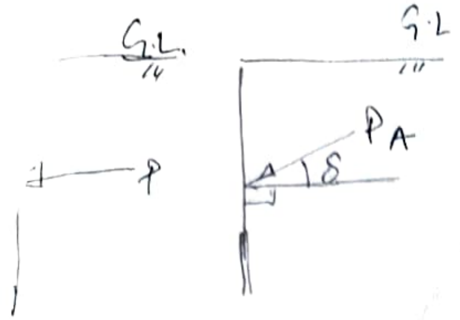
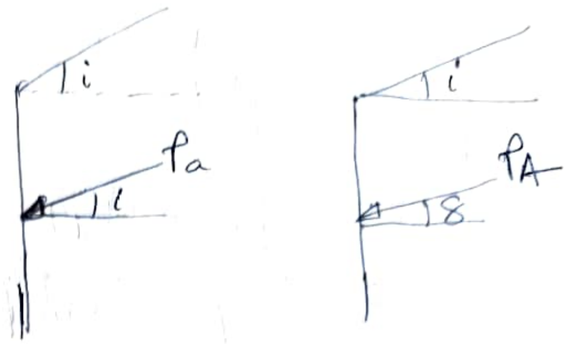
Coulomb's Earth Pressure theory



Rankine's theory

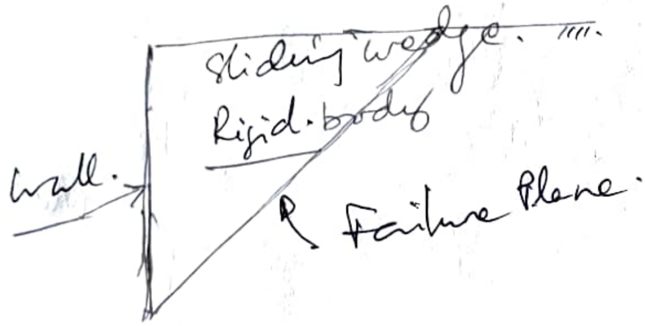
Coulomb's theory

δ = friction angle between soil & wall



Rankine theory

Coulomb's theory



Active Earth Pressure (For Sand)

$\beta = 180^\circ - (\theta + \alpha)$

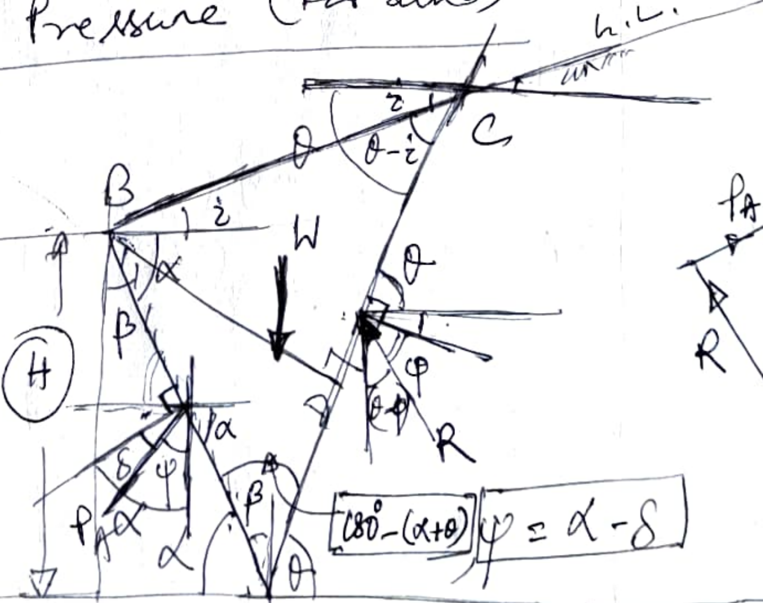
$\sin\beta = \sin(\alpha + \theta)$

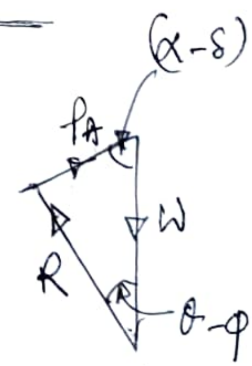
$W = \frac{1}{2} \times \gamma \times AC \times BD.$

$\frac{AC}{\sin(\alpha + i)} = \frac{AB}{\sin(\theta - i)}$

$\Rightarrow AC = AB \sin(\alpha + i) / \sin(\theta - i)$

$BD = AB \sin(\alpha + \theta)$





Force Polygon

$\boxed{AB = \frac{H}{\sin\alpha}}$

$W = \left[\frac{1}{2} \times \gamma \times AB \sin(\alpha + i) / \sin(\theta - i)\right] \times AB \sin(\alpha + \theta)$

$$= \frac{\gamma}{2} \times \frac{H}{\sin\alpha} \times \frac{\sin(\alpha+i)}{\sin(\theta-i)} \times \frac{H}{\sin\alpha} \times \sin(\alpha+\theta)$$

$$= \frac{\gamma H^2}{2\sin^2\alpha} \frac{\sin(\alpha+i)\sin(\alpha+\theta)}{\sin(\theta-i)}$$

$$\boxed{\alpha = 90^\circ - \beta}$$

$$\frac{P_A}{\sin(\theta-\varphi)} = \frac{W}{\sin[180^\circ-(\alpha-\delta)-(\theta-\varphi)]}$$

$$\Rightarrow P_A = \frac{W\sin(\theta-\varphi)}{\sin(180^\circ-\alpha-\theta+\varphi+\delta)}$$

for maximum value of P_A

$$\frac{dP_A}{d\theta} = 0$$

$\frac{d^2P}{d\theta^2} = -ve$ Maxm.

$\frac{d^2P}{d\theta^2} = +ve$ Mini

$$P_A = \frac{1}{2}\gamma H^2 K_A$$

where, $$K_A = \frac{\sin^2(\alpha+\varphi)}{\sin^2\alpha\,\sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-i)}{\sin(\alpha-\delta)\sin(\alpha+i)}}\right]^2}$$

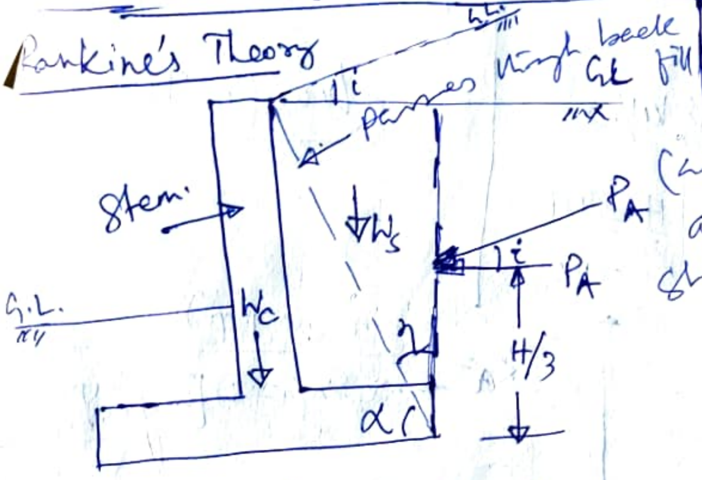
or $$\boxed{K_A = \frac{\cos^2(\varphi-\beta)}{\cos^2\beta\cos(\delta+\beta)\left[1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-i)}{\cos(\delta+\beta)\cos(i-\beta)}}\right]^2}}$$

If, $i=\delta=0$ & $\alpha = 90^\circ$

$$K_A = \frac{1-\sin\varphi}{1+\sin\varphi}$$

Some important considerations while designing retaining wall by Rankine Theory & Coloumb's theory

Rankine's Theory



$\alpha = 45^\circ + \frac{\varphi}{2}$, $\eta = 45^\circ - \frac{\varphi}{2}$

To be checked.

