3.1 Baru Pumping Station

3.1.2 Stability Analysis of Gate

Dimension of Gate Body

- Given conditions

Highest tide level:	El 0.45 m
Surplus height from highest tide level:	0.4 m
Design river bed elevation of Baru river:	El -2.40 m
Gate width:	4.0 m
Surplus height for hoisting:	1.0 m
Control room height:	3.0 m

- Thickness of footing (h)

Thickness of footing is assumed to be 1 m.

- Assumed dimension of gate body for stability analysis

Width of pier:	1.0 m
Width of OM bridge:	3.0 m
Length of upper pier:	2.0 m
Length of control room:	4.0 m
Length o lower pier:	9.0 m

(length of control room:4m+width of OM bridge:3m+space for stop log:1m +top & tail of pier:1m = 9 m)

Assumed dimension of gate body is shown in attached figure-1.

Calculation of Load for Stability Analysis

Conditions of stability analysis are both under normal and earthquake condition at the load mentioned in the figure-1

1) Self weight of gate body (calculation details are shown in attached table-1 and figure-2)

Total volume:	153.59 m^3
Unit weight of reinforced concrete:	2.5 t/m^3
Weight of gate body:	383.975 t

(x, y, z) = (5.080, 2.739, 0.000)

2) Mud sediment weight (calculation details are shown in attached table-2 and figure-3)

Sediment area:	10.215 m^2
Unit weight of sediment:	0.6 t/m^3
Thickness of sediment:	1.0 m
Weight of sediment:	6.129 t

(x, y, z) = (10.096, 1.5, 0)

3) Water weight (calculation details are shown in attached table-2 and figure-3)

Area:	10.215 m^2
Unit weight of sediment:	1.0 t/m^3
Water depth:	2.85 m
Weight of sediment:	29.112 t

(x, y, z) = (10.096, 2.425, 0)

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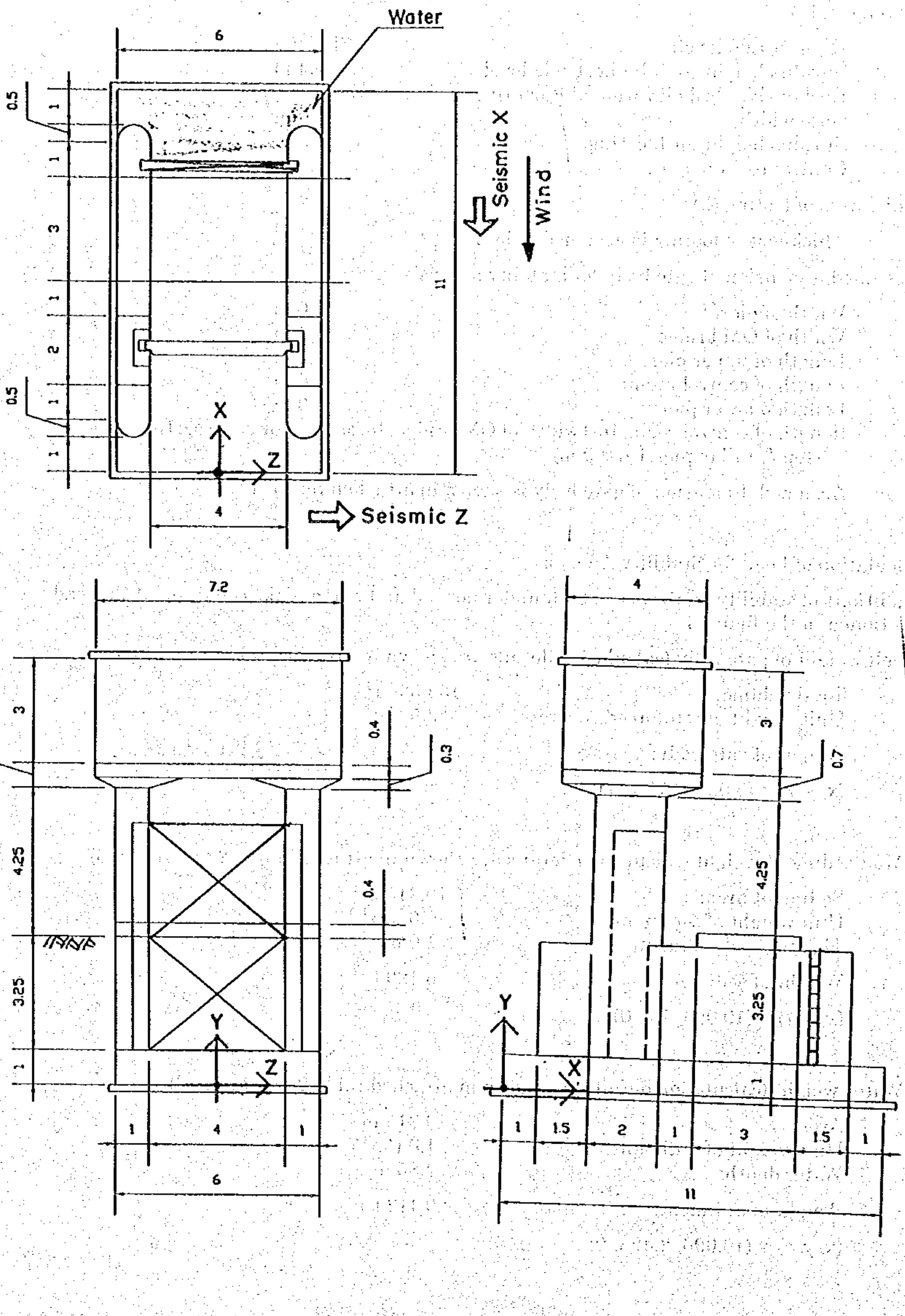

Figure - 1 Condition for Stability Analysis

Table - 1

Self weight of gate body 2.500 :unit weight of rainforced concrete (t/m3)

No.			V(m3)	W(t)	VX	VY	VZ
1	11.520	1	11.520	28.800	100.800	259.200	0.000
2	1.860	2	3.720	9.300	32.550	80.445	0.000
3	2.000	2	4.000	10.000	35.000	80.000	0.000
4	6.208	2	12.415	31.038	108.631	182.345	0.000
5	1.276	2	2.553	6.381	61.977	16.751	0.000
6	1.625	2	3.250	8.125	75.156	21.328	0.000
7	0.683	2	1.365	3.413	29.518	8.958	0.000
8	15.925	2	31.850	79.625	497.656	209.016	0.000
9	0.683	2	1.365	3.413	12.456	8.958	0.000
10	6.500	2	13.000	32.500	81.250	85.313	0.000
11	1.276	2	2.553	6.381	8.218	16.751	0.000
12	0.000	1	0.000	0.000	0.000	0.000	0.000
13	0.000	1	0.000	0.000	0.000	0.000	0.000
14	0.000	1	0.000	0.000	0.000	0.000	0.000
15	0.000	1	0.000	0.000	0.000	0.000	0.000
16	0.000	1	0.000	0.000	0.000	0.000	0.000
17	0.000	1	0.000	0.000	0.000	0.000	0.000
18	0.000	1	0.000	0.000	0.000	0.000	0.000
19	0.000	1	0.000	0.000	0.000	0.000	0.000
20	0.000	1	0.000	0.000	0.000	0.000	0.000
21	0.000	1	0.000	0.000	0.000	0.000	0.000
22	0.000	1	0.000	0.000	0.000	0.000	0.000
23	66.000	1	66.000	165.000	907.500	82.500	0.000
24	0.000	1	0.000	0.000	0.000	0.000	0.000
			153.590	383.975	1,950.713	1,051.564	0.000
					X	Y	Z
					5.080	2.739	0.000

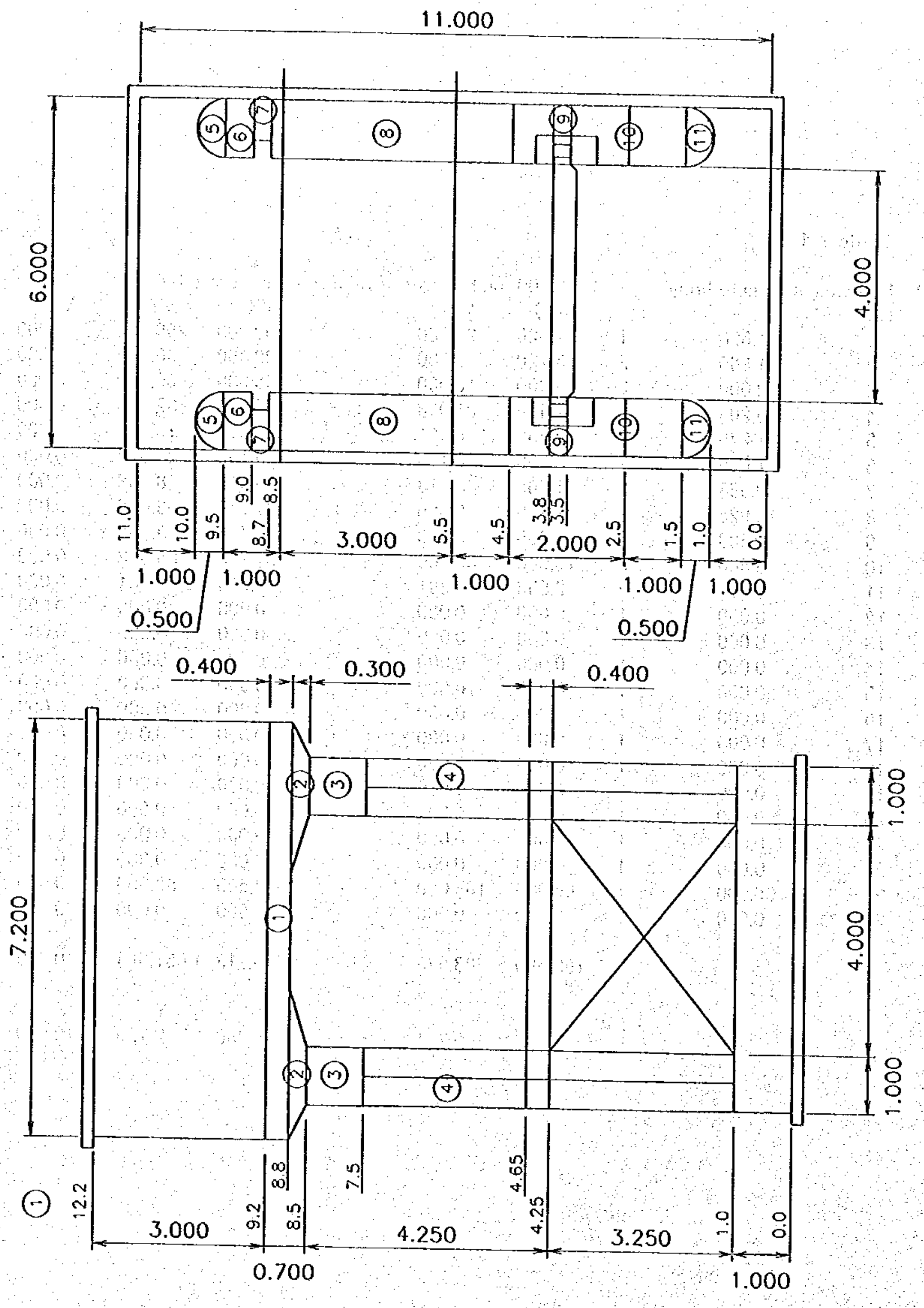

Figure 2. Self-Weight Calculation

Table - 2 Water Weight and Sediment Weight

Water and Sediment weitht

area

1	1.000	1.500	1.000	1.000	1.500	2.000	3.000
2	3.142	0.500	0.500	0.500	0.393	-2.000	-0.785
3	4.000	2.000	1.000	1.000	8.000	1.000	8.000
4							
							10.215

