5.12.2 Structural Analysis

1) Type W1-1 Wall

1. CALCULATION OF STEM

· THICKNESS ; T = 0.50 (M)

H (M)	M (T·M)	S (T)	ASR (CM2)
1.50	1.24	2.07	2.56
2.00	1.54	2.57	3.18
2.50	1.85	3.08	3.81
3.00	2.15	3.58	4.43
3.50	2.45	4.08	5.05
4.00	2.75	4.58	5.67
4.50	3.05	5.09	6.30
5.00	3.35	5.59	6.92
5.50	3.66	6.09	7.54
6.00	3.96	6.60	8.17
6.50	4.26	7.10	8.79
7.00	4.56	7.60	9.41
7.50	4.86	8.11	10.03
8.00	5.17	8.61	10.66
10.50	6.68	11.13	13.77

2. CALCULATION OF COUNTERFORT

· THICKNESS OF FLANGE ; B = 0.50 (M)
· THICKNESS OF WEB ; T = 1.00 (M)
· CALCULATION SPAN ; L = 2.75 (M)

L (M)	FL (M)	M (T·M)	S (T)	ASR (CM2)
1.50	0.20	2.39	3.94	6.13
2.00	0.40	4.92	6.26	7.57
2.50	0.60	8.74	9.09	9.60
3.00	0.80	14.09	12.41	12.04
3.50	1.00	21.23	16.24	14.85
4.00	1.20	30.42	20.58	18.00
4.50	1.40	41.89	25.41	21.49
5.00	1.60	55.91	30.75	25.30
5.50	1.80	72.73	36.59	29.45
6.00	2.00	92.59	42.94	33.92
6.50	2.20	115.75	49.79	38.72
7.00	2.40	142.46	57.14	43.84
7.50	2.60	172.98	65.00	49.29
8.00	2.80	207.54	73.36	55.06
10.50	3.80	449.99	122.70	88.76

3. CALCULATION OF TOE

- THICKNESS ; T = 1.50 (M)

L (M)	T (M)	M (T·M)	S (T)	ASR (CM2)
0.00	1.50	-80.99	-77.45	51.41
0.50	1.50	-46.55	-60.08	29.55
1.00	1.50	-21.13	-41.38	13.41
1.50	1.50	-5.39	-21.35	3.42
2.00	1.50	0.00	0.00	0.00

4. CALCULATION OF HEEL

- THICKNESS ; T = 1.00 (M)

L (M)	M (T·M)	S (T)	ASR (CM2)
0.00	-6.76	-11.27	6.56
0.50	-4.37	-7.29	4.25
1.00	-1.99	-3.31	1.93
1.50	0.40	0.67	0.37
2.00	2.79	4.65	2.56
2.50	5.18	8.63	4.75
3.00	7.57	12.61	6.94
3.50	9.95	16.59	9.13

5. CALCULATION OF HORIZONTAL TIES

H (M)	S (M)	ASR (CM2)
1.50	2.07	1.48
2.00	2.57	1.84
2.50	3.08	2.20
3.00	3.58	2.56
3.50	4.08	2.92
4.00	4.58	3.27
4.50	5.09	3.63
5.00	5.59	3.99
5.50	6.09	4.35
6.00	6.60	4.71
6.50	7.10	5.07
7.00	7.60	5.43
7.50	8.11	5.79
8.00	8.61	6.15
8.50	9.11	6.51
9.00	9.62	6.87
9.50	10.12	7.23
10.00	10.62	7.59
10.50	11.13	7.95

2) Type W2-1 Wall

1. CALCULATION OF STEM

· THICKNESS ; T = 0.50 (M)

H (M)	M (T·M)	S (T)	ASR (CM2)
1.50	2.13	2.71	4.39
2.00	2.65	3.37	5.46
2.50	3.17	4.03	6.53
3.00	3.68	4.69	7.60
3.50	4.20	5.35	8.67
4.00	4.72	6.01	9.74
4.50	5.24	6.67	10.81
5.00	5.76	7.32	11.88
5.50	6.28	7.98	12.94
6.00	6.79	8.64	14.01
6.50	7.31	9.30	15.08
7.00	7.83	9.96	16.15
7.50	8.35	10.62	17.22
8.00	8.87	11.28	18.29
11.00	11.97	15.23	24.70

2. CALCULATION OF COUNTERFORT

· THICKNESS OF FLANGE ; B = 0.50 (M)
· THICKNESS OF WEB ; T = 1.00 (M)
· CALCULATION SPAN ; L = 2.98 (M)

L (M)	FL (M)	M (T·M)	S (T)	ASR (CM2)
1.50	0.32	3.13	5.16	6.28
2.00	0.65	6.45	8.20	7.32
2.50	0.97	11.44	11.90	9.07
3.00	1.30	18.46	16.26	11.23
3.50	1.62	27.81	21.28	13.74
4.00	1.95	39.84	26.95	16.56
4.50	2.27	54.88	33.29	19.68
5.00	2.60	73.24	40.28	23.11
5.50	2.92	95.27	47.94	26.83
6.00	3.25	121.29	56.25	30.85
6.50	3.57	151.63	65.22	35.15
7.00	3.90	186.63	74.86	39.75
7.50	4.22	226.60	85.15	44.63
8.00	4.55	271.88	96.10	49.81
11.00	6.50	673.55	175.64	86.94

3. CALCULATION OF TOE

· THICKNESS ; T = 1.00 (M)

L (M)	T (M)	M (T·M)	S (T)	ASR (CM2)
0.00	1.00	0.00	-0.01	0.00
0.00	1.00	0.00	0.00	0.00

4. CALCULATION OF HEEL

· THICKNESS ; T = 1.00 (M)

L (M)	M (T·M)	S (T)	ASR (CM2)
0.00	-46.73	-59.45	45.36
0.50	-41.51	-52.81	40.29
1.00	-36.29	-46.17	35.23
1.50	-31.07	-39.53	30.16
2.00	-25.85	-32.89	25.10
2.50	-20.64	-26.25	20.03
3.00	-15.42	-19.62	14.97
3.50	-10.20	-12.98	9.90
4.00	-4.98	-6.34	4.84
4.50	0.24	0.30	0.22
5.00	5.45	6.94	5.00
5.50	10.67	13.58	9.78
6.00	15.89	20.21	14.57

5. CALCULATION OF HORIZONTAL TIES

H (M)	S (M)	ASR (CM2)
1.50	2.71	1.94
2.00	3.37	2.41
2.50	4.03	2.88
3.00	4.69	3.35
3.50	5.35	3.82
4.00	6.01	4.29
4.50	6.67	4.76
5.00	7.32	5.23
5.50	7.98	5.70
6.00	8.64	6.17
6.50	9.30	6.64
7.00	9.96	7.12
7.50	10.62	7.59
8.00	11.28	8.06
11.00	15.23	10.88

3) Type W3 Wall

1. CALCULATION OF STEM

H (M)	D (M)	M (T · M)	S (T)	ASR (CM^2)
0.00	0.600	19.71	8.69	31.90
0.50	0.577	15.60	7.44	26.30
1.00	0.553	12.11	6.29	21.30
1.50	0.530	9.18	5.23	16.89
2.00	0.506	6.76	4.27	13.05
2.50	0.483	4.81	3.41	9.77
3.00	0.459	3.28	2.64	7.20
3.50	0.436	2.11	1.97	4.78
4.00	0.412	1.25	1.40	3.02
4.50	0.389	0.67	0.92	1.72
5.00	0.366	0.30	0.54	0.83
5.50	0.342	0.10	0.26	0.29
6.00	0.319	0.01	0.08	0.05

2. CALCULATION OF HEEL AND TOE

ITEM			HEEL	TOE
	H	(M)	0.600	0.600
DIMENSION	D	(M)	0.500	0.450
	B	(M)	1.000	1.000
MOMENT	M	(T · M)	8.38	0.20
SHEARING FORCE	S	(T)	6.78	9.36
REQUIRED REINF.	ASR	(CM^2)	13.02	0.33

TABLE 5-3 RESULT OF STRUCTURAL ANALYSIS FOR RETAINING WALL (1/3)

Type	Item		Unit	Stem		Toe	Heel	Counterfort	
W1−1	Position			H = 0.0 m	H = 4.7 m	−	−	H = 0.0 m	H = 4.7 m
	Moment		(t·m)	6.68	3.84	80.99	9.95	449.99	84.65
	Shear Force		(t)	11.13	6.40	50.93	16.59	122.70	40.40
	Reg'd Reinf.		(cm^2)	13.77	7.92	51.41	9.13	88.76	32.13
	Reinf.		(cm^2)	D16@125=16.08	D16@250=8.04	D16,D25@125=55.36	D20@250=12.56	(D28+D25)×9=99.63	D25×9=44.19
	Stress	σc	(kg/cm^2)	37 < 94.5	28 < 94.5	41 < 94.5	18 < 94.5	17 < 94.5	11 < 94.5
		σs	(kg/cm^2)	1131 < 1400	1271 < 1400	1229 < 1400	984 < 1400	1209 < 1400	654 < 1400
		τ	(kg/cm^2)	3.2 < 4.2	1.8 < 4.2	4.7 < 8.4	2.3 < 4.2	4.0 < 4.2	2.7 < 4.2
W1−2	Position			H = 0.0 m	H = 4.7 m	−	−	H = 0.0 m	H = 4.7 m
	Moment		(t·m)	6.68	3.84	−	26.44	449.99	84.65
	Shear Force		(t)	11.13	6.40	−	44.06	122.70	40.40
	Reg'd Reinf.		(cm^2)	13.77	7.92	−	22.03	61.09	22.32
	Reinf.		(cm^2)	D16@125=16.08	D16@250=8.04	−	D25@125=39.28	(D20+D25)×9=72.45	D20×9=28.26
	Stress	σc	(kg/cm^2)	37 < 94.5	28 < 94.5	−	31 < 94.5	10 < 94.5	7 < 94.5
		σs	(kg/cm^2)	1131 < 1400	1271 < 1400	−	867 < 1400	1011 < 1400	659 < 1400
		τ	(kg/cm^2)	3.2 < 4.2	1.8 < 4.2	−	3.0 < 5.3	2.8 < 4.2	1.7 < 4.2
W1−3	Position			H = 0.0 m	H = 5.6 m	−	−	H = 0.0 m	H = 5.6 m
	Moment		(t·m)	7.22	3.84	−	27.24	596.67	84.65
	Shear Force		(t)	12.03	6.40	−	22.70	143.54	40.40
	Reg'd Reinf.		(cm^2)	14.89	7.92	−	26.44	71.34	22.50
	Reinf.		(cm^2)	D16@125=16.08	D16@250=8.04	−	D25@125=39.28	(D20+D25)×9=72.45	D20×9=28.26
	Stress	σc	(kg/cm^2)	40 < 94.5	28 < 94.5	−	32 < 94.5	11 < 94.5	7 < 94.5
		σs	(kg/cm^2)	1223 < 1400	1271 < 1400	−	894 < 1400	1184 < 1400	602 < 1400
		τ	(kg/cm^2)	3.5 < 4.2	1.8 < 4.2	−	3.1 < 5.3	3.0 < 4.2	1.8 < 4.2

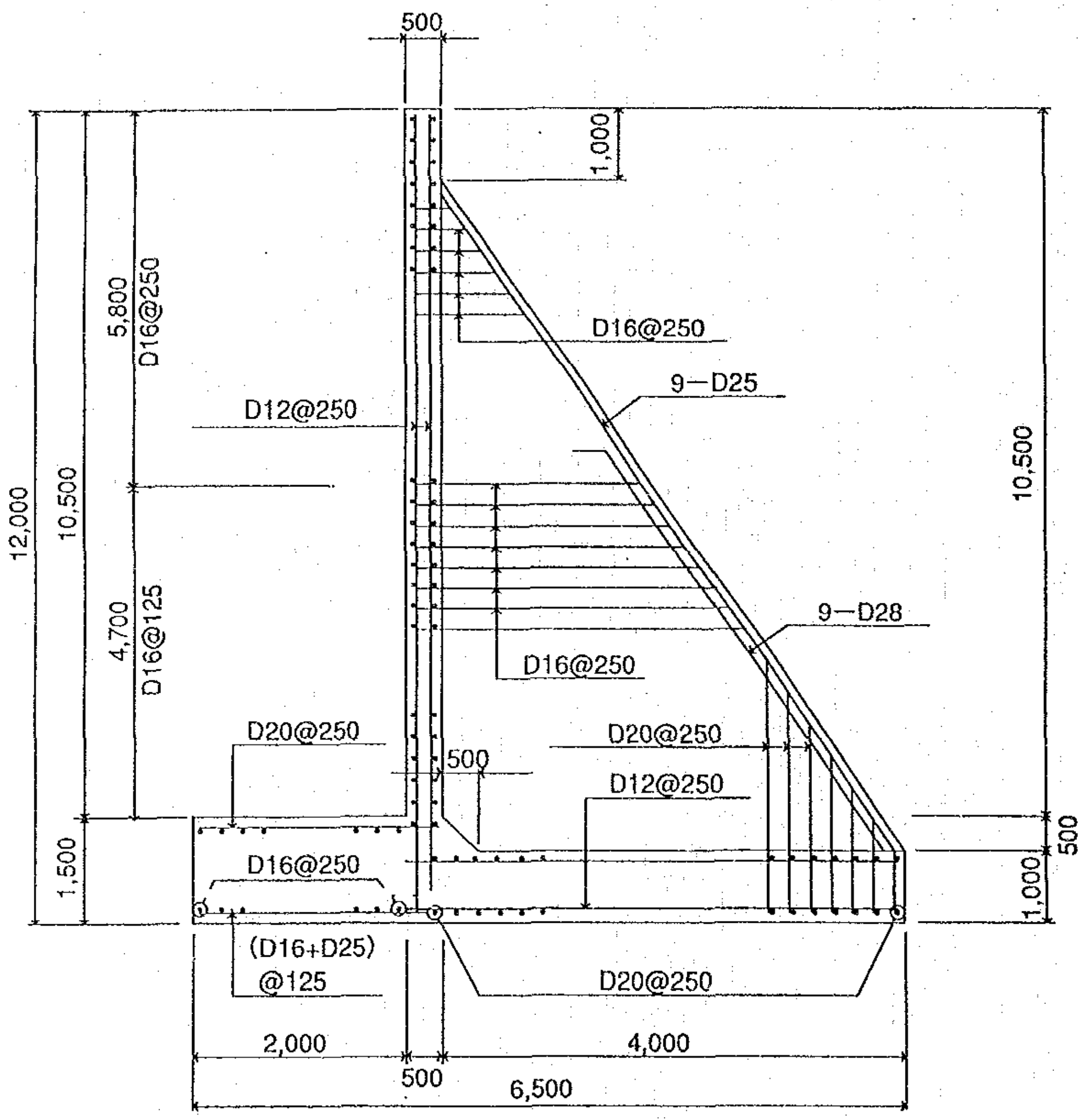
TABLE 5-4 RESULT OF STRUCTURAL ANALYSIS FOR RETAINING WALL (2/3)

Type	Item		Unit	Stem		Toe	Heel	Counterfort	
W1−4	Position			H = 0.0 m	H = 5.6 m	−	−	H = 0.0 m	H = 5.6 m
	Moment		(t·m)	7.22	3.84	84.76	8.80	569.67	84.65
	Shear Force		(t)	12.03	6.40	53.08	14.66	143.54	40.40
	Reg'd Reinf.		(cm^2)	14.89	7.92	53.80	8.06	100.20	31.32
	Reinf.		(cm^2)	D16@125=16.08	D16@250=8.04	D25,D16@125=55.36	D20@250=12.56	D28×18=110.88	D28×9=55.44
	Stress	σc	(kg/cm^2)	40 < 94.5	28 < 94.5	43 < 94.5	16 < 94.5	17 < 94.5	10 < 94.5
		σs	(kg/cm^2)	1223 < 1400	1271 < 1400	1286 < 1400	870 < 1400	1206 < 1400	574 < 1400
		τ	(kg/cm^2)	3.5 < 4.2	1.8 < 4.2	4.5 < 8.4	2.0 < 4.2	4.0 < 4.2	2.5 < 4.2
W2−1	Position			H = 0.0 m	H = 5.7 m	−	−	H = 0.0 m	H = 5.7 m
	Moment		(t·m)	11.97	6.07	−	46.73	673.55	86.46
	Shear Force		(t)	10.03	5.09	−	28.22	175.64	42.63
	Reg'd Reinf.		(cm^2)	24.70	12.52	−	45.36	86.94	23.66
	Reinf.		(cm^2)	D20@125=25.12	D20@250=12.56	−	D28@150=49.28	D25×18=88.38	D25×9=44.19
	Stress	σc	(kg/cm^2)	56 < 94.5	37 < 94.5	−	50 < 94.5	12 < 94.5	5 < 94.5
		σs	(kg/cm^2)	1322 < 1400	1304 < 1400	−	1234 < 1400	1190 < 1400	476 < 1400
		τ	(kg/cm^2)	2.9 < 4.2	1.5 < 4.2	−	3.8 < 4.2	3.2 < 4.2	1.5 < 4.2
W2−2	Position			H = 0.0 m	H = 3.3 m	−	−	H = 0.0 m	H = 3.3 m
	Moment		(t·m)	12.91	9.48	−	48.25	844.29	334.18
	Shear Force		(t)	10.82	7.95	−	45.89	204.13	110.16
	Reg'd Reinf.		(cm^2)	26.62	19.57	−	46.84	100.95	46.90
	Reinf.		(cm^2)	D25@125=39.28	D25@250=19.64	−	D28@125=49.28	D28×18=110.88	D28×9=55.44
	Stress	σc	(kg/cm^2)	51 < 94.5	49 < 94.5	−	51 < 94.5	11 < 94.5	8 < 94.5
		σs	(kg/cm^2)	932 < 1400	1325 < 1400	−	1274 < 1400	1032 < 1400	1014 < 1400
		τ	(kg/cm^2)	3.1 < 4.2	2.3 < 4.2	−	4.1 < 4.2	3.3 < 4.2	2.2 < 4.2

TABLE 5-5 RESULT OF STRUCTURAL ANALYSIS FOR RETAINING WALL (3/3)

Type	Item		Stem		Toe	Heel	Counterfort	
W3	Position		H = 0.0 m	H = 1.2 m	—	—	—	—
	Moment	(t·m)	19.71	10.94	0.20	8.38	—	—
	Shear Force	(t)	8.69	5.87	5.86	6.25	—	—
	Req'd Reinf.	(cm^2)	31.90	19.54	0.33	13.02	—	—
	Reinf.	(cm^2)	D25@125=39.28	D25@250=19.64	D16@250=8.04	D16@250=16.08	—	—
	Stress σc	(kg/cm^2)	54 < 94.5	47 < 94.5	1 < 94.5	38 < 94.5	—	—
	Stress σs	(kg/cm^2)	1126 < 1400	1372 < 1400	59 < 1400	1256 < 1400	—	—
	Stress τ	(kg/cm^2)	2.0 < 4.2	1.5 < 4.2	1.5 < 4.2	1.6 < 4.2	—	—

FIGURE 5-5 ARRANGEMENT OF REINF. FOR RETAINING WALL



5. 13 Analysis of Foundation for Retaining Wall

Analysis of Pile Foundation for Retaining Wall

** Type W1-1 **

Pile Groupe		1	2	3
n (Number of Piles)	(pcs.)	5	4	5
D (Diameter)	(mm)	800.00	800.00	800.00
t (Thickness)	(mm)	12.00	12.00	12.00
A (Cross Section)	(m^2)	0.02469	0.02469	0.02469
I (Moment of Inertia of Pile Cross Section)	(m^4)	0.191E-2	0.191E-2	0.191E-2
θ (Angle Between Pile and Vertical))	(deg)	0.00	0.00	0.00
Xi (Coordinate of Pile Head))	(m)	2.00	0.00	-2.00
L (Length of Piles)	(m)	10.00	10.00	10.00
Kh (Coefficient of Horizontal Subgrade Reaction)	(kg/cm^2)	6.85	6.85	6.85
β	(m^{-1})	0.43	0.43	0.43

Simultaneous Equation:

$$\begin{bmatrix} 178395. & 0. & -207408. \\ 0. & 693943. & 0. \\ -207408. & 0. & 2464972. \end{bmatrix} \begin{bmatrix} \delta x \\ \delta y \\ \alpha \end{bmatrix} = \begin{bmatrix} 705.00 \ (H) \\ 1780.00 \ (V) \\ 1210.00 \ (M) \end{bmatrix}$$

Displacement : $\delta x = 0.005013$ (m)

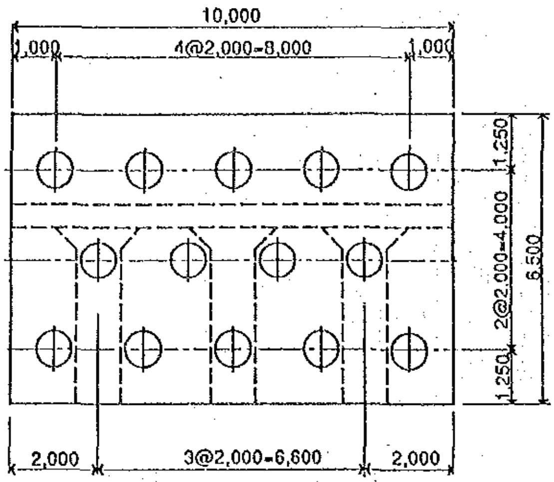
$\delta y = 0.002565$ (m)

$\alpha = 0.000913$ (rad)

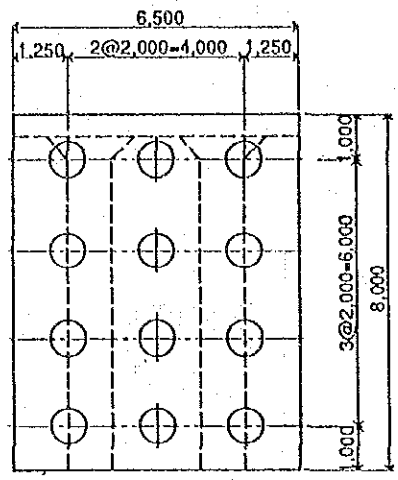
		1	2	3
Xi (Horizontal Displacement)	(m)	0.005	0.005	0.005
Yi (Vertical Displacement)	(m)	0.004	0.003	0.001
PVi (Vertical Load)	(t/pcs.)	217.62	127.14	36.66
PHi (Horizontal Load)	(t/pcs.)	50.36	50.36	50.36
Mi (Moment)	(t·m/pcs.)	-42.83	-42.83	-42.83
Σ Hi	(t)	704.999		
Vi	(t)	1779.999		
Mi	(t·m)	1209.999		

Check of Stress		Pile Groupe - 1	Pile Groupe - 3
Compressive Stress	(kg/cm^2)	1775	1042
Tensile Stress	(kg/cm^2)	0	-745
Allowable Stress	(kg/cm^2)	1900	1900

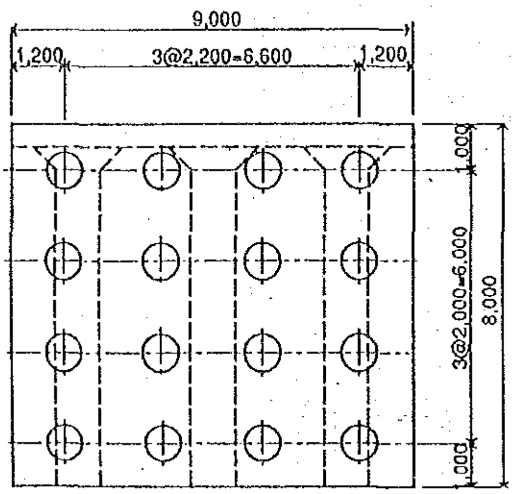
FIGURE 5-6 PILE ARRANGEMENT FOR TYPICAL RETAINING WALL



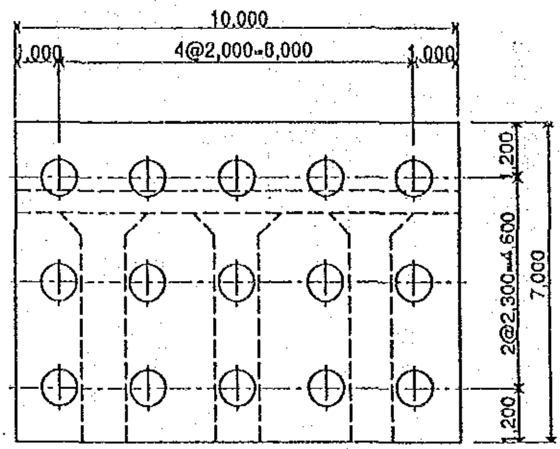
TYPE W1-1



TYPE W1-2



TYPE W1-3



TYPE W1-4

5. 14 Examination for Fish Way

A fish way is required when fishery is important for the river and certain normal discharge must be maintained. The type and scale of the fish way shall be planed by the design standards for headworks in land improvement projects, established by Agricultural Structure Improvement Bureau, Ministry of Agricultural, Forestry and Fisheries of Japan and the example of the Choa Phraya Dam in Thailand.

5. 14. 1 Conditions Suitable for Fish Way

1) The fish way shall be provided at both sides in case of overflow with full section.
2) The wider the fish way the better, however, it directly reflects the water volume and construction cost. Depending on the conditions and places where fish ways are established, the width of fish ways shall generally range from 2 m to 6 m and be about 3% of riverbed width during low water level.
3) The climbing entrance of fish way shall be perpendicular to the original river course.
4) The climbing entrance shall be located near the closure dam.
5) It is not effect in case of low water.
6) The desirable height of one drop is less than 30 cm.
7) The velocity shall be less than 2 m/sec.
8) The hydraulic gradient shall generally range from 1/10 to 1/16.
9) The partition walls between cisterns in fish way shall be provided with a notch.
10) Submerged holes shall be provided in partition walls.

5. 14. 2 Hydraulic Structures of Each Part of Fish Way

1) The width of fish ways are 3 m × 2 sets and are 3% of riverbed width of 200 m. The fish way shall be provided at both sides.
2) The velocity in fish way shall be less than 60 cm/sec.
3) The height of one drop shall be 0.30 m.

4) The overflow depth of notch shall be 0.30 m.
5) The overflow depth of other parts except notch shall be 0.20 m.
6) The width of notch shall be 1.0 m of one-third of overall width of fish way.
7) The depth of cisterns shall be 1.0 m.
8) The length of cisterns (in flow direction) shall be 3.0 m.
9) The submerged holes (50 cm × 20 cm × 2 holes) shall be provided on partition walls.

5. 14. 3 Discharge Capacity of Fish Way

Discharge capacity per one set is as follows.

1) Partition Wall

$Q_1 = C_1 B_1 H_1{}^{3/2}$
$\varepsilon_1 = 0.55 (D_1 - 1) = 0.55 (1.10 - 1) = 0.55$
$C_1 = 1.785 (0.00295/0.20 + 0.237 \times 0.20/1.10) (1 + 0.055) = 1.85$
$Q_1 = 1.85 \times 2.00 \times 0.20^{3/2} \underline{= 0.33\ m^3/s}$

2) Notch

$Q_2 = C_2 B_2 H_2{}^{3/2}$
$\varepsilon_2 = 0.55 (D_2 - 1) = 0.55 (1.0 - 1) = 0$
$C_2 = 1.785 (0.00295/0.30 + 0.237 \times 0.301/100) = 1.87$
$Q2 = 1.87 \times 1.00 \times 0.30^{3/2} \underline{= 0.31\ m^3/s}$

3) Submerged Holes

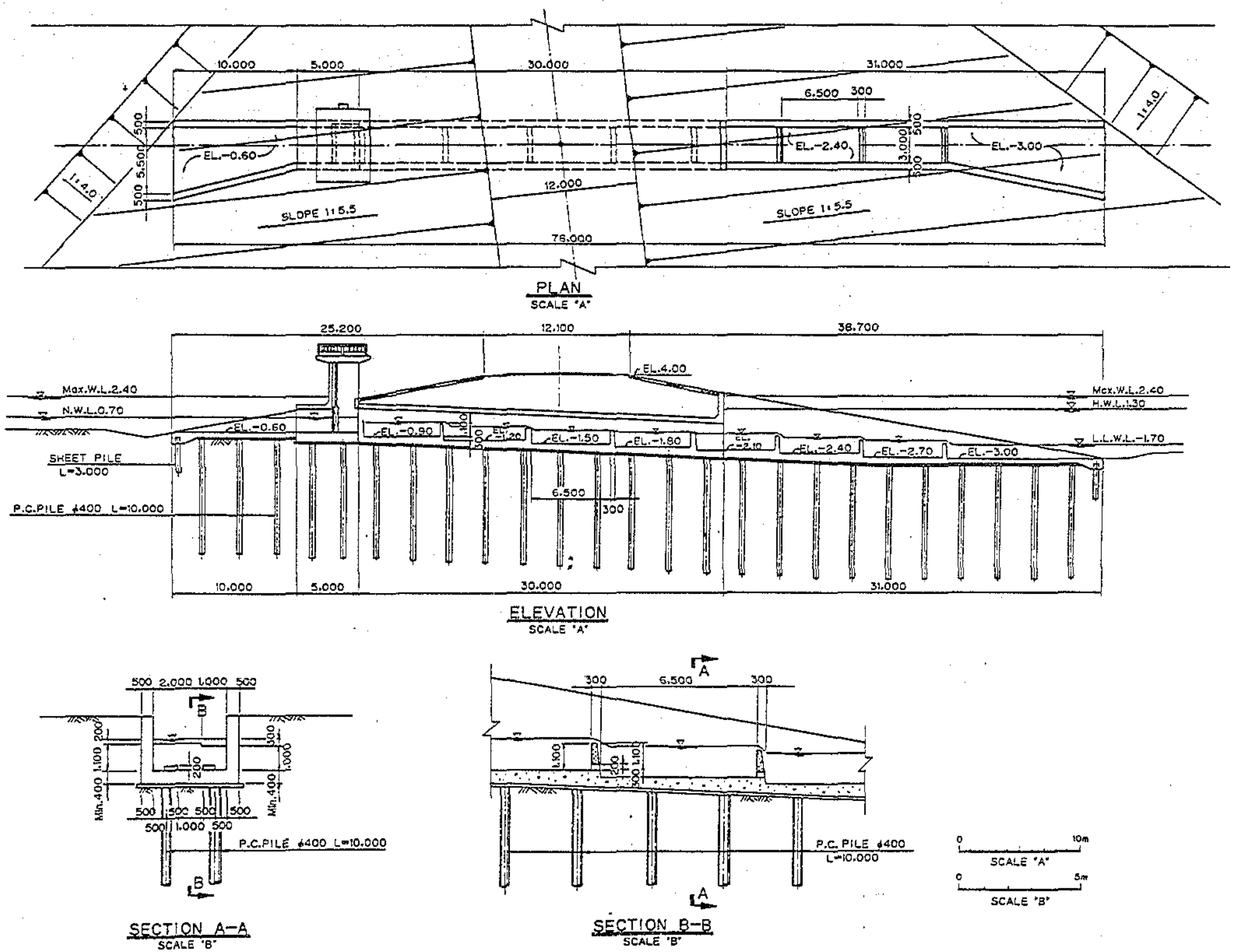
$Q_3 = C_3\, a\, B_3 \sqrt{2g\, h_o}$
$h_2/a = 1.00/0.20 = 5.00,\ h/a = 1.30/0.20 = 6.50$
$C_3 = 0.30$
$Q_3 = 0.30 \times 0.20 \times 0.50 \times 2 \times \sqrt{2 \times 9.8 \times 1.30} \underline{= 0.30\ m^3/s}$

4) Discharge Capacity of Fish Way

$Q = Q_1 \times Q_2 + Q_3 = 0.33 + 0.31 + 0.30 = 0.94\ m^3/s/set$

Total capacity $\Sigma Q = 2 \times 0.94\ m^3/s < $ River maintenance $2.49\ m^3/s$

FIGURE 5-7 LAYOUT OF FISH WAY



APPENDIX - 6 : DESIGN OF CLOSURE DAM

APPENDIX - 6. DESIGN OF CLOSURE DAM

LIST OF CONTENTS

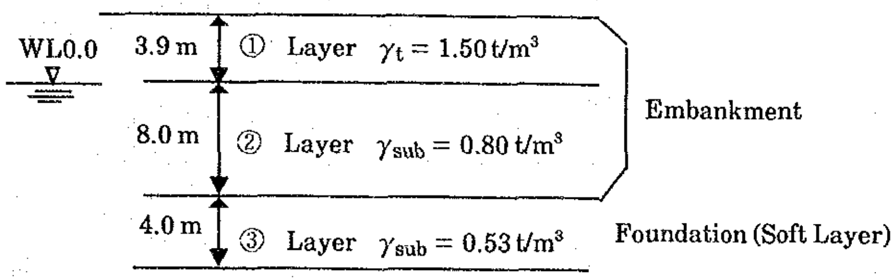
6. 1 Design of Foundation

6. 1. 1 Analysis on Settlement and Stability of Soft Foundation at Riverbed

The followings show the results of the consolidation analysis and the stability analysis by the slip circle slice method for embankment of the closure dam with soft foundation left intact at the river bed.

