RETAINING WALL PROBLEMS

P1. CANTILEVER RETAINING WALL

Question

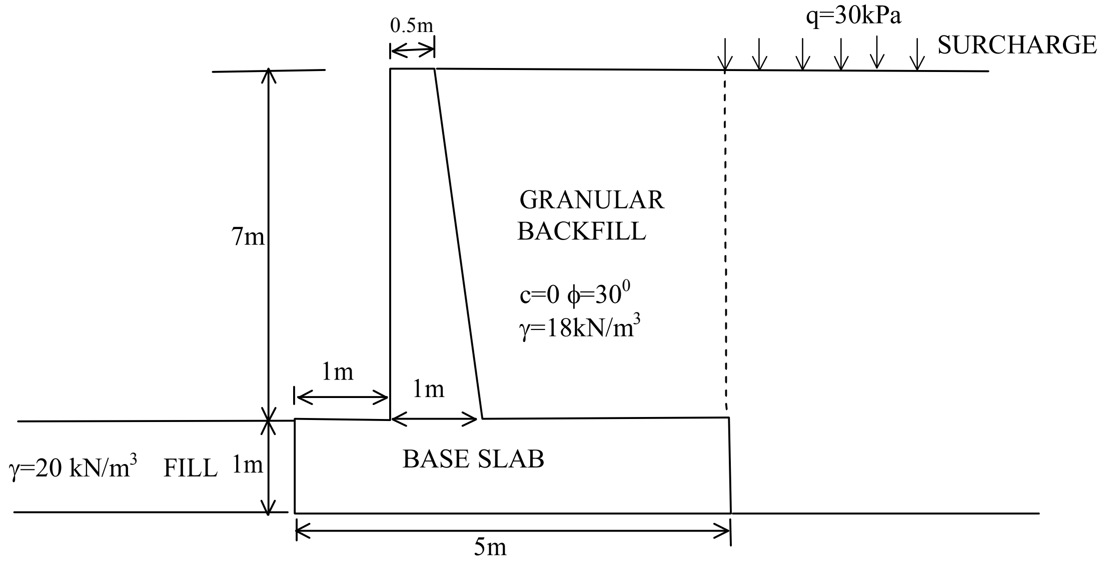
For the retaining wall and the profile shown below, calculate:

a. The safety factor against overturning,

b. The safety factor against sliding (minimum required F.S. =1.5)
   Do not consider the passive resistance of the fill in front of the wall.

c. If the overturning safety is not satisfactory, extend the base to the right and satisfy the overturning stability requirement.

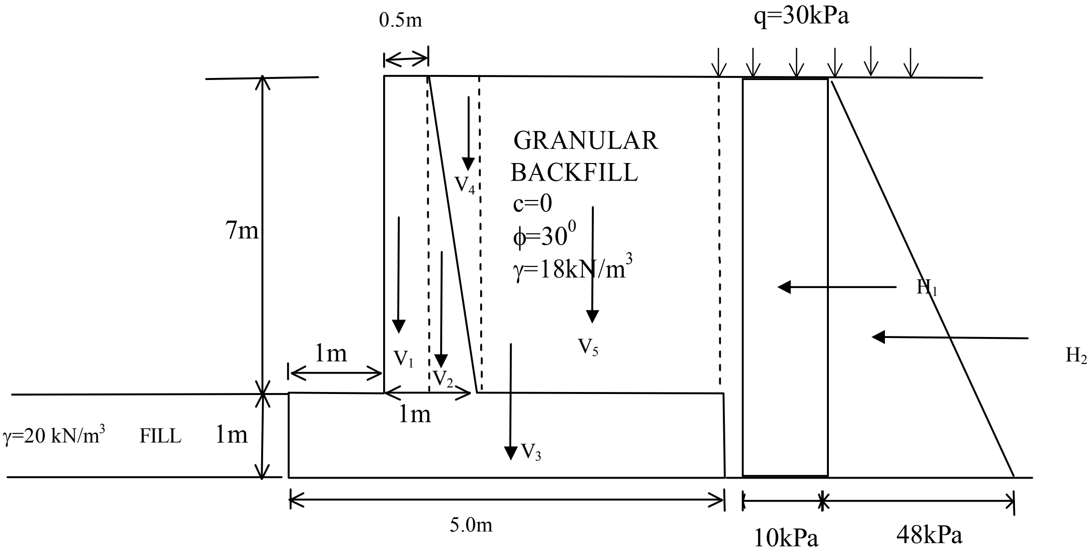
   If the sliding is not satisfactory, design a shear key (location, thickness, depth) under the base slab to satisfy the sliding stability. Take advantage of passive resistance of the foundation soil. Calculate the vertical stress starting from the top level of the base but consider the passive resistance starting from the bottom level of the base slab (i.e. in the sand). Use a factor of safety of 2.0 with respect to passive resistance.



