3. Stress-Strain Calculation

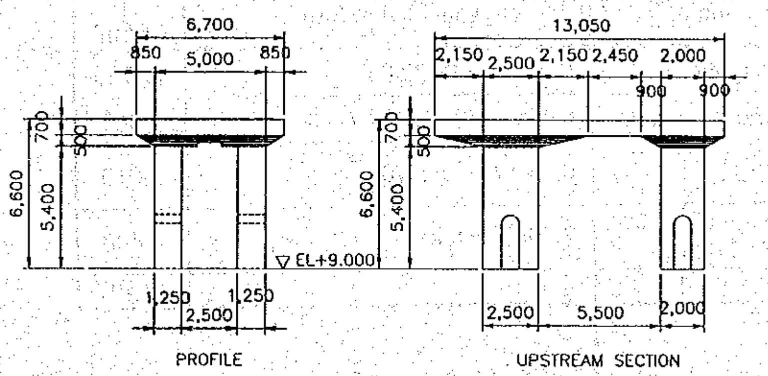
Stress-strain calculations of the structure are made to decide proper reinforcing bar arrangement. Described below are the bar arrangement for the center pier. Deformed steel bars are used for all parts of structure, and bar spacing will be 125 mm or 250 mm.

(1) Gate column and Control Deck

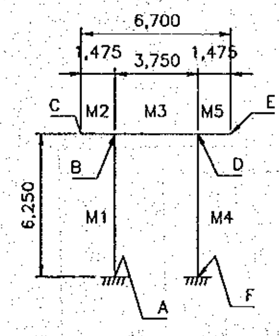
Direction of flowing water

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	$\frac{1}{12}\times0.85\times1.25^3\times2+\frac{1}{12}\times0.60\times1.25^3\times2$	0.47201
2	$\frac{1}{12}\times13.05\times0.70^3$	0.37301
3	$\frac{1}{12}\times13.05\times0.70^3$	0.37301
4	$\frac{1}{12}\times0.85\times1.25^3\times2+\frac{1}{12}\times0.60\times1.25^3\times2$	0.47201
5	$\frac{1}{12}\times13.05\times0.70^3$	0.37301

Section area

Member	Calculation	Area (m^2)
1	$(0.85\times1.25)\times2+(0.60\times1.25)\times2$	3.625
2	13.05×0.70	9.135
3	13.05×0.70	9.135
4	$(0.85\times1.25)\times2+(0.60\times1.25)\times2$	3.625
5	13.05×0.70	9.135

b) Loading Calculation

Normal Case

[Control deck]

<u>Distributed load</u>

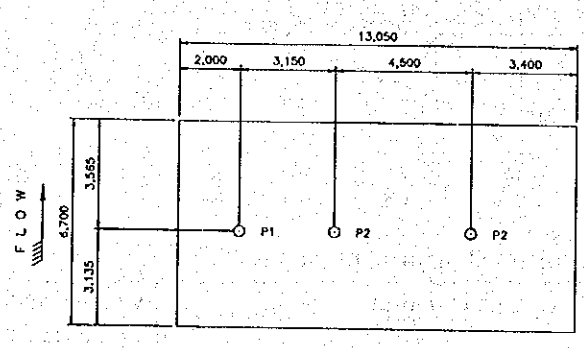
Position of load	Calculation form	W (tf / m)
Weight of body	$0.70\times6.70\times2.50tf/m^3$	11.725
Cinder concrete	$0.15\times6.70\times2.35tf/m^3$	2.362
Sidewake live load	$0.30tf/m^2\times6.70$	2.010
Weight of control house	$2.20tf/m\times2$	4.400
Total		20.497

<u>Load of control house</u>

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

$p_0 = 2.20tf/m\times13.05m = 28.71tf$

<u>Hoisting load</u>



Condition	P1 (tf / place)	P2 (tf / place)
Normal case	105.00	35.00
Seismic case	25.00	12.00

Wind pressure

For control house

$Hw = (4.00+0.70)\times 13.05\times 0.15\ tf/m^2 = 9.20\ tf$

$Mw = 9.20\times(1/2\times 4.00+0.35) = 21.62\ tf-m$

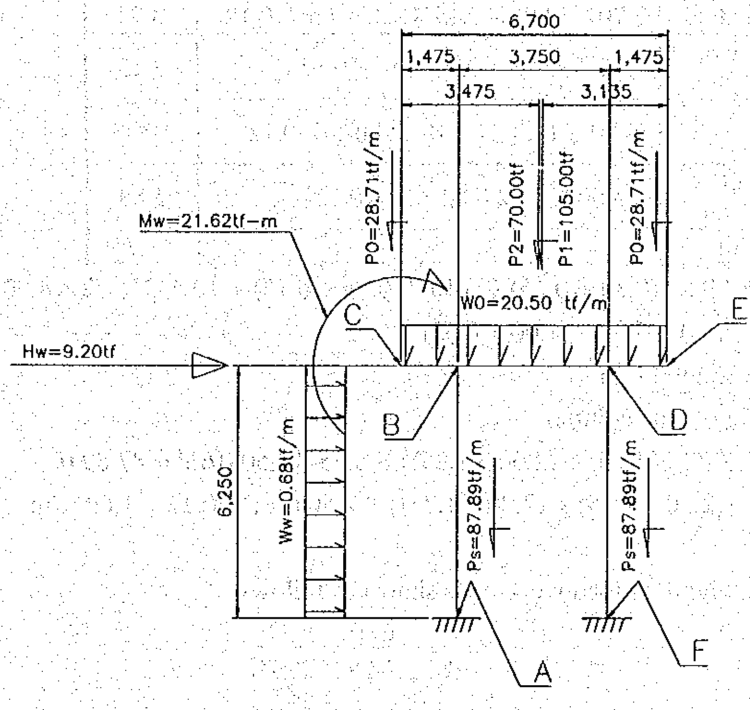
For gate column

$Ww = (2.50 + 2.00)\times 0.15\ tf/m^2 = 0.68\ tf/m$

Weight of gate column

$Ps = \{(2.50\times 1.25) + (2.00\times 1.25)\}\times 2.50\ tf/m^3\times 6.25 = 87.89\ tf$

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



Seismic case

[Control deck]

<u>Distributed load</u>

W0 = 20.50 tf/m (refer to normal case)

<u>Load of control house</u>

P0 = 28.71 tf (refer to normal case)

<u>Seismic load</u>

Position of load	Calculation form	W (tf / m)
Weight of body	$6.70\times13.05\times0.70\times2.50\ tf/m^3\times0.12$	18.36
Cinder concrete	$6.70\times13.05\times0.15\times2.35\ tf/m^3\times0.12$	3.94
Sidewalk live load	$6.70\times13.05\times0.30\ tf/m^2\times0.12$	3.15
Weight of control house	$(6.70+13.05)\times2\times2.20\ tf/m\times0.12$	10.43
Weight of hoisting system	$(25.00\ tf+12.00\ tf\times2)\times0.12$	5.88
Total (He)		41.76

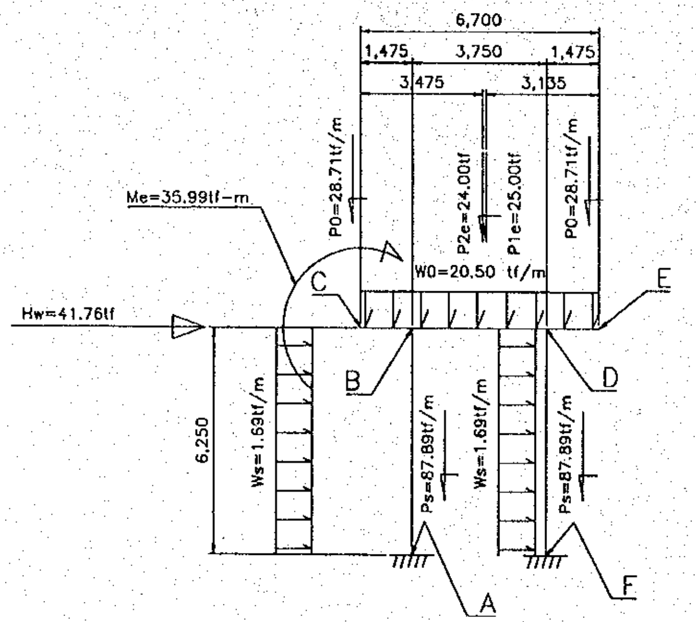
$$Me = 3.94\times(0.35+0.15)+3.15\times(0.35+0.15)+10.43\times(1/2\times4.00+0.35)+5.88\times(1.00+0.35)$$
$$= 35.99\ tf-m$$

<u>Weight of gate column</u>

$Ps = \{(2.50\times1.25)+(2.00\times1.25)\}\times6.25\times2.50\ tf/m^3 = 87.89\ tf$

$Ws = \{(2.50\times1.25)+(2.00\times1.25)\}\times2.50\ tf/m^3\times0.12 = 1.69\ tf/m$

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	6.250	7.320	2.060	-250.166
		Minimum	0.000	-18.835	6.310	-250.166
	M2	Maximum	0.000	23.010	-28.710	-9.490
		Minimum	1.475	-41.637	-58.948	-9.490
	M3	Maximum	2.000	131.339	62.328	-7.430
		Minimum	3.750	-87.777	-148.547	-7.430
	M4	Maximum	6.250	23.129	7.430	-295.384
		Minimum	0.000	-23.309	7.430	-295.384
	M5	Maximum	1.475	0.000	28.710	0.000
		Minimum	0.000	-64.647	58.948	0.000
Seismic Case	M1	Maximum	6.250	65.206	18.549	-166.850
		Minimum	0.000	-83.732	29.111	-166.850
	M2	Maximum	0.000	24.200	-28.710	-36.570
		Minimum	1.475	-40.447	-58.948	-36.570
	M3	Maximum	0.976	34.527	0.000	-18.021
		Minimum	3.750	-127.833	-105.862	-18.021
	M4	Maximum	6.250	63.186	18.021	-252.700
		Minimum	0.000	-82.454	28.584	-252.700
	M5	Maximum	1.475	0.000	28.710	0.000
		Minimum	0.000	-64.647	58.947	0.000

Effective area and bar arrangement

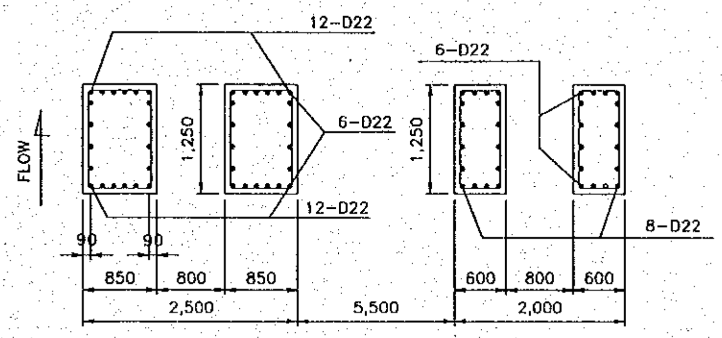
<Gate column>

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 2.90 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.



