4.5.4 Left Bank Intake Gate

1. DESIGN ITEM

1.1 Design Condition

- Type of Gate		Steel Roller Gate	
- Number of Gate		2	Gates
- Clear span		2.000	m
- Height of Gate		2.000	m
- Design Water Depth			
	Upstream	4.000	m
	Downstream	0.000	m
- Water Depth for Operation			
• Lifting Time			
	Upstream	2.200	m
	Downstream	0.000	m
• Lowering time			
	Upstream	1.840	m
	Downstream	0.000	m
- Gate Floor Level		EL 4.000	m
- Method for Watertightness		Rear front 4 faces watertight	
- Hoisting System		One spindle rod	
- Hoisting Speed		About 0.3 m/min	
- Total Head		2.500	m
- Operation Method		Local manual operation and Remote manual operation	
- Power		Electricity	
- Sources of Power		220 V, 50 Hz.	

1.2 Design Condition

- General Item		
• Horizontal Seismic Intensity	0.120	
• Deffelence in Temperature	——	
• Wind Load	150	kgf/m^2
• Deformation of Main Girder	1/ 800	
• Margin of Thickness		

Skin Plate (Side facing water) 1.00 mm

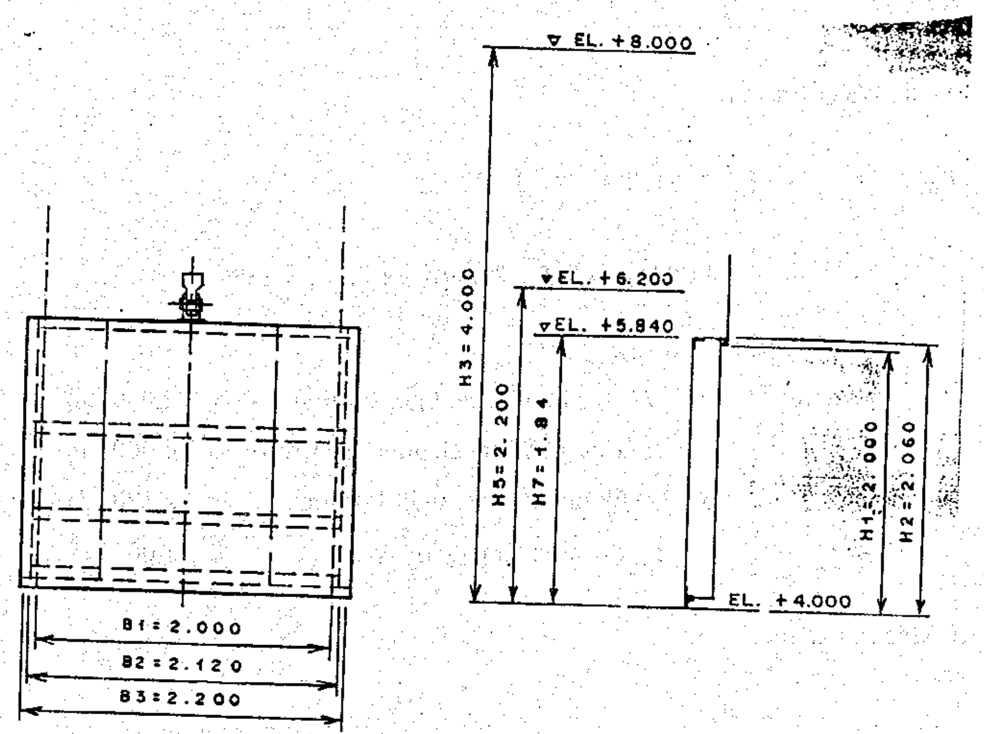
Other Members (Side facing water) 1.00 mm

- Major Material of Gate — Main beam: SS400; Skin Plate: SS400

- Allowable Stress

Steel	Technical Manual for Dam and Weir Chapter 2, 2-0-7		
	Correction Factor	Normal Case	Seismic Case
		1.000	1.500
Concrete	Bearing stress	55.0	kgf/cm^2
	Share stress	4.0	kgf/cm^2

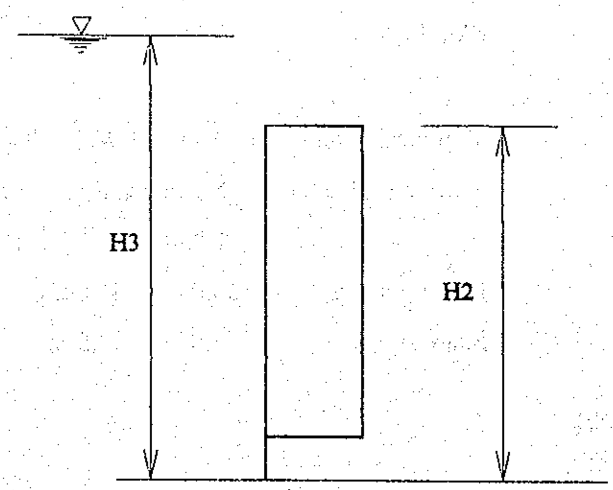
2. BASIC DIMENSIONS



B1	:	Clear Span	2.000	m
B2	:	Length of Watertight Area	2.120	m
B3	:	Span Between Rollers	2.200	m
H1	:	Height of Gate	2.000	m
H2	:	Height of Watertight Area	2.060	m
H3	:	Design Water Depth Upstream	4.000	m
H4	:	Design Water Depth Downstream	0.000	m
H5	:	Water Depth for Operation (Upstream)	2.200	m
H7	:	Water Depth for Operation (Downstream)	1.840	m

3. ACTING LOADS

3.1 Loads in Normal Time



(1) Static Water Pressure

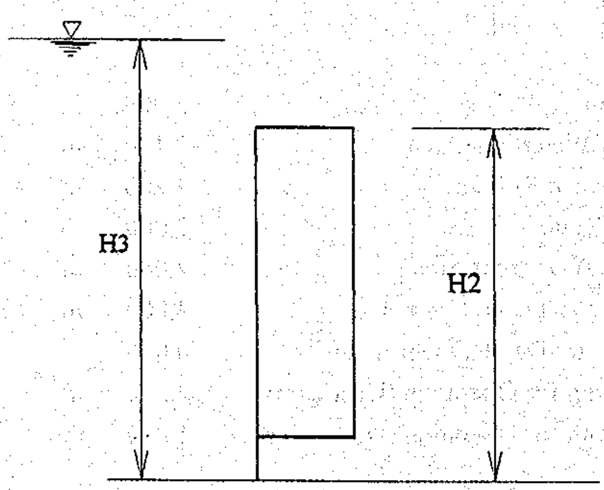
$$P_W = \frac{1}{2} \cdot \{H3^2 - (H3 - H2)^2\} \cdot B2 \cdot \gamma$$

$$= \frac{1}{2} \times \{4.000^2 - (4.000 - 2.060)^2\} \times 2.120 \times 1.00$$

$$= 12.971 \quad \text{tf}$$

Where,	H2	:	Height of Watertight Portion	2.060	m
	H3	:	Design Water Depth (Upstream)	4.000	m
	B2	:	Width of Watertight Portion	2.120	m
	γ	:	Specific Gravity of Water	1.00	

3.2 Loads in Seismic Time



(1) Wave Height in Seismic Time (hwe)

$$hwe = \frac{k \cdot \tau}{2 \cdot \pi} \cdot (g \cdot Hee)^{0.5}$$

Where,

k	:	Design seismic intensity	0.12	
τ	:	Seismic cycle	1.0	sec
g	:	Acceleration of gravity	9.8	m/sec²
Hee	:	Distance between riverbed and water surface	3.700	m

$$= \frac{0.12 \times 1.0}{2 \cdot \pi} \times (9.8 \times 3.700)^{0.5}$$

$$= 0.115 \quad tf$$

(2) Static Water Pressure in Seismic Time (Peg)

$$Peg = \frac{1}{2} \cdot \{(H3 + hwe)^2 - (H3 + hwe - H2)^2 \cdot B2$$

$$= \frac{1}{2} \times (4.000 + 0.115)^2 - (4.000 + 0.115 - 2.060)^2\} \times 2.120$$

$$= 13.473 \quad tf$$

(3) Dynamic Water Pressure in Seismic Time (Pdo)

$$Pdo = \frac{7}{12} \cdot k \cdot Hee^{05} \cdot (H3^{1.5} - h^{1.5}) \cdot B2$$

Where,

k	:	Seismic intensity	0.12	
Hee	:	Distance between riverbed and water surface	3.700	m
H3	:	Design Water Depth	4.000	m
h	:	H3 – H2	1.940	m
B2	:	Width Watertight portion	2.120	m

$$= \frac{7}{12} \times 0.12 \times 3.700^{0.5} \times (4.000^{1.5} - 1.940^{1.5}) \times 2.120$$

$$= 1.512 \quad tf$$

(4) Force of Inertia in Seismic Time (Wge)

$$Wge = k \cdot Wg$$

Where,

Wg	: Self weight of gate	1.300	tf

$$= 0.12 \times 1.3$$

$$= 0.156 \quad tf$$

(5) Total Force in Seismic Time (Pe)

$$Pe = Peg + Pdo + Wge$$

$$= 13.473 + 1.512 + 0.156$$

$$= 15.141 \quad tf$$

(6) Comparison between Pe and Pa

Allowable stress in seismic time is 1.5 times of that of normal time. Therefor, Pe which is divided by 1.5, is used for comparison as follow:

$$\frac{Pe}{1.5} = \frac{18.1795.141}{1.5} = 10.094 < Pw = 12.971$$

Therefor, the design of gate is done using loads of normal time.

4. LOADS FOR GATE OPERATION

4.1 Lifting Time

$$Pu = \frac{1}{2} \cdot \{H5^2 - (H5 - H2)^2\} \cdot B2 \cdot \gamma$$

$$= \frac{1}{2} \times \{ 2.200^2 - (2.200 - 2.060)^2 \} \times 2.120 \times 1.00 = 5.110 \text{ tf}$$

Where,

H2	: Height of watertight portion	2.060	m
H5	: Water depth for Operation (Upstream)	2.200	m
B2	: Width of watertight portion	2.120	m
γ	: Specific gravity of water	1.00	

4.2 Lowering Time

$$Pd = \frac{1}{2} \cdot H7^2 \cdot B2 \cdot \gamma$$

$$= \frac{1}{2} \times 1.840^2 \times 2.120 \times 1.00 = \quad 3.589 \text{ tf}$$

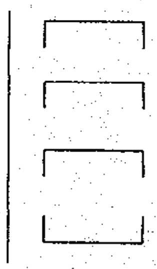
Where,

H7 : Water depth for Operation (Upstream) 1.840 m

5. MAIN BEAM

5.1. Mean Beam and Allotted Load

(1) Arrangement of Beam



No	Distance L (m)	Water Pressure p(tf/m^2)
1		p(1) = 1.940
	L(1) = 0.750	
2		p(2) = 2.690
	L(2) = 0.630	
3		p(3) = 3.320
	L(3) = 0.600	
4		p(4) = 3.920
	L(4) = 0.080	
At the gate floor		p(5) = 4.000

