CLASS TEST

S.No.: 06_CE_JP_18032025

RPSC

Rajasthan Public Service Commission

ASSISTANT ENGINEER EXAMINATION

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SOIL MECHANICS

CIVIL ENGINEERING

Date of Test : 18/03/2025

ANSWER KEY

1.	(d)	11.	(a)	21.	(d)	31.	(d)	41.	(d)
2.	(c)	12.	(a)	22.	(a)	32.	(c)	42.	(d)
3.	(a)	13.	(a)	23.	(b)	33.	(d)	43.	(b)
4.	(d)	14.	(b)	24.	(b)	34.	(c)	44.	(a)
5.	(a)	15.	(c)	25.	(b)	35.	(c)	45.	(b)
6.	(b)	16.	(c)	26.	(d)	36.	(d)	46.	(c)
7.	(c)	17.	(d)	27.	(b)	37.	(a)	47.	(d)
8.	(a)	18.	(c)	28.	(c)	38.	(a)	48.	(a)
9.	(c)	19.	(b)	29.	(b)	39.	(c)	49.	(b)
10.	(a)	20.	(b)	30.	(c)	40.	(d)	50.	(d)

Detailed Explanations

1. (d)

$$\text{Void ratio, } e = \frac{V_v}{V_s}$$

$$V_v = V_a + v_w = 140 + 180$$

$$V_v = 320 \text{ cc}$$

$$V = 700 \text{ cc}$$

$$V = V_S + 320$$

$$V_s = 380 \text{ cc}$$

$$e = \frac{320}{380} = 0.842$$

2. (c)

$$D_{60} < 4.75 \text{ mm}$$

$\Rightarrow$ Sand > Gravel

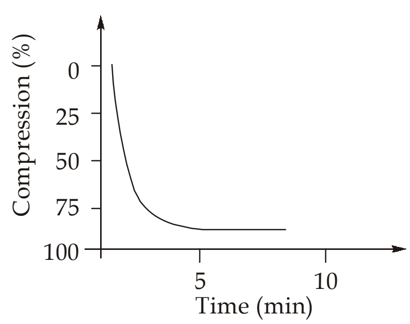
$$\text{Coefficient of curvature, } C_c = \frac{D_{30}^2}{D_{10} \times D_{60}} = \frac{0.29^2}{0.36 \times 0.16} = 1.46$$

$$\text{Coefficient of uniformity, } C_u = \frac{D_{60}}{D_{10}} = \frac{0.36}{0.16} = 2.25$$

$$1 < C_c < 3 \quad \text{But } C_u < 6$$

Hence, soil is classified as *SP*.

3. (a)



Because of very high permeability of the soil it does not take much time for pore water to drain out.

4. (d)

$\therefore$

$$k = C_v m_v \gamma_w$$

$$C_v = \frac{k}{m_v \gamma_w}$$

$$\frac{C_{v_2}}{C_{v_1}} = \frac{k_2}{k_1} \times \frac{m_{v_1}}{m_{v_2}}$$

$$\frac{C_{v_2}}{C_{v_1}} = \frac{1}{2} \times \frac{1}{2}$$

$$C_{v_2} = \frac{1}{4}C_{v_1}$$

6. (b)

The uniformity of a soil is expressed qualitatively by a term known as uniformity coefficient, C_u, given by,

$$C_u = \frac{D_{60}}{D_{10}}$$

The larger the numerical value of C_u, the more is the range of particles

7. (c)

8. (a)

$$S = \tau = 57.7 \text{ kPa}$$
$$\sigma = 100 \text{ kPa}$$
$$C = 0 \text{ (for sand)}$$
$$\tau = \sigma\tan\phi$$
$$\tan\phi = 0.577 = \frac{1}{\sqrt{3}}$$
$$\phi = 30°$$

9. (c)

10. (a)

Net ultimate bearing capacity of clay so is unaffected due to rise of water table to the ground level.

11. (a)

The effect of increasing the amount of compactive effort is to increase the maximum dry density and to decrease the optimum water content.

14. (b)

According to IS2911 (Part I)-1979.

