

CLASS TEST

S.No. : 01 ND_CE_NW_220919

Soil Mechanics



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CLASS TEST 2019-2020

CIVIL ENGINEERING

Date of Test : 22/09/2019

ANSWER KEY > Soil Mechanics

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

DETAILED EXPLANATIONS

1. (b)

Water table correction in SPT is given by

$$N = 15 + \frac{1}{2}(N' - 15) = 15 + \frac{1}{2}(32 - 15)$$

$$= 15 + 8.5 = 23.5$$

 \therefore Corrected SPT value = 23

2. (a)

$$k \propto D_{10}^2$$

$$\therefore \frac{k_1}{k_2} = \frac{(D_{10})_1^2}{(D_{10})_2^2} = \frac{(0.6)^2}{(0.3)^2} = 4$$

3. (a)

$$C_C = \frac{D_{30}^2}{D_{60}D_{10}} \text{ and } C_u = \frac{D_{60}}{D_{10}}$$

$$\therefore \frac{D_{30}}{D_{10}} = \sqrt{C_C C_u} = \sqrt{1.5 \times 6} = 3.00$$

4. (a)

$$h = \frac{4\sigma}{\gamma d} = \frac{4 \times 0.073}{9.81 \times 0.05} \quad (\because \text{Surface tension for water, } \sigma = 0.073 \text{ N/m})$$

$$h = 0.62 \text{ m}$$

$$\therefore \text{Capillary tension} = h \cdot \gamma_w$$

$$= 0.62 \times 9.81 = 6.1 \text{ kN/m}^2$$

5. (b)

$$\text{Given } \frac{\mu_1}{\mu_2} = 1.5 \text{ and } \frac{1-\mu_1}{1-\mu_2} = 0.9$$

$$\therefore \frac{K_1}{K_2} = \left(\frac{\mu_1}{1-\mu_1} \right) \times \left(\frac{1-\mu_2}{\mu_2} \right)$$

$$= \frac{1.5}{0.9} = 1.666$$

6. (b)

$$\text{Exit gradient} = \frac{\text{Potential drop in last field}}{\text{Length of flow line of last square}} = \frac{18/12}{2} = 0.75$$

7. (b)

It is a coarse-soil, as only 7% are finer than 75 micron sieve.

It is a sandy soil because more than 50% of the coarse fraction passes through 4.75 mm sieve.

Size of aggregates varies linearly

To calculate D_{60}

$$\frac{80 - 7}{4.75 - 0.075} = \frac{80 - 60}{4.75 - D_{60}}$$

$$\Rightarrow D_{60} = 3.47 \text{ mm}$$

To calculate D_{30}

$$\Rightarrow \frac{80 - 7}{4.75 - 0.075} = \frac{80 - 30}{4.75 - D_{30}}$$

$$\Rightarrow D_{30} = 1.548 \text{ mm}$$

Similarly, $D_{10} = 0.267 \text{ mm}$

$$\Rightarrow \text{Uniformity Coefficient } C_u = \frac{D_{60}}{D_{10}} = 12.99 > 6$$

and Coefficient of curvature $C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} = 2.59$

$$\Rightarrow 1 < C_c < 3$$

\therefore The soil is well graded and finer particles are non-plastic, so it is silt.

\therefore It is SW – SM

8. (d)

As we know the relation,

$$\frac{s_f}{s_p} = \left[\frac{B_f (B_p + 30)}{B_p (B_f + 30)} \right]^2$$

$$\Rightarrow s_f = 12 \left[\frac{300 \left(\frac{30 + 30}{300 + 30} \right) \right]^2$$

$$\therefore s_f = 39.67 \text{ mm}$$

9. (c)

For 50% consolidation, the value of time factor (T_v) will be same for both soils.

$$T_v = \frac{C_{v1} t_1}{(d_1)^2} = \frac{C_{v2} t_2}{(d_2)^2}$$

where,

$$C_{v2} = 2C_{v1}$$

$$d_1 = \frac{d}{2}$$

$$d_2 = d$$

$$t_1 = 6 \text{ months}$$

$$\Rightarrow \frac{C_{v_1} \times 6}{(d/2)^2} = \frac{2C_{v_1} \times t_2}{(d)^2}$$

$$\therefore t_2 = 12 \text{ months}$$

10. (a)

$$\phi' = 28^\circ, \gamma_{\text{sat}} = 19 \text{ kN/m}^3, F = 3, \beta = ?, \gamma' = 19 - 10 = 9 \text{ kN/m}^3$$

