Class Test Solution (Soil) 27-04-2019

Answer key

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

1. **(d)**

Average degree of consolidation = $\frac{A_1}{A_1 + A_2}$

= 66.67 %

$T_{v_2} > T_{v_1}$ as time increase T_v and degree of consolidation also increase.

2. **(d)**

If the load is applied to the piston with the valve kept closed, the piston load is apportioned by the water and the spring in relation to the stiffness of each. The piston will move very little when the load is applied because the water is relatively incompressible. Essentially all of the applied load is resisted by an increase in the fluid pressure within the chamber. Next we open the valve. The fluid pressure within the chamber will force water through this valve. As water escapes the spring shortens and begins to carry a significant fraction of the applied load. There must be a corresponding decrease in the pressure within the chamber fluid. Eventually a condition is reached in which all of the applied load is carried by the spring and the pressure in the water has returned to the original hydrostatic condition. Once this stage is reached, there is no further flow of water. Sharing the load between the mineral and pore phases also occurs in actual soil problems, although the pore fluid will not always carry all of the applied load initially.

3. (d)

4. **(c)**

The secondary consolidation depends upon the shear stresses and on the degree of the disturbances of the sample. The rate increases with a decrease in the thickness of the specimen used in the test.

5. **(b)**

6. **(c)**

Clay X	Clay Y
$w_L = 42$ $w_P = 20$ $w_N = 30$	$w_L = 56$ $w_P = 34$ $w_N = 50$

Plasticity index,

Clay X

$I_p = w_L - w_P = 42 - 20 = 22$

Clay Y

$I_P = 56 - 34 = 22$

Settlement of clay layer

$$\Delta H = \frac{C_c H_0}{1+e_0}\log_{10}\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)$$

$\Delta H \propto C_c$

Coefficient of compression,

$C_c = 0.009\ (w_L - 10)$

$C_c \propto w_L$

Hence $\Delta H \propto w_L$

- Clay y will experience larger settlement.
- I_P is same in both the samples, w_L of clay y is more so it's dry strength will be less and plasticity is a measure of dry strength so plasticity of sample x will be more.
- Consistency $I_c = \frac{w_L - w_N}{I_P}$

$$(I_c)_x = \frac{42-30}{22} = 0.54$$

$$(I_c)_y = \frac{56-50}{22} = 0.27$$

I_c of y is less so y is softer in consistency.

7. **(d)**

Larger the gradation higher will be shear strength and angle of friction.

8. (d)

Uniformity coefficient,

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.8}{0.1} = 8$$

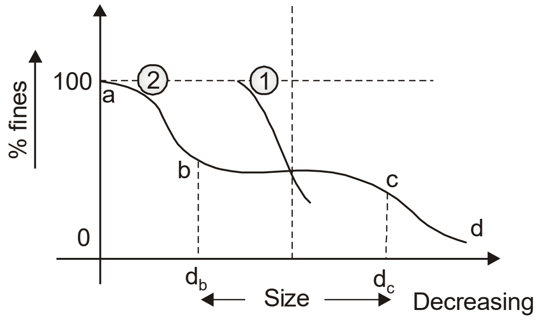
Coefficient of curvature,

$$C_c = \frac{D_{30}^2}{D_{60}D_{10}} = \frac{(0.3)^2}{0.8 \times 0.1} = 1.125$$

9. **(d)**

A curve with a hump represents the soil in which some of the intermediate particles are missing. A flat s-curve represents a soil which contains the particles of different sizes in good proportion.

10. (a)



- Curve 1 : Poorly graded or uniformly graded shows prevalence of nearly uniform grain size.
- In curve 2 flat part bc represents missing particle of diameter between d_b and d_c.
- A conspicuous break in the continuity of the grain size curve may also indicate the simultaneous deposition of the soil by two different agents. For instance, one fraction might be washed into a glacial lake by a river and another fraction dropped from melting ice floats.

