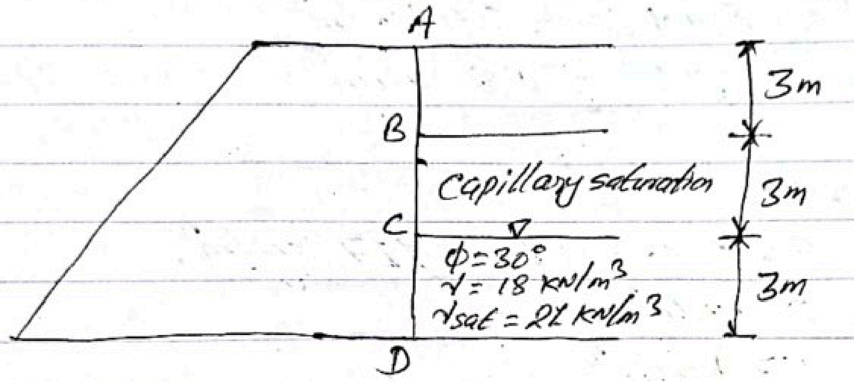
Final Assessment 2079

Numerical Solution

5HZVA

1.(b) Solution

Calculate the active force due to the following backfill at the back of the retaining wall.



Solution,

Active earth pressure coefficient (Ka) $= \dfrac{1-\sin\phi}{1+\sin\phi}$

$$= \frac{1-\sin 30°}{1+\sin 30°}$$

$$= 1/3.$$

At level A

pore water pressure $(u) = 0\ kN/m^2$

Total stress $(\sigma) = 0\ kN/m^2$.

$\therefore$ Effective stress $(\sigma_v') = \sigma - u = 0\ kN/m^2$.

$\therefore$ Active earth pressure $(Pa) = Ka\,\sigma_v' - 2c\sqrt{Ka}$

$$= \tfrac{1}{3}\times 0 - 2\times 0\sqrt{0}$$

$$= 0\ kN/m^2$$

At level B (Just above)

$\sigma = 3\times 18 = 54\ kN/m^2$

$u = 0\ kN/m^2$.

$\sigma_v' = 54\ kN/m^2$.

$Pa = \tfrac{1}{3}\times 54 = 18\ kN/m^2$.

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At level B (Just below)

~~$\sigma = 3\times18 = 54\ kN/m^2$ γ_{sat}~~

$\sigma = 3\times18 = 54\ kN/m^2$

$U = -10\times3 = -30\ kN/m^2$

$\sigma_v' = 54+30 = 84\ kN/m^2$

$P_a = \frac{1}{3}\times84 = 28\ kN/m^2$

At level C

$\sigma = 3\times18 + 3\times21 = 117\ kN/m^2$

$U = 0\ kN/m^2$

$\sigma_v' = 117\ kN/m^2$

$P_a = \frac{1}{3}\times117 = 39\ kN/m^2$

At level D

$\sigma = 3\times18 + 6\times21 = 180\ kN/m^2$

$U = 10\times3 = 30\ kN/m^2$

$\sigma_v' = 150\ kN/m^2$

$P_a = \frac{1}{3}\times150 = 50\ kN/m^2$

Pressure distribution diagram:

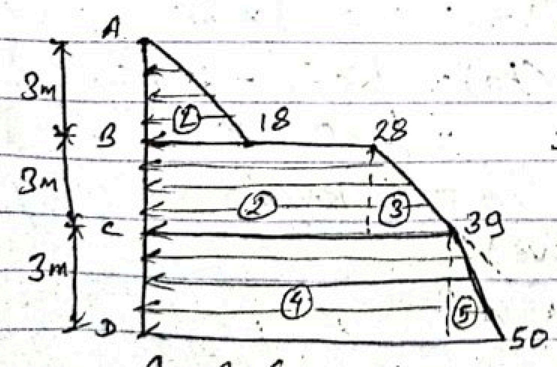
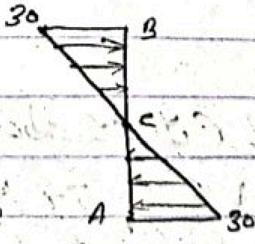
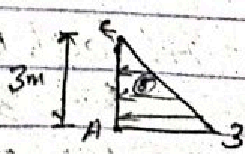


fig: Active earth pressure



Pore water pressure



Pore water pressure neglecting tension zone.