(2) Allotted Load

R (1) =

p (B) = {p (2) + 2 · p (1)} · L(1)/6 = 0.821

R(1) = 0.821 tf/m

R (2) =

p (U) = {p (1) + 2 · p (2)} · L(1)/6 = 0.915

p (B) = {p (3) + 2 · p (2)} · L(2)/6 = 0.914

R(2) = 1.829 tf/m

R (3) =

p (U) = {p (2) + 2 · p (3)} · L(2)/6 = 0.980

p (B) = L1 · [3 · L(3) · {p (3) + p (5)}
 − L1 · {p (3) + 2 · p (5)}] / 6 / L (3) = 1.035

R(3) = 2.015 tf/m

R (4) =

p (T) = {p (3) + 2 · p (5)} · $L1^2$ / 6 / L (3) = 1.454

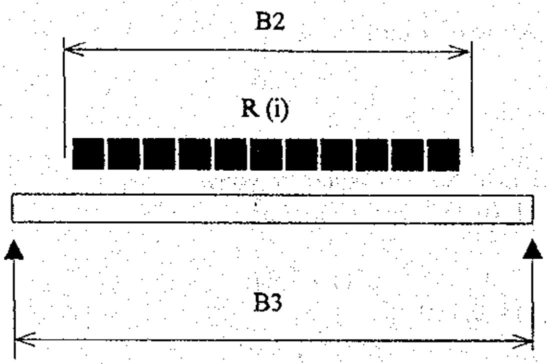
R(5) = 1.454 tf/m

Σ 6.119 tf/m

Where, L1 = L(3) + L(4)

= 0.600 + 0.080 = 0.680 m

5.2. Sectional Force of Main Beam



Where,

B2 : Distance of watertight rubbers 2.120 m

B3 : Span length 2.200 m

R(i) : Unit load tf/m

(1) Bending Moment

$$M(i) = \frac{1}{8} \cdot R(i) \cdot B2 \cdot (2 \cdot B3 - B2) \text{ tf-m}$$

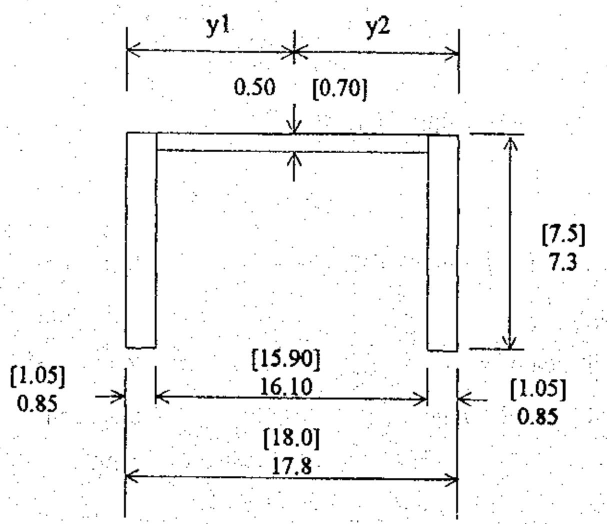
(2) Shearing Force

$$S(i) = \frac{1}{2} \cdot R(i) \cdot B2 \text{ tf}$$

No	R (i) (tf/m)	M (i) (tf/m)	S (i) (tf)
1	0.821	0.496	0.870
2	1.829	1.105	1.939
3	2.015	1.217	2.136
4	1.454	0.879	1.541

5.3. Stress of Main Beam

(a) Sectional Dimension of Beam [180 x 75 x 7.0 x 10.5



Moment of Area

I = 1.066 cm^4

Section Modulus

Zt = 120 cm^3

Zc = 120 cm^3

Sectional area of web

Aw = 8.1 cm^2

Sectional area of flange

Ac = 6.2 cm^2

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 0.85 x 7.3	6.2	0.43	2.67	1.15	0.37
t 0.50 x 16.1	8.1	8.90	72.09	641.60	173.89
t 0.85 x 7.3	6.2	17.38	107.76	1,872.80	0.37
Total	20.5	-----	182.52	2,690.18	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{182.52}{20.5} = 8.90 \text{ cm}$$

$$y2 = 17.8 - 8.90 = 8.90 \text{ cm}$$

Moment of area (I)

$$I = \Sigma(A \cdot y^2 + I\,o) - \Sigma A \cdot y1^2$$

$$= 2{,}690.18 - 20.5 \times 8.90^2 = 1{,}066 \text{ cm}^4$$

Section Modulus (Z)

$$Zt = \frac{I}{y2} = \frac{1{,}066}{8.90} = 120 \text{ cm}^3$$

$$Zc = \frac{I}{y1} = \frac{1{,}066}{8.90} = 120 \text{ cm}^3$$

(b) Bearing Stress

$$\sigma c = \frac{M(i)}{Zc}$$

....

$$\frac{9}{K} < \frac{L}{b} \le 30 \qquad \sigma ac = 1{,}200 - 11 \cdot \left(K \cdot \frac{L}{b} - 9\right)$$

$$\frac{9}{2.0} < \frac{70.0}{7.3} \le 30 \quad : \sigma ac = 1{,}200 - 11 \cdot \left(2.0 \cdot \frac{70.0}{7.3} - 9\right)$$

$$4.5 < 9.6 \le 30 \qquad = 1{,}088 \quad kgf/cm^2$$

Where,

L	:	Distance between fixed points of flange	70.0	cm
b	:	Width of flange	7.3	cm
Aw	:	Total sectional area of web	8.1	cm^2
Ac	:	Total sectional area of flange	6.2	cm^2
fa	:	Correction factor	1.00	

$$K = \sqrt{3 + \frac{1}{2} \cdot \frac{Aw}{Ac}} \qquad \text{In case } \frac{Aw}{Ac} < 2 \rightarrow K = 2$$

$$\text{Where, } \frac{Aw}{Ac} = \frac{8.1}{6.2} = 1.3$$

$$= 2$$

Allowable bearing stress

$$\sigma ac = fa \cdot \sigma ac = 1.00 \times 1{,}088 = 1{,}088 \ kgf/cm^2$$

Beam No.	**M (i) (tf-m)**	**σ c (kgf/cm^2)**	**Allowable Stress (kgf/cm^2)**
1	0.496	4.13	1,088
2	1.105	921	1,088
3	1.217	1,014	1,088
4	0.879	733	1,088

(c) Shearing Stress

$$\tau = \frac{S(i)}{Aw}$$

Allowable shearing stress

$$\tau a = fa \cdot \tau a = 1.00 \times 700 = 700 \ kgf/cm^2$$

No	**S (i) (tf**	**τ (kgf/cm^2)**	**Allowable Stress (kgf/cm^2)**
1	0.870	107	700
2	1.939	239	700
3	2.136	264	700
4	1.541	190	700

5.4. Deformation Rate of Main Beam

Deformation

$$\delta(i) = \frac{R(i)\cdot B2}{48\cdot E\cdot I}\cdot\left(B3^3 - \frac{1}{2}\cdot B3\cdot B2^2 + \frac{1}{8}\cdot B2^3\right) \quad \text{cm}$$

Where,

R(i)	:	Unit Load		kgf/cm
B2	:	Width of watertight portion	212	cm
B3	:	Span	220	cm
E	:	Modulus of elasticity	2.1×10^6	kgf/cm^2
I	:	Moment of section		vm^4

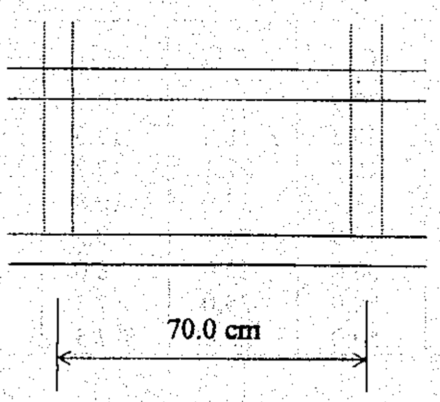
Deformation Rate

$$\frac{1}{Rd(i)} = \frac{\delta(i)}{B3}$$

Beam No.	R(i) Kgf/cm)	I (cm^4)	δ (i) (cm)	$\frac{1}{Rd(i)}$	Allowable Deformation
1	8.21	1,066	0.112	$\frac{1}{1,964}$	$\frac{1}{800}$
2	18.29	1,066	0.249	$\frac{1}{884}$	$\frac{1}{800}$
3	20.15	1,066	0.274	$\frac{1}{803}$	$\frac{1}{800}$
4	14.54	1,066	0.198	$\frac{1}{1,111}$	$\frac{1}{800}$

6. SKIN PLATE

6.1 Beam Allocation and Water Pressure



Water Pressure (kgf/cm^2)	Block No.	Length of Block (cm)	Span Length (cm)
0.2585 Average value between flanges of Main Beam →	1	67.5	67.5
0.3310 Average value between flanges of Main Beam →	2	55.5	55.5
0.3858 Average value between flanges of Main Beam →	3	45.0	45.0

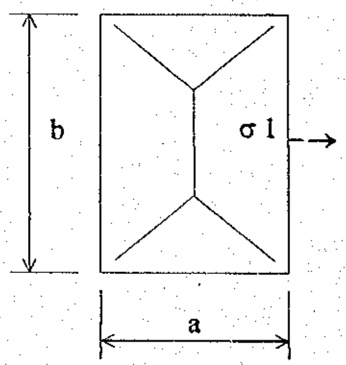
6.2 Stress on Skin Plate

Bearing moment on skin plate due the water pressure is estimated by the formula of DIN 10704 as follows:

$$\sigma = \frac{1}{100} \cdot k \cdot L^2 \cdot \frac{p}{(t-\varepsilon)^2}$$

Where,

σ	:	Stress on Skin Plate		kgf/cm^2
k	:	Coefficient shown in table below		
a	:	Length of shorter side of bloc		cm
b	:	Length of longer side of bloc		cm
L	:	Span Length as a shorter side		cm
p	:	Acting water pressure		kgf/cm^2
t	:	Thickness of plate	0.90	cm
ε	:	Margin of thickness of plate	0.20	cm



k Value

b / a	σ 1	σ 1	σ 1	σ 1
1.00	30.9	13.7	13.7	30.9
1.25	40.3	18.8	13.5	33.9
1.50	45.5	22.1	12.2	34.3
1.75	48.4	23.9	10.8	34.3
2.00	49.9	24.7	9.5	34.3
2.50	50.0	25.0	8.0	34.3
3.00	50.0	25.0	7.5	34.3
b/a > 3	50.0	25.0	7.5	34.3

Calculation Result

Allowable Stress : $\sigma a = fa \cdot \sigma a = 1.00 \times 1{,}200 = 1{,}200$ kgf/cm²

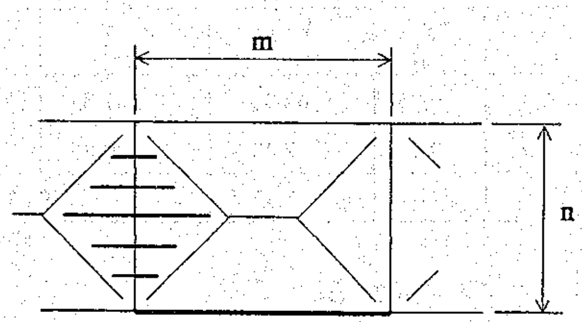
Block No.	a cm	b cm	b / a	k	L cm	p kgf/cm²	t - ε cm	σ kgf/cm²
1	67.5	70.0	1.04	32.29	67.5	Average value 0.2353	0.70	706
2	55.5	70.0	1.26	40.53	55.5	Average value 0.3043	0.70	775
3	45.0	70.0	1.56	1.56	45.0	Average value 0.3620	0.70	690

Based on the calculation result, the thickness of 9.0 mm is employed for the skin plate.

