RAJIV GANDHI PROUDYOGIKI VISHWAVIDYALAYA, BHOPAL

New Scheme Based On AICTE Flexible Curricula

Civil Engineering, VII-Semester

CE701- Geotechnical Engg

UNIT-I

INTRODUCTION, INDEX PROPERTIES AND CLASSIFICATION OF SOILS: Definition and scope of soil mechanics, Origin of soil, formation of soil, clay minerals, Soil structure. 3-phase soil system, Basic terminology, and their relations, index properties of soil - Water content, Field density, Specific gravity, Grain size distribution by sieve and hydrometer analysis, Relative density, Atterberg limits and their determination, Various indices -Flow-Index, Plasticity Index, Toughness Index, Liquidity Index, Activity Ratio.

Different Systems of Soil Classification - Particle Size, Textural, Unified, HRB and IS classification. Field identification of soils

UNIT –II

PERMEABILITY, EFFECTIVE STRESS AND SEEPAGE THROUGH SOILS : Types of soil water, Capillarity in soils, Flow of water through soils, Darcy's Law, permeability, Factors affecting permeability, Laboratory & field tests for determination of coefficient of permeability, Permeability of layered soils.

Seepage pressure, total, Neutral and effective stress, Upward and downward seepage through soils, , Flow nets: characteristics, Methods of construction of flow net, Application of flow net, Quick condition, Laplace Equation for two –dimensional flow, Seepage through anisotropic soil and non-homogenous soil, Seepage through earth dam.

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UNIT-III

STRESS DISTRIBUTION IN SOILS AND COMPACTION: Boussinesq's and Westergard's theories for point load, Uniformly loaded circular and rectangular areas, Pressure bulb, Variation of vertical stress under point load along vertical and horizontal plane, Newmark's influence chart for irregular areas. Contact pressure distribution in sands and clays.

Mechanism importance of compaction, Standard Proctor compaction test, Modified compaction test Factors affecting compaction, Effects of compaction on soil properties, Field compaction equipment and compaction quality control.

UNIT-IV

CONSOLIDATION : Types of compressibility, Spring analogy, Immediate settlement, Primary consolidation and secondary consolidation, Stress history of clay, e-p and e-log p curves, Normally consolidated soil, Over consolidated soil and under consolidated soil, Pre-consolidation pressure and its determination, Consolidation test, Terzaghi's 1-D consolidation theory, Coefficient of consolidation, Square root time and Logarithm of time fitting methods, Computation of total settlement.

UNIT-V

SHEAR STRENGTH & STABILIZATION OF SOILS:

SHEAR STRENGTH: Definition and importance of shear strength, Mohr and coulomb failure theories, Mohr's Stress Circles, Measurement of shear strength-Different types of Shear Test namely, Direct Shear Test, Unconfined Compression Test, Tri Axial Compression Test & Vane Shear Test for strength parameters, Strength tests based on drainage conditions, Measurement of pore pressure, Pore pressure parameters, Strength envelops, shear strength of sands, Critical void ratio, Liquefaction, Shear strength of clays. Factors affecting shear strength of granular soils and cohesive soils.

STABILIZATION OF SOIL: Introduction, Mechanical stabilization, Cement stabilization, Lime stabilization, Bituminous stabilization, Chemical stabilization, Thermal stabilization, Electrical stabilization, Stabilization by grouting, Use of geo-synthetic materials, Types, Functions and applications of geo-synthetics, Reinforced earth structures-components and construction.

LIST OF EXPERIMENTS:

1. Determination of water content by Oven drying method.
2. Determination of water content by Pycnometer
3. Determination of soil field density by core cutter method
4. Determination of soil field density by sand replacement method
5. Determination of Specific Gravity By Pycnometer.
6. Determination of Consistency Limits (i) Liquid Limit (ii) Plastic Limit (iii) Shrinkage Limit
7. Determination of liquid limit of soil by cone penetrometer.
8. Grain size analysis by sieve shaking method
9. Grain size analysis of fine grained soil by sedimentation using (i)pipette (ii) hydrometer.
10. Determination of coefficient of permeability of soil by- (a) constant head method (b) variable head method.
11. Determination of compaction parameters by- (a) light compaction, (b) heavy compaction.
12. Direct Shear test
13. Triaxial Test
14. Unconfined Compression Strength Test

Books and References

1. Punamia B.C., Soil Mechanics & Foundations., Firewall Media, 2017 (16th edition)

2. Alam Singh, Modern Geotechnical Engineering., CBS Publishers & Distributors, 2012 (3rd edition)

3. Gopal Ranjan & ASR Rao, Basic & Applied Soil Mechanics. New Age International, 2016 (3rd edition)

4. S.K Grag, Geotechnical Engineering., Khanna Publishers, 2016 (10th edition)

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UNIT-I

Basic Definitions & Index Properties: Definition and scope of soil mechanics, Historical development. Formation of soils, Soil composition, Minerals, Influence of clay minerals on engineering behavior, Soil structure, Three phase system, Index properties and their determination, Consistency limits, Classification systems based on particle size and consistency limits.

- **Definition and scope of soil mechanics:**

Soil mechanics is a branch of soil physics and applied mechanics that describes the behavior of soils. It differs from fluid mechanics and solid mechanics in the sense that soils consist of a heterogeneous mixture of fluids (usually air and water) and particles (usually clay, silt, sand, and gravel) but soil may also contain organic solids and other matter. Along with rock mechanics, soil mechanics provides the theoretical basis for analysis in geotechnical engineering, a sub discipline of civil engineering, and engineering geology. Soil mechanics is used to analyze the deformations of and flow of fluids within natural and man-made structures that are supported on or made of soil, or structures that are buried in soils. For example, applications in building and bridge foundations, retaining walls, dams, and buried pipeline systems. Principles of soil mechanics are also used in related disciplines such as engineering geology, geophysical engineering, coastal engineering, agricultural engineering, and hydrology and soil physics.

- **Historical development:**

Soil Mechanics is one of the subjects of Civil Engineering involving the study of soil, its behavior and application as an engineering material. According to Terzaghi : "Soil Mechanics is the application of laws of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulations of solid particles produced by the mechanical and chemical deformation and transformation of rocks regardless of whether or not they contain an admixture of an organic constituent."

- **Formation of soils:**

Soil formation or pedogenesis is the combined effect of physical, chemical, biological and anthropogenic processes working on soil parent material. Soil is said to be formed when organic matter has accumulated and colloids are washed downward, leaving deposits of clay, humus, iron oxide, carbonate, and gypsum, producing a distinct layer called the B horizon. This is a somewhat arbitrary definition as mixtures of sand, silt, clay and humus will support biological and agricultural activity before that time. These constituents are moved from one level to another by water and animal activity. As a result, layers

formation of distinctive soil horizons. However, more recent definitions of soil embrace soils without any organic matter, such as those regolith’s that formed on Mars and analogous conditions in planet Earth deserts. An example of the development of a soil would begin with the weathering of lava flow bedrock, which would produce the purely mineral-based parent material from which the soil texture forms.

Soil development would proceed most rapidly from bare rock of recent flows in a warm climate, under heavy and frequent rainfall. Under such conditions, plants (in first stage nitrogen-fixing lichens and cyan bacteria then epileptic higher plants) become established very quickly on basaltic lava, even though there is very little organic material. The plants are supported by the porous rock as it is filled with nutrient-bearing water that carries minerals dissolved from the rocks.

The weathering of parent material takes the form of physical weathering (disintegration), chemical weathering (decomposition) and chemical transformation. Generally, minerals that are formed under high temperatures and pressures at great depths within the Earth's mantle are less resistant to weathering, while minerals formed at low temperature and pressure environment of the surface are more resistant to weathering.[citation needed] Weathering is usually confined to the top few meters of geologic material, because physical, chemical, and biological stresses and fluctuations generally decrease with depth. Physical disintegration begins as rocks that have solidified deep in the Earth are exposed to lower pressure near the surface and swell and become mechanically unstable. Chemical decomposition is a function of mineral solubility, the rate of which doubles with each 10 °C rise in temperature, but is strongly dependent on water to effect chemical changes. Rocks that will decompose in a few years in tropical climates will remain unaltered for millennia in deserts. Structural changes are the result of hydration, oxidation, and reduction. Chemical weathering mainly results from the excretion of organic acids and chelating compounds by bacteria and fungi, thought to increase under present-day greenhouse effect.

Physical disintegration is the first stage in the transformation of parent material into soil. Temperature fluctuations cause expansion and contraction of the rock, splitting it along lines of weakness. Water may then enter the cracks and freeze and cause the physical splitting of material along a path toward the center of the rock, while temperature gradients within the rock can cause exfoliation of "shells". Cycles of wetting and drying cause soil particles to be abraded to a finer size, as does the physical rubbing of material as it is moved by wind, water, and gravity. Water can deposit within rocks minerals that expand upon drying, thereby stressing the rock. Finally, organisms reduce parent material in size and create

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Grinding of parent material by rock-eating animals also contributes to incipient soil formation.

Chemical decomposition and structural changes result when minerals are made soluble by water or are changed in structure. The first three of the following list are solubility changes and the last three are structural changes.

The solution of salts in water results from the action of bipolar water molecules on ionic salt compounds producing a solution of ions and water, removing those minerals and reducing the rock's integrity, at a rate depending on water flow and pore channels. Hydrolysis is the transformation of minerals into polar molecules by the splitting of intervening water. This results in soluble acid-base pairs. For example, the hydrolysis of orthoclase-feldspar transforms it to acid silicate clay and basic potassium hydroxide, both of which are more soluble. In carbonation, the solution of carbon dioxide in water forms carbonic acid. Carbonic acid will transform calcite into more soluble calcium bicarbonate.

Hydration is the inclusion of water in a mineral structure, causing it to swell and leaving it stressed and easily decomposed. Oxidation of a mineral compound is the inclusion of oxygen in a mineral, causing it to increase its oxidation number and swell due to the relatively large size of oxygen, leaving it stressed and more easily attacked by water (hydrolysis) or carbonic acid (carbonation). Reduction, the opposite of oxidation, means the removal of oxygen, hence the oxidation number of some part of the mineral is reduced, which occurs when oxygen is scarce. The reduction of minerals leaves them electrically unstable, more soluble and internally stressed and easily decomposed. It mainly occurs in waterlogged conditions.

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➢ **Soil composition:**

Soils are formed from materials that have resulted from the disintegration of rocks by various processes of physical and chemical weathering. The nature and structure of a given soil depends on the processes and conditions that formed it:

- **Breakdown of parent rock: weathering, decomposition, erosion.**
- **Transportation to site of final deposition: gravity, flowing water, ice, wind.**
- **Environment of final deposition: flood plain, river terrace, glacial moraine, lacustrine or marine.**
- **Subsequent conditions of loading and drainage: little or no surcharge, heavy surcharge due to ice or overlying deposits, change from saline to freshwater, leaching, contamination.**

Silts, sands and gravels are classified by their size, and hence they may consist of a variety of minerals.

Owing to the stability of quartz compared to other rock minerals, quartz is the most common

constituent of sand and silt. Mica and feldspar are other common minerals present in sands and silts. The mineral constituents of gravel may be more similar to that of the parent rock.

The common clay minerals are Montmorillonite, Illite, and Kaolinite. These minerals tend to form in sheet or plate like structures, with length typically ranging between 10^{-7} m and $4x10^{-6}$ m and thickness typically ranging between 10^{-9} m and $2x10^{-6}$ m, and they have a relatively large specific surface area. The specific surface area (SSA) is defined as the ratio of the surface area of particles to the mass of the particles. Clay minerals typically have specific surface areas in the range of 10 to 1,000 square meters per gram of solid. Due to the large surface area available for chemical, electrostatic, and van der Waals interaction, the mechanical behavior of clay minerals is very sensitive to the amount of pore fluid available and the type and amount of dissolved ions in the pore fluid. To anticipate the effect of clay on the way a soil will behave, it is necessary to know the kinds of clays as well as the amount present. As home builders and highway engineers know all too well, soils containing certain high-activity clays make very unstable material on which to build because they swell when wet and shrink when dry. This shrink-and-swell action can easily crack foundations and cause retaining walls to collapse. These clays also become extremely sticky and difficult to work with when they are wet. In contrast, low-activity clays, formed under different conditions, can be very stable and easy to work with.

The minerals of soils are predominantly formed by atoms of oxygen, silicon, hydrogen, and aluminum, organized in various crystalline forms. These elements along with calcium, sodium, potassium, magnesium, and carbon constitute over 99 per cent of the solid mass of soils.

- **<u>Grain size distribution:</u>**

Soils consist of a mixture of particles of different size, shape and mineralogy. Because the size of the particles obviously has a significant effect on the soil behavior, the grain size and grain size distribution are used to classify soils. The grain size distribution describes the relative proportions of particles of various sizes. The grain size is often visualized in a cumulative distribution graph which, for example, plots the percentage of particles finer than a given size as a function of size. The median grain size D_{50}, is the size for which 50% of the particle mass consists of finer particles. Soil behavior, especially the hydraulic conductivity, tends to be dominated by the smaller particles; hence, the term "effective size", denoted by D_{10}, is defined as the size for which 10% of the particle mass consists of finer particles.

Sands and gravels that possess a wide range of particle sizes with a smooth distribution of particle sizes are called well graded soils. If the soil particles in a sample are predominantly in a relatively narrow range of sizes, the sample is uniformly graded. If a soil sample has distinct gaps in the gradation curve, e.g., a mixture of gravel and fine sand, with no coarse sand, the sample may be gap graded. Uniformly graded and gap graded soils are both considered to be poorly graded. There are many methods for measuring particle-size distribution. The two traditional methods are sieve analysis and hydrometer analysis. For measuring the distribution of particle sizes in a soil sample, it is necessary to conduct different particle-size tests. Wet sieving is carried out for separating fine grains from coarse grains by washing the soil specimen on a 75 micron sieve mesh. Dry sieve analysis is carried out on particles coarser than 75 micron. Samples (with fines removed) are dried and shaken through a set of sieves of descending size. The weight retained in each sieve is measured. The cumulative percentage quantities finer than the sieve sizes (passing each given sieve size) are then determined. The resulting data is presented as a distribution curve with grain size along x-axis (log scale) and percentage passing along y-axis (arithmetic scale).

Sedimentation analysis is used only for the soil fraction finer than 75 microns. Soil particles are allowed to settle from a suspension. The decreasing density of the suspension is measured at various time intervals. The procedure is based on the principle that in a suspension, the terminal velocity of a spherical particle is governed by the diameter of the particle and the properties of the suspension.

In this method, the soil is placed as a suspension in a jar filled with distilled water to which a deflocculating agent is added. The soil particles are then allowed to settle down. The concentration of particles remaining in the suspension at a particular level can be determined by using a hydrometer. Specific gravity readings of the solution at that same level at different time intervals provide information about the size of particles that have settled down and the mass of soil remaining in solution.

The results are then plotted between % finer (passing) and log size.

- **Minerals:**
  - **Montmorillonite**

Montmorillonite clay is made of four planes of oxygen with two silicon and one central aluminum plane intervening. The aluminum-silicate Montmorillonite clay is said to have a 2:1 ratio of silicon to aluminum. The seven planes together form a single crystal of Montmorillonite. The crystals are weakly held together and water may intervene, causing the clay to swell up to ten times its dry volume. It occurs in soils which have had little leaching, hence it is found in arid regions. As the crystals are not bonded face to face, the entire surface is exposed and available for surface reactions, hence it has a high cation exchange capacity (CEC).

- **Illite**

Illite is clay similar in structure to Montmorillonite but has potassium bridges between the faces of the clay crystals and the degree of swelling depends on the degree of weathering of the potassium. The active surface area is reduced due to the potassium bonds. Illite originates from the modification of mica, a primary mineral. It is often found together with Montmorillonite and its primary minerals. It has moderate CEC, Vermiculite is a mica-based clay similar to Illite, but the crystals of clay are held together more loosely by hydrated magnesium and it will swell, but not as much as does Montmorillonite. It has very high CEC. Chlorite is similar to vermiculite, but the loose bonding by occasional hydrated magnesium, as in vermiculite, is replaced by a hydrated magnesium sheet that firmly bonds the planes above and below it. It has two planes of silicon, one of aluminum and one of magnesium; hence it is a clay.

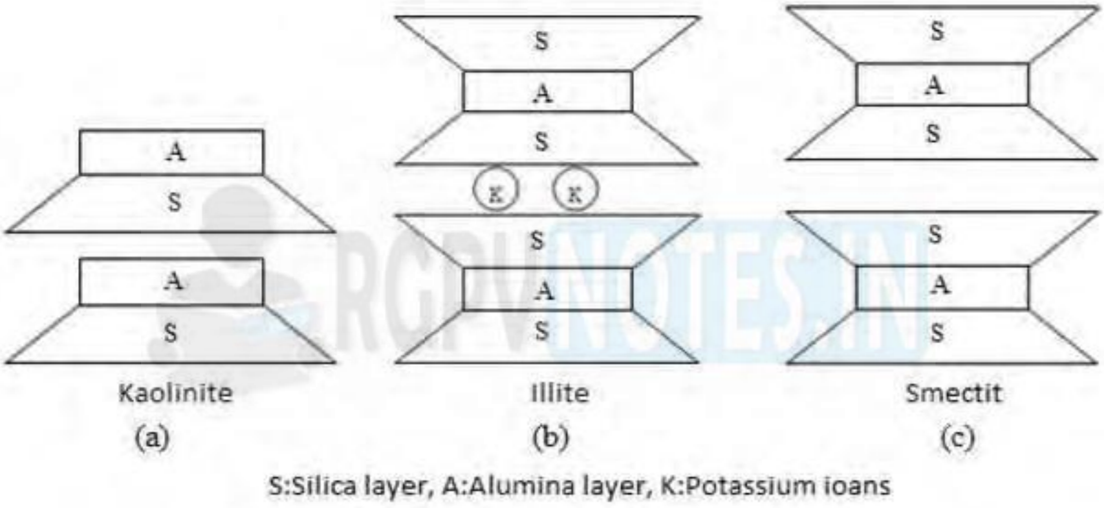

Fig: 1.1 Clay Minerals

- **Kaolinite**

Kaolinite is very common, highly weathered clay, and more common than Montmorillonite in acid soils. It has one part of silica and one part alumina plane per crystal; hence it is type clay. One plane of silica of Montmorillonite is dissolved and is replaced with hydroxyls, which produces strong hydrogen bonds to the oxygen in the next crystal of clay. As a result, Kaolinite does not swell in water and has a low specific surface area, and as almost no isomorphs substitution has occurred it has a low CEC. Where rainfall is high, acid soils selectively leach more silica than alumina from the original clays, leaving Kaolinite. Even heavier weathering results in sesquioxide clays.

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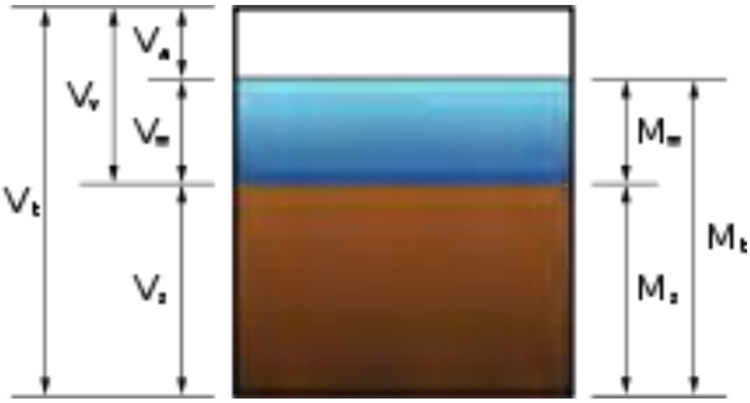
- **Soil structure & Three phase system:**



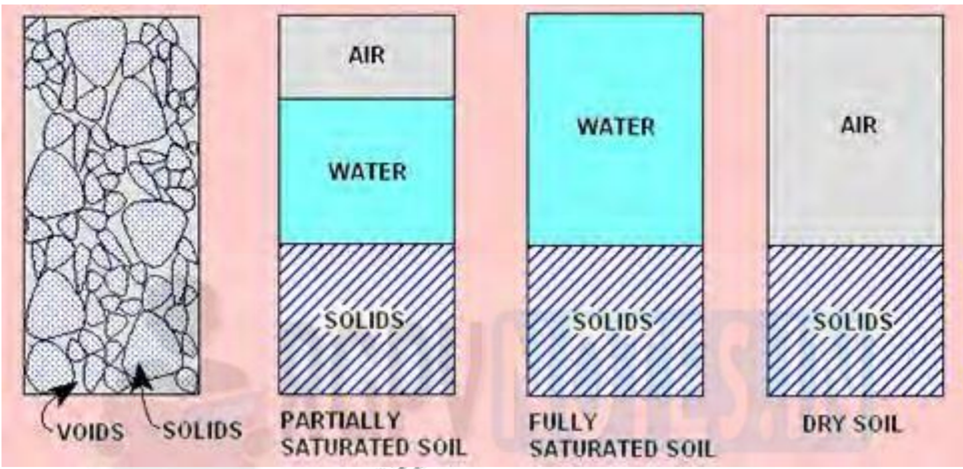


Fig: 1.2 Three Phase System of Soil

For the purpose of engineering analysis and design, it is necessary to express relations between the weights and the volumes of the three phases. The various relations can be grouped into:

- **Volume relations**
- **Weight relations**
- **Inter-relations**

As the amounts of both water and air are variable, the volume of solids is taken as the reference quantity. Thus, several relational volumetric quantities may be defined. The following are the basic volume relations:

- **Void ratio (e)**

Void ratio is the ratio of the volume of voids (Vv) to the volume of soil solids (Vs)

$$e = \frac{Vv}{Vs} x100$$

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- **Porosity (n)**

Porosity (n) is the ratio of the volume of voids to the total volume of soil (V), and is expressed as a percentage.

$$n = \frac{Vv}{V} x100$$

Void ratio and porosity are inter-related to each other as follows:

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

- **Degree of saturation (S)**

The volume of water (V_w) in a soil can vary between zero (i.e. a dry soil) and the volume of voids. This can be expressed as the degree of saturation (S) in percentage.

$$S = \frac{Vw}{Vv} x100$$

- **Air Content (ac)**

Air content (ac) is the ratio of the volume of air (V_a) to the volume of voids.

$$a_c = \frac{Va}{Vv} x100$$

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➢ **<u>Index properties of Soil and their determination:</u>**

Index properties are the properties of soil that help in identification and classification of soil. These properties are generally determined in the laboratory. In situ density and relative density require undisturbed sample extraction while other quantities can be determined from disturbed soil sampling. Following are the major properties of soils:

- **Water Content (w)**
- **Unit weight of Soil/ In-situ density (γ)**
- **Specific Gravity (G)**
- **Consistency Limits**
- **Particle Size Distribution**
- **Sensitivity and activity of Clays**

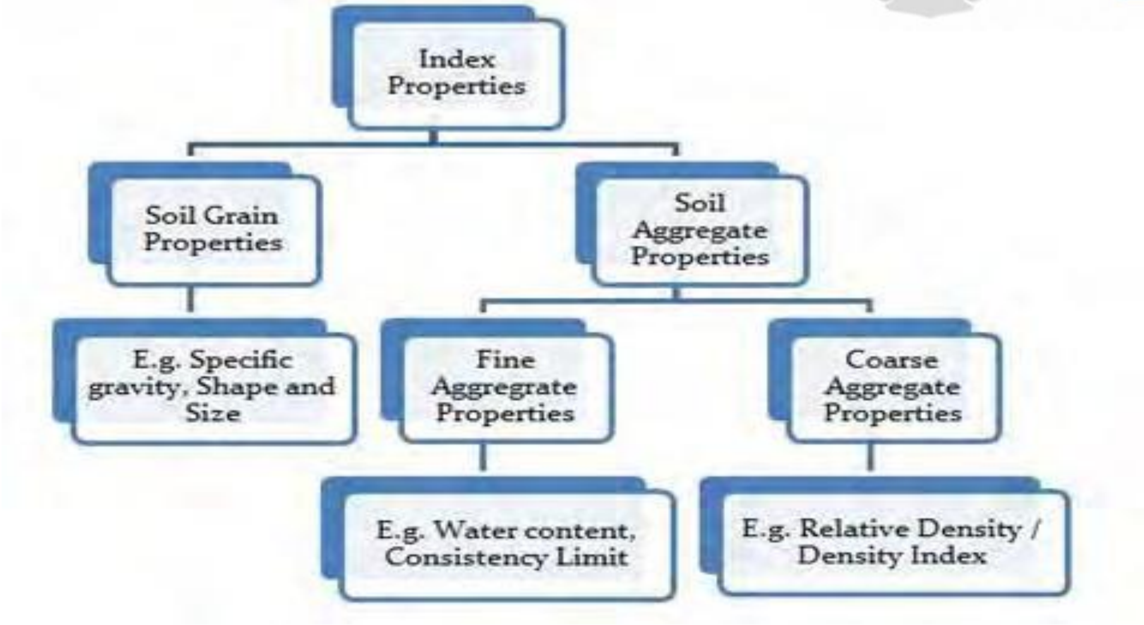

Fig: 1.3 Index Properties of Soil

Soil index properties are properties which facilitate identification and classification of soils for engineering purposes. The nature of some properties differs for coarse- and fine-grained soils.

