CT QUESTION

Date:

Dr Eqramul Haque

CT 1

(1) A 4m strip footing on a sand surface has a ultimate bearing capacity of 5250 lb/ft². What would be the net allowable bearing pressure for a 10×10 m square footing on the same sand surface?

Ans

$D_f = 0$

For strip footing,

$q_u = 5250 \text{ lb/ft}^2 = \gamma D_f N_q + 0.5 B \gamma N_\gamma$

$= 0.5 \times 4 \times \gamma N_\gamma$

$\gamma N_\gamma = 2625 \text{ lb/ft}^3$

For square footing,

$q_{nu} = \gamma D_f (N_q - 1) + 0.4 \times B \times \gamma N_\gamma$

$= 0 + 0.4 \times 10 \times 2625 = 10500 \text{ lb/ft}^3$

$q_{na} = \dfrac{q_{nu}}{3} + \gamma D_f$

$= \boxed{3500 \text{ lb/ft}^3}$

(2) Write down the safe allowable bearing pressure determination steps from @ net allowable bearing pressure?

If settlement criteria (25 mm max for footing and 50 mm maximum for raft foundation) then,

$q_{safe} = q_{na}$.

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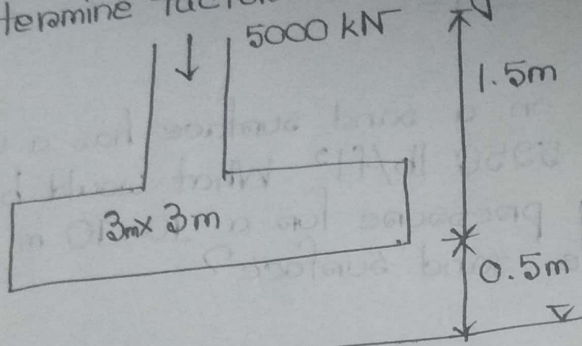
③ Determine factor of safety.



$c = 0$

$\phi = 38^\circ$

$\gamma = 19.5 \ kN/m^3$

$N_q = 65.52$

$N_\gamma = 77.2$

Assuming soil to be saturated above base, $\gamma = \gamma_{sat}$.

$$q_u = \gamma D_f (N_q - 1) R_{w_1} + 0.4 B \gamma N_\gamma R_{w_2}$$

$$= 19.5 \times 1.5 \times (65.52 - 1) \times 1 + 0.4 \times 4 \times 19.5 \times 77.2 \times \frac{2}{3}$$

$$\left[\begin{aligned} R_{w_1} &= 0.5\left(1 + \frac{1.5}{1.5}\right) = 1 \\ R_{w_2} &= 0.5\left(1 + \frac{0.5}{1.5}\right) = \frac{2}{3} \end{aligned} \right]$$

$q_{nu} = 3493 \ kN/m^2$, $q_u = 3522 \ kN/m^2$

$$q_{na} = \frac{Q_{na}}{B^2} = \frac{5000}{3^2} = \cancel{\frac{q_{nu}}{FS} + \gamma D_f} = \cancel{\frac{3493}{FS}}$$

$$= \frac{q_u}{FS} - \gamma D_f$$

$$= \frac{3522}{FS} - 19.5 \times 1.5$$

$$\boxed{FS = 6.02}$$

(4) How does depth of footing help increase bearing capacity?

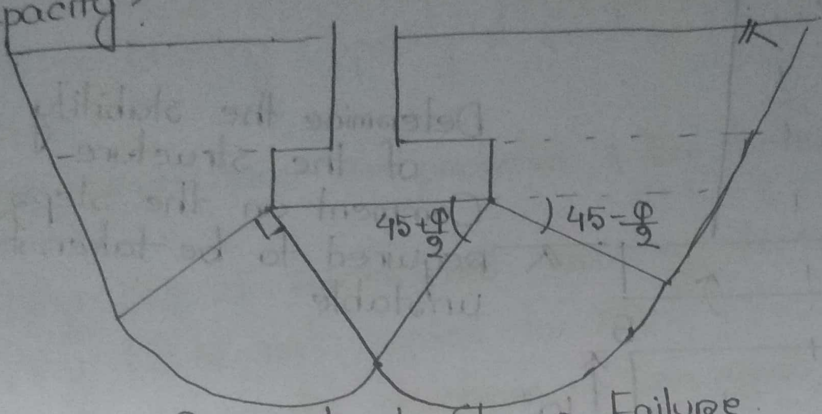
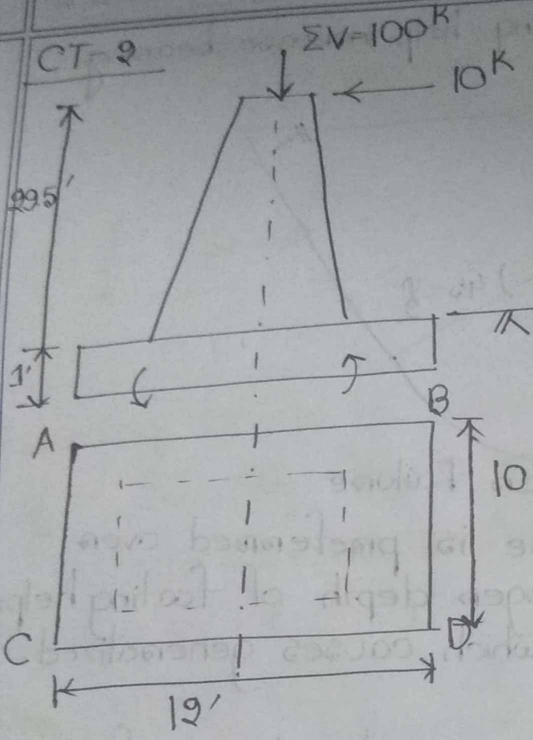


Fig. Generalized Shear Failure.

(1) Generalized shear failure is preferred over localized shear failure. Larger depth of footing helps increase the arc length which causes generalized shear failure.

(2) Larger depth of footing generates larger frictional resistance.

CT-2



Determine the stability of the structure.
· Comment on the steps required to be taken if unstable.

Ans

$M = 10 \times 30.5 = 305$ k-ft

$$\sigma_B = \frac{P}{A} - \frac{Mc}{I}$$

$P = 100^k$

$A = 10 \times 12 = 120 \text{ ft}^2$

$M = 305$ k-ft

$c = 6'$

$$I = \frac{10 \times 12^3}{12} = 1440 \text{ ft}^4$$

$\sigma_B = -0.4375$ ksi (Tension)

Soil cannot resist any tension, so structure will be unstable.