7. VERTICAL SUPPORTING BEAM

The relation between main beam and vertical beam is illustrated as follows. In the figure the bold horizontal lines are acting on vertical beam.

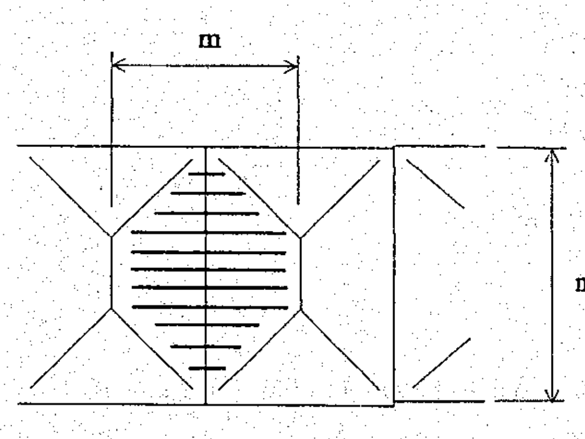
In case $m \geq n$



Bearing Moment

$$M = \frac{n^2 \cdot w}{12} \cdot m \quad \text{or} \quad \frac{n^3}{12} \cdot p \quad \text{(kgf-cm)}$$

In case $m < n$

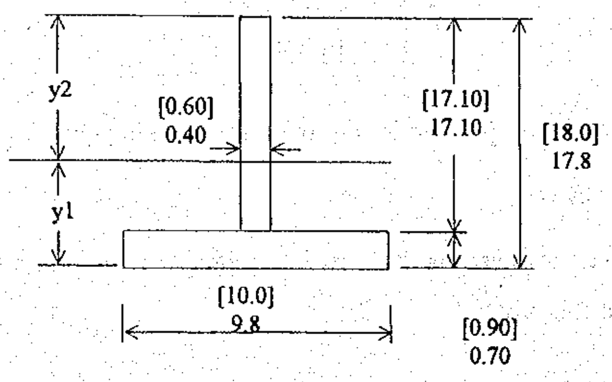


Bearing Moment

$$M = \frac{1}{24} \cdot (3 \cdot n - \frac{m^2}{n}) \cdot w \quad \text{(kgf-cm)} \quad \text{or,}$$

$$M = \frac{1}{24} \cdot (3 \cdot n^2 - m^2) \cdot m \cdot p \quad \text{(kgf-cm)}$$

(1) Sectional Dimensions of Beam PL 180 x 100 x 6 x 9



Moment of Section

$I = 438 \text{ cm}^4$

Section Modulus

$Zt = 92 \text{ cm}^3$

Sectional area of web

$Aw = 6.8 \text{ cm}^2$

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 0.40 x 17.1	6.8	9.25	62.90	581.83	166.67
t 0.70 x 9.8	6.9	0.35	2.42	0.85	0.28
Total	13.7	-----	65.32	749.63	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{65}{13.7} = 4.77 \text{ cm}$$

$$y2 = 17.8 - 4.77 = 13.03 \text{ cm}$$

Moment of Section (I)

$$I = \Sigma (A \cdot y^2 + I\,o) - \Sigma A \cdot y1^2$$

$$= 750 - 13.7 \times 4.77^2 = 438 \text{ cm}^4$$

Section Modulus (Z)

$$Z = \frac{I}{y2} = \frac{438}{4.77} = 92 \text{ cm}^3$$

(2) Stress on Vertical Beam

$$\sigma = \frac{M}{Z}$$

$$\frac{9}{K} < \frac{L}{b} \leq 30 \; : \; \sigma a = 1{,}200 - 11 \cdot \left(K \cdot \frac{L}{b} - 9 \right)$$

Where,

L : Distance between fixed points of flange n cm

b	:	Width of flange	9.8	cm
Aw	:	Total sectional area of web	6.8	cm^2
Ac	:	Total sectional area of flange	6.9	cm^2
fa	:	Correction factor		1.0

$$K = \sqrt{3+\frac{1}{2}\cdot\frac{Aw}{Ac}} \qquad \text{In case } \frac{Aw}{Ac} < 2 \rightarrow K = 2$$

$$\text{Where, } \frac{Aw}{Ac} = \frac{6.8}{6.9} = 1.0$$

$$= 2$$

$\sigma a = fa \cdot \sigma a = 1.00 \times \sigma a$

Block No.	m cm	n cm	p kgf/cm^2	M kgf/cm^2	σ kgf/cm^2	Allowable Stress kgf/cm^2
1	70.0	75.0	0.2353	8218	89	1,131
2	70.0	63.0	0.3043	6341	69	1,158
3	70.0	60.0	0.3620	6516	71	1,164

8. GUIDE FRAME

8.1 Bearing Stress of Slide Plate

$$\sigma s = \frac{p \cdot B2}{2 \cdot b}$$

Where,

p	: Water pressure	0.400	kgf/cm^2	(State of rest)
		0.220	kgf/cm^2	(State of move)
B2	: Width of watertight area	212.0	cm	
b	: Width of slide plate	8.0	cm	

$$= \frac{0.400 \times 212.0}{2 \times 8.0} = 5.3 \quad kgf/cm^2 < \sigma sa \text{ (State of rest)}$$

$$= \frac{0.220 \times 212.0}{2 \times 8.0} = 2.9 \quad kgf/cm^2 < \sigma sa \text{ (State of move)}$$

Allowable Bearing Stress

σsa = 180 kgf/cm^2 (State of rest)

= 60 kgf/cm^2 (State of move)

8.2 Bearing Stress of Concrete

$$\sigma c = \frac{p \cdot B2}{2 \cdot bf}$$

Where,

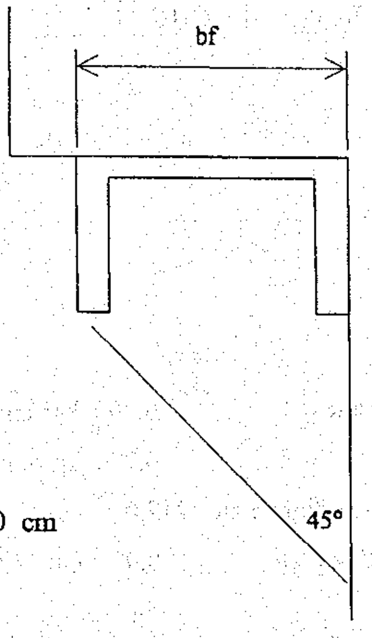
p	: Water pressure		0.400	kgf/cm^2
B2	: Width of watertight area	212.0		kgf/cm^2
bf	: Width of stress receiving area	20.0		cm

$$= \frac{0.400 \times 212.0}{2 \times 20.0} = 2.1 \quad kgf/cm^2 < \sigma ca$$

Allowable Bearing Stress

$\sigma ca = 55 \quad kgf/cm^2$

8.3 Shear Stress of Concrete



bf = 20.0 cm

Length of shearing stress : L

$$L = 2 \cdot bf$$

$$= 2 \times 20.0 = 40 \text{ cm}$$

Shearing stress of concrete : τc

$$\tau c = \frac{\sigma c \cdot bf}{L}$$

$$= \frac{2.1 \times 20.0}{40} = 1.1 \ kgf/cm^2 < \tau ca$$

Allowable shearing stress

τca = 4.0 kgf/cm^2

9. LOADS FOR GATE OPENING AND CLOSING

9.1 Loads in Gate Operation

(1) Gate Lifting Time

Pu = 5.110 tf (Water pressure)

(2) Gate Lowering Time

Pd = 3.539 tf (Water pressure)

9.2 Loads Acting on Gate

(1) Total Weight of Gate Wg

Self Weight	Wg1	= 1.300 tf
Weight of Spindle	Wg2	= 0.300 tf
Total Weight	Wg	= Wg1 + Wg2 = 1.600 tf

(2) Friction Due to Slide

(Gate lifting time)

Fgu = $\mu s \cdot Pu$

Where,

μs : Friction coefficient of sliding face 0.6

Pu : Loads in gate lifting time 5.110 tf

= 0.6 x 5.110

= 3.066 tf.

(Gate lowering time)

Fgd = $\mu s \cdot Pd$

Where,

μs : Friction coefficient of sliding face 0.6

Pd : Loads in gate lifting time 3.589 tf

= 0.6 x 3.589

= 2.153 tf

(3) Friction of Rubber

(Gate Lifting Time)

Fru = $\mu r \cdot \{ q \cdot (L1 + 2 \cdot L2) + p1 \cdot b \cdot L1 + 2 \cdot p2 \cdot b \cdot L2 \}$

Where,

μr	: Friction coefficient of rubber	0.7	
q	: Initial pressure	0.100	tf/m
p1	: Average force on upper face	0.140	tf/m^2
p2	: Average force of side face	1.170	tf/m^2
b	: Effective width of stress receiving area	0.050	m
L1	: Length of watertight rubber of upper side	2.120	m
L2	: Length of watertight rubber of side face	2.060	m

$= 0.7 \times \{0.100 \times (2.120 + 2 \times 2.060) + 0.140 \times 0.050 \times 2.120$

$+ 2 \times 1.170 \times 0.050 \times 2.060\}$

$= 0.616$ tf.

(Gate lowering time)

$Frd = \mu r \cdot \{ q \cdot (L1 + 2 \cdot L2) + 2 \cdot p2 \cdot b \cdot L2 \}$

Where,

μr	: Friction coefficient of rubber	0.7	
q	: Initial pressure	0.100	tf/m
p2	: Average force of side face	0.822	tf/m^2
b	: Effective width of stress receiving area	0.050	m
L1	: Length of watertight rubber of upper side	2.120	m
L2	: Length of watertight rubber of side face	2.060	m

$= 0.7 \times \{0.100 \times (2.120 + 2 \times 2.060) + 2 \times 0.822 \times 0.050 \times 2.060\}$

$= 0.555$ tf.

(4) Up Lift

(Gate Lowering Time)

$$Fbd = \frac{Wg}{\gamma} \cdot \frac{s}{100}$$

Where,

Wg	: Self weight of gate	1.000	tf
γ	: Specification gravity of steel	7.85	
s	: Portion of gate submerged	20.0	%

$$= \frac{1.000}{7.850} \times \frac{20.0}{100} = 0.025 \text{ tf}$$

(5) Downward Force Acting on Upper Portion of Gate

(Gate Lifting TIme)

$Ftdu = p \cdot t \cdot B \cdot k \cdot \Delta H + D \cdot B$

Where,

p	: Acting pressure	0.140	tf/m^2
t	: Width of stress receiving area	0.249	m
B	: Length of stress receiving area	2.120	m
k	: Coefficient of water overflowed	0.000	
ΔH	: Water level difference	0.140	m
D	: Thickness of gate body	0.209	m

= 0.140 x 0.249 x 2.120 + 0.000 x 0.140 x 0.209 x 2.120

= 0.074 tf.

(6) Upward Force Acting in Lower Portion of Gate

(Gate Lowering TIme)

Fbud = p · t · B

Where,

p	: Acting pressure	1.840	tf/m^2
t	: Width of stress receiving area	0.050	m
B	: Length of stress receiving area	2.120	m

= 1.840 x 0.050 x 2.120

= 0.195 tf.