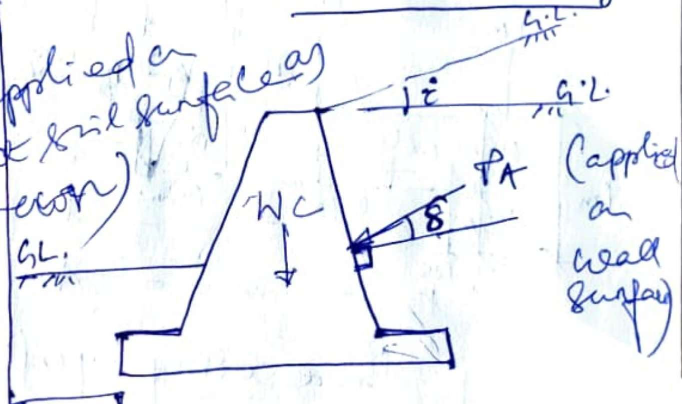
- The dotted line should not cross the ~~sys~~ stem of wall. If it crosses the stem Rankine's theory can't be applied for design of retaining wall.
- For inclined back fill (i)

$$\alpha = \left(45^\circ + \frac{\varphi}{2}\right) - \frac{i}{2} + \sin^{-1}\left(\frac{\sin i}{\sin\varphi}\right)$$

$$\eta = \left(45^\circ + \frac{i}{2}\right) - \frac{\varphi}{2} - \sin^{-1}\left(\frac{\sin i}{\sin\varphi}\right)$$

- Both W_C & W_s are considered for stability analysis of Retaining wall.

Coloumb's Theory



- When α crosses the stem, Coloumb's theory can be applied.
- Here only W_C is considered.

No use of wt of soil not considered

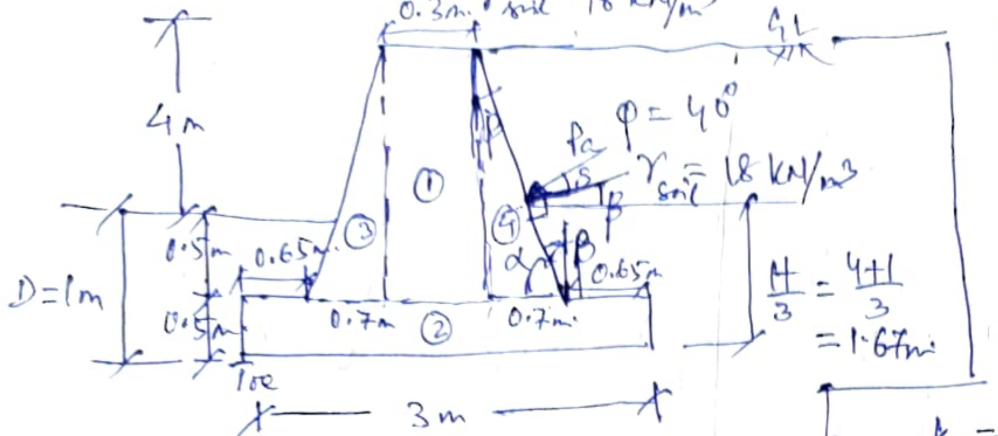
Example. Gravity Retaining wall.

$\gamma_c = 24$ kN/m³

$H = 4m$, $\varphi = 40°$, $\delta = \frac{2}{3}\varphi = 26.7°$, $i = 0$,

$\alpha = \tan^{-1}\left(\frac{4+0.5}{0.7}\right) =$

$\beta = 90° - 81° = 9°$.



Coloumb's Theory

$$K_a = \frac{\sin^2(\alpha+\varphi)}{\sin^2\alpha \sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\varphi+\delta)\times\sin(\varphi-i)}{\sin(\alpha-\delta)\cdot\sin(\alpha+i)}}\right]^2}$$

$= 0.256$

Base soil properties

$C = 25$ kN/m² $\varphi = 20°$

α = adhesion factor = 0.9

$C_a = \alpha C = 0.9 \times 25 =$

$\delta = 0.75\varphi = 15°$

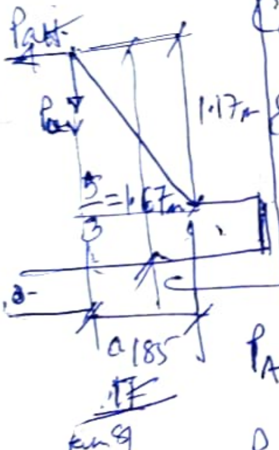
$\gamma = 18$ kN/m³.

$P_A = \frac{1}{2}\gamma K_a H'^2$

$H' = H + D$
$= 4 + 1 = 5m$

$= \frac{1}{2} \times 18 \times 0.256 \times 5^2$

$= 57.6$ kN/m.



$P_{AH} = P_A \cos(\delta+\beta) = 57.6 \times \cos(26.7+9) = 46.8$ kN/m.

$P_{AV} = P_A \sin(\delta+\beta) = 57.6 \sin(26.7+9) = 33.61$ kN/m

	Vertical force (kN/m) ①	Horizontal force (kN/m) ②	Lever arm (m) ③	M_R (kN-m/m) ④ = ① × ③	M_o (kN-m/m) ⑤ = ② × ③
1.	$W_1 = 0.3 \times 4.5 \times 24 = 32.4$	—	$3/2 = 1.5$	$32.4 \times 1.5 = 48.6$	
2.	$W_2 = 3 \times 0.5 \times 24 = 36$	—	$3/2 = 1.5$	54	
3.	$W_3 = \frac{1}{2} \times 4.5 \times 0.7 \times 24 = 37.8$	—	$0.65 + \frac{2}{3} \times 0.7 = 1.12$	42.3	
4.	$W_4 = 37.8$	—	$0.65 + 0.7 + 0.3 + \frac{0.7}{3} = 1.88$	71.1	
5.	$P_{AV} = 33.61$		$0.65 + 0.7 + 0.3 + (0.7 - 0.185) = 3.165$	72.8	
		$P_{AH} = 46.8$ kN/m	1.67		$46.8 \times 1.67 = 78.2$
	$\Sigma V = 177.61$ kN/m	$\Sigma H = 46.8$ kN/m		$\Sigma M_R = 288.8$ kN/m	$\Sigma M_o = 78.2$ kN-m/m

① $(F.O.S)_{sliding} = \frac{C_a(B'\times 1) + (\Sigma V)(\tan\delta)^{\approx \mu}}{\Sigma H}$

$$\boxed{\frac{C_a(B'\times1)+(\Sigma V)\tan\delta}{\Sigma H} = 2.27 > 1.5 \text{ hence safe.}}$$

$$= \frac{(25\times0.9)\times(3\times1)+177.61\times\tan 15^\circ}{46.8}$$

$= 2.5 > 1.5$ (ok) hence safe.

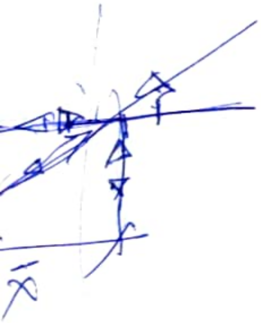
② $(F.O.S)_{Overturning} = \frac{\Sigma M_R}{\Sigma M_O} = \frac{288.8}{78.2}$

$= 3.7 > 1.5$

safe.

③ No tension ~~check~~. condition.

$$\bar{x} = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V} = \frac{288.8-78.2}{177.61} = 1.2\,m.$$



$$e = \frac{b}{2} - \bar{x} = \frac{3}{2} - 1.2 = 0.3m < \left(\frac{b}{6} = \frac{3}{6} = 0.5m\right)$$

hence safe.