MEDIUM DENSE SILTY SAND

c=0, $\phi=32^0$, $\gamma=20kN/m^3$, $\tan\delta=0.5$(base friction) , $\gamma_{conc}=24kN/m^3$

Solution:



$K_a = \tan^2(45-\phi/2)$

For granular backfill $\Rightarrow K_a = \tan^2(45-30/2) = 0.333$

Active pressure, $\mathbf{p_a = (q+\gamma z)K_a - 2c\sqrt{Ka}}$

$z=0 \Rightarrow p_a = 30x0.333 = 10\ kN/m^2$

$z=8 \Rightarrow \sigma_a = (30+18x8)0.333 = 58\ kN/m^2$

Force(kN/m)	Arm,about toe(m)	Moment(kN.m/m)
V_1=0.5x7x24=84	1.25	105
V_2=0.5x7x1/2x24=42	1.67	70
V_3=1x5x24=120	2.5	300
V_4=0.5x7x1/2x18=31.5	1.83	57.75
V_5=3x7x18=378	3.5	1323
ΣV=655.5		ΣM_r=1855.75
H_1=10x8=80	4	320
H_2=(58-10)x8x1/2=192	8/3	512
ΣH=272.0		ΣM_{ov}=832

a)

$$(F.S.)_{ov} = \frac{\sum M_r}{\sum M_{ov}} = \frac{1855.75}{832} = 2.23$$

$(F.S.)_{ov}=2.23>2.0$ O.K.

b)

c=0(at the base)   do not consider

$$(F.S.)_{sliding} = \frac{\sum V.\tan\delta + (2/3cB) + P_p}{\sum H}$$

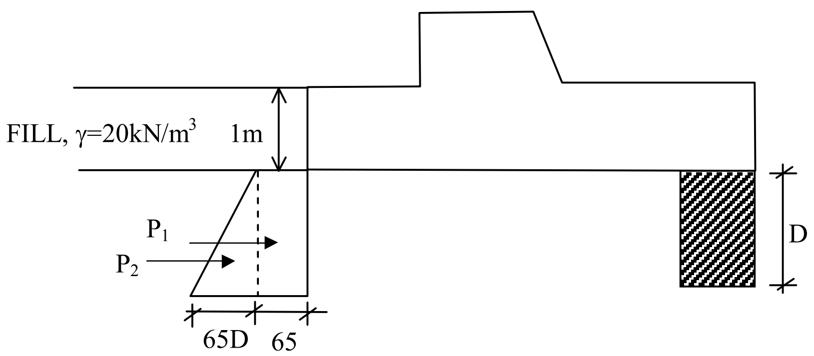
$$(F.S.)_{sliding} = \frac{\sum V.\tan\delta}{\sum H} = \frac{655.5x0.5}{272} = 1.20$$

$(F.S.)_{sliding}=1.20<1.5$ NOT O.K. <u>DESIGN BASE KEY</u>

c)

<u>Base key design:</u>

Passive resistance at the base key;



$K_p=\tan^2(45+32/2)=3.25$

$z=0 \Rightarrow p_p=1x20x3.25=65$ kPa

$z=D \Rightarrow p_p=(1x20+20xD)x3.25=65+65D$ kPa

$P_p=P_1+P_2=65D+1/2x65D^2$

Use F.S.=2.0 w.r.t. passive resistance $\Rightarrow P_p=1/2(65D+1/2x65D^2)$

$$(F.S.)_{sliding} = \frac{\sum V.\tan\delta + P_p}{\sum H} = \frac{655.55x0.5 + 1/2(65D + 1/2x65D_2)}{272} = 1.5$$

