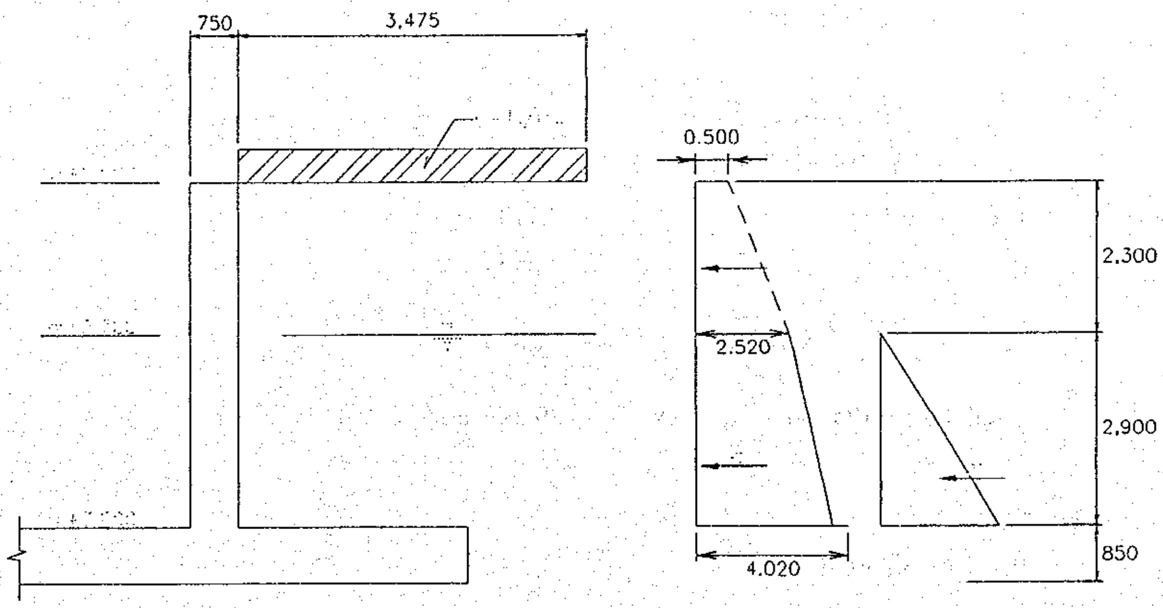
4.3.2.2 Retaining Wall Type I (Side Wall)



<u>Wall</u>

- $P_1 = \dfrac{0.5+2.57}{2}\times 2.3$ = 3.530 tf

  $Z_1 = 2.9 + \dfrac{2\times 0.5+2.57}{3(0.5+2.57)}\times 2.3$ = 3.801 m

  $M_1 = 3.530 \times 3.801$ = 13.418 tf.m

- $P_2 = \dfrac{2.57+4.02}{2}\times 2.90$ = 9.556 tf

  $Z_2 = \dfrac{2\times 2.57+4.02}{3(2.57+4.02)}\times 2.9$ = 1.343 m

  $M_2 = 1.343 \times 9.556$ = 12.834 tf.m

- $P_3 = ½ \times 2.9^2$ = 4.205 tf

  $Z_3 = \dfrac{2.9}{3}$ = 0.967 m

  $M_3 = 4.025 \times 0.967$ = 3.892 tf.m

  $\Sigma P = 3.530 + 9.556 + 4.025$ = 17.111 tf

ΣM = 13.418 + 12.834 + 3.892 = 30.144 tf.m

Shear Stress

$\tau = \dfrac{17111}{\frac{7}{8} \times 100 \times 66} = 2.963 \text{ kg f/cm}^2 < \bar{\tau} = 6.5 \text{ kgf/cm}^2$

$Ca = \dfrac{66}{\sqrt{\dfrac{15 \times 3014400}{100 \times 1600}}} = 3.926$

$Ca = 3.926$, $\delta = 0$ } $\phi = 2.173$, $nw = 0.0725$

$A = \dfrac{0.0275}{25} \times 100 \times 66 = 31.896 \text{ cm}^2$

Used D25 a 125 → A = 39.279 cm^2

For Slab

- P_1 = 3.530 tf
  Z_1 = 0.425 + 3.801 = 4.226 m
  M_1 = 3.530 x 4.226 = 14.920 tf.m

- P_2 = 9.556 tf
  Z_2 = 0.425 + 1.343 = 1.768 m
  M_2 = 9.556 x 1.768 = 16.901 tf.m

- $P_3 = \frac{1}{2} \times 2.9^2$ = 4.205 tf
  $Z_3 = 0.425 \times \dfrac{2.9}{3}$ = 1.392 m
  M_3 = 4.205 x 1.392 = 5.853 tf.

ΣP = 3.530 + 9.556 + 4.025 = 17.111 tf

ΣM = 14.920 + 16.900 + 5.853 = 37.673 tf.m

M = 37.673 tfm

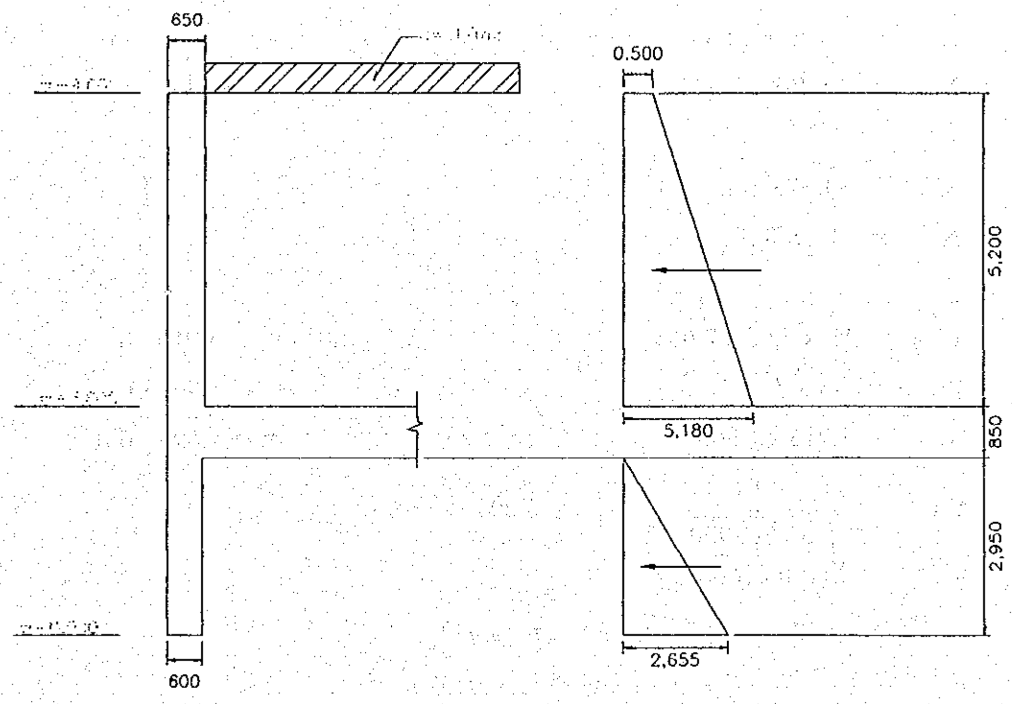
$Ca = \dfrac{76}{\sqrt{\dfrac{15 \times 3767300}{100 \times 1600}}} = 4.044$

$Ca = 4.044$, $\delta = 0$ } $\phi = 2.571$, $nw = 0.0686$

$$A = \frac{0.0686}{15} \times 76 \times 100 = 34.757 \ cm^2$$

Used D25 – 125 → A = 39.269 cm^2

4.3.2.3 Retaining Wall Type II (Breast Wall at Floodway Side)



<u>Wall</u>

$$P = \frac{0.5 + 5.18}{2} \times 5.2 = 14.768 \ tf$$

$$Z = \frac{2 \times 0.50 + 5.18}{3(0.5 + 5.18)} \times 5.2 = 1.889 \ m$$

$$M = 14.768 \times 1.889 = 27.896 \ tf.m$$

Shear Stress

$$\tau = \frac{14768}{7/8 \times 100 \times 56} = 3.0138 \ kg f/cm^2 < \bar{\tau} = 6.5 \ kgf/cm^2$$

$$M = 27.896 \ tf.m$$

$$Ca = \frac{56}{\sqrt{\frac{15 \times 2789600}{100 \times 1600}}} = 3.464$$

$$Ca = 3.464 \quad \} \quad \phi = 1.855$$

δ = 0 nw = 0.0944

A = $\frac{0.0944}{15}$ x 100 x 56 = 35.250 cm^2

Used D25 – 125 → A = 39.270 cm^2

For Slab

P = 14.768 tf

Z = 0.425 + 1.889 = 2.314 m

M = 14.768 x 2.314 = 34.173 tf.m

M = 34.175 tfm

N = -14.768 tf

P = ½ x 2.95 x 2.655 = 3.916 tf

Z_1 = 2/3 x 2.95 = 1.9666 m

M = 3.916 x 1.960 = 7.699 tf.m

$$Ca = \frac{51}{\sqrt{\frac{15\times 769900}{100\times 1600}}} = 6.003$$

Ca = 6.003 } ϕ = 3.630

δ = 0 } nw = 0.02976

A = $\frac{0.02976}{15}$ x 100 x 51 = 10.118 cm^2

Used D25 a 250 → A = 19.635 cm^2

Slab I

Z = 1.9666 + 0.425 = 2.3186 m

M = 2.3816 x 3.916 = 9.3263 tf.

N = –3.916 tf

$$e = \frac{9.3265}{-3.916} = -2.382 \text{ m}$$

$$e_a = -2.382 + \frac{0.65}{2} - 1.65 = -2.222 \text{ m}$$

Ne_a = –3.916 x –2.222 = 8.701 tf.m

$$Ca = \frac{48.5}{\sqrt{\frac{15\times 870100}{100\times 1800}}} = 5.696$$

$$\left.\begin{aligned} Ca &= 5.696 \\ \delta &= 0 \end{aligned}\right\} \quad \begin{aligned} \phi &= 3.386 \\ nw &= 0.0367 \\ \zeta &= 0.927 \end{aligned}$$

$$i = \frac{1}{1-0.927\cdot\frac{0.485}{-2.222}} = 0.832$$

$$iA = \frac{0.0367}{15} \times 100 \times 48.5 = 11.866 \text{ cm}^2$$

$$A = 11.866 \text{ cm}^2$$

$$A = \frac{11.832}{0.832} = 14.262 \text{ cm}^2$$

Used D16 – 125

<u>Slab II</u>

$$\Delta M = 34.173 - 9.3263 = 24.847$$

$$H = 3.916 + 14.768 = 18.384$$

$$e = \frac{24.847}{-18.384} = -1.352 \text{ m}$$

$$e_a = -1.352 + \frac{0.85}{2} - 0.09 = -1.017 \text{ m}$$

$$Ne_a = -1.017 \times -14.768 = 15.019 \text{ tf.m}$$

$$Ca = \frac{76}{\sqrt{\frac{15\times1501900}{100\times1600}}} = 6.405$$

$$\left.\begin{aligned} Ca &= 6.405 \\ \delta &= 0 \end{aligned}\right\} \quad \begin{aligned} \phi &= 3.902 \\ nw &= 0.02614 \\ \zeta &= 0.932 \end{aligned}$$

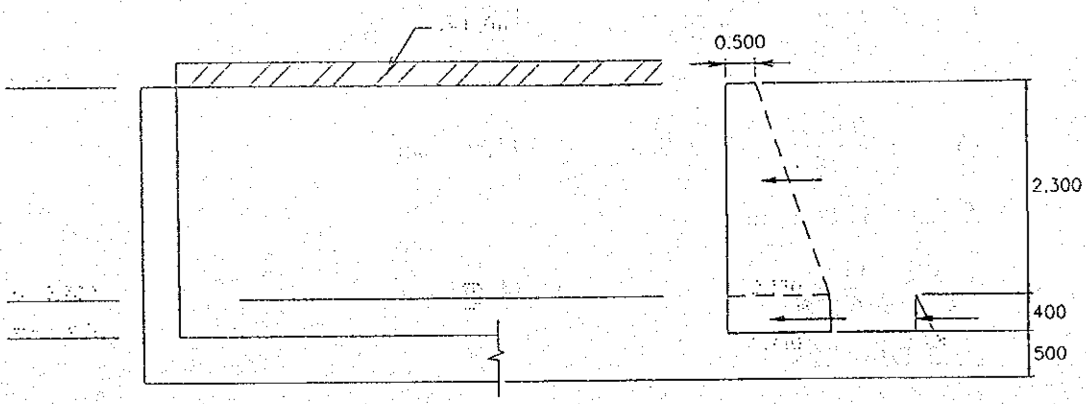
$$i = \frac{1}{1-0.932\cdot\frac{0.76}{-1.017}} = 0.590$$

$$iA = \frac{0.0.2614}{15} \times 76 \times 100 = 13.244 \text{ cm}^2$$

$$A = \frac{13.244}{0.590} \quad 22.447 \text{ cm}^2$$

Used D22 a 125 → A = 30.4106 cm²

4.3.2.4 Retaining Wall Type III (Breast Wall at Semarang River Side)



Wall

- $P_1 = \frac{0.5+2.57}{2} \times 2.3$ = 3.530 tf

  $Z_1 = 0.4 + \frac{2 \times 0.5 + 2.57}{3(0.5+2.57)} \times 2.3$ = 1.292 m

  M_1 = 3.5305 x 1.292 = 4.561 tf.m

- $P_2 = \frac{2.54+2.74}{2} \times 0.4$ = 1.056 tf

  $Z_2 = \frac{2 \times 2.54 + 2.74}{3(2.54+2.74)} \times 0.4$ = 0.193 m

  M_2 = 1.0.56 x 0.193 = 0.204 tf.m

- P_3 = ½ x 0.4^2 = 0.08 tf

  Z_3 = 1/3 x 0.4 = 0.1333 m

  M_3 = 0.08 x 0.1333 = 0.0107 tf.

