Class Test Solution (SOIL) 30-07-2017
Answer key

1.	(a)	**16.**	(c)	**31.**	(c)	**46.**	(d)	**61.**	(d)
2.	(a)	**17.**	(b)	**32.**	(c)	**47.**	(d)	**62.**	(c)
3.	(a)	**18.**	(c)	**33.**	(d)	**48.**	(d)	**63.**	(a)
4.	(a)	**19.**	(c)	**34.**	(b)	**49**	(b)	**64.**	(d)
5.	(d)	**20.**	(d)	**35.**	(c)	**50.**	(d)	**65.**	(c)
6.	(c)	**21.**	(d)	**36.**	(d)	**51.**	(a)	**66.**	(c)
7.	(c)	**22.**	(b)	**37.**	(c)	**52.**	(c)	**67.**	(d)
8.	(b)	**23.**	(b)	**38.**	(d)	**53**	(b)	**68.**	(d)
9.	(d)	**24**	(d)	**39.**	(a)	**54.**	(b)	**69.**	(d)
10.	(d)	**25.**	(c)	**40.**	(c)	**55.**	(c)	**70.**	(a)
11.	(b)	**26.**	(a)	**41.**	(d)	**56.**	(b)	**71.**	(b)
12.	(c)	**27.**	(d)	**42.**	(d)	**57.**	(a)	**72.**	(d)
13.	(c)	**28.**	(b)	**43.**	(b)	**58.**	(b)	**73.**	(a)
14.	(d)	**29.**	(a)	**44.**	(c)	**59.**	(c)	**74.**	(d)
15.	(c)	**30.**	(d)	**45.**	(b)	**60.**	(c)	**75.**	(c)

CLASS TEST [SOIL] SOLUTIONS

1. (a)

Consider the base area of 1 m^2

$$n = \frac{e}{1+e} = \frac{0.6}{1.6}$$

$$\frac{V_v}{V} = \frac{0.6}{1.6} \quad \text{... (i)}$$

$$S = 0.5$$

$$\frac{V_w}{V_v} = 0.5 \quad \text{... (ii)}$$

$$V_w = \frac{0.60 \times V}{2 \times 1.6} = \frac{0.30 \times 500}{1.6}$$

$$V_w = 93.75 \text{ mm}$$

2. (a)

Asticity index,

$I_P = W_L - W_P = 48 - 26 = 22\%$

$$\text{Activity} = \frac{I_P}{\text{Clay content}} = \frac{22}{55} = 0.4\,.$$

3. (a)

$$I_T = \frac{I_P}{I_F} = \frac{0.45 - 25}{\dfrac{w_1 - w_2}{\log_{10}\left(\dfrac{N_2}{N_1}\right)}}$$

$$= \frac{0.20}{\dfrac{0.20}{\log_{10} 10}} = 1$$

4. (a)

1) $$R = \frac{V_i - V_d}{V_d} \times 100 / W_i - W_s$$

$$= \frac{1.36 V_d - V_d}{V_d} \times \frac{100}{(45 - 11)}$$

$$= \frac{36}{34} = 1.05$$

2) $$V_s = \frac{V_i - V_d}{V_d} \times 100$$

$$= 0.36 \times 100 = 36\%$$

3) $$S_r = \frac{V_i - V_d}{V_i} \times 1000$$

$$= \frac{1.36\ V_d - V_d}{1.36\ V_d} \times 10$$

$$= 0.264 \times 100$$

$$= 26.4\%.$$

5. (d)

$$0.15 = \frac{(0.20) \times x + (1 - x) \times 0}{1}$$

$\Rightarrow$ $x = 0.75$

$\therefore$ $1 - x = 0.25$

$\Rightarrow$ Hence % of sand = 25.

6. (c)

7. (c)

Measuring flask method is used for determination of specific gravity of soil particles.

8. (b)

Plasticity of the soil is due to adsorbed water.