Then, $65D+32.5D^2=160.5$ $\Rightarrow$ $\underline{D=1.43m}$

<u>If passive resistance (with a F.S. of 2.0) is subtracted from the driving horizontal forces, (i.e. used in the denominator)</u>

Use F.S.=2.0 w.r.t. passive resistance $\Rightarrow$ $P_p=1/2(65D+1/2x65D^2)$

$$(F.S.)_{sliding} = \frac{\sum V.\tan\delta}{H-Pp} = 1.50$$

Then, $\underline{D=1.07m}$

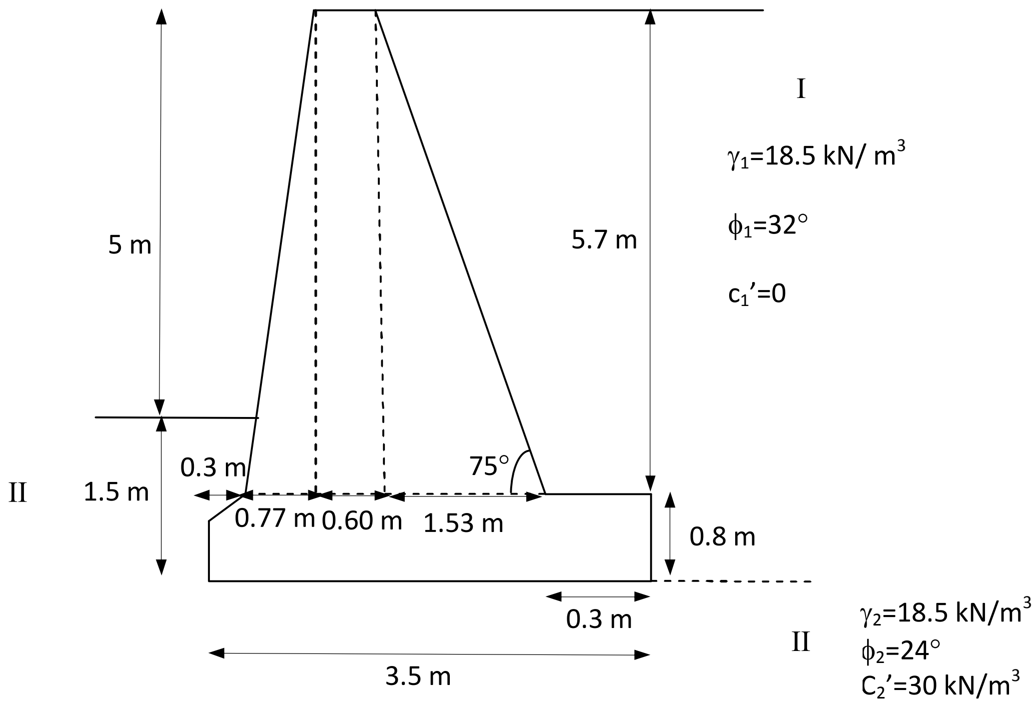
Take **D=1.43m** as it is on safe side.

P2. GRAVITY RETAINING WALL

Question

A gravity retaining wall is shown below. Use δ=2/3 ϕ and Coulomb active earth pressure theory. Determine

a. The factor of safety against overturning
b. The factor of safety against sliding
c. Calculate base pressures for both cases;
   - considering the passive pressure, and
   - neglecting it.

