2. Stability Analysis

Type of Pile

Foundation structure of the major parts of weir shall be designed as the structure that transmits loads to the stable ground so that the uneven settlement, which arises due to the possible loading, be avoided to maintain stability. As discussed in the previous section, pile foundation has been selected for the geological and technical reasons.

There are several choices as to which type of pile should be used. Conceivable pile types are reinforced concrete pile, prestressed concrete pile, steel pipe pile, and cast-in-place concrete pile. The most suitable pile type is selected through the preliminary comparative study regarding foundation pile for the center pier, and the results are summarized in the table below.

	RC Pile □ 500	PC Pile ϕ 600	Steel Pipe Pile ϕ 600	Cast-in-place Pile ϕ 1000
Number of Pile	72 (6 x 12)	36 (4 x 9)	24 (3 x 8)	21 (3 x 7)
Displacement of top portion of pile	0.6 mm (1.2 mm)	1.0 mm (1.2 mm)	1.2 mm (1.4 mm)	0.8 mm
Maximum Bending Moment (Pile body)	2.8 tf-m (5.84 tf-m)	8.1 tf-m (13.7 tf-m)	19.0 tf-m (31.3 tf-m)	24.0 tf-m (39.5 tf-m)
Material cost	Rp 87.3 x 10^6	Rp 58.2 x 10^6	Rp 261.6 x 10^6	Rp 110.4 x 10^6
Cost for driving	Rp 15.4 x 10^6	Rp 7.7 x 10^6	Rp 5.1 x 10^6	Rp 15.5 x 10^6
Total Cost	Rp 102.7 x 10^6	Rp 65.9 x 10^6	Rp 266.7 x 10^6	Rp 125.9 x 10^6
Evaluation	Not applicable	Applicable	Not applicable	Not applicable

Clearly, prestressed concrete pile is most advantageous economically among all types of pile. So, the prestressed concrete pile is adopted for the foundation pile of center piers. This type of pile is applied to the foundation pile for the end piers and gate floor slabs as well.

Pile Diameter and Arrangement

There are many cases for the combination of pile diameter and pile arrangement (number of pile). Judging from the structural size, geological and soil mechanical conditions, the following three alternatives are selected for comparative study. It is noted that the maximum pile diameter which is available in this country, is 600 mm.

Alternative-1	PC Pile Dia.450 mm, type A 60 piles
Alternative-2	PC Pile Dia.500 mm, type A 50 piles
Alternative-3	PC Pile Dia.600 mm, type A 36 piles

Calculation Case and Method

Pile stability analysis is carried out for the above three (3) alternatives to select structurally the most stable and economically the most reasonable alternative. The method of stability analysis is presented in the Design Criteria. In the calculation the following four (4) cases are considered..

① Normal Case, ② Flooding Case, ③ Seismic Case, ④ Construction Case

The items to be checked in the stability analysis are 1) displacement of pile, 2) stress generated in pile body, supporting capacity of ground at pile tip.

Allowable Stress and Displacement

The stress generated in the pile body shall not exceed the strength of pile. Pile interaction curve is used to compare these two stress and strength of pile.

At Simongan Weir and the surrounding area, there is Damar Formation form that is hard sand layer with the N-value of more than 50. Damar Formation forms are underneath design riverbed at the depth of about 10.0 m.

So, it is difficult to drive PC pile into the layer with general pile driving.

Therefore, in case of stability calculation, it regards that PC piles are driven until Damar Formation form surface.

The allowable bearing capacity of ground is estimated with result of geological survey at Simongan weir. Safety factors are set 3 for Normal Case and 2 for

Seismic Case.

Regarding the displacement of piles, the allowable displacements are set to be 10 mm for Normal Case and 15 mm for Seismic Case.

The allowable bearing capacity of ground is shown in each alternative pile as follows.

Layer	Li (m)	N – Value Average	Fi (tf / m)	Li·Fi (tf / m)
As	5.30	17	3.40	18.02
Ac	3.50	16	9.60	33.60
Dc	1.40	35	15.00	21.00
Total	10.20			72.62

Alternative – 1 (Dia 450 mm)

$$A = \pi/4 \, D^2 = \pi/4 \times (0.45)^2 = 0.159\, m^2$$

$$qd = 500 tf / m^2 \times A = 79.50\, tf$$

$$U = \pi D = \pi \times 0.45 = 1.414\, m$$

$$Ru = 79.50 + 1.414 \times 72.62 = 182.18\, tf / pile$$

Allowable bearing capacity
for PC pile (Dia 450 mm)

Case	Safety factor	Allowable bearing capacity (tf / pile)
Normal	3	60.73
Seismic	2	91.09

Alternative – 2 (Dia 500 mm)

$$A = \pi/4 \, D^2 = \pi/4 \times (0.50)^2 = 0.196\, m^2$$

$$qd = 500\, tf / m^2 \times A = 98.00\, tf$$

$$U = \pi D = \pi \times 0.50 = 1.571\, m$$

$$Ru = 98.00 + 1.571 \times 72.62 = 212.09\, tf / pile$$

Allowable bearing capacity
for PC pile (Dia 500 mm)

Case	Safety factor	Allowable bearing capacity (tf / pile)
Normal	3	70.70
Seismic	2	106.04

Alternative – 3 (Dia 600 mm)

$$A = \pi/_4 D^2 = \pi/_4 \times (0.60)^2 = 0.283\, m^2$$

$$qd = 500\, tf / m^2 \times A = 141.50\, tf$$

$$U = \pi D = \pi \times 0.60 = 1.885\, m$$

$$Ru = 141.50 + 1.885 \times 72.62 = 278.39\, tf / pile$$

Allowable bearing capacity
for PC pile (Dia 600 mm)

Case	Safety factor	Allowable bearing capacity (tf / pile)
Normal	3	92.80
Seismic	2	139.19

Calculation Results

Pile stability analyses for Alternative-1, Alternative-2 and Alternative-3 were conducted based on the conditions mentioned above. As a result Alternative-3 (pile dia.=600mm, N=36) was selected for the economical reason. The calculation results are shown as follows.

COMPARATIVE STUDY ON PILE FOUNDATION FOR CENTER PIER

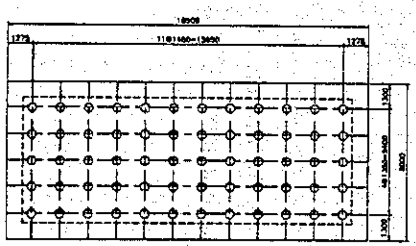
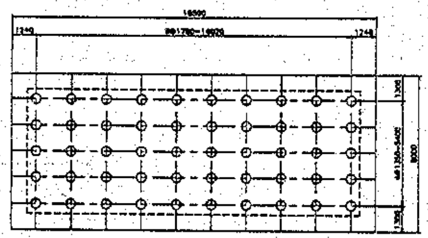
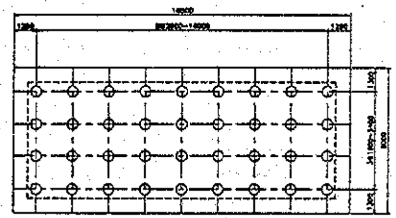
Kind of pile	Alternative-1 Dia. 450 (A)			Alternative-2 Dia. 500 (A)			Alternative-3 Dia. 600 (A)		
Pile Arrangement	 FLOW			 FLOW			 FLOW		
Number of necessary pile	60 piles (L=10.30m)			50 piles (L=10.30m)			36 piles (L=10.30m)		
Displacement (Horizontal)	Calculation (mm)	Allowable capacity (mm)		Calculation (mm)	Allowable capacity (mm)		Calculation (mm)	Allowable capacity (mm)	
Normal Case	1.0	10		1.0	10		0.90	10	
Design flooding Case	0.1	10		0.1	10		0.89	10	
Constructional Case	0.1	10		0.1	10		0.07	10	
Seisimic Case	1.0	15		1.0	15		1.11	15	
Force / Moment Acting on Pile	Axial load (tf/pile)	Bending moment (tf-m)		Axial load (tf/pile)	Bending moment (tf-m)		Axial load (tf/pile)	Bending moment (tf-m)	
		Calculation	Allowable capacity		Calculation	Allowable capacity		Calculation	Allowable capacity
Normal Case	23.00	4.39	4.77	27.59	5.82	6.50	42.18	9.27	11.35
Design flooding Case	21.88	0.23	4.68	26.34	0.30	6.39	34.15	1.48	10.51
Constructional Case	29.72	0.25	5.28	35.78	0.33	7.19	49.17	1.30	12.07
Seisimic Case	15.67	6.23	6.48	18.99	8.26	8.91	34.41	14.46	15.79
Bearing capacity (tf) Normal	50.13			54.96			92.80		
Bearing capacity (tf) Seismic	75.19			82.45			110.89		
Summary of cost	Rp 96.5 million			Rp 98.3 million			Rp 85.3 million		
Evaluation	Not adopted			Not adopted			Adopted		

