CLASS TEST

S.No. : 03_ SK_CE_S+G_190922

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SOIL MECHANICS

CIVIL ENGINEERING

Date of Test : 19/09/2022

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (b)

$$\because \qquad K = \frac{aL}{At}\ln\left(\frac{h_1}{h_2}\right)$$

$h_1 = 40$ cm, $h_2 = 20$ cm

$a = \frac{\pi}{4}\times 0.5^2 \text{ cm}^2$; $L = 15$ cm; $A = \frac{\pi}{4}\times 10^2 \text{ cm}^2$; $t = 1 \times 60 \times 60 = 3600s$

$$\therefore \qquad K = \frac{\frac{\pi}{4}(0.5)^2\times 15}{\frac{\pi}{4}(10)^2\times 3600}\ln\left(\frac{40}{20}\right)$$

$$= 7.22 \times 10^{-6} \text{ cm/s}$$

$$= 4.33 \times 10^{-4} \text{ cm/min}$$

2. (b)

$$\sigma_z = \frac{2q}{\pi z} = \frac{2\times 150}{\pi \times 5}$$

$$= 19.099 \text{ kN/m}^2$$

3. (b)

$D = 0.4$ m, $L = 15$ m, $c_u = \frac{80}{2} = 40 \text{ kN/m}^2$, $\alpha = 0.8$, FOS = 3

$$Q_{\text{up}} = 9c\cdot\frac{\pi}{4}d^2 + \alpha\, c\,\pi dL$$

$$Q_{up} = 9\times 40\times\frac{\pi}{4}\times 0.4^2 + 0.8\times 40\times\pi\times 0.4\times 15$$

$$Q_{up} = 648.42 \text{ kN}$$

$$\therefore \qquad \text{Design load capacity of pile} = \frac{648.42}{3} = 216.14 \text{ kN}$$

4. (c)

The total compression is always constant for a given load and does not depend on C_v and k.

7. (b)

$$q = h\sqrt{k_x k_y}\times\frac{N_f}{N_d}$$

$$= 8\times\sqrt{5\times 10^{-6}\times 6\times 10^{-6}}\times\frac{6}{18}$$

$$= 14.6 \times 10^{-6} \text{ m}^3\text{/s/m}$$

8. (a)

As per Skempton's theory, net ultimate bearing capacity is given by,

$$q_{nu} = cN_c$$

$$D_f/B = \frac{1.5}{2} = 0.75$$

$$\Rightarrow \quad 0 < \frac{D_f}{B} < 2.5$$

$\therefore$ For square footing,

$$N_c = 6\left[1+\frac{0.2D_f}{B}\right] = 6\times[1+0.2\times0.75] = 6.9$$

$$\Rightarrow \quad q_{nu} = cN_c = 30 \times 6.9 = 207 \text{ kN/m}^2$$

9. (a)

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

$= 0.73\,(60 - 20) = 29.2$

As soil lies above A-line, the soil will be clay.

Now, as $w_L > 50$, soil will be classified as CH.

10. (b)

For sandy soils,

$$q_u = \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma \qquad (\because \quad c = 0)$$

If water table rises to ground surface from great depth, γ will be replaced by γ' in both terms. Since γ' is approximately half of γ, ultimate bearing capacity will also become approximately half.

11. (b)

Presence of organic matter reduces the specific gravity of soil.

12. (a)

Maximum depth of unsupported excavation,

$$H_C = \frac{4c}{\gamma\sqrt{k_a}}$$

For pure clay, $\phi = 0$ and thus $k_a = 1$

$$\therefore \quad H_C = \frac{4c}{\gamma} = \frac{4\times80}{20} = 16 \text{ m}$$

$\therefore$ Active earth pressure at base level of excavation is

$$P_a = k_a\gamma H_c - 2c\sqrt{k_a}$$

$$= 20 \times 16 - 2 \times 80$$

$$= 160 \text{ kN/m}^2$$

13. (b)

At the steady state condition neither flow nor pore water pressure will change with time.

14. (b)

Deformations of soils are function of effective stress.

15. **(d)**

Quick clay has sensitivity > 30.