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S.N.	Description	Force (KN/m)	Lever arm (m)	moment
1.	$\frac{1}{2}\times 18\times 3$	27	7	189
2.	28×3	84	4.5	378
3.	$\frac{1}{2}\times 11\times 3$	16.5	4	66
4.	39×3	117	1.5	175.5
5.	$\frac{1}{2}\times 11\times 3$	16.5	1	16.5
6.	$\frac{1}{2}\times 30\times 3$	45	1	45
	Total	306		870

$\therefore$ Total active force = 306 KN/m

Point of application $= \dfrac{870}{306} = 2.84$ m above base.

Ans//

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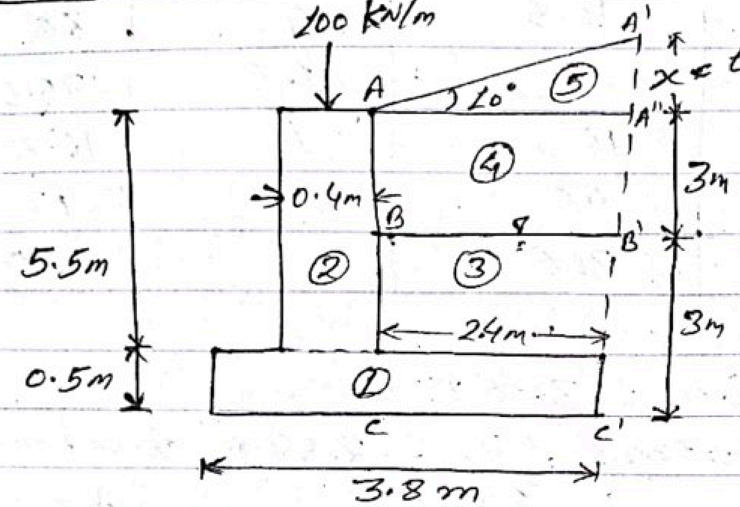
2(a) solution

Check the stability of the retaining wall shown below:



$x = \tan 10° \times 2.4 = 0.42$ m

$\gamma_{soil} = 17\ kN/m^3$

$\phi = 36°$

$\gamma_{wall} = 23.5\ kN/m^3$

Solution;

Here, back fill is inclined

$\therefore$ Active earth pressure coefficient $(K_a) = \cos i \times \dfrac{\cos i - \sqrt{\cos^2 i - \cos^2\phi}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}}$

$$= \cos 10° \times \frac{\cos 10° - \sqrt{\cos^2 10° - \cos^2 36°}}{\cos 10° + \sqrt{\cos^2 10° - \cos^2 36°}}$$

$$= 0.269$$

At level A-A''

Total stress $(\sigma) = 0\ kN/m^2$

Pore water pressure $(U) = 0\ kN/m^2$

Effective stress $(\sigma_v') = 0\ kN/m^2$.

Active earth pressure $(P_a) = K_a \times \sigma_v' - 2c\sqrt{K_a}$

$= 0.269 \times 0 - 2 \times 0$

$= 0\ kN/m^2$.

At level B-B'

$\sigma = 17 \times 3 = 51\ kN/m^2$

$U = 0\ kN/m^2$.

$\sigma_v' = 51\ kN/m^2$

$\therefore P_a = 0.269 \times 51 = 13.719 \ kN/m^2$

At level c-c'

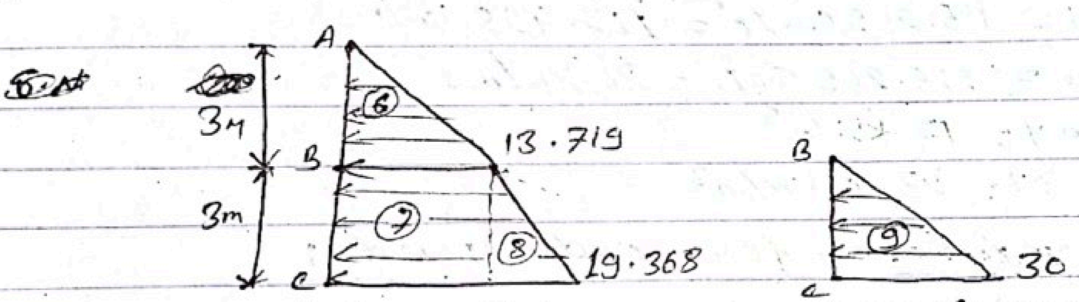
$\sigma = 17 \times (3+3) = 102 \ kN/m^2$

