MULTIPLE CHOICE OBJECTIVE TYPE QUESTIONS

1. Soils are basically
(*a*) Organic materials
(*b*) Inorganic materials
(*c*) Mineral materials
(*d*) Organic and Inorganic materials

2. Cohesionless soils are formed by
(*a*) Physical disintegration of rocks
(*b*) Chemical weathering of rocks
(*c*) Consistent blowing of wind in the same direction
(*d*) Deposition in a delta region

3. 'Talus' is the soil transported by
(*a*) Wind (*b*) Water
(*c*) Glacier (*d*) Gravitational force

4. 'Losses' is silty clay formed by the action of
(*a*) Water (*b*) Glacier
(*c*) Wind (*d*) Gravitational force

5. 'Drift' is the material picked up, mixed disintegrated, transported and redeposited by
(*a*) Wind (*b*) Gravitational force
(*c*) Glacier water (*d*) All of the above

6. Non-cohesive soil is
(*a*) Sand (*b*) Silt
(*c*) Clay (*d*) Clay and silt

7. The soil transported by flowing water is called
(*a*) Aeolian soil (*b*) Marine soil
(*c*) Alluvial soil (*d*) Lacustrine soil

8. The soil transported by wind is called
(*a*) Aeolian soil (*b*) Marine soil
(*c*) Alluvial soil (*d*) Lacustrine soil

9. Lacustrine soils are those which are
(*a*) Deposited in sea water
(*b*) Deposited at the bottom of lakes
(*c*) Transported by wind
(*d*) Deposited due to moving of ice sheets

10. Clay can be classified as
(*a*) Highly cohesive soil
(*b*) Limited cohesive soil
(*c*) Cohesionless soil
(*d*) None of the above

11. Which of the following is cohesive soil?
(*a*) Kankar (*b*) Black cotton soil
(*c*) Loose coarse sand (*d*) Sand with clay

12. Which of the following soils has the finest grains?
(*a*) Sands (*b*) Silts
(*c*) Fine sands (*d*) Clays

13. Silts have the following property:
(*a*) Plasticity (*b*) Limited plasticity
(*c*) No plasticity (*d*) Elasticity

14. The minimum size of grains of silt soil is about
(*a*) 0.01 mm (*b*) 0.3 mm
(*c*) 0.06 mm (*d*) 0.002 mm

15. The maximum size of grains of silt soil is about
(*a*) 0.06 mm (*b*) 0.1 mm
(*c*) 0.5 mm (*d*) 2 mm

16. Which of the following will have highest percentage of land?
(*a*) Sandy clay loam (*b*) Silty clay loam
(*c*) Clay (*d*) Silty clay

17. The maximum size of clay particle is
(*a*) 0.1 mm (*b*) 0.03 mm
(*c*) 0.002 mm (*d*) 0.0002 mm

18. The ratio between the total volume of voids and the total volume of solids is called
(*a*) Porosity (*b*) Void fraction
(*c*) Void ratio (*d*) Solid ratio

19. As per I.S.S. the specific gravity of soil is determined at
(*a*) 10°C (*b*) 17°C
(*c*) 27°C (*d*) 47°C

20. The ratio of volume of voids to the total volume of the given soil mass, is known as
(*a*) Porosity (*b*) Void ratio
(*c*) Specific gravity (*d*) Watercontent

21. The representation of the three constituents of a soil, *i.e.*, solid, water and air, by the three spaces of a diagram is called
(*a*) bi-phase diagram (*b*) One phase diagram
(*c*) Three phase diagram (*d*) Two phase diagram

22. The ratio of weight of water to the weight of solids in a given mass of soil, is known as
(*a*) Void ratio (*b*) Porosity
(*c*) Specific gravity (*d*) Water content

23. The degree of saturation in soils can be defined as the ratio of
(*a*) Water by weight to the dry soil weight
(*b*) Volume of water to the gross volume of soil.
(*c*) Volume of water to volume of voids in soil
(*d*) Weight of the water to weight of soil

24. The relationship between void ratio e and porosity ratio *n* is
(*a*) $n = \dfrac{e}{1-e}$ (*b*) $e = n\,(1 + e)$
(*c*) $e = \dfrac{1+n}{1-e}$ (*d*) $n = \dfrac{1+e}{1-e}$

25. When the pores of-a-soil are full of water then the soil will be called
(*a*) Moist soil (*b*) Fully saturated soil
(*c*) Plastic soil (*d*) Hydrated soil

26. Dry density of a soil is
(*a*) Always greater than the saturated density
(*b*) Ratio of the weight of soil solids to the volume of solids.
(*c*) Ratio of the weight of soil solids to the total volume
(*d*) Determined at 100°C

27. The ratio of volume of air voids to the volume of voids, is called
(*a*) Void density (*b*) Air content
(*c*) Porosity (*d*) Permeability

28. In which sample the percentage of the porosity will be least?
(*a*) Loose sand
(*b*) Compact sand
(*c*) Soft clay
(*d*) Soft clay of organic nature

29. The percentage of the void ratio will be highest in
(*a*) Organic clays
(*b*) Soft clay slightly organic in nature
(*c*) Stiff clay
(*d*) Loose sand

30. The dry density of which sample is expected to be highest?
(*a*) Organic clay (*b*) Bentonite
(*c*) Stiff clay (*d*) Dense sand

31. The percentage of the void ratio will be least in case of
(*a*) Uniform dense sand (*b*) Organic clays
(*c*) Soft clay slightly organic in nature
(*d*) Bentonite

32. A soil sample is having a specific gravity of 2.60 and a void ratio of 0.78. The water content in percentage required to fully saturated the soil at that void ratio would be
(*a*) 10 (*b*) 30 (*c*) 50 (*d*) 70

33. A dry soil has a mass specific gravity of 1.35. If the specific gravity of solids is 2.7 then the void ratio will be
(*a*) 0.5 (*b*) 1.0
(*c*) 1.5 (*d*) 2.0

34. The dry density of which sample may be expected to be least?
(*a*) Stiff clay (*b*) Bentonite
(*c*) Dense sand (*d*) Loose sand

35. If the pores of a soil are completely full of air only, the soil is said to be:
(*a*) Wet soil (*b*) Dry soil
(*c*) Fully saturated soil (*d*) Partially saturated soil

36. The unit weight of a soil at zero air voids depends on
(*a*) Specific gravity (*b*) Water content
(*c*) Unit weight of water (*d*) All of the above

37. The weight of water per unit of volume of water is called
(*a*) Moisture content (*b*) Density
(*c*) Unit weight of water (*d*) Degree of saturation

38. The ratio of bulk density of a sample of soil mass to the density of water at 4°C is called
(*a*) Bulk specific gravity (*b*) Wet density
(*c*) Saturation density (*d*) Moisture content

39. The specific gravity of sandy soil is
(*a*) 1.5 (*b*) 2.0
(*c*) 2.2 (*d*) 2.6

40. The natural water content of the soil sample was found to be 40%, specific gravity is 2.7 and void ratio 1.2; then the degree of saturation of the soil will be
(*a*) 100 % (*b*) 69 %
(*c*) 87 % (*d*) 90 %

41. Two different granular soils are placed in a permeameter tube and flow is allowed to take place under a constant total head. The total head and pressure head at point A in centimeter, are respectively

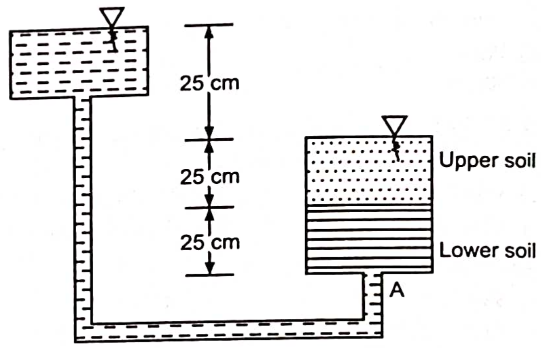


Fig. 8.24.

(*a*) 75, 75 (*b*) 25, 75
(*c*) 25, 25 (*d*) 75, 25

42. For the purpose of designing a well point system for lowering ground water table in a sandy silt deposit, the coefficient of permeability of the soil is to be determined which one of the following methods would be most suitable?
(*a*) Constant head permeameter test
(*b*) Variable head permeameter test
(*c*) Pumping out test in field
(*d*) Pumping in test in field

43. If a flow net of a cofferdam foundation had 6 numbers of flow channels and 12 numbers of equipotential drops, with the head of water being 6 m through a foundation having $k = 4 \times 10^{-5}$ m/min, then seepage loss per meter length of dam will be
(*a*) 24×10^{-5} m³/min (*b*) 18×10^{-5} m³/min
(*c*) 12×10^{-5} m³/min (*d*) 9×10^{-5} m³/min

44. Two sheet piles are drivers 12 m apart to 6 m depth in water loged sand bank of a river. The sand shows a specific gravity of 2.7 and void ratio 0.7. The factor of safety against quick condition when inside excavation and dewatering reaches 4 m depth (from ground level) will be
(*a*) 0.5 (*b*) 1.0 (*c*) 2.0 (*d*) 4.0

45. A uniform sand stratum 2.5 m thick has a specific gravity of 2.62 and a natural void ratio of 0.62. The hydraulic head required to cause quick sand condition in the sand stratum is

(a) 0.5 m (b) 1.5 m
(c) 2.5 m (d) 3.5 m

In a wet soil mass, air occupies one sixth of its volume and water occupies one third of its volume. The void ratio of the soil is

(a) 0.25 (b) 0.5
(c) 1.00 (d) 1.50

7. If during a permeability test on a soil sample with a falling head permeameter equal time intervals are noted for drop of head from h_1 to h_2 and again from h_2 to h_3 then which one of the following relation would hold good?

(a) $h_3^2 = h_1 h_2$ (b) $h_2^2 = h_1 h_3$
(c) $h_1^2 = h_2 h_3$ (d) $(h_1 - h_2) = (h_2 - h_3)$

8. The bulk density of soil can be defined as

(a) Ratio of the weight of the solids to the volume of solids
(b) Ratio of unit weight of soil to that of water
(c) Unit weight of soil
(d) Unit weight of soil under saturated conditions.

49. The submerged density of a soil is the ratio of

(a) Weight of soil in water to its volume
(b) Weight of soil minus weight of equivalent water to volume of soil.
(c) Weight of soil less weight of water is to the volume of solids plus voids in the soil.
(d) Unit weight of saturated soil.

50. A functional equation for specific gravity (G), water content (ω), void ratio (e) and degree of saturation (S_r) is

(a) $\omega = \dfrac{S_r, (G)}{e}$ (b) $e = \dfrac{S_r, \omega}{G}$
(c) $S_r = \dfrac{\omega G}{e}$ (d) $G = \dfrac{S_r\ \omega}{e}$

51. The porosity of a soil sample having its void ratio equal to unity would be

(a) 33.34% (b) 50.0%
(c) 66.66% (d) 75.0%

52. A soil sample has a porosity of 40%. The specific gravity of solids in 2.70. The void ratio for the sample is

(a) 0.250 (b) 0.667
(c) 0.750 (d) 0.800

53. The size of aperture of I.S. sieve 300 micron is

(a) 0.0300 mm (b) 0.300 mm
(c) 0.297 mm (d) 0.305 mm

54. Sieve analysis of a soil sample is done if the particles do not pass through square openings of

(a) 0.075 mm (b) 0.150 mm
(c) 4.75 mm (d) 0.300 mm

55. Wet analysis of fine particles is done if nearly all soil particles pass through square sieve openings of

(a) 0.075 mm (b) 0.045 mm
(c) 0.212 mm (d) 0.300 mm

56. Coarse sieve analysis is done for that material which retains on the I.S. sieve of opening size

(a) 75 mm (b) 4.75 mm
(c) 2 mm (d) 150 micron

57. The effective size of a soil is

(a) D_{10} (b) D_{20} (c) D_{30} (d) D_{40}

58. The uniformity co-efficient of soil is defined as the ratio of

(a) $\dfrac{D_{10}}{D_{20}}$ (b) $\dfrac{D_{20}}{D_{30}}$
(c) $\dfrac{D_{60}}{D_{10}}$ (d) $\dfrac{D_{40}}{D_{50}}$

59. A soil having uniformity co-efficient more than 10, is called

(a) Uniform soil (b) Poor soil
(c) Well graded soil (d) Coarse soil

60. Group symbols assigned to silty sand and clay sand are respectively

(a) SS and CS (b) SM and CS
(c) SM and SC (d) MS and CS

(GATE 2008)

61. A soil having uniformity co-efficient more than 10 is called

(a) Uniform (b) Fine
(c) Coarse (d) Well graded soil

62. The soil having uniformity co-efficient less than 4 is called

(a) Uniform (b) Fine
(c) Coarse (d) Well graded soil

63. The particle diameter corresponding to 20 percent finer on the grain size curve is called

(a) D_{120} (b) D_{80}
(c) D_{20} (d) D_{200}

64. As per the Indian standard soil classification system, a sample of silty clay with liquid limit of 40%, and plasticity index of 28% is classified as

(a) CH (b) CI
(c) CL (d) CL-ML **(GATE 2012)**

65. The most accurate method of determining the water content in a sample of soil is

(a) Oven drying method (b) Sand bath method
(c) Alcohol method (d) Calcium carbide method

66. In the oven drying method of determination of water content of a soil sample, the temperature range of soil must be up to

(a) 40°C (b) 110°C (c) 60°C (d) 80°C

67. A flownet is drawn to obtain

(a) Seepage coefficient of permeability and uplift pressure
(b) Coefficient of permeability, uplift pressure and exit gradient.
(c) Exit gradient, uplift pressure, seepage quality
(d) Exit gradient, seepage and coefficient of permeability.

