

UNIT-I

1.1. Soil Formation:

Generally soils are formed by either i) Physical disintegration
ii) Chemical decomposition

1.1.1. Physical disintegration:

Physical disintegration or mechanical weathering of rocks occurs due to the following physical process.

- 1) Temperature changes: different material of rock have different coefficient of thermal expansion. Unequal expansion or contraction occurs due to temperature changes. When the stresses induced due to such changes are repeated many times, the particles get separated from the rocks and the soils are formed.
- 2) Wedging action of ice: water in the pores and minute cracks of rocks get frozen in very cold climates. As the volume of ice is formed more than that of water, expansion occurs. Because of this rock get broken into pieces when large stresses develop in the cracks due to wedging action of ice formed.
- 3) Spreading of roots and plants: As the roots of trees and shrubs grow in the cracks and fissure of the rocks, forces act on the rock. The segments of rocks are forced apart and disintegration of rocks occurs.
- 4) Abrasion: as water, wind and glaciers move over the surface of rocks, abrasion and scouring takes place. It results in formation of soil.

1.1.2. Chemical decomposition:

When chemical decomposition or chemical weathering of rocks takes place, original rock minerals are transformed into new minerals by chemical reactions.

- 1) Hydration: In hydration, water combines with rock mineral and result in formation of a new chemical compound. The chemical reaction causes changes in volume and decomposition of rock into small particles.
- 2) Carbonation: In this process, the CO_2 in the atmosphere combines with water to form carbonic acid. The carbonic acid react chemically with rocks and causes decomposition
- 3) Oxidation: It occurs when oxygen ions combine with minerals in rocks. Oxidation results in decomposition of rocks.
- 4) Solution: some of the rock minerals form a solution with water when they get dissolved in water. Because of this, chemical reactions take place and soils are formed.
- 5) Hydrolysis: in this process, water get dissociated into H^+ and OH^- ions. The hydrogen cations replace the metallic ions such as Calcium, sodium and potassium in rock mineral and soils are formed with a new chemical composition.

1.2. Transportation of soils:

The soils are formed at a place may be transported to other places because of the water, wind, ice and gravity.

1) Water transported soils:

Flowing water is one of the most important agents of transportation of soils. The size of the soil particles carried by water depends upon the velocity. The swift water can carry the particles of large size such as boulders and gravels. With a decrease in velocity, the coarser particles get deposited. The finer particles are carried further downstream and deposited when the velocity reduces. A delta is formed when the velocity slows down to almost zero at the confluence with a receiving body of still water such as lake, a sea or an ocean.

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

2) Wind transported soils:

Soil particles are transported by winds. The particle size of the soil depends on the velocity of wind. The finer particles are carried far away from the place of the formation. Soil deposits by wind are known as Aeolian deposits.

Large sand dunes are formed by winds. Sand dunes occur in arid regions and on the lee ward side of the sea with sandy beaches.

Loess is a silt deposit made by wind. These deposits have low density and high compressibility. The bearing capacity of such soils is very low. The permeability in vertical direction is large.

3) Glacier- deposited soils:

Glaciers are large masses of ice formed by the compaction of snow. As the glaciers grow and move, they carry with them soils varying in size from fine grained to huge boulders. Soils get mixed with ice and are transported far away from their original position.

4) Gravity- Deposited soils:

Soils can be transported through short distances under the action of gravity. Colluvial soils such as talus have been deposited by the gravity which consists of irregular, coarse particles.

5) Soils transported by combined action:

Sometimes, two or more agents of transportation act jointly and transport the soil.

1.3. Soil Structure:

The geometrical arrangement of soil particles with respect to one another is known as soil structure.

1) Single grained structure: Cohesionless soils, such as gravel and sand are composed of bulky grains in which the gravitational forces are more predominant than surface forces. In this each particle is in contact with those surrounding it. Depending upon the relative position of the particles, the soil may have a loose structure or a dense structure.

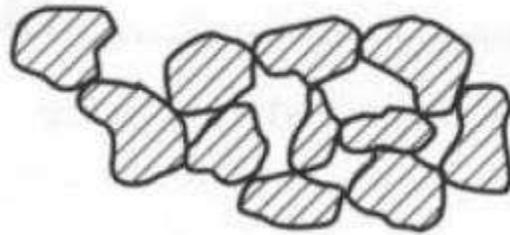


Fig 1.1: Loose Structure

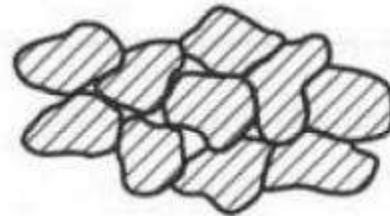


Fig 1.2: Dense Structure

2) Honeycomb structure: Fine sands or silts get deposited such that the particles when settling develop a particle-to-particle contact that bridges over large voids of soil mass. The particles wedge between one another into a stable condition and form a skeleton like arch to carry the weight of overlying material. This structure usually develops when the particles size is between 0.002mm and 0.02mm. the soils in honey comb structure are loose.

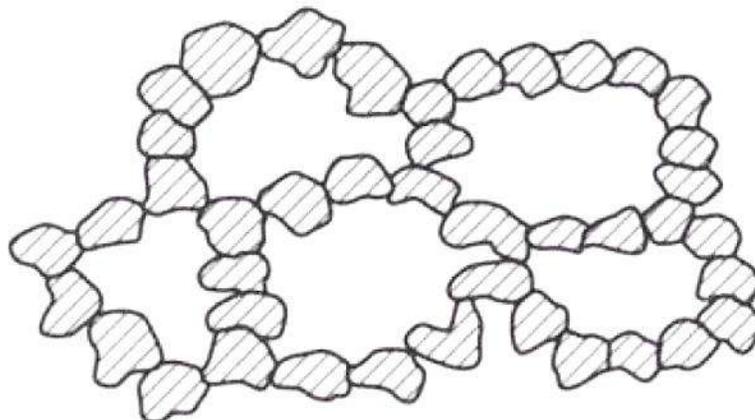


Fig 1.3: Dense Structure

3) Flocculated structure: It occurs in clays and the clay particles have large surface area and, therefore the electric forces are important in such soils. The clay particles have a negative charge on the surface and a positive charge on the edges. Interparticle contact develops between the positively charged edges and negatively charged faces. This results in flocculated structure.

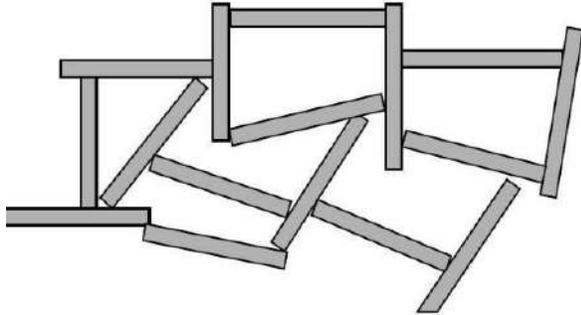


Fig 1.4: Flocculated structure

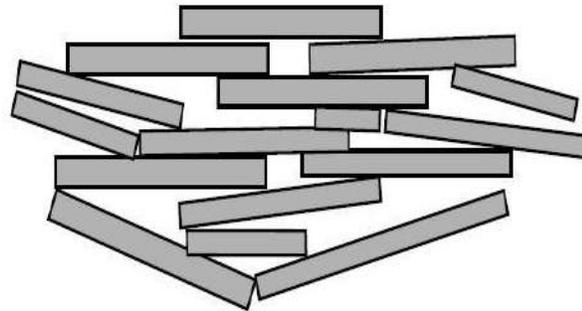


Fig 1.5: Dispersed Structure

When clay particle settles in water, deposits formed have a flocculent structure.

4) Dispersed structure: Dispersed structure develops that have been remoulded. The particles develop more or less a parallel orientation. The clay which is flocculated will be remoulded when it is transported to other places. Remoulding converts edge-face orientation to face-face orientation (Fig 1.5).

5) Coarse grained Skeleton: A coarse grained skeleton is a composed structure which is formed when the soil contains particles of different types. When the amount of bulky, cohesionless particles is large compared with that of fine grained clayey particles, the bulky grains are in particle to particle contact. These particles form a skeleton and space is filled by clayey particles known as binders (Fig 1.6(a)).

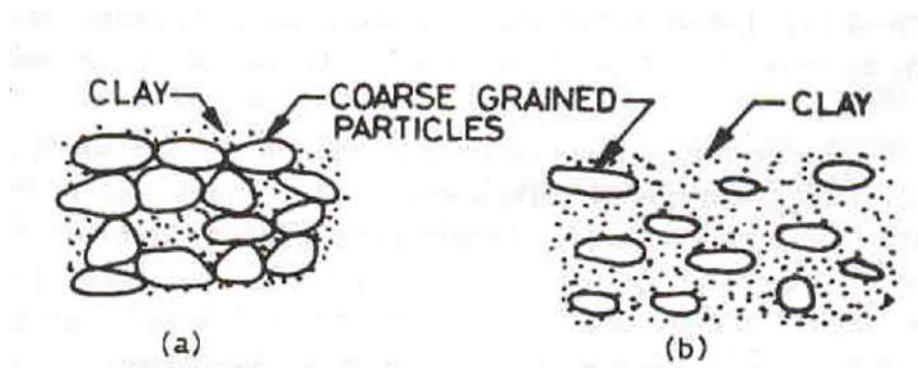


Fig 1.6 composite structure (a) coarse grained skeleton (b) clay matrix

6) Clay Matrix Structure: Clay matrix structure is also composed of different types of soil particles. In this case the amount of clayey particles is larger than coarse grained particles (Fig 1.6 (b)).

1.4. Clay Mineralogy:

Clay minerals are composed of two basic structural units. They are

- 1) Tetrahedral unit
- 2) Octahedral unit
- 3)

1) Tetrahedral unit: A tetrahedral unit consists of a silicon atom (Si^{4++}) surrounded by four oxygen atoms (O^{2-}), forming the shape of a tetrahedron. Oxygen atoms locate at the tips and silicon at its centre (Fig 1.7). An individual tetrahedron unit cannot exist in nature.

2) Octahedral unit: An octahedral consists of six hydroxyls (OH^-) forming a configuration of an octahedron and having one aluminum atom at the centre (Fig 1.8). Because of net negative charge, an octahedral unit cannot exist in isolation. Several octahedral units combine to form a gibbsite sheet.

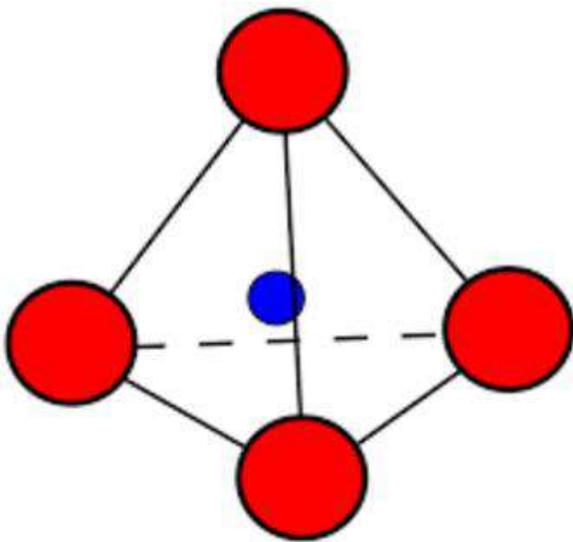


Fig 1.7 Tetrahedral unit

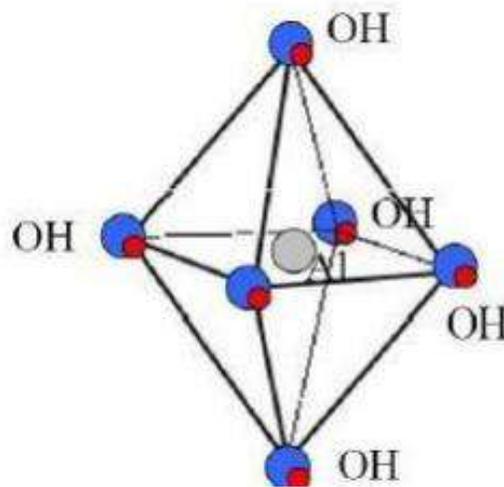


Fig 1.8 Octahedral unit

1.4.1. Kaolinite Mineral:

Its basic structural unit consists of an aluminum sheet (gibbsite) (G) combined with silica sheet (S) (Fig 1.9). Tips of the silica sheet and one base of alumina sheet forms a common interface. The total thickness of structural unit is about 7.2 Angstrom units.

As the bond is fairly strong, the mineral is stable. This mineral is electrically neutral.

Ex: China Clay

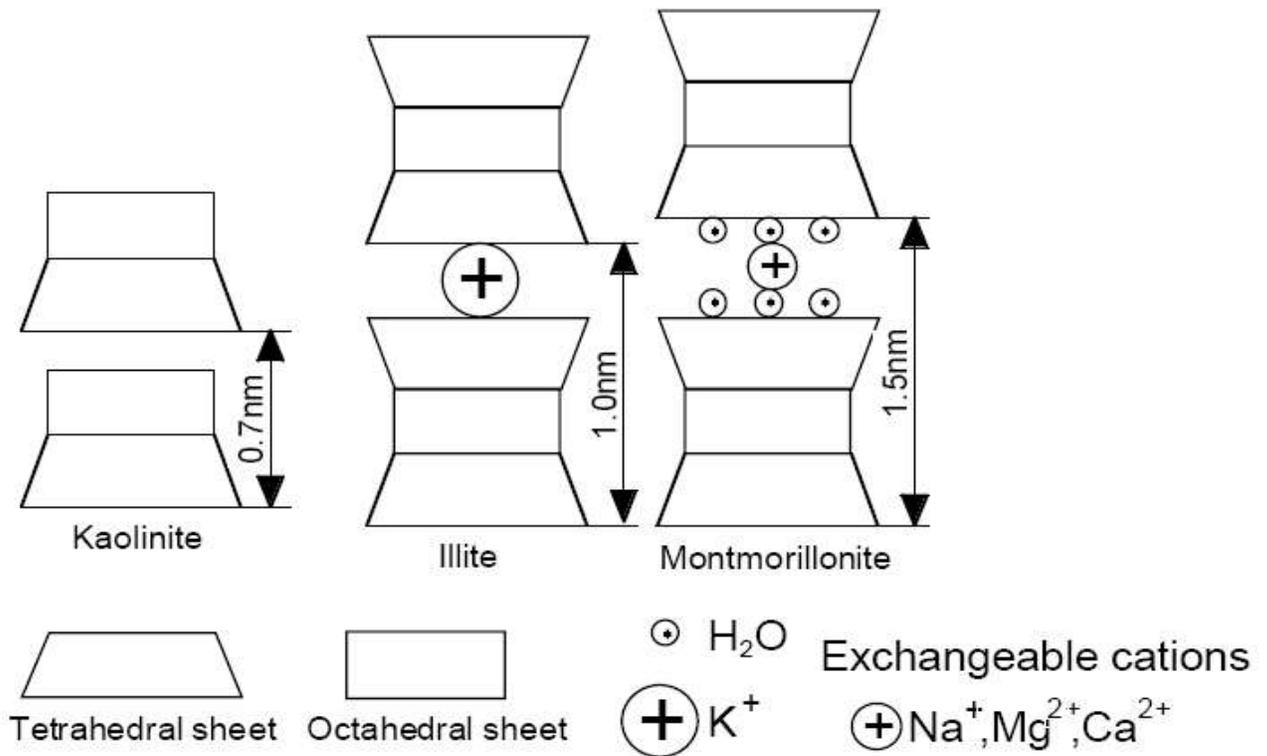


Fig 1.9 Clay Minerals

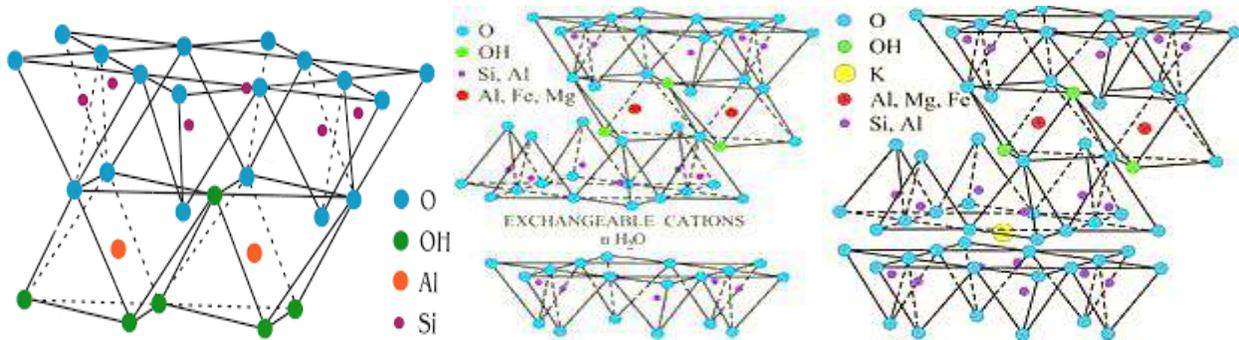


Fig 1.9 (a) Kaolinite

Fig 1.9 (b) Montmorillonite

Fig 1.9 (c) Illite

1.4.2. Montmorillonite:

It consists of an alumina sheet sandwiched between two silica sheets. The thickness of each structural unit is about 10 Angstrom units. Two successive structural units are joined by a link between oxygen ions of silica sheets.

The negatively charged silica sheet attracts water in the space between two structural units. This results in an expansion of the mineral. The soil containing large amount of montmorillonite exhibits high shrinkage and high swelling.

1.4.3. Illite Mineral:

The basic structural unit of illite is similar to montmorillonite except the link between the structural units is through non-exchangeable potassium (K^+) and not water.

The lattice of illite is stronger than montmorillonite and comparatively the swelling is less.

1.4.4. Diffused Double layer:

Diffused double layer (DDL) is the result of clay-water-electrolyte interaction. Cations are held strongly on the negatively charged surface of dry fine-grained soil or clays. These cations are termed as adsorbed cations. Those cations in excess of those needed to neutralize electronegativity of clay particles and associated anions are present as salt precipitates. When dry clays come in contact with water, the precipitates can go into solution. The adsorbed cations would try to diffuse away from the clay surface and tries to equalize the concentration throughout pore water. However, this movement of adsorbed cations are restricted or rather minimized by the negative surface charge of clays. The diffusion tendency of adsorbed cations and electrostatic attraction together would result in cation distribution adjacent to each clay particle in suspension. Fig. 1.10 presents such a distribution of ions adjacent to a single clay particle. The charged clay surface and the distributed ions adjacent to it are together termed as diffuse double layer (DDL). Close to the surface there is high concentration of ions which decreases outwards. Thus there are double layers of ions (a) compressed layer and (b) diffused layer and hence the name double layer. The variation in concentration of cations and anions in pore water with distance from clay surface is also presented in Fig. 1.10. A high concentration of cations close to clay surface gradually reduces, and reaches equilibrium concentration at a distance away from clay surface. For anions, concentration increases with distance from clay surface.

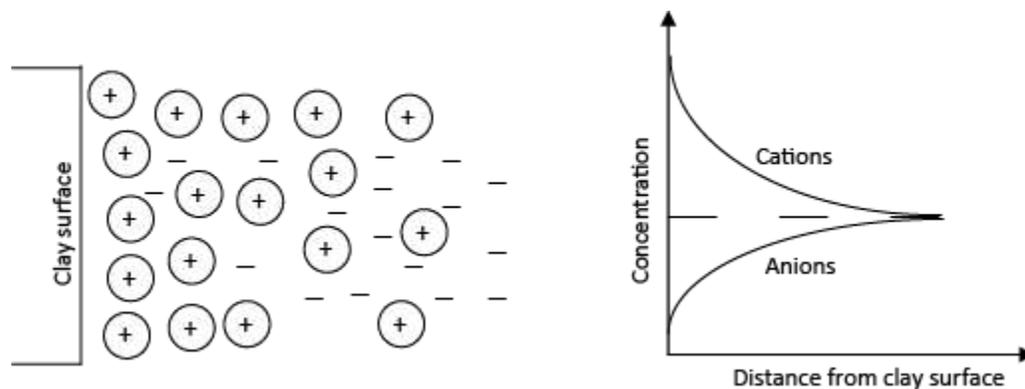


Fig. 1.10 Distribution of ions adjacent to clay surface

1.4.5. Adsorbed Water:

The innermost layer of doublelayer water, which is held very strongly by clay, is known as adsorbed water (Stern layer). This water is more viscous than is free water. The orientation of water around the clay particles gives clay soils their plastic properties.