  ΣP = 3.530 + 1.056 + 0.08 = 4.666 tf

  ΣM = 4.561 + 0.204 + 0.0107 = 4.776 tf.m

Shear Stress

$$\tau = \frac{4666}{7/8 \times 100 \times 41} = 1.3006 \text{ kg f/cm}^2 < \bar{\tau} = 6.5 \text{ kgf/cm}^2$$

$$M = 4.776 \text{ tfm}$$

$$Ca = \frac{31}{\sqrt{\frac{15 \times 477600}{100 \times 1600}}} = 4.632$$

$$\left.\begin{array}{l} Ca = 4.632 \\ \delta = 0 \end{array}\right\} \quad \begin{array}{l} \phi = 2.662 \\ nw = 0.0513 \\ \zeta = 0.909 \end{array}$$

$$A = \frac{0.0563}{15} \text{ x } 100 \text{ x } 31 = 6.5111 \text{ cm}^2$$

Used D13 – 125 → $A = 10.619 \text{ cm}^2$

<u>For Bottom Slab</u>

- $P_1 = 3.5305$ tf
  $Z_1 = 0.25 + 1.292 = 1.542$ m
  $M_1 = 3.5305 \text{ x } 1.542 = 5.444$ tf.m

- $P_2 = 1.056$ tf
  $Z_2 = 0.25 + 0.193 = 0.443$ m
  $M_2 = 1.056 \text{ x } 0.443 = 0.468$ tf.m

- $P_3 = 0.08$ tf
  $Z_3 = 0.133 + 0.25 = 0.383$ m
  $M_3 = 0.08 \text{ x } 0.3833 = 0.031$ tf.

$\Sigma P = 3.505 + 1.056 + 0.08 = 17.111$ tf

$\Sigma M = 5.444 + 0.468 + 0.031 = 5.943$ tf.m

$M = 5.943$ tfm

$N = -4.666$ tf

$$e = \frac{5.943}{-4.666} = -1.274 \text{ m}$$

$$e_s = -1.274 + \frac{0.50}{2} - 0.09 = -1.114 \text{ m}$$

Ne_a = −1.114 x −4.666 = 5.198 tf.m

$$Ca = \frac{41}{\sqrt{\dfrac{15 \times 519800}{100 \times 1600}}} = 5.873$$

Ca = 5.873 ; δ = 0 } ϕ = 3.528 ; nw = 0.031 ; ζ = 0.926

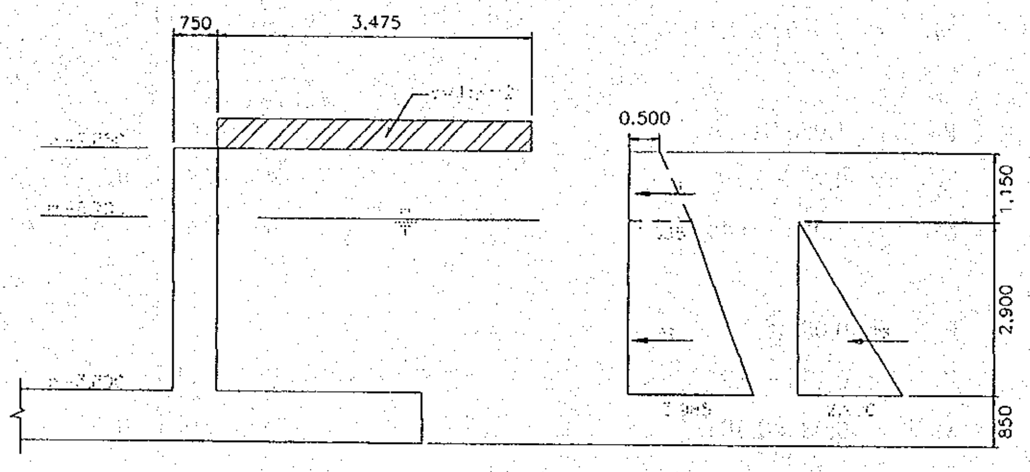
$$i = \frac{1}{1 - 0.926 \cdot \dfrac{0.41}{-1.114}} = 0.746$$

$$IA = \frac{8.554}{15} \times 100 \times 41 = 8.554 \text{ cm}^2$$

$$A = \frac{8.554}{0.746} = 11.471 \text{ cm}^2$$

Used D16 – 12.50 → A = 16.085 cm^2

4.3.2.5 Retaining Wall Type IV (Side Wall)



- $$P_1 = \frac{0.5 + 1.535}{2} \times 1.150 = 1.170 \text{ tf}$$

$$Z_1 = 2.90 + \frac{2 \times 0.5 + 1.535}{3(0.5 + 1.535)} \times 1.150 = 3.374 \text{ m}$$

$$M_1 = 1.170 \times 3.374 = 3.948 \text{ tf.m}$$

- $$P_2 = \frac{1.535 + 3.985}{2} \times 2.90 = 6.554 \text{ tf}$$

$Z_2 = \frac{2 \times 1.535 + 2.985}{3(1.535 + 2.985)} \times 2.9 = 1.295$ m

$M_2 = 1.295 \times 6.554 = 7.899$ tf.m

- $P_3 = ½ \times 2.90^2 = 4.205$ tf

$Z_3 = \frac{2.9}{3} = 0.967$ m

$M_3 = 4.205 \times 0.967 = 4.066$ tf.

$\Sigma M = 3.948 + 8.487 + 4.066 = 16.501$ tf

$Ca = \frac{66}{\sqrt{\frac{15 \times 1606100}{100 \times 1600}}} = 5.379$

$Ca = 5.379$, $\delta = 0$ } $\phi = 3.184$, $nw = 0.03753$

$A = \frac{0.03753}{25} \times 100 \times 66 = 16.5131$ cm²

Used D19 a 125 → A = 22.632 cm²

<u>For Slab</u>

- $P_1 = 1.170$ tf

$Z_1 = 0.425 + 3.347 = 3.772$ m

$M_1 = 1.170 \times 3.772 = 4.413$ tf.m

- $P_2 = 6.557$ tf

$Z_2 = 0.425 + 1.295 = 1.720$ m

$M_2 = 6.557 \times 1.720 = 11.278$ tf.m

- $P_3 = 4.203$ tf

$Z_3 = 0.425 + 0.967 = 1.392$ m

$M_3 = 1.392 \times 4.203 = 5.851$ tf.

$\Sigma M = 4.413 + 11.278 + 5.851 = 21.542$ tf.m

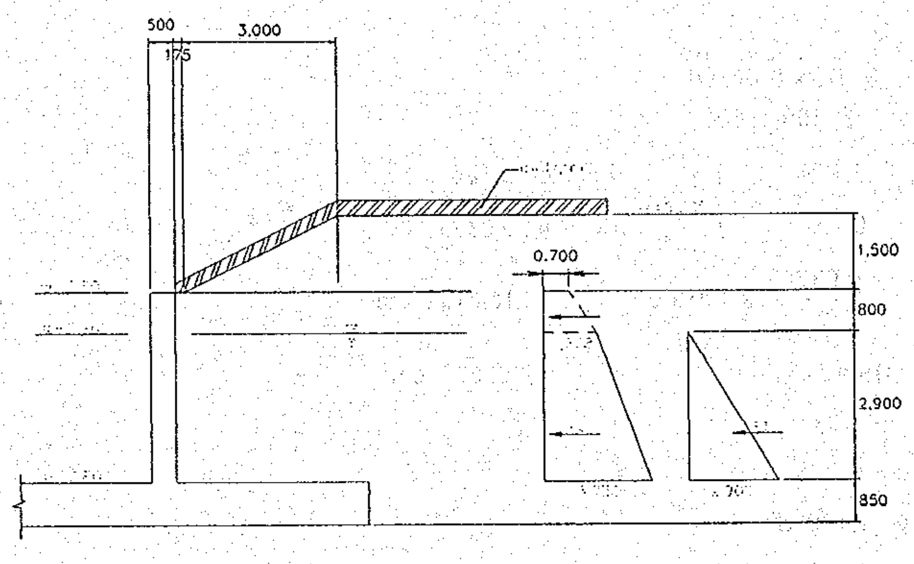
$$Ca = \frac{76}{\sqrt{\frac{15 \times 2154200}{100 \times 1600}}} = 5.380$$

$Ca = 5.380$, $\delta = 0$ } $\phi = 3.184$, $nw = 0.03753$

$$A = \frac{0.03755}{15} \times 100 \times 76 = 19.025 \ cm^2$$

Used D19 a 125 → $A = 22.682 \ cm^2$

4.3.2.6 Retaining Wall Type V (Breast Wall at Semarang River Side)



Assume the active earth coefficient K is 0.70

- $P_1 = \frac{0.70 + 1.708}{2} \times 0.80 = 0.963$ tf

  $Z_1 = \frac{2 \times 0.70 + 1.708}{3(0.70 + 1.708)} \times 0.8 + 2.90 = 3.244$ m

  $M_1 = 3.244 \times 0.9632 = 3.124$ tf.m

- $P_2 = \frac{1.708 + 3.738}{2} \times 2.90 = 7.897$ tf

  $Z_2 = \frac{2 \times 1.708 + 3.738}{3(1.708 + 3.738)} \times 2.90 = 1.270$ m

  $M_2 = 7.897 \times 1.270 = 10.029$ tf.m

- $P_3 = ½ \times 2.9^2 = 4.205$ tf

$Z_3 = \frac{2.9}{3} = 0.967$ m

$M_3 = 4.205 \times 0.967 = 4.065$ tf.

$\Sigma P = 0.9632 + 7.897 + 4.205 = 13.065$ tf

$\Sigma M = 3.224 + 10.029 + 4.066 = 17.21900$ tf.m

Shear Stress

$\tau = \frac{13065}{7/8 \times 100 \times 41} = 3.892$ kg f/cm^2 $< \bar{\tau} = 6.5$ kgf/cm^2

$Ca = \frac{41}{\sqrt{\frac{15 \times 1721900}{100 \times 1600}}} = 3.227$

$\left.\begin{array}{l} Ca = 3.227 \\ \delta = 0 \end{array}\right\} \begin{array}{l} \phi = 1695 \\ nw = 0.1094 \end{array}$

$A = \frac{0.1094}{15} \times 100 \times 41 = 29.9030$ cm^2

Used D22 – a125

Slab

- $P_1 = 0.963$ tf
  $Z_1 = 0.425 + 3.244 = 3.699$ m
  $M_1 = 0.963 \times 3.669 = 3.533$ tf.m

- $P_2 = 7.897$ tf
  $Z_2 = 1.270 + 0.425 = 1.695$ m
  $M_2 = 7.897 \times 1.695 = 13.385$ tf.m

- $P_3 = 4.205$ tf
  $Z_3 = 0.967 + 0.425 = 1.392$ m
  $M_3 = 1.392 \times 4.205 = 5.853$ tf.

$M = 3.533 + 13.385 + 5.853 = 22.771$ tfm

$P = 13.065$

$e = \frac{22.771}{-13.065} = -1.743$ m

$$e_a = -1.743 + \frac{0.85}{2} - 0.09 = -1.408 \text{ m}$$

$$Ne_a = -1.408 \text{ x } -13.065 = 18.3955 \text{ tf.m}$$

$$Ca = \frac{76}{\sqrt{\frac{15 \times 1839550}{100 \times 1600}}} = 5.787$$

$$\left.\begin{array}{ll} Ca & = 5.787 \\ \delta & = 0 \end{array}\right\} \begin{array}{ll} \phi & = 3.464 \\ nw & = 0.0323 \\ \zeta & = 0.925 \end{array}$$

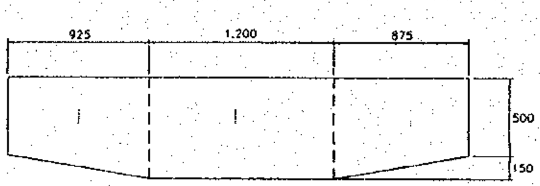
$$i = \frac{1}{1 - 0.925 \cdot \frac{0.76}{-1.408}} = 0.777$$

$$iA = \frac{0.0323}{15} \text{ x } 76 \text{ x } 100 = 16.365 \text{ cm}^2$$

$$A = \frac{16.363}{0.777} = 23.379 \text{ cm}^2$$

Used D22 a 125

4.3.2.7 Control Tower



$$\text{Weight of concrete I} = 0.925 \text{ x } \frac{(0.65+0.5)}{2} \text{ x } 2.5 = 1.329 \text{ tf/m}$$

$$\text{Weight of concrete II} = 0.1.20 \text{ x } \frac{0.65}{2} \text{ x } 2.5 = 1.950 \text{ tf/m}$$

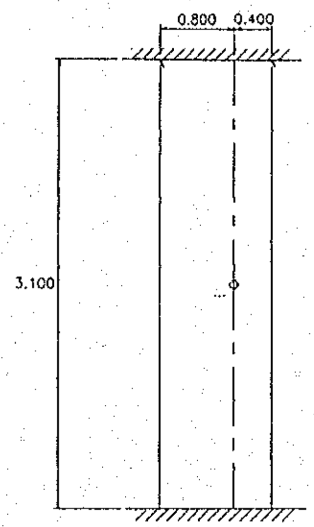
$$\text{Weight of concrete III} = 0.875 \text{ x } \frac{(0.65+0.5)}{2} \text{ x } 2.5 = 1.258 \text{ tf/m}$$

$$\text{Weight of live load} = 3 \text{ x } 0.3 = 0.900 \text{ tf/m}$$

$$= 5.437 \text{ tf/m}$$

Weight of gate = 2.5 t

$$M_o = \tfrac{1}{8} \text{ x } 5.437 \text{ x } 3.1^2 + \tfrac{1}{4} \text{ x } 2.5 \text{ x } 3.1 = 8.469 \text{ tfm}$$



$$Sa = \frac{a + \frac{1}{3} re_x}{e_y + \frac{1}{3} re_x} \cdot e_y + v$$

$$Sa = \frac{0 + \frac{1}{3} \times 1 \times 3.10}{1.20 + \frac{1}{3} \times 3.10} \cdot 1.2 + 0.80$$

$$= 1.355 \text{ m}$$

Where,

$r = 1$

$$Ca = \frac{56}{\sqrt{\frac{15 \times 846900}{135 \times 1800}}} = 7.795$$

$$\left.\begin{array}{ll} Ca & = 7.795 \\ \delta & = 0 \end{array}\right\} \quad \begin{array}{ll} \phi & = 4.848 \\ nw & = 0.01764 \end{array}$$

$$A = \frac{0.01764}{15} \times 100 \times 56 = 6.586 \text{ cm}^2$$

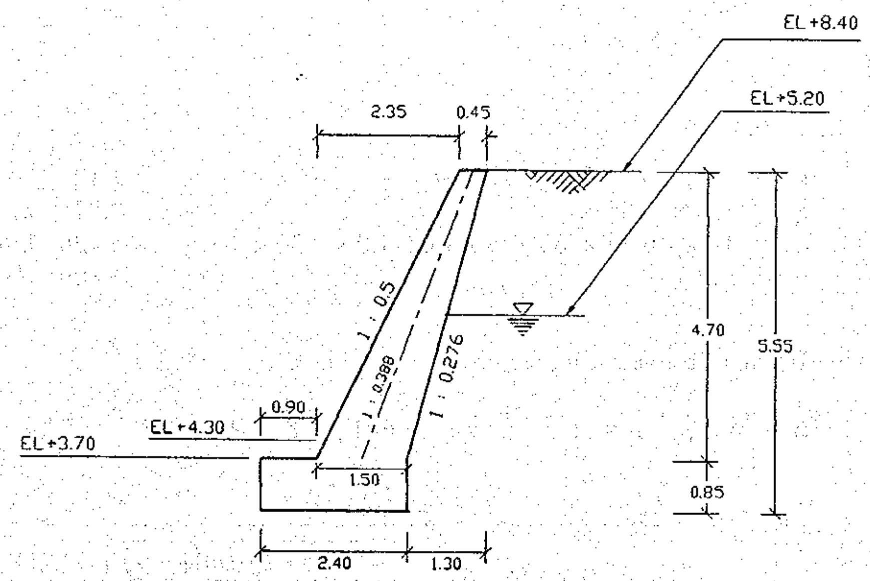
Used D13 – a125 → A = 10.619 cm^2

4.4 Connecting Wall, Revetment, Riverbed Protection, etc.

4.4.1 Connecting Wall at Intake Structure

(1) Design Condition

(a) Proposed Section



(b) The Material Data

(i) Wet Stone Masonry

Unit Weight	γ_m	=	2.30	t/m^3
Allowable Compressive stress	σ_{ca}	=	15	kg/cm^2
Allowable shear strength	τca	=	4.5	kg/cm^2