Coarse-grained (non-cohesive) soil index properties are:

- **Particle-size distribution**
- **Particle shape**
- **Relative density**
- **Consistency**
- **Clay and clay minerals content**

Fine-grained (cohesive) soil index properties are:

- **Consistency**
- **Clay and clay minerals content**
- **Water content**

One of soil index properties which describe non-cohesive soils is particle size distribution. Soil that contains wide range of particle sizes is named well-graded. The opposite type of soil, which contains narrow range of particle sizes, is categorized as poorly graded. Well-graded soils can be more densely packed. Particle shape also influences how closely particles can be packed together. The density of soil (especially of coarse-grained) is the indication of strength and stiffness. The relative density is the ratio of the actual bulk density and the maximum possible density of the soil. Relative density is a good

indicator of potential increases in density, and thus deformations that may occur under the different loads.

Consistency is the resistance of soils to deformation and rupture. The unconfined compression strength is often used as an indication of consistency. In practice, the terms soft, medium, stiff, very stiff, and hard are applied to rate consistency of soil. This soil index property describes both cohesive and non-cohesive soils. Consistency at non-cohesive soil depends primarily on particle shape and size distribution, while at cohesive soils this property primarily depends on water content.

Clay and clay minerals content is important soil index characteristic for both coarse- and fine-grained soils. Clay minerals are fine-sized platy silicates which are highly plastic. Therefore, depending on percentage and type of clay minerals, clayey soils are less or more plastic.

Water content is very important soil index property of fine-grained soils since their behavior largely changes with water concentration variations. According to Atterberg's there are four states: liquid, plastic, semi-solid and solid. Marginal water contents that separate these states are known as Atterberg limits and these are: shrinkage (SL), plastic (PL) and liquid limit (LL). These limits have different values for different types of fine-grained soils.

The Unified Soil Classification gives each soil type a two-letter designation. For coarse grained soils, the first letter, either G for gravel or S for sand, refers to the dominant particle size in the soil. The second letter is either W, for well graded or P, for poorly graded. The second letter can also be M for silt or C for clay if coarse-grained soils contain more than 12% of silt or clay. The first letter of the designation for fine-grained soils is M or C (silt or clay). The second letter, either H (high) or L (low), refers to the plasticity of the soil.

- **Consistency limits:**

The consistency of a fine-grained soil refers to its firmness, and it varies with the water content of the soil. A gradual increase in water content causes the soil to change from solid to semi-solid to plastic to liquid states. The water contents at which the consistency changes from one state to the other are called consistency limits (or Atterberg limits).

Two of these are utilized in the classification of fine soils:

- Liquid limit (W_L) - change of consistency from plastic to liquid state.
- Plastic limit (W_P) - change of consistency from brittle/crumbly to plastic state.

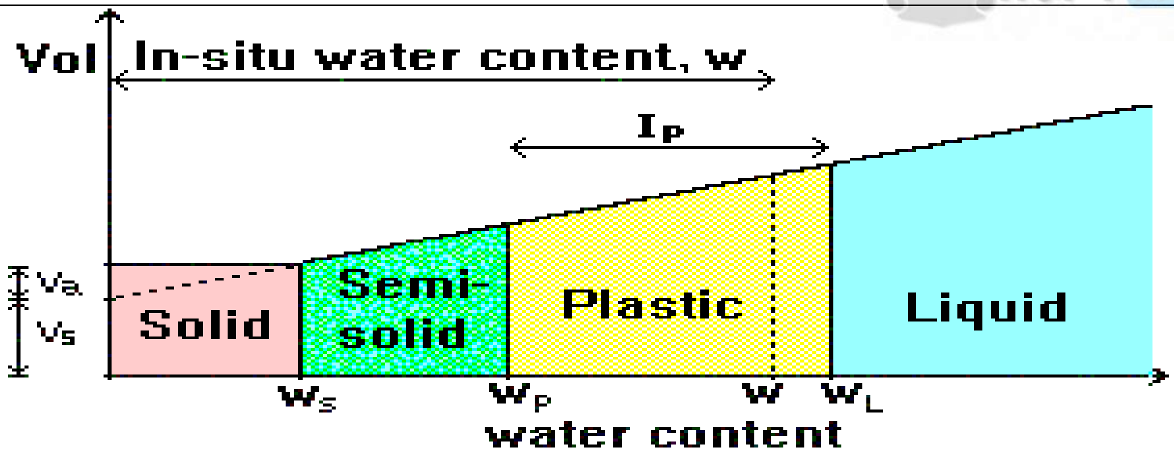

Fig: 1.4 Consistency Limits

The difference between the liquid limit and the plastic limit is known as the plasticity index (I_P), and it is in this range of water content that the soil has a plastic consistency. The consistency of most soils in the field will be plastic or semi-solid.

The three limits are known as the shrinkage limit (WS), plastic limit (WP), and liquid limit (WL) as shown. The values of these limits can be obtained from laboratory tests.

- **Classification Based on Grain Size:**

The range of particle sizes encountered in soils is very large: from boulders with dimension of over 300 mm down to clay particles that are less than 0.002 mm. Some clay contains particles less than 0.0021 mm in size which behave as colloids, i.e. do not settle in water.

In the Indian Standard Soil Classification System (ISSCS), soils are classified into groups according to size, and the groups are further divided into coarse, medium and fine sub-groups.

The grain-size range is used as the basis for grouping soil particles into boulder, cobble, gravel, sand, silt or clay.

Table 1.1 Classification of soil based on Grain Size

Very coarse soils	Boulder size		> 300 mm
	Cobble size		80 - 300 mm
Coarse soils	Gravel size (G)	Coarse	20 - 80 mm
		Fine	4.75 - 20 mm
	Sand size (S)	Coarse	2 - 4.75 mm
		Medium	0.425 - 2 mm
		Fine	0.075 - 0.425 mm
Fine soils	Silt size (M)		0.002 - 0.075 mm
	Clay size (C)		< 0.002 mm

- **Indian Standard Soil Classification System (ISSCS):**

According to this system, the symbols of the various soils are as: Gravel (G), Sand (S), Silt or Silty (M), Clay or Clayey (C), Organic (O), Peat (Pt), well graded (W), poorly graded (P). To classify the fine-grained soil, plasticity chart (as shown in Figure 1.5) is used. The difference between the plasticity charts used for Unified Soil Classification System (USCS) and Indian Standard Soil Classification System (ISSCS) is that in USCS, the soil is classified as High Plasticity (if liquid limit >50%) or Low Plasticity (if liquid limit < 50%) soil, but in ISSCS, the soil is classified as High Plasticity (if liquid limit >50%) or Intermediate Plasticity (if liquid limit is in between 35% to 50%) or Low Plasticity (if liquid limit < 35%). For example, if a soil sample has liquid limit (w_L) 45% and plasticity index (I_P) 25, according to the Unified Soil Classification System (USCS) the point is above 'A' line (point A in Figure 1.5) and it is classified as CL. However, according to Indian Standard Soil Classification System (ISSCS) the point is also above 'A' line (point B in Figure 1.5), but it is classified as CI.

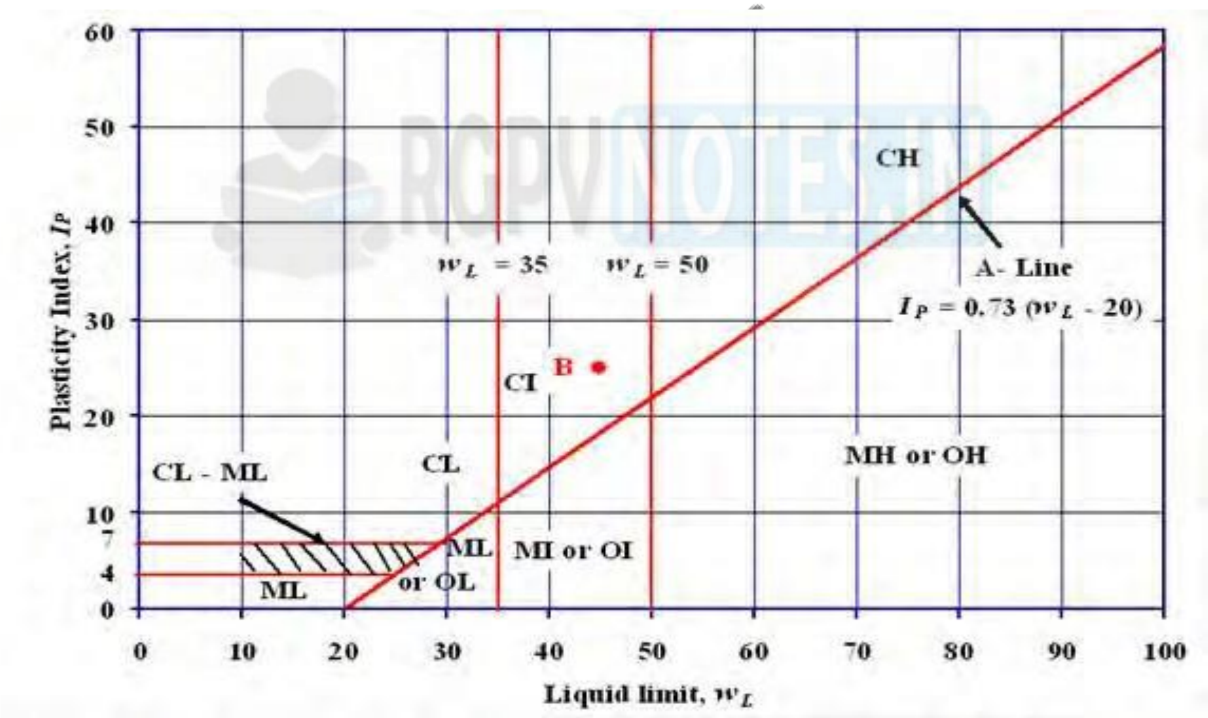

Fig: 1.5 Plasticity chart as per Indian Standard Soil Classification System (ISSCS)

The soil is called fine-grained soil if 50% or more soil is passed through 0.075 mm sieve. The fine-grained soils are classified based on plasticity chart (as shown in Figure 1.5). The soil has low plasticity (CL: Clay with low plasticity, ML: Silt with low plasticity) if the liquid limit of the soil is less than 50% and if the liquid limit of the soil is greater than 50% the soil has high plasticity (CH: Clay with high plasticity, MH:

Silt with high plasticity). However, more than one group can be termed as boundary soils (like GW-GM: Well graded gravel mixed with silt).

- **AASHTO Soil Classification System:**

According to the AASHTO soil classification system, the soils are classified based on the Group Index (*GI*) value which can be calculated as:

$$GI = 0.2\,a + 0.005\,ac + 0.01\,bd$$

Where,

a is that part of the percent passing through the 75 m (0.075 mm) sieve greater than 35 and not exceeding 75, expressed as a positive whole number (range 1 to 40).

b is that part of the percent passing through the 75 m (0.075 mm) sieve greater than 15 and not exceeding 55, expressed as a positive whole number (range 1 to 40).

c is that part of liquid limit greater than 40 and not exceeding 60, expressed as a positive whole number (range 1 to 20).

d is that part of plasticity index greater than 10 and not exceeding 30, expressed as a positive whole number (range 1 to 20).

The group index should be rounded off to the nearest whole number. If the calculated group index value is negative, then it is taken as zero. A group index value equal to zero indicates a good sub grade material, whereas group index value equal to or greater than 20 indicates a very poor sub grade material.

Unit – II

Soil Water and Consolidation: Soil water, Permeability Determination of permeability in laboratory and in field, Seepage and seepage pressure, Flow-nets, uses of a flow-net, Effective, neutral and total stresses, Compressibility and consolidation, Relationship between pressure and void ratio, Theory of one dimensional consolidation, Consolidation test, Fitting Time curves, Normally and over consolidated clays, Determination of pre-consolidation pressure, settlement analysis, Calculation of total settlement.

➢ Soil water:

The soil water content is the amount of water held in the soil at any given time and can be expressed as volumetric or gravimetric water content. Volumetric water content is the ratio of volume of water per unit volume of dry soil and is the most useful way of expressing water content for developing a water budget. Gravimetric water content is the mass of water per unit mass of dry soil. Soil water is the term for water found in naturally occurring soil. There are four main types of soil water-

- Gravitational Water or Ground Water
- Capillary Water
- Hygroscopic Water and
- Chemically Combined Water

- **Gravitational Water:**

Gravitational water is free water moving through soil by the force of gravity. It is largely found in the macro pores of soil and very little gravitational water is available to plants as it drains rapidly down the water table in all type of soil except the most compact of soils.

- **Capillary Water:**

Capillary water is water held in the microspores of the soil, and is the water that composes the soil solution. Capillary water is held in the soil because the surface tension properties (cohesion and adhesion) of the soil microspores are stronger than the force of gravity. However, as the soil dries out, the pore size increases and gravity starts to turn capillary water into gravitational water and it moves down. Capillary water is the main water that is available to plants as it is trapped in the soil solution right next to the roots if the plant.

- **Hygroscopic Water:**

Hygroscopic water forms as a very thin film surrounding soil particles and is generally not available to the plant. This type of soil water is bound so tightly to the soil by adhesion properties that very little of it can be taken up by plant roots. Since hygroscopic water is found on the soil particles and not in the pores, certain types of soils with few pores (clays for example) will contain a higher percentage of it.

- **Chemically Combined Water:**

The water chemically combined in the structure of soil minerals is known as combined water. After the elimination of hygroscopic water by heating soil to about 150°C., the only water that remains is in the hydrated oxides of aluminum, iron, silicon, etc. This water is absolutely unavailable to the plants and can only be driven off from the soil by resorting to very high temperature but not before bringing about irreversible changes in the physical and chemical composition of the soil itself.

Permeability:

The ability of soil to allow flow of water through it is called as permeability of soil. It is very important factor for the structures which are in contact with water. Flow of water in soil takes place through void spaces, which are interconnected. Water does not flow in a straight line, but in a winding path. However in soil mechanics flow is considered to be in a straight line at an effective velocity. The velocity of flow depends on size of pores. The soil permeability is a measure indicating the capacity of the soil or rock to allow fluids to pass through it. It is often represented by the permeability coefficient (k) through the Darcy's equation:

$$V = Ki$$

Where v is the apparent fluid velocity through the medium i is the hydraulic gradient, and K is the coefficient of permeability (hydraulic conductivity) often expressed in m/s, K depends on the relative permeability of the medium for fluid constituent (often water) and the dynamic viscosity of the fluid as follows. The ratio of permeability's of typical sands/gravels to those of typical clays is of the order of 10^6. A small proportion of fine material in a coarse-grained soil can lead to a significant reduction in permeability.

Constant head permeability test:

The constant head permeability test is a laboratory experiment conducted to determine the permeability of soil. The soils that are suitable for this test are sand and gravels. Soils with silt content cannot be tested with this method. The test can be employed to test granular soils either reconstituted or disturbed. Constant head permeameter is recommended for coarse-grained soils only since for such soils, flow rate is measurable with adequate precision. As water flows through a sample of cross-section area A, steady total head drop h is measured across length L. The objective of constant head permeability test is to determine the coefficient of permeability of a soil. Coefficient of permeability helps in solving issues related to:

- Yield of water bearing strata
- Stability of earthen dams
- Embankments of canal bank
- Seepage in earthen dams
- Settlement Issues

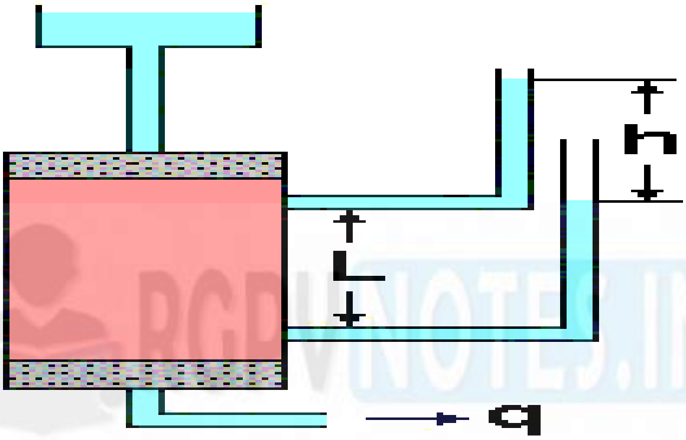

Fig 2.1 Constant head permeability test

The coefficient of permeability, k is defined as the rate of flow of water under laminar flow conditions through a porous medium area of unit cross section under unit hydraulic gradient. The coefficient of permeability (k) is obtained from the relation:

$$k = \frac{qL}{Ah} = \frac{QL}{Aht}$$

Where q= discharge, Q=total volume of water, t=time period, h=head causing flow, L= length of specimen, A= cross-sectional area.

- <u>Falling head permeability test:</u>

The falling head permeability test involves flow of water through a relatively short soil sample connected to a standpipe which provides the water head and also allows measuring the volume of water passing through the sample. The diameter of the standpipe depends on the permeability of the tested soil. The test can be carried out in a Falling Head permeability cell or in an oedometer cell. Falling head permeameter is recommended for fine-grained soils. Total head h in standpipe of area A is allowed to

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fall. Hydraulic gradient varies with time. Heads h_1 and h_2 are measured at times t_1 and t_2. At any time t, flow through the soil sample of cross-sectional area A is

$$q = k.h.\frac{A}{L} \quad \text{.............. (1)}$$

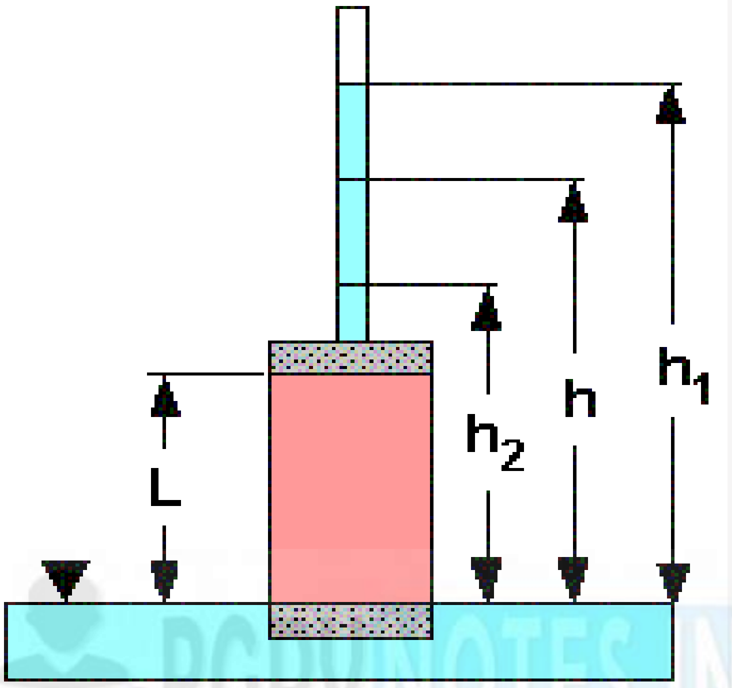


Fig 2.2 Falling head permeability test

Flow in unit time through the standpipe of cross-sectional area a is = a x (-dh/dt)................ (2)

Equating (1) and (2)

$$-a.\frac{dh}{dt} = k.h.\frac{A}{L}$$

$$-\frac{dh}{h} = \left(\frac{kA}{La}\right)dt$$

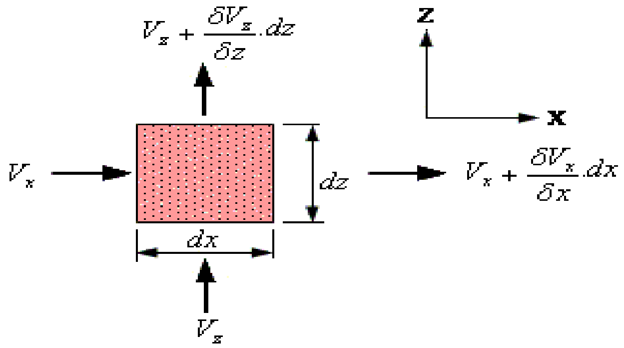
Integrating between the limits

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

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

$$= \frac{2.3L.a\log_{10}\left(\frac{h_1}{h_2}\right)}{A(t_2 - t_1)}$$

Seepage Velocity:



A rectangular soil element is shown with dimensions dx and dz in the plane, and thickness dy perpendicular to this plane. Consider planar flow in to the rectangular soil element. In the x-direction, the net amount of the water entering and leaving the element is

$$\frac{\delta V_x}{\delta x}.dx.dy.dz$$

Similarly in the z-direction, the difference between the water inflow and outflow is

$$\frac{\delta V_z}{\delta z}.dz.dx.dy$$

For a two-dimensional steady flow of pore water, any imbalance in flows into and out of an element in the z-direction must be compensated by a corresponding opposite imbalance in the x-direction. Combining the above and dividing by dx.dy.dz, the continuity equation is expressed as

$$\frac{\delta V_x}{\delta x} + \frac{\delta V_z}{\delta z} = 0$$

From Darcy's law,

$$V_x = k_x . \frac{\delta h}{\delta x}$$

$$V_z = k_z . \frac{\delta h}{\delta z}$$

Where h is the head causing flow.

