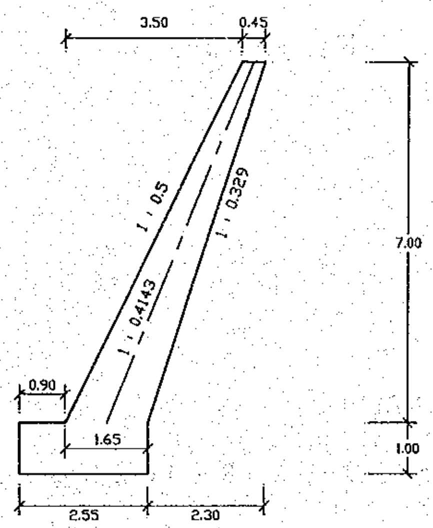
3.2.2 Leaning

3.2.2.1 Leaning Wall for Concrete Type

Consideration Section: WF.110L +15.0 m to WF.111 L – 15.0 m

Design Condition

(a) Proposed Section



(b) The Material Data

(i) Plain concrete type D (K 175)

Unit Weight	γ_c	=	2.35	t/m^3
Allowable Compressive stress	σ_{ca}	=	60	kg/cm^2
Allowable shear strength	τ_{ca}	=	5.50	kg/cm^2

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

N – value	N	=	20	
Soil internal friction angle	ϕ	=	32°	
Unit weight	γ_s	=	1.80	t/m^3
Cohesion	c	=	0.0	t/m^2

(c) Design Load

(i) Earth Pressure

Coulomb Formula

Friction angle at wall $\delta = \phi = 31°$

Angle between back side of wall & vertical plane $\phi = -18.189°$ (atn. $^{2.3}/_{7.0}$)

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

(1) Allowable bearing capacity beneath slab

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

For $\phi = 32$, $N_c = 38$
$N_q = 22$
$N_\gamma = 20$

Df = 1.20 m

$k = 1 + 0.3 \times 1.20/2.10 = 1.17$

$q = 1.8 \times 1.2 = 2.16\ t/m^2$

$q_u = 1.17 \times 2.16 \times 22 + ½ \times 1.8 \times 1.0 \times 2.55 \times 20 = 101.498\ t/m^2$

$q_a = q_u / 3 = 33.833\ t/m^2$

(2) Coefficient of active earth pressure

$\phi = 32°$, $\alpha = 0$, $\theta = -18.189°$, $\delta = \phi = 32°$

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

$$= \frac{\cos^2(32 + 18.189)}{\cos^2(-18.189) \times \cos(-18.189 + 32) \left[1 + \sqrt{\frac{\sin(32 + 32) \times \sin(32)}{\cos(-18.189 + 32) \times \cos(-18.189)}}\right]^2}$$

$= 0.158$

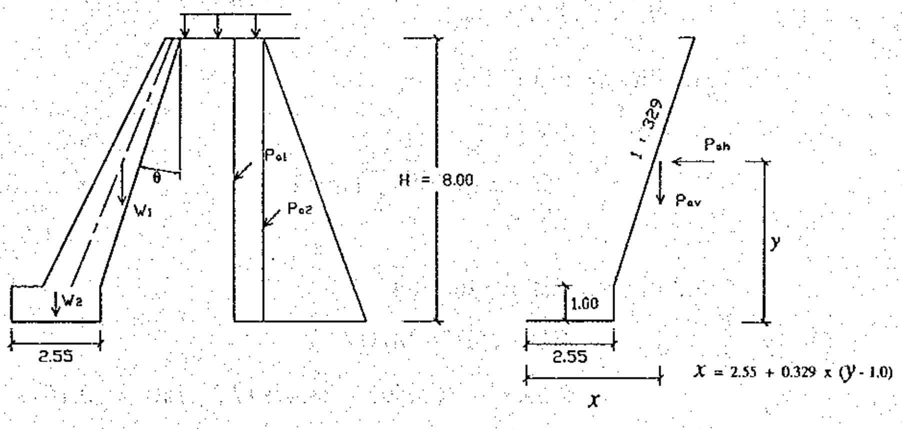
(3) Center Weight of structure

W_i (t/m)			x_i (m)		$W_i \cdot x_i$ (tm/m)	y_i (m)		$W_i \cdot y_i$ (tm/m)
W_1	$(0.45 + 1.65) \times {}^{7.0}/_{2} \times 2.35$	17.273	$0.9 + 1.65/2 + 0.4143 \times 2.833$	2.899	50.074	$1.0 + {}^{(2 \times 0.45 + 1.65)}/_{2.1} \times {}^{7}/_{3}$	3.833	66.207
W_2	$1.0 \times 2.55 \times 2.35$	5.992	2.55/2	1.275	7.640	10 / 2	0.500	1.000
Σ		23.265			57.714			67.207

$$x_o = \Sigma W_i \cdot x_i / \Sigma W = 57.714 / 23.265 = 2.481 \text{ m}$$
$$y_o = \Sigma W_i \cdot y_i / \Sigma W = 67.207 / 23.265 = 2.889 \text{ m}$$

(4) Stability Analysis

(i) Load Condition



$Pa_1 = q \cdot H \cdot Ka$

$= 1.0 \times 8.0 \times 0.158 = 1.264$ t/m

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

$= ½ \times (8.0)^2 \times 1.8 \times 0.158 = 9.101$ t/m

Horizontal Earth Pressure

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

$= 1.264 \times \cos(32 - 18.189) = 1.227$ t/m

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

$= 9.101 \times \cos(13.811) = 8.838$ t/m

Vertical Earth Pressure

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

$= 1.264 \times \sin(13.811) = 0.302$ t/m

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

$= 9.101 \times \sin(13.811) = 2.173$ t/m

(iii) Vertical Load and Moment

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
W	23.265	–		57.714
Pav_1	0.302	2.55 + 0.329 x (4.0 – 0.9)	3.570	1.078
Pav_2	2.173	2.55 + 0.329 x (8.0/3 – 0.9)	3.131	6.804
FV	25.740		MV	65.596

(iv) Horizontal Force and Moment

Horizontal Force (t/m)		Arm (m)		Moment (tm/m)
P_{ah1}	1.227	8.0/2	4.000	4.908
P_{ah2}	8.838	8.0/3	2.667	23.571
FH	10.065		MH	28.479

(v) Check of Stability

Stability against tilting

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

$= 25.740/10.065 \times \tan 32 \quad = 1.580$

> 1.5

Stability against overturning

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

$= 2.55/2 - (65.596 - 28.479) / 25.740 = 0.167 \text{ m} (-)$

$< {}^{B}/_{6} (=0.425 \text{ m})$

Stability of bearing strata

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

$q_{max} = {}^{25.740}/_{2.55} \times (1 + 6 \times 0.167/2.55) = 14.061 \text{ t/m}^2$

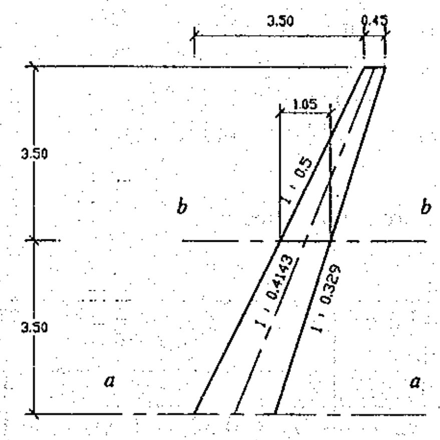
$< q_a (=33.833 \text{ t/m}^2)$

$q_{max} = {}^{25.740}/_{2.55} \times (1 - 6 \times 0.167/2.55) = 6.128 \text{ t/m}^2$

$\succ 0$

Stress strain analysis of wall

(i) Load Condition



(ii) Weight of structure

Sect. a	W_a	=	(0.45 + 1.65) x 7.0/2 x 2.35	=	17.273	t/m
	y_a	=	(2 x 0.45 + 1.65)/2.10 x 7.0/3	=	2.833	m
	x_a	=	2.833 x 0.4143 + 1.65/2	=	1.999	m
Sect. b	W_b	=	(0.45 + 1.05) x 3.5/2 x 2.35	=	6.169	t/m
	y_b	=	(2 x 0.45 + 1.05)/1.50 x 3.5/3	=	1.517	m
	x_b	=	1.517 x 0.4143 + 1.05/2	=	1.153	m

(iii) Earth Pressure

Section a.

$P_{a\,1a} = q \cdot Ka \cdot H$
$= 1.0 \times 0.158 \times 7.0 = 1.106$ t/m
$P_{a\,2a} = ½ \cdot H^2 \cdot \gamma_s \cdot Ka$
$= ½ \cdot (7.0)^2 \times 1.8 \times 0.158 = 6.968$ t/m

Horizontal Earth Pressure
$P_{ah\,1a} = P_{a\,1a} \times \cos(\delta + \theta)$
$= 1.106 \times \cos(13.811) = 1.074$ t/m
$P_{ah\,2a} = 6.968 \times \cos(13.811) = 6.766$ t/m

Vertical Earth Pressure
$P_{av\,1a} = P_{a\,1a} \times \sin(\delta + \theta)$
$= 1.106 \times \sin(13.811) = 0.264$ t/m
$P_{av\,2a} = 6.968 \times \sin(13.811) = 1.663$ t/m

Section b.
$P_{a\,1b} = 1.0 \times 0.158 \times 3.50 = 0.553$ t/m
$P_{a\,2b} = ½ \cdot (3.5)^2 \times 1.8 \times 0.158 = 1.742$ t/m

Horizontal Earth Pressure

$P_{ah\ 1b}$ = $P_{a\ 1b}$ x cos ($\delta + \theta$)

= 0.553 x cos (13.811) = 0.537 t/m

$P_{ah\ 2b}$ = 1.742 x cos (13.811) = 1.692 t/m

Vertical Earth Pressure

$P_{ah\ 1b}$ = $P_{a\ 1b}$ x sin ($\delta + \theta$)

= 0.553 x sin (13.811) = 0.132 t/m

$P_{av\ 2b}$ = 1.742 x sin (13.811) = 0.416 t/m

(iv) Vertical Force and Moment

at section a

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
W_a	17.273		1.999	34.529
P_{av1a}	0.264	${}^{7.0}/_2$ x 0.329 + 1.65	2.802	0.740
P_{av2a}	1.663	${}^{7.0}/_3$ x 0.329 + 1.65	2.418	4.021
FV_a	19.200		MV_a	39.290

at section b

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
W_b	6.169		1.153	7.113
P_{av1b}	0.132	${}^{3.5}/_2$ x 0.329 + 1.05	1.626	0.215
P_{av2b}	0.416	${}^{3.5}/_3$ x 0.329 + 1.05	1.434	0.597
FV_b	6.171		MV_b	7.924

(v) Horizontal Force and Moment

at section a

Horizontal Force (t/m)		Arm (m)		Moment (tm/m)
P_{ah1a}	1.074	7.0/2	3.500	3.759
P_{ah2a}	6.766	7.0/3	2.333	15.785
FH_a	7.840		MH_a	19.544

at section b

Horizontal Force (t/m)		Arm (m)		Moment (tm/m)
P_{ah1b}	0.553	3.5/2	1.750	0.968
P_{ah2b}	1.742	3.5/3	1.167	2.033
FH_b	2.295		MH_b	3.001

(vi) Check of Stresses

$e = B/2 - (MV - MH)/FV$
$q = FV/b \cdot (1 \pm 6e/B)$

at section a.