9.3 Total Loads for Gate Opening and Closing

Item	Lifting Force	Lowering Force
Self weigh	1.300	1.300
Friction of supporting plate	3.066	-2.153
Friction of rubber	0.616	-0.555
Up Lift	-----	-0.025
Downward force of gate	0.074	-----
Upward force of gate	-----	-0.195
Total	5.056	-1.628

Nota : Lifting Force 5.500 tf
Lowering Force 2.000 tf

10. GATE HOISTING SYSTEM

10.1 Specification of System

Method	Spindle Rod (1)
Middle Shoe	Non
Operation Method	Electrical and manual operation

Control Method	Local manual operation & remote manual operation		
Total Head	2.500 m		
Hoisting Sped	0.3 m/min		
Hoisting Load	Lifting time	W1 = 5,5000 kgf	
	Lowering time	W2A = 2,000 kgf	
		(by the calculation of gate opening and closing loads)	
	Lowering time	W2B = 8,276 kgf	
		(by motor output)	
Power	220 V, 50 Hz		
Applied Standard	Technical Standard for Dam/Weir Facilities		

10.2 Spindle

Specification on Spindle Screw

- Name of Spindle Screw — Tr – 90 (30° Trapezoidal screw)
  - Outer diameter — 90 mm
  - Diameter in Vally Portion — 78.0 mm
  - Effective Diameter — 84.0 mm
  - Interval — 12 mm
  - Number of screw line — 1 line
- Sectional Dimension of Screw in Vally Portion
  - Area (A) — 47.8 cm^2
  - Moment of Area — 181.7 cm^4
- Spindle Factor — FS = 0.010707
- Supporting Interval of Spindle — 270.0 cm
- Condition of Spindle End (Driving Portion) $P_{in} - P_{in}$ (Coefficient = 1.00)
- Material of Spindle (Driving Portion) — SUS 304

10.3 Force on Spindle

(1) Spindle of Torque

$$TS = W \cdot FS$$
$$= 5{,}500 \times 0.010707$$
$$= 58.89 \text{ kgf·m}$$

10.4 Selection of Limi Torque

(1) Selection of Limi Torque

Diameter of Spindle, Thrust Force, Torque → JMB-3 selected

Allowable Spindle Diameter	57 ~ 127	<>	90	mm
Allowable Thrust Force	44,100	>	5,500	kgf
Allowable Torque	406	>	58,89	kgf·m

(2) Rotation Number of Limi Torque Axis

$$NB = \frac{V}{L} = \frac{300}{12} = 25.00 \text{ rpm}$$

Where,

V : Lifting Speed 300 mm/min

L : Screw lead 12 mm

(3) Ratio of Reduction Speed

$$ia = \frac{NM}{NB} = \frac{1{,}450}{25.00} = 58.00$$

Where,

NM : Design rotation number of motor 1,450 rpm

Based on the above, the ratio of reduction speed applied is 61.50

(4) Calculation of Motor

$$KW = \frac{TS \cdot NM}{974 \cdot i \cdot \eta L}$$

Where,

ηL : Operation efficiency of limi torque 0.39

$$= \frac{58.89 \times 1{,}450}{974 \times 61.50 \times 0.39}$$

$$= 3.66 \text{ kw}$$

Therfore, a motor with 5.50 kw is used

(5) Backling Stress of Spindle

$$Wc = \frac{974 \cdot KW \cdot \eta l \cdot i}{NM \cdot FS}$$

$$= \frac{974 \times 5.50 \times 0.39 \times 61.50}{1{,}450 \times 0.010707}$$

$$= 8{,}276 \text{ kgf}$$

(6) Safety Factor Against Backling Load on Spindle

(Screw Axis of Driving Portion)

- Slenderness ratio at boundary (λ_o)

$$\lambda_o = \pi \cdot \sqrt{\frac{E}{0.6 \cdot \sigma y}}$$

Where,

E	: Young's modulus	1.97×10^6	kgf/cm²
σ y	: Yield point or proof stress of material	2,100	kgf/cm²

$$= 3.14 \times \sqrt{\frac{1.97 \times 10^6}{0.6 \times 2{,}100}} = 124.2$$

- Slenderness ratio (λ)

$$\lambda = \beta \cdot L / r$$

Where,

β	: Condition at material end	1.00	
L	: Design Length	270.0	cm
r	: Radius of gyration of spindle		

$$= \sqrt{I / A} = \sqrt{181.7 / 47.8} = 1.95 \text{ cm}$$

$$= 1.00 \times 270.0 / 1.95 = 138.5$$

- $\lambda > \lambda_o$ therefore, Euler's equation is adopted

$$\sigma k = \frac{\pi^2 \cdot E}{\lambda^2}$$

$$= \frac{3.14^2 \times 1.97 \times 10^6}{138.5^2} = 1{,}014 \text{ kg/cm}^2$$

- Compressive Stress σ c

$$\sigma c = \frac{Wc}{n \cdot A}$$

$$= \frac{8{,}276}{1 \times 47.8} = 173 \text{ kg/cm}^2$$

- Safety factor Sf

Normal use Sf $= \frac{\sigma k}{\sigma c} = \frac{1,014}{173} = 5.86 > 4.0$

At maximum torque Sf $= \frac{\sigma k}{3 \cdot \sigma c} = \frac{1,014}{3 \times 173} = 1.95 > 1.1$

(7) Calculation in Manual Operation

$$F = \frac{TS}{HR \cdot \eta H \cdot R \cdot HRR \cdot \eta R}$$

Where,

HR	: Standard ratio of reduction speed	41.00	
ηH	: Efficiency rate under manual operation	0.33	
R	: Radius of handle	0.305	m
HRR	: Ratio of reduction speed of manual reduction gear	5.50	
ηR	: Efficiency manual reduction gear	0.95	

$$= \frac{58.89}{41.00 \times 0.33 \times 0.305 \times 5.50 \times 0.95}$$

$= 2.7$ kgf

10.5 Selection Result

With all calculation results mentioned above, the hoisting equipment is designed as follows:

- Limi Torque JMB-3
- Electrical Motor 5.50 kw
  With manual reduction gear
- Spindle Screw Tr-90 1 line screw

10.6 Pressure on Stem Nut Surface

$$Q = \frac{Wc}{n \cdot \frac{\pi}{4} \cdot (D^2 - d^2) \cdot Z}$$

$$= \frac{8,276}{1 \times 0.785 \times (9.0^2 - 7.8^2) \times 19.83} = 26.4 \text{ kgfcm}^2 < Qa$$

Where,

Ws : Bucking load of spindle 8,276 kgf

n	:	Number of spindle	1	
D	:	Outer diameter of spindle	9.0	cm
d	:	Spindle diameter of vally portion	7.8	cm
Z	:	Number of screw		

$$Z = \frac{L}{P} = \frac{23.8}{1.2} = 19.83$$

L	:	Length of stem nut	23.8	cm
P	:	Internal of screw	1.2	cm

Allowable surface pressure Qa = 60 kgf/cm^2

4.5.5 Temporary Gate (Steel Stop Log)

1. DESIGN ITEM

1.1 Design Condition

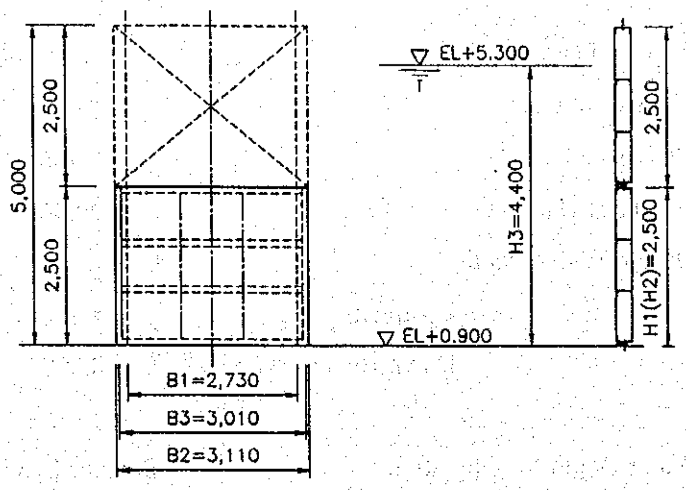
- Type of Gate	Steel Slide Gate	
- Number of Gate	12	Gates
- Clear span	2.730	m
- Height of Gate	2.500	m
- Design Water Depth		
Upstream	4.400	m
Downstream	0.000	m
- Water Depth for Operation	Balance by water pressure	
- Gate Floor Level	EL 0.900	m
- Method for Watertightness	For front 3 faces watertightness	
- Hoisting System	by Truck Crane	

1.2 Design Condition

- General Item

• Horizontal Seismic Intensity		——		
• Deffelence in Temperature		——		
• Wind Load		——		
• Deformation of Main Girder		1/ 600		
• Margin of Thickness				
Skin Plate (Side facing water)		0.00	mm	
Other Members (Side facing water)		0.00	mm	
• Major Material of Gate	Main beam	SS400		
	Skin Plate	SS400		
• Allowable Stress				
Steel	Technical Manual for Dam and Weir Chapter 2, 2-0-7			
	Correction Factor	Normal Case		Seismic Case
		1.000		1.650
Concrete	Bearing stress	55.0	kgf/cm^2	
	Share stress	4.0	kgf/cm^2	

2. BASIC DIMENSIONS

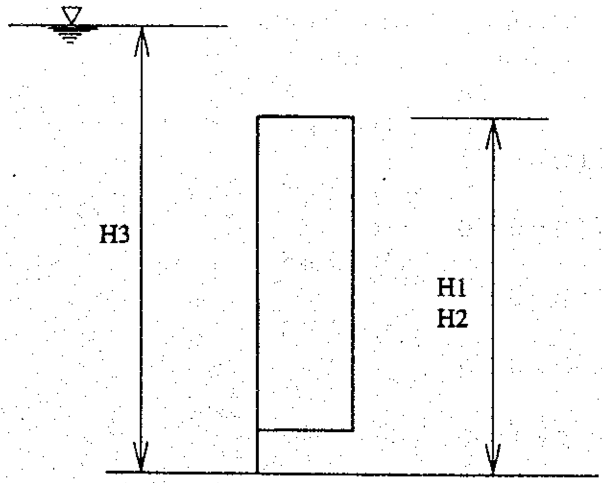


Where,

B1	:	Clear Span	2.730	m
B2	:	Length of Watertight Area	3.110	m
B3	:	Span Between Rollers	3.010	m
H1	:	Height of Gate	2.500	m
H2	:	Height of Watertight Area	2.500	m
H3	:	Design Water Depth Upstream	4.400	m
H4	:	Design Water Depth Downstream	0.000	m

3. ACTING LOADS

3.1 Loads in Normal Time



(1) Static Water Pressure

$$P_W = \frac{1}{2} \cdot \{H3^2 - (H3 - H2)^2\} \cdot B2 \cdot \gamma$$

$$= \frac{1}{2} \times \{4.400^2 - (4.400 - 2.500)^2\} \times 3.110 \times 1.00$$

$$= 24.491 \quad \text{tf}$$

Where,	H2	:	Height of Watertight Portion	2.500	m
	H3	:	Design Water Depth (Upstream)	4.400	m
	B2	:	Width of Watertight Portion	3.110	m
	γ	:	Specific Gravity of Water	1.00	