1.5. Volumetric relationships:

In general, a soil mass consists of solid particles, water and air. To represent the soil in an easier manner the three phase diagram is very useful.

The notations in figure 1.11 represent

V = Volume of soil

V_v = Volume of voids

V_s = Volume of solids

V_a = Volume of air

V_w = Volume of water

W = Weight of soil

W_a = Weight of air

W_w = Weight of water

W_s = Weight of solids

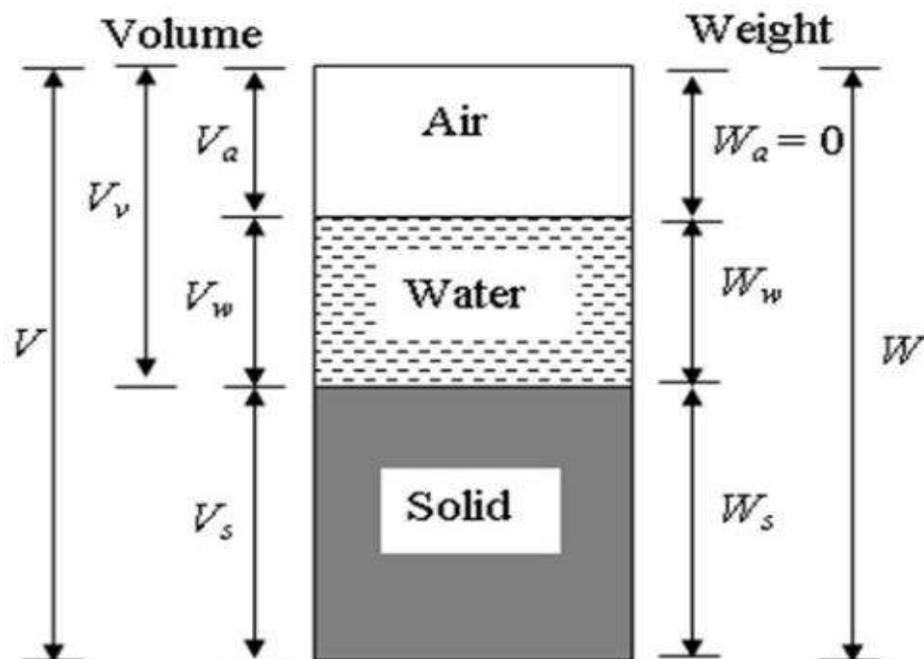


Fig 1. 11 Three Phase Diagram of Soil

The following five volumetric relationships are widely used in geotechnical engineering.

1) **Void Ratio (e):** It is defined as ratio of volume of voids to volume of solids.

$$e = \frac{V_v}{V_s}$$

2) **Porosity (n):** It is defined as ratio of volume of voids to total volume of soil.

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

3) **Degree of saturation (S):** The degree of saturation is the ratio of volume of water to the volume of voids.

$$s = \frac{V_w}{V_v}$$

4) **Percentage air voids (n_a):** It is the ratio of volume of air to the total volume.

$$n_a = \frac{V_a}{V}$$

5) **Air content (a_c):** It is the ratio of volume of air to volume of voids.

$$a_c = \frac{V_a}{V_v}$$

6) **Water content (W):** It is defined as the ratio of mass of water to the mass of solids

$$W = \frac{M_w}{M_s}$$

7) **Bulk mass density (ρ):** it is defined as total mass per unit volume

$$\rho = \frac{M}{V}$$

The bulk density is expressed in kg/m^3

8) **Dry mass density (ρ_d):** It is defined as the mass of solids per unit volume.

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

9) **Saturated mass density (ρ_{sat}):** It is defined as bulk mass density of soil when it is fully saturated.

$$\rho_{sat} = \frac{M_{sat}}{V}$$

10) **Submerged mass density (ρ'):** When the soil exists below water, it is in a submerged condition. The submerged mass density (ρ') of the soil is defined as the submerged mass per unit total volume.

$$\rho' = \frac{M_{sat}}{V}$$

1.5.1 Relationship among various parameters:

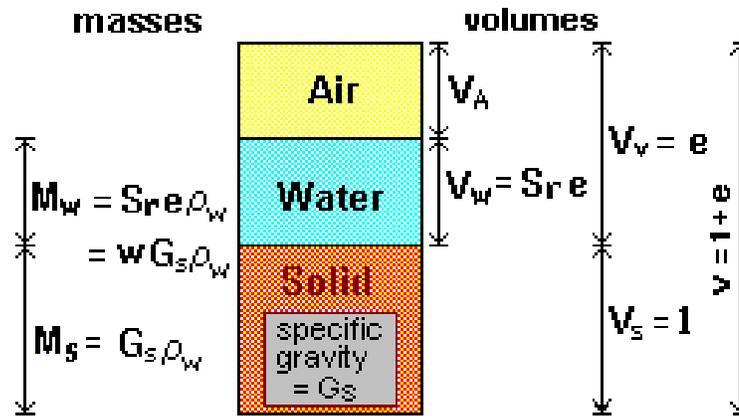


Fig 1.11 (a) Three phase diagram in terms of void ratio

Three Phase Diagram in terms of

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

$$n = \frac{V_v}{V} = \frac{e}{1 + e}$$

$$S = \frac{V_w}{V_v}$$

$$n_a = \frac{V_a}{V}$$

$$a_c = \frac{V_a}{V_v}$$

$$n_a = \frac{V_a}{V_v} \times \frac{V_v}{V} = n a_c$$

For Convenience, the volume of solids is taken as unity

Area of cross-section of soil mass is taken as unity

$$S = \frac{V_w}{V_v} = \frac{e_w}{e}$$

The Volume of water (V_w) = $S e$

Obviously, the volume of air (V_a) = $e - S e = e(1 - S)$ (from figure)

Therefore Percentage of air voids, $(n_a) = \frac{V_a}{V} = \frac{e(1-S)}{1+e}$

Air content, $(a_c) = \frac{V_a}{V_v} = \frac{e(1-S)}{e} = 1 - S$

$$\rho = \frac{M}{V} = \frac{M_s + M_w}{1 + e} = \frac{G\rho_w + Se\rho_w}{1 + e} = \frac{(G + Se)\rho_w}{1 + e}$$

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

$$\rho_{sat} = \frac{M_{sat}}{V}$$

For saturated soil, $S=1$

$$\rho_{sat} = \frac{M_{sat}}{V} = \frac{(G + e)\rho_w}{1 + e}$$

$$\rho' = \rho_{sat} - \rho_w = \frac{(G + e)\rho_w}{1 + e} - \rho_w = \frac{(G + e - 1 - e)\rho_w}{1 + e} = \frac{(G - 1)\rho_w}{1 + e}$$

Incase of soil not fully saturated

$$\rho' = \rho - \rho_w = \frac{(G + Se)\rho_w}{1 + e} - \rho_w = \frac{(G + Se)\rho_w - (1 + e)\rho_w}{1 + e} = \frac{[(G - 1) - e(1 - S)]\rho_w}{1 + e}$$

Relation between the void ratio and water content

$$w = \frac{M_w}{M_s} = \frac{V_w\rho_w}{V_s\rho_s} = \frac{SV_v}{V_sG} = \frac{Se}{G}$$

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

From ρ_d ,

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

$$\rho_d = \frac{\rho}{1 + w}$$

Relation between dry mass density and percentage of air voids:

$$V = V_s + V_w + V_a$$

$$1 = \frac{V_s}{V} + \frac{V_w}{V} + \frac{V_a}{V}$$

$$1 = \frac{V_s}{V} + \frac{V_w}{V} + n_a$$

$$1 - n_a = \frac{V_s}{V} + \frac{V_w}{V}$$

$$1 - n_a = \frac{M_s/G\rho_w}{V} + \frac{M_w/\rho_w}{V} = \frac{\rho_d}{G\rho_w} + \frac{(wM_s)}{V} = \frac{\rho_d}{G\rho_w} + \frac{w\rho_d}{\rho_w} = \left(w + \frac{1}{G}\right) \frac{\rho_d}{\rho_w}$$

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

1.6. Relative density (Density Index) (D_r):

The most important index aggregate property of a cohesionless soil is its relative density. The Engineering property of a mass of cohesionless soil depends largely on its relative density. The relative density defined as

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

e_{max} = maximum void ratio of soil in loosest condition

e_{min} = minimum void ratio of soil in densest condition

ρ_{max} = dry density in dense condition

ρ_{min} = dry density in loose condition

ρ_d = dry density in natural condition

GEOTECHNICAL ENGINEERING

UNIT-II

1. Grain Size Analysis:

The grain size analysis is, also known as particle size analysis, is a method of separation of soil particles into different fractions based on the particle size.

The grain size analysis can be done in two stages. First one is sieve analysis and the second one is sedimentation analysis. The sieve analysis is meant for coarse grained soils ($>75\mu$) and sedimentation analysis meant for fine grained soils ($<75\mu$). As most of the times soil may contain particles of both size, both the analysis are required.

1.1.1 Sieve Analysis:

In this the soil is sieved through set of sieves. The sieves are designated by the size of square opening in mm or microns. Sieves of various sizes ranging from 80mm to 75μ are available.

a) Dry sieve analysis:

- 1) The soil sample has to be taken in suitable quantity, as given in table 1. The larger the particle size, the greater the quantity of the soil required. The soil should be oven dry and it should not contain any lumps.
- 2) The sample is sieved through a 4.75mm IS sieve. The portion retained on the sieve is the gravel fraction.
- 3) The gravel fraction is sieved through the set of coarse sieves manually or using mechanical shaker and weight determined on each sieve has to be determined.
- 4) The minus 4.75mm fraction is sieved through set of fine sieves. Normally 10mins of shaking is enough for most soils. The mass of the soil is measured to nearest 0.1gm.

b) Wet sieve analysis:

If the soil contains a substantial quantity of fines ($>5\%$) a wet sieve analysis is required.

- 1) In this method, all the lumps are broken into individual particles and a representative soil sample in the required quantity is taken.
- 2) The dried sample is taken in a tray and soaked with water. If deflocculation is required sodium-hexameta-phosphate is added.

- 3) The sample is stirred and left for soaking for a period of at least one hour. The slurry is sieved through a 4.75mm IS sieve and washed with jet of water.
- 4) The material passed through 4.75mm is sieved through a 75 μ . The material retained on 75 μ is collected and dried in oven.
- 5) It is then sieved through set of fine sieves.

Stokes' law: Stokes' law states that the velocity at which soil particles settle in a suspension depend on shape, size and weight of particles. Assuming spherical particles and same density,

$$v = \frac{1}{18} D^2 \frac{\gamma_s - \gamma_w}{\eta}$$

Here, v = Terminal velocity of sinking soil particle

D = Diameter of soil particle

γ_s = Unit weight of soil solids

γ_w = Unit weight of water

G = Specific Gravity of Soil Solids

η = Viscosity of water

1.1.2 Hydrometer method:

Hydrometer method is used to determine the particle size distribution of fine-grained soils passing 75 μ sieve. It works based on stokes' law. The hydrometer measures the specific gravity of the soil suspension at the center of its bulb. The specific gravity depends upon the mass of solids present, which in turn depends upon the particle size.

In this process, a hydrometer with a long stem is used. The stem is marked from top to bottom, generally in the range of 0.995 to 1.330.

Procedure:

Part-I: Calibration of hydrometer

1. Take about 800ml of water in one measuring cylinder. Place the cylinder on a table and observe the initial reading.
2. Immerse the hydrometer in the cylinder. Take the reading after the immersion.
3. Determine the volume of the hydrometer (V_H) which is equal to the difference between the final and initial readings. Alternatively weigh the hydrometer to the nearest 0.1g. The volume of the hydrometer in ml is approximately equal to its mass in grams.

4. Determine the area of cross section (A) of the cylinder. It is equal to the volume indicated between any two graduations divided by the distance between them. The distance is measured with an accurate scale.
5. Measure the distance (H) between the neck and the bottom of the bulb. Record it as the height of the bulb (h).
6. Measure the distance (H) between the neck to each marks on the hydrometer (R_h).
7. Determine the effective depth (H_e), corresponding to each of the mark (R_h) as
[Note: the factor V_H/A should not be considered when the hydrometer is not taken out when taking readings after the start of the sedimentation at ½, 1, 2, and 4 minutes.]
8. Draw a calibration curve between H_e and R_h . Alternatively, prepare a table between H_e and R_h . The curve may be used for finding the effective depth H_e corresponding to reading R_h .

Part-II: Meniscus Correction

1. Insert the hydrometer in the measuring cylinder containing about 700ml of water.
2. Take the readings of the hydrometer at the top and at the bottom of the meniscus.
3. Determine the meniscus correction, which is equal to the difference between the two readings.
4. The meniscus correction C_m is positive and is constant for the hydrometer.
5. The observed hydrometer reading R_h' is corrected to obtain the corrected hydrometer reading R_h as

$$R_h = R_h' + C_m$$

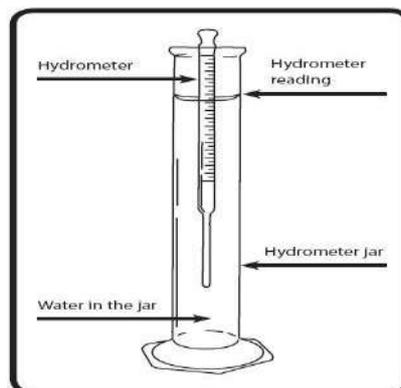


Fig 1.1: Hydrometer Method

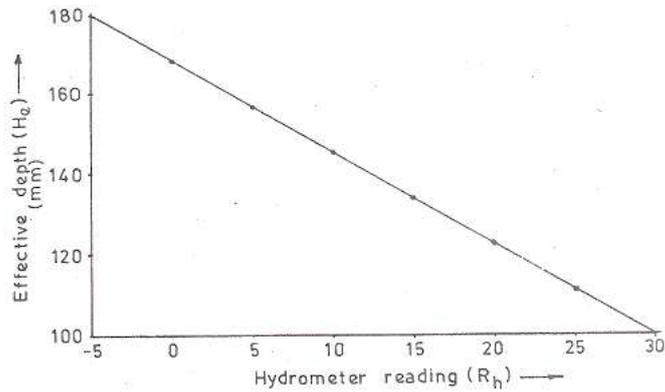


Fig 1.2: Hydrometer Calibration Chart

Part-III: Pretreatment and Dispersion

1. Weigh accurately, to the nearest 0.01g about 50g air-dried soil sample passing 2mm IS sieve, obtained by riffing from the air-dried sample passing 4.75mm IS sieve.

Place the sample in a wide mouthed conical flask.

2. Add about 150ml of hydrogen peroxide to the soil sample in the flask. Stir it gently with a glass rod for a few minutes.

3. Cover the flask with a glass plate and leave it to stand overnight.

4. Heat the mixture in the conical flask gently after keeping it in an evaporating dish. Stir the contents periodically. When vigorous frothing subsides, the reaction is complete. Reduce the volume to 50ml by boiling. Stop heating and cool the contents.

5. If the soil contains insoluble calcium compounds, add about 50ml of hydrochloric acid to the cooled mixture. Stir the solution with a glass rod for a few minutes. Allow it to stand for one hour or so. The solution would have an acid reaction to litmus when the treatment is complete.

6. Filter the mixture and wash it with warm water until the filtrate shows no acid reaction.

7. Transfer the damp soil on the filter and funnel to an evaporating dish using a jet of distilled water. Use the minimum quantity of distilled water.

8. Place the evaporating dish and its contents in an oven and dry it at 105°C to 110° C. transfer the dish to a desiccator and allow it to cool.

9. Take the mass of the oven dried soil after pretreatment and find the loss of mass due to pretreatment.

10. Add 100ml of sodium hexa-metaphosphate solution to the oven – dried soil in the evaporating dish after pretreatment.

11. Warm the mixture gently for about 10minutes.

12. Transfer the mixture to the cup of a mechanical mixture. Use a jet of distilled water to wash all traces of the soil out of the evaporating dish. Use about 150ml of water. Stir the mixture for about 15minutes.

13. Transfer the soil suspension to a 75 μ IS sieve placed on a receiver (pan). Wash the soil on this sieve using a jet of distilled water. Use about 500ml of water.

14. Transfer the soil suspension passing 75 μ IS sieve to a 1000ml measuring cylinder. Add more water to make the volume exactly equal to 1000ml.

15. Collect the material retained on 75 μ IS sieve. Dry it in an oven. Determine its mass. If required, do the sieve analysis of this fraction.

Part-IV: Sedimentation Test

1. Place the rubber bung on the open end of the measuring cylinder containing the soil suspension. Shake it vigorously end-over-end to mix the suspension thoroughly.

2. Remove the bung after the shaking is complete. Place the measuring cylinder on the table and start the stop watch.

3. Immerse the hydrometer gently to a depth slightly below the floating depth, and then allow it to float freely.

4. Take hydrometer reading (R_h) after 1/2, 1, 2 and 4 minutes without removing the hydrometer from the cylinder.

5. Take out the hydrometer from the cylinder, rinse it with distilled water.

6. Float the hydrometer in another cylinder containing only distilled water at the same temperature as that of the test cylinder.

7. Take out the hydrometer from the distilled water cylinder and clean its stem. Insert it in the cylinder containing suspension to take the reading at the total elapsed time interval of 8minutes. About 10 seconds should be taken while taking the reading. Remove the hydrometer, rinse it and place it in the distilled water after reading.

8. Repeat the step (7) to take readings at 15, 30, 60, 120 and 240minutes elapsed time interval.

9. After 240 minutes (4 hours) reading, take readings twice within 24 hours. Exact time of reading should be noted.

10. Record the temperature of the suspension once during the first 15minutes and thereafter at the time of every subsequent reading.

11. After the final reading, pour the suspension in an evaporating dish, dry it in an oven and find its dry mass.

12. Determine the composite correction before the start of the test and also at 30min, 1, 2 and 4 hours. Thereafter just after each reading, composite correction is determined.

13. For the determination of composite correction (C), insert the hydrometer in the comparison cylinder containing 100ml of dispersing agent solution in 1000 ml of distilled water at the same temperature. Take the reading corresponding to the top of meniscus. The negative of the reading is the composite correction.

1.2. Consistency limits (Atterberg Limits):

The term Atterberg limits is named as per the Swedish agriculturist *Albert Atterberg*. When water is added into a soil mass, it changes its state from solid to liquid. He divided the entire range from solid to liquid into four stages:

1. The solid state
2. The semi-solid state
3. The plastic state
4. The liquid state

He set arbitrary limits, known as Atterberg limits or consistency limits, for these division in terms of water content. Thus the Atterberg limits are the water content at which the soil mass passes from one state to the next state. These limits are presented as percentage of moisture present inside the soil. The Atterberg limits which are commonly used for engineering purposes are:

1. Liquid limit
2. Plastic limit
3. Shrinkage limit

Liquid limit (LL):Liquid limit is the water content corresponding to the arbitrary limit between liquid and plastic state of consistency of a soil. It is defined as the minimum water content at which the soil is still in the liquid state, but has a small shearing strength against flowing.

Plastic limit (PL):Plastic limit is the water content corresponding to an arbitrary limit between the plastic and semi-solid states of consistency of a soil. It is defined as the minimum water content at which a soil will just begin to crumble when rolled into a thread approximately 3 mm in diameter.

Shrinkage limit: Shrinkage limit is defined as the maximum water content at which a reduction in water content will not cause a decrease in the volume of a soil mass. It is lowest water content at which a soil can still be completely saturated

Determination of Liquid Limit of Soil & Plastic Limit of Soil (IS: 2720(Part 5)-1985)

The liquid limit and plastic limit of the soil are tested using casagrande apparatus

1.2.1 Test for liquid Limit:

- 1) After receiving the soil sample it is dried in air or in oven (maintained at a temperature of 60°C). If clods are there in soil sample then it is broken with the help of wooden mallet. The soil passing 425 micron sieve is used in this test.
- 2) About 120 gm. of air dried soil from thoroughly mixed portion of material passing 425 micron IS sieve is obtained.
- 3) Distilled water is mixed to the soil thus obtained in a mixing disc to form uniform paste. The paste shall have a consistency that would require 30 to 35 drops of cup to cause closer of standard groove for sufficient length.
- 4) A portion of the paste is placed in the cup of casagrande device and spread into portion with few strokes of spatula.
- 5) It is trimmed to a depth of 1 cm. at the point of maximum thickness and excess of soil is returned to the dish.
- 6) The soil in the cup is divided by the firm strokes of the grooving tool along the diameter through the centre line of the follower so that clean sharp groove of proper dimension is formed.
- 7) Then the cup is dropped by turning crank at the rate of two revolutions per second until two halves of the soil cake come in contact with each other for a length of about 12 mm. by flow only.
- 8) The number of blows required to cause the groove close for about 12 mm. is recorded.
- 9) A representative portion of soil is taken from the cup for water content determination.
- 10) The test is repeated with different moisture contents at least 3 times for blows between 10 and 40.