$U = 10 \times 3 = 30 \ kN/m^2$

$\sigma_v' = 102 - 30 = 72 \ kN/m^2$

$\therefore P_a = 0.269 \times 72 = 19.368 \ kN/m^2$

Pressure distribution diagram:



Active earth pressure

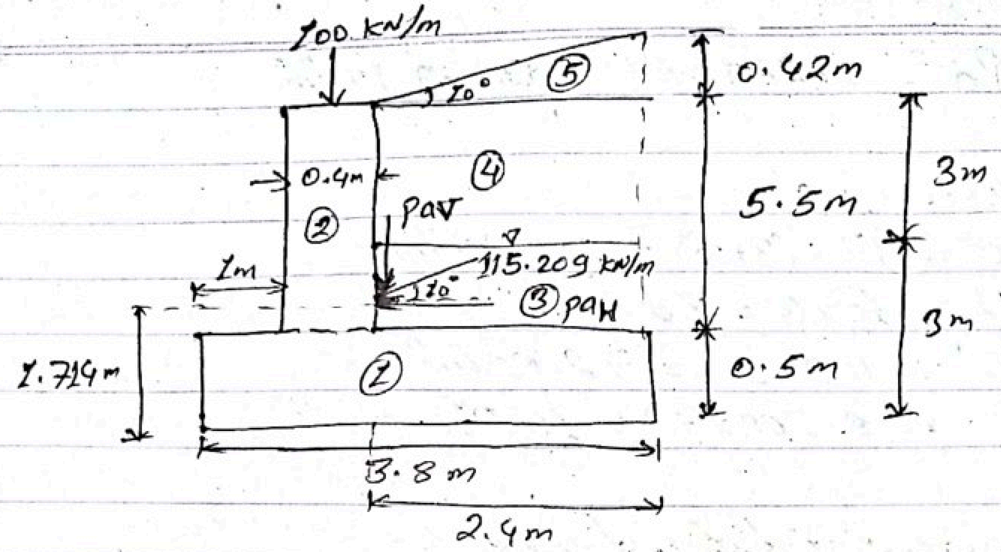
pore water pressure

Computation of total active force:

S.N.	Description	Force (kN/m)	Lever arm (m)	Moment
6	$\frac{1}{2} \times 13.719 \times 3$	20.578	4	82.312
7	13.719×3	41.157	1.5	61.735
8	$\frac{1}{2} \times 5.649 \times 3$	8.474	1	8.474
9	$\frac{1}{2} \times 30 \times 3$	45	1	45
	Total	115.209		197.521

$\therefore$ Total active force = 115.209 kN/m acting parallel to inclined backfill.

Point of application $= \dfrac{197.521}{115.209} = 1.714$ m above base.



$Pa_H = 115.209 \cos 20° = 113.458$ kN/m

$Pa_V = 115.209 \sin 20° = 20$ kN/m.

$\gamma_{soil} = 17$ kN/m^3

$\gamma_{wall} = 23.5$ kN/m^3

Computation of forces and moments:

S. NO.	Description	Forces (kN/m)		Lever arm (m)	moment (kN-m/m)	
		Vertical	Horizontal		Clockwise	Anticlockwise
1	0.5×3.8×23.5	44.65		1.9	84.835	
2	0.4×5.5×23.5	51.7		1.2	62.04	
3	2.4×2.5×17	102		2.6	265.2	
4	2.4×3×17	122.4		2.6	318.24	
5	½×2.4×0.42×17	8.568		3	25.704	
6.	PaV	20		1.4	28	
7.	PaH		113.458	1.714		194.467
8.	100	100		1.2	120	~~194.467~~
	Total	449.318	113.458		904.019	194.467

Stability check:

Factor of safety against overturning,

$$F_O = \frac{\Sigma M_r}{\Sigma M_O} = \frac{904.019}{194.467} = 4.65 > 2 \quad (\text{safe})$$

Factor of safety against sliding;

$$\delta = \frac{2}{3}\phi = \frac{2}{3} \times 36^\circ = 24^\circ$$

$$\mu = \tan\delta$$

$$\therefore F_S = \frac{\Sigma V}{\Sigma H} \times \mu = \frac{\Sigma V}{\Sigma H} \times \tan\delta$$

$$= \frac{449.318}{113.458} \times \tan 24^\circ$$

$$= 1.763 > 1.5 \quad (\text{safe})$$

Hence, the retaining wall is stable against sliding and overturning.