(ii) Soil Material
(Base on geological investigation on SB 3)

N – value	N	=	41	
Soil internal friction angle	ϕ	=	40°	
Wet unit weight	γ_s	=	2.0	t/m^3
Submerged unit weight	γ'_s	=	1.0	t/m^3
Cohesion	c	=	0.0	t/m^2

(c) Design Load

(i) Earth Pressure

Coulomb Formula

Friction angle at wall $\delta = \phi = 40°$ (in ordinary)

Angle between back side of wall & vertical plane $\phi = -15.461°$

(atn. 1.3/4.7)

(ii) Surcharge Load $q = 1.00\ t/m^2$

(2) Allowable bearing capacity beneath slab

$$qu = \alpha \cdot k \cdot c \cdot Nc + k \cdot g \cdot Nq + \frac{1}{2} \cdot \gamma \cdot \beta \cdot B \cdot N_\gamma$$

$$k = 1 + 0.3\ Df/B$$

$$q = \gamma \cdot Df$$

For $\phi = 40$, $N_c = 75$

$N_g = 63$

$N_\gamma = 83$

Df = 0.85 m

$k = 1 + 0.3 \times 0.85/2.15 = 1.12$

$q = 1.0 \times 0.85 = 2.85\ t/m^2$

$q_u = 1.12 \times 0.85 \times 63 + ½ \times 1.0 \times 1.0 \times 2.30 \times 83 = 155.426\ t/m^2$

$q_a = q_u / 3 = 55.809\ t/m^2$

(3) Coefficient of active earth pressure

$\phi = 40°$, $\alpha = 0$, $\theta = -15.641$, $\delta = \phi = 40°$

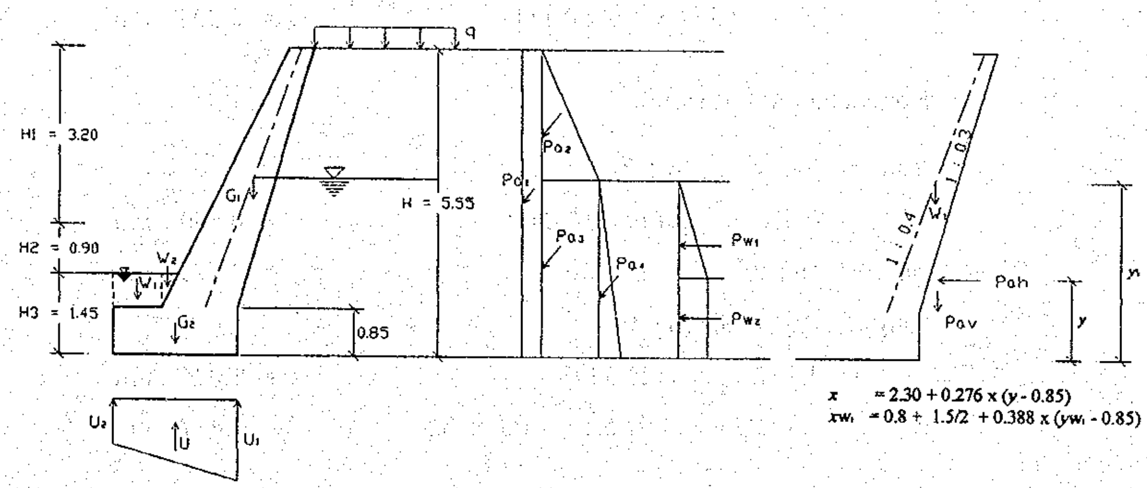
$$Ka = \frac{\cos^2(\phi-\theta)}{\cos^2\theta \cdot \cos(\phi+\delta) \cdot \left[1+\sqrt{\frac{(\sin\phi+\delta)\cdot\sin(\phi-\alpha)}{\cos(\theta+\delta)\cdot\cos(\theta-\alpha)}}\right]^2}$$

$$= \frac{\cos^2(40+16.7)}{\cos^2(-16.7)\times\cos(-16.7+40)\times\left[1+\sqrt{\frac{\sin(40+40)\times\sin(40)}{\cos(-16.7+40)\times\cos(-16.7)}}\right]^2}$$

$= 0.111$

(4) Stability Analysis

(i) Load Condition



y_1	=	0.85 + (2 x 0.45 + 1.50)/1.95 x 4.7/3	= 2.778	m
x_1	=	0.80 + 1.50/2 + 0.388 (2.778 – 0.85)	= 2.298	m
G_2	=	0.85 x 2.30 x 2.30	= 4.497	t/m
y_2	=	0.85 / 2	= 0.425	m
x_2	=	2.30 / 2	= 1.150	m

(iii) Uplift Pressure

U_1	=	(EL+5.20) – (EL+3.70) + 0.85	= 2.350	t/m^2
U_2	=	(EL+4.30) – (EL+3.70) + 0.85	= 1.450	t/m^2
U	=	(U1 + U2) x 8/2 x γ_w		
	=	(3.35 + 1.45) x 2.30/2	= 4.370	t/m
x_u	=	2.30 – (2.35 + 2 x 1.45) / 3.80 x 2.3/3	= 1.241	m

(iv) Earth Pressure

Pa_1	=	q · H · Ka		
	=	1.0 x 5.55 x 0.111	= 0.616	t/m^2

$Pa_2 = ½ \cdot H_1^2 \cdot \gamma_s \cdot Ka$

$= ½ \times (3.2)^2 \times 2.0 \times 0.111 = 1.137$ t/m²

$Pa_3 = H_1 \cdot \gamma_s \cdot Ka \cdot (H_2 + H_3)$

$= 3.2 \times 2.0 \times 0.111 \times 2.35 = 1.669$ t/m²

$Pa_4 = ½ \cdot (H_2 + H_3)^2 \cdot \gamma'_s \cdot Ka$

$= ½ \times (2.35)^2 \times 1.0 \times 0.111 = 0.306$ t/m²

Horizontal Earth Pressure

$Pah_1 = Pa_1 \times \cos(\delta + \theta)$

$= 0.616 \times \cos(40 - 15.641) = 0.560$ t/m

$ya_1 = H/2 = 5.55/2 = 2.775$ m

$Pah_2 = Pa_2 \times \cos(\delta + \theta)$

$= 1.137 \times \cos(24.539) = 1.034$ t/m

$ya_2 = H - 2/3\ H_1 = 5.55 - 2/3 \times 3.2 = 3.417$ m

$Pah_3 = Pa_3 \times \cos(\delta + \theta)$

$= 1.669 \times \cos(24.539) = 1.519$ t/m

$ya_3 = (H_2 + H_3)/2 = 2.35/2 = 1.175$ m

$Pah_4 = Pa_4 \times \cos(\delta + \theta)$

$= 0.360 \times \cos(24.539) = 0.279$ t/m

$ya_4 = (H_2 + H_3)/3 = 2.35/3 = 0.783$ m

Vertical Earth Pressure

$Pav_1 = Pa_1 \times \sin(\delta + \theta)$

$= 0.616 \times \sin(24.539) = 0.256$ t/m

$xa_1 = 2.30 + 0.276\ (ya_1 - 0.85) = 2.831$ m

$Pav_2 = Pa_2 \times \sin(\delta + \theta)$

$= 1.137 \times \sin(24.539) = 0.472$ t/m

$xa_2 = 2.30 + 0.276\ (ya_2 - 0.85) = 3.008$ m

$Pav_3 = Pa_3 \times \sin(\delta + \theta)$

$= 1.669 \times \sin(24.539) = 0.693$ t/m

$xa_3 = 2.30 + 0.276\ (ya_3 + 0.85) = 2.390$ m

$Pav_4 = Pa_4 \times \sin(\delta + \theta)$

$= 0.306 \times \sin(24.539) = 0.127$ t/m

$xa_4 = 2.30 + 0.276\ (ya_4 - 0.85) = 2.300$ m

Passive earth pressure : to be omitted

(v) Water Pressure

Vertical water pressure

$W_1 = 0.80 \times 0.6 \times 1.0 = 0.480$ t/m

$x_{w1} = 0.80/2 = 0.400$ m

$y_{w1} = 0.85 + 0.6/2 = 1.150$ m

$W_2 = ½ \times 0.6 \times (0.6/2) \times 1.0 = 0.090$ t/m

$x_{w2} = 0.80 + 0.3/3 = 0.900$ m

$y_{w2} = 1.45 - 0.6/3 = 1.250$ m

Vertical water pressure

$P_{w1} = ½ (H_2)^2 \cdot \gamma_w$

$= ½ \times (0.9)^2 \times 1.0 = 0.405$ t/m

$y_{w1} = H_3 + H_2/3 = 1.45 + 0.9/3 = 1.750$ m

$P_{w2} = H_2 \cdot \gamma_w \cdot H_3$

$= 0.9 \times 1.0 \times 1.45 = 1.305$ m

$y_{w2} = H_3/2 = 1.45/2 = 0.725$ m

(vi) Vertical Force and Moment

Vertical Force (t/m)		Arm (m)	Moment (tm/m)
G_1	10.540	2.298	24.221
G_2	4.497	1.150	5.172
W_1	0.480	0.400	0.192
W_2	0.090	0.900	0.081
Pav_1	0.256	2.831	0.725
Pav_2	0.472	3.008	1.420
Pav_3	0.693	2.390	1.656
Pav_4	0.127	2.300	0.292
U	-4.370	1.241	-5.423
FV	12.785	MV	28.335

(vii) Horizontal Force and Moment

Horizontal Force (t/m)		Arm (m)	Moment (tm/m)
Pah_1	0.560	2.775	1.554
Pah_2	1.034	3.417	3.533
Pah_3	1.519	1.175	1.785
Pah_4	0.279	0.783	0.218
Pw_1	0.405	1.750	0.709
Pw_2	1.305	0.725	0.946
FH	5.102	MH	8.745

(viii) Check of Stability

Stability against tilting

$SF = FV / FH \cdot \tan \phi$

$= 12.785/5.102 \times \tan 40 \qquad = 2.103$

> 1.5

Stability against overturning

$e = B/2 - (MV - MH)/FV$

$= 2.30/2 - (28.335 - 8.745)/12.785 \qquad = 0.382 \text{ m } (-)$

$< {}^{B}/_{6} \ (=0.383 \text{ m})$

Stability of bearing strata

$q = FV / B\ (1 \pm 6e/B)$

$q_{max} = 12.785/_{2.30} \times (1 + 6 \times 0.382/2.30) \qquad = 11.098 \text{ t/m}^2$

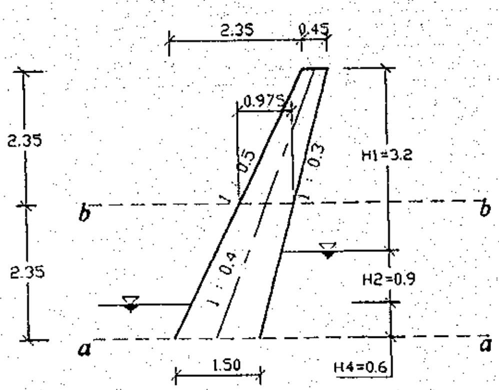
$< q_a\ (= 51.809 \text{ t/m}^2)$

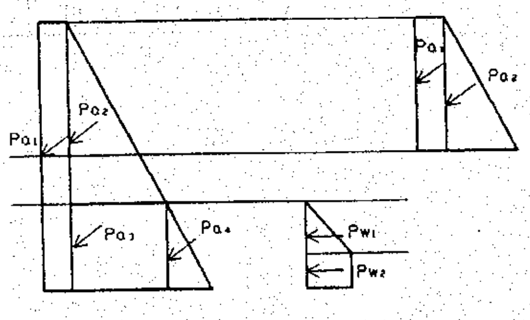
$q_{min} = 12.785/_{2.30} \times (1 + 6 \times 0.382/2.30) \qquad = 0.019 \text{ t/m}^2$

> 0

(5) Stress Strain Analysis of Wall

(i) Load Condition





Section a Section b

(ii) Load Condition

Section a

W_a	=	(0.45 + 1.50) x 4.70/2 x 2.30	= 10.540	t/m
y_a	=	(2 x 0.45 + 1.5)/1.95 x 4.7/3	= 1.928	m
x_a	=	1.928 x 0.388 + 1.50/2	= 1.498	m

Section a

W_b	=	(0.45 + 0.975) x 2.35/2 x 2.30	= 3.851	t/m
y_b	=	(2 x 0.45 + 0.975)/1.425 x 2.35/3	= 1.031	m
x_b	=	1.031 x 0.388 + 0.975/2	= 0.877	m

(iii) Load Condition

Section a

Pa_1 a	=	$q \cdot Ka \cdot H$		
	=	1.0 x 0.111 x 4.7	= 0.522	t/m
Pa_2 a	=	$\frac{1}{2}\ H_1^2 \cdot \gamma_s \cdot Ka$		
	=	$\frac{1}{2}$ x $(3.2)^2$ x 2.0 x 0.111	= 1.137	t/m
Pa_3 a	=	$H_1 \cdot \gamma_s \cdot Ka \cdot (H_2 + H_4)$		
	=	3.2 x 2.0 x 0.111 x 1.5	= 1.066	t/m
Pa_4 a	=	$\frac{1}{2}\ (H_2 + H_4)^2 \cdot \gamma_s' \cdot Ka$		
	=	$\frac{1}{2}$ x $(1.5)^2$ x 1.0 x 0.111	= 0.125	t/m