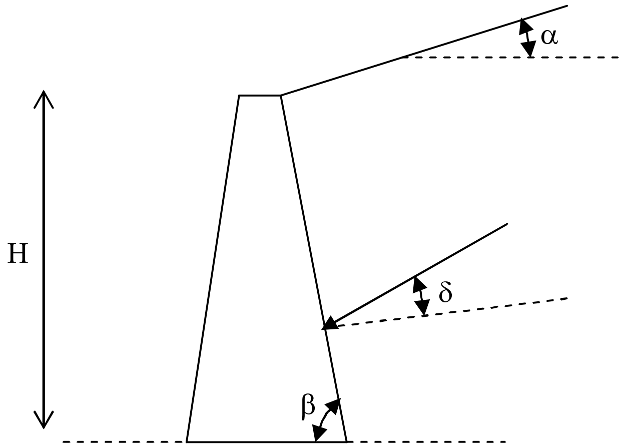


Soil properties: **I)** γ_1=18.5 kN/m^3 , ϕ_1=32^0, c_1= 0

II) γ_2=18.0 kN/m^3 , ϕ_2=24^0, c_2= 30 kN/m^2

$\gamma_{concrete}$=24 kN/m^3

Note: In Coulomb's active earth pressure theory, the forces to be considered are **only** P_a(Coulomb) and weight of the wall i.e. the weight of the soil above the back face of the wall is not taken into account.



Coulomb active forces;

$$P_a = \frac{1}{2}\gamma H^2 K_a \quad \text{where}$$

H =Height of the wall

K_a = Coulomb's active earth pressure coefficient

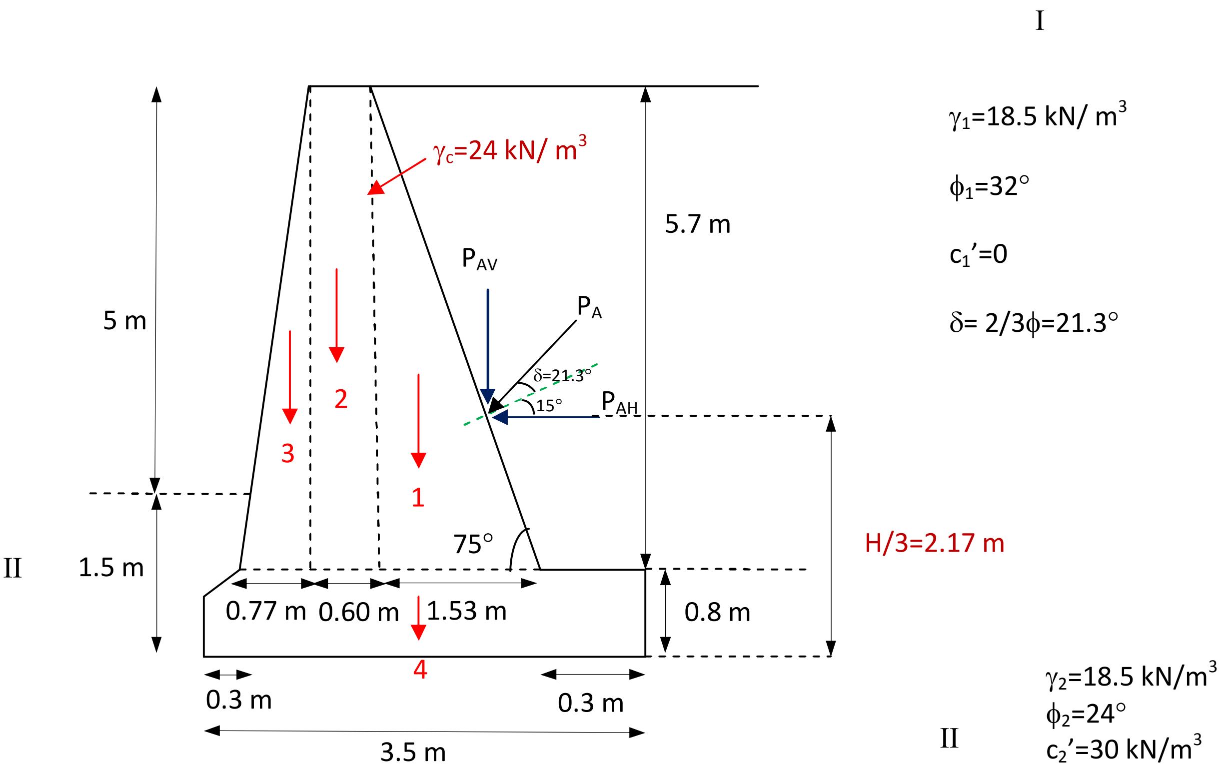
$$K_a = \frac{\sin^2(\beta+\phi)}{\sin^2\beta.\sin(\beta-\delta).(1+\sqrt{\dfrac{\sin(\phi+\delta).\sin(\phi-\alpha)}{\sin(\beta-\delta).\sin(\alpha+\beta)}})^2}$$

