MODULE – 1

INTRODUCTION

“KARL VON TERZAGHI is rightly regarded as Father of Modern Soil Mechanics”

1.1 INTRODUCTION:

1.1.1 Definition of Soil: To an agriculturist “soil is the substance existing on the earth’s surface, which grows and develops plant life”.

To a geologist “it means the disintegrated rock material which has not been transported from the place of origin”.

To a civil engineer “the loose unconsolidated inorganic material on the earth’s crust produced by the disintegration of rocks, overlaying hard rock with or without organic matter”. It includes different materials like boulders, sands, gravels, clays and silts.

1.1.2 Soil Mechanics: is the study of the engineering behaviour of soil when it is used either as a construction material or as a foundation material.

1.1.3 Foundation Engineering: is a branch of civil engineering, which is associated with the design, construction, maintenance and renovation of footings, foundation walls, pile foundation, caissons and all other structural members which form the foundation of buildings and other engineering structures.

1.1.4 Soil Engineering: may be considered as the branch of engineering involving the study of soil, its behaviour and application as an engineering material.

1.2 ORIGIN AND SOIL FORMATION:

Soil has originated from the Latin word “Solum”. Soil is formed by the process of “Weathering” of rocks, i.e., disintegration and decomposition of rocks and minerals at or near the earth’s surface through the actions of natural or mechanical and chemical agents into smaller and smaller grains. The factors of weathering may be atmospheric, such as changes in temperature and pressure, erosion and transportation by wind, water and glaciers, chemical action such as crystal growth, oxidation, hydration, carbonation and leaching by water, especially rainwater with time. It is to be noted that 95% of the earth’s crust consists of igneous rocks and only the remaining 5% consists of sedimentary and metamorphic rocks. However, sedimentary rocks are present on 80% of the earth’s surface area. ‘Leaching’ is the process whereby water-soluble parts in the soil such as calcium carbonate are dissolved and washed out form the soil by rain fall.

1.3 HISTORY OF SOIL MECHANICS:

The use of soil for engineering purposes dates back to prehistoric times. Soil was used not only for foundations but also as construction material for embankments. The knowledge of soils for foundations, bunds and roads was gained by trial and error experiences. Through ancient times and even within the last few generations practically all improvement was the result of a continuously broadening by empirical knowledge. The hanging gardens of Babylon were supported by huge retaining walls, the construction of which should have required some knowledge, through empirical, of earth pressures. The large public buildings, harbours, aqueducts, bridges, roads and sanitary works of Romans certainly indicated some knowledge of the engineering behaviour of soil. Coulomb, a French Engineer, published his wedge theory of earth pressure in 1776, which is the first major contribution to the scientific study of soil behaviour. He was the first to introduce the concept of shearing resistance of the soil as composed of two components – cohesion and internal friction. Poncelet, Culmann and Rebhann were the other men who extended the work of Coulomb D’ Arcy and Stokes were notable for their laws for the flow of water through soil and settlement of a solid particle in liquid medium, respectively. These laws are still valid and play an important role in soil mechanics. Rankine gave his theory of earth pressure in 1857; he did not considered cohesion, although he knew of its existence.

1.4 FIELDS OF APPLICATIONS OF SOIL MECHANICS:

1.4.1 Foundation Design and Construction: foundation is an important element of all civil engineering structures. It is, therefore, necessary to know the bearing capacity of the soils, the pattern of stress distribution in the soil beneath the loaded area, the probable settlement of the foundation, effect of ground water and the effect of vibrations etc..

1.4.2 Pavement Design: may consists of the design of flexible or rigid pavements, and its performance depends upon the subsoil on which its rests. Problems peculiar to the design of pavements are the effect of repetitive loading, swelling and shrinkage of sub-soil and frost action.

1.4.3 Design of Underground Structures and Earth Retaining Structures: underground structures such as drainage structures, pipelines and tunnels. Earth retaining structures such as gravity retaining wall, anchored bulkheads and coffer dams can be designed and constructed only by using the principles of soil mechanics.

1.4.4 Design of Embankments and Excavations: when the surface of the soil structure is not horizontal, the component of gravity tends to move the soil downwards and may disturb the stability of the earth structures. A thorough knowledge of shear strength and related properties of soil is essential to design the slope and height or depth of the embankment.

1.4.5 Design of Earth Dams: the construction of an earth dam mainly requires the soil which may either be homogeneous or composite section, its design involves the determination of the following physical properties of soil: index properties such as density, plasticity, specific gravity, particle size distribution, permeability, consolidation, compaction and shear strength parameters.

1.5 SOIL AS A THREE PHASE SYSTEM:

Phase means any homogeneous part of the system different from other parts of the system and separated by transition. A system consisting of more than one phase is said to be heterogeneous. A soil mass is a three phase system consisting of solid particles, water and air. The void space between the soil grains is filled partly with water and partly with air. However, if we take a dry soil mass, the voids are filled with air only. In case of a perfectly saturated soil, the voids are filled completely with water. In general, the soil mass has three constituents which do not occupy separate spaces but are blended together forming a complex material [Fig 2.1 (a)] the properties of which depend upon the relative percentages of these constituents, their arrangement and a variety of other factors. For calculation purposes, it is always more convenient to show these constituents occupying separate spaces, as shown in [Fig 2.1 (b) (i) and (b) (ii)].

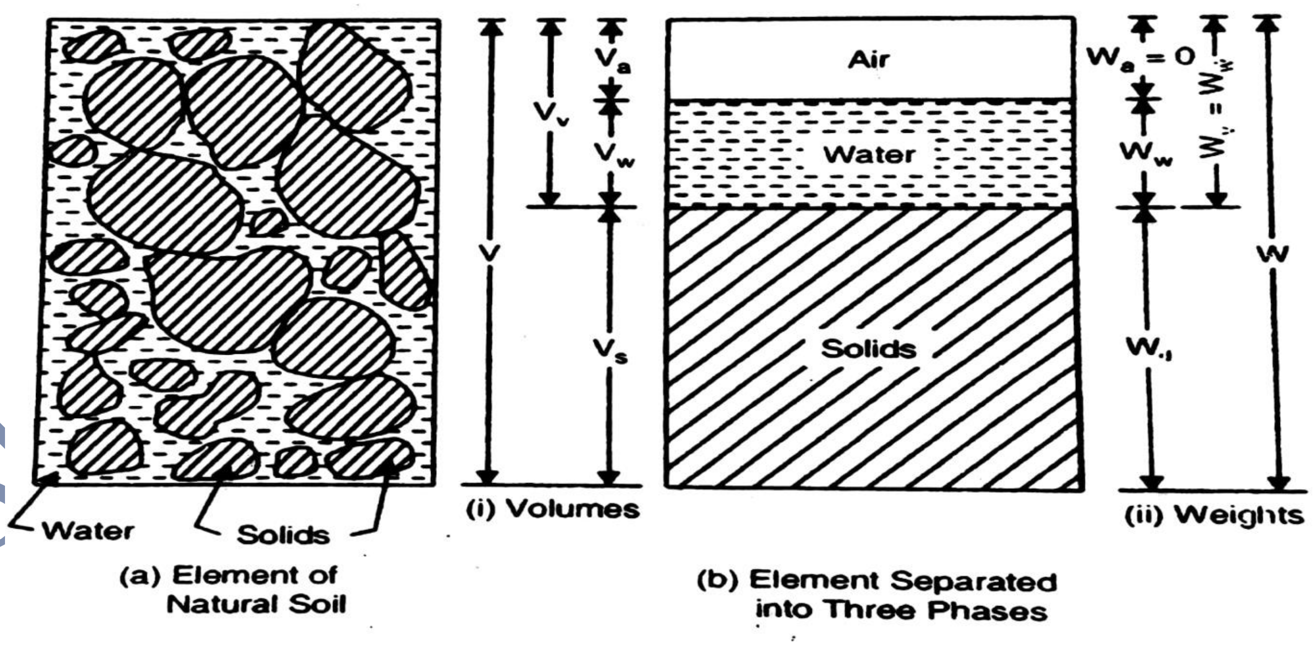


FIG. 2.1. SOIL AS A THREE PHASE SYSTEM

1.6 IMPORTANT DEFINITIONS:

1.6.1 Water Content or Moisture Content (w): is defined as the ratio of the weight of water W_W to the weight of solids W_S of the soil mass. It is usually expressed in percentage.

$$w = (W_W / W_S) \times 100$$

1.6.2 Density of Soil: is defined as the mass of the soil per unit volume.

1.6.3 Bulk Density or Moist Density (ρ): is defined as the total mass M of the soil per unit of its total volume V.

$$\rho = M / V \text{ (it is expressed in terms of g/cm}^3 \text{ or kg/m}^3\text{)}$$

1.6.4 Dry Density (ρ_d): is defined as the mass of solids M_d per unit of total volume of the soil mass V.

$$\rho_d = M_d / V$$

1.6.5 Density of Solids (ρ_s): is defined as the mass of solids M_d per unit of total volume of the solids V_S.

$$\rho_s = M_d / V_S$$

1.6.6 Saturated Density (ρ_{sat}): is defined as the ratio of the total soil mass of saturated sample M_{sat} to its total volume V.

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

1.6.7 Submerged Density or Buoyant Density (ρ'): is defined as the submerged mass of soil solids $(M_d)_{sub}$ per unit of total volume V of the soil mass.

$$\rho' = (M_d)_{sub} / V$$

$$\rho' = \rho_{sat} - \rho_w$$

1.6.8 Unit Weight of Soil Mass: is defined as the weight of soil per unit volume.

1.6.9 Bulk Unit Weight or Moist Unit Weight (ɣ): is defined as the total weight W of a soil mass per unit of its total volume V.

$$\gamma = W / V$$

1.6.10 Dry Unit Weight (γ_d): is defined as the weight of solids W_d per unit of total volume of soil mass V.

$$\gamma_d = W_d / V$$

1.6.11 Unit Weight of Solids (γ_s): is defined as weight of soil solids W_d per unit volume of solids V_s.

$$\gamma_s = W_d / V_s$$

1.6.12 Unit Weight of Water (γ_w): is defined as the weight of water W_w per unit volume of water V_w.

$$\gamma_w = W_w / V_w$$

1.6.13 Saturated Unit Weight (γ_{sat}): is defined as the ratio of the total weight of saturated sample W_{sat} to its total volume V.

$$\gamma_{sat} = W_{sat} / V$$

1.6.14 Submerged Unit Weight or Buoyant Unit Weight (γ'): is defined as the submerged weight of soil solids $(W_d)_{sub}$ per unit of total volume V of the soil mass.

$$\gamma' = (W_d)_{sub} / V$$

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

1.6.15 Mass Specific Gravity or Bulk Specific Gravity or Apparent Specific Gravity (G_m): is defined as the ratio of mass of the soil to the unit weight of water at a standard temperature.

1.6.16 Specific Gravity of Solids (G or G_s): is defined as the ratio of the weight of given volume of soil solids to the weight of an equal volume of water at a standard temperature.

1.6.17 Specific Gravity of Water (G_w): is defined as the ratio of the weight of water to the weight of water at a standard temperature.

1.6.18 Voids Ratio (e): is defined as the ratio of the volume of voids V_v to the volume of solids V_s in the soil mass.

$$e = V_v / V_s$$

1.6.19 Porosity (n): is defined as the ratio of the volume of voids V_v to the total volume of the soil mass V. It is usually expressed in percentage.

$$n = (V_v / V) \times 100$$

1.6.20 Degree of Saturation (S or S_r): is defined as the ratio of the volume of water V_w to the volume of voids V_v. It is usually expressed in percentage.

$$S \text{ or } S_r = (V_w / V_v) \times 100$$

1.6.21 Percentage Air Voids (n_a): is defined as the ratio of the volume of air voids V_a to the total volume of the soil mass V. It is usually expressed in percentage.

$$n_a = (V_a / V) \times 100$$

1.6.22 Air Content (a_c): is defined as the ratio of the volume of air voids V_a to the volume of voids V_v. It is usually expressed in percentage.

$$a_c = (V_a / V_v) \times 100$$

1.6.23 Density Index or Relative Density or Degree of Density (I_D): is defined as the ratio of the difference between the voids ratio of the soil in its loosest state e_{max} and its natural voids ratio e to the difference between the voids ratios in the loosest and densest state e_{min}.

$$I_D = (e_{max} - e) / (e_{max} - e_{min})$$

MODULE – 1

INDEX PROPERTIES OF SOIL AND THEIR DETERMINATION

1.8 GENERAL:

The determination of water content, specific gravity, particle size distribution, consistency limits, in-situ density and density index. These properties are known as "Index Properties".

Or

For a proper evaluation of the suitability of soil for use as foundation or construction material, information about its properties, in addition to classification is frequently necessary. Those properties which help to assess the engineering behaviour of a soil and which assist in determining its classification accurately are termed as "Index properties".

1.9 WATER CONTENT:

1.9.1 The water content of a soil sample can be determined by the following methods:

1.9.1.1 Oven Drying Method

1.9.1.2 Sand Bath Method

1.9.1.3 Alcohol Method

1.9.1.4 Calcium Carbide Method

1.9.1.5 Pycnometer Method

1.9.1.6 Radiation Method

1.9.1.7 Torsion Balance Method

1.9.1.1 Oven Drying Method: this is the most accurate method of determining the water content, and is, therefore, used in the laboratory. A specimen of soil sample is kept in a clean container and put in a thermostatically controlled oven with interior of non-corroding material to maintain the temperature between 105 to 110°C. Usually the sample is kept for about 24 hours in the oven so that complete drying is assured.

Procedure:

- A clean non-corrodible container is taken and its mass is found with its lid, on a balance accurate to 0.01g.
- A specimen of the moist soil is placed in the container and the lid is replaced.

- The mass of the container and the contents is determined.
- With the lid removed, the container is then placed in the oven for drying.
- After drying, the container is removed from the oven and allowed to cool in a desiccator.
- The lid is then replaced and the mass of container and the dry soil is found.
- The water content is calculated from the following expression:

$$w = (M_2 - M_3) / (M_3 - M_1) \times 100$$

Where, M_1 = mass of container with lid

M_2 = mass of container with lid and wet soil

M_3 = mass of container with dry soil

1.9.1.2 Sand Bath Method: this is a field method of determining rough value of the water content, where the facility of an oven is not available.

Procedure:

- First prepare a sand bath using a metal container.
- The container with the wet soil is placed on a sand bath.
- The sand bath is heated over a kerosene stove.
- The soil becomes dry within ½ to 1 hour.
- The water content is then determined using the following expression:

$$w = ((M_2 - M_3) / (M_3 - M_1)) \times 100$$

Where, M_1 = mass of container with lid

M_2 = mass of container with lid and wet soil

M_3 = mass of container with lid and dry soil

1.9.1.3 Alcohol Method: this is also a field method.

Procedure:

- The wet sample is kept in an evaporating dish and mixed with sufficient quantity of methylated spirit.
- The dish is then properly covered and the mixture is ignited.

- The mixture is kept stirred by a wire during ignition.
- Hence the dry weight of sample is taken after the ignition.
- The water content is determined from the following expression:

$$w = ((M_2 – M_3) / (M_3 – M_1)) \times 10$$

Where, M_1 = mass of empty dish

M_2 = mass of dish and wet soil

M_3 = mass of dish and dry soil

1.9.1.4 Calcium Carbide Method: a device known as “Rapid Moisture Tester” has been developed for rapid determination of water content of a soil sample. Rapid moisture tester is a portable equipment which can be conveniently used in the field as well for the determination of water content.

Procedure:

- In this method, 6 g of wet soil sample is placed in an air-tight container (called moisture tester).
- Then soil is mixed with sufficient quantity of fresh calcium carbide powder.
- The mixture is shaken vigorously.
- The acetylene gas, produced by the reaction of the moisture of the soil and the calcium carbide, exerts pressure on a sensitive diaphragm placed at the end of the container.
- The dial gauge located at the diaphragm reads the water content directly.
- However, the calibration of the dial gauge is such that it gives the water content based on the wet weight of the sample.
- Knowing the water content based on wet weight, the water content based on dry weight can be found from the equation:

$$w = (w' / (1 - w'))$$

Where, w' = water content based on wet weight

w = water content based on dry weight

The method is very quick– the result can be obtained in 5 to 10 minutes. The field kit contains the moisture tester, a small single-pan weighing balance, a bottle containing calcium carbide and a brush.

1.9.1.5 Pycnometer Method: this is also a quick method of determining the water content of those soils whose specific gravity G is accurately known. Pycnometer is a large size density bottle of about 900 ml capacity. A conical brass cap, having a 6 mm diameter hole at its top is screwed to the open end of the pycnometer. A rubber washer is placed between conical cap and the rim of the bottle so that there is no leakage of water.

Procedure:

- Take a clean, dry pycnometer, and find its mass with its cap and washer (M_1).
- Put about 200 g to 400 g of wet soil sample in the pycnometer and find its mass with its cap and washer (M_2).
- Fill the pycnometer to half its height and mix it thoroughly with the glass rod. Add more water, and stir it.
- Replace the screw top and fill the pycnometer flush with the hole in the conical cap. Dry the pycnometer from outside, and find its mass (M_3).
- Empty the pycnometer, clean it thoroughly, and fill it with clean water to the hole of the conical cap, and find its mass (M_4).
- The water content is then calculated from the following expression:

$$w = \{ ((M_2 - M_1) / (M_3 - M_4) \times ((G - 1) / G)) - 1 \} \times 100$$

1.9.1.6 Radiation Method: this method is extremely useful for the determination of water content of soil deposit in the in-situ condition.

Procedure:

- It uses two steel casings – A and B which are placed in two bore holes at some distance apart, in the soil deposit the field moisture content of which is to determined.
- A device containing some radio-active isotope material (such as cobalt – 60) is placed in a capsule which in turn is lowered into casing A.
- Similarly, a detector unit is lowered in steel casing B.
- Small opening are made in both casing A and B, facing each other.
- When the radioactive device is activated, it emits neutrons.
- When these neutrons strike with the hydrogen atoms of water in the sub-soil, they lose energy.
- The loss of energy is evidently equal to water content in the soil.

- The detector device is calibrated to give directly the water content of the subsoil, at that level of emission.
- However, proper shielding precautions should be taken to avoid radiation problems.

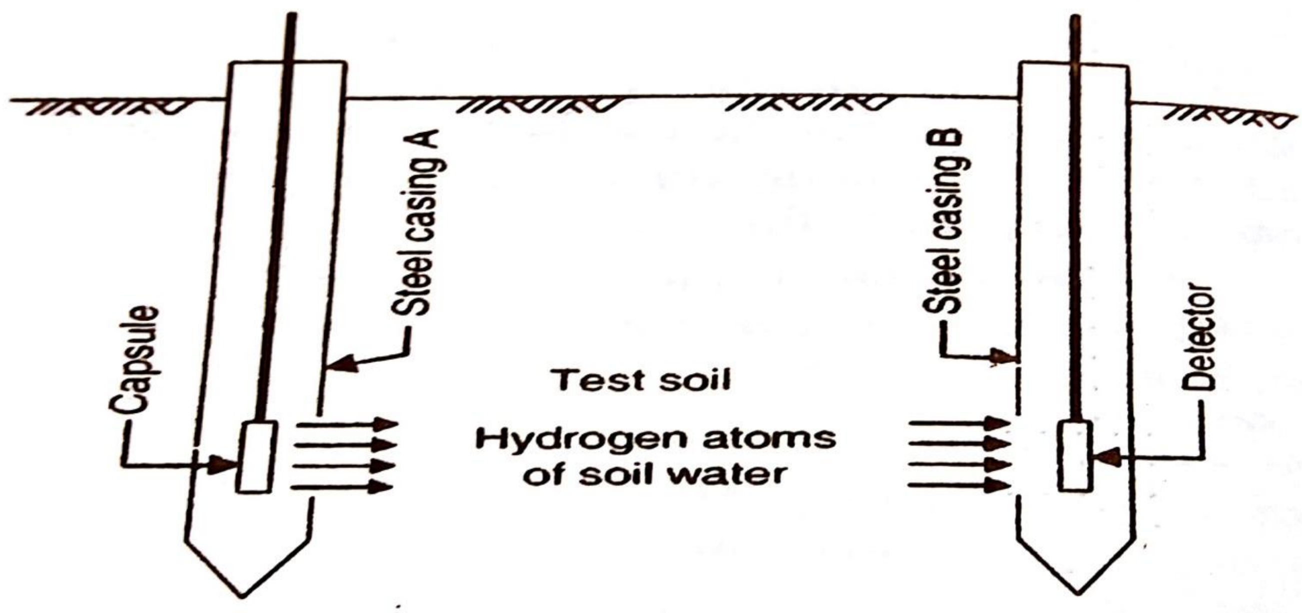


FIG. 3.2 RADIATION METHOD

1.9.1.7 Torsion Balance Method: the equipment has two main parts (i) infra-red lamp and (ii) torsion balance. The infra-red radiation is provided by 250 watt lamp built in the balance for use with alternating current 220-230 V, 50 cycle, and single phase mains supply. The weighing mechanism, a torsion balance, has a built-in magnetic damper to reduce pan vibration during quick drying. The balance scale is divided in terms of water percentages from 1 to 100 water content in 0.2 percent division. The moisture meter is generally calibrated to use 25g of soil.

Procedure:

- The test specimen is kept in a suitable container so that the water content to be determined is not affected by ambient conditions.
- Torque is applied to one end of the torsion wire by means of a calibrated drum to balance the loss of weight of the sample as it dries out under infrared lamp.
- To determine the percent reduction of mass at any instant, rotate the drum scale by turning the drum drive knob until the point returns to the index.
- The percent is read directly from the scale.
- However, this percent (w') is the percent of water based upon the initial mass (i.e wet mass) of the sample.

- The water content (w) based on the dry mass is computed from the equation:

$$w = w' / (1 - w')$$

1.10 SPECIFIC GRAVITY:

The specific gravity of soil solids is determined by: (i) a 50 ml density bottle, (ii) a 500 ml flask, or (iii) a 500 ml pycnometer. The density bottle method is the most accurate, and is suitable for all types of soils. The flask or pycnometer is used only for coarse grained soils. The density bottle method is the standard method used in the laboratory.

Procedure:

- The mass M_1 of the empty, dry, bottle is first taken.
- A sample of oven dried soil, cooled in a desiccator, is put in the bottle, and mass M_2 is taken.
- The bottle is then filled with distilled water gradually, removing the entrapped air either by applying vacuum or by shaking the bottle.
- The mass M_3 of the bottle, soil and water is taken.
- Finally, bottle is emptied completely and thoroughly washed, and clean water is filled to the top, and the mass M_4 is taken.
- Based on these four observation, the specific gravity can be computed as follows:

$$G = (M_2 - M_1) / (M_2 - M_1) - (M_3 - M_4)$$

Soil Type	Specific Gravity
Quartz sand	2.64 – 2.65
Silt	2.68 – 2.72
Silt with Organic matter	2.40 – 2.50
Clay	2.44 – 2.92
Bentonite	2.34
Loess	2.65 – 2.75
Lime	2.70
Peat	1.26 – 1.80
Humus	1.37

1.11 PARTICLE SIZE DISTRIBUTION:

The percentage of various sizes of particles in a given dry soil sample is found by a particle size analysis or mechanical analysis. By mechanical analysis is meant the separation of a soil into its different size fractions. The mechanical analysis is performed in two stages (i) sieve analysis and (ii) sedimentation analysis or wet mechanical analysis. The first stage is meant for coarse grained soils only, while the second stage is performed for fine grained soils. In general, a soil sample may contain both coarse grained particles as well as fine particles, and hence both the stages of the mechanical analysis may be necessary. The sieve analysis is, however, the true representative of grain size distribution, since the test is not affected by temperature etc.

1.11.1 Sieve Analysis: in the BS and ASTM standards, the sieve sizes are given in terms of the number of openings per inch. The number of openings per square inch is equal to the square of the number of the sieve. In the Indian Standards (IS : 460-1962), the sieves are designated by the size of the aperture in mm. The complete sieve analysis can be divided into two parts – the coarse analysis and fine analysis. An oven-dried sample of soil is separated into two fractions by sieving it through a 4.75 mm IS sieve. The portion retained on it is termed as the gravel fraction and is kept for the coarse analysis, while the portion passing through it termed as fine fraction and is kept for fine analysis.

Procedure:

- Take 1000g of oven dried soil sample for the sieve analysis.
- Arrange the set of sieve: 4.75mm, 2.36mm, 1.18mm, 600μ, 425μ, 300μ, 212μ, 150μ, 75μ and pan.
- Take the empty weight of each sieve.
- Place the set of sieve on a sieve shaking machine.
- Pour 1000g of oven dried soil sample on the top sieve.
- Switch on the sieve shaker machine and sieve it for about 10 minutes.
- Then note down the mass of soil which is retained on the each sieves.
- On the basis of total weight of sample taken and the weight of soil retained on each sieve, the percentage of the total weight of soil passing through each sieve can be calculated as below:

Percentage Retained on Particular Sieve = (Weight of soil retained on that sieve) / (Total weight of soil taken) x 100

Cumulative Percentage Retained = Sum of percentage retained on all sieves of larger sizes and the percentage retained on that particular sieve.

Percentage Finer = 100 % - Cumulative percentage retained

- Draw a graph by keeping particle diameter (mm) on x-axis and percentage finer on y-axis in a semi log sheet thus a smooth "S" curve is obtained.
- Using this "S" curve we have to determine the uniformity co-efficient, coefficient of curvature and sorting coefficient.

Sieve No.	Weight of Sieve	Weight of Sieve + Soil	Weight of Soil	Percentage Weight Retained	Cumulative Percentage Retained	Percentage Finer
4.75mm						
2.36mm						
1.18mm						
600µ						
425µ						
300µ						
212µ						
150µ						
75µ						
Pan						

1.11.2 Sedimentation Analysis or Wet Mechanical Analysis: in the sedimentation analysis, the soil fraction, finer than 75 micron size is kept in suspension in a liquid (usually water) medium. The analysis is based on Stoke's Law, according to which the velocity at which grains settle out of suspension, all other factors being equal, is dependent upon the shape, weight and size of the grain. However, in the usual analysis it is assumed that the soil particles are spherical and have the same specific gravity. With this assumption, the coarser particles settle more quickly than the finer ones. If v is the terminal velocity of sinking of a spherical particle, it is given by:

$$v = (2/9) \times r^2 \times (\gamma_s - \gamma_w) / \eta$$

or

$$v = (1/18) \times D^2 \times (\gamma_s - \gamma_w) / \eta$$

r = radius of the spherical particle (m)

D = diameter of the spherical particle (m)

v = terminal velocity (m/sec)

γ_s = unit weight of particle (kN/m^3)

γ_w = unit weight of water (kN/m^3)

η = viscosity of water or liquid (kN-s/m^2) = μ / g

μ = viscosity in absolute units of poise

g = acceleration due to gravity

1.11.2.1 Assumptions:

- Soil particle are spherical.
- Particles settle independent of other particles and the neighbouring particles do not have any effect on its velocity of settlement.
- The walls of jar, in which the suspension is kept, also do not affect the settlement.
- An average value of specific gravity is used.
- The finer grains of the soil carry charge on their surface and have a tendency for floc formation.

1.11.2.2 Limitations:

- In actual practice, the fine particles of soil, for which this analysis is primarily meant, are not truly spherical.
- The particles of fine grained soils are thin platelets which do not settle out of suspension in the same manner and at the same rate as smooth spheres.
- When the particle diameter is about 0.2mm beyond which liquid tends to develop a turbulent motion at the boundaries of the particles.
- For particle smaller than 0.0002mm, Stokes law no longer remains valid.

1.11.3 Hydrometer Method: the hydrometer method of sedimentation analysis differs from the pipette analysis in the method of taking the observation – the principles of the test being the same in both the cases. In the hydrometer analysis, M_D is computed indirectly by reading the density of the soil suspension at a depth H_e at various time intervals. In the hydrometer test, the sampling depth H_e (also known as the effective depth) goes on increasing as the particles settle with the increase in the time intervals. It is therefore, necessary to calibrate the hydrometer and the sedimentation jar before the start of the sedimentation test, to find a relation between H_e and the density readings of the hydrometer.

1.11.3.1 Calibration of Hydrometer:

- Fig 3.6 (a) shows the hydrometer.
- The readings on the hydrometer stem give the density of the soil suspension situated at the centre of the bulb at any time.
- For convenience, the hydrometer readings are recorded after subtracting 1 and multiplying the remaining digits by 1000.
- Such a reduced reading is designated as R_h.
- If the density reading at the intersection of horizontal surface of soil suspension with the stem, is 1.010, it is recorded as 10 (i.e., $R_h = 10$).
- Similarly, a density reading of 0.995 is recorded as $R_h = -5$.
- As indicated in Fig. 3.6 (a) the hydrometer readings increase in downward direction towards the hydrometer bulb.
- Let H be the height, in cm, between any hydrometer reading R_h and the neck, and h the height of the bulb.
- Fig. 3.6 (b) shows the jar, containing the soil suspension.
- When the hydrometer is immersed in the jar [Fig. 3.6 (c)], the water level aa rises to a_1a_1, the rise being equal to the volume V_h of the hydrometer divided by the internal area of cross-section A of the jar.
- Similarly, the level bb rises to b_1b_1.
- The rise between bb and b_1b_1 will be approximately equal to $V_h / 2A$.
- Therefore, we have:

$$H_e = \{H + (h / 2) + (V_h / 2A) – (V_h / A)\}$$

$$H_e = H + 0.5 (h – V_h / A)$$

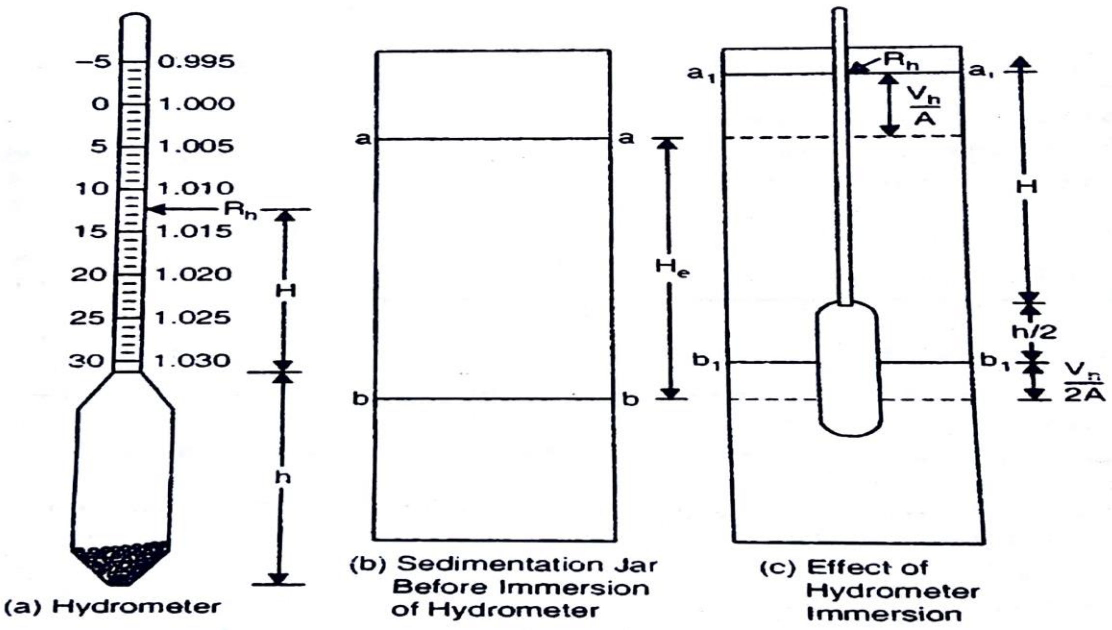


FIG. 3.6. HYDROMETER ANALYSIS.

Procedure:

- A soil of about 50g dry mass is mixed with distilled water and made into a thin paste.
- The paste is mixed well with a suitable quantity of deflocculating agent.
- Transfer the paste into a sedimentation jar and add the distilled water to bring the level to the 1000ml.
- The suspension is mixed well by placing the palm of the hand over the open end turning the jar upside down and back.
- The jar is next placed on the table.
- The hydrometer is inserted carefully into the suspension and the timer started.
- Readings are taken at the time interval of ½, 1, 2 and 4 minutes.
- After 4 minutes remove the hydrometer.
- The suspension is remixed as before, the hydrometer is inserted.
- The subsequent readings are taken at an time interval of 8, 15, 30, 60, 120, 240, 480 and 1440 minutes.
- After each reading in the suspension, the hydrometer should take out and inserted into another jar of distilled water.

1.11.3.2 Correction to the Hydrometer Readings: the following corrections are applied to the hydrometer readings:

1.11.3.2.1 Meniscus correction

1.11.3.2.2 Dispersing agent correction

1.11.3.2.3 Temperature correction

- The hydrometers are generally calibrated at 27° C.
- If the temperature of the soil suspension is not 27° C, a temperature correction C_t should be applied to the observed hydrometer readings.
- If the test temperature is more than 27° C, the hydrometer readings will naturally be less and hence the temperature correction will be positive.
- If the test temperature is lower than 27° C, the temperature correction will be negative.
- Since the soil suspension is opaque, the hydrometer reading is taken at the top of the meniscus.
- Actual reading, to be taken at the water level, will be more since the readings increase in the downward direction. Hence the meniscus correction C_m is always positive.
- Its magnitude can be found by immersing hydrometer in a jar containing clear water, and finding the difference between the reading corresponding to the top and bottom of the meniscus.
- The addition of dispersing agent in water increases its density, and hence the dispersing agent correction C_d is always negative.
- Thus the corrected hydrometer reading R is given by:

$$R = R_h' + C_m \pm C_t - C_d$$

Where, R_h' = observed hydrometer reading at the top of the meniscus

1.11.4 Particle Size Distribution Curve:

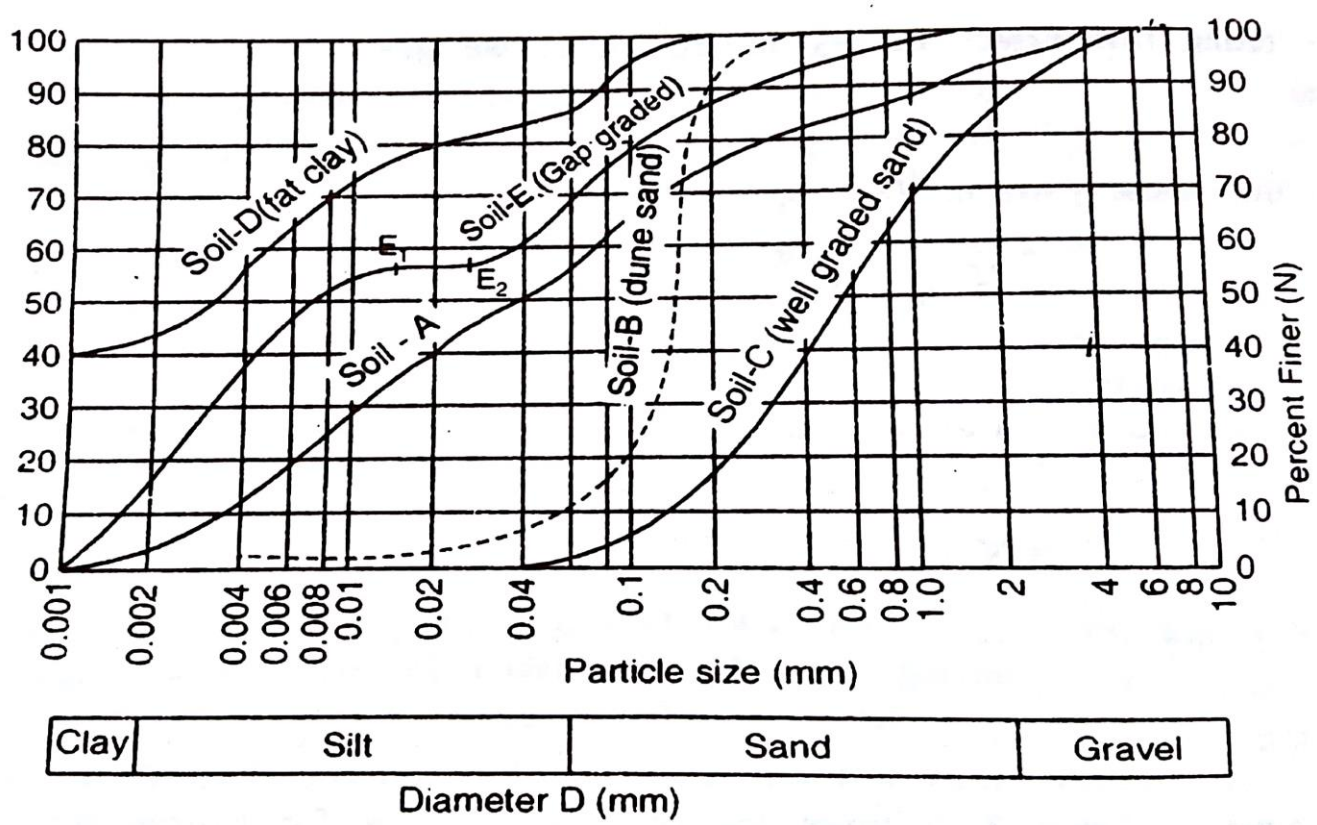


FIG. 3.8. PARTICLE SIZE DISTRIBUTION CURVE

- The results of the mechanical analysis are plotted to get a particle-size distribution curve with the percentage finer N as the ordinate and the particle diameter as the abscissa, the diameter being plotted on logarithmic scale.
- Fig. 3.8 shows some typical curves for various soils.
- A particle size distribution curves gives an idea about the type and gradation of the soil.
- A curve situated higher up or to the left represents a relatively fine grained soil while a curve situated to the right represents a coarse grained soil.
- A soil is said to be well graded when it had good representation of particles of all sizes.\
- On the other hand, a soil is said to be poorly graded if it has an excess of certain particles and deficiency of other.
- Soil contains most of the particles of about the same size, it is known as a uniformly graded soil.
- Thus soil A is well graded while the soil B is uniformly graded.
- A curve with a flat portion represents a soil in which some intermediate size particles are missing (i.e., Soil E). Such a soil is known as gap graded or skip graded.

1.11.4.1 Parameters: The particle size distribution curve can be used to determine the following parameters for a given soil:

1.11.4.1.1 Effective Size

1.11.4.1.2 Uniformity Coefficient or Coefficient of Uniformity

1.11.4.1.3 Coefficient of Curvature or Coefficient of gradation

1.11.4.1.1 Effective Size or Effective Diameter (D_{10}): the diameter of the particle size distribution curve corresponding to 10 percent finer.

1.11.4.1.2 Uniformity Coefficient or Coefficient of Uniformity (C_u): it is defined as the ratio of diameter of the particle corresponding to 60 percent finer to the diameter of particle corresponding to 10 percent finer.

$$C_u = D_{60} / D_{10}$$

1.11.4.1.3 Coefficient of Curvature or Coefficient of gradation (C_c): it is defined as the ratio of diameter of particle corresponding to square of 30 percent finer to the diameter of particle corresponding to 60 and 10 percent finer.

$$C_c = (D_{30})^2 / (D_{60} \text{ x } D_{10})$$

1.12 CONSISTENCY OF SOILS:

By consistency is meant the relative ease with which soil can be deformed. This term is mostly used for the fine grained soils for which the consistency is related to a large extent to water content. Consistency denotes degree of firmness of the soil which may be termed as soft, stiff or hard. Fine grained soil may be mixed with water to form a plastic paste which can be moulded into any form by pressure. The addition of water reduces the cohesion making the soil still easier to mould. Further addition of water reduces the cohesion until the material no longer retains its shape under its own weight, but flows as a liquid. If water is evaporated from such a soil suspension, the soil passes through various stages of consistency. In 1911, the Swedish agriculturist Atterberg divided the entire range from liquid to solid state into four stages: (i) the liquid state, (ii) the plastic state, (iii) the semi-solid state and (iv) the solid state. He set arbitrary limits, known as consistency limits or Atterberg limits, for these divisions in terms of water content. Thus, the consistency limits are the water contents at which the soil mass passes from one state to the next. Fig. 3.9 shows the four stages of consistency, with the appropriate consistency limits. The Atterberg limits which are most useful for engineering purposes are: liquid limit, plastic limit and shrinkage limit. These limits are expressed as per cent water content.

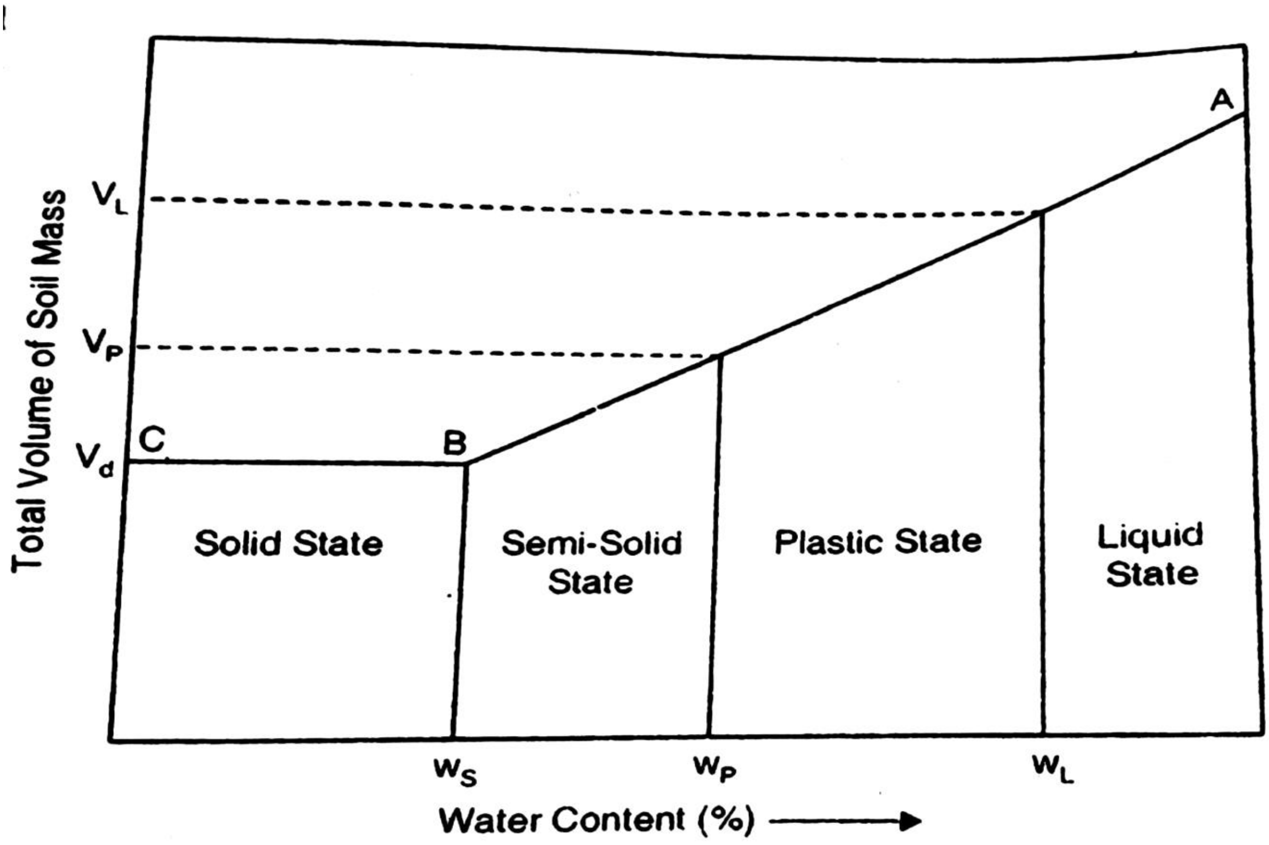


FIG. 3.9. CONSISTENCY LIMITS

1.12.1 Liquid Limit (W_L): with reference to the above figure, liquid limit is the water content corresponding to the arbitrary limit between liquid and plastic state of consistency of a soil.

Or

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 which can be measured by standard available means.

Or

With reference to the standard liquid limit device, it is defined as the minimum water content at which a part of soil cut by a groove of standard dimensions will flow together for a distance of 12mm under an impact of 25 blows in the device.

Or

It is defined as the water content at which the soil changes from the plastic state to liquid state.

1.12.2 Plastic Limit (W_P): with reference to the above figure, plastic limit is the water content corresponding to an arbitrary limit between the plastic and the semi-solid states of consistency of a soil.

Or

It is defined as the minimum water content at which a soil will just begin to crumble when rolled into a thread approximately 3mm in diameter.

Or

It is defined as the water content at which the soil changes from the plastic state to semi-solid state.

1.12.3 Shrinkage Limit (W_S): 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.

Or

It is defined as the water content at which the soil changes from semi-solid to solid state.

1.12.4 Plasticity Index (I_P): is defined as the numerical difference between the liquid limit and the plastic limit of a soil.

$$I_P = W_L - W_P$$

In case of sandy soils, plastic limit should be determined first. When plastic limit cannot be determined, the plasticity index is reported as non-plastic. When the plastic limit is equal to or greater than the liquid limit, the plasticity index is reported as zero.

1.12.5 Plasticity: is defined as that property of a soil which allows it to be deformed rapidly, without rupture, without elastic rebound and without volume change.