Flow-nets & uses of a flow-net:

A flow-net is a graphical representation of two-dimensional steady-state groundwater flow through aquifers. Construction of a flow-net is often used for solving groundwater 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 analyzing two-dimensional irrotational flow problems. Flow net technique is a graphical representation method.

A Flow net is a graphical representation of flow of water through a soil mass. It is a curvilinear net formed by the combination of flow lines and equipotential lines. Properties and application of flow net are explained in this article. Flow lines represent the path of flow along which the water will seep through the soil. Equipotential lines are formed by connecting the points of equal total head.

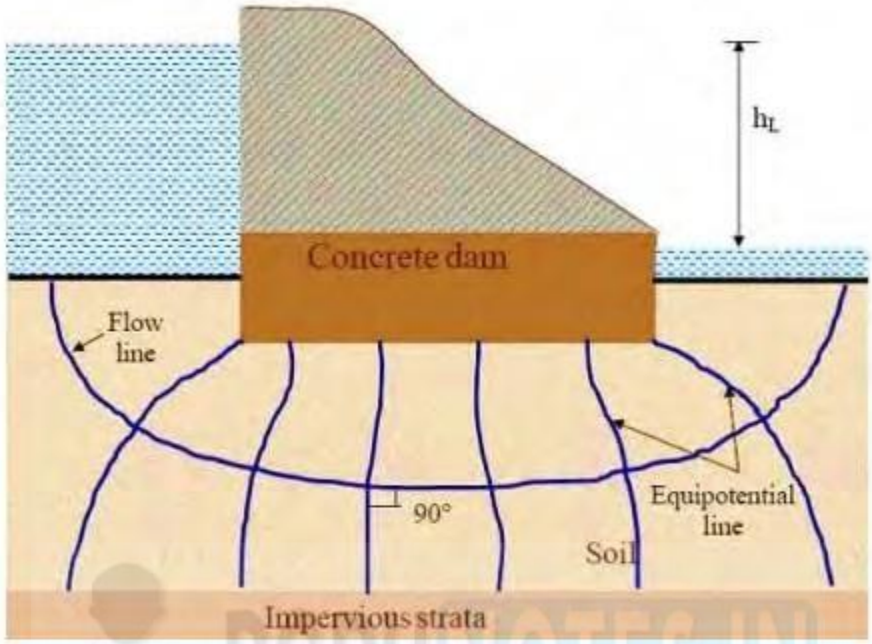


Fig 2.3 Flow Net

Properties of flow net are as follows:

- The angle of intersection between each flow line and an equipotential line must be 90o which means they should be orthogonal to each other.
- Two flow lines or two equipotential lines can never cross each other.
- Equal quantity of seepage occurs in each flow channel. A flow channel is a space between two flow lines.
- Head loss is the same between two adjacent potential lines.
- Flow nets are drawn based on the boundary conditions only. They are independent of the permeability of soil and the head causing flow.
- The space formed between two flow lines and two equipotential lines is called a flow field. It should be in a square form.
- Either flow lines or equipotential lines are smoothly drawn curves.

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Applications of Flow Net:

Flow net is useful to determine the following parameters in seepage analysis of soil:

- **Rate of Seepage loss**
- **Seepage Pressure**
- **Uplift Pressure**
- **Exit Gradient**

- **Rate of Seepage loss (Q)**

Using flow net, the rate of seepage loss or seepage quantity can be determined using the below expression:

$$Q = k.H.\frac{N_f}{N_d}.\frac{\Delta B}{\Delta L}$$

Where,
k = coefficient of permeability

H = Head causing flow

N_f = Number of flow lines

N_d = Number of Equipotential lines

ΔB = width of flow field

ΔL = Length of flow field

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- **Seepage Pressure (P_s):**

Seepage pressure at any point is determined by using the below mentioned formula:

$$P_s = Y_w.h$$

Where, Y_w = Unit weight of water

h = Hydraulic potential after "n" potential drops. It can be expressed as:

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

$$\Delta H = \frac{H}{N_d}$$

Where,

ΔH = Potential drop or Drop in head between two adjacent equipotential lines.

- **Uplift Pressure (Pu)**

The uplift pressure at any point within the soil mass can be found using the under mentioned formula. It is also called as hydrostatic pressure.

$$P_u = Y_w.h_w$$

Where,

Y_w = Unit weight of water

h_w = Piezometric head or Pressure head = total head – Elevation head

$$h_w = h \pm z$$

- **Exit Gradient (i_{exit}):**

The exit gradient is the hydraulic gradient at the downstream end of flow line where seepage water from the soil mass joins with free water at the downstream. Exit gradient can be expressed as:

$$i_{exit} = \frac{\Delta H}{\Delta L}$$

Where,

ΔL = Length of flow field

ΔH = Potential Drop

- **Effective, Neutral and Total stresses:**

As mention before, the soil consists of the solid particles which distributed randomly with void spaces in between, the void spaces are occupied by water and/or air. We need to known the cross section of soil profile under Ground Surface (GS) or (G.L) ground level to calculate the stresses on the element. It means the column of the soil weight in the nature case. There is many types of stresses the total stress, natural stress and effective stress.

$$S_T = \gamma h$$

S_T : Total stress at point A

γ : Unit weight of soil

h : The high of the soil column

For Multilayer soil case

$$S_T = \gamma_1 h_1 + \gamma_2 h_2 + \gamma_3 h_3$$

The stress has two components:

The stress due to pore pressure (u); u = h.γ_w where, γ_w = density of water h = depth (ft)

Stress due to the weight of the rectangle; σ = h.γ_w

The principle effective stress is $\sigma' = \sigma - u$ the principle of effective stress is the most important principle in soil mechanics. Deformations of soils are a function of effective stresses not total stresses. The principle of effective stresses applies only to normal stresses and not shear stresses.

Total stress (σ) is equal to the sum of effective stress (σ') and pore water pressure (u) or, alternatively, effective stress is equal to total stress minus pore water pressure.

Now assume the block is sand, what if the ground water level is at a depth hw below the ground level,

$$\sigma = \gamma.h_w + \gamma_{sat}(h - h_w)$$

The pore pressure is $u = \gamma_w (h - h_w)$

So the effective stress is; $\sigma' = \sigma - u = \gamma.h_w + \gamma_{sat}(h - h_w) - \gamma_w(h - h_w)$

- **Compressibility and Consolidation:**

Consolidation is a process by which soils decrease in volume. According to Karl von Terzaghi "consolidation is any process which involves a decrease in water content of saturated soil without replacement of water by air." In general it is the process in which reduction in volume takes place by expulsion of water under long term static loads. It occurs when stress is applied to a soil that causes the soil particles to pack together more tightly, therefore reducing its bulk volume. When this occurs in a soil that is saturated with water, water will be squeezed out of the soil. The magnitude of consolidation can be predicted by many different methods. In the Classical Method, developed by Terzaghi, soils are tested with an odometer test to determine their compression index. This can be used to predict the amount of consolidation.

When stress is removed from a consolidated soil, the soil will rebound, regaining some of the volume it had lost in the consolidation process. If the stress is reapplied, the soil will consolidate again along a recompression curve, defined by the recompression index. The soil which had its load removed is considered to be over consolidated. This is the case for soils which have previously had glaciers on them. The highest stress that it has been subjected to is termed the pre consolidation stress. The over consolidation ratio or OCR is defined as the highest stress experienced divided by the current stress. A soil which is currently experiencing its highest stress is said to be normally consolidated and to have an OCR of one. A soil could be considered under consolidated immediately after a new load is applied but before the excess pore water pressure has had time to dissipate.

When a soil layer is subjected to vertical stress, volume change can take place through rearrangement of soil grains, and some amount of grain fracture may also take place. The volume of soil grains remains constant, so change in total volume is due to change in volume of water. In saturated soils, this can happen only if water is pushed out of the voids. The movement of water takes time and is controlled by the permeability of the soil and the locations of free draining boundary surfaces.

It is necessary to determine both the magnitude of volume change (or the settlement) and the time required for the volume change to occur. The magnitude of settlement is dependent on the magnitude of applied stress, thickness of the soil layer, and the compressibility of the soil.

When soil is loaded untrained, the pore pressure increases. As the excess pore pressure dissipates and water leaves the soil, settlement takes place. This process takes time, and the rate of settlement decreases over time. In coarse soils (sands and gravels), volume change occurs immediately as pore pressures are dissipated rapidly due to high permeability. In fine soils (silts and clays), slow seepage occurs due to low permeability.

- **Relationship between pressure and void ratio**

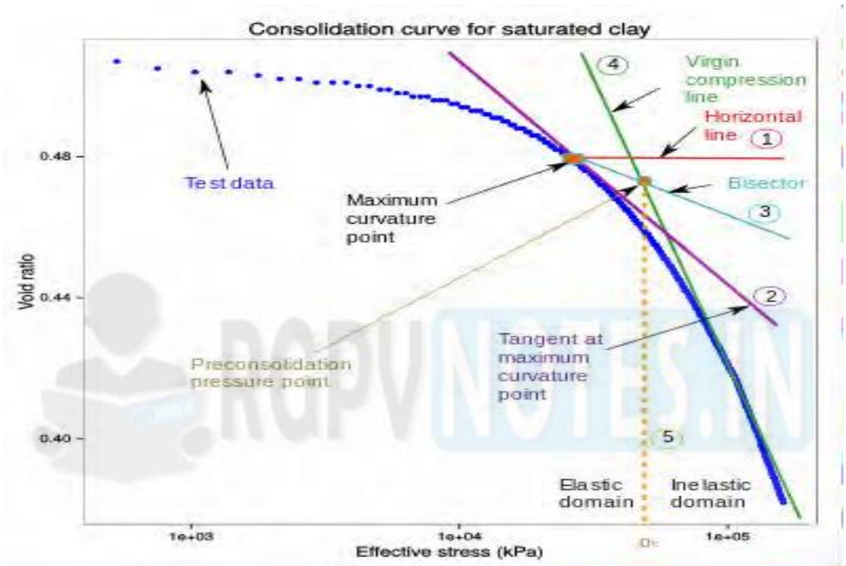

Fig 2.4 Relationship between pressure and void ratio

- <u>**Theory of one dimensional consolidation**</u>

The process of consolidation is often explained with an idealized system composed of a spring, a container with a hole in its cover, and water. In this system, the spring represents the compressibility or the structure of the soil itself, and the water which fills the container represents the pore water in the soil.

- **The container is completely filled with water, and the hole is closed. (Fully saturated soil)**
- **A load is applied onto the cover, while the hole is still unopened. At this stage, only the water resists the applied load. (Development of excess pore water pressure)**
- **As soon as the hole is opened, water starts to drain out through the hole and the spring shortens. (Drainage of excess pore water pressure)**
- **After some time, the drainage of water no longer occurs. Now, the spring alone resists the applied load. (Full dissipation of excess pore water pressure. End of consolidation)**

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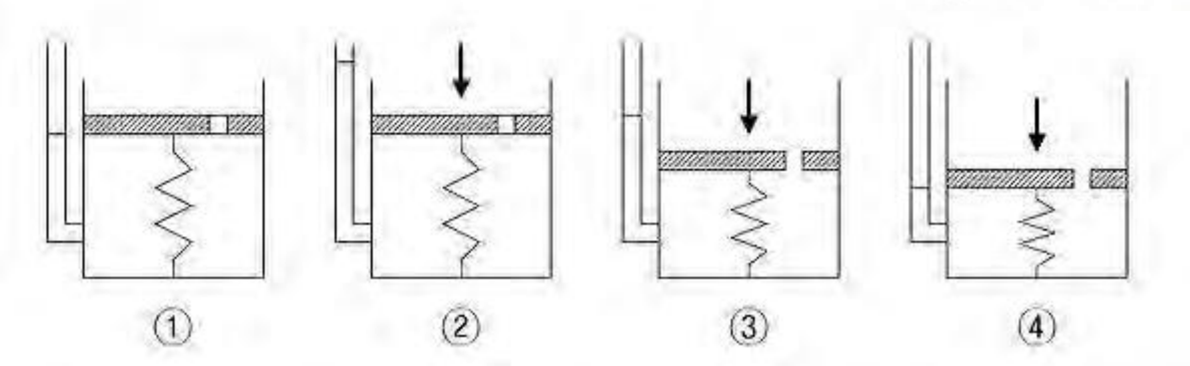

Fig 2.5 Consolidation test setup

Consolidation test & Fitting Time curves

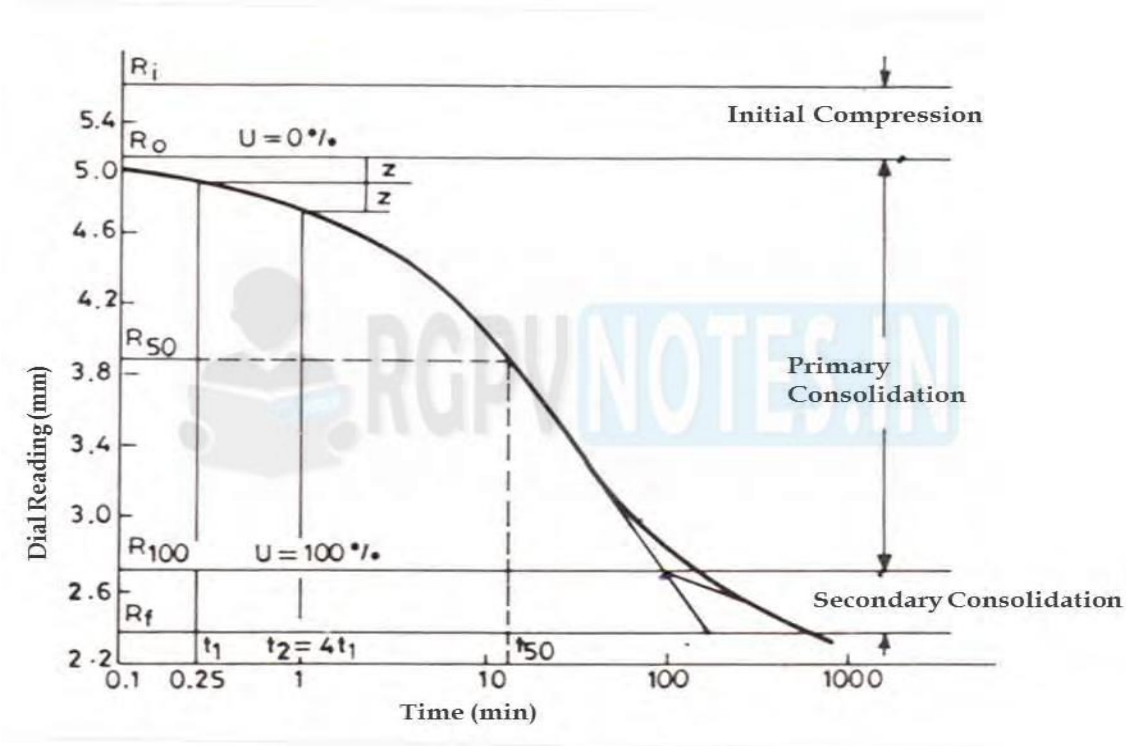

Fig 2.6 Fitting Time curves

After the coefficient of consolidation (c_v) has been determined from laboratory data calculations are possible for site settlements. It is important to note that c_v is not a constant, but varies with both the level of stress and degree of consolidation. For practical site settlement calculations, however, it is sufficiently accurate to measure c_v relative to the loading range applicable on site and then assume this value to be approximately constant for all degrees of consolidation

The basic equation used is:

$$c_v = \frac{T_v \, d^2}{t}$$

- **<u>Normally and over consolidated clays:</u>**

Normally consolidated clays are these that are currently experiencing the maximum vertical overburden effective pressure they have ever experienced in their history. Over consolidated clays had experienced a higher overburden stress in the past. The pre-consolidation stress's is defined to be the maximum effective stress experienced by the soil. This stress is identified in comparison with the effective stress in its present state. For soil at state Q or F, this would correspond to the effective stress at point P.

If the current effective stress, s', is equal (note that it cannot be greater than) to the pre-consolidation stress, then the deposit is said to be normally consolidated (NC). If the current effective stress is less than the pre-consolidation stress, then the soil is said to be over-consolidated (OC). OP corresponds to initial loading of the soil. PQ corresponds to unloading of the soil. QFR corresponds to a reloading of the soil. Upon reloading beyond P, the soil continues along the path that it would have followed if loaded from O to R continuously.

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

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

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

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

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

$$C_C = \frac{\Delta e}{\log_{10}\left(\frac{\sigma_2'}{\sigma_1'}\right)}$$

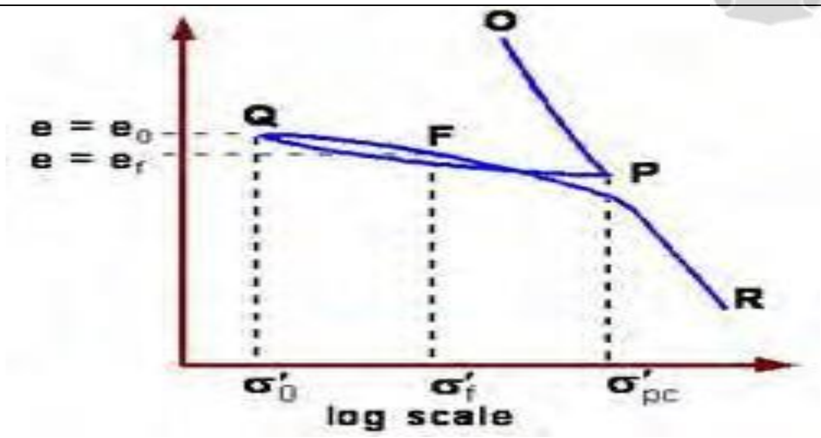

Fig 2.7 Curve between stresses and void ratio

- **Settlement analysis & Calculation of total settlement:**

The total settlement of a loaded soil has three components: Elastic settlement, primary consolidation, and secondary compression. Elastic settlement is on account of change in shape at constant volume, i.e. due to vertical compression and lateral expansion. Primary consolidation (or simply consolidation) is on account of flow of water from the voids, and is a function of the permeability and compressibility of soil. Secondary compression is on account of creep-like behavior.

Primary consolidation is the major component and it can be reasonably estimated. A general theory for consolidation, incorporating three-dimensional flow is complicated and only applicable to a very limited range of problems in geotechnical engineering. For the vast majority of practical settlement problems, it is sufficient to consider that both seepage and strain take place in one direction only, as one-dimensional consolidation in the vertical direction.

The components of settlement of a foundation are:

- **Immediate settlement**
- **Consolidation Settlement, and**
- **Secondary compression (creep)**

$$\Delta H = \Delta Hi + U\ \Delta Hc + \Delta Hs$$

ΔH = total settlement, ΔHc = consolidation settlement, ΔH = secondary compression, U = average degree of consolidation. Generally, the final settlement of a foundation is of interest and U is considered equal to 1 (i.e. 100% consolidation)

- **Immediate Settlement:**

  - **Immediate settlement takes place as the load is applied or within a time period of about 7 days.**
  - **Predominates in cohesion less soils and unsaturated clay**
  - **Immediate settlement analysis are used for all fine-grained soils including silts and clays with a degree of saturation < 90% and for all coarse grained soils with large co-efficient of permeability (say above 10.2 m/s)**

- **Consolidation Settlement (ΔHc):**

  - **Consolidation settlements are time dependent and take months to years to develop. The leaning tower of Pisa in Italy has been undergoing consolidation settlement for over 700 years. The lean is caused by consolidation settlement being greater on one side. This, however, is an extreme case. The principal settlements for most projects occur in 3 to 10 years.**
  - **Dominates in saturated/nearly saturated fine grained soils where consolidation theory applies. Here we are interested to estimate both consolidation settlement and how long a time it will take or most of the settlement to occur.**

- **Secondary Settlement/Creep (ΔHs):**

  - **It occurs under constant effective stress due to continuous rearrangement of clay particles into a more stable configuration.**
  - **Predominates in highly plastic clays and organic clays.**

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Unit – III

STRESS DISTRIBUTION IN SOILS AND COMPACTION: Boussinesq's and Westergard's theories for point load, Uniformly loaded circular and rectangular areas, Pressure bulb, Variation of vertical stress under point load along vertical and horizontal plane, Newmark's influence chart for irregular areas. Contact pressure distribution in sands and clays. Mechanism importance of compaction, Standard Proctor compaction test, Modified compaction test Factors affecting compaction, Effects of compaction on soil properties, Field compaction equipment and compaction quality control.

Stress distribution in soils:

Stress in the soil is caused by the following:

- Self-Weight of soil
- Structural loads, applied at or below the surface

Many problems in foundation engineering require a study of the transmission and distribution of stresses in large and extensive masses of soil. Some examples are wheel loads transmitted through embankments to culverts, foundation pressures transmitted to soil strata below footings, pressures from isolated footings transmitted to retaining walls, and wheel loads transmitted through stabilized soil pavements to sub-grades below. In such cases, the stresses are transmitted in all downward and lateral directions. Estimation of vertical stresses at any point in a soil mass due to external loading is essential to the prediction of settlements of buildings, bridges, and embankments. The theory of elasticity, which gives primarily the interrelationships of stresses and strains, has been the basis for the determination of stresses in a soil mass. According to the elastic theory, constant ratios exist between stresses and strains. For the theory to be applicable, the real requirement is not that the material necessarily is elastic but that there must be constant ratios between stresses and the corresponding strains.