11. (c)

Activity (A) of soil is the ratio of the plasticity index and the percentage of clay fraction (minus 2 μ size)

$$A = \frac{I_P}{F} \quad \text{... (i)}$$

I_P = Plasticity index

F = Clay fraction (percentage finer than 2μ size)

$I_P = W_L - W_P$

W_L = Liquid limit = 65%

W_P = Plastic limit = 29%

$I_P = 65 - 29 = 36\%$

F = 24

Using (i)

$$A = \frac{36}{24} = 1.5$$

A > 1.25 are active soils

12. (*)

13. (b)

All types of soils carried and deposited by water are known as alluvial deposits. Deposits made in lakes are called lacustrine deposits. Marine deposits are formed when the flowing water carries soils to ocean or sea. Soils deposited by wind are known as Aeolian deposits.

∴ Correct option is (b).

14. (a)

Shrinkage ratio

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

SR = 1.905

15. (d)

All the limits and indices with the exception of the shrinkage limit are determined on soils that have been thoroughly worked into a uniform soil-water mixture.

16. (d)

17. (d)

The shearing strength of the soils is extremely low. The soils have very low bearing capacity. It is extremely difficult to work with such soils.

18. (c)

Cohesion is the term used to discribe the strength of clay sample when it is unconfined being due to negative pressure in the water filling the voids space, of very small size, between particles.

19. (d)

20 (a)

For montmorillonite, K<Na<H<ca

For kaolinite Na<K<a<H

21. (a)

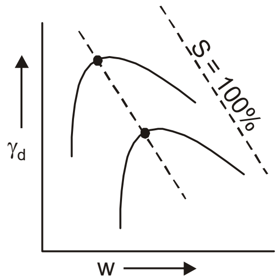
Relative compaction= Degree of compaction achieved as a percentage of the laboratory compaction R(%)

$$= \frac{\gamma_d \text{in the field}}{\text{Maximum } \gamma_d \text{from the proctor test}} \times 100$$

$$\text{Relative density} = \frac{e_{max} - e_{natural}}{e_{max} - e_{min}} \times 100$$

Note: Relative density can be zero but relative compaction can never be zero.

22. **(d)**



23. **(d)**

24. **(b)**

25. **(c)**

A well graded sand attains a much higher dry density than a poorly graded sand for same compactive effort.

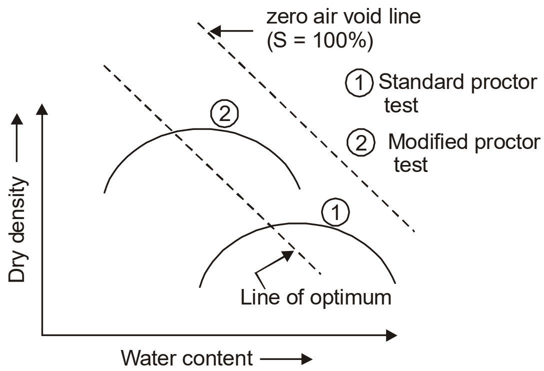
Cohesive soil has high air voids. This soil attains a relatively lower maximum dry density as compared to the cohesionless soil. It also requires more water than cohesionless soil and therefore the optimum water content is high. Heavy clay of very high plasticity have very low dry density and a very high optimum water content.

Hence the correct option is (c).

26. **(c)**

Cohesive soils have high air voids. These soils attain a relatively lower maximum dry density as compared with the cohesionless soils. Such soils require more water than cohesionless soil and therefore, the optimum water content is high.

Double layer formation is not accured in cohesionless soil so there is a little use of compaction curve in cohesionless soil.



For sandy soils, the relative density is used as a criterion for measurement of compactness.

27. **(a)**

When comparing the properties of two soils with equal values of plastic index, it is found that as the liquid limit increases, the dry strength and toughness decrease, whereas compressibility and permeability increase.

When comparing the properties of two soils with equal liquid limits, it is found that as the plasticity index increases, the dry strength and toughness increase, whereas the permeability decreases. However, the compressibility remains almost the same.