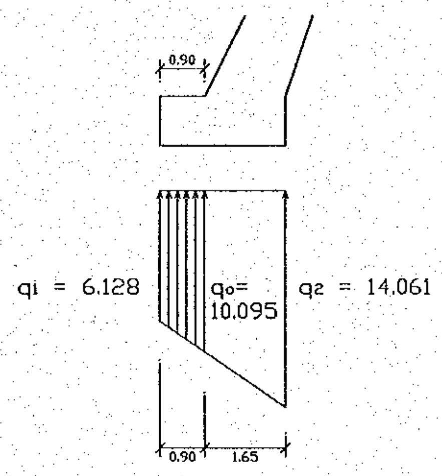
$e = 1.65/2 - (39.290 - 19.544)/19.200 = 0.203$ m (–)
$q_{max} = 19.200/1.65 \times (1 + 6 \times 0.203/1.65) = 20.226$ t/m^2
$= 2.023$ kg/cm^2
$< \sigma_{ca} (= 60$ kg/cm$^2)$

at section b.

$e = 1.05/2 - (7.924 - 3.001)/6.717 = 0.208$ m (–)
$q_{max} = 6.717/1.05 \times (1 + 6 \times 0.208/1.05) = 14.001$ t/m^2
$= 1.400$ k/cm^2
$< \sigma_{ca} (= 60$ kg/cm$^2)$

(5) Stress strain analysis of slab

(i) Load Condition



(ii) Weight of Structure

$W = 0.9 \times 1.0 \times 2.35 = 2.115$ t/m
$M_w = 2.115 \times 0.9/2 = 0.952$ tm/m

(iii) Reaction Beneath Slab

$Q = (6.128 + 10.095) \times 0.9/2 = 7.300$ t/m
$x_q = (6.128 \times 2 + 10.095)/16.233 \times 0.9/3 = 0.413$ m
$M_q = 7.300 \times 0.413 = 3.015$ t/m

(iv) Check of Stress

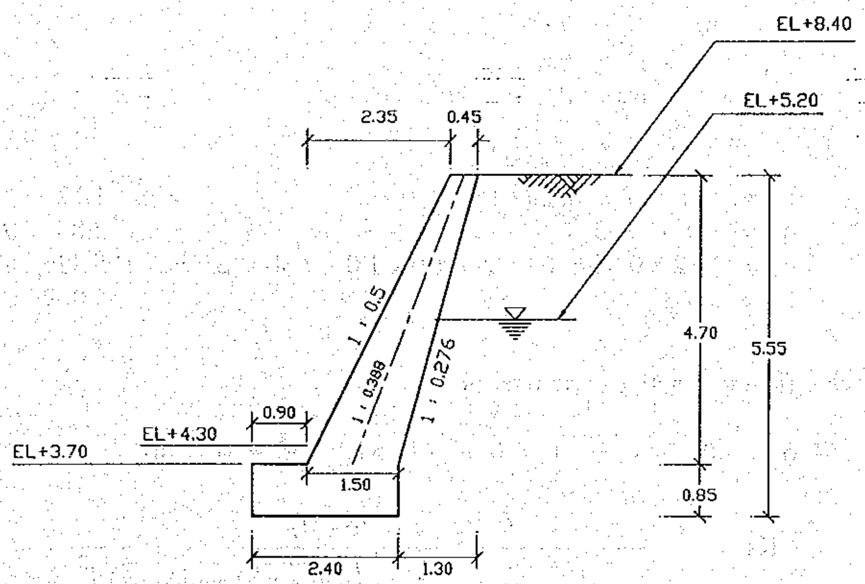
M =	$M_q - M_W$	= $3.015 - 0.952$	=	2.063	tm/m
S =	$Q - W$	= $7.300 - 2.115$	=	5.185	t/m
I =	$bh^3 / 12$	= $100 \times 100^3 / 12$	=	8,333,333.000	cm^4
Z =	I / y	= $8,333,333 / 50$	=	166,666.667	cm^3
σ_c =	M / Z	= $206,300/166,666.667$	=	1.238	kg/cm^2
			<	$\sigma_{ca} = 60$	kg/cm^2
τ_c =	S / A	= $5185 / (100 \times 100)$	=	0.519	kg/cm^2
			<	$\tau_{ca} = 5.5$	kg/cm^2

3.2.2.2 Leaning Wall for Wet Stone Masonry Type

Consideration Section: WF.105L to WF.110L

(1) Design Condition

(a) Proposed Section



(b) The Material Data

(iii) 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

(iv) 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

(iii) 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)

(iv) Surcharge Load $q = 1.00$ t/m²

(2) Allowable bearing capacity beneath slab

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

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/m²
$q_u = 1.12 \times 0.85 \times 63 + ½ \times 1.0 \times 1.0 \times 2.30 \times 83$	=	155.426	t/m²
$q_a = q_u / 3$	=	55.809	t/m²

(3) Coefficient of active earth pressure

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

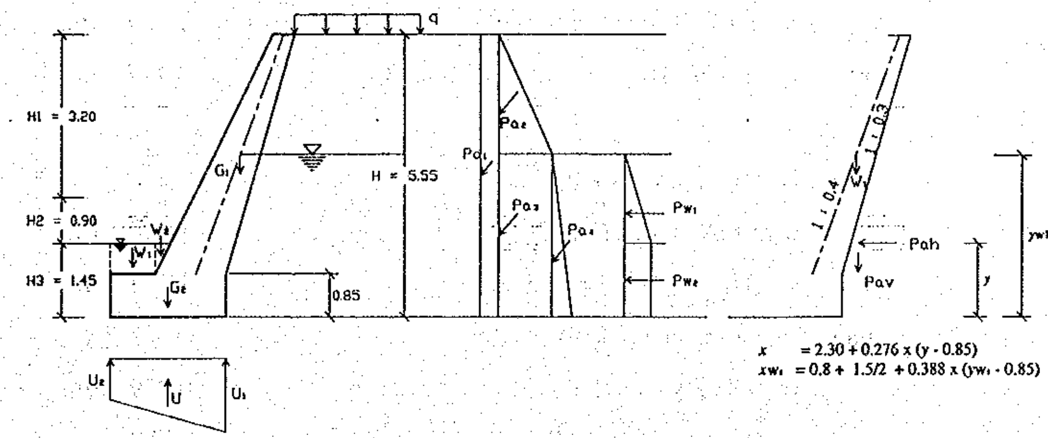
$$Ka = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cdot \cos(\phi + \delta) \cdot \left[1 + \sqrt{\dfrac{(\sin\phi + \delta) \cdot \sin(\phi - \alpha)}{\cos(\theta + \delta) \cdot \cos(\theta - \alpha)}}\right]^2}$$

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

$$= 0.111$$

(4) Stability Analysis

(vi) Load Condition



x_2 = 2.30 / 2 = 1.150 m

(viii) 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

(ix) Earth Pressure

Pa_1 = $q \cdot H \cdot Ka$

= 1.0 x 5.55 x 0.111 = 0.616 t/m^2

Pa_2 = $\frac{1}{2} \cdot H_1^2 \cdot \gamma_s \cdot Ka$

= ½ x $(3.2)^2$ x 2.0 x 0.111 = 1.137 t/m^2

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

= 3.2 x 2.0 x 0.111 x 2.35 = 1.669 t/m^2

Pa_4 = $\frac{1}{2} \cdot (H_2 + H_3)^2 \cdot \gamma_s \cdot Ka$

= ½ x $(2.35)^2$ x 1.0 x 0.111 = 0.306 t/m^2

Horizontal Earth Pressure

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

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

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

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

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

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

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

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

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

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

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

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

Vertical Earth Pressure

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

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

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

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

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

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

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

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

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

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

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

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

Passive earth pressure : to be omitted

(x) Water Pressure

Vertical water pressure

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

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

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

W_2 = ½ x 0.6 x (0.6/2) x 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 x $(0.9)^2$ x 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 x 1.0 x 1.45 = 1.305 m

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

(xi) 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

(xii) 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

(xiii) Check of Stability

Stability against tilting

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

$$= 12.785/5.102 \ \text{x} \ \tan 40 \qquad = 2.103$$

$$> 1.5$$

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

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

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

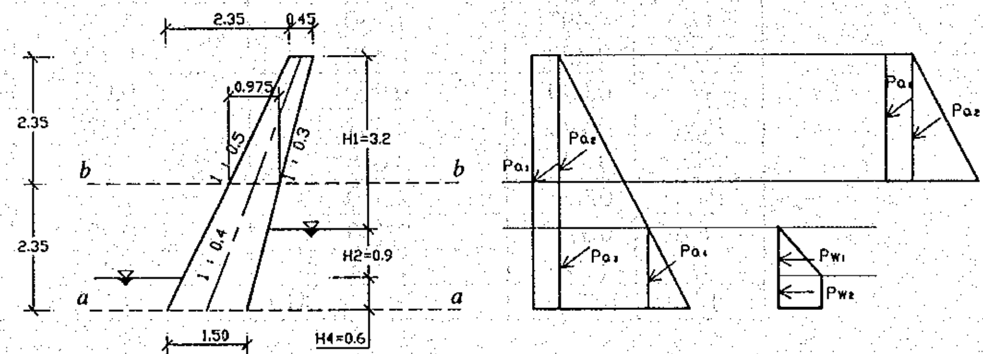
$$< q_a \ (= 51.809 \ \text{t/m}^2)$$

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

$$> 0$$

(5) Stress Strain Analysis of Wall

(i) Load Condition



Section a Section b

(ii) Load Condition

Section a

$W_a = (0.45 + 1.50) \text{ x } 4.70/2 \text{ x } 2.30 \qquad = 10.540 \text{ t/m}$

$y_a = (2 \text{ x } 0.45 + 1.5)/1.95 \text{ x } 4.7/3 \qquad = 1.928 \ \ \text{m}$

$x_a = 1.928 \text{ x } 0.388 + 1.50/2 \qquad = 1.498 \ \ \text{m}$

Section a

$W_b = (0.45 + 0.975) \text{ x } 2.35/2 \text{ x } 2.30 \qquad = 3.851 \ \ \text{t/m}$