17. **(d)**

Coefficient of compressibility,

$$a_v = \frac{\Delta e}{\Delta \bar{\sigma}} = \frac{0.05}{100-50} = 0.001\ \text{m}^2/\text{kN}$$

Coefficient of volume change,

$$m_v = \frac{a_v}{1+e_0} = \frac{0.001}{1+0.7} = \frac{0.001}{1.7}$$

$$\Rightarrow \quad m_v = 5.88 \times 10^{-4}\ \text{m}^2/\text{kN}$$

18. **(b)**

At liquid limit, $w_L = \frac{W_w}{W_s} = \frac{m_w}{m_s}$

$$0.45 = \frac{m_w}{m_s}$$

$$\Rightarrow \quad m_w = 0.45\, m_s$$

Volume of soil sample, $V = V_w + V_s$

$$= \frac{m_w}{\rho_w} + \frac{m_s}{\rho_s}$$

$$= \frac{0.45 m_s}{\rho_w} + \frac{m_s}{G.\rho_w} \qquad (\because \rho_s = G\rho_w)$$

$$\Rightarrow \quad 23 = \frac{0.45 m_s}{1} + \frac{m_s}{2.73 \times 1}$$

$$\Rightarrow \quad m_s = 28.18\ \text{g}$$

The minimum volume will be attained by soil at shrinkage limit.

At shrinkage limit, $w_s = \frac{m_w}{m_s} = 0.18$

$$\Rightarrow \quad m_w = 0.18\, m_s$$

Minimum volume, $V_{min} = V_w + V_s$

$$= \frac{m_w}{\rho_w} + \frac{m_s}{\rho_s}$$

$$= \frac{0.18 m_s}{\rho_w} + \frac{m_s}{G\rho_w}$$

$$= \frac{0.18 \times 28.18}{1} + \frac{28.18}{2.73 \times 1} = 15.39 \simeq 15.4\ \text{cc} \qquad (\because \ m_s = \text{constant})$$

20. (d)

$$\text{Total stress at } A, \sigma = \gamma_{sat} \times 1$$
$$= 19.62 \text{ kN/m}^2$$
$$\text{Pore water pressure at } A, u = -2 \times \gamma_w$$
$$= -2 \times 9.81$$
$$= -19.62 \text{ kN/m}^2$$
$$\text{Effective stress at } A, \bar{\sigma} = \sigma - u$$
$$= 19.62 - (-19.62) = 39.24 \text{ kN/m}^2$$

21. (d)

$$\text{Void ratio, } e = \frac{wG}{S} = \frac{0.4 \times 2.65}{1} \qquad [\because \text{ Fully saturated}]$$
$$\Rightarrow \quad e = 1.06$$

Saturated unit weight of clay,

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w = \left(\frac{2.65+1.06}{1+1.06}\right) \times 9.81$$
$$= 17.667 \text{ kN/m}^3$$

Effective stress at centre of clay layer due to clay = 17.667 × 3 = 53 kN/m^3

Total intial overburden pressure = 260 + 53 = 313 kN/m^3

$$\text{Consolidation settlement, } S = \frac{H_0 C_c}{1+e_0}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$
$$= \frac{6 \times 0.5}{1+1.06}\log_{10}\left(\frac{313+100}{313}\right) = 0.1754 \text{ m}$$
$$= 17.54 \text{ cm}$$

22. (c)

$$\text{Effective normal stress, } \bar{\sigma} = \sigma - u$$
$$= 328 - 114$$
$$= 214 \text{ kPa}$$
$$\text{Shear resistance, } \tau = c' + \bar{\sigma}\tan\phi'$$
$$= 25 + 214 \times \tan 30°$$
$$= 148.55 \text{ kPa}$$

23. (b)

$$n = 0.5; \quad \therefore \quad e = \frac{n}{1-n} = \frac{0.5}{0.5} = 1$$

$\because$ $\qquad Se = wG$

$S = 0.7$; $e = 1$; $G = 2.7$

$$\therefore \qquad w = \frac{Se}{G} = \frac{0.7 \times 1}{2.7} = 0.259$$

$$\therefore \qquad \gamma = \frac{(G+Se)\gamma_w}{1+e} = \frac{G\gamma_w(1+w)}{1+e} = \frac{2.7 \times 10 \times 1.259}{2}$$
$$= 16.99 \text{ kN/m}^3$$

