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PART - A

2 Marks Questions

1.

Sol. Porosity (n) = V_v/V_{Total}

$$n = V_v / (V_s + V_v)$$

Void ratio (e) = V_v/V_s

$$\text{Now, } n = \frac{V_v/V_s}{V_s + V_v/V_s}$$

$$n = \frac{V_v/V_s}{1 + V_v/V_s} = \frac{e}{1 + e}$$

$$\boxed{n = \frac{e}{1 + e}}$$

2.

Sol. The ratio of volume of air voids to the total volume of voids. It represents the fraction of voids which are filled with air.

3.

Sol.

$$W = 0.4$$

$$G = 2.7$$

Now,
Given

$$WG = Se$$

$$S = 1$$

$$e = 0.4 \times 2.7$$

$$e = 1.08$$

4.

Sol. Given

$$W = 10\%$$

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

$$G = 2.7$$

Now,

$$\gamma_d = \frac{\gamma_b}{1 + w} = \frac{20}{1 + 0.1} = 18.18 \text{ Kn / m}^3$$

$\therefore$

$$\gamma_d = \frac{G\gamma_w}{(1 + e)} \Rightarrow 18.18 = \frac{2.7 \times 9.81}{1 + e}$$

$$e = 0.457$$

5.

Sol. Relative density/Density index/Degree of Denseness - It is used to represent the relative degree of Compactness of Natural Cohesionless soil in absolute term unlike void ratio and dry density

$$\boxed{I_D = \frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}}$$

e_{max} - Void ratio of soil in loosest state.

e - Void ratio of soil in natural state.

e_{min} - void ratio of soil in densest state.

6.

Sol. Index properties of soil are those properties which help to access the engineering behaviour of a soil and which assist in determining its classification accurately.

The index properties for coarse and fine soils are tabulated below:

Type of soil	Index property
Coarse soil	Particle size distribution, Grain shape, Relative density
Fine soil	Atterberg's limit and consistency

7.
Sol. The specific gravity of solid particles is determined in the laboratory using the following methods:
- Density bottle method
 - Pycnometer method
 - Measuring flask method
 - Gas jar method
 - Shrinkage limit method

8.
Sol. Coefficient of uniformity represents the particle size range of soil in distribution curve. It is defined as the ratio of D_{60} size of particle to D_{10} size of particle.

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

9.
Sol. Minimum volume of soil mass at full saturation below which no volume change occurred.
10.
Sol. The liquid limit is determined in the laboratory by Casagrande's apparatus. i.e. water content corresponding to 25 blows.

11.
Sol. Plasticity Index : Plasticity index (I_p or PI) is the range of water content over which the soil remains in the plastic state. It is equal to the difference between the liquid (w_l) and the plastic limit (w_p). Thus,

$$I_p = w_l - w_p$$

Toughness index : Toughness index (I_t) of a soil is defined as the ratio of the plasticity index (I_p) to the flow index (I_f). Thus

$$I_t = \frac{I_p}{I_f}$$

12.
Sol. In-situ density is determined by
- Core cutter method
 - Sand replacement method
 - Water displacement method
13.
Sol. Stokes law is valid for particle sizes between 0.2 mm to 0.2 μm .
14.
Sol. When fines are between 5 to 12% dual symbols are required such as GP-GM, SW-SC etc.
15.
Sol. It is used for the classification of the soil and their suitability for subgrade design.
16.
Sol. **Water occurs in soil in four main Modes:** Gravitational (Ground/Free), Capillary water, Hygroscopic water and Chemically Combined (Structural Water) these types are defined by how tightly they are held within soil.

17.
Sol. **Pore water-** It is the water, filling the spaces between soil particles, moving through the soil. Ground water & Capillary water is also termed as pore water.
Adsorbed water - Adsorbed water is a thin film of water molecules tightly bound to the Surface of soil particles through adhesion and other force. Hygroscopic water and Film water also known as adsorbed water.

18.
Sol. The arrangement of soil particles in soil mass is called as structure of soil.

19.

Sol. It is the stress which is being transpered in soil mass by grain-to-grain contact which tends to force the particles to come into Closer State of Contact resulting im its decreased void ratio, increased degree of denseness & mobilization of shear strength.

20.

Sol. On lowering of water table effective stress increases.

21.

Sol. Many factoris affecting the Compaction are as follow

1. Water Content
2. Degree of Compactness
3. Method of compaction
4. Types of Soil.

22.

Sol. It is used for Soil Compaction, in this test Volume of mould is 945CC

- * No. of Layers are = 3
- * No. of blows on each layer = 25
- * Wt. of Hammer = 5.5 lb (2.445 kg)
- * Height of free fall of Hammer = 304.8 mm or 12 inches

23.

Sol. In this test volume of mould = 945CC

- * No. of layers = 5
- * No. of blows on each layer = 25
- * Wt. of Hammer = 10lb (4.54kg)
- * Height of free fall = 18 inches = 457.8mm.

24.

Sol. On increasing compacting effort OMC decreases and MDD increases.

25.

Sol. The flow of free water through soil is governed by Darcy's law. Darcy demonstrated experimentally that for laminar flow in a homogeneous soil, the velocity of flow (v) is given by

$$v = k.i$$

Where k = coefficient of permeability, i = hydraulic gradient.

The velocity of flow is also known as the discharge velocity or the superficial velocity.

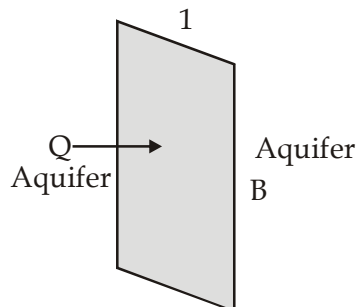
26.

Sol. Factors affecting permeability include grain size, void ratio, particle shape, and soil structure, as well as the properties of the pore fluid (like its temperature and viscosity), degree of saturation, and the presence of impurities or adsorbed water.

27.

Sol. Coefficient of transmissibility : Consider an aquifer of depth B and width unity. The discharge passing through this aquifer is

$$Q = vA = kiA = ki (B \times 1) = kiB$$



when hydraulic gradient (i) = 1, then discharge (Q) = Transmissibility (T) i.e.

$$T = k_B$$

28.

Sol. Discharge velocity is the average velocity of water if it flowed through the entire cross-sectional area of a porous medium, while seepage velocity is the actual average velocity of water flowing only through the void spaces, making it always greater than discharge velocity.

29.

Sol. Seepage pressure is the force exerted by water flowing through saturated soil on the soil particles, acting in the direction of the flow. Seepage pressure is directly related to the hydraulic gradient (i) and the unit weight of water (γ_w). The seepage force per unit volume is given by the formula $i \cdot \gamma_w$.

30.

Sol. Void ratio, $e = 0.7$
Specific gravity, $G_s = 2.67$
Critical hydraulic gradient

$$i_{cr} = \frac{G_s - 1}{1 + e} = \frac{2.67 - 1}{1 + 0.7} = 0.98$$

31.

Sol. The stress conditions occurring in a soil mass are primarily categorized into three main types: total stress, pore water pressure (or neutral stress), and effective stress. Total stress is the overall force from the weight of the soil and any surface loads. Pore water pressure is the stress exerted by water within the soil's pores. Effective stress is the difference between total stress and pore water pressure, and it's the stress carried by the soil grains, which governs its strength and deformation.

32.

Sol. The applications of flow nets in seepage analysis include determining the Quantity of seepage, Estimating seepage and pore pressures, Calculation of Uplift Pressure and Evaluation of exit gradient and piping potential.

33.

Sol. Primary consolidation is the process where a saturated soil layer decreases in volume due to the gradual expulsion of pore water under a sustained load. This process occurs because, initially, the incompressible water in the soil pores carries the majority of the applied load, generating excess pore water pressure. Over time, this excess water pressure dissipates as the water slowly drains out of the soil's voids. The applied load is then gradually transferred from the pore water to the soil solid particles, which causes the soil skeleton to compress and settle.

34.

Sol. Secondary consolidation settlement (also known as secondary compression or creep) is the change in volume of a fine-grained, saturated soil due to the plastic rearrangement of soil particles under a constant effective stress. This process occurs after the primary consolidation phase is largely complete, meaning most of the excess pore water pressure has already dissipated.

35.

Sol. The time taken for complete removal of water is called the hydrodynamic lag.

36.

Sol. **Coefficient of Compressibility :** The coefficient of compressibility (a_v) is defined as decrease in void ratio per unit increase in effective stress. It is equal to the slope of the $e - \bar{\sigma}$ curve at the point under consideration.

Thus,
$$a_v = \frac{-de}{d\bar{\sigma}} = \frac{-\Delta e}{\Delta \bar{\sigma}}$$

Coefficient of Volume Change : The coefficient of volume change (or the coefficient of volume compressibility) is defined as the volumetric strain per unit increase in effective stress. Thus

$$m_v = \frac{-\Delta V / V_0}{\Delta \bar{\sigma}}$$

Where, m_v = Coefficient of volume change
 V_0 = Initial volume,
 ΔV = Change in volume
and $\Delta \bar{\sigma}$ = Change in effective stress.

37.

Sol. Consolidated-drained test.

38.

Sol. The unconfined compression (UC) test is a simple and quick laboratory method used in geotechnical engineering to determine the unconfined compressive strength (q_u) of cohesive soils and weak rocks. This test measures the maximum axial compressive stress a cylindrical specimen can withstand without any lateral support or confining pressure.

39.

Sol. Liquefaction of sand is the phenomenon where saturated, loose sand temporarily loses its strength and stiffness from earthquake shaking or other rapid loading, behaving like a liquid. This occurs because the sudden shaking increases the water pressure within the soil's pore spaces, which reduces the contact forces between the sand grains and causes them to lose their solid structure.

40.

Sol. Cohesion of soils is the internal molecular attraction which resists the rupture or shear of a material.

41.

Sol. Active earth pressure is the minimum lateral pressure exerted by soil on a retaining structure when the structure moves or yields away from the backfill. This slight movement allows the soil mass to expand laterally until its internal shear strength is fully mobilized and it reaches a state of plastic equilibrium (the verge of failure).

42.

Sol. Passive earth pressure is the maximum resistance that a soil mass can offer when a retaining structure is pushed towards it, causing the soil to be compressed and mobilizing its full internal shear strength. This condition represents a state of limiting plastic equilibrium where the soil is on the verge of failure.

43.

Sol. Assumptions made in Rankine theory to analyze Active Earth Pressure are as follows:

1. The soil mass is homogenous and semi-infinite.
2. The soil is dry and cohesionless.
3. A ground surface is a plane, which may be horizontal or vertical.
4. The back of the retaining wall is smooth and vertical.
5. The soil element is in a state of plastic equilibrium.

44.

Sol. Given, $H = 3.6\text{m}$, $\phi = 30^\circ$, $\rho = 1.9\text{ gm/cc}$

Active earth Pressure at a depth of 3.6m (p_a) = $k_a \cdot \gamma \cdot H$

$$= \left(\frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} \right) \times 1.9 \times 9.81 \times 3.6$$

$$= 22.37 \text{ kN/m}^2$$

45.

Sol. A retaining wall's stability should be checked for Overturning, Sliding, Bearing capacity failure, and Settlement.

46.

Sol. Analyzing slope stability is crucial in engineering and environmental management because it is essential for protecting human life and property, ensuring the safety and longevity of infrastructure, and minimizing environmental degradation.

47.

Sol. Infinite slopes: They have dimensions that extend over great distances and the soil mass is inclined to the horizontal.

Finite slopes: A finite slope is one with a base and top surface, the height being limited. The inclined faces of earth dams, embankments and excavation and the like are all finite slopes.

48.

Sol. For an infinite slope of cohesionless soil under steady seepage parallel to the slope, the factor of safety (FS) is given by the formula:

$$FS = \left(\frac{\gamma'}{\gamma_{\text{sat}}} \right) \frac{\tan \phi'}{\tan \beta}$$

Where:

- γ' is the buoyant (submerged) unit weight of the soil ($\gamma_{\text{sat}} - \gamma_w$)
- γ_{sat} is the saturated unit weight of the soil
- ϕ' is the effective angle of internal friction of the soil
- β is the angle of inclination of the slope.

49.

Sol. Slope protection techniques include reinforcement methods like retaining walls, rock bolts, and soil nails; erosion control measures such as geotextiles, vegetation, and riprap; and drainage solutions that manage water runoff. Other techniques involve modifying the slope geometry, like terracing, or applying protective layers, such as shotcrete or mesh systems.

50.

Sol. A shallow foundation is a type of building foundation that transfers structural loads to the soil layers near the ground surface, typically at a depth that is less than or equal to its width. They are generally used when the soil close to the surface has sufficient load-bearing capacity to support the structure without excessive settlement.

51.

Sol. It is the net loading intensity at which neither the soil fails in shear nor there is excessive settlement detrimental to the structure.

52.

Sol. The three primary modes of shear failure in soil foundations are general shear failure, local shear failure, and punching shear failure.

53.

Sol. Local shear failure differs from general shear failure in that it occurs in looser or weaker soils, has an ill-defined failure surface that does not extend to the ground surface, and is characterized by significant settlement rather than sudden collapse. General shear failure, which occurs in dense or stiff soils, results in a well-defined failure surface that reaches the ground, leading to sudden collapse and soil bulging.

54.

Sol. The main corrections applied to observed Standard Penetration Test (SPT) values are for overburden pressure and dilatancy.

55.

Sol. Bearing capacity of footing on purely cohesive soil does not depend upon size of footing.

$$q_{\text{up}} = q_{\text{uf}} \text{ (for clay)}$$

∴ Ultimate bearing capacity of a 2m wide square footing = 180 kPa

56.

Sol. Piles can be classified by their function as end-bearing piles (transferring load to a strong layer like rock), friction piles (transmitting load through skin friction), compaction piles (increasing the density of loose soil), and tension piles (resisting uplift forces). Other types include batter piles (inclined piles for lateral support) and sheet piles (for retaining walls).

57.

Sol. Piles are primarily classified into three main types based on their method of installation: driven piles, bored piles (or cast-in-situ), and driven and cast-in-situ piles.

58.

Sol. Pile driving is the process of installing long, strong columns (piles) into the ground to create a deep foundation for structures. The primary methods for pile driving are impact driving, vibratory driving, the press-in method (jacking), and combined methods like jetting or augering with driving.

59

Sol. Fender Pile : It is an upright, free standing pile driven usually into the sea or river to protect the dock wall or wharf from impact of ships.

60.

Sol. The zone influence of a pile group is much deeper than of a single pile.

61.

Sol. It is a type of deep foundation in the shape of a hollow prismatic box which is built above the ground level and then sunk into the ground at the required depth as a single unit. It is a water tight chamber which is used for laying foundations under water like rivers, lakes etc.

62.

Sol. The well depth below the maximum scour level is called the grip length.

63.

Sol. **Auger boring** : Partially saturated sand, silt and medium to stiff cohesive soil.

Wash boring : All types of soil except hard soils or rock.

Percussion boring : Only method suitable for drilling boreholes in bouldery and gravelly strata.

Rotary boring : Useful if the soil is highly resistant to auguring or wash boring.

64.

Sol. Soil exploration techniques include open excavations (like test pits and trenches), boring methods (such as auger, wash, percussion, and rotary boring), and geophysical methods (including seismic refraction and electrical resistivity)

65.

Sol. Auger boring, also known as horizontal earth boring or jack and bore, is a trenchless construction method used to install pipes and conduits underground with minimal surface disruption. This technique is ideal for crossing beneath obstacles like roads, railways, and existing structures without open-cut excavation.

66.

Sol. Soil samplers are categorized into different types based on their design and the type of sample they collect, which can be either disturbed or undisturbed. Key types include open-tube samplers (like Shelby tubes) for undisturbed samples, split-spoon samplers for disturbed samples, and piston samplers for soft, saturated soils. Other samplers include rotary samplers and scraper bucket samplers.

67.

Sol. The spacing of boreholes is a crucial aspect of geotechnical site investigation, primarily determined by the type and size of the proposed structure and the variability of the underlying subsoil conditions.

68.

Sol. Disturbed soil samples have their natural structure and properties significantly altered during the collection process, while undisturbed soil samples are collected using specialized techniques to preserve their in-situ structure, moisture content, and density as closely as possible.

69.

Sol. **(a) Representative sample:** If only soil structure gets modified but moisture content & mineral content remains the same while sampling, the sample is called a representative sample.

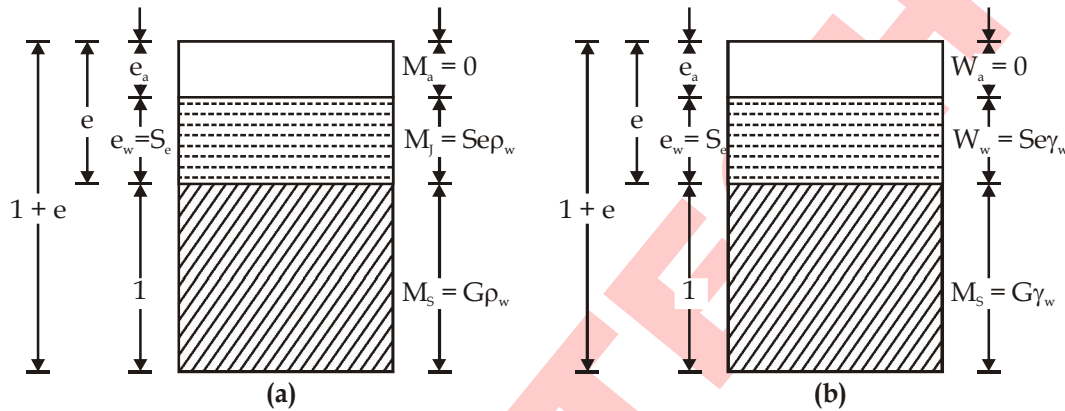
(b) Non-representative sample: If all the 3 i.e. soil structure, mineral content, moisture content get modified by sampling, samples are known as non-representative samples.

PART - B

5 Marks Questions

70.

Sol. If the area of cross-section of the soil mass is also taken unity, the volume of solids is also equal to the height of solids. Fig. shows the phase diagram with volume of solids V_s equal to unity. Since the void ratio is equal to the ratio of the volume of voids to the volume of solids, the volume of voids in Fig. becomes equal to e . The total volume (V) is obviously equal to $(1 + e)$. The volume of air is shown by e_a and the volume of water, by e_w .



Three-phase diagram in terms of voids ratio

71.

Sol. Height of solids method : In this method, equivalent height of solids is determined from the dry mass of the soil. The height of solids is given by

$$H_s = \frac{V_s}{A} = \left(\frac{M_s}{G \rho_w} \right) \cdot \frac{1}{A}$$

where

H_s = height of solids,

V_s = volume of solids,

M_s = dry mass of sample,

G = specific gravity of solids,

A = cross-sectional area of specimen.

From definition of void ratio,

$$e = \frac{\text{Volume of voids}}{\text{Volume of solids}} = \frac{V - V_s}{V_s} \dots (a)$$

Eq. (a) can be written as,

$$e = \frac{(A \times H) - (A \times H_s)}{(A \times H_s)} = \frac{H - H_s}{H_s}$$

where H is the total height (total thickness)

Thus, the voids ratio is determined from the total height (H) and the height of solids. The total thickness of the sample is measured at least once during the test, usually either before the start of the test or at the end of the test. At other stages of loading, the thickness H is worked out from the measured thickness and the difference in dial gauge readings.

Therefore,

$$H = H_0 \pm \Sigma \Delta H$$

where

H_0 = Initial height and

ΔH = Change in height

72.

Sol. Data given,

Bulk unit weight, $\gamma_t = 19 \text{ kN/m}^3$ Moisture content, $w = 15\%$ Specific gravity, $G = 2.7$

We know that,
$$\gamma_t = \frac{G + Se}{1 + e} \gamma_w = \frac{G + wG}{1 + e} \gamma_w$$

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

$$\Rightarrow e = 0.6031$$

Also,
$$S \cdot e = wG$$

$$\Rightarrow S \times 0.6031 = 0.15 \times 2.7$$

$$\Rightarrow S = 0.6715 = 67.15\%$$

Water content for 100% saturation (i.e., $S = 1$),

$$w = \frac{e}{G} = \frac{0.6031}{2.7} = 0.2234$$

$$= 22.34\%$$

Thus, additional water content required = $22.34 - 15 = 7.34\%$

73.

Sol. Given

$$\gamma_d = 18.5 \text{ kN/m}^3;$$

$$w = 15\%; G = 2.7$$

Assuming,

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

Bulk unit weight,
$$\gamma = \gamma_d(1 + w) = 18.5(1.15) = 21.275 \text{ kN/m}^3$$

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

$$18.5 = \frac{2.7 \times 10}{1 + e}$$

$$\therefore e = 0.4595$$

Saturated unit weight,
$$\gamma_{\text{sat}} = \left(\frac{G + e}{1 + e} \right) \gamma_w = \frac{2.7 + 0.4595}{1.4595} \times 10 = 21.6478 \text{ kN/m}^3$$

Submerged unit weight
$$\gamma' = \gamma_{\text{sat}} - \gamma_w = 21.6478 - 10 = 11.6478 \text{ kN/m}^3$$

74.