Horizontal earth pressure

Pah_1 a	=	**Pa_1 a x cos ($\delta + \theta$)**		
	=	**0.522 x cos (24.539)**	= 0.475	t/m
Pah_2 a	=	**1.137 x cos (24.539)**	= 1.034	t/m
Pah_3 a	=	**1.066 x cos (24.539)**	= 0.969	t/m
Pah_4 a	=	**0.125 x cos (24.539)**	= 0.114	t/m

Vertical earth pressure

Pav_1 a	=	**Pa_1 a x sin ($\delta + \theta$)**		
	=	**0.522 x sin (24.539)**	= 0.217	t/m
Pav_2 a	=	**1.137 x sin (24.539)**	= 0.472	t/m
Pav_3 a	=	**1.066 x sin (24.539)**	= 0.443	t/m
Pav_4 a	=	**0.125 x sin (24.539)**	= 0.052	t/m

Section b

$Pa_1\, b = q \cdot Ka \cdot H$

$= 1.0 \times 0.111 \times 2.35 \quad = 0.261$ t/m

$Pa_2\, b = ½\, H_1^2 \cdot \gamma_s \cdot Ka$

$= ½ \times (2.35)^2 \times 2.0 \times 0.111 \quad = 0.613$ t/m

Horizontal earth pressure

$Pah_1\, b = 0.261 \times \cos(24.539) \quad = 0.237$ t/m

$Pah_2\, b = 0.613 \times \cos(24.539) \quad = 0.558$ t/m

Vertical earth pressure

$Pav_1\, b = 0.261 \times \sin(24.539) \quad = 0.108$ t/m

$Pav_2\, b = 0.613 \times \sin(24.539) \quad = 0.255$ t/m

(iv) Water Pressure

(v) Vertical Force and Moment

at section a

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
W_a	10.540		1.498	15.789
P_{av1a}	0.217	$^{4.7}/_2$ x 0.276 + 1.50	2.149	0.466
P_{av2a}	0.472	(1.5+ 3.2/3) x 0.276 + 1.50	2.208	1.042
P_{av3a}	0.443	$^{1.5}/_2$ x 0.276 + 1.50	1.707	0.756
P_{av4a}	0.052	$^{1.5}/_3$ x 0.276 + 1.50	1.638	0.085
FV_a	11.724		MV_a	18.139

at section b

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
W_b	3.851		0.877	3.377
P_{av1b}	0.108	$^{2.35}/_2$ x 0.276 + 0.975	1.299	0.140
P_{av2b}	0.255	$^{2.35}/_3$ x 0.276 + 0.975	1.191	0.304
FV_b	4.214		MV_b	3.821

(vi) Horizontal Force and Moment

at section a

Horizontal Force (t/m)		Arm (m)		Moment (tm/m)
P_{av1a}	0.475	$^{4.7}/_2$	2.350	1.116
P_{av2a}	1.034	1.5+ 3.2/3	2.567	2.654
P_{av3a}	0.969	$^{1.5}/_2$	0.750	0.727
P_{av4a}	0.114	$^{1.5}/_3$	0.500	0.057
FH_a	2.592		MV_a	4.554

at section b

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
P_{av1b}	0.237	$^{2.35}/_2$	1.175	0.278
P_{av2b}	0.558	$^{2.35}/_3$	0.785	0.437
FV_b	0.795		MV_b	0.715

(vii) Check of Stresses

$e \quad = \quad B/2 - (MV - MH)/FV$

$q \quad = \quad FV/b \cdot (1 \pm 6e/B) \;\; \text{for} \;\; e < B/6$

at section a.

$e \quad = \quad 1.50/2 - (18.139 - 4.554)/11.724 = \quad 0.409 \;\; \text{m}$

$q_{max} \quad = \quad 11.734/1.5 \times (1 + 6 \times 0.409/1.5) = \quad 20.603 \;\; \text{t/m}^2$

$= \quad 2.060 \;\; \text{kg/cm}^2$

$< \; \sigma_{ca}(= 15 \; \text{kg/cm}^2)$

at section b.

$e \quad = \quad 0.975/2 - (3.821 - 0.715)/4.214 \quad = \quad 0.250 \;\; \text{m}$

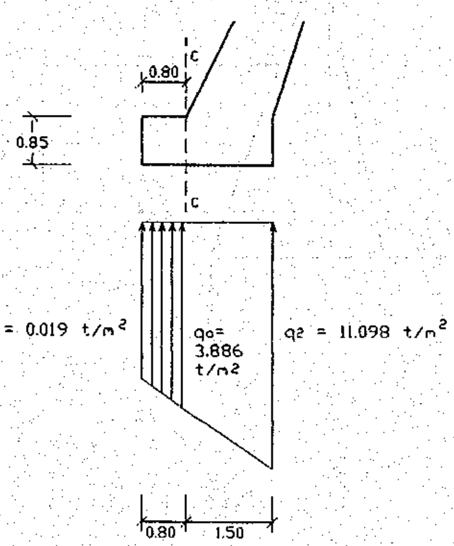
$q_{max} \quad = \quad 4.214/0.975 \times (1+6 \times 0.250/0.975) \quad = 10.971 \;\; \text{t/m}^2$

$= \quad 1.097 \;\; \text{kg/cm}^2$

$< \sigma_{ca}(= 15 \; \text{kg/cm}^2)$

(6) Stress strain analysis of slab

(i) Load Condition



(ii) Weight of Structure

$$W = 0.8 \times 0.85 \times 2.3 = 1.564 \text{ t/m}$$
$$M_w = 1.564 \times 0.8/2 = 0.626 \text{ tm/m}$$

(iii) Reaction Beneath Slab

$$Q = (0.019 + 3.886)/2 \times 0.8 = 1.562 \text{ t/m}$$
$$x_q = (2 \times 0.019 + 3.886)/3.905 \times 0.8/3 = 0.268 \text{ m}$$
$$M_q = 1.562 \times 0.268 = 0.419 \text{ t/m}$$

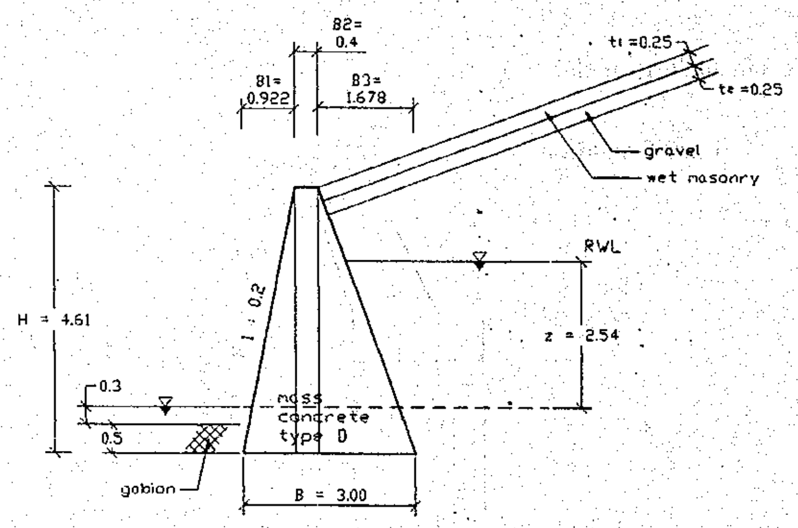
(iv) Check of Stress

$$M = Mq - M_W = 0.626 - 0.419 = 0.207 \text{ tm/m}$$
$$S = Q - W = 1.564 - 1.562 = 0.002 \text{ t/m}$$
$$I = bh^3 / 12 = 100 \times 85^3 / 12 = 5{,}117{,}708.000 \text{ cm}^4$$
$$Z = I / y = 5{,}117{,}708 / 42.5 = 120{,}416.667 \text{ cm}^3$$
$$\sigma_c = M / Z = 20{,}700 / 120{,}416.667 = 0.172 \text{ kg/cm}^2 < \sigma_{ca} (= 15 \text{ kg/cm}^2)$$
$$\tau_c = S / A = 2 / (100 \times 85) = 0.0002 \text{ kg/cm}^2 < \tau_{ca} (= 4.5 \text{ kg/cm}^2)$$

4.4.2 Earth Retaining Wall at Downstream of Simongan Weir

(1) Design Condition

(a) Proposed Section



(b) The Material Data

(i) Soil Material
(Base on geological investigation on RB 30)

N – value	N	=	17	
Soil internal friction angle	ϕ	=	31	°
Wet Unit weight	γ_s	=	1.8	t/m^3
Submerged unit weight	γ'_s	=	1.0	t/m^3
Cohesion	c	=	0.0	t/m^2

(ii) Others

Unit weight of concrete	γ_c	=	2.35	t/m^3
Unit weight of wet stone masonry	γ_m	=	2.30	t/m^3
Unit weight of gravel	γ_g	=	2.00	t/m^3
Unit weight of water	γ_w	=	1.00	t/m^3

(c) Design Load

(i) Earth Pressure

Coulomb Formula

Friction angle at wall	δ	=	ϕ	(in ordinary)
	δ	=	$\phi/2$	(in seismic)
Angle between back side of wall and vertical plane	ϕ	=	20°	(atn. $^{1.678}/_{4.61}$)
Angle between ground surface and horizontal plane	α	=	26.56°	(atn 0.5)

(ii) Surcharge due to revetment

$dh = (t1 \times \gamma m + t2 \times \gamma g)/\gamma s - (t1 + t2) =$ 0.097 ~ 0.10 m.

(2) Allowable bearing capacity beneath slab

$$qu = \alpha \cdot k \cdot c \cdot Nc + k \cdot g \cdot Ng + \tfrac{1}{2} \cdot \gamma \cdot \beta \cdot B \cdot N_\gamma$$
$$k = 1 + 0.3\ Df/B$$
$$q = \gamma \cdot Df$$

For ϕ = 31, N_c = 32
N_g = 20
N_γ = 16

Df = 0.5 m

$k = 1 + 0.3 \times 0.5/3.0 = 1.05$
$q = 1.0 \times 0.3 + 2.0 \times 0.5 = 0.5\ t/m^2$
$q_u = 1.05 \times 1.3 \times 20 + \tfrac{1}{2} \times 1.0 \times 1.0 \times 3.0 \times 16 = 51.3\ t/m^2$
$q_a = q_u / 3 = 17.100\ t/m^2$ (in ordinary)
$q_{ea} = q_u / 2 = 25.650\ t/m^2$ (in seismic)

(3) Coefficient of active earth pressure

(in ordinary)

ϕ = 31 °, α = 26.56 , θ = 20°, $\delta = \phi$ = 31°

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

$$= \frac{\cos^2(31-20)}{\cos^2 20\times\cos(20+31)\left[1+\sqrt{\frac{\sin(31+31)\times\sin(31-26.56)}{\cos(20+31)\times\cos(20-26.56)}}\right]^2}$$

= 0.979

(in seismic)

ϕ = 31 °, α = 26.56 , θ = 20°, $\delta = \phi/2$ = 15.5 °

Kh' =	2.0/1.0 x 0.12	=	0.24
θ_o =	atn 0.12	=	6.84°
θ'_o =	atn 0.24	=	13.5°

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

$$= \frac{\cos^2(31-6.84-20)}{\cos(6.84)\times\cos^2(20)\times\cos(20+6.84+15.5)\left[1+\sqrt{\frac{\sin(31+15.5)\times\sin(31-26.56-6.84)}{\cos(20+6.84+15.5)\times\cos(20-26.56)}}\right]^2}$$

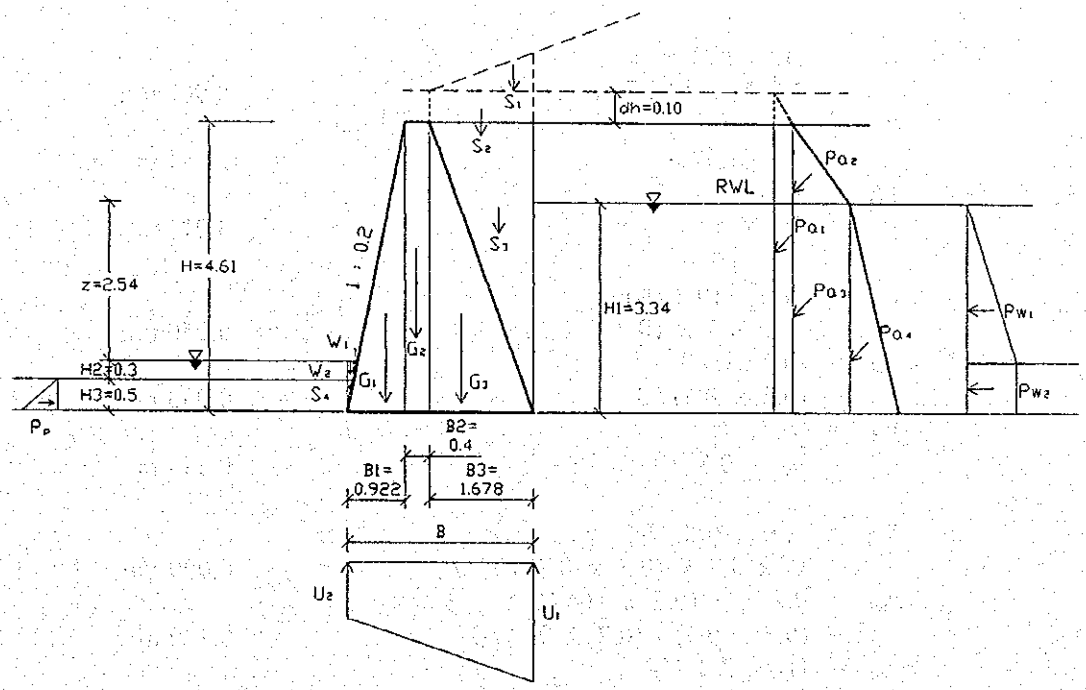
= 1.535

$$K'l_e = \frac{\cos^2(31-13.5-20)}{\cos(13.5)\times\cos^2(20)\times\cos(20+13.5+15.5)\left[1+\sqrt{\frac{\sin(31+15.5)\times\sin(31-26.56-13.5)}{\cos(20+13.5+15.5)\times\cos(20-26.56)}}\right]^2}$$

