CLASS TEST

S.No.: 01_JP_CE_12082025

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SOIL MECHANICS

CIVIL ENGINEERING

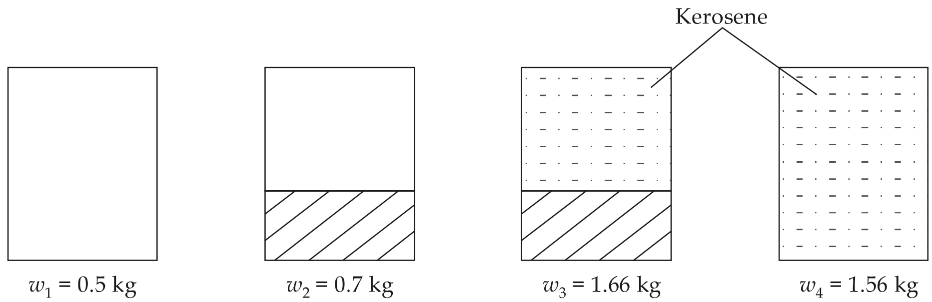
Date of Test : 12/08/2025

ANSWER KEY

1.	(c)	7.	(c)	13.	(b)	19.	(b)	25.	(a)
2.	(d)	8.	(b)	14.	(c)	20.	(c)	26.	(a)
3.	(c)	9.	(a)	15.	(c)	21.	(a)	27.	(a)
4.	(c)	10.	(c)	16.	(b)	22.	(b)	28.	(d)
5.	(c)	11.	(c)	17.	(c)	23.	(b)	29.	(b)
6.	(c)	12.	(a)	18.	(a)	24.	(d)	30.	(b)

DETAILED EXPLANATIONS

1. (c)

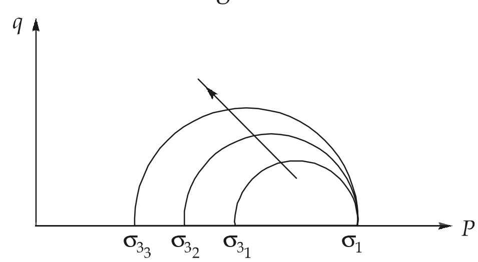


$$G_s = \left(\frac{w_2 - w_1}{w_4 - w_3 + w_2 - w_1}\right) \times G_k = \left(\frac{0.7 - 0.5}{1.56 - 1.66 + 0.7 - 0.5}\right) \times 0.8 = \left(\frac{0.2}{0.1}\right) \times 0.8$$

$$G_s = 1.6$$

2. (d)

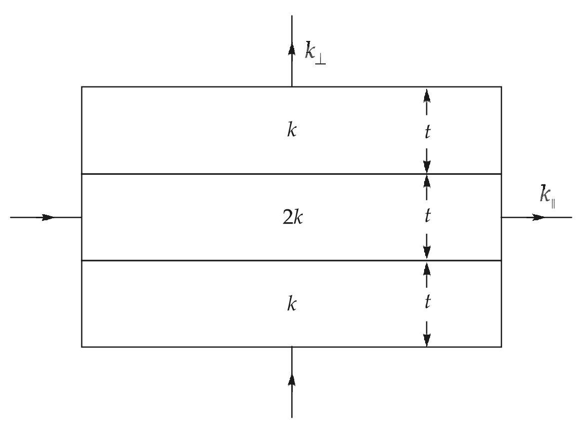
Stress path will be from right to left upwards as minor principal stress is decreasing over the time in the case of active earth pressure on retaining walls.



3. (c)

- With decrease in size, maximum dry density is lower and occur at higher water content.
- With increase in compactive effort, maximum dry density is higher and occurs at lower water content.