water depth 2.85 m unit wiegh 1 t/m3

	area(m2)	Weigit(t)	X	Y	Z	WX	WY	WZ
1	3.000	8.550	10.250	2.425	0.000	87.638	20.734	0.000
2	-0.785	-2.238	9.712	2.425	0.000	-21.740	-5.428	0.000
3	8.000	22.800	10.000	2.425	0.000	228.000	55.290	0.000
4	0.000	0.000				0.000	0.000	0.000
	Σ	29.112				293.898	70.596	0.000
						10.096	2.425	0.000

sediment depth 1 m unit wiegh 0.6 t/m3

	area(m2)	Weigit(t)	X	Y	Z	WX	WY	WZ
1	3.000	1.800	10.250	1.500	0.000	18.450	2.700	0.000
2	-0.785	-0.471	9.712	1.500	0.000	-4.577	-0.707	0.000
3	8.000	4.800	10.000	1.500	0.000	48.000	7.200	0.000
4	0.000	0.000				0.000	0.000	0.000
	Σ	6.129				61.873	9.193	0.000
						10.096	1.500	0.000

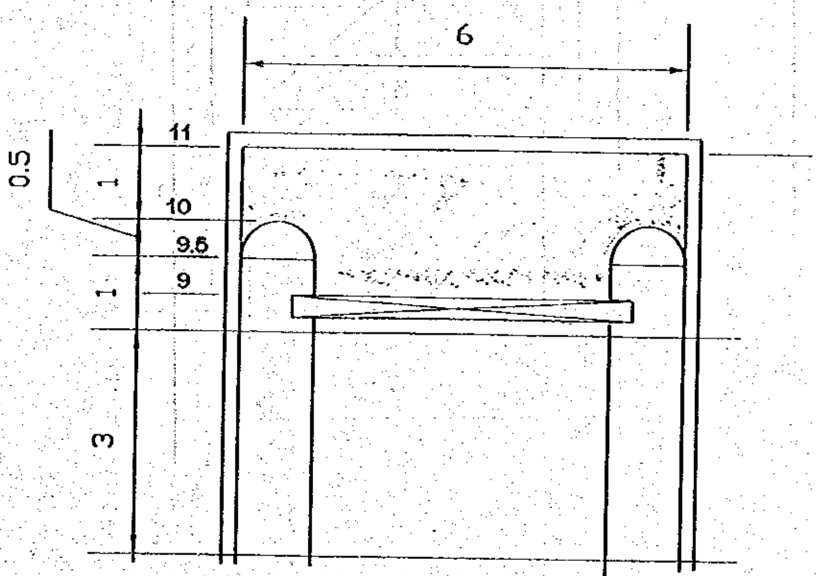

Figure - 3 Area for Water and Sediment Weight

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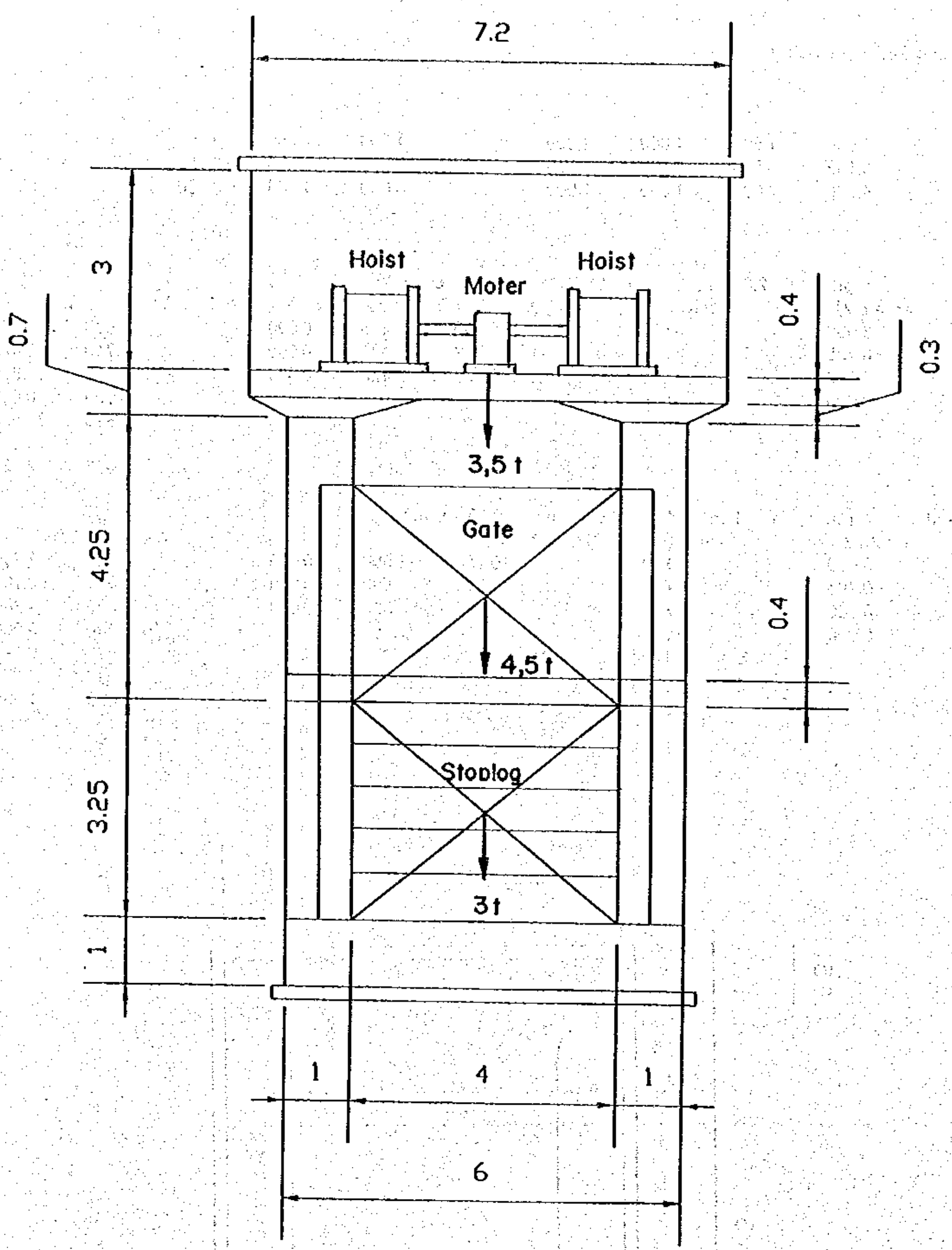


4) Weight of gate, hoist, stop-log etc. (details see figure-4)

Opened gate:	4.5 t	(x, y, z) = (3.65, 5.875, 0)
Stop-log:	3.0 t	(x, y, z) = (8.65, 2.625, 0)
Hoist:	3.5 t	(x, y, z) = (3.65, 9.2, 0)
	(3.5 t/unit x 1 units)	
Weight of control room:	50.0 t	(x, y, z) = (3.5, 10.7, 0)
	(1.5 t/m^2 x 7.2 m x 4 m = 43.2 → 50 t)	
Embedded material:	3.0 t	(x, y, z) = (3.65, 3.6, 0)
	(3.0 t/unit x 1units)	

5) Weight of O/M bridge (details see figure-5)

5)-1 Self-weight

Width of bridge:	3.0 m	
Length of bridge:	6.0 m	
Thickness of bridge:	0.4 m	
Unit weight of reinforced concrete:	2.5 t/m^3	
Self-weight of bridge:	18.0 t	(x, y, z) = (7, 4.45, 0)

5)-2 Weight of O/M equipment

Load area of equipment:	L=4 m x W=3 m	
Unit weight of equipment:	1.0 t/m^2	
Weight of O/M equipment	12.0 t	(x, y, z) = (7, 4.65, 0)

6) Weight of stairway

Area for stairway:	6.0 m^2	
Unit weight:	0.4 t/m^2	
Weight of stairway	2.4 t	(x, y, z) = (3.5, 6.725, 4)

7) Uplift

7)-1 Length of sheet pile against piping

Creep ratio is calculated by Lane's formula as follows: $C \leq \dfrac{\frac{1}{3}\Sigma L + \Sigma l}{\Delta H}$

assuming C = 8.5 (very fine sand or silt)
L = 11.0 m
Σl = 1 m x 2 + 4l

Necessity length of sheet pile against piping: $l \geqq 4.64$ m

→ l = 5.0 m

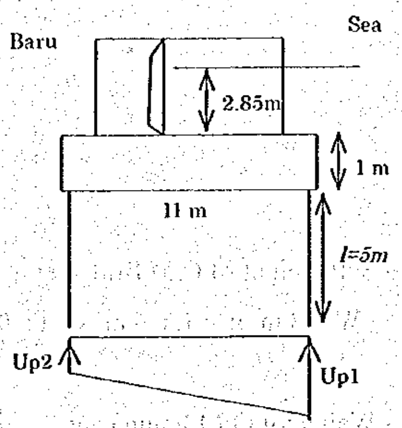


Figure-6

7)-2 Calculation of uplift

Uplift at the point of x calculated by following formula:

$$Upx = (h_2 + \Delta h \cdot \mu \cdot \frac{\Sigma l - lx}{\Sigma l} + d_2) \cdot W_0$$

h_2=0, Δh=2.85 m, μ=1.00, d_2=1.5, W_0=1.00
Σl = 1.0 x 2 + 5 x 4 + 11 = 33 m

- Sea side (Up1)