Ans//

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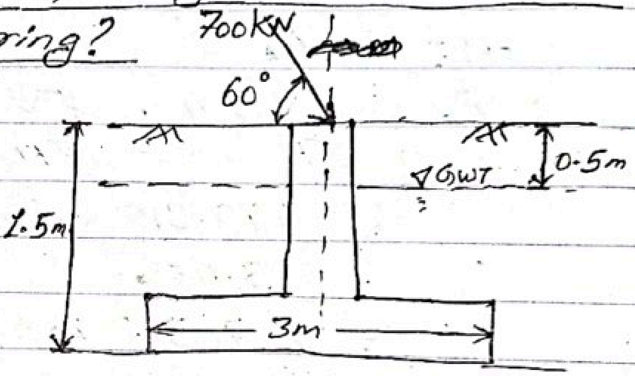
3(b) Solution

For the rectangular foundation (2m × 3m) shown below:

(i) Compute the net allowable bearing capacity (FS=3)

(ii) If the water table is lowered by 2m. What effect on bearing capacity would occur due to the water lowering?



$\gamma_d = 18\ kN/m^3$

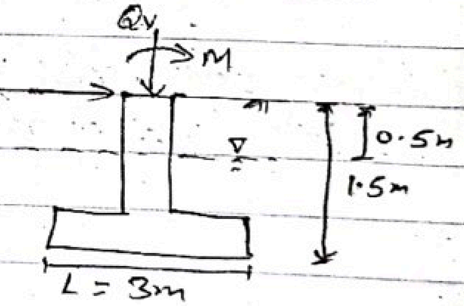
$\phi = 25°$

$c = 0$

$\gamma_{sat} = 21\ kN/m^3$

Solution;

Here;



Dimension $(B \times L) = 2m \times 3m$

$D_f = 1.5m$

$Z_{w_1} = 0.5m$

$Z_{w_2} = 0$

∴ Correction factor; CR

$$R_{w_1} = 0.5\left(1 + \frac{Z_{w_1}}{D_f}\right) = 0.5\left(1 + \frac{0.5}{1.5}\right) = 2/3$$

$$R_{w_2} = 0.5\left(1 + \frac{Z_{w_2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

The inclined load is divided into two component,

Vertical component = $700 \sin 60° = 606.2$ kN

Horizontal component = $700 \cos 60° = 350$ kN.

The horizontal component will exert moment on the foundation in the direction as shown in figure.

$\therefore$ Moment $(M) = 350 \times 1.5 = 525$ kN-m.

$\therefore$ Eccentricity $(e) = \dfrac{\text{Overall moment}}{\text{Vertical load}}$

$= \dfrac{525}{606.2} = 0.866$ m.

Here, foundation is eccentrically loaded in the direction of $(L = 3m)$;

$\therefore$ Change in dimension of foundation;

$B' = B = 2m$

$L' = L - 2 \times e = 3 - 2 \times 0.866 = 1.268$ m

Now,

$N\phi = \tan^2(\pi/4 + \phi/2)$

$= \tan^2(45 + 25/2) = 2.463$

The general bearing capacity equation by Meyerof is given as;

$Q_{ult} = CN_c s_c d_c i_c + \gamma' D_f N_q s_q d_q i_q R_{w_1} + 0.5 B \gamma' N_\gamma s_\gamma d_\gamma i_\gamma R_{w_2}$

for $c = 0$;

$Q_{ult} = \gamma' D_f N_q s_q d_q i_q R_{w_1} + 0.5 B \gamma' N_\gamma s_\gamma d_\gamma i_\gamma R_{w_2}$

Now,

Bearing capacity factor;

$N_q = e^{\pi \tan\phi} N\phi = e^{\pi \tan 25^\circ} \times 2.463$

$= 10.65$

$N_\gamma = (N_q - 1)\tan(1.4\phi)$

$= (10.65 - 1)\tan(1.4 \times 25) = 6.75$

Depth factor;

$d_q = 1 + 0.1\sqrt{N\phi}\,[D_f/B']$

$= 1 + 0.1\sqrt{2.463}\,[1.5/2]$

$= 1.117$

$d_\gamma = d_q = 1.117$ $(\because \phi > 10^\circ)$

__Shape factor__

$S_q = S_\gamma = 1 + 0.1 \times N_\phi \left[\frac{B'}{L'}\right]$

$= 1 + 0.1 \times 2.463 \left[\frac{2}{1.268}\right]$

$= 1.388$

__Inclination factor__

$i_q = i_c = \left[1 - \frac{\alpha}{90}\right]^2 ; \left[\because \alpha = 90 - 60^\circ = 30^\circ\right]$

