Class Test Solution (SOIL) 25-07-2018
Answer key

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

CLASS TEST [SOIL] SOLUTIONS

1. (d)

For uniformly graded coarse grained soil maximum void ratio occurs when particles are arranged in a cubical array and corresponding maximum void ratio is **91%**.

Minimum possible void ratio will be when particles are arranged in prizmoidal array. The minimum void ratio is 35%.

2. (b)

3. (c)

$W_L = 60\%$

$$W_P = 20\%$$

$$I_P = W_L - W_P = 40\%$$

Particle greater than 2 mm size = 80%

% of clayey particle (less than 2 micron) = 20%

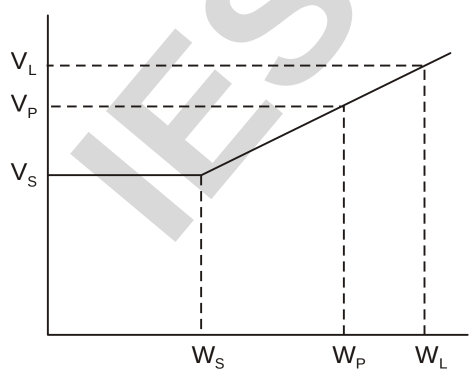
Activity of sample = $\dfrac{I_P}{C}$

$$A_C = \frac{40}{20} = 2\%$$

4. (c)

$V_P - V_d = 0.25\ V_P$ (given) ...(ii)

$V_L - V_d = 0.34\ V_L$ (given) ...(i)



Subtracting (i) from (ii), we get

$V_L - V_P = 0.34\ V_L - 0.25\ V_P$

or $0.66\ V_L = 0.75\ V_P$

or $V_L = 1.136\ V_P$

From geometry of fig. we can write

$$\frac{W_P - W_S}{V_P - V_d} = \frac{W_L - W_P}{V_L - V_P};$$

where $W_P = 0.25$

$$W_L = W_P + I_P = 0.25 + 0.08 = 0.33$$

W_S = shrinkage limit.

$$\therefore \quad \frac{0.25 - W_S}{V_P - V_d} = \frac{0.33 - 0.25}{1.136\ V_P - V_P} = \frac{0.08}{0.136\ V_P}$$

From (i), $0.75\ V_P = V_d$.

$$\therefore \quad \frac{0.25 - W_S}{V_P - V_d} = \frac{0.08}{0.136\ V_P}$$

$$\text{or} \quad \frac{0.25 - W_S}{0.25\ V_P} = \frac{0.08}{0.136\ V_P}$$

or $\quad W_S = 0.1029$, i.e. 10.29%

5. (a)

$$\text{Also Shrinkage ratio (SR)} = \left(\frac{\dfrac{V_1 - V_2}{V_d}}{W_1 - W_2}\right)$$

$$\text{or} \quad \text{S.R} = \frac{V_L - V_P}{V_d \cdot (W_L - W_P)}$$

$$= \frac{1.136\,V_P - V_P}{0.75\,V_P\,(0.08)} = \frac{0.136}{0.75 \times 0.08} = 2.27$$

6. (b) Poorly graded sands will have particle of uniform size leading to poor packing of soil structure. Hence they will have lower friction angle.

7. (b)

$$P_a = k_a \gamma z - 2C\sqrt{k_a}$$

At the surface z = 0

$$P_a = -2C\sqrt{k_a}$$

When vertical surcharge of q kN/m^2 is applied then

$$P_a = K_a q$$

For total active thrust to be zero

$$K_a q - 2c\sqrt{K_a} = 0$$

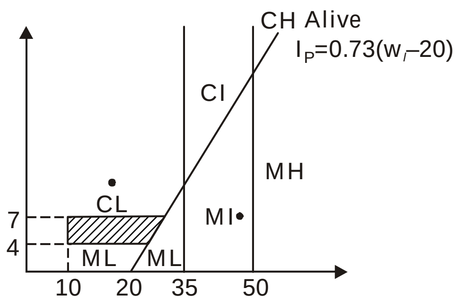
$$q = \frac{2c\sqrt{k_a}}{k_a}$$

$$q = \frac{2c}{\sqrt{k_a}}$$

8. (a)

