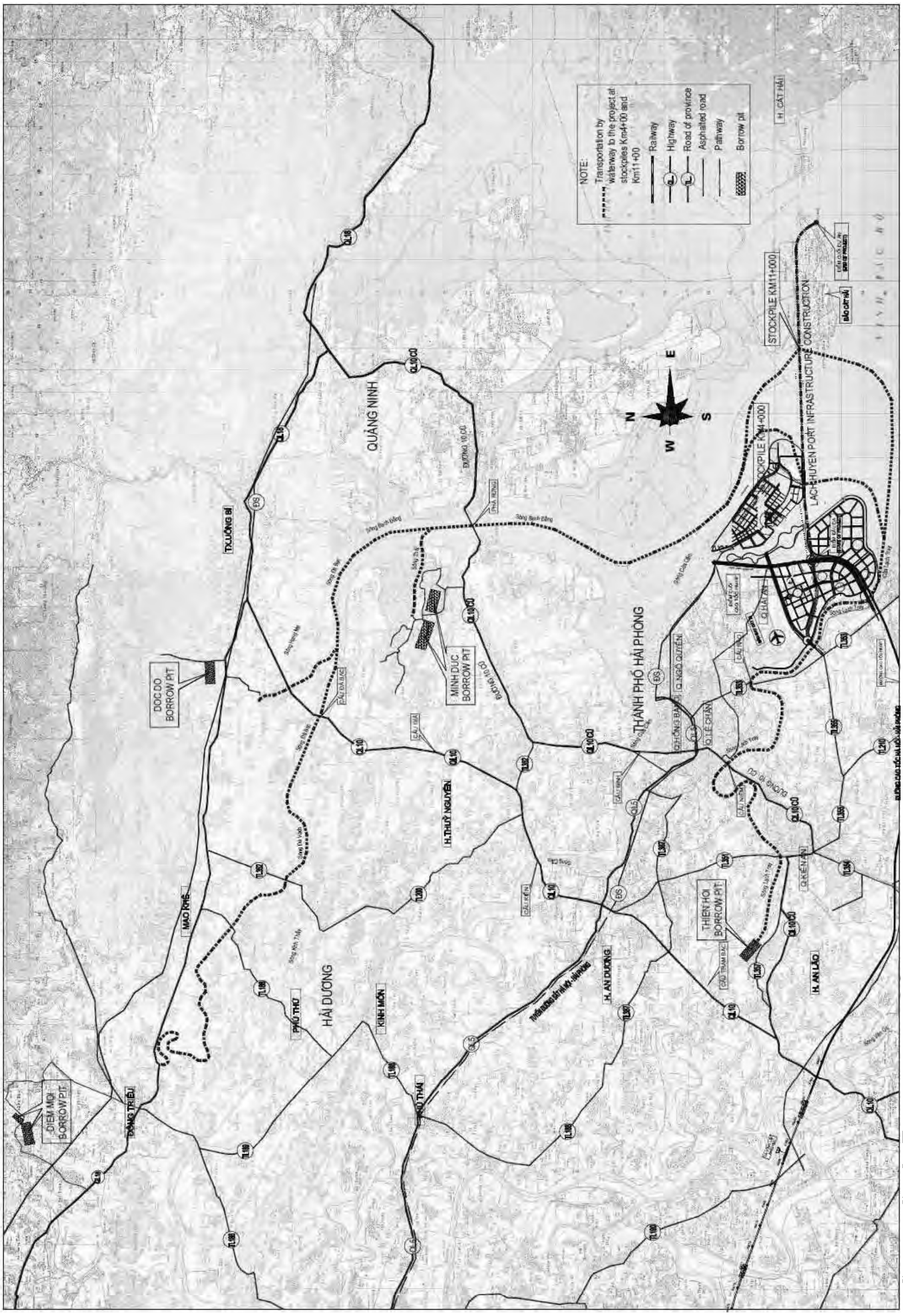
Potential reserves and capacities at the time of the survey are listed in the table below.

Table 4.4.2-1 List of Material Sources

No	**Lacation and name of borrow pits**	**Reserve** (m^3)	**Providing capacity** (m^3/day)
1	Thiên Hoi borrow pit An Tien commune – An Lao district – Hai Phong city.	1,000,000	2,000
2	Minh Duc borrow pit Minh Duc town – Thuy Nguyen district – Hai Phong city	1,000,000	2,000
3	Doc Đo borrow pit Doc Đo 2 zone - Yen Tu road – Phuong Dong commune – Uong Bi district-Quang Ninh province	100,000	500
4	Diem Moi borrow pit Diem Moi commune – Dong Trieu district – Quang Ninh province	3,560,000	3,000
Total		**5,660,000**	**7,500**

Source: Study Team

Figure 4.4.2-1 Location of Borrow Pits



Source: Study Team

4.4.2.2 Quality (Results of Laboratory Test)

The test results of samples taken at the borrow pits are presented in the table below.

Table 4.4.2-2 Results of Laboratory Tests of Samples at Borrow Pits

No	Properties	Unit	Thien Hoi	Minh Duc	Doc Do	Diem Moi	Tech. Requirement
1	Passing 50mm sieve	%	100	100	100	100	
2	Passing 0.075mm	%	13.3	57.83	75.5	43.27	
3	Liquid Limit	%	39.00	39.35	32.96	36.17	≤ 40
4	Plasticity index	%	16.73	14.53	12.37	14.60	≤ 17
5	Max dry density	g/cm^3	1.91	1.89	1.91	1.95	
6	Optimum moisture content	%	14.02	14.95	13.95	12.65	
7	CBR (at K98% soaked)	%	28.13	19.00	27.03	30.05	≥ 10
8	Classification		CL (A2-6)	CL (A2-6)	CL (A2-6)	CL (A2-6)	
9	Conclusion		Embankment Subgrade	Embankment Slope protection	Embankment Slope protection	Embankment	

Source: Study Team

4.4.2.3 Exploitation and Transportation Condition

The borrow pits are exploiting favorable conditions, can be exploited by the machines. Distance transport from the borrow pits to the project are listed in the table below.

Table 4.4.2-3 Transportation distance

Location of mines/ stockpiles	Reserve (m^3)	Supplied capacity (m^3/day)	Route from quarry / sand stockpile to the Project	Transportation distance	
				Highway (Km)	Water way (Km)
Thien Hoi borrow pit	1.000.000	2000	From the borow pits to material stockpile at Km11 +00 (weter way) & *Km4 (highway)*	30.1	42.3
Minh Duc borrow pit	1.000.000	2000		34.2	29.1
Doc Do borrow pit	100.000	500		40.8	44.5
Diem Moi borrow pit	3.000.000	3000		71.6	60.7

Source: Study Team

4.4.3 Sand Resources for Embankment

4.4.3.1 Location of Sand Resources

Sand resources of exploitation and supply for embankment consist of:

- Sand stockpile of along PR 353 : Anh Dung ward – Duong Kinh district, sand was exploited from Thai Binh and Kinh Thay rivers to gather in the stockpile;
- Rao bridge stockpile : Anh Dung Ward – Duong Kinh district and Dang Giang Ward – Ngo Quyen district, sand was exploited from Thai Binh and Kinh Thay rivers to gather in the stockpile.
- Niem bridge stockpile : Vinh Niem ward – Le Chan district and Quan Tru ward – Kien An district, sand was exploited from Thai Binh and Kinh Thay rivers to gather in the stockpile.
- Dong Hai stockpile : Dong Hai Ward – Hai An district, sand was exploited from Kinh Thay rivers to gather in the stockpile;
- Tram Bac and Quy Cao stockpiles along highway No10 belong to Quoc Tuan commune – An Lao district and Giang Bien commune – Vinh Bao district, sand was exploited from Thai Binh rivers to gather in the stockpile.

Location of stockpiles is presented in the figure on the next page.

Currently, the sand stockpiles existing and providing sand for projects in the area.

4.4.3.2 Quality (Results of Laboratory Test)

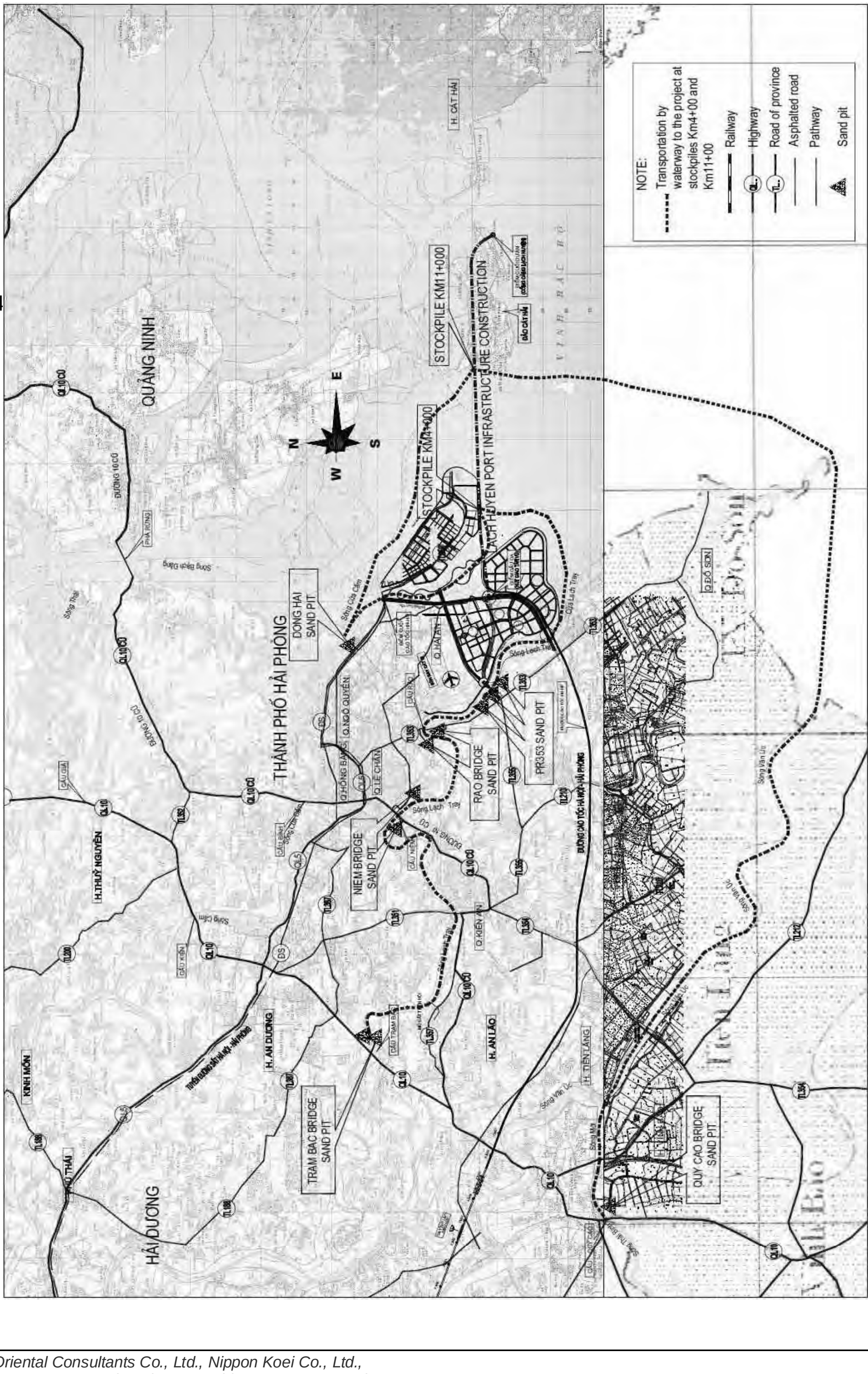
The test results of samples taken at the sand resources are presented in the table below.

Table 4.4.3-1 Results of Laboratory Tests of Sand Resources for Embankment

Properties	Kinh Thay river & Thái Bình river (PR 353 Stockpile)	Kinh Thay river & Thái Bình river (Niem Bridge stockpile)	Thai Binh riever (Quy Cao stockpile)	Specification
Sieve analysis % (percent passing 0.075mm)	3.7	5.2	4.3	$\leq$ 10
Liquid Limit (%)	NP	NP	NP	NP
Plasticity Index	NP	NP	NP	NP
Classification	SM(A-3)	SM(A-3)	SM(A-3)	A-3
Max dry density (γ_{cmax},g/cm^3)	1.63	1.628	1.68	
Optium moisture content (Wop)	13.50	14.50	12.50	
CBR at density K=95 (%)	9.2	9.8	12.4	$\geq$8
Conclusion	Meet the requirements of specification applied for embankment			

Source: Study Team

Figure 4.4.3-1 Location of Sand Resources for Embankment



Source: Study Team

Reserve, Capacities and Transportation Condition

The amount of reserve, capacities and distance transport from the sand resources to the project are shown in the table below.

Table 4.4.3-2 Reserve, Capacities and Transportation Distance

Location of mines/ stockpiles	Material resource	Reserve (m^3)	Supplied capacity (m^3/day)	Route from quarry / sand stockpile to the Project	Transportation distance	
					Highway (Km)	Water way (Km)
PR 353 stockpile	Thai Binh and Kinh Thay sand pit river	100.000x4	1000	From the quarry/ stockpile to material stockpile at Km11+00 (weter way) & *Km4 (highway)*	20.6	20.2
Rao bridge stockpile		100.000x3	1000		18.1	22.4
Niem bridge stockpile		50.000x2	200		15.6	28.7
Dong Hai stockpile	Kinh Thay sand pit river	50.000x2	200		7.4	13.7
Tram Bac stockpile	Kinh Thay and Thai Binh sand pit river	100.000	200		30.5	44.0
Quy Cao stockpile	Thai Binh sand pit river	50.000	200		41.6	54.8

Source: Study Team

4.4.4 Sand Resources for Soft Soil Treatment

4.4.4.1 Location of Sand Resources

Sand resources for soft soil treatment were exploited along Lo river – Viiet Tri city – Phu Tho province and gathered sand stockpiles in follows:

-Sand stockpile of along PR 353 : Anh Dung ward – Duong Kinh district – Hai Phong city;

-Rao bridge stockpile : Anh Dung Ward – Duong Kinh district and Dang Giang Ward – Ngo Quyen district – Hai Phong city.

-Niem bridge stockpile : Vinh Niem ward – Le Chan district and Quan Tru ward – Kien An district – Hai Phong city.

-Dong Hai stockpile : Dong Hai Ward – Hai An district – Hai Phong city

Location of stockpiles is presented in the figure on the next page.

4.4.4.2 Reserve, Capacity and Transportation Condition

The amount of reserve, capacities and distances from the sand resources to the project are listed in the table below.

Table 4.4.4-1 Reserve, Capacities and Transportation Distance

Location of mines/ stockpiles	Material resource	Reserve (m^3)	Supplied capacity (m^3/day)	Route from quarry / sand stockpile to the Project	Transportation distance	
					Highway (Km)	Water way (Km)
PR 353 stockpile	Lo sand pit, Viet Tri, Phu Tho	100.000x4	200	From the quarry/ stockpile to material stockpile at Km11+00 (weter way) & *Km4 (highway)*	20.6	20.2
Rao bridge stockpile		100.000x3	200		18.1	22.4
Niem bridge stockpile		50.000x2	200		15.6	28.7
Tram Bac stockpile		100.000	200		30.5	44.0
Dong Hai stockpile		50.000x2	200		7.4	13.7
Quy Cao stockpile		50.000	200		41.6	54.8
Lo river, Viet Tri-Phu Tho		Very large				272

Source: Study Team

4.4.4.3 Quality (Results of Laboratory Test)

The test results of samples taken at the sand resources are presented in the table below.

Table 4.4.4-2 Results of Laboratory Tests of Sand Resources for Soft Soil Treatment

Properties	Lo river Viet Tri (PR353 stockpile)	Lo river Viet Tri (Niem bridge stockpile)	Lo river Viet Tri (Quy Cao bridge stockpile)	Technical Requirement For Sand Drain
Sieve analysis, percent retaining				
19m	100	100	100	
9.5 mm	96.4	100	100	
4.75 mm	88.4	98.6	95.9	
3.26 mm	78.5	96.4	79.2	
1.18 mm	62.6	79.0	49.1	
0.6 mm	31.8	52.2	24.5	
0.25 mm	8.2	15.3	7.6	<50%
0.15 mm	4.2	6.7	5.4	
0.075 mm	2.4	4.6	4.2	<5%
Specific gravity (g/cm^3)	2.67	2.66	2.67	
Modulus size	3.15	2.5	3.34	
Dust, mud, clay content (%)	0.75	0.83	0.67	< 5%
Liquid limit (%)	-	-		
Plasticity Index (%)	NP	NP		
Coefficient of uniformity Cu	4.1	3.9	5.4	
Coefficient of curvature Cc	1.1	0.9	1.2	
Permeability coefficient (cm/s)	-	-		
Conclusion	Meet the technical requirement for sand drain			

Source: Study Team

4.4.5 Rock Quarries for Asphalt Concrete and Cement Concrete

4.4.5.1 Location of Rock Quarries

Rock quarries for the project were explored in the following locations,

Lien Khe quarry : Lien Khe commune – Thuy Nguyen district – Hai Phong city;

Phuong Mai quarry : Phuong Nam commune – Uong Bi district – Quang Ninh province;

Thong Nhat quarry : Phu Thu town – Kinh Mon district – Hai Duong province.

Location of the rock quarries is presented in the figure on the next page.

4.4.5.2 Reserve, Capacity and Transportation Condition

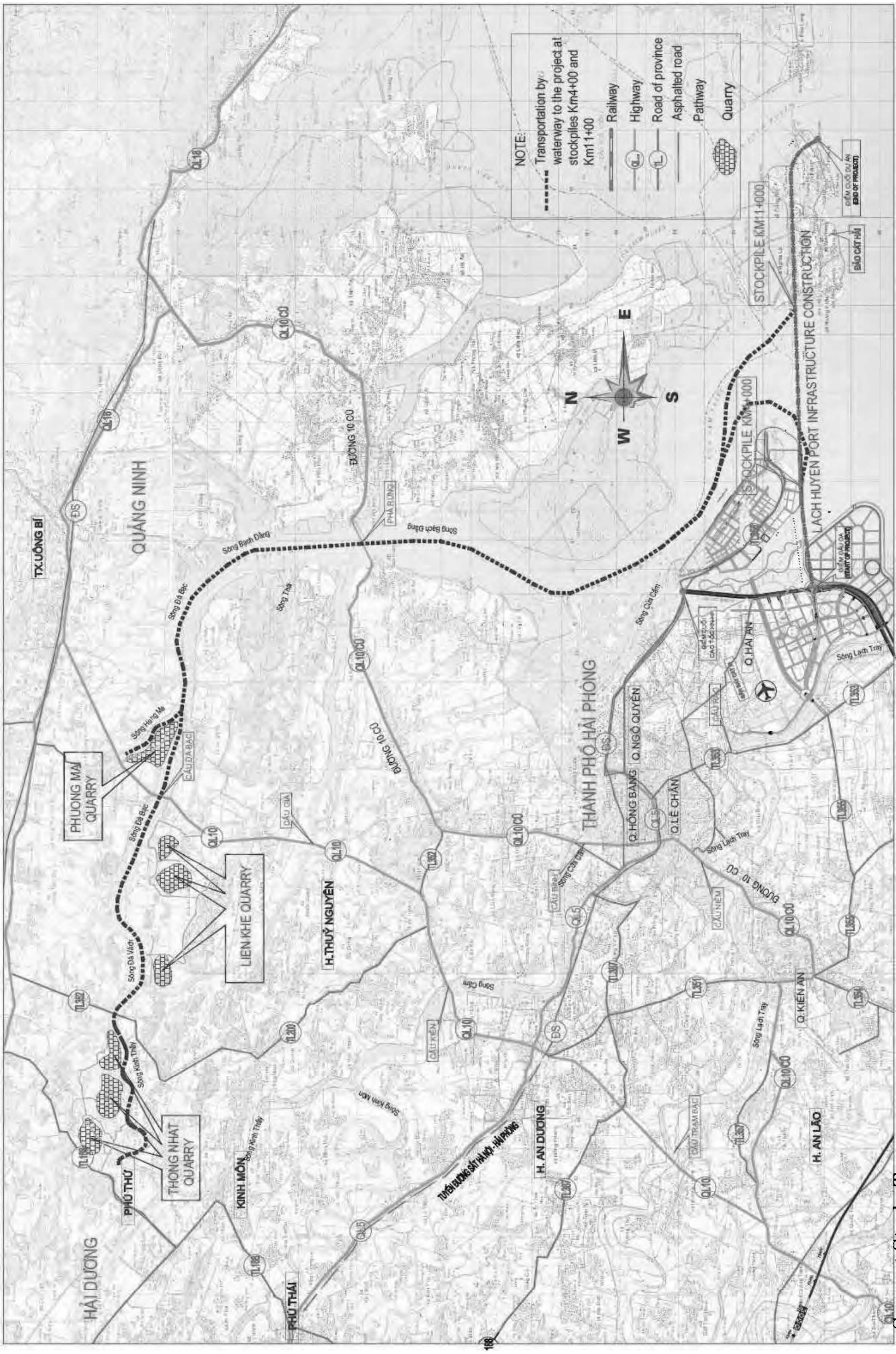
The quarries are exploited by machine chains. The amount of reserve, capacities and distances from the rock quarries to the project are listed in the table below. Existing quarries are producing coarse aggregate 1x2, 2x4, 4x6 ..., quarry stone and providing products for the construction in the area. Base and subbase shall be produced according to the request of customers.

Table 4.4.5-1 Reserve, Capacities and Transportation Distance

Location	Material resource	Reserve (m^3)	Supplied capacity (m^3/day)	Route from quarry / sand stockpile to the Project	Transportation distance	
					Highway (Km)	Water way (Km)
Lien Khe quarry	Limestone	600.000	800	From the quarry to material stockpile at Km11+00 (weter way) & *Km4 (highway)*	33.0	36.9
Phuong Mai quarry		>1.000.000	200		33.7	34.5
Thong Nhat quarry		>1.500.000	6000		47.5	48.5

Source: Study Team

Figure 4.4.5-1 Location of Rock Quarries



Source: Study Team

4.4.5.3 Quality (Results of Laboratory Test)

The test results of rock samples taken at the quarries are presented in the table below.

The rock samples from these quarries are satisfactory to produce aggregate for the subbase/base and coarse aggregate for AC pavement/ cement concrete. According to available testing laboratory data from quarries, these values are within the requirements of the standard material.

Table 4.4.5-2 Results of Laboratory Tests of Rock Samples

Properties	Quarries and testing laboratory results			Requirement specification of aggregate for asphalt concrete	Requirement specification of aggregate for cement concrete
	Lien Khe	Phuong Mai	Thong Nhat		
Compression strength of natural rock (Dry/ saturated, KG/cm^2)	894.8/853.8	864.4/836.7	898.8/859.1	≥800 22TCN 249-98	≥850 (for concrete C45) TCVN 7570-2006
Abrasion of Aggregate LA (%)	21.44	21.9	22.42	≤40	≤ 50
Bituminous agglutinate degree (%)	96	95	96	≥ 95	

Source: Study Team

4.4.6 Fine Aggregate for Asphalt Concrete and Cement Concrete

4.4.6.1 Location of Sand Resources for Fine Aggregate

The supply resources of fine aggregates for asphalt concrete and cement concrete are exploited from the Lo River (Phu Tho), transported to gather in the sand stockpiles as described in Section 4.4.4, the sand resources for soft soil treatment.

4.4.6.2 Reserve, Capacity and Transportation Condition

The amount of reserve, capacities and transportation conditions of the sand resources to the project are listed in the table below.

Table 4.4.6-1 Reserve, Capacities and Transportation Distance

Location	Material resource	Reserve (m^3)	Supplied capacity (m^3/day)	Route from quarry / sand stockpile to the Project	Transportation distance	
					Highway (Km)	Water way (Km)
PR 353 stockpile	Lo sand pit, Viet Tri, Phu Tho	100.000x4	200	From the quarry/ stockpile to material stockpile at Km4+00/ Km11+00 (weter way) & *Km4 (highway)*	20.6	20.2
Rao bridge stockpile		100.000x3	200		18.1	22.4
Niem bridge stockpile		50.000x2	200		15.6	28.7
Tram Bac stockpile		100.000	200		30.5	44.0
Dong Hai stockpile		50.000x2	200		8.5	13.7
Quy Cao stockpile		50.000	200		41.6	54.8
Lo river, Viet Tri-Phu Tho		Very large	As required			272

Source: Study Team

4.4.6.3 Quality (Results of Laboratory Test)

The test results of sand samples taken at the resources are presented in the table below.

Recently, sand material at these resources used as fine aggregate for asphalt concrete and cement concrete for construction projects in the region.

Table 4.4.6-2 Results of Laboratory Tests of Sand Samples

Properties	PR 353	Rao bridge	Niem bridge	Dong Hai	Tram Bac bridge	Quy Cao bridge
Analysis grain (% Passing sive)						
10 mm	2.1	1.6	0.4		0.7	2.0
5 mm	11.7	5.6	1.1	5.8	6.9	3.4
2.5 mm	14.0	8.4	2.5	7.6	12.9	14.2
1.25 mm	18.3	17.8	19.2	15.8	14.8	28.2
0.63 mm	24.4	21.1	28.1	18.5	16.7	20.7
0.316 mm	20.1	27.1	33.1	35.3	29.4	18.9
0.14 mm	5.0	10.6	10.0	12.1	11.5	8.2
<0.14 mm	4.4	7.8	5.7	5.0	7.1	4.4
Gravity density	2.67	2.66	2.66	2.66	2.66	2.67
Prorous volume mass	1.42	1.39	1.38	1.36	1.40	1.41
Size modulus	3.03	2.60	2.54	2.54	2.64	3.09
Mica content (%)	0.09	0.10	0.11	0.12	0.10	0.09
Dust, clay, content (%)	0.75	0.75	0.83	0.88	0.81	0.72
Conclusion	Meet the requirement specification of fine aggregate for AC (22TCN249-98) / cement concrete (TCVN7570-2006)					

Source: Study Team

4.4.7 Asphalt and Cement Concrete Mixing Plants

4.4.7.1 Conditions of Mixing Plants

The conditions, such as location, capacities and technology of mixing plnats are summarized in the table below.

Table 4.4.7-1 Location, Actuality, and Technology of Mixing Plants

No	Mixing plant	Location	Management Unit	Actuality	capacity	Technology
1	Niem bridge cement concrete mixing plant	Along the bank of Lach Tray river, right of PR360 (Km1), far way from Niem bridge is 300m. It belong to Vinh Niem ward – Le Chan district – Hai Phong.	Bach Dang consultant and Investment JSC. Head office : No 1 – Le Thanh Tong street – Ngo Quyen district – Hai Phong city. Detail please contact Mr Bui Xuan Phong Mobi: 0912.499.443 – Director. Fax: 0313.552.815	The plant is operating well. There are 2 machine groups of mixing.	60m³/h	Chinese Technology
2	PR 353 cement concrete mixing plant	Along the bank of Lach Tray river. Left of PR 353 (Km3-Pham Van Dong road), far way 1,5km from Rao bridge Belong to Anh Dung ward - Duong Kinh district - Hai Phong city.	Hoang Truong construction and Transport Company Ltd. Head office : No14C – Cat Bi road – Cat Bi ward - Hai An district – Hai Phong city. Detail please contact Mr Sinh, Tel : 0313.814.040, Mobi : 0983.246.724	The plant is operating well. There are 4 machine groups of mixing. Area of stockpile is 10.000m²	120m³/h	Chinese Technology
3	PR 353 asphalt concrete mixing plant	Along the bank of Lach Tray river. The left PR353 (Km2- pham Van Dong road), far way 1,0km from Rao bridge. Belong to Anh Dung ward - Duong Kinh district - Hai Phong city.	Hoang Truong construction and Transport Company Ltd. Head office : No14C – Cat Bi road – Cat Bi ward - Hai An district – Hai Phong city. Detail please contact Mr Sinh, Tel : 0313.814.040, Mobi : 0983.246.724	The plant is operating well. There is 01 machine groups of mixing.	120T/h	Korean Technology
4	Dinh Vu asphalt concrete mixing plant	Dinh Vu Industrial zone - Hai An district - Hai Phòng city.	Hoang Truong construction and Transport Company Ltd. Head office : No14C – Cat Bi road – Cat Bi ward - Hai An district – Hai Phong city. Detail please contact Mr Sinh, Tel : 0313.814.040, Mobi : 0983.246.724	The plant is operating well. There is 01 machine groups of mixing..	120T/h	Korean Technology

Source: Study Team

4.5 Conclusions and Recommendations

4.5.1 Conclusions

(1) Locations, Reserves and Capacities

At the survey time, reserves of borrow pits, sand stockpiles and rock quarries to meet the materials required volume of the project, favorable exploiting and supplied conditions.

Transport distance from borrow pits, rock quarries to the project to be rather large, It will be increased construction costs. Transport distance from the sand stockpiles to work relatively close.

The asphalt concrete and cement concrete mixing plants are located in areas relatively close distance to the project.

(2) Quality of the borrow pits:

The quality of the borrow pits basically meets the technical requirements of the project. However, Thien Hoi borrow pit, components are heavily weathered sandstone, silty stone clay with clayey sand, contains many grain with large size (large gravel, cobble-stone), must be removed before use. If other borrow pits have sufficient reserves to provide for the project, should not use Thien Hoi borrow pit.

(3) Quality of sand resources :

- Fine sand for embankment meets technical requirements of the project.

- Coarse sand used to make fine aggregate for asphalt concrete and cement concrete meets technical requirements of the project.

- Cine sand for soft soil treatment meets technical requirements of the project.

(4) Quality of rock quarries:

The qualities of rock quarries are satisfactory as coarse aggregate for asphalt concrete and cement concrete and base/ subbase.

4.5.2 Recommendations

- It is recommended to study carefully the location of mines for having alternatives reasonable using and transporting to the project to lower construction cost.
- For fine sand (A-3) for embankment, especially for subgrade, it is recommended to choose Minh Duc and Diem Moi borrow pits.
- During construction stage, adequate testing samples to control the quality of the materials exploited from quarries and stockpiles as directed technical specification projects.
- The products are asphalt and cement concrete from the mixing plants before deciding to use for the works needed to check equipment, production technology and product quality.

CHAPTER 5 HIGHWAY DESIGN

5.1 DESIGN CONDITION OF HIGHWAY

5.1.1 Future development plan of Dinh Vu - Cat Hai Economic Zone

5.1.1.1 Dinh Vu - Cat Hai Economic Zone Master Plan

Tan Vu-Lach Huyen Highway runs through Dinh Vu - Cat Hai Economic Zone which is under management of the Hai Phong Economic Zone Authority (HEZA). There is a master plan for this area which is being prepared by Nikken Sekkei Civil Engineering Ltd. According to information from Hai Phong City as of April 2011, the master plan was being reviewed by Hai Phong City and prepared to be submitted for approval of Prime Minister. And the procedure for getting the approval would take several months after the submission.

Therefore, the Study Team made a judgment that it would not be in time for implementation of Detailed Design of the highway. For the purpose of proper implementation of the highway design related to the industrial zone, the Study Team got design conditions, such as finish grade of industrial zone, classification of internal road in industrial zone, current states and time schedule of land development work, etc., by carrying out questionnaire survey.

The Study Team held a meeting with HEZA on 27th May 2011 and discussed design condition related to industrial zone. Result of the discussion is summarized in Minutes of Meetings.

Regarding future railway plan, the Study Team has not got any updated information from SAPROF study.

5.1.1.2 Current States and Future Development Plan of Dinh Vu Industrial Zone

Dinh Vu Industrial Zone (DVIZ) is invested by Dinh Vu Industrial Zone Joint Stock Company (DVIZ JSC). According to answer for our questionnaire survey, land development work is being carried out based on Notice No.304-TB-UB dated 29th December 2004 issued by Hai Phong City People's Committee on detailed plan of Dinh Vu General Economic Zone with DVIZ. Summary of past background and current states of land development work is as follows;

➢For DVIZ with the scope of 944.49 ha, development of the project is implemented in accordance with the detailed plan of scale of 1/2000 which was approved by Ministry of Construction in Decision 774/QD-BXD dated 11 May 2006.

➢During execution, Hai Phong City People Committee issued some official documents such as the Decision 327/QD-UBND on 2 March 2007 regarding to its approval for the detailed plan and issuance of the rules on construction management for the Industrial Group of Vinashin Corporation in DVIZ, the Decision 1392/QD-UBND dated 31 July 2007 in respect of acquiring the land to be handed over to Pha Rung Shipbuilding One Member Ltd., Co., for development of Dinh Vu Vinashin Industrial Group and Synthetic Port Construction in Dong Hai 2 ward, Hai An district; the Decision 1135/QD-UBND dated 18 June 2009 in respect of its approval for amendment of the detail plan of scale of 1/500 and Regulations on Construction Management for Dinh Vu Vinashin Industrial Group and Synthetic Port in DVIZ in Dong Hai 2 ward, Hai An district.

➢DVIZ JSC undertakes the development of phase 1 project with the scope of 164 ha and phase 2 project with the scope of 377.46ha. The detailed plan with scale of 1/500 of phase 2 project was approved by Hai Phong City People's Committee as stated in Decision 2278/QD-UB dated 5 November 2009. At present, DVIZ JSC has, for going well with the actual conditions of project, made some adjustments for the detailed plan with scale of 1/500 for phase 2 project. Such adjustments were approved in principle by Hai Phong City People's Committee as stated in the letter 6541/UBND-GT dated 5th November 2010.

➢Regarding current state of project implementation, phase 1 project is now basically completed by DVIZ JSC and phase 2 project is now in process.

➢Regarding phase 2 project, DVIZ JSC has fulfilled the land acquisition and resettlement and hand-over of 135.6/377.46 ha for infrastructure construction. Development work for the remaining land is now in the process of compensation and site clearance. It is expected to be completed at the end of year 2011.

➢Finish grade of the land is +5.0m from sea chart level.

➢DVIZ JSC is conducting 2 docks of 10,000 to 20,000 DWT Liquid Cargo serving for the secondary enterprises in the Chemical and Petrochemical sectors of DVIZ. Up to now, there are 35 secondary investment projects of domestic and oversea running in DVIZ with total rental land area of 123.011 ha. Total investment capital of the secondary projects is USD 1,001.742 million.

➢Development of the Project is subject to the progress of land hand-over and normally delayed due to long time and complexity of land acquisition and resettlement procedures. For improving the state, DVIZ JSC produced the policies that the infrastructure development and investment attraction will be started as soon as land hand-over with the basic schedule as follows:

The basic infrastructure structure for phase 2 will be completed in 3 years as from the land is handed over.

Whole infrastructure structure of the industrial zone for phase 2 will be completed in consecutive 4 years.

The Study Team has received a master plan drawing from DVIZ JSC (see Figure 7.1.1-1). It does not fit together with overall master plan of Dinh Vu - Cat Hai Economic Zone and there is an overlapped section at the Southern portion. The Study Team discussed with DVIZ JSC and reached agreement that they would change their plan and omit the plan road at the overlapped section. However, construction of dike at the edge of DVIZ was on-going at the site when the Study Team visited the site on 8th June 2011.

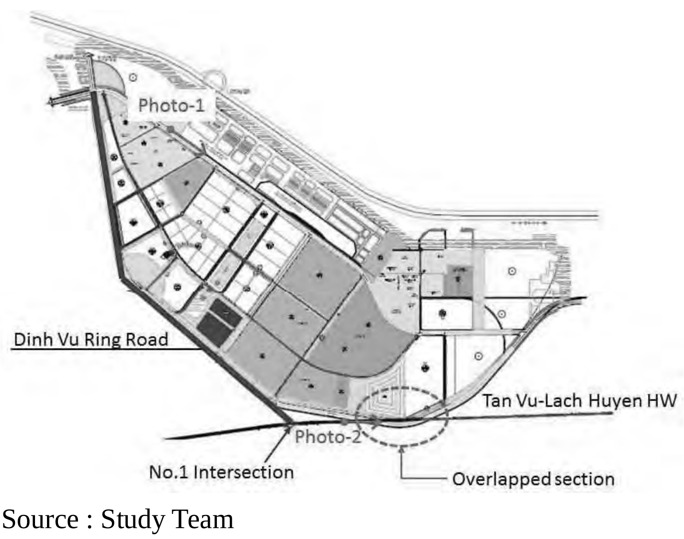


Photo-1 Northern Part

Photo-2 overlapped section

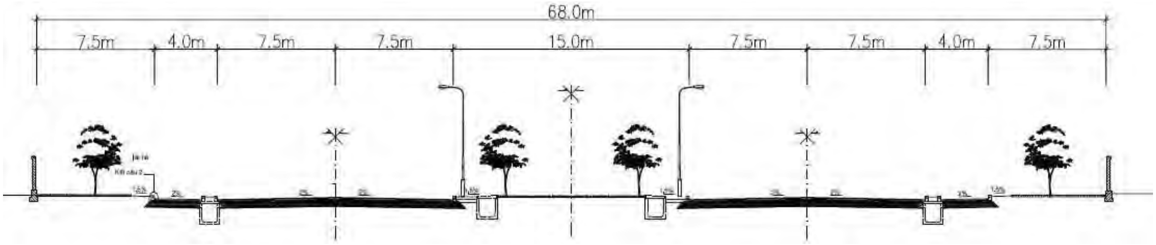
Source : Study Team

Figure 5.1.1-1 Plan of DVIZ

Classification of Dinh Vu Ring Road

Dinh Vu Ring Road will connect to the highway at No.1 Intersection and at-grade intersection with roundabout is planned. Classification of the road is as follows;

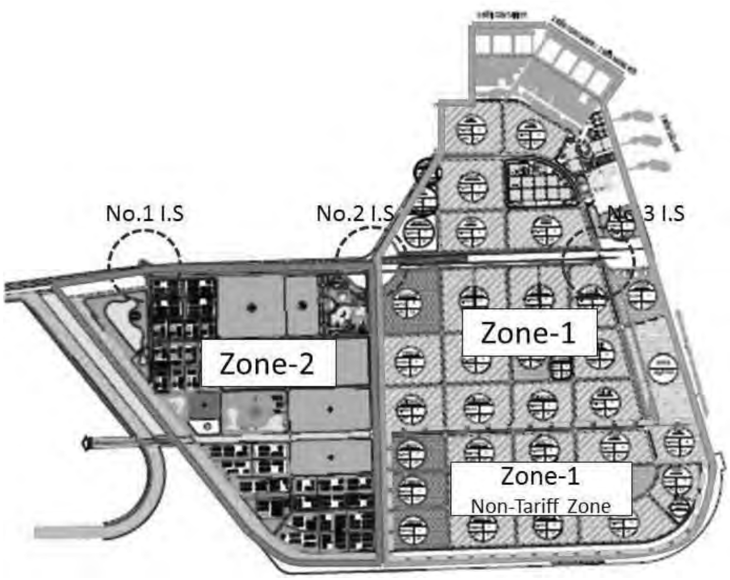
- ➢Total width: W=68.0m
- ➢Design speed: 80km/h
- ➢Cross section



The Study Team carried out design of No.1 intersection considering the classification of Dinh Vu Ring Road.

5.1.1.3 Current States and Future Plan of Nam Dinh Vu Industrial Zone

Nam Dinh Vu Industrial Zone (NDVIZ) is divided into two zones, Zone-1 and Zone-2, as shown in Figure 7.1.1-2. Nam Dinh Vu Investment Joint Stock Company (NDVIJSC) is an investor for Zone-1 and HAPACO Joint Stock Company (HAPACO) is an investor for Zone-2.



Source : Study Team

Figure 5.1.1-2 Plan of NDVIZ

The Study Team carried out questionnaire suvey to both NDVIJSC and HAPACO in Basic Design. According to answer from them, the land development work is being carried out based on both Decision No.795/QD-UBND dated 05th May 2009 issued by Hai Phong City People's Committee on detailed plan of None-tariff Zone and Nam Dinh Vu Industrial Zone (Zone-1) and Decision No.644/QD-UBND dated 16th April 2009 issued by Hai Phong City People's Committee on detailed plan of Nam Dinh Vu Industrial Zone (Zone-2). Summary of past background and current states of land development is as follows;

(1) Zone-1: NDVIJSC

Total Area of Zone-1 is 1,354ha (None-tariff zone:448ha (southern part), Industrial zone (northern part): 906ha) and the breakdown is shown in below table;

Table 5.1.1-1 Area of None-Tariff Zone

No.	Types of Land	Area (ha)	Ratio (%)
1	Production Area	118.0	26.34
2	Storage + Yard	98.5	21.99
3	Services & Trade	70.0	15.62
4	Green tree, Sport area	73.5	16.40
5	Technical Node Area	2.5	0.56
6	Transportation Area, Car Parking Area	80.5	17.97
7	Military area	5.0	1.12
	Total	**448.0**	**100.00**

Source : Study Team

Table 5.1.1-2 Area of Industrial Zone

No.	Types of Land	Area (ha)	Ratio (%)
1	Land of manufacturing area	307.0	33.88
2	Warehouse + container yard	187.5	20.70
3	Port area	143.6	15.85
4	Operation Centre + logistic service	56.0	6.18
5	Green tree area, sport & gymnastic area	91.0	10.04
6	Technical Node Area	7.0	0.77
7	Transportation land, parking	113.9	12.58
	Total	**906.0**	**100.00**

Source : Study Team

- Finish grade of the land is +5.0m from sea chart level.
- Land development work is being carried out from North to South and 370ha of Northern industrial area will be completed by the end of 2013. At present, 100ha of landfill work has been completed already.
- Land development work at Southern part of the highway can be started after completion of Northern area, at the soonest from 2014.

(2) Zone2: HAPACO

➢Total Area is 658ha and the breakdown is shown in below table;

Table 5.1.1-3 Area of Industrial Zone

No.	Types of Land	Area (ha)	Ratio (%)
1	Production area	190.0	28.88
	Heavy industry	114.5	
	Light industry	75.5	
2	Store + yard	201.0	30.55
3	Operation Centre + services	29.0	4.41
4	Green tree, sport & gym mastic area	67.0	10.18
5	Isolation green tree	45.0	6.84
6	Technical Node Area	8.0	1.22
7	Transportation area, car parking area	118.0	17.92
	Total	**658.0**	**100.00**

Source : Study Team

➢Finish grade of the land is +5.0m from sea chart level.

➢At present, land development work has not been started yet. Therefore, construction of the highway should be considered to be executed without any landfill of industrial zone.

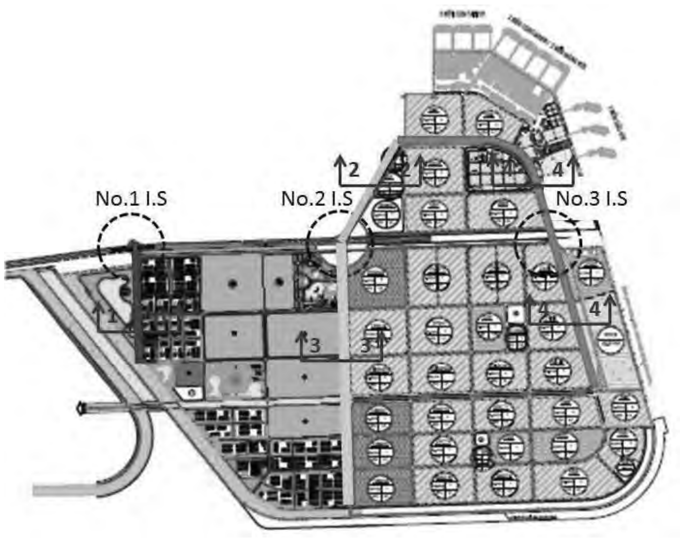
The Study Team has got information that Zone-2 would be divided into two parts and another investor was planning to join.

Source : Study Team

Figure 5.1.1-3 Photo at Zone-2 (Land development work not started yet)

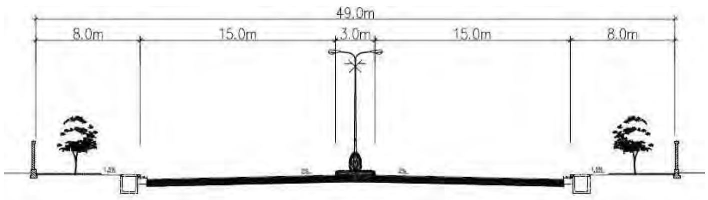
(3) Classification of Internal Road in Industrial Zone

There are several internal roads in industrial zone which will have a crossing with the highway (see Figure 7.1.1-4). Typical cross section of each road is shown in Figure 7.1.1-5. No.1 intersection is at-grade intersection, while No.2 and No.3 intersections are inaccessible due to ensuring navigation clearance for future railway.

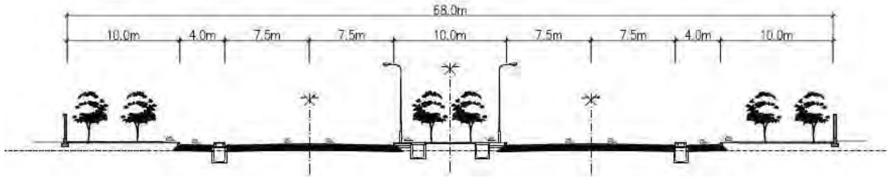


Source : Study Team

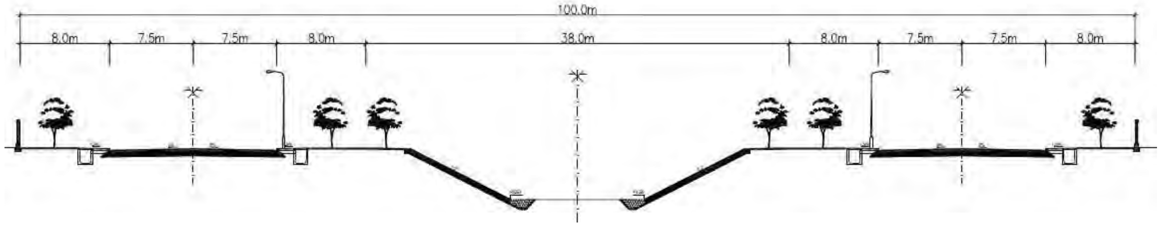
Figure 5.1.1-4 Crossing Roads



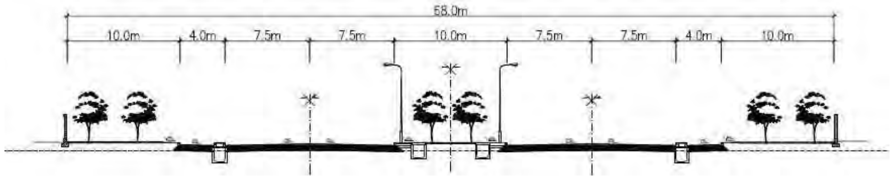
1-1 section (W=49m, V=60km/h)



2-2 section (W=68m)



3-3 section (W=100m)



4-4 section (W=68m)

Source : Study Team

Figure 5.1.1-5 Cross Section of IZ Internal Roads

5.1.2 Design concept in Cat Hai area

As mentioned above, a master plan for Dinh Vu - Cat Hai Economic Zone which is being prepared by Nikken Sekkei Civil Engineering Ltd. has not been officially approved yet. Therefore, the Study Team carries out Detailed Design in Cat Hai area considering present life of local people. The Study Team reviewed SAPROF study and modified size and location of box culverts, pipe culverts, frontage road, relocation of channel, etc. based on site checking in Basic Design. Then, meetings were held between local authorities and the Study Team discussing on the updated plan. The Study Team received some comments and requests from local authorities after the discussion and sent a letter to local authorities to request them to approve the updated plan on 1st July.

CHAPTER 6 TRAFFIC DEMAND FORECAST

6.1 General

Two traffic demand forecasts were carried out in the past in order to determine traffic flows for the Tan Vu-Lach Huyen Highway (i.e., the Project road). The final results of the first forecast were submitted in July 2009 as part of a Feasibility Study (hereafter referred to as the FS) by a consortium consisting of Japan Bridge & Infrastructure Institute and Hyder Consulting & Highway Engineering Consultants, with Vietnam Infrastructure Development and Finance Investment being the Client.

Then, a second forecast was executed by the consortium of Nippon Koei and Japan Bridge & Infrastructure Institute under a JICA preparatory survey (hereafter referred to as the SAPROF), which was requested by the Vietnamese Government, as there was a wish to implement the Project road together with Lach Huyen Port as a Japanese-funded scheme instead of the originally planned BOT scheme.

The results of the SAPROF were submitted in July 2010, and based on these results the Governments of Japan and Vietnam agreed to carry out the Lach Huyen Port Infrastructure Construction Project (hereafter referred to in this chapter as the "LH Project"), which will produce detailed designs for both the port and road/bridge facilities. Project staff was mobilized in March 2011.

6.2 Objective

The objective of this chapter is to review the traffic demand forecasts of the FS and SAPROF and to make revisions as necessary in order to produce a final traffic demand forecast for the design of the Tan Vu-Lach Huyen Highway. Given the time and resource constraints of the Project, no new large-scale surveys were planned and existing data and modeling was applied to the fullest extent possible.

6.3 About the Project Area & Road

The Project area, which is located in Hai Phong City, consists of the three islands of Dinh Vu, Cat Hai, and Cat Ba, with the Project road connecting the former two islands with the mainland at Hai An District (see Figure 6.5.2-1 on page 6-9). The Project road is to be a corridor strongly linear in nature having high transport capacity. At present, there will be three major locations with access to the Project road, which consists of the two termini of Tan Vu IC at the junction with Hanoi-Hai Phong Expressway and the entrance of Lach Huyen Port at the eastern end of Cat Hai Island, together with an intersection on Dinh Vu Island (i.e., Intersection#1). Note that there is a plan to add a railway to the corridor in the distant future.

6.4 Review of FS & SAPROF Traffic Demand Forecasts

Below a review of the FS and SAPROF traffic demand forecasts is carried out. The SAPROF applied the same methodology as the FS for estimating traffic demand but updated and adjusted the variables to arrive at a more reliable result. This report continues the refinement process in order to improve accuracy even further.

6.4.1 Target Years

The target years that were applied in the FS and SAPROF traffic forecasts are as shown in Table 6.4.1-1, with the SAPROF having an additional target year of 2035. It was decided that this JICA report would apply the same target years as that of the SAPROF in the interest of continuity.

Table 6.4.1-1 FS & SAPROF Target Years

Study	Target Years	Remarks
FS	2015, 2020, 2030	Target years of 2022 & 2032 also taken up to coordinate with the Hanoi-Haiphong Expressway Study by Yooshin-KPT.
SAPROF	2015, 2020, 2030, 2035	Target year of 2035 added at request of Vietnam Ministry of Transport.

Source : Study Team

6.4.2 Traffic Forecasting Methodology

As the Project road is strongly linear in its make-up, both the FS and SAPROF applied a simplified traffic demand (i.e., corridor) model to calculate future traffic flows. That is, no trip distribution or traffic assignment models were utilized and modal split was determined exogenously. Given this, a trip generation/attraction model assuming trips would only use the Project road was developed by the FS, which was later updated and revised by the SAPROF. The output of the model consists of peak-hour passenger car unit (PCU) trips derived from variables quantifying industrial area, the amount of freight handled by ports, population, and numbers of tourists. Note that a trip generation/attraction model was constructed for each of the three islands of Dinh Vu, Cat Hai, and Cat Ba, and the values of their variables are as shown in Tables 6.4.2-1 to 6.4.2-3.

Table 6.4.2-1 Dinh Vu Island Trip Generation/Attraction Variables & Values for FS & SAPROF

Variable	Unit	2015		2020		2030	
		FS	SAPROF	FS	SAPROF	FS	SAPROF
Industrial Area	$100m^2$	32,780	16,375	65,568	32,750	78,680	97,680
Port Area Freight	tons/hr	4.5 mil	4.5 mil	6 mil	6 mil	10 mil	10 mil
Apt. for Rent	units	174,350	162,500	348,700	325,000	523,050	650,000

Source : Study Team

Table 6.4.2-2 Cat Hai Island Trip Generation/Attraction Variables & Values for FS & SAPROF

Variable	Unit	2015		2020		2030	
		FS	SAPROF	FS	SAPROF	FS	SAPROF
Population	persons	31,000	19,000	33,000	19,300	38,500	20,100
Port Area Freight	tons/hr	4.2 mil	5.394 mil	33.0 mil	29.525 mil	80.0 mil	120.0 mil
Tourists	persons/year	500,000	500,00	1.6 mil	1.6 mil	2.6 mil	2.6 mil

Source : Study Team

Table 6.4.2-3 Cat Ba Island Trip Generation/Attraction Variables & Values for FS & SAPROF

Variable	Unit	2015		2020		2030	
		FS	SAPROF	FS	SAPROF	FS	SAPROF
Population	persons	12,000	12,000	14,500	13,000	16,500	14,600
Tourists	persons/year	500,00	500,00	1.6 mil	1.6 mil	2.6 mil	2.6 mil

Source : Study Team

As the above tables indicate there were significant downward revisions in the SAPROF regarding development of industrial and residential areas for the years 2015 and 2020, but then a large upward revision for 2030. As for population, the SAPROF made large downward revisions, as it seems that the FS data was mistakenly double counted. The SAPROF also greatly increased the freight to be handled by Lach Huyen Port based on data from a 2010 JICA survey and on information from the Vietnam Ministry of Transport.

Note that the land-use and population data in the FS was extracted from the “Lach Huyen Gateway Port – Hai Phong, Closing Statement” and the “Investment Report of Dinh Vu – Cat Hai Bridge Construction Project in Hai Phong City”. As for the SAPROF, its land-use and population data was supplemented from more recent documents that included “Decision No. 644/QD-UBND” of 16 April 2009, “Decision No. 795/QD-UNBD” of 29 May 2009, and the “Statistical Yearbook 2008 of Hai Phong”.

In Table 6.4.2-4, the trip generation and attraction rates for the above variables are shown. Note that these rates are based on data from China on the premise that the growth pattern Vietnam is now experiencing is similar to that of China.

Table 6.4.2-4 Trip Generation & Attraction Rates

Variable	Trip Generation Rate (AM Peak Hour)	Trip Attraction Rate (AM Peak Hour)
Industrial Area	0.110 pcu/hr/100m^2	0.150 pcu/hr/100m^2
Port Area Freight	0.082 pcu/ton	0.082 pcu/ton
Apt. for Rent	0.250 pcu/hr/unit	0.080 pcu/hr/unit
Population*	0.250 pcu/hr/household	0.080 pcu/hr/household
Tourists	0.400 pcu/hr/person	0.400 pcu/hr/person

**: Divided by four to estimate number of households*

Source : Study Team

Based on the preceding, the FS and SAPROF estimated traffic flows for the Tan Vu - Dinh Vu and Dinh Vu - Cat Hai sections of the Project road for the years 2015, 2020, and 2030, and which are shown in Table 6.4.2-5. From this table the following can be stated:

- The traffic volume for the SAPROF is about 40% to 44% lower than that for the FS for the year 2015.
- The traffic volume for the SAPROF is about 41% to 45% lower than that for the FS for the year 2020, except for the Dinh Vu to Cat Hai direction (which is approx. 36% lower).
- The traffic volume for the SAPROF in the year 2030 is lower than that of the FS in all cases, varying from a decrease of approximately 3% to 31%, except for the direction of Tan Vu to Dinh Vu, which saw the traffic for the SAPROF increase by about 13% over that of the FS.

Table 6.4.2-5 Comparison of FS & SAPROF Traffic Forecasts for Morning Peak Hour (unit: PCU)

Section	Direction	Year 2015		Year 2020		Year 2030	
		FS	SAPROF	FS	SAPROF	FS	SAPROF
Tan Vu IC- Dinh Vu	To Tan Vu IC	2,272	1,276	3,789	2,149	4,624	4,140
	From Tan Vu IC	1,304	745	2,457	1,451	3,515	3,967
Dinh Vu - Cat Hai	Cat Hai to Dinh Vu	1,680	927	2,691	1,494	2,888	2,002
	Dinh Vu to Cat Hai	583	351	1,157	745	1,392	1,350

Note: Railway assumed to be in place in 2030.

Source : Study Team

6.5 Updating of Traffic Demand Forecast

As stated earlier, the purpose of this chapter is to refine the previous work of the FS and SAPROF by updating the traffic demand models utilized in those reports. Below, the traffic model variables and assumptions of those models for the three areas comprising the Project (i.e., Dinh Vu, Cat Hai, Cat BA) are updated and revised as necessary.

6.5.1 Traffic Model Variables

6.5.1.1Dinh Vu

Based on information received from the Dinh Vu Industrial Zone (DVIZ) Company, the size and pace of development for the industrial and residential areas was confirmed and updated as shown in Table 6.5.1-1. As for the amount of freight to be handled by Dinh Vu Port, this was set taking into account planning data from Decision No 2190/QD-TTg issued by the Prime Minister regarding the planning of Vietnam seaport systems, information from DVIZ Co., and the total expected capacity of ports in Hai Phong as estimated by the LH Project team in charge of port design.

Table 6.5.1-1 Dinh Vu Island Trip Generation/Attraction Variables for LH Project

Variable	Unit	Year 2015	Year 2020	Year 2030
Industrial Area	$100m^2$	34,800	59,800	65,500
Port Area Freight	tons/hr	5.72 mil	9.77 mil	15 mil
Apt. for Rent	units	100,000	300,000	650,000

Source : Study Team

The amount of freight expected to be handled by Dinh Vu Port is larger than that of the FS and SAPROF by 1.27, 1.63, and 1.50 times for the years 2015, 2020, and 2030, respectively. As for residential development, the FS values are larger than the values of this report by about 1.74 and 1.16 times for the years 2015 and 2020, respectively, while for the year 2030 the FS value is about 19.5% smaller. In the case of the SAPROF, its values are 1.62 and 1.08 times larger for the years 2015 and 2020, while for the year 2030 the values of this report and the SAPROF for residential development are the same.

6.5.1.2 Southern Dinh Vu

In order to account for trips that would use Intersection#1 of the Project road coming from the south of Dinh Vu Island, a separate trip generation/attraction model for Southern Dinh Vu industrial zone is constructed and its forecasting variable is as shown in Table 6.5.1-2. Note that neither the FS nor the SAPROF modeled for this movement explicitly. Information for industrial area development was obtained from Decision No. 795/QD-UBND and from Nam Dinh Vu Industrial Co.

Table 6.5.1-2 Southern Dinh Vu Island Trip Generation/Attraction Variable for LH Project

Variable	Unit	2015	2020	2030
Industrial Area	100m^2	10,000	21,000	59,450

Source : Study Team

As the above table indicates, industrial development in Southern Dinh Vu is quite small till 2015. Note that when comparing the industrial area of Dinh Vu with earlier reports, the values of Table 6.5.1-2 and 6.5.1-1 must be added. Given this, assumed industrial development in this report is 1.37, 1.23, and 1.58 times larger than that of the FS and 2.74, 2.47, and 1.27 times greater than that of the SAPROF for the years 2015, 2020, and 2030, respectively.

6.5.1.3 Cat Hai Island

The population for Cat Hai was revised downward significantly and is based on time-series data received from the Hai Phong People's Committee (HPPC) as well as on the document "Adjustment of the General Construction Planning of Hai Phong City till 2025 – Vision 2050". That is, the FS population values are 2.38, 2.34, and 2.35 times larger than that of this report, while the SAPROF values are 1.46, 1.37, and 1.23 times greater for the years of 2015, 2020, and 2030, respectively.

In regards to the amount of freight to be handled at Lach Huyen Port, this was determined using the same data sources as that for Dinh Vu Port described above. Compared to the FS, this report has values that are 8% and 22% smaller for 2015 and 2020 while for 2030 it is 1.25 times larger. As for the SAPROF, its values are about 1.40, 1.01, and 1.20 times greater than that of this report for the years 2015, 2020, and 2030, respectively.

Concerning the number of tourists visiting Cat Hai Island, this is also based on time-series data provided by the HPPC and future forecasts were derived from trend analysis. Compared to the FS and SAPROF, the values of this report are 3.0, 1.26, and 1.15 times greater for the years 2015, 2020, and 2030, respectively.

Table 6.5.1-3 Cat Hai Island Trip Generation/Attraction Variables for LH Project

Variable	Unit	2015	2020	2030
Population	persons	13,000	14,100	16,400
Port Area Freight	tons/hr	3.85 mil	29.14 mil	100.00 mil
Tourists	persons/year	1.51 mil	2.01 mil	2.99 mil

Source : Study Team

6.5.1.4 Cat Ba Island

As in the case of Cat Hai Island, time-series data for Cat Ba's population and tourists visiting the island was obtained from the HPPC and future forecasts derived via trend analysis (see Table 6.5.1-4). Population values for this report are greater than the FS and SAPROF values by 1.56, 1.46, 1.00 times and 1.56, 1.63, 1.13 times, respectively, for the years of 2015, 2020, and 2030. In the case of tourists, the variations between this report and the FS and SAPROF are the same as that for Cat Bai.