With horizontal backfill; α=0°

With vertical retaining wall; β=90°

δ: friction between the wall and adjacent soil

Solution:



For $\alpha=0^0$

$\beta=75^0 \quad \Rightarrow K_a=0.4023$ (use eqn. 1)

$\phi=32^0$

$\delta=(2/3)x32=21.3^0$

$$P_a=\frac{1}{2}.\gamma.H^2.K_a = \frac{1}{2}x18.5x6.5^2\,x0.4023 = 157.22\ kN/m$$

$$P_h = P_a.\cos(15+\delta)=157.22\,x\cos 36.3=126.65\ kN/m$$

$$P_v = P_a.\sin(15+\delta)=157.22\,x\sin 36.3= 93.15\ kN/m$$

Force (kN/m)	Moment arm about pt. A (m)	Moment (kN.m/m)
1) $(\frac{1}{2}x1.53x5.7)x24 = 104.65$	2.18	228.14
2) $(0.6x5.7)x24 = 82.08$	1.37	112.45
3) $(\frac{1}{2}x0.77x5.7)x24 = 52.67$	0.81	42.66
4) $(3.5x0.8)x23.58 = 67.20$	1.75	117.60
P_v= 93.15	2.83	263.61
ΣV=399.75		**$\Sigma M_{resisting}$ =764.46**

$\Sigma \mathbf{M}_{\mathbf{overturning}}$= P_h x H/3 = 126.65x 2.17 = **274.83 kN.m/m**

a) $(F.S.)_{overt.} = \frac{\sum M_r}{\sum M_o} = \frac{764.46}{274.83} = 2.78 > 2.0$ O.K.

Note: if there is cohesionless soil at the base (c=0) ignore this term

if passive pressure is considered

b) $$(F.S.)_{sliding} = \frac{\sum V.\tan\delta + (\frac{2}{3}.c_2 .B) + P_p}{\sum H}$$

$\delta=(2/3)x\phi_2$

- **<u>P_p is ignored</u>**

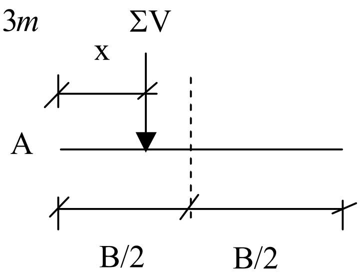
$$(F.S)_{sliding} = \frac{399.75x\tan(\frac{2}{3}x24) + (\frac{2}{3}x30x3.5)}{126.65} = 1.46$$

c. Pressure on soil at toe and heel

-If P_p is ignored

ΣM_{net}=764.46-274.83=489.63 kN.m/m

$$x = \frac{\sum M_{net}}{\sum V} = \frac{489.63}{399.75} = 1.22m \qquad \Rightarrow e = \frac{B}{2} - x = \frac{3.5}{2} - 1.22 = 0.53m$$



$$q_{\min}^{\max} = \frac{399.75}{3.5}\left[1 \pm \frac{6x0.53}{3.5}\right]$$

q_{max}=217.99 kN/ m^2 / m (toe)

q_{min}=10.44 kN/ m^2 / m (heel)

-If P_p is considered

K_p = (1+sin24°)/(1-sin24°)=2.37

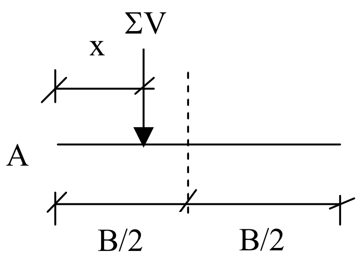
p_p @z=0 = $K_p(\gamma z)$+ $2c(K_p)^{0.5}$=2x30x$2.37^{0.5}$=92.40 kPa

p_p @z=1.5 = $K_p(\gamma z)$+ $2c(K_p)^{0.5}$=2.37x18.5x1.5+ 92.40=65.80+92.40=155.20 kPa