= 1.839

(4) Stability Analysis

(i) Load Condition



(ii) Center Weight of Structure

G_1	$= B_1 \times H/2 \times \gamma_c$			
	$= 0.992 \times 4.61/2 \times 2.35$		=	4.994 t/m
x_{g1}	$= 2/3\ B_1$	$= 2/3 \times 0.922$	=	0.615 m
y_{g1}	$= H/3$	$= 4.61/3$	=	1.537 m
G_2	$= B_2 \times H \times \gamma_c$			
	$= 0.40 \times 4.61 \times 2.35$		=	4.333 t/m
x_{g2}	$= B_1 + B_2/2$	$= 0.922 + 0.4/2$	=	1.122 m
y_{g2}	$= H/2$	$= 4.61/2$	=	2.305 m
G_3	$= B_3 \times H/2 \times \gamma_c$			
	$= 0.678 \times 4.61/2 \times 2.35$		=	9.089 t/m
x_{g3}	$= B_1 + B_2 + B_3/3$	$= 0.922 + 0.4 + 1.678/3$	=	1.881 m
y_{g3}	$= H/3$	$= 4.61/3$	=	1.537 m
S_1	$= ½ \times B_3 \times B_3/2 \times \gamma_s$			
	$= ½ \times (1.678)^2/2 \times 1.80$		=	1.267 t/m
x_{s1}	$= B1 + B2 + {}^2/_3\ B_3$			
	$= 0.922 + 0.4 + 2/3 \times 1.678$		=	2.441 m
y_{s1}	$= H + dh + B_3/2/3$			
	$= 4.61 + 0.10 + 1.678/6$		=	4.990 m

$S_2 = dh \times B_3 \times \gamma_s$
$= 0.10 \times 1.678 \times 1.8 = 0.302$ t/m
$x_{s2} = B_1 + B_2 + B_3/2$
$= 0.922 + 0.4 + 1.678/2 = 2.161$ m
$y_{s2} = H + dh/2$
$= 4.61 + 0.10/2 = 4.660$ m

$S_3 = B_3 \times H/2 \times \gamma_s$
$= 1.678 \times 4.61/2 \times 1.8 = 6.962$ t/m
$x_{s3} = B - B_3/3$
$= 3.0 - 1.678/3 = 2.441$ m
$y_{s3} = {}^2/_3 H$
$= 2/3 \times 4.61 = 3.073$ m

$S_4 = ½ H_3 \cdot (0.2\, H_3) \cdot \gamma_s$
$= ½ \times 0.5 \times (0.2 \times 0.5) \times 2.0 = 0.050$ t/m
$x_{s4} = (0.2\, H_3) / 3$
$= (0.2 \times 0.5) / 3 = 0.033$ m
$y_{s4} = {}^2/_3 H_3$
$= {}^2/_3 \times 0.5 = 0.333$ m

$W_1 = ½ H_2 \cdot (0.2\, H_2) \cdot \gamma_w$
$= ½ \times 0.3 \times (0.2 \times 0.3) \times 1.0 = 0.009$ t/m
$x_{w1} = 0.2\, H_3 + (0.2 \times H_2) / 3$
$= (0.2 \times 0.5) + (0.2 \times 0.3) / 3 = 0.120$ m
$y_{w1} = H_3 + 2/3\ H_2$
$= 0.5 + 2/3 \times 0.3 = 0.700$ m

$W_2 = 0.2\, H_3 \times H_2 \times \gamma_w$
$= (0.2 \times 0.5) \times 0.3 \times 1.0 = 0.03$ t/m
$x_{w2} = (0.2 \times H_3) / 2$
$= (0.2 \times 0.5)/2 = 0.05$ m
$y_{w2} = H_3 + H_2/2$
$= 0.5 + 0.3/2 = 0.65$ m

$U_1 = H_1 \times \gamma_w$
$= 3.34 \times 1.0 = 3.340$ t/m

$U_2 = (H_2 + H_3) \times \gamma_w$
$= 0.8 \times 1.0 = 0.800$ t/m

$U = (U_1 + U_2)/2$
$= (3.34 + 0.800)/2 = 2.070$ t/m

$x_u = 2/3\, B \times (2 \times U_2 + U_1) / (U_2 + U_1)$
$= 2/3 \times 3.0 \times (2 \times 0.8 + 3.34) / 4.14 = 2.386$ m
$y_u = 0$

Wi (t/m)		xi (m)	$W_i \cdot x_i$ (tm/m)	y_i (m)	$W_i \cdot y_i$ (tm/m)
G_1	4.994	0.615	3.071	1.537	7.676
G_2	4.333	1.122	4.862	2.305	9.988
G_3	9.089	1.881	17.096	1.537	13.970
S_1	1.267	2.441	3.093	4.990	6.322
S_2	0.302	2.161	0.653	4.660	1.407
S_3	6.962	2.441	16.994	3.073	21.394
S_4	0.050	0.033	0.002	0.333	0.017
W_1	0.009	0.120	0.001	0.700	0.006
W_2	0.030	0.050	0.002	0.650	0.020
U	2.070	3.860	7.990	0	0
Σ	29.106		53.763		60.799

$x_o = \Sigma W_i \cdot x_i / \Sigma W_i$ = 1.847 m

$y = \Sigma W_i \cdot x_i / \Sigma W_i$ = 2.089 m

(iii) Earth Pressure

Active earth pressure
(in ordinary)

$P_{a1} = dh \cdot \gamma_s \cdot K_a \cdot H$
$= 0.10 \times 1.8 \times 0.979 \times 4.61$ = 0.812 t/m

$P_{a2} = ½ (H - H_1)^2 \cdot \gamma_s \cdot K_a$
$= ½ \times (4.61 - 3.34)^2 \times 1.8 \times 0.979$ = 1.421 t/m

$P_{a3} = (H - H_1) \cdot \gamma_s \cdot K_a \cdot H_1$
$= (4.61 - 3.34) \times 1.8 \times 0.979 \times 3.34$ = 7.475 t/m

$P_{a4} = ½ H_1 \cdot \gamma'_s \cdot K_a$
$= ½ \times (3.34)^2 \times 1.0 \times 0.979$ = 5.461 t/m

Horizontal active earth pressure

$P_{ah1} = P_{a1} \times \cos(\theta + \delta)$
$= 0.812 \times \cos(20° + 31°)$ = 0.511 t/m
$y_{a1} = H/2$
$= 4.61/2$ = 2.305 m

$P_{ah2} = P_{a2} \times \cos(\theta + \delta)$
$= 1.421 \times \cos(51°)$ = 0.894 t/m
$y_{a2} = H_1 + (H - H_1)/3$
$= 3.34 + 1.27/3$ = 3.763 m

$P_{ah3} = P_{a3} \times \cos(\theta + \delta)$
$= 7.475 \times \cos(51°)$ = 4.704 t/m
$y_{a3} = H_1 / 2$

$= 3.34/2$ $= 1.670$ m

$P_{ah4} = P_{a4} \times \cos(\theta + \delta)$
$= 5.461 \times \cos(51°)$ $= 3.437$ t/m
$y_{a4} = H_1 / 3$
$= 3.34/3$ $= 1.113$ m

Vertical active earth pressure

$P_{av1} = P_{a1} \times \sin(\theta + \delta)$
$= 0.812 \times \sin(51°)$ $= 0.631$ t/m
$x_{a1} = B$ $= 3.000$ m

$P_{av2} = P_{a2} \times \sin(\theta + \delta)$
$= 1.421 \times \sin(51°)$ $= 1.104$ t/m
$x_{a2} = B$ $= 3.000$ m

$P_{av3} = P_{a3} \times \sin(\theta + \delta)$
$= 7.475 \times \sin(51°)$ $= 5.809$ t/m
$x_{a3} = B$ $= 3.000$ m

$P_{av4} = P_{a4} \times \sin(\theta + \delta)$
$= 5.461 \times \sin(51°)$ $= 4.244$ t/m
$x_{a4} = B$ $= 3.000$ m

(in seismic)

$P_{ea1} = dh \cdot \gamma_s \cdot K_{ea} \cdot H$
$= 0.10 \times 1.8 \times 1.535 \times 4.61$ $= 1.274$ t/m

$P_{ea2} = ½ (H - H_1)^2 \cdot \gamma_s \cdot K_{ea}$
$= ½ \times (1.27)^2 \times 1.8 \times 1.535$ $= 2.228$ t/m

$P_{ea3} = (H - H_1) \cdot \gamma_s \cdot K_{ea} \cdot H_1$
$= 1.27 \times 1.8 \times 1.535 \times 3.34$ $= 11.720$t/m

$P_{ea4} = ½ H_1 \cdot \gamma'_s \cdot K_{ea}$
$= ½ \times (3.34)^2 \times 1.0 \times 1.535$ $= 0.856$ t/m

Horizontal active earth pressure

$P_{eah1} = P_{ea1} \times \cos(\theta + \delta)$
$= 1.274 \times \cos(20° + 15.5°)$ $= 1.037$ t/m
$y_{ea1} = y_{a1}$ $= 2.305$ m

$P_{eah2} = P_{ea2} \times \cos(\theta + \delta)$
$= 2.228 \times \cos(35.5°)$ $= 1.814$ t/m
$y_{ea2} = y_{a2}$ $= 3.763$ m

$P_{eah3} = P_{ea3} \times \cos(\theta + \delta)$
$= 11.720 \times \cos(35.5°)$ $= 9.542$ t/m

y_{ea3} = y_{a3} = 1.670 m

P_{eah4} = P_{ea4} x cos (θ + δ)
= 0.856 x cos (35.5°) = 6,970 t/m

y_{ea4} = y_{a4} = 1.113 m

Passive earth pressure
to be omitted

(iv) Residual water pressure

P_{w1} = ½ · z^2 · γ_w
= ½ x (2.54)2 x 1.0 = 3.226 t/m

y_{w1} = (H – z) + z/3
= 0.8 + 2.54/3 = 1.647 t/m

P_{w3} = z · γ_w · (H_1 – z)
= 2.54 x 1.0 x 0.8 = 2.032 t/m

y_{w2} = (H – z) / 3
= 0.8 / 2 = 0.400 t/m

(v) Vertical Force and Moment

(in ordinary)

Vertical Force (t/m)		Arm (m)	Moment (tm/m)
W	29.106	-	53.763
P_{av1}	0.631	3.000	1.893
P_{av2}	1.104	3.000	3.312
P_{av3}	5.809	3.000	17.427
P_{av4}	4.244	3.000	12.732
FV	40.894	MV	89.127

(in seismic)

Vertical Force (t/m)		Arm (m)	Moment (tm/m)
W	29.106	-	53.763
P_{eav1}	0.740	3.000	2.220
P_{eav2}	1.294	3.000	3.882
P_{eav3}	6.806	3.000	20.418
P_{eav4}	4.972	3.000	14.916
FVe	42.918	MVe	95.199

(vi) Horizontal Force and Moment

(in ordinary)

Horizontal Force (t/m)		Arm (m)	Moment (tm/m)
P_{ah1}	0.511	2.305	1.178
P_{ah2}	0.894	3.763	3.364
P_{ah3}	4.704	1.670	7.856
P_{ah4}	3.437	1.113	3.825
P_{w1}	3.226	1.647	5.313
P_{w2}	2.032	0.400	0.813
FH	14.804	MH	22.349

(in seismic)

Horizontal Force (t/m)		Arm (m)	Moment (tm/m)
P_{eah1}	1.037	2.305	2.390
P_{eah2}	1.814	3.763	6.826
P_{eah3}	9.542	1.670	15.935
P_{eah4}	6.970	1.113	7.758
P_{w1}	3.226	1.647	5.313
P_{w2}	2.032	0.400	0.813
We	3.493	2.089	7.297
FHe		MHe	46.332

(vii) Check of stability

(in ordinary)

Stability against overturning

$e = B/2 - (MV - MH) / FV$

$= 3.0/2 - (89.127 - 23.349) / 40.894 \quad = 0.109 \text{ m } (-)$

$< B/6\ (= 0.50 \text{ m})$

Stability of bearing strata

$q = FV/B\ (1 \pm 6e / B)$

$q_{max} = 40.894/3.0 \times (1 + 6 \times 0.109 / 3.0) \quad = 16.603 \text{ t/m}^2$

$< q_a\ (= 17.100 \text{ t/m}^2)$

(in seismic)

Stability against overturning

$e = 30/2 - (95.199 - 46.332) / 42.918 \quad = 0.361 \text{ m}$

$< B/3\ (= 1.0 \text{ m})$

Stability of bearing strata

$q_{max} = 42.918/3.0 \times (1 + 6 \times 0.361 / 3.0) \quad = 24.635 \text{ t/m}^2$

$< q_{ea}\ (= 25.650 \text{ t/m}^2)$

Notes : It is not required to check the stability against tilting because of PC sheet pile on the toe.