1.12.6 Shrinkage Index (I_S): is defined as the difference between the plastic limit and shrinkage limit of a soil.

$$I_S = W_P – W_S$$

1.12.7 Consistency Index or Relative Consistency (I_C): is defined as the ration of the difference between liquid limit and water content to the plasticity index of a soil.

$$I_C = (W_L – w) / I_P$$

1.12.8 Liquidity Index (I_L): is defined as the ratio of the natural water content minus its plastic limit to its plasticity index.

$$I_L = (w - W_P) / I_P$$

1.12.9 Toughness Index (I_T): is defined as ratio of plasticity index to flow index of soil.

$$I_T = I_P / I_F$$

1.12.10 Flow Index (I_F): is defined as

$$I_F = (W_1 - W_2) / \text{Log}_{10} (N_2 / N_1)$$

1.12.11 Determination of Liquid Limit: the liquid limit can be determined by two methods:

1.12.11.1 Casagrande Liquid Limit Method

1.12.11.2 Static Cone Penetration Method

1.12.11.1 Casagrande Liquid Limit Method: the liquid limit is determined in the laboratory with the help of the standard liquid limit apparatus designed by Casagrande. The apparatus (Fig. 3.10) consists of a hard rubber base of B S hardness 21-25, over which a brass cup drops through a desired height. The brass cup can be raised and lowered to fall on the rubber base with the help of a cam operated by a handle. The height of fall of the cup can be adjusted with the help of adjusting screws. Before starting the test, the height of fall of the cup is adjusted to 1cm. Two types of grooving tools are used (i) the Casagrande tool and (ii) ASTM tool. The Casagrande tool cuts a groove of size 2 mm wide at the bottom, 11 mm wide at the top and 8 mm high. ASTM tool cuts a groove 2 mm wide at the bottom, 13.6 mm at the top and 10 mm deep. The ASTM tool is used only for more sand soils where the Casagrande tool tends to tear the sides of the groove.

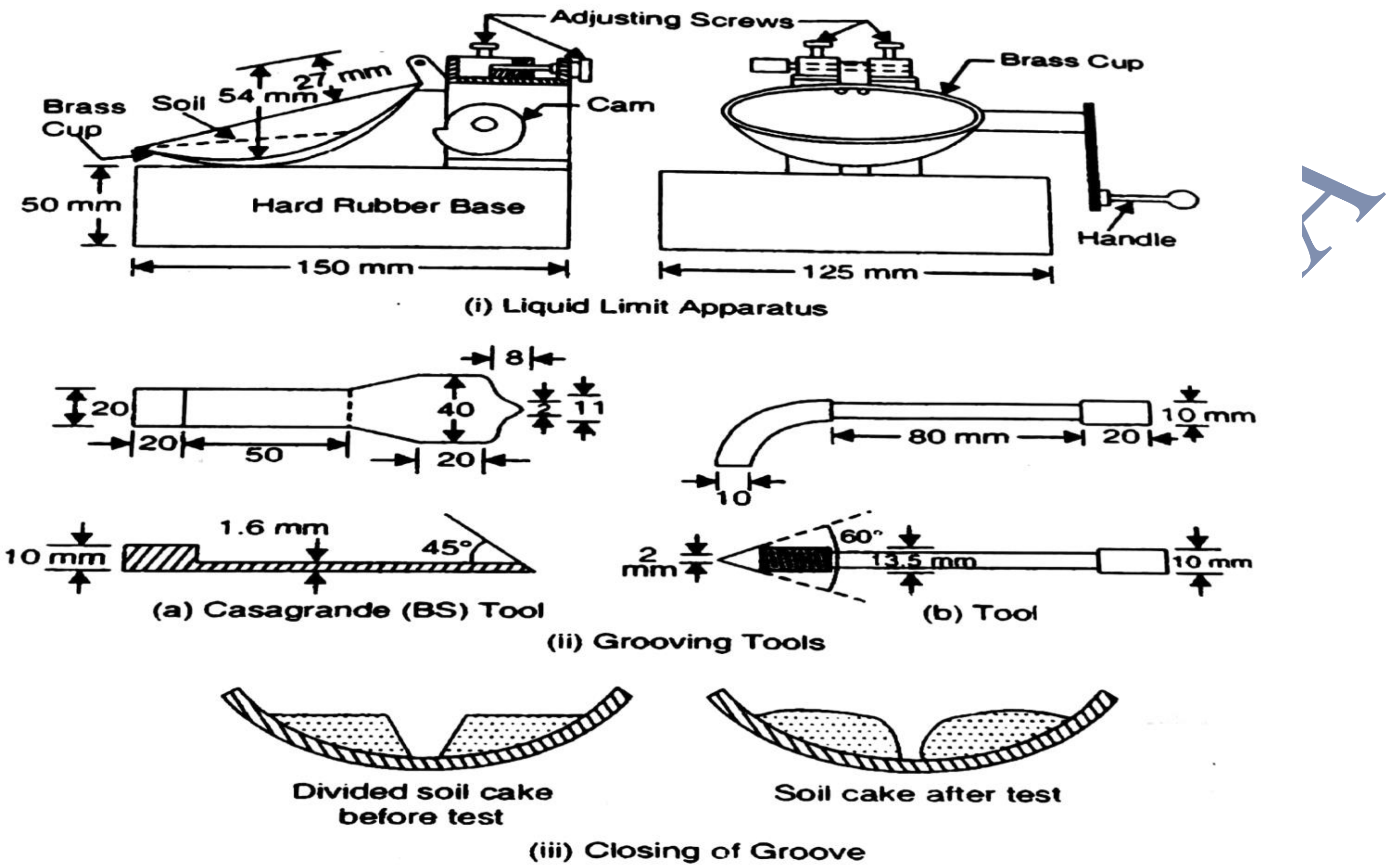


FIG. 3.10. LIQUID LIMIT APPARATUS.

Procedure:

- About 120g of the specimen passing through 425 micron sieve is mixed thoroughly with distilled water in the evaporation dish or on a marble plate to form a uniform paste.
- A portion of the paste is placed in the cup over the spot where the cup rests on the base, squeezed down and spread into position and the groove is cut in the soil pat (Fig. 3.10 (iii)).
- The handle is rotated at a rate about 2 revolutions per second, and the numbers of blows are counted until the two parts of the soil sample come into contact at the bottom of the groove along a distance of 10 mm.
- Some soils tend to slide on the surface of the cup instead of the flowing.
- If it occurs, the result should be discarded and the test repeated until flowing does not occur.
- After recording the number of blows, approximately 10 gram of soil from near the closed groove is taken for water content determination.

- The liquid limit is determined by plotting a graph between number of blows as abscissa on a logarithmic scale and the corresponding water content as ordinate. This graph is known as the flow curve, is straight line.
- For plotting the flow curve (Fig. 3.11), at least four to five sets of reading in the range of 10 to 50 blows should be taken. The water content corresponding to 25 blows is taken as the liquid limit.
- Flow index can also be determined form the flow curve.
- If the flow curve is extended at either end so as to intersect the ordinates corresponding to 10 and 100 blows the numerical difference in water content at 10 and 100 blows gives directly the flow index.

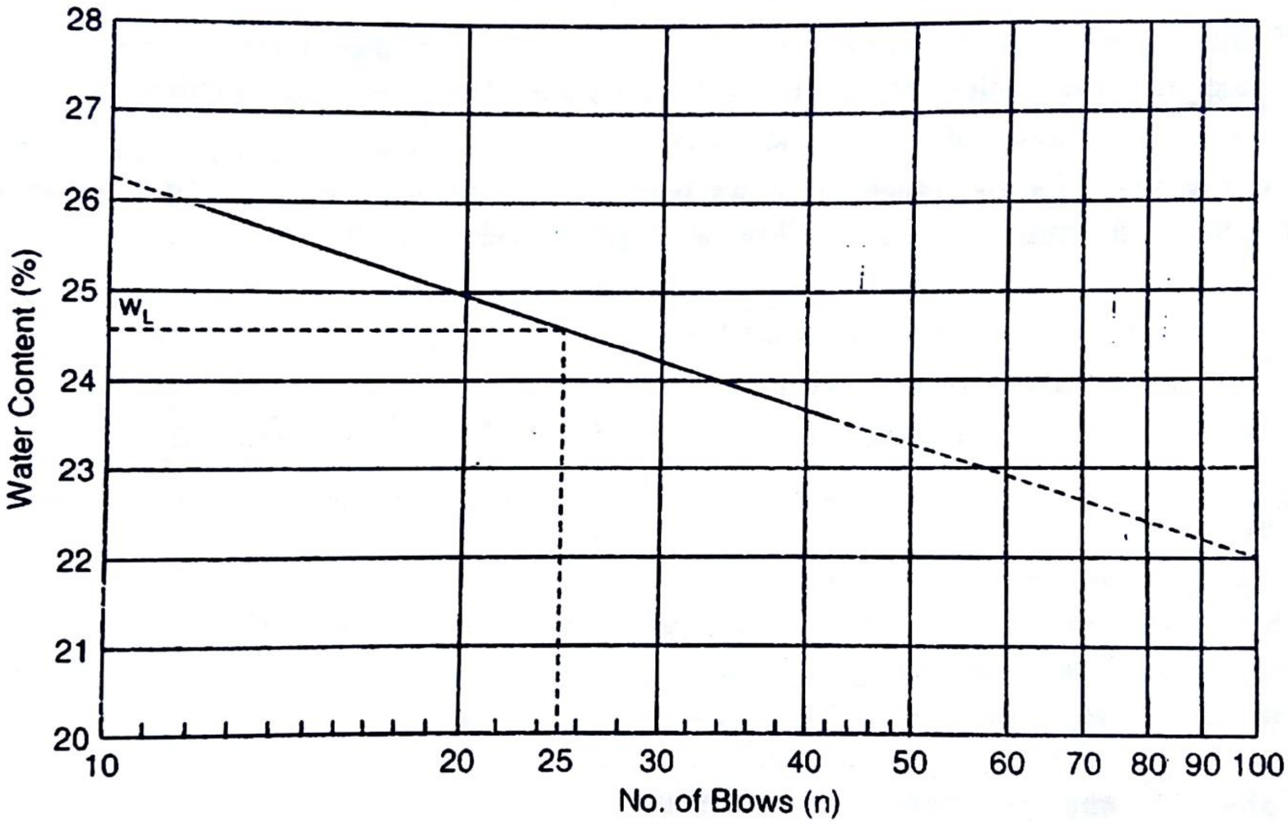


FIG. 3.11. FLOW CURVE.

1.12.11.2 Static Cone Penetration Method: the Soviet liquid limit device is based on the principle of static penetration. A 30° cone of stainless steel is made to penetrate the soil pat, under a mass of 75 grams inclusive of the mass of the cone. If the cone penetrates through a depth of 1cm in 5 seconds, the soil pat is at the liquid limit. IS: 2720 (Part V): 1985 specifies a similar penetrometer (Fig. 3.12) for the determination of liquid limit. The cone has a central angle of 31° and the total sliding mass of 80 g. The soil pat is kept in a cylindrical trough, 5 cm in diameter and 5 cm in high, below the cone. The liquid limit of the soil corresponds to the water content of a paste which would give 20 mm penetration of the soil.

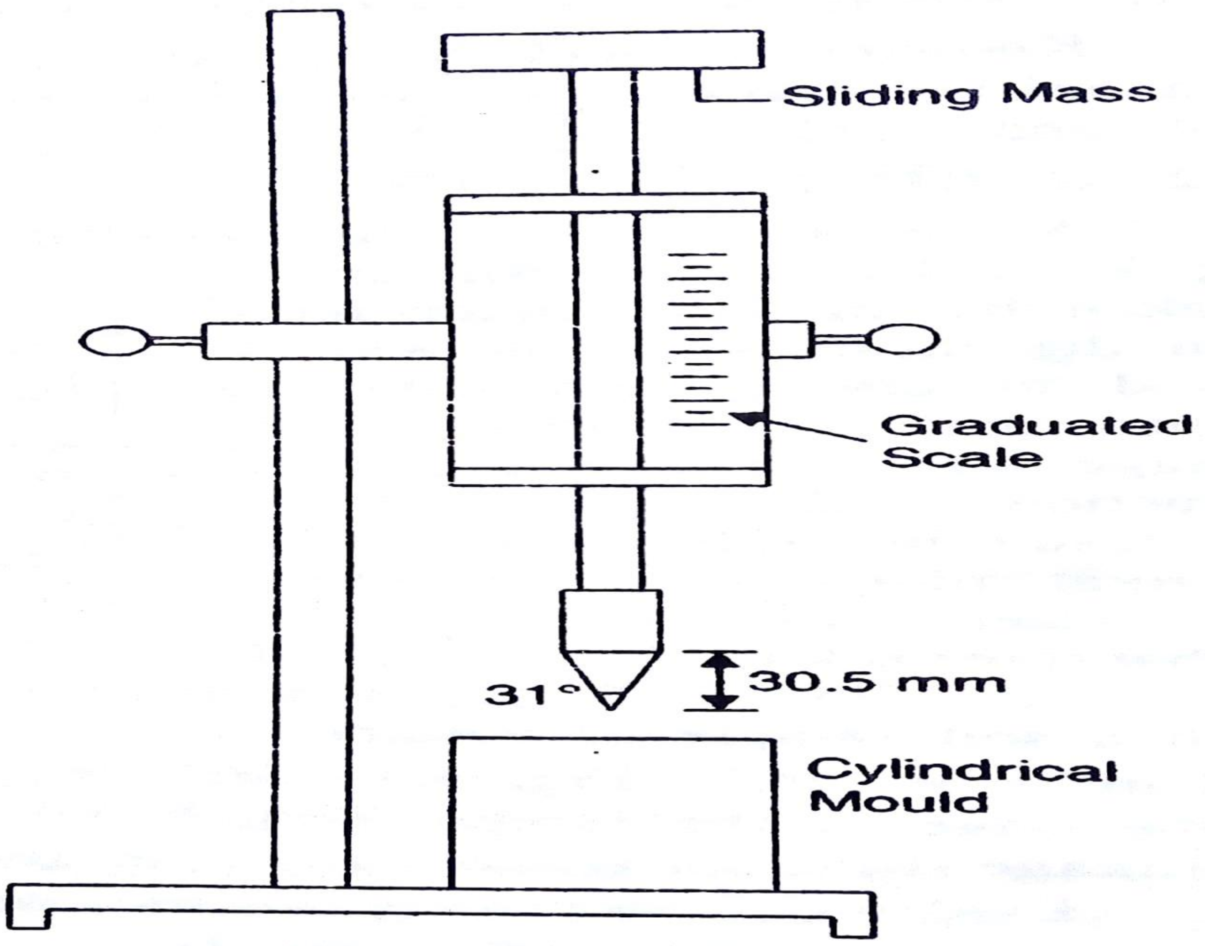


FIG. 3.12. CONE PENETRATION. IS : 2720 (PART V)-1965.

Procedure:

- Soil pats are prepared at various water contents and depth of penetration (x) for each pat is noted.
- A graph is plotted representing water content (w) on the y-axis and cone penetration (x) on the x-axis.
- The best fitting straight line is drawn. The water content corresponding to a cone penetration of 20 mm is then taken as the liquid limit.
- The set of values used for the graphs should be such that the values of penetration are in the range of 14 to 28 mm.

- Alternatively, W_L can be determined by one point method using cone penetrometer, from any one of the following relationships:

$$W_L = w / (0.65 + 0.0175\ x\ )$$

- The above expression is applicable only if the depth of penetration is between 20 to 30 mm.

1.12.12 Determination of Plastic Limit:

Procedure:

- To determine the plastic limit, the soil specimen, passing through 425 micron sieve, is mixed thoroughly with distilled water until the soil mass becomes plastic enough to be easily moulded with fingers.
- The plastic soil mass should be left for enough time to allow water to permeate through the soil mass.
- A ball is formed with about 8 g of this plastic soil mass and rolled between the fingers and a glass plate with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length.
- When a diameter of 3 mm is reached, the soil is remoulded again into a ball.
- This process of rolling and remoulding is repeated until the thread starts just crumbling at a diameter of 3 mm.
- The crumbled threads are kept for the water content determination.
- The test is repeated twice more with fresh samples.
- The plastic limit W_P is then taken as the average of three water contents. The plasticity and toughness index are also determined.

1.12.13 Determination of Shrinkage Limit: shrinkage limit of a soil can be determined by any one of the following methods:

1.12.12.1 Determination of shrinkage limit, when the specific gravity of the soil is unknown.

1.12.12.2 Determination of shrinkage limit, when the specific gravity of the soil is known.

Procedure:

- The apparatus consists of a porcelain evaporating dish of about 12 cm diameter with bottom, a stainless steel shrinkage dish, 45 mm in diameter and 15 mm high, two glass plates of each 75 mm x 75 mm one is a plain glass and the other having three metal prongs, a glass cup of 50 mm in diameter and 25 mm high, straight edge, spatula, oven, mercury desiccator, balance and sieves.

- The volume of the shrinkage dish is first determined by filling it with mercury, remove the excess mercury by pressing a flat glass plate over the top and then take the mass of the dish filled with mercury.
- The mass of the mercury divided by its unit weight gives the volume of the wet soil sample.
- Take 50 g of oven dry soil passing through 425 micron IS sieve.
- The soil is mixed with sufficient quantity of water to bring the soil to a consistency.
- The inside of the shrinkage dish is coated with a thin layer of vaseline.
- The soil mixture is placed in the shrinkage dish in three equal quantities so as to fill the dish.
- The excess soil is removed with a straight edge and the dish is weighed with soil.
- The dish is then placed in an oven at 110C and the soil pat is allowed to dry up.
- The weight of dry soil pat is found by weighing.
- In order to find the volume of dry soil pat the glass cup is filled with mercury and excess is removed by pressing the glass plate with three prongs firmly over the top.
- Then place the glass cup in the evaporating dish.
- The dry soil pat is placed on the surface of the mercury in the cup and carefully pressed by means of the glass plate with prongs.
- The mass of the mercury so displaced divided by its density gives the volume of dry soil pat.
- The shrinkage limit is then calculated by using the formula:

$$W_S = w - ((V_w - V_d) / M_d) \times 100$$

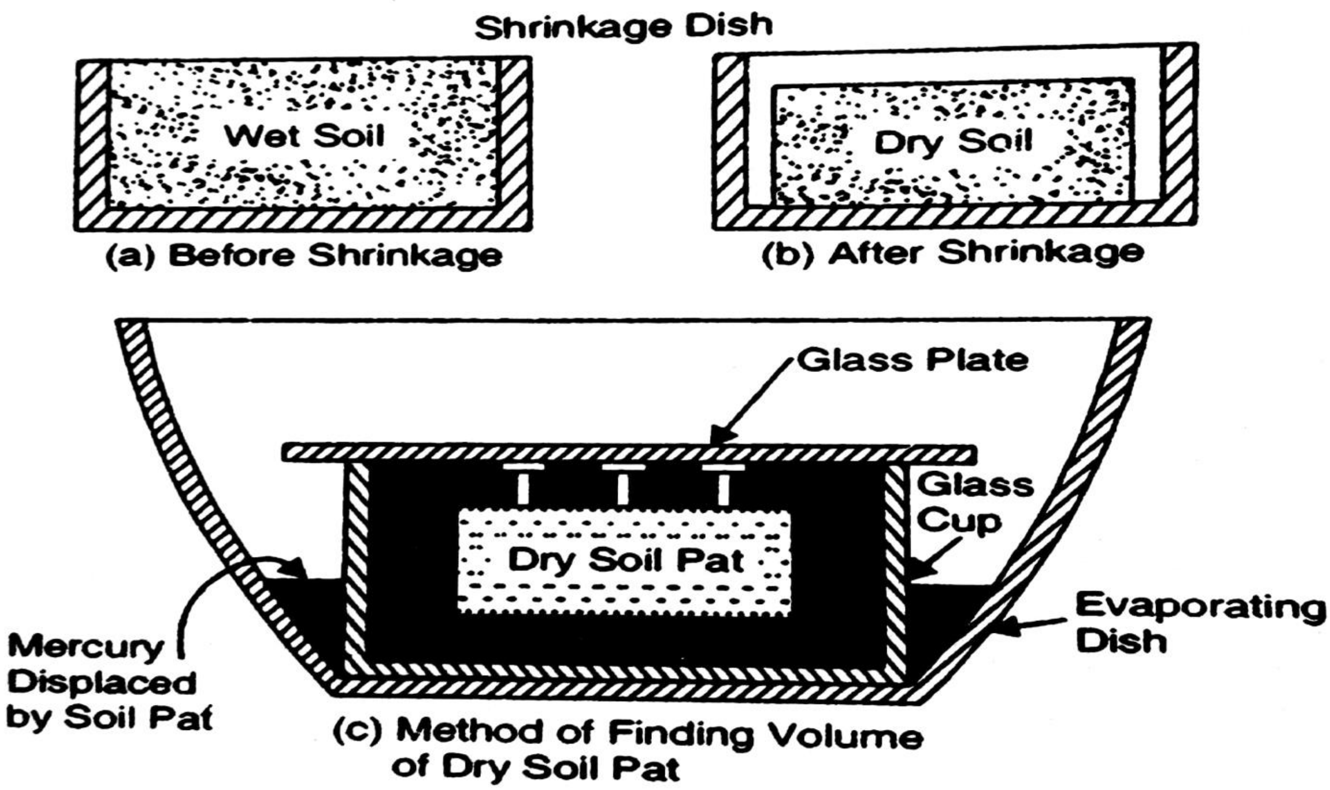


FIG. 3.14. APPARATUS OF SHRINKAGE LIMIT DETERMINATION.

1.12.14 Shrinkage Ratio (SR): is defined as the ratio of a given volume change expressed as a percentage of dry volume, to the corresponding change in water content above the shrinkage limit expressed as a percentage of the weight of the oven dried soil.

$$SR = \{(V_1 - V_2) / V_d \times 100\} / (w_1 - w_2)$$

1.12.15 Volumetric Shrinkage (VS): is defined as the decrease in the volume of a soil mass, expressed as a percentage of the dry volume of the soil mass, when the water content is reduced from a given percentage to the shrinkage limit.

$$VS = ((V_1 - V_d) / V_d) \times 100$$

1.12.16 Linear Shrinkage (L_S): is defined as the decrease in one dimension of a soil mass expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit.

$$L_S = 100\ [1- \{100 / (VS + 100)\}^{1/3}]$$

1.13 DETERMINATION OF IN-SITU OR FIELD DENSITY:

The field density of a natural soil deposit or of a compacted soil can be determined by the following methods:

1.13.1 Sand replacement method

1.13.2 Core cutter method

1.13.3 Water displacement method

1.13.4 Rubber balloon method

1.13.1 Sand Replacement Method: the equipment consists of (i) sand pouring cylinder mounted above a pouring cone and separated by a valve or shutter, (ii) calibrating container, (iii) tray with central circular hole, (iv) chisel, scoop, balance etc. The procedure consists of (a) calibration of the cylinder, (b) measurement of a soil density, (c) determination of water content and dry density.

Procedure:

- Calibration of the cylinder consists of the determination of the weight of sand required to fill the pouring cone of the cylinder, and the determination of the bulk density of sand.
- Uniformly graded, dry, clean sand preferably passing through a 600 micron sieve and retained on 300 micron IS sieve is used in the cylinder.
- The cylinder is filled upto a height 1 cm below the top, and its initial mass M_1 is taken.
- The sand is run out of cylinder, equal in volume to that of the calibrating container.
- The cylinder is then placed over a plane surface and the sand is allowed to run out to fill the cone below.
- When no further sand runs out, the value is closed. The sand filled in the cone is collected, and its mass M_2 is found.
- All the sand is refilled in the cylinder so that the total mass of sand and cylinder is equal to the original mass M_1.
- The cylinder is then put centrally above the calibrating container, and the sand is allowed to run into the calibrating container.
- The value is closed when there is no further movement of sand and mass of cylinder with sand is found M_3.
- The mass M' of the sand required to fill the calibrating container will be equal to $M_1 – M_2 – M_3$.
- The mass M' divided by the volume of the calibrating container gives the bulk density of the sand. All the sand is then refilled in the cylinder.

- For measurement of soil density the site is cleaned and levelled, and the tray placed over it.
- A test hole, approximately of a depth equal to that of the calibrating container is excavated in the ground, and the soil is collected in the tray.
- The mass M of the excavated soil is found.
- The cylinder is centrally placed over the hole, and the sand is allowed to run in it. The value is closed when no further movement of sand takes place.
- The mass M_4 of the cylinder and the remaining sand in it is measured.
- The mass M'' will be equal to $M_1 - M_3 - M_4$. Dividing M'' by the bulk density of sand, the volume of the hole and hence the volume V of the excavated soil is known.
- Dividing the mass M by the volume V, the bulk density of the soil excavated is known.
- A suitable sample of the excavated soil is kept for water content determination. The dry density of the soil will be equal to the bulk density divided by (1+w).

1.13.2 Core Cutter Method: a core cutter, consisting of a steel cutter, 10 cm in diameter and about 13 cm high, and a 2.5 cm high dolly, weighing balance, knife, steel scale and oven.

Procedure:

- First calculate the volume of the core cutter.
- Note down the empty weight of core cutter as M_1 g.
- Place the core cutter on the surface of soil where the field density is to be determined.
- Place the dolly on the above of the core cutter.
- The core cutter with dolly is driven in the cleaned surface with the help of a suitable rammer, till about 1 cm of the dolly protrudes above the surface.
- The cutter, containing the soil, is dug out of the ground, the dolly is removed and the excess soil is trimmed off.
- Note down the mass of core cutter along with soil as M_2 g.
- Mass of soil in cutter is found by $M_2 - M_1$ g.
- By dividing it by the volume of cutter the bulk density is determined i.e., $(M_2 - M_1) / V$.
- The water content of the excavated soil is found in the laboratory. The dry density of the soil will be equal to the bulk density divided by (1+w).

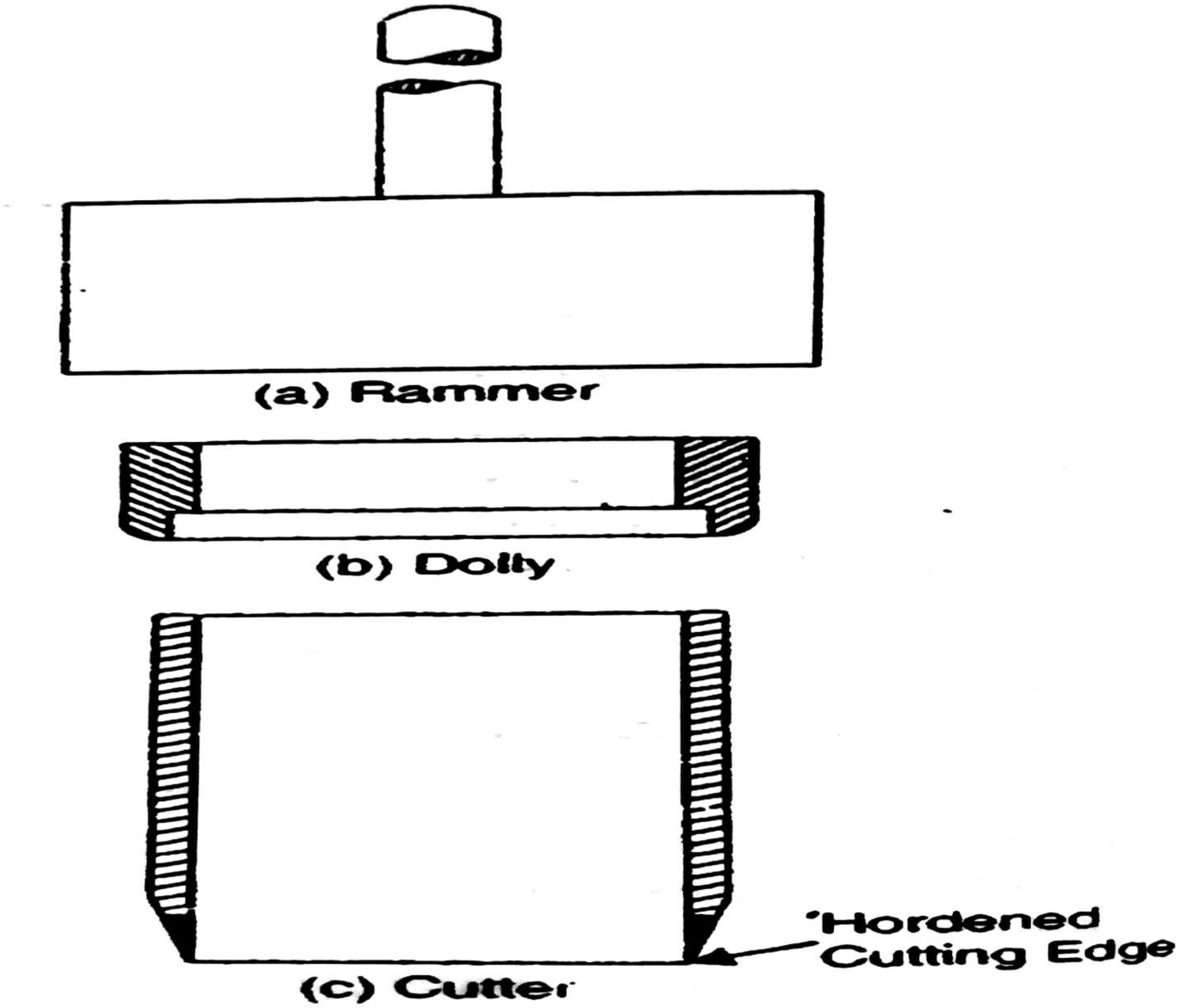


FIG. 3.20. CORE CUTTER.

1.13.3 Water Displacement Method: the method is suitable only for cohesive soil samples brought from the field.

Procedure:

- A small specimen is trimmed to a more or less regular shape, from a larger sample, and its mass M_1 is found.
- The specimen is covered with a thin layer of paraffin wax and the mass M_2 of the coated specimen is taken.
- A metal container is filled above the overflow level, and excess water is allowed to run off through the overflow outlet.

- The coated specimen is then slowly immersed in the container, and the overflow water is collected in a measuring jar.
- The volume V_w of the displaced water is thus known.
- The volume of the uncoated specimen is then calculated from the relation:

$$V = V_w - ((M_2 - M_1) / G_p)$$

Where G_p = density of paraffin wax (g/ml)

1.13.4 Rubber Balloon Method: in this method, the volume of the excavated hole is measured with the help of an inflated rubber balloon. The apparatus consists of (i) a graduated glass or lucite cylinder enclosed in an air-tight aluminium case, with an opening in the bottom, (ii) a tray with central circular hole of 10 cm diameter. The cylinder is partially filled with water. Pressure or vacuum can be applied to the bottom of the cylinder with the help of a double acting rubber bulb.

Procedure:

- The ground surface, where the density is to be determined is cleaned and levelled, and tray is placed over it.
- The cylinder is then placed centrally over the tray.
- The air value is opened and air is pumped into the cylinder until the balloon is completely inflated against the surface of the soil in the opening of the tray.
- The water level is read in the cylinder. The cylinder is then removed, and a hole is excavated in the ground.
- The excavated soil is weighed, and a sample is kept for water content determination.
- The cylinder is then placed over the opening in the tray, air value is opened and air is forced in the cylinder to inflate the bottom, until the base of the instrument is raised off the tray at least by 1 cm.
- The air valve is closed and both feet are placed firmly on the base plate so that the balloon is forced into any irregularities in the hole.
- The water level is read in the cylinder.
- The volume of the hole is found from the difference between the initial and final water level, in the glass cylinder.
- Knowing the mass, volume and water content, the bulk density and dry density can be computed.

1.14 ACTIVITY OF CLAYS:

The properties of clays and their behaviour is influenced by presence of certain clay minerals even in small quantities. The thickness of the oriented water around a clay particle is dependent on type of clay mineral. Thus, the plasticity of a clay depends upon (i) the nature of clay mineral present, (ii) amount of clay mineral present. On the basis of lab tests, Skempton (1953) observed that for a given soil the plasticity index is directly proportional to the percent of clay-size fraction (i.e. percent by weight finer than 0.002 mm in size). He introduced the concept of activity, by relating the plasticity to the quantity of clay-size particles, and defined the activity A_c as the ratio of plasticity index to the percent by weight of soil particles of diameter smaller than two microns present in the soil. Thus

$$A_c = I_p / C_w$$

Where, I_p = plasticity index

C_w = percentage, by weight of clay-sizes, i.e. of particles of size less than 2 microns

Activity can be determined from the results of usual laboratory tests such as wet analysis, liquid limit and plastic limit.

1.15 SENSITIVITY OF CLAYS:

The consistency of an undisturbed sample of clay is altered, even at the same water content, if it is remoulded. It is because the original structure of clay is altered by remoulding. Since the strength of clay soil is related to its structure, remoulding results in decrease of it strength. The degree of disturbance of undisturbed clay sample due to remoulding is expressed by sensitivity (S_t) which is defined as "the ratio of its unconfined compression strength in the natural or undisturbed state to that in the remoulded state, without change in the water content".

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

The sensitivity of most clays generally falls in a range of 1 to 8.

1.16 THIXOTROPY OF CLAYS:

When sensitive clays are used in construction, they loose strength due to remoulding during construction operations. However, with passage of time, the strength again increases, though not to the same original level. This phenomenon of "strength loss-strength gain" with no change in volume or water content is called thixotropy (from the Greek thixis, meaning 'touch' and tropein, meaning 'to change'). Thus thixotropy is defined as "an isothermal, reversible, time dependent process which occurs under constant composition and volume, thereby a material softens, as a result of remoulding, and then gradually returns to its original strength when allowed to rest".

The loss of strength due to remoulding is partly due to (i) permanent destruction of the structure due to in-situ layers, (ii) reorientation of the molecules in the adsorbed layers. The gain in strength is due to rehabilitation of the molecular structure of the soil, and is due to its thixotropic property.

1.17 UNIFIED SOIL CLASSIFICATION SYSTEM (USCS):

Originally developed by Casagrande (1940), the Unified Soil Classification System (USCS) was used for air field construction during World War II. It was later (1952) modified slightly by the Bureau of Reclamation and Corps of Engineers of USA, to make it applicable to other construction like foundations, earth dams, earth canals, earth slopes etc. The system has also been adopted by American Society of Testing Materials (ASTM) and later by Bureau of Indian Standards (1970).

- According to USCS, the coarse grained soils are classified on the basis of their grain size distribution.
- While the fine grained soils, whose behaviour are controlled by plasticity, are classified on the basis of their plasticity.
- Various soils are classified into four major groups: (i) Coarse grained, (ii) Fine grained, (iii) Organic soils, (iv) Peat.

The soils are first classified into two categories:

1.17.1 Coarse grained soils   1.17.2 Fine grained soils

1.17.1 Coarse Grained Soils: if more than 50% of the soil is retained on No. 200 US sieve (0.075 mm), it is designated as coarse grained soil. A coarse grained soil is designated as gravel (G). If 50% or more of the coarse fraction (plus 0.075 mm) is retained on No. 4 (4.75 mm) US sieve; otherwise it is termed as sand (S). Coarse grained soils, containing less than 5% fines, are designated by symbols GW and SW if they are well graded and by symbol GP and SP if they are poorly graded. If however the percentage of fines in more than 12%, the coarse grained soils are designated by symbols GM, GC, SM or SC. If the percentage of fines lies between 5 to 12%, coarse grained soils are designated by dual symbols GW-GM or SP-SM.

1.17.2 Fine Grained Soils: a soil is termed as fine grained if more than 50% of the soil sample passes No. 200 US sieve. Fine grained soil are subdivide into silt (M) and clay (C), based on their liquid limit and plasticity index. Organic soils are also included in this group. Fig. 4.3 shows the plasticity chart devised by Casagrande (1948), and used for the USCS system. The A – line in the chart has the equation $I_p = 0.73\ (W_L - 20)$. This A – line generally separates the more clay like materials from those that are silty and also the organic soils from inorganic soils.

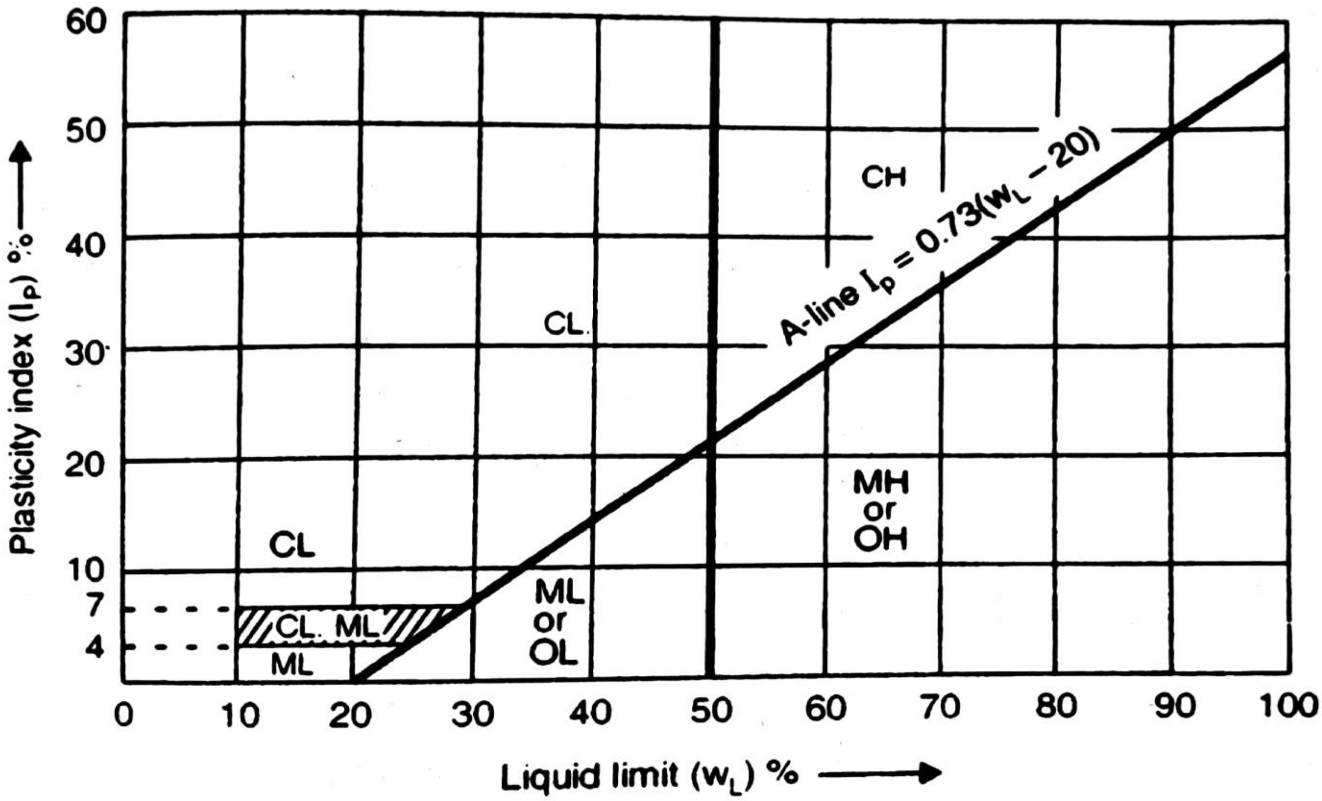


FIG. 4.3 CASAGRANDE'S PLASTICITY CHART (USCS)

1.18 INDIAN STANDARD CLASSIFICATION SYSTEM, ISCS (IS: 1498 – 1970):

The Indian Standard Soil Classification System (ISCS), first developed in 1959, was revised in 1970. This revised version is essentially based on USCS with the modification. Soils are broadly divided into three divisions:

1.18.1 Coarse grained soil

1.18.2 Fine grained soil

1.18.3 Highly organic soils and other miscellaneous soil materials

1.18.1 Coarse Grained Soil: are further divided into two sub-divisions:

1.18.1.1 Gravel (G): in these soils, more than half the coarse fraction (+75 micron) is larger than 4.75 mm IS sieve size.

1.18.1.2 Sands (S): in these soils, more than half the coarse fraction (+75 micron) is smaller than 4.75 mm IS sieve size.

1.18.2 Fine Grained Soil: are further divided into three sub-divisions:

1.18.2.1 Inorganic silts and very fine sands (M)

1.18.2.2 Inorganic clays (C)

1.18.2.3 Organic silts and clays and organic matter (O)

The fine grained soils are further divided into the following groups on the basis of the following arbitrarily selected values of liquid limit which is good index of compressibility:

1.18.2.4 Silts and clays of low compressibility: having a liquid limit less than 35, and represented by symbol L.

1.18.2.5 Silts and clays of medium compressibility: having liquid limit greater than 35 and less than 50, and represented by symbol I.

1.18.2.6 Silts and clays of high compressibility: having liquid limit greater than 50, and represented by a symbol H.

Laboratory classification of fine grained soil is done with the help of plasticity chart shown in Fig. 4.4. The A – line, dividing inorganic clay form silt and organic soil has the following equation:

$$I_p = 0.73\ (W_L - 20)$$

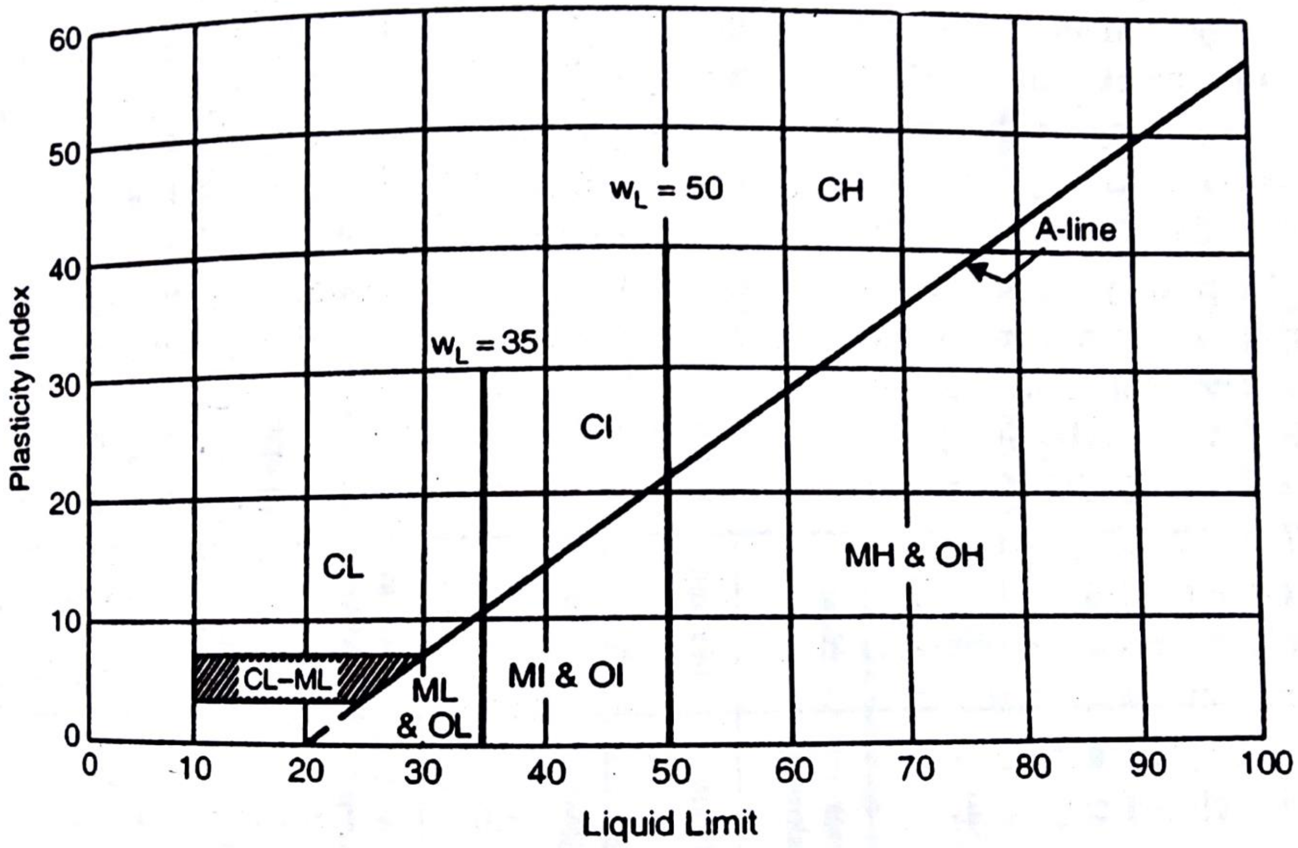


FIG. 4.4. PLASTICITY CHART (IS SOIL CLASSIFICATION SYSTEM).

MODULE – 2

SOIL STRUCTURE AND CLAY MINERALOGY

2.1 SOIL STRUCTURE:

Soil structure is usually defined as the arrangement and state of aggregation of soil particles in a soil mass. This term includes, in large sense, consideration of the mineralogical composition, electrical properties, shape and orientation of solid particles; the nature and properties of soil water and its ionic composition; and the interaction forces between soil particles, soil water, and their adsorption complexes. Soil structure is an important factor which influences many soil properties, such as permeability, compressibility and shear strength etc. The following types of soil structure are generally recognised:

2.1.1 Single Grained: an arrangement composed of individual soil particles.\

2.1.2 Honeycomb: an arrangement of soil particles having a comparatively loose, stable structure resembling a honeycomb.

2.1.3 Flocculent: an arrangement composed of 'flocs' of soil particles instead of individual soil particles. The particles are oriented 'edge-to-edge' or 'edge-to-face' with respect to one another.