Table 6.5.1-4 Cat Hai Island Trip Generation/Attraction Variables for LH Project

Variable	Unit	2015	2020	2030
Population	persons	18,750	21,200	16,500
Tourists	persons/year	1.51 mil	2.01 mil	2.99 mil

Source : Study Team

6.5.2 Traffic Model Assumptions

6.5.2.1 General

(1) Transport Network

The future transport network, which was discussed in detail with the Hai Phong Department of Transport, is essentially the same as that of the FS and SAPROF except for the following important exceptions:

- No railway is assumed to be in place by 2030 that will link Lach Huyen Port with Tan Vu and Dinh Vu. This is based on the unclear status of said project, as well as on the fact that only 3.7% of marine freight is hauled by rail from Hai Phong (see December 2007 World Bank report: "Urban Transport in Vietnam's Medium Cities"). Note also that with the construction of the Hanoi-Hai Phong Expressway it is expected that rail will become even more uncompetitive.
- No bridge will connect Cat Hai with TL359 to the north by 2030, which is planned to be built by 2050.
- Ring Road #3 will not be completed until 2030; whereas, in the FS and SAPROF it was assumed to be finished in 2015.

For reference, the transport network assumed to be completed by the year 2030 is as shown in Figure 6.5.2-1.

(2) Peak Hour Factors

The peak hour factors applied in the FS, which were also adopted by the SAPROF, were revised as it was considered more accurate to apply factors derived from local traffic conditions in Hai Phong rather than based on overseas sources. Note that the FS peak hour factors for motorcycle and car were 7% and for light goods vehicle (LGV), heavy goods vehicle (HGV), and bus 5%.

Table 6.5.2-1 Peak Hour Factors by Vehicle Type

Vehicle Type	Motorcycle	Car	LGV	HGV	Bus
Peak Hour Factor	11%	10%	12%	9%	10%

Note: Estimated from 2009 traffic survey by the Hai Phong Urban Development Project

Source : Study Team

(3) PCU Conversion Factors

The PCU conversion factors utilized in this report are the same as those used in the FS and SAPROF reports, which are from the Vietnam standard of TCVN-4054-2005, and are as shown in Table 6.5.2-2.

Table 6.5.2-2 PCU Conversion Factors

Vehicle Type	PCU Factor
Motorcycle	0.3
Car	1.0
LGV (trucks of 2 axles & mini-buses	2.0
HGV (trucks with more than 3 axles & large buses)	2.5

Source : Study Team

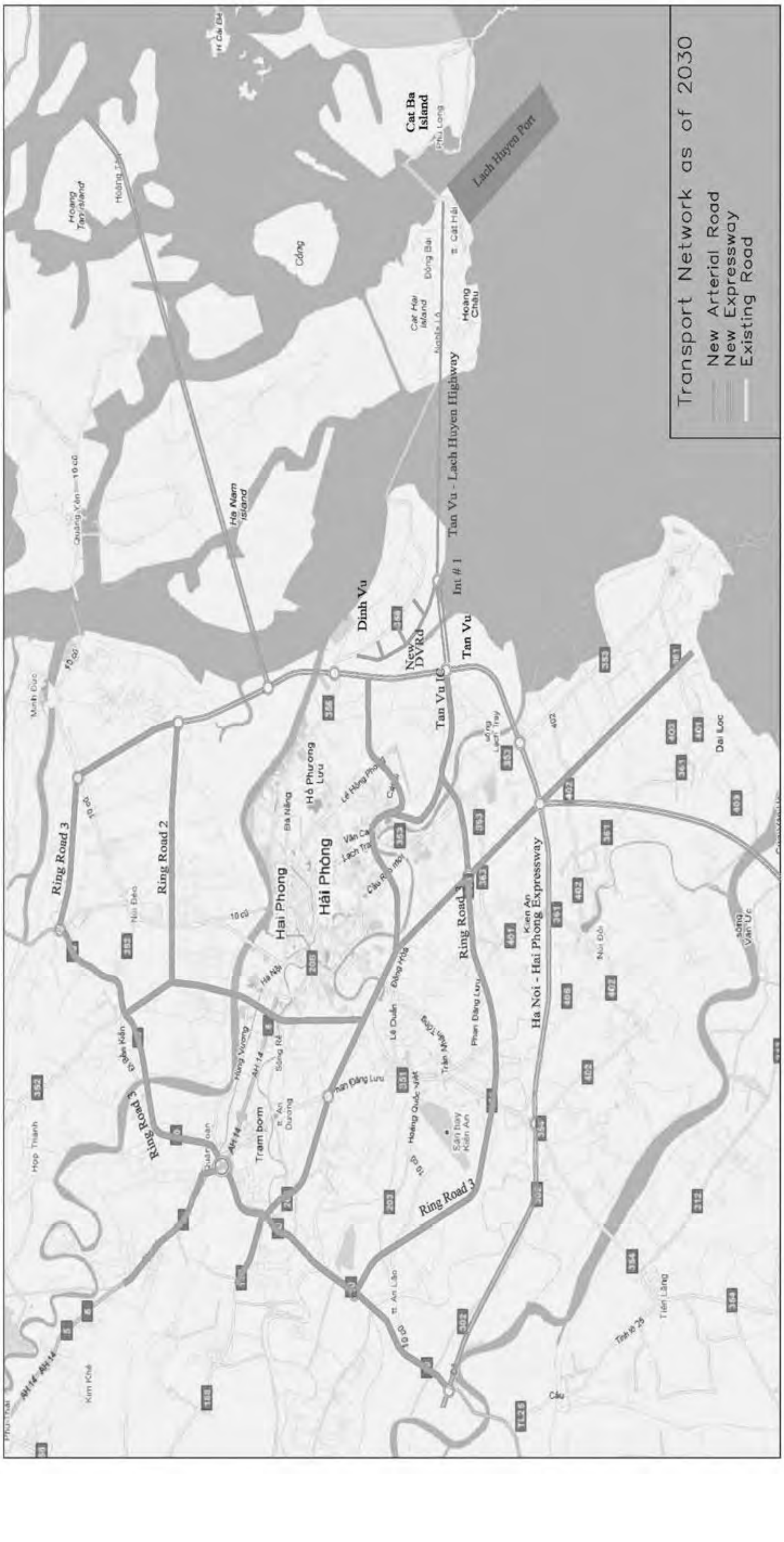


Figure 6.5.2-1 Future Road Network in 2030

Source : Study Team

6.5.2.2 Area Specific

Depending on the area of the Project there are differences in trip generation/ attraction. Below, assumptions regarding the three zones comprising the Project area are examined.

1) Dinh Vu Island

Assumptions regarding trip generation/attraction for Dinh Vu were made for the six items listed below.

- **Industrial Ground Floor Area (GFA):** One of the land-use components that produces trips and is represented as the ratio of industrial ground floor area to total land area. It was assumed in the FS, SAPROF, and in this report that the GFA for Dinh VU was 30%.
- **Peak Hour Factor:** Meant here as the percentage of traffic in the most congested hour of the day in relation to total daily traffic. In the FS and SAPROF, the figure of 5% was adopted based on overseas data. In this report, 9% was applied based on 2009 traffic survey data from the Hai Phong Urban Development Project.
- **Apartment Footprint:** The other land-use component that produces trips. All three studies assumed that the apartment footprint, which is the ratio of land an apartment building occupies within a residential block, would be 50% with a plot ratio of 5.
- **Modal Split of Industrial Employees:** Modal split assumed in this report for industrial employees differs from that of the FS and SAPROF, which are the same, and it is derived from the abovementioned 2009 traffic survey and car ownership time-series data for the period of 2002 to 2009 from the Development Strategy Institute. As Table 6.5.2-3 indicates, this report assumes that growth in car use is much slower and that motorcycles will remain the dominant mode of transport, while public transport use will remain static due to increases in personal income and the ability for people to own a vehicle.

Table 6.5.2-3 Modal Split of Industrial Employees in Dinh Vu

Mode	2015		2020		2030	
	FS& SAPROF	LH Project	FS& SAPROF	LH Project	FS& SAPROF	LH Project
Motorcycle	70%	89%	50%	85%	30%	75%
Car	30%	6%	30%	10%	50%	20%
Public Transport	-	5%	20%	5%	20%	5%

Note: The FS and SAPROF assume the existence of a railway in 2030.

Source : Study Team

- **Modal Split of Residents:** This report assumes that Dinh Vu residents will work in the nearby companies of the DVIZ and therefore not use public transport. Based on the same logic and data as in the preceding item, growth in car use is much slower than that assumed by the FS and SAPROF, which have adopted the same modal split values, and motorcycles remain the dominant mode of transport (see Table 6.5.2-4).

Table 6.5.2-4 Modal Split of Residents in Dinh Vu

Mode	2015		2020		2030	
	FS& SAPROF	LH Project	FS& SAPROF	LH Project	FS& SAPROF	LH Project
Motorcycle	50%	94%	30%	90%	20%	80%
Car	50%	6%	70%	10%	60%	20%
Public Transport	-	-	-	-	20%	-

Note: The FS and SAPROF assume the existence of a railway in 2030.

Source : Study Team

- **Project Road Usage:** Use of the Project road by those working/residing in Dinh Vu is an important factor affecting traffic flows. The assumption by the FS for both North Dinh Vu and South Dinh Vu, which essentially means those working/residing north and south of Intersection#1 of the Project road, respectively, is 100%. In the case of the SAPROF, the usage was assumed to be 50% for North Dinh Vu and 80% for South Dinh Vu. The reason behind this being that those people in North Dinh Vu have other options such as using NH5. As for this report, an interview with 17 Dinh Vu companies was carried out in May 2011 and based on the results of said survey it was assumed that 80% and 90% of those residing/working in North Dinh Vu and South Dinh Vu, respectively, would use the Project road.

Table 6.5.2-5 Project Road Usage

Area	FS	SAPROF	LH Project
North Dinh Vu	100%	50%	80%
South Dinh Vu	100%	80%	90%

Source : Study Team

2) Cat Hai Island

Assumptions regarding trip generation/attraction for Cat Hai were made for the four items listed below.

- **Average Household Size:** This was assumed to be 4 in both the FS and SAPROF, and is used to convert population into household units for PCU trip calculation purposes. This report, after obtaining data from the Development Strategy Institute, decided also to adopt this value.
- **Peak Hour Factor:** The logic is the same as that stated above for Dinh Vu.
- **Modal Split:** The modal split of the FS and SAPROF are the same. Modal split for this report was estimated by referring to ferry data for the period of 2002 to 2009. As for future growth by mode, the logic and sources of information are essentially the same as that stated above for Dinh Vu, meaning that motorcycles will remain the dominant mode of transport but less so owing to greater public transport use, which is due to the greater travel distances required for traveling between Cat Hai centers of activity in Hai Phong (see Table 6.5.2-6).

Table 6.5.2-6 Modal Split for Cat Hai (unit: %)

Mode	2015		2020		2030	
	FS& SAPROF	LH Project	FS& SAPROF	LH Project	FS& SAPROF	LH Project
Motorcycle	50%	81%	30%	76%	20%	66%
Car	50%	5%	70%	10%	60%	20%
Public Transport	-	14%	-	14%	20%	5%

Source : Study Team

- **Tourist Travel Behavior**: The FS and SAPROF reports assumed that modal choice for tourists would be 20% bus and 80% other modes of transport and to remain static for the 2015 to 2030 period. This report, on the other hand, assumes that modal choice for tourists is 90% bus and 10% car, 80% bus and 20% car, and 70% bus and 30% car for the years 2015, 2020, and 2030, respectively. Calculations were again based on the abovementioned ferry data, with future growth in car ownership growth based on trend analysis of data from the Development Strategy Institute.

  ●As for the peak hour for tourists, both the FS and SAPROF assumed 6%. This report considered this figure too low and revised it to 10%. Moreover, this report, based on information from the Hai Phong Tourist Dept., assumes that 60% of all tourists going to Cat Hai will use the Project road..

3) Cat Ba Island

Assumptions regarding trip generation/attraction for Cat Ba were made for the three items listed below.

- **Average Household Size:** Same as Cat Hai.
- **Modal Split:** The modal split values assumed by the FS and SAPROF are again the same, with the values for Cat Ba equal to those for Cat Bai. In this report, modal split was estimated for Cat Ba using the same sources mentioned above in Cat Hai. Note that bus usage is greater for Cat Ba than Cat Hai given the greater distance to the mainland and therefore the difficulty of using a motorcycle.

Table 6.5.2-7 Modal Split for Cat Ba (unit: %)

Mode	2015		2020		2030	
	FS& SAPROF	LH Project	FS& SAPROF	LH Project	FS& SAPROF	LH Project
Motorcycle	50%	72%	30%	85%	20%	75%
Car	50%	7%	70%	10%	60%	20%
Public Transport	-	21%	-	5%	20%	5%

Source : Study Team

- **Tourist Travel Behavior**: It was assumed by the three reports that tourists going to Cat Bai would also visit Cat Ba due to their close distance.

6.6 Future Traffic Flows on Project Road

With the updating and revision of the preceding variables and assumptions, initial future traffic flows for the Project road in the morning peak hour were estimated using the trip generation/attraction models. Then, these initial traffic flows were adjusted and finalized by taking into account the following two important conditions:

- **Ring Road#3 (RR#3):** Unlike the FS and SAPROF, which assume completion of RR#3 by 2015, this report, after careful examination of the situation on the ground, assumes that RR#3 will not be completed until after 2020. This means that vehicles wishing to access Hai Phong from the Project road will most likely exit at Intersection#1 and travel up through Dinh Vu Island instead of exiting at Tan Vu IC. Likewise, vehicles from Hai Phong wishing to use the Project road are assumed to access it from Intersection#1.
- **Motorcycle Traffic:** It was assumed in the FS and SAPROF that motorcycles would be able to use the Hanoi-Hai Phong Expressway. However, as motorcycles are banned from using the Expressway, motorcycle traffic originally expected to use Tan Vu IC going right (i.e., towards Dinh Vu IC) is rerouted through Intersection#1 and traffic going left towards Hanoi is assumed to shift to bus.

The final traffic flows for the Project road for the peak hour are as shown in Figures 6.6-1, 6.6-2, 6.6-3, and 6.6-4 for the years 2015, 2020, 2030, and 2035, respectively. Note that traffic on the road to the left of the Tan Vu IC (i.e., RR#3) in 2015 and 2020, which is indicated as a dotted line, is zero as it does not yet exist. With the construction of RR#3, there is a large shift of traffic in 2030 from the Intersection#1-Dinh Vu Road to the Dinh Vu-Tan Vu Road. Daily traffic was estimated from the peak hour traffic using the factors described in 5.5.2. As Table 6.6-1 indicates, daily traffic for the Project area was about 121,000 vehicles in 2015, 198,000 in 2020, and 413,000 in 2030. Average annual growth in daily traffic after Project road completion in 2015 is 10.35% till 2020 and from 2020 till 2030 it is 7.60% (see Table 6.6-2). Tables 6.6-3, 6.6-4, and 6.6-5 contain daily traffic flow data by direction and vehicle type for the years 2015, 2020, and 2030, respectively.

Table 6.6-1 Daily Traffic by Year for Project Area

	2015	2020	2030
Daily Traffic (veh)	121,281	198,409	412,634

Source : Study Team

Table 6.6-2 Average Annual Growth in Daily Traffic for Project Area

	2015-2020	2020-2030
Daily Traffic (veh)	10.35%	7.60%

Source : Study Team

Turning movement traffic flows for the morning peak hour for the Tan Vu IC and Intersection#1 are as indicated in Figures 6.6-5 and 6.6-6. In the case of the Tan Vu IC, turning movement ratios from the FS together with traffic flow data from the Hanoi – Hai Phong Expressway Project and the trip generation/attraction model of this report were used for estimation purposes. As for Intersection#1, the trip/generation model flows of this report and the concept of gravitational attraction were used to calculate turning movements. Traffic flows at the Tan Vu interchange are again much less than the FS, as motorcycle traffic was restricted and RR#3 was not completed until after 2020.

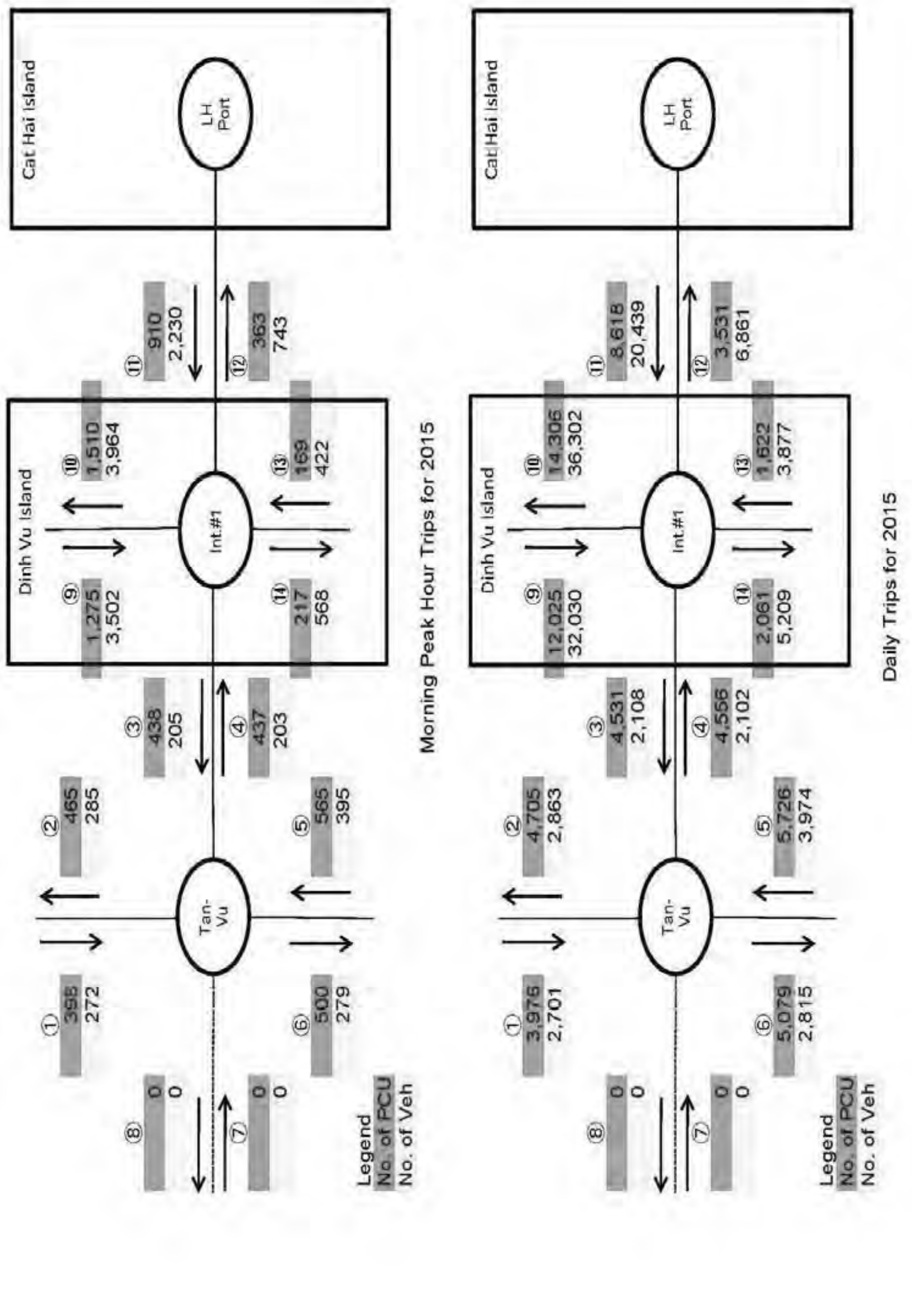
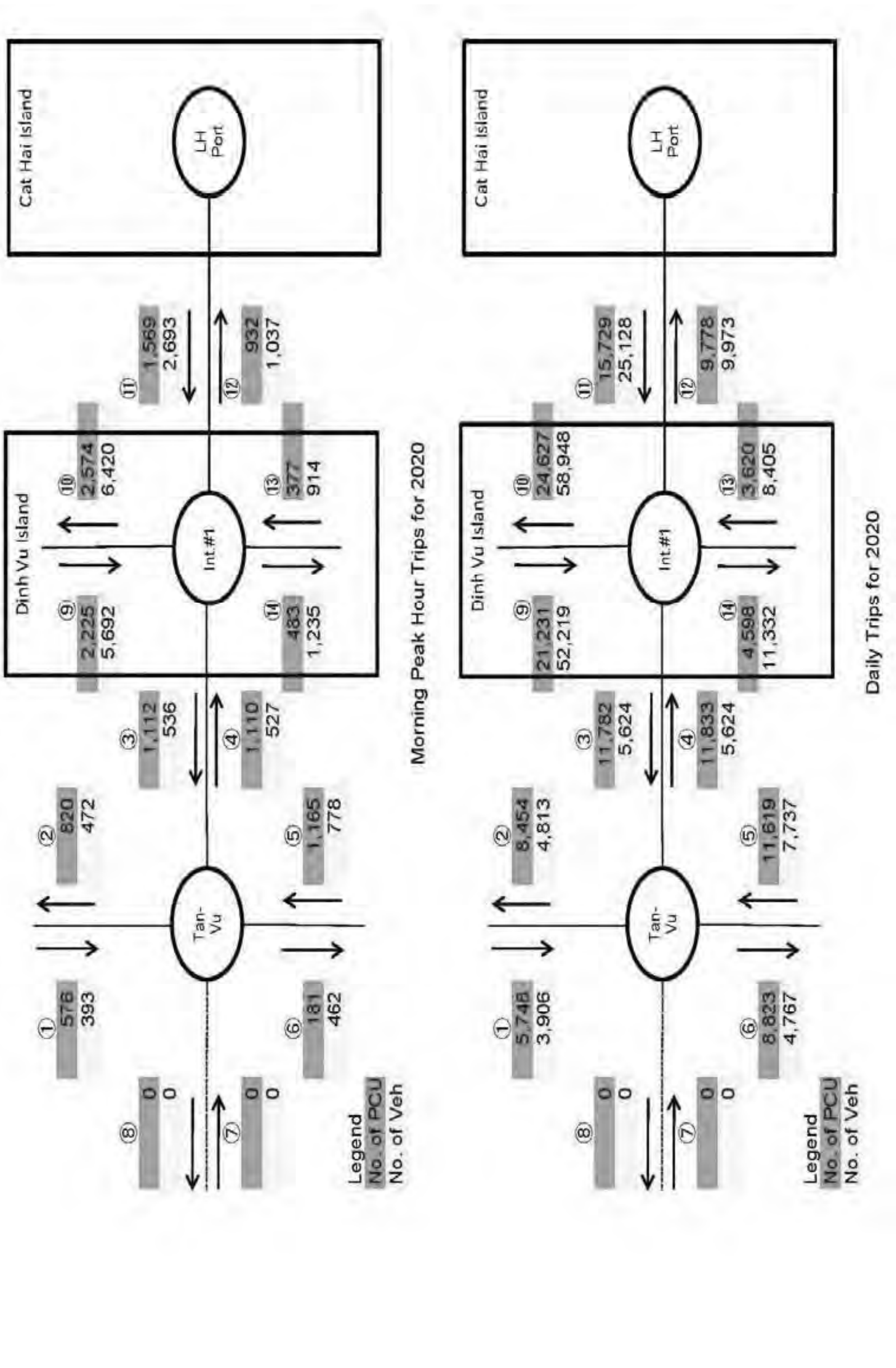


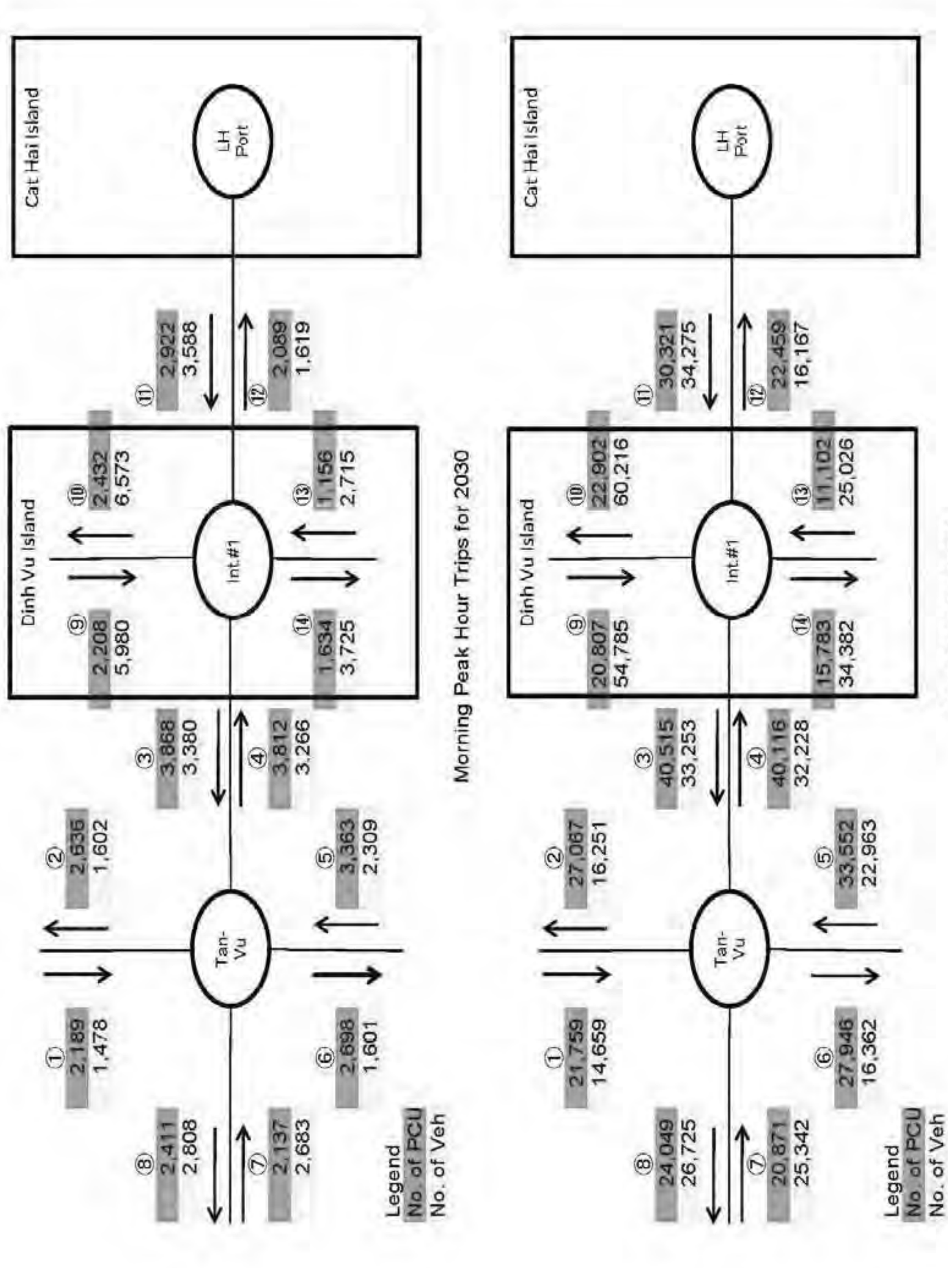
Figure 6.6-1 Peak Hour & Daily PCU & Vehicle Trips for Project Road for 2015

Source : Study Team



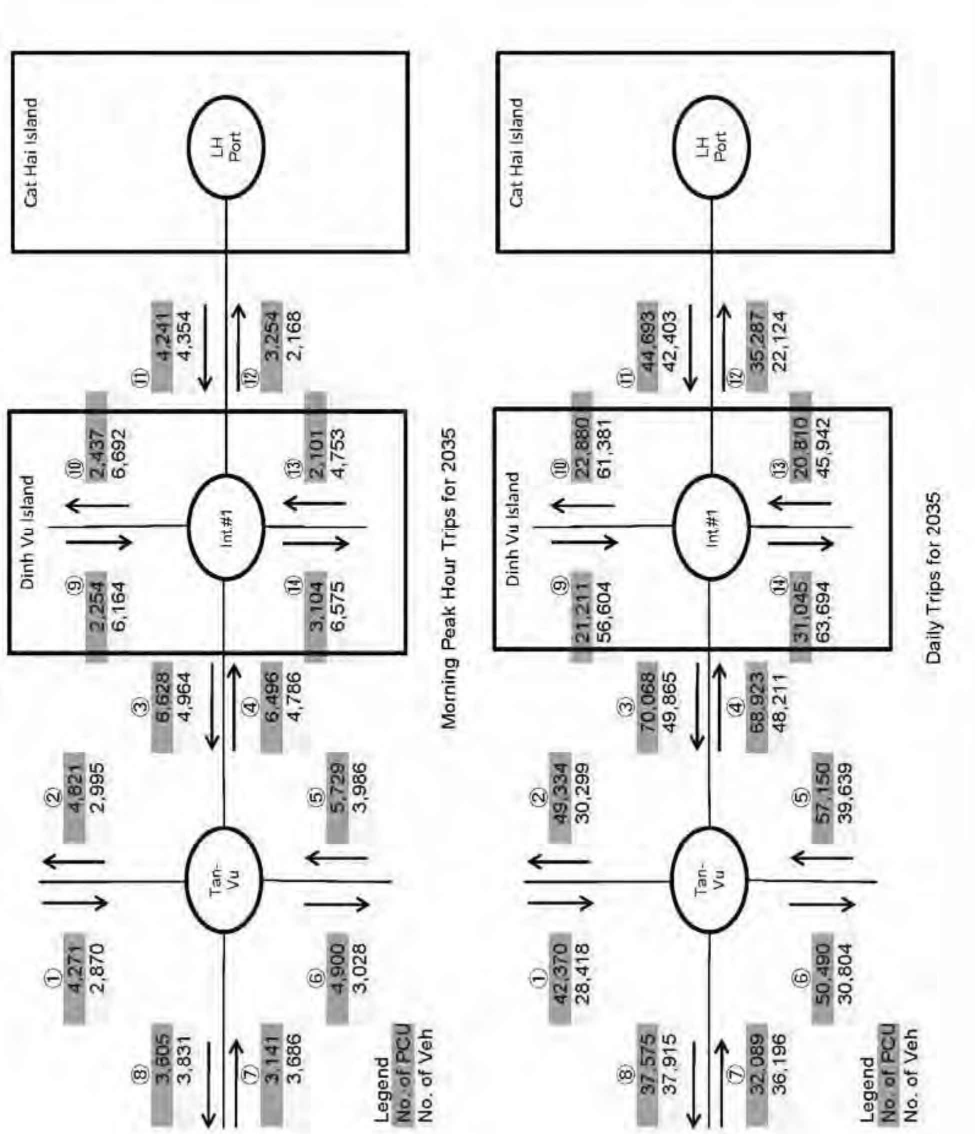
Source : Study Team

Figure 6.6-2 Peak Hour & Daily PCU & Vehicle Trips for Project Road for 2020



Source : Study Team

Figure 6.6-3 Peak Hour & Daily PCU & Vehicle Trips for Project Road for 2030



Source : Study Team

Figure 6.6-4 Peak Hour & Daily PCU & Vehicle Trips for Project Road for 2035

Table 6.6-3 Daily Traffic by Vehicle Type per Direction for 2015

Direction	MC	Car	LGV	HGV	Bus	Total Veh	Total PCU
(1)	0	1,753	295	432	221	2,701	3,976
(2)	0	1,560	226	595	482	2,863	4,705
(3)	0	493	0	605	1,010	2,108	4,531
(4)	0	466	0	755	881	2,102	4,556
(5)	0	2,731	226	673	344	3,974	5,726
(6)	0	1,251	164	580	820	2,815	5,079
(7)	0	0	0	0	0	0	0
(8)	0	0	0	0	0	0	0
(9)	30,645	421	0	774	190	32,030	12,025
(10)	34,288	677	0	925	412	36,302	14,306
(11)	18,977	487	0	346	629	20,439	8,618
(12)	6,073	174	0	346	268	6,861	3,531
(13)	3,614	80	0	156	27	3,877	1,622
(14)	4,908	109	0	156	36	5,209	2,061

Source : Study Team

Table 6.6-4 Daily Traffic by Vehicle Type per Direction for 2020

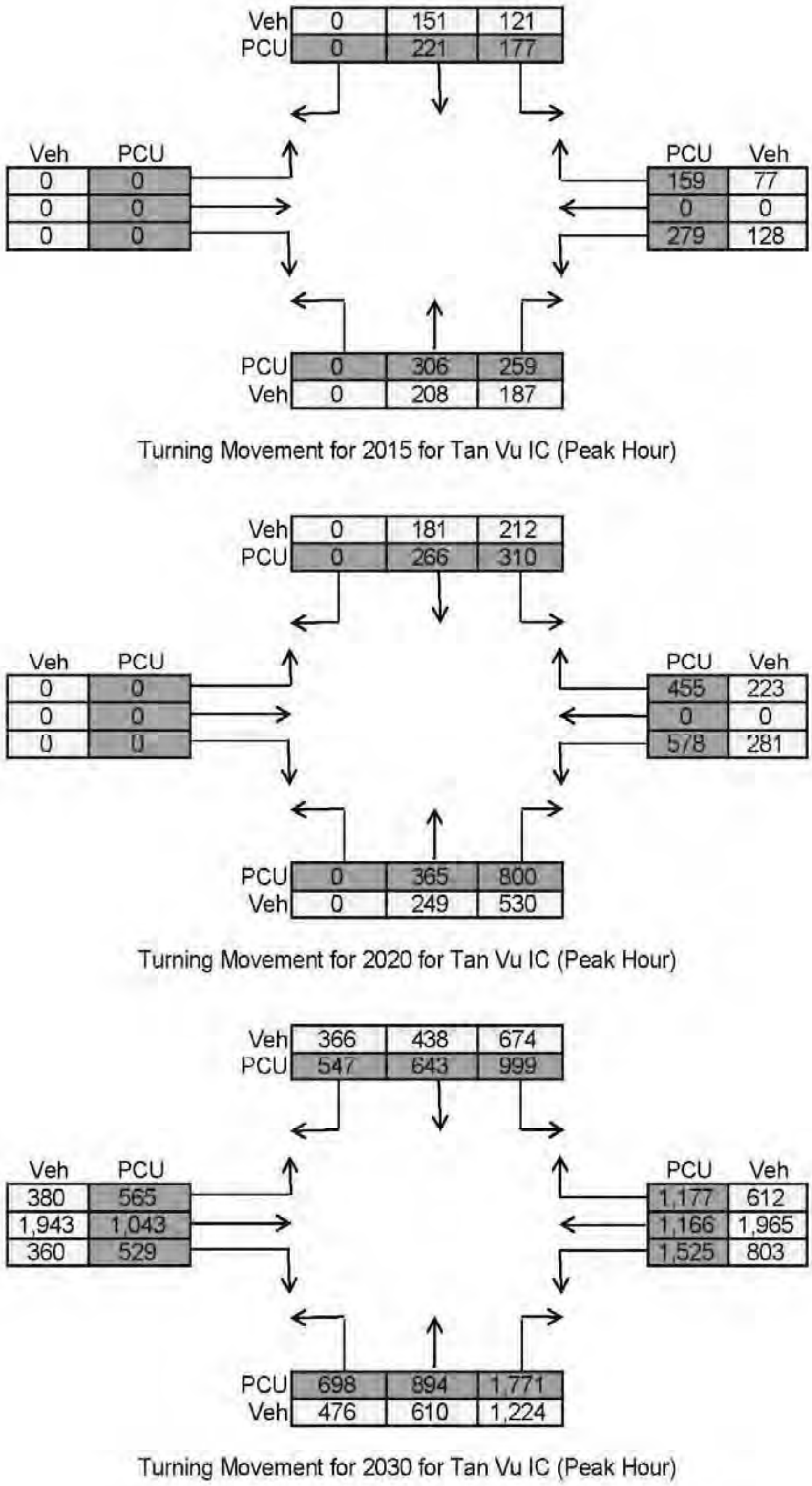
Direction	MC	Car	LGV	HGV	Bus	Total Veh	Total PCU
(1)	0	2,536	426	625	319	3,906	5,748
(2)	0	2,296	269	1,468	780	4,813	8,454
(3)	0	1,519	0	2,632	1,473	5,624	11,782
(4)	0	1,373	0	2,917	1,267	5,557	11,833
(5)	0	4,841	924	1,365	607	7,737	11,619
(6)	0	1,998	196	1,848	725	4,767	8,823
(7)	0	0	0	0	0	0	0
(8)	0	0	0	0	0	0	0
(9)	48,786	1,325	0	1,790	318	52,219	21,231
(10)	54,597	1,753	0	2,087	511	58,948	24,627
(11)	20,661	1,091	0	2,619	757	25,128	15,729
(12)	6,611	407	0	2,619	336	9,973	9,778
(13)	7,702	299	0	344	60	8,405	3,620
(14)	10,509	408	0	333	82	11,332	4,598

Source : Study Team

Table 6.6-5 Daily Traffic by Vehicle Type per Direction for 2030

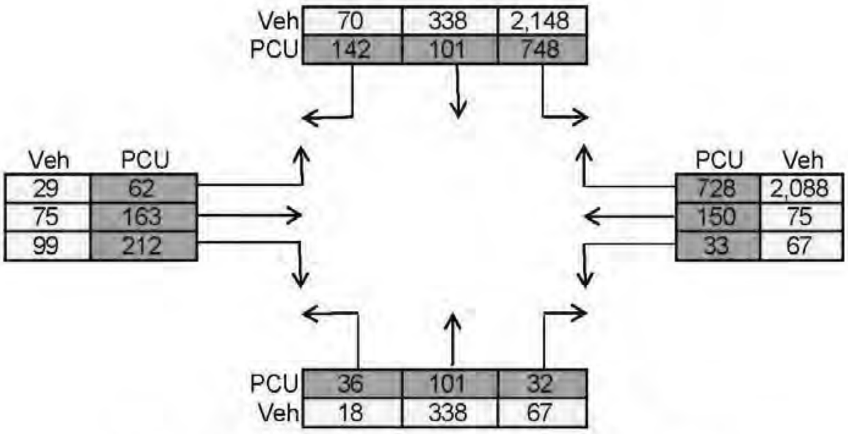
Direction	MC	Car	LGV	HGV	Bus	Total Veh	Total PCU
(1)	0	9,328	1,793	2,341	1,197	14,659	21,759
(2)	0	8,634	1,220	4,788	1,609	16,251	27,067
(3)	14,612	6,981	0	9,090	2,570	33,253	40,515
(4)	14,020	6,407	0	9,645	2,156	32,228	40,116
(5)	0	15,088	2,448	3,591	1,836	22,963	33,552
(6)	0	8,352	863	5,246	1,901	16,362	27,946
(7)	14,018	7,320	1,321	1,803	878	25,340	20,870
(8)	14,612	6,728	1,050	3,177	1,158	26,725	24,049
(9)	51,257	2,260	0	1,078	190	54,785	20,807
(10)	56,171	2,708	0	1,078	259	60,216	22,902
(11)	23,271	2,780	0	7,189	1,035	34,275	30,321
(12)	7,447	1,050	0	7,189	481	16,167	22,459
(13)	22,068	1,942	0	822	194	25,026	11,102
(14)	30,091	2,648	0	1,378	265	34,382	15,783

Source : Study Team

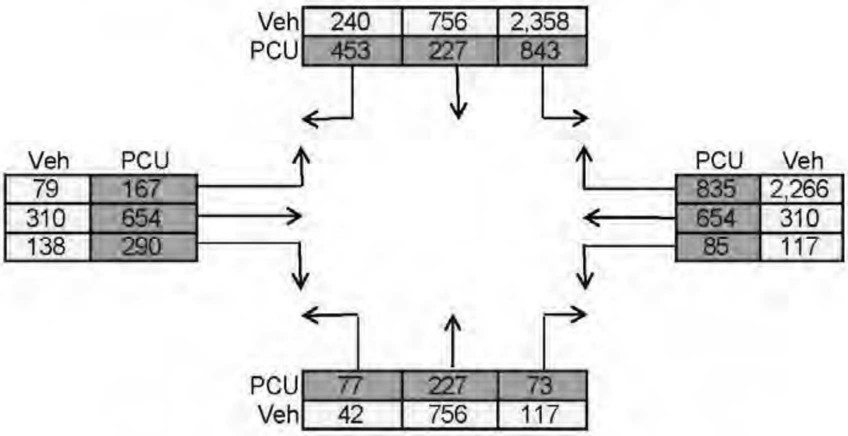


Source : Study Team

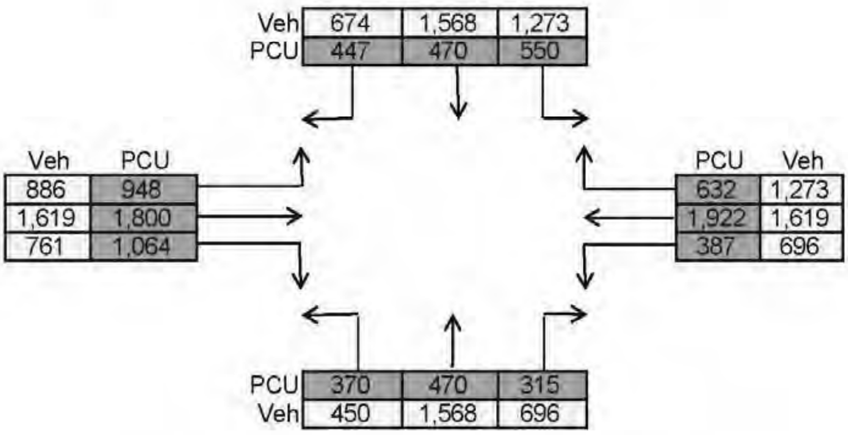
Figure 6.6-5 Peak Hour Turning Movement Traffic Flows at Tan Vu Interchange (2015-2030)



Turning Movement for 2015 for Intersection #1 (Peak Hour)



Turning Movement for 2020 for Intersection #1 (Peak Hour)



Turning Movement for 2030 for Intersection #1 (Peak Hour)

Source : Study Team

Figure 6.6-6 Peak Hour Turning Movement Traffic Flows at Intersection#1 (2015-2030)

6.7 Conclusion

Comparisons of the traffic demand forecasts for the LH Project, FS, and SAPROF are given in Tables 6.7-1 and 6.7-2. As both tables indicate, the traffic flows forecasted by this report are significantly less than that of the FS and SAPROF for the years 2015 and 2020 for the Tan Vu-Dinh Vu Section, which can be partially explained by the LH Project's assumption that Ring Road#3 will not be completed until after 2020. As for 2030, the difference between the traffic flows of this report and the FS are insubstantial (i.e., variations are from 8% to 16%), while in the case of the SAPROF they are minimal (with differences ranging from 4% to 7%).

On the other hand, in the case of the Dinh Vu-Cat Hai Section, the traffic flows forecasted by this report are significantly less than that of the FS for the years 2015 and 2020, while in 2030 the LH Project's traffic flow for the Dinh Vu-Cat Hai direction exceeds that of the FS by 1.5 times. This can be attributed to the FS's assumption that a train will be in operation between LH Port and Dinh Vu, which is not assumed by this report. In the case of the SAPROF, the trend is the same, with the traffic flows of this report being significantly greater in 2030 (i.e., from 1.46 to 1.56 times larger). In 2015, the traffic flows of the SAPROF and this report are almost the same, while in 2020 the traffic flows of this report are greater by 1.05 to 1.25 times and which can be attributed to the larger number of tourists assumed by the LH Project.

Table 6.7-1 Comparison of FS & LH Project Traffic Forecasts for Morning Peak Hour (unit: PCU)

Section	Direction	Year 2015			Year 2020			Year 2030		
		FS (A)	LH Project (B)	Change (B/A)	FS	LH Project (B)	Change (B/A)	FS	LH Project (B)	Change (B/A)
Tan Vu - Dinh Vu	To Tan Vu	2,272	438	0.19	3,789	1,112	0.29	4,624	3,868	0.84
	From Tan Vu	1,304	437	0.34	2,457	1,110	0.45	3,515	3,812	1.08
Dinh Vu - Cat Hai	To Dinh Vu	1,680	910	0.54	2,691	1,569	0.58	2,888	2,922	1.01
	From Dinh Vu	583	743	0.62	1,157	932	0.81	1,392	2,089	1.50

Note: LH Project assumes no railway to be in place by 2030 while the FS assumes the opposite.

Source : Study Team

Table 6.7-2 Comparison of SAPROF & LH Project Traffic Forecasts for Morning Peak Hour(unit: PCU)

Section	Direction	Year 2015			Year 2020			Year 2030		
		SAPROF (A)	LH Project (B)	Change (B/A)	SAPROF (A)	LH Project (B)	Change (B/A)	SAPROF (A)	LH Project (B)	Change (B/A)
Tan Vu IC - Dinh Vu	To Tan Vu	1,276	438	0.34	2,149	1,112	0.52	4,140	3,868	0.93
	From Tan Vu	745	437	0.59	1,451	1,110	0.76	3,967	3,812	0.96
Dinh Vu - Cat Hai	To Dinh Vu	927	910	0.98	1,494	1,569	1.05	2,002	2,922	1.46
	From Dinh Vu	351	743	1.03	745	932	1.25	1,350	2,089	1.55

Note: LH Project assumes no railway to be in place by 2030 while the SAPROF assumes the opposite

Source : Study Team

Required number of lanes was calculated based on Vietnamese Standard (TCVN4054-2005) and Japanese Standard.

In Vietnamese Standard, the number of lanes is determined by following formula.

$n_{lane} = N_{rush\text{-}hour} / Z \times N_{actual\text{-}capacity}$

Where:

n_{lane} : Required number of lanes

$N_{rush\text{-}hour}$: Rush-hour design traffic capacity

$N_{actual\ capacity}$: Actual capacity of through traffic flow. If there is no study and calculation, 1,800 pcu/h/lane is adopted for the highway with median separator and side separator.

Z : 0.55 for design speed $\geq$ 80 km/h

In Japanese Standard, the number of lanes is determined by following formula.

n_{lane} = Traffic Volume / Basic Traffic Capacity x Correction Factor

Where:

Traffic Volume : From traffic demand forecast (pcu/h)

Basic Traffic Capacity : 2,200 pcu/h/lane

Correction Factor: 0.75 for 47 % of large vehicles factor

Required number of lanes calculated in Vietnamese Standard and Japanese Standard is shown in Table 6.7-3.

According to the calculation based on Vietnamese Standard, 6-lane and 8-lane will be required in 2024 and 2027 for Tan Vu - Dinh Vu section, and 6-lane will be required in 2026 for Dinh Vu - Cat Hai section.

According to the calculation based on Japanese Standard, 6-lane will be required in 2029 for Tan Vu - Dinh Vu section, although 4-lane will be enough for capacity even in 2030 for Dinh Vu - Cat Hai section.

Table 6.7-3 Required Number of Lanes

Year	Tan Vu – Dinh Vu								Dinh Vu – Cat Hai							
	Traffic Volume (pcu/h)		Required Number of lanes						Traffic Volume (pcu/h)		Required Number of lanes					
	to TV	from TV	In VN Standard			In JP Standard			to DV	from DV	In VN Standard			In JP Standard		
2015	438	437	0.9	->	2	0.5	->	2	910	743	1.7	->	2	1.0	->	2
2020	1,112	1,110	2.2	->	4	1.3	->	2	1,569	932	2.5	->	4	1.5	->	2
2021	1,388	1,380	2.8	->	4	1.7	->	2	1,704	1,048	2.8	->	4	1.7	->	2
2022	1,663	1,650	3.3	->	4	2.0	->	4	1,840	1,163	3.0	->	4	1.8	->	2
2023	1,939	1,921	3.9	->	4	2.3	->	4	1,975	1,279	3.3	->	4	2.0	->	2
2024	2,214	2,191	4.4	->	6	2.7	->	4	2,110	1,395	3.5	->	4	2.1	->	4
2025	2,490	2,461	5.0	->	6	3.0	->	4	2,246	1,511	3.8	->	4	2.3	->	4
2026	2,766	2,731	5.6	->	6	3.3	->	4	2,381	1,626	4.0	->	6	2.6	->	4
2027	3,041	3,001	6.1	->	8	3.7	->	4	2,516	1,742	4.3	->	6	2.6	->	4
2028	3,317	3,272	6.7	->	8	4.0	->	4	2,651	1,858	4.6	->	6	2.7	->	4
2029	3,592	3,542	7.2	->	8	4.3	->	6	2,787	1,973	4.8	->	6	2.9	->	4
2030	3,868	3,812	7.8	->	8	4.7	->	6	2,922	2,089	5.1	->	6	3.0	->	4

Source : Study Team

CHAPTER 7 HIGHWAY DESIGN

7.1 HIGHWAY DESIGN

7.1.1 Design Standard

Based on Decision No.3139/QD-BGTVT dated 29th October 2010, following design standard is applied for the highway design;

➢Design standard: TCVN4054-2005

7.1.2 Basic Design Concept

7.1.2.1 Design Vehicle

Design vehicles shall be the prevailed vehicle type in the traffic flow used for calculation of the highway factor. Dimensions for design vehicles are specified in TCVN4054-05 (Item 3.2).

Table 7.1.2-1 Dimensions for Design Vehicles

Dimensions are in meters

Vehicle Type	Overall Length	Overall Width	Height	Front Overhang	Rear Overhang	Wheel base
Car	6	1.8	2	0.8	1.4	3.8
Truck	12	2.5	4	1.5	4	6.5
Semi-trailer	16.5	2.5	4	1.2	2	4.00 ÷ 8.80

Source : Study Team

7.1.2.2 Geometric Design

Based on Decision No.3139/QD-BGTVT dated 29th October 2010, road classification of the highway is decided as follows;

➢Design Grade: Grade III, plain terrain

➢Design speed: 80km/h

➢Total width: W=29.5m

➢Number of lane: 4 lanes

Geometric design criteria for the highway are summarized in below table;

Table 7.1.2-2 Summary of the Geometric Design Criteria for the Highway

No.	Item			Unit	Design Criteria
1	Design Speed			Km/h	80
2	Carriageway Width			m	2x3.5
3	Outer Shoulder Width (paved portion)			m	2.0
4	Outer Shoulder Width (non-paved portion)			m	0.5
5	Inner Shoulder Width			m	0.75
6	Median Width Including Inner Shoulder			m	9.0
7	Crossfall of Carriageway			%	2.0
8	Crossfall of Outer Shoulder (non-paved portion)			%	6.0
9	Maximum Superelevation			%	8.0
10	Minimum Radius			m	400
11	Minimum Length of Curve			m	220 (250<R<275) 200 (275<R<300) 170 (300<R<350) 140(350<R)
12	Minimum Length of Clothoid			m	110 (250<R<275) 100 (275<R<300) 85 (300<R<350) 70(350<R)
13	Maximum Grade			%	5
14	Maximum Length of Longitudinal Grade			m	900 (4%) 700 (5%)
15	Vertical Curves	Crest	Minimum	m	4000
			Normal	m	5000
		Sag	Minimum	m	2000
			Normal	m	3000
		Minimum Length of Curves			70
16	Stopping Sight Distance			m	100
17	Minimum Radius Without Superelevation			m	2500

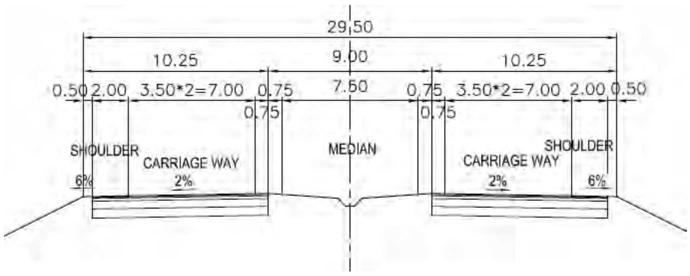
Source : Study Team

7.1.2.3 Cross Section Layout

Cross sections of the main carriageway are shown below. In Phase-2, pavement is planned to be extended toward inside to 6 lanes. Scope of this project is Phase-1 construction only.

<Phase-1>

Carriageway	2 x 3.50m both side (total 4 lanes)
Shoulder	2.50m both side
(Paved portion	2.00m both side)
(Non-paved portion	0.50m both side)
Median	9.00m (including inner shoulder)
Total width	29.50m

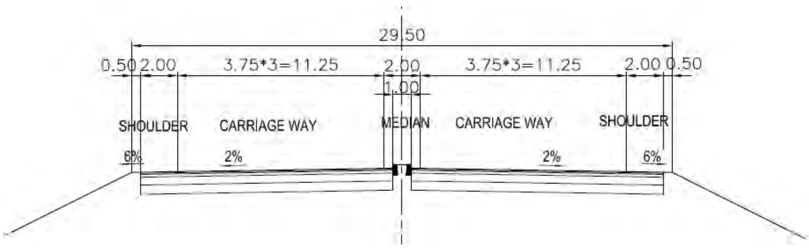


Source : Study Team

Figure 7.1.2-1 Cross Section (Phase-1)

<Phase-2>

Carriageway	2 x 3.75m both side (total 6 lanes)
Shoulder	2.50m both side
(Paved portion	2.00m both side)
(Non-paved portion	0.50m both side)
Median	2.00m
Median strip	1.00m
Median safe lane	2 x 0.50m
Total width	29.50m



Source : Study Team

Figure 7.1.2-2 Cross Section (Phase-2)

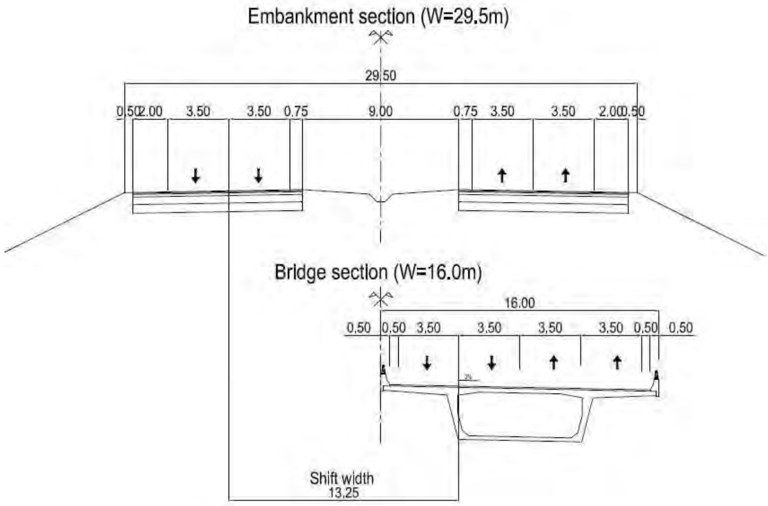
7.1.2.4 Transition section

Total width of approach bridge in Phase-1 is 16m only, while total width of embankment is 29.5m. Therefore, transition of carriage ways is necessary at the section behind both A1 and A2 abutment of approach bridge.

Length of transition section is calculated based on Vietnamese Design Standard 22TCN 237 2001 as shown below;

Length of transition: L=0.625*V*W=0.625*80km/h*13.25m=<u>662.5m</u>

Where, V: design speed (V>60km/h), W: shift width



Source : Study Team

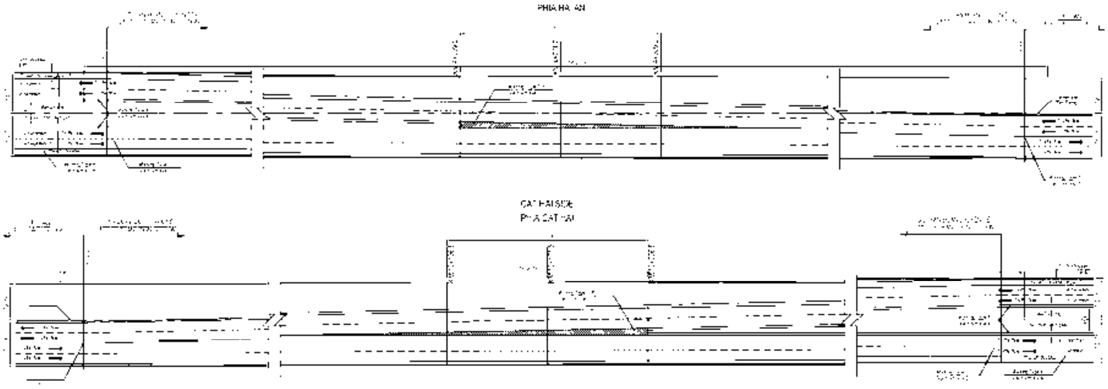
Figure 7.1.2-3 Concept of lane shift

Superelevation at north side lane (traffic flow is from Lach Huyen to Tan Vu) is changed from -2% into 2% at the transition section. Length of Superelevation runoff is 70m and the location is as follows;

(Hai An side) from Km4+239.0 to Km4+309.0, L=70m

(Cat Hai side) from Km10+136.5 to Km10+206.5, L=70m

Embankment is built with completion width (W=29.5m), while pavement construction is partially omitted in Phase-1.



Source : Study Team

Figure 7.1.2-4 Details of transition section

7.1.3 Typical Cross Section

Regarding typical cross section, the Study Team had a lot of discussion with PMU2 and DRVN during Detailed Design Stage because it would affect land acquisition area. The project site is located at fish pond area and the highway is to be built in the pond water. This is a special feature of this project.

In such a case, following matters should be considered carefully for determination of typical cross section;

- Construction of temporary access road for highway construction
- Minimizing bad effect to the surrounding area (treatment for polluted water runoff from the road surface to the ponds)
- Stability of fill slope

Detailed explanation for each item is summarized below;

(1) Construction of temporary access road

Temporary access road can be constructed inside or outside of land acquisition area of the highway. However there is no regulation for temporary compensation during construction at present in Hai Phong City, therefore, the Study Team decided to construct temporary access road inside of land acquisition area.

(2) Minimizing bad influence to the surrounding area

The Study Team proposed to provide berm with side ditch at both sides of embankment slope in order to prevent polluted water from road surface from pouring into fish ponds, shrimp ponds and salt fields. The need to prevent polluted water runoff from the road surface is described in the EIA (Environmental Impact Assessment) Report (approved on 27th May 2010) by MOT Decision 1420/QD-BGTVT). It is also required by the owner of the aquaculture ponds during the socio-economic survey carried out by the JICA Study Team in July 2011 (refer to the "Report of the Socio-Economic Survey, Tan Vu-Lach Huyen Highway Construction Project, JICA D/D Study Team, July 2011").

Replying to PMU2's request, the Study Team made a comparison to prove the necessity of our suggestion. The alternatives for the comparison are as follows;

<u>Alternative-1</u>: No side ditch (land acquisition area is decided based on F/S slope)

<u>Alternative-2</u>: Side ditch at shoulder (width of shoulder is changed from 0.5 to 1.0m)

<u>Alternative-3</u>: Side ditch at berm

<u>Alternative-4</u>: Side ditch at berm (same land acquisition area as Alt.-1, applying steel sheet pile)

Result of the comparison is shown in Table 7.1.3-1.

Alternative-3 is selected considering drainage condition, environmental impact and cost impact comprehensively. Regarding drainage condition, depth of concrete side ditch is to be more than 1m in Alternative-2 although it is designated to be less than 0.8m in the Highway Standard. Because 0% of minimum longitudinal gradient is applied in the project in order to reduce embankment height on the soft ground and bottom level of the ditch should be changed lower to ensure flow of water. On top of that, width of shoulder should be widened from 0.5m to 1.0m at both sides in order to put concrete ditch, therefore, additional filling and soft soil treatment are required and they will result in increasing construction cost.

Regarding environmental impact, polluted water from road surface cannot be treated suitably and bad influence will happen to the surrounding area such as fish pond and salt field in case of Alternative-1.