It is known that, only at relatively small magnitudes of stresses, the proportionality between strains and stresses exists in the case of the soil. Fortunately, the order of magnitudes of stresses transmitted into soil from structural loadings is also small and hence the application of the elastic theory for the determination of stress distribution in soil gives reasonably valid results.

The most widely used theories regarding the distribution of stress in soil are those of Boussinesq's and Westergard's. They have developed first for point loads and later, the values for point load have been integrated to give stresses below uniform strip loads, uniformly loaded circular and rectangular areas.

If there are imposed structural loadings also on the soil, the resultant stress may be obtained by adding algebraically the stress due to self-weight and stress transmitted due to structural loadings.

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➢ **Boussinesq's Theory:**

Boussinesq's (1885) has given the solution for the stresses caused by the application of a point load at the surface of a homogeneous, elastic, isotropic, and semi-infinite medium, with the aid of the mathematical theory of elasticity. A semi-infinite medium is one bounded by a horizontal boundary plane, which is the ground surface for soil medium. The following is an exhaustive list of assumptions made by Boussinesq's in the derivation of his theory:

- **The soil medium is an elastic, homogeneous, isotropic, and semi-infinite medium, which extends infinitely in all directions from a level surface. (Homogeneity indicates identical properties at all points in identical directions, while isotropy indicates identical elastic properties in all directions at a point).**
- **The medium obeys Hooke's law.**
- **The self-weight of the soil is ignored.**
- **The soil is initially unstressed.**
- **The change in volume of the soil upon application of the loads onto it is neglected.**
- **The top surface of the medium is free of shear stress and is subjected to only the point load at a specified location.**
- **Continuity of stress is considered to exist in the medium.**
- **The stresses are distributed symmetrically with respect to Z-axis**

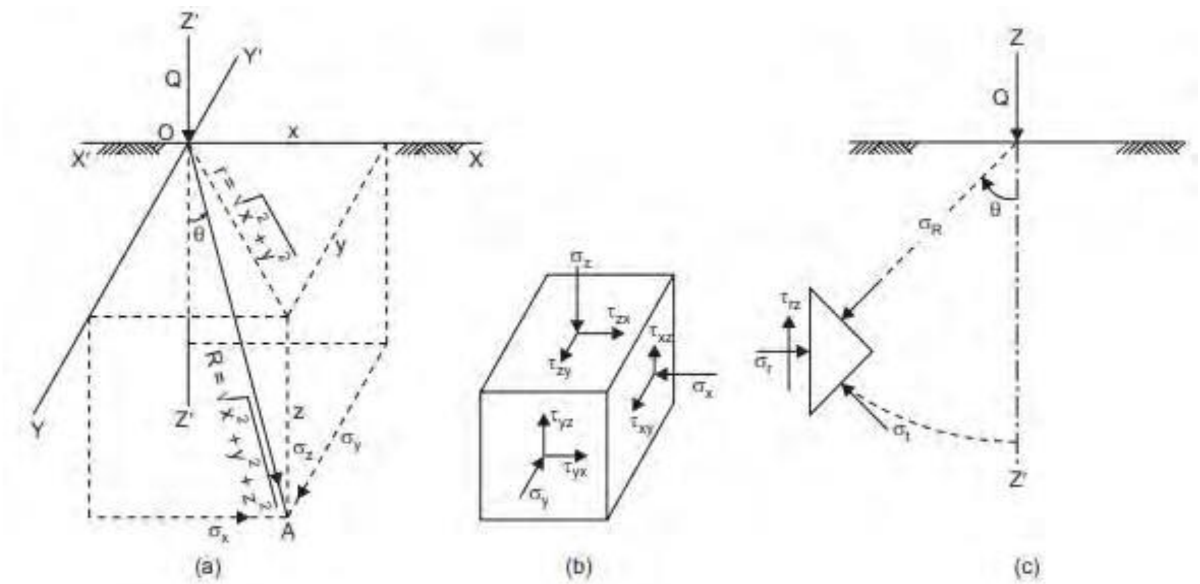

Fig 3.1 Boussinesq's analysis

The Boussinesq's equations are as follows:

In Fig. 3.1 (a), the origin of co-ordinates is taken as the point of application of the load Q and the location of any point A in the soil mass is specified by the co-ordinates x, y, and z. The stresses acting at point A

on planes normal to the co-ordinate axes are shown. In Fig. 3.1 (b). σ's are the normal stresses on the planes normal to the co-ordinate axes; τ's are the shearing stresses. The first subscript of τ denotes the axis normal to which the plane containing the shear stress is, and the second subscript indicates the direction of the axis parallel to which the shear stress acts. In Fig. 3.1 (c), the cylindrical co-ordinates and the corresponding normal stresses radial stress σ_r, tangential stress σ_t , and the shear stress τ_{rz} are shown; σ_z is another principal stress in the cylindrical co-ordinates; the polar radial stress σ_R is also shown.

Boussinesq's equations:

$$\sigma_z = \frac{3Q}{2\pi}\cdot\frac{z^3}{R^5}$$

$$= \frac{3Q}{2\pi}\cdot\frac{\cos^2\theta}{z^2}$$

$$= \frac{3Q}{2\pi}\cdot\frac{z^3}{(r^2+z^2)^{5/2}}$$

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

$$\sigma_x = \frac{Q}{2\pi}\left[\frac{3x^2z}{R^5} - (1-2\upsilon)\left\{\frac{x^2-y^2}{Rr^2(R+z)} + \frac{y^2z}{R^3r^2}\right\}\right]$$

$$\sigma_y = \frac{Q}{2\pi}\left[\frac{3y^2z}{R^5} - (1-2\upsilon)\left\{\frac{y^2-x^2}{Rr^2(R+z)} + \frac{x^2z}{R^3r^2}\right\}\right]$$

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

$$\sigma_r = \frac{Q}{2\pi}\left[\frac{3zr^2}{R^2} - \frac{(1-2\upsilon)}{R(R+z)}\right]$$

$$= \frac{Q}{2\pi}\left[\frac{3zr^2}{(r^2+z^2)^{5/2}} - \frac{(1-2\upsilon)}{r^2+z^2+z\sqrt{r^2+z^2}}\right]$$

$$= \frac{Q}{2\pi z^2}\left[3\sin^2\theta\cos^3\theta - \frac{(1-2\upsilon)\cos^2\theta}{(1+\cos\theta)}\right]$$

$$\sigma_t = -\frac{Q}{2\pi z^2}(1-2\upsilon)\left[\cos^3\theta - \frac{\cos^2\theta}{1+\cos\theta}\right]$$

$$= \frac{-Q}{2\pi}(1-2\upsilon)\left[\frac{z}{(r^2+z^2)^{3/2}} - \frac{1}{r^2+z^2+z\sqrt{r^2+z^2}}\right]$$

$$\tau_{rz} = \frac{3Q}{2\pi}\cdot\frac{rz^2}{R^5}$$

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

$$= (3Q/2\pi z^2).\ (\sin\theta\cos^4\theta)$$

Here υ is 'Poisson's ratio' of the soil medium.

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➢ Westergard's Theory:

Westergard's (1938) has obtained an elastic solution for stress distribution in soil under a point load based on conditions analogous to the extreme condition of this type. The material is assumed to be laterally reinforced by numerous, closely spaced horizontal sheets of negligible thickness but of infinite rigidity, which prevents the medium from undergoing lateral strain; this may be viewed as representative of an extreme case of the non-isotropic condition.

Natural clay strata have thin lenses of coarser material within them; this accentuates the no isotropic condition commonly encountered in sedimentary soils, which is the primary reason for resistance to lateral strain in such cases.

The vertical stress σ_z caused by a point load, as obtained by Westergard's, is given by:

$$\sigma_z = \frac{Q}{z^2} \cdot \frac{\frac{1}{2\pi}\sqrt{\frac{1-2\upsilon}{2-2\upsilon}}}{\left[\left(\frac{1-2\upsilon}{2-2\upsilon}\right)+\left(\frac{r}{z}\right)^2\right]^{3/2}}$$

The symbols have the same meaning as in the case of Boussinesq's solution; v is Poisson's ratio for the medium and may be taken to be zero for large lateral restraint. (They give, in fact, the flattest curve for stress distribution, as shown in Fig. 3.2, a flat curve being the logical shape for a case of large lateral restraint). Then the equation for σ_z reduces to:

$$\sigma_z = \frac{Q}{z^2} \cdot \frac{1/\pi}{[1+2(r/z)^2]^{3/2}}$$

or $$\sigma_z = K_w \cdot \frac{Q}{z^2}$$

where $$K_w = \frac{1/\pi}{[1+2(r/z)^2]^{3/2}}$$

K_w is Westergard's influence coefficient, the variation of which with (r/z) is shown in Fig. 3.2; for comparison, the variation of K_b is also super-imposed:

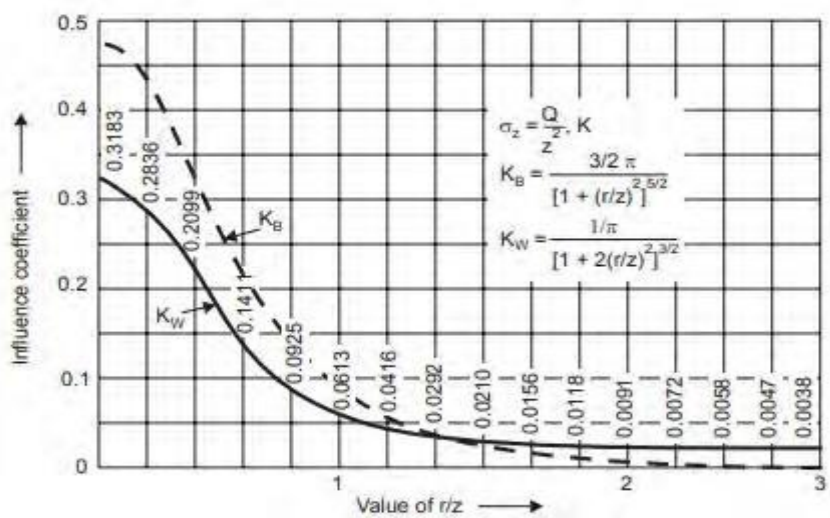

Fig 3.2 Influence coefficients for vertical stress due to concentrated load

Stress Isobar or Pressure Bulb Concept:

An 'isobar' is a stress contour or a line that connects all points below the ground surface at which the vertical pressure is the same. In fact, an isobar is a spatial curved surface and resembles a bulb in shape; this is because the vertical pressure at all points in a horizontal plane at equal radial distances from the load is the same. Thus, the stress isobar is also called the 'bulb of pressure' or simply the 'pressure bulb'. The vertical pressure at each point on the pressure bulb is the same.

Pressure at points inside the bulb is greater than that at a point on the surface of the bulb; and pressures at points outside the bulb are smaller than that value. Any number of pressure bulbs may be drawn for any applied load since each one corresponds to an arbitrarily chosen value of stress. A system of isobars indicates the decrease in stress intensity from the inner to the outer ones and reminds one of an 'Onion bulb'. Hence the term 'pressure bulb'. An isobar diagram, consisting of a system of isobars appears somewhat as shown in Fig. 3.3:

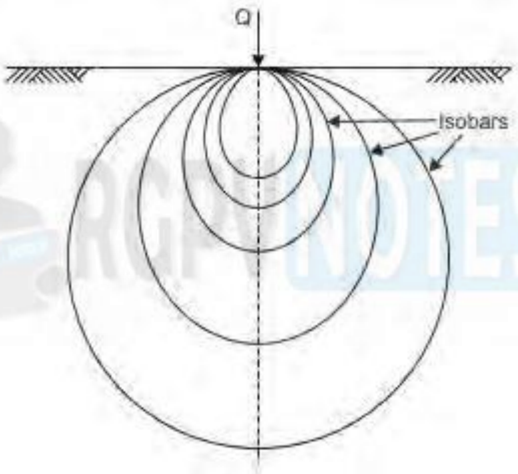

Fig 3.3 Isobar diagram (A system of pressure bulbs for a point load—Boussinesq's)

Let it be required to plot an isobar for which $\sigma_z = 0.1$ Q per unit area (10% isobar).

Newmark's influence chart:

Newmark's (1942) derived a simple, graphical procedure for computing the vertical stress in the interior of a soil medium, loaded by uniformly distributed, vertical load at the surface. The chart devised by him for this purpose is called an 'Influence Chart'. This is applicable to a semi-infinite, homogeneous, isotropic and elastic soil mass (and not for a stratified soil).

The vertical stress underneath the centre of a uniformly loaded circular area has been shown to be:

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

Where a = radius of the loaded area, z = depth at which the vertical stress is required, and q = intensity of the uniform load. Here (a/z) may be interpreted as relative sizes or radii of circular-loaded areas required to cause particular values of the ratio of the vertical stress to the intensity of the uniform loading applied.

If a series of values is assigned for the ratio σ_z/q, such as 0, 0.1, 0.2. 0.9, and 1.00, a corresponding set of values for the relative radii, a/z, may be obtained. If a particular depth is specified, then a series of concentric circles can be drawn. Since the first has a zero radius and the eleventh has infinite radius, in practice, only nine circles are drawn. Each ring or annular space causes a stress of q/10 at a point beneath the centre at the specified depth z, since the number of annular spaces (c) is ten.

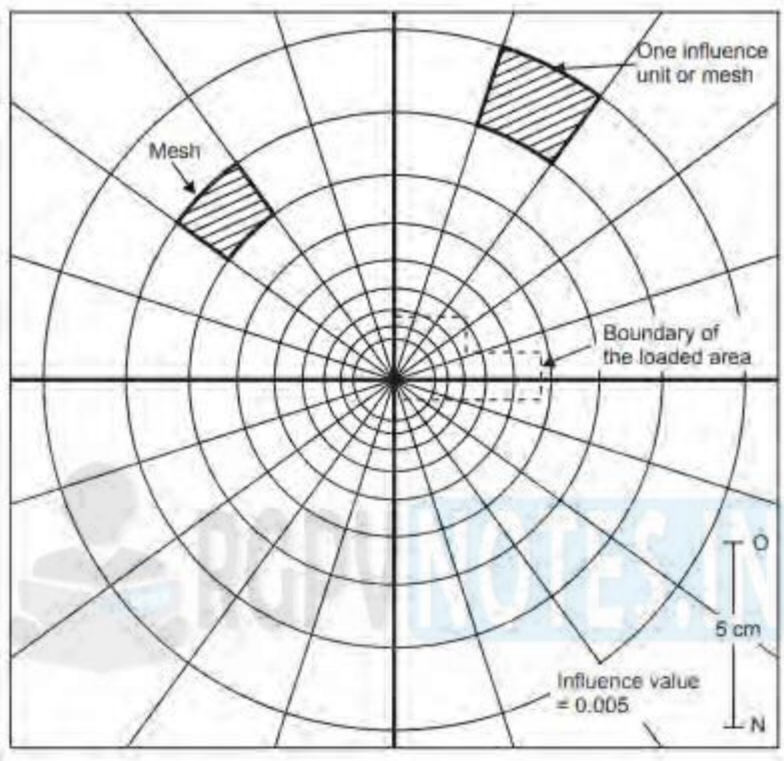

Fig 3.4 Newmark's influence chart

The construction of Newmark's influence chart, as this is usually called, may be given somewhat as follows:

For the specified depth z (say, 10 m), the radii of the circles, a, are calculated from the relative radii of (2.70 m, 4.00 m, 5.18 m and so on). The circles are then drawn to a convenient scale (say, 1 cm = 2m).

A suitable number of uniformly spaced rays (say, 20) are drawn, emanating from the centre of the circles. The resulting diagram will appear as shown in Fig. 3.4; on it is drawn a vertical line ON, representing the depth z to the scale used in drawing the circles (if the scale used is 1 cm = 2 m, ON will be 5 cm). The influence value for this chart will be 1 10 20 × or 0.005. The diagram can be used for other values of the depth z by simply assuming that the scale to which it is drawn alters; thus, if z is to be 5 m the line ON now represents 5 m and the scale is therefore 1 cm = 1 m (similarly, if z = 20 m, the scale becomes 1 cm = 4 m).

The uses of the Newmark's chart are as follows:

The chart can be used for any uniformly loaded area of whatever shape that may be. First, the loaded area is drawn on a tracing paper, using the same scale to which the distance ON on the chart represents the specified depth; the point at which the vertical stress is desired is then placed over the centre of the circles on the chart. The number of influence units encompassed by or contained in the boundaries of the loaded area are counted, including fractional units, if any let this total equivalent number be N. The stress σ_z at the specified depth at the specified point is then given by:

$$\sigma_z = I.\,N.\,q, \text{ where } I = \text{influence value of the chart.}$$

Although it appears remarkably simple, Newmark's chart has also some inherent deficiencies:

- Many loaded areas have to be drawn; alternatively, many influence charts have to be drawn.
- For each different depth, counting of the influence meshes must be done. Considerable amount of guesswork may be required in estimating the influence units partially covered by the loaded area.

However, the primary advantage is that it can be used for loaded area of any shape and that it is relatively rapid. This makes it attractive.

➢ <u>Contact pressure distribution in sands and clays:</u>

'Contact pressure' is the actual pressure transmitted from the foundation to the soil. It may also be looked upon as the pressure, by way of reaction, exerted by the soil on the underside of the footing or foundation. A uniformly loaded foundation will not necessarily transmit a uniform contact pressure to the soil. This is possible only if the foundation is perfectly 'flexible'; the contact pressure is uniform for a flexible foundation irrespective of the nature of the foundation soil.

If the foundation is 'rigid', the contact pressure distribution depends upon the type of the soil below the foundation as shown in Fig. 3.5. On the assumption of a uniform vertical settlement of the rigid foundation, the theoretical value of the contact pressure at the edges of the foundation is found to be infinite from the theory of elasticity, in the case of perfectly elastic material such as saturated clay (ϕ = 0). However, local yielding of the soil makes the pressure at the edges finite, as shown in Fig. 3.5(a). Under incipient failure conditions the pressure distribution, tends to be practically uniform.

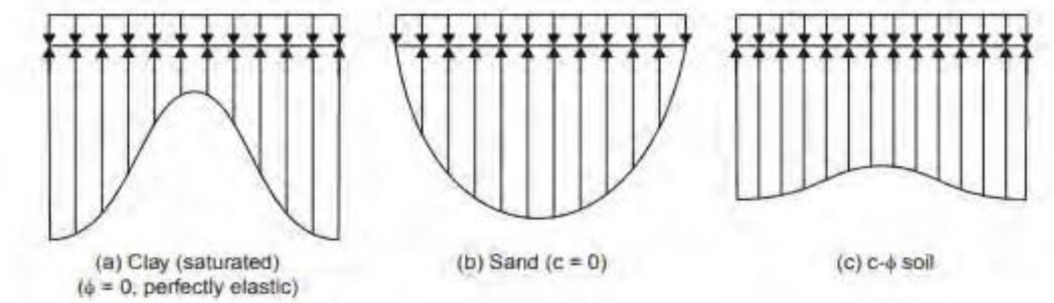

Fig 3.5 Contact pressure distribution under a uniformly loaded rigid foundation

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For a rigid foundation, placed at the ground surface on sand (c = 0), the contact pressure at the edges is zero, since no resistance to shear can be mobilized for want of over-burden pressure; the pressure distribution is approximately parabolic, as shown in Fig. 3.5(b). The more the foundation is below the surface of the sand, the more the shear resistance developed at the edges due to increase in overburden pressure, and as a consequence, the contact pressure distribution tends to be more uniform. For a general cohesive-frictional soil (c – ϕ soil) the contact pressure distribution will be intermediate between the extreme cases of (a) and (b), as shown in Fig. 3.5(c). Also for a foundation such as a reinforced concrete foundation which is neither perfectly flexible nor perfectly rigid, the contact pressure distribution depends on the degree of rigidity, and assumes an intermediate pattern for flexible and rigid foundation. However, in most practical cases the assumption of uniform contact pressure distribution yields sufficiently accurate design values for moments, shears and vertical stresses, and hence is freely adopted.

- **Compaction of soil:**

'Compaction' of soil may be defined as the process by which the soil particles are artificially rearranged and packed together into a state of closer contact by mechanical means in order to decrease its porosity and thereby increase its dry density. This is usually achieved by dynamic means such as tamping, rolling, or vibration. The process of compaction involves the expulsion of air only.

The process of compaction is accompanied by the expulsion of air only. In practice, soils of medium cohesion are compacted by means of rolling, while cohensionless soils are most effectively compacted by vibration. Prior to the advent of rolling equipment, earth fills were usually allowed to settle over a period of years under their own weight before the pavement or other construction was placed.

The degree of compaction of a soil is characterized by its dry density. The degree of compaction depends upon the moisture content, the amount of compactive effort or energy expended and the nature of the soil. A change in moisture content or compactive effort brings about a change in density. Thus, for compaction of soil, a certain amount of water and a certain predetermined amount of rolling are necessary.

The following are the important effects of compaction:

- Compaction increases the dry density of the soil, thus increasing its shear strength and bearing capacity through an increase in frictional characteristics ;
- Compaction decreases the tendency for settlement of soil ; and,
- Compaction brings about a low permeability of the soil.

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- **Moisture Content—Dry Density Relationship:**

The relation between moisture content and dry density of a soil at a particular compaction energy or effort is shown in Fig. 3.6

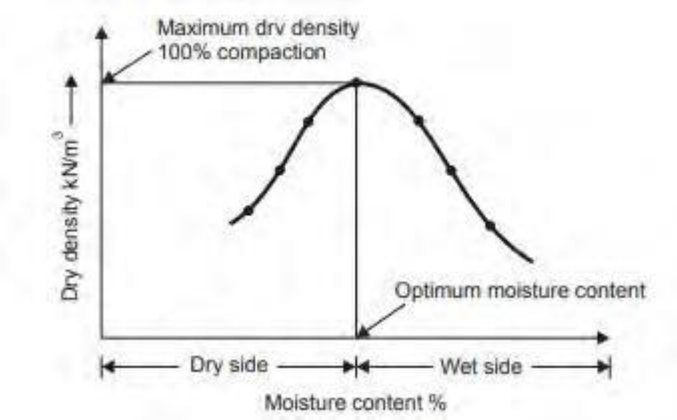

Fig 3.6 Moisture content versus dry density at a particular compactive effort

The addition of water to a dry soil helps in bringing the solid particles together by coating them with thin films of water. At low water content, the soil is stiff and it is difficult to pack it together. As the water content is increased, water starts acting as a lubricant, the particles start coming closer due to increased workability and under a given amount of compactive effort, the soil-water-air mixture starts occupying less volume, thus effecting gradual increase in dry density. As more and more water is added, a stage is reached when the air content of the soil attains a minimum volume, thus making the dry density a maximum. The water content corresponding to this maximum dry density is called the 'optimum moisture content'. Addition of water beyond the optimum reduces the dry density because the extra water starts occupying the space which the soil could have occupied.