1) Consolidation Analysis

The models adopted for the analysis are illustrated below.



The settlement shall be calculated by the following equation.

$$S = \frac{e_0 - e_1}{1 + e_0} \cdot H$$

Where : S = Settlement (cm)
e_0 = Initial void ratio before loading
e_1 = Void ratio after loading
H = Thickness of the soft layers

The value of e_0 and e_1 in the above equation are taken at those average values found between depth of 3.0 and 8.5 m in the e-log P curve to shown in Figure 6-1.

- Calculation of vertical stress

Layer	Depth [*1] (m)	Z [*2] (m)	H [*3] (m)	γt, γsub [*4] (t/m^3)	H×γt, γsub (t/m^2)	P [*5] (tf/m^2)
①	0.0 ~ 3.9	1.95	3.9	1.50	5.85	2.925
②	3.9 ~ 11.9	7.90	8.0	0.80	6.40	9.050
③	11.9 ~ 15.9	13.90	4.0	0.53	2.12	13.310

[*1] Depth of each layer [*2] Estimated depth of each layer [*3] Thickness of each layer
[*4] γt ; Wet density γsub ; Submerged density [*5] Effective pressure after loading

- Calculation of consolidation settlement

Layer	H (m)	Pz [*1] (tf/m^2)	e_0 [*2]	e_1 [*3]	Sc [*3] (cm)	ΣSc [*5] (cm)
①	3.9	-	-	-	-	-
②	8.0	-	-	-	-	-
③	4.0	1.060	2.31	1.81	60.4	60.4

[*1] Effective pressure before loading
[*2] Initial void ratio
[*3] Void ratio after loading of embankment
[*4] Consolidation settlement
[*5] Total consolidation settlement

Therefore, consolidation settlement become 60.4 cm

- Calculation of time spent for consolidation degree to reach 80%

$$t = \frac{Tu \cdot H^2}{Cv} = \frac{0.567 \times 400^2}{8.64} = 10{,}500 \text{ day} \doteqdot 28.8 \text{ year}$$

Where : t = Time required for consolidation
Tu = Time factor (0.567 by 80% for consolidation degree)
H = Thickness of soft layers (400 cm)
Cv = Consolidation coefficient
(Cv curve are shown in Figure 6-2,
$Cv = 6.0 \times 10^{-3} cm^2/min = 8.64 cm^2/day$)

It is, therefore, about 29 years required for consolidation degree to reach 80 percent.

FIGURE 6-1 RELATIONSHIP BETWEEN CONSOLIDATION PRESSURE AND VOID RATIO

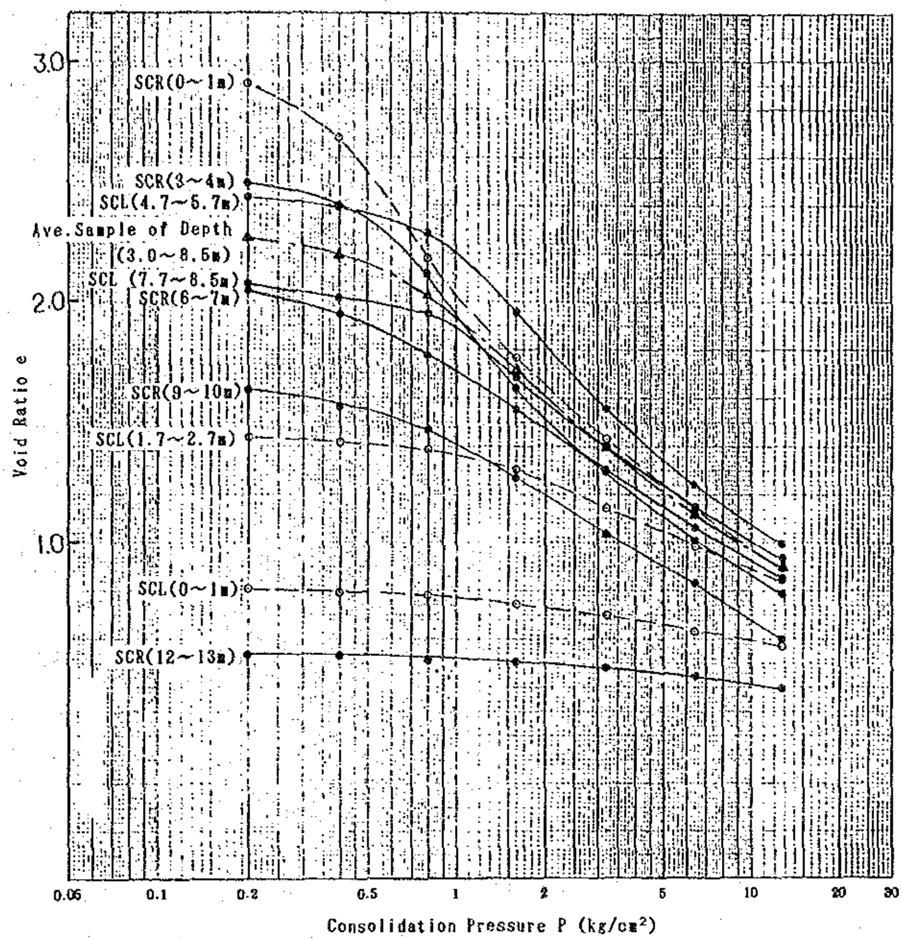
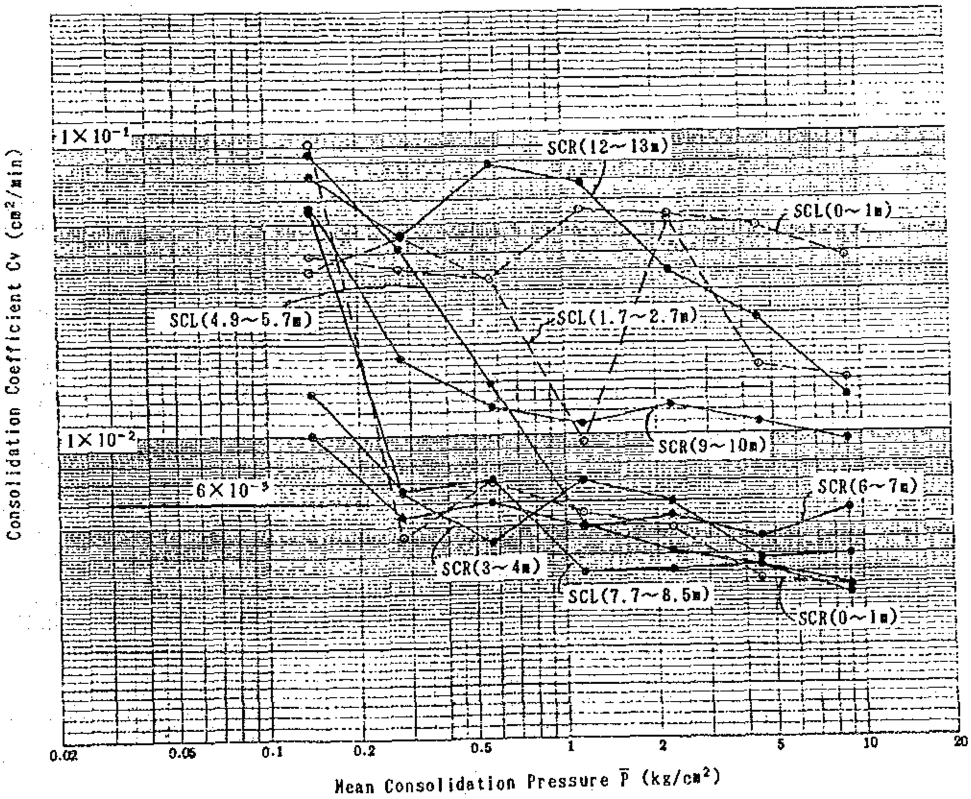


FIGURE 6-2 RELATIONSHIP BETWEEN CV AND $\overline{P}$



2) Stability Analysis

The stability analysis shall be made by the slip circle slice method with the following equation.

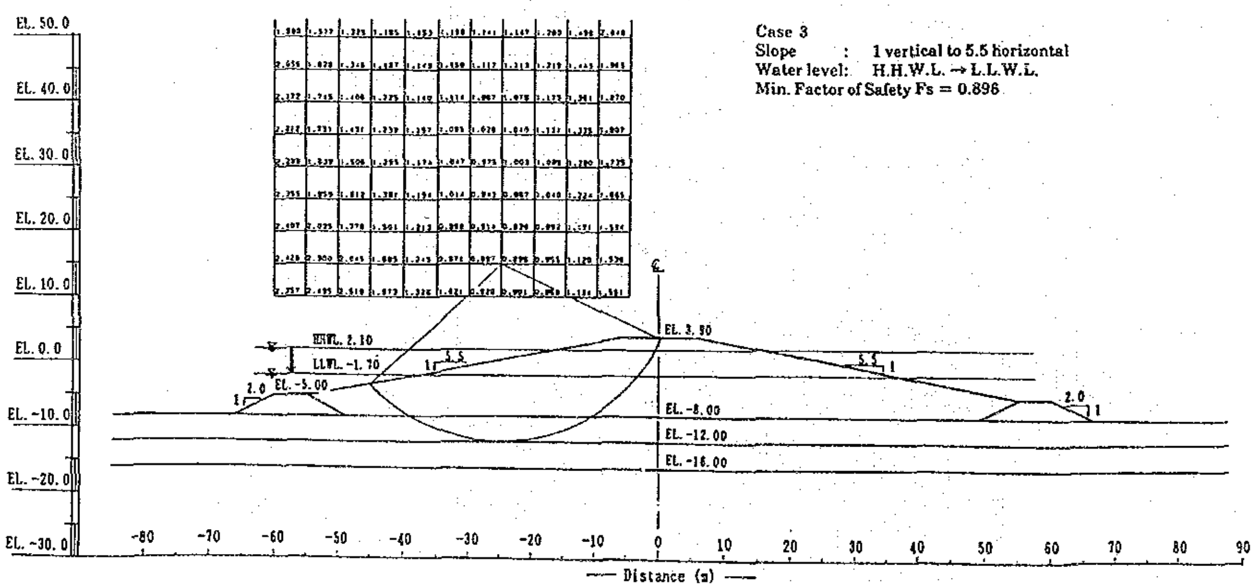
$$F.S = \Sigma[(N - U) \cdot \tan\phi + C \cdot l] / \Sigma T$$

where,
- F.S ; Factor of safety,
- N ; Normal force acting on slip circle of each slice,
- U ; Residual pore pressure acting on slip circle of each slice,
- ø ; Internal friction angle of materials on slip circle of each slice,
- C ; Cohesion of materials on slip circle of each slice,
- l ; Arc length of slip circle of each slice,
- T ; Tangential force acting on slip circle of each slice.

The water level conditions in analysis shall be in drawdown from H.H.W.L. to L.L.W.L. with allowable safety factor by 1.10

The analysis results are shown in Figure 6-3. The safety factor can be expressed Fs = 0.896 < 1.10 and there will be fear for deep slip circle through soft layers.

FIGURE 6-3 RESULT OF STABILITY ANALYSIS
(UNIMPROVED SOFT FOUNDTION AT THE RIVERBED)



6.1.2 Study as to Sand Compaction Pile for Soft Foundation at Riverbed

As to foundation treatment at the riverbed, the replacement method and sand compaction pile method are considered.

In this case, the design of sand compaction pile method at the riverbed is reported herein.

1) Decision of Pile Interval

Sand compaction piles, in considering construction on the water, shall be placed with diameters as large as 2000 mm and casing diameter of 1500 mm. The pile intervals shall be determined by using the following Barron's equation.

Barron's Equation

$$U(Th) = 1 - \exp\left(-\frac{8Th}{F(n)}\right)$$

$$F(n) = \frac{n^2}{n^2-1}\ \ell_n n - \frac{3n^2-1}{4n^2}$$

$$n = de/dw$$

$$Tn = (Cv/de^2)t$$

Where :U(Th); Consolidation degree for time factor
Th; Time coefficient for consolidation degree
de; Circular conversion of water collecting capacity in diameter per pile (cm)
dw; Diameter of sand compaction pile (200 cm)
Cv; Consolidation coefficient in horizontal direction (cm^2/day)
t; Time factor for necessary consolidation degree (day)

Calculation of F(n)

①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫
d	de	dw	n	n^2	n^2	ln (n)	$3n^2$-1	$4n^2$	⑧/⑨	⑥×⑦	F (n)
(m)	(cm)	(cm)	②/③		(n^2-1)						⑪-⑩
2.2	248.6	200	1.243	1.55	2.818	0.218	3.65	6.2	0.589	0.614	0.025
2.3	259.9	〃	1.300	1.69	2.449	0.262	4.07	6.8	0.599	0.642	0.043
2.4	271.2	〃	1.356	1.84	2.190	0.305	4.52	7.4	0.611	0.668	0.057
2.5	282.5	〃	1.413	2.00	2.000	0.346	5.00	8.0	0.625	0.692	0.067
2.6	293.8	〃	1.469	2.16	1.862	0.385	5.48	8.6	0.637	0.717	0.080
2.7	305.1	〃	1.526	2.33	1.752	0.423	5.99	9.3	0.644	0.741	0.097
2.8	316.4	〃	1.582	2.50	1.667	0.459	6.50	10.0	0.650	0.765	0.115
2.9	327.7	〃	1.639	2.69	1.595	0.494	7.07	10.8	0.655	0.788	0.133

Calculation of U(Th)

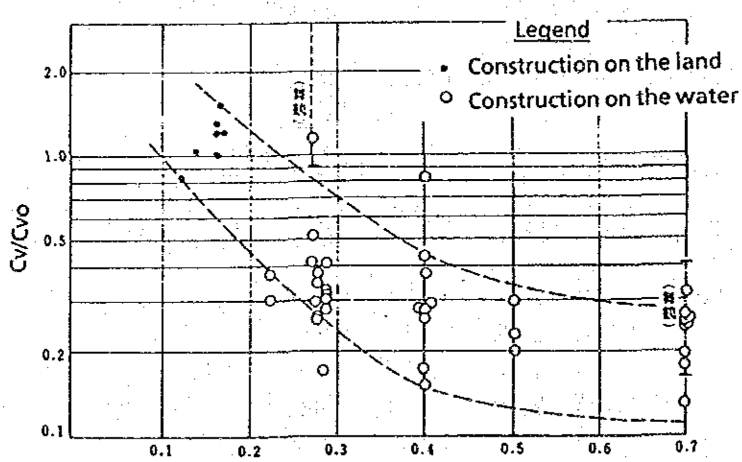
① d (m)	⑬ de²	⑭ Cv*t *1	⑮ Th ⑭/⑬	⑯ -8Th	⑰ ⑯/⑫	⑱ Exp ⑰	⑲ U 1-⑱	As *2
2.2	61802	946.0	0.015	-0.120	-4.800	0.008	0.992	0.649
2.3	67548	〃	0.014	-0.112	-2.605	0.074	0.926	0.593
2.4	73549	〃	0.013	-0.104	-1.825	0.161	0.839	0.545
2.5	79806	〃	0.012	-0.096	-1.433	0.239	0.761	0.502
2.6	86318	〃	0.011	-0.088	-1.100	0.333	0.667	0.464
2.7	93086	〃	0.010	-0.080	-0.825	0.438	0.562	0.430
2.8	100109	〃	0.009	-0.072	-0.626	0.535	0.465	0.400
2.9	107387	〃	0.009	-0.072	-0.541	0.582	0.418	0.373

*1 $Cv = 1.2 \times 1.0^{-3}$ cm²/min = 1.73 cm²/day t= 18 month
*2 As : Replacement ratio

Consequently, pile intervals by 2.4 m and the replacement ratio by about 55 percent are necessarily required for consolidation degree to reach more than 80 percent during 1.5 years form commencement of foundation improvement to completion of the Project construction works.

For the above calculation, Cv of consolidation coefficient, as exceeding 20 percent in its replacement ratio, is obtained by multiplying the values of Cv shown in Figure 6-2 by 0.2 in taking into consideration delay in consolidation in construction works. (refer to Figure 6-4)

FIGURE 6-4 DELAY IN CONSOLIDATION S.C.P. LAND IMPROVEMENT



$Cv/Cvo = 0.2$
$Cv = 0.2 \times Cvo$
$= 0.2 \times 6 \times 10^{-3}$ cm²/min
$= 1.2 \times 10^{-3}$ cm²/min = 1.73 cm²/day

Where : Cv ; Consolidation coefficient for consolidation analysis
Cvo ; Consolidation coefficient obtained form soil tests (6×10^{-3} cm²/min)

2) Stability Analysis of Sand Compaction Pile

For further information, in case of implementation of sand compaction pile, the necessary stability analysis shall be made by slip circle slice method.

The water level conditions for the stability analysis are taken as follows.

Case	Water Level	Remarks III
Case 1	Constant W.L.	H.H.W.L. 2.10
Case 2	〃	L.L.W.L. -1.70
Case 3	Drawdown W.L.	H.H.W.L. → L.L.W.L.

The stability analysis shall be made with FS 1.20 for Case 1 and 2, 1. 10 for Case 3, respectively, in paying attention to the fact that the water level conditions are difference in frequency in each case.

And the respective design values used for stability analysis are shown in Table 5-1.

TABLE 6-1 DESIGN VALUES FOR STABILITY ANALYSIS

Zone		Density			Shear Strength	
		γt (t/m³) *1	γsat (t/m³) *2	γsub (t/m³) *3	C (tf/m²) *4	ϕ (°) *5
Earthfill Zone		1.50	1.80	0.80	0	25
Rockfill Zone		1.80	2.20	1.20	0	35
Soft Layerr	Unimproved Ground	1.55	1.55	0.55	Cu = 1.5(P ≦ 7.5 tf/m³) *6 Cu = 1.5 + 0.2(P - 7.5)U (P > 7.5 tf/m²)	0
	Sand Compaction Pile	1.80	2.00	1.00	0	30
	Composite Ground (As = 0.55)	1.69	1.80	0.80	0.45 Cu	17.6
	Intermediate Layer	1.98	1.98	0.98	6.5	0
	Foundation Layer	2.07	2.07	1.07	21.7	0

*1 Wet density *2 Saturated density *3 Submerged density *4 Cohesion
*5 Friction angle *6 P : Effective load of objective ground
U : Consolidation degree of objective ground

The design values are obtained based on the followings.

- Dam body (earthfill zone, rockfill zone)
  ; refer to analysis results in Appendix 6. 2. 1
- Foundation (unimproved portions of soft layer)

Design density depends on the results of the soil test to be conducted.

Design shear strength shall be obtained by the following equation in considering initial strength (Cu_0), rate of strength increase (m), and consolidation degree (U).

$Cu = Cuo \qquad (P \leqq 7.5\ tf/m^2)$
$Cu = Cuo + m(P - 7.5)U \qquad (P > 7.5\ tf/m^2)$

where : Cuo ; 1.5 tf/m^2 (by the result of in-situ test)
m ; 2.0 (by Skempton's equation $m = 0.11 + 0.0037\ Ip = 0.184 \fallingdotseq 0.2$ Ip : Plasticity index)

- Foundation (improved portions of soft layer)

  Design density and shear strength shall be obtained as composite ground.

- Foundation (Intermediate layer and foundation layer)

  Both the design density and shear strength have been obtained by soil tests and in-site tests.

The results of stability analysis are shown in Table 6-2 and Figure 6-5.

TABLE 6 - 2 RESULTS OF STABILITY ANALYSIS
(IMPROVED SOFT FOUNDATION AT THE RIVERVBED)

Case	Water Level	Calculation		
		Safety Factor		Allowable Safety Factors
Case 1	Constant W.L. (H.H.W.L.)	2.074	>	1.20
Case 2	〃 W.L. (L.L.W.L.)	1.978	>	1.20
Case 3	Drawdown W.L. (H.H.W.L.) → (L.L.W.L.)	1.120	>	1.10

Judged from the results of stability analysis, the allowable safety factors can be satisfied in any cases.

FIGURE 6 - 5(1) RESULT OF STABILITY ANALYSIS
(IMPROVED SOFT FOUNDATION AT THE RIVERBED)

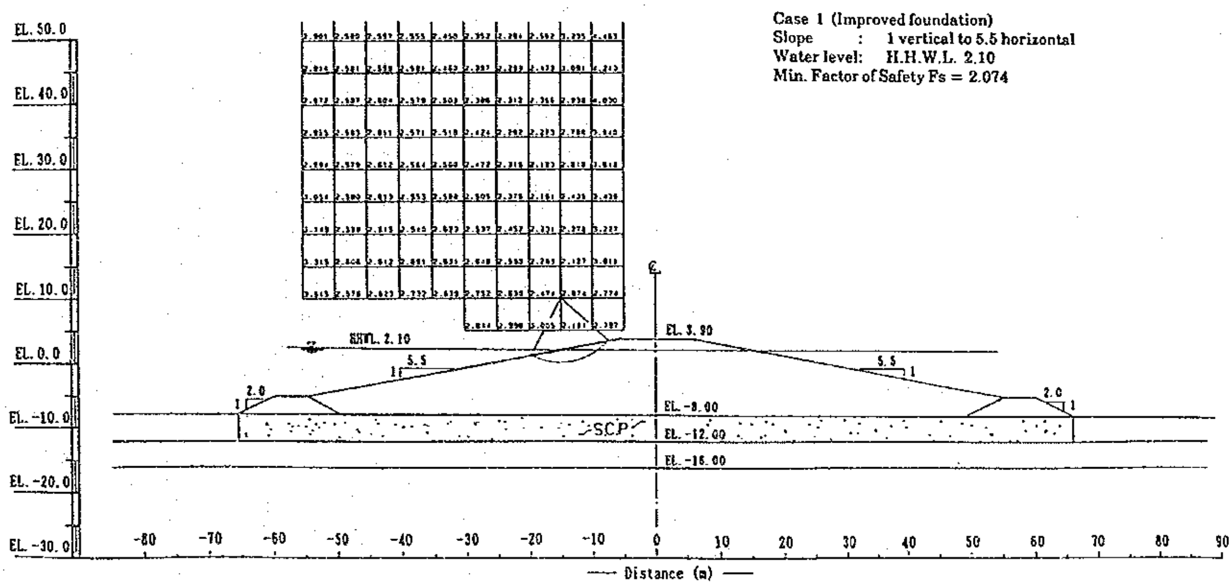
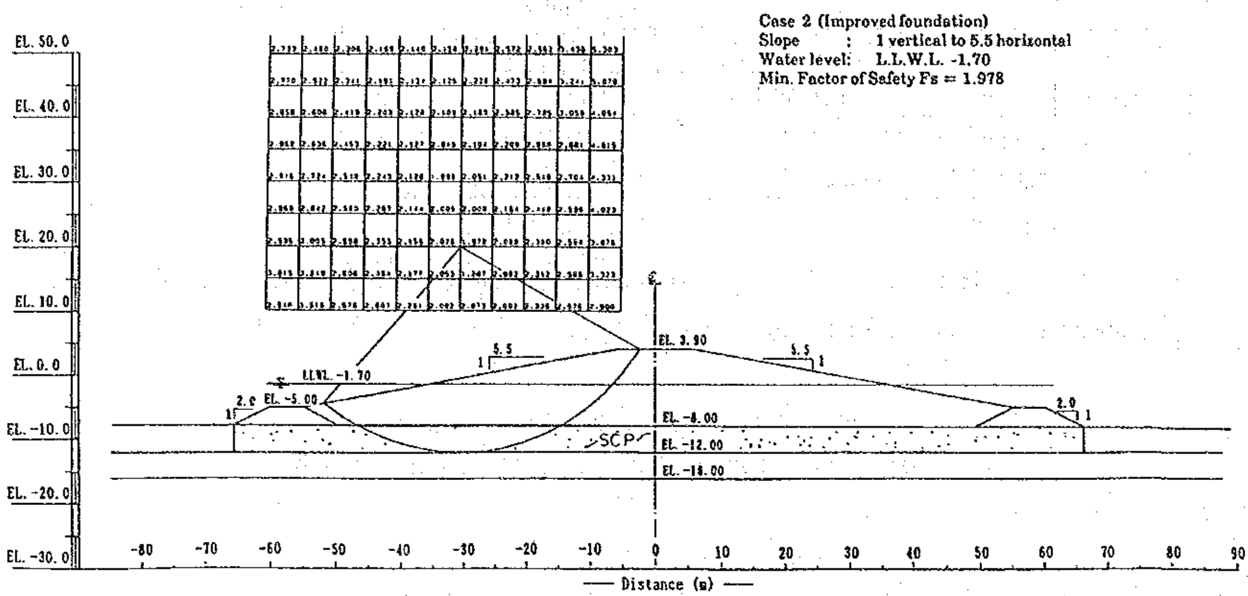
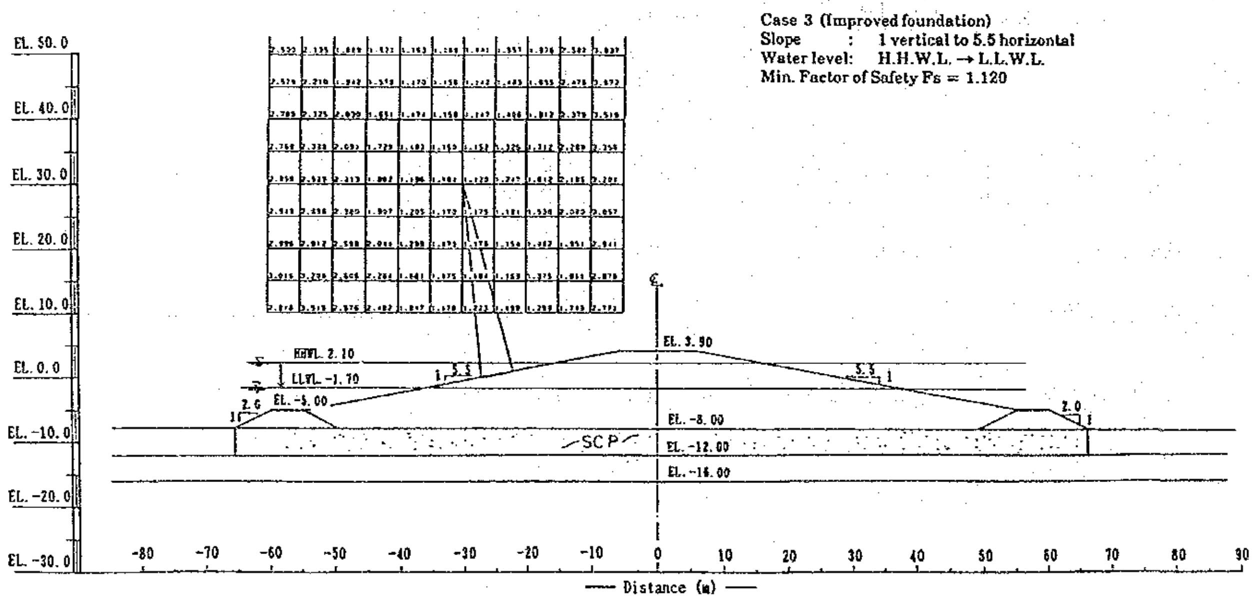


FIGURE 6-5(2) RESULT OF STABILITY ANALYSIS
(IMPROVED SOFT FOUNDATION AT THE RIVERBED)





6.1.3 Design of Sand Compaction Piles at Abutment Foundation

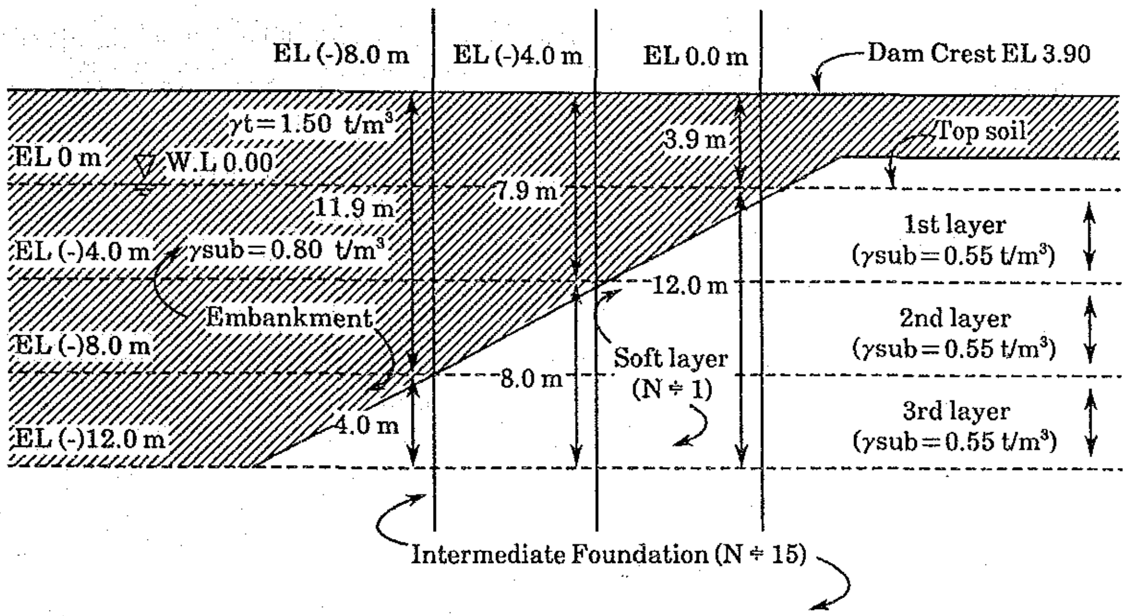
Foundation improvement by sand compaction pile method shall be carried out soft layers at both abutments.

