CLASS TEST

S.No. : 01_IG_CE_A+C_140423

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SOIL MECHANICS

CIVIL ENGINEERING

Date of Test : 14/04/2023

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (a)

Equivalent permeability,

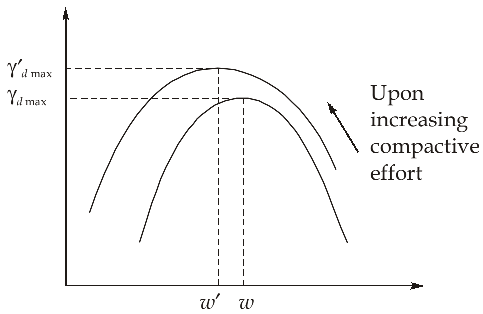
$$k_e = \sqrt{k_x k_y}$$

$$13 \times 10^{-7} = \sqrt{3\times10^{-7} \times k_y}$$

$$k_y = \frac{169\times10^{-14}}{3\times10^{-7}}$$

$$k_y = 5.63 \times 10^{-6} \text{ cm/s}$$

2. (b)



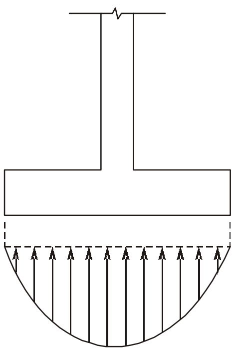
$$\gamma'_{d\max} > \gamma_{d\max} \text{ and } w' < w$$

3. (c)

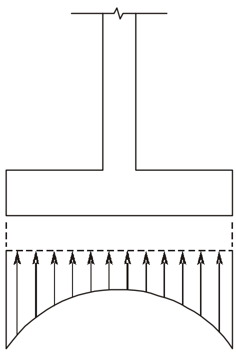
$$S_n = \frac{C}{\gamma H} = \frac{60}{18\times6} = 0.56$$

4. (d)

Contact pressure distribution:



Rigid footing on granular soil



Rigid footing on clayey soil

5. (d)

As per IS : 2911 (Part-1) 2010

Minimum spacing for point bearing piles = 2.5 D

and Minimum spacing for friction piles = 3 D

6. (c)

$$B = \frac{\Delta U_3}{\Delta \sigma_3} = \frac{0.19 - 0.09}{0.3 - 0.1} = \frac{0.1}{0.2} = 0.5$$

7. (a)

$$\begin{aligned} i_c &= (G - 1)\,(1 - n) \\ &= (2.5 - 1)\,(1 - 0.5) \\ &= 1.5 \times 0.5 \\ &= 0.75 \end{aligned}$$

8. (b)

$$Ar = \frac{D_0^2 - D_i^2}{D_i^2} = \frac{50^2 - 40^2}{40^2} = 0.5625 \text{ or } 56.25\% \simeq 56\%$$

9. (a)

$$k_0 = \frac{\mu}{1-\mu} = \frac{0.25}{1 - 0.25} = \frac{1}{3}$$

10. (d)

In plate load test:

For clayey soil, $q_{uf} = q_{up}$

Given, $q_{up} = 180 \text{ kN/m}^2$

$\therefore$ $q_{uf} = 180 \text{ kN/m}^2$

$\therefore$ Difference of ultimate bearing capacity of foundation and plate = 0

Note: For cohesionless soil,

$$q_{uf} = q_{up} \times \frac{B_f}{B_p}$$

11. (d)

$$\text{Factor of safety } F = \left(1 - \frac{\gamma_w h}{\gamma_{avg} z}\right)\frac{\tan\phi}{\tan i}$$

$$\gamma_{\text{avg}} = \frac{20 \times 5 + 15 \times 5}{10} = 17.5 \text{ kN/m}^3$$

$$\therefore \quad F = \left(1 - \frac{10 \times 5}{17.5 \times 10}\right) \times \frac{\tan 45^\circ}{\tan 30^\circ} = \frac{5\sqrt{3}}{7} = 1.24$$

12. (d)

The effect of overburden pressure on SPT value may be approximated by the equation.

$$N = N'\left(\frac{350}{\bar{\sigma} + 70}\right)$$

$\bar{\sigma}$ = Effective overburden pressure at test level

$= 18 \times 6 = 108 \text{ kN/m}^2 \not> 280 \text{ kN/m}^2$ (OK)

$$\therefore \quad N = 28 \times \left(\frac{350}{108 + 70}\right) = 55$$

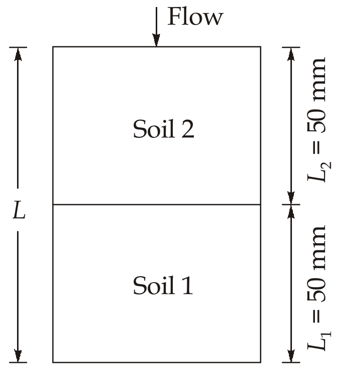
13. (c)