$= \left[1 - \frac{30}{90}\right]^2 = \frac{4}{9} = 0.44$

$i_\gamma = \left[1 - \frac{\alpha}{\phi}\right]^2 = \left[1 - \frac{30}{25}\right]^2 = 0.04$

Now,

$Q_{ult} = \gamma_{avg} \times D_f \times N_q \times S_q \times d_q \times i_q \times R_{w_1} + 0.5 \times B \times \gamma_{sat} \times N_\gamma \times d_\gamma \times S_\gamma \times i_\gamma \times R_{w_2}$

$= \left[\frac{(0.5 \times 18) + (1 \times 21)}{1.5}\right] \times 1.5 \times 10.65 \times 1.388 \times 1.117 \times \frac{2}{3} \times 0.44 +$

$0.5 \times 2 \times 21 \times 6.75 \times 1.117 \times 1.388 \times 0.04 \times 0.5$

$= 145.303 + 4.395$

$= 149.698 \ kN/m^2$

$Q_{net} = Q_{ult} - \gamma_{avg} \times D_f$

$= 149.698 - 20 \times 1.5$

$= 119.698 \ kN/m^2$

$Q_{ns} = \frac{Q_{net}}{F} = \frac{119.698}{3} = 39.89 \ kN/m^2$

(i) $\therefore$ __Net allowable bearing capacity (Q_{ns}) = 39.89 kN/m².__

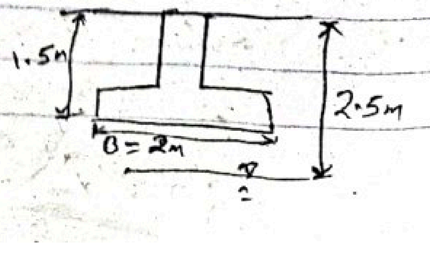
(ii) __If the water table is lowered by 2m;__

$Z_{w_1} = 1.5 m$

$Z_{w_2} = 1 m$

$\therefore R_{w_1} = 1$

$\therefore R_{w_2} = 0.5\left(1 + \frac{1}{2}\right) = 0.75$



$\therefore \gamma_{avg} = \left(\frac{1\times 18 + 1\times 21}{2}\right) = 19.5\ kN/m^3$

$\therefore Q_{ult} = \gamma_d \times D_f \times N_q \times S_q \times d_q \times i_q \times R_{w_1} + 0.5 B \gamma_{avg} N_\gamma S_\gamma i_\gamma R_{w_2}$

$= 18\times 1.5\times 10.65\times 1.388\times 1.117\times 0.44\times 1 +$

$0.5\times 2\times 19.5\times 6.75\times 1.117\times 1.388\times 0.04\times 0.75$

$= 196.159 + 6.122$

$= 202.281\ kN/m^2$

Hence, if the water table is lowered by 2m, the bearing capacity will increases due to water lowering. Ans.

4(b) solution

Q.) A group of end bearing piles of 50 cm diameter is embedded in soil and is subjected to net load of 5000 kN. A soil profile consist of a surface layer of sand 3m thick ($\gamma = 16\ kN/m^3$), the 2nd layer of 3m dense sand ($\gamma = 19\ kN/m^3$), 3rd layer is of 10m thick medium dense clay ($\gamma_{sat} = 20\ kN/m^3$, $C_c = 0.25$, $e_0 = 0.75$) and the 4th layer consists of dense clay 10m thick ($\gamma_{sat} = 21\ kN/m^3$, $C_c = 0.2$, $e_0 = 0.65$). Total number of 9 piles of 12m are arranged at spacing of 0.5m in square grid. Estimate the consolidation settlement.