4. (c)



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$$k_{\perp} = \frac{\Sigma k_i z_i}{\Sigma z_i} = \frac{kt + 2kt + kt}{3t} = \frac{4kt}{3t} = \frac{4k}{3}$$

$$k_{\parallel} = \frac{\Sigma z_i}{\Sigma \frac{z_i}{k_i}} = \frac{3t}{\frac{t}{k} + \frac{2t}{k} + \frac{t}{k}} = \frac{3t}{\frac{3}{2}\frac{t}{k}} = 2k$$

$$\text{Ratio} = \frac{K_{\perp}}{K_{\parallel}} = \frac{\frac{4}{3}k}{2k} = \frac{2}{3} = 0.67$$

5. (c)

$$Q_{up} = Q_{eb} + Q_{sf}$$

$$Q_{up} = 0 + Q_{sf} \qquad \text{(Neglecting end bearing)}$$

$$Q_{up} = \alpha C_u(\pi DL)$$

$$= 0.75 \times \frac{100}{2} \times \pi \times 0.5 \times 16$$

$$Q_{up} = 940 \text{ kN}$$

6. (c)

Given, initial area, $A_0 = 18 \text{ cm}^2$

Failure strain = 25% = 0.25

$$\text{Corrected area} = \frac{A_0}{1-\varepsilon} = \frac{18}{1-0.25} = \frac{18}{0.75} = 24 \text{ cm}^2$$

7. (c)

Given, $C_c = 1.2$

But, $$C_c = \frac{D_{30}^{\,2}}{D_{60} \times D_{10}}$$

$\Rightarrow$ $$1.2 = \frac{(3.2)^2}{D_{60} \times 0.6}$$

$$D_{60} = \frac{3.2 \times 3.2}{1.2 \times 0.6} = 14.22 \text{ mm}$$

So, $$C_u = \frac{D_{60}}{D_{10}} = \frac{14.22}{0.6} = 23.70$$

$\therefore$ $C_u > 6$ and C_c lies between 1 and 3.

Hence the soil is well graded sand.

8. (b)

For Taylor's square root of time fitting method,

$$U = 90\%$$

For $U = 90\% > 60\%$

$$T_v = 1.781 - 0.933 \times \log_{10}(100 - \%U)$$

$$= 1.781 - 0.933 \times \log_{10}(10)$$

$$= 0.848$$

9. (a)

For line load, we know that

$$\sigma_z = \frac{2q}{\pi z}\left[\frac{1}{1+\left(\frac{x}{z}\right)^2}\right]^2 = \frac{2\times169}{\pi\times3}\times\left[\frac{1}{1+\left(\frac{2}{3}\right)^2}\right]^2$$

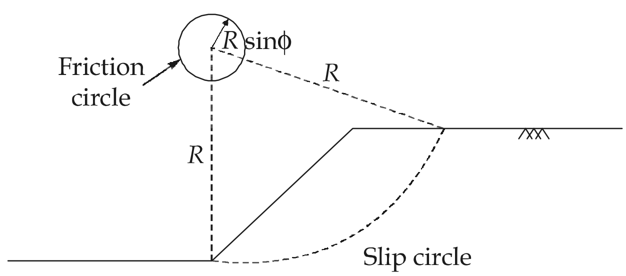
$$= \frac{2\times169}{\pi\times3}\times\frac{9^2}{13^2} = \frac{2\times169}{3\times\pi}\times\frac{81}{169}$$

$\therefore \quad \sigma_z = 17.19 \text{ kN/m}^2$

10. (c)

Friction circle method:

- Based on total stress analysis.
- In this method it is assumed that the resultant force *R* on the rupture surface is tangential to a circle of radius *R* sin ϕ, which is concentric with trial slip circle.



