4.5.2 Sediment Flush Gate

1. DESIGN ITEM

1.1 Design Condition

- Type of Gate		Steel Roller Gate
- Number of Gate	2	Gates
- Clear span	5.500	m
- Height of Gate	4.350	m
- Design Water Depth		
Upstream	4.700	m
Downstream	0.000	m
- Water Depth for Operation		
• Lifting Time		
Upstream	4.700	m
Downstream	0.000	m
• Lowering time		
Upstream	1.000	m
Downstream	0.000	m
- Depth Sedimentation	1.000	m
- Gate Floor Level	EL 1.500	m
- Method for Watertightness		Fore front 3 faces watertight
- Hoisting System		Winch by 1 motor and 2 drum
- Hoisting Speed		About 0.3 m/min
- Total Head	7.500	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

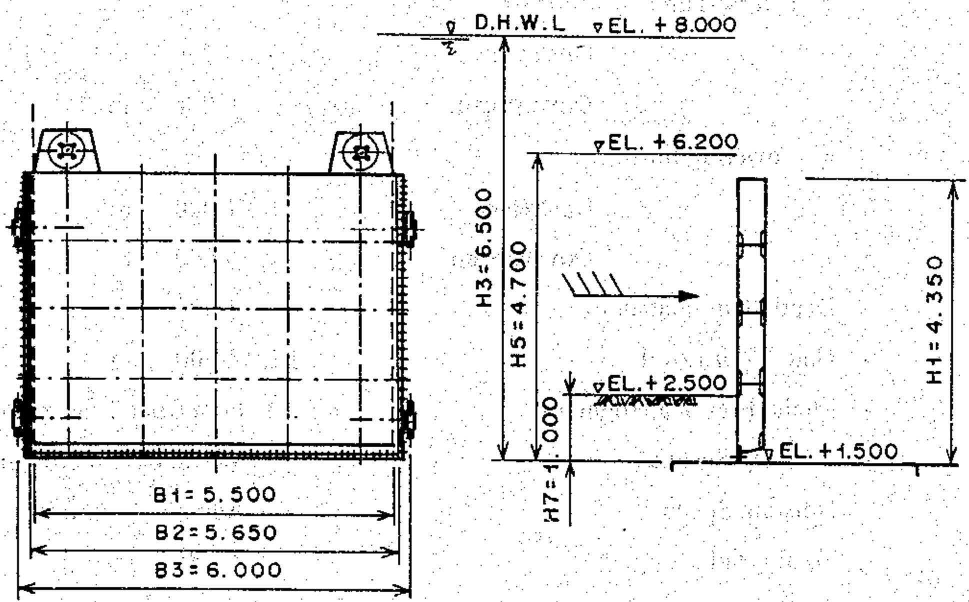
- Major Material of Gate

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

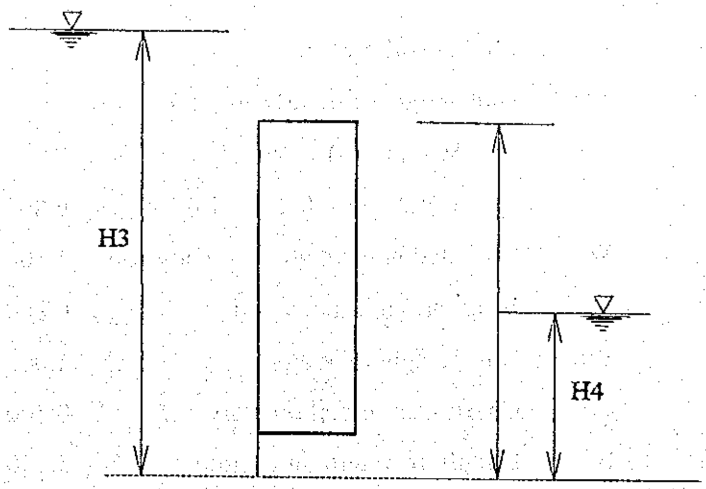
2. BASIC DIMENSIONS



B1	:	Clear Span	5.500	m
B2	:	Length of Watertight Area	5.650	m
B3	:	Span Between Rollers	6.000	m
H1	:	Height of Gate	4.350	m
H2	:	Height of Watertight Area	4.350	m
H3	:	Design Water Depth Upstream	4.700	m
H4	:	Design Water Depth Downstream	0.000	m
H5	:	Water Depth for Operation (Upstream)	4.700	m
H7	:	Water Depth for Operation (Downstream)	1.000	m

3. ACTING LOADS

3.1 Loads in Normal Time



(1) Static Water Pressure

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

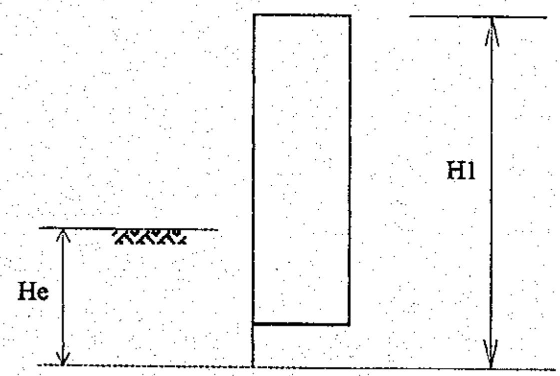
$$= \frac{1}{2} \times \{4.700^2 - (4.700 - 4.350)^2\} \times 5.650 \times 1.00$$

$$= 62.058 \quad tf$$

Where,

H2	:	Height of Watertight Portion	4.350	m
H3	:	Design Water Depth (Upstream)	4.700	m
B2	:	Width of Watertight Portion	5.650	m
γ	:	Specific Gravity of Water	1.00	

(2) Pressure of Muddy Soil (Sediment Deposit)



Technical Manual for Dam and Weir is used for calculation of muddy soil pressure.

$$Ps = \frac{1}{2} \cdot W1 \cdot He^2 \cdot Ce \cdot B2$$

Where,

Ps	:	Muddy soil pressure		
W1	:	Unit weight of muddy soil in water		tf/m^3
		$= W - (1 - v) \cdot Wo$		
		$= 1.500 - (1 - 0.30) \times 1.00 =$	0.800	
W	:	Saturated unit weight of muddy soil	1.500	tf/m^3
v	:	Void ratio of muddy soil	0.300	
Wo	:	Unit Weight of Water	1.000	tf/m^3
Ce	:	Coefficient of soil pressure	0.500	
B2	:	Length of watertight portion	5.650	m
He	:	Height of muddy soil	1.000	m

$$= \frac{1}{2} \times 0.800 \times 1.000^2 \times 0.5 \times 5.650$$

$$= 1.130 \quad tf$$

(3) Design Loads in Normal Time

$$Pa = Pw + Ps$$

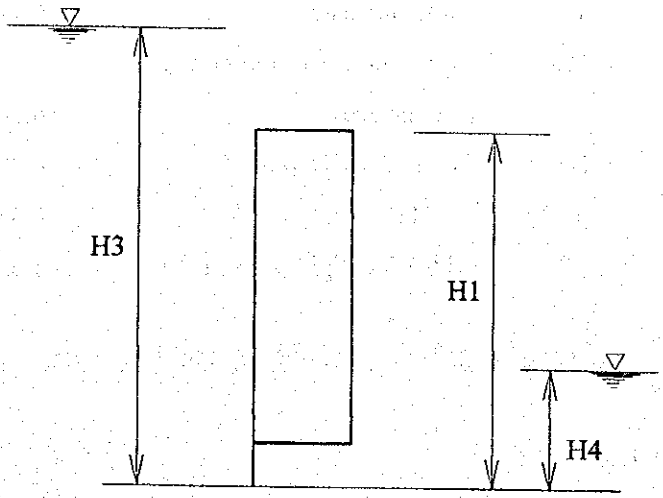
Where,

Pw	:	Water pressure	62.058	tf
Ps	:	Muddy soil pressure	1.130	tf

$$= 62.058 + 1.130$$

$$= 63.188 \quad tf$$

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	6.500	m

$$= \frac{0.12 \times 1.0}{2 \cdot \pi} \times (9.8 \times 6.500)^{0.5}$$

$= 0.152$ tf

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

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

$$= \frac{1}{2} \times (4.700 + 0.152)^2 - (4.700 + 0.152 - 4.350)^2\} \times 5.650$$

$= 65.794$ 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	6.500 m
H3	:	Design Water Depth	4.700 m
h	:	H3 – H2	0.350 m
B2	:	Width Watertight portion	5.650 m

$$= \frac{7}{12} \times 0.12 \times 6.500^{0.5} \times (4.700^{1.5} - 0.350^{1.5}) \times 5.650$$

$$= 10.065 \quad \text{tf}$$

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

$$Wge = k \cdot Wg$$

Where,

Wg	:	Self weight of gate	10.500 tf

$$= 0.12 \times 10.5$$

$$= 1.260 \quad \text{tf}$$

(5) Total Force in Seismic Time (Pe)

$$Pe = Peg + Pdo + Wge + Ps$$

$$= 65.794 + 10.065 + 1.260 + 1.130$$

$$= 78.249 \quad \text{tf}$$

(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:

$$\frac{Pe}{1.5} = 78.249 / 1.5 = 52.166 < Pa = 63.188$$

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

4. LOADS FOR GATE OPERATION

4.1 Lifting Time

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

$$= \frac{1}{2} \times \{ 4.700^2 - (4.700 - 4.350)^2 \} \times 5.650 \times 1.00 = 62.058 \text{ tf}$$

Where,

H2	: Height of watertight portion	4.350	m
H5	: Water depth for Operation (Upstream)	4.700	m
B2	: Width of watertight portion	5.650	m
γ	: Specific gravity of water	1.00	

4.2 Lowering Time

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

$$= \frac{1}{2} \times 1.000^2 \times 5.650 \times 1.00 = 2.825 \text{ tf}$$

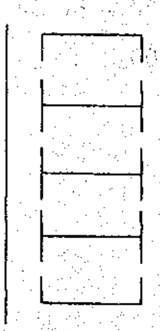
Where,

H7	: Water depth for Operation (Upstream)	1.000	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)	Soil Pressure q(tf/m^2)	Total Q(tf/m^2)
1		p(1) = 0.350	p(1) = 0.000	Q(1) = 0.350
	L(1) = 1.200			
2		p(2) = 1.550	p(2) = 0.000	Q(2) = 1.550
	L(2) = 1.100			
3		p(3) = 2.650	p(3) = 0.000	Q(3) = 2.650
	L(3) = 1.000			
4		p(4) = 3.650	p(4) = 0.000	Q(4) = 3.650
	L(4) = 0.900			
5		p(5) = 4.550	p(5) = 0.340	Q(1) = 4.890
	L(5) = 0.150			
At the gate floor		4.700	0.400	5.100

(2) Allotted Load

R (1) =

$p\ (B) = \{p\,(2) + 2 \cdot p\,(1)\} \cdot L(1)/6$		=	0.450	
	R(1)	=	0.450	tf/m

R (2) =

$p\ (U) = \{p\,(1) + 2 \cdot p\,(2)\} \cdot L(1)/6$		=	0.690	
$p\ (B) = \{p\,(3) + 2 \cdot p\,(2)\} \cdot L(2)/6$		=	1.054	
	R(2)	=	1.744tf/m	

R (3) =

$p\ (U) = \{p\,(2) + 2 \cdot p\,(3)\} \cdot L(2)/6$		=	1.256	
$p\ (B) = \{p\,(4) + 2 \cdot p\,(3)\} \cdot L(3)/6$		=	1.492	
	R(3)	=	2.748	tf/m