TABLE OF STABILITY CALCULATION FOR CENTER PIER

Direction		Direction of flowing water				Direction of weir axis			
Case		Normal case	Design Flooding case	Construction case	Seismic case	Normal case	Design Flooding case	Construction case	Seismic case
Quantity of displacement for footing (m)	Horizontal (δ X m)	0.0010524	0.0001016	0.0001187	0.0011130	0.0000389	0.0000231	0.0000774	0.0007930
	Allowable	0.010	0.010	0.010	0.015	0.010	0.010	0.010	0.015
	Vertical (δ Y m)	0.0009044	0.0008851	0.0010240	0.0008319	0.0009044	0.0008851	0.0010240	0.0008319
	Allowable	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015
Axial force (tf/pile)	No.1	42.1761	34.1462	49.1719	34.4060	46.8333	46.3136	52.8222	27.5063
	No.2	43.9954	37.7094	50.8775	37.1769	48.5800	47.7040	54.9370	39.4951
	No.3	45.8147	41.2727	52.5832	39.9477	50.3267	49.0944	57.0519	51.4838
	No.4	47.6340	44.8359	54.2888	42.7186	52.0734	50.4847	59.1667	63.4726
	No.5	49.4533	48.3992	55.9944	45.4894	-	-	-	-
	No.6	51.2726	51.9624	57.7001	48.2603	-	-	-	-
	No.7	53.0919	55.5257	59.4057	51.0312	-	-	-	-
	No.8	54.9113	59.0889	61.1114	53.8020	-	-	-	-
	No.9	56.7306	62.6521	62.8170	56.5729	-	-	-	-
Allowable bearing capacity (tf/pile)		92.80	92.80	92.80	139.19	92.80	92.80	92.80	139.19
Shearing stress (tf)		8.2264	1.0575	1.0444	15.4906	0.4286	0.2808	0.7511	6.9806
Bending moment (tf-m/pile)		9.2717	1.4787	1.3042	14.4611	0.6182	0.4253	1.0058	8.7184
Allowable bending moment (tf-m/pile)		11.3463	10.5112	12.0739	15.7942	11.8307	11.7766	12.4535	15.0767

Number of piles: n=36 piles

Pile head condition: Fixing

Pile condition

1. Diameter: Dia.600 mm
2. Geometrical moment of area: $I = 0.0052200\ m^4$
3. Section area of pile $A = 0.167800\ m^2$

3. Stress-Strain Calculation

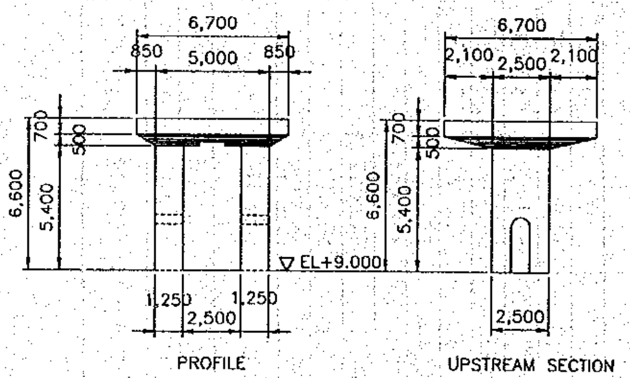
Stress-strain calculations of the structure are made to decide proper reinforcing bar arrangement. Described below are the bar arrangement for the center pier. Deformed steel bars are used for all parts of structure, and bar spacing will be 125 mm or 250 mm.

(1) Gate column and Control Deck

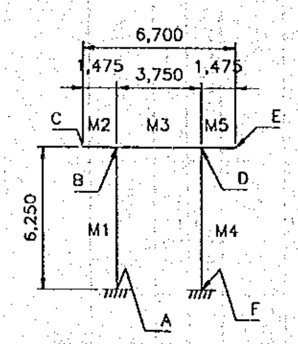
Direction of flowing water

a) Dimension

Gate column that is fixed on gate pier is regarded as rigid frame structure. And section of gate column without gallery section is regarded as standard section.



Dimension of rigid frame structure is shown as follows.



Geometrical moment of inertia

Member	Calculation	Geometrical moment of inertia (m^4)
1	$\frac{1}{12} \times 0.85 \times 1.25^3 \times 2$	0.27669
2	$\frac{1}{12} \times 6.70 \times 0.70^3$	0.19151
3	$\frac{1}{12} \times 6.70 \times 0.70^3$	0.19151
4	$\frac{1}{12} \times 0.85 \times 1.25^3 \times 2$	0.27669
5	$\frac{1}{12} \times 6.70 \times 0.70^3$	0.19151

Section area

Member	Calculation	Area (m^2)
1	0.85×1.25×2	2.125
2	6.70×0.70	4.690
3	6.70×0.70	4.690
4	0.85×1.25×2	2.125
5	6.70×0.70	4.690

b) Loading Calculation

Normal Case

[Control deck]

<u>Distributed load</u>

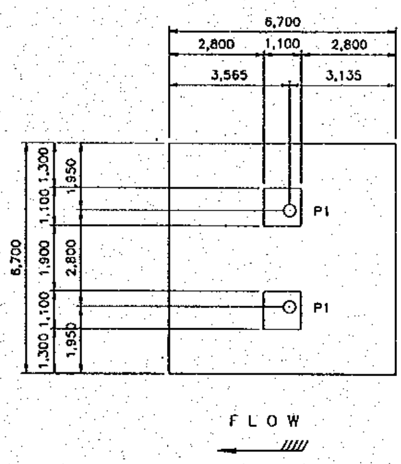
Position of load	Calculation form	W (tf / m)
Weight of body	$0.70 \times 6.70 \times 2.50 tf / m^3$	11.725
Cinder concrete	$0.15 \times 6.70 \times 2.35 tf / m^3$	2.362
Sidewake live load	$0.30 tf / m^2 \times 6.70$	2.010
Weight of control house	$2.30 tf / m \times 2$	4.600
Total		20.697

<u>Load of control house</u>

Both ends of control deck are affected by control house weight.