9. (d)

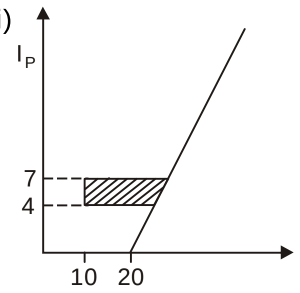
10. (d)

11. (b)

Explanation

(i)Fine content between 5 – 12% only, dual symbols are used

(ii)



dual symbols are used only in the hatched region.

12. (c)

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13. (c)

14. (d)

15. (c)

16. (c)

- Dispersed structure is formed when there is a NET REPULSIVE FORCE.
- Remoulding and compaction convertes the edge to face orientation to face to face orientation (dispersed structure).
- Concentration of dissolved mineral decrease the change of clay particle stabilize then hence flocculated structure forms.

17. (b)

Fine grained soil = 60%

Use sheep foot roller because it is better for clayey/fine grained soil.

18. (c)

Sometimes tube core of earth dam is compacted at wet of optimum.

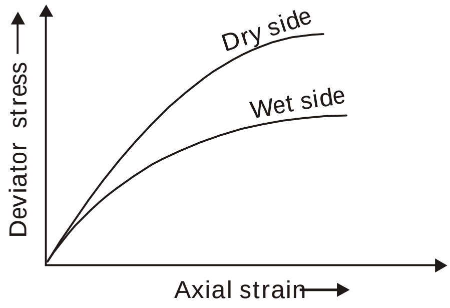
19. (c)

Core of earthendam is compacted on wet side of optimum, so as to reduce shrinkage/ cracking of the core and make the core of dam less permeable.

20. (d)

The procedure for conducting the test is similar to that in the standard proctor test but the soil is compacted only in 2 layers. It is claimed that the optimum water content and the dry density obtained in the test are almost equal to that in the standard proctor test.

The soils compacted wet of the optimum have relatively flatter stress-strain curve and a corresponding lower value of the modulus of elasticity.



21. (d)

Zero air void density

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

and $e = wG$

Hence, the correct option is (d).

22. (b)

$\because$ In variable head permeability test,

$$K = \frac{2.3aL}{At}\log_{10}\frac{H_1}{H_2}$$

where a = inside area of the burette

L = length of sample

H_1 = initial head

H_2 = final head

$\because$ the soil is the same in both cases;

$$\therefore \frac{2.3aL}{A\times 10}\log_{10}\frac{27}{3} = \frac{2.3aL}{At}\log_{10}\frac{27}{9}$$

or $$\frac{1}{10}\log 9 = \frac{1}{t}\log 3$$

or $$t = \frac{10\log 3}{2\log 3} = \frac{10}{2} = 5 \text{ minute}$$

23. (b)

elevation head = 25 cm

$$\text{pressure head} = \frac{\left(100+\frac{19.6}{9.8}\times 100\right)}{2} = 150 \text{ cm}$$

total = 175 cm.

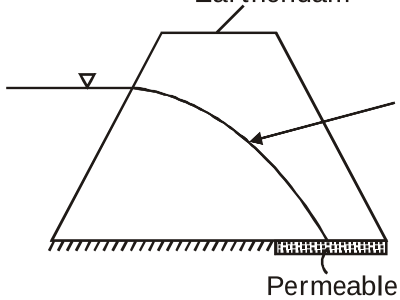
24. (d)

25. (c)

26. (a)

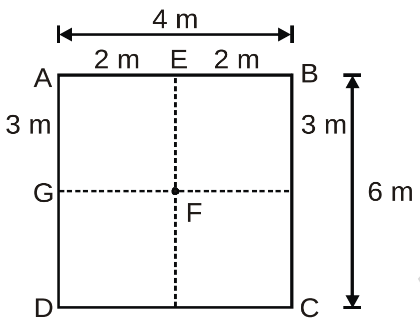
- at very high discharge quick sand condition can occur in gravel also.
- submerged weight not moist weight

27. (d)



Phreatic line is the top flow line of the seepage flow. Pressure on the surface of phreatic line is atmospheric and below is hydro static.

