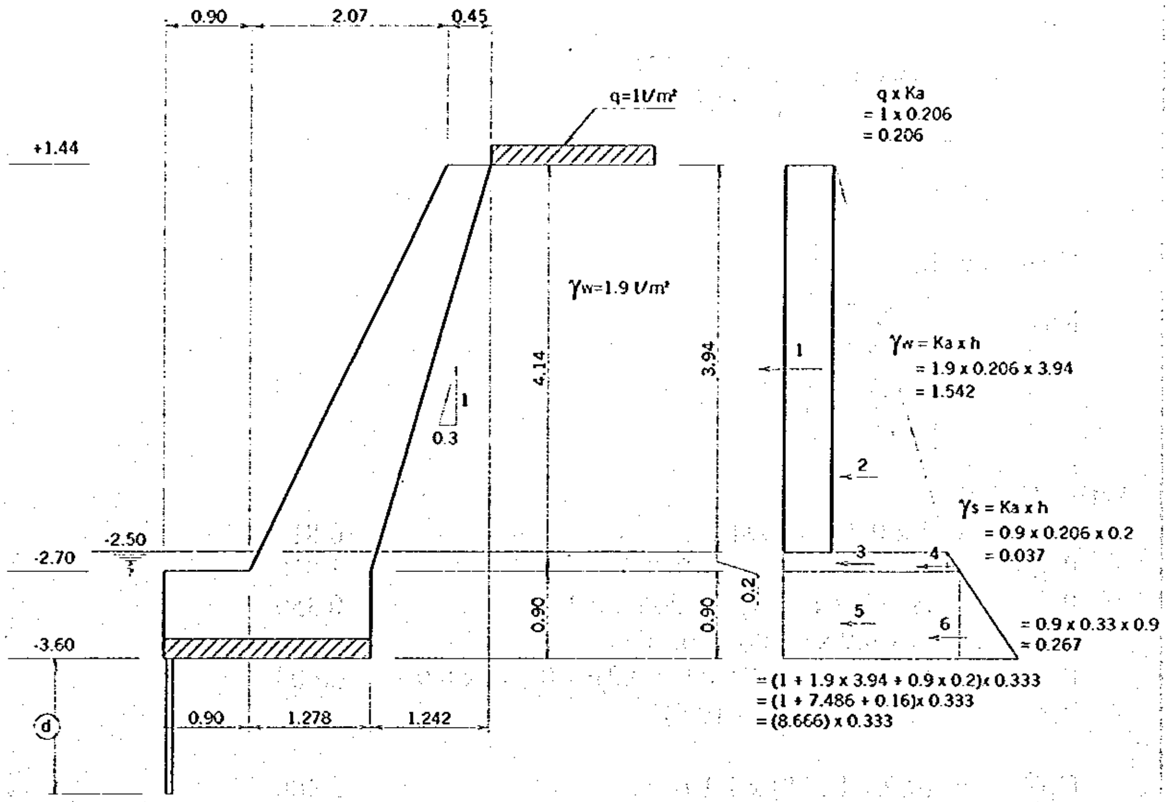
2.1 Asin Pumping Station

2.1.6 Structural Calculation of Asin Revetment

Name of Structure	RETAINING WALL OF ASIN RIVER	Category Calculation	**Stability**	Page	1/4



tg Θ = 0.3

Θ = -16.699°   α = 0

Backfill material

φ = 30°

γ = 1.9 t/m³

$$\delta = \frac{30}{3} = 10°$$

C = 0

$$Ka = \frac{Cos^2(\varphi - \theta)}{Cos^2\theta \cdot Cos(\varphi + \theta) \cdot \left[1 + \sqrt{\frac{Sin(\varphi + \theta) \cdot Sin(\varphi - \alpha)}{Cos(\theta + \delta) \cdot Cos(\theta - \alpha)}}\right]^2}$$

Cos²(φ - Θ)	= Cos²(46.699)	=	0.470
Cos²Θ	= Cos²(-16.699)	=	0.918
Cos(Θ + δ)	= Cos(-16.699 + 10)	=	0.993
Sin(φ + δ)	= Sin(30 + 10)	=	0.642
Sin(φ - α)	= Sin(30 - 0)	=	0.5
Cos(Θ + δ)	= Cos(-16.699 + 10)	=	0.993
Cos(Θ - α)	= Cos(-16.699 - 0)	=	0.957

$$Ka_1 = \frac{0.47}{0.918(0.933)\cdot\left[1+\sqrt{\frac{0.642(0.5)}{0.993(0.957)}}\right]^2}$$

$$= \frac{0.47}{2.279} = 0.206$$

Vertical wall, $\delta = 0$

$$Ka_2 = \frac{1-Sin\varphi}{1+Sin\varphi}$$

$$= \frac{1-0.5}{1+0.5} = 0.333$$

Active Force

P_{a1} = 1 x 0.206 x 3.94 = 0.811

P_{a2} = ½ x 1.9 x (3.94)² x 0.206 = 3.038

P_{a3} = (1 + 1.9 x 3.94) x 0.206 x 0.2 = 0.350

P_{a4} = ½ x 0.9 x 0.206 x (0.2)² = 0,008

P_{a5} = (1 + 1.9 x 3.94 + 0.9 x 0.2)² x 0.333 x 0.9 = 2,600

P_{a6} = ½ x 0.9 x 0.33 x (0.9)² = 0,120

Up↑ = (0.9 + 1.278) x 1.10 = 2.400

Horizontal Force

P_{aH} = Pa Cos (θ + δ)
= Pa Cos (-16.699 + 10)
= Pa (0.993)

Vertical Force

P_{aV} = Pa Sin (θ + δ)
= Pa Sin (-16.699 + 10)
= Pa (0.117)