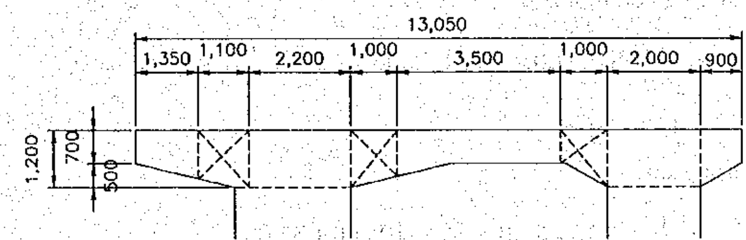
SECTION OF GATE COLUMN

Results of Strength calculation for gate columns are shown as follows.

		Member 1			
		Outside	Inside	Inside	Outside
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	7.32	-18.84	65.21	-83.73
N	tf	0	0	0	0
S	tf	2.06	6.31	18.55	29.11
B	cm	290	290	290	290
D	cm	116	116	116	116
Ac	cm^2	33640	33640	33640	33640
As	cm^2	(12+8)-D22 =76.0	(12+8)-D22 =76.0	(12+8)-D22 =76.0	(12+8)-D22 =76.0
P=As/(B×D)		0.00226	0.00226	0.00226	0.00226
N=Es/Ec		15	15	15	15
X0	cm	26.5	26.5	26.5	26.5
K=X0/D		0.229	0.229	0.229	0.229
M/(B×D^2)	kgf/cm^2	0.188	0.483	1.671	2.146
S/(B×D)	kgf/cm^2	0.061	0.188	0.551	0.865
(C)		9.469	9.469	9.469	9.469
(S)		31.943	31.943	31.943	31.943
(Z)		1.083	1.083	1.083	1.083
σ c	kgf/cm^2	1.8	4.6	15.8	20.3
σ s	kgf/cm^2	90	231	801	1028
τ	kgf/cm^2	0.06	0.19	0.55	0.87
σ ca	kgf/cm^2	75	75	112.5	112.5
σ sa	kgf/cm^2	1800	1800	2700	2700
τ a	kgf/cm^2	3.8	3.8	5.7	5.7
Note		Normal case	Normal case	Seismic case	Seismic case

		Member 4			
		Outside	Inside	Inside	Outside
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-23.31	23.13	63.19	-82.45
N	tf	0	0	0	0
S	tf	7.43	7.43	18.02	28.58
B	cm	290	290	290	290
D	cm	116	116	116	116
Ac	cm^2	33640	33640	33640	33640
As	cm^2	(12+8)-D22 =76.0	(12+8)-D22 =76.0	(12+8)-D22 =76.0	(12+8)-D22 =76.0
P=As/(B×D)		0.00226	0.00226	0.00226	0.00226
N=Es/Ec		15	15	15	15
X0	cm	26.5	26.5	26.5	26.5
K=X0/D		0.229	0.229	0.229	0.229
$M/(B\times D^2)$	kgf/cm^2	0.597	0.593	1.619	2.113
S/(B×D)	kgf/cm^2	0.221	0.221	0.536	0.85
(C)		9.469	9.469	9.469	9.469
(S)		31.943	31.943	31.943	31.943
(Z)		1.083	1.083	1.083	1.083
σ c	kgf/cm^2	5.7	5.6	15.3	20
σ s	kgf/cm^2	286	284	776	1012
τ	kgf/cm^2	0.22	0.22	0.54	0.85
σ ca	kgf/cm^2	75	75	112.5	112.5
σ sa	kgf/cm^2	1800	1800	2700	2700
τ a	kgf/cm^2	3.8	3.8	5.7	5.7
Note		Normal case	Normal case	Seismic case	Seismic case

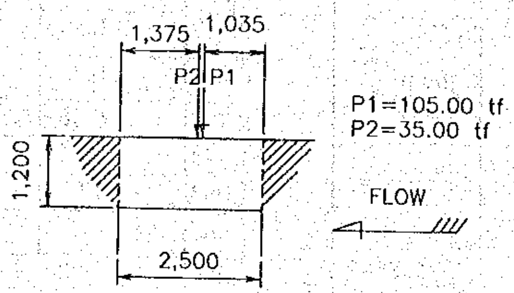
<Control deck>



<Riverside control deck>

Distributed load

Position of load	Calculation form	W (tf / m^2)
Weight of body	$1.20 \times 2.50 tf / m^3$	3.000
Cinder concrete	$0.15 \times 2.35 tf / m^3$	0.350
Sidewake live load		0.300
Weight of control house		0.500
Total		4.150



$q0 = 4.15\ tf/m^2 \times 2.20 = 9.13\ tf/m$

Bending moment and Shearing stress

Edge section moment

$$Me = -\frac{1}{8} \times (105.00 \times 2.50 + 35.00 \times 2.50) - \frac{1}{12} \times 9.13 \times 2.50^2$$

$= -48.505$ tf-m

Middle section moment

$$Mm = \frac{1}{8} \times (105.00 \times 2.50 + 35.00 \times 2.50) + \frac{1}{24} \times 9.13 \times 2.50^2$$

$= 46.127$ tf-m

Shearing stress

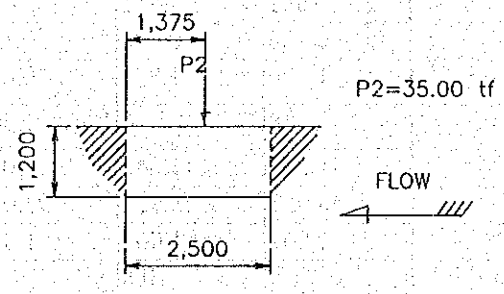
$$S = \frac{1}{2} \times 9.13 \times 2.50 + \frac{1}{2} \times (105.00 + 35.00)$$

$= 81.413$ tf

<Land side control deck>

Distributed load

Position of load	Calculation form	W (tf/m^2)
Weight of body	$1.20 \times 2.50 tf/m^3$	3.000
Cinder concrete	$0.15 \times 2.35 tf/m^3$	0.350
Sidewake live load		0.300
Weight of control house		0.500
Total		4.150



$$q0 = 4.15\ tf/m^2 \times 2.00 = 8.30\ tf/m$$

Bending moment and Shearing stress

Edge section moment

$$Me = -\frac{1}{8} \times (35.00 \times 2.50) - \frac{1}{12} \times 8.30 \times 2.50^2$$

$$= -15.260\ tf\text{-}m$$

Middle section moment

$$Mm = \frac{1}{8} \times (35.00 \times 2.50) + \frac{1}{24} \times 8.30 \times 2.50^2$$

$$= 13.099\ tf\text{-}m$$

Shearing stress

$$S = \frac{1}{2} \times 8.30 \times 2.50 + \frac{1}{2} \times (35.00)$$

$$= 27.875\ tf$$

Results of structural calculation for control deck are shown as follows.

		Riverside section		Land side section	
		Edge section	Middle section	Edge section	Middle section
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-48.51	46.13	-15.26	13.1
N	tf	0	0	0	0
S	tf	81.41	0	27.88	0
B	cm	220	220	200	200
D	cm	107	107	107	107
Ac	cm^2	23540	23540	21400	21400
As	cm^2	9-D22 =34.20	9-D22 =34.20	8-D16 =16.08	8-D16 =16.08
P=As/(B×D)		0.00145	0.00145	0.00075	0.00075
N=Es/Ec		15	15	15	15
X0	cm	20.1	20.1	14.9	14.9
K=X0/D		0.188	0.188	0.139	0.139
M/(B×D^2)	kgf/cm^2	1.926	1.831	0.666	0.572
S/(B×D)	kgf/cm^2	3.458	0	1.303	0
(C)		11.343	11.343	15.058	15.058
(S)		48.957	48.957	93.043	93.043
(Z)		1.067	1.067	1.049	1.049
σc	kgf/cm^2	21.8	20.8	10	8.6
σs	kgf/cm^2	1414	1345	930	798
τ	kgf/cm^2	3.46	0	1.3	0
σca	kgf/cm^2	75	75	75	75
σsa	kgf/cm^2	1800	1800	1800	1800
τa	kgf/cm^2	3.8	3.8	3.8	3.8

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) Loading Calculation

Normal Case

Distributed load for control deck

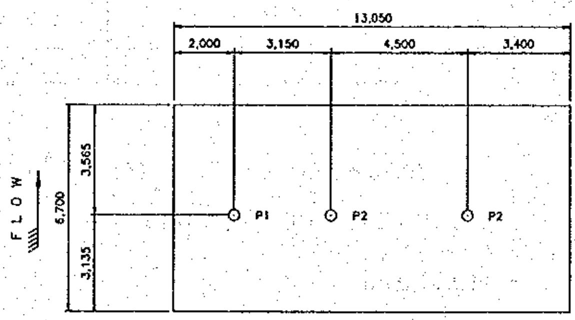
Position of load	Calculation form	W (tf / m)
Weight of body	$0.70 \times 13.05 \times 2.50 tf/m^3$	22.838
Cinder concrete	$0.15 \times 13.05 \times 2.35 tf/m^3$	4.600
Sidewake live load	$0.30 tf/m^2 \times 13.05$	3.915
Weight of control house	$2.20 tf/m \times 2$	4.400
Total		35.753

Load of control house

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

$p_0 = 2.20tf / m \times 6.70m = 14.74tf$

Hoisting load



Condition	P1 (tf / place)	P2 (tf / place)
Normal case	105.00	35.00
Seismic case	25.00	12.00

Wind pressure

For control house

$Hw = (4.00+0.70) \times 6.70 \times 0.15\ tf/m^2 = 4.72\ tf$

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

For gate column

$Ww = 1.25 \times 2 \times 0.15\ tf/m^2 = 0.38\ tf/m$

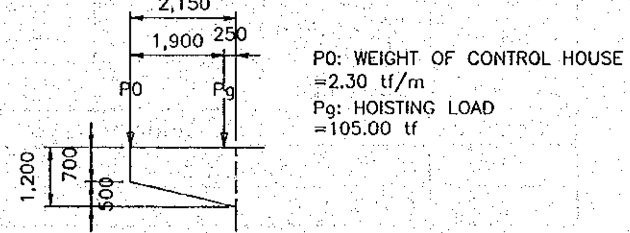
Weight of gate column

$Ps1 = (2.50 \times 1.25) \times 6.25 \times 2.50\ tf/m^3 \times 2 = 97.66\ tf$

$Ps2 = (2.00 \times 1.25) \times 6.25 \times 2.50\ tf/m^3 \times 2 = 78.13\ tf$

Stage section for control deck

Load for stage section is transformed into bending moment and shearing stress, and the force is adopted in structural calculation.