24. (a)

$$\eta_g = 1 - \frac{\theta}{90}\left[\frac{(n-1)m+(m-1)n}{mn}\right]$$

Here, $m = 4$, $n = 5$

$$\theta = \tan^{-1}\left(\frac{d}{s}\right) = \tan^{-1}\left(\frac{400}{1000}\right) = 21.8°$$

$$\therefore \quad \eta_g = 1 - \frac{21.8}{90}\left[\frac{(5-1)4+(4-1)5}{4\times5}\right]$$

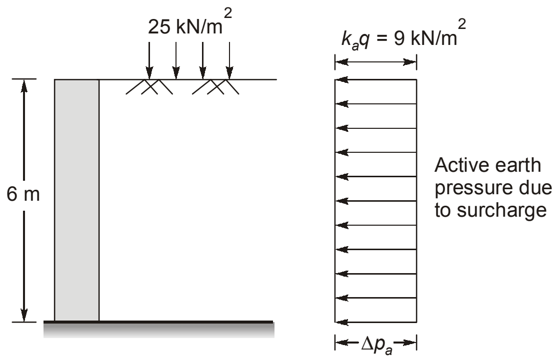
$$= 0.6246 = 62.46\%$$

$$\therefore \quad Q_g = n \cdot Q_u \cdot \eta_g$$

$$= 20\times380\times\frac{62.46}{100} = 4746.96 \text{ kN}$$

$$\approx 4747 \text{ kN}$$

25. (c)



$$k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 28°}{1+\sin 28°} = 0.36$$

Increase in active earth pressure,

$$\Delta p_a = k_a q = 0.36 \times 25 = 9 \text{ kN/m}^2$$

Increase in total active thrust,

$$\Delta P_a = \Delta p_a h = 9 \times 6$$

$$= 54 \text{ kN/m length of the wall}$$

26. (a)

When the infinite slope is subjected to full depth seepage (submerged condition)

$$\text{Factor of safety, } F = \frac{c' + \gamma' H \cos^2\beta \tan\phi}{\gamma_{sat} H \cos\beta \sin\beta}$$

Here, $F = 1$ and $H = H_c$

$$\therefore \quad H_C = \frac{c'}{\cos^2\beta(\gamma_{sat}\tan\beta - \gamma'\tan\phi')}$$

$$= \frac{10}{\cos^2 18°\{17\times\tan 18° - (17-9.81)\tan 14°\}} = 2.96 \text{ m}$$

27. (c)

For rectangular footing,

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

$\because$ For sand, $c = 0$

$$\therefore \quad q_u = 19\times2\times25+\left(1-0.2\times\frac{2}{3}\right)\times\frac{1}{2}\times19\times2\times18$$

$$= 950 + 296.4$$

$$= 1246.4 \text{ kN/m}^2$$

28. (b)

$$c' = 13 \text{ kN/m}^2, \quad \phi' = 30°$$

$$\bar{\sigma}_{3f} = 50 \text{ kN/m}^2$$

$$\text{Compressive strength} = \bar{\sigma}_{1f} - \bar{\sigma}_{3f}$$

We know that

$$\bar{\sigma}_{1f} = \sigma_{3f}\tan^2\left(45°+\frac{\phi'}{2}\right)+2c\tan\left(45°+\frac{\phi'}{2}\right)$$

$$= 50\tan^2\left(45°+\frac{30°}{2}\right)+2c\tan\left(45°+\frac{30°}{2}\right)$$

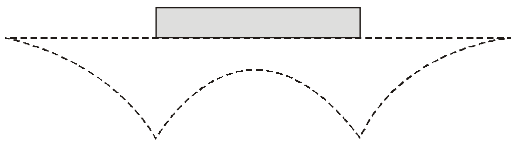
$$= 50\times\left(\sqrt{3}\right)^2+2\times13\times\sqrt{3}$$

$$\simeq 195 \text{ kN/m}^2$$

$$\therefore \quad \text{Compressive strength} = \bar{\sigma}_{1f} - \bar{\sigma}_{3f}$$

$$= 195 - 50 = 145 \text{ kN/m}^2$$

29. (a)



This is the distribution of deformation for sand in case of flexible footing.

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