Active Horizontal Force

P_{aH1} = 0.811 x (0.993) = 0.805 ←

P_{aH2} = 3.038 x (0.993) = 3.017 ←

P_{aH3} = 0.350 x (0.993) = 0.348 ←

P_{aH4} = 0.008 x (0.993) = 0.008 ←

P_{aH5} = 2.60 x (0.993) = 2.582 ←

P_{aH6} = 0.12 x (0.993) = 0.119 ←

Total = 6.879

Passive Horizontal Force

φ soil assume 20°

$$Kp = \frac{1+\sin\varphi}{1-\sin\varphi} = \frac{1+0.342}{1-0.342} = \frac{1.342}{0.658} = 2.039$$

P_{p1} = ½ x 0.6 x 2.039 x (0.9)² = 0.495 →

Passive Vertical Force

P_{aV1} = 0.811 x 0.117 = 0.095 ↑

P_{aV2} = 3.017 x 0.117 = 0.353 ↑

P_{aV3} = 0.868 x 0.117 = 0.101 ↑

P_{aV4} = 0.023 x 0.117 = 0.003 ↑

$W_1 = \frac{0.45+1.278}{2} \times 4.14 \times 2.3 = 9.2 \downarrow$

W_2 = 2.178 x 0.9 x 2.3 = 4.50 ↓

Center of Wall

Force	Distance	D	Moment
1. 2.52 x 4.14	= 10.433	$\frac{2.52}{2} = 1.26$	13.146
2. 1.242 x 4.14 x ½	= 2.57	$1.278 + \frac{1.242}{3} \times 2 = 2.106$	-5.412
3. 2.07 x 4.14 x ½	= 4.285	$\frac{2.07}{3} = 0.69$	-2.956
4. (0.45 + 1.278)x ½ x 4.14	= 3.577	1.335	-4.778

Active Moment at A

Forces		Distance	Moment
P_{aH1}	0.805	$1.10 + \frac{3.94}{2} = 3.07$	2.47
P_{aH2}	3.017	$1.10 + \frac{3.94}{3} = 2.413$	7.28
P_{aH3}	0.348	$0.9 + \frac{0.2}{2} = 1$	0.348
P_{aH4}	0.008	$0.9 + \frac{0.2}{3} = 0.966$	0.008
P_{aH5}	2.582	$\frac{0.9}{2} = 0.45$	1.162
P_{aH6}	0.119	$\frac{0.9}{3} = 0.30$	0.036
P_{aV1}	0.095	$3.42 - \frac{3.94 \times 0.3}{2} = 2.829$	0.269
P_{aV2}	0.353	$3.42 - 3.94 \times 0.3 \times 2/3 = 2.632$	0.929
P_{aV3}	0.101	$(0.9 + 1.278) \times 0.2/2 = 2.278$	0.230
P_{aV4}	0.003	$(0.9 + 1.278) \times 0.2/3 = 2.245$	0.007
UP ↑	2.40	$\frac{(0.9 + 1.278)}{2} = 1.089$	2.614
Total			15.353

Passive Moment at A

Forces		Distance	Moment
P_{P1}	0.495	$\frac{0.9}{3} = 0.3$	0.148
w_1	8.22	$0.9 + 1.335 = 2.235$	18.371
w_2	4.50	$(0.9 + 1.278) \times \frac{1}{2} = 1.089$	4.900
Total			23.419

Control of Bearing

$$S_F = \frac{23.419}{15.353} = 1.525$$

LOG PILE

Allowable bearing capacity for pile.

Vertical 4 t
Horizontal 1.0 t

Number of Log

Total horizontal force = + active horizontal force
- passive horizontal force

$$= 6.879 - 0.495$$
$$= 6.384$$

Number of Log $= \frac{6.879 - 0.495}{1.0} \approx 6.5 Nos$

$\approx 7 Nos$

Total vertical force $= w_1 + w_2$

$= 8.22 + 4.50 =$ **12.72 ton**

Number of log $= \frac{12.72}{4} = 4 Nos$

As the final design, the number of logs per meter stretch is eight (0.5 m interval)

2.1 Asin Pumping Station

2.1.7 Structural Calculation of Steel Sheet Pile

7.2.1 Design of Sheet Pile Coffering

7.2.2 Acting Force and Load Calculation

(0) Design Condition

Design River Bed Elevation EL -2.435m

Design Water Level (normal and earthquake) EL +0.375m

Design Crest Elevation of Coffering EL +1.875m

Design Elevation of Tie-Rod EL +0.875m

(1) Hydrostatic Pressure

$P = \gamma_w \times h^2/2$ where P; hydrostatic pressure (tf/m)

h; height of water outside(m)

γ_w; unit weight of water (tf/m^3)

as h = 0.375-(-2.435)=2.81 m

γ_w= 1.0 tf/m^3

P =1×2.81^2/2

= 3.95 tf/m

(2) Residual Water Pressure in the Wall

$P' = \gamma_w/2 \times (2/3 \times h)^2$ P' : water pressure in the wall (tf/m)

h : height of water outside (m)

γ_w : unit weight of water (tf/m^3)

as h = 2.81m

γ_w = 1.0 tf/m^3

P =1/2×(2/3×2.81)^2

=1/2×1.873^2

=1.754 tf/m

acting point =EL-2.435+1.873/3

=EL-2.435+0.624

=EL-1.811

(3) Weight of Soil in the Wall

$W = B(\gamma H_1 + \gamma_b H_2)$

W : weight of soil in the wall(tf/m)

B : width of wall(m)

H_1 : thickness of upper layer(above water surface) (m)

H_2 : thickness of lower layer(below water surface) (m)

γ : unit weight of soil (tf/m^3)

γ_b : saturated unit weight of soil (tf/m^3)

as γ = 1.9(sandy soil, wet, compacted)

γ_b = 1.9(sandy soil, saturated, compacted)