The curve with the peak shown in Fig. 3.6 is known as the 'moisture-content dry density curve' or the 'compaction curve'. The state at the peak is said to be that of 100% compaction at the particular compactive effort; the curve is usually of a hyperbolic form, when the points obtained from tests are smoothly joined. The wet density and the moisture content are required in order to calculate the dry density as follows:

$$\gamma_d = \frac{\gamma}{(1+w)}, \text{ where}$$

γ_d = dry density,

γ = wet (bulk) density,

w = water content, expressed as a fraction.

Increase in compactive effort or the energy expended will result in an increase in the maximum dry density and a corresponding decrease in the optimum moisture content.

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➢ **Standard Proctor Test (AASHO Test):**

This test was developed by R.R. Proctor (1933) in connection with the construction of earth dams in California (U.S.A.).The apparatus for the test consists of

- A cylindrical mould of internal diameter 102 mm and an effective height of 117 mm, with a volume of 0.945 liter,
- A detachable collar of 50 mm effective height (60 mm total height),
- A detachable base plate, and
- A 50 mm diameter metal rammer of weight 25 N, and a height of fall of 300 mm, moving in a metallic outer sleeve (Fig. 3.7).

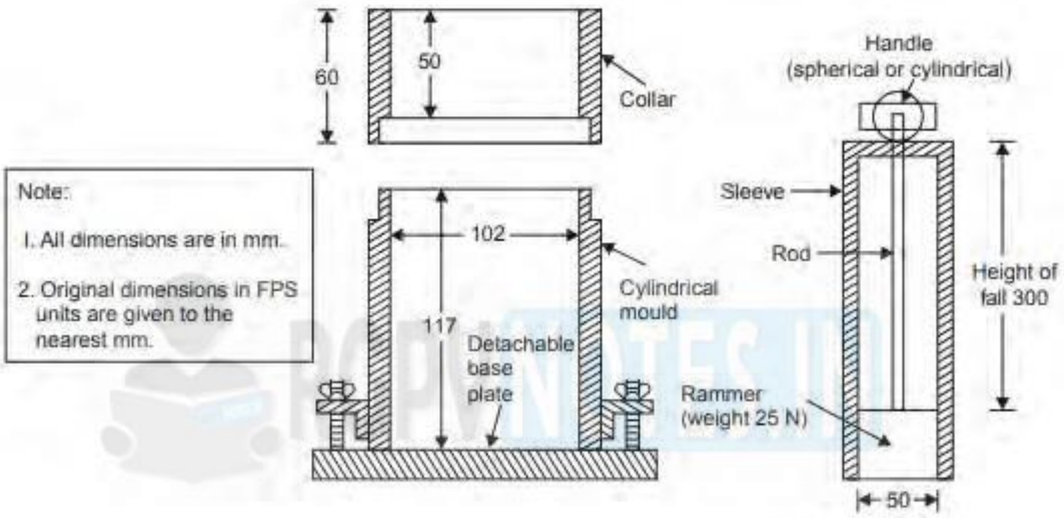

Fig 3.7 Apparatus for standard proctor test

The Test Procedure Consists of the Following Steps:

- About 30 N of air-dried soil passing 20 mm sieve is taken.
- A reasonable amount of water is added to the soil and it is thoroughly mixed.
- The mould is filled with this moist soil in three equal layers to give a total compacted depth of 130 mm. Each layer is compacted by giving 25 blows with the standard rammer, pulling the rammer in the sleeve to the maximum height and then allowing it to fall freely. The position of the rammer is changed each time to distribute the compactive energy evenly to the soil. Each layer is raked with a spatula before placing fresh soil to provide proper bond.
- The collar is removed and the extra soil trimmed off to the top of the mould and the weight of the mould obtained. The wet weight (W) of the soil is got by subtracting the weight of the empty mould. The bulk unit weight (γ) of the soil is obtained by dividing the wet weight of soil by the volume of the soil (V) which is the same as that of the soil. $\gamma = W/V$

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- A representative sample of the wet soil is taken and the moisture content (w %) determined in the standard manner through oven-drying.

The dry unit weight (γ_d) is obtained as:

$$\gamma_d = \frac{\gamma}{\left(1+\frac{w}{100}\right)}$$

- The soil is broken with hand an remixed with increased water so that the moisture content increases by 2 to 4% nearly.
- The test is repeated with a least six different water contents. The wet weight of the soil itself gives an indication whether the number of readings is adequate or not, because it first increases with an increase in water content, up to a certain value, and thereafter decreases. The test must be done such that this peak is established.
- The moisture content-dry density curve, called the 'compaction curve' is drawn. (i) The optimum moisture content and the corresponding maximum dry unit weight are read off from the graph.

The compactive effort or energy transmitted to the soil is considered to be about 605 Nm per 1000 cm3 of the soil. This test has been adopted as the standard test by the AASHO (American Association of State Highway Officials) initially. For coarse-grained soils, an initial water content of 4% and for fine-grained soils, a value of 10% are considered to be reasonable values, since the optimum value is likely to be more for the latter than that for the former.

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➢ <u>Modified Proctor Test (Modified AASHO Compaction Test):</u>

- This test was developed to deliver greater compactive effort with a view to simulating the heavier compaction required for the construction of airport pavements. The mould used is almost the same as that for Standard Proctor Test but with an effective height of 127 mm. The weight of the rammer is 45 N and the height of fall is 450 mm.
- The mould is filled in five layers, each layer being compacted with 25 blows. The compactive energy delivered is of the order of 2726 Nm per 1000 cm3 of soil, which is about 4.5 times that of the Standard Proctor Test.
- The moisture content-dry density relationship may be obtained by adopting a similar procedure as in the previous case. Since the compactive effort is more for this test than for the Standard Proctor Test, the compaction curve in this case may be expected to lie higher when superimposed over the curve for the latter, with the peak placed to the left.

➢ <u>Factors affecting compaction:</u>

Compaction of the soil is the process where the dry density of soil is increased by reducing air content or air voids present in the soil. This process is an essential part of the construction of any structure, as it strengthens the soil. Many factors influence the degree with which the soil is compacted. The various factors which affect the compaction of soil are as follows:

- Moisture content: To achieve the desired density of the soil, the moisture content of that soil has to be controlled properly. If the water content is low, it leads to soil being stiff which will resist compaction. When the water content is increased, lubrication takes place between the soil particles and the soil becomes more workable. Dry density of the soil increases with an increase in the water content till the optimum water content is reached. Adding more water at this stage, will reduce the dry density. The amount of water content added to the soil till it reaches its maximum dry density is called the moisture content of the soil.
- Types of soil: The soil type influences the compaction of that soil to a great extent. The coarse grained soils can be compacted to higher dry density than the fine grained soils. The maximum dry density decreases if the quantity of fines is increased to an amount more than that required to fill voids in the coarse grained soils. Hence it's safe to say a well graded soil obtains a much higher dry density than a poorly graded soil. Cohesive soils such as heavy clays, clays & silts provide higher resistance to compaction as they attain lower maximum dry density. Cohensionless soils such as sandy soils and coarse grained or gravelly soils are susceptible for easy compaction.
- Amount of compaction: The optimum water content required for compaction decreases with an increase in the compaction effort. This effect of increase in compaction is significant only until the water content reaches its optimum level. After that level, the volume of air voids becomes almost constant and the effect of increased compaction is not significant. It should be noted that the maximum dry density does not go on increasing with an increase in the compactive effort.
- Speed of Rolling: The speed of rolling is the speed with which the soil is compacted is a significant factor. There are two important things to be considered. The first thing is, the higher the speed of rolling, the more length of embankment can be compacted in a day. Secondly, at higher speed of rolling there is likely a chance of insufficient time required for the deformations to take place and hence more passes may be required to achieve the desired compaction.
- Contact Pressure: Contact pressure is the pressure between the soil and wheels of the equipment used for compaction. This pressure depends on the weight of the roller wheel and the contact area. A higher contact pressure increases the dry density and lowers the optimum moisture content.

➢ <u>Field compaction equipment:</u>

Certain types of in-situ compaction equipment are described below:

- Rollers:
  1. Smooth-wheeled rollers: This type imparts static compression to the soil. There may be two or three large drums; if three drums are used, two large ones in the rear and one in the front is the

common pattern. The compaction pressure are relatively low because of a large contact area. This type appears to be more suitable for compacting granular base courses and paving mixtures for highway and airfield work rather than for compacting earth fill. The relatively smooth surface obtained acts as a sort of a 'seal' at the end of a day's work and drains off rain water very well. The roller is self-propelled by a diesel engine and has a weight distribution that can be altered by the addition of ballast to the rolls. The common weight is 80 kN to 100 kN (8 to 10 t), although the range may be as much as 10 kN to 200 kN (1 to 20 t). The pressure may be of the order of 300 N (30 kg) per lineal cm of the width of rear rolls. The number of passes varies with the desired compaction; usually eight passes may be adequate to achieve the equivalent of standard Proctor compaction.

2. **Pneumatic-tyred rollers: This type compacts primarily by kneading action. The usual form is a box or container—mounted on two axles to which pneumatic-tyred wheels are fitted; the front axle will have one wheel less than the rear and the wheels are mounted in a staggered fashion so that the entire width between the extreme wheels is covered. The weight supplied by earth ballast or other material placed in the container may range from 120 kN (12 t) to 450 kN (45 t), although an exceptionally heavy capacity of 2000 kN (200 t) may be occasionally used. Some equipment is provided with a "Wobble-wheel" effect, a design in which a slightly weaving path is tracked by the travelling wheels; this facilitates the exertion of a steady pressure on uneven ground, which is very useful in the initial stages of a fill.**
3. **Sheep foot rollers: This type of roller consists of a hollow steel drum provided with projecting studs or feet; the compaction is achieved by a combination of tamping and kneading. The drum can be filled with water or sand to provide and control the dead weight. As rolling is done, most of the roller weight is imposed through the projecting feet. The feet are usually club-shaped (100 × 75 mm) or tapered (57 × 57 mm), the number on a 50 kN (5 t) roller ranging from 64 to 88. The contact pressures of the feet may range from 700 kN/m2 (7 kg/cm2) to 4200 kN/m2 (42 kg/cm2) and weight per drum from 25 kN (2.5 t) to 130 kN (13 t). Initially, the projections sink into the loose soil and compact the soil near the lowest portion of the layer. In subsequent passes with the roller, the zone of compaction continues to rise until the surface is reached, when the roller is said to "Walk-out". The length of the studs, the contact area and the weight of roller are related to the roller performance.**
4. **Grid rollers: This type consists of rolls made from 38 mm steel bars at 130 mm centers, with spaces of 90 mm square. The weight of the roller ranges from 55 kN (5.5 t) to 110 kN (11 t). This is usually a towed unit which is suitable for many types of soil including wet clays and silts.**

- **Rammers:**

This type includes the dropping type and pneumatic and internal commission type, which are also called 'frog rammers'. They weigh up to about 1.5 kN (150 kg) and even as much as 10 kN (1 t) occasionally. This type may be used for cohensionless soils, especially in small restricted and confined areas such as beds of drainage trenches and back fills of bridge abutments.

- **Vibrators**

These are vibrating units of the out-of-balance weight type or the pulsating hydraulic type. Such a type is highly effective for cohensionless soils. Behind retaining walls where the soil is confined, the backfill, much deeper in thickness, may be effectively compacted by vibration type of compactors. A few of this type are dealt with below:

1. **Vibrating drum: A separate motor drives an arrangement of eccentric weights so as to cause a high-frequency, low-amplitude, vertical oscillation to the drum. Smooth drums as well as sheep foot type of drums may be used. Layers of the order of 1 meter deep could be compacted to high densities.**
2. **Vibrating pneumatic tyre: A separate vibrating unit is attached to the wheel axle. The ballast box is suspended separately from the axle so that it does not vibrate. A 300 mm thick layer of granular soil will be satisfactorily compacted after a few passes.**
3. **Vibrating plate: This typically consists of a number of small plates, each of which is operated by a separate vibrating unit. These have a limited depth of effectiveness and hence are used in compacting granular base courses for highway and airfield pavements.**
4. **Vibroflotation: A method suited for compacting thick deposits of loose sandy soil is called the 'vibroflotation' process. The improvement of density is restricted to the surface zone in the case of conventional compaction equipment. The vibroflotation method first compact deep zone in the soil and then works its way towards the surface. A cylindrical vibrator weighing about 20 kN (2 t) and approximately 400 mm in diameter and 2 m long, called the 'Vibroflot', is suspended from a crane and is jetted to the depth where compaction is to start.**

- **Control of Compaction in the Field:**

Control of compaction in the field consists of checking the water content in relation to the laboratory optimum moisture content and the dry unit weight achieved in-situ in relation to the laboratory maximum dry unit weight from a standard compaction test. Typically, each layer is tested at several random locations after it has been compacted.

The required density can be specified either by 'relative compaction' (also called 'degree of compaction') or by the final air-void content. Relative compaction means the ratio of the insitu dry unit weight achieved by compaction to the maximum dry unit weight obtained from an appropriate standard

compaction test in the laboratory. Usually, the relative compaction of 90 to 100% (depending upon the maximum laboratory value), corresponding to about 5 to 10% air content, is specified and sought to be achieved. Typical values of dry unit weights achieved may be as high as 22.5 kN per m^3 for well-graded gravel and may be as low as 14.4 kN per m3 (1440 kg/m3) for clays. Approximate ranges of optimum moisture content may be 6 to 10% for sands, 8 to 12% for sand-silt mixtures, 11 to 15% for silts and 13 to 21% for clays (as got from modified AASHO tests). A variation of 5 to 10% is allowed in the field specification of dry unit weight at random locations, provided the average is about the specified value.

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Unit IV

CONSOLIDATION :Types of compressibility, Spring analogy, Immediate settlement, Primary consolidation and secondary consolidation, Stress history of clay, e-p and e-log p curves, Normally consolidated soil, Over consolidated soil and under consolidated soil, Pre-consolidation pressure and its determination, Consolidation test, Terzaghi 1-D consolidation theory, Coefficient of consolidation, Square root time and Logarithm of time fitting methods, Computation of total settlement.

Compressibility of soil:

A soil is a particulate material, consisting of solid grains and void spaces enclosed by the grains. The voids may be filled with air or other gas, with water or other liquid, or with a combination of these.

The volume decrease of soil under stress might be conceivably attributed to:

- Compression of the solid grains;
- Compression of pore water or pore air;
- Expulsion of pore water or pore air from the voids, thus decreasing the void ratio or porosity.

Under the loads usually encountered in geotechnical engineering practice, the solid grains, as well as pore water, may be considered to be incompressible. Thus, compression of pore air and expulsion of pore water are the primary sources of volume decrease of a soil mass subjected to stresses. A partially saturated soil may experience appreciable volume decrease through the compression of pore air before any expulsion of pore water takes place; the situation is thus more complex for such a soil. However, it is reasonable to assume that the volume decrease of a saturated soil mass is, for all practical purposes, only due to the expulsion of pore water by the application of load. Sedimentary deposits and submerged clay strata are invariably found in nature in the fully saturated condition and problems involving volume decrease and the consequent ill-effects are associated with these.

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Specifically, the compressibility of a soil depends on the structural arrangement of the soil particles, and in fine-grained soils, the degree to which adjacent particles are bonded together. A structure that is more porous, such as a honey-combed structure, is more compressible than a dense structure. The Soil that is composed predominantly of flat grains is more compressible than one with mostly spherical grains. A soil in a remolded state may be more compressible than the same soil in its natural state.

➢ **Settlement:**

The total settlement may be considered to consist of the following contributions:

- Initial settlement or elastic compression.
- Consolidation settlement or primary compression.
- Secondary settlement or secondary compression
- Initial Settlement or Elastic Compression:

This is also referred to as the 'distortion settlement' or 'contact settlement' and is usually taken to occur immediately on application of the foundation load. Such immediate settlement in the case of partially saturated soils is primarily due to the expulsion of gases and to the elastic compression and rearrangement of particles. In the case of saturated soils immediate settlement is considered to be the result of vertical soil compression, before any volume change occurs.

1. Immediate Settlement in Cohesion-less Soils:

The elastic, as well as the primary compression effects, occur more or less together in the case of cohesion-less soils because of their high permeability. The resulting settlement is termed 'immediate settlement'. The methods available for predicting this settlement are far from perfect; either the standard penetration test or the use of charts is resorted to.

This can be used for determining the settlement, S_f, of a square foundation on a deep layer of cohesion-less soil by using Terzaghi and Peck's formula:

$$S_f = S_p \left(\frac{2B}{B + 0.3} \right)^2$$

Where Sp = Settlement of a 305 mm-square plate, B = Width of foundation (meters) the chart is applicable for deep layers only, that is, for layers of thickness not less than 4B below the foundation.

2. Immediate Settlement in Cohesive Soils:

If saturated clay is loaded rapidly, excess hydrostatic pore pressures are induced; the soil gets deformed with virtually no volume change and due to the low permeability of the clay little water is squeezed out of the voids. The vertical deformation due to the change in shape is the immediate settlement. The immediate settlement of a flexible foundation, according to Terzaghi (1943), is given by:

$$S_i = q \cdot B \left(\frac{1 - \nu^2}{E_s} \right) . I_t$$

Where S_i = immediate settlement at a corner of a rectangular flexible foundation of size L × B,

B = Width of the foundation,

q = Uniform pressure on the foundation,

Es = Modulus of elasticity of the soil beneath the foundation,

ν = Poisson's ratio of the soil, and

I_t = Influence Value, which is dependent on L/B

For a perfectly flexible square footing, the immediate settlement under its center is twice that at its corners

- **Consolidation Settlement or Primary Compression:**

The phenomenon of consolidation occurs in clays (chapter seven) because the initial excess pore water pressures cannot be dissipated immediately owing to the low permeability. The theory of one-dimensional consolidation, advanced by Terzaghi, can be applied to determine the total compression or settlement of a clay layer as well as the time rate of dissipation of excess pore pressures and hence the time-rate of settlement. The settlement computed by this procedure is known as that due to primary compression since the process of consolidation as being the dissipation of excess pore pressures alone is considered. Total settlement: The total consolidation settlement, S_c, may be obtained from one of the following equations:

$$S_c = \frac{H \, . \, C_c}{(1+e_0)} \log_{10}\left(\frac{\overline{\sigma}_0 + \Delta\overline{\sigma}}{\overline{\sigma}_0}\right)$$

$$S_c = m_v \, . \, \Delta\overline{\sigma} \, . \, H$$

$$S_c = \frac{\Delta e}{(1+e_0)} \, . \, H$$

These equations and the notation have already been dealt with in chapter seven. The vertical pressure increment Δσ at the middle of the layer have to be obtained by using the theory of stress distribution in soil.

Time-rate of settlement: Time-rate of settlement is dependent, in addition to other factors, upon the drainage conditions of the clay layer. If the clay layer is sandwiched between sand layers, pore water could be drained from the top as well as from the bottom and it is said to be a case of double drainage. If drainage is possible only from either the top or the bottom, it is said to be a case of single drainage. In the former case, the settlement proceeds much more rapidly than in the latter. The calculations are based upon the equation:

$$T = \frac{C_v t}{H^2}$$

Again, the use of this equation and the notation has been given in chapter seven. A large wheel load passing on a roadway resting on a clay layer will cause immediate settlement, which is, theoretically speaking, completely recoverable after the load has passed. If the load is applied for a long time, consolidation occurs. Judgment may be necessary for deciding what portion of the superimposed load carried by a structure will be sustained long enough to cause consolidation.

In the case of foundation of finite dimensions, such as a footing resting on a thick bed of clay, lateral strains will occur and the consolidation is no longer one dimensional. Lateral strain effects in the field may induce non-uniform pore pressures and may become one of the sources of differential settlements of a foundation.

• Secondary Settlement or Secondary Compression:

Settlement due to secondary compression is believed to occur during and mostly after the completion of primary consolidation or complete dissipation of excess pore pressure. A few theories have been advanced to explain this phenomenon, known as 'secondary consolidation', and have already been given at the end of chapter seven. In the case of organic soils, the secondary compression is comparable to the primary compression; in the case of all other soils, secondary settlement is considered insignificant. Further discussion of the concept of secondary settlement, being of an advanced nature, is outside the scope of the present work.

➢ **Consolidation:**

Consolidation is a process by which soils decrease in volume. According to Karl von Terzaghi "consolidation is any process which involves a decrease in water content of saturated soil without replacement of water by air." In general, it is the process in which reduction in volume takes place by the expulsion of water under long-term static loads. It occurs when stress is applied to a soil that causes the soil particles to pack together more tightly, therefore reducing its bulk volume. When this occurs in soil that is saturated with water, water will be squeezed out of the soil. The magnitude of consolidation can be predicted by many different methods. In the Classical Method, developed by Terzaghi, soils are tested with an odometer test to determine their compression index. This can be used to predict the amount of consolidation.

When stress is removed from a consolidated soil, the soil will rebound, regaining some of the volumes it had lost in the consolidation process. If the stress is reapplied, the soil will consolidate again along a recompression curve, defined by the recompression index. The soil which had its load removed is considered to be over consolidated. This is the case for soils that have previously had glaciers on them. The highest stress that it has been subjected to is termed the pre consolidation stress. The over consolidation ratio or OCR is defined as the highest stress experienced divided by the current stress. A soil that is currently experiencing its highest stress is said to be normally consolidated and to have an OCR of one. Soil could be considered under consolidated immediately after a new load is applied but before the excess pore water pressure has had time to dissipate.

When a soil layer is subjected to vertical stress, volume change can take place through rearrangement of soil grains, and some amount of grain fracture may also take place. The volume of soil grains remains constant, so the change in total volume is due to a change in the volume of water. In saturated soils, this can happen only if water is pushed out of the voids. The movement of water takes time and is controlled by the permeability of the soil and the locations of free-draining boundary surfaces.