20 × 10 × 0.005 = 1T/m^2

9. (d)



10. (b) More than 50% of total wt is finer than 75μ sieve so soil is fine soil.

$$I_P = 38 - 20 = 18$$

I_P = (A line) = 0.73 (W_L – 20) = 0.73 × (38 – 20) = 13.14

Plot the PI and LL on the plasticity chart the plot lies above A line so classification is CI.

11. (c)

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

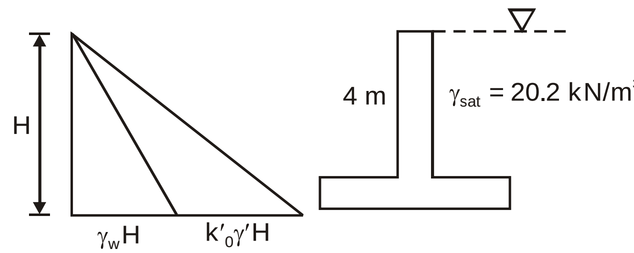
$$C_C = \frac{D_{30}^2}{D_{60} \times D_{10}}$$

$$C_u \times C_c = \frac{D_{60}}{D_{10}} \times \frac{D_{30}^2}{D_{60} \times D_{10}} = \left[\frac{D_{30}^2}{D_{10}^2}\right]$$

$$\left[\frac{D_{30}}{D_{10}}\right]^2 = 9 \times 1$$

or $$\left[\frac{D_{30}}{D_{10}}\right] = 3$$

12. (d)



$$H = 4 \text{ m}$$

$$\mu = 0.2$$

$$\gamma_{sat} = 20.2 \text{ kN/m}^3$$

$$\gamma' = \gamma_{sat} - \gamma_w$$

$$\gamma' = 20.2 - 9.81$$

$$\gamma' = 10.39 \text{ kN/m}^3$$

$$k_0 = \frac{\mu}{1-\mu} = \frac{0.2}{1-0.2} = 0.25$$

Earth pressure at bottom of the retaining wall

p_H = 9.81 × 4 + 0.25 × 10.39 × 4

p_H = 49.63 kN/m^2

Total earth pressure force

$$P_0 = \frac{1}{2} p_H \times H$$

$$P_0 = \frac{1}{2} \times 49.63 \times 4$$

P_0 = 99.26 kN (per unit length of wall)

13. (d)

$$\phi = 0$$

$$K_a = 1$$

$$Cu = 20 \text{ kPa}$$

$$P_a =$$

$$\frac{1}{2} K_a \gamma H^2 - 2CH\sqrt{K_a} + \frac{2C^2}{\gamma}$$

$$P_a = \frac{1}{2} \times 1 \times 19 \times 36 - 2 \times 20 \times 6 \times \sqrt{1} + \frac{2 \times (20)^2}{19}$$

$$P_a = 342 - 240 + 42.10$$

$$P_a = 144.1 \text{ kN}$$

14. (c)

15. (c) These soils are quite strong and can resist external forces because of a strong bond due to attraction between particles.

