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Introduction to Soil Mechanics

The term "**soil**" can have different meanings, depending upon the field in which it is considered.

To a geologist, it is the material in the relative thin zone of the Earth's surface within which roots occur, and which are formed as the products of past surface processes. The rest of the crust is grouped under the term "rock".

To a pedologist, it is the substance existing on the surface, which supports plant life.

To an engineer, it is a material that can be:

- **built on:** foundations of buildings, bridges
- **built in:** basements, culverts, tunnels
- **built with:** embankments, roads, dams
- **supported:** retaining walls

Soil Mechanics is a discipline of Civil Engineering involving the study of soil, its behaviour and application as an engineering material.

Soil Mechanics is the application of laws of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulations of solid particles, which are produced by the mechanical and chemical disintegration of rocks, regardless of whether or not they contain an admixture of organic constituents.

Soil consists of a multiphase aggregation of solid particles, water, and air. This fundamental composition gives rise to unique engineering properties, and the description of its mechanical behavior requires some of the most classic principles of engineering mechanics.

Engineers are concerned with soil's mechanical properties: permeability, stiffness, and strength. These depend primarily on the nature of the soil grains, the current stress, the water content and unit weight.

Formation of Soils

In the Earth's surface, rocks extend upto as much as 20 km depth. The major rock types are categorized as igneous, sedimentary, and metamorphic.

- **Igneous rocks:** formed from crystalline bodies of cooled magma.
- **Sedimentary rocks:** formed from layers of cemented sediments.
- **Metamorphic rocks:** formed by the alteration of existing rocks due to heat from igneous intrusions or pressure due to crustal movement.

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.

All soils originate, directly or indirectly, from different rock types.

Physical weathering reduces the size of the parent rock material, without any change in the original composition of the parent rock. Physical or mechanical processes taking place on the earth's surface include the actions of water, frost, temperature changes, wind and ice. They cause disintegration and the products are mainly coarse soils.

The main processes involved are exfoliation, unloading, erosion, freezing, and thawing. The principal cause is climatic change. In exfoliation, the outer shell separates from the main rock. Heavy rain and wind cause erosion of the rock surface. Adverse temperature changes produce fragments due to different thermal coefficients of rock minerals. The effect is more for freeze-thaw cycles.

Chemical weathering not only breaks up the material into smaller particles but alters the nature of the original parent rock itself. The main processes responsible are hydration, oxidation, and carbonation. New compounds are formed due to the chemical alterations.

Rain water that comes in contact with the rock surface reacts to form hydrated oxides, carbonates and sulphates. If there is a volume increase, the disintegration continues. Due to leaching, water-soluble materials are washed away and rocks lose their cementing properties.

Chemical weathering occurs in wet and warm conditions and consists of degradation by decomposition and/or alteration. The results of chemical weathering are generally fine soils with altered mineral grains.

The effects of weathering and transportation mainly determine the basic **nature** of the soil (size, shape, composition and distribution of the particles).

The environment into which deposition takes place, and the subsequent geological events that take place there, determine the **state** of the soil (density, moisture content) and the **structure** or fabric of the soil (bedding, stratification, occurrence of joints or fissures)

Transportation agencies can be combinations of gravity, flowing water or air, and moving ice. In water or air, the grains become sub-rounded or rounded, and the grain sizes get sorted so as to form poorly-graded deposits. In moving ice, grinding and crushing occur, size distribution becomes wider forming well-graded deposits.

In running water, soil can be transported in the form of suspended particles, or by rolling and sliding along the bottom. Coarser particles settle when a decrease in velocity occurs, whereas finer particles are deposited further downstream. In still water, horizontal layers of successive sediments are formed, which may change with time, even seasonally or daily.

Wind can erode, transport and deposit fine-grained soils. Wind-blown soil is generally uniformly-graded.

A glacier moves slowly but scours the bedrock surface over which it passes.

Gravity transports materials along slopes without causing much alteration.

Soil Types

Soils as they are found in different regions can be classified into two broad categories:

- (1) **Residual soils**
- (2) **Transported soils**

Residual Soils

Residual soils are found at the same location where they have been formed. Generally, the depth of residual soils varies from 5 to 20 m.

Chemical weathering rate is greater in warm, humid regions than in cold, dry regions causing a faster breakdown of rocks. Accumulation of residual soils takes place as the rate of rock decomposition exceeds the rate of erosion or transportation of the weathered material. In humid regions, the presence of surface vegetation reduces the possibility of soil transportation.

As leaching action due to percolating surface water decreases with depth, there is a corresponding decrease in the degree of chemical weathering from the ground surface downwards. This results in a gradual reduction of residual soil formation with depth, until unaltered rock is found.

Residual soils comprise of a wide range of particle sizes, shapes and composition.

Transported Soils

Weathered rock materials can be moved from their original site to new locations by one or more of the transportation agencies to form transported soils. Transported soils are classified based on the mode of transportation and the final deposition environment.

(a) Soils that are carried and deposited by rivers are called ***alluvial deposits***.

(b) Soils that are deposited by flowing water or surface runoff while entering a lake are called ***lacustrine deposits***. Alternate layers are formed in different seasons depending on flow rate.

(c) If the deposits are made by rivers in sea water, they are called ***marine deposits***. Marine deposits contain both particulate material brought from the shore as well as organic remnants of marine life forms.

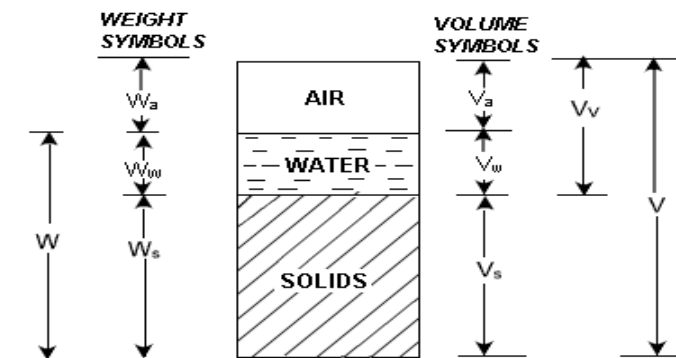
(d) Melting of a glacier causes the deposition of all the materials scoured by it leading to formation of ***glacial deposits***.

(e) Soil particles carried by wind and subsequently deposited are known as ***aeolian deposits***.

Phase Relations of Soils

Soil is not a coherent solid material like steel and concrete, but is a particulate material. Soils, as they exist in nature, consist of solid particles (mineral grains, rock fragments) with water and air in the voids between the particles. The water and air contents are readily changed by changes in ambient conditions and location.

As the relative proportions of the three phases vary in any soil deposit, it is useful to consider a soil model which will represent these phases distinctly and properly quantify the amount of each phase. A schematic diagram of the three-phase system is shown in terms of weight and volume symbols respectively for soil solids, water, and air. The weight of air can be neglected.



