Class Test Solution (SOIL) 02-09-2019
Answer key

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

CLASS TEST [SOIL] SOLUTIONS

1. (b)

$$G_m = \frac{\gamma_t}{\gamma_w} = \frac{G + Se}{1 + e}$$

For dry sample, $S = 0$

$$G_m = \frac{\gamma_t}{\gamma_w} = \frac{G}{1+e}$$

$$e = \frac{G - G_m}{G_m}$$

$$e = \frac{2.68 - 2.02}{2.02} = 0.326$$

2. (c)

$$Se = w \cdot G$$

S = 1 at saturation

$$\therefore \quad w_{sat} = e/G \qquad ....(i)$$

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

Substitute in equation (i)

$$w_{sat} = \frac{\frac{G\gamma_w}{\gamma_d} - 1}{G} \qquad \left\{\therefore G = \frac{\gamma_s}{\gamma_w}\right\}$$

$$w_{sat} = \frac{\gamma_w}{\gamma_d} - \frac{\gamma_w}{\gamma_s} = \gamma_w\left[\frac{1}{\gamma_d} - \frac{1}{\gamma_s}\right]$$

3. (b)

$$\rho_{max} = \frac{M}{840} \qquad \left(M = \text{Mass of the sample}\right)$$

$$\rho_{min} = \frac{M}{1370}$$

$$\rho_d = \frac{M}{1000}$$

$$D_r = \frac{\rho_{max}}{\rho_d}\left[\frac{\rho_d - \rho_{min}}{\rho_{max} - \rho_{min}}\right] \times 100$$

$$D_r = \frac{M/840}{M/1000}\left[\frac{\frac{M}{1000} - \frac{M}{1370}}{\frac{M}{840} - \frac{M}{1370}}\right] \times 100$$

$$D_r = 0.6981 \times 100$$

$$= 69.81\%.$$

4. (b)

$$V = 7.6 \times \frac{\pi \times 3.8^2}{4}$$

$$= 86.19 \text{ cm}^3$$

$$\rho_s = G_s\rho_w = 2.71 \times 1$$

$$= 2.71 \text{ g/cm}^3$$

$$V_s + V_w + V_a = 86.19$$

$$\frac{M_s}{2.71} + \frac{0.16\,M_s}{\rho_w} + 0.06 \times 86.19 = 86.19$$

$$\rho_w = 1 \text{ g/cm}^3$$

$$M_s = 153.1 \text{ gm}$$

$$M_w = 153.1 \times 0.16 \text{ gm}$$

$$M = M_s + M_w$$

$$= 153.1 + 0.16 \times 153.1$$

$$= 177.6 \text{ gm}$$

5. (b)

$V_A = 2 \text{ m}^3$ $\quad e_A = 0.6$

$V_B = 3 \text{ m}^3$ $\quad e_B = 0.8$

$V_C = 4 \text{ m}^3$ $\quad e_C = ?$

Soil A and B are mixed and compacted to form soil C. In the whole process of volume change only volume of voids will be changed, volume of soil solids will remain constant. So from equation

$$V_S = \frac{V_A}{1+e_A} + \frac{V_B}{1+e_B}$$

$$\frac{V_C}{1+e_C} = \frac{2}{1+0.6} + \frac{3}{1+0.8}$$

$$\frac{4}{1+e_C} = 1.25 + 1.67$$

$$\frac{4}{1+e_C} = 2.916$$

$$\boxed{e_C = 0.37}$$

Porosity of mixed soil

$$n = \frac{e_C}{1+e_C}$$

$$n = \frac{0.37}{1+0.37}$$

$$\boxed{n = 0.27}$$

6. (c)

7. (a)

Soil type dry unit weight (kN/m^3) in loosest state

Gravel	16
Coarse sand	15
Fine sand	14
Coarse silt	13

8. (b)

% passing on 4.75 mm sieve = 65

% retained on 75 m sieve = 92

so, the soil will be sand.