H_1 = (1.875-0.375)+1/3×(0.375-(-2.435))=1.5+0.937=2.437m

H_2 = 2/3×(0.375-(-2.435))=1.873m

W = B×(1.9×2.437+1.9×1.873)=8.189B tf/m

(4) Earth Pressure

$p_a = K_a(\Sigma \gamma_i h_i)$ p_a: active earth pressure

$p_p = K_p(\Sigma \gamma_i h_i)$ p_p: passive earth pressure

K_a: coefficient of active earth pressure

K_p: coefficient of passive earth pressure

γ_i: unit weight of soil in the ith layer

h_i: thickness of ith soil layer

For layer 1; sand (γ_1=1.9tf/m^3, ϕ=30°)

K_a=0.309, K_p=2.24
K_{ea}=0.404, K_{ep}=2.80

For layer2; sandy clay (γ_2=1.9tf/m^3, ϕ=26.5°)
K_a=0.368, K_p=2.00
K_{ea}=0.474, K_{ep}=2.34

<u>active earth pressure of clay in the wall;</u>
(EL-0.562m)
d_1 = 1.875-(-0.562)=2.437m
p_{a1} = 1.9×2.437×0.309=1.431 tf/m/m
p_{ea1} = 1.9×2.437×0.404=1.871 tf/m/m
P_{a1} = 1.431×2.437/2=1.744tf/m
P_{ea1} = 1.871×2.437/2=2.280tf/m
acting point = 2.437/3-0.562=EL+0.250m
(EL-1.000m)
d_2 = -0.562-(-1.000)=0.438m
p_{a2} = (1.9×2.437×0.309+0.9×0.438×0.309)=1.431+0.122=1.553 tf/m/m
p_{ea2} = (1.9×2.437×0.404+0.9×0.438×0.404)=1.871+0.159=2.030 tf/m/m
P_{a2} = $(p_{a1}+p_{a2})\times d_2/2$=(1.431 +1.553)×0.438/2=0.653tf/m
P_{ea2} = $(p_{ea1}+p_{ea2})\times d_2/2$=(1.871 +2.030)×0.438/2=0.854tf/m
acting point = EL-1.000m+ $(d_2/2\times p_{a1}+d_2/3\times(p_{a2}-p_{a1}))/P_{a2}$
= EL-1+(0.438/2*1.431+0.438/3*(1.553-1.431))/0.653
= EL-1+0.507
= EL-0.493
(EL-2.435m)
d_3 = -2.435-(-1.000)=1.435m
p_{a3} = (1.9×2.437×0.309+0.9×0.438×0.309+0.9×1.435×0.368)
= 1.431+0.122+0.475
= 2.028 tf/m/m
p_{ea3} = (1.9×2.437×0.404+0.9×0.438×0.404+0.9×1.435×0.474)
= 1.871+0.159+0.612
= 2.642 tf/m/m
P_{a3} = $(p_{a2}+p_{a3})\times d_3/2$=(1.553 +2.028)×1.435/2=2.569tf/m
P_{ea3} = $(p_{ea2}+p_{ea3})\times d_3/2$=(2.030 +2.642)×1.435/2=3.352tf/m
acting point = EL-2.435+ $(d_3/2\times p_{a2}+d_3/3\times(p_{a3}-p_{a2}))/P_{a3}$
= EL-2.435+(1.435/2*1.553+1.435/3*(2.028-1.553))/2.569
= EL-2.435+0.522
= EL-1.913

<u>active earth pressure of clay upstream;</u>
(EL-2.435m)
d_1 = -1-(-2.435)=1.435m
p_{a1} = 0.9×1.435×0.368=0.475tf/m/m
P_{a1} = 1.435×0.475/2=0.341tf/m
acting point = EL-2.435+1.435/3
= EL-2.435+0.478
= EL-1.957

7.2.3 Study of Stability of Wall

(1) Stability against Shearing Force

$FM_d <= M_r$ F: safety factor (1.2; normal and seismic conditions)
M_d: shearing moment (tfm)
M_r: resistance moment (tfm)

(2) Shearing Moment
Shearing moment is calculated as follows;
a. shearing moment by hydrostatic pressure
(normal condition and earthquake condition)

$M_d=\gamma_w \times h^3/6$
$=1.0\times 2.81^3/6$
$=3.698$ tf.m

b. shearing moment by inertia of soil in the wall (earthquake condition)

$M_d=Kh(((BH_1\gamma)(H_2+H_1/2))+(B\gamma_b H_2^2/2))$

γ_w: unit weight of water (tf/m^3) (=1.0)
γ : unit weight of soil (tf/m^3) (=1.9)
γ_b: unit weight of saturated soil (tf/m^3) (=1.9)
K_h: design seismic coefficient (=0.11)

$M_d = 0.11\times(((B\times 2.437\times 1.9)\times(1.873+2.437/2))+(B\times 1.9\times 1.873^2/2))$
$= 0.11*(4.630B*3.092+3.333B))$
$= 1.9414B$

Therefore,
(normal condition)
$M_d=3.698$ tf.m
(earthquake condition)
$M_d=3.698 +1.9414B$

(3) Resistance Moment
Resistance moment is calculated as follows;

$M_r=1/6\times\gamma_m(RH^3)$

B: width of wall (m)
$\nu = B/H$;(B/4.31)=0.2320B
ϕ: internal friction angle of soil in the wall (°)(=25.6)
γ_m: converted unit weight of soil in the wall (tf/m^3) (=1.9)
γ_m': submerged unit weight of soil in the wall (tf/m^3) (=0.9)

<u>Normal condition:</u>

$R = 2/3^2(3-\nu\cos\phi)\tan\phi\sin\phi$
$= 2/3*(0.2320B)^2(3-0.2320B*\cos25.6)\tan25.6*\sin25.6$
$= 0.1380*(3-0.2092B)*(0.2320B)^2$
$= (0.4140-0.02887B)*0.05382B^2$
$= 0.02228B^2-0.001554B^3$