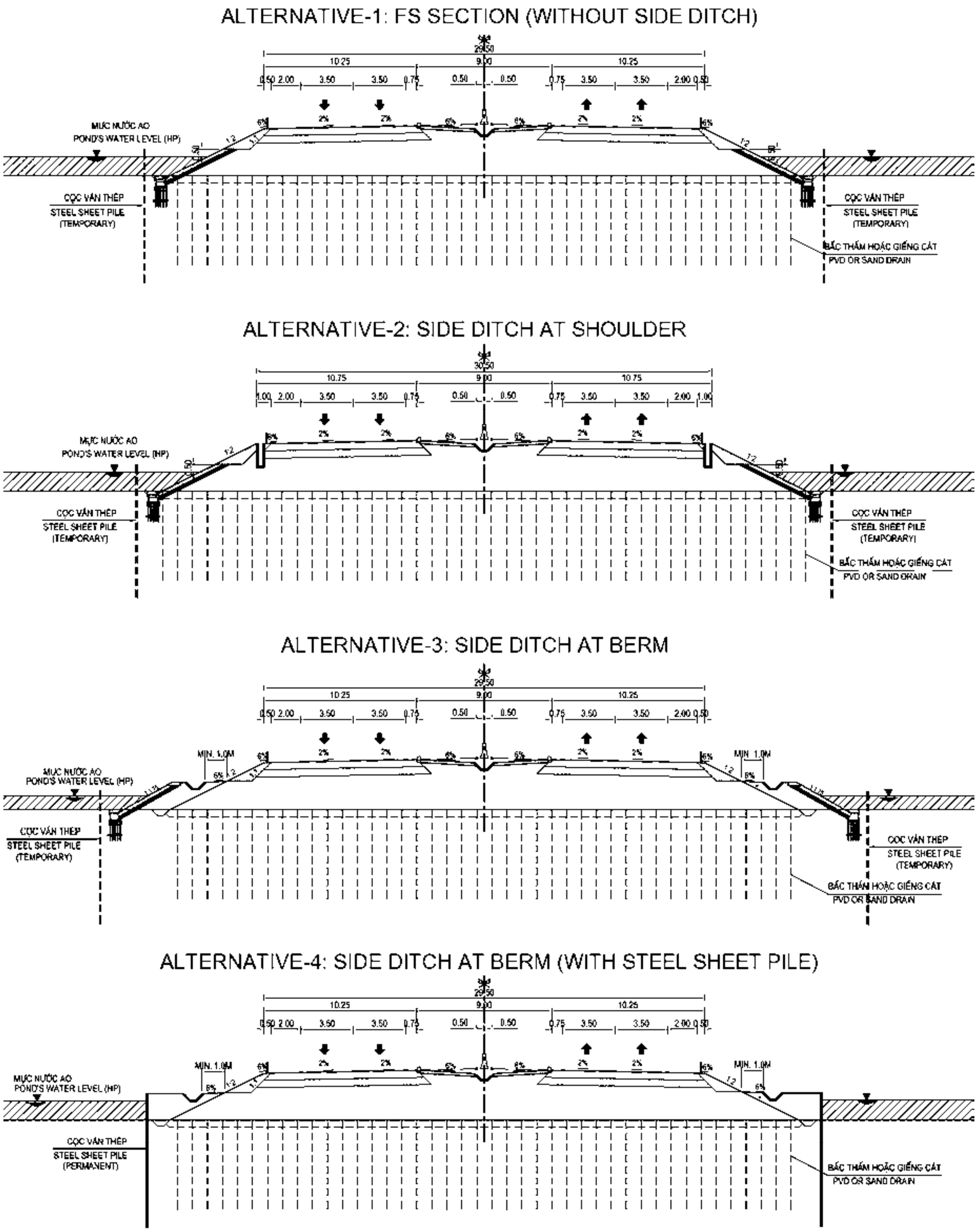
Regarding cost impact, Alternative-3 is cheaper than any other alternatives except for Alternative-1 which can be rejected due to environmental aspect.

As a result of these comparisons, Alternative-3 can be the best selection for the road cross section.

Table 7.1.3-1 Comparison table of Alternatives of Design of Road Cross Section

	Alternative-1: No side ditch (land acquisition area is decided based on F/S slope)		**Alternative-2: Side ditch at shoulder (width of shoulder is changed from 0.5 to 1.0m)**		**Alternative-3: Side ditch at berm**		**Alternative-4: Side ditch at berm (same land acquisition area as Alternative-1, applying steel sheet pile)**	
Additional land to be acquired comparing to F/S (i.e.2m from the embankment slope based on F/S design)	Hai An side	+0ha	Hai An side	+0.4ha	Hai An side	+2.7ha	Hai An side	+0ha
	Cat Hai side	+0ha	Cat Hai side	+0.6ha	Cat Hai side	+3.4ha	Cat Hai side	+0ha
	Total (ha)	+0ha	Total (ha)	+1.0ha	Total (ha)	+6.1ha	Total (ha)	+0ha
Cost for additional land acquisition to be acquired comparing to F/S	Hai An side	0	Hai An side	192,000,000	Hai An side	1,296,000,000	Hai An side	0
	Cat Hai side	0	Cat Hai side	198,000,000	Cat Hai side	1,122,000,000	Cat Hai side	0
	Total (VND)	**0**	**Total (VND)**	**390,000,000**	**Total (VND)**	**2,418,000,000**	**Total (VND)**	**0**
Cost for construction of drainage facilities and temporary works	Embankment (Berm)	0	Embankment (Berm)	0	Embankment (Berm)	21,945,600,000	Embankment (Berm)	30,723,840,000
	Embankment (Highway)	0	Embankment (Highway)	52,876,800,000	Embankment (Highway)	0	Embankment (Highway)	0
	Side Ditch	0	Side Ditch	38,506,400,000	Side Ditch	2,247,900,000	Side Ditch	2,247,900,000
	Grouted Riprap (Highway slope)	30,968,899,200	Grouted Riprap (Highway slope)	30,968,899,200	Grouted Riprap (Berm)	25,807,416,000	Grouted Riprap	0
	Additional soft soil treatment (PVD)	0	Additional soft soil treatment (PVD)	1,762,250,000	Additional soft soil treatment (PVD)	0	Additional soft soil treatment (PVD)	0
	Steel sheet pile (Temporary)	158,160,000,000	Steel sheet pile (Temporary)	158,160,000,000	Steel sheet pile (Temporary)	158,160,000,000	Steel sheet pile (Permanent)	381,000,000,000
	Total (VND)	**189,128,899,200**	**Total (VND)**	**282,274,349,200**	**Total (VND)**	**208,160,916,000**	**Total (VND)**	**413,971,740,000**
Total additional cost (VND)	**189,128,899,200**		**282,664,349,200**		**210,578,916,000**		**413,971,740,000**	
Advantage	- Land acquisition area is minimum. - Additional cost is minimum.		- Land acquisition area is reduced compared with Alternative -3.		- Poluted water from road pavement can be treated suitably. - Maintenance of ditch is easy.		- Poluted water from road pavement can be treated suitably. - Maintenance of ditch is easy.	
Disadvantage	- There is no consideration to surrounding environment		- Maximum depth of ditch becomes more than 1m in order to keep gradient for drainage (maximum depth of ditch should be 80cm (TCVN4054: 2005)). - Additional embankment and soft soil treatment are required due to widening of shoulder (change from 0.5m to 1.0m). - Maintenance of ditch and ensuring safety are not easy because of deep ditch.		- Land acquisition area is maximum.		- Cost for steel sheet pile is too expensive.	
Consultant's recommendation					Recommended			

Source : Study Team



Source : Study Team

Figure 7.1.3-1 Alternatives for comparison

(3) Stability of fill slope

➢Fill slope gradient

According to the standard, the JICA Study Team decided to apply the value of “1:2.0” as fill slope gradient of main road and “1:1.75” as that of berm with side ditch in case of using fine sand for the material of embankment.

Table 7.1.3-2 Fill slope gradient

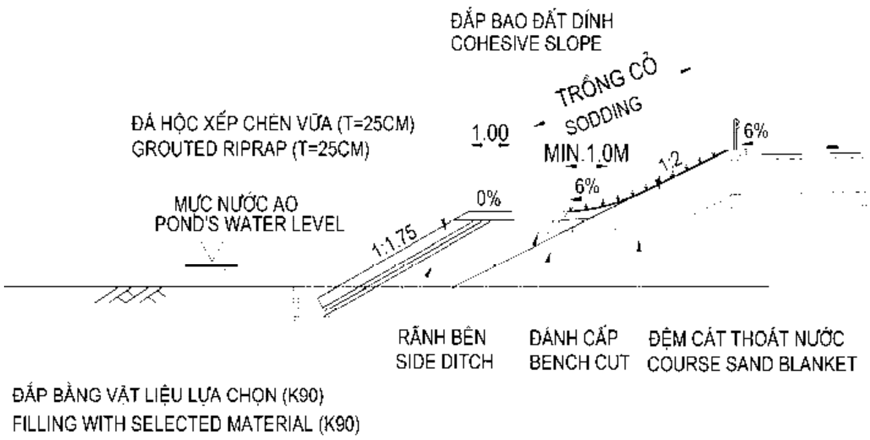
Type of soil/rock	**Slope of the embankment when height of fill slope**	
	under 6m	**from 6m to 12m**
Lightly weathered rocks	1 : 1 ÷ 1 : 1.3	1 : 1.3 ÷ 1.5
Slightly weathered rock with size more than 25cm, dry rip rap	1 : 0.75	1 : 1.0
Crushed stone, graveled stone, sand mixed with gravel, clinker.	1:1.3	1:1.3 ÷1.5
Large and medium size sand, clay, clayey sand, easily weathered rock	1:1.5	1:1.75
Silty soil, fine sand	1:1.75 ÷ 2	1:1.75 ÷2

Source : TCVN4054 2005, Table 25

➢Slope protection

(Fish pond section)

As mentioned above, berm with side ditch is provided at fish pond section in order to treat polluted water from road surface. Grouted riprap, which is popular method in Vietnam, is planned at the fill slope of berm in the pond. For the fill slope of main road, cohesive soil and sodding are planned. Geotextile is used for ensuring slope stability of embankment during soft soil treatment.

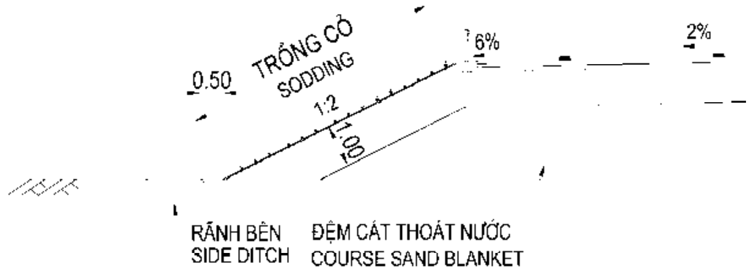


Source : Study Team

Figure 7.1.3-2 Slope protection at fish pond section

(Residential area and salt field in Cat Hai)

Cohesive slope (thickness=1.0m) with sodding is planned at the fill slope.

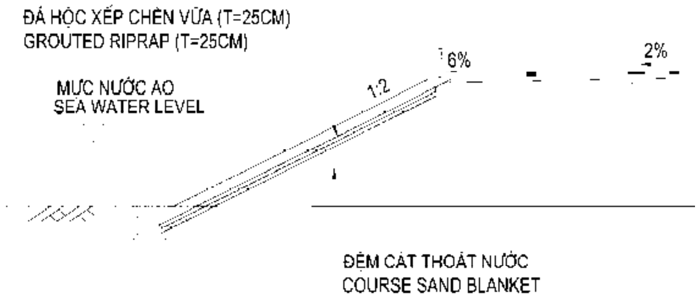


Source : Study Team

Figure 7.1.3-3 Slope protection at Residential area and salt field in Cat Hai

(Sea and dike relocation section)

Concrete panel is planned in order to protect the slope from erosion by tide and wave.

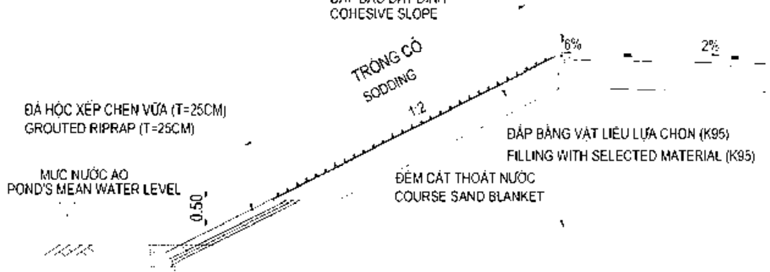


Source : Study Team

Figure 7.1.3-4 Slope protection at sea and dike relocation section

(Northern side of superelevation section)

No side ditch is provided at northern side of superelevation section because treatment of water from road surface is not necessary. Grouted riprap is planned in fish pond only up to 50cm above mean water level, because there is little affection by tide and wave of pond. Cohesive soil and sodding is planned at upper part of fill slope as shown in below figure.

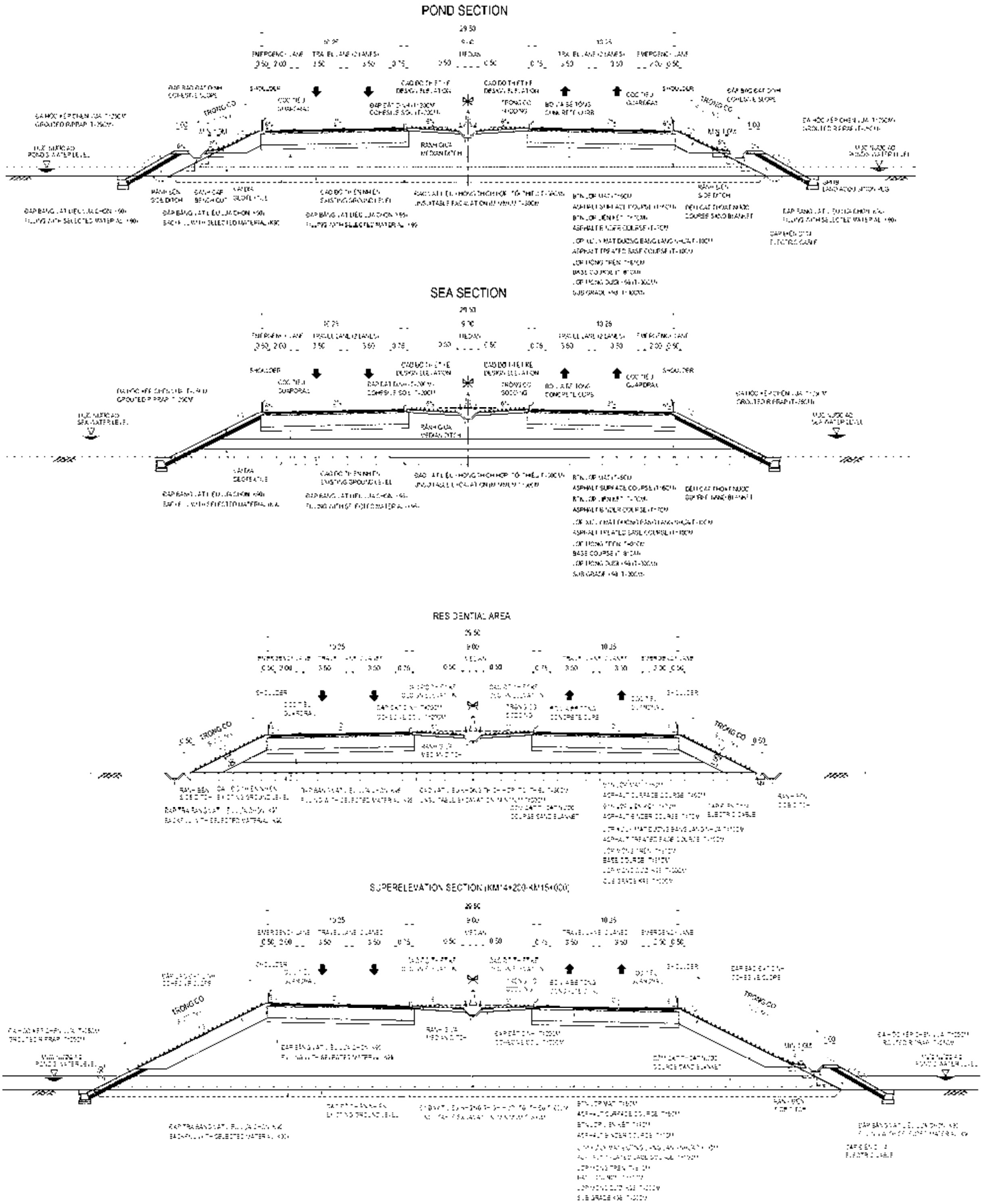


Source : Study Team

Figure 7.1.3-5 Slope protection at Northern side of superelevation section

(4) Typical cross section drawings

After studying the above mentioned matters, the Study Team finalized the typical cross section for each section as shown in Figure 7.1.3-6.



Source : Study Team

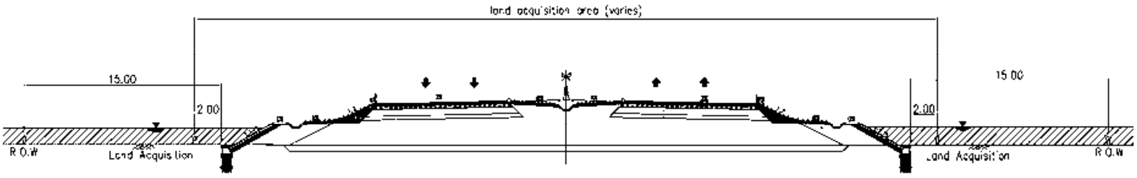
Figure 7.1.3-6 Typical cross section

7.1.4 Land acquisition and ROW

Decree No.11/2010/ND-CP issued by the Government of Vietnam dated 24th February 2010 is applied for land acquisition of the project. According to the Decree, PMU2 decided the land acquisition area for the project as shown below;

- Land acquisition area: 2m from the edge of riprap foundation
- ROW: 15m from the end of fill slope

Note) Area between land acquisition and ROW (13m width) will not be acquired by the project owner.



Source : Study Team

Figure 7.1.4-1 Concept for land acquisition area and ROW

7.1.5 Horizontal and Longitudinal Alignment

7.1.5.1 Horizontal Alignment

(1) General Design Policy

Tan Vu-Lach Huyen Highway has straight lines, circles and clothoid curves, which are to be used as transition curves. The general policy for horizontal alignment design is as follows;

➢The alignment should be suitable to the topography.

➢The alignment should be continuous with no rapid changes.

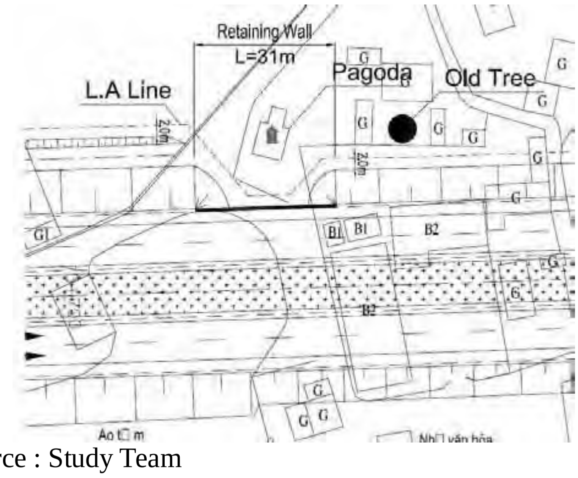
➢Sufficient curve length should be maintained to prevent an illusion in which the curve looks less sharp than it actually is.

(2) Review of F/S Alignment

F/S horizontal alignment which length is 15.630km was approved by MOT by his letter No. 3139/QD-BGTVT dated 29 October 2010. In this Study, the F/S alignment was reviewed especially considering social impact along the alignment. As a result, almost all sections were appropriate except for one location in Cat Hai Island. Following location was studied carefully;

➢Km13+700-Km14+000: where the alignment runs through Thon Trung Village, Dong Bai Commune.

Loss of residential lands of more than 35 households will be caused in this village. This also cause affect to an office building of Dong Bai Commune's People Committee and pagoda nearby. Therefore, a meeting was held on 08 April 2011 at Dong Bai Commune's People Committee and an interview was carried out among stakeholders. As a result of the meeting, we could confirm that the affected persons had already reached agreement to be resettled and the office building could be demolished. In response to the results of the meeting, we decided to keep the alignment unchanged. However, the pagoda land will be inside the land acquisition area in case of normal fill slope. Therefore, we propose to build a retaining wall in order to avoid the pagoda.



Source : Study Team

Photo of Pagoda

Figure 7.1.5-1 Plan and Photo at Thon Trung Village, Dong Bai Commune

(3) Alignment Elements

Alignment elements and coordinates of IP are shown below table;

Table 7.1.5-1 Alignment Elements and Coordinates of IP

Alignment elements

No.	STA	X	Y	Start Radius	A	End Radius	Length	Direction	Type
1	0+000	2301375.271	604148.960				2401.5	82-10-15.37	Tangent
2	2+401.50	2301702.399	606528.073		916.515	6000	140.0	82-10-15.37	Clothoid
3	2+541.50	2301720.929	606666.840	6000		6000	482.0	82-50-21.79	Circular curve
4	3+023.49	2301761.747	607146.972	6000	916.515		140.0	87-26-31.49	Clothoid
5	3+163.49	2301766.907	607286.876				10960.0	88-6-37.91	Tangent
6	14+123.48	2302128.275	618240.907		409.878	1200	140.0	88-6-37.91	Clothoid
7	14+263.48	2302130.169	618380.873	1200		1200	1144.5	91-27-10.02	Circular curve
8	15+407.96	2301599.893	619346.435	1200	409.878		140.0	146-5-50.90	Clothoid
9	15+547.96	2301480.764	619419.936				81.98	149-26-23.02	Tangent
10	15+629.94	2301410.172	619461.618						

IP Coordinate

No	STA	X	Y
BP	0+000	2301375.271	604148.960
IP1		2301754.336	606905.801
IP2		2302154.055	619022.383
EP	15+629.94	2301410.172	619461.618

Source : Study Team

7.1.5.2 Longitudinal Alignment

(1) Basic concept and control points for longitudinal alignment design

Basic concept for longitudinal alignment design was determined as follows in SAPROF report in 2010.

➢Design elevation of approach road in Hai An is equal to the planned formation of Dinh Vu Industrial Zone.

➢Design elevation for bridge section is decided to ensure vertical clearances of future railways and navigation clearance for Bach Dang River.

➢Design elevation of approach road in Cat Hai is equal to the planned elevation of Lach Huyen Port.

➢Minimum vertical gradient (0%) is applied both road and bridge section.

In addition to the above concept, follows are added in Detailed Design Stage;

➢Vertical alignment is set as lower as possible in order to minimize filling volume.

➢Design elevation of approach road in both Hai An and Cat Hai is determined considering design high water level (HWL), which is updated based on hydrological survey result in Detailed Design Stage, to ensure the control levels of hydraulic frequency (P=1%).

➢The elevation of shoulder edge is set 0.5 or 0.6m higher than the flooded water level (P=1%). According to Highway Design Standard, Chapter 7.3.2 in TCVN 4054: 2005, minimum 0.5m must be kept at the edge of shoulder. Therefore, 0.5 or 0.6m is applied for safety which satisfies the Standard.

➢Minimum vertical gradient (0.3%) is applied for bridge section in order to drain surface water properly.

Main control points are also adjusted based on updated HWL. Following table shows comparison of control points between SAPROF and Detailed Design.

The main difference is 15cm higher than SAPROF in Hai An section.

Table 7.1.5-2 Comparative Main Control Points for Longitudinal Design

	SAPROF	D/D
Hai An Section	Lowest design elevation---3.44m Planned formation of Dinh Vu Industrial Zone 5.0-1.90=3.10m 3.10+0.335(Transverse gradient)=3.44m	Lowest design elevation ---3.79m Hydraulic frequency (P=1%)=2.98m 2.98+0.5(Distance from HWL1% at the edge of shoulder)+0.305(Transverse gradient) =3.79m
Bridge section	[Approach Bridge] Vertical clearance for future railways---9.65m 3.10+6.55=9.65m (Clearance for railways is higher than that for road at Intersection No.2, 3.) [Main Bridge] 3.86m---HWL for Bach Dang River 2.45(HWL:5%)+1.41(wave height)=3.86m	[Approach Bridge] Vertical clearance for future railways---9.65m (No change) [Main Bridge] 3.96m---HWL for Bach Dang River 2.55(HWL:5%)+1.41(wave height)=3.96m
Cat Hai section	Lowest design elevation---3.60m Planned formation of Lach Huyen Port was 3.60m	Lowest design elevation---3.63m Hydraulic frequency (P=1%)=2.72m 2.72+0.6(Distance from HWL1% at the edge of shoulder)+0.305(Transverse gradient) =3.63m

Source : Study Team

(2) Other Control Points for Longitudinal alignment design

In addition, some control points are added in Detailed Design as follows;

- Installation of box culverts for local approach road in Cat Hai
- Installation of box culverts for irrigation and drainage
- Design elevation of future dike in Cat Hai (H=4.5m)
- Planning height of Hanoi-Hai Phong Express way at Tan Vu IC
- Free board for Cam River Bridge

Details of these control points are summarized in below table;

Table 7.1.5-3 Summary of additional control points for longitudinal alignment design

	SAPROF	D/D
Underpass box culverts	1)Km10+128(4.0 x 3.2) FH of road in box=1.6m 2)Km13+600(4.0 x 3.2) FH of road in box=0.8m *Thickness of earth covering on the box is 60cm only.	1) Km10+414(4.0 x 3.2) FH of underpass box (center position)=1.54m ---Location has been changed by request from local people. 2)Km13+600(4.0 x 3.2) FH of underpass box (center position)=1.65m *Distance between bottom surface of median ditch and top of culvert should be kept about 0.3m.
Irrigation/ Drainage box culverts	-No control (Because all boxes are lower than lowest design elevation for each section.)	Km 15+100 (3 x 4.0 x 6.0) Height of box is changed from 4.0m to 6.0m based on request from people's committee in Cat Hai.
Dike relocation section	-	Lowest design elevation at Km9+907-Km12+013---4.82m 4.50(Design elevation of dike)+0.305(Transverse gradient) =4.81m
Hanoi-Haiphong Exp.	5.276m---Design elevation at Km0	4.765m--- Design elevation at Km0
Cam River Bridge (Km1+665.5-Km1+734.5: L=69m)	-	Required lowest design elevation at Km1+657 and Km1+743---4.680m 2.50m(HWL1%)+0.5m(free board)+1.30(Height of girder)+0.075m(pavement)+0.305m(transverse gradient)=4.680m

Source : Study Team

From the above table, design elevation of road is a little higher than SAPROF. The reasons are as follows;

a) Change of location of underpass box culvert at Km10+414

Location is shifted 286m to the east based on request from local people. Therefore design elevation becomes higher, because distance between the culvert and bridge abutment is longer than that of SAPROF.

b) Change of design elevation of local approach road at underpass box culvert at Km13+600

Design elevation of local approach road becomes about 0.9m higher than that of SAPROF based on updated survey result. This point is one of the crest points for longitudinal alignment design.

c) Change of dimension of irrigation box culvert at Km15+100

Height of box culvert is changed from 4.0m to 6.0m due to request from Cat Hai People's Committee. This point is one of the crest points for longitudinal alignment design.

d) Dike relocation section Km9+907 - Km11+880

Lowest design elevation of highway is set to be the same as design elevation of future dike (H=4.5m) according to request from Department of Agriculture and Rural Development of Hai Phong City.

e) Cam River Bridge at Km1+700

In order to ensure free board(0.5m) from HWL(1%), the longitudinal alignment is shifted upper.

7.1.6 Local Approach Road Design

7.1.6.1 Basic Design Policy

Existing roads, which cannot be connected to the highway after completion of the construction, shall be compensated to ensure the existing access. An Underpass box culvert shall be constructed in order to cross the highway. In case of applying underpass box culvert, vertical alignment of the highway becomes high and height of embankment becomes high as well. It is necessary to design frontage road to connect the existing roads instead of box culvert and minimize the number of box culvert. Location of box culvert should be decided considering request from local people well.

7.1.6.2 Design Concept for Underpass Box Culvert

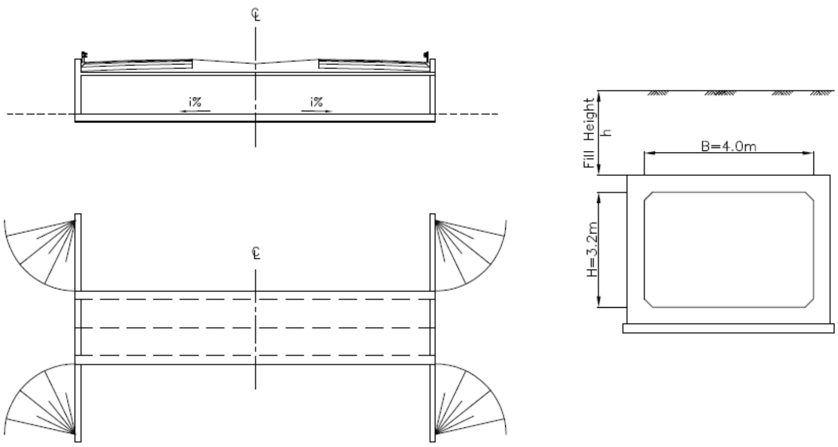
According to the Standards, vertical clearance for local approach road is defined as follows;

Table 7.1.6-1 vertical clearance for local approach road

4.75m	Apply for Highway class I, II, III	(Item4.10.2-TCVN4054-2005)
4.5m	Apply for Highway class IV, V, VI	(Item4.10.2-TCVN4054-2005)
3.2m	Road for motorized vehicles	(Item7.10.1-TCVN5729-97)
2.7m	Road for motorbike, bicycle and pedestrian	(Item7.10.1-TCVN5729-97)
2.5m	Apply to each minimum horizontal length of 4.0 m in underpass with only pedestrian, bicycle and other non-motorized vehicles.	(Item4.8-TCVN5729-97)

Source : Study Team

The Study Team applies 3.2m clearance for all local approach roads based on existing road condition. Size of underpass box culver is B4.0m x H3.2m uniformly and agreement between local authorities and the Study Team has been made.



Source : Study Team

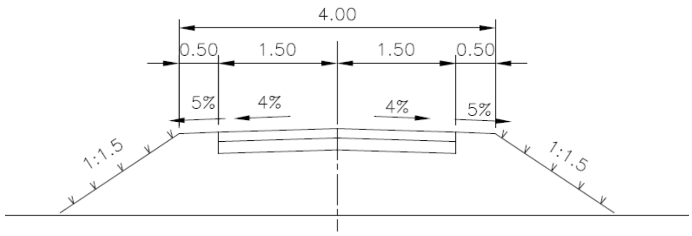
Figure 7.1.6-1 General Layout of Underpass Box Culvert (4.0x3.2)

7.1.6.3 Design Concept for frontage road

According to the highway Standard, frontage road should be arranged at both sides along the highway class I and II. Tan Vu-Lach Huyen Highway is classified as class III, therefore, it is basically not necessary to design frontage road throughout the alignment except for sections where local people requested.

Roadbed width is defined as 4.0m (one-way road) as same as opening of underpass box culvert and cross section is shown below figure. Rural road design standard, 22TCN210-92, shall be applied in Detailed Design stage.

Frontage road will connect to the existing road which does not satisfy geometrical factor of the Standard. Therefore, JST designed the frontage roads considering the existing road condition. In addition, there is a master plan of Cat Hai Island and these roads shall be reconstructed in the future based on the master plan. However, the master plan has not approved yet at the moment, therefore, JST designed the frontage roads as same as existing road condition.



Source : Study Team

Figure 7.1.6-2 Typical Cross Section of Frontage Road

7.1.6.4 Hai An Side

The alignment almost runs through fish/shrimp ponds which are owned by local people. The Study Team carried out site visit and found that there was no existing road which public vehicles could run, though there were some private roads and narrow paths at the edge of pond. Moreover, this area is located in future industrial zone site, therefore, the Study Team decided not to apply any underpass box culvert and frontage road at Hai An side in order to keep elevation of the highway as same as future industrial zone embankment.

The Study Team had a meeting with Hai An District People’s Committee and our proposal was accepted by local authorities.

Narrow paths at the edge of pond

Private road owned by company

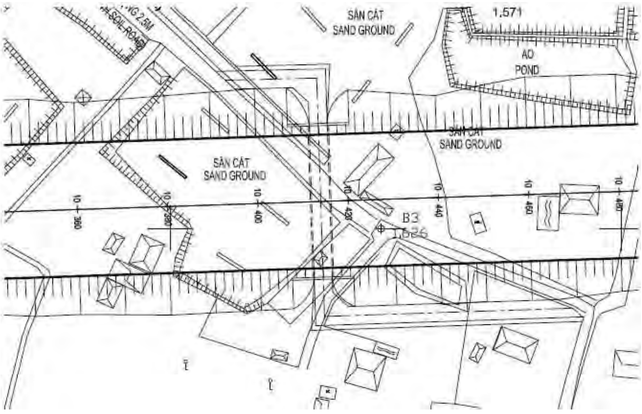
Source : Study Team

Figure 7.1.6-3 Photos in Hai An district

7.1.6.5 Cat Hai side

Two underpass box culverts for local approach road are designed in Cat Hai at following location;

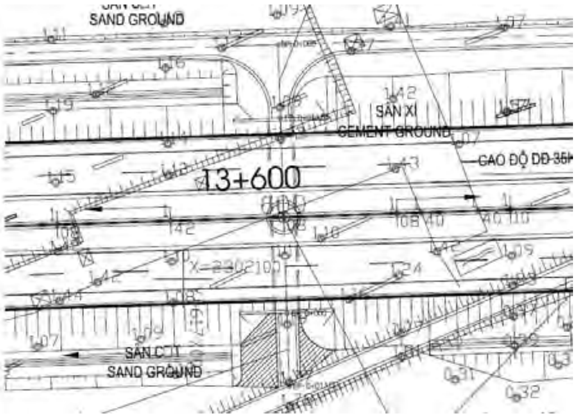
(1) Underpass at Km10+414



Concrete paved road (w=2.1m)

Figure 7.1.6-4 Plan and photo at Km10+414

(2) Underpass at Km13+600



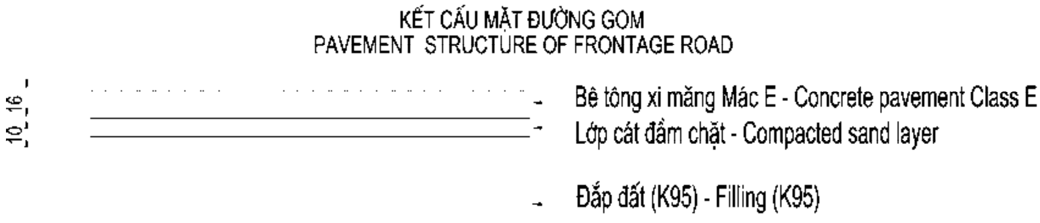
Concrete Paved Road to Dong Bai Commune (w=3.0m)

Source : Study Team

Figure 7.1.6-5 Plan and photo at Km13+600

7.1.6.6 Pavement structure

Concrete pavement is applied as shown in Figure 7.1.6-6 in order to make consistent with the existing road condition.



Source : Study Team

Figure 7.1.6-6 Pavement structure of frontage road

7.1.6.7 Box culvert for dike road relocation

Three underpass box culverts are designed for dike road relocation in Cat Hai based on a meeting on 29th June 2011 with Department of Agriculture and Rural Development of Hai Phong City which is in charge of management of the dike.

- Km10+075, B4.0mxH3.2m
- Km14+651.5, B4.0mxH3.2m
- Km15+340, B4.0mxH3.2m

7.1.6.8 Summary of underpass box culvert

Summary of underpass box culvert is shown in below table.

Table 7.1.6-2 Summary of underpass box culvert

No	Location			Size			Length (m)	Invert Elevation (m)			Skew (Degree)	Remarks
	Km..	+	.....	B		H		Left	Center	Right		
1	Km10	+	75.00	4.0	x	3.2	35.480	1.854	2.209	1.854	90.00	Relocation of dike road
2	Km10	+	414.00	4.0	x	3.2	34.085	1.277	1.540	1.788	90.00	
3	Km13	+	600.00	4.0	x	3.2	34.255	1.599	1.650	1.701	90.00	
4	Km14	+	651.50	4.0	x	3.2	35.215	0.635	0.121	-0.421	90.00	Relocation of dike road
5	Km15	+	340.00	4.0	x	3.2	35.370	0.436	0.491	0.440	90.00	Relocation of dike road

Source : Study Team

7.1.6.9 Summary of frontage road

Summary of frontage road is shown in below table.

Table 7.1.6-3 Summary of frontage road

No	Location			L - R	Width (m)	Length (m)
	Km..	-	Km..			
1	Km10+047	-	Km10+075	L	4.0	39.2
2	Km10	+	75.00	R	4.0	25.2
3	Km10+391	-	Km10+414	L	4.0	36.0
4	Km10+414	-	Km10+471	R	4.0	70.0
5	Km13+302	-	Km13+731	L	4.0	428.7
6	Km13	+	600.00	L	4.0	13.0
7	Km13	+	600.00	R	4.0	13.0
8	Km14	+	651.50	L	4.0	30.4
9	Km14+651.5	-	Km14+685	R	4.0	45.4
10	Km15	+	340.00	L	4.0	44.7
11	Km15	+	340.00	R	4.0	44.8

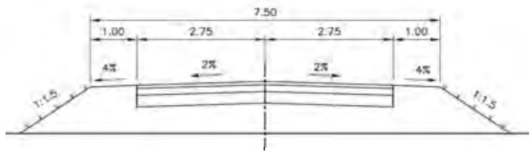
7.1.6.10 Additional Intersections in Cat Hai Island

In SAPROF study, there was no plan of intersection in Cat Hai Island. It was impossible for local people to use the highway even after completion of the highway construction. In order to increase the efficiency of the highway, the Study Team had a meeting and discussed about adding intersection with local authority. Replying to a request from local people, following two intersections with local approach road are added in Basic Design.

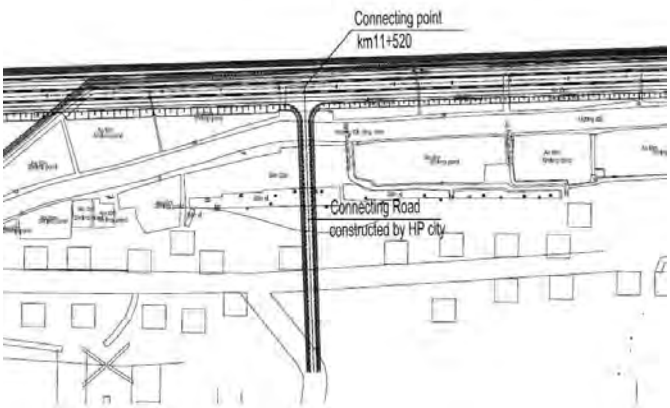
(1) At-grade Intersection at Km11+520

Local authority requested to add a connecting road which will be located between the highway and District Road 356 at right side of Km11+520. Classification of the road is defined as follows based on Highway Standards;

- ➢Design Grade: Grade V, plain terrain
- ➢Design speed: 40km/h
- ➢Width of roadbed: W=7.50m
- ➢Number of lane: 2 lanes



Plan of the intersection and the connecting road is shown in Figure 7.2.6-6. As a result of discussion with local authorities, it was confirmed that design and construction of connecting road would be done by Hai Phong City and the Study Team would carry out design of intersection only within ROW of the highway.



Plan of intersection at Km11+520 Photo at future intersection point

Source : Study Team

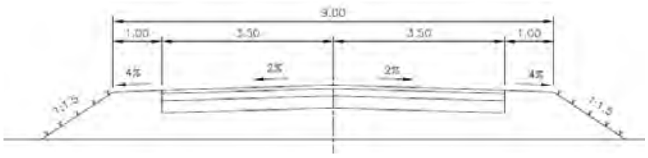
Figure 7.1.6-7 Plan and Photo of Intersection at Km11+520

(2) At-grade Intersection at Km15+576

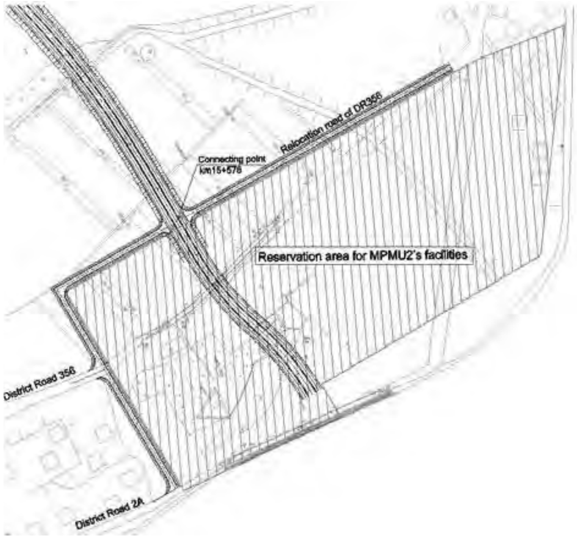
There is a reservation area for MPMU2's facilities at the end point of the highway as shown in Figure 7.2.6-7. Two existing roads, District Road 356 and 2A, will be occupied in the reservation area, therefore, relocation of the roads is required.

Local authority requested to consider a relocation of District Road 356 road which connects to both District Road 356 and 2A and an at-grade intersection is added at Km15+576. Classification of the relocated road is defined as follows based on Highway Standards;

- Design Grade: Grade IV, plain terrain
- Design speed: 60km/h
- Width of roadbed: W=9.0m
- Number of lane: 2 lanes



As a result of discussion with local authorities, it was confirmed that design and construction of the relocated road would be done by Hai Phong City and the Study Team would carry out design of intersection only within ROW of the highway.



District Road 356

District Road 2A

Source : Study Team

Figure 7.1.6-8 Plan and Photos of Intersection and Relocated Road at Km15+576

(3) Design criteria for local approach road

Design criteria for local approach road which will be constructed by Hai Phong City is shown in Table 7.1.6.4.

Table 7.1.6-4 Design criteria for local approach road

No	Standard	Unit	Criteria
1	Design standard		**TCVN 4054 - 2005**
2	Design grade		Grade V, plain terrain
3	Design speed	km/h	40
4	Minimum horizontal curve radius	m	R_{min} = 60
5	Maximum longitudinal gradient	%	i_{dmax} =7%
6	Minimum length of grade change (or: Minimum length for change of grade)	m	120
7	Minimum vertical curve radius (crest)	m	$R_{crest-min}$ = 1000 (700)
8	Minimum vertical curve radius (sag)	m	$R_{sag-min}$ = 700 (450)
9	Minimum length of vertical curve	m	35
10	Stopping sight distance	m	S_2 =80
11	Required elastic modulus	MPa	$E_{y/c}$ = 80
12	Design load for bridge, culvert design		H30-XB80
13	Design frequency		
14	- Road, culvert, small bridge pavements	%	4

Source : Study Team

7.2 Pavement Design

7.2.1 Design Condition

7.2.1.1 Pavement Design Specification

Vietnamese Standard for Pavement Design, 22TCN 211-06 (hereinafter "the specification") was proposed to be used in the design of Pavement structure in this Project. (Pavement designs in F/S and SAPROF stage were calculated based on 22TCN274-01 and AASHTO.)

7.2.1.2 Road Condition

Project Road is classified to "Public Highway" with 6-lane. Pavement type is Flexible Pavement. (Rigid Pavement is no longer utilized because of no plan to install Toll-gate.)

7.2.1.3 Design Values

Summary of Design Values with reference are as following. Basically values of Vehicle load are referred to Chapter 6 Traffic Demand Forecast, and other values are referred to the specification 22 TCN 211-06.

Table 7.2.1-1 Summary of Design Values

Design Input Requirements			Value	Reference (22TCN211-06)
1	Vehicle Load	Traffic Volume(Vehicles/day) in 2030	Truck=18,735 Bus=4,726	Refer Chapter 6 (in this Report)
		Traffic annual growth rate(%): 2015=>2020 2020=>2030	10.35% 7.60%	
		Design Period (years)	15 (2015-2030)	
		Conversion Coefficient from 6-lane to 1-lane, f_l	0.30	Clause 3.3.2.
		Standard calculation axle load, P_{tt}(kN)	120.0	Table 3.1
		Calculation pressure on pavement, p(Mpa)	0.60	Table 3.1
		Diameter of wheel track, D(cm)	36.0	Table 3.1
		Others (C1, C2, Pi) for each vehicle types	See Table D4-2, Clause 7.2.3	Table A-2
2	Material Properties	For each materials, *Elastic Modulus, E (Mpa) *Flexural tensile strength, R_{ku} (Mpa) *Friction angle, φ (degree) *Cohesive force, C(Mpa)	See Table D6-1, Clause 7.2.3	For As, Base: Table C-1 For Embankment: Table B-3
3	Others	Other necessary values for design are described in Clause 7.2.3, design sheet.		

Source : Study Team

7.2.2 Design Result

Following Pavement Structure is proposed based on the pavement calculation result by Design Sheet (refer clause 7.2.3).

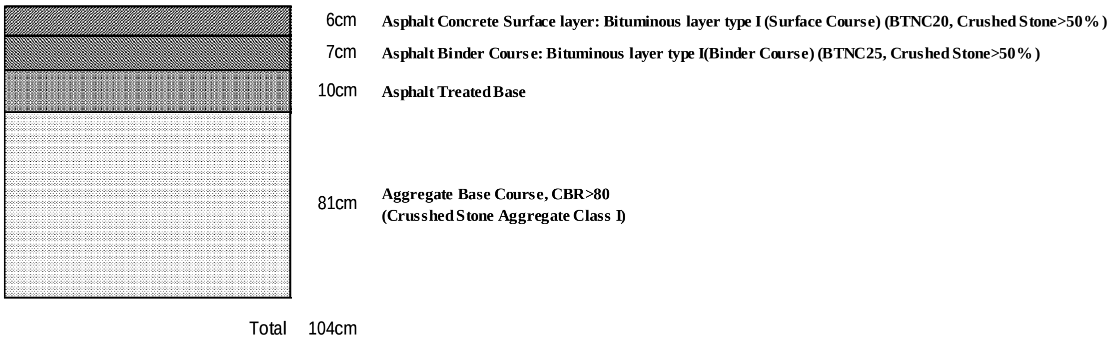


Figure 7.2-1 Pavement Structure

7.2.3 Design Sheet

Pavement Design is shown from next page. The Design was done by “Design Sheet” which is programmed by Excel based on 22TCN 211-06.

Pavement Design Sheet
for Detailed Design Study Lach Huyen Infrastructure Construction Project in Vietnam

Note: *Table No. and Figure No. is following 22TCN211-06.

D-1. Design Standard:
22TCN 211-06

D-2. Road Category:
Highway (2015 ~ 2030, 15years)

- Characteristics of Standard calculation axle load are as follwing

Table 3.1: Characteristics of Standard calculation axle load

Standard calculation axle load, P_{tt}(kN)	Calculation pressure on pavement, p(Mpa)	Diameter of wheel track, D (cm)
100	0.6	33
120	0.6	36

- From above table,

P_{tt}= 120 **kN**
p= 0.6 **Mpa**
D= 36 **cm**

D-3. Number of Lane:
6-lane -> f_l= 0.30 (f1; coefficient from 6-lane to 1-lane)

D-4. Calculation of Vehicle Load
*From Capter2 (Direction 3+4, Dailey Vehicle)

Table D4-1 Traffic Volume for each Vehicle type

Section: Tan Vu-Dinh Vu	MC	Car	Truck	Bus
2015	0	959	1,360	1,891
2020	0	2,892	5,549	2,740
2030	28,632	13,388	18,735	4,726

(Average annual growth rate 2015-2020) = 10.35 %
(Average annual growth rate 2020-2030) = 7.60 %

Types and Number of Trucks & Buses are spporsed as following;

		%	Number
Bus	(Total)		4,726
	- Small Bus	90.0%	4,253
	- Large Bus	10.0%	473
Truck			
	(Total)		18,735
	- Light	30.0%	5,621
	- Medium	20.0%	3,747
	- Heavy	20.0%	3,747
	- Heavy	15.0%	2,810
	- Heavy	15.0%	2,810

- Calculation of a number of axles converted into a number of standard axles (Final year of design period)

Table D4-2 Calculation of Total Number of acles, N (2030)

Type of Vehicle		P_i(kN)	C1	C2	ni	$C_1 * C_2 * n_i * (Pi/P_{tt})^{4.4}$
Small Bus	Front	26.4	1	6.4	4,253	35
	Rear	45.2	1	1	4,253	58
Large Bus	Front	56.0	1	6.4	473	106
	Rear	95.8	1	1	473	175
Light Truck	Front	18.0	0	6.4	5,621	0
	Rear	56.0	1	1	5,621	197
Medium Truck	Front	25.8	1	6.4	3,747	28
	Rear	69.6	1	1	3,747	341
Heavy Truck	Front	48.2	1	6.4	3,747	433
	Rear	100.0	1	1	3,747	1,680
Heavy Truck	Front	45.4	1	6.4	2,810	250
	Rear	90.0	2.2	1	2,810	1,744
Heavy Truck	Front	23.1	0	6.4	2,810	0
	Rear	73.2	2	1	2,810	639
Total					N=	5,684

<-(3-1)

*Ptt = 120 kN

*Values of Pi, C1, C2 are determined by refering Table E-1

Ntt= f1 *N = 0.30 * 5,684 (3-3)

= 1,705 vehicles/day (2030)

- To calculate number of standard axle accumulated in the period of 15 years

$$N_e = \frac{[(1+q)^t - 1]}{q(1+q)^{t-1}} *365*N_{tt} \text{ (axles)} \qquad \text{(A-3)}$$

t1=	5	(years),	q1=	0.1035	(2015-2020)
t2=	10	(years),	q2=	0.0760	(2020-2030)
Ntt1=	537	(vehicles/day), (See below calculation)			(2015-2020)
Ntt2=	1,705	(vehicles/day), (See previous page)			(2020-2030)

Table D4-3 Calculation of Total Number of acles, N (2020)

Type of Vehicle		P_i(kN)	C1	C2	ni	$C_1*C_2*n_i*(Pi/P_{tt})^{4.4}$ <-(3-1)
Small Bus	Front	26.4	1.0	6.4	2,466	20
	Rear	45.2	1.0	1.0	2,466	34
Large Bus	Front	56.0	1.0	6.4	274	61
	Rear	95.8	1.0	1.0	274	102
Light Truck	Front	18.0	0.0	6.4	1,665	0
	Rear	56.0	1.0	1.0	1,665	58
Medium Truck	Front	25.8	1.0	6.4	1,110	8
	Rear	69.6	1.0	1.0	1,110	101
Heavy Truck	Front	48.2	1.0	6.4	1,110	128
	Rear	100.0	1.0	1.0	1,110	498
Heavy Truck	Front	45.4	1.0	6.4	832	74
	Rear	90.0	2.2	1.0	832	516
Heavy Truck	Front	23.1	0.0	6.4	832	0
	Rear	73.2	2.0	1.0	832	189
Total					N=	1,790

Ntt(2020)= 1,790 * 0.30 = 537 (veh/day) (2020)

Therefore,

$$N_e = \frac{[(1+q_1)^{t1} - 1]}{q_1(1+q_1)^{t1-1}} *365*N_{tt1} + \frac{[(1+q_2)^{t2} - 1]}{q_2(1+q_2)^{t2-1}} *365*N_{tt2}$$

= 812,484 + 4,576,282 = 5,388,766 (axles)

D-5. Determination of strength coefficient for design reliability and Required elastic modulus value

$E_{ch} \geq K_{cd}^{dv} * E_{yc}$ = 1.10 * 214

= 235 (Mpa) (3-4)

K_{cd}^{dv}: Strength coefficient on deflection depending on reliability

E_{yc}: Required elastic modulus value

Table 3-2: Determination of strength coefficient on deflection depending on reliability

Reliability	0.98	0.95	0.90	0.85	0.80
Strength coefficient K_{cd}^{dv}	1.29	1.17	1.10	1.06	1.02

Table 3-3 : Selection of design reliability by road type and class

Road type, class		**Design reliability**		
1. Expressway		0.90	0.95	0.98
2. Highway/road	-I, II Class	0.00	0.95	0.98
	-III, IV Class	0.85	0.90	0.95
	-V, VI Class	0.80	0.85	0.90
3. Urban road	-Expressway and urban arterial road	0.90	0.95	0.98
	-Other urban road	0.85	0.90	0.95
4. Specialized road		0.80	0.85	0.90

K_{cd}^{dv}= 1.10

Table3-4 Required elastic modulus value

Types of standard axle load		**Required elastic modulus value Eyc(Mpa), Corresponding to a number of calculation axles (vehicle/ day/lane)**									
		10	20	50	100	200	500	1000	2000	5000	7000
10	High-grade A1			133	147	160	178	192	207	224	235
	High-grade A2		91	110	122	135	153				
	Low-grade B1		64	82	94						
12	High-grade A1		127	146	161	173	190	204	218	235	253
	High-grade A2	90	103	120	133	146	163				
	Low-grade B1		79	98	111						

E_{yc}= 204+(218-204)*(1705-1000)/(2000-1000)

= 214 Mpa

D-6. Design Condition

- Material Condition

Table D6-1 Material Properties for Each layer

Materal	E(Mpa)			R_{ku}	C	φ	t
	Sliding	Deflection	Tensil and Flexture	(Mpa)	(Mpa)	(degree)	(cm)
Surface Course	300	420	1800	2.8			6
Binder Course	350	350	1600	2.0			7
Asphalt Treated Base	350	350	800				10
Base	300	300	300				81
Embankment	50	50	50		0.028	21	-

Surface Course : Bituminous layer type I (Surface Course) (BTNC20, Crushed Stone>50%)
Binder Course : Bituminous layer type I(Binder Course) (BTNC25, Crushed Stone>50%)
Asphalt Treated Base : Black crushed stone mixed with compact asphalt
Base : Crushed Stone Aggregate Base Class I
Embankment : Clay and loam, CBR=8

Note: *Values of Asphalt and Base were determined by refering to Table C-1
*E of Embankment was desided by formula B-5, (CBR=8)
$E=4.68*CBR+12.48=50$(Mpa)
*Values of Embankment were determined by refering to Table B-3 ($W/W_{nh}=0.65$)
*Minimum Asphalt thickness should be 12.5cm. ($N_e>4.0*10^6$) (According to Table 2-2)

Therefore design condition is as following

Ntt=	1,705	(vehicles/lane/day in 2030)
Ne=	5,388,766	(axle for 15 years)
P_{tt}=	120	(kN)
p=	0.60	(Mpa)
D=	36	(cm)

E_{yc}= 214 (Mpa), K_{cd}^{dv}= 1.1

		t(cm)
Surface Course	uminous layer type I (Surface Course) (BTNC20, Crushed Stone>5	6
Binder Course	uminous layer type I(Binder Course) (BTNC25, Crushed Stone>5(	7
Asphalt Treated Base	Black crushed stone mixed with compact asphalt	10
Base	Crushed Stone Aggregate Base Class I	81
Embankment	Clay and loam, CBR=8	

D-7. Cheking Deflection

No.		$K_{cd}^{dv}*E_{yc}=$	235	Mpa		
4	Surface Course	$E_4=$	420	Mpa	6	cm
3	Binder Course	$E_3=$	350	Mpa	7	cm
2	Asphalt Treated Base	$E_{2=}$	350	Mpa	10	cm
1	Base	$E_1=$	300	Mpa	81	cm
		$E_0=$	50	Mpa		

$$E_{TB} = E_1 \cdot \left[\frac{\left(1+k.t^{1/3}\right)}{(1+k)}\right]^3 \text{ Mpa} \qquad k=\frac{h_1}{h_2} \quad t=\frac{E_2}{E_1} \qquad (3\text{-}5)$$

The results are described in the below table:

Table D7-1 Calculation results of E_{tbi}

Layer	Material Courses	E_i (Mpa)	t $=E_2/E_1$	h_i (cm)	K $=h_2/h_1$	h_{tbi} (cm)	E_{tbi} (Mpa)
4	Surface Course	420	1.36	6	0.06	104	314
3	Binder Course	350	1.15	7	0.08	98	308
2	Asphalt Treated Base	350	1.17	10	0.12	91	305
1	Base	300		81		81	300

$E_{tb5}=$ 314 daN/cm^2

$\beta =$ 1.265 (from below)

$E_{TB}^{tt} = \beta \cdot E_{tb}$

= 1.265 * 314

= 397 Mpa

Table 3-6 Adjustment coefficient β

H/D Ratio	0.51	0.75	1.00	1.25	1.50	1.75	2.00
β Coefficient	1.033	1.069	1.107	1.136	1.178	1.198	1.210

$\beta = 1.114*(H/D)^{0.12}$ (in case of H/D>2.0) (3-6)

Where as;

D= 36 cm (Diameter of Wheel track)

H= 104 cm (Hight of the Layer)

Therefore

H/D= 2.89

From Table/formula 3-6,

$\beta = 1.114*(104/36)^{0.12}$

= 1.265

E_0= 50 Mpa , E_{TB}^{tt}= 397 Mpa

E_0/E_{TB}^{tt}= 0.126

H/D= 2.89

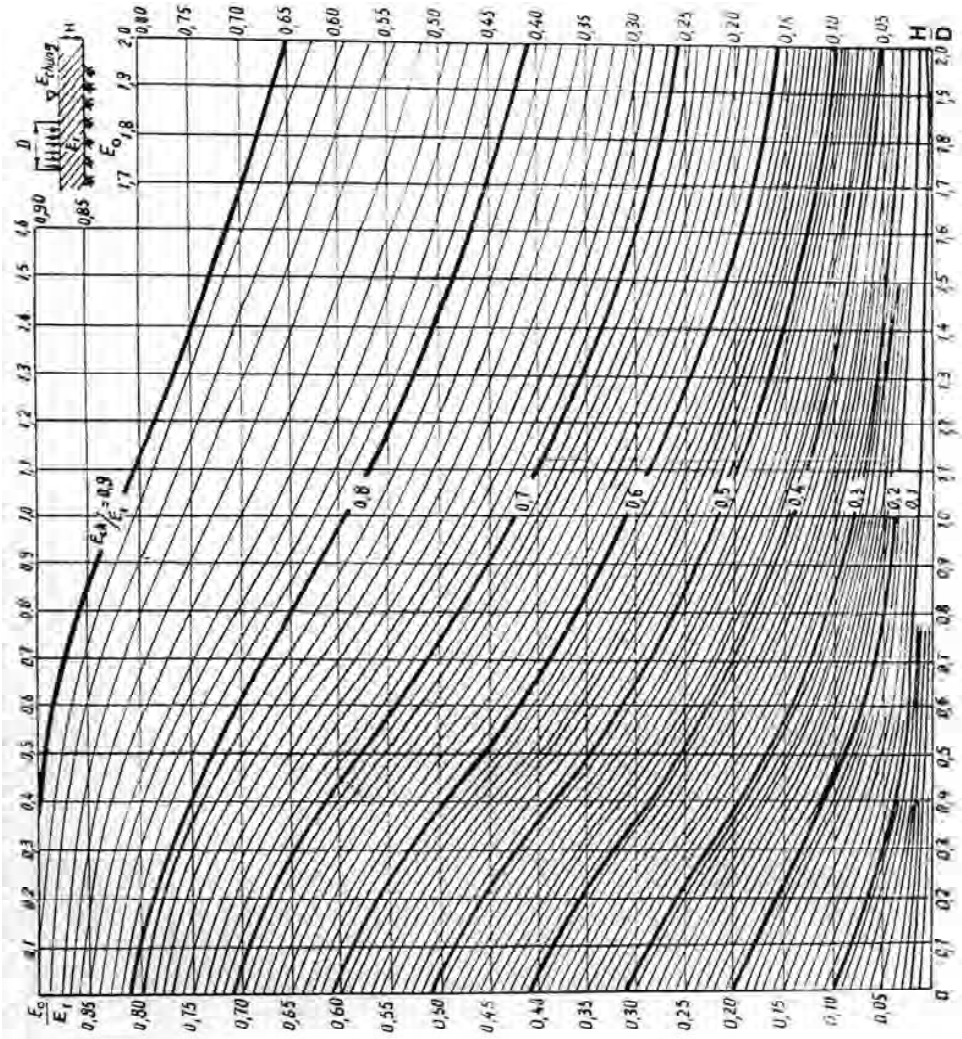


Figure 3-1. Monograph for determination of general elastic modulus of double-layer system E_{ch} (H/D<2)

$$E_{ch} = \frac{1.05.E_0}{\dfrac{1+\dfrac{E_0}{E_1}}{\sqrt{1+4\left(\dfrac{H}{D}\right)^2\cdot\left(\dfrac{E_o}{E_1}\right)^{-0.67}}}+\dfrac{E_o}{E_1}} \qquad \text{(F-1)} \quad \text{(H/D>2)}$$

From above Table/Formula, E_{ch}= 236 Mpa ≧ 235 **Mpa (=K_{cd}^{dv}*E_{yc})** **OK**

D-8. Cheking Sliding

$$\mathbf{T_{ax} + T_{av}} < \frac{\mathbf{C_{tt}}}{\mathbf{K_{cd}^{tr}}} \qquad (3\text{-}7)$$

T_{ax} : From Table 3-2 (H/D<2.0), or From Table 3-3 (H/D >2.0)
T_{av} : From Table 3-4
C_{tt} : From formula(3-8)
K_{cd}^{tr} : From Table 3-7

- Detarmination of K_{cd}^{tr}

$\mathbf{K_{cd}^{tr}}$ = 0.94 (From Table 3-7)

Table 3-7 Selection of coefficient of shear strength depending on reliability

Reliability	0.98	0.95	0.90	0.85	0.80
Coef. K_{cd}^{tr}	1.10	1.00	0.94	0.90	0.87

- Calculation of Ctt

$\mathbf{C_{tt}}$= $C*K_1*K_2*K_3$ = 0.0164 (Mpa)

C: 0.028 (Mpa) Cohesive force of fundation soil
K_1: 0.6 for pavement carriageway
K_2: 0.65 from Table 3-8

Table 3-8 Determination of Coefficient K2 depending on a number of design axles

A number of design axles (N_{tt}) (axle/day/lane)	Under 100	Under 1000	Under 5000	Over 5000
Coefficient K_2	1.00	0.80	0.65	0.60

*Ntt= 1,705

K_3: 1.5 For types of cohesive soil (clay, clay loam, clay sand etc.)