1) Determination of Improvement Extent for the Soft Foundation

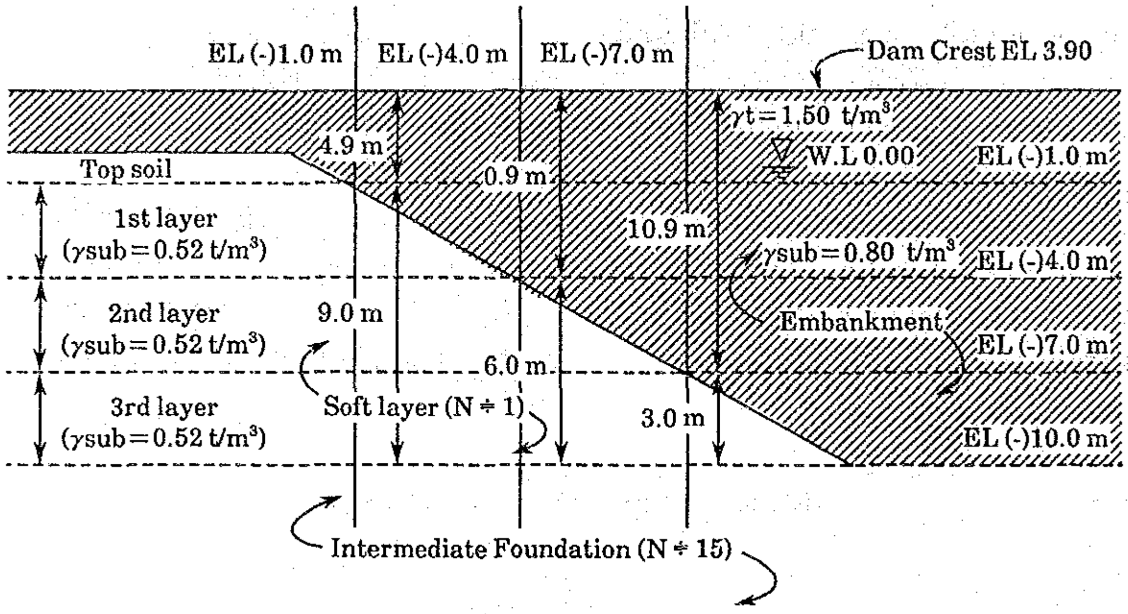
Determination of the extent of soft layer improvement shall be made based on the results of stability analysis, which is performed in conditions of dam foundation by EL 0m, EL (-)4.0 m and EL (-)8.0 m at the right abutment, and EL (-)1.0 m, EL (-) 4.0 m and EL (-)7.0 at the left abutment, respectively, as shown in Figure 6-6.

FIGURE 6-6 PROFILE OF CLOSURE DAM

(RIGHT ABUTMENT)



(LEFT ABUTMENT)



The design vales of the soft layers found in the both abutment areas are shown in the following table.

TABLE 6 - 3 DESIGN VALUES OF THE STABILITY ANALYSIS (SOFT FOUNDATION AT THE ABUTMENT)

Position		Density			Shear Strength	
		γt (t/m³) *1	γsat (t/m³) *2	γsub (t/m³) *3	C (tf/m²) *4	ø (°) *5
Soft Layer	Right Abutment	1.55	1.55	0.55	Cu=1.5(P≦7.5 tf/m³) *6	0
	Left Abutment	1.52	1.52	0.52	Cu=1.5+0.2(P-7.5)U (P>7.5 tf/m³)	0

*1 Wet density *2 Saturated density *3 Submerged density *4 Cohesion
*5 Friction angle *6 P : Effective load of objective ground
U : Consolidation degree of objective ground

For further references, the design values for the intermediate layers and the foundation layers for the dam embankment foundation shall be the same as shown in Table 6-1. The result of the stability analysis are shown in Table 6-4 and Figure 6-7.

TABLE 6-4 RESULTS OF STABILITY ANALYSIS FOR UNIMPROVED LAYERS

<RIGHT ABUTMENT FOUNDATION>

*1 Profile for Analysis	Stability Analysis by Slip Circle Method					Consolidation (Settlement)	
	Case	Water Level *2	Safety Factor		Allowable Safety Factor	Total Consolidation (cm)	Time Required to Reach U_{80} (yrs.)
I	Case 1	Constant W. L. (H.H.W.L.)	1.498	>	1.20	50.9	28.8
	Case 2	〃 W. L. (L.L.W.L.)	1.117	<	1.20		
	Case 3	Drawdown W. L. (H.H.W.L. → L.L.W.L.)	0.896	<	1.10		
II	Case 1	Constant W. L. (H.H.W.L.)	1.410	>	1.20	89.9	115.1
	Case 2	〃 W. L. (L.L.W.L.)	1.024	<	1.20		
	Case 3	Drawdown W. L. (H.H.W.L. → L.L.W.L.)	0.872	<	1.10		
III	Case 1	Constant W. L. (H.H.W.L.)	1.901	>	1.20	94.8	258.9
	Case 2	〃 W. L. (L.L.W.L.)	1.460	>	1.20		
	Case 3	Drawdown W. L. (H.H.W.L. → L.L.W.L.)	1.314	>	1.10		

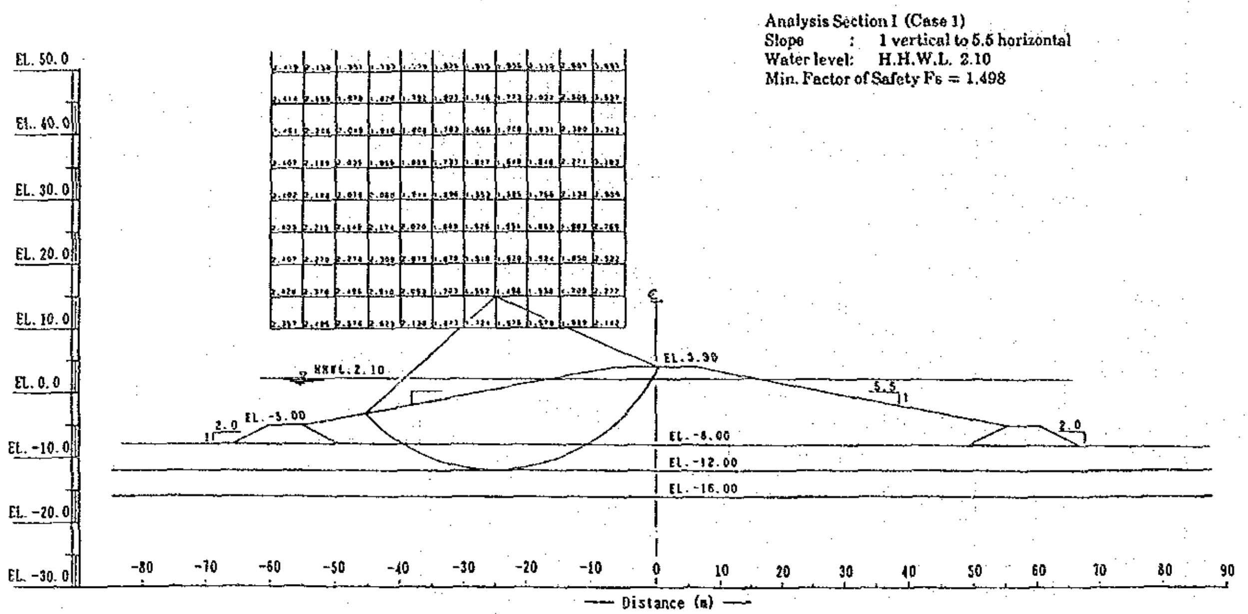
*1 Profile for analysis I: Foundation excavation elevation EL (-)8.0 m
Profile for analysis II: Foundation excavation elevation EL (-)4.0 m
Profile for analysis III: Foundation excavation elevation EL (-)0.0 m
*2 H.H.W.L. 2.10 m L.L.W.L. (-)1.70 m

<LEFT ABUTMENT FOUNDATION>

*1 Profile for Analysis	Stability Analysis by Slip Circle Method					Consolidation (Settlement)	
	Case	Water Level	Safety Factor		Allowable Safety Factor	Total Consolidation (cm)	Time Required to Reach U_{80} (yrs.)
I	Case 1	Constant W. L. (H.H.W.L.)	1.618	>	1.20	30.7	16.2
	Case 2	〃 W. L. (L.L.W.L.)	1.228	>	1.20		
	Case 3	Drawdown W. L. (H.H.W.L. → L.L.W.L.)	0.953	<	1.10		
II	Case 1	Constant W. L. (H.H.W.L.)	1.526	>	1.20	44.1	64.7
	Case 2	〃 W. L. (L.L.W.L.)	1.115	<	1.20		
	Case 3	Drawdown W. L. (H.H.W.L. → L.L.W.L.)	0.931	<	1.10		
III	Case 1	Constant W. L. (H.H.W.L.)	2.074	>	1.20	44.5	145.6
	Case 2	〃 W. L. (L.L.W.L.)	1.601	>	1.20		
	Case 3	Drawdown W. L. (H.H.W.L. → L.L.W.L.)	1.416	>	1.10		

*1 Profile for analysis I: Foundation excavation elevation EL (-)7.0 m
Profile for analysis II: Foundation excavation elevation EL (-)4.0 m
Profile for analysis III: Foundation excavation elevation EL (-)1.0 m

FIGURE 6 - 7(1) RESULTS OF STABILITY ANALYSIS
(UNIMPROVED SOFT FOUNDATION AT THE RIGHT ABUTMENT FOUNDATION)



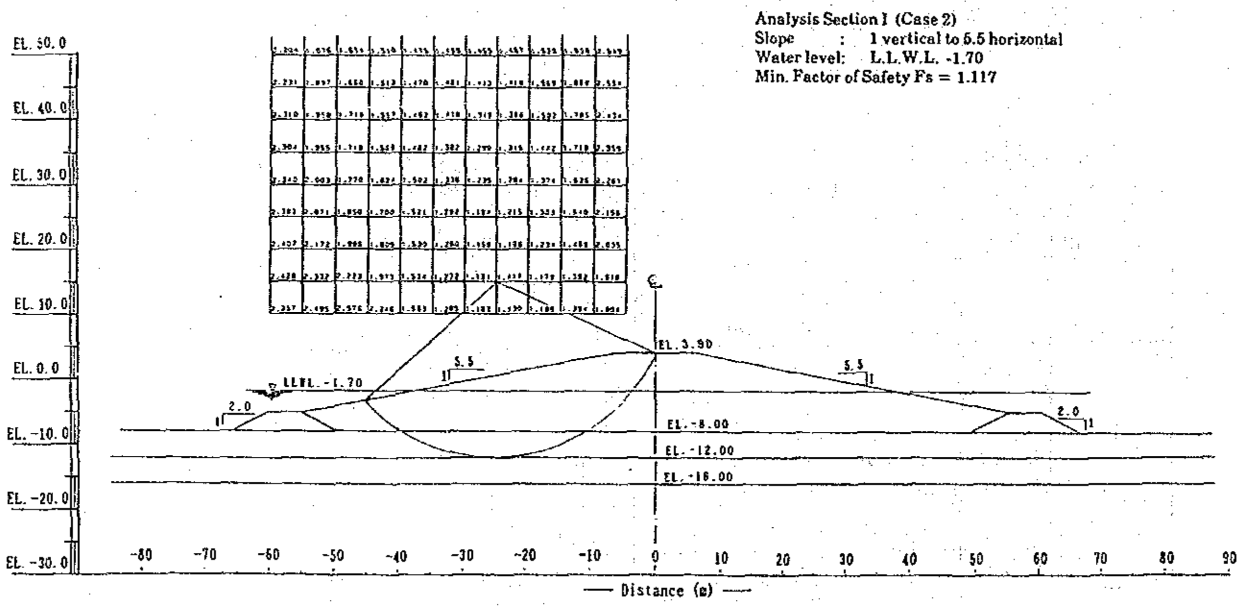
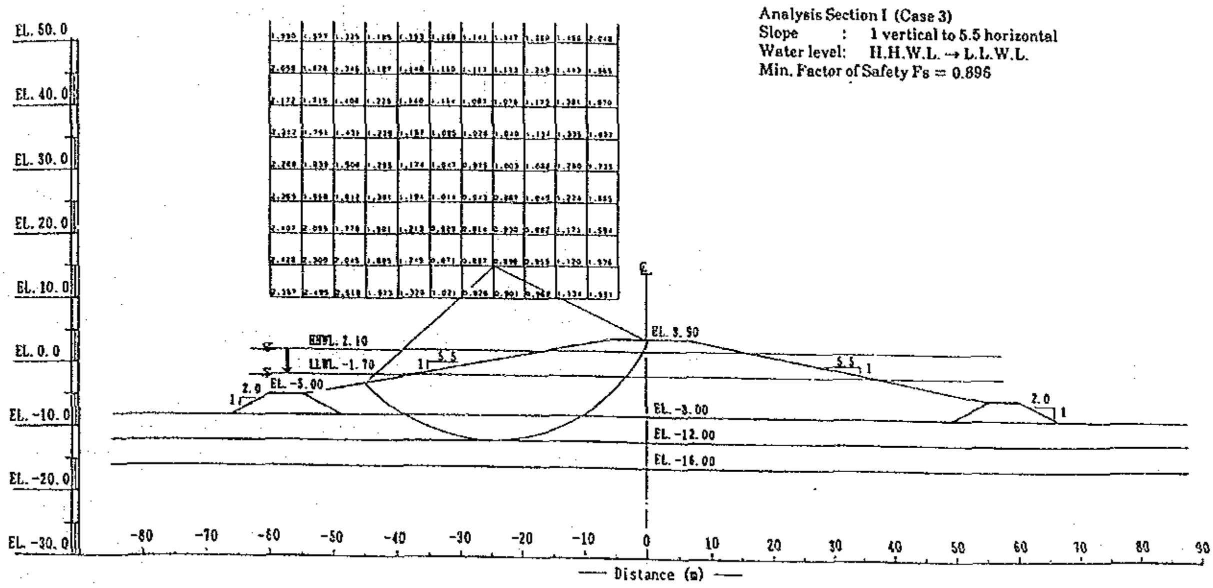


FIGURE 6 - 7(2) RESULTS OF STABILITY ANALYSIS (UNIMPROVED SOFT FOUNDATION AT THE RIGHT ABUTMENT FOUNDATION)



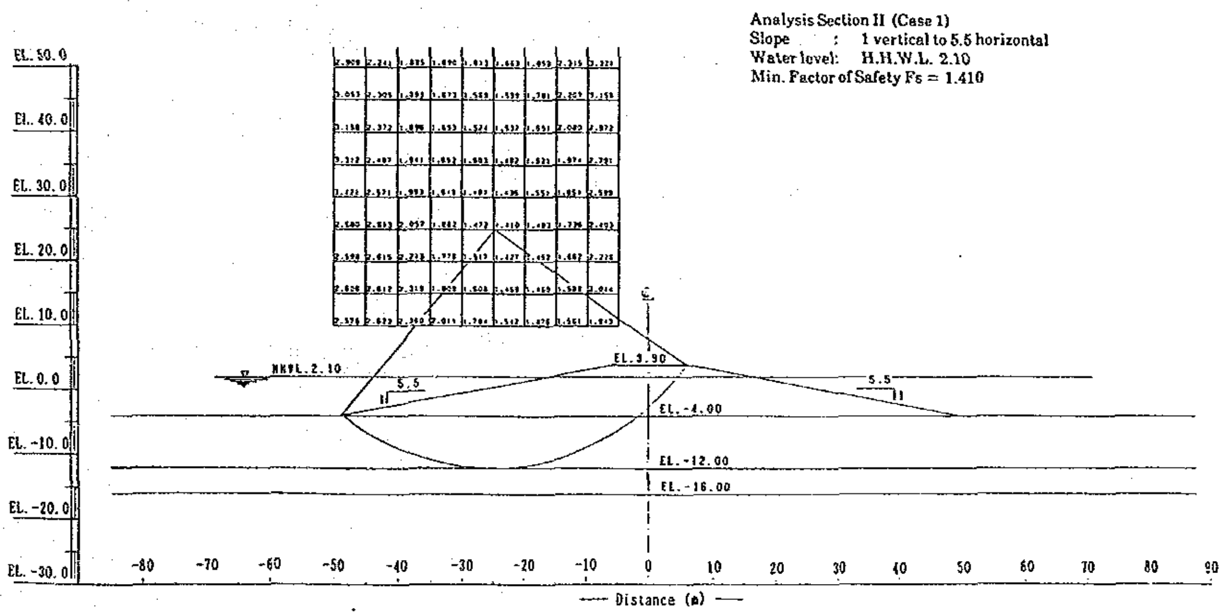
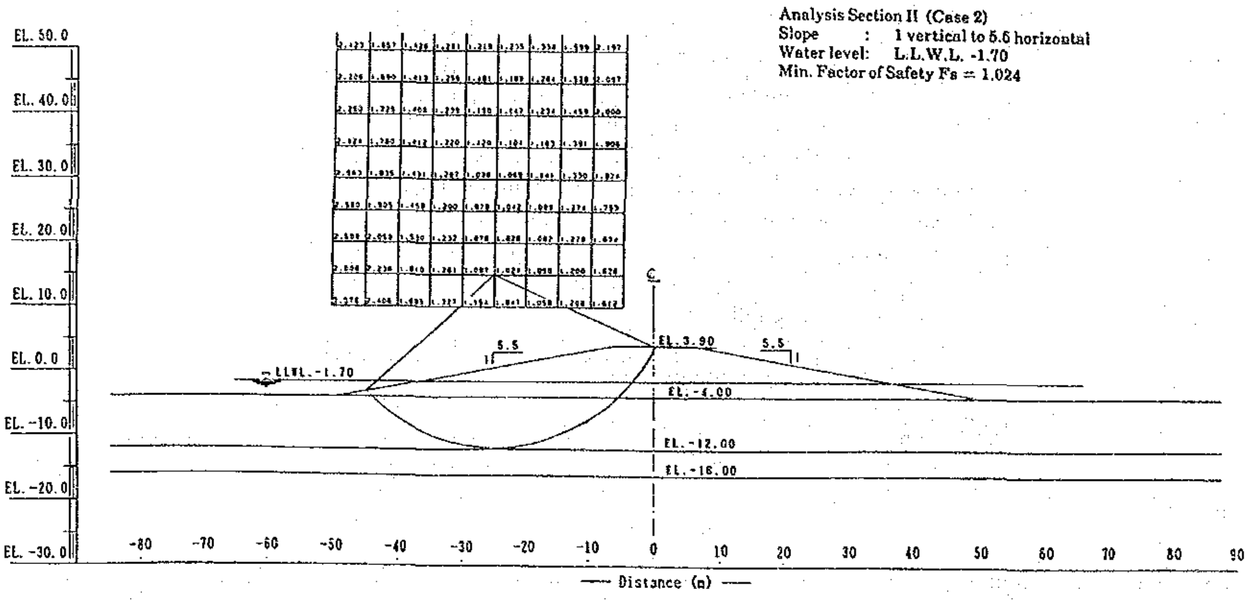


FIGURE 6 - 7(3) RESULTS OF STABILITY ANALYSIS
(UNIMPROVED SOFT FOUNDATION AT THE RIGHT ABUTMENT FOUNDATION)



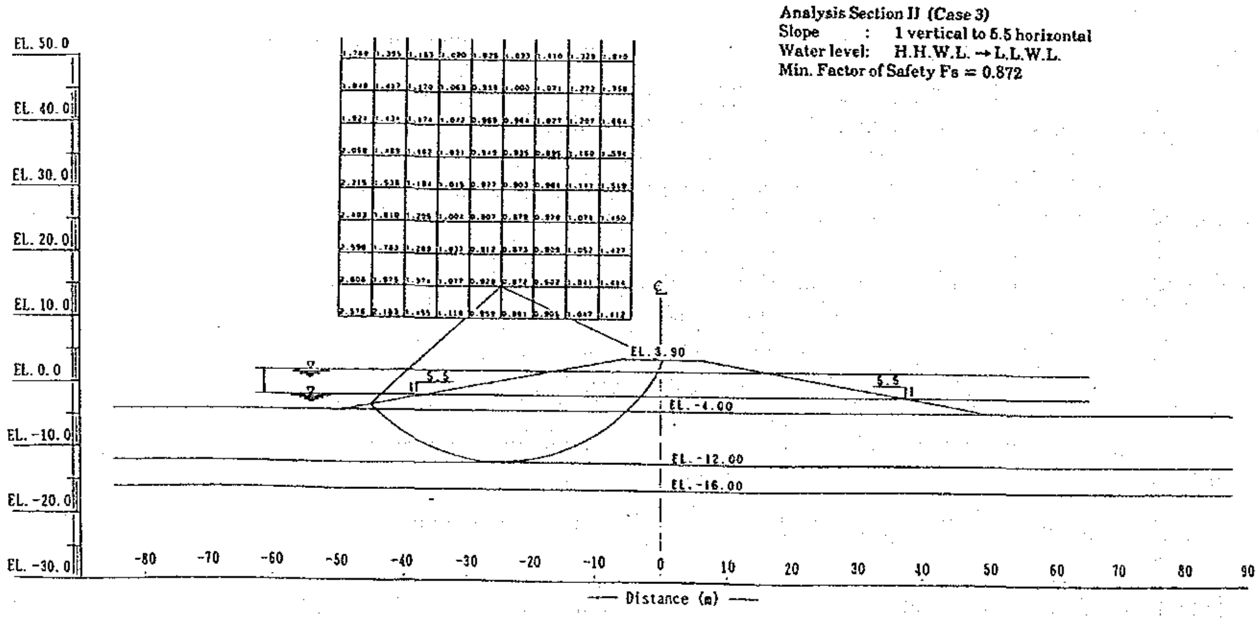
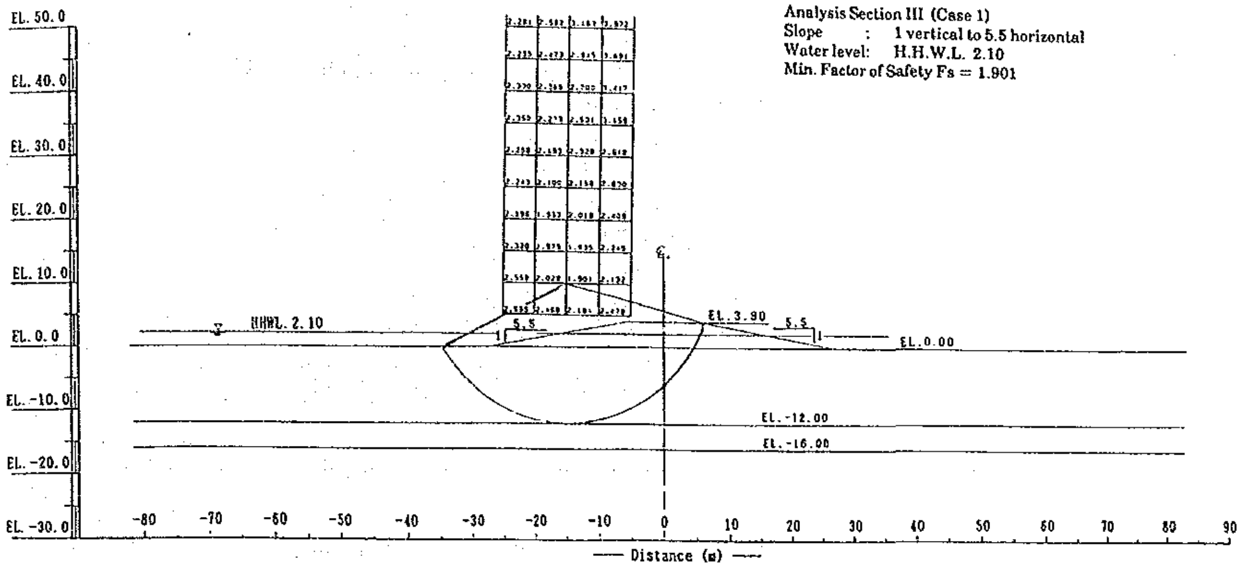


FIGURE 6 - 7(4) RESULTS OF STABILITY ANALYSIS
(UNIMPROVED SOFT FOUNDATION AT THE RIGHT ABUTMENT FOUNDATION)



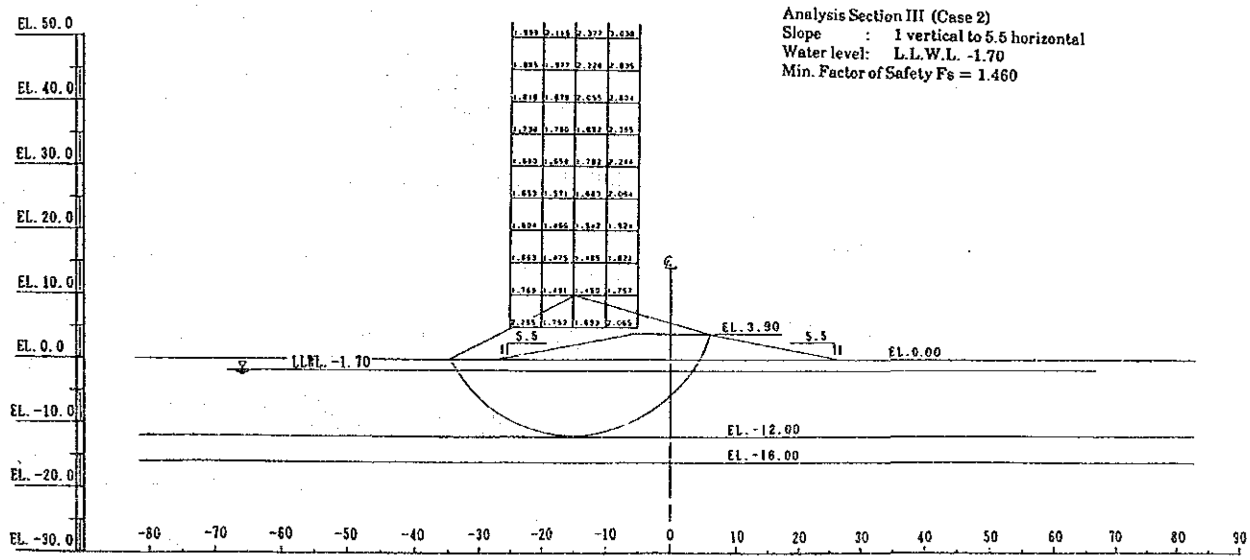
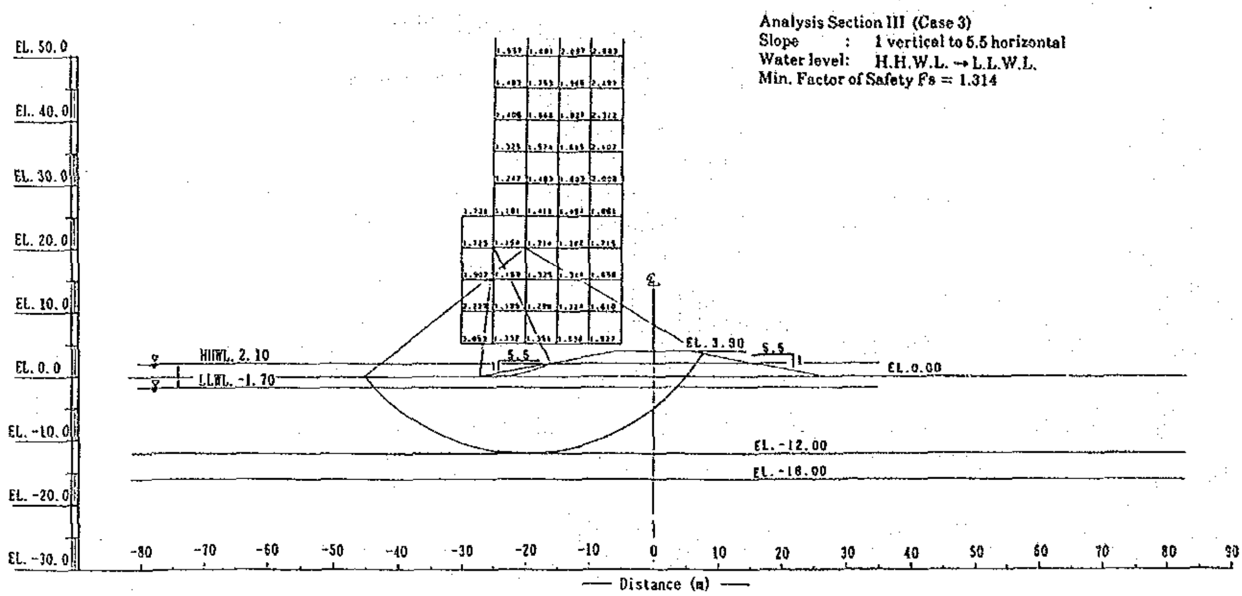


FIGURE 6 - 7(5) RESULTS OF STABILITY ANALYSIS

(UNIMPROVED SOFT FOUNDATION AT THE RIGHT ABUTMENT FOUNDATION)



(UNIMPROVED SOFT FOUNDATION AT THE LEFT ABUTMENT FOUNDATION)