Sol. Liquidity Index (I_L)

- Liquidity index is defined as the ratio of the 'difference between natural water content (w)' of soil and the plastic limit (w_p)' to the plasticity index i.e.,

$$I_L = \frac{w - w_p}{I_p} = \frac{w - w_p}{w_L - w_p}$$

- This index indicates as to in what part of its plastic range, a given soil mass lies.
- The soil is softer in consistency if its natural moisture content is closer to liquid limit i.e., when

$$I_L = 1$$

- Similarly, the soil is stiff if $I_L = 0$

Activity Number (A_c)

- Activity number is the ratio of the plasticity index (I_p) to clay fraction (i.e., percentage by mass of particles finer than 2 microns)



Thus, $A_c = \frac{I_p}{\% \text{ of clay size particles (i.e. } < 2\mu) \text{ in soil}}$

- It signifies the swelling characteristics of clays. Higher activity number implies higher swelling.
- The soil containing the clay mineral montmorillonite have very high activity (>4). The soil containing the mineral Kaolinite are least active ($A < 1$), whereas the soils containing the mineral illite are moderately active ($A = 1$ to 2)
- Depending upon activity, the soils are classified into three types:

S.No.	Activity	Soil type
1.	$A < 0.75$	Inactive
2.	$A = 0.75$ to 1.25	Normal
3.	$A > 1.25$	Active

75.

Sol. **Thixotropy:**

- It is that property of soil due to which loss of strength (shear strength) on remoulding can be regained if left undisturbed for some time.
- Increase in strength with passage of time is due to tendency of clay soil to regain their chemical equilibrium with the reorientation of water molecule in adsorbed layer.
- Thixotropy of soils is of great practical importance in soil engineering. For example, if pile is driven into soil, it disturbs the soil. Frictional resistance of driven pile in clay soil immediately after installation will be much less than its value after a month.

Sensitivity (S_t):

- Sensitivity is defined as the ratio of the unconfined compressive strength of an undisturbed soil sample to the unconfined compressive strength of the specimen of the same soil after remoulding it at an unaltered moisture content.

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

- It helps in measuring the effect of remoulding of soil on its strength.

76.

Sol. Given :

$$w_L = 55\%$$

$$w_P = 25\%$$

$$w_s = 15\%$$

$$w = 30\%$$

Plasticity index, $I_p = w_L - w_P = (55 - 25)\% = 30\%$

Liquidity index, $I_L = \frac{w - w_P}{w_L - w_P} = \frac{(30 - 25)}{(55 - 25)} = \frac{5}{30} = 0.1667 = 16.67\%$

77.

Sol.

$$\rho_{\min} = \frac{M_{\min}}{V_m} = \frac{410}{290} = 1.414 \text{ gm / ml}$$

$$\rho_{\max} = \frac{M_{\max}}{V_m} = \frac{410}{215} = 1.907 \text{ gm / ml}$$

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

or

$$D_r = \frac{1.907}{1.816} \left(\frac{1.816 - 1.414}{1.907 - 1.414} \right) \times 100$$

or $D_r = 85.63\%$

78.

Sol.

Given,

Volume of soil, $V = 600,000 \text{ m}^3$

Void ratio of soil, $e = 0.75$

$$\begin{aligned} \text{Volume of solid, } V_s &= \frac{V}{1+e} \\ &= \frac{600000}{1+0.75} \\ &= 342857.14 \text{ m}^3 \end{aligned}$$

Borrow Pit A

Fill

Given,

In situ void ratio of soil, $e_A = 0.80$

Transportation cost = Rs. 10 m^3

Volume of solid, $V_s = 342857.14 \text{ m}^3$

$$\begin{aligned} \therefore \text{Volume of soil, } V_A &= V_s(1 + e_A) \\ &= 372857.14 \text{ (1.8)} \\ &= 617142.85 \text{ m}^3 \end{aligned}$$

Now, Total transportation cost = $617142.85 \times 10 = \text{Rs. } 6171428.5$

Borrow Pit B

Given,

In situ void ratio of soil, $e_B = 1.7$

Transportation cost = Rs. 5/ m^3

Volume of solid, $V_s = 342857.14 \text{ m}^3$

$$\begin{aligned} \therefore \text{Volume of soil, } V_B &= 342857.14 (1 + e_B) \\ &= 342857.14 \times 2.7 \\ &= 925714.278 \text{ m}^3 \end{aligned}$$

Now, Total transportation cost = $925714.278 \times 5 = \text{Rs. } 4628571.39$

Hence, soil borrowed from Pit B is more economical.

79.

Sol.

Hydrometer method is used to determine the particle size distribution of fine-grained soils passing 75 μm sieve. The hydrometer measures the specific gravity of the soil suspension at the centre of its bulb.

Limitations of stokes law :-

- Particles under going settlement are assumed to be spherical
- Medium in which settlement takes place is infinite
- Particles are assumed to be under going discrete settling
- Stokes law is valid for particles having size 0.2 mm to 0.2 μm

80.

Sol. **Sheep foot rollers :**

Sheep foot rollers are ideally suited for compaction of cohesive soils.

Cohesive clays are generally found deposited in chunks. There is no effective method of reducing the voids within a chunk - neither vibrations nor short duration loading can do this. All that can be done is to compact the different chunks together by reducing the space between the different chunks.

This can be satisfactorily done by sheep foot rollers.

These rollers have cylindrical projections protruding out of the steel drum surface. As a result, their penetration into soil and the contact pressure applied on soil is much higher. They impart a kneading type compactive effort to the entire depth of the layer to be compacted and also simultaneously break down lumps or clods present in the soil. They also induce an improvement in the uniformity of distribution of water in the soil layer with each pass of the roller.

Vibratory rollers :

Vibratory rollers are most effective for cohesion less soils and produce excellent densification of soils such as sand, gravel and relatively large stones.

As these materials are vibrated, the particles shift their position and nestle more closely with adjacent particles to increase the density of the mass.

Vibratory rollers are available as:

(a) Vibrating steel drum rollers

These are suitable for compacting granular soils, with no fines, in layers up to 1 m.

(b) Vibrating pneumatic tyred rollers

These are suitable for compacting granular soils having appreciable percentage of fines with thickness of layers reduced to about 30 cm.

81.

Sol. The most suitable equipment depends primarily on the soil type (cohesive vs. granular) and the required compaction method (vibration, kneading, pressure, or impact).

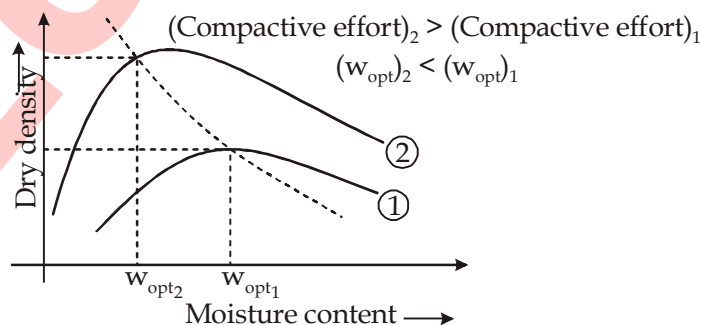
Equipment Suitability by Soil Type :

Compaction Equipment	Suitable Soil Types
(a) Sheepsfoot Rollers (Padfoot/Tamping)	Cohesive soils (heavy and silty clays) and fine-grained soils with more than 20% fines.
(b) Smooth-Wheeled Rollers	Granular soils (crushed rock, gravel, sand) and moderately cohesive soils.
(c) Vibratory Compactors (Rollers/Plates)	Cohesionless/granular soils (sand, gravel, crushed stone).
(d) Pneumatic-Tyred Rollers	Both cohesive and granular soils (versatile for mixed soils, silty sands, and clayey gravels).
(e) Rammers (Jumping Jacks)	All types of soil, including cohesive, in confined areas such as trenches, around foundations, and behind retaining walls.

82.

Sol. As water content at compaction is increased (from completely dry condition), the dry unit weight obtained increases. This trend continues upto a limiting water content beyond which further increase in water content results in a decrease in dry unit weight. This limiting water content is known as the optimum moisture content.

The phenomenon described is shown in the figure given below :



- Compaction at optimum moisture content produces the maximum dry unit weight possible for a given amount of compactive effort. The optimum moisture content is different for different compactive efforts. Typically, an increase in compactive effort reduces the optimum moisture content and produces an increase in the maximum dry unit weight to which a soil can be compacted. It is to be noted here that
- The typical inverted V shape of a compaction curve applies only to soil possessing some amount

of plasticity.

- In case of pure sandy soils, the maximum dry unit weight is obtained either when the soil is completely dry or at complete saturation condition.

83.

Sol. The factors affecting permeability of soil are as follows :

- Temperature** : As the viscosity of the pore fluid decrease with the temperature, permeability increases with temperature, as unit weight of pore fluid does not change much with change in temperature.
- Properties of Pore Fluid** : Pore fluids are fluids that occupy pore spaces in a soil or rock. Permeability is directly proportional to the unit weight of pore fluid and inversely proportional to viscosity of pore fluid.
- Shape of Particles** : Permeability is inversely proportional to square of specific surface area (i.e. with increase in specific surface area, permeability decreases).
- Degree of Saturation** : Permeability increases with increase of degree of saturation.
- Adsorbed Water** : Adsorbed water means a thin microscopic film of water surrounding individual soil grains. This water is not free to move and hence reduces the effective pore space and thus decreases coefficient of permeability.
- Grain Size** : The permeability varies approximately as the square of grain size. It depends on the effective diameter of the grain size (D_{10})

$$K = C (D_{10})^2$$
 where,
 K = Permeability in cm/sec
 C = Constant and generally lies between 100 to 150.
- Void Ratio** : Increase in the void ratio increase the area available for flow hence permeability increases.
- Foreign Impurities** : (Foreign Agents) clog the voids due to which permeability decreases.
- Structure of Soil Mass** : For same void ratio the permeability is more for flocculant structure as compared to the dispersed structure.

84.

Sol. Given :

Hydraulic conductivity, (K) = 15.24 m/day

Density, (ρ_w) = 998.2 kg/m³

Viscosity, (μ) = 1.002×10^{-3} kg/ms

$$K = 15.24 \text{ m/day} = \frac{15.24}{24 \times 60 \times 60} = 1.764 \times 10^{-4} \text{ m/s}^{-1}$$

We know that

$$\text{Intrinsic permeability, } (K_0) = K \frac{\mu}{\gamma_w} = k \frac{\mu}{\rho_w g}$$

$$\Rightarrow K_0 = \frac{1.764 \times 10^{-4} \left(\frac{\text{m}}{\text{s}} \right) \times (1.002 \times 10^{-3}) \left(\frac{\text{kg}}{\text{m-s}} \right)}{998.2 \left(\frac{\text{kg}}{\text{m}^3} \right) \times 9.81 \left(\frac{\text{m}}{\text{s}^2} \right)}$$

$$\Rightarrow K_0 = 18.05 \times 10^{-12} \text{ m}^2$$

$$\text{Also } 1 \text{ Darcy} = 9.87 \times 10^{-13} \text{ m}^2$$

$$\Rightarrow k_0 = \frac{18.05 \times 10^{-12}}{9.87 \times 10^{-13}} = 18.288 \text{ Darcy}$$

85.

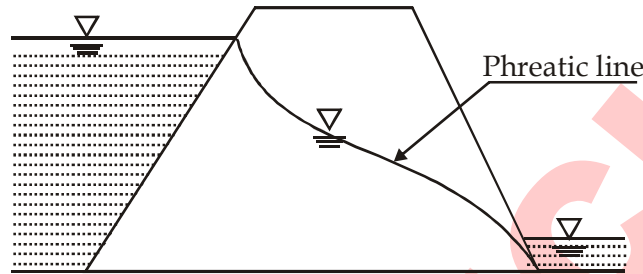
- Sol.** (i) **Aquifers** : (High porosity and high permeability) An aquifer is a saturated formation of earth material which not only stores water but yields it in appreciable quantity relatively easily due to its high permeability.

Deposits of sand and gravel form good aquifers.

- (ii) **Aquitard** : (High permeability and low porosity) It is a formation through which only seepage is possible and thus the yield is insignificant compared to an aquifer.
A sandy clay unit is an example of aquitard.
- (iii) **Aquiclude** : (High porosity and low permeability) These formations are highly porous but are not permeable due to very small size of pores.
Example:- Clay

86.

Sol. Phreatic line is the top most flow line in an earthen dam. Above phreatic line pressure is atmospheric and below phreatic line pressure is hydro static.



87.

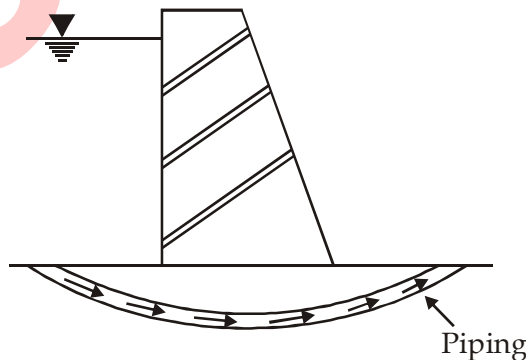
Sol.

$$\begin{aligned}\text{Head available, } h &= 30 - 5 = 25\text{m} \\ \text{Coefficient of permeability, } k &= 6 \times 10^{-4} \text{ cm/s} \\ &= 6 \times 10^{-6} \text{ m/s} \\ \text{No. of flow channels, } N_f &= 4 \\ \text{No. of potential drops, } N_d &= 10\end{aligned}$$

$$\begin{aligned}\text{Discharge per meter run of dam, } q &= k \times h \times \frac{N_f}{N_d} \times 1 \\ &= 6 \times 10^{-6} \times 25 \times \frac{4}{10} \times 1 \\ &= 6 \times 10^{-5} \text{ m}^3/\text{sec}/\text{m}\end{aligned}$$

88.

Sol. **Piping** : Often hydraulic structures like dam, weirs, barage etc. fail by formation of small pipe shaped channels in foundation which is called as piping failure. Such type of failure occurs when water flowing through the foundation carries very high hydraulic gradient which carries away the soil particles. It is form of quick sand condition.



89.

Sol. **Terzaghi's filter design approach:**

Use of filters is employed in preventing piping failures. They prevent erosion and reduces uplift pressure.

A rational approach to the design of filters has been given by Terzaghi.

It is based on the principle that a properly graded filter permit flow of water but prevent the movement of soil particles. It consists of layers of pervious material such that soil particles in a particular layer are coarser than that in the preceding layer

However, the difference of sizes of the particles in the two layers should not be excessive, otherwise the particles of the preceding layer will be carried into the next layer.

According to him, the following filter criteria should be satisfied.

$$(a) \quad \frac{D_{15}(\text{filter})}{D_{85}(\text{protected material})} < 4(\text{to } 5)$$

It ensures no significant passing of particles from the protected material to the filter.

$$(b) \quad 4(\text{to } 5) < \frac{D_{15}(\text{filter})}{D_{15}(\text{protected material})}$$

It ensures sufficient head is lost in filter without build up of seepage pressure.

90.

Sol. Pressure intensity (q) due to point load (Q)

$$Q = 25 \text{ kN}$$

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

$$= \frac{3 \times 25}{2\pi \times 6^2} \left[\frac{1}{1 + \left(\frac{4}{6}\right)^2} \right]^{5/2}$$

$$= 0.132 \text{ kN/m}^2$$

91.

Sol.

- When a soil mass is subjected to a compressive force, its volume decreases.
- The decrease in volume of soil under stress may be because of :
 - (i) Compression and expulsion of pore air (it is called compaction)
 - (ii) Compression and expulsion of pore water (it is called primary consolidation)
 - (iii) Gradual readjustment of clay particles into more stable configuration (it is called secondary consolidation).

S.No.	Compaction	Consolidation
1.	Instantaneous phenomenon.	Time dependent phenomenon.
2.	Soil is always partially saturated.	Soil is completely saturated.
3.	Densification is due to reduction in the volume of air voids at a given water content.	Volume reduction is due to expulsion of pore water from void.
4.	Specific compaction techniques are used.	Consolidation occurs on account of a static load placed on the soil.

92.

Sol.

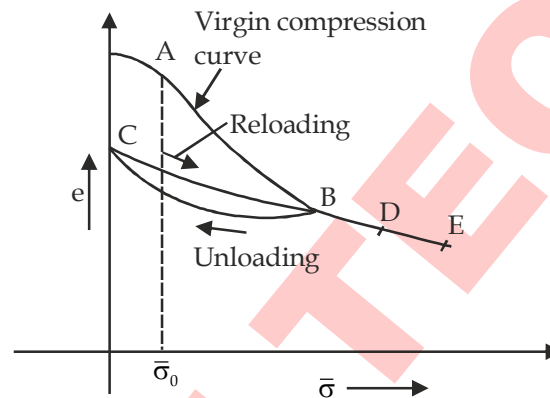
- The soil is homogeneous and isotropic.
- The soil is fully saturated.
- The solid particles and water in the voids are incompressible. The consolidation occurs due to expulsion of water from the voids.
- The coefficient of permeability of the soil has the same value at all points, and it remains constant

during the entire period of consolidation.

- Darcy's law is valid throughout the consolidation process.
- Soil is laterally confined, and the consolidation takes place only in axial direction. Drainage of water also occurs only in the vertical direction.
- The time lag in consolidation is due to entirely the low permeability of the soil.
- There is a unique relationship between the void ratio and the effective stress, and this relationship remains constant during the load increment. In other words, the coefficient of compressibility and the coefficient of volume change are constant.

93.

- Sol.**
- Pre consolidation stress ($\bar{\sigma}_0$) is the maximum value of stress that the soil has ever experienced in its history.
 - Soils tend to retain the effect of stress changes that have taken place in their geological history, in the form of their structure.



- When stresses in soil exceeds $\bar{\sigma}_0$, some kind of breakdown in the soil structure occurs, resulting in a much higher consolidation indicated by a steeper void ratio (e) v/s effective stress curve ($\bar{\sigma}_0$) as shown in the figure. Whereas, the initial portion of $e - \bar{\sigma}$ curve (when $\bar{\sigma} < \bar{\sigma}_0$) is relatively flatter and is called recompression curve.
- A knowledge of pre-consolidation pressure is significant in the sense that within the range of recompression, the change of void ratio is small and hence the settlement would be small. Thus, a soil can be loaded up to its pre consolidation pressure with little worry about settlement.

94.

Sol.

$$m_v = \frac{\frac{\Delta e}{1+e_0}}{\Delta \sigma} = \frac{a_v}{1+e_0}$$

$$= \frac{(1.1-0.9)}{2.1} / 125$$

$$= \frac{0.2}{2.1 \times 125}$$

$$= 7.619 \times 10^{-4} \text{ m}^2/\text{kN}$$

95.

- Sol.** Shear strength of soil is defined as its maximum resistance to shearing stresses before it fails or deforms by continuous sliding of its particles along internal surfaces. This resistance is primarily derived from three components: Cohesion, Internal Friction, Interlocking between particles.

Importance of Determining Shear Strength: Determining the shear strength of soil is crucial in geotechnical engineering because all stability analyses of ground-supported structures rely on this

property. Without this knowledge, the risk of structural failure increases significantly.

Key applications and importance include:

- Foundation Design
- Slope and Embankment Stability
- Design of Earth-Retaining Structures
- Pavement and Road Design
- Seismic Engineering and Environmental Planning

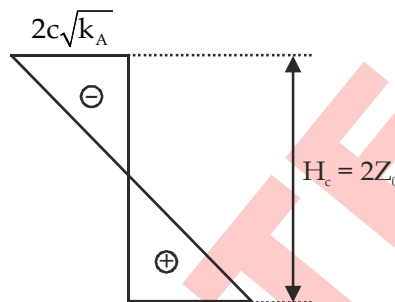
96.

Sol. For $c - \phi$ soils,

Active earth pressure at depth Z ,

$$p_A = K_A \gamma Z - 2c\sqrt{K_A}$$

The critical height of an unsupported cut (H_c) in $c - \phi$ soils is equal to $2Z_0$



Where,

$$Z_0 = \frac{2c\sqrt{K_A}}{\gamma K_A} = \frac{2c}{\gamma\sqrt{K_A}}$$

i.e.,

$$H_c = 2Z_0 = \frac{4c}{\gamma\sqrt{K_A}} \quad \dots(i)$$

As given in the equation:

$$H_c = 2.8 \text{ m}$$

$$\phi = 0 \quad (\because \text{soil is cohesive})$$

$\Rightarrow$

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

$$\text{Bulk unit weight, } \gamma = 17 \text{ kN/m}^3$$

$\Rightarrow$

$$2.8 = \frac{4c}{17\sqrt{1}}$$

$\Rightarrow$

$$c = 11.9 \text{ kN/m}^2$$

97.

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Key applications and importance include:

- Foundation Design
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- Design of Earth-Retaining Structures



Pavement and Road Design
Seismic Engineering and Environmental Planning

98.