Calculation:

- 1) A 'flow curve' is to be plotted on a semi-logarithmic graph representing water content in arithmetic scale and the number of drops on logarithmic scale.

- 2) The flow curve is a straight line drawn as nearly as possible through four points
- 3) The moisture content corresponding to 25 blows as read from curve is the liquid limit of that soil

1.2.2. Test for Plastic Limit:

1. A soil sample of 20 gm. passing 425 micron IS sieve is to be taken.
2. It is to be mixed with distilled water thoroughly in the evaporating dish till the soil mass becomes plastic enough to be easily moulded with fingers.
3. It is to be allowed to season for sufficient time, to allow water to permeate throughout the soil mass.
4. 10 gms. of the above plastic mass is to be taken and is to be rolled between fingers and glass plate with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length. The rate of rolling shall be between 60 and 90 stokes per minute.
5. The rolling is to be continued till the thread becomes 3 mm. in diameter.
6. The soil is then kneaded together to a uniform mass and rolled again.
7. The process is to be continued until the thread crumbled with the diameter of 3 mm.
8. The pieces of the crumbled thread are to be collected in a air tight container for moisture content determination.
9. The Plastic limit is to be determined for at least three portions of soil passing 425 micron IS sieve. The average of the results calculated to the nearest whole number is to be reported as the plastic limit of the soil.

1.2.3. Plasticity Index (PI or I_p)

The plasticity index (PI) is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. The PI is the difference between the liquid limit and the plastic limit ($PI = LL - PL$). Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 tend to have little or no silt or clay.

$$I_p = LL - PL$$

1.2.4. Liquidity Index (LI or I_l)

The liquidity index of soil indicates the nearness of water content to its liquid limit

$$I_l = \frac{w - w_p}{I_p} \times 100$$

It is also known as water-plasticity ratio.

1.2.5. Consistency Index (CI or I_c):

It shows the nearness of the soil to its plastic limit.

$$I_c = \frac{w_l - w}{I_p} \times 100$$

1.2.6 Flow Index:

It is the slope of flow curve obtained between number of blows and the water content is casagrande's method of determination of liquid limit as shown in fig

$$I_f = \frac{w_1 - w_2}{\log_{10} \frac{N_2}{N_1}}$$

N_1 = Number of blows at water content of w_1

N_2 = number of blows at water content of w_2

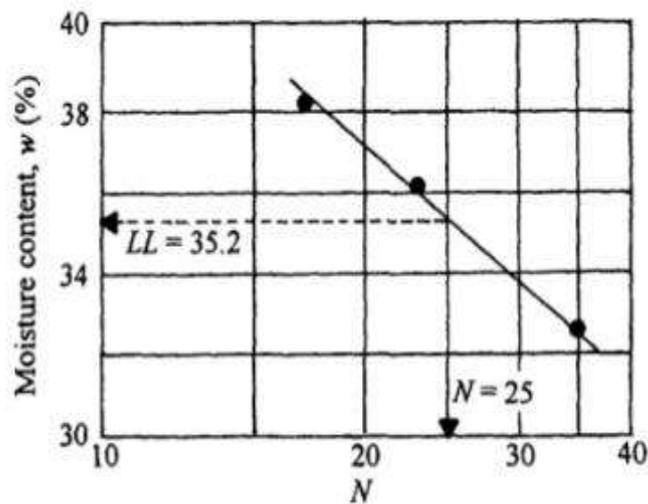


Fig 1.3 Graph to determine flow index

1.2.7. Toughness Index:

The ratio between the index of plasticity and the flow index of a soil is known as toughness index.

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

Toughness index is a measure of soil shear strength at the plastic limit.

1.2.8. Determination of Shrinkage limit of Soil (IS: 2720(Part 6)-1985)

1. 100 gm. of soil sample from a thoroughly mixed portion of the material passing through 425 micron IS sieve is taken.
2. About 30 gm. of above soil sample is placed in the evaporating dish and thoroughly mixed with distilled water to make a paste.
3. The weight of the clean empty shrinkage dish is determined and recorded.
4. The dish is filled in three layers by placing approximately 1/3rd of the amount of wet soil with the help of spatula.
5. Then the dish with wet soil is weighed and recorded immediately.
6. The wet soil cake is air dried until the colour of the pat turns from dark to light. Then it is oven dried at a temperature of 105^o C to 110^o C for 12 to 16 hours. The weight of the dish with dry sample is determined and recorded. Then the weight of oven dry soil pat is calculated (W_0).
7. The shrinkage dish is placed in the evaporating dish and the dish is filled with mercury, till it overflows slightly. Then it is be pressed with plain glass plate firmly on its top to remove excess mercury. The mercury from the shrinkage dish is poured into a measuring jar and the volume of the shrinkage dish is calculated. This volume is recorded as the volume of the wet soil pat (V).
8. A glass cup is placed in a suitable large container and the glass cup removed by covering the cup with glass plate with prongs and pressing it. The outside of the glass cup is wiped to remove the adhering mercury. Then it is placed in the evaporating dish which is clean and empty.
9. Then the oven dried soil pat is placed on the surface of the mercury in the cup and pressed by means of the glass plate with prongs, the displaced mercury being collected in the evaporating dish.
10. The mercury so displaced by the dry soil pat is weighed and its volume (V_0) is calculated by dividing this weight by unit weight of mercury.

$$\text{Shrinkage limit } (W_s) = W \frac{V - V_0}{w_0} \times 100$$

11. The test is repeated at least 3 times for each soil sample and the average of the result is reported.

1.3. Unified soil classification system:

It was originally developed by Casagrande (1948) and was known as airfield classification system. It was adopted with some modification by the U.S. Bureau of Reclamation and the U.S. Corps of Engineers. This system is based on both grain size and plasticity characteristics of soil. The major symbols used are given in table 1.1.

Table 1.1 Symbols used in USC system

	Symbols	Description
Primary	G	Gravel
	S	Sand
	M	Silt
	C	Clay
	O	Organic
	Pt	Peat
Secondary	W	Well-graded
	P	Poorly-graded
	M	Non-plastic fines
	C	Plastic fines
	L	Low plasticity
	H	High plasticity

- 1) Coarse-grained soils: If 50 % or more of the total material by weight retained on 75 μ m IS sieve. There are 8 groups of soils come into this category
- 2) Fine-grained soils: If more than 50 % of the total material passes 75 μ m IS sieve. There are 6 groups of soils come into this category

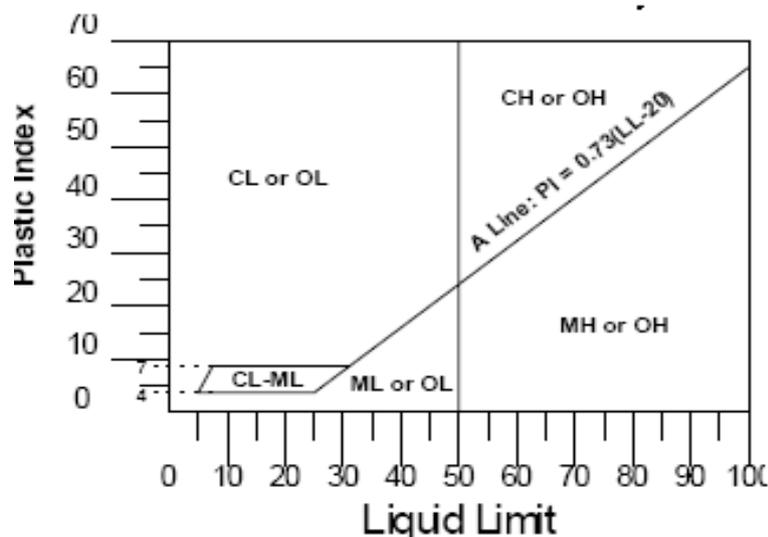


Fig 1.4: Plasticity chart for USC

Table 1.2. Unified soil classification system

Major Division			Group Symbols	Typical names	Classification Criteria				
Coarse-Grained Soils (more than 50% retained on 75 μ)	Gravel (50% or more of coarse fraction retained on 4.75 mm)	Clean Gravels	GW	Well graded gravels	Percentage of fines a) Less than 5% passing 75 μ , GW, GP, SW, SP b) More than 12% passing No. 200, GM, GC, SM, SC c) 5 to 12% passing use of dual symbol as GW-GM	$C_u > 4$, $C_c = 1$ to 3			
			GP	Poorly graded gravels		Not meeting both criteria for GW			
		Gravels with fines	GM	Silty gravels		Atterberg limits below A-line or PI less than 4	Atterberg limits in hatched area GM-GC		
			GC	Clayey gravels		Atterberg limits above A-line and PI greater than 7			
	Sand (more than 50% of coarse fraction passing 4.75 mm)	Clean sands	SW	Well graded sand		$C_u > 4$, $C_c = 1$ to 3			
			SP	Poorly graded sand		Not meeting both criteria for GW			
		sands with fines	SM	Silty sand		Atterberg limits below A-line or PI less than 4	Atterberg limits in hatched area SM-SC		
			SC	Clayey sand		Atterberg limits above A-line and PI greater than 7			
			Fine Grained Soils (50% or more passing through 75 μ sieve)	Silts and clays Liquid limit 50% or less		ML		Inorganic silts of low plasticity	See plasticity chart
						CL		Inorganic clays of low to medium plasticity	
OL	organic silts of low plasticity								
Silts and clays Liquid limit greater than 50%	MH	Inorganic silts of high plasticity							
	CH	Inorganic clays of high plasticity							
	OH	organic silts of medium to high plasticity							
	Highly organic soils	Pt		Peat, muck and other highly organic soils	Visual-Manual Identification				

1.4 Indian Standard Soil Classification System:

Indian Standard Soil Classification system is in many respects similar to the Unified system. However, there is one basic difference in the classification of fine-grained soils. The fine-grained soils in this system are sub-divided into three categories of low, medium and high compressibility instead of two categories of low and high compressibility in Unified soil classification system.

Soils are divided into three broad divisions:

- 1) Coarse-grained soils, when 50 % or more of the total material by weight retained on 75 μm IS sieve.
- 2) Fine-grained soils, when more than 50 % of the total material passes 75 μm IS sieve.
- 3) If the soil is highly organic and contains a large percentage of organic matter and particles of decomposed vegetation, it is kept in a separate category marked as peat (Pt).

In all, there are 18 groups of soils

Coarse-grained soils – 8 groups

Fine-grained soils – 9 groups

Peat - 1 group

1. Coarse-Grained Soils: The classification of coarse-grained soils is done on the basis of their grain and gradation characteristics as illustrated in Table, when the fines (75 μm) present in them are less than 5 % by weight. Coarse-grained soils are sub-divided into gravel and sand. The soil is termed gravel (G) when more than 50 % of coarse fraction (>75 μm) is retained on 4.75 mm IS sieve, and termed sand (S) if more than 50 % of coarse fraction is smaller than 4.75 mm IS sieve.

These are further sub-divided as given in into 8 groups. Coarse-grained soils which contain more than 12 % fines (< 75 μm) are classified as GM or SM if fines are silty in character (meaning, the limits plot below the A-line on the plasticity chart). On the other hand they are classified as GC or SC if the fines are clayey in character (meaning the limits plot above the A-line on the plasticity chart).

2. Fine-grained Soils: The fine-grained soils are classified on the basis of their plasticity characteristics using the I.S Plasticity chart shown in Fig. The fine-grained soils are further divided into three sub-divisions depending upon the values of the liquid limit:

- a) Silts and clays of low compressibility – These soils have a liquid limit less than 35 % (represented by a symbol L)
- b) Silts and clays of medium compressibility – These soils have a liquid limit > 35 % but < 50 % (represented by a symbol I)
- c) Silts and clays of high compressibility – These soils have a liquid limit > 50 % (represented by a symbol H).

Fine-grained soils are further sub-divided, as given in table in 9 groups.

Boundary Classifications: Sometimes, it is not possible to classify a soil into any one of 18 groups discussed above. A soil may possess characteristics of two groups, either in particle distribution or in plasticity. For such cases, boundary classifications occur and dual symbols are used.

a) Boundary classification for coarse-grained soils: Coarse-grained soils having 5 % to 12 % fines are borderline cases and given a dual symbol. While giving dual symbols, first assume a coarse soil and then a finer soil i.e., the first part of the symbol is indicative of the gradation of coarse fraction, while the second part indicates the nature of fines. For example, a soil with the dual symbol SW-SC is well-graded sand with ‘clayey’ fines that plot above A-line.

(i) Boundary classifications within gravel or sand groups can occur. The following classifications are common GW-GP, GM-GC, GW-GM, GW-GC, GP-GM, SW-SP, SM-SC, SW-SM, SW-SC, and SP-SM.

(ii) Boundary classifications can occur between the gravel and sand groups such as GW-SW, GP-SP, GM-SM, and GC-SC Note: The rule for correct classification is to favour the non-plastic classification. For eg. A gravel with 10 % fines, $C_u = 20$ and $C_c = 2$ and $IP = 6$ will be classified as GW – GM, and not GW – GC.

b) Boundary classification for fine-grained soils: Fine-grained soils also can have dual symbols. (i) If the limits plot in the hatched zone on the plasticity chart, i.e., IP between 4 and 7, the soil has a group symbol CL – ML. (ii) If the position of the soil on the plasticity chart falls close to the A-line, dual symbol is used, such as CI – MI , CH – MH (iii) If the liquid limit is very close to 35 % or 50 %, dual symbols are used, such as ML – MI, MI – MH, CL – CI, CI – CH, OL – OI, OI – OH.

c) Boundary classification between coarse-grained and fine-grained soils: Dual symbols can also be used when the soils have about equal percentage of coarse – grained and fine – grained fractions. The possible dual symbols in this case are GM – ML, GM – MI, GM – MH, GC – CL, GC – CI, GC – CH, SM – ML, SM – MI, SM – MH, SC – CL, SC – CI, SC – CH.

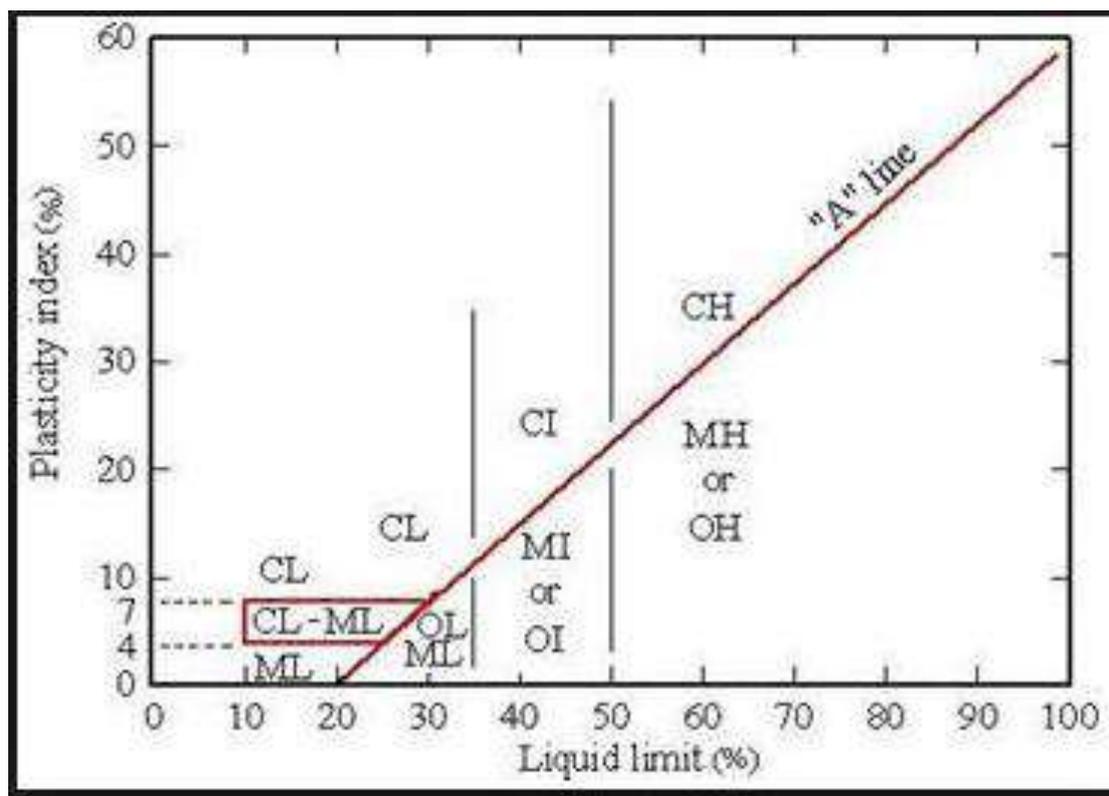


Fig 1.5: Plasticity chart for IS classification

Table1.3: Classification of coarse grained soils (ISC System)

Division	Sub Division		Group Symbols	Typical names	Laboratory Criteria	
Coarse-Grained Soils (more than 50% retained on 75 microns)	Gravel (50% or more of coarse fraction retained on 4.75 mm)	Clean Gravels (Fines less than 5%)	GW	Well graded gravels	$C_u > 4$, $C_c = 1$ to 3	
			GP	Poorly graded gravels	Not meeting all gradation requirements for GW	
		Gravels with appreciable amount of fines (Fines more than 12%)	GM	Silty gravels	Atterberg limits below A-line or PI less than 4	Atterberg limits plotting above A-line with PI between 4 to 7 or border line cases requiring use of dual symbol GM-GC
			GC	clayey gravels	Atterberg limits above A-line and PI greater than 7	
	Sand (more than 50% of coarse fraction passing 4.75 mm)	Clean sands (Fines less than 5%)	SW	Well graded sand	$C_u > 6$, $C_c = 1$ to 3	
			SP	Poorly graded sand	Not meeting all gradation requirements for SW	
		Sand with appreciable amount of fines (Fines more than 12%)	SM	Silty sand	Atterberg limits below A-line or PI less than 4	Atterberg limits plotting above A-line with PI between 4 to 7 or border line cases requiring use of dual symbol SM-SC
			SC	Clayey sand	Atterberg limits above A-line and PI greater than 7	

Table1.4: Classification of fine grained soils (ISC System)

Division	Sub Divison	Group Symbols	Typical names	Laboratory Criteria	
Fine-Grained Soils (more than 50% pass 75 microns)	Low Compressibility (L) (LL < 35%)	ML	Inorganic silts with none to low plasticity	Atterberg limits plot below A-line or PL < 7	Atterberg limits plot above A-line with PL between 4 to 7 ML-CL
		CL	Inorganic clays of low plasticity	Atterberg limits plot above A-line or PL > 7	
		OL	organic silts of low plasticity	Atterberg limits plot below A-Line	
	Intermediate compressibility (I) (35% > LL < 50%)	MI	Inorganic silts of medium plasticity	Atterberg limits plot below A-Line	
		CI	Inorganic clays of medium plasticity	Atterberg limits plot above A-Line	
		OI	organic silts of medium plasticity	Atterberg limits plot below A-Line	
	Highly compressibility (H) (LL > 50%)	MH	Inorganic silts of high compressibility	Atterberg limits plot below A-Line	
		CH	Inorganic clays of high plasticity	Atterberg limits plot above A-Line	
		OH	Organic clays of medium to high plasticity	Atterberg limits plot below A-Line	
	Highly organic soils		Pt	Peat and other highly organic soils	Readily identified by colour, odour, spongy feel and fibrous texture

See Plasticity chart

Soil Water:

There are different types of soil water. Classification of soil water can be done based on two criteria; one based on adhesive property between water and solids and the other based on how water is held in the pores formed by soil solids.

Based on adhesive property between water and solids, the soil water is classified into 'Adsorbed water' and 'Pore water'. Adsorbed water is the thin layer of water formed around the solids. This is very close to the solid particles and is attracted towards them due to the adhesion between water and solids. Since adsorbed water is bound to the solids, it is known as bound water. Pore water is the one which is held in the pores of soil mass. As this is free to move, it is known as free water. Pore water has same properties as that of ordinary water, whereas the adsorbed water has high boiling point, low freezing point, high viscosity, high surface tension.