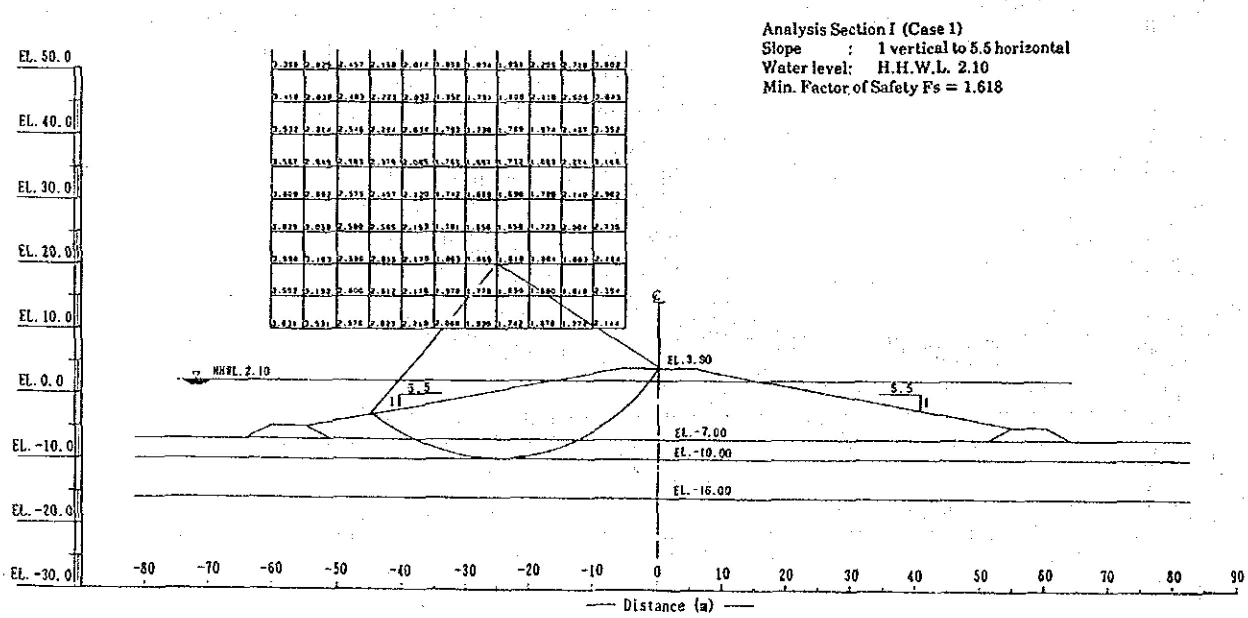
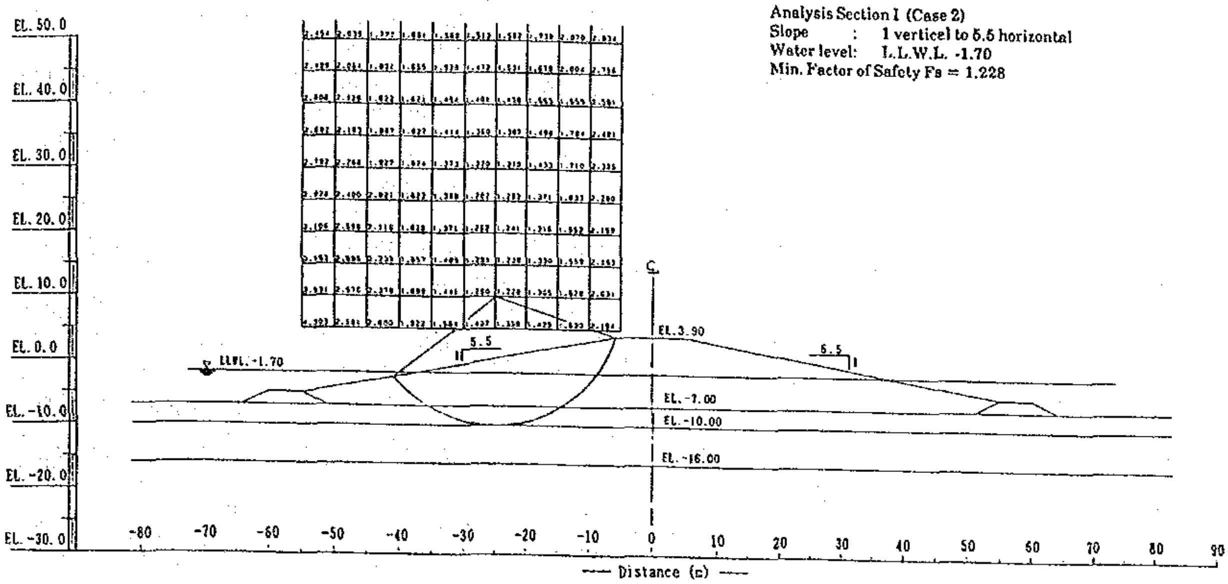


FIGURE 6 - 7(6) RESULTS OF STABILITY ANALYSIS
(UNIMPROVED SOFT FOUNDATION AT THE LEFT ABUTMENT FOUNDATION)



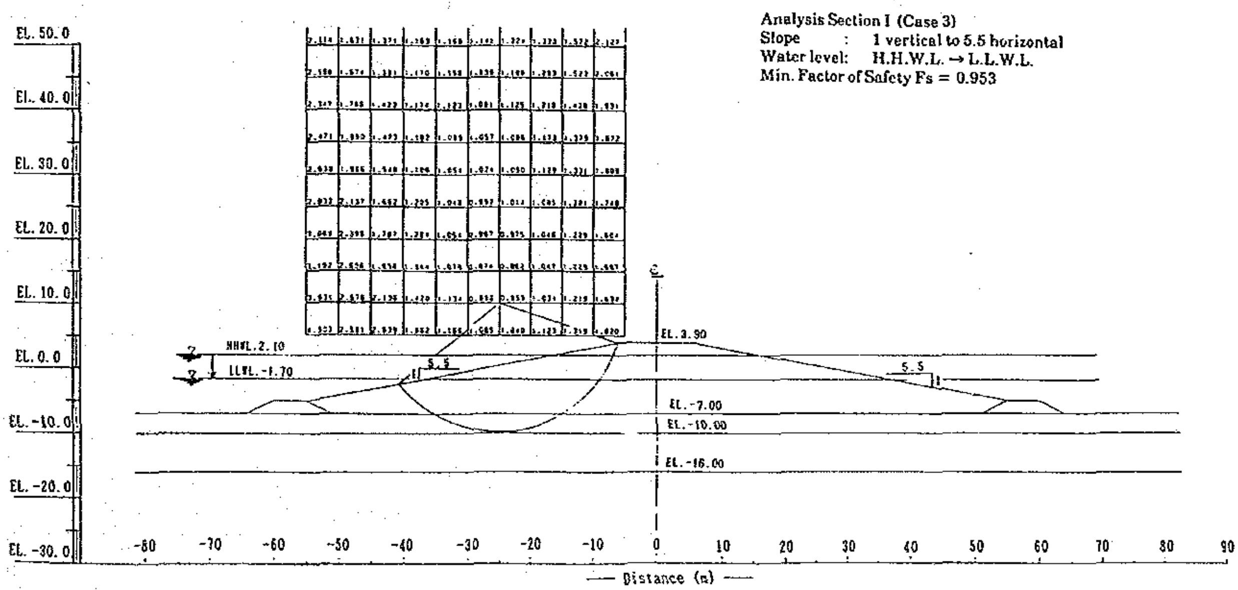
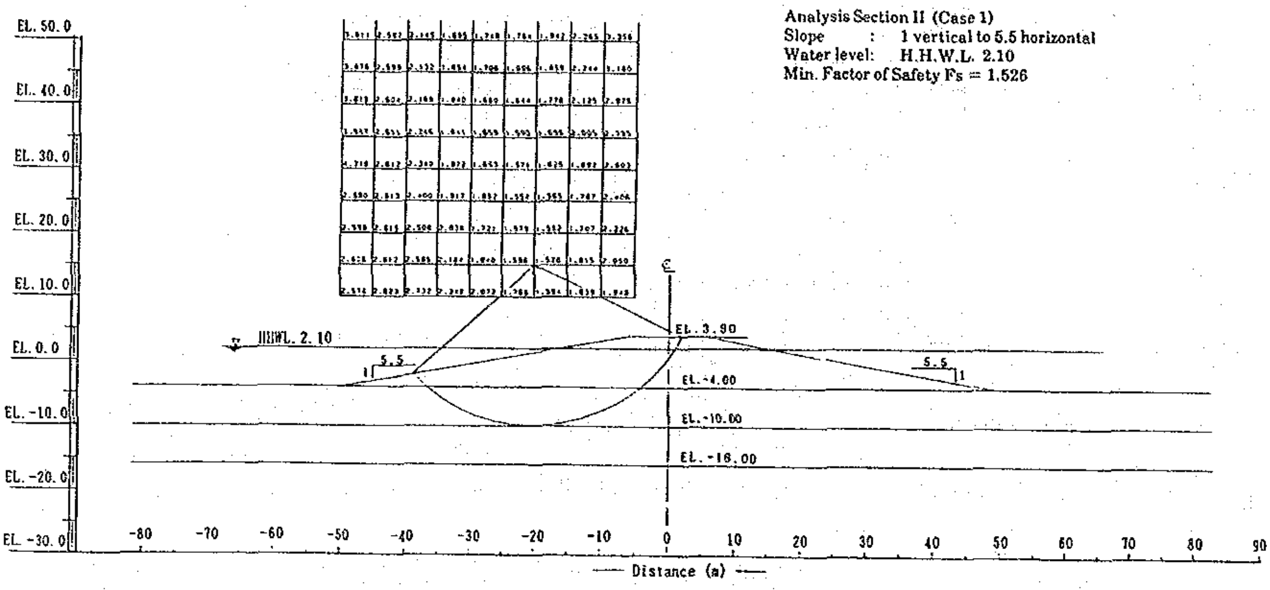


FIGURE 6 - 7(7) RESULTS OF STABILITY ANALYSIS
(UNIMPROVED SOFT FOUNDATION AT THE LEFT ABUTMENT FOUNDATION)



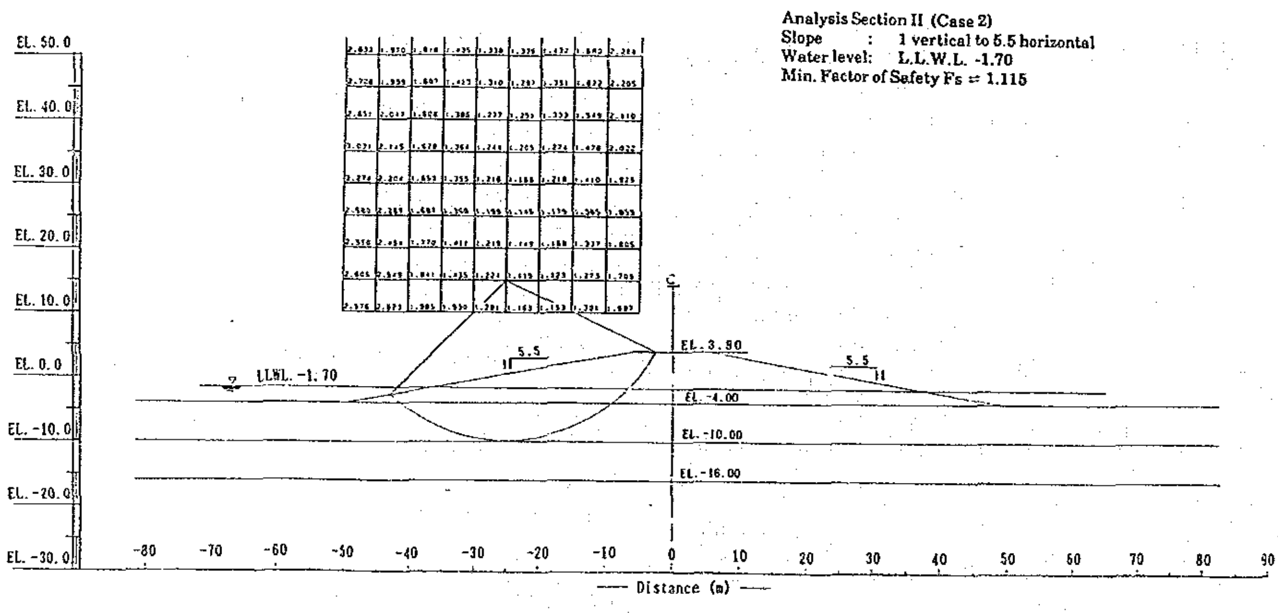
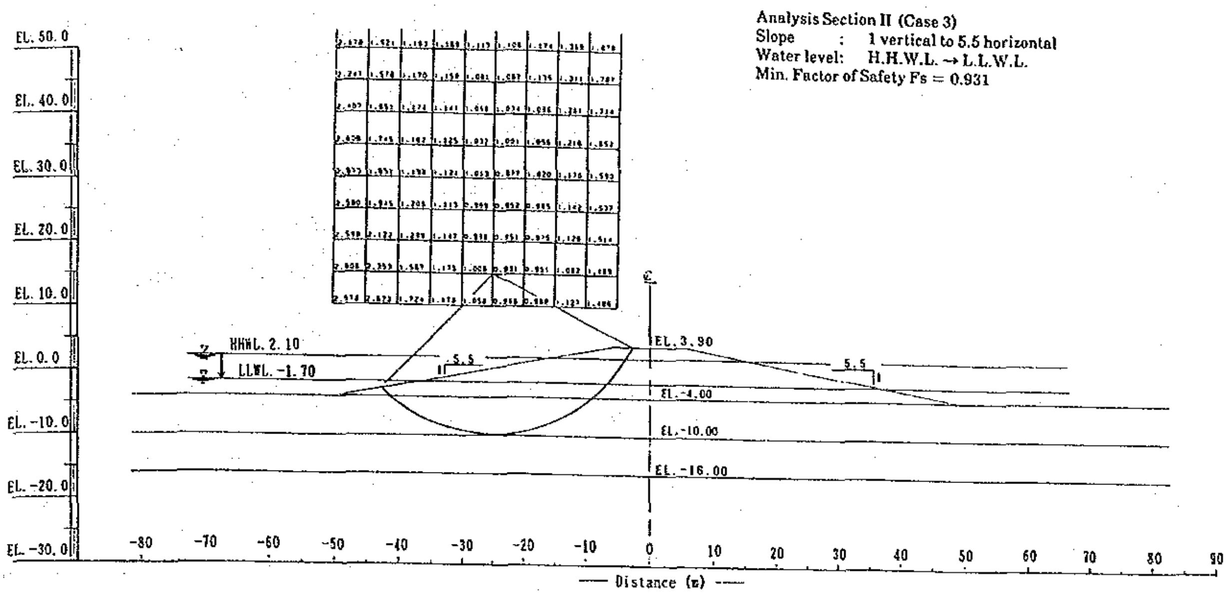


FIGURE 6 - 7(8) RESULTS OF STABILITY ANALYSIS (UNIMPROVED SOFT FOUNDATION AT THE LEFT ABUTMENT FOUNDATION)



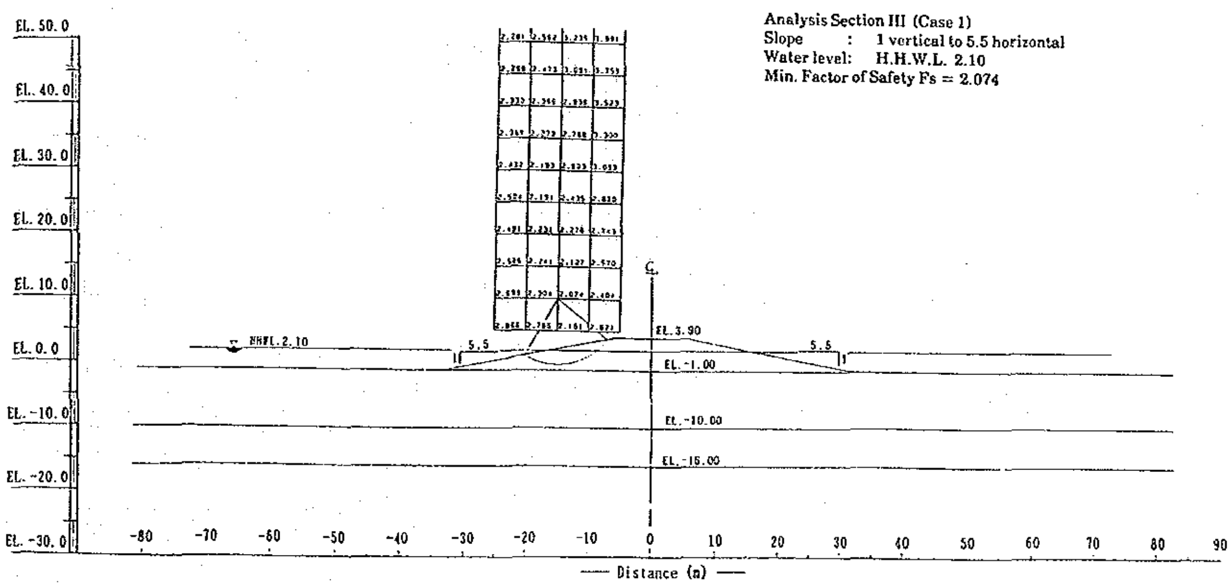
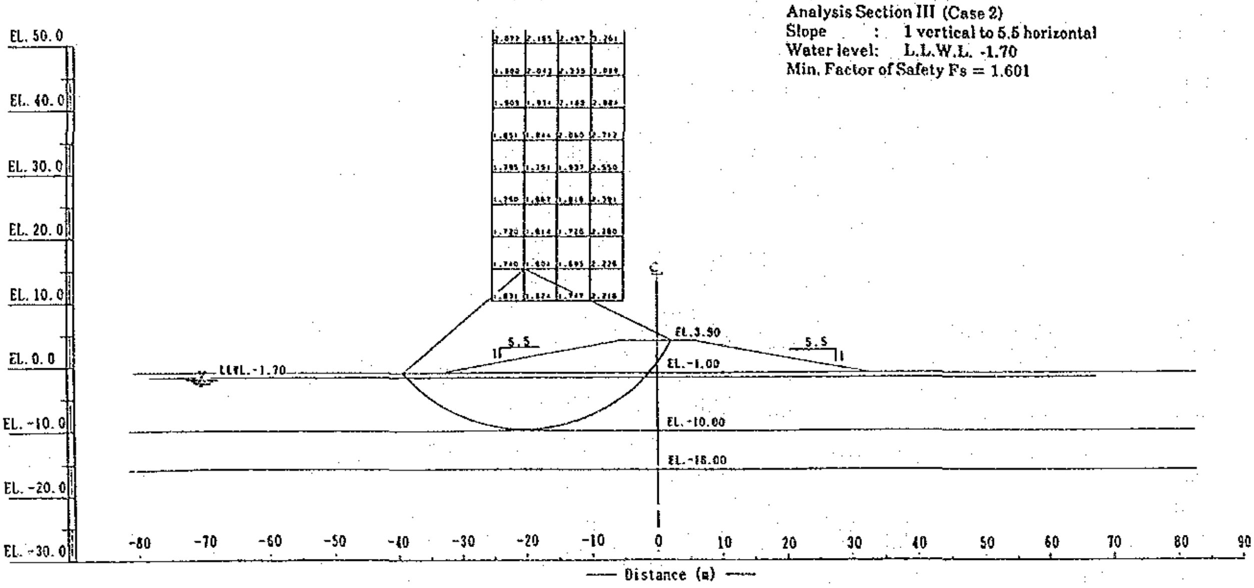
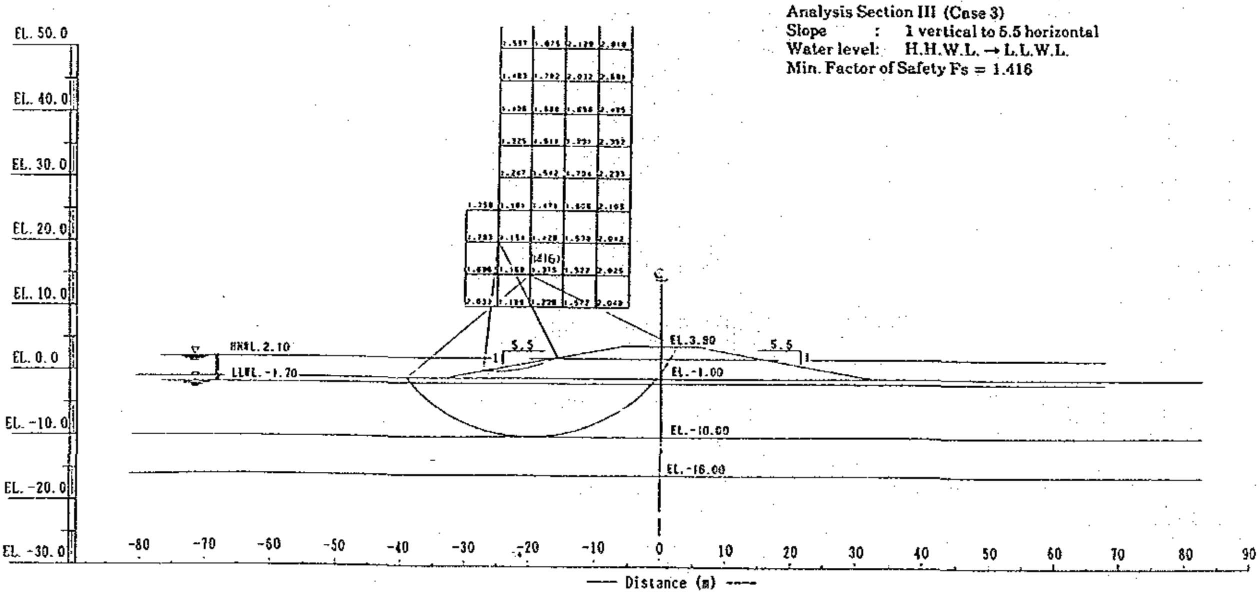


FIGURE 6 - 7(9) RESULTS OF STABILITY ANALYSIS
(UNIMPROVED SOFT FOUNDATION AT THE LEFT ABUTMENT FOUNDATION)

Analysis Section III (Case 2)
Slope : 1 vertical to 5.5 horizontal
Water level: L.L.W.L. -1.70
Min. Factor of Safety Fs = 1.601



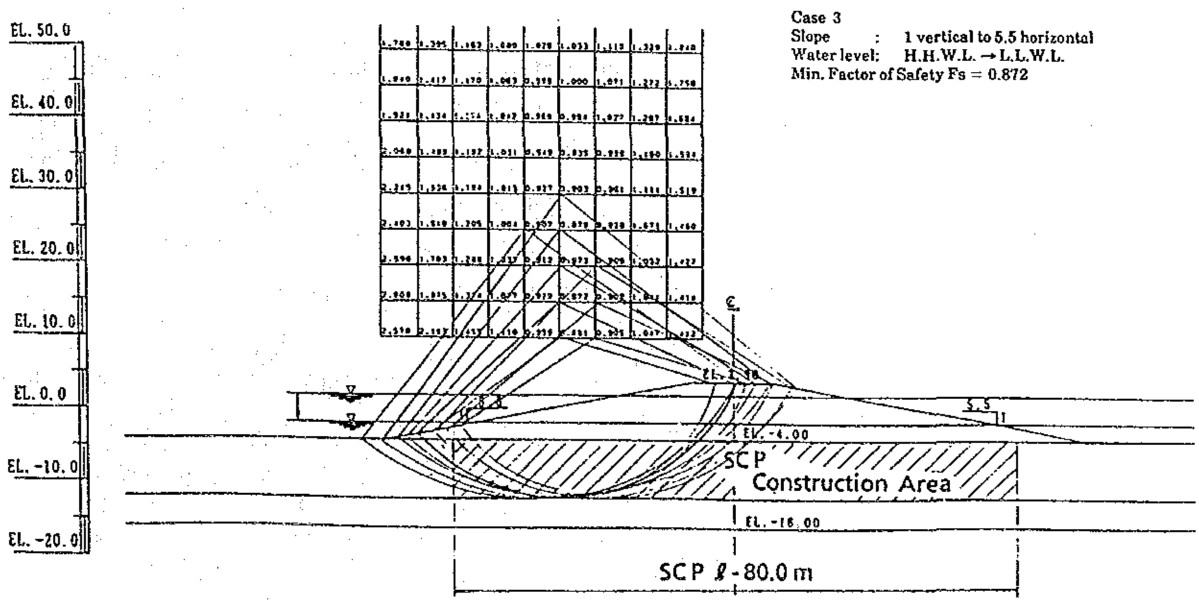
Analysis Section III (Case 3)
Slope : 1 vertical to 5.5 horizontal
Water level: H.H.W.L. → L.L.W.L.
Min. Factor of Safety Fs = 1.416



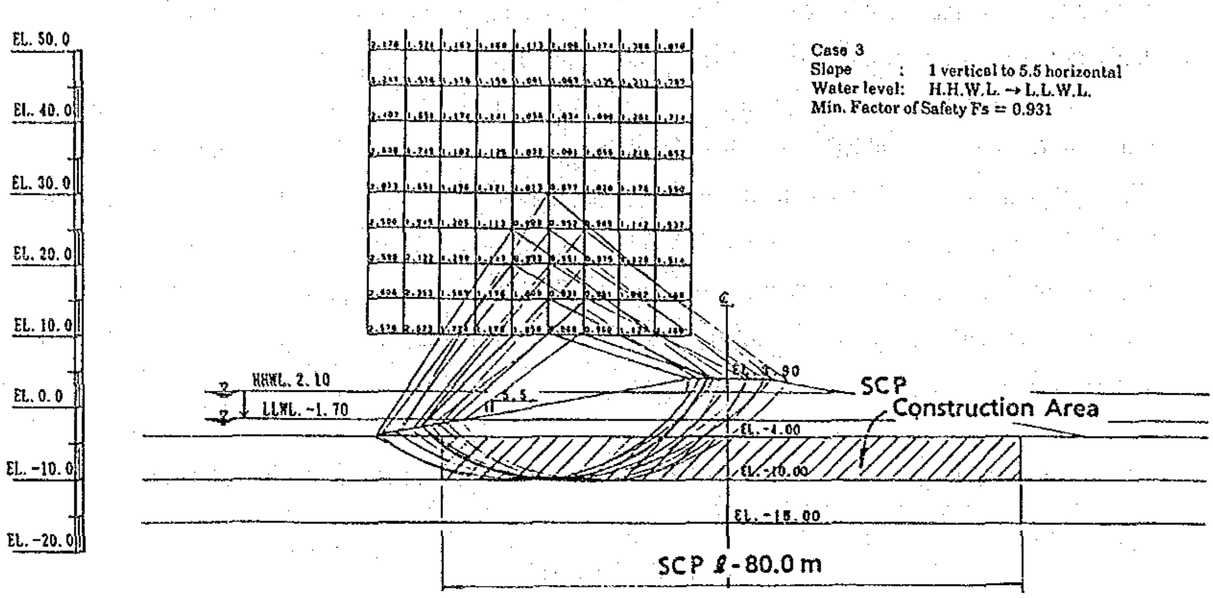
As clearly learned from Table 6-4, at the both abutments, in which case the elevation of dam foundation is higher than EL (-)2.0 m : or in which case the dam height is lower than 5.9 m, the dam embankment stability can be ensured even in conditions where the soft foundation will not be improved or will remain intact. Under such conditions, the extent of improvement of the soft foundation in profile of the closure dam is determined as the deeper part by EL (-)2.0 m of the foundation, while the one in cross section is determined to an extent 40 m in each direction the both up-and-downstream sides from the center of dam at the both abutments, in taking into account the passing line of the reasonable slip circle slice below the necessary safety factor as shown in Figure 6-8.

FIGURE 6-8 RESULT OF STABILITY ANALYSIS IN UNIMPROVED FOUNDATION

(RIGHT ABUTMENT)



(LEFT ABUTMENT)



Furthermore, the consolidation settlement of the closure dam under construction is estimated to shown below. In this estimate, settlement at both the abutments shall be estimated for the following three sections in view of embankment portions by location of foundation excavation ground.

RIGHT ABUTMENT FOUNDATION

Section	EL. of Foundation Excavation	Thickness of Soft Layers	Thickness of Embankment		
			Dry Part	Submerged Part	Total
I	EL (-)8.0 m	4.0 m	3.9 m	8.0 m	11.9 m
II	EL (-)4.0 m	8.0 m	3.9 m	4.0 m	7.9 m
III	EL 0.0 m	12.0 m	3.9 m	-	3.9 m

LEFT ABUTMENT FOUNDATION

Section	EL. of Foundation Excavation	Thickness of Soft Layers	Thickness of Embankment		
			Dry Part	Submerged Part	Total
I	EL (-)7.0 m	3.0 m	3.9 m	7.0 m	10.9 m
II	EL (-)4.0 m	6.0 m	3.9 m	4.0 m	7.9 m
III	EL (-)1.0 m	9.0 m	3.9 m	1.0 m	4.9 m

The results of calculation are shown in Table 6-5. And consolidation time (80% in consolidation degree) can be estimated as follows.

RIGHT ABUTMENT FOUNDATION

Section I	$t = \dfrac{Tu \cdot H^2}{Cu} = \dfrac{0.567 \times 400^2}{8.64} = 10{,}500 \fallingdotseq 28.8$ year
Section II	$t = \dfrac{0.567 \times 800^2}{8.64} = 42{,}000 \fallingdotseq 115.1$ year
Section III	$t = \dfrac{0.567 \times 1{,}200^2}{8.64} = 94{,}500 \fallingdotseq 258.9$ year

LEFT ABUTMENT FOUNDATION

Section I	$t = \dfrac{Tu \cdot H^2}{Cu} = \dfrac{0.567 \times 300^2}{8.64} = 5{,}906 \fallingdotseq 16.2$ year
Section II	$t = \dfrac{0.567 \times 600^2}{8.64} = 23{,}625 \fallingdotseq 64.7$ year
Section III	$t = \dfrac{0.567 \times 900^2}{8.64} = 53{,}156 \fallingdotseq 145.6$ year

Under the conditions, the consolidation settlement will have been not finished by the time of completion of the construction works.

2) Design of Sand Compaction Pile

The sand compaction piles proposed shall be 400 mm by casing diameter and 700 mm by sand compaction pile diameter. The studies of pile intervals are made as to the each analyzed section II in Figure 6-6, where the elevation of the both abutment foundations are at EL (-)4.0 m.

First of all, pile drilling intervals shall be roughly determined so that the consolidation degree can reach more than 80 percent for about one year from starting to completing of the works.

In this connection, the consolidation speed will not slow down when the replacement ratio is less than 15 percent, on the other hand, its will slow down the value of about 80 percent when the replacement ratio is about 20 percent. Because of this, the consolidation coefficient is taken as Cv = 4.8 ×

TABLE 6 - 5 (1) CALCULATION OF CONSOLIDATION SETTLEMENT (RIGHT ABUTMENT FOUNDATION)

Zone		Depth (m)	H *1 (m)	Z *2 (m)	γt, γsub *3 (t/m^2)	Pe *4 (tf/m^2)	Pz *5 (tf/m^2)	e_0 *6	e_1 *7	Sc *8 (cm)	Σ Sc *9 (cm)
Section I	Embankment	EL 3.9 ~ EL 0 m	3.9	1.95	1.50	-	2.93	-	-	-	-
		EL 0 ~ EL (-)4.0 m	4.0	5.9	0.80	-	7.45	-	-	-	-
		EL (-)4.0 ~ EL (-)8.0 m	4.0	9.9	0.80	-	10.65	-	-	-	-
	Soft Layer	EL (-)8.0 ~ EL (-)12.0 m	4.0	13.9	0.55	1.10	13.35	1.670	1.330	50.9	50.9
Section II	Embankment	EL 3.9 ~ EL 0 m	3.9	1.95	1.50	-	2.93	-	-	-	-
		EL 0 ~ EL (-)4.0 m	4.0	5.9	0.80	-	7.45	-	-	-	-
	Soft Layer	EL (-)4.0 ~ EL (-)8.0 m	4.0	9.9	0.55	1.10	10.15	2.110	1.705	52.1	52.1
		EL (-)8.0 ~ EL (-)12.0 m	4.0	13.9	0.55	3.30	12.35	1.595	1.350	37.8	89.9
Section III	Embankment	EL 3.9 ~ EL 0 m	3.9	1.95	1.50	-	2.93	-	-	-	-
	Soft Layer	EL 0 ~ EL (-)4.0 m	4.0	5.9	0.55	1.10	6.95	2.525	2.190	38.0	38.0
		EL (-)4.0 ~ EL (-)8.0 m	4.0	9.9	0.55	3.30	9.15	1.975	1.740	31.6	69.6
		EL (-)8.0 ~ EL (-)12.0 m	4.0	13.9	0.55	5.50	11.35	1.540	1.380	25.2	94.8

*1 Thickness of each layer *2 Estimated Depth of each layer
*3 γt : Wet density γsub : Submerged density *4 Initial stress (before loading)
*5 Stress in vertical direction after loading
*6 Initial void ratio *7 Void ratio after loading
*8 Consolidation settlement *9 Total consolidation settlement

TABLE 6 - 5 (2) CALCULATION OF CONSOLIDATION SETTLEMENT (LEFT ABUTMENT FOUNDATION)

Zone		Depth (m)	H *1 (m)	Z *2 (m)	γt, γsub *3 (t/m^2)	Pe *4 (tf/m^2)	Pz *5 (tf/m^2)	e_0 *6	e_1 *7	Sc *8 (cm)	Σ Sc *9 (cm)
Section I	Embankment	EL 3.9 ~ EL 0 m	3.9	1.95	1.50	-	2.93	-	-	-	-
		EL 0 ~ EL (-)4.0 m	4.0	5.9	0.80	-	7.45	-	-	-	-
		EL (-)4.0 ~ EL (-)7.0 m	3.0	9.4	0.80	-	10.25	-	-	-	-
	Soft Layer	EL (-)7.0 ~ EL (-)10.0 m	3.0	12.4	0.52	0.78	12.23	2.125	1.805	30.7	30.7
Section II	Embankment	EL 3.9 ~ EL 0 m	3.9	1.95	1.50	-	2.93	-	-	-	-
		EL 0 ~ EL (-)4.0 m	4.0	5.9	0.80	-	7.45	-	-	-	-
	Soft Layer	EL (-)4.0 ~ EL (-)7.0 m	3.0	9.4	0.52	0.78	9.83	2.465	2.210	22.1	22.1
		EL (-)7.0 ~ EL (-)10.0 m	3.0	12.4	0.52	2.34	11.39	2.070	1.845	22.0	44.1
Section III	Embankment	EL 3.9 ~ EL 0 m	3.9	1.95	1.50	-	2.93	-	-	-	-
		EL 0 ~ EL (-) 1.0 m	1.0	4.4	0.80	-	6.25	-	-	-	-
	Soft Layer	EL (-) 1.0 ~ EL (-)4.0 m	3.0	6.4	0.52	0.78	7.43	2.465	2.300	14.3	14.3
		EL (-)4.0 ~ EL (-)7.0 m	3.0	9.4	0.52	2.34	8.99	2.425	2.245	15.8	30.1
		EL (-)7.0 ~ EL (-)10.0 m	3.0	12.4	0.52	3.90	10.55	2.020	1.875	14.4	44.5

*1 Thickness of each layer *2 Estimated Depth of each layer
*3 γt : Wet density γsub : Submerged density *4 Initial stress (before loading)
*5 Stress in vertical direction after loading
*6 Initial void ratio *7 Void ratio after loading
*8 Consolidation settlement *9 Total consolidation settlement

10^{-3} cm^2/min (=6.0 × 10^{-3} cm^2/min × 0.8) in the right abutment foundation and 6.0 × 10^{-3} cm^2/min in the left abutment foundation, respectively.