Remediation:

1. For $\sigma_B = 0$ Area or depth footing can be increased
2. Pile or other suitable deep foundation can be chosen

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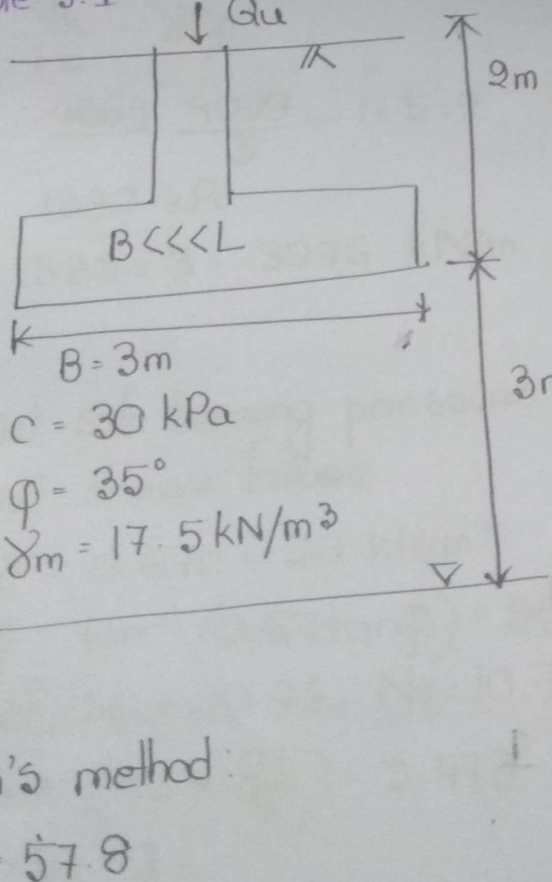
Classnote Maths + Questions

Ultimate Bearing Capacity of Shallow Foundation

Theory

1. Why foundations are provided 1.5m below ground surface?
2. Why γD_f is subtracted from q_u to get q_{nu}?
3. Why FS is not applied to γD_f?
4. Draw schematic diagram of generalized shear failure.

Example 5.1



$B = 3m$

$c = 30\ kPa$

$\phi = 35°$

$\gamma_m = 17.5\ kN/m^3$

Find q_u, q_{nu}, q_{na} and allowable load per m on footing.

Ans

Terzaghi's method:

$N_c = 57.8$

$N_q = 51.4$

$N_\gamma = 42.4$

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$$q_u = cN_c + \gamma D_f N_q + \frac{1}{2} B \gamma N_\gamma$$
$$= 30 \times 57.8 + 17.5 \times 2 \times 51.4 + 0.5 \times 3 \times 17.5 \times 42.4$$
$$= 4646 \text{ kN/m}^2$$

$$q_{nu} = q_u - \gamma D_f$$
$$= 4646 - 17.5 \times 2$$
$$= 4611 \text{ kN/m}^2$$

$$q_{na} = \frac{q_u}{FS} - \gamma D_f$$
$$= \frac{4646}{3} - 17.5 \times 2$$
$$= 1514 \text{ kN/m}^2$$

$$Q_{na} = q_{na} \times B$$
$$= 1514 \times 3$$
$$= 4541 \text{ kN/m}$$

Meyerhof's factor:

$N_c = 46$

$N_q = 33$

$N_\gamma = 37$

$$N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right)$$
$$= \tan^2\left(45 + \frac{35}{2}\right)$$
$$= 3.69$$

$$S_c = 1 + 0.2 N_\phi \left(\frac{B}{L}\right) = 1$$

$$S_q = 1 + 0.1 N_\phi \left(\frac{B}{L}\right) = 1$$

$$S_\gamma = S_q = 1$$

$$d_c = 1 + 0.2 \sqrt{N_\phi} \left(\frac{D_f}{B}\right) =$$
$$= 1 + 0.2 \times \sqrt{3.69} \times \frac{2}{3} = 1.257$$

$d_q = 1 + 0.1\sqrt{N_q}\left(\frac{D_f}{B}\right)$
$= 1.128$
$d_\gamma = d_q = 1$

$q_u = cN_c s_c d_c + \gamma D_f N_q s_q d_q + \frac{1}{2} B\gamma N_\gamma s_\gamma d_\gamma$
$= 30\times46\times1\times1.257 + 17.5\times2\times33\times1\times1.128 +$
$0.5\times3\times17.5\times37\times1\times1.128$
$= 4099 \text{ kPa}$

$q_{nu} = q_u - \gamma D_f$
$= 4064 \text{ kPa}$

$q_{na} = \frac{q_u}{FS} - \gamma D_f$
$= \cancel{4064}\ \frac{4099}{3} - 17.5\times2$
$= 1332 \text{ kPa}$

$Q_{na} = 1332\times3 = 3996 \text{ kN/m}$.

Extra:

5.2. Net safe bearing pressure if the soil fails by localized shear failure.

Ans: $\bar{c} = 0.67c = 20 \text{ kN/m}^2$
$\bar{\phi} = \tan^{-1}(0.67\tan\phi) = 25°$

Meyerhof, $N_c = 20.71$, $N_q = 10.7$, $N_\gamma = 6.8$

$N_\phi = \tan^2\left(45 + \frac{25}{2}\right) = 2.473$

$s_c = s_q = s_\gamma = 1$

$d_c = 1.21$

$d_\gamma = d_q = 1.105$

$q_u = 1115 \text{ kN/m}^2$, $q_{nu} = 1080 \text{ kN/m}^2$, $q_{na} = 337 \text{ kN/m}^2$;
$Q_{na} = 1010 \text{ kN/m}$.

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5.3, 5.4.