R (4) =

$p\ (U) = \{p\,(3) + 2 \cdot p\,(4)\} \cdot L(3)/6$		=	1.658	
$p\ (B) = L1 \cdot [\,3 \cdot L(4) \cdot \{p\,(4) + p\,(6)\} - L1 \cdot \{p\,(4) + 2 \cdot p\,(6)\}] / 6 / L\,(4)$		=	1.719	
$q\ (T) = q\,(6)^2 \cdot \{q(6) - 3 \cdot L(5)\} / 6 / L\,(4)$		=	-0.001	
	R(4)	=	3.376	tf/m

R (5) =

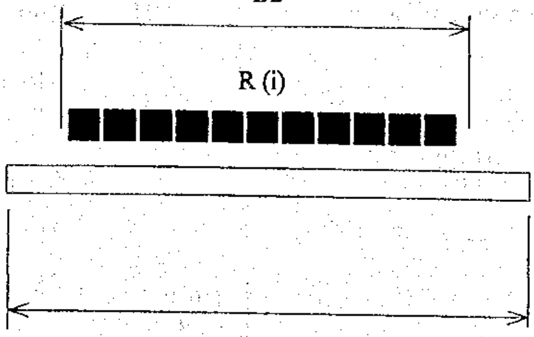
$p\ (T) = \{p\,(4) + 2 \cdot p\,(6)\} \cdot L1^2 / 6 / L\,(4)$		=	2.664	
$q\ (T) = p\,(6)^2 \cdot \{3 \cdot L1 - p(6)\} / 6 / L\,(4)$		=	0.081	
	R(5)	=	2.745	tf/m
	Σ		11.063	tf/m

$L1 = L(4) + L(5)$

$= 0.900 + 0.150 = 1.050$ m

Distance between surface of deposit and next beam = 0.000 m

5.2. Sectional Force of Main Beam



Where,

B2 : Distance ot watertight rubbers 5.650 m

B3 : Span length 6.000 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

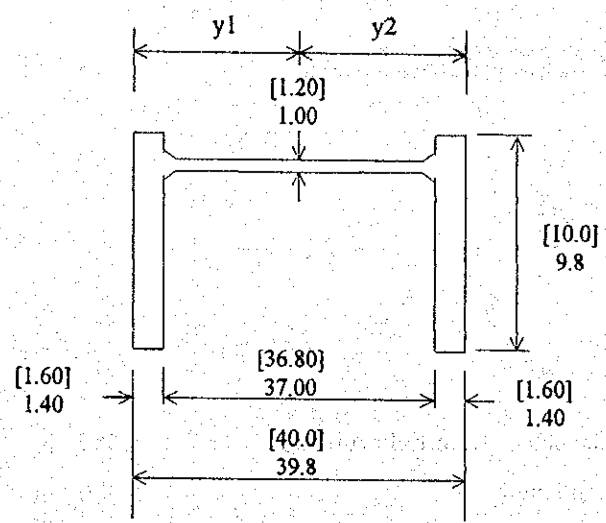
$$S(i) = \frac{1}{2} \cdot R(i) \cdot B2 \ \text{tf}$$

No	R (i) (tf/m)	M (i) (tf/m)	S (i) (tf)
1	0.450	2.018	1.271
2	1.744	7.821	4.927
3	2.748	12.324	7.763
4	3.376	15.140	9.537
5	2.745	12.310	7.755

5.3. Stress of Main Beam

(1) Beam – No.1

(a) Sectional Dimension of Beam BH 400 x 100 x 12.0 x 16.0



Moment of Area

I = 14,326 cm^4

Section Modulus

Zt = 720 cm^3

Zc = 720 cm^3

Sectional area of web

Aw = 37.0 cm^2

Sectional area of flange

Ac = 13.7 cm^2

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 1.40 x 9.8	13.7	0.70	9.59	6.71	2.24
t 1.00 x 37.0	37.0	19.90	736.30	14,652.37	4,221.08
t 1.40 x 9.8	13.7	39.10	535.67	20,944.70	2.24
Total	64.4	-----	1,281.56	39,829.34	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{1,281.58}{64.4} = 19.90 \text{ cm}$$

$$y2 = 39.8 - 19.90 = 19.90 \text{ cm}$$

Moment of area (I)

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

$$= 39,829.34 - 64.4 \times 19.90^2 = 14,326 \text{ cm}^4$$

Section Modulus (Z)

$$Zt = \frac{I}{y2} = \frac{14,326}{19.90} = 720 \text{ cm}^3$$

$$Zc = \frac{I}{y1} = \frac{14,326}{19.90} = 720 \text{ cm}^3$$

(b) Bearing Stress

$$\sigma c = \frac{M(i)}{Zc} = 2.018\times10^5 / 720 = 280 \ \text{kgf/cm}^2 < \sigma \text{ac}$$

Allowable bearing stress

σ ac = fa $\cdot$ σ ac = 1.00 x 1,200 = 1,200 kgf/cm^2

(c) Shearing Stress

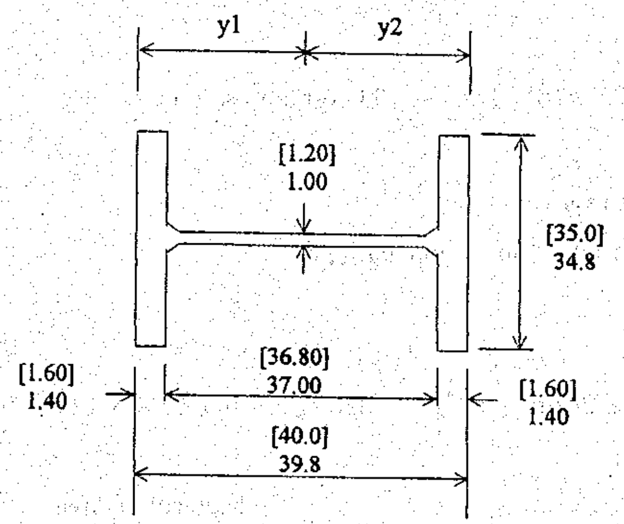
$$\tau = \frac{S(i)}{Aw} = 1.271\times10^3 / 37.0 = 34 \ \text{kgf/cm}^2 < \tau \text{ a}$$

Allowable shearing stress

τ a = fa $\cdot$ τ a = 1.00 x 700 = 700 kgf/cm^2

(2) Beam – No.2

(a) Sectional Dimension of Beam BH 400 x 350 x 12.0 x 16.0



Moment of Area

I = 40,143 cm^4

Section Modulus

Zt = 2,017 cm^3

Zc = 2,017 cm^3

Sectional area of web

Aw = 37.0 cm^2

Sectional area of flange

Ac = 48.7 cm^2

Member cm	**A** cm^2	**y** cm	**A · y** cm^3	**A · y^2** cm^4	**I o** cm^4
t 1.40 x 34.8	48.7	0.70	34.09	23.86	7.96
t 1.00 x 37.0	37.0	19.90	736.30	14,652.37	4,221.08
t 1.40 x 34.8	48.7	39.10	1,904.17	74,453.04	7.96
Total	134.4	——	2,674.56	93,366.27	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{2{,}674.56}{134.4} = 19.90 \ \text{cm}$$

$y2 = 39.8 - 19.90 = 19.90$ cm

Moment of area (I)

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

$$= 93{,}366.27 - 134.4 \times 19.90^2 = 40.143 \ cm^4$$

Section Modulus (Z)

$$Zt = \frac{I}{y2} = \frac{40{,}143}{19.90} = 2{,}017 \ cm^3$$

$$Zc = \frac{I}{y1} = \frac{40{,}143}{19.90} = 2{,}017 \ cm^3$$

(b) Bearing Stress

$$\sigma c = \frac{M(i)}{Zc} = 7.821 \times 10^5 / 2{,}017 = 388 \ kgf/cm^2 < \sigma ac$$

Allowable bearing stress

$$\sigma ac = fa \cdot \sigma ac = 1.00 \times 1{,}200 = 1{,}200 \ kgf/cm^2$$

(c) Shearing Stress

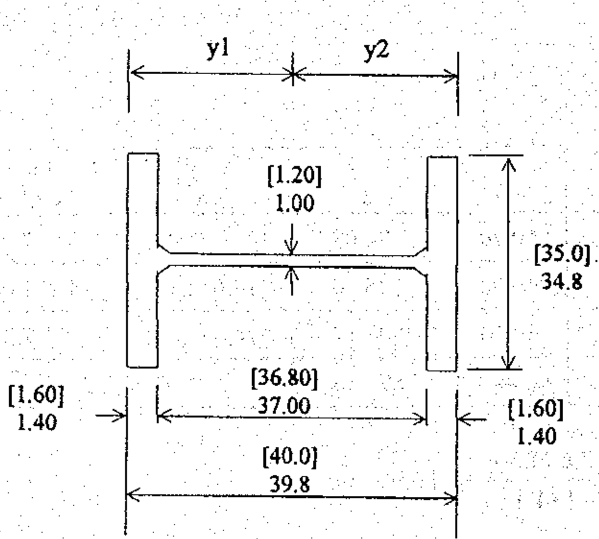
$$\tau = \frac{S(i)}{Aw} = 4.927 \times 10^3 / 37.0 = 133 \ kgf/cm^2 < \tau a$$

Allowable shearing stress

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

(3) Beam – No.3

(a) Sectional Dimension of Beam BH 400 x 350 x 12.0 x 16.0



Moment of Area

$I = 40{,}143 \ cm^4$

Section Modulus

$Zt = 2{,}017 \ cm^3$

$Zc = 2{,}017 \ cm^3$

Sectional area of web

$Aw = 37.0 \ cm^2$

Sectional area of flange

$Ac = 48.7 \ cm^2$

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 1.40 x 34.8	48.7	0.70	34.09	23.86	7.96
t 1.00 x 37.0	37.0	19.90	736.30	14,652.37	4,221.08
t 1.40 x 34.8	48.7	39.10	1,904.17	74,453.04	7.96
Total	134.4	----	2,674.56	93,366.27	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{2,674.56}{134.4} = 19.90 \text{ cm}$$

$$y2 = 39.8 - 19.90 = 19.90 \text{ cm}$$

Moment of area (I)

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

$$= 93,366.27 - 134.4 \times 19.90^2 = 40.143 \text{ cm}^4$$

Section Modulus (Z)

$$Zt = \frac{I}{y2} = \frac{40,143}{19.90} = 2,017 \text{ cm}^3$$

$$Zc = \frac{I}{y1} = \frac{40,143}{19.90} = 2,017 \text{ cm}^3$$

(b) Bearing Stress

$$\sigma c = \frac{M(i)}{Zc} = 12.324 \times 10^5 / 2,017 = 611 \text{ kgf/cm}^2 < \sigma ac$$

Allowable bearing stress

$$\sigma ac = fa \cdot \sigma ac = 1.00 \times 1,200 = 1,200 \text{ kgf/cm}^2$$

(c) Shearing Stress

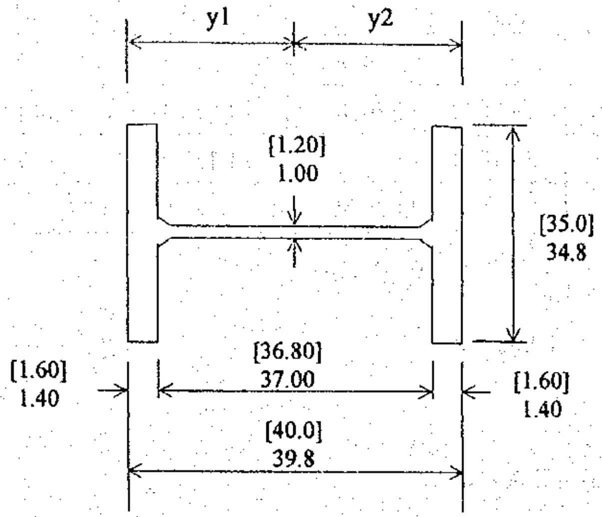
$$\tau = \frac{S(i)}{Aw} = 7.763 \times 10^3 / 37.0 = 210 \text{ kgf/cm}^2 < \tau a$$