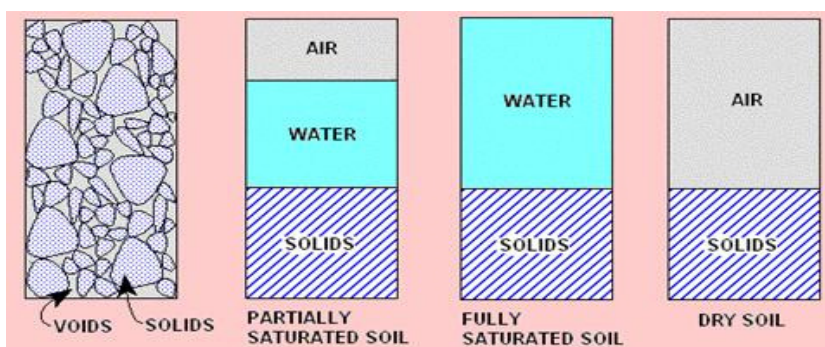
The soil model is given dimensional values for the solid, water and air components.

Total volume, $V = V_s + V_w + V_v$

Three-phase System

Soils can be partially saturated (with both air and water present), or be fully saturated (no air content) or be perfectly dry (no water content).

In a saturated soil or a dry soil, the three-phase system thus reduces to two phases only, as shown.



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**

Volume 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**:

1.Void ratio (e) is the ratio of the volume of voids (V_v) to the volume of soil solids (V_s), and is expressed as a decimal.

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

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

$$n = \frac{V_v}{V} \times 100$$

percentage.

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

$$e = \frac{n}{1-n} \quad \text{and} \quad n = \frac{e}{(1+e)}$$

3. 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.

$$S = \frac{V_w}{V_v} \times 100$$

For a dry soil, $S = 0\%$, and for a fully saturated soil, $S = 100\%$.

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

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

5.Percentage air voids (n_a) is the ratio of the volume of air to the total volume.

$$n_a = \frac{V_a}{V} \times 100 = n \times \alpha_c$$

Weight Relations

Density is a measure of the quantity of mass in a unit volume of material. Unit weight is a measure of the weight of a unit volume of material. Both can be used interchangeably. The units of density are ton/m³, kg/m³ or g/cm³. The following are the **basic weight relations**:

1. The ratio of the mass of water present to the mass of solid particles is called the **water content (w)**, or sometimes the **moisture content**.

$$w = \frac{W_w}{W_s}$$

Its value is 0% for dry soil and its magnitude can exceed 100%.

2. The mass of solid particles is usually expressed in terms of their **particle unit weight** (γ_s) or **specific gravity** (G_s) of the soil grain solids .

$$\gamma_s = \frac{W_s}{V_s} = G_s \cdot \gamma_w$$

where γ_w = Unit weight of water

For most inorganic soils, the value of G_s lies between 2.60 and 2.80. The presence of organic material reduces the value of G_s .

3. **Dry unit weight** (γ_d) is a measure of the amount of solid particles per unit volume.

$$\gamma_d = \frac{W_s}{V}$$

4. **Bulk unit weight** (γ_t or γ) is a measure of the amount of solid particles plus water per unit volume.

$$\gamma_t = \gamma = \frac{(W_s + W_w)}{(V_s + V_v)}$$

5. **Saturated unit weight** (γ_{sat}) is equal to the bulk density when the total voids is filled up with water.

6. **Buoyant unit weight** (γ') or **submerged unit weight** is the effective mass per unit volume when the soil is submerged below standing water or below the ground water table.

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

Inter-Relations

It is important to quantify the state of a soil immediately after receiving in the laboratory and prior to commencing other tests. The water content and unit weight are particularly important, since they may change during transportation and storage.

Some physical state properties are calculated following the practical measurement of others. For example, dry unit weight can be determined from bulk unit weight and water content. The following are some **inter-relations**:

1.

$$w = \frac{W_w}{W_s} = \frac{\gamma_w V_w}{G_s \gamma_w V_s} = \frac{V_w}{G_s V_s} = \frac{S V_v}{G_s V_s} = \frac{S e}{G_s}$$

2.

$$\gamma = \frac{(G_s + S e) \gamma_w}{1 + e}$$

3.

$$\gamma = \frac{(1 + w) G_s \gamma_w}{1 + e}$$

4.

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

5.

$$\gamma_d = \frac{\gamma}{1 + w}$$

6.

$$\gamma' = \frac{[(G_s - 1) + (S - 1)e] \gamma_w}{1 + e}$$

7.

$$\gamma' = \frac{(G_s - 1) \gamma_w}{1 + e}$$

Worked Examples

Example 1: A soil has void ratio = 0.72, moisture content = 12% and $G_s = 2.72$. Determine its

- (a) Dry unit weight
- (b) Moist unit weight, and the
- (c) Amount of water to be added per m^3 to make it saturated.

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

Solution:

$$(a) \gamma_d = \frac{G_s \cdot \gamma_w}{1+e} = \frac{2.72 \times 9.81}{1+0.72} = 15.51 \text{ kN/m}^3$$

$$(b) \gamma = \gamma_d(1+w) \\ = \frac{1+0.12}{1+0.72} \times 2.72 \times 9.81 = 17.38 \text{ kN/m}^3$$

$$(c) \gamma_{sat} = \frac{G_s + e}{1+e} \cdot \gamma_w \\ = \frac{2.72+0.72}{1+0.72} \times 9.81 = 19.62 \text{ kN/m}^3$$

Water to be added per m^3 to make the soil saturated

$$= \gamma_{sat} - \gamma = 19.62 - 17.38 = 2.24 \text{ kN}$$

Example 2: The dry density of a sand with porosity of 0.387 is 1600 kg/m^3 . Find the void ratio of the soil and the specific gravity of the soil solids. [Take $\gamma_w = 1000 \text{ kg/m}^3$]

$$n = 0.387$$

$$\gamma_d = 1600 \text{ kg/m}^3$$

Solution:

$$(a) e = \frac{n}{1-n} = \frac{0.387}{1-0.387} = 0.631$$

$$(b) \gamma_d = \frac{G_s \cdot \gamma_w}{1+e}$$

$$\therefore G_s = \frac{(1+e)}{\gamma_w} \cdot \gamma_d = \frac{1+0.631}{1000} \times 1600 = 2.61$$

Soil Classification

It is necessary to adopt a formal system of soil description and classification in order to describe the various materials found in ground investigation. Such a system must be meaningful and concise in an engineering context, so that engineers will be able to understand and interpret.

It is important to distinguish between description and classification:

Description of soil is a statement that describes the physical nature and state of the soil. It can be a description of a sample, or a soil *in situ*. It is arrived at by using visual examination, simple tests, observation of site conditions, geological history, etc.

Classification of soil is the separation of soil into classes or groups each having similar characteristics and potentially similar behaviour. A classification for engineering purposes should be based mainly on mechanical properties: permeability, stiffness, strength. The class to which a soil belongs can be used in its description.

The aim of a classification system is to establish a set of conditions which will allow useful comparisons to be made between different soils. The system must be simple. The relevant criteria for classifying soils are the **size distribution** of particles and the **plasticity** of the soil.

Particle Size Distribution

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).