$C_c = 2.5 \quad 1 < C_c < 3$

$C_u = 7 \quad C_u > 6$

So, the soil is well graded sand

% passing 75 μ sieve = 8

This is

between 5 and 12, so soil will have dual symbol

P.I of soil = 3

on plasticity chart equal of A–line

$I_P = 0.73 \cdot (W_L - 20)$

$= 0.73 \cdot (15 - 20)$

$I_P = 0$

Plasticity index is less than 4% so fine soil is silt.

So, correct classification SW – SM

9. (c)

	Retained	% Retained	% Cumulative Retained	% finer
4.25	4	4	4	96
2.75	21	21	25	75
1.8	15	15	40	60
0.425	30	30	70	30
75 μ	20	20	90	10
< 75 μ	10	10	100	0

$D_{60} = 1.8$ mm

$D_{30} = 0.425$

$D_{10} = 0.075$ mm

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.8}{0.075} = 24$$

$$C_c = \frac{D_{30}^2}{D_{60} \cdot D_{10}} = \frac{(0.425)^2}{1.8 \times 0.075}$$

$= 1.33$

10. (d)

11. (b)

12. (c)

Montmorillonite has largest specific surface areas hence can assimilate more water.

Atterberg limit values for clay minerals with adsorbed cations.

	w_L	I_P
Na - Kaolinite	29	I
Na - Illite	41	27
Na - montmorillonite	344	251

13. (b)

14. (b)

$\rho_d = 1.75$ g/cc

$n_a = 5\%$

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

$$1.75 = \frac{(1-0.05)\times G \times 1}{1+0.16\times G}$$

$G = 2.612$

Maximum $(\rho_d)_{theoretical}$

$$= \left(\frac{G}{1+wG}\right)\times \rho_w$$

$$= \frac{2.612\times 1}{1+0.16\times 2.612}$$

$$\boxed{(\rho_d)_{theoretical} = 1.842\text{ g/cc}}$$

15. (c)

16. (d)

17. (a)

$$\frac{V}{n} = V_s$$

$$n = \frac{2.5\times 10^{-2}}{5.85\times 10^{-2}}$$

$$n = 0.427$$

$$e_1 = \frac{n}{1-n} = 0.745$$

$$e_2 = \frac{0.39}{1-0.39} = 0.639$$

$$\frac{k_1}{k_2} = \frac{e_1^3}{(1+e_1)}\frac{(1+e_2)}{e_2^3}\times\frac{\mu_2}{\mu_1}$$

$$\frac{1.826\times 10^{-1}}{K_2} = \frac{0.745^3}{1.745}\times\frac{1.639}{(0.639)^3}\times\frac{10.09}{8.95}$$

$$K_2 = 1.088\ 0\times 10^{-1}\text{ cm/s}$$

18. (c)

The effective stress at a depth below 2.5 m

$$\bar{\sigma} = \sigma_0 - u$$

$$= 18.5\times 2 + 20.5\times 0.5 - (-0.5\times 9.81)$$

$$\boxed{\bar{\sigma} = 52.155\text{ kN/m}^2}$$

19. (b)

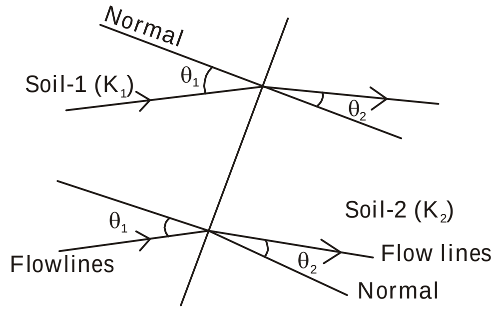
20. (a)

$$h_c = \frac{C}{eD_{10}}$$

$$= \frac{0.3}{0.05\times 10^{-1}\times 0.85}$$

$$= 70.6\text{ cm} = 0.706\text{ m}$$

21. (b)



When two different soils are used in a soil mass, thus making it non-homogeneous. The flow lines and equipotential lines get deflected at the interface. The flow net thus get modified.