$RH^3 = (0.02228B^2-0.001554B^3)*4.31^3$
$= 1.784B^2-0.1244B^3$

$\gamma_m = (\gamma H_1+\gamma' H_2)/H$
$= (1.9*2.437+1.9*1.873)/4.31$
$= 1.9$

therefore,

$M_r = 1/6\times 1.9\times(1.784B^2-0.1244B^3)$
$= 0.5649B^2-0.03939B^3$

<u>Seismic condition:</u>

$R = \nu^2(3-\nu\cos\phi)\sin\phi$

$= 0.05382B^2*(3-0.2320B*cos25.6)*sin25.6$
$= 0.02325B^2*(3-0.2092B)$
$= 0.06975B^2-0.004864B^3$
$RH^3 = (0.06975B^2-0.004864B^3)*4.31^3$
$= 5.5844B^2-0.3894B^3$
$\gamma_m' = (\gamma H_1+\gamma' H_2')/H$
$= 1.9$
$M_r = 1/6\times\gamma_m(RH^3)$
$= 1/6*1.9*(5.5844B^2-0.3894B^3)$
$= 1.7684B^2-0.1233B^3$

(2) Design of Wall Width
M_r-Fs* M_d>=0

Normal condition
$0.5649B^2-0.03939B^3-1.20*3.698>=0$
$0.03939B^3-0.5649B^2+4.4376<=0$
B>=3.18m

Earthquake condition
$1.7684B^2-0.1233B^3-1.2*(3.698+1.9414B)>=0$
$0.1233B^3-1.7684B^2+2.3297B+4.4376<=0$
B>=2.12 m
Therefore, the width of the wall is designed as 3.2m

7.2.3.3.Stability against sliding
(1) Basic formula
Basic formula for stability against sliding is as follows;
$F=B(W\mu+c)/\Sigma H$
F: safety factor (1.2 for normal condition, 1.0 for seismic condition)
W: unit weight of soil in the wall (tf/m^3)
Normal condition $W=\gamma_m H$
Seismic condition $W'=\gamma_m' H$
μ: $\tan\phi$
ϕ: angle of internal friction of existing ground (°)
c: cohesion of existing ground (tf/m^2)
ΣH: horizontal force (tf/m)

(2) Horizontal force acting on the wall

Normal condition
Hydrostatic pressure: 3.95 tf/m

Earthquake condition
Hydrostatic pressure: 3.95 tf/m
Inertia acting on the wall:
$F_d=0.11\times((B\times2.437\times1.9)+(B\times1.9\times1.873))$
$=0.11*(4.630B+3.559B)$
$=0.9008B$
$=0.9008*3.2$
$=2.88$ tf/m
Total Force=3.95+2.88=6.83 tf/m

(3) Resistance force of soil in the wall

Normal condition, Earthquake condition

Rh=((2.437×1.9)+(1.9×1.873))×3.2×tan25.6=8.189*3.2*.479=12.55 tf/m

(4) Safety against sliding

Normal condition

Fs=Rh/H=12.55/3.95=3.18 >1.20

Earthquake condition

Fs=Rh/H=12.55/6.83=1.84 >1.0

7.2.4 Bearing Capacity of Foundation

(1) Basic formula

F =Qu/W

Qu =A'(KCNc+Kγ_2DfNq+½γ_1B'N_γ)

F : safety factor (normal condition 1.2, seismic condition 1.0)

Qu : ultimate bearing capacity of ground (tf/m)

W : weight of soil in the wall (tf/m)

A : effective loading area (m^2)

B' : effective loading width taking into account eccentricity

B'=B-2e

B : width of the wall

e : eccentricity of load (e=Ms/W)

M_B : moment acting on the ground

K : incremental coefficient determined by B and D

K=1+0.3×(B/D)

C : cohesion

Df : depth up to the point of calculation (m)

γ_1 : unit weight of soil above the depth Df

γ_2 : unit weight of soil below the depth Df

Nc, Nq, Nr: bearing capacity coefficient of soil

Tanθ = ΣH/W

ΣH : horizontal force

(3) Weight of soil in the wall

Normal condition and earthquake condition

W=((2.437×1.9)+(1.9×1.873))×3.2=8.189*3.2=26.20 tf/m

(4) Ultimate bearing capacity

driving moment is

normal condition ;

M_d = 3.698 tf/m

earthquake condition

M_d = 3.698 +1.9414B

= 3.698+1.9414*3.2

= 9.9105 tf/m

distance of eccentricity is

normal condition ;

e = M_d /W = 3.698/26.20=0.1411 m

earthquake condition

e' = M_d/W = 9.911/26.20=0.3783 m

effective loading area per unit length is

normal condition ;

A=L×B'=L×(B-2e)=1.0*(3.2-2*0.1411)=2.918 m^2

earthquake condition

A'=L×B'=L×(B-2e')=1.0*(3.2-2*0.3783)=2.443 m^2

On the other hand, the coefficient of bearing capacity is calculated as follows;

Normal condition

Horizontal total force; H=3.95 tf/m
Weight of soil in the wall; W=26.20tf/m
Tanθ=3.95/26.20=0.151
ϕ=25.6°
C=1.0 tf/m^2
Nr=4
Nc=15

Earthquake condition

Horizontal total force; H=6.83 tf/m
Weight of soil in the wall; W=26.20tf/m
Tanθ=6.83/26.20=0.261
ϕ=25.6°
C=1.0 tf/m^2
Nr=2
Nc=12