11. (c)

Given, $\gamma_b = 19 \text{ kN/m}^3$, $w = 17\%$

So, dry density, $\gamma_d = \frac{\gamma_b}{1+w} = \frac{19}{1+0.17} = 16.24 \text{ kN/m}^3$

Also, $\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.7\times9.81}{1+e} = 16.24$

$\Rightarrow \quad e = \frac{2.7\times9.81}{16.24} - 1 = 0.631$

When the soil is fully saturated, $S = 1$,

$\therefore \quad S \cdot e = w \cdot G$

So, new moisture content,

$$w = \frac{S\cdot e}{G} = \frac{1\times0.631}{2.7} = 0.2337 \text{ or } 23.37\%$$

$\therefore$ Additional moisture content required

$= 23.37 - 17 = 6.37\%$

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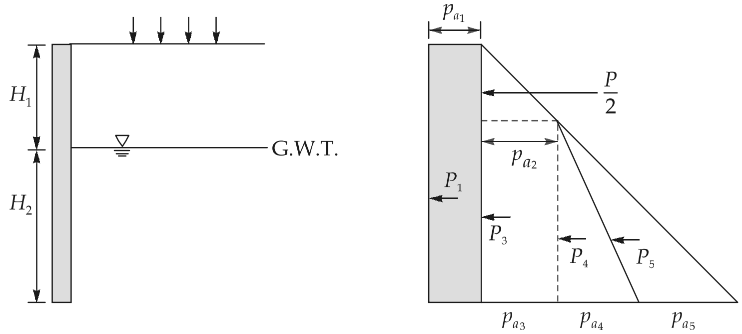
12. (a)

$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right) = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1}{3}$$

$$\gamma_{sat} = \frac{G + e}{1 + e} \cdot \gamma_w = \frac{2.65 + 0.65}{1 + 0.65} \times 9.81 = 19.62 \text{ kN/m}^3$$

$\therefore$ $$\gamma' = \gamma_{sat} - \gamma_w = 19.62 - 9.81 = 9.81 \text{ kN/m}^3$$



$$p_{a_1} = K_a \times q = \frac{1}{3} \times 14 = 4.67 \text{ kN/m}^2$$

$$p_{a_2} = K_a \cdot \gamma_d \times H_1 = \frac{1}{3} \times 15.755 \times 3 = 15.755 \text{ kN/m}^2$$

$$p_{a_3} = p_{a_2} = 15.755 \text{ kN/m}^2$$

$$p_{a_4} = K_a \gamma' H_2 = \frac{1}{3} \times 9.81 \times 7 = 22.89 \text{ kN/m}^2$$

$$p_{a_5} = \gamma_w \cdot H_2 = 9.81 \times 7 = 68.67 \text{ kN/m}^2$$

$$P_1 = p_{a_1} \times \text{H} = 4.67 \times 10 = 46.7 \text{ kN/m}$$

$$P_2 = \frac{1}{2} \cdot p_{a_2} \times H_1 = \frac{1}{2} \times 15.755 \times 3 = 23.633 \text{ kN/m}$$

$$P_3 = p_{a_3} H_2 = 15.755 \times 7 = 110.285 \text{ kN/m}$$

$$P_4 = \frac{1}{2} \times p_{a_4} \cdot H_2 = \frac{1}{2} \times 22.89 \times 7 = 80.115 \text{ kN/m}$$

$$P_5 = \frac{1}{2} \times p_{a_5} H_2 = \frac{1}{2} \times 68.67 \times 7 = 240.345 \text{ kN/m}$$

$\therefore$ Total $P_a = 46.7 + 23.633 + 110.285 + 80.115 + 240.345 = 501.08$ kN/m

13. (b)

Given:

Weight of pile, P_s = 22 kN

Shaft diameter, D_0 = 340 mm

Under-ream dia, D_u = 700 mm

Undrained shear strength, C = 60 kPa

α = 0.3, $N_C = 9$

Ultimate tensile capacity will be due to

1. Friction along the length of pile (P_1)
2. Bearing action caused by under-reamed portion (P_2)
3. Self weight of pile (P_3)