As we know that,

$$\text{Factor of safety} = \frac{\gamma' \tan \phi'}{\gamma_{\text{sat}} \tan \beta}$$

$$\Rightarrow 3 = \frac{9 \times \tan 28^\circ}{19 \times \tan \beta}$$

$$\Rightarrow \tan \beta = 0.08395$$

$$\therefore \beta = 4.80^\circ$$

11. (b)

$$\text{Shrinkage Ratio} = \frac{\left(\frac{V_L - V_d}{V_d}\right) \times 100}{(w_L - w_s)} = \frac{\left(\frac{10 - 6}{6}\right) \times 100}{(50 - 15)} = 1.905\%$$

12. (a)

$$K_x = \frac{Zk + Z\frac{k}{3} + Z\frac{k}{2} + Z2k}{4Z} = \frac{23}{24}k$$

$$K_z = \frac{4Z}{\frac{Z}{k} + \frac{3Z}{k} + \frac{2Z}{k} + \frac{Z}{2k}} = \frac{8}{13}k$$

13. (b)

Soil A - 1 kg

$$w_C = 100\%$$

\therefore

$$w_S = 500 \text{ gm}$$

$$w_W = 500 \text{ gm}$$

Soil B - 1 kg

$$w_C = 50\%$$

\therefore

$$w_S = 666 \text{ gm}$$

$$w_W = 333 \text{ gm}$$

$$\text{Water content of mixed soil} = \frac{\text{Wt. of water}}{\text{Wt. of soil solid}} = \frac{500 + 333}{500 + 666} = 0.7144 \text{ or } 71.44\%$$

14. (d)

1. Zone 1 : Makes an angle of ϕ with horizontal

2. Zone 2 : It remains in plastic equilibrium

3. Zone 3 : Makes an angle of $\left(45^\circ - \frac{\phi}{2}\right)$ with horizontal

15. (d)

Ultimate bearing capacity is given by
Circular footing:

$$Q_{uc} = 1.3 cN_c + \gamma D_f N_q + 0.3 b\gamma N_\gamma$$

Square footing:

$$Q_{us} = 1.3 cN_c + \gamma D_f N_q + 0.4 b\gamma N_\gamma$$

For cohesionless soil :

$$c = 0$$

For footing on surface of soil:

$$D_f = 0$$

$$\therefore \frac{Q_{uc}}{Q_{us}} = \frac{0.3}{0.4} = 0.75$$

16. (b)

17. (a)

$$c_u = 100/2 = 50 \text{ kN/m}^2$$

$$\begin{aligned} \text{Ultimate bearing capacity of single} &= \alpha c_u (\pi D)L && \text{(Neglecting End Bearing)} \\ &= 0.8 \times 50 \times (\pi \times 0.5) \times 15 \\ &= 942.5 \text{ kN} \end{aligned}$$

18. (a)

$$A_t = \frac{I_p}{C}$$

where,

$$C = \text{Fraction of clay particles} = \frac{0.34}{1.5}$$

$$I_p = 0.6 - 0.26 = 0.34$$

$$\therefore A_t = \frac{(0.6 - 0.26)}{(0.34 / 1.5)} = 1.5$$

19. (b)

$$T_V = \frac{C_v t}{d^2} = \frac{\pi}{4} U^2 \quad \text{(For } U < 60\% \text{ consolidation)}$$

$$\therefore t \propto U^2$$

$$\frac{t}{178} = \left(\frac{60}{40}\right)^2$$

$$\therefore t = 400.5 \text{ days}$$

$$\text{Extra time required} = 400.5 - 178 = 222.5 \text{ days}$$

20. (a)

$$w = \left[\frac{(M_2 - M_1)}{(M_3 - M_4)} \cdot \left(\frac{G-1}{G}\right) - 1 \right] \times 100$$

Where,

$$M_2 - M_1 = 1000 \text{ gm}$$

$$M_3 = 2000 \text{ gm}$$

$$M_4 = 1480 \text{ gm}$$

$$\begin{aligned} \therefore w &= \left[\frac{1000}{2000 - 1480} \times \left(\frac{2.67 - 1}{2.67} \right) - 1 \right] \times 100 \\ &= 20.3\% \end{aligned}$$

21. (c)

$$\frac{D_f}{B} = \frac{1.2}{2.5} = 0.48 < 2.5$$

\therefore It is a shallow foundation

$$\begin{aligned} N_c &= 6 \left[1 + 0.2 \left(\frac{D_f}{B} \right) \right] \\ &= 6 \left[1 + 0.2 \times \frac{1.2}{2.5} \right] = 6.576 \end{aligned}$$

$$\begin{aligned} \therefore q_{nu} &= cN_c \\ &= 2.75 \times 6.576 \quad \left(\because c = \frac{q_u}{2} = \frac{5.5}{2} = 2.75 \text{ t/m}^2 \right) \\ &= 18.084 \text{ t/m}^2 \end{aligned}$$

$$\begin{aligned} \therefore q_s &= \left(\frac{q_{nu}}{\text{FOS}} \right) + \gamma D_f \\ &= \frac{18.084}{3} + 1.8 \times 1.2 = 8.2 \text{ t/m}^2 \end{aligned}$$