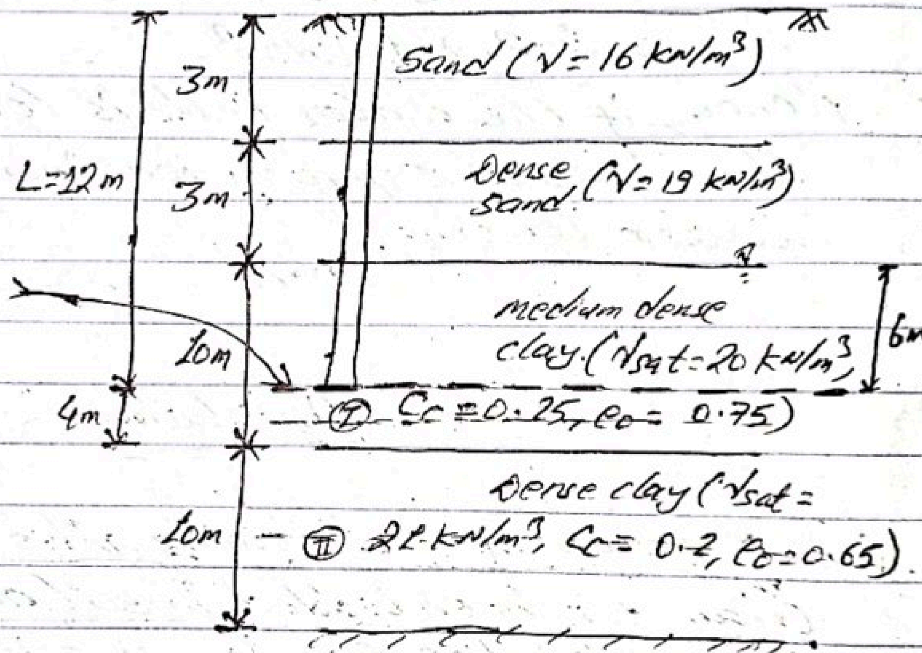
Solution;

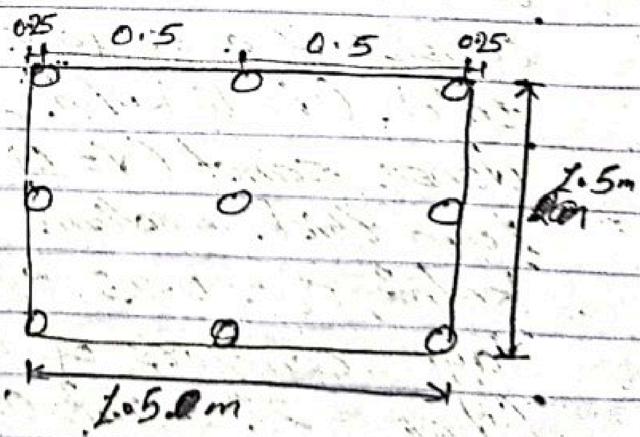
Here; dia. of pile = 50 cm = 0.5 m

Length of a pile(L) = 12 m

Spacing = 0.5 m

Net load = 5000 kN.





The size of the footing is ~~2×2 m~~ 1.5×1.5 m

The piles are end bearing piles. So, the 5000 kN load act at the level of 12 m from the surface. i.e. fictitious footing is at depth 12 m from surface and spreads out at a 2:1 slope.

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Now, two layers are assumed to contribute to the settlement of the foundation.

Layers are:

Layer-I: From 12m to 16m below ground level = 4m thick.

Layer-II: From 16m to 26m = 10 m thick.

The total settlement is computed using the relation;

$$S = \sum \left(\frac{C_c}{1+e_0}\right) \log_{10}\left(\frac{\sigma_0' + \Delta\sigma_v'}{\sigma_0'}\right) * H_i$$

where, C_c = Compression index.

e_0 = Initial void ratio.

σ_0' = ~~Initial~~ Effective overburden stress or pressure at the middle of each layer.

$\Delta\sigma_v'$ = Change in effective stress.

Now,

⇒ Computation of σ_0';

For layer-I;

$\sigma_0' = 3\times16 + 3\times19 + (10-2)\times(20-10) = 185\ kN/m^2$ ($\gamma_{sat} - \gamma_w$)

For layer-II;

$\sigma_0' = 3\times16 + 3\times19 + 10(20-10) + (10-5)\times(21-10)$

$= 260\ kN/m^2$

⇒ Computation of $\Delta\sigma_v'$;

For layer-I:

Area at 2m depth (Z) below fictitious footing.