④ Bearing Capacity → Meyerhof (As load is inclined)

$$q_{nu} = q_u - \gamma D_f = cN_c s_c d_c i_c + \gamma D_f N_q s_q d_q i_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma - \gamma D_f$$

$\phi = 20^\circ \rightarrow N_c = 14.8,\ N_q = 6.4,\ N_\gamma = 2.9$

$s_c = 1 + 0.2K_p(B'/L) = 1.$ $\quad [L >> B' \Rightarrow B'/L = 0]$

$\Rightarrow s_c = s_q = s_\gamma = 1$ for strip footing

④

$C = 25\ kN/m^2$, $\gamma = 18\ kN/m^3$,

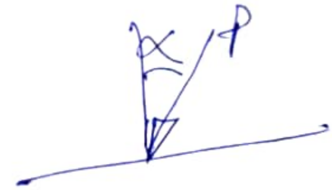
$D_f = 1m$ $\quad B' = B - 2e = 3 - 2\times0.3 = 2.4m$

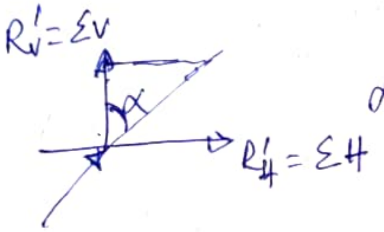
$d_c = 1 + 0.2\tan(45^\circ + \frac{\phi}{2})(\frac{D_f}{B'}) = 1 + 0.2\tan(45^\circ + \frac{20}{2})(\frac{1}{2.4})$

$= 1.12$

$d_q = d_\gamma = 1 + 0.1\tan(45^\circ + \frac{\phi}{2})(\frac{D_f}{B'}) = 1.06$

$i_c = i_q = (1 - \frac{\psi}{90^\circ})^2$ or $(1 - \frac{\alpha}{90^\circ})^2$





$\alpha = \tan^{-1}(\frac{\Sigma H}{\Sigma V}) = \tan^{-1}(\frac{46.8}{177.61}) = 14.8^\circ$

$i_c = i_q = (1 - \frac{14.8^\circ}{90^\circ})^2 = 0.7$

$i_\gamma = (1 - \frac{\alpha^\circ}{\phi^\circ})^2 = (1 - \frac{14.8^\circ}{20^\circ})^2 = 0.07$

$q_{nu} = 25\times14.8\times1\times1.12\times0.7 + 18\times1\times6.4\times1\times1.06\times0.7$

$+ \frac{1}{2}\times18\times2.4\times2.9\times1\times1.06\times0.07 - 1\times18 = 362.2\ kN/m^2$ ✓

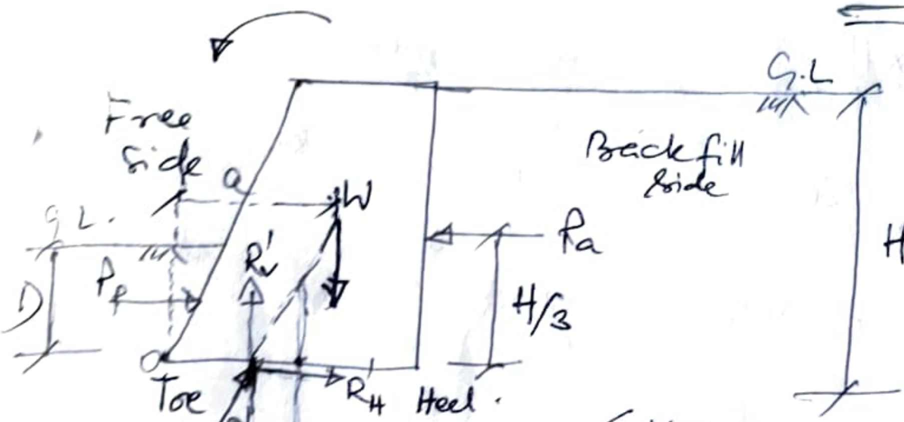
$Q_{nu} = q_{nu}\times(B'\times1) = 362.2\times(2.4\times1) = 869.3\ kN/m$

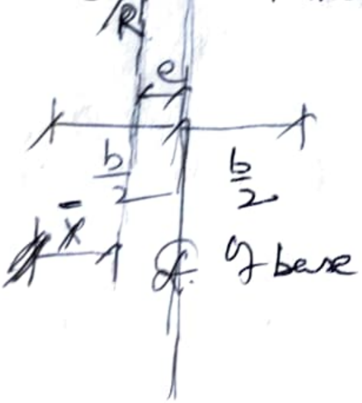
$(F.O.S.)_{bearing} = \frac{Q_{nu}}{\Sigma V} = \frac{869.2}{177.61} = 4.9 > 2.5 - 3$

hence safe

Gravity Retaining Wall

Design steps/ checks





$\Sigma V = 0$

$\Rightarrow R_v' = W$

$\Sigma H = 0 \Rightarrow R_H' = P_a - P_p$ (P_p is neglected as it will give more safety) but certain cases it is considered.

$\Sigma M_{toe} = 0$

$$-R_v' \bar{x} + W \times a - P_a \times \frac{H}{3} = 0$$

$$\Rightarrow \boxed{\bar{x} = \frac{Wa - P_a H/3}{R_v'} = \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}}$$

$$e = \frac{b}{2} - \bar{x} \Rightarrow \boxed{e \ngtr \frac{b}{6}}$$

Checks.

1. No sliding condition.

$$FOS = \frac{R_v' \times \mu}{R_H'} = \frac{\Sigma V \times \mu}{\Sigma H} \geq 1.5 - 2$$

μ = Coefficient of friction between wall & base soil ($= \tan\delta$)

② No. overturning Condition.

$$F.O.S. = \frac{\sum M_R}{\sum M_O} \geq 1.5-2.$$

$$= \frac{Wa}{\left(P_a \times \frac{H}{3}\right)}$$ [In the present case under consideration

$\sum M_R$ = Sum of the resisting moment about the toe

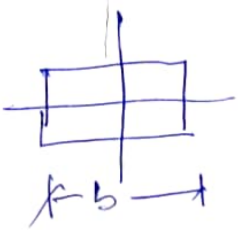
$\sum M_O$ = Sum of the overturning moment about the toe.

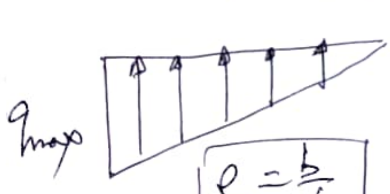
③ No tension condition

(No tension should be developed at the base of the foundation).

$\boxed{e \not> \frac{b}{6}}$

$\boxed{e < \frac{b}{6}}$ ✓ No tension. pr. Distribution at the base.

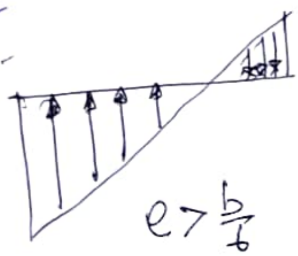




$\boxed{e = \frac{b}{6}}$ ✓ No tension.

$$\frac{W}{A} \pm \frac{W \times e}{I/y}$$

$$\frac{R_v'}{(b \times 1)} + \frac{R_v' \times e}{\frac{1 \times b^3}{12} \times \frac{2}{b}} = \frac{R_v'}{b} + \frac{6 R_v' \times e}{b^2}$$



$e > \frac{b}{6}$ ✗ as tension develops at base & there will be separation between base & soil as soil can't take tension.

④ No bearing failure.