It is necessary to determine both the magnitude of volume change (or the settlement) and the time required for the volume change to occur. The magnitude of settlement is dependent on the magnitude of applied stress, thickness of the soil layer, and the compressibility of the soil.

When soil is loaded untrained, the pore pressure increases. As the excess pore pressure dissipates and water leaves the soil, settlement takes place. This process takes time, and the rate of settlement decreases over time. In coarse soils (sands and gravels), volume change occurs immediately as pore pressures are dissipated rapidly due to high permeability. In fine soils (silts and clays), slow seepage occurs due to low permeability.

- Relationship between pressure and void ratio

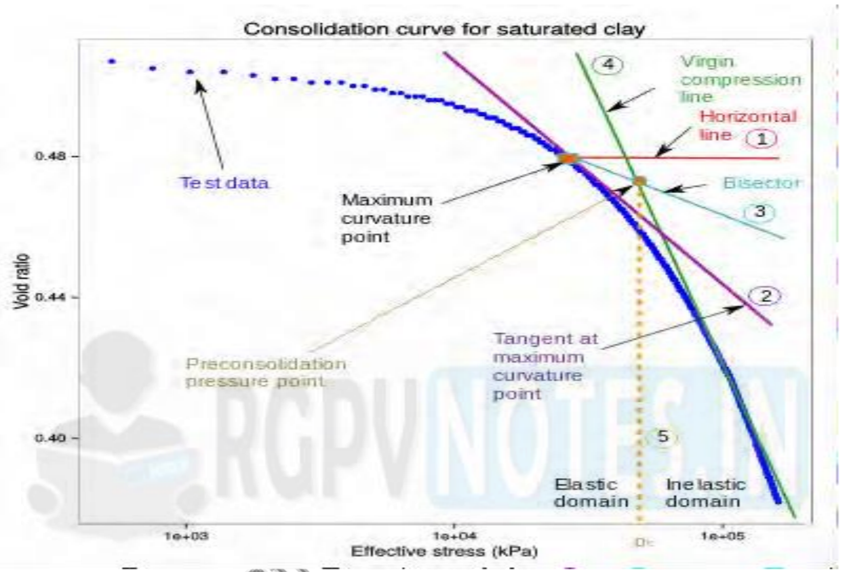

Figure 4.1 Relationship between pressure and void ratio

➢ <u>One dimensional consolidation</u>

The process of consolidation is often explained with an idealized system composed of a spring, a container with a hole in its cover, and water. In this system, the spring represents the compressibility or the structure of the soil itself, and the water which fills the container represents the pore water in the soil.

- The container is completely filled with water, and the hole is closed. (Fully saturated soil)
- A load is applied onto the cover, while the hole is still unopened. At this stage, only the water resists the applied load. (Development of excess pore water pressure)
- As soon as the hole is opened, water starts to drain out through the hole and the spring shortens. (Drainage of excess pore water pressure)
- After some time, the drainage of water no longer occurs. Now, the spring alone resists the applied load. (Full dissipation of excess pore water pressure. End of consolidation)

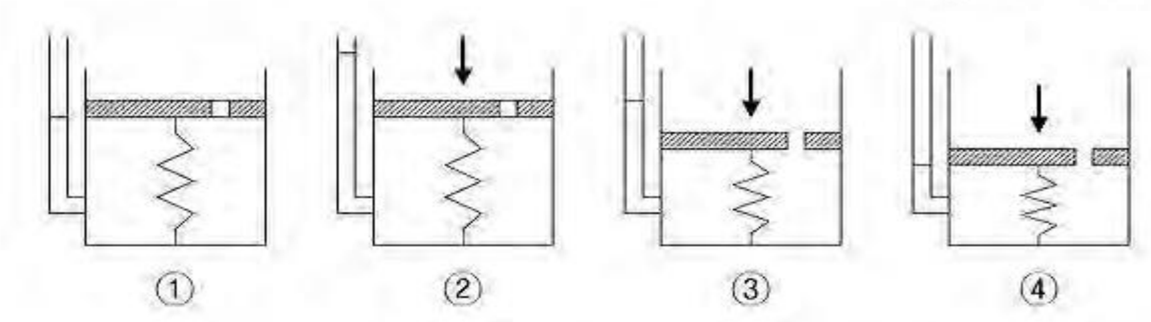

Figure 4.2 Consolidation Test Setup

Consolidation Test & Fitting Time curves

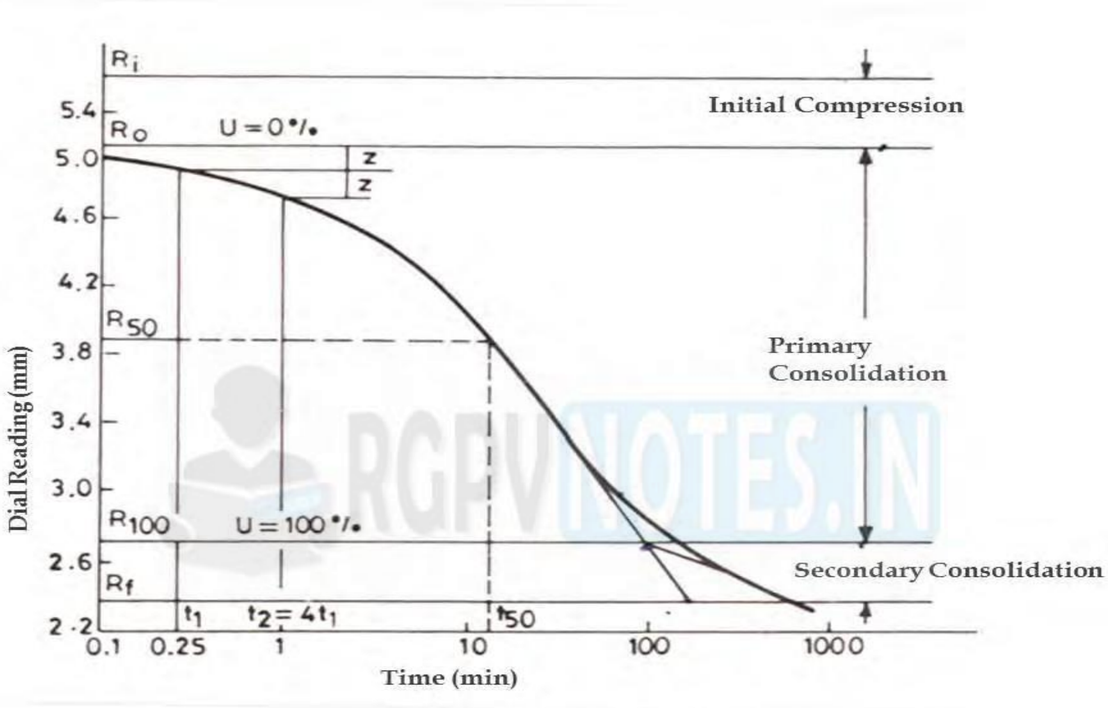

Figure 4.3 Fitting Time Curves

After the coefficient of consolidation (c_v) has been determined from laboratory data calculations are possible for site settlements. It is important to note that c_v is not a constant, but varies with both the level of stress and degree of consolidation. For practical site settlement calculations, however, it is sufficiently accurate to measure c_v relative to the loading range applicable on site and then assume this value to be approximately constant for all degrees of consolidation The basic equation used is:

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

- **<u>Normally and over consolidated clays:</u>**

Normally consolidated clays are these that are currently experiencing the maximum vertical overburden effective pressure they have ever experienced in their history. Over consolidated clays had experienced a higher overburden stress in the past. The pre-consolidation stress's is defined to be the maximum

effective stress experienced by the soil. This stress is identified in comparison with the effective stress in its present state. For soil at state Q or F, this would correspond to the effective stress at point P.

If the current effective stress, s', is equal (note that it cannot be greater than) to the pre-consolidation stress, then the deposit is said to be normally consolidated (NC). If the current effective stress is less than the pre-consolidation stress, then the soil is said to be over-consolidated (OC). OP corresponds to initial loading of the soil. PQ corresponds to unloading of the soil. QFR corresponds to a reloading of the soil. Upon reloading beyond P, the soil continues along the path that it would have followed if loaded from O to R continuously.

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

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

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

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

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

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

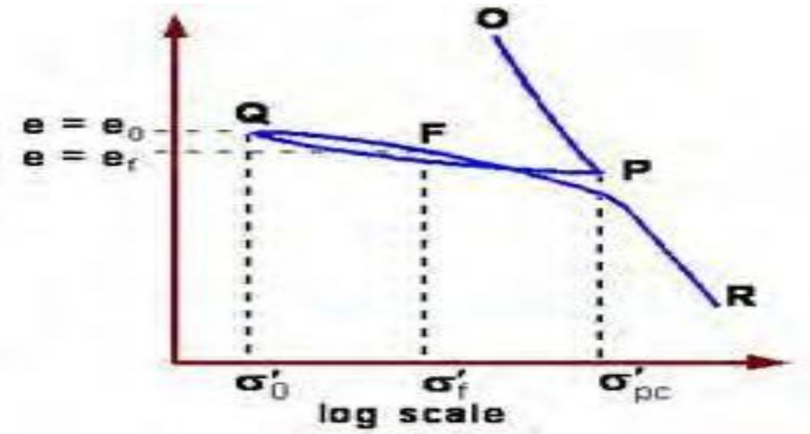

Figure 4.4 Curve between stresses and void ratio

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- **Terzaghi theory of one-dimensional consolidation:**

Terzaghi (1925) advanced his theory of one-dimensional consolidation based upon the following assumptions, the mathematical implications being given in parentheses:

- **The soil is homogeneous (k_z is independent of z).**
- **The soil is completely saturated (S = 100%).**
- **The soil grains and water are virtually incompressible (γw is constant and volume change of soil is only due to change in void ratio).**
- **The behavior of infinitesimal masses in regard to expulsion of pore water and consequent consolidation is no different from that of larger representative masses (Principles of calculus may be applied).**
- **The compression is one-dimensional (u varies with z only).**
- **The flow of water in the soil voids is one-dimensional, Darcy's law being valid.**
- **Certain soil properties such as permeability and modulus of volume change are constant; these actually vary somewhat with pressure. (k and m_v are independent of pressure).**
- **The pressure versus void ratio relationship is taken to be the idealized one**
- **Hydrodynamic lag alone is considered and plastic lag is ignored, although it is known to exist. (The effect of k alone is considered on the rate of expulsion of pore water).**

The first three assumptions represent conditions that do not vary significantly from actual conditions. The fourth assumption is purely of academic interest and is stated because the differential equations used in the derivation treat only infinitesimal distances. It has no significance for the laboratory soil sample or for the field soil deposit. The fifth assumption is certainly valid for deeper strata in the field owing to lateral confinement and is also reasonably valid for an oedometer sample. The sixth assumption regarding flow of pore water being one-dimensional may be taken to be valid for the laboratory sample, while its applicability to a field situation should be checked. However, the validity of Darcy's law for flow of pore water is unquestionable.

The seventh assumption may introduce certain errors in view of the fact that certain soil properties which enter into the theory vary somewhat with pressure but the errors are considered to be of minor importance. The eighth and ninth assumptions lead to the limited validity of the theory. The only justification for the use of the eighth assumption is that, otherwise, the analysis becomes unduly complex. The ninth assumption is necessitated because it is not possible to take the plastic lag into account in this theory. These two assumptions also may be considered to introduce some errors.

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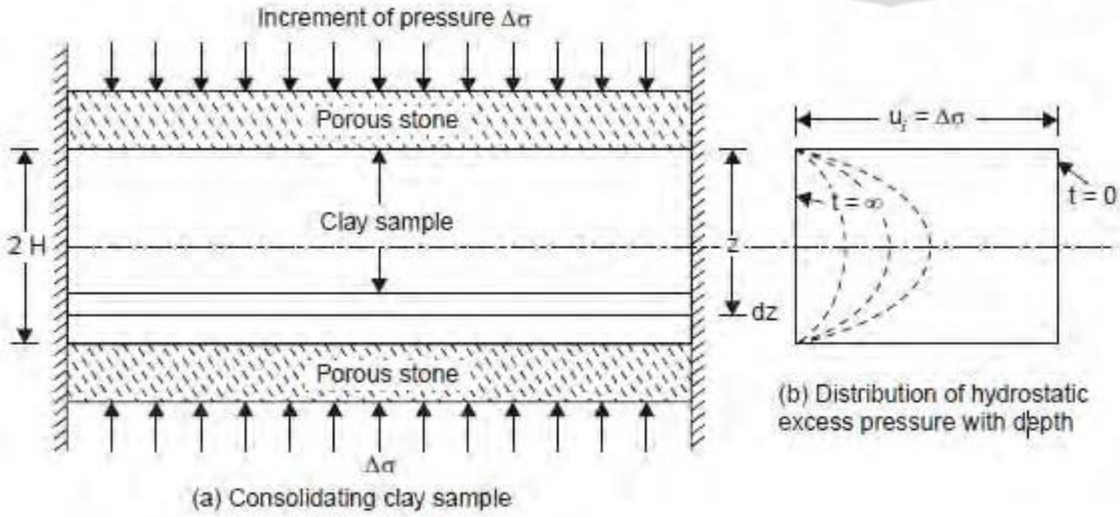

Figure 4.5 Consolidation of a clay sample with double drainage

Let us consider a layer of unit area of cross-section and of elementary thickness dz at depth z from the pervious boundary. Let the increment of pressure applied be Δσ. immediately on application of the pressure increment, pore water starts to flow towards the drainage faces. Let ∂*h* be the head lost between the two faces of this elementary layer, corresponding to a decrease of hydrostatic excess pressure ∂*u*.

$$k_x \cdot \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = \frac{1}{(1+e)}\left[e \cdot \frac{\partial s}{\partial t} + S \cdot \frac{\partial e}{\partial t}\right]$$

For one-dimensional flow situation, this reduces to:

$$k_z \cdot \frac{\partial^2 h}{\partial z^2} = \frac{1}{(1+e)}\left[e \cdot \frac{\partial s}{\partial t} + S \cdot \frac{\partial e}{\partial t}\right]$$

During the process of consolidation, the degree of saturation is taken to remain constant at 100%, while void ratio changes causing reduction in volume and dissipation of excess hydrostatic pressure through expulsion of pore water; that is

$$S = 100\% \text{ or unity, and } \frac{\partial S}{\partial t} = 0.$$

$$k_z \cdot \frac{\partial^2 h}{\partial z^2} = -\frac{1}{(1+e)} \cdot \frac{\partial e}{\partial t} = -\frac{\partial}{\partial t}\left[\frac{e}{1+e}\right],$$

Negative sign denoting decrease of e for increase of h. Since volume decrease can be due to a decrease in the void ratio only as the pore water and soil grains are virtually incompressible

$$\frac{\partial}{\partial t}\left(\frac{e}{1+e}\right)$$

Represents time-rate of volume change per unit volume

The flow is only due to the hydrostatic excess pressure,

$$h = \frac{u}{\gamma_w}, \text{ where } \gamma_w = \text{unit weight of water.}$$

$$\therefore \quad \frac{k}{\gamma_w} \cdot \frac{\partial^2 u}{\partial z^2} = -\frac{\partial V}{\partial t}$$

(This can also be considered as the continuity equation for a non-zero net out-flow, while Laplace's equation represents inflow being equal to out-flow). Here k is the permeability of soil in the direction of flow, and ∂V represents the change in volume per unit volume. The change in hydrostatic excess pressure, ∂u, changes the inter-granular or effective stress by the same magnitude, the total stress remaining constant. The change in volume per unit volume, ∂V, may be written, as per the definition of the modulus of volume change, m_v

$$\partial V = m_v . \partial \bar{\sigma} = -m_v . \partial u, \text{ since an increase } \partial \bar{\sigma} \text{ represents a decrease } \partial u.$$

Differentiating both sides with respect to time,

$$\frac{\partial V}{\partial t} = -m_v \cdot \frac{\partial u}{\partial t}$$

$$\frac{\partial u}{\partial t} = \frac{k}{\gamma_w . m_v} \cdot \frac{\partial^2 u}{\partial z^2}$$

This is written as:

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

$$\text{where } c_v = \frac{k}{\gamma_w . m_v}$$

C_V is known as the "Coefficient of consolidation". u represents the hydrostatic excess pressure at a depth z from the drainage face at time t from the start of the process of consolidation. The coefficient of consolidation may also be written in terms of the coefficient of compressibility:

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

This is the basic differential equation of consolidation according to Terzaghi theory for one-dimensional consolidation. The coefficient of consolidation combines the effect of permeability and compressibility characteristics on volume change during consolidation. Its units can be shown to be mm^2/s or L^2T^{-1}.

The initial hydrostatic excess pressure, u_i, is equal to the increment of pressure $\Delta\sigma$, and is the same throughout the depth of the sample, immediately on application of the pressure, and is shown by the heavy line. The horizontal portion of the heavy line indicates the fact that, at the drainage face, the hydrostatic excess pressure instantly reduces to zero, theoretically speaking. Further, the hydrostatic excess pressure would get fully dissipated throughout the depth of the sample only after the lapse of infinite time, as indicated by the heavy vertical line on the left of the figure. At any other instant of time, the hydrostatic excess pressure will be the maximum at the farthest point in the depth from the drainage faces, that is, at the middle and it is zero at the top and bottom. The distribution of the hydrostatic excess pressure with depth is sinusoidal at other instants of time, as shown by dotted lines. These curves are called “Isochrones”.

- **Settlement analysis & Calculation of total settlement:**

The total settlement of a loaded soil has three components: Elastic settlement, primary consolidation, and secondary compression. Elastic settlement is on account of change in shape at constant volume, i.e. due to vertical compression and lateral expansion. Primary consolidation (or simply consolidation) is on account of flow of water from the voids, and is a function of the permeability and compressibility of soil. Secondary compression is on account of creep-like behavior.

Primary consolidation is the major component and it can be reasonably estimated. A general theory for consolidation, incorporating three-dimensional flow is complicated and only applicable to a very limited range of problems in geotechnical engineering. For the vast majority of practical settlement problems, it is sufficient to consider that both seepage and strain take place in one direction only, as one-dimensional consolidation in the vertical direction.

The components of settlement of a foundation are:

- Immediate settlement
- Consolidation Settlement, and
- Secondary compression (creep)

$$\Delta H = \Delta H_i + U\,\Delta H_c + \Delta H_s$$

ΔH = total settlement, ΔHc = consolidation settlement, ΔH = secondary compression, U = average degree of consolidation. Generally, the final settlement of a foundation is of interest and U is considered equal to 1 (i.e. 100% consolidation)

- **Immediate Settlement(ΔH_i)**
  - Immediate settlement takes place as the load is applied or within a time period of about 7 days.
  - Predominates in cohesion less soils and unsaturated clay
  - Immediate settlement analysis are used for all fine-grained soils including silts and clays with a degree of saturation < 90% and for all coarse grained soils with large co-efficient of permeability (say above 10.2 m/s)

- **<u>Consolidation Settlement (ΔH_c)</u>**
    - **Consolidation settlements are time dependent and take months to years to develop. The leaning tower of Pisa in Italy has been undergoing consolidation settlement for over 700 years. The lean is caused by consolidation settlement being greater on one side. This, however, is an extreme case. The principal settlements for most projects occur in 3 to 10 years.**
    - **Dominates in saturated/nearly saturated fine grained soils where consolidation theory applies. Here we are interested to estimate both consolidation settlement and how long a time it will take or most of the settlement to occur.**
- **<u>Secondary Settlement/Creep (ΔH_S)</u>**
    - **It occurs under constant effective stress due to continuous rearrangement of clay particles into a more stable configuration.**
    - **Predominates in highly plastic clays and organic clays.**

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Unit V

SHEAR STRENGTH: Definition and importance of shear strength, Mohr and coulomb failure theories, Mohr's Stress Circles, Measurement of shear strength-Different types of Shear Test namely, Direct Shear Test, Unconfined Compression Test, Tri-Axial Compression Test & Vane Shear Test for strength parameters, Strength tests based on drainage conditions, Measurement of pore pressure, Pore pressure Parameters, Strength envelops shear strength of sands, Critical void ratio, Liquefaction, Shear strength of clays. Factors affecting shear strength of granular soils and cohesive soils.

STABILIZATION OF SOIL: Introduction, Mechanical stabilization, Cement stabilization, Lime stabilization, Bituminous stabilization, Chemical stabilization, Thermal stabilization, Electrical stabilization, Stabilization by grouting, Use of geosynthetic materials, Types, Functions and applications of geo-synthetics, Reinforced earth structures-components and construction.

- **Shear Strength of Soil:**

The shear strength of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it. One must understand the nature of shearing resistance in order to analyze soil stability problems such as bearing capacity, slope stability, and lateral pressure on earth retaining structures. A lot of maturity and skill may be required on the part of the engineer in interpreting the results of the laboratory tests for application to the conditions in the field.

A soil derives its shearing strength from the following:

- **Resistance due to the interlocking of particles.**
- **Frictional resistance between the individual soil grains, which may be sliding friction, rolling friction, or both.**
- **Adhesion between soil particles or 'cohesion'.**

Soils consist of individual particles that can slide and roll relative to one another. The shear strength of a soil is equal to the maximum value of shear stress that can be mobilized within a soil mass without failure taking place. The shear strength of a soil is a function of the stresses applied to it as well as how these stresses are applied. Knowledge of the shear strength of soils is necessary to determine the bearing capacity of foundations, the lateral pressure exerted on retaining walls, and the stability of slopes.

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- **Mohr's Theory:**

In soil testing, cylindrical samples are commonly used in which radial and axial stresses act on principal planes. The vertical plane is usually the minor principal plane whereas the horizontal plane is the major principal plane. The radial stress (σ_r) is the minor principal stress (σ_3), and the axial stress (σ_a) is the major principal stress (σ_1).

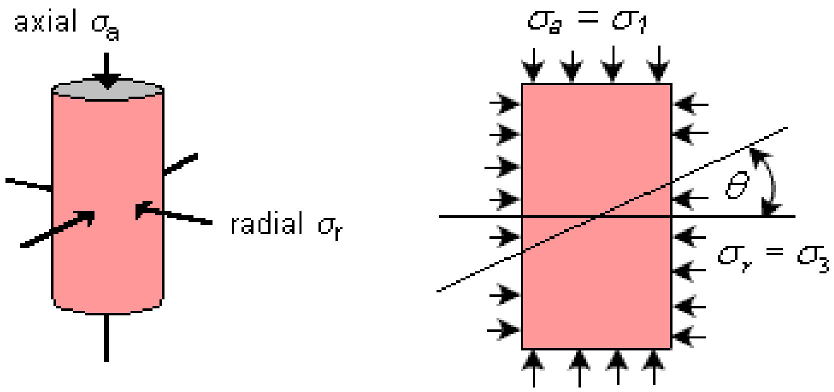

Fig 5.1 Stresses acting on soil sample

To visualize the normal and shear stresses acting on any plane within the soil sample, a graphical representation of stresses called the Mohr circle is obtained by plotting the principal stresses. The sign convention in the construction is to consider compressive stresses as positive and angles measured counter-clockwise also positive.