$p_0 = 2.30 tf / m \times 6.70 m = 15.41 tf$

<u>Hoisting load</u>



P1=105.00 tf / place

<u>Wind pressure</u>

For control house

Hw = (4.00+0.70)×6.70×0.15 tf/m^2 = 4.72 tf

$Mw = 4.72 \times (1/2 \times 4.00 + 0.35) = 11.09$ tf –m

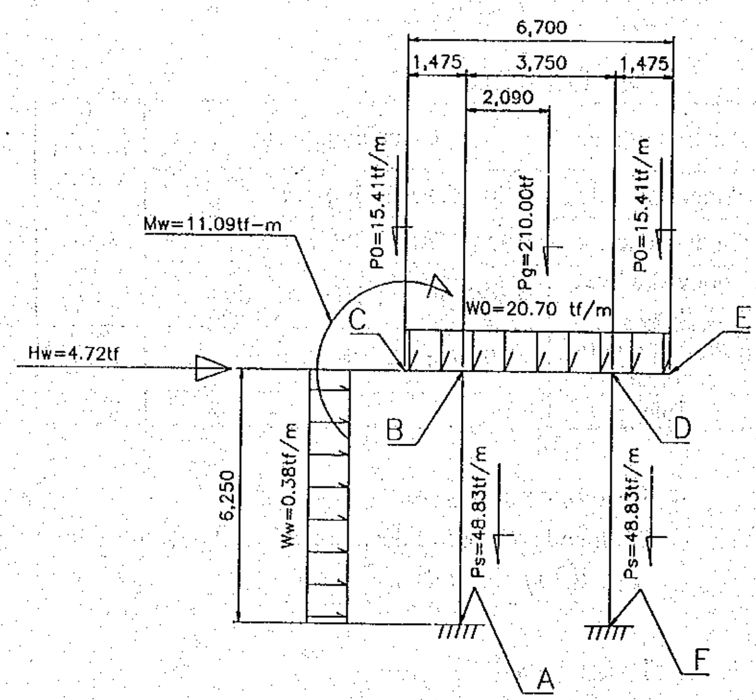
For gate column

$Ww = 2.50 \times 0.15$ tf/m^2 = 0.38 tf/m

<u>Weight of gate column</u>

$Ps = 2.50 \times 1.25 \times 2.50$ tf/m^3 $\times 6.25 = 48.83$ tf

Figure of load working in normal case is shown as follows.



Seismic case

[Control deck]

Distributed load

W0 = 20.70 tf/m (refer to normal case)

Load of control house

P0 = 15.41 tf (refer to normal case)

Seismic load

Position of load	Calculation form	W (tf / m)
Weight of body	$6.70 \times 6.70 \times 0.70 \times 2.50\ tf/m^3 \times 0.12$	9.43
Cinder concrete	$6.70 \times 6.70 \times 0.15 \times 2.35\ tf/m^3 \times 0.12$	1.90
Sidewalk live load	$6.70 \times 6.70 \times 0.30\ tf/m^2 \times 0.12$	1.62
Weight of control house	$15.41\ tf \times 4 \times 0.12$	7.40
Weight of hoisting system	$25.00\ tf \times 2 \times 0.12$	6.00
Total (He)		26.35

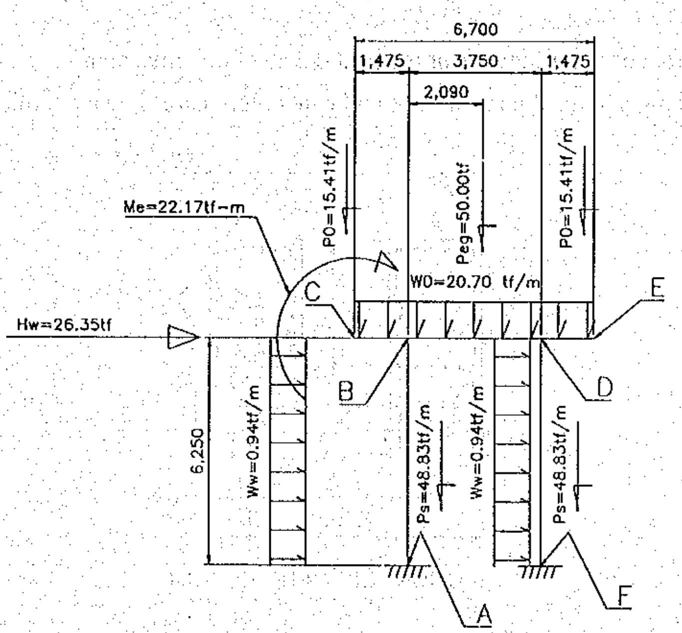
$$Me = 1.90 \times 0.425 + 1.62 \times 0.35 + 7.40 \times 2.00 + 6.00 \times 1.00$$
$$= 22.17\ tf-m$$

Weight of gate column

$Ps = 2.50 \times 1.25 \times 2.50\ tf/m^3 \times 6.25 = 48.83\ tf$

$Ws = (2.50 \times 1.25 \times 2.50\ tf/m^3) \times 0.12 = 0.94\ tf/m$

Figure of load working is shown as follows.



c) Consideration of bending moment and shearing stress

Summary of calculation result is shown as follows.

	Member	Condition	Distance (m)	Bending moment M (tf –m)	Shearing stress S (tf)	Axial Force N (tf)
Normal Case	M1	Maximum	0.000	13.605	-8.211	-219.643
		Minimum	6.250	-45.138	-10.586	-219.643
	M2	Maximum	0.000	11.810	-15.410	-4.720
		Minimum	1.475	-33.437	-45.943	-4.720
	M3	Maximum	2.090	137.194	81.608	-15.306
		Minimum	3.750	-104.458	-162.754	-15.306
	M4	Maximum	6.250	59.210	15.306	-257.527
		Minimum	0.000	-36.454	15.306	-257.527
	M5	Maximum	1.475	0.000	15.410	0.000
		Minimum	0.000	-45.247	45.942	0.000
Seismic Case	M1	Maximum	6.250	27.215	9.507	-134.293
		Minimum	0.000	-50.562	15.382	-134.293
	M2	Maximum	0.000	14.290	-15.410	-21.770
		Minimum	1.475	-30.957	-45.943	-21.770
	M3	Maximum	1.909	33.984	0.000	-12.263
		Minimum	3.750	-84.087	-88.104	-12.263
	M4	Maximum	6.250	38.839	12.263	-182.877
		Minimum	0.000	-56.165	18.138	-182.877
	M5	Maximum	1.475	0.000	15.410	0.000
		Minimum	0.000	-45.247	45.942	0.000

<u>Effective area and bar arrangement</u>

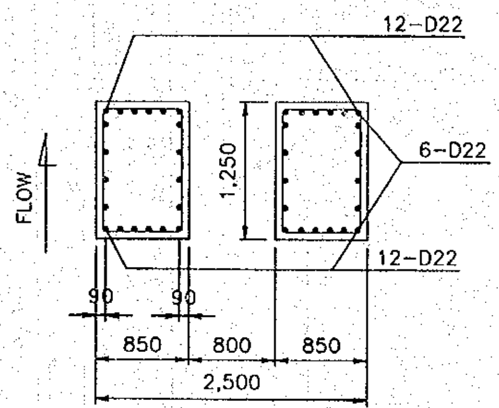
Effective area of Gate column and control deck gets smaller than original area. Because each members has opening for wire rope and blockout.

Gate column has gallery, then effective width of gate column section is 1.70 m.

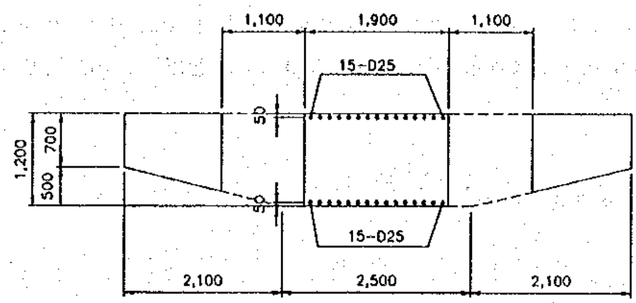
Control deck has opening for wire rope, then effective width of control deck section is 1.90 m.

