CLASS TEST

S.No. : 04 IGCE_IJKL_300823

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SOIL MECHANICS

CIVIL ENGINEERING

Date of Test : 30/08/2023

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (c)

Coefficient of curvature, C_c = 1.2

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

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

$$\Rightarrow \quad D_{60} = \frac{3.2^2}{1.2 \times 0.6} = 14.22 \text{ mm}$$

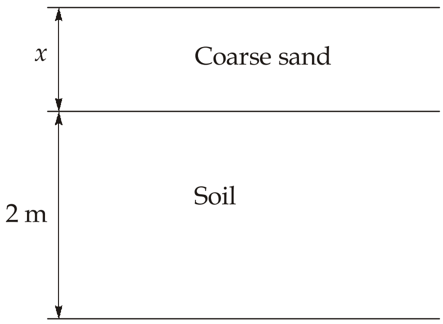
So,
$$C_u = \frac{D_{60}}{D_{10}} = \frac{14.22}{0.6} = 23.70$$

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

Hence, the soil is well graded sand.

2. (b)

Critical hydraulic gradient,



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

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

$$\Rightarrow \quad = 1.19$$

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

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

$$\therefore \quad (2 + x) \times 0.7933 = 1.90$$

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

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

3. (b)

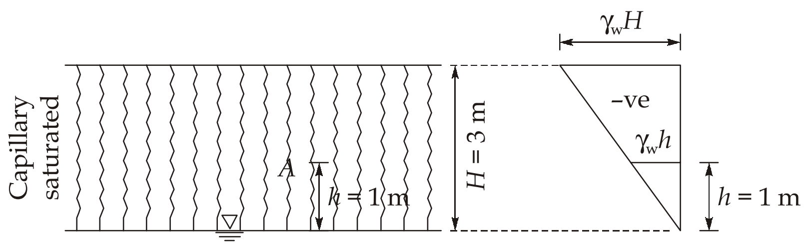
For Taylor's square root of time fitting method,

Degree of consolidation, U = 90%

For U = 90% > 60%

$$T_v = 1.781 - 0.933 \times \log_{10}(100 - \%U)$$
$$= 1.781 - 0.933 \times \log_{10}(100 - 90)$$
$$= 0.848$$

4. (c)



Pore water pressure 1m above G.W.T. is

$$u = -\gamma_w h$$

For h = 1 m, $\quad u = -10 \times 1$

$\Rightarrow \quad u = -10 \text{ kN/m}^2$

5. (b)

Given, $\quad U_r = 30\%$

$$U = 70\%$$

We know, $\quad (1 - U) = (1 - U_r)(1 - U_v)$

$\Rightarrow \quad (1 - 0.7) = (1 - 0.3)(1 - U_v)$

$\Rightarrow \quad U_v = 1 - \dfrac{0.3}{0.7} = \dfrac{4}{7}$

$\Rightarrow \quad U_v = 57.14\%$

6. (c)

For normally consolidated clay,

Given, $\quad \tau_f = \bar{\sigma}\tan 30°$

$\Rightarrow \quad \phi = 30° \qquad [\because C' = 0]$

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

$$\bar{\sigma}_1 = 69\tan^2\left(45° + \frac{30°}{2}\right)$$

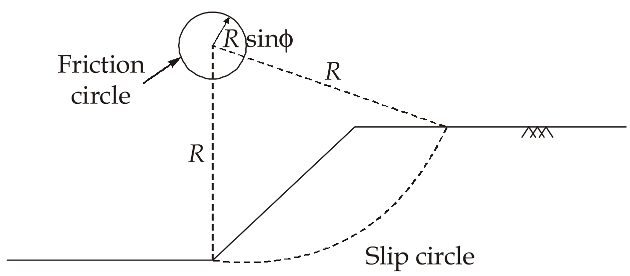
$\Rightarrow \quad \bar{\sigma}_1 = 207 \text{ kN/m}^2$

$\therefore \quad (\Delta\bar{\sigma}_d)_f = \bar{\sigma}_1 - \bar{\sigma}_3 = 207 - 69 = 138 \text{ kN/m}^2$

7. (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 $R \sin\phi$, which is concentric with trial slip circle.