68. A flownet is drawn for a weir. The total head loss is 6 m number of potential drops is 10 and the length of flow path of the last square is 1 m. The exit gradient is
(*a*) 0.6 (*b*) 0.7
(*c*) 1.0 (*d*) 1.6

69. A flow net for seepage under a sheet pile wall has $n_f = 4$, $n_d = 8$ and the permeabilities of the soil in the horizontal and vertical directions are $K_h = 8 \times 10^{-5}$ m/sec and $K_v = 2 \times 10^{-5}$ m/sec. If the head loss through the soil is 2 m, the quantity of seepage per meter length of the wall will be
(*a*) 2×10^{-5} m^3/sec (*b*) 4×10^{-5} m^3/sec
(*c*) 8×10^{-5} m^3/sec (*d*) 16×10^{-5} m^3/sec
(IES 2011)

70. An isobar is a line which connects all points below the ground surface at which
(*a*) The local ground elevation is same
(*b*) The settlement is same
(*c*) The vertical stress is the same
(*d*) The ground elevation is varying **(IES 2012)**

71. Standard Newmark's influence chart is shown in given figure if loaded equally the areas marked 1 and 2 will yeild pressure at centre such that

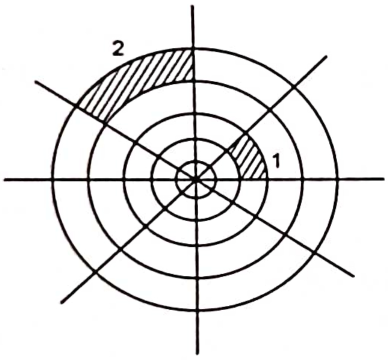


Fig. 8.25.

(*a*) 1 yeilds more than 2
(*b*) 2 yields more than 1
(*c*) 1 and 2 yeild the same
(*d*) 1 yeilds exactly half of that of 2

72. Consider the following characteristics of soil layer.
1. Poisson ratio 2. Young's modulus
3. Finite nature of soil 4. Effect of water table
5. Rigidity of footing
Westerguard's analysis for pressure distribution in soil utilises.
(*a*) 1, 3, 4 and 5 (*b*) 2, 3, 4 and 5
(*c*) 3, 4 and 5 (*d*) 1 and 5

73. The method of determination of water content of soils suitable for coarse grained soils only, is
(*a*) Sand bath method (*b*) alcohol method
(*c*) Oven drying method (*d*) Pycnometer method

74. The shrinkage ratio of a soil is
(*a*) Equal to the mass specific gravity of the soil in its dry state
(*b*) Equal to the saturated density
(*c*) Equal to the specific density
(*d*) The same as dry density.

75. The maximum water content at which a reduction in water content will not cause a decrease in volume of a soil mass is known as
(*a*) Plastic limit (*b*) Shrinkage limit
(*c*) Liquid limit (*d*) Consistency limit

76. The minimum moisture content at which the soil remains in liquid state but has just started exhibiting a small shear strength against flowing is called
(*a*) Elastic limit (*b*) Plastic limit
(*c*) Consistency limit (*d*) Liquid limit

77. The minimum moisture content at which the soil just begins to crumble when rolled into 3 mm threads is known as
(*a*) Elastic limit (*b*) Plastic limit
(*c*) Consistency limit (*d*) Liquid limit

78. The plastic limit exists in case of
(*a*) Clays (*b*) Silty soils
(*c*) Sandy soils (*d*) Gravel soils

79. The relation between dry density (γ_d) bulk density (γ) and water content (w) is
(*a*) $\gamma = \dfrac{\gamma_d}{1+w}$ (*b*) $\gamma_d = \dfrac{\gamma}{1+w}$
(*c*) $w = \dfrac{\gamma}{1+\gamma_d}$ (*d*) $w = \dfrac{\gamma}{1-\gamma_d}$

80. The saturated density of soil can be expressed as
(*a*) Weight of soil in water to its volume
(*b*) Weight of soil minus weight of equivalent water to volume of soil
(*c*) Weight of soil less weight of water in voids to the volume of soilds plus voids in the soil
(*d*) Unit weight of saturated soil

81. The percentage of the porosity of loose sand is in the range of
(*a*) 5 to 15% (*b*) 15 to 30%
(*c*) 30 to 40% (*d*) 40 to 50%

82. The percentage of the porosity of compacted sand is about
(*a*) 5 to 15% (*b*) 15 to 30%
(*c*) 30 to 40% (*d*) 40 to 50%

83. The liquid limit is defined as
(*a*) Amount of water which makes the soil go into the solids state from the liquid state
(*b*) Amount of water content which makes the soil go into the liquid state
(*c*) Limit of water that makes the soil flow
(*d*) Minimum water content at which soil can be rolled into 3 mm dia threads

The ratio in between liquid limit minus the natural water content and plasticity index of a soil is known as
(*a*) Relative consistency (*b*) Liquidity ratio
(*c*) Relative density (*d*) Shrinkage ratio

The shear strength of a soil in the plastic limit state is
(*a*) Zero
(*b*) Reasonable
(*c*) Small
(*d*) Close to saturated soil strength

The shear strength of a soil in the liquid limit state is
(*a*) Zero
(*b*) Very small
(*c*) Reasonable
(*d*) Close to saturated soil strength

. The liquid and plastic limits exist in
(*a*) Sandy soils (*b*) Silty soils
(*c*) Gravel soils (*d*) Clay soils

. The plasticity index is
(*a*) Liquid limit-shrinkage limit
(*b*) Liquid limit-plastic limit
(*c*) Plastic limit-liquid limit
(*d*) Plastic limit-shrinkage limit

). When the plastic limit of a soil is greater than the liquid limit, than the plasticity index is
(*a*) Negative (*b*) Zero
(*c*) One (*d*) More than one

). When the plasticity index of a soil is zero, the soil is
(*a*) Clay (*b*) Silt
(*c*) Sand (*d*) Silty sand

1. The liquidity index is defined as ratio expressed as a percentage of

(*a*) $\dfrac{\text{Plastic limit} - \text{natural water content}}{\text{Plasticity index}}$

(*b*) $\dfrac{\text{Natural water content} - \text{Plastic limit}}{\text{Plasticity index}}$

(*c*) $\dfrac{\text{Natural water content} + \text{Plastic limit}}{\text{Plasticity index}}$

(*d*) $\dfrac{\text{Liquid limit} - \text{Natural water content}}{\text{Plasticity index}}$

2. The shrinkage index is equal to
(*a*) Liquid limit – Plastic limit
(*b*) Liquid limit – Shrinkage limit
(*c*) Plastic limit – Liquid limit
(*d*) Plastic limit – Shrinkage limit

93. The plasticity index of loose sand is about
(*a*) 20 – 40 (*b*) 40 – 70
(*c*) 70 – 90 (*d*) None of the above

94. The ratio in between the difference between void ratio of a cohesionless soil in the loosest state and any given void ratio, to the difference between its void ratios in the loosest and in the densest states is called
(*a*) Relatively consistency (*b*) Relative density
(*c*) Relative porosity (*d*) Optimum void ratio

95. When the plasticity index of a sample of soils is 6 or nearly 6, then the sample must be of
(*a*) Sand (*b*) Silt
(*c*) Silty clay (*d*) Clay

96. Consistency index of soil is given by

(*a*) $\dfrac{\text{Natural water content} - \text{Liquid limit}}{\text{Plasticity index}}$

(*b*) $\dfrac{\text{Natural water content} - \text{Liquid limit}}{\text{Liquidity index}}$

(*c*) $\dfrac{\text{Liquid limit} - \text{Natural water content}}{\text{Plasticity index}}$

(*d*) None of the above

97. The flow index in soils indicates
(*a*) Shear strength variation with water content
(*b*) Variation of plastic limit
(*c*) Ratio of liquid limit to plastic limit
(*d*) None of the above

98. The plasticity index of highly plastic soil is about
(*a*) 5–10 (*b*) 10–20
(*c*) 20–40 (*d*) More than 40

99. The soil in India are classified according to
(*a*) MIT classification
(*b*) Unified soil classification system
(*c*) Particle size classification
(*d*) International classification system

100. The graph which is prepared on the basis of observations and results obtained from liquid limit tests taking water content as ordinate on arithmetic scale and the number of blows as abscissa on logarithmic scale, then the graph will be named as
(*a*) Plasticity curve (*b*) Saturation curve
(*c*) Liquidity curve (*d*) Flow curve

101. The slope of the flow curve obtained from a liquid limit test expressed as the difference in water contents at 10 blows and 100 blows is called
(*a*) Flow index (*b*) Porosity index
(*c*) Liquidity (*d*) Percolation index

102. The results of four compaction tests (curves *A*, *B*, *C* and *D*) on different soils are shown in graph tests.
1. Silty sand, modified test
2. Silty sand, standard test
3. Fat clay, modified test
4. Fat clay, standard test

Curve *A*, *B* *C* and *D* respond respectively to tests

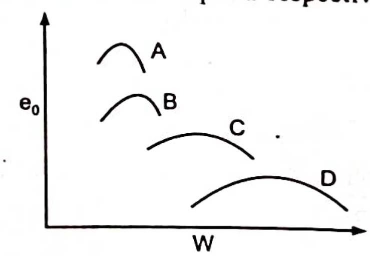


Fig. 8.26.

(a) 1, 3, 2 and 4 (b) 1, 2, 3 and 4
(c) 2, 1, 3 and 4 (d) 2, 1, 4 and 3

103. A triaxial test was conducted on a granular soil. At failure $\frac{\sigma_1'}{\sigma_3'} = 4$. The effective minor principal stress at failure was 100 kPa. The values of approximate ϕ and the principal stress difference at failure are, respectively
(a) 45° and 570 kPa (b) 40° and 400 kPa
(c) 37° and 300 kPa (d) 30° and 200 kPa

104. In a saturated soil deposit having a density 22 kN/m^3 the effective normal stress on a horizontal plane at 5m depth will be
(a) 22 kN/m^2 (b) 50 kN/m^2
(c) 60 kN/m^2 (d) 110 kN/m^2

105. The intensity of vertical stress σ_z at a depth due to a point load acting on the surface of a semi infinite elastic soil mass is
(a) Directly proportional to depth
(b) Inversely proportional to depth
(c) Directly proportional to the square of depth
(d) Inversely proportional to the square of depth

106. In Newmark's influence chart for stress distribution there one 10 concentric circles and ten radial lines. The influence factor of the chart is
(a) 0.1 (b) 0.01
(c) 0.001 (d) 0.0001

107. A concentrated load of 50t acts vertically at a point on the soil surface. If Boussinesq's equation in applied for computation of stress then the ratio of vertical stresses at depths of 3 m and 5 m respectively below the point of application of load will be
(a) 0.36 (b) 0.60
(c) 1.66 (d) 2.77

108. The ratio of the energies imparted to soil sample in modified proctor's compaction test and the standard proctor's compaction test is about
(a) 10.0 (b) 4.5
(c) 2.2 (d) 1.8

109. Westergaard's analysis for stress distribution beneath loaded area is applicable to
(a) Sandy soil (b) Clayey soil
(c) Stratified soil (d) Silty soil

110. Using Mohar's diagram, the relation between major principal stress σ_1 and minor principal stress σ_3 and shear parameters c and ϕ is given by $\sigma_1 = \sigma_3 N_\phi + 2c \sqrt{N\phi}$ where N_ϕ is equal to

(a) $\frac{\sin\phi}{(1+\sin\phi)}$ (b) $\frac{\sin\phi}{(1-\sin\phi)}$

(c) $\frac{(1-\sin\phi)}{(1+\sin\phi)}$ (d) $\frac{(1+\sin\phi)}{(1-\sin\phi)}$

111. The results of a unsolidated drained triaxial shear test on a normally consolidated clay are shown in the figure. The angle of internal friction is

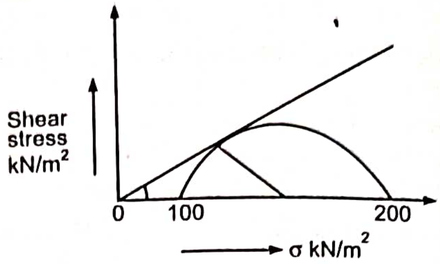


Fig. 8.27.