Distributed load

Position of load	Calculation form	W (tf / m^2)
Weight of body	$\frac{1}{2}\times(0.70+1.20)\times 2.50 tf/m^3$	2.375
Cinder concrete	$0.15\times 2.35 tf/m^3$	0.353
Sidewake live load		0.300
Weight of control house		0.500
Total		3.528

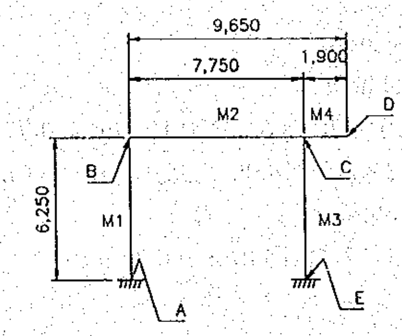
$$M_0 = \frac{1}{2}\times(6.70+5.00)\times 2.15\times 3.528\, t/m^2 \times 1.127 + 2.35\, tf/m^2 \times 6.70\times 2.15 + 105.00\, tf \times 0.25$$

$$= 110.11 \text{ tf-m}$$

$$S0 = \frac{1}{2}\times(6.70+5.00)\times 2.15\times 3.528\, t/m^2 + 2.35\, tf/m^2 \times 6.70 + 105.00\, tf$$

$$= 165.12 \text{ tf}$$

Dimension of rigid frame structure is shown as follows.



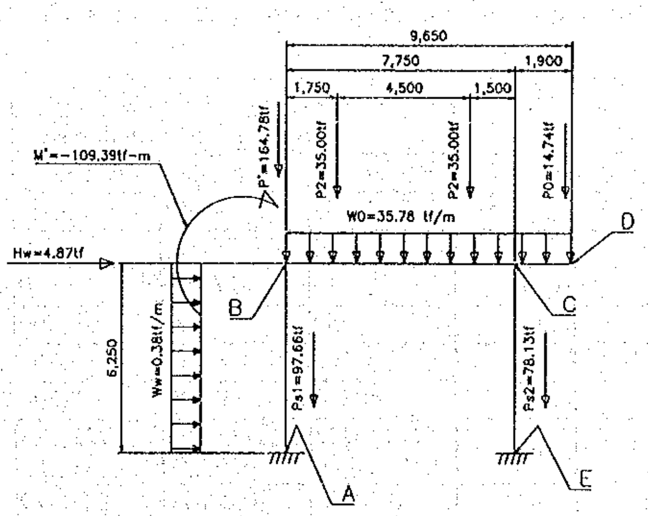
Geometrical moment of inertia

Member	Calculation	Geometrical moment of inertia (m^4)
1	$\frac{1}{12}\times 1.25\times 0.85^3\times 2\times 2$	0.2559
2	$\frac{1}{12}\times 6.70\times 0.70^3$	0.1915
3	$\frac{1}{12}\times 1.25\times 0.60^3\times 2\times 2$	0.0900
4	$\frac{1}{12}\times 6.70\times 0.70^3$	0.1915

Section area

Member	Calculation	Area (m^2)
1	(0.85×1.25)×2×2	4.250
2	6.70×0.70	4.690
3	(0.60×1.25)×2×2	3.000
4	6.70×0.70	4.690

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



Member	Condition	Distance (m)	Bending moment M (tf -m)	Shearing stress S (tf)	Axial Force N (tf)
M1	Maximum	0.000	7.719	-12.788	-437.523
	Minimum	6.250	-79.628	-15.163	-437.523
M2	Maximum	3.915	146.454	0.000	-20.033
	Minimum	0.000	-189.018	175.083	-20.033
M3	Maximum	6.250	76.551	20.033	-333.064
	Minimum	0.000	-48.655	20.033	-333.064
M4	Maximum	1.900	0.000	14.740	0.000
	Minimum	0.000	-92.589	82.722	0.000

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

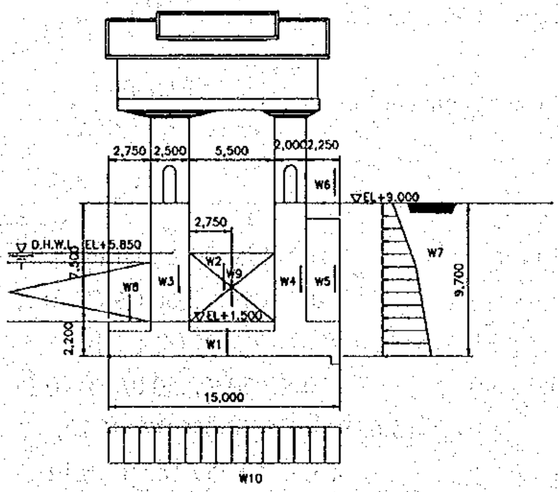
		Gate column	Control deck		Stage section
			Upper side	Bottom side	
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-79.63	-189.02	146.45	-109.39
N	tf	0	0	0	0
S	tf	15.16	175.08	0	164.78
B	cm	250	500	500	390
D	cm	161	65	65	65
Ac	cm^2	40250	32500	32500	25350
As	cm^2	(6+6)-D22 =45.6	D25-125 =201.31	D22-125 =155.8	D25-125 =265.14
P=As/(B×D)		0.00113	0.00619	0.00479	0.01046
N=Es/Ec		15	15	15	15
X0	cm	27.1	22.6	20.4	27.6
K=X0/D		0.168	0.348	0.314	0.425
M/(B×D^2)	kgf/cm^2	1.229	8.948	6.933	6.639
S/(B×D)	kgf/cm^2	0.377	5.387	0	6.5
(C)		12.601	6.5	7.112	5.484
(S)		62.339	12.175	15.533	7.426
(Z)		1.059	1.131	1.117	1.165
σ c	kgf/cm^2	15.5	58.2	49.3	36.4
σ s	kgf/cm^2	1149	1634	1615	739
τ	kgf/cm^2	0.38	5.39	0	6.5
σ ca	kgf/cm^2	75	75	75	75
σ sa	kgf/cm^2	1800	1800	1800	1800
τ a	kgf/cm^2	3.8	3.8	3.8	3.8

(2) Gate pier and Gate floor slab

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

- It regards that both gate piers are fixed on gate floor slab.
- Standard section and maintenance bridge section are calculated.
- Gate pier at riverside is referred to center pier.

<u>Standard section</u>



<Normal case>

Loading calculation

W1 (Weight of slab)	2.20×2.50 tf/m³	$= 5.50$ tf/m²
W2 (Weight of water)	3.70×1.00 tf/m³	$= 3.70$ tf/m²
W3 (Weight of pier: Riverside)	$7.50 \times 2.50 \times 2.50$ tf/m³	$= 46.89$ tf/m
W4 (Weight of pier: Land side)	$7.50 \times 2.00 \times 2.50$ tf/m³	$= 37.50$ tf/m
W5 (Weight of earth)	3.70×1.00 tf/m³ + 3.80×1.80 tf/m³	
		$= 10.54$ tf/m²
W6 (Surcharge load)		$= 1.00$ tf/m²
W7 (Earth load)		$= 0.297$ tf/m²
		$= 3.926$ tf/m²
W8 (Weight of gate: Flood discharge gate)	$50.0\,tf / 18.50\,m$	$= 2.70$ tf/m
W9 (Weight of gate: Sediment flush gate)	$10.0\,tf / 5.50\,m$	$= 1.82$ tf/m
W10 (Uplift)		$= 6.14$ tf/m²

<Seismic case>

Loading calculation

W1 (Weight of slab)	2.20×2.50 tf/m³	$= 5.50$ tf/m²
W2 (Weight of water)	3.70×1.00 tf/m³	$= 3.70$ tf/m²

W3 (Weight of pier: Riverside) $7.50 \times 2.50 \times 2.50$ tf/m^3 = 46.89 tf/m

W4 (Weight of pier: Land side) $7.50 \times 2.00 \times 2.50$ tf/m^3 = 37.50 tf/m

W5 (Weight of earth) 3.70×1.00 tf/m^3 + 3.80×1.80 tf/m^3

= 10.54 tf/m^2

W6 (Surcharge load) = 0.50 tf/m^2

W7 (Earth load) = 0.192 tf/m^2

= 6.258 tf/m^2

W8 (Weight of gate: Flood discharge gate) $50.0\,tf \big/ 18.50\,m$ = 2.70 tf/m

W9 (Weight of gate: Sediment flush gate) $10.0\,tf \big/ 5.50\,m$ = 1.82 tf/m

W10 (Uplift) = 6.14 tf/m^2

Seismic load

(Gate floor slab) $2.20 \times 15.00 \times 2.50$ tf/m3 $\times 0.12$ = 9.90 tf/m

(Gate pier at riverside) $2.20 \times 7.50 \times 2.50$ tf/m3 $\times 0.12$ = 4.50 tf/m