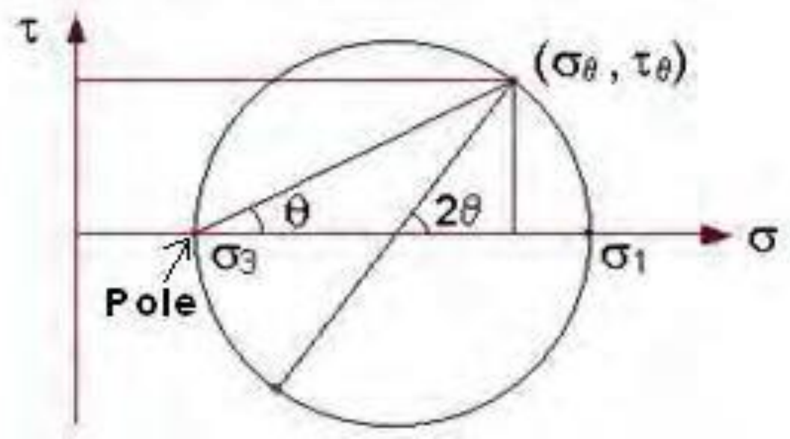

Fig 5.2 Graphical representation of Mohr Circle

Draw a line inclined at an angle with the horizontal through the pole of the Mohr circle to intersect the circle. The coordinates of the point of intersection are the normal and shear stresses acting on the plane, which is inclined at an angle within the soil sample. A 'Principal plane' is defined as a plane on which the stress is wholly normal or one which does not carry shearing stress. From mechanics, it is known that

there exist three principal planes at any point in a stressed material. The normal stresses acting on these principal planes are known as the 'principal stresses'. The three principal planes are to be mutually perpendicular. In the order of decreasing magnitude, the principal stresses are designated the 'major principal stress, the 'intermediate principal stress and the 'minor principal stress', the corresponding principal planes being designated exactly in the same manner.

- **Mohr-Coulomb Failure Criterion:**

The soil sample has failed; the shear stress on the failure plane defines the shear strength of the soil. Thus, it is necessary to identify the failure plane. Is it the plane on which the maximum shear stress acts, or is it the plane where the ratio of shear stress to normal stress is the maximum? For the present, it can be assumed that a failure plane exists and it is possible to apply principal stresses and measure them in the laboratory by conducting a triaxial test. Then, the Mohr circle of stress at failure for the sample can be drawn using the known values of the principal stresses. If data from several tests, carried out on different samples up to failure is available, a series of Mohr circles can be plotted. It is convenient to show only the upper half of the Mohr circle. A line tangential to the Mohr circles can be drawn and is called the Mohr-Coulomb failure envelope.

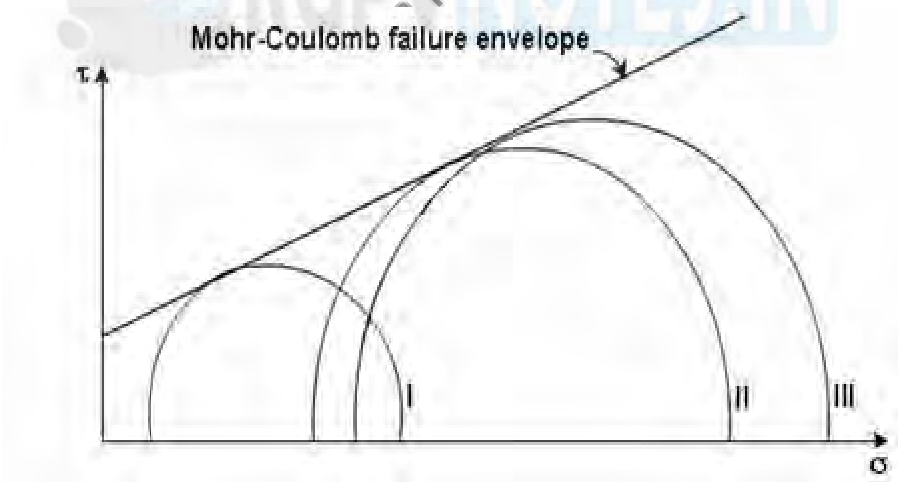

Fig 5.3 Mohr-Coulomb failure envelope

If the stress condition for any other soil sample is represented by a Mohr circle that lies below the failure envelope, every plane within the sample experiences shear stress which is smaller than the shear strength of the sample. Thus, the point of tangency of the envelope to the Mohr circle at failure gives a clue to the determination of the inclination of the failure plane. The orientation of the failure plane can be finally determined by the pole method.

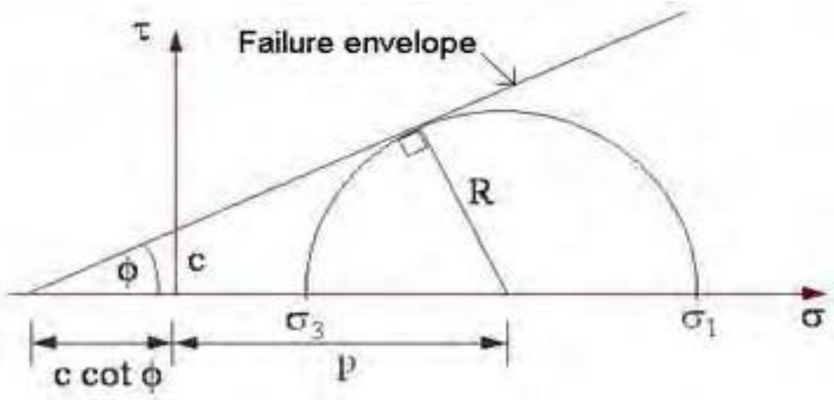

Fig 5.4 Mohr-Stress failure envelope

The Mohr-Coulomb failure criterion can be written as the equation for the line that represents the failure envelope. The general equation is

Where σ_f = shear stress on the failure plane

c = apparent cohesion

σ_n = normal stress on the failure plane

f = angle of internal friction

The failure criterion can be expressed in terms of the relationship between the principal stresses. From the geometry of the Mohr circle,

$$\sin\phi = \frac{R}{c.\cot\phi + p} = \frac{\frac{\sigma_1 - \sigma_3}{2}}{c.\cot\phi + \frac{\sigma_1 + \sigma_3}{2}}$$

$$\sigma_1 = \sigma_3\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

Rearranging,

$$\frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left[\frac{\pi}{4} + \frac{\phi}{2}\right]$$

$$\sigma_\theta = \frac{(\sigma_1 + \sigma_3)}{2} + \frac{(\sigma_1 - \sigma_3)}{2}\cos 2\theta$$

Normal stress

Shear stress

The plane inclined at an angle of to the horizontal has acted on it the maximum shear stress equal to $\frac{\sigma 1 - \sigma 3}{2}$ and the normal stress on this plane is equal to $\frac{\sigma 1 + \sigma 3}{2}$ The plane with the maximum ratio of shear

stress to normal stress is inclined at an angle of $45 + \frac{\alpha}{2}$ to the horizontal, where a is the slope of the line tangent to the Mohr circle and passing through the origin.

➢ **<u>Methods of Shear Strength Determination:</u>**

- **Triaxial Test:**

The Triaxial test is carried out in a cell on a cylindrical soil sample having a length to diameter ratio of 2. The usual sizes are 76 mm x 38 mm and 100 mm x 50 mm. Three principal stresses are applied to the soil sample, out of which two are applied water pressure inside the confining cell and are equal. The third principal stress is applied by a loading ram through the top of the cell and is different to the other two principal stresses. A typical triaxial cell is shown.

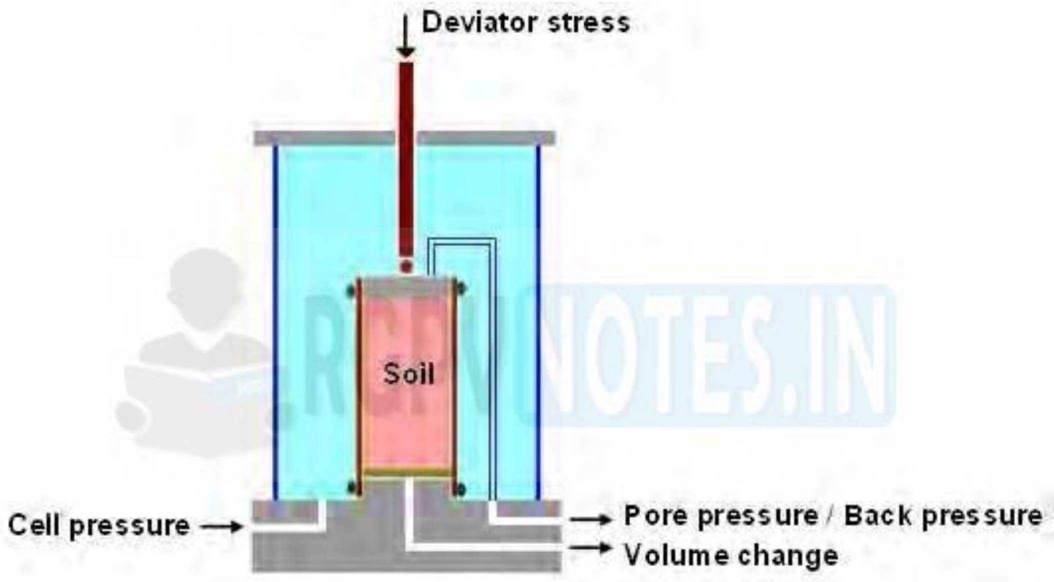

Fig 5.5 Triaxial test apparatus

The soil sample is placed inside a rubber sheath which is sealed to a top cap and bottom pedestal by rubber O-rings. For tests with pore pressure measurement, porous discs are placed at the bottom, and sometimes at the top of the specimen. Filter paper drains may be provided around the outside of the specimen to speed up the consolidation process. Pore pressure generated inside the specimen during testing can be measured through pressure transducers.

The Triaxial compression test consists of two stages:

First stage: In this, a soil sample is set in the triaxial cell and confining pressure is then applied.

Second stage: In this, additional axial stress (also called deviator stress) is applied which induces shear stresses in the sample. The axial stress is continuously increased until the sample fails.

During both stages, the applied stresses, axial strain, and pore water pressure or change in sample volume can be measured.

There are several test variations, and those used mostly in practice are:

- UU (unconsolidated undrained) test: In this, cell pressure is applied without allowing drainage. Then keeping cell pressure constant, deviator stress is increased to failure without drainage.
- CU (consolidated undrained) test: In this, drainage is allowed during cell pressure application. Then without allowing further drainage, deviator stress is increased keeping cell pressure constant.
- CD (consolidated drained) test: This is similar to the CU test except that as deviator stress is increased, drainage is permitted. The rate of loading must be slow enough to ensure no excess pore water pressure develops.
- In the UU test, if pore water pressure is measured, the test is designated by In the CU test, if pore water pressure is measured in the second stage

Significance of Triaxial Testing:

The first stage simulates in the laboratory the in-situ condition that soil at different depths is subjected to different effective stresses. Consolidation will occur if the pore water pressure which develops upon application of confining pressure is allowed to dissipate. Otherwise, the effective stress on the soil is the confining pressure (or total stress) minus the pore water pressure which exists in the soil.

During the shearing process, the soil sample experiences axial strain, and either volume change or development of pore water pressure occurs. The magnitude of shear stress acting on different planes in the soil sample is different. When at some strain the sample fails, this limiting shear stress on the failure plane is called the shear strength.

The Triaxial test has many advantages over the direct shear test:

- The soil samples are subjected to uniform stresses and strains.
- Different combinations of confining and axial stresses can be applied.
- Drained and undrained tests can be carried out.
- Pore water pressures can be measured in undrained tests.
- The complete stress-strain behaviour can be determined.

The choice as to which of these tests is to be used depends upon the types of soil and the problem on hand. For problems of short-term stability of foundations, excavations and earth dams UU-tests are appropriate. For problems of long-term stability, either CU-test or CD tests are appropriate, depending upon the drainage conditions in the field.

UU Tests:

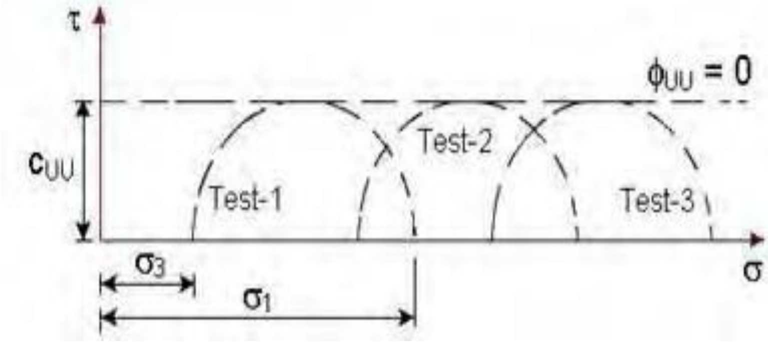

Fig 5.6 UU test failure envelope

All Mohr circles for the UU test plotted in terms of total stresses have the same diameter. The failure envelope is a horizontal straight line and hence it can be represented by the equation:

$$\tau_f = c_{UU} = \frac{\sigma_1 - \sigma_3}{2}$$

CU & CD Tests:

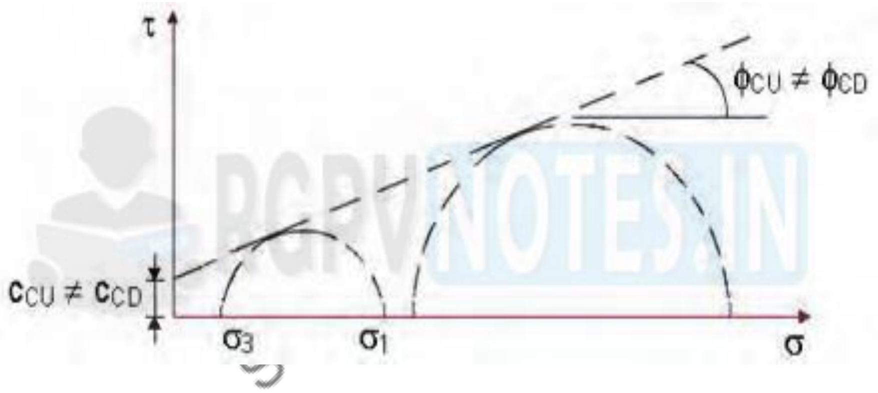

Fig 5.7 CU & CD test failure envelope

For tests involving drainage in the first stage, when Mohr circles are plotted in terms of total stresses, the diameter increases with the confining pressure. The resulting failure envelope is an inclined line with an intercept on the vertical axis. It can be stated that for identical soil samples tested under different Triaxial conditions of UU, CU and CD tests, the failure envelope is not unique.

- **Direct Shear Test:**

The test is carried out on a soil sample confined in a metal box of square cross-section which is split horizontally at mid-height. A small clearance is maintained between the two halves of the box. The soil is sheared along a predetermined plane by moving the top half of the box relative to the bottom half. The box is usually square in a plan of size 60 mm x 60 mm. A typical shear box is shown.

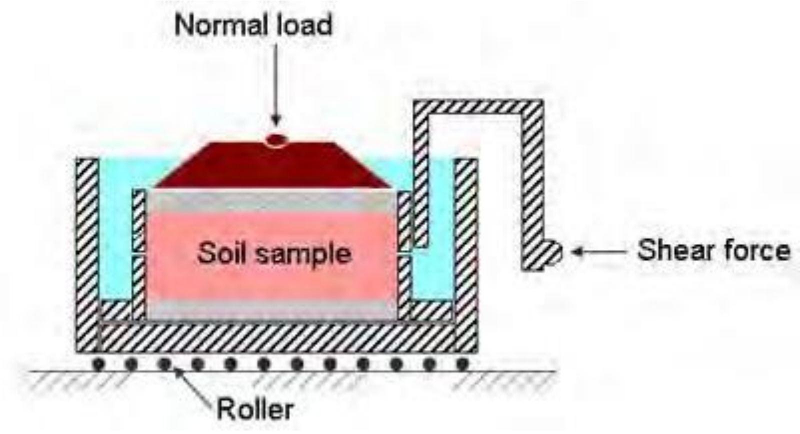

Fig 5.8 Direct shear test apparatus

If the soil sample is fully or partially saturated, perforated metal plates and porous stones are placed below and above the sample to allow free drainage. If the sample is dry, solid metal plates are used. A load normal to the plane of shearing can be applied to the soil sample through the lid of the box.

Tests on sands and gravels can be performed quickly, and are usually performed dry as it is found that water does not significantly affect the drained strength. For clays, the rate of shearing must be chosen to prevent excess pore pressures from building up.

As a vertical normal load is applied to the sample, shear stress is gradually applied horizontally, by causing the two halves of the box to move relative to each other. The shear load is measured together with the corresponding shear displacement. The change of thickness of the sample is also measured.

A number of samples of the soil are tested each under different vertical loads and the value of shear stress at failure is plotted against the normal stress for each test. Provided there is no excess pore water pressure in the soil, the total and effective stresses will be identical. From the stresses at failure, the failure envelope can be obtained.

The test has several advantages:

- **It is easy to test sands and gravels.**
- **Large samples can be tested in large shear boxes, as small samples can give misleading results due to imperfections such as fractures and fissures, or may not be truly representative.**
- **Samples can be sheared along predetermined planes when the shear strength along fissures or other selected planes are needed.**
- **The failure plane is always horizontal in the test, and this may not be the weakest plane in the sample. Failure of the soil occurs progressively from the edges towards the centre of the sample.**
- **There is no provision for measuring pore water pressure in the shear box and so it is not possible to determine effective stresses from undrained tests.**

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- **The shear box apparatus cannot give reliable undrained strengths because it is impossible to prevent localized drainage away from the shear plane.**

Effective Stress Parameters:

If the same Triaxial test results of UU, CU and CD tests are plotted in terms of effective stresses taking into consideration the measured pore water pressures, it is observed that all the Mohr circles at failure are tangent to the same failure envelope, indicating that shear strength is a unique function of the effective stress on the failure plane.

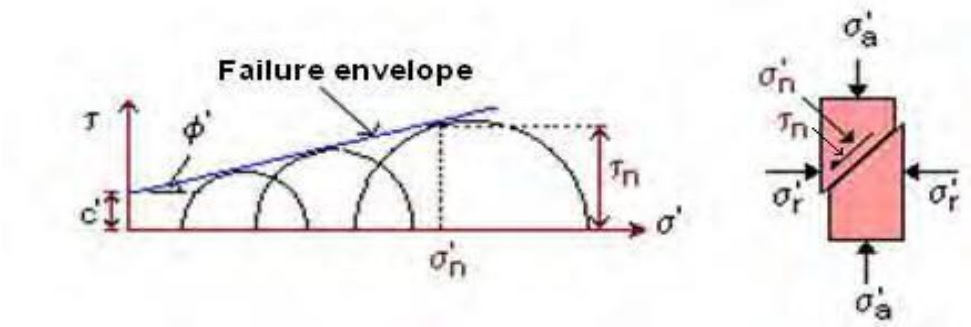

Fig 5.9 Mohr's circle failure envelope for effective stress parameter

This failure envelope is the shear strength envelope which may then be written as:

Where c' = cohesion intercept in terms of effective stress

f' = angle of shearing resistance in terms of effective stress

It is the effective stress acting on the rupture plane at failure, is the shear stress on the same plane and is, therefore, the shear strength. The relationship between the effective stresses on the failure plane is

$$\sigma_1' = \sigma_3'\left(\frac{1+\sin\phi'}{1-\sin\phi'}\right) + 2c'\sqrt{\frac{1+\sin\phi'}{1-\sin\phi'}}$$

- **Skempton's Pore Water Pressure Parameters:**

The difference between the total and effective stresses is simply the pore water pressure u. Consequently, the total and effective stress Mohr circles have the same diameter and are only separated along with the s - axis by the magnitude of the pore water pressure. It is easy to construct a series of total stress Mohr Circles but the inferred total stress parameters have no relevance to actual soil behaviour. In principle, the effective strength parameters are necessary to check the stability against failure for any soil construction in the field. To do this, the pore water pressure in the ground under the changed loading conditions must be known and in general, they are not.

In an undrained triaxial test with pore pressure measurement, this is possible and the effective stresses can then be determined. Alternatively, in drained tests, the loading rate can be made sufficiently slow so

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as to allow the dissipation of all excess pore water pressure. For low permeability soils, the drainage will require longer times.

In undrained tests, the general expression relating total pore water pressure developed and changes in applied stresses for both the stages is:

$$D_u = D_{u1} + D_{u2} = B.D_{s3} + B.A.\ (D_{s1} - D_{s3}) = B\ [D_{s3} + A\ (D_{s1} - D_{s3})]$$

Where D_{u1} = pore water pressure developed in the first stage during the application of confining stress D_{s3},

D_{u2} = pore water pressure developed in the second stage during the application of deviator stress (D_{s1} - D_{s3}),

B and A are Skempton's pore water pressure parameters.

Parameter B is a function of the degree of saturation of the soil (= 1 for saturated soils, and = 0 for dry soils). Parameter A is also not constant, and it varies with the over-consolidation ratio of the soil and also with the magnitude of deviator stress. The value of A at failure is necessary for plotting the effective stress Mohr circles. Consider the behaviour of saturated soil samples in undrained triaxial tests. In the first stage, increasing the cell pressure without allowing drainage has the effect of increasing the pore water pressure by the same amount.

Thus, there is no change in the effective stress. During the second shearing stage, the change in pore water pressure can be either positive or negative.