(a) $\sin^{-1}\left(\frac{1}{3}\right)$ (b) $\sin^{-1}\left(\frac{1}{2}\right)$

(c) $\sin^{-1}\left(\frac{2}{3}\right)$ (d) $\sin^{-1}\left(\frac{1}{\sqrt{2}}\right)$

112. A undrained triaxial compression test is carried out on a saturated clay sample under a cell pressure of 100 kN/m^2. The sample failed at a deviator stress of 200 kN/m^2. The cohesion of the given sample of clay is
(a) 50 kN/m^2 (b) 100 kN/m^2
(c) 200 kN/m^2 (d) 300 kN/m^2

113. The property of a soil which permits flow of water through its interconnected voids is called
(a) Seepability (b) Porosity
(c) Permeability (d) Void ratio

114. The type of flow in which all liquid particles move in parallel paths without crossing the paths of other particles is called
(a) Laminar flow (b) Turbulent flow
(c) Eddy flow (d) Viscous flow

115. Permeability of soil is affected by
(a) Grain size
(b) Void ratio of the soil
(c) Structural arrangement of the soil particles
(d) All of the above

116. The law that states that laminar flow in a saturated soil, the velocity is directly proportional to the hydraulic gradient is called
(a) Reynold's law (b) Bligh's law
(c) Darcy's law (d) Lacey's law

117. Darcy's law can also be represented as

(a) $Q = \frac{1}{K} iA$

(b) $Q = \frac{K}{i} A$

(c) $Q = K\ i\ A$ (d) $Q = K\ A$

118. Permeability of soil varies
(a) Inversely as square of grain size
(b) Inversely as grain size
(c) as grain size
(d) Square of grain size

Darcy's law is not applicable to seepage through soils if

(a) The soil is homogenous
(b) The soil is isotropic
(c) The flow conditions are laminar in the soil
(d) The flow conditions are turbulent in the soil

The dimensions of coefficient of permeability are
(a) cm (b) g/cm^2
(c) cm/sec (d) g/cm

The coefficient of permeability is proportional to void ratio (e) as
(a) 1/e (b) e
(c) e^2 (d) $\frac{e^3}{1+e}$

If the hydraulic gradient is unity, then the ratio of flow across unit area of soil is called
(a) Co-efficient of seepage
(b) Co-efficient of permeability
(c) Co-efficient of viscosity
(d) Co-efficient of discharge

When a soil has co-efficient of permeability as 10 cm/sec then the soil should be
(a) Very fine like clay (b) Silt
(c) Coarse like sand (d) Gravel

When a soil has co-efficient of permeability as 1 mm/sec, then the soil should be
(a) Clay (b) Silt
(c) Sand (d) Gravel

Compaction by vibratory roller is the best method of compaction in case of
(a) Moist silty sand (b) Well graded dry sand
(c) Clay of medium compressibility
(d) Silt of high compressibility **(GATE 2008)**

When a soil has co-efficient of permeability as 0.00001 cm/sec, then the soil should be
(a) Clay (b) Silt
(c) Sand (d) Gravel

Soil with co-efficient of permeability 10^{-5} cm/sec could be classified as
(a) Pervious (b) Semi pervious
(c) Impervious (d) All the above are incorrect

According to I.S.I. classification the soil whose co-efficient of permeability is more than 10^{-2} cm/sec, the soil will be classified as
(a) Pervious
(b) Semi pervious
(c) Impervious
(d) All the above are incorrect

According to ISI classification the soil whose co-efficient of permeability is more than 10^{-4} cm/sec, the soil will be classified as
(a) Previous (b) Semi pervious
(c) Impervious (d) None of the above

130. The line of demarcation between pervious and impervious soils is given by a permeability of
(a) 0.1 μ/sec (b) 0.5 μ/sec
(c) 1 μ/sec (d) 5 μ/sec

131. The soil which is most suitable for constructing dam core or a impervious blanket should have co-efficient of permeability less than
(a) 10^{-5} cm/sec (b) 10^{-4} cm/sec
(c) 10^{-3} cm/sec (d) 10^{2} cm/sec

132. Coefficient of permeability of an underground stratum is 0.001 m/s. Discharge obtained from a well of area 20 m^2 dug into this stratum (with drawdown of 2 m)
(a) 2400 lpm (b) 2000 lpm
(c) 1200 lpm (d) 1000 lpm

133. The curve showing the relationship between dry unit weight and the water content of a soil for a given compactive effort, is called
(a) Compression curve (b) Moisture density curve
(c) Settlement curve (d) Porosity curve

134. The densification of a soil by machines is called
(a) Consolidation (b) Compression
(c) Compaction (d) Soil stabilisation

135. The ratio expressed as a percentage of dry unit weight of a soil to maximum unit weight (density) obtained in a laboratory compaction test is called.
(a) Percent compaction (b) Percent porosity
(c) Percent void ratio (d) Percent consolidation

136. The slope of the linear portion of the pressure void ratio curve on a semi log plot is called
(a) Consolidation index (b) Compression index
(c) Tension index (d) None of the above

137. The gradual reduction in volume of a soil mass resulting from an increase in and continued application of compressive stress and is due to expulsion of water from the pores is called
(a) Compaction (b) Consolidation
(c) Settlement (d) Depression

138. A comparatively sudden reduction in volume of a soil mass under an applied load is called
(a) Primary compression (b) Initial consolidation
(c) Primary time effect (d) Secondary compression

139. The ratio of amount of consolidation at a given distance from a drainage surface and at a given time to the total amount of consolidation obtainable at that point under a given stress increment is called.
(a) Compaction ratio (b) Degree of compaction
(c) Consolidation ratio (d) All the above are correct

140. The reduction in volume of a soil mass caused by the application of a sustained load to the mass and due to the adjustment of the internal structure of the soil mass is called
(a) Initial consolidation (b) Primary consolidation
(c) Primary compression (d) Secondary consolidation

141. The e-log p curve shown in the figure is representative of

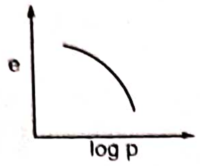


Fig. 8.28.

(a) Normally consolidated clay
(b) Over consolidated clay
(c) Under consolidated clay
(d) Normally consolidated clayey sand **(GATE 2010)**

142. Which of the following is not an assumption in Terzaghics theory of one dimensional consolidation?
(a) Time lag in consolidation is due entirely to permeability.
(b) Deformation of the soil, is due entirely to change in volume.
(c) Water is compressible while soil particles are incompressible.
(d) Darcy's law is perfectly valid.

143. A clay sample has a void ratio of 0.50 in dry state and specific gravity of solids = 2.70. Its shrinkage limit will be
(a) 12 % (b) 13.5 %
(c) 18.5 % (d) 22 %

144. A bed of sand consists of three horizontal layers of equal thickness. The value of Daray's k for the upper and lower layers is 1×10^{-1} cm/sec. The ratio of permeability of the bed in the horizontal direction to that in the vertical direction is,
(a) 10.0 to 1 (b) 2.8 to 1
(c) 2.0 to 1 (d) 1 to 10

145. A soil has liquid limit of 0%, plastic limit of 35% and shrinkage limit of 20% and it has a natural moisture content of 50%. The liquidity index of soil is
(a) 1.5 (b) 1.25
(c) 0.6 (d) 0.4

146. Due to rise in temperature, the viscosity and unit weight of percolating fluid are reduced to 70% and 90% respectively. Other things being constant, the change in cofficient of permeability will be
(a) 20.0 % (b) 28.6 %
(c) 63.0 % (d) 77.8 %

147. The standard plasticity chart to classify fine grained soils is shown in figure. The area marked 'X' represents

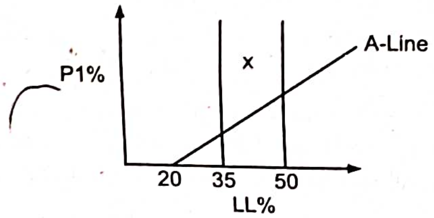


Fig. 8.29.

(a) Silt of low plasticity
(b) Clay of high plasticity
(c) Organic soil of medium plasticity
(d) Clay of intermediate plasticity

148. A deposit of fine sand has a porosity 'n' and specific gravity of soil solids is G. The hydraulic gradient of the deposit to develop boiling condition of sand is given by
(a) $i_c = (G - 1)(1 - n)$ (b) $i_c = (G - 1)(1 + n)$
(c) $i_c = \dfrac{G-1}{1-n}$ (d) $i_c = \dfrac{G-1}{1+n}$

149. An upward hydraulic gradient i of a certain magnitude will initiate the phenomenon of boiling in granular soils. The magnitude of this gradient.
(a) $0 \leq i \leq 0.5$ (b) $0.5 < i < 0.1$
(c) $i = 1.0$ (d) $1 < i \leq 2$

150. The porosity of a certain soil sample was found to be 80% and its specific gravity was 2.7; the critical hydraulic gradient will be estimated as
(a) 0.34 (b) 0.92
(c) 1.0 (d) 1.5 **(IES 2012)**

151. In the schematic flownet shown in the given figure, the hydraulic potential at point A is

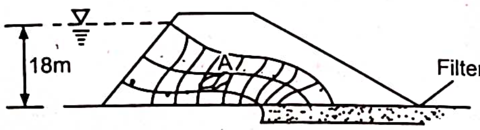


Fig. 8.30.

(a) 5 m of water (b) 12 m of water
(c) 15 m of water (d) 25 m of water

152. Consider the following statements in relation to the given sketch.

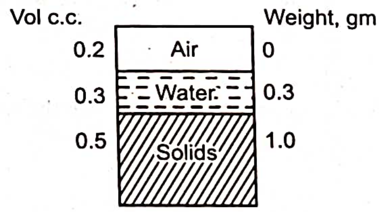


Fig. 8.31.

1. Soil is partially saturated at degree of saturation = 60 %
2. Void ratio = 40 %
3. Water content = 30 %
4. Saturated unit weight = 1.5 gm/cc.

Of these statements
(a) 1, 2 and 3 are correct
(b) 1, 3 and 4 are correct
(c) 2, 3 and 4 are correct
(d) 1, 2 and 4 are correct

153. The effective stress friction angle of a saturated, cohesionless soil is 38°. The ratio of shear stress to normal effective stress on the failure plane is
(a) 0.781 (b) 0.616 (c) 0.488 (d) 0.438
(GATE 2012)

154. Which of the following laboratory triaxial test parameters should one specify to be carried out in connection with the initial stability of footing on saturated clay?

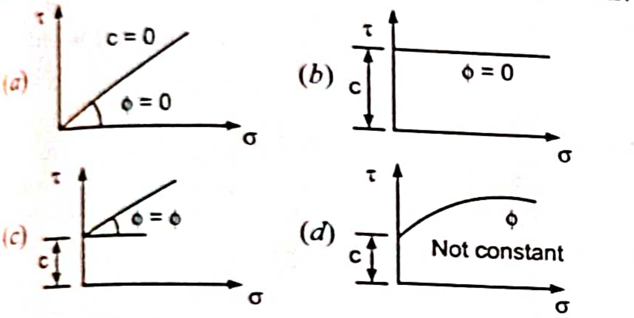
1. C_{cu}, ϕ_{cu} – Consolidated undrained
2. C_u, ϕ_u – Undrained
3. C_d', ϕ_d' – Drained

Select the correct answer using the codes given below:

Codes:

(a) 1 alone (b) 2 alone
(c) 1 and 3 (d) 1, 2 and 3

155. Which one of the following figures gives the failure envelope for a normally consolidated saturated clay sample tested in triaxial test under drained condition?



156. Active earth pressure per meter length on the retaining wall with a smooth vertical back as shown in figure will be

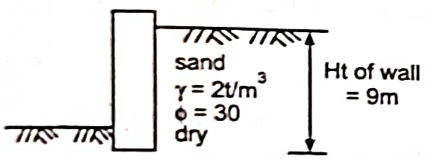


Fig. 8.32.