Tensile capacity due to friction

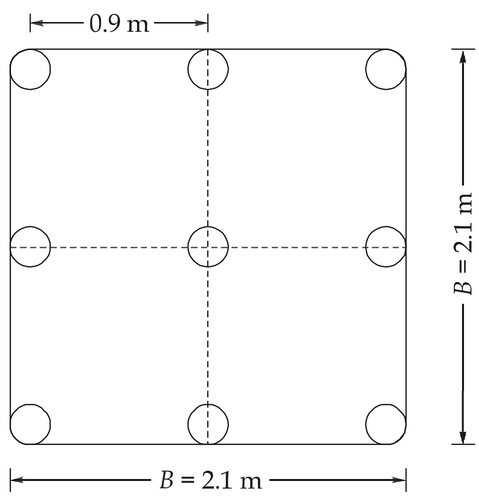
$$P_1 = f_s \times A_s$$
$$= \alpha \,.\, C\,(\pi D_0)\,(L - \text{Depth of under-ream})$$
$$= 0.3 \times 60 \times \pi \times 0.34 \times (10 - 0.42) = 184.19 \text{ kN}$$

Tensile capacity due to bearing action

$$P_2 = C N_C \cdot A$$
$$= \frac{60 \times 9\pi(D_u^2 - D_o^2)}{4} = \frac{60 \times 9 \times \pi(0.7^2 - 0.34^2)}{4} = 158.79 \text{ kN}$$

$\therefore$
$$P = P_1 + P_2 + P_3$$
$$= 184.19 + 158.79 + 22 = 364.98 \simeq 365 \text{ kN}$$

14. (c)



$$C_u = \frac{q_u}{2} = \frac{1.5}{2} = 0.75 \text{ kg/cm}^2 = 7.5 \text{ t/m}^2$$
$$B = 2 \times 0.9 + 0.3 = 2.1 \text{ m}$$

(a) Pile acting individually

$$P_n = n \cdot \alpha \cdot C \cdot A_s$$
$$= 9 \times 0.9 \times 7.5 \times (\pi \times 0.3 \times 10)$$
$$= 572.6 \text{ t}$$

(b) Piles acting in a group

$$P_g = C\,(4\,\text{BL}) = 7.5 \times 4 \times 2.1 \times 10 = 630 \text{ t}$$

$\therefore$ Efficiency for pile group,

$$\eta = \frac{P_g}{P_n} = \frac{630}{572.6} = 1.1$$

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15. (c)

- Local shear failure, generally occurs in soil having somewhat plastic stress-strain curve e.g., loose sand and soft clays.
- Cyclic pile load test is carried out when it is required to required to determine, skin friction and end bearing capacity separately for a pile load on a single pile.

16. (b)

$$q_u = 1.3\, C\, N_C + \gamma D_f N_q + 0.4\, \gamma B N_\gamma \cdot R_\gamma$$

$$C = 0$$

$$\therefore \quad q_u = \gamma D_f N_q + 0.4\, \gamma B N_\gamma \cdot R_\gamma$$

$$R_\gamma = 0.5\left(1+\frac{D}{B}\right) = 0.5\left(1+\frac{2.5}{3}\right) = 0.917$$

$$\therefore \quad q_u = 18 \times 1 \times 21 + 0.4 \times 20 \times 3 \times 17 \times 0.917$$

$$= 752.136 \text{ kN/m}^2$$

$$\therefore \quad q_{nu} = q_u - \gamma D_f = 752.136 - 18 \times 1$$

$$= 734.136 \text{ kN/m}^2$$

$$\therefore \quad q_{ns} = \frac{q_{nu}}{\text{FOS}} = \frac{734.136}{2.5} = 293.65 \text{ kN/m}^2$$

17. (c)

Shrinkage limit, $w_s = w_1 - \Delta w$

$$= w_1 - \frac{\Delta V \cdot \rho_w}{M_S}$$

$$= \frac{M_1 - M_d}{M_d} - \frac{(V_1 - V_d)\rho_w}{M_d}$$

$$= \frac{55.4 - 39.8}{39.8} - \frac{(29.2 - 21.1) \times 1}{39.8} = 0.188$$

i.e., $w_s = 18.8\%$

18. (a)

$$S \times e = wG$$

$$\Rightarrow \quad e = \frac{2.7 \times 0.2222}{1} \simeq 0.6$$

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w = \left(\frac{2.7+0.6}{1+0.6}\right) \times 10$$

$$= 20.625 \text{ kN/m}^3$$

Effective stress at centre of clay layer

$$\bar{\sigma}_0 = (18 - 10) \times 2 + (20.625 - 10) \times 0.5$$

$$\Rightarrow \quad \bar{\sigma}_0 = 21.31 \text{ kN/m}^2$$

Load distribution dimensions at the centre of clay layer = 2 + 0.75 + 0.75 = 3.5 m