- End bearing piles = 2.5D
  Friction piles = 3D
  In loose sands or fill deposits = 2D

16. (c)

$$\text{Settlement ratio} = \left(\frac{4B+2.7}{B+3.6}\right)^2$$

$$\lim_{B\to\infty}\left(\frac{4B+2.7}{B+3.6}\right)^2 = 16$$

17. (d)

$$V_s = k_p i$$

Where, V_s = seepage velocity

k_p = coeff. of percolation

18. (c)

Critical hydraulic gradient $= i_c = \dfrac{G-1}{1+e}$

also, $e = \dfrac{n}{1-n}$

$\therefore$ $i_c = \dfrac{G-1}{1+\dfrac{n}{1-n}} = (G-1)(1-n)$

19. (b)

For loose and medium sand critical depth of pile = 15D

$$= 15 \times (2 \times 0.4)$$
$$L_c = 12 \text{ m}$$

21. (d)

$$n = \frac{1}{3}$$
$$G = 2.65$$
$$\text{Critical exit gradient } i_c = \frac{G-1}{1+e} = (G-1)(1-n)$$
$$i_c = (2.65-1)\left(1-\frac{1}{3}\right)$$
$$i_c = 1.1$$
$$\text{Head required} = i_c \times \text{thickness of sand stratum}$$
$$H = 1.1 \times 12.5$$
$$H = 13.75 \text{ m}$$

22. (a)

The average degree of consolidation depends upon the non-dimensional time factor T_V. The time factor T_V depends upon the coefficient of consolidation C_v, time t and drainage path d.

23. (b)

With increase in temperature viscosity of water decreases and coefficient of permeability increases. As coefficient of consolidation

$$C_v = \frac{k}{\gamma_w m_v}$$

$\therefore$ C_v increases and rate of consolidation increase.

24. (b)

Only excess pore pressure completely dissipates and effective stresses increase.

25. (b)

Coefficient of consolidation,

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

T_v is constant and unit less

d is drainage path, unit is centimeter (cm) t is time of consolidation, unit is sec.

$\therefore$ unit of C_v = cm^2/sec

26. (d)

$$w_L = 55\%$$
$$I_p = 18\% = w_L - w_P$$
$$\Rightarrow \quad w_P = 37\%$$
$$I_s = w_P - w_S$$
$$24 = 37 - w_s$$
$$w_s = 13\%$$

27. (b)

The slow rate of consolidation of saturated clays of low permeability may be accelerated by means of vertical sand drains which provide for radial drainage, resulting in shortening of drainage path.

28. (c)

For 90% consolidation,

$$\frac{t_1}{t_2} = \frac{H_1^2}{H_2^2} \qquad [\text{As } t \propto H^2]$$

Where t_1 = 15 years, H_1 = H and H_2 = 2H

$$\therefore \quad \frac{15}{t_2} = \frac{H}{(2H)^2}$$

$$\therefore \quad t_2 = 15 \times 4 = 60 \text{ years}$$

As the clay is 3 times more permeable and 4 times more compressible therefore time required for 90% consolidation

$$= 60 \times \frac{4}{3} = 80 \text{ years}$$

29. (b)

$$\text{Permeability} \propto D_{10}^{\ 2}$$
$$\frac{k_1}{k_2} = \frac{0.6^2}{0.3^2}$$
$$\frac{k_1}{k_2} = 4$$

30. (c)

Skempton's B-parameter is given by,

$$B = \frac{1}{1 + n\left(\frac{C_v}{C_s}\right)}$$

C_v is volume compressibility of pore fluid under isotropic conditions,

C_s is the coefficient of compressibility of the soil skeleton; n is porosity

In a fully saturated soil, the compressibility of the pore water (C_v) is negligible compared with the compressibility of the soil mass (C_s). Therefore, the ratio (C_v/C_s) tends to zero and the coefficient B becomes equal to unity.

In a partially saturated soil, the compressibility of the air in the voids is high. The ratio (C_v/C_s) has a value greater than unity, and, therefore, the pore pressure coefficient B has a value of less than unity.

31. (d)

For remolded clays

$$w_L = 57\%$$
$$C_c = 0.007(w_L - 7)$$
$$C_c = 0.35$$

32. (c)

When ratio of $\frac{\sigma_1 - \sigma_3}{2}$ to $\frac{\sigma_1 + \sigma_3}{2}$ is equal to unity $\frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_1 + \sigma_3}{2}$

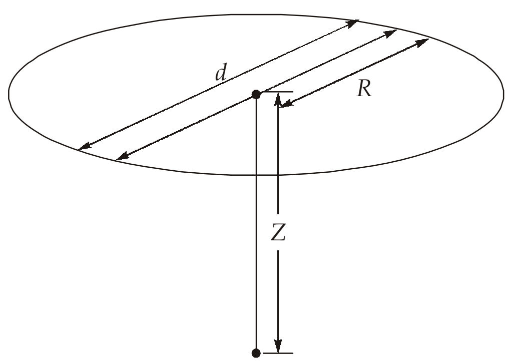
$\sigma_3 = 0$ (Confining pressure zero)
This case is represented by unconfined compression test.