4. LOADS FOR GATE OPERATION

4.1 Lifting Time

Non

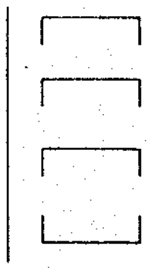
4.2 Lowering Time

Non

5. MAIN BEAM

5.1. Mean Beam and Allotted Load

(1) Arrangement of Beam



No	Distance L (m)	Water Pressure p(tf/m^2)
1		p(1) = 1.900
	L(1) = 0.825	
2		p(2) = 2.725
	L(2) = 0.800	
3		p(3) = 3.525
	L(3) = 0.800	
4		p(4) = 4.325
	L(4) = 0.075	
At the gate floor		p(5) = 4.400

(2) Allotted Load

R (1) =

p (B) = {p (2) + 2 · p (1)} · L(1)/6 = 0.897

R(1) = 0.897 tf/m

R (2) =

p (U) = {p (1) + 2 · p (2)} · L(1)/6 = 1.011

p (B) = {p (3) + 2 · p (2)} · L(2)/6 = 1.197

R(2) = 2.208 tf/m

R (3) =

p (U) = {p (2) + 2 · p (3)} · L(2)/6 = 1.303

p (B) = L1 · [3 · L(3) · {p (3) + p (5)}
− L1 · {p (3) + 2 · p (5)}] / 6 / L (3) = 1.501

R(3) = 2.804 tf/m

R (4) =

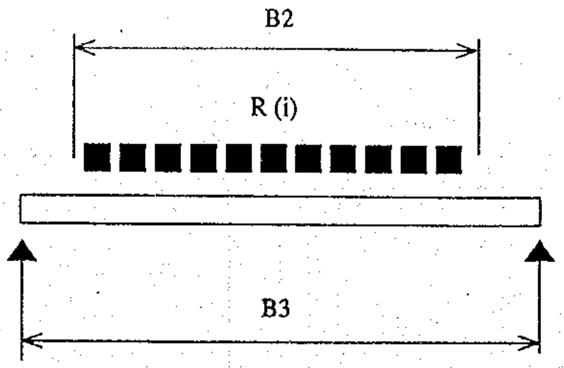
p (T) = {p (3) + 2 · p (5)} · L1^2 / 6 / L (3) = 1.966

R(5) = 1.966 tf/m

Σ 7.875 tf/m

Where, L1 = L(3) + L(4)
= 0.800 + 0.075 = 0.875 m

5.2. Sectional Force of Main Beam



Where,

B2 : Distance of watertight rubbers 3.110 m

B3 : Span length 3.010 m

R(i) : Unit load tf/m

(1) Bending Moment

$$M(i) = \frac{1}{8} \cdot R(i) \cdot B2 \cdot (2 \cdot B3 - B2) \text{ tf-m}$$

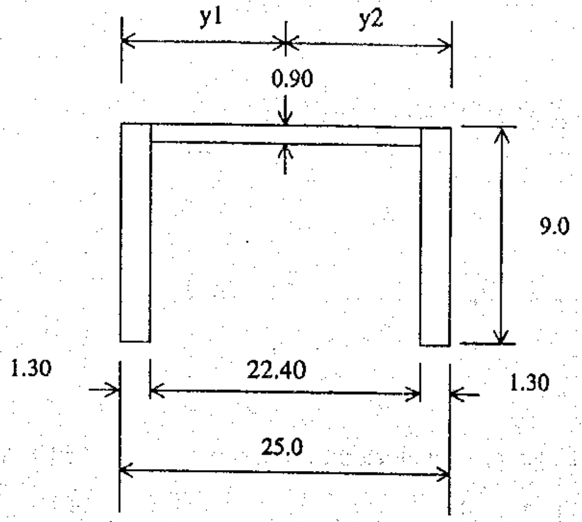
(2) Shearing Force

$$S(i) = \frac{1}{2} \cdot R(i) \cdot B2 \text{ tf}$$

Beam No	R (i) (tf/m)	M (i) (tf/m)	S (i) (tf)
1	0.897	1.015	1.395
2	2.208	2.498	3.433
3	2.804	3.172	4.360
4	1.966	2.224	3.057

5.3. Stress of Main Beam

(a) Sectional Dimension of Beam ⊏ 380 x 125 x 16.0 x 25.0



Moment of Area

I = 4,132 cm^4

Section Modulus

Zt = 331 cm^3

Zc = 331 cm^3

Sectional area of web

Aw = 20.2 cm^2

Sectional area of flange

Ac = 11.7 cm^2

Member cm	A cm^2	y cm	$A \cdot y$ cm^3	$A \cdot y^2$ cm^4	I o cm^4
t 1.30 x 9.0	11.7	0.65	7.61	4.94	1.65
t 0.90 x 22.4	20.2	12.50	252.50	3,156.25	842.96
t 1.30 x 9.0	11.7	24.35	284.89	6,937.19	1.65
Total	43.6	-----	545.00	10,944.64	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{545.00}{43.6} = 12.50 \text{ cm}$$

$$y2 = 25.0 - 12.50 = 12.50 \text{ cm}$$

Moment of area (I)

$$I = \Sigma (A \cdot y^2 + I o) - \Sigma A \cdot y1^2$$

$$= 10{,}944.64 - 43.6 \times 12.50^2 = 4.132 \text{ cm}^4$$

Section Modulus (Z)

$$Zt = \frac{I}{y2} = \frac{4{,}132}{12.50} = 331 \text{ cm}^3$$

$$Zc = \frac{I}{y1} = \frac{4{,}132}{12.50} = 331 \text{ cm}^3$$

(b) Bearing Stress

$$\sigma c = \frac{M(i)}{Zc}$$

Allowable bearing stress

$\sigma ac = fa \cdot \sigma ac = 1.10 \times 1{,}200 = 1{,}320$ kgf/cm^2

Beam No	M (i) (tf/m)	σc (kgf/cm^2)	Allowable Stress (kgf/cm^2)
1	1.015	307	1,320
2	2.498	755	1,320
3	3.172	958	1,320
4	2.224	672	1,320

(c) Shearing Stress

$$\tau = \frac{S(i)}{Aw}$$

Allowable shearing stress

$\tau a = fa \cdot \tau a = 1.10 \times 700 = 770$ kgf/cm^2

Beam No	M (i) (tf/m)	τ (kgf/cm^2)	Allowable Stress (kgf/cm^2)
1	1.395	69	770
2	3.433	170	770
3	4.360	216	770
4	3.057	151	770

5.4. Deformation Rate of Main Beam

Deformation

$$\delta (i) = \frac{R(i) \cdot B2}{48 \cdot E \cdot I} \cdot (B3^3 - \frac{1}{2} \cdot B3 \cdot B2^2 + \frac{1}{8} \cdot B2^3) \qquad \text{cm}$$

Where,				
	R(i) :	Unit Load		kgf/cm
	B2 :	Width of watertight portion	311	cm
	B3 :	Span	301	cm
	E :	Modulus of elasticity	2.1×10^6	kgf/cm^2
	I :	Moment of section		vm^4

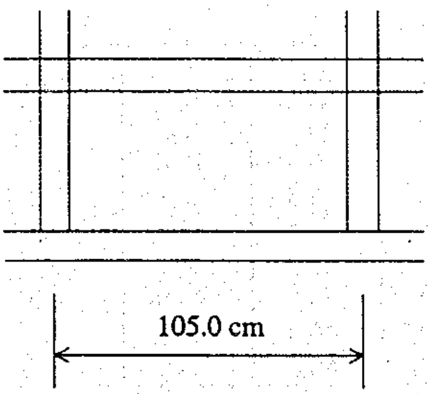
Deformation Rate

$$\frac{1}{Rd(i)} = \frac{\delta(i)}{B3}$$

Beam No.	R(i) Kgf/cm)	I (cm^4)	δ (i) (cm)	$\frac{1}{Rd(i)}$	Allowable Deformation
1	8.97	4,132	0.110	$\frac{1}{2,736}$	$\frac{1}{600}$
2	22.08	4,132	0.272	$\frac{1}{1,107}$	$\frac{1}{600}$
3	28.04	4,132	0.345	$\frac{1}{872}$	$\frac{1}{600}$
4	19.66	4,132	0.242	$\frac{1}{1,244}$	$\frac{1}{600}$

6. SKIN PLATE

6.1 Beam Allocation and Water Pressure



Water Pressure (kgf/cm^2)	Block No.	Length of Block (cm)	Span Length (cm)
0.2358 Average value between flanges of Main Beam →	1	73.5	73.5
0.3170 Average value between flanges of Main Beam →	2	71.0	71.0
0.3925 Average value between flanges of Main Beam →	3	62.0	62.0

6.2 Stress on Skin Plate

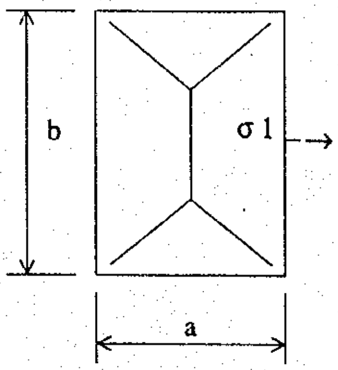
Bearing moment on skin plate due the water pressure is estimated by the formula of DIN as follows:

$$\sigma = \frac{1}{100} \cdot k \cdot L^2 \cdot \frac{p}{(t-\varepsilon)^2}$$

Where,				
	σ	:	Stress on Skin Plate	kgf/cm^2
	k	:	Coefficient shown in table below	
	a	:	Length of shorter side of bloc	cm
	b	:	Length of longer side of bloc	cm
	L	:	Span Length as a shorter side	cm
	p	:	Acting water pressure	kgf/cm^2

t : Thickness of plate 0.90 cm

ε : Margin of thickness of plate 0.00 cm



k Value

b / a	σ 1	σ 1	σ 1	σ 1
1.00	30.9	13.7	13.7	30.9
1.25	40.3	18.8	13.5	33.9
1.50	45.5	22.1	12.2	34.3
1.75	48.4	23.9	10.8	34.3
2.00	49.9	24.7	9.5	34.3
2.50	50.0	25.0	8.0	34.3
3.00	50.0	25.0	7.5	34.3
b/a > 3	50.0	25.0	7.5	34.3

Calculation Result

Allowable Stress : σ a = fa · σ a = 1.10 x 1,200 = 1,320 kgf/cm^2

Block No.	a cm	b cm	b / a	k	L cm	p kgf/cm^2	t - ε cm	σ kgf/cm^2
1	73.5	105.0	1.43	44.01	73.5	Average value 0.2358	0.90	692
2	71.0	105.0	1.48	45.06	71.0	Average value 0.3170	0.90	889
3	62.0	105.0	1.69	47.75	62.0	Average value 0.3925	0.90	889

Note : Indicates the thickness of imaginary plate.

Based on the calculation result, the thickness of 9.0 mm is employed for the skin plate.