Therefore, the ultimate bearing capacity is

Normal condition

Qu = A(1.0×C×Nc+1/2×γ_1×B'×Nr)
= 2.815*(1*1*15+1/2*1.0*2.815*4)
= 58.07 tf/m

Earthquake condition

Qu' = A'(1.0×C×Nc +1/2×γ_1×B'×Nr)
= 2.416*(1*1*12+1/2*1.0*2.416*2)
= 34.83 tf/m

safety factor is

Normal condition

Fs = Qu/W
= 58.07/26.20 = 2.22>1.2

Earthquake condition

Fs = Qu'/W
= 34.83/26.20 = 1.33>1.0

7.2.5 Design of Steel Sheet Pile

(1) Basci formula

$Mp \Rightarrow FM_A$

$MA = P_A l_A + Pwlw$

$Mp = Pp + Pwlw$

Mp : moment around tie rod hook by passive earth pressure (tfm/m)
M_A : moment around tie rod hook by active earth pressure (tfm/m)
D : calculated depth of imbedded sheet pile (m)
Pp : passive earth pressure (tf/m)
Lp : arm length of action of Pp from tie rod (m)
P_A : active earth pressure (tf/m)
l_A : arm length of action of P_A from tie rod (m)
Pw : residual water pressure (tf/m)
lw : arm length of action of Pw from tie rod (m)
F : safety factor (1.2)

(2) Calculation of moment around the tie rod
Assuming the depth of imbedded sheet pile as D

<u>Normal condition</u>

<u>Residual Water Pressure in the Wall</u>

P_{h1} = 1.754 tf/m
Arm = 0.875-(-1.811)
= 2.686
P_{h2} = 1.873*D*1/2
= 0.9365D tf/m
Arm = 0.875-(-2.435)+D/3
= 3.31+D/3

Active earth pressure

(EL-0.562m)
P_{a1} = 1.744tf/m
Arm = 0.875-0.250
= 0.625

(EL-1.000m)
P_{a2} = 0.653tf/m
Arm = 0.875-(-0.493)
= 1.368

(EL-2.435m)
P_{a3} = 2.569tf/m
Arm = 0.875-(-1.841)
= 2.716

(below El-2.435m)
d_4 = D
(i) component-1
$P_{a4\text{-}1} = p_{a3} \times d_4 = 2.028 \times D$
acting point
= EL-2.435-D/2
Arm = 0.875-(-2.435-D/2)
= 3.31+D/2
(ii) component-2
$P_{a4\text{-}2} = 1/2 * D^2 * 0.9 * 0.368 = 0.1656 \times D^2$
acting point

= EL-2.435-2/3D
Arm = 0.875-(-2.435-2/3D)
= 3.31+2/3D

Passive earth pressure
(below El-2.435m)
d_4 = D
P_{p4} = $1/2*D^2*0.9*2=0.9\times D^2$
acting point
= EL-2.435-2/3D
Arm = 0.875-(-2.435-2/3D)
= 3.31+2/3D

total driving moment around the tie rod is
Md = 1.754*2.686
+ 0.9365D*(3.31+0.333D)
+ 1.744*0.625
+ 0.653*1.368
+ 2.569*2.716
+ (2.028*D)*(3.31+D/2)
+ $(0.1656*D^2)*(3.31+2/3D)$
= $4.711+3.100D+0.312D^2+1.090+0.893+6.977$
+ $6.71D+1.014D^2+0.5481D^2+0.1104D^3$
= $0.1104D^3+1.8741D^2+9.81D+13.67$

resistance moment around the tie rod is
Mr = $(0.9\times D^2)*(3.31+2/3D)$
= $0.600D^3+2.979D^2$

safety factor should be over 1.2 for both normal and earthquake conditions
Mr>=Fs*Md
$0.600D^3+2.979D^2>=1.2*(0.1104D^3+1.8741D^2+9.81D+13.67)$
$0.4675D^3+0.7301D^2-11.77D-16.40>=0$
D>=3.46m

Earthquake condition

Residual Water Pressure in the Wall
P_h = 1.754 tf/m
Arm = 0.875-(-1.811)
= 2.686
P_{h2} = 1.873*D*1/2
= 0.9365D tf/m
Arm = 0.875-(-2.435)+D/3
= 3.31+D/3

Active earth pressure
(EL-0.562m)
P_{a1} = 2.280 tf/m
Arm = 0.875-0.250
= 0.625
(EL-1.000m)
P_{a2} = 0.854tf/m
Arm = 0.875-(-0.493)
= 1.368

(EL-2.435m)

P_{a3} = 3.352tf/m

Arm = 0.875-(-1.841)

= 2.716

(below El-2.435m)

d_4 = D

(i) component-1

$P_{ea4\text{-}1}$ = $p_{ea3} \times d_4 = 2.642 \times D$

acting point

= EL-2.435-D/2

Arm = 0.875-(-2.435-D/2)

= 3.31+D/2

(ii) component-2

$P_{ea4\text{-}2}$ = $1/2 * D^2 * 0.9 * 0.474 = 0.2133 \times D^2$

acting point

= EL-2.435-2/3D

Arm = 0.875-(-2.435-2/3D)

= 3.31+2/3D

<u>Passive earth pressure</u>

(below El-2.435m)

d_4 = D

$P_{p4} = 1/2 * D^2 * 0.9 * 2.34 = 1.053 \times D^2$

acting point

= EL-2.435-2/3D

Arm = 0.875-(-2.435-2/3D)

= 3.31+2/3D

total driving moment around the tie rod is

Md = 1.754*2.686
+ 0.9365D*(3.31+0.333D)
+ 2.280*0.625
+ 0.854*1.368
+ 3.352*2.716
+ (2.642*D)*(3.31+D/2)
+ $(0.2133*D^2)$*(3.31+2/3D)
= $4.711+3.100D+0.312D^2+1.425+1.168+9.104+8.745D$
$+ 1.321D^2+0.7060D^2+0.1422D^3$
= $0.1422D^3+2.339D^2+11.854D+16.41$

resistance moment around the tie rod is

Mr = $(1.053 \times D^2)*(3.31+2/3D)$
= $0.702D^3+3.485D^2$

safety factor should be over 1.2 for both normal and earthquake conditions

Mr>=Fs*Md

$0.702D^3+3.485D^2 >= 1.2*(0.1422D^3+2.339D^2+11.854D+16.41)$

$0.5314D^3+0.6782D^2-14.225D-19.69 >= 0$

D>=3.61m

Therefore, the depth is determined in earthquake condition and D=3.61m

The total length of sheet pile is

L=1.875+2.435+3.61=7.92m → <u>8m (D=3.69m)</u>

(iii) Reaction of imaginary supporting point

It is assumed that 0.3 times the wall height(above the ground) is the distance of imaginary

supporting point as the foundation soil is clay.
H'=0.3*4.31=1.293m
The load on the imaginary supporting point is calculated as follows;