Based on how water is held in the pores of soil, it is classified into 'hygroscopic water' and 'capillary water'. Hygroscopic water is the one that is held in the pores of soil due to Gravity. The water below the ground water table could be considered as hygroscopic water. Capillary water is the one that is held in the pores of soil due to the surface tension and capillary action. If a glass tube is inserted in a beaker containing water, the water rises into the glass tube till a particular height. If the water in the beaker is similar to hygroscopic water of soils, then the water in the glass tube could be taken to be similar to the capillary water of soils.

In a natural soil deposit, the Zone below the ground water table is known as 'Zone of Saturation' and the one above the ground water table is known as 'Zone of Aeration'. The zone of capillary rise is called 'Capillary Fringe'.

Permeability:

Flow takes place in a pipe from a point of higher energy to a point of lower energy. Since soil is permeable due to the presence of interconnected voids, flow can take place in soils too when there is a difference in energy. The property of soil to allow water to pass through the interconnected voids is known as Permeability of soil.

The study of permeability of soil is important because the measure of permeability is required to calculate the quantity of underground seepage under various seepage conditions, analyze problems involving the pumping of water for underground constructions, and carry out stability analysis of earthen dam and earth retaining structures subjected to seepage forces.

Darcy's Law:

The flow of water through soil is governed by Darcy's law. In 1856, Darcy demonstrated experimentally that for laminar flow in a homogeneous soil, the velocity of flow is given by

$$v = ki$$

Where k = coefficient of permeability, i = hydraulic gradient

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

The above equation is known as Darcy's law, which is one of the corner stones of soil engineering. The discharge q is obtained by multiplying the velocity of flow by the cross sectional area of soil A normal to the direction of flow. Thus,

$$q = vA = kiA$$

The area A includes both the solids and the voids.

The coefficient of permeability can be defined by the above equation. If the hydraulic gradient is unity, the coefficient of permeability is equal to the velocity of flow. In other words, the coefficient of permeability is defined as the velocity of flow which would occur under unit hydraulic gradient. The coefficient of permeability has the dimensions of velocity [L/T]. It is measured in mm/sec, cm/sec, m/sec, m/day or other velocity units.

Table1. Typical Values of the coefficient of permeability

Sl No	Soil type	Coefficient of permeability	Drainage properties
1.	Clean Gravel	10^{+1} to 10^{+2}	Very good
2.	Coarse and Medium sands	10^{-2} to 10^{+1}	Good
3.	Fine Sands, Loose silt	10^{-4} to 10^{-2}	Fair
4.	Dense Silts, Clayey silts	10^{-5} to 10^{-4}	Poor
5.	Silty Clay, Clay	10^{-8} to 10^{-5}	Very Poor

DETERMINATION OF COEFFICIENT OF PERMEABILITY:

The coefficient of permeability of a soil can be determined using the following methods.

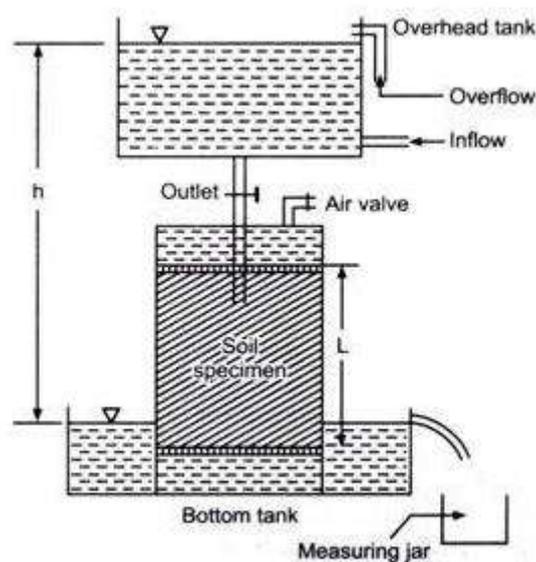
- a) Laboratory Methods: Two methods are available
 - i) Constant Head permeability test
 - ii) Variable Head Permeability test

The instruments used are known as permeameters.

- b) Field Methods: Coefficient of permeability is determined from in-situ soil condition
 - i) Pumping out tests
 - ii) Pumping in tests
- c) Indirect Methods: Coefficient of permeability of soil can also be determined indirectly from soil parameters
 - i) Computation from the particle size or its specific surface
 - ii) Computation from the consolidation test data.
- d) Capillarity Permeability test: Coefficient of permeability of an unsaturated soil can be determined by capillarity permeability test.

Constant Head Permeability Test:

The Coefficient of permeability of a relatively more permeable soil can be determined in a laboratory by the constant head permeability test. The test is conducted in an instrument known as constant head permeameter. It consists of a metallic mould, 100mm internal diameter, 127.3mm effective height and 1000ml capacity according to IS:2720(Part XVII). The mould is provided with a detachable extension collar, 100mm diameter and 60mm high, required during compaction of soil. The mould is provided with a drainage base plate with a recess for porous stone. The mould is fitted with a drainage cap having an inlet valve and an air release valve. The drainage base and cap have fittings for clamping to the mould.



Constant head permeability test

Figure shows a schematic sketch. The soil sample is placed inside the mould between two porous discs. The porous discs should be at least ten times more permeable than the soil. The porous discs should be deaired before these are placed in the mould. The water tubes should be deaired. The sample can also be prepared in the permeameter by pouring the soil into it and tamping it to obtain the required density. The base is provided with a dummy plate, 12mm thick and 108mm in diameter, which is used when the sample is compacted in the mould.

In this test, as the name itself indicates, a constant head of water is maintained throughout the test. This is done by supplying water continuously through a source and having an arrangement for overflow at the top level. The soil sample is placed in mould and the mould facilitates the easy flow of water through the porous stones apart from preventing the solids to flow. The soil can be either undisturbed or compacted at a density at which coefficient of permeability is required. Before the test is conducted, the sample is saturated. For saturation, a very high head of water is maintained for a very long time and the water is allowed to get into the voids of the soil. Till the voids are filled, the flow of water in outlet would be unsteady. A steady flow of water at the outlet is an indication of saturation of sample. Once the soil is saturated, the test can be carried out and measuring the volume of water collected in the jar in a given time.

As per Darcy's Law; $q = Aki$

If V is the volume of water collected in time t, A is the cross sectional area of soil sample, h is the constant head maintained throughout the test and L is the length of the sample, then

$$\frac{V}{t} = Ak \frac{h}{L}$$

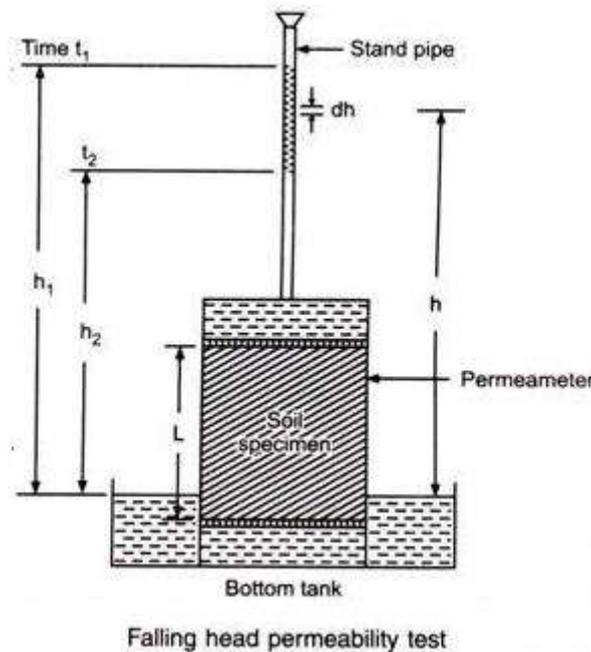
$$k = \frac{V}{t} \frac{1}{A} \frac{L}{h}$$

Falling Head Permeability test

If a soil with a very low coefficient of permeability (Eg: Clay) is subjected to constant head permeability test, the volume of water collected in a given time is too low quantity. As per theory of errors, the less the size of sample, the greater the error involved. Hence, in such cases, the coefficient of permeability is erroneously determined in constant head method. However, the falling head method could be used to find permeability to such soils.

Unlike the constant head test, a continuous source of water supply is not required in this test. A stand pipe is used for maintaining the head here. At time t_1 , the head h_1 , is noted and depending on the coefficient of permeability of soil, after some time say time t_2 , the head h_2 is noted down.

As constant head is not maintained, Darcy's law cannot be directly applied for this case. However, integration can be resorted to, by considering a small strip of water column between h_1 and h_2 at a height h , the thickness of the strip being dh . Let dt be the short time taken by water to pass through the strip and a be the area of cross section of the stand pipe, then the small discharge in small period of time is $dh/dt(a)$.



Since this is happening in a small period of time, the head h , can be assumed to be constant. Applying Darcy's law,

$$q = Aki$$

$$-a \frac{dh}{dt} = Ak \frac{h}{L}$$

Negative sign is because as t increases, h decreases

$$-\frac{dh}{h} = \frac{Ak}{aL} dt$$

Integrating on both sides,

$$\int_{h_1}^{h_2} -\frac{dh}{h} = \frac{Ak}{aL} \int_{t_1}^{t_2} dt$$

$$- [\log_e(h)]_{h_1}^{h_2} = \frac{Ak}{aL} [t]_{t_1}^{t_2}$$

$$- (\log_e h_2 - \log_e h_1) = \frac{Ak}{aL} (t_2 - t_1)$$

$$k = \frac{aL}{A(t_2 - t_1)} \log_e \left(\frac{h_1}{h_2} \right)$$

If $t_2 - t_1$ is taken as the time period t , for head to drop from h_1 to h_2

$$k = \frac{2.303aL}{At} \log_{10} \left(\frac{h_1}{h_2} \right)$$

Factors affecting permeability:

$$k = C \frac{\gamma_w}{\mu} \left(\frac{e^3}{1+e} \right) D^2$$

The following are the factors that affect the permeability of soils

1. **Particle Size:** From the above equation, the coefficient of permeability is proportional to the square of the particle size (D). Permeability of coarse grained soils is very large as compared to that of the fine grained soils. The permeability of coarse sand may be more than one million times as much that of clay.
2. **Structure of the soil mass:** The coefficient C in above equation takes into account the shape of flow passage. The size of the flow passage depends upon the structural arrangement. For the same void ratio, the permeability is more in the case of flocculated structure as compared to that of dispersed structure.

Stratified soil deposits have greater permeability parallel to the plane of stratification than that perpendicular to this plane. Permeability of a soil deposit also depends upon the shrinkage cracks, joints, fissures and shear zones. Loose deposits have greater permeability in the vertical direction than in the horizontal direction.

The permeability of a natural soil deposit should be determined in undisturbed condition; the disturbance caused during sampling may destroy the original structure and affect the permeability. The effect of disturbance is more pronounced in the case of fine grained soils than in the case of coarse grained soils.

3. **Shape of the particle:** The permeability of a soil depends upon the shape of the particles. Angular particles have greater specific surface area as compared to the rounded particles. For the same void ratio, the soils with angular particles are less permeable than those with rounded particles, as the permeability is inversely proportional to the specific surface. However, in a natural deposit the void ratio for a soil with angular particles may be greater than that for rounded particles, the soil with angular particles may be actually more permeable.
4. **Void Ratio:** From the above equation, the coefficient of permeability varies as $e^3/(1 + e)$. For a given soil, the greater the void ratio, the higher is the value of coefficient of permeability. Based on the other concepts, it has been established that the permeability of soil varies as e^2 or $e^2/(1 + e)$. whatever may be the exact relationship, all soils have e versus $\log k$ plot as a straight line.

The permeability of a soil at a given void ratio may not have any relationship with that of another soil at the same void ratio. Paradoxically, the soils with the largest void ratio are the least pervious. This is due to the fact that the individual void passages in clays are extremely small through which water cannot flow easily.

If the permeability of a soil at a void ratio of 0.85 is known, its value at another void ratio of e can be determined using following equation given by casagrande

$$k = 1.4k_{0.85}e^2$$

Where $k_{0.85}$ = permeability at a void ratio of 0.85

k = permeability at a void ratio of e

5. **Properties of water:** As indicated in equation, the coefficient of permeability is directly proportional to the unit weight of water and is inversely proportional to its viscosity. The unit weight of water doesnot vary much over the range of temperature ordinarily encountered in soil engineering problems. However, there is a large variation in the value of coefficient of viscosity. The coefficient of permeability increases with an increase in temperature due to the reduction in viscosity.

It is usual practice to report the k at 27°C . The following equation can be used for conversion of the permeability to 27°C

$$k_{27} = k_t \frac{\mu_t}{\mu_{27}}$$

6. **Degree of saturation:** if the soil is not fully saturated, it contains air pockets formed due to entrapped air or due to the air liberated from percolating water. Whatever may be the cause of presence of the air in soils, the permeability is reduced due to the

presence of air which causes blockage of the passage. Consequently, the permeability of a partially saturated soil is considerably smaller than that of a fully saturated soil. In fact, Darcy's law is not strictly applicable to those soils.

7. **Adsorbed Water:** The fine grained soils have a layer of adsorbed water strongly attached to their surface. This adsorbed water layer is not free to move under gravity. It causes an obstruction to flow of water in the pores and hence reduces the permeability of soils.
8. **Impurities in water:** Any foreign matter in water has a tendency to plug the flow passage and reduces the effective voids and hence loses the permeability of soils.

ONE-DIMENSIONAL FLOW:

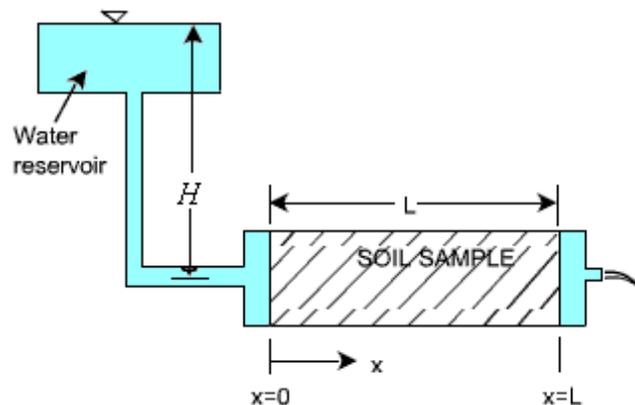
For this, the **Laplace Equation** is $\frac{d^2h}{dx^2} = 0$

Integrating twice, a general solution is obtained.

$$\frac{dh}{dx} = c_1$$

$$h = c_2 + c_1x$$

The values of constants can be determined from the specific boundary conditions.



As shown, at $x = 0$, $h = H$, and at $x = L$, $h = 0$

Substituting and solving,

$$c_2 = H, c_1 = -\frac{H}{L}$$

The specific solution for flow in the above permeameter is

$$h = H - \frac{H}{L}x$$

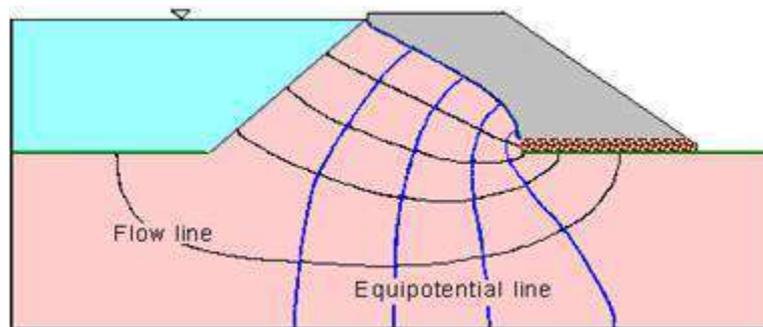
Which states that head is dissipated in a linearly uniform manner over the entire length of the permeameter.

Two-dimensional Flow:

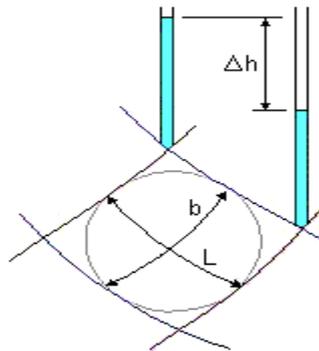
Flow Nets Graphical form of solutions to **Laplace equation** for two-dimensional seepage can be presented as flow nets. Two orthogonal sets of curves form a flow net:

- **Equipotential lines** connecting points of equal total head **h**
- **Flow lines** indicating the direction of seepage down a hydraulic gradient

Two flow lines can never meet and similarly, two equipotential lines can never meet. The space between two adjacent flow lines is known as a **flow channel**, and the figure formed on the flownet between any two adjacent flow lines and two adjacent equipotential lines is referred to as a **field**. Seepage through an embankment dam is shown.



Calculation of flow in a channel If standpipe piezometers were inserted into the ground with their tips on a single equipotential line, then the water would rise to the same level in each standpipe. The pore pressures would be different because of their different elevations. There can be no flow along an equipotential line as there is no hydraulic gradient.



Consider a field of length **L** within a flow channel. There is a fall of total head **Δh**. The average hydraulic gradient is

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

As the flow lines are **b** apart and considering unit length perpendicular to field, the flow rate is

$$\Delta q = kb \frac{\Delta h}{L}$$

There is an advantage in sketching flow nets in the form of **curvilinear 'squares'** so that a circle can be inscribed within each four-sided figure bounded by two equipotential lines and two flow lines.

In such a square, $b = L$, and the flow rate is obtained as $\Delta q = k \cdot \Delta h$. Thus the flow rate through such a flow channel is the permeability k multiplied by the uniform interval between adjacent equipotential lines.

Calculation of total flow For a complete problem, the flow net can be drawn with the overall head drop h divided into N_d so that $\Delta h = h / N_d$. If N_f is the no. of flow channels, then the total flow rate is

$$q = \Delta q N_f = kh \frac{N_f}{N_d}$$

UNIT-III

EFFECTIVE STRESS & STRESS DISTRIBUTION IN SOIL

CONCEPT OF STRESSES:

In a loaded soil mass which is below water, there are two types of stresses that act within a saturated soil mass.

- i. Effective stress
- ii. Neutral stress

Effective stress: It is also known as inter granular pressure. It is transmitted to soil grains through their point of contact of the interconnected particles of soil. It decreases the void ratio. It changes shearing resistance. Compression takes place under this stress. The effective stress is represented by $\bar{\sigma}$ or σ'

Neutral stress: It is also known as pore water pressure. It is transmitted to the soil base through the pore water. It does not decrease the void ratio. It does not change shearing resistance. Compression does not take place under this stress. It is represented by u .

The effective stress cannot be measured directly in the laboratory. It is deduced from total stress and pore water pressure.

$$\bar{\sigma} \text{ or } \sigma' = \sigma - u$$

Where $\bar{\sigma}$ or σ' = effective stress

σ = total stress

u = pore water pressure

3.1. Geostatic stress:

When the ground surface is horizontal and the properties of the soil do not change along a horizontal plane the stresses due to self weight are known as geostatic stresses.

In such case, the stresses are normal to the horizontal and vertical planes, and there is no shear stress on these planes.

a) Vertical stresses:

Let us consider the horizontal plan A-A at a depth below the ground surface.