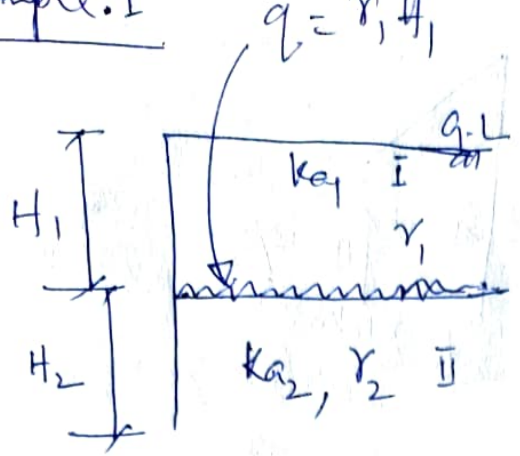
$$\boxed{FOS = \frac{q_{nu}}{q_{max}} \geq 2 \text{ to } 3}$$

$= \frac{Q_{nu}}{\sum V}$ (stress coming from the wall) calculated

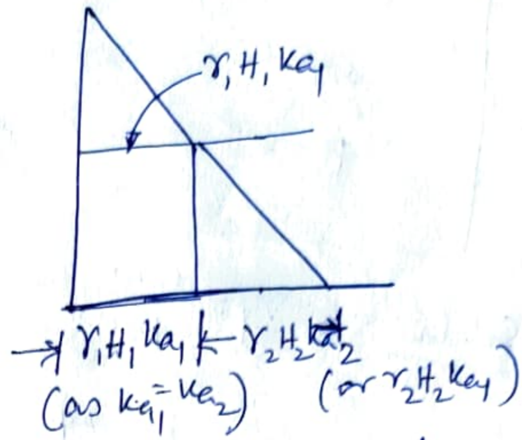
$$q_{max} \text{ or } q_{min} = \frac{R_v'}{b}\left(1 \pm \frac{6e}{b}\right)$$

$$q_{max} = \frac{R_v'}{b}\left(1 + \frac{6e}{b}\right) \ \& \ q_{min} = \frac{R_v'}{b}\left(1 - \frac{6e}{b}\right)$$

Example.1



Case-I
Active Earth Pr. Dist. Diagram

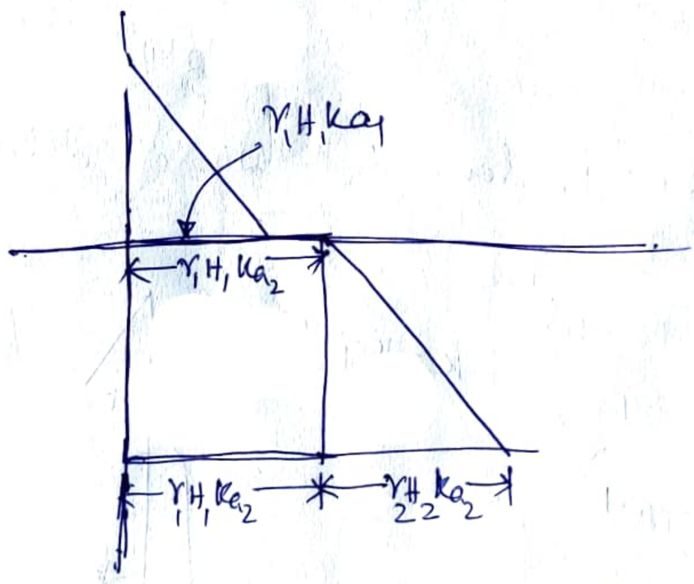


Case – I $\quad ka_1 = ka_2$

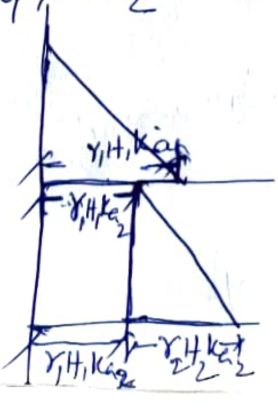
For 2nd layer layer I will act as surcharge (q)

Case-II $\quad ka_1 < ka_2 \Rightarrow ka_2 > ka_1$

$q = \gamma_1 H_1$



Case-III $\quad ka_1 > ka_2 \Rightarrow ka_2 < ka_1 \quad ; \quad q = \gamma_1 H_1$



② **No overturning condition.**

$$F.O.S. = \frac{\Sigma M_R}{\Sigma M_O} \geq 1.5 - 2.$$

$$= \frac{Wa}{\left(R_h \times \frac{H}{3}\right)}$$ [In the present case under consideration.

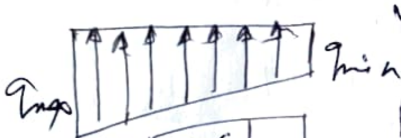
ΣM_R = Sum of the resisting moment about the toe

ΣM_O = Sum of the overturning moment about the toe.

③ **No tension condition**

(No tension should be developed at the base of the foundation).

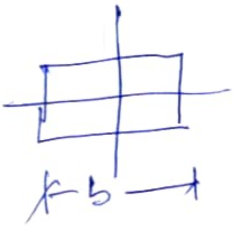
$\boxed{e \not> \frac{b}{6}}$



No tension.

pr. Distribution at the base.

$\boxed{e < \frac{b}{6}}$

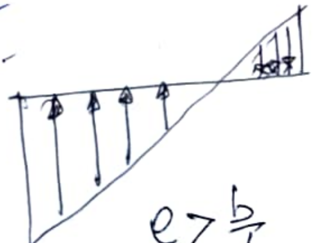


No tension.

$\boxed{e = \frac{b}{6}}$

$\frac{W}{A} \pm \frac{W \times e}{I/y}$

$\frac{R_v'}{(b \times 1)} + \frac{R_v' \times e}{\frac{1 \times b^3}{12} \times \frac{2}{b}} = \frac{R_v'}{b} + \frac{6 R_v' \times e}{b^2}$



✗ as tension develops at base & there will be separation between base & soil as soil can't take tension.

$e > \frac{b}{6}$

④ **No bearing failure.**

$$\boxed{FOS = \frac{q_{nu}}{q_{max}} \geq 2 \text{ to } 3}$$

$q_{nu} = (q_{nu} \ldots)$

$= \frac{Q_{nu}}{\Sigma V}$

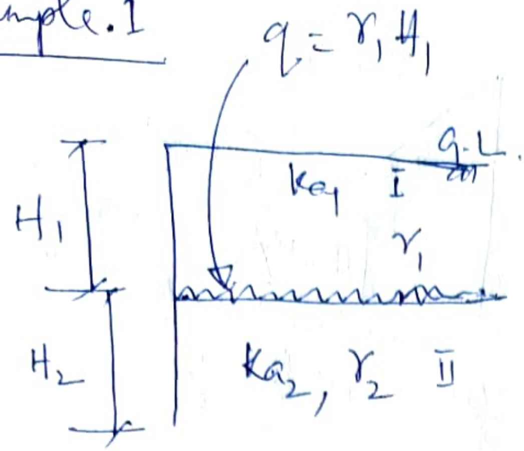
stress coming from the wall

calculated

$$q_{max} \text{ or } q_{min} = \frac{R_v'}{b}\left(1 \pm \frac{6e}{b}\right)$$

$$q_{max} = \frac{R_v'}{b}\left(1 + \frac{6e}{b}\right) \;\&\; q_{min} = \frac{R_v'}{b}\left(1 - \frac{6e}{b}\right)$$

Example.1

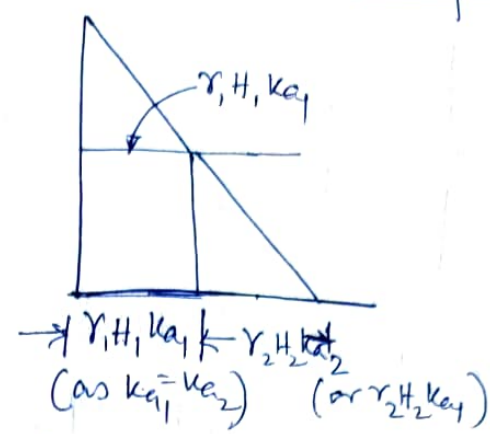


Case-I

Active

Earth Pr. Dist. Diagram

(1)