No.		C_{tt}/K_{cd}^{tr}=	0.017		
4	Surface Course	E_4=	300	Mpa	6 cm
3	Binder Course	E_3=	350	Mpa	7 cm
2	Asphalt Treated Base	$E_{2=}$	350	Mpa	10 cm
1	Base	E_1=	300	Mpa	81 cm

E_0= 50 Mpa
C= 0.028 Mpa
φ= 21 degree

- Determination of T_{ax}

Table D8-1 Calculation results of E_{tbi}

Layer	Material Courses	E_i (Mpa)	t $=E_2/E_1$	h_i (cm)	K $=h_2/h_1$	h_{tbi} (cm)	E_{tbi} (Mpa)
4	Surface Course	300	0.97	6	0.06	104	308
3	Binder Course	350	1.15	7	0.08	98	308
2	Asphalt Treated Base	350	1.17	10	0.12	91	305
1	Base	300		81		81	300

E_{tb}= 308 daN/cm^2
β = 1.265 (from below)
$E_{TB}^{tt} = \beta \cdot E_{tb}$
= 1.265 * 308
= 389 Mpa

Table 3-6 Adjustment coefficient β

H/D Ratio	0.51	0.75	1.00	1.25	1.50	1.75	2.00
β Coefficient	1.033	1.069	1.107	1.136	1.178	1.198	1.210

$\beta = 1.114*(H/D)^{0.12}$ (in case of H/D>2.0) (3-6)

Where as;
D= 36 cm (Diameter of Wheel track)
H= 104 cm (Hight of the Layer)
Therefore
H/D= 2.89
From Table/formula 3-6,
β = 1.114*(104/36)0.12
= 1.265

E_0= 50 Mpa , E_{TB}^{tt}= 389 Mpa

E_{TB}^{tt}/E_o= 7.79

H/D= 2.89 φ= 21 degree

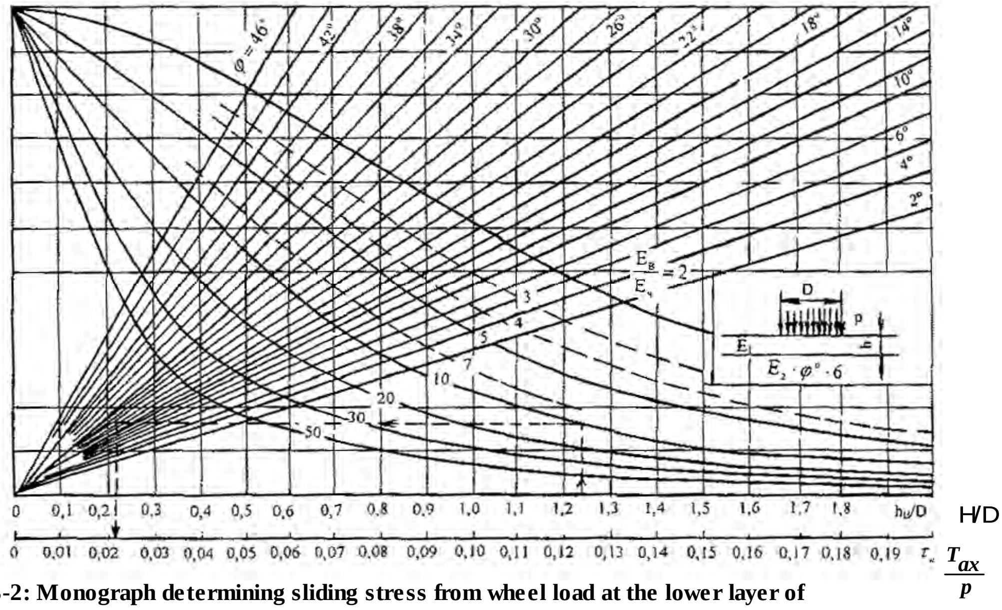


Figure 3-2: Monograph determining sliding stress from wheel load at the lower layer of the double-layer system (H/D = 0 ~ 2)

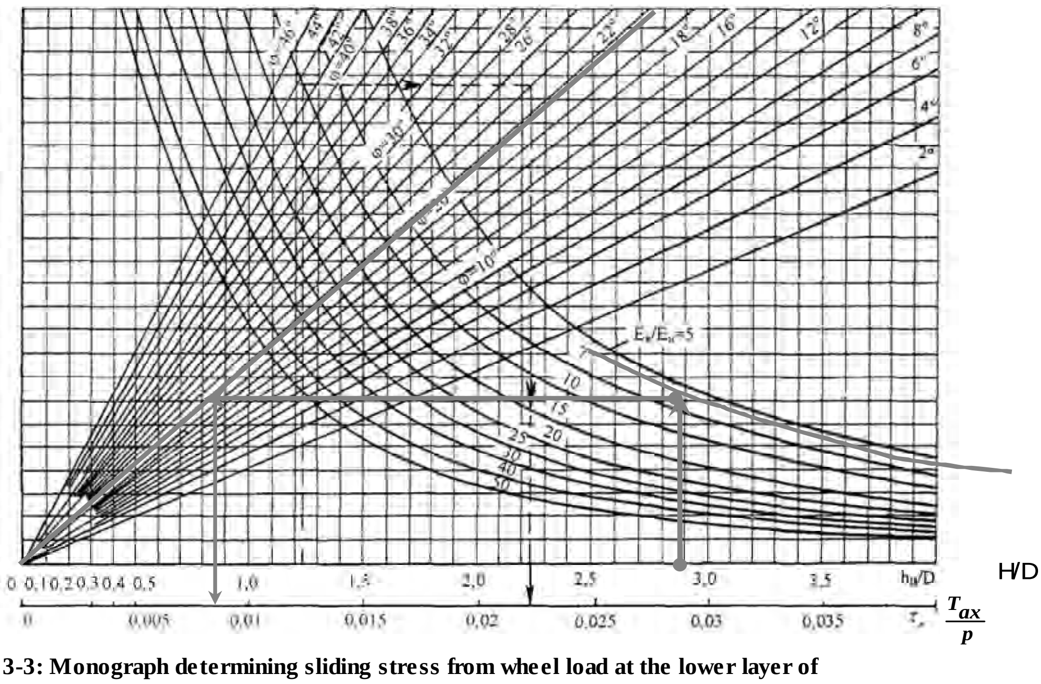


Figure 3-3: Monograph determining sliding stress from wheel load at the lower layer of the double-layer system (H/D = 0 ~ 4)

From above Figure, T_{ax}/p= 0.008 , p= 0.60 Mpa

Therefore T_{ax}= 0.0048 Mpa

- Determination of T_{av}

H= 104 cm
φ= 21 degree

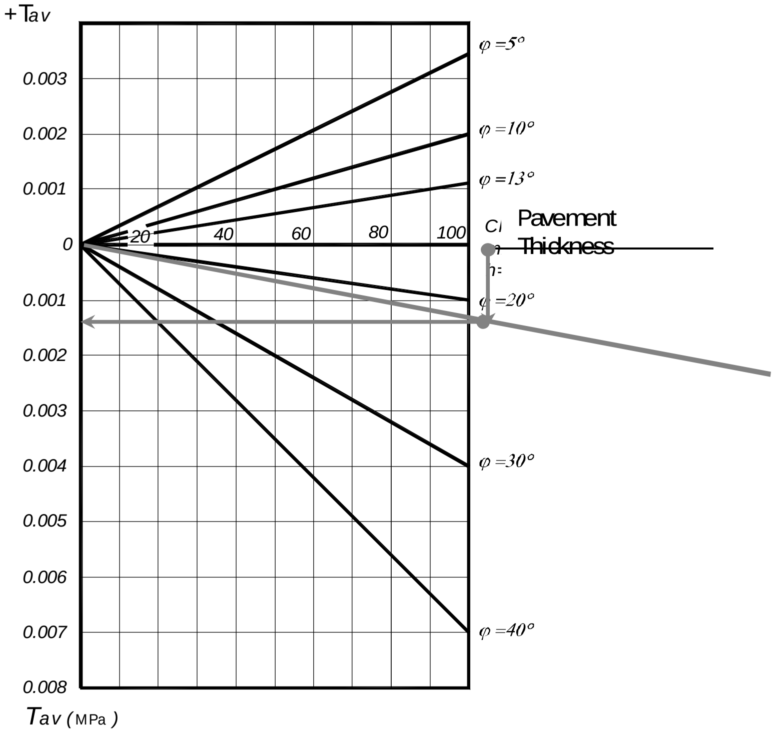


Figure 3-4. Monograph for finding active shearing stress T_{av} by self waight of pavement

From above Figure,

T_{AX}= -0.0013 Mpa

- Cheking Sliding

$$T_{ax} + T_{av} < \frac{C_{tt}}{K_{cd}^{tr}} \qquad (3\text{-}7)$$

T_{ax} =	0.0048	Mpa
T_{av} =	-0.0013	Mpa
C_{tt} =	0.0164	Mpa
K_{cd}^{tr} =	0.94	Mpa

$T_{ax} + T_{av}$ =	0.0048	+	-0.0013	=	0.0035
C_{tt}/K_{cd}^{tr}=	0.01638	/	0.94	=	0.0174

$T_{ax} + T_{av} < C_{tt}/K_{cd}^{tr}$ **OK**

D-9. Cheking Flexural Strength

$$\sigma_{ku} \leq \frac{R_{tt}^{ku}}{K_{cd}^{ku}} \qquad (3\text{-}9)$$

$$\sigma_{ku} = \overline{\sigma}_{Ku} * p * K_b \qquad (3\text{-}10)$$

$\overline{\sigma}$ ku: the value shall be determined by Figure 3-5, 3-6, (See after)

p: = 0.60 Mpa

Kb= 1.00 (for the heaviest special axle load)

$R_{tt}^{ku} = k_1 * k_2 * R_{ku}$ (3-11)

$k_1 = 11.11/N_e^{0.22}$ (for asphalt concrete material) (3-12)

$= 11.11/ \; 5{,}388{,}766^{0.22} = \underline{0.37}$

$N_e =$ 5,388,766 (axle for 15 years)

$k_2 =$ 1.00 (for material consolidated with inorganic material)

$R_{ku} =$ 2.80 (for Surface Course, See D-7 Design Condition)

2.00 (for Binder Course, See D-7 Design Condition)

$K_{cd}^{ku} =$ 0.94 (From Table 3-7)

- Design Condition

No.				
4 Surface Course	E_4=	1800	Mpa	6 cm
3 Binder Course	E_3=	1600	Mpa	7 cm
2 Asphalt Treated Base	E_{2}=	800	Mpa	10 cm
1 Base	E_1=	300	Mpa	81 cm
	E_0=	50	Mpa	

Table D9-1 Calculation results of E_{tbi}

Layer	Material Courses	E_i (Mpa)	t $=E_2/E_1$	h_i (cm)	K $=h_2/h_1$	h_{tbi} (cm)	E_{tbi} (Mpa)
4	Surface Course	1800	4.60	6	0.06	104	438
3	Binder Course	1600	4.71	7	0.08	98	392
2	Asphalt Treated Base	800	2.67	10	0.12	91	340
1	Base	300		81		81	300

A) Cheking of Surface Course

h_1= 6 cm, E_1= 1800 Mpa

Etb4= 392 Mpa (from Table D9-1)

β = 1.256 (from below)

$E_{TB}{}^{tt} = \beta \cdot E_{tb}$

= 1.256 * 392

= 492 Mpa

Table 3-6 Adjustment coefficient β

H/D Ratio	0.51	0.75	1.00	1.25	1.50	1.75	2.00
β Coefficient	1.033	1.069	1.107	1.136	1.178	1.198	1.210

$\beta = 1.114*(H/D)^{0.12}$ (in case of H/D>2.0) (3-6)

Where as;

D= 36 cm (Diameter of Wheel track)

H= 98 cm (Hight of the Layer)

Therefore

H/D= 2.72

From Table/formula 3-6,

$\beta = 1.114*(98/36)^{0.12}$

= 1.256

E_0= 50 Mpa , E_{TB}^{tt}= 492 Mpa

E_0/E_{TB}^{tt}= 0.102

H/D= 2.72

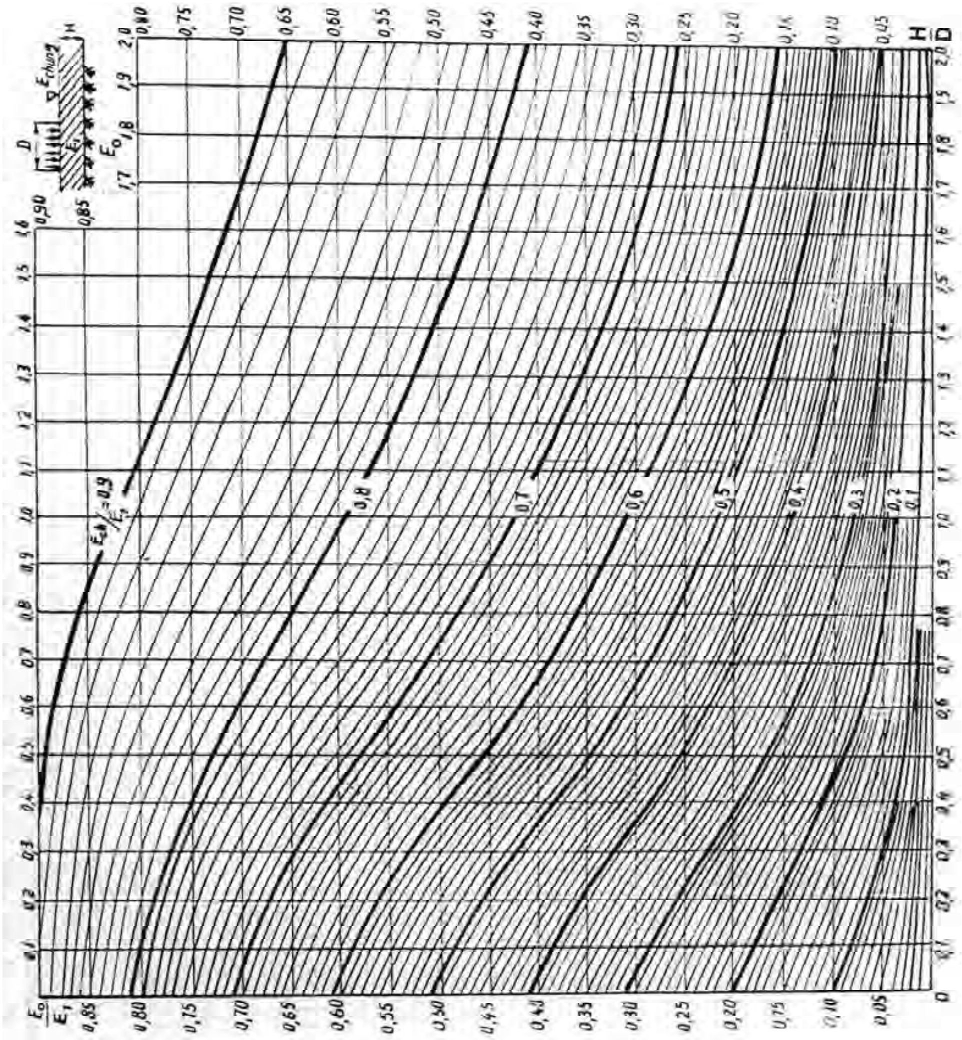


Figure 3-1. Monograph for determination of general elastic modulus of double-layer system E_{ch} (H/D<2)

$$E_{ch} = \frac{1.05.E_0}{\dfrac{1+\dfrac{E_0}{E_1}}{\sqrt{1+4\left(\dfrac{H}{D}\right)^2 . \left(\dfrac{E_o}{E_1}\right)^{-0.67}}} + \dfrac{E_o}{E_1}} \qquad \text{(F-1)} \qquad \text{(H/D>2)}$$

From above Table/Formula, E_{ch}= 269 Mpa

h_1=	6 cm	⇨	H/D=	0.1667
E_1=	1800 Mpa	⇨	E_1/E_{ch}=	6.6999

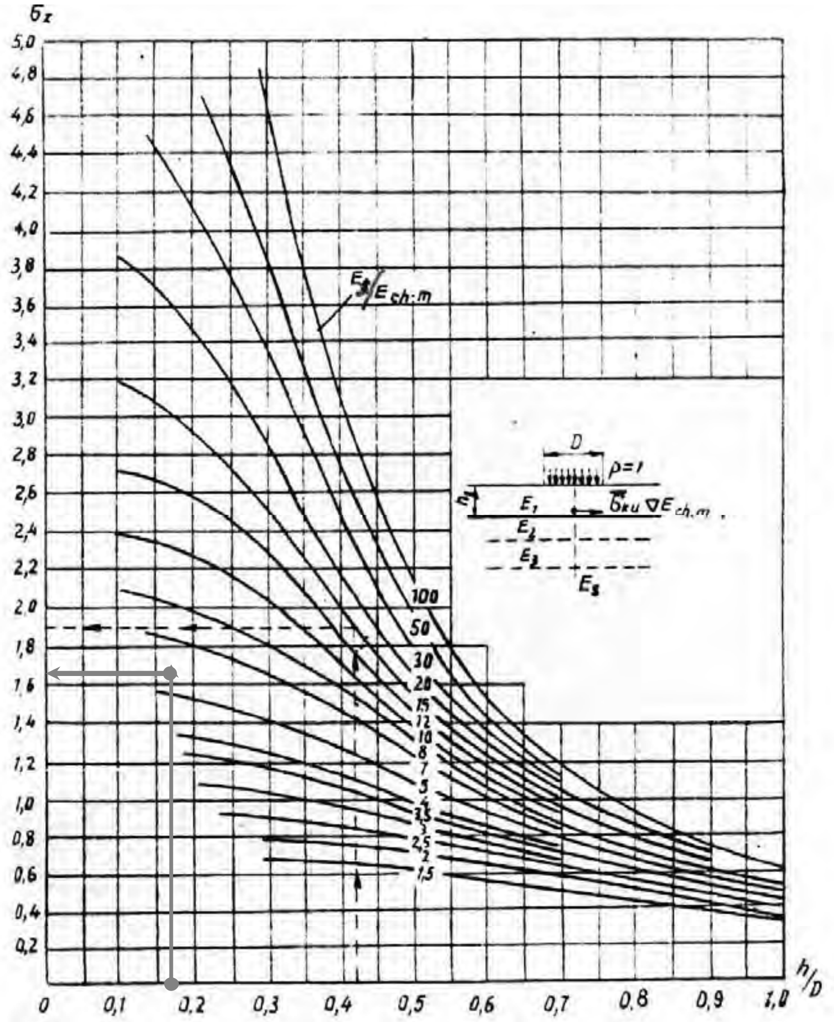


Figure 3-5: Monograph determining unit bending and tensile stress σ_{ku} in layers of surface layer

From above Figure,

$\overline{\sigma_{ku}}$ = 1.65

Terefore,

$\sigma_{ku} = \overline{\sigma_{Ku}} * p * K_b$ = 1.65 * 0.60 * 1.00

= 0.99

And,

$R_{tt}^{ku} = k_1 * k_2 * R_{ku}$ = 0.37 * 1.00 * 2.80

= 1.03

K_{cd}^{ku} = 0.94

$$\sigma_{ku} \leq \frac{R_{tt}^{ku}}{K_{cd}^{ku}} = \frac{1.03}{0.94} = 1.09 \quad \textbf{OK}$$

B) Cheking of Binder Course

$h_1 = 6 + 7 = \underline{13}$ cm

$$E_1 = \frac{1800 * 6 + 1600 * 7}{6 + 7}$$

$= \underline{1692}$ Mpa

Etb4= 340 Mpa (from Table D9-1)

$\beta =$ 1.245 (from below)

$E_{TB}^{tt} = \beta \cdot E_{tb}$

$= 1.245 * 340$

$= \underline{423}$ Mpa

Table 3-6 Adjustment coefficient β

H/D Ratio	0.51	0.75	1.00	1.25	1.50	1.75	2.00
β Coefficient	1.033	1.069	1.107	1.136	1.178	1.198	1.210

$\beta = 1.114*(H/D)^{0.12}$ (in case of H/D>2.0) (3-6)

Where as;

D= 36 cm (Diameter of Wheel track)

H= 91 cm (Hight of the Layer)

Therefore

H/D= 2.53

From Table/formula 3-6,

$\beta = 1.114*(91/36)^{0.12}$

$= 1.245$

E_0= 50 Mpa , E_{TB}^{tt}= 423 Mpa

E_0/E_{TB}^{tt}= 0.118

H/D= 2.53

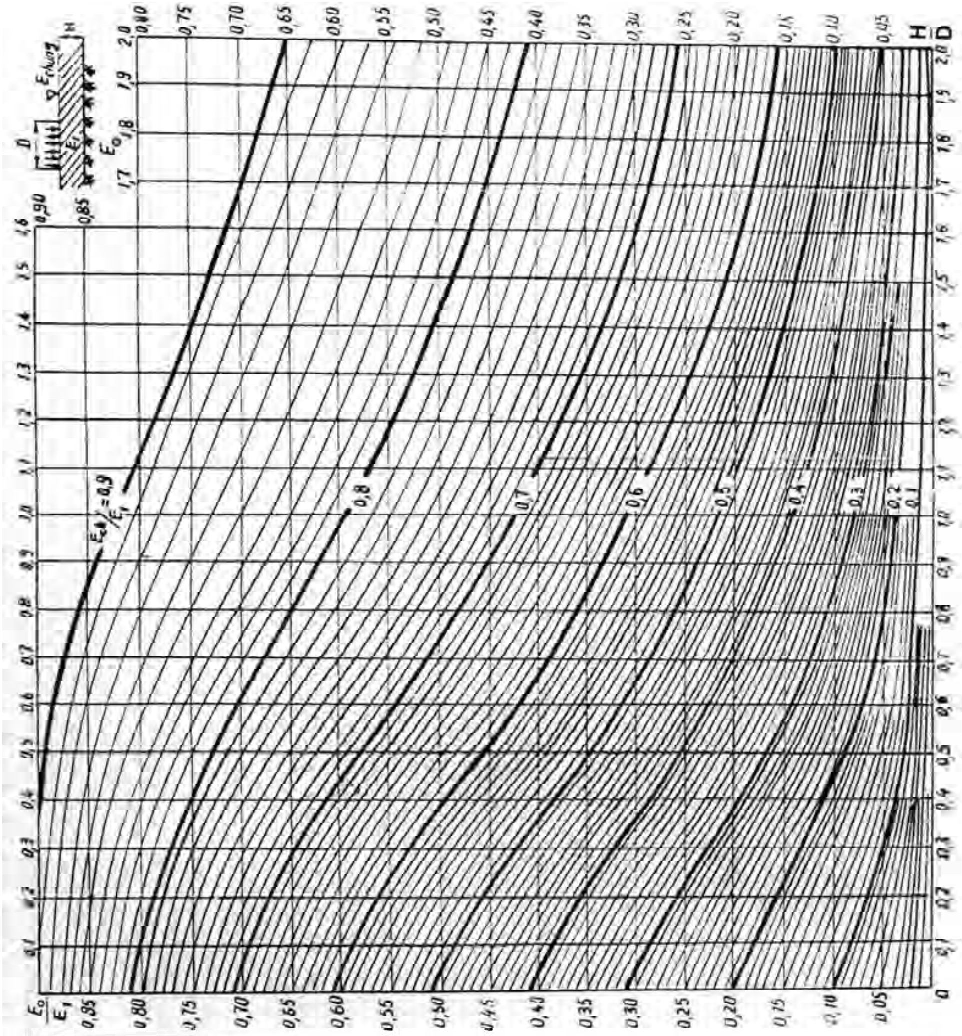


Figure 3-1. Monograph for determination of general elastic modulus of double-layer system E_{ch} (H/D<2)

$$\mathbf{E_{ch}} = \frac{1.05.E_0}{\dfrac{1+\dfrac{E_0}{E_1}}{\sqrt{1+4\left(\dfrac{H}{D}\right)^2\cdot\left(\dfrac{E_o}{E_1}\right)^{-0.67}}}+\dfrac{E_o}{E_1}} \qquad \text{(F-1)} \qquad \text{(H/D>2)}$$

From above Table/Formula, E_{ch}= 233 Mpa

h_1=	13	cm	⇨	H/D=	0.36
E_1=	1692	Mpa	⇨	E_1/E_{ch}=	7.28

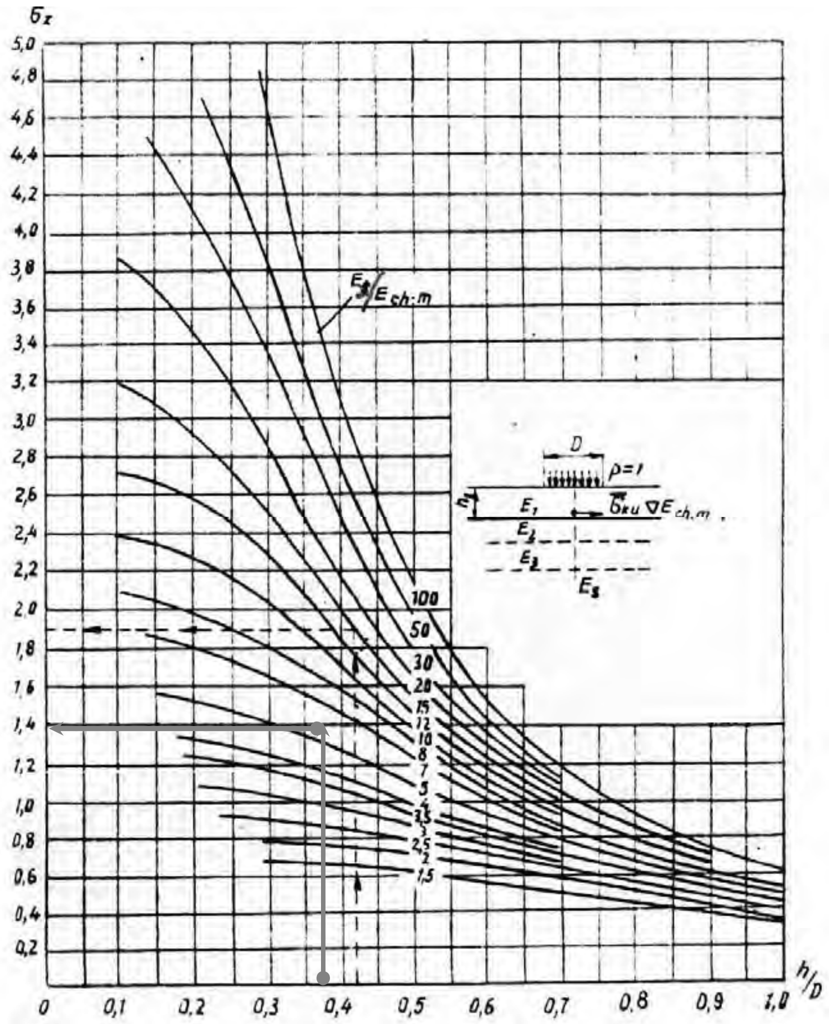


Figure 3-5: Monograph determining unit bending and tensile stress σ_{ku} in layers of surface layer

From above Figure,

$\overline{\sigma_{ku}}$ = 1.40

Terefore,

$\sigma_{ku} = \overline{\sigma_{Ku}} * p * K_b$ = 1.40 * 0.60 * 1.00

= 0.84

And,

$R_{tt}^{ku} = k_1 * k_2 * R_{ku}$ = 0.37 * 1.00 * 2.80

= 1.03

K_{cd}^{ku} = 0.94

$$\sigma_{ku} \le \frac{R_{tt}^{ku}}{K_{cd}^{ku}} = \frac{1.03}{0.94} = 1.09 \qquad \textbf{OK}$$

7.3 Interchange/Intersection Design

7.3.1 Location of Interchange/Intersections

Six interchanges and intersections are planned in the Tan Vu-Lach Huyen Highway.

The location of interchanges/intersections is shown in Table 7.3.1-1.

Table 7.3.1-1 Location of Intersections

Km	Name	Leg	Note
0+000	Tan Vu Interchange	Three-leg	Connected with Hanoi-Haiphong Expressway at Km 100+891.11.
2+836.32	No.1 Intersection	Four-leg	Connected with Dinh Vu Ring Road, which will be constructed as the trunk road in the Dinh Vu Industrial Zone area.
5+149.11	No.2 Intersection	Four-leg	Tan Vu-Lach Huyen Highway will pass over the Dinh Vu Ring Road by bridge. Two roads will not be connected, but pier position will be planned so as not to disturb the intersection development.
7+521.05	No.3 Intersection	Four-leg	Tan Vu-Lach Huyen Highway will pass over the Dinh Vu Ring Road by bridge. Two roads will not be connected, but pier position will be planned so as not to disturb the intersection development.
11+520	-	Three-leg	Connected with local road in Cat Hai.
15+576	-	Four-leg	Connected with local road which is the way to the existing ferry terminal in Cat Hai.

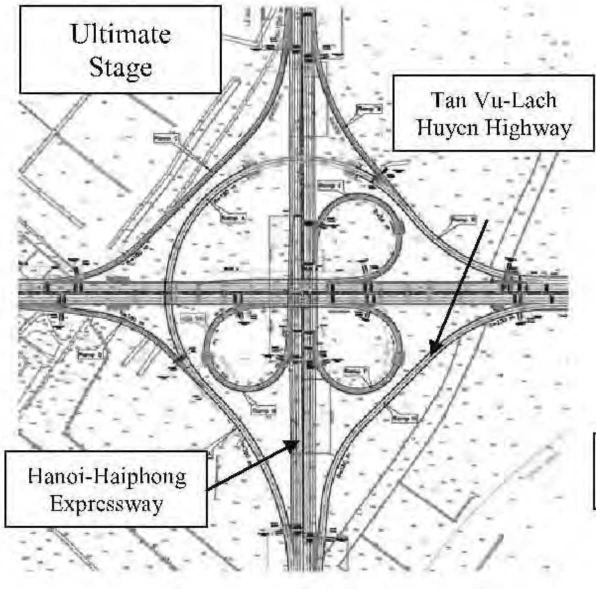
Source : Study Team

7.3.1.1 Interchange/Intersection Design

(1) Tan Vu Interchange

1) Introduction

It was planned that Tan Vu Interchange is developed as “cloverleaf with semi-direct connection” in the ultimate stage (see Figure 7.3.1-1).



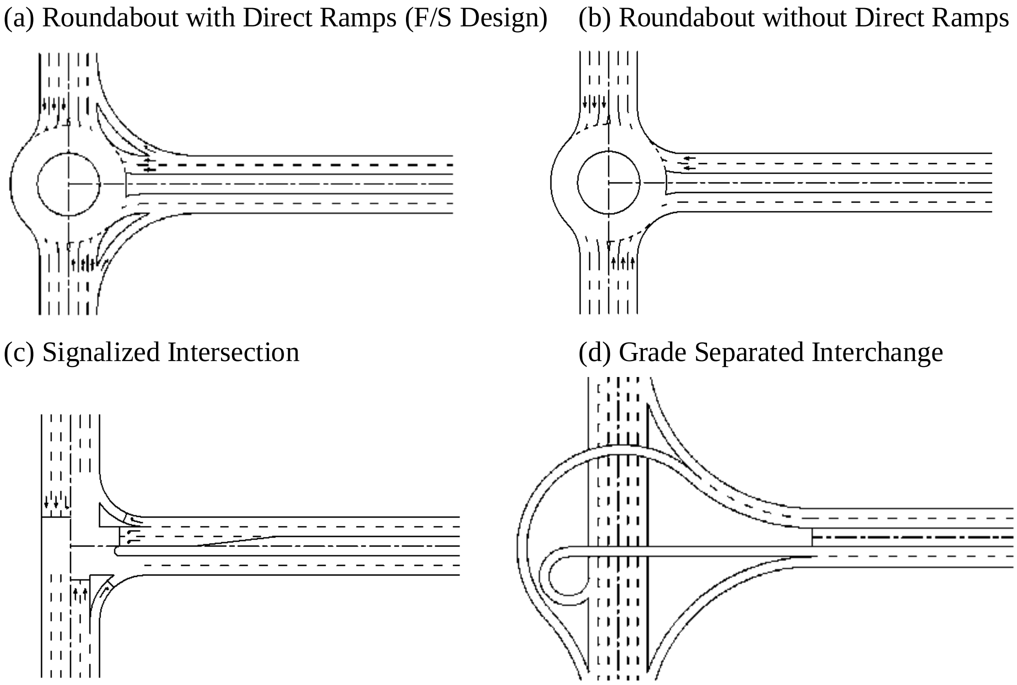
Source: SAPROF Report (July 2010)

Figure 7.3.1-1 Tan Vu Interchange in Ultimate Stage

However, adoption of other interchange/intersection types including at grade intersection and grade separated interchange is studied for the initial stage, since it is forecasted that there is not much traffic before Haiphong Ring Road No.3 is connected to Hanoi-Haiphong Expressway.

2) Alternatives

The following four alternatives are studied.



Source : Study Team

Figure 7.3.1-2 Alternative Interchange/Intersection types

3) Evaluation

Four alternatives are evaluated with comprehensive aspects.

Summary of evaluation is shown in Table 7.3.1-2. As a result, “Signalized Intersection” is selected as the appropriate intersection type for Tan Vu Interchange at the initial stage from the following viewpoints.

- Enough traffic capacity (Capable until 2026)
- Cheaper initial construction cost ((c) Signalized Intersection: 15 billion VND, (d) Grade Separated Interchange: 354 billion VND)
- High traffic safety
- No additional land acquisition
- Easy future upgrade

Table 7.3.1-2 Evaluation of Interchange/Intersection Type for Tan Vu Interchange

Evaluation Item (Basic Score) \ Alternative	(a) Roundabout with Direct Ramps (F/S Design)	(b) Roundabout without Direct Ramps	(c) Signalized Intersection	(d) Grade Separated Interchange
Description	- At grade intersection. - Two roads are connected with roundabout. - Additional 2 direct ramps (east to north, south to east) are provided to secure smooth traffic flow on these directions.	- At grade intersection. - Two roads are connected with roundabout.	- At grade intersection. - Two roads are connected with signalized intersection.	- Grade separated interchange. - Two roads are connected with ramps - Two bridges are required.
Capacity Analysis *2 (30)	- Degree of saturation for all entries is less than 1.0 in Year 2015 and 2020. - Enough traffic capacity is secured. (24, Moderate)	- Degree of saturation for all entries is less than 1.0 in Year 2015 and 2020. - Enough traffic capacity is secured. (24, Moderate)	- v/c ratio for all lane groups and intersection itself are less than 1.0 in Year 2015 and 2020. - Enough traffic capacity is secured. (24, Moderate)	- Two roads are grade separated. - Enough traffic capacity is secured. (30, Good)
Construction Cost (20)	- Higher since two ramps are constructed. (16, Moderate)	- Cheap since only earth work is made. (20, Good)	- Cheap since only earth work is made. (20, Good)	Highest since four ramps including two bridges are constructed. (10, Poor)
Safety (20)	- Safety is secured by installing traffic signs to inform drivers of the existence of roundabout before entering roundabout. (16, Moderate)	- Safety is secured by installing traffic signs to inform drivers of the existence of roundabout before entering roundabout. (16, Moderate)	- Each traffic is separated by signal phasing. (20, Good)	- Each traffic is separated by grade separation. (20, Good)
Land Acquisition (20)	- Approximately 44,000 m2 of additional land acquisition is required. (16, Moderate)	- No additional land acquisition is required. (20, Good)	- No additional land acquisition is required. (20, Good)	- Approximately 84,000 m2 of additional land acquisition is required. (10, Poor)
Future Upgrade (10)	- Easy since only earthwork is made at initial stage. (8, Moderate)	- Easy since only earthwork is made at initial stage. (8, Moderate)	- Easy since only earthwork is made at initial stage. (8, Moderate)	- Easiest since initial construction is made in consideration of ultimate interchange type. (10, Good)
Recommendation (100) *1	(80)	(88)	Recommended (92)	(80)

Note: *1: Score is basic score x evaluation (Good=1.0, Moderate=0.8, Poor=0.5).

*2: Detailed capacity analysis is shown in Table 7.6.2-2.

Source : Study Team

Table 7.3.1-3 Capacity Analysis of Tan Vu Interchange

(a) Roundabout with Direct Ramps (F/S Design)

Year 2015

Traffic Volume

Entry Flow	North Bound	East Bound	South Bound	West Bound
Left Turn			177	279
Through	306		221	
Right Turn				
Total of Entry Flow (D) (pcu/h)	306	0	398	279

Circulating Flow	NB Circ	EB Circ	SB Circ	WB Circ
Circulating Flow (pcu/h)	177	677	279	306

Capacity

	North Bound	East Bound	South Bound	West Bound
Input from Geometric Design				
e: Entry Width (m)	17.84		17.84	14.98
v: Approach Half Width (m)	15.00		15.00	9.75
l': Effective Flare Length (m)	29.16		29.16	32.88
D: Inscribed Circle Diameter (m)	100.00		100.00	100.00
ø: Enty Angle (Degrees)	34		36	39
r: Entry Radius (m)	70.00		70.00	70.00
Entry Capacity (C) (pcu/h)	5,141		5,009	3,782

Degree of Saturation

	North Bound	East Bound	South Bound	West Bound
Entry Flow / Entry Capacity (D/C)	**0.06**		**0.08**	**0.07**

Year 2015

Year 2020

Traffic Volume

Entry Flow	North Bound	East Bound	South Bound	West Bound
Left Turn			310	578
Through	365		266	
Right Turn				
Total of Entry Flow (D) (pcu/h)	365	0	576	578

Circulating Flow	NB Circ	EB Circ	SB Circ	WB Circ
Circulating Flow (pcu/h)	310	1,154	578	365

Capacity

	North Bound	East Bound	South Bound	West Bound
Input from Geometric Design				
e: Entry Width (m)	17.84		17.84	14.98
v: Approach Half Width (m)	15.00		15.00	9.75
l': Effective Flare Length (m)	29.16		29.16	32.88
D: Inscribed Circle Diameter (m)	100.00		100.00	100.00
ø: Enty Angle (Degrees)	34		36	39
r: Entry Radius (m)	70.00		70.00	70.00
Entry Capacity (C) (pcu/h)	5,013		4,724	3,736

Degree of Saturation

	North Bound	East Bound	South Bound	West Bound
Entry Flow / Entry Capacity (D/C)	**0.07**		**0.12**	**0.15**

Year 2020

(b) Roundabout without Direct Ramps

Year 2015

Traffic Volume

Entry Flow	North Bound	East Bound	South Bound	West Bound
Left Turn			177	279
Through	306		221	
Right Turn	259			159
Total of Entry Flow (D) (pcu/h)	565	0	398	438

Circulating Flow	NB Circ	EB Circ	SB Circ	WB Circ
Circulating Flow (pcu/h)	177	677	279	306

Capacity

	North Bound	East Bound	South Bound	West Bound
Input from Geometric Design				
e: Entry Width (m)	17.84		17.84	14.98
v: Approach Half Width (m)	15.00		15.00	9.75
l': Effective Flare Length (m)	29.16		29.16	32.88
D: Inscribed Circle Diameter (m)	100.00		100.00	100.00
ø: Enty Angle (Degrees)	34		36	39
r: Entry Radius (m)	70.00		70.00	70.00
Entry Capacity (C) (pcu/h)	5,141		5,009	3,782

Degree of Saturation

	North Bound	East Bound	South Bound	West Bound
Entry Flow / Entry Capacity (D/C)	**0.11**		**0.08**	**0.12**

Year 2015

Year 2020

Traffic Volume

Entry Flow	North Bound	East Bound	South Bound	West Bound
Left Turn			310	578
Through	365		266	
Right Turn	800			455
Total of Entry Flow (D) (pcu/h)	1,165	0	576	1,033

Circulating Flow	NB Circ	EB Circ	SB Circ	WB Circ
Circulating Flow (pcu/h)	310	1,154	578	365

Capacity

	North Bound	East Bound	South Bound	West Bound
Input from Geometric Design				
e: Entry Width (m)	17.84		17.84	14.98
v: Approach Half Width (m)	15.00		15.00	9.75
l': Effective Flare Length (m)	29.16		29.16	32.88
D: Inscribed Circle Diameter (m)	100.00		100.00	100.00
ø: Enty Angle (Degrees)	34		36	39
r: Entry Radius (m)	70.00		70.00	70.00
Entry Capacity (C) (pcu/h)	5,013		4,724	3,736

Degree of Saturation

	North Bound	East Bound	South Bound	West Bound
Entry Flow / Entry Capacity (D/C)	**0.23**		**0.12**	**0.28**

Year 2020

(c) Signalized Intersection

Traffic Volume

	North Bound			East Bound			South Bound			West Bound		
	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn
Flow Rate (pcu/h)		306	259				177	221		279		159
Lane Group		TH	RT				LT	TH		LT		RT
Phase Number		1	1/3				2	1		3		2/3
Phasing	1	=>	2	=>	3							
Flow Rate in Lane Group (v) (pcu/h)		**306**	**259**				**177**	**221**		**279**		**159**

Saturation Flow

	North Bound LT	North Bound TH	North Bound RT	East Bound LT	East Bound TH	East Bound RT	South Bound LT	South Bound TH	South Bound RT	West Bound LT	West Bound TH	West Bound RT
S_o: Base Sturation Flow		1,900	1,900				1,900	1,900		1,900		1,900
N: Number of Lanes		2	1				1	2		2		1
f_{HV}: Heavy-vehicle Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_G: Grade Adjustment Factor		0.999	0.999				1.001	1.001		0.998		0.998
f_P: Parking Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_{BB}: Bus Blockage Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_A: Area Type Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_{LU}: Lane Utilization Adjustment Factor		0.952	1.000				1.000	0.952		0.971		1.000
f_{LT}: Left-turn Adjustment Factor		1.000	1.000				0.950	1.000		0.950		1.000
f_{RT}: Right-turn Adjustment Factor		1.000	0.850				1.000	1.000		1.000		0.850
f_{Lpb}: Left-turn Ped/Bike Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_{Rpb}: Right-turn Ped/Bike Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
Adjusted Saturation Flow (pcu/h)		**3,614**	**1,613**				**1,807**	**3,621**		**3,497**		**1,611**

Capacity Analysis

	North Bound LT	North Bound TH	North Bound RT	East Bound LT	East Bound TH	East Bound RT	South Bound LT	South Bound TH	South Bound RT	West Bound LT	West Bound TH	West Bound RT
Cycle Length (s)		100	100				100	100		100		100
Effective Green Time (s)		28	60				32	28		28		64
Los Time (s)		4	4				4	4		4		4
Green Ratio		0.280	0.600				0.320	0.280		0.280		0.640
Lane Group Capacity (c) (pcu/h)		1,012	968				578	1,014		979		1,031
v/c Ratio for Lane Group		**0.302**	**0.268**				**0.306**	**0.218**		**0.285**		**0.154**
Flow Ratio		0.085	0.161				0.098	0.061		0.080		0.099
Critical Lane Group/Phase		*					*			*		
Sum of Critical Flow Ratios	0.262											
v/c Ratio for Intersection	**0.298**											

Year 2015

Traffic Volume

	North Bound			East Bound			South Bound			West Bound		
	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn
Flow Rate (pcu/h)		365	800				310	266		578		455
Lane Group		TH	RT				LT	TH		LT		RT
Phase Number		1	1/3				2	1		3		2/3
Phasing	1	=>	2	=>	3							
Flow Rate in Lane Group (v) (pcu/h)		**365**	**800**				**310**	**266**		**578**		**455**

Saturation Flow

	North Bound LT	North Bound TH	North Bound RT	East Bound LT	East Bound TH	East Bound RT	South Bound LT	South Bound TH	South Bound RT	West Bound LT	West Bound TH	West Bound RT
S_o: Base Sturation Flow		1,900	1,900				1,900	1,900		1,900		1,900
N: Number of Lanes		2	1				1	2		2		1
f_{HV}: Heavy-vehicle Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_G: Grade Adjustment Factor		0.999	0.999				1.001	1.001		0.998		0.998
f_P: Parking Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_{BB}: Bus Blockage Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_A: Area Type Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_{LU}: Lane Utilization Adjustment Factor		0.952	1.000				1.000	0.952		0.971		1.000
f_{LT}: Left-turn Adjustment Factor		1.000	1.000				0.950	1.000		0.950		1.000
f_{RT}: Right-turn Adjustment Factor		1.000	0.850				1.000	1.000		1.000		0.850
f_{Lpb}: Left-turn Ped/Bike Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_{Rpb}: Right-turn Ped/Bike Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
Adjusted Saturation Flow (pcu/h)		**3,614**	**1,613**				**1,807**	**3,621**		**3,497**		**1,611**

Capacity Analysis

	North Bound LT	North Bound TH	North Bound RT	East Bound LT	East Bound TH	East Bound RT	South Bound LT	South Bound TH	South Bound RT	West Bound LT	West Bound TH	West Bound RT
Cycle Length (s)		100	100				100	100		100		100
Effective Green Time (s)		28	60				32	28		28		64
Los Time (s)		4	4				4	4		4		4
Green Ratio		0.280	0.600				0.320	0.280		0.280		0.640
Lane Group Capacity (c) (pcu/h)		1,012	968				578	1,014		979		1,031
v/c Ratio for Lane Group		**0.361**	**0.826**				**0.536**	**0.262**		**0.590**		**0.441**
Flow Ratio		0.101	0.496				0.172	0.073		0.165		0.282
Critical Lane Group/Phase			*				*					
Sum of Critical Flow Ratios	0.667											
v/c Ratio for Intersection	**0.758**											

Year 2020

Traffic Volume

	North Bound			East Bound			South Bound			West Bound		
	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn
Flow Rate (pcu/h)		436	1,449				470	320		937		810
Lane Group		TH	RT				LT	TH		LT		RT
Phase Number		1	1/3				2	1		3		2/3
Phasing	1 => 2 => 3											
Flow Rate in Lane Group (v) (pcu/h)		**436**	**1,449**				**470**	**320**		**937**		**810**

Saturation Flow

	North Bound			East Bound			South Bound			West Bound		
	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn
S_o: Base Sturation Flow		1,900	1,900				1,900	1,900		1,900		1,900
N: Number of Lanes		1	2				1	2		2		1
f_{HV}: Heavy-vehicle Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_G: Grade Adjustment Factor		0.999	0.999				1.001	1.001		0.998		0.998
f_P: Parking Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_{BB}: Bus Blockage Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_A: Area Type Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_{LU}: Lane Utilization Adjustment Factor		1.000	0.885				1.000	0.952		0.971		1.000
f_{LT}: Left-turn Adjustment Factor		1.000	1.000				0.950	1.000		0.950		1.000
f_{RT}: Right-turn Adjustment Factor		1.000	0.850				1.000	1.000		1.000		0.850
f_{Lpb}: Left-turn Ped/Bike Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
f_{Rpb}: Right-turn Ped/Bike Adjustment Factor		1.000	1.000				1.000	1.000		1.000		1.000
Adjusted Saturation Flow (pcu/h)		**1,898**	**2,856**				**1,807**	**3,621**		**3,497**		**1,611**

Capacity Analysis

	North Bound			East Bound			South Bound			West Bound		
	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn
Cycle Length (s)		100	100				100	100		100		100
Effective Green Time (s)		28	60				32	28		28		64
Los Time (s)		4	4				4	4		4		4
Green Ratio		0.280	0.600				0.320	0.280		0.280		0.640
Lane Group Capacity (c) (pcu/h)		531	1,713				578	1,014		979		1,031
v/c Ratio for Lane Group		**0.820**	**0.846**				**0.812**	**0.316**		**0.957**		**0.786**
Flow Ratio		0.230	0.507				0.260	0.088		0.268		0.503
Critical Lane Group/Phase			*				*					
Sum of Critical Flow Ratios	0.767											
v/c Ratio for Intersection	**0.872**											

Year 2026

Source : Study Team

4) Geometric Design

a) Design Vehicle

Trailer is used as design vehicle for intersection geometric design since much volume of trailer is expected in Tan Vu-Lach Huyen Highway which is connecting to Lach Huyen Port.

Dimension of trailer is shown in Table 7.3.1-4.

Table 7.3.1-4 Dimension of Trailer

(m)

Vehicle Type	Overall Length	Overall Width	Height	Front Overhang	Rear Overhang	Wheel Base
Trailer	16.50	2.50	4.00	1.20	2.00	4.00 / 8.80

Source : Study Team

b) Design Scale

According to 22 TCN 273-01, intersection treatment should be selected from following three types based on the traffic volume.

- Simple open throat
- Simple open throat with auxiliary lane
- Channelization

Recommended intersection treatment according to the traffic volume is shown in Table 7.3.1-5.

Table 7.3.1-5 Intersection Treatment

Turning Movement	Vehicles/h for the Design HR		
	Simple	Auxiliary Lanes	Channelization
Right	30 <	< 30 < 60	< 60
Left	30 <	< 30 < 50	< 50

Source: 22 TCN 273-01

Traffic demand forecast for Tan Vu IC in 2020 is shown in Figure 7.3.1-3.

Traffic volumes on each right turn and left turn are more than 60 vehicles/hour and more than 50 vehicles/hour, respectively.

Therefore, Tan Vu IC should be designed as “channelized intersection”.

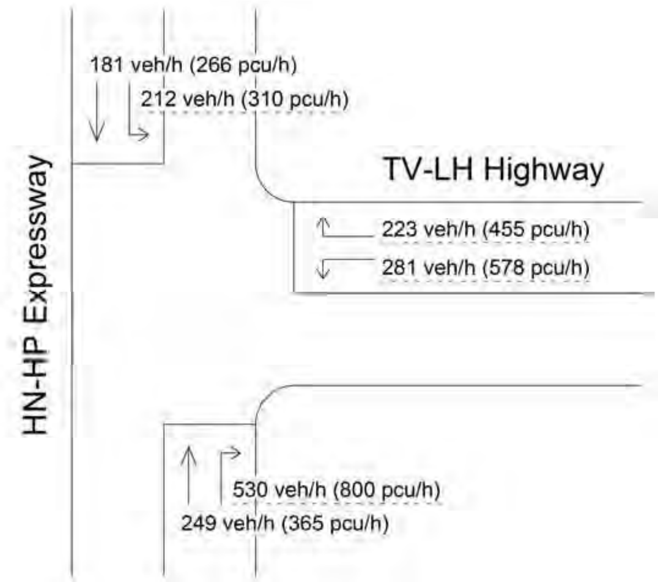


Figure 7.3.1-3 Traffic Demand Forecast for Tan Vu IC in 2020

c) Design Speed

i) Design Speed of HN - HP Expressway

120 km/h was applied as the design speed of HN - HP Expressway, since it was planned as an expressway without any at-grade intersection.

However, it is proposed to reduce the design speed from 120 km/h to 80 km/h, which is the same design speed as that of TV - LH Highway, for the Tan Vu IC area, since it will have a problem for traffic safety to run through at-grade intersection at high speed of 120 km/h.

ii) Design Speed of Ramp

It is preferable to apply higher design speed as much as possible in order to provide the smooth traffic flow at the intersection. Higher design speed, however, requires larger minimum curve radius (see Table 7.3.1-6), and therefore requires larger land acquisition as well.

It is proposed to apply 40 km/h as the design speed of ramp.

Table 7.3.1-6 Minimum Curve Radius for Right Turn Lane

Design Speed of Ramp (km/h)	30	35	40	50
Minimum Curve Radius (m)	25	30	45	80

Source: 22 TCN 273-01

d) Right Turn Lane

i) Deceleration Taper

Where number of right turn lane is 1-lane, 110 m is applied as the deceleration taper length, as shown in Table 7.3.1-7.

Table 7.3.1-7 Length of Deceleration Taper

Design Speed of Ramp (km/h)	30	35	40	50
Highway Design Speed (km/h)	-			
50	55	45	40	-
60	75	70	60	50
70	100	95	85	70
80	125	115	110	95
90	140	135	130	115
100	160	155	150	135
110	180	175	170	155

Source: 22 TCN 273-01

Since number of right turn lane is 2-lane in Tan Vu IC, 120 % of above-stated length is applied in reference to Japanese standard.

- Deceleration Taper Length: 110 m x 120 % = 132 m => 140 m

ii) Acceleration Taper

Where number of right turn lane is 1-lane, 115 m is applied as the acceleration taper length, as shown in Table 7.3.1-8.

Table 7.3.1-8 Length of Acceleration Taper

Design Speed of Ramp (km/h)	30	35	40	50
Highway Design Speed (km/h)	-			
50	40	-	-	-
60	50	50	50	-
70	90	80	70	60
80	135	125	115	85
90	-	180	170	145

Source: 22 TCN 273-01

Since number of right turn lane is 2-lane in Tan Vu IC, 120 % of above-stated length is applied in reference to Japanese standard.

- Acceleration Taper Length: 110 m x 120 % = 138 m => 140 m

iii) Ramp Design

Right turn ramp is designed with following policies according to the 22 TCN 273-01.

- Minimum curve radius: R = 45 m (for ramp design speed: 40 km/h)
- Transition curve parameter: A = 40 (for ramp design speed: 40 km/h)
- Lane width at nose position: W = 9.5 m (4.75 m x 2-lane)
- Lane width in curve: W = 10.25 m (sufficient for all vehicles including trailer)
- Nose offset: W = 1.6 m (for highway design speed: 80 km/h)
- Nose size: R = 0.6 m
- Nose taper length: L = 30 m

Summary of right turn ramp design is shown in Figure 7.3.1-4.

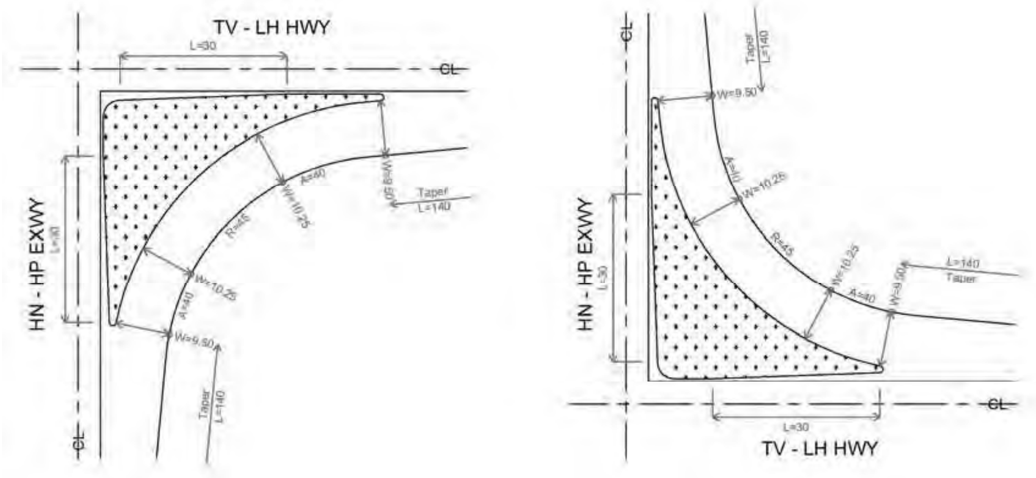


Figure 7.3.1-4 Summary of Right Turn Ramp Design

e) Left Turn Lane

i) Left Turn Storage Lane

Lengths of left turn storage lane for west bound and south bound are determined according to Figure 7.3.1-5 with following parameters.

aa) West Bound (TV - LH Highway to HN - HP Expressway)

- Traffic volume for left turn: 578 pcu/hour (see Figure 7.3.1-3)
- Cycle length: 100 s
- Number of left turn lane: 2 lanes

bb) South Bound (HN - HP Expressway to TV - LH Highway)

- Traffic volume for left turn: 310 pcu/hour (see Figure 7.3.1-3)
- Cycle length: 100 s
- Number of left turn lane: 2 lanes

In case two lanes are provided for left turn lane, it is reasonable to expect a 75 % increase in capacity for second lane.

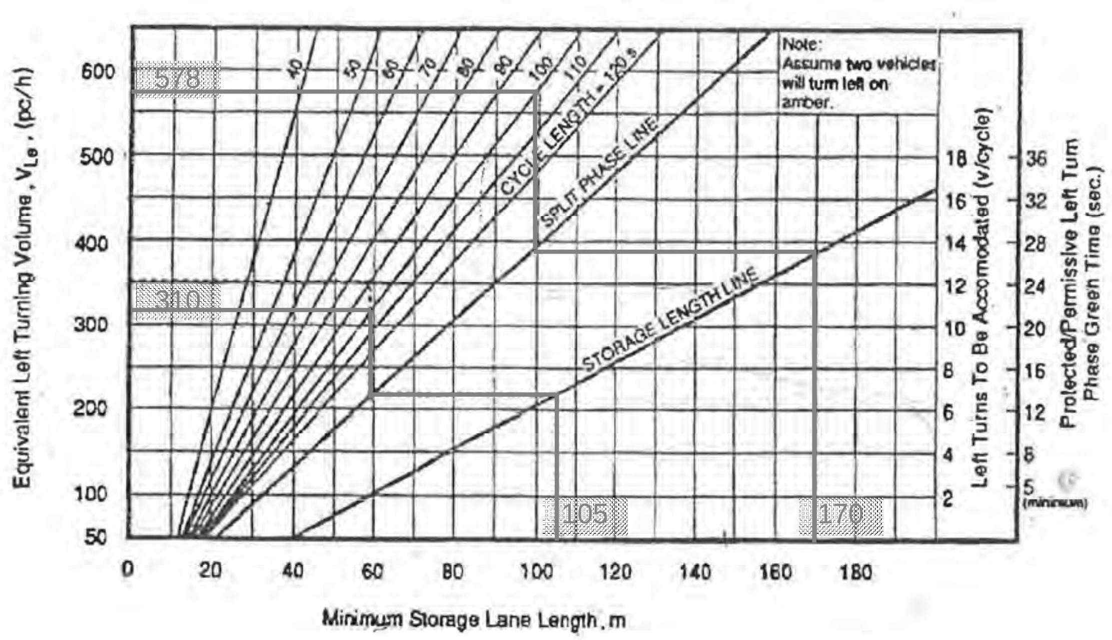
Therefore following length is applied for the left turn storage lane.

aa) West Bound (TV - LH Highway to HN - HP Expressway)

- 100 m <= 97m (97 m + 0.75 x 97 m = 170 m)

bb) South Bound (HN - HP Expressway to TV - LH Highway)

- 60 m (60 m + 0.75 x 60 m = 105 m)



Source: 22 TCN 273-01

Figure 7.3.1-5 Length of Left Turn Storage Lane

ii) Deceleration Lane

39 m and 140 m are applied as the length of taper and deceleration lane according to Table 7.3.1-9.

Table 7.3.1-9 Taper and Deceleration lane length

Highway Design Speed (km/h)	Taper	Deceleration Lane
50	28	80
60	31	100
70	35	120
80	39	140
90	42	160
100	46	180
110	49	200

Source: 22 TCN 273-01

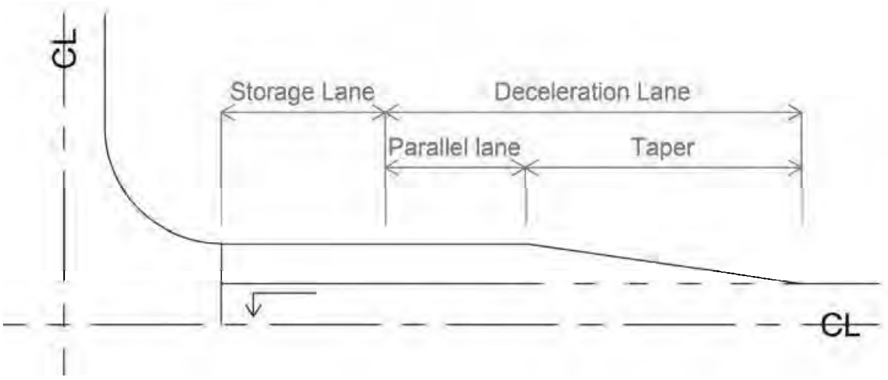


Figure 7.3.1-6 Storage Lane and Deceleration lane for Left Turn Lane

Following length is applied for taper and parallel lane in the 140 m of deceleration lane.

aa) West Bound (TV - LH Highway to HN - HP Expressway)

- No taper and parallel lane are required.

bb) South Bound (HN - HP Expressway to TV - LH Highway)

- Taper: 78 m (for 2 lane shift)
- Parallel lane: 62 m (140 - 78)

f) Turning Path

Turning pass in TV IC is shown in Figure 7.3.1-7. It was designed in consideration of trailer's smooth pass in the intersection.