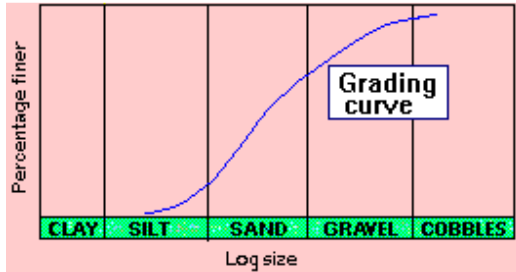
Sedimentation analysis is used only for the soil fraction finer than 75 microns. Soil particles are allowed to settle from a suspension. The decreasing density of the suspension is measured at various time intervals. The procedure is based on the principle that in a suspension, the terminal velocity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

Grain-Size Distribution Curve

The size distribution curves, as obtained from coarse and fine grained portions, can be combined to form one complete **grain-size distribution curve** (also known as **grading curve**). A typical grading curve is shown.

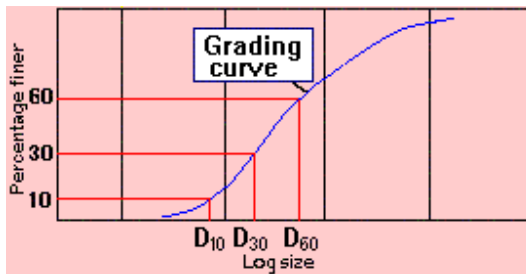


From the complete grain-size distribution curve, useful information can be obtained such as:

1. **Grading characteristics**, which indicate the uniformity and range in grain-size distribution.
2. **Percentages** (or **fractions**) of gravel, sand, silt and clay-size.

Grading Characteristics

A grading curve is a useful aid to soil description. The geometric properties of a grading curve are called **grading characteristics**.



To obtain the grading characteristics, three points are located first on the grading curve.

D_{60} = size at 60% finer by weight

D_{30} = size at 30% finer by weight

D_{10} = size at 10% finer by weight

The grading characteristics are then determined as follows:

1. **Effective size** = D_{10}
2. **Uniformity coefficient**,
$$C_u = \frac{D_{60}}{D_{10}}$$
3. **Curvature coefficient**,
$$C_c = \frac{(D_{30})^2}{D_{60} \cdot D_{10}}$$

Both C_u and C_c will be 1 for a single-sized soil.

$C_u > 5$ indicates a **well-graded soil**, i.e. a soil which has a distribution of particles over a wide size range.

C_c between 1 and 3 also indicates a well-graded soil.

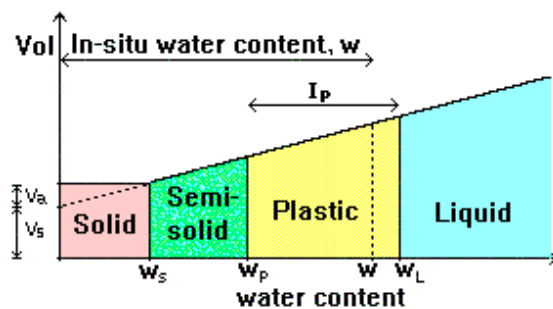
$C_u < 3$ indicates a **uniform soil**, i.e. a soil which has a very narrow particle size range.

Consistency of Soils

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**).

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



Two of these are utilised 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/crumby to plastic state

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.

Indian Standard Soil Classification System

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 clays contain particles less than 0.001 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.

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

Gravel, sand, silt, and clay are represented by **group symbols G, S, M, and C** respectively.

Physical weathering produces very coarse and coarse soils. Chemical weathering produce generally fine soils.

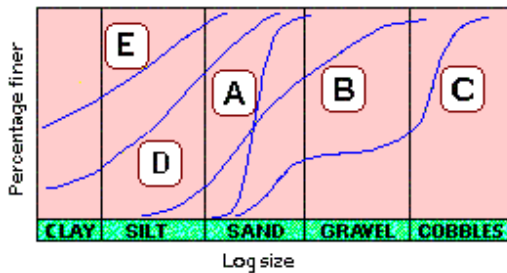
Indian Standard Soil Classification System

Coarse-grained soils are those for which more than 50% of the soil material by weight has particle sizes greater than 0.075 mm. They are basically divided into either **gravels (G)** or **sands (S)**.

According to **gradation**, they are further grouped as well-graded (**W**) or poorly graded (**P**). If **fine soils** are present, they are grouped as containing silt fines (**M**) or as containing clay fines (**C**).

For example, the combined symbol **SW** refers to well-graded sand with no fines.

Both the position and the shape of the grading curve for a soil can aid in establishing its identity and description. Some typical grading curves are shown.



Curve A - a poorly-graded medium SAND

Curve B - a well-graded GRAVEL-SAND (i.e. having equal amounts of gravel and sand)

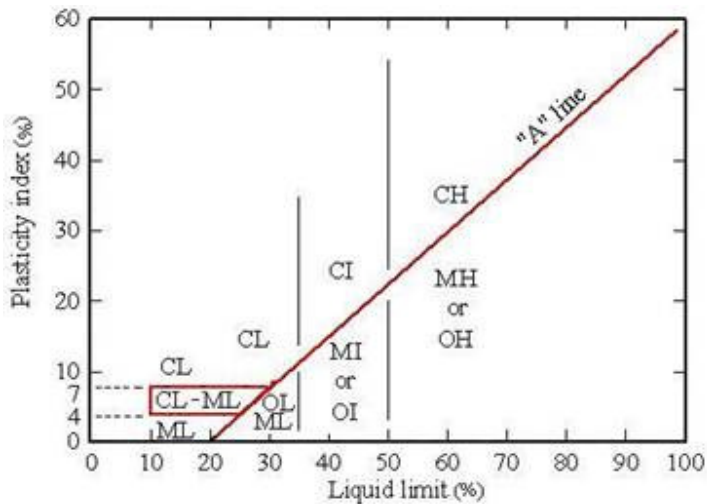
Curve C - a gap-graded COBBLES-SAND

Curve D - a sandy SILT

Curve E - a silty CLAY (i.e. having little amount of sand)

Fine-grained soils are those for which more than 50% of the material has particle sizes less than 0.075 mm. Clay particles have a **flaky** shape to which water adheres, thus imparting the property of **plasticity**.

A **plasticity chart**, based on the values of liquid limit (W_L) and plasticity index (I_p), is provided in **ISSCS** to aid classification. The '**A**' line in this chart is expressed as $I_p = 0.73 (W_L - 20)$.



Depending on the point in the chart, fine soils are divided into **clays (C)**, **silts (M)**, or **organic soils (O)**. The organic content is expressed as a percentage of the mass of organic matter in a given mass of soil to the mass of the dry soil solids. Three divisions of plasticity are also defined as follows.

Low plasticity	$W_L < 35\%$
Intermediate plasticity	$35\% < W_L < 50\%$
High plasticity	$W_L > 50\%$

The 'A' line and vertical lines at W_L equal to **35%** and **50%** separate the soils into various classes.

For example, the combined symbol **CH** refers to clay of high plasticity.

Soil classification using group symbols is as follows:

Group Symbol	Classification
Coarse soils	
GW	Well-graded GRAVEL
GP	Poorly-graded GRAVEL
GM	Silty GRAVEL
GC	Clayey GRAVEL
SW	Well-graded SAND
SP	Poorly-graded SAND
SM	Silty SAND
SC	Clayey SAND
Fine soils	
ML	SILT of low plasticity
MI	SILT of intermediate plasticity
MH	SILT of high plasticity
CL	CLAY of low plasticity

CI	CLAY of intermediate plasticity
CH	CLAY of high plasticity
OL	Organic soil of low plasticity
OI	Organic soil of intermediate plasticity
OH	Organic soil of high plasticity
Pt	Peat

Activity

"Clayey soils" necessarily do not consist of 100% clay size particles. The proportion of clay mineral flakes (< 0.002 mm size) in a fine soil increases its tendency to swell and shrink with changes in water content. This is called the **activity** of the clayey soil, and it represents the degree of plasticity related to the clay content.