Allowable shearing stress

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

(4) Beam – No.4

(a) Sectional Dimension of Beam BH 400 x 350 x 12.0 x 16.0



Moment of Area

I = 40,143 cm^4

Section Modulus

Zt = 2,017 cm^3

Zc = 2,017 cm^3

Sectional area of web

Aw = 37.0 cm^2

Sectional area of flange

Ac = 48.7 cm^2

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 1.40 x 34.8	48.7	0.70	34.09	23.86	7.96
t 1.00 x 37.0	37.0	19.90	736.30	14,652.37	4,221.08
t 1.40 x 34.8	48.7	39.10	1,904.17	74,453.04	7.96
Total	134.4	-----	2,674.56	93,366.27	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{2,674.56}{134.4} = 19.90 \text{ cm}$$

$$y2 = 39.8 - 19.90 = 19.90 \text{ cm}$$

Moment of area (I)

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

$$= 93,366.27 - 134.4 \times 19.90^2 = 40.143 \text{ cm}^4$$

Section Modulus (Z)

$$Zt = \frac{I}{y2} = \frac{40,143}{19.90} = 2,017 \text{ cm}^3$$

$$Zc = \frac{I}{y1} = \frac{40,143}{19.90} = 2,017 \text{ cm}^3$$

(b) Bearing Stress

$$\sigma c = \frac{M(i)}{Zc} = 15.140 \times 10^5 / 2,017 = 751 \text{ kgf/cm}^2 < \sigma\,ac$$

Allowable bearing stress

σ ac = fa · σ ac = 1.00 x 1,200 = 1,200 kgf/cm^2

(c) Shearing Stress

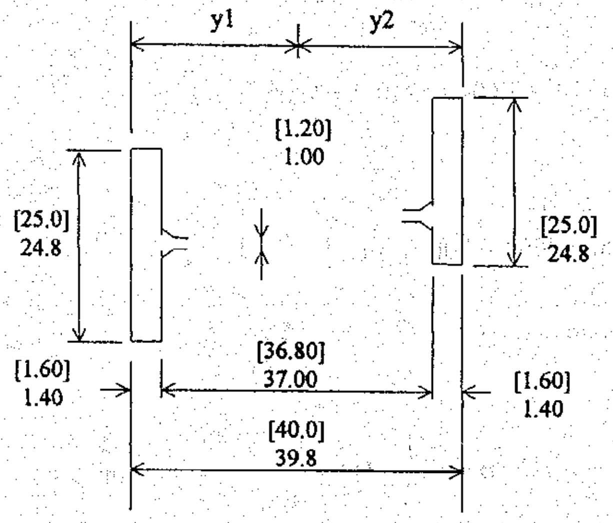
$$\tau = \frac{S(i)}{Aw} = 9.537\times10^3 / 37.0 = 258 \ \text{kgf/cm}^2 < \tau a$$

Allowable shearing stress

τ a = fa · τ a = 1.00 x 700 = 700 kgf/cm^2

(5) Beam – No.5

(a) Sectional Dimension of Beam BH 400 x 250 x 12.0 x 16.0



Moment of Area

I = 29,816 cm^4

Section Modulus

Zt = 1,498 cm^3

Zc = 1,498 cm^3

Sectional area of web

Aw = 37.0 cm^2

Sectional area of flange

Ac = 34.7 cm^2

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 1.40 x 24.8	34.7	0.70	24.29	17.00	5.67
t 1.00 x 37.0	37.0	19.90	736.30	14,652.37	4,221.08
t 1.40 x 24.8	34.7	39.10	1,356.77	53,049.71	5.67
Total	106.4	-----	2,117.36	71,951.50	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{2{,}117.36}{106.4} = 19.90 \ \text{cm}$$

$$y2 = 39.8 - 19.90 = 19.90 \ \text{cm}$$

Moment of area (I)

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

$= \quad 71{,}951.50 - 106.4 \times 19.90^2 = 29{,}816 \ \text{cm}^4$

Section Modulus (Z)

$$Zt = \frac{I}{y2} = \frac{29{,}816}{19.90} = 1{,}498 \ \text{cm}^3$$

$$Zc = \frac{I}{y1} = \frac{29{,}816}{19.90} = 1{,}498 \ \text{cm}^3$$

(b) Bearing Stress

$$\sigma c = \frac{M(i)}{Zc} = 12.310 \times 10^5 / 1{,}498 = 822 \ \text{kgf/cm}^2 < \sigma \text{ac}$$

Allowable bearing stress

σ ac = fa $\cdot$ σ ac = 1.00 x 1,200 = 1,200 kgf/cm^2

(c) Shearing Stress

$$\tau = \frac{S(i)}{Aw} = 7.755 \times 10^3 / 37.0 = 210 \ \text{kgf/cm}^2 < \tau \text{a}$$

Allowable shearing stress

τ a = fa $\cdot$ τ a = 1.00 x 700 = 700 kgf/cm^2

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	565	cm
	B3	:	Span	600	cm
	E	:	Modulus of elasticity	2.1 x 10^6	kgf/cm^2
	I	:	Moment of section		vm^4

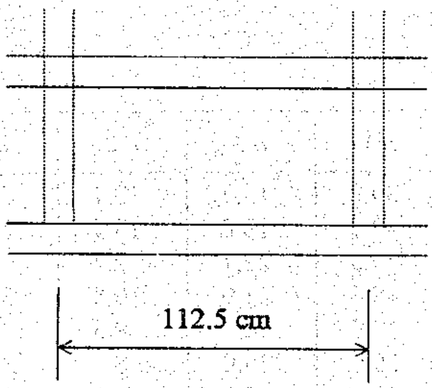
Deformation Rate

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

Beam No.	R(i) Kgf/cm)	I (cm^4)	δ (i) (cm)	$\frac{1}{Rd(i)}$	Allowable Deformation
1	4.50	14,326	0.251	1/2,390	$\frac{1}{800}$
2	17.44	40,143	0.348	1/1,724	$\frac{1}{800}$
3	27.48	40,143	0.548	1/1,095	$\frac{1}{800}$
4	33.76	40,143	0.673	1/892	$\frac{1}{800}$
5	27.45	29,816	0.737	1/814	$\frac{1}{800}$

6. SKIN PLATE

6.1 Beam Allocation and Water Pressure



Water Pressure (kgf/cm^2)	Block No.	Length of Block (cm)	Span Length (cm)
0.2628 Average value between flanges of Main Beam →	1	78.5	78.5
0.3423 Average value between flanges of Main Beam →	2	70.0	70.0
0.3471 Average value between flanges of Main Beam →	3	70.0	70.0
0.3621 Average value between flanges of Main Beam →	4	67.5	67.5

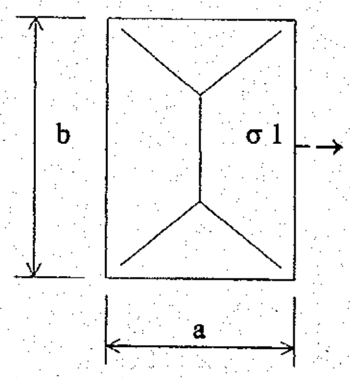
6.2 Stress on Skin Plate

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	1.20	cm
ε	:	Margin of thickness of plate	0.20	cm



k Value

b / a	σ1	σ2	σ3	σ4
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 : $\sigma a = fa \cdot \sigma a = 1.00 \times 1{,}200 = 1{,}200\ kgf/cm^2$

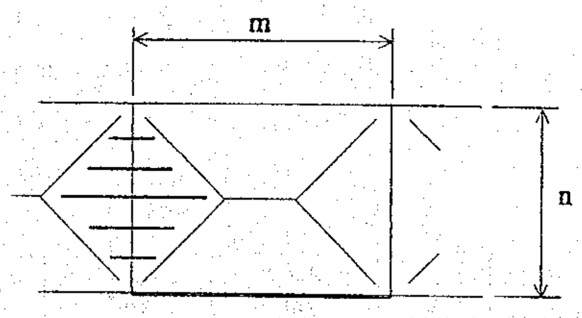
Block No.	a cm	b cm	b / a	k	L cm	p kgf/cm^2	t - ε cm	σ kgf/cm^2
1	78.5	112.5	1.42	44.11	78.5	Average value 0.0833	1.00	226
2	70.0	112.5	1.61	46.74	70.0	Average value 0.1925	1.00	441
3	70.0	112.5	1.61	46.74	70.0	Average value 0.2975	1.00	681
4	67.5	112.5	1.67	47.43	67.5	Average value 0.4143	1.00	895

Based on the calculation result, the thickness of 12.0 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.

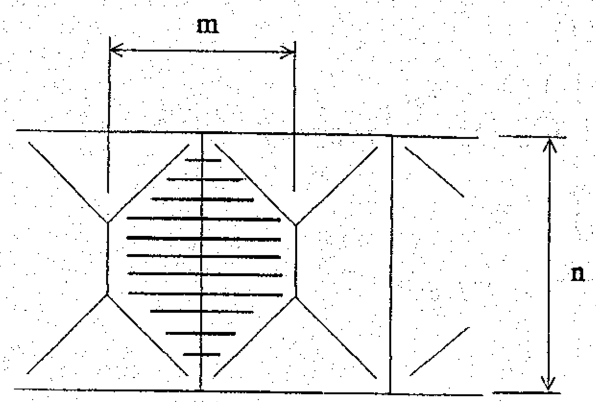
In case $m \geq n$



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 $m < n$

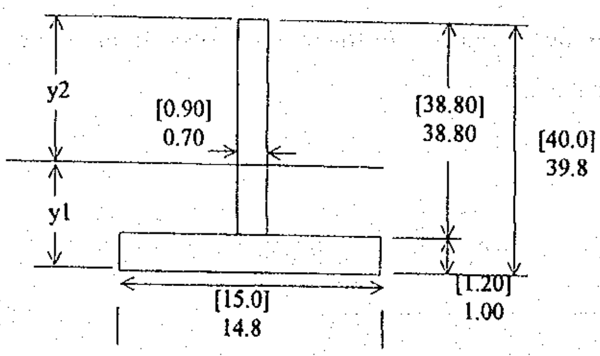


Bearing Moment

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

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

(1) Sectional Dimensions of Beam PL 400 x 150 x 9 x 12



Moment of Section
I = 7,202 cm^4

Section Modulus
Z = 273 cm^3

Sectional area of web
Aw = 27.2 cm^2

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 0.70 x 38.8	27.2	20.40	554.88	11,319.55	3,407.31
t 1.00 x 14.8	0.0	0.50	7.40	3.70	1.23
Total	42.0	-----	562.28	14,731.79	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{562}{42.0} = 13.39 \text{ cm}$$

$$y2 = 39.8 - 13.39 = 26.41 \text{ cm}$$

Moment of Section (I)