2.1.4 Dispersed: an arrangement composed of particles having a 'face-to-face' or parallel orientation.

2.1.5 Coarse Grained Skeleton: an arrangement of coarse grains forming a skeleton with its interstices partly filled by a relatively loose aggregation of the finest soil grains.

2.1.6 Cohesive Matrix: an arrangement in which a particle-to-particle contact of coarse fraction is not possible.

The single-grained structure is characteristic of coarse grained soils. The honeycomb, flocculent and dispersed structures are found in fine grained soils. The skeleton and matrix structures represent composite soils.

2.2 SINGLE GRAINED STRUCTURE:

- Coarse grained soils (diameter > 0.02 mm) settle out of suspension in water as individual grains independently of other grains.
- The major force causing their deposition is gravitational and the surface forces are too small to be of practical importance.
- The weight of the grains causes them to settle, and get particle-to-particle contact on deposition.

- They may be deposited in a loose state having high voids ratio or in a dense state having a low voids ratio.

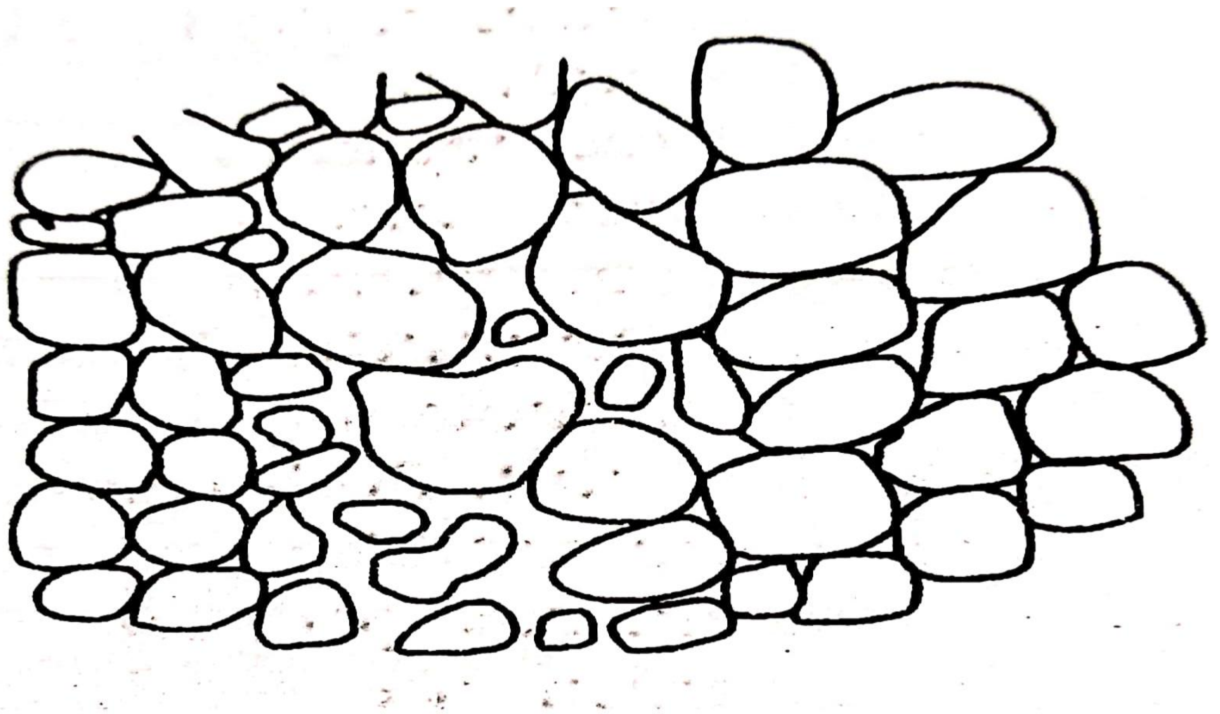


FIG. 5.10. SINGLE GRAINED STRUCTURE.

2.3 HONEYCOMB STRUCTURE:

- Such a structure exists in grains of silts or rock flour smaller than 0.02 mm diameter and larger than 0.0002 mm.
- When such grains settle under gravity, the surface forces also play an equally important role.
- The surface forces at the contact areas, as the grains come in contact at the bottom of suspension, are large enough compared to the submerged weight to prevent the grains from rolling down immediately into position of equilibrium among the grains already deposited.
- The grains coming in contact are held until miniature arches are formed, bridging over relatively large voids spacing and forming a honeycomb structure (Fig. 5.11).
- Each cell in the honeycomb structure is supposed to be made up of numerous single mineral grains.
- Comparatively large amount of water is enclosed within voids built up of aggregates of minerals glued to each other by the adhesion forces.

- The structure so formed has high voids ratio and is capable of carrying relatively heavy loads without volume change.

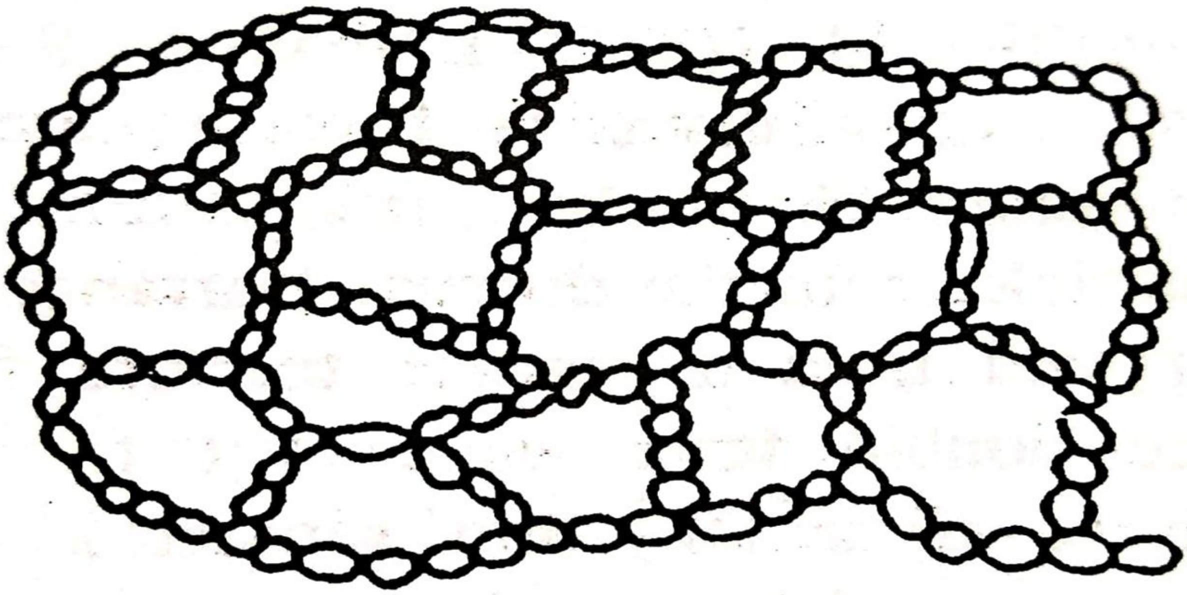


FIG. 5.11. HONEYCOMB STRUCTURE.

2.4 FLOCCULENT AND DISPERSED STRUCTURE:

- According to Lambe (1953) a flocculated structure of clay platelets is formed when there are edge-to-edge contacts between the platelets (Fig. 5.12 (a)).
- Such a structure is formed if the net electrical forces between adjacent soil particles at the time of deposition are attraction forces.
- If there is a concentration of dissolved minerals in the water the tendency of flocculation is increased.
- On the other hand, a dispersed or oriented structure is formed when the platelets have face-to-face contact in more or less parallel array (Fig. 5.12 (b)).
- Such a structure is formed if the net electrical forces between adjacent soil particles at the time of deposition are repulsion.
- Remoulding and compacting clays tends to produce a dispersed structures.

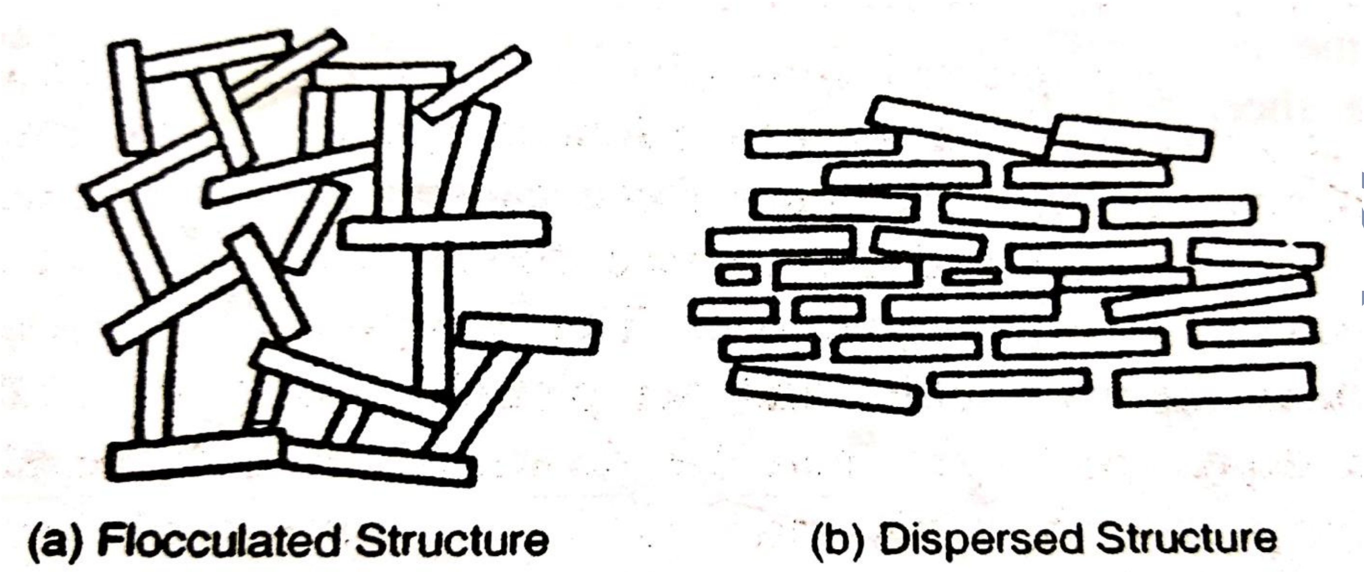


FIG. 5.12. GOLDSCHMIDT-LAMBE CONCEPT OF CARDHOUSE STRUCTURE.

2.5 ATOMS AND ATOMIC BONDS:

Molecules of minerals are composed of atoms of chemical elements and these are the basic building blocks of all matter. The atoms of an element are formed of basic particles i.e. protons, neutrons and electrons. An atom consists of a nucleus with one or more protons, each carrying a positive electromagnetic charge and one or more neutrons carrying no charge.

2.5.1 Valence: valence of an element is the number of electrons that is in excess or deficient at a shell level. If the electrons are in excess it is termed as positive valence. If the electrons are not in excess i.e. deficient it is termed as negative valence.

2.5.2 Atomic Bonds: there are two main groups of atomic bonds i.e.

2.5.2.1 Primary valence bonds 2.5.2.2 Secondary valence bonds

2.5.2.1 Primary Valence Bonds:

- Primary valence bond is a chemical combination of two or more elements because of the lack of a complete complement of electrons in their outermost shells.
- One atom joins with another atom by adding electrons to its outer shell or by losing them to arrive at a stable compound.
- The number of electrons an atom gains or loses depends upon the valence of the element.

- Atoms which lose or gain electrons in this manner are called ions and the forces binding them together are called ionic bonds.
- The ionic bonds are the strongest of the bonds that hold.

2.5.2.2 Secondary Valence Bonds: atoms in one molecule bonding to atoms in another molecule i.e. intermolecular bond are called secondary bonds. These bonds are of two types:

2.5.2.2.1 Vander waals forces 2.5.2.2.2 Hydrogen bond

2.5.2.2.1 Vander Waals Forces: are the attractive forces between the surface of two parallel clay mineral particles separated by water. These forces depend essentially on the crystal structure of the minerals and on the distance separating the particles. This force increases with the decrease in the distance between the surfaces of the minerals.

2.5.2.2.2 Hydrogen Bond: the positively charged hydrogen is capable of attracting an oxygen atom of another molecule, thus forming a hydrogen bond. Hydrogen bond is weaker than the primary valence bond, but much stronger than the vander waals forces and is not easily broken under the stresses normally applied on a soil system.

2.6 ELECTRICAL DIFFUSE DOUBLE LAYER:

- The soil colloid suspended in water carries nearly always a net negative charge.
- The acidic nature of soil minerals suggests that hydrogen atoms come off in the presence of water and thereby give the minerals a net negative charge.
- The net electrical charge of the entire soil water suspension must be zero.
- The charge on each colloid must be neutralized by ions from the water which swarm around each colloid.
- These ions are called counter ions or exchangeable ions, since they can be replaced.
- If there were no thermal activity possessed by these ions, and if there were no attraction exerted on them by other ions and colloids, these counter ions would all swarm to the surface of the particles to neutralize the surface charge of the particle.
- The counter ions thus constitute the diffuse double layer of the colloid.

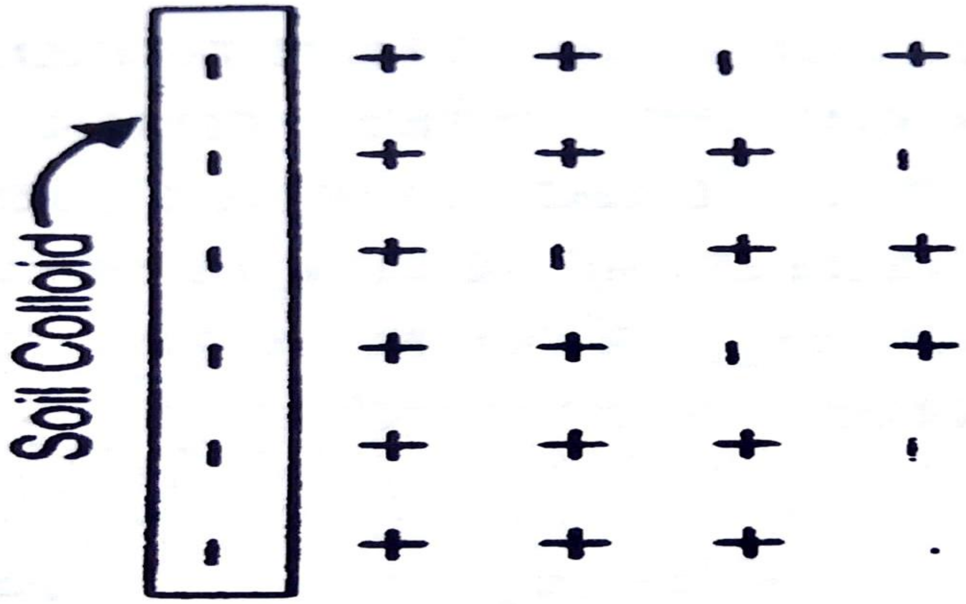
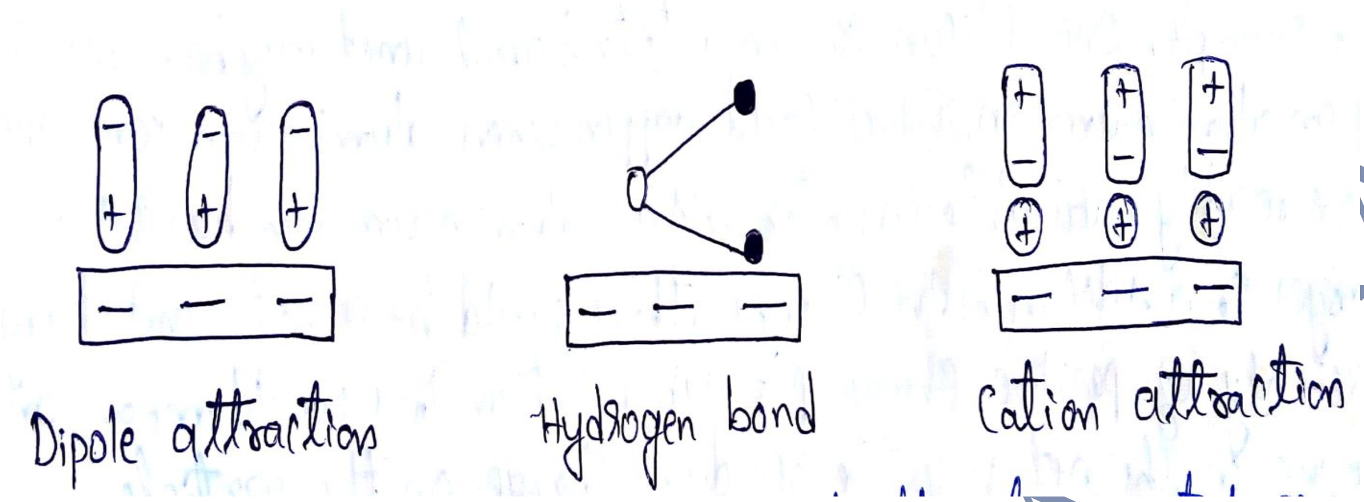


FIG. 5.6. DIFFUSE DOUBLE LAYER

2.7 ADSORBED WATER:

- The layers of water which is surrounding the clay crystal are called adsorbed water.
- First, water molecule being dipole, is electrostatically attracted to the negatively charged surface of the crystal.
- Secondly, water is adsorbed by hydrogen bonding i.e. hydrogen atom of water molecule is attracted to the oxygen on the surface of the clay crystal.
- The third reason is that the cations present in water get attracted to the negatively charged surface of the clay particles and water dipoles are in turn, attracted to the cations.
- It seems natural that the attraction of water to the particle surface is very strong near the surface and tappers off with distance.



2.8 BASE EXCHANGE CAPACITY OR CATION EXCHANGE CAPACITY:

The process of replacing cations of one kind by those of another in an adsorption complex is known as base exchange. By base exchange is meant the capacity of colloidal particle to change the cations adsorbed on their surface. Thus hydrogen clay can be changed to sodium clay by a constant percolation of water containing dissolved Na salts. Such changes can be used to decrease the permeability of a soil. The quantity of exchangeable cations in a soil is termed as exchange capacity.

2.9 ISOMORPHOUS SUBSTITUTION:

In a clay mineral, metallic ions of one kind may be substituted by other metallic ions of a lower valence, but of the same physical size, such a substitution is called isomorphous substitution.

Example: one silicon ion in a tetrahedral unit may be substituted by an aluminium ion, which could happen when aluminium ions are more readily available in water. As an aluminium ion has 3 positive charges and a silicon 4 positive charge, there would be a net unit charge deficiency of positive charge per substitution. This would mean an increase in the net negative residual charge on the particle.

2.10 STRUCTURE OF CLAY MINERALS:

Two fundamental building blocks are involved in the formation of clay mineral structure. They are

2.10.1 Tetrahedral unit   2.10.2 Octahedral unit

2.10.1 Tetrahedral Unit: the tetrahedral unit consists of a four oxygen atoms placed at the tips of a tetrahedron enclosing a silicon atom. The oxygen at the bases of all the units lie in a common plane. the oxygen atoms are negatively charged with two negative charges each and the silicon with four positive charges. Each of the three oxygen ions at the base shares its charges with the adjacent tetrahedral unit.

2.10.2 Octahedral Unit: the octahedral unit is a combination of six hydroxyl ions at tips of an octahedral enclosing an aluminium ion at the centre. Iron or magnesium ions may replace aluminium ions in some unit. These octahedral units are bound together in a sheet structure with each hydroxyl ion common to three octahedral units. This sheet is sometimes termed as gibbsite sheet.

2.11 CLAY MINERALS:

2.11.1 Kaolinite 2.11.2 Montmorillonite 2.11.3 Illite

2.11.1 Kaolinite:

- Kaolinite is the most common mineral of the kaolin group.
- The kaolinite structural unit is made up of gibbsite sheets joined to silica sheet through the hydrogen bond.
- The thickness of each layer is about 7A°.
- Since the hydrogen bond is fairly strong, it is extremely difficult to separate the layers, and as a result kaolinite is relatively stable and water is unable to penetrate between the layers.
- Kaolinite consequently shows relatively little swell on wetting.
- China clay is almost pure kaolinite.

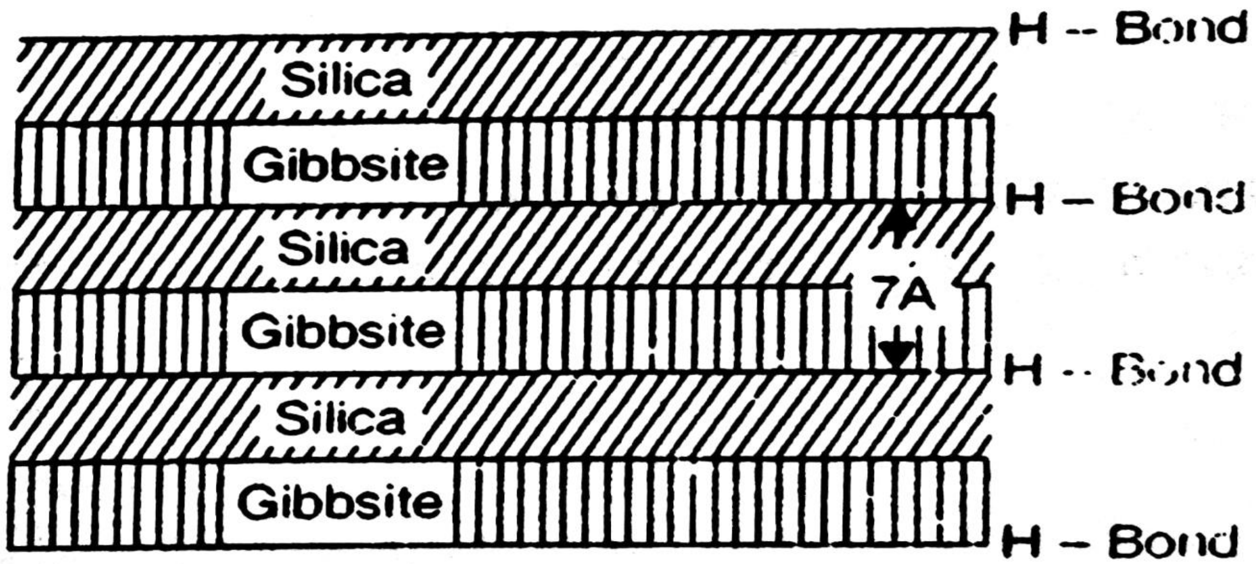


FIG. 5.16 (b). STRUCTURE OF KAOLINITE.

2.11.2 Montmorillonite:

- This is the most common of all the clay minerals in expansive clay soils.
- The mineral is made up of sheet like units.
- The basic structure of each unit is made up of gibbsite sheet (i.e. the octahedral sheet) sandwiched between two silica sheets with water bond.
- The thickness of each unit is about 10 A°.
- There is very weak bonding between the successive sheets and water may enter between the sheets causing the minerals to swell.
- The spacing between the elemental silica-gibbsite-silica sheets depends upon the amount of available water to occupy the space. For this reason, montmorillonite is said to have an expanding lattice.
- Black cotton soil belongs to this group.

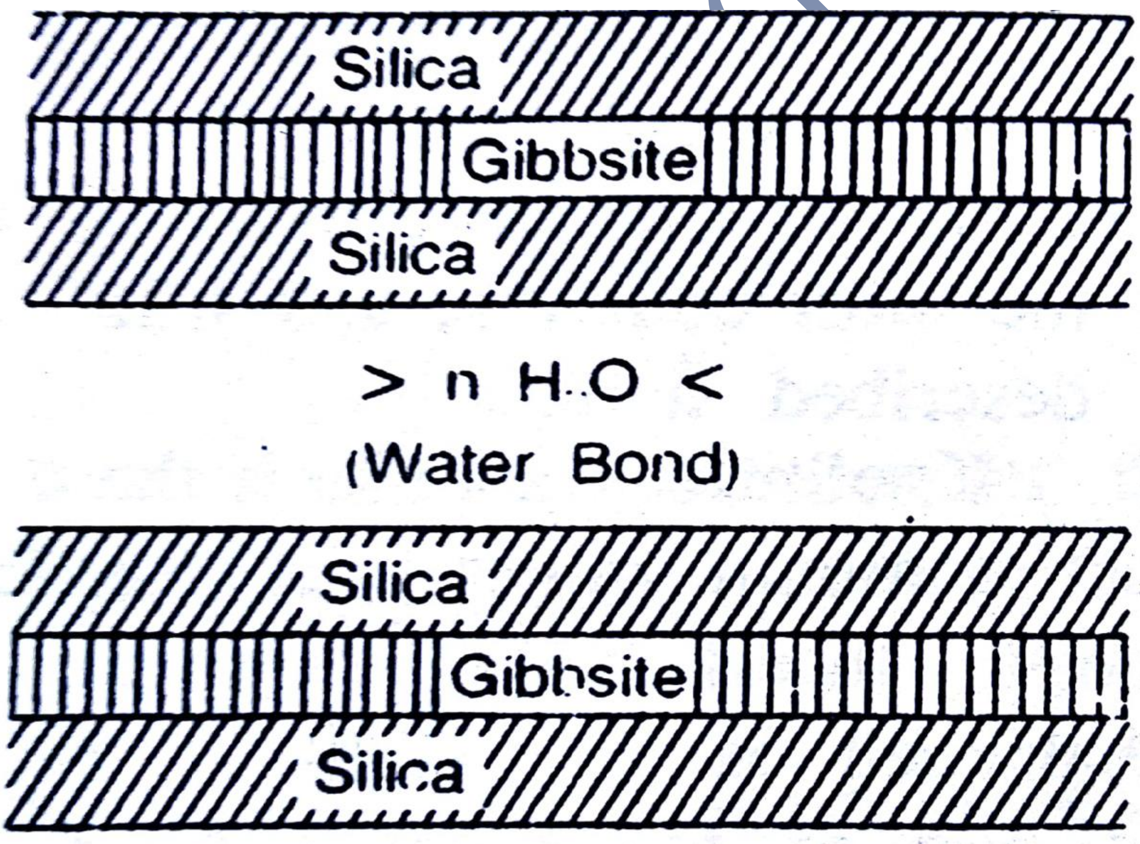


FIG. 5.17 (*b*). STRUCTURE OF MONTMORILLONITE.

2.11.3 Illite:

- The structure of illite is similar to that of montmorillonite except that some of the silicons are always replaced by aluminium atoms and the resultant charge deficiency is balanced by potassium ions.
- The potassium ions occur between unit layers.
- The cation bond of illite is weaker than the hydrogen bond of kaolinite, but stronger than the water bond of montmorillonite.
- Due this, the illite crystal has a greater tendency to split into ultimate platelets consisting of gibbsite layer between two silica layers, than that in kaolinite.

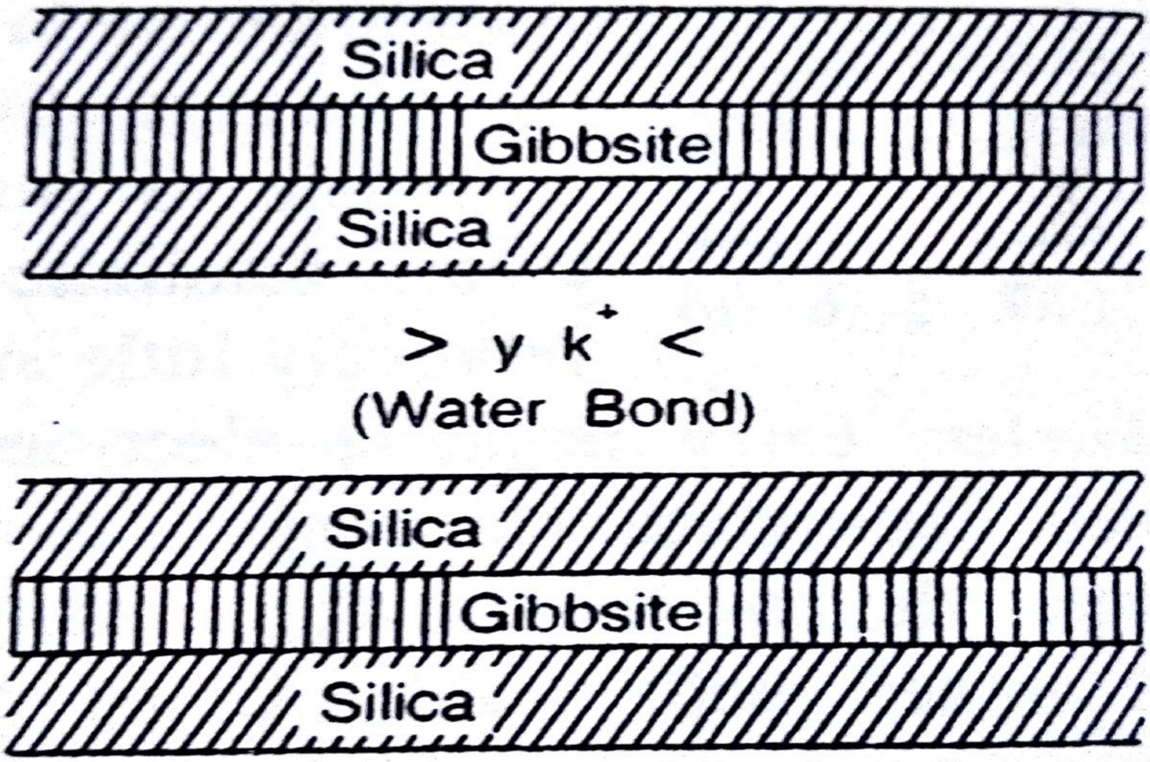


FIG. 5.18(*b*). STRUCTURE OF ILLITE.

MODULE – 2

COMPACTION OF SOILS

2.12 INTRODUCTION:

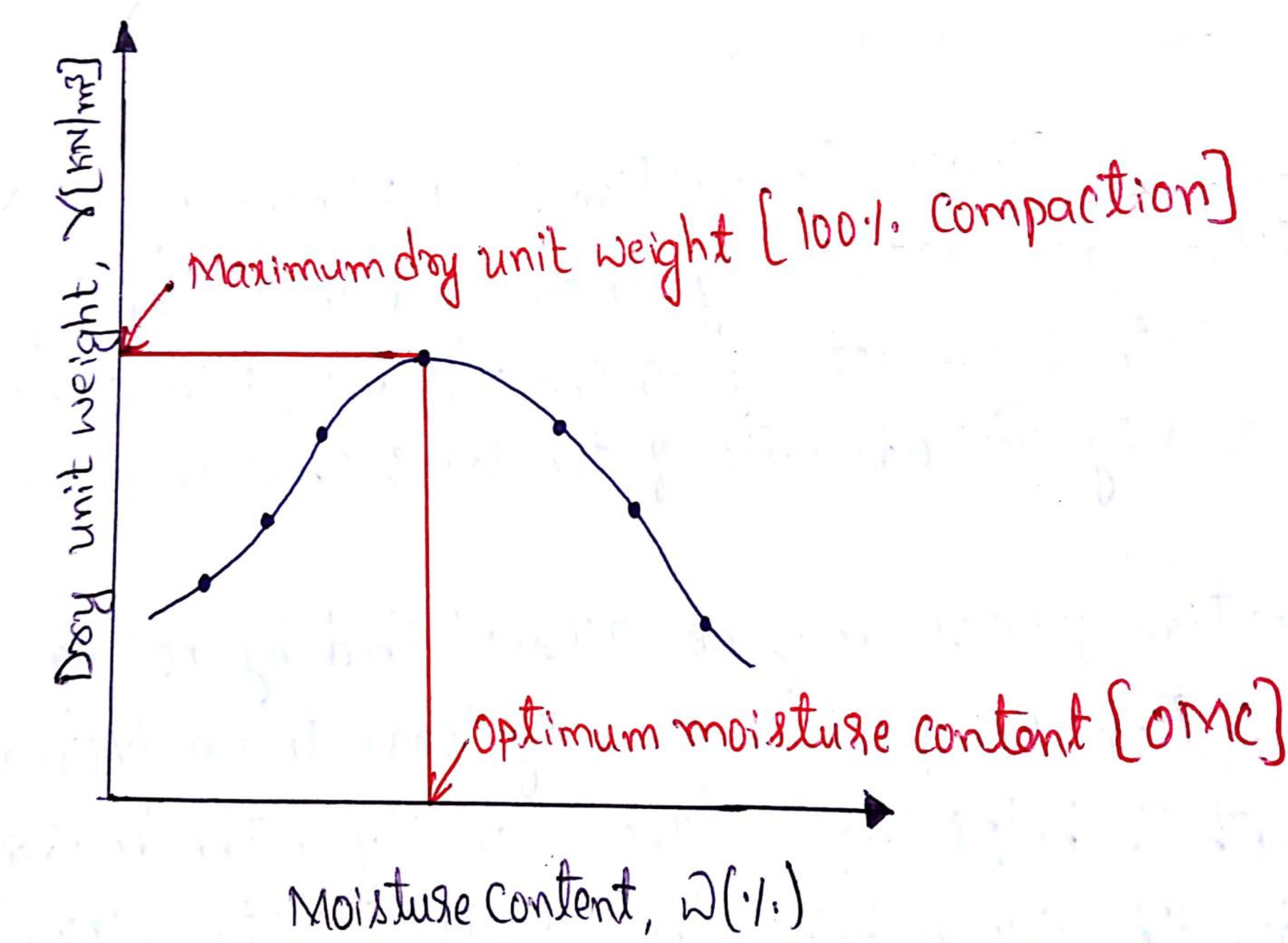
Compaction is a process by which the soil particles are artificially rearranged and packed together into a closer state of contact by mechanical means in order to decrease the porosity (or voids ratio) of soil and thus increase its dry density. The compaction process may be accomplished by rolling, tamping or vibration. Compaction is somewhat different from consolidation. While consolidation is a gradual process of volume reduction under sustained loading, compaction refers to a more or less rapid reduction mainly in the air voids under a loading of short duration.

2.13 EFFECTS OF COMPACTION:

- Increases the dry density of the soil.
- Increases the shear strength of the soil.
- Increases the bearing capacity of soil.
- Brings about a low permeability of the soil.

2.14 PRINCIPLE OF COMPACTION:

- Compaction in general, is the densification of soil by removal of air, which requires mechanical energy.
- The degree of compaction of a soil is measured in terms of its dry unit weight.
- When water is added to the soil during compaction, it acts as a softening agent on the soil particles.
- The dry unit weight after compacting increases as the water content increases.
- To certain limit the dry unit weight decreases, any increase in the water content.
- The water content at which the maximum dry unit weight is attained is generally referred as the "optimum moisture content".
- Standard proctor test and modified proctor test are mainly used as compaction process for any type of soil.



2.15 AIR VOID AND ZERO AIR VOID LINE:

A line which shows the water content dry density relation for the compacted soil containing a constant percentage of air voids is known as an air-voids line.

A line which shows the water content dry density relation for the compacted soil containing no air voids is known as a zero air-voids line.

Zero air-void line can be determined by using a formula:

$$\gamma_d = G\gamma_w / (1 + (wG / S))$$

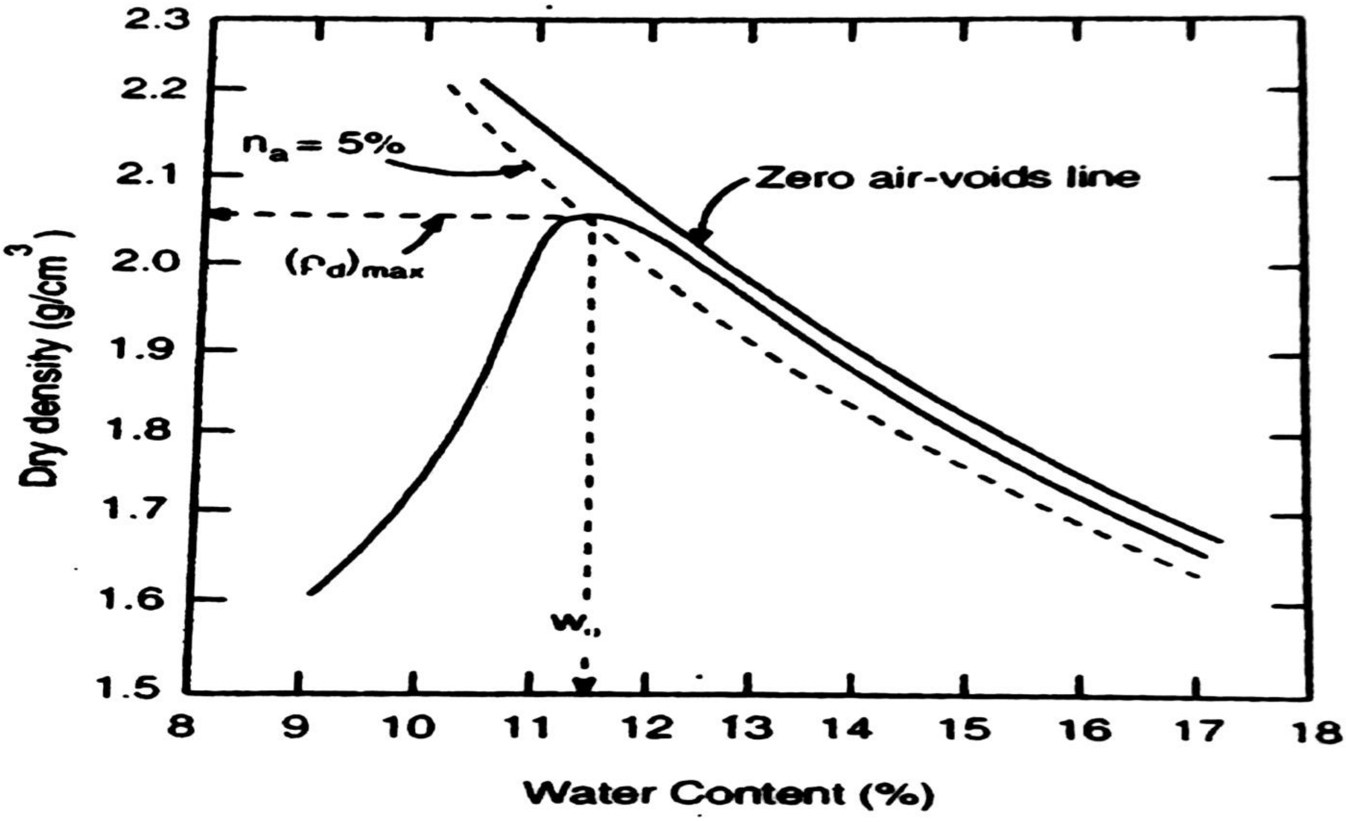


FIG. 17.2. WATER CONTENT DENSITY RELATIONSHIP.

2.16 FACTORS AFFECTING COMPACTION:

The various factors which affect the compacted density are as follows:

2.16.1 Water content

2.16.2 Amount of compaction

2.16.3 Method of compaction

2.16.4 Type of soil

2.16.5 Addition of admixtures

2.16.1 Water Content: it has been seen by laboratory experiments that as the water content are increased, the compacted density goes on increasing, and till a maximum dry density is achieved after which further addition of water decreases the density.

2.16.2 Amount of Compaction: the amount of compaction greatly affects the maximum dry density and optimum water content of a given soil. The effect of increasing the compactive energy results in an increase in the maximum dry density and decreases in the optimum water content. However, the increase in maximum dry density does not have a linear relationship with increase of compactive effort.

2.16.3 Method of Compaction: the density obtained during compaction, for a given soil, greatly depends upon the type of compaction or the manner in which the compactive effort is applied. The various variables in this aspect are (i) weight of the compacting equipment, (ii) the manner of operation such as dynamic or impact or rolling or static, (iii) time and area of contact between the compacting element and the soil.

2.16.4 Type of Soil: the maximum dry density achieved corresponding to a given compactive energy largely depends upon the type of soil. Well graded coarse grained soils attain a much higher density and lower optimum water contents then fine grained soils which require more water for lubrication because of the greater specific surface.

2.16.5 Addition of Admixtures: the compaction properties of a soil can be modified by a number of admixtures other than soil material. These admixtures have special application in stabilised soil construction.

2.17 EFFECT OF COMPACTION ON SOIL PROPERTIES:

The main aim of compacting a soil is to improve some desirable properties of the soil, such as reduction of compressibility, water absorption and permeability, increase in soil strength, bearing capacity etc. and change in swelling and shrinkage characteristics.

2.17.1 Soil structure

2.17.2 Permeability

2.17.3 Shrinkage

2.17.4 Swelling

2.17.5 Pore pressure

2.17.6 Compressibility

2.17.7 Stress strain characteristics

2.17.8 Shear strength

2.17.1 Soil Structure: when a soil is compacted at low effort with less water content i.e. less than optimum moisture content it exhibits a flocculated structure as shown in point A and also exhibits the same structure in high compactive effort as shown in point C. But when soil is compacted with higher percentage of water i.e. more than optimum moisture content it exhibits a dispersed structure for both low and heavy compactive effort which is shown in point D and B, but D point tends a more dispersed structure when compared to point B of heavy compactive effort.

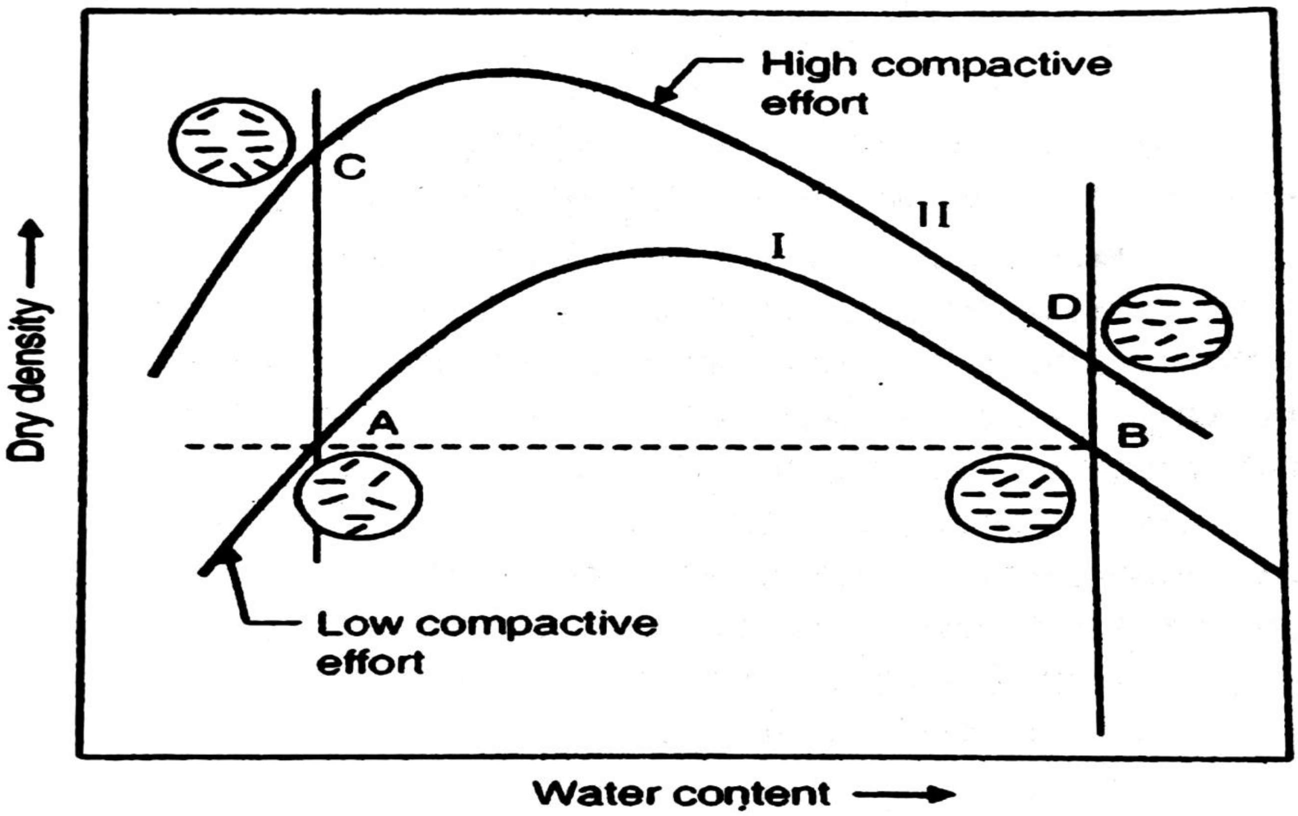


FIG. 17.11. EFFECTS OF COMPACTION ON STRUCTURE OF CLAY (AFTER LAMBE, 1958)

2.17.2 Permeability: permeability of soil mainly depends on the size of voids. When soil is compacted dry it is said to be more permeable then those soil compacted wet i.e. more than the optimum moisture content it said to be less permeable in nature. Thus, dry density increases but permeability goes on decreases.

2.17.3 Shrinkage: soil sample compacted dry of optimum shrink less than the sample compacted wet of optimum. This so because the soil particles having dispersed structure have nearly parallel orientation and can pack more efficiently.

2.17.4 Swelling: a soil sample compacted dry of optimum water content has a high water deficiency and exert greater swelling pressure and also swells more on higher water content from wet side compaction.

2.17.5 Pore Pressure: soil is compacted dry optimum, tends to develop low pore pressure than that of soil compacted wet of optimum will have higher pore pressure.

2.17.6 Compressibility: soil compacted wet of optimum is more compressible than the soil compacted dry of optimum. However, in the high pressure range, a sample compacted dry of optimum is more compressible than the one compacted wet of the optimum.