22. (b)

Given: $e_0 = 0.85$, $e_f = 0.73$, $\Delta\sigma_0 = (2 - 1) \text{ kg/cm}^2 = 1 \text{ kg/cm}^2$
 $\Delta e = (0.85 - 0.73) = 0.12$, $k = 3.3 \times 10^{-4} \text{ cm/sec}$

\therefore Coefficient of volume change,

$$m_v = \frac{\Delta e}{(1 + e_0)} \times \frac{1}{(\Delta\sigma_0)} = \frac{0.12}{(1 + 0.85)} \times \frac{1}{1}$$

$$m_v = 0.065 \text{ cm}^2/\text{kg}$$

\therefore Coefficient of consolidation,

$$C_v = \frac{k}{(m_v \gamma_w)} = \frac{3.3 \times 10^{-4}}{(0.065 \times 10^{-3} \times 1)} = 5.08 \text{ cm}^2/\text{sec}$$

23. (b)

As we know the relation,

$$\gamma_d = \frac{G\gamma_w}{1 + e}$$

$$\therefore e_1 = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.65 \times 9.81}{16.65} - 1 = 0.56$$

$$V_1 = 2070 \text{ m}^3$$

Now,
$$e_2 = \frac{n}{1-n} = \frac{0.33}{0.67} = 0.493$$

and,
$$\frac{V_1}{1+e_1} = \frac{V_2}{1+e_2}$$

$$\therefore V_2 = \frac{2070 \times (1.493)}{1.56} = 1981.1 \text{ m}^3$$

24. (c)

The horizontal distance between two points along the sand stratum = 21 m

Vertical distance between these two points = $\frac{21}{15} = 1.4 \text{ m}$

$$\therefore \text{Actual length of sand sample between two points} = \sqrt{(21)^2 + (1.4)^2} = 21.047 \text{ m}$$

Now,
$$Q = kiA$$

where,
$$i = \frac{H_L}{L} = \frac{3.5}{21.047}$$

$$Q = 5 \text{ litres per hour} = \frac{5000}{60 \times 60} \text{ cm}^3 / \text{sec}$$

(\because 1 litre = 1000 ml = 1000 cc)

$$A = 4 \text{ m depth} \times 1 \text{ m width}$$

$$A = 40000 \text{ cm}^2$$

Substituting these values, we get

$$\Rightarrow \frac{5000}{60 \times 60} = k \left(\frac{3.5}{21.047} \right) \times 40000$$

$$\therefore k = 2.09 \times 10^{-4} \text{ cm/sec}$$

25. (d)

Effective stress at 5 m depth, $\bar{\sigma}_1 = \gamma_d \times 2.4 + \gamma_{\text{sub}} \times 2.6$

using
$$\gamma_d = \frac{G \gamma_w}{1+e}$$

$$\Rightarrow 16.68 = \frac{2.7 \times 9.81}{1+e}$$

$$\Rightarrow e = 0.588$$

$$\gamma_{\text{sub}} = \left(\frac{G-1}{1+e} \right) \gamma_w = \frac{2.7-1}{1+0.588} \times 9.81 = 10.5 \text{ kN/m}^3$$

$$\therefore \bar{\sigma}_1 = (16.68 \times 2.4 + 10.5 \times 2.6) = 67.33 \text{ kN/m}^2$$

The shear strength of the soil,

$$\tau_{f1} = \bar{\sigma}_1 \tan \phi = 67.33 \times \tan 36^\circ = 48.92 \text{ kN/m}^2$$

Now water table rises upto ground level,

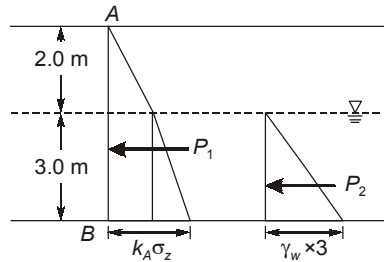
$$\bar{\sigma}_2 = \gamma_{\text{sub}} \times 5 = 10.5 \times 5 = 52.5 \text{ kN/m}^2$$

and,

$$\tau_{f_2} = \bar{\sigma}_2 \tan \phi = 52.5 \times \tan 36^\circ = 38.14 \text{ kN/m}^2$$

$$\therefore \text{Decrease in shear strength} = 48.92 - 38.14 = 10.78 \text{ kN/m}^2$$

26. (c)



$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1 - \sin 28^\circ}{1 + \sin 28^\circ} = 0.36$$

$$\rho_d = 1.61 \text{ gm/cc} \quad (\text{given})$$

$$\gamma_d = \rho_d \times g = 1.61 \times 9.81 = 15.79 \text{ kN/m}^3$$

$$\gamma_{\text{sub}} = \rho_{\text{sub}} \times g = 0.95 \times 9.81 = 9.32 \text{ kN/m}^3$$