Let the area of cross section of prism be A . If the unit weight of soil (γ) is considered the vertical stress (σ_z)

$$\sigma_z = \frac{\text{weight of the soil}}{\text{area of base}} = \frac{\gamma Z A}{A} = \gamma Z$$

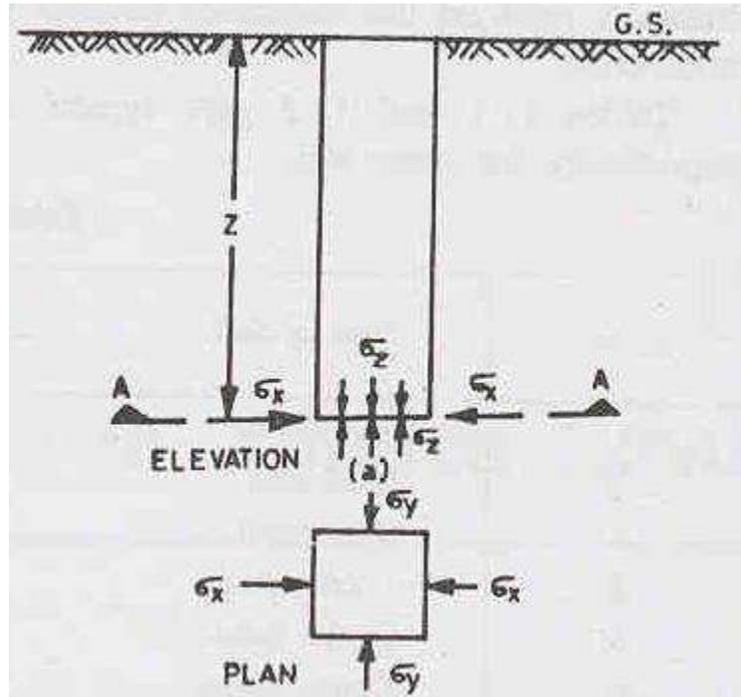


Fig 1: Geostatic stress

If the soil is stratified, having n layers

$$\sigma_z = \sum_{i=1}^n \gamma_i Z_i$$

In natural deposits generally the density of the soil increases with an increase in depth due to weight of soil above

In such cases

$$\sigma_z = \int_0^{Z_1} \gamma_1 dz + \int_0^{Z_2} \gamma_2 dz + \int_0^{Z_3} \gamma_3 dz + \dots + \int_0^{Z_n} \gamma_n dz$$

b) Horizontal stresses:

The horizontal stresses (σ_x and σ_y) act on vertical as shown in fig.

The horizontal stresses at a point in a soil mass are highly variable. These depend not only upon the vertical stresses, but also on the type of soil and on the conditions where soil is stretched or compressed laterally. Here it assumed that $\sigma_x = \sigma_y$

The ratio of horizontal stress (σ_x and σ_y) to the vertical stresses (σ_z) is known as coefficient of lateral stress.

$$\text{Thus } K = \frac{\sigma_x}{\sigma_z}$$

$$\sigma_x = K\sigma_z$$

In natural deposits, generally there is no lateral strain. The lateral stress coefficient for this case is known as the coefficient of lateral pressure at rest (K_0). The value of its coefficient can be obtained from the theory of elasticity, as explained below.

The strain in x-direction is given by

$$\epsilon_x = \frac{1}{E} [\sigma_x - \nu(\sigma_y + \sigma_z)]$$

For condition of no lateral strain $\epsilon_x = 0$

Thus

$$\sigma_x = \nu(\sigma_y + \sigma_z)$$

Taking

$$\sigma_x(1 - \nu) = \nu\sigma_z$$

$$\sigma_x = \frac{\nu\sigma_z}{(1 - \nu)}$$

But we know that

$$\sigma_x = K_0\sigma_z$$

So

$$K_0 = \frac{\nu}{1 - \nu}$$

The value of ' K_0 ' is determined if the Poisson's ratio is known.

3.2. Vertical stresses due to a point load:

3.2.1. Boussinesq's Solution:

Boussinesq gave the theoretical solutions for the stress distribution in an elastic medium subjected to a concentrated load on its surface. The following assumptions are made while

- 1) The soil mass is an elastic continuum, having a constant value of modulus of elasticity.
- 2) The soil is homogenous
- 3) The soil is isotropic
- 4) The soil mass is semi-infinite
- 5) The soil is weightless and is free from residual stresses before the application of the load

Fig 2 shows a horizontal surface of the elastic continuum subjected to a point load Q at a point O. the origin of the coordinates is taken at O.

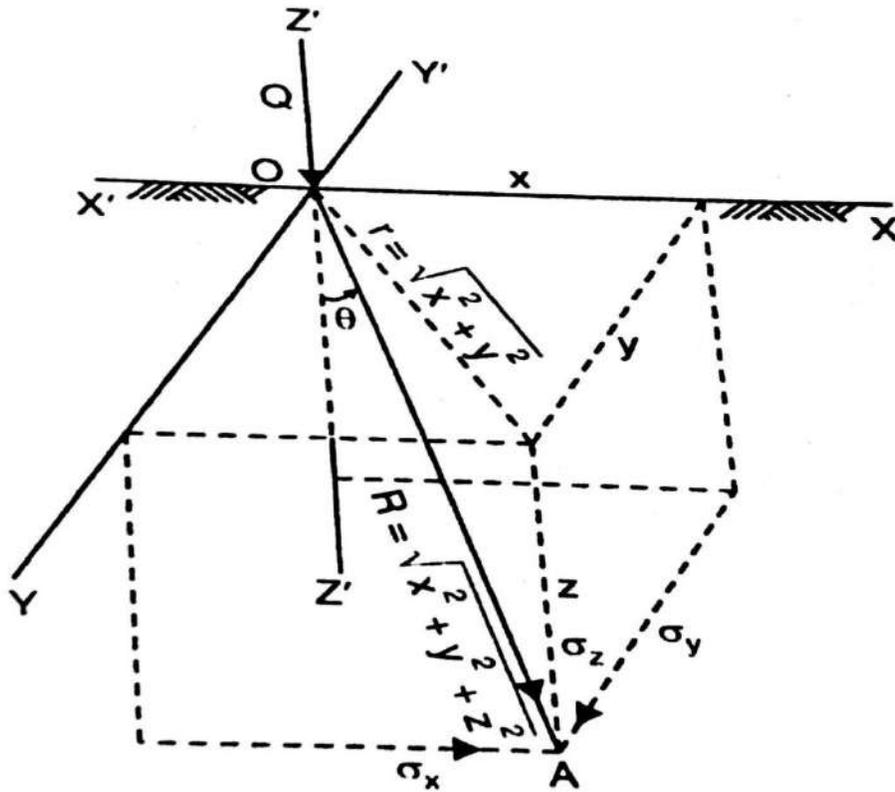


Fig 2: Stresses due to a concentrate load

Boussinesq proved that the polar stress σ_R at a point P(X,Y,Z) is given by

$$\sigma_R = \frac{3}{2\pi} \frac{Q \cos\phi}{R^2}$$

Where R= Polar distance b/w the origin O and point P.

ϕ = angle which the line OA makes with the vertical

As shown in the fig

$$R = \sqrt{(x^2 + y^2 + z^2)}$$

But from the figure

$$R = \sqrt{(r^2 + z^2)}$$

$$\text{Here } \sin\phi = \frac{r}{R} \text{ and } \cos\phi = \frac{z}{R}$$

The vertical stress (σ_z) at point p is given by

$$\begin{aligned}
 \sigma_z &= \sigma_R \cos^2 \phi \\
 &= \frac{3}{2\pi} \frac{Q \cos \phi}{R^2} \cos^2 \phi \\
 &= \frac{3}{2\pi} \frac{Q \cos^3 \phi}{R^2} \\
 \sigma_z &= \frac{3Q}{2\pi} \cdot \frac{\left(\frac{Z}{R}\right)^3}{R^2} \\
 \sigma_z &= \frac{3Q}{2\pi} \cdot \frac{(Z)^3}{R^5} \\
 \sigma_z &= \frac{3Q}{2\pi} \cdot \frac{Z^5}{R^5} \cdot \frac{1}{Z^2} \\
 \sigma_z &= \frac{3Q}{2\pi} \cdot \frac{Z^5}{(r^2 + z^2)^{5/2}} \cdot \frac{1}{Z^2} \\
 \sigma_z &= \frac{3Q}{2\pi} \cdot \frac{1}{Z^2} \cdot \frac{1}{\left(1 + \left(\frac{r}{z}\right)^2\right)^{5/2}} \\
 \sigma_z &= I_B \cdot \frac{(Q)}{Z^2}
 \end{aligned}$$

I_B is known as the boussinesq influence coefficient for the vertical stress.

Isobar diagram:

An isobar is a curve joining all points of equal stress intensity.

$$\sigma_z = I_B \cdot \frac{(Q)}{Z^2}$$

Taking $\sigma_z = 0.1Q$

$$0.1Q = I_B \cdot \frac{(Q)}{Z^2}$$

$$I_B = 0.1 Z^2$$

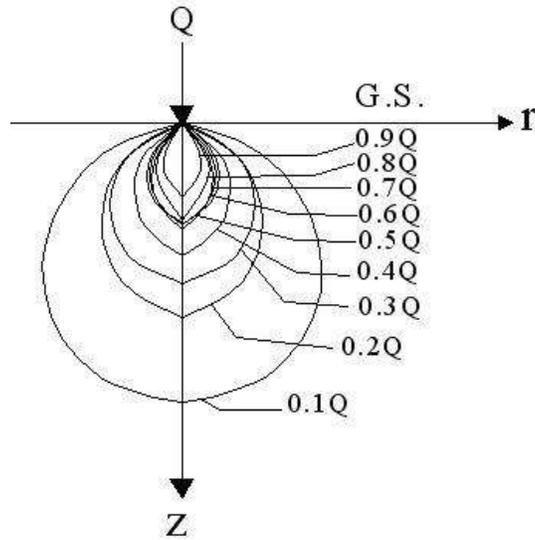


Fig 3: Isobars

Table 1: calculations for Isobar for 0.1Q

Depth z (units)	Influence coefficients K_B	r/z	r (units)	σ_z
0.5	0.0250	1.501	0.750	0.1Q
1.0	0.1000	0.932	0.932	0.1Q
1.5	0.2550	0.593	0.890	0.1Q
2.0	0.4000	0.271	0.542	0.1Q
2.185	0.4775	0	0	0.1Q

3.2.2. Westergaard's Solution:

Boussinesq solution assumes that the soil deposit is isotropic. Actual sedimentary deposits are generally anisotropic. There are generally thin layers of sand embedded in homogeneous clay strata. Westergaard's solution assumes that there are thin sheets of rigid materials sandwiched in a homogeneous soil mass. These thin sheets are closely spaced and are of infinite rigidity and are, therefore, incompressible. These permit only downward displacement of the soil mass as a whole without any lateral displacement. Therefore, Westergaard's solution represents more closely the actual sedimentary deposits.

According to Westergaard, the vertical stress at a point P at a depth below the concentrated load Q is given by

$$\sigma_z = \frac{\left(\frac{c}{2\pi}\right) \frac{Q}{Z^2}}{\left(c^2 + \left(\frac{r}{cz}\right)^2\right)^{\frac{3}{2}}}$$

Where c is depends on the poissons ratio

$$C = \sqrt{\frac{1-2\nu}{2-2\nu}}$$

For an elastic material, the value of ν varies from 0.0 to 0.50.

For the case when the $C = \frac{1}{\sqrt{2}}$ where $\nu=0$ the equation can be written as

$$\sigma_z = \frac{1}{\pi(1+2\left(\frac{r}{z}\right)^2)^{\frac{3}{2}}} \frac{Q}{z^2}$$

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

Where I_w is known as westergaard influence coefficient.

3.3. Vertical stress due to line load

Let the vertical line load of intensity q per unit length acting along the surface of a semi-infinite soil mass as shown in fig.

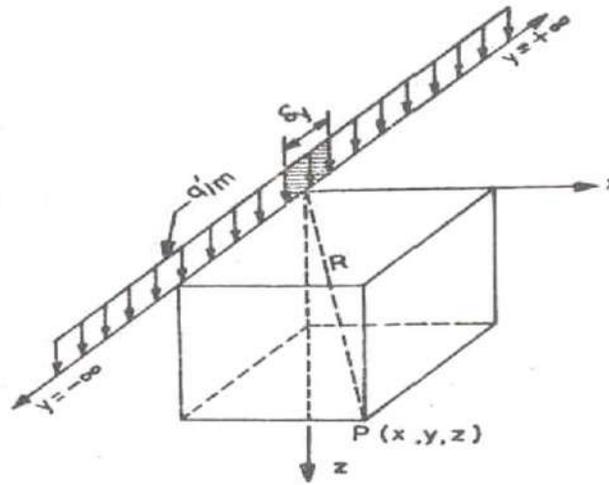


Fig 4: Line load on soil

Let us consider the load acting on a small length δy . The load can be taken as $q' \times \delta y$

$$\Delta\sigma_z = \frac{3q'\delta y}{2\pi} \cdot \frac{z^3}{(r^2 + z^2)^{\frac{5}{2}}}$$

$$\sigma_z = \frac{3q'}{2\pi} z^3 \cdot \int_{-\infty}^{+\infty} \frac{dy}{(r^2 + z^2)^{\frac{5}{2}}}$$

$$\sigma_z = \frac{3q'}{2\pi} z^3 \cdot \int_{-\infty}^{+\infty} \frac{dy}{(x^2 + y^2 + z^2)^{\frac{5}{2}}}$$

Substituting $x^2 + z^2 = u^2$

$$\sigma_z = \frac{3q'}{2\pi} z^3 \cdot \int_{-\infty}^{+\infty} \frac{dy}{(u^2 + y^2)^{\frac{5}{2}}}$$

Let $y = u \tan\theta$ $dy = u \sec^2\theta d\theta$

$$\sigma_z = \frac{3q'}{2\pi} z^3 \cdot \int_0^{\pi/2} \frac{u \sec^2\theta}{u^5 \sec^5\theta} d\theta$$

$$\sigma_z = \frac{3q'}{2\pi u^4} z^3 \cdot \int_0^{\pi/2} \cos^3\theta d\theta$$

Let $\sin\theta = t$ then $\cos\theta d\theta = dt$

$$\sigma_z = \frac{3q'}{\pi u^4} z^3 \cdot \int_0^1 (1 - t^2) dt$$

$$\sigma_z = \frac{3q'}{\pi u^4} z^3 \cdot \frac{2}{3} = \frac{2q' z^3}{\pi(x^2 + z^2)^2}$$

$$\sigma_z = \frac{2q'}{\pi z} \cdot \left(\frac{1}{1 + \left(\frac{x}{z}\right)^2} \right)^2$$

Above equation can be used to determine the vertical stress at point P.

When the point P lies vertically below the line load, $x=0$

$$\sigma_z = \frac{2q'}{\pi z}$$

$$\sigma_x = \frac{2q' z x^2}{\pi(x^2 + z^2)^2}$$

$$\tau_{xz} = \frac{2q' x z^2}{\pi(x^2 + z^2)^2}$$

3.4. Vertical stresses under a circular area

The loads applied to the soil surfaces by footings are not concentrated loads. These are usually spread over a finite area of footing. It is generally assumed that the footing is flexible and contact pressure is uniform. In other words the load is assumed to be uniformly distributed over the area of the base of the footing.

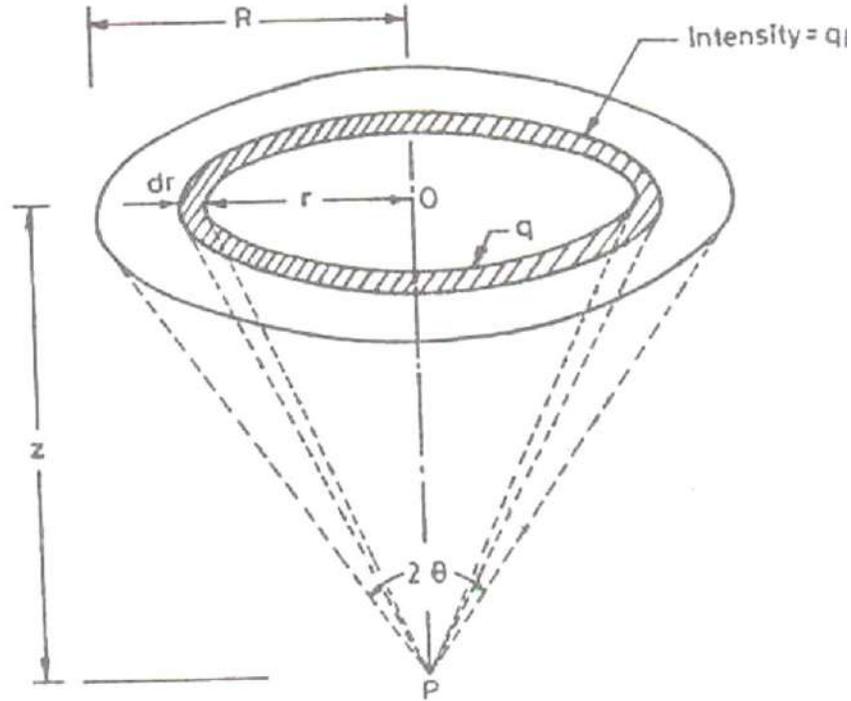


Fig 5: Circular Load

Let us determine the vertical stress at point P at depth Z below the centre of a uniformly loaded circular area. Let the intensity of the load be q per unit area and R be the radius of the loaded area. Boussinesq's solution can be used to determine σ_z . The load on the elementary ring of radius r and width dr is equal to $q \cdot 2\pi r dr$. The load acts at a constant radial distance r from the point P . From Boussinesq's equation for point load

$$\Delta\sigma_z = \frac{3(q \cdot 2\pi r dr)\delta y}{2\pi} \cdot \frac{1}{\left(1 + \left(\frac{r}{z}\right)^2\right)^{\frac{5}{2}}} \cdot \frac{1}{z^2}$$

$$\sigma_z = 3qz^3 \cdot \int_0^R \frac{r dr}{(r^2 + z^2)^{\frac{5}{2}}}$$

Let $r^2 + z^2 = u$ then $2r dr = du$

$$\sigma_z = 3qz^3 \cdot \int_{z^2}^{(R^2+z^2)} \frac{du}{2u^{\frac{5}{2}}}$$

$$= 3qz^3 \cdot \left(-\frac{2}{3}\right) \left[u^{-3/2}\right]_{z^2}^{(R^2+z^2)}$$

$$= qz^3 \left[\frac{1}{z^3} - \frac{1}{(R^2 + z^2)^{3/2}} \right]$$

$$\sigma_z = q \left[1 - \left\{ \frac{1}{1 + (R/z)^2} \right\}^{3/2} \right]$$

$$\sigma_z = I_c q$$

$$I_c = \left[1 - \left\{ \frac{1}{1 + (R/z)^2} \right\}^{3/2} \right]$$

$$\tan \theta = R/z$$

$$I_c = \left[1 - \left\{ \frac{1}{1 + \tan^2 \theta} \right\}^{3/2} \right]$$

$$I_c = \left[1 - \{ \cos^2 \theta \}^{3/2} \right] = 1 - \cos^3 \theta$$

3.5. Vertical stress at any point under rectangular area

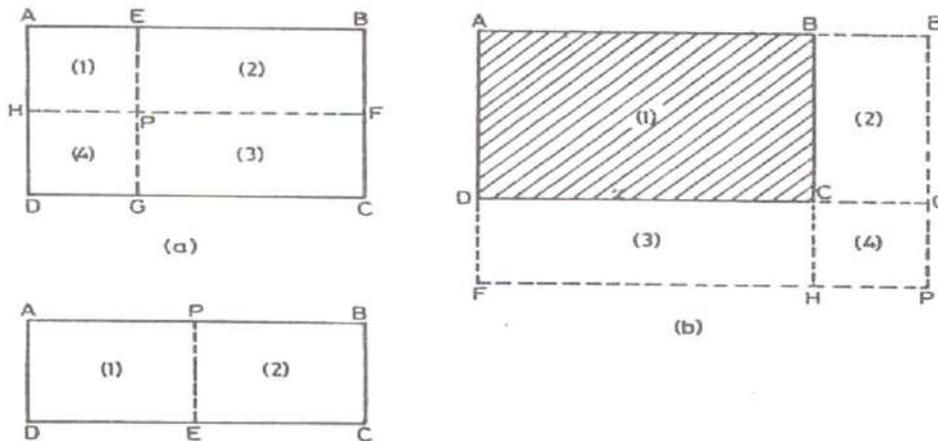


Fig 6: Rectangular Load

1) Point anywhere below the rectangular area.

Fig a shows the location of the point P, below the rectangular area ABCD. The given rectangle is subdivided into 4 small rectangles AEPH, EBFP, HPGD and PFCG, each having one corner at P. The vertical stress at P due to the given rectangular load is equal to that from small rectangles.

Therefore, using $\sigma_z = I_N q$

$$\sigma_z = q[(I_N)_1 + (I_N)_2 + (I_N)_3 + (I_N)_4]$$

Where $(I_N)_1, (I_N)_2, (I_N)_3$ and $(I_N)_4$ are Newmark's influence factors obtained from table for the four rectangles marked (1), (2), (3) and (4)

For the special case, when the point P is at the centre of the rectangle ABCD, all the four small rectangles are equal and equation becomes

$$\sigma_z = 4I_N$$

Where I_N is the influence factor for the small rectangle.

2) Point outside the loaded area.

fig b shows the point P outside the loaded area ABCD. In this case, a large rectangle AEPF is drawn with its one corner at P.