4.5 DESIGN OF GATE

4.5.1 Flood Discharge Gate

1 Design Item

1.1 Design Condition

- Type of Gate	Steel Roller Gate	
- Number of Gate	3	Gates
- Clear span	18.500	m
- Height of Gate	3.700	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 1 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 ——

- Deformation of Main Girder 1/ 800
- Margin of Thickness

Skin Plate (Side facing water)		1.000	mm
Other Members (Side facing water)		1.000	mm

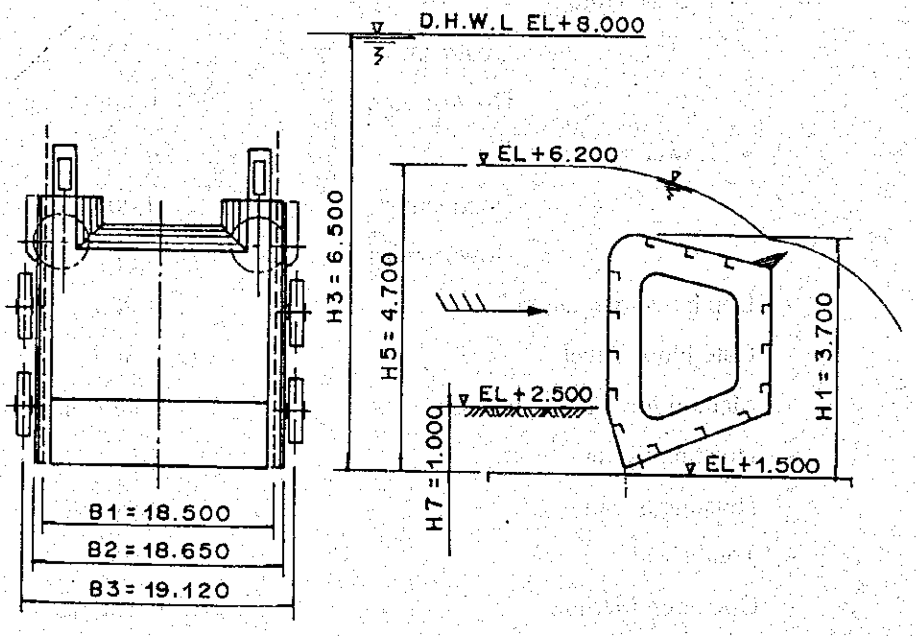
- Major Material of Gate

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

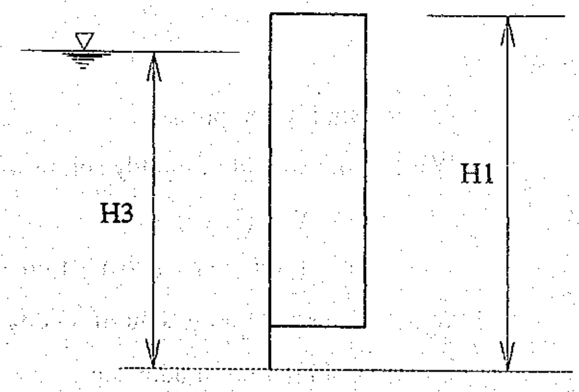
2. BASIC DIMENSIONS



B1	:	Clear Span	18.500	m
B2	:	Length of Watertight Area	18.650	m
B3	:	Span Between Rollers	19.120	m
H1	:	Height of Gate	3.700	m
H2	:	Height of Watertight Area	3.700	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

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

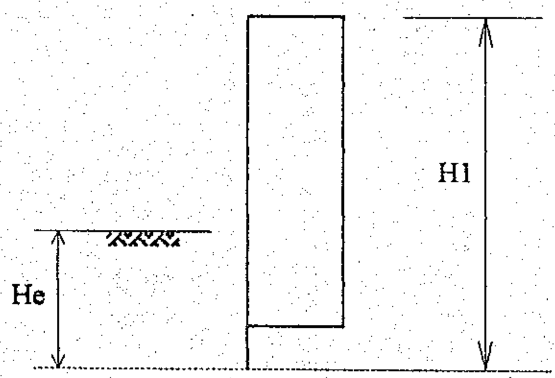
$$= \frac{1}{2} \times [\, 4.700^2 - (\, 4.700 - 3.700 \,)^2 \,]\ 18.650 \times 1.00$$

$$= 196.664 \quad \text{tf}$$

Where,

H2	:	Height of Watertight Portion	3.700	m
H3	:	Design Water Depth (Upstream)	4.700	m
B2	:	Width of Watertight Portion	18.650	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	18.650	m
He	:	Height of muddy soil	1.000	m

$$= \frac{1}{2} \times 0.800 \times 1.000^2 \times 0.5 \times 18.650$$

$$= 3.730 \quad tf$$

(3) Design Loads in Normal Time

$$Pa = Pw + Ps$$

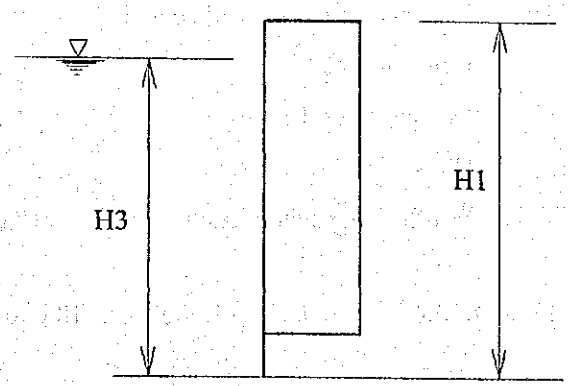
Where,

Pw	:	Water pressure	196.664	tf
Ps	:	Muddy soil pressure	3.730	tf

$$= 196.664 + 3.730$$

$$= 200.394 \quad tf$$

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/sec^2
Hee	:	Distance between riverbed and water surface	4.700	m

$$= \frac{0.12 \times 1.0}{2 \cdot \pi} \times (9.8 \times 4.700)^{0.5}$$

$$= 0.130 \quad tf$$

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

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

$$= \frac{1}{2} \times [(4.700 + 0.130)^2 - (4.700 + 0.130 - 3.700)^2] \times 18.650$$

$$= 205.635 \quad tf$$

(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	4.700	m
H3	:	Design Water Depth	4.700	m
h	=	H3 - H2	1.000	m
B2	:	Width Watertight portion	18.650	m

$$= \frac{7}{12} \times 0.12 \times 4.700^{0.5} \times (4.700^{1.5} - 1.000^{1.5}) \times 18.650$$

= 26.008 tf

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

Wge = k · Wg

Where,

Wg : Self weight of gate 52.000 tf

= 0.12 x 52.0

= 6.240 tf

(5) Total Force in Seismic Time (Pe)

Pe = Peg + Pdo + Wge + Ps

= 205.635 + 26.008 + 6.240 + 3.730

= 241.613 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:

$$\frac{Pe}{1.5} = 241.613 / 1.5 = 161.075 < Pa = 200.394$$

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

4. LOADS FOR GATE OPERATION

4.1 Lifting Time

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

$$= \frac{1}{2} \times \{ 4.700^2 - (4.700 - 3.700)^2 \} \times 18.650 \times 1.00$$

$$= 196.664 \quad tf$$

Where,

H2	: Height of watertight portion	3.700	m
H5	: Water depth for Operation (Upstream)	4.700	m
B2	: Width of watertight portion	18.650	m
γ	: Specific gravity of water	1.00	

4.2 Lowering Time

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

$$= \frac{1}{2} \times 1.000^2 \times 18.650 \times 1.00$$

$$= 9.325 \quad tf$$

Where,

H7	: Water depth for Operation (Upstream)	1.000	m

5. GATE BODY

5.1 Sectional Dimensions

(1) Sectional Dimensions Regarding Y Axis

No	Dimension of Member	A	x	A · x	A · x²	I
1.	PL 14 x 460.2	55.23	10.85	599.36	6504	
2.	PL 14 x 2400.0	228.00	0.70	201.60	141	
3.	PL 14 x 1118.0	134.16	25.78	3459.10	89185	279
4.	PL 14 x 2173.0	260.76	147.33	38415.90	5659620	8869
5.	PL 14 x 2335.7	280.29	249.30	69875.10	17419900	
6.	PL 14 x 2245.6	269.47	139.86	37687.50	5270960	10868
7.	L– 150 x 150 x 12	28.60	12.26	350.64	4299	
8.	L– 150 x 150 x 12	28.60	12.26	350.64	4299	
9.	L– 150 x 150 x 12	28.60	12.26	350.64	4299	
10.	L– 150 x 150 x 12	28.60	12.26	350.64	4299	
11.	L– 150 x 150 x 12	28.60	25.12	718.57	18054	
12.	L– 150 x 150 x 12	28.60	55.12	1576.57	86908	
13.	L– 150 x 150 x 12	28.60	119.85	3427.74	410818	
14.	L– 150 x 150 x 12	28.60	189.85	5429.74	1030840	
15.	L– 150 x 150 x 12	28.60	237.74	6799.36	1616480	
16.	L– 150 x 150 x 12	28.60	237.74	6799.36	1616480	
17.	L– 150 x 150 x 12	28.60	237.74	6799.36	1616480	
18.	L– 150 x 150 x 12	28.60	181.88	5201.76	946095	
19.	L– 150 x 150 x 12	28.60	121.88	3485.76	424843	
20.	L– 150 x 150 x 12	28.60	61.88	1769.76	109512	
Total		1688.30		193649.00	36340000	20017

At center of balance: Ex

$$Ex = \frac{\Sigma A \cdot x}{\Sigma A} = \frac{193649.00}{1688.30} = 114.7 \text{ cm}$$

Sectional Secondary Momentum: Iy

$$Iy = \Sigma\ (A \cdot x^2 + I) - \Sigma\ A \cdot Ex^2$$

$$= (36340000 + 2001780) - 1688.30 \times 114.7^2 = 16130100 \text{ cm}^4$$

(2) Sectional Dimensions Regarding X Axis

No	Dimension of Member	A	y	A · y	A · y^2	I
1.	PL 14 x 460.2	55.23	10.85	599.36	6504	
2.	PL 14 x 2400.0	228.00	150.00	43200.00	6480000	13824
3.	PL 14 x 1118.0	134.16	319.37	42848.50	13684700	1118
4.	PL 14 x 2173.0	260.76	319.35	83272.40	26593000	1390
5.	PL 14 x 2335.7	280.29	163.21	45746.60	7466510	12742
6.	PL 14 x 2245.6	269.47	23.21	6255.52	145129	455
7.	L– 150 x 150 x 12	28.60	64.14	1834.40	117659	
8.	L– 150 x 150 x 12	28.60	124.14	3550.40	440747	
9.	L– 150 x 150 x 12	28.60	184.14	5266.40	969756	
10.	L– 150 x 150 x 12	28.60	244.14	6982.40	1704680	
11.	L– 150 x 150 x 12	28.60	298.15	8527.05	2542330	
12.	L– 150 x 150 x 12	28.60	358.15	10234.00	3668530	
13.	L– 150 x 150 x 12	28.60	320.84	9176.11	2944090	
14.	L– 150 x 150 x 12	28.60	293.13	8383.38	2457380	
15.	L– 150 x 150 x 12	28.60	229.14	6553.40	1501650	
16.	L– 150 x 150 x 12	28.60	169.14	4837.40	818199	
17.	L– 150 x 150 x 12	28.60	109.14	3121.40	340670	
18.	L– 150 x 150 x 12	28.60	49.72	1421.89	70692	
19.	L– 150 x 150 x 12	28.60	37.44	1070.89	40098	
20.	L– 150 x 150 x 12	28.60	25.17	719.89	18121	
Total		1688.30		293610.00	72010500	295306

At center of balance: Ey

$$Ey = \frac{\Sigma A \cdot y}{\Sigma A} = \frac{293610.00}{1688.30} = 173.9 \text{ cm}$$

Moment of Area: Ix

$$Ix = \Sigma\ (A \cdot y^2\ + I) - \Sigma A \cdot Ey^2$$

$$= (72010500 + 2953060) - 1688.30 \times 173.9^2 = 23902100 \text{ cm}^4$$

5.2 Loads in Perpendicular Direction

(1) Self Weight of Gate

Wg = 52.00 tf

(2) Upward Force Due to Water Pressure in Lower Portion

$$Wu = (H3 - \frac{h'}{2}) \cdot L1 \cdot B1$$

Where,

H3	:	4.700	m
h′	:	1.000	m
L1	:	0.500	m
B1	:	18.500	m

$$= (4.700 - \frac{1.000}{12}) \times 0.500 \times 18.500 = 38.850 \quad tf$$

(3) Weight of Water At Upper Portion

$$Wd = k \cdot Hfr \cdot Dep \cdot B5$$

Where,

Hfr	:	1.000	m	
k	:	0.5	m	
Dep	:	Thickness of Gate	2.500	m
B5	:	Width of Overflow	16.800	m

$$= 0.5 \times 1.000 \times 2.500 \times 16.800 = 21.000 \quad tf$$

(4) Total Load in Perpendicular Direction

$$Pv = Wg - Wu + Wd$$

$$= 52.000 - 38.850 + 21.000 = 34.150 \quad tf$$

5.3 Sectional Force of Gate

(1) Momentum in Horizontal Direction

$$Mh = \frac{Pa}{8} \cdot (2 \cdot B3 - B2)$$

Where,

Pa	:	Force Acting in Horizontal direction	200.394	tf
B3	:	Span Length between rollers	19.120	m
B2	:	Width of watertight portion	18.650	m

$$= 200.394 / 8 \times (2 \times 19.120 - 18.650)$$

$$= 490.715 \quad tf\text{-}m$$

(2) Momentum in Vertical Direction

$$Mv = \frac{Pv}{8} \cdot B4$$

Where,

Pv	: Force acting in vertical direction	34.150	tf
B4	: Span length at hanging points	18.000	m

$= 34.150 / 8 \times 18.000 = 76.838$ tf-m

(3) Stress on Shell

Horizontal force: σy

$$\sigma y = \frac{Mh}{Iy} \cdot e\,y = 490.715 \times 10^5 / 16130100 = 3.042 \cdot e\,y \ \text{kgf/cm}^2$$

Vertical force : σx

$$\sigma x = \frac{Mv}{Ix} \cdot e\,y = 76.838 \times 10^5 / 23902100 = 0.321 \cdot e\,x \ \text{kgf/cm}^2$$

Total Stress:

$$\sigma x y = \sigma y + \sigma x$$

Calculation Point	e y	e x	σy	σx	σxy
1.	-114.7	-143.9	-349	-46	-395
2.	-114.7	-113.9	-349	-37	-385
3.	-114.7	-83.9	-349	-27	-376
4.	-114.7	-53.9	-349	-17	-366
5.	-114.7	-23.9	-349	-8	-357
6.	-114.7	6.1	-349	2	-347
7.	-114.7	36.1	-349	12	-337
8.	-114.7	66.1	-349	21	-328
9.	-114.7	96.1	-349	31	-318
10.	-99.7	126.1	-303	40	-263
11.	-84.7	156.1	-258	50	-208
12.	-69.7	186.1	-212	60	-152
13.	-69.7	186.1	-212	60	-152
14.	-69.7	186.1	-212	60	-152
15.	-64.7	196.1	-197	63	-134
16.	5.3	157.0	16	50	67
17.	75.3	129.3	229	42	271
18.	135.3	106.1	412	34	446
19.	135.3	51.1	412	16	428
20.	135.3	-8.9	412	-3	409
21.	135.3	-68.9	412	-22	389
22.	135.3	-128.9	412	-41	370
23.	65.3	-137.1	199	-44	155
24.	5.3	-149.4	16	-48	-32
25.	-54.7	-161.6	-166	-52	-218
26.	-84.7	-173.9	-258	-56	-313
27.	-98.3	-113.9	-299	-37	-336
28.	-98.3	-53.9	-299	-17	-316
29.	-98.3	6.1	-299	2	-297
30.	-98.3	66.1	-299	21	-278
31.	-84.7	119.4	-258	38	-219
32.	-54.7	179.4	-166	58	-109
33.	-0.2	141.6	-1	45	45
34.	69.8	113.8	212	37	249
35.	118.9	51.1	362	16	378
36.	118.9	-8.9	362	-3	359
37.	118.9	-68.9	362	-22	340
38.	62.3	-121.0	190	-39	151
39.	2.3	-133.2	7	-43	-36
40.	-57.7	-145.5	-176	-47	-222

5.4 Stress on Skin Plate

Bending moment on skin plate due to water pressure is estimated

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

By the formula of DIN 19704 as follows:

Where,

σ	: Stress	kgf/cm^2
k	: Coefficient show in table below	
a	: Length of shorter side of block	cm
b	: Length of longer side of block	cm
P	: Water pressure or water pressure + soil pressure	kgf/cm^2
t	: Thickness of plate	cm
ε	: Margin of thickness of plate	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 Point	a	b	P	k1	k4	t	σ1	0.3·σ1	σ4	0.3·σ4
	60	300	0.13	50.0	34.3	1.40	163	49	111	33
	60	300	0.19	50.0	34.3	1.40	238	71	163	49
	60	300	0.25	50.0	34.3	1.40	313	94	214	64
	60	300	0.31	50.0	34.3	1.40	388	116	266	80
	60	300	0.37	50.0	34.3	1.40	463	139	317	95
	60	300	0.43	50.0	34.3	1.40	538	161	369	111
	0	300	0.46	50.0	34.3	1.40	0	0	0	0

5.5 Horizontal Stiffness Plate

5.5.1 Sectional Dimensions

No	Dimensions	A	y	A · y	$A \cdot y^2$	I
1.	PL 14 x 288.0	34.56	0.60	20.74	12	4
2.	L– 150 x 150 x 12	28.60	12.06	344.92	4160	622
	Total	63.16		365.65	4172	626

At Center of Balance

$$E\,i = \frac{\Sigma A \cdot x}{\Sigma A} = \frac{365.65}{63.16} = 5.79 \text{ cm}$$

$$E\,o = (1.20 + 15.00) - 5.79 = 10.21 \text{ cm}$$

Moment of Area (I)

$$I = \Sigma\ (A \cdot x^2 + I) - A \cdot EI^2$$

$$= (4172 + 626) - 63.16 \times 5.79^2 = 2681 \text{ cm}^4$$

Section Modulus (Z)

$$Z\,i = \frac{I}{Ei} = \frac{2681}{5.79} = 463 \text{ cm}^3$$

$$Z\,o = \frac{I}{Eo} = \frac{2681}{10.21} = 263 \text{ cm}^3$$

5.5.2 Bending Moment of Horizontal Stiffness Members

Horizontal members are designed as consecutive beams supported by vertical stiffness member.