16. (b)

$$\eta_h = 80\%; \quad \eta_b = 60\%$$
$$W = 80 \text{ kN}; \quad H = 60 \text{ cm}$$
$$S = 20 \text{ mm} = 2 \text{ cm}$$
$$C = 25 \text{ mm} = 2.5 \text{ cm}$$
$$Q_u = \frac{\eta_h \times \eta_b \times WH}{S + \frac{C}{2}}$$
$$Q_u = \frac{0.8 \times 0.6 \times 80 \times 60}{2 + \frac{2.5}{2}}$$
$$Q_u = 708.9 \text{ kN}$$

17. (a)

18. (b)

(1) Bulk density $= \dfrac{1.855 \text{ kg}}{10^{-3} \text{ m}^3} = 1855 \text{ kg/m}^3$

$$\rho_d = \frac{\rho_b}{1+w} = \frac{1855}{1+0.15} = 1613.04 \text{ kg/m}^3$$

(2) $$\rho_d = \frac{(1-n_a)G\rho_w}{1+wG}$$

$$\Rightarrow 1613.04 = \frac{(1-n_a) \times 2.7 \times 1000}{1+0.15 \times 2.7}$$

$$n_a = 0.1607 = 16.07 \%.$$

19. (d)

$$e_{max} = 0.8$$
$$\dot{e}_{min} = 0.4$$
$$e = 0.6$$

Max dry density

$$\gamma_{d,max} = \frac{\gamma_w (G_s)}{1+e_{min}}$$
$$\gamma_{d,max} = \frac{9.81 \times 2.67}{1+0.4} = 18.71 \text{ kN/m}^3$$

Field density

$$\gamma_{d,field} = \frac{9.81 \times 2.67}{1+0.6} = 16.37 \text{ kN/m}^3$$

Relative compaction $= \dfrac{16.37}{18.70} = 0.875$

$\simeq 87.5\%$

20. (d)

21. (b)

22. (c)

Total pressure at

x = 4.5 + 1.5 × 0.5 + 2.25 × 1.5 = 8.625 t/m^2.

23. (c)

$$L = \frac{1.5}{\cos 10}$$
$$L = 1.523$$

total head at A = $(h_p)_A + (h_z)_A$

= 1.75 + 1.25 = 3m

total head at B = 0.95 m

L = 1.523

$$i = \frac{\Delta H}{L} = \frac{3-0.95}{1.523} = 1.346$$

24. (a)

$$q = 0.1 \text{ cc/sec}$$
$$\gamma_{sub} = 2 - 1 = 1 \text{ t/m}^2 = 1 \text{ gm/cc}$$
$$i = \frac{q}{AK} = \frac{0.1}{50 \times 2 \times 10^{-3}} = 1$$

eff. stress upward = $h\,\gamma_{sub} - iz\gamma_w$

[h = z = 0.5 cm]

0.5 × 1 – 1 × 0.5 × 1 = 0

25. (d)

26. (d)

According, to converse-lebarre equation

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

where $\theta = \tan^{-1}\left(\dfrac{d}{s}\right) = \tan^{-1}\left(\dfrac{d}{3d}\right)$

$= 18.43°$

As its square pile group so m = n = 4

$$\eta_g = 1 - \frac{18.43}{90}\left(\frac{4\times3+3\times4}{4\times4}\right)$$

$$= 69.28\%.$$

27. (a) Consistency represents the relative ease with which a soil can be deformed. This term is mainly used for clayey soil and is related to water content *i.e.*, how with change in water content the consistency of soil changes.

28. (b)

The dynamic head drop per equipotential is,

$$(\Delta h) = \frac{H_A - H_B}{N_{eq} =} = \frac{10-2}{14 \text{ drops}} = .57 \text{ m/drop}$$

Pressure head at A = 10 + 2 – Δh (2)

$= 10.86$ m

Pressure head at B = 10 + 2 – Δh (10)

$= 6.3$ m

The uplifting force F is,

$$F = L\left(\frac{P_A + P_B}{2}\gamma_w\right) = 50\left(\frac{10.86+6.3}{2}\right)(9.81)$$

$= 4208.49$ kN/m.