Water Table i) At ground level

ii) At foundation base

iii) 1.25m below ground level

iv) 1.25m below base foundation

$\gamma_{sat} = 18.5 kN/m^3$ $\gamma_b = 18.5 - 9.81 = 8.69$ kN/m^3

i) $q_u = 30 \times 46 \times 1.257 + (8.69 \times 2 \times 33 \times 1.128 + 0.5 \times 3 \times 37 \times 8.69 \times 1.128$

$= 2926$ kN/m^2

$q_{nu} = 2908$ kN/m^2

$q_{na} = 958$ kN/m^2 (FS = 3)

$Q_{na} = 2874$ kN/m.

ii) $q_u = 30 \times 46 \times 1.257 + 17.5 \times 2 \times 33 \times 1.128 + 0.5 \times 3 \times 37 \times 8.69 \times 1.128$

$= 3582$ kN/m^2

$q_{nu} = 3547$ kN/m^2

$q_{na} = 1159$ kN/m^2

$Q_{na} = 3477$ kN/m^2

iii) Terzaghi's method,

$R_{w_1} = 0.8125$

$R_{w_2} = 0.5$

$q_u = 30 \times 57.8 \times + 18.5 \times 2 \times 51.4 \times 0.8125 + 0.5 \times 3 \times 42.4 \times 18.5 \times 0.5$

$= 3868$ kN/m^2

$q_{nu} = 3831$ kN/m^2 $[\gamma = \gamma_{sat}]$

$q_{na} = 1252$ kN/m^2

$Q_{na} = 3757$ kN/m

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IV) Terzaghi,

$R_{w_1} = 0.5\left(1 + \frac{9}{9}\right) = 1$

$R_{w_2} = 0.5\left(1 + \frac{1.25}{9}\right) = 0.8125$

$q_u = 4592\ kN/m^2$

$q_{nu} = 4555\ kN/m^2$

$q_{na} = 1494\ kN/m^2$

$Q_{na} = 4481\ kN/m^2$

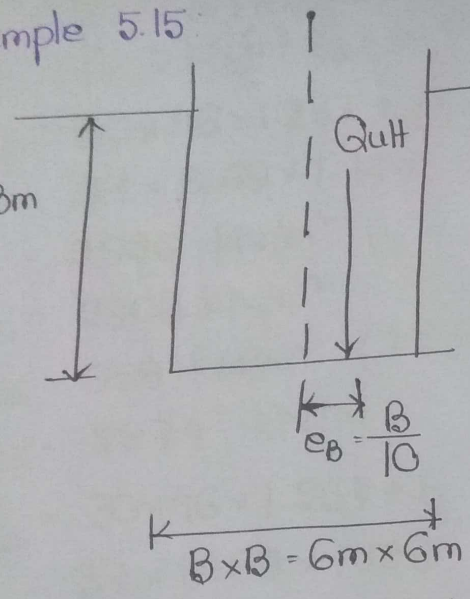
Date :

Bearing Capacity of Foundation Subjected to Eccentric Loading

Theory:

1. Develop kern and explain it's meaning.

Example 5.15



$c = 0$
$\gamma = 18.5\ kN/m^3$
$\varphi = 33^\circ,\ N_{cor} = 20$

$\varphi = 0.3N_{cor} + 27 = 33^\circ$

$B' = B - 2e = 6 - 2\times0.6 = 4.8m$

$L' = L = 6m$

$q_u = \gamma D_f N_q S_q d_q + 0.5B'\gamma N_\gamma S_\gamma d_\gamma$

$N_q = 26.3,\ N_\gamma = 26.55$

$N_\varphi = 1.842$

$S_q = \cancel{1.34} = S_\gamma = 1 + 0.1 N_\varphi \left(\frac{B'}{L}\right) = 1.147$

$d_q = 1.115 = d_\gamma = 1 + 0.1\sqrt{N_\varphi}\left(\frac{D_f}{B'}\right)$

$q_u = \cancel{3942\ kN/m^2}\ 3374\ kN/m^2$

$Q_{ult} = \cancel{113,530\ kN}\ q_u \times 6 \times 4.8 = 97181\ kN$

Example 5.16

$B' = 6 - 2\times0.75 = 4.5m$

$L' = 6 - 2\times0.6 = 4.8m$

$\phi = 0.3\times N + 27 = 33^\circ$

$N_q = 26.3\times \quad N_\gamma = 26.55$

$\gamma = 18.5\ kN/m^3, \ N_\phi = 1.842$

$s_q = s_\gamma = 1 + 0.1 N_\phi \cdot \frac{B'}{L'} = 1.173$

$d_q = d_\gamma = 1.072$

$q_u = 1888\ kN/m^2$

$Q_{ult} = 40776\ kN$

Date :

Safe Bearing Pressure and Settlement Calculation

Theory:

1. Why $\Delta\sigma$ is taken as unfactored?
2. Why calculation is done assuming clay to be saturated?
3. Reasons for using β (Settlement coefficient).

Example 6.12

$q_n = 100\ kN/m^2$

2V : 1H load spread.

Layer	H (m)	C_c	e_o	$P_o\ (kN/m^2)$	$\Delta p\ (kN/m^2)$	S_{oc} (m)
1	4	0.16	0.93	48.4	68.6	0.127
2	4	0.14	0.84	78.14	38.1	0.0525
3	3	0.11	0.76	105.8	25.5	0.0176
4	5	0.09	0.73	139.8	17.5	0.0133

$P_{o_1} = 17.\times 2 + 7.19\times 2 = 48.4\ kN/m^2$

$\Delta P_1 = 100\times \dfrac{8\times 12}{(8+2)(12+2)} = 68.6\ kN/m^2$

$S_{oc} = 0.2104$

$\beta = 0.8$

$S_c = \beta S_{oc} = 0.168\ m$

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Pile Capacity for Vertical Loading

Theory

1. Construction problems in boring pile.
2. Write down the steps for footing design from planning
3. Why $FS_b > FS_f$?
4. Why do we choose pile tip to be rested on the granular deposition rather than stiff clay?
5. Areas where negative skin friction can occur. Mitigation of negative skin friction.

Lecture problems

1. A concrete pile of 15" dia, 40' long is driven into a homogenous stratum of clay with the water table at the GL. The clay is of medium stiff consistency with the undrained shear strength $c_u q_u = 3600$ lb/ft². Compute Q_b by skempton's method and Q_f by α-method. Also determine the pullout capacity and the allowable pullout load with FS = 3.0. Determine Q_a for overall FS = 2.5.