Mw-1 = 4.50 tf/m $\times (7.50 \times 1/2 + 1.10)$ = 21.825 tf-m

(Gate pier at land side) $2.50 \times 7.50 \times 2.50$ tf/m3 $\times 0.12$ = 5.625 tf/m

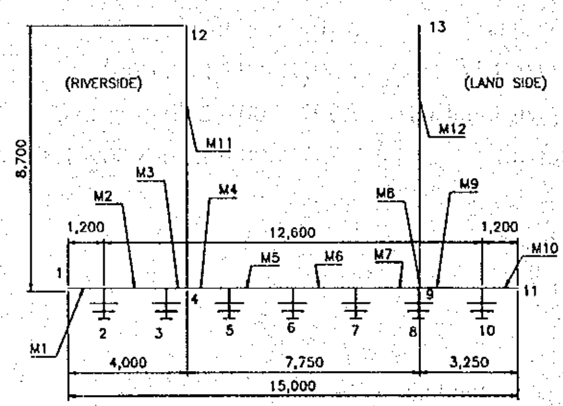
Mw-2 = 5.625 tf/m $\times (7/50 \times 1/2 + 1.10)$ = 27.281 tf-m

(Gate: Discharge gate) 50.0 tf $\times$ 2.75 / 18.50 $\times$ 0.12 = 0.892 tf/m

(Gate: Sediment gate) 10.0 tf $\times$ 0.12 = 1.20 tf/m

$\Sigma H = 9.90 + 4.50 + 5.625 + 0.892 + 1.20$ = 22.117 tf/m

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



Geometrical moment of inertia

Member	Calculation	Geometrical moment of inertia (m^4)
1~10	$\frac{1}{12} \times 1.00 \times 2.20^3$	0.88733
11	$\frac{1}{12} \times 1.00 \times 2.50^3$	1.30208
12	$\frac{1}{12} \times 1.00 \times 2.00^3$	0.66666

Section area

Member	Calculation	Area (m^2)
1~10	2.20×1.00	2.200
11	2.50×1.00	2.500
12	2.00×1.00	2.000

Axial spring constant

$$Kv = a\frac{Ap \times Ep}{L}$$

$a = 0.013 \times (L/D) + 0.61$ (for prestressed concrete pile)

a = 0.013×(9.80 / 0.60) + 0.61 = 0.822333

Ap = 0.1678 m^2

Ep = 4.0×10^6 tf/m^2

Kv = 56321.42 tf/m

Kh = 1/3×Kv = 18773.81 tf/m

Summary of calculation results is shown as follows.

(Normal case)

Member	Condition	Distance (m)	Bending moment M (tf –m)	Shearing stress S (tf)	Axial Force N (tf)
M1	Maximum	0.000	0.000	0.000	0.000
	Minimum	1.200	-4.147	-6.912	0.000
M2	Maximum	2.100	14.229	2.702	-4.049
	Minimum	0.000	-4.147	14.798	-4.049
M3	Maximum	0.700	32.043	23.433	-8.124
	Minimum	0.000	14.229	27.465	-8.124
M4	Maximum	0.000	32.043	-23.457	-8.124
	Minimum	1.300	-2.576	-29.801	-8.124
M5	Maximum	0.000	-2.576	-3.686	-12.252
	Minimum	2.100	-21.076	-13.934	-12.252
M6	Maximum	2.100	-4.001	3.007	-16.461
	Minimum	0.000	-21.076	13.255	-16.461
M7	Maximum	2.100	53.826	22.413	-20.780
	Minimum	0.000	-4.001	32.661	-20.780
M8	Maximum	0.150	61.879	53.322	-25.238
	Minimum	0.000	53.826	54.054	-25.238
M9	Maximum	1.084	-3.696	0.000	4.441
	Minimum	0.000	-12.269	15.822	4.441
M10	Maximum	1.200	0.000	0.000	0.000
	Minimum	0.000	-10.512	17.520	0.000
M11	Maximum	0.000	0.000	0.000	-46.890
	Minimum	8.600	0.000	0.000	-46.890
M12	Maximum	0.000	74.149	-29.679	-37.500
	Minimum	8.600	0.000	0.000	-37.500

(Seismic case)

Member	Condition	Distance (m)	Bending moment M (tf−m)	Shearing stress S (tf)	Axial Force N (tf)
M1	Maximum	0.000	0.000	0.000	0.000
	Minimum	1.200	-4.147	-6.912	0.000
M2	Maximum	2.100	20.788	5.826	-8.352
	Minimum	0.000	-4.147	17.922	-8.352
M3	Maximum	0.700	42.342	28.776	-16.761
	Minimum	0.000	20.788	32.808	-16.761
M4	Maximum	0.000	15.061	-18.114	-16.761
	Minimum	1.300	-12.611	-24.458	-16.761
M5	Maximum	0.587	-11.769	0.000	-25.276
	Minimum	2.100	-17.351	-7.382	-25.276
M6	Maximum	2.100	15.458	10.499	-33.959
	Minimum	0.000	-17.351	20.747	-33.959
M7	Maximum	2.100	89.543	30.155	-42.870
	Minimum	0.000	15.458	40.403	-42.870
M8	Maximum	0.150	98.386	58.583	-52.067
	Minimum	0.000	89.543	59.315	-52.067
M9	Maximum	1.495	-7.982	0.000	-12.812
	Minimum	0.000	-23.745	21.083	-12.813
M10	Maximum	1.200	0.000	0.000	-22.117
	Minimum	0.000	-10.152	16.920	-22.117
M11	Maximum	0.000	0.000	0.000	-46.890
	Minimum	8.600	0.000	0.000	-46.890
M12	Maximum	0.000	100.306	-39.255	-37.500
	Minimum	8.600	0.000	0.000	-37.500

Results of strength calculation are shown as follows.

Table of strength calculation result at standard section

		Gate floor slab				Gate pier at land side	
		Top side	Bottom side	Top side	Bottom side	Normal case	Seismic case
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-21.08	61.88	23.75	98.39	74.15	100.31
N	tf	0	0	0	0	0	0
S	tf	13.26	53.32	21.08	58.58	29.68	39.26
B	cm	100	100	100	100	100	100
D	cm	211	203.5	211	203.5	191	191
Ac	cm2	21100	20350	21100	20350	19100	19100
As	cm2	D16-125 =20.32	D25-125 =39.28	D16-125 =20.32	D25-125 =39.28	D22-125 =30.40	D22-125 =30.40
P=As/(B×D)		0.00096	0.00193	0.00096	0.00193	0.00159	0.00159
N=Es/Ec		15	15	15	15	15	15
X0	cm	32.9	43.4	32.9	43.4	37.4	37.4
K=X0/D		0.156	0.213	0.156	0.213	0.196	0.196
M/(B×D2)	kgf/cm^2	0.473	1.494	0.533	2.376	2.033	2.75
S/(B×D)	kgf/cm^2	0.628	2.62	0.999	2.879	1.554	2.055
(C)		13.512	10.089	13.512	10.089	10.92	10.92
(S)		73.027	37.184	73.027	37.184	44.813	44.813
(Z)		1.055	1.077	1.055	1.077	1.07	1.07
σ c	kgf/cm^2	6.4	15.1	7.2	24	22.2	30
σ s	kgf/cm^2	519	833	584	1325	1366	1848
τ	kgf/cm^2	0.63	2.62	1	2.88	1.55	2.06
σ ca	kgf/cm^2	75	75	112.5	112.5	75	112.5
σ sa	kgf/cm^2	1600	1600	2400	2400	1600	2400
τ a	kgf/cm^2	3.8	3.8	5.7	5.7	3.8	5.7
Note		Normal case		Seismic case			

<u>Maintenance bridge section</u>

<Normal case>

Loading calculation

W1 (Weight of slab)	1.60×2.50 tf/m^3	= 4.00 tf/m^2
W2 (Weight of water)	0.60×1.00 tf/m^3	= 0.60 tf/m^2
W3 (Weight of pier: Riverside)	8.10×2.50×2.50 tf/m^3	= 50.625 tf/m
W4 (Weight of pier: Land side)	9.72×2.00×2.50 tf/m^3	= 48.600 tf/m
W5 (Weight of earth)	3.70×1.00 tf/m^3 + 5.42×1.80 tf/m^3	
		= 13.456 tf/m^2
W6 (Surcharge load)		= 1.00 tf/m^2
W7 (Earth load)		= 0.297 tf/m^2
		= 3.926 tf/m^2
W8 (Weight of Maintenance bridge: L = 21.00m)		= 27.77 tf/m
W9 (Weight of Maintenance bridge: L = 5.50m)		= 9.600 tf/m
W10 (Uplift)		= 3.47 tf/m^2

<Seismic case>

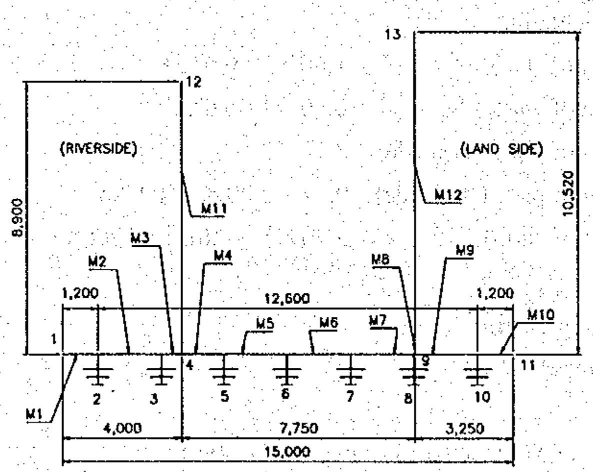
Loading calculation

W1 (Weight of slab)	1.60×2.50 tf/m^3	= 4.00 tf/m^2
W2 (Weight of water)	0.60×1.00 tf/m^3	= 0.60 tf/m^2
W3 (Weight of pier: Riverside)	$8.10 \times 2.50 \times 2.50$ tf/m^3	= 50.625 tf/m
W4 (Weight of pier: Land side)	$9.72 \times 2.00 \times 2.50$ tf/m^3	= 48.600 tf/m
W5 (Weight of earth)	3.70×1.00 tf/m^3 + 5.42×1.80 tf/m^3	
		= 13.456 tf/m^2
W6 (Surcharge load)		= 0.50 tf/m^2
W7 (Earth load)		= 0.192 tf/m^2
		= 7.890 tf/m^2
W8 (Weight of Maintenance bridge: L = 21.00m)		= 27.77 tf/m
W9 (Weight of Maintenance bridge: L = 5.50m)		= 9.600 tf/m
W10 (Uplift)		= 3.47 tf/m^2