Sol. The shear strength is determined using a direct shear test or a box shear test. It is better suited to cohesionless soils. The shear strength of soil is the resistance offered by the soil grain against shear deformation. Soil shear strength is determined by particle cohesion and the resistance of particles sliding over one other due to friction or interlocking. The cohesion and angle of internal friction of soil can be determined using a direct shear test. To calculate the shear strength of soil, Direct Shear Test is performed on a soil sample that is contained in a metal box with a square cross-section that is split horizontally at mid-height. A small gap exists between the two halves of the box. By moving the top half of the box relative to the bottom half, the soil is sheared along a predetermined plane. The box is typically square in plan, measuring 60 mm x 60 mm.

Advantages and Limitations:

Advantages:

- The apparatus is relatively simple and inexpensive.
- The test is simple and quick to perform, especially for cohesionless soils like sands and gravels, as they drain quickly.

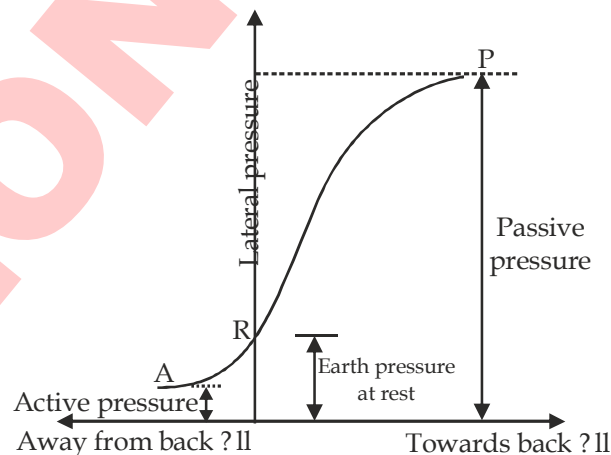
Limitations:

- The failure plane is forced to occur along a predetermined horizontal plane, which may not be the soil's weakest plane, potentially leading to an overestimation of shear strength.
- The stress distribution along the failure plane is not uniform (higher at the edges).
- Pore water pressure cannot be measured, making it difficult to control or monitor drainage conditions precisely, especially for cohesive soils where drained tests can take days.
- The state of stress is only known at failure.

99.

Sol. • A structure which is used to hold back a soil mass is called a retaining structure. In design of these retaining structures, it becomes imperative to know the magnitude and line of action of the lateral force exerted by the soil which is known as earth pressure.

• It has been found out that the value of lateral earth pressure depends upon the movement of retaining structure with respect to back fill soil as shown in figure below.



- Accordingly lateral earth pressure can be divided into three categories.

(a) Earth pressure at rest:

- 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 earth pressure at rest (represented by point R in the figure)

(b) Active earth pressure:

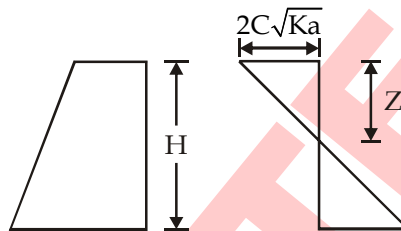
- When wall moves away from the soil, the block of soil has the tendency to move downwards and outwards. Hence, shear stress is mobilized on the soil
- Because of the nature of the shear stress developed in direction opposite to movement of wall, active earth pressure on wall will fall below the earth pressure at rest.
- The pressure developed on the wall when the soil is on the verge of failure is called active earth pressure.

(c) Passive earth pressure:

- When wall moves towards the soil, the block of soil has tendency to move up and inside. Hence, shear stress is mobilized on the soil.
- Because of the nature of the shear stress developed in direction opposite to movement of wall, pressure on the wall increases beyond the earth pressure at rest condition and this is called passive earth pressure.
- Full value of passive earth pressure is mobilized when the soil is on the verge of shear failure.

100.

Sol. Acc. to Rankine's earth pressure theory, for cohesive soils a negative pressure exists which tries to cause a pull on the wall. The value of this negative pressure is $p_a = -2C\sqrt{Ka}$



The pressure is negative in the top region.

As shown fig. the pressure is negative in the top region. It becomes zero at a depth Z_c . If the wall has a height of $2Z_c$, the total earth pressure is zero. This height is known as the critical height (H_c).

$$H_c = 2Z_c$$

If the height of an unsupported vertical cut is smaller than H_c , it should be able to stand.

101.

Sol. The assumptions made in Coulomb's wedge theory are as follows:

- The backfill soil is considered to be dry and cohesionless (primarily granular, like sand).
- The soil is assumed to be homogeneous and isotropic, meaning its properties (such as unit weight and angle of internal friction) are the same everywhere and in all directions.
- The backfill is assumed to be an ideally plastic material (elastically non-deformable but breakable at failure).
- The ground or backfill surface is a plane surface, which may be horizontal or inclined.
- The back of the wall is a plane surface and can be vertical or inclined (battered).
- The wall surface is considered rough, and the friction between the wall and the backfill (angle of wall friction, δ) is taken into account.
- The failure surface is a single, planar wedge that passes through the heel of the wall.
- The resultant earth pressure acts at an angle to the normal of the wall, equal to the wall friction angle (δ).

102.

Sol. (a) Factor of safety with respect to shear strength :

$$F_s = \frac{s}{\tau_m}$$

where,

F_s = Factor of safety with respect to shear strength

s = Shear strength

τ_m = Mobilised shear strength (equal to applied shear stress).

(b) **Factor of safety with respect to cohesion :** The factor of safety with respect to cohesion (F_c) in the

ratio of the available cohesion intercept (c) and the mobilised cohesion intercept.

Thus
$$F_c = \frac{c}{c_m}$$

where

c = Cohesion intercept

c_m = Mobilised cohesion intercept

F_c = Factor of safety with respect to cohesion

- (c) **Factor of safety with respect to friction** : The factor of safety with respect to friction is the ratio of the available frictional strength to the mobilised frictional strength. Thus

$$F_\phi = \frac{\bar{\sigma} \tan \phi}{\bar{\sigma} \tan \phi_m}$$

or

$$F_\phi = \frac{\tan \phi}{\tan \phi_m}$$

where

F_ϕ = Factor of safety with respect to friction

ϕ = Angle of shearing resistance,

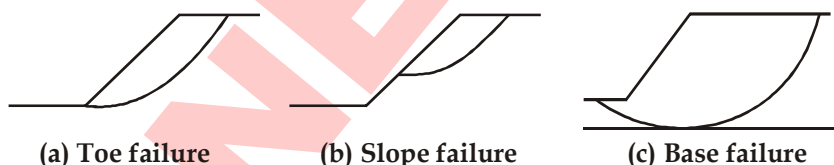
ϕ_m = Angle of mobilised shearing resistance.

103.

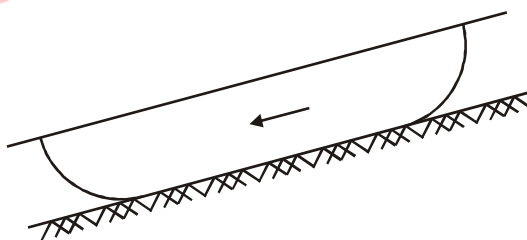
Sol. A slope may have any one of the following types of failures.

- (1) **Rotational failure** : This type of failure occurs by rotation along a slip surface by downward and outward movement of the soil mass. The slip surface is generally circular for homogeneous soil conditions and non-circular in case of non-homogeneous conditions. Rotational slips are further divided into 3 types.

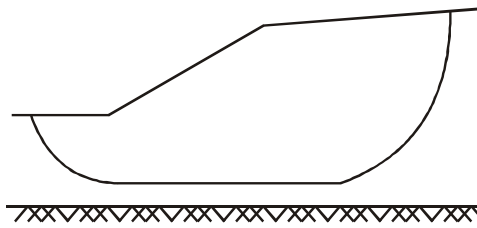
- **Toe failure**, in which the failure occurs along the surface that passes through the toe [Figure (a)].
- **Slope failure**, in which the failure occurs along a surface that intersects the slope above the toe [Figure (b)].
- **Base failure**, in which the failure surface passes below the toe [Figure (c)].



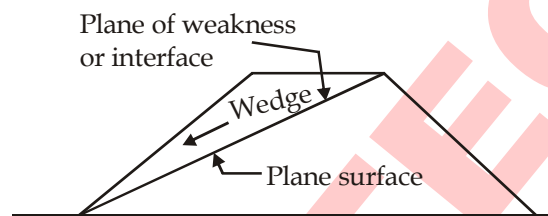
- (2) **Translational Failure** : Translational failure occurs in an infinite slope along a long failure parallel to the slope. The shape of the failure surface is influenced by the presence of any hard stratum at a shallow depth below the slope surface. Translational failures may also occur along slopes of layered materials.



- (3) **Compound Failure** : A compound failure is a combination of the rotational slips and the translational slip (Figure). A compound failure surface is curved at the two ends and plane in the middle portion. A compound failure generally occurs when a hard stratum exists at considerable depth below the toe.



- (4) **Wedge Failure** : A failure along an inclined plane is known as plane failure or wedge failure or block failure (Figure). It occurs when distinct blocks and wedges of the soil mass become separated. A plane failure is similar to translational failure in many respects. However, unlike translational failure which occurs in an infinite slope, a plane failure may occur even in a finite slope consisting of two different materials or in a homogeneous slope having cracks, fissures, joints or any other specific plane of weakness.
- (5) **Miscellaneous Failures** : In addition to above four types of failures, some complex types of failures in the form of spreads and flows may also occur.



104.

Sol. Foundation design requires considering a combination of geotechnical, structural, and environmental factors to ensure safety, stability, and durability.

Geotechnical Factors (Soil Conditions):

The ground conditions are paramount, as the soil must be able to support the imposed loads without excessive settlement or shear failure.

- Soil Type and Properties.
- Bearing Capacity
- Water Table Level
- Settlement Potential

Structural Factors (Building Requirements):

- Type and Magnitude of Loads
- Type of Structure
- Size and Layout

Environmental and External Factors:

- Seismic Activity
- Climate and Weather
- Adjacent Structures
- Building Codes and Regulations
- Cost and Constructability

105.

Sol. Factors affecting bearing capacity are as follows:

1. Nature of soil and its physical and engineering properties.
2. Nature of foundation and other details such as the size, shape, depth and rigidity of the structure.
3. Location of the ground water table relative to the foundation level.
4. The total and differential settlements that the structure can withstand without functional failure.
5. Initial stress condition of the soil due to prehistory or due to the existing structure near proposed foundation.
6. Nature and type of loading i.e. centric or eccentric.
7. Mode of shear failure i.e. General, Local or Punching shear failure.

106.

Sol. Classification of foundation

(a) As per Terzaghi basis

If $\left(\frac{D_f}{B}\right) \leq 1 \rightarrow$ The foundation is called shallow foundation.

If $1 < \left(\frac{D_f}{B}\right) \leq 15 \rightarrow$ The foundation is moderate deep foundation

If $\left(\frac{D_f}{B}\right) \geq 15 \rightarrow$ The foundation is deep foundation

(b) As per skempton's basis

If $\left(\frac{D_f}{B}\right) \leq 2.5 \rightarrow$ The foundation is shallow foundation

If $\frac{D_f}{B} > 2.5 \rightarrow$ The foundation is deep foundation

107.

Sol. Net ultimate bearing capacity :

• **Definition:** The maximum net pressure intensity that the soil at the foundation base can support before it fails in shear.

• **Calculation:** It's the difference between the ultimate bearing capacity (gross pressure at failure) and the pressure from the soil's own weight above the foundation's base.

• **Formula:**

$$q_{nu} = q_u - \gamma D_f$$

q_u = Ultimate bearing capacity (gross pressure)

γ = Unit weight of the soil

D_f = Depth of the foundation

Net safe bearing capacity :

• **Definition:** The maximum net pressure intensity that the soil can be subjected to without risk of shear failure. It is the net ultimate bearing capacity divided by a safety factor.

• **Purpose:** To provide a margin of safety against collapse.

• **Calculation:** It's determined by dividing the net ultimate bearing capacity by a pre-selected factor of safety to account for uncertainties.

• **Formula:**

$$q_{ns} = \frac{q_{nu}}{F}$$

q_{nu} = Net ultimate bearing capacity

F = Factor of safety against shear failure (typically 2.5 to 3.0)

108.

Sol. As per Terzaghi theory,

Ultimate bearing capacity for a square footing,

$$q_u = 1.3 C N_c + q N_q + 0.4 B \gamma N_\gamma$$

For purely cohesive soil,

$$N_c = 5.7, N_q = 1 \text{ and } N_\gamma = 0$$

But for local shear failure in place of 'C', we use C_m i.e.

$$C_m = \frac{2C}{3}$$

and in place of N_c and N_q we use N_c' and N_q'

Since, we are not given values of N_c' and N_q'

So, we assume $N_c' = N_c = 5.7$

and $N_q' = N_q = 1$

Thus,

$$q_u = 1.3 \times \left(\frac{2C}{3} \right) \times 5.7 + (\gamma \times 1.2) \times 1 + 0$$

$$q_u = 1.3 \times \left(\frac{2 \times 40}{3} \right) \times 5.7 + (18 \times 1.2) \times 1$$

$$q_u = 219.2 \text{ kN/m}^2$$

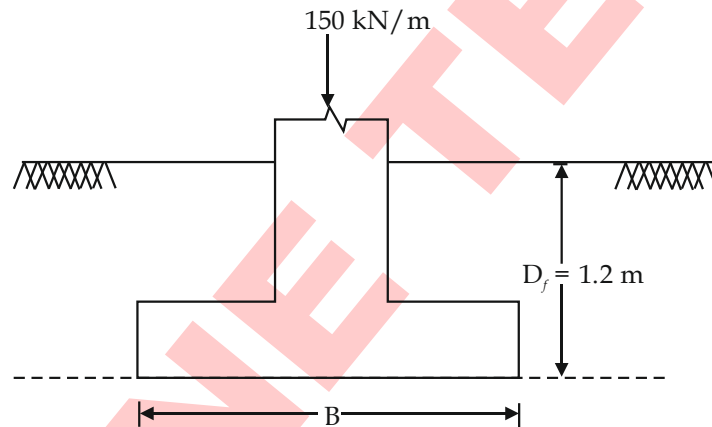
109.

Sol. Assumptions in Terzaghi's Bearing Capacity Theory :

- Depth of foundation is less than or equal to its width.
- Base of the footing is rough.
- Soil above bottom of foundation has no shear strength; is only a surcharge load against the overturning load.
- Surcharge upto the base of footing is considered.
- Load applied is vertical and non-eccentric.
- The soil is homogenous and isotropic.
- L/B ratio is infinite.

110.

Sol.



Given :

Load carried by the footing, $q = 150 \text{ kN/m}$ Depth of footing, $D_f = 1.2 \text{ m}$ Unit weight of soil, $\gamma = 18.8 \text{ kN/m}^3$ Unconfined compressive strength of soil, $q_u = 160 \text{ kN/m}^2$

Factor of safety, FOS = 3

Now, Cohesion of soil, $C = \frac{q_u}{2} = \frac{160}{2} = 80 \text{ kN/m}^2$

Net ultimate bearing capacity for cohesive soil, $q_{nu} = 5.7C$

Net safe bearing capacity of soil, $q_{ns} = \frac{q_{nu}}{\text{FOS}} = \frac{5.7C}{3}$

Assuming 150 kN/m to be the net external load acting on the footing.

$$\therefore \frac{150}{1 \times B} = \frac{5.7C}{3}$$

$$\Rightarrow \frac{150}{1 \times B} = \frac{5.7 \times 80}{3}$$

$$\Rightarrow B = 0.986 \text{ m}$$



111.

Sol.

$$B_1 = 600 \text{ mm} \\ = 0.6 \text{ m}$$

Settlement

$$S_1 = 15 \text{ mm}$$

$$B_2 = 1 \text{ m}$$

For cohesion less soils

$$\frac{S_2}{S_1} = \left[\frac{B_2 (B_1 + 0.3)}{B_1 (B_2 + 0.3)} \right]^2$$

$$\Rightarrow \frac{S_2}{15} = \left[\frac{1 (0.6 + 0.3)}{0.6 (1 + 0.3)} \right]^2$$

$$\Rightarrow S_2 = 19.97 \text{ mm}$$

112.

Sol.

Four-storey hospital building on erratic soil: An erratic soil mass is one in which the soil or soil strata are not uniform or consistent in properties, elevation, thickness, or extent.

Raft or mat foundation is provided when soil is sufficiently erratic.

Two-storey residential building of load-bearing walls resting on medium dense sand: Strip footing as preferred in this case.

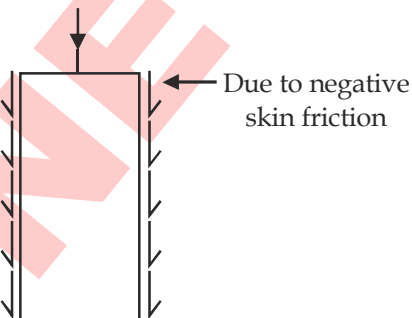
113.

Sol.

Negative skin friction

When the settlement in the surrounding soil is more than the settlement of the pile, it is called as the negative skin friction.

Due to friction, there is a downward drag on the pile and hence the pile load carrying capacity is reduced.



Cause of Negative skin friction

(i) Due to sudden draw down of the water table.

(ii) Due to, recent loose fill deposit

114.

Sol.

Pile load test conducted for determining the load carrying capacity of a pile. The test involves installing a test pile between two anchor piles and applying incremental loads through a hydraulic jack while monitoring settlement. Loads are applied until the pile reaches twice its safe load or a specified settlement. A load-settlement curve is plotted to determine the ultimate load and safe load based on settlement criteria. The test provides values for maximum load, permissible working load, and pile settlement under different loads.

115.

Sol.

The ratio of the ultimate load capacity of the pile group (Q_{ug}) to the sum of the individual load capacities of all the piles in the group, is called the group efficiency (η). It can be written as

$$\eta = \frac{Q_{ug}}{nQ_u}$$

Where n is the number of piles in the group and Q_u is the load capacity of one pile.

The group efficiency depends on the spacing between piles, type of soil in which the piles are installed and the manner of pile installation, that is, driven, bored, cast in situ, etc.

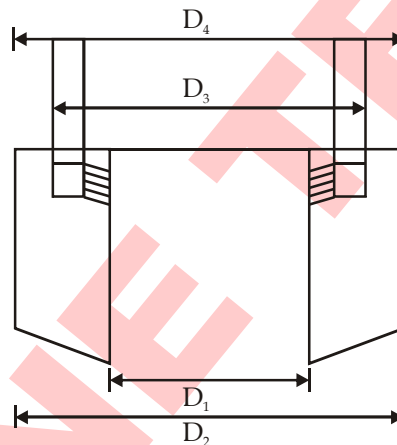
116.

Sol. A structure is never founded on a single pile. The bearing capacity of a pile group is not necessarily the capacity of the individual pile multiplied by the number of piles in the group. The phenomenon by virtue of which this difference occurs is known as 'group action of piles'. Pile group action is the behavior of a group of piles acting together to support a load, which differs from the sum of the individual piles capacities due to factors like overlapping stress zones and the pile cap connecting them. The efficiency of the group is determined by the ratio of the group's actual load capacity to the sum of the individual piles capacities, and it is affected by pile spacing, soil type, and pile geometry.

117.

Sol. (1) Area ratio $(A_r) = \left(\frac{D_2^2 - D_1^2}{D_1^2} \times 100 \right)$
 $< 20\%$ for stiff clay
 $< 10\%$ for sensitive clay

It is important for the disturbed and undisturbed sample.



(2) **Inside clearance**

$$C_i = \left(\frac{D_3 - D_1}{D_1} \right) \times 100 \quad (1 \text{ to } 3\%)$$

It is due to elastic expansion.

Higher the value of C_i higher the lateral expansion.

(3) **Design of non-returning valve :** In some situations, at strategic location, non-returning valve or check valve is employed to prevent or to reduce the water hammer pressure water column separation occurs, the installation of non-return valve downstream of the pocket could prevent flow reversal and subsequent over pressure.

PART - C

20 Marks Questions

118.