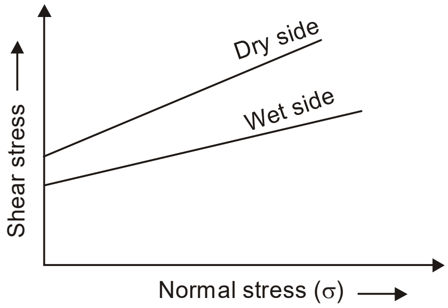
28. **(c)**

Relationship between atterberg limits and engineering properties of soil:

Properties	**Fix W_L and increase I_P**	**Fix I_P and increase W_L**
Strength	increases	decreases
Toughness	increases	decreases
Compressibility	constant	increases
Permeability	decreases	increases
Rate of vol. change	decreases	increases

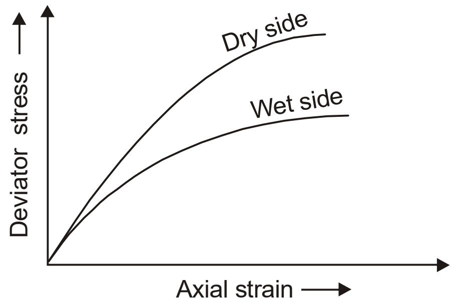
29. **(d)**

Soils compacted at a water content more than the optimum water content usually have a dispersed structure if the compaction induces large shear strains and a flocculated structure if the shear strain are relatively small.



Failure Envelops

The soils compacted dry of the optimum have a steeper stress-strain curve than those on the wet side.



30. (a)

31. **(b)**

32. **(c)**

Smooth Wheeled rollers are useful for finishing operations after compaction of fills and for compacting granular base course of highways. (can be used for fine soils)

Pneumatic tyred rollers are effective for cohesive as well as cohesionless soils. (compacts by kneading action)

Sheep foot rollers are ideally suitable for compaction of cohesive soils. (compacts by combination of tamping and kneading action).

Vibratory Compactors are used for granular soils. (not for fine grained soil)

Hence most appropriate option is (c).

33. **(d)**

34. **(d)**

As per Indian standard light compaction test a hammer of 2.6 kg is allowed to fall from a height of 310 mm and 3 layers are tamped 25 times in a mould of volume 1000 cc

$\therefore$ Energy imparted,

$= 2.6 \times 25 \times 3 \times 9.81 \times \dfrac{310}{1000}$

$= 593.01$ Nm

For embankment compaction, the volume of soil covered by rammer,

$= 0.05 \times 10^4 \times \dfrac{500}{10}$

$= 25000\,\text{cc}$

Energy imparted in each pass,

$= (1 + 50\%) \times 400$

$= \left(1 + \dfrac{50}{100}\right) \times 400$

$= 600$ N.m

If n number of passes are required to develop equivalent energy to Indian standard light compaction test then,

$n \times \dfrac{600}{25000} = \dfrac{593.01}{1000}$

$\Rightarrow n = 24.71$

$\simeq 25$

35. **(b)**

Volume of levee = Area of trapezoidal × length of levee $V = \dfrac{1}{2}(5+30) \times 5 \times 130$

$= 11375\ m^3$

<u>For earth levee</u>

$R.C = \dfrac{\gamma_d\ \text{field}}{\gamma_d\ \text{proctor}} = 0.95$

$\gamma_{df} = 0.95 \times 16.5 = 15.675\ kN/m^3$

$\gamma_d = \dfrac{G\gamma_w}{1+e} = 15.675 = \dfrac{2.68 \times 9.81}{1+e}$

$e = 0.677$

$e = \dfrac{V_T - V_S}{V} = 0.677 = \dfrac{11375 - V_S}{V_S}$

$V_S = 6782.94\ m^3$

For borrow pits

$$17.80 = \frac{G\gamma_w(1+w)}{1+e} = \frac{2.68\times 9.81\times(1+0.18)}{1+e}$$

$e = 0.74$

$$e = \frac{V_T - V_S}{V_S}$$

$$0.74 = \frac{V_T - 6782.4}{6782.4}$$

$V_T = 11801.37\ m^3$

36. **(a)**

$$\text{No. of trucks} = \frac{V_T \text{ excavated}}{V_T \text{ Truck}}$$

e = 1.47 for excavated soil

$$e = \frac{V_T - V_S}{V_S}$$

$$1.47 = \frac{V_T - 6782.4}{6782.4}$$

$V_T = 16752.528\ m^3$

$$\text{Number of truck} = \frac{16752.528}{6}$$

= 2792.08 trucks.