Case-1: Soil sample I

$$k_1 = \frac{aL_1}{At_1}\ln\left(\frac{h_1}{h_2}\right)$$

$$\Rightarrow \quad \frac{L_1}{k_1} = \frac{At_1}{a\ln\left(\frac{h_1}{h_2}\right)} = \frac{4560\times 60}{130\ln\left(\frac{900}{135}\right)} = 1109.37 \text{ sec}$$

Case-2: Soil sample II placed on soil sample I for permeability test on both soils



$$k_v = \frac{aL}{At}\ln\left(\frac{h_1}{h_2}\right)$$

$$\Rightarrow \quad k_v = \frac{130\times L}{4560\times 140}\ln\left(\frac{900}{135}\right)$$

$$\Rightarrow \quad \frac{L}{k_v} = 2588.54 \text{ sec}$$

$$\because \quad \frac{L}{k_v} = \frac{L_1}{k_1}+\frac{L_2}{k_2}$$

$$\Rightarrow \quad 2588.54 = 1109.37+\frac{50}{k_2}$$

$$\Rightarrow \quad k_2 = 0.0338 \text{ mm/s}$$

14. (d)

$$\because \quad T_v = \frac{C_v t}{H^2}$$

$$\Rightarrow \quad T_v = \frac{5\times 10^{-2}\times 3\times 365\times 24\times 60\times 60}{\left(\frac{6500}{2}\right)^2} = 0.4478$$

But $\quad T_v = \frac{\pi}{4}\left(\frac{U_z}{100}\right)^2 = 0.4478$ for $U_z < 60\%$

$$\Rightarrow \quad U_z = 75.51\% > 60\%$$

Thus, $$T_v \neq \frac{\pi}{4}\left(\frac{U_z}{100}\right)^2$$

$\therefore$ $$T_v = 1.781 - 0.933 \log_{10}(100 - U_z)$$

$\Rightarrow$ $$0.4478 = 1.781 - 0.933 \log_{10}(100 - U_z)$$

$\Rightarrow$ $$100 - U_z = 26.85$$

$\Rightarrow$ $$U_z = 73.15\% > 60\% \quad \text{(OK)}$$

Thus consolidation due to fill material is 73.15%.

15. (b)

Cu test is conducted on a normally consolidated clay, thus

$$C' = 0$$

$$\sigma_1 = \sigma_3 + \sigma_d = 75 + 60 = 135 \text{ kPa}$$

$$\bar{\sigma}_1 = \sigma_1 - U = 135 - 35 = 100 \text{ kPa}$$

$$\bar{\sigma}_3 = \sigma_3 - U = 75 - 35 = 40 \text{ kPa}$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2\left(45° + \frac{\phi}{2}\right)$$

$\Rightarrow$ $$100 = 40\tan^2\left(45° + \frac{\phi'}{2}\right)$$

$\Rightarrow$ $$\phi' = 25.38°$$

16. (c)

Given: $$B = 5 \text{ m}, L = 7 \text{ m}, D = 2 \text{ m}, C = 60 \text{ kN/m}^2$$

$\because$ $$q_u = \left(1 + 0.3\frac{B}{L}\right)CN_c + qN_q + 0.5\left(1 - 0.2\frac{B}{L}\right) \times BN_\gamma \gamma$$

In case of clays $$N_c = 5.7, N_q = 1, N_\gamma = 0$$

$\therefore$ $$q_u = \left(1 + 0.3 \times \frac{5}{7}\right) \times 60 \times 5.7 + 1 \times \gamma D_f + 0$$

$\therefore$ $$q_{u\ net} = q_u - \gamma D_f$$

$$= \left(1 + 0.3 \times \frac{5}{7}\right) \times 60 \times 5.7$$

$$= 415.29 \text{ kN/m}^2$$

17. (d)

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1 + e_0}$$

$\Rightarrow$ $$\frac{35 - 25}{35} = \frac{1.2 - e_f}{1 + 1.2}$$

$\Rightarrow$ $$e_f = 0.57$$

18. (b)

$$\gamma_{sat} = 17.5 \text{ kN/m}^3 \qquad \text{(Given)}$$
$$\gamma' = \gamma_{sat} - \gamma_w$$
$$= 17.5 - 9.81$$
$$= 7.69 \text{ kN/m}^3$$
$$\therefore \quad i_c = \frac{\gamma'}{\gamma_w} = \frac{7.69}{9.81} = 0.784$$

19. (b)

Given:

% passing 75 micron sieve = 30%, w_L = 40%, w_p = 20%

$$\text{Group index, GI} = 0.2a + 0.005ac + 0.01\,bd$$

where
$$a = \text{\% passing 75 micron} - 35 \not> 40$$
$$b = \text{\% passing 75 micron} - 15 \not> 40$$
$$c = w_L - 40 \not> 20$$
$$d = I_p - 10 \not> 20$$

So,
$$a = 30 - 35 = -5 = 0 \qquad \text{(as negative value not possible)}$$
$$b = 30 - 15 = 15$$
$$c = 40 - 40 = 0$$
$$d = (w_L - w_p) - 10 = 20 - 10 = 10$$

So,
$$\text{GI} = 0.2a + 0.005ac + 0.01 \times bd$$
$$= 0.01 \times 15 \times 10 = 1.5$$

20. (c)

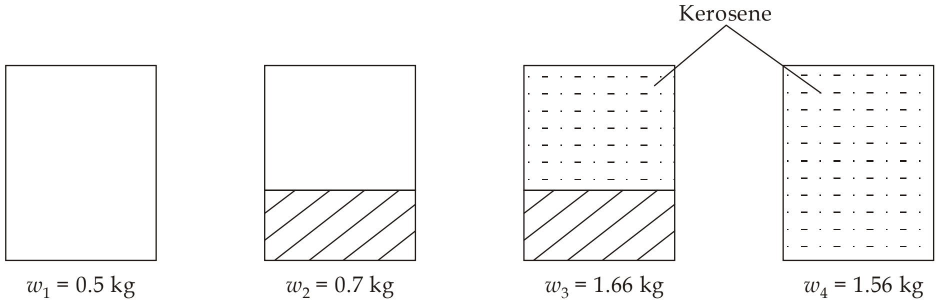
- Local shear failure, generally occurs in soil having somewhat plastic stress-strain curve e.g., loose sand and soft clays.
- Cyclic pile load test is carried out when it is required to determine, skin friction and end bearing capacity separately for a pile load on a single pile.

21. (c)

$$q_{safe} = \frac{q_{nu}}{FOS} + \gamma D_f$$
$$= \frac{160}{3} + 18 \times 1.5 = 80.33 \text{ kN/m}^2$$

$\therefore$ Allowable load,
$$Q = q_{safe} \times \text{Area}$$
$$= 80.33 \times 3 \times 3 = 723 \text{ kN}$$

22. (c)

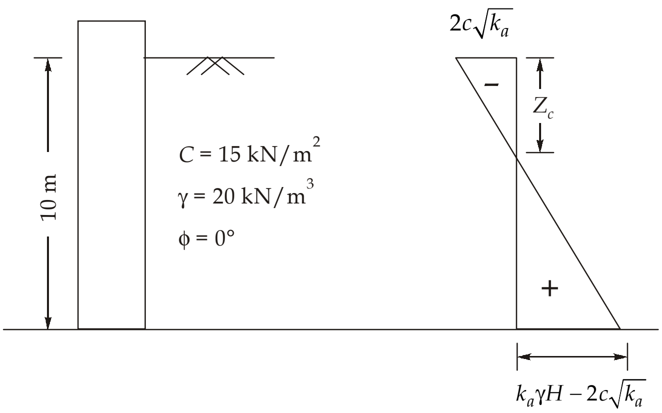


$$G_s = \left(\frac{w_2 - w_1}{w_4 - w_3 + w_2 - w_1}\right) \times G_k$$

$$= \left(\frac{0.7 - 0.5}{1.56 - 1.66 + 0.7 - 0.5}\right) \times 0.8 = \left(\frac{0.2}{0.1}\right) \times 0.8$$

$$G_s = 1.6$$

23. (c)



Active earth pressure coefficient,

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

$$k_a = 1$$

For $Z_c \Rightarrow$

$$k_a \gamma Z_c = 2c\sqrt{k_a}$$

$$Z_c = \frac{2c}{\gamma\sqrt{k_a}}$$

$$Z_c = \frac{2 \times 15}{20 \times 1} = 1.5 \text{ m}$$

If tension cracks are developed,

Active thrust,

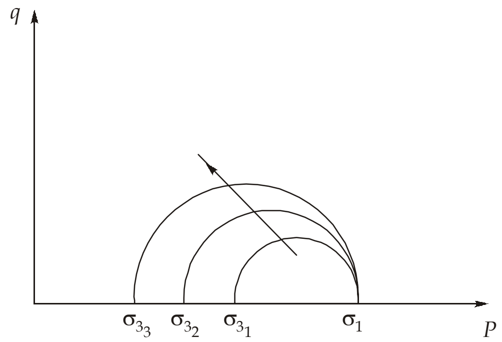
$$P_a = \frac{1}{2} \times (H - Z_c) \times \left(k_a \gamma H - 2c\sqrt{k_a}\right)$$

$$P_a = \frac{1}{2} \times 8.5 \times 170$$

$$P_a = 722.5 \text{ kN/m}$$

24. **(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.