$= (B+Z)(L+Z) = (B+Z)^2$ [∵ B=L for square]

$= (1.5+2)^2 = 12.25\ m^2$

$$\therefore \Delta\sigma_v' = \frac{Q}{Area} = \frac{5000}{12.25} = 408.16 \text{ kN/m}^2$$

For layer-II;

Area at 9m depth below fictitious footing

$$= (1.5+9)^2 = 110.25 \text{ m}^2$$

$$\therefore \Delta\sigma_v' = \frac{5000}{110.25} = 45.35 \text{ kN/m}^2$$

⇒ Settlement computation;

Layer-I; $S_1 = \frac{C_c}{1+e_0} \log_{10}\left(\frac{\sigma_0' + \Delta\sigma_v'}{\sigma_0'}\right) \times H_1$

$$= \frac{0.25}{1+0.75} \times \log_{10}\left(\frac{185+408.16}{185}\right) \times 4$$

$$= 0.289 \text{ m}$$

Layer-II; $S_2 = \frac{0.2}{1+0.65} \log_{10}\left(\frac{260+45.35}{260}\right) \times 10$

$$= 0.084$$

$\therefore$ Total settlement (S) $= S_1 + S_2$

$$= 0.289 + 0.084$$

$$= 0.373 \text{ m}$$

$$= 37.30 \text{cm}$$

Hence, consolidation settlement = 37.30cm Ans.

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5(b) solution

Q) Determine the group efficiency of a rectangular group of piles with 4 rows and 3 piles per row. If the individual pile capacity is 100 kN, what is the group capacity?

Solution;

Here;

No. of rows = 4 = m

No. of piles per row = 3 = n

∴ Total number of piles = 4×3 = 12 nos.

Individual pile capacity [$Q_{u(s)}$] = 100 kN

Group capacity, $Q_{u(g)}$ = ?

Now, using Converse - Labarre formula;

Group efficiency, $\eta_g = 1 - \theta \times \left\{ \frac{(n-1)\times m + (m-1) n}{90 \times m \times n} \right\}$

$\theta = \tan^{-1}\left(\frac{d}{s}\right)$

Let, the spacing between piles be twice the diameter of pile. i.e. $s = 2d$.

$\therefore \theta = \tan^{-1}\left(\frac{d}{2d}\right) = \tan^{-1}\left(\frac{1}{2}\right) = 26.56°$

$\therefore$ Group efficiency, $\eta_g = 1 - \theta \times \left\{ \frac{(n-1)\times m + (m-1) n}{90\, mn} \right\}$

$= 1 - 26.56 \left\{ \frac{(3-1)\times 4 + (4-1)\times 3}{90\times 4\times 3} \right\}$

$= 58.19\%$

$\therefore$ Group capacity, $Q_{u(g)} = \eta \times [n \times Q_{u(s)}]$

$= 58.19\% \times [12 \times 100]$

$= 698.28$ kN Ans//

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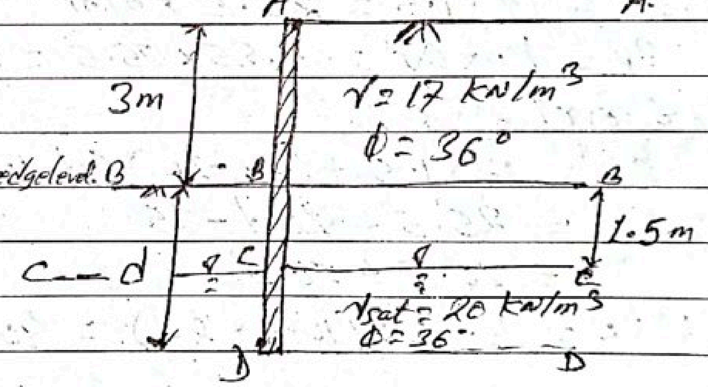
5(a) Solution

(b) The side of excavation 3m. deep in sand are to be supported by cantilever sheet pile walls. The water table is 1.5m below from bottom of excavation. The sand has saturated unit weight of 20 kN/m³ and unit weight of 17 kN/m³ above the water table and $\phi = 36°$. Determine the depth of penetration of pilling below the bottom of excavation to give a factor of safety 2.

Solution;



Assume simplified method.

$K_a = \frac{1-\sin\phi}{1+\sin\phi}$

$= \frac{1-\sin 36°}{1+\sin 36°}$

$= 0.259$

$K_p = \frac{1}{K_a} = 3.85.$

Considering right side of sheet pile;

At level A-A

$\sigma_v' = 0$; $P_a = K_a\sigma_v' = 0\ kN/m^2$.