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

$$= 14{,}732 - 42.0 \times 13.39^2 = 7{,}202 \text{ cm}^4$$

Section Modulus (Z)

$$Z = \frac{I}{y2} = \frac{7{,}202}{13.39} = 273 \text{ cm}^3$$

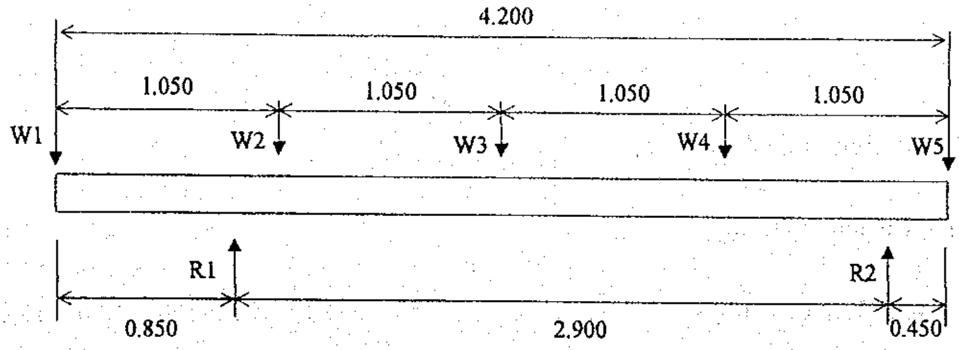
(2) Stress on Vertical Beam

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

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

Block No.	m cm	n cm	p kgf/cm^2	M kgf/cm^2	σ kgf/cm^2	Allowable Stress kgf/cm^2
1	112.5	105.0	0.0833	8036	29	1,200
2	112.5	105.0	0.1925	18570	68	1,200
3	112.5	105.0	0.2975	28699	105	1,200
4	112.5	105.0	0.4195	40469	148	1,200

8. FORCE OF END BEAM AND REACTING FORCE OF ROLLER



8.1 Load on Beam

Beam No.		Load (tf)
No 1		1.040
No 2		4.153
No 3		7.269
No 4		10.238
No 5		8.560
Total		31.260

Reacting Force of Roller

Roller No.		Force
No 1	R1	10.137
No 2	R2	21.123
Total		31.260

8.2 Bending Moment and Shear Force

Position	Bending Moment	Shear Force
No 1	0.000	1.040
No 1	88.400	1.040 -9.097
No 2	-93.543	-9.097 -4.944
No 3	-612.680	-4.944 2.325
No 4	-368.571	2.325 12.563
No 2	385.200	12.563 -8.560
No 5	0.000	-8.560

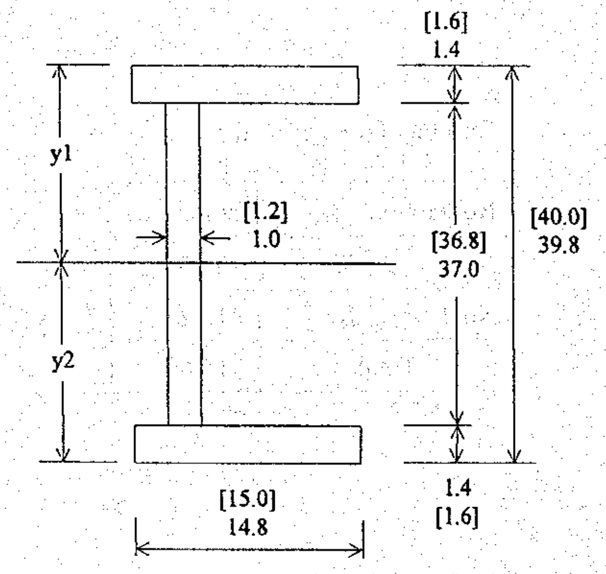
Maximum bearing moment at Roller   Mmax   = 385.200 tr-cm
Maximum shear force at Roller   Smax   = 12.563 tr

Maximum bearing moment on Beam	Mmax	= 612.680 tr-cm
Maximum shear force on Beam	Smax	= 12.563 tr

8.3 Calculation of End Beam

Sectional Dimension of Beam

(Portion without Roller Bearing) BH 400 x 150 x 12.0 x 16.0



Moment of Section

I = 19,490 cm^4

Section Modulus

Z1 = 979 cm^3

Z2 = 979 cm^3

Sectional area of web

Aw = 37.0 cm^2

Sectional area of flange

Ac = 20.7 cm^2

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 1.40 x 14.8	20.7	0.70	14.49	10.14	3.38
t 1.40 x 37.0	37.0	19.90	736.30	14,652.37	4,221.08
t 1.40 x 14.8	20.7	39.10	809.37	31,646.37	3.38
Total	78.4	——	1,560.16	50,536.72	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{1,560.16}{78.4} = 19.90 \text{ cm}$$

$$y2 = 39.8 - 19.90 = 19.90 \text{ cm}$$

Moment of Section (I)

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

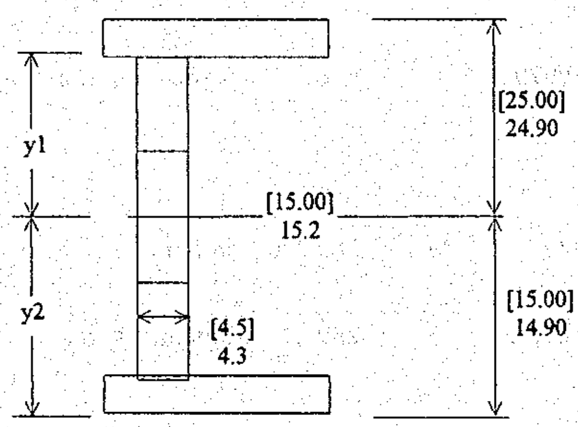
$$= 50,536.72 - 78.4 \times 19.90^2 = 19,490 \text{ cm}^4$$

Section Modulus (Z)

$$Z1 = \frac{I}{y1} = \frac{19{,}490}{19.90} = 979 \ \text{cm}^3$$

$$Z2 = \frac{I}{y2} = \frac{19{,}490}{19.90} = 979 \ \text{cm}^3$$

(Portion with Roller Bearing)



Moment of Section

I = 29,767 cm^4

Section Modulus

Z1 = 1,703 cm^3

Z2 = 1,334 cm^3

Sectional area of web

Aw = 25.4 cm^2

Sectional area of flange

Ac = 20.7 cm^2

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 1.40 x 14.8	20.7	0.70	14.49	10.14	3.38
t 1.30 x 15.9	68.4	9.35	639.54	5,979.70	1,440.38
t 1.30 x 5.9	25.4	35.45	900.43	31,920.24	73.59
t 1.40 x 14.8	20.7	39.10	809.37	31,646.37	3.38
Total	135.2	-----	2,363.83	71,077.18	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{2{,}363.83}{135.2} = 17.48 \ \text{cm}$$

$$y2 = 39.8 - 17.48 = 22.32 \ \text{cm}$$

Moment of Section (I)

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

$$= 71{,}077.18 - 135.2 \times 17.48^2 = 29.767 \ cm^4$$

Section Modulus (Z)

$$Z1 = \frac{I}{y1} = \frac{29{,}767}{17.48} = 1{,}703 \ cm^3$$

$$Z2 = \frac{I}{y2} = \frac{29.767}{22.32} = 1{,}334 \ cm^3$$

(1) Maximum Bending Stress

(Portion without Roller Bearing)

Maximum Bending Moment $M = 612.680$ kgf-cm

$$\sigma = \frac{M}{Z1} = 612.680/979 = 626 \ kgf/cm^2 < \sigma a$$

(Portion with Roller Bearing)

Maximum Bearing Moment $M = 385.200$ kgf-cm

$$\sigma = \frac{M}{Z2} = 385.200/1{,}334 = 289 \ kgf/cm^2 < \sigma a$$

Allowable Stress

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

$$\frac{9}{2.0} < \frac{105.0}{14.8} \leq 30 \qquad \sigma a = 1{,}200 - 11 \cdot \left(2.0 \cdot \frac{105.0}{14.8} - 9 \right)$$

$4.5 < 7.1 \leq 30$

$$= 1{,}143 \ kgf/cm^2$$

Where,

L	:	Distance fixed points of compressive flange	105.0	cm
b	:	With of compressive flange	14.8	cm
Aw	:	Total sectional area of web	37.0	cm^2
Ac	:	Total sectional area of flange	20.7	cm^2
fa	:	Correction factor	1.0	

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

$$\text{Where, } \frac{Aw}{Ac} = \frac{37.0}{20.7} = 1.8$$

$$= 2$$

$$\sigma a = f a \cdot \sigma a = 1.00 \times 1{,}143 = 1{,}143 \text{ kgf/cm}^2$$

(2) Maximum Shearing Stress

(Portion without Roller Bearing)

Maximum Shearing Force P = 12,563 kgf

$$\tau = \frac{P}{Aw} = 12.563 / 37.0 = 340 \text{ kgf/cm}^2 < \tau a$$

(Portion with Roller Bearing)

Maximum Shearing Stress P = 12,563 kgf

$$\tau = \frac{P}{Aw} = 12.563 / 25.4 = 495 \text{ kgf/cm}^2 < \tau a$$

Allowable Stress

$$\tau a = f a \cdot \tau a = 1.00 \times 700 = 700 \text{ kgf/cm}^2$$

9. ROLLER

9.1 Load of Roller

Load of Roller P = 21.123 kgf

9.2 Dimensions of Roller

Outer diameter of roller	RD	= 55.0	cm
Diameter of roller shaft (Bearing side)	D1	= 15.0	cm
Diameter of roller shaft (Bush side)	D2	= 13.0	cm
Width roller boss	B	= 12.0	cm
Width roller shoe	b1	= 10.0	cm
Effective width of roller shoe	b2	= 9.0	cm
Material of roller	SCMnCr3B		
Stiffness of roller	HB	= 210	
Material of roller shaft	SUS304		
Correction coefficient by standard	1.000		

9.3 Strength of Roller (Line Contact)

$$\rho = 0.418 \cdot \sqrt{\frac{P \cdot 2.1 \cdot 10^6}{b2 \cdot D/2}}$$

$$= 0.418 \times \sqrt{\frac{21.123 \times 2.1 \times 10^6}{9.0 \times 55.0/2}} = 5{,}596 \quad kg/cm^2 < \rho a$$

$$C = 1.52 \cdot \sqrt{\frac{P \cdot D/2}{b2 \cdot 2.1 \cdot 10^6}}$$

$$= 1.52 \times \sqrt{\frac{21{,}123 \times 55.0/2}{9.0 \times 2.1 \times 10^6}} = 0.266 \quad cm$$

$$Z = 0.78 \cdot C$$

$$= 0.78 \times 0.266 = 0.208 \quad cm$$

Where,

ρ	:	Strength of Herz at contacting face		kgf/cm^2
P	:	Load of roller (acting)	21,123	kgf
b2	:	Effective width of roller shoe	9.0	cm
D	:	Outer diameter of roller	55.0	cm
C	:	Hart width of contacting face	0.266	cm
Z	:	Depth at maximum shearing stress	0.208	cm

Allowable Stress at Contacting Face

$$\rho a = \frac{100}{2 \cdot \upsilon} \cdot HB$$

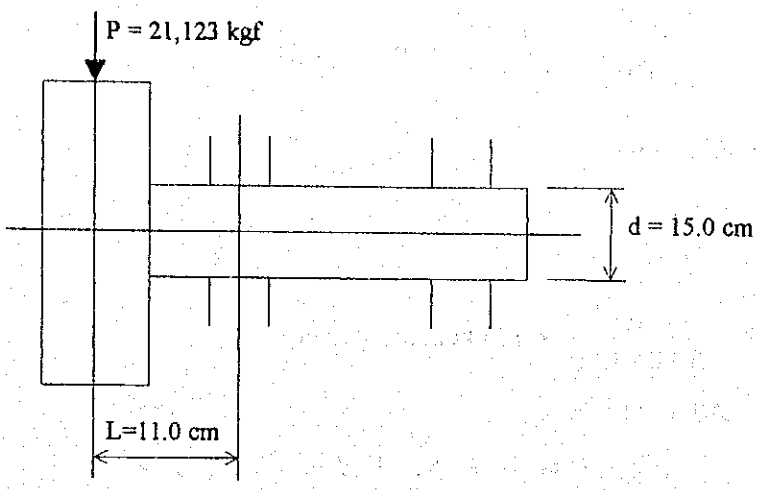
Where,

υ	:	Safety factor	1.0	
HB	:	Burinel stiffness of roller	210	kgf/mm^2

$$= \frac{100}{2 \times 1.3} \times 210 = 8{,}077 \quad kgf/cm^2$$

9.4 Roller Shaft

(1) Bearing Moment acting on Roller Axis



$$M = P \cdot L$$

$$= 21{,}123 \times 11.0 = 232{,}353 \quad \text{kgf-cm}$$

(2) Dimensions of Roller Shaft

Diameter (Shaft side)	d1	=	15.0	cm
Diameter (Bush side)	d2	=	13.0	cm
Section modulus	Z	=	331	cm^3
Section Area	A	=	177	cm^2
Material	SUS304			