K_1 and K_2 are related as

$$\frac{K_1}{K_2} = \frac{\tan\theta_1}{\tan\theta_2}$$

22. (d)

uniqueness of flow net means the value of n_f/ n_d ratio to be same

23. (b)

Tension Piles : These piles are in tension. These piles are used to anchor down structures subjected to hydrosatic uplift forces or overturning forces.

Friction Piles : Friction piles do not reach the hard stratum. These piles transfer the load through skin friction between the embedded surface of the pile and the surrounding soil. Friction piles are used when a hard stratum does not exist at a resonable depth. The ultimate load (Q_u) carried by the pile is equal to the load transmitted by skin friction (Q_s). The term friction pile is actually a misnomers, as in the clayey soils, the load is tranferred by adhesion and not friction between the pile surface and the soil. The friction piles are also known as floating piles as these do not reach the hard stratum.

Compaction Piles : These piles are driven into loose grannular soils to increase the relative density. The bearing capacity of the soil is increased due to densification caused by virbations.

Batter Piles: Vertical piles have

conventionally not been used for taking up any lateral (horizontal) load although they are capable to withstand small lateral loads. When the applied horizontal load per pile exceeds the value suitable for the said vertical piles, inclined piles called batter piles or raker piles are used in combination with vertical piles. The degree of batter i.e., the angle made by the pile with the vertical may be as high as 30°. Such lateral loads may quite possibly be caused on pile foundations due to wind and seismic forces in buildings, earth pressures in earth retaining structures like retainig walls and abutments, water pressures in water front structurs etc.

24. (d)

$$\gamma_{sat_1} = 20.2 \text{ kN/m}^3$$

$$\gamma_{sat2} = 19.1 \text{ kN/m}^3$$

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

$$\gamma_1' = 20.2 - 9.81$$

$$\gamma_1' = 10.39 \text{ kN/m}^3$$

$$\gamma_2' = 19.1 - 9.81$$

$$\gamma_2' = 9.29 \text{ kN/m}^3$$

$$h = 2.5 \text{ m}$$

Effective stress at AB

$$= \gamma_1' H_1 + \gamma_2' H_2 + h\gamma_w$$

$$\boxed{\bar{\sigma}_{AB} = 44.2 \text{ kN/m}^3}$$

25. (a)

26. (b)

$$e = \frac{n}{1-n}$$

$$e = \frac{0.40}{0.60} = 0.667$$

$$\text{Critical gradient} = \frac{G_s - 1}{1+e} = \frac{2.65-1}{1+0.667}$$

$$\boxed{i_{cr} = 0.99}$$

Saturated density

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\rho_w$$

$$\gamma_{sat} = \left(\frac{2.65+0.667}{1+0.667}\right) \times 9.81$$

$$\boxed{\gamma_{sat} = 19.52 \text{ kN/m}^3}$$

Let x be the depth of sand layer effective pressure at bottom

$$= (1.50 + x)(19.52 - 9.81)$$

$$= (1.50 + x)\ 9.71$$

Upward pressure at bottom

$$= 1.95 \times 9.81$$

Now, factor of safety

$$\text{F.S.} = \frac{(1.50 + x) \times 9.71}{1.95 \times 9.81} = 2.50$$

$$\boxed{x = 3.42 \text{ m}}$$

27. (a)

$$\sigma_z = I_f qN$$

$$= 0.005 \times 10 \times 10$$

$$\Rightarrow \quad 0.5 \text{ t/m}^2.$$

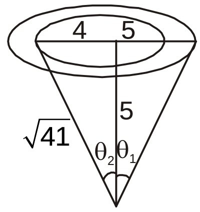
28. (b)

$$\theta_1 = 45°$$

$$\cos\theta_2 = \frac{5}{\sqrt{41}}$$

$$\sigma_z = q\left(1 - \cos^3\theta_1\right) - q\left(1 - \cos^3\theta_2\right)$$

$$= 100\left(1 - \frac{1}{2\sqrt{2}}\right) - 100\left(1 - \left(\frac{5}{\sqrt{41}}\right)^3\right)$$



$$= 100(1 - 0.353) - 100(1 - 0.476)$$

$$\Rightarrow 100(0.647) - 100(0.524)$$

$$\Rightarrow 12.3 \text{ kN/m}^2.$$

29. (a)

$$\frac{\sigma_1}{\sigma_2} = \frac{z_2^2}{z_1^2} = \frac{25}{9} = \frac{25}{9} = 2.77$$