M_{res} (due to P_p)=92.4x1.5^2x0.5+0.5x65.80x(1/3)x1.5^2=128.63 kN.m/m

ΣM_{res}=764.46+128.63 = 893.09 kN.m/m

ΣM_{net}=893.09-274.83=618.26 kN.m/m



$$x = \frac{\sum M_{net}}{\sum V} = \frac{618.26}{399.75} = 1.55m \qquad \Rightarrow e = \frac{3.5}{2} - 1.55 = 0.20m$$

$$q_{\min}^{\max} = \frac{399.75}{3.5}\left[1 \pm \frac{6x0.20}{3.5}\right]$$

q_{max}=153.37 kN/ m^2 / m (toe)

q_{min}=75.05 kN/ m^2 / m (heel)

P3. REINFORCED EARTH WALL

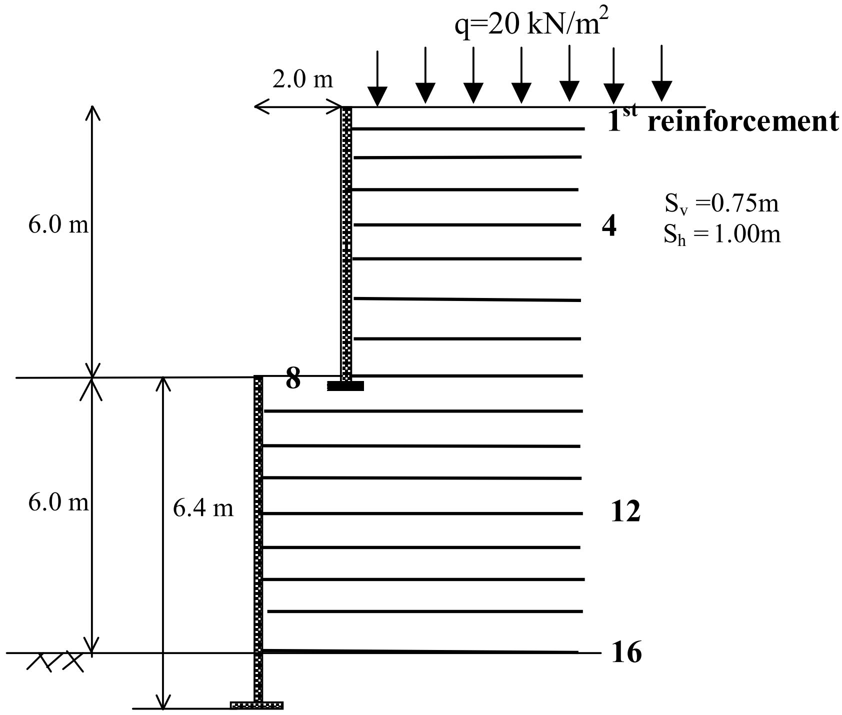
Question:

A reinforced earth wall is to be constructed as shown in the figure below. The material that will be used as backfill shall have the following properties, γ =17 kN/m^3, ϕ=30^0, c=0. The strips will be galvanized steel and will have a width of 75mm. The yield stress for strip material is f_y=3x10^5 kN/m^2.

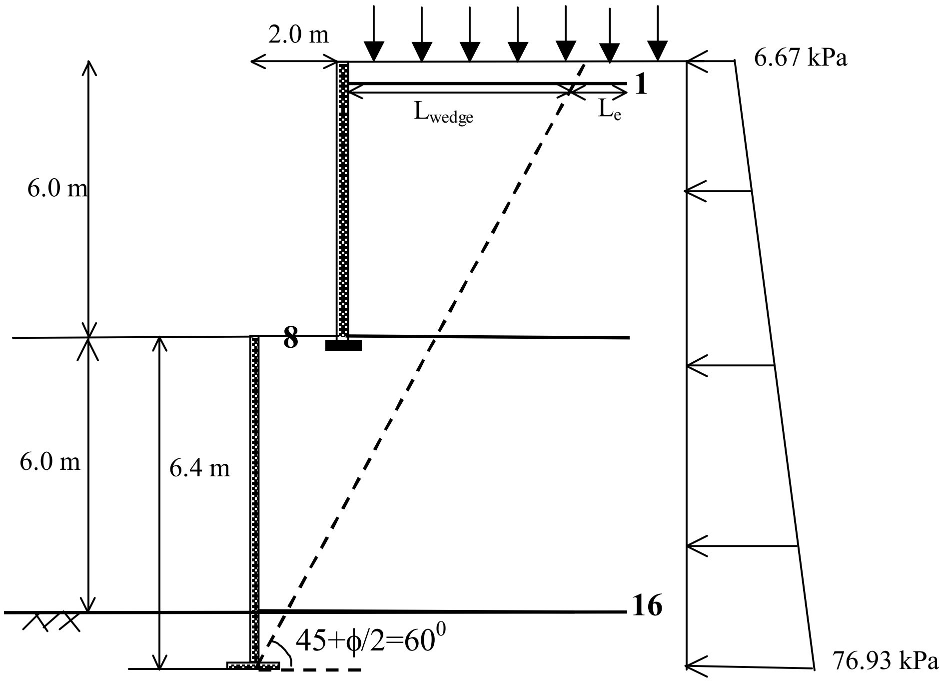
a. Design the reinforcements (i.e. determine the length and thickness) by using a factor of safety of 3.0 for both tie-breaking and pull-out.