(3) Bearing Stress

$$\sigma = \frac{M}{Z} = 232.353 / 331 = 702 \quad \text{kgf/cm}^2 < \sigma a$$

Allowable bearing stress = Yielding point / 2 · Correction factor

$$\sigma a = 2{,}100 / 2 \times 1.000 = 1{,}050 \quad \text{kgf/cm}^2$$

(4) Shearing Stress

$$\tau = \frac{4 \cdot P}{3 \cdot A} = \frac{4 \times 21{,}123}{3 \times 177} = 159 \quad \text{kgf/cm}^2 < \tau a$$

Allowable shearing stress = $\sigma a \ / \ 3^{0.5}$

$$\tau a = 1{,}050 / 1.732 = 606 \quad \text{kgf/cm}^2$$

(5) Composite Stress

$$Sf = \left(\frac{\sigma}{\sigma a}\right)^2 + \left(\frac{\tau}{\tau a}\right)^2$$

$$= \left(\frac{702}{1{,}050}\right)^2 + \left(\frac{159}{606}\right)^2 = 0.5 < 1.2$$

(6) Bush's Bearing Stress

$$\sigma c = \frac{P}{d \cdot B}$$

$$= \frac{21{,}123}{13.0 \times 12.0} = 135 \text{ kgf/cm}^2 < \sigma ca$$

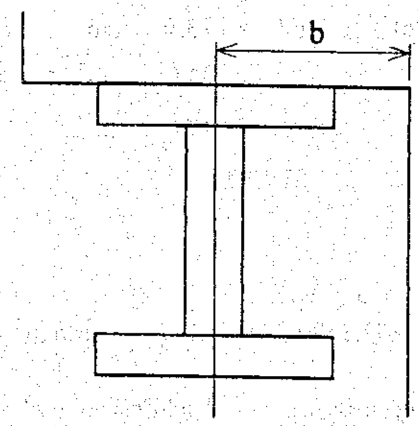
Allowable stress

σca = 250 kgf/cm^2

Oil less Metal #500 SP-SL4

10. GUIDE FRAME AND ROLLER RAIL

10.1 Distance between Side of Pier and Center of Roller Rail



$b > 20 + 0.1 \cdot R$

Where, b : Distance between side of Pier and Center of Roller Rail (cm)

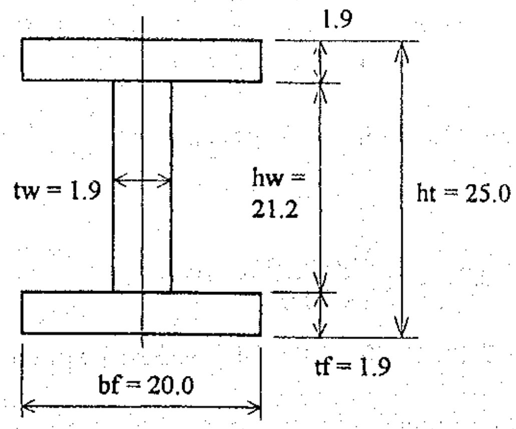
R : Load of Roller (1 Roller) (tf)

25.0 cm > 20 + 0.1 x 21.123 = 22.1 cm

10.2 Section Efficiency of Roller Rail

Moment of Area

$I = 11{,}670 \text{ cm}^4$



Section Modulus

Z = 934 cm^3

Where,

$$I = \frac{bf \cdot ht^3}{12} - \frac{(bf - tw) \cdot hw^3}{12}$$

$$= \frac{20.0 \times 25.0^3}{12} - \frac{(20.0 - 1.9) \times 21.2^3}{12} = 11{,}670 \ cm^4$$

$$Z = \frac{2 \cdot I}{ht}$$

$$= \frac{2 \times 11{,}670}{25.0} = 934 \ cm^3$$

10.3 Strength of Roller Rail

Andle's formula is used for estimating the strength of roller rail.

$$k = 0.0588 \cdot \frac{R}{\sqrt[3]{bf^2 \cdot I}}$$

$$a = 0.75 \cdot \frac{R}{k \cdot bf}$$

$$M = \frac{k \cdot a^2 \cdot bf}{4}$$

Where,

k	:	Bearing stress of concrete	kgf/cm^2
R	:	Load per one roller	kg
bf	:	Bottom flange width of roller rail	cm
2a	:	Distribution length of concrete stress of bottom of the rail	cm

M : Bearing moment on roller rail kgf-cm

$$k = 0.0588 \times \frac{21{,}123}{\sqrt[3]{20.0^2 \times 11{,}670}} = 7.4 \text{ kgf/cm}^2 < ka$$

Allowable bearing stress of concrete.

ka = 55.0 kgf/cm²

$$a = 0.75 \times \frac{21{,}123}{7.4 \times 20.0} = 107.0 \text{ cm}$$

$$M = 7.4 \times 107.0^2 \times 20.0 / 4 = 423{,}613 \text{ kgf-cm}$$

Bearing stress of roller rail σr

$$\sigma r = \frac{M}{Z} = \frac{423{,}613}{934} = 454 \text{ kgf/cm}^2 < \sigma ra$$

Allowable bearing stress

σra = fa · σa

= 1.00 x 1,200

= 1,200 kgf/cm²

10.4 Thickness of Rail

tr > 4 · Zr

Where,

tr	:	Thickness of rail		cm
Zr	:	Depth at maximum shearing stress	0.208	cm

1.0 cm > 4 x 0.208 = 0.832 cm

10.5 Stress Arissing on Web Portion

$$\sigma b = \frac{R}{bp \cdot tw}$$

Where,

σ b	=	Stress on web portion		kgf/cm²
R	=	Acting Roller load	21,123	kg
bp	=	Width of pressure receiving portion	6.3	cm
tw	=	Thickness of web	1.9	cm
2C	=	Width of roller contacting area	0.5	cm

tf	=	Thickness of upper flange	1.9	cm
tr	=	Thickness of rail	1.0	cm
C	=	Half of contacting area	0.3	cm

$$= \frac{21{,}123}{6.3 \times 1.9} = 1{,}765 \ \text{kgf/cm}^2 < \sigma\,\text{ba}$$

Allowable Bearing Stress

$$\sigma\,\text{ba} = \text{f a} \cdot \sigma\,\text{a}$$

$$= 1.00 \times 1{,}800$$

$$= 1{,}800 \ \text{kgf/cm}^2$$

10.6 Bearing Stress of Lower Flange for Roller Rail

$$\text{Mf} = \frac{\text{K} \cdot \text{bf}^2}{8}$$

$$\sigma\,\text{f} = \frac{\text{Mf}}{\frac{\text{tf}^2}{6}}$$

Where,

Mf	=	Bending moment	kgf-cm/cm
k	=	Bearing stress of concrete	kgf/cm^2
bf	=	Width of lower flange of roller rail	cm
σ f	=	Bending Stress	kgf/cm^2
tf	=	Thickness of roller flange to roller rail	cm

$$\text{Mf} = \frac{7.4 \times 20.0^2}{8} = 370 \ \text{kgf-cm/cm}$$

$$\sigma\,\text{f} = \frac{370}{\frac{1.9^2}{6}} = 615 \ \text{kgf/cm}^2 < \sigma\,\text{fa}$$

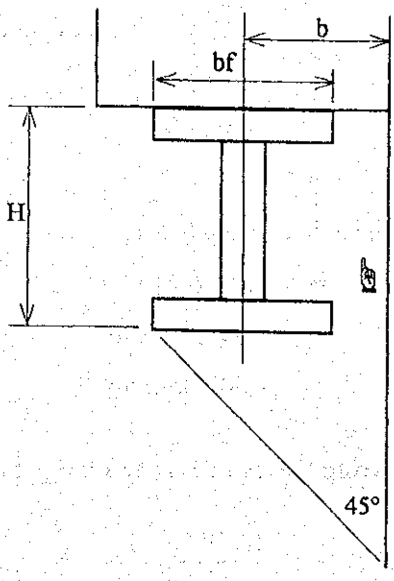
Allowable Bending Stress

$$\sigma\,\text{ba} = \text{f a} \cdot \sigma\,\text{a}$$

$$= 1.00 \times 1{,}200$$

$$= 1{,}200 \ \text{kgf/cm}^2$$

10.7 Shearing Strength of Concrete



b = 25.0 cm

bf = 20.0 cm

H = 25.0 cm

Shearing length of concrete : L

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

$$= 2 \times \left(25.0 + \frac{20.0}{2}\right) + 25.0 = 95 \text{ cm}$$

Shearing strength of concrete : τ c

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

$$= 7.4 \times 20.0 / 95 = 1.6 \text{ kgf/cm}^2 < \tau ca$$

Allowable shearing strength

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

11. LOADS FOR GATE OPENING AND CLOSING

11.1 Loads in Gate Operation

(1) Gate Lifting Time

Pu = 62.058 + 1.130 = 63.188 tf (water pressure + soil pressure)

(2) Gate Lowering Time

Pd = 2.825 tf (water pressure)

11.2 Loads Acting on Gate

(1) Total Weight of Gate

Self Weight Wg = 10.500 tf

(2) Friction Due to Roller Rotation

(Gate Lifting Time)

$$Fgu = \frac{\mu 1 + \mu 2 + \cdot d/2}{D/2} \cdot Pu$$

Where,

D	:	Diameter of roller	55.00	cm
d	:	Center diameter of bearing	13.00	cm
μ1	:	Friction Coefficient by roller rolling	0.1	
μ2	:	Friction Coefficient by bearing sliding	0.02	
Pu	:	Loads in Gate lifting time	63.188	tf

$$= \frac{0.1 + 0.02 \times 13.00/2}{55.00/2} \times 63.188$$

= 0.528 tf

(Gate Lowering Time)

$$Fgd = \frac{\mu 1 + \mu 2 + \cdot d/2}{D/2} \cdot Pd$$

Where,

D	:	Diameter of roller	55.00	cm
d	:	Center diameter of bearing	13.00	cm
μ1	:	Friction Coefficient by roller rolling	0.1	
μ2	:	Friction Coefficient by bearing sliding	0.02	
Pd	:	Loads in Gate lifting time	2.825	tf

$$= \frac{0.1 + 0.02 \times 13.00/2}{55.00/2} \times 2.825$$

= 0.024 tf

(3) Friction Due to Sediment

(Gate Lifting Time)

Fe = μe · Pe

Where,

μe	:	Friction coefficient	0.40	
Pe	:	Muddy soil pressure	1.130	tf

= 0.40 x 1.130

= 0.452 tf

(4) Friction of Rubber

(Gate Lifting Time)

Fru = $2 \cdot \mu r \cdot (q + p \cdot b) \cdot L$

Where,

μr :	Friction coefficient of rubber	0.7	
q :	Inicial pressure	0.100	tf/m
p :	Average force acting on side	2.525	tf/m^2
b :	Effective width of receiving pressure	0.050	m
L :	Length of watertight rubber at side	4.350	tf