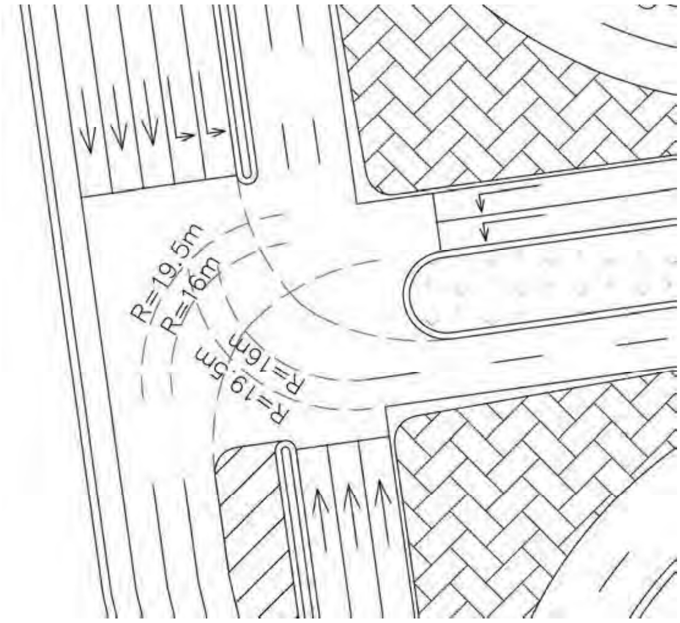


Figure 7.3.1-7 Radius of Each Turning Path

g) Plan and Typical Cross Section

Plan and typical cross section of Tan Vu IC is shown in Figure 7.3.1-8.

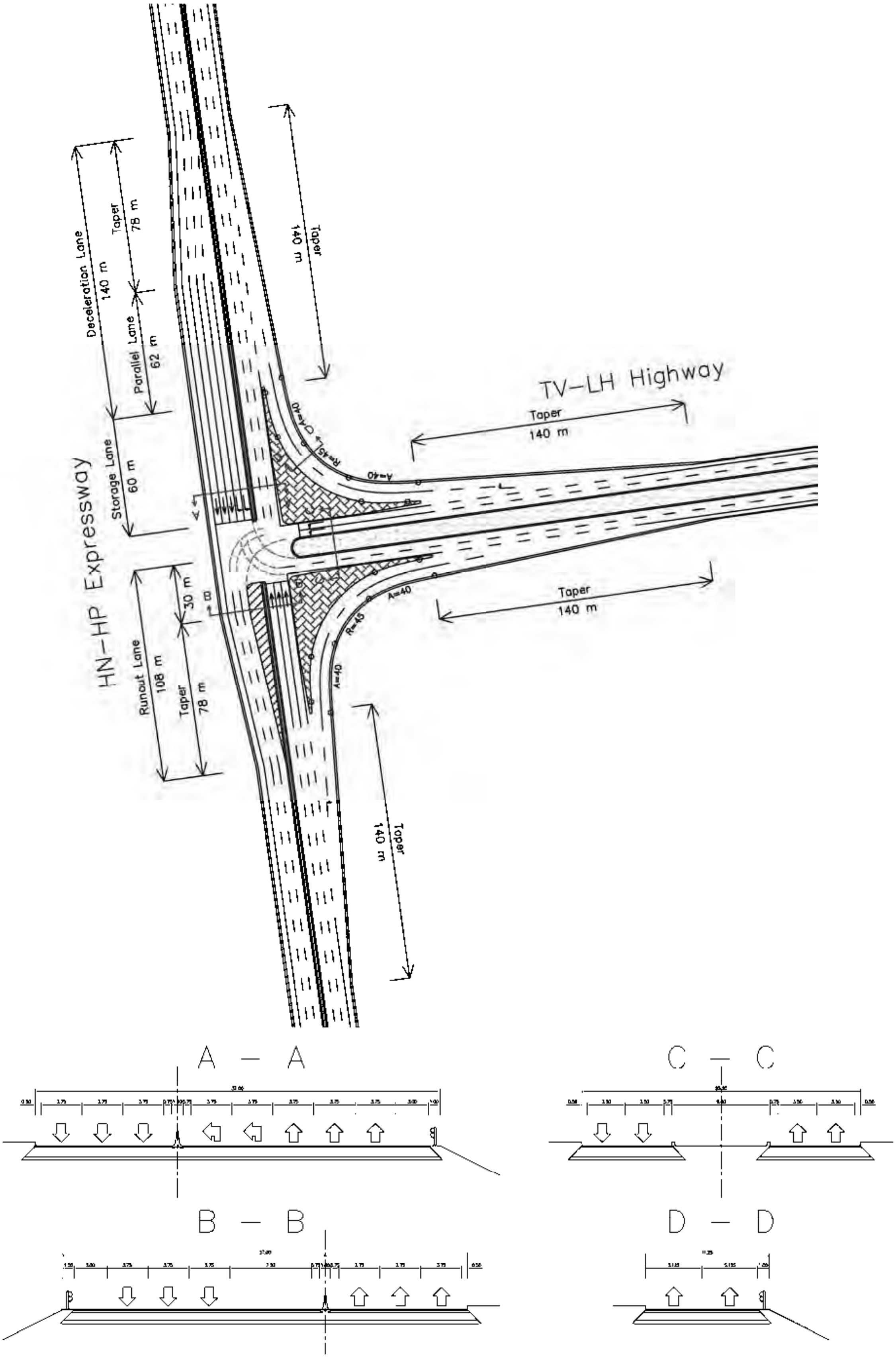


Figure 7.3.1-8 Plan and Typical Cross Section

h) Safety Plan

For the traffic safety in the intersection, road signs are installed as shown in Figure 7.3.1-9.

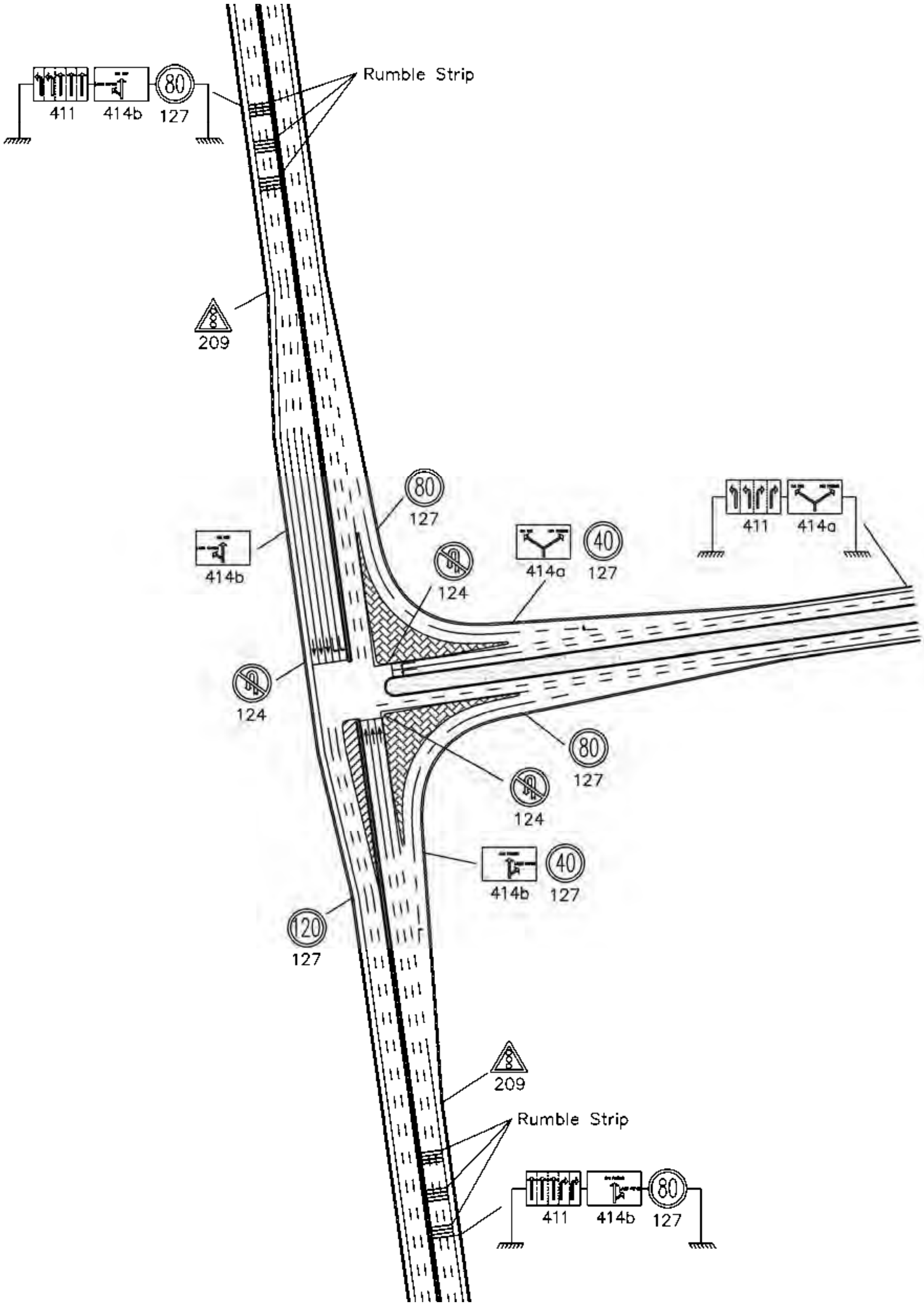


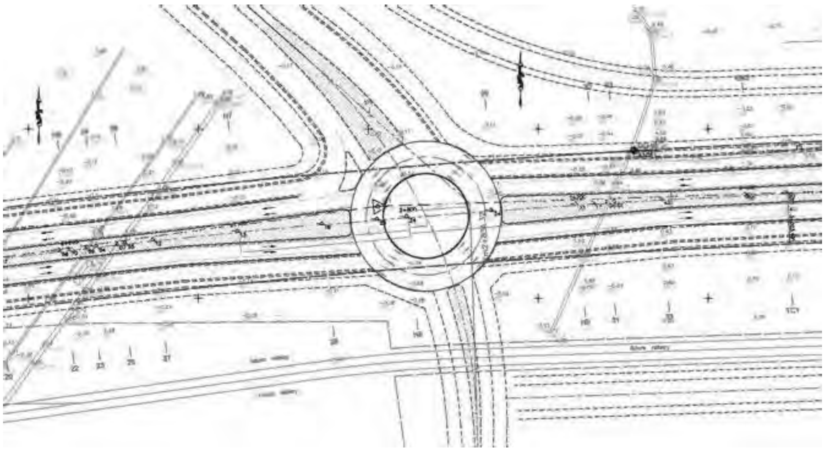
Figure 7.3.1-9 Road Sign Plan

(2) No.1 Intersection

1) Introduction

No.1 Interchange is planned in the Dinh Vu - Cat Hai Economic Zone Master Plan.

The number of lane of crossing road, i.e., Dinh Vu Ring Road, is eight lanes, and it is planned that only four lanes are connected to Tan Vu-Lach Huyen Highway through a roundabout, although remaining four lanes are connected to the frontage road of Tan Vu-Lach Huyen Highway (see Figure 7.3.1-10).



Source: SAPROF Repot (July 2010)

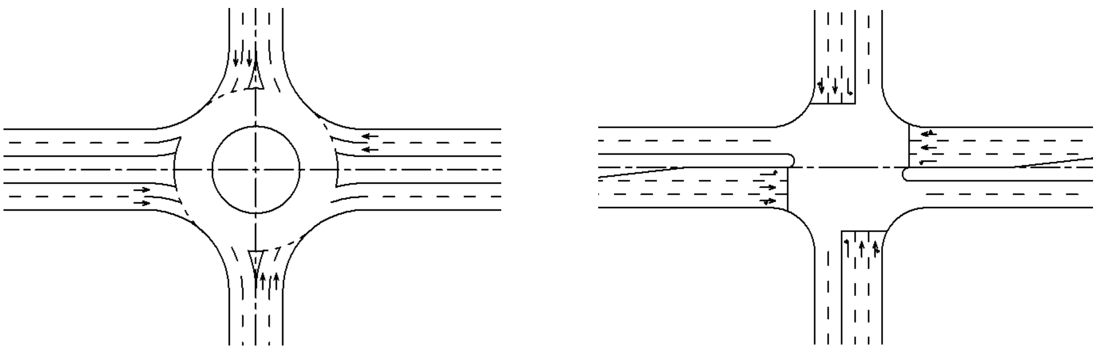
Figure 7.3.1-10 No.1 Intersection Proposed in Dinh Vu - Cat Hai Economic Zone Master Plan

The frontage road will, however, not be constructed in the Tan Vu-Lach Huyen Highway Project. Therefore, the roundabout is planned between four-lane of Tan Vu-Lach Huyen Highway and four-lane of Dinh Vu Ring Road as the initial phase of development.

2) Alternatives

Following two alternatives are studied.

(a) Roundabout (b) Signalized Intersection



Source : Study Team

Figure 7.3.1-11 Alternative Intersection types

3) Evaluation

Two alternatives are evaluated with comprehensive aspects.

Summary of evaluation is shown in Table 7.3.1-10. As a result, “Roundabout” is selected as the appropriate intersection type for No.1 Intersection at the initial stage.

Table 7.3.1-10 Evaluation of Interchange/Intersection Type for No.1 Interchange

Alternative / Evaluation Item (Basic Score)	(a) Roundabout	(b) Signalized Intersection
Description	- At grade intersection. - Two roads are connected with roundabout.	- At grade intersection. - Two roads are connected with signalized intersection.
Capacity Analysis *2 (30)	- Degree of saturation for all entries is less than 1.0 in Year 2015 and 2020. - Enough traffic capacity is secured. (24, Moderate)	- v/c ratio for all lane groups and intersection itself are less than 1.0 in Year 2015. - However, v/c ratio for some lane groups and intersection itself exceed 1.0 in Year 2020. - Enough traffic capacity is not secured in Year 202. (15, Poor)
Construction Cost (20)	- Cheap since only earth work is made. (20, Good)	- Cheap since only earth work is made. (20, Good)
Safety (20)	- Safety is secured by installing traffic signs to inform drivers of the existence of roundabout before entering roundabout. (16, Moderate)	- Each traffic is separated by signal phasing. (20, Good)
Land Acquisition (20)	- No additional land acquisition is required. (20, Good)	- No additional land acquisition is required. (20, Good)
Future Upgrade (10)	- Easy since only earthwork is made at initial stage. (8, Moderate)	- Easy since only earthwork is made at initial stage. (8, Moderate)
Recommendation (100) *1	Recommended (88)	(83)

Note: *1: Score is basic score x evaluation (Good=1.0, Moderate=0.8, Poor=0.5).

*2: Detailed capacity analysis is shown in Table 7.3.1-11.

Source : Study Team

Table 7.3.1-11 Capacity Analysis of No.1 Interchange

(a)(a) Roundabout

Year 2015

Traffic Volume

Entry Flow				
	North Bound	East Bound	South Bound	West Bound
Left Turn	36	62	748	33
Through	101	163	101	150
Right Turn	32	212	142	728
Total of Entry Flow (D) (pcu/h)	169	437	991	911

Circulating Flow				
	NB Circ	EB Circ	SB Circ	WB Circ
Circulating Flow (pcu/h)	973	882	219	199

Capacity

	North Bound	East Bound	South Bound	West Bound
Input from Geometric Design				
e: Entry Width (m)	12.56	14.86	14.46	14.12
v: Approach Half Width (m)	7.50	9.75	7.50	9.75
l': Effective Flare Length (m)	32.66	45.30	52.10	40.30
D: Inscribed Circle Diameter (m)	100.00	100.00	100.00	100.00
ø: Enty Angle (Degrees)	35	29	32	31
r: Entry Radius (m)	80.00	100.00	80.00	90.00
Entry Capacity (C) (pcu/h)	2,694	3,545	3,695	3,916

Degree of Saturation

	North Bound	East Bound	South Bound	West Bound
Entry Flow / Entry Capacity (D/C)	**0.06**	**0.12**	**0.27**	**0.23**

Year 2015

Year 2020

Traffic Volume

Entry Flow				
	North Bound	East Bound	South Bound	West Bound
Left Turn	77	167	843	85
Through	227	654	227	654
Right Turn	73	290	453	835
Total of Entry Flow (D) (pcu/h)	377	1,111	1,523	1,574

Circulating Flow				
	NB Circ	EB Circ	SB Circ	WB Circ
Circulating Flow (pcu/h)	1,664	1,155	816	471

Capacity

	North Bound	East Bound	South Bound	West Bound
Input from Geometric Design				
e: Entry Width (m)	12.56	14.86	14.46	14.12
v: Approach Half Width (m)	7.50	9.75	7.50	9.75
l': Effective Flare Length (m)	32.66	45.30	52.10	40.30
D: Inscribed Circle Diameter (m)	100.00	100.00	100.00	100.00
ø: Enty Angle (Degrees)	35	29	32	31
r: Entry Radius (m)	80.00	100.00	80.00	90.00
Entry Capacity (C) (pcu/h)	2,220	3,322	3,243	3,702

Degree of Saturation

	North Bound	East Bound	South Bound	West Bound
Entry Flow / Entry Capacity (D/C)	**0.17**	**0.33**	**0.47**	**0.43**

Year 2020

(b) Signalized Intersection

Traffic Volume

	North Bound			East Bound			South Bound			West Bound		
	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn
Flow Rate (pcu/h)	36	101	32	62	163	212	748	101	142	33	150	728
Lane Group	LT	LT	TH+RT	LT	LT	TH+RT	LT	LT	TH+RT	LT	LT	TH+RT
Phase Number	2	1	1	4	3	3	2	1	1	4	3	3
Phasing	1 =>		2 =>		3 =>		4					
Flow Rate in Lane Group (v) (pcu/h)	**36**		**133**	**62**		**375**	**748**		**243**	**33**		**878**

Saturation Flow

S_o: Base Sturation Flow	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900
N: Number of Lanes	1	1	2	1	1	2	1	1	2	1	1	2
f_{HV}: Heavy-vehicle Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_G: Grade Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_P: Parking Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_{BB}: Bus Blockage Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_A: Area Type Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_{LU}: Lane Utilization Adjustment Factor	1.000	0.952	0.952	1.000	0.952	0.952	1.000	0.952	0.952	1.000	0.952	0.952
f_{LT}: Left-turn Adjustment Factor	0.950	0.456	1.000	0.950	0.125	1.000	0.950	0.583	1.000	0.950	0.383	1.000
f_{RT}: Right-turn Adjustment Factor	1.000	0.964	0.964	1.000	0.915	0.915	1.000	0.912	0.912	1.000	0.876	0.876
f_{Lpb}: Left-turn Ped/Bike Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_{Rpb}: Right-turn Ped/Bike Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Adjusted Saturation Flow (pcu/h)	**1,805**	**795**	**3,487**	**1,805**	**207**	**3,311**	**1,805**	**961**	**3,301**	**1,805**	**607**	**3,168**

Capacity Analysis

Cycle Length (s)	100	100	100	100	100	100	100	100	100	100	100	100
Effective Green Time (s)	29	21	21	5	28	28	29	21	21	5	28	28
Los Time (s)	4	4	4	4	4	4	4	4	4	4	4	4
Green Ratio	0.290	0.250	0.210	0.050	0.320	0.280	0.290	0.250	0.210	0.050	0.320	0.280
Lane Group Capacity (c) (pcu/h)	523	199	732	90	66	927	523	240	693	90	194	887
v/c Ratio for Lane Group	**0.069**	**0.000**	**0.182**	**0.687**	**0.000**	**0.405**	**1.000**	**0.934**	**0.351**	**0.366**	**0.000**	**0.990**

Flow Ratio	0.020	0.000	0.038	0.034	0.000	0.113	0.290	0.234	0.074	0.018	0.000	0.277
Critical Lane Group/Phase				*			*	*				*
Sum of Critical Flow Ratios	0.835											
v/c Ratio for Intersection	**0.949**											

Year 2015

Traffic Volume

	North Bound			East Bound			South Bound			West Bound		
	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn	Left Turn	Through	Right Turn
Flow Rate (pcu/h)	77	227	73	167	654	290	843	227	453	85	654	835
Lane Group	LT	LT	TH+RT	LT	LT	TH+RT	LT	LT	TH+RT	LT	LT	TH+RT
Phase Number	2	1	1	4	3	3	2	1	1	4	3	3
Phasing	1 =>		2 =>		3 =>		4					
Flow Rate in Lane Group (v) (pcu/h)	**77**		**300**	**167**		**944**	**843**		**680**	**85**		**1,489**

Saturation Flow

S_o: Base Sturation Flow	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900
N: Number of Lanes	1	1	2	1	1	2	1	1	2	1	1	2
f_{HV}: Heavy-vehicle Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_G: Grade Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_P: Parking Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_{BB}: Bus Blockage Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_A: Area Type Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_{LU}: Lane Utilization Adjustment Factor	1.000	0.952	0.952	1.000	0.952	0.952	1.000	0.952	0.952	1.000	0.952	0.952
f_{LT}: Left-turn Adjustment Factor	0.950	0.160	1.000	0.950	0.125	1.000	0.950	0.397	1.000	0.950	0.125	1.000
f_{RT}: Right-turn Adjustment Factor	1.000	0.964	0.964	1.000	0.954	0.954	1.000	0.900	0.900	1.000	0.916	0.916
f_{Lpb}: Left-turn Ped/Bike Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
f_{Rpb}: Right-turn Ped/Bike Adjustment Factor	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Adjusted Saturation Flow (pcu/h)	**1,805**	**279**	**3,486**	**1,805**	**216**	**3,451**	**1,805**	**646**	**3,256**	**1,805**	**207**	**3,313**

Capacity Analysis

Cycle Length (s)	100	100	100	100	100	100	100	100	100	100	100	100
Effective Green Time (s)	29	21	21	5	28	28	29	21	21	5	28	28
Los Time (s)	4	4	4	4	4	4	4	4	4	4	4	4
Green Ratio	0.290	0.250	0.210	0.050	0.320	0.280	0.290	0.250	0.210	0.050	0.320	0.280
Lane Group Capacity (c) (pcu/h)	523	70	732	90	69	966	523	161	684	90	66	928
v/c Ratio for Lane Group	**0.147**	**0.000**	**0.410**	**1.000**	**1.112**	**0.977**	**1.000**	**1.980**	**0.994**	**0.942**	**0.000**	**1.605**

Flow Ratio	0.043	0.000	0.086	0.050	0.356	0.274	0.290	0.495	0.209	0.047	0.000	0.449
Critical Lane Group/Phase				*			*	*				*
Sum of Critical Flow Ratios	1.284											
v/c Ratio for Intersection	**1.459**											

Year 2020

Source : Study Team

4) Geometric Design

a) Design Speed

Designing the geometry of a roundabout involves choosing between trade-offs of safety and capacity.

Roundabouts operate most safely when their geometry forces traffic to enter and circulate at slow speeds. Horizontal curvature and narrow pavement widths are used to produce this reduced-speed environment.

Conversely, the capacity of roundabouts is negatively affected by these low-speed design elements. As the widths and radius of entry and circulatory roadways are reduced, so also the capacity of the roundabout is reduced.

For No.1 Intersection, higher design speed should be applied with ensuring safety so that Tan Vu-Lach Huyen Highway keeps higher service level. 50 km/h of design speed is adopted for No.1 Intersection, which is the highest design speed proposed in “Roundabouts An Informational Guide, Federal Highway Administration (FHWA), USA, 2000”.

Table 7.3.1-12 Design Speed of No.1 Intersection

Site Category	Entry Design Speed
Mini-Roundabout	25 km/h
Urban Compact	25 km/h
Urban Single Lane	35 km/h
Urban Double Lane	40 km/h
Rural Single Lane	40 km/h
Rural Double Lane	**50 km/h**

Source: Roundabouts An Informational Guide, Federal Highway Administration (FHWA), USA, 2000

b) Design Vehicle

Trailer is used as design vehicle for intersection geometric design since much volume of trailer is expected in Tan Vu-Lach Huyen Highway which is connecting to Lach Huyen Port.

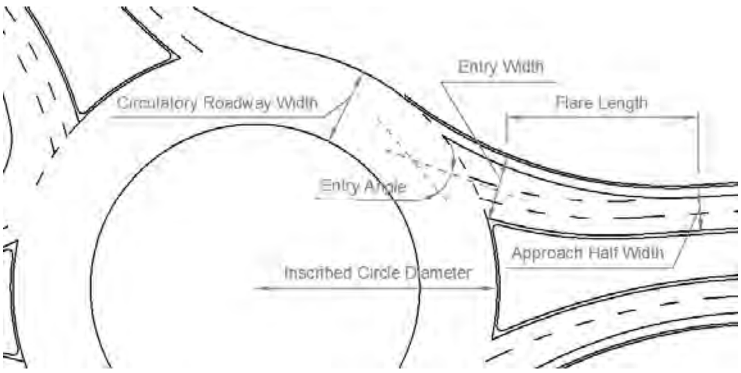
c) Geometric Elements and Capacity

The geometric elements of the roundabout affect the capacity of entry flow.

The most important geometric element is the widths of the entry and circulatory roadways, or the number of lanes at the entry and on the roundabout.

Two entry lanes permit nearly twice the rate of entry flow as does one lane. Wider circulatory roadways allow vehicles to travel alongside, and so provide longer gaps between bunches of vehicles.

The flare length also affects the capacity. The inscribed circle diameter and the entry angle have minor effects on capacity.



Source : Study Team

Figure 7.3.1-12 Geometric Elements

d) Inscribed Circle Diameter

In general, smaller inscribed diameters are better for overall safety since they help to maintain lower speeds.

However, in high-speed environments, larger inscribed diameters generally allow for the provision of better approach geometry, which leads to a decrease in vehicle approach speeds. Larger inscribed diameters also reduce the angle formed between entering and circulating vehicle paths, thereby reducing the relative speed between these vehicles and leading to reduced entering-circulating crash rates.

Therefore, roundabouts in high-speed environments such as No.1 Intersection may require diameters that are somewhat larger than those for low-speed environments.

However, very large diameters should not be used since they will have high circulating speeds and more crashes with greater severity.

100 m of inscribed circle diameter is adopted for No.1 Intersection, which is maximum value recommended in “Design Manual for Roads and Bridges: Volume 6 Road Geometry, Department for Transport, UK, 2006”.

e) Entry Width

Entry width is the largest determinant of a roundabout’s capacity. The capacity of an approach is not dependent only on the number of entering lanes, but on the total width of the entry. In other words, the entry capacity increases steadily with incremental increases to the entry width.

On the other hand, larger entry width increases crash frequency. To maximize the roundabout’s safety, entry widths should be kept to a minimum.

The entry width involves a trade-off between capacity and safety. The design should provide the minimum width necessary for capacity and accommodation of the design vehicle in order to maintain the highest level of safety.

The entry width on No.1 Intersection is determined based on the capacity analysis with attention to safety.

Table 7.3.1-13 shows the adopted entry width for each entry.

Table 7.3.1-13 Entry Width for No.1 Intersection

Bound	Entry Width (m)
North Bound	12.6
East Bound	14.9
South Bound	14.5
West Bound	14,1

Source : Study Team

f) Circulatory Roadway Width

The required circulatory roadway width is determined from the width of the entries and the turning requirements of the design vehicle.

It should always be as wide as the maximum entry width and up to 120 percent of the maximum entry width, and should remain constant throughout the roundabout.

Circulatory roadway width on No.1 Intersection is determined as 16 m, which is 107 percent of the maximum entry width, based on the capacity analysis with attention to safety.

g) Flare Length

Flaring is an effective means of increasing capacity since entry width becomes wider by providing flaring, and it does not require additional right of way in full lane.

The flare length on No.1 Intersection is determined based on the capacity analysis with attention to safety.

Table 7.3.1-14 shows the adopted flare length for each entry.

Table 7.3.1-14 Flare Length for No.1 Intersection

Bound	Flare Length (m)
North Bound	33
East Bound	45
South Bound	52
West Bound	40

Source : Study Team

h) Entry Angle

Entry angle should lie between 20 and 60 degrees.

Low entry angles force drivers to look over their shoulders or use their mirrors to merge with circulating traffic.

Large entry angles tend to have lower capacity and may produce excessive entry deflection which can lead to sharp braking at entries, accompanied by shunt accidents, especially when approach speeds are high.

The entry angle on No.1 Intersection is determined based on the capacity analysis with attention to safety.

Table 7.3.1-15 shows the adopted entry angle for each entry.

Table 7.3.1-15 Entry Angle for No.1 Intersection

Bound	Entry Angle (degrees)
North Bound	35
East Bound	29
South Bound	32
West Bound	31

Source : Study Team

(3) Local Intersection at Km11+520

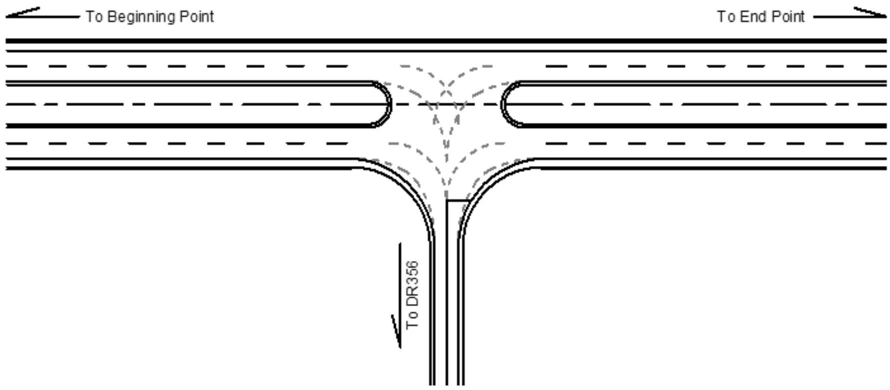
By the request from Cat Hai District People's Committee, three-leg intersection is constructed at Km11+520, between Tan Vu-Lach Huyen Highway and new north-south road, which will be constructed by Haiphong City.

Geometric elements of the new road are as follows.

- Design Category:V
- Design Speed:40 Km/h
- Number of Lane:2
- Cross Section:7.5 m (2 x 2.75 + 2 x 1.00)

This intersection is planned as unsignalized intersection since there will be not much traffic in this intersection.

The plan of the intersection is shown in Figure 7.3.1-13.



Source : Study Team

Figure 7.3.1-13 Plan of Local Intersection at Km11+520

(4) Local Intersection at Km15+576

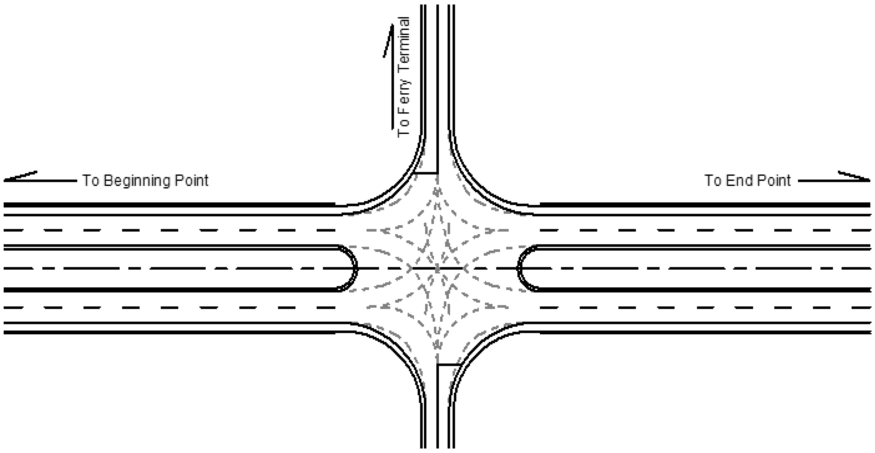
By the request from Cat Hai District People's Committee, four-leg intersection is constructed at Km11+576, between Tan Vu-Lach Huyen Highway and District Road No. 356, which will be replaced by Haiphong City since the existing road is located in the future port area.

Geometric elements of the replaced District Road No. 356 are as follows.

- Design Category:V
- Design Speed:40 Km/h
- Number of Lane:2
- Cross Section:9.0 m (2 x 3.50 + 2 x 1.00)

This intersection is planned as unsignalized intersection since there will be not much traffic in this intersection.

The plan of the intersection is shown in Figure 7.3.1-14.



Source : Study Team

Figure 7.3.1-14 Plan of Local Intersection at Km11+576

7.4 DRAINAGE DESIGN

7.4.1 Road Surface Drainage

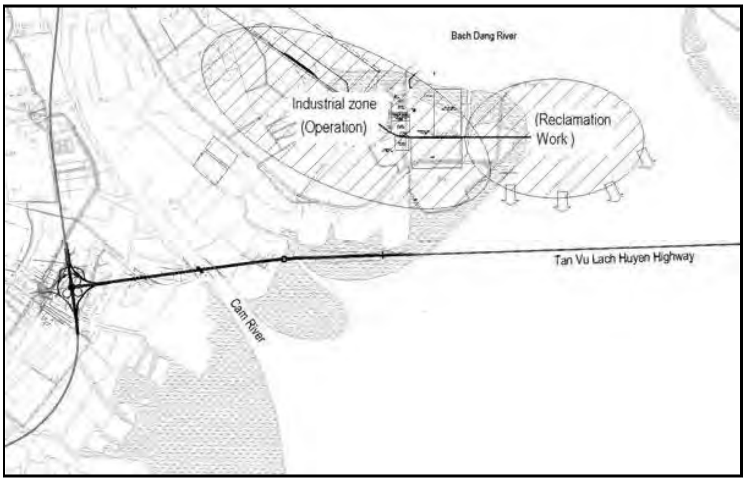
7.4.1.1 Design Policies and Conditions

The drainage design is executed based on the following policies and conditions:

- ➢TCVN 4054-2005
- ➢22 TCN 220 - 95

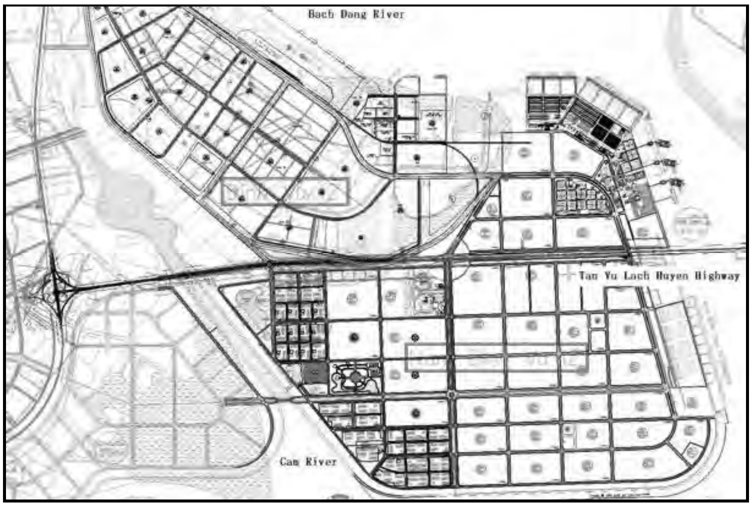
Scope of work in this project is "Phase-1 construction" only. "Phase-1 construction" is the 1st step of road service, and drainage system should be covered "future stage" generally. However, site condition in Hai An section is assumed to have large difference between "Phase-1 construction" and "future stage". Because land reclamation for industrial zone will be continuing, when Tan Vu Lach Huyen Highway opens road service.

Next figures show envisioned status in Hai An section, times of TVLH highway service begins and completes all.



Source : Study Team

Figure 7.4.1-1 Envisioned Status of Hai An Section (Phase-1 construction is completed)



Source : Study Team

Figure 7.4.1-2 Envisioned Status of Hai An Section (Future stage)

Land reclamation for industrial zone will complete more than 10 years after “Phase-1 construction” will have finished.

Thus drainage system in Hai An section should be considered “Phase-1 construction”. and drainage design is aimed hatching portion in the table below.

Table 7.4.1-1 Existing and Anticipated Main Land Use along Tan Vu Lach Huyen Highway

Stage	Hai An section	Cat Hai section
Present	Shrimp ponds	Salt farms, Shrimp ponds, Houses
Phase-1 construction	Shrimp ponds	Salt farms, Shrimp ponds, Houses
Future stage	Industrial zone	Salt farm, Shrimp pond (There is no definitive master plan currently.)

Source : Study Team

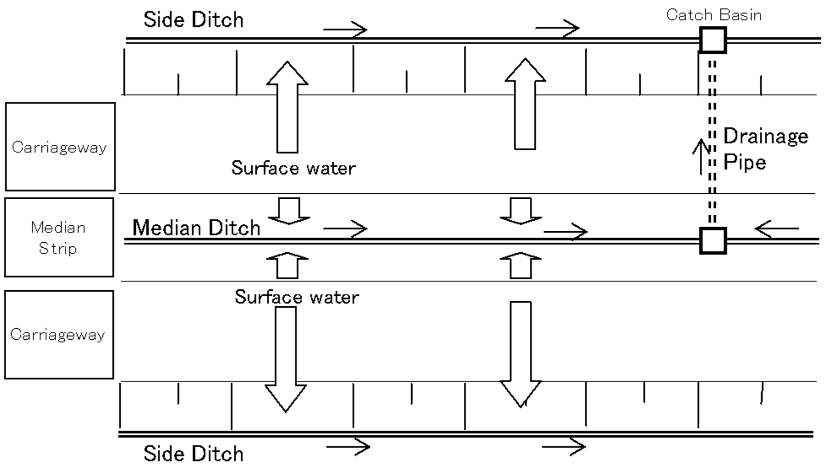
Drainage system in Hai An section should be respond to change depending on the stage of the construction of industrial zone.

7.4.1.2 Discharge of Drainage

Discharge spots for drainage should be avoid flowing down directly to shrimp ponds and salt farms, when considering the impact for environment and land users. So side ditches are set beside slope toe of highway and they lead surface water to channels, rivers and sea.

7.4.1.3 Drainage System

Catchment area is divided carriageway and median and they has ditches at both side and median as a general rule. Typical drainage system is shown in figure below.



Source : Study Team

Figure 7.4.1-3 Typical Drainage System

The future time highway upgrade to 6 lanes, this system should be reconsidered because of reduction of median width.

7.4.1.4 Method of Drainage Design

Method of drainage design to decide size of ditches is followed “22- TCN 220-95”.

The formula of drainage design is applied the followings:

(i) The formula is applied for the flow:

$$Qp = Ap\times\psi\times Hp\times F\times\delta 1$$

Where:

Qp : Design flow, m3/s

AP : flood peak module corresponding to the design frequency

φ : flood flow coefficient

HP : the maximum daily rainfall corresponding to the design frequency P%, mm

F : catchment area, Km2

δ_1 : decrease coefficient by ponds and lakes

(ii) Calculation of Ditch Drainage

$$Qo = (1/n)\times A\times R2/3\times I1/2$$

Where:

Qo : ditch drainage capacity, m3/s

A : ditch sectional area, m2

R : hydraulic radius, m

I : hydraulic gradient

Checking flow

+ If Qo >= Qp => sufficient capacity for drainage

+ If Qo < Qp => Insufficient capacity for drainage

Calculation results show the appendix.

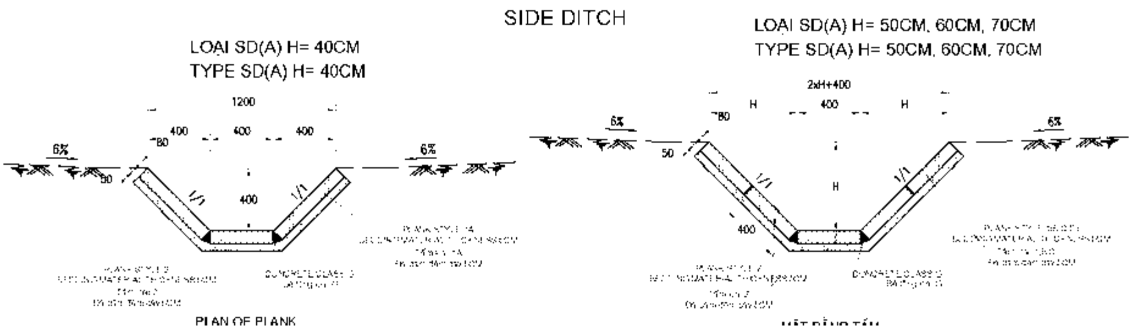
7.4.1.5 Drainage Items

Drainage system is covered the following drainage items.

(1) Side Ditch

Longitudinal side ditches at toe of slope

Sizes (depth) of ditches are determined by drainage design calculation. Minimum depth is 40cm.



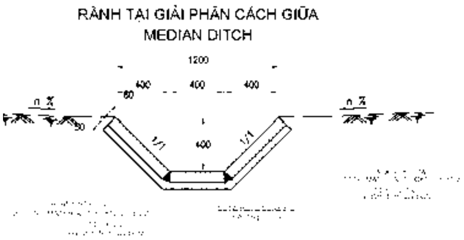
Source : Study Team

Figure 7.4.1-4 Side Ditch

(2) Median Ditch

Longitudinal median ditch is installed in median strip

Depth of ditch is fixed 40cm.Gradient of ditch is controlled by cross fall of median surface soil.



Source : Study Team

Figure 7.4.1-5 Median Ditch

(3) Crossing Drainage Pipe

Crossing drainage pipe is set to flow down surface water from median ditch. Diameter of pipe is determined as follows.

Table 7.4.1-2 Drainage Pipe List

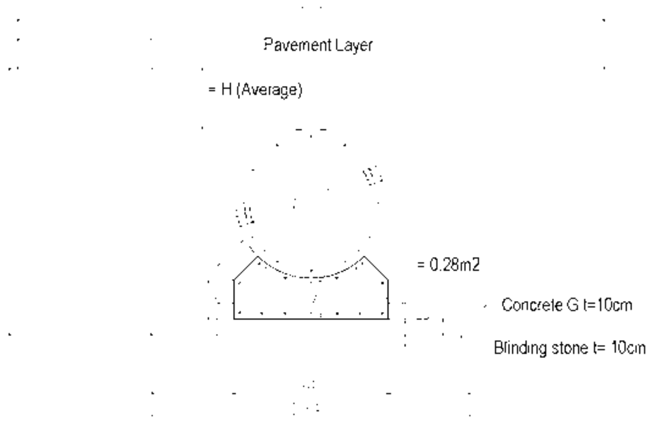
Hai An

No	Location			Size D (m)	Length (m)	Skew (Degree)	Remarks
	Km..	**+**	**.....**				
1	Km1	+	000	0.75	15.34	90.00	Drainage culvert
2	Km1	+	450	0.75	13.77	90.00	Drainage culvert
3	Km1	+	960	0.75	13.85	90.00	Drainage culvert
4	Km2	+	360	0.75	14.86	90.00	Drainage culvert
5	Km2	+	775	1.00	19.50	-	Drainage culvert
6	Km2	+	832	1.00	19.50	-	Drainage culvert
7	Km3	+	290	1.00	22.76	90.00	Drainage culvert
8	Km4	+	075	1.00	20.56	90.00	Drainage culvert

Cat Hai

No	Location			Size D (m)	Length (m)	Skew (Degree)	Remarks
	Km..	**+**	**.....**				
1	Km10	+	061	0.75	16.00	-	Irrigation culvert
2	Km11	+	310	0.75	13.50	90.00	Drainage culvert
3	Km12	+	560L	1.00	15.00	90.00	Drainage culvert
4	Km12	+	560R	1.00	16.00	90.00	Drainage culvert
5	Km13	+	230(1)	0.75	14.40	90.00	Drainage culvert
6	Km13	+	230(2)	0.75	5.00	90.00	Drainage culvert
7	Km14	+	250	0.75	14.40	90.00	Drainage culvert
8	Km14	+	600	0.75	14.50	90.00	Drainage culvert
9	Km14	+	700	0.75	14.50	90.00	Drainage culvert
10	Km15	+	320	0.75	14.50	90.00	Drainage culvert
11	Km15	+	552	0.75	13.10	90.00	Drainage culvert
12	Km15	+	568R	0.75	14.00	-	Drainage culvert
13	Km15	+	569L	0.75	14.00	-	Drainage culvert
14	Km15	+	599	0.75	13.10	90.00	Drainage culvert

Source : Study Team



Source : Study Team

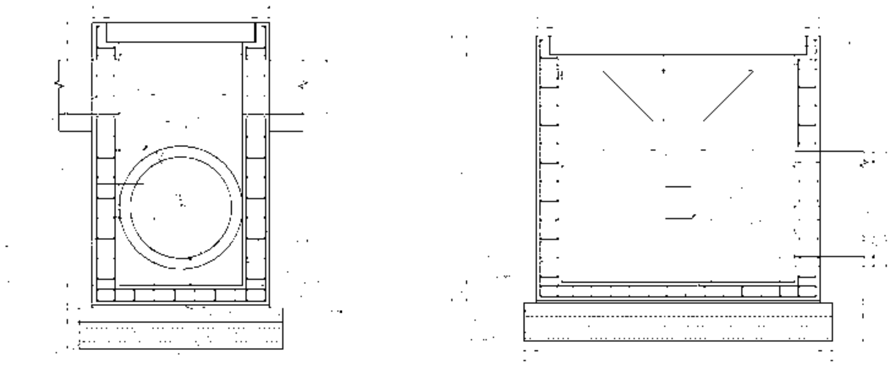
Figure 7.4.1-6 Drainage Pipe (D=0.75)

(4) Catch Basin

Types of catch basins are named below generally.

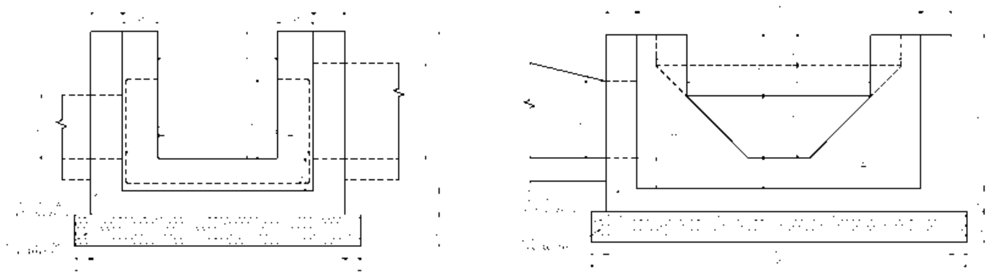
Catch Basin in median strip -------- Catch Basin Type A

Catch Basin at road side --------- Catch Basin Type B



Source : Study Team

Figure 7.4.1-7 Catch Basin Type A1



Source : Study Team

Figure 7.4.1-8 Catch Basin Type B1

Catch Basin Type B1 and B2 have no reinforcement bar, because locations of them have no impact from traffic load and soil pressure. And they do not have any catch basin covers the reason why is, their depth of hole is not deep.

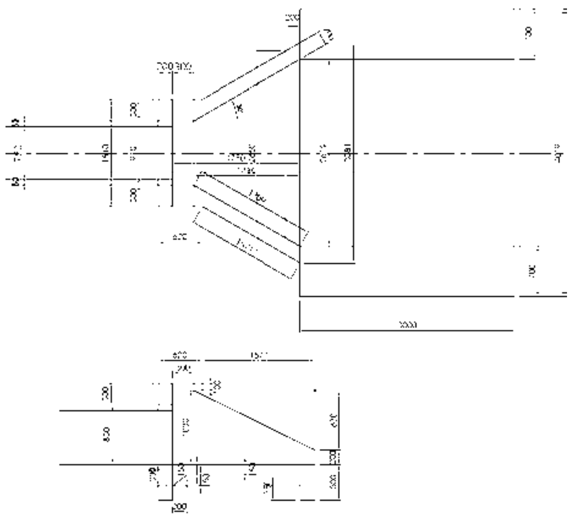
(5) Outlet of Drainage Pipe

Types of outlet drainage pipe are defined table below.

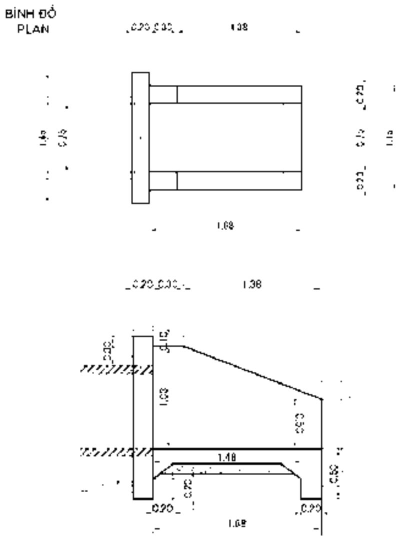
Table 7.4.1-3 Types of Drainage Pipe Outlet

	Case	Remark
Outlet Type A	Flow to river (channel) directly	Normal outlet
Outlet Type B	Flow to river (channel) directly	Between pipe and existing river(channel) is grate fall
Outlet Type C	Flow to Catch basin connected side ditch	Wing wall is parallel to connect catch basin

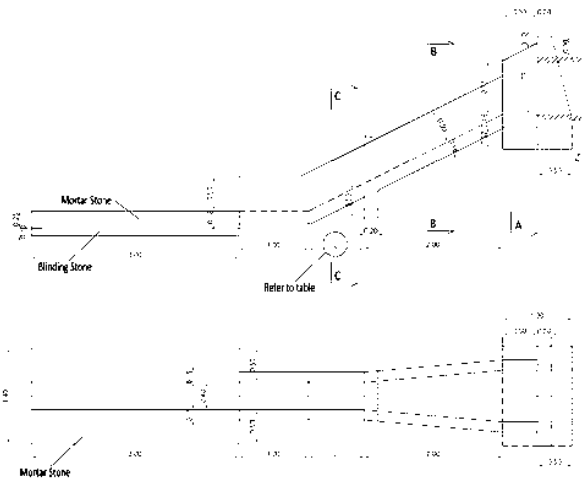
Source : Study Team



Outlet Type A



Outlet Type C



Source : Study Team

Outlet Type B

Figure 7.4.1-9 Outlet Type A , B , C

7.4.2 Irrigation

7.4.2.1 Irrigation Culvert

Function of Irrigation Culverts is classified 4 types below.

(1) Crossing River and Channel

(2) Connect between pond and pond

(3) Navigation

(4) Water Gate Relocation

Case “(2)” is mainly used in this project.

(1) Crossing River and Channel

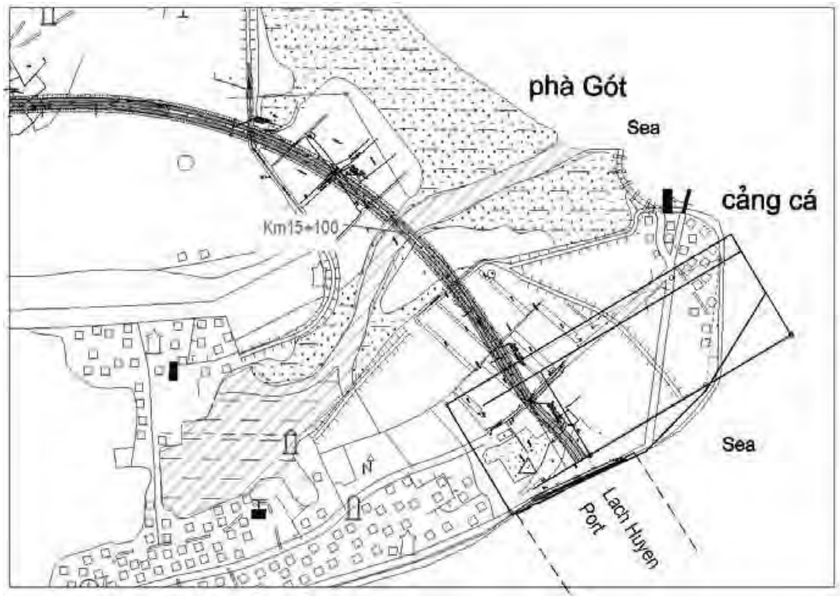
Culvert are generally set and connects river and channel which are divided by TVLH Highway.

(2) Connect between pond and pond

This type is connection of shrimp ponds divided by TVLH Highway. Culverts in Hai An section will be unnecessary, if land reclamation work start along the TVLH Highway, or land user renounce work of shrimp aquaculture industry.

(3) Navigation

There are ships crossing at the location of Km15+100. Therefore culvert at this location should have function for navigation. Dimension of height is changed from into 6m based on request from Cat Hai People’s Committee.



Source : Study Team

Figure 7.4.2-1 Navigation Area at Km15+100

(4) Water Gate Relocation

It is located Km 14+670, this point has existing water gate at center of T.L.Highway. This water gate is used to abstract water for shrimp pond. Thus water gate should be relocated for pond after construction of Highway. Location of new water gate is selected left side of highway (Km14+650).

Irrigation culvert according to review of F/S and site reconnaissance are as follows. And Table 7.4.2-2 is comparison between F/S and D/D.

Table 7.4.2-1 List of Irrigation Culvert

No	D/D Stage										Installation purpose
	Location			Direction of water	Type	Size			Angle deg.	Length m	
	Km..	+	....			No	F (B)	H			
	Irrigation Culvert										
	Hai An district										
1	Km0	+	225.00		Pipe		1.50		115	39.820	Ditch
2	Km0	+	788.00	R-L	Box		2.00 x	2.00	120	40.497	Shrimp pond
3	Km0	+	915.00	L-R	Box		3.00 x	3.00	60	39.426	Channel
4	Km2	+	390.00	L-R	Box		2.00 x	2.00	120	44.824	Shrimp pond
5	Km2	+	650.00		Box		2.00 x	2.00	120	46.833	Shrimp pond
6	Km4	+	140.00	L-R	Box	2 x	3.00 x	3.00	60	37.326	Channel and Sea
	Cat Hai district										
7	Km10	+	90.00	R-L	Pipe		1.50		90	48.280	Ditch
8	Km10	+	659.00		Pipe		1.50		100	43.270	Ditch
9	Km10	+	805.00	L-R	Box		3.00 x	3.00	60	42.475	Channel
10	Km14	+	620.00	L-R	Box		4.00 x	4.00	90	33.941	Pond
11	Km14	+	650.00	No Crossing	Box	2 x	2.00 x	4.00	90	4.094	Watergate for pond(Out of Highway)
12	Km14	+	907.00		Box		2.00 x	2.00	105	40.156	Shrimp pond
13	Km15	+	100.00		Box	3 x	4.00 x	6.00	90	33.773	Channel and Navigation
14	Km15	+	520.00	R-L	Pipe		1.50		75	36.220	Ditch

- Function of Culverts in Hai An district will finish working by the reclamation of industrial zone.

Source : Study Team

Table 7.4.2-2 Comparison Irrigation Culverts between F/S and D/D

No	F/S study					D/D Stage					States	Reason
	Location		Size			Location		Size				
	Km.. + ...	Type	No	F (B)	H	Km.. + ...	Type	No	F (B)	H		
Hai An District												
1	-					Km0 + 225.00	Pipe		1.50		new	request from Trang Cat Commune
2	-					Km0 + 788.00	Box		2.00	x 2.00	new	request from Trang Cat Commune
3	Km0 + 950.00	Box		3.00	x 3.00	Km0 + 915.00	Box		3.00	x 3.00	location changed	request from Trang Cat Commune
4	Km1 + 698.00	Box	8 x	4.00	x 4.00	Km1 + 700.00	Bridge				changed	request from Hai Phong city
5	Km2 + 390.00	Pipe		2.00		Km2 + 390.00	Box		2.00	x 2.00	changed	request from Dong Hai 2 Commune
6	-					Km2 + 650.00	Box		2.00	x 2.00	new	request from Dong Hai 2 Commune
7	Km4 + 100.00	Box	3 x	4.00	x 4.00	Km4 + 140.00	Box	2 x	3.00	x 3.00	changed	request from Dong Hai 2 Commune
Cat Hai District												
8	Km10 + 58.30	Pipe		1.25		Km10 + 90.00	Pipe		1.50		changed	Consultant's judgement based on site visit
9	Km10 + 400.00	Pipe		1.25		-					deleted	Consultant's judgement based on site visit
10	Km10 + 659.00	Pipe		1.25		Km10 + 659.00	Pipe		1.50		size changed	Length more than 30m
11	Km10 + 818.00	Box	2 x	4.00	x 3.00	Km10 + 805.00	Box		3.00	x 3.00	changed	Consultant's proposal based on site visit
12	Km13 + 980.00	Pipe		1.25		-					deleted	Consultant's judgement based on site visit
13	-					Km14 + 620.00	Box		4.00	x 4.00	new	Consultant's proposal based on site visit
14	Km14 + 669.00	Box		1.50	x 3.00	Km14 + 650.00	Box	2 x	2.00	x 4.00	relocation of watergate	Consultant's judgement based on site visit
15	Km14 + 926.00	Box		3.00	x 3.00	Km14 + 907.00	Box		2.00	x 2.00	changed	Consultant's judgement based on site visit
16	Km15 + 150.00	Box	3 x	4.00	x 4.00	Km15 + 100.00	Box	3 x	4.00	x 6.00	changed	Request from Cat Hai PC
17	Km15 + 521.50	Pipe		1.25		Km15 + 520.00	Pipe		1.50		size changed	Length more than 30m
18	Km15 + 688.00	Pipe		1.25		-					deleted	Consultant's judgement based on site visit

Source : Study Team

7.4.2.2 Channel relocation

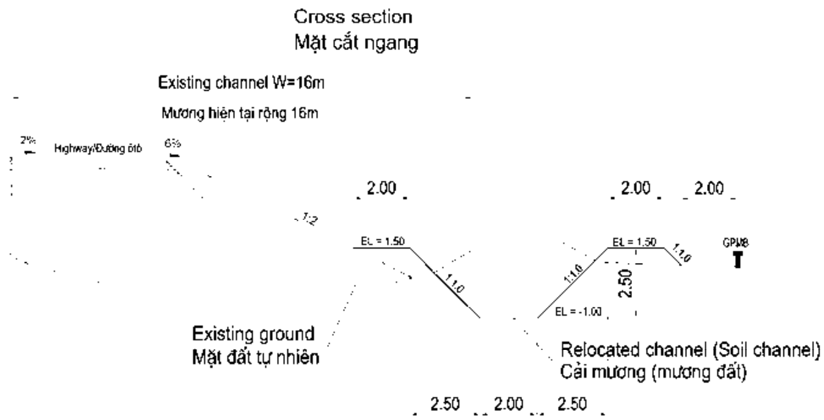
Tan Vu-Lach Huyen Highway runs through Cat Hai island and there is a section where the embankment overlaps existing water channel at section Km12+200 to Km13+240. According to a request from Cat Hai PC on relocation of the channel, Cat Hai PC and JST had a discussion in order to decide the size and type of the relocated channel.

Source : Study Team

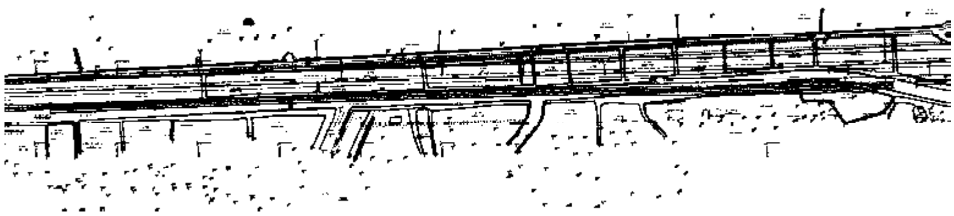
Figure 7.4.2-2 Photo of Existing Channel (From Km 13+400 to west)

An agreement was made between Cat Hai PC and JST in the discussion held on 20 May 2011 as shown in below figure.

<u>Cross section</u>



<u>Plan</u>



➢Size of relocated channel:
Bottom width=2.0m, Bottom elevation=-1.0m, Depth=2.5m, Slope gradient=1:1
➢Type of relocated channel:
Soil channel (same as existing condition)

Source : Study Team

Figure 7.4.2-3 Details of relocation channel

7.5 SOFT SOIL TREATMENT

7.5.1 Design Criteria

(1) Design standards

The following standards are applied for soft soil treatment design:

- Standard for Investigation and Design of Embankment on Soft Ground 22TCN262-2000,
- Highway Design Standard TCVN4054-2005.
- Flexible pavement design – 22TCN 211-06 (refference only)
- Japanese Standard for Sand Compaction Pile

(2) Condition of settlement and consolidation

Soft ground shall be treated to satisfy following conditions;

1) Residual settlement (Sr) is decided to be less than: 10cm for section behind bridge abutment and box culvert (H>2.0m), 20cm for sections including small size culvert (H≤2.0m) and 30 cm for normal embankment section. Value of allowable settlement for each section is summarized in Table 7.5.3-1.

Table 7.5.1-1 Allowable residual settlement after construction of pavement

Highway classification	**Embankment location on soft soil**		
	Near abutment	**At the place of culverts or under public highway**	**At normal embankment**
1. Expressway and highway class 80	≤ 10 cm	≤ 20 cm	≤ 30 cm
2. Highway under class 60 with surfacing class A1	≤ 20 cm	≤ 30 cm	≤ 40 cm

Source: 22TCN 262-2000, Item II.2.3.

Note) Allowable residual settlement is reduced into 10cm for box culvert section in this project, although 20cm is recommended in the standard (see Table 7.5.1-1). The reason why 10cm is applied is to minimize the risk of differential settlement because shallow foundation is applied for box culvert foundation instead of pile foundation.

2) Settlement due to creep consolidation is ignored in residual settlement.

3) Total period of treatment is decided to be less than: <u>16 months</u> for normal embankment section and <u>12 months</u> for box culvert section and section behind bridge abutment which is calculated from construction schedule.