Least area of bar arrangement is about 0.2 % of effective area.

Bar arrangements of control deck and gate column are shown as follows.



SECTION OF GATE COLUMN



SECTION OF CONTROL DECK

Results of Strength calculation on each members are shown as follows.

		Control Deck		Top of Gate Column		Bottom of Gate Column	
		Top side	Bottom side	Inside	Outside	Outside	Outside
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-104.46	137.19	27.22	-59.21	-45.14	-56.17
N	tf	0	0	0	0	0	0
S	tf	162.75	81.61	9.51	15.31	10.59	18.14
B	cm	190	190	170	170	170	170
D	cm	115	115	111	111	111	111
Ac	cm^2	21850	21850	18870	18870	18870	18870
As	cm^2	15-D25 =73.65	15-D25 =73.65	12-D22 =45.6	12-D22 =45.6	12-D22 =45.6	12-D22 =45.6
$P=As/(B\times D)$		0.00337	0.00337	0.00242	0.00242	0.00242	0.00242
N=Es/Ec		15	15	15	15	15	15
X0	cm	31.2	31.2	26.1	26.1	26.1	26.1
K=X0/D		0.271	0.271	0.235	0.235	0.235	0.235
$M/(B\times D^2)$	kgf/cm^2	4.157	5.46	1.3	2.827	2.155	2.682
$S/(B\times D)$	kgf/cm^2	7.449	3.735	0.504	0.811	0.561	0.961
(C)		8.101	8.101	9.218	9.218	9.218	9.218
(S)		21.746	21.746	29.937	29.937	29.937	29.937
(Z)		1.099	1.099	1.085	1.085	1.085	1.085
σc	kgf/cm^2	33.7	44.2	12	26.1	19.9	24.7
σs	kgf/cm^2	1356	1781	584	1269	968	1204
τ	kgf/cm^2	7.45	3.74	0.5	0.81	0.56	0.96
σca	kgf/cm^2	75	75	112.5	75	75	112.5
σsa	kgf/cm^2	1800	1800	2700	1800	1800	2700
τa	kgf/cm^2	7.6	7.6	5.7	3.8	3.8	5.7
Note		Normal case	Normal case	Seismic case	Normal case	Normal case	Seismic case

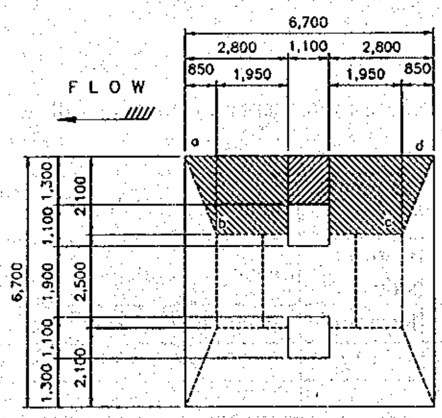
Direction of weir axis

In calculation of direction of weir axis, control desk and gate columns are fixed on gate pier. Then, positions of control deck base and bottom gate columns are considered.

a) Consideration of control deck base

Loading calculation

Loading condition is regarded as same thing direction of flowing water.



Distributed load

Position of load	Calculation form	W (tf/m^2)
Weight of body	$\frac{1}{2}\times(0.70+1.30)\times 2.50 tf/m^3$	2.500
Cinder concrete	$0.10\times 2.35 tf/m^3$	0.235
Sidewake live load		0.300
Weight of control house		0.500
Total		3.535

Weight of control house

P0 = 2.30 tf/m

Hoisting load

Pg = 105.0 tf   X = 0.15 m

Bending moment and Shearing stress

$$M_0 = \frac{1}{2}\times(6.70+5.00)\times 2.10\times 3.535\, tf/m^2 \times 1.101 + 2.30\, tf/m \times 6.70\times 2.10 + 105.0\times 0.15$$

95.92 tf-m

$$S_0 = \frac{1}{2}\times(6.70+5.00)\times 2.10\times 3.535\, tf/m^2 + 2.30\, tf/m + 105.0 \quad = 163.84 \text{ tf}$$

b) Consideration of control deck base with blockout

Load calculation

<u>Distributed load</u>

$W_0 = 3.535$ tf/m^2

<u>Weight of control house</u>

$P_0 = 2.30$ tf/m

<u>Hoisting load</u>

$Pg = 105.0$ tf

$q_0 = 3.535 \times 1.300 + 2.30 = 6.900$ tf/m

<u>Bending moment and Shearing stress</u>

Moment of edge position

$$Mf = \frac{1}{12} \times 6.900 \times 1.10^2 + \frac{1}{8} \times 105.0 \times 1.10 \quad = 15.13 \text{ tf-m}$$

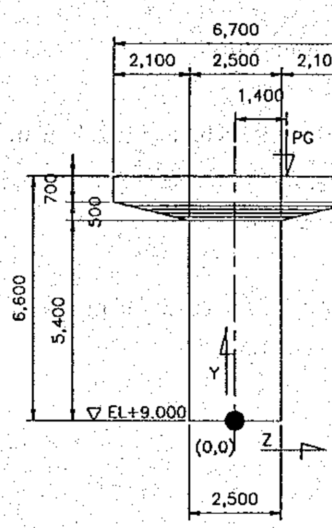
Moment of middle position

$$M_{max} = \frac{1}{8} \times 6.900 \times 1.10^2 + \frac{1}{4} \times 105.0 \times 1.10 \quad = 29.92 \text{ tf-m}$$

Shearing stress at edge position

$$S = \frac{1}{2} \times 6.900 \times 1.10 + \frac{1}{2} \times 105.0 \quad = 56.30 \text{ tf}$$

c) Consideration of gate column base



Weight of body

Position of load	Calculation form	W (tf)	Y (m)	Z (m)
Weight of control deck	$6.70 \times 6.70 \times 0.70 \times 2.50$ tf/m^3	78.56	6.25	0.00
Weight of control deck haunch	$(2.50+6.70) \times 1/2 \times 0.50 \times (5.00+6.70) \times 1/2 \times 2.50$ tf/m^3	33.64	5.69	0.00
Weight of gate column (1)	$5.40 \times 1.25 \times 2.50 \times 2.50$ tf/m^3	42.19	2.70	0.00
Weight of gate column (2)		42.19	2.70	0.00
Cinder concrete	$6.70 \times 6.70 \times 0.10 \times 2.35$ tf/m^3	10.55	6.65	0.00
Weight of control house		60.00	8.60	0.00
Weight of hoisting system		25.00	7.60	0.00
Total		292.13	5.77	0.00

Hoisting load

PG = 105.00 tf

Z = 1.40 m

Wind load

Position of load	Calculation form	W (tf)	Y (m)	Z (m)
Gate column	$(5.40+0.50) \times (1.25 + 1.25) \times 0.15$ tf/m^2	2.21	2.95	0.00
Control deck	$6.70 \times 4.70 \times 0.15$ tf/m^2	4.72	8.25	0.00
Total		6.93	6.56	0.00

Bending moment and shearing stress

< Normal case >

$M_0 = 105.00 \times 1.40 + 6.93 \times 6.56 = 192.46$ tf-m

$S_0 = 6.93$ tf

$N_0 = 292.13 + 105.00 = 397.13$ tf

< Seismic case >

$M_0 = 292.13 \times 0.12 \times 5.77 = 202.27$ tf-m

$S_0 = 292.13 \times 0.12 = 35.06$ tf

$N_0 = 292.13$ tf

Strength calculation results of direction of weir axis are shown as follows.