UU tests on saturated soils, Pore water pressure is not dissipated in both the stages ($D_u = D_{u1} + D_{u2}$),

CU tests on saturated soils, Pore water pressure is not dissipated in the second stage only ($D_u = D_{u2}$)

- **Stress-Strain Behavior of Sands**

Sands are usually sheared under drained conditions as they have relatively higher permeability. This behaviour can be investigated in direct shear or triaxial tests. The two most important parameters governing their behaviour are the relative density (I_D) and the magnitude of the effective stress (). The relative density is usually defined in percentage as

$$I_D = \frac{e_{\max} - e}{e_{\max} - e_{\min}} x100$$

Where e_{max} and e_{min} are the maximum and minimum void ratios that can be determined from standard tests in the laboratory, and e is the current void ratio. This expression can be re-written in terms of dry density as

$$I_D = \left(\frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}}\right) x \frac{\gamma_{d\max}}{\gamma_d} x100$$

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Where γ_{dmax} and γ_{dmin} are the maximum and minimum dry densities and γ_d is the current dry density. Sand is generally referred to as dense if ID > 65% and loose if < 35%. The influence of relative density on the behaviour of saturated sand can be seen from the plots of CD tests performed at the same effective confining stress. There would be no induced pore water pressures existing in the samples.

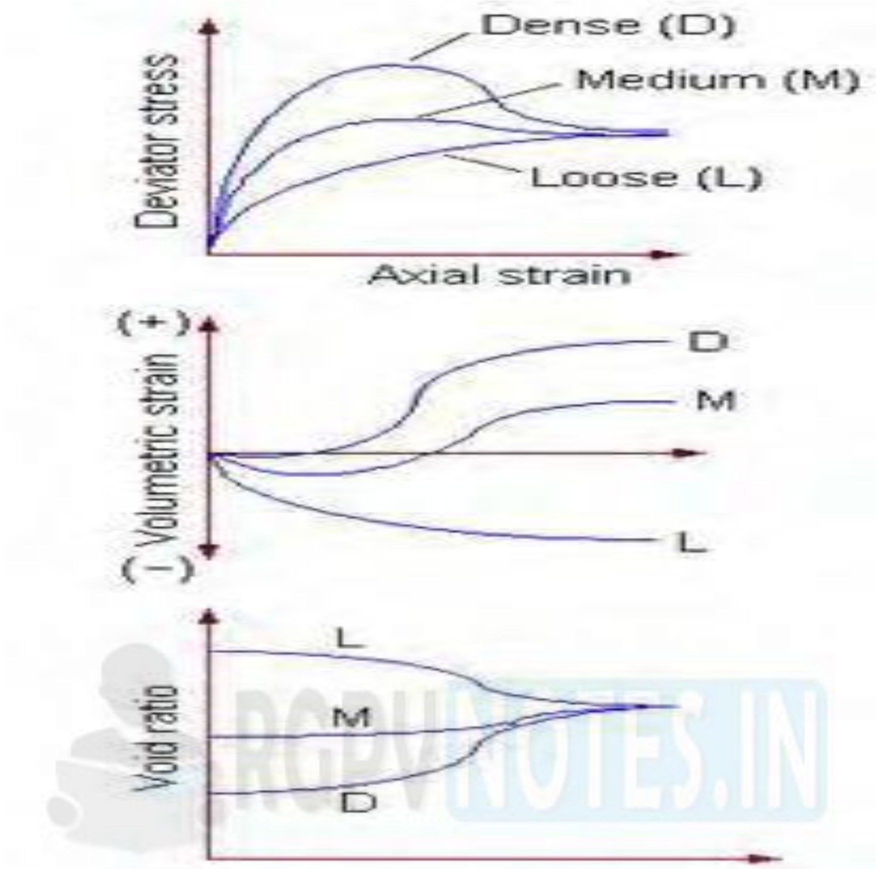


Fig 5.9 Behavior of different types of soil under loading conditions

For the dense sand sample, the deviator stress reaches a peak at a low value of axial strain and then drops down, whereas, for the loose sand sample, the deviator stress builds up gradually with axial strain. The behaviour of the medium sample is in between. The following observations can be made:

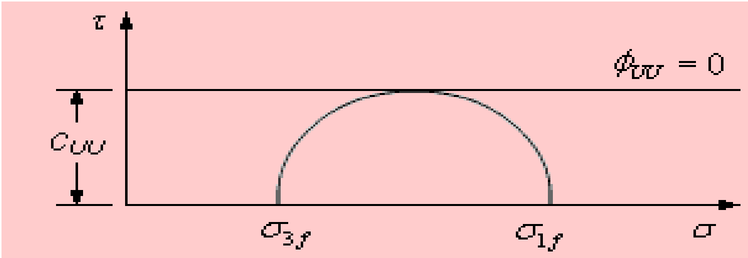
- **All samples approach the same ultimate conditions of shear stress and void ratio, irrespective of the initial density.**
- **The denser sample attains a higher peak angle of shearing resistance in between. Initially, dense samples expand or dilate when sheared and initially loose samples compress.**

Worked Examples

Example 1: A UU test is carried out on a saturated normally consolidated clay sample at a confining pressure of 3 kg/cm2.The deviator stress at failure is 1 kg/cm2.

(a) Determine its total stress strength parameters.

(b) If another identical sample is tested at a confining pressure of 4 kg/cm2, what will be the vertical axial stress at failure?



$$\sigma_{3f} = 3\ kg/cm^2$$

$$\sigma_{1f} - \sigma_{3f} = 1\ kg/cm^2$$

(a) From the plot, note that Suu = 0 and

$$c_{UU} = \frac{\sigma_{1f} - \sigma_{3f}}{2} = 0.5\ kg/cm^2$$

$$\sigma_{3f} = 4\ kg/cm^2$$

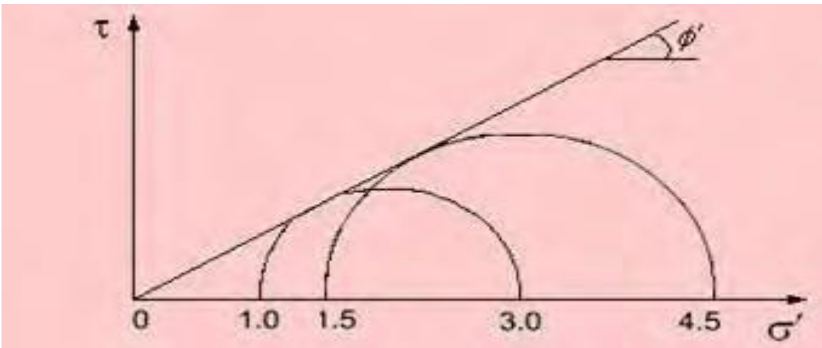
UU tests on identical samples yield the same failure deviator stress $(\sigma_{1f} - \sigma_{3f})$ **at all confining pressures.**

Therefore, the vertical axial stress at failure, $\sigma_{1f} = 4 + 1 = 5\ kg/cm^2$

Example 2: Results of tests conducted on two saturated clay samples are given. Determine the shear strength parameters.

	Sample1	Sample2
Confining pressure -----	4.8 kg/cm2	6.3 kg/cm2
Axial stress at failure --------------	6.8 kg/cm2	9.3 kg/cm2
Pore water pressure at failure ---	3.8 kg/cm2	4.8 kg/cm2

Solution:



For sample 1:

$$\sigma'_{3f} = \sigma_{3f} - u_f = 4.8 - 3.8 = 1.0\, kg/cm^2$$
$$\sigma'_{1f} = \sigma_{1f} - u_f = 6.8 - 3.8 = 3.0\, kg/cm^2$$

For sample 2:

$$\sigma'_{3f} = \sigma_{3f} - u_f = 6.3 - 4.8 = 1.5\, kg/cm^2$$
$$\sigma'_{1f} = \sigma_{1f} - u_f = 9.3 - 4.8 = 4.5\, kg/cm^2$$

From the plot, one can obtain

$$c' \approx 0$$
$$\phi' = 30°$$

- **Stabilization of soil:**

Soil stabilization is a method of improving soil properties by blending and mixing other materials.

Three types of soil stabilization Techniques stabilization have been performed for thousands of years; it wasn't too long after roads were developed that primitive engineers began looking for ways to improve them. Believe it or not, some of the original methods (or at least their spiritual descendants) are still employed today. Let's take a look at the three basic types of soil stabilization and how they work:

- **Mechanical – The oldest types of soil stabilization are mechanical in nature. Mechanical solutions involve physically changing the property of the soil somehow, in order to affect its gradation, solidity, and other characteristics. Dynamic compaction is one of the major types of soil stabilization; in this procedure, a heavyweight is dropped repeatedly onto the ground at regular intervals to quite literally pound out deformities and ensure a uniformly packed surface. Vibro compaction is another technique that works on similar principles, though it relies on vibration rather than deformation through kinetic force to achieve its goals.**
- **Chemical – Chemical solutions are another of the major types of soil stabilization. All of these techniques rely on adding additional material to the soil that will physically interact with it and change its properties. There are a number of different types of soil stabilization that rely on chemical additives of one sort or another; you will frequently encounter compounds that utilize cement, lime, fly ash, or kiln dust. Most of the reactions sought are either cementitious or pozzolanic in nature, depending on the nature of the soil present at the particular site you are investigating.**
- **Polymer/Alternative – Both of the previous types of soil stabilization have been around for hundreds of years, if not more; only in the past several decades has technology opened up new types of soil stabilization for companies to explore. Most of the newer discoveries and techniques**

developed thus far are polymer-based in nature, such as those developed by Global Road Technology. These new polymers and substances have a number of significant advantages over traditional mechanical and chemical solutions; they are cheaper and more effective in general than mechanical solutions, and significantly less dangerous for the environment than many chemical solutions tend to be.

All three types of soil stabilization are still employed on construction projects all across the globe. Following are the various soil stabilization methods and materials:

- Soil Stabilization with Cement: The soil stabilized with cement is known as soil cement. The cementing action is believed to be the result of chemical reactions of cement with siliceous soil during hydration reaction. The important factors affecting the soil-cement are the nature of soil content, conditions of mixing, compaction, curing and admixtures used.

The appropriate amounts of cement needed for different types of soils may be as follows:

- Gravels – 5 to 10%
- Sands – 7 to 12%
- Silts – 12 to 15%, and
- Clays – 12 – 20%

The quantity of cement for compressive strength of 25 to 30 kg/cm^2 should normally be sufficient for tropical climate for soil stabilization. If the layer of soil having a surface area of A (m^2), thickness H (cm) and dry density r_d (tones/m^3), has to be stabilized with p percentage of cement by weight on the basis of dry soil, cement mixture will be

$$\frac{100 \times p}{100 + p}$$

And, the amount of cement required for soil stabilization is given by

Amount of cement required, in tones = $\left(\frac{AHr_d}{100}\right) \times \left(\frac{p}{100 + p}\right)$

Lime, calcium chloride, sodium carbonate, sodium sulphate and fly ash are some of the additives commonly used with cement for cement stabilization of soil.

- Soil Stabilization using Lime: Slaked lime is very effective in treating heavy plastic clayey soils. Lime may be used alone or in combination with cement, bitumen or fly ash. Sandy soils can also be stabilized with these combinations. Lime has been mainly used for stabilizing the road bases and the subgrade. Lime changes the nature of the adsorbed layer and provides pozzolanic action. The plasticity index of highly plastic soils is reduced by the addition of lime with soil. There is an increase in the optimum water content and a decrease in the maximum compacted density and the strength

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and durability of soil increases. Normally 2 to 8% of lime may be required for coarse-grained soils and 5 to 8% of lime may be required for plastic soils. The amount of fly ash as an admixture may vary from 8 to 20% of the weight of the soil.

- **Soil Stabilization with Bitumen: Asphalts and tars are bituminous materials that are used for the stabilization of soil, generally for pavement construction. Bituminous materials when added to soil, it imparts both cohesion and reduced water absorption. Depending upon the above actions and the nature of soils, bitumen stabilization is classified into the following four types:**
  - **Sand bitumen stabilization**
  - **Soil Bitumen stabilization**
  - **Waterproofed mechanical stabilization, and**
  - **Oiled earth.**
- **Chemical Stabilization of Soil: Calcium chloride being hygroscopic and deliquescent is used as a water-retentive additive in mechanically stabilized soil bases and surfacing. The vapour pressure gets lowered, surface tension increases and the rate of evaporation decreases. The freezing point of pure water gets lowered and it results in the prevention or reduction of frost heave. The depressing the electric double layer, the salt reduces the water pick up and thus the loss of strength of fine-grained soils. Calcium chloride acts as a soil flocculent and facilitates compaction. Frequent application of calcium chloride may be necessary to make up for the loss of chemicals by leaching action. For the salt to be effective, the relative humidity of the atmosphere should be above 30%. Sodium chloride is the other chemical that can be used for this purpose with a stabilizing action similar to that of calcium chloride. Sodium silicate is yet another chemical used for this purpose in combination with other chemicals such as calcium chloride, polymers, chrome lignin, alkyl chlorosilanes, silicones, amines and quaternary ammonium salts, sodium hexametaphosphate, phosphoric acid combined with a wetting agent.**

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Figure 5.10: Chemical Stabilization of Soil

- **Electrical Stabilization of Clayey Soils: Electrical stabilization of clayey soils is done by a method known as electro-osmosis. This is an expensive method of soil stabilization and is mainly used for the drainage of cohesive soils.**
- **Soil Stabilization by Grouting: In this method, stabilizers are introduced by injection into the soil. This method is not useful for clayey soils because of their low permeability. This is a costly method for soil stabilization. This method is suitable for stabilizing buried zones of a relatively limited extent. The grouting techniques can be classified as follows:**
  - **Clay grouting**
  - **Chemical grouting**
  - **Chrome lignin grouting**
  - **Polymer grouting, and**
  - **Bituminous grouting**
- **Soil Stabilization by Geotextile and Fabrics: Geotextile is porous fabrics made of synthetic materials like polyethene, polyester, nylons polyvinyl chloride. Woven, non-woven and grid form varieties of geotextile are available. Geotextiles have high strength. When properly embedded in the soil, it contributes to its stability. It is used in the construction of unpaved roads over soft soils. Reinforcing the soil for stabilization by metallic strips into it and providing an anchor or tie back to restrain a facing skin element.**

Geosynthetics for Ground Improvement: Long ago, when difficult sites for construction purposes were to be dealt with, the conventional practice was limited to either the replacement of unsuitable soils or adopting suitable foundations which sometimes increases the cost of foundations. Innovative soil modification approaches are evolved to solve soil-related problems. One among them is the usage of geo-synthetics. When used to enhance the soil strength they have the following advantages.

- **They are space-saving,**
- **Better material quality control,**
- **Better construction quality control,**
- **Cost savings,**
- **Technical superiority,**
- **Construction time saving,**
- **Material deployment,**
- **Material availability,**
- **Environmental sensitivity.**

- **Geosynthetics**

Geosynthetics are man-made materials used to improve soil conditions. The word is derived from: Geo = earth or soil + Synthetics = man-made.

Geosynthetics are typically made from petrochemical-based polymers ("plastics") that are biologically inert and will not decompose from bacterial or fungal action. While most are essentially chemical inert, some may be damaged by petrochemicals and most have some degree of susceptibility to ultraviolet light (sunlight). Geosynthetics materials are placed on or in the soil to do one of four things (some may perform more than one of these functions simultaneously):

- **Separation/confinement/distribute loads**
  Improve level-grade soil situations such as roads, alleys, and laneways
  Improve sloped-grade situations such as banks, hillsides, stream access points
- **Reinforce soil**
  Soil walls, bridge abutments, box culverts/bridges, and soil arches
- **Prevent soil movement (piping)**
  While letting water move through the material - such as in drainage systems and backfill around water intakes
- **Controlling water pressure**
  Allowing flow (drainage) in the plane of the material such as on foundation walls to allow water to move down to perimeter drains.

A wide variety of Geosynthetics products can be used in environmental protection projects, including geomembranes, Geosynthetics clay liners (GCL), Geonets, geocomposites and pipes. Geomembranes are continuous flexible sheets manufactured from one or more synthetic materials. They are relatively impermeable and are used as liners for fluid or gas containment and as vapour barriers. Geosynthetics clay liners (GCLs) are geocomposites that are prefabricated with a Bentonite clay layer typically incorporated between a top and bottom geotextile layer or bonded to a geomembrane or single layer of geotextile. When hydrated they are effective as a barrier for liquid or gas and are commonly used in landfill liner applications often in conjunction with a geomembrane.

Geonets are open grid-like materials formed by two sets of coarse parallel, extruded polymeric strands intersecting at a constant acute angle. The network forms a sheet with in-plane porosity that is used to carry relatively large fluid or gas flows. Geocomposites are Geosynthetics made from a combination of two or more Geosynthetics types. Examples include geotextile-Geonets, geotextile-geogrid, Geonets-geomembranes, or a Geosynthetics clay liner (GCL). Geo pipes are perforated or solid-wall polymeric

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pipes used for the drainage of liquids or gas (including leachate or gas collection in landfill applications). In some cases, the perforated pipe is wrapped with a geotextile filter.

There are a number of different Geosynthetics materials, and with the similarity of many of the names, as well as many similar-sounding trade names, it can be confusing without an understanding of the basic categories. For the types typically used in agricultural applications, they fall into these categories:

- Geotextile: They are used for drainage, separation and reinforcement are in two forms
  Woven - cloth-like materials with fibres woven perpendicular to each other
  Non-woven - felt-like materials with randomly oriented fibres
- Geogrid: They are open mesh-like materials used for stabilization and reinforcement
- Geocells: They are cavity-like materials in a web used for stabilization
- Geomembranes: They are very low permeability liner or fluid containment materials
- Erosion control: These materials that are biodegradable or non-biodegradable

<u>Geotextile</u>

Geotextile is defined as "any permeable textile used with foundation soil, rock, earth, or any other geotechnical engineering-related material as an integral part of a human-made project, structure, or system". They are typically the most used Geosynthetics material for agriculture purposes.

These are fabric or cloth-like materials that are classified based on the method used to place the threads or yarns in the fabric: either woven or non-woven. Geotextiles typically come in rolls up to approximately 5.6m (18 ft) wide and 50 to 150m (160 to 500 ft) long.

Woven:

These cloth-like fabrics are formed by the uniform and regular interweaving of threads or yarns in two directions. These products have a regular visible construction pattern, and where present, have distinct and measurable openings. Woven Geotextile is typically used for soil separation, reinforcement, load distribution, filtration, and drainage. They can have high tensile strength and relative low strain or limited elongation under load (typically up to 15%).

Non-Woven:

These felt-like fabrics are formed by random placement of threads in a mat and bonded by heat-bonding, resin-bonding or needle punching. These products do not have any visible thread pattern. Non-woven Geotextile is typically used for soil separation, stabilization, load distribution, and drainage but not for soil reinforcement such as in retaining walls. They have a relatively high strain and stretch considerably under load (about 50%)

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Geogrid:

These are open grid-like materials of integrally connected polymers; they are used primarily for soil reinforcement. Their strength can be greater than the more common Geotextile. Geogrid have a low strain and stretch only about 2 to 5% under load. Where practicable they would likely be used in heavy load or high demand agricultural situations.

Soil reinforcement has seen the entry of the third type of geogrid, welded strapping (also described as strips or bars), which is rigid in structure. Produced in both polyester and polypropylene, the welded strapping grid is used in both uniaxial and biaxial applications. Properties of interest are strong junctions, excellent creep characteristics in the polyester form, and high chemical resistance. In the biaxial form, two bars are employed in the cross-machine direction giving a three-dimensional structure to aid in confinement applications.

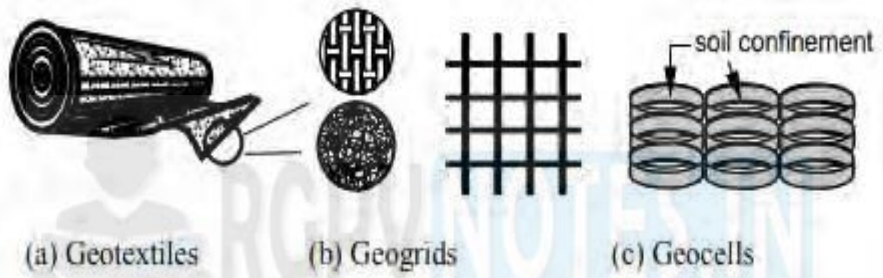

Fig 5.11 Geosynthetics commonly used for soil reinforcement

Geocells

Whereas Geotextile and Geogrid are 'flat' materials, Geocells or Geowebs have 'depth'. They are typically formed from polyethene sheets and expand out like an accordion when opened up to use. They are meant to contain soil, gravel or other fill material within their maze of cells or pockets and may be porous to allow water movement. They are used on slopes with soft subgrades and in erosion control in channels. They may be used over the top of a geotextile or geogrid. While they come in compact bundles when collapsed, they typically cover an area 2.5m (8 ft) wide by 6 to 12m (20 to 40 ft) long when expanded.

Geomembranes

Whereas Geotextile, geogrid and Geocells are usually porous to allow water to filter through them, geomembranes are polymer sheets used to control fluid movement. These materials have very low permeability and would be used for lining ponds, pits etc to control leachate. They may be used over the top of a geotextile.

Erosion Control

Various Geosynthetics materials such as tubes, mats, etc are used for erosion control. Unlike other Geosynthetics, these materials may be biodegradable and may include straw and coconut matting.

Functions of Geosynthetics

The functions include:

- **Separation: Geosynthetics, placed between two dissimilar materials, maintains the integrity and functionality of the two materials. It may also involve providing long-term stress relief. Key design properties to perform this function include those used to characterize the survivability of the Geosynthetics during installation.**
- **Filtration: Geosynthetics allows liquid flow across its plane while retaining fine particles on its upstream side. Key design properties to fulfil this function include the Geosynthetics permittivity (cross-plane hydraulic conductivity per unit thickness) and measures of the Geosynthetics pore-size distribution (e.g. apparent opening size).**
- **Reinforcement: Geosynthetics develops tensile forces intended to maintain or improve the stability of the soil Geosynthetics composite. A key design property to carry out this function is the Geosynthetics tensile strength.**
- **Stiffening: Geosynthetics develops tensile forces intended to control the deformations in the soil-Geosynthetics composite. Key design properties to accomplish this function include those used to quantify the stiffness of the soil-Geosynthetics composite.**
- **Drainage: Geosynthetics allows liquid (or gas) to flow within the plane of its structure. A key design property to quantify this function is the Geosynthetics transmissivity (in-plane hydraulic conductivity integrated over thickness).**
- **Hydraulic/Gas Barrier: Geosynthetics minimizes the cross-plane flow, providing containment of liquids or gasses. Key design properties to fulfil this function include those used to characterize the long-term durability of the Geosynthetics material.**
- **Protection: Geosynthetics provides a cushion above or below other material (e.g. geomembranes) in order to minimize damage during placement of overlying materials. Key design properties to quantify this function include those used to characterize the puncture resistance of the Geosynthetics material.**