28. (b)



Rectangular area of 4 m × 6 m can be divided into 4 strip of equal size of 2 m × 3 m.

Stress below the centre of strip 4 m × 6 m

= 80 MPa

Stress below the corner of strip 2 m × 3 m

$= \frac{80}{4} = 20 \text{ MPa}$

29. (a)

30. (d)

31. (c)

The point loads applied below ground surface cause somewhat smaller stresses than are caused by surface loads, and, therefore, the boussinesq solution is not strictly applicable. However, the solution in frequently used for shallow footings, in which z is measured below the base of the footing.

32. (c)

33. (d)

Total consolidation = 7 cm

For 4 cm settlement, U_1 = 4/7 × 100 = 57.14%,

For 2 cm settlement, U_2 = 2/7 × 100 = 28.57%

$t_1 = 90$ days

For, $U \le 60\%$,

$$\frac{c_v t}{H^2} = t \propto U^2$$

$$\therefore \quad \frac{t_1}{t_2} = \frac{U_1^2}{U_2^2}$$

$$\therefore \quad t_2 = \frac{U_2^2}{U_1^2} \times t_1$$

$$= \frac{(28.57)^2}{(57.14)^2} \times 90$$

$$= 22.5 \text{ days.}$$

34. (b)

35. (c)

The pore water pressure is $u_0 = \Delta p = \gamma_w h$ = 481 kN/m^2

$$h = \frac{\Delta p}{\gamma_w} = 48.1 \text{ m}$$

The degree of consolidation at A is U_A (%) when h = 15 m :

$$U_A\% = \left(1 - \frac{u_A}{u_0}\right)100 = \left(1 - \frac{(15)(10)}{(48.1)(10)}\right)100$$

$$= 68.8\%$$

for 60% consolidation,

$$60 = \left(\frac{48.1 - h}{48.1}\right) \times 100$$

$$h = 19.24 \text{ m}$$

36. (d)

T_v is a factor which depends on the degree of consolidation and inversily on square of drainage path.

C_v is more $\Rightarrow$ Rate of settlement is more as $C_v \; \alpha \; k$

$$C_c = 0.009\,(W_L - 10)$$

37. (c)

$\Delta\bar{\sigma} = 4 \text{ kN/m}^2$

$$(m_v)_z = 0.0004z^2 + 0.08$$

$$S_f = \int_0^2 m_v (\Delta\bar{\sigma})\, dz$$

$$= 4\int_0^2 (0.0004z^2 + 0.008)\,dz$$

$$= 4\left[\frac{0.0004z^3}{3} + 0.008z\right]_0^2$$

$$= 0.0683 \text{ m}$$

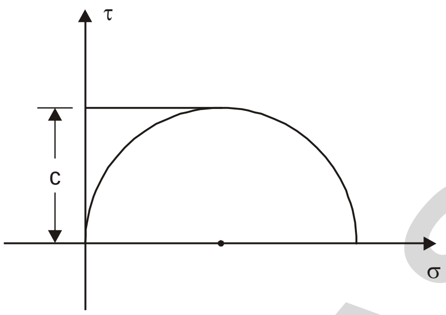
$$= 6.83\text{cm}$$

38. (d)

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

$$\theta_c = 45^o + \frac{\phi}{2}$$

$$\Rightarrow \quad 45^o = 45^o + \frac{\phi}{2} \Rightarrow \phi = 0$$



$$\sigma_3 = 30 \text{ kN/m}^2$$

$$\sigma_1 = \sigma_3 + 2c$$

$$(\sigma_1 - \sigma_3) = 2c$$

$$\Rightarrow \quad \sigma_d = 2 \times 50 = 100 \text{ kN/m}^2.$$

39. (a)