(a) 81 t (b) 27 t
(c) 2 t (d) 1 t

157. The angle between the horizontal and the maximum slope that a soil assumes through natural processes is called

(a) Angle of internal friction
(b) Angle of repose
(c) Cohesiveness
(d) Hydraulic gradient

158. The angle of internal friction

(a) Varies with the density of sand
(b) Varies with the normal direct pressure
(c) Depends upon the particle shape and roughness
(d) All of the above

159. The angle of internal friction of round grained loose sand is about

(a) 5° to 25° (b) 25° to 30°
(c) 30° to 35° (d) 32° to 37°

160. The angle of internal fricrtion of round grained dense sand is about

(a) 5° to 25° (b) 25° to 30°
(c) 30° to 35° (d) 32° to 37°

161. The shear strength of a soil

(a) Is proportional to the cohesion of the soil
(b) Is proportional to the tangent of the angle of internal friction
(c) Increases with the increases in normal stress of soil
(d) All of the above

162. The expansion of soil due to shear at a constant value of pressure is called

(a) Apparent cohesion. (b) True cohesion
(c) Dialatancy (d) Consistency

163. Considerable loss of shear strength due to shock or disturbance is exhibited by

(a) Under consolidated clays
(b) Normally consolidated clays
(c) Over consolidated clays
(d) Organic soil **(IES 2010)**

164. The strength of soil is usually identified by

(a) Direct tensile stress
(b) Direct compressive stress
(c) Ultimate shear stress
(d) Effective stress

165. The force of attraction between the individual particles of soil which keeps the soil particles bound together is known as

(a) Compaction (b) Cohesion
(c) Friction (d) Dilatancy

166. Shear strength of very plastic cohesive soils is found out by means of

(a) Cone test (b) Penetration test
(c) Vane shear test (d) Torsional shear test

167. For the determination of shear strength parameters, c and ϕ, of soil in the laboratory the test to be conducted will be

(a) Triaxial compression test
(b) Sieve analysis
(c) Compaction test
(d) Relative density test **(IES 2012)**

168. The density of sand at which there is no change in volume under the influence of shearing strain produced due to shear stress is called

(a) Relative density (b) Apparent density
(c) Critical density (d) Any one of the above

169. The field density and field moisture content of a soil can be determined by

1. Core cutter method
2. Sand replacement method
3. Proctor compaction test
4. Modified proctor compaction test

(a) 1, 2, 3 and 4 (b) 1, and 2 only
(c) 2 and 3 only (d) 2 and 4 only

170. Surface tension is expressed in

(a) Newtons
(b) Newtons per millimetre length
(c) Newtons per square millimeter
(d) Newtons per cubic millimeters

171. Which law gives the diameter of a hypothetical sphere which will settle at the same terminal velocity.
(*a*) Darcy's law (*b*) Stoke's law
(*c*) Young's law (*d*) Hooke's law

172. The validity of stoke's law is limited to particle size not smaller than
(*a*) 0.0002 mm (*b*) 0.002 mm
(*c*) 0.02 mm (*d*) 0.2 mm

173. According to the stoke's law, the terminal velocity depends on
(*a*) Shape of the grains (*b*) Size of the grains
(*c*) Weight of the grains (*d*) All of the above

174. The terminal velocity of soil particles
(*a*) Varies as the radius of the particles
(*b*) Varies as the square of the radius of particles
(*c*) Varies as the cube of the radius of particles
(*d*) Varies inversely as the radius of the particles

175. The seepage force in soils is proportional to
(*a*) Exit gradient
(*b*) Head of water at upstream
(*c*) Head of water at downstream
(*d*) All of the above

176. The coefficient of active earth pressure is given by
(*a*) $k_a = \frac{1+\sin\phi}{1-\sin\phi}$ (*b*) $k_a = \frac{1-\sin\phi}{1+\sin\phi}$
(*c*) $k_a = \frac{1-\tan\phi}{1+\tan\phi}$ (*d*) $k_a = \frac{1+\tan\phi}{1-\tan\phi}$

177. The bearing capacity of a soil depends on
(*a*) The nature of load
(*b*) The extent of load
(*c*) The size of the footing
(*d*) The depth of foundation

178. A grillage foundation is essentially a
(*a*) Shallow foundation (*b*) Deep foundation
(*c*) Spread foundation (*d*) Pile foundation

179. The shear strength of a soil
(*a*) Is directly proportional to the angle of internal friction
(*b*) Is inversely proportional to the angle of internal friction.
(*c*) Is proportional to the tangent of the angle of internal friction
(*d*) Is inversely proportional to the tangent of the angle of internal friction.

180. Foundations for buildings on sandy soils must have a depth of atleast
(*a*) 0.1 metre (*b*) 0.5 metre
(*c*) 0.8 to 1 metre (*d*) 2 to 5 metre

181. Maximum bearing capacity can be expected from
(*a*) Laminated rocks (*b*) Compact coarse sand
(*c*) Granite rocks (*d*) Soft rocks

182. The bearing capacity of a soil
(*a*) Increases with the increase in the area of footing
(*b*) Reduces with decrease in the area of footing
(*c*) Increases with decrease in the area of footing
(*d*) Is not related with the size of footing.

183. Which soil is expected to have least bearing capacity?
(*a*) Laminated rocks
(*b*) Compact dry medium sand
(*c*) Losse fine sand
(*d*) Back cotton soil

184. Quick sand
(*a*) Is one that loses moisture rapidly
(*b*) Is a type of pure silica sand
(*c*) Is a condition wherein a cohesionless soil loses its strength because of upward flow of water
(*d*) Is a sand consisting of spherical sane particles only.

185. A cohesive soil on drying
(*a*) Expands (*b*) Shrinkage
(*c*) Swells (*d*) Change colour

186. Least bearing capacity may be expected from
(*a*) Loose gravel (*b*) Stiff clay
(*c*) Loose fine sand (*d*) Compacted coarse sand

187. Pile foundations are used
(*a*) In sandy soils
(*b*) In wet soils
(*c*) For multistoreyed buildings
(*d*) When requisite bearing area is not available.

188. The shape of precast concrete pile is generally
(*a*) Square (*b*) Round
(*c*) Square or octangonal (*d*) Square or circular

189. Consider the following statements regrading negative skin friction in piles.
1. It is developed when the pile is driven through a recently deposited clay layer
2. It is developed when the pile is driven through a layer of dense sand
3. It is developed due to a sudden drawdown of the water table

Of these statements
(*a*) 1 alone is correct (*b*) 1 alone is correct
(*c*) 2 and 3 are correct (*d*) 1 and 3 are correct

190. The allowable bearing capacity at 25 mm allowable settlement for a footing in sandy soil is 15 t/m^2. The allowable bearing capacity for the same footing permitting a settlement of 40 mm is
(*a*) 24 t/m^2 (*b*) 30 t/m^2
(*c*) 35 t/m^2 (*d*) 40 t/m^2

191. All theoretical approaches indicate that at greater depths the bearing capacity of pile base in sands should be practically independent of its size and be proportional to
(*a*) Shape of pile
(*b*) Overburden pressure σ
(*c*) Friction angle ϕ
(*d*) Shape of pile and friction angle ϕ

192. A test concrete block is subjected to vertical vibration and resonance occourred at a frequency of 20 cycles per sec. If mass of vibration is 6 kg. and mass of

ANSWERS

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

61. (d)	62. (a)	63. (c)	64. (b)	65. (d)	66. (b)	67. (a)	68. (d)	69. (b)	70. (c)
71. (c)	72. (d)	73. (d)	74. (a)	75. (b)	76. (c)	77. (b)	78. (b)	79. (b)	80. (d)
81. (d)	82. (c)	83. (d)	84. (a)	85. (c)	86. (b)	87. (d)	88. (b)	89. (b)	90. (c)
91. (b)	92. (b)	93. (d)	94. (b)	95. (b)	96. (c)	97. (a)	98. (c)	99. (c)	100. (d)
101. (a)	102. (a)	103. (c)	104. (c)	105. (c)	106. (c)	107. (d)	108. (b)	109. (c)	110. (c)
111. (b)	112. (b)	113. (c)	114. (a)	115. (d)	116. (c)	117. (c)	118. (d)	119. (d)	120. (c)
121. (d)	122. (b)	123. (d)	124. (c)	125. (b)	126. (a)	127. (c)	128. (a)	129. (c)	130. (c)
131. (b)	132. (c)	133. (b)	134. (c)	135. (a)	136. (b)	137. (b)	138. (b)	139. (c)	140. (d)
141. (b)	142. (c)	143. (c)	144. (b)	145. (c)	146. (b)	147. (d)	148. (a)	149. (c)	150. (a)
151. (b)	152. (b)	153. (a)	154. (b)	155. (b)	156. (b)	157. (b)	158. (d)	159. (b)	160. (d)
161. (c)	162. (c)	163. (a)	164. (c)	165. (b)	166. (c)	167. (a)	168. (c)	169. (b)	170. (b)
171. (b)	172. (a)	173. (d)	174. (b)	175. (a)	176. (b)	177. (c)	178. (c)	179. (b)	180. (c)
181. (c)	182. (c)	183. (c)	184. (c)	185. (b)	186. (c)	187. (d)	188. (d)	189. (d)	190. (a)
191. (b)	192. (d)	193. (d)	194. (b)	195. (b)	196. (a)	197. (a)	198. (d)	199. (d)	200. (c)
201. (d)	202. (d)	203. (c)	204. (c)	205. (d)	206. (b)	207. (d)	208. (c)	209. (b)	210. (d)
211. (b)	212. (c)	213. (a)	214. (d)	215. (b)	216. (b)	217. (d)	218. (c)	219. (b)	220. (a)
221. (b)	222. (c)	223. (c)	224. (b)	225. (c)	226. (c)	227. (a)	228. (d)	229. (a)	230. (c)
231. (d)	232. (d)	233. (c)	234. (c)	235. (b)	236. (c)	237. (c)	238. (d)	239. (a)	240. (b)
241. (b)	242. (b)	243. (b)	244. (a)	245. (d)	246. (a)	247. (c)	248. (a)	249. (c)	250. (a)
251. (c)	252. (b)	253. (c)	254. (a)	255. (b)	256. (d)	257. (b)	258. (a)	259. (b)	260. (a)
261. (b)	262. (d)	263. (c)	264. (c)	265. (c)	266. (d)	267. (b)	268. (b)	269. (a)	270. (b)
271. (b)	272. (a)	273. (a)	274. (c)	275. (d)	276. (c)	277. (b)	278. (b)	279. (c)	280. (c)
281. (c)	282. (d)	283. (c)	284. (c)	285. (d)	286. (d)	287. (a)	288. (d)	289. (c)	290. (a)
291. (b)	292. (a)	293. (c)	294. (a)	295. (b)	296. (a)	297. (d)	298. (b)	299. (a)	300. (c)
301. (d)	302. (a)	303. (a)	304. (c)	305. (d)	306. (c)	307. (c)	308. (b)	309. (b)	310. (c)
311. (b)	312. (c)	313. (a)	314. (c)	315. (b)	316. (b)	317. (a)	318. (b)	319. (a)	320. (a)
321. (b)	322. (d)	323. (d)	324. (c)	325. (b)	326. (a)	327. (b)	328. (b)	329. (b)	330. (b)
331. (b)	332. (a)	333. (b)	334. (c)	335. (d)	336. (a)	337. (b)	338. (c)	339. (d)	340. (b)
341. (a)	342. (d)	343. (c)	344. (d)	345. (a)	346. (a)	347. (c)	348. (b)	349. (b)	350. (d)
351. (b)	352. (c)	[illegible]	[illegible]	[illegible]					

Q.1 Soil is to be excavated from a borrow pit which has a density of 1.75 gm/cc and water content of 12%. The specific gravity of soil particles is 2.7. The soil is compacted so that water content is 18% and dry density is 1.65 gm/cc. For 1000 m^3 of soil in fill, estimate

(i) quantity of soil to be excavated from the pit in m^3,

(ii) amount of water to be added.

Also, determine the void ratios of the soil in borrow pit and fill.

[15 marks : 1995]

Solution:

Using the subscripts 1 and 2 for borrow pit and fill respectively. All the physical quantities have their usual meanings.