(3) Traffic Load

Traffic load is evaluated in accordance with 22TCN262-2000 from following equations:

$$q = \frac{n \times G}{B \times l} \qquad (1\text{-}1)$$

$$B = n \times b + (n-1) \times d + e \qquad (1\text{-}2)$$

Where, (see Figure 7.5.1-1)

n: Number of vehicle,

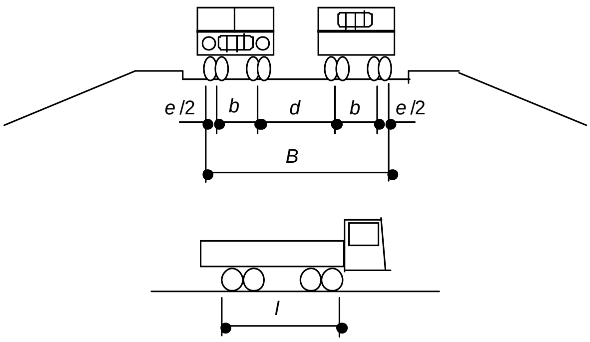
G: Weight of vehicle (=30 ton in case of H30),

B: Width of traffic load (Max. 13.75m as designated, 1 side),

l: distance between front wheel and rear wheel (=6.6m, in case of H30),

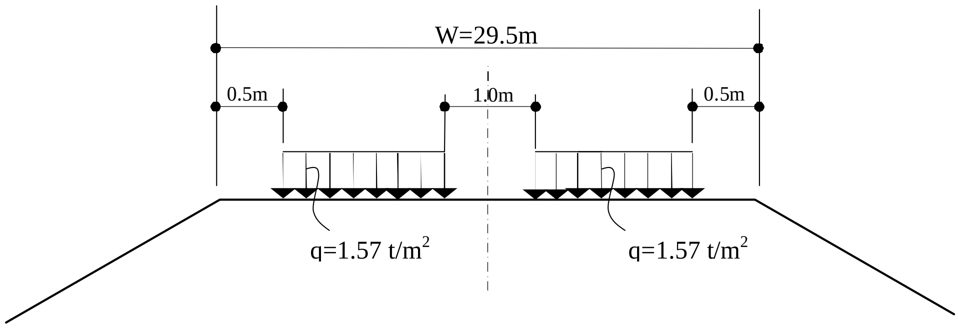
b = 1.8 m, e = 0.5m, d = 1.3m

Results: B=11.6m, n=4, and q=1.57 t/m2 and will be distributed in the carriage ways as sketched for calculation as Figure 7.5.1-2.



Source : Study Team

Figure 7.5.1-1 Traffic Load Calculation Diagram



Source : Study Team

Figure 7.5.1-2 Traffic Load Value and Distribution

7.5.2 Method of analysis

(1) Settlement analysis

The total settlement S can be given as:

S = Se + Sc + Screep

where :

Se: Immediate Settlement

Sc: Consolidation Settlement

Screep: Creep Consolidation. (to be ignored)

Immediate settlement takes place as the load is applied or within a time period of about seven days, while consolidation settlement takes months to years for dispersion of excess pore pressure.

For calculation of consolidation settlement, the value of over-consolidation ($\Delta\sigma oc$) of the respective cohesive layers has been taken into consideration. The computation formulae are as per range of vertical stress whether less than or exceeding yield stress (Pc).

(1)When effective overburden pressure (σ') plus stress increment (Λp) is still less than yield stress (Pc)

$Sc = Ca / (1+e0) \log ((\sigma' + \Delta p) / \sigma')$, where, Ca = Cc / 10

(2)When effective overburden pressure (σ') plus stress increment (Δp) exceeds yield stress (Pc)

$$Sc = Ca / (1+e0) \log ((\sigma' + \Delta \sigma oc) / \sigma') + Cc/(1+e0) \log ((\sigma' + \Delta p) / Pc)$$

where, Ca = Cc / 10 and $Pc = \sigma' + \Delta \sigma oc$

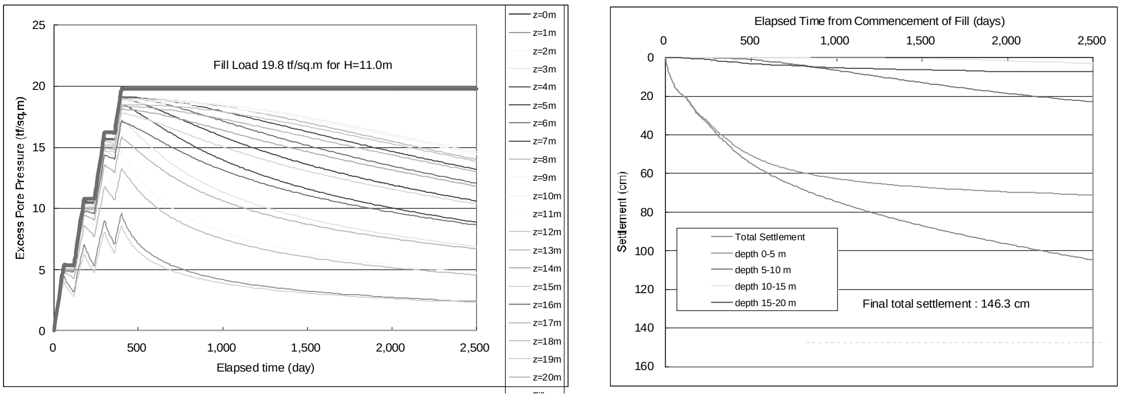
Numerical analysis for one dimensional consolidation settlement has been carried out in compliance with Terzaghi's consolidation equation (1) given below, which considers only vertical (z-direction) dispersion of excess pore pressure.

$\partial uz / \partial t = cv\, \partial 2uz / \partial z2$ (1)

Applying differential equation to above equation (1), increment/dispersion of excess pore pressure at the depth of z in the period of dt (from t = t – dt through t = t) is described with explicit solution as follows.

$$uz{:}t = uz{:}t\text{-}dt + cv\, dt\, (uz{+}dz{:}t\text{-}dt - 2\, uz{:}t\text{-}dt + uz\text{-}dz{:}t\text{-}dt) / (dz)2 \qquad (2)$$

The result of numerical analysis with deference equation is exemplarily shown below in terms of increment/dispersion of excess pore pressure and settlement curve.



Source : Study Team

Figure 7.5.2-1 Exemplary Analysis of Excess Pore Pressure and Settlement

(2) Design of Vertical Drain

The purposes of vertical drains' installation are as follows:

1.To accelerate consolidation settlement,

2.To decrease post-construction settlement accordingly, and

3.To increase rate of strength gain due to consolidation.

The purposes of item 1 and 2 will be applied for settlement problems, and the purpose of item 3 will be for stability problems.

Numerical analysis for one dimensional consolidation settlement considering installation of vertical drain also has been carried out in compliance with Rendulic's consolidation equation (3) given below, which considers both vertical (z-direction) and horizontal (r-direction) dispersion of excess pore pressure simultaneously.

$$\partial u / \partial t = c_v\, \partial^2 u / \partial z^2 + c_h\, (\partial^2 u / \partial r^2 + \partial u / r \partial r) \qquad (3)$$

Horizontal dispersion of excess pore pressure due to installation of vertical drain (equivalent radius : a) is formulated as equation (4).

$$\partial u_r / \partial t = c_h\, (\partial^2 / \partial r^2 + \partial / r \partial r)\, u_r \qquad (4)$$

Average excess pore pressure within the influential area (equivalent radius: b) of vertical drain can be analytically given by equation (5).

$$u_{r:t} = u_{r:0} \quad \exp [- 2\, c_h\, t / \{ b^2 F(n)\}] \qquad (5)$$

where, $F(n) : n^2 / (n^2 - 1) \quad \log n - (3 n^2 - 1) / 4 n^2$

$n : b / a$ ($n=5$ induces $F(5)=0.9365$)

a : equivalent radius of vertical drain (supposed to be 0.1m)

b : equivalent radius of influential area of each vertical drain

Vertical drain has been designed using Hansbo's theory (1979) as presented below;

$$U_h = 1 - \exp(\frac{-8T_h}{F})$$

$$F = F(n) + F_s + F_r$$

Where F is the factor which expresses the additive effect due to the spacing of the drains, F(n); smear effect, Fs; and well-resistance, Fr .

Degree of consolidation at node k can be defined with the ratio of remaining excess pore pressure to initial pore pressure u0 k:t (equal to total stress at node k at z = z and t = t) with equations (6) and (7), respectively.

Uk z:t = 1 – uz:t / u0 k:t (6)

Uk r:t = 1 – ur:t / u0 k:t (7)

Degree of consolidation Uk:t at the specific node of k at a specific time t = t is determined with the following equation (8), called as Carillo's equation, using degrees of consolidation Uk z:t and Uk r:t resulted from vertical and horizontal dispersion of excess pore pressure, respectively.

Uk:t = 1 – (1 – Uk z:t) (1 – Uk r:t) (8)

The consolidation degree, namely, consolidation time for consolidation settlement is mainly dependent on the horizontal coefficient of consolidation Ch and the effective diameter of the drain well such as arrangement pattern and spacing of Prefabricated Vertical Drain (PVD).

(3) Stability against sliding

Following conditions are to be confirmed for stability against sliding:

- Factor of safety (Fs) is not less than 1.2 in period of filling and waiting for consolidation, and
- Factor of safety (Fs) is not less than 1.4 at the end of final period of waiting for consolidation.

(4) Sand Compaction Pile

a) Overview

Sand Compaction Pile (SCP) is penetrated with vibration load on soft soil. It shall contribute to increase of bearing capacity, decrease of consolidation settlement, increase of horizontal resistance, uniformity of ground, drainage by consolidation and increase of ground density. This method is used in mainly sand soil, clay soil and organic soil conditions.

b) Design

Replacement ratio

Replacement ratio is defined by the following figure (Figure 7.5.2-1), and calculated by the following formula according to patterns such as square and triangular patterns.

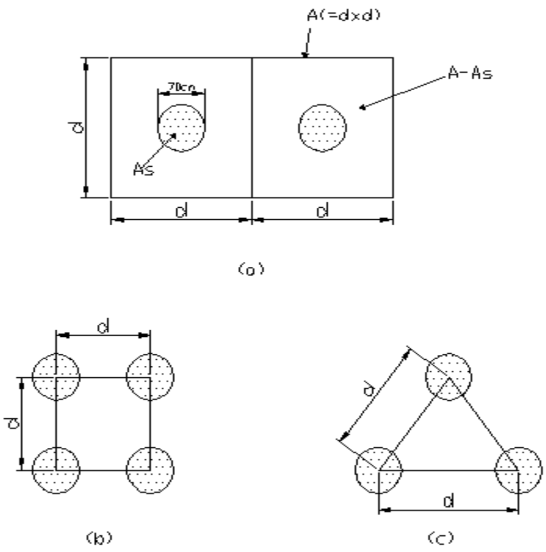


Figure 7.5.2-2 Arrangement and design concept of SCP

$$Fv = \frac{As}{A} = \frac{As}{d^2}$$ in case of square pattern (7-1)

$$Fv = \frac{As}{A} = \frac{2}{\sqrt{3}}\frac{As}{d^2}$$ in case of triangular pattern (7-2)

Where,

As: Cross-sectional area of SCP

d: Center-to-Center spacing

<u>Shear strength</u>

Soft ground after being treated by SCP will be considered as a composite ground comprising of SCP and surrounding soft soil. Shear strength of the composite ground SC is calculated as follows.

τ sc=(1-Fv)(Co+Cu/p• (Po-Pc+μc• z)•U+ Fv • (s'•Z+μs• z)tan s• (cosθ)² (7-3)

τ sc=(1- Fv)(Co+Cu/p• (Po-Pc+μc•□z)·U+(‾'m•Z+‾z)•μs •Fv •tan‾s •(cosθ)² (7-4)

Where,

μc: Reduction coefficient of stress, $\mu c = \frac{\sigma c}{\sigma} = \frac{1}{1+(n-1)Fv}$

μs: Increase coefficient of stress, $\mu s = \frac{\sigma s}{\sigma} = \frac{n}{1+(n-1)Fv}$

=2.5~3.5 typically, (3.0 in average).

n: Ratio of stress division, $n = \frac{\sigma s}{\sigma c}$

Cu/p: Ratio of strength increase

γs': Sub water unit weight of sand

Z: Depth to the failure surface

φs: Friction angle of sand

θ: Angle between acting surface and horizontal surface

σz: Increased stress at failure surface due to embankment loading

σ: Average stress

σc: Stress acting on surrounding soil

σs: Stress acting on SCP

γm' = Average sub water unit weight of composite soil

Friction angle of sand (of SCP) and ratio of stress division depending on replacement ratio is shown in Table 7.5.2-1.

Table 7.5.2-1 Friction angle and Ratio of stress division depending on replacement ratio

Replacement Ratio, Fv	Friction Angle of sand, φs	Ratio of stress division, n
0 ~ 0.4	30	3
0.4 ~ 0.7	30	2
0.7 ~ 1	30~35	1

Cohesion and internal friction angle of the composite soil used for slope stability analysis are evaluated by following formula (7-5) and (7-6) respectively, which is derived from formula (7-3).

$$\varphi = \tan^{-1}(m \times \tan \varphi s) \qquad (7\text{-}5)$$

C=(1-Fv)(Co+Cu/p·(Po-Pc+μc·ΔP)·U (7-6)

Where,

m= Fv×μs

Po : Effective overburden pressure

Pc : Preconsolidation pressure

ΔP: Embankment pressure

<u>Settlement</u>

Settlement of the composite ground is less than non-treated ground because SCP shares load acting upon the ground. Accordingly, SCP reduces stress acting upon soil. Following formula is used to get settlement of the composite ground.

$$S = \frac{Cc}{1+e_o} H \log\left(\frac{Po + \mu c \times \Delta P}{Po}\right)$$ For normal consolidation (7-7)

$$S_c = \frac{C_s}{1+e_o} H \log \frac{P_o + \mu c \times \Delta P}{P_o}$$ For over consolidation and $P_c > P_0 + \Delta P$ (7-8)

$$S_c = \frac{C_s}{1+e_o} H \log\frac{P_c}{P_0} + \frac{C_c}{1+e_0} H \log\frac{P_0 + \mu c \times \Delta P}{P_c}$$ For overconsolidation and $P_c < P_0 + \Delta P$ (7-9)

7.5.3 Result of the analysis

(1) Sectioning

Based on soil profile prepared in Detailed Design stage, sectioning for the calculation is updated. In addition to the soil investigation result, location of box culvert is also considered for the sectioning because of different value of allowable settlement from normal embankment section. The updated sectioning is shown in Table 7.5.3-1.

(2) Soft soil layer for settlement calculation

Soft soil layer to be considered for calculation of total settlement is as follows;

➢Hai An side: all the clay layer up to layer-8 (layer-8 is included)

➢Cat Hai side: all the clay layer up to layer-7B (layer-7B is included)

Thickness of each layer is summarized in Table 7.5.3-1. Boring data which is conducted near the section is used for determination of layer thickness respectively. Referred boring No. for each section is indicated in Table 7.5.3-1 also.

(3) Soil parameters for settlement calculation

Soil parameters for settlement calculation are summarized in Table 7.5.3-2.

Table 7.5.3-1 Sectioning for Detailed Design (Hai An side)

No.	SECTION	Type of section	Location(STA)			Length (m)	Drilled Boring No.	Height of embankment for calculation (m)	Allowable residual settlement(cm)	Soil layer thickness (m)								
										1	2	3	4	5	6	7a	7b	8
1	HA-1	Normal	Km+20.00	~	Km+260	240	BA-4	3.3	30			27.0					2.5	
2	HA-2	Normal	Km+260	~	Km+762	502	BA-7	3.4	30			26.0					9.0	
3	HA-3	DR-BOX(2x2)	Km+762.00	~	Km+814	52	BA-7	3.9	20			24.5					9.0	
4	HA-4	Normal	Km+814.00	~	Km+888	74	BA-7	4.9	30			24.0					9.0	
5	HA-5	DR-BOX	Km+888.00	~	Km+980.00	92	BA-6	5.3	10			30.0						
6	HA-6a	Normal	Km+980.00	~	Km1+160	180	BA-6	3.5	30			30.0						5.0
7	HA-6b	Normal	Km+160	~	Km1+260	100	BA-10	3.3	30			15.0			10.5		10.5	
8	HA-6c	Normal	Km1+260	~	Km1+475.0	215	BA-11	3.4	30			21.0			9.5		4.0	
9	HA-7	Normal	Km1+475.0	~	Km1+640.0	165	BA-12	5.5	10			25.0			3.5		5.8	
10	HA-8	Behind bridge abutment	Km1+640.0 6	~	Km1+665	25	BA-13	6.7	10			22.0			4.5			
11	Cam River Bridge	Bridge	Km1+665	~	Km1+735		-	-	-									
12	HA-9	Behind bridge abutment	Km1+735	~	Km1+760	25	BA-14	6.7	10			23.5	1.0		1.0		7.0	
13	HA-10	Normal	Km1+760	~	Km2+000	240	BA-15	5.5	10			17.0	1.0		2.0		12.5	
14	HA-11	Normal	Km2+000.0	~	Km2+364	364	BA-17	3.5	30			10.5			11.0		10.0	
15	HA-12	DR-BOX(2x2)	Km2+364.0	~	Km2+416	52	BA-18	4.2	20			10.0			5.5		17.0	1.0
16	HA-13	Normal	Km2+416.0	~	Km2+624	208	BA-19	4.4	30			11.0			6.0		11.0	1.3
17	HA-14	DR-BOX(2x2)	Km2+624.0	~	Km2+676	52	BA-19	4.5	20			10.0			6.0		11.0	1.3
18	HA-15	Normal	Km2+676.0	~	Km3+000	324	BA-21	6.0	30			5.5	4.5		12.0		8.0	
19	HA-16	Normal	Km3+000.0	~	Km3+375	375	BA-22	5.4	30			10.5			3.5		18.0	
20	HA-17	Normal	Km3+375.0	~	Km3+675	300	BA-25	4.5	30			17.5			6.0		7.0	
21	HA-18	Normal	Km3+675.0	~	Km4+111	436	BA-27	4.5	30			8.5			6.0		14.0	
22	HA-19	DR-BOX	Km4+111.0	~	Km4+169	58	BA-20	4.4	10			11.0			15.0		7.0	2.0
23	HA-20a	Normal	Km4+169.0	~	Km4+280	111	BA-29	4.6	10			12.0			18.0			5.0
24	HA-20b	Normal	Km4+280	~	Km4+457	177	BA-30	6.7	10			16.5			5.5		11.0	

Source : Study Team

Sectioning for Detailed Design (Cat Hai side)

No.	SECTION	Type of section	Locaton(STA.)			Length (m)	Refferd Boring No.	Height of embankment for calculation (m)	Allowable residual settlement(cm)	Soil layer thickness (m)							
										1	2	3	4	5	6	7a	7b
1	CH-1	Behind bridge abutment (Piled slab)	Km9+948	~	Km9+973	25	BP-92	6.8	-			11.0	9.0		6.0		4.5
2	CH-2	Normal embankment	Km9+973	~	Km10+048	75	BC-1	6.7	10			12.5	5.0		5.0		5.5
3	CH-3	UP-BOX	Km10+048	~	Km10+103	55	BC-7	6.5	10			10.0	7.5		4.0		6.5
4	CH-4	Normal embankment	Km10+103	~	Km10+387	284	BC-3	5.6	30			12.5	5.0		8.5		9.0
5	CH-5	UP-BOX	Km10+387	~	Km10+442	55	BC-4	5.6	10		1.5	11.5	9.0				5.0
6	CH-6	Normal embankment	Km10+442	~	Km10+625	183	BC-5	5.5	30			12.0	5.0				3.5
7	CH-7	Normal embankment	Km10+625	~	Km10+778	153	BC-35	5.8	30		2.0	7.5			10.5		2.5
8	CH-8	DR-BOX	Km10+778	~	Km10+832	54	BC-6	5.8	10			10.0			4.0		7.5
9	CH-9	Normal embankment	Km10+832	~	Km11+250	418	BC-8	5.7	30	3.0	2.5	5.5			2.0		8.0
10	CH-10	Normal embankment	Km11+250	~	Km11+650	400	BC-11	4.9	30	2.0	3.5	6.0			4.0		2.0
11	CH-11	Normal embankment	Km11+650	~	Km11+790	140	BC-12	4.9	30	2.0	2.0	7.0				7.5	1.5
12	CH-12	Normal embankment	Km11+790	~	Km12+050	260	BC-13	4.9	30	2.0	2.5	6.5			2.0		6.0
13	CH-13	Normal embankment	Km12+050	~	Km12+300	250	BC-15	5	30	2.0	2.0	6.5			2.0	9.5	
14	CH-14	Normal embankment	Km12+300	~	Km12+580	280	BC-39	5	30	1.5	2.5	2.0	2.0		1.5	4.5	
15	CH-15	Normal embankment	Km12+580	~	Km12+950	370	BC-19	5.2	30		3.5	4.5	3.5	2.0		8.5	
16	CH-16	Normal embankment	Km12+950	~	Km13+573	623	BC-20	5.8	30	2.5	3.0	4.0	2.0	3.5			12.5
17	CH-17	UP-BOX	Km13+573	~	Km13+628	55	BC-23	5.9	10		6.0	6.0					6.0
18	CH-18	Normal embankment	Km13+628	~	Km13+910	282	BC-24	5.9	30		5.5	6.0					7.0
19	CH-19	Normal embankment	Km13+910	~	Km14+070	160	BC-26	6.5	30	2.0		6.5		2.0			2.0
20	CH-20	Normal embankment	Km14+070	~	Km14+370	300	BC-38	6.8	30	6.0		4.0		2.5			
21	CH-21	Normal embankment	Km14+370	~	Km14+593	223	BC-28	5.3	30		6.0	4.0		5.5			
22	CH-22	DR-BOX & UP-BOX	Km14+593	~	Km14+679	86	BC-29	7	10	0.5	4.5		3.0	2.0			3.0
23	CH-23	Normal embankment	Km14+679	~	Km14+881	202	BC-30	5.8	30	2.0	4.5	3.0		3.5			
24	CH-24	DR-BOX(2x2)	Km14+881		Km14+933	52	BC-31	6.1	20		4.5	9.0					5.0
25	CH-25	Normal embankment	Km14+933		Km15+068	135	BH-16	7.3	30		5.5	3.5	4.5	1.5			8.0
26	CH-26	DR-BOX	Km15+068		Km15+132	64	BC-32	7.6	10		3.5	4.0	2.0	4.5			6.0
27	CH-27	Normal embankment	Km15+132		Km15+313	181	BC-33	6.9	30		2.5	3.0	4.5	4.0			4.0
28	CH-28	UP-BOX	Km15+313		Km15+368	55	BC-34	5.1	10		5.5	2.0	3.0	3.0			3.5
29	CH-29	Normal embankment	Km15+368	~	Km15+630	262	B11	4.4	30		3.0	6.0	2.5	2.5			
30	CH-30	Terminal road	Km15+630	~	Km15+884	254	B11	2.5	30		3.0	6.0	2.5	2.5			

Source : Study Team

Table 7.5.3-2 Soil parameters for settlement calculation (Hai An side)

Item					3 (Clay)	4 (Clay)	6 (Clay)	7B (Clay)	8 (Clay)	9 (Clay)	10A (Sand)	10B (Sand)
N-value					2	7	13	7	14	6	21	49
Unit Weight			γt	(g/cm^3)	1.70	(1.76)	1.86	1.76	1.88	(1.76)	(2.00)	(2.05)
Shear Strength	For Short Term		Su or Cd	(kg/cm^2)	Note (2)	0.25	0.50	0.25	0.60	(0.25)	0.00	0.00
			ϕu or ϕd	(degree)	0.0	0.0	0.0	0.0	0.0	0.0	35.0	40.0
	For Long Term	Total Stress	Ccu	(kg/cm^2)	0.13	0.06	0.16	0.06	0.16	0.06	-	-
			ϕcu	(degree)	14.3	17.1	15.9	17.1	15.9	17.1	-	-
		Effective Stress	C'	(kg/cm^2)	0.06	0.09	0.12	0.09	0.12	0.09	-	-
			ϕ'	(degree)	26.5	22.5	23.3	22.5	23.3	22.5	-	-
	Rate of Strength Increase		m		0.2	0.2	0.2	0.2	0.2	0.2	-	-
Consolidation	Initial Void Ratio		eo		1.384	1.225	0.943	1.225	0.851	1.225	-	-
	Compression Index		Cc		0.435	0.420	0.302	0.420	0.262	0.420	-	-
	Swell Index		Cs		0.050	0.060	0.039	0.060	0.039	0.060	-	-
	Preconsolidation Pressure		Pc	(kg/cm^2)	0.74	1.59	1.70	1.59	1.73	1.59	-	-
	Coefficient of Consolidation		Cv	(cm^2/day)	50	70	100	70	100	70	-	-

Note (1): The value in ( ) is assumed.
Note (2): Su = 0.1 kg/cm^2 (Down to EL 0.0m), Su = 0.1 + 0.005 x Z kg/cm^2 (below EL 0.0m, Zo = EL 0.0m)
Note (3): Adopt the soil parameters (Ccu, ϕcu, C', ϕ') of Layer-6 for Layer-8
Note (4): Adopt the soil parameters (Ccu, ϕcu, C', ϕ', eo, Cc, Cs, Pc, Cv) of Layer-7B for Layer-4 and Layer-9

Source : Study Team

Soil parameters for settlement calculation (Cat Hai side)

Item					Layer													
					1	2	3	4	5	L5-1	6	L6-1	7A	7B	8	9	10A	10B
					(Clay)	(Sand)	(Clay)	(Clay)	(Sand)	(Sand)	(Clay)	(Sand)	(Clay)	(Clay)	(Clay)	(Clay)	(Sand)	(Sand)
N-value					2	5	3	5	10	57	11	9	4	5	14	6	14	42
Unit Weight			γt	(g/cm^3)	1.65	1.90	1.73	1.73	1.93	(2.10)	1.86	(1.93)	1.75	1.81	1.93	1.76	(1.95)	(2.05)
Shear Strength	For Short Term		Su or Cd	(kg/cm^2)	Note (2)	0.00	0.15	Note (3)	0.00	0.00	0.60	0.00	0.20	0.25	0.80	0.25	0.00	0.00
			φu or φd	(degree)	0.0	23.0	0.0	0.0	25.0	45.0	0.0	25.0	0.0	0.0	0.0	0.0	30.0	40.0
	For Long Term	Total Stress	Ccu	(kg/cm^2)	0.11	-	0.01	0.19	-	-	0.22	-	0.31	0.31	0.08	0.20	-	-
			φcu	(degree)	13.5	-	21.2	9.7	-	-	17.6	-	19.5	19.5	20.0	13.8	-	-
		Effective Stress	C'	(kg/cm^2)	0.10	-	0.02	0.14	-	-	0.16	-	0.12	0.12	0.03	0.11	-	-
			φ'	(degree)	25.2	-	30.5	21.5	-	-	24.9	-	26.3	26.3	25.2	24.5	-	-
	Rate of Strength Increase		m		0.2	-	0.2	0.2	-	-	-	-	0.2	0.2	-	-	-	-
Consolidation	Initial Void Ratio		eo		1.595	-	1.294	1.317	-	-	0.962	-	1.185	1.078	0.825	1.200	-	-
	Compression Index		Cc		0.527	-	0.397	0.386	-	-	0.220	-	0.430	0.351	0.193	0.457	-	-
	Swell Index		Cs		0.070	-	0.054	0.054	-	-	0.042	-	0.059	0.054	0.031	0.067	-	-
	Preconsolidation Pressure		Pc	(kg/cm^2)	0.34	-	0.66	0.83	-	-	1.46	-	1.15	1.13	1.44	1.51	-	-
	Coefficient of Consolidation		Cv	(cm^2/day)	40	-	60	70	-	-	150	-	80	80	150	80	-	-

Note (1): The value in ( ) is assumed.
Note (2): Su = 0.1 kg/cm^2 (Down to EL 0.0m), Su = 0.1 + 0.02 x Z kg/cm^2 (below EL 0.0m, Zo = EL 0.0m)
Note (3): Su = 0.1 + 0.02 x Z kg/cm^2 (below EL -5m, Zo = EL -5.0m)
Note (4): Adopt the soil parameters (Ccu, φcu, C', φ') of Layer-7B for Layer-7A

Source : Study Team

(4) Selection of Countermeasures and result of calculation

Soft soil treatment methods are studied among the several possible treatment methods as shown in Table 7.5.3-4. Recommended countermeasures are selected as shown in Table 7.5.3-3 taking into account the technical requirements and the clarifications based on the practical experiences of highway projects in Vietnam, especially Hanoi-Hai Phong expressway project which is located nearby this project.

Table 7.5.3-3 Recommended Countermeasures and Applicable Condition in Detailed Design

Case	**Classification**	**Countermeasures for soft soil treatment**	**Remark**
I-1	Low embankment (He < 3.0m) and Soft soil layer Thickness(D) < 5-10m	Pre-loading	Apply reinforced geotextile or not
I-2	Low embankment (He < 3.0m) and Soft soil layer Thickness(D) ≥ 10m	PVD or Sand drain (SD)	ditto
II	Embankment (6.0 ≥ He ≥ 3.0m) and/or Depth of installation ≤ 30m	PVD or Sand drain (SD)	ditto
III	Embankment(He) > 6m and In case of not satisfied the requirement conditions by only sand drain	PVD or SD or Sand compaction pile (SCP)	ditto
IV	Special cases near bridge abutment and pagoda section	SCP or Piled Slab (LRS)	ditto

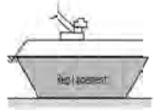
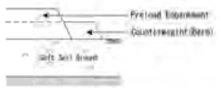
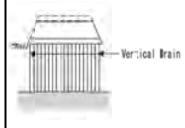
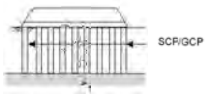
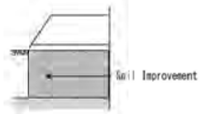
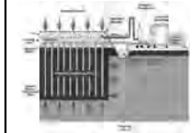
Source : Study Team

The Study Team basically follows the concept of Basic Design to the Detailed Design. However, soil investigation work has not been completed and boring data was not available during Basic Design stage. PVD cannot be driven into stiff clay layer, therefore, sand drain was selected as main countermeasure for safety because the Study Team could not judge whether PVD could be applied or not.

After getting boring data, the Study Team reviewed it and found that PVD could be applied at some sections judging from boring data and N-value. Applying PVD is actually better than applying sand drain from the view point of availability of material and ease of execution, therefore, the Study Team tried to apply PVD as much as possible in Detailed Design stage.

Table 7.5.3-5 and Table 7.5.3-6 show selected countermeasure for each section and result of calculation of the treatment.

Table 7.5.3-4 Soft Soil Treatment Methods

Improvement Method	Excavation Replacement with Sand Mat	Pre-loading with Counterweight	Vertical Drain with Pre-loading	Sand/Gravel Compaction Pile	Deep Mixing Method (Cement, Lime Improvement)	Dynamic Compaction	Vacuum Consolidation
Sketch							
Introduction	This method involves the removal of all or part of the soft deposits and replaces it with material, which increases the strength and decreases the settlement.	This method, which places counterweight fill to prevent sliding accelerate the settlement by pre-loading.	This method places material like sand and cardboard vertically to drain the soft ground and to accelerate the consolidation by shortening the drainage path.	With this method, sand/gravel colums of large diameter are placed in the ground and compacted by vibration or impact to increase the strength of ground.	This method, use of chemical improvement is more attractive than conventional methods such as vertical drains, sand compaction pile and counterweight fill methods to prevent affecting neighboring area.	With this method, sand or gravel column are made by free fall of heavy weight using special improved crane.	With this method, water in soft soil is pumped up by vacuum pressure through PVD.
Effectiveness	• To increase the bearing capacity by replacement. . Necessary sand mat to ensure the traffic-ability prior to excavation.	• Pre-loading to accelerate the consolidation settlement and increase the shear strength. • To ensure the stability of embankment by counterweight.	• Pre-loading with dewatering through vertical drains to accelerate the consolidation settlement and increase the shear strength.	• To increase the shear strength by the sand columns and ensure the stability of embankment. • To reduce the consolidation settlement. •To accelarate the settlement.	• To improvement the soil properties by the chemical improvement (Cement, Lime etc.) and ensure the stability of embankment.	•To make the sand or gravel column by forced replacement of soft soil. • To encrease the shear strength by sand or gravel column. • To reduce the consolidation settlement.	• Consolidation settlement will occur by this suction without any embankment. • To accelerate consolidation settlement by vacuum load as surcharge.
Criteria for Application	Shallow deposit less than 6m.	Shallow deposit is effective.	More advantageous for deep deposit.	More advantageous for deep deposit.	Deep deposit is preferable	Shallow Deposit less than 6m	Expected maximum load by suction is about 65/m2
	Clay, Peat or organic soil.	Clay, Peat or Organic Soil.	Clay, Peat or Organic Clay	Clay, Peat or Organic soil.	Peat or Organic Clay, loose cohesionless soil.	Clay Peat and Organic soil, loose cohesionless soil	Clay, Peat, Organic Soil
	Widely used	Widely used	Technique and equipment can be available in Vietnam.	Technique and equipment can be available in Vietnam.	Necessary technique and equipment from abroad.		Necessary technique and equipment from abroad
Clarifications or other attentions to pay for at construction or design stage	• Difficult complete excavation may leave the small pockets of peat that may cause severe settlement after the completion. • Very slow shear strength peat may cause failure of the side slope excavation that results in increasing the quantities. • Rarely used in recent time because difficult estimating of material volume. • The settlement often continously occurred after completion.	•This may result in very long construction times depend on the depth of peat. •Large land acquisition area is required. •The height of preloading is generally 2m or about 1m, where residual settlement is low due to shallow peat deposit or where differential residual settlement may be difficult to achieve on flat land.	• Generally, sand mats are necessary prior to the placement of the sand columns because the equipment used is very heavily. • The sand drain method does not effect appreciable acceleration of settlement, but it is incorporated into design as a composite foundation with the strength of the sand columns.	• Necessary monitoring must be carried out to pay attention of uplift or deformation of surroundings and adjacent structure during placing of sand column. • In small and medium rivers with narrow beds there may be blockage of low-lying waterways.	• This method is not necessary to consider greatly influence adjacent ground during construction however, depending on ground condition and work condition, uplift or lateral deformation of the adjacent ground may occur so that necessary treatment shall be considered. • Consideration shall be given to great variation in the strength of improved columns after mixing and stirring.	• As number of tamping is very much for one location, it takes time very much. • Confirmation of improvement effectiveness to required depth shall be checked area by area. • Material for sand column shall be sand or gravel. • Trial execution is necessary to confirm the design criteria. • No excavation and No disposale site for removed soil are necessary. • Uncommon	• Sand ream in the soft soil layer reduce the effectiveness of vacuum because of leakage of air and water throught it. • Monitoring of vacuum pressure and water volume pumped up must be carried out to confirm the effectiveness of vacuum pressure. •Effectiveness of soil improvement by this method shall be checked by monitoring the settlement and strength of soil. • Air tightness is essential for this method. Holes or tear of sheet and other cause leakage shall be checked stricktly. •Uncommon
Cost effectiveness	Cost effective but applicable depth is less than estimate 6m	Cost effective but takes time	Cost effective	Cost effective	Costly	Cost effective but takes time	Costly
Consultant's selection			O	O			

Source : Study Team

Table 7.5.3-5 Selected countermeasures and result of calculation of the treatment (Hai An side)

No.	SECTION	Type of section	Location(STA.)	Length (m)	Height of embankment (m)	Method of treatment	Height of surcharge (m)	No. of Geotextile layer(200KN/m)	No. of Geotextile layer(400KN/m)	No. of embankment step	Treatment depth(m)	Spacing of drain (m)	Treatment period(month)	Degree of consolidation (%)	Total Settlement (cm)	Residual settlement Sr (cm)	Allowable residual settlement (cm)	Sand mat thickness (m)	Filling progress: 1st Step Height(m)	1st Step Δh	1st Step embank speed (cm/day)	1st Step waiting time for consolidation (mon)	2nd Step Height(m)	2nd Step Δh	2nd Step Embankment speed (cm/day)	2nd Step Waiting time for consolidation (mon)	3rd Step Height(m)	3rd Step Δh	3rd Step Embankment speed (cm/day)	3rd Step Waiting time for consolidation	Total treatment period (day)	Safety factor of slope stability: 1st Step	2nd Step	3rd Step	Finish improvement
1	HA-1	Normal embankment	Km+20.00 ~ Km+260	240	3.3	PVD	0.0	-	2	2	29.5	1.2	15.5	90.6	107.7	10.1	30	0.6	2.9		10	3	FG+Δh	0.8	10	11.0					465	1.563	1.272		1.676
2	HA-2	Normal embankment	Km+260 ~ Km+762	502	3.4	PVD	0.0	-	2	2	30.0	0.8	16.1	90.27	121.22	11.8	30	0.6	2.9		10	3	FG+Δh	0.9	10	11.5					482	1.586	1.237		1.669
3	HA-3	DR-BOX(2x2)	Km+762.00 ~ Km+814	52	3.9	PVD	1.0	-	4	2	30.0	0.8	11.9	93.28	141.07	9.5	20	0.7	3.9		10	4	FG+Δh	1.9	10	6.0					358	1.506	1.226		1.908
4	HA-4	Normal embankment	Km+814.00 ~ Km+888	74	4.7	PVD	0.0	-	4	2	30.0	0.8	15.7	90.69	167.44	15.6	30	0.8	3.9		10	4	FG+Δh	1	10	9.5					472	1.510	1.229		1.647
5	HA-5	DR-BOX	Km+888.00 ~ Km+980.00	92	5.3	SD	0.0	-	3	2	30.0	1.6	11.4	96.68	167.54	5.6	10	0.9	4.8		10	4	FG+Δh	0.6	10	5.0					341	1.236	1.260		1.475
6	HA-6a	Normal embankment	Km+980.00 ~ Km1+160	180	3.5	PVD	0.0	-	2	2	30.0	0.8	14.6	91.41	116.02	9.97	30	0.6	3.9		10	4	FG+Δh	0.7	10	9.0					438	1.267	1.303		1.662
7	HA-6b	Normal embankment	Km1+160 ~ Km1+260	100	3.3	PVD	1.0	-	3	2	15.0	1.2	15.5	83.55	87.91	14.5	30	0.5	4		10	4	FG+Δh	1.7	10	10.0					466	1.367	1.245		1.976
8	HA-6c	Normal embankment	Km1+260 ~ Km1+475.00	215	3.4	PVD	1.0	-	4	2	21.0	1.1	15.6	94.62	100.72	5.4	30	0.5	4		10	4	FG+Δh	1.8	10	10.0					469	1.543	1.304		2.146
9	HA-7	Normal embankment	Km1+475.00 ~ Km1+640.00	165	5.5	SD	0.0	-	3	3	30.0	1.5	11.5	94.21	170.88	9.9	10	0.9	3.7		10	3	5		10	3.0	FG+Δh	0.5	10	3	344	1.505	1.379	1.295	1.451
10	HA-8	Behind bridge abutment	Km1+640.00 ~ Km1+665	25	6.7	LRS	-	-	-	-	-	-	-	-	-	-	10	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
11	Cam River Bridge	Bridge	Km1+665 ~ Km1+735	70	-		-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
12	HA-9	Behind bridge abutment	Km1+735 ~ Km1+760	25	6.7	LRS	-	-	-	-	-	-	-	-	-	-	10	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
13	HA-10	Normal embankment	Km1+760 ~ Km2+000	240	5.5	SD	0.0	-	3	3	30.0	1.5	11.5	95.83	169.28	7.1	10	0.8	3.7		10	3	5		10	3.0	FG+Δh	0.5	10	3	344	1.530	1.361	1.294	1.447
14	HA-11	Normal embankment	Km2+000.00 ~ Km2+364	364	3.5	SD	0.0	-	2	2	30.0	2.0	11.4	90.88	63.69	5.8	30	0.5	3.4		10	4	FG+Δh	0.4	10	6.0					347	1.394	1.382		1.598
15	HA-12	DR-BOX(2x2)	Km2+364.00 ~ Km2+416	52	4.2	SD	0.0	-	2	2	30.0	2.0	11.8	90.87	99.35	9.1	20	0.7	3.9		10	4	FG+Δh	0.5	10	6.0					353	1.260	1.234		1.466
16	HA-13	Normal embankment	Km2+416.00 ~ Km2+624	208	4.4	SD	0.0	-	2	2	29.3	2.0	11.8	93.27	95.29	6.0	30	0.8	3.9		10	4	FG+Δh	0.5	10	6.0					355	1.260	1.215		1.440
17	HA-14	DR-BOX(2x2)	Km2+624.00 ~ Km2+676	52	4.5	SD	0.0	-	3	2	28.3	2.0	10.8	91.36	90.55	7.8	20	0.5	3.9		10	1	FG+Δh	0.5	10	5.0					325	1.428	1.324		1.570
18	HA-15	Normal embankment	Km2+676.00 ~ Km3+000	324	6.0	SD	0.0	-	4	2	30.0	2.0	13.3	93.97	71.42	4.3	30	0.6	3.9		10	3	FG+Δh	0.5	10	8.0					398	1.904	1.339		1.501
19	HA-16	Normal embankment	Km3+000.00 ~ Km3+375	375	5.4	PVD	0.0	-	3	3	32.0	0.8	14.3	90.77	140.89	13.0	30	0.8	3.9		10	3	5.5		10	3.0	FG+Δh	0.6	10	6	429	1.410	1.274	1.211	1.429
20	HA-17	Normal embankment	Km3+375.00 ~ Km3+675	300	4.5	PVD	0.0	-	3	2	17.5	0.8	15.9	86.36	118.91	16.2	30	0.6	3.8		10	4	FG+Δh	0.7	10	10.0					478	1.447	1.296		1.596
21	HA-18	Normal embankment	Km3+675.00 ~ Km4+111	436	4.5	SD	0.0	-	3	2	26.5	2.0	11.8	92.12	92	7.3	30	0.6	3.9		10	4	FG+Δh	0.5	10	6.0					355	1.453	1.321		1.556
22	HA-19	DR-BOX	Km4+111.00 ~ Km4+169	58	4.4	SD	0.0	-	3	2	30.0	2.0	11.7	90.86	72.96	6.7	10	0.6	3.9		10	4	FG+Δh	0.4	10	6.0					352	1.400	1.342		1.551
23	HA-20a	Normal embankment	Km4+169.00 ~ Km4+280	111	4.6	SD	0.0	-	3	2	12.0	1.8	11.8	93.32	66.28	4.4	10	0.5	3.9		10	4	FG+Δh	0.3	10	6.0					353	1.427	1.337		1.522
24	HA-20b	Normal embankment	Km4+280 ~ Km4+457	177	6.7	SCP	0.0	-	1	3	30.0	1.5	10.7	96.49	134.03	4.7	10	0.9	4.5		10	3	FG+Δh	0.5	10	5.0					322	1.554	1.320		1.410
25	HA-21	Behind bridge abutment	Km4+457 ~ Km4+497	40	7.3	LRS	-	-	-	-	-	-	-	-	-	-	10	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Notes:

1)Installation of Geotextile (number, location, etc.) should be decided by the Engineer with careful checking during construction stage based on soil test result.

2)Installation of Course Sand Blanket (setting elevation, thickness, etc.) should be decided by the Engineer during construction checking the site condition.

Source: Study Team

Table 7.5.3-6 Selected countermeasures and result of calculation of the treatment (Cat Hai side)

No.	SECTION	Type of section	Locator (STA)			Length (m)	Height of embankment (m)	Method of treatment	Height of surcharge (m)	No. of geotextile layer (400KN/m)	No. of geotextile layer (200KN/m)	No. of embankment step	Treatment depth (m)	Spacing of drain (m)	Treatment period (mon)	Degree of consolidation (%)	Total Settlement (cm)	Residual settlement Sr (cm)	Allowable residual settlement (cm)	Sand mat thickness (m)
1	CH-1	Behind bridge abutment (Filed slab)	Km9+948	~	Km9+973	25	6,8	LRS	0,0				20,0							
2	CH-2	Normal embankment	Km9+973	~	Km10+048	75	6,7	SCP	0,0		1	3	28,0	1,5	11,7	99,94	128,15	0,08	10	0,9
3	CH-3	UP-BOX	Km10+048	~	Km10+103	55	6,5	SD	0,0	3		3	28,0	1,4	11,7	99,28	162,82	1,18	10	1,0
4	CH-4	Normal embankment	Km10+103	~	Km10+387	284	5.6	SD	0.0		1	3	20.0	1.5	15.4	82.08	158.84	28.47	30	0.8
5	CH-5	UP-BOX	Km10+387	~	Km10+442	55	5.6	SD	0.0		4	3	27.0	1.8	11.3	94.6	135.99	7.34	10	0.8
6	CH-6	Normal embankment	Km10+442	~	Km10+625	183	5,5	PVD	0,0		4	3	20,5	1,4	15,8	90,22	126,84	12,41	30	0,7
7	CH-7	Normal embankment	Km10+625	~	Km10+778	153	5,8	PVD	0,0		3	3	9,5	0,8	15,2	69,92	87,96	26,46	30	0,6
8	CH-8	DR-BOX	Km10+778	~	Km10+832	54	5.8	PVD	0.0		3	3	21.5	0.8	12.0	98.06	114.2	2.22	10	0.8
9	CH-9	Normal embankment	Km10+832	~	Km11+250	418	5.7	PVD	0.0		2	2	13.0	0.8	15.3	79.06	127	26.59	30	0.7
10	CH-10	Normal embankment	Km11+250	~	Km11+650	400	4,9	PVD	0,0		2	2	11,5	1,2	14,9	85,35	84,09	12,32	30	0,6
11	CH-11	Normal embankment	Km11+650	~	Km11+790	140	4.9	PVD	0.0		3	2	18.5	1.2	15.0	93.86	120.62	7.41	30	0.7
12	CH-12	Normal embankment	Km11+790	~	Km12+050	260	4.9	PVD	0.0		2	2	11.0	1.2	15.0	76.03	100.2	24.02	30	0.6
13	CH-13	Normal embankment	Km12+050	~	Km12+300	250	5,0	PVD	0,0		3	2	22,0	1,2	14,1	93,4	121,92	8,05	30	0,7
14	CH-14	Normal embankment	Km12+300	~	Km12+580	280	5,0	PVD	0,0		3	2	14,0	1,4	14,9	91,76	66,61	5,49	30	0,5
15	CH-15	Normal embankment	Km12+580	~	Km12+950	370	5.2	PVD	0.0		2	2	22.0	1.5	14.0	93	91.28	6.6	30	0.6
16	CH-16	Normal embankment	Km12+950	~	Km13+573	623	5.8	PVD	0.0		2	2	11.5	1.2	16.0	77.51	131.8	29.64	30	0.7
17	CH-17	UP-BOX	Km13+573	~	Km13+628	55	5,9	PVD	1,5		3	2	12,0	0,8	11,8	88,68	83,59	9,46	10	0,7
18	CH-18	Normal embankment	Km13+628	~	Km13+910	282	5,9	PVD	0,0		2	2	11,5	1,4	15,2	78,47	82,1	17,68	30	0,6
19	CH-19	Normal embankment	Km13+910	~	Km14+070	160	6.5	SD	0.0	3		2	8.5	1.5	13.5	98.98	92.29	0.94	30	0.7
20	CH-20	Normal embankment	Km14+070	~	Km14+370	300	6.8	SD	0.0	3		2	10.0	1.5	13.7	98.66	126.18	1.69	30	0.8
21	CH-21	Normal embankment	Km14+370	~	Km14+593	223	5,3	No need	0,0		1	2	10,0		11,9	81,97	41,31	7,45	30	0,5
22	CH-22	DR-BOX & UP-BOX	Km14+593	~	Km14+679	86	7.0	SD	0.0		2	2	8.0	1.8	11.5	91.39	52.75	4.54	10	0.6
23	CH-23	Normal embankment	Km14+679	~	Km14+881	202	5.8	SD	0.0		2	2	9.5	2.0	13.1	90.85	61.22	5.6	30	0.6
24	CH-24	DR-BOX(2x2)	Km14+881	~	Km14+933	52	6,1	SD	0,0		4	2	13,5	1,6	11,8	79,92	91,94	18,46	20	0,7
25	CH-25	Normal embankment	Km14+933	~	Km15+068	135	7,3	SD	0,0		4	3	13,5	1,6	14,7	81,46	105,41	19,54	30	0,7
26	CH-26	DR-BOX	Km15+068	~	Km15+132	64	7.6	SD	0.0	3		3	20.0	1.5	11.8	99.35	88.87	0.58	10	0.8
27	CH-27	Normal embankment	Km15+132	~	Km15+313	181	6,9	SD	0,0		4	3	10,0	1,8	11,6	89,46	80,95	8,53	30	0,7
28	CH-28	UP-BOX	Km15+313	~	Km15+368	55	5,1	PVD	0,0		1	2	10,5	1,5	11,9	92,82	54,7	3,93	10	0,5
29	CH-29	Normal embankment	Km15+368	~	Km15+630	262	4,4	PVD	0,0		2	2	11,5	1,5	12,6	91,85	53,61	4,37	30	0,5
30	CH-30	Terminal road	Km15+630	~	Km15+884	254	2,5	No need	0,0			2	11,5		10,9	44,74	33,62	18,58	30	0,5

No.	SECTION	Filling process: 1st Step Height (m)	Δh	Embankment speed (cm/day)	Waiting time for consolidation (mon)	2nd Step Height (m)	Δh	Embankment speed (cm/day)	Waiting time for consolidation (mon)	3rd Step Height (m)	Δh	Embankment speed (cm/day)	Waiting time for consolidation (mon)	Total treatment period (day)	Safety factor of slope stability: 1st Step	2nd Step	3rd Step	Finish improvement
1	CH-1																	
2	CH-2	3,5		10	3	5,2		10	3	FG+Δh	0,2	10	3	350	1,848	1,526	1,406	1,464
3	CH-3	3,6		10	3	5		10	3	FG+Δh	0,2	10	3	351	1,733	1,539	1,345	1,44
4	CH-4	3.7		10	3	5.2		10	3	FG+Δh	0.5	10	7	462	1.189	1.33	1.299	1.446
5	CH-5	3.8		10	3	5.2		10	3	FG+Δh	0.2	10	3	339	1.411	1.323	1.344	1.439
6	CH-6	3,9		10	3	5,6		10	3	FG+Δh	0,5	10	7,5	473	1,32	1,229	1,27	1,413
7	CH-7	3,9		10	3	5,6		10	3	FG+Δh	0,4	10	7	457	1,403	1,263	1,249	1,48
8	CH-8	3.8		10	3	5.3		10	3	FG+Δh	0.1	10	3.7	360	1.358	1.239	1.261	1.463
9	CH-9	3,7		10	4	FG+Δh	0,6	10	9					460	1,515	1,251		1,429
10	CH-10	3,9		10	4	FG+Δh	0,4	10	9					447	1,384	1,267		1,438
11	CH-11	3.9		10	4	FG+Δh	0.6	10	9					451	1.347	1.223		1.516
12	CH-12	3.9		10	4	FG+Δh	0.6	10	9					449	1.316	1.218		1.476
13	CH-13	3,8		10	4	FG+Δh	0,5	10	8					422	1,361	1,246		1,516
14	CH-14	3,9		10	4	FG+Δh	0,3	10	9					446	1,571	1,357		1,484
15	CH-15	3.8		10	4	FG+Δh	0.4	10	8					421	1.109	1.238		1.423
16	CH-16	3.7		10	4	FG+Δh	0.6	10	9.6					480	1.52	1.218		1.444
17	CH-17	3,8		10	4	FG+Δh	1,8	10	5,5					354	1,68	1,207		1,57
18	CH-18	3,9		10	4	FG+Δh	0,4	10	9					457	1,449	1,213		1,435
19	CH-19	3.8		10	3	5.5		10	3	FG+Δh	0.1	10	5	404	1.666	1.415	1.34	1.42
20	CH-20	3.8		10	3	5.3		10	3	FG+Δh	0.3	10	5	410	1.637	1.47	1.295	1.424
21	CH-21	3,8		10	4	FG+Δh	0,1	10	6					356	1,522	1,335		1,429
22	CH-22	3.9		10	3	5.7		10	3	FG+Δh	0.1	10	3	345	1.823	1.528	1.417	1.458
23	CH-23	3.9		10	4	FG+Δh	0.2	10	7					394	1.565	1.345		1.495
24	CH-24	3,9		10	4	FG+Δh	0,4	10	5,5					355	1,471	1,246		1,465
25	CH-25	3,8		10	3	5,5		10	3	FG+Δh	0,3	10	6	443	1,817	1,596	1,395	1,457
26	CH-26	3.7		10	3	5.4		10	3	FG+Δh	0.1	10	3	355	1.991	1.648	1.388	1.474
27	CH-27	3,9		10	3	5,6		10	3	FG+Δh	0,2	10	3	347	1,739	1,446	1,356	1,431
28	CH-28	3,8		10	4	FG+Δh	0,1	10	6					356	1,611	1,456		1,503
29	CH-29	3,9		10	4	FG+Δh	0,2	10	7					379	1,309	1,327		1,468
30	CH-30	FG+Δh	0,2	10	10									328	1,389			1,518

Notes:

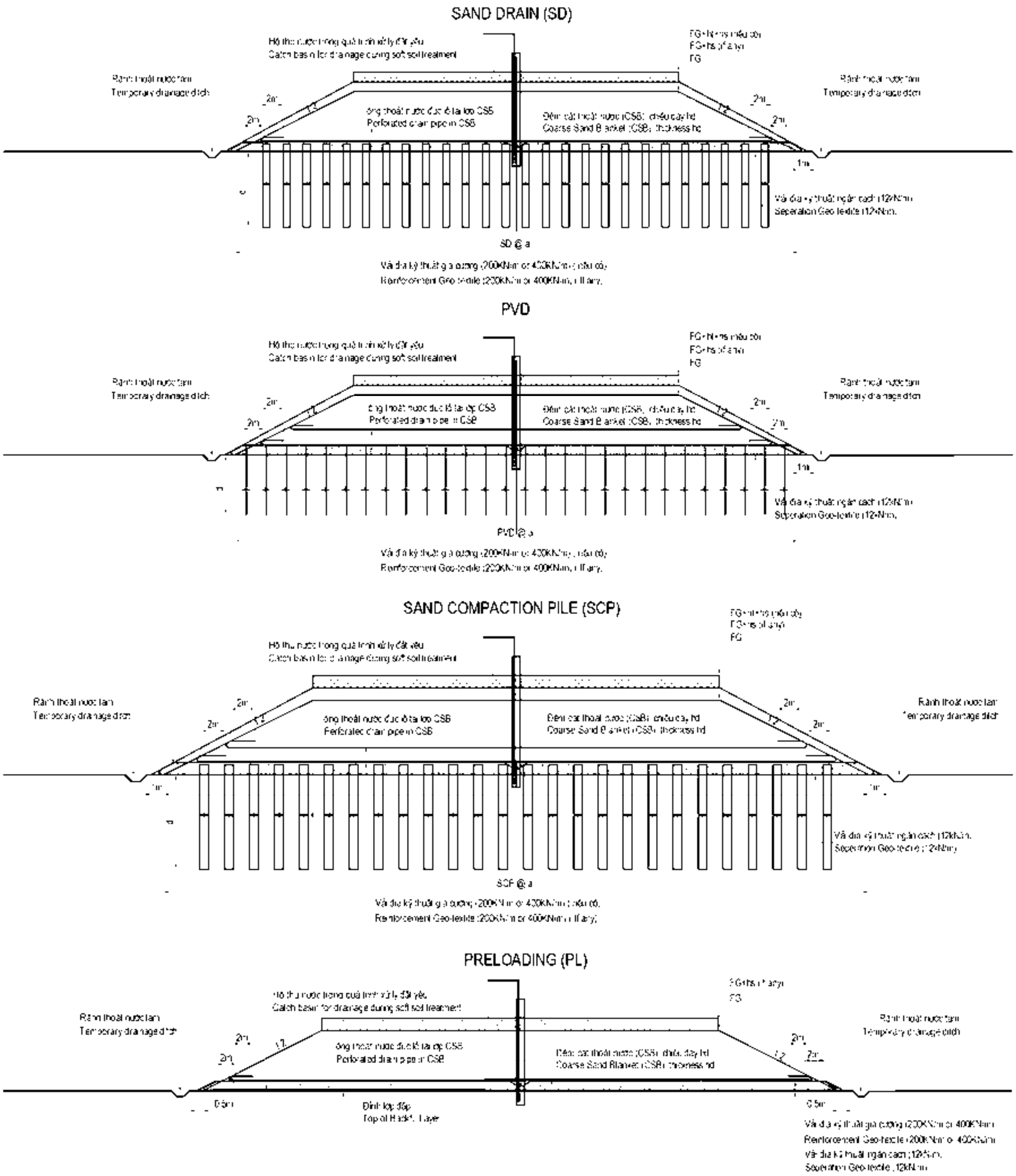
1)Installation of Geotextile (number, location, etc.) should be decided by the Engineer with careful checking during construction stage based on soil test result.

2)Installation of Course Sand Blanket (setting elevation, thickness, etc.) should be decided by the Engineer during construction checking the site condition.

Source : Study Team

7.5.4 Typical Cross Section of Soft Soil Treatment

Typical cross section of soft soil treatment is as shown in Figure 7.5.4-1.



Source : Study Team

Figure 7.5.4-1 Typical cross section of soft soil treatment

7.6 ROAD STRUCTURE DESIGN

7.6.1 Road Structures

Road structures in this project are listed below (exclude bridge structure);

- ➢Underpass box culvert
- ➢Drainage/Irrigation box culvert and pipe culvert
- ➢Retaining wall
- ➢Piled slab behind bridge abutment

7.6.1.1 Underpass Box Culvert

Location of underpass box culvert is summarized in Chapter 7.1.6.

Road class of underpass is selected as Class VII (TCVN5729-97 item4.8) by conducting site investigations.

Size of box culvert is agreed by local authorities.

1) Carriageway width : 3.0m
2) Shoulder: 0.5m*2=1.0m
3) Required vertical clearance: 3.2m

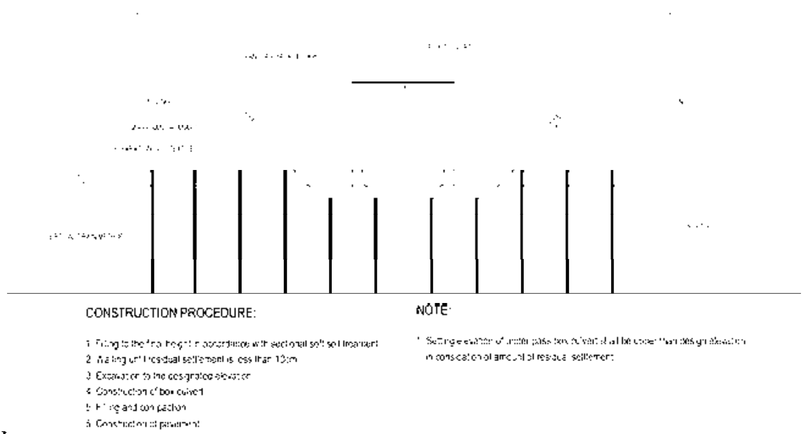
<u>Foundation of culvert</u>

Shallow foundation without piles is selected for this project in order to avoid the unevenness and ensure comfortableness for driver. Pile foundation is popular method for box culvert in Vietnam so far, however, unevenness at the connection point between box and embankment is usually seen because of difference of amount of residual settlement.

Instead of using pile foundation, soft soil treatment (Sand drain or PVD, see chapter 7.5) is applied under box culvert to decrease residual settlement. Allowable residual settlement is reduced into 10cm for box culvert section, although 20cm is defined in the standard (see Table 7.5.1-1). The reason why 10cm is applied is to minimize the risk of differential settlement because shallow foundation is applied for box culvert foundation instead of pile foundation.

Box culverts are constructed after completion of settlement of embankment.

Construction procedure is shown in Figure 7.6.1-1.



Source : Study Team

Figure 7.6.1-1 Construction procedure of box culvert

7.6.1.2 Drainage/Irrigation box culvert and pipe culvert

Details of drainage/irrigation box culvert and pipe culvert are summarized in Chapter 7.5.

All of the size of irrigation culvert was investigated and studied about the necessary size, and based on agreement with local authorities. In case length of pipe culvert is over 30m, minimum diameter of pipe culvert is defined as 1.50m according to TCVN 4054-2005.

Culvert type for design is classified as follows.