8. (b)

The ratio of τ_{ff} and σ_{ff} is maximum on the plane of maximum obliquity and not on the plane of maximum shear stress.

9. (d)

To prevent possibility of erosion and piping, filter must have grain sizes that satisfy following requirements:

(i) $$\frac{D_{15\,(\text{filter})}}{D_{85(\text{protected material})}} < 5$$

It ensures that no significant invasion of particles from the protected material to the filter.

(ii) $$4 < \frac{D_{15(\text{filter})}}{D_{15(\text{protected material})}} < 20$$

It ensures that sufficient head is lost in filter without build-up of seepage pressure (specifies the lower limit of material.

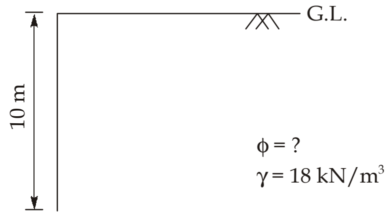
(iii) $$\frac{D_{50(\text{filter})}}{D_{50(\text{protected material})}} < 25$$

This is the additional guideline for the selection of material.

10. (b)

$$p_a = K_a(\gamma z)$$

where K_a is coefficient of active earth pressure



$\Rightarrow$ $$60 = K_a \times 18 \times 10$$

$\Rightarrow$ $$K_a = \frac{1}{3}$$

Now, $$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1}{3}$$

$\Rightarrow$ $$\phi = 30°$$

11. (b)

At liquid limit, $w_L = 60\%$, $V_L = 20$ cc, $S = 1$

At shrinkage limit, $w_s = 28\%$, $V_s = ?$, $S = 1$

We know that, $$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{G\gamma_w}{1+\frac{wG}{S}}$$

and $$\gamma_d = \frac{W_s}{V}$$

So, at liquid limit, $$\frac{W_s}{20} = \frac{G\gamma_w}{1+0.6\times 2.7} \quad \text{...(i)}$$

At shrinkage limit, $$\frac{W_s}{V} = \frac{G\gamma_w}{1+0.28\times 2.7} \quad \text{...(ii)}$$

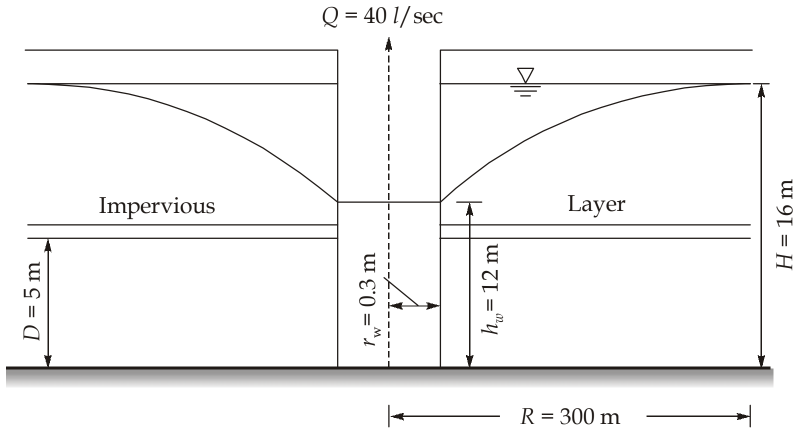
From eq. (i) and (ii)

$$\frac{1+0.28\times2.7}{V} = \frac{1+0.6\times2.7}{20}$$

$$\Rightarrow \quad \frac{V}{20} = \frac{1+0.28\times2.7}{1+0.6\times2.7} = \frac{1.756}{2.62}$$

$$\Rightarrow \quad V = 13.40 \text{ cc}$$

12. (a)



For confined aquifer,

By Dupit's equation, $$k = \frac{2.303 Q \log_{10}\left(\frac{R}{r_w}\right)}{2\pi D(H-h_w)}$$

Given, $Q = 40\ l/\text{sec} = 0.04\ \text{m}^3/\text{sec}$, $R = 300$ m, $r_w = 0.3$ m, $D = 5$ m, $H = 16$ m, $h_w = 12$ m

Hence, $$k = \frac{2.303\times0.04\times\log\left(\frac{300}{0.3}\right)}{2\pi\times5\times(16-12)}$$

$$= 2.199 \times 10^{-3} \text{ m/sec} \simeq 2.2 \text{ mm/sec}$$

13. (a)