Skempton's method,

$$Q_b = cN_c A_b$$

$$= 300 \times 9 \times \pi \left(\frac{7.5}{12}\right)^2 \qquad Q_b = 2508 \text{ lb}$$

$$= 3313 \text{ lb} \quad \left[\begin{matrix}14.5 \text{ ton/m}^2 \\ > 11 \text{ ton/m}^2\end{matrix}\right] \quad \begin{matrix} d = 0.381 \text{m} \\ L = 12.192\end{matrix}$$

Not okay

α- method

$$Q_f = c\alpha_c A_f$$

$$= 300 \times 0.55 \times \pi \times \frac{15}{12} \times 40$$

$$= 25918 \text{ lb} \qquad [0.9 \text{ ton/m}^2 < 7 \text{ ton/m}^2 \text{ Ok}]$$

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$$Qa_1 = \frac{Qb}{3} + \frac{Qf}{1.5}$$

$$= \frac{2508}{3} + \frac{25918}{1.5} = 18115 \text{ lb}$$

$$Qa_2 = \frac{Qb + Qf}{2.5} = 11370 \text{ lb (Governs)}$$

Pullout capacity,

$$Q_{pull} = \frac{Qf}{FS} + W_{pile}$$

$$= \frac{25918}{3} + 150 \times \pi \left(\frac{7.5}{12}\right)^2 \times 40$$

$$= 16002 \text{ lb}$$

2. A group pile of 9 friction piles arranged in a square pattern is to be proportioned in a deposit of medium stiff clay. Assume the piles are 30 cm diameter and 10 cm long. Assume $\alpha = 0.8$ and $C_u = 50$ kPa. The bottom of the pile cap embedding all the piles rest at a depth of 1.5m below the ground surface. Find the optimum spacing for the piles.

Single pile action,

$$Qf = \alpha C A_f$$

$$= 0.8 \times 50 \times \pi \times 0.3 \times 10 = 377 \text{ kN}$$

$$Q_b = c N_c A_b$$

$$= 50 \times 9 \times \pi \times 0.15^2 = 31.8 \text{ kN} \approx 0$$

$$Q_u = Q_f \times 9 = 3393 \text{ kN}$$

Group pile action

$Q_{fg} = \alpha c A_{fg}$

$= 1 \times 50 \times \left[5+5+2\times\frac{d}{2}\right] \times 4 \times 10$

$= 200\,[25+0.3] \times 10$

$Q_{ug} = 200\,[25+0.3] \times 10$

$\eta = \frac{Q_{ug}}{Q_u} = 1$

$Q_{ug} = Q_u$

$s = 0.7\,m = 2.33d$

3. For Q2 determine uplift capacity

$Q_{u,i} = \frac{2}{3} Q_f + W_{pile}$

$= \frac{2}{3} \times 3393 + 24 \times \pi \times \left(\frac{0.3}{2}\right)^2 \times 10 \times 9 \quad [\gamma_{conc} = 24\ kN/m^3]$

$= \cancel{2279\ kN/m^2}\ 2415\ kN \cancel{/m^2}$

$Q_{u,g} = \frac{2}{3} Q_{fg} + W_{soil}$

$= \frac{2}{3} \times 3393 + (2 \times 0.7 + 0.3)^2 \times 10 \times 18 \quad [\gamma_{soil} = 18\ kN/m^3]$

$= 2782.2\ kN$

$Q_a = \frac{\frac{2}{3} \times 3393}{2} + W_{pile}$

$= 1284\ kN$

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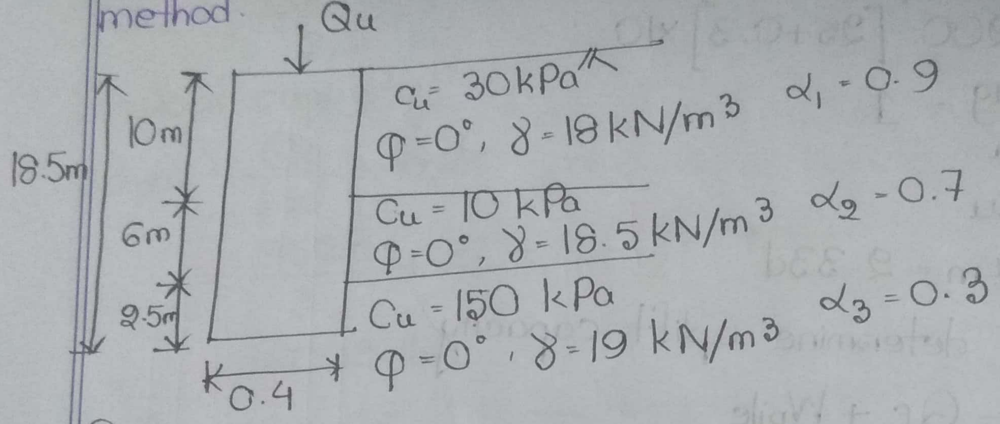
4. A pile of 40 cm dia and 18.5m length passes through two layers of clay and is embedded in a third layer.
Compute Q_f by α-method and Q_b by skempton's method.



Q_f

$Q_{f} = Q_{f_1} + Q_{f_2} + Q_{f_3}$

$= 0.9\times30\times\pi\times0.4\times10 + 0.7\times10\times\pi\times0.4\times6 +$
$0.3\times150\times\pi\times0.4\times2.5$

$= 533$ kN

$Q_b = cN_cA_b$

$= 150\times9\times\pi\times(0.2)^2$

$= 170$ kN

$Q_u = 703$ kN

$Q_a = \frac{703}{2.5} = 281$ kN

5. Find the optimum spacing if 9 piles of Q5 are arranged in a square pattern.

$Q_{u,i} = 703\times9 = 6327$ kN

Group action,

$Q_{fg} = 1\times 30\times 4\times(25+0.4)\times 10 + 1\times 10\times 4\times(25 + 0.4)\times 6$
$+ 1\times 150\times 4\times(25+0.4)\times 2.5$

$= 2940\,(25+0.4)$

$Q_{bg} = 150\times 9\times(25+0.4)^2$

$Q_{ug} = Q_{fg} + Q_{bg}$

$\eta = 1 = \dfrac{Q_{ug}}{Q_{ui}}$

$150\times 9\times(25+0.4)^2 + 2940\,(25+0.4) = 6327.$

$s = 0.5m = 1.168d$
$\approx 2.5 - 3d$ [Code recommended]

If the loading was upward,

$Q_{a,i} = \dfrac{\frac{2}{3}\times 6327}{2} + 24\times 9\times \pi\times 0.2^2\times 18.5$

$= 2611\ kN$

$Q_{a,g} = \dfrac{\frac{2}{3}\times 2940\,(25+0.4)^2}{2} + (25+0.4)^2\times(10\times 18 + 6\times 18.5 + 2.5\times 19)$