Activity = (Plasticity index) / (% clay particles by weight)

Classification as per activity is:

Activity	Classification
< 0.75	Inactive
0.75 - 1.25	Normal
> 1.25	Active

Liquidity Index

In fine soils, especially with clay size content, the existing state is dependent on the current water content (**w**) with respect to the consistency limits (or Atterberg limits). The **liquidity index (LI)** provides a quantitative measure of the present state.

$$LI = \frac{w - W_p}{I_p}$$

Classification as per liquidity index is:

Liquidity index	Classification
> 1	Liquid
0.75 - 1.00	Very soft
0.50 - 0.75	Soft
0.25 - 0.50	Medium stiff
0 - 0.25	Stiff
< 0	Semi-solid

Visual Classification

Soils possess a number of physical characteristics which can be used as aids to identification in the field. A handful of soil rubbed through the fingers can yield the following:

SAND (and coarser) particles are visible to the naked eye.

SILT particles become dusty when dry and are easily brushed off hands.

CLAY particles are sticky when wet and hard when dry, and have to be scraped or washed off hands.

Worked Example

The following test results were obtained for a fine-grained soil:

$W_L = 48\%$; $W_P = 26\%$

Clay content = 55%

Silt content = 35%

Sand content = 10%

In situ moisture content = 39% = w

Classify the soil, and determine its activity and liquidity index

Solution:

Plasticity index, $I_P = W_L - W_P = 48 - 26 = 22\%$

Liquid limit lies between 35% and 50%.

According to the Plasticity Chart, the soil is classified as CI, i.e. clay of intermediate plasticity.

$$\Rightarrow \text{Activity} = \frac{I_P}{\text{Clay content}} = \frac{22}{25} = 0.88$$

$$LI = \frac{w - W_P}{I_P} = \frac{39 - 26}{22} = 0.59$$

Liquidity index ,

The clay is of normal activity and is of soft consistency.

Formation of Clay Minerals

A soil particle may be a mineral or a rock fragment. A mineral is a chemical compound formed in nature during a geological process, whereas a rock fragment has a combination of one or more minerals. Based on the nature of atoms, minerals are classified as silicates, aluminates, oxides, carbonates and phosphates.

Out of these, silicate minerals are the most important as they influence the properties of clay soils. Different arrangements of atoms in the silicate minerals give rise to different silicate structures.

Basic Structural Units

Soil minerals are formed from two basic structural units: tetrahedral and octahedral. Considering the valencies of the atoms forming the units, it is clear that the units are not electrically neutral and as such do not exist as single units.

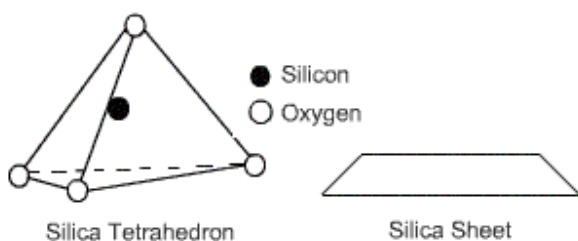
The basic units combine to form sheets in which the oxygen or hydroxyl ions are shared among adjacent units. Three types of sheets are thus formed, namely **silica sheet**, **gibbsite sheet** and **brucite sheet**.

Isomorphous substitution is the replacement of the central atom of the tetrahedral or octahedral unit by another atom during the formation of the sheets.

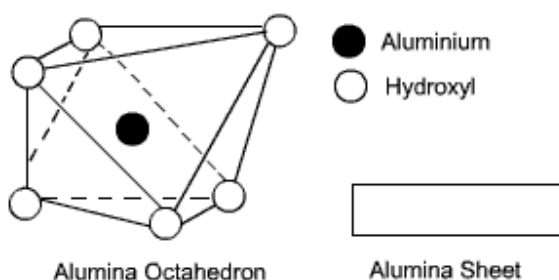
The sheets then combine to form various two-layer or three-layer sheet minerals. As the basic units of clay minerals are sheet-like structures, the particle formed from stacking of the basic units is also plate-like. As a result, the surface area per unit mass becomes very large.

Structure of Clay Minerals

A tetrahedral unit consists of a central silicon atom that is surrounded by four oxygen atoms located at the corners of a tetrahedron. A combination of tetrahedrons forms a **silica sheet**.



An octahedral unit consists of a central ion, either aluminium or magnesium, that is surrounded by six hydroxyl ions located at the corners of an octahedron. A combination of aluminium-hydroxyl octahedrons forms a **gibbsite sheet**, whereas a combination of magnesium-hydroxyl octahedrons forms a **brucite sheet**.



Two-layer Sheet Minerals

Kaolinite and halloysite clay minerals are the most common.

Kaolinite Mineral

The basic kaolinite unit is a two-layer unit that is formed by stacking a gibbsite sheet on a silica sheet. These basic units are then stacked one on top of the other to form a lattice of the mineral. The units are held together by hydrogen bonds. The strong bonding does not permit water to enter the lattice. Thus, kaolinite minerals are stable and do not expand under saturation.

Kaolinite is the most abundant constituent of residual clay deposits.

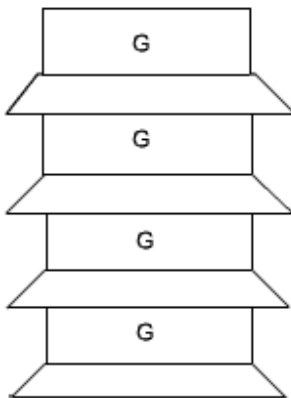


Fig: Kaolinite Mineral

Halloysite Mineral

The basic unit is also a two-layer sheet similar to that of kaolinite except for the presence of water between the sheets.

Three-layer Sheet Minerals

Montmorillonite and illite clay minerals are the most common. A basic three-layer sheet unit is formed by keeping one silica sheet each on the top and at the bottom of a gibbsite sheet. These units are stacked to form a lattice as shown.

Montmorillonite Mineral

The bonding between the three-layer units is by van der Waals forces. This bonding is very weak and water can enter easily. Thus, this mineral can imbibe a large quantity of water causing swelling. During dry weather, there will be shrinkage.

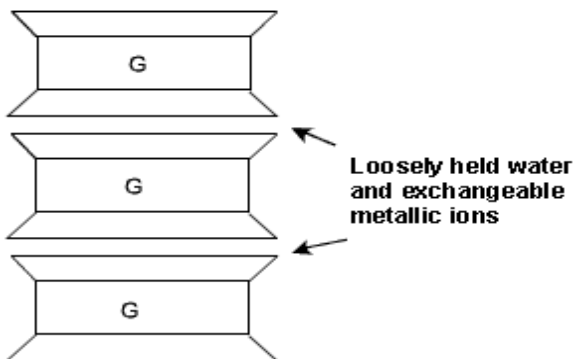


Fig: Montmorillonite Mineral

Illite Mineral

Illite consists of the basic montmorillonite units but are bonded by **secondary valence forces** and **potassium ions**, as shown. There is about 20% replacement of aluminium with silicon in the gibbsite sheet due to **isomorphous substitution**. This mineral is very stable and does not swell or shrink.