2.17.7 Stress Strain Characteristics: for a given soil, a sample compacted dry side of optimum has a steeper stress-strain curve and hence has a higher modulus of elasticity, than the one which is compacted wet of optimum, at the same density.

2.17.8 Shear Strength: the shear strength of compacted clays depends upon (i) dry density, (ii) moulding water content, (iii) soil structure, (iv) method of compaction, (v) strain used to define strength, (vi) drainage condition, (vii) type of soil.

2.18 STANDARD PROCTOR TEST:

The standard proctor test was developed by R R Proctor (1933) for the construction of earth fill dams in the state of California. The test equipment consists of (i) cylindrical metal mould, having an internal diameter of 10.15 cm, an internal effective height of 11.7 cm and a capacity of 0.945 litre, (ii) detachable base plate, (iii) collar 5 cm in effective height, and (iv) rammer 2.5 kg in mass falling through a height of 30.5 cm.

Procedure:

- About 3 kg of air-dried and pulverised soil, passing a 4.75 mm sieve, is mixed thoroughly with a small quantity of water.
- The mixture is covered with wet cloth, and left for a maturing time of about 5 to 30 minutes to permit proper absorption of water.
- The quantity of water to be added for the first test depends upon the probable optimum water content of soil.
- The initial water content may be taken 4% for coarse grained soils and 10% for the fine grained soils.
- The empty mould attached to the base plate is weighed without collar. The collar is then attached to the mould.
- The mixed and mature soil is placed in the mould and compacted by giving 25 blows of the rammer uniformly distributed over the surface, such that the compacted height of soil is about 1/3 the height of the mould.
- Before putting the second instalment of soil, the top of the first compacted layer is scratched with the help of any sharp edge.
- The second and the third layer are similarly compacted, each layer being given 25 blows.
- The last compacted layer should project not more than 6 mm into the collar. The collar is removed and the excess soil is trimmed off to make it level with the top of mould.

- The weight of the mould, base plate and the compacted soil is taken. A representative sample is taken from the centre of the compacted specimen and kept for water content determination.
- The bulk density and the corresponding dry density for the compacted soil are calculated.
- The compacted soil is taken out of the mould, broken with hand and remixed with raised water content.
- After allowing for the maturing time, the soil is compacted in the mould in three equal layers, as described above, and the corresponding dry density and water content are thus determined.
- The test is repeated on soil samples with increasing water contents, and the corresponding dry density obtained is thus determined.
- A compaction curve is plotted between the water contents as abscissae and the corresponding dry densities as ordinates.
- The dry density goes on increasing as the water content is increased, till maximum density is reached. The water content corresponding to the maximum density is called the optimum water content.

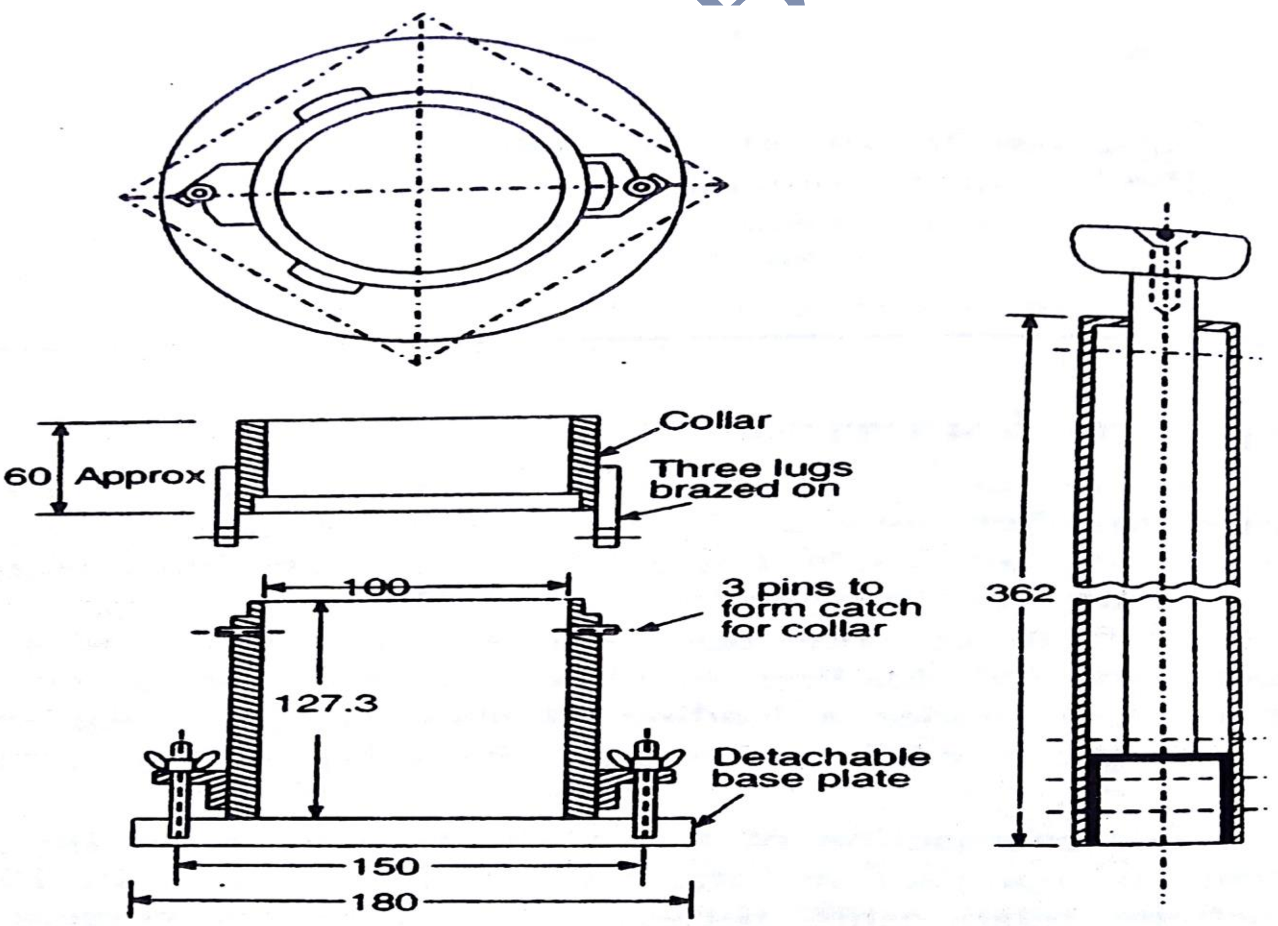


FIG. 17.1. EQUIVALENT FOR STANDARD PROCTOR TEST.
[IS : 2720 a (PART VII) : 1965 : LIGHT COMPACTION]

2.19 MODIFIED PROCTOR TEST:

The modified proctor test was developed by R R Proctor (1933) for the construction of earth fill dams in the state of California. The test equipment consists of (i) cylindrical metal mould, having an internal diameter of 10.15 cm, an internal effective height of 11.7 cm and a capacity of 0.945 litre, (ii) detachable base plate, (iii) collar 5 cm in effective height, and (iv) rammer 4.9 kg in mass falling through a height of 30.5 cm.

Procedure:

- About 3 kg of air-dried and pulverised soil, passing a 4.75 mm sieve, is mixed thoroughly with a small quantity of water.
- The mixture is covered with wet cloth, and left for a maturing time of about 5 to 30 minutes to permit proper absorption of water.
- The quantity of water to be added for the first test depends upon the probable optimum water content of soil.
- The initial water content may be taken 4% for coarse grained soils and 10% for the fine grained soils.
- The empty mould attached to the base plate is weighed without collar. The collar is then attached to the mould.
- The mixed and mature soil is placed in the mould and compacted by giving 45 blows of the rammer uniformly distributed over the surface, such that the compacted height of soil is about 1/3 the height of the mould.
- Before putting the second instalment of soil, the top of the first compacted layer is scratched with the help of any sharp edge.
- The second, third, fourth and fifth layer are similarly compacted, each layer being given 45 blows.
- The last compacted layer should project not more than 6 mm into the collar. The collar is removed and the excess soil is trimmed off to make it level with the top of mould.
- The weight of the mould, base plate and the compacted soil is taken. A representative sample is taken from the centre of the compacted specimen and kept for water content determination.
- The bulk density and the corresponding dry density for the compacted soil are calculated.
- The compacted soil is taken out of the mould, broken with hand and remixed with raised water content.
- After allowing for the maturing time, the soil is compacted in the mould in three equal layers, as described above, and the corresponding dry density and water content are thus determined.

- The test is repeated on soil samples with increasing water contents, and the corresponding dry density obtained is thus determined.
- A compaction curve is plotted between the water contents as abscissae and the corresponding dry densities as ordinates.
- The dry density goes on increasing as the water content is increased, till maximum density is reached. The water content corresponding to the maximum density is called the optimum water content.

2.20 FIELD COMPACTION METHODS:

The equipment that are normally used for compaction consists of

2.20.1 Smooth wheel rollers   2.20.3 Sheepsfoot rollers

2.20.2 Pneumatic rubber tired rollers   2.20.4 Vibratory rollers

2.20.1 Smooth Wheel Rollers: are suitable for subgrade and for finishing operation of fills with sandy and clayey soils. There are two types of smooth wheel rollers. One type has two large wheels, one in the rear and in the front. The equipment weighs from 50 to 125 kN. The other type is the tandem roller. This roller has large single drums in the front and rare and weighs from 10 to 200 kN.

2.20.2 Pneumatic Rubber Tired Rollers: are suitable for sandy and clayey soil compaction. The maximum weight of this roller may reach 2000 kN. It is consists of 9 to 11 tires which are closely spaced. The contact pressure under the tires can range from 600 to 700 kN/m^2 and they produce 70 to 80% coverage.

2.20.3 Sheepsfoot Rollers: are drums with a large number of projections. Drum widths ranging from 120 to 180 cm and diameter ranging from 90 to 180 cm. The area of each projection may range from 25 to 85 cm^2. These rollers are most effective in compacting clayey soils. The contact pressure under the projections can range from 1400 to 7000 kN/m^2.

2.20.4 Vibratory Rollers: are extremely efficient in compacting granular soils. Vibrators can be attached to smooth wheel roller, pneumatic rubber tired roller or sheepsfoot rollers to provide vibratory effect to the soil. The weights of vibratory rollers range from 120 to 300 kN.

2.21 FIELD COMPACTION CONTROL:

The field compaction control consists of the determination of:

(i) Water content at which the soil has been compacted.

(ii) The dry density and hence the degree of compaction.

Proctor Needle method is commonly used for the determination of water content and also the dry density in the field.

2.21.1 Proctor Needle Method:

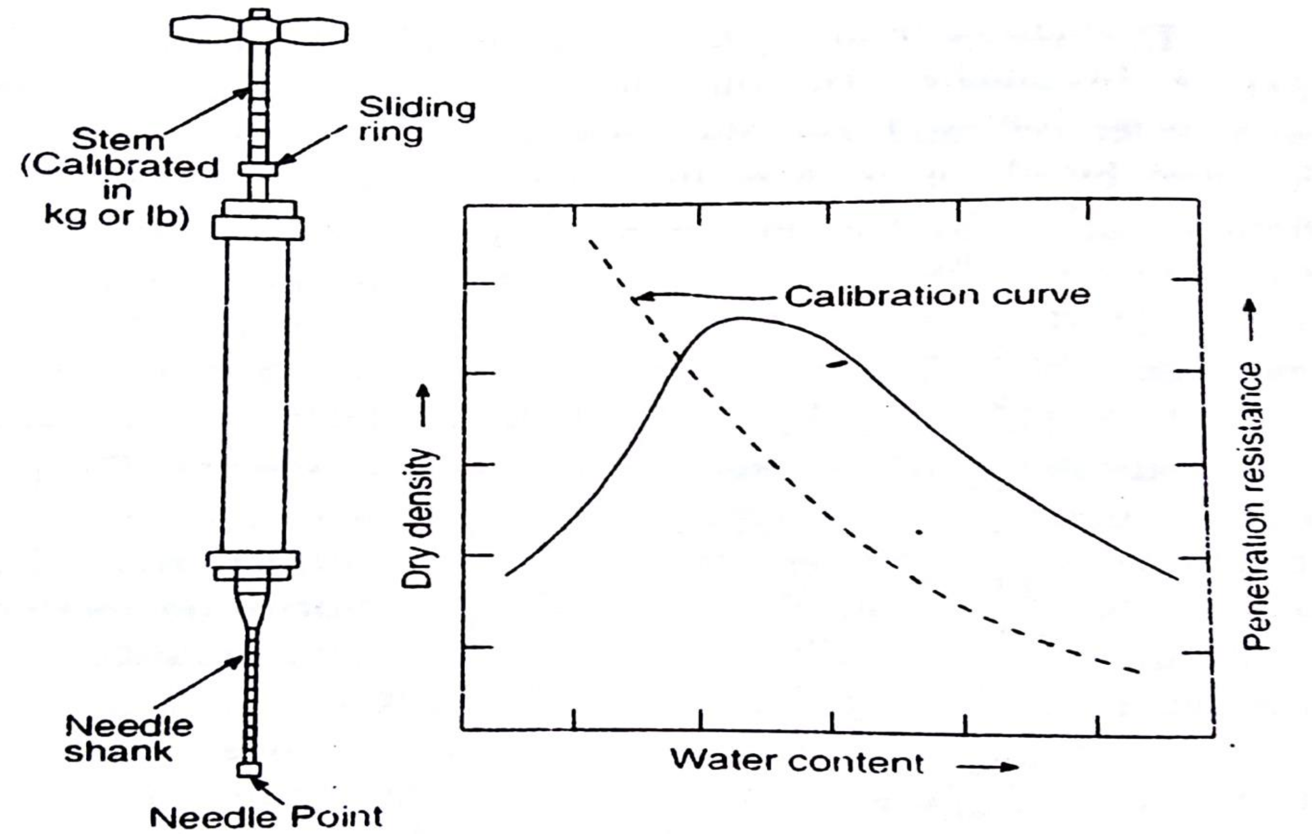


FIG. 17.6. PROCTOR NEEDLE AND CALIBRATION CURVE.

Procedure:

- The proctor needle consists of a needle consists of a needle point, attached to graduated needle shank which in turn is attached to a spring loaded plunger.
- The needle points of varying cross-sectional area are available so that a wide range of penetration resistance can be measured.

- The penetration force is read on a loaded gauge fixed over the handle.
- To use the needle in the field, a calibration curve is plotted in the laboratory between the penetration resistance as the ordinate and the water content as the abscissa.
- The laboratory penetration resistance is measured by inserting the Proctor needle in the compacted soil in the Proctor mould.
- The penetration resistance corresponding to various water contents are thus noted at the end of each Proctor compaction, and a calibration curve is plotted.
- This curve can be used to determine the placement water content.
- The penetration resistance of the compacted soil in the field is determined with the Proctor's needle, and its water content is read off from the calibration curve.

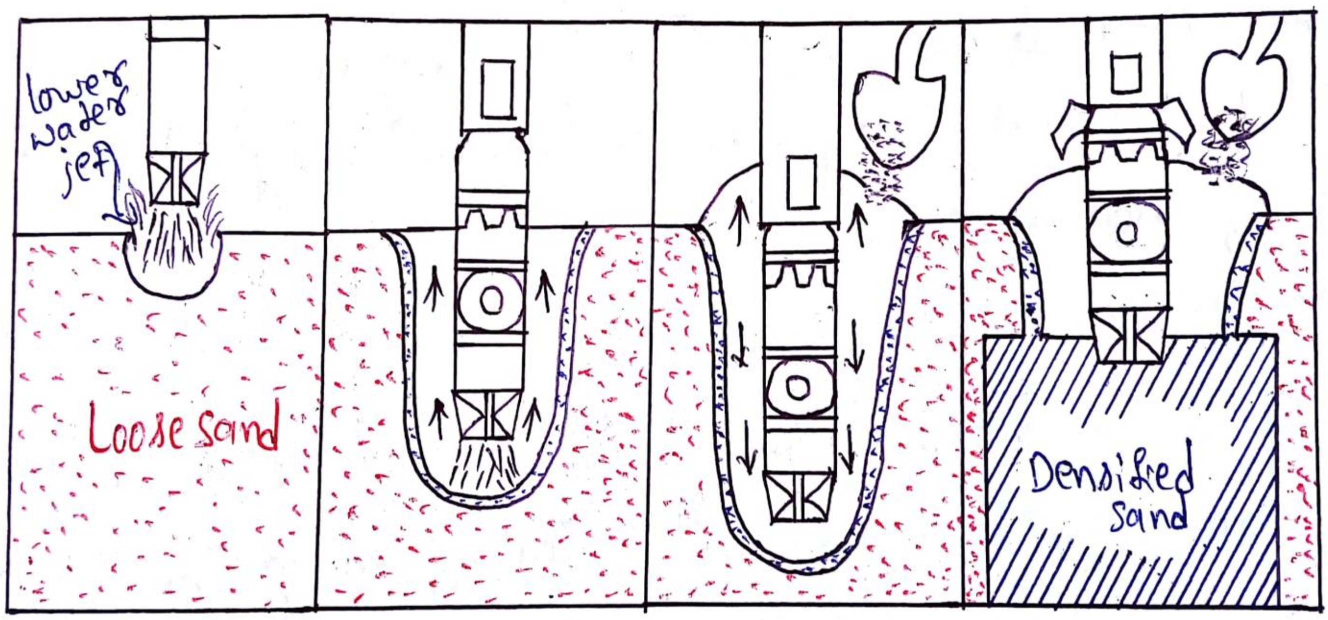
2.22 SPECIAL COMPACTING TECHNIQUES:

2.22.1 Vibroflotation

2.22.2 Dropping of a heavy weight or Dynamic compaction

2.22.3 Blasting

2.22.1 Vibroflotation:



Compaction by vibroflotation process

- The vibroflotation technique is used for compacting granular soils only upto a depth of 30m.
- The vibrofloat is a cylindrical tube containing water jet at top and bottom.
- It also consists of a rotating eccentric weight, which develops a horizontal vibratory motion.
- The vibrofloat is sunk into the soil by using lower water jets, during which the surrounding material is compacted by the vibration process.
- Granular material is poured from the top of the hole.
- The water from the lower jet is transferred to the top jet of the vibrating unit through which the granular materials are settled down the hole.
- This method is very effective for increasing the density of a sand deposit for depths upto 30m.
- Hence a higher density is obtained by this process.

2.22.2 Dynamic Compaction: this process consists of a crane to lift a steel rammer weighing upto 500 kN and upto a height of 40 to 50 cm, and dropped repeatedly on the ground at regular intervals. The process is then repeated either at the same location or over other parts of the area to be compacted. The degree of compaction achieved at a given site depends on the following three factors:

- Weight of rammer.
- Height of rammer drop.
- Spacing of locations at which the rammer is dropped.

2.22.3 Blasting: blasting is a technique that has been used for the densification of granular soils. Here the densification of granular soils is done by blasting or by using a dynamite at a certain depth below the ground surface in saturated soil. The lateral spacing of the charges varies from about 3 to 10 m. Compaction upto relative density of about 80% and upto a depth of about 20 m over a large area can easily be achieved by using this process.

2.23 DIFFERENCE BETWEEN COMPACTION AND CONSOLIDATION:

Compaction	Consolidation
It is an instantaneous phenomenon	It is a time dependent phenomenon
Applies the cohesive as well as cohesionless soils	Applies to cohesive soils
Reduction in the volume of air voids	Reduction in the volume due to pore water
Dynamic loading is commonly applied	Static loading is commonly applied
Relatively quick process	Relatively slow process
Specific compaction techniques	No specified techniques

MODULE – 3

FLOW THROUGH SOILS

3.1 INTRODUCTION:

Soils are permeable due to the presence of interconnected voids through which water can flow from points of high energy to points of low energy. The study of the flow of water through permeable soil media is important in soil mechanics. It is necessary for estimating the quantity of underground seepage under various hydraulic conditions, for investigating problems involving the pumping of water for underground construction, and for making stability analyses of earth dams and earth retaining structures that are subjected to seepage forces.

3.1.1 Permeability: is defined as the property of a porous material which permits the passage or seepage of water (or other fluid) through its interconnecting voids.

3.1.2 Permeable: a material having continuous voids is called permeable. Gravles are highly permeable while stiff clay is the least permeable, and hence such a clay may be termed as impermeable for all practical purposes. The flow of water through soils may either be a laminar flow or turbulent flow.

3.1.3 Laminar Flow: in laminar flow, each fluid particle travels along a definite path which never crosses the path of any other particle.

3.1.4 Turbulent Flow: in turbulent flow, the paths are irregular and twisting, crossing and recrossing at random. In most of the practical flow problems in soil mechanics, the flow is laminar.

The study of seepage of water through soil is important for the following engineering problems:

- Determination of rate of settlement of a saturated compressible soil layer.
- Calculation of seepage through the body of earth dams, and stability of slopes.
- Calculation of uplift pressure under hydraulic structure and their safety against piping.
- Ground water flow towards wells and drainage of soil.

3.2 DRACY'S LAW:

The law of flow of water through soil was first studied by Darcy (1856) who demonstrated experimentally that for laminar flow conditions in a saturated soil, the rate of flow or the discharge per unit time is proportional to the hydraulic gradient.

$$q \propto i$$

$$q = k\, i\, A \quad (3.2.1)$$

Or

$$v = (q / A) = k\, i \quad (3.2.2)$$

Where,

q = discharge per unit time

A = total cross sectional area of soil mass, perpendicular to the direction of flow

i = hydraulic gradient

k = Dracy's coefficient of permeability

v = velocity of flow, or average discharge velocity

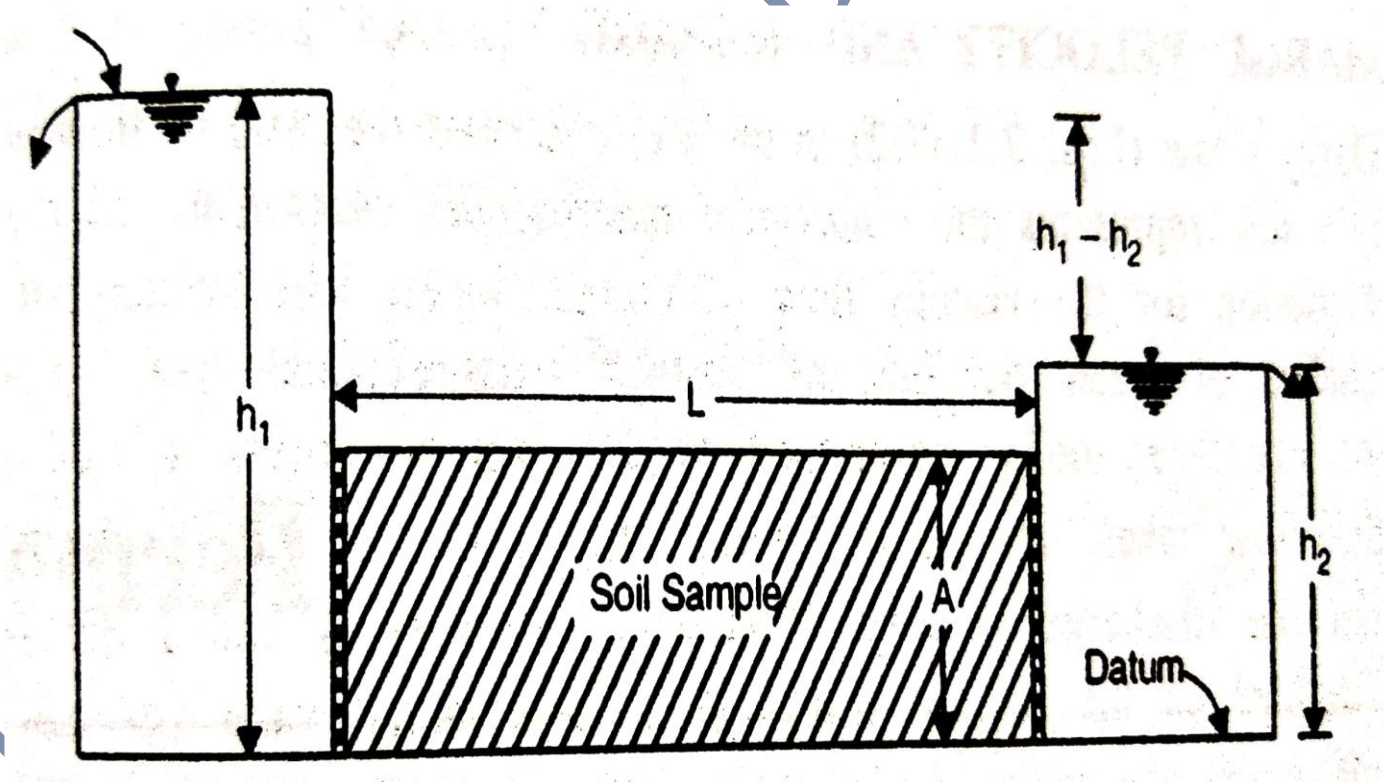


FIG. 7.1. FLOW OF WATER THROUGH SOIL.

3.2.1 Assumptions of Dracy's Law:

- The soil mass is homogeneous and fully saturated.
- The flow of water passing through the soil mass is one dimensional.
- The velocity of flow is based on total cross section area of the soil mass.
- The soil mass structure is unaltered by the flow of water.
- The void ratios remain constant during the flow.

3.3 VALIDITY OF DRACY'S LAW:

- Darcy's law of linear dependency between velocity of flow (v) and hydraulic gradient (i) is valid only for laminar flow conditions in the soil.
- From the experiments on flow through pipes, Reynolds found that the flow is laminar so long as the velocity of flow is less than a lower critical velocity v_c expresses in terms of Reynolds number as follows:

$$(v_c\ d\ \rho_w / \eta\ g) = 2000$$

Where,

v_c = lower critical velocity in the pipe (cm/sec)

d = diameter of pipe (cm)

ρ_w = density of water (g/ml)

η = viscosity of water (g sec/cm^2)

g = acceleration due to gravity (cm/sec^2)

- Based on this analogy, the flow through soils may be assumed to depend upon the dimensions of the pore spaces.
- In coarse grained soils, where the pore dimensions are larger, there will be greater possibility of flow becoming turbulent.
- Francher, Lewis and Branes (1933) demonstrate experimentally that flow through sands remains laminar and the Dracy's law valid so long as the Reynolds number, expressed in the form below, is equal to or less than unity:

$$(v\ D_a\ \rho_w / \eta\ g) \le 1$$

Where,

v = velocity of flow (cm/sec)

D_a = diameter of average particle (cm)

ρ_w = density of water (g/ml)

η = viscosity of water (g sec/cm^2)

g = acceleration due to gravity (cm/sec^2)

3.4 FACTORS AFFECTING PERMEABILITY:

The Poiseuille's law adapted for the flow through the soil pores. Comparing it with the Dracy's law: q = kiA, we get

$$k = D^2_s \times (\gamma_w / \eta) \times (e^3 / 1+e) \times C \quad - (1)$$

The factors affecting permeability are:

3.4.1 Grain size

3.4.2 Properties of the pore fluid

3.4.3 Voids ratio of the soil

3.4.4 Structural arrangement of the soil particles

3.4.5 Entrapped air and foreign matter

3.4.6 Adsorbed water in clayey soils

3.4.1 Effect of Grain Size and Shape of Particles: permeability varies approximately as the square of the grain size. Since soils consist of many different sized grains, some specific grain size has to be used for comparison. Allen Hazen (1892), based on his experiments on filter sands of particle size between 0.1 and 3mm, found that the permeability can be expresses as:

$$k = CD^2_{10}$$

Where,

k = coefficient of permeability (cm/sec per unit hydraulic gradient)

C = constant, approximately equal to 100 when D_{10} is expressed in centimetre

D_{10} = effective diameter (cm)

Attempt have been made to correlate the permeability with specific surface of the soil particles. One such relationship is given by Kozeny (1907):

$$k = (1 / K_k \eta S^2_s) \times (n^3 - (1 - n^2)$$

Where,

k = coefficient of permeability (cm/sec per unit hydraulic gradient)

n = porosity

S_s = specific surface of particle (cm^2/cm^3)

η = viscosity ($g\text{-}sec/cm^2$)

K_k = constant, equal to 5 for spherical particles

3.4.2 Effect of Properties of the Pore Fluid: Eq (1) indicates that the permeability is directly proportional to the unit weight of water and inversely proportional to its viscosity. Though the unit weight of water does not change much with the change in temperature, there is great variation in viscosity with temperature. Hence, when other factors remain constant, the effect of the property of water on the values of permeability can be expressed as:

$$k_1 / k_2 = \eta_2 / \eta_1$$

3.4.3 Effect of Voids Ratio: Eq (1) indicates that the effect of voids ratio on the values of permeability can be expressed as:

$$k_1 / k_2 = (C_1\ e_1^3 / 1 + e_1) / (C_2\ e_2^3 / 1 + e_2)$$

Laboratory experiments have shown that the factor C changes very little with the change in the voids ratio of un-stratified sand samples. However, for clays, it varies appreciably. Thus, for coarse grained soils, equation reduces to:

$$k_1 / k_2 = (e_1^3 / 1+e_1) \times (1+e_2 / e_2^3)$$

3.4.4 Effect of Structural Arrangement of Particles and Stratification: the structural arrangement of the particle may vary, at the same voids ratio, depending upon the method of deposition or compacting the soil mass. The structure may be entirely different for a disturbed sample as compared to an undisturbed sample which may possess stratification. The effect of structural disturbance on permeability is much pronounced in fine-grained soils.

3.4.5 Effect of Degree of Saturation and other Foreign Matter: the permeability is greatly reduced if air is entrapped in the voids thus reducing its degree of saturation. The dissolved air in the pore fluid (water) may get liberated, thus changing the permeability. Ideal condition of test are when air-free distilled water is used and the soil is completely saturated by vacuum saturation, for measuring the permeability. However, since the percolating water in the field may have some gas content, it may appear more realistic to use the actual field water for testing in the laboratory.

3.4.6 Effect of Adsorbed Water: the adsorbed water surrounding the fine soil particles is not free to move, and reduces the effective pore space available for the passage of water. According to a crude approximation after Casagrande, 0.1 may be taken as the voids ratio occupied by adsorbed water, and the permeability may be roughly assumed to be proportional to the square of the net voids ratio of (e – 0.1).

3.5 COEFFICIENT OF ABSOLUTE PERMEABILITY:

- The coefficient of permeability (k) depends not only on the properties of the soil mass (such as size, shape, specific surface, structural arrangement, stratification, voids ratio etc.,) but also on the properties of the permeant (i.e. water) which flows through it.
- Let us now introduce a coefficient which does not depend upon the properties of permant. Such a coefficient, known as coefficient of absolute permeability (K) is defined by the expression:

$$K = k\,(\eta / \gamma_w)$$

$$K = C\,(e^3 / 1+e)\,D^2$$

- The above equation indicates that the coefficient of absolute permeability is independent of the properties of permeant (i.e. water) and it depends solely on the properties of soil mass.
- Dimension of K is given as:

$$[\,K\,] = [\,L / T\,] \times [\,F\,T / L^2\,] \times [\,L^3 / F\,] = [\,L^2\,]$$

- Hence K has the dimension of area. The units of K are: mm^2, cm^2, m^2 or darcy.
- 1 darcy = 0.987×10^{-8} cm^2.
- It is interesting to note that for a given voids ratio and structural arrangement of particles, the coefficient of absolute permeability is constant, irrespective of type of fluid.

3.6 DETERMINATION OF COEFFICIENT OF PERMEABILITY:

The coefficient of permeability can be determined by the following methods:

3.6.1 Laboratory Methods:

3.6.1.1 Constant head permeability test

3.6.1.1 Falling head permeability test

3.6.2 Field Methods:

3.6.2.1 Pumping out tests

3.6.2.2 Pumping in tests

3.6.3 Indirect Methods:

3.6.3.1 Computation from grain size or specific surface

3.6.3.2 Horizontal capillarity test

3.6.3.3 Consolidation test data

3.6.1 LABORATORY METHODS:

3.6.1.1 Constant Head Permeability Test:

Figure 7.5 shows the diagrammatical representation of constant head test.

- Water flows from the overhead tank consisting of three tubes: the inlet tube, the overflow tube and the outlet tube.
- The constant hydraulic gradient i causing the flow is the head h (i.e. difference in the water levels of the overhead and bottom tanks) divided by the length L of the sample.
- If the length of the sample is large, the head lost over a length of specimen is measured by inserting piezometric tubes, as shown in Fig. 7.5 (b).
- If Q is the total quantity of flow in a time interval t, we have from Darcy's law,

$$q = Q / t = k\ i\ A$$

$$k = (Q / t)\ (1 / i\ A) = (Q / t)\ (L / h)\ (1 / A)$$

Where, A = total cross sectional area of sample.

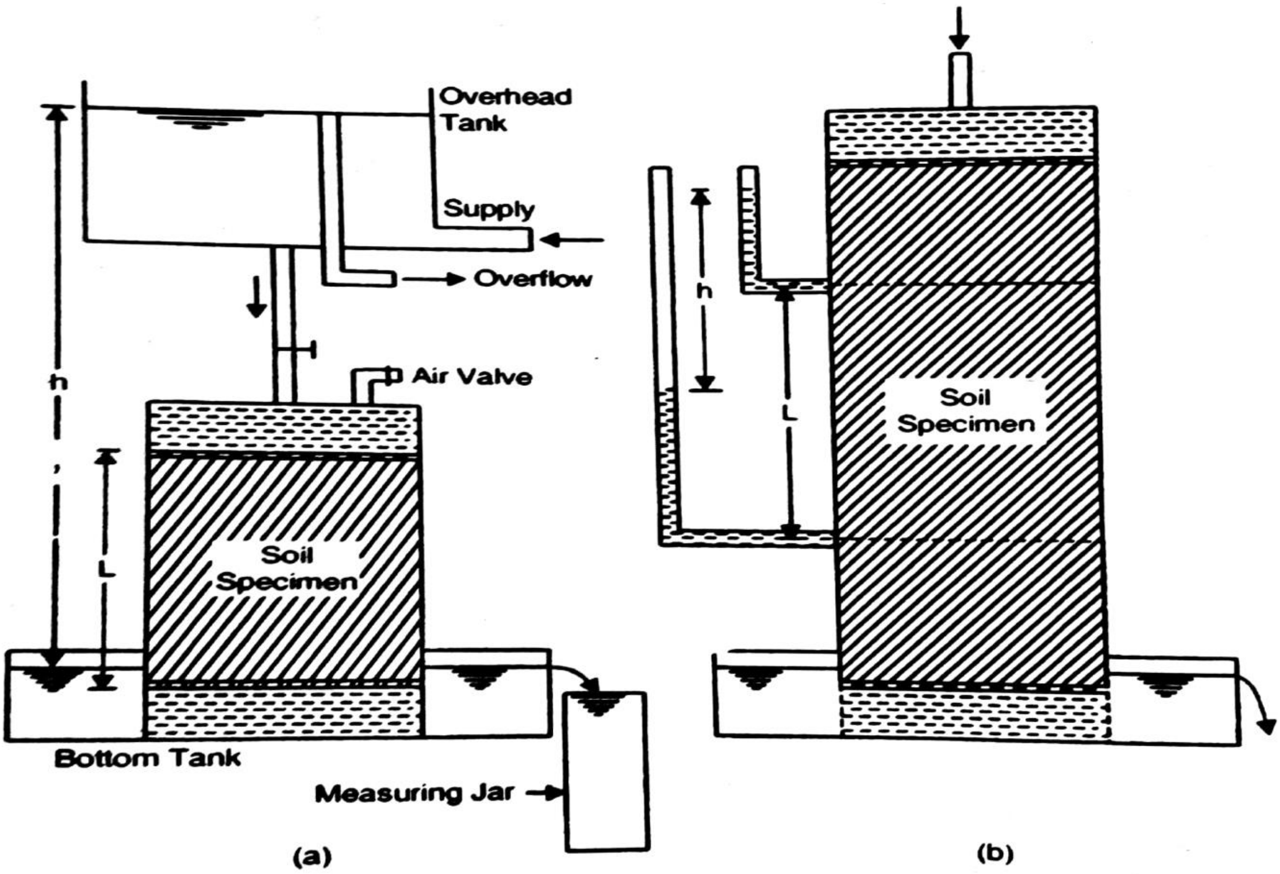


FIG. 7.5. CONSTANT HEAD TEST.

3.6.1.1 Falling Head Permeability Test:

- The constant head permeability test is used for coarse grained soil only when a reasonable discharge can be collected in a given time.
- However, the falling head test is used for relatively less permeable soils where the discharge is small. Fig. 7.6 shows the diagrammatical representation of a falling head test arrangement.
- A stand pipe of known cross-sectional area a is fitted over the permeameter and water is allowed to run down.
- The water level in the stand pipe constantly falls as water flows. Observations are started after steady state of flow has reached.
- The head at any time interval t is equal to the difference in the water level in the stand pipe and the bottom tank. Let h_1 and h_2 be heads at time intervals t_1 and t_2 ($t_2 > t_1$) respectively.
- Let h be the head at any intermediate time interval t, and –dh be the change in the head in a smaller time interval dt (minus sign has been used since h decreases as t increase).
- Hence, from Darcy's law, the rate of flow q is given by:

$$q = (-dh \,.\, a) / dt = kiA$$

Where, i = hydraulic gradient at time t = (h / L)

$$(k\, h / L)\, A = - (dh / dt)\, a \text{ or } (Ak / aL)\, dt = - (dh / h)$$

- Integrating between two time limits, we get

$$(AK / aL) \int_{t1}^{t2} dt = - \int_{h1}^{h2} (dh / h) = \int_{h2}^{h1} (dh / h)$$

$$(AK / aL)\,(t_2 - t_1) = \log_e (h_1 / h_2)$$

Denoting $t_2 - t_1 = t$, we get

$$k = (aL / At) \log_e (h_1/h_2) = 2.3\, (aL/At) \log_{10} (h_1/h_2)$$

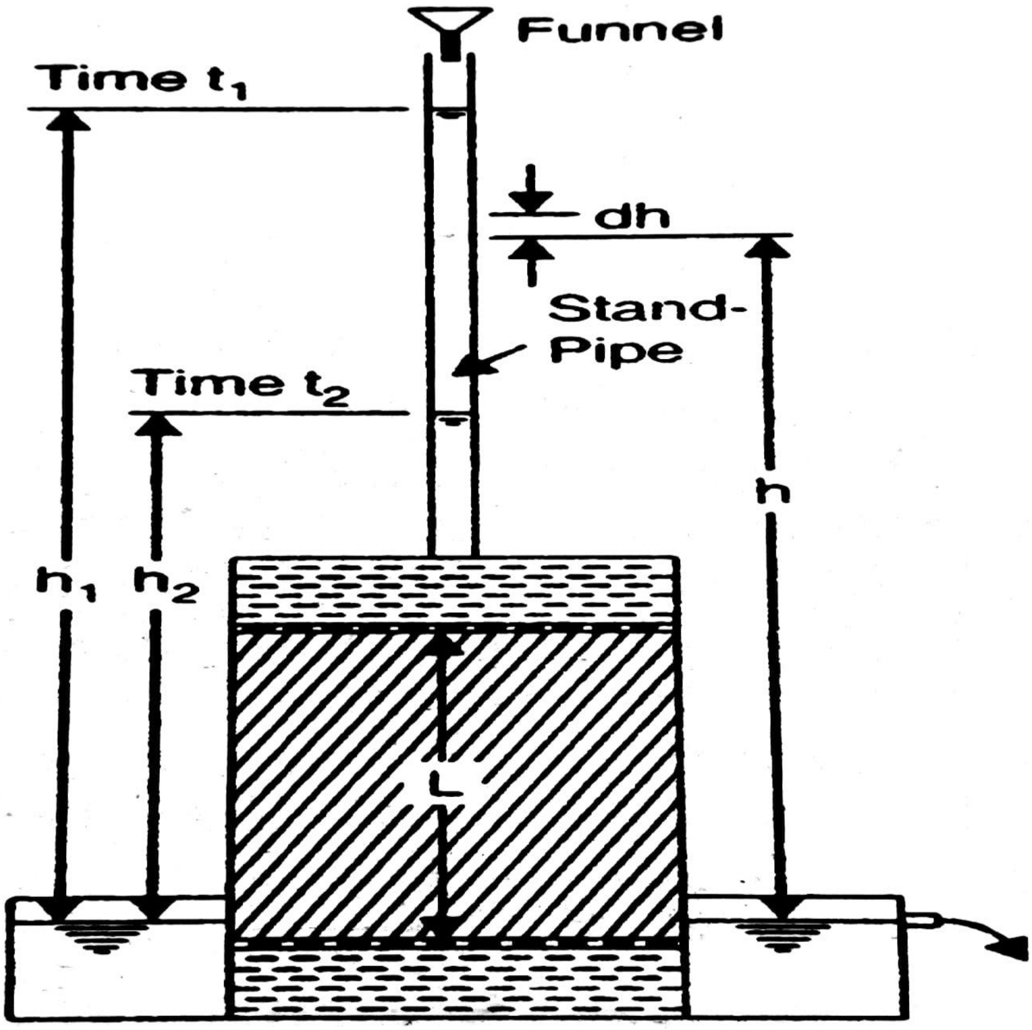


FIG. 7.6. FALLING HEAD TEST.

3.6.2 FIELD METHODS:

- Compared to laboratory tests, field permeability tests are more reliable. They give the in-situ value of permeability with minimum disturbance.
- The value of coefficient so obtained is an overall average for a large area. The permeability can be determined by either a pumping out test or pumping in test.

3.6.2.1 PUMPING OUT TESTS:

3.6.2.1.1 Pumping out Test in Unconfined Aquifer: the equations as follows:

$$k = q / \pi (H^2 - h^2) \times \log_e (R/r) = q / 1.36 (H^2 - h^2) \log_{10} (R/r)$$

$$k = q / \pi (h_2^2 - h_1^2) \times \log_e (r_2/r_1) = q / 1.36 (h_2^2 - h_1^2) \log_{10} (r_2/r_1)$$

3.6.2.1.2 Pumping out Test in Confined Aquifer:

$$k = q / 2\pi b (H - h) \times \log_e (R/r) = q / 2.72b (H - h) \log_{10} (R/r)$$

$$k = q / 2\pi b (h_2 - h_1) \times \log_e (r_2/r_1) = q / 2.72b (h_2 - h_1) \log_{10} (r_2/r_1)$$

3.6.2.2 PUMPING IN TESTS:

3.6.2.2.1 Open End Test:

- An open end pipe is sunk in the strata and the soil is taken out of the pipe just to the bottom.
- Clean water, having temperature slightly higher than the ground water is added through a metering system to maintain gravity flow under constant head.
- Water may also be allowed to enter the hole under some pressure head.
- The permeability is calculated from the following expression determined from the electrical analogy experiments:

$$k = q / 5.5rh$$

Where,

h = differential head of water

r = radius of casing

q = constant rate of flow

3.6.2.2.2 Packer Tests:

- An uncased portion of the drill hole or a perforated portion of the casing is used for performing the test.
- In case the test is performed during drilling, a top packer just inside or below the casing. Water is pumped in the lower portion of the hole.
- To perform the test after completion of the hole, which can stand without casing, two packer set on a pipe or drill stem keeping the perforated portion of the pipe between the plugs.
- The bottom of the pipe is plugged. The length of packer on expansion should be five times the diameter of the hole. Testing is started from the bottom of the hole and continued upwards.
- The coefficient of permeability is determined from the following expressions:

$$k = q / 2\pi Lh \log_{10} (L/r);\ L \geq 10r$$

$$k = q / 2\pi Lh \sinh^{-1} (L/2r);\ 10r > L \geq r$$

Where, L = length of portion of the hole tested

3.7 PERMEABILITY OF STRATIFIED SOIL DEPOSITS:

- In nature, soil mass may consists of several layers deposited one above the other. These bedding planes may be horizontal, inclined or vertical.
- Each layer, assumed to be homogenous and isotropic, has its own value of coefficient of permeability.
- The average permeability of the whole deposit will depend upon the direction of flow with relation to the direction of the bedding planes.
- We shall consider both the cases of flow:

3.7.1 Parallel to the bedding planes 3.7.2 Perpendicular to the bedding planes

3.7.1 Parallel to the Bedding Planes:

- Let Z_1, Z_2…….Z_n = thickness of layer and k_1, k_2…….k_n = permeabilites of the layers. For flow to be parallel to the bedding planes, the hydraulic gradient i will be same for all the layers.
- However, since v = ki and since k is different, the velocity of flow will be different in different layers. Let k_x = average permeability of the soil deposit parallel to the bedding plane.
- Total discharge through the soil deposit = Sum of discharge through the individual layers.

$$q = q_1 + q_2 + \ldots\ldots q_n.$$

$$q = k_x\, i\, Z = k_1\, i\, Z_1 + k_2\, i\, Z_2 + \ldots\ldots. K_n\, i\, Z_n$$

$$k_x = (k_1 Z_1 + k_2 Z_2 + \ldots\ldots. + k_n Z_n / Z) \text{ (Where } Z = Z_1 + Z_2 + \ldots\ldots Z_n)$$

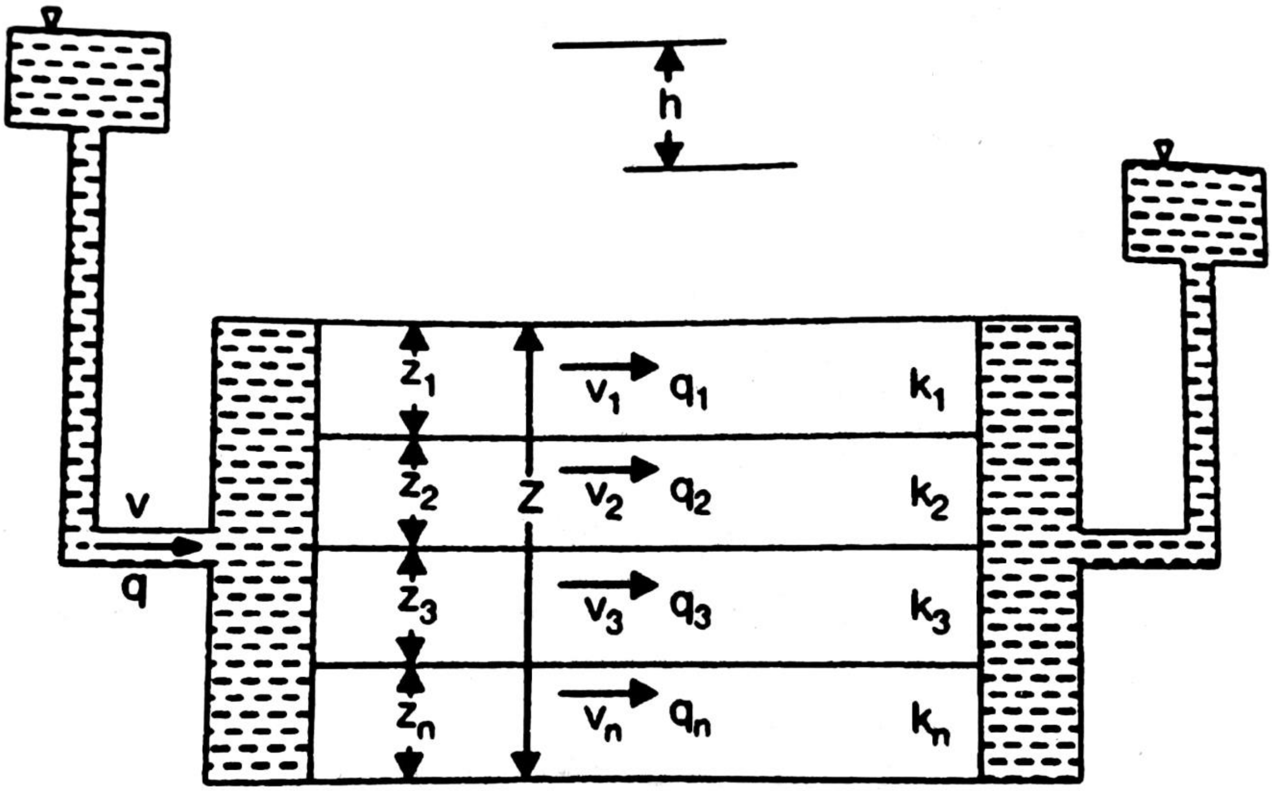


FIG. 7.8. FLOW PARALLEL TO BEDDING PLANE.