Up1 = 2.85 x 1 x (33-11)/33 + 1
= 2.90 t/m^2

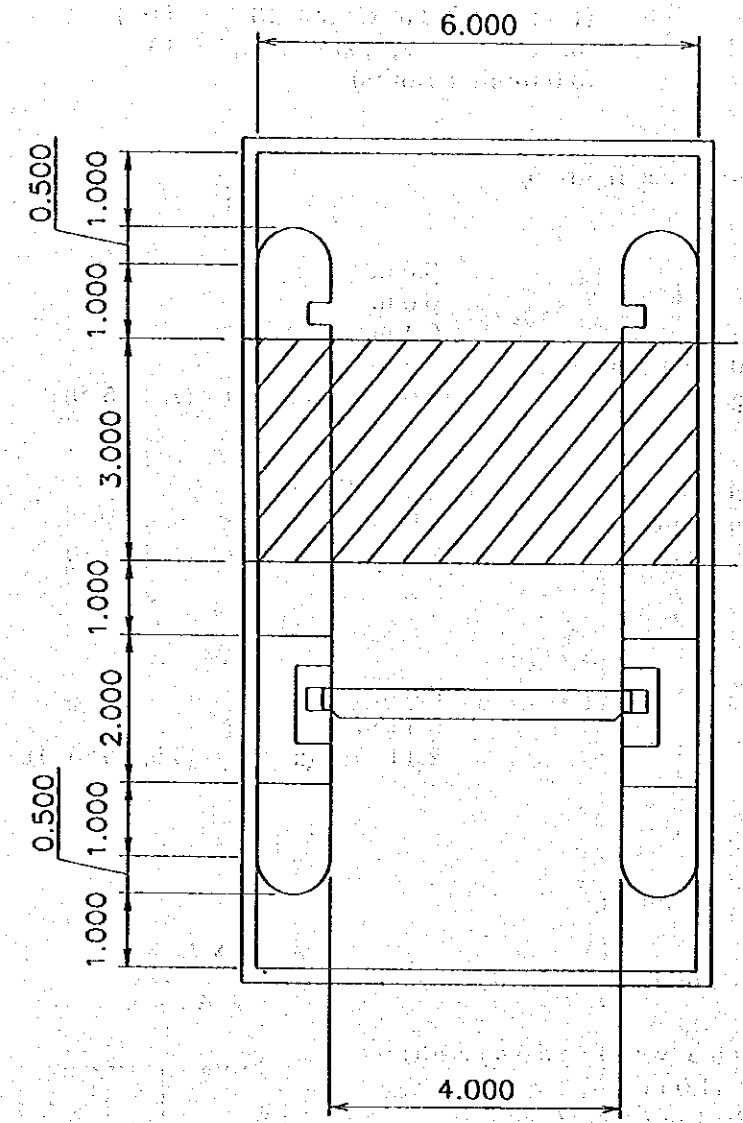


Figure 5. O/M Bridge

– Self-weight of O/M Bridge

W : 3 m x L : 6 m x t : 0.4 m x $2.5\ ^{t}/m^3 = 18^{t}$

$(x, y, z) = (7, 4.45, 0)$

– Weight of O/M Equipment

W : 3 m x L : 4 m x $1.0\ ^{t}/m^2 = 12^{t}$

$(x, y, z) = (7, 4.65, 0)$

- Baru river side (Up2)

Up2 = 2.85 x 1 x (33-22)/33 + 1.0
= 1.95 t/m^2

Up = (1/2) x (2.9+1.95) x 11 x 6 = 160.05 t

X= 5.14

8) Static water pressure

Static water pressure (SWP) of gate axis direction is balanced, therefore only SWP of flow direction is considered.

8)-1 Water level under normal condition

Wave pressure caused by wind is assumed as a static water pressure of the wave height.

Wave height caused by wind is calculated by following formula.

hw = 0.00077 x V x F$^{1/2}$

hw: wave height (m), V: 10 minutes average wind velocity (m/sec)
blow distance (m)

V: 40 km/hr = 11.1 m/sec → 11.1 x 1.5 (safety factor) = 16.7 m/sec → 20 m/sec
F: 100 m (Sea side)

hw = 0.00077 x 20 x 100$^{1/2}$ = 0.154 m → 0.16 m

8)-2 Water level under earthquake condition

Wave pressure caused by earthquake is is assumed as a static water pressure of a half of the wave height.

he = (K τ x $\sqrt{}$(g H_0))/ π

he: wave height (m), K: horizontal earthquake factor, τ : period of earthquake (1.0 sec)
H_0: water depth (m), g: acceleration of gravity (9.8 m/sec^2)

K: K=ad / g

Ad = n (ac x z)^m = 109.508 cm/sec^2
ad: design shock acceleration (cm/sec^2)
ac: basic shock acceleration (160 cm/s^2: 100-year return period)
z: factor depending on geographic condition (0.56)
n, m: coefficients for soil types (n=0.29, m=1.32: soft alluvium)

Therefore , K = 109.508 / 980 = 0.11

he/2 = ((0.11 x 1.0 x $\sqrt{}$(9.8 x 2.86))/ π)/2 = 0.09 m

8)-3 Static water pressure under normal condition (details see figure-7)

P = -45.24 t y = 1.42

8)-4 Static water pressure under earthquake condition (details see figure-7)

P = -43.57 t y = 1.40

9) Dynamic water pressure (DWP) under earthquake condition

DWP is calculated following formula: $Pd = \frac{7}{12} \cdot W_o \cdot Kh \cdot H^{1/2} \cdot h^{1/2}$ $y = \frac{2}{5} \cdot h$

Pd: dynamic water pressure at h
W_0: unit weight of water (1.0 t/m^3)
Kh: horizontal earthquake factor (0.11)

(Normal)

S.W.P to Gate + Pier (P_1)

$P_1 = -½ \times 3.01 \times 3.01 \times 1.0 \times 6 = -27.18$

$Y = 3.01 \times \frac{1}{3} + 1 = 2.00$

S.W.P to Footing (P_2, P_3)

$P_2 = -½ (3.01 + 4.01) \times 1.0 \times 1.0 \times 6 = 3$

$= -21.06$

$Y = 1 - \frac{2 \times 3.01 + 4.01}{3.01 + 4.01} \times \frac{1}{3} = 0.52$

$P_3 = ½ \times 1.0 \times 1.0 \times 1.0 \times 6 = 3$

$Y = 0.33$

Total S.W.P

$P = -27.18 + (-21.06) + 3 = -45.24$

$Y = \frac{-27.18 \times 2 - 21.06 \times 0.52 + 3 \times 0.33}{-45.24}$

$= 1.42$

(Siesmic)

$P_1 = -½ \times 2.94 \times 2.94 \times 1.0 \times 6 = -25.93$

$Y = 1.98$

$P_2 = -½ \times (2.94 + 3.94) \times 1.0 \times 1.0 \times 6$

$= -20.64$

$Y = 0.52$

$P_3 = 3 \quad Y = 0.33$

$P = -43.57$

$Y = 1.40$

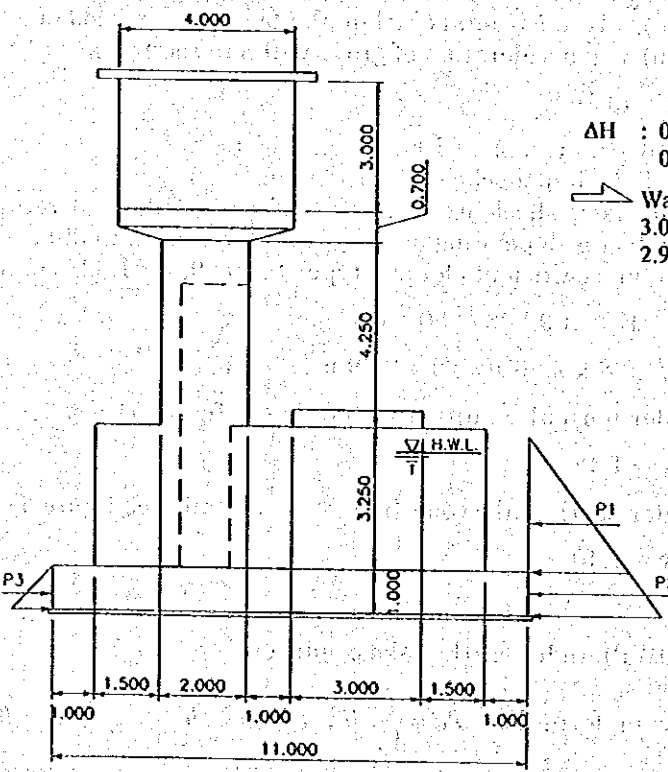
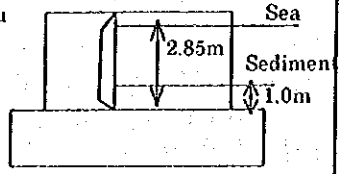


Figure 7. Static Water Pressure

H: total effective depth (m)
H = 2.85 (total depth) - 1.00 (sediment) = 1.85

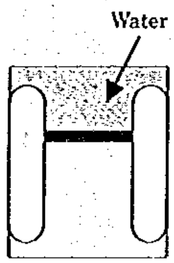


9)-1 Direction of earthquake load Z

$Pd = 7/12 \times 1.0 \times 0.11 \times 1.85^{1/2} \times 1.85^{3/2} \times 1.0 \times 2$
$= 0.44$ t

$Y = 2/5 \times 1.85 + 1 + 1 = 2.74$ m

9)-2 Direction of earthquake load X



$Pd = 7/12 \times 1.0 \times 0.11 \times 1.85^{1/2} \times 1.85^{3/2} \times 6 = 1.32$ t

Y = 2.74 m

10) Sediment pressure

pe = Ce x W x d (t/m)

Pe: sediment pressure
Ce: coefficient of sediment pressure (0.6: silt)
W: unit weight of seidment (0.6 t/m^3: submarged loose clayly soil)
d: sediment thickness (1 m)

$Pe = 1/2 \times (0.6 \times 0.6 \times 1) \times 1 \times 6 = 1.08$ t $Y = 1.0 + 1/3 = 1.33$