Normal condition

P (D=1.293m)
  = pad+pw'-ppd
  pad; active earth pressure underground
  ppd; pasive earth pressure underground
  pw'; residual water pressure underground
pad = 2.028+0.9*0.368*1.293
  = 2.456
ppd = 2.0*1.293
  = 2.586
pw' = 1.873-1.873*1.293/3.69
  = 1.217
P = 2.456+1.217-2.586
  = 1.087 tf/m^2 pressure at the imaginary supporting point

Earthquake condition

P (D=1.293m)
  = pad+pw'-ppd
  pad; active earth pressure underground
  ppd; pasive earth pressure underground
  pw'; residual water pressure underground
pad = 2.642+0.9*0.474*1.293
  = 3.194
ppd = 2.34*1.293
  = 3.026
pw' = 1.873-1.873*1.293/3.69
  = 1.217
P = 3.194+1.217-3.026
  = 1.385 tf/m^2

The reaction at the imaginary supporting point is calculated as follows;

Normal condition

Reaction at Imaginary Supporting Point ; Rd = 5.933 tf/m
Reaction at Tie Rod; Ap = 4.416 tf/m

Seismic condition

Reaction at Imaginary Supporting Point ; Rd = 7.076 tf/m
Reaction at Tie Rod; Ap = 5.459 tf/m

(3) Maximum Bending Moment
Assuming that the maximum moment occurs at x m above the ground elevation (EL-2.435m),

Mmax = Rd(x+H')-S6'(x+1/3*H')-S7'(x+2/3*H')
  -(PA3+Pw)*x*1/2x+(PA3+Pw-PA2)x/dR*1/2x*1/3x
where dR; depth between W.L and D.L.(=-0.562+2.435=1.873m)

Normal Condition

Mmax = 5.933*(x+1.293)-2.713*(x+1/3*1.293)-0.703*(x+2/3*1.293)

-(2.324+1.873)*x*1/2*x+(2.324+1.873-1.553)*x/1.873*1/2*x*1/3*x
= $5.933x+7.671-2.713x-1.169-0.703x-0.606-2.099x^2+0.235x^3$
= $0.235x^3-2.099x^2+2.517x+5.986$

The shear stress is zero at the point of maximum moment. Therefore,

dMmax/dx=0
$0.705x^2-4.198x+2.517=0$
x=0.677
Mmax=6.80 tfm/m

<u>Seismic Condition</u>

Mmax = 7.076*(x+1.293)-3.147*(x+1/3*1.293)-0.895*(x+2/3*1.293)
-(2.994+1.873)*x*1/2*x+(2.994+1.873-2.030)*x/1.873*1/2*x*1/3*x
= $7.076x+9.149-3.147x-1.356-0.895x-0.771-2.434x^2+0.252x^3$
= $0.252x^3-2.434x^2+3.034x+7.022$

The shear stress is zero at the point of maximum moment. Therefore,

dMmax/dx=0
$0.504x^2-4.868x+3.034=0$
x=0.670
Mmax=8.040 tfm/m

(4) Cross Section of Steel Sheet Pile
(i) Normal Condition
Z=Mmax/sa=6.800*10^5/1800=378 cm^3/m
(ii)Seismic Condition
Z=Mmax/sa=8.040*10^5/2700=298 cm^3/m

Therefore, the type of the steel sheet pile is Type-III

7.2.6.1 Design of tie rod and
(1) Tie rod
T=Ap×l
T; tensile load of tie rod (tf)
Ap; reaction of tie rod (tf/m)
L; inerval of tie rod (m) (l=1.6m)

<u>Normal Condition</u>
T=4.416*1.6=7.07 tf
<u>Seismic Condition</u>
T=5.459*1.6=8.73 tf

(2) Cross Section of Tie Rod
d = sqr(4T/πσa)
A: cross section of tie rod (cm^2)
T: tensile load of tie rod (tf)
σa: allowable stress (kg/cm^2)

Normal Condition
D=(4*7.07*10^3/3.14/900)^(1/2)=3.16 cm

Seismic Condition
D=(4*8.73*10^3/3.14/1400)^(1/2)=2.82 cm

7.2.6.2 Reinforcing Beam

(1) Bending Moment

M=Tl/4

M: design moment (tfm)

T: tensile load of rod (tf)

L: interval of tie rod (m) (l=1.6m)

<u>Normal Condition</u>

M=7.07*1.6/4=2.83 tfm

<u>Seismic Condition</u>

M=8.73*1.6/4=3.49 tfm

(2) Cross Section of Beam

Z=M/σa

Z: cross section coefficient (cm^3/m)

σa: allowable stress (kgf/cm^2)

Normal Condition

$Z=2.83*10^5/1400=202\ cm^3$

Seismic Condition

$Z=3.49*10^5/2100=166\ cm^3$

7.2.6.3 Check of Water tightness

L1/h1>=F

F = safety factor (3.0)

L1; seepage length (m)

Fs=(3.69*2+3.2)/(0.375+2.435)=10.58/2.81=3.77>3.0

Steel sheet plate	Type-III
Tie rod	Ø35

2.1 Asin Pumping Station

2.1.8 Structural Calculation of Concrete Sheet Pile

Design of Concrete Sheet Pile

Stability analysis of concrete sheet piles was made as follows;

Earth Pressure Coeficient (phai 22.5, c=0.41kg/cm2)