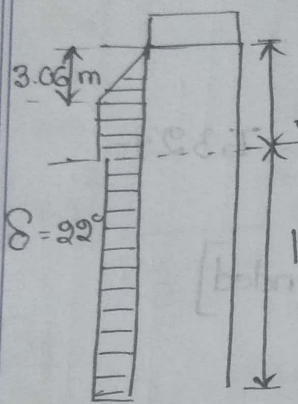
$= 1318.5\,(25+0.4)^2$

$s = 0.5m \approx 1\ m$ [Code]

Date :

Sheet 16-1 Page 963 Bowles

A 460 mm diameter pipe pile is driven closed end 15m into a cohesionless soil with an estimated φ angle of 34°. The soil has a $\gamma_d = 16.5\ kN/m^3$ $\gamma' = 8.60\ kN/m^3$. The GWT is 6m below the ground surface. Estimate the ultimate pile capacity P_u using Terzaghi's method and friction angle $\delta = 22°$



$\gamma_d = 16.5\ kN/m^3$ (assumed)

$A_b = \frac{\pi}{4}(0.46)^2 = 0.166\ m^2$

Critical depth,

for $\varphi = 34°$, $\frac{Z_c}{d} = 0.275 \times 34 - 2.7 = 6.65$

$\therefore Z_c = 6.65 \times 0.46 = 3.06$

$\sigma_{Z_c} = \gamma z =$

$A_{f_1} = \pi \times 0.46 \times 3.06 = 4.42$

$A_{f_2} = \pi \times 0.46 \times (6 - 3.06) = 4.25$

$A_{f_3} = \pi \times 0.46 \times 10 = 14.45$

$k_o = 0.8 \sim 1.0$

$q_f = [\sigma_v K_o] \tan\delta A_f$

Critical depth:

$y = \frac{Z_c}{d}, x = \varphi°$

$\frac{y-5}{5-7.2} = \frac{x-28}{28-36}$

$\Rightarrow y = 0.275x - 2.7$

$\varphi < 36°$

$\varphi > 36°$

$\frac{y-7.2}{7.2-15} = \frac{x-36}{36-43}$

$y = 1.95x - 63$

End bearing $q_b = qN_q$ $[q = 3.06 \times 16.5]$

$= 51 \times 30 = 1530\ kN$

$Q_b = 1530 \times 0.166 = 254\ kN$

Frictional component: $k = 0.8$ $(0.8 \sim 1.0)$

Part 1: $Q_{f_1} = \frac{1}{2} \times (51 \times 0.8 \tan 22°) \times 4.42$

$= 36.4\ kN$

$Q_{f_2} = (51 \times 0.8 \tan 22°) \times \pi \times (0.46) \times (16 - 3.06)$

$= 310\ kN$

$(Q_{ult})_1 = 254 + 36.4 + 310 = 599\ kN$

$(Q_{ult}) = (Q_{ult})_1 - W_{self\,wt}$

$= 599 - 64 = 535\ kN$

Date:

Ex 16-3 Page-9 Bowles
Estimate the ultimate pile capacity of a 300mm round concrete pile that is 30m long with 24m driven into a soft clay soil of average $q_u = 24 kN/m^2$. Assume $\gamma' = 8.15 kN/m^3$ for the soil. The water surface is 2m above the ground line.

For Point bearing: $q_b = cN_c A_b$

$= \frac{q_u}{2} \times N_c \times \pi \times (\frac{d}{2})^2$

$= \frac{24}{2} \times 6.2 \times 0.071 = 5.3 kN$

↑ surface friction pile. better not to use $\frac{D_f}{B}$ factor.

Friction pile: $q_f = (\alpha_c C) A_f$

$= 0.9 \times \frac{24}{2} \times \pi \times 0.3 \times 30 = 244.3 kN$

$\alpha = 0.9$ (since the soil is soft)

Self weight of the pile, $W_p = [0.071 \times 30] * 24 kN$

$= 51 kN$

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

$\therefore q_{ult} = q_{ult,1} - W_p = 244.3 + 5.3 - 51 = 199 kN$

Tension piles:

1. Used beneath ~~bridges~~ buildings to ~~support~~ resist uplift from hydraulic pressure.
2. To support structures over expansive soils.
3. Over turning caused by wind, ice loads and broken wires may produce tension forces for power transmission towers.

In this type of situation the piles or piers supporting the tower legs must be designed for both compressive and tension forces.

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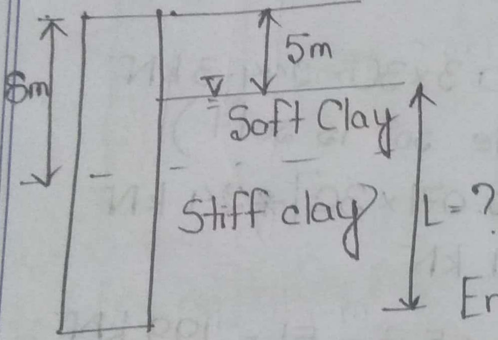
Excersize 16.3 A pile is driven through a soft cohesive
Page 963 deposit overlying a stiff clay. The GWT
Bowles is 5m below the ground surface and the stiff
clay is at 8m depth.

Other data:

Soft clay	Stiff clay
$\gamma_{wet} = 17.5$	$19.3\ kN/m^3$
$\gamma' = 9.5$	$10.6\ kN/m^3$
$S_u = 50$	$165\ kN/m^2$

Estimate the length of a 550 dia pile carrying an allowable load $P_a = 420$ kN using an SF = 4.0 and Terzaghi's method

Solution:



$c = S_u$

$A_b = \frac{\pi}{4} \times (0.55)^2 = 0.238\ m^2$

$A_{f_1} = \pi \times 0.55 \times 8 = 13.8\ m^2$

$A_{f_2} = \pi \times 0.55 \times L = 1.73L$

Assume end point is extended into stiff clays.

End bearing $= q_p = [c N_c A_b]$

$= 165 \times 6.2 \times 0.238$

$= 243$ kN

Skin friction:

$q_{f_1} = [\alpha_a c_1] A_{f_1} = 0.85 \times 50 \times 13.8 = 586.5$ kN

[$\alpha_a = 0.85$; soft to medium clay]

$q_{f_2} = [\alpha_a c_2] A_{f_2} = 0.5 \times 165 \times 1.73 \times L = 142.7L$

[$\alpha_a = 0.5$; stiff clay]

$q_{ult} = q_p + q_{f_1} + q_{f_2} = 243 + 586.5 + 142.7L$

$q_{allow} = \frac{q_{ult}}{FS} = \frac{829.5 + 142.7L}{4} = 420$

$L = 6$ m

Total length of pile = 8+6 = 14 m.