Increase in stress due to load = $\dfrac{200}{3.5 \times 3.5}$

$$\Delta\sigma = 16.33 \text{ kN/m}^2$$

$$\Delta H = \frac{C_c H}{1+e}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\sigma}{\bar{\sigma}_0}\right)$$

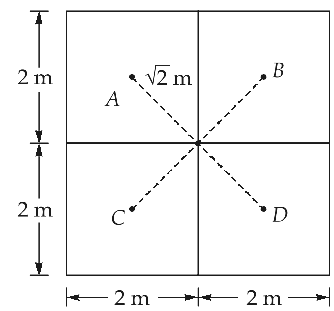
$$= \frac{0.4\times 1}{1+0.6}\log_{10}\left(\frac{21.31+16.33}{21.31}\right)$$

$$= 0.06176 \text{ m} = 61.67 \text{ mm}$$

19. (b)

Total load, $Q = 200 \times 4 \times 4 = 3200$ kN

Divide this load in four equal squares of 2 m × 2 m size, as shown in figure,



∴ Load in each part square = $\frac{3200}{4} = 800$ kN

The distance from A to O i.e. $AO = \sqrt{2}$ m

By symmetry, the stress σ_z at O at 4 m depth is four times of that caused by one load.

$$\sigma_z = \frac{4\times 800}{4^2}\times\frac{3}{2\pi}\times\left[\frac{1}{1+\left(\frac{\sqrt{2}}{4}\right)^2}\right]^{5/2}$$

$$= 71.136 \text{ kN/m}^2 \simeq 71.14 \text{ kN/m}^2$$

20. (c)

$$L = 10 \text{ m}, d = 0.3 \text{ m}, W = 25 \text{ kN}, H = 2 \text{ m}$$

Penetration in 5 blows = 40 mm

∴ In 1 blow, penetration i.e. $S = \frac{40}{5} = 8$ mm = 0.8 cm

$$Q_{\text{safe}} = \frac{WH}{S+C}\times\frac{1}{\text{FOS}}$$

$$= \frac{1}{6}\left[\frac{25\times 2}{0.8+2.5}\right]\times 100 = 252.53 \text{ kN}$$

21. (a)

As more than 50% is retained on 75 μ IS sieve, the soil is coarse-grained.

Coarse fraction = 100 – 45 = 55%

Gravel fraction = 100 – 60 = 40%

Sand fraction = 55 – 40 = 15%

As more than half the coarse fraction is larger than 4.75 mm sieve, the soil is gravel.

Also, $I_P = w_L - w_P = 40 - 12 = 28\%$

A-line, $I_P = 0.73\,(w_L - 20)$

$= 0.73\,(40 - 20) = 14.6\%$

∵ I_P is above A-line, therefore the soil should be GC as per IS classification.

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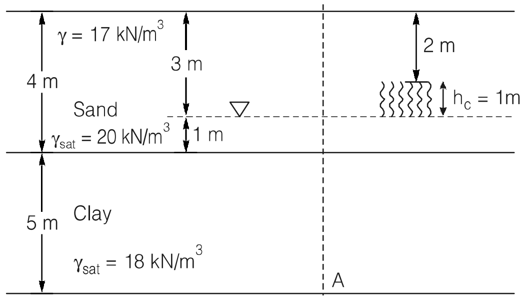
22. (b)



Effective stress at A,

$$(\bar{\sigma}_A)_i = 3\gamma_{sand} + 1(\gamma_{sat} - \gamma_w)_{sand} + 5\,(\gamma_{sat} - \gamma_w)_{clay}$$
$$= 3 \times 17 + 1\,(20 - 9.81) + 5\,(18 - 9.81)$$
$$= 51 + 10.19 + 40.95$$
$$= 102.14 \text{ kN/m}^2$$