* Cv = 6.0 × $10^{-3}$$cm^2$/min; the value is obtained from the results of soil mechanical test.

The relationship obtained for pile intervals and consolidation degree by the Barron's equation can be shown as follows.

Calculation of F(n) by Barron's Equation

①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪	⑫
d	de	dw	n	n^2	n^2	ln (n)	$3n^2$-1	$4n^2$	⑧/⑨	⑥×⑦	F (n)
(m)	(cm)	(cm)	②/③		(n^2-1)						⑪-⑩
1.1	124.3	70	1.776	3.15	1.465	0.574	8.45	12.6	0.670	0.841	0.171
1.2	135.6	〃	1.937	3.75	1.364	0.661	10.25	15.0	0.683	0.902	0.219
1.3	146.9	〃	2.099	4.41	1.293	0.741	12.23	17.6	0.695	0.958	0.263
1.4	158.2	〃	2.260	5.11	1.243	0.815	14.33	20.4	0.702	1.013	0.311
1.5	169.5	〃	2.421	5.86	1.206	0.884	16.58	23.4	0.709	1.066	0.357
1.6	180.8	〃	2.583	6.67	1.176	0.949	19.01	26.7	0.712	1.116	0.404
1.7	192.1	〃	2.744	7.53	1.153	1.009	21.59	30.1	0.717	1.163	0.446
1.8	203.4	〃	2.906	8.44	1.134	1.067	24.32	33.8	0.720	1.210	0.490

Calculation of U (Th)

(RIGHT ABUTMENT FOUNDATION)

①	⑬	⑭	⑮	⑯	⑰	⑱	⑲	
d	de^2	Cv×t	Th	-8Th	⑯/⑫	Exp ⑰	U	As*2
(m)		*1	⑭/⑬				1-⑱	
1.1	15450	2522.0	0.163	-1.304	-7.626	0.001	0.999	0.318
1.2	18387	〃	0.137	-1.096	-5.005	0.007	0.993	0.267
1.3	21580	〃	0.117	-0.936	-3.559	0.028	0.972	0.227
1.4	25027	〃	0.101	-0.808	-2.598	0.074	0.926	0.196
1.5	28730	〃	0.088	-0.704	-1.972	0.139	0.861	0.171
1.6	32689	〃	0.077	-0.616	-1.525	0.218	0.782	0.150
1.7	36902	〃	0.068	-0.544	-1.220	0.295	0.705	0.133
1.8	41372	〃	0.061	-0.488	-0.996	0.369	0.631	0.118

*1 Cv = 4.8 × 10^{-3} cm^2/min = 6.91 cm^2/day t= 12 month

*2 As: Replacement ratio

(LEFT ABUTMENT FOUNDATION)

① d (m)	⑬ de^2	⑭ Cv×t *1	⑮ Th ⑭/⑬	⑯ -8Th	⑰ ⑯/⑫	⑱ Exp ⑰	⑲ U 1-⑱	As*2
1.1	15450	3153.0	0.204	-1.632	-9.544	0.000	1.000	0.318
1.2	18387	〃	0.172	-1.376	-6.283	0.002	0.998	0.267
1.3	21580	〃	0.146	-1.168	-4.441	0.012	0.988	0.227
1.4	25027	〃	0.126	-1.008	-3.241	0.039	0.961	0.196
1.5	28730	〃	0.110	-0.880	-2.465	0.085	0.915	0.171
1.6	32689	〃	0.096	-0.768	-1.901	0.149	0.851	0.150
1.7	36902	〃	0.085	-0.680	-1.525	0.218	0.782	0.133
1.8	41372	〃	0.076	-0.608	-1.241	0.289	0.711	0.118

*1 $Cv = 6 \times 10^{-3}\ cm^2/min = 8.64\ cm^2/day$ t= 12 month

*2 As: Replacement ratio

Information provided by the above analysis shows that the pile intervals should be 1.5 m or less at the right abutment foundation and 1.6 m or less at the left abutment foundation, respectively, in which cases consolidation degree will exceed 80 percent.

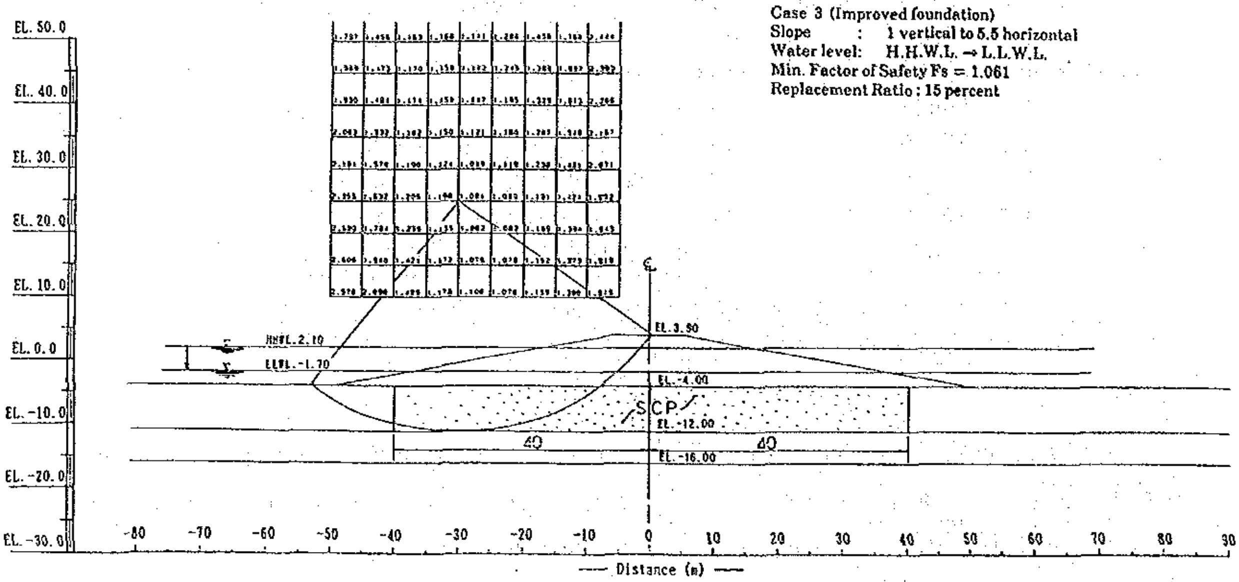
The ground after implementation of the sand compaction pile works, shall be treated as composite ground, and the results of the stability analysis made by the slip circle slice method are shown as follows, in taking the replacement ratio by 15.0 percent, 17.1 percent and 19.6 percent, respectively.

Replacement Ratio	Right Abutment Found	Left Abutment Found
0%	Fs = 0.872	Fs = 0.931
15.0%	Fs = 1.061	$\boxed{Fs = 1.102}$
17.1%	Fs = 1.083	-
19.6%	$\boxed{Fs = 1.104}$	-

As shown in the above table, the allowable safety factors can be satisfied on the condition that the replacement ratio are adopted by 19.6 percent (pile interval 1.4 m in square position) at the right abutment foundation and 15.0 percent (pile interval 1.6 m) at the left abutment foundation, respectively.

FIGURE 6 - 9 (1) RESULTS OF STABILITY ANALYSIS
(IMPROVED SOFT FOUNDATION AT THE RIGHT ABUTMENT FOUNDATION)

Case 3 (Improved foundation)
Slope : 1 vertical to 5.5 horizontal
Water level: H.H.W.L. → L.L.W.L.
Min. Factor of Safety Fs = 1.061
Replacement Ratio : 15 percent



Case 3 (Improved foundation)
Slope : 1 vertical to 5.5 horizontal
Water level: H.H.W.L. → L.L.W.L.
Min. Factor of Safety Fs = 1.083
Replacement Ratio : 17.1 percent

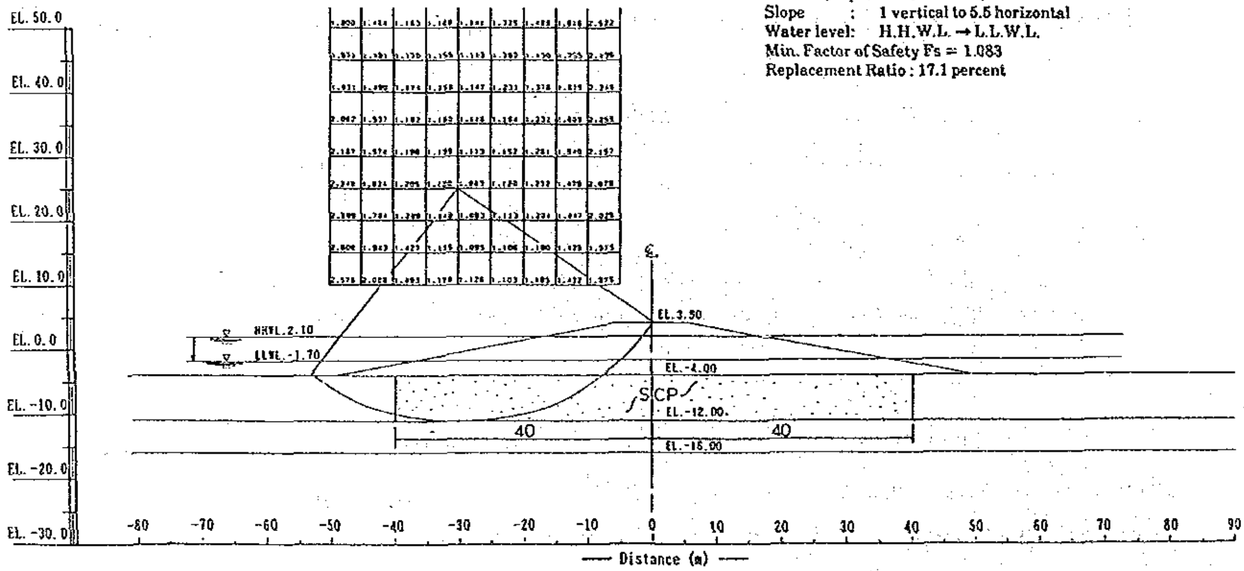
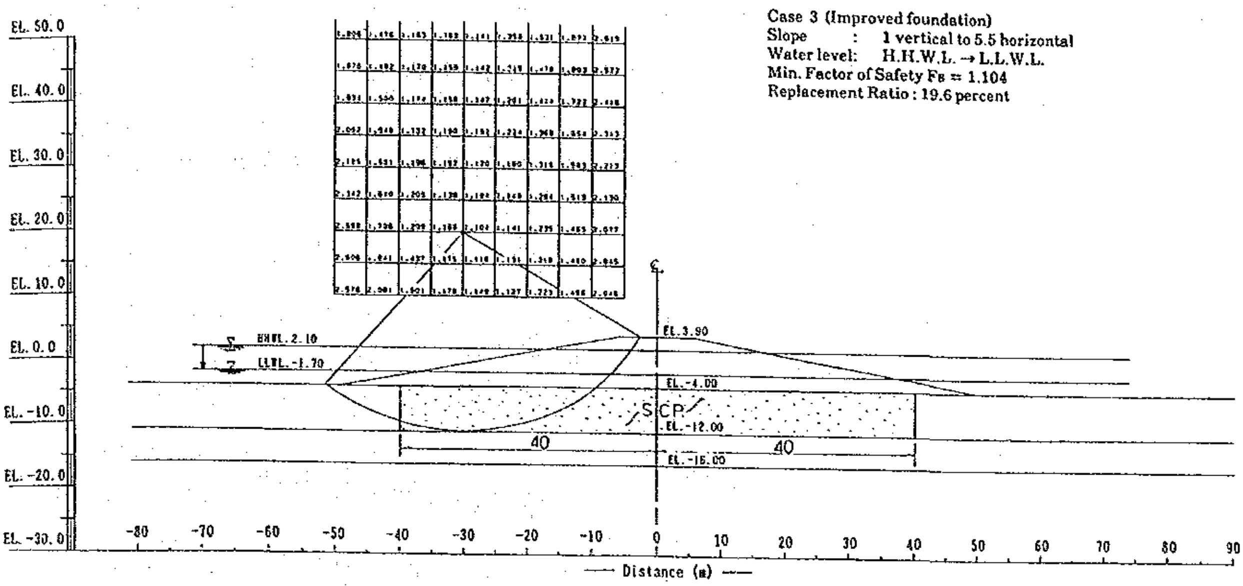
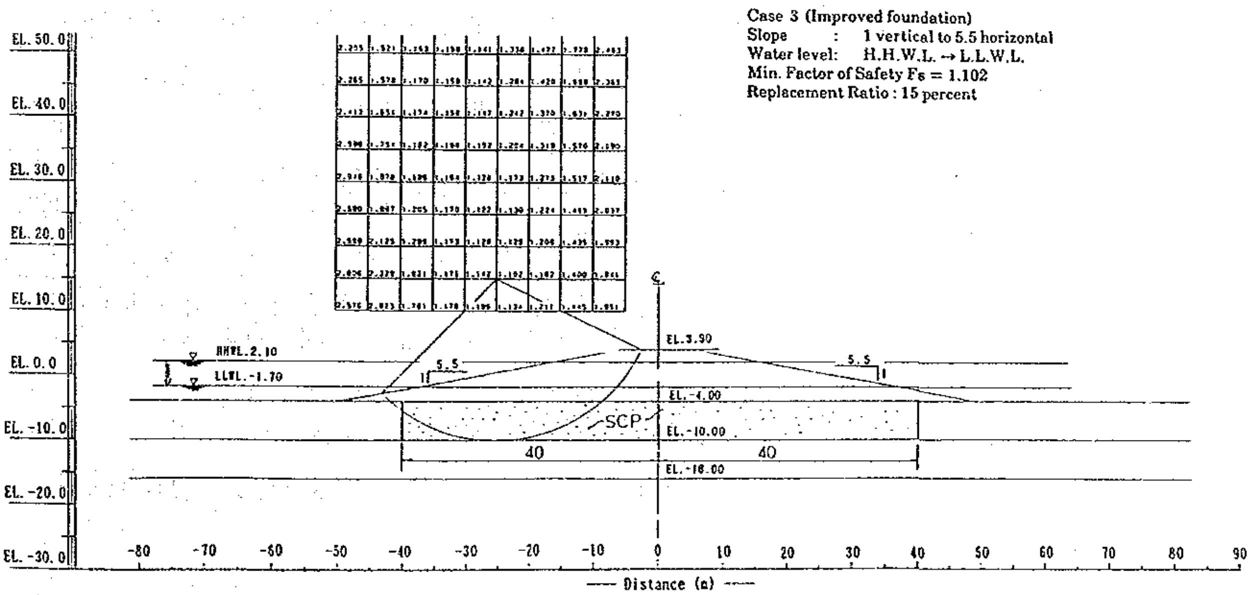


FIGURE 6 - 9 (2) RESULTS OF STABILITY ANALYSIS

(IMPROVED SOFT FOUNDATION AT THE RIGHT ABUTMENT FOUNDATION)



(IMPROVED SOFT FOUNDATION AT THE LEFT ABUTMENT FOUNDATION)



On the basis of the above results, the specifications of the sand compaction piles are determined as follows.

<RIGHT ABUTMENT FOUNDATION>

Conditions for execution of the Piling works :	Land implementation (ground elevation by EL 1.50 m)
Pile diameter :	Sand pile diameter by 700 mm (casing diameter by 400 mm)
Pile intervals :	1.4 m in square position
Replacement ratio :	19.6 percent
Extent for implementation:	Up to 40 m each for up-and-downsteam from dam axis (total length 80 m)
	Elevation of the excavation line: EL. (-)2.0 to (-)12.0 m
	Depth of construction: Up to EL.(-)12.0 m

<LEFT ABUTMENT FOUNDATION>

Conditions for execution of the Piling works :	Land implementation (ground elevation by EL 1.50 m)
Pile diameter :	Sand pile diameter by 700 mm (casing diameter by 400 mm)
Pile intervals :	1.6 m in square position
Replacement ratio :	15.0 percent
Extent for implementation:	Up to 40 m each for up-and-downsteam from dam axis (total length 80 m)
	Elevation of the excavation line: EL. (-)2.0 to (-)10.0 m
	Depth of construction: Up to EL.(-)10.0 m

6. 2 Dam Embankment

6. 2. 1 Stability Analysis (Dam Embankment with Borrow Area Material)

1) Outline of Design

The closure dam shall have a crest elevation of 3.9 m and a crest width of 12 m, taking into consideration the factor of the Bang Pakong River design flood water level of EL 2.40 m and road use of the dam crest after completion but excepting extra-banking of 0.3 m.

The closure dam shall be designed in symmetry with the dam axis, consisting mostly of an earthfill zone and partly of rockfill zone at the feet of the up-and-downstream slope for preventing the earth embankment materials from washout. Riprap works shall be provided on the earthfill zones for both up-and-downstream slopes for protecting embankment materials from erosion by waves and rainwater.

2) Determination of Embankment Slope

The respective design value as to embankment materials, which are compoesd of earthfill zone and rockfill zone, are devided as follows, based on the past data obtained in the similar materials in Japan.

Zone	Density			Shear Strength	
	γt (t/m^3)*1	γsat (t/m^3)*2	γsat (t/m^3)*3	C (tf/m^2)*4	ϕ (°)*5
Earthfill Zone	1.50	1.80	0.80	0	25
Rockfill Zone	1.80	2.20	1.20	0	35

*1 Wet density *2 Saturated density
*3 Submerged density *4 Cohesion *5 Friction angle

The proposed embankment slope shall be determined by the following equation for surface sliding.

$$F_s = \frac{\gamma sub}{\gamma sat} \cdot \frac{\tan\phi}{m} = \frac{0.8}{1.8} \cdot \frac{\tan 25^\circ}{0.182} = 1.139 > 1.10$$

Where : F_s = Allowable safety factor
γsat = Saturated density of embankment material (1.8 t/m^3)
γsub = Submerged density of embankment material (0.8 t/m^3)
ϕ = Internal friction angle of embankment materials (25°)
m = Grade of embankment slope (slope 1 : 5.5 = 0.182)

Consequently, the embankment slope of earthfill zone shall be 1 : 5.5, while that of rockfill zone shall be 1 : 2.0 by same equation for surface sliding.

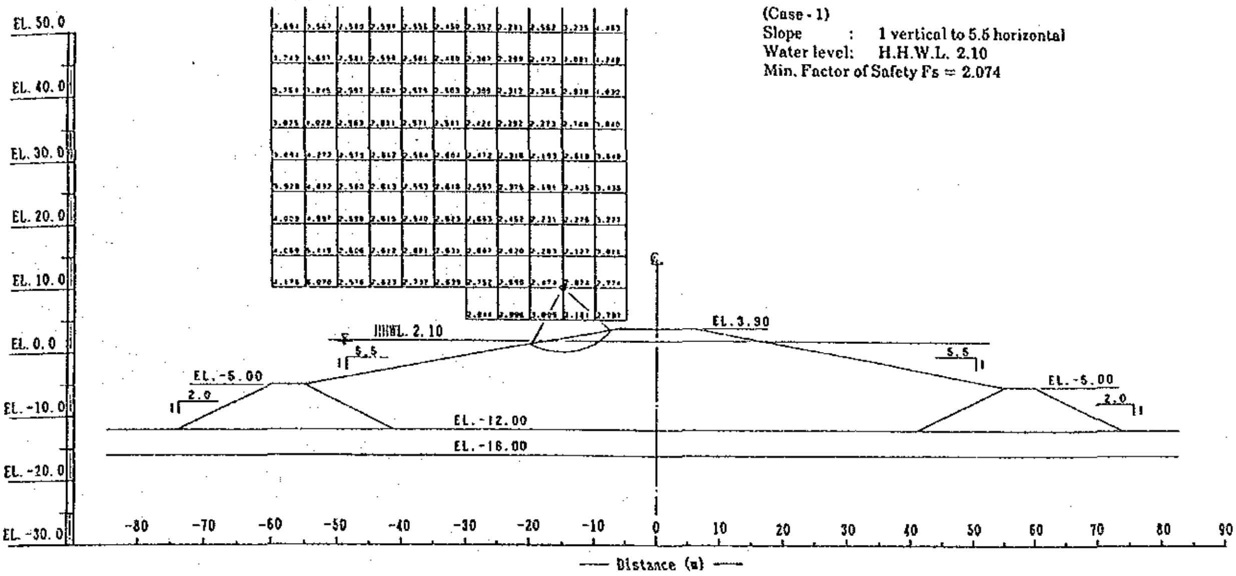
Based on the above mentioned typical dam section, stability analysis shall be carried out by slip circle slice method. The design values of the embankment and foundation are shown in Table 6-1, and the stability analysis results are shown in the Table 6-6 and Figure 6-10.

TABLE 6-6 RESULTS OF STABILITY ANALYSIS
(EMBANKMENTS WITH BORROW MATERIALS)

Case	Water Level	Calculation		
		Safety Factor		Allowable Safety Factors
Case 1	Constant W.L. (H.H.W.L.)	2.074	>	1.20
Case 2	〃 W.L. (L.L.W.L.)	2.076	>	1.20
Case 3	Drawdown W.L. (H.H.W.L.) → (L.L.W.L.)	1.120	>	1.10

As clarified in the above table, the safety factors are more than the allowable safety factor in any cases.

FIGURE 6 - 10(1) RESULTS OF STABILITY ANALYSIS (EMBANKMENT WITH BORROW MATERIALS)



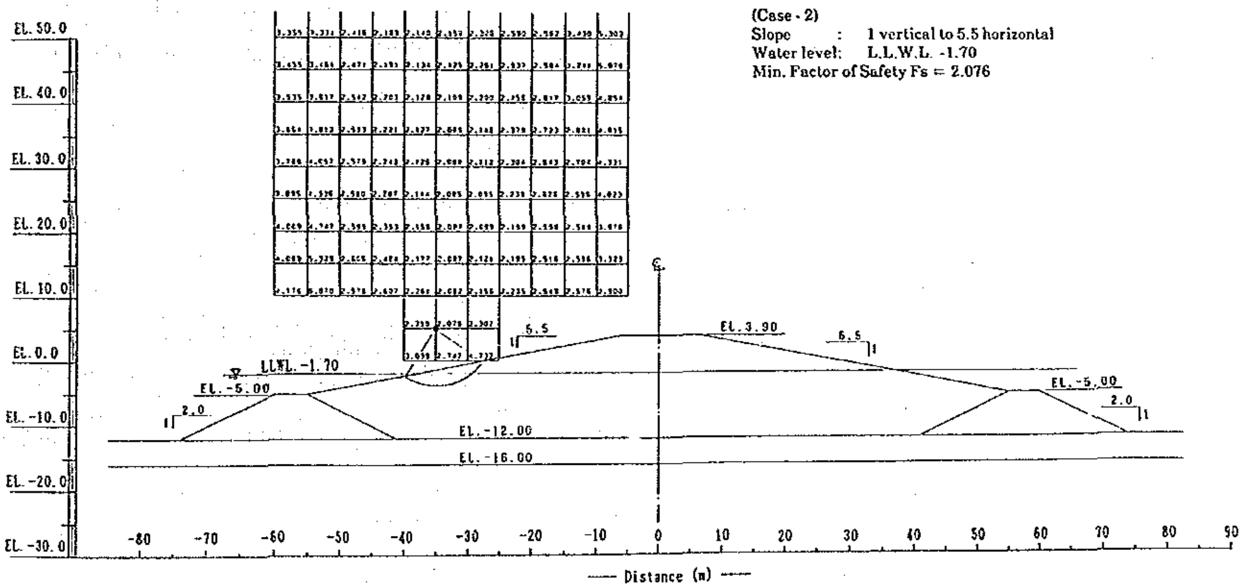
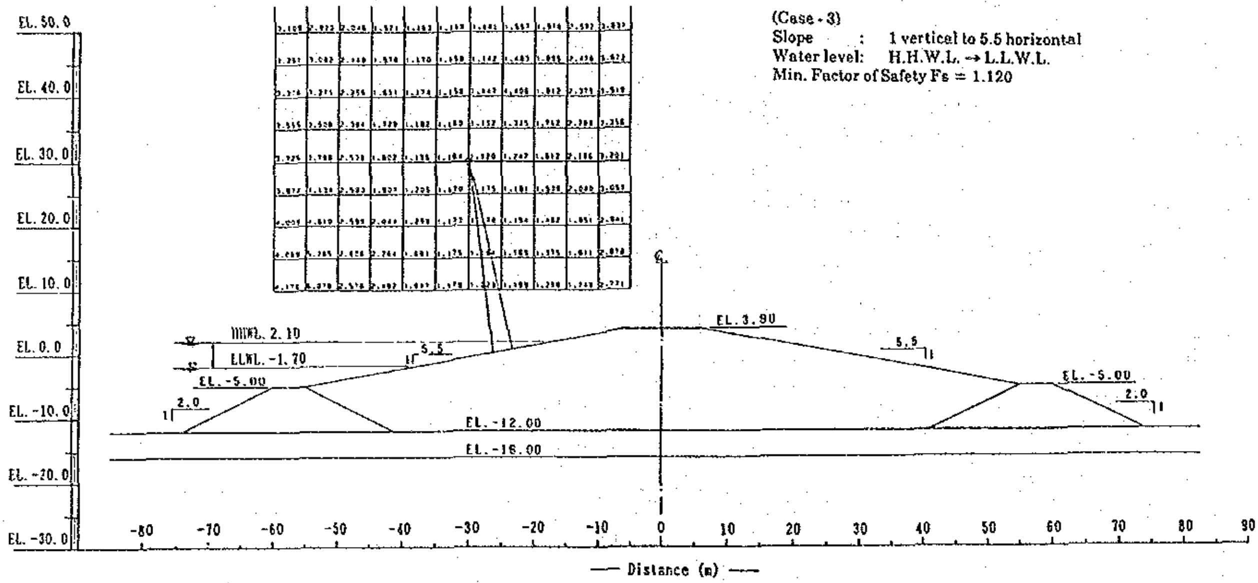
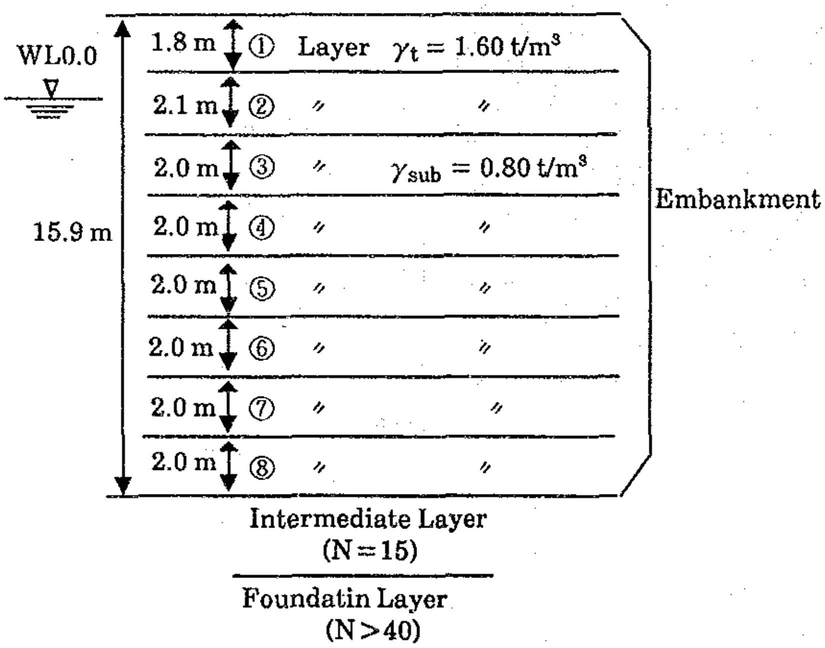


FIGURE 6 - 10(2) RESULTS OF STABILITY ANALYSIS
(EMBANKMENT WITH BORROW MATERIALS)



3) Calculation of Consolidation Settlement

Consolidation settlement to be caused in construction of the dam embankment shall be calculated by the method with the embankment divided into eight layers, and the model to be employed for the purpose can be shown as follows.