30. (d)

31. (a)

Since this soil layer is singly drained

$$H = 8m$$

Similarly for 90% consolidation

$$T_{90} = \frac{C_v t_{90}}{H^2}$$

$$(\because \quad T_{90} = 1.781 - 0.933 \log(100 - 90) = 0.848)$$

$$t_{90} = \frac{0.848 \times 8^2}{2 \times 10^{-3} \times 10^{-4}} \text{sec}$$

$$= 8.6 \text{ yr}$$

32. (c)

33. (b)

The reckoning of time is conventionally done from mid-way through construction of loading period.

$$S_4 = 117 \text{ mm}$$

$$t = 4 \text{ years}$$

$$S_c = 360 \text{ mm}$$

$$S_9 = ?; \quad t = 4 \text{ year}$$

$$\frac{S_4}{S_9} = \frac{u_4}{u_9} = \sqrt{\frac{T_{v_4}}{T_{v_9}}} = \sqrt{\frac{t_4}{t_9}} = \sqrt{\frac{t_4}{t_9}}$$

$$\left[\because \frac{C_v}{H^2} = \text{constant}\right]$$

$$\frac{117}{S_g} = \sqrt{\frac{4}{9}}$$

$$S_g = \frac{3}{2} \times 117$$

$$= 175.5 \text{ mm.}$$

34. (d)

Thickness of clay layer = 2000 mm

Pressure increment, $\Delta\sigma$ = 25 kPa

Initial stress, σ_1 = 100 kPa

Specific gravity, G_S = 2.7

Moisture content, w = 40%

Compressive index, C_c = 0.40

Degree of saturation, S = 100%

We know $Se = WG_S$

$$1 \times e_0 = 0.40 \times 2.7$$

$$e_0 = 1.08$$

$$C_c = \frac{\Delta e}{\log \frac{\sigma_2}{\sigma_1}}$$

$$0.40 = \frac{\Delta e}{\log\left(\frac{\sigma_1 + \Delta\sigma}{\sigma_1}\right)}$$

$$0.40 = \frac{\Delta e}{\log\left(\frac{125}{100}\right)}$$

$$\Delta e = 0.038$$

For one dimensional consolidation

$$\frac{\Delta H}{H} = \frac{\Delta e}{1 + e_0}$$

$$\Delta H = 2000 \times \frac{0.038}{1 + 1.08}$$

$$\Delta H = 36.5 \text{ mm.}$$

35. (c)

$$E = 80 \times 10^3 \text{ kPa}$$

Poisson's ratio, μ = 0.35

Influence factor, I_f = 2.0

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

$$q = 40 \text{ kN}$$

$$= \frac{40 \times 2 \times (1 - 0.35^2)}{80 \times 10^3} \times 2.0$$

$$= 1.75 \text{ mm}$$

36. (b)

37. (a)

38. (b)

$$\frac{\Delta H}{H} = \frac{\Delta V}{V}$$

$$\frac{\Delta H}{H} = \frac{\Delta V}{V} = \frac{\Delta e}{1+e_0} = \frac{e_0 - e}{1+e_0}$$

$$\frac{5\text{ mm}}{50\text{ mm}} = \frac{1.25 - x}{1+1.25}$$

$$x = 1.025$$

39. (c)

Casagrande's logarithm of time fitting method is used to determine coefficient of consolidation C_v.

40. (a)

- For 100% consolidation

  Time factor, $T_v \to \infty$

hence, time take for 100% consolidation $t \to \infty$

- According to Terzaghi's one dimensional theory of consolidation there is a unique relationship between void ratio and effective stress. But, in secondary compression relationship between void ratio and effective stress is not unique, as secondary compression occurs at constant effective stress.