37. **(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

38. **(b)**

Piezometric head = Pressure head + Potential head

$$= \frac{P}{\gamma} + Z$$

Apply energy between D and C

$$\frac{P_D}{\gamma} + \frac{V_D^2}{2g} + Z_D = \frac{P_c}{\gamma} + \frac{V_c^2}{2g} + Z_c + h_L$$

$h_L = 0$ {Because head loss occurs in only resisting medium}

$$0 + 0 + 40 = \frac{P_c}{\gamma} - 80 + 0$$

$$\frac{P_c}{\gamma} = 120\text{ cm}$$

And potential head

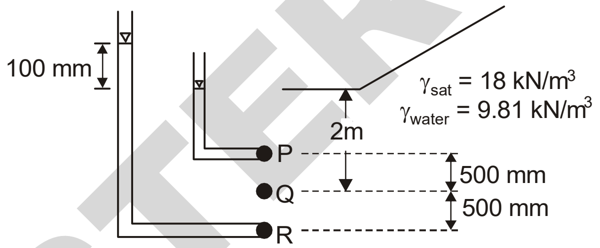
$Z_c = -80$ cm

$$\text{Piezometric head} = \frac{P_c}{\gamma} + Z_c$$

= 120 – 80 = 40 cm

Hence, answer is (b).

39. **(b)**



Head loss from R to Q = difference of piezometric Level = 100 mm

$$\text{Loss per unit length} = \frac{100}{500+500}$$

= 0.1

$= \frac{1}{10}$ Hence, loss upto

$$Q = \frac{500}{10} = 50\text{ mm}$$

$\Rightarrow$ Piezometric level at

Q = 2.050 m

$\Rightarrow$ Pore water pressure at

Q = 2.050 γ_w

Seepage is taking place in upward direction

Effective stress at = 2.05 × 9.81 = 15.89 kN/m^2

40. **(d)**

$\gamma_{sat_1} = 19.8\ kN/m^3$

$\gamma_{sat_2} = 20.3\ kN/m^3$

$\gamma_1' = 9.99\ kN/m^3$

$\gamma_2' = 10.04 \text{ kN/m}^3$

Piezometric head at C = 5 m

Piezometric head at B = 2 m

The seepage flow is occuring under head of

h = 5.0 – 2 – 1 – 0.5

h = 1.5 m

Effective stress at C

$= \gamma_1' \times 1 + \gamma_2' \times 2 - h\gamma_w$

$\boxed{\bar{\sigma}_c = 15.355 \text{ kN/m}^3}$

41. (c)

Horizontal seepage per unit volume

$= i\gamma_w = 0.25 \times 9.81$

= 2.45 kN

Vertical force/volume = 15 + 19.95

= 34.95

resultant body force = $\sqrt{2.45^2 + (34.95)^2}$

= 35.03 kN

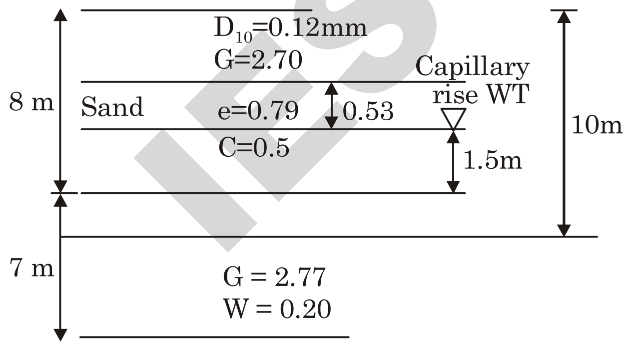
42. (b)

The seepage force per unit volume is equal to the product of the hydraulic gradient (i) and the unit weight of water.

43. (d)

The capillary potential is minimum when the water content is minimum.

44. (a)



Height of capillary rise

$$h_c = \frac{C}{e_{D10}} = \frac{0.5}{0.79 \times 0.012} = 52.74 \text{ cm}$$

= 0.53 m

$$\gamma_{sat\,sand} = \left(\frac{G+e}{1+e}\right)\gamma_w = 19.12 \text{ kN/m}^2$$

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

$$= \left(\frac{2.77 + 0.2 \times 2.77}{1 + 0.0 \times 2.77}\right) \times 9.81$$