= 2 x 0.7 x (0.100 + 2.525 x 0.050) x 4.350

= 1.378 tf

(Gate Lowering Time)

Frd = $2 \cdot \mu r \cdot (q + p \cdot b) \cdot L$

Where,

μ :	Friction coefficient of rubber	0.7	
q :	Inicial pressure	0.100	tf/m
p :	Average force acting on side	0.115	tf/m^2
b :	Effective width of receiving pressure	0.050	m
L :	Length of watertight rubber at side	4.350	tf

= 2 x 0.7 x (0.100 + 0.115 x 0.050) x 4.350

= 0.644 tf

(5) Uplift Pressure

(Gate Lifting Time)

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

Where,

Wg :	Self weight of Gate	10.500	tf
γ :	Specific gravity of steel	7.85	
s :	Volume ratio of submerged portion of gate	100.0	%

= $\frac{10.500}{7.850} \times \frac{100.0}{100}$

= 1.338 tf

(Gate Lowering Time)

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

Where,

Wg :	Self weight of Gate	10.500	tf
γ :	Specific gravity of steel	7.85	
s :	Volume ratio of submerged portion of gate	25.0	%

$$= \frac{10.500}{7.850} \times \frac{25.0}{100}$$

$= 0.334$ tf

(6) Downward Force Acting on Upper Portion of Gate

(Gate Lifting Time)

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

Where,

p :	Acting pressure	0.350	tf/m^2
t :	Width of stress receiving area	0.412	m
B :	Length of stress receiving area	5.650	m
k :	Coefficient of over flow water	1.000	
ΔH :	Water level difference	0.350	m
D :	Width of Gate	0.412	m

$= 0.350 \times 0.412 \times 5.650 + 1.000 \times 0.350 \times 0.412 \times 5.650$

$= 1.630$ tf

(7) Upward Force Acting on Lower Portion of Gate

(Gate Lowering Time)

$$Fbud = p \cdot t \cdot B$$

Where,

p :	Acting pressure	1.000	tf/m^2
t :	Width of stress receiving area	0.050	m
B :	Length of stress receiving area	5.650	m

$= 1.000 \times 0.050 \times 5.650$

$= 0.283$ tf

(8) Downward Force Acting on Lower Portion of Gate

(Lifting Time)

Fbdu = $p \cdot t \cdot B + k \cdot \Delta h \cdot D \cdot B$

Where,

p	:	Acting force	0.000	tf/m^2
t	:	Width of stress receiving area	0.000	m
B	:	Length of stress receiving area	5.650	m
k	:	Coefficient of Downplay	1.000	
ΔH	:	Water level difference	4.700	m
D	:	Thickness of Gate	0.412	m

= 0.000 x 0.000 x 5.650 + 1.000 x 4.700 x 0.412 x 5.650

= 10.941 tf

(Gate Lowering Time)

Fbdd = $p \cdot t \cdot B + k \cdot \Delta h \cdot D \cdot B$

Where,

p	:	Acting force	0.000	tf/m^2
t	:	Width of stress receiving area	0.000	m
B	:	Length of stress receiving area	5.650	m
k	:	Coefficient of Downplay	1.000	
ΔH	:	Water level difference	1.000	m
D	:	Thickness of Gate	0.412	m

= 0.000 x 0.000 x 5.650 + 1.000 x 1.000 x 0.412 x 5.650

= 2.328 tf

11.3 Total of Loads for Gate Opening and Closing

Item	Lifting	Lowering
Self weight	10.500	10.500
Friction due to roller rotation	0.528	-0.024
Friction of rubber	1.378	-0.644
Friction due to sediment	0.452	-
Uplift pressure	-1.338	-0.334
Downward Force of Gate	1.630	-
Upward Force of Gate	-	-0.283
Downward Force on Lower Gate Body	10.941	2.328
Total	24.091	11.543

Lifting Force	25.000 tf
Lowering Force	Self weight lowering

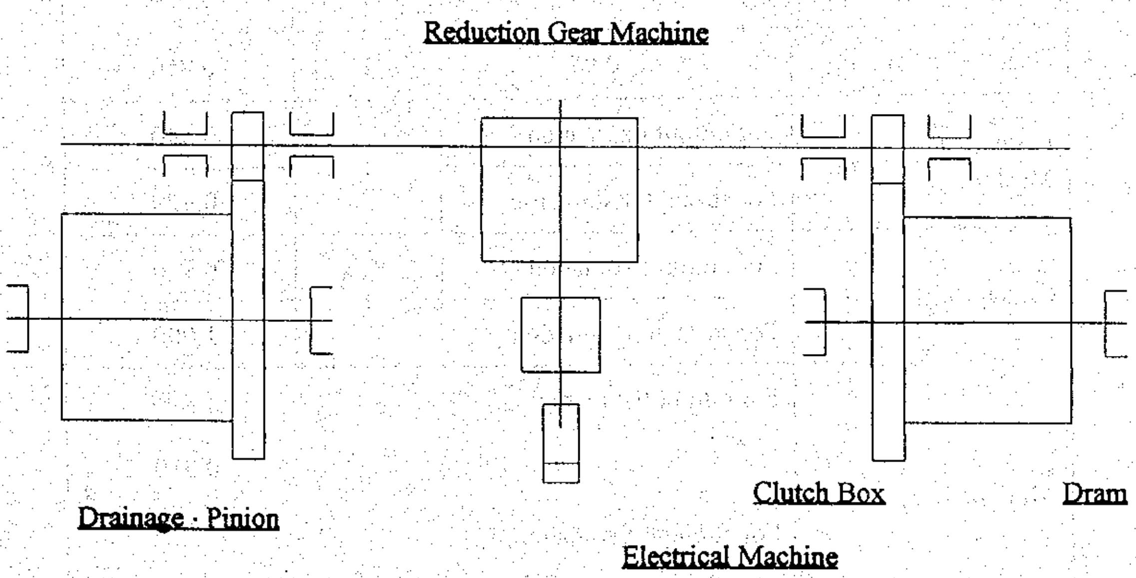
12. GATE HOISTING SYSTEM

12.1 Specification of System

Method	1 Motor + 1 Dram Winch
Operation Method	(Local manual operation & Remote manual operation)
Power	Electricity
Hoisting Speed	0.300 m/min
Total Head	7.500 m
Hoisting Load	
Lifting time	25.000 tf
Lowering time	Falling by Self-Weight
Source Power	220 V, 50 Hz
Safety Factor	

12.2 Layout of Hoisting Machine

(1) Layout Plan



12.3 Hoisting Speed

$$V = RPM \cdot \frac{1}{X(1)} \cdot \frac{Z1(1)}{Z2(1)} \cdot \frac{1}{N} \cdot \frac{D \cdot \pi}{1000}$$

Where,

V	:	Hoisting Speed (Calculation Value)		m/min
RPM	:	Revolution Number	1,140	rpm
X(1)	:	Ratio Reduction Gear-1	1/520	
Z1(1)	:	Number of Cog-wheel of Pinion	26	
Z2(1)	:	Number of Cog-wheel of Gear	88	
N	:	Number of Wire	4	
D	:	Diameter of Dram	600	mm

$$= 1{,}140 \times \frac{1}{520} \times \frac{26}{88} \times \frac{1}{4} \times \frac{600 \times \pi}{1000} = 0.305 \text{ m/min}$$

12.4 Efficiency of Machine

Parts		Unit Efficiency	Number	Total Efficiency
Sheave	ηs	0.95	3	0.927
Dram	ηd	0.95	1	0.950
Gear	ηg	0.95	1	0.950
Reduction Machine–1	ηr	For output Estimation	1	0.390
		For strength Estimation		0.429
Clutch Box	ηc	For output Estimation	1	0.950
		For strength Estimation		1.000
All Efficiency		For output Estimation	$\eta 1$ =	0.310
		For strength Estimation	$\eta 2$ =	0.359
Remarks		Sheave (One side) 1		

12.5 Generating Power

$$Kw = \frac{Wu \cdot V}{6.12 \cdot \eta 1}$$

Where,

Kw	:	Generating Power		kw
Wu	:	Hoisting Load	25.000	rpm
V	:	Hoisting Speed	0.305	m/min
$\eta 1$	:	Machine Efficiency	0.310	

$$= \frac{25.000 \times 0.305}{6.12 \times 0.310} = 4.02 \text{ kw}$$

Therefore, the generating power of 5.50 kw will be used.

(3 alternating current 6P)

4.5.3 Right Bank Intake Gate

1. DESIGN ITEM

1.1 Design Condition

- Type of Gate		Steel Roller Gate
- Number of Gate		4 Gates
- Clear span		2.250 m
- Height of Gate		2.000 m
- Design Water Depth		
	Upstream	4.200 m
	Downstream	0.000 m
- Water Depth for Operation		
• Lifting Time		
	Upstream	2.400 m
	Downstream	0.000 m
• Lowering time		
	Upstream	2.040 m
	Downstream	0.000 m
- Gate Floor Level		EL 3.800 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

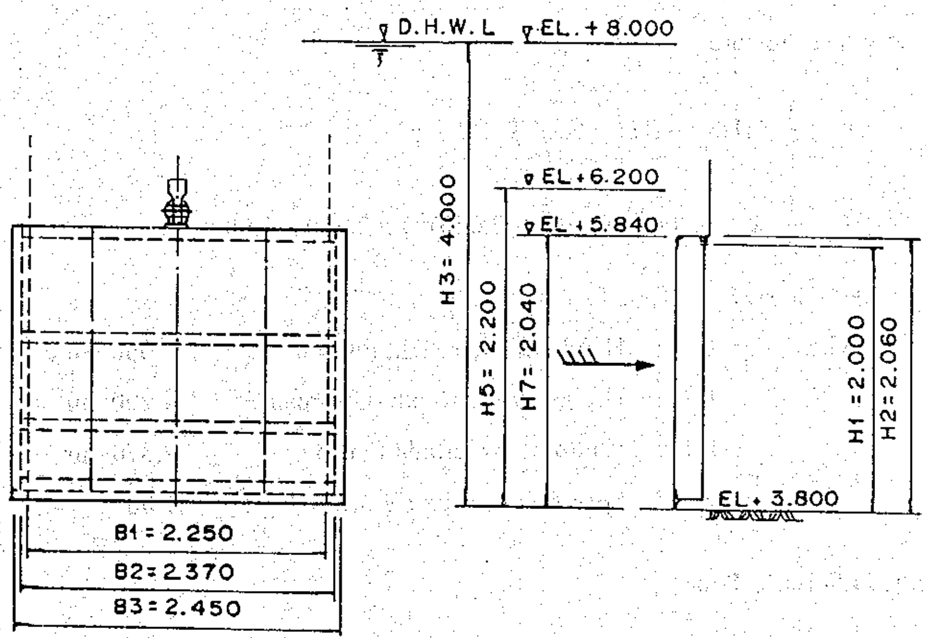
• Horizontal Seismic Intensity	0.120
• Deffelence in Temperature	———
• Wind Load	150 kgf/m^2
• Deformation of Main Girder	1/ 800
• Margin of Thickness	