Table 7.6.1-1 List of Drainage/Irrigation box culvert type

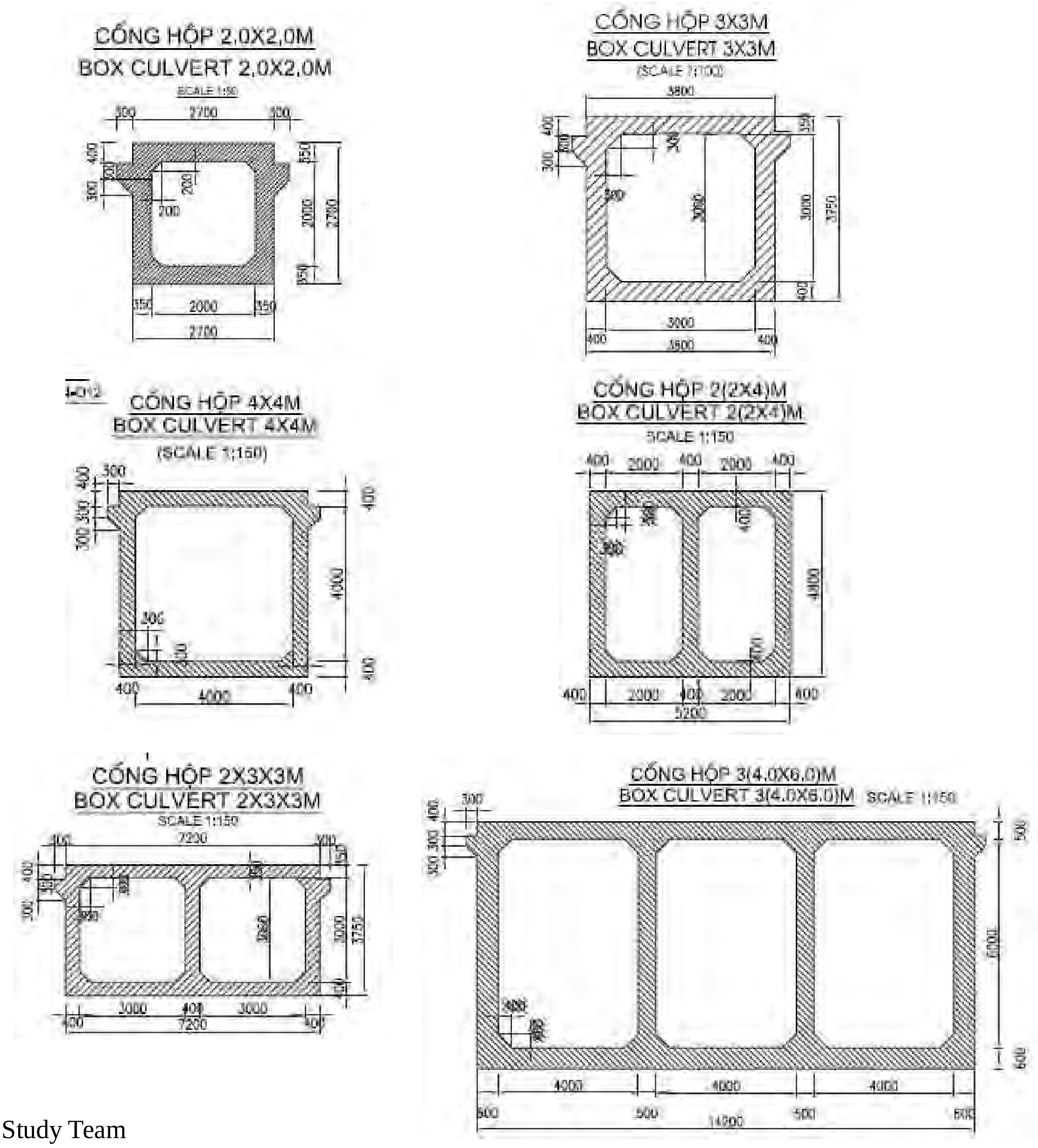
Type	B(m) x H(m)	Location	Note
1	2.00 x 2.00	Km 0+788, Km 2+390, Km 2+650, Km 14+907	
2	3.00 x 3.00	Km 0+915, Km 10+805	
3	4.00 x 4.00	Km 14+620	
4	2 x 2.00 x 4.00	Km 14+650 (Left side)	With water gates
5	2 x 3.00 x 3.00	Km 4+140	
6	3 x 4.00 x 6.00	Km 15+100	

Source : Study Team

Table 7.6.1-2 List of Drainage/Irrigation pipe culvert type

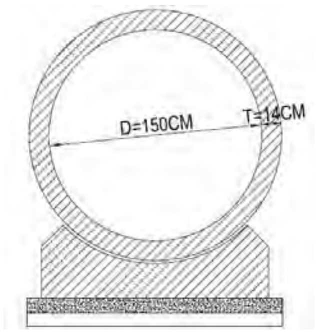
Type	Diameter(m)	Location	Note
1	1.50	Km 0+225, Km 10+090, Km 10+659,Km 15+520	Length is over 30m for all culverts.

Source : Study Team



Source : Study Team

Figure 7.6.1-2 Size of Drainage/Irrigation box culvert

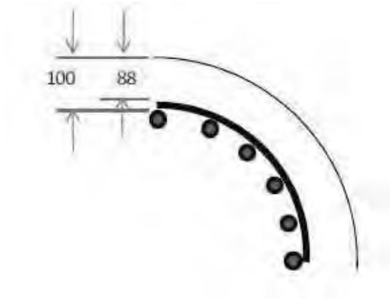


Source : Study Team

Figure 7.6.1-3 Size of Drainage/Irrigation pipe culvert

Protection of reinforcing bars of box culvert from salt water

For the purpose of protection of reinforcing bars from salt water, concrete covers for main bar is decided to be 100mm following bridge standard (22 TCN-272-05). This concept is same as bridge foundation (see Figure 7.6.1-4).



Note) This figure shows an example for bridge foundation.

Source : Study Team

Figure 7.6.1-4 Minimum concrete cover for box culvert

Foundation of culvert

Shallow foundation without pile foundation is selected for drainage/irrigation box culvert as well as underpass box culvert. Regarding allowable residual settlement, 10cm for large size box culvert (H>2.0m), 20cm for small size box culvert (H≤2.0m) and 30 cm for pipe culvert section are applied.

Box culverts and pipe culverts are constructed after completion of settlement of embankment.

7.6.1.3 Retaining wall

1)Retaining wall behind abutment of approach bridge

In SAPROF, retaining walls were planned near abutment of approach bridge. They are deleted in Detailed Design stage after conducting comparative study with using updated topographic survey data and geological survey data.

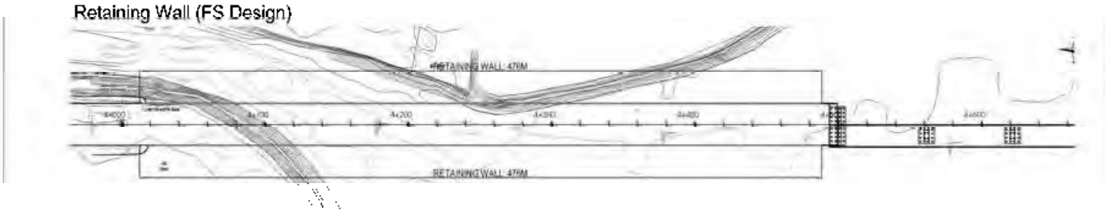
Table 7.6.1-3 Technical comparison for retaining wall between SAPROF and D/D

	SAPROF	D/D	Reason of change
Hai An	Km4+20-Km4+500 (Behind A1 abutment of approach bridge) L=480m, U-type	Deleted (Change to embankment slope with soft soil treatment and piled slab)	The plan road along the highway has not been specified yet. It will be constructed by Hai Phong city after completion of the highway. Therefore, there is no reason to consider land restriction by using retaining wall for the future road.
Cat Hai	Km9+945-Km10+126 (Behind A2 abutment of approach bridge) L=181m, U-type	Deleted (Change to embankment slope with soft soil treatment and piled slab)	There is no problem of effect of lateral displacement and land slide by earth pressure at A2 abutment section and no difficulty for land acquisition at this section.

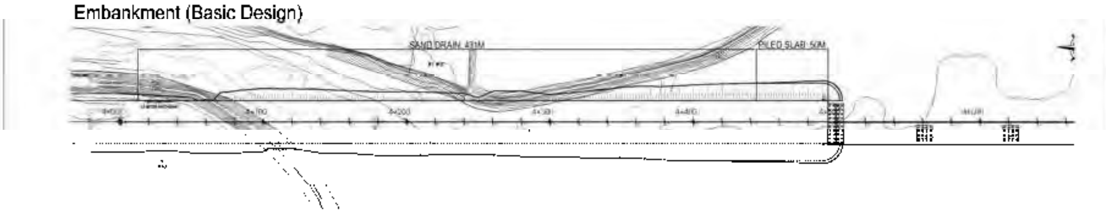
Source : Study Team

Comparison between Retaining wall and Embankment (behind A1 abutment)

Retaining Wall (FS Design)



Embankment (Basic Design)



Cost comparison Table

FS Design

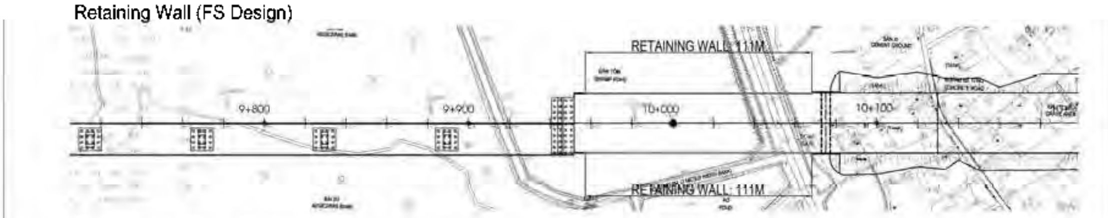
Items	Unit	Quantity	Unit price(mil.VND)	Cost(mil.VND)
Retaining Wall (incl. pile foundation)	m	952	300	285,600
Embankment	m3	79,000	0.15	11,850
Piled Slab	m2	0	12.6	0
Sand drain	m3	0	0.20	0
Grouted Riprap	m2	0	0.55	0
Total				297,450

Basic Design

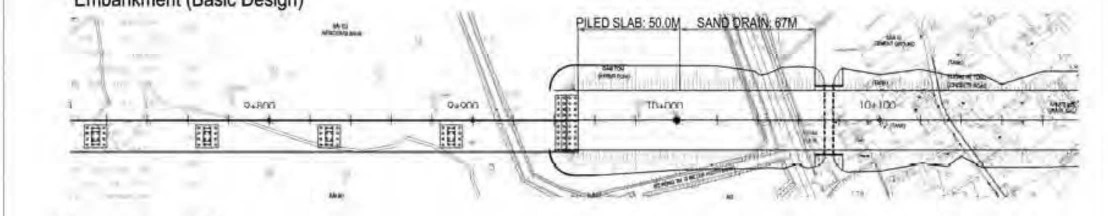
Items	Unit	Quantity	Unit price(mil.VND)	Cost(mil.VND)
Retaining Wall (incl. pile foundation)	m	0	300	0
Embankment	m3	108,000	0.15	16,200
Piled Slab	m2	2,850	12.6	35,910
Sand drain	m3	407,000	0.20	81,400
Grouted Riprap	m2	11,600	0.55	6,380
Total				139,890

Comparison between FS and Basic Design (behind A2 abutment)

Retaining Wall (FS Design)



Embankment (Basic Design)



Cost comparison Table

FS Design

Items	Unit	Quantity	Unit price(mil.VND)	Cost(mil.VND)
Retaining Wall (incl. pile foundation)	m	222	300	66,600
Embankment	m3	21,000	0.15	3,150
Piled Slab	m2	0	12.6	0
Sand drain	m3	0	0.20	0
Grouted Riprap	m2	0	0.55	0
Total				69,750

Basic Design

Items	Unit	Quantity	Unit price(mil.VND)	Cost(mil.VND)
Retaining Wall (incl. pile foundation)	m	0	300	0
Embankment	m3	31,000	0.15	4,650
Piled Slab	m2	2,800	12.6	35,280
Sand drain	m3	40,000	0.20	8,000
Grouted Riprap	m2	2,000	0.55	1,100
Total				49,030

Source : Study Team

Figure 7.6.1-5 Cost comparison for retaining wall between SAPROF and D/D

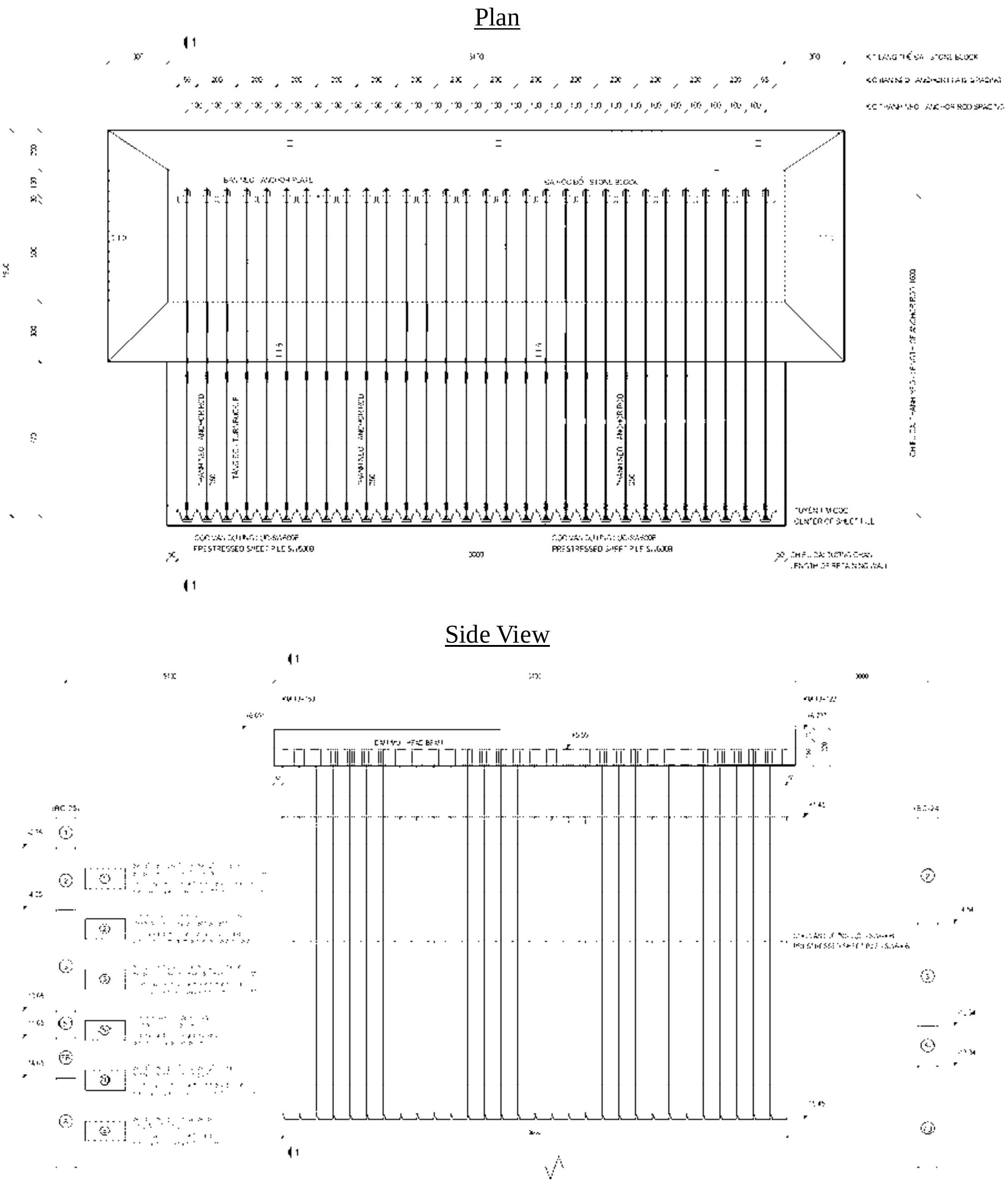
2) Retaining wall in Cat Hai Island

Retaining wall is designed at following section;

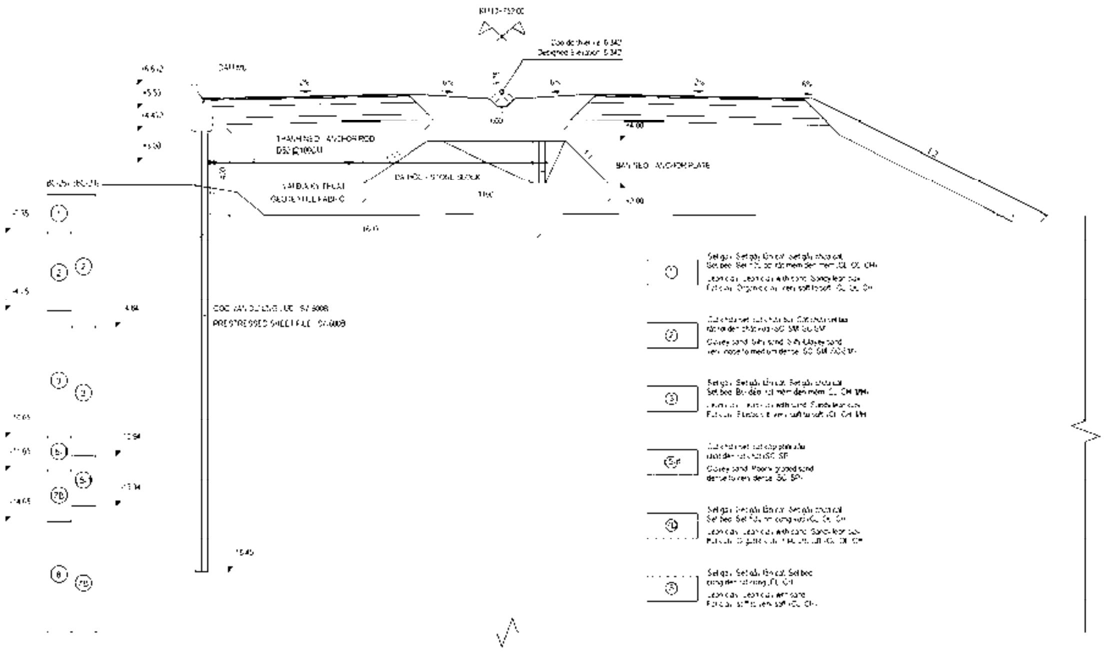
- Km13+722-Km13+753: L=31m, Dong Bai Commune, near pagoda (see Figure 7.1.5-1)

Type of retaining wall is "Concrete sheet pile wall (with anchor rod)" as shown in Figure 7.6.1-6. In case of applying normal retaining wall, excavation for the foundation will affect the pagoda area and wall of the pagoda needs to be removed. And it is also necessary to ensure passage of local people in front of the pagoda even during construction. Therefore, concrete sheet pile wall which doesn't need temporary excavation is applied in order to minimize the impact by the construction.

The retaining wall will be constructed in soft soil area. Soft soil treatment such as PVD and sand blanket installation behind retaining wall should be carried out in the same manner as that in normal embankment section after retaining wall is constructed.



Cross section



Source : Study Team

Figure 7.6.1-6 Details of Concrete Sheet Pile Wall

7.6.1.4 Piled Slab

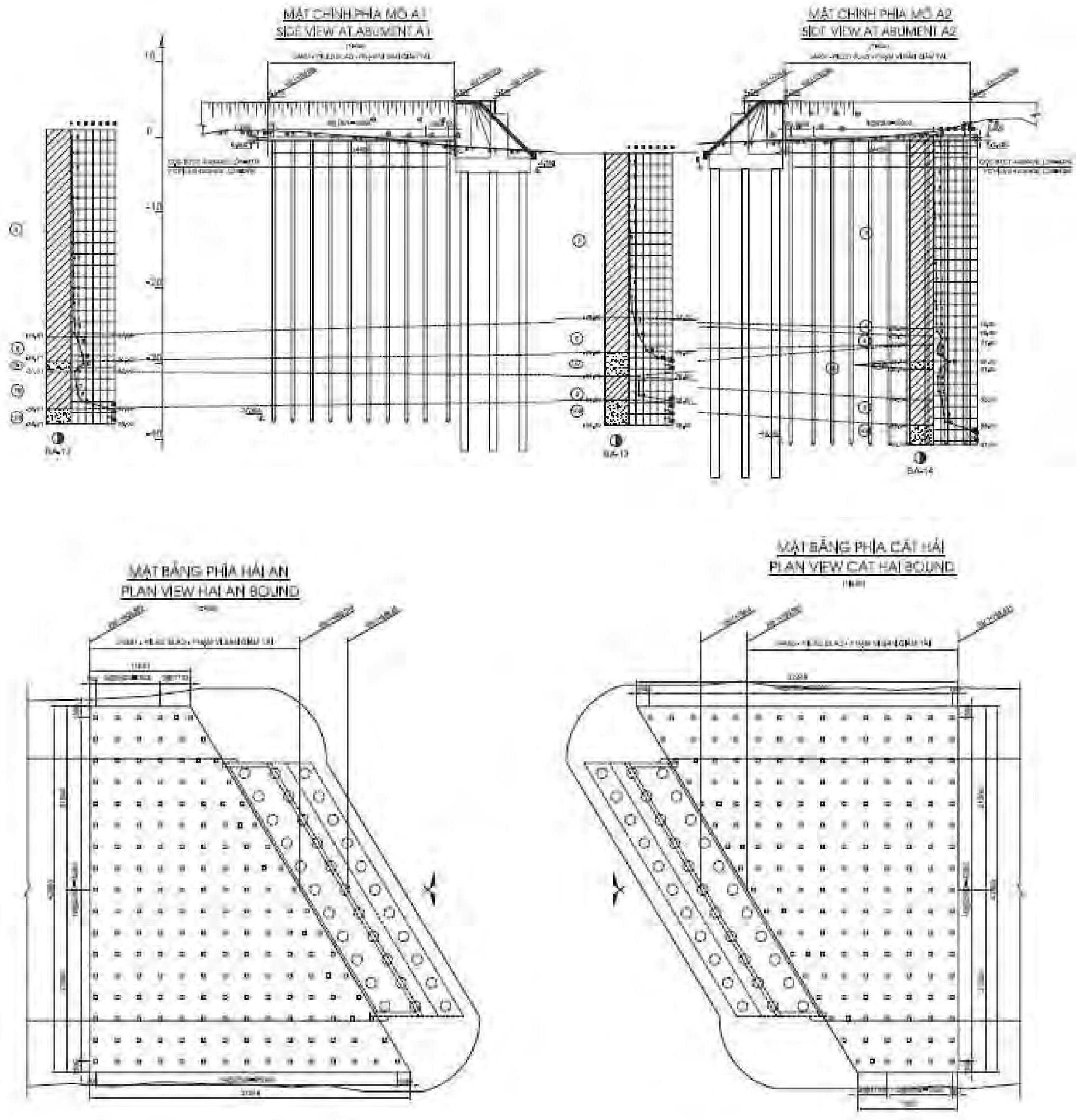
At the high embankment section near bridge abutment, piled slab is applied instead of soft soil treatment.

(1) Location:

1) Km1+635.569-Km1+660.019 (L=24.45m, A1 abutment of Cam River Bridge)
2) Km1+739.981-Km1+764.431 (L=24.45m, A2 abutment of Cam River Bridge)
3) Km4+456-Km4+497 (L=41m, A1 abutment of Approach Bridge)
4) Km9+948.5-Km9+971.9 (L=23.4m, A2 abutment of Approach Bridge)

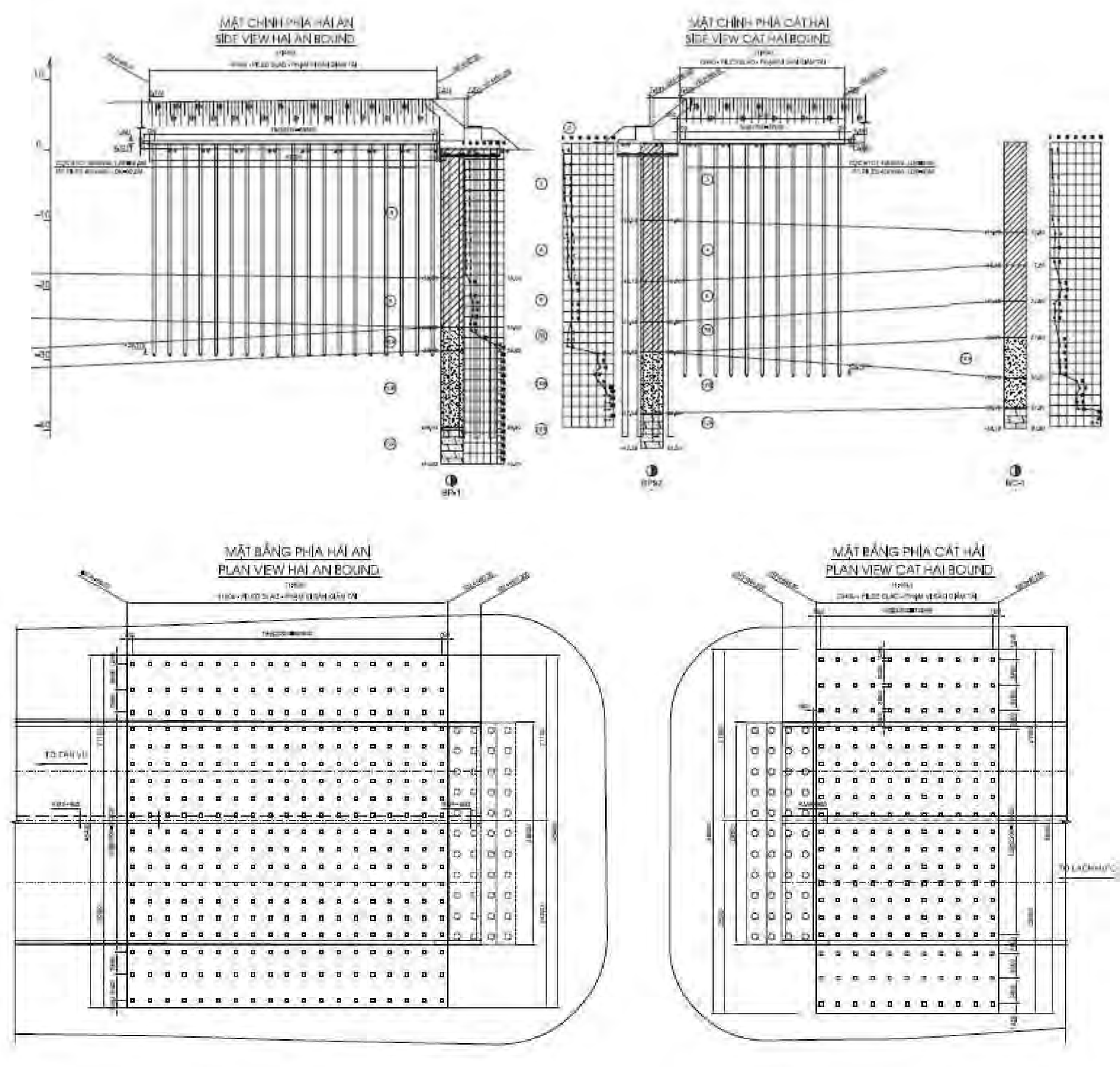
(2) Details of Piled Slab

Details of piled slab are shown below.



Source : Study Team

Figure 7.6.1-7 Details of piled slab at Cam River Bridge



Source : Study Team

Figure 7.6.1-8 Details of piled slab at Approach Bridge

Note)

-Length of pile in drawing is estimated, so the real length of pile shall be decided at the site by the engineer based on the result of static load test and verified by driving formula.

-Number of test is 2% of total piles for each piled slab based on provisions of AASHTO LRFD 1998.

-Length of pile in drawing is from the soffit of piled slab to pile tip. Therefore, the manufactured length of pile shall be 1 m longer from this length.

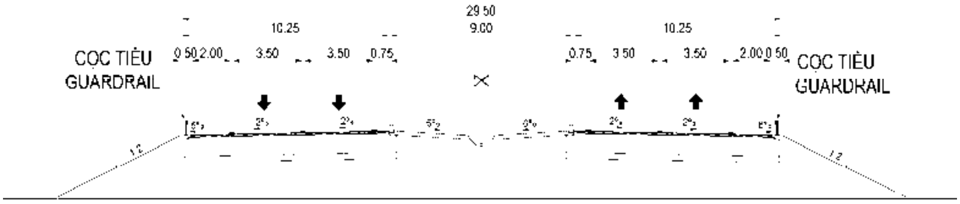
7.7 Traffic Safety

Traffic safety devices are defined as devices to direct and assist road users so that they may be able to travel safely and efficiently on a road network.

7.7.1 Guardrail

Guardrail refers to a facility that has the purpose of preventing a vehicle from running the wrong way or from straying out of its lane into an opposing lane, as well as restoring a wayward vehicle back to its correct path with minimum damage to the vehicle and its passengers. A secondary function of guardrail is to guide driver's eyes.

In this project, guardrail is installed both sides at outer shoulder as shown in Figure 7.7.1-1.



Source : Study Team

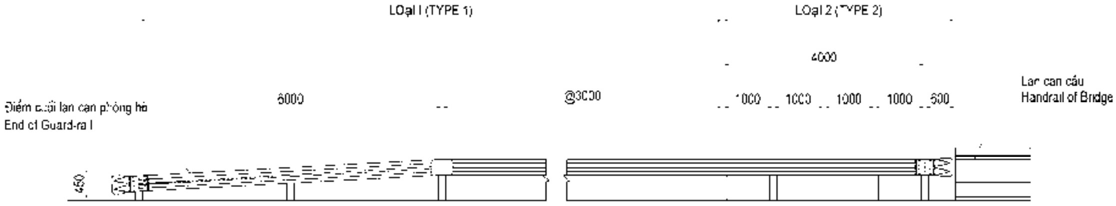
Figure 7.7.1-1 Installation of guardrail (cross section)

Regarding type of guardrail, there are two types, Type 1 for normal embankment section and Type 2 for near bridge wingwall section. Location of guardrail is summarized in Table7.7.1-1. Installation interval is 3m for Type 1 and 1m for Type 2 as shown in Figure 7.7.1-2.

Table 7.7.1-1 Schedule of guardrail

Trái - Left						Phải - Right					
TT No	Đoạn (Km...-Km....) Section (Sta---to Sta)		Đơn vị Units	Loại Type	Chiều dài Lenght	TT No	Đoạn (Km...-Km....) Section (Sta---to Sta)		Đơn vị Units	Loại Type	Chiều dài Lenght
Tuyến Chính - MainRoad											
1	Km0+000.00	Km1+652.00	m	Loại 1 (Type 1)	1655.48	1	Km0+000.00	Km1+660.00	m	Loại 1 (Type 1)	1663.48
2	Km1+652.00	Km1+657.00	m	Loại 2 (Type 2)	5.00	2	Km1+660.00	Km1+665.00	m	Loại 2 (Type 2)	5.00
3	Km1+735.00	Km1+740.00	m	Loại 2 (Type 2)	5.00	3	Km1+743.00	Km1+748.00	m	Loại 2 (Type 2)	5.00
4	Km1+740.00	Km4+496.00	m	Loại 1 (Type 1)	2775.5	4	Km1+748.00	Km4+496.30	m	Loại 1 (Type 1)	2768.27
5	Km4+496.00	Km4+501.00	m	Loại 2 (Type 2)	5.00	5	Km4+496.30	Km4+501.30	m	Loại 2 (Type 2)	5.00
6	Km9+944.00	Km9+949.00	m	Loại 2 (Type 2)	5.00	6	Km9+944.20	Km9+949.20	m	Loại 2 (Type 2)	5.00
7	Km9+949.00	Km15+560.00	m	Loại 1 (Type 1)	5626.9	7	Km9+949.20	Km11+506.00	m	Loại 1 (Type 1)	1556.69
8	Km15+590.00	Km15+629.94	m	Loại 1 (Type 1)	39.94	8	Km11+540.00	Km15+560.00	m	Loại 1 (Type 1)	4002.2
						9	Km15+590.00	Km15+629.94	m	Loại 1 (Type 1)	39.94
Đoạn chuyển tiếp vào Cảng - Transition Section of terminal road											
1	Km15+629.94	Km15+883.86	m	Loại 1 (Type 1)	253.92	1	Km15+629.94	Km15+883.86	m	Loại 1 (Type 1)	253.92
	Total Length				10371.74						10304.5

Source : Study Team

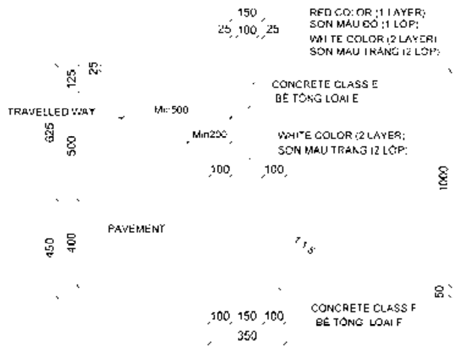


Source : Study Team

Figure 7.7.1-2 Guardrail layout

7.7.2 Guard post

Guard post is installed only for frontage road with 20m interval both sides at shoulder for the purpose of guiding the driver's eyes. Details of guard post are shown in Figure 7.7.2-1.

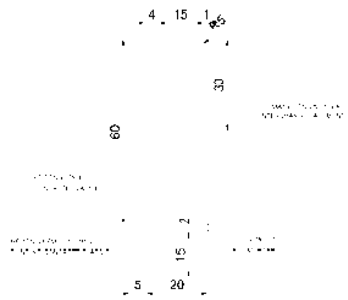


Source : Study Team

Figure 7.7.2-1 Details of guard post

7.7.3 Concrete curb

Concrete curbs are installed at the median of throughway. Details of concrete curb are shown in Figure 7.7.3-1.

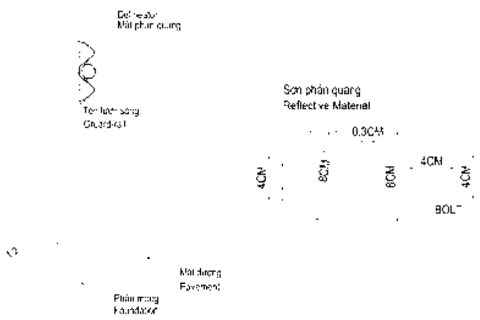


Source : Study Team

Figure 7.7.3-1 Details of concrete curb

7.7.4 Delineators

Delineators are installed at column of guardrail with interval of 48m. Purpose of installation of delineators is to indicate the roadway alignment by using light retro-reflecting material. Details of delineator are shown in Figure 7.7.4-1.



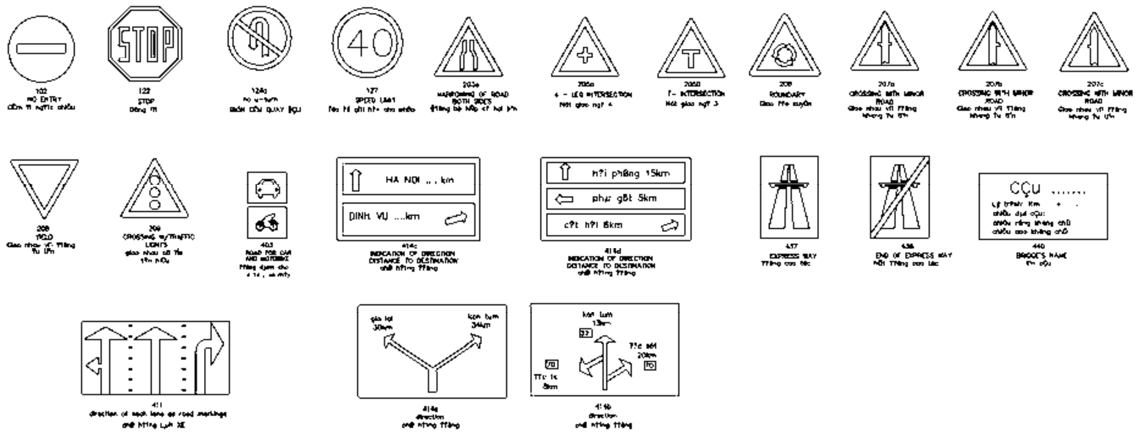
Source : Study Team

Figure 7.7.4-1 Details of delineator

7.7.5 Traffic signs

Road signs give drivers the necessary guidance, warnings and instructions to ensure a safe and smooth traffic flow. Traffic signs are similar to ordinary commercial signs and billboards in the sense that they transmit information to the person who sees them. However, they are of much greater public importance.

Design of traffic signs should follow the Vietnamese Standard 22 TCN237-01. Details of traffic signs which are used in this project are shown in Figure 7.7.5-1. Sign installation plan can be checked in the Detailed Design Drawings.



Source : Study Team

Figure 7.7.5-1 Details of traffic signs

7.7.6 Road marking

Road markings essentially consist of continuous lines, broken lines and symbols. These may be applied on the road surface in different arrangements to convey distinct messages to the road user. These markings may be either of road paint or of thermoplastic material. Reflective studs are alSourced along with road marking for a better effect both during the day and the night.

Design of road marking should follow the Vietnamese Standard 22 TCN237-01. Details of road marking can be checked in the Detailed Design Drawings.

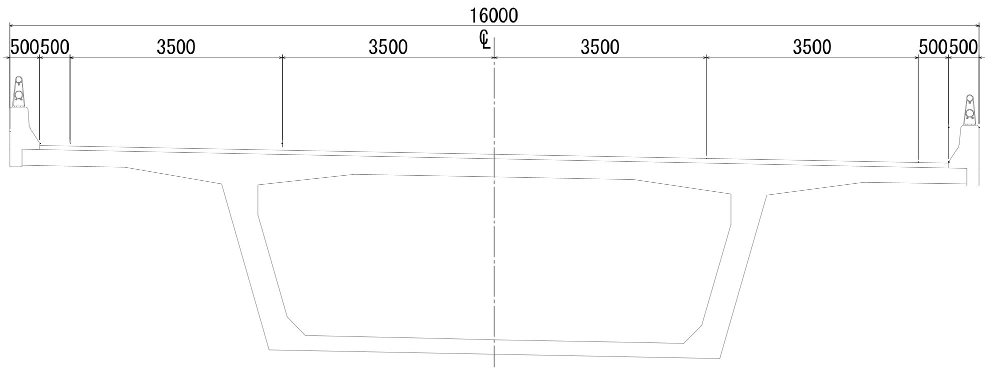
CHAPTER 8 DESIGN OF BRIDGES

8.1 Design Conditions

8.1.1 Basic Conditions

- Bridge Type : Prestressed Concrete Bridge
- Structure Type : PC continuous box girder (except for Cam River Bridge)
- Bridge Length : 4433.7m + 490.0m + 519.2m (except for Cam River Bridge)
- Road Spec : Highway class III , plain terrain
- Design Speed : 80km/h
- Live Load : AASHTO LRFD

- Width Composition



Source: Study Team

Figure 8.1.1-1 Width Composition of Superstructure

- Longitudinal Grade : less than 5.0%
- Cross Grade : 2.0%

8.1.2 Material to be used

8.1.2.1 Superstructure

- Concrete for Main Girder : σck = 40N/mm^2 [for main bridge]
  : σck = 40, 50N/mm^2 [for approach bridge]
  : σck = 30, 40N/mm^2 [for Cam River bridge]
- Reinforcing Bars : SD345
- PC Tendon: 19S15.2 [Internal for Main Bridge, External for Approach Bridge] SWPR7BL
  : 12S15.2 [Internal for Approach Bridge] SWPR7BL
  : 1S28.6 [Transversal for Main Bridge and Approach Bridge] SWPR19BL
  : 7S12.7 [Internal for Cam River Bridge] SWPR7BL
  : 1S19.3 [Transversal for Cam River Bridge] SWPR19L

Table 8.1.2-1 Specification for PC Cable

		Longitudinal Cable			Transversal Cable		(N/mm^2)
		19S15.2	12S15.2	7S12.7	1S28.6	1S19.3	
		SWPR7BL	SWPR7BL	SWPR7BL	SWPR19L	SWPR19L	
Ultimate tensile strength		1850.0	1850.0	1850.0	1800.0	1800.0	
Yield strength		1600.0	1600.0	1600.0	1500.0	1500.0	
Allowable tensile strength	during prestressing	1440.0	1440.0	1440.0	1350.0	1350.0	0.9σpy
	after prestressing	1295.0	1295.0	1295.0	1260.0	1260.0	0.7σpu
	at design load	1110.0	1110.0	1110.0	1080.0	1080.0	0.6σpu
Remarks		JSHB-2002, JIS 3536-1999					

Source: Study Team

8.1.2.2 Substructure & Foundation

- Concrete : : σck = 40, 28N/mm2 [for Substructure]
  : σck = 30N/mm2 [for Pile]
- Reinforcing bars : SD345
- Steel pipe pile : SKK400
- Steel sheet pike : SKY400, SKY490

8.1.3 Conditions of Design Load

(1) Self weight

Unit weight of reinforcement concrete**24.50** kN/m^3

Unit weight of structural steel**77.01** kN/m^3

(2) Superimposed dead loads

a) Wearing surface weight:

Traffic lanes:

7.0 cm asphalt + 0.4 cm water proof membrane: **2250** kg/m^3

For B=16.0m bridge:

0.074m x 15.0m x (2250 x 0.00981 kN/m^3)=***24.5 kN/m***

b) Curb and rail at side:

Concrete median + steel rail:

0.31m^2 x 2 x 24.5kN/m^3 + 5.0kN/m=***20.2 kN/m***

c) Miscellaneous dead loads:

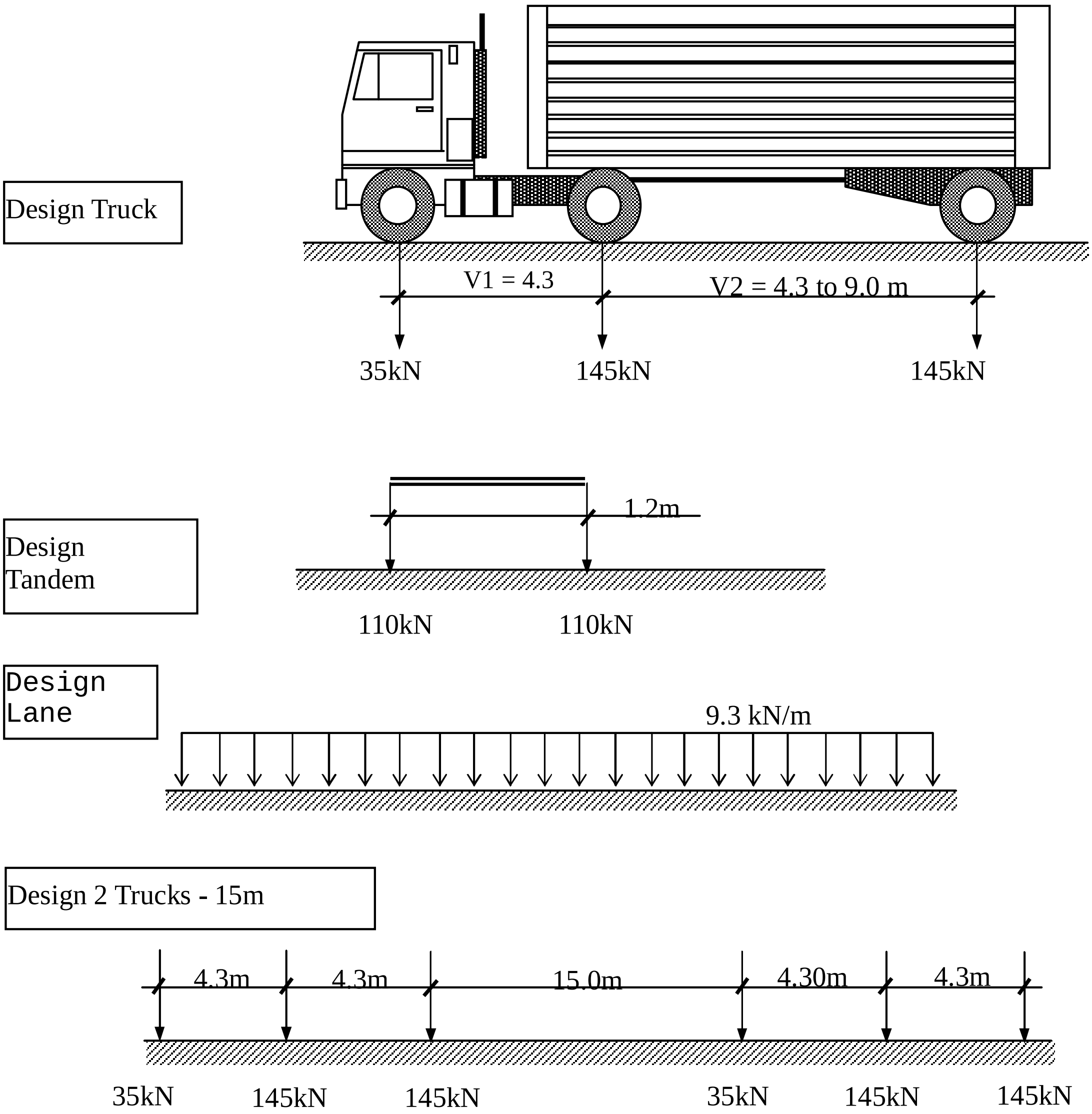
(traffic lights, water drainage …)=***2.20 kN/m***assumed

Total of superimposed dead load =46.9 kN/m

(3) Live Loads

Vehicular live loading on the roadways of bridges or incidental structures, designated HL-93, shall consist of a combination of the:

- Design Truck or design Tandem, and
- Design lane load



Source: 22TCN 272-05

Figure 8.1.3-1 Conditions of Live Load

Dynamic load allowance IM of vehicular takes as below table.

Table 8.1.3-1 Dynamic Load Allowance

Components	IM
Deck Joints - all limit states	75%
All other components	
• Fatigue and fracture limit state	15%
• All other limit states	25%

Source: Study Team

Dynamic load allowance need not be applied to foundation components that are entirely below ground level.

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

- The effect of the design tandem combined with the effect of the design lane load, or
- The effect of one design truck with the variable axle combined with the effect of the design lane load, and
- For both negative moment between points of contra flexure under a uniform load on all spans, and reaction at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 15 000 mm between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 145 000-N axles of each truck shall be taken as 4300 mm.

Multiple presence factors “m” for live load.

Table 8.1.3-2 Multiple Presence Factor

Number of loaded lanes	Multiple presence factors “m”
1	1.20
2	1.00
3	0.85
>3	0.65

Source: Study Team

(4) Braking force

The braking force shall be taken as 25 percent of the axle weights of the design truck or design tandem. This braking force shall be placed in all design lanes which are considered to be loaded in accordance with Article 3.6.1.1.1 and which are carrying traffic headed in the same direction. These forces shall be assumed to act horizontally at a distance of 1800 mm above the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future. The multiple presence factors specified in Article 3.6.1.1.2 shall apply.

For B=15.7m bridge:

Total number of traffic lanes:

Round down(14.00m/3.5m , 0)=4 lanes for safety

Multiple presence factors “m”=0.65

25% of design truck or design tandem

25% x 4lanes x (35+145*2)kN x 0.65=211.25 kN

25% x 4lanes x (110*2)kN x 0.65=143.0 kN

(5) Temperature load

1) Uniform temperature:

The metrological data recorded from 1984 to 2008 at Hon Dau Station located in Do Son District, Hai Phong City, were collected and a statistic study was performed for obtaining the maximum and minimum shade air temperatures appropriate to the 100 year return period for the site.

The distribution model used for the statistic analysis is "Log Pearson Type III Distribution".

The results are shown in the table below.

Considering the results, the design temperature range is defined as 40 degree in Celsius.

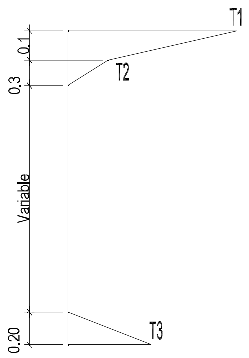
Table 8.1.3-3 Temperature Ranges (degree in Celsius)

	Recorded In 1984-2008	20 year return period	33 year return period	100 year return period
Max. Temperature	38.7	38.45	38.79	39.46
Min. Temperature	6.6	4.11	3.35	1.76
Temperature Range	32.1	34.34	35.44	37.7

Source: Study Team

2) Gradient temperature:

Positive gradient

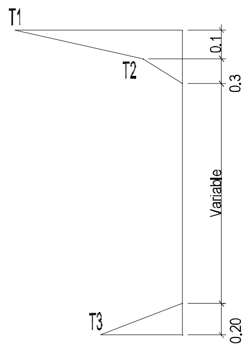


$T1 = 23^0\,C$

$T2 = 6^0\,C$

$T3 = 3^0\,C$

Negative gradient



$T1 = -7^0\,C$

$T2 = -1^0\,C$

$T3 = 0^0\,C$

(6) Wind load

1) Static wind load:

The bridge locates at Hai Phong province. According to *TCVN 2737-1995* - Appendix E, bridge is in wind region IV. Following *22TCN-272-05*, sec. 3.8.1:

Design wind speed $V = S.V_B$ where V_B = ***59.0 m/s***

V_B – 3 second gust wind velocity with 100 years return period can take from table as below:

Table 8.1.3-4 Wind Velocity

Wind region	V_B (m/s)
I	38
II	45
III	53
IV	**59**

Source: TCVN 2737-1995

S – correct coefficient for wind zone and elevation of deck slab can takes from table as below:

Table 8.1.3-5 Correct Coefficient for Wind Zone and Elevation

Elevation of deck slab upper ground area or water plane	Exposed area	Forest, houses area with maximum trees, houses height 10m	houses area with houses height > 10m
10	1.09	1.00	0.81
20	1.14	1.06	0.89
30	1.17	1.10	0.94
40	1.20	1.13	0.98
50	1.21	1.16	1.01

Source: 22TCN 272-05

Elevation of deck slab upper ground area - exposed areaH_{ele}=23.2m

Horizontal wind pressure $P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)

C_d– obstacle coefficient depends on ratio b/d

b– overall width between handrails

d– superstructure height including solid parapet

For B=15.00m bridge:

b=15.00m

d= (7.5+3.2)/2 + 1 = 6.35m

S= 1.17

b/d =15.00/6.35= 2.36

C_d = 1.34

$P_D = 0.0006 \times (59.0 \times 1.17)^2 \times 1.34 =$ ***4.1 kN/m²***

Horizontal wind load = 4.1*6.35 = 26.1 KN/m

Longitudinal wind load = 25%*26.1 = 6.5 KN/m

For piers:

C_d = 1.9

$P_D = 0.0006 \times (59.0 \times 1.17)^2 \times 1.9 =$ ***5.43 kN/m²***

2) Wind load on vehicular:

For strength combination III, wind load on vehicular and on structure have to simultaneously consider (wind speed 25m/s). Wind load on vehicular in transversal direction, is 1.5 kN/m at 1.8m height from asphalt surface. Wind load on vehicular in longitudinal direction is 0.75 kN/m at 1.8m height from asphalt surface.

(7) Earth Quake

1) Acceleration coefficient, site coefficient and response spectrum:

As for earth quake distribution map of Viet Nam in *22TCN-221-95*, the bridge locates in seismic zone with grade 7 following MSK-64 scale. According to *22TCN-272-05* sec 3.10.4, the site of the bridge is located in seismic zone No.2. With 0.09 <A< 0.19.

Table 8.1.3-6 Acceleration and Seismic Zone

Acceleration coefficient	Seismic zone	MSK-64 scale
$A \leq 0.09$	1	grade ≤ 6.5
$0.09 < A \leq 0.19$	2	$6.5 <$ grade ≤ 7.5
$0.19 < A < 0.29$	3	$7.5 <$ grade ≤ 8

Source: 22TCN 272-05

According to “Seismic standard *TCXDVN375-2006”*, peak ground acceleration coefficient A=0.1291g.

Coefficient site S: according to the geological survey, ***soil profile type is II***.

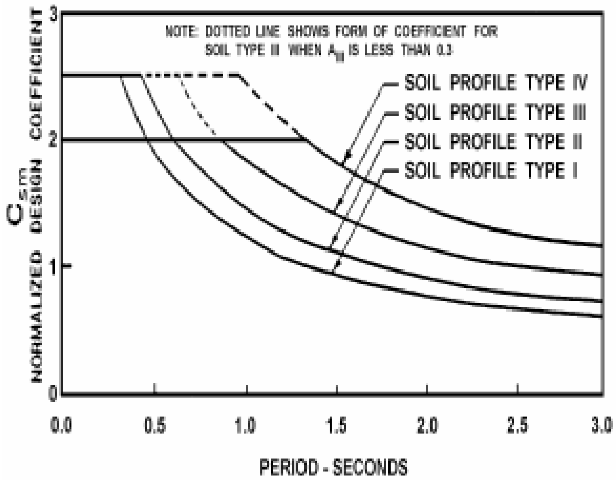
Table 8.1.3-7 Site Coefficient

Site coefficient	Soil profile type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

Source: 22TCN 272-05, Study Team

Seismic design response spectrum for soil profile type III for the bridge is stipulated in specification for bridge design 22TCN-272-05.

Response spectrum:



Source: 22TCN 272-05

Figure 8.1.3-2 Response Spectrum

2) Response modification factor:

Because of important of the bridge - ***Critical category***, response modification factors show in table as below are proposed in design.

Table 8.1.3-8 Response Modification Factor

Components	R
Single column	1.5
Multiple column bents	1.5
Connection: columns, piers, or pile bents to cap beam or superstructure; columns or piers to foundations	1.0
Foundations	1.0

Source: 22TCN 272-05, Study Team

3) Damping ratio:

Damping ratio for structures members is 5%.

4) Analysis:

Seismic demands shall be determined by elastic response spectrum analysis. The number of modes included in the analysis shall be sufficient to get a participating mass of approximately 85-90%. The seismic response spectrum is defined according to Section 3.10.6 of 22TCN-272-05. Complete Quadratic Combination (CQC) method is used. Combination of seismic force effects in different directions is used as follows: 100% for one of the perpendicular directions combined with 30% for the other perpendicular direction.

Response Spectrum - Single mode method is used for result checking.

5) Liquefaction evaluation:

Liquefaction is a phenomenon that results in a loss of shear strength due to a decrease in effective stress resulting from generation of excess pore pressures in sandy soil. The potential for liquefaction shall be based on findings from soil investigation. For an alluvial sandy layer having all of the following three conditions, liquefaction assessment shall be conducted.

- Saturated soil layer having ground water level higher than 10m below the ground surface and located at a depth less than 20m below the ground surface.
- Soil layer containing a fine content (*FC*) of 35% or less, or soil layer having plasticity index, I_p less than 15, even if FC is larger than 35%.
- Soil layer having a mean particle size (D_{50}) less than 10mm and a particle size at 10% pass on the grading curve (D_{10}) is less than 1mm.

When liquefaction is estimated in layers surrounding the foundation, lateral resistance shall be reduced in the foundation design.

(8) Vehicular Collision force

Abutments and piers located within a distance 9m to the edge of roadway or within a distance of 15m to the centerline of a railway track, shall be designed for an equivalent static force of ***1800 kN***. Which is assumed to act in any direction in a horizontal plane, at a distance of ***1.2m*** above ground.

(9) Ship Collision force

1) Design Vessel: 1000DWT
2) Design Collision Velocity: 3.5 m/s
3) Ship Collision Force on Pier: $PS= 1.2*10^5 V(DWT)^{0.5}$=13,100kN

(10) Settlement load

Settlement load caused by non-uniform settlement of piers and abutments for continuous spans, settlement value is assumed ***20mm*** for each piers or abutments. Some combinations of non-uniform settlement shall be considered for the unfavorable case.

(11) Creep and Shrinkage

The effects of creep and shrinkage are based on CEB-FIB model code according to the article 5.4.2.3 in 22TCN272-05, depending on humidity, construction schedule, material characteristic and structural dimensions. Average annual humidity for the CEB-FIB model code is ***80%.***

8.1.4 Load Modifier Factors and Load Combinations

(1) Load modifier factors

Each component and connection shall satisfy equation as below for each limit state, unless otherwise specified. For service and extreme event limit states, resistance factors shall be taken as 1.0, except for bolts, and for concrete columns in Seismic Zones 3 and 4. All limit states shall be considered of equal importance.

$$\sum \eta i.\ \gamma i.\ Qi \quad \leq \quad \phi.\ Rn = Rr$$

Where:

ηi:load modifier factors

γi:load factors

ϕ:resistance factors

Qi: forces effect

Rn: nominal resistance

Rr: factored resistance

For loads for which a maximum value of γi is appropriate:$\eta i = \eta_D \times \eta_R \times \eta_I \geq 0.95$

For loads for which a minimum value of γi is appropriate:$\eta i = 1/(\eta_D \times \eta_R \times \eta_I) \leq 1.0$

For this bridge:

Table 8.1.4-1 Load Modifier Factors

Factors	Sign	Limit states	Value
Factor relating to ductility	η_D	Strength	1.0
		Other	1.0
Factor relating to redundancy	η_R	Strength	1.0
		Other	1.0
Factor relating to operational importance	η_I	Strength	1.0
		Other	1.0

Source: Study Team

(2) Load combinations

Design load combinations:

Table 8.1.4-2 Load Combinations

Load Combinations / Limit states	Comb. code	DC	DW	LL IM CE BR PL LS EL	WS	WL	TU CR SH	TG	SE	EQ	CV
Service - I	Comb1	1.00	1.00	1.00	0.30	1.00	1.00	0.50	1.00	-	-
Service - III	Comb2	1.00	1.00	0.80	0.30	1.00	1.00	0.50	1.00	-	-
Strength - I	Comb3	1.25	1.50	1.75	-	-	0.50	-	1.00	-	-
Strength - II	Comb4	1.25	1.50	1.35	-	-	0.50	-	1.00	-	-
Strength - III	Comb5	1.25	1.50	-	1.40	-	0.50	-	1.00	-	-
Strength - IV	Comb6	1.50	1.50	-	-	-	0.50	-	-	-	-
Strength - V	Comb7	1.25	1.50	1.35	0.40	1.00	0.50	-	1.00	-	-
ExtremeT - I	Comb8	1.25	1.50	0.50	-	-	-	-	-	1.00	-
ExtremeT - II	Comb9	1.25	1.50	0.50	-	-	-	-	-	-	1.00

Where:

DC	Dead load	TG	Gradient temperature
DW	Pavement dead load	TU	Uniform temperature
BR	Braking force	WA	Water pressure
IM	Impact load	WL	Wind load on vehicular
LL	Live load	WS	Wind load on structure
PL	Pedestrian load	EQ	Earth quake
SE	Settlement	CT	Vehicular collision force
CR	Creep	EL	Accumulated locked-in force
SH	Shrinkage		

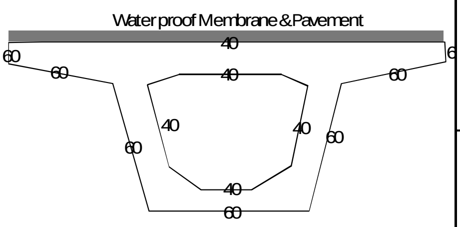
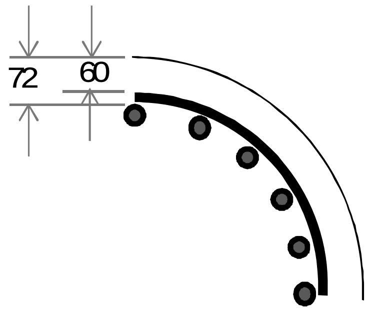
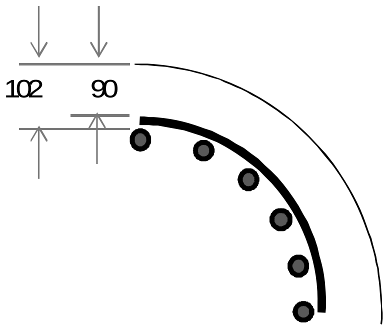
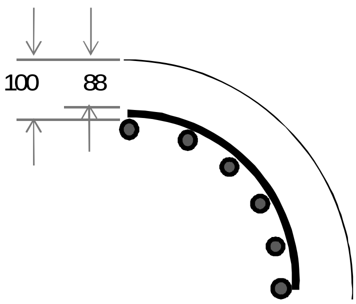
Source: 22TCN272-05, Study Team

Other loads, such as: horizontal earth pressure (EH), earth surcharge load (ES), vertical pressure from dead load of earth fill (EV) are used for substructure design, load factors of these conform to bridge design specification *22TCN 272-05*.

8.1.5 Concrete Cover

Basic concept for protection of reinforcing steels and PC tendons is made following Vietnamese standards (Article 5.12.3 of 22 TCN-272-05 and Table1 in Article 4 of TCXDVN327:2004), and concrete covers are recommended as follows:

Table 8.1.5-1 Minimum Concrete Cover

	Schematic	22TCN-272-05	TCXDVN327:2004	Adopt
PC Box Girder		Outside (Coastal) 75mm * 0.8 = 60mm (W/C < 0.4)	Outside (Above water level, sea water) 40mm+additional 20mm for longer life = 60mm (f'c=50MPa*)	Outside 60mm (f'c=50MPa*)
		Inside 40mm * 0.8=32mm (W/C<0.4)	N/A	Inside 40mm
Pier (above Highest High Water Level)		Main Bar (Coastal) 75mm (0.4<W/C<0.5)	Main Bar Cover for Stirrup + 12mm =60mm+12mm=72mm	Main Bar 72mm (f'c=40MPa*)
		Stirrup 75mm-12mm= 63mm	Stirrups (Above water level, sea water) 60mm (f'c = 30MPa + additional 10MPa for longer life =40MPa*)	Stirrup 60mm (f'c=40MPa*)
Pier (Under Highest High Water Level)		Main Bar (Direct Exposure to Salt Water) 100mm (0.4<W/C<0.5)	Main Bar Cover for Stirrup + 12mm =90 + 12 = 102mm	Main Bar 102mm (f'c=50MPa*)
		Stirrup 10mm - 12mm = 88mm	Stirrups(Variable Water Level) 70mm + additional 20mm for longer life=90mm (f'c=40MPa+additional 10MPa for longer life=50MPa*)	Stirrup 90mm (f'c=50MPa*)
Foundation		Main Bar (Direct Exposure to Salt Water) 100mm (0.4<W/C<0.5)	Main Bar Cover for Stirrup + 12mm =70 + 12 = 82mm	Main Bar 100mm
		Stirrup 10mm-12mm= 88mm	Stirrups (Submerged) 50mm+additional 20mm for longer life=70mm (f'c=30MPa+additional 10MPa for longer life=40MPa*)	Stirrup 88mm

*f'c is based on cubic samples.