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

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

$= 1.0 \times 0.111 \times 4.7 = 0.522$ t/m

$Pa_2\,a = \frac{1}{2}\ H_1^2 \cdot \gamma_s \cdot Ka$

$= \frac{1}{2} \times (3.2)^2 \times 2.0 \times 0.111 = 1.137$ t/m

$Pa_3\,a = H_1 \cdot \gamma_s \cdot Ka \cdot (H_2 + H_4)$

$= 3.2 \times 2.0 \times 0.111 \times 1.5 = 1.066$ t/m

$Pa_4\,a = \frac{1}{2}\ (H_2 + H_4)^2 \cdot \gamma_s' \cdot Ka$

$= \frac{1}{2} \times (1.5)^2 \times 1.0 \times 0.111 = 0.125$ t/m

Horizontal earth pressure

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

$= 0.522 \times \cos(24.539) = 0.475$ t/m

$Pah_2\,a = 1.137 \times \cos(24.539) = 1.034$ t/m

$Pah_3\,a = 1.066 \times \cos(24.539) = 0.969$ t/m

$Pah_4\,a = 0.125 \times \cos(24.539) = 0.114$ t/m

Vertical earth pressure

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

$= 0.522 \times \sin(24.539) = 0.217$ t/m

$Pav_2\,a = 1.137 \times \sin(24.539) = 0.472$ t/m

$Pav_3\,a = 1.066 \times \sin(24.539) = 0.443$ t/m

$Pav_4\,a = 0.125 \times \sin(24.539) = 0.052$ t/m

Section b

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

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

$Pa_2\,b = \frac{1}{2}\ H_1^2 \cdot \gamma_s \cdot Ka$

$= \frac{1}{2} \times (2.35)^2 \times 2.0 \times 0.111 = 0.613$ t/m

Horizontal earth pressure

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

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

Vertical earth pressure

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

$Pav_2\,b$ = 0.613 x sin (24.539) = 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 = B/2 – (MV – MH)/FV

q = FV/b · (1 ± 6e/B) for e < B/6

at section a.

$e = 1.50/2 - (18.139 - 4.554)/11.724 = 0.409$ m

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

$= 2.060$ kg/cm^2

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

at section b.

$e = 0.975/2 - (3.821 - 0.715)/4.214 = 0.250$ m

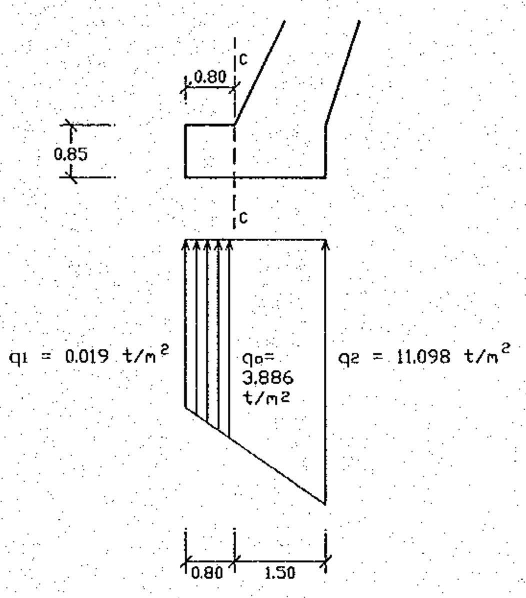
$q_{max} = 4.214/0.975 \times (1+6 \times 0.250/0.975) = 10.971$ t/m^2

$= 1.097$ kg/cm^2

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

(6) Stress strain analysis of slab

(v) Load Condition



(vi) Weight of Structure

$W = 0.8 \times 0.85 \times 2.3 = 1.564$ t/m

$M_w = 1.564 \times 0.8/2 = 0.626$ tm/m

(vii) Reaction Beneath Slab

$Q = (0.019 + 3.886)/2 \times 0.8 = 1.562$ t/m

x_q = (2 x 0.019 + 3.886)/3.905 x 0.8/3 = 0.268 m

M_q = 1.562 x 0.268 = 0.419 t/m

(viii) Check of Stress

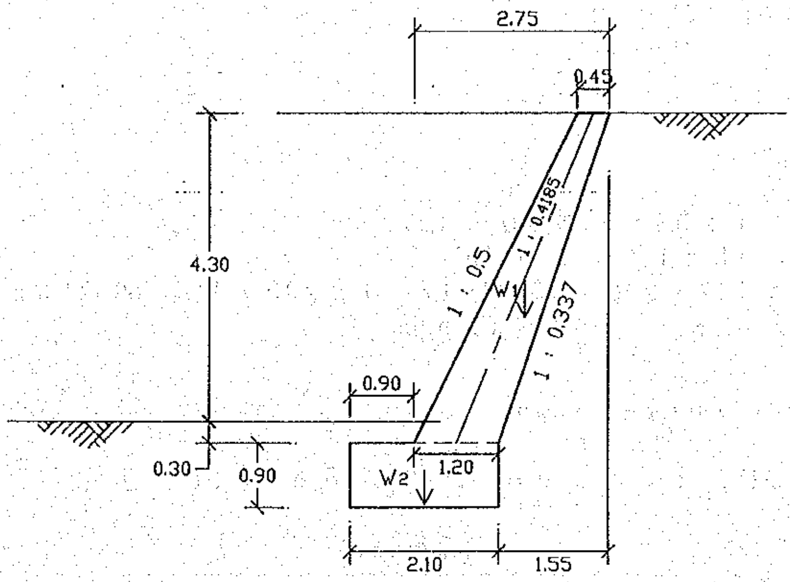
M =	Mq – Mw	= 0.626 – 0.419	=	0.207	tm/m
S =	Q – W	= 1.564 – 1.562	=	0.002	t/m
I =	$bh^3 / 12$	= $100 \times 85^3 / 12$	=	5,117,708.000	cm^4
Z =	I / *y*	= 5,117,708 / 42.5	=	120,416.667	cm^3
σ_c =	M / Z	= 20,700 / 120,416.667	=	0.172	kg/cm^2
			<	σ_{ca} (= 15	kg/cm^2)
τ_c =	S / A	= 2 / (100 x 85)	=	0.0002	kg/cm^2
			<	τ_{ca} (= 4.5	kg/cm^2)

3.2.2.3 Leaning Wall for Wet Stone Masonry Type

Consideration Section: WF.91R+25.0 to WF.94R+22.0

(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 RB 26)

N – value	N	=	17	
Soil internal friction angle	ϕ	=	31°	
Unit weight	γ_s	=	1.80	t/m^3
Cohesion	c	=	0.0	t/m^2

(c) Design Load

(i) Earth Pressure

Coulomb Formula

Friction angle at wall δ = ϕ (in ordinary)

Angle between back side of wall & vertical plane ϕ = 18.622°
(atn. $^{1.55}/_{4.6}$)

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

(2) Allowable bearing capacity beneath slab

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

For $\phi = 31$, $N_c = 30$
$N_g = 17$
$N_\gamma = 14$

$Df = 1.20\ m$

$k = 1 + 0.3 \times 1.20/2.10 = 1.17$
$q = 1.8 \times 1.20 = 2.16\ t/m^2$
$q_u = 1.17 \times 2.16 \times 17 + ½\ 1.8 \times 1 \times 2.10 \times 14 = 69.422\ t/m^2$
$q_a = q_u / 3 = 23.14\ t/m^2$ (in ordinary)

(3) Coefficient of active earth pressure

$\phi = 31°$, $\alpha = 0$, $\theta = -18.622$, $\delta = \phi = 31°$

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

$$= \frac{\cos^2(31+18.622)}{\cos^2(-18.622)\times\cos(-18.622+31)\times\left[1+\sqrt{\frac{\sin(31+31)\times\sin(31)}{\cos(-18.622+31)\times\cos(-18.622)}}\right]^2}$$

$$= 0.170$$

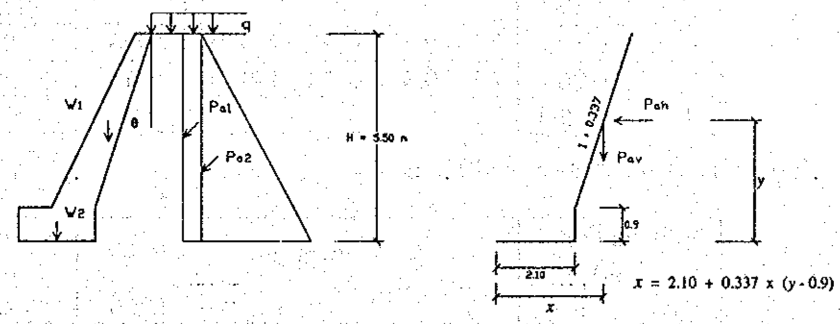
(4) Center Weight of structure

W_i (t/m)			x_i (m)		Wi x x_i (tm/m)	y_i (m)		Wi x y_i (tm/m)
W_1	$(0.45 + 1.2) \times {}^{4.6}/_2 \times 2.3$	8.729	$0.9 + 0.6 + 0.4185 \times 1.952$	2.317	20.225	$0.9 + {}^{(2\times0.45+1.2)}/_{1.65} \times {}^{4.6}/_3$	2.852	24.895
W_2	$0.9 \times 2.1 \times 2.3$	4.347	${}^{2.1}/_2$	1.050	4.564	${}^{0.9}/_2$	0.450	1.856
Σ		13.076			24.789			26.851

$x_o = \Sigma W_i \cdot x_i / \Sigma W = 24.789 / 13.076 = 1.895\ m$
$y_o = \Sigma W_i \cdot y_i / \Sigma W = 26.851 / 13.076 = 2.053\ m$

(5) Stability Analysis

(i) Load Condition



(ii) Earth Pressure

$Pa_1 = q \cdot H \cdot Ka$

$= 1.0 \times 5.5 \times 0.165 \qquad = 0.908$ t/m

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

$= ½ \cdot (5.5)^2 \cdot 1.8 \cdot 0.165 \qquad = 4.492$ t/m

Horizontal Earth Pressure

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

$= 0.908 \times \cos(31 - 18.622) \qquad = 0.886$ t/m

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

$= 4.492 \times \cos(12.378) \qquad = 4.338$ t/m

Vertical Earth Pressure

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

$= 0.908 \times \sin(12.378) \qquad = 0.195$ t/m

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

$= 4.492 \times \sin(12.378) \qquad = 0.963$ t/m

(iii) Vertical Load and Moment

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
W	13.076	–		22.607
Pav_1	0.195	2.1 + 0.337 x (2.75 – 0.9)	2.723	0.531
Pav_2	0.963	2.1 + 0.337 x (1.833–0.9)	2.414	2.325
FV	14.234		MV	25.463

(iv) Horizontal Force and Moment

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
Pav_1	0.886	$^{5.5}/_2$	2.750	2.436
Pav_2	4.338	$^{5.5}/_3$	1.833	7.953
FH	5.224		MH	10.389

(v) Check of Stability

Stability against tilting

$$SF = FV / FH \cdot \tan\phi$$
$$= 14.234/5.224 \ \text{x} \ \tan 31 \qquad = 1.637$$
$$\sim 1.5$$