- Calculation of vertical stress

Layer	Depth *1 (m)	Z *2 (m)	H *3 (m)	$\gamma t, \gamma sub$*4 (t/m^3)	H×γt, γsub (tf/m^2)	P *5 (tf/m^2)
①	0.0 ~ 1.8	0.9	1.8	1.50	2.70	1.35
②	1.8 ~ 3.9	2.85	2.1	1.50	3.15	4.28
③	3.9 ~ 5.9	4.9	2.0	0.80	1.60	6.65
④	5.9 ~ 7.9	6.9	2.0	0.80	1.60	8.25
⑤	7.9 ~ 9.9	8.9	2.0	0.80	1.60	9.85
⑥	9.9 ~ 11.9	10.9	2.0	0.80	1.60	11.45
⑦	11.9 ~ 13.9	12.9	2.0	0.80	1.60	13.05
⑧	13.9 ~ 15.9	14.9	2.0	0.80	1.60	14.65

*1 Depth of each layer
*2 Estimated depth of each layer
*3 Thickness of each layer
*4 γt : Wet density γsub : Submerged density
*5 Effective pressure after loading

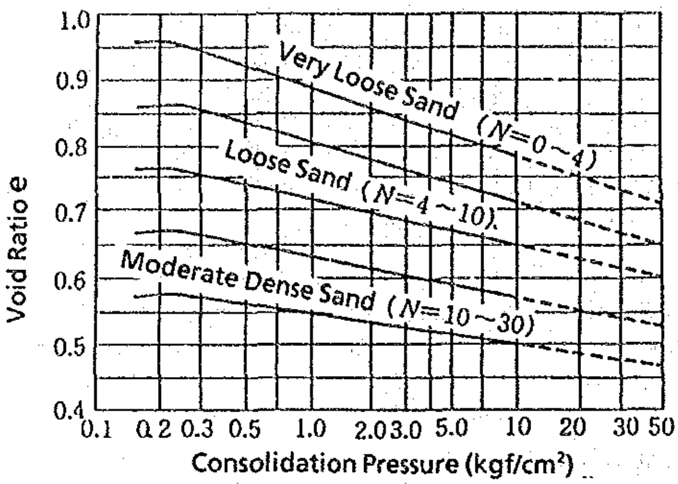
- Calculation of consolidation settlement

Layer	H (m)	Pz *1 (tf/m^2)	e_0 *2	e_1 *3	Sc *4 (cm)	Σ Sc *5 (cm)
①	1.8	1.35	0.960	0.960	0	0
②	2.1	1.58	0.960	0.930	3.2	3.2
③	2.0	0.80	0.960	0.910	5.1	8.3
④	2.0	0.80	0.960	0.890	7.1	15.4
⑤	2.0	0.80	0.960	0.890	7.1	22.5
⑥	2.0	0.80	0.960	0.880	8.2	30.7
⑦	2.0	0.80	0.960	0.880	8.2	38.9
⑧	2.0	0.80	0.960	0.870	9.2	48.1

*1 Initial stress (before loading)
*2 Initial void ratio
*3 Void ratio after loading
*4 Consolidation settlement
*5 Total settlement

In the above calculation, e_0 as initial void ratio and e_1 as void ratio after embankment are adopted in their values as very loose sand materials into the following figure.

FIGURE 6-11 RELATIONSHIP BETWEEN CONSOLIDATION PRESSURE AND VOID RATIO (BY B.K. HOUGH)



The settlement of the intermediate layer shall be calculated by the following equation applied to the elastic models.

$$S = (1 - \mu^2) \cdot \frac{P}{D} \cdot H$$

$$= (1 - 0.4^2) \times \frac{15.45}{1000} \times 4.0$$

$$= 0.05 \text{ m}$$

Where: S = Consolidation settlement (m)
μ = Poisson ratio (0.4)
P = Load strength (15.45 ton/m^2)
D = Deformation coefficient of foundation (1000 tf/m^2)
H = Thickness of objective layer (4.0 m)

Therefore, the settlement taking place in construction the dam embankment is estimated at 48 cm for dam body and 5 cm for foundation to make a total settlement of 53 cm. And, since the materials to be used for the dam body are sandy soils with higher consolidation coefficient, the settlement of the said parts is considered to be finished mostly within a construction period for the residual settlement not to give any adverse effects to the dam body.

4) Riprap Works for Embankment Slope

Thickness of the proposed riprap works shall be obtained by the following Hudson's equation.

$$
\begin{aligned}
W &= \frac{\gamma t H^3}{Kp\,(Gs-1)^3 \cot \alpha} \\
&= \frac{1.80 \times 1.00^3}{2.0 \times (2.60-1)^3 \cot 10.3^\circ} \\
&= \frac{1.80}{45.06} \\
&= 0.040\ t
\end{aligned}
$$

Where:
- W = Weight of rocks (t)
- γt = Unit weight of riprap materials (1.80 tf/m^3)
- H = Total wave height (1.00 m by S.M.B. Method)
- Gs = Specific gravity (2.60)
- α = Angle of slop against water surface (10.3°)

In the above conditions, weight per piece of riprap materials will be 40 kg/piece. Furthermore, based on the results of following calculation, riprap materials are required with 10 to 50 cm diameter in taking the shape of rocks as sphere,

$$
\begin{aligned}
D50 &= 1.24\sqrt[3]{\frac{W50}{Gs}} \\
&= 1.24\sqrt[3]{\frac{0.040}{2.60}} \\
&\doteqdot 31\ cm
\end{aligned}
$$

The riprap thickness shall be about 70 cm almost doubled by the required riprap diameter, in taking into consideration the submerged construction.

In this connection , the sand and gravel bedding zone in thickness 30 cm shall be provided between the two zones of the riprap and earth fill zone, so that earth fill materials can be protected form washout, because the riprap and earth fill materials are different in their grain size at the 50 percent ratio on the grain size distribution curve as 30 cm and 0.3 mm, respectively. Consequently, the protection zone on the surface of dam body including sand and gravel bedding zone will be total 1.0 m in thickness.

* The materials of sand and gravel bedding zone shall be selected by grain size from 1.0 to 30.0 mm in diameter.

6.2.2 Stability Analysis (Dam Embankment with Diversion Canal Excavation Material)

This chapter deals with the results of study, including consolidation analysis and stability analysis, for the case with of dam embankment with excavtion materials deeper than 8.0 m at the diversion canal site.

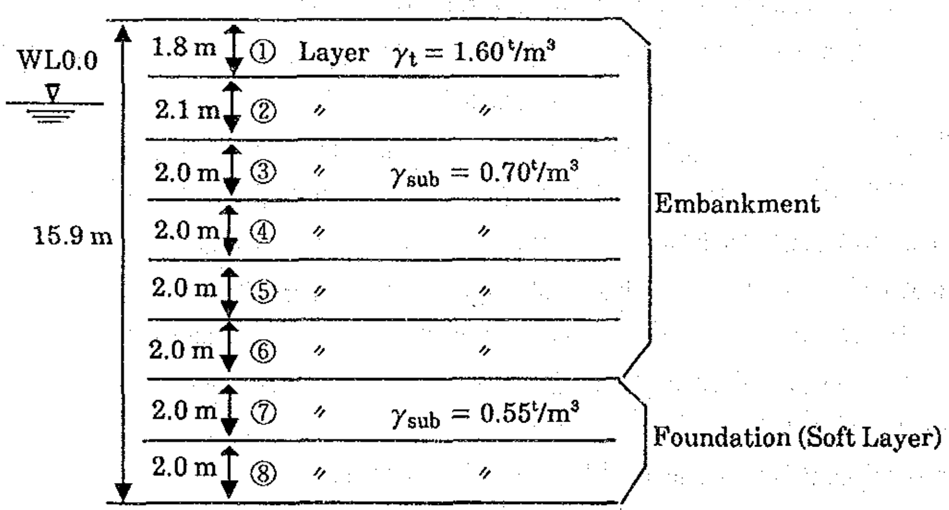
1) In the Case that the Excavated Materials are appropriated to Embankment Materials without Foundation Improvement

The results of consolidation analysis and stability analysis in the case that the materials excavated at the diversion canal site appropriated without foundation improvement are shown as follows.

- Consolidation Analysis

Since the soft layers found at the foundation are deemed similar in their cohesive soil and strength to the embankment materials, excavation shall be made only one-meter deep from the ground surface and the deeper materials shall remain intact.

The calculation of the consolidation settlement can be shown in the following figure in the way to divide the embankment portion into six and the foundation soft layers into two.



· Calculation of vertical stress

	Layer	Depth *1 (m)	Z *2 (m)	H *3 (m)	$\gamma t \cdot \gamma_{sub}$ *4 (t/m^3)	$H \times \gamma t, \gamma sub$ (t/m^2)	P *5 (tf/m^2)
Embankment	①	0.0 ~ 1.8	0.9	1.8	1.60	2.88	1.44
	②	1.8 ~ 3.9	2.85	2.1	1.60	3.36	4.56
	③	3.9 ~ 5.9	4.9	2.0	0.70	1.40	6.94
	④	5.9 ~ 7.9	6.9	2.0	0.70	1.40	8.34
	⑤	7.9 ~ 9.9	8.9	2.0	0.70	1.40	9.74
	⑥	9.9 ~ 11.9	10.9	2.0	0.70	1.40	11.14
Soft Layer	⑦	11.9 ~ 13.9	12.9	2.0	0.55	1.10	12.39
	⑧	13.9 ~ 15.9	14.9	2.0	0.55	1.10	13.49

*1 Depth of each layer   *2 Estimated depth of each layer
*3 Thickness of each layer   *4 γt : Wet density. γ_{sub}: Submerged Density
*5 Effective pressure after loading

· Calculation of consolidation settlement

	Layer	H (m)	Cc *1	e_o *2	Pz *3 (tf/m^2)	ΔP *4 (tf/m^2)	Logα *5	e_1 *6	Sc *7 (cm)	ΣSc *8 (cm)
Embankment	①	1.8	0.3	1.650	1.440	-	-	-	-	-
	②	2.1	0.3	1.650	1.680	2.88	0.434	-	10.3	10.3
	③	2.0	0.3	1.650	0.700	6.24	0.996	-	22.5	32.8
	④	2.0	0.3	1.650	0.700	7.64	1.076	-	24.3	57.2
	⑤	2.0	0.3	1.650	0.700	9.04	1.143	-	25.9	83.0
	⑥	2.0	0.3	1.650	0.700	10.44	1.202	-	27.2	110.2
Soft Layer	⑦	2.0	-	1.710	0.550	-	-	1.345	26.9	137.1
	⑧	2.0	-	1.650	1.650	-	-	1.330	24.2	161.3

*1 Compression index   *2 Initial void ratio
*3 Initial stress (before loading)   *4 Increased stress with loading (P - Pz)
*5 $\alpha = (Pz + \Delta P)/Pz$ *6 Yoid ratio after loading
*7 Consolidation settlement   *8 total settlement

The consolidation settlement, therefore, will be about 160 cm.

- Calculation of consolidation settlement

Condition: Single drainage condition (No sand mat is provided on the soft layers. Drainage distance H = 10.5 m)*1

U (%) *2	10	20	30	40	50	60	70	80	90
Sc (cm)*3	16.1	32.3	48.4	64.5	80.6	96.8	112.9	129.0	145.1
Tv*4	0.008	0.031	0.071	0.126	0.197	0.287	0.403	0.567	0.848
t (day)	1,025	3,971	9,094	16,139	25,234	36,762	51,621	72,627	108,621
t (year)*5	2.8	10.9	24.9	44.2	69.1	100.7	141.4	199.0	297.6

*1 : H : $h_1 (Cvf/Cve)^{0.5} + h_2 (Cvf/Cve)^{0.5} = 11.9 \times (6\times10^{-3}/2\times10^{-2})^{0.5} + 4.0 = 10.5$

H : Corrected thickness h_1 : Embankment thickness

h_2 : Foundation thickness   Cvf : Consolidation coefficient of foundation

Cve: Consolidation coefficient of embankment

*2 Consolidation degree   *3 Consolidation settlement

*4 Time factor   *5 Consolidation time

Conditions: Double drainage condition (Two meter thick sand mat is provided on the soft layers).

- Part of embankment (Drainage distance H = 4.95 m)

U (%)	10	20	30	40	50	60	70	80	90
Sc (cm)	8.3	16.6	24.9	33.2	41.5	49.8	58.1	66.4	74.7
Tv	0.008	0.031	0.071	0.126	0.197	0.287	0.403	0.567	0.848
t (day)	68	264	604	1,072	1,676	2,442	3,429	4,824	7,215
t (year)	0.2	0.7	1.7	2.9	4.6	6.7	9.4	13.2	19.8

- Part of embankment (Drainage distance H = 4.0 m)

U (%)	10	20	30	40	50	60	70	80	90
Sc (cm)	5.1	10.2	15.3	20.4	25.5	30.7	35.8	40.9	46.0
Tv	0.008	0.031	0.071	0.126	0.197	0.287	0.403	0.567	0.848
t (day)	148	574	1,315	2,333	3,648	5,315	7,463	10,500	15,704
t (year)	0.4	1.6	3.6	6.4	10.0	14.6	20.4	28.8	43.0

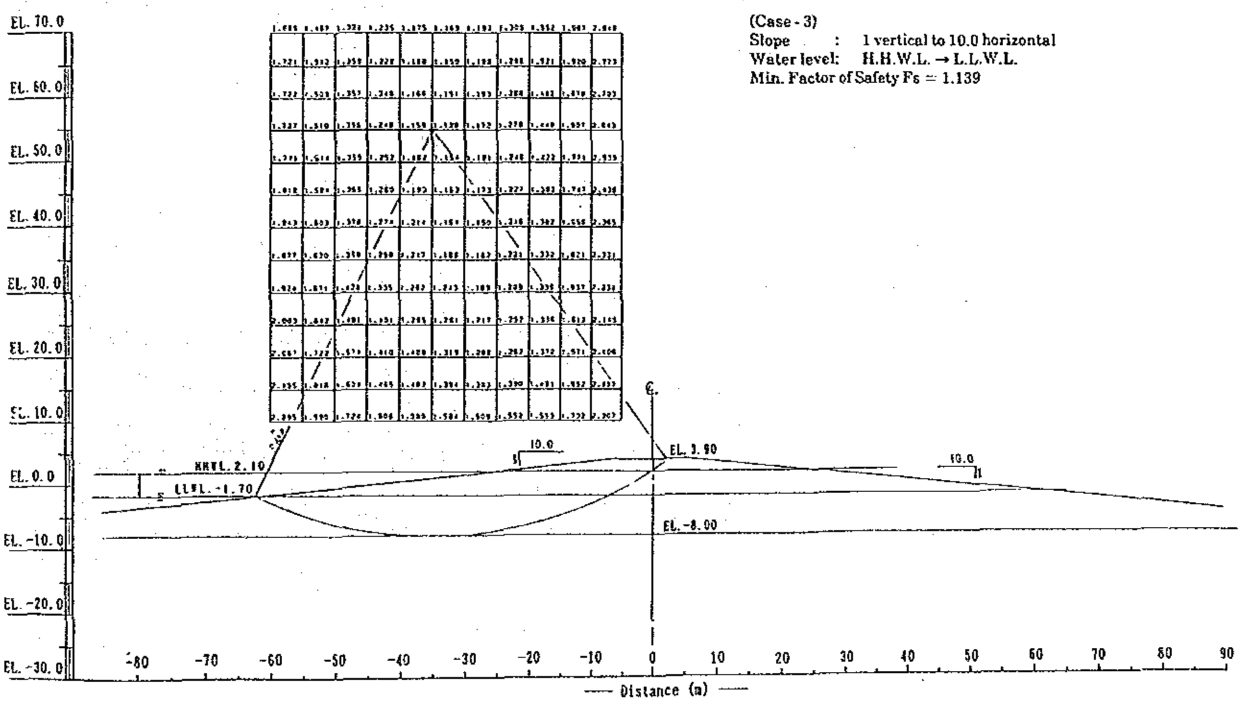
Therefore, the time required for consolidation settlement to reach consolidation degree 80% will be about 199 years by single drainage condition, while about 29 years by double drainage condition.

- Stability analysis

In this case, the gentle slope at 1:10 is necessary, in condition of FS = 1.062 for the slope at 1:9.0 and FS = 1.139 for the slope at 1:10.0 under the condition of rapid drawdown. (refer to Figure 6-12)

Under such conditions, the excavated materials at the diversion canal site cannot be used for embankment in considering the fact that much more time and heavier settlement will be inevitably employed without making foundation improvement works.

FIGURE 6-12 RESULT OF STABILITY ANALYSIS
(THE CASE OF EMBANKMENT WITHOUT FOUNDATION IMPROVEMENT)



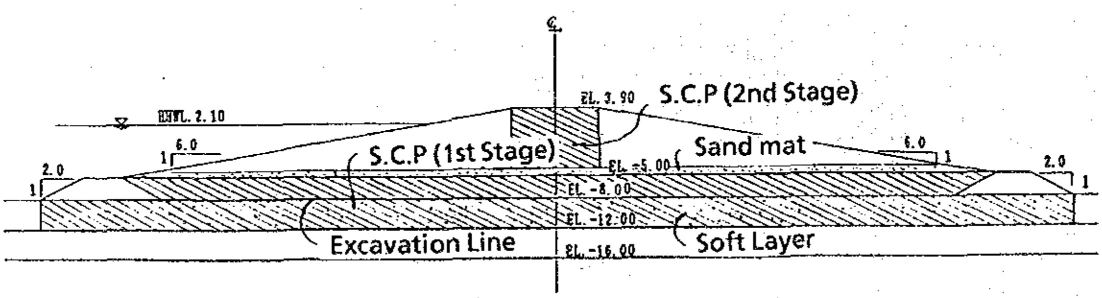
2) In the Case that the Excavated Material are appropriated to Embankment Materials with Foundation Improvement

As mentioned before, the study was made on availability of the materials excavated at the diversion canal site for embankment materials without foundation improvement to reject them for the purpose, while hereinafter on availability of the same materials to be appropriated with foundation improvement works.

Since the improvement works aim to stabilize the embankment foundation as well as to accelerate consolidation settlement, the sand compaction pile method shall be made to increase bearing capacity and shear strength in taking into consideration its drain effects and works as composite ground.

The sand compaction pile method shall be implemented in such a manner that as the first stage when the embankment works will come to EL(-) 5.0 m, sand mat of one meter thickness shall be provided and the works shall be performed through both embankment and foundation portion in submerged works. And in the second stage, the works shall be proceed from embankment crest to sand mat surface in dry works.

FIGURE 6-13 IMPROVED PLAN BY COMPACTION PILE METHOD
(WITH EXCAVATED MATERIALS APPROPRIATED)



* The sand compaction pile method requires some flat or very gently sloped area in its implementation, and consequently, the area around slope position at up-and-downstream will be difficult in implementation of the foundation improvement works.

* The result of stability analysis has found that the slope of 1:5.5 cannot satisfy the safety factor required for stability of the embankment. In this respect, the necessary slope grade shall be designed at 1:6.0.

The designed specifications of the proposed sand compaction pile shall be determined by the Barron's equations in the condition that the consolidation settlement should reach about 80% of consolidation degree until the completion of the construction works and these are shown in Table 6-7.

TABLE 6 - 7 SPECIFICATIONS OF SAND COMPACTION PILE

Items	1st Stage (Const. on the Water)	2nd Stage (Const. on the Land)
Casing dia.	1,500 mm	400 mm
Sand pile dia.	2,000 mm	700 mm
Pile intervals	2.4 mm	1.4 mm
Pile pattern	Square position	Square position
Replacement ratio	55%	19.6%

Taking ground condition after implementation of sand compaction pile into consideration, stability analysis shall be made by the slip circle slice method. In this respect, the embankment and foundation will be improved to show the characteristic features as composite ground and the design values are shown in Table 6 - 8.

TABLE 6-8 DESIGN VALUES FOR STABILITY ANALYSES

<table>
<tr><th colspan="3" rowspan="2">Zone</th><th colspan="3">Density</th><th colspan="2">Shear Strength</th></tr>
<tr><th>γt (t/m³)*1</th><th>γsat (t/m³)*2</th><th>γsub (t/m³)*3</th><th>C (tf/m²)*4</th><th>ø(°)*5</th></tr>
<tr><td rowspan="4">Embankment</td><td colspan="2">Embankment (Unimprovement)</td><td>1.60</td><td>1.70</td><td>0.70</td><td>Cu=1.0 (P≦5tf/m²)*6
Cu=1.0+0.2(P-5)U (P>5tf/m²)</td><td>0</td></tr>
<tr><td colspan="2">Embankment (improvement by SCP)</td><td>1.80</td><td>2.00</td><td>1.00</td><td>0</td><td>30.0</td></tr>
<tr><td rowspan="2">Composite ground</td><td>Construction on the water (As = 0.55)</td><td>1.71</td><td>1.87</td><td>0.87</td><td>0.45 Cu</td><td>17.6</td></tr>
<tr><td>Construction on the land (As = 0.196)</td><td>1.64</td><td>1.76</td><td>0.76</td><td>0.80 Cu</td><td>6.5</td></tr>
<tr><td rowspan="2">Soft Layer</td><td colspan="2">Soft layer (unimprovement)</td><td>1.55</td><td>1.55</td><td>0.55</td><td>Cu = 1.5 (P≦7.5 tf/m²)
Cu = 1.5 + 0.2 (P-7.5)U (P >7.5 tf/m²)</td><td>0</td></tr>
<tr><td colspan="2">Composite ground (As = 0.55)</td><td>1.69</td><td>1.80</td><td>0.80</td><td>0.45 Cu</td><td>17.6</td></tr>
</table>

*1 Wet density *2 Saturated density *3 Submerged density *4 Cohesion
*5 Friction angle *6 P: Effective load of objective ground
U: Consolidation degree of objective ground

The results of stability analyses are shown in Table 6-9 and Figure 6-14.

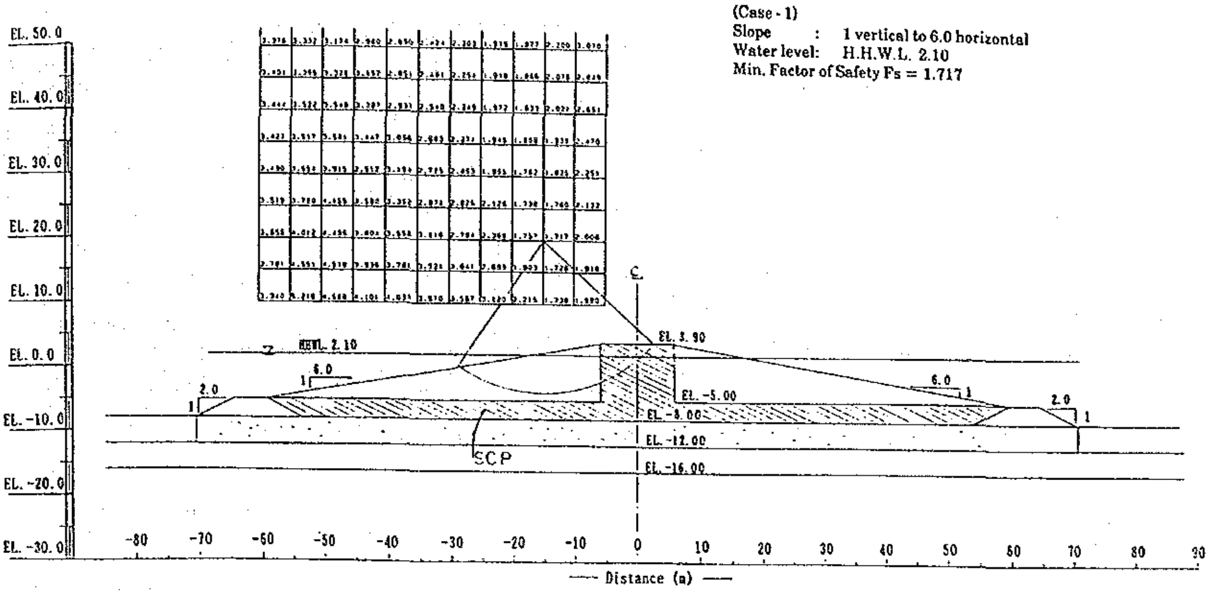
TABLE 6-9 RESULTS OF STABILITY ANALYSIS (EMBANKMENT WITH EXCAVATED MATERIALS ON THE CONDITON WITH SOIL IMPROVEMENT)

Case	Water Level	Safety Factor		Allowable Safety Factor
Case 1	Constant W.L.(H.H.W.L) *1	1.717	>	1.20
Case 2	Constant W.L.(L.L.W.L)*2	1.232	>	1.20
Case 3	Drawdown W.L. (H.H.W.L →L.L.W.L)	1.183	>	1.10

*1 H.H.W.L 2.10 m *2 L.L.W.L -1.70 m

The above scale of this method, therefore, is judged sufficient to ensure the stability of the embankment.

FIGURE 6 - 14(1) RESULTS OF STABILITY ANALYSIS (EMBANKMENT WITH EXCAVATED MATERIALS ON THE CONDITON WITH SOIL IMPROVEMENT)



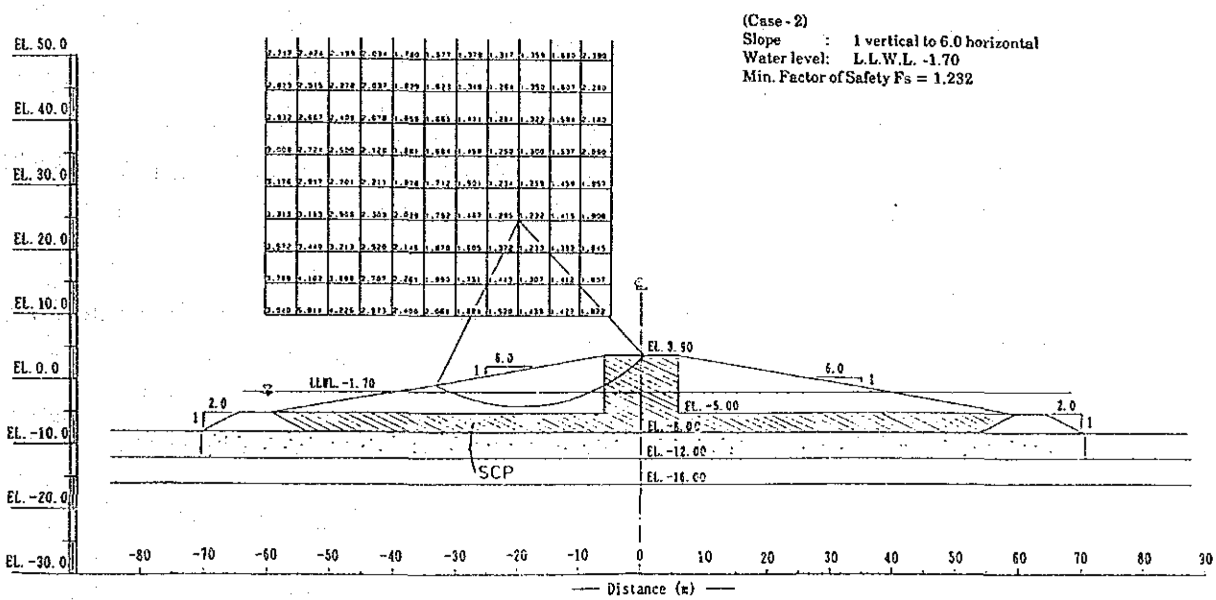
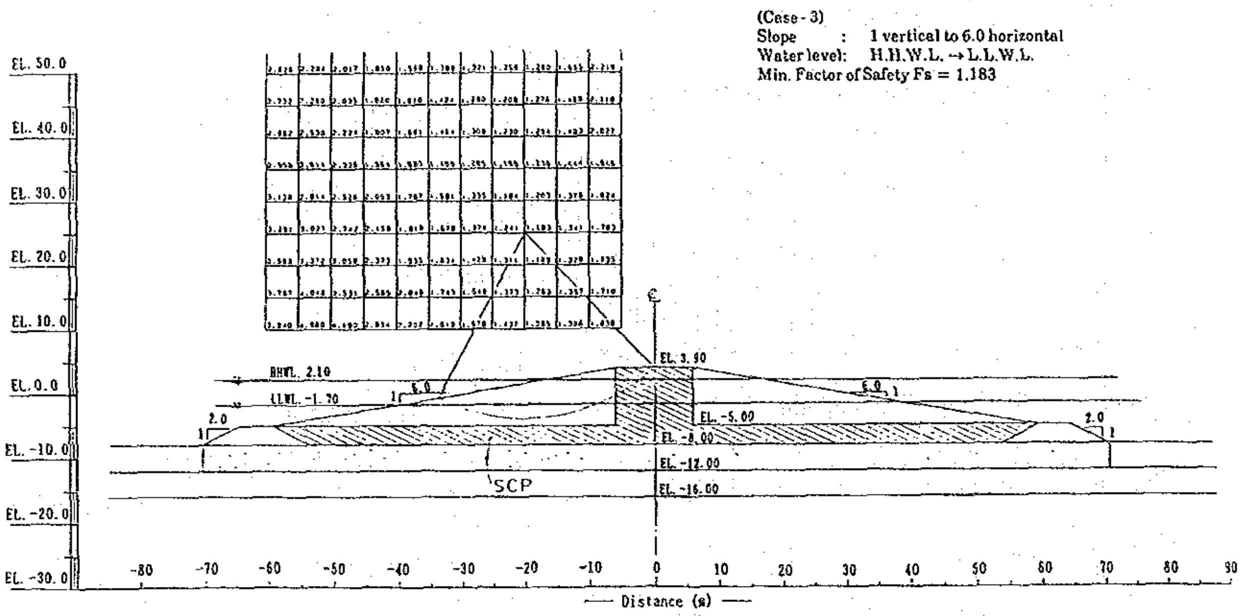


FIGURE 6 - 14(2) RESULTS OF STABILITY ANALYSIS
(EMBANKMENT WITH EXCAVATED MATERIALS ON THE CONDITON
WITH SOIL IMPROVEMENT)



6. 2. 3 Determination of Embankment Materials

For selection of embankment materials, the economic studies were made on the following two plans, and

① Borrow materials are used.
② Excavated materials at the diversion canal site deeper than 8.0 m are appropriated.

The results of economic comparison of the above two are shown in Table 6-10.

TABLE 6-10 COMPARISON OF CONSTRUCTION COST WITH BORROW MATERIALS AND APPROPRIATION OF EXCAVATED MATERIALS

Work Item	Unit Price (฿)	① The Case of use with Borrow Material		② The Case of use with Excavated Materials	
		Quantity	Total	Quantity	Total
1. Foundation Excavation	80	845.0 m^3	67,600	147.0 m^3	11,760
2. Embankment					
2. 1 Riprap	820	100.4 m^3	82,328	109.4 m^3	89,708
2. 2 Embank. with Borrow Material	120	1,132.1 m^3	135,852	-	-
2. 3 Embank. with Excavation Materials	50	-	-	712.6 m^3	35,630
2. 4 Embank. with Rock Materials	700	2,660 m^3	186,200	110.0 m^3	77,000
3. Sand Compaction Pile Works Pile I					
3. 1 Sand Mat	250	-	-	1,140 m^3	28,500
3. 2 Sand Compaction Pile	2,100 (Const. on the water)	-	-	500.5 m^3	1,051,050
	4,200 (Const. on the land)	-	-	18.8 m^3	78,960
Const. Cost (unit price per meter in direction along the dam axis)		471,980			1,372,608

As shown in the above table, the construction cost with borrow materials is more economical in terms fo unit cost per meter in direction along the dam axis than that with excavated materials from diversion canal site by about ฿900,000. In comparison of the two, the construction with borrow materials is easier in implementation, short in work period because of being needless to have foundation improvement works, and more economical in construction cost.

The construction works of embankment, therefore, are considered more advantageous in using the borrow materials than in appropriation of the materials excavated at the diversion canal site.

APPENDIX - 7 : DESIGN OF ROAD AND ROAD BRIDGE

APPENDIX - 7. DESIGN OF ROAD AND ROAD BRIDGE

LIST OF CONTENTS

7.1 Road

7.1.1 Route Alignment

RID plans to build a new road beginning from an existing road which branches from the trunk road 304 on the left bank of the Bang Pakong River and reaches to Chuknua village on the river bank. The new road will link up with an existing road on the right bank, after crossing the river and the diversion canal. The existing branch road on the left bank has been paved with laterite materials with a total width of 9.0 meters and a favourable plane alignment.