11) Wind load

11)-1 Wind pressure

$Pa = V^2/16$

Pa: wind pressure (kg/m^2)
V: wind verosity (km/hr)   V = 40 km/hr

$Pa = 40 \times 40 / 16 \times 1.5$ (safety factor) $= 150\ kg/m^2$

11)-2 Wind load (details see table-3 and figure-9)

Wind direction: -X (Sea to Baru)

Pw = Pa x A (t)

A: Area above HWL (49.46 m^2)

$Pw = 49.46 \times 0.150 = 7.419$ t,   Y = 8.222

12) Earth pressure

γ : unit weight of soil (1.9 t/m^3: embankment sandy soil)
γ': unit weight of submarged soil (0.9 t/m^3: embankment sandy soil)
ϕ : internal friction angle (30° assumed internal friction angle of embankment sandy soil)
C: soil cohesion (0)
ground water level (GWL): 0.6 m below ground level (GL)

12)-1 Earth pressure under normal condition

$pa = Ka \times \gamma \times h\ (+ Ka \times q)$

pa: acting earth pressure (tf/m^2)
Ka: coefficient of acting earth pressure
γ : unit weight of soil (tf/m^3)
h: earth depth to acting point (m)
q: surcharge in normal condition (tf/m^2)

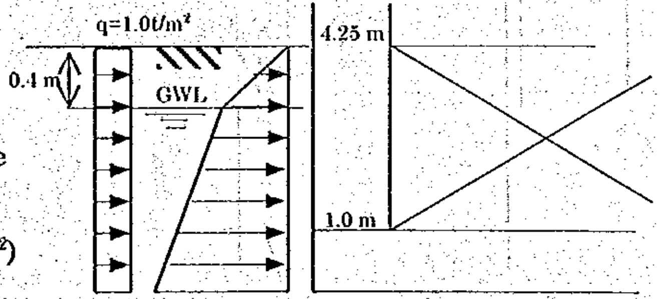


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Table - 3

wind load

Area No.							A(m2)
1	7.2	3.4	1	1	24.480	1	24.480
2	3.6	0.3	0.5	1	0.540	2	1.080
3					0.000	1	0.000
4	1	3.85	1	1	3.850	2	7.700
5	6	0.8	1	1	4.800	1	4.800
6	4	2.85	1	1	11.400	1	11.400
7					0.000	1	0.000
							49.460

Wind load 0.15 t/m2

No.	A(m2)	Ww	X	Y	Z	WX	WY	WZ
1	24.480	3.672		10.500	0.000	0.000	38.556	0.000
2	1.080	0.162		8.650	0.000	0.000	1.401	0.000
3	0.000	0.000			0.000	0.000	0.000	0.000
4	7.700	1.155		6.575	0.000	0.000	7.594	0.000
5	4.800	0.720		4.250	0.000	0.000	3.060	0.000
6	11.400	1.710		6.075	0.000	0.000	10.388	0.000
7	0.000	0.000				0.000	0.000	0.000
Σ	49.460	7.419				0.000	61.000	0.000
						0.000	8.222	0.000

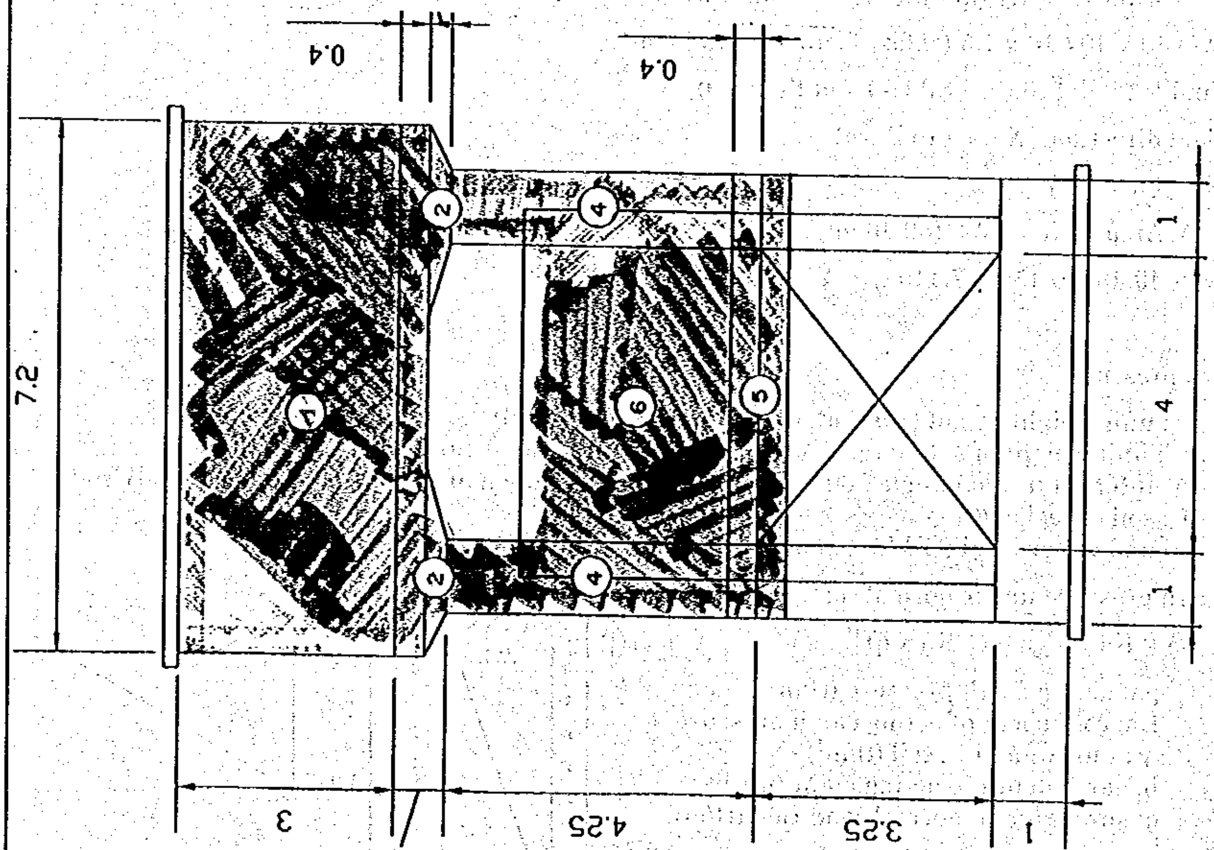


Figure - 9

$$Ka = \frac{\cos^2(\phi-\theta)}{\cos^2\theta\cdot\cos(\theta+\delta)\cdot\left(1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\alpha)}{\cos(\theta+\delta)\cdot\cos(\theta-\alpha)}}\right)^2}$$

θ =0°, ϕ =30°, α =0, $\delta = \phi/3$=10°

$$Ka = \frac{\cos^2 30}{\cos^2 0\cdot\cos 10\cdot\left(1+\sqrt{\frac{\sin 40\cdot\sin 30}{\cos 10\cdot\cos 0}}\right)^2} = 0.308$$

-Earth pressure at GWL (pa1)

pa1 = 0.308 x 1.9 x 0.4 = 0.23 tf
Pa1= 1/2 x 0.23 x 0.4 x 8 = 0.37 tf Y = 3.98

-Earth pressure at bottom (pa2)

pa2 = 0.308 x 0.9 x (4.25-0.4) + 0.23 = 1.30 tf
Pa2= 1/2 x (0.23+1.30) x 3.85 x 8 = 23.56 tf Y = 1.48

-Total earth pressure Pa

Pa = Pa1 + Pa2 = 23.93 tf
Y = (3.98 x 0.37 + 1.48 x 23.93) / 23.93 = 1.52 X=5.5

*Horizontal force: Pa_H = Pa x cos δ = 23.93 x cos10 = 23.57 tf
*Vertical force: Pa_V = Pa x sin δ = 23.93 x sin10 = 4.16 tf

-Pressure caused by surcharge (Pa3)

q = 1 tf/m2
Pa3 = Ka x q x H x L = 0.308 x 1.0 x 4.25 x 8 = 10.47 tf Y=2.125

*Horizontal force: $Pa3_H$ = Pa3 x cos δ = 10.47 x cos10 = 10.31 tf
*Vertical force: $Pa3_V$ = Pa3 x sin δ = 10.47 x sin10 = 1.82 tf

12)-2 Earth pressure under earthquake condition

pae = Kea x γ x h – 2C$\sqrt{\text{Kea}}$ + Kea x q'

pae: acting earth pressure (tf/m²)
Kae: coefficient of acting earth pressure, γ : unit weight of soil (tf/m³)
h: earth depth to acting point (m), C: soil cohesion (tf/m²)
q': surcharge (tf/m²)

$$Kae = \frac{\cos^2(\phi-\theta_e-\theta)}{\cos\theta_e\cdot\cos^2\theta\cdot\cos(\theta+\theta_e+\delta)\cdot\left(1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\alpha-\theta_e)}{\cos(\theta+\theta_e+\delta)\cdot\cos(\theta-\alpha)}}\right)^2}$$

θ = 0, ϕ = 30°, δ =0, α =0
tan θ_0 = Kh / (1-Kv) = 0.11/(1-0) = 0.11, θ_0= 6.3

$$Kae = \frac{\cos^2 23.7}{\cos 6.3\cdot\cos^2 0\cdot\cos 6.3\cdot\left(1+\sqrt{\frac{\sin 30\cdot\sin 23.7}{\cos 6.3\cdot\cos 0}}\right)^2} = 0.404 \quad \text{(above GWL)}$$

Kae' = 0.500 (below GWL)

Kh' = Kh x (γ / (1- γ)) = 0.11 x (1.9 / (1.9-1)) = 0.23
γ ': unit weight of submarged soil (tf/m³)
tan= 0.23, θ_0=13.0

-Earth pressure at GWL (pae1)

pae1 = 0.404 x 1.9 x 0.4 = 0.31 tf
Pae1= 1/2 x 0.31 x 0.4 x 8 = 0.50 tf Y = 3.98

-Earth pressure at bottom (pae2)

pae2 = 0.500 x 0.9 x (4.25-0.4) + 0.31 = 2.04 tf
Pae2= 1/2 x (0.31+2.04) x 3.85 x 8 = 36.19 tf Y = 1.45

-Total earth pressure Pa

Pae = Pae1 + Pae2 = 36.69 tf
Y = 1.48 X=5.5

*Horizontal force: Pae_H = Pae x cos δ = 36.69 x cos0 = 36.69 tf
*Vertical force: Pae_V = Pae x sin δ = 36.69 x sin0 = 0 tf