Now rectangle ABCD = rectangle AEPF - rectangle BEPH- rectangle DGPF + rectangle CGPH

The last rectangle CGPH is given plus sign because this area has been deduced twice, once in rectangle BEPH and once in DGPF.

Stress at point P due to a load on rectangle ABCD is given by

$$\sigma_z = q[(I_N)_1 - (I_N)_2 - (I_N)_3 + (I_N)_4]$$

Where $(I_N)_1, (I_N)_2, (I_N)_3$ and $(I_N)_4$ are Newmark's influence coefficients for rectangles AEPF, BEPH, DGPF, CGPH.

3) Point below the edge of the loaded area

If the point P is below the edge of the loaded area ABCD, the given rectangle is divided into two small rectangles APED and PBCE.

$$\sigma_z = q[(I_N)_1 + (I_N)_2]$$

Where, $(I_N)_1$ and $(I_N)_2$, are Newmark's influence coefficient for rectangles APED and PBCE respectively.

3.6. Newmark's Influence chart:

The methods for the determination of vertical stresses under a strip, a circular and a rectangular area have been discussed in the preceding sections. In practice, sometimes one has to find the vertical stresses under a uniform loaded area of other shapes. For such cases, Newmark's influence charts are extremely useful.

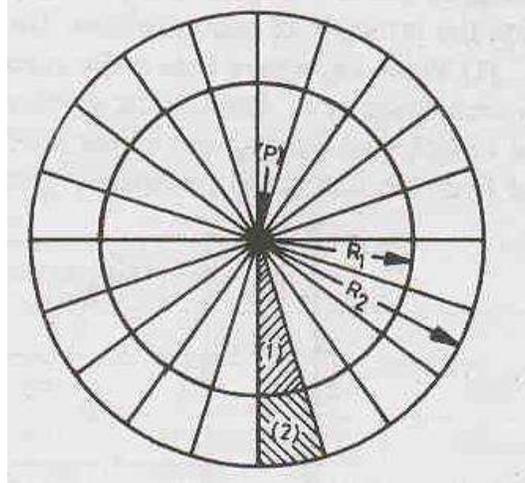


Fig 7: Concentric circles for R_1 and R_2

Newmark's chart is based on the concept of the vertical stress below the center of the circular area. Let us consider a uniformly loaded circular area of radius R_1 , divided into 20 equal sectors. The vertical stress at a point P at depth z just below the centre of loaded area due to load on one sector will be $1/20$ of that due to load on full circle.

$$\sigma_z = \frac{1}{20} q \left[1 - \left\{ \frac{1}{1 + (R_1/z)^2} \right\}^{3/2} \right]$$

If the vertical stress (σ_z) is given an arbitrary fixed value, say $0.005q$, Equation becomes

$$0.005q = \frac{1}{20} q \left[1 - \left\{ \frac{1}{1 + (R_1/z)^2} \right\}^{3/2} \right]$$

Solving Equation $\frac{R_1}{z} = 0.270$

Thus every one-twentieth sector of the circle, with a radius R_1 equal to $0.270Z$, would give a vertical stress $0.005q$ at its centre.

Similarly solving $\frac{R_2}{z} = 0.270$

Use of Newmark's chart

The chart can be used to determine the vertical stress at point P below the loaded area. A plan of the loaded area is drawn on a tracing paper to a scale such that length AB(=2cm in this case) is

equal to depth(z) of the point P below the surface. For example, if the pressure is required at a depth of 1m, the plan should be drawn to a scale 2cm= 1, or R.F=1/50. The traced plan of the loaded area is placed over the Newmark chart such that the point P at which the pressure is required coincides with center of the chart. The vertical stress at point P is given by

$$\sigma_z = I \times N \times q$$

N=number of small area units covered by the plan

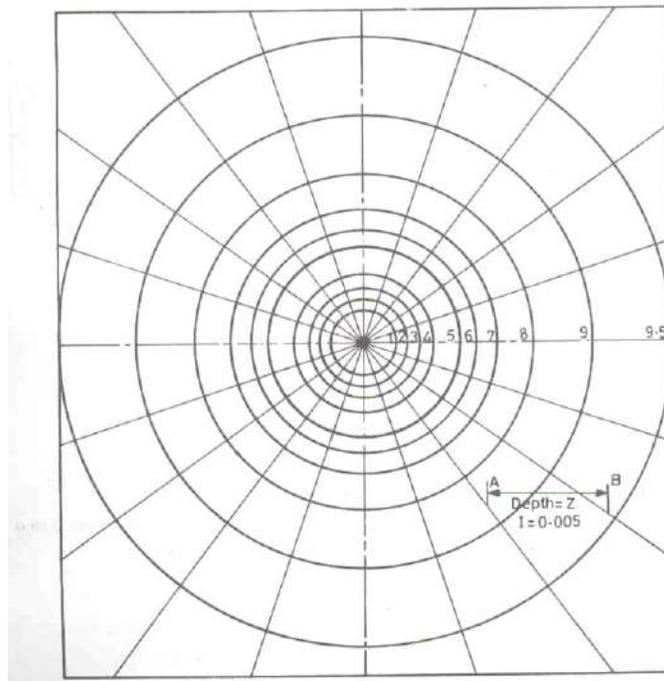


Fig 8: Newmark's influence chart

GEOTECHNICAL ENGINEERING-I

UNIT-IV – COMPACTION

Objective: To know about the compaction technique.

Syllabus: Compaction

Mechanism of compaction – factors affecting – effects of compaction on soil properties – Methods of compaction used in field.

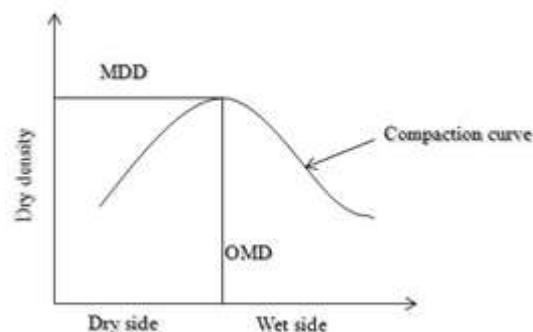
Outcomes:

Student will be able to

- Know the importance of compaction in real life
- Idealize the terms optimum moisture content and maximum dry density variations corresponding to the various factors
- Know about the methods available in practical to achieve the compaction in the field

LEARNING MATERIAL

Soil, in its natural state provides support to the footings carrying loads from structures. Apart from this, it is also extensively used as construction material. The reason for using as construction material are its easy availability and low cost. Whenever soil is used as a construction material, it should be in its densest state. *The densest state can be achieved by artificial process of bringing solids close to each other and reducing void ratio using mechanical energy is termed as COMPACTION.* The compaction results in increased density and shear strength whereas decrease in compressibility and permeability.



The term compaction is very often confused with the term consolidation in the civil engineering field. The points of comparison between compaction and consolidation are listed below

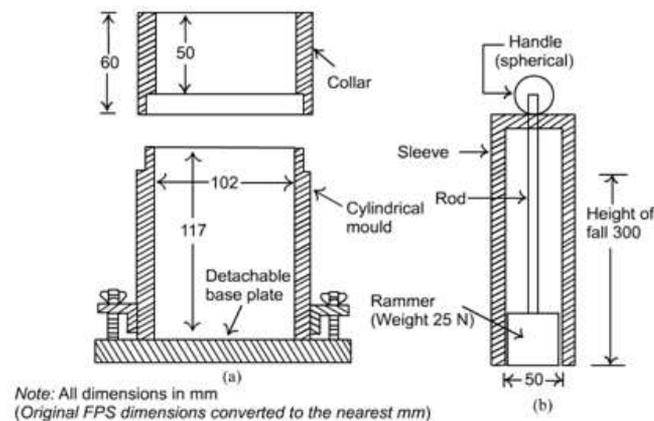
Compaction

1. Removal of air voids from the soil
2. Artificial process
3. Instantaneous phenomenon
4. Soil remain partially saturated

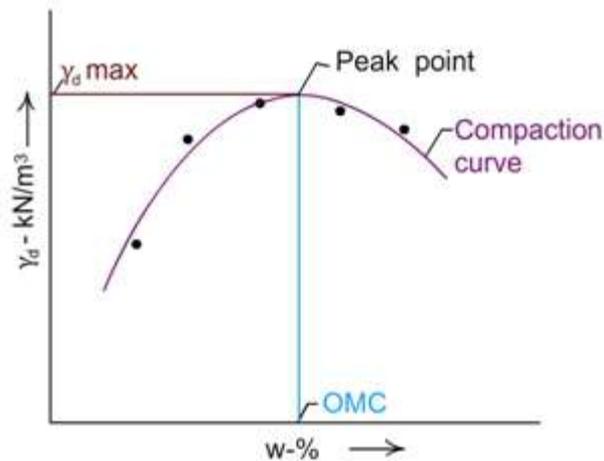
Consolidation

1. Expulsion of water from the pores of soil
2. Natural process
3. Slow and time-independent process
4. Applicable for saturated clays only

Standard Proctor test:

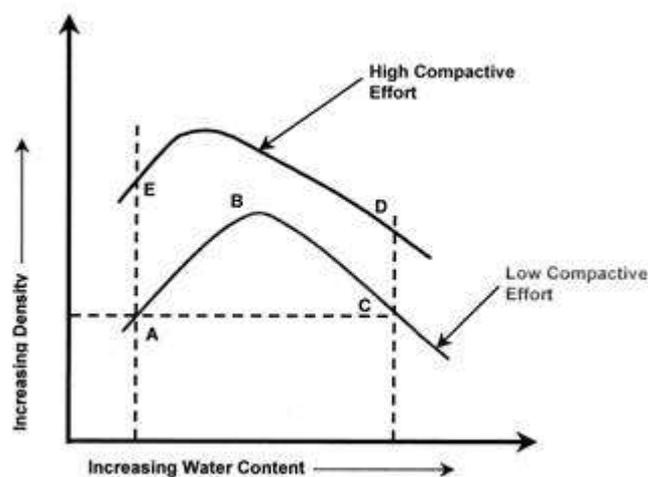


The apparatus for conducting the test consists of 1) a mould for carrying the sample soil – called the Proctor mould, and 2) a rammer for delivering the compaction energy (Fig 8.7). The mould is in three parts, viz. the mould proper which is a cylinder open at both the ends, base and collar. Procedure Measure the inner height and diameter of the mould. Take the weight of the empty mould with base. Air-dry the sample soil, break lumps and sieve through a 4.75 mm sieve. Collect about 15 kg (mass) of the sieved material for conducting a full test for a minimum of five values of w . Take the first portion of 2.5 kg from the above, add little water and mix thoroughly using both hands. Fill it in the mould in three layers delivering 25 blows per layer after placing each layer by simply lifting the rammer and letting it fall freely, through the cylindrical sleeve, on the fill vertically. The surface of the layer below must be scarified for ensuring good bond with the succeeding layer. At the end of compaction detach the collar, cut off the protruding soil at the top and take the weight with the base. The difference with the empty weight gives the wet weight which when divided by the volume of the mould gives the wet density. Determine moisture content using three samples collected from different heights of the compacted soil removed from the mould with the help of a sample extractor. The dry density is determined from the wet density and water content. Now take another 2.5 kg of the sample soil and add about 2 to 3 % more water over the previous quantity and repeat the test as before. Continue with the test till the 6 weight reaches a maximum and falls by two or three values. Plot γ_d vs. w and join the experimental results by a smooth curve, locate the peak and determine $\gamma_{d \max}$ and OMC



Modified Proctor test:

In the modified proctor test, the mould used is the same as in the standard proctor test. However, the rammer used is much heavier and has a greater drop than that in standard proctor test. Its mass is 4.89kg and free drop is 450mm. The face diameter is 50mm as in the standard proctor test. The soil is compacted in five equal layers, each layer is given 25 blows. The Compactive effort in modified proctor test is about 4.56 times that in the standard proctor test.



Zero air void line:

At particular moisture content, maximum dry density is possible only if all the air voids are removed. This is only a theoretical possibility. Practically, all the voids cannot be removed irrespective of the magnitude of Compactive energy applied at that moisture content. The dry density corresponding to the theoretical case where all the air voids are removed at particular moisture content is known as 'Maximum theoretical dry density'. But in practice either in the laboratory or in the field, the dry density that could be achieved at the same moisture content

is less than this dry density because it is impossible to remove all the air voids. This means that for every moisture contents, this maximum theoretical dry density could be calculated as follows:

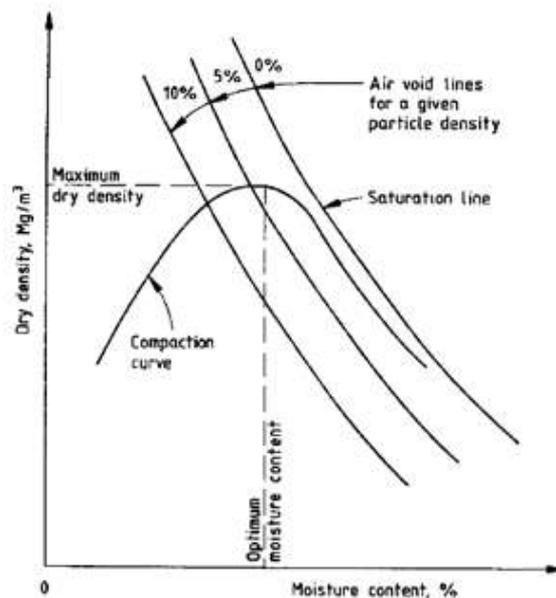
When the entire air voids are removed, the degree of saturation becomes equal to 100%. This implies that the void ratio, $e=wG$.

Where $\rho_{d,max}$ is the maximum theoretical dry density and ρ_w is the Density of water.

For calculating this dry density at particular moisture content, only the specific gravity of solids of soil is required. For each moisture contents, this value could be calculated and plotted along with the compaction curve.

Though the zero air voids line appears to be straight line, especially for a small range of moisture contents, it is a curve. It depends upon the Compactive energy and saturation of soil.

This air voids line or curve varies depending upon the degree of saturation. The dry density corresponding to a particular degree of saturation can be calculated as follows



Factors effecting compaction:

1. **Water content:** At low water content, the soil is stiff and offers more resistance to compaction. As the water content is increased, the soil particles get lubricated. The soil mass becomes more workable and the particles have closer packing. The dry

density of soil increases with an increase in water content till the optimum moisture content is reached. At that stage, the air voids attain approximately a constant volume. With further increase in water content, the air voids do not decrease, but the total voids increase and the dry density decreases. Thus the higher dry density is achieved upto the optimum water content due to forcing air out from the soil voids. After the optimum water content is reached, it becomes more difficult to force air out and to further reduce the air voids.

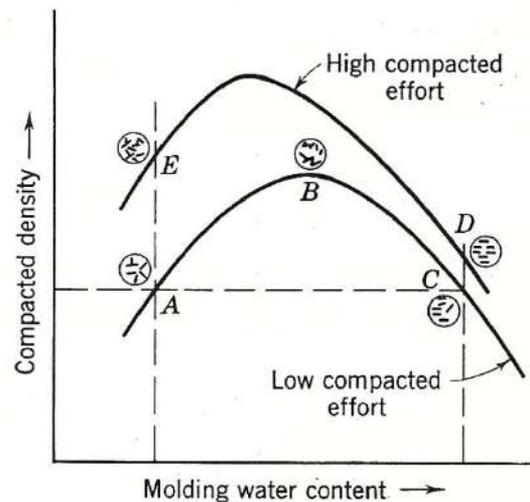
2. **Amount of compaction:** The effect of increasing the amount of Compactive effort is to increase the maximum dry density and to decrease the optimum water content. At water content less than the optimum, the effect of increased compaction is more predominant. At water content more than the optimum, the volume of air voids becomes almost constant and the effect of increased compaction is not significant.
3. **Type of soil:** The dry density achieved depends upon the type of soil. In general, coarse grained soils can be compacted to higher dry density than fine grained soils. With the addition of even a small quantity of fines to a coarse grained soil, the soil attains a much higher dry density for the same Compactive effort. However, if the quantity of fines is increased to a value more than that required to fill the voids of the coarse grained soils, the maximum dry density decreases. A well graded sand attains a much higher dry density than a poorly graded soils. Cohesive soils have higher air voids. Such soils have less dry density and more water content for optimum compared to cohesionless soils.
4. **Method of compaction:** The dry density achieved depends not only upon the amount of compaction but also on the method of compaction. For the same amount of compactive effort, the dry density will depend upon whether the method of compaction utilizes kneading action, dynamic action or static action. Different method of compaction gives their own compactive effort with different optimums.
5. **Admixture:** The compaction characteristics of the soils are improved by adding other materials, known as admixtures. The most commonly used admixtures are lime, cement or bitumen. The dry density achieved depends upon the type and amount of admixtures.

Effect of compaction on properties of soils:

The engineering properties of soil are improved by compaction. The desirable properties are achieved by proper selection of soil type, the mode of placement and the method of

compaction. The effects of compaction are as follows:

1. **Soil Structure:** The water content at which the soil is compacted plays an important role in the engineering properties of soil. Soils compacted at a water content less than the optimum water content generally have a flocculated structure. Soils compacted at water content more than the optimum water content usually have a dispersed structure.

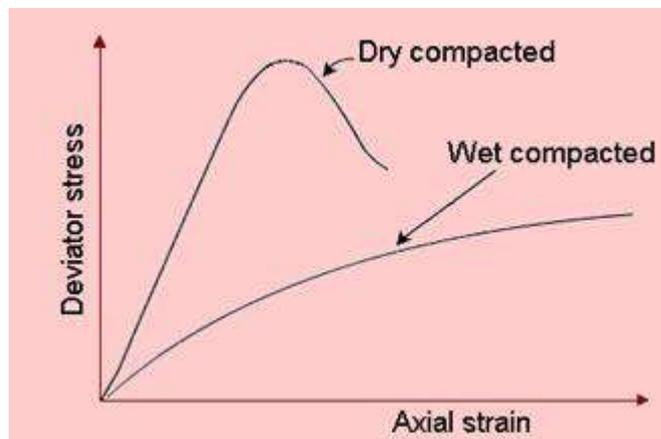


2. **Permeability:** The Permeability of a soil depends upon the size of voids. The permeability of soil decrease with an increase in water content on the dry side of the optimum water content. There is an improved orientation of the particles and a corresponding reduction in the size of the voids which cause a decrease in permeability. The minimum permeability occurs at or slightly above the optimum water content. But the permeability after the optimum would be less than that of dry of optimum.
3. **Swelling:** A soil compacted dry of the optimum water content has higher water deficiency and more random orientation of particles. Consequently, it imbibes more water than the sample compacted wet of the optimum, and has more swelling.
4. **Shrinkage:** Soils compacted dry of the optimum shrink less on drying compared with those compacted wet of optimum. The soils compacted wet of optimum shrink more because the soil particles in the dispersed structure have nearly parallel orientation of particles and can pack more efficiently.
5. **Pore water pressure:** A sample compacted dry of optimum has low water content. The pore pressure developed for the soil compacted dry of optimum is therefore less than that for the same soil compacted wet of optimum.

6. **Compressibility:** The flocculated structure developed on dry side of optimum offers greater resistance to compression than the dispersed structure on the wet side. Consequently the soils on dry side are less compressible.

Compressibility also depends upon degree of compaction, method of compaction, soil structure when compacted in kneading action and stresses caused while compacted with failure of structure of soil.

7. **Stress strain relationship:** The soils compacted dry of optimum have a steeper stress strain curve than those on the wet side. The modulus of elasticity for the soils compacted dry of optimum is high.



8. **Shear strength:** In general, at given water content, the shear strength of the soil increases with an increase in the compactive effort, till a critical degree of saturation is reached. With further increase in Compactive effort, the shear strength decreases. The shear strength of the soil depends upon the soil type, the moulded water content, drainage conditions and the method of compaction.

Methods of Compaction used in field:

Several methods are used for compaction of soil in field. The choice of the method will depend upon the soil type, the maximum dry density required and economic consideration. Some of the common methods are as follows:

1. **Tampers:** A hand operated tamper or rammer consists of a block of iron, about 3 to 5 kg in mass, attached to a wooden rod. The tamper is lifted for about 0.30m and

dropped on the soil to be compacted. A mechanical rammer is operated by compressed air or gasoline power. It is much heavier, about 30 to 150 kg. Mechanical rammers have been used upto a mass of 1000kg in some special cases.