The total active pressure at the base is given by

$$P_A = K_a \sigma_z + 3\gamma_w$$

Where,

$$\sigma_z = \gamma_{\text{dry}} \times 2.0 + \gamma_{\text{sub}} \times 3$$

$$P_A = 0.36 [15.79 \times 2.0 + 9.32 \times 3.0] + 3 \times 9.81$$

$$P_A = 50.86 \text{ kN/m}^2$$

27. (c)

Bulk density,
$$\rho = \frac{1.855}{0.945 \times 10^{-3}} = 1962.96 \text{ kg/m}^3$$

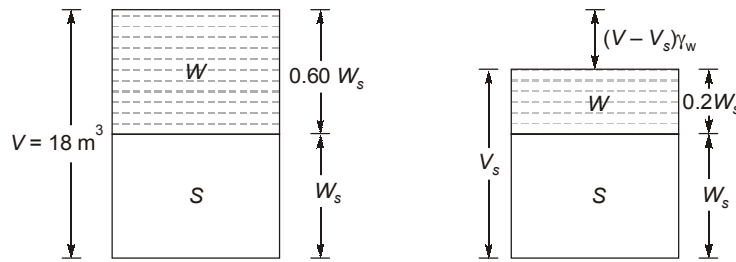
Dry density,
$$\rho_d = \frac{\rho}{1+w} = \frac{1962.96}{1+0.16} = 1692.21 \text{ kg/m}^3$$

and
$$\rho_d = \frac{(1-\eta_a)G\rho_w}{1+wG}$$

$$\Rightarrow 1 - \eta_a = \frac{1692.21 \times (1 + 0.16 \times 2.68)}{2.68 \times 1000} = 0.9022$$

$$\therefore \eta_a = 0.0978 \text{ or } 9.78 \%$$

28. (c)



At liquid state

$$w = 60\%$$

$$V = 18 \text{ m}^3$$

$$S = 1$$

$$\gamma_d = \frac{G_s \gamma_w}{1 + \frac{wG_s}{S}}$$

At liquid limit,

$$\frac{W_s}{18} = \frac{G_s \gamma_w}{1 + 0.60 \times 2.6} \quad \dots(i)$$

At shrinkage limit,

$$\frac{W_s}{V} = \frac{G_s \gamma_w}{1 + 0.20 \times 2.6} \quad \dots(ii)$$

Dividing equation (i) by the equation (ii),

$$\Rightarrow \frac{V}{18} = \frac{1 + 0.20 \times 2.6}{1 + 0.60 \times 2.6}$$

$$\therefore V = 10.7 \text{ m}^3$$

29. (b)

Equivalent height of surcharge is,
$$z_e = \frac{q}{\gamma} = \frac{36}{18} = 2 \text{ m}$$

Thus the problem reduces to the calculation of active earth pressure due to height

$$z = H + z_e = 4 + 2 = 6 \text{ m}$$

$$k_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{1}{3}$$

The pressure intensity at the base of the wall is given as

$$p_a = k_a \gamma z = \frac{1}{3} \times 18 \times 6 = 36 \text{ kN/m}^2$$

The pressure intensity at the top of the wall is
$$= k_a \times q = \frac{1}{3} \times 36 = 12 \text{ kN/m}^2$$

The pressure distribution is thus trapezoidal, therefore magnitude of active earth pressure,

$$P = \frac{1}{2}(12 + 36) \times 4 = 96 \text{ kN/m}$$

The distance of point of application from the base is

$$\bar{z} = \frac{H}{3} \left(\frac{2a+b}{a+b} \right) = \frac{4}{3} \times \left(\frac{2 \times 12 + 36}{12 + 36} \right) = 1.67 \text{ m}$$

30. (a)

Original length of specimen, $L = 9 \text{ cm}$

Initial cross-sectional area, $A_1 = \frac{\pi}{4} \times (4)^2 = 12.57 \text{ cm}^2$

Change in length at failure, $\Delta L = 1 \text{ cm}$

$$\text{Area at failure, } A_2 = \frac{A_1}{1 - \frac{\Delta L}{L}} = \frac{12.57}{1 - \frac{1}{9}}$$

$$A_2 = 14.14 \text{ cm}^2$$

\therefore Unconfined compressive strength, $q_u = \frac{\text{Failure load}}{A_2} = \frac{465}{14.14} = 32.88 \text{ N/cm}^2$

or $q_u = 328 \text{ kN/m}^2$ or 328 kPa

\therefore Shear strength, $c_u = \frac{q_u}{2} = \frac{328}{2} = 164 \text{ kPa}$