$$\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c\tan(45 + \phi/2)$$

For sand Specimen $C = 0$

$$\sigma_1 = \sigma_3 \tan^2(45 + \phi/2)$$

$$900 = 250\tan^2\left(45 + \frac{\phi}{2}\right)$$

$$\phi = 34.41°.$$

$$\tau_f = C + \sigma_n \tan\phi$$

$$(\because \ \tau_f = 100 \text{ kN/m}^2)$$

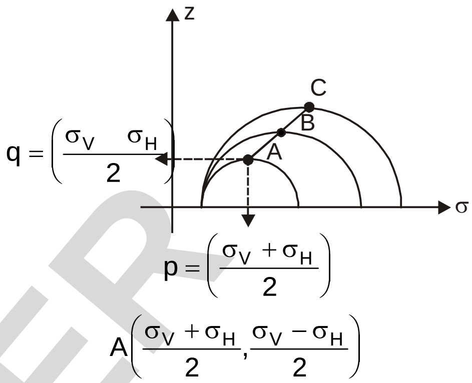
$$\tau_f = 100 \times 0.685$$

$$\tau_f = 68.516 \text{ kN/m}^2.$$

40. (c)

only 1 and 4 are correct

Stress path is made by joining the points of maximum shear stress of mohr's circle or point A, B and C are top most points of mohr circle.



So stress path traced by total stress analysis will be a straight line joining top most points of mohr's circle.

Now of there is pore pressure u then

$$p = \frac{\sigma_V + \sigma_H}{2}$$

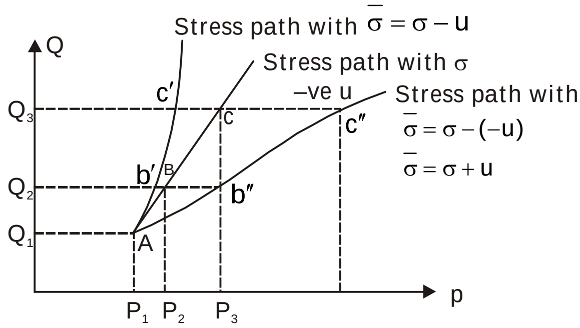
$$q = \frac{\sigma_V - \sigma_H}{2}$$

i.e. ordinate of total stress path will not change

$$\overline{\sigma_V} = \sigma_V - u$$

$$\overline{\sigma_H} = \sigma_H - u$$

So if u is +ve then effective stress path will shift towards the left where as if u is –ve then effective stress path will shift towards right.



41. (d)

Cell pressure stage (B) : The parameter B represents the ratio of change in pore pressure to the change in cell pressure.

$$B = \frac{\Delta U_c}{\Delta \sigma_c}$$

for fully saturated soil B = 1 and for dry soil B = 0

$$\boxed{0 \le B \le 1}$$

Deviator stress stage : The parameter A is defined in term of $\bar{A}$, which represents change in pore pressure due to change inddeviator stress

$$\bar{A} = \frac{\Delta U_d}{\Delta \sigma_d}$$

$$\bar{A} = \frac{\Delta U_d}{\Delta \sigma_1 - \Delta \sigma_3}$$

42. **(d)** Axial strain, $\varepsilon = \frac{13}{80} = 0.162$

$$A_f = \frac{A_0}{1-\varepsilon} = \frac{1105}{1-0.162} = 1315 \text{ mm}^2$$

$$\therefore q_u = \frac{28}{1315} \frac{\text{N}}{\text{mm}^2} = 21.3 \text{ kN/m}^2$$

$$C_u = \frac{q_u}{2} = \frac{21.3}{2} = 10.65 \text{ kN/m}^2$$

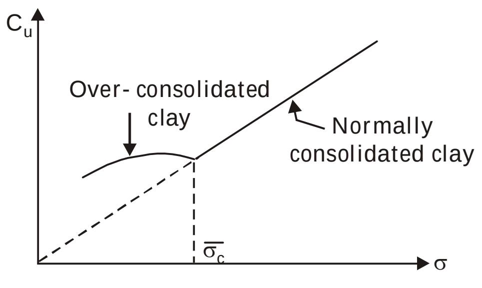
43. **(b)**

$$A = \frac{\Delta u_d}{\Delta\sigma_1 - \Delta\sigma_3} = \frac{10}{20-0} = 0.5$$

44. **(c)**

The vane shear test does not give accurate results when the failure envelope is not horizontal.