Sol. Given that

$$e = 0.7$$

$$s = 0.5$$

$$G_s = 2.7$$

Find η , γ_{bulk} , γ_{dry}

$$\text{So, Porosity } (n) = \frac{e}{e+1} = \frac{0.7}{1+0.7} = \frac{0.7}{1.7}$$

$$\boxed{n = 0.412}$$

$$\therefore Gw = Se$$

$$\Rightarrow 2.7 \times w = 0.5 \times 0.7$$

$$\Rightarrow \text{Water content} = w = 0.1296 \text{ or } w = 12.96\%$$

$$\gamma_{\text{bulk}} = \frac{G(1+w)}{1+e} \times \gamma_w = \frac{2.7(1+0.1296)}{1+0.7} \times 9.81$$

$$\therefore \boxed{\gamma_{\text{bulk}} = 17.6 \text{ kN/m}^3}$$

$$\gamma_{\text{dry}} = \left(\frac{G}{1+e} \right) \gamma_w = \left(\frac{2.7}{1+0.7} \right) \times 9.81$$

$$\boxed{\gamma_{\text{dry}} = 15.58 \text{ kN/m}^3}$$

For saturating the soil at current void ratio.

$$\Rightarrow S = 1$$

$$\therefore Gw = Se$$

$$\Rightarrow 2.7 \times w = 1 \times 0.7$$

$$w = 0.7/2.7$$

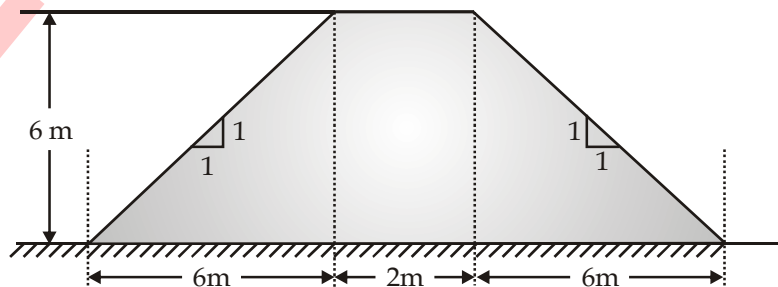
$$w = 0.2592$$

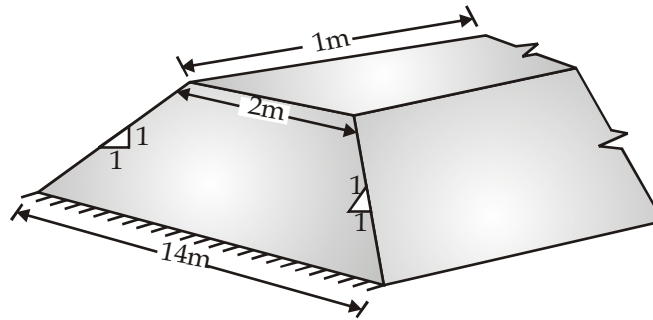
$$\text{So, } \Delta w = 0.2592 - 0.1296$$

 $\therefore$ Water content can be changed by 12.96% without changing dry density (γ_d).

119.

Sol.





We will use subscript 1 for borrow pit and 2 for embankment respectively.

Bulk unit weight of borrow pit soil,

$$\gamma_{t_1} = 18 \text{ kN/m}^3$$

Water content of borrow pit soil,

$$w_1 = 8\%$$

Dry density of embankment soil,

$$\gamma_{d_2} = 20 \text{ kN/m}^3$$

Water content of embankment soil,

$$w_2 = 10\%$$

Specific gravity of soil solids,

$$G_s = 2.7$$

Volume of embankment per unit of length,

$$= \frac{1}{2} \times (2 + 14) \times 6 \times 1 = 48 \text{ m}^3$$

Calculations for finding volume of solids (V_s) in 1 m length of embankment

$$\gamma_{d_2} = \frac{W_s}{V_{\text{soil}}} = \frac{V_{\text{solids}} \times G \times \gamma_w}{V_{\text{soil}}}$$

$$\Rightarrow V_{\text{solids}} (V_s) = \frac{\gamma_{d_2} \times V_{\text{soil}}}{G \times \gamma_w} = \frac{20 \times 48}{2.7 \times 9.81} = 36.244 \text{ m}^3$$

Calculations for finding quantity of earth to be excavated from borrow pit

Bulk unit weight of borrow pit soil,

$$\gamma_{t_1} = \frac{W_{\text{solids}} \times (1 + w)}{V_{\text{soil}}}$$

$$\Rightarrow \gamma_{t_1} = \frac{G \times \gamma_w \times V_{\text{solids}} \times (1 + w_1)}{V_{\text{soil}}}$$

$$\Rightarrow V_{\text{soil}} = \frac{2.7 \times 9.81 \times 36.244 \times (1 + 0.08)}{18} = 57.6 \text{ m}^3$$

Thus, 57.6 m³ of earth is required to be excavated from borrow pit area.

Weight of 57.6 m³ of borrow pit soil,

$$W_1 = 57.6 \times 18 = 1036.8 \text{ kN}$$

Number of truck loads required

$$= \frac{1036.8}{80} = 12.96$$

Thus 13 number of truck loads are required per metre length of embankment.

Calculations for porosity (n) and degree of saturation (S) of embankment soil

$$\gamma_{d_2} = \frac{G \times \gamma_w}{1 + e_2}$$

$$\Rightarrow 20 = \frac{2.7 \times 9.81}{1 + e_2}$$

$$\Rightarrow e_2 = 0.324$$

$$\text{Now, } n_2 = \frac{e_2}{1 + e_2} = \frac{0.324}{1 + 0.324} = 0.245$$

$$\text{Also, } eS = wG$$

$$\Rightarrow 0.324 \times S_2 = 0.1 \times 2.7$$

$$\Rightarrow S_2 = 0.8333 = 83.33\%$$

120.

Sol.

Mass of empty core cutter = 1071 gm

Mass of core cutter + Mass of soil = 2970 gm

Hence, Mass of soil = (2970 - 1071)gm = 1899 gm

 $\therefore$ Volume of core cutter = Volume of soil sample

$$V = \frac{\pi}{4} \times (10.2)^2 \times 12.6 \text{ cc}$$

$$V = 1029.58 \text{ cc}$$

$$\text{In-situ density, } \rho = \frac{M}{V} = \frac{1899}{1029.58} = 1.84 \text{ gm/cc}$$

$$\text{Dry density, } \rho_d = \frac{\rho}{1 + w} = \frac{1.84}{1 + 0.06} = 1.73 \text{ gm/cc}$$

$$\therefore \text{In-situ dry unit weight, } \gamma_d = \rho_d \cdot g = 16.97 \text{ kN/m}^3$$

$$\text{As we know that, } \rho_d = \left(\frac{G}{1 + e} \right) \rho_w$$

$$\text{Hence, } 1.73 = \left(\frac{2.69}{1 + e} \right) \times 1$$

$$\Rightarrow e = 0.55$$

$$\& \text{ Porosity, } n = \frac{e}{1 + e} = \frac{0.55}{1.55} = 0.3548 = 35.48\%$$

If embankment gets fully saturated and no volume change takes place, then 'e' remains same

$$S.e = w'.G$$

$$1 \times e = w'.G$$

$$\text{or } w' = \frac{0.55}{2.69} = 0.2045 = 20.45\%$$

$$\therefore \text{Increase in water content} = w' - w = (20.45 - 6)\% = 14.45\%$$

Now bulk density = Saturated density

$$\rho_{\text{sat}} = \left(\frac{G + e}{1 + e} \right) \rho_w = \frac{2.69 + 0.55}{1.55} \times 1 = 2.09 \text{ gm/cc}$$

& Bulk unit weight = saturated unit weight

$$\gamma_{\text{sat}} = \rho_{\text{sat}} \times g = 2.09 \times \frac{10^{-3}}{10^{-6}} \times 9.81 \times 10^{-3} \text{ kN/m}^3$$

$$\gamma_{\text{sat}} = 20.5029 \text{ kN/m}^3$$

121.

Sol. Given

$$\text{Liquid Limit } W_L = 60\%$$

$$\text{Plastic Limit } W_P = 36\%$$

$$\text{Plastic Index } I_P = W_L - W_P$$

$$\Rightarrow I_P = 60 - 36$$

$$I_P = 24\%$$

Plastic index as per A line

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

$$\Rightarrow I_P = 0.73(60 - 20)$$

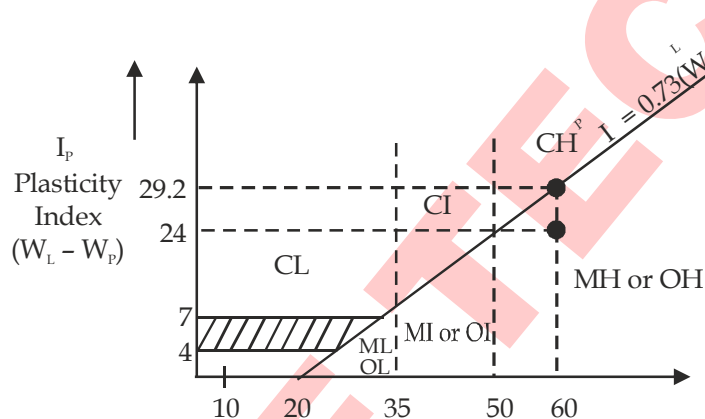
$$\Rightarrow I_P = 29.2\%$$

$$\Rightarrow I_P = 29.2\% > \text{Actual Plastic Index}$$

So, soil is silt.

Because $W_L > 50\%$, soil is highly compressible

So, given soil should be MH or OH



122.

Sol. Data Given :

$$\text{Specific gravity of dry soil, } G = 2.74$$

$$\text{Mass of dry soil, } W_s = 133.7 \text{ gm}$$

$$\text{Total volume of suspension, } V = 1000 \text{ cc}$$

$$\text{Dynamic viscosity, } \mu = 0.0102 \text{ poise}$$

(i) Density of Suspension immediately after it is prepared.

$$\rho_w = 1 \text{ gm/cc}$$

$$\text{Density of soil solids } (\rho_s) = \frac{\text{Mass of soil solids}}{\text{Volume of soil solids}} = \frac{W_s}{V_s}$$

$$\rho_s = G\rho_w$$

$$\rho_s = 2.74\rho_w = 2.74 \text{ gm/cc}$$

or

$$\rho_s = \frac{133.7}{V_s} = 2.74$$

Volume of solids in the suspension

$$V_s = 48.79 \text{ cc}$$

Volume of water in suspension,

$$V_w = V - V_s = 1000 - 48.79$$

$$V_w = 951.21 \text{ cc}$$

$$\text{Mass of water in suspension} = W_w = \rho_w \times V_w$$

$$W_w = 1 \times 951.21 \text{ gm}$$

$$W_w = 951.21 \text{ gm}$$

$$\text{Density of suspension} = \frac{W_s + W_w}{V} = \frac{133.7 + 951.21}{1000}$$

$$\rho_{\text{susp}} = 1.085 \text{ gm/cc}$$

(ii) Volume of Sample = 10 cm^3

$$H_s = 21 \text{ cm}$$

$$t = 2 \text{ min } 30 \text{ sec} = (2 \times 60 + 30) \text{ sec}$$

$$\mu = 0.0102 \text{ poise}$$

From stoke's law,

$$V_t = \frac{(\gamma_s - \gamma_w) \times D^2}{18\mu}$$

or

$$V_t = \frac{g(\rho_s - \rho_w) \times D^2}{18\mu}$$

or

$$D = \sqrt{\frac{18\mu}{g(\rho_s - \rho_w)}} \times V_t$$

Let D (in cm) be the diameter of the particles settled to a depth of 21 cm at $t = 2 \text{ min } 30 \text{ sec}$ with a uniform velocity V_t .

$$V_t = \frac{H_s}{t} = \frac{21}{(2 \times 60 + 30)} = 0.14 \text{ cm/sec}$$

Adopting,

$$g = 981 \text{ cm/sec}^2$$

$$D = \sqrt{\frac{18 \times 0.0102}{981 \times (2.74 - 1)}} \times 0.14$$

$$D = 0.00388 \text{ cm}$$

$$D = 0.0388 \text{ mm}$$

At time $t = 0$, weight of solid present in 1000 cc of suspension = 133.7 gm

Hence, weight of solid present in 10 cm^3 of suspension = 1.337 gm

At time $t = 2.5 \text{ min}$, weight of solids present in 10 cc of suspension = 0.406 gm

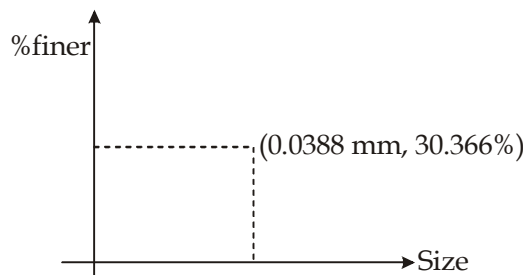
$$\% \text{ finer i.e. } \% N, \quad \% N = \frac{\text{Concentration at } H_s \text{ after time } t}{\text{Initial concentration}} \times 100$$

$$\Rightarrow \% N = \frac{0.406}{1.337} \times 100 = 30.366\%$$

Hence the co-ordinate of the required point on the particle size distribution curve are :

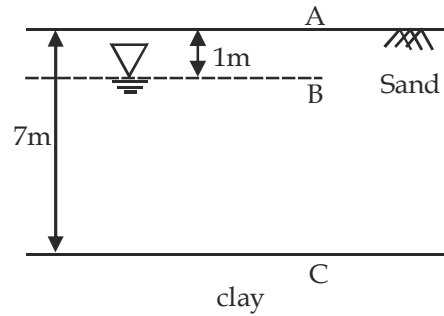
$$D = 0.0388 \text{ mm}$$

$$N = 30.366\%$$



123.

Sol. Given data,



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

$$\gamma_d = 17 \text{ kN/m}^3$$

Stress at A,

$$\sigma_A = \gamma_{\text{sat}} \times Z = 0$$

and,

$$u_A = 1 \times 9.81 = -9.81 \text{ kN/m}^2$$

and,

$$\begin{aligned}\bar{\sigma}_A &= \sigma_A - u_A = 0 - (-9.81) \\ &= 9.81 \text{ kN/m}^2\end{aligned}$$

Stress at B,

$$\sigma_B = \gamma_{\text{sat}} \times Z$$

 $\Rightarrow$

$$\sigma_B = 1 \times 20 = 20 \text{ kN/m}^2$$

and,

$$\bar{\sigma}_B = \sigma_B - u_B = 20 - 0 = 20 \text{ kN/m}^2$$

Stress at C

$$\sigma_C = \gamma_{\text{sat}} \times Z$$

 $\Rightarrow$

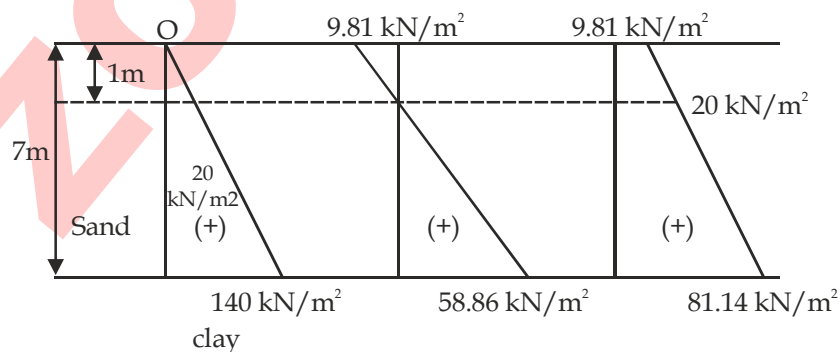
$$\sigma_C = 7 \times 20 = 140 \text{ kN/m}^2$$

and,

$$u_C = 6 \times 9.81 = 58.86 \text{ kN/m}^2$$

and,

$$\begin{aligned}\bar{\sigma}_C &= \sigma_C - u_c \\ &= 140 - 58.86 = 81.14 \text{ kN/m}^2\end{aligned}$$



124.

Sol. Bulk density, $\rho = \left(\frac{G + Se}{1 + e} \right) \rho_w$

or $\rho = \left(\frac{2.65 + 0.45 \times 0.70}{1 + 0.70} \right) \times 1000 = 1744.12 \text{ kg/m}^3$

or $\gamma = 1744.12 \times 9.81 \times 10^{-3} = 17.11 \text{ kN/m}^3$

For saturated soils, $S = 1.0$

and

$$\rho = \left(\frac{2.65 + 0.70}{1 + 0.70} \right) \times 1000 = 1970.59 \text{ kg/m}^3$$

or $\gamma = 1970.59 \times 9.81 \times 10^{-3} = 19.33 \text{ kN/m}^3$

Fig. shows the soil profile. The stresses at section A-A, B-B and C-C are as under :

Section A-A :

$$\sigma = 0$$

$$u = 0$$

$$\bar{\sigma} = 0$$

Section B-B :

$$\sigma = 17.11 \times 3 = 51.33 \text{ kN/m}^2$$

$$u = 0$$

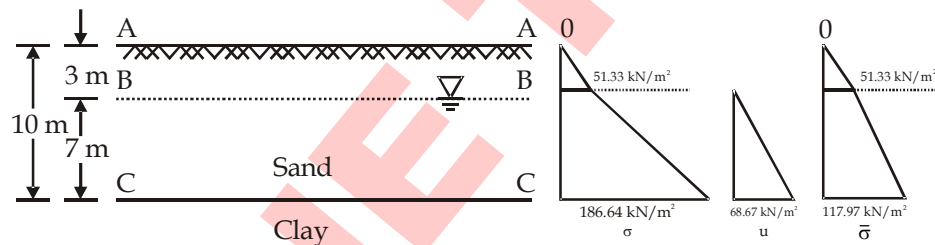
$$\bar{\sigma} = 51.33 \text{ kN/m}^2$$

Section C-C

$$\sigma = 17.11 \times 3 + 19.33 \times 7 = 186.64 \text{ kN/m}^2$$

$$u = 7 \times 9.81 = 68.67 \text{ kN/m}^2$$

$$\bar{\sigma} = 186.64 - 68.67 = 117.97 \text{ kN/m}^2$$



125.

Sol. Given data :

Thickness of sand layer = 10 m

Thickness of clay layer = 6 m

$$\begin{aligned} \rho_{\text{sat(sand)}} &= 2.1 \text{ g/cm}^3 \\ \gamma_{\text{sat(sand)}} &= 20.6 \text{ kN/m}^3 \\ \rho_{\text{sat(clay)}} &= 1.8 \text{ g/cm}^3 \\ \gamma_{\text{sat(clay)}} &= 17.66 \text{ kN/m}^3 \end{aligned}$$

Dry density of sand

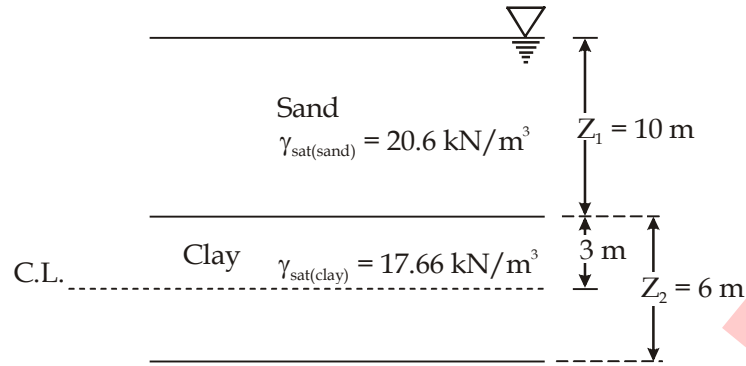
$$\begin{aligned} \rho_{\text{dry}} &= 1.7 \text{ g/cm}^3 \\ \gamma_{\text{dry}} &= 16.68 \text{ kN/m}^3 \end{aligned}$$

Degree of saturation of partially saturated soil

$$s = 20\%$$

Increase in the effective stress due to lowering in water table

(i) Initial position of water table :



Total stress at center line of clay ($z = 13 \text{ m}$)

$$\sigma = \gamma_{\text{sat(sand)}} \times Z_1 + \gamma_{\text{sat(clay)}} \times \frac{Z_2}{2}$$

$$\sigma = 20.6 \times 10 + 17.66 \times \frac{6}{2}$$

$$\sigma = 258.98 \text{ kN/m}^2$$

Pore water pressure at center line of clay ($Z = 13 \text{ m}$)

$$u = \gamma_w \left(Z_1 + \frac{Z_2}{2} \right) = 9.81 \left(10 + \frac{6}{2} \right)$$

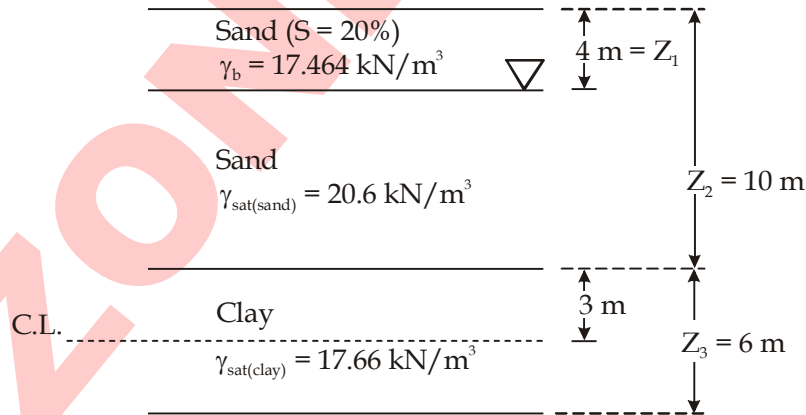
$$u = 127.53 \text{ kN/m}^2$$

Effective stress at center line of clay ($Z = 13 \text{ m}$)

$$\bar{\sigma} = \sigma - u = 258.98 - 127.53$$

$$\bar{\sigma} = 131.45 \text{ kN/m}^2$$

(ii) Final position of water table



$$\gamma_{\text{dry}} = \frac{G\gamma_w}{1+e} \quad \dots(i)$$

$$\gamma_{\text{sat}} = \frac{(G+e)\gamma_w}{1+e} = \frac{G\gamma_w}{1+e} + \frac{e\gamma_w}{1+e} \quad \dots(ii)$$

Substitute (i) and (ii)

$$\gamma_{\text{sat}} = \gamma_{\text{dry}} + \frac{e\gamma_w}{1+e} \quad \dots(iii)$$

Bulk density, $\gamma_b = \frac{(G + Se)\gamma_w}{1 + e}$

$$\gamma_b = \frac{G\gamma_w}{1 + e} + \frac{Se\gamma_w}{1 + e} \quad \dots(iv)$$

Substitute (i) and (iii) in (iv)

$$\begin{aligned}\gamma_b &= \gamma_{dry} + S \times (\gamma_{sat} - \gamma_{dry}) \\ \gamma_b &= \gamma_{dry} + S \times (\gamma_{sat(sand)} - \gamma_{dry}) \\ \gamma_b &= 16.68 + 0.2 \times (20.6 - 16.68) \\ \gamma_b &= 17.464 \text{ kN/m}^3\end{aligned}$$

Total stress at center of clay ($z = 13 \text{ m}$)

$$\sigma = \gamma_b \times Z_1 + \gamma_{sat(sand)}(Z_2 - Z_1) + \gamma_{sat(clay)} \times \frac{Z_3}{2}$$

$$\sigma = 17.464 \times 4 + 20.6 \times (10 - 4) + 17.66 \times \frac{6}{2}$$

$$\sigma = 246.436 \text{ kN/m}^2$$

Pore water pressure at center line of clay ($Z = 13 \text{ m}$)

$$u = \gamma_w(Z_2 - Z_1) + \gamma_w \frac{Z_3}{2}$$

$$u = 9.81 \times (10 - 4) + 9.81 \times \frac{6}{2}$$

$$u = 88.29 \text{ kN/m}^2$$

Effective stress at center line of clay ($Z = 13 \text{ m}$)

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

$$\bar{\sigma} = 246.436 - 88.29 = 158.146 \text{ kN/m}^2$$

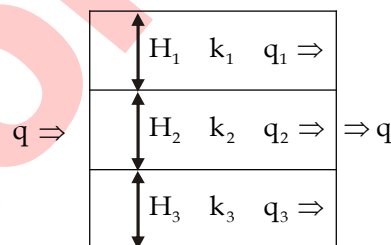
Increase in the effective stress due to lowering of water table

$$\Delta\bar{\sigma} = 158.146 - 131.45$$

$$\Delta\bar{\sigma} = 26.696 \text{ kN/m}^2$$

126.