If the proposed branch road was to be constructed as an extension of the existing branch road on the left bank, Chuknua village houses would have to be removed as obstacles of access road construction. Furthermore, the plane alignment as a whole would not be favourable due to the fact that the road would have to be a considerable distance to the west of the road bridge.

Therefore, the proposed road will be constructed about 300 meters east of the end of the existing road on the left bank avoiding the residential area of Chuknua village.

The crossing point of the road over the diversion canal is decided 200 meters downstream from the diversion dam so as to lessen the effect of the river discharge through the diversion dam.

1) Radius of Curve

The following table shows the relationship between the design speed (V), and the curve radius (R), length (L), width (ΔW), and the superelevation of curve (i).

TABLE 7-1 DIMENSIONS OF ROAD CURVE

Design Speed (km/hr)	Radius of Curve		Length of Curve (m)	width of Curve (m)	Superelevation of Curve (%)
	Min (m)	Stand. (m)			
60	150	200	100	0.25 (0)	9 (8)
80	280	400	140	0	9 (7)

Note : The figures in ( ) are for the standard radius.

As learned from the above table, the curve radius for a road with a design speed of 60 km/hr is decided at more than 200 meters and that of 80 km/hr at more than 400 meters.

Since the distance from the starting point to the point IP_1 is as short as 178.0meters with the intersecting angle of 62°13'16", the necessary curve radius is 200 meters. Consequently, the design speed for the portion of 300 meters from Sta. 0 + 000 to Sta. 0 + 300 will be 60 km/hr and the others 80 km/hr. The curve radius for the IP_2 and IP_3 will be designed at 500 meters.

2) Transition Portion

Curve widening in the proposed road is not required because the related curve radius is more than 200 meters, and there is no need to provide transition portions for the proposed road.

7.1.2 Longitudinal Section

1) Longitudinal Slope

The road design criteria set by the Highway Department of Thailand indicate that the longitudinal slope should be four (4) percent maximum for roads that are classified Class 4 with a design speed in the range of 60 to 80 km/hr and with flat gentle topography. The proposed road, providing no drains, should be flat with minimum longitudinal slope.

2) Design Crest Elevation of the Road

The design crest elevation of the road at each point should be as follows.

a) The design crest elevation of the starting point (Sta. 0) will be taken at 2.26 meters, the same elevation as that of the existing road crest.

b) The design crest elevation at the closure dam from the point (Sta. 0 + 920) to the point (Sta. 1 + 200) will be EL 4.20 meters to meet the closure dam crest elevation including the amount of camber.

c) The proposed road surface elevation in the distance from Sta. 2 + 160 to Sta. 2 + 440 in the area for the O/M buildings shall be at EL.3.90 m in leveling with the embankment crest of the diversion canal.

d) The design road crest elevation at the road bridge from the point (Sta. 2 + 640) to the point (Sta. 2 + 960) will be EL 5.20 meters to meet the road bridge elevation.

e) Since the proposed road must be constructed to cross the Bang Pakong river via the high water channel, the road crest elevation shall be EL.2.50 m in adding 0.1 m to the maximum water level of Max. W.L. 2.40 m.

7.1.3 Road Cross Section

1) Road Width

As a result of consultative discussion with RID staff concerned, the road width is decided as follows.

a) For the vehicle lanes in the general road, 3.0 m × 2 = 6.0 m and for the shoulder, 1.5 m × 2 = 3.0 m. Therefore, the total road width is 9.0 m.

b) For the vehicle lane of the closure dam embankment, 3.0 m × 2 = 6.0 m and for shoulder, 1.5 m × 2 = 3.0 m together with shoulder protection by 1.5 m × 2 = 3.0 m. Therefore, the total road width is 12.0 m.

c) The road running through the area for the O/M building shall have vehicle road width as 3.0 m × 6 = 18.0 m. The shoulder shall have width 1.0 m × 2 = 2.0 m, and for the pedestrian walks, 4.0 m × 2 = 8.0 m. Therefore, the total road width is 28.0 m.

d) For the vehicle lanes of the road bridge, 4.0 m × 2 = 8.0 m with the shoulder 0.5 m × 2 = 1.0 m, and the pedestrian walks as 1.5 m × 2 = 3.0 m. Therefore, the total road width is 12.0 m.

2) Cross Slope

According to the design criteria of the Highway Department of Thailand, the cross slope of the proposed road should be 3.5 percent.

3) Embankment Slope

The slope of the road embankment will be 1 to 2.0 in accordance with the design criteria of the Highway Department.

7.1.4 Pavement Works

1) Design Conditions

- Daily traffic capacity (large size vehicles)
  : A - traffic (100 - 250/day)
- Design CBR : 3%
- Minimum thickness of each paved layer
  - Surface course : 5.0 cm
  - Base course : 10.0 cm
  - Sub-base course : 10.0 cm

2) Pavement thickness required (TA)

$$T_A = \frac{3.84 \cdot N^{0.16}}{C.B.R.^{0.3}}$$

where, T_A : Thickness required (cm)

N : Number of wheels of vehicles passing for 10 years. (150,000 wheels/dir.)

C.B.R. : C.B.R. for road grade (3%)

T_A is obtained as about 19 cm.

3) Design Pavement Thickness

$$T'_A = a_1 \cdot T_1 + a_2 . T_2 + a_3 . T_3$$

where, T'_A : Design pavement thickness (cm)

T_1, T_2 & T_3 : Thickness of surface course, base course and sub-base course, respectively (cm)

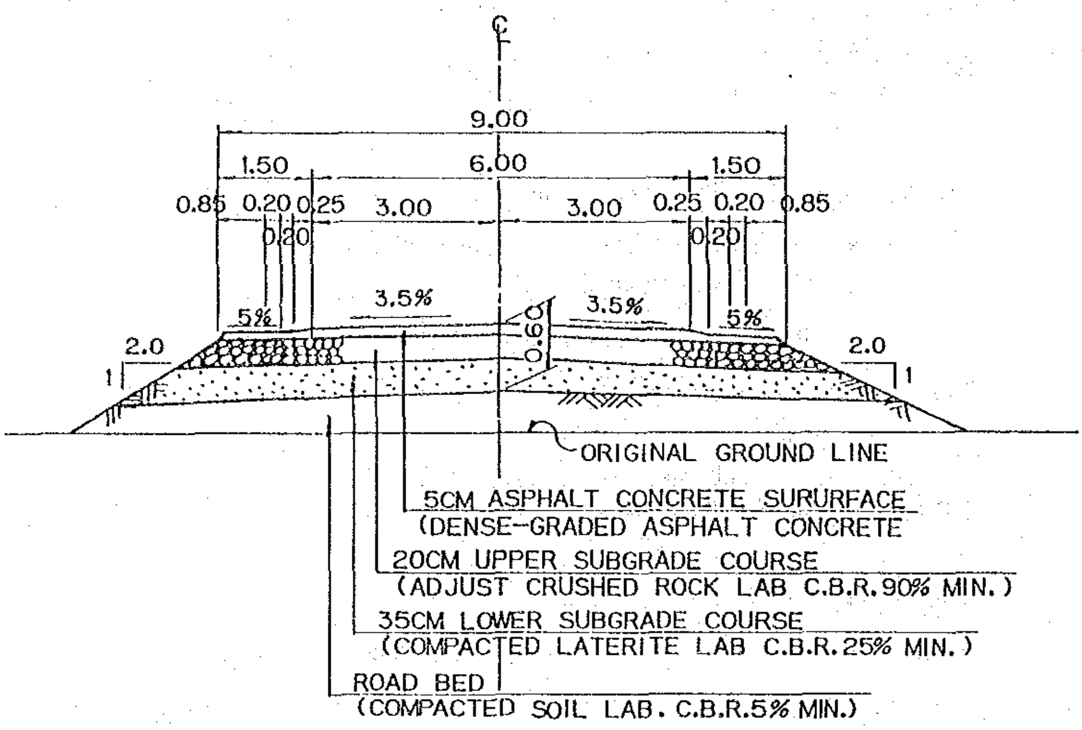
a_1, a_2 & a_3 : Equivalent conversion coefficient of each course by 1.00, 0.35 and 0.20, respectively.

TABLE 7-2 COMPARISON OF DESIGN PAVEMENT THICKNESS OF COURSES

Scheme	Surface C. 2000฿/m³	Base Course 450 ฿/m³	Sub-base C. 265฿/m³	Con. Thick. T'A (cm)	Total Thick. H (cm)	Paving Cost (฿/m²)
Case - A	5 cm	10 cm	55	19.5	60	
	100 ฿/m²	45 ฿/m²	146			291
Case - B	5	15	45	19.3	65	
	100	68	119			287
Case - C	5	20	35	19.0	60	
	100	90	93			283
Case - D	5	25	30	19.8	60	
	100	113	80			293

From the above table, the pavement construction of the proposed road will have a thickness of 5.0 cm for the surface course, 20 cm for the base course, 35 cm for the sub-base course, and a total thickness of 60 cm.

FIGURE 7-1 TYPICAL CROSS SECTION OF ROAD



7. 1. 5 Lighting Facilities for Roads

1) Type A Road (Road width: 9.0 m)

Width of vehicle lanes	:	W = 6.0 m
Physical condition	:	C (Little light gives adverse effects to the road lighting and traffic)
Standard brightness	:	L = 0.5 cd/m^2
Light source	:	Fluorescent mercury lamp or high-voltage sodium lamp
Light distribution	:	Cut-off type by C in the trunk road
Equipment arrangement	:	One side arrangement in one unit N = 1
Lighting height	:	$H \geqq 1.5\,W = 1.5 \times 6.0 = 9.0\,m \rightarrow H = 10\,m$
Interval for lighting	:	$S \leqq 3.5\,H = 3.5 \times 10\,m = 35\,m$
Illumination coefficient	:	$W_1/H = 6.0/10.0 = 0.6$, $U = 0.26$
Conservation rate	:	M = 0.65

Conversion coefficient for
average intensity of illumination: K = 15 ℓx/cd/m^2 (Asphalt)
Calculation for illumination

$$F/S = \frac{W \times K \times L}{N \times U \times M} = \frac{6.0 \times 15 \times 0.5}{1 \times 0.26 \times 0.65} = 266\ \ell m/m$$

Standard brightness (cd/m^2)	Illumi.	Light distribut.	Arrange of lighting	Light. height (m)	Comser. co-efficient	Inter. (m)	Per 1 km	
							No. of Lamp	Watt (kw)
0.5	F.M Lamp H.F 200x (9,200ℓm)	Cutoff type	One side One set	10.0	0.65	30	33	7.26
	H.S. Lamp N.H 150F (13,000 ℓm)					35	29	5.08

The lighting for road, therefore, shall be made with 10 m-elevated high-voltage sodium lamps of NH150F (Light flux: 13,000 ℓm, Average life: 12,000 hr, Power supply: 175 watt) installed at intervals of 35 m on one side arrangement.

2) Type B Road (Road width : 28.0 m)

Width of vehicle lanes	:	$W = 20.0$ m
Physical condition	:	C (Little light gives adverse effects to the road lighting and traffic)
Standard brightness	:	$L = 0.5$ cd/m^2
Light source	:	Fluorescent mercury lamp or high-voltage sodium lamp
Light distribution	:	Cut-off type by C in the trunk road
Equipment arrangement	:	Double in opposition $N = 2$
Lighting height	:	$H \geqq 0.5 \cdot W = 0.5 \times 20.0 = 10.0$ m
Interval for lighting	:	$S \leqq 3.0H = 3.0H = 3.0 \times 10$ m $= 30$ m
Illumination coefficient	:	$W_1/H = 10.0/10.0 = 1.0, U_1 = 0.33$ $W_2/H = 4.0/10.0 = 0.4, U_2 = 0.14$ $U = 0.33 + 0.14 = 0.47$
Conservation rate	:	$M = 0.65$

Conversion coefficient for

average intensity of illumination: $K = 15$ ℓx/cd/m^2 (Asphalt)

Calculation for illumination

$$F/S = \frac{W \times K \times L}{N \times U \times M} = \frac{20.0 \times 15 \times 0.5}{2 \times 0.47 \times 0.65} = 245\ \ell m/m$$

Standard brightness (cd/m²)	Illumi.	Light distribut.	Arrange of lighting	Light. height (m)	Conser. co-efficient	Inter. (m)	Per 1 km	
							No. of Lamp	Watt (kw)
0.5	F.M Lamp H.F 200x (9,200ℓm)	Cutoff type	Both-side with facing two sets	10.0	0.65	30	33	7.26
	H.S. Lamp N.H 150F (13,000 ℓm)					35	29	5.08

The 10 m-elevated equipment, therefore, shall be provided with high-voltage sodium lamps or NH150F (Light flux: 13,000 ℓm. Average life: 12,000 hr, Power supply: 175 watt) at the intervals of 30 m on one side arrangement.

3) Type C Road (Road width: 12.0 m)

Width of vehicle lanes	:	$W = 9.0$ m
Physical condition	:	C (Little light gives adverse effects to the road lighting and traffic)
Standard brightness	:	$L = 0.5$ cd/m²
Light source	:	Fluorescent mercury lamp or high-voltage sodium lamp
Light distribution	:	Cut-off type by C in the trunk road
Equipment arrangement	:	Double in opposition $N = 2$
Lighting height	:	$H \geqq 1.0 \cdot W = 1.0 \times 9.0 = 9.0$ m $\rightarrow H = 10$ m
Interval for lighting	:	$S \leqq 3.5H = 3.5 \times 10$ m $= 35$ m
Illumination coefficient	:	$W_1/H = 9.0/10.0 = 0.9$, $U = 0.33$
Conservation rate	:	$M = 0.65$

Conversion coefficient for
average intensity of illumination: $K = 15$ ℓx/cd/m² (Asphalt)

Calculation for illumination

$$F/S = \frac{W \times K \times L}{N \times U \times M} = \frac{9.0 \times 15 \times 0.5}{1 \times 0.33 \times 0.65} = 315 \text{ ℓm/m}$$

Standard brightness (cd/m²)	Illumi.	Light distribut.	Arrange of lighting	Light. height (m)	Conser. co-efficient	Inter. (m)	Per 1 km	
							No. of Lamp	Watt (kw)
0.5	F.M Lamp H.F 250x (11,800 ℓm)	Cutoff type	One side One set	10.0	0.65	35	29	7.83
	H.S. Lamp N.H 150F (13,000 ℓm)					35	29	5.08

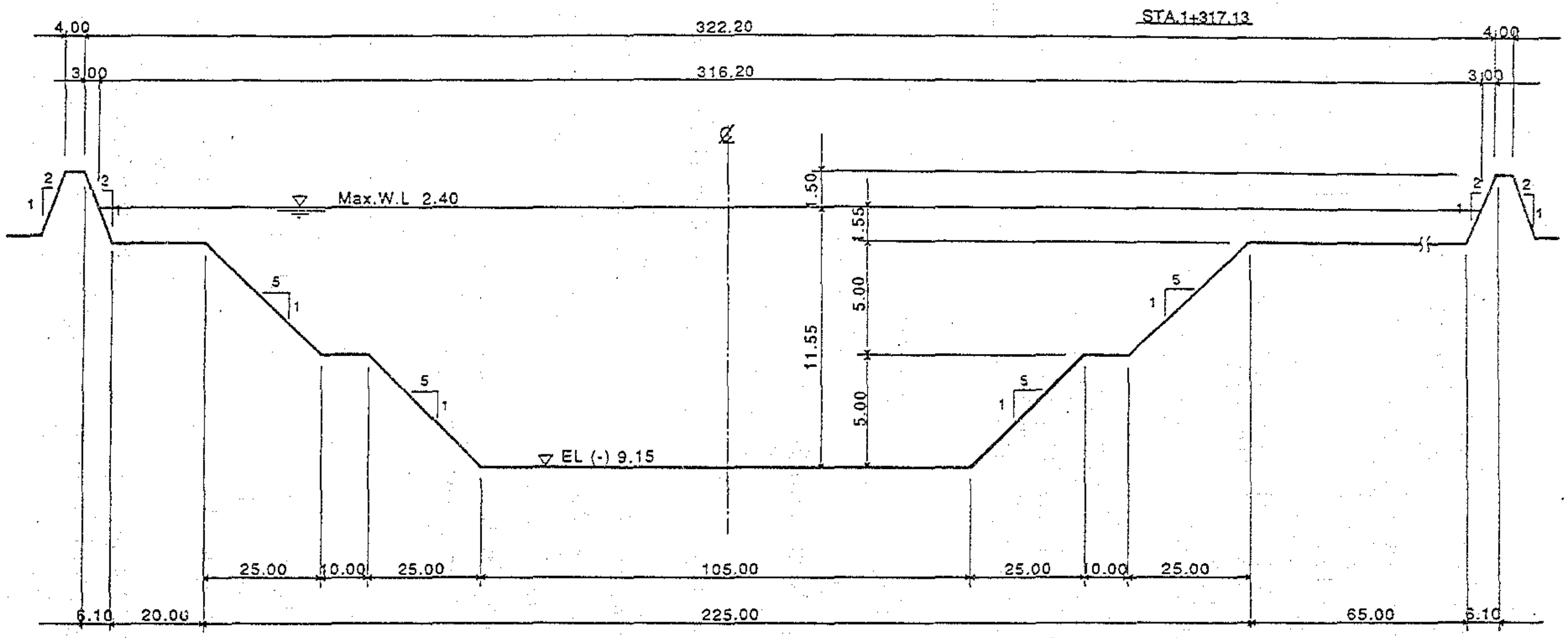
The 10 m-elevated equipment, therefore, shall be provided with high-voltage sodium lamps or NH150F (Light flux: 13,000 ℓm, Average life: 12,000 hr, Power supply: 175 watt) at the intervals of 35 m on one side arrangement.

7.2 Road Bridge

7.2.1 Basic Design Conditions

1)	Road class	: Class 4 by design criteria of the Highway Department in Thailand
2)	Design speed	: 80 km/hr
3)	Design traffic	: 300 to 1000 vehicles/day
4)	Bridge class	: Class 1 (TL - 20)
5)	Bridge length	: 226.85 meters
6)	Bridge construction	: For vehicle 2 lanes × 4.0 m = 8.0 m For shoulder 2 × 0.5 m = 1.0 m For Side-walk 2 × 1.50 m = 3.0 m Total width 12.0 m
7)	Route alignment	: straight
8)	Inclined angle	: 90 degrees
9)	Pavement	: Asphalt pavement with 6.0 cm thickness for vehicles lanes.
10)	Cross slope	: 3.5 % for vehicles lane
11)	Longitudinal slope	: Level
12)	Special load	: Lighting facilities

TABLE 7-2 DESIGN CROSS SECTION OF DIVERSION CANAL



13) River planning

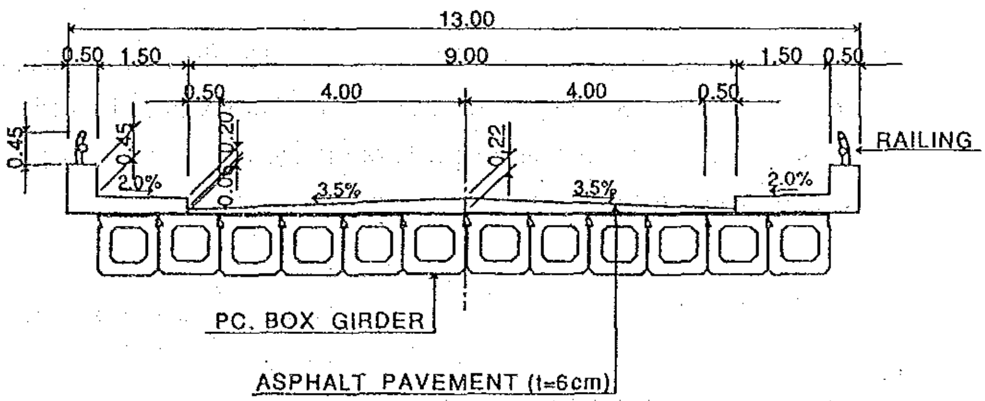
River name	: Bang Pakong river
Location of bridge	: Sta. 1 + 317.3
Design flood discharge	: Q = 1600 m^3/s
Maximum water level	Max. W. L. 2.40 m
Design crest dike elevation	: EL. 3.90 m
Design river bed elevation	: EL. (-) 9.15 m
Design river bed slope	: I = 1/4,000
Design cross section	: as Figure 7-2

7.2.2 Alignment Plan

1) Cross Alignment

The cross alignment of the road bridge will be the same as that of the road as shown in Figure 7-3.

FIGURE 7-3 TYPICAL CROSS SECTION OF ROAD BRIDGE



2) Longitudinal Alignment

The longitudinal alignment of the proposed road bridge will be level. The road bridge surface elevation will be EL.5.20 meters allowing for the fact that the girder seat elevation should be kept at more than EL.3.90 meters, which is the dike elevation.

Bridge surface elevation = Design dike elevation + Girder height + Pavement thickness
= EL. 3.90 m + 1.00 m + 0.22 m
= EL. 5.12 m $\fallingdotseq$ EL. 5.20 m

7. 2. 3 Bridge Length

The bridge length will be determined taking into account the fact that the river cross section must have the ability to cope with the flow of design flood discharge of Q = 1,600m^3/s at the maximum water level (Max. W. L. 2.40 m) in making the abutments front surface (EL. 0.85 m) contact the berm shoulder. The following equation should be applied to obtain the bridge length.

Bridge length = River width + 2 × Girder seat width
= 225 m + 2 × 0.80 m
= 226.60 m $\fallingdotseq$ 226.85 m

7. 2. 4 Type of Superstructure and Span

The type of superstructure and span length should be determined taking into consideration economy, ease of construction and of O and M works, along with a comprehensive and comparative study of the following seven (7) types and their respective span lengths.

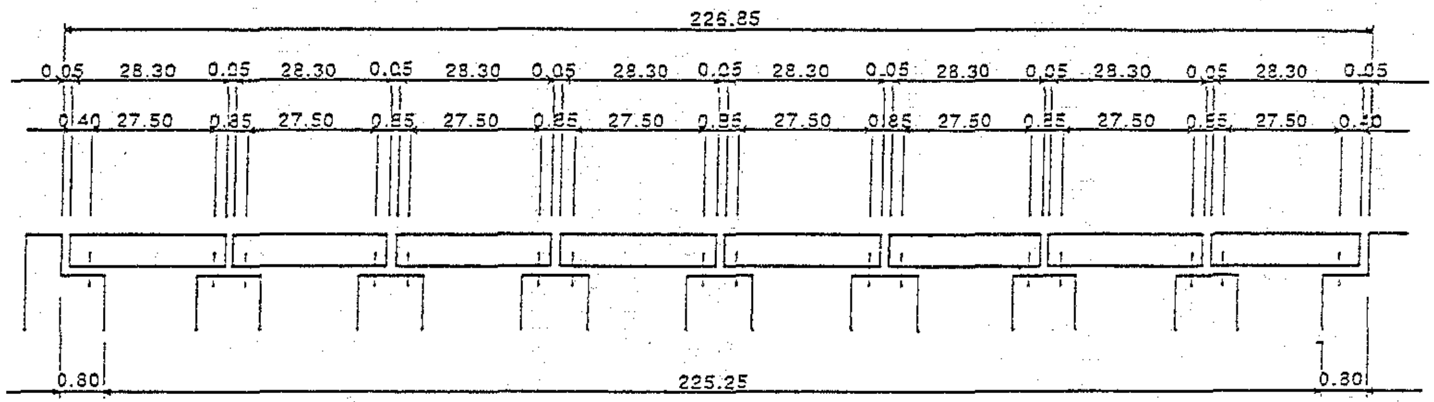
Superstructure	Span	Name of Case
P.C. I - Section Girder	8 spans	A - 1
	9 spans	A - 2
P.C. Hollow Box Girder	7 spans	B - 1
	8 spans	B - 2
	9 spans	B - 3
Steel Simple Composite Girder	5 spans	C - 1
	6 spans	C - 2

The most suitable type of superstructure and the related span length is determined to be Case B-2 (Hollow Box P.C. bridge : 27.5 m × 8 span) in view of the results of a comparative study on economy, merits and demerits of construction and O and M works as shown in Table 7-1.

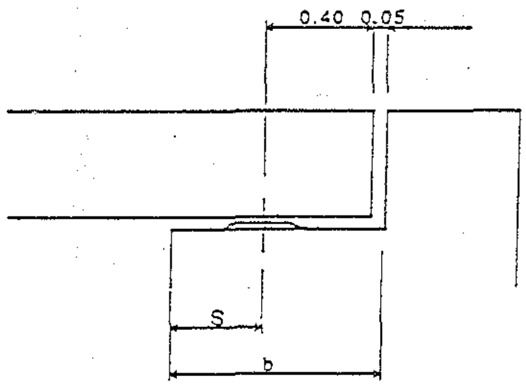
TABLE 7-3 COMPARISON OF SUPERSTRUCTURE TYPE AND SPAN

Scheme	Type of Superstructure	Span	Economy	Specific Features	Overall Appraisal
A - 1	Prestressed Concrete Bridge Girder I - Section	27.5 m × 8	137%	- Dead load reaction is largest - O and M works are not required - Economically, initial cost is largest - No need for bent method and easy in construction	△
A - 2		24.5 m × 9	141%		△
B - 1	Prestressed Concrete Hollow Box Girder	31.6 m × 7	104%	- O and M works are not required - Most economical - No need for bent method and easy in construction	△
B - 2		27.5 m × 8	100%		◎
B - 3		24.5 m × 9	101%		○
C - 1	Simple Composite Steel Girder	44.7 m × 5	118%	- Dead load reaction is least - Bent method is required and high technique is necessary in construction	△
C - 2		37.2 m × 6	113%		△

FIGURE 7-4 ELEVATION OF ROAD BRIDGE



Elevation of Road Bridge



$S = 20 + 0.5 \times l$
$= 20 + 0.5 \times 27.50$
$= 34cm$ — $35.0cm$

$b = 0.35 + 0.40 + 0.05 = 0.80$ m

Detail of Girder Seat

7.2.5 Infrastructure

1) Abutment

The abutment type will be a common inverted T-shape type because of having a height of 6.30 meters with pile foundation.

2) Pier

The piers will be an ellipse balcony type made of reinforced concrete, taking into consideration the fact that they will be constructed beneath the river discharge.

7.2.6 Foundation Works

The foundation works will be a type of P.C concrete pile foundation with a 500 m dia. and 10 to 15 m length for the piers, and a type of steel pile with a 450 mm dia. and 21 m length taking into account the fact that the bearing layer is a clay layer (N > 20) at EL.(-) 21.00 m and the groundwater table is high because the bearing layer is based 10 meters lower than the design river bed.

7.2.7 Stability Analysis of Infrastructure

1) Abutment

a) Type: Abut. A-1 Case-1

1. Design Criteria

1 - 1 Dimensions (m)

H1 = 1.30 H2 = 4.20 H3 = 0.00 H4 = 0.80 H5 = 0.50
H6 = 1.95 H7 = 0.00 HT = 6.30
B1 = 0.80 B2 = 0.30 B3 = 0.60 B4 = 0.80 B5 = 0.50
B6 = 3.20 BT = 4.50 TL = 13.00

Water Level (Rear) h1 = 2.10
(Front) h2 = 0.00

1 - 2 Unit Load (t/m^3)

Reinforced Concrete	:	Rc = 2.40
Wet Earth	:	Rs = 1.80
Saturated Earth	:	Rw = 2.00
Water	:	Wo = 1.00

1 - 3 Coefficient of Active Earth Pressure : KA = 0.355 (0.367)

1 - 4 Load

Reaction from Superstructure Dead Rd = 445.0 (t)
Live Rt = 88.0 (t)

Live Load Q = 1.00 (t/m^2)

2. Check of Stability

(1) Support

Load	V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1. Wall	217.62	1.75			380.11	
2. Live Load	37.70	3.05			114.98	
3. Earth and Water (Front)	0.00	0.00			0.00	
4. Earth and Water (Rear)	329.61	3.06			1,007.90	
5. Reaction	533.00	1.15			612.95	
6. Buoyancy	0.0	0.00			0.00	
7. Earth Pressure	78.10	4.50			351.45	
8. Earth Pressure			167.48	2.33		391.01
9. Groundwater Pressure			28.66	0.70		20.07
Sum	1,196.03		196.15		2,467.39	411.08

Horaizontal Force H = 1,196.03 (t)
Vatical Force V = 196.15 (t)
Moment M = 2,467.39 − 411.08 = 2,056.31 (t · m)

1) Stability Against Overturning

X = M / H = 1.72 (m)
E = L/2 − X = 4.5/2 − X = 0.53 (m) < B/6 = 0.75 (m)

2) Soil Reaction

Q1 = V/L/B × (1 + 6 × E/L) = 34.91 (t/m^2)
Q2 = V/L/B × (1 − 6 × E/L) = 5.98 (t/m^2)

(2) Sliding and Overturning

Load	V (t)	X (m)	H (t)	Y (m)	V・X (t・m)	H・Y (t・m)
1. Wall	217.62	1.75			380.11	
2. Live Load	0.00	0.00			0.00	
3. Earth and Water (Front)	0.00	0.00			0.00	
4. Earth and Water (Rear)	329.61	3.06			1,007.90	
5. Reaction	445.00	1.15			511.75	
6. Buoyancy	0.00	0.00			0.00	
7. Earth Pressure	78.10	4.50			351.45	
8. Earth Pressure			167.48	2.33		391.01
9. Groundwater Pressure			28.66	0.70		20.07
Sum	1,070.33		196.15		2,251.21	411.08

Horaizontal Force $H = 1{,}070.33$ (t)
Vatical Force $V = 196.15$ (t)
Moment $M = 2{,}251.21 - 411.08 = 1{,}840.13$ (t・m)

1) Factor of Safety Against Sliding

Coefficient of Friction : $\mu = 0.60$
$Fs = \mu \times V/H = 3.27$

2) Stability Against Overturning

$X = M/H = 1.72$ (m)
$E = L/2 - X = 4.5/2 - X = 0.53$ (m) $< B/6 = 0.75$ (m)

3) Soil Reaction

$Q1 = V/L/B \times (1 + 6 \times E/L) = 31.24$ (t/m^2)
$Q2 = V/L/B \times (1 - 6 \times E/L) = 5.35$ (t/m^2)

b) Type: Abut. A-1 Case-2

1. Design Criteria

1 - 1 Dimensions (m)

$H1 = 1.30$ $H2 = 4.20$ $H3 = 0.00$ $H4 = 0.80$ $H5 = 0.50$
$H6 = 1.95$ $H7 = 0.00$ $HT = 6.30$
$B1 = 0.80$ $B2 = 0.30$ $B3 = 0.60$ $B4 = 0.80$ $B5 = 0.50$
$B6 = 3.20$ $BT = 4.50$ $TL = 13.00$
Water Level (Rear) $h1 = 3.50$
(Front) $h2 = 3.50$

1 - 2 Unit Loads (t/m³)

Reinforced Concrete	:	Rc = 2.40
Wet Earth	:	Rs = 1.80
Saturated Earth	:	Rw = 2.00
Water	:	Wo = 1.00

1 - 3 Coefficient of Active Earth Pressure : KA = 0.355 (0.367)

1 - 4 Load

Reaction from Superstructure	Dead	Rd =	445.0 (t)
	Live	Rt =	88.0 (t)