Mass of paraffin wax, $M_p = 584.3 - 575.8 = 8.5$ gm

Weight of soil, $W = 575.8$ gm

Volume of paraffin wax, $V_p = \frac{M_p}{G_p \times \rho_w} = \frac{8.5}{0.85\times1} = 10$ ml $\quad (G_p = 0.85)$

Volume of soil, $V = 342 - 10 = 332$ ml

Bulk density, $\rho_b = \frac{W}{V} = 1.734$ g/cc $\quad (\because 1 \text{ ml} = 1 \text{ cc})$

$\therefore$ Dry density, $\rho_d = \frac{\rho_b}{1+w} = \frac{1.734}{1+0.2} = 1.445$ g/cc

14. (d)

The passive earth pressure coefficient is

$$k_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1+0.259}{1-0.259} = 1.699$$

The passive pressure at a depth z is given by,

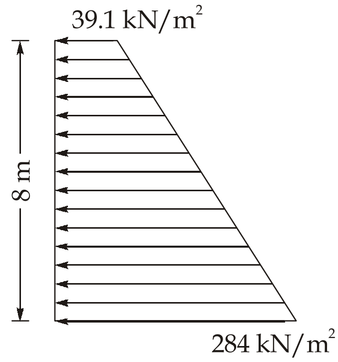
$$p_p = k_p \gamma z + 2C\sqrt{k_p}$$

At z = 0 m, $$p_p = 2\times 15\times\sqrt{1.699} = 39.1 \text{ kN/m}^2$$

At z = 8 m, $$p_p = 18\times 1.699\times 8 + 2\times 15\sqrt{1.699}$$

$$= 283.76 \text{ kN/m}^2$$

$\therefore$ Total passive thrust, $$p_p = \frac{1}{2}[39.1+283.76]\times 8 = 1291.44 \text{ kN/m}$$



15. (d)

For 50% consolidation,

$$T_V = \frac{C_V t}{h^2}$$

$\Rightarrow$ $$C_V = \left(\frac{T_V \times h^2}{t}\right)$$

In the laboratory test,

$$T_{V\,50} = \frac{\pi}{4}\times 0.5^2 = 0.196$$

$$t = 15 \text{ min}$$

$$h = \frac{H}{2} = \frac{3}{2} = 1.5 \text{ cm} \qquad [\because \text{ Double drainage}]$$

So, $$C_V = \frac{T_V \times h^2}{t} = \frac{0.196\times 1.5^2}{15}$$

$\therefore$ $$= 0.0294 \text{ cm}^2/\text{min}$$

In the case of actual building,

$$h = \frac{8\times 100}{2} = 400 \text{ cm}$$

$\therefore$ $$t = \frac{T_V \times h^2}{C_V} = \frac{0.196\times 400^2}{0.0294\times 60\times 24}$$

$\Rightarrow$ $$t = 740.74 \text{ days} \simeq 741 \text{ days}$$

16. (c)

Given, γ_b = 19 kN/m^3, w_1 = 17%

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

Also, $$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.7\times 9.81}{1+e} = 16.24$$

$$\Rightarrow \qquad e = \frac{2.7 \times 9.81}{16.24} - 1 = 0.631$$

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

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

So, new moisture content,

$$w_2 = \frac{S \cdot e}{G} = \frac{1 \times 0.631}{2.7} = 0.2337 \text{ or } 23.37\%$$

$\therefore$ Additional moisture content required

$$= 23.37 - 17 = 6.37\%$$

17. (d)

For constant head permeability test,

$$k = \frac{Q}{Ai} = \frac{626}{\frac{\pi}{4} \times 7.5^2 \times 60 \times \frac{24.7}{18}} = 1.72 \times 10^{-1} \text{ cm/s}$$

Now, Discharge velocity, $V = ki$

$$= 1.72 \times 10^{-1} \times \frac{24.7}{18} = 0.236 \text{ cm/s}$$

Seepage velocity, $\frac{V}{n} = \frac{0.236}{0.44}$

$$V_s = 0.536 \text{ cm/s}$$

18. (c)