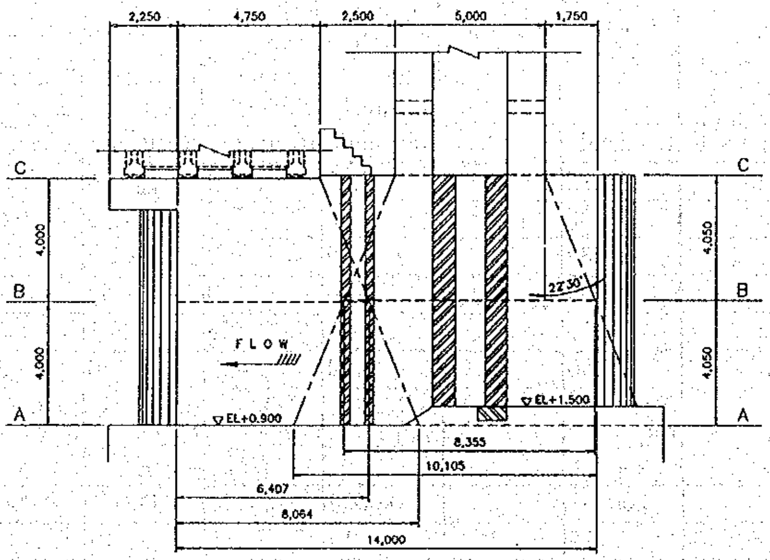
		Control deck base		Control deck base with blockout		Gate column base	
				Edge Section	Middle section	Normal case	Seismic case
Member of shape		Rectangle		Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-95.92		-15.13	29.92	192.46	202.27
N	tf	0		0	0	397.13	292.13
S	tf	163.84		56.29	0	6.93	35.06
B	cm	390		130	130	250	250
D	cm	115		65	65	241	241
Ac	cm^2	44850		8450	8450	60250	60250
As	cm^2	(9+9)-D22 =68.4		8-D22 =30.4	8-D22 =30.4	(6+6)-D22 =45.6	(6+6)-D22 =45.6
P=As/(B×D)		0.00153		0.0036	0.0036	0.00076	0.00076
N=Es/Ec		15		15	15	15	15
X0	cm	22.1		18.1	18.1	217.5	162.9
K=X0/D		0.192		0.279	0.279	0.902	0.676
$M/(B\times D^2)$	kgf/cm^2	1.86		2.755	5.447	1.325	1.393
S/(B×D)	kgf/cm^2	3.653		6.662	0	0.115	0.582
(C)		11.116		7.904	7.904	11.051	10.465
(S)		46.706		20.43	20.43	1.195	5.013
(Z)		1.068		1.103	1.103	1.619	1.984
σc	kgf/cm^2	20.7		21.8	43.1	14.6	14.6
σs	kgf/cm^2	1303		844	1669	24	105
τ	kgf/cm^2	3.65		6.66	0	0.12	0.58
σca	kgf/cm^2	75		75	75	75	112.5
σsa	kgf/cm^2	1800		1800	1800	1800	2700
τa	kgf/cm^2	7.6(α=2)		7.6(α=2)	7.6(α=2)	3.8	5.7
Note							

(2) Gate pier

Principle of structural calculation for gate pier is shown as follows.

- Scattering angle for weight of gate column and maintenance bridge is 22° 30'(= 45° /2).
- Semicircular section at both side and blockout section for gate sheet are not effective width.
- Bar arrangement for gate pier is considered dividing into two parts about gate column section and maintenance bridge section.

Positions of calculation and dimension are shown as follows.



(i) Consideration of gate column section

< Normal case >

a) Effecting load from gate column

$Vt = 292.13 + 105.00 \times 2 = 502.13$ tf

$Ht = 6.93$ tf (for wind pressure)

$Zt = 0.00$m

$Yt = 6.59$ m

(Distance from center of gravity for wind pressure to position C – C)

= 10.64 m

(Distance from center of gravity for wind pressure to position B –B)

= 14.69 m

(Distance from center of gravity for wind pressure to position A –A)

b) Weight of gate pier

<u>Position B – B</u>

$Vb = 8.355 \times 4.05 \times 2.50 \times 2.50$ tf/m^3 = 211.49 tf

$Zb = 0.00$ m

<u>Position A – A</u>

$Va = 10.105 \times 8.10 \times 2.50 \times 2.50$ tf/m^3 = 511.57 tf

$Za = 0.00$ m

c) Wind pressure

<u>Position B – B</u>

$Wb = (EL\ 9.00 - EL\ 5.20) \times 8.355 \times 0.15$ tf/m^2 = 4.76 tf

$Zb = 1.90$ m

Position A – A

$Wa = (EL\ 9.00 - El\ 5.20) \times 10.105 \times 0.15$ tf/m^2 = 5.76 tf

$Za = 1.90 + 4.05 = 5.95$ m

d) Total of load for normal case

Position	Item	Vertical force			Horizontal force		
		V (tf)	Z (m)	Mz (tf-m)	H (tf)	Y (m)	Hy (tf-m)
A – A	Weight of gate column	502.13	0.00	0.00	6.93	14.69	101.80
	Weight of gate pier	511.57	0.00	0.00			
	Wind pressure				5.76	5.95	34.27
	Total	1,013.70			12.69		136.07
B – B	Weight of gate column	502.13	0.00	0.00	6.93	10.64	73.74
	Weight of gate pier	211.49	0.00	0.00			
	Wind pressure				4.76	1.90	9.04
	Total	713.62			11.69		82.78
C - C	Weight of gate column	502.13	0.00	0.00	6.93	6.59	45.67
	Weight of gate pier						
	Wind pressure						
	Total	502.13			6.93		45.67

< Seismic case >

a) Effecting load from gate column

$Vts = 292.13$ tf

$Hts = 292.13 \times 0.12 = 35.06$ tf (for earthquake load)

$Zte = 0.00$m

$Yte = 5.77$ m

(Distance from center of gravity at gate column to position C – C)

$= 9.82$ m

(Distance from center of gravity at gate column to position B –B)

$= 13.87$ m

(Distance from center of gravity at gate column to position A –A)

b) Weight of gate pier

<u>Position B – B</u>

$Vbs = 8.355 \times 4.05 \times 2.50 \times 2.50$ tf/m^3 $= 211.49$ tf

$Hbs = Vbs \times 0.12 = 25.38$ tf

$Zbs = 0.00$ m

$Ybs = 4.05 / 2 = 2.03$ m

<u>Position A – A</u>

$Vas = 10.105 \times 8.10 \times 2.50 \times 2.50$ tf/m^3 $= 511.57$ tf

$Has = Vas \times 0.12 = 61.39$ tf

Za = 0.00 m

Yas = 8.10 / 2 = 4.05 m

c) Total of load for seismic case

Position	Item	Vertical force			Horizontal force		
		V (tf)	Z (m)	Mz (tf-m)	H (tf)	Y (m)	Hy (tf-m)
A – A	Weight of gate column	292.13	0.00	0.00	35.06	13.87	485.47
	Weight of gate pier	511.57	0.00	0.00	61.39	4.05	248.63
	Total	803.70			96.45		734.10
B – B	Weight of gate column	292.13	0.00	0.00	35.06	9.82	344.29
	Weight of gate pier	211.49	0.00	0.00	25.38	2.03	51.52
	Total	503.62			60.44		395.81

(ii) Consideration of maintenance bridge section

Loading condition is seismic time. Effective width is 4.75 m.

a) Load of maintenance bridge

V_0 = 344.00 tf

H_0 = V0×0.12 = 41.28 tf

Z_0 = 0.00 m

Y_0 = 1.50 / 2 + 4.00 = 4.75 m

(Distance from center of gravity at maintenance bridge to position B – B)

= 1.50 / 2 + 8.00 = 8.75 m

(Distance from center of gravity at maintenance bridge to position A – A)

b) Weight of gate pier

<u>Position B - B</u>

Vbs = 6.407×4.00×2.50×2.50 ft/m³ = 160.18 tf

Hbs = Vbs×0.12 = 19.22 tf

Zbs = 0.00 m

Ybs = 4.00 / 2 = 2.00 m

<u>Position A – A</u>

Vbs = 8.064×8.00×2.50×2.50 tf/m³ = 403.20 tf

Hbs = Vbs×0.12 = 48.38 tf

Zbs = 0.00 m

Ybs = 8.00 / 2 = 4.00 m

c) Total of load for seismic case

Position	Item	Vertical force			Horizontal force		
		V (tf)	Z (m)	Mz (tf-m)	H (tf)	Y (m)	Hy (tf-m)
A – A	Weight of maintenance bridge	344.00	0.00	0.00	41.28	8.75	361.20
	Weight of gate pier	403.20	0.00	0.00	48.38	4.00	193.52
	Total	747.20			89.66		554.72
B – B	Weight of maintenance bridge	344.00	0.00	0.00	41.28	4.75	196.08
	Weight of gate pier	160.18	0.00	0.00	19.22	2.00	38.44
	Total	504.18			60.50		234.52

Results of strength calculation for gate pier are shown as follows.