Stability against overturning

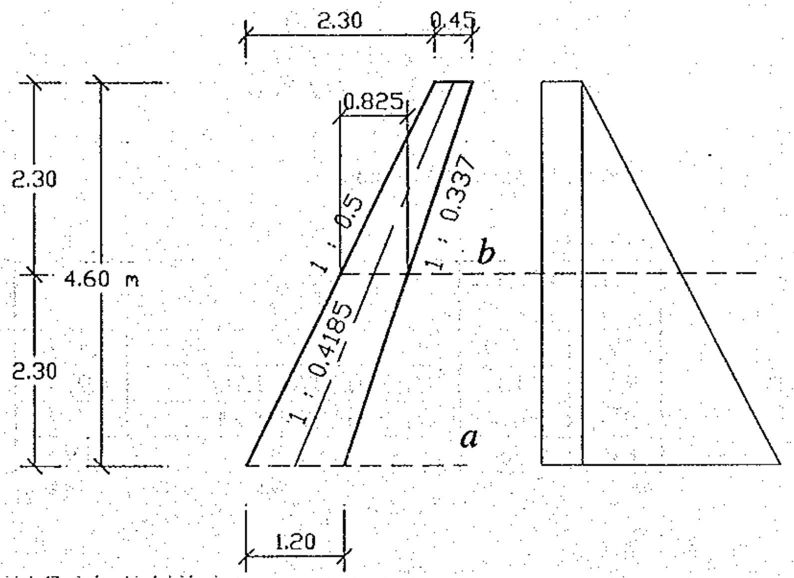
$$e = B/2 - (MV - MH) / FV$$
$$= 2.10 / 2 - (25.463 - 10.389) / 14.234 = 0.009 \ \text{m} (-)$$
$$< {}^{B}/_{6} \ (=0.35 \text{ m})$$

Stability of bearing strata

$$q = FV / B\,(1 \pm 6e/B)$$
$$q_{max} = 14.234/_{2.1} \ \text{x} \ (1 + 6 \ \text{x} \ 0.009/2.1) = 6.790 \ \text{t/m}^2$$
$$< q_a \ (=23.141 \text{ t/m}^2)$$
$$q_{max} = 14.234/_{2.1} \ \text{x} \ (1 - 6 \ \text{x} \ 0.009/2.1) = 6.604 \ \text{t/m}^2$$
$$> 0$$

(6) Stress strain analysis of wall

(vii) Load Condition



(vi: ,

Sect. a	W_a	=	(0.45 + 1.2)/2 x 4.6 x 2.3	=	8.729	t/m
	y_a	=	(2 x 0.45 + 1.2)/1.65 x 4.6/3	=	1.952	m
	x_a	=	1.952 x 0.4185 + 0.6	=	1.417	m
Sect. b	W_b	=	(0.45 + 0.825)/2 x 2.30 x 2.30	=	3.505	t/m
	y_b	=	(2 x 0.45 + 0.825)/1.275 x 2.3/3	=	1.037	m
	x_b	=	1.037 x 0.4185 + 0.6	=	1.034	m

(ix) Earth Pressure

Section a.

$P_{a\,1a} = q \cdot Ka \cdot H$

$= 1.0 \times 0.165 \times 4.6 = 0.759$ t/m

$P_{a\,2a} = \frac{1}{2} \cdot H^2 \cdot \gamma_s \cdot Ka$

$= \frac{1}{2} \cdot (4.6)^2 \times 1.8 \times 0.165 = 3.142$ t/m

Horizontal Earth Pressure

$P_{ah\,1a} = P_{a\,1a} \times \cos(\delta + \theta)$

$= 0.759 \times \cos(12.378) = 0.741$ t/m

$P_{ah\,2a} = 3.142 \times \cos(12.378) = 3.069$ t/m

Vertical Earth Pressure

$P_{av\,1a} = P_{a\,1a} \times \sin(\delta + \theta)$

$= 0.759 \times \sin(12.378) = 0.163$ t/m

$P_{av\,2a} = 3.142 \times \sin(12.378) = 0.674$ t/m

Section b.

$P_{a\,1b} = 1.0 \times 0.165 \times 2.30 = 0.380$ t/m

$P_{a\,2b} = \frac{1}{2} \cdot (2.3)^2 \times 1.8 \times 0.165 = 0.786$ t/m

Horizontal Earth Pressure

$P_{ah\ 1b} = P_{a\ 1b}\ x\ \cos(\delta + \theta)$

$= 0.380\ x\ \cos(12.378) \quad = \quad 0.371 \quad t/m$

$P_{ah\ 2b} = 0.786\ x\ \cos(12.378) \quad = \quad 0.767 \quad t/m$

Vertical Earth Pressure

$P_{ah\ 1b} = P_{a\ 1b}\ x\ \sin(\delta + \theta)$

$= 0.380\ x\ \sin(12.378) \quad = \quad 0.081 \quad t/m$

$P_{av\ 2b} = 0.786\ x\ \sin(12.378) \quad = \quad 0.168 \quad t/m$

(x) Vertical Force and Moment

at section a

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
W_a	8.729		1.417	12.369
P_{av1a}	0.163	$^{4.6}/_2$ x 0.337 + 1.20	1.975	0.322
P_{av2a}	0.674	$^{4.6}/_3$ x 0.337 + 1.20	1.717	1.157
FV_a	9.566		MV_a	13.848

at section b

Vertical Force (t/m)		Arm (m)		Moment (tm/m)
W_b	3.505		1.034	3.624
P_{av1b}	0.081	$^{2.3}/_2$ x 0.337 + 0.825	1.213	0.098
P_{av2b}	0.168	$^{2.3}/_3$ x 0.337 + 0.825	1.083	0.182
FV_b	3.754		MV_b	3.904

(xi) Horizontal Force and Moment

at section a

Horizontal Force (t/m)		Arm (m)		Moment (tm/m)
P_{ah1a}	0.741	$^{4.6}/_2$	2.300	1.704
P_{ah2a}	3.069	$^{4.6}/_3$	1.533	4.705
FH_a	3.810		MH_a	6.409

at section b

Horizontal Force (t/m)		Arm (m)		Moment (tm/m)
P_{ah1b}	0.371	$^{2.3}/_2$	1.150	0.427
P_{ah2b}	0.767	$^{2.3}/_3$	0.767	0.588
FH_b	1.138		MH_b	1.015

(xii) Check of Stresses

$$e = B/2 - (MV - MH)/FV$$
$$q = FV/b \cdot (1 \pm 6e/B)$$

at section a.

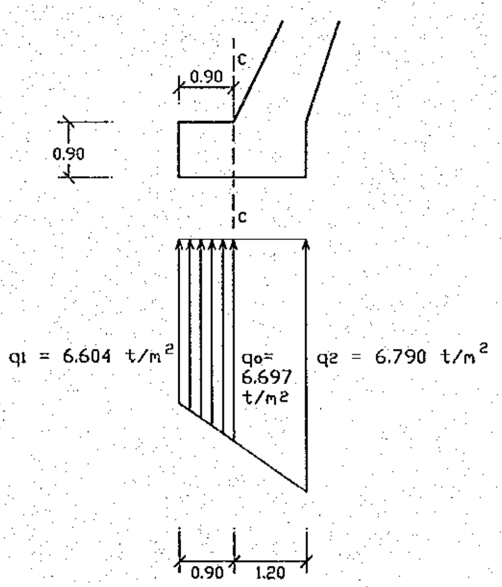
$$e = 1.20/2 - (13.848 - 6.409)/9.566 = 0.18 \ \text{m}$$
$$q_{max} = 9.566/1.2 \times (1 + 6 \times 0.18/1.2) = 15.146 \ \text{t/m}^2 = 1.515 \ \text{kg/cm}^2 < \sigma_{ca}(= 15 \ \text{kg/cm}^2)$$

at section b.

$$e = 0.825/2 - (3.904 - 1.015)/3.754 = 0.357 \ \text{m}$$
$$q_{max} = 3.754/0.825 \times (1 + 6 \times 0.357/0.825) = 16.365 \ \text{t/m}^2 = 1.637 \ \text{kg/cm}^2 < \sigma_{ca}(= 15 \ \text{kg/cm}^2)$$

(7) Stress strain analysis of slab

(ix) Load Condition



(x) Weight of Structure

$$W = 0.9 \times 0.9 \times 2.3 = 1.863 \ \text{t/m}$$
$$M_w = 1.863 \times 0.9/2 = 0.838 \ \text{tm/m}$$

(xi) Reaction Beneath Slab

Q	=	$(6.604 + 6.697) \times 0.9/2$	=	5.985	t/m
x_q	=	$(6.604 \times 2 + 6.697)/13.301 \times 0.9/3$	=	0.449	m
M_q	=	5.985×0.449	=	2.687	t/m

(xii) Check of Stress

$M = Mq - M_W$	$= 2.687 - 0.838$	=	1.849	tm/m
$S = Q - W$	$= 5.985 - 1.863$	=	4.122	t/m
$I = bh^3/12$	$= 100 \times 90^3/12$	=	6,075,000	cm^4
$Z = I/y$	$= 6,075,000/45$	=	135,000	cm^3
$\sigma_c = M/Z$	$= 184,900/135,000$	=	1.370	kg/cm^2
		<	σ_{ca} (= 15	kg/cm^2)
$\tau_c = S/A$	$= 4122/(100 \times 90)$	=	0.458	kg/cm^2
		<	τ_{ca} (= 4.5	kg/cm^2)

CHAPTER 4 SIMONGAN WEIR

4.1 Scope of Work

Aiming at flood control in Garang River and water supply for PDAM intake and intake facilities for Semarang River and the left bank irrigation channel, the existing fixed type weir is to be reconstructed as a gated weir composed of the following structures. The weir is constructed at the same location as the existing one.

① Flood discharge gates,
② Sediment flush gates,
③ Main weir body comprised of gate piers, gate floor slabs and operation decks,
④ Stilling basin, concrete apron and side approach walls,
⑤ Intake structures for Semarang River and left bank irrigation channel,
⑥ Maintenance and approach bridges,
⑦ Gate control houses, Operation/Management office building, and
⑧ Protection works for riverbed/riverbank of up and downstream channels

Based on the definitive plan (basic design), the detailed design of the weir is being carried out. The progress of the detailed design works is presented below.

Design Conditions and Structural Dimensions

Design conditions determined in the basic design are summarized below.

(a) Channel Dimensions and Elevations of Structure

- Design flood discharge	790 m^3/s
- Width of channel (Flood discharge portion)	60.5 m
- Water level	
High water level	EL. 8.000
Normal water level (upstream channel)	EL. 5.200
- Elevation of structure	
Design dike crown	EL. 9.000
Gate floor elevation (design Riverbed)	EL. 1.500
Floor level of stilling basin	EL. 0.900
Lower end of riverbed protection	EL. -0.100
- Underside elevation of gate when opened	EL. 9.000
- Top elevation of gate pier	EL. 15.600
- Floor elevation of intake (right bank)	EL. 3.800
- Floor elevation of intake (left bank)	EL. 4.000

(b) Structural Dimensions of Weir and Gate

The structural dimensions of weir are summarized in the following table.