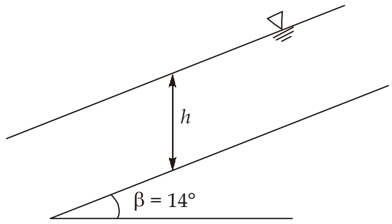
$$q_{u \text{ (undisturbed)}} = \frac{T}{\pi d^2 \left[\frac{h}{2} + \frac{d}{6}\right]} = \frac{35 \times 1000}{\pi \times 60^2 \times \left[\frac{100}{2} + \frac{60}{6}\right]}$$

$$= 0.05158 \text{ N/mm}^2 = 51.58 \text{ kN/m}^2$$

$$q_{u \text{ (remoulded)}} = \frac{5 \times 1000 \times 10^3}{\pi \times 60^2 \left[\frac{100}{2} + \frac{60}{6}\right]} \text{ kN/m}^2 = 7.368 \text{ kN/m}^2$$

$$\therefore \quad \text{Sensitivity of the clay} = \frac{q_{u\text{(undisturbed)}}}{q_{u\text{(remoulded)}}} = \frac{51.58}{7.368} = 7$$

19. (a)



$$\phi = 30°$$
$$\gamma_{sat} = 20 \text{ kN/m}^3$$

FOS against failure =

$$\frac{\gamma_{sub}}{\gamma_{sat}} \times \frac{\tan\phi}{\tan\beta} = \frac{(20-10)}{20} \times \frac{\tan(30°)}{\tan(14°)} = 1.16$$

20. (a)

- Experimentally it has been shown that the compression of soil layer does not stop when excess pore water pressure has dissipated to zero but continues at a gradually decreasing rate at constant effective stress. This phenomenon is known as secondary compression.
- Since, secondary consolidation is not governed by the dissipation of excess hydrostatic pressure, Terzaghi's theory of consolidation cannot be applied to find the rate of secondary consolidation.

21. (b)

As per Tergazhi, for rectangular footing
Ultimate bearing capacity,

$$q_u = \left(1 + 0.3\frac{B}{L}\right) c N_c + \gamma \cdot D_f N_q + \left(1 - 0.2\frac{B}{L}\right) \cdot \frac{1}{2} B\gamma N_\gamma$$

Now, $C = 0$,

So,
$$q_u = 18 \times 1 \times 24 + \left(1 - 0.2 \times \frac{1.5}{3.0}\right) \times 1.5 \times \frac{1}{2} \times 18 \times 20$$
$$= 432 + 243 = 675 \text{ kN/m}^2$$

Net ultimate bearing capacity,

$$q_{nu} = q_u - \gamma \cdot D_f = 675 - 18 \times 1 = 657 \text{ kN/m}^2$$

Net safe bearing capacity,

$$q_{ns} = \frac{q_{nu}}{\text{FOS}} = \frac{657}{3} = 219 \text{ kN/m}^2$$

22. (b)

Given, $W = 80$ kN, $H = 1.5$ m = 150 cm, $S = 4$ mm = 0.4 cm, $C = 2.5$ cm for drop hammer

Hence,
$$Q_c = \frac{WH}{(S + C)\text{FOS}} = \frac{80 \times 150}{(0.4 + 2.5)6} = 689.66 \text{ kN} \simeq 689 \text{ kN (say)}$$

23. (a)

As per Skempton, $q_{nu} = CN_C$ (where C is the cohesion)

Given, UCS = 100 kN/m^2

$$\Rightarrow \quad C = \frac{UCS}{2} = 50 \text{ kN/m}^2$$

For $\dfrac{D_f}{B} \geq 2.5$, $\left[\dfrac{D_f}{B} = \dfrac{3}{1} = 3\right]$

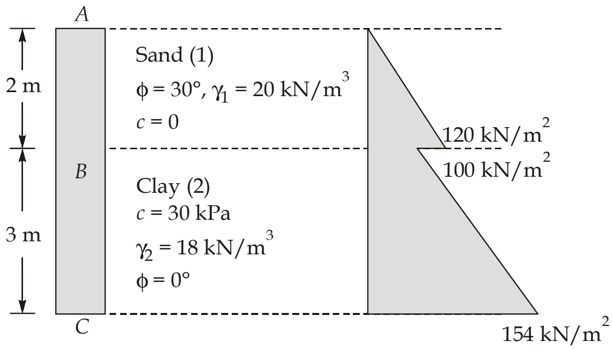
N_c for rectangular footing

$$= 7.5\left(1+\frac{0.2B}{L}\right) \leq 9$$

$$= 7.5\left(1+\frac{0.2\times 1}{3}\right) = 8 \leq 9$$

$$q_{nu} = 8 \times 50 = 400 \text{ kN/m}^2$$

24. (d)