29. (a)

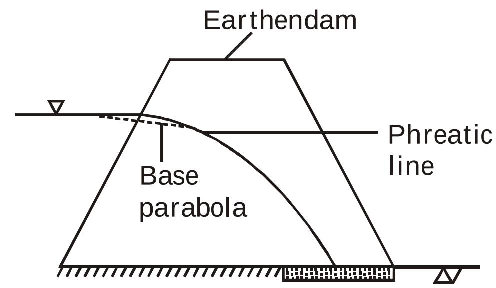
total head loss at x = $\frac{100}{40}\times30 = 75$ cm

total head = 100 – 75 = 25 cm

elevation head = 30 cm

$h_p = 25 - 30 = -5$

30. (c)



In a earthen dam

- Phreatic line is a flow line or stream line which is perpendicular to u/s wetted surface which is also called 100% equipotential line.
- Phreatic line represents top flow line of seepage flow and follows path of base parabola with small correction at entry point.
- Equation (x^2 – 6y) represents a parabolic profile thus it is correct.

31. (c)

$$q_{nu} = 5\left(1+.2\frac{D_f}{b}\right)\left[1+.2\frac{B}{L}\right]C$$

$$= 5\left(1+.2\times\frac{5}{10}\right)\left[1+.2\times\frac{10}{10}\right]\times30$$

$= 198$ kN/m^2

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

$= 79.2$ kN/m^2.

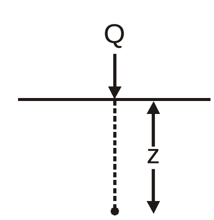
32. (d)

$$i_c = \frac{\gamma_{sub}}{\gamma_w} = \frac{G-1}{I+e},$$

Seepage pressure = $h_L\gamma_w$

33. (b)

34. (c) Stress at depth z just below the point load Q.



$$\sigma_z = k_B\frac{Q}{z^2}$$

$$k_B = \frac{3}{2\pi}\left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$$

$$\sigma_z = k_B \times \frac{Q}{z^2}$$

$$\sigma_z = 0.4775 \times \frac{50}{(5)^2}$$

$$\sigma_z = 0.955 \text{ kN/m}^2$$

35. (c)

36. (c)

$$N_C = N_R \frac{350}{\bar{\sigma}+70}$$

$$N_C = \frac{350}{18\times 6+70}\times 28$$

$$N_C = 55.06 \text{ say } 55.$$

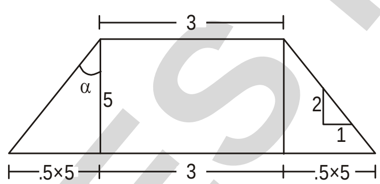
$$\frac{N_C}{N_R} = \frac{55}{28} = 1.96$$

which is less than 2 and greater than 1.5

So corrected N value = 55.

37. (c)

$$\tan\alpha = 0.5 = \frac{P}{B}$$



$$\sigma_z = \frac{1800}{(3+2\times 0.5\times 5)^2}$$

$$\sigma_z = \frac{1800}{8^2}$$

$$\sigma_z = 28.125 \text{ kN/m}^2$$

38. (d) Since ϕ = 20°, it is local shear failure. It is also given that C = 350 kN/m^2, γ = 17.5 kN/m^3, B = 1.5 m and D = 1m. For ϕ = 20°, N_C = 12.60, N_γ = 1.34 and N_q = 3.80. Let us assume the factor of safety to be 2.5

Strip footing

$$q_f = \frac{2}{3}C \,.\, N_C + \gamma D N_q + 0.5\gamma B N_\gamma$$

$$= \left(\frac{2}{3}\times 350\times 12.60\right) + (17.5\times 1\times 3.80) + (0.5\times 17.5\times 1.5\times 1.34)$$

$$= 2940 + 66.5 + 17.50 = 3024.09 \text{ kN/m}^2$$

$$q_{nf} = q_f - \gamma D$$

$$= 3024.09 - (17.5 \times 1) = 3006.59 \text{ kN/m}^2$$

$$q_{ns} = \frac{q_{nf}}{F} = \frac{3006.59}{2.5} = 1202.64 \text{ kN/m}^2$$

$$q_s = q_{ns} + \gamma D$$

$$= 1202.64 + (17.5 \times 1)$$

$$= 1220.14 \text{ kN/m}^2.$$

39. (d)