3.7.2 Perpendicular to the Bedding Planes:

- In this case, the velocity of flow, and hence the unit discharge, will be the same through each layer. However, the hydraulic gradient, and hence the head loss through each layer will be different.
- Denoting the head loss through the layers by h1, h2…… hn and the total head loss h, we have:

$$h = h_1 + h_2 \ldots\ldots. h_n$$

$$h_1 = i_1 Z_1 ,\ h_2 = i_2 Z_2 \ldots\ldots\ldots.. h_n = i_n Z_n$$

$$h = i_1 Z_1 + i_2 Z_2 + \ldots\ldots\ldots i_n Z_n$$

Now, if k_z = average permeability perpendicular to the bedding plane, we have

$$v = k_z i = k_z (h / Z) \text{ or } h = (vZ / k_z)$$

$$\text{Also } i_1 = (v/k_1) , i_2 = (v/k_2) , i_n = (v/k_n)$$

$$k_z = (Z / (Z_1/k_1) + (Z_2/k_2) + \ldots\ldots (Z_n/k_n))$$

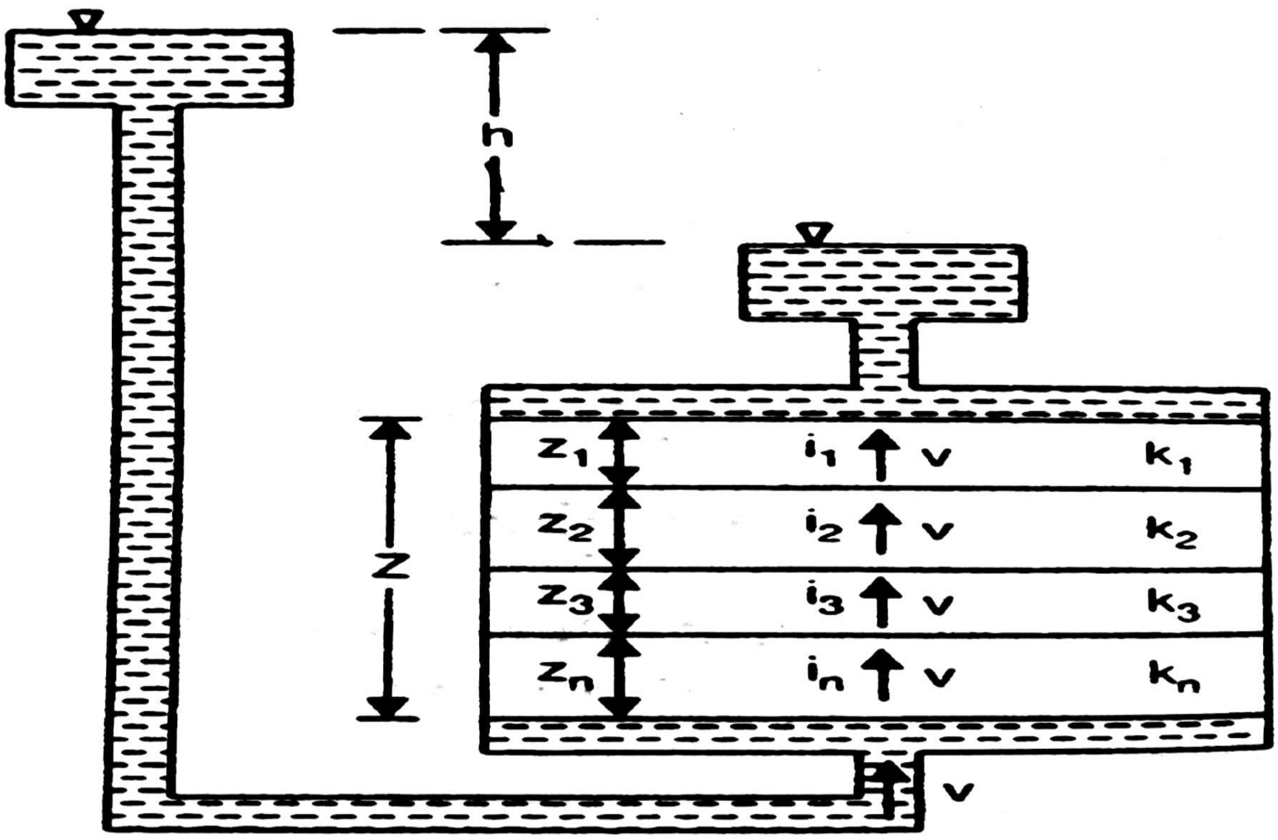


FIG. 7.9. FLOW PERPENDICULAR TO BEDDING PLANE

3.8 SEEPAGE VELOCITY:

It is defined as the rate of discharge of percolating water per unit cross-sectional area of voids perpendicular to the direction of flow.

3.9 SUPERFICIAL VELOCITY:

It is defined as the velocity of flow or the rate of flow of water through the soil mass after entering in and emerging out of the total voids present in the entire soil mass.

3.10 COEFFICIENT OF PERCOLATION:

The seepage velocity is proportional to hydraulic gradient. The k_p is known as the coefficient of percolation. Its value is always greater than the coefficient of permeability (k).

3.11 CAPILLARY RISE:

The height above a free water elevation to which water will rise due to capillary action.

3.12 CAPILLARY PHENOMENON IN SOILS:

- The voids in a natural deposit act as capillary tubes and water rises in the continuous voids to a certain height above ground water table or free surface.
- The height to which water rises is called capillary rise, which depends on particle size and void ratio.
- All other factors being equal, the capacity rise in the fine grained soils is always greater than in coarse grained soils.

The capillary rise in soils depends on:

- The size of void that is effective. It varies inversely with the size of voids.
- The particle size and density of soil.
- The capillary rise is most pronounced in soils composed mainly of fine sands, silts, silty clays and colloids.

MODULE – 3

SEEPAGE ANALYSIS

3.13 INTRODUCTION:

Seepage is the infiltration or percolation of water through rock or soil. The term seepage is usually restricted to the very slow movement of ground water. When the seepage velocity is great enough, erosion can occur because of the frictional drag exerted on the soil particles. Vertically upwards seepage is a source of danger on the downstream side of sheet piling and beneath the toe of a dam. Erosion of the soil, known as piping, can lead to failure of the structure and sinkhole formation. The term sand boil is used to describe the appearance of the discharging end of an active soil pipe.

3.14 LAPLACE EQUATION:

The quantity of water flowing through a saturated soil mass, as well as the distribution of water pressure can be estimated by the theory of flow of fluids through porous medium. While computing these quantities with the help of theoretical analysis that follow, the following assumptions as made:

3.14.1 Assumptions:

- The saturated porous medium is incompressible. The size of the pore spaces does not change with time, regardless of water pressure.
- The seeping water flows under a hydraulic gradient which is due only to gravity head loss, or Dracy's law for flow through porous medium is valid.
- There is no change in the degree of saturation in the zone of soil through which water seeps and the quantity of water flowing into any element of volume is equal to the quantity which flows out in the same length of time.
- The hydraulic boundary conditions at entry and exit are known.
- Water is incompressible.

3.14.2 Limitations:

- Flow is laminar.
- Degree of saturation is 100%.
- Coefficient of permeability is constant everywhere in the soil medium.
- Coefficient of permeability is same in all direction.

- No expansion or contraction.
- No volume change occurs.

3.14.3 Derivation of Laplace Equation:

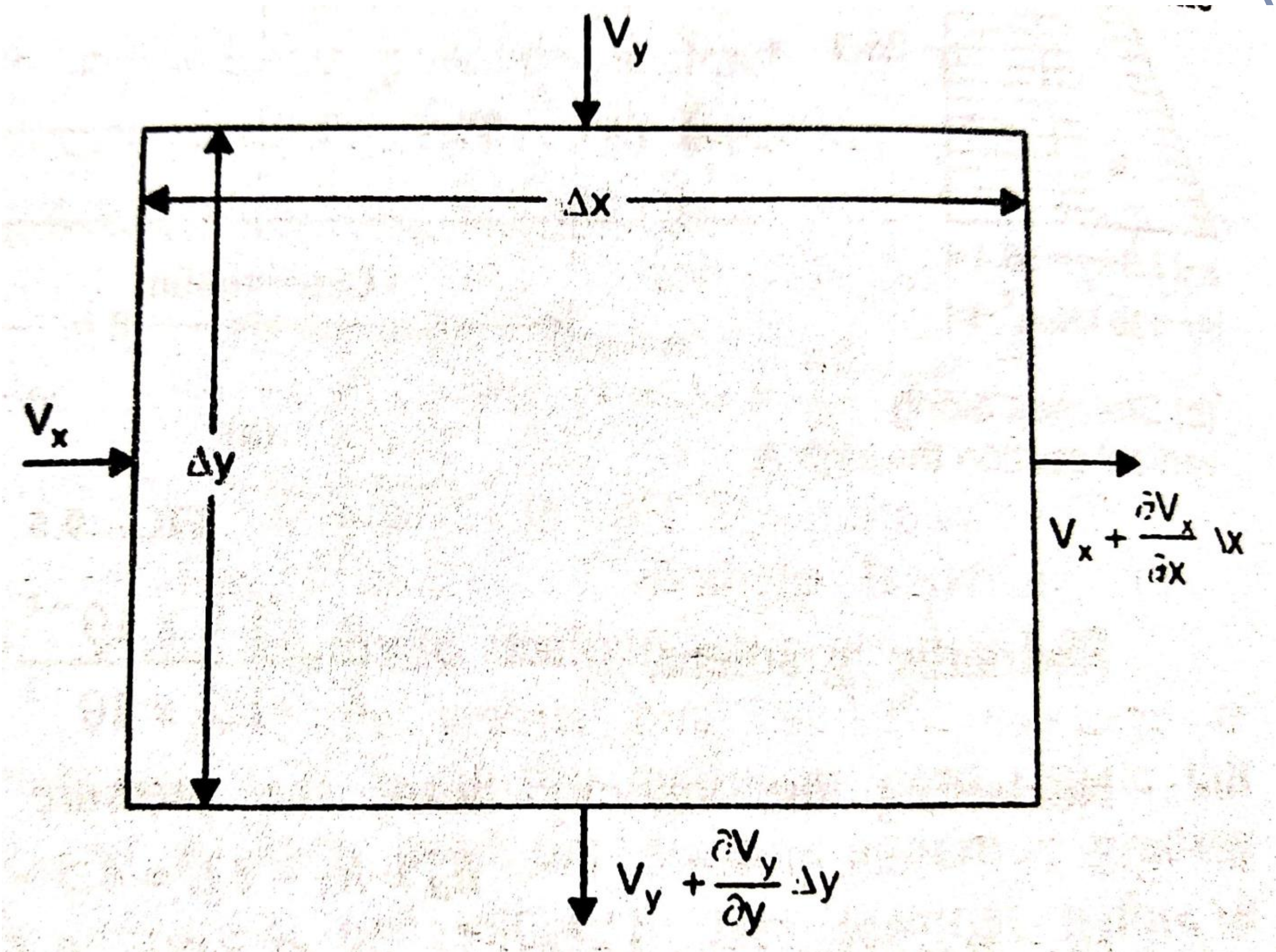


FIG. 9.6. TWO DIMENSIONAL FLOW.

- Consider an element of soil of size Δ x, Δ y and of unit thickness perpendicular to the plane of the paper. Let v_x and v_y be the entry velocity components in x and y directions.
- Then $(v_x + (\partial v_x / \partial x)\ \Delta x)$ and $(v_y + (\partial v_y / \partial y)\ \Delta y)$ will be the corresponding velocity components at the exit of the element.
- According to assumption 3 stated above, the quantity of water entering the element is equal to the quantity of water leaving it.
- $v_x (\Delta y \cdot 1) + v_y (\Delta x \cdot 1) = (v_x + (\partial v_x / \partial x) \cdot \Delta x)(\Delta y \cdot 1) + (v_y + (\partial v_y / \partial y) \cdot \Delta y)(\Delta x \cdot 1)$
- From which, $(\partial v_x / \partial x) + (\partial v_y / \partial y) = 0$ Eq. 9.9. This is the continuity equation.

- According to assumption 2, $v_x = k_x \cdot i_x = k_x (\partial h / \partial x)$ and $v_y = k_y \cdot i_y = k_y (\partial h / \partial y)$

Where h = hydraulic head under which water flows

k_x and k_y = coefficient of permeability in x and y directions.

- Substituting these in Eq. 9.9, we get $(\partial^2 (k_x h)) / \partial x^2) + (\partial^2 (k_y h)) / \partial y^2) = 0$
- For an isotropic soil, $k_x = k_y = k$ (say) $(\partial^2 h/\partial x^2) + (\partial^2 h/\partial y^2) = 0$
- Substituting $\Phi = kh$ = velocity potential, we get $(\partial^2\Phi/\partial x^2) + (\partial^2\Phi/\partial y^2) = 0$ – Laplace Equation.

3.14.4 Velocity Potential (Φ): the velocity potential may be defined as a scalar function of space and time such that its derivative with respect to any direction gives the fluid velocity in that direction. This is evident, since we have $\Phi = kh$.

$$(\partial\Phi / \partial x) = k (\partial h / \partial x) = ki_x = v_x$$

$$(\partial\Phi / \partial y) = k (\partial h / \partial y) = ki_y = v_y$$

The solution can be obtained by

(i) Analytical Methods
(ii) Graphical Methods
(iii) Experimental Methods

3.15 FLOWNET:

- A flow-net is a graphical representation of two dimensional steady state ground water flows through aquifers.
- Construction of a flow-net is often used for solving ground water flow problems where the geometry makes analytical solutions impractical.
- The method is often used in Civil Engineering, hydrogeology or soil mechanics as a first check for problems of flow under hydraulic structures like dams or sheet pile walls.
- As such, a grid obtained by drawing a series of equipotential lines is called a flow-net. The flow-net is an important tool in analysing two dimensional irrotational flow problems.

3.15.1 Characteristics or Properties of Flow-Net:

- The flow lines and equipotential lines meet at right angles to one another.

- The fields are approximately squares, so that a circle can be drawn touching all the four sides of the square.
- The quantity of water flowing through each flow channel is the same. Similarly, the same potential drop occurs between two successive equipotential lines.
- Smaller the dimensions of the filed, greater will be the hydraulic gradient and velocity of flow through it.
- In a homogeneous soil, every transition in the shape of the curves is smooth, being either elliptical or parabolic in shape.

3.15.2 Application or Uses of Flow-Net: a flow-net can be used for the following purposes

3.15.2.1 Determination of seepage

3.15.2.2 Determination of hydrostatic pressure

3.15.2.3 Determination of seepage pressure

3.15.2.4 Determination of exit gradient

3.15.2.1 Determination of Seepage:

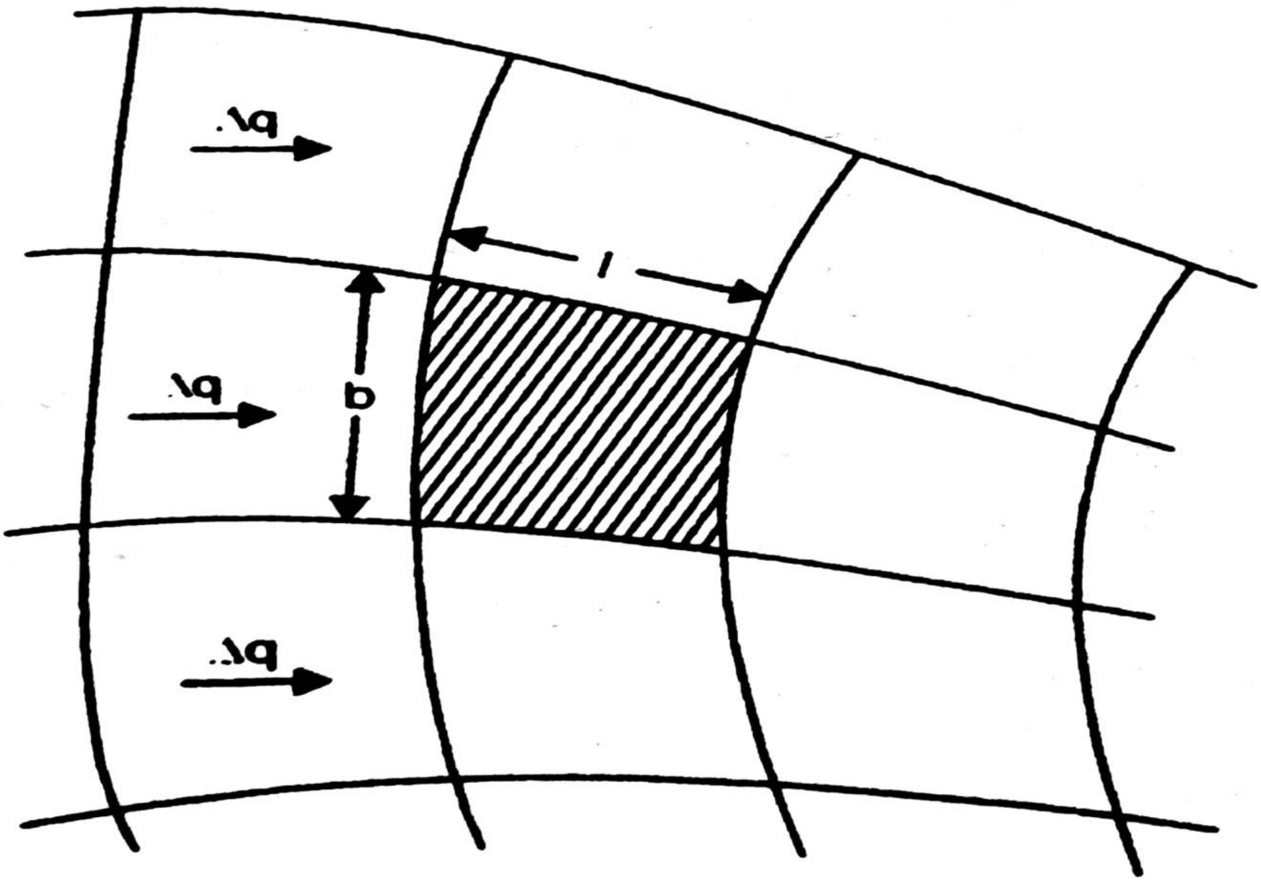


FIG. 9.7. PORTION OF A FLOW NET.

- Fig. 9.7 shows a portion of a flow-net. The portion between any two successive flow lines is known as at flow channel.
- The portion enclosed between two successive equipotential lines and successive flow lines is known as field such as that shown hatched in Fig. 9.7.
- Let b and *l* be the width and length of the field, Δ h = head drop through the field, Δ q = discharge through the flow channel, H = total hydraulic head causing flow = difference between upstream and downstream heads.
- Then, from Darcy's law of flow through soils:

$$\Delta q = k (\Delta h / l) \times (b \times 1) \text{ (Considering unit thickness)}$$

- If N_d = total number of potential drops in the complete flow net then, $\Delta h = H / N_d$.

$$\Delta q = k \times (H / N_d) \times (b / l)$$

- The total discharge through the complete flow net is given by

$$q = \sum \Delta q = k \times (H / N_d) \times (b / l) \times N_f = k H \times (N_f / N_d) \times (b / l)$$

Where, N_f = total number of flow channels in the net. The field is square; hence b = *l*

$$b = k H (N_f / N_d)$$

- This is the required expression for the discharge passing through a flow net and is valid for isotropic soils in which $k_x = k_y = k$.

3.15.2.2 Determination of Hydrostatic Pressure:

- The hydrostatic pressure at any point within the soil mass is given by $u = h_w \times \gamma_w$
- Where, u = hydrostatic pressure, h_w = piezometric head.
- The hydrostatic pressure in terms of piezometric head h_w is calculated from the following relation $h_w = h – Z$.
- Where, h = hydraulic potential at the point under consideration.
- Z = position head of the point above datum, considered positive upwards.
- All the three quantities h_w, h and Z can be expressed as the percentage of the total hydraulic head H.

3.15.2.3 Determination of Seepage Pressure:

- The hydraulic potential h at any point located after n potential drops, each of value Δh is given by

$$h = H - n \Delta h$$

- The seepage pressure at any point equals the hydraulic potential or the balance hydraulic head multiplied by the unit weight of water and hence, is given by

$p_s = h \gamma_w = (H - n \Delta h) \gamma_w$. The pressure acts in the direction of flow.

3.15.2.4 Determination of Exit Gradient:

- The exit gradient is the hydraulic gradient at the downstream end of the flow line where the percolating water leaves the soil mass and emerges into the free water at the downstream.
- The exit gradient can be calculated from the following expression, in which Δh represents the potential drop and *l* the average length of last field in the flow net at exit end:

$$i_e = \Delta h / l$$

3.15.3 Procedure for Drawing Flow-Nets:

- A flow-net is to be drawn by trial and error. For a given set of boundary conditions, the flow net will remain the same even if the direction of flow is reversed.
- Figure shows the flow-net for flow around a sheet pile. In figure lines a-a-a, b-b-b and c-c-c, are one set of lines called as stream lines or flow lines.
- Flow occurs along the stream lines and there is no flow across a stream line. The other set of lines are orthogonal to the streamlines and are known as equipotential lines. Along an equipotential line the total head is constant.

To draw a flow-net the following procedure may be followed:

- Draw the geometry of the structure correctly on the paper. It implies that the horizontal and vertical scales should be the same. Otherwise, the square shape requirement (x = y) cannot be met.
- Identify the boundary flow lines and boundary equipotential lines in the drawing. In an example in figure, the upstream ground surface and downstream surface are the initial and the final equipotential lines, respectively. The front and back sides of the sheet pile and the surface of the impervious layer are the boundary flow lines. In figure for example: (i) CDE is a flow line, (ii) MM' is a flow line, (iii) OC is an equipotential line, (iv) EO' is an equipotential line.

- Draw a set of stream lines and equipotential lines within the boundaries, keeping in mind the two points that (i) stream lines do not intersect each other, and (ii) stream lines and equipotential lines intersect each other at right angles.
- Adjust the number of equipotential lines and flow lines such that the network is made of curvilinear squares.

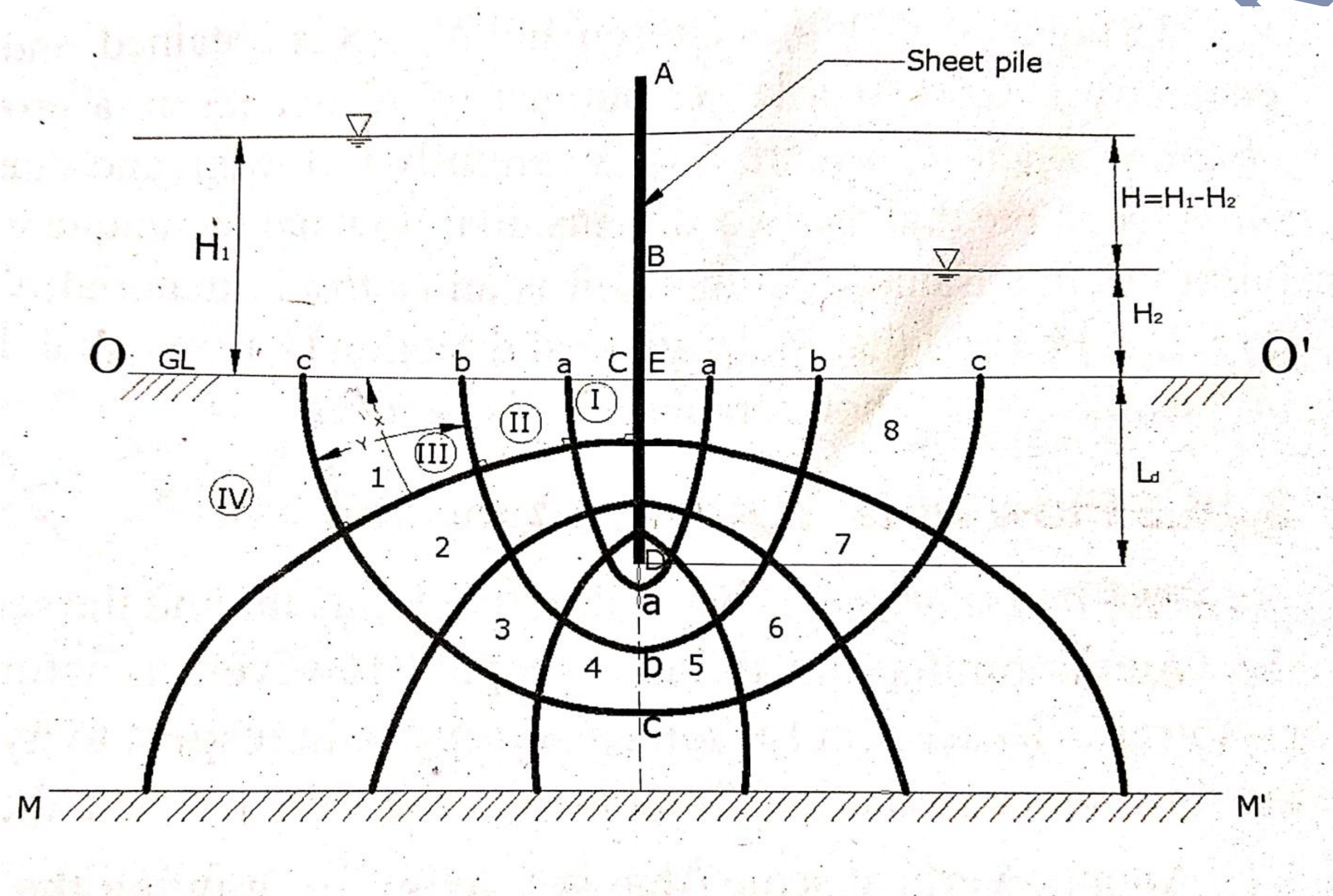


Fig. 3.16 : Flow Net for flow around sheet pile

3.15.4 Flow-Nets in Anisotropic Soil:

- Let us now consider the case of an anisotropic flow medium in which $k_x \neq k_y$. For such case, the flow equation becomes:

$$k_x\,(\partial^2 h / \partial x^2) + k_y\,(\partial^2 h / \partial y^2) = 0$$

- Which is not a Laplacian equation. Hence flow-net cannot be directly drawn by rewriting it, we get:

$$(k_x / k_y)\,(\partial^2 h / \partial x^2) + (\partial^2 h / \partial y^2) = 0$$

- Let us put $x_n = x \sqrt{(k_y / k_x)}$
- Where x_n is the new co-ordinate variable in the x – direction.
- Then the above equation becomes:

$$(\partial^2 h / \partial x^2) + (\partial^2 h / \partial y^2) = 0. \text{ This is in Laplacian form.}$$

- To plot the flow-net for such a case, the cross section through anisotropic soil is plotted to a natural scale in the y – direction, but to a transformed scale in the x – direction, all the dimensions parallel to x – axis being reduced by multiplying by the factor $\sqrt{(k_y / k_x)}$.
- The flow-net for this transformed section will now be constructed in the normal manner as if the soil were isotropic.
- The actual flow-net is then obtained by re-transforming the cross-section including the flow-net, back to the natural scale by multiplying by a factor $\sqrt{(k_x / k_y)}$.
- The actual flow-net thus, will not have orthogonal set of curves.
- As shown in figure, the field of transformed section will be a square one, while the field of the actual section (re-transformed) will be rectangular one having its length in x – direction equal to $\sqrt{(k_x / k_y)}$ times the width in the y – direction.
- Let k_x = permeability coefficient in x – direction of the actual anisotropic soil field, k' = equivalent permeability of the transformed field.
- Then, for the transformed section:

$$\Delta q = k' (\Delta h / l) \times (l \times 1)$$

- For the actual field:

$$\Delta q = k_x (\Delta h / l \sqrt{(k_x / k_y)}) \times (l \times 1)$$

- Since the quantity of flow is the same. Hence, the discharge is given by

$$q = k' H (N_f / N_d) = \sqrt{(k_x / k_y)} H (N_f / N_d)$$

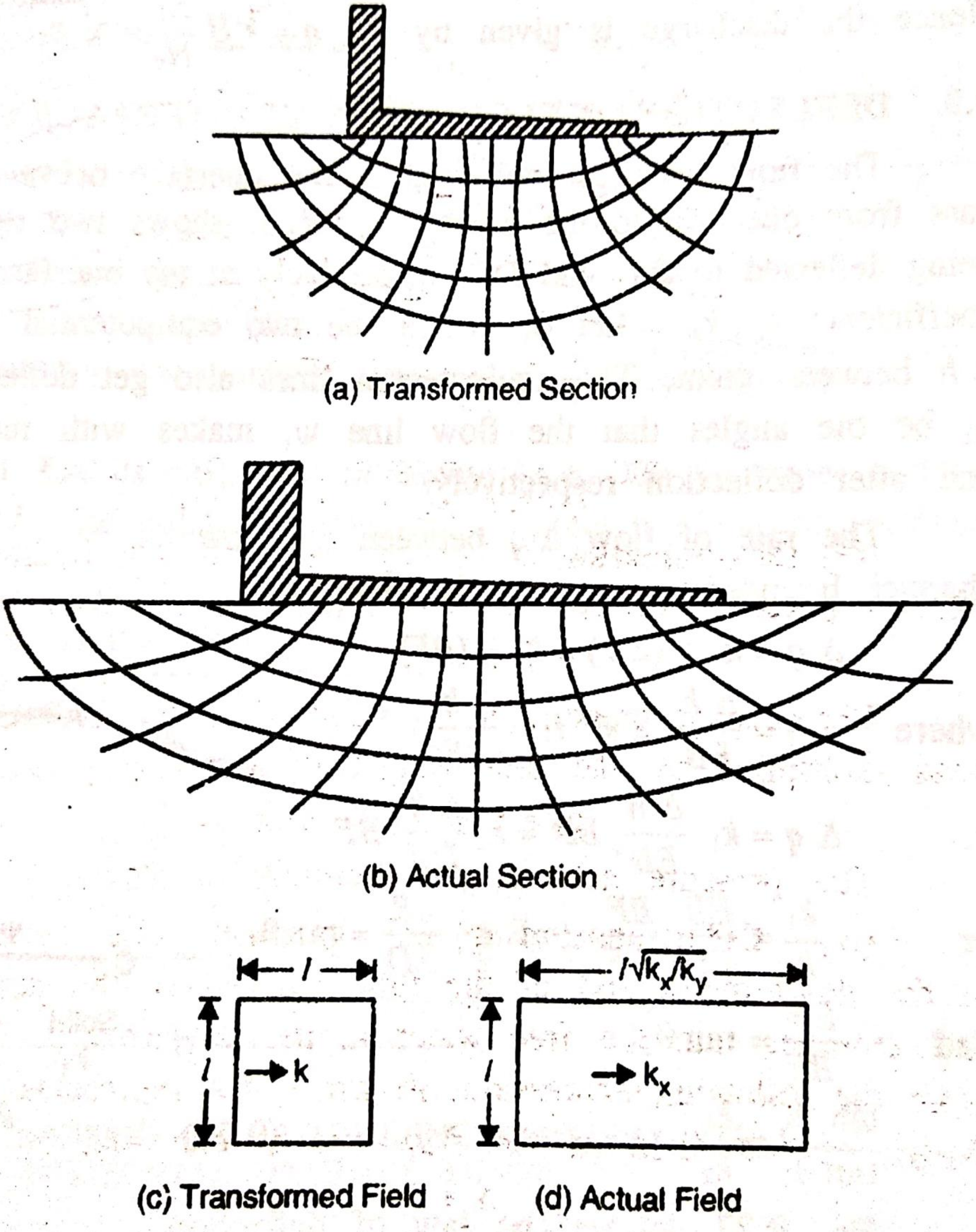


FIG. 9.13. ANISOTROPIC FLOW.

3.15.5 Typical Illustrations of Flow-Net:

Following figures demonstrate the typical flow-nets drawn for different kinds of seepage problems pertaining to flow beneath dam, dam with sheet pile as heel cut-off wall and dam with sheet pile as toe cut-off respectively.

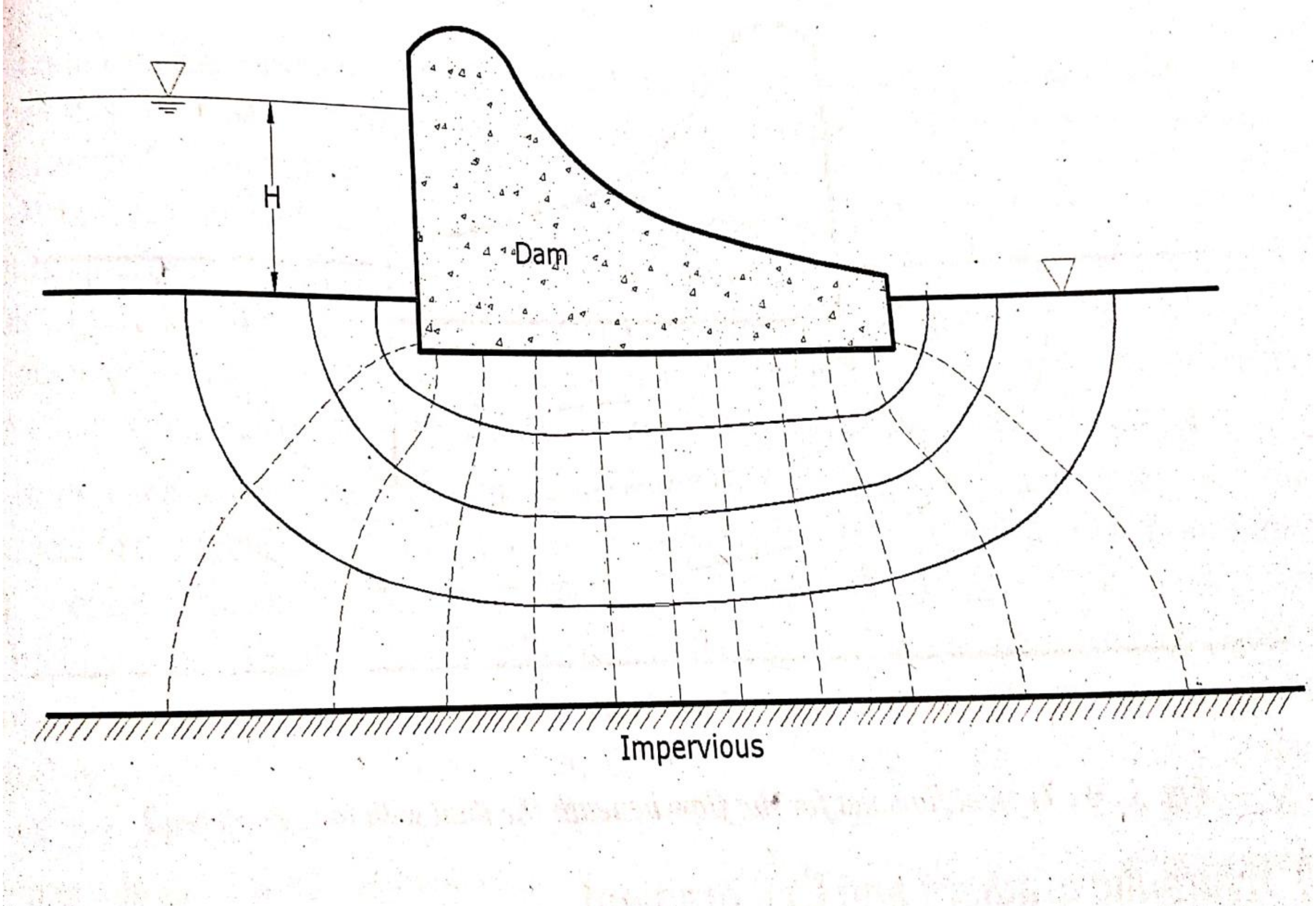


Fig. 3.18 : Typical flow net for the flow beneath the dam without any cutoff wall

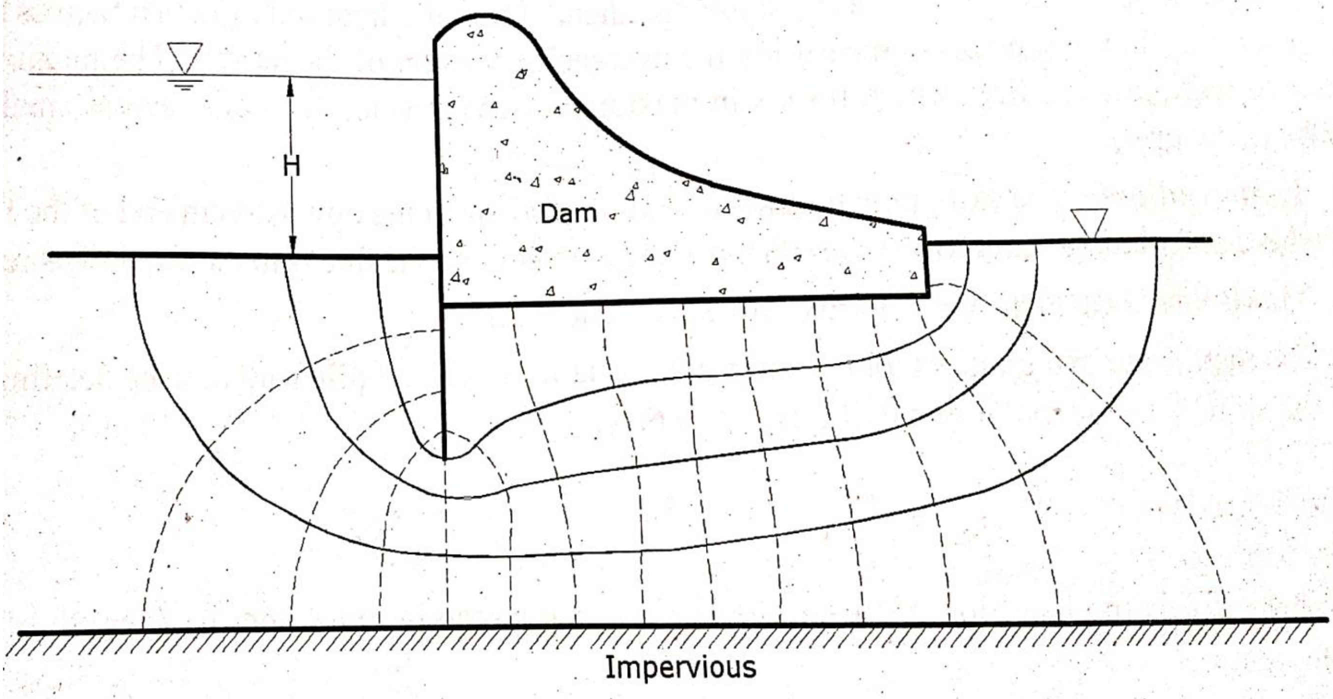


Fig. 3.19 : Typical flow net for the flow beneath the dam with heel cutoff wall

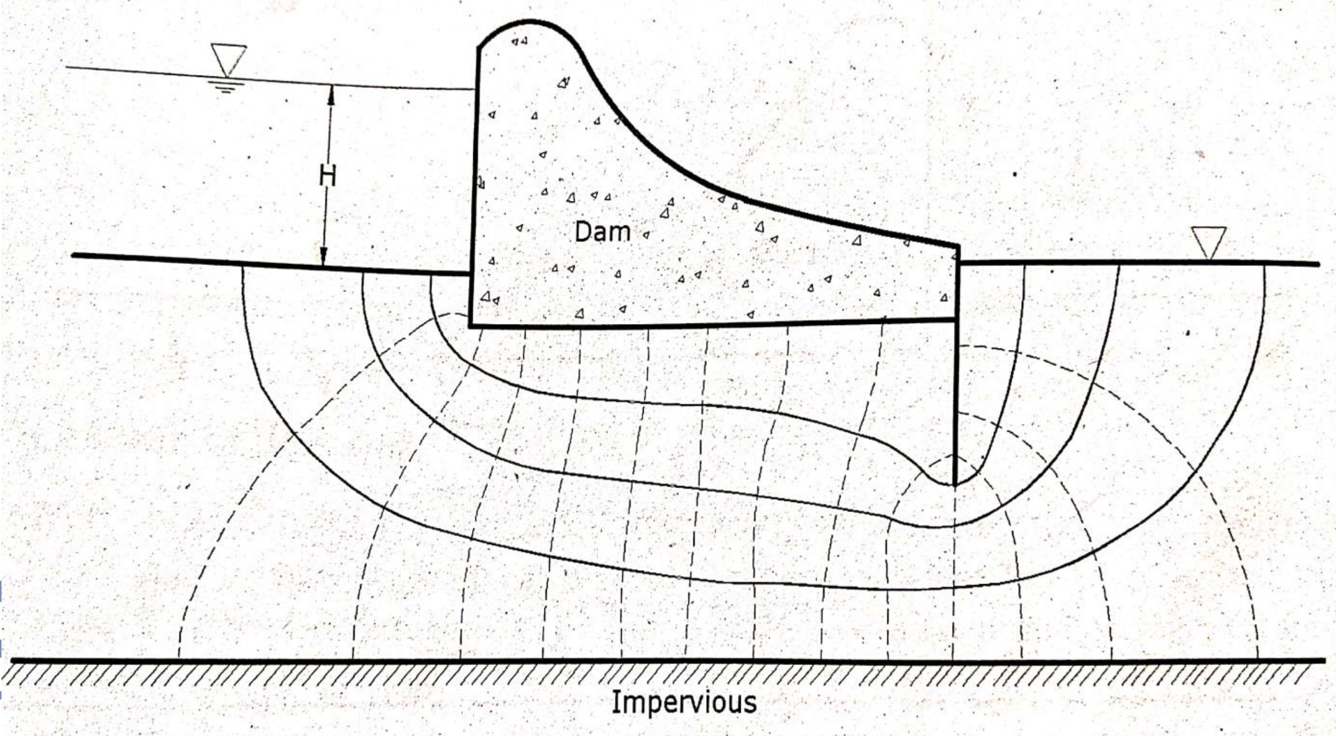


Fig. 3.20 : Typical flow net for the flow beneath the dam with toe cut off wall

3.16 CONFINIED AND UNCONFINED FLOW:

3.16.1 Confined Flow:

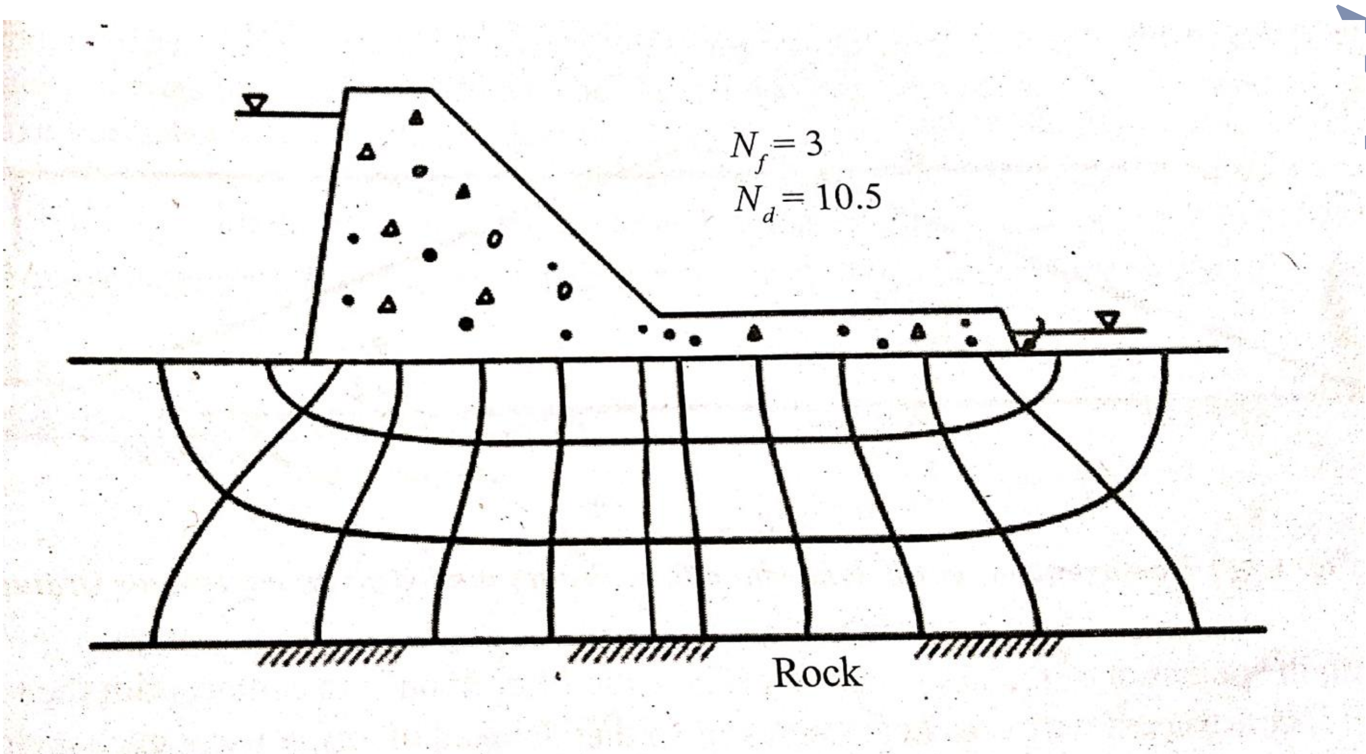


Fig. 3.23 : Flow net for flow through previous foundation of a weir with $N_d = 10.5$

- In confined flow, the flow occurs under pressure. The flow takes place between non-leaky or impervious boundaries. Flow under a concrete dam is an example of confined flow.
- Examples of flow-nets depicted in pervious sections were for boundary conditions which produce confined flow, that is, there were enough boundaries to define the flow space completely and the flow was thus forced to confine itself to that space.
- Figure. 3.23 is another example of such flow, and is presented here only to describe one more features of flow-nets which apply to confined as well as to unconfined flow.
- For real boundary conditions, the situation where the correct flow-net turns out to have a whole number of flow paths and/or a whole number of equipotential drops is rarely likely to be encountered.
- For fractional flow paths and fractional head drops the figures would not be squares. For a half flow path the figures would be rectangles with the length of the rectangle along the flow line twice that of the length along the direction normal to it since the latter represents the cross-sectional area of the flow path.