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

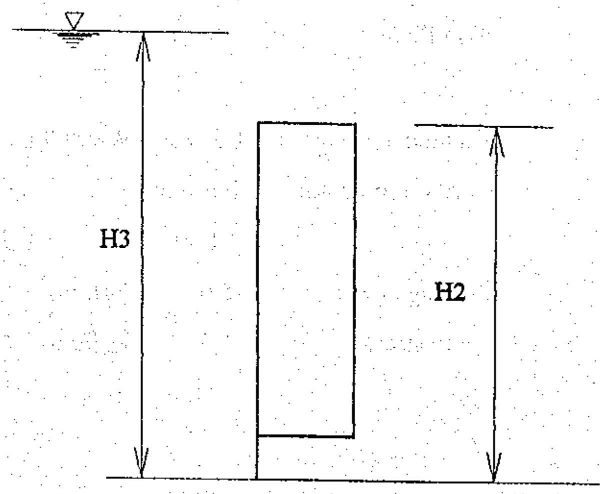
2. BASIC DIMENSIONS



B1	:	Clear Span	2.250	m
B2	:	Length of Watertight Area	2.370	m
B3	:	Span Between Rollers	2.450	m
H1	:	Height of Gate	2.000	m
H2	:	Height of Watertight Area	2.060	m
H3	:	Design Water Depth Upstream	4.200	m
H4	:	Design Water Depth Downstream	0.000	m
H5	:	Water Depth for Operation (Upstream)	2.400	m
H7	:	Water Depth for Operation (Downstream)	2.040	m

3. ACTING LOADS

3.1 Loads in Normal Time



(1) Static Water Pressure

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

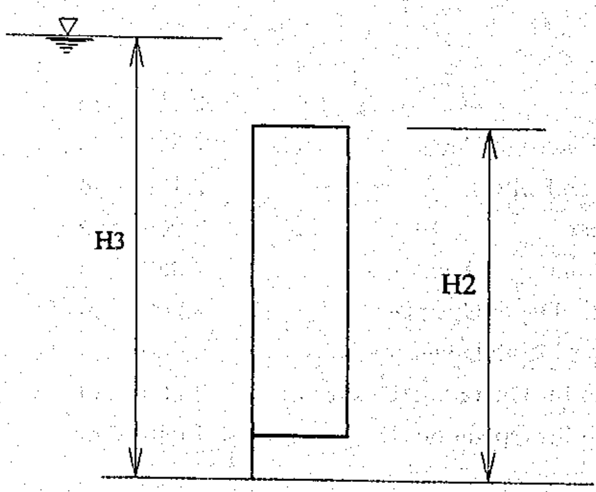
$$= \frac{1}{2} \times \{4.200^2 - (4.200 - 2.060)^2\} \times 2.370 \times 1.00$$

$$= 15.477 \quad \text{tf}$$

Where,

H2	:	Height of Watertight Portion	2.060	m
H3	:	Design Water Depth (Upstream)	4.200	m
B2	:	Width of Watertight Portion	2.370	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/sec^2
Hee	:	Distance between riverbed and water surface	4.200	m

$$= \frac{0.12 \times 1.0}{2 \cdot \pi} \times (9.8 \times 4.200)^{0.5}$$

$$= 0.123 \quad tf$$

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

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

$$= \frac{1}{2} \times (4.200 + 0.123)^2 - (4.200 + 0.123 - 2.060)^2\} \times 2.370$$

$$= 16.077 \quad tf$$

(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	4.200	m
H3	:	Design Water Depth	4.200	m
h	:	H3 – H2	2.140	m
B2	:	Width Watertight portion	2.370	m

$$= \frac{7}{12} \times 0.12 \times 4.200^{0.5} \times (4.200^{1.5} - 2.140^{1.5}) \times 2.370$$

$$= 1.862 \quad tf$$

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

$$Wge = k \cdot Wg$$

Where,

Wg : Self weight of gate 2.000 tf

$$= 0.12 \times 2.0$$

$$= 0.240 \quad tf$$

(5) Total Force in Seismic Time (Pe)

$$Pe = Peg + Pdo + Wge$$

$$= 16.077 + 1.862 + 0.240$$

$$= 18.179 \quad tf$$

(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:

$$\frac{Pe}{1.5} = \frac{18.179}{1.5} = 12.119 < Pw = 15.477$$

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

4. LOADS FOR GATE OPERATION

4.1 Lifting Time

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

$$= \frac{1}{2} \times \{2.400^2 - (2.400 - 2.060)^2\} \times 2.370 \times 1.00 = 6.689 \text{ tf}$$

Where,

H2	: Height of watertight portion	2.060	m
H5	: Water depth for Operation (Upstream)	2.400	m
B2	: Width of watertight portion	2.370	m
γ	: Specific gravity of water	1.00	

4.2 Lowering Time

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

$$= \frac{1}{2} \times 2.040^2 \times 2.370 \times 1.00 = 4.931 \text{ tf}$$

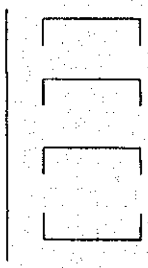
Where,

H7 : Water depth for Operation (Upstream) 2.040 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) = 2.140
	L(1) = 0.800	
2		p(2) = 2.940
	L(2) = 0.650	
3		p(3) = 3.590
	L(3) = 0.535	
4		p(4) = 4.125
	L(4) = 0.075	
At the gate floor		p(5) = 4.200

(2) Allotted Load

R (1) =

p (B) = {p (2) + 2 · p (1)} · L(1)/6 = 0.963

R(1) = 0.963 tf/m

R (2) =

p (U) = {p (1) + 2 · p (2)} · L(1)/6 = 1.069

p (B) = {p (3) + 2 · p (2)} · L(2)/6 = 1.026

R(2) = 2.095 tf/m

R (3) =

p (U) = {p (2) + 2 · p (3)} · L(2)/6 = 1.096

p (B) = L1 · [3 · L(3) · {p (3) + p (5)}

– L1 · {p (3) + 2 · p (5)}] / 6 / L (3) = 0.986

R(3) = 2.082 tf/m

R (4) =

p (T) = $\{p(3) + 2 \cdot p(5)\} \cdot L1^2 / 6 / L(3)$ = 1.390

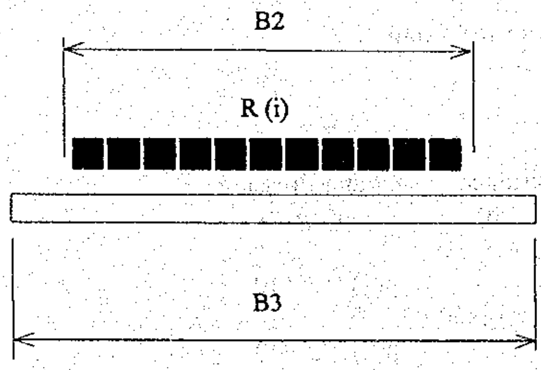
R(5) = 1.390 tf/m

Σ 6.530 tf/m

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

= 0.535 + 0.075 = 0.610 m

5.2. Sectional Force of Main Beam



Where,

B2 : Distance of watertight rubbers 2.370 m

B3 : Span length 2.450 m

R(i) : Unit load tf/m

(1) Bending Moment

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

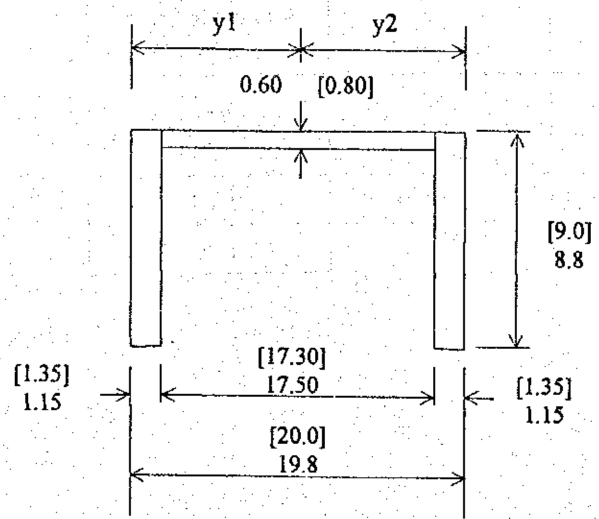
(2) Shearing Force

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

No	R (i) (tf/m)	M (i) (tf/m)	S (i) (tf)
1	0.963	0.722	1.141
2	2.095	1.570	2.483
3	2.082	1.560	2.467
4	1.390	1.042	1.647

5.3. Stress of Main Beam

(a) Sectional Dimension of Beam ⊏ 200 x 90 x 8.0 x 13.5



Moment of Area

I = 2,029 cm^4

Section Modulus

Zt = 205 cm^3

Zc = 205 cm^3

Sectional area of web

Aw = 10.5 cm^2

Sectional area of flange

Ac = 10.1 cm^2

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 1.15 x 8.8	10.1	0.58	5.86	3.40	1.12
t 0.60 x 17.5	10.5	9.90	103.95	1,029.10	267.97
t 1.15 x 8.8	10.1	19.23	194.22	3,734.91	1.12
Total	30.7	——	304.03	5,037.62	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{304.03}{30.7} = 9.90 \text{ cm}$$

$$y2 = 19.8 - 9.90 = 9.90 \text{ cm}$$

Moment of area (I)

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

$$= 5{,}037.62 - 30.7 \times 9.90^2 = 2{,}029 \text{ cm}^4$$

Section Modulus (Z)

$$Zt = \frac{I}{y2} = \frac{2{,}029}{9.90} = 205 \text{ cm}^3$$

$$Zc = \frac{I}{y1} = \frac{2{,}029}{9.90} = 205 \text{ cm}^3$$

(b) Bearing Stress

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

Allowable bearing stress

σ ac = fa · σ ac = 1.00 x 1,200 = 1,200 kgf/cm^2

No	M (i) (tf-m)	σ c (kgf/cm^2)	Allowable Stress (kgf/cm^2)
1	0.722	352	1,200
2	1.570	766	1,200
3	1.560	761	1,200
4	1.042	508	1,200

(c) Shearing Stress

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

Allowable shearing stress

τ a = fa · τ a = 1.00 x 700 = 700 kgf/cm^2

No	S (i) (tf	τ (kgf/cm^2)	Allowable Stress (kgf/cm^2)
1	1.141	109	700
2	2.483	236	700
3	2.467	235	700
4	1.647	157	700

5.4. Deformation Rate of Main Beam

Deformation

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

Where,				
R(i) :	Unit Load		kgf/cm
B2 :	Width of watertight portion	237	cm
B3 :	Span	245	cm
E :	Modulus of elasticity	2.1 x 10^6	kgf/cm^2
I :	Moment of section		vm^4

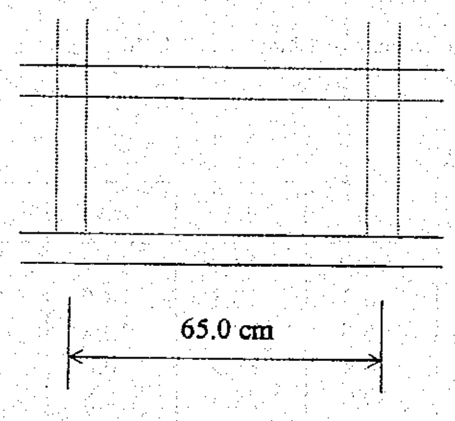
Deformation Rate

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

Beam No.	R(i) Kgf/cm)	I (cm^4)	δ (i) (cm)	$\frac{1}{Rd(i)}$	Allowable Deformation
1	9.63	2,029	0.106	$\frac{1}{2,311}$	$\frac{1}{800}$
2	20.95	2,029	0.230	$\frac{1}{1,065}$	$\frac{1}{800}$
3	20.82	2,029	0.229	$\frac{1}{1,070}$	$\frac{1}{800}$
4	13.90	2,029	0.153	$\frac{1}{1,601}$	$\frac{1}{800}$