Supporting point $M1 = \frac{-1}{12} \cdot P \cdot a \cdot b^2$ (kgf-cm)

Center point $M2 = \frac{1}{24} \cdot P \cdot a \cdot b^2$ (kgf-cm)

Where,

P	:	Average Water Pressure	kgf/cm^2
a	:	Width of horizontal member	cm
b	:	Span of horizontal membercm	

Calculation Point	a	b	P	M1	M2
	60	300	0.160	-72000	36000
	60	300	0.220	-99000	49500
	60	300	0.280	-126000	63000
	60	300	0.340	-153000	76500
	60	300	0.440	-198000	99000
	60	300	0.445	-100125	50063

Calculation Point	Sectional Coefficient		Stress at Supporting Point		Stress at Center Point	
	Z i	Z o	σ1 i	σ1 o	σ2 I	σ2 o
	463	263	-156	-274	78	137
	463	263	-214	-376	107	188
	463	263	-272	-479	136	240
	463	263	-330	-582	165	291
	463	263	-428	-753	214	376
	463	263	-216	-381	108	190

5.6 Vertical Stiffness Member

5.6.1 Sectional Dimensions

No	Dimensions	A	x	A · x	A · x²	I
1.	PL 14 x 288.0	34.56	0.60	20.74	12	4
2.	PL 7 x 500.0	25.10	26.30	660.13	17361	5271
3.	PL 12 x 150.0	14.80	51.90	768.12	39865	1
	Total	74.46		1448.99	57239	5276

At center of balance: (E)

$$E\,i = \frac{\Sigma A \cdot x}{\Sigma A} = \frac{1448.99}{74.46} = 19.46 \text{ cm}$$

$$E\,o = (1.20 + 50.20 + 1.00) - 19.46 = 32.94 \text{ cm}$$

Moment of Area: (I)

$$I = \Sigma\,(A \cdot x^2 + I) - \Sigma\,A \cdot Ei^2$$

$$= (57239 + 5276) - 74.46 \times 19.46^2 = 34319 \text{ cm}^4$$

Section Modulus : (Z)

$$Z\,i = \frac{I}{Ei} = \frac{34319}{19.46} = 1764 \text{ cm}^3$$

$$Z\,o = \frac{I}{Eo} = \frac{34319}{32.94} = 1042 \text{ cm}^3$$

5.6.2 Bending Moment of Vertical Stiffness Member

The member is designed ad a simple beam subjected to trapezoidal loads.

$$M = \frac{1}{9 \times 3^{0.5}} \cdot (P2 - P1) \cdot h^2 \cdot b + \frac{1}{8} \cdot P1 \cdot h^2 \cdot b \quad \text{(kgf-cm)}$$

Where,

P1 : Water pressure at upper portion kgf/cm^2

P2 : Water pressure at lower portion kgf/cm^2

b : Distance between vertical members cm

h : Span of vertical member cm

$$M = \frac{1}{9 \times 3^{0.5}} \text{ x } (0.4600 - 0.1000) \text{ x } 300.0^2 \text{ x } 360.0 + \frac{1}{8} \text{ x } 0.1000 \text{ x } 360.0^2 \text{ x } 300.0$$

$$= 1234246 \text{ kgf-cm}$$

Bending Moment

$$\sigma 3 = \frac{-M}{Zi} = -1234246 / 1764 = -700 \text{ kgf/cm}^2$$

$$\sigma 3' = \frac{M}{Zo} = 1234246 / 1042 = 1184 \text{ kgf/cm}^2$$

5.7 Total Stress

5.7.1 Total Stress in Horizontal Direction

Total stresses in the horizontal direction are summed up using stresses of shell and skin plate as follows:

$\sigma h\,1 = \sigma x y + 0.3\,\sigma 1 + \sigma s\,2$ (Center Point)

$\sigma h\,2 = \sigma x y + \sigma a\,1$ (Supporting Point)

$\sigma h\,3 = \sigma x y + \sigma 4$

Calculation Point	Stress of Shell	Stress of Skin Plate		Stress of Horizontal Stiffness Member		Total Stress
	Z i	0. 3 $\sigma 1$	$\sigma 4$	$\sigma s 2$	$\sigma s 1$	σh
1	-395		-111			-507
2	-385	-49		78		-356
3	-376		-163			-539
4	-366	-71		107		-331
5	-357		-214			-571
6	-347	-94		136		-305
7	-337		-266			-603
8	-328	-116		165		-279
9	-318		-317			-635
10	-263	-139		214		-188
11	-208		-369			-576
12	-152	-161		108		-205
13	-152		0			-152
14	-152	0				-152
15	-134		0			-134
16	67	0				67
17	271					271
18	446					446
19	428					428
20	409					409
21	389					389
22	370					370
23	155					155
24	-32					-32
25	-218					-218
26	-313	-111				-424
27	-336				-274	-609
28	-316				-376	-693
29	-297				-479	-776
30	-278				-582	-860
31	-219				-753	-972
32	-109				-381	-490
33	45					45
34	249					249
35	378					378
36	359					359
37	340					340
38	151					151
39	-36					-36
40	-222					-222

Allowable : 1200 kgf/cm^2

5.7.2 Total Stress in Vertical Direction

$\sigma v 1 = 0.3 \cdot \sigma 4 + \sigma 3$ (with diaflum)

$\sigma v 2 = \sigma 1$ (without diaflum)

Where,

σ 1, σ 4 : Stress of Skin Plate

σ 3 : Stress Vertical Stiffness member

Calculation Point	$\sigma 1$	$0.3 \cdot \sigma 4$	$\sigma 3$	σv
1		33	700	733
2	238			238
3		49	700	749
4	313			313
5		64	700	764
6	388			388
7		80	700	780
8	463			463
26	538			538

Allowable : 1200 kgf/cm^2

5.8 Composite Stress of Gate

$$\sigma g = (\sigma h^2 + \sigma h v^2 - \sigma h \cdot \sigma v + 3 \cdot \tau)^{0.5}$$

Where,

σg : Composite Stress kg/cm^2

σh : Composite stress in horizontal direction kg/cm^2

σv : Composite stress in vertical direction kg/cm^2

τ : Shearing stress kg/cm^2

$\tau = 0$ At the maximum moment point (Center of Gate)

Calculation Point	$\sigma 1$	$0.3 \cdot \sigma 4$	$\sigma 3$	σv
1	-507	733	0	1080
2	-356	238	0	518
3	-539	749	0	1120
4	-331	313	0	557
5	-571	764	0	1160
6	-305	388	0	601
7	-603	780	0	1201
8	-279	463	0	648
26	-424	538	0	835

Allowable : 1.1×1200 kgf/cm^2 = 1320 kg/cm^2

5.9 Shearing Stress

(1) Shearing Stress in Horizontal Direction

Shearing stress in the horizontal direction is bearded by shell plate.

$$\tau x = \frac{Fx}{Ax} \cdot 10^3$$

Where,

Fx : Horizontal shearing stress

$= \frac{Pnh}{2} = 200.394 / 2 \quad = \quad 100.197$ tf

Ax : Sectional area of shell plate

$= 260.76 + 269.47 \quad = \quad 530.223$ cm^2

$= 100.197 / 530.223 \times 10^3 = 189$ kg/cm^2

Allowable stress 700 kgf/cm^2

(2) Vertical shearing stress

Vertical shearing stress is bearded by shell plate

$$\tau y = \frac{Fy}{Ay} \cdot 10^3$$

Where,

Fy : Vertical shearing stress

$= \frac{Pv}{2} = 34.150 / 2 \quad = \quad 17.075$ tf

Ay : Sectional area of shell plate

$= 288.00 + 280.29 \quad = \quad 568.285$ cm^2

$= 17.075 / 568.285 \times 10^3 = 30$ kg/cm^2

Allowable stress 700 kgf/cm^2

5.10

1.10.1 Horizontal Direction

(1) Deformation

$$\delta x = \frac{Pnh}{48 \cdot E \cdot Iy} \cdot (B3^3 - \frac{B3 \cdot B2^2}{2} + B2^3/8)$$

Where,

δ x	: Deformation in horizontal direction		
E	: Yang's modulus	2.1×10^6	kgf/cm^2
Iy	: Sectional secondary moment on I y	16130100	cm^4
Pnh	: Horizontal loads	200.394	kgf
B2	: Width of watertight portion	1865.0	cm
B3	: Span between rollers	1912.0	cm

$$= 200394 / (48 \times 2.1 \times 10^6 \times 16130100) \times (1912.0^3 - \frac{1912.0 \times 1865.0^2}{2} + \frac{1865.0^3}{8})$$

$$= 0.55 \text{ cm}$$

(2) Rate of deformation

$$R x = \frac{\delta x}{B3} = 1 / 3466.0 < \frac{1}{800}$$

1.10.2 Vertical Direction

(1) Deformation

$$\delta y = \frac{5 \cdot Pv}{348 \cdot E \cdot Ix} \cdot B4^3$$

Where,

δ y	: Vertical Deformation		
E	: Yang's modulus of steel	2.1×10^6	kgf/cm^2
Ix	: Sectional secondary moment on I x	23902100	cm^4
Pv	: Vertical loads	34150	kgf
B4	: Distance between hanging points	1800.0	cm

$$= 5 \times 34150 / (384 \times 2.1 \times 10^6 \times 23902100) \times 1800.0^3$$

$= \quad 0.05 \text{ cm}$

(2) Rate of Deformation

$$Ry = \frac{\delta v}{B4} = 1/34840 < \frac{1}{800}$$

5.11 Rollers and Reaction

Where,

A	: Distance between center of roller and reacting point	0.70	m
B	: Distance between center of roller and gate floor	0.90	m
C	: Distance between upper-lower rollers	1.50	m
D	: Distance between center of roller and center of water pressure	0.95	m
E	: Distance between forefront of gate and center of upper water	1.17	m
F	: Distance between horizontal load and gate floor	1.43	m
L	: Distance between forefront of gate and center of roller	1.20	m
Ex	: Distance between forefront of gate and shell's center of balance	1.15	m
Wu	: Upward force by gate leaning in front	38.850	tf
Wg	: Self weight of gate	52.000	tf
Wd	: Weight of water on gate	21.000	tf
Pu	: Reacting force at lower point	34.150	t
Pnh	: Horizontal loads	200.394	tf
R1	: Reacting force of upper roller		tf
R2	: Reacting force of lower roller		tf

5.12 Supporting Force of Roller

Supporting force of roller is calculated by balance around the roller axis of lower part.