<u>Total Calculated Load and Moment</u> (details see table-4)

1) Normal condition

Total vertical load and moment: ΣV = 361.55 tf, ΣMx = 1849.44 tm
Total horizontal load and moment: ΣH = 63.53 tf, ΣMz = 49.40 tm
Acting point of compsition of forces: (x, z) = (5.12, 0.14)

2)Earthquake condition

2)-1 Earthquake condition: Z

Total vertical load and moment: ΣV = 357.57 tf, ΣMx = 1880.79 tm
Total horizontal load and moment: ΣH = 100.81 tf, ΣMz = 263.01 tm
Acting point of compsition of forces: (x, z) = (5.29, 0.74)

2)-2 Earthquake condition: X

Total vertical load and moment: ΣV = 359.73 tf, ΣMx = 1701.06 tm
Total horizontal load and moment: ΣH = 101.98 tf, ΣMz = 32.95 tm
Acting point of compsition of forces: (x, z) = (4.73, 0.09)

<u>Stability against Overturning</u> (details see table-4)

1) Normal condition

Acting point of compsition of forces: (x,z) = (5.12, 0.14)

Distance between center of footing and acting point (r):

$r = \sqrt{(ex^2+ez^2)} = \sqrt{((5.12-5.5)^2+0.14^2)} = 0.41$
ex, ez: deviation from footing center to acting point along X, Z axis

Stability against overturning: $r_{max} = \frac{1}{6} \cdot \frac{L^2 \cdot \sin^2\theta + B^2 \cdot \cos^2\theta}{L \cdot \sin\theta + B \cdot \cos\theta} > r$

L: 11 m, B: 6 m, tan θ = ex/ez, θ = 1.23

r_{max} = 1.50 > 0.41 → OK

Table - 4 (1)

Construction completed, Normal condition, Maintenace Normal

No.	Item		Vertical force V (t)	X (m)	Y(m)	Z (m)	V*X	V*Z	Holizontal (flow direction) Hx (t)	Y (m)	Hx*Y	Holizontal (Gate axis direciton) Hz (t)	Y (m)	Hz*Y
1)	Self-weight of gate body		383.98	5.08	2.74	0.00	1,950.59	0.00			0.00			0.00
2)	Sediment weight		6.13	10.10	1.50	0.00	61.88	0.00			0.00			0.00
3)	Water weight		29.11	10.10	2.43	0.00	293.91	0.00			0.00			0.00
4)	Gete, Hoist and Misc.						0.00	0.00			0.00			0.00
		Opened gate	4.50	3.65	5.88	0.00	16.43	0.00			0.00			0.00
		Closed gate					0.00	0.00			0.00			0.00
		Stop log	3.00	8.65	2.63	0.00	25.95	0.00			0.00			0.00
		Hoist weight	3.50	3.65	9.20	0.00	12.78	0.00			0.00			0.00
		Control room	50.00	3.50	10.70	0.00	175.00	0.00			0.00			0.00
		Embedded material	3.00	3.65	3.60	0.00	10.95	0.00			0.00			0.00
5)	O/M bridge						0.00	0.00			0.00			0.00
		Self-weight of bridge	18.00	7.00	4.45	0.00	126.00	0.00			0.00			0.00
		Weight of O/M equipment	12.00	7.00	4.65	0.00	84.00	0.00			0.00			0.00
6)	Stairwaty		2.40	3.50	6.73	4.00	8.40	9.60			0.00			0.00
7)	Uplift		-160.05	5.14	0.00	0.00	-822.66	0.00			0.00			0.00
8)	Static water pressure						0.00	0.00			0.00			0.00
		nomal					0.00	0.00	-45.24	1.42	-64.24			0.00
10)	Sediment pressure						0.00	0.00	-1.08	1.33	-1.44			0.00
11)	Wind load						0.00	0.00	-7.42	8.22	-61.00			0.00
12)	Earth pressure						0.00	0.00			0.00			0.00
	nomal	earth pressure	4.16	5.50	1.52	-3.00	22.88	-12.48			0.00	23.57	1.52	35.83
		pressure by surcharge	1.82	5.50	2.13	-3.00	10.01	-5.46			0.00	10.31	2.13	21.91
		Total	361.55				1,976.12	-8.34	-53.74		-126.68	33.88		57.74
							5.47	-0.02			2.36			1.70

ΣV	ΣH	ΣMx	ΣMz	X	Z
361.55	63.53	1,849.44	49.40	5.12	0.14

ex	ez	r
0.38	0.14	*0.41*

ex/ez	θ	
2.82	1.23	70.44

sin θ	sin2 θ	cos θ	cos2 θ
0.94	0.89	0.33	0.11

rmax *1.50* 1.09

Table - 4(2)

Construction completed, Earthquake Condition, Maintenace Seismic Z

		Vertical force						Holizontal (flow direction)			Holizontal (Gate axis direciton)		
		V (t)	X (m)	Y(m)	Z (m)	V*X	V*Z	Hx (t)	Y (m)	Hx*Y	Hz (t)	Y (m)	Hz*Y
1)	Self-weight of gate body	383.98	5.08	2.74	0.00	1,950.59	0.00			0.00	42.24	2.74	115.69
2)	Sediment weight	6.13	10.10	1.50	0.00	61.88	0.00			0.00	0.67	1.50	1.01
3)	Water weight	29.11	10.10	2.43	0.00	293.91	0.00			0.00			0.00
4)	Gete, Hoist and Misc.					0.00	0.00			0.00	0.00	0.00	0.00
	Opened gate	4.50	3.65	5.88	0.00	16.43	0.00			0.00	0.50	5.88	2.91
	Closed gate					0.00	0.00			0.00	0.00	0.00	0.00
	Stop log	3.00	8.65	2.63	0.00	25.95	0.00			0.00	0.33	2.63	0.87
	Hoist weight	3.50	3.65	9.20	0.00	12.78	0.00			0.00	0.39	9.20	3.54
	Control room	50.00	3.50	10.70	0.00	175.00	0.00			0.00	5.50	10.70	58.85
	Embedded materials	3.00	3.65	3.60	0.00	10.95	0.00			0.00	0.33	3.60	1.19
5)	O/M bridge					0.00	0.00			0.00	0.00	0.00	0.00
	Self-weight of bridge	18.00	7.00	4.45	0.00	126.00	0.00			0.00	1.98	4.45	8.81
	Weight of O/M equipment	12.00	7.00	4.65	0.00	84.00	0.00			0.00	1.32	4.65	6.14
6)	Stairwaty	2.40	3.50	6.73	4.00	8.40	9.60			0.00			0.00
7)	Uplift	−160.05	5.14	0.00	0.00	−822.66	0.00			0.00			0.00
8)	Static water pressure					0.00	0.00			0.00			0.00
	seismic					0.00	0.00	−43.57	1.40	−61.00			0.00
9)	Dynamic water pressure					0.00	0.00			0.00	0.44	2.74	1.21
10)	Sediment pressure					0.00	0.00	−1.08	1.33	−1.44			0.00
11)	Wind load					0.00	0.00			0.00			0.00
12)	Earth pressure					0.00	0.00			0.00			0.00
	earthquake	0.00	0.00	0.00	0.00	0.00	0.00			0.00	36.69	1.45	53.20
	Total	355.57				1,943.23	9.60	−44.65		−62.43	90.38		253.41
						5.47	0.03			1.40			2.80

ΣV	ΣH	ΣMx	ΣMz	X	Z
355.57	100.81	1,880.79	263.01	5.29	0.74

ex	ez	r
0.21	0.74	*0.77*

ex/ez		θ
0.28	0.28	15.88

sin θ	sin2 θ	cos θ	cos2 θ
0.27	0.07	0.96	0.93

rmax *1.61* 0.84

Table - 4 (3)

Construction completed, Earthquake Condition, Maintenace   Seismic X

No.	Item	Vertical force V (t)	X (m)	Y(m)	Z (m)	V*X	V*Z	Holizontal (flow direction) Hx (t)	Y (m)	Hx*Y	Holizontal (Gate axis direciton) Hz (t)	Y (m)	Hz*Y
1)	Self-weight of gate body	383.98	5.08	2.74	0.00	1,950.59	0.00	-42.24	2.74	-115.69			0.00
2)	Sediment weight	6.13	10.10	1.50	0.00	61.88	0.00	-0.67	1.50	-1.01			0.00
3)	Water weight	29.11	10.10	2.43	0.00	293.91	0.00			0.00			0.00
4)	Gete, Hoist and Misc.					0.00	0.00			0.00			0.00
	Opened gate	4.50	3.65	5.88	0.00	16.43	0.00	-0.50	5.88	-2.91			0.00
	Closed gate									0.00			0.00
	Stop log	3.00	3.65	2.63	0.00	25.95	0.00	-0.33	2.63	-0.87			0.00
	Hoist weight	3.50	3.65	9.20	0.00	12.78	0.00	-0.39	9.20	-3.54			0.00
	Control room	50.00	3.50	10.70	0.00	175.00	0.00	-5.50	10.70	-58.85			0.00
	Embedded materials	3.00	3.65	3.60	0.00	10.95	0.00	-0.33	3.60	-1.19			0.00
5)	O/M bridge					0.00	0.00			0.00			0.00
	Self-weight of bridge	18.00	7.00	4.45	0.00	126.00	0.00	-1.98	4.45	-8.81			0.00
	Weight of O/M equipment	12.00	7.00	4.65	0.00	84.00	0.00	-1.32	4.65	-6.14			0.00
6)	Stairwaty	2.40	3.50	6.73	4.00	8.40	9.60			0.00			0.00
7)	Uplift	-160.05	5.14	0.00	0.00	-822.66	0.00			0.00			0.00
8)	Static water pressure					0.00	0.00			0.00			0.00
	seismic					0.00	0.00	-43.57	1.40	-61.00			0.00
9)	Dynamic water pressure					0.00	0.00	-1.32	2.74	-3.62			0.00
10)	Sediment pressure					0.00	0.00	-1.08	1.33	-1.44			0.00
11)	Wind load					0.00	0.00			0.00			0.00
12)	Earth pressure					0.00	0.00			0.00			0.00
	normal	4.16	5.50	1.52	-3.00	22.88	-12.48			0.00	23.57	1.52	35.83
	Total	359.73				1,966.11	-2.88	-99.22		-265.05	23.57		35.83
						5.47	-0.01			2.67			1.52