6. SKIN PLATE

6.1 Beam Allocation and Water Pressure



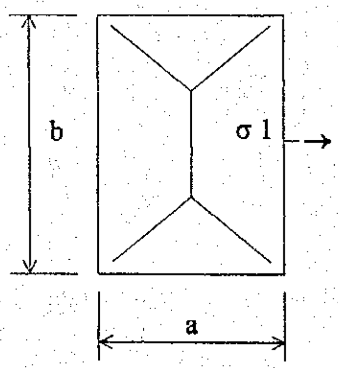
Water Pressure (kgf/cm^2)	Block No.	Length of Block (cm)	Span Length (cm)
0.2585 Average value between flanges of Main Beam →	1	71.0	65.0
0.3310 Average value between flanges of Main Beam →	2	56.0	56.0
0.3858 Average value between flanges of Main Beam →	3	35.5	35.5

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/cm²
	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/cm²
	t	:	Thickness of plate	0.90	cm
	ε	:	Margin of thickness of plate	0.20	cm



k Value

b / a	σ1	σ1	σ1	σ1
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 = 1.00 x 1,200 = 1,200 kgf/cm²

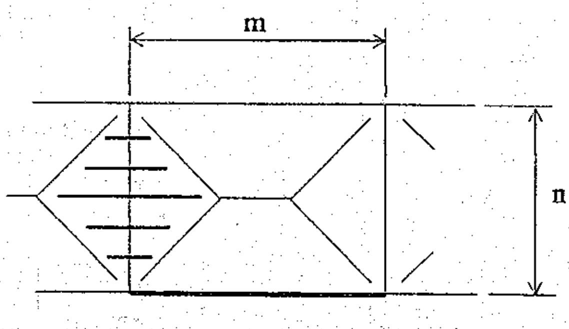
Block No.	a cm	b cm	b / a	k	L cm	p kgf/cm²	t - ε cm	σ kgf/cm²
1	65.0	71.0	1.09	34.37	65.0	Average value 0.2585	0.70	766
2	56.0	65.0	1.16	36.94	56.0	Average value 0.3310	0.70	783
3	35.5	65.0	1.83	48.89	35.5	Average value 0.3858	0.70	485

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.

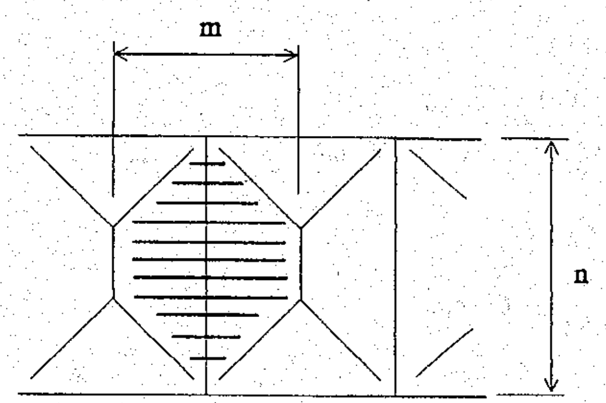
In case $m \geq n$



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 $m < n$

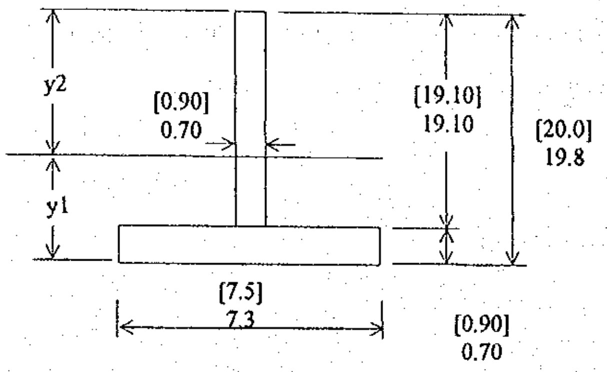


Bearing Moment

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

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

(1) Sectional Dimensions of Beam PL 200 x 75 x 9 x 9



Moment of Section

I = 769 cm^4

Section Modulus

Zt = 63 cm^3

Sectional area of web

Aw = 13.4 cm^2

Member cm	A cm^2	y cm	A · y cm^3	A · y^2 cm^4	I o cm^4
t 0.70 x 19.1	13.4	10.25	137.35	1,407.84	406.46
t 0.70 x 7.3	0.0	0.35	1.79	0.62	0.21
Total	18.5	-----	139.14	1,815.13	

$$y1 = \frac{\Sigma(A \cdot y)}{A} = \frac{139}{18.5} = 7.52 \text{ cm}$$

$$y2 = 19.8 - 7.52 = 12.28 \text{ cm}$$

Moment of Section (I)

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

$$= 1,815 - 18.5 \times 7.52^2 = 769 \text{ cm}^4$$

Section Modulus (Z)

$$Z = \frac{I}{y2} = \frac{769}{7.52} = 63 \text{ cm}^3$$

(2) Stress on Vertical Beam

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

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

Block No.	m cm	n cm	p kgf/cm^2	M kgf/cm^2	σ kgf/cm^2	Allowable Stress kgf/cm^2
1	65.0	80.0	0.2585	10484	166	1,200
2	65.0	65.0	0.3310	7575	120	1,200
3	65.0	53.5	0.3858	4923	78	1,200

8. GUIDE FRAME

8.1 Bearing Stress of Slide Plate

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

Where,

p	: Water pressure	0.420	kgf/cm^2 (State of rest)
		0.240	kgf/cm^2 (State of move)
B2	: Width of watertight area	23.70	cm
b	: Width of slide plate	3.0	cm

$$= \frac{0.420 \times 237.0}{2 \times 3.0} = 16.6 \quad kgf/cm^2 < \sigma sa \text{ (State of rest)}$$

$$= \frac{0.240 \times 237.0}{2 \times 3.0} = 9.5 \quad kgf/cm^2 < \sigma sa \text{ (State of move)}$$

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,

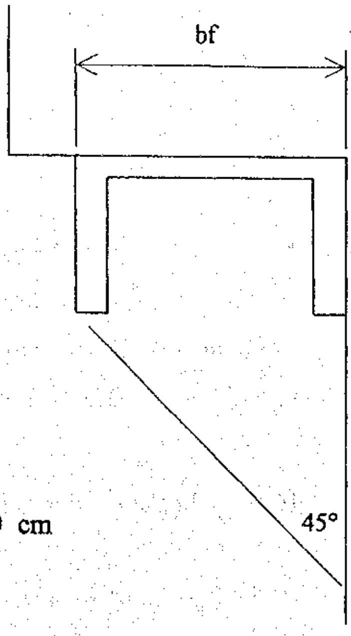
p	: Water pressure	0.420	kgf/cm^2
B2	: Width of watertight area	237.0	kgf/cm^2
bf	: Width of stress receiving area	20.0	cm

$$= \frac{0.420 \times 237.0}{2 \times 20.0} = 2.5 \quad kgf/cm^2 < \sigma ca$$

Allowable Bearing Stress

σca = 55 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 : τ c

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

$$= \frac{2.5 \times 20.0}{40} = 1.3 \text{ kgf/cm}^2 < \tau ca$$

Allowable shearing stress

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

9. LOADS FOR GATE OPENING AND CLOSING

9.1 Loads in Gate Operation

(1) Gate Lifting Time

Pu = 6.689 tf (Water pressure)

(2) Gate Lowering Time

Pd = 4.931 tf (Water pressure)

9.2 Loads Acting on Gate

(1) Total Weight of Gate Wg

Self Weight	Wg1	=	1.000 tf
Weight of Spindle	Wg2	=	0.300 tf
Total Weight	Wg	=	Wg1 + Wg2 = 1.300 tf

(2) Friction Due to Slide

(Gate lifting times)

Fgu = μs · Pu

Where,

μs	: Friction coefficient of sliding face	0.6	
Pu	: Loads in gate lifting time	6.689	tf

= 0.6 x 6.689 = 4.013 tf

(Gate lowering time)

Fgd = μs · Pd

Where,

μs	: Friction coefficient of sliding face	0.6	
Pd	: Loads in gate lifting time	4.931	tf

= 0.6 x 4.931 = 2.959 tf

(3) Friction of Rubber

(Gate Lifting Time)

Fru = μr · { q · (L1 + 2 · L2) + p1 · b · L1 + 2 · p2 · b · L2 }

Where,

μr	: Friction coefficient of rubber	0.7	
q	: Initial pressure	0.100	tf/m
p1	: Average force on upper face	0.340	tf/m^2
p2	: Average force of side face	1.370	tf/m^2
b	: Effective width of stress receiving area	0.050	m
L1	: Length of watertight rubber of upper side	2.370	m
L2	: Length of watertight rubber of side face	2.060	m

= 0.7 x {0.100 x (2.370 + 2 x 2.060) + 0.340 x 0.050 x 2.370
+ 2 x 1.370 x 0.050 x 2.060}

= 0.680 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	1.010	tf/m^2
b	: Effective width of stress receiving area	0.050	m
L1	: Length of watertight rubber of upper side	2.370	m
L2	: Length of watertight rubber of side face	2.060	m

= 0.7 x {0.100 x (2.370 + 2 x 2.060) + 2 x 1.010 x 0.050 x 2.060}

= 0.600 tf.

(4) 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	0.340	tf/m^2
t	: Width of stress receiving area	0.249	m
B	: Length of stress receiving area	2.370	m
k	: Coefficient of water overflowed	0.000	
ΔH	: Water level difference	0.340	m
D	: Thickness of gate body	0.209	m

= 0.340 x 0.249 x 2.370 + 0.000 x 0.340 x 0.209 x 2.370

= 0.201 tf.

(5) Upward Force Acting in Lower Portion of Gate

(Gate Lowering TIme)

$$Fbud = p \cdot t \cdot B$$

Where,

p	: Acting pressure	2.040	tf/m^2
t	: Width of stress receiving area	0.050	m
B	: Length of stress receiving area	2.370	m

= 2.040 x 0.050 x 2.370

= 0.242 tf.

9.3 Total Loads for Gate Opening and Closing

Item	Lifting Force	Lowering Force
Self weigh	1.600	1.600
Friction of supporting plate	4.013	-2.959
Friction of rubber	0.680	-0.600
Downward force of gate	0.201	——
Upward force of gate	——	-0.242
Total	6.494	-2.201

Nota : Lifting Force 7.000 tf
Lowering Force 3.000 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 = 7,000 kgf
	Lowering time	W2A = 3,000 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 300.0 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

$$TS = W \cdot FS = 7{,}000 \times 0.010707 = 74.95 \text{ kgf·m}$$

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	>	7,000	kgf
Allowable Torque	406	>	74,95	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

$$= \frac{74.95 \times 1{,}450}{974 \times 61.50 \times 0.39}$$

$= 4.65$ kw

Therfore, a motor with 5.50 kw is used

(5) Backling Stress of Spindle

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

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

$= 8{,}276$ 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/cm^2
σy	: Yield point or proof stress of material	2,100	kgf/cm^2

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

- Slenderness ratio (λ)

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

Where,

β	: Condition at material end	1.00	
L	: Design Length	300.0	cm

r : Radius of gyration of spindle

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

$$= 1.00 \times 300.0 / 1.95 = 153.8$$

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

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

$$= \frac{3.14^2 \times 1.97 \times 10^6}{153.8^2} = 821 \text{ kg/cm}^2$$

- Compressive Stress σc

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

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

- Safety factor Sf

Normal use $Sf = \frac{\sigma k}{\sigma c} = \frac{821}{173} = 4.74 > 4.0$

At maximum torque $Sf = \frac{\sigma k}{3 \cdot \sigma c} = \frac{821}{3 \times 173} = 1.58 > 1.1$

(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	

$$= \frac{74.95}{41.00 \times 0.33 \times 0.305 \times 5.50 \times 0.95}$$

$$= 3.5 \text{ kgf}$$

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