CLASS TEST

S.No. : 02_ SK_CE_H+I_120822

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SOIL MECHANICS

CIVIL ENGINEERING

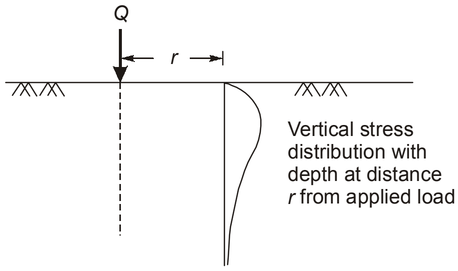
Date of Test : 12/08/2022

ANSWER KEY

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

DETAILED EXPLANATIONS

1. (c)



3. (c)

$$\text{Shrinkage ratio, R} = \frac{(V_1 - V_d)/V_d}{w_1 - w_s} \times 100$$

$$= \frac{(10 - 5.94)/5.94}{50 - 15} \times 100$$

$$= 1.95$$

4. (b)

Permeability is related to void ratio as

$$\frac{k_2}{k_1} = \frac{e_2^3}{1+e_2} \times \frac{1+e_1}{e_1^3}$$

$$\Rightarrow \quad k_2 = k_1\left(\frac{e_2^3}{1+e_2}\right) \times \left(\frac{1+e_1}{e_1^3}\right)$$

$$= \left(1 \times 10^{-3}\right) \times \left(\frac{0.6^3}{1+0.6}\right)\left(\frac{1+0.4}{0.4^3}\right)$$

$$= 2.95 \times 10^{-3} \text{ cm/s}$$

11. (a)

$$\text{Mass specific gravity} = \frac{\gamma_d}{\gamma_w} = 1.84$$

Hence, $\gamma_d = 1.84 \times 1 = 1.84$ g/cc

Void ratio, $e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 1.0}{1.84} - 1$

$= 0.47$

12. (d)

In plate load test:

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

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

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

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

Note: For cohesionless soil,

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

13. (c)

$$S_n = \frac{C}{F_C \gamma H}$$

$$\Rightarrow \quad S_n = \frac{C}{\gamma H_C} \qquad (\because \text{ Factor of safety, } F_C = 1 \text{ for critical height})$$

$$\Rightarrow \quad H_C = \frac{C}{\gamma S_n}$$

$$= \frac{30}{24 \times 0.05} = 25\,\text{m}$$

14. (a)

Effective normal stress, $\bar{\sigma} = \sigma - u$

$= 328 - 114$

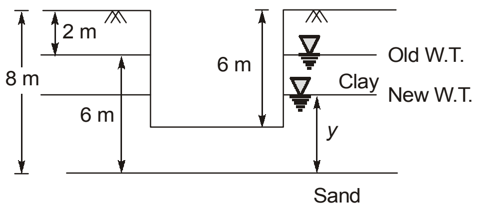
$= 214$ kPa

Shear resistance, $\tau = c' + \bar{\sigma} \tan \phi'$

$= 25 + 214 \times \tan 30°$

$= 148.55$ kPa

16. (b)



When uplift exceeds the soil weight, the soil becomes unstable.

If excavation is carried out upto 6 m depth,

Downward weight of soil = Uplift force of water

$(8 - 6)\text{ m} \times \gamma_{sat} = y \times \gamma_w$

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w = \left(\frac{2.72+0.72}{1+0.72}\right) \times 9.81 = 19.62\text{ kN/m}^3$$

$\therefore \quad 2 \times 19.62 = y \times 9.81$

$\Rightarrow \quad y = 4$ m

Water table should be lowered by = 6 – 4= 2 m

17. (d)

(i)Plasticity index for soil A = 30 – 16 = 14

Plasticity index for soil B = 52 – 19 = 33

Since plasticity index of soil B is greater, it is more plastic than soil A.

(ii) Consistency index for soil A $= \dfrac{w_L - w}{I_P} = \dfrac{30-32}{14} = -0.143$

Consistency index for soil B $= \dfrac{w_L - w}{I_P} = \dfrac{52-40}{33} = 0.364$

As consistency index for soil A is negative, it will turn into slurry when remoulded hence it is not a suitable foundation material. Soil B, however, is suitable for foundations.