So secondary consolidation does not obey Terzaghi's 1 D theory of consolidation.

- Initial excess pore water pressure distribution leads to change in the time factor. For example, time factor for the initial pore water pressure as shown in Fig. (a) is less than that for Fig. (b)

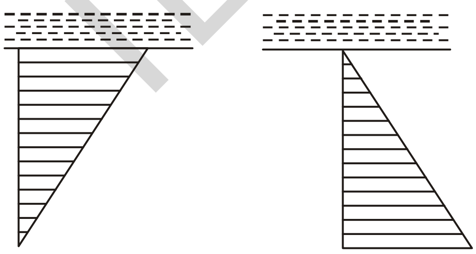


Fig (a) Fig (b)

41 (d)

42. (d)

Test No. 1 : $\sigma_3 = 50$

$$\sigma_1 = 50 + 120 = 170$$

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

$$170 = 50\tan^2\alpha + 2c\tan\alpha \quad ....(1)$$

Test No. 2 : $\sigma_3 = 100$

$$\sigma_1 = 240$$

$$240 = 100\tan^2\alpha + 2c\tan\alpha \quad ....(2)$$

(2) & (1) $\quad 70 = 50\tan^2\alpha$

$$\tan\alpha = 1.18$$

$$\alpha = 49.79° \quad \left\{\because \alpha = 45 + \frac{\phi}{2}\right\}$$

$$\phi = 9.6°$$

Substituting :

$$170 = 50\left(\frac{70}{50}\right) + 2c(1.18)$$

$$100 = 2 \times 1.18 \times C$$

$$C = 42.372 \text{ kN/m}^2.$$

43. (c)

$$\tan\phi = \tau/\sigma = 4/10 = 0.4$$

Hence, the angle of internal friction

$$\phi = 21°48'$$

The angle of inclination

$$\theta = 45° + \frac{\phi}{2}$$

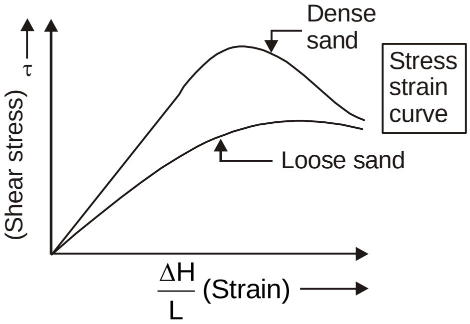
$$= 45° + \frac{21°48'}{2} = 55°54'$$

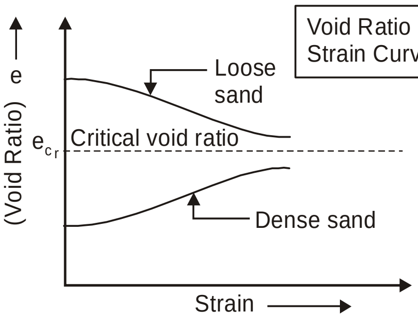
44. (c)

Meyerhof and Adams gave equations for the pull-out resistance of piles which involve diameter of enlarged base, undrained cohesion, length pile, angle of shearing resistance, etc.

45. (a)

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



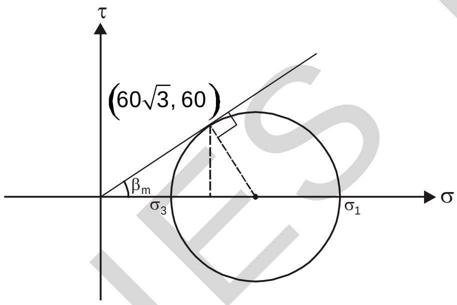


46. (d)

In the case of loose sand, the specimen bulges and ultimately fails by sliding simultaneously on numerous planes. The failure is known as the plastic failure. In the case of dense sand, the specimen shows a clear failure plane and the failure is known as the brittle failure.