Seismic load

(Gate floor slab)	$1.60 \times 15.00 \times 2.50$ tf/m^3 $\times 0.12$	= 7.20 tf/m
(Gate pier at riverside)	$2.00 \times 9.72 \times 2.50$ tf/m^3 $\times 0.12$	= 5.832 tf/m
	Mw-1 = 5.832 tf/m$\times$(9.72$\times$1/2 + 0.80)	= 33.009 tf-m
(Gate pier at land side)	$2.50 \times 8.10 \times 2.50$ tf/m^3 $\times 0.12$	= 6.075 tf/m
	Mw-2 = 6.075 tf/m$\times$(8.10$\times$1/2 + 0.80)	= 29.464 tf-m
(Maintenance bridge: L = 5.50m)		
	9.600 tf/m$\times 0.87 \times 1/2 \times 0.12$	= 0.500 tf/m
(Maintenance bridge: L = 21.00m)		
	27.77 tf/m$\times 1.82 \times 1/2 \times 0.12$	= 3.032 tf/m
ΣH = 7.20 + 5.83 + 6.08 + 9.60$\times$0.12 + 27.77$\times$0.12		= 23.591 tf/m

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



Geometrical moment of inertia

Member	Calculation	Geometrical moment of inertia (m^4)
1~10	$\frac{1}{12} \times 1.00 \times 1.60^3$	0.34133
11	$\frac{1}{12} \times 1.00 \times 2.50^3$	1.30208
12	$\frac{1}{12} \times 1.00 \times 2.00^3$	0.66666

Section area

Member	Calculation	Area (m^2)
1~10	1.60×1.00	1.600
11	2.50×1.00	2.500
12	2.00×1.00	2.000

Axial spring constant

$$Kv = a\frac{Ap \times Ep}{L}$$

$a = 0.013 \times (L/D) + 0.61$ (for prestressed concrete pile)

a = 0.013×(9.80 / 0.60) + 0.61 = 0.822333

Ap = 0.1678 m^2

Ep = 4.0×10^6 tf/m^2

Kv = 56321.42 tf/m

Kh = 1/3×Kv = 18773.81 tf/m

Summary of calculation results is shown as follows.

(Normal case)

Member	Condition	Distance (m)	Bending moment M (tf-m)	Shearing stress S (tf)	Axial Force N (tf)
M1	Maximum	0.000	0.000	0.000	0.000
	Minimum	1.200	-0.814	-1.356	0.000
M2	Maximum	2.100	36.669	16.662	-5.938
	Minimum	0.000	-0.814	19.035	-5.938
M3	Maximum	0.700	67.304	43.368	-11.915
	Minimum	0.000	36.669	44.159	-11.915
M4	Maximum	0.000	67.304	-44.627	-11.915
	Minimum	1.300	8.333	-46.096	-11.915
M5	Maximum	0.000	8.333	-15.403	-17.968
	Minimum	2.100	-26.504	-17.776	-17.968
M6	Maximum	2.100	1.723	12.255	-24.142
	Minimum	0.000	-26.504	14.628	-24.142
M7	Maximum	2.100	100.015	45.619	-30.476
	Minimum	0.000	1.723	47.992	-30.476
M8	Maximum	0.150	112.490	83.081	-37.014
	Minimum	0.000	100.015	83.250	-37.014
M9	Maximum	1.332	-8.631	0.000	6.513
	Minimum	0.000	-25.196	24.881	6.513
M10	Maximum	1.200	0.000	0.000	0.000
	Minimum	0.000	-13.454	22.423	0.000
M11	Maximum	0.000	0.000	0.000	-87.995
	Minimum	8.900	0.000	0.000	-87.995
M12	Maximum	0.000	137.685	-43.527	-58.200
	Minimum	10.520	0.000	0.000	-58.200

(Seismic case)

Member	Condition	Distance (m)	Bending moment M (tf-m)	Shearing stress S (tf)	Axial Force N (tf)
M1	Maximum	0.000	0.000	0.000	0.000
	Minimum	1.200	-0.814	-1.356	0.000
M2	Maximum	2.100	44.600	20.439	-11.197
	Minimum	0.000	-0.814	22.812	-11.197
M3	Maximum	0.700	80.113	50.339	-22.468
	Minimum	0.000	44.600	51.130	-22.468
M4	Maximum	0.000	47.117	-37.656	-22.468
	Minimum	1.300	-2.791	-39.125	-22.468
M5	Maximum	0.000	-2.791	-5.989	-33.882
	Minimum	2.100	-17.860	-8.362	-33.882
M6	Maximum	2.100	35.325	24.139	-45.523
	Minimum	0.000	-17.860	26.512	-45.523
M7	Maximum	2.100	161.744	59.013	-57.467
	Minimum	0.000	35.325	61.386	-57.467
M8	Maximum	0.150	175.776	93.461	-69.796
	Minimum	0.000	161.744	93.630	-69.796
M9	Maximum	1.887	-13.206	0.000	-11.157
	Minimum	0.000	-46.474	35.261	-11.157
M10	Maximum	1.200	0.000	0.000	-23.591
	Minimum	0.000	-13.454	22.423	-23.591
M11	Maximum	8.900	3.532	0.000	-87.995
	Minimum	0.000	3.532	0.000	-87.995
M12	Maximum	0.000	189.241	-58.639	-58.200
	Minimum	0.500	0.000	0.000	-58.200

Results of strength calculation are shown as follows.

Table of strength calculation result at standard section

		Gate floor slab				Gate pier at land side	
		Top side	Bottom side	Top side	Bottom side	Normal case	Seismic case
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-26.5	112.49	-46.47	175.68	137.69	189.24
N	tf	0	0	0	0	58.2	58.2
S	tf	14.63	83.08	35.26	58.58	43.53	58.64
B	cm	100	100	100	100	100	100
D	cm	151	203.5	151	203.5	191	191
Ac	cm2	15100	20350	15100	20350	19100	19100
As	cm2	D16-125 =20.32	D25-125 =39.28	D16-125 =20.32	D25-125 =39.28	D25-125 =39.28	D25-125 =39.28
P=As/(B×D)		0.00135	0.00193	0.00135	0.00193	0.00206	0.00206
N=Es/Ec		15	15	15	15	15	15
X0	cm	27.4	43.4	27.4	43.4	57.2	52.7
K=X0/D		0.182	0.213	0.182	0.213	0.299	0.276
M/(B×D2)	kgf/cm²	1.162	2.716	2.038	4.242	3.774	5.187
S/(B×D)	kgf/cm²	0.969	4.083	2.335	2.879	2.279	3.07
(C)		11.714	10.089	11.714	10.089	10.416	10.333
(S)		52.736	37.184	52.736	37.184	24.376	27.141
(Z)		1.064	1.077	1.064	1.077	1.167	1.127
σ c	kgf/cm²	13.6	27.4	23.9	42.8	39.3	53.6
σ s	kgf/cm²	919	1515	1612	2366	1380	2112
τ	kgf/cm²	0.97	3.08	2.34	2.88	2.28	3.07
σ ca	kgf/cm²	75	75	112.5	112.5	75	112.5
σ sa	kgf/cm²	1600	1600	2400	2400	1600	2400
τ a	kgf/cm²	3.8	3.8	5.7	5.7	3.8	5.7
Note		Normal case		Seismic case			

(3) Pile head treatment

a) Vertical bearing pressure for footing concrete

Pnmax = 104.677 tf/pile (Direction of weir axis in construction case)

$$\sigma_{cv} = \frac{P_{N\max}}{\pi D^2 / 4} = \frac{104677}{\pi/4 \times 60^2} = 37.02 \text{ kgf/cm}^2$$

$\leqq \sigma$ ca = 60.0 kgf/cm² ....O.K

b) Punching shear stress for footing concrete

$$\tau_V = \frac{P_{N\max}}{\pi(D+h)h} = \frac{104677}{\pi(60+125)\times 125} = 1.44 \text{ kgf/cm}^2$$

$\leqq \tau$ 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 = 64.41 tf/pile (Direction of flowing water in seismic case)

M = 18.60 tf-m

S = 25.42 tf

$$\sigma_{ch} = \frac{25420}{60 \times 10} = 42.37 \text{ kgf/cm}^2 \leqq \sigma\text{ ca} = 60.0 \times 1.5 = 90.0 \text{ kgf/cm}^2$$

................O.K

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

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

where

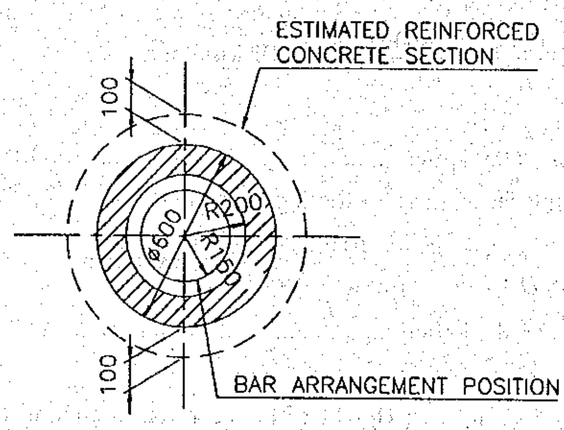
h'	: Effective thickness to vertical punching stress on footing (cm)
l	: Stuffing length of pile (cm)
D	: Pile diameter (cm)
H	: Shearing pressure (kgf)

$h' = 55$ cm (Direction of weir axis)

$$\tau_h = \frac{25420}{55 \times (2 \times 10 + 60 + 2 \times 55)} = 2.43 \text{ kgf/cm}^2 \leqq \tau\text{ ca3} = 8.8 \text{ kgf/cm}^2$$

....................O.K

e) Strength of estimated reinforced concrete section



$P_N$min = 53.80 tf/pile (Direction of weir axis in seismic case)

M = 18.60 tf-m
S = 25.42 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	-18.60	X0	cm	23.20
N	tf	0.00	K= X0/H		0.29
S	tf	25.42	M/(B×H²)	kgf/cm²	3.633
B	cm	80.00	S/(B×H)	kgf/cm²	3.972
H	cm	80.00	(C)		27.9
D	cm	55.00	(S)		38.261
DD	cm	25.00	(Z)		3.804
DG	cm	25.00	σ c	kgf/cm²	101.4
B0 , R	cm	40.00	σ s	kgf/cm²	2085.00
H0 , R0	cm	20.00	τ	kgf/cm²	0.00
AC	cm²	3769.90	σ ca	kgf/cm²	112.50
AS , AS1	cm²	12-D16 = 45.60	σ sa	kgf/cm²	2400.00
P , P1		0.0121	τ a	kgf/cm²	5.70
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 = 50×9.0 +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

h) Depth of concrete filling for concrete type-D

Position of bottom of concrete filing is adopted with bending moment value in case of fix type and hinge type of pile head treatment.

$$L_4 = La + Lf$$

Where;

L_4: Length from pile head to bottom of concrete filling (m)

La: Length for position where maximum bending moment value is half (1/2) (m)

Lf: Installed pile length in footing (m)

Calculation results of bending moment are shown in both normal case and seismic case as follows.