(1) Gate Closing Time

$$R1 = \frac{1}{2 \cdot C} \cdot \{A \cdot Pu + D \cdot Wu + (F - B) \cdot Pnh + (E - L) \cdot Wd + (Ex - L) \cdot Wg\}$$

$$= \frac{1}{2 \times 1.500} \times \{0.700 \times 34.150 + 0.950 \times 38850 + (1.433 - 0.900) \times$$

$$200.394 + (1.167 - 1.200) \times 21.000 + (1.147 - 1.200) \times 52.000\}$$

$$= 54.725 \quad \text{tf}$$

$$R2 = \frac{Pnh}{2} - R1$$

$$= 200.394 / 2 - 54.725 = 45.472 \quad tf$$

(2) Gate Lifting Time

$$R1 = \frac{1}{2 \cdot C} \cdot \{D \cdot Wu + (F - B) \cdot Pnh + (E - L) \cdot Wd + (Ex - L) \cdot Wg\}$$

$$= \frac{1}{2 \times 1.500} \times \{0.950 \times 38.850 + (1.433 - 0.900) \times 200.394 + (1.167 - 1.200) \times 21.000 + (1.147 - 1.200) \times 52.000\}$$

$$= 46.756 \quad tf$$

$$R2 = \frac{Pnh}{2} - R1$$

$$= 200.394 / 2 - 46.756 = 53.441 \quad tf$$

Load of Roller: P = 54.725kgf

6. ROLLER

6.1 Load of Roller

Load of Roller P = 54,725 kgf

6.2 Dimensions of Roller

Outer diameter of roller	RD	=	%100.0	cm
Diameter of roller shaft (Bearing side)	D1	=	29.0	cm
Diameter of roller shaft (Bush side)	D2	=	20.0	cm
Width roller boss	B	=	20.0	cm
Width roller shoe	b1	=	20.0	cm
Effective width of roller shoe	b2	=	19.0	cm
Material of roller	SCMn3B			
Stiffness of roller	HB	=	200	
Material of roller shaft	SUS304			
Correction coefficient by standard	1.000			

6.3 Strength of Roller (Line Contact)

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

$= 0.418 \times [54.725 \times 2.1 \times 10^6 / (19.0 \times \%100.0/2)]^{1/2} = 4{,}597\ kg/cm^2 < \rho a$

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

$= 1.52 \times [54.725 \times \%100.0/2 / (19.0 \times 2.1 \times 10^6)]^{1/2} = 0.398$ cm

$Z = 0.78 \cdot C$

$= 0.78 \times 0.398 = 0.310$ cm

Where,

ρ	:	Strength of Herz at contacting face		kgf/cm^2
P	:	Load of roller (acting)	54,725	kgf
b2	:	Effective width of roller shoe	19.0	cm
D	:	Outer diameter of roller	100.0	cm
C	:	Hart width of contacting face	0.398	cm
Z	:	Depth at maximum shearing stress	0.310	cm

Allowable Stress at Contacting Face

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

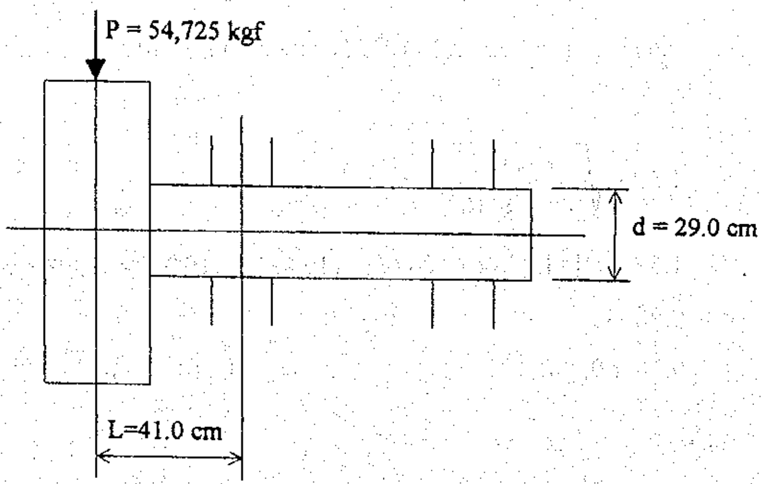
Where,

υ	:	Safety factor	1.0	
HB	:	Burinel stiffness of roller	200	kgf/mm^2

$= \frac{100}{2 \times 1.3} \times 200 = 7{,}692\ kgf/cm^2$

6.4 Roller Shaft

(1) Bearing Moment acting on Roller Axis



$M = P \cdot L$

$= 54,725 \times 41.0 = 2,243,725$ kgf-cm

(2) Dimensions of Roller Shaft

Diameter (Shaft side)	$d1 = 29.0$	cm
Diameter (Bush side)	$d2 = 20.0$	cm
Section modulus	$Z = 2394$	cm^3
Section Area	$A = 661$	cm^2
Material	SUS304	

(3) Bearing Stress

$$\sigma = \frac{M}{Z} = 2,243,725 / 2,394 = 937 \text{ kgf/cm}^2 < \sigma a$$

Allowable bearing stress = Yielding point / 2 · Correction factor

σa = 2,100 / 2 x 1.000 = 1,050 kgf/cm^2

(4) Shearing Stress

$$\tau = \frac{4 \cdot P}{3 \cdot A} = 4 \times 54,725 / (3 \times 661) = 110 \text{ kgf/cm}^2 < \tau a$$

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

τa = 1,050 / 1.732 = 606 kgf/cm^2

(5) Composite Stress

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

$$= (937 / 1{,}050)^2 + (110 / 606)^2 = 0.8 < 1.2$$

(6) Bush's Bearing Stress

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

$$= 54.725 / 20 \times 20 = 137 \text{ kgf/cm}^2 < \sigma ca$$

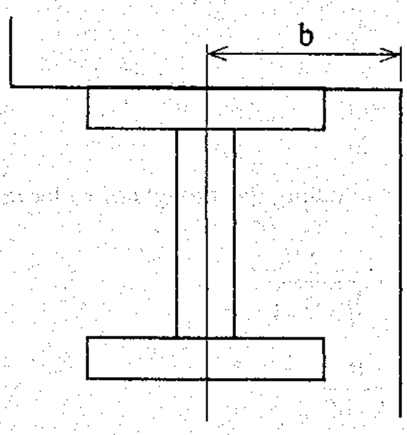
Allowable stress

$\sigma ca = 250$ kgf/cm^2

Oil less Metal #500 SP-SL4

7. GUIDE FRAME AND ROLLER RAIL

7.1 Distance between Side of Pier and Center of Roller Rail



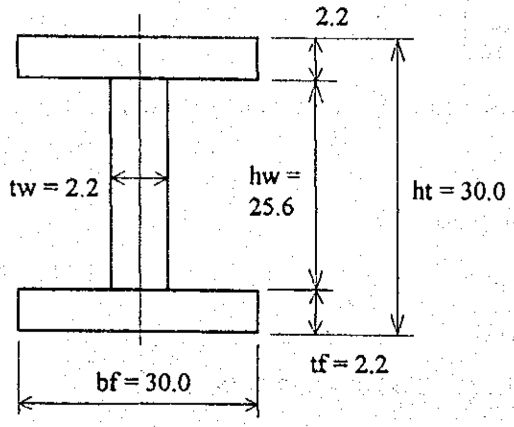
$b > 20 + 0.1 \cdot R$

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

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

$31.0 \text{ cm} > 20 + 0.1 \times 54.725 = 25.5 \text{ cm}$

7.2 Section Efficiency of Roller Rail



Moment of Area

$I = 28{,}633 \ cm^4$

Section Modulus

$Z = 1{,}909 \ cm^3$

Where,

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

$$= \frac{30.0 \cdot 30.0^3}{12} - \frac{(30.0 - 2.2) \times 25.6^3}{12} = 28{,}633 \ cm^4$$

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

$$= \frac{2 \times 28{,}633}{30.0} = 1{,}909 \ cm^3$$

7.3 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

$k = 0.0588 \times 54.725 / (30.0^2 \times 28.633)^{1/3} = 10.9$ kgf/cm² < ka

Allowable bearing stress of concrete.

$ka = 55.0$ kgf/cm²

$a = 0.75 \times 54.725 / (10.9 \times 30.0) = 125.5$ cm

$M = 10.9 \times 125.5^2 \times 30.0 / 4 = 1,287,583$ kgf-cm

Bearing stress of roller rail σr

$$\sigma r = \frac{M}{Z} = 1,287,583 / 1,909 = 674 \text{ kgf/cm}^2 < \sigma ra$$

Allowable bearing stress

$\sigma ra = fa \cdot \sigma a$

$= 1.00 \times 1,200$

$= 1,200$ kgf/cm²

7.4 Thickness of Rail

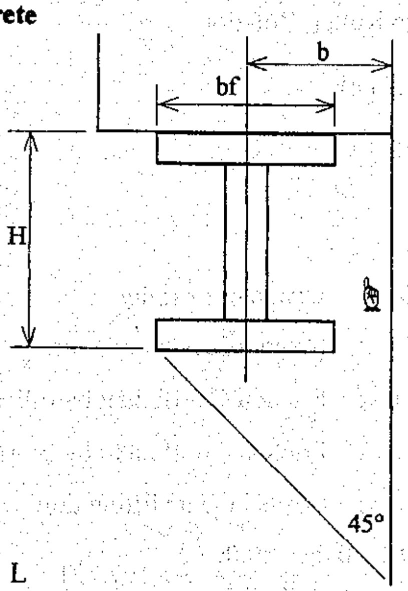
$tr > 4 \cdot Zr$

Where,

tr	:	Thickness of rail		cm
Zr	:	Depth at maximum shearing stress	0.310	cm

1.6 cm > $4 \times 0.310 = 1.240$ cm

7.5 Shearing Strength of Concrete



$b = 31.0$ cm

$bf = 30.0$ cm

$H = 30.0$ cm

Shearing length of concrete : L

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

$$= \quad 2 \times \left(31.0 + \frac{30.0}{2}\right) + 30.0 = 122 \ \text{cm}$$

Shearing strength of concrete : τc

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

$$= 10.9 \times 30.0 / 122 = 2.7 \ \text{kgf/cm}^2 < \tau ca$$

Allowable shearing strength

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

8. LOADS FOR GATE OPENING AND CLOSING

8.1 Loads in Gate Operation

(1) Gate Lifting Time

$$Pu = 196.664 + 1.130 = 197.794 \ \text{tf} \ \text{(water pressure + soil pressure)}$$

(2) Gate Lowering Time

$$Pd = 9.325 \ \text{tf} \ \text{(water pressure)}$$

8.2 Loads Acting on Gate

(1) Total Weight of Gate

Self Weight $Wg = 52.000$ 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	100.00	cm
d	:	Center diameter of bearing	20.00	cm
$\mu 1$	:	Friction Coefficient by roller rolling	0.1	
$\mu 2$	:	Friction Coefficient by bearing sliding	0.02	
Pu	:	Loads in Gate lifting time	197.794	tf

$$= \frac{0.1 + 0.02 \times 20.00/2}{100.00/2} \times 197.794$$

$$= 1.187 \ \text{tf}$$

(Gate Lowering Time)

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

Where,

D	:	Diameter of roller	100.00	cm
d	:	Center diameter of bearing	20.00	cm
μ1	:	Friction Coefficient by roller rolling	0.1	
μ2	:	Friction Coefficient by bearing sliding	0.02	
Pd	:	Loads in Gate lifting time	9.325	tf

$$= \frac{0.1 + 0.02 \times 20.00/2}{100.00/2} \times 9.325$$

= 0.056 tf

(3) Friction Due to Sediment

(Gate Lifting Time)

$Fe = \mu e \cdot Pe$

Where,

μ e	:	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,

μ r	:	Friction coefficient of rubber	0.7	
q	:	Inicial pressure	0.100	tf/m
p	:	Average force acting on side	2.850	tf/m^2
b	:	Effective width of receiving pressure	0.050	m
L	:	Length of watertight rubber at side	3.700	tf

= 2 x 0.7 x (0.100 + 2.850 x 0.050) x 3.700

= 1.256 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	0.135	tf/m^2
b	:	Effective width of receiving pressure	0.050	m
L	:	Length of watertight rubber at side	3.700	tf

$= 2 \times 0.7 \times (0.100 + 0.135 \times 0.050) \times 3.700$

$= 0.553$ tf

(5) Uplift Pressure

(Gate Lifting Time)

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

Where,

Wg	:	Self weight of Gate	52.000	tf
γ	:	Specific gravity of steel	7.85	
s	:	Volume ratio of submerged portion of gate	100.0	%

$$= \frac{52.000}{7.850} \times \frac{100.0}{100}$$

$= 6.624$ tf

(Gate Lowering Time)

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

Where,

Wg	:	Self weight of Gate	52.000	tf
γ	:	Specific gravity of steel	7.85	
s	:	Volume ratio of submerged portion of gate	25.0	%

$$= \frac{52.000}{7.850} \times \frac{25.0}{100}$$

$= 1.656$ 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	1.000	tf/m^2
t	:	Width of stress receiving area	2.500	m
B	:	Length of stress receiving area	18.650	m
k	:	Coefficient of over flow water	0.000	
ΔH	:	Water level difference	1.000	m
D	:	Width of Gate	2.500	m

= 1.000 x 2.500 x 18.650 + 0.000 x 1.000 x 2.50 x 18.650

= 46.625 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	18.650	m

= 1.000 x 0.050 x 18.650

= 0.933 tf

8.3 Total of Loads for Gate Opening and Closing

Item	Lifting	Lowering
Self weight	52.000	52.000
Friction due to roller rotation	1.187	-0.056
Friction of rubber	1.256	-0.553
Friction due to sediment	0.452	-
Uplift pressure	-6.624	-1.656
Downward Force of Gate	46.625	-
Upward Force of Gate	-	-0.933
Total	94.896	48.802

Lifting Force 95.000 tf

Lowering Force Self weight lowering

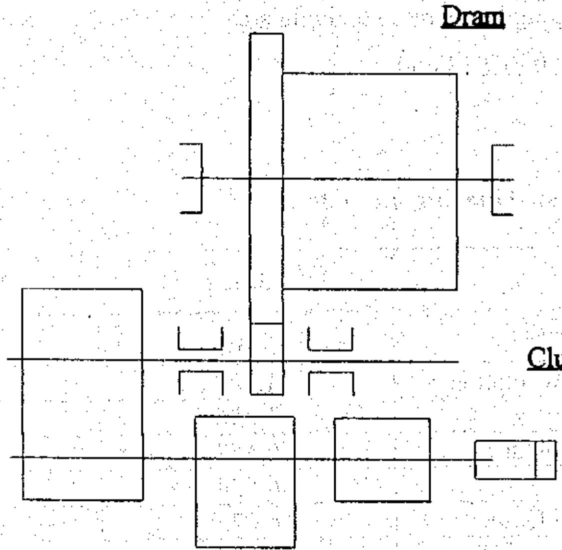
9. GATE HOISTING SYSTEM

9.1 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	95.000 tf
Lowering time	Falling by Self-Weight
Source Power	220 V, 50 Hz
Safety Factor	

9.2 Layout of Hoisting Machine

(1) Layout Plan



9.3 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	960	rpm
X(1)	:	Ratio Reduction Gear-1	1/500	
Z1(1)	:	Number of Cog-wheel of Pinion	21	
Z2(1)	:	Number of Cog-wheel of Gear	136	
N	:	Number of Wire	4	
D	:	Diameter of Dram	1,400	mm

$$= 960 \text{ x } \frac{1}{500} \times \frac{21}{136} \times \frac{1}{4} \times \frac{1{,}400 \times \pi}{1000} = 0.326 \text{ m/min}$$

9.4 Efficiency of Machine

Parts		Unit Efficiency	Number	Total Efficiency
Sheave	ηs	0.95	3	0.881
Dram	ηd	0.95	1	0.950
Gear	ηg	0.95	1	0.950
Reduction Machine–1	ηr	For output Estimation	1	0.800
		For strength Estimation		0.940
Clutch Box	ηc	For output Estimation	1	0.790
		For strength Estimation		0.920
All Efficiency		For output Estimation	η1 =	0.503
		For strength Estimation	η2 =	0.688
Remarks		Sheave (One side) 1		

9.5 Generating Power

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

Where,

Kw	:	Generating Power		kw
Wu	:	Hoisting Load	95.000	rpm
V	:	Hoisting Speed	0.326	m/min
η1	:	Machine Efficiency	0.503	

$$= \frac{95.000 \times 0.326}{6.12 \times 0.503} = 10.06 \text{ kw}$$

Therefore, the generating power of 11.00 kw will be used.

(3 alternating current)