		For gate column		Control deck base with blockout		For maintenance bridge	
		Position A - A	Position B – B	Position A - A	Position B - B	Position A - A	Position B – B
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	136.07	734.1	82.78	395.81	554.72	234.52
N	tf	1013.7	803.7	713.62	503.62	747.2	504.18
S	tf	12.69	96.45	11.69	60.44	89.66	60.5
B	cm	651	651	476	476	696	406
D	cm	241	241	241	241	241	241
Ac	cm^2	156891	156891	114716	114716	167736	97846
As	cm^2	D22-250 =98.95	D22-250 =98.95	D22-250 =72.35	D22-250 =72.35	D22-250 =105.79	D22-250 =61.71
$P=As/(B\times D)$		0.00063	0.00063	0.00063	0.00063	0.00063	0.00063
N=Es/Ec		15	15	15	15	15	15
X0	cm	241	115.1	241	139.8	149.6	222.8
K=X0/D		1	0.477	1	0.58	0.621	0.924
$M/(B\times D^2)$	kgf/cm^2	0.36	1.942	0.299	1.432	1.372	0.995
$S/(B\times D)$	kgf/cm^2	0.081	0.615	0.102	0.527	0.535	0.618
(C)		23.62	11.555	26.416	10.827	10.654	11.228
(S)		-11.951	12.649	-14.747	7.835	6.504	0.918
(Z)		1.453	2.224	1.453	2.166	2.108	1.59
σc	kgf/cm^2	8.5	22.4	7.9	15.5	14.6	11.2
σs	kgf/cm^2	65	368	66	168	134	14
τ	kgf/cm^2	0.08	0.61	0.1	0.53	0.53	0.62
σca	kgf/cm^2	75	112.5	75	112.5	112.5	112.5
σsa	kgf/cm^2	1600	2400	1600	2400	2400	2400
τa	kgf/cm^2	3.8	5.7	3.8	5.7	5.7	5.7
Note		Normal case	Seismic case	Normal case	Seismic case	Seismic case	Seismic case

(iii) Consideration of downstream apron stage section

Maintenance bridge has 4 main girders. Then, weight of maintenance bridge is equally divided into 4 parts.

Load

a) Normal case

<u>Weight of maintenance bridge</u>

$V_0 = 344.00$ tf

$V_1 = V_0 / 4 = 86.00$ tf

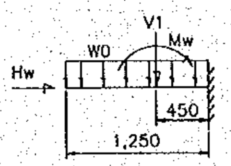
<u>Wind pressure</u>

$Hw = (2.40 \times 18.50 + 1.00 \times 2.50) \times 0.15 \text{ tf/m}^2 = 7.04$ tf

$Mw = 7.04 \times (2.40 / 2 + 0.50) = 11.97$ tf-m

<u>Weight of apron stage</u>

$W_0 = 1.00 \times 1.25 \times 2.50 \text{ tf/m}^3 = 3.13 \text{ tf/m}^2$



b) Seismic case

<u>Weight of maintenance bridge</u>

$V_0 = 344.00$ tf

$V_1 = V_0 / 4 = 86.00$ tf

<u>Weight of apron stage</u>

$W_0 = 1.00 \times 1.25 \times 2.50 \text{ tf/m}^3 = 3.13 \text{ tf/m}^2$

<u>Seismic load</u>

$W_1 = 1.25 \times 2.50 \times 1.00 \times 2.50 \text{ tf/m}^3 \times 0.12 = 0.94$ tf

$W_2 = V1 \times 0.12 = 10.32$ tf

$He = W1 + W2 = 11.26$ tf

$Me = 10.32 \times 0.50 = 5.16$ tf-m

c) Strength calculation

Geometrical moment of inertia

$$I = 2.50 \times 1.00^3 / 12 = 0.208333 \text{ m}^4$$

Area of section

$A = 2.50 \times 1.00 = 2.500 \text{ m}^2$

Normal case	M (tf-m)	38.06	Seismic case	M (tf-m)	35.99
	N (tf)	7.04		N (tf)	11.26
	S (tf)	89.91		S (tf)	89.91

Results of strength calculation are shown as follows.

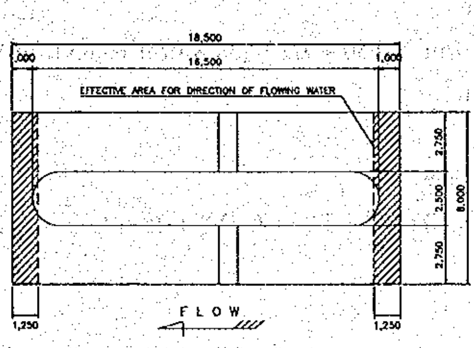
		For apron stage section	
		Normal case	Seismic case
Member of shape		Rectangle	Rectangle
M	tf-m	38.02	35.99
N	tf	7.04	11.26
S	tf	89.91	89.91
B	cm	250	250
D	cm	91	91
Ac	cm^2	22750	22750
As	cm^2	9-D19 =25.56	9-D19 =25.56
P=As/(B×D)		0.00112	0.00112
N=Es/Ec		15	15
X0	cm	16.4	17.2
K=X0/D		0.18	0.189
M/(B×D^2)	kgf/cm^2	1.836	1.738
S/(B×D)	kgf/cm^2	3.952	3.952
(C)		12.822	12.91
(S)		58.443	55.445
(Z)		1.064	1.072
σ c	kgf/cm^2	23.5	22.4
σ s	kgf/cm^2	1610	1446
τ	kgf/cm^2	3.95	3.95
σ ca	kgf/cm^2	75	112.5
σ sa	kgf/cm^2	1800	2700
τ a	kgf/cm^2	7.6	11.4
Note			