= 20.98 kN/m²

$$\gamma_{dry\,sand} = \left(\frac{G\gamma_w}{1+e}\right)$$

$$= \frac{2.70 \times 9.81}{1.79} = 14.79 \text{ kN/m}^2$$

total stress at 10 m below

= 5.97 × 14.79 + 2.03 × 19.12 + 20.98 × 2

= 169.06 kN/m²

pore water prces = 3.5 × 9.81

= 34.33 kN/m²

$\bar{\sigma}$ = 169.06 – 34.33

= 134.73 kN/m²

45. (a)

$$h_c = \frac{C}{eD_{10}} = \frac{0.3}{0.05 \times 10^{-1} \times 0.85} = 70.6 \text{ cm} =$$

0.706 m

46. (b)

Given,

Average bulk density,

$\gamma_t = 20 \text{ kN/m}^3$

$\gamma_w = 10 \text{kN/m}^3$

Since soil is saturated

hence, $\gamma_t = \gamma_{sat} = 20 \text{kN/m}^3$

$\gamma_{submerged} = \gamma_{sat} - \gamma_w$

= 20 – 10

= 10 kN/m³

Effective stress at a depth 10 m below the river bed

$\bar{\sigma} = \gamma_{sub} \times H$

$= 10 \times 10$

$= 100$ kN/m^2

Note : Effective stress at any depth does not depend upon the change of water level above the soil.

47. (c)

The effective stress at a depth below 2.5 m

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

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

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

48. (d)

$Q = KAi$

$$Q_A = K \cdot \frac{\pi}{4} D^2 \cdot \frac{h}{2L}$$

$$Q_B = K \cdot \frac{\pi}{4} \times 4D^2 \cdot \frac{h}{L}$$

$$\therefore \quad \frac{Q_A}{Q_B} = \frac{K \cdot \frac{\pi}{4} D^2 \cdot \frac{h}{2L}}{K \cdot \frac{\pi}{4} \cdot 4D^2 \cdot \frac{h}{L}} = \frac{1}{8} = 0.125$$

49. (b)

Assuming Darcy's law to be valid

Discharge velocity,

$$v = Ki = 30 \times \frac{(50-25)}{1500} = 0.5 \text{ m/d}$$

Seepage velocity,

$$v_s = \frac{v}{\text{Porosity}}$$

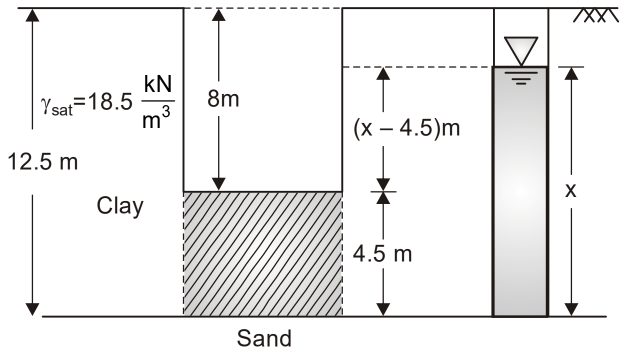
$$= \frac{0.5}{0.25} = 2 \text{ m/d}$$

Time of travel of an inert tracer from one well to another is t,

$$t = \frac{\text{Distance}}{v_s} = \frac{1500}{2} = 750 \text{ days}$$

50. (a)

51. (a)



For quick sand condition

Bouyant weight = seepage force

$4.5\gamma_{sub} = (x - 4.5)\gamma_w$

$4.5(18.5 - 10) = (x - 4.5) \times 10$

$x = 8.325$ m.

Hence the correct option is (a).

Note: **Quick sand condition:** In case of upward seepage flow, if the upward seepage force become equal to the bouyant weight of soil, the effective stress in the soil becomes zero.

52. (b)

- In both the conditions, quick condition and liquefaction of saturated sand, effective stress reduces and soil looses it's shear strength.
- In liquefaction, effective stress reduces due to increase in pore water pressure. So liquefaction is not possible in dry condition.