If the soil gets saturated by capillary action then,

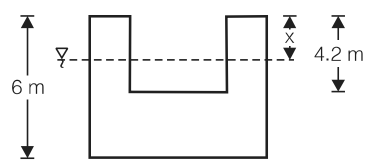
$$(\bar{\sigma}_A)_f = 2\,\gamma_{sand} + 1\,\gamma_{sat.,\,sand} + 1\gamma'_{sand} + 5\gamma'_{clay}$$
$$= 2 \times 17 + 1\,(20) + (20 - 9.81) + 5(18 - 9.81)$$
$$= 34 + 20 + 10.19 + 40.95$$
$$= 105.14 \text{ kN/m}^2$$

$\therefore$ Increase in effective stress at A = $(\bar{\sigma}_A)_f - (\bar{\sigma}_A)_i$ = 105.14 – 102.14

$$= 3 \text{ kN/m}^2$$

23. (b)

Let x be the depth of ground water table initially.



Total upward water force on the sand stratum at the bottom of excavation

$$= (6 - x) \times \gamma_w$$

Total downward force at the bottom of excavation

= Weight of soil in saturated condition

$$= \gamma_{sat} \times (6 - 4.2)$$
$$= (\gamma_{sub} + \gamma_w) \times 1.8$$

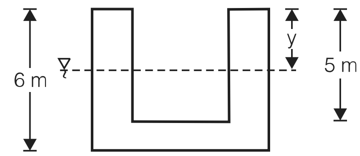
When quicksand condition occurs, then total upward water force becomes equal to total downward force i.e.

$$(6 - x) \times \gamma_w = (\gamma_{sub} + \gamma_w) \times 1.8$$

$\Rightarrow$ $$(6 - x) \times 10 = (11 + 10) \times 1.8$$

$\Rightarrow$ $$x = 6 - \frac{21 \times 1.8}{10}$$

$\Rightarrow$ $$x = 2.22 \text{ m}$$



Similarly, if the depth of displaced ground water table is y from the surface, then

$$(6 - y) \times \gamma_w = (6 - 5) \times (\gamma_{sub} + \gamma_w)$$

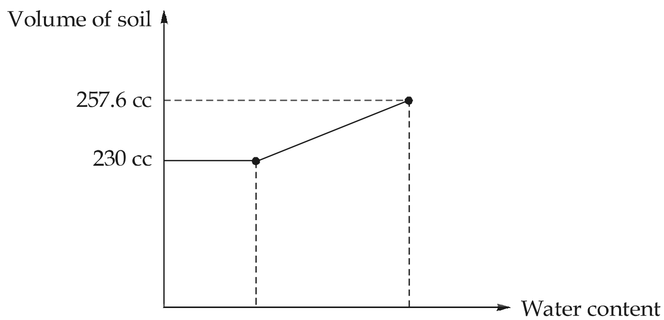
$\Rightarrow$ $$(6 - y) \times 10 = 1 \times (11 + 10)$$

$\Rightarrow$ $$y = 6 - 2.1 = 3.9 \text{ m}$$

$\therefore$ Lowering of ground water table required $= y - x$

$$= 3.9 - 2.2 = 1.68 \text{ m}$$

24. (d)