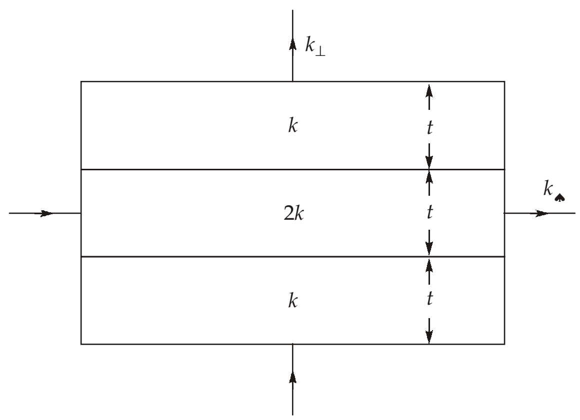
25. **(c)**

- 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.

26. **(d)**

- Pycnometer test is most suitable for cohesionless soils as it is difficult to saturate the fine-grained soil completely.
- Calcium carbide test is the quickest field test. It takes only 5-7 minutes.

27. **(c)**



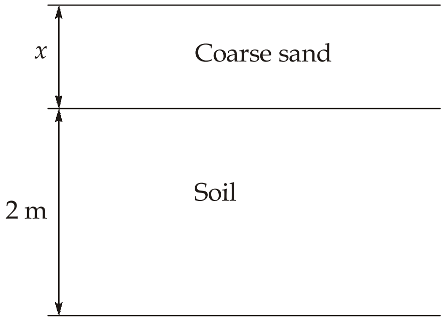
$$k_\perp = \frac{\Sigma k_i z_i}{\Sigma z_i} = \frac{kt + 2kt + kt}{3t} = \frac{4kt}{3t} = \frac{4k}{3}$$

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

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

28. **(a)**

29. **(b)**



$$i_{cr} = \frac{G-1}{1+e} = (G-1)(1-n)$$

$$\Rightarrow \quad i_{cr} = (2.7 - 1)\ (1 - 0.3)$$

$$\Rightarrow \quad i_{cr} = 1.19$$

$$i_{\text{allowable}} = \frac{1.19}{\text{FOS}}$$

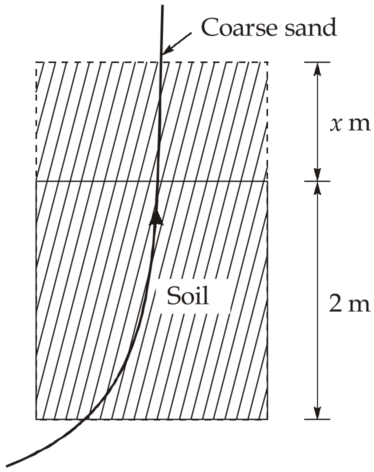
$$\Rightarrow \quad i_{\text{allowable}} = \frac{1.19}{1.5} = 0.7933$$

$$\therefore \quad (2 + x) \times 0.7934 = 1.90$$

$$\Rightarrow \quad 2 + x = 2.395$$

$$\Rightarrow \quad x = 0.395 \text{ m} \simeq 0.4 \text{ m}$$

Alternatively



Given, $n = 0.3,\ G_s = 2.7$

Seepage head, $H_L = 1.9$ m

$$\text{FOS} = 1.5$$

$$\text{FOS} = \frac{\text{Buoyand weight}}{\text{Seepage force}}$$

$$= \frac{\gamma_{\text{sub}}(2+x)A}{\gamma_w h_L A} = \frac{\left(\dfrac{G_s - 1}{1+e}\right)\gamma_w (2+x)}{\gamma_w \times h_L}$$

$$= (G_s - 1)\frac{(1-n)(2+x)}{h_L}$$

$$1.5 = (2.7-1)\frac{(1-0.3)(2+x)}{1.9}$$

$$(2 + x) = \frac{1.5 \times 1.9}{1.7 \times 0.7}$$

$$x = \frac{1.5 \times 1.9}{1.7 \times 0.7} - 2$$

$$= 0.395 \simeq 0.4 \text{ m}$$

30. (c)

$$D_{60} = 0.45 \text{ mm}$$
$$D_{30} = 0.2 \text{ mm}$$
$$D_{10} = 0.04 \text{ mm}$$

$$\therefore \text{Uniformity coefficient, } C_u = \frac{D_{60}}{D_{10}} = \frac{0.45}{0.04} = 11.25$$

$$\text{Coefficient of curvature, } C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} = \frac{0.2^2}{0.45 \times 0.04} = 2.22$$

■■■■