53. (d)

54. (d)

$$K_{x_{eq}} = \frac{K_1H_1 + K_2H_2 + K_3H_3}{H_1 + H_2 + H_3}$$

$$= \frac{(1 \times 23 + 1.5 \times 5.2 + 2 \times 0.5) \times 10^{-6}}{3}$$

$= 10.6 \times 10^{-6}$ cm/s

$$K_{z_{eq}} = \frac{H_1 + H_2 + H_3}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3}} = \frac{3}{\frac{1}{10^{-6}}\left[\frac{1}{23} + \frac{1.5}{5.2} + \frac{0.5}{2}\right]}$$

$$\frac{K\,H_{eq}}{K_{Z_{eq}}} = \frac{10.6}{5.55} = 2.056$$

55. (d)

$$q = k \cdot i \times A$$

$$= 7\times10^{-4}\times\frac{0.5}{2}\times1\times10^{-2}(cm^3/s/m^2)$$

$$= 1.75 \times 10^{-6}\ m^3 \times 24 \times 60 \times 60\ m^3/d/m^3$$

$$= 0.1512\ m^3/d/m^2$$

56. (a)

The smaller diameter pipes are required for less pervious soils.

57. (b)

Given,

K = 0.0013 mm/second

= 0.11232 m/day

H_L = 9 m

N_f = 5

N_d = 8

We know that,

$$q = KH_L\frac{N_f}{N_d}$$

$$= 0.11232\times9\times\frac{5}{8}$$

$$= 0.6318\frac{m^3}{day}/m$$

Total seepage loss,

Q = q × Length of dam

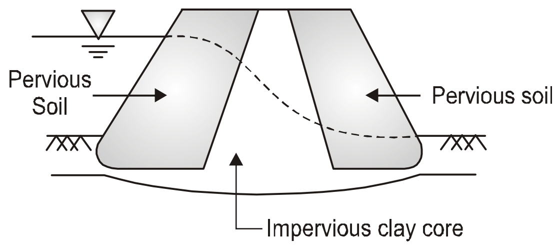
= 0.6318 × 90

= 56.862 m^3/day

Hence, the correct option is (b).

58. (a)

1. Graded filter drains water without losing fines from the soil. It permits water but prevents the soil particles to flow. Thus prevents piping failure.
2. Lime treatment is used to stabilize the black cotton soils.
3. Impervious clay core is able to hold the water and seal the excavation. It is provided in earthen dams to prevent seepage of water through body of dam.



4. Curtain grouting is done to protect a dam from seepage by strengthening its foundation. Ground curtains are used under dams where foundation would otherwise pass too much seepage.

Hence, correct match is-

A – 4, B – 3, C – 1, D – 2

59. (a)

60. (d)

For the filter to provide free drainage, its coefficient of permeability should be 25 times or more than the coefficient of permeability of the soil to be protected. As the coefficient of permeability varies as the square of the particle size, the ratio of the particle diameter should be attest 5.

61. (b)

62. (d)

$$OCR = \frac{\bar{\sigma}_C}{\bar{\sigma}}$$

$\bar{\sigma}_C$ = maximum overburden pressure experienced by soil in its history

$\bar{\sigma}$ = present pressure

Before construction

$\bar{\sigma}_C$ = 150 kN/m^2, $\bar{\sigma}$ = 100 kN/m^2

Since $\bar{\sigma}_C > \bar{\sigma}$ it is OC soil

$$OCR = \frac{150}{100} = 1.5$$

$\bar{\sigma}_C$ = 150 kN/m^2, $\bar{\sigma}$ = 200 kN/m^2

Since $\bar{\sigma}_C < \bar{\sigma}$ it is NC soil

For NC soil

OCR = 1

63. (a)

64. (a)

- A soil is said to be normally consolidated when the existing effective stress $\bar{\sigma}$ is the maximum that it has ever experienced in its stress history.
- Over consolidation ratio is the ratio of preconsolidation stress to the present vertical effective stress, hence for normally consolidated soil it's value is unity.
- Laboratory samples are slightly disturbed when they are taken out. The disturbance causes slight decrease in the slope of compression curve obtained in the laboratory. Higher the disturbance, more is decrease in the slope of compression curve.
- During consolidation, water is gradually removed from the pores. The time taken in complete removal of water is called hydrodynamic lag. It depends on permeability of soil and drainage path.