$w_s = 0.151$ $w_1 =$ unknown

$w_{soil} = 450$ gm $w_{solid} = 391$ gm

$w_{solid} = 391$ gm

$w_w = 450 - 391 = 59$ gm

$w_s = 0.51$

$$\therefore \quad \gamma_d = \frac{391}{230} = 1.7 \text{ g/cc}$$

$$V_2 = 1.12 \times 230 = 257.6 \text{ cc}$$

{$\because$ 12% increment in original volume}

Now, shrinkage ratio, $$R = G_D = \frac{\gamma_d}{\gamma_w} = 1.7$$

We, also know, $$R = \frac{\frac{V_1 - V_s}{V_s}}{w_1 - w_s}$$

$\Rightarrow$ $$1.7 = \frac{\frac{(257.6 - 230)}{230}}{w_1 - 0.151}$$

$\Rightarrow$ $$w_1 = 0.22 \simeq 22\%$$

$$w_w = w_1 \times w_{solid}$$

$\therefore$ $$w_w = w_1 \times w_{solid} = 0.22 \times 391 = 86.64 \text{ gm}$$

$$\Delta w_w = 86.64 - 59 = 27.02 \text{ gm}$$

$$\Delta V_w = 27.64 \text{ cc} \qquad (\because \gamma_w = 1 \text{ g/cc})$$

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25. (a)

σ_3 = 1000 kPa, $\sigma_3' = \sigma_3 - u$; σ_d = 1600 kPa; $\sigma_1' = \sigma_1 - u$; σ_1 = 2600 kPa; as ($\sigma_1 = \sigma_3 + \sigma_d$)

We know,

$$\sigma_1' = \sigma_3' \tan^2\left(45+\frac{\phi'}{2}\right)+2c'\tan\left(45+\frac{\phi'}{2}\right)$$

$$(\sigma_1 - u) = (\sigma_3 - u)\tan^2\left(45+\frac{\phi'}{2}\right)+2c'\tan\left(45+\frac{\phi'}{2}\right)$$

$$\Rightarrow \quad (2600 - u) = (1000-u)\tan^2\left(45+\frac{25}{2}\right)+2(220)\tan\left(45+\frac{25}{2}\right)$$

$$\Rightarrow \quad (2600 - u) = (1000 - u)(2.463) + 691$$

$$\Rightarrow \quad 2600 - u = 2463 - 2.463u + 691$$

$$\Rightarrow \quad 1.463u = 2463 + 691 - 2600$$

$$\Rightarrow \quad u = 378.67 \text{ kPa} \simeq 378 \text{ kPa (say)}$$

26. (a)

$$\text{FOS} = \frac{c' + \gamma' H\cos^2\beta\tan\phi}{\gamma_{sat} H\cos\beta\sin\beta}$$

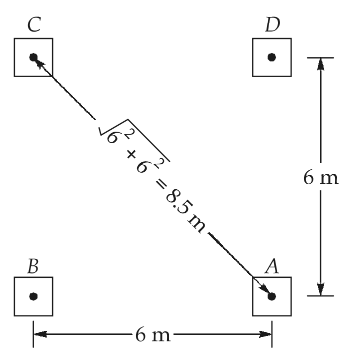
For FOS = 1, at $H = H_C$

$$\therefore \quad H_C = \frac{C'}{\cos^2\beta[\gamma_{sat}\tan\beta - \gamma'\tan\phi']}$$

$$\Rightarrow \quad H_C = \frac{12}{\cos^2(18°)[19\tan 18° - (19-9.81)\tan 15°]}$$

$$\Rightarrow \quad H_C = 3.57 \text{ m}$$

27. (a)



For footing A, B, C, D, the $\frac{r}{Z}$ ratios are 0, $\frac{6}{3}, \frac{6}{3}$ and $\frac{8.5}{3}$ respectively.

$$\sigma_z = \frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{r}{Z}\right)^2}\right]^{5/2}\frac{Q}{Z^2}$$

$\therefore$ $$\sigma_z = \frac{3Q}{2\pi Z^2}\left[\left(\frac{1}{1+(0)^2}\right)^{5/2} + 2\left(\frac{1}{1+\left(\frac{6}{3}\right)^2}\right)^{5/2} + \left(\frac{1}{1+\left(\frac{8.5}{3}\right)^2}\right)^{5/2}\right]$$

$\Rightarrow$ $$\sigma_z = \frac{3\times600}{2\pi(3)^2}[1.0398]$$

$\Rightarrow$ $$\sigma_z = 33.09 \text{ kN/m}^2 \simeq 33 \text{ kN/m}^2$$

28. (d)