ΣV	ΣH	ΣMx	ΣMz	X	Z
359.73	101.98	1,701.06	32.95	4.73	0.09

ex	ez	r
0.77	0.09	*0.78*

ex/ez		θ
-8.42	-1.45	-83.23

sin θ	sin2 θ	cos θ	cos2 θ
-0.99	0.99	0.12	0.01

rmax		
	-3.91	-4.69

2) Earthquake condition

2)-2 Earthquake direciton- Flow (Z)

Acting point of compsition of forces: (x, z) = (5.29, 0.74)
Distance between center of footing and acting point (r):
r = 0.77

Stability against overturning: $$r_{max} = \frac{1}{3}\cdot\frac{L^2\cdot\sin^2\theta + B^2\cdot\cos^2\theta}{L\cdot\sin\theta + B\cdot\cos\theta} > r$$

L: 11 m, B: 6 m, $\tan\theta$ = ex/ez, θ = 0.28

r_{max} = 1.61 > 0.77 → OK

2)-2 Earthquake direciton- Gate Axis (X)

Acting point of compsition of forces: (x, z) = (4.73, 0.09)
Distance between center of footing and acting point (r):
r = 0.78

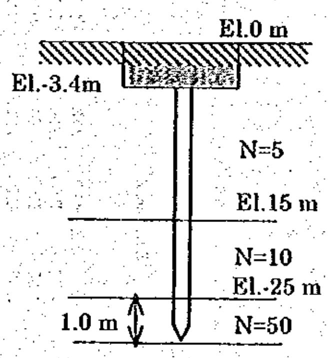
L: 11 m, B: 6 m, $\tan\theta$ = ex/ez, θ = 0.78

r_{max} = 3.43 > 0.78 → OK

Pile Foundation

1) N-value for design of pile foundation

Geological condition at the site is assumed as shown in figure-11.



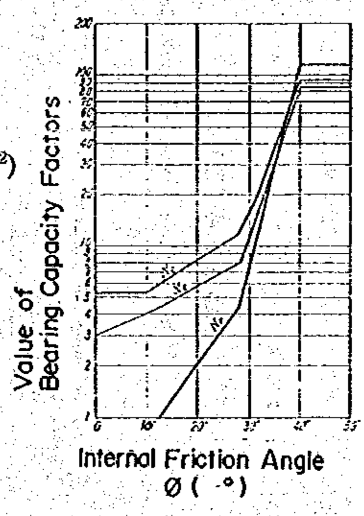
N-value at pile tip (Nt): 50
average N-value 3.75D above the tip to pile tip (N2):
3.75 D = 3.75 x 0.5 = 1.875
N2 = (0.875 x 10 + 1.0 x 50) / 1.875 =31.3 → 31

N-value for pile design (N): N = (50+31)/2 = 40.5 → 40

2) Estimation of internal friction angle

$\phi = 15 + \sqrt{(15 \times N)} = 15 + \sqrt{(15 \times 40)} = 39.5 \rightarrow 40$

3) Allowable compressive bearing capacity (Ra)



Ra = { qd x A + u Σ(li x fi)}/SF

qd: ultimate bearing capacity per unit are at pile tip (tf/m²)
A: Area of pile tip (= $\pi R^2/4$ = 0.196 m²)
li: stratum depth (l_1= 11.6 m, l_2= 10 m, l_3= 1 m)
u: circumferential length of pile (= 1.571 m)
fi: maximum skin firiction of stratum
(f_1= 2.5 tf/m², f_2= 3 tf/m², f_3= 3 tf/m²)
SF: safety factor (normal: 3, earthquake:2)

- ultimate bearing capacity (qd)

$$qd = 1.3 \times c \times Nc + 0.3 \times R \times \gamma_1 \times N_\gamma + \gamma_2 \times Df \times Nq$$

c: cohesion (= 0)
Nc, N_γ, Nq: bearing capacity foctors
Nc = 92, N_γ= 110, Nq = 85
γ_1: unit weight of soil below pile tip (= 0.8 tf/m³)

γ_2: unit weight of soil above pile tip (= 0.8 tf/m^3)
R: diameter of pile (= 0.5 m)
Df: Pile rength (= 22.6 m)

qd = 0.3 x 0.5 x 0.8 x 110 + 0.8 x 22.6 x 85 = 1516 tf/m^2

Ra = {1550 x 0.196 + 1.571 x (11.6 x 2.5 + 10 x 3 + 1 x 3)}/SF = 401.2 / SF

-Noramal condition: Ra = 133.7 tf
-Earthquake condition: Ra = 200.6 tf

4) Allowable pull-out capacity (Pa)

Pa = Pu / SF + w

Pu: ultimate axial pull-out capacity of pile determined by ground conditions (tf)
Pu = U Σ (li x fi) = 97.4
w: effective weight of pile (= 1.6 tf/m x 22.6 m = 36.16 tf)
SF: safety factor (normal: 6, earthquake:3)
-Noramal condition: Pa = 52.39 tf
-Earthquake condition: Pa = 68.63 tf

5) Allowable lateral bearing capacity (Ha)

Ha = (k x D / β) x δ a

k: coefficient of lateral reaction of foundation ground (kgf/cm^3)
D: pile diameter(= 0.5 m)
β : charactaristic value of pile (cm^{-1})

$$\beta = \sqrt[4]{\frac{k \cdot D}{4 \cdot E \cdot I}}$$

E: coefficient of elasticity of pile body (= 400,000 kgf/cm^2)
I: momet of interia of corss section of pile body (= 260,604.6 m^4)
δ a: allowable displacement of pile (normal: 1.0 cm, earthquake: 1.5 cm)

5)-1 Estimation of coefficient of lateral reaction of foundation ground (k)

$k = k_0(B_H/30)^{-3/4}$

k_0 = 1/30 x α x E_0,
E0 = 28N = 140, α =1 (normal), α =2 (earthquake)
$B_H = \sqrt{(D/\beta)}$
D = 0.5 m

k = 1.70 (normal condition)
k = 3.40 (earthquake condition) (details see table-5)

5)-2 Allowable lateral bearing capacity (Ha)

Ha = (k x D / β) x δ a

K: 1.70 kgf/cm^3 (normal), 3.40 kgf/cm^3 (earthquake)
D: pile diameter(= 50 cm)
β : 0.0035 cm^{-1}, 1/β = 264.5 cm
: allowable displacement of pile (normal: 1.0 cm, earthquake: 1.5 cm)

Ha = 22.5 t (normal),
Ha = 45.0 t (earthquake)

Table – 5

estimate of lateral reaction of foundation

K	1.341591	K	1.665043	K	1.699102	K	1.702331	K	1.702634	K	1.702663	K	1.702665	K	1.702665	K	1.702665
K0	4.666667	K0	4.666667	K0	4.666667	K0	4.666667	K0	4.666667	K0	4.666667	K0	4.666667	K0	4.666667	K0	4.666667
BH	158.1139	BH	118.5486	BH	115.3907	BH	115.099	BH	115.0717	BH	115.0691	BH	115.0689	BH	115.0689	BH	115.0689
α	1	α	1	α	1	α	1	α	1	α	1	α	1	α	1	α	1
E0	140	E0	140	E0	140	E0	140	E0	140	E0	140	E0	140	E0	140	E0	140
φ	50	φ	50	φ	50	φ	50	φ	50	φ	50	φ	50	φ	50	φ	50
d	50	d	50	d	50	d	50	d	50	d	50	d	50	d	50	d	50
β	0.002	β	0.003558	β	0.003755	β	0.003774	β	0.003776	β	0.003776	β	0.003776	β	0.003776	β	0.003776
1/β	500	1/β	281.0755	1/β	266.3003	1/β	264.9556	1/β	264.8299	1/β	264.8181	1/β	264.817	1/β	264.8169	1/β	264.8169
	0.133970	k	1.341591	k	1.665043	k	1.699102	k	1.702331	k	1.702634	k	1.702663	k	1.702665	k	1.702665
E	400000	E	400000	E	400000	E	400000	E	400000	E	400000	E	400000	E	400000	E	400000
/	261675	/	261675	/	261675	/	261675	/	261675	/	261675	/	261675	/	261675	/	261675
1/β	[illegible]500																
β	0.002																
k	0.133978	k	1.341591	k	1.665043	k	1.699102	k	1.702331	k	1.702634	k	1.702663	k	1.702665	k	1.702665
N	[illegible]	N	5	N	5	N	5	N	5	N	5	N	5	N	5	N	5

6) Load and moment for a pile

6)-1 Load and moment at footing center

(normal condition)

- flow direction (X) Vertical: V = 361.55 t
  Mxv = 323.05 x (5.47-5.5) = -9.7 tm
  Horizontal: Hx = -53.74 t
  Mxh = -126.7 tm
  ΣMx = -9.7 – 126.7 = -136.4 tm
- Gete axis direction (Z) Vertical: V = 361.55 t
  Mzv = -8.3 tm
  Horizontal: Hz= 33.88 t
  Mzh = 57.7 tm
  ΣMz= 49.4 tm

(earthquake condition: Z)

- flow direction (X) Vertical: V = 355.57 t
  Mxv = -10.7 tm
  Horizontal: Hx = -44.65 t
  Mxh = -62.4 tm
  ΣMx = 73.1 tm
- Gete axis direction (Z) Vertical: V = 355.57 t
  Mzv = 9.6 tm
  Horizontal: Hz= 90.38 t
  Mzh = 253.4 tm
  ΣMz = 263.0 tm

(earthquake condition: X)

- flow direction (X) Vertical: V = 359.73 t
  Mxv = -10.8 tm
  Horizontal: Hx = -99.22 t
  Mxh = -265.1 tm
  ΣMx = -275.9 tm
- Gete axis direction (Z) Vertical: V = 359.73 t
  Mzv = -2.9 tm
  Horizontal: Hx = 23.57 t
  Mzh = 35.8 tm
  ΣMz = 32.9 tm

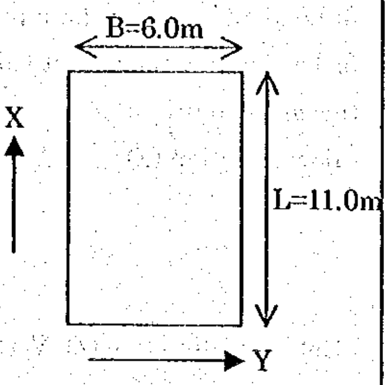
	flow direction: X			gate axis direction: Z			
	V	H	M	V	H	M	ΣH
Normal	361.55	-53.74	-136.4	361.55	33.88	49.4	63.53
earthquake Z	355.57	-44.65	73.1	355.57	86.15	263.0	100.81
earthquake X	359.73	-99.22	-275.9	359.73	23.57	32.9	101.98