Live Load $Q = 1.00\ (t/m^2)$

2. Check of Stability

(1) Support

Load	V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1. Wall	217.62	1.75			380.11	
2. Live Load	37.70	3.05			114.8	
3. Earth and Water (Front)	0.00	0.00			0.00	
4. Earth and Water (Rear)	282.87	3.08			871.03	
5. Reaction	533.00	1.15			612.95	
6. Buoyancy	−66.15	1.92			−126.79	
7. Earth Pressure	71.22	4.50			320.48	
8. Earth Pressure			152.73	2.42		369.92
9. Groundwater Pressure			0.00	0.00		0.00
Sum	1,076.25		152.73		2,173.65	369.92

Horaizontal Force	H =	1,076.25 (t)
Vatical Force	V =	152.73 (t)
Moment	M =	2,173.65 − 369.92 = 1,803.73 (t · m)

1) Stability Against Overturning

$X = M/H = 1.68\ (m)$

$E = L/2 - X = 4.5/2 - X = 0.578\ (m) < B/6 = 0.75\ (m)$

2) Soil Reaction

$Q1 = V/L/B \times (1+6\times E/L) = 32.48\ (t/m^2)$

$Q2 = V/L/B \times (1-6 \times E/L) = 4.32\ (t/m^2)$

(2) Sliding and Overturning

Load	V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1. Wall	217.62	1.75			380.11	
2. Live Load	0.00	0.00			0.00	
3. Earth and Water (Front)	0.00	0.00			0.00	
4. Earth and Water (Rear)	282.87	3.08			871.93	
5. Reaction	445.00	1.15			511.75	
6. Buoyancy	− 66.15	1.92			−126.79	
7. Earth Pressure	71.22	4.50			320.48	
8. Earth Pressure			152.73	2.42		369.92
9. Groundwater Pressure			0.00	0.00		0.00
Sum	950.55		152.73		1,957.47	369.92

Horaizontal Force $H = 950.55$ (t)
Vatical Force $V = 152.73$ (t)
Moment $M = 1{,}957.47 - 369.92 = 1{,}587.55$ (t · m)

1) Factor of Safety Against Sliding

Coefficient of Friction : $\mu = 0.60$
$Fs = \mu \times V/H = 3.73$

2) Stability Against Overturning

$X = M/H = 1.67$ (m)
$E = L/2 - X = 4.5/2 - X = 0.58$ (m) $< B/6 = 0.75$ (m)

3) Soil Reaction

$Q1 = V/L/B \times (1 + 6 \times E/L) = 28.69$ (t/m^2)
$Q2 = V/L/B \times (1 - 6 \times E/L) = 3.81$ (t/m^2)

c) Type: Abut. A-1 Case-3

1. Design Criteria

1 - 1 Dimensions (m)

$H1 = 1.30$ $H2 = 4.20$ $H3 = 0.00$ $H4 = 0.80$ $H5 = 0.50$
$H6 = 1.95$ $H7 = 0.00$ $HT = 6.30$
$B1 = 0.80$ $B2 = 0.30$ $B3 = 0.60$ $B4 = 0.80$ $B5 = 0.50$
$B6 = 3.20$ $BT = 4.50$ $TL = 13.00$

1 - 2 Unit Loads (t/m^3)

Reinforced Concrete	:	Rc = 2.40
Wet Earth	:	Rs = 1.80
Saturated Earth	:	Rw = 2.00
Water	:	Wo = 1.00

1 - 3 Coefficient of Active Earth Pressure : KA = 0.00 (0.00)

1 - 4 Load

Reaction from Superstructure — Dead Rd = 0.00 (t)

Live Rt = 0.00 (t)

Live Load Q = 0.00 (t/m^2)

2. Check of Stability

Load	V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1. Wall	217.62	1.75			380.11	
2. Live Load	0.00	0.00			0.00	
3. Earth and Water (Front)	74.88	2.90			217.15	
4. Earth and Water (Rear)	0.00	0.00			0.00	
5. Reaction	445.00	1.15			511.75	
6. Buoyancy	0.00	0.00			0.00	
7. Earth Pressure	0.00	0.00				
8. Earth Pressure			0.00	0.00		0.00
9. Groundwater Pressure			0.00	0.00		0.00
Sum	737.50		0.00		1,109.01	0.00

Horaizontal Force	H =	0.00	(t)
Vatical Force	V =	737.50	(t)
Moment	M =	1,109.01	(t · m)

1) Stability Against Overturning

X = M/H = 1.50 (m)

E = L/2 - X = 4.5/2 - X = 0.75 (m) = B/6 = 0.75 (m)

2) Soil Reaction

Q1 = V/L/B × (1+6×E/L) = 25.21 (t/m^2)

Q2 = V/L/B × (1 − 6 × E/L) = 0.00 (t/m^2)

2) Pier

a) Type: Pier P-1

1. Design Criteria

1 - 1 Dimensions (m)

H1 = 1.00 H2 = 0.50 H3 = 8.00 B1 = 1.60 B2 = 14.00 B3 = 0.80
B4 = 9.00 F1 = 0.00 F2 = 1.00 F3 = 3.00 F4 = 10.00 F5 = 1.00
Water Level = 9.00

1 - 2 Unit Load (t/m^3)

Reinforced Concrete	:	Rc = 2.40
Wet Earth	:	Rs = 1.80
Saturated Earth	:	Rw = 2.00
Water	:	Wo = 1.00

1 - 3 Load

Reaction from Superstructure	Dead	Rd = 889.0 (t)
	Live	Rt = 134.0 (t)
Wind Load		Hw = 14.00 (t)

2. Check of Stability

(1) Case-1

	Load		V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1.	Upper Slab	1	49.92	0	0.00	10.00	0	0.00
2.	〃	2	16.16	0	0.00	9.27	0	0.00
3.	Column		135.60	0	0.00	5.00	0	0.00
4.	Bottom Slab	1	0.00	0	0.00	1.00	0	0.00
5.	〃	2	72.00	0	0.00	0.50	0	0.00
6.	Reaction from S. Str.		1,027.00	0	14.00	10.50	0	147.00
7.	Earth and Water		59.87	0	0.00	0.00	0	0.00
8.	Bouyancy		0.00	0	0.00	0.00	0	0.00
	Sum		1,360.55		14.00			147.00

Horaizontal Force	H	=	14.00	(t)
Vatical Force	V	=	1,360.55	(t)
Moment	M	=	147.00	(t · m)

1) Stability Against Overturning

E = 0.108 (m) < L/6 = 3.33 (m)

2) Soil Reaction

$Q1 = V/L/B \times (1 + 6 \times E/L) \quad = \quad 48.29 \ (t/m^2)$
$Q2 = V/L/B \times (1 - 6 \times E/L) \quad = \quad 42.41 \ (t/m^2)$

(2) Case-2

Load		V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1. Upper Slab	1	49.92	0	0.00	10.00	0	0.00
2. 〃	2	16.16	0	0.00	9.27	0	0.00
3. Column		135.60	0	0.00	5.00	0	0.00
4. Bottom Slab	1	0.00	0	0.00	1.00	0	0.00
5. 〃	2	72.00	0	0.00	0.50	0	0.00
6. Reaction from S. Str.		1,027.00	0	14.00	10.50	0	147.00
7. Earth and Water		0.00	0	0.00	0.00	0	0.00
8. Bouyancy		−86.50	0	0.00	0.00	0	0.00
Sum		1,214.18		14.00			147.00

Horaizontal Force	H =	14.00	(t)
Vatical Force	V =	1,214.18	(t)
Moment	M =	147.00	(t · m)

1) Stability Against Overturning

$E = 0.121 \ (m) \quad < L/6 = 3.33 \ (m)$

2) Soil Reaction

$Q1 = V/L/B \times (1 + 6 \times E/L) \quad = \quad 43.413 \ (t/m^2)$
$Q2 = V/L/B \times (1 - 6 \times E/L) \quad = \quad 37.533 \ (t/m^2)$

(3) Case-3

Load		V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1. Upper Slab	1	49.92	1.50	0	0	74.88	0
2. 〃	2	16.16	1.50	0	0	24.24	0
3. Column		135.60	1.50	0	0	203.40	0
4. Bottom Slab	1	0.00	1.50	0	0	0.00	0
5. 〃	2	72.00	1.50	0	0	108.00	0
6. Reaction from S. Str.		445.00	1.10	0	0	489.50	0
7. Earth and Water		0.00	1.50	0	0	0.00	0
8. Bouyancy		0.00	1.50	0	0	0.00	0
Sum		718.68		0		900.02	0

Horaizontal Force	H =	0.00	(t)
Vatical Force	V =	717.76	(t)
Moment	M =	900.02	(t · m)

1) Stability Against Overturning

$E = 0.248\ (m) < L/6 = 3.33\ (m)$

2) Soil Reaction

$Q1 = V/L/B \times (1 + 6 \times E/L) = 35.93\ (t/m^2)$
$Q2 = V/L/B \times (1 - 6 \times E/L) = 11.98\ (t/m^2)$

b) Type: Pier P-2, P-3

1. Design Criteria

1 - 1 Dimensions (m)

H1 = 1.00 H2 = 0.50 H3 = 13.00 B1 = 1.60 B2 = 16.00 B3 = 0.80
B4 = 9.00 F1 = 0.00 F2 = 1.00 F3 = 3.00 F4 = 10.00 F5 = 1.00
Water Level = 14.00

1 - 2 Unit Load (t/m^3)

Reinforced Concrete	:	Rc = 2.40
Wet Earth	:	Rs = 1.80
Saturated Earth	:	Rw = 2.00
Water	:	Wo = 1.00

1 - 3 Load

Reaction from Superstructure	Dead	Rd = 889.0 (t)
	Live	Rt = 134.0 (t)
Wind Load		Hw = 14.00 (t)

2. Check of Stability

(1) Case-1

Load			V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1.	Upper Slab	1	49.92	0	0.0	15.00	0.0	0.0
2.	〃	2	16.16	0	0.0	14.27	0.0	0.0
3.	Column		220.35	0	0.0	7.50	0.0	0.0
4.	Bottom Slab	1	0.00	0	0.0	1.00	0.0	0.0
5.	〃	2	72.00	0	0.0	0.50	0.0	0.0
6.	Reaction from S. Str.		1,027.00	0	14.00	15.50	0.0	217.00
7.	Earth and Water		68.12	0	0.0	0.0	0.0	0.0
8.	Bouyancy		0.0	0	0.0	0.0	0.0	0.0
	Sum		1,453.56		14.00			217.00

Horaizontal Force	H	=	14.00	(t)
Vatical Force	V	=	1,453.56	(t)
Moment	M	=	217.00	(t · m)

1) Stability Against Overturning

E = 0.149 (m) < L/6 = 3.33 (m)

2) Soil Reaction

Q1 = V/L/B × (1 + 6 × E/L) = 52.79 (t/m^2)
Q2 = V/L/B × (1 − 6 × E/L) = 44.11 (t/m^2)

(2) Case-2

Load		V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1. Upper Slab	1	49.92	0	0.0	15.00	0.0	0.0
2. 〃	2	16.16	0	0.0	14.27	0.0	0.0
3. Column		220.35	0	0.0	7.50	0.0	0.0
4. Bottom Slab	1	0.00	0	0.0	1.00	0.0	0.0
5. 〃	2	72.00	0	0.0	0.50	0.0	0.0
6. Reaction from S. Str.		1,027.00	0	14.00	15.50	0.0	217.00
7. Earth and Water		0.00	0	0.0	0.0	0.0	0.0
8. Bouyancy		−121.81	0	0.0	0.0	0.0	0.0
Sum		1,263.62		14.00	0.0		217.00

Horaizontal Force H = 14.00 (t)
Vatical Force V = 1,263.62 (t)
Moment M = 217.00 (t · m)

1) Stability Against Overturning

E = 0.172 (m) < L/6 = 3.33 (m)

2) Soil Reaction

Q1 = V/L/B × (1 + 6 × E/L) = 46.46 (t/m^2)
Q2 = V/L/B × (1 − 6 × E/L) = 37.78 (t/m^2)

(3) Case-3

Load			V (t)	X (m)	H (t)	Y (m)	V · X (t · m)	H · Y (t · m)
1.	Upper Slab	1	49.92	1.50	0.0	15.60	74.88	0.0
2.	〃	2	16.16	1.50	0.0	14.27	24.24	0.0
3.	Column		220.35	1.50	0.0	7.50	330.53	0.0
4.	Bottom Slab	1	0.00	1.50	0.0	1.00	0.00	0.0
5.	〃	2	72.00	1.50	0.0	0.50	108.00	0.0
6.	Reaction from S. Str.		445.00	1.10	0.0	0.0	489.50	0.0
7.	Earth and Water		0.0	1.50	0.0	0.0	0.0	0.0
8.	Bouyancy		0.0	1.50	0.0	0.0	0.0	0.0
	Sum		803.43		0.0		1,027.15	0.0

Horaizontal Force	H	=	0.0	(t)
Vatical Force	V	=	803.43	(t)
Moment	M	=	1,027.15	(t · m)

1) Stability Against Overturning

$E = 0.222\ (m) < L/6 = 3.33\ (m)$

2) Soil Reaction

$Q1 = V/L/B \times (1 + 6 \times E/L) = 38.56\ (t/m^2)$

$Q2 = V/L/B \times (1 - 6 \times E/L) = 15.00\ (t/m^2)$

7.2.8 Structural Calculation

TABLE 7-4 RESULT OF STRUCTURAL CALCULATION FOR ABUTMENT

Type	Item			Stem		Toe	Heel	Parapet	Wing
	Position			H = 0.0	H = 0.50	Bottom side	Bottom side	inside	inside
	Moment		(t · m)	343.49	257.95	130.12	150.80	71.50	20.86
	Shear Force		(t)	190.06	154.25	164.87	164.62	142.87	6.78
A-1	Reg'd Reinf.		(cm^2)	497.98	312.08	157.20	176.58	291.56	42.43
	Reinf.		(cm^2)	D28@125=646.80	D28@250=320.32	D16@125=211.05	D16@125=211.05	D28@250=320.32	D28@125=49.28
		σc	(kg/cm^2)	89 < 94.5	88 < 94.5	25 < 94.5	29 < 94.5	81 < 94.5	77 < 94.5
	Stress	σs	(kg/cm^2)	1,053 < 1,400	1,391 < 1,400	1,016 < 1,400	1,178 < 1,400	1,281 < 1,400	1,216 < 1,400
		τ	(kg/cm^2)	3.7 < 4.2	1.0 < 4.2	2.0 < 4.2	2.0 < 4.2	3.7 < 4.2	1.7 < 4.2

TABLE 7-5 RESULT OF STRUCTURAL CALCULATION FOR PIERS

Type	Item			Upper Slab	Bottom Slab	Column
	Position			-	Axiol Direction	-
	Moment		(t · m)	174.76	266.46	133.00
	Axiol Force		(t)	-	-	1,229.00
	Shear Force		(t)	207.01	217.55	14.00
	Reg'd Reinf.		(cm^2)	101.73	240.90	-
P1	Reinf.		(cm^2)	D28@125 × 2 = 127.66	D20@125 = 254.34	D16@150 = 358.20
		σc	(kg/cm^2)	48 < 94.5	37 < 94.5	12 < 94.5
	Stress	σs	(kg/cm^2)	1,127 < 1,400	1,328 < 1,400	- < 1,400
		τ	(kg/cm^2)	9.5 > 4.2	2.7 < 4.2	0.1 < 4.2
	Stirrup Reg'd Reinf.		(cm^2)	9.54	-	-
	Stirrup Reinf.		(cm^2)	12.56	-	-
	Position			–	Axiol Direction	-
	Moment		(t · m)	191.25	295.89	203.00
	Axiol Force		(t)	–	-	1,313.00
	Shear Force		(t)	215.51	224.49	14.00
P2	Reg'd Reinf.		(cm^2)	111.78	268.49	0.00
	Reinf.		(cm^2)	D28@125 × 2 = 127.66	D25@125 = 397.71	D16@250 = 358.20
P3		σc	(kg/cm^2)	53 < 94.5	34 < 94.5	13 < 94.5
	Stress	σs	(kg/cm^2)	1,233 < 1,400	959 < 1,400	– < 1,400
		τ	(kg/cm^2)	9.9 > 4.2	3.0 < 4.2	0.1 < 4.2
	Stirrup Reg'd Reinf.		(cm^2)	10.01	–	–
	Stirrup Reinf.		(cm^2)	12.56	–	–

7.2.9 Analysis of Foundation for Infrastructure

1) Abutment

Pile Groupe			1	2	3
n	(Number of Piles)	(pcs.)	11	11	11
D	(Diameter)	(mm)	450.00	450.00	450.00
t	(Thickness)	(mm)	9.00	9.00	9.00
A	(Cross Section Area)	(m^2)	0.00965	0.00965	0.00965
I	(Moment of Inertia of Pile Cross Section)	(m^4)	0.233E-3	0.233E-3	0.233E-3
θ	(Angle Between Pile and Vertical)	(deg)	0.00	0.00	0.00
Xi	(Coordinate of Pile Head)	(m)	-1.50	0.00	1.50
L	(Length of Piles)	(m)	21.00	21.00	21.00
Kh	(Coefficient of Horizontal Subgrade Reaction)	(kg/cm^3)	0.30	0.30	0.30
β		(m^{-1})	0.29	0.29	0.29

Simultaneous Equation:

$$\begin{pmatrix} 15467. & 0. & -27687. \\ 0. & 506972. & 0. \\ -27687. & 0. & 883350. \end{pmatrix} \begin{pmatrix} \sigma x \\ \sigma y \\ a \end{pmatrix} = \begin{pmatrix} 196.150 \ (H) \\ 1196.030 \ (V) \\ 633.900 \ (M) \end{pmatrix}$$

Displacement: σx = 0.014797 (m)
σy = 0.002359 (m)
a = 0.001181 (rad)

			1	2	3
Xi	(Horizontal Displacement)	(m)	0.015	0.015	0.015
Vi	(Vertical Displacement)	(m)	0.001	0.002	0.004
PVi	(Vertical Load)	(t/pcs)	9.301	37.376	65.451
PHi	(Horizontal Load)	(t/pcs)	6.130	6.130	6.130
Mi	(Moment)	(t·m/pcs)	-8.759	-8.759	-8.759

Σ		
Hi	(t)	196.150
Vi	(t)	1196.030
Mi	(t·m)	633.900

Check of Stress		Pile Groupe - 1	Pile Groupe - 3
Compressive Stress	(kg/cm^2)	857	1367
Tensile Stress	(kg/cm^2)	-688	-178
Allowable Stress	(kg/cm^2)	1400	1400

2) P1 Pier

Pile Groupe			1	2	3	4	5	6	
n	(Number of Piles)	(pcs.)	2	2	2	2	2	2	
D	(Diameter)	(mm)	500.00	500.00	500.00	500.00	500.00	500.00	
t	(Thickness)	(mm)	90.00	90.00	90.00	90.00	90.00	90.00	
A	(Cross Section Area)	(m^2)	0.11592	0.11592	0.11592	0.11592	0.11592	0.11592	
I	(Moment of Inertia of Pile Cross Section)	(m^4)	0.261E-2	0.261E-2	0.261E-2	0.261E-2	0.261E-2	0.261E-2	
θ	(Angle Between Pile and Vertical)	(deg)	0.00	0.00	0.00	0.00	0.00	0.00	
Xi	(Coordinate of Pile Head)	(m)	-3.750	-2.250	-0.750	0.750	2.250	3.750	
L	(Length of Piles)	(m)	15.00	15.00	15.00	15.00	15.00	15.00	
Kh	(Coefficient of Horizontal Subgrade Reaction)	(kg/cm^3)	0.90	0.90	0.90	0.90	0.90	0.90	
β		(m^{-1})	0.32	0.32	0.32	0.32	0.32	0.32	

Simultaneous Equation:

$$\begin{pmatrix} 116118. & 0. & -104039. \\ 0. & 482247. & 0. \\ -104039. & 0. & 4142363. \end{pmatrix} \begin{pmatrix} \sigma x \\ \sigma y \\ a \end{pmatrix} = \begin{pmatrix} 14.000\ (H) \\ 1360.548\ (V) \\ 146.940\ (M) \end{pmatrix}$$

Displacement: σx = 0.000970 (m)
σy = 0.004702 (m)
a = 0.000087 (rad)

			1	2	3	4	5	6
Xi	(Horizontal Displacement)	(m)	0.001	0.001	0.001	0.001	0.001	0.001
Vi	(Vertical Displacement)	(m)	0.004	0.005	0.005	0.005	0.005	0.005
PVi	(Vertical Load)	(t/pcs)	105.515	108.660	111.806	114.952	118.098	121.244
PHi	(Horizontal Load)	(t/pcs)	1.167	1.167	1.167	1.167	1.167	1.167
Mi	(Moment)	(t·m/pcs)	-1.518	-1.518	-1.518	-1.518	-1.518	-1.518

Σ	Hi	(t)	14.000
	Vi	(t)	1360.548
	Mi	(t·m)	146.940

Check of Stress		Pile Groupe - 1	Pile Groupe - 6
Compressive Stress	(kg/cm^2)	146	159
Tensile Stress	(kg/cm^2)	0	0
Allowable Stress	(kg/cm^2)	170 (C)	0 (T)

3) P2, P3 Pier

Pile Groupe			1	2	3	4	5	6	7	8
n	(Number of Piles)	(pcs.)	2	2	2	2	2	2	2	2
D	(Diameter)	(mm)	500.00	500.00	500.00	500.00	500.00	500.00	500.00	500.00
t	(Thickness)	(mm)	90.00	90.00	90.00	90.00	90.00	90.00	90.00	90.00
A	(Cross Section Area)	(m^2)	0.11592	0.11592	0.11592	0.11592	0.11592	0.11592	0.11592	0.11592
I	(Moment of Inertia of Pile Cross Section)	(m^4)	0.261E-2	0.261E-2	0.261E-2	0.261E-2	0.261E-2	0.261E-2	0.261E-2	0.261E-2
θ	(Angle Between Pile and Vertical)	(deg)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Xi	(Coordinate of Pile Head)	(m)	-4.375	-3.125	-1.875	-0.625	0.625	1.875	3.125	4.375
L	(Length of Piles)	(m)	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00
Kh	(Coefficient of Horizontal Subgrade Reaction)	(kg/cm^3)	8.10	8.10	8.10	8.10	8.10	8.10	8.10	8.10
β		(m^{-1})	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56

Simultaneous Equation:

$$\begin{bmatrix} 116118. & 0. & -104039. \\ 0. & 482247. & 0. \\ -104039. & 0. & 4142363. \end{bmatrix} \begin{bmatrix} \sigma x \\ \sigma x \\ a \end{bmatrix} = \begin{bmatrix} 14.000 \text{ (H)} \\ 1453.560 \text{ (V)} \\ 216.600 \text{ (M)} \end{bmatrix}$$

Displacement : σx = 0.000171 (m)
σy = 0.003014 (m)
a = 0.000057 (rad)

			1	2	3	4	5	6	7	8
Xi	(Horizontal Displacement)	(m)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Vi	(Vertical Displacement)	(m)	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003
PVi	(Vertical Load)	(t/pcs)	83.385	85.517	87.649	89.781	91.914	94.046	96.178	98.310
PHi	(Horizontal Load)	(t/pcs)	0.875	0.875	0.875	0.875	0.875	0.875	0.875	0.875
Mi	(Moment)	(t·m/pcs)	-0.454	-0.454	-0.454	-0.454	-0.454	-0.454	-0.454	-0.454

Σ	Hi	(t)	14.000
	Vi	(t)	1453.559
	Mi	(t·m)	216.601

Check of Stress		Pile Groupe - 1	Pile Groupe - 8
Compressive Stress	(kg/cm^2)	117	130
Tensile Stress	(kg/cm^2)	0	0
Allowable Stress	(kg/cm^2)	170 (C)	0 (T)

TABLE 7-6 RESULT OF DISPLACEMENT METHOD FOR INFRASTRUCTURE

Type	Case	Number of Piles	Vertical Load V	Vertical Load Va	Horizontal Load H	Horizontal Load Ha	Compressive Stress σ	Compressive Stress σa	Tensile Stress σ	Tensile Stress σa	Horizontal Displacement δ	Horizontal Displacement δa
		(pcs.)	(t/pcs.)	(t/pcs.)	(t/pcs.)	(t/pcs.)	(kg/cm^2)	(kg/cm^2)	(kg/cm^2)	(kg/cm^2)	(cm)	(cm)
ABUT.	A1	32	65	< 110	6.1	< 7.3	1,367	< 1,400	688	< 1,400	1.45	< 1.5
PIRE	P1	12	121	< 128	1.2	< 21.0	146	< 170	0	< 0	0.10	< 1.5
	P2,P3	16	98	< 99	0.9	< 108.9	130	< 170	0	< 0	0.02	< 1.5

APPENDIX - 8 DESIGN OF PUMPING STATION

APPENDIX - 8. DESIGN OF PUMPING STATION

LIST OF CONTENTS

8. 1 Site Selection of Pumping Station

Site selection of the proposed pumping station will be made so as to find the most advantageous site through alternative study taking the following conditions into consideration:

i) Topographic conditions

- The length of main irrigation canal between the pumping site and the service area should be shorter as possible.

- The site should be located in the upstream of the diversion and closure dams, limit entering of sediment and drifting materials, and ensure to divert the water by pumping continuously.

ii) Geological conditions

- It must be possible to carry out foundation works safely and economically.

iii) Environmental conditions

- In construction of the proposed pumping station any damages should not be given to its neighboring facilities, buildings, etc.

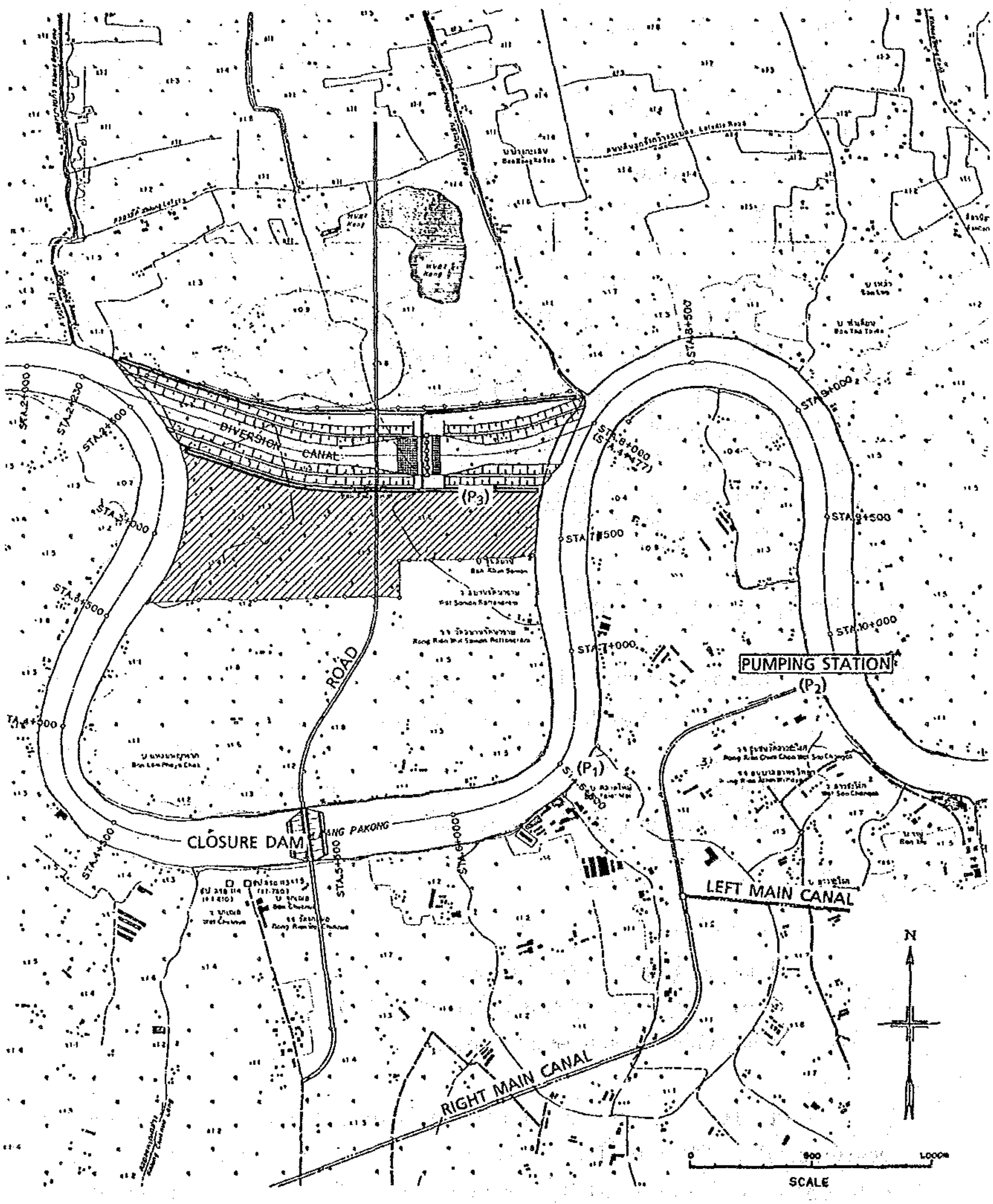
- Noise and vibration from the pump operation must not become a public unisance to the neighboring area.

iv) Others

- Land acquisition should be easy.
- Power supply for pump driver must be facilitated.
- Easy O/M works must be ensured.

As a result of the alternative study for the pumping site at the three locations, (P1), (P2) and (P3), illustrated in Figure 8-1, considering the above-mentioned conditions, site (P2) has been selected for the following reasons:

FIGURE 8-1 LOCATION OF PUMPING STATION



i) Site (P1) is located nearly in the middle of the left bank of the Bang Pakong River between the entrance of the diversion canal and the closure dam. After completion of the Project, the flow velocity of the river in this portion would be below 0.01 m/s even with full pumping operation. Accordingly, the river water would remain almost motionless. The sand and other organic materials would deposit in the upstream portion of the station together with suspended materials in the river resulting in a decrease of the cross sectional area of the river. And also, waste water from animal husbandry and kitchen services might pollute the river and result in poor water quality and thick growth of water weed. Smooth flushing in the wet season should be impeded resulting in great O/M costs.

Site (P1), therefore, is considered unsuitable for the proposed pumping site.

ii) Site (P3) is located along the diversion canal upstream of the diversion dam. Although there is advantageous in the operation and maintenance works of the pumping station, disadvantages of this site were found in the main irrigation canal of (P3) to be laid at a right angle to the Bang Pakong river flow. It is, therefore, considered unfavorable from the river control viewpoint because the canal embankment with a crest level of EL. 5.00 m would become an obstacle for smooth discharge of flood water overflowing the river course. Additionally, the main canal would cross over the existing river course parallel with the closure dam which would increase the volume of construction and the total main canal length would be 1.0 km or more longer than that in the case of (P2). Site (P3), therefore, would be considerably less economical than (P2).

iii) Site (P2) is located at about 2.3 km upstream from the entrance of the diversion canal on the left bank of the Bang Pakong River. The site is at a point where little sand sediment may occur in the Bang Pakong River upstream of the diversion canal entrance, and the water course runs near the left bank. Furthermore, rather easy linking of the main canals can be secured at this point. Although there is an elementary school and a junior high school about 300 m upstream from the (P2) site, little fear exists for noise pollution from the various works and operations. And selection of (P2) will not bring any trouble for river control works and will be more economical than (P3).