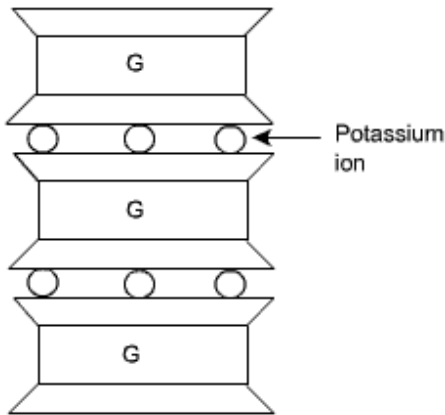
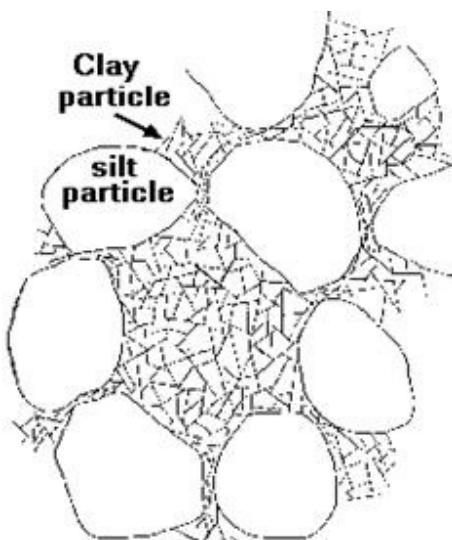


Fig: Illite Mineral

Fine Soil Fabric

Natural soils are rarely the same from one point in the ground to another. The content and nature of grains varies, but more importantly, so does the arrangement of these. The arrangement and organisation of particles and other features within a soil mass is termed its **fabric**.

CLAY particles are **flaky**. Their thickness is very small relative to their length & breadth, in some cases as thin as 1/100th of the length. They therefore have high specific surface values. These surfaces carry negative electrical charge, which attracts positive ions present in the pore water. Thus a lot of water may be held as adsorbed water within a clay mass.



Nature of Water in Clay

The presence of exchangeable cat ions on the surface of clay particles was discussed in the preceding section. Some salt precipitates (cat ions in excess of the exchangeable ions and their associated anions) are also present on the surface of dry clay particles. When water is added to clay, these cat ions and anions float around the clay particles (**figure 1.16**).

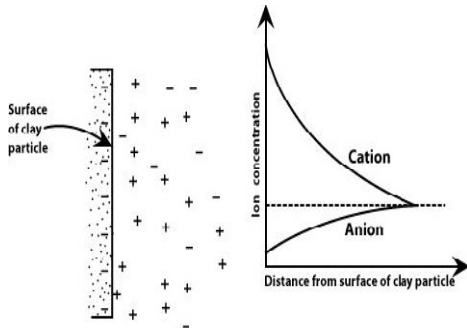


Figure 1.16 Diffuse double layer.

At this point, it must be pointed out that water molecules are dipolar, since the hydrogen atoms are not symmetrically arranged around the oxygen atoms (**figure 1.17a**). This means that a molecule of water is like a rod with positive and negative charges at opposite ends (**figure 1.17b**). There are three general mechanisms by which these dipolar water molecules, or *dipoles*, can be electrically attracted toward the surface of the clay particles (**figure 1.18**).

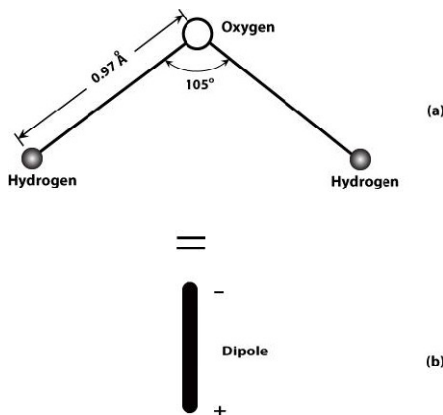


Figure 1.17 Dipolar nature of water

- Attraction between the negatively charged faces of clay particles and the positive ends of dipoles.
- Attraction between cations in the double layer and the negatively charged ends of dipoles. The cations are in turn attracted by the negatively charged faces of clay particles
- Sharing of the hydrogen atoms in the water molecules by hydrogen bonding between the oxygen atoms in the clay particles and the oxygen atoms in the water molecules.

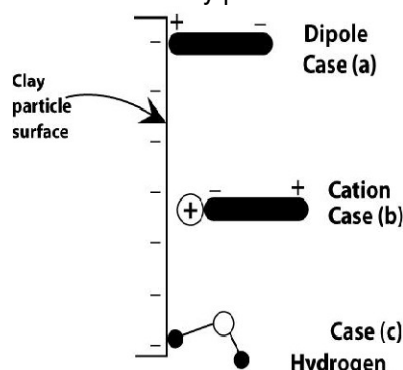


Figure 1.18 Dipolar water molecules in diffuse

Flocculation and Dispersion of Clay Particles

In addition to the repulsive force between the clay particles there is an attractive force, which is largely attributed to the Van de Waal's force. This is a secondary bonding force that acts between all adjacent pieces of mater. The force between two flat parallel surfaces varies inversely as $1/x^3$ to $1/x^4$, which x is the distance between the two surfaces. Van der Waal's force is also dependent on the dielectric constant of the medium separating the surfaces. However, if water is the separating medium, substantial changes in the magnitude of the force will not occur with minor changes in the constitution of water. The behavior of clay particles in a suspension can be qualitatively visualized from our understanding of the attractive and repulsive forces between the particles and with the aid of **figure 1.29**. Consider a dilute suspension of clay particles in water. These colloidal clay particles will undergo Brownian movement and, during this random movement, will come close to each other at distance within the range of interparticle forces. The forces of attraction and repulsion between the clay particles vary at different rates with respect to the distance of separation. The force of repulsion decreases exponentially with distance, whereas the force of attraction decreases as the inverse third or fourth power of distance, as shown in **figure 1.29**. Depending on the distance of separation, if the magnitude of the repulsive force is greater than the magnitude of the attractive force, the net result will be repulsion. The clay particles will settle individually and form a dense layer at the bottom; however, they will remain separate from their neighbors (**figure 1.30a**). This is referred to as the *dispersed state* of the soil. On the other hand, if the net force between the particles is attraction, flocs will be formed and these flocs will settle to the bottom. This is called *flocculated clay* (**figure 1.30b**).