$$Q = C\, N_C\, A_b + \alpha\, C\, A_S$$

$$\Rightarrow 4\times 9\times\frac{\pi}{4}(0.5)^2 + \left(\frac{4}{2}\right)\times\pi\times 0.5\times 10$$

$$\Rightarrow 9\pi(0.5)^2 + (2\pi\times 5)$$

$$\Rightarrow \pi\left(9\times\frac{1}{4}+10\right)$$

$$\Rightarrow \frac{49\pi}{4}.$$

40. (d)

$$U = \frac{e_0 - e}{e_0 - e_f} = \frac{0.77-0.63}{0.77-0.58}\times 100 = 74\%.$$

41. (d) Tv is a factor which depends on the degree of consolidation and distribution of initial excess pore water pressure.

C_V is more $\Rightarrow$ Rate of settlement is more

C_c = 0.009 (W_L – 10)

42. (c) In comparison to atterberg limits of normal soils, expansive soils have

- More liquid limit
- Less plastic limit
- Less shrinkage limit
- More volumetric shrinkage.

This essentially means expansive soils have a large plastic region, represented by higher values of plasticity index.

43. (c)

$$\frac{t_1}{t_2} = \frac{H_1^2}{H_2^2}$$

$$H_1 = H, \quad H_2 = 2H$$

$$\frac{15}{t} = \frac{H^2}{2H^2}$$

$$t = 60 \text{ years}$$

The clay layer is 3 times more permeable and 4 times more compressible therefore time required for 90% consolidation = $60 \times \frac{4}{3}$.

$$t = 80 \text{ years}$$

44. (a)

- When over burden exits, the effective stress in soil at certain level is high but once it is eroded, effective stress reduces so that soil become preconsolidated.
- For a soil below glacier, glacier acts as overburden, after melting of glacier effective stress on soil got reduced.
- Due to lowering of ground water table capillary pressure sets up which increases the effective stress of soil.

45. (a)

46. (d)

$$S_c = \frac{H_0.C_c}{1+e_0}\log_{10}\left(\frac{\sigma_0+\Delta\sigma}{\sigma_0}\right)$$

$$C_c = 0.009\ (w_L - 10) = 0.234$$

$$S_c = \frac{300\times 0.234}{1+0.82}\log_{10}\left(\frac{2+1.5}{2}\right)$$

$$= 38.57 \times 0.24 = 9.26\,\text{cm}$$

47. (c)

48. (b)

49. (a)

$$D = 7.5 \text{ cm} = 0.075 \text{ m}$$

$$H = 11.25 \text{ cm} = 0.1125 \text{ m}$$

$$T = 40 \text{ N-m} = 40\times10^{-3} \text{ kN-m}$$

$$S = \frac{T}{\pi\left(D^2\frac{H}{2}+\frac{D^3}{6}\right)}$$

$$S = \frac{T}{\pi D^2\left(\frac{H}{2}+\frac{D}{6}\right)}$$

$$S = \frac{40\times10^{-3}}{3.14\times(0.075)^2\times\left[\frac{0.1125}{2}+\frac{0.075}{6}\right]}$$

$$S = 32.92 \text{ kN/m}^2$$

50. (c)

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

$$B = \frac{10-(-60)}{100-0} = 0.70$$

$$B = 0.70$$

$$AB = \frac{\Delta U_d}{\Delta\sigma_d}$$

$$AB = \frac{-70-10}{500-0}$$

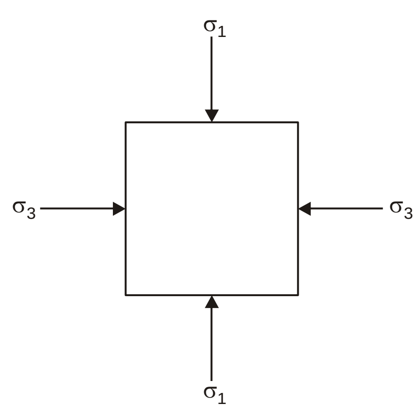
$$A \times 0.70 = \frac{-80}{500}$$

$$A = -0.23$$

51. (b)

52. (a)

In undrained triaxial test.