Sol. For calculation of equivalent permeability parallel to bedding plane,



Head loss per unit length will be constant for each layer.

\$\Rightarrow\$ Total discharge = Summation of discharge through each layer.

$$\Rightarrow i_1 = i_2 = i_3 = i$$

$$\text{and } q_1 + q_2 + q_3 = q = k_H A i = k_H [H_1 + H_2 + H_3] \times 1 i$$

$$q_1 = k_1 i A_1 = k_1 i (H_1 \times 1)$$

$$q_2 = k_2 i A_2 = k_2 i (H_2 \times 1)$$

$$q_3 = k_3 i A_3 = k_3 i (H_3 \times 1)$$

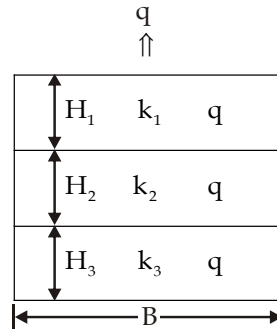
$$\Rightarrow k_H (H_1 + H_2 + H_3) i = k_1 H_1 i + k_2 H_2 i + k_3 H_3 i$$

$$\Rightarrow k_H = \frac{k_1 H_1 + k_2 H_2 + k_3 H_3}{H_1 + H_2 + H_3}$$

Hence, Equivalent horizontal permeability i.e. k_H is

$$k_H = \frac{\sum_{i=1}^n k_i H_i}{\sum_{i=1}^n H_i}$$

Calculation of equivalent permeability perpendicular to bedding plane,



Area ($B \times 1$) and discharge (q) through each layer will be constant,

$$q = q_1 = q_2 = q_3$$

$$h = h_1 + h_2 + h_3$$

...(i)

where, h is representing head loss

$$q = k_v i A = k_v \frac{h}{(H_1 + H_2 + H_3)} (B \times 1)$$

$$q = k_v \frac{h}{\sum H_i} B$$

$\therefore$

$$h = \frac{q \sum H_i}{k_v B}$$

...(ii)

$$q_1 = q = k_1 i_1 A_1$$

$$q_1 = q = k_1 \frac{h_1}{H_1} (B \times 1)$$

$\therefore$

$$h_1 = \frac{q H_1}{k_1 B}$$

...(iii)

Similarly,

$$h_2 = \frac{q H_2}{k_2 B}$$

...(iv)

$$h_3 = \frac{q H_3}{k_3 B}$$

...(v)

Using equation (ii), (iii), (iv) and (v) in equation (i)

$$\frac{q \sum H_i}{k_v B} = \frac{q H_1}{k_1 B} + \frac{q H_2}{k_2 B} + \frac{q H_3}{k_3 B}$$

Hence, Equivalent vertical permeability i.e. k_v is

$$k_v = \frac{\sum_{i=1}^n H_i}{\sum_{i=1}^n \frac{H_i}{k_i}}$$



127.

Sol.

$$D = 100 \text{ mm}$$

$$L = 120 \text{ mm}$$

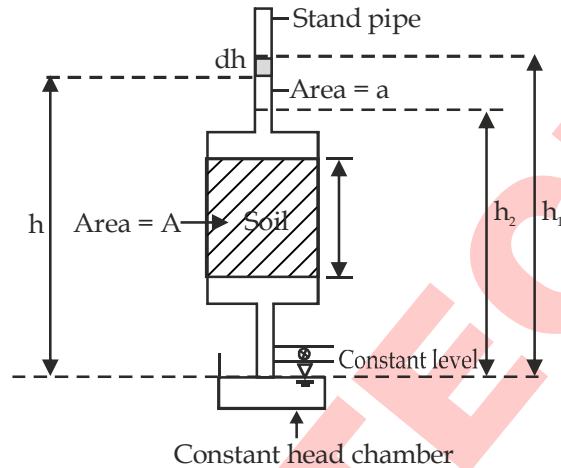
$$h_1 = 750 \text{ mm}$$

$$h_2 = 250 \text{ mm}$$

$$t = 1 \text{ hour}$$

$$= 3600 \text{ sec}$$

$$d = 6 \text{ mm}$$



$$\begin{aligned} \text{Area of standpipe, } a &= \frac{\pi}{4} \times (6 \times 10^{-3})^2 \\ &= 2.827 \times 10^{-5} \text{ m}^2 \end{aligned}$$

$$\begin{aligned} \text{Area of sample, } A &= \frac{\pi}{4} \times (100 \times 10^{-3})^2 \\ &= 7.854 \times 10^{-3} \text{ m}^2 \end{aligned}$$

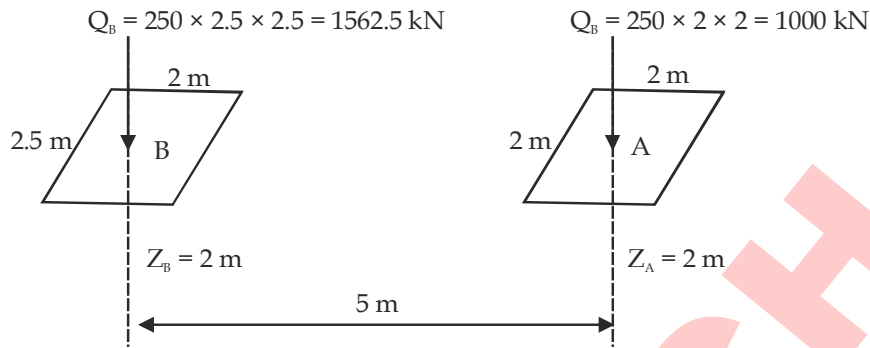
$$\begin{aligned} \text{Coefficient of permeability, } K &= \frac{2.303aL}{At} \log_{10} \left(\frac{h_1}{h_2} \right) \\ &= \frac{2.303 \times 2.827 \times 10^{-5} \times 120 \times 10^{-3}}{7.854 \times 10^{-3} \times 3600} \log_{10} \left(\frac{750}{250} \right) \\ &= 1.32 \times 10^{-7} \text{ m / sec} \\ &= 0.475 \text{ mm / hr} \end{aligned}$$

128.

Sol. Given data

$$q = 250 \text{ kPa}$$

$$Z = 2 \text{ m}$$



(i) Vertical stress below footing (A) as per Boussinesq equation

$$\sigma_A = \frac{3}{2\pi} \frac{Q_A}{Z^2} \left[\frac{1}{1 + \left(\frac{r_A}{Z}\right)^2} \right]^{5/2} + \frac{3Q_B}{2\pi Z^2} \left[\frac{1}{1 + \left(\frac{r_B}{Z}\right)^2} \right]^{5/2}$$

Here, $Q_A = 1000 \text{ kN}$, $Q_B = 1562.5 \text{ kN}$, $r_A = 0$, $r_B = 5 \text{ m}$, $Z = 2 \text{ m}$

$$\text{So, } \sigma_A = \frac{3 \times 1000}{2 \times \pi \times 2^2} \left[\frac{1}{1 + \left(\frac{0}{2}\right)^2} \right]^{5/2} + \frac{3 \times 1562.5}{2 \times \pi \times 2^2} \left[\frac{1}{1 + \left(\frac{5}{2}\right)^2} \right]^{5/2}$$

$$\Rightarrow \sigma_A = 119.366 + 1.318$$

$$\Rightarrow \sigma_A = 120.64 \text{ kN/m}^2$$

(ii) Vertical Stress below footing B:

$$\sigma_B = \frac{3Q_A}{2\pi \times Z^2} \left[\frac{1}{1 + \left(\frac{r_A}{Z}\right)^2} \right]^{5/2} + \frac{3Q_B}{2\pi \times Z^2} \left[\frac{1}{1 + \left(\frac{r_B}{Z}\right)^2} \right]^{5/2}$$

Here, $Q_A = 1000 \text{ kN}$, $Q_B = 1562.5 \text{ kN}$, $r_A = 5 \text{ m}$, $r_B = 0$, $Z = 2 \text{ m}$

$$\sigma_B = \frac{3 \times 1000}{2 \times \pi \times 2^2} \left[\frac{1}{1 + \left(\frac{5}{2}\right)^2} \right]^{5/2} + \frac{3 \times 1562.5}{2 \times \pi \times 2^2} \left[\frac{1}{1 + \left(\frac{0}{2}\right)^2} \right]^{5/2}$$

$$\sigma_B = 0.8434 + 186.510$$

$$\sigma_B = 187.35 \text{ kN/m}^2$$

(iii) Stress at mid point between the footing:

 $Q_A = 1000 \text{ kN}$, $Q_B = 1562.5 \text{ kN}$, $r_A = 2.5 \text{ m}$, $r_B = 2.5 \text{ m}$, $Z = 2 \text{ m}$

$$\text{So, } \sigma_C = \frac{3 \times 1000}{2\pi \times \pi \times 2^2} \left[\frac{1}{1 + \left(\frac{2.5}{2}\right)^2} \right]^{5/2} + \frac{3 \times 1562.5}{2 \times \pi \times 2^2} \left[\frac{1}{1 + \left(\frac{2.5}{2}\right)^2} \right]^{5/2}$$

$$= 11.356 + 17.744$$

$$\sigma_C = 29.10 \text{ kN/m}^2$$

129.

Sol. (i) **Initial compression:**

It the soil is initially partially saturated, expulsion of air as well as compression of pore air may take place with the application of external loads which is called initial compression. It is an immediate phenomenon. After the initial compression, soil reaches into fully saturated state, Initial compression can also occur if significant lateral strain takes place. This is due to deformation of soil under undrained condition.

The result of initial compression is the immediate settlement which is usually determined by using elastic theory, even though the deformation itself is not truly elastic.

(ii) **Primary consolidation**

After initial consolidation, further reduction in volume occurs due to expulsion of water from voids. When a saturated soil is subjected to a pressure, initially all the applied pressure is taken up by water as an excess pore water pressure, as water is almost incompressible as compared with solid particles. A hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs. The decrease depends upon the permeability of the soil and is therefore, time dependent. This reduction in volume is called primary consolidation,

In fine-grained soils, the primary consolidation occurs over a long time. On the other hand, in coarse grained soils, the primary consolidation occurs rather quickly due to high permeability.

(iii) **Secondary consolidation**

Experimentally, it has been shown that compression of soil layer do not cease when excess pore water pressure is fully dissipated and the primary consolidation is complete. This additional reduction in volume is called secondary consolidation. It continues at a gradually decreasing rate under constant effective stress.

Secondary consolidation is thought to be due to gradual readjustment of clay particles into a more stable configuration following the structural disturbance caused by the decrease in void ratio.

(iv) **Primary compression ratio**

Total settlement of soil (S_T) = Immediate settlement (S_i) + primary consolidation (S_1) + Secondary consolidation (S_2)

Primary compression ratio is the ratio of primary consolidation to total settlement. It is denoted by r_p

$$r_p = \frac{S_1}{S_T} = \frac{R_0 - R_{100}}{R_i - R_f}$$

Similarly, initial compression ratio, $r_0 = \frac{R_i - R_0}{R_i - R_f}$

and secondary compression ratio, $r_s = 1 - (r_0 + r_p)$

Where,

R_i = initial dial gauge reading

R_0 = dial gauge reading corresponding to beginning of 1⁰ consolidation.

R_{100} = dial gauge reading corresponding to completion of 1⁰ - consolidation. It is calculated from curve fitting technique of Taylor's method.

R_f = final dial gauge reading.

130.

Sol.

$$H = 25 \text{ cm}$$

$$e_1 = 0.92$$

$$e_2 = 0.78$$

$$\sigma_1 = 60 \text{ kN/m}^2$$

$$\sigma_2 = 120 \text{ kN/m}^2$$

Coefficient of compressibility

$$a_v = \frac{-\Delta e}{\Delta \sigma} = \frac{0.92 - 0.78}{120 - 60}$$

$$a_v = 2.333 \times 10^{-3} \text{ m}^2 / \text{kN}$$

Coefficient of volume change

$$m_v = \frac{a_v}{1 + e_0} = \frac{2.333 \times 10^{-3}}{1 + 0.92}$$

$$m_v = 1.215 \times 10^{-3} \text{ m}^2 / \text{kN}$$

$$\text{at, } t = 5 \text{ min} = 5 \times 60 = 300 \text{ sec}$$

$$U = 50\%$$

$$T_v = \frac{\pi}{4} \times 0.5^2 = 0.196$$

Due to double drainage

$$\text{Max. drainage path, } (H) = 12.5 \text{ mm} = 0.0125 \text{ m}$$

So,

$$T_v = \frac{c_v t}{(H)^2}$$

$\Rightarrow$

$$C_v = \frac{0.196 \times (0.0125)^2}{300}$$

$$C_v = 1.021 \times 10^{-7} \text{ m}^2 / \text{sec}$$

$\therefore$

$$K = C_v m_v \gamma_w$$

$\Rightarrow$

$$K = 1.021 \times 10^{-7} \times 1.2153 \times 10^{-3} \times 9.81$$

$$= 12.17 \times 10^{-10} \text{ m} / \text{s}$$

$$= 1.217 \times 10^{-7} \text{ cm} / \text{s}$$

Final thickness calculation

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

$$\Rightarrow \frac{0.92 - 0.78}{1 + 0.92} = \frac{\Delta H}{25}$$

$\Rightarrow$

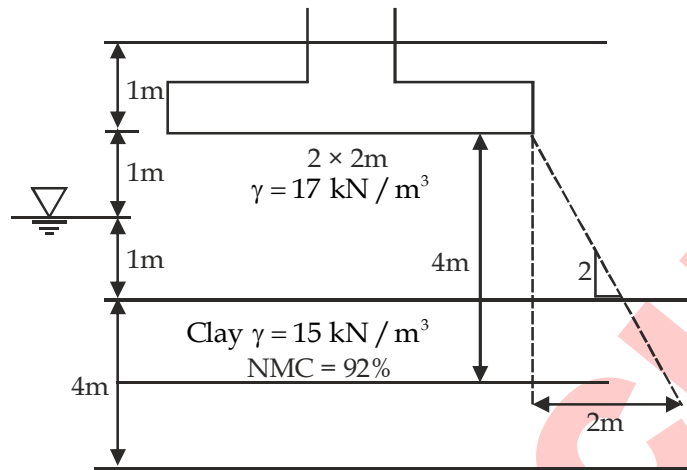
$$\Delta H = 1.823 \text{ mm}$$

$$\text{Final thickness} = (25 - 1.823) \text{ mm}$$

$$= 23.177 \text{ mm}$$

131.

Sol. Given data:



$$\gamma = \left(\frac{G + e}{1 + e} \right) \gamma_w = \frac{G(1 + w)}{1 + Gw} \times \gamma_w$$

$$\Rightarrow 15 = \frac{G(1 + 0.92)}{1 + G \times 0.92} \times 9.81$$

$$15 + 13.8G = 18.835G$$

$$G = 2.979$$

$$\text{Void ratio} = Gw = 2.979 \times 0.92$$

$$\Rightarrow \text{Void ratio} = Gw = 2.979 \times 0.92$$

$$\Rightarrow e = 2.74$$

Initial effective stress at middle of clay layer

$$\bar{\sigma} = 2 \times 17 + 1 \times (17 - 9.81) + 2 \times (15 - 9.81)$$

$$\bar{\sigma} = 51.57 \text{ kN/m}^2$$

Stress increase at middle of clay layer

$$\Delta \sigma = \frac{400}{\text{Area}} = \frac{400}{6 \times 6} = 11.11 \text{ kN/m}^2$$

$$\Rightarrow C_c = 1.05$$

$$H = 400 \text{ cm}$$

Consolidation given as

$$\Delta H = \frac{HC_c}{1 + e_0} \log_{10} \left(\frac{\bar{\sigma} + \Delta \sigma}{\bar{\sigma}} \right)$$

Put value

$$\Delta H = \frac{400 \times 1.05}{1 + 2.74} \log_{10} \left(\frac{51.57 + 11.11}{51.57} \right)$$

$$\Rightarrow \Delta H = 9.515 \text{ cm} = 95.15 \text{ mm}$$

Permissible settlements in shallow foundation for isolated footing on sand = 40 mm < ΔH

So, settlement is not within permissible limits.

132.

Sol.

Thickness of clay layer = $H = 8$ m

Drainage condition is double drainage, Hence

$$\text{Drainage path} = d = \frac{H}{2} = 4 \text{ m}$$

$$\text{Ultimate settlement} = \Delta H = 248 \text{ mm}$$

$$\text{Settlement in 4 years} = (\Delta h)_1 = 99.2 \text{ mm}$$

(a) Hence degree of consolidation in 4 years i.e. U_1

$$U_1 = \frac{(\Delta h)_1}{\Delta H} \times 100$$

$$U_1 = \frac{99.2}{248} \times 100$$

$$U_1 = 40\%$$

(b) Since

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

where,

 T_v = Time factor corresponding to 'U' t_u = Time required to achieve 'U' degree of consolidation d = Drainage path = H (for 1 way drainage)

$$= \frac{H}{2} \text{ (for 2 way drainage)}$$

Now for

 C_v = Coefficient of consolidation

$$U = 40\%$$

$$T_{40} = \frac{C_v t_{40}}{d^2}$$

$$\{t_{40} = 4 \text{ years}, d = 4 \text{ m}\}$$

$$\frac{\pi \left(\frac{40}{100} \right)^2}{4} = \frac{C_v \times 4}{(4)^2}$$

$$C_v = 0.5028 \text{ m}^2/\text{year}$$

For

$$U = 60\%$$

$$T_{60} = \frac{C_v \times t_{60}}{d^2}$$

$$\frac{\pi \left(\frac{60}{100} \right)^2}{4} = \frac{0.5028 \times t_{60}}{4^2}$$

$$t_{60} = 9 \text{ years}$$

Hence time required to achieve 60% degree of consolidation = 9 years

(c)

For

$$t = 5 \text{ years}$$

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

$$\frac{\pi \left(\frac{U}{100} \right)^2}{4} = \frac{0.5028 \times 5}{(4)^2}$$

$$U = 44.73\%$$

Hence settlement in 5 years = $(\Delta h)_2 = U \times (\Delta H)$

$$(\Delta h)_2 = 0.4473 \times 248$$

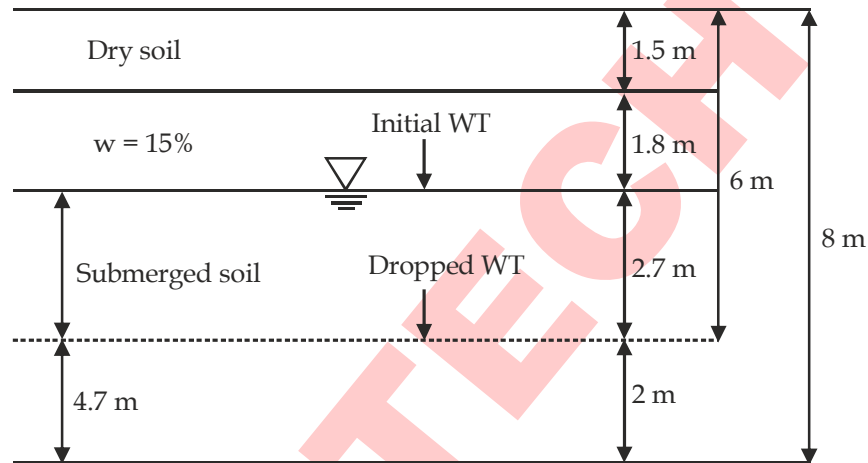


$$= 110.93 \text{ mm}$$

- (d) Since, $K = C_v (m_v) \gamma_w$
 where, C_v = Coefficient of consolidation
 m_v = Coefficient of volume compressibility
 γ_w = Unit weight of water
 K = Coefficient of permeability
 Hence, $K = (0.5028) \times (4.766 \times 10^{-4}) \times (9.81)$
 $K = 2.35 \times 10^{-3} \text{ m/year}$

133.