6)-2 Layout of pile

ϕ 500 PHC pile X x Z = 5 x 3 n=15

@X = 2.125 m @Z = 1.75 m

6)-3 Calculation of displacement



Simplified deformation method

$H = -n \times K_2 \times \alpha \ + n \times K_1 \times \delta x$
$V = n \times Kv \times \delta y$
$M = (\Sigma Kv \times Xi^2 + n \times K_4) \times \alpha \ - n \times K_3 \times \delta x$

H: horizontal load acting to footing (t)
V: vertical load acting to footing (t)
M: moment around footing center (t.cm)
δ x: horizontal displacement (cm)
δ y: vertical displacement (cm)
α : angle of roatation (radian)
Kv: axial forces induce the unit axial deflection at pile top (t/cm)
$Kv = a \times Ap \times Ep / l, \quad a = 0.013 \times (l/D) + 0.61 = 1.18$
$Kv = 1.18 \times 1{,}159 \times 400{,}000 / 2209 = 247.6$ t/cm
Xi: horizontal distance of pile i from footing center (cm)
K_1, K_2, K_3 and K_4: coefficients
$K_1 = 4EI\beta^3, \quad K_2 = 2EI\beta^2, \quad K_3 = K_2, \quad K_4 = 2EI\beta$

$\beta = \sqrt[4]{\dfrac{k \cdot D}{4 \cdot E \cdot I}}$ $\quad \beta = 3.78 \times 10^{-3}$ (normal), $\beta = 4.49 \times 10^{-3}$ (earthquake)

results are as follows:(details see table-6)

		δ H (cm)			α H (x 10^{-5} radian)		
	δ y	δ x	δ z	δ H	α x	α z	α H
Normal	0.097	-0.163	0.115	0.199	-3.95	11.52	15.47
earthquake: Z	0.096	-0.078	0.200	0.215	0.440	43.30	43.74
Earthquake: X	0.096	-0.183	0.049	0.189	-7.33	7.14	14.47

Allowable: normal condition; δ y= δ H = 1.0 cm, α = 1/1000 = 1 x 10^{-3}
earthquacke condition; δ y= 1.0 cm, δ H = 1.5 cm, α = 1/1000 = 1 x 10^{-3}

6)-3 Load for pile

$P_V i = Kv \times (Xi\,\alpha \ + \ \delta y)$

$P_H i = H / n$

$P_V i$: vertical load acting on pile i (t)
$P_H i$: horizontal load acting on pile i (t)

results are shown below; (details see table-7)

	P_V (t)		
	P_{Vmax}	P_{Vmin}	P_H (t)
Normal	28.2	19.9	4.24
earthquake: Z	24.3	23.4	6.72
Earthquake: X	31.5	16.1	6.8

7) Stress for a pile

7)-1 Load and moment for calculation of pile stress

$M_0 = H / 2\beta$ $\quad \beta = 0.378 \times 10^{-2}$: normal, $\beta = 0.449 \times 10^{-2}$: earthquake
$M_m = -0.2079\, M_0$

Table 6 displacement calculation

V	361.55			n			5	3	15		
H	-53.74						2	1			
M	-13640			@			2.125	212.5			
											6
							E	I	β		11
K1	22.6					4	400000	261675	0.00378		
K2	2,991.1					2	400000	261675	0.00378		
K3	2,991.1										
K4	791,305.2					2	400000	261675	0.00378		
Kv	247.6										
ΣXi	2100000										
ΣKV·Xi	519960000										
H=		−	44867.00484	α	+			339.2	δx	=	−53.74
V=			3714	δy						=	361.55
M=			531829578	α	−			44867	δx	=	−13640
		−	2013048123	α				15218644	δx	=	−2411152.84
			1.80394E+11	α				15218644	δx	=	−4626613.752
			1.78381E+11	α						=	−7037766.592
				α						=	−3.94536E−05
											−25346.20145
			−20982.6147		−			44867	δx	=	−13640
									δx	=	−0.163652882
									δy	=	0.097347873

normarl X

displacement calculation

V	361.55	n		5	3		15
H	33.88			2	1		
M	4940	@		1.75	175		
				E	I	β	
K1	22.6		4	400000	261675	0.00378	
K2	2,991.1		2	400000	261675	0.00378	
K3	2,991.1						
K4	791,305.2		2	400000	261675	0.00378	
Kv	247.6						
ΣZi	306250						
ΣKV·Zi	75827500						

H=	− 44867.00484	α	+	339.2	δZ	=	33.88
V=	3714	δy				=	361.55
M=	87697078	α	−	44867	δZ	=	4940
	− 2013048123	α		15218644	δZ	=	1520094.124
	29746371486	α		15218644	δZ	=	1675621.11
	27733323363	α				=	3195715.234
		α				=	0.00011523
							8678.283682
	10105.34816		−	44867	δZ	=	4940
					δZ	=	0.115125763

normal Z

displacement calculation

V	355.57	n	5	3	15
H	-44.65		2	1	
M	7310	@	2.125	212.5	

			E	I		β
K1	37.9	4	400000		261675	0.00449
K2	4,220.3	2	400000		261675	0.00449
K3	4,220.3					
K4	939,936.6	2	400000		261675	0.00449

Kv	247.6
ΣXi	2100000
ΣKV·Xi	519960000

H=	-	63304.73001 α	+	568.5	δx	=	-44.65
V=		3714 δy				=	355.57
M=		534059049 α	-	63304.73001	δx	=	7310
	-	4007488842 α		35987249.8	δx	=	-2826556.195
		3.036E+11 α		35987249.8	δx	=	4155563.036
						=	1329006.841
		2.99593E+11 α				=	4.43605E-06
		α					225425.8653
		2369.111674	-	63304.73001	δx	=	7310
					δx	=	-0.078049276
					δy	=	0.095737749

seismic-Z X

Name of Structure	Baru Gate	Category of calculation	Stability Analysis	Page	26/34

displacement calculation

V	355.57	n	5	3	15
H	86.15		2	1	
M	26300	@	1.75	175	

			E	I	β
K1	37.9	4	400000	261675	0.00449
K2	4,220.3	2	400000	261675	0.00449
K3	4,220.3				
K4	939,936.6	2	400000	261675	0.00449

Kv	247.6
ΣZi	306250
ΣKV·Zi	75827500

H=	− 63304.73001	α	+	568.5	δZ	=	86.15
V=	3714	δy				=	355.57
M=	89926549	α	−	63304.73	δZ	=	26300
	− 4007488842	α		35987250	δZ	=	5453702.49
	51121127628	α		35987250	δZ	=	14950931.31
	47113638787	α				=	20404633.8
		α				=	0.000433094
							2308.967623
	38946.64789		−	63304.73	δZ	=	26300
					δZ	=	0.199774138

seismic-Z Z

displacement calculation

V	359.73	n		5	3		15
H	−99.22			2	1		
M	−27590	@		2.125	212.5		
			E	I		β	
K1	37.9		4	400000	261675	0.00449	
K2	4,220.3		2	400000	261675	0.00449	
K3	4,220.3						
K4	939,936.6		2	400000	261675	0.00449	
Kv	247.6						
ΣXi	2100000						
ΣKV·Xi	519960000						

H=	− 63304.73001 α	+	568.5 δx	=	−99.22
V=	3714 δy			=	359.73
M=	534059049 α	−	63304.73001 δx	=	−27590
	− 4007488842 α		35987249.8 δx	=	−6281095.312
	3.036E+11 α		35987249.8 δx	=	−15684265.96
	2.99593E+11 α			=	−21965361.27
	α			=	−7.33175E−05
					−13639.31662
	−39155.851	−	63304.73001 δx	=	−27590
			δx	=	−0.182701213
			δy	=	0.096857835

seismic−X X

displacement calculation

V	321.73	n		5	3	15
H	23.57			2	1	
M	3290	@		1.75	175	

			E	I	β
K1	37.9	4	400000	261675	0.00449
K2	4,220.3	2	400000	261675	0.00449
K3	4,220.3				
K4	939,936.6	2	400000	261675	0.00449

Kv	247.6
ΣZi	306250
ΣKV·Zi	75827500

H=	− 63304.73001	α	+	568.5	δZ	=	23.57
V=	3714	δy				=	321.73
M=	89926549	α	−	63304.73	δZ	=	3290
	− 4007488842	α		35987250	δZ	=	1492092.486
	51121127628	α		35987250	δZ	=	1870287.604
	47113638787	α				=	3362380.091
		α				=	7.13674E−05
							14011.99077
	6417.828165		−	63304.73	δZ	=	3290
					δZ	=	0.049409075

seismic−X Z

Results are shown below; (details see table-8)

	P_V (t)				
	P_{Vmax}	P_{Vmin}	P_H (t)	M_O (tm)	M_m (tm)
Normal	28.2	19.9	4.24	5.61	1.17
earthquake: Z	24.3	23.4	6.72	7.48	1.56
Earthquake: X	31.5	16.1	6.8	7.57	1.57

7)-2 Effective pre-stress

- basic condition

  ϕ 500 PHC pile Type A

  r_0 = 25.0 cm, r_1 = 16.0 cm, r_p = 21.0 cm
  PC wire ϕ 9mm x 9
  cross sectional area (A0): A0 = $\pi (r_0^2 - r_1^2)$ = 1159.2 cm^2

- Area of PC cable (Ap): Ap = 0.64 x 9 = 5.8 cm^2
- Area of concrete (Ac): Ac = 1159 – 5.8 =1153.4 cm^2
- Effective cross sectional area (Ae): Ae = Ac + n x Ap = 1153.4 + 5 x 5.8 = 1182.4 cm^2
  n = Ep / Ec = 5
- Moment of inertia (Ie): Ie = $(\pi/4) \times (r_0^4 - r_1^4) + 1/2 \times (n-1) \times Ap \times r_p^2$ = 260604.6 cm^4
- Coefficient of effective cross seciton (Ze): Ze = Ie / r_0 = 260604.6 / 25 = 10416.2
- Tensile strength of PC wire (σ pi): σ pi = 0.7 x 14500 = 10150 kg/cm^2
- Tensile stress after elastic deflection): σ pt = σ pi / (1 + n' x (Ap / Ac)) = 9819.2 kg/cm^2
  n' = Ep / Ec = 6.7
- Initial prestress (σ cpt): σ cpt = σ pt x Ap / Ac = 9819.2 x 5.8 / 1153.4 = 49.4 kg/cm^2
- Reduce of tensile strength

$$\Delta\sigma_{p\varphi} = \frac{n \cdot \varphi \cdot \sigma_{c\varphi} + Ep \cdot \varepsilon_s}{1 + n \cdot \frac{\sigma_{c\varphi}}{\sigma_{pt}} \cdot (1 + \frac{\varphi}{2})} \qquad \phi = 3.0, \quad \varepsilon s = 15 \times 10^{-5}$$