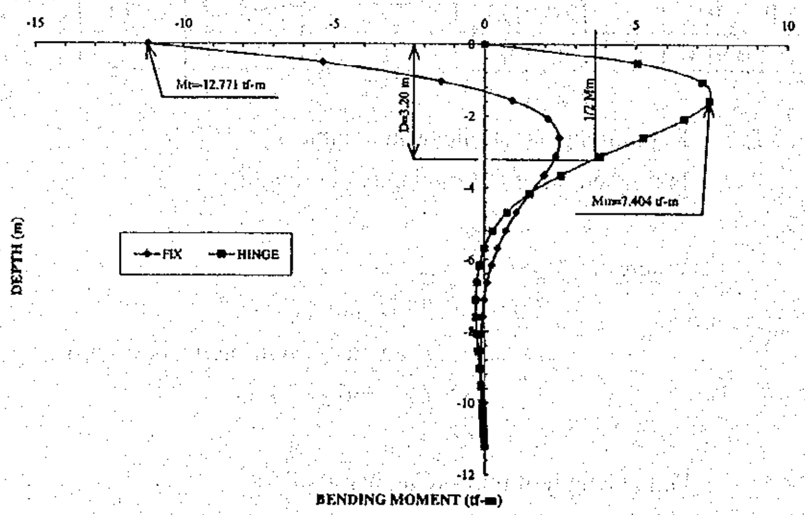
[Normal case]

Mm = 7.404 tf-m

1/2 Mm = 3.702 tf-m

D = 3.20 m ≒ 3.50 m

NORMAL CASE



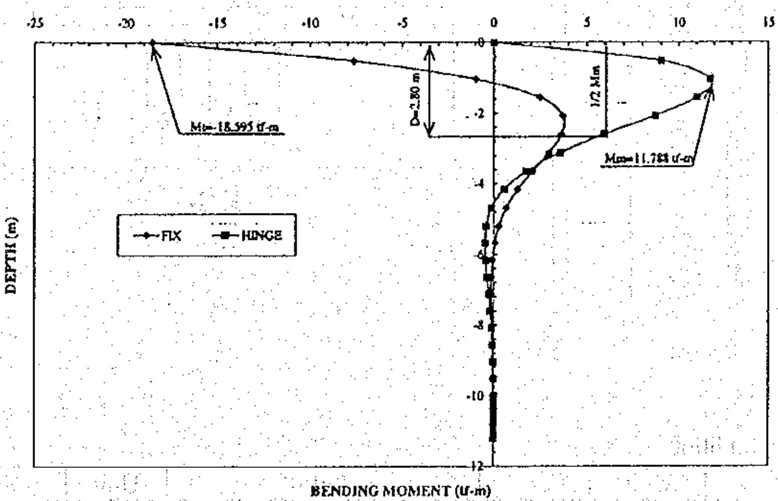
[Seismic case]

Mm = 11.788 tf-m

1/2 Mm = 5.894 tf-m

D = 2.80 m ≒ 3.00 m

SEISMIC CASE

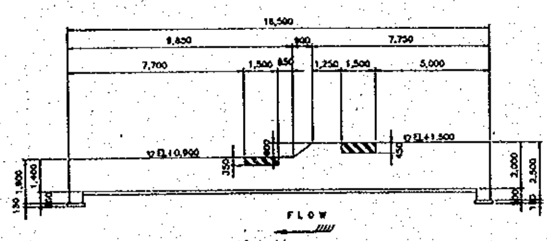


Therefore, depth from pile head to bottom of concrete filling adopts 3.50 m.

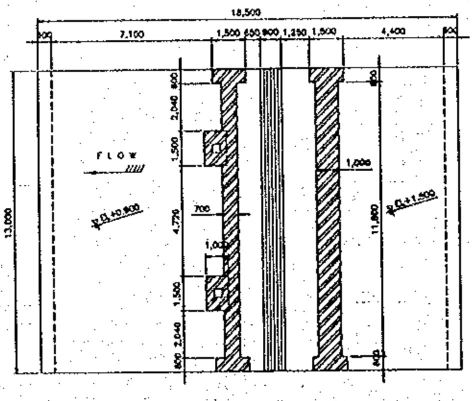
4.2.3 Gate Floor Slab, Stilling Basin and Apron

4.2.3.1 Gate Floor Slab

1) Loading calculation



PROFILE



PLAN

Design Condition

Item	Unit		Item	Unit	
Thickness at upstream of gate	m	2.000	Unit weight of concrete	tf / m^3	2.500
Thickness at downstream of gate	m	1.400	Unit weight of water	tf / m^3	1.000
Length of slab at flowing water	m	18.500	Unit weight of soil	tf / m^3	1.800
Width of slab at weir axis	m	13.000	Submerged unit weight	tf / m^3	1.000
Length of slab at upstream of gate	m	7.750	Angle of shearing resistance		30.000
Length of slope section	m	0.900	Horizontal seismic intensity		0.120
Length of slab at downstream of gate	m	9.850	Apparently horizontal seismic intensity		0.240
Width of muddy soil	m	1.000	Coefficient of earth pressure in Normal case		0.3085
Weight of flood discharge gate	tf/m	9.700	Coefficient of earth pressure in Seismic case above water		0.4107
Distance of installed gate position	m	5.000	Coefficient of earth pressure in seismic case below water		0.5089
Surcharge load in construction case	tf / m^2	1.000	Top elevation of slab	EL m	1.500
			Bottom elevation of slab	EL m	-0.500
			Normal water level	EL m	5.850
			Water level in seismic case	EL m	5.200
			Design high water level	EL m	8.000

1.Weight of body

	Weight W(t)	Vertical length X(m)	Moment MX(t-m)	Horizontal length Y(m)	Moment MY(t-m)	Note
1	841.750	9.250	7786.188	0.700	589.225	Conc.
2	151.125	3.875	585.609	1.700	256.913	Conc.
3	8.775	8.050	70.639	1.600	14.040	Conc.
Total	1001.650	8.429	8442.436	0.859	860.178	

2.Weight of muddy soil

Normal case Seismic case	Weight W(tf)	Vertical length X(m)	Moment MX(t-m)	Horizontal length Y(m)	Moment MY(t-m)
	65.000	2.500	162.500	2.500	162.500

3. Surcharge load

Construction	Weight W(tf)	Vertical length X(m)
	240.500	9.250

4. Weight of water

Normal case Seismic case	Weight W(tf)	Vertical length X(m)	Moment MX(t-m)	Horizontal length Y(m)	Moment MY(t-m)	Note
Upstream	240.500	2.500	601.250	3.850	925.925	
Downstream	76.830	13.575	1042.967	1.700	130.611	
Total	317.330	5.181	1644.217	3.329	1056.536	

Design flooding case	Weight W(tf)	Vertical length X(m)	Moment MX(t-m)	Horizontal length Y(m)	Moment MY(t-m)	Note
	1563.250	9.250	14460.063	5.250	8207.063	
Total	1563.250	9.250	14460.06	5.250	8207.063	

5. Earth pressure

Normal case	(tf/m^2)		(tf/m)		(m)
Pa0=	0.000	E1=	8.020	Y1=	0.667
Pa1=	0.617				
Seismic case	(tf/m^2)		(tf/m)		(m)
Pea0=	0.000	Ee1=	13.231	Ye1=	0.667
Pea1=	1.018				

6. Hydrostatic pressure

Normal, Seismic			(tf/m)		(m)
Wa0=	3.700				
Wa1=	6.350	P1=	130.650	Y3=	0.912
Design flooding case			(tf/m)		(m)
Wae0=	6.500				
Wae1=	8.500	Pe1=	195.000	Ye3=	0.956

Normal Case

	Vertical			Horizontal		
	V (tf)	X (m)	Mx (tf-m)	H (tf)	Y (m)	My (tf-m)
Weight of body	1001.650	8.429	8442.908			
Weight of gate	126.100	6.000	756.600			
Horizontal earthquake load (Main body)						
Weight of muddy soil	65.000	2.500	162.500			
Horizontal earthquake load (Muddy soil)						
Surcharge load						
Weight of water	317.330	5.181	1644.217			
Earth pressure				8.020	0.667	5.347
Hydrostatic pressure				130.650	0.912	119.167
Uplift	-770.705	8.546	-6586.438			
Total	739.375	5.978	4419.787	138.670	0.898	124.513

Acting force at toe of Gate floor slab

V0=	739.375 tf
H0=	138.670 tf
M0=Mx+My=	4544.300 tf-m

Acting force at middle of bottom slab of Gate floor slab

Vc=	739.375 tf
Hc=	138.670 tf
Mc=Mx+My=	2294.917 tf-m

Construction Case

	Vertical			Horizontal		
	V(t)	X(m)	Mx(t-m)	H(t)	Y(m)	My(t-m)
Weight of body	1001.650	8.429	8442.908			
Horizontal earthquake load (Main body)						
Weight of muddy soil						
Horizontal earthquake load (Muddy soil)						
Surcharge load	240.500	9.250	2224.625			
Weight of water						
Earth pressure						
Hydrostatic pressure						
Uplift						
Total	1242.150	8.588	10667.533	0.000		0.000