Assume at rest

$$\sigma_3 = \sigma_1 K_0$$

$$\Rightarrow \quad \sigma_3 = \sigma_1 \times \frac{\mu}{1-\mu}$$

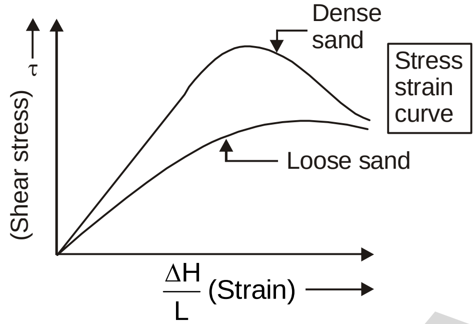
$$\Rightarrow \quad \frac{1-\mu}{\mu} = \frac{\sigma_1}{\sigma_3}$$

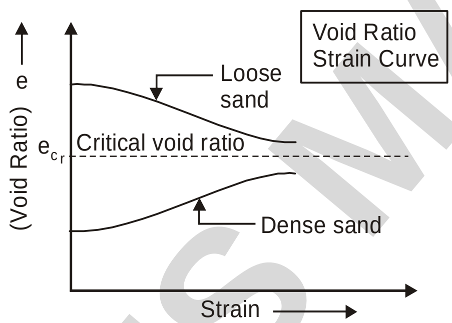
$$\Rightarrow \quad \frac{1}{\mu} = \frac{\sigma_1}{\sigma_3}+1$$

$$\Rightarrow \quad \frac{1}{\mu} = \frac{\sigma_1+\sigma_3}{\sigma_3}$$

$$\Rightarrow \quad \mu = \frac{\sigma_3}{\sigma_1+\sigma_3}$$

53. (a) Generally, the failure strain is 2 to 4% for dense sand and 12 to 16% for loose sand.





54. (a)

Assumptions of Terzaghi's theory

(i) Foundation is shallow ($D_f \le B$)

(ii) The base of foundation is rough.

(iii) Footing is continuous such as strip footing, which makes analysis two dimensional.

(iv) Terzaghi consider only base resistance and ignored side resistance.

55. (b)

Given, slope 1:1 i.e. $\beta = 45°$

for $\phi = 10°$ & $\beta = 45°$

$S_n = 0.108$

$$S_n = \frac{C}{\gamma HF}$$

$$0.108 = \frac{40}{16\times 5\times F}$$

$$\Rightarrow \quad F = \frac{40}{16\times 5\times 0.108}$$

$= 4.63$

$\therefore$ Critical height for slope

$H_C = 4.63 \times 5 = 23.15$ m.

56. (c)

Resisting Moment = CL × r

$$= 70 \times \left(\frac{2\pi\times r}{360°}\times 95°\right)\times 10.2$$

$$= 70 \times \frac{2\times\pi\times 10.2}{360}\times 95\times 10.2$$

$= 12075.34$ kN.m.

Actuating moment = W × 4 = 1050 × 4

= 4200 kNm

FOS =

$$\frac{\text{Resisting moment}}{\text{Actuating moment}} = \frac{12075.345}{4200} = 2.875.$$

57. (c) Maximum inclination in a cohesionless soil in infinite slope is equal to angle of internal friction. Stability is affected neither by the unit weight of soil nor by water content.

58. (a)

59. (d)

60. (d) Statement 3 and 4 are obviously correct as included in all options.