7. GUIDE FRAME AND OTHERS

7.1 Bearing Stress of Slide Plate

$$\sigma s = \frac{p \cdot B2}{2 \cdot b}$$

Where,

p : Water pressure 0.440 kgf/cm^2 (in state of rest)

0.000 kgf/cm^2 (in state of motion)

B2 : Width of watertight area 311.0 cm

b : Width of slide plate 15.0 cm

$$= \frac{0.440 \times 311.0}{2 \times 15.0} = 4.6 \quad kgf/cm^2 < \sigma sa \text{ (in state of rest)}$$

$$= \frac{0.000 \times 311.0}{2 \times 15.0} = 0.0 \quad kgf/cm^2 < \sigma sa \text{ (in state of motion)}$$

Allowable Bearing Stress

σsa = 180 kgf/cm^2 (in state of rest)

= 60 kgf/cm^2 (in state of motion)

7.2 Bearing Stress of Concrete

$$\sigma c = \frac{p \cdot B2}{2 \cdot bf}$$

Where,

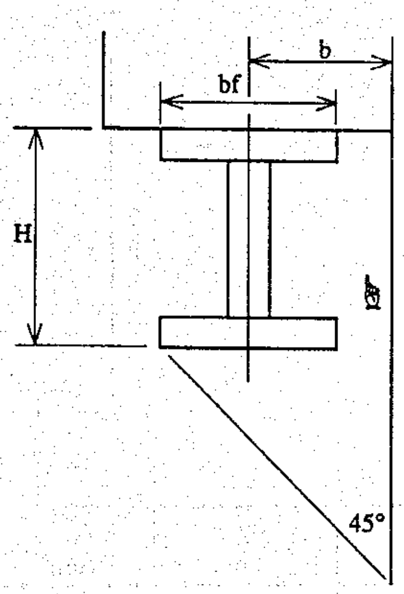
p	: Water pressure	0.440	kgf/cm^2
B2	: Width of watertight area	311.0	kgf/cm^2
bf	: Width of stress receiving area	15.0	cm

$$= \frac{0.440 \times 311.0}{2 \times 15.0} = 4.6 \quad kgf/cm^2 < \sigma ca$$

Allowable Bearing Stress

σca = 55 kgf/cm^2

7.3 Shear Stress of Concrete



b = 17.0 cm

bf = 15.0 cm

H = 19.4 cm

Shearing length of concrete : (L)

$$L = 2 \cdot \left(b + \frac{bf}{2}\right) + H$$

$$= \quad 2 \times \left(17.0 + \frac{15.0}{2}\right) + 19.4 = 68 \text{ cm}$$

Shearing strength of concrete : τc

$$\tau c = \frac{\sigma c \cdot bf}{L}$$

$$= \frac{4.6 \times 15.0}{68} = 1.0 \text{ kgf/cm}^2 < \tau ca$$

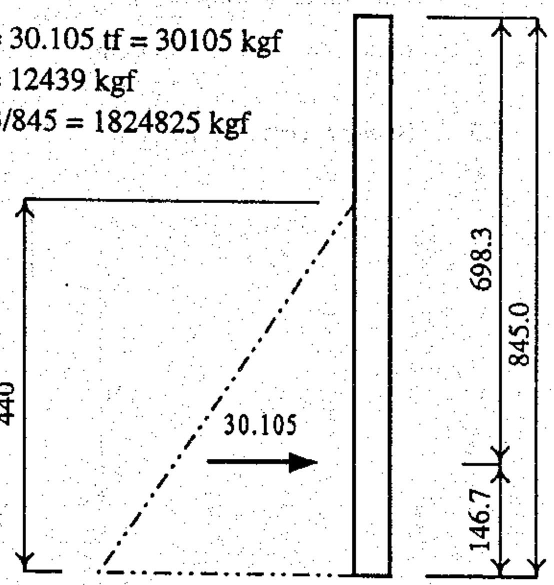
Allowable shearing strength

$\tau = 4.0 \text{ kgf/cm}^2$

8. CENTER POST

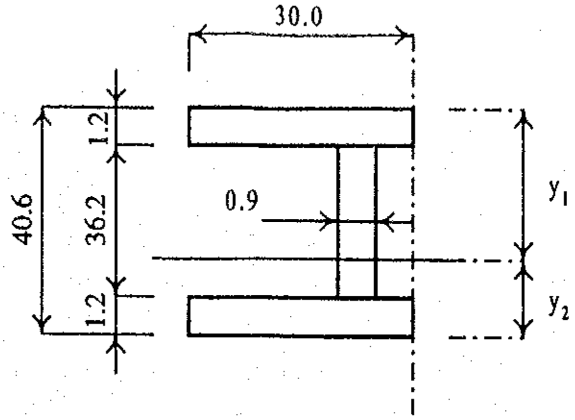
Loading Calculation

Water Pressure $Pw = 1/2 \times 4.40^2 \times 3.11 = 30.105 \text{ tf} = 30105 \text{ kgf}$
Shearing Stress $S = Pw/2 \times 698.3/845.0 = 12439 \text{ kgf}$
Bending Moment $M = Pw/2 \times 146.7 \times 698.3/845 = 1824825 \text{ kgf}$



Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	Io cm^4
30.0×1.2	36.00	0.60	21.60	12.96	4.32
0.9×38.2	34.38	20.30	697.91	14167.65	7180.72
30.0×1.2	36.00	40.00	1440.00	57600.00	4.32
Σ	106.38	—	2159.51	75969.98	

$y_1 = \Sigma(A \cdot y)/\Sigma A = 2159.51/106.38 = 20.30\text{cm}$



$y_2 = 40.6 - 20.30 = 20.30$cm

Geometrical moment of inertia

$I = \Sigma(A \cdot y^2 + Io) - \Sigma A \times y1^2 = 32132\ cm^4$

Modulus of section

$Z = I / y1 = 1583\ cm^3$

Bending Stress

$\sigma = M / Z = 1153\ kgf/cm^2 < 1200 \times 1.1 = 1320\ kgf/cm^2$ …… O.K.

Shearing Stress

$\tau = S / Aw = 362\ kgf/cm^2 < 700 \times 1.1 = 770\ kgf/cm^2$ …… O.K.

Deflection

$y = Pw \cdot a^2 \cdot b^2/6/E/I/L = 0.923$ cm $\qquad \sigma = 1 / 915 < 1 / 600$ …… O.K

E: 2.1×10 ^ 6 kgf/cm^2

4.6 Maintenance Bridge

4.6.1 Maintenance Bridge

(a) Features of Bridges

The features of maintenance bridges are shown in the table below.

Bridge Name	Span Length (m)	Bridge Length (m)	Effective Width (m)	Width(m)		
				Driving	Sidewalk	Total
Maintenance Bridge No.1	5.5	8.3	6.4	5.0	1.4	7.0
Maintenance Bridge No.2	18.5	21.0	6.4	5.0	1.4	7.0
Maintenance Bridge No.3	18.5	21.0	6.4	5.0	1.4	7.0
Maintenance Bridge No.4	18.5	21.0	6.4	5.0	1.4	7.0
Maintenance Bridge No.5	5.5	8.3	6.4	5.0	1.4	7.0

Note: Sidewalk is adopted at one side only.

(b) Type of Substructure

There are various types of super structure which are adaptable for bridges with these spans and load conditions. From the viewpoint of maintenance, concrete structures are preferable, as they require less maintenance efforts.

Therefore, a reinforced concrete type (RC type) and a post-tension pre-stressed concrete type (PC type) are recommended for superstructure here. When the structures of RC type and PC type compared, PC type has thinner girder than RC type. On the other hand, RC type is commonly cheaper and more often adopted than PC type when the length is smaller than 20 m (refer to Fig.6.4.5). Therefore, for type selection, the length of 20 m shall be the border between RC type and PC type, as far as the depth of girder brings about no problem.

In case of the maintenance bridges for Simongan Weir, the length of bridge No.1 and 5 is less than 20 m and there is no problem of girder depth. Therefore, RC type girder is selected for these two bridges.

For bridge No.2, 3 and 4 of which lengths are longer than 20m, then PC type is selected. For design of superstructure, a standard design of BINAMARUGA was adopted.

(c) Design Criteria

The following design criteria are used to set up the loading conditions on the superstructures of the proposed bridges.

- Peraturan Perencanaan Teknik Jembatan 1992 BINA MARGA (BMS)

  (Bridge Design Code)

- Design Manual, December 1992 BINA MARGA

The design criteria for the bridge design are stated in the "INTERIM REPORT (4), VOLUME Ⅲ: DESIGN CRITERIA".

Live load

- Wheel load (T) = 10 tf/wheel (Truck Crane)

- Side walk load = 350 kgf/m^2

Earthquake force is applied in accordance with "Technical Design of Bridge (Peraturan Prencanaan Teknik Jembatan Tahun 1922)" (hereinafter called the Code). The minimum earthquake design load is derived from the following formula:

$Teq = Kh \cdot I \cdot Wr$

where,

Teq: total base shear force in the direction being considered (kN)

Kh : coefficient of horizontal seismic loading

$Kh = C \cdot S$

Where,

C : base shear coefficient for the appropriate zone, period and side condition (=0.15, zone 4 (refer to INTERIM REPORT (4), VOLUME Ⅲ DESIGN CRITERIA))

S : structure type factor (= 1.0 for RC type or 1.3 for PC type)

I : safety factor of importance of structure (= 1.0 road bridge)

Wr: total nominal weight of structure object to seismic acceleration taken as dead load

superimposed dead load (kN)

Therefore, the design seismic loads are as calculated follows:

- For No.1 and No.5 bridges

$$Teq = 0.15 \times 1.0 \times 1.0 \times Wr = 0.15\ Wr$$

- For No.2,3 and 4 bridges

$$Teq = 0.15 \times 1.3 \times 1.0 \times Wr = 0.195Wr \Rightarrow 0.2\ Wr$$

- Horizontal force from temporary gate: 9.25 tf/m

4.6.2 Approach Bridge

(a) Features of Bridges

The features of approach bridges are shown in the table below.

Bridge Name	Span Length (m)	Bridge Length (m)	Effective Width (m)	Width (m)	Load
Approach Bridge No.1	11.46	13.0	6.4	7.0	vehicle
Approach Bridge No.2	7.46	9.0	6.4	7.0	vehicle

(b) Geology at the Site

The boring data clarified that Damar Formation, which is the base rock, appears below EL.2.0 m at the bridge site, when the ground elevation of bridge side is EL.8.634 m (hole No. SB-3). Above that elevation, an alluvial layer exists. The base rock is composed of Sedimentary Rock and Pyroclastic Sedimentary Rock. The N values of the layer are 11 to 18 above EL.2.0 m and over 40 below El.2.0 m was selected as the bearing layer.

(c) Type of Superstructure

As the bridge lengths are less than 20m in both No.1 and No.2 bridges, RC type girder is selected for these two bridges.

For design of superstructure, a standard design of BINAMARUGA was adopted.

- Peraturan Perencanaan Teknik Jembatan 1992 BINA MARGA (BMS) (Bridge Design Code)

- Design Manual, December 1992 BINA MARGA

(d) Type of Substructure

The substructures consist of two (2) abutments (A1 and A2) and one (1) pier (P1) with PC pile foundation. The bottom elevations of footings are determined as follows.

Item	A1	P1	A2
Bottom elevation of Footing	6.057m	4.658m	5.644m

(e) Design Criteria

The design criteria for the bridge design are stated in "VOLUME Ⅲ DESIGN CRITERIA". For seismic load, following coefficient was adopted as for RC type bridge.

$$Teq = 0.15 \times 1.0 \times 1.0 \times Wr = 0.15Wr$$

(f) Structure Calculation of Sub-Structure

Structure Calculation of the sub-structures are summarized in the following tables.