2. Rollers: Rollers of different types are used for compaction of soils and depends upon the following factors
 - a) Contact pressure: The compaction increases with an increase in contact pressure.
 - b) Number of passes: The compaction of soil increases with an increase in number of passes made.
 - c) Layer thickness: The compaction of a soil increases with a decrease in the thickness of layer.
 - d) Speed of roller: The compaction depends upon the speed of the roller. The speed should be so adjusted that the maximum effect is achieved.

Types of rollers:

- i) Smooth wheel rollers: This has one large wheel in the front and two smaller wheels at the rear. Since the rear wheels are placed apart at a distance equal to the length of the front wheel, a single pass compacts the entire width from end to end of the rear wheels.
 - ii) Pneumatic tired rollers: These have pneumatic tyres fixed on two axles - with one number less on the front axle - spaced in such a way that together they cover the complete width, as in the smooth wheeled rollers.
 - iii) Sheep foot rollers: In this case the drum (wheel) incorporates projections which go their entire depth down compacting the soil locally with the drum compacting the full width. (The name comes from the resemblance of the projections to a sheep's foot.) These are of two types, (a) the club foot type, and (b) the tapered foot type. These are normally towed by pneumatic tyred tractors, which can tow up to three rollers in tandem.
3. Vibratory compactors: Weights placed eccentrically across a rotating shaft attached to the drum produce vibrations in the vertical direction. With regard to the selection of the type of roller, while all the types are suitable for cohesive soils, the vibratory type is most effective in cohesionless soils.

GEOTECHNICAL ENGINEERING-I

UNIT-V – CONSOLIDATION

Objective: To study about the process of consolidation in soil.

Syllabus: Consolidation

Spring Analogy mechanism –Terzaghi's theory of one dimensional consolidation – Over consolidated and normally consolidated clay – Determination of void ratio at various load increments –Determination of coefficient of consolidation – pre consolidation pressure.

Outcomes:

Student will be able to

- know the importance of consolidation in real life
- calculate consolidation settlement for cohesion soils.
- realise the determination of coefficient of consolidation

LEARNING MATERIAL

The property of the soil due to which a decrease in volume occurs under compressive forces is known as the compressibility of soil. The compression can be due to the following cases

1. Compression of solid particles and water in the voids
2. Compression and expulsion of air in the voids
3. Expulsion of water in the voids

The compression of the saturated soil under a steady static pressure is known as consolidation. It is entirely due to the expulsion of the water from voids. It is similar to the action of squeezing of water from a sponge under pressure.

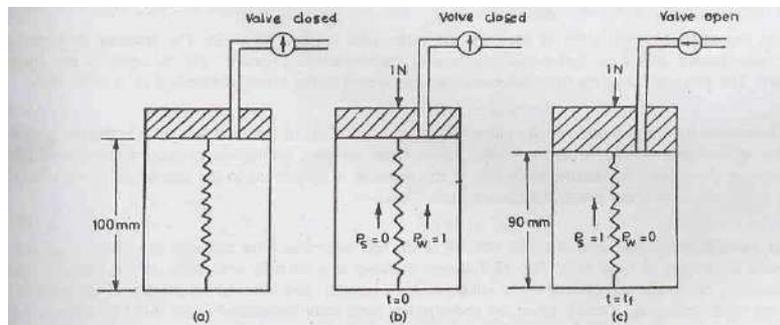
The consolidation of a soil deposit can be divided into 3 stages:

1. Initial consolidation: when a load is applied to a partially saturated soil, a decrease in volume occurs due to expulsion and compression of air in voids. A small decrease in volume also occurs due to the compression of the solids. The reduction of volume of soil just after the application of the load is known as initial consolidation.
2. Primary consolidation: After initial consolidation, further reduction in volume occurs due to the 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 excess pore water pressure. A hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs. This reduction in volume is called primary consolidation.

- Secondary consolidation: the reduction in volume continues at a very slow rate even after the excess hydrostatic pressure developed by the applied pressure is fully dissipated and the primary consolidation is complete. The additional reduction in the volume is called as secondary consolidation

Spring Analogy:

The process of primary consolidation can be explained with the help of the spring analogy given by Terzaghi. Figure shows a cylinder fitted with a tight fitting piston having a valve the cylinder is fitted with water and contains a spring of specified stiffness. Let the initial length of the spring be 100mm and the stiffness of spring be 10mm/N. Let us assume the piston is weightless and the spring and water are initially free of stress.

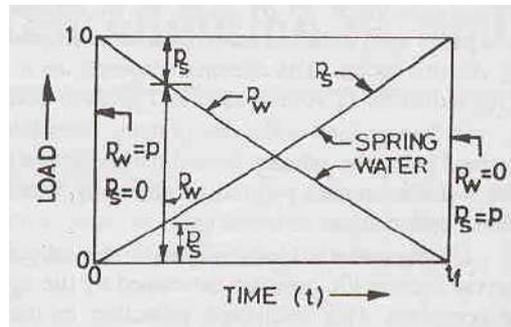


When a load P (say 1N) is applied to the piston, with its valve closed, the entire load is taken by water. The stiffness of the spring is negligible compared with that of water, and consequently, no load is taken by spring. From equilibrium,

Initially when valve is closed, . Therefore

If the valve is now gradually opened, water starts escaping from the cylinder. The spring starts sharing some load and a decrease in its length occurs. When a portion () of the load is transferred from the water to the spring, becomes

As more and more water escapes, the load carried by the spring increases. Graph shows the transfer of the load from the water to the spring. Eventually when the steady conditions are established, the water stops escaping. Finally, at some time, the entire load is taken by spring. Thus and.



Determination of void ratio at various load increments:

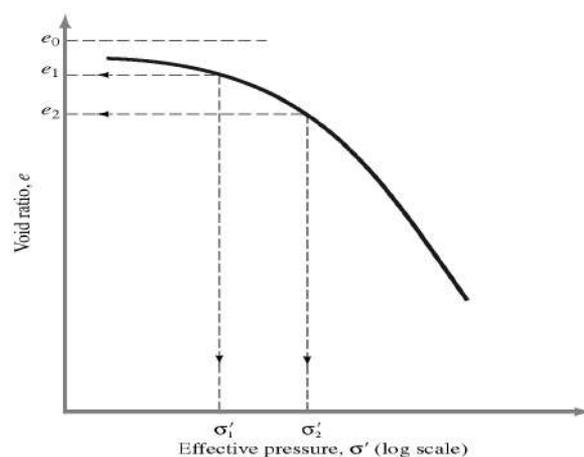
The result of a consolidation test is plotted in the form of a plot between the void ratio and the effective stress. It is therefore, required to determine the void ratio increments. There are two methods:

1. Heights of solids: 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 = 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.

2. Change in void ratio method: In this method, the final void ratio, corresponding to complete swelling conditions after the load has been removed, is determined from its water content, using the equation, assuming soil is saturated.

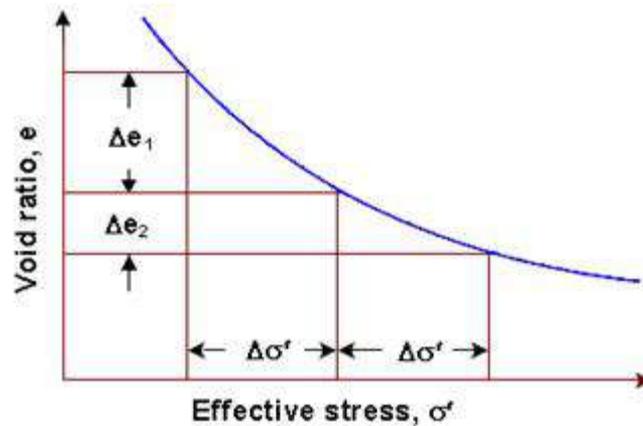
Compressibility characteristics:



Soils are often subjected to uniform loading over large areas, such as from wide foundations, fills or embankments. Under such conditions, the soil which is remote from the edges of the

loaded area undergoes vertical strain, but no horizontal strain. Thus, the settlement occurs only in one-dimension.

The compressibility of soils under one-dimensional compression can be described from the decrease in the volume of voids with the increase of effective stress. This relation of void ratio and effective stress can be depicted either as an arithmetic plot or a semi-log plot.



In the arithmetic plot as shown, as the soil compresses, for the same increase of effective stress $\Delta\sigma'$, the void ratio reduces by a smaller magnitude, from Δe_1 to Δe_2 . This is on account of an increasingly denser packing of the soil particles as the pore water is forced out. In fine soils, a much longer time is required for the pore water to escape, as compared to coarse soils.

It can be said that the compressibility of a soil decreases as the effective stress increases. This can be represented by the slope of the void ratio – effective stress relation, which is called the coefficient of compressibility, a_v .

$$a_v = -\frac{de}{d\sigma'}$$

$$a_v = -\frac{\Delta e}{\Delta\sigma'}$$

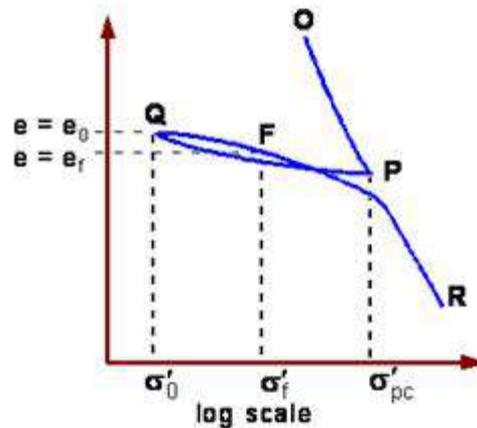
For a small range of effective stress, the -ve sign is introduced to make a_v a positive parameter.

If e_0 is the initial void ratio of the consolidating layer, another useful parameter is the coefficient of volume compressibility, m_v , which is expressed as

$$m_v = \frac{a_v}{1 + e_0}$$

It represents the compression of the soil, per unit original thickness, due to a unit increase of pressure.

The figure shows the relation of void ratio and effective stress of a clay soil as a semi-log plot.



OP corresponds to initial loading of the soil. PQ corresponds to unloading of the soil. QFR corresponds to a reloading of the soil. Upon reloading beyond P, the soil continues along the path that it would have followed if loaded from O to R continuously.

The preconsolidation stress, s'_{pc} , is defined to be the maximum effective stress experienced by the soil. This stress is identified in comparison with the effective stress in its present state. For soil at state Q or F, this would correspond to the effective stress at point P.

If the current effective stress, s' , is equal (note that it cannot be greater than) to the preconsolidation stress, then the deposit is said to be normally consolidated (NC). If the current effective stress is less than the preconsolidation stress, then the soil is said to be over-consolidated (OC).

It may be seen that for the same increase in effective stress, the change in void ratio is much less for an overconsolidated soil (from e_0 to e_f), than it would have been for a normally consolidated soil as in path OP. In unloading, the soil swells but the increase in volume is much less than the initial decrease in volume for the same stress difference.

The distance from the normal consolidation line has an important influence on soil behaviour. This is described numerically by the overconsolidation ratio (OCR), which is defined as the ratio of the **preconsolidation stress** to the current effective stress.

$$OCR = \frac{\sigma'_{pc}}{\sigma'}$$

Note that when the soil is normally consolidated, $OCR = 1$

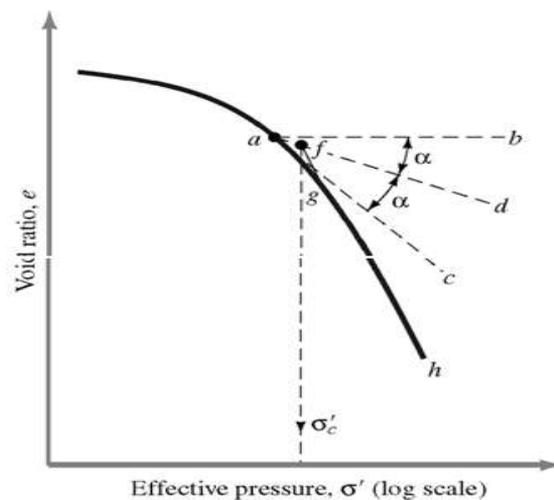
Settlements will generally be much smaller for structures built on overconsolidated soils. Most soils are overconsolidated to some degree. This can be due to shrinking and swelling of the soil on drying and rewetting, changes in ground water levels, and unloading due to erosion of overlying strata.

For NC clays, the plot of void ratio versus log of effective stress can be approximated to a straight line, and the slope of this line is indicated by a parameter termed as **compression index, C_c** .

$$C_c = \frac{\Delta e}{\log_{10} \left(\frac{\sigma'_2}{\sigma'_1} \right)}$$

Determination of Preconsolidation Pressure:

It is possible to determine the preconsolidation stress that the soil had experienced. The soil sample is to be loaded in the laboratory so as to obtain the void ratio - effective stress relationship. Empirical procedures are used to estimate the preconsolidation stress, the most widely used being Casagrande's construction which is illustrated.



The steps in the construction are:

- Draw the graph using an appropriate scale.
- Determine the point of maximum curvature A.
- At A, draw a tangent AB to the curve.
- At A, draw a horizontal line AC.
- Draw the extension ED of the straight line portion of the curve.
- Where the line ED cuts the bisector AF of angle CAB that point corresponds to the preconsolidation stress.

Terzaghi one dimensional consolidation equation

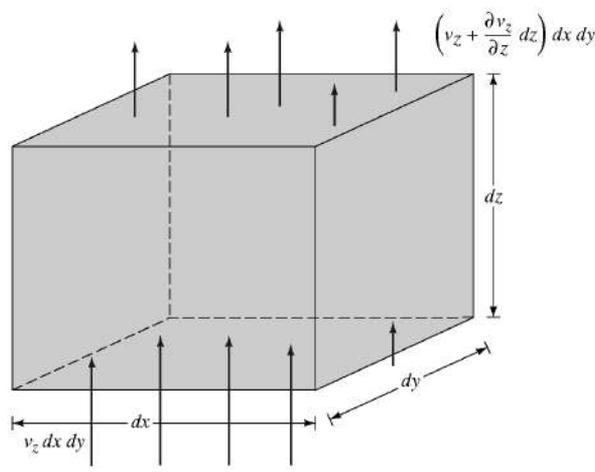
Assumptions:

1. The clay-water system is homogenous.
2. Saturation is complete ($S = 100\%$).
3. Compressibility of water is negligible.

4. Compressibility of soil grains is negligible (but soil particles rearrange).
5. Flow of water is in one direction only.
6. Darcy's Law is Valid.

Derivation:

The total stress increases when additional vertical load is first applied. Instantaneously, the pore water pressure increases by exactly the same amount. Subsequently there will be flow from regions of higher excess pore pressure to regions of lower excess pore pressure causing dissipation. The effective stress will change and the soil will consolidate with time. This is shown schematically.



On the assumption that the excess pore water drains only along vertical lines, an analytical procedure can be developed for computing the rate of consolidation. Consider a saturated soil element of side's dx, dy and dz.

The initial volume of soil element = dx.dy.dz

If n is the porosity, the volume of water in the element = n.dx.dy.dz

the continuity equation for one-dimensional flow in the vertical direction is

$$\frac{\delta V_z}{\delta z} dx.dy.dz = - \frac{\delta}{\delta t} (n.dx.dy.dz)$$

Only the excess head (h) causes consolidation, and it is related to the excess pore water pressure (u) by

$h = u/\gamma_w$. The Darcy equation can be written as

$$V_z = -k_z \frac{\delta h}{\delta z} = - \frac{k_z}{\gamma_w} \frac{\delta u}{\delta z}$$

The Darcy eqn. can be substituted in the continuity eqn., and the porosity n can be expressed in terms of void ratio e, to obtain the flow equation as

$$\frac{k_z}{\gamma_w} \frac{\delta^2 u}{\delta z^2} dx dy dz = \frac{\delta}{\delta t} \left(\frac{e}{1+e} dx dy dz \right)$$

If e_0 is the initial void ratio of the consolidating layer, the initial volume of solids in the element is $(dx dy dz) / (1 + e_0)$, which remains constant. The change in water volume can be represented by small changes Δe in the current void ratio e .

The flow eqn. can then be written as

$$\frac{k_z}{\gamma_w} \frac{\delta^2 u}{\delta z^2} dx dy dz = \frac{dx dy dz}{1+e_0} \cdot \frac{\delta e}{\delta t}$$

$$\frac{k_z}{\gamma_w} \frac{\delta^2 u}{\delta z^2} = \frac{1}{1+e_0} \cdot \frac{\delta e}{\delta t}$$

This is the hydrodynamic equation of one-dimensional consolidation. If a_v = coefficient of compressibility, the change in void ratio can be expressed as $\Delta e = a_v \cdot (-\Delta s') = a_v \cdot (\Delta u)$ since any increase in effective stress equals the decrease in excess pore water pressure. Thus,

$$\frac{\delta e}{\delta t} = a_v \cdot \frac{\delta u}{\delta t}$$

The flow eqn. can then be expressed as

$$\frac{k_z}{\gamma_w} \frac{\delta^2 u}{\delta z^2} = \frac{a_v}{1+e_0} \cdot \frac{\delta u}{\delta t}$$

$$\frac{k_z}{a_v} \cdot \frac{(1+e_0)}{\gamma_w} \cdot \frac{\delta^2 u}{\delta z^2} = \frac{\delta u}{\delta t}$$

By introducing a parameter called the coefficient of consolidation,

$$c_v = \frac{k_z (1+e_0)}{a_v \gamma_w} = \frac{k_z}{m_v \gamma_w},$$

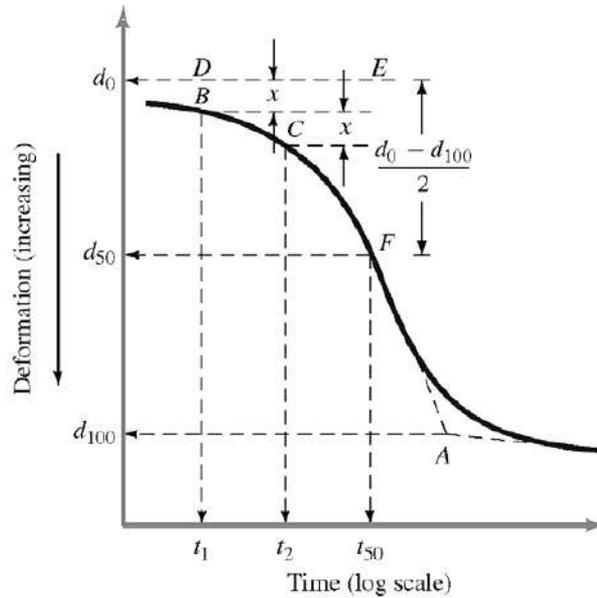
the flow eqn. then becomes

$$c_v \cdot \frac{\delta^2 u}{\delta z^2} = \frac{\delta u}{\delta t}$$

This is Terzaghi's one-dimensional consolidation equation. A solution of this for a set of boundary conditions will describe how the excess pore water pressure u dissipates with time t and location z . When all the u has dissipated completely throughout the depth of the compressible soil layer, consolidation is complete and the transient flow situation ceases to exist.

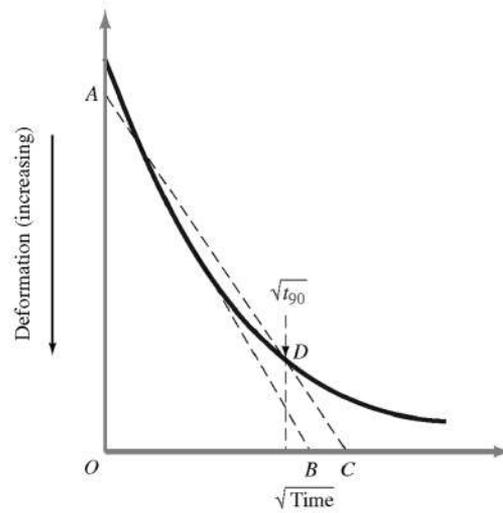
Determination of coefficient of consolidation C_v :

1. Logarithm of Time Method:



- Extend the straight line portion of primary and secondary consolidations to intersect at Point A. Point A represents d_{100} (Deformation at 100% primary consolidation).
- The initial curved portion of the deformation plot versus $\log t$ is approximated to be a parabola on a natural scale. Select times t_1 and t_2 on the curved portion such that $t_2 = 4t_1$. Let the difference of the specimen deformation between $(t_2 - t_1)$ be equal to x .
- Draw a line horizontal to DE such that the vertical distance BD is equal to x . The deformation corresponding to the line DE is d_0 (Deformation at 0% primary consolidation).
- The ordinate of Point F on the consolidation curve represents the deformation at 50% primary consolidation (d_{50}).
- For 50% average degree of consolidation ($U = 50\%$), $T_v = 0.197$

2. Square Root of Time Method:



- Draw a line AB through the early portion of the curve.
- Draw a line AC such that $OC = 1.15OB$.
- The time value for Point D (i.e. the intersection of line AC and the data) is the square root of time for t_{90} (i.e. the time to 90% primary consolidation).
- For 90% consolidation, $T_v = 0.848$

UNIT – VI

Shear Strength of Soils

Objective: To study the shear strength parameters

Syllabus:

Basic mechanism of shear strength –Mohr Coulomb Failure theories –Liquefaction- shear strength determination – various drainage conditions

Outcomes:

Student will be able to

- Learn the test procedure of determining the shear parameters
- Determine shear strength parameters of soils.