b. Find the factor of safety along sliding on the base and calculate the base pressures for the foundation soil.

- Design life for structure 50 yrs.
- Corrosion=0.025 mm/yr
- Use Rankine Earth Pressure Theory and take the friction angle between soil and reinforcement as 20^0



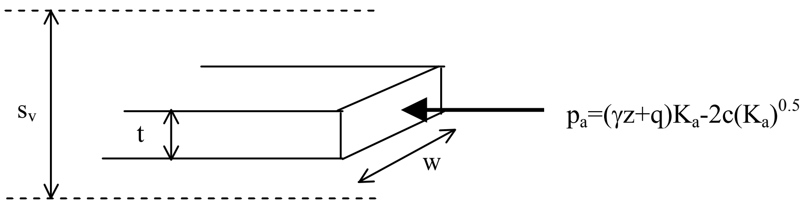
Solution:



$\phi=30^0 \Rightarrow K_a = 1/3$

a) Design of reinforcement

As far as the tie breaking is concerned, bottom reinforcement (16) is the most critical one since the lateral pressure is maximum at that level.



$$(F.S.)_{breaking} = \frac{w.t.f_y}{T_{max}} = 3.0$$

$$T = S_v.S_h.(\gamma z + q)\ K_a$$

$$T_{max} = 0.75x1.0x\ (12x17 + 20)x\frac{1}{3} = 56\ kN$$

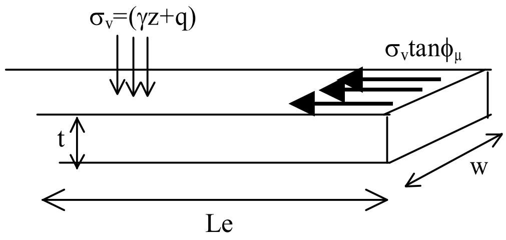
$$(F.S.)_{breaking} = \frac{0.075\,x\,t\,x3x10^5}{56} = 3.0 \qquad \Rightarrow t = 7.46\ mm$$

Corrosion rate $\Rightarrow$ 0.025mm/yr. x 50 =1.25mm

t=7.46+1.25 = 8.71mm

USE t_{design} =9mm

- As far as tie pull-out is concerned,



Frictional resistance is available on both surface (top and bottom)

Friction angle between soil and reinforcement

$$(F.S.)_{pull-out} = \frac{2\cancel{(\gamma z + q)}\tan\phi_\mu L_e w}{\cancel{(\gamma z + q)}K_a S_v S_h} = \frac{2\tan\phi_\mu L_e w}{K_a S_v S_h} = 3.0$$

$$(F.S.)_{pull-out} = \frac{2x0.075xL_e\,x\tan 20}{\frac{1}{3}x0.75x1.0} = 3.0 \qquad \Rightarrow L_e = 13.7m$$