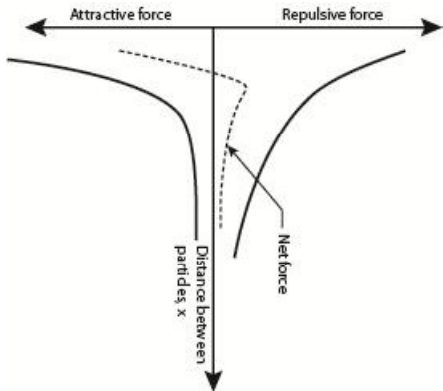


figure 1.29 Dispersion and flocculation of clay in a suspension



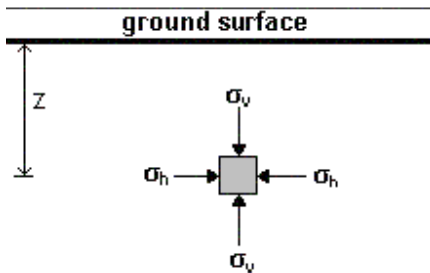
Figure 1.30 (a) Dispersion and (b) flocculation of clay

Stresses in the Ground

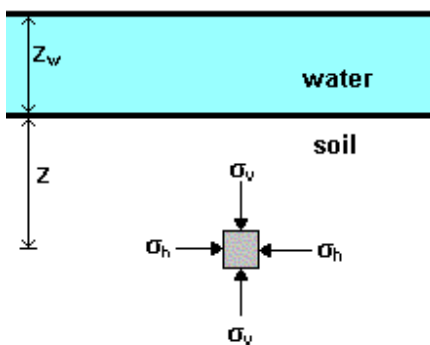
Total Stress

When a load is applied to soil, it is carried by the solid grains and the water in the pores. The **total vertical stress** acting at a point below the ground surface is due to the weight of everything that lies above, including soil, water, and surface loading. Total stress thus increases with depth and with unit weight.

Vertical total stress at depth z , $\sigma_v = \gamma \cdot Z$



Below a water body, the total stress is the sum of the weight of the soil up to the surface and the weight of water above this. $\sigma_v = \gamma \cdot Z + \gamma_w \cdot Z_w$

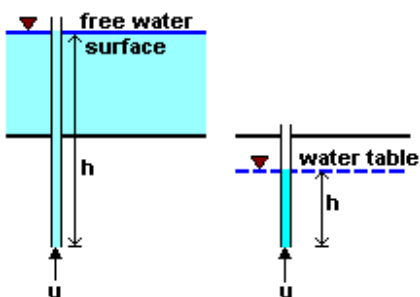


The total stress may also be denoted by σ_z or just σ . It varies with changes in water level and with excavation.

Pore Water Pressure

The pressure of water in the pores of the soil is called **pore water pressure (u)**. The magnitude of pore water pressure depends on:

- the depth below the water table.
- the conditions of seepage flow.



Under hydrostatic conditions, no water flow takes place, and the pore pressure at a given point is given by $u = \gamma_w \cdot h$

where h = depth below water table or overlying water surface

It is convenient to think of pore water pressure as the pressure exerted by a column of water in an imaginary standpipe inserted at the given point.

The natural level of ground water is called the **water table** or the **phreatic surface**. Under conditions of no seepage flow, the water table is horizontal. The magnitude of the pore water pressure at the water table is zero. Below the water table, pore water pressures are positive.

Principle of Effective Stress

The **principle of effective stress** was enunciated by **Karl Terzaghi** in the year 1936. This principle is valid only for saturated soils, and consists of two parts:

1. At any point in a soil mass, the effective stress (represented by $\bar{\sigma}$ or σ') is related to total stress (σ) and pore water pressure (u) as

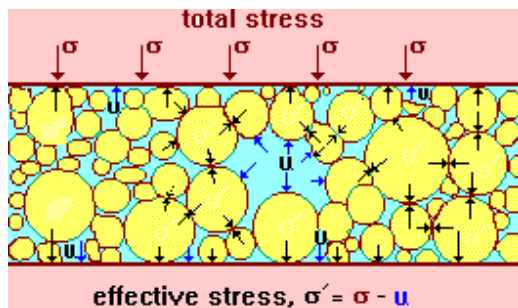
$$\bar{\sigma} = \sigma - u$$

Both the total stress and pore water pressure can be measured at any point.

2. All measurable effects of a change of stress, such as compression and a change of shearing resistance, are exclusively due to changes in effective stress.

$$\text{Compression} = f_1(\bar{\sigma})$$

$$\text{Shear Strength} = f_2(\bar{\sigma})$$



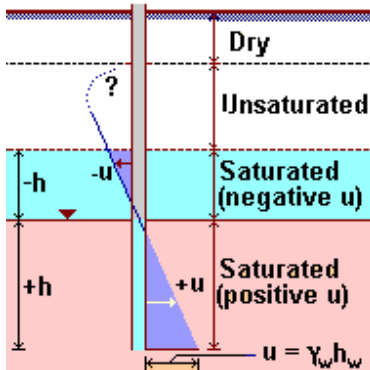
In a saturated soil system, as the voids are completely filled with water, the pore water pressure acts equally in all directions.

The effective stress is not the exact contact stress between particles but the distribution of load carried by the soil particles over the area considered. It cannot be measured and can only be computed.

If the total stress is increased due to additional load applied to the soil, the pore water pressure initially increases to counteract the additional stress. This increase in pressure within the pores might cause water to drain out of the soil mass, and the load is transferred to the solid grains. This will lead to the increase of effective stress.

Effective Stress in Unsaturated Zone

Above the water table, when the soil is saturated, pore pressure will be negative (less than atmospheric). The height above the water table to which the soil is saturated is called the **capillary rise**, and this depends on the grain size and the size of pores. In coarse soils, the capillary rise is very small.



Between the top of the saturated zone and the ground surface, the soil is partially saturated, with a consequent reduction in unit weight. The pore pressure in a partially saturated soil consists of two components:

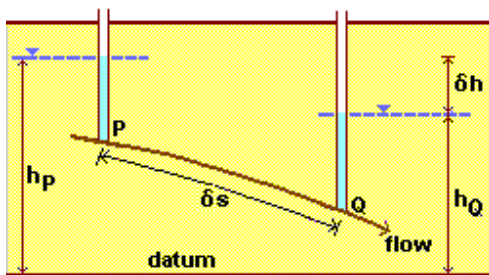
Pore water pressure = u_w

Pore air pressure = u_a

Water is incompressible, whereas air is compressible. The combined effect is a complex relationship involving partial pressures and the degree of saturation of the soil.

Effective Stress Under Hydrodynamic Conditions

There is a change in pore water pressure in conditions of **seepage flow** within the ground. Consider seepage occurring between two points **P** and **Q**. The potential driving the water flow is the hydraulic gradient between the two points, which is equal to the head drop per unit length. In steady state seepage, the gradient remains constant.



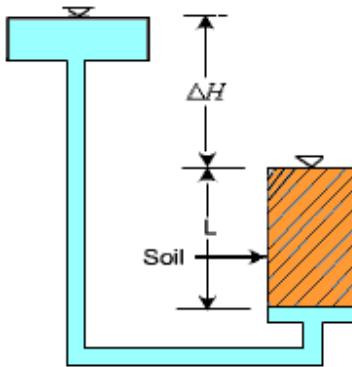
Hydraulic gradient from P to Q, $i = \delta h / \delta s$

As water percolates through soil, it exerts a drag on soil particles it comes in contact with. Depending on the flow direction, either downward or upward, the drag either increases or decreases inter-particle contact forces.

A downward flow increases effective stress.