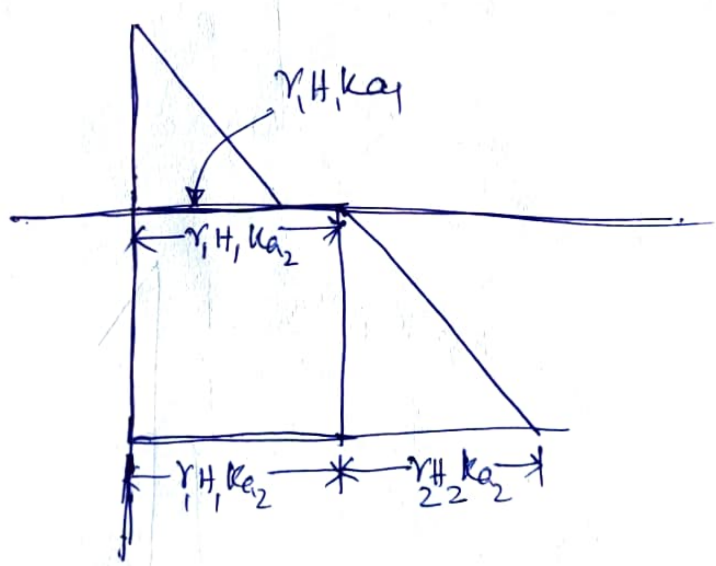


Case – I $\quad ka_1 = ka_2$

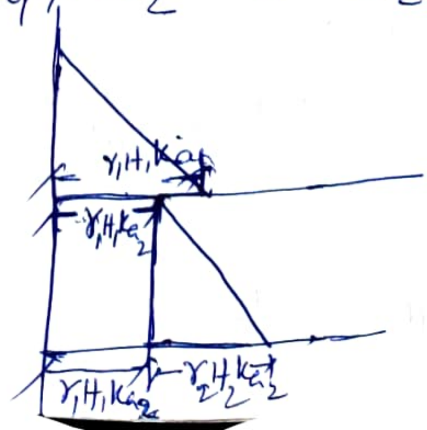
For 2nd layer layer I will act as surcharge (q)

Case-II $\quad ka_1 < ka_2 \Rightarrow ka_2 > ka_1$

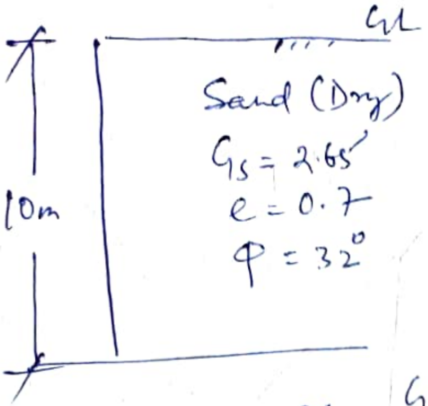
$q = \gamma_1 H_1$

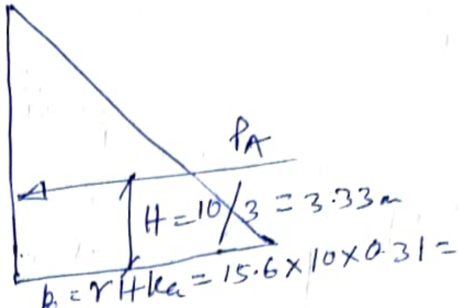


Case-III $\quad ka_1 > ka_2 \Rightarrow ka_2 < ka_1 \; ; \; q = \gamma_1 H_1$



Example - II





Solution : $\gamma_d = \dfrac{G\gamma_w}{1+e} = \dfrac{2.65\times 10}{1+0.7} = 15.6\ kN/m^3$.

Case - I.

$$k_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 32^\circ}{1+\sin 32^\circ} = 0.31$$

Eactive earth pr. at base of the wall

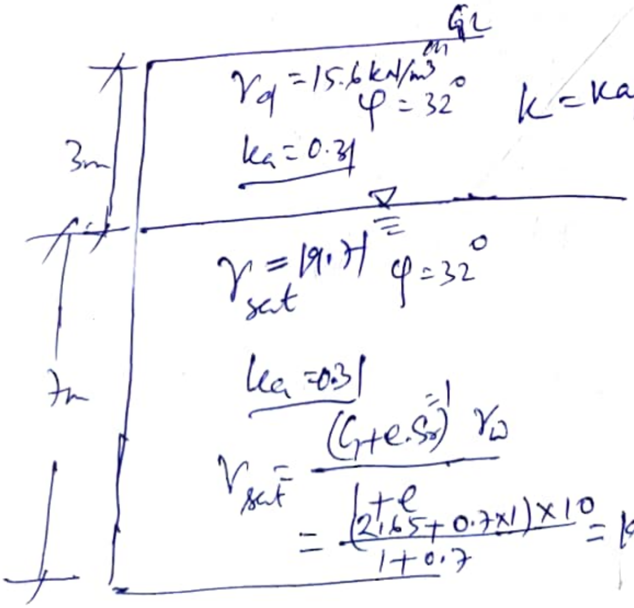
$p_a = \gamma H k_a = 15.6 \times 10 \times 0.31 =$

Total earth pr. $P_A = \frac{1}{2}\gamma H^2 k_a$.

$= \frac{1}{2} \times 15.6 \times 10^2 \times 0.31$

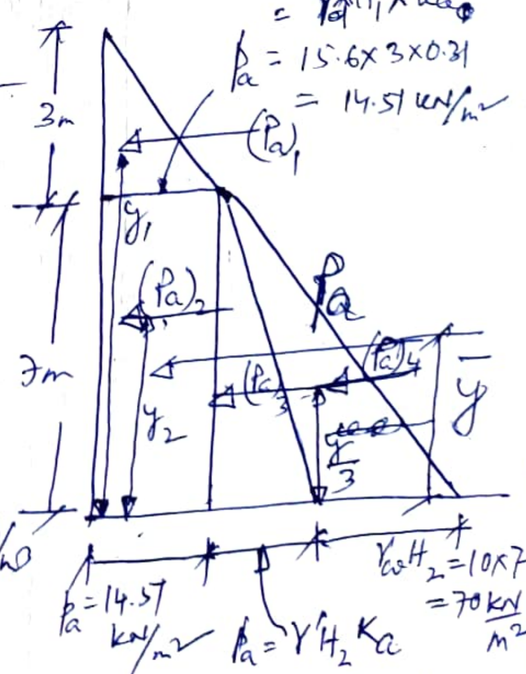
$= 241.8\ kN/m^2$

Case - II. Same Problem with W.T. at a depth 3m below GL.



$k = k_{a_1} = k_{a_2}$

$$\gamma_{sat} = \frac{(G+e.S_r)\gamma_w}{1+e} = \frac{(2.65+0.7\times 1)\times 10}{1+0.7} = 19.71\ kN/m^3$$



$= \gamma_d H_1 \times k_a$

$p_a = 15.6 \times 3 \times 0.31$

$= 14.51\ kN/m^2$

$p_a = \gamma' H_2 K_a$

$= 9.71 \times 7 \times 0.3$

$= 21.1\ kN/m^2$

$\gamma' = \gamma_{sat} - \gamma_w = 19.71 - 10 = 9.71\ kN/m^3$

$$P_a = (P_a)_1 + (P_a)_2 + (P_a)_3 + (P_a)_4$$

$$= \frac{1}{2}\times 14.51\times 3 + 14.51\times 7 + \underbrace{\frac{1}{2}\times 21.1\times 7 + \frac{1}{2}\times 70\times 7}_{318.9}$$

$$= 21.77 + 101.6 + 318.9$$

$$= 442.2\ kN/m.$$

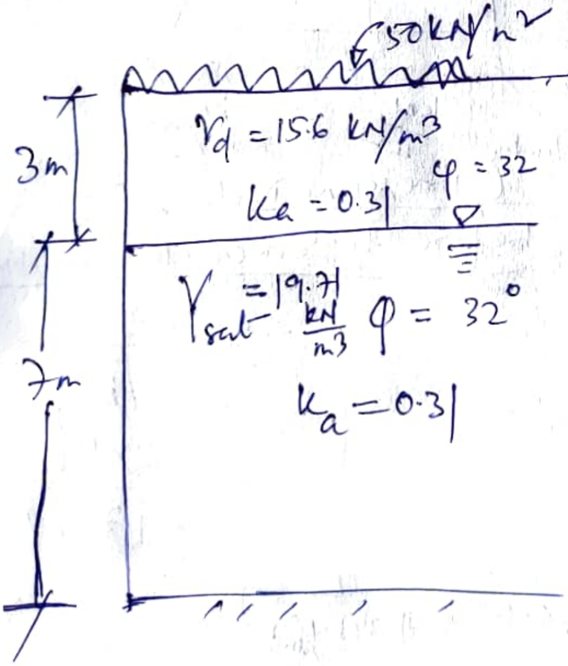
$$P_a \times \bar{y} = (P_a)_1 \times y_1 + (P_a)_2 \times y_2 + \{(P_a)_3 + (P_a)_4\} y_3$$