47. (b)

Assuming that plane of failure is the plane of maximum obliquity.



$$(\sigma_f, \tau_f) = (60\sqrt{3}, 60)$$

$$\tan\beta_m = \frac{60}{60\sqrt{3}} = \frac{1}{\sqrt{3}}$$

$$\Rightarrow \quad \beta_m = 30°$$

$$\sigma_1 = \frac{\sigma_f}{1-\sin\beta_m} = \frac{\sigma_f}{1-\sin 30°}$$

$$= \frac{60\sqrt{3}}{1-\frac{1}{2}} = 120\sqrt{3}\ \text{N/mm}^2$$

48. (b)

Effective stress = Total stress – pore water pressure

$$\bar{\sigma} = \sigma - u$$

$$= 300 - 150$$

$$= 150\ \text{kN/m}^2$$

Shear strength,

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

$$= 10 + 150\tan 30^o$$

$$= 96.6\ \text{kN/m}^2$$

49. (b) Limitation of direct test:

(i) This test cannot be performed as an undrained test because there is no arrangement to measure excess pore pressure which develops in undrained tests.

(ii) In this test, the lateral pressures and stresses on planes other than the plane of shear are not known, unlike in a triaxial compression test.

(iii) The horizontal failure plane is pre-determined, which may not be the weakest.

(iv) As the test progresses, the area under shear gradually decreases. The corrected area (A_f) at failure should be used in determining the value of σ and τ.

(v) There is some effect of lateral restrain caused by the side walls of the shear box.

50. (d)

The soil sample is encased by a flexible membrane or jacket and two end caps. Thus, the confining fluid does not penetrate into the pore spaces.

51. (c)

Here, $D_f = \frac{12}{10} = 1.2$

and $S_n = \frac{C}{\gamma HF} = \frac{30}{19 \times 10 \times 1.5} = 0.105$

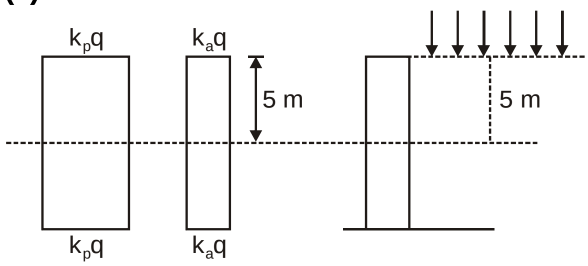
Slope; $i = 30 + \frac{15-30}{0.101-0.143} \times (0.105 - 0.143)$

$= 16.43°$

52. (c)

53. (b)

54. (c)



Angle of internal friction

$$\phi = 30°$$

Active earth pressure coefficient

$$k_a = \frac{1-\sin\phi}{1+\sin\phi}$$

$$k_a = \frac{1-\sin 30°}{1+\sin 30°} = \frac{1/2}{3/2} = \frac{1}{3}$$

Passive earth pressure coefficient

$$k_a \times k_p = 1$$

$$k_p = 3$$

Maximum horizontal stress at depth 5 m

$$= 30 \times 3 \times 1 = 90 \text{ kN/m}^2$$

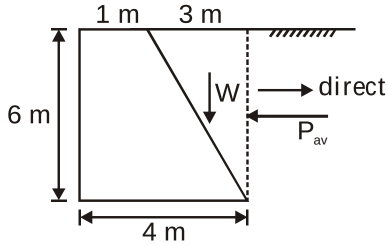
Minimum horizontal stress at depth 5 m

$$= 30 \times \frac{1}{3} = 10 \text{ kN/m}^2$$

55. (c)

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30°}{1+\sin 30°}$$

$$K_a = \frac{1}{3}$$



$$P_{av} = \frac{1}{2} K_a \gamma H^2$$

$$P_{av} = \frac{1}{2} \times \frac{1}{3} \times 19.2 \times 36$$

$$P_{av} = 115.2 \text{ kN}$$

(per unit length of wall)

$$w = \frac{1}{2} \times 3 \times 6 \times 19.2 \times 1$$

$$w = 172.8 \text{ kN}$$

Resultant pressure force

$$R = \sqrt{(P_{av})^2 + w^2}$$

$$\boxed{R = 207.67 \text{ kN}}$$

56. (b)