	Active	Passive
Normal	0.42	1.840
Seismic	0.53	2.065

Safety Factor

Normal	1.20
Seimic	1.00

Boundary Condition

River	Location	River Bed Elevation	Dike Top Elevation	allowanc	H1	H2	H	SF(N)	SF(E)
		m	m	m	m	m	m		
Semarang River	No.30	-2.4	1.1	0.5	4	8	12.0	1.39	1.24
Asin Retarding Pond	east	-2.7	1.1	0	3.8	8.2	12.0	1.30	1.16
Asin Pumping St. Com.1	all	-2.4	1.2	0.5	4.1	7.9	12.0	1.35	1.20
Asin Pumping St. Com.2	all	-2.7	1.2	0	3.9	8.1	12.0	1.26	1.12
Asin Pumping St.	north	-2.4	-0.3	0.5	2.6	5.4	8.0	1.26	1.12
Asin River 1	left	-3.7	0.3	0	4	8	12.0	1.39	1.24
Asin River 2	right	-2.7	0.3	0	3	7	10.0	1.47	1.31
Baru River 1	left	-2.4	0.85	0	3.3	6.8	10.0	1.35	1.21
Baru River 2	right	-3.4	1.2	0	4.6	8.4	13.0	1.32	1.18
Baru Pumping St. Com.	all	-3.4	1.2	0	4.6	8.4	13.0	1.22	1.09
Baru Pumping St.	north	-3.4	-0.3	0.5	3.6	7.4	11.0	1.38	1.23

Normal Condition

Concrete sheet pile (normal condition)

phai= 22.5 Ka= 0.42 w= 0.8
c= 0.41 Kp= 1.84 q= 1

Semarang River(river bed -2.4m, bank crown +1.1m, allowance 0.5m)

D= 4 m total= 12 m
h= 8 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	24	-4.13	5.04	25.10	47.10	6.04	53.14
M	97	-24.80	30.24	102.21	125.61	16.09	141.70
FS							1.39

Asin Retarding Pond (river bed -2.7m, dike crown +1.1m, allowance+0.0m)

D= 3.8 m total= 11 m
h= 7.2 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	20	-3.79	4.62	21.16	38.15	5.43	43.59
M	76	-21.22	25.87	80.55	91.57	13.04	104.61
FS							1.30

Asin Pumping Station Complex 1 (bed =-2.4m, dike crown +1.2m, allowance 0.5m)

D= 4.1 m total= 12 m
h= 7.9 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	24	-4.133	5.04	25.10	45.93	5.96	51.89
M	96	-24.59	29.99	101.36	120.96	15.69	136.65
FS							1.35

Asin Pumping Station Complex 2 (bed =-2.7m, dike crown +1.2m, allowance 0.0

D= 3.9 m total= 11 m
h= 7.1 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	20	-3.788	4.62	21.16	37.10	5.36	42.46
M	75	-21.03	25.64	79.83	87.81	12.68	100.48
FS							1.26

Asin Pumping Station (bed =-2.4m, dike crown -0.3m, allowance 0.5m)

D= 2.6 m total= 8 m
h= 5.4 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	11	-2.755	3.36	11.36	21.46	4.07	25.54
M	34	-12.95	15.79	36.53	38.63	7.33	45.96
FS							1.26

Asin River 1 (bed =-3.7 m, dike crown 0.3m, allowance 0.0m)

D= 4 m total= 12 m
h= 8 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	24	-4.133	5.04	25.10	47.10	6.04	53.14
M	97	-24.8	30.24	102.21	125.61	16.09	141.70
FS							1.39

Asin River 2 (bed =-2.7 m, dike crown 0.3m, allowance 0.0m)

D= 3 m total= 10 m
h= 7 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	17	-3.444	4.20	17.56	36.06	5.28	41.34
M	62	-18.94	23.10	65.76	84.15	12.32	96.47
FS							1.47

Baru River 1 (bed =-2.4 m, dike crown 0.85m, allowance 0.0m)

D= 3.25 m total= 10 m
h= 6.75 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	17	-3.444	4.20	17.56	33.53	5.09	38.63
M	60	-18.51	22.58	64.26	75.45	11.46	86.91
FS							1.35

Baru River 2 (bed =-3.4 m, dike crown 1.2m, allowance 0.0m)

D= 4.6 m total= 13 m
h= 8.4 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	28	-4.477	5.46	29.37	51.93	6.34	58.27
M	117	-27.76	33.85	123.45	145.41	17.74	163.15
FS							1.32

Baru Pumping Station Complex(bed =-3.7 m, dike crown 1.2m, allowance 0.0m)

D= 4.9 m total= 13 m
h= 8.1 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	28	-4.477	5.46	29.37	48.29	6.11	54.40
M	115	-27.09	33.03	120.46	130.38	16.50	146.88
FS							1.22

Baru Pumping Station (bed =-3.4 m, dike crown 0.3m, allowance 0.5m)

D= 3.6 m total= 11 m
h= 7.4 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	20	-3.788	4.62	21.16	40.30	5.58	45.89
M	77	-21.59	26.33	81.99	99.41	13.77	113.19
FS							1.38

Seismic Condition

Concrete sheet pile (seismic condition)

phai= 22.5 Ka= 0.53 w= 0.8
c= 0.41 Kp= 2.07 q= 1

Semarang River(river bed -2.4m, bank crown +1.1m, allowance 0.5m)

D= 4 m total= 12 m
h= 8 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	31	-5.22	6.36	31.67	52.99	6.79	59.78
M	122	-31.29	38.16	128.98	141.31	18.11	159.42
FS							1.24

Asin Retarding Pond (river bed -2.7m, dike crown +1.1m, allowance+0.0m)

D= 3.8 m total= 11 m
h= 7.2 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	26	-4.78	5.83	26.70	42.92	6.11	49.03
M	96	-26.77	32.65	101.64	103.02	14.67	117.68
FS							1.16