Gate

Item	Dimension
- Flood discharge gate	3 gates
Gate type	Shell type steel roller gate
Height	3.70 m
Clear span length	18.50 m
- Sediment discharge gate	2 gates
Gate type	Girder type steel roller gate
Height	4.35 m
Clear span length	5.50 m
- Right intake gate	for Semarang River
Gate type	Steel slide gate
Dimension	H=2.0m x W=2.25m x 4
Water intake	0.50 m^3/s
- Left intake gate	for Irrigation Channel
Gate type	Steel slide gate
Dimension	H=2.0m x W=2.00m x 2
Water intake	0.15 m^3/s

Weir Structure

Item	Dimension
- Number of pier	Center pier : 4, End pier : 2
- Center pier	
Length (flow direction)	16.5 m
Width (right angle to flow)	2.5 m x 4 units
Height	14.1 m
Footing	18.5 m x 8.0 m x 2.2/1.6 m
- End pier	
Length (flow direction)	16.5 m
Width (right angle to flow)	2.00 m x 2 units
Height	14.1 m
Footing	18.5 m x 15.0 m x 2.2/1.6 m
- Gate floor slab	18.5 m x 13.0 m x 2.0/1.4 m
- Stilling basin	L=20.0 m, Depth=0.6 m
- Concrete apron (Downstream-1)	76.5m x 15.0m x 1.2m
- Concrete apron (Downstream-2)	79.5m x 10.0m x 1.0m
- Concrete apron (Upstream)	76.5m x 10.0m x 7.5m
- Approach wall (Downstream-1)	H=9.1m, L=15.0m
- Approach wall (Downstream-2)	H=7.5m, L=10.0m
- Approach wall (Upstream)	H=8.5m, L=15.0m
- Deck of Control House	6.7m x 6.7m x 2units, 13.05m x 6.70m x 2units

4.2 Main Weir

4.2.1 Design Condition

(a) Established position of Simongan Weir	WF.99+18.50 m
(b) Design dike crown level	EL +9.00 m
(c) Design riverbed level	EL +1.50 m
(d) Top of flood discharge gate elevation	EL +5.20 m

(d) Hydraulic condition

On gate operation rule of Simongan Weir, when Garang river will get design discharge that is above a 70.0 m^3/s (overflowing water depth is about 0.65 m), flood discharge gates of Simongan Weir will be opened.

Maintenance water requirement is usually discharged from Jatibarang Dam. Therefore, in stability and structure calculation, hydraulic conditions are decided as follows.

(i) Normal condition	
Design discharge	Q= 70.0 m^3/s
Upstream water level at gate	EL+5.85 m
Downstream water level at gate	EL+1.55 m
(ii) Design flood condition	
Design discharge	Q= 790.0 m^3/s
Upstream water level at gate	EL+8.00 m
Downstream water level at gate	EL+8.00 m
(iii) Constructional condition	
Design discharge	none
Upstream water level at gate	Bottom of structure
Downstream water level at gate	Bottom of structure
(iv) Seismic condition	
Design discharge	Q= 70.0 m^3/s
Upstream water level at gate	EL+5.85 m
Downstream water level at gate	EL+1.55 m

(1) Design Criteria

(a) Material

(i) Concrete

Reinforced concrete	σ ck = 250 kgf/cm^2 or equivalent
Leveling concrete	σ ck = 175 kgf/cm^2 or equivalent
Secondary concrete	σ ck = 225 kgf/cm^2 or equivalent

(ii) Concrete fabrication

PC pile	σck = 500 kgf/cm^2 or equivalent
Concrete sheet pile	σck = 350 kgf/cm^2 or equivalent

(iii) Reinforcing bar

Deformed bar	JIS SD 295 or equivalent
Round bar	JIS SR 235 or equivalent

(iv) Structural steel

H-steel pile	JIS SS 400 or equivalent
Steel sheet pile	JIS SY 295 or equivalent
Gate	

(v) Backfill and filling materials

Soil	Sand

(vi) Others

Other materials are followed by " DESIGN CRITERIA REPORT (VOLUME one (1))".

(b) Unit weight

(i) Concrete

Reinforced concrete	$\gamma c = 2.50$ tf/m^3
Plain concrete	$\gamma c' = 2.35$ tf/m^3
(ii) Steel material	$\gamma s = 7.85$ tf/m^3

(iii) Backfill and filling materials

Wet unit weight	$\gamma t = 1.80$ tf/m^3
Submerged unit weight	$\gamma' = 1.00$ tf/m^3
Internal friction angle	$\phi = 30.00^\circ$
Cohesion	C= 0.00 tf/m^2
(iv) Timber	$\gamma = 0.30$ tf/m^3
(v) Asphalt pavement	$\gamma = 2.30$ tf/m^3

(vi) Others

Other materials are followed by " DESIGN CRITERIA REPORT (VOLUME one (1))".

(c) Allowable strength capacity of materials

(i) Reinforced concrete

Allowable compressive stress due to bending	$\sigma ca = 75.0$ kgf/cm^2
Allowable axial compressive stress	$\sigma ca = 60.0$ kgf/cm^2
Allowable shearing stress	$\tau ca1 = 3.8$ kgf/cm^2
Allowable punching shear stress	$\tau ca3 = 8.8$ kgf/cm^2
Allowable bond stress (for unformed bar)	$\tau ba = 15.0$ kgf/cm^2

The value of allowable bearing stress at bottom slab concrete for pile head treatment work is described as follows.

Consideration item		Allowable stress (kgf/cm^2)	
		Normal	Earthquake
Allowable vertical bearing stress	σbav	112	168
Allowable horizontal bearing stress	σbah	67	100
Allowable punching shear stress	τca3	8.8	

(ii) Reinforced bar

Allowable tensile stress σsa = 1,600 kgf/cm^2 (underwater)

= 1,800 kgf/cm^2 (Aerial)

(iii) Steel material (SS 400)

Allowable axial tensile stress σsa = 1,400 kgf/cm^2

Allowable bending tensile stress σsa = 1,400 kgf/cm^2

Allowable axial compressive stress

In case of $l/r \leq 20$ $\sigma sa = 1{,}400\ kgf/cm^2$

In case of $20 < l/r \leq 93$ $\sigma sa = 1{,}400 - 8.4 \times (l/r - 20)\ kgf/cm^2$

In case of $l/r > 93$ $\sigma sa = \dfrac{12{,}000{,}000}{6{,}700 + (l/r)}\ kgf/cm^2$

Where, l : Member of effective buckling length (cm)

r : Radius of gyration in member (cm)

Allowable shearing stress τsa = 800 kgf/cm^2

But in temporary construction, the above values show a 50 % increase.

(iv) Others

Other materials are followed by " DESIGN CRITERIA REPORT (VOLUME one (1))".

(d) Extra allowable stress

In case of temperature change a 15 % increase

In case of earthquake a 50 % increase

In case of temperature change with earthquake a 65 % increase

(e) Design load

(i) Earth pressure

The calculation of earth pressure follows Coulomb's formula. The value of friction angle at wall (δ) is shown as follows.

Friction angle at wall (δ)

Kind of calculation	Item	Normal condition	Earthquake condition
Stability calculation	Soil to soil	ϕ	$\phi/2$
Structural calculation	Soil to concrete	$\phi/3$	0

Where, ϕ : internal friction angle of soil (degree)

(ii) Surcharge load

Normal condition $q = 1.00$ tf/m^2

Construction $q = 1.00$ tf/m^2

Design flood condition $q = 1.00$ tf/m^2

Earthquake condition $q' = 0.50$ ft/m^2

(iii) Wind load W= 0.15 tf/m^2

(to horizontal projective area)

(iv) Sidewalk live load W= 0.30 tf/m^2

(v) Shed load W= 0.50 tf/m^2

(vi) Temperature change

The assumed ambient temperature for design purpose is 30 ℃, and structure are designed for a variation of minimum 15 ℃ to maximum 40 ℃. The coefficients of liner expansion of 1.0×1.0^{-5} and 1.2×1.0^{-5} are used for concrete member and steel member.

(vii) Muddy soil pressure

Flood discharge gates of Simongan Weir have to be always shut to take the waters of Garang river.

It assumes that muddy soil is accumulated on gate floor slab until one (1) meter depth.

(viii) Earthquake load

There is Simongan Weir that is located between two zone which has a difference factor (Z).on geographic position, therefore, the average value of those different factors will be adopted for detailed design. Formula is shown on " DESIGN CRITERIA REPORT (VOLUME one (1))",while any others factor are shown as follows.

ac = 160 cm/s^2 (design shock acceleration: return period 100 years)

n = 1.56 (coefficient for soil type : alluvium)

m = 0.89 (coefficient for soil type : alluvium) and

g = 980 cm/s^2 (acceleration of gravity)

In case of Z = 0.56

$$ad_1 = n(ac \times Z)^m = 1.56 \times (160 \times 0.56)^{0.89} = 85.25 \text{ cm/s}^2$$

In case of Z = 1.00

$$ad_2 = n(ac \times Z)^m = 1.56 \times (160 \times 1.00)^{0.89} = 142.82 \text{ cm/s}^2$$

$$Ave(ad) = (ad_1 + ad_2)/2 \neq 114.04$$

Therefore,

$$E = ad/g = 114.04/980 = 0.116 \fallingdotseq 0.12$$

(ix) Others

Other materials are followed by " DESIGN CRITERIA REPORT (VOLUME one (1))".

(f) The numbers

(i) Elastic modulus

In case of calculation of elastic deformation

Steel materials	Es = 2.1×10^6 kgf/cm^2
Concrete	Ec =2.45×10^5 kgf/cm^2

In case of stress calculation of member

Reinforced bar	Es = 2.1×10^6 kgf/cm^2
Concrete	Ec = 1.4×10^5 kgf/cm^2

(ii) Safety factor of foundation

The pile foundation with bearing layer is introduced into Simongan Weir, because of thinking about resistance of ground subsidence around Semarang City and importance of intake structure.

Safety factor of pile foundation is shown as follows.

Allowable compressive bearing capacity

Loading condition	Safety factor
	Bearing pile
Normal case	3
Seismic case	2

Allowable axial pull-out capacity

Loading condition	Safety factor
	Bearing pile
Normal case	6
Seismic case	3

Allowable displacement

Kind of displacement	Normal case	Seismic case
Horizontal displacement	10 mm	15 mm
Vertical displacement	10～20 mm	10～20 mm
Angle of inclination	1/500～1/1,000 radian	1/500～1/1,000 radian

(iii) Minimum thickness of structure member

Minimum thickness of reinforced concrete member shall be 30 cm for river structures subjecting to river current about workability.

(iv) Minimum concrete cover

Protective cover for reinforced bar shall not be less than the following.