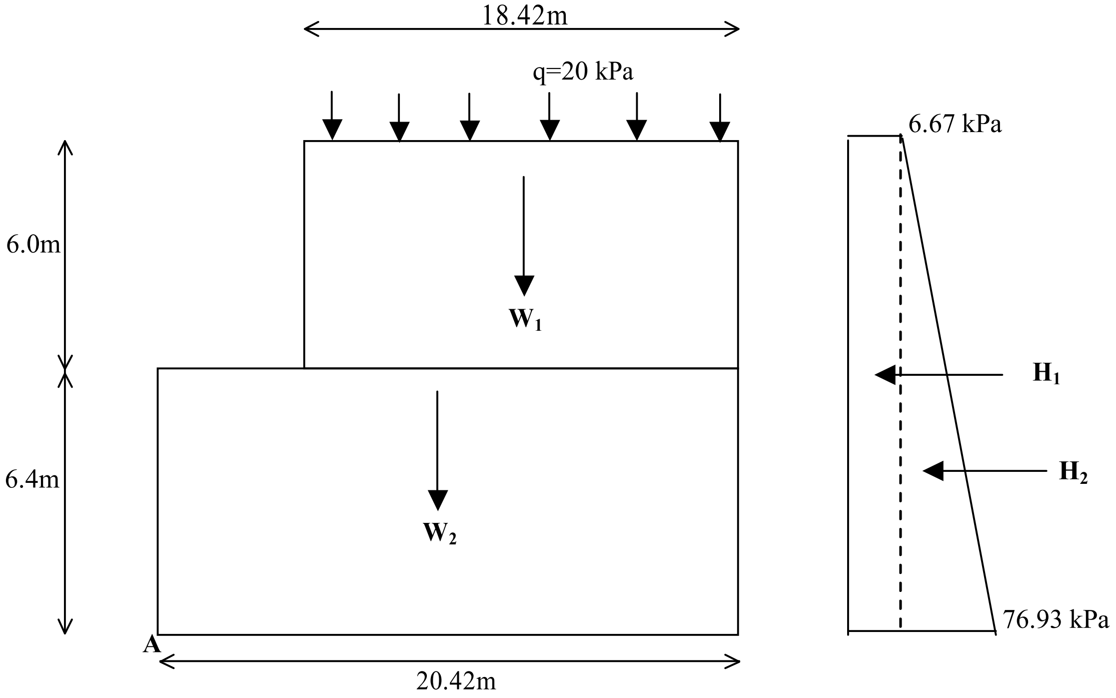
<u>Since first reinforcement (1) is the most critical one when the pull-out criterion is concerned,</u>

$$\tan(45-\phi/2) = \frac{L_{wedge} + 2}{12.4 - 0.75} \qquad \Rightarrow L_{wedge} = 4.72m$$

- Total tie length $L=L_{wedge}+L_e$=13.7+4.72=18.42m for upper 6m of the wall

- For lower 6m of the wall , L=20.42m

b) $(F.S.)_{sliding}$ and Base Pressure



Forces (kN/m)	Moment arm, about A (m)	Moment (kN.m/m)
W_1=18.42x6.0x17=1878.8	11.21	21061
W_2 = (18.42+2)x6.4x17 = 2221.7	10.21	22684
Load = 20x18.42 = 368.4	11.21	4130
ΣF_v = 4469		**ΣM_r = 47875**
H_1 = 6.67x12.4 = 82.7	12.4 /2	512.7
H_2 = (76.93-6.67)x12.4x(1/2) = 435.7	12.4 /3	1800
ΣF_h = 518		**ΣM_{ov} = 2313**

$(FS)_{sliding} = (\Sigma F_v \cdot \tan\delta) / \Sigma F_h$

In gravity or cantilever retaining walls, at the base of the wall, we would use tan δ for soil-wall friction. However in this problem, we see that, at the bottom of the wall, there is soil-soil interface, therefore we should use the friction angle of the soil in the F.s. sliding equation. (If two soils have different internal friction angles, the lower value should be used.

$(FS)_{sliding} = (4469 \times \tan 30) / 518$

$= \mathbf{4.98}$

$X = \Sigma M_{net} / \Sigma F_v = (47875 - 2313) / 4469 = 10.2$ m

$e = B/2 - X = (18.42 + 2)/2 - 10.2 = 0.01 \sim 0$ $\Rightarrow$ no eccentricity

$q_{max} = \Sigma F_v / B = 4469 / 20.42$

$= \mathbf{218.85\ kN/m^2 / m}$