For over-consolidated clays, the plot between C_u and σ is curved until a pressure equal to the preconsolidation pressure $\bar{\sigma}_c$.



C_u–σ Plot for over-consolidated clay

45. **(b)**

From equation

$$S_n = \frac{C}{F_c \times \gamma H}$$

Taking $F_c = 1$

$$0.261 = \frac{30}{1 \times 16 \times H}$$

$$\boxed{H = 7.18 \text{ m}}$$

46. **(d)**

$$\text{where } Z = \frac{4c}{\gamma \cdot \sqrt{K_A}}$$

$$\therefore 6 \text{ m} = \frac{4 \times c}{20 \frac{\text{kN}}{\text{m}^3} \sqrt{1}} \left(\because K_A = \frac{1-\sin 0^\circ}{1+\sin 0^\circ} = 1 \right)$$

$$\Rightarrow c = \frac{6 \times 20}{4} \text{kN/m}^2 = 30 \text{ kN/m}^2$$

47. **(d)**

Rankine derived its theory for dry cohesionless soil but later it was also extended to submerged and cohesive soils also hence statement 1 and 2 are correct.

Rankine's theory considered stress in soil mass when it is in plastic equilibrium, it assumed every point in soil mass experiences shear failure under the developed shear stress.

48. **(d)**

$$\eta_h = 80\%$$

$$\eta_b = 60\%$$

$$W = 80 \text{ kN}$$

$$H = 60 \text{ cm}$$

$$S = 20 \text{ mm} = 2 \text{ cm}$$

$$C = 25 \text{ mm} = 2.5 \text{ cm}$$

$$Q_u = \frac{\eta_h \times \eta_b \times WH}{S + \frac{C}{2}}$$

$$Q_u = \frac{0.8 \times 0.6 \times 80 \times 60}{2 + \frac{2.5}{2}}$$

$$\boxed{Q_u = 708.9 \text{ kN}}$$

49. (b)

$$q_u = 200\ \text{kN/m}^2$$
$$\gamma = 20\ \text{kN/m}^3$$
$$D_f = 3\ \text{m}$$
$$\text{F.O.S.} = 2.0$$

Safe bearing capacity

$$q_{safe} = \frac{q_u - \bar{\sigma}}{\text{F.O.S.}} + \bar{\sigma}$$
$$= \frac{200 - 20 \times 3}{2.0} + 20 \times 3$$
$$= \frac{140}{2} + 60$$
$$= 130\ \text{kN/m}^2$$

50. (d)

$$q_{nu} = 5\left(1 + .2\frac{D_f}{B}\right)\left[1 + .2\frac{B}{L}\right]C$$
$$= 5\left(1 + .2 \times \frac{5}{10}\right)\left[1 + .2 \times \frac{10}{10}\right] \times 30$$
$$= 198\ \text{kN/m}^2$$

Net safe bearing capacity $= \dfrac{198}{2.5}$

$= 79.2\ \text{kN/m}^2$.

51. (a)

52. (c)

G.S.F.

$$C = 30\ \text{kN/m}^2$$
$$\phi = 37°$$

L.S.F. (Modified shear and friction angle)

$$C' = \frac{2}{3}C = \frac{2}{3} \times 30 = 20\ \text{kN/m}^2$$
$$\tan\phi = \frac{2}{3}\tan\phi$$
$$\phi = \tan^{-1}\left(\frac{2}{3}\tan\phi\right) \simeq 26.67°$$

53. (b)

54. (b)