Tables 4.6.1 and 4.6.2 show the reinforcing bar calculation of the Abutment-1 (A1) and Abutment-2 (A2) respectively. Table 4.6.3 shows the reinforcing bar calculation of the Pier (P1).

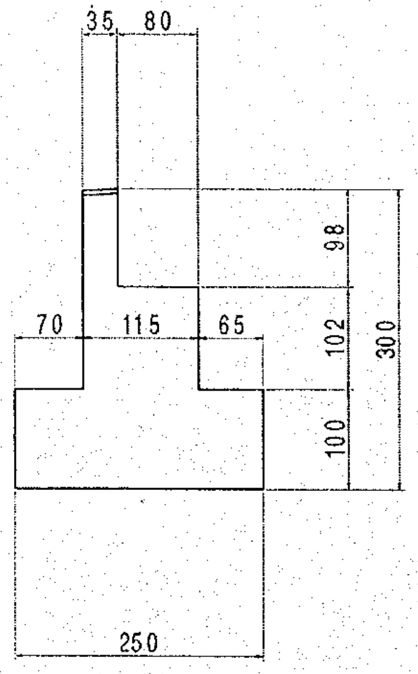
Table 4.6.4 shows stress calculation of the foundations piles of A1, A2 and P1.

Table 4.6.1 Reinforcing Bar Calculation of Abutment 1(A1)

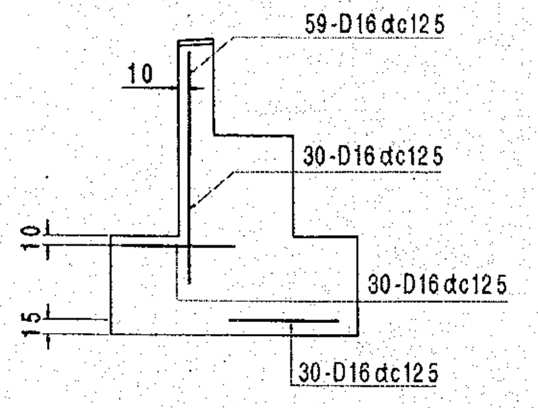
SUBSTRUCTURE No.		
COEFFICIENT OF EARTH PRESSURE	NORMAL	0.308
	SEISMIC	0.433

FIGURE

DIMENSION



BAR ARRANGEMENT



SUBSTRUCTURE DESIGN RECORD ABUTMENT 1

NAME	SIMONGAN BRIDGE

SECTION STRESS CHECK

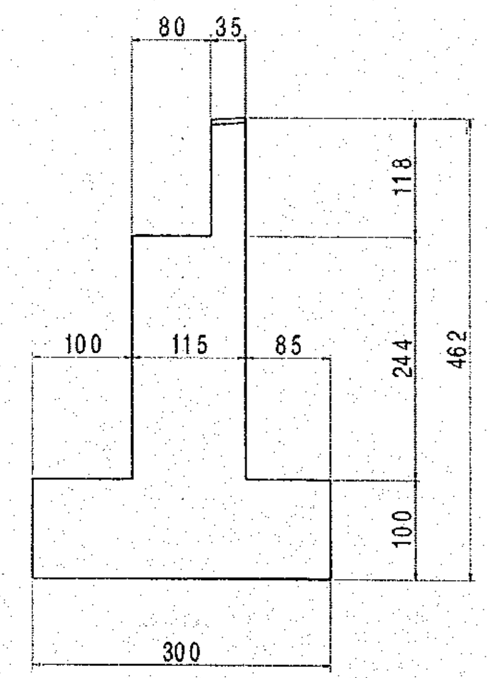
MEMBERS&POSITION	LOAD	FORCES		BAR ARRANGEMENT			REMARKS
PARAPET ① (BACK)	NORMAL	M(tf · m)	19.492	760; 10, 20, 30; FRONT BAC 0.002bd≦As 0.02bd ≧As As= 117.174cm^2 D16-125 ctc	σ_c	38.7＜75	DECIDED BY LEAST MAIN BAR
					σ_s	952＜1800	
		S(tf)	32.254		τ	2.12＜2.2	
PARAPET ② (BACK)	SEISMIC	M(tf · m)					
		S(tf)					
BODY④ (FOOT)	NORMAL	M(tf · m)	15.125	As, As′; 760; 10, 100, 110; FRONT BAC 0.0015A＜As 0.008A′ ＜As As′ = cm^2 D - ctc As=59.58 cm^2 D 16 - 250 ctc	σ_c	2.30＜75	DECIDED BY LEAST MAIN BAR
		N(tf)	112.995		σ_s	8.20＜1800	
		S(tf)	13.391		τ	0.18＜2.2	
BODY④ (FOOT)	SEISMIC	M(tf · m)	19.467	As, As′; 760; 10, 100, 110; FRONT BAC 0.0015A＜As 0.008A′ ＜As As′ = cm^2 D - ctc As=59.58 cm^2 D 16 - 250 ctc	σ_c	2.26＜112.5	DECIDED BY LEAST MAIN BAR
		N(tf)	69.995		σ_s	7.35＜2700	
		S(tf)	23.553		τ	0.31＜2.6	
FOOTING ⑤ (BACK)	SEISMIC	M(tf · m)	12.613	10, 90, 100; 250 0.002bd≦As 0.02bd ≧As As=59.58 cm^2 D16-250ctc	σ_c	2.89＜112.5	MORE THAN 1/3 OF BELOW BARS DOUBLE PITCH
					σ_s	247.53＜2700	
		S(tf)	——		τ	——＜——	
FOOTING ⑥ (FRONT)	SEISMIC	M(tf · m)	-1.239	85, 100, 15; 250 0.002bd≦As 0.02bd ≧As As=59.58 cm^2 D16-250ctc	σ_c	0.31＜112.5	DECIDED BY STRESS 30-D16 =59.58
					σ_s	25.79＜2700	
		S(tf)	——		τ	——＜——	

Table 4.6.2 Reinforcing Bar Calculation of Abutment 2 (A2)

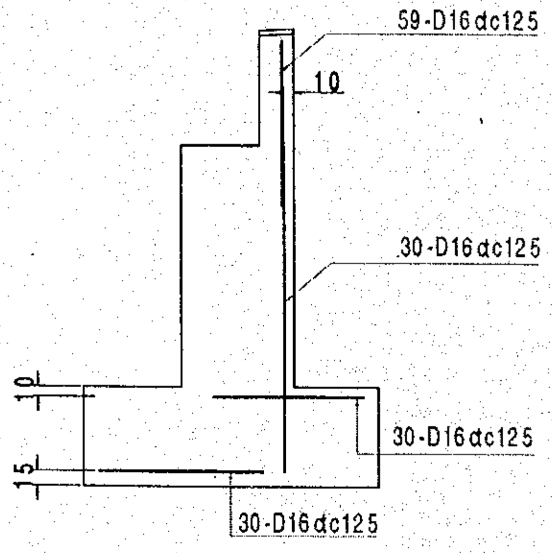
SUBSTRUCTURE No.		
COEFFICIENT OF EARTH PRESSURE	NORMAL	0.308
	SEISMIC	0.433

FIGURE

DIMENSION



BAR ARRANGEMENT



SECTION STRESS CHECK

SUBSTRUCTURE DESIGN RECORD ABUTMENT2

NAME	SIMONGAN BRIDGE

MEMBERS&POSITION	LOAD	FORCES		BAR ARRANGEMENT			REMARKS
PARAPET ① (BACK)	NORMAL	M(tf · m)	26.777	760; 10, 20, 30; FRONT BAC; 0.002bd≦As; 0.02bd ≧As; As= 117.174cm²; D16-125 ctc	σ_c	36.7<75	DECIDED BY LEAST MAIN BAR
					σ_s	1033.9<1800	
		S(tf)	36.107		τ	1.90<2.2	
PARAPET ② (BACK)	SEISMIC	M(tf · m)					
		S(tf)					
BODY④ (FOOT)	NORMAL	M(tf · m)	59.618	760; As; As′; 10, 100, 110; FRONT BAC; 0.0015A<As; 0.008A <As; As′ = cm²; D - ctc; As=59.58 cm²; D 16 - 250 ctc	σ_c	6.18<75	DECIDED BY LEAST MAIN BAR
		N(tf)	181.618		σ_s	31.85<1800	
		S(tf)	36.736		τ	0.46<2.2	
BODY④ (FOOT)	SEISMIC	M(tf · m)	92.345	760; As; As′; 10, 100, 110; FRONT BAC; 0.0015A<As; 0.008A <As; As′ = cm²; D - ctc; As=59.58 cm²; D 16 - 250 ctc	σ_c	14.30<112.5	DECIDED BY LEAST MAIN BAR
		N(tf)	129.618		σ_s	541.71<2700	
		S(tf)	62.603		τ	0.76<2.6	
FOOTING ⑤ (BACK)	SEISMIC	M(tf · m)	26.291	10, 90, 100; 250; 0.002bd≦As; 0.02bd ≧As; As=59.58 cm²; D16-250ctc	σ_c	6.03<112.5	MORE THAN 1/3 OF BELOW BARS DOUBLE PITCH
					σ_s	515.94<2700	
		S(tf)	——		τ	——<——	
FOOTING ⑥ (FRONT)	SEISMIC	M(tf · m)	-67.249	85, 100, 15; 250; 0.002bd≦As; 0.02bd ≧As; As=59.58 cm²; D16-250ctc	σ_c	16.86<112.5	DECIDED BY STRESS
					σ_s	1399.31<2700	30-D16 =59.58
		S(tf)	——		τ	——<——	

Table 4.6.3 Reinforcing Bar Calculation of Pier (P1)