3.16.2 Unconfined Flow:

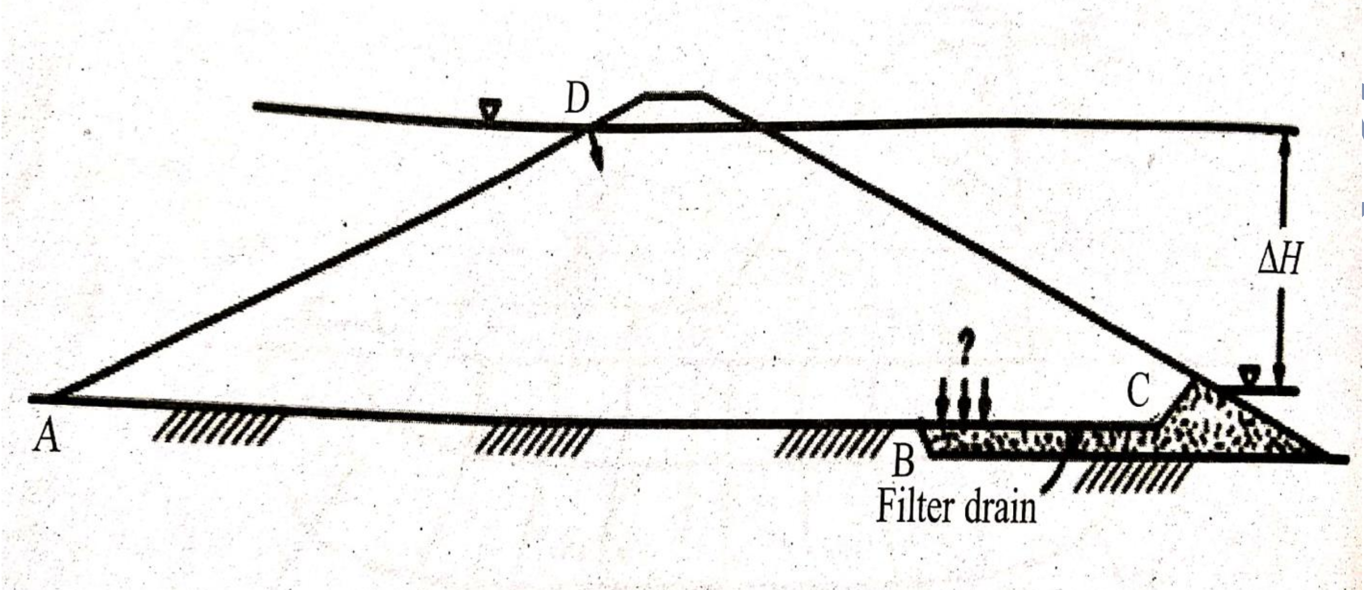


Fig. 3.24 : Homogeneous earth dam with a filter drain founded on an impervious stratum

- The flow occurs under gravity and there is a phreatic surface of flow which is open to atmosphere. Unconfined flow in which the top flow line in the flow through earth dam is the phreatic surface.
- There are numerous real situations in which some boundary conditions are not from the soil profile and therefore, the flow space is not completely defined; the flow in such space is then described as unconfined flow.
- One such situation, depicted in Figure, is that of flow through a homogeneous earth dam with a filter drain founded on an impervious stratum.

3.17 PHREATIC LINE OF AN EARTH DAM:

Let us consider the case of a homogeneous earth dam with a horizontal filter, as shown in Fig. (b). In order to draw the flow-net, it is first essential to find the location and shape of the phreatic line or the top flow line separating the saturated and unsaturated zones.

The phreatic line or seepage line is defined as “the line within a dam section below which there are positive hydrostatic pressures in the dam”. The hydrostatic pressure on the phreatic line itself is atmospheric. The phreatic line can be located by

3.17.1 Analytical Method

3.17.2 Graphical Method

3.17.3 Experimental Method

We shall first discuss the graphical method of determination of phreatic line suggested by Casagrande.

3.17.4 Casagrande Method:

- An analytically derived flow-net has been given by Kozeny (1931) for the case of water flowing above an impervious, infinite, horizontal plane which at a certain point A becomes permeable.
- Kozeny's solution consists of a family of confocal parabolas representing the flow lines and equipotential lines.
- The point A where the floor becomes permeable represents the focus for the parabolas.
- Casagrande (1940) established that the actual top flow line or the phreatic line in a homogeneous earth dam corresponds very closely to the solution of Kozeny, except at the entrance and exit ends, where it has to suit the boundary conditions.
- Kozeny's top flow line is called the basic or base parabola. Casagrande assumed the starting point F of the filter FE as the focus.

Procedure for Phreatic Line in an Earth Dam with Filter:

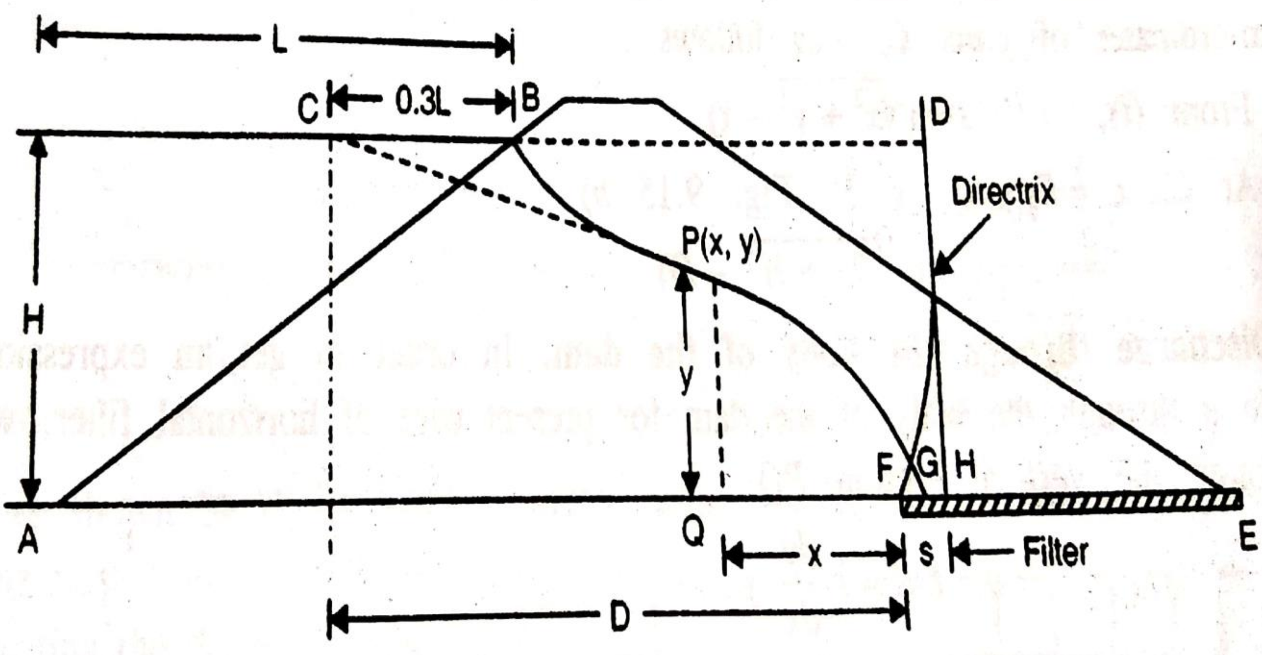


FIG. 9.15. (*b*) CASAGRANDE'S METHOD OF DETERMINING PHREATIC LINE IN A DAM WITH HORIZONTAL DRAINAGE FILTER.

- AB is the upstream face. Let its horizontal projection be L. On the water surface, measure a distance BC = 0.3 L. Then the point C is the starting point of the base parabola.
- To locate the directrix of parabola, we utilise the principle that any point on the parabola is equidistant from the focus as well as from the directrix. Hence with point C as the centre and CF as the radius, draw an arc to cut the horizontal line through CB in D. Draw a vertical tangent to the curve FD at D. Evidently, CD = CF. Hence the vertical line DH is the directrix.
- The last point G on the parabola will lie midway between F and H.
- In order to locate the intermediate points on the parabola we use the principle that its distance from the focus and directrix must be equal. For example, to locate any point P, draw vertical line QP at any distance x from F. Measure QH. With F as the centre and QH as the radius, draw an arc to cut vertical line through Q in point P.
- Join all these points to get the base parabola. However, some correction is to be made at the entry point. The phreatic line must start from B and not from C. Also, the phreatic line is a flow line, and must start perpendicularly to the U/S face AB which is a 100% equipotential line.
- Hence the portion of the phreatic line at B is sketched free hand in such a way that it starts perpendicularly to AB and meets the rest of the parabola tangentially without any kink. The base parabola should also meet the D/S filter perpendicularly at G.

Procedure for Phreatic Line in an Earth Dam with no Filter:

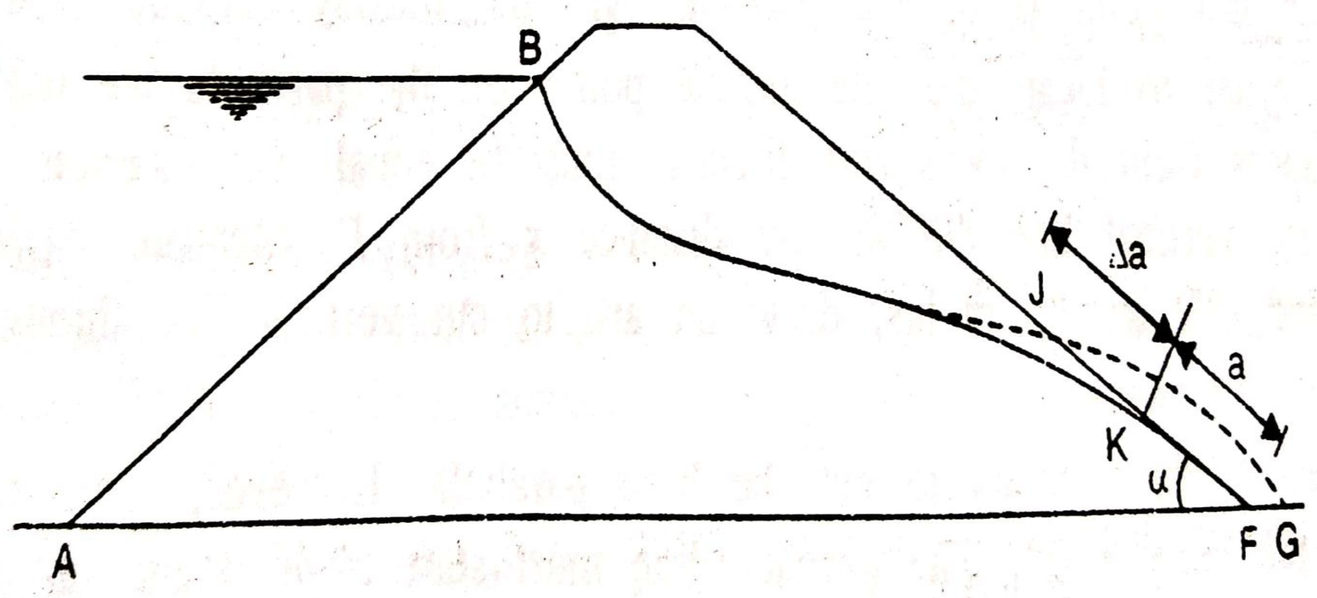


FIG. 9.16. DAM WITH NO DRAINAGE FILTER.

- Figure shows a homogenous earth dam with no horizontal drainage filter at the D/S side.
- The focus in this case will be lower point F of the D/S slope, and the base parabola BJG will evidently cut the D/S slope at J and extend beyond the limit of the dam, as shown by dotted line.
- However, according to exit conditions, the phreatic line must emerge out at some point K meeting the D/S face tangentially there. The portion KF is then known as the discharge face, and always remains wet.
- The correction Δ a, by which the parabola is to be shifted downwards, is found by the values of (Δa / a + Δa) given by Casagrande for various values of slope α of the discharge face.
- The slope α can even exceed the value of 90°, especially in the case of rockfill toe.

3.18 FLOW THROUGH EARTH DAM:

- The flow through an earth dam differs from the other cases in that the top flow line is not know in advance of sketching the flow net.
- Thus, it is case of unconfined flow. The determination of the top flow line will be dealt with in a later section.
- The top flow line as well as the flow-net will be dependent upon the nature of internal drainage for the earth dam. Typical cases are shown in Fig. 6.8; the top flow line only is shown.
- Assuming that the top flow line is determined, a typical flow net for an earth dam with a rock toe, resting on an impervious foundation is shown in Fig. 6.9.
- AB is known to be an equipotential and AD a flow line. BC is the top flow line at all points of this line the pressure head is zero.
- Thus BC is also the phreatic line or, on this line the total head is equal to the elevation head. Line CD is neither an equipotential nor a flow line, but the total head equals the elevation head at all the points of CD.

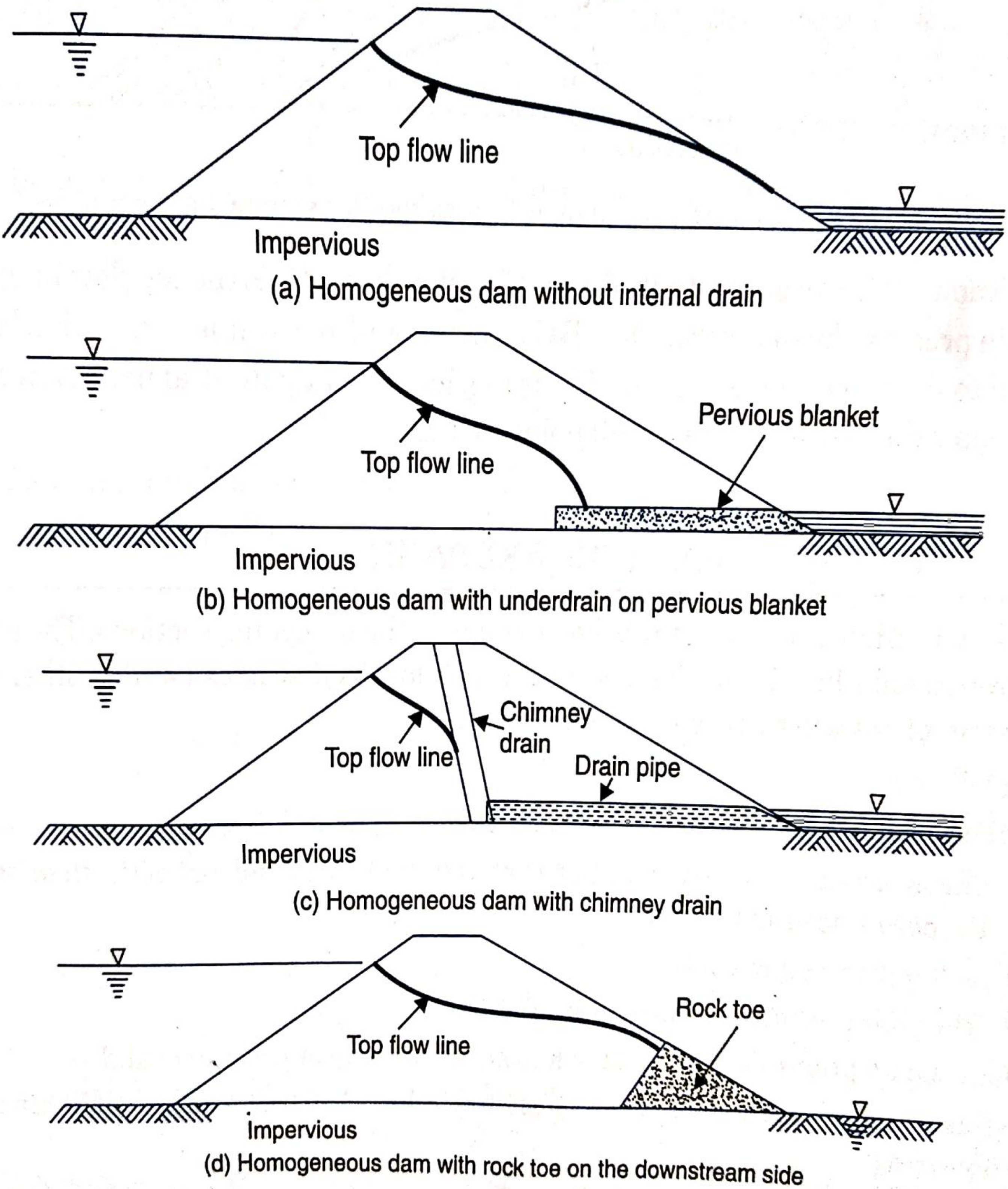


Fig. 6.8 Top flow line for typical cases of homogeneous earth dams on impervious foundation with different internal drainage arrangements

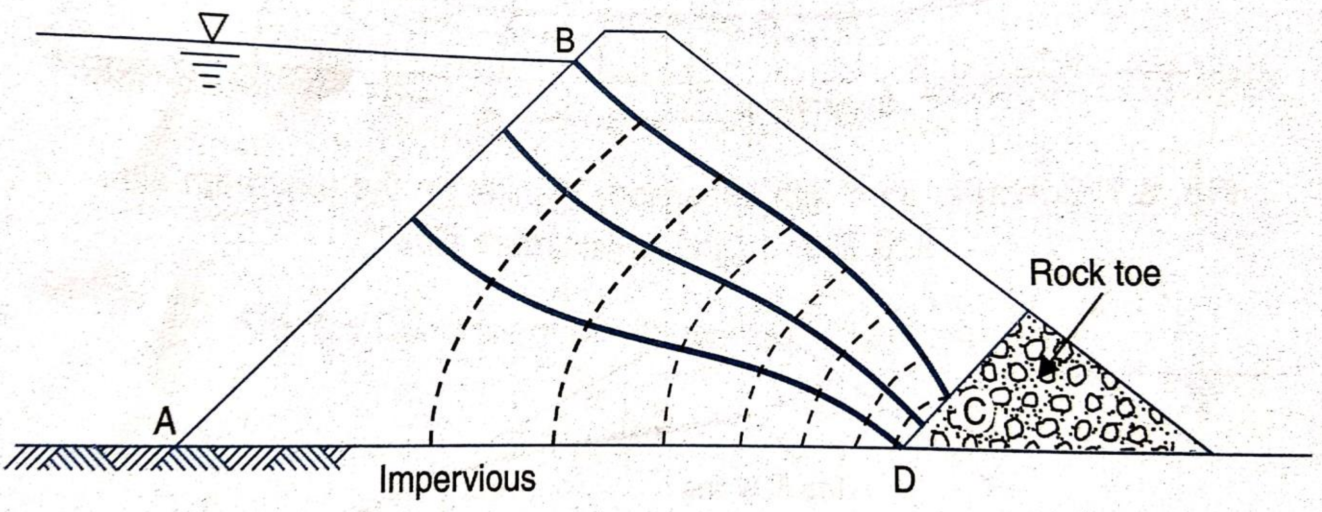


Fig. 6.9 Flow net for an earth dam with rock toe (for steady state seepage)

3.19 FILTER DESIGN:

- A protective filter is designed to provide for drainage of water from a body of soil without allowing movement of soil particles by lowering water.
- It can be a single layer filter or a multiple layer filter. In the latter case each subsequent layer will be increasingly coarser than the previous one and is sometimes referred to as reverse filter.
- The filter design for the drainage layers and internal zoning of a dam is a critical part of the embankment design.
- It is essential that the individual particles in the foundation and embankment are held in place and do not move as a result of seepage forces.
- This is accomplished by ensuring that the zones of material meet filter criteria with respect to adjacent materials.
- According to Terzaghi, the two requirements are satisfied if the gradation of filler material is such that:

$$(D_{15(filter)} / D_{85(protected\ soil)}) < 4 \text{ to } 5$$

$$(D_{15(filter)} / D_{15(protected\ soil)}) > 4 \text{ to } 5$$

- The US crops of Engineers have recommended that

$$(D_{50(filter)} / D_{50(protected\ soil)}) \leq 25$$

- The soil protected by filter is referred to as base material. Fillets are usually multi-layered and each layer should satisfy the requirements with respect to proceeding layers.
- A graded filter consists of layers of pervious material which permit flow of water but prevent the movement of soil particles.
- The soil particles in a particular layer are coarser than that in the preceding layer. However, the difference of sizes of the particles in two layers should not be excessive otherwise the particles of the preceding layer will be carried into the next layer.

The particle sizes of different layers are fixed according to the design criteria given below:

- The filter material should be coarse enough so that the percolating water moves easily without any build-up of water pressure in the filter.
- The filler material should be fine enough that the soil particles of the base material are not washed through the filter.
- The material of the last layer should be coarse enough not to be carried away through the openings of the perforated drainage pipes, if provided.
- The grain size curve of the filter material should be roughly parallel to that of the base material.
- To avoid segregation, filter should not contain the particles of size larger than 75 mm.
- For proper working, the filter material should not contain more than 5% of the fines passing 75 micron IS Sieve.
- The thickness and area of the filter should be sufficient to carry the seepage discharge safely.

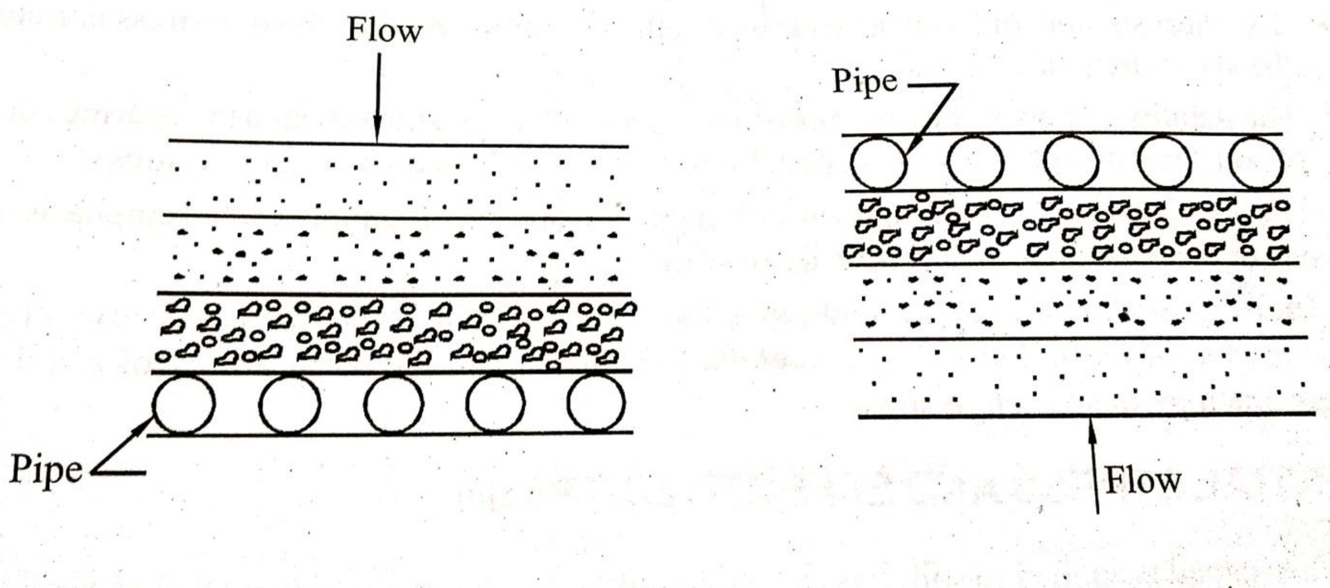


Fig. 3.34 : Graded filter

MODULE – 3

EFFECTIVE STRESS ANALYSIS

3.20 INTRODUCTION:

Terzaghi (1925) developed the effective stress concept, which became a key concept in modern soil mechanics. Effective stress in soil contributes to its strength and volume change. It also influences the capillary rise, seepage force due to water flow, quick sand and heaving at the bottom of the excavation.

3.21 IMPORTANCE OF EFFECTIVE STRESS:

- Compression and shear strength of soil are dependent on the effective stress.
- As the effective stress in a soil increases, the compression of the soil occurs. This causes settlement of structures built on soils.
- The shear strength of a soil depends on its effective stress. As the effective stress is changed, the shear strength changes.
- The stability of slopes, the earth pressures against retaining structures and the bearing capacity of soils depends upon the shear strength of the soil and hence, the effective stress.
- The importance of shear strength in soil engineering problem cannot be over emphasised. It is one of the most important properties of soils.

3.22 EFFECTIVE AND NEUTRAL PRESSURES OR STRESSES:

- At any plane in a soil mass, the total stress or unit pressure σ is the total load per unit area. This pressure may be due to (i) self-weight of soil and (ii) over-burden on the soil.
- The total pressure consists of two components: inter granular pressure or effective pressure and the neutral pressure or pore pressure.
- Effective pressure σ' is the pressure transmitted from particle through their point of contact through the soil mass above the plane. Such a pressure, also termed as inter granular pressure, is effective in decreasing the voids ratio of the soil mass and in mobilising its shear strength.
- The neutral pressure or the pore water pressure or pore pressure is the pressure transmitted through the pore fluid. This pressure equal to water load per unit area above the plane, does not have any measurable influence on the voids ratio or any other mechanical property of the soil, such as shearing resistance. Therefore, this pressure is also called as neutral pressure u.

- Since the total pressure at any plane is equal to the sum of the effective pressure and the pore pressure, we have

$$\sigma = \sigma' + u$$

- At any plane, the pore pressure is equal to piezometric head h_w times the unit weight of water i.e. $u = h_w \times \gamma_w$.

3.23 QUICK SAND PHENOMENA:

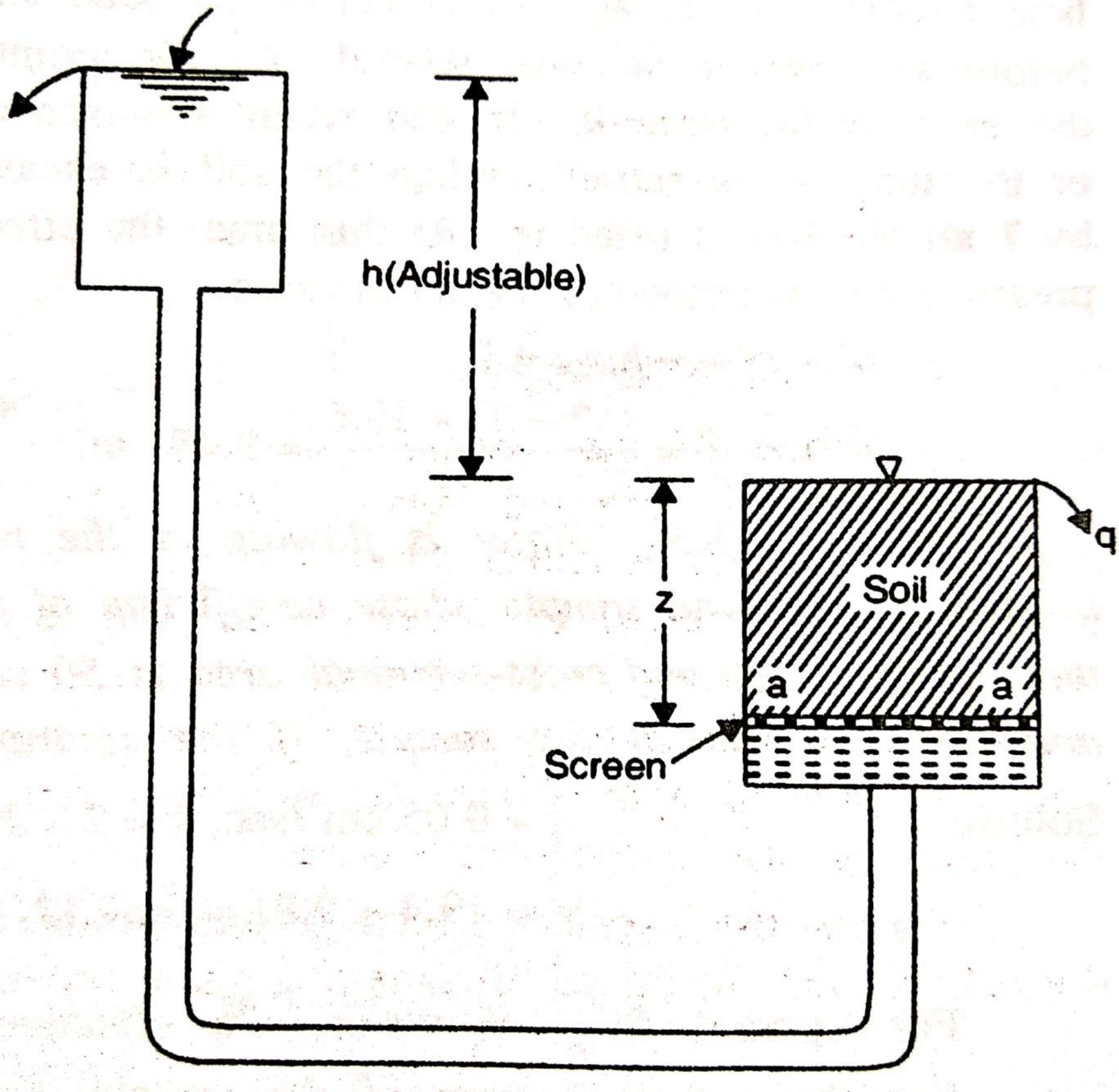


FIG. 9.2. QUICK SAND CONDITION.

- When flow takes place in an upward direction, the seepage pressure also acts in the upward direction and the effective pressure is reduced.
- If the seepage pressure becomes equal to the pressure due to submerged weight of soil, the effective pressure is reduced to zero.
- In such a case, cohesionless soil losses all its shear strength and the soil particles have a tendency to move up in the direction of flow. This phenomenon of lifting of soil particles is called quick condition or boiling condition or quick sand.
- Thus, during the quick condition:

$$\sigma' = z\gamma' - p_s = 0$$

$$p_s = z\gamma' \text{ or } iz\gamma_w = z\gamma'$$

$$i = i_c = (\gamma' / \gamma_w)$$

- The hydraulic gradient at such a critical state is called the critical hydraulic gradient.
- For loose deposit of sand or slit, if voids ratio e is taken as 0.67 and G as 2.67, the critical hydraulic gradient works out to be unity.
- It should be noted that quick sand is not a type of sand but a flow condition occurring within a cohesionless soil when its effective pressure is reduced to zero due to upward flow of water.
- Figure shows a set up to demonstrate the phenomenon of quick sand. Water flows in an upward direction through a saturated soil sample of thickness z under a hydraulic head h.
- This head can be increased or decreased by moving the supply tank in the upward or downward direction.
- When the soil particles are in the state of critical equilibrium, the total upward force at the bottom of the soil becomes equal to the total weight of all the materials above the surface considered.

MODULE – 4

CONSOLIDATION OF SOIL

4.1 INTRODUCTION:

When a compressive load is applied to a soil mass, a decrease in its volume takes place. The decrease in the volume of soil mass under stress is known as compression and the property of soil mass pertaining to its susceptibility to decrease in volume under pressure is known as compressibility. Soils are composed of small solid particles not bonded together, except by the small van der waals force and adsorbed double layer water. When a stress is applied to it, the elastic deformation of solid particles is negligible small compared to the deformation caused by change in relative position of the discrete particles and the resulting decrease in the volume of voids. The rate at which this deformation can take place depends upon the permeability of soil and upon the distance the water must travel to reach a drainage surface. The compressibility of clays may also be caused by three factors (i) the expulsion of double layer water from between the grains, (ii) slipping of the particles to new positions of greater density, (iii) bending of particles as elastic sheets. The permeability of clay being very small, time is an important factor in the consolidation of clays.

4.2 DEFINITATION:

The process of gradual compression due to the expulsion of pore water under steady pressure is referred as consolidation.

Or

According to Terzaghi, "every process involving a decrease in the water content of a saturated soil without replacement of the water by air is called a process of consolidation.

Or

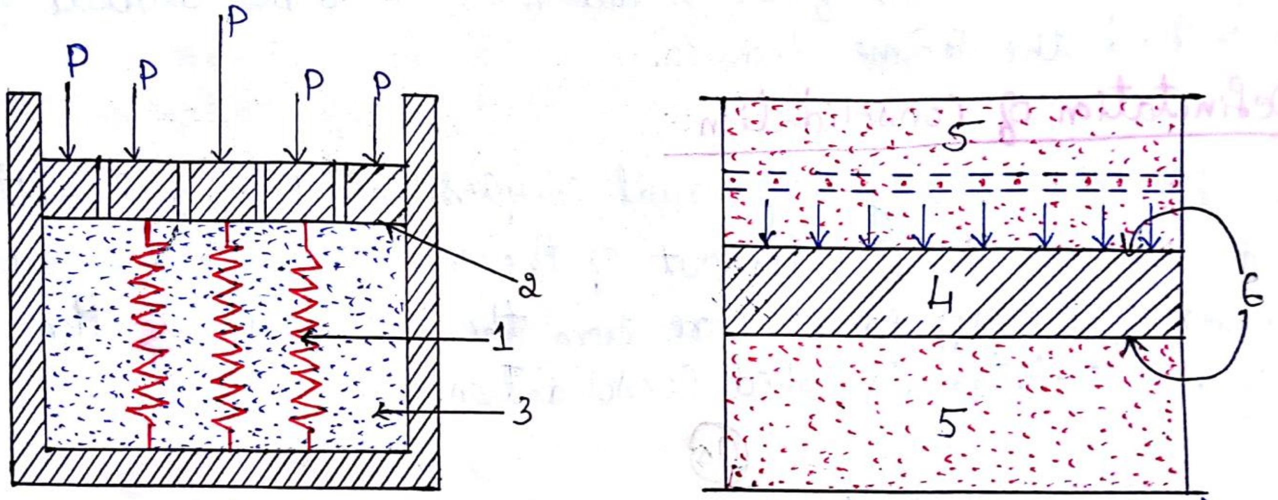
The decrease in soil volume by the squeezing out of the pore water on account of gradual dissipation of excess hydrostatic pressure induced by an imposed total stress is defined as consolidation.

Or

In a saturated soil mass having its voids filled with incompressible water, decrease in volume or compression can takes place when water is expelled out of the voids. Such compression resulting from a long term static load and the consequent escape of pore water is termed as consolidation.

4.3 MASS – SPRING ANALOGY:

It is also termed as Principle of Consolidation or Mechanics of Consolidation or The Consolidation Process.



Spring Analogy

Clay layer sandwiched b/w sand layer

1 ⟶ Spring

2 ⟶ Piston

3 ⟶ Pore water

4 ⟶ Soft Clay layer

5 ⟶ Sand

6 ⟶ Drainage faces

- The process of consolidation of clay – soil – water system may be explained with the help of a spring analogy model as described by Terzaghi and Frohlich.
- The model consists of a cylinder with a frictionless piston.
- The piston is supported on one or more helical metallic springs.
- The space underneath the piston is completely filled with water.
- The springs represent the mineral skeleton in the actual soil mass and the water below the piston is the pore water under saturated conditions in the soil mass.

- When a load per unit area is applied on the piston, this stress is fully transferred to the water and the water pressure increases.
- The pressure in the water is U = P.
- This is analogous to pore water pressure U that would be developed in clay – water system under external pressures.
- If the whole model is leak proof without any holes in the piston, there is no chance for the water to escape.
- Such a condition represents highly impermeable clay – water system.
- If a few holes are made in the piston, the water will immediately escape through holes.
- With the escape of water through the holes a part of the load carried by the water is transferred to the springs.
- This process of transference of load from water to spring goes on until the flow stops when all the load will be carried by the spring and none of water.
- The time required to attain this condition depends upon the number and size of the holes made in the piston.
- A few small holes represent a clay soil with poor drainage characteristics.
- After the spring – water system attains equilibrium condition under the imposed load, the settlement of the piston is analogous to the compression of the clay – water system under external pressure.

4.4 TERZAGHI'S THEORY OF ONE DIMENSIONAL CONSOLIDATION:

4.4.1 Assumptions:

- The soil is homogeneous and fully saturated.
- Soil particles and water are incompressible.
- The deformation of the soil is due entirely to change in volume.
- Dracy's law for the velocity of flow of water through soil is perfectly valid.
- Coefficient of permeability is constant during consolidation.
- Load is applied in one direction only and deformation occurs only in the direction of the load application, i.e. the soil is restrained against lateral deformation.
- Excess pore water drains out only in the vertical direction.
- The boundary is a free surface offering no resistance to the flow of water from the soil.
- The change in thickness of the layer during consolidation is insignificant.
- The time lag in consolidation is due entirely to the permeability of soil.

4.4.2 Limitations:

- Flow is assumed to be one dimensional but is reality flow takes place in three dimensional.
- Application of external load is assumed to produce excess pore water pressure over entire soil stratum but in some cases the excess pore water pressure does not develop over entire clay stratum.
- In reality flow may not be laminar i.e. it may be turbulent hence Dracy's law is not valid.
- Soil may not get completely saturated.

4.5 PRE – CONSOLIDATION PRESSURE:

The temporary overburden pressure to which a soil has been subjected and under which it got consolidated is known as pre – consolidation pressure.

4.5.1 Determination of Pre – Consolidation Pressure by Casagrande's Method:

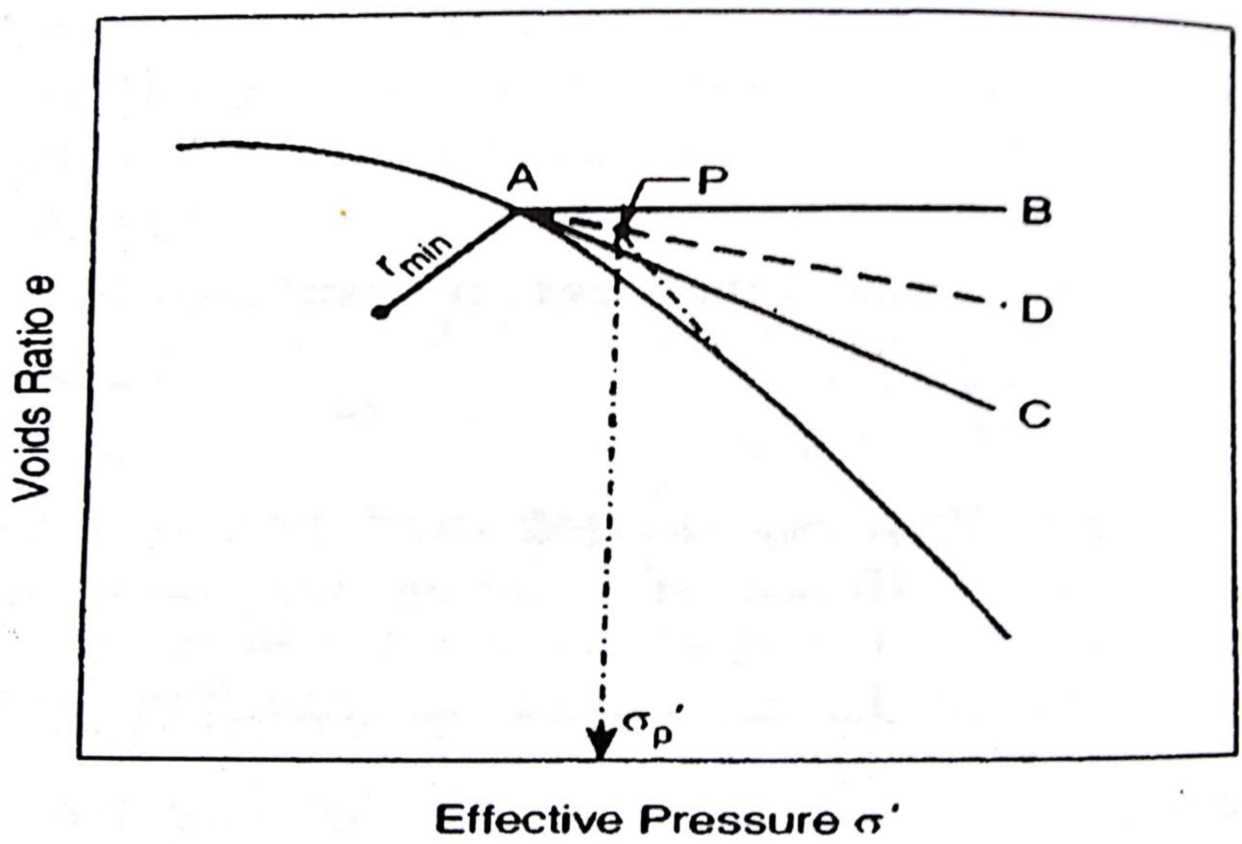


FIG. 15.4. PRE-CONSOLIDATION PRESSURE.

Procedure:

- To find the preconsolidation pressure, an undisturbed sample of clay is consolidated in the laboratory and the pressure voids ratio relationship is plotted on a semi-log plot as shown in Fig. 15.4.
- The initial portion of the curve is flat and resembles the recompression curve is flat and resembles the recompression curve of a remoulded specimen.
- The lower portion of the curve, which is a straight line, is the laboratory virgin curve.
- The approximate value of the preconsolidation pressure may be determined by the following empirical method of A.
- The point A of maximum curvature is selected and horizontal line AB is drawn.
- A tangent AC is drawn to the curve and the bisector AD, bisecting angle BAC is drawn.
- The straight portion of the virgin curve is extended back to meet the bisector AD in P.
- The point P corresponds to the preconsolidation pressure.

4.6 IMPORTANT DEFINITATION:

4.6.1 Normally Consolidated Soil: soil is one which has never been subjected to an effective pressure greater than the existing overburden pressure and which is also completely consolidated by the existing overburden.

4.6.2 Under Consolidated Soil: a soil which is not fully consolidated under the existing overburden pressure is called an under consolidated soil.

4.6.3 Over Consolidated Soil: a soil is said to be over consolidated, if it has ever been subjected to a pressure in excess of its present overburden pressure.