Sol.



Data given,
 Porosity,

$$n = 35\%$$

Specific gravity, $G = 2.7$

Moisture content 1.5 m below G.L.,

$$w = 15\%$$

$$\text{Void ratio, } e = \frac{n}{1 - n} = \frac{0.35}{1 - 0.35} = 0.538$$

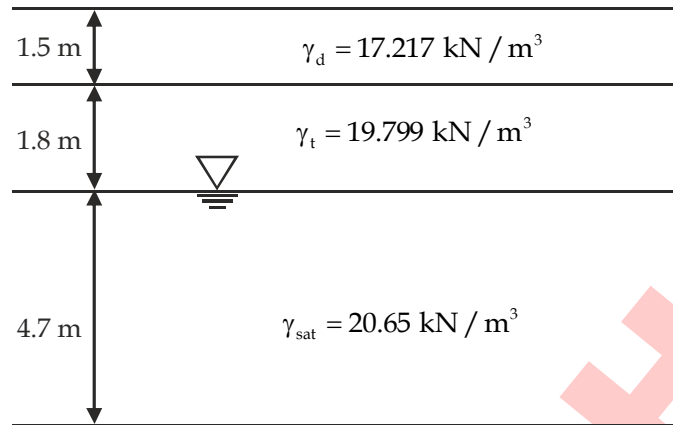
For finding effective pressure and shear strength, we need to calculate various unit weights of sand.

$$\text{Dry unit weight, } \gamma_d = \frac{G \gamma_w}{1 + e} = \frac{2.7 \times 9.81}{1 + 0.538} = 17.217 \text{ kN / m}^3$$

$$\text{Bulk unit weight (for } w = 15\%), \gamma_t = \frac{G(1 + w) \gamma_w}{1 + e} = \frac{2.7 \times (1 + 0.15) \times 9.81}{1 + 0.538} = 19.799 \text{ kN / m}^3$$

$$\text{Saturated unit weight, } \gamma_{\text{sat}} = \frac{G + e}{1 + e} \gamma_w = \left(\frac{2.7 + 0.538}{1 + 0.538} \right) \times 9.81 = 20.65 \text{ kN / m}^3$$

(i)



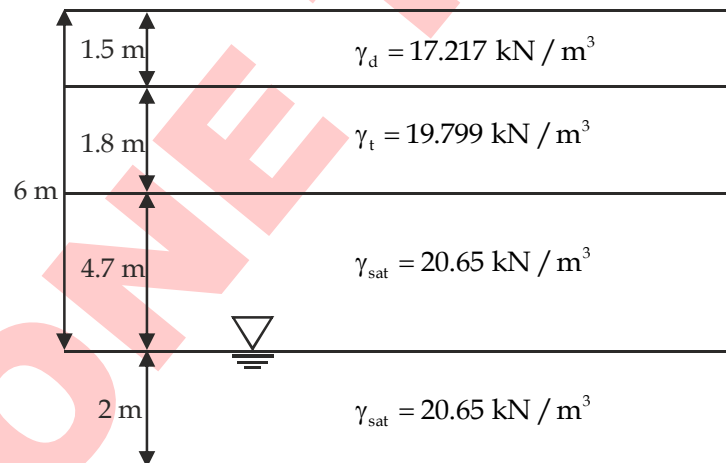
Effective stress of the depth of 8 m, $\bar{\sigma} = \text{Total stress } (\sigma) - \text{Pore water pressure } (u)$

$$\begin{aligned} \text{Total stress, at depth of 8 m, } \sigma &= \gamma_d \times 1.5 + \gamma_t \times 1.8 + \gamma_{\text{sat}} \times 4.7 \\ &= 17.217 \times 1.5 + 19.799 \times 1.8 + 20.65 \times 4.7 \\ &= 158.519 \text{ kN/m}^2 \end{aligned}$$

$$\text{Pore water pressure at depth of 8 m, } u = 4.7 \times \gamma_w = 46.107 \text{ kN/m}^2$$

$$\text{Effective stress at depth of BM, } \bar{\sigma} = \sigma - u = 158.519 - 46.107 = 112.412 \text{ kNm}^2$$

(ii)



When WT suddenly drops by 2.7 m, then soil in this depth of 2.7 m, will remain saturated rather than being submerged. Consequently the effective stress will increase.

$$\begin{aligned} \text{Total stress at depth of 8 m, } \delta &= 1.5\gamma_d + 1.8\gamma_t + (2.7 + z)\gamma_{\text{sat}} \\ &= 1.5 \times 17.217 + 1.8 \times 19.799 + 4.7 \times 20.65 \\ &= 158.519 \text{ kN/m}^2 \end{aligned}$$

$$\text{Pore water pressure at depth of 8m, } u = 2\gamma_w = 2 \times 9.81 = 19.62 \text{ kN/m}^2$$

$$\begin{aligned} \text{Effective stress at depth of 8 m, } \bar{\sigma} &= \sigma - u = 158.519 - 19.62 \\ &= 138.899 \text{ kN/m}^2 \end{aligned}$$

(iii) Shear strength of soil is given by,

$$\tau = C + \bar{\sigma} \tan \phi$$

For sand,

$$C = 0$$

and

$$\phi = 30^\circ \text{ (given)}$$

Thus,

$$\tau = 0 + \bar{\sigma} \tan 30^\circ$$

Case I:

$$\tau = 112.412 \times \tan 30^\circ = 64.901 \text{ kN / m}^2$$

Case II:

$$\tau = 138.899 \times \tan 30^\circ = 80.193 \text{ kNm}^2$$

134.

Sol.

Sl. No.	Confining Pressure (kPa)	Deviator stress (kPa)	Pore pressure (kPa)
1	100	150	40
2	200	220	75

(i) When construction is done at a faster rate, total stress parameter will use Sample 1.

$$\sigma_3 = 100 \text{ KPa}$$

$$\sigma_d = 150 \text{ KPa}$$

$$\sigma_1 = \sigma_3 + \sigma_d = 100 + 150 = 250 \text{ KPa}$$

We know,

$$\sigma_1 = \sigma_3 \tan^2 \theta + 2C \tan \theta \quad \dots(A)$$

Put value,

Sample 2

$$250 = 100 \tan^2 \theta + 2C \tan \theta \quad \dots(i)$$

$$\sigma_3 = 200 \text{ KPa}$$

$$\sigma_d = 220 \text{ KPa}$$

$$\sigma_1 = \sigma_3 + \sigma_d = 200 + 220 = 420 \text{ KPa}$$

Again from eqn. (A)

$$420 = 200 \tan^2 \theta + 2C \tan \theta \quad \dots(ii)$$

From eqn. (ii) - (i)

$$170 = 100 \tan^2 \theta$$

$$\tan \theta = 1.304$$

$$\theta = 45 + \frac{\phi}{2} = 52.51$$

$$\phi = 15.03^\circ$$

From eqn. (ii)

$$120 = 200 \tan^2 52.51 + 2C \tan 52.51$$

$$C = 30.68 \text{ KPa}$$

(ii) When construction is done slowly, effective stress parameter will use.

Sample 1 :

$$\bar{\sigma}_3 = 100 - 40 = 60 \text{ KPa}$$

$$\bar{\sigma}_1 = 250 - 40 = 210 \text{ KPa}$$

From eqn. (A)

$$210 = 60 \tan^2 \theta + 1\bar{C} \tan \theta \quad \dots(iii)$$

Sample 2 :

$$\bar{\sigma}_3 = 200 - 75 = 125 \text{ KPa}$$

$$\bar{\sigma}_1 = 420 - 75 = 345 \text{ KPa}$$

From eqn. (A)

$$345 = 125 \tan^2 \theta + 2\bar{C} \tan \theta \quad \dots(\text{iv})$$

From eqn. (iv) - (iii)

$$135 = 65 \tan^2 \theta$$

$$\tan^2 \theta = 2.077$$

$$\theta = 45 + \frac{\phi'}{2} = 55.24$$

$$\phi' = 20.49^\circ$$

From eqn. (iv)

$$345 = 125 \tan^2 55.24 + 2\bar{C} \tan 55.24$$

$$\bar{C} = 29.63 \text{ KPa}$$

135.

Sol.

In the triaxial shear test, the significance of pore water pressure is that it helps in simulating the in field conditions. In field, during loading or unloading, all the load changes are not immediately transferred to the soil particles and rather a part of it is taken up by water present in pores which is known as pore water pressure.

This vital property of soil (i.e., the effective stress being borne by soil particles in short as well as long term which in turn varies with pore water pressure over the period of time) needs to be assessed in order to analyse the safety of any structure being constructed. Hence, we conduct different types of triaxial tests to simulate different field conditions. The following examples will illustrate this point.

(i) **Consolidated Drained Test (CD Test) :**

- Drainage is not allowed in both stages. Loading rate is slow to allow water to expel out so that pore water pressure does not build up.
- This test helps in analysing the gradual loading condition such as the long term stability of embankment which has been in existence since long ago.

(ii) **Consolidated Undrained Test (CU Test)**

- Drainage is permitted in 1st stage where as it is not allowed in second stage. Thus pore water pressure develops during the second stage.
- This test helps in analysing the sudden unloading situation such as dewatering or draw down condition. The example of such a situation may be checking the stability of an embankment that has lived some of its life and is now being unloaded.

(iii) **Unconsolidated Undrained Test (UU Test)**

- Drainage is not permitted in both the stages.
- This test is suitable for soil of low permeability or when loading is very fast such as rapid construction.

The example may be analysing the stability under construction pore water pressure i.e., during construction only.

Pore Pressure parameters as given by A.W. Skempton :

- Pore water pressures, as has been discussed already, play a very important role in determining the shear strength of soil.
- A.W. Skempton gave a theoretical approach to evaluate change in pore water pressure (Δu) in terms of the change in the principal stresses (σ_1 and σ_3) by equation :

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

or

$$\Delta u = \Delta u_1 + \Delta u_2$$

where,

$$\Delta u_1 = B \Delta \sigma_3$$

$$\Delta u_2 = A.B(\Delta \sigma_1 - \Delta \sigma_3) = A_t(\Delta \sigma_1 - \Delta \sigma_3)$$

In above equation,

$\Delta\sigma_1$ and $\Delta\sigma_3$ are change in major and minor principal stresses respectively, and A and B are the constants which are known as Skempton's pore pressure parameters.

- Parameter B represents the change in the pore pressure (Δu_1) with respect to change in the cell pressure ($\Delta\sigma_3$) in the first stage of the triaxial test.

i.e.,
$$B = \frac{\Delta u_1}{\Delta\sigma_3}$$

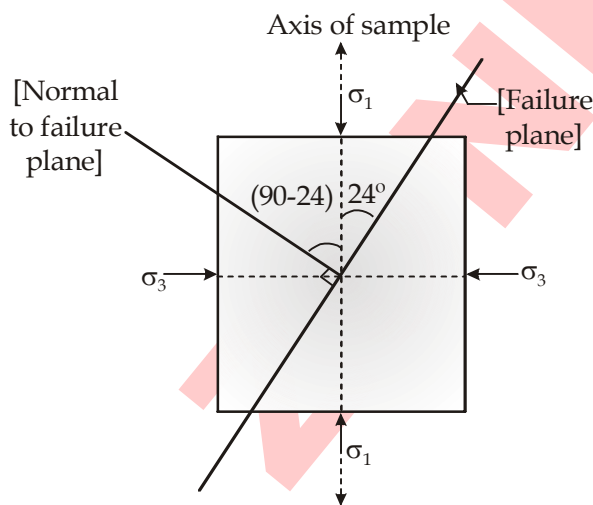
- B varies with the degree of saturation
For a completely saturated soil, $B = 1$
For a completely dry soil, $B = 0$
- In lab, B can be determined in the first stage of UU test.
- The factor A_f ($A_f = A \times B$) represents the change in pore pressure (Δu_2) due to increase in deviator stress ($\Delta\sigma_1 - \Delta\sigma_3$)

i.e.,
$$A_f = A \times B = \frac{\Delta u_2}{\Delta\sigma_1 - \Delta\sigma_3}$$

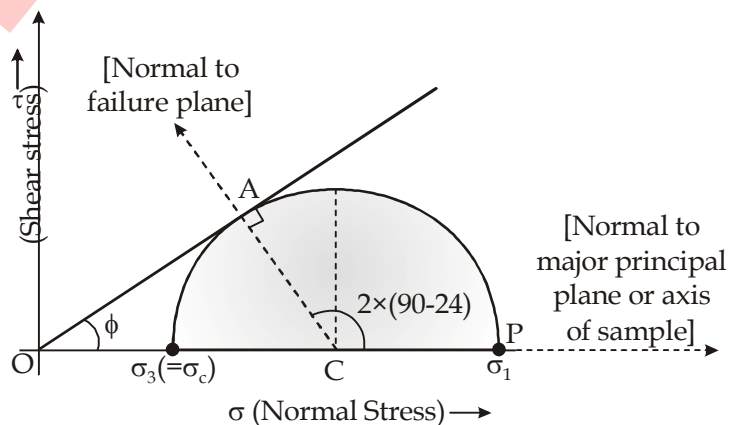
- The value of A_f can be measured during second part of the test knowing the value of A_f and B, the value of A can be easily found out.
- It is to be noted that unlike B, A varies with stress change. Thus the value of B must correspond to the pressure range in the deviator part of the test.
- Apart from stress condition, parameter A also depends on strain, anisotropy, sample disturbance and OCR.
- Value of A may be as large as 2 to 3 for very loose saturated fine sand. It can be -0.5 to 0 for heavily over consolidated clays. For NC clays $A = 0.5 - 1$.
- For OC clays $A = f(\text{OCR})$. For heavily over consolidated clay, $A < 0$, typical value of A are -0.2 to -0.3.

136.

Sol.



Mohr circle for the Triaxial Test :



Specimen is of fine dry sand, thus $c = 0$ and failure envelope (OA) passes through origin. It is given in the question that deviator stress = 400 kN/m^2

$$\Rightarrow \sigma_1 - \sigma_3 = 400 \text{ kN/m}^2 \quad \dots(i)$$

In ΔOAC ,

Exterior angle $ACP = \text{Sum of interior and opposite angles } (\angle AOC + \angle OAC)$

$$\Rightarrow 2 \times (90 - 24) = \phi + 90^\circ$$

$$\Rightarrow \phi = 42^\circ$$

$$\sin \phi = \frac{AC}{OC}$$

$$\Rightarrow \sin \phi = \frac{\frac{(\sigma_1 - \sigma_3)}{2}}{\frac{(\sigma_1 + \sigma_3)}{2}} \quad [\because \text{Stress OC corresponds to } \frac{\sigma_1 + \sigma_3}{2}]$$

$$\Rightarrow \frac{400}{\sigma_1 + \sigma_3} = \sin(42^\circ)$$

$$\Rightarrow \sigma_1 + \sigma_3 = 597.791 \text{ kN/m}^2 \quad \dots(ii)$$

Solving equation (i) and (ii), we get

$$\sigma_3 = 98.895 \text{ kN/m}^2$$

Thus, the lateral pressure to which the specimen is subjected = 98.895 kN/m²

137.

Sol. Calculation for cohesion parameter of soil, 'c'

Undrained compressive strength of soil, $q_u = 1.2 \text{ kg/cm}^2$

$\therefore$ Undrained cohesion or cohesion parameter of soil, $c = \frac{q_u}{2} = 0.6 \text{ kg/cm}^2$

Calculations for ϕ (angle of internal friction)

We will assume that value of 'c' obtained from unconfined compression test is valid even for triaxial test. [Though in actual scenario, two different tests may or may not give same shear strength parameters]

From tri-axial test,
$$\sigma_1 = \sigma_3 \left(\frac{1 + \sin \phi}{1 - \sin \phi} \right) + 20 \times \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}}$$

Here,

$$\sigma_3 = 0.4 \text{ kg/cm}^2 = \sigma_c$$

and

$$\begin{aligned} \sigma_1 &= \sigma_c + \Delta \sigma_a = 0.4 + 1.6 \\ &= 2 \text{ kg/cm}^2 \end{aligned}$$

Also,

$$\frac{1 + \sin \phi}{1 - \sin \phi} = \tan^2 \left(45 + \frac{\phi}{2} \right)$$

i.e.,

$$2 = 0.4 \times \tan^2 \left(45 + \frac{\phi}{2} \right) + 2 \times 0.6 \times \tan \left(45 + \frac{\phi}{2} \right)$$

Solving, we get

$$\tan \left(45 + \frac{\phi}{2} \right) = 1.19258$$

i.e.,

$$\phi = 2 \times [\tan^{-1}(1.19258) - 45^\circ] = 10.039^\circ$$

Calculation for total stress at a depth of 4m

Assuming no capillary action, i.e., soil above water table to be dry.

$$\sigma = \rho_d \times 2.5 + [\rho_{\text{sat}} \times 1.5]$$

ρ_d = dry-density of soil = 1.7 gm/cc

$$\rho_{\text{sat}} = \text{saturated-density of soil} = \frac{G \rho_w (1 + w)}{1 + e}$$

Now,

$$\rho_d = \frac{\rho_w G}{1 + e}$$

$$\Rightarrow 1.7 = \frac{1 \times 2.7}{1 + e}$$

$$\Rightarrow e = 0.5882$$

Also,
For 100% saturation,
i.e.,

$$\Rightarrow 0.5882 \times 1 = w \times 2.7$$

$$\Rightarrow w = 0.21786$$

$$\therefore \rho_{\text{sat}} = \frac{\rho_w \times (1 + w)}{1 + e} = \frac{2.7 \times 1 \times (1 + 0.21786)}{1 + 0.5882}$$

$$= 2.0704 \text{ gm/cm}^3$$

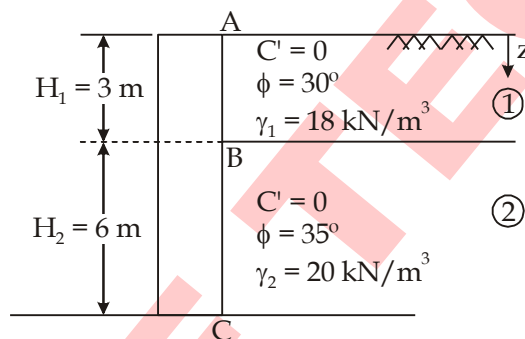
$$\Rightarrow \sigma = \rho_d \times 250 + \rho_{\text{sat}} \times 150$$

$$= 735.562 \text{ gm/cm}^2$$

$$= 0.736 \text{ kg/cm}^2$$

138.