For test 1

$$\sigma_3 = 200 \text{ kPa};\ \sigma_d = 600 \text{ kPa};\ \sigma_1 = 200 + 600 = 800 \text{ kPa}$$

For test 2

$$\sigma_3 = 350 \text{ kPa};\ \sigma_d = 1050 \text{ kPa};\ \sigma_1 = 350 + 1050 = 1400 \text{ kPa}$$

We know $$\sigma_1 = \sigma_3 \tan^2\left(45+\frac{\phi}{2}\right) + 2C\tan\left(45+\frac{\phi}{2}\right)$$

$$\sigma_1 = \sigma_3\tan^2\alpha + 2C\tan\alpha$$

Let $$\tan\alpha = X$$

$\therefore$ $$\sigma_1 = \sigma_3 X^2 + 2CX$$

$$800 = 200X^2 + 2CX \quad ...(1)$$

$$1400 = 350X^2 + 2CX \quad ...(2)$$

$$-600 = -150X^2 \quad \Rightarrow \quad X = 2$$

$\therefore$ $$\tan\left(45+\frac{\phi}{2}\right) = 2$$

$\therefore$ $$\phi = 36.87°$$

$\therefore$ $$C = 0$$

At X-X section

$$\tau = C + \sigma'\tan\phi$$

$\Rightarrow$ $$\tau = 0 + [(17.5\times2) + (21.5-9.81)(3)]\tan 36.87°$$

$\Rightarrow$ $$\tau = 52.55 \text{ kPa}$$

29. (b)

As the flow is in upward direction.

$\therefore$ $$P_H = H_1 + Z + iZ$$

Where, H_1 = Height of water above soil surface = 2 m

Z = Vertical depth of section = 1 m

i = Hydraulic gradient = $\frac{\Delta h}{L} = \frac{2}{4} = 0.5$

$\therefore$ $$P_H = 2 + 1 + 0.5\,(1)$$

$\Rightarrow$ $$P_H = 3.5 \text{ m}$$

Datum head = 3 m

$\therefore$ Total head = 3 + 3.5 = 6.5 m

Head loss = Total available head – Total head at P

= (4 + 2 + 2) – (7.5) = 1.5 m

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30. (b)

For sample A:
$\sigma_d = 105$ kPa
$\sigma_3 = 150$ kPa
$\sigma_1 = \sigma_3 + \sigma_d = 255$ kPa

i.e. $\sigma_1 = \sigma_3 \tan^2 \theta_c + 2c \tan \theta_c \left[\text{where } \theta_c \left(45° + \frac{\phi}{2}\right)\right]$

$\Rightarrow \quad \sigma_1 = \sigma_3 X^2 + 2c X$

$\Rightarrow \quad 255 = 150 X^2 + 2c X \quad \text{...(i)}$

For sample B:
$\sigma_d = 200$ kPa
$\sigma_3 = 300$ kPa
$\sigma_1 = \sigma_3 + \sigma_d = 500$ kPa

i.e. $\sigma_1 = \sigma_3 \tan^2 \theta_c + 2c \tan \theta_c$

$\sigma_1 = \sigma_3 X^2 + 2c X$

$500 = 300 X^2 + 2c X \quad \text{...(ii)}$

Solving equations (i) and (ii)

$$255 = 150 X^2 + 2c X$$
$$\underline{-}500 = \underline{-}300 X^2 \underline{-} 2c X$$
$$-245 = -150 X^2$$

$\Rightarrow \quad X^2 = 1.633$

$\therefore \quad X = \tan\left(45° + \frac{\phi}{2}\right) = 1.278$

$\therefore \quad \phi = \left[\tan^{-1}(1.278) - 45\right] \times 2 = 13.92°$

Put value of ϕ in equation (i)

$$255 = 150 \tan^2\left(45° + \frac{13.92}{2}\right) + 2c \tan\left(45° + \frac{13.92}{2}\right)$$

$\Rightarrow \quad c = 3.899 \text{ kPa} \simeq 3.9 \text{ kPa}$

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