4.6.4 Primary Consolidation (S_C): the squeezing out of pore water from a loaded saturated soil causing a time – dependent decrease in volume.

4.6.5 Secondary Consolidation (S_S): even after the reduction of all excess hydrostatic pressure to zero, some compression of soil takes place at a very slow rate. This is known as secondary consolidation.

4.6.6 Immediate Settlement (S_E): this is caused by the elastic deformation of dry soil and of moist and saturated soils without any change in the moisture content. This is known as elastic or immediate settlement.

4.6.7 Total Settlement (S_T): is given as S_T = Primary Consolidation Settlement (S_C) + Secondary Consolidation Settlement (S_S) + Immediate Settlement (S_E).

4.6.8 Compression Index (C_c): the compression index represents the slope of the linear portion of the pressure voids ratio curve and remains constant within a fairly large range of pressure.

$$C_c = ((e_o - e) / \log_{10}(\sigma' / \sigma_o')) = (\Delta e / \Delta \log_{10} \sigma')$$

4.6.9 Coefficient of Compressibility (a_v): is defined as the decrease in voids ratio per unit increase of pressure.

$$a_v = (e_o - e) / (\sigma' - \sigma_o') = -\Delta e / \Delta\sigma'$$

4.6.10 Coefficient of Volume Change (m_v): is defined as the change in volume of a soil per unit of initial volume due to a given unit increase in pressure.

$$m_v = -((\Delta e / 1 + e_o) \times (1 / \Delta\sigma'))$$

4.6.11 Coefficient of Consolidation (C_v): it is defined as the ratio between the coefficient of permeability and the product of coefficient of volume change with unit weight of water.

$$C_v = (K_z (1 + e_o) / (a_v \times \gamma_w))$$

The coefficient of consolidation is the soil parameter governing the time rate of consolidation.

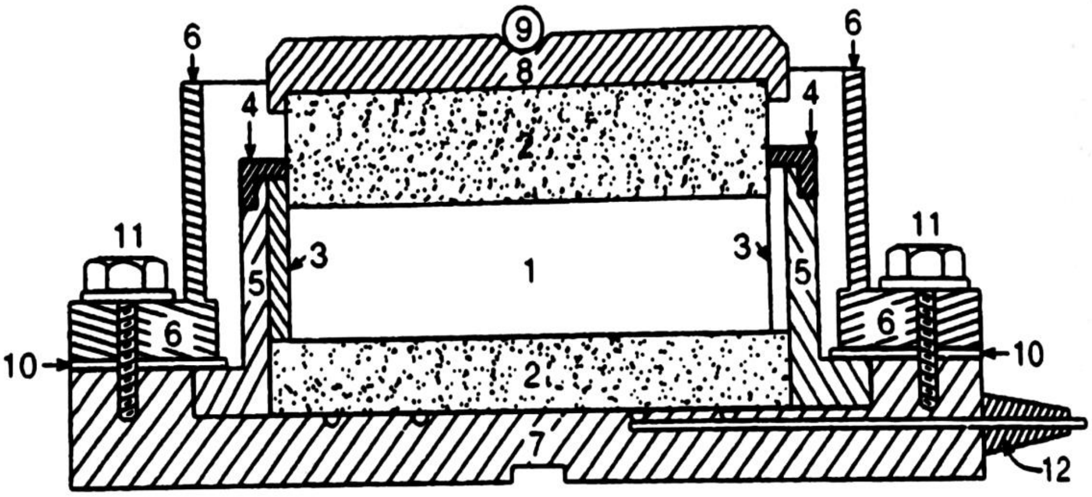
4.7 LABORATORY ONE DIMNESIONAL CONSOLIDATION TEST:

The laboratory consolidation test is conducted with an apparatus known as Consolidometer or Oedometer consisting essentially of a loading frame and consolidation cell in which the specimen is kept. Porous stones are put on the top bottom ends of the specimen. It is conducted by using Fixed ring cell and Floating ring cell methods.

Procedure:

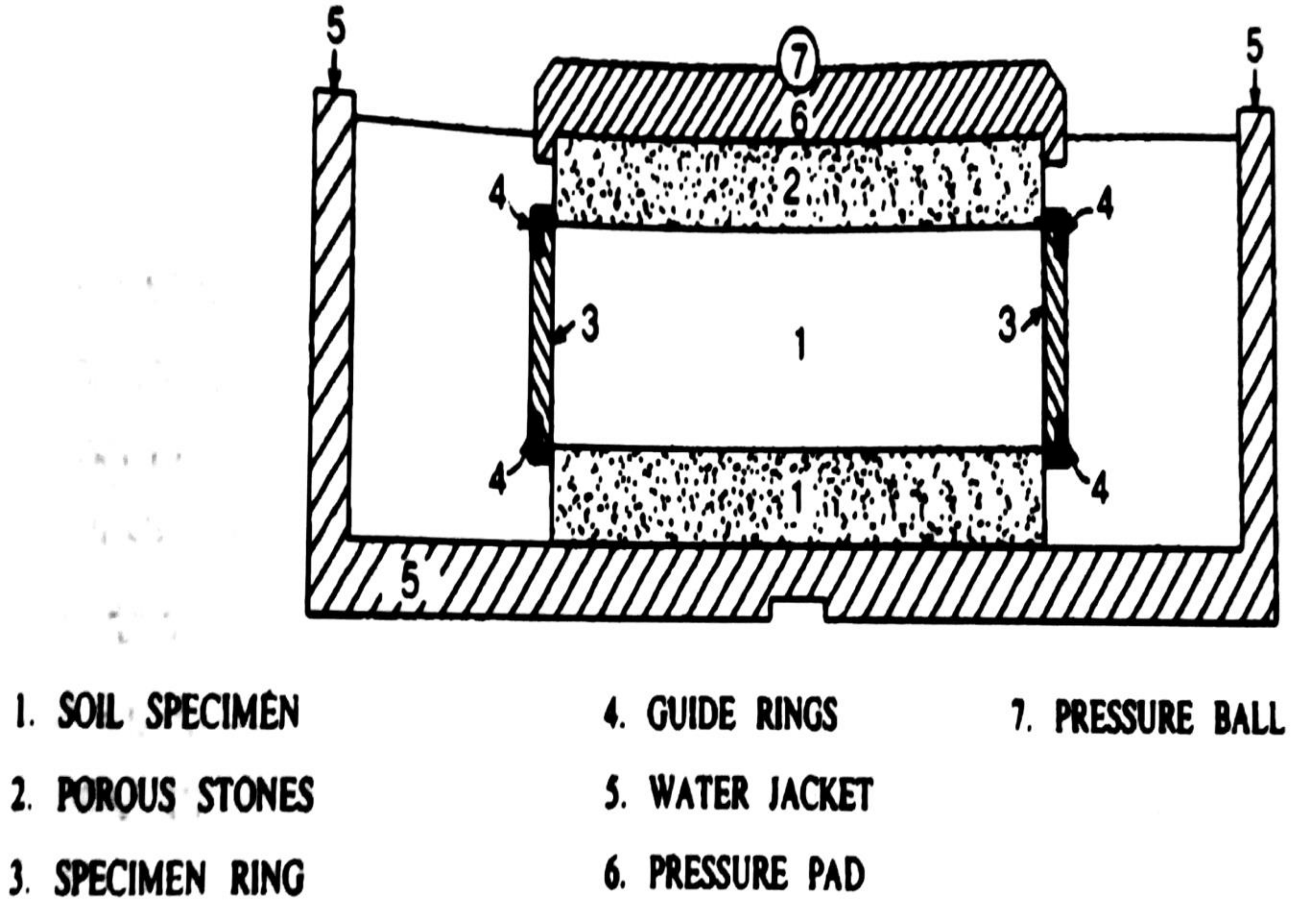
- In the fixed ring cell, only the top porous stone is permitted to move downward as the specimen compresses.
- In the floating ring cell, both top and bottom porous stones are free to compress the specimen towards the middle.
- Direct measurement of permeability of the specimen at any stage of loading can be made only in the fixed ring type.

- However, the floating ring cell has the advantage of having smaller effects of friction between the specimen ring and the soil specimen.
- The loading machine is usually capable of applying steady vertical pressure upto 800 or 1000 kN/m^2 to the soil sample.
- During the test, the specimen is allowed to consolidate under a number of increments of vertical pressure, such as 10, 20, 50, 100, 200, 400, 800, 1000 kN/m^2 and each pressure increment is maintained constant until the compression virtually ceases, generally 24 hours.
- The vertical compression of the specimen is measured by means of a dial gauge.
- Dial gauge readings are taken after the application of each pressure increment at the following total elapsed times: 0.25, 1.00, 2.25, 4.00, 6.25, 9.00, 12.25, 16.00, 20.25, 25, 36, 49, 60 minutes and 2, 4, 8 and 24 hours.
- The dial gauge readings showing the final compression under each pressure increment are also recorded.
- After the completion of consolidation under the desired maximum vertical pressure, the specimen is unloaded and allowed to swell.
- The final readings corresponding to the completion of swelling is recorded and the specimen is taken out, and dried to determine its water content and the weight of the soil solids.



1. SOIL SPECIMEN
2. POROUS STONES
3. SPECIMEN RING
4. GUIDE RING
5. OUTER RING
6. WATER JACKET
7. BASE
8. PRESSURE PAD
9. PRESSURE BALL
10. RUBBER GASKET
11. BOLTS
12. DRAIN TUBE.

FIG. 15.7. FIXED RING CONSOLIDATION RING.



4.7 DETERMINATION OF COEFFICIENT OF CONSOILDATION:

The coefficient of consolidation Cv can be determined by compacting the characteristics of the theoretical relationship between Tv and U to the relationship between elapsed time t and degree of consolidation of the specimen obtained in the laboratory. Out of the many methods available, the following two methods will be described here:

4.7.1 Square root of time fitting method

4.7.2 Logarithm of time fitting method

4.7.1 Square Root of Time Fitting Method: Fig. 15.9 shows a theoretical characteristic curve between $\sqrt{T_v}$ and U. The curve is straight up to U = 60% and the abscissa at U = 90% is equal to 1.15 times the abscissa at U = 60%. The use of this characteristic of the theoretical curve determine the 90% U point on a laboratory time consolidation curve was suggested by Taylor (1948).

Procedure:

- The method consists of drawing the curve between root of time √t as abscissa and the dial reading R, representing the compression of the specimen, as ordinate for any pressure increment in the consolidation test.
- Fig. 15.10 shows such a plot drawn for test data.
- The initial dial reading R_0 corresponds to the time t = 0 and U = 0.
- The straight portion (line A) is produced back to meet the ordinate at reading R_c, which is called the corrected zero reading and the consolidation between R_0 and R_c is called initial consolidation.
- From R_c, another line B is so drawn that is abscissa at every point is 1.15 times that of line A.
- The intersection of line B with the consolidation curve gives a point P corresponding to 90% U whose dial reading and time may be designated as R_{90} and t_{90} respectively.
- The coefficient of consolidation is calculated from the equation:

$$C_v = ((T_v)_{90} \times d^2 / t_{90})$$

$$C_v = (0.848 \times d^2 / t_{90})$$

Where, d = average drainage path for the pressure increment

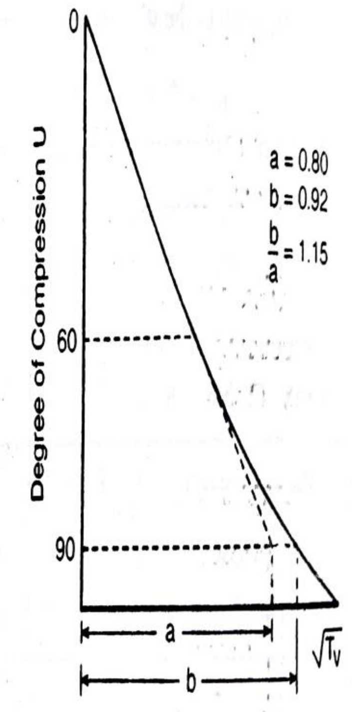


FIG. 15.9. THEORETICAL CURVE BETWEEN $\sqrt{T_v}$ and U.

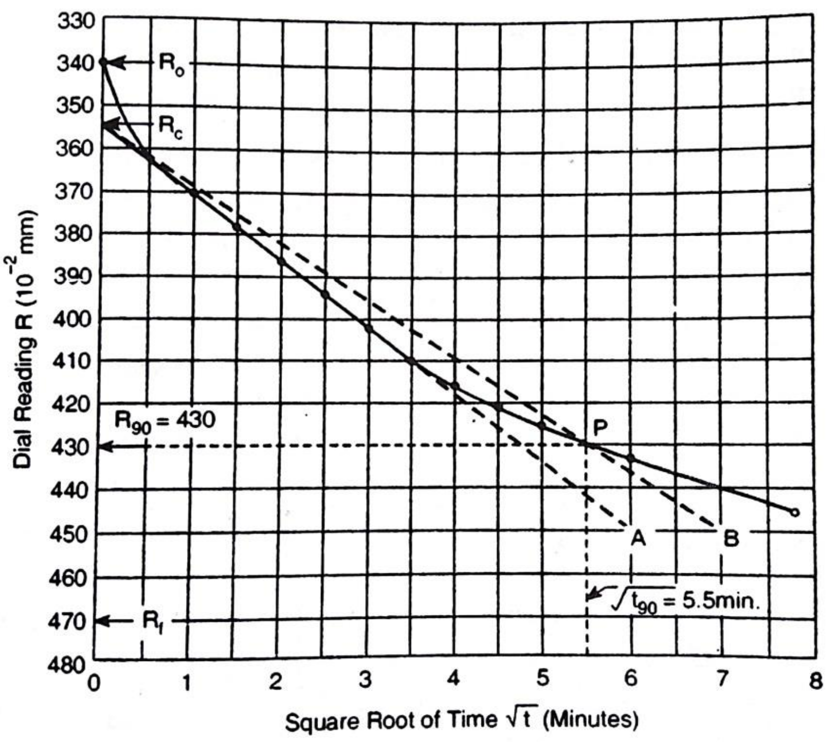


FIG. 15.10. TIME-CONSOLIDATION CURVE (TABLE 15.5).

4.7.2 Logarithm of Time Fitting Method: this method suggested by A. Casagrande (1930), is based on the characteristic of U – $\mathrm{Log}_{10}\ T_v$ plot in which intersection of a tangent at the point of inflection and the asymptote of the lower portion is at the ordinate of 100% U. This characteristic is used to determine the point of 100% U on the semi log plot of laboratory time consolidation curve. Fig. 15.11 shows the theoretical curve while Fig. 15.12 shows the plotting of the test data.

Procedure:

- The corrected zero reading R_c in the laboratory curve is obtained on the assumption that the curve portion at the beginning of the graph (on semi-log plot) is parabola.
- A time t_1 = 1 minute is selected and its corresponding point A is marked on the curve.
- Another point B is so selected on the curve that its corresponding time is ¼ minute.
- A horizontal line is then drawn at a vertical height z above B, where z is the vertical height between A and B.
- The ordinate corresponding to this horizontal line is the corrected reading R_c corresponding to U =0.
- The dial reading R_{100} corresponding to U = 100% is given by extending the straight portion of the curve to meet the point P which is the point of 100% consolidation.
- The consolidation from R_c to R_{100} is the primary consolidation while the consolidation from R_{100} to R_f is the secondary consolidation.
- After locating R_c and R_{100}, the dial reading R_{50}, and hence t_{50}, corresponding to 50% U can be found out from the plot.
- The coefficient of consolidation corresponding to 50% U is obtained from the relation:

$$C_v = ((T_v)_{50} \times d^2 / t_{50})$$

$$C_v = (0.197 \times d^2 / t_{50})$$

Where, d = average drainage path for the pressure increment

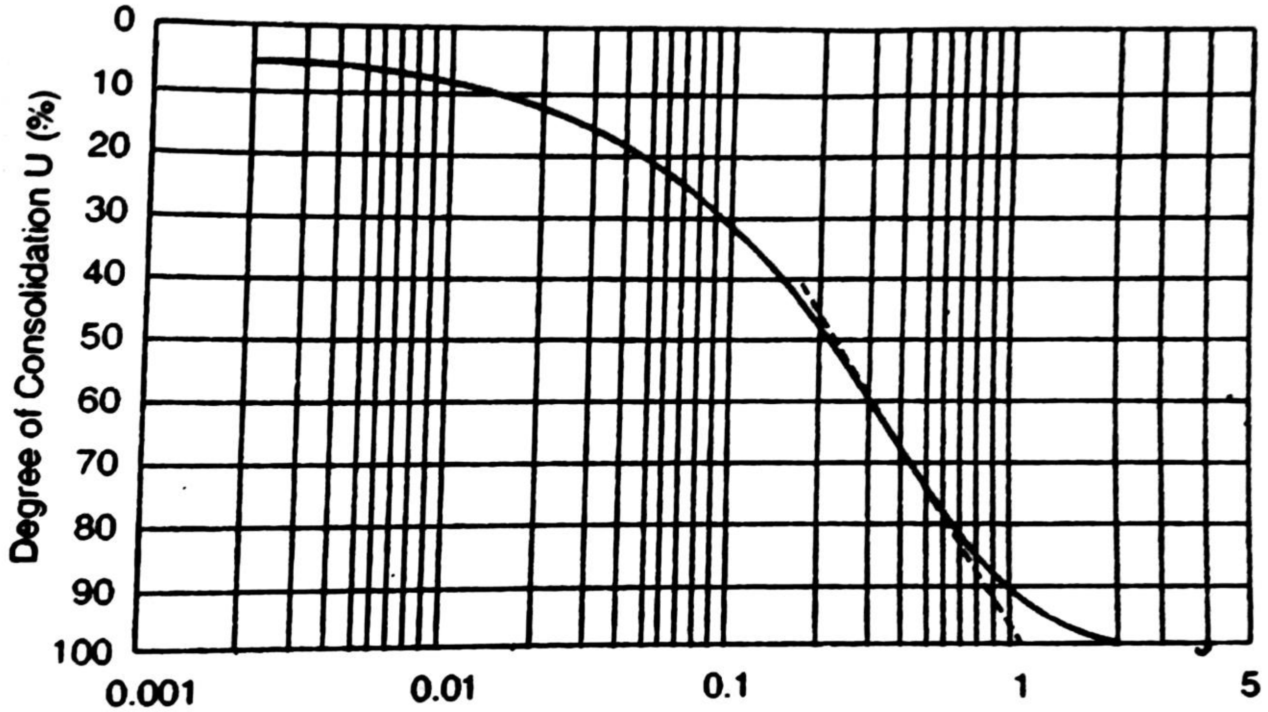


FIG. 15.11. THEORETICAL CURVE LOG T_v AND U (REF. TABLE 15.1).

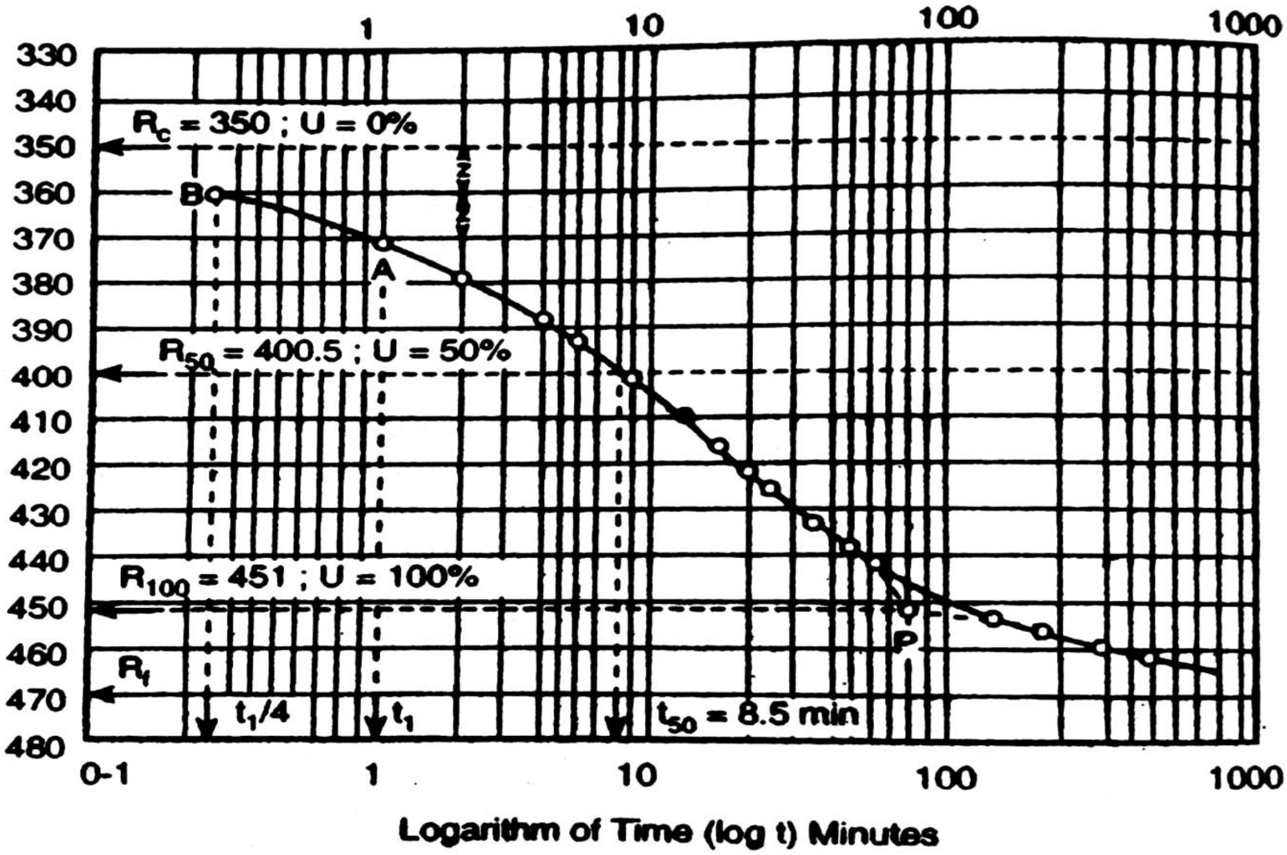
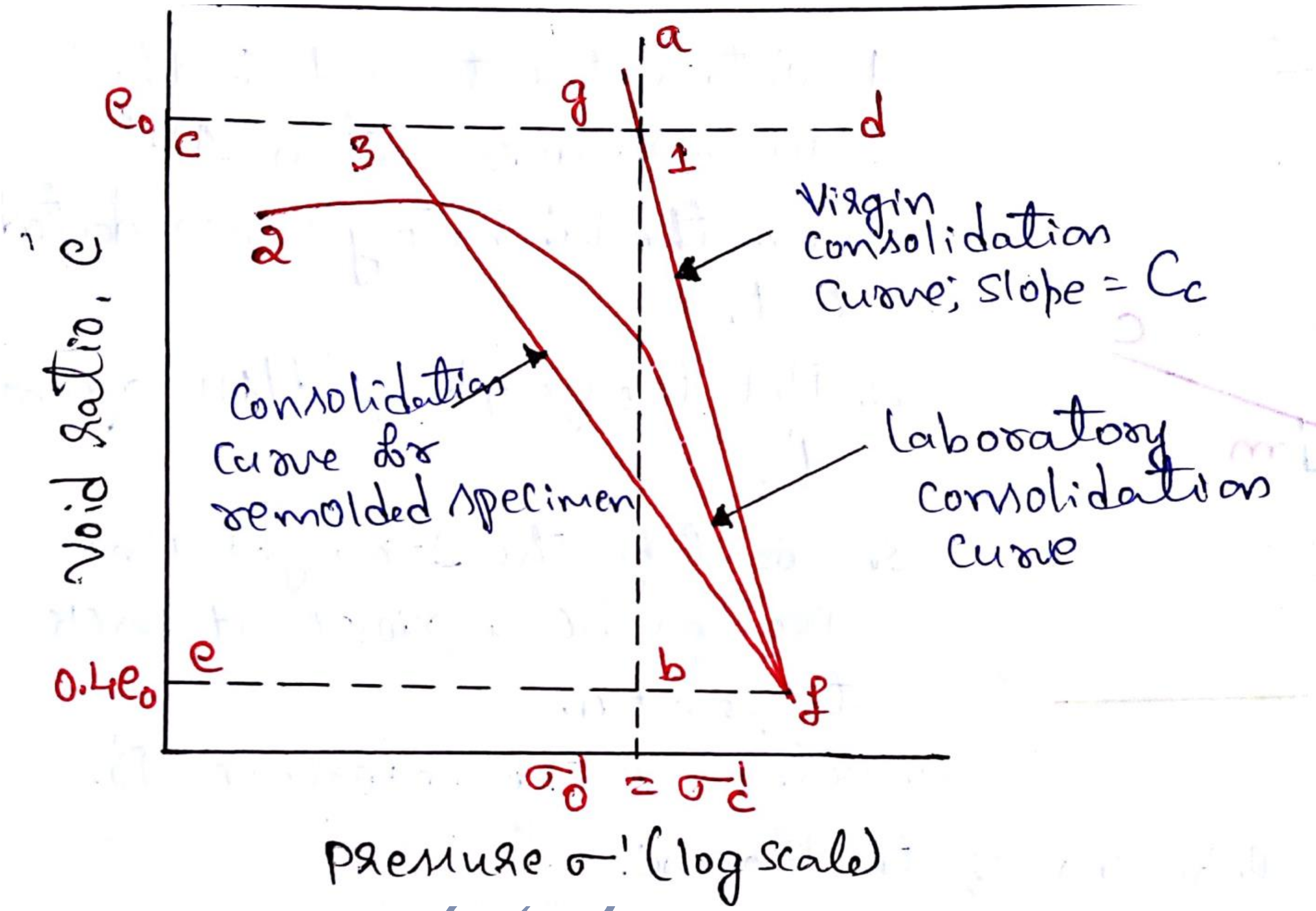


FIG. 15.12. TIME CONSOLIDATION CURVE (TABLE 15.5).

4.8 DETERMINATION OF COMPRESSION INDEX (C_c):



Procedure:

- If a remoulded soil is laterally confined in a consolidometer, consisting of a metal ring, and porous stones are placed both at its top and bottom faces, the compression or consolidation of soil sample takes place under a vertical pressure applied on the top of porous stone.
- The porous stones provided free drainage of water and air from or into the soil sample.
- Under a given applied pressure, a final settlement and equilibrium voids ratio is attained after certain time.
- At the equilibrium stage, the applied pressure naturally becomes the effective pressure on the soil.
- Thus a relationship can be obtained between the effective pressure and the equilibrium voids ratio in the form of curve.
- Skempton (1944) suggested the following empirical expression for the compression index for undisturbed clay:

$$C_c = 0.009\,(W_L - 10).$$

MODULE – 5

SHEAR STRENGTH OF SOIL

5.1 INTRODUCTION:

When soil is loaded, shearing stresses are induced in it. When the shearing stresses reach a limiting value, shear deformation takes place, leading to the failure of the soil mass. The failure may be in the form of sinking of a footing, or movement of a wedge of soil behind a retaining wall forcing it to move out, or the slide in an earth embankment. The shear strength of soil is the resistance to deformation by continuous shear displacement of soil particles or on masses upon the action of a shear stress. All stability analysis in soil mechanics involves a basic knowledge of the shearing properties and shearing resistance of the soil. The shearing resistance of soil is constituted basically of the following components:

(i) The structural resistance to displacement of the soil because of the interlocking of the particles.

(ii) The frictional resistance to translocation between the individual soil particles at their contact points.

(iii) Cohesion or adhesion between the surface of the soil particles.

5.2 THEORETICAL CONSIDERATION: MOHR'S STRESS CIRCLE

- Through a point in a loaded soil mass, innumerable planes pass and stress components on each plane depends upon the direction of the plane.
- It can be shown that there exist three typical planes, mutually orthogonal to each other, on which the stress is wholly normal and no shear stress acts.
- These planes are called the principal planes and the normal stresses acting on these planes are called the principal stresses.
- In the order of decreasing magnitude of the normal stress, these planes are called major, intermediate and minor principal planes and the corresponding normal stresses on them are called major principal stress σ_1, intermediate principal stress σ_2 and minor principal stress σ_3.
- Many problems in soil engineering can be approximated by considering two dimensional stress conditions.
- Fig. 18.1 (a) shows a soil element subjected to two dimensional stress system.

- From the consideration of the equilibrium of the element, one gets the following expressions for the normal stress σ and shearing stress τ on any plane MN inclined at an angle of α with the x direction:

$$\sigma = (\sigma_y + \sigma_x) / 2 + ((\sigma_y - \sigma_x) / 2) \cos 2\alpha + \tau_{xy} \sin 2\alpha$$

$$\tau = ((\sigma_y - \sigma_x) / 2) \sin 2\alpha - \tau_{xy} \cos 2\alpha$$

- Where σ_y and σ_x = normal stresses on planes perpendicular to y and x axes, respectively ($\sigma_y > \sigma_x$).
- Squaring the above two equations and adding, we get the following results:

$$(\sigma - ((\sigma_y + \sigma_x) / 2)^2 + \tau^2 = ((\sigma_y - \sigma_x) / 2)^2 + \tau^2_{xy}$$

- The co-ordinates of points on the circle represent the normal and shearing stresses on inclined planes at a given point. This circle is known as Mohr's circle of stress.
- To draw the Mohr circle, the normal stresses σ_x and σ_y are marked on the abscissa, at points B and A and a circle is drawn with point C, mid-way between A and B, as the centre, with radius equal to $CB_1 = CA_1$ where BB_1 and AA_1 are perpendiculars drawn at B and A of magnitude equal to τ_{xy}.
- The sign conventions are shown in Fig. 18.1 (b).
- Fig. 18.1 (c) shows the Mohr circle so drawn. The co-ordinates of any point F (σ, τ) represent the stress conditions on plane which makes an angle α with the x direction.
- If from a point B_1 [Fig. 18.1 (d)] on a circle representing the state of stress on vertical plane, a line is drawn parallel to this plane (i.e. vertical), it intersects the circle at a point P.
- Also, if from the point A1 on the circle representing the stresses on the horizontal plane, a line is drawn parallel to this latter plane (i.e. horizontal) it will also intersect the circle in the same plane P.
- In general, if through a point F representing the stresses on a given plane, a line is drawn parallel to that plane, it will also intersect the circle in the point P.
- The point P is therefore, a unique point called the origin of planes or the pole.
- Let us now take the case of soil element whose sides are the principal planes, i.e. consider the state of stress where only normal stresses are acting on the faces of the element.
- Fig. 18.2 (a) shows the element, and Fig. 18.2 (b) shows the Mohr circle.
- In Fig. 18.2 (a) the major principal plane is horizontal. Hence the pole P is located by drawing a horizontal line through point A [Fig. 18.2 (b)] representing the major principal stress σ_1. This intersects the circle at B.

- If a line PF is drawn through P at an angle α with the horizontal, it will intersect the circle at F which represents the stress conditions on a plane inclined at angle of α with the direction of the major principal plane.
- Fig. 18.2 (c) shows an element in which the principal planes are not horizontal and vertical, but are inclined to y and x directions.
- Fig. 18.2 (d) shows the corresponding stress circle. Point A represents the major principal stress (σ_1, 0) and B represents the minor principal stress (σ_3, 0).
- Hence to get the position of the pole, a line is drawn through A, parallel to the major principal plane, to intersect the circle in P. Evidently, PB gives the direction of minor principal plane.
- To find the stress components on any plane MN inclined at an angle of α with the major principal plane, a line is drawn through P, at an angle α with PA, to intersect the circle at F.
- The co-ordinates (σ, τ) of point F give the stress components on the plane MN. Analytical expression for σ, τ are:

$$\sigma = ((\sigma_1 + \sigma_3)/2) + ((\sigma_1 - \sigma_3)/2)\cos 2\alpha$$

$$\tau = ((\sigma_1 - \sigma_3)/2)\sin 2\alpha$$

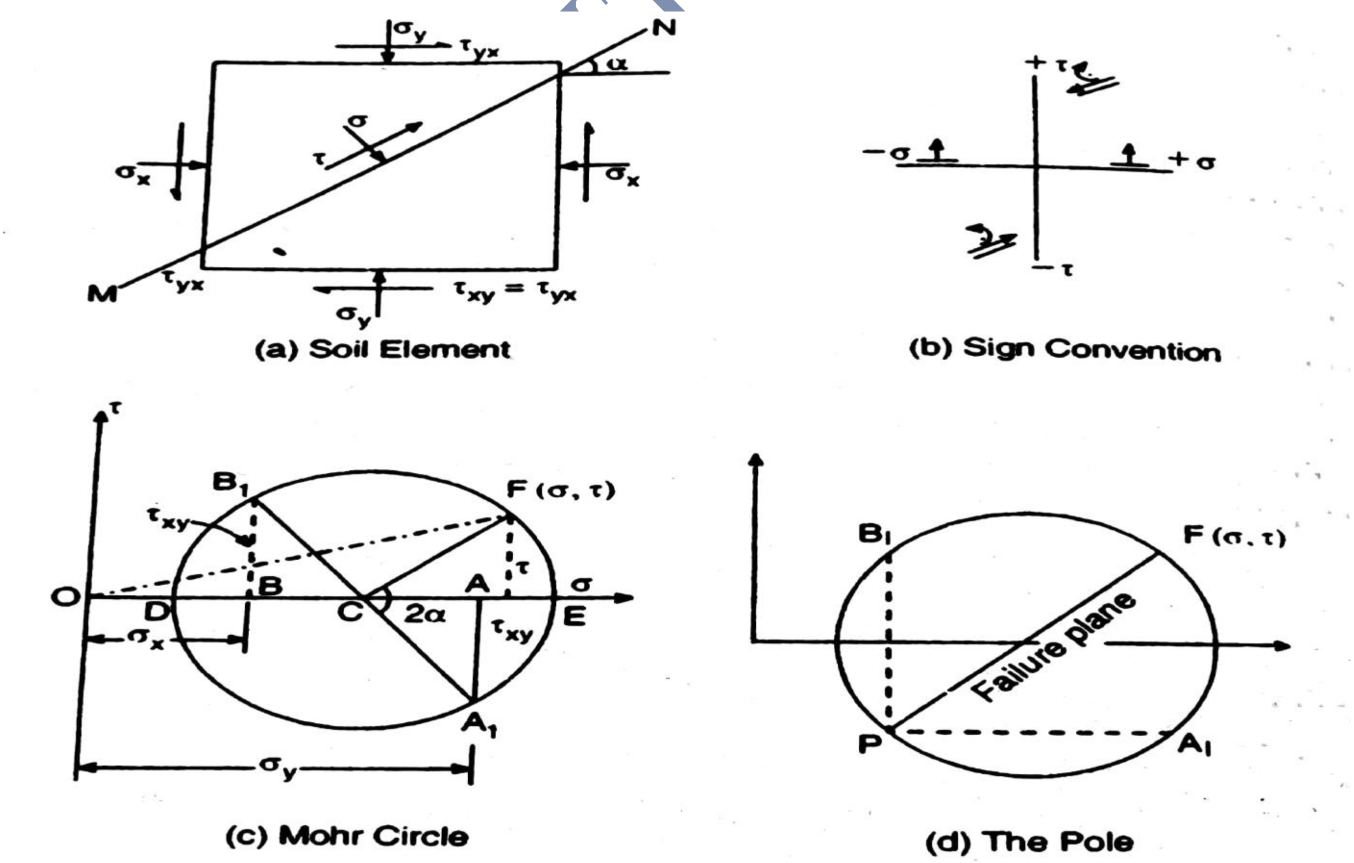


FIG. 18.1 MOHR'S STRESS CIRCLE.

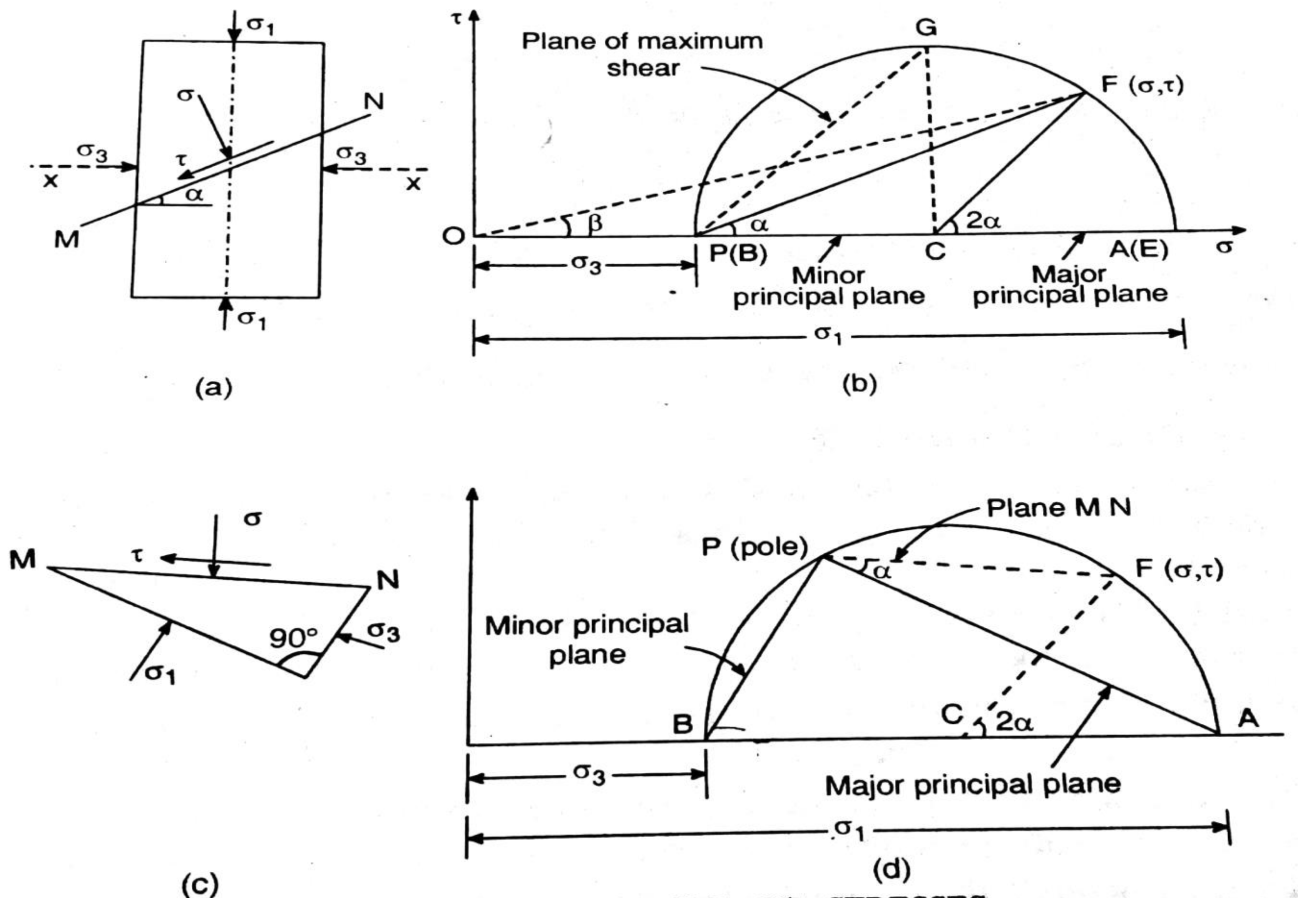


FIG. 18.2 MOHR CIRCLE OF STRESSES.

5.3 MOHR – COULOMB FAILURE THEORY:

Of the many theories of failure that have been proposed, only that formulated by Mohr (1900) has been useful in case of soils. The following are essential points of Mohr's strength theory:

(i) Material fails essentially by shear. The critical shear stress causing failure depends upon the properties of the material as well as on normal stress on the failure plane.

(ii) The ultimate strength of the material is determined by the stresses on potential failure plane (or plane of shear).

(iii) When the materials subjected to three dimensional principal stresses (i.e. σ_1, σ_2, σ_3) the intermediate principal stress does not have any influence on the strength of material.

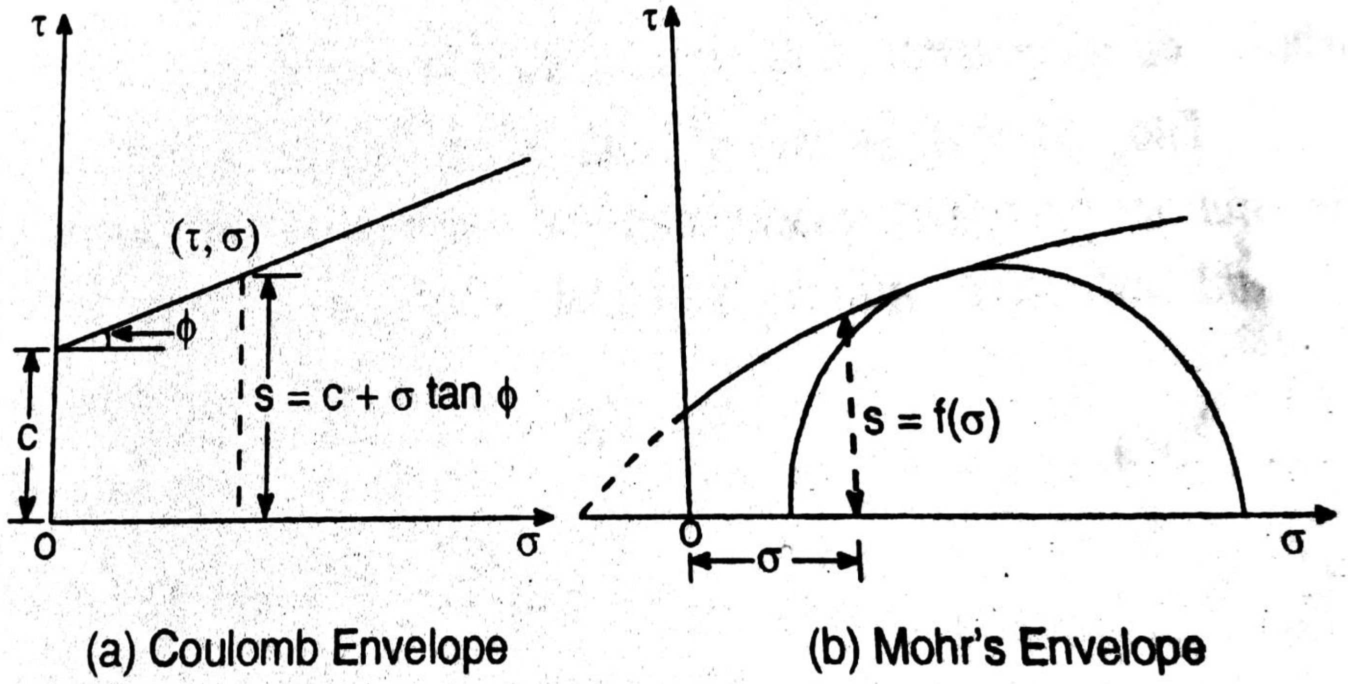


FIG. 18.3. FAILURE ENVELOPES.

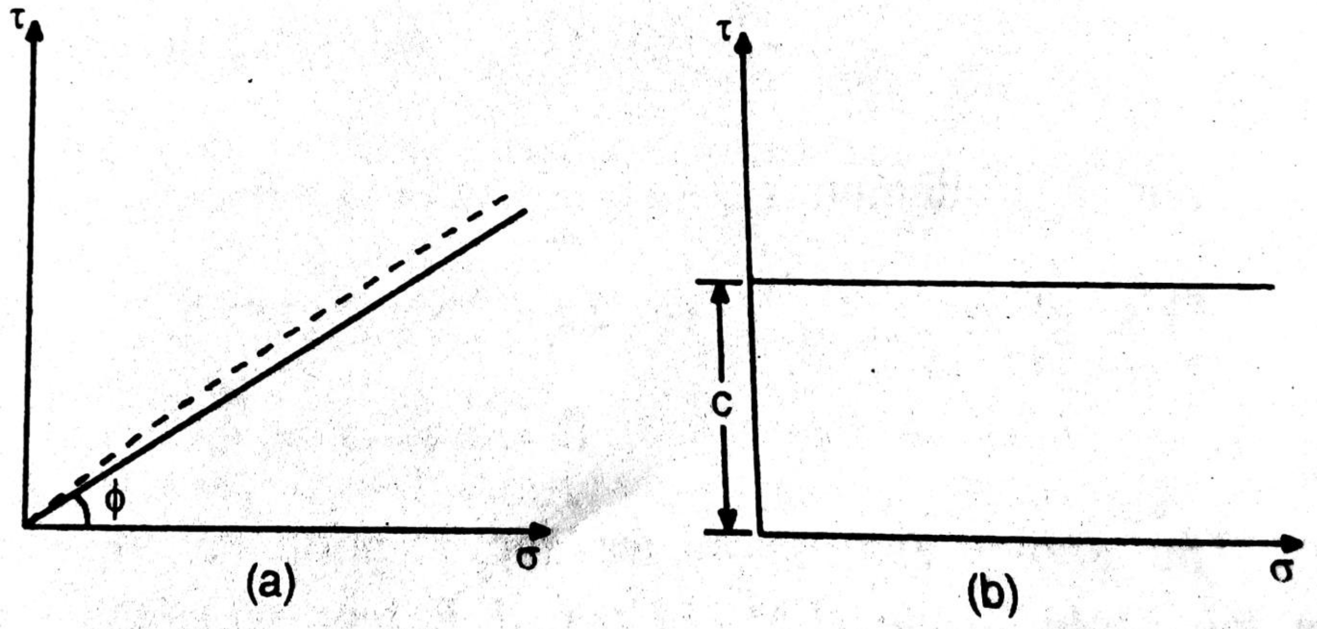


FIG. 18.4. STRAIGHT FAILURE ENVELOPE.