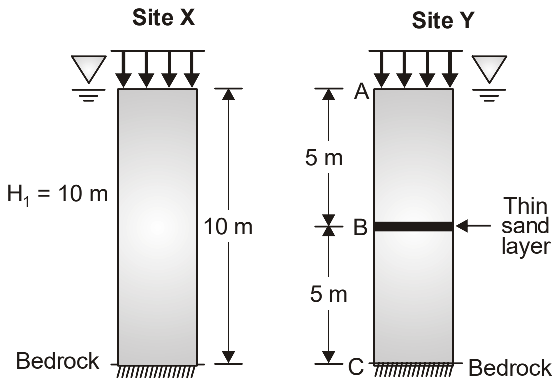
Now it has been observed that even though the consolidation is 100% complete due to the phenomenon of hydrodynamic lag (primary consolidation), the settlement of soil continues due to readjustment of soil particles at a constant effective stress. This phenomenon is known as plastic lag or secondary consolidation.

According to Terzaghi's 1D consolidation theory, there is a fixed relationship between void ratio and effective stress, as secondary consolidation takes place at constant effective stress, so it is ignored means plastic lag is ignored.

65. (a)

The recompression index is appreciably smaller than the compression index. It is usually in the range of $\frac{1}{10}$ to $\frac{1}{5}$ of the compression index.

66. (c)



Time factor, $T_v = \frac{C_v t}{H^2}$

H = Maximum distance that water has to travel to reach a drainage face

Degree of consolidation and soil conditions are same at both the sites so time factor T_v and coefficient of consolidation C_v are same,

$\therefore \quad t \propto H^2$

At site X, primary consolidation completes in 36 months $\Rightarrow \quad t_X = 36$ month

At site Y, in part AB double drainage condition occurs while in part BC single drainage condition occurs hence part BC will take more time for completion of primary consolidation.

$\therefore \quad H_Y = 5$ m

$$\frac{t_X}{t_Y} = \left(\frac{H_X}{H_Y}\right)^2$$

$$\Rightarrow \frac{36}{t_Y} = \left(\frac{10}{5}\right)^2$$

$\Rightarrow t_Y = 9$ months

67. (c)

We know that

$$(T_V)_{90} = \frac{C_{v_1} t_1}{H_1^2}$$

$$(T_V)_{90} = \frac{C_{v_1} \times 15}{H_1^2} \qquad \text{... (i)}$$

Again, $\boxed{K = C_v m_v \gamma_w}$

$$\frac{K_1}{K_2} = \frac{C_{v_1}}{C_{v_2}} \times \frac{m_{v_1}}{m_{v_2}} \times \frac{\gamma_{w_1}}{\gamma_{w_2}}$$

$$\frac{K}{3K} = \frac{C_{v_1}}{C_{v_2}} \times \frac{m_v}{4m_v}$$

$$C_{v_2} = \frac{3}{4} C_{v_1}$$

Time required to achieve 90% consolidation

$$(T_V)_{90} = \frac{C_{v_2} t_2}{(H_2)^2}$$

From (i)

$$\frac{C_{v_1} \times 15}{H_1^2} = \frac{3}{4} C_{v_1} \times \frac{t_2}{(2H_1)^2}$$

t_2 = 80 years

68. (c)

Final settlement in layer 1 = $\frac{x}{2}$

Total settlement in two layer = x

Out of 10 cm settlement, settlement in 5m thick = 5 cm

$\Rightarrow \frac{5}{x/2} = \frac{10}{x}$ = degree of settlement at the sand layer

$$\Rightarrow \frac{C_v \times 1}{\left(\frac{10}{2}\right)^2} = T_v = \frac{C_v \times t}{\left(\frac{5}{2}\right)^2}$$

$\Rightarrow t = \frac{1}{4}$year

69. (b)

- For 100% consolidation

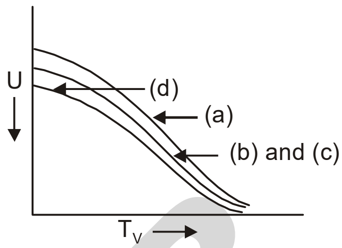
Time factor, $T_v \to \infty$

hence time taken 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.

70. (a)



71. (a)

72. (b)

Sheep foot rollers are considered most suitable for compacting clayey soils. They consists of a hollow drum with a large number of small projections (known as feet) on its surface. These projections penetrate the soil during rolling operations and cause compaction. The drum can be filled with water or sand to increase the mass. The rollers compact the soil by a combination of tamping and kneading action. Pressure applied on the soil or efficiency depends upon weight of roller and number of feet in contact with the ground at a time.

73. (a)

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.

74. (c)

75. (a)