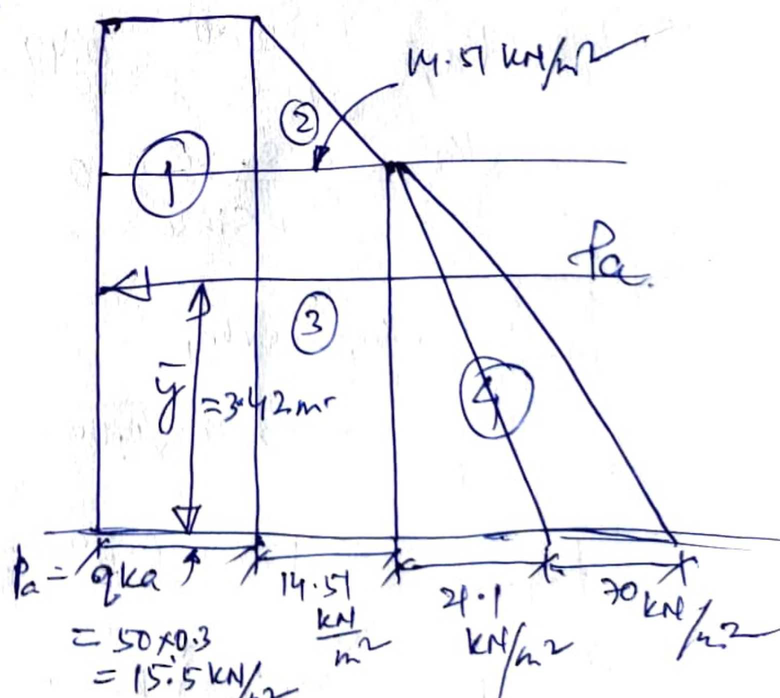
$$= \left[21.77\times\left(7+\frac{3}{3}\right) + 101.6\times\frac{7}{2} + 318.9\times\frac{7}{3}\right]$$

$$\Rightarrow \bar{y} = \frac{\left[21.77\times 8 + 101.6\times 3.5 + 318.9\times\frac{7}{3}\right]}{442.2}$$

$$= 2.88\ m.$$

Case-III Same problem with a surcharge.





$P_a = qK_a$
$= 50\times 0.3$
$= 15.5\ kN/m^2$

$$P_a = 15.5\times 10 + \frac{1}{2}\times 14.51\times 3 + 14.51\times 7 + \frac{1}{2}(21.1+70)\times 7$$

$$= 597.2\ kN/m$$

$$P_a \times \bar{y} = (P_a)_1 \times y_1 + (P_a)_2 \times y_2 + (P_a)_3 \times y_3 + (P_a)_4 \times y_4$$

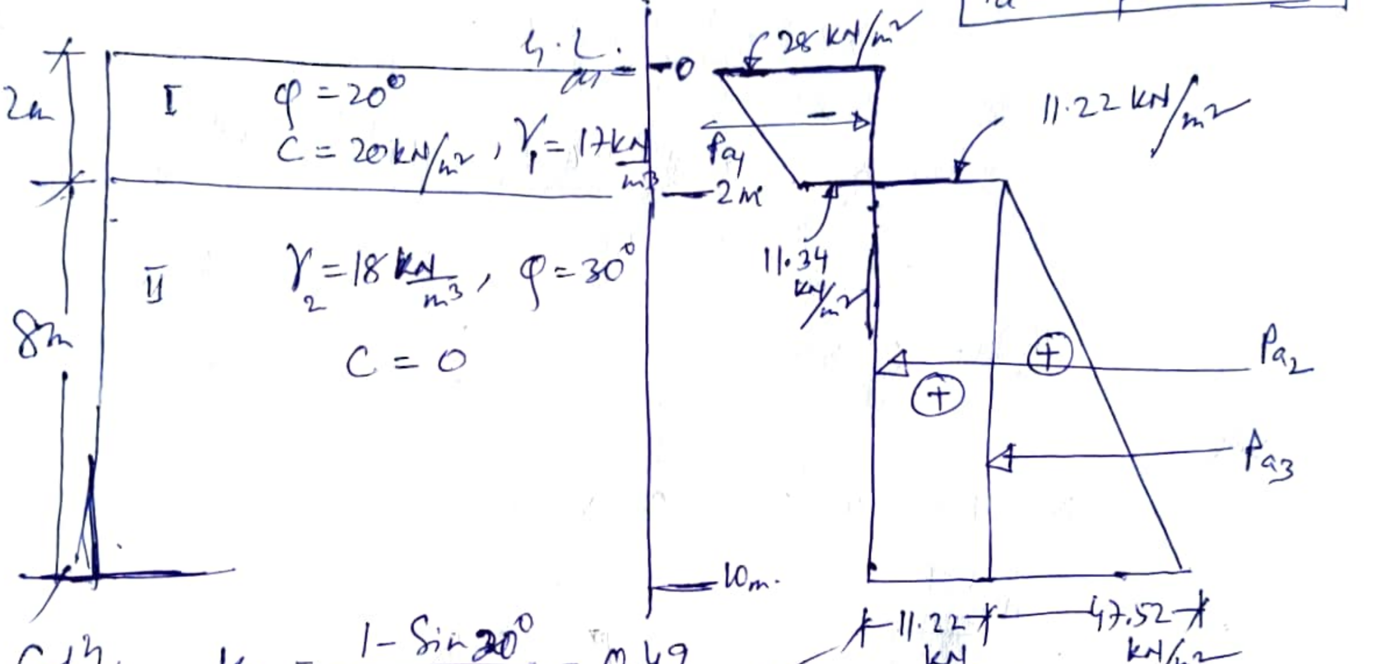
$$\Rightarrow \bar{y} = \frac{(15.5\times 10)\times\frac{10}{2} + \left(\frac{1}{2}\times 14.51\times 3\right)\times\left(\frac{3}{3}+7\right) + (14.51\times 7)\times\frac{7}{2} + 318.9\times\frac{7}{3}}{597.2}$$

$$= \frac{155\times 5 + 21.76\times 8 + 101.57\times 3.5 + 318.9\times 2.33}{597.2}$$

$$= 3.42\,m$$

Example. C-φ soil. Active Case.

$$\boxed{P_a = -P_{a_1} + P_{a_2} + P_{a_3}}$$



Active Earth Pressure Distribution (before tension crack Development)

Soln.