(3) Footing of center pier

< Direction of flowing water >

Normal case

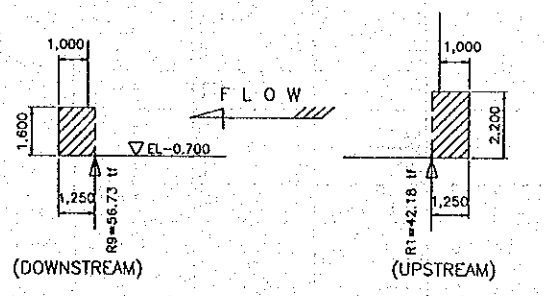
a) Effective area



Design effective length

$= 1.00 + D / 10 = 1.00 + 2.50 / 10 = 1.25$ m

b) Reaction force



c) Weight of water

<u>Upstream side</u>

$Wu = 3.70 \times 1.25 \times 8.00 \times 1.00$ tf/m^3 = 37.00 tf

$Xw = 1 / 2 \times 1.25 = 0.63$ m

<u>Downstream side</u>

$Wd = 0.60 \times 1.25 \times 8.00 \times 1.25$ tf/m^3 = 7.50 tf

$Xw = 1 / 2 \times 1.25 = 0.63$ m

d) Weight of muddy soil

<u>Upstream side</u>

$Ws = 1.00 \times 1.25 \times 8.00 \times 1.00$ tf/m^3 = 10.00 tf

$Xs = 1 / 2 \times 1.25 = 0.63$ m

e) Weight of footing

<u>Upstream side</u>

$W0u = 2.20 \times 1.25 \times 8.00 \times 2.50$ tf/m^3 = 55.00 tf

$X0u = 1 / 2 \times 1.25 = 0.63$ m

<u>Downstream</u>

$W0d = 1.60 \times 1.25 \times 8.00 \times 2.50$ tf/m^3 = 40.00 tf

$X0d = 1 / 2 \times 1.25 = 0.63$ m

f) Uplift

Upstream side

Upu = {6.14 tf/m^2 $\times$ 0.50 + 1/2 $\times$ (3.88 tf/m^2+3.85 tf/m^2) $\times$ 0.75} $\times$ 8.00

= 47.75 tf

Xpu = 0.70 m

Downstream side

Upd = 1/2 $\times$ (3.20 tf/m^2 + 3.25 tf/m^2) $\times$ 1.25 $\times$ 8.00 = 32.25 tf

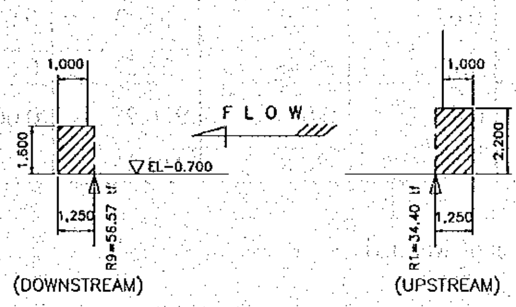
Xpd = 0.62 m

g) Total of load

	Upstream side			Downstream side		
	V (tf)	X (m)	Mx (tf-m)	V (tf)	X (m)	Mx (tf-m)
Weight of water	-37.00	0.63	-23.31	-6.00	0.63	-3.78
Weight of muddy soil	-10.00	0.63	-6.30	0.00	0.00	0.00
Weight of footing	-55.00	0.63	-34.65	-40.00	0.63	-25.20
Uplift	47.75	0.70	33.43	32.25	0.63	20.32
Reaction force	168.72	0.00	0.00	226.92	0.00	0.00
Total	114.47		-30.83	213.17		-8.66

Seismic case

a) Reaction force



b) Weight of water

<u>Upstream side</u>

Wu = 3.70×1.25×8.00×1.00 tf/m^3 = 37.00 tf

Xw = 1 / 2×1.25 = 0.63 m

<u>Downstream side</u>

Wd = 0.60×1.25×8.00×1.25 tf/m^3 = 7.50 tf

Xw = 1 / 2×1.25 = 0.63 m

c) Weight of muddy soil

<u>Upstream side</u>

Ws = 1.00×1.25×8.00×1.00 tf/m^3 = 10.00 tf

Xs = 1 / 2×1.25 = 0.63 m

d) Weight of footing

<u>Upstream side</u>

W0u = 2.20×1.25×8.00×2.50 tf/m^3 = 55.00 tf

X0u = 1 / 2×1.25 = 0.63 m

<u>Downstream</u>

W0d = 1.60×1.25×8.00×2.50 tf/m^3 = 40.00 tf

X0d = 1 / 2×1.25 = 0.63 m

e) Uplift

Upstream side

$Upu = \{6.14\ tf/m^2 \times 0.50 + 1/2 \times (3.88\ tf/m^2 + 3.85\ tf/m^2) \times 0.75\} \times 8.00$

$= 47.75\ tf$

$Xpu = 0.70\ m$

Downstream side

$Upd = 1/2 \times (3.20\ tf/m^2 + 3.25\ tf/m^2) \times 1.25 \times 8.00 = 32.25\ tf$

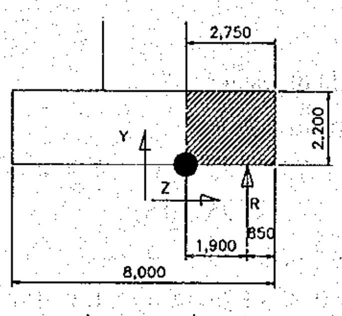
$Xpd = 0.62\ m$

f) Total of load

	Upstream side			Downstream side		
	V (tf)	X (m)	Mx (tf-m)	V (tf)	X (m)	Mx (tf-m)
Weight of water	-37.00	0.63	-23.31	-6.00	0.63	-3.78
Weight of muddy soil	-10.00	0.63	-6.30	0.00	0.00	0.00
Weight of footing	-55.00	0.63	-34.65	-40.00	0.63	-25.20
Uplift	47.75	0.70	33.43	32.25	0.63	20.32
Reaction force	137.60	0.00	0.00	226.28	0.00	0.00
Total	83.35		-30.83	212.53		-8.66

< Direction of weir axis >

Normal case



a) Reaction force

$R = \Sigma(R1 + \ldots\ldots + R9) = 445.07\ tf$

$Z = 1.90\ m$

b) Weight of water

<u>Upstream side</u>

$W1 = (EL\ 5.20 - EL\ 1.50) \times 2.75 \times (7.75 - 2.50) \times 1.00\ tf/m^3$

$= 53.42\ tf$

<u>Downstream side</u>

$W2 = 0.60 \times 9.70 \times 1.00\ tf/m^3 = 5.87\ tf$

$\Sigma W = W1 + W2 = 59.29\ tf$

$Z = 1/2 \times 2.75 = 1.38\ m$

c) Weight of muddy soil

$Ws = 1.00 \times 2.75 \times (7.75 - 2.50) \times 1.00$ tf/m3 = 14.44 tf

$Zs = 1/2 \times 2.75 = 1.38$ m

d) Weight of footing

$W0 = \{1.60 \times 18.50 + (8.65 + 7.75) \times 1/2 \times 0.60\} \times 2.75 \times 2.50$ tf/m3

= 237.33 tf

$Z = 1/2 \times 2.75 = 1.38$ m

e) Uplift

$Up = \{6.14$ tf/m2 $\times 0.50 + 1/2 \times (3.88$ tf/m2 $+ 3.20$ tf/m2$) \times 18.00\}$

$\times 2.75 = 183.67$ tf

$Zp = 1/2 \times 2.75 = 1.38$ m

f) Total of load

	V (tf)	Z (m)	Mz (tf-m)
Weight of water	-59.24	1.38	-81.75
Weight of muddy soil	-14.44	1.38	-19.93
Weight of footing	-237.33	1.38	-327.52
Uplift	183.67	1.38	253.46
Reaction force	445.07	1.90	845.63
Total	317.73		669.89

Seismic case

a) Reaction force

$R = \Sigma(R1 + \ldots\ldots + R9) = 409.41$ tf

$Z = 1.90$ m

b) Weight of water

<u>Upstream side</u>

$W1 = (EL\ 5.20 - EL\ 1.50) \times 2.75 \times (7.75 - 2.50) \times 1.00$ tf/m^3

= 53.42 tf

<u>Downstream side</u>

$W2 = 0.60 \times 9.70 \times 1.00$ tf/m^3 = 5.87 tf

$\Sigma W = W1 + W2 = 59.29$ tf

$Z = 1/2 \times 2.75 = 1.38$ m

c) Weight of muddy soil

$Ws = 1.00 \times 2.75 \times (7.75 - 2.50) \times 1.00$ tf/m3 = 14.44 tf

$Zs = 1/2 \times 2.75 = 1.38$ m

d) Weight of footing

$W0 = \{1.60 \times 18.50 + (8.65 + 7.75) \times 1/2 \times 0.60\} \times 2.75 \times 2.50$ tf/m3

= 237.33 tf

$Z = 1/2 \times 2.75 = 1.38$ m

e) Uplift

$Up = \{6.14$ tf/m2 $\times 0.50 + 1/2 \times (3.88$ tf/m2 $+ 3.20$ tf/m2$) \times 18.00\}$

$\times 2.75 = 183.67$ tf

$Zp = 1/2 \times 2.75 = 1.38$ m

f) Total of load

	V (tf)	Z (m)	Mz (tf-m)
Weight of water	-59.24	1.38	-81.75
Weight of muddy soil	-14.44	1.38	-19.93
Weight of footing	-237.33	1.38	-327.52
Uplift	183.67	1.38	253.46
Reaction force	409.41	1.90	777.88
Total	282.07		602.14

Results of strength calculation are shown as follows.