Sol. Given :



Coefficient of active earth pressure

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

$$K_{a1} = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{1}{3}$$

$$K_{a2} = \frac{1 - \sin 35^\circ}{1 + \sin 35^\circ} = 0.271$$

$$P_a = K_a \bar{\sigma}_z - 2c\sqrt{K_a}$$

Since soil is cohesionless soil

$$\therefore c = 0$$

$$\therefore P_a = K_a \bar{\sigma}_z = K_a \gamma z$$

(a) Active earth pressure ($0 \leq z \leq 3$)At point A, $z = 0$

$$P_A = K_{a1} \gamma_1 z$$

$$P_A = 0$$

At point B, $z = 3$

$$P_B = K_{a1} \gamma_1 z$$

$$P_B = \frac{1}{3} \times 18 \times 3$$

$$P_B = 18 \text{ kN/m}^2$$

(b) Active earth pressure ($3 \leq z \leq 9$)

At point B, $z = 3$

$$p_B = K_{a2} \gamma_1 z = 0.271 \times 18 \times 3$$

$$p_B = 14.634 \text{ kN/m}^2$$

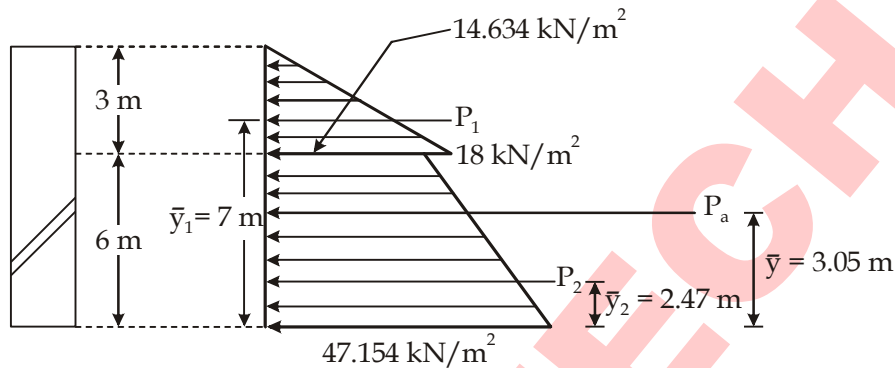
At point C, $z = 9$

$$p_C = K_{a2} \gamma_1 H_1 + K_{a2} \gamma_2 H_2$$

$$p_C = 0.271 \times 18 \times 3 + 0.271 \times 20 \times 6$$

$$p_C = 47.154 \text{ kN/m}^2$$

(i) Pressure distribution diagram



(ii) Total active thrust on the wall

$$\bar{y}_1 = \left(6 + \frac{3}{3}\right) = 7 \text{ m from base of retaining wall}$$

$$\bar{y}_2 = \left(\frac{2 \times 14.634 + 47.154}{14.634 + 47.154}\right) \times \frac{6}{3} = 2.47 \text{ m from the base of retaining wall}$$

$$P_1 = \frac{1}{2} \times 3 \times 18 = 27 \text{ kN/m}$$

$$P_2 = \frac{1}{2} \times 6 \times (14.634 + 47.154) = 185.364 \text{ kN/m}$$

$$P_a = P_1 + P_2$$

$$P_a = 27 + 185.364 = 212.364 \text{ kN/m}$$

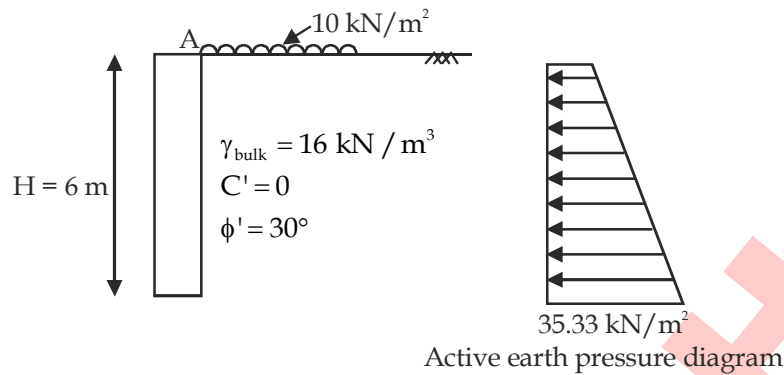
Location of this total active earth pressure thrust i.e. $\bar{y}$

$$\bar{y} = \frac{P_1 \bar{y}_1 + P_2 \bar{y}_2}{P_1 + P_2} = \frac{27 \times 7 + 185.364 \times 2.47}{27 + 185.364}$$

$$\bar{y} = 3.05 \text{ m from the base}$$

139.

Sol. (i)



$$\text{Coefficient of active earth pressure, } K_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{1}{3}$$

$$\text{Active earth pressure at top of wall} = K_a q = \frac{1}{3} \times 10 = 3.33 \text{ kN/m}^2$$

$$\begin{aligned} \text{Active earth pressure at bottom of wall} &= K_a \gamma_{\text{bulk}} H + K_a q \\ &= \frac{1}{3} \times 16 \times 6 + 3.33 = 35.33 \text{ kN/m}^2 \end{aligned}$$

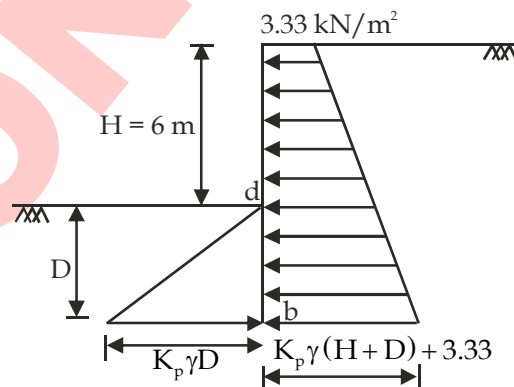
Active thrust = Area of pressure diagram

$$= (3.33 \times 6) + \frac{1}{2} \times 6 \times (35.33 - 3.33)$$

$$= 116 \text{ kN/m} - \text{length of wall}$$

(ii) Let the depth of embedment of cantilever sheet pile be D m.

Using approximate analysis



Taking moment of all forces about b

$$\frac{1}{2} K_p \gamma D^2 \times \frac{D}{3} = 3.33(H + D) \times \frac{(H + D)}{2} + \frac{1}{2} K_a \gamma (H + D)^2 \times \left(\frac{H + D}{3} \right)$$

$$\text{Coefficient of passive earth pressure, } K_p = \frac{1}{K_a} = 3$$

$$\therefore \frac{1}{2} \times 3 \times 16 \frac{D^3}{3} = \frac{3.33}{2} (6+D)^2 + \frac{1}{2} \times \frac{1}{3} \times 16 \times \frac{(6+D)^3}{3}$$

$$8D^3 = 1.665(36 + D^2 + 12D) + 0.889(216 + D^3 + 180^2 + 108D)$$

$$8D^3 = 59.94 + 1.665D^2 + 20D + 192 + 0.889D^3 + 16D^2 + 96D$$

$$7.11D^3 - 17.665D^2 - 116D - 251.94 = 0$$

$$D = 6.1 \text{ m}$$

Since it is an approximate method, hence allowing 20% extra,

$$\begin{aligned} \text{Depth of embankment} &= 1.2 \times 6.1 \\ &= 7.32 \text{ m} \end{aligned}$$

140.

Sol. Dry Density of the soil above W.T

$$\begin{aligned} \gamma_d &= \frac{G\gamma_w}{(1+e)} = \frac{(2.65 \times 9.81)}{(1+0.65)} \\ &= 15.76 \text{ kN/m}^3 \end{aligned}$$

$$\text{Surcharge Intensity } q = 14 \text{ kN/m}^2$$

Submerged density of the soil below W.T

$$\gamma_{\text{sub}} = \left(\frac{G-1}{1+e} \right) \gamma_w = \frac{(2.65-1) \times 9.81}{1+0.65} = 9.81 \text{ kN/m}^3$$

$$K_A = \frac{(1 - \sin 30^\circ)}{(1 + \sin 30^\circ)} = \left(\frac{1}{3} \right)$$

$\therefore$ At point A $Z = 0$

$$p_A = (K_A q + K_A \gamma \times 0)$$

$\therefore$

$$p_A = K_A q = \left(\frac{1}{3} \times 14 \right) = 4.67 \text{ kN/m}^2$$

In Sand

At point B $Z = 3\text{m}$,

$$\text{Equivalent surcharge, } q_1 = (q + \gamma_d h_1) = (14 + 15.76 \times 3)$$

$$q_1 = 61.28 \text{ kN/m}^2$$

$$p_B = K_a q_1$$

$$= \frac{1}{3} \times 61.28 = 20.43 \text{ kN/m}^2$$

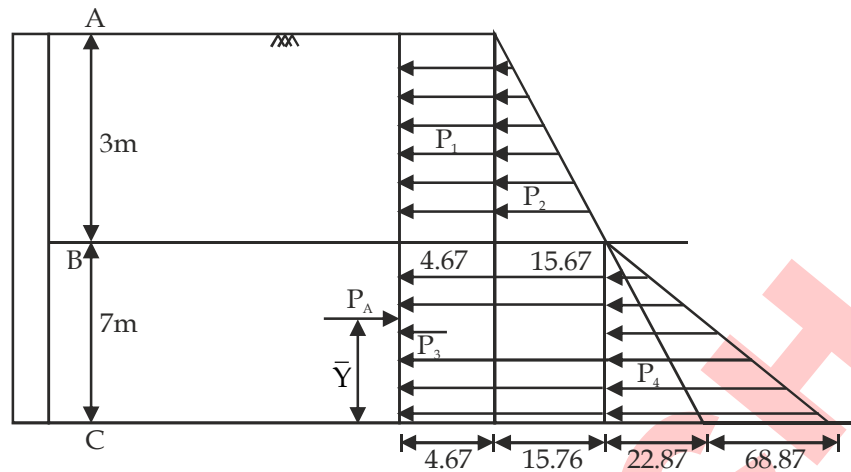
At Point C

$$p_c = (K_a q_1 + \gamma_w h_2 + \gamma_{\text{sub}} h_2 K_a)$$

$$= (20.43 + 68.67 + 22.87) = 111.97 \text{ kN/m}^2$$



Earth Pressure Diagram:



$$P_1 = (3 \times 4.67) = 14.01 \text{ kN / m},$$

$$Y_1 = \left(7 + \frac{3}{2}\right) = 8.5\text{m}$$

$$P_2 = \left(\frac{1}{2} \times 15.7 \times 3\right) = 23.64 \text{ kN / m},$$

$$Y_2 = 7 + \frac{1}{3} \times 3 = 8\text{m}$$

$$P_3 = (4.67 + 15.76) \times 7 = 143.01 \text{ kN / m},$$

$$Y_3 = \frac{7}{2} = 3.5\text{m}$$

$$P_4 = \frac{1}{2} (22.87 + 68.67) \times 7 = 320.39 \text{ kN / m},$$

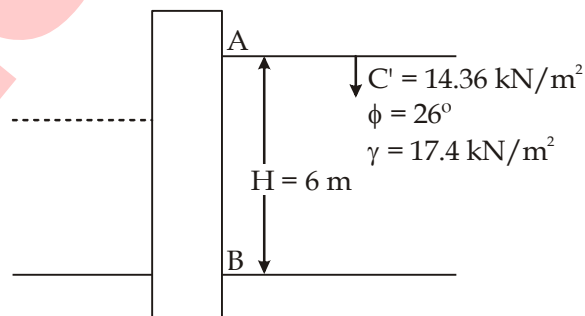
$$Y_4 = \frac{7}{3} = 2.33\text{m}$$

Total active Thrust on the wall

$$P_A = \sum_{i=1}^n P_i = (P_1 + P_2 + P_3 + P_4) = 501.05 \text{ kN / m}$$

$$\text{Resultant of action thrust} = \frac{P_1 Y_1 + P_2 Y_2 + P_3 Y_3 + P_4 Y_4}{P_A} = 3.105\text{m}$$

141.
Sol.



(i) calculation for Rankine active force per unit length

$$p_A = K_a \gamma z - 2C\sqrt{K_a}$$

Here,

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

$$K_a = \frac{1 - \sin 26^\circ}{1 + \sin 26^\circ} = 0.39$$

At point A,

$$z = 0$$

$$p_A = -2C\sqrt{K_a}$$

$$p_A = -2 \times (14.36) \times \sqrt{0.39}$$

$$p_A = -17.94 \text{ kN/m}^2$$

At point B,

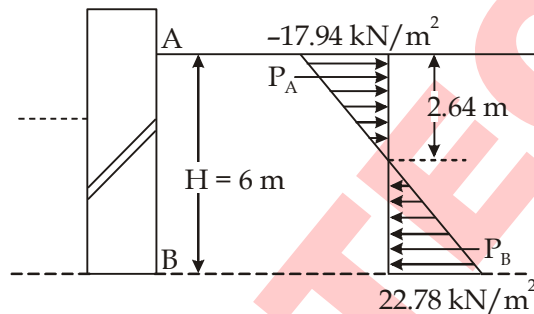
$$z = 6 \text{ m}$$

$$p_B = K_a (\gamma H) - 2C\sqrt{K_a}$$

$$p_B = 0.39 \times 17.4 \times 6 - 2 \times 14.36 \times \sqrt{0.39}$$

$$p_B = 22.78 \text{ kN/m}^2$$

Hence, the pressure distribution diagram



The depth of tension crack

$$p_a = 0 = K_a \gamma H_c - 2C\sqrt{K_a}$$

$$H_0 = \frac{2C\sqrt{K_a}}{K_a \gamma}$$

$$H_0 = \left(\frac{2C}{\sqrt{K_a} \gamma} \right)$$

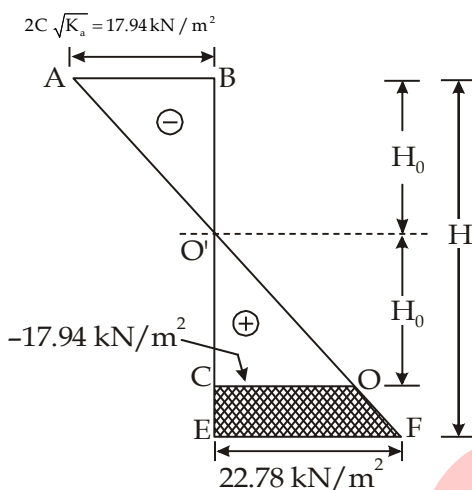
$$H_0 = \frac{2 \times 14.36}{\sqrt{0.39} \times 17.4} = 2.64 \text{ m}$$

The resultant active thrust before the tensile crack occurs.

= Area under COEF

$$= \frac{1}{2} \times (17.94 + 22.78) \times (6 - 5.28)$$

$$= 14.659 \text{ kN/m}^2$$



(ii) Calculation for critical depth

Net total active earth pressure is zero upto a depth $2H_0$

$$H_C = 2H_0 = \frac{2 \times 2C}{\sqrt{K_a} \gamma}$$

$$H_C = 2 \times 2.64 = 5.28 \text{ m}$$

142.

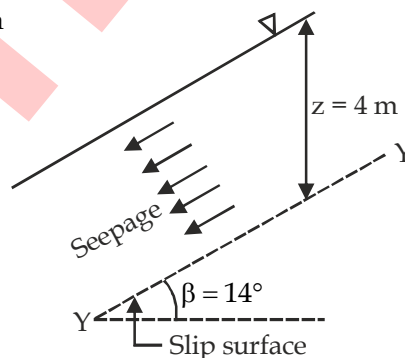
Sol. Given,

$$\gamma_t = 22 \text{ kN/m}^3$$

$$\phi'_r = 20^\circ$$

$$\gamma_{\text{sub}} = \gamma_t - \gamma_w$$

$$= 12.19 \text{ kN/m}^3$$



Slip surface is along YY as shown in figure.

$$\text{Factor of safety, } F = \frac{\tau_f}{\tau}$$

where,

τ_f = Resisting shear force by soil

$$\tau_f = C + \bar{\sigma} \tan \phi'_r$$

where,

$$\bar{\sigma} = \bar{\sigma}_z \cos \beta$$

$$= \left(\frac{\gamma_{\text{sub}} z \times b \cos \beta}{b} \right) \cos \beta$$

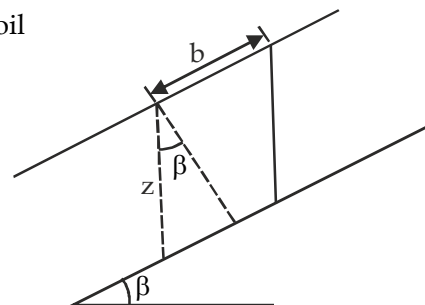
$$\bar{\sigma} = \gamma_{\text{sub}} z \cos^2 \beta$$

$$\tau_f = \gamma_{\text{sub}} \times z \cos^2 \beta \times \tan \phi'_r$$

τ = shear force due to weight of soil and seepage

$$= \sigma_z \sin \beta = \frac{\gamma_{\text{sat}} \times z \times b \cos \beta \times \sin \beta}{b}$$

$$= \gamma_{\text{sat}} z \cos \beta \sin \beta$$



$$F = \frac{\tau_f}{\tau} = \frac{\gamma_{\text{sub}} z \cos^2 \beta \tan \phi'_r}{\gamma_{\text{sat}} z \sin \beta \cos \beta} = \frac{\tan \phi'_r}{\tan \beta} \times \frac{\gamma_{\text{sub}}}{\gamma_{\text{sat}}}$$

$$F = \frac{\tan 20^\circ}{\tan 14^\circ} \times \frac{12.19}{22}$$

$$F = 0.81$$

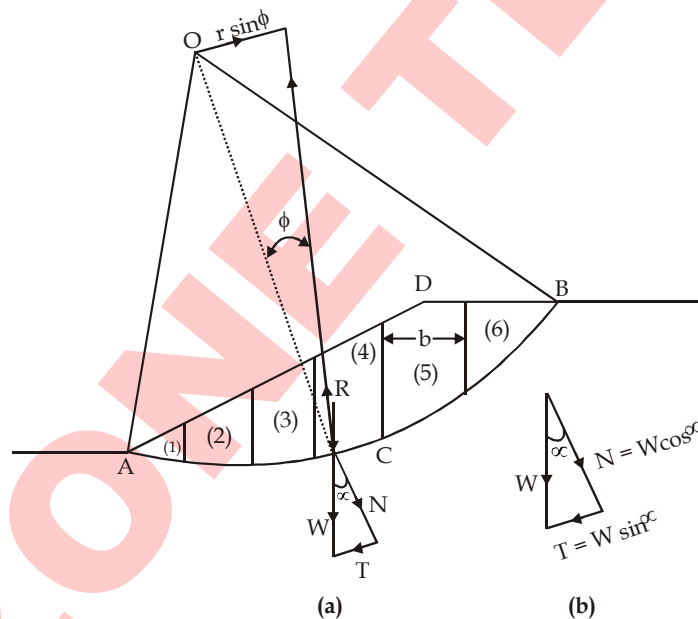
143.

Sol. Swedish Circle Method :

The actual shape of a slip surface in the case of finite slopes is curvilinear. For convenience, it is approximated as circular. The assumption of a circular slip surface and its application for stability analysis of slopes was developed in Sweden. The method is known as the Swedish circle method or the method of slices.

Figure (a) shows a slope. Let AB be a circular surface with radius r and centre O. The trial failure wedge above the slip surface is divided into vertical slices by drawing vertical lines, as shown. The slices are usually of equal width, but not necessarily so. In case of non-homogenous slopes where the slip surface passes through more than one type of material, a vertical line is always located at the point where the slip surface passes from one material to the other.

Let us consider the equilibrium of one slice (say, No. 4). The slice is in equilibrium under the following forces.



- (1) Weight (W) acting vertically through its centre of gravity.
- (2) Cohesive force (C) acting along the curved surface in the direction opposite to the direction of probable movement of the wedge.
- (3) Reaction (R) at the base inclined at angle ϕ to the normal, assuming the slippage is imminent.
- (4) Reactions on the two vertical sides of the slice due to adjacent slices. However, in the Swedish circle method, it is assumed that the reactions on the two sides are equal and opposite and are, therefore, in equilibrium and do not affect the stability of the slice. Accordingly, only the first three forces are considered for the analysis.

The weight W is resolved into its normal component (N) and tangential component (T). Let us take the moments about the centre of rotation O of all the 3 forces.

Actuating or overturning moment,

$$M_0 = T \times r \quad \dots(a)$$

The moment due to N-components is zero, as those components always pass through O.

Resisting moment,

$$M_R = (C \Delta L) \times r + R(r \sin \phi) \quad \dots(b)$$

where ΔL is the length of the curved surface of the slice.

Resolving the forces in radial direction,

$$N = R \cos f \text{ or } R = N / \cos f \quad \dots(c)$$

or

$$R \sin f = N \tan f \quad \dots(d)$$

From Eqs. (b) and (d),

$$M_R = (C \Delta L)r + N r \tan \phi \quad \dots(e)$$

The factor of safety for the slice is equal to the ratio of the resisting moment (M_R) and the overturning moment (M_0). Thus

$$F_s = \frac{r[c\Delta L + N \tan \phi]}{Tr} = \frac{c \Delta L + N \tan \phi}{T}$$

The factor of safety of entire wedge is given by

$$F_s = \frac{\Sigma c \Delta L + \Sigma N \tan \phi}{\Sigma T} \quad \dots(i)$$

If c and ϕ are constant,

$$F_s = \frac{cL_a + \tan \phi \Sigma N}{\Sigma T} \quad \dots(ii)$$

where

$$L_a = \text{length of the entire slip surface} = \Sigma \Delta L$$

The components N and T are determined by drawing force triangles as shown in Fig. (b). If the angle α which the normal makes with the vertical is measured, the components can be computed as under

$$N = W \cos \alpha, \text{ and } T = W \sin \alpha$$

The length ΔL of the arc is given by $b \sec \alpha$, where b is the width of the slice.

Therefore, Eq. (i) can be written as

$$F_s = \frac{\Sigma c b \sec \alpha + \Sigma (W \cos \alpha) \tan \phi}{\Sigma W \sin \alpha}$$

It may be noted that the tangential component T may be negative i.e. in the direction opposite to that of movement for some of the slices near the toe.

The procedure can be summarised as under :

- Take a trial wedge and divide it into 6 to 12 vertical slices.
- Determine the weight of each slice and its line of action.

For convenience, the weight is generally taken proportional to the middle ordinate of the slice and it is assumed to have line of its action through the middle of the slice.

- The weight is resolved (analytically or graphically) into normal and tangential components.
- The curved length ΔL of each slice is measured or computed.
- The factor of safety is determined from eq. (iii) or Eq. (i)

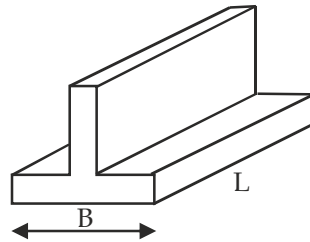
The calculations are generally done in a tabular form. The stability analysis is repeated for a number of trial surfaces. The circle which gives the minimum factor of safety is the most critical circle.

144.

Sol. The situation in which different types of shallow foundation are adopted.