$$K_{a_1} = \frac{1-\sin 20^\circ}{1+\sin 20^\circ} = 0.49$$

$$K_{a_2} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.33$$

Layer-I

$$P_a = \gamma z K_{a_1} - 2c\sqrt{K_{a_1}}$$

$$P_a\big|_{z=0} = 0 - 2\times 20\times\sqrt{0.49} = -28\,kN/m^2$$

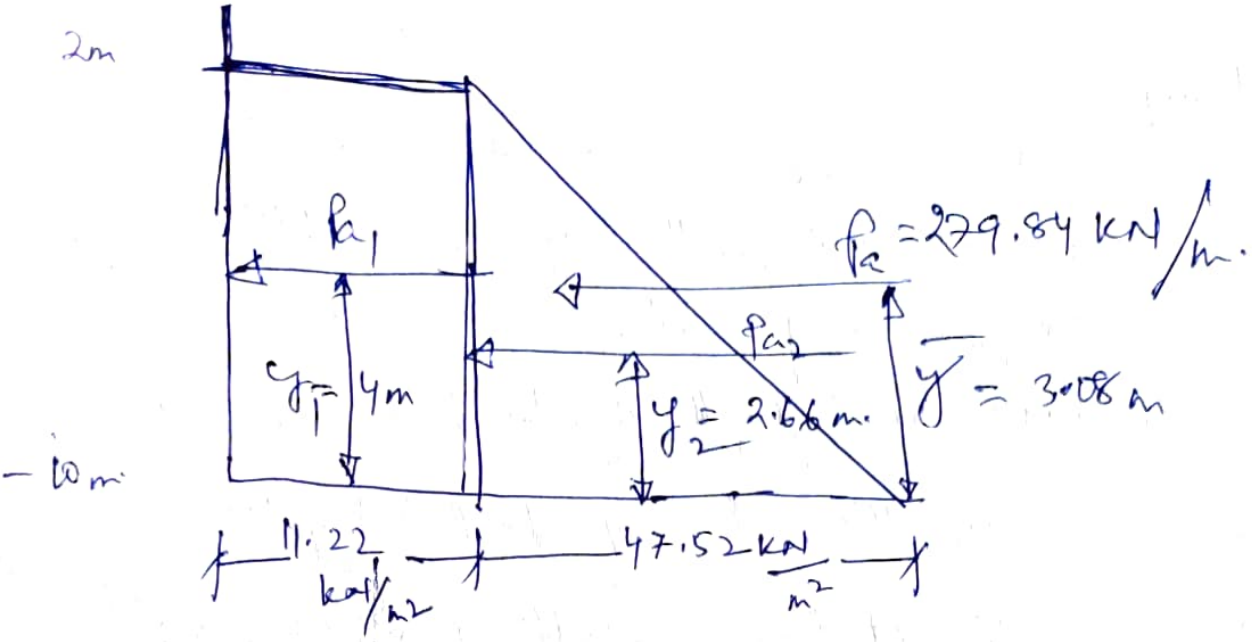
$$P_a\big|_{z=2m} = 17\times 2\times 0.49 - 2\times 20\sqrt{0.49} = -11.34\,kN/m^2$$

$$Z_0 = \frac{2c}{\gamma\sqrt{K_{a_1}}} = \frac{2\times 20}{17\times\sqrt{0.49}} = 3.36\,m.$$

Layer-II

$$P_a\big|_{z=2m} = \overset{q}{(\gamma_1 z)}\times K_{a_2} = 17\times 2\times 0.33 = 11.22\,kN/m^2$$

$$P_a\big|_{z=10m} = 11.22 + \gamma_2 H_2 K_{a_2} = 11.22 + 18\times 8\times 0.33 = 58.74\,\frac{kN}{m^2}$$



(Active Earth Pressure Distribution after tension crack Development)

~~$P_{a_1} = 15.5 \times 10 = 155$ kN/m , $y_1 = \frac{10}{2} = 5$m.~~

~~$P_{a_2} = \frac{1}{2} \times 14.51 \times 3 = 21.76$ kN/m, $y_2 = \frac{3}{3}$~~

$P_{a_1} = 11.22 \times 8 = 89.76$ kN/m , $y_1 = \frac{8}{2} = 4$m.

$P_{a_2} = \frac{1}{2} \times 47.52 \times 8 = 190.08$ kN/m , $y_2 = \frac{8}{3} = 2.66$m

$P_a = P_{a_1} + P_{a_2} = 89.76 + 190.08 = 279.84$ kN/m

$$\bar{y} = \frac{89.76 \times 4 + 190.08 \times 2.66}{279.84}$$

$= 3.08$m.

Example

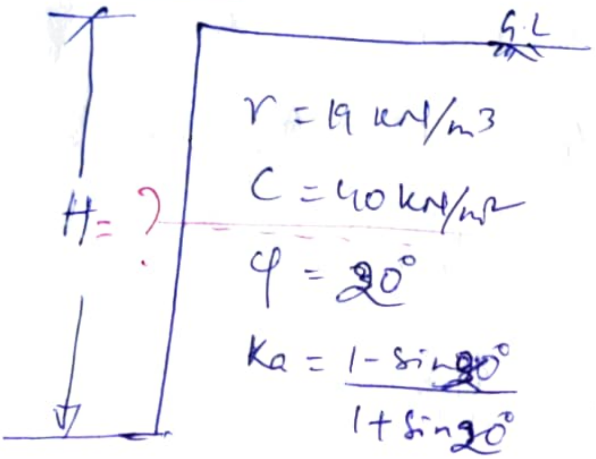
An unsupported excavation is to be m in a clay layer with unit weight $19\ kN/m^3$ & $C = 40\ kN/m^2$ & $\varphi = 20°$.

a) Calculate the depth of tension cracks

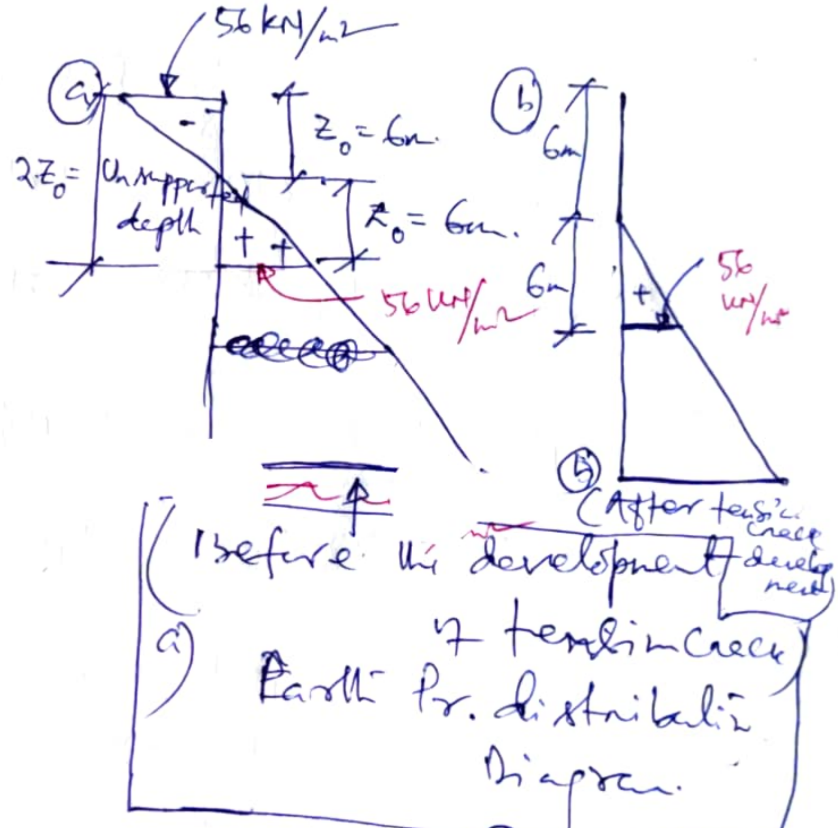
b) Calculate the unsupported depth

c) Draw the Active earth pressure diagram for the above.

Solution :



$K_a = \frac{1-\sin 20°}{1+\sin 20°} = 0.49$



a) C-φ Soil

$P_a = \gamma z K_a - 2c\sqrt{K_a}$

at $z = 0$, $P_a = -2c\sqrt{K_a}$

$= -2 \times 40 \sqrt{0.49}$

$= -56\ kN/m^2$

$P_a = 0 \Rightarrow z = z_0$ = depth of tension crack

$\gamma z_0 K_a - 2c\sqrt{K_a} = 0$

or $z_0 = \frac{2c}{\gamma\sqrt{K_a}} = \frac{2 \times 40}{19 \times \sqrt{0.49}} = 6m$ Ans