$$K_{P_1} = \frac{1+\sin\phi}{1-\sin\phi} = 3$$

$$K_{P_2} = 1 \qquad (\because \ \phi_{clay} = 0°)$$

$$p_B = K_{P_1}\gamma_1 z_1 = 3\times 20\times 2 = 120 \text{ kN/m}^2$$

$$p'_B = K_{p_2}\gamma_1 z_1 + 2c\sqrt{K_{P_2}}$$

$$= 1\times 20\times 2 + 2\times 30\sqrt{1} = 100 \text{ kN/m}^2$$

$$p_c = K_{p2}\times\gamma_1 z_1 + K_{p_2}\gamma_2 z_2 + 2c\sqrt{K_{P_2}}$$

$$= 1\times 20\times 2 + 1\times 18\times 3 + 2\times 30\sqrt{1} = 154 \text{ kN/m}^2$$

Therefore total passive earth pressure per unit length of the wall is

$$p_a = \frac{1}{2}\times 120\times 2 + \frac{1}{2}(154+100)\times 3$$

$$= 120 + 381 = 501 \text{ kN/m}$$

25. (c)

$$\phi = 16°$$

$$k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin(16°)}{1+\sin(16°)} = 0.57$$

At A:

$$p_a = k_a \cdot \bar{\sigma}_z - 2c\sqrt{k_a} \quad (\because c = 0)$$

$$p_a = k_a \cdot \bar{\sigma}_z = 0.57 \times (4.5 + 20 \times 6) = 70.965 \text{ kN/m}^2$$

26. (b)

$$\gamma_{sat} = \frac{G+e}{1+e} \times \gamma_w = \frac{2.65+1}{1+1} \times 9.81 = 17.9 \text{ kN/m}^3$$

$$\gamma' = \gamma_{sat} - \gamma_w = 17.9 - 9.81 = 8.1 \text{ kN/m}^3$$

For sudden drawdown $\gamma = \gamma_{sat}$

$$\phi_w = \frac{\gamma'}{\gamma_{sat}} \times \phi_u = \frac{8.1}{17.9} \times 15 = 6.8°$$

$$F_C = \frac{C_u}{S_n \times \gamma_{sat} \times H} = \frac{12}{0.126 \times 17.9 \times 5} = 1.06$$

27. (a)

- Equivalent point load method is an approximate method.
- Boussinesq's equation can't be applied to sedimentary deposits.
- West

28. (c)

$$T_v = C_v \frac{t}{d^2}$$

$$T_v = \frac{k}{m_v \gamma_w} \times \frac{t}{d^2}$$

$\because$ Percentage consolidation required is same in both the cases.

$\therefore$ $$T_v = T_v'$$

$$\frac{k}{m_v \gamma_w} \times \frac{t}{d^2} = \frac{k'}{m_v' \gamma_w} \times \frac{t}{d^2}$$

$$\frac{k}{m_v} \times \frac{15}{d^2} = \frac{3k}{4m_v} \times \frac{t}{(d/2)^2} \qquad [\because k' = 3k \; \& \; m_v' = 4\text{m}]$$

$$15 = 3t$$

$$t = 5 \text{ years}$$

29. (a)

$$i = \frac{G-1}{1+e}$$

$$n = 0.3$$

$$\therefore \quad e = \frac{n}{1-e} = \frac{0.3}{1-0.3} = \frac{0.3}{0.7} = 0.43$$

$$G = 1.43i + 1$$

30. (c)

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

Failure strain = 25% = 0.25

$$\text{Corrected area} = \frac{A_0}{1-\varepsilon} = \frac{18}{1-0.25}$$

$$= \frac{18}{0.75} = 24\ \text{cm}^2$$

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