At level C-C

$\sigma_v' = 17 \times 4.5 = 76.5\ kN/m^2$

$P_a = 0.259 \times 76.5 = 19.814\ kN/m^2$

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At level D-D

$\sigma_v' = 17 \times 4.5 + (20-10) \times (d-1.5)$

$= 76.5 + 10d - 15$

$= 61.5 + 10d$

$\therefore P_a = 0.259 \times (61.5 + 10d)$

$= 15.928 + 2.59d.$

Considering left side of sheet pile;

At level B-B

$\sigma_v' = 0 \;;\; P_p = 0 \text{ kN/m}^2$

At level C-C;

$\sigma_v' = 17 \times 1.5 = 25.5 \text{ kN/m}^2.$

$P_p = k_p \times \sigma_v' = 3.85 \times 25.5 = 98.175 \text{ kN/m}^2.$

At level D-D;

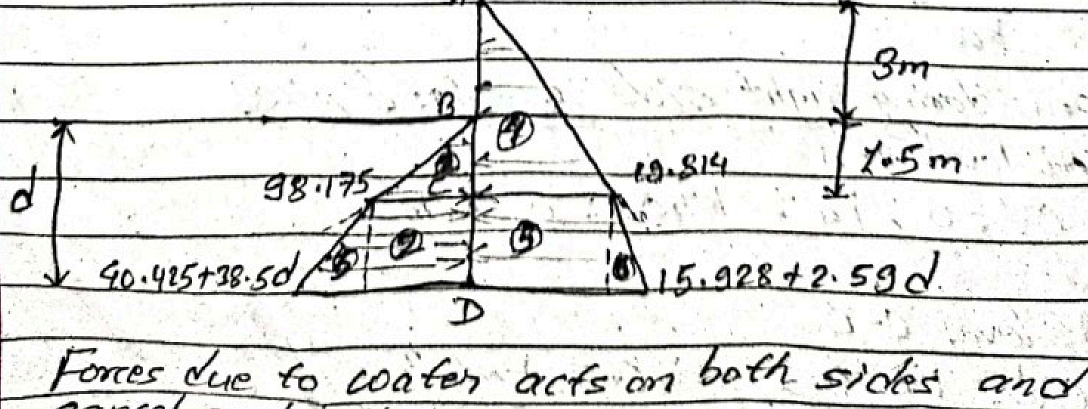
$\sigma_v' = 17 \times 1.5 + (20-10)(d-1.5)$

$= 25.5 + 10d - 15$

$= 10.5 + 10d.$

$\therefore P_p = 3.85 \times (10.5 + 10d) = 40.425 + 38.5d.$

Pressure distribution diagram



Forces due to water acts on both sides and cancel each other.

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Taking moment about base i.e. (A) $\Sigma M_{0} = 0$

$$\frac{1}{2}\times 98.175\times 1.5\times\left[d - \frac{2\times 1.5}{3}\right] + 98.175\times(d-1.5)\times\left(\frac{d-1.5}{2}\right)$$

$$+ \frac{1}{2}\times[40.425 + 38.5d - 98.175]\times(d-1.5)\times\left(\frac{d-1.5}{3}\right)$$

$$- \frac{1}{2}\times 19.814\times 4.5\times\left(d - 1.5 + \frac{4.5}{3}\right) - 19.814\times$$

$$(d-1.5)\times\left(\frac{d-1.5}{2}\right) - \frac{1}{2}\times(15.928 + 2.59d - 19.814)$$

$$\times(d-1.5)\times\left(\frac{d-1.5}{3}\right) = 0$$

or, $73.631(d-1) + 49.087\times(d-1.5)^2 + \frac{1}{6}\times($

$38.5d - 57.75)(d-1.5)^2 - 44.582 d -$

$9.907(d-1.5)^2 - \frac{1}{6}\times(2.59d - 3.886)(d-1.5)^2 = 0$

or, $73.631d - 73.631 + 49.087d^2 - 147.261d + 110.445$
$+ 6.417d^3 - 28.875d^2 + 43.313d - 21.656 - 44.582d$
$- 9.907d^2 + 29.721d - 22.291 - 0.432d^3 + 1.943d^2$
$- 2.914d + 1.457 = 0$

or, $5.985d^3 + 12.248d^2 - 48.092d - 5.676 = 0$

On solving;

Minimum depth of penetration 'd' $= 2.065$ m

$\therefore$ Actual depth of penetration 'D' $= 1.4d = 2.891$ m

say, $D = 3$ m; this provides a FOS of about 2.

Hence, the depth of penetration of pilling below the bottom of excavation to give a factor of safety 2 is 3m. Ans.