$\rho_1 = 1.75$ gm/cc

$w_1 = 12\%$

$G = 2.7$

V_{s_1} = Volume of solids in borrow pit

$V_2 = 1000\ m^3$

$w_2 = 18\%$

$\rho_{d_2} = 1.65$ gm/cc

V_{s_2} = Volume of solids in fill

$V_1 = ?$

(i) We know that volume of solids in borrow pit and the fill is same i.e.

$$V_{s_1} = V_{s_2}$$

Also
$$\rho_{d_1} = \frac{\rho_1}{1+w_1} = \frac{G\rho_w}{1+e_1}$$

$$\Rightarrow \quad \frac{1.75}{1+0.12} = \frac{2.7\times 1}{1+e_1}$$

$$\Rightarrow \quad 1 + e_1 = \frac{2.7\times 1.12}{1.75}$$

$$\Rightarrow \quad e_1 = 0.728$$

and
$$\rho_{d_2} = \frac{G\rho_w}{1+e_2}$$

$$\Rightarrow \quad 1.65 = \frac{2.7\times 1}{1+e_2}$$

$$\Rightarrow \quad 1+e_2 = \frac{2.7}{1.65}$$

$$\Rightarrow \quad e_2 = 0.636$$

We know that, $$e = \frac{V_v}{V_s}$$

$$\Rightarrow \quad e+1 = \frac{V_v}{V_s} + 1 = \frac{V_s + V_v}{V_s}$$

$$\Rightarrow \quad V_s = \frac{V}{1+e}$$

But $$V_{s_1} = V_{s_2}$$

$$\therefore \quad \frac{V_1}{1+e_1} = \frac{V_2}{1+e_2}$$

$$\Rightarrow \quad V_1 = \left(\frac{1+e_1}{1+e_2}\right)\times V_2$$

$$\Rightarrow \quad V_1 = \left(\frac{1+0.728}{1+0.636}\right)\times 1000$$

$$\Rightarrow \quad V_1 = 1056.23\ m^3$$

Hence, 1056.23 m^3 of soil is to be excavated from the borrow pit.

(ii) We know that

$$V_{s_2} = \frac{V_2}{1+e_2}$$

$$\Rightarrow \quad V_{s_2} = \frac{1000}{1+0.636} = 611.25\ m^3$$

$\therefore$ Weight of solids $= W_s = V_s \gamma_s = V_s G \gamma_w = 611.25 \times 2.7 \times 9.81$

$$W_s = 16190.18\ kN$$

Now, $$w_1 = \frac{W_{w_1}}{W_s}$$

where W_{w_1} = weight of water in pit $= w_1 \times W_s = 0.12 \times 16190.18 = 1942.8$ kN

W_{w_2} = weight of water in fill $= w_2 \times W_s = 0.18 \times 16190.18 = 2914.23$ kN

$\therefore$ Amount of water added $= W_{w_2} - W_{w_1}$

$$= 2914.23 - 1942.8 = 971.43\ kN = \frac{971.43\times 10^3}{9.81}\ kg$$

$$= 99.02 \times 10^3\ kg$$

Q.20 The void ratio and specific gravity of a sample of clay are 0.73 and 2.7 respectively. If the voids are 92% saturated, find the bulk density, the dry density and the water content.
What would be the water content for complete saturation, the void ratio remaining the same?

[10 marks : 1999]

Solution:

Given data:

$e = 0.73,\quad G = 2.7,\quad S = 92\%$

We know that $\quad Se = Gw$

$$\Rightarrow \quad w = \frac{Se}{G}$$

$$\Rightarrow \quad w = \frac{0.92 \times 0.73}{2.7}$$

$$\Rightarrow \quad w = 0.2487 \text{ or } 24.87\%$$

Now,

$$\gamma = \frac{(G + Se)\gamma_w}{1 + e}$$

$$\Rightarrow \quad \gamma = \left[\frac{2.7 + (0.92 \times 0.73)}{1 + 0.73}\right] \times 9.81$$

$$\Rightarrow \quad \gamma = 19.12 \text{ kN/m}^3$$

But

$$\text{Bulk density, } \rho = \frac{\gamma}{g} = \frac{19.12 \times 10^3}{9.81} = 1949.0316 \text{ kg/m}^3$$

$$\text{Dry density, } \rho_d = \frac{\rho}{1 + w} = \frac{1949.0316}{1 + 0.2487} = 1560.85 \text{ kg/m}^3$$

When $\quad S = 100\%$

then $\quad Se = Gw$

$$\Rightarrow \quad w = \frac{Se}{G} = \frac{1 \times 0.73}{2.7}$$

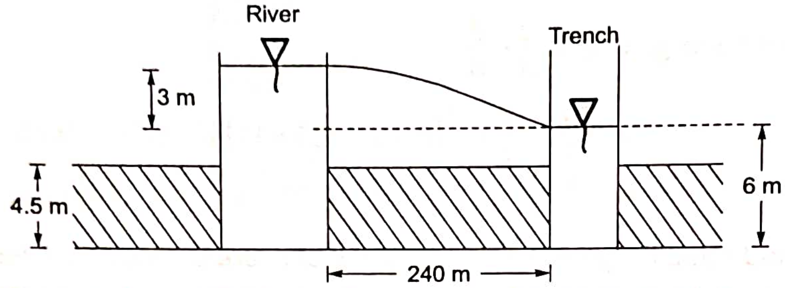
$$\Rightarrow \quad w = 0.2703 \text{ or } 27.03\%$$

Q.21 **A trench is to be excavated 240 m away from a river and in a confined aquifer 4.5 m thick. The trench has to run parallel to the river and is to be 300 m long. Water level in the trench is to be maintained 6 m above the lower confining layer and 3 m below the water level in the river. Determine the rate at which water should be pumped from the trench, if the hydraulic conductivity of the aquifer is 4.5 m/day. Assume there is no contribution to flow from the land side of the trench. State the condition when the assumption can be valid.**

[12 marks : 1999]

Solution:

As there is no contribution of flow from land side of the trench, the hydraulic gradient will vary linearly between the river and trench, otherwise the flow could have been radial.



Given data:

Hydraulic conductivity, $k = 4.5$ m/day
Length of trench, $L = 300$ m
Thickness of aquifer, $B = 4.5$ m

Hydraulic gradient between river and trench, $i = \dfrac{(6+3)-6}{240} = \dfrac{3}{240}$

Let the rate of discharge be Q, then

$$Q = kiA = kiLB = 4.5 \times \frac{3}{240} \times 300 \times 4.5 = 75.94 \text{ m}^3\text{/day}$$

where A is area through which water flows in the trench.

The above given assumption can only be valid when the flow is laminar i.e. Darcy's law is valid. It also means that the soil to be dewatered is homogeneous and isotropic.

Q.22 **A retaining wall 6 m high supports earth with its face vertical. The earth is cohesionless with particle specific gravity 2.69, angle of internal friction 35° and porosity 40.5%. The earth surface is horizontal and level with the top of the wall. Determine the earth thrust and its line of action on the wall if the earth is water logged to level 2.5 m below the top surface. Neglect wall friction. Draw the pressure diagrams.**

[15 marks : 1999]

Solution:

Given data:

Height of retaining wall, $H = 6$ m
Specific gravity of soil particles, $G = 2.69$
Angle of internal friction, $\phi = 35°$
Porosity, $n = 40.5\% = 0.405$
Depth of water table below top surface = 2.5 m

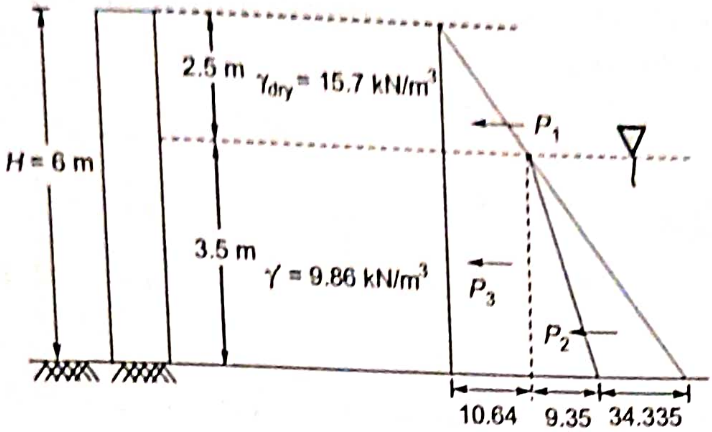
We know that $\quad n = \dfrac{e}{1+e}$

$\therefore \quad e = \dfrac{n}{1-n} = \dfrac{0.405}{1-0.405} = 0.681$

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w = \left(\frac{2.69+0.681}{1+0.681}\right)\times 9.81 = 19.67 \text{ kN/m}^3$$

$$\gamma_{dry} = \frac{G\gamma_w}{1+e} = \frac{2.69\times 9.81}{1+0.681} = 15.7 \text{ kN/m}^3$$

$$\gamma' = \gamma_{sat} - \gamma_w = 19.67 - 9.81 = 9.86 \text{ kN/m}^3 .$$



$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 35^\circ}{1+\sin 35^\circ} = 0.271$$

for $z = 2.5$ m

$$P_v = K_a\gamma z = 0.271 \times 15.7 \times 2.5 = 10.64 \text{ kN/m}^2$$

∴Total active thrust for z = 2.5 m is given by

$$P_1 = \frac{1}{2}\times P_v \times 2.5 = \frac{1}{2}\times 10.64\times 2.5 = 13.3 \text{ kN/m}$$

For $z = 3.5$ m

$$P_{v_1} = K_a\gamma' z = 0.271 \times 9.86 \times 3.5 = 9.35 \text{ kN/m}^2$$

$$P_{v_2} = \gamma_w z = 9.81\times 3.5 = 34.335 \text{ kN/m}^2$$

$$P_{v_1} + P_{v_2} = 9.35 + 34.335 = 43.685 \text{ kN/m}^2$$

∴Total active thrust for z =3.5 m is given by

$$P_2 + P_3 = \frac{1}{2}\times(43.685)\times 3.5 + 10.64 \times 3.5 = 76.45 + 37.24 = 113.69 \text{ kN/m}$$

∴Total active thrust on the retaining wall = $P_1 + P_2 + P_3 = 13.3 + 76.45 + 37.24 = 126.99$ kN/m

$$\text{Location of total thrust from base} = \frac{13.3\times\left(3.5+\frac{2.5}{3}\right)+37.24\times\left(\frac{3.5}{2}\right)+76.45\times\left(\frac{3.5}{3}\right)}{126.99} = 1.67 \text{ m}$$

Q.29 Work out the theoretical maximum dry density for a soil sample having specific gravity of 2.7 and OMC = 16%. Also explain the difference in OMC values in case of Proctor Test and Modified Proctor Test for cohesive soils and granular soils.

[10 marks : 2001]

Solution:

Given data:

$G = 2.7$, $OMC = w = 16\%$, $\gamma_w = 9.81$ kN/m^3

We know that

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

But

$$Se = Gw$$

$\Rightarrow$

$$e = \frac{Gw}{S}$$

$\therefore$

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

where γ_d is dry unit weight of soil and all other symbols have their usual meanings.

Now, for a given water content, the theoretical maximum value of dry unit weight for a compacted soil is obtained corresponding to the situation where no air voids are left i.e. when the degree of saturation becomes equal to 100% i.e. S = 1

$\therefore$

$$\gamma_{d,max} = \frac{G\gamma_w}{1+\frac{Gw}{1}}$$

$\Rightarrow$

$$\gamma_{d,max} = \frac{G\gamma_w}{1+Gw} = \frac{2.7\times 9.81}{1+2.7\times 0.16} = 18.49 \text{ kN/m}^3$$

The only difference between Proctor Test and Modified Proctor Test is that the latter requires more compactive effort than the former. The dry density increases with the compactive effort but the water content decreases along with it. The compaction curve for the Modified Proctor Test shifts to the top and to the left w.r.t. the compaction curve for Proctor Test.

There is a difference between OMC values of cohesive and granular soils as well. Cohesive soils have large value of OMC than granular soils.

Q.35 What will be the ratio of average permeability in the horizontal direction to that in the vertical direction for a soil deposit consisting of three horizontal layers if the thickness and permeability of the second layer are twice those of the first and those of the third layer twice those of the second?