The following numbers are shown the distance between concrete surface and center of reinforced bar.

Important concrete footing and slab exposed to soil	115 mm
Lower side of footing supported by pile foundation	165 mm
Important structures exposed to weather or backfilling soil or under water	90 mm
Not exposed to earth or weather	50 mm

4.2.2 Gate pier

4.2.2.1 Center Pier

1. Loading Calculation

Weight of body

Total weights of center weir pier are shown as follows.

Table of total weight of center pier

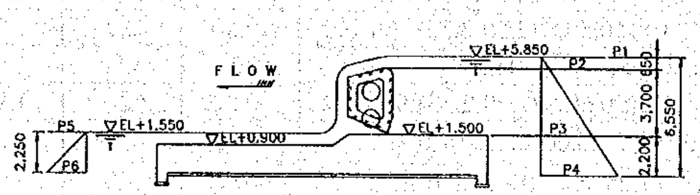
No.	Calculation form	Weight (tf)	X (m)	Y (m)	Z (m)
1	$1.60\times18.50\times8.00\times2.50$ tf/m^3	592.00	9.25	0.80	4.00
	$-(1.10\times3.05\times0.50\times(2.50-2.35)tf/m^3\times2$	-0.50	10.75	1.35	4.00
2	$7.75\times0.60\times8.00\times2.50$ tf/m^3	93.00	3.88	1.90	4.00
	$-(1.00\times3.45\times0.50\times(2.50-2.35)tf/m^3\times2$	-0.52	6.51	1.95	4.00
3	$(0.90\times0.60)/2\times8.00\times2.50$ tf/m^3	5.40	8.05	1.80	4.00
4	$0.50\times\pi\times1.25^2\times7.50\times2.50$ tf/m^3	46.02	1.72	5.95	4.00
5	$3.00\times7.50\times2.50\times2.50$ tf/m^3	140.63	3.75	5.95	4.00
6	$(0.75\times7.50\times0.70\times2.35$ tf/m$^3)\times2$	18.51	5.63	5.95	4.00
7	$1.10\times2.50\times7.50\times2.50$ tf/m^3	51.56	6.50	5.95	4.00
8	$(0.75\times7.50\times0.70\times2.35tf/m^3)\times2$	18.51	7.38	5.95	4.00
9	$(0.90\times0.60)/2\times2.50\times2.50$ tf/m^3	1.69	8.35	2.00	4.00
10	$1.05\times0.60\times2.50\times2.50$ tf/m^3	3.94	9.18	1.90	4.00
11	$(0.30\times8.10\times0.50\times2.35tf/m^3)\times2$	5.71	9.85	5.65	4.00
12	$(0.50\times0.30\times8.10\times2.35tf/m^3)\times2$	5.71	10.65	5.65	4.00
13	$1.50\times1.10\times8.10\times2.50$ tf/m^3	33.41	10.25	5.65	4.00
14	$5.45\times8.10\times2.50\times2.50$ tf/m^3	275.91	14.03	5.65	4.00
15	$1.00\times2.25\times2.50\times2.50$ tf/m^3	14.06	17.38	9.20	4.00
16	$0.50\times\pi\times1.25^2\times7.10\times2.50$tf/m^3	43.57	16.78	5.15	4.00
17	$(0.85\times1.25\times5.40\times2.50tf/m^3)\times4$	57.38	6.50	12.40	4.00
18	$(0.40\times0.80-0.50\times\pi\times0.40^2)\times1.25\times2\times2.50$ tf/m^3	0.43	6.50	12.05	4.00
19	$(0.80\times1.25\times3.00\times2.50$ tf/m$^3)\times2$	15.00	6.50	13.60	4.00
20	$(1.20\times1.20)/2\times0.80\times2.50$ tf/m^3	1.44	9.80	10.10	4.00
21	$(0.30\times0.30)/2\times0.80\times2.50$ tf/m$^3\times4$	0.36	9.60	10.30	4.00
22	$0.50\times1.50\times1.50\times2.50$ tf/m^3	2.81	10.75	10.45	4.00
23	$0.5/6\times\left\{\begin{array}{l}(7.00\times7.00)+(2.50\times5.00)\\+4\times(6.00\times4.75)\end{array}\right\}\times2.50$	36.56	6.50	15.36	4.00
24	$7.00\times7.00\times0.70\times2.50$ tf/m^3	85.75	6.50	15.95	4.00
25	$7.50\times1.95\times2.50\times2.50$ tf/m^3	91.41	8.73	5.95	4.00
26	Stair	1.07	2.00	11.78	4.00
	Sub total	1640.80	8.85	4.83	4.00

No.	Calculation form	Weight (tf)	X (m)	Y (m)	Z (m)
	Control house $7.00 \times 7.00 \times (0.50+0.30)$ tf/m^2	39.20	6.50	16.30	4.00
	Machine of gate system	49.00	6.50	16.30	4.00
	Gate	90.00	6.50	4.05 (11.55)	4.00
	Maintenance bridge	344.00	15.00	10.70	4.00
	Total	2163.00	9.63	6.20 (6.51)	4.00

The number in parentheses shows condition of opening flood discharge gates.

Hydrostatic pressure

(Normal case)



In case of upstream flood discharge gate

$$P1 = 0.00 \times 1.00 \text{ tf/m}^3 = 0.00 \text{ tf/m}^2$$

$$P2 = 0.65 \times 1.00 \text{ tf/m}^3 = 0.65 \text{ tf/m}^2$$

$$P3 = 4.35 \times 1.00 \text{ tf/m}^3 = 4.35 \text{ tf/m}^2$$

$$P4 = 6.55 \times 1.00 \text{ tf/m}^3 = 6.55 \text{ tf/m}^2$$

In case of downstream flood discharge gate

$$P5 = 0.00 \times 1.00 \text{ tf/m}^3 = 0.00 \text{ tf/m}^2$$

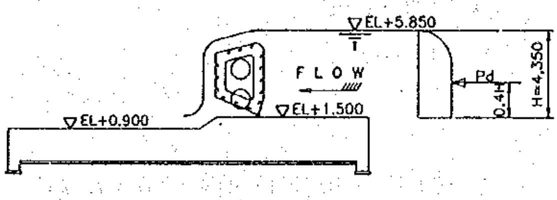
$$P6 = 2.25 \times 1.00 \text{ tf/m}^3 = 2.25 \text{ tf/m}^2$$

Position of load	Calculation form	W (tf)	Y (m)	Z (m)
Upstream of gate	$(0.65+4.35)\times \frac{1}{2} \times 3.70 \times 18.50$	171.13	3.59	4.00
Upstream of pier	$\frac{1}{2} \times 4.35^2 \times 2.50$	23.65	3.65	4.00
Upstream of slab	$(4.35+6.55)\times \frac{1}{2} \times 2.20 \times 8.00$	95.92	1.03	4.00
Downstream of slab	$-\frac{1}{2} \times 2.25^2 \times 8.00$	-20.25	0.75	4.00
Total		270.45	2.90	4.00

(Seismic case)

Refer to normal condition in hydrostatic pressure.

Hydrodynamic pressure due to earthquake



$$P_d = \frac{7}{12} W_0 \times Kh \times H^2 \times B$$

Where,
P_d : Hydrodynamic pressure (tf)
W_0 : Unit weight of water (tf/m^3)
Kh : Design seismic coefficient
H : Water depth (m)
B : Affected width of load (m)

(Direction of flowing water)

$W_0 = 1.00$ tf/m^3
Kh = 0.12
H = 4.35 m and
B = (18.50 + 2.50) = 21.00 m

$$P_d = \frac{7}{12} \times 1.00 \times 0.12 \times 4.35^2 \times 21.00 = 27.82 \text{ tf}$$

Y = 2.20+0.40×4.35 = 3.94 m

(Direction of weir axis)

$W_0 = 1.00$ tf/m^3
Kh = 0.12
H = 4.35 m and
B = 5.50 m

$$P_d = \frac{7}{12} \times 1.00 \times 0.12 \times 4.35^2 \times 5.50 = 7.29 \text{ tf}$$

Y = 2.20+0.40×4.35 = 3.94 m

Following water force

$$P = K \times V^2 \times A$$

Where,
K : coefficient of pier resistance
V : maximum flow velocity (m/s)
A : projective area of pier in vertical direction (m^2)

In design flooding case, there is supercritical flow around Simongan weir, because downstream position of Simongan weir has about 2.70 m drop between top of slab and downstream design riverbed.

Velocity of supercritical flow is faster than uniform flow. Therefore, velocity of supercritical flow is adopted.
The velocity is used in design discharge by non-uniformed flow calculation at WF.98.

$Q = 790\ m^3/s$

$H = 2.80\ m$

$V = 4.20\ m/s$

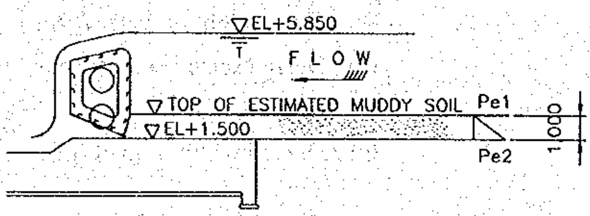
$K = 0.04$

Therefore

$P = 0.04 \times 4.20^2 \times (2.50 \times 4.35) = 7.67\ tf$

$Y = 0.60 \times 2.80 + 2.20 = 3.88\ m$

Muddy soil pressure



The pressure of muddy soil is estimated as follows

$Pe = Ce \times W1 \times D$

Where
Pe : muddy soil pressure (tf/m^2)
Ce : coefficient of muddy soil pressure ($Ce = 0.4 \sim 0.6$)
W1 : unit weight of muddy soil (tf/m^3)
D : depth from the surface (m)

$Ce = 0.5$

$W1 = 1.00\ tf/m^2$

$D = 1.00\ m$

$Pe_1 = 0.00\ tf/m^2$

$Pe_2 = 0.5 \times 1.00 \times 1.00 = 0.50\ tf/m^2$

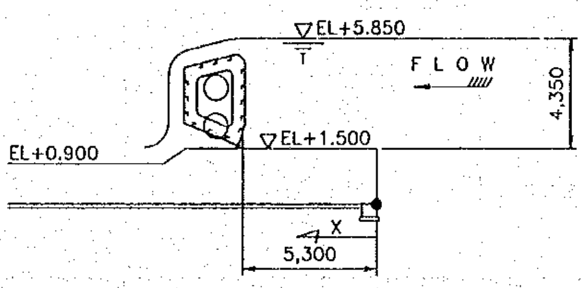
Therefore

$P_E = \frac{1}{2} \times 0.50 \times 1.00 \times (18.50 + 2.50) = 5.25\ tf$

$Y = \frac{1}{3} \times 1.00 + 2.20 = 2.53\ m$

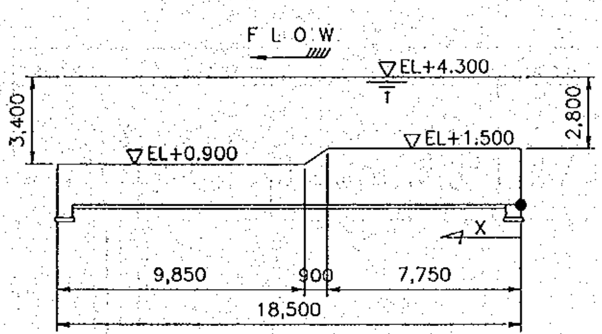
Weight of water

(Normal case)