Acting force at toe of Gate floor slab

V0=	1242.150 tf
H0=	0.000 tf
M0=Mx+My=	10667.533 tf-m

Acting force at middle of bottom slab of Gate floor slab

Vcc=	1242.150 tf
Hcc=	0.000 tf
Mcc=Mx+My=	822.355 tf-m

Design Flooding Case

	Vertical			Horizontal		
	V (tf)	X (m)	Mx (tf-m)	H (tf)	Y (m)	My (tf-m)
Weight of body	1001.650	8.429	8442.908			
Horizontal earthquake load (Main body)						
Weight of muddy soil						
Horizontal earthquake load (muddy soil)						
Surcharge load						
Weight of water	1563.250	9.250	14460.063			
Earth pressure				8.020	0.667	5.347
Hydrostatic pressure						
Uplift	-2044.250	9.250	-18909.313			
Total	520.650		3993.658	8.020		5.347

Acting force at toe of Gate floor slab

V0=	520.650 tf
H0=	8.020 tf
M0=Mx+My=	3999.005 tf-m

Acting force at middle of bottom slab of Gate floor slab

Vcf=	520.650 tf
Hcf=	8.020 tf
Mcf=Mx+My=	817.008 tf-m

Seismic Case

	Vertical			Horizontal		
	V (tf)	X (m)	Mx (tf-m)	H (tf)	Y (m)	My (tf-m)
Weight of body	1001.650	8.429	8442.908			
Horizontal earthquake load (Main body)				120.198	0.859	103.250
Weight of gate	126.100	6.000	756.600			
Weight of muddy soil	65.000	2.500	162.500			
Horizontal earthquake load (muddy soil)				7.800	2.500	19.500
Surcharge load						
Weight of water	317.330	5.181	1644.217			
Earth pressure				13.231	0.667	8.821
Hydrostatic pressure				130.650	0.912	119.167
Uplift	-770.705	8.546	-6586.438			
Total	739.375		4419.787	271.879		250.738

Acting force at toe of Gate floor slab

V0=	739.375 tf
H0=	271.879 tf
M0=Mx+My=	4670.524 tf-m

Acting force at middle of bottom slab of Gate floor slab

Vcs=	739.375 tf
Hcs=	271.879 tf
Mcs=Mx+My=	2168.693 tf-m

2) Stability Analysis

Type of Pile

The prestressed concrete pile is adopted for the foundation pile of gate floor slab.

Refer to stability calculation of center pier.

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 that is available in this country, is 600 mm.

Alternative-1	PC Pile Dia.450 mm, type A	56 piles
Alternative-2	PC Pile Dia.500 mm, type A	40 piles
Alternative-3	PC Pile Dia.600 mm, type A	30 piles

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)

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

$$qd = 500\ \mathrm{tf/m^2} \times A = 79.50\ \mathrm{tf}$$

$$U = \pi D = \pi \times 0.45 = 1.414\ \mathrm{m}$$

$$Ru = 79.50 + 1.414 \times 72.62 = 182.18\ \mathrm{tf/pile}$$

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 / m²×A = 98.00 tf

U = π D = π ×0.50 = 1.571 m

Ru = 98.00 + 1.571×72.62 = 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 / m²×A = 141.50 tf

U = π D = π ×0.60 = 1.885 m

Ru = 141.50 + 1.885×72.62 = 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	139.20

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=30 piles) was selected for the economical reason. The calculation results are shown as follows.

COMPARATIVE STUDY ON PILE FOUNDATION FOR GATE FLOOR SLAB

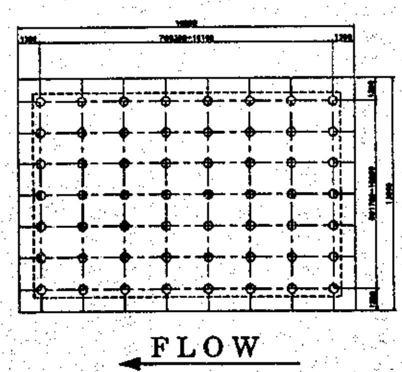
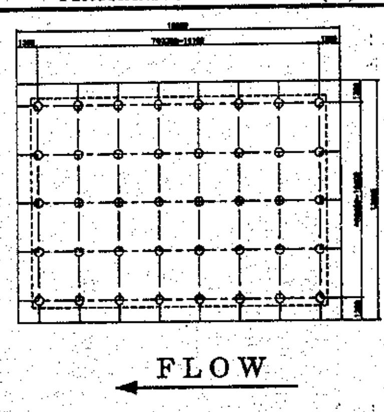
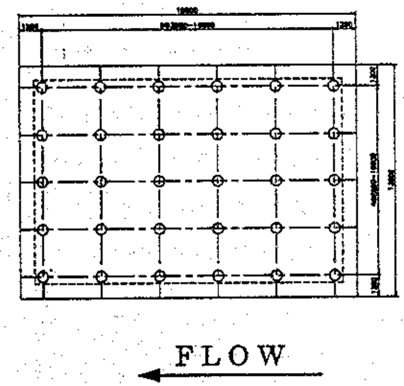
kind of pile		Alternative-1 Dia. 450 (A)			Alternative-2 Dia. 500 (A)			Alternative-3 Dia. 600 (A)		
Pile Arrangement										
Number of necessary pile		56 piles (L=10.20m)			40 piles (L=10.20m)			30 piles (L=10.20m)		
Displacement (Horizontal)		Calculation (mm)	Allowable capacity (mm)		Calculation (mm)	Allowable capacity (mm)		Calculation (mm)	Allowable capacity (mm)	
Normal Case		0.4	10		0.5	10		0.7	10	
Design flooding Case		0.7	10		0.8	10		0.0	10	
Constructional Case		0.0	10		0.0	10		0.0	10	
Seisimic Case		0.5	15		0.6	15		0.7	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		0.372	2.438	3.05	0.626	3.775	4.23	1.820	4.911	7.15
Design flooding Case		6.941	3.133	3.55	9.852	4.849	5.01	10.023	0.138	8.00
Constructional Case		14.938	0.066	4.16	20.916	0.104	5.94	34.105	0.157	10.51
Seisimic Case		1.405	3.746	5.39	2.128	5.800	7.49	2.122	7.925	12.44
Bearing capacity (tf)	Normal	60.73			70.70			92.80		
	Seisimic	91.09			106.04			139.20		
Summary of cost		Rp 91.9 million			Rp 78.7 million			Rp 73.5 million		
Evaluation		Not adopted			Not adopted			Adopted		

TABLE OF STABILITY CALCULATION FOR GATE FLOOR SLAB

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)	0.0006998	0.0000534	0.0000185	0.0007354	-	-	-	-
	Allowable	0.010	0.010	0.010	0.015	0.010	0.010	0.010	0.015
	Vertical (δ Y m)	0.004765	0.0003174	0.0007572	0.0004765	-	-	-	-
	Allowable	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015
Axial force (tf/pile)	No.1	50.3058	24.6868	48.7053	49.9859	-	-	-	-
	No.2	40.6051	21.7541	45.7852	40.4132	-	-	-	-
	No.3	30.9054	18.8214	42.8651	30.8405	-	-	-	-
	No.4	21.2039	15.8887	39.9449	21.2678	-	-	-	-
	No.5	11.5032	12.9559	37.0248	11.6951	-	-	-	-
	No.6	1.8026	10.0232	34.1047	2.1224	-	-	-	-
	No.7	-	-	-	-	-	-	-	-
	No.8	-	-	-	-	-	-	-	-
	No.9	-	-	-	-	-	-	-	-
Allowable bearing capacity (tf/pile)		92.80	92.80	92.80	139.20	-	-	-	-
Shearing stress (tf)		4.9040	0.2673	0.0000	9.3443	-	-	-	-
Bending moment (tf-m/pile)		4.9113	0.1383	0.1573	7.9246	-	-	-	-
Allowable bending moment (tf-m/pile)		7.15	8.00	10.51	12.44	-	-	-	-

Number of piles: n=30 piles

Pile head condition: Fixing

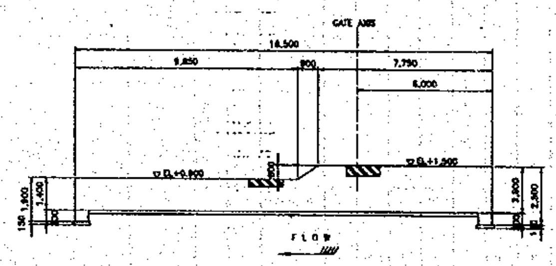
<u>Pile condition</u>

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

(i) Main body

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.



PROFILE

Loading calculation

W1 (Weight of body:1)	1.40×2.50 tf/m^3	= 3.500 tf/m^2
W2 (Weight of body:2)	2.00×2.50 tf/m^3	= 5.000 tf/m^2
W3 (Weight of muddy soil)		
	1.00×1.00 tf/m^3	= 1.000 tf/m2
W4 (Weight of water)	3.70×1.00 tf/m^3	= 3.700 tf/m^2
Wg (Weight of gate)		= 9.700 tf/m
W6-1 (Uplift: L =0.60 m)		= 6.14 tf/m^2
W6-2 (Uplift: L = 17.90 m)	= 3.88 tf/m^2	
		= 3.20 tf/m^2

Horizontal pressure

<Normal case>

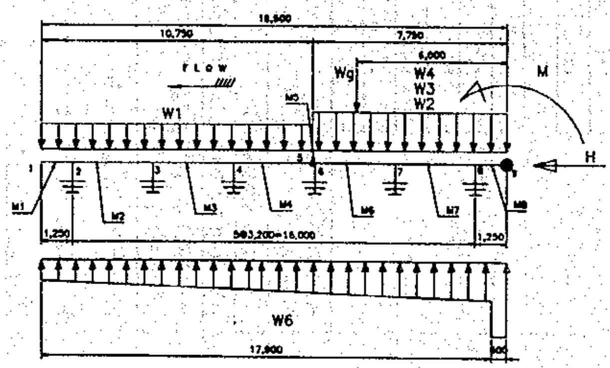
H = 10.667 tf/m (Refer to stability analysis.)