[10 marks : 2002]

Solution:

Given data:

For 1st layer H_1, k_1

For 2nd layer $H_2 = 2H_1;\ k_2 = 2k_1$

For 3rd layer $H_3 = 2H_2 = 4H_1;\ k_3 = 2k_2 = 4k_1$

When the flow is in the horizontal direction, the hydraulic gradient is same through each layer but the discharge and velocity of flow through each layer is different.

$$\therefore \quad k_H = \frac{k_1H_1 + k_2H_2 + k_3H_3}{H_1 + H_2 + H_3} = \frac{k_1H_1 + (2k_1 \times 2H_1) + (4k_1 \times 4H_1)}{H_1 + 2H_1 + 4H_1}$$

$$= \frac{k_1H_1 + 4k_1H_1 + 16k_1H_1}{7H_1} = \frac{21\,k_1H_1}{7H_1}$$

$$\therefore \quad k_H = 3k_1$$

Now when the flow is vertical, then the discharge and velocity of flow remains same but hydraulic gradient changes in each layer

$$\therefore \quad k_V = \frac{H_1 + H_2 + H_3}{\dfrac{H_1}{k_1} + \dfrac{H_2}{k_2} + \dfrac{H_3}{k_3}} = \frac{H_1 + 2H_1 + 4H_1}{\dfrac{H_1}{k_1} + \dfrac{2H_1}{2k_1} + \dfrac{4H_1}{4k_1}} = \frac{7H_1}{3H_1/k_1} = \frac{7}{3}k_1$$

$$\therefore \quad \frac{k_H}{k_V} = \frac{3k_1}{\dfrac{7}{3}k_1} = \frac{3 \times 3}{7} = \frac{9}{7}$$

Q.37 In a consolidation test, the void ratio of the specimen which was 1.068 under the effective pressure of 214 kN/m^2, changed to 0.994 when the pressure was increased to 429 kN/m^2. Calculate the coefficient of compressibility, compression index and the coefficient of volume compressibility. Find the settlement of foundation resting on above type of clay if thickness of layer is 8 m and the increase in pressure is 10 kN/m^2.

[15 marks : 2002]

Solution:

Given data:

$e_1 = 0.994$; $e_0 = 1.068$; $\bar{\sigma}_1 = 429$ kN/m^2; $\bar{\sigma}_0 = 214$ kN/m^2; $H_0 = 8$ m; $\Delta\bar{\sigma}_0 = 10$ kN/m^2

Coefficient of compressibility, $a_v = -\dfrac{\Delta e}{\Delta\bar{\sigma}}$

where Δe is change in void ratio $= e_1 - e_0$

and $\Delta\bar{\sigma}$ is change in effective pressure $= \bar{\sigma}_1 - \bar{\sigma}_0$

$$\therefore \quad a_v = -\left(\frac{0.994 - 1.068}{429 - 214}\right) = 3.44 \times 10^{-4} \text{ m}^2/\text{kN}$$

$$\text{Coefficient of volume compressibility, } m_v = \frac{a_v}{1+e_0} = \frac{3.44 \times 10^{-4}}{1+1.068} = 1.66 \times 10^{-4} \text{ m}^2/\text{kN}$$

$$\text{Compression index, } C_c = -\frac{\Delta e}{\log_{10}\left(\dfrac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)} = -\frac{(0.994 - 1.068)}{\log_{10}\left(\dfrac{429}{214}\right)} = 0.245$$

Settlement of foundation, $\Delta H = m_v H \Delta\sigma' = 13.315$ mm

Q.45 In a Proctor compaction test, the soil specimen of one of the observations had a bulk density of $19\ kN/m^3$ with a moisture content of 15%. Find,

(i) degree of saturation of the specimen if $G = 2.7$

(ii) additional moisture content required for saturating the soil specimen.

[7 marks : 2004]

Solution:

Given data

$\gamma = 19\ kN/m^3$, $w = 15\%$, $G = 2.7$

(i) We know that
$$\gamma_d = \frac{\gamma}{1+w}$$

$$\Rightarrow \quad \gamma_d = \frac{19}{1+0.15} = 16.52\ kN/m^3$$

But
$$\gamma = \frac{G+Se}{1+e} \times \gamma_w \qquad [\because Se = Gw]$$

$$\Rightarrow \quad \gamma = \frac{G+Gw}{1+e} \times \gamma_w$$

$$\Rightarrow \quad 1+e = \frac{(1+w)\,G\gamma_w}{\gamma}$$

$$\Rightarrow \quad e = \frac{(1+0.15)\times 2.7\times 9.81}{19} - 1 = 0.603$$

But
$$S = \frac{Gw}{e} = \frac{2.7\times 0.15}{0.603} = 0.6716 = 67.16\%$$

Thus, degree of saturation is 67.16%

(ii) For degree of saturation = 100%, $S = 1$

But $Se = Gw$

$$w = \frac{Se}{G}$$

$$\Rightarrow \quad w = \frac{1\times 0.603}{2.7} \times 100 = 22.33\%$$

$\therefore$ Additional water content required = 22.33 − 15 = 7.33% **(Ans.)**

... clay it failed at a depth of 2.8 m of

Q.48 Explain the following: (i) Initial compression (ii) Primary consolidation (iii) Secondary consolidation (iv) Primary compression ratio.

[8 marks : 2004]

Solution:

(i) **Initial Compression**

If soil is partially saturated, then immediately after the application of load, the volume decreases due to expulsion of air as well as due to compression of pore air which is called initial compression. At the end of initial compression of soil, it becomes fully saturated if load is sufficiently large. The result of initial compression is the immediate settlement which is usually determined by using elastic theory, even though the deformation itself is not truely elastic. Computation of immediate settlement has to be made in the design of shallow foundations.

(ii) **Primary Consolidation**

After the initial compression, soil is fully saturated and further decrease in volume occurs due to the expulsion of pore water and compression of pore water (water is incompressible, hence volume change due to compression of pore water is negligible). It is a time dependent phenomenon which depends upon permeability of soil and magnitude of load applied. The rate of flow is controlled by pore pressure, the permeability and compressibility of soil with the passage of time as the pore pressure dissipates, the rate of flow decreases and eventually, flow ceases altogether, leading to a condition of constant effective stress. This signifies the end of primary consolidation.

(iii) **Secondary Consolidation**

After the completion of primary settlement when pore water pressure ceases to zero at the top surface, and no decrease in volume may be expected hence forth. But in practice there is always a certain decrease in volume after primary consolidation after a long time which is called secondary consolidation . The actual cause for secondary consolidation is not well established but it may be attributed to plastic readjustment of soil solids. The secondary settlement in granular soils is insignificant but in highly plastic soils it is 10-20% of total consolidation.

(iv) **Primary Compression Ratio**

It is the ratio of primary compression to total compression and is denoted by r_p.

$$\therefore \qquad r_p = \frac{\text{Primary compression}}{\text{Total compression}}$$

Primary compression ratio and other ratios like initial compression ratio and secondary compression ratios are used while determining the coefficient of consolidation (C_v) in laboratory. The two methods namely Casagrande logarithm of time fitting method and Taylor square root of time method use these ratios effectively. In these methods dial gauge readings which give compression of sample are plotted against logarithm of time or square root of time for various degree of consolidation. Thus

$$r_p = \frac{10}{9} \times \frac{(R_0 - R_{90})}{(R_i - R_f)} = \frac{R_0 - R_{100}}{R_i - R_f}$$

Where R_i = initial dial gauge reading

R_f = final dial gauge reading

R_0 = dial gauge reading at zero per cent consolidation

R_{90} = dial gauge reading at 90% consolidation

R_{100} = dial gauge reading at 100% consolidation

Q.49 Under a certain loading, a layer of clay is expected to undergo full settlement of 18 cm. Also it is expected to settle by 5 cm in the period of first 2 months of loading. Find the time required for the clay layer to settle by 10 cm.

[7 marks : 2004]

Solution:

(i) We know that degree of consolidation is given by

$$U = \frac{\Delta h}{\Delta H} \times 100$$

Where Δh is settlement at any stage and ΔH is total settlement

Given $\Delta H = 18$ cm, $\Delta h_1 = 5$ cm, $t_1 = 2$ months, $\Delta h_2 = 10$ cm, $t_2 = ?$

Now
$$U_1 = \frac{5}{18} \times 100 = 27.78\% = 0.2778$$

$$U_2 = \frac{10}{18} \times 100 = 55.55\% = 0.5555$$

Now, we know that, $T_v = \frac{C_v t}{d^2}$

where T_v is time factor $= \frac{\pi}{4} U^2$

C_v is coefficient of consolidation which is constant for a particular soil.

d is length of drainage path

t is time required to complete any stage of consolidation

$\therefore \quad t \propto T_v \qquad$ [But $T_v \propto U^2$ where U is in fraction]

$\Rightarrow \quad t \propto U^2$

$$\Rightarrow \quad \frac{t_1}{t_2} = \left(\frac{U_1}{U_2}\right)^2$$

$$\Rightarrow \quad t_2 = \left(\frac{U_2}{U_1}\right)^2 \times t_1 = \left(\frac{0.5555}{0.2777}\right)^2 \times 2 = 8 \text{ months}$$

Q.53 Water flows in a confined aquifer from a fully penetrating river with a piezometric head of 12 m towards a river with a piezometric head of 10 m located 400 m away. If the aquifer hydraulic conductivity and effective porosity are 3 m/day and 0.15 respectively, estimate the seepage velocity in the aquifer. Determine the draw-down and piezometric head 150 m from an upstream river.

[12 marks : 2006]

Solution:

Given data:

$h_1 = 12$ m; $h_2 = 10$ m; $L = 400$ m; $k = 3$ m/day; $n = 0.15$; $v_s = ?$

From Darcy's law, we have

$$v = ki$$

where 'v' is discharge velocity, 'k' is hydraulic conductivity and 'i' is hydraulic gradient

$$i = -\frac{(h_2 - h_1)}{L} = -\frac{(10-12)}{400} = \frac{2}{400} = \frac{1}{200}$$

Now,
$$v = ki = 3 \times \frac{1}{200} = \frac{3}{200} \text{ m/day}$$

Seepage velocity,
$$v_s = \frac{v}{n} = \frac{3/200}{0.15} = 0.1 \text{ m/day}$$

Now, for determining drawdown and piezometric head at 150 m from the upstream river we have.

$h_1 = 12$ m; $L = 150$ m; $v = \frac{3}{200}$ m/day; $k = 3$ m/day; $h_2 = ?$

Again, we have
$$v = ki$$

$$\Rightarrow \quad v = -k \times \left(\frac{h_2 - h_1}{L}\right)$$

$$\Rightarrow \quad \frac{3}{200} = -3 \times \left(\frac{h_2 - 12}{150}\right)$$

$$\Rightarrow \quad \frac{15}{20} = 12 - h_2$$

$$\Rightarrow \quad h_2 = 12 - 0.75$$

$$\Rightarrow \quad h_2 = 11.25 \text{ m}$$

But drawdown,
$$s = h_1 - h_2 = 12 - 11.25 = 0.75 \text{ m}$$

$\therefore$ Piezometric head at 150 m away from upstream river = 11.25 m and drawdown = 0.75 m

Q.60 State and discuss different factors influencing compaction of soil in the field. [15 marks : 2006]

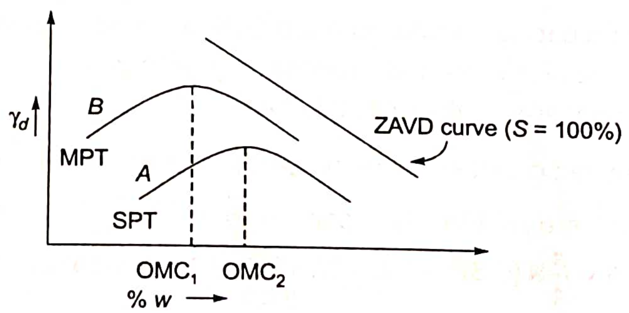
Solution:

There are four main factors which influence compaction and they are as follows:

(i) **Water Content:** There are two theories to explain the typical water content-dry unit weight relationship. They are the Lubrication theory by Proctor and the Electrical Double Layer theory by Lambe.

According to Lubrication theory at lower water contents, the soil is stiff and the soil grains offer more resistance to compaction. As the water content increases, the dry density increases and air voids are decreased till the optimum water content is reached, a stage when lubrication effect is maximum. With further increase in moisture content, however the water starts to replace the soil particles and since $\gamma_w << \gamma_s$, the dry unit weight starts decreasing.

Lambe uses concept of soil structure and the Electrical Double Layer theory to explain the effect of water content on dry unit weight. In case of cohesive soils, there is an attractive force namely the Van-der Waal's forces which acts between two soil particles and a repulsive force which is due to double layers of adsorbed water tending to come into contact with each other. While the attractive forces remains same in magnitude, the repulsive force is directly related to the size of double layers. If the net force between the particles is attractive, flocculated structure is the result; if it is repulsive, the particles tends to move away - 'disperse'. At low water contents attractive forces are predominant which makes it difficult for the particles to move about when compactive effort is applied. A low dry unit weight is the consequence. As the water content is increased, the double layer expands and inter particle repulsive forces increase. The particles easily slide over one another and get packed more closely, resulting in higher dry unit weight. The maximum expansion of the double layer is at the OMC, beyond that, the addition of water does not add any further to the expansion of double layer but the water tends to occupy space which otherwise would have been occupied by soil particles. Hence a decrease in unit weight. It also explains why the shape of the compaction curve is not the typical inverted V shape in the case of soils which are not cohesive and plastic in nature.