Position of load	Calculation form	W (tf)	X (m)	Z (m)
Section of gate	5.30×4.35×(8.00-2.50)×1.00	126.80	2.65	4.00
Section of pier	(1.25+1.00)×2.50×4.35×1.00	24.47	1.13	4.00
	$-\pi/4 \times 2.50^2 \times 1/2 \times 4.35 \times 1.00$	-10.68	1.72	4.00
Total		140.59	2.46	4.00

(Design flooding case)

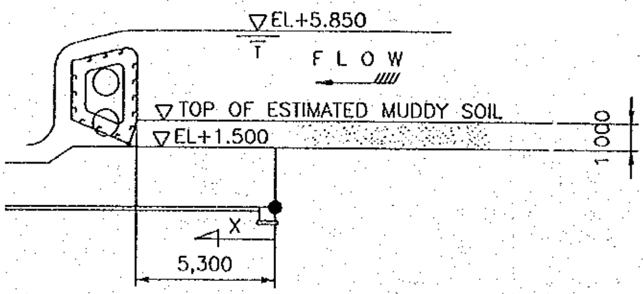


Position of load	Calculation form	W (tf)	X (m)	Z (m)
Upstream section	7.75×2.80×(8.00-2.50)×1.00	119.35	3.88	4.00
	(1.25+1.00)×2.50×2.80×1.00	15.75	1.13	4.00
	$-\pi/4 \times 2.50^2 \times 1/2 \times 2.80 \times 1.00$	-6.87	1.72	4.00
Slop section	$(2.80+3.40) \times 1/2 \times 0.90 \times (8.00-2.50) \times 1.00$	15.35	7.61	4.00
Downstream section	9.85×3.40×(8.00-2.50)×1.00	184.20	13.58	4.00
	(1.25+1.00)×2.50×3.40×1.00	19.13	17.37	4.00
	$-\pi/4 \times 2.50^2 \times 1/2 \times 3.40 \times 1.00$	-8.34	16.78	4.00
Total		338.57	9.69	4.00

(Seismic case)

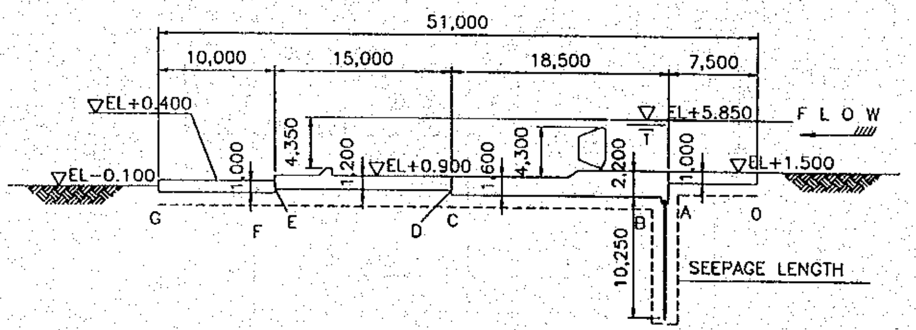
Refer to normal case.

Weight of muddy soil



Position of load	Calculation form	W (tf)	X (m)	Z (m)
Section of gate	5.30×1.00×(8.00-2.50)×1.00	29.15	2.65	4.00
Section of pier	(1.25+1.00)×2.50×1.00×1.00	5.63	1.13	4.00
	$-\pi/_4 \times 2.50^2 \times 1/_2 \times 1.00 \times 1.00$	-2.45	1.72	4.00
Total		32.33	2.46	4.00

Uplift



Vertical creep length

$$Lv := (2.20-1.00)+10.25\times2+(1.60-1.20)+((1.00+0.50)-1.20) = 22.40 \text{ m}$$

Horizontal creep length $Lh = 51.00$ m

Weighted creep length $Lw = 51.00/3 + 22.40 = 39.40$ m

Different water level $\Delta h = 4.35$ m

In between "0" point to "A" point weight creep length

$$Lwa = 7.50/3 + (2.20-1.00) = 3.70 \text{ m}$$

In between "0" point to "B" point weight creep length

$$Lwb = 7.50/3 + (2.20-1.00) + 10.25\times2 = 24.20 \text{ m}$$

In between "0" point to "C" point weight creep length

$$Lwc = 26.00/3 + (2.20 - 1.00) + 10.25 \times 2 = 30.37 \text{ m}$$

Uplift at "A" point $Ua = (4.35 + 2.20) - 3.70/39.40 \times 4.35 = 6.14 \text{ tf/m}^2$

Uplift at "B" point $Ub = (4.35 + 2.20) - 24.20/39.40 \times 4.35 = 3.88 \text{ tf/m}^2$

Uplift at "C" point $Uc = (4.35 + 2.20) - 30.37/39.40 \times 4.35 = 3.20 \text{ tf/m}^2$

Position of load	Calculation form	U (tf)	X (m)	Z (m)
Upstream section of seepage blocking	$6.14 \times 0.60 \times 8.00$	29.47	0.30	4.00
Downstream section of seepage blocking	$(3.88 + 3.20) \times 1/2 \times 17.90 \times 8.00$	506.93	9.54	4.00
Total		536.40	9.03	4.00

(Design flooding case)

$\Delta h = (EL + 4.30 \text{ m} - EL + 1.50 \text{ m}) + 2.20 = 5.00 \text{ m}$

$Ua = Ub = Uc = 5.00 \times 1.00 \text{ tf/m}^3 = 5.00 \text{ tf/m}^2$

Therefore

$U = 5.00 \times 18.50 \times 8.00 = 740.00 \text{ tf}$

$X = 1/2 \times 18.50 = 9.25 \text{ m}$

$Z = 4.00 \text{ m}$

(Seismic case)

Refer to Normal condition in uplift.

<u>Wind pressure</u>

(Normal case: Direction of Flowing water)

Direction of wind: from upstream to downstream

Position of load	Calculation form	U (tf)	Y (m)
Control house section	$7.00 \times 4.85 \times 0.15 \text{ tf/m}^2$	5.09	18.03
Haunch of control rack section	$(7.00 + 2.50) \times 1/2 \times 0.50 \times 0.15 \text{ tf/m}^2$	0.36	15.39
Gate post section	$3.40 \times 2.50 \times 0.15 \text{ tf/m}^2$	1.28	13.40
Maintenance bridge section	$2.00 \times 21.00 \times 0.15 \text{ tf/m}^2$	6.30	10.70
Gate pier section	$(EL + 9.00 - EL + 5.85) \times 2.50 \times 0.15 \text{ tf/m}^2$	1.18	8.13
Total		14.21	13.47

(Normal case: Direction of weir axis)

Position of load	Calculation form	U (tf)	Y (m)
Control house section	$7.00 \times 4.85 \times 0.15$ tf/m^2	5.09	18.03
Haunch of control rack section	$(7.00+2.50) \times \frac{1}{2} \times 0.50 \times 0.15$ tf/m^2	0.36	15.39
Gate post section	$5.40 \times 2.50 \times 0.15$ tf/m^2	2.03	12.40
Gate pier section	$1.00 \times 17.50 \times 0.15$ tf/m^2	2.63	9.20
	(EL + 8.00-EL +5.85)$\times 16.50 \times 0.15$ tf/m^2	5.32	7.63
Total		15.43	12.14

(Design flooding case: Direction of flowing water)
Direction of wind: from upstream to downstream

Position of load	Calculation form	U (tf)	Y (m)
Control house section	$7.00 \times 4.85 \times 0.15$ tf/m^2	5.09	18.03
Haunch of control rack section	$(7.00+2.50) \times \frac{1}{2} \times 0.50 \times 0.15$ tf/m^2	0.36	15.39
Gate post section	$5.40 \times 2.50 \times 0.15$ tf/m^2	2.03	12.40
Maintenance bridge section	$2.00 \times 21.00 \times 0.15$ tf/m^2	6.30	10.70
Gate pier section	(EL + 9.00-EL +8.00)$\times 2.50 \times 0.15$ tf/m^2	0.38	9.20
Gate section	$18.50 \times 3.70 \times 0.15$	10.27	11.55
Total		24.43	12.77

(Design flooding case: Direction of weir axis)

Position of load	Calculation form	U (tf)	Y (m)
Control house section	$7.00 \times 4.85 \times 0.15$ tf/m^2	5.09	18.03
Haunch of control rack section	$(7.00+2.50) \times \frac{1}{2} \times 0.50 \times 0.15$ tf/m^2	0.36	15.39
Gate post section	$5.40 \times 2.50 \times 0.15$ tf/m^2	2.03	12.40
Gate pier section	$1.00 \times 17.50 \times 0.15$ tf/m^2	2.63	9.20
Total		10.11	14.51

(Constructional case: Direction of flowing water)

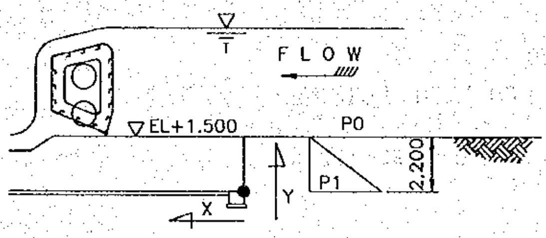
Direction of wind: from upstream to downstream

Position of load	Calculation form	U (tf)	Y (m)
Control house section	7.00×4.85×0.15 tf/m²	5.09	18.03
Haunch of control rack section	$(7.00+2.50)\times \frac{1}{2}\times 0.50\times 0.15$ tf/m²	0.36	15.39
Gate post section	5.40×2.50×0.15 tf/m²	2.03	12.40
Maintenance bridge section	2.00×21.00×0.15 tf/m²	6.30	10.70
Gate pier section	(EL+9.00-EL+1.50)×2.50×0.15 tf/m²	2.81	5.95
Gate section	18.50×3.70×0.15 tf/m²	10.27	11.55
Total		26.86	12.11

(Construction case: Direction of weir axis)

Position of load	Calculation form	U (tf)	Y (m)
Control house section	7.00×4.85×0.15 tf/m²	5.09	18.03
Haunch of control rack section	$(7.00+2.50)\times \frac{1}{2}\times 0.50\times 0.15$ tf/m²	0.36	15.39
Gate post section	5.40×2.50×0.15 tf/m²	2.03	12.40
Gate pier section	1.00×17.50×0.15 tf/m²	2.63	9.20
	6.50×16.50×0.15 tf/m²	16.09	5.45
	$(9.75+8.85)\times \frac{1}{2}\times 0.60\times 0.15$ tf/m²	0.84	1.90
Total		27.04	8.73

Earth pressure



Earth pressure is adopted in case of direction of flowing water.