Exe 16.4
Page 964
Bowles

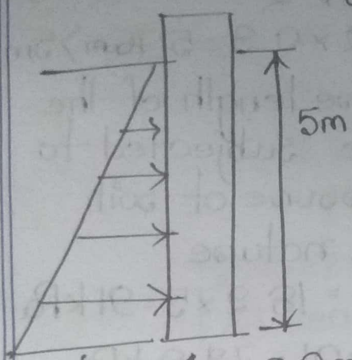
What is the approximate ultimate pullout resistance T_u for a tension pile in a medium dense sand with $\varphi = 36°$, $\gamma = 18.2\ kN/m^3$ and using an 800 mm diameter concrete pile with a length of 5m (and no bell)?

$\delta = \frac{2}{3}\varphi = 24°$

for $\varphi = 36°$, $\frac{Z_c}{d} = 0.275\varphi - 2.7 = 7.2$

$Z_c = 7.2 \times 0.8 = 5.76\ m > 5m$

So the entire pile will be subjected to lateral earth pressure with hydrostatic variation.



$d = 0.8m$

$\sigma_v K$
$= \gamma Z K$
$= 18.2 \times 5 \times 0.8$
$= 72.8\ kPa$

$k = 0.8 \sim 1.0$ (should be more than that occurs in rest condition)

$(T_u)_{sc} = [\sigma_v k \tan\delta] A_f$
$= [72.8 \times 0.8 \times \tan 24]\ \pi \times 0.8 \times 5$
$= 408\ kN$

$T_u = (T_u)_{soil} + W_{pile}$
$= 408 + \left[\frac{\pi}{4} \times 0.8^2 \times 5 \times 24\right]$
$= 468\ kN$

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

$k \rightarrow$ lateral pressure coefficient to evaluate q_f for both sand and clay.

Use, $k = 0.8$ (both tension and comp) for low volume displacement pile)

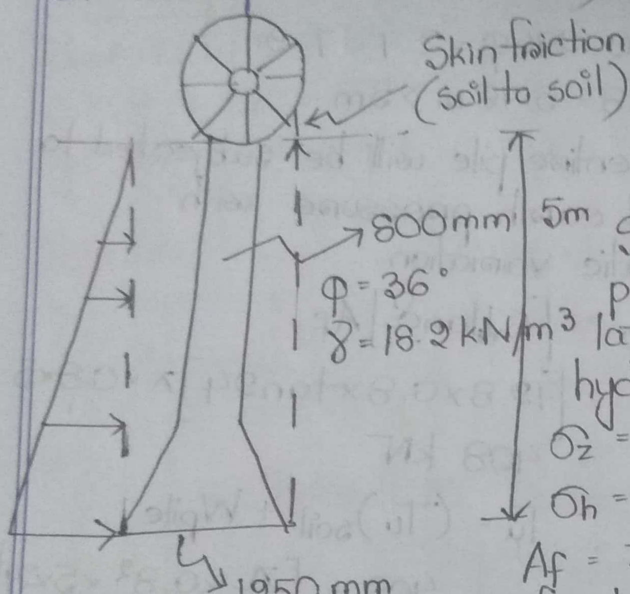
$= 1.0$ for displacement piles.

(Usually $k > k_o = 1 - \sin\varphi$)

Use the same value of clay for both sand and clay.

Date:

Q4] What is the approximate ultimate pullout resistance T_u for a tension pile in a medium dense sand with $\phi = 36°$, $\gamma = 18.5\ kN/m^3$ and using an 800 mm dia concrete pile with a length of 5m (and with a bell at the tip of dia 1.25m)



For $\phi = 36°$, $\frac{Z_c}{d} = 0.275\phi - 2.7$
$= 0.275 \times 36 - 2.7$
$= 7.2$

$Z_c = 7.2 \times 0.8 = 5.76m > 5m$

So, the entire length of the pile will be subjected to lateral pressure of soil, hydraulic in nature.

$\sigma_z = \gamma D_f = \gamma Z_c = 18.2 \times 5 = 91\ kPa$

$\sigma_h = k\sigma_z = 0.8 \times 91 = 72.8\ kPa$

$A_f = \pi \times 0.8 \times 5 = 12.57\ m^2$

Skin friction: $Q_f = [\sigma_z k \tan\delta] A_f$

$W_{pile} = \frac{\pi}{\cancel{48}\ 4}(0.8)^2 \times 5 \times 24$
$= 60 kN$

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

Since a bell is at the tip, so friction will occur between soil to soil (not soil to pile)

ϕ at the surrounding soil of the pile tip $= 36°$

$\tan\delta = \tan\phi = \tan 36°$

$Q_f = 72.8 \times \tan 36 \times 12.57 = 665\ kN$

End bearing: This component will be derived by the shaded area of the pile X-section

$N_q = 38$

$Q_b = [qN_q]A_b = 91 * 38 \times \frac{\pi}{4}[1.25^2 - 0.8^2]$
$= \cancel{4244\ kN}\ 2505\ kN$

$Q_{ult} = \cancel{Q_b} + Q_f + W_{pile} + Q_b$
$= \cancel{665 + 60 + 60 + 665 = 725\ kN}$
$= 3230\ kN$

Group Capacity of Piles

Ex 13.2
Vargese
Page 276

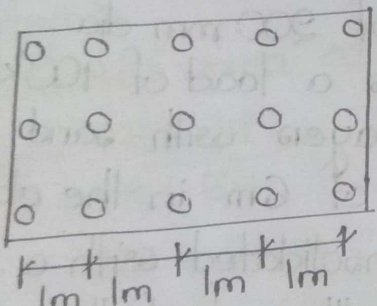
A group of friction piles in clay consists of 15 piles of 500 mm dia (grouped as 5×3) spaced at 1m apart. If the undrained shear strength of the clay is 0.3 kg/cm² and the piles are 20 m in length. Estimate the group ~~ga~~ capacity and its efficiency? [Note: the pile cap will extend 25 cm around the piles and the pile cap will touch the soil but as the soil is clay the strength of pile cap need not be considered]

Solution: Step 1: Capacity of each pile and that of the group.