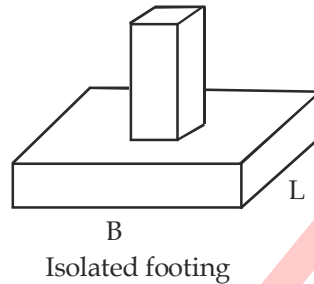
1. Strip footing

These are also known as wall footing to support wall. If $L \gg B$ then Strip footing adopted.



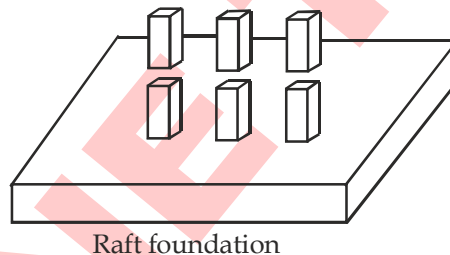
2. Isolated footing

These are also known as spread footing. Isolated footing is used below the column.



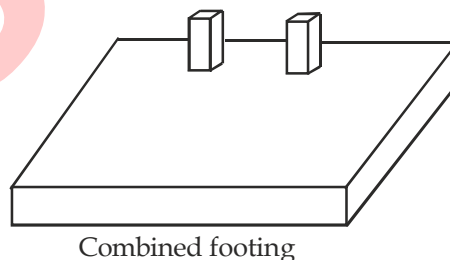
3. Raft/Mat foundation

These type of foundations are large continues footing which support all columns and walls of a structure and are constructed when soil is weak and load from the structure is heavy



4. Combined Footings

These footings are usually constructed due to space limitations and support two or more columns. They may be either rectangular or trapezoidal in shape.



145.

Sol. Data given:

$$\phi = 35^\circ ; C = 0 ; N_q = 33 ; N_\gamma = 34$$

Width of square footin, $B = 3 \text{ m}$

Depth of footing, $D_f = 2 \text{ m}$

Bulk unit weight of sand above water table,

$$\gamma_t = 18 \text{ kN} / \text{m}^3$$

Saturated unit weight of sand, $\gamma_{\text{sat}} = 20 \text{ kN/m}^3$

As given by Terzaghi,

Ultimate bearing capacity for a square footing,

$$q_u = 1.3 C N_c + q N_q + 0.4 b \gamma N_\gamma$$

For

$$c = 0,$$

$$q_u = q N_q + 0.4 B \gamma N_\gamma$$

The value of q and γ depends upon relative position of water table

(i) When water table is at ground surface:

$$q = (\gamma_{\text{sat}} - \gamma_w) D_f$$

$$q_u = (20 - 9.81) \times 2 \times 33 + 0.4 \times 3 \times (20 - 9.81) \times 34 \\ = 1088.292 \text{ kN/m}^2$$

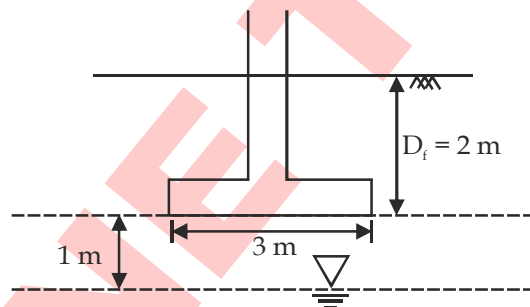
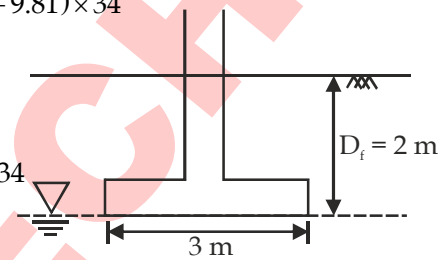
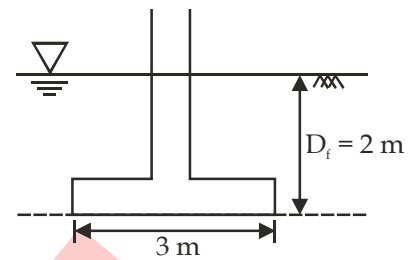
(ii) When water table is at level of footing:

$$q = \gamma_t \times D_f$$

$$q_u = 18 \times 2 \times 33 + 0.4 \times 3 \times (20 - 9.81) \times 34 \\ = 1603.752 \text{ kN/m}^2$$

(iii) When water table is 1 m below the footing:

$$q_u = 18 \times 2 \times 33 + 0.4 [1 \times 18 + 2 \times (20 - 9.81)] \times 34 \\ = 1709.968 \text{ kN/m}^2$$



146.

Sol. Given that

Depth of footing = 1.3 m

Factor of safety = 3

Safe load = 800 kN

Void ratio (e) = 0.55

Degree of saturation = 50%

$G = 2.67$

$C = 8 \text{ kN/m}^2$

$\phi = 30^\circ$

$N_c = 37.2$

$N_q = 22.5$

$N_\gamma = 19.7$

We know that for square footing

$$q_u = 1.3 C N_c + \gamma D_f N_q + 0.4 B \gamma N_\gamma$$

$$\gamma = \frac{G + Se}{1 + e} \gamma_w = \frac{2.67 + 0.5 \times 0.55}{1 + 0.55} \times 10 = 19 \text{ kN/m}^3$$

$$q_u = 1.3 \times 8 \times 37.2 + 19 \times 1.3 \times 22.5 + 0.4 B \times 19 \times 19.7$$

$$q_{\text{nu}} = 1.3 \times 8 \times 37.2 + 19 \times 1.3 \times 22.5 - 19 \times 1.3 + 0.4 B \times 19 \times 19.7$$

∴ Safe load i.e. $Q_s = 800 \text{ kN}$

$$\text{So, } q_{\text{safe}} = \frac{800}{B^2} = \frac{q_{\text{nu}}}{f} + \gamma D_f$$

$$\Rightarrow \frac{800}{B^2} = [1.3 \times 8 \times 37.2 + 19 \times 1.3 \times 22.5 - 19 \times 1.3 + 0.4 B \times 19 \times 19.7] \frac{1}{3} + 19 \times 1.3$$

$$\frac{800}{B^2} = 305.98 + 49.91B + 24.7$$

$$800 = 330.68B^2 + 49.91B^3$$

$$B = 1.41 \text{ m}, -6.2 \text{ m}, -1.83 \text{ m}$$

Hence width (B) = 1.41 m.

147.

Sol. In clay soil:

Net bearing capacity of a shallow foundation in clayey soil is given below.

$$q_{\text{nu}} = CN_c \quad \dots(1)$$

It can be seen from equation (1) that the effect of a rise of water table is unaffected in net ultimate bearing capacity.

In sandy soil:

Net ultimate bearing capacity of a shallow foundation in sandy soil is given below:

$$q_{\text{nu}} = \gamma D_f (N_q - 1) + 0.5B\gamma N_\gamma \quad \dots(2)$$

From equation (2), the effect of a rise of water table from below the failure zone upto the ground surface in a free-draining soil (sandy soil) is to reduce the bearing capacity of a footing by nearly 50 percent, because if we increase water table we use effective overburden pressure (γ_{sub} in place of γ_{sat}). Since γ_{sub} is roughly one-half of γ_{sat} , the bearing capacity contribution from the surcharge, that is, qN_q and from the soil beneath the footing $0.5\gamma BN_\gamma$ will each be reduced by 50 percent.

148.

Sol. The split-spoon sampler is driven into the soil for a distance of 450 mm by blows of a drop hammer (monkey) of 65 kg falling vertically and freely from a height of 750 mm. The number of blows required for the last 300 mm of penetration is added together and recorded as the N value at that particular depth of the borehole. The number of blows required to effect the first 150 mm of penetration, called the seating drive, is disregarded.

1. Over burden correction

- It is necessary because the N-value will have effect on it due to confinement of soil at various depth.
- Two granular soils possessing the same relative density but having different confining pressures are tested, the one with a higher confining pressure will give higher N value.
- Since the confining pressure increases with depth, the N values at shallow depths are underestimated and the N values at larger depths are overestimated.
- Therefore, if no correction is applied to recorded N values, the relative densities at shallow depths will be under estimated and at higher depths, they will be overestimated.
- If N_0 = observed S.P.T. value Then,

$$N_1 = N_0 \times \frac{350}{(\bar{\sigma} + 70)}$$

Where, $\bar{\sigma}$ = Effective stress at level of test (kN/m^2)

N_1 = Corrected N-value of overburden.

- Overburden correction will not be applied if $\bar{\sigma} > 280 \text{ kN/m}^2$

2. Dilatancy Correction

- It is applied to the already corrected N-values for overburden pressure. Dilatancy correction is

required only if $[N_1 > 15]$ in saturated fine sand and silt (ue, when water table is above test level).

- $(N_1 > 15)$ basically represents the Dense sand which will have the tendency to dilate under rapid loading (untrained condition) and (-ve) pore water pressure will develop. Hence, observed (N) value will be more because shear resistance will increase.

$$\text{Corrected } N \text{ value after Dilatancy Correction} = N_2 = 15 + \frac{1}{2}(N_1 - 15)$$

- This correction becomes more significant for fine dense sand.

149.

Sol. Given,

Parameters	Contractor 1	Contractor 2
Size of plate	45 cm × 45 cm	75 cm × 75 cm
Load	100 kN	175 kN
Settlement at above load	10 mm	10 mm

Using Hausel method

q_p = constant corresponding bearing pressure
 q_c = constant corresponding to perimeter shear

$$Q = A_p q_p + p_p q_c$$

For contractor 1,

$$\begin{aligned} \text{Area of plate } (A_p) &= 45 \text{ cm} \times 45 \text{ cm} = (0.45 \times 0.45) \text{ m}^2 \\ \text{Perimeter of plate } (p_p) &= (4 \times 0.45) \text{ m} \end{aligned}$$

So,

$$Q = (0.45)^2 q_p + 4 \times 0.45 q_c \quad \dots(1)$$

or

$$100 = 0.2025 q_p + 1.8 q_c$$

For contractor 2,

$$\begin{aligned} \text{Area of plate } (A_p) &= (0.75 \times 0.75) \text{ m}^2 \\ \text{Perimeter of plate } (p_p) &= 4 \times 0.75 = 3 \text{ m} \end{aligned}$$

So,

$$Q = (0.75)^2 q_p + 4 \times 0.75 q_c \quad \dots(2)$$

or

$$175 = 0.5625 q_p + 3 q_c$$

From (1) and (2),

$$q_p = 37.04 \text{ kN/m}^2$$

and

$$q_c = 51.39 \text{ kN/m}^2$$

For a footing size 3 m × 3 m

$$\text{Maximum load carried } (Q_f) = A_f q_p + p_f q_c$$

Here,

A_f = Area of footing

p_f = perimeter of footing

So,

$$Q_f = (3)^2 \times 37.04 + 4 \times 3 \times 51.39 = 950.04 \text{ kN}$$

Hence, Maximum load carried by footing = 950.04 kN.

150.

Sol. Given data

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

$$\text{UCS} = 200 \text{ kPa}$$

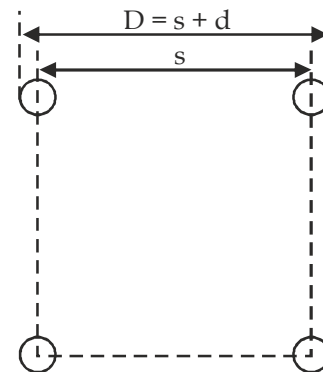
$$d = 0.3 \text{ m}$$

$$L = 12 \text{ m}$$

$$\text{Spacing} = 0.75 \text{ m}$$

$$\alpha = 0.6$$

$$C = \frac{\text{UCS}}{2} = \frac{200}{2} = 100 \text{ kPa}$$



- (i) Load capacity of the group based on the individual pile failure. (For friction pile, capacity of bearing may be ignore for calculation of single pile capacity)

$$Q_u = \alpha \bar{C} A_s$$

$$\begin{aligned}
 &= 0.6 \times 100 \times \pi \times 0.3 \times 12 \\
 Q_u &= 678.58 \text{ kN} \\
 Q_u &= 4 \times 678.58 \\
 Q_u &= 2714.34 \text{ kN} \\
 \text{So,} \quad & \\
 \text{(ii) Load capacity of the group based on the block failure.} \\
 D &= s + d = 0.75 + 0.3 \\
 D &= 1.05 \text{ m} \\
 Q_{ug} &= 9CA_b + \alpha \bar{C}A_s \quad [\alpha = 1] \\
 Q_{ug} &= 9 \times 100 \times 1.05 \times 1.05 + 1 \times 100 \times 4 \times 1.05 \times 12 \\
 Q_{ug} &= 6032.25 \text{ kN}
 \end{aligned}$$

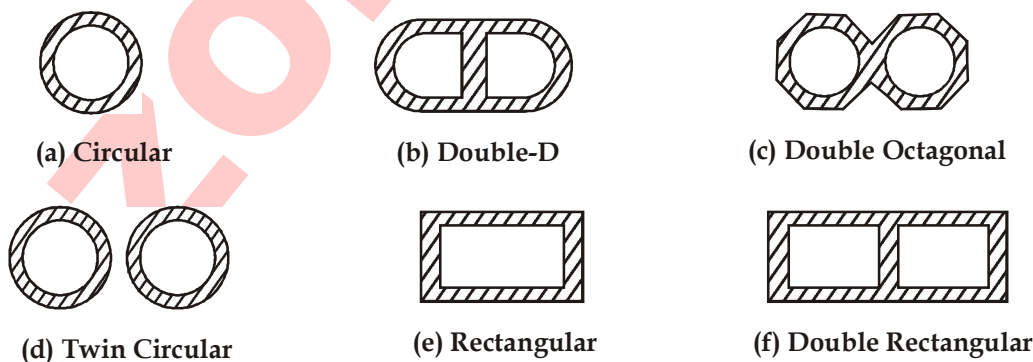
(ii) Design Load capacity of group

$$\begin{aligned}
 &= \text{Min} \left[\frac{Q_u}{\text{FOS}}, \frac{Q_{ug}}{\text{FOS}} \right] \\
 &= \text{Min} \left[\frac{2714.34}{2}, \frac{6032.25}{3} \right] \\
 &= \text{Min} [1357.17 \text{ kN}, 2010.75 \text{ kN}] \\
 (Q_U)_{\text{design}} &= 1357.17 \text{ kN}
 \end{aligned}$$

151.

Sol. Different Shapes of Wells :-

- (1) **The most commonly used shape is circular [Figure (a)],** as it has high structural strength and is convenient in sinking. The chances of tilting are also minimum in this shape. The shape is quite suitable for piers of the single-line railway bridges and the double-lane road bridges.
- (2) **Double-D wells [Figure (b)]** are generally used for the piers and abutments of bridges which are too long to be accommodated on a circular well of 9 m diameter. The wells of this shape can also be sunk easily.
- (3) **Double-octagonal wells [Figure (c)]** are better than the double-D wells in many respects. The square corners are eliminated and bending stresses are considerably reduced. However, they offer greater resistance than double-D wells against sinking on account of increased surface area. Moreover, the construction is more difficult.



- (4) **Twin-Circular wells [Figure (d)]** are two independent wells placed very close to each other and having a common well cap. The wells are sunk simultaneously. These wells are suitable where the length of the pier is considerable, which cannot be accommodated on a double-D or double-octagonal well. Twin circular wells are advantages when the depth of sinking is small and the bearing capacity of the soil is high. The disadvantage of twin circular wells is that there is a possibility of the relative settlement of the two wells even if a heavy R.C.C. top cap is provided unless the wells are founded on an incompressible soil.



- (5) **Rectangular wells [Figure (e)]** are generally used for bridge foundation having depths upto 7-8 m. For large foundations, double-rectangular wells [Figure (f)] are used. For piers and abutments of very large size, rectangular wells with multiple dredge holes (not shown in the figure) are used. Bending stresses in the steining are very high in rectangular wells.

152.

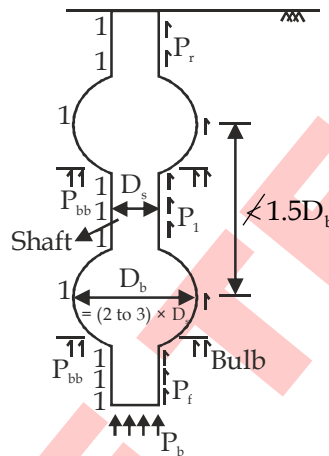
Sol.

Bulb diameter = 2 to 3 times shaft diameter.

Spacing between centre to centre of bulb $\geq 1.5 \times \text{dia of bulb}$.

Mechanism involved in resisting uplift in swelling soil:

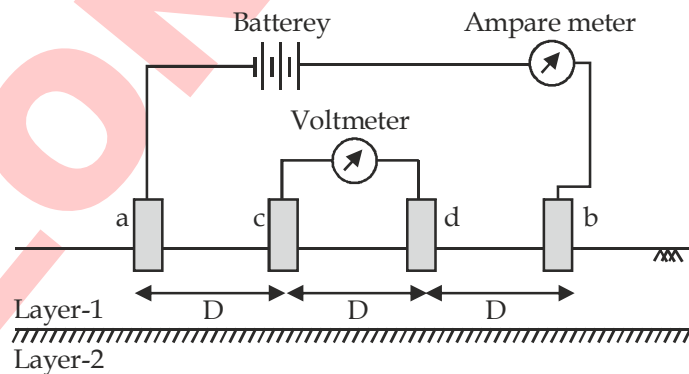
- (1) **Base bearing at end of pile (P_b):** The uplift pressure is resisted at the base of pile by point bearing.
- (2) **Frictional resistance on sides of pile (P_f):** The sides of pile resists the uplift pressure by mechanism of frictional resistance
- (3) **Bearing resistance at enlarged bulb of pile (P_{bb}):** The enlarge portion of bulb outside the shaft resists the uplift pressure by point bearing.



153.

Sol.

Electrical resistivity method of soil exploration is based on the principle that electrical resistance offered by Different types of soils to the flow of current is different.



Procedure:

- The electrodes a and b are driven into the ground at a considerable distance, say $3D$.
- An electrical potential is established between these electrodes with the help of a battery and an amperemeter is connected with battery to measure the current.
- The electrical potential (V) at intermediate points is measured with the help of two other electrodes c and d installed at these points. It is to be noted that all four electrodes are placed at equal spacing of D .
- This spacing of electrodes (D) determines the effective depth upto which the lines of electric current flow and hence the depth of the soil influencing the rate of drop of potential (V) from one

electrode to other. The larger the spacing (D), larger is the depth of soil involved in circuit.

- The mean resistivity or apparent resistivity (δ) is given by:

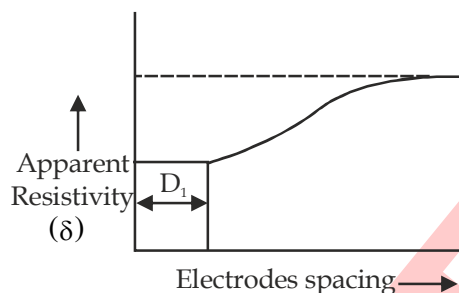
$$\delta = 2\pi D \left(\frac{V}{I} \right)$$

In fact, apparent resistivity (δ), represents the weighted average of true resistivity in a large volume of soil with the soil nearer to surface contributing more than that far away.

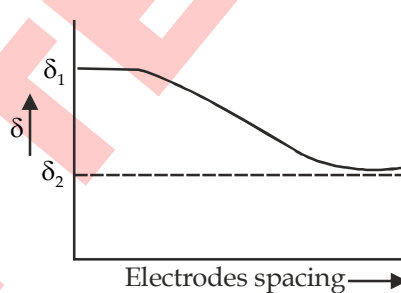
- The value of δ remains constant for a uniform soil stratum, irrespective of value of D. Hence, so long as soil profile is not changing, δ will remain constant even with increasing D.

If, however, δ is observed to be changing with D, this implies another soil type has come within the influence of electric currents.

- The change point where δ starts changing with D represents the critical electrode spacing and equal to the depth of the first stratum.
- Further the δ goes on changing as D increases (or more and more of second layer comes into the influencing zone of electric current) until it becomes equal to the resistivity of the second layer.
- The apparent resistivity, δ , increases with D if lower stratum has higher resistivity than upper one; and vice versa.
- The figures below illustrate the above points



(a) Case of increasing resistivity with depth



(b) Case of falling resistivity with depth

Seismic Refraction Method:

- Basic principal involved in this method is that seismic waves have different velocities in different types of soil or rock, and waves get refracted when they cross the boundary between different types of soil.
- This method enables the determination of the general soil profile and approximate depth of boundaries of strata or the bed rock.
- In this method a shock wave is generated either by striking a plate on the soil with a hammer or by exploding a charge at or near the ground surface.
- These radiating shock waves are recorded by a device called geophone, which records the time of travel of the waves.
- These geophones are installed at suitable known distances on the ground in a line from the source of shock.
- Some of the waves, termed direct or primary waves travel directly from the shock-source along the ground surface in the direction of the geophones. While other waves travel in a downward direction at various angles to the horizontal and will be refracted if they pass into a stratum of different seismic velocity.
- These reflected waves travel much faster if the underlying layer is denser.
- As the distance between the shock source and the geophone increases, refracted waves reach geophone earlier than the direct waves.