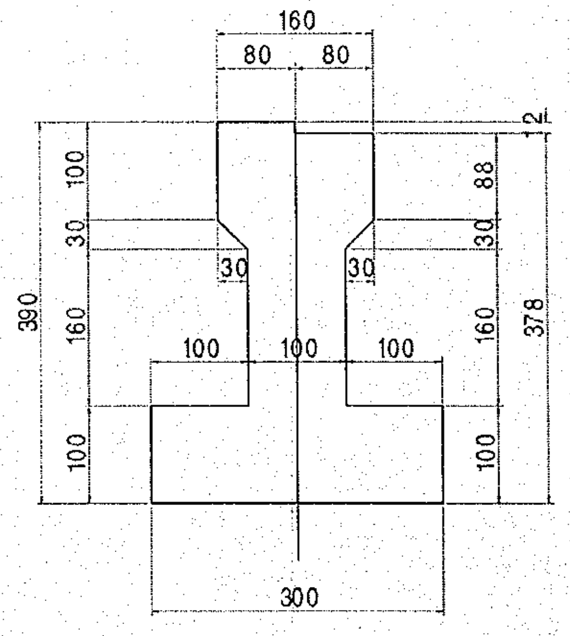
SUBSTRUCTURE DESIGN RECORD PIER

NAME	SIMONGAN PIER

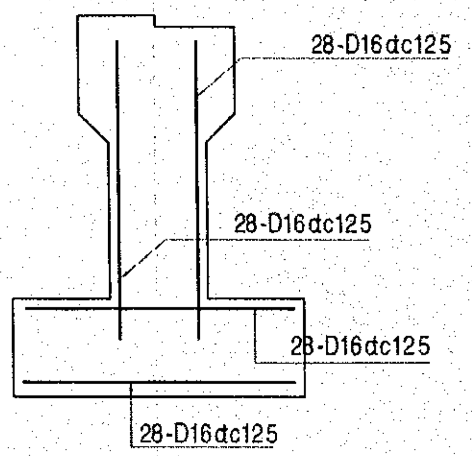
SUBSTRUCTURE No.		
COEFFICIENT OF EARTH PRESSURE	NORMAL	0.308
	SEISMIC	0.433

FIGURE

DIMENSION



BAR ARRANGEMENT



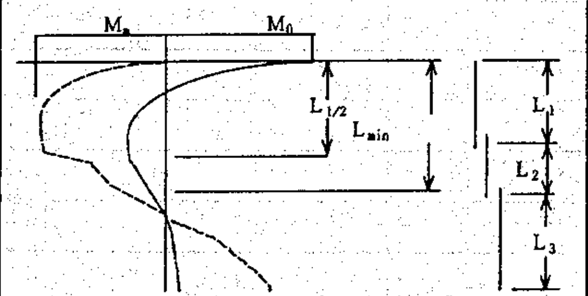
SECTION STRESS CHECK

MEMBER&POSITION	LOAD	FORCES		BAR ARRANGEMENT			REMARKS
BODY (FOOT)	LONGITUDE NORMAL	M(tf · m)	1.181	700, As, As', 10, 80, 10, 100, FRONT, BAC; 0.0015A<As; 0.008A <As	σ_c	3.52<75	DECIDED BY LEAST MAIN BAR
		N(tf)	246		σ_s	-50.24<1800	
		S(tf)	0	As= 55.608cm²; D16-250 ctc	τ	0<2.2	
BODY (FOOT)	LONGITUDE SEISMIC	M(tf · m)	70	700, As, As', 10, 80, 10, 100, FRONT, BAC; 0.0015A<As; 0.008A <As	σ_c	11.89<112.5	DECIDED BY LEAST MAIN BAR
		N(tf)	170		σ_s	204.58<2700	
		S(tf)	27	As= 55.608cm²; D16-250 ctc	τ	0.43<3.3	
BODY (FOOT)	TRANSVERSAL NORMAL	M(tf · m)	0	700, As, As', 10, 80, 10, 100, FRONT, BAC; 0.0015A<As; 0.008A <As	σ_c	3.50<75	DECIDED BY LEAST MAIN BAR
		N(tf)	246		σ_s	-52.45<1800	
		S(tf)	0	As= 7.944cm²; D16-250 ctc	τ	0<2.2	
BODY (FOOT)	TRANSVERSAL SEISMIC	M(tf · m)	65	700, As, As', 10, 80, 10, FRONT, BAC; 0.0015A<As; 0.008A <As	σ_c	3.21<112.5	DECIDED BY LEAST MAIN BAR
		N(tf)	171		σ_s	-25.04<2700	
		S(tf)	27	As= 7.944cm²; D16-250 ctc	τ	0.39<3.3	
FOOTING (BACK)	SEISMIC	M(tf · m)	-54.636	10, 90, 100, 300; 0.002bd≦As; 0.02bd ≧As; As' = cm²; D16 - 250 ctc	σ_c	14.79<112.5	MORE THAN 1/3 OF BELOW BARS DOUBLE PITCH
					σ_s	1218.45<2700	
		S(tf)	0	As=55.608cm²; D 16 - 250 ctc	τ	————	
FOOTING (FRONT)	SEISMIC	M(tf · m)	-56.866	85, 100, 15, 250; 0.002bd≦As; 0.02bd ≧As; As' = cm²; D 16 - 250 ctc	σ_c	15.39<112.5	DECIDED BY LEAST MAIN BAR
					σ_s	1268.18<2700	
		S(tf)	——	As=55.608 cm²; D 16 - 250 ctc	τ		

Table 4.6.4 Pile Stress Calculation of Abutments and A Pier

SUBSTRUCTURE DESIGN RECORD PC PILES

		SUBSTRUCTURE No.		APPROACH BRIDGE A1	APPROACH BRIDGE PIER	APPROACH BRIDGE A2
CONDITION		PILES TYPE		PC DRIVEN PILES	PC DRIVEN PILES	PC DRIVEN PILES
		DIAMETERS	mm	φ 500	φ 500	φ 500
		LENGTH OF PILES		5.5	4.0	5.0
		NUMBER OF PILES		6	8	8*
		JOIN METHOD		TYPE A · TYPE B	TYPE A · TYPE B	TYPE A · TYPE B
		FACTOR OF LIQUEFACTION D_E		0 · 1/3 · 2/3 · 1	0 · 1/3 · 2/3 · 1	0 · 1/3 · 2/3 · 1
		SECTION OF LIQUEFACTION m (WEAK CLAY SECTION)		—	—	—
		HORIZONTAL REFLECT FACTOR OF GROUND Kh(1 / β SECTION)		—	—	—
		DECIDED FACTOR OF PILES NUMBER		B · D · S	B · D · S	B · D · S
RESULTS	FORCE	DECIDED CASE		NORMAL CASE	NORMAL CASE	NORMAL CASE
		BEND MOMENT	tf·m	2.538	(N) 0.064	3.702
		VERTICAL FORCE	t	39.656	(N) 43<72	37.688
		HORIZONTAL FORCE	t	0	(N) 0	0
	STRESS	σ_c, σ_{ca}	kgf/cm²	136.9<170	(N) 117.1<170	146.1<170
		σ_p, σ_{pa}	kgf/cm²	8047.2<8700	(S) 8112.7<8700	8100.9<8700.0
		τ, τ_{ta}	kgf/cm²	—	—	—
			M_0	2.538	0	3.702
			M_m	—	0	—
			$M_{1/2}$	1.269	0	1.851
			$L_{1/2}$	2.873	0	2.665
			L_{min}	4.025	1.356	5
			L_1	4.025	1.356	5
			L_2			
			L_3			
			D1 or t1	φ 500	φ 500	φ 500
			D2 or t2			
			D3 or t3			
			A_{min}			
REMARKS						



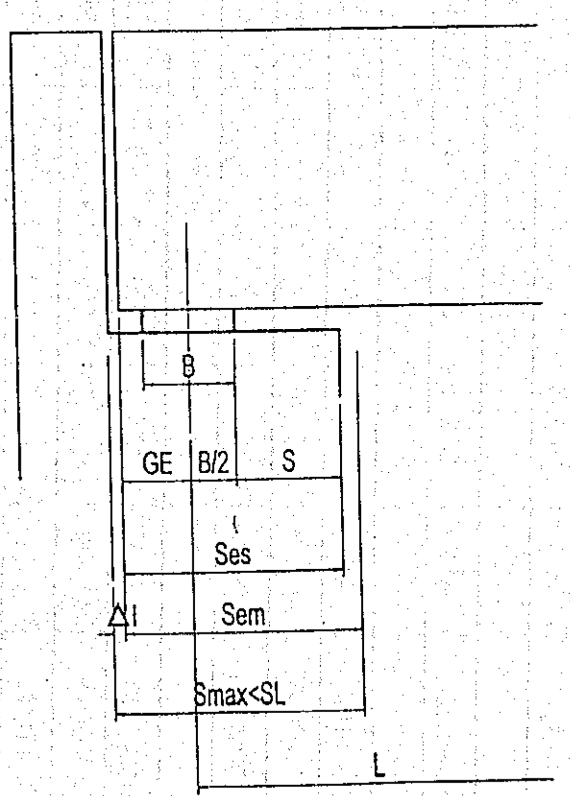
Note D1～D3:Bar arrangement of cast-in-place piles
t1～ t3:The thickness of steel tube, express by the types of piles

REMARK: (N) mains Normal case, (S) mains Seismic case, * mains corrected.

CALCULATION OF HORIZONTAL SEISMIC COEFFICIENT

NAME		LONGITUDE WIDTH	LATERAL WIDTH	HEIGHT	YOUNG'S MODULUS E	WEIGHT OF SUPERSTRUCTURE Wu	WEIGHT OF SUBSTRUCTURE Wp	SECTION MODULUS I y	PERIOD T	BASE SHEAR COEFFICIENT C	STRUCTURE TYPE FACTOR S	COEFFICIENT OF SEISMIC Kh	IMPORTANCE FACTOR I	Kh· I
		m	m	m	t/m²	t	t	m⁴	s					
ASIN No.1	A1	1.4	10.2	5	2.6E+06	202.8	178.5	2.3324	0.090	0.15	1.3	0.195	1	0.20
	A2	1.4	10.2	5	2.6E+06	202.8	178.5	2.3324	0.090	0.15	1.3	0.195	1	0.20
ASIN No.2	A1	1.4	8.85	5	2.6E+06	165.7	154.875	2.0237	0.088	0.15	1.3	0.195	1	0.20
	A2	1.4	8.85	5	2.6E+06	166.7	154.875	2.0237	0.088	0.15	1.3	0.195	1	0.20
ASIN PUMP	A1	1.4	5.6	5	2.6E+06	101.9	98	1.2805	0.087	0.15	1.3	0.195	1	0.20
	A2	1.4	5.6	5	2.6E+06	101.9	98	1.2805	0.087	0.15	1.3	0.195	1	0.20
DAM	A1				2.6E+06			0	#DIV/0!	0.15	1.0	0.150	1	0.15
	P1				2.6E+06			0	#DIV/0!	0.15	1.0	0.150	1	0.15
	P2				2.6E+06			0	#DIV/0!	0.15	1.0	0.150	1	0.15
	P3				2.6E+06			0	#DIV/0!	0.15	1.0	0.150	1	0.15
	A2				2.6E+06			0	#DIV/0!	0.15	1.0	0.150	1	0.15
SIMONGAN	A1	1.15	7	2	2.6E+06			0.8872	0.000	0.15	1.0	0.150	1	0.15
	P1	1	7	3	2.6E+06			0.5833	0.000	0.15	1.0	0.150	1	0.15
	A2	1.15	7	3.6	2.6E+06			0.8872	0.000	0.15	1.0	0.150	1	0.15
RAIL BRIDGE	A1				2.6E+06			0	#DIV/0!	0.15	1.2	0.180	1	0.18

Calculation of SL



CALCULATION OF SL

GE	GE = 300 15 < l < 20
Sem	Sem=70+0.5L
S	S = 20 + 0.5L

		L m	GE mm	Δl mm	B mm	S mm	Ses	Sem mm	Smax mm	SL mm
ASIN No1	A1,2	21.2	300	30	280	306	746	806	836	900
ASIN No2	A1,2	21.2	300	30	280	306	746	806	836	900
ASIN PUNP	A1,2	21.2	300	30	280	306	746	806	836	900
DUM										
DUM ISLAND	A1,2	16.4	300	30	300	282	732	782	812	850
	P1~3	16.4	300	30	300	282	732	782	1594	1600
SIMONGAN	RC7.0	6.4	300	30	200	232	632	732	762	800
	PC21.0	20.4	300	30	280	302	742	802	1634	1700
SIMONGAN SIDE	A1	8.4	300	30	300	242	692	742	772	800
	P11	8.4	300	30	300	242	692	742	1514	
	P12	12.6	300	30	300	263	713	763	1556	1600
	A2	12.6	300	30	300	263	713	763	793	800