In contrast, an upward flow opposes the force of gravity and can even cause to counteract completely the contact forces. In such a situation, effective stress is reduced to zero and the soil behaves like a very

viscous liquid. Such a state is known as **quick sand condition**. In nature, this condition is usually observed in coarse silt or fine sand subject to artesian conditions.



At the bottom of the soil column,

$$\sigma = \gamma \cdot L$$

$$u = \gamma_w (L + \Delta H)$$

During quick sand condition, the effective stress is reduced to zero.

$$\gamma \cdot L = \gamma_w (L + \Delta H)$$

$$L(\gamma - \gamma_w) = \gamma_w \cdot \Delta H$$

$$L \cdot \gamma_s = \gamma_w \cdot \Delta H$$

$$\frac{\Delta H}{L} = \frac{\gamma_s}{\gamma_w} = i_{cr} \approx 1$$

where i_{cr} = **critical hydraulic gradient**

This shows that when water flows upward under a hydraulic gradient of about 1, it completely neutralizes the force on account of the weight of particles, and thus leaves the particles suspended in water.

The Importance of Effective Stress

At any point within the soil mass, the magnitudes of both total stress and pore water pressure are dependent on the ground water position. With a shift in the water table due to seasonal fluctuations, there is a resulting change in the distribution in pore water pressure with depth.

Changes in water level **below ground** result in changes in effective stresses below the water table. A rise increases the pore water pressure at all elevations thus causing a decrease in effective stress. In contrast, a fall in the water table produces an increase in the effective stress.

Changes in water level **above ground** do not cause changes in effective stresses in the ground below. A rise above ground surface increases both the total stress and the pore water pressure by the same amount, and consequently effective stress is not altered.

In some analyses it is better to work with the *changes* of quantity, rather than in absolute quantities. The effective stress expression then becomes:

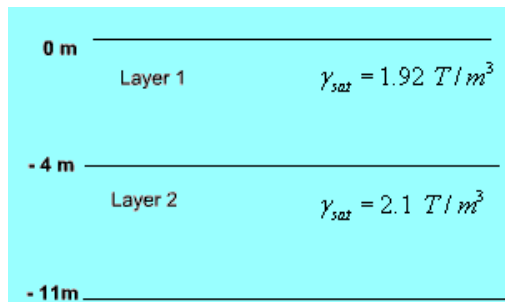
$$\Delta \sigma' = \Delta \sigma - \Delta u$$

If both total stress and pore water pressure change by the same amount, the effective stress remains constant.

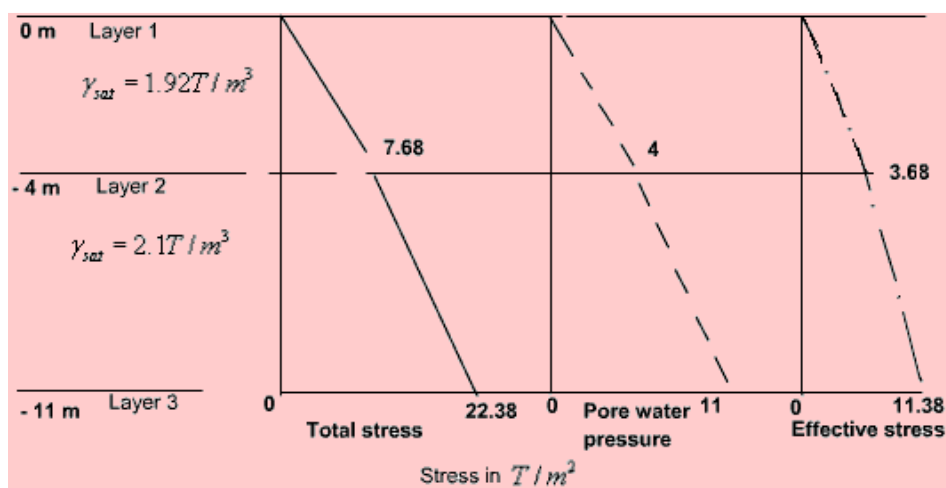
Total and effective stresses must be distinguishable in all calculations. Ground movements and instabilities can be caused by changes in total stress, such as caused by loading by foundations and unloading due to excavations. They can also be caused by changes in pore water pressures, such as failure of slopes after rainfall.

Worked Examples

Example 1: For the soil deposit shown below, draw the total stress, pore water pressure and effective stress diagrams. The water table is at ground level.



Solution:



Total stress

At - 4 m, $\sigma = 1.92 \times 4 = 7.68 \text{ T/m}^2$

At -11 m, $\sigma = 7.68 + 2.1 \times 7 = 22.38 \text{ T/m}^2$

Pore water pressure

At - 4 m, $u = 1 \times 4 = 4 \text{ T/m}^2$

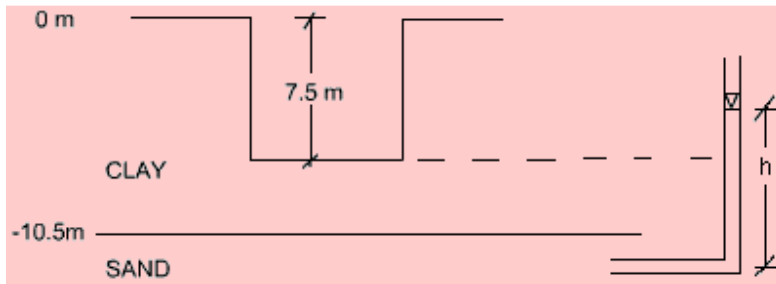
At -11 m, $u = 1 \times 11 = 11 \text{ T/m}^2$

Effective stress

At - 4 m, $\bar{\sigma} = 7.68 - 4 = 3.68 \text{ T/m}^2$

At -11 m, $\bar{\sigma} = 22.38 - 11 = 11.38 \text{ T/m}^2$

Example 2: An excavation was made in a clay stratum having $\gamma_t = 2 \text{ T/m}^3$. When the depth was 7.5 m, the bottom of the excavation cracked and the pit was filled by a mixture of sand and water. The thickness of the clay layer was 10.5 m, and below it was a layer of pervious water-bearing sand. How much was the artesian pressure in the sand layer?



Solution:

When the depth of excavation was 7.5 m, at the interface of the CLAY and SAND layers, the effective stress was equal to zero.

Downward pressure due to weight of clay = Upward pressure due to artesian pressure

$$(10.5 - 7.5) \gamma_t = \gamma_w h, \text{ where } h = \text{artesian pressure head}$$

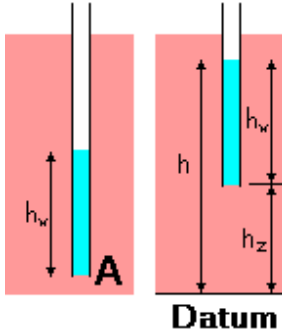
$$3 \times 2 = 1 \times h$$

$$\therefore h = 6 \text{ m} = 0.6 \text{ kg/cm}^2 \text{ or } 6 \text{ T/m}^2 \text{ artesian pressure}$$

Permeability of Soils

Pressure, Elevation and Total Heads

In soils, the interconnected pores provide passage for water. A large number of such flow paths act together, and the average rate of flow is termed the coefficient of permeability, or just permeability. It is a measure of the ease that the soil provides to the flow of water through its pores.