Otherwise : K_0 for OC clays 1 - 4

K_0 (OC clay) $=K_0$ (NC clay) $\sqrt{\text{O.C.R}}$

For Statement 1,

For perfectly cohesionless soil (c = 0)

$$K_0 = 1-\sin\phi$$

So it is also correct.

as $\sin\phi$ will be between 0 to 1 so $K_0 < 1$

Hence statement 2 is also correct.

For statement 5,

for dense sand $k_0 \rightarrow 0.4 - 0.5$

for loose sand $k_0 \rightarrow 0.45 - 0.5$

After mechanical compaction

$k_0 \rightarrow 0.8 - 1$

Hence, it is also correct.

Option (d) $\rightarrow$ All are correct.

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

62. (a)

63. (a)

64. (a) The Rankine's theory assumed that the wall surface is smooth whereas in practice, a lot of friction may develop between the wall surface and the soil fill. This friction will depend upon the wall material. This friction leads to the development of smaller active pressure and larger passive pressure than that estimated by Rankine's theory.

Thus, the estimation of the active pressure using Rankine's theory will be slightly higher than the actual (reduced due to friction) Passive pressure will be slightly lower.

65. (c) In actual practice, the friction leads to the development of smaller active pressure than that estimated by Rankine's theory and the larger passive pressure than the theoretical.

66. (b) The characteristics of flow net can be summarised as under :–

- The fundamental condition that is to be satisfied is that every intersection between a flow line and an equipotential line should be at right angles.
- The second condition to be satisfied is that the discharge between any two adjacent flow lines is constant and the drop of head between the two adjacent equipotential lines is constant.
- The ratio of the length and width of each field is constant. The ratio is generally taken as unity for convenience. In other words, the flow net consists of approximate squares.

67. (b) An isobar is a curve joining the points of equal stress intensity. In other words, an isobar is a contour of equal stress. An isobar is a spatial curved surface of the shape of an electrical bulb or an onion. The curved surface is symmetrical about the vertical axis passing through the load point.

68. (d) Stress path can be plotted for stress condition during triaxial test.

69. (d)

Process of consolidation

There are four main stages as follows:

1. **Initially:** equilibrium (or steady state) pore water pressure, u_0 is constant (and is simply the head of water, or hydrstatic pressure = $\gamma_w z$)
2. Load applied to soil surface increases total stress on soil sample, which generated a rise in pore water pressure – Soil particles try to move closen together– but prevented by incompresible pore water. Water pressure rises Δu (excess pore water pressure), to equal the total stresses increase;

   $u = \gamma_w z$ + applied stress = $u_0 + \Delta u$
3. **Dissipation of pore water pressure:** Over a period of time (Months to years), the excess pwp, Δu, dissipates (drains slowly out of the clay due to low permeability under sustained load – i.e squeezing from voids of pressurised pore fluid). The clay paricles take up new positions, resulting in settlement and increase in effective stress:

total (σ)	=	effective (σ')	+	p.w.p (u)
$\downarrow$		$\downarrow$		$\downarrow$
constant effective (σ')	=	increasing total (σ)		decreasing – p.w.p (u)
$\downarrow$		$\downarrow$		$\downarrow$
increasing		constant		decreasing

4. **Full dissipation of excess pwp ($\Delta u = 0$):** In the long term, excess pwp becomes zero and there is a maximum increase in effective stress, and pwp returns to its original value, u_0.

70. (c) Sand with an initial void ratio equal to critical void ratio will show no volume change when subject to shearing strain.

71. (b)

72. (a) Since flow through one large channel will be much greater than flow through a number of small channels having the same size of total channel area as the one large channel.

73. (c) A person can easily walk on damp sand near the sea beach because it possesses strength due to capillary moisture. On the same sand in saturated conditions, it becomes difficult to walk as the capillary action is destroyed.

74. (a)

75. (d) Under-reamed piles have a enlarged bulb, due to which bearing capacity is more than that of a straight bored pile of the same diameter.