33. (d)

If initial void ratio is less than critical void ratio then the sample is dense sand. So in this case the volume will decrease initially to get fully compacted and then to achieve critical void ratio the volume will increase. If initial void ratio is more than critical void ratio then the sample is loose sand. In this case volume initially decreases and then remains constant.

34. (c)

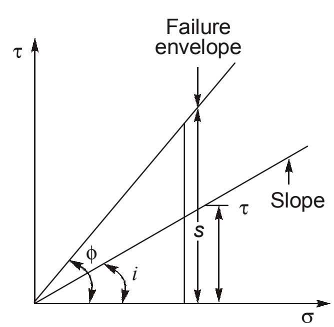
$$\sigma_z = q\left[1 - \frac{4z^2}{4z^2 + R^2}\right]^{3/2}$$



35. (c)

For an infinite slope in cohesionless soil.

Factor of Safety, (F) = $\frac{\tan\phi}{\tan i} = \frac{s}{\tau}$.



For $\phi > i$; F > 1 it means that the slope remains stable as long as angle of slope is less than angle of shearing resistance.

36. (d)

Taylor's stability No.

$$S_n = \frac{c_m}{\gamma H} = \frac{c}{F_c \gamma H}$$

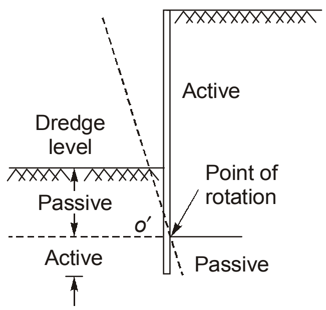
$$\because \quad F_c = \frac{H_c}{H}$$

$$\therefore \quad S_n = \frac{c}{\gamma H_c}$$

37. (a)

In Taylor's stability chart, the stability number is based on the assumption that factor of safety with respect to friction is unity. Thus effective angle is used in the chart and it is equal to the mobilised angle.

38. (a)



39. (c)

40. (d)

$$K_o = \frac{\mu}{1-\mu}$$

$$\Rightarrow \quad \frac{1}{K_o} = \frac{1-\mu}{\mu}$$

41. (d)

As per Terzaghi's bearing capacity equation,

$$q_{ult} = cN_c + \gamma D_f N_q + 0.5\,\gamma B N_\gamma$$

For a purely cohesive soil,

$$N_q = 1 \text{ and } N_\gamma = 0$$

$$\therefore \quad q_{ult} = cN_c + \gamma D_f$$

But $\quad q_{net,ult} = q_{ult} - \gamma D_f$

$$\Rightarrow \quad q_{net,ult} = cN_c + \gamma D_f - \gamma D_f$$

$$\Rightarrow \quad q_{net,ult} = cN_c$$

42. (d)

Terzaghi's bearing capacity factors N_c, N_q and N_γ are function of angle of friction only.

$N_c > N_q > N_\gamma$ for $\phi < 30°$

$N_c > N_\gamma > N_q$ for $30° < \phi < 40°$

$N_\gamma > N_c > N_q$ for $\phi < 40°$

43. (b)

$$B = \frac{\Delta U_1}{\Delta \sigma_3} = \frac{325-250}{600-500} = \frac{75}{100}$$

$$B = 0.75$$

44. (a)

According to **Hansen**, the ultimate bearing capacity is given by

$q_u = cN_c\, s_c\, d_c\, i_c + qN_q\, s_q\, d_q\, i_q$
$+ 0.5\, \gamma\, B\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma$

s_c, s_q and s_γ are shape factors
d_c, d_q and d_γ are depth factors
and i_c, i_q and i_γ are inclination factors

47. (d)

$$K_o = 1 - \sin\phi$$

$$K_o = 1 - \sin 30° = 0.5$$

$$\sigma_h = k_0 \Delta\sigma$$

$$= 0.5 \times 200$$

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

48. (a)

For strip footing on clay,

$q_{nu} = CN_c$

for circular footing on clay,

$q_{nu} = 1.3\ CN_c > CN_c$

49. (a)

For flexible footing

$$I_f = 1.12$$

$$S_i = \frac{qB(1-\mu^2)\times I_f}{E}$$

$$= \frac{200\times 4\times(1-0.5^2)\times 1.12}{50\times 10^3}\times 10^3$$

$$S_i = 13.44\ \text{mm}$$

50. (d)

Under-reamed pile is a special type of bored pile which is provided with a bulb/pedestal at the end. The usual size of such piles are 150 to 200 mm shaft diameter, 3 to 4 m long. The diameter of under-reamed portion is usually 2 to 3 times the shaft diameter.

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