(b) at $2z_0 = 12m$

$P_a = \gamma z K_a - 2c\sqrt{K_a}$

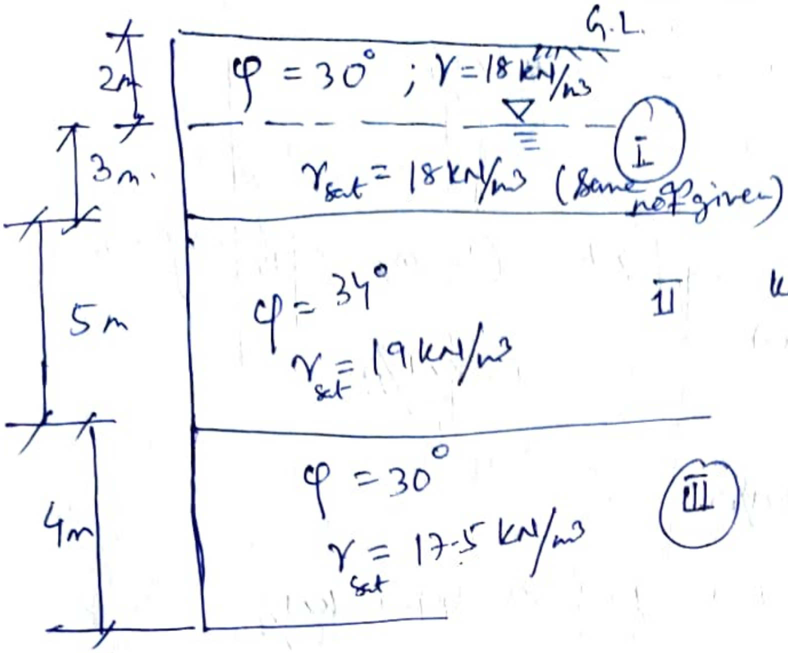
$= 19 \times 12 \times 0.49 - 2 \times 40\sqrt{0.49}$

$= +56\ kN/m^2$

Unsupported depth $= 2Z_0 = 6 \times 2 = 12m$ Ans

IN) Z=10m

Cohesionless Soil. (3 layered backfill)



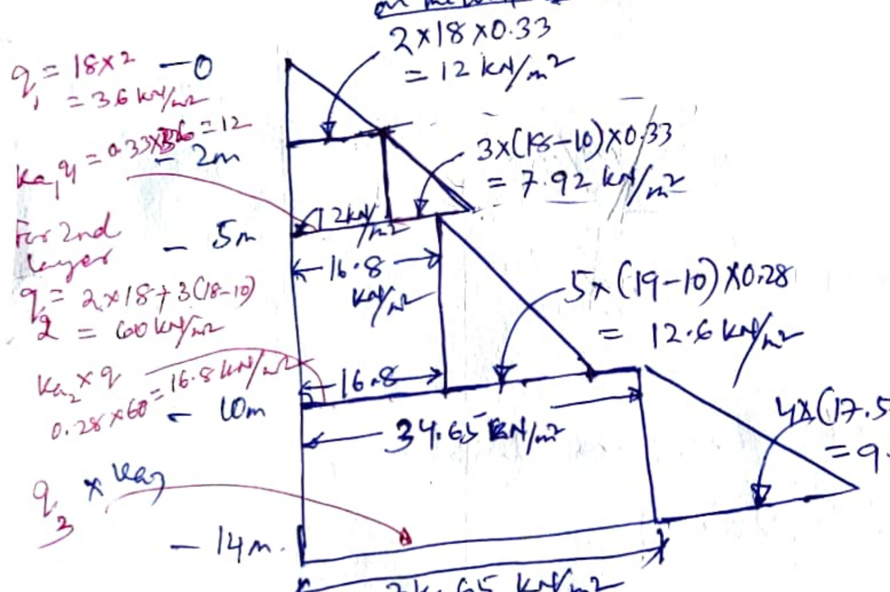
$k_{a_1} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.33$

$k_{a_2} = \frac{1-\sin 34^\circ}{1+\sin 34^\circ} = 0.28$

$k_{a_3} = 0.33$

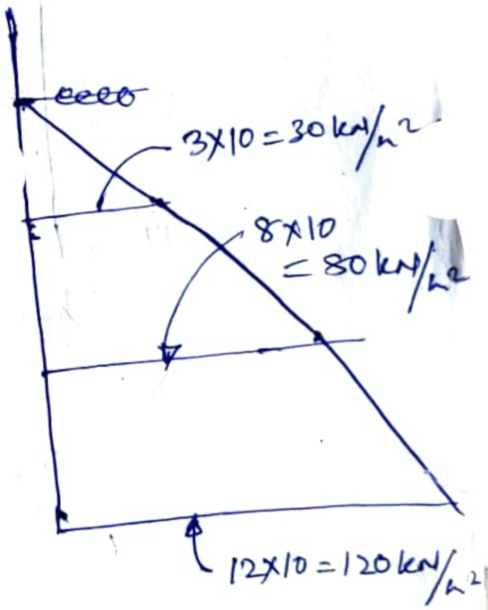
Solution.

Earth Pressure Dist. on the wall Due to soil



+

Water Pressure Dist. on the wall



(I) First layer will be surcharge for the layer II.

q = ~~Effective~~ Surcharge = Effective over burden pressure

$= [2 \times 18 + 3 \times (18-10)] = 36 + 24 = 60$

$p = q k_{a_2} = 60 \times 0.28 = 16.8\ kN/m^2$ = uniform pressure due to surcharge in 2nd layer

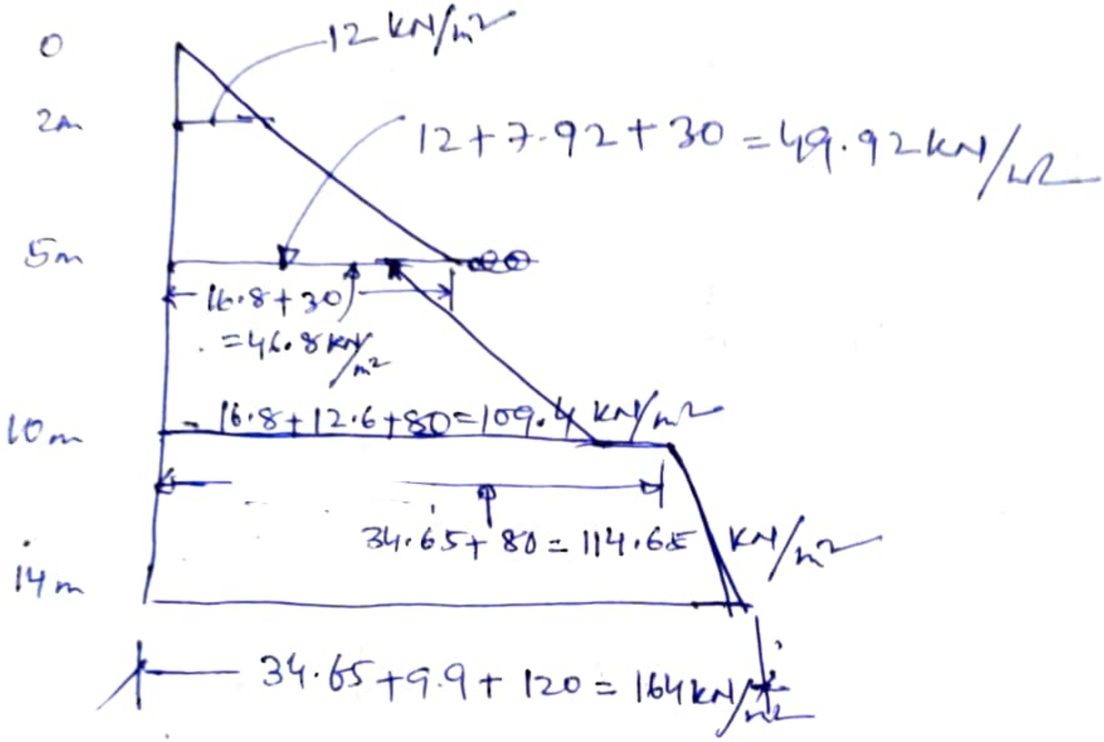
$p_a \big|_{z=5m\ (1st\ layer)} = 12 + 7.92 = 19.92\ kN/m^2$

$$p_a\big|_{z=5m\,(2nd\ layer)} = qk_{a_2} = 16.8\ kN/m^2$$

$$p_a\big|_{z=10m\,(2nd\ layer)} = 16.8 + 12.6 = 29.4\ kN/m^2$$

$$p_a\big|_{z=10m\,(for\ 3rd\ layer)} = qk_{a_3} = \left[2\times 18 + 3(18-10) + 5(19-10)\right]\times 0.33$$

$$= 34.65\ kN/m^2$$



(Total Earth Pressure Dist. Diagram)