The settlement corresponds to $q = 400\ \text{kN/m}^2$

Footing load, $q_f = \dfrac{1800}{2 \times 2} = 450\ \text{kN/m}^2$

Settlement of plate corresponding to 450 kN/m²

$$\frac{S_{p450}}{S_{p400}} = \frac{450}{400} \Rightarrow S_{p450} = 6 \times \frac{450}{400}$$
$$= 6.75\ \text{mm}$$

$$\frac{S_f}{S_p} = \left[\frac{B_f(B_p + 0.3)}{B_p(B_f + 0.3)}\right]^2 = \left[\frac{2(0.3 + 0.3)}{0.3(2 + 0.3)}\right]^2 = 3.025$$

$S_f = S_p \times 3.025 = 6.75 \times 3.025 = 20.42$ mm

55. (c)

Raft foundations on sands are quite useful. Their bearing capacity failure is generally out of question, because the bearing capacity in sands increases with the width of the footing and this width in rafts is quite large. The differential settlements in rafts are also generally smaller as compared to isolated footings even for the same load intensity because a raft eliminates the influence of local loose soils.

56. (b)

$$D/B = \frac{1.2}{2.5} = 0.48 < 2.5$$

$$\therefore N_c = \left[1 + 0.2\frac{D}{B}\right] \times 6$$

$$\Rightarrow \left[1 + 0.2 \times \frac{1.2}{2.5}\right] \times 6$$

$$N_c = 6.57$$

Safe bearing capacity $= \dfrac{cN_c}{2.5} + \gamma d = 9.38$

57. (a) The load carrying capacity of a driven pile can be estimated from the resistance against penetration developed during driving operation. The methods give fairly good results only in the case of free-draining sands and hard clays in which high pore water pressures do not develop during the driving of piles. In saturated fine grained soils, high pore water pressure develops during the driving operation and the strength of the soil is considerably changed and the methods do not give reliable results.

58. (b)

$$q_u = 9c_u \times \frac{\pi}{4}(\phi)^2 + \alpha\bar{c}\pi d l$$

$$q_u = 9\times4 \times \frac{\pi}{4}(0.60)^2 +$$

$$1/2 \times 4 \times 3.14\times0.6 \times 12$$

$$q_u = 10.17 + 45.21$$

$$q_u = 55.38 \text{ tonne}$$

59. (c)

Loose soil settles faster than pile, so it is a case of negative skin friction.

Load bearing capacity of pile

$$Q_{up} = 9C\times\frac{\pi}{4}(d)^2 + \alpha_2\bar{C}_2\pi DL_2 - \alpha_1\bar{C}_1\pi DL_1$$

$$Q_{up} = 9\times40\times\frac{\pi}{4}(0.5)^2 +$$

$$0.8\times30\times3.14\times0.5\times8$$

$$-0.6\times20\times0.5\times3.14\times4$$

$$\boxed{Q_{up} = 296.73 \text{ kN}}$$

60. (c)

The unit skin friction along the pile is found by the equation $f_s = k\sigma'_v \tan\delta$

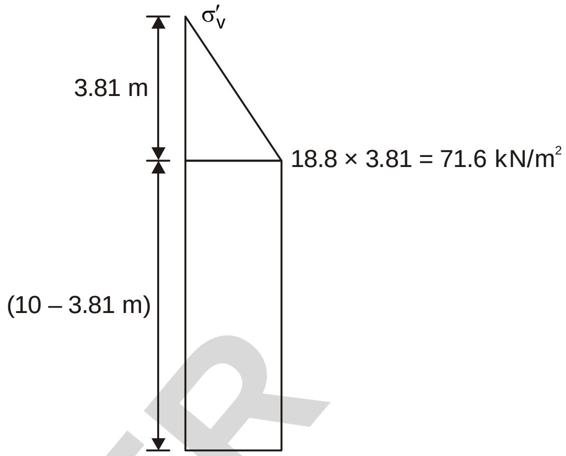
The critical depth is 15B. For a depth, z, from 0 to 15 B, $\sigma'_v = \gamma z = 18.8z$

For a depth greater or equal to 15B,

$$\sigma'_v = \gamma z = \gamma 15B = \left(18.8\frac{\text{kN}}{\text{m}^3}\right)(15)(0.254 \text{ m})$$

= 71.6 kN/m² (71.6 kPa)

The resulting vertical effective stress distribution is shown.