  $\Delta \sigma_{p\phi}$ = 979.4

  $\Delta \sigma_r$ = r x σ pt = 0.07 x 9819.2 = 687.3

- Effective tensile stress of pile (σ pe): σ pe = σ pt –($\Delta \sigma_{p\phi} + \Delta \sigma_r$) = 8152.5 kg/cm^2
- Effective prestress (σ ce): σ ce = σ pe x Ap / Ac = 41.0 kg/cm^2

7)-3 Analysis on pile stress

- Allowable stress of concrete (σ ca, σ ta)

  σ ca = 500 kg/cm2 / 3 = 166 kg/cm^2 normal
  σ ca' = 1.5 x σ ca = 250 kg/cm^2 earthquake

  σ ta = not allowed normal
  σ ta' = 30 kg/cm^2 40 < σ ce < 70 kg/cm^2 earthquake

- Allowable stress of PC cable (σ pa)

  σ pa = 0.60 x 14500 = 8700 kg/cm^2

Table 7
calc of load for a pile n 15

X	α		Z			δy	Pv	H	Ph
normal								63.53	4.24
	-3.95			11.52					
425	-0.0000395	-0.01679	175	0.0001152	0.02016	0.097	19.889		
212.5	-0.0000395	-0.00839	175	0.0001152	0.02016	0.097	21.967		
0	-0.0000395	0	0	0.0001152	0	0.097	24.046		
0	-0.0000395	0	0	0.0001152	0	0.097	24.046		
-212.5	-0.0000395	0.008394	-175	0.0001152	-0.02016	0.097	26.124		
-425	-0.0000395	0.016788	-175	0.0001152	-0.02016	0.097	28.202		
earthquake Z								100.81	6.72
	0.44			43.3					
425	0.0000044	0.00187	175	0.000433	0.075775	0.096	24.340		
212.5	0.0000044	0.000935	175	0.000433	0.075775	0.096	24.108		
0	0.0000044	0	0	0.000433	0	0.096	23.877		
0	0.0000044	0	0	0.000433	0	0.096	23.877		
-212.5	0.0000044	-0.00094	-175	0.000433	-0.07578	0.096	23.645		
-425	0.0000044	-0.00187	-175	0.000433	-0.07578	0.096	23.414		
earthquake X								101.98	6.80
	-7.33			7.14					
425	-0.0000733	-0.03115	175	0.0000714	0.012495	0.096	16.074		
212.5	-0.0000733	-0.01558	175	0.0000714	0.012495	0.096	19.931		
0	-0.0000733	0	0	0.0000714	0	0.096	23.787		
0	-0.0000733	0	0	0.0000714	0	0.096	23.787		
-212.5	-0.0000733	0.015576	-175	0.0000714	-0.0125	0.096	27.644		
-425	-0.0000733	0.031153	-175	0.0000714	-0.0125	0.096	31.501		

- Stress of pile

$\sigma c = \sigma ce + (M / Ie) \times r_0 + (V / Ae) \quad = \sigma ce + (M / Ze) + (V / Ae)$
$\sigma c' = \sigma ce - (M / Ie) \times r_0 + (V / Ae) \quad = \sigma ce - (M / Ze) + (V / Ae)$
$\sigma p = \sigma pe + n \times (M / Ie) \times r_p - n \times (V / Ae)$

σ c: compressive stress of concrete (kg/cm^2)
σ c': tensile stress of concrete (kg/cm^2)
σ p: tensile stress of PC cable (kg/cm^2)

Results are shown below; (details see table-8)

(normal condition)

$\sigma c = 118.7$ kg/cm^2 < 166 kg/cm^2 → OK
$\sigma c' = 4.0$ kg/cm^2 > 0 → OK
$\sigma p = 8294.4$ kg/cm^2 < 8700 kg/cm^2 → OK

(earthquake condition)

$\sigma cz = 133.4$ kg/cm^2 < 250 kg/cm^2 → OK
$\sigma c'z = -11.0$ kg/cm^2 > -30 kg/cm^2 → OK
$\sigma pz = 8354.9$ kg/cm^2 < 8700 kg/cm^2 → OK

$\sigma cx = 140.3$ kg/cm^2 < 250 kg/cm^2 → OK
$\sigma c'x = -18.1$kg/cm^2 > -30 kg/cm^2 → OK
$\sigma px = 8389.4$ kg/cm^2 < 8700 kg/cm^2 → OK

<u>Analysis on Joint between Pile and Footing</u>

1) Compressive stress

(normal) V = 4.09 t, H = 4.24 t
$r_0 = 25$ cm, $r_1 = 16$ cm

$\sigma v = V / \{\pi \times (r_0^2 - r_1^2)\} = 35.3$ kg/cm^2 < 75 kg/cm^2 (design criteria)
$\sigma_H = H / Dl = 9.4$ kg/cm^2 < 75 kg/cm^2 → OK
D = 45 cm, l = 10 cm

(earthquake) V = 44.9 t, H = 6.80 t

$\sigma v = 38.7$ kg/cm^2 < 112.5 kg/cm^2 = 75 kg/cm^2 x 1.5
$\sigma_H = 15.1$ kg/cm^2 < 112.5 kg/cm^2 → OK

2) Shear stress

(normal) $\tau = V / (\pi \times D \times l') = 3.21$ kg/cm^2 < 7.5 kg/cm^2 → OK
l' = 90 cm : distance between pile top and footing top

(earthquake) $\tau = 3.53$ kg/cm^2 < 7.5 kg/cm^2 → OK

<u>Reinforcement of Pile Head</u>

Rinforcement of pile head is done by embedding the reinforcing bar basket into the inside of PC Pile with filling of concrete. In this case, the pile head is asuumed as a RC with circle shape corss section.

Condtions of calculation is as follows:

Table 8
Pile stress

normal															
c	41	+	5.61	x	100000	/	10416.2	+	28.2	x	1000	/	1182.4	=	118.7
c'	41	−	5.61	x	100000	/	10416.2	+	19.9	x	1000	/	1182.4	=	4.0
p	8152.5	+	589.1	x	100000	/	260604.6	−	99.5	x	1000	/	1182.4	=	8,294.4
earthquake Z															
c	41	+	7.48	x	100000	/	10416.2	+	24.3	x	1000	/	1182.4	=	133.4
c'	41	−	7.48	x	100000	/	10416.2	+	23.4	x	1000	/	1182.4	=	−11.0
p	8152.5	+	785.4	x	100000	/	260604.6	−	117	x	1000	/	1182.4	=	8,354.9
earthquake X															
c	41	+	7.57	x	100000	/	10416.2	+	31.5	x	1000	/	1182.4	=	140.3
c'	41	−	7.57	x	100000	/	10416.2	+	16.1	x	1000	/	1182.4	=	−18.1
p	8152.5	+	794.9	x	100000	/	260604.6	−	80.5	x	1000	/	1182.4	=	8,389.4

(Dimension and bar diameter)

Assumed diameter of cross section: 50 cm (pirl diameter)+ 20 cm = 70 cm
Rainforcement bar to be used: D13 x 9

(Load condition)

The most tough case for pile top is having large moment and small axial stress at earthquake condition. Loads for calculation are as follows:

Bending moment at pile top: 7.57 t.m → 8 t.m
Axisal force: 27.4 t → 27 t

(Result of calculation)

$\sigma c = 62.0 \text{ kg/cm}^2 < 112.5 \text{ kg/cm}^2$ → OK
$\sigma s = 797.3 \text{ kg/cm}^2 < 2100 \text{ kg/cm}^2$ → OK

(See Table – 9)

Check of Footing Rigidity

hmin $\{(3 \times k \times \lambda^4) / E\}^{1/3}$

$\lambda = a \times (\lambda'^2 + e^2) / (\lambda' + e) = 2.2$

$\lambda' = \max(b, l) = 2 \quad (l=0)$

$k = kv \times (m \times n) / (L \times B) = 6752$

hmin $\{(3 \times k \times \lambda^4) / E\}^{1/3} = 0.58$ m

h = 1.0 m > 0.58 m = hmin → OK

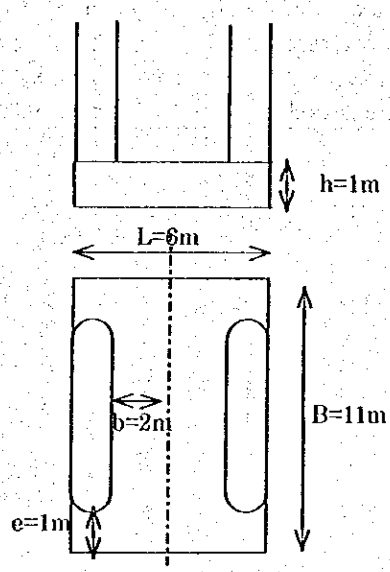
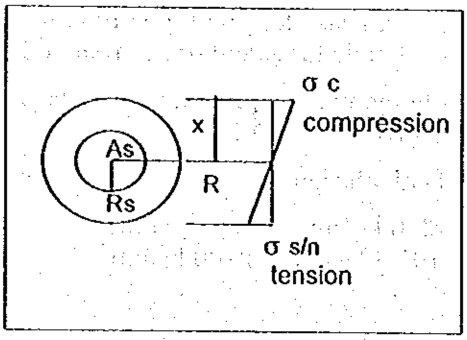


Table-9

item	symbol	unit	amount			
concrete radius	R	cm	35.0			
radius to the bar	Rs	cm	11.0			
cover	d	cm	24.0			
bending moment	M	tf*m	8.000			
axial force	N	tf	27.000			
shearing force	Q	tf	0.000			
eccentricity	e	cm	29.6			
bar diameter	D	mm	13			
no of bars	h		9			
bar area	As	cm^2	11.401			
elasticity ratio	n	-	15.0			
bar ratio	p	-	0.00296			
location of nutral axis	x	cm	24.8			
compression stress of concrete	σ c	kgf/cm^2	62.012	≦	σ c = 112.5	OK
tensile stress of bar	σ s	kgf/cm^2	797.287	≦	σ s = 2100.0	OK



circular section