LEARNING MATERIAL

Shear strength may be defined as the resistance to shearing stresses and a consequent tendency for shear deformation.

Soil derives its shearing strength from the following

1. resistance due to interlocking of particles
2. frictional resistance between the individual soil grains
3. adhesion between soil particles or cohesion

PRINCIPAL PLANES AND PRINCIPAL STRESSES

At a point in a stressed material, every plane will be subjected to a normal or direct stress and a shearing stress. A principal plane is defined as a plane on which the stress is fully normal or one which does not carry shearing stress. The normal stress acting on this principal planes are known as principal stresses. There exist three principal planes at any point in a stressed material. These three principal planes are mutually perpendicular. In the order of decreasing magnitude the principal planes are designated as major principal plane, minor principal plane and intermediate principal plane and the corresponding principal stresses are designated in the same manner.

σ_1 = major principal stress
 σ_3 = minor principal stress

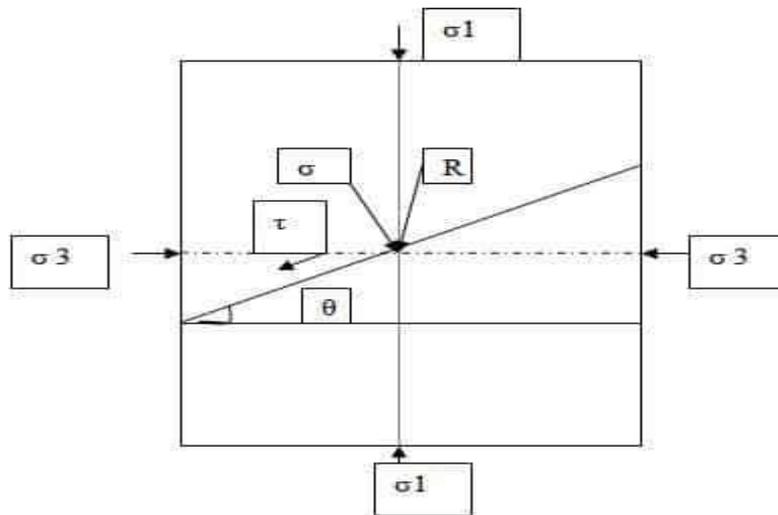


Fig 1: Soil Under Stresses

From this figure,

These equations will give the stresses on the inclined plane making an angle θ with the major principal plane.

MOHR'S CIRCLE

Otto Mohr, a German scientist devised a graphical method for the determination of stresses on a plane inclined to the major principal planes. The graphical construction is known as Mohr's circle. In this method, the origin O is selected and the normal stresses are plotted along the horizontal axis and the shear stresses on the vertical axis.

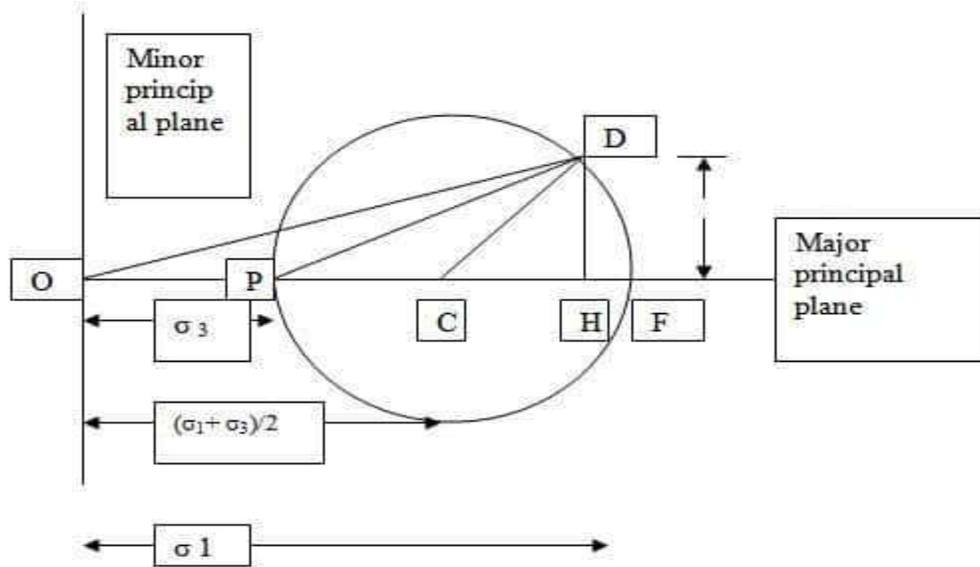


Fig 2. Mohr Circle

To construct a Mohr circle, first mark major and minor principal stress on X axis, Mark the centre point of that as C. A circle is drawn with c as centre and CF as radius. Each point on the circle gives the stresses on a particular plane. The point E is known as the pole of the circle.

1. Mohr's circle can be drawn for stress system with principal planes inclined to co-ordinate axes
2. Stress system with vertical and horizontal planes are not the principal planes

MOHR-COULOMB THEORY

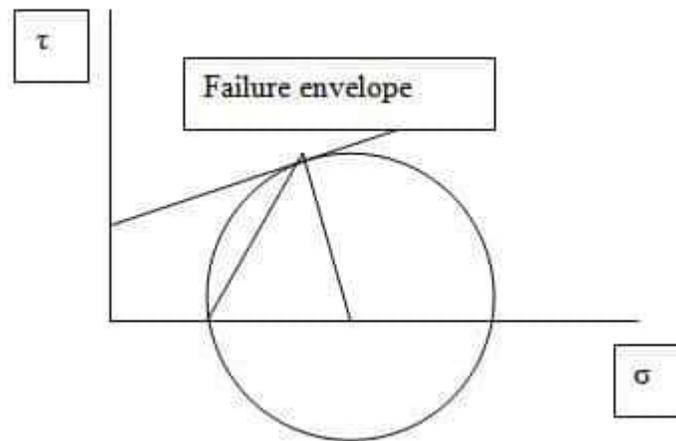


Fig 3. Mohr Coulomb Envelop

The soil is a particulate material. The shear failure in soils is by slippage of particles due to shear stresses. According to Mohr, the failure is caused by a critical combination of normal and shear stresses. The soil fails when the shear stress on the failure plane at failure is a unique function of the normal stress acting on that plane. Since the shear stress of the failure plane is defined as the shear strength (s) the equation for that can be written as

$$S = f(\theta)$$

The Mohr theory is concerned with the shear stress at failure plane at failure. A plot can be made between the shear stresses and the normal stress at failure. The curve defined by this is known as the failure envelope.

The shear strength of a soil at a point on a particular plane was expressed by Coulomb as a linear function of the normal stress on that plane as,

In this C is equal to the intercept on Y axis and phi is the angle which the envelope make with X axis

DIFFERENT TYPES OF SHEAR TESTS AND DRAINAGE CONDITIONS

The following tests are used to measure the shear strength of the soil

1. Direct shear test
2. Triaxial compression test
3. Unconfined compression test
4. Vane shear test

Depending upon the drainage conditions, there are three types of tests

- Unconsolidated-Undrained condition
- Consolidated – Undrained condition
- Consolidated-Drained condition

DIRECT SHEAR TEST

Apparatus

The test is conducted in a soil specimen in a shear box which is split in to two halves along the horizontal plane at its middle. The size of the shear box is 60 x 60 x 50 mm. the box is divided horizontally such that the dividing plane passes through the centre. The two halves are held together by locking pins the box is also provided with gripper plates plain or perforated according to the testing conditions.

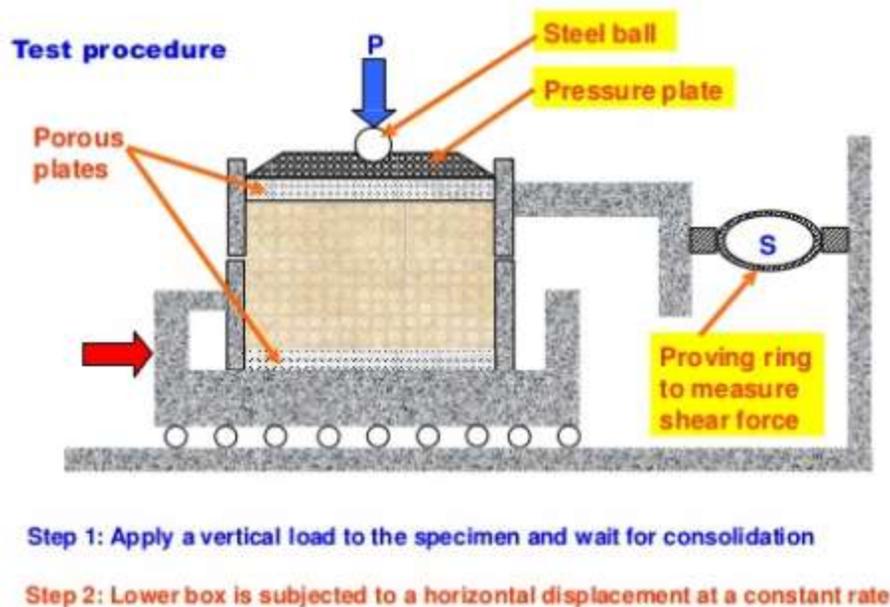


Fig 4. Direct Shear Test

Test

A soil specimen of size 60 x 60 x 25 mm is taken. It is placed in the direct shear box and compacted. The upper grid plate, porous stone and pressure pad is placed on the specimen. Normal load and shear load is be applied till failure

Presentation of results

- Stress – strain curve
- Failure envelope
- Mohr's circle

Merits

1. the sample preparation is easy
2. as the thickness of the sample is very less, the drainage is quick
3. it is ideally suited for conducting drained tests on cohesionless soils
4. the apparatus is relatively cheap

Demerits

1. the stress conditions are known only at failure
2. the stress distribution on the failure plane is not uniform
3. the area of shear gradually decreases as the test progresses
4. the orientation of the failure plane is fixed
5. control of drainage conditions is very difficult
6. measurement of pore water pressure is not possible

Direct Shear Test on Sands:

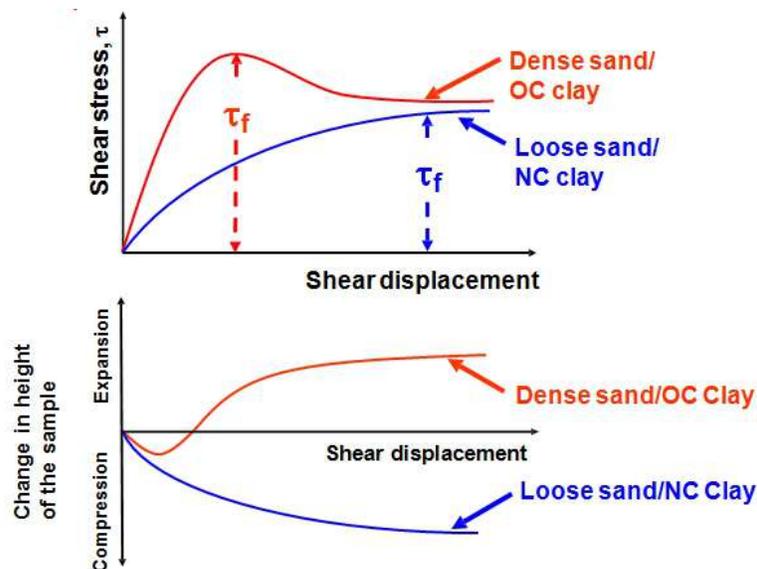


Fig 5. Graphs for Triaxial tests on sands

TRIAXIAL COMPRESSION TEST

It is used for the determination of shear characteristics of all types of soils under different drainage conditions. In this a cylindrical specimen is stressed under conditions of axial symmetry.

In the first stage of the test, the specimen is subjected to an all round confining pressure on the sides, top and bottom. This stage is known as the consolidation stage. In the second stage of the test called shearing stage, an additional axial stress called deviator stress is applied on the top of the specimen through a ram. Thus the total stress in the axial direction at the time of shearing is equal to the confining stress plus the deviator stress. The vertical sides of the specimen are principal planes. The confining pressure is the minor principal stress. The sum of the confining stress and deviator stress is the major principal stress. Triaxial apparatus consists of a circular base with a central pedestal. The specimen is placed on the pedestal. The pedestal has one or two holes which are used in the drainage function or pore pressure measurement. A triaxial cell is placed to the base plate. It is a Perspex cylinder. There are three tie rods which support the cell. A central ram is there for applying axial stress. An air release valve and an oil release valve are attached to the cell. The apparatus also have special features like,

- Mercury control system
- Pore water pressure measurement device
- Volume changes measurement

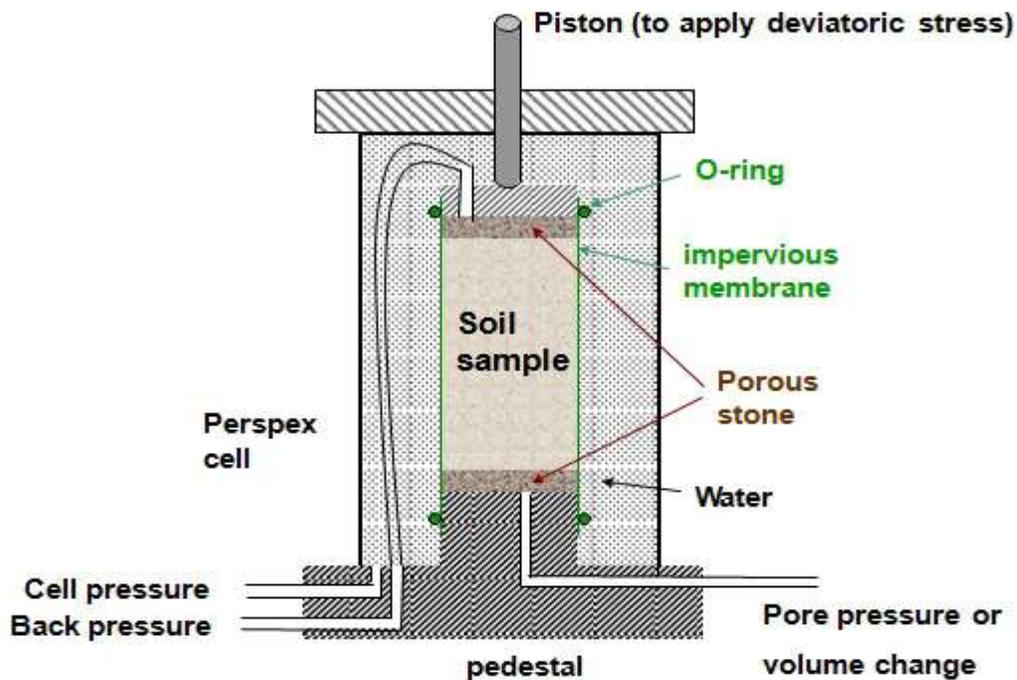


Fig 6: Triaxial test apparatus

Triaxial test on cohesive soil

CU, UU and CD tests can be conducted on soil specimen. The specimen is placed in the pedestal inside a rubber membrane. The confining pressure and axial pressure is applied till failure.

Triaxial test on cohesionless soil

The procedure is same as that in the cohesive soil only the sample preparation is different. A metal former, a membrane and a funnel are used for the sample preparation.

Merits

1. There is complete control over the drainage conditions
2. Pore pressure changes and volumetric changes can be measured directly
3. The stress distribution in the failure plane is uniform
4. The specimen is free to fail on the weakest plane
5. The state of stress at all intermediate stages up to failure is known
6. The test is suitable for accurate research work

Demerits

1. The apparatus is elaborate, costly and bulky
2. The drained test takes a longer period as compared with that in a direct shear test
3. The strain condition in the specimen are not uniform
4. It is not possible to find out the cross sectional area of the specimen accurately under large strains
5. The test simulates only axi symmetric problems
6. The consolidation of the specimen in the test is isotropic whereas in the field, consolidation is generally anisotropic.

Computation of various parameters

1. Post consolidation dimensions

$$V_o = L_o \times \frac{\pi D_o^2}{4}$$
$$D_o = \left[\frac{V_o}{(\pi / 4) \times L_o} \right]^{\frac{1}{2}}$$

2. Cross sectional area during shearing stage

$$A = \frac{A_o}{1 - \xi_1}$$

3. Stresses

Deviator stress = P/A

Principal stresses $\sigma_1 = \sigma_3 + (\sigma_1 - \sigma_3)$

4. Compressive strength

The deviator stress at failure is known as the compressive strength of soil

Presentation of results of triaxial test

- Stress-strain curves
- Mohr envelopes in terms of total stress and effective stress

UNCONFINED COMPRESSION TEST

The unconfined compression test is a special form of triaxial test in which the confining pressure is zero. The test can be conducted only on clayey soils which can stand without confinement. There are two types of UCC machines machine with a spring and machine with a proving ring

A compressive force is applied to the specimen till failure. The compressive load can be measured using a proving ring.

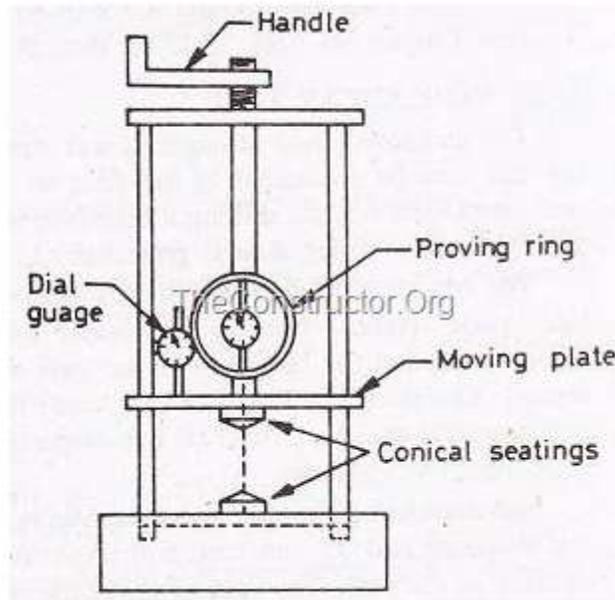


Fig 7. Unconfined Compression Apparatus

Presentation of results

In this test the minor principal stress is zero. The major principal stress is equal to the deviator stress. The Mohr circle can be drawn for stress conditions at failure.

Merits

1. The test is convenient, simple and quick
2. It is ideally suited for measuring the unconsolidated undrained shear strength of intact saturated clays
3. The sensitivity of the soil can be easily determined

Demerits

1. The test cannot be conducted on fissured clays
2. The test may be misleading for soils of which the angle of shearing resistance is not zero.

VANE SHEAR TEST

The undrained strength of soft clays can be determined in a laboratory by vane shear test. The test can also be conducted in the field on the soil at the bottom of bore hole. The apparatus consists of a vertical steel rod having four thin stainless steel blades or vanes fixed at its bottom end. Height of the vane should be equal to twice the diameter. For conducting test in a laboratory, a specimen of diameter 38mm and height 75mm is prepared and fixed to the base of the apparatus. The vane is slowly lowered in to the specimen till the top of the vane is at a depth of 10 to 20 mm below the top of the specimen. The readings of the strain indicator and torque indicator are taken

Shear strength S

$$S = \frac{T}{\pi \left(\frac{D^2 H_1}{2} + \frac{D^3}{12} \right)}$$

Where T =Torque applied

D = Diameter of vane

H₁= Height of vane

Merits

1. The test is simple and quick
2. It is ideally suited for determination of the in-situ undrained shear strength of non fissured, fully saturated clay
3. The test can be conveniently used to determine the sensitivity of the soil

Demerits

1. The test cannot be conducted on the fissured clay or the clay containing silt or sand laminations
2. The test does not give accurate results when the failure envelope is not horizontal