Asin Pumping Station Complex 1 (bed =-2.4m, dike crown +1.2m, allowance 0.5n

D= 4.1 m total= 12 m
h= 7.9 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	31	-5.215	6.36	31.67	51.68	6.70	58.38
M	121	-31.03	37.84	127.91	136.08	17.66	153.73
FS							1.20

Asin Pumping Station Complex 2 (bed =-2.7m, dike crown +1.2m, allowance 0.0n

D= 3.9 m total= 11 m
h= 7.1 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	26	-4.781	5.83	26.70	41.74	6.03	47.77
M	95	-26.53	32.36	100.74	98.78	14.26	113.04
FS							1.12

Asin Pumping Station (bed =-2.4m, dike crown -0.3m, allowance 0.5m)

D= 2.6 m total= 8 m
h= 5.4 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	14	-3.477	4.24	14.33	24.14	4.58	28.73
M	43	-16.34	19.93	46.10	43.46	8.25	51.71
FS							1.12

Name of Structure	CONCRETE SHEET PILE	Category Calculation	Stability Analysis	Page	4/4

Asin River 1 (bed =-3.7 m, dike crown 0.3m, allowance 0.0m)

D= 4 m total= 12 m
h= 8 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	31	-5.215	6.36	31.67	52.99	6.79	59.78
M	122	-31.29	38.16	128.98	141.31	18.11	159.42
FS							1.24

Asin River 2 (bed =-2.7 m, dike crown 0.3m, allowance 0.0m)

D= 3 m total= 10 m
h= 7 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	21	-4.346	5.30	22.15	40.57	5.94	46.51
M	78	-23.9	29.15	82.98	94.67	13.86	108.53
FS							1.31

Baru River 1 (bed =-2.4 m, dike crown 0.85m, allowance 0.0m)

D= 3.25 m total= 10 m
h= 6.75 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	21	-4.346	5.30	22.15	37.73	5.73	43.45
M	76	-23.36	28.49	81.09	84.88	12.89	97.77
FS							1.21

Baru River 2 (bed =-3.4 m, dike crown 1.2m, allowance 0.0m)

D= 4.6 m total= 13 m
h= 8.4 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	36	-5.65	6.89	37.07	58.42	7.13	65.55
M	148	-35.03	42.72	155.78	163.59	19.96	183.55
FS							1.18

Baru Pumping Station Complex(bed =-3.7 m, dike crown 1.2m, allowance 0.0m)

D= 4.9 m total= 13 m
h= 8.1 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	36	-5.65	6.89	37.07	54.33	6.87	61.20
M	145	-34.18	41.68	152.01	146.68	18.56	165.24
FS							1.09

Baru Pumping Station (bed =-3.4 m, dike crown -0.3m, allowance 0.5m)

D= 3.6 m total= 11 m
h= 7.4 m

	Pa P1	pc	pq	PT	Pp P1	pc	PT
P	26	-4.781	5.83	26.70	45.34	6.28	51.62
M	97	-27.25	33.23	103.46	111.84	15.49	127.33
FS							1.23

2.1 Asin Pumping Station

2.1.9 Seepage Analysis

Name of Structure	Sheet Pile For Seepage Control	Category of calculation	Seepage Analysis	Page	1/1

Seepage Control Analysis

(1) Asin Pumping Station

The length of steel sheet pile for seepage control is the same as the gate foundation.
L=5.0m

(2) Asin Retarding Pond

Required length of sheet pile is calculated by using Lane's formula as follows

$C<(L/3+2l)/H)$

Where; C=8.5 (very fine sand or silt)

H=+0.25-(-2.5)=2.75 m (M.H.W.L of the sea-Pond D.L.W.L)

L=16.8m (bottom width of dike)

Therefore,

$l > 8.9$ m

The total length of the concrete sheet pile is

L=8.9+3.8=12.7-----------13.0m

2.1 Asin Pumping Station

2.1.10 Alignment Calculation of Asin River

Name of Structure	Asin River	Category of calculation	Allgment	Page	1/2

ASIN RIVER

DRAW NO.	IP	N	E	R	DISTANCE		α			TL		CL		Acc. Dis
1	IP3/BPA	9,231,172.0	434,808.0											
					90.554	95.07				-			90.554	90.554
	IPA1	9,231,081.8	434,816.0	-			178.12					-	-	90.554
					309.977	276.95		96.95		-	-		309.977	400.531
	IPA2	9,230,774.1	434,853.5	-					179.29			-	-	400.531
					455.796	96.23		276.23		-	-		455.796	856.327
	IPA3	9,230,321.0	434,903.0	-			179.20					-	-	856.327
					328.171	277.03		97.03		-	-		328.171	1,184.498
	IPA4	9,229,995.3	434,943.2	-					179.67			-	-	1,184.498
					23.964	96.71		276.71		-	-		23.964	1,208.462
	EPA	9,229,971.5	434,946.0	-			-96.71					-	-	1,208.462
										-			-	

Name of Structure	Asin River	Category of calculation	Aligment	Page	2/2

ASIN BOX CULVERT

DRAW NO.	IP	N	E	R	DISTANCE		α		α	TL		CL		Acc. Dis	
1	BPCA	9,229,981.1	434,948.6												
					24.221	136.05				8.726			15.495	15.495	
	IPCA1	9,229,964.3	434,966.0	25			141.498					16.799	16.799	32.294	1.479
					83.726	277.55		97.55		8.726	8.502		66.498	98.792	
	IPCA2	9,229,881.3	434,977.0	90					169.202			16.961	16.961	115.753	0.401
				6	82.599	108.35		288.35		~~8.624~~	8.502		65.423	181.176	
	IPCA3	9,229,802.9	435,003.0	~~10~~			98.082					14.297	14.297	195.473	3.238
					7.297	10.26		190.26		~~8.674~~			(1.377)	194.096	
	EPCA	9,229,804.2	435,010.2	-					-190.263				-	194.096	

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