(II) **Compactive Effort:**

For a given type of compaction, the higher the compactive effort, the higher the maximum dry unit weight and lower the OMC. In the above figure compaction curve B corresponds to the higher compactive effort in a MPT, comparing it with the compaction curve A for SPT, one can see the compaction curve shifts to the top and to the left when compactive effort is increased. However, the margin of increase becomes smaller and smaller even on the dry side of the OMC while on the wet side of OMC, there is hardly any increase at all. If the peaks of compaction curves for different compactive efforts are joined together a 'line of optimums' is obtained which is nearly parallel to zero air void line. This brings out the fact that even a higher compactive effort does not result in a higher efficiency of compaction.

(iii) **Types of Soil:**

(a) Coarse grained, well graded soils compact to high dry unit weight especially if they contain some fines.

(b) Poorly graded sands lead to lowest dry unit weight values.

(c) In clay soils, the maximum dry unit weight tends to decrease as plasticity increases.

(d) Cohesive soils have generally high values of OMC.

(e) Heavy clays with high plasticity have very low maximum dry density and very high OMC.

(iv) **Methods of Compaction:**

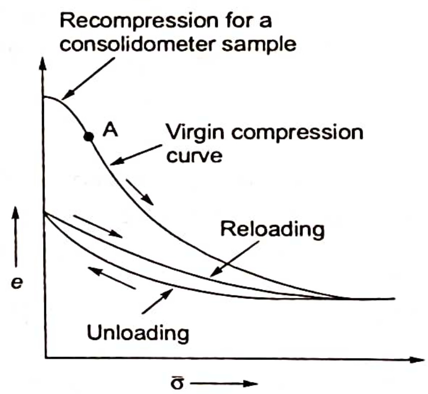
Ideally speaking, the laboratory test must reproduce a given field compaction procedure, because the mode of compaction does influence somewhat the shape and the position of the 'γ_d' vs 'w' plot. Since the field compaction is essentially a kneading type compaction or rolling type compaction and the laboratory tests use the dynamic impact type compaction, one must expect some divergence in the OMC and $\gamma_{d(max)}$ in the two cases.

Q.61 Write the significance of pre-consolidation pressure in soil. How would you determine the pre-consolidation pressure using Casagrande method.

[10 marks : 2006]

Solution:

Soils tend to retain the effect of stress changes that have taken place in their geological history, in the form of their structure. When a soil is stressed to a level greater than maximum stress to which it was ever subjected in the past, perhaps some kind of a break down in the soil structure occurs, resulting in a much higher compressibility indicated by a steep void ratio-effective stress curve ($e-\bar{\sigma}$)



The initial flatter portion of the $e-\bar{\sigma}$ curve is called recompression curve and the steeper portion after the break in the curve (attributed to a breakdown in a structure) is called virgin compression curve, because the soil is experiencing first time the increased stresses in this part. Some where between these two parts of the curve lies the (point A) corresponding to the maximum value of stress the soil has ever experienced, called the preconsolidation pressure or stress $\bar{\sigma}_c$.

A soil is said to be preconsolidated or over consolidated , if the existing effective stress is less than the preconsolidation stress, that is $\bar{\sigma} < \bar{\sigma}_c$. Here comes into picture the term OCR (over consolidation ratio) which is the ratio of the preconsolidation stress to the present vertical effective stress; that OCR = $\frac{\bar{\sigma}_c}{\bar{\sigma}}$

For normally consolidated soil OCR = 1 and for a preconsolidated soil OCR > 1

Q.85 A deposit of fine sand has a porosity of 45%. Estimate the critical hydraulic gradient to develop quicksand condition if the specific gravity of grain is 2.7.

[5 marks : 2009]

Solution:

Given data:

Porosity of sand, $n = 45\%$

Specific gravity of sand grains, $G = 2.7$

The critical hydraulic gradient to develop quicksand condition is given by

$$i_c = \frac{G-1}{1+e}$$

But void ratio,

$$e = \frac{n}{1-n} = \frac{0.45}{1-0.45} = 0.82$$

$\therefore$

$$i_c = \frac{2.7-1}{1+0.82} = 0.9341$$

$\ldots 6.81 \times 1.2 = 270.61 \text{ kN/m}^2$

Q.96 Discuss pore pressure parameters.

[3 marks : 2010]

Solution:

If it is not possible to determine pore water pressure practically, it can be calculated by using theoretical approach given by A.W. Skempton. The pore pressure parameter represent the response of pore pressure due to change in the total stress under undrained conditions. Such parameters are A and B.

The pore water pressure can be classified in two stages:

(i) Consolidation stage or cell pressure stage

(ii) Shear stage or deviator stress stage

The value of B can be given as

$$B = \frac{\Delta u_c}{\Delta \sigma_c} = \frac{\Delta u_c}{\Delta \sigma_3}$$

Here B is the ratio of pore pressure developed to the change in confining pressure. Δu_c represents increase in pore pressure due to increase in cell pressure, $\Delta\sigma_c$. For a fully saturated soil $B=1$ and for a fully dried soil $B=0$. The value of B can also be determined by using soil properties. Thus, the value of B can be given as

$$B = \frac{1}{1+n\dfrac{C_v}{C_c}}$$

Where n is porosity, C_v is coefficient of consolidation, C_c is coefficient of compression

A is defined as

$$\bar{A} = AB$$

where $\bar{A}$ is related to change in pore water pressure due to change in deviator stress i.e.

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

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

where Δu_d is change in pore water pressure due to change in deviator stress.

Now

$$\Delta u = \Delta u_c + \Delta u_d = B \cdot \Delta\sigma_3 + \bar{A} \cdot \Delta\sigma_d$$

$$= B \cdot \Delta\sigma_3 + AB \cdot \Delta\sigma_d = B \cdot \Delta\sigma_3 + AB(\Delta\sigma_1 - \Delta\sigma_3)$$

$\therefore$

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \sigma_3)]$$

The value of A may be as high as 2 to 3 for saturated fine sand and may be as low as –0.5 for heavily over consolidated clays. For a given soil A depends upon the strain, anisotropy, sample disturbance and the over consolidation ratio.

Q.97 **In a laboratory vane shear test a vane 100 mm long and 60 mm diameter was pressed into the soft cohesive soil (G = 2.72). A torque of 40 kN-mm was required to achieve the failure. Same soil when remoulded required 15 kN-mm to achieve failure. Calculate the cohesion in both cases and value of sensitivity. Take the void ratio of soil as 30%.**

[5 marks : 2010]

Solution:

Given that

$$d = 60 \text{ mm}, h = 100 \text{ mm}, G = 2.72, e = 30\%$$
$$T_1 = 40 \text{ kN-mm}, T_2 = 15 \text{ kN-mm}$$

Assuming that both upper and lower end of the vane shear the soil.

In undisturbed condition,

$$T_1 = c_u \pi d^2 \left[\frac{h}{2} + \frac{d}{6}\right]$$

$$\Rightarrow \quad 40 \times 10^{-3} = c_u \times \pi \times (0.06)^2 \times \left[\frac{0.1}{2} + \frac{0.06}{6}\right]$$

$$\Rightarrow \quad c_u = \frac{40 \times 10^{-3}}{6.786 \times 10^{-4}}$$

$$\Rightarrow \quad c_u = 58.95 \text{ kN/m}^2$$

Since the soil is soft clay, the unit undrained shear resistance will be the shear strength of the soil. Therefore shear strength of soil in undisturbed state is 58.95 kN/m^2.

In remoulded condition,

$$T_2 = c_{ur} \pi d^2 \left[\frac{h}{2} + \frac{d}{6}\right]$$

$$\Rightarrow \quad 15 \times 10^{-3} = C_{ur} \times \pi \times (0.06)^2 \times \left[\frac{0.1}{2} + \frac{0.06}{6}\right]$$

$$\Rightarrow \quad c_{ur} = \frac{15 \times 10^{-3}}{6.786 \times 10^{-4}}$$

$$\Rightarrow \quad c_{ur} = 22.1 \text{ kN/m}^2$$

Thus shear strength of soft clay in remoulded condition is 22.1 kN/m^2

$$\text{Sensitivity of soil} = \frac{\text{Undisturbed soil strength}}{\text{Remoulded soil strength}} = \frac{58.95}{22.1} = 2.67$$

Thus the soil is normal as the value of sensitivity lies between 1 and 4

Q.98 **In a consolidation test done in laboratory a sample of 20 mm thick consolidated 50% in 15 minutes with double drainage. How much time a 5.0 m thick layer of same soil will consolidate 50% and 30%? If the soil layer has a rock below, how much time it will take to consolidate 50% and 30%?**

[8 marks : 2010]

Solution:

As per Taylor's formula, time factor is given by

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

where C_v is coefficient of consolidation, t is time to complete any stage of consolidation and d is length of drainage path.

When degree of consolidation i.e. $U \leq 60\%$; $T_v = \frac{\pi}{4} U^2$

$\therefore$ For U = 50%,

$$T_v = \frac{\pi}{4}\times(0.5)^2 = 0.1963$$

$$d = \frac{20}{2} = 10\text{ mm (double drainage)}$$

$$t = 15\text{ minutes for 50\% consolidation}$$

$\therefore$

$$C_v = \frac{T_v d^2}{t} = \frac{0.1963\times(10)^2}{15} = 1.31\text{ mm}^2/\text{min}$$

For $U = 30\%$,

$$T_v = \frac{\pi}{4}\times(0.3)^2 = 0.0707$$

(i) For

$$U = 50\%,\ d = \frac{5}{2} = 2.5\text{ m} = 2500\text{ mm}$$

$$t = \frac{T_v d^2}{C_v} = \frac{0.1963\times(2500)^2}{1.31}\times\frac{1}{60\times 24} = 650.38\text{ days} = 1.78\text{ years}$$

For $U = 30\%$,

$$t = \frac{T_v d^2}{C_v} = \frac{0.0707\times(2500)^2}{1.31}\times\frac{1}{60\times 24} = 234.24\text{ days}$$

(ii) For

$$U = 50\%,\ d = 5\text{ m} = 5000\text{ mm}$$

$$t = \frac{T_v d^2}{C_v} = \frac{0.1963\times(5000)^2}{1.31}\times\frac{1}{60\times 24}\times\frac{1}{365} = 7.13\text{ years}$$

For $U = 30\%$

$$D = 5\text{ m} = 5000\text{ mm}$$

$$t = \frac{T_v d^2}{C_v} = \frac{0.0707\times(5000)^2}{1.31}\times\frac{1}{60\times 24}\times\frac{1}{365} = 2.57\text{ years}$$

Q.99 A 1.5 m layer of soil is subjected to an upward seepage head of 1.95 m. What depth of coarse sand will be required above this soil to provide a factor of safety of 1.5 against piping. Coarse sand and soil have specific gravity 2.67 and porosity as 30%.

[8 marks : 2010]

Solution:

Given that seepage head, h = 1.95 m, Factor of safety, F = 1.5, Specific gravity, G = 2.67 and porosity, $n = 30\%$

Void ratio of the soil is given by

$$e = \frac{n}{1-n} = \frac{0.3}{1-0.3} = \frac{0.3}{0.7} = \frac{3}{7} = 0.43$$

Critical hydraulic gradient is given by

$$i_c = \frac{G-1}{1+e} = \frac{2.67-1}{1+0.43} = 1.1678$$

With a factor of safety of 1.5 against piping, the gradient will be

$$i = \frac{i_c}{F} = \frac{1.1678}{1.5} = 0.7785$$

But,

$$i = \frac{h}{L}$$

$\Rightarrow$

$$L = \frac{h}{i} = \frac{1.95}{0.7785} = 2.5\text{ m}$$

Available flow path i.e. thickness of existing soil = 1.5 m

$\therefore$ Depth of coarse sand required = $L - 1.5 = 2.5 - 1.5 = 1$ m

Q.102 An oven dry soil sample of volume 225 cm^3 weighs 390 g. If the specific gravity is 2.72, determine the void ratio and shrinkage limit. What will be the water content which will fully saturate the sample and cause an increase in volume equal to 8% of the original dry volume?