For the four sides of the square pile (4BL) and the average pressure, the skin-friction capacity along the pile from depth 0 to 15D is

$$Q_f = 4BL'\frac{1}{2}k\sigma'_v \tan\delta$$

$$= (4)\ (0.254 \text{ m})\ (15)\ (0.254 \text{ m})\left(\frac{1}{2}\right)\times(1.6)$$

$$\left(71.6\frac{\text{kN}}{\text{m}^2}\right)(\tan 0.6\times35^\circ) = 85.1 \text{ kN}$$

The skin friction capacity along the pile from depth 15 D to 10 m is

$$Q_f = 4B\ (L - L')\ f_{z-15D}$$

$$= (4)\ (0.254 \text{ m})\ (10 \text{ m} - 3.81 \text{ m})\times$$

$$(1.6)\left(71.6\frac{\text{kN}}{\text{m}^2}\right)\tan(0.6\times35^\circ) = 276.6 \text{ kN}$$

The total frictional resistance along the pile is

$$Q_{s\,total} = \Sigma Q_s = 85.1 \text{ kN} + 276.6 \text{ kN} = 362 \text{ kN}$$

(360 kN)

61. (d)

62. (c)

Most of the shear tests are conducted as strain-controlled. The stress-strain characteristic are easily obtained in these tests, as the shape of the stress-strain curve beyond peak point, can be observed only in a strain-controlled test.

Stress-controlled tests are preferred for conducting shear tests at a very low rate, because an applied load can easily be kept constant for any given period of time. Further, the loads can be conveniently applied and removed. The stress-controlled test represents the field conditions more closely.

63. (a)

64. (d)

Stress path can be plotted for stress condition during triaxial test.

65. (c)

66. (c)

The correlation is not perfect. A liberal factor of safety should be provided if the design is based only on index properties.

67. (d)

68. (d)

$$\text{Consistency index} = \frac{w_L - w_n}{w_L - w_P}$$

When soil is at liquid limit,

$$w_n = w_L$$

$$\text{Consistency index} = \frac{w_L - w_L}{w_L - w_P} = 0$$

69. (d)

A soil having symbol SM is classified silty sand.

where S = Sand and M = Silt

SM is a coarse soil [i.e. Coarse fraction > Fine fraction] in which sand is the major part of coarse fraction [Sand > Gravel]

Coarse soil having fines greater than 12% are classified as either SM, SC GM, or GC.

70. (a)

The water held by electrochemical forces existing on the soil surface is known as adsorbed water or hygroscopic water. The quantity of adsorbed water depend upon the colloidal fraction in the soil, the chemical composition of the clay mineral and the environment surrounding the particle. The adsorbed water is important only for clayey soils. For coarse grained soils, its amount is negligible or zero.

71. (b)

72. (d)

Permeability on wet side of optimum is less than the dry side of optimum.

Note :

Project	Compaction water content	Reason
Core of an earth dam.	Wet of optimum	To reduce permeability and prevent cracking in core.
Homogenous earth dam	Dry of optimum	To have a stronger soil & to prevent build up of high pore water pressure.
Sub-grade of pavement	Wet of optimum	To limit volume change.

73. (a)

74. (d)

75. (c)

Secondary Consolidation

The reduction in volume of a soil mass caused by the application of a sustained load to the mass, due principally to the adjustment of the internal structure of the soil mass after most of the load has been transferred from the soil water to the soil solids.

Immediate Compression

The immediate compression occurs as soon as the load from the building is applied on the foundation soil and is a result of expulsion of air from voids of soil grains.

Primary Consolidation

The force applied on the soil strata also increases the pressure in the pore water. This is higher as compared to the pore water pressure in the surrounding soil, thus a hydraulic gradient is developed causing the flow of water to the surrounding areas. The time taken for the expulsion of pore water from a soil depends on the permeability of that soil.