(Normal case)

The earth pressure acting is calculated by the following Coulom's formulas.

Unit weight of soil $\gamma' = 1.00$ tf/m³ (under water)

Coefficient of active earth pressure $Ka = 0.308$

Depth $\Delta h = 2.20$ m

Therefore

$P0 = 0.00\ tf/m^2$

$P1 = 0.308 \times 1.00 \times 2.20 = 0.678\ tf/m^2$

$P = \frac{1}{2} \times 0.678 \times 2.20 \times 8.00 = 5.97\ tf$

$Y = \frac{1}{3} \times 2.20 = 0.73\ m$

$Z = 4.00\ m$

(Design flooding case)

Refer to Normal case in earth pressure.

(Construction case)

Unit weight of soil	$\gamma t = 1.80\ tf/m^3$
Coefficient of active earth pressure	$Ka = 0.308$
Depth	$\Delta h = 2.20\ m$

Therefore

$P0 = 0.00\ tf/m^2$

$P1 = 0.308 \times 1.80 \times 2.20 = 1.220\ tf/m^2$

$P = \frac{1}{2} \times 1.220 \times 2.20 \times 8.00 = 10.74\ tf$

$Y = \frac{1}{3} \times 2.20 = 0.73\ m$

$Z = 4.00\ m$

(Seismic case)

Unit weight of soil	$\gamma' = 1.00\ tf/m^3$ (underwater)
Coefficient of active earth pressure	$Kae = 0.492$ (underwater)
Depth	$\Delta h = 2.20\ m$

Therefore

$Pe0 = 0.00\ tf/m^2$

$Pe1 = 0.492 \times 1.00 \times 2.20 = 1.082\ tf/m^2$

$Pe = \frac{1}{2} \times 1.082 \times 2.20 \times 8.00 = 9.52\ tf$

$Y = \frac{1}{3} \times 2.20 = 0.73\ m$

$Z = 4.00\ m$

Load Combination

Load combinations for stability analysis are made as follows.

Load \ Condition		Normal case	Design flooding case	Construction case	Seismic case
The Vertical load	Weight of body	○	○	○	○
	Weight of gate	○	○	○	○
	Weight of maintenance bridge	○	○	○	○
	Weight of control house	○	○	○	○
	Weight of machine	○	○	○	○
	Weight of earth	○	○	○	○
	Weight of water	○	○		○
	Weight of muddy soil	○			○
	Uplift	○	○		○
The horizontal load	Hydrostatic pressure	○	○	○	○
	Hydrodynamic pressure due to earthquake				○
	Flowing water force		○		
	Earth pressure	○	○	○	○
	Muddy soil pressure	○			○
	Wind pressure	○	○	○	
	Horizontal earthquake load				○

Symbol " ○ " shows that it puts in the calculation.

Calculation loads are categorized for each calculation cases as follows.

[Normal case: Direction of flowing water]

	Vertical			Horizontal		
	V(tf)	X(m)	Mx(tf-m)	H(tf)	Y(m)	My(tf-m)
Weight of body	1640.80	8.85	14524.36			
Weight of muddy soil	32.33	2.46	79.53			
Weight of water	140.59	2.46	345.85			
Weight of earth						
Weight of maintenance bridge	388.80	15.00	5832.00			
Weight of control house	39.20	6.50	254.80			
Weight of machine	25.00	6.50	162.50			
Weight of gate	50.00	6.50	325.00			
Hydrostatic pressure				270.45	2.90	784.31
Flowing water force				0.27	4.81	1.28
Earth pressure				5.97	0.73	4.38
Muddy soil pressure				5.25	2.53	13.28
Wind pressure				14.21	13.47	191.41
Uplift	-536.40	9.03	-4843.69			
Total	1780.32		16680.35	296.15		994.66

Vn = 1,780.32 tf

Hn = 296.15 tf

Mn = Mx + My = 17,625.01 tf m

[Normal case: Direction of weir axis]

	Vertical			Horizontal		
	V(tf)	Z(m)	Mz(tf-m)	H(tf)	Y(m)	My(tf-m)
Weight of body	1640.80	4.00	6563.20			
Weight of muddy soil	32.33	4.00	129.32			
Weight of water	140.59	4.00	562.36			
Weight of earth						
Weight of maintenance bridge	388.80	4.00	1555.20			
Weight of control house	39.20	4.00	156.80			
Weight of machine	25.00	4.00	100.00			
Weight of gate	50.00	4.00	200.00			
Hydrostatic pressure						
Flowing water force						
Earth pressure						
Muddy soil pressure						
Wind pressure				15.43	12.14	187.32
Uplift	-536.40	4.00	-2145.60			
Total	1780.32		7121.28	15.43		187.32

Vn = 1,780.32 tf

Hn = 15.43 tf

Mn = Mz + My = 7,308.60 tf m

[Design flooding case: Direction of flowing water]

	Vertical			Horizontal		
	V(tf)	X(m)	Mx(tf-m)	H(tf)	Y(m)	My(tf-m)
Weight of body	1640.80	8.85	14524.36			
Weight of muddy soil						
Weight of water	338.57	9.69	3280.74			
Weight of earth						
Weight of maintenance bridge	388.80	15.00	5832.00			
Weight of control house	39.20	6.50	254.80			
Weight of machine	25.00	6.50	162.50			
Weight of gate	50.00	6.50	325.00			
Hydrostatic pressure						
Flowing water force				7.67	3.88	29.76
Earth pressure				5.97	0.73	4.38
Muddy soil pressure						
Wind pressure				24.43	12.77	311.97
Uplift	-740.00	9.25	-6845.00			
Total	1742.37		17534.40	38.07		346.11

Vf = 1,742.37 tf

Hf = 38.07 tf

Mf = Mx + My = 17,880.51 tf m

[Design flooding case: Direction of weir axis]

	Vertical			Horizontal		
	V(tf)	Z(m)	Mz(tf-m)	H(tf)	Y(m)	My(tf-m)
Weight of body	1640.80	4.00	6563.20			
Weight of muddy soil						
Weight of water	338.57	4.00	1354.28			
Weight of earth						
Weight of maintenance bridge	388.80	4.00	1555.20			
Weight of control house	39.20	4.00	156.80			
Weight of machine	25.00	4.00	100.00			
Weight of gate	50.00	4.00	200.00			
Hydrostatic pressure						
Flowing water force						
Earth pressure						
Muddy soil pressure						
Wind pressure				10.11	14.51	146.70
Uplift	-740.00	4.00	-2960.00			
Total	1742.37		6969.48	10.11		146.70

Vf = 1,742.37 tf

Hf = 10.11 tf

Mf = Mz + My = 7,116.18 tf m

[Construction case: Direction of flowing water]

	Vertical			Horizontal		
	V(tf)	X(m)	Mx(tf-m)	H(tf)	Y(m)	My(tf-m)
Weight of body	1640.80	8.85	14524.36			
Weight of muddy soil						
Weight of water						
Weight of earth						
Weight of maintenance bridge	260.80	15.00	3912.00			
Weight of control house	39.20	6.50	254.80			
Weight of machine	25.00	6.50	162.50			
Weight of gate	50.00	6.50	325.00			
Hydrostatic pressure						
Flowing water force						
Earth pressure				10.74	0.73	7.87
Muddy soil pressure						
Wind pressure				26.86	12.11	325.27
Uplift						
Total	2015.80		19178.66	37.60		333.15

Vc= 2,015.80 tf

Hc = 37.60 tf

Mc = Mx + My = 19,511.81 tf m

[Construction case: Direction of weir axis]

	Vertical			Horizontal		
	V(tf)	Z(m)	Mz(tf-m)	H(tf)	Y(m)	My(tf-m)
Weight of body	1640.80	4.00	6563.20			
Weight of muddy soil						
Weight of water						
Weight of earth						
Weight of maintenance bridge	260.80	4.00	1043.20			
Weight of control house	39.20	4.00	156.80			
Weight of machine	25.00	4.00	100.00			
Weight of gate	50.00	4.00	200.00			
Hydrostatic pressure						
Flowing water force						
Earth pressure						
Muddy soil pressure						
Wind pressure				27.04	8.73	236.06
Uplift						
Total	2015.80		8063.20	27.04		236.06

Vc= 2,015.80 tf

Hc = 27.04 tf

Mc = Mz + My = 8,299.26 tf m

[Seismic case: Direction of flowing water]

	Vertical			Horizontal		
	V(tf)	X(m)	Mx(tf-m)	H(tf)	Y(m)	My(tf-m)
Weight of body	1640.80	8.85	14524.36	196.90	4.83	951.01
Weight of muddy soil	32.33	2.46	79.53	3.88	2.70	10.47
Weight of water	140.59	2.46	345.85			
Weight of earth						
Weight of maintenance bridge	260.80	15.00	3912.00	31.30	10.51	328.92
Weight of control house	24.50	6.50	159.25	2.94	18.38	54.02
Weight of machine	25.00	6.50	162.50	3.00	16.45	49.35
Weight of gate	50.00	6.50	325.00	6.00	4.05	24.30
Hydrostatic pressure				270.45	2.90	784.31
Hydrodynamic pressure				27.82	3.94	109.61
Flowing water force				0.27	4.81	1.28
Earth pressure				9.86	0.73	7.23
Muddy soil pressure				5.25	2.53	13.28
Wind pressure						
Uplift	-536.40	9.03	-4843.69			
Total	1637.62		14664.80	557.66		2333.79

Vs= 1,637.62 tf

Hs = 557.66 tf

Ms = Mx + My = 16,998.59 tf m

[Seismic case: Direction of weir axis]

	Vertical			Horizontal		
	V(tf)	Z(m)	Mz(tf-m)	H(tf)	Y(m)	My(tf-m)
Weight of body	1640.80	4.00	6563.20	196.90	4.83	951.01
Weight of muddy soil	32.33	4.00	129.32	3.88	2.70	10.47
Weight of water	140.59	4.00	562.36			
Weight of earth						
Weight of maintenance bridge	260.80	4.00	1043.20	31.30	10.51	328.92
Weight of control house	24.50	4.00	98.00	2.94	18.38	54.02
Weight of machine	25.00	4.00	100.00	3.00	16.45	49.35
Weight of gate	50.00	4.00	200.00	6.00	4.05	24.30
Hydrostatic pressure						
Hydrodynamic pressure				7.29	3.94	28.72
Flowing water force						
Earth pressure						
Muddy soil pressure						
Wind pressure						
Uplift	-536.40	4.00	-2145.60			
Total	1637.62		6550.48	251.30		1446.80

Vs= 1,637.62 tf

Hs = 251.30 tf

Ms = Mz + My = 7,997.28 tf m