Source: Study Team

8.1.6 Site Condition

8.1.6.1 Soil Condition

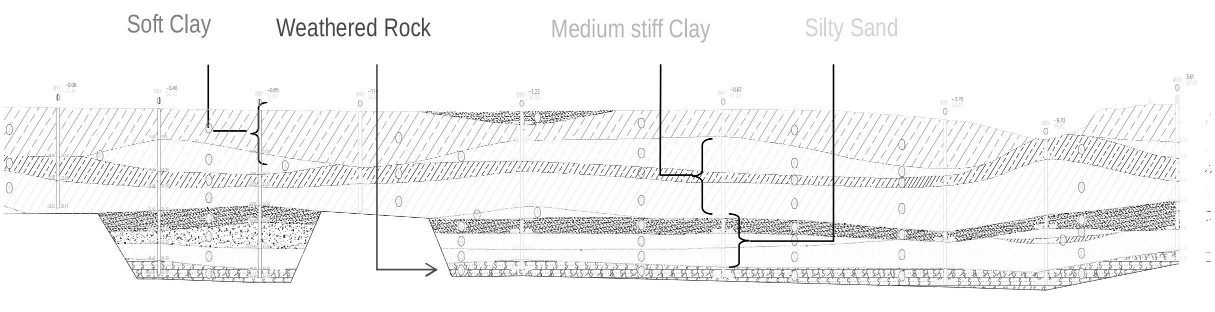
The ground conditions by D/D study are shown in below illustrations and following tables. The weathered rock layer that can be regarded as the bearing layer is distributed E.L.-40.0m to E.L.-50.0m depth, and has a thick surface layer predominant with clay on top. Specialty, very sensitive clay is thickly deposited from ground surface to GL-15m, of which N-value is 0 to 2, will be affected by consolidation occurs.

Table 8.1.6-1 Design data for Approach Bridge at Boring NoBB7

Layer Name	Soil Type	Average N-value	Wet Densit above the water level γ (kN/m3)	Wet Densit under the water level γ (kN/m3)	Shear Strength C (kN/m2)	Internal Friction φ (°)	Horizontal spring constant αEo (kN/m2)(3)
2	Sand	3	19.0	10.0	-	21.0	8400
3	Clay	2	17.0	8.0	15.0	-	5600
4	Clay	6	18.0	9.0	25.0	-	16800
5	Sand	11	20.0	11.0	-	25.0	30800
6	Clay	12	20.0	11.0	60.0	-	33600
L 6-1	Sand	26	20.0	11.0	-	30.0	72800
7A	Clay	4	18.0	9.0	25.0	-	11200
7B	Clay	7	18.0	9.0	30.0	-	19600
8	Clay	15	19.0	10.0	60.0	-	42000
9	Clay	7	18.0	9.0	30.0	-	19600
L 10A-1	Clay	10	19.0	10.0	60.0 (1)	-	28000
L 10A-3	Clay	6	19.0	10.0	36.0 (1)	-	16800
10A	Sand	21	20.0	11.0	-	35.0	58800
10B	Sand	38	21.0	12.0	-	40.0	106400
11	Clay	18	20.0	11.0	120.0	-	50400
12A	Clay	50	21.0	12.0	600.0 (2)	-	140000
12B	Clay	50	26.0	17.0	600.0 (2)	-	140000

※Note
(1) According to Terzaghi and Peck(qu=12.5N->Cu=6N)
(2) Adopt the minimum value among 12A and 12B from Rock Test(Unconfined Strength)
Cu=qu/2
(3)According to JSHB-2002
2800×(AverageN-value)

Source: Study Team



Source: Study Team

Figure 8.1.6-1 Soil Profile

8.1.6.2 Structure Height and Seawater Depth

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

Table 8.1.6-2 Pier Height and Seawater Depth

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

*pier Height : Column + Pile Cap Height

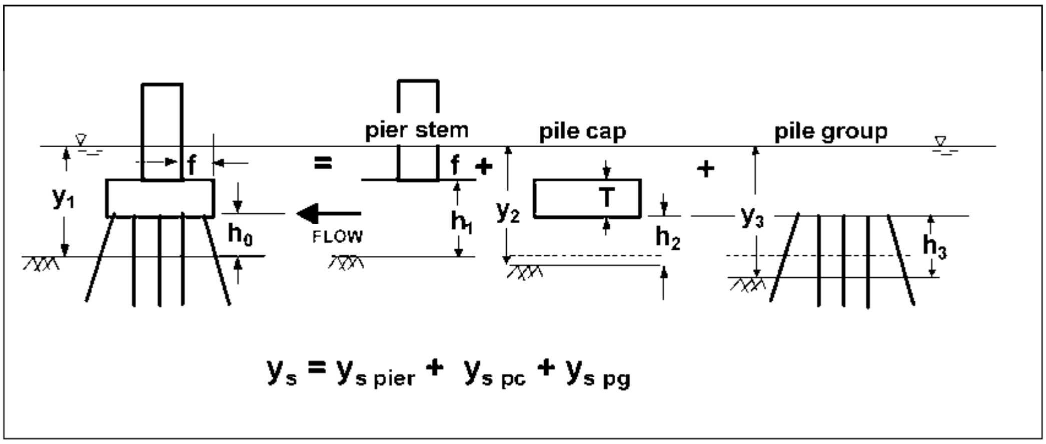
Source: Study Team

8.1.6.3 Design Scour Depth

1) Superposition of Scour Components Method of Analysis

The local scour depth is considered by Superposition of Scour Components Method of Analysis of "Hydraulic Engineering Circular No.18" (FHWA NHl01-001, May 2001), and the contraction scour is according for F/S study.

The components of a complex pier are illustrated in Figure 8.1.5-2. This is followed by a definition of the variables. Note that the pile cap can be above the water surface, at the water surface, in the water or on the bed. The location of the pile cap may result from design or from long-term degradation and/or contraction scour. The pile group, as illustrated, is in uniform (lined up) rows and columns. This may not always be the case. The support for the bridge in many flow fields and designs may require a more complex arrangement of the pile group. In more complex pile group arrangements, the methods of analysis given in this manual may give smaller or larger scour depths.



Source: Chapter 6, FHWA NHl01-001

Figure 8.1.6-2 Definition sketch for scour components for a complex pier

The variables illustrated in Figure 8.1.5-2 and others used in computations are as follows:

f = Distance between front edge of pile cap or footing and pier, m
h_o = Height of the pile cap above bed at beginning of computation, m
h_1 = ho + T = height of the pier stem above the bed before scour, m
h_2 = ho + ys $_{pier}$/2 = height of pile cap after pier stem scour component has been computed, m
h_3 = ho + ys $_{pier}$/2 + ys $_{pc}$/2 = height of pile group after the pier stem and pile cap scour components have been computed, m
S = Spacing between columns of piles, pile center to pile center, m
T = Thickness of pile cap or footing, m
y_1 = Approach flow depth at the beginning of computations, m
y_2 = y_1 + ys $_{pier}$/2 = adjusted flow depth for pile cap computations m
y_3 = y_1 + ys $_{pier}$/2 + ys $_{pc}$/2 = adjusted flow depth for pile group computations, m
V_1 = Approach velocity used at the beginning of computations, m/sec
V_2 = V1(y_1/y_2) =adjusted velocity for pile cap computations, m/sec
V_3 = V1(y_1/y_3) =adjusted velocity for pile group computations, m/sec

Total scour from superposition of components is given by:

$$ys = ys_{pier} + ys_{pc} + ys_{pg}$$

where:

ys = Total scour depth, m

ys_{pier} = Scour component for the pier stem in the flow, m

ys_{pc} = Scour component for the pier cap or footing in the flow, m

ys_{pg} = Scour component for the piles exposed to the flow, m

2) For Pier scour

The need to compute the pier stem scour depth component occurs when the pier cap or the footing is in the flow and the pier stem is subjected to sufficient flow depth and velocity as to cause scour. The first computation is the scour estimate, $y_{s\,pier}$, for a full depth pier that has the width and length of the pier stem using the basic pier equation.

$$\frac{y_{spier}}{y_1} = k_{hpier}\left[2.0K_1K_2K_3K_4\left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{gy_1}}\right)^{0.43}\right]$$

Where:

$y_{s\,pier}$ = Scour component for the pier stem in the flow, m

y_1 = Flow depth directly upstream of the pier, m

$K_{h\,pier}$ = Coefficient to account for the height of the pier stem above the bed and the shielding effect by the pile cap overhang distance "f" in front of the pier stem (from Figure 8.1.5-2)

K_1 = Correction factor for pier nose shape from Figure 8.1.5-3 and Table 8.1.5-3 (6.1)

K_2 = Correction factor for angle of attack of flow from Table 8.1.5-3(6.2)

K_3 = Correction factor for bed condition from Table 8.1.5-3(6.3)

K_4 = Correction factor for armoring by bed material size (D50<2mm or D95< 20mm, then K4=1)

a = Pier width, m

V_1 = Mean velocity of flow directly upstream of the pier, m/s

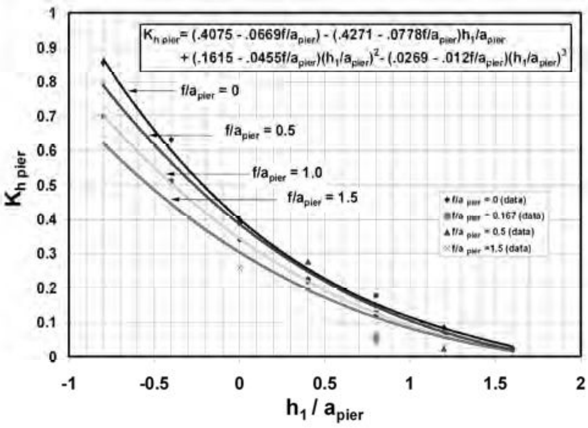
g = Acceleration of gravity (9.81 m/s^2)

Table 8.1.6-3 Correction factor for pier scour

Table 6.1. Correction Factor, K_1, for Pier Nose Shape.	
Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

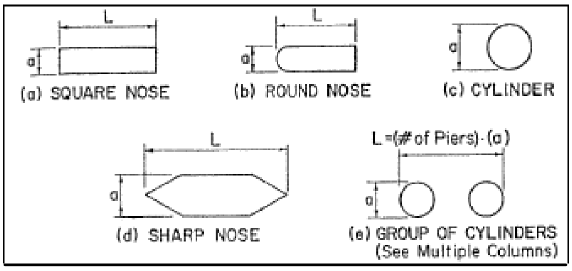
Table 6.2. Correction Factor, K_2, for Angle of Attack, θ, of the Flow.			
Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier, m			

Table 6.3. Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition.		
Bed Condition	Dune Height m	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	3> H ≥ 0.6	1.1
Medium Dunes	9> H ≥ 3	1.2 to 1.1
Large Dunes	H ≥ 9	1.3



Source: Chapter 6, FHWA NHl01-001

Figure 8.1.6-3 Suspended pier scour ratio.



Source: Chapter 6, FHWA NHl01-001

Figure 8.1.6-4 Common pier shapes.

3) For Pile cap scour

As described below, there are two cases to consider in estimating the scour caused by the pile cap (or footing).

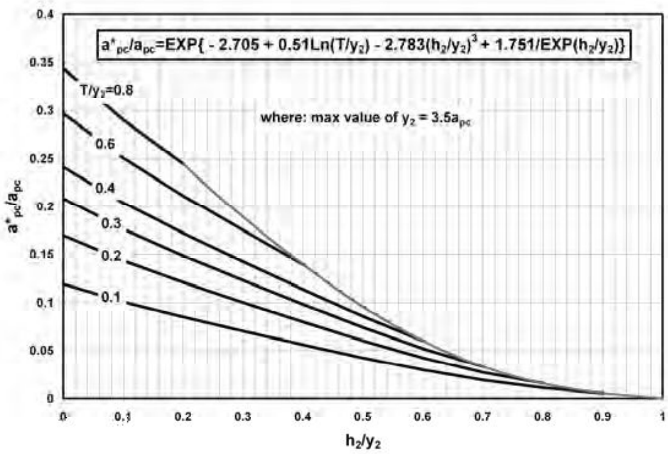
<u>Case 1: The bottom of the pile cap is above the bed</u> and in the flow either by design or after the bed has been lowered by scour caused by the pier stem component. The strategy is to reduce the pile cap width, a_{pc}, to an equivalent full depth solid pier width, a_{*pc}, using Figure 8.3.5-5.

$$\frac{y_{spc}}{y_2} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a*_{pc}}{y_2}\right)^{0.65} \left(\frac{V_2}{\sqrt{g y_2}}\right)^{0.43}$$

T = Thickness of the pile cap exposed to the flow, m
$h_2 = h_o + y_{s\ pier}/2$, m
$y_2 = y1 + y_{s\ pier}/2$, = adjusted flow depth, m
$V_2 = V_1(y_1/y_2)$ = adjusted flow velocity, m/s

where:

h_o = Original height of the pile cap above the bed, m
y_1 = Original flow depth at the beginning of the computations before scour, m
$y_{s\ pier}$ = Pier stem scour depth component, m
V_1 = Original approach velocity at the beginning of the computations, m/s



Source: Chapter 6, FHWA NHl 01-001

Figure 8.1.6-5 Pile cap equivalent width

Case 2: The bottom of the pile cap or footing is on or below the bed. The strategy is to treat the pile cap or exposed footing like a short pier in a shallow stream of depth equal to the height to the top of the footing above bed. The portion of the flow that goes over the top of the pile cap or footing is ignored.

$$\frac{y_{spc}}{y_f} = 2.0 K_1 K_2 K_3 K_4 \left(\frac{a_{pc}}{y_f}\right)^{0.65} \left(\frac{V_f}{\sqrt{g y_f}}\right)^{0.43}$$

4) For Pile Group scour

Research by Salim and Jones and by Smith has provided a basis for determining pile group scour depth by taking into consideration the spacing between piles, the number of pile rows and a height factor to account for the pile length exposed to the flow.

The effective width of an equivalent full depth pier is the product of the projected width of piles multiplied by a spacing factor and a number of aligned rows factor.

$a^*_{pg} = a_{proj} K_{sp} K_m$

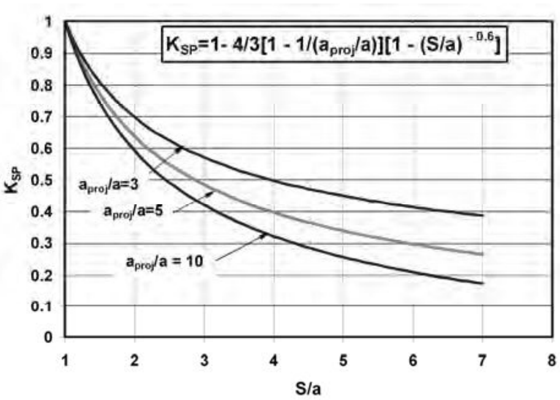
where:

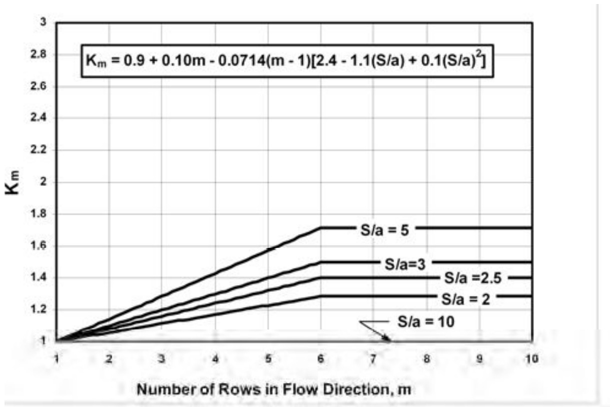
a_{proj} = Sum of non-overlapping projected widths of piles

K_{sp} = Coefficient for pile spacing (Figure 8.1.5-6)

K_m = Coefficient for number of aligned rows, m, (Figure 8.1.5-7)

K_m = 1.0 for skewed or staggered pile groups





Source: Chapter 6, FHWA NHl 01-001

Figure 8.1.6-6 Pile spacing factor

Figure 8.1.6-7 Adjustment factor for number of aligned rows

The scour equation for a pile group can then be written as follows:

$$\frac{y_{spg}}{y_3} = K_{hpg} \left[2.0 K_1 K_3 K_4 \left(\frac{a^*_{pg}}{y_f}\right)^{0.65} \left(\frac{V_3}{\sqrt{g y_3}}\right)^{0.43} \right]$$

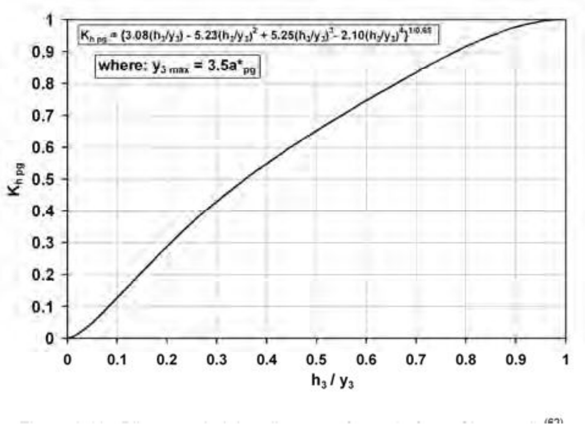
where:

$K_{h\,pg}$ = Pile group height factor given in Figure 8.1.6-9

$h_3 = h_0 + y_{s\,pier}/2 + y_{s\,pc}/2$

$y_3 = y_1 + y_{s\,pier}/2 + y_{s\,pc}/2$

$V_3 = V_1 (y_1/y_3)$



Source: Chapter 6, FHWA NHl 01-001

Figure 8.1.6-8 Pile group height adjustment factor

5) Design scour depth

The results on the design scour depth are shown in the following table. The depth of construction scour is assumed around 1.0m. Furthermore, the relationship between depth of pile cap and elevation of each scour are shown in following Figure.

However, due to the local scouring with design flow velocity ware not continued in the long-term, the seismic design was considered to 50% of Normal condition(as service and strength condition).

Table 8.1.6-4 Results of Design Scour Depth

Pier No	Ground Elevation (m)	Depth of Ground Elevation (m)	Depth of contraction scour (m)	Depth of local scour (m)	Total depth of scour (m)	Level after scour of Normal(m)	Level after scour of Seismic (m)	Foundation Type
P61~P75	-4.96	7.51	0.35	6.81	**7.15**	-12.11	-8.71	Pile Foundation
Pier 76	-4.39	6.94	0.18	6.24	**6.42**	-10.81	-7.69	SPSP
Pier 77	-6.12	8.67	0.73	7.46	**8.19**	-14.31	-10.58	
Pier 78	-8.25	10.80	1.57	11.20	**12.77**	-21.01	-15.41	
Pier 79	-8.98	11.53	1.90	6.32	**8.22**	-17.19	-14.03	Multi Column Pile
Pier 80	-8.58	11.13	1.72	6.06	**7.78**	-16.36	-13.33	
Pier 81	-7.25	9.80	1.16	5.66	**6.82**	-14.07	-11.24	
Pier 82	-5.32	7.87	0.46	6.75	**7.21**	-12.53	-9.16	Pile Foundation
Pier 83	-1.20	3.75	0.00	6.17	**6.17**	-7.37	-4.29	
Pier 84	0.14	2.42	0.00	5.89	**5.89**	-5.75	-2.81	
Pier 85	0.44	2.11	0.00	6.08	**6.08**	-5.65	-2.61	
Pier 86	0.71	1.84	0.00	5.80	**5.80**	-5.09	-2.19	
Pier 87	1.10	1.46	0.00	5.78	**5.78**	-4.69	-1.80	

Note) Design water EL = 2.72m(1% HHWL)

Design flow velocity = 1.45m/s (assumed at ebb and flow condition by F/S report)

Source: Study Team

8.1.7 Concept on Comparative Study for Structure Optimization

For the selection of the most appropriate alternative, the Consultant has prepared the evaluation criteria as shown in tables below, using score (point) ranking to evaluate priority of each category.

Table 8.1.7-1 Evaluation Criteria of Alternative Study

No.	Category	Evaluation Criteria	Maximum Score (Point)
1.	Economic Viability (50 points)	Construction Cost	40
2.		STEP Clearance	10
3.	Technical Viability (35 point)	Structure Stability (including earthquake Resistance)	10
4.		Construction Plan (Difficulty) and Construction Period	10
5.		Maintenance	15
6.	Other Viability (15 points)	Environmental Impact	5
7.		New Technology	5
8.		Aesthetics	5
Total Points			100

Source: Study Team

Table 8.1.7-2 Scoring System for Evaluation of Alternative Structure

Description		Structural Aspect and Stability	Construction Plan and Period	Maintenance	Construction Cost		STEP Clearance		Aesthetics	New Technology	Environmental Traffic Management
Grade	Rate	(10)	(10)	(15)	(40)	Ratio	(10)	Persent	(5)	(5)	(5)
Very Good	100%	10	10	15	40	(1.00)	10	Over 50%	5	5	5
Good	80%	8	8	12	32	(1.01～1.15)	8	(21～49%)	4	4	4
Fare	60%	6	6	9	24	(1.15～1.30)	6	(11～20%)	3	3	3
Bad	40%	4	4	6	16	(1.31～1.99)	4	(1～10%)	2	2	2
Very Bad	0%	0	0	0	0	Over 2.00	0	0%	0	0	0

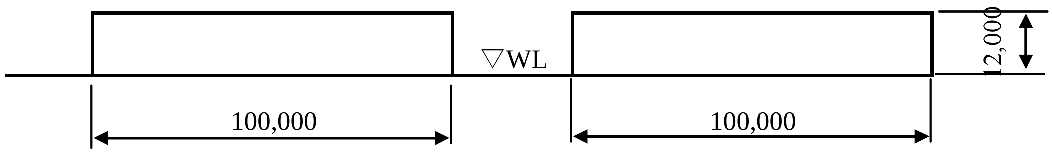
Source: Study Team

8.2 Span Length and Span Arrangement

8.2.1 Study on Span Length of Main Bridge

8.2.1.1 Navigation Clearance

The navigation channel for large vessels will be shifted to the northern side of deep sea port. The bridge shall have navigation for vessels of 1,000 DWT. The navigation clearance at Nam Trieu Channel was agreed with Bina Marine by the letter No. 192/TB-BGTVT dated 17 May 2009 as shown in the figure below.



Source: Study Team

Figure 8.2.1-1 Navigation Clearance

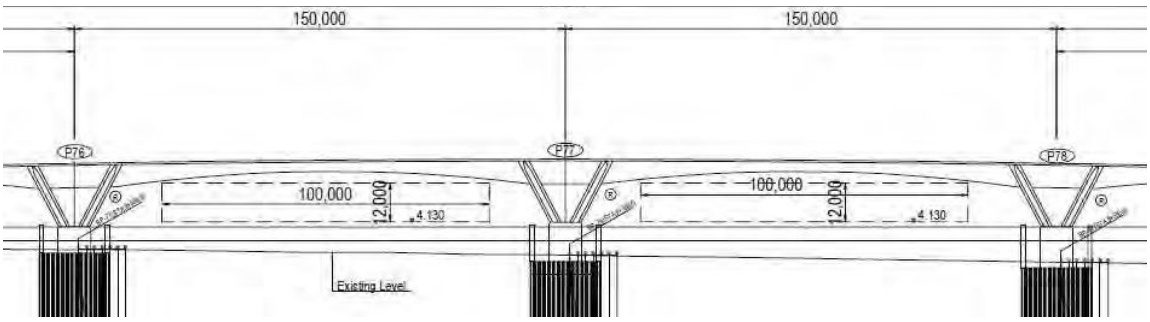
WL (Water Level) for navigation clearance is calculated as follows:

WL = 2.72m (DHWL) + 1.41m (Effect of wave) = 4.13m

DHWL: Design High Water Level = 2.72m (High tide water level at 1% probability)

8.2.1.2 Span Length of Main Bridge

Two main spans are subjected to clearance of navigation channel and construction limit. The proposed main bridge (Prestressed Concrete Box Girder with V-shaped Pier) is required 150.0m for main span length in consideration with the navigation clearance at the level of WL (4.13m) as shown in the figure below. During construction of the main bridge, construction limits for temporary facilities and anchorage of barges or tug boat are considered to determine the main span length. The span length of 150.0m determined from the navigation clearance is enough space for temporary construction facilities such as stagings and anchorage of barge.



Source: Study Team

Figure 8.2.1-2 Span Length determined from Navigation Clearance

Two main spans are secure to be 150.0m each. Side span length of the main bridge are economically determined by ratio of main span length (L) which is 0.60～0.65*L. Side span on both sides are also secured to be 95.0m which is 0.633*L. The end of side span is constructed on the falsework supported steel staging in the sea.Study on Span Length of Approach Bridge

8.2.1.3 General

(1) Study Objections

In order to realize appropriate spanning plan to adjust bridge length in accordance with the site conditions such as intersections with cross roads and railways, comparison study for the appropriate span of the approach bridge is performed.

(2) Conditions of Study

In this study, the erection method of Span-By-Span (SBS) with Precast Girder Segments is assumed, as recommended by SAPROF Study. The comparison study between SBS and Movable Scaffolding System (MSS) will be presented later.

(3) Scope of Study

The study was implemented by checking the following items.

1) Optimization of Span Length

In SAPROF Study, the span length with n@ (60m + 5m) with double wall piers is recommended and approved by MOT. In MOD between MOT and JICA on 18th of June, 2010, single wall pier for the approach bridge is mentioned in the page 5 of Annex I.

In this study, the following four alternatives are studied.

(i) Alternative-1: 60m span with double wall piers (10@ (5m+ 60m), SAPROF)
(ii) Alternative-2: 65m span with single wall piers (10@65m)
(iii) Alternative-3: 60m span with single wall piers (10@60m)
(iv) Alternative-4: 50m span with single wall piers (10@50m)

2) Items to be studied

Items to be studied are quantities of girders, piers and foundations, and approximate construction cost.

8.2.1.4 Contents of Study

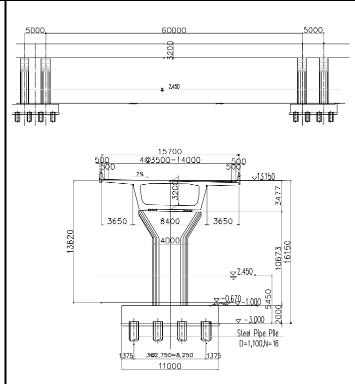
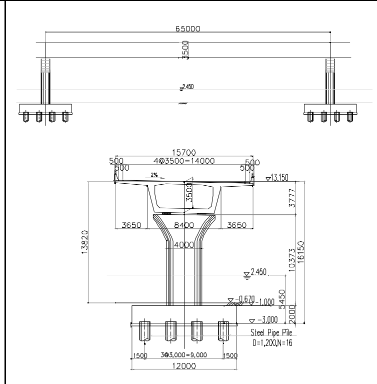
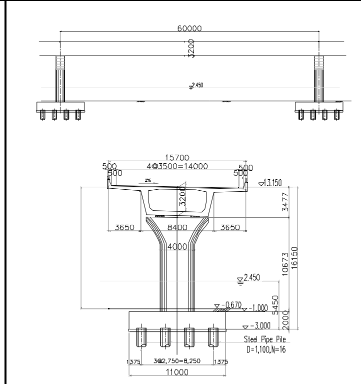
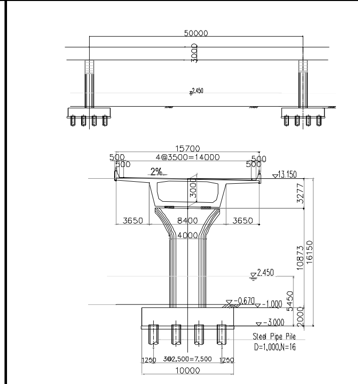
(1) Comparison Study

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

(2) Study Result

As shown in the following pages, Alternative-3:60m span with single wall pier is recommended from comprehensive view points, economic, technical and other viabilities.

Table 8.2.2-1 Comparison Table on Span Length and Shape of Piers of Approach Bride

Evaluation Items	Max. Point	Alternative-1 Span Length: 60.0m+5.0m with Double Wall Piers		Alternative-2 Span Length: 65.0m with Single Wall Piers		Alternative-3 Span Length: 60.0m with Single Wall Piers		Alternative-4 Span Length: 50.0m with Single Wall Piers	
Schematics		 SAPROF Study							
Structural Aspect and Stability	10	Structurally Stable Large erection girders are needed Concrete volume is large Large number of bearings (6 pieces/100m) Large number of piers	8	Structurally Stable Election girder is the largest among alternatives Concrete volume is the largest Smal. number of bearings (3 pieces/100m) Number of piers is the smallest	8	Stracturally Stable Large election girders are needed Relatively small concrete volume Small number of bearings (3.3 pieces/100m) Small number of piers	8	Stracturally Stable Erection girder is the smallest Concrete volume is the smallest Number of bearings is 4 per 100m Number of piers is the largest	8
Construction Cost	40	Superstructure (per 100m) 27,751,306,472 VND Substructure (per 100m) 24,938,167,752 VND Total 52,689,474,224 VND Ratio 1.054	32	Superstructure (per 100m) 26,180,045,368 VND Substructure (per 100m) 24,746,335,692 VND Total 50,926,381,060 VND Ratio 1.019	32	Superstructure (per 100m) 25,515,586,192 VND Substructure (per 100m) 24,473,686,000 VND Total 49,989,272,192 VND Ratio 1.000	40	Superstructure (per 100m) 23,847,705,665 VND Substructure (per 100m) 26,613,553,200 VND Total 50,461,258,865 VND Ratio 1.009	32
Construction Plan and Period	10	Workability is inferior due to large pier head segments	6	Workability is inferior due to heaby precast segments	6	Workability is superior with single wall piers Precast segments are not as heavy as Alt.-2	8	Workability is superior with single wall piers and light weight segments Construction Peirod: 1.5 month / 100m	10
Maintenance	15	Inferior in Maintenance due to large number of bearings to be replaced (6 pieces /100m)	6	Supeiror in Maintenance with small number of bearings (3 pieces/100m)	10	Superior in maintentance with small number of bearings (3.3 pieces /100m)	10	Number of bearing is 4 per 100m	8
STEP Clearance	10	53% (Preliminary Estimate) Large election girder make a contribution	10	54%(Preliminary Estimate) Large election girder make a contribution	10	53%(preliminary Estimate) Large election girder make a contribution	10	52% (preliminary Estimate) Large number of steel piles make a contribution	10
Aesthetics	5	Slender appearance Girder/Span ratio is less than 1/18 Knotty impression with double piers	3	Slender appearnace with long span Girder/Span ratio is less than 1/18	4	Slender outfit with long span Girder/Span ratio is less than 1/18	4	Ordinary outfit Girder / Span ratio is more than 1/17	3
New Technology	5	Span-by-span (SBS) Method is new technology in Vietnam	5	Span-by-span (SBS) Method is new technology in Vietnam One of the wold longest span by SBS Method	5	Span-by-span (SBS) Method is new technology in Vietnam	5	Span-by-span (SBS) Method is new technology in Vietnam	5
Traffic Management/ Environmental Aspect	5	Large number of piers (3 piers / 100m) limits traffic management under the bridges.	3	Superior in traffic and environment with small number of piers (1.5 piers / 100m)	4	Superior in traffic and environment with small number of piers (1.7 piers / 100m)	4	Inferior in traffic managemet and environmental aspect with small number of piers (2 piers / 100m)	3
Evaluation	100	Construction cost is highest Double wall pier results in large number of bearings and large pier head segments Not Recommended	73	Construction Cost is a little higher than Alt.-3 Large erection girders are needed Longest span results in smallest number of piers and bearings Less Recommended	79	Minimum Construction Cost with efficient workability. Relatively long span results in small number of piers and bearings Most Recommended	89	Construction Cost is a little higher than Alt.3 Superior in Workability Relatively short span results in large number of piers and bearings Less Recommended	79

Source : Study Team

8.2.2 Study on Number on Continuous Spans of Approach Bridge

8.2.2.1 General

(1) Study Objections

In order to determine appropriate number of continuous girder spans to adjust span arrangement in accordance with the site conditions such as intersections with cross roads and railways, comparison study for the appropriate span of the approach bridge is performed.

(2) Conditions of Study

In this study, the erection method of Span-By-Span (SBS) with Precast Girder Segments is assumed, as recommended by SAPROF Study. The comparison study between SBS and Movable Scaffolding System (MSS) is presented in Section 8.4. 1.

(3) Scope of Study

The study was implemented by checking the following items.

1) Optimization of Number of Continuous Spans of Approach Bridge

In SAPROF Study, the number of continuous spans of Approach Bridge is not specified but it seems that 8 or 9 continuous spans were assumed . In this study, the following four alternatives are studied.

(i) Alternative-1: 4-Span Continuous Girder (4@60.0m = 240.0m)
(ii) Alternative-2: 5-Span Continuous Girder (5@60.0m = 300.0m)
(iii) Alternative-3: 6-Span Continuous Girder (6@60.0m = 360.0m)
(iv) Alternative-4: 7-Span Continuous Girder (7@60.0m = 420.0m)

(4) Items to be studied

Items to be studied are quantities of bearings and expansion joints, and approximate construction cost.

8.2.2.2 Contents of Study

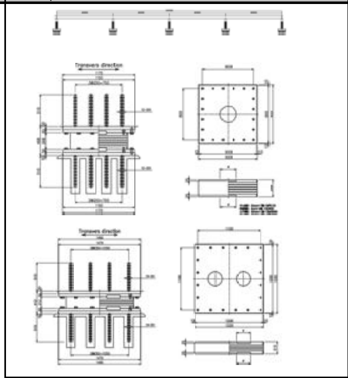
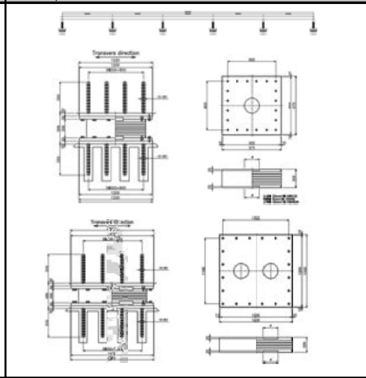
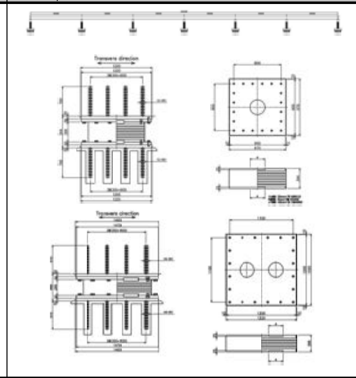
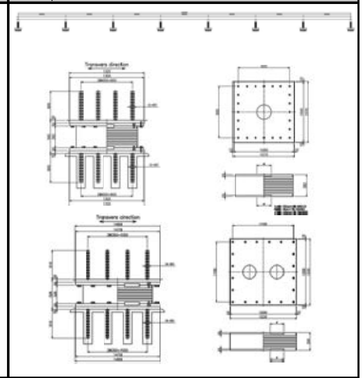
(1) Comparison Study

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

(2) Study Result

As shown in the table on the following page, Alternatives 1 and 2: 4 or 5 continuous spans are recommended from comprehensive view points, economic, technical and other viabilities.

Table 8.2.3-1 Comparison on Number of Continuous Spans of Approach Bridge

Evaluation Items	Max. Point	Alternative-1 4-Span Continous Girder (4@60.0m = 240.0m)		Alternative-2 5-Span Continuous Girder (5@60.0m = 300.0m)		Alternative-3 6-Span Continous Girder (6@60.0m = 360.0m)		Alternative-4 7 Span Continuous Girder (7@60.0nm=420.0m)	
Schematics		 End Support : 900 x 900 x 32 x 5Layers N=4 Middle Support : 1200 x 1200 x 32 x 3 Layers N=6		 End Support : 950 x 950 x 32 x 6Layers N=4 Middle Support : 1200 x 1200 x 32 x 4 Layers N=8		 End Support : 950 x 950 x 32 x 7Layers N=4 Middle Support : 1200 x 1200 x 32 x 5 Layers N=10		 End Support : 1050 x 1050 x 32 x 8Layers N=4 Middle Support : 1200 x 1200 x 32 x 6Layers N=12	
Structural Aspect and Stability	10	- Least number of continuous spans - Inferior in seismic stability because of small number of statically indeterminate degrees - Superior in adopting to site constraines such as location of intersections - Dimensions of bearings are the smallest	8	- 2nd least number of continuous spans - Seismically stable because of enough number of statically indeterminate degrees - Superior in adopting to site constraines such as location of intersections - Dimensions of bearings are the second smallest	10	- 2nd most number of continuous spans - Seismically stable because of enough number of statically indeterminate degrees - Inferiour in adopting to site constraines such as location of intersections - Dimensions of bearings are the second largest	8	- Most number of continuous spans - Seismically stable because of enough number of statically indeterminate degrees - Inferior in adopting to site constraines such as location of intersections - Dimensions of bearings are the largest	8
Construction Cost	40	Bearing Shoe (per 100m) 5,884,400,000 VND Expansion Joint (per 100m) 1,065,333,333 VND Total 6,949,733,333 VND Ratio 1.00	40	Bearing Shoe (per 100m) 6,336,853,333 VND Expansion Joint (per 100m) 852,266,667 VND Total 7,189,120,000 VND Ratio 1.03	32	Bearing Shoe (per 100m) 6,657,288,889 VND Expansion Joint (per 100m) 710,222,222 VND Total 7,367,511,111 VND Ratio 1.06	32	Bearing Shoe (per 100m) 7,367,809,524 VND Expansion Joint (per 100m) 608,761,905 VND Total 7,976,571,429 VND Ratio 1.15	24
Construction Plan and Period	10	Workability is inferior due to large number of bearings and joints	6	Workability is superior because of smaller number of bearings and joints	8	Workability is superior because of small number of bearings and joints	8	Workability is superior because of sumall number of bearings and joints	10
Maintenance	15	Inferior in maintenance due to large number of bearings and joints to be replaced (Bearings: 4.2 pieces /100m, Joints: 0.42 locations/100m)	9	Superior in maintenance due to smaller number of bearings and joints to be replaced (Bearings: 4 pieces /100m, Joints: 0.3 locations/100m)	12	Superior in maintenance because of smaller number of bearings and joints to be replaced (Bearings: 3.9 pieces /100m, Joints: 0.28 locations/100m)	12	Superior in maintenance because of smaller number of bearings and joints to be replaced (Bearings: 4.2 pieces /100m, Joints: 0.24 locations/100m)	12
STEP Clearance	10	53% (Preliminary Estimate) No signuficant differences among the alternatives	10	54%(Preliminary Estimate) No signuficant differences among the alternatives	10	55%(preliminary Estimate) No signuficant differences among the alternatives	10	55% (preliminary Estimate) No signuficant differences among the alternatives	10
Aesthetics	5	No signuficant differences among the alternatives	5	No signuficant differences among the alternatives	5	No signuficant differences among the alternatives	5	No signuficant differences among the alternatives	5
New Technology	5	Rubber bearing for seismic force sharing is new technology in Vietnam	5	Rubber bearing for seismic force sharing is new technology in Vietnam	5	Rubber bearing for seismic force sharing is new technology in Vietnam	5	Rubber bearing for seismic force sharing is new technology in Vietnam	5
Traffic Management/ Environmental Aspect	5	No signuficant differences among the alternatives	4	No signuficant differences among the alternatives	4	No signuficant differences among the alternatives	4	No signuficant differences among the alternatives	4
Evaluation	100	Construction cost is low and superior in adopting site constraines Recommended	87	Construction cost is low and superior in adopting site constraines Recommended	86	Construction cost is high and inferior in adopting the site constrains Less Recommended	84	Construction cost is high and inferior in adopting the site constrains Less Recommended	78

Source : Study Team

8.2.3 Study on Span Length of Flyover Bridge

8.2.3.1 General

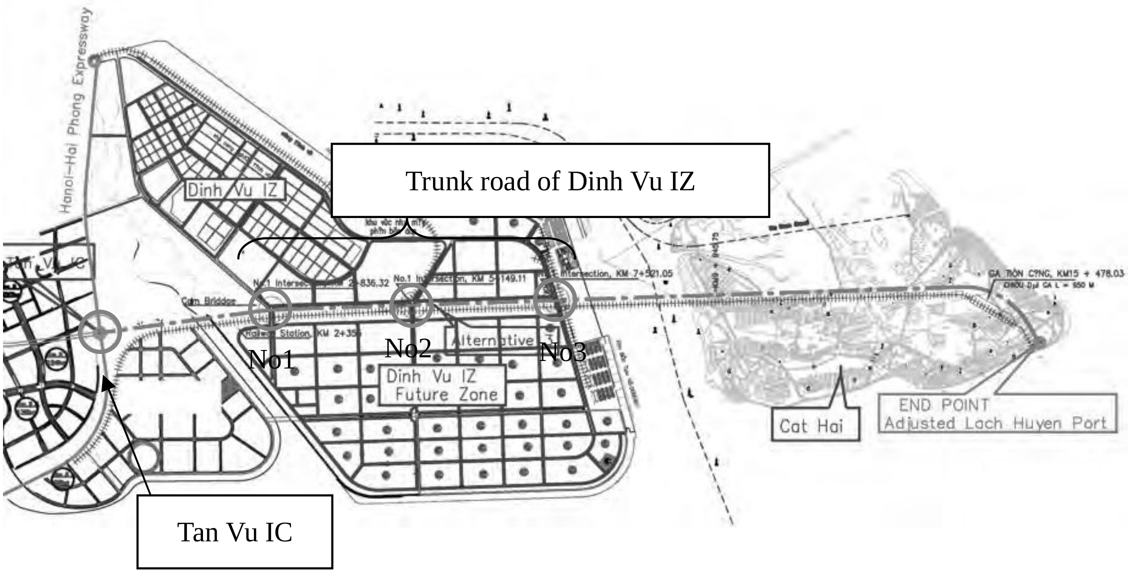
(1) Study Objections

This study is prepared to optimize the type and span length of Flyover Bridge at the intersections in Nam Dinh Vu Industrial Zone.

(2) Conditions of Study

1) Location of Intersections

In this study, the location of the cross roads and railways in Nam Dinh Vu erection is based on the General Plan Adjustment of Haiphong City in 2025 and Orientation to 2050.



Source: Study Team

Figure 8.2.3-1 Location of Interchange and Intersections

a) No.2 Intersection

No.2 Intersection is located at Km 5+149.11, where Dinh Vu Ring Road will pass under the highway. The railway will pass under the highway at Km 5+084.11.

b) No.3 Intersection

No.3 Intersection is located at Km 7+542.05, where Dinh Vu Ring Road will pass under the highway. The railway will pass under the highway at Km 7+642.43.

Located at Km 2+836.32, No.1 Intersection will be in the road section.

2) Planning Elevation of Nam Dinh Vu Industrial Zone

Based on the letter No.297/CV-QH dated 18th July 2008 of Hai Phong Construction Service of Hai Phong Planning Institute, the planning elevation of Nam Dinh Vu Industrial Zone is as follows,

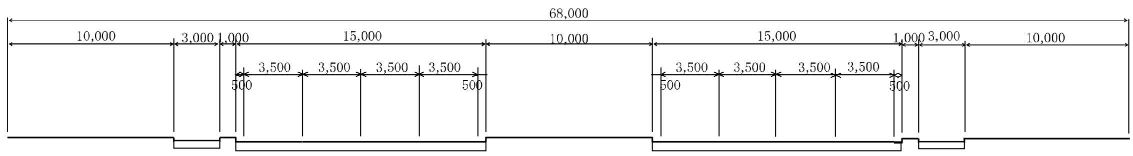
Planning Elevation: Construction foundation of +5.0m (Sea chart elevation)

For road design elevation = +5.0m – 1.90m = 3.10m

3) Clearance of Cross Road

Based on Article 4.10.2 of TCVN4054-2005, the vertical clearance of 4.75m over driven lanes and shoulders is applied.

The typical cross section of Dinh Vu Ring Road is as shown in the figure below.



Source: Study Team

Figure 8.2.3-2 Typical Cross Section of Dinh Vu Ring Road

4) Clearance of Railway

Based on Article 26 of Railway National Law, the clearance of railway is as follows,

- Vertical : 6.55m
- Horizontal : perpendicular 7.00m from outer edges of outer rails.

➢ RAILWAY NATIONAL LAW (RNL)

Article 26 -

PROTECTION LIMIT OF RAILWAY CORRIDOR

Protection Limit of Railway Corridor (PLRC) consists of a special room limit above railway, 2 land surface band limits at both sides of railway center line and an under ground limit on which the railway line stretches out.

1- Special room above railway line: is defined as a vertical 6.55 m height above top of rail on 1435 railway line

The interval between railway and electrical transmission line is defined by National Electricity Law

2- Two Land surface band limits at both sides of railway line center is defined as below:

+ perpendicular 7.00 m length from outer edges of outer rails towards both sides of railway line (in the case that the railway is on flat ground surface on which the cross section of railway line is not a filling or a cutting - one)..

+ perpendicular 5.00 m length from the bottom of a filling embankment towards both sides of railway line (in the case that the railway cross section is a filling embankment)

+ perpendicular 5.00 m length from the top of cutting embankment or from the outer edges of longitudinal ditches towards both sides of railway line (in the case that railway cross section is a cutting embankment)

3- Under ground limit on which the railway stretches out: as defined in Article 32 of RNL

(3) Scope of Study

The study was implemented by checking the following items.

1) Optimization of Span Length

In SAPROF Study, the span length with 71.25m + 83.5m + 71.25m with double wall pier is recommended and approved by MOT. In this study, the following three alternatives are studied.

a) Alternative-1: Span arrangement of 71.25m+83.5m+71.25m with double wall piers

Cast-in-place PC Box Girder with Cantilever Method is assumed as recommended in SAPROF Study.

Side span of 71.25m is long for the central span of 83.5m. Also, the clearance to the railway at the Intersection No.3 is not enough. Therefore, Alternative-2 with shorter side span is introduced. For continuity with Approach Bridges, the type of piers is assumed as single wall which is recommended for Approach Bridge in the previous section.

b) Alternative-2: Span arrangement of 64m+84m+64m with single wall piers

Cast-in-place PC Box Girder with Cantilever Method is assumed same as Alternative-1.

The span length of 60m is recommended for the approach bridge as mentioned in the previous section. The Flyover Bridge with same span length is introduced as follows.

c) Alternative-3: General span length of 60m with single wall piers

2) Items to be studied

Items to be studied are made on economic, technical and other viabilities.

8.2.3.2 Contents of Study

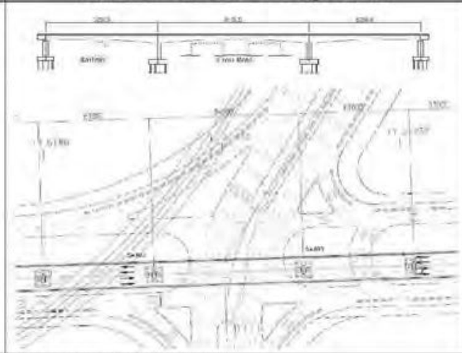
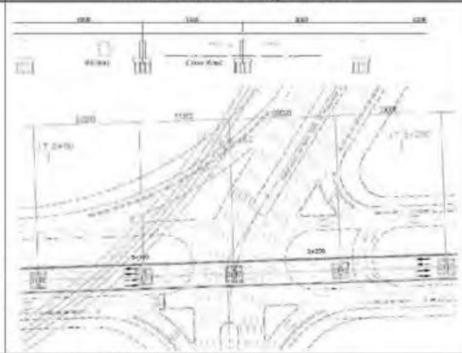
(1) Comparison Study

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

(2) Study Result

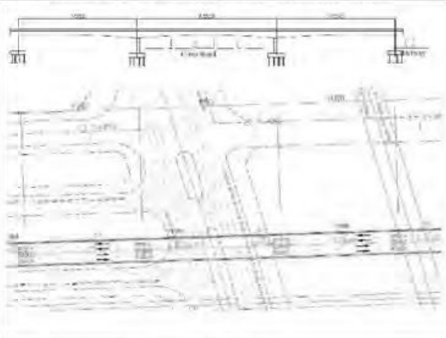
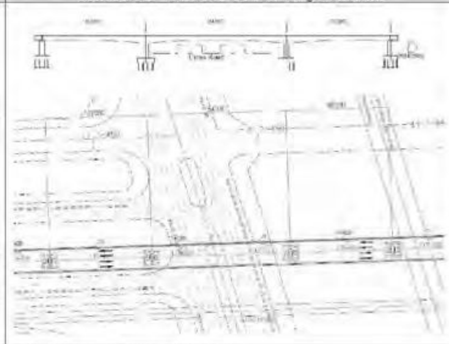
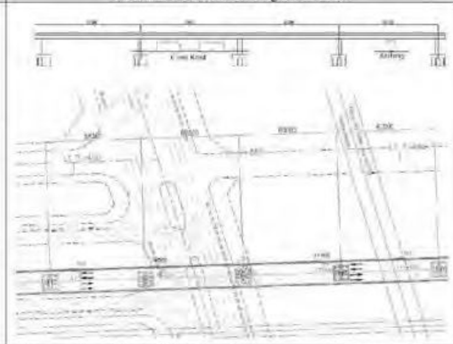
As shown in the following pages, Alternative-3:60m span with single wall pier is recommended from comprehensive view points. The one of the significant advantages of Alternative-3 is the fact that the bridge type and the erection method are the same as the adjacent approach bridges, so that the erection girder can be commonly utilized and the construction can be continuously arranged, which results in the lowest construction cost and the shortest construction period among the alternatives.

Table 8.2.4-1 Comparison in Flyover Bridge for Intersection No.2

Alternatives			Alternative-1: 71.25m+83.5m+71.25m with Double Wall Piers		Alternative-2: 64m+84m+64m with Single Wall Piers		Alternative-3: 60m with Single Wall Piers	
Features								
Features			- Span arrangement: 71.25m + 83.5m + 71.25m - Cast-in-place PC Box Girder with Cantilever Method. - Proposed in SAPROF and approved by MOT. - Double wall piers.		- Span arrangement: 64 m + 84m + 64m - Cast-in-place PC Box Girder with Cantilever Method. - Single wall piers		- The central span length of 60m is applied. - Side Span of 51.3m is applied for adjusting the distance between the railway and the intersection. - SBS Method with Precast PC Box Girders is assumed to be applied as the erection method, same as adjacent approach bridges.	
Category	Criteria	Score	Evaluation	Score	Evaluation	Score	Evaluation	Score
Economic Viability 50 Points	Construction Cost	40	1.18 (207,000 JPY/m^2, Based on Unit Price in SAPROF Study) Traveler forms for Cantilever Method are needed. Large volume of scaffoldings is needed for long side span. Concrete volume of the girder is largest (12.5m^3/m).	24	1.15 (202,000 JPY/m^2, Based on Unit Price in SAPROF Study) Traveler forms for Cantilever Method are needed. Volume of scaffoldings are smaller than Alternative-1. Concrete Volume of the girder is larger than Alternative-3 (12m^3/m)	24	1.0 (176,000 JPY/m^2, Based on Unit Price in SAPROF Study) Erection Girder for SBS is commonly used as a part of approach bridges. Traveler forms and Scaffoldings are not needed. Concrete volume of the girder is smallest (9m^3/m).	40
	STEP Clearance	10	42%(Based on Unit Price in SAPROF Study) Amount procured from Japan: 85,100 JPY/m^2 (Cement:2,800, Rebar:8,500, PC Strand:6,700, Steel Pile: 36,900, sheet pile: 15,900, Overhead: 14,300)	8	42%(Based on Unit Price in SAPROF Study) Amount procured from Japan: 84,800 JPY/m^2 (Cement:2,800, Rebar:8,500, PC Strand:6,600, Steel Pile: 36,900, Sheet Pile: 15,900, Overhead: 14,100)	8	52%((Based on Unit Price in SAPROF Study) Erection girder for SBS method is procured from Japan. Amount procured from Japan: 92,200 JPY/m^2 (Erection Girder: 7,900, Cement:2,600, Rebar:8,500, PC Strand:6,800, Steel Pile: 38,000, Sheet Pile:17,000, Overhead: 11,400)	10
Technical Viability 35 Points	Structure Stability	10	Side-span-ratio is 0.85, which is relatively high.	6	Side-span-ratio is optimum (0.76).	8	The span of 60m is relatively long as girders by SBS Method but applicable.	8
	Construction Plan and Construction Period	10	Period: 13 months (Foundation, Piers and Superstructure) Traveler forms for Cantilever Method are needed. Large volume of scaffoldings is needed for long side span. Girder erection of adjacent approach bridges is separated.	6	Period: 13 months (Foundation, Piers and Superstructure) Traveler forms for Cantilever Method are needed. Volume of scaffoldings are smaller than Alternative-1	6	Period: 6 months (Foundation, Piers and Superstructure) Erection Girder for SBS is commonly used as a part of approach bridges. Traveler forms and Scaffoldings are not needed.	10
	Maintenance	15	The number of bearings is 8. The number of joints is 4.	8	The number of bearings is 4. The number of joints is 2.	10	The number of bearings is 12. The number of joints is 1.	6
Other Viability 15 Points	Traffic Management/ROW/Environmental Impact	5	No problems in keeping clearance	5	No problems in keeping clearance	5	In order to keep the clearance, a pier is located at the center median of the projected road in Nam Dinh Vu Industrial Zone.	4
	New Technology	5	Cantilever Method is commonly practiced in Vietnam.	3	Cantilever Method is commonly practiced in Vietnam.	3	SBS is a new technology in Vietnam.	5
	Aesthetics	5	Discontinuous at the intersection with Flyover bridge. There are a couple of irregular spans for adjustment	4	Discontinuous at the intersection with Flyover bridge. There are a couple of irregular spans adjacent to the Flyover Bridge	4	Continuous with adjacent approach bridges. There is an irregular span for adjustment.	4
Total Evaluation		100	**Not recommended** Construction cost is approximately 18% more than Alternative-3. Construction period is estimated double comparing to Alternative-3.	64	**Not recommended** Construction cost is approximately 18% more than Alternative-3. Construction period is estimated double comparing to Alternative-3.	68	**Recommended** The bridge type and the erection method are the same as the adjacent approach bridges, so that the erection girder can be commonly utilized and the construction can be continuously arranged.	87

Source : Study Team

Table 8.2.4-2 Comparison in Flyover Bridge for Intersection No.3

Alternatives			Alternative-1: 71.25m+83.5m+71.25m with Double Wall Piers		Alternative-2: 64m+84m+64m with Single Wall Piers		Alternative-3: 60m with Single Wall Piers	
Features								
Features			- Span Arrangement: 71.25m + 83.5m + 71.25m - Cast-in-place PC Box Girder with Cantilever Method is assumed. - Proposed in SAPROF and approved by MOT. - Double wall piers.		- Span Arrangement: 64 m + 84m + 64m - Cast-in-place PC Box Girder with Cantilever Method is assumed. - Single wall piers.		- The central span length of 60m is applied. - Side Span of 58.38m is applied for keeping clearance at the intersection. - SBS Method with Precast PC Box Girders is assumed to be applied as the erection method, same as adjacent approach bridges.	
Category	Criteria	Score	Evaluation	Score	Evaluation	Score	Evaluation	Score
Economic Viability 50 Points	Construction Cost	40	1.18(Preliminary Estimate) (= 2,524USD/m^2) Traveler forms for Cantilever Method are needed. Large volume of scaffoldings is needed for long side span. Concrete volume of the girder is largest (12.5m^3/m).	24	1.15(Preliminary Estimate) (=2,465USD/m^2) Traveler forms for Cantilever Method are needed. Volume of scaffoldings are smaller than Alternative-1. Concrete Volume of the girder is larger than Alternative-3 (12m^3/m)	24	1.0 (Preliminary Estimate) (=2,146USD/m^2) Erection Girder for SBS is commonly used as a part of approach bridges. Traveler forms and Scaffoldings are not needed. Concrete volume of the girder is smallest (9m^3/m).	40
	STEP Clearance	10	46%(Preliminary Estimate) Amount procured from Japan: 1,154USD/m^2 (Cement:34, Rebar:115, PC Strand:90, Steel Pile: 513, sheet pile: 230, Overhead: 181)	8	47%(Preliminary Estimate) Amount procured from Japan: 1,151 USD/m^2 (Cement:33, Rebar:112, PC Strand:85, Steel Pile: 513, Sheet Pile: 230, Overhead: 178)	8	58%(Preliminary Estimate) Erection girder for SBS method is procured from Japan. Amount procured from Japan: 1,246 USD/m^2 (Erection Girder: 107, Cement:35, Rebar:115, PC Strand:91, Steel Pile: 512, Sheet Pile:231, Overhead: 155)	10
Technical Viability 35 Points	Structure Stability	10	Side-span-ratio is 0.85, which is relatively high.	6	Side-span-ratio is optimum (0.76).	8	The span of 60m is relatively long as girders by SBS Method but applicable.	8
	Construction Plan and Construction Period	10	Period: 13 months (Foundation, Piers and Superstructure) Traveler forms for Cantilever Method are needed. Large volume of scaffoldings is needed for long side span. Girder erection of adjacent approach bridges is separated.	6	Period: 13 months (Foundation, Piers and Superstructure) Traveler forms for Cantilever Method are needed. Volume of scaffoldings are smaller than Alternative-1.	6	Period: 6 months (Foundation, Piers and Superstructure) Erection Girder for SBS is commonly used as a part of approach bridges. Traveler forms and Scaffoldings are not needed.	10
	Maintenance	15	The number of bearings is 8. The number of joints is 4.	8	The number of bearings is 4. The number of joints is 2.	10	The number of bearings is 12. The number of joints is 1.	6
Other Viability 15 Points	Traffic Management/ROW/Environmental Impact	5	Clearance to the railway is not enough.	2	No problems in keeping clearance.	5	In order to keep the clearance, a couple of piers are located just outside of the sidewalks of the projected road in Nam Dinh Vu Industrial Zone.	4
	New Technology	5	Cantilever Method is commonly practiced in Vietnam.	3	Cantilever Method is commonly practiced in Vietnam.	3	SBS is a new technology in Vietnam.	5
	Aesthetics	5	Discontinuous at the intersection with Flyover bridge. There are a couple of irregular spans for adjustment	4	Discontinuous at the intersection with Flyover bridge. There are a couple of irregular spans adjacent to the Flyover Bridge	4	Continuous with adjacent approach bridges. There is an irregular span for adjustment.	4
Total Evaluation		100	**Not recommended** Construction cost is approximately 18% more than Alternative-3. Construction period is estimated double comparing to Alternative-3.	61	**Not recommended** Construction cost is approximately 15% more than Alternative-3. Construction period is estimated double comparing to Alternative-3.	68	**Recommended** The bridge type and the erection method are the same as the adjacent approach bridges, so that the erection girder can be commonly utilized and the construction can be continuously arranged.	87

Source : Study Team

8.2.4 Recommended Span Arrangement in Bridge Section

Recommendations in the studies on span length of Approach Bridge and Flyover Bridge is as follows,

- Approach Bridge: 60m span length with single wall piers
- Flyover Bridge: 60m span length with single wall piers
- Number of Continuous Spans: 4 or 5 spans

In consequence, optimum bridge spans are arranged for the entire bridge section as shown in the table below. Several adjusting spans less than 60m are introduced around the intersections and Main Bridge.

Table 8.2.4-1 Recommended Span Arrangement in comparison with SAPROF Study

	Recommended by JICA Study Team	SAPROF Study
	Span Arrangement	Span Arrangement
Approach Bridge(1)	2@(5@60m) +(51.5m+4@60m) +6@(5@60m) +(4@60m+58.36m) +(5@60m) +4@(52.98m+3@60m+52.98m)	46.6m+7@65m+46.6m
Flyover Bridge(1)		71.25m+83.5m+71.25m
Approach Bridge (2)		53.5m+32@65m
Flyover Bridge(2)		71.25m+83.5m+71.25m
Approach Bridge (3)		20@65m
Main Bridge	95m+150m+150m+95m	95m+150m+150m+95m
Approach Bridge (4)	(54.8m+3@60m+54.8m) +(54.8m+2@60m+54.8m)	7@65m+64.2m
Total Length	5,442.9m	5,442.9m

Source: Study Team