19. (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)

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

21. (d)

Total stress at A, $\sigma = \gamma_{sat} \times 1$

$= 19.62 \text{ kN/m}^2$

Pore water pressure at A, $u = -2 \times \gamma_w$

$= -2 \times 9.81$

$= -19.62 \text{ kN/m}^2$

Effective stress at A, $\bar{\sigma} = \sigma - u$

$= 19.62 - (-19.62) = 39.24 \text{ kN/m}^2$

22. (b)

Given, head loss through soil B is 19 times that through soil A,

$$\Delta H_B = 19\,\Delta H_A$$

Total head loss $\quad \Delta H = \Delta H_A + \Delta H_B = 200 \text{ mm}$

$\Rightarrow \quad \Delta H_A + 19\,\Delta H_A = 200 \text{ mm}$

$\Delta H_A = 10 \text{ mm}$

$\Rightarrow \quad \Delta H_B = 19 \times 10 = 190 \text{ mm}$

$$i_A = \frac{\Delta H_A}{L} = \frac{10}{200} = 0.05$$

$$i_B = \frac{\Delta H_B}{L} = \frac{190}{200} = 0.95$$

As per Darcy's equation,

$$v = ki$$

As discharge through the sample is constant and area of both sample is same

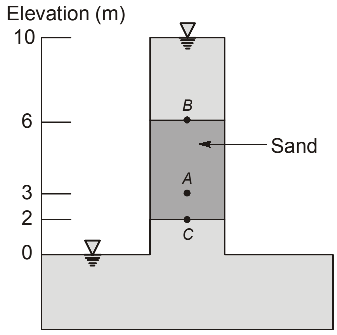
$\therefore \quad v_A = v_B$

$\Rightarrow \quad k_A i_A = k_B i_B$

$$k_B = \frac{k_A i_A}{i_B} = \frac{3\times10^{-5}\times0.05}{0.95}$$

$\Rightarrow \quad k_B = 1.58 \times 10^{-6} \text{ m/s}$

23. (c)



$$\text{Length of sample, } L = 4 \text{ m}$$

$$\text{Total head at B, } h_B = \text{Elevation head + Piezometric head}$$
$$= 6 + 4 = 10 \text{ m}$$
$$\text{Total head at C, } h_C = 0 + 2 = 2 \text{ m}$$
$$\text{Head difference} = h_B - h_C$$
$$= 10 - 2 = 8 \text{ m}$$

$$\Rightarrow \quad \text{Hydraulic gradient, } i = \frac{\Delta h}{L} = \frac{8}{4} = 2$$

$$\text{Head loss from point B to A} = i \times 3 = 2 \times 3 = 6 \text{ m}$$
$$\therefore \quad \text{Total head at A} = 10 - 6 = 4 \text{ m}$$
$$\Rightarrow \quad \text{Pressure head} = \text{Total head} - \text{Elevation head}$$
$$= 4 - 3 = 1 \text{ m}$$

24. (b)

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

$$\Rightarrow \quad e = 1.06$$

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

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

25. (c)

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

$$= 8 \times \sqrt{5\times10^{-6} \times 6\times10^{-6}} \times \frac{6}{18}$$

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

26. (c)

Given: $\beta = 50°;\quad \sigma_3 = 0;\quad \sigma_1 = 1.2 \text{ kg/cm}^2$

$$\sigma_1 = \sigma_3 \tan^2\beta + 2c \tan\beta$$

$$\Rightarrow \quad 1.2 = 0 + 2c \tan 50°$$

$$\Rightarrow \quad c = \frac{1.2}{2\tan 50°} = \frac{1.2}{2\times1.192}$$

$$= 0.503 \text{ kg/cm}^2$$

27. (b)

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,

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

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

28. (d)

Gravel = 100 – 82 = 18%

Sand = 82 – 9 = 73%

Fines = 9%

Therefore, the soil is predominantly sand. As fines lie between 5% and 12%, this soil will be classified by dual symbol representation.

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.12}{0.11} = 10.18 \qquad (\therefore\ C_u > 6)$$

$$C_c = \frac{D_{30}^2}{D_{60}\times D_{10}} = \frac{0.45^2}{1.12\times0.11} = 1.64 \qquad (\therefore\ 1 < C_c < 3)$$

Therefore sand is well graded.

Plasticity index, $I_P = w_L - w_P$

$= 22 - 12 = 10\% > 7\%$

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

$= 0.73\ (22 - 20) = 1.46$

$\therefore$ Soil lies above A-line i.e. it contains clay.

Soil is SW – SC : Well graded sand containing clay in sand.

29. (c)

Tap water contains a considerable amount of air. During permeability test, this air gets struck and remains trapped between sand grains thereby lowering the permeability.

30. (d)

At optimum moisture content the degree of saturation is less than 100%. Thus Statement (I) is incorrect.

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