- The theory was first expressed by Coulomb (1776) and later generalised by Mohr. The theory can be expressed algebraically by the equation:

$$\tau_f = s = F(\sigma).$$

- Where τ_f = s = shear stress on failure plane, at failure = shear resistance of materials, $F(\sigma)$ = function of normal stress.
- If the normal and shear stress corresponding to failure are plotted, then a curve is obtained. The plot or the curve is called the strength envelope.
- Coulomb defined the function F (σ) as a linear function of σ and gave the following strength equation:

$$s = c + \sigma \tan \Phi.$$

- Where the empirical constants c and Φ represent respectively, the intercepts on the shear axis, and the slope of the straight line of equation $s = c + \sigma \tan \Phi$ [Fig. 18.3 (a)].
- These parameters are usually termed as cohesion and angle of internal friction or shearing resistance respectively.
- Fig. 18.3 (b) shows the Mohr's envelope, which is the graphical representation of $s = F(\sigma)$.
- Coulomb considered that the relationship between shear strength and normal stress could be adequately represented by the straight line.
- The generalised Mohr theory also recognises that the shear strength depends on the normal stress, but indicates that the relation is not linear.
- The strength theory upon which the Coulomb and Mohr strength lines are based indicates that definite relationship exists among the principal stresses, the angle of internal friction and the inclination of the failure plane.
- The curved failure envelope of Mohr is often referred to as a straight line for most of the calculations regarding the stability of soil mass.
- For an ideal pure friction material, such a straight line passes through the origin [Fig. 18.4 (a)]. However, dense sands exhibit a slightly curved strength line, indicated by dashed line.
- Fig. 18.4 (b) represents purely cohesive (plastic) material, for which the straight line is parallel to the σ – axis.
- The strength of such a material is independent of the normal stress acting on the plane of failure.
- It can, therefore, be concluded that the Mohr envelope can be considered to be straight if the angle of internal friction Φ is assumed to be constant.

- Depending upon the properties of a material the failure envelope may be straight or curved, and it may pass through the origin of stress or it may intersect the shear stress axis.

5.4 THE EFFECTIVE STRESS PRINCIPLE:

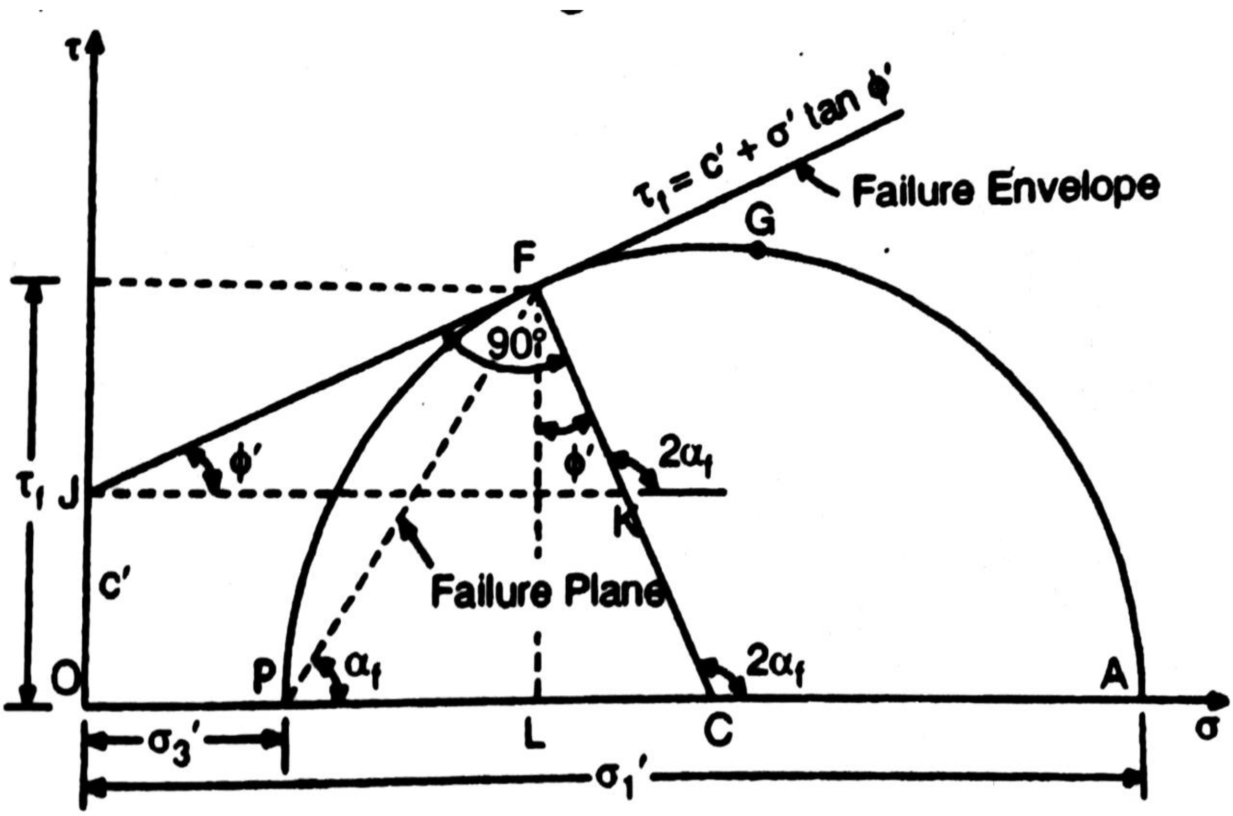


FIG. 18.5

- In equation s = c + σ tan Φ, it is assumed that the total normal stress governs the shear strength of soil.
- This assumption is not always correct. Extensive tests on remoulded clays have sustained beyond doubt Terzaghi’s early concept that the effective normal stresses control the shearing resistance of soils.
- Therefore, a failure criterion of greater general applicability is obtained by expressing the shear strength as a function of the effective normal stress σ', given by the equation:

$$\tau_f = c' + \sigma' \tan \Phi'$$

Or

$$\tau_f = c' + (\sigma - u) \tan \Phi'$$

- Where, c' = effective cohesion intercept and Φ' = effective angle of shearing resistance.
- In terms of total stresses, the equation takes the form:

$$\tau_f = c_u + \sigma \tan \Phi_u$$

- Where, c_u = apparent cohesion and Φ_u = apparent angle of shearing resistance.
- The normal stress σ' and shear stress τ on any plane inclined at an angle α to the major principal plane can be expressed in terms of effective major principal stress σ_1' and effective minor principal stress σ_3' from the equations $\sigma = ((\sigma_1 + \sigma_3)/2) + ((\sigma_1 - \sigma_3)/2) \cos 2\alpha$, $\tau = ((\sigma_1 - \sigma_3)/2) \sin 2\alpha$ as under:

$$\sigma' = ((\sigma_1' + \sigma_3')/2) + ((\sigma_1' - \sigma_3')/2) \times \cos 2\alpha \quad (18.11)$$

$$\tau = ((\sigma_1' - \sigma_3')/2) \times \sin 2\alpha \quad (18.12)$$

- Substituting the values of σ' in equation $\tau_f = c' + \sigma' \tan \Phi'$, we get

$$\tau_f = c' + \tan \Phi' \left[((\sigma_1' + \sigma_3')/2) + ((\sigma_1' - \sigma_3')/2) \times \cos 2\alpha \right]$$

- The most dangerous plane i.e. the plane on which failure will take place is the one on which the difference $(\tau_f - \tau)$, between the shear strength and shear stress is minimum.

$$(\tau_f - \tau) = c' + ((\sigma_1' + \sigma_3')/2) \tan \Phi' + ((\sigma_1' - \sigma_3')/2) \times \cos 2\alpha \times \tan \Phi' - ((\sigma_1' - \sigma_3')/2) \times \sin 2\alpha$$

- Differentiating this with respect to α, we get.

$$d/d\alpha\,(\tau_f - \tau) = -(\sigma_1' - \sigma_3') \sin 2\alpha \times \tan \Phi' - (\sigma_1' - \sigma_3') \cos 2\alpha$$

- For minimum $(\tau_f - \tau)$, $d/d\alpha\,(\tau_f - \tau) = 0$.
- This gives $\cos 2\alpha = -\sin 2\alpha \times \tan \Phi'$ and $\alpha = \alpha_f = 45° + (\Phi'/2)$.
- The above expression for the location of the failure plane can be directly derived from the Mohr circle [Fig. 18.5].
- JF represents the failure envelope given by the straight line $\tau_f = c' + \sigma' \tan \Phi'$.
- The pole P will be the point with stress co-ordinates as $(\sigma_3', 0)$.
- The Mohr circle is tangential to the Mohr envelope at the point F. PF represents the direction of the failure plane, inclined at an angle of α_f with the direction of the major principal plane.
- From the geometry of Fig. 18.5, we get from triangle JFK: $2\alpha_f = 90° + \Phi'$ or $\alpha_f = 45° + (\Phi'/2)$.

- It should be noted that for any combination of the applied principle effective stress σ_1' and σ_3', failure will occur only if the stress circle touches the failure envelope.
- Also, the co-ordinates of the failure point F represent the stress components σ' and τ at failure.
- As it is evident from Fig. 18.5, the τ_f at failure is less than the maximum shear stress, corresponding to point G, acting on the plane PG.
- Thus, the failure plane does not carry maximum shear stress, and the plane which has the maximum shear stress is not the failure plane.

5.5 MEASURMEENT OF SHEAR STRENGTH PARAMETERS:

The measurement of shear strength of soil involves certain test observations at failure with the help of which the failure envelope can be plotted corresponding to a given set of conditions. Shearing resistance can be determined in the laboratory by the following four methods:

5.5.1 Direct shear test

5.5.2 Triaxial shear test

5.5.3 Unconfined compression test

5.5.4 Vane shear test

Again depending upon the drainage conditions, three types of shear test have been developed:

5.5.5 Undrained test or Quick test

5.5.6 Consolidated undrained test

5.5.7 Drained test

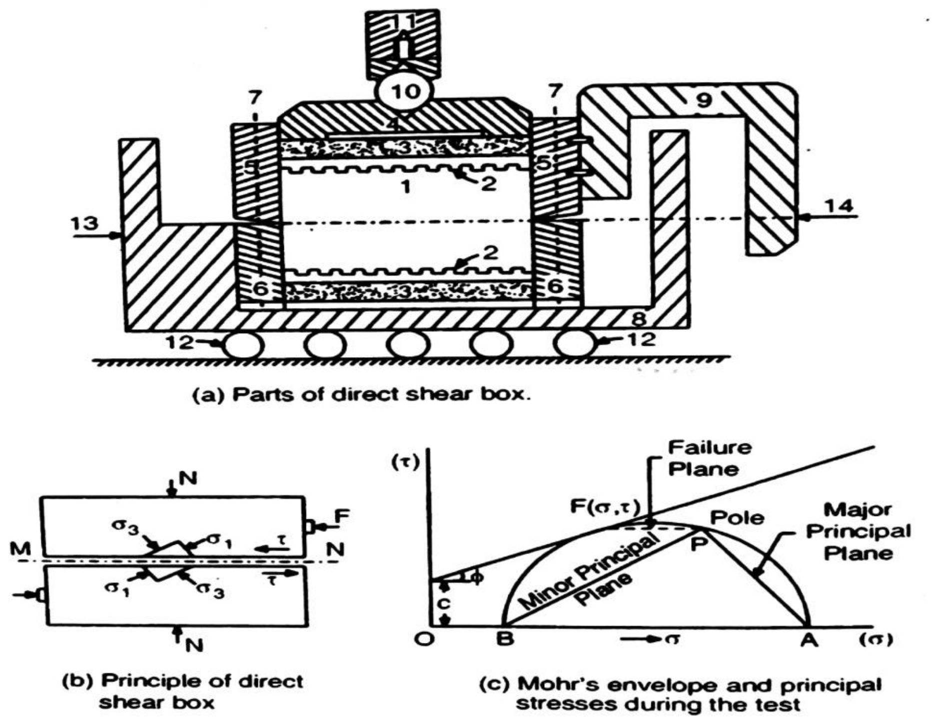
5.5.5 Undrained Test or Quick Test: no drainage of water is permitted. Hence there is no dissipation of pore pressure during entire test.

5.5.6 Consolidated Undrained Test: drainage is permitted under the initially applied normal stress only and full primary consolidation or softening is allowed to take place. No drainage is allowed afterwards.

5.5.7 Drained Test: drainage is permitted throughout the test during the application of both normal and shear stresses, so that full consolidation occurs and no excess pore pressure is set up at any stage of the test.

The parameters c and Φ are not fundamental properties of the soil; they may be simply be considered merely coefficients derived from the geometry of the graph obtained by the plotting shear stress at failure against normal stress. They vary with the drainage conditions of the test.

5.5.1 Direct Shear Test:



1. SOIL SPECIMEN
2. METAL GRIDS
3. POROUS STONES
4. LOADING PAD
5. UPPER PART
6. LOWER PART
7. SCREWS TO FIX TWO HALVES OF SHEAR BOX
8. CONTAINER FOR SHEAR BOX
9. U-ARM
10. STEEL BALL
11. LOADING YOKE
12. ROLLERS
13. SHEAR FORCE APPLIED BY JACK
14. SHEAR RESISTANCE MEASURED BY PROVING RING

FIG. 18.6. SHEAR BOX TEST.

- This is a simple and commonly used test and is performed in a shear box apparatus (Fig. 18.6). The apparatus consists of a two piece shear box of square or circular cross section.
- The lower half of the box is rigidly held in position in a container which rests over slides or rollers and which can be pushed forward at a constant rate by geared jack, driven either by electric motor or by hand.
- The upper half of the box butts against a proving ring. The soil sample is compacted in the shear box, and is held between metal grids and porous stones or plates.
- As shown in Fig. 18.6 (a), the upper half of the specimen is held in the upper box and the lower half in the lower box, and the joint between the two parts of the box is at a level of the centre of the specimen.
- Normal load is applied on the specimen from a loading yoke bearing upon steel ball of pressure pad.

- When a shearing force is applied to the lower box through the geared jack, the movement of the lower part of box is transmitted through the specimen to the upper part of the box and hence on the proving ring. The deformation of proving ring indicates the shear force.
- The volume change during the consolidation and during the shearing process is measured by mounting a dial gauge at the top of the box.
- The soil specimen can be compacted in the shear box by clamping both the parts together with the help of two screws. These screws are, however, removed before the shearing force is applied.
- Metal grids, placed above the top and below the bottom of the specimen may be perforated if drained test is required or plain if undrained test is required.
- The metal grids have linear slots to have proper grip with the soil specimen, and are so oriented that the slots are perpendicular to the direction of the shearing force.
- The specimen of the shear box is sheared under a normal load N. The shearing strain is made to increase at a constant rate, and hence the test is called the strain controlled shear box test.
- The other type of test is the stress controlled shear box test, in which there is an arrangement to increase the shear stress at a desired rate and measure the shearing strain.
- Fig. 18.6 (a) shows the strain controlled shear box. The shear force F, at failure, corresponding to the normal load N is measured with the help of the proving ring.
- A number of identical specimens are tested under increasing normal loads and the required maximum shear force is recorded.
- A graph is plotted between the shear force F as the ordinate and the normal load N as the abscissa. Such a plot gives the failure envelope for the soil under the given test conditions.
- Fig. 18.6 (c) shows such a failure envelope plotted as a function of the shear stress s and the normal stress σ.
- The scales of both s and σ are kept equal so that the angle of shearing resistance can be measured directly from the plot.
- Any point F (σ, τ) on the failure envelope represents the state of stress in the material during failure, under a given normal stress. In direct shear test, the failure plane MN is predetermined, and is horizontal. Fig. 18.6 (b) shows the stress conditions during failure.
- In order to find the direction of principal planes at failure, we first locate the position of the pole on the Mohr circle [Fig. 18.6 (c)] on the principle that the line joining any point on the circle to the pole P gives the direction of the plane on which the stresses are those given by the co-ordinates of that point.

- Hence, through point F a horizontal line is drawn to intersect the circle at the point P which is the pole.
- Since points A and B represent respectively, the major and minor principal stresses, PA and PB give the directions of major and minor principal planes.
- Tests can be performed under all the three conditions of drainage. To conduct undrained test, plain grids are used. For the drained test, perforated grids are used.

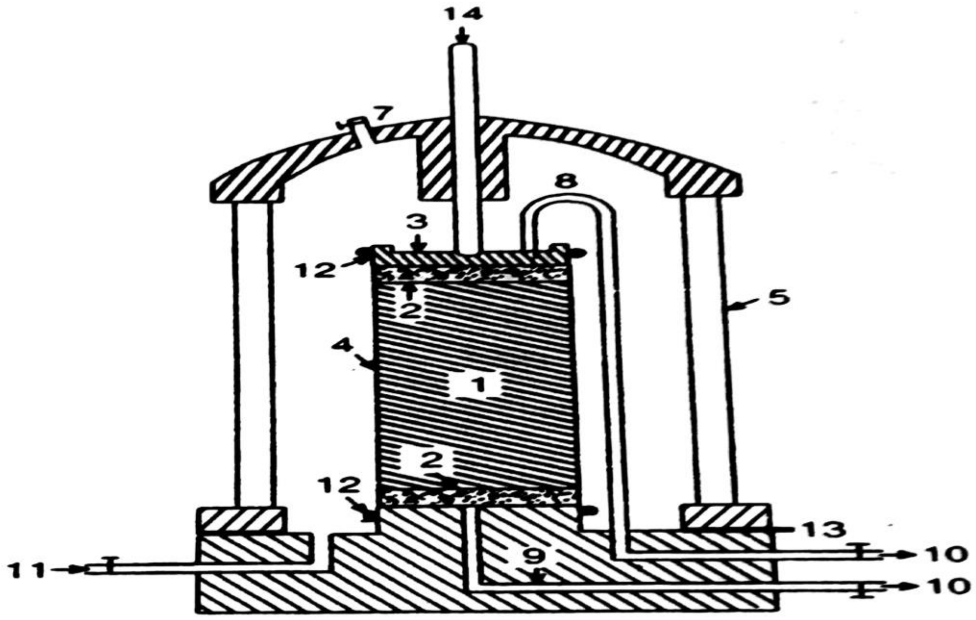
5.5.1.1 Advantages of Direct Shear Test:

- The direct shear test is a simple test and relatively fast especially for granular soils.
- The basic principle is easily understood which can be extended to gravelly soils and other materials containing large particles.
- Quick drainage and quick dissipation of pore pressure developed in sample is usually easy to achieve because of relatively thin thickness of sample.
- The apparatus is relatively cheap.
- It is not difficult to prepare re-compacted test samples.

5.5.1.2 Disadvantages of Direct Shear Test:

- It is difficult to control the drainage conditions if an undrained test is conducted.
- The stress conditions across the soil sample are very complex. The failure occurs progressively from the edges towards the centre.
- The plane of shear failure is predetermined and may not necessarily be the weakest one.
- The pore water pressure cannot be measured and therefore only the total stresses are known.
- As the test progress, the area under the shear gradually decreases. The connected area at failure should be used in determining the values of normal stress and shear stress.

5.5.2 Triaxial Shear Test:



1. SOIL SPECIMEN
2. POROUS DISC
3. TOP CAP
4. RUBBER MEMBRANE
5. PERSPEX CYLINDER
6. LOADING RAM
7. AIR RELEASE VALVE
8. TOP DRAINAGE TUBE
9. BOTTOM DRAINAGE TUBE
10. CONNECTIONS FOR DRAINAGE OR PORE PRESSURE MEASUREMENT
11. CELL FLUID INLET
12. RUBBER RINGS
13. SEALING RING
14. AXIAL LOAD THROUGH PROVING RING

FIG. 18.7. THE TRIAXIAL CELL.

- The strength test more commonly used in a research laboratory is the triaxial compression test, first introduced in the U.S.A by A. Casagrande and Karl Terzaghi in 1936-37.
- The solid specimen, cylindrical in shape, is subjected to direct stresses acting in three mutually perpendicular directions.
- In the common solid cylindrical specimen test, the major principal stress σ_1 is applied in the vertical direction and the other two principal stresses σ_2 and σ_3 ($\sigma_2 = \sigma_3$) are applied in the horizontal direction by the fluid pressure round the specimen.
- The test equipment specially consists of a high pressure cylindrical cell, made of perspex or other transparent material, fitted between the base and the top cap.
- Three outlet connections are generally provided through the base: cell fluid inlet, pore water outlet from the bottom of the specimen and the drainage outlet from the top of the specimen.
- A separate compressor is used to apply fluid pressure in the cell. Pore pressure developed in the specimen during the test can be measured with the help of a separate pore pressure measuring equipment.
- The cylindrical specimen is enclosed in a rubber membrane.

- A stainless steel piston running through the centre of the top cap applies the vertical compressive load (called the deviator stress) on the specimen under test.
- The load is applied through a proving ring, with the help of a mechanically operated load frame.
- Depending upon the drainage conditions of the test, solid nonporous discs or end caps, or porous discs are placed on the top and bottom of the specimen and the rubber membrane is sealed on to these end caps by rubber rings.
- The length of the specimen is kept about 2 to 2 ½ times its diameter. The cell pressure $\sigma_2 = \sigma_3$ acts all-round the specimen; it acts also on the top of the specimen as well as the vertical piston meant for applying the deviator stress.
- The vertical stress applied by the loading frame, through the proving ring is equal to $(\sigma_1 - \sigma_3)$, so that the total stress on the top of the specimen = $(\sigma_1 - \sigma_3) + \sigma_3 = \sigma_1$ = major principal stress.
- This principal stress difference $(\sigma_1 - \sigma_3)$ is called the deviator stress recorded on the proving ring dial. Another dial measures the vertical deformation of the sample during testing.
- A particular confining pressure σ_3 is applied during one observation, giving the value of the other stress σ_1 at failure. A Mohr circle corresponding to this set of (σ_1, σ_3) can thus be plotted.
- Various sets of observations are taken for different confining pressures σ_3 and the corresponding values of σ_1 are obtained. Thus, a number of Mohr circles, corresponding to failure conditions, are obtained.
- A curve, tangential to these stress circles, gives the failure envelope for the soil under the given drainage conditions of the test.
- Shear test can be performed in the triaxial apparatus under all the three drainage conditions. For undrained test, solid (nonporous) end caps are placed on the top and bottom of the specimen.
- In the consolidated-undrained test, porous discs are used. The specimen is allowed to consolidated under the desired confining pressure by keeping the pore water outlet open.
- When the consolidation is complete, the pore water outlet is closed, and the specimen is sheared under undrained conditions. The pore water pressure can be measured during the undrained part of the test.
- In drained test, porous discs are used, and the pore water outlet is kept open throughout the test. The compression test is carried out sufficiently slowly to allow for the full drainage during the test.
- Fig. 18.10 (a) shows the effective stresses acting on the soil specimen during triaxial testing. The minor principal stress and the intermediate principal stress are equal.
- The effective minor principal stress is equal to the cell pressure minus the pore pressure.

- The major principal stress is equal to the deviator stress plus the cell pressure.
- The effective major principal stress σ_1' is equal to the major principal stress minus the pore pressure.
- The stress components on the failure plane MN are σ' and τ_f, and the failure plane is inclined at an angle of α' to the major principal plane. Fig. 18.10 (b) shows the failure envelope JF and a Mohr circle corresponding to any failure point F.

5.5.2.1 Advantages of Triaxial Shear Test:

- The shear tests under all the three drainage conditions can be performed with complete control.
- Precise measurements of the pore pressure and volume change during the test are possible.
- The stress distribution on the failure plane is uniform.
- The state of stress within the specimen during any stage of the test, as well as at failure is completely determinate.
- The specimen is free to fail along the weakest plane.

5.5.2.2 Disadvantages of Triaxial Shear Test:

- The triaxial test is much more complicated than the direct shear test.
- The plane of failure is not predetermined. It can occur anywhere and the test may reveal a surface of weakness relating to some natural feature of the soil structure.
- Tests on smaller diameter give unrealistically high strength of stiff clays.
- Consolidation and drainage of cohesive samples in a triaxial cell take a much longer time compared to that in a direct shear test.
- Samples of cohesionless soil such as sands can be difficult to prepare and are perhaps more conveniently tested in a direct shear test.

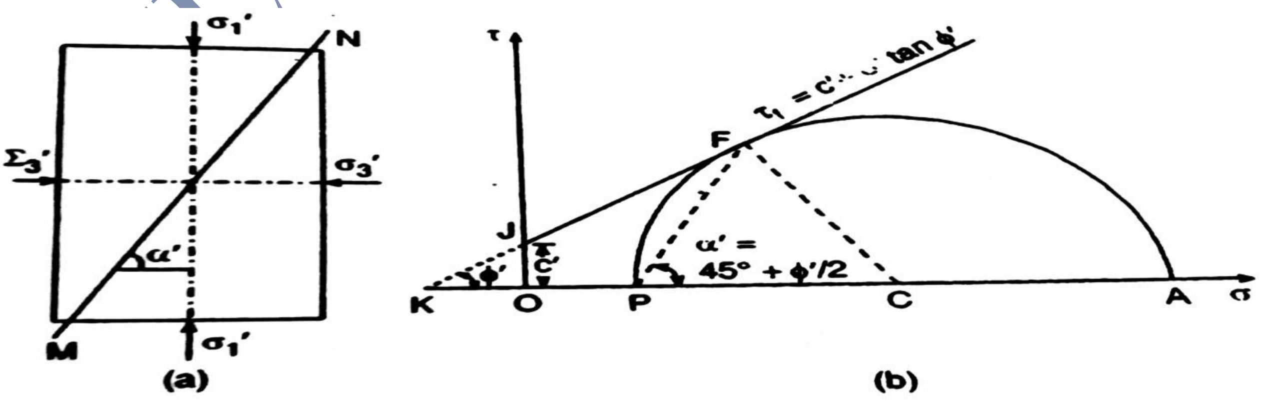


FIG. 18.10. STRESS CONDITIONS AND FAILURE ENVELOPE IN TRIAXIAL COMPRESSION TESTS.

5.5.3 Unconfined Compression Test:

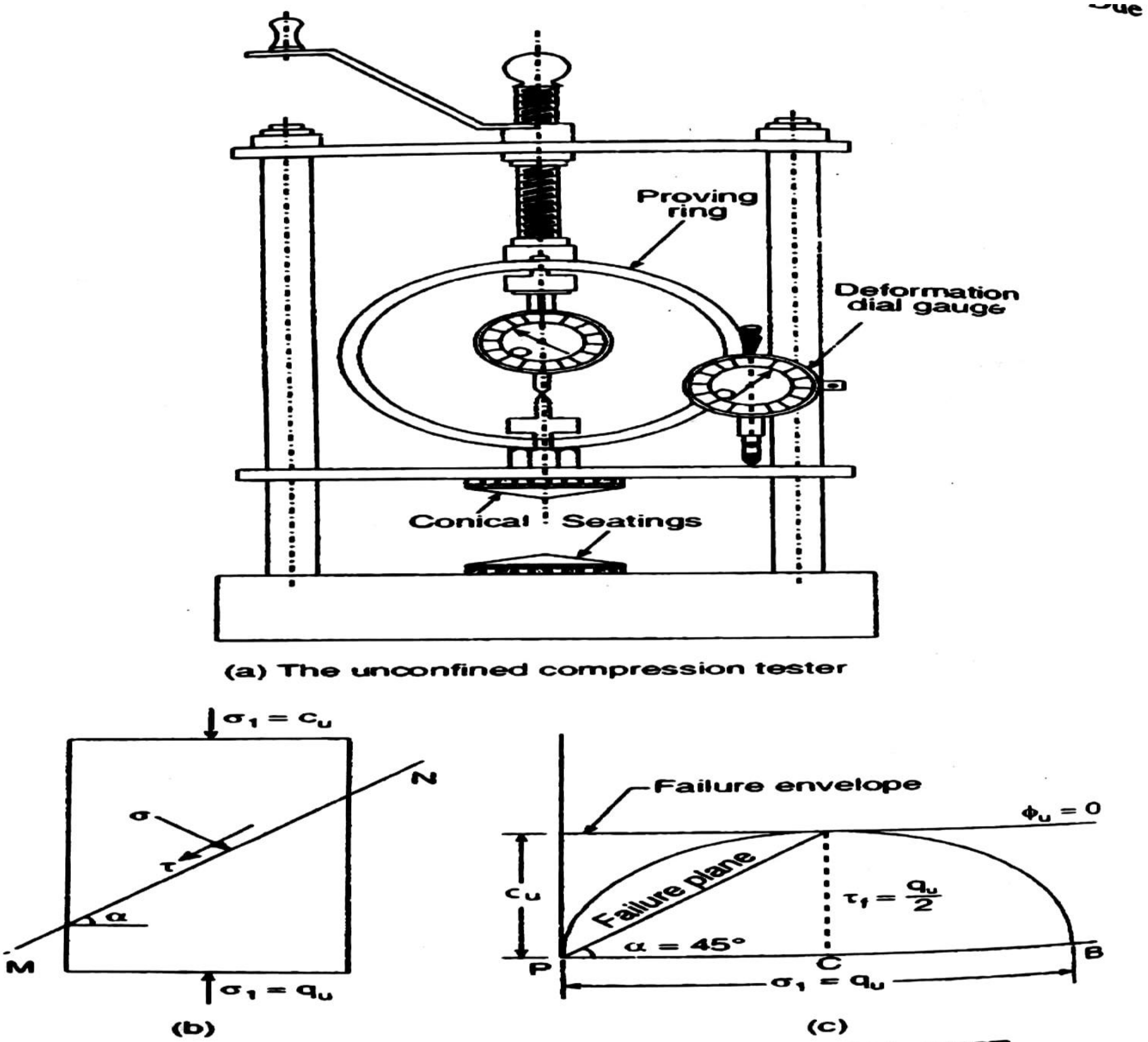


FIG. 18.12. UNCONFINED COMPRESSION TEST.

- The unconfined compression test is a special case of triaxial compression test in which $\sigma_2 = \sigma_3 = 0$. The cell pressure in the triaxial cell is also called the confining pressure. Due to the absence of such a confining pressure, the uniaxial test is called the unconfined compression test.
- The cylindrical specimen of soil is subjected to major principal stress σ_1 till the specimen fails due to shearing along a critical plane of failure.
- In its simplest form, the apparatus consists of a small load frame fitted with a proving ring to measure the vertical stress applied to the soil specimen.
- Fig. 18.12 (a) shows an unconfined compression tester. The deformation of the sample is measured with the help of a separate dial gauge.

- The ends of the cylindrical specimen are hollowed in the form of cone. The cone seating reduce the tendency of the specimen to become barrel shaped by reducing end restraints.
- During the test, load versus deformation readings are taken and a graph is plotted. When a brittle failure occurs, the proving ring dial indicates a definite maximum load which drops rapidly with the further increase of strain.
- In the plastic failure, no definite maximum load is indicated. In such a case, the load corresponding to 20% strain is arbitrarily taken as the failure load.
- Fig. 18.12 (b), (c) shows the stress conditions, at failure, in the unconfined compression test which is essentially an undrained test. Since $\sigma_3 = 0$, the Mohr circle passes through the origin which is also the pole.
- From Eq. $\sigma_1 = \sigma_3 \tan^2\alpha + 2c_u \tan\alpha$, we get

$$\sigma_1 = 2c_u \tan \alpha = 2c_u \tan (45° + \Phi_u/2)$$

- In the above equation, there are two unknowns c_u and Φ_u, which cannot be determined by the unconfined test since a number of tests on the identical specimens give the same value of σ_1.
- Therefore, the unconfined compression test is generally applicable to saturated clays for which the apparent angle of shearing resistance Φ_u is zero. Hence

$$\sigma_1 = 2c_u$$

- When the Mohr circle is drawn, its radius is equal to $\sigma_1 / 2 = c_u$. The failure envelope is horizontal. PF is the failure plane, and the stresses on the failure plane are

$$\sigma = \sigma_1 / 2 = q_u / 2 \quad \text{and} \quad \tau_f = \sigma_1 / 2 = q_u / 2 = c_u$$

5.5.3.1 Advantages of Unconfined Compression Test:

- The unconfined compression test is a quick, simple and widely used test.
- This test has a significant cost advantage over a triaxial test.
- This test is quite suitable for studying sensitivity of clays.

5.5.3.2 Disadvantages of Unconfined Compression Test:

- The sample must be fully saturated so that apparent angle of cohesion may be assumed zero.
- The test under estimates in-site strength because of the sampling disturbance.
- This test is suitable for intact homogeneous clays.

5.5.4 Vane Shear Test:

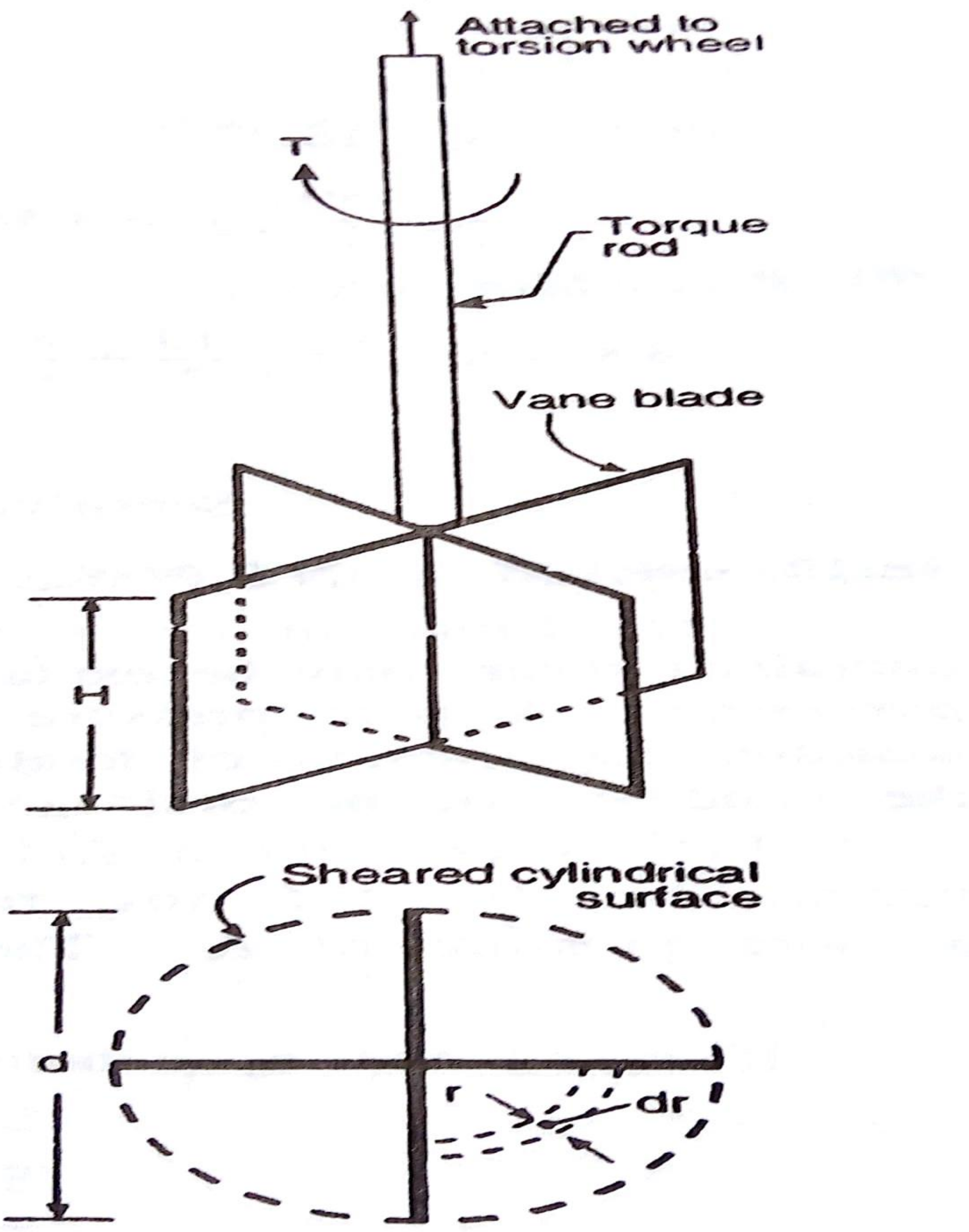


FIG. 18.22. VANE SHEAR TESTS.

- Vane shear test is a quick test, used either in the laboratory or in the field, to determine the undrained shear strength of cohesive soil.
- The vane shear tester consists of four thin steel plates, called vanes, welded orthogonally to a steel rod.

- A torque measuring arrangement, such as a calibrated torsion spring, is attached to the rod which is rotated by a worm gear and worm wheel arrangement.
- After pushing the vanes gently into the soil, the torque rod is rotated at a uniform speed. The rotation of the vane shears the soil along cylindrical surfaces.
- The rotation of the spring in degrees is indicated by a pointer moving on a graduated dial attached to the worm wheel shaft. The torque T is then calculated by multiplying the dial reading with the spring constant.
- A typical laboratory vane is 20 mm high and 12 mm in diameter with a blade thickness from 0.5 to 1 mm, the blades being made of high tensile steel.
- The field shear vane is from 10 to 20 cm in height and from 5 to 10 cm in diameter, with blade thickness of about 2.5 mm.
- The torque is calculated using the equation:

$$T = \pi d^2 \tau_f \left(H/2 + d/12 \right)$$

5.6 DETERMINATION OF SHEAR STRENGTH PARAMETERS BY DIRECT SHEAR TEST:

The object of the test is to determine the shear parameters of a soil with the help of shear box test.

5.6.1 Procedure:

5.6.1.1 Preparation of Specimen:

- The undisturbed specimen is prepared by pushing a cutting ring of size 10 cm in diameter and 2 cm high, in the undisturbed soil sample obtained from the field. The square specimen of size 6 cm x 6 cm is then cut from the circular specimen so obtained.
- In order to obtain remoulded specimen of cohesive soil, the soil may be compacted to the required density and water content, in a separate bigger mould. The sample is then extracted and trimmed to the required size.
- Alternatively, the soil may be compacted at the required density and water content directly into the shear box after fixing two halves of the shear box together by means, of the fixing screws.
- Non-cohesive soils may be tamped in the shear box itself with the base plate and grid plate or porous stone as required in place at the bottom of the box.
- In all three cases mentioned above, water content and dry density of the soil compacted in the shear box should be determined.

5.6.1.2 Undrained Test:

- The shear box with the specimen plain grid plate over the base plate at the bottom of the specimen, and plain grid plate over the top of the specimen, should be fitted into position. The serrations of the grid plates should be placed at right angles to the direction of shear. As the porous stones are not used in the undrained tests, plain plates of equal thickness should be placed, one at the bottom and the other at the top of the two grids, so as to maintain the shear plane in the sample in the middle of its thickness. Place the loading pad on the top of the plain grid plate. Both the parts of the box should be tightened together by the fixing screws.
- Put the water inside the water jacket so that the sample does not get dried during the test.
- Mount the shear box assembly on the load frame. Set the lower part of the shear box to bear against the load jack and upper part of the box to bear against the proving ring. Set the dial of the proving ring to zero.
- Put the loading yoke on the top of the loading pad, and adjust the dial gauge to zero to measure the vertical displacement in the soil sample. Put proper normal weight on the hanger of the loading yoke, so that this weight plus the weight of the hanger equals the required normal load. Note the reading of the vertical displacement dial gauge.
- Remove the locking screws so that the parts are freed to move against each other. By turning the spacing screws, raise the upper part slightly above the lower parts by about 1 mm.
- Conduct the test by applying horizontal shear load to failure or to 20% longitudinal displacement, which ever occur first. The rate of strain may vary from 1 to 2.5 mm per minute.
- Start the stop watch immediately at the start of the application of the shear load. Take the readings of proving ring dial gauge, longitudinal displacement gauge and vertical displacement gauge at regular time intervals.
- At the end of the test, remove specimen from the box and determine its final water content. Repeat the above steps on three or four identical specimens, under laying normal loads.

5.6.1.3 Consolidated Undrained Test:

- Assemble the box in the similar manner as described above for the undrained test, except that instead of plain grid plates, perforated grid plates should be used, and saturated porous stones, should be used one at the top of the top perforated grid plate and other at the bottom of the bottom perforated grid plate.
- Step (2), (3) and (4) same as in undrained test.

- The shear test should be conducted only after complete consolidation has occurred under a particular normal stress. After the application of the normal load, the vertical compression of the soil with time should be recorded, as is done in the consolidation test.
- Apply the horizontal shear load. The rate of shear should be such that water does not drain from the specimen at the time of application of shear load. Take the reading of the three dial gauges at constant interval of time.
- Remove the specimen from the box, at the end of the test, and determine its final water content. Repeat the test on three of four identical specimens, under varying normal loads.

5.6.1.3 Consolidated Drained Test:

- Assemble the shear box with sample, perforated grid plates and porous stones as in consolidated undrained test. Allow the sample to consolidate under the normal load. After the application of the normal load, vertical compression of the soil with time should be recorded, as is done in the consolidation test.
- When consolidation has completely occurred, the shear test should be done at a slow rate so that complete drainage can occur and at least 95% pressure dissipation occurs during the test.
- At the end of the test, remove the specimen from the box and determine its final water content. Repeat the test on three or four identical specimens, under varying normal loads.

5.7 DETERMINATION OF SHEAR STRENGTH PARAMETERS BY TRIAXIAL SHEAR TEST:

The object of the experiment is to determine shear parameters of undisturbed (or remoulded) soil specimen in the triaxial compression apparatus by unconsolidated undrained test without the measurement of pore pressure.

5.7.1 Procedure:

5.7.1.1 Preparation of Specimen:

5.7.1.1.1 Undisturbed Specimen:

- If the undisturbed sample has been collected in a thin walled tube having the same internal diameter as that of the specimen required for testing, the sample may be extruded out with the help of sample extruder and pushed into the split mould.

- The sample should be extruded from the tube pushing from the cutting edge side. The ends of the specimen are trimmed flat and normal to its axis. The split mould should be lightly oiled from inside.
- The specimen is taken out carefully from the split mould, and its length, diameter, weight should be measured to an accuracy enabling the bulk density to be calculated.
- A portion of soil trimmings is placed for water content determination. The specimen is then placed on one of the end caps and the other end cap is put on the top of the specimen.
- The rubber membrane is then placed around the specimen using the membrane stretcher. The membrane is sealed to the end caps by means of rubber rings. The specimen is then ready to be placed on the pedestal in the triaxial cell.

5.7.1.1.2 Remoulded Specimen:

- Remoulded specimens may be prepared by compacting the soil, at required water content and dry density, in a big size mould by static or dynamic method, and then preparing the cylindrical specimen of required dimensions by the method described in step 2 above.

5.7.1.1.3 Undrained Triaxial Compression Test:

- Cover the pedestal in the triaxial cell with a solid end cap or keep drainage valve closed. Place the specimen assembly centrally on the pedestal.
- Assemble the cell, with the loading ram initially clear of the top of the specimen, and place it in the loading machine.
- Admit the operating fluid in the cell, and raise its pressure to the desired value. Adjust the loading machine to bring the loading ram a short distance away from the set on the top cap of the specimen.
- Read the initial reading to the load measuring gauge. Adjust the loading machine further so that the loading ram comes just in contact with the seat on the top of the specimen. Note the initial reading of the dial measuring axial compression.
- Apply the compressive force at constant rate of axial compression, such that failure is produced in a period of approximately 5 to 15 minutes.
- Take the simultaneous reading of load and deformation dials, define the stress-strain curve. Continue the test until the maximum value of stress has been passed or until an axial strain of 20 percent has been passed.

- Unload the specimen and drain off the cell fluid. Dismantle the cell and take out the specimen. Remove the rubber membrane and note down the mode of failure.
- Weigh the specimen. Keep samples for water content determination. Repeat the test on three or more identical specimens under different cell pressure.

5.8 DETERMINATION OF UNCONFINED COMPRESSIVE STRENGTH OF SOIL:

The object of the experiment is to determine the unconfined compressive strength of clayey soil using controlled strain. The purpose of the test is to obtain a quantitative value of compressive and shearing strength of soils in an undrained state.

5.8.1 Procedure:

5.8.1.1 Preparation of Specimen:

- Undisturbed cylindrical specimen may be cut from the bigger undisturbed sample obtained from the field. A wire saw may be used to trim the ends parallel to each other.
- A lathe or trimmer may be used to trim the specimen to circular cross section. Alternatively, field sample may be obtained directly in a thin sampling tube having the same internal diameter as the specimen to be tested.
- The split mould is oiled lightly from inside and the sample is then pushed out of the tube into the split mould. The split mould is opened carefully and sample is taken out.
- Remoulded sample may be prepared by compacting the soil at the desired water content and dry density in a bigger mould, and then cut by the sampling tube. Alternatively, remoulded specimen may be prepared directly in the split mould.

5.8.1.2 Compression Test:

- Measure the initial length and diameter of the specimen. Put the specimen on the bottom plate of the loading device. Adjust the upper plate make contact with the specimen. Set the load dial gauge and the strain (compression) dial gauge to zero.
- Compress the specimen until cracks have definitely developed or until a vertical deformation of 20% is reached. Take the load dial readings approximately at every 1 mm deformation of the specimen.
- Sketch the failure pattern; measure the angle between the cracks and the horizontal, if possible.