M = 11.317 tf/m $\times$ 0.898 = 10.163 tf-m/m

<Seismic case>

H = 20.914 tf/m (Refer to stability analysis)

M = 20.914 tf/m $\times$ 0.922 = 19.283 tf-m/m



Geometrical moment of inertia

Member	Calculation	Geometrical moment of inertia (m^4)
1~5	$\frac{1}{12}\times1.00\times1.40^3$	0.22867
6~8	$\frac{1}{12}\times1.00\times2.20^3$	0.88733

Section area

Member	Calculation	Area (m^2)
1~5	1.40×1.00	1.400
6~8	2.20×1.00	2.200

Axial spring constant

$$Kv = a\frac{Ap\times Ep}{L}$$

$a = 0.013\times(L/D)+0.61$ (for prestressed concrete pile)

a = 0.013×(10.20 / 0.60) + 0.61 = 0.83100
Ap = 0.1678 m^2
Ep = 4.0×10^6 tf/m^2
Kv = 54683.06 tf/m
Kh = 1/3×Kv = 18227.69 tf/m

Summary of calculation results is shown as follows.

(Normal case)

Member	Condition	Distance (m)	Bending moment M (tf-m)	Shearing stress S (tf)	Axial Force N (tf)
M1	Maximum	0.000	0.000	0.000	0.000
	Minimum	1.250	-0.691	-1.095	0.000
M2	Maximum	0.665	-0.506	0.000	-1.743
	Minimum	3.200	-3.062	-1.975	-1.743
M3	Maximum	0.217	-3.045	0.000	-3.514
	Minimum	3.200	-6.091	-1.987	-3.514
M4	Maximum	3.100	0.304	1.240	-5.339
	Minimum	0.000	-6.091	2.947	-5.339
M5	Maximum	0.100	0.418	1.041	-5.339
	Minimum	0.000	0.304	1.240	-5.339
M6	Maximum	1.727	19.037	0.000	-7.246
	Minimum	0.000	0.418	12.401	-7.246
M7	Maximum	1.315	17.653	0.000	-9.231
	Minimum	3.200	7.186	-11.084	-9.231
M8	Maximum	1.250	10.446	0.000	-11.317
	Minimum	0.000	7.186	5.927	-11.317

(Seismic case)

Member	Condition	Distance (m)	Bending moment M (tf –m)	Shearing stress S (tf)	Axial Force N (tf)
M1	Maximum	0.000	0.000	0.000	0.000
	Minimum	1.250	-0.222	-0.345	0.000
M2	Maximum	0.000	-0.222	-0.572	-3.322
	Minimum	0.000	-3.138	-0.872	-6.696
M3	Maximum	0.000	-3.138	-0.872	-6.696
	Minimum	3.200	-6.393	-1.097	-6.696
M4	Maximum	3.100	1.702	2.718	-10.173
	Minimum	0.000	-6.393	2.565	-10.173
M5	Maximum	0.100	1.967	2.579	-10.173
	Minimum	0.000	1.702	2.718	-10.173
M6	Maximum	1.933	22.858	0.000	-13.807
	Minimum	0.000	1.967	13.643	-13.807
M7	Maximum	1.536	25.040	0.000	-17.590
	Minimum	3.200	16.894	-9.776	-17.590
M8	Maximum	1.250	20.154	0.000	-21.564
	Minimum	0.000	16.894	5.927	-21.564

Results of strength calculation are shown as follows.

Table of strength calculation result at standard section

Normal case					
		Top side of Downstream slab	Bottom side of Downstream slab	Bottom side of Upstream slab	Bottom side of Blockout section of Upstream slab
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-6.10	0.42	19.04	19.04
N	tf	0.00	0.00	0.00	0.00
S	tf	1.99	1.04	0.00	0.00
B	cm	100.00	100.00	100.00	100.00
D	cm	131.00	123.50	203.50	153.50
Ac	cm^2	13100	12350	20350	15350
As	cm^2	D16-250 =8.04	D19-250 =11.36	D19-250 =11.36	D19-250 =11.36
P=As/(B×D)		0.00061	0.00092	0.00056	0.00074
N=Es/Ec		15	15	15	15
X0	cm	16.60	123.50	24.70	21.20
K=X0/D		0.127	1.000	0.121	0.138
M/(B×D2)	kgf/cm^2	0.355	0.028	0.460	0.808
S/(B×D)	kgf/cm^2	0.152	0.084	0.000	0.000
(C)		16.469	5.762	17.182	15.159
(S)		113.417	5.762	124.457	94.436
(Z)		1.044	1.435	1.042	1.048
σ c	kgf/cm^2	5.90	0.20	7.90	12.20
σ s	kgf/cm^2	605.00	2.00	858.00	1145.00
τ	kgf/cm^2	0.15	0.08	0.00	0.00
σ ca	kgf/cm^2	75.00	75.00	75.00	75.00
σ sa	kgf/cm^2	1600.00	1600.00	1600.00	1600.00
τ a	kgf/cm^2	3.80	3.80	3.80	3.80

Seismic case					
		Top side of Downstream slab	Bottom side of Downstream slab	Bottom side of Upstream slab	Bottom side of Blockout section of Upstream slab
Member of shape		Rectangle	Rectangle	Rectangle	Rectangle
M	tf-m	-6.39	1.97	22.86	22.86
N	tf	0.00	0.00	0.00	0.00
S	tf	2.57	2.58	0.00	0.00
B	cm	100.00	100.00	100.00	100.00
D	cm	131.00	123.50	203.50	153.50
Ac	cm^2	13100	12350	20350	15350
As	cm^2	D16-250 =8.04	D19-250 =11.36	D19-250 =11.36	D19-250 =11.36
P=As/(B×D)		0.00061	0.00092	0.00056	0.00074
N=Es/Ec		15	15	15	15
X0	cm	16.60	18.90	24.70	21.20
K=X0/D		0.127	0.153	0.121	0.138
M/(B×D2)	kgf/cm^2	0.372	0.129	0.552	0.970
S/(B×D)	kgf/cm^2	0.196	0.209	0.000	0.000
(C)		16.469	13.784	17.182	15.159
(S)		113.417	76.369	124.457	94.436
(Z)		1.044	1.054	1.042	1.048
σ c	kgf/cm^2	6.10	1.80	9.50	14.70
σ s	kgf/cm^2	633.00	148.00	1031.00	1374.00
τ	kgf/cm^2	0.20	0.21	0.00	0.00
σ ca	kgf/cm^2	112.50	112.50	112.50	112.50
σ sa	kgf/cm^2	2400.00	2400.00	2400.00	2400.00
τ a	kgf/cm^2	5.70	5.70	5.70	5.70

(ii) Pile head treatment

a) Vertical bearing pressure for footing concrete

Pnmax = 50.306 tf/pile (in Normal case)

$$\sigma_{cv} = \frac{P_{N\max}}{\pi D^2/4} = \frac{50306}{\pi/4 \times 60^2} = 17.79 \text{ kgf/cm}^2$$

$\leqq \sigma$ ca = 60.0 kgf/cm^2...O.K

b) Punching shear stress for footing concrete

$$\tau_V = \frac{P_{N\max}}{\pi(D+h)h} = \frac{50306}{\pi(60+60)\times 60} = 2.22 \text{ kgf/cm}^2$$

$\leqq \tau$ ca3 = 8.8 kgf/cm^2...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)

$P_N$max = 49.99 tf/pile (in Seismic case)
M = 7.92 tf-m
S = 9.34 tf

$\sigma_{ch} = \frac{9340}{60 \times 10} = 15.57$ kgf/cm² $\leqq \sigma$ ca = 60.0×1.5 = 90.0 kgf/cm²

................O.K

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

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

where

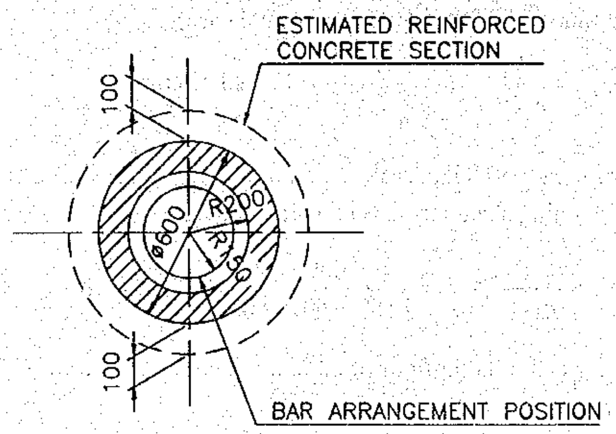
h' : Effective thickness to vertical punching stress on footing (cm)
l : Stuffing length of pile (cm)
D : Pile diameter (cm)
H : Shearing pressure (kgf)

h' = (90-1/2×60) = 60 cm

$$\tau_h = \frac{9340}{60 \times (2 \times 10 + 60 + 2 \times 60)} = 0.78 \text{ kgf/cm}^2$$

$\leqq \tau$ ca3 = 8.8 kgf/cm² O.K

e) Strength of estimated reinforced concrete section



$P_N$min = 2.12 tf/pile (in Seismic case)
M = 7.92 tf-m
S = 9.34 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	-7.92	X0	cm	17.10
N	tf	0.00	K= X0/H		0.21
S	tf	9.34	$M/(B\times H^2)$	kgf/cm^2	1.55
B	cm	80.00	$S/(B\times H)$	kgf/cm^2	1.46
H	cm	80.00	(C)		42.17
D	cm	55.00	(S)		93.26
DD	cm	25.00	(Z)		3.53
DG	cm	25.00	σc	kgf/cm^2	65.20
B0 , R	cm	40.00	σs	kgf/cm^2	2164.00
H0 , R0	cm	20.00	τ	kgf/cm^2	0.00
AC	cm^2	3769.90	σca	kgf/cm^2	112.50
AS , AS1	cm^2	8-D16 =16.08	σsa	kgf/cm^2	2400.00
P , P1		0.00427	τa	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 × 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