[10 marks : 2011]

Solution:

Given: Volume of dry sample of soil $= V = 225\ cm^3$

Weight of dry sample of soil $= W = 390\ g$

$$G = 2.72$$

$$\text{Dry density of sample} = \gamma_d = \frac{W}{V} = \frac{390}{225} = 1.73\ g/cm^3$$

Using

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

$$\Rightarrow \quad 1.73 = \frac{2.72 \times 1}{1+e}$$

$$\Rightarrow \quad e = 0.57$$

$$\text{Shrinkage limit} = \frac{e}{G} = \frac{0.57}{2.72} = 20.95\%$$

Water content for 100% saturation and increase in volume corresponding to 8%

$$V = 225\ cm^3$$

$$V_V + V_S = 225\ cm^3$$

$$\frac{V_V}{V_S} = 0.57$$

$$V_V = 81.7\ cm^3$$

$$V_S = 143.3\ cm^3$$

Increase in volume is 8%, hence $V_V = 225 \times 0.08 + 81.7 = 99.7\ cm^3$

Now,

$$e = \frac{V_V}{V_S} = \frac{99.7}{143.3} = 0.696$$

$$S \times e = W \times G$$

$$1 \times 0.696 = W \times 2.72$$

$$W = 25.6\%$$

Q.110 What do you understand by "index properties of soil"? Explain and list the properties under different categories?

[4 marks : 2012]

Solution:

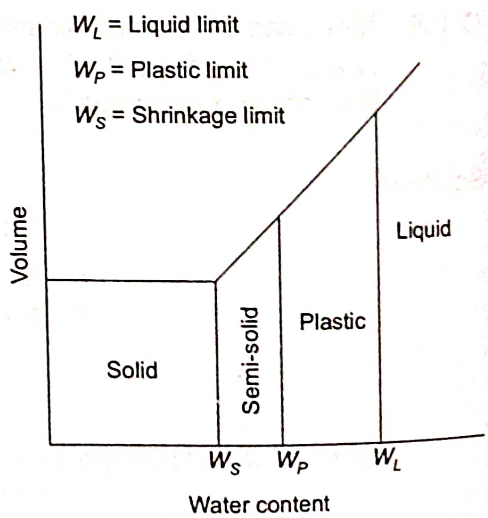
Index properties: These are indicative of engineering properties of soil and can be determined more easily then to determine the engineering properties of soil. The index properties of soils can be divided into two categories, namely, (i) soil grain properties, and (ii) soil aggregate properties. Soil grain properties are those properties which are dependent on the individual grains of the soil and are independent of the manner of soil formation. The properties in this category are mineralogical composition, specific gravity of solids, size and shape of grains. Soil aggregate properties are those properties which are dependent on the soil mass as a whole and, thus, represent the collective behaviour of a soil. The index properties are listed as below:

Grain Shape: The shape of soil grains is a useful soil grain property in the case of coarse-grained soils where it is important in influencing the engineering behaviour of these soils. The shape of grains in a coarse-grained soil can be examined with naked eyes, whereas fine-grained soils require microscopic examination.

Grain-size distribution or the percentage of various sizes of soil grains present in a given dry soil sample, is an important soil grain property. Grain size analysis of coarse-grained soils is carried out by sieve analysis, whereas fine-grained soils are analysed by the hydrometer method or the pipette method. In general, as most soils contain both coarse and fine-grained constituents, a combined analysis is usually carried out. In the combined grain size analysis, a soil sample in the dry state is first subjected to sieve analysis and then the finer fraction is analysed by the hydrometer or pipette method.

Consistency is a term which is used to describe the degree of firmness of a soil in a qualitative manner by using descriptions such as soft medium, firm, stiff or hard. It indicates the relative ease with which a soil can be deformed. In practice, the property of consistency is associated only with fine-grained soils, especially clays.

The physical properties of clays are considerably influenced by the amount of water present in them. Depending upon the water content, the following four stages or states of consistency are used to describe the consistency of a clay soil : (i) the liquid state; (ii) the plastic state; (iii) the semi-solid state, and (iv) the solid state. The boundary water contents at which the soil undergoes a change from one state to another are called "consistency limits" Atterberg, first demonstrated the significance of these limits. Hence, they are also known as the Atterberg Limits. These limits of water content, though empricial in nature, are of great significance in understanding the behaviour of clays.



Atterberg limits

Significance of other soil aggregate properties: In addition to the Atterberg's limits, there are other soil aggregate properties which are of significance. These are:

Permeability: The permeability of a soil is an important soil aggregate property and is useful in several engineering problems, e.g., seepage through soils, drainage, rate of settlement of compressible layers, etc. The property of soil which permits water to percolate through it, is called permeability.

Unconfined Compressive Strength: The unconfined compressive strength of a cohesive soil is related to the consistency of clays. Unconfined compressive strength is defined as the load per unit area at which an unconfined prismatic or cylindrical specimen of standard dimensions of a soil fails in a simple compression test. It is twice the value of shear strength of a clay soil under undrained condition.

Sensitivity and Thixotropy: It is observed that cohesive soils, upon remoulding, lose a part of their strength. The loss in strength upon remoulding is attributed partly to the breaking down of the original structure of the soil and partly to the disturbance caused to water molecules in the adsorbed layer. Sensitivity is a measure of the loss in strength of soils as a result of remoulding and is, thus, indicative of the effect of remoulding on the consistency of a cohesive soil.

Sensitivity, S_t is defined as the ratio of the unconfined compressive strength of an undisturbed specimen of the soil to the unconfined compressive strength of a specimen of the same soil after remoulding at unaltered water content.

$$S_t = \frac{(q_u)\text{ undisturbed}}{(q_u)\text{ remoulded}}$$

Relative density: The degree of denseness or looseness of natural deposits of coarse-grained soils can be measured in terms of their relative density. Relative density is, for a coarse-grained soil, the equivalent of relative consistency for a clay soil. It is the most important soil aggregate property of a coarse-grained soil.

Relative density, $D_r (I_D)$ is defined as the ratio of the difference between the void ratio of a cohesionless soil in the loosest state and void ratio in its natural state to the difference between its void ratios in the loosest and densest states.

$$D_r \text{ or } I_D = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}} \times 100\%$$

where e_{max} = void ratio in the loosest state

e_{mat} = void ratio obtained in the field in the natural state

e_{min} = void ratio in the densest state

Void ratio, Porosity and unit weight are other important soil aggregate properties.

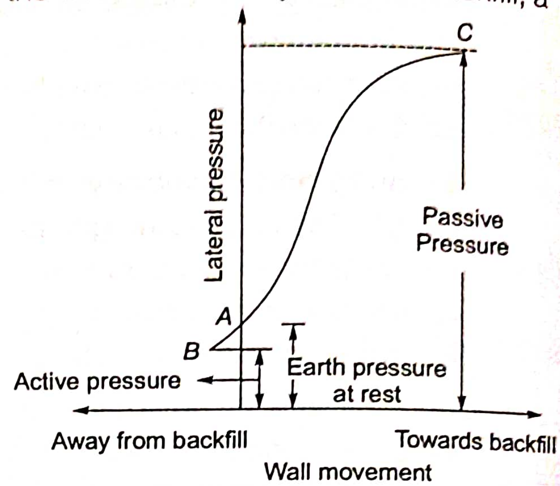
Q.111 What is meant by "earth pressure at rest", active earth pressure and passive earth pressure? Explain the difference in terms of wall movement. [4 marks : 2012]

Solution:

Structures which are used to hold back a soil mass are called retaining structure. Retaining walls, sheet pile walls, crib walls, sheetings in excavations, basement walls, etc., are examples of retaining structures. A retaining wall helps in maintaining the ground surface at different elevations on either side of it. Without such a structure, the soil at higher elevation would tend to move down till it acquires its natural, stable slope configuration.

Consequently, the soil that is retained at a slope steeper than it can sustain by virtue of its shearing strength, exerts a force on the retaining wall. This force is called the earth pressure and the material that is retained by the wall is referred to as backfill.

The given figure illustrates how the magnitude of earth pressure varies with the movement of the wall. When the wall is rigid and unyielding, the soil mass is in a state of rest and there are no deformations and displacements. The earth pressure corresponding to this state is called the earth pressure at rest (represented by point A). If the wall rotates about its toe, thus moving away from the backfill, the soil mass expands, resulting in a decrease of the earth pressure. This is a consequence of the mobilisation of shearing resistance in a direction opposing the movement of the earth mass. When the wall moves away from the backfill, a portion of the backfill located next to the retaining wall tends to break away from the rest of the soil mass and tends to move downwards and outwards relative to the wall. Since the shearing resistance is mobilised in directions away from the wall, there is a resultant decrease in earth pressure which continues, until, at a certain amount of displacement, failure will occur in the backfill and slip surfaces will be developed. At this state, the entire shearing resistance has been mobilised. The force acting on the wall at this stage (represented by point B) does not decrease any more beyond this point even with further wall movement. This force is called the active earth pressure.



Relationship between wall movement and lateral pressure

On the other hand, if the wall is pushed towards the backfill the soil is compressed and the soil offers resistance to this movement by virtue of its shearing resistance. Since the shearing resistance builds up in directions towards the wall, the earth pressure gradually increases. It this force reaches a value which the backfill cannot withstand, failure again ensues and slip surfaces develop. The pressure reaches a maximum value represented by point *C* when the entire shearing resistance has been mobilised and does not increase any more with further wall movement. This pressure is called the passive earth pressure.

Q.112 A solid sample has a porosity of 40%. The specific gravity of solids is 2.7. Calculate the (i) void ratio (ii) dry density (iii) unit weight if the soil is 50% saturated and (iv) unit weight if the solid is completely saturated.

[5 marks : 2012]

Solution:

Given data

Porosity of soil sample, $n = 40\%$

Specific gravity of soil solids $G_s = 2.7$

Now, as we know

(i)
$$n = \frac{e}{1+e}$$

$\therefore$
$$e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.667$$

(ii) Also
$$\gamma_d = \left(\frac{G_s}{1+e}\right)\gamma_w$$

Taking
$$\gamma_w = 9.81 \text{ kN/m}^3$$

$$\gamma_d = \frac{2.7}{1.667} \times 9.81$$

$\Rightarrow$
$$\gamma_d = 15.889 \text{ kN/m}^3$$

(iii) When soil is 50% saturated

$$\gamma = \left(\frac{G_s + Se}{1+e}\right)\gamma_w$$

where

$$S = 0.5$$

$$\therefore \quad \gamma = \left(\frac{2.7+0.5\times0.667}{1+0.667}\right)\times 9.81 = 17.852 \text{ kN/m}^3$$

(iv) When the soil is completely saturated, $S = 1$

$$\gamma_{sat} = \left(\frac{2.7+1\times0.667}{1+0.667}\right)\times 9.81 = 19.814 \text{ kN/m}^3$$

Q.113 A horizontal stratified deposit consists of four layers each uniform in itself. The permeabilities of the layers are 7.5 × 10⁻⁴ cm/sec, 49 × 10⁻⁴ cm/sec, 13 × 10⁻⁴ cm/sec and 17 × 10⁻⁴ cm/sec and their thicknesses are 5 m, 4 m, 17 m and 6 m respectively. Find the effective average permeabilities of the deposit in horizontal and vertical directions.

[5 marks : 2012]

Solution:

Given data

For 1st layer $H_1 = 5$ m, $K_1 = 7.5 \times 10^{-4}$ cm/sec

For 2nd layer $H_2 = 4$ m, $K_2 = 49 \times 10^{-4}$ cm/sec

For 3rd layer $H_3 = 17$ m, $K_3 = 13 \times 10^{-4}$ cm/sec

For 4th layer $H_4 = 6$ m, $K_4 = 17 \times 10^{-4}$ cm/sec

As we know, effective average permeability in horizontal direction, K_H is given by

$$\Rightarrow \quad K_H = \frac{K_1H_1 + K_2H_2 + K_3H_3 + K_4H_4}{H_1 + H_2 + H_3 + H_4}$$

$$\Rightarrow \quad K_H = \frac{7.5\times10^{-4}\times5 + 49\times10^{-4}\times4 + 13\times10^{-4}\times17 + 17\times10^{-4}\times6}{5+4+17+6}$$

$$\Rightarrow \quad K_H = \frac{10^{-4}\times(7.5\times5 + 49\times4 + 13\times17 + 17\times6)}{32}$$

$$\Rightarrow \quad K_H = 17.39\times10^{-4} \text{ cm/sec}$$

and effective permeability in the vertical direction, K_V is given by

$$K_V = \frac{H_1 + H_2 + H_3 + H_4}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3} + \frac{H_4}{K_4}}$$

$$\Rightarrow \quad K_V = \frac{5+4+17+6}{\frac{5}{7.5\times10^{-4}} + \frac{4}{49\times10^{-4}} + \frac{17}{13\times10^{-4}} + \frac{6}{17\times10^{-4}}} = \frac{32}{2.4089\times10^4}$$

$$\Rightarrow \quad K_V = 13.284\times10^{-4} \text{ cm/sec}$$