Individual =

End bearing: $Q_b = A_b c N_c = \left[\frac{\pi}{4}(0.5)^2\right] \times [0.3 \times 100]$

$= 35 \text{ kN}$



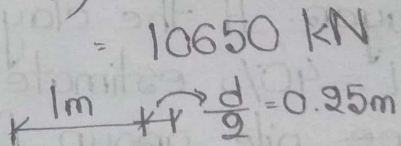
$Q_f = \alpha c \Sigma A_f$

$= 0.9 \times 30 \times \pi \times 0.5 \times 20$

$= \cancel{69}\ 675 \text{ kN}$

$\therefore Q_u = n_p (Q_b + Q_f) = 15 \times (675 + 35)$

$= 10650 \text{ kN}$



$c = 0.3 \text{ kg/cm}^2$

$= 30 \text{ kN/m}^2$

$\alpha = 0.9$ (soft clay)

Step 2: Capacity as a group:

group perimeter:

L: 5 piles @ 1m apart $= \left[4 \times 1 + \frac{0.5}{2} \times 2\right] = 4.5\text{m}$

B: 3 piles @ 1m apart $= \left[2 \times 1 + \frac{0.5}{2} \times 2\right] = 2.5\text{m}$

Projected area by perimeter for end bearing -

$= 4.5 \times 2.5 = 11.25 \text{ m}^2$

Perimeter = $[4.5+2.5] \times 2 = 14m$

Depth ratio = $\frac{D}{B} = \frac{20}{2.5} = 8 > 2.5m$.

N_c from skempton's curve (P-117, Figure 6.4 Vargese)

for $\frac{D}{B} > 2.5$, $N_c = 9.0$

$Q_b = A_b C N_c = 11.25 \times 30 \times 9 = 3038 \text{ kN}$

$Q_f = \alpha c \Sigma A_f = 1 \times 30 \times 14 \times 20 = 8400 \text{kN}$ ↳ Perimeter

here $\alpha = 1$ [soil to soil friction]

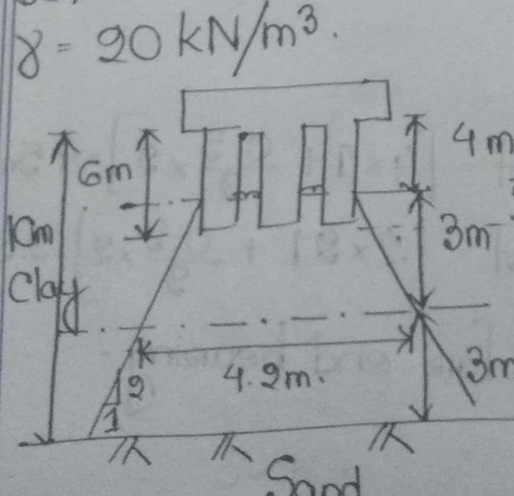
$Q_u = 3038 + 8400 = 11438 \text{ kN}$

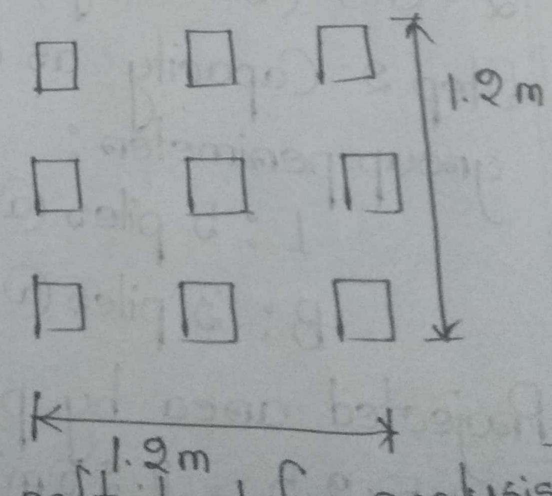
Lesser of the above two Q_u is the group capacity

thus $Q_u = 10650 \text{ kN}$

Settlement

Ex 4.7 Page 93 Vargese | A group of nine piles of 200 mm dia spaced at 0.5m transfer a load of 400kN into a 10m thick clay layer with sand below. It penetrates to a depth of 6m in the clay layer. If the clay is normally consolidated with a LL = 40%, estimate the probable settlement of the ~~pipe~~ pile group. Assume saturated water content, 39%, $G_s = 2.7$ and the water level is at GL. Let $\gamma = 20 \text{ kN/m}^3$.





[Assume raft level for analysis]

Date :

Step 1:

Determine characteristics of soil.

$C_c = 0.009\,(LL-10)$

$= 0.009\,(40-10) = 0.27$

$e_0 = \frac{wG_s}{S_r} = \frac{0.39\times 2.7}{1} = 1.05.$

Step 2:

Calculate 2/3 depth, the assumed level of raft where ΔP is released from the top of the pile.

$\frac{2}{3}L = \frac{2\times 6}{3} = 4\,m$ from top of the pile.

Thickness of compressible layer for this pile arrangement, $H = 10-64 = 6m$. i.e, 1 m below pile tip.

mid depth, $\frac{H}{2} = \frac{6}{2} = 3m$

Step 3:

Enlarged area at CG $= \left[\frac{3}{2}+1.2+\frac{3}{2}\right]^2 = 4.2^2 = 17.64m^2$

$\Delta P = \frac{400\,kN}{17.64}$ ← If not given in KN then convert it to kN by multiplying with Aold, and then divide by Anew.

$= 22.7\ kN/m^2$

$P_0 = (20-10)\times(4m+3.3) = 70\,kN/m^2$

$\downarrow$ initial $\downarrow$ mid-depth

$\Delta P_{top} = 228\ kN/m^2$, $\Delta P_{avg} = 125.35\ kN/m^2$ (assuming submerged)

Step 4:

Settlement, $S_{oc} = H\frac{C_c}{1+e_0}\log\frac{P_0+\Delta P_{avg}}{P_0}$

$= \cancel{0.0964\,m} = \cancel{9.64\,cm}$

$= 35\,cm$

P-9

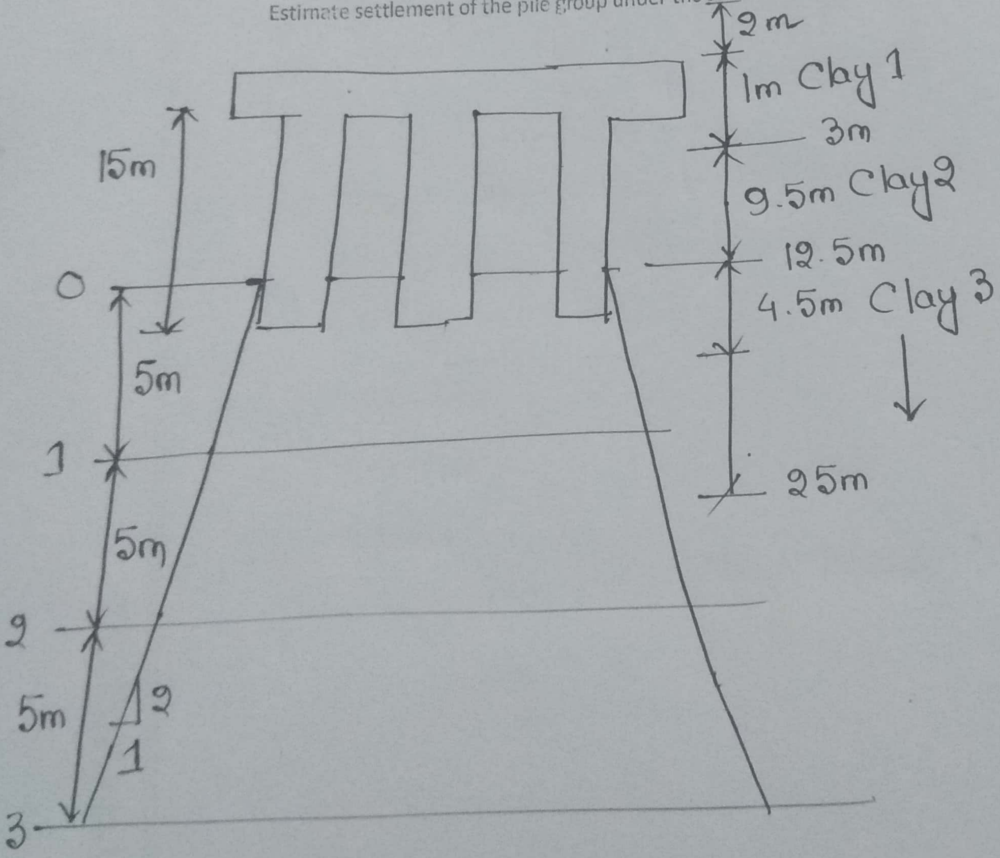
CE 343 St. ID:.......................

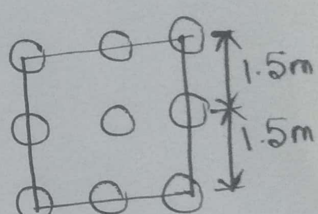
Dt: 31-05-2011

Q.1. A group of nine piles (each 600 mm diameter, 15 m long and 1500 mm spacing) is driven into layered clay-dominant soils to carry a design load of 12000 kN (dead load= 8000 kN and live load= 4000 kN). The piles are grouped as 3 x3 with cut-off level at 2.0 m below ground level and ground water table at GL. The underlying soil characteristics are as follows:

Depth from GL (m)	: 0~3.0	3.0~12.5	12.5~25.0
Cohesion, c (kN/m^2)	: 25	40	125
Liquid limit, LL (%)	: 40	38	40
Moisture content, w (%)	: 28	27	25
Dry density, γ (kN/m^3)	: 16.5	17.0	17.5
Saturated density, γ (kN/m^3)	: 19.5	19.5	20.0
Specific gravity, G_s	: 2.68	2.69	2.71

Estimate settlement of the pile group under the sustained loading.





Load transfer level at 0-0, $\frac{2}{3}L = 4.5 = 3.0$ m
(L = transferable clayey layer = clay 3)

Initial stress at 0-0 (before)

i) for 3.0m of clay 1 = $19.5\times3 = 58.5$ kPa
ii) for 9.5m of clay 2 = $19.5\times9.5 = 185.25$ kPa
iii) for 3.0m of clay 3 = $20.0\times3 = 60.0$ kPa
iv) less $3+9.5+3 = 15.5$ m of water = $9.8\times15.5 = 152$ kPa

Effective P_0' at level 0-0 = $58.5+185.25+60-152 = 152$ kPa

Effective P_0' at level 1-1 (5m below 0-0) = $152+[5\times20-5\times9.8] = 203$ kPa

Effective P_0' at level 2-2 (5m below 1-1) = $203+[5\times20-5\times9.81] = 254$ kPa

Effective P_0' at level 3-3 (5m below 2-2) = $254+51 = 305$ kPa.

Step 2: Consolidation increase of pressure assuming 2:1 distribution of various levels down to dead load.

Component of abutment load:
dead load = 8000 kN

$$\Delta P \text{ at 0-0 level} = \frac{8000}{\frac{\pi}{4}\times\left(1.5+1.5+\frac{0.6}{2}\times2\right)^2} = 786 \text{ kPa}$$

$$\Delta P \text{ at 1-1 level} = \frac{8000\times\cancel{\frac{\pi}{4}}\times\cancel{(3.6)^2}}{\frac{\pi}{4}(3.6+5)^2} = 138 \text{ kPa}.$$

$$\Delta P \text{ at 2-2 level} = \frac{786\times3.6^2}{(3.6+10)^2} = 55.0 \text{ kPa} < 10\%\,\Delta P_{0-0}$$

$$\Delta P \text{ at 3-3 level} = \frac{786\times3.6^2}{(3.6+15)^2} = 29.4 \text{ kPa}.$$

Step 3: P_0' and Δp at mid heights.

Date :

i) Mid heights of levels (0-0) and (1-1)

$P_o' = \frac{152 + 203}{2} = 177.5$ kPa

$\Delta P = 462$ kPa

ii) Mid heights of levels (1-1) and (2-2)

$P_o' = 229$ kPa

$\Delta P = 96.5$ kPa

iii) Mid heights of levels (2-2) and (3-3)

$P_o' = 280$ kPa

$\Delta P = 42$ kPa

Step 4: Find e_o, C_c etc

Soil will compress in clay 3

$C_c = 0.009(LL-10) = 0.009 \times (40-10) = 0.27$

$e_o = \frac{\omega G_s}{S_r} = \frac{25 \times 2.71}{100 \times 100} = 0.68$

Step 5: Settlement calculation:

$S = \frac{C_c}{1+e_o} H \log \frac{P_o + \Delta P}{P_o}$

$S_1 = 450$ mm

$S_2 = 120$ mm

$S_3 = 48$ mm

$S_T = \Sigma S_i = 450 + 120 + 48 = 618$ mm $= 61.8$ cm