57. (a)

$P_p = \frac{1}{2} k_p \gamma H^2 = \frac{1}{2} \times 3 \times 1.8 \times 3^2 =$ **24.3 t**

58. (b)

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 than that estimated on Rankine's theory and the larger passive pressure than the theoretical. Thus, the estimation of the active pressure from the Rankine's theory will be slightly higher than the actual (reduced due to friction) and passive pressure will be slightly lower. It, thus, remains on a safer side and hence the design of the retaining walls can be done safely on estimation of active and passive pressures computed by the Rankine's theory.

59. (a)

As the soil is purely cohesive, ultimate bearing capacity will not be dependent on shape of the footing so

Ratio = 1

60. (a)

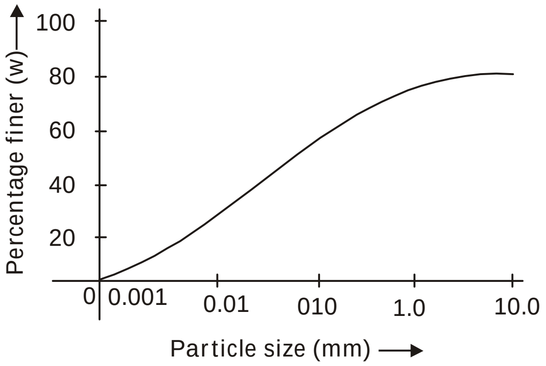
61. (b)

62. (d)

This method for the determination of water content is quite suitable for coarse-grained soils from which the entrapped air can be easily removed.

63. (a)

64. (b)



The semi-log plot for the particle size distribution, as shown in figure, has the following advantages over natural plots.

1. The soils of equal uniformity exhibit the same shape, irrespective of the actual particle size.
2. As the range of the particle sizes is very large, for better representation, a log scale is required.

65. (c)

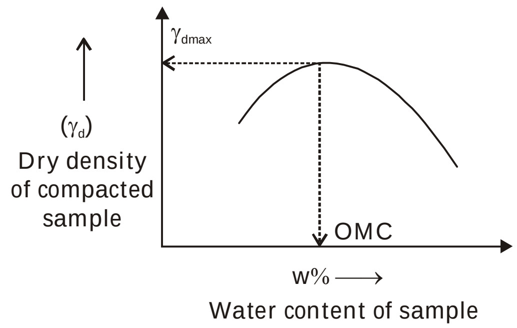
Soil particles smaller than 0.0002 mm equivalent diameter develop Brownian movement and affects their settlement.

66. (c)

The magnitude of permeability variation with soil composition ranges widely. The lower the ion exchange capacity of a soil, the lower, of course, the effect of exchangeable ion on permeability.

67. (a)

Cohesive soils have lower $\gamma_{d\max}$ and high values of OMC because they require more water for lubrication because of high specific surface area.



Heavy clays with high plasticity have very low values of $\gamma_{d\max}$ and very high values of OMC.

An increase in organic content in clay soils leads to an increase in OMC value and decrease in $\gamma_{d\max}$.

68. (d)

69. (a)

$\frac{n_f}{n_d}$ ratio, is called shape factor.

70. (c)

Water pressure is also called neutral stress because it acts on all sides of particles but does not cause the soil particles to press against adjacent particles hence does not cause compression in the soil.

71. (a)

72. (a)

The tendency to decrease in volume causes an increase in pore water pressure which cannot dissipate under undrained condition. Indeed, there is an increase in pore water pressure under successive cylces of loading. If the pore water pressure becomes equal to maximum effective stress component, normally the over burden pressure the value of effective stress in zero, i.e., interpartical force will be zero, and the sand soil will exist in a liquid state with negligible shear strength.

73. (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.

74. (d)

75. (c)

Assumptions of Terzaghi's theory

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

(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.