		Direction of flowing water		Direction of flowing water		Direction of weir axis	
		Upstream side	Downstream side	Upstream side	Downstream side	Normal case	Seismic case
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-30.83	-30.83	-8.66	-8.66	669.89	602.14
N	tf	0	0	0	0	0	0
S	tf	114.47	83.35	213.17	212.53	317.73	282.07
B	cm	250	250	250	250	1183.5	1183.5
D	cm	211	211	151	151	143.5	143.5
Ac	cm^2	52750	52750	37750	37750	169832.3	169832.3
As	cm^2	D19-250 =93.72	D19-250 =93.72	D19-250 =93.72	D19-250 93.72	D22-125 =359.8	D22-125 =359.8
P=As/(B×D)		0.00178	0.00178	0.00248	0.00248	0.00212	0.00212
N=Es/Ec		15	15	15	15	15	15
X0	cm	43.4	43.4	36	36	31.9	31.9
K=X0/D		0.206	0.206	0.238	0.238	0.222	0.222
M/(B×D^2)	kgf/cm^2	0.277	0.277	0.152	0.152	2.749	2.471
S/(B×D)	kgf/cm^2	2.17	1.58	5.647	5.63	1.871	1.661
(C)		10.436	10.436	9.121	9.121	9.716	9.716
(S)		40.286	40.286	29.169	29.169	33.986	33.986
(Z)		1.074	1.074	1.086	1.086	1.08	1.08
σc	kgf/cm^2	2.9	2.9	1.4	1.4	26.7	24
σs	kgf/cm^2	167	167	66	66	1401	1260
τ	kgf/cm^2	2.17	1.58	5.65	5.63	1.87	1.66
σca	kgf/cm^2	75	112.5	75	112.5	75	112.5
σsa	kgf/cm^2	1600	2400	1600	2400	1600	2400
τa	kgf/cm^2	7.6	11.4	7.6	11.4	7.6	11.4
Note		Normal case	Seismic case	Normal case	Seismic case		

(4) Pile head treatment

a) Vertical bearing pressure for footing concrete

Pnmax = 63.47 tf/pile (Direction of weir axis in seismic case)

$$\sigma_{cv} = \frac{P_{N\max}}{\pi D^2/4} = \frac{63470}{\pi/4 \times 60^2} = 22.45 \text{ kgf/cm}^2$$

< σ ca = 60.0×1.5 = 90.0 kgf/cm² ....O.K

b) Punching shear stress for footing concrete

$$\tau_V = \frac{P_{N\max}}{\pi(D+h)h} = \frac{63470}{\pi(60+125)\times 125} = 0.87 \text{ kgf/cm}^2$$

< τ ca3 = 8.8 kgf/cm²..O.K

where

h : Height of between top of footing and pile head (cm)

c) Horizontal bearing pressure for footing concrete

$$\sigma_{ch} = \frac{H}{D\ell}$$

where

ℓ : Stuffing length of pile (cm)

D : Pile diameter (cm)

H : Shearing pressure (kgf)

Pnmax = 56.57 tf/pile (Direction of flowing water in seismic case)

M = 15.79 tf-m

S = 15.49 tf

$$\sigma_{ch} = \frac{15490}{60\times 10} = 25.82 \text{ kgf/cm}^2 < \sigma \text{ ca} = 60.0\times 1.5 = 90.0 \text{ kgf/cm}^2$$

.... O.K

d) Vertical punching shearing stress to pile on edge of footing

$$\tau_h = \frac{H}{h'(2\ell + D + 2h')}$$

where

h' : Effective thickness to vertical punching stress on footing (cm)

l : Stuffing length of pile (cm)

D : Pile diameter (cm)

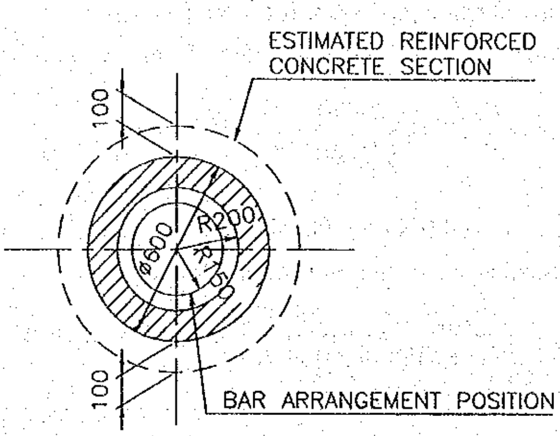
H : Shearing pressure (kgf)

h' = 55 cm (Direction of weir axis)

$$\tau_h = \frac{15490}{55\times(2\times 10 + 60 + 2\times 55)}$$

= 1.48 kgf/cm² < τ ca3 = 8.8 kgf/cm²........................O.K

e) Strength of estimated reinforced concrete section



Pnmin = 27.51 tf/pile (Direction of weir axis in seismic case)

M = 15.08 tf-m

S = 6.98 tf

D = 60 + 10×2 = 80 cm

a = 20.0 cm×2 = 40.0 cm

d = 25.0 cm

Result of strength calculation is shown as follows.

Member of shape		Circle			
M	tf-m	-15.08	X0	cm	22.50
N	tf	27.51	K= X0/H		0.281
S	tf	6.98	M/(B×H^2)	kgf/cm^2	2.945
B	cm	80.00	S/(B×H)	kgf/cm^2	1.091
H	cm	80.00	(C)		32.518
D	cm	55.00	(S)		47.031
DD	cm	25.00	(Z)		3.406
DG	cm	25.00	σ c	kgf/cm^2	95.80
B0 , R	cm	40.00	σ s	kgf/cm^2	2078.00
H0 , R0	cm	20.00	τ	kgf/cm^2	0.00
AC	cm^2	3769.9	σ ca	kgf/cm^2	112.50
AS , AS1	cm^2	8-D16 = 16.08	σ sa	kgf/cm^2	2400.00
P , P1		0.00427	τ a	kgf/cm^2	5.40
N= ES/EC		15			

f) Reinforcing bar at pile head treatment

Fixing length of reinforcing bar at footing

$$L_1 \geqq L_0$$

Where

L0 : 35 D (mm)

D : Diameter of reinforcing bar (mm)

$L_1 = 35 \times 16 = 560 \fallingdotseq 600$ mm

Fixing length of reinforcing bar at pile

$L_2 \geqq 50\phi + L_0$

Where

ϕ : Diameter of PC steel bar (mm)

$L2 = 50 \times 9.0 + 600 = 1050$ mm

g) Depth of concrete filling

Depth of concrete filling is the same fixing length of reinforcing bar at pile.

$L_3 = 1050$ mm