At point **A**, the pore water pressure (**u**) can be measured from the height of water in a standpipe located at that point.

The height of the water column is the **pressure head** (h_w).

$$h_w = u/\gamma_w$$

To identify any difference in pore water pressure at different points, it is necessary to eliminate the effect of the points of measurement. With this in view, a datum is required from which locations are measured.

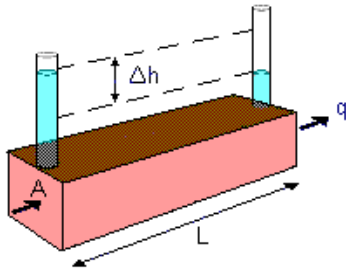
The **elevation head** (h_z) of any point is its height above the datum line. The height of water level in the standpipe above the datum is the **piezometric head** (**h**).

$$h = h_z + h_w$$

Total head consists of **three components**: elevation head, pressure head, and velocity head. As seepage velocity in soils is normally low, velocity head is ignored, and total head becomes equal to the piezometric head. Due to the low seepage velocity and small size of pores, the flow of water in the pores is steady and laminar in most cases. Water flow takes place between two points in soil due to the difference in total heads.

Darcy's Law

Darcy's law states that there is a linear relationship between flow velocity (**v**) and hydraulic gradient (**i**) for any given saturated soil under steady laminar flow conditions.



If the rate of flow is **q** (volume/time) through cross-sectional area (**A**) of the soil mass, Darcy's Law can be expressed as

$$v = q/A = k \cdot i$$

where **k** = permeability of the soil

$$i = \Delta h/L$$

Δh = difference in total heads

L = length of the soil mass

The flow velocity (**v**) is also called the Darcian velocity or the **superficial velocity**. It is different from the actual velocity inside the soil pores, which is known as the **seepage velocity, v_s** . At the particulate level, the water follows a tortuous path through the pores. Seepage velocity is always greater than the superficial velocity, and it is expressed as:

$$v_s = \frac{q}{A_v} = \frac{q}{A_v} \cdot \frac{A}{A} \approx \frac{v}{n}$$

where **A_v** = Area of voids on a cross section normal to the direction of flow

n = porosity of the soil

Permeability of Different Soils

Permeability (**k**) is an engineering property of soils and is a function of the soil type. Its value depends on the average size of the pores and is related to the distribution of particle sizes, particle shape and soil structure. The ratio of permeabilities of typical sands/gravels to those of typical clays is of the order of **10⁶**. A small proportion of fine material in a coarse-grained soil can lead to a significant reduction in permeability.

For different soil types as per grain size, the orders of magnitude for permeability are as follows:

Soil	k (cm/sec)
Gravel	10 ⁰
Coarse sand	10 ⁰ to 10 ⁻¹
Medium sand	10 ⁻¹ to 10 ⁻²
Fine sand	10 ⁻² to 10 ⁻³
Silty sand	10 ⁻³ to 10 ⁻⁴
Silt	1 x 10 ⁻⁵
Clay	10 ⁻⁷ to 10 ⁻⁹

Factors affecting Permeability

The coefficient of permeability depends on several factors, most of which are listed below: Shape and size of the soil particles.

1. **Void ratio** : Permeability increases with increase of void ratio.

2. **Degree of saturation** : Permeability increases with increase of degree of saturation. The variation of the value of k with degree of saturation for Madison sand is shown in figure 1.2. **Figure 1.3** show the effect of the degree of saturation on the value of k for a silty clay specimens were prepared by kneading compaction to a dry unit weight of 16.98 kN/m^3 . The molding moisture contents w were varied.

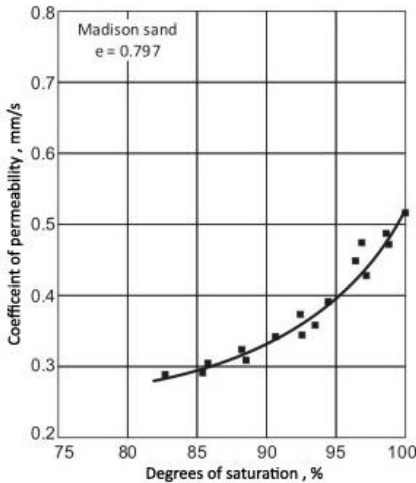


Figure 1.2 Influence of degree of saturation on permeability of Madison sand

Composition of soil particles: For sands and silts this is not important; however, for soils with clay minerals this is one of the most important factors. Permeability in this case depends on the thickness of water held to the soil particles, which is a function of the cation exchange capacity, valence of the cations, etc. other factors remaining the same, the coefficient of permeability decreases with increasing thickness of the diffuse double layer

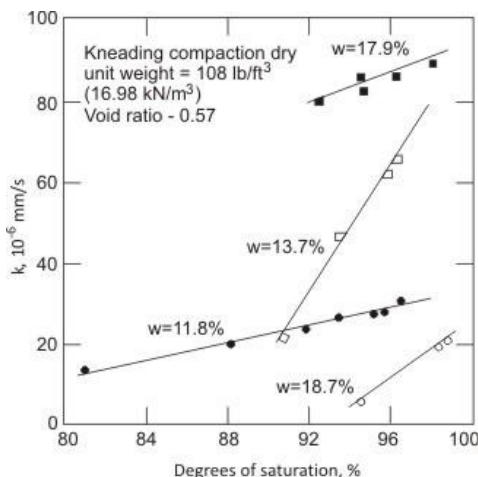


Figure 1.3 Influence of degree of saturation on permeability of compacted silty clay.

3. **Soil structure**: Fine-grained soils with a flocculated structure have a higher coefficient of permeability than those with a dispersed structure. This fact is demonstrated in **figure 1.4** for the case of a silty clay. The test specimens were prepared to a constant dry unit weight by kneading compaction. The

molding moisture content was varied. Note that with the increase of moisture content the soil becomes more and more dispersed. With increasing degree of dispersion, the permeability decreases.

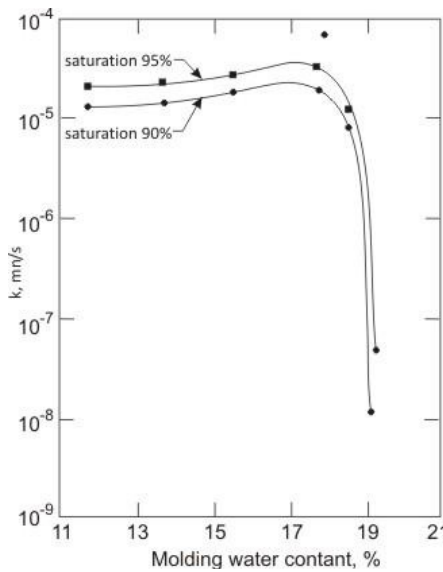


Figure 1.4 Dependence of permeability on the structure of a silty clay.

4. Viscosity of permeant
5. Density and concentration of permeant

In soils, the permeant or pore fluid is mostly water whose variation in property is generally very less. Permeability of all soils is strongly influenced by the density of packing of the soil particles, which can be represented by void ratio (**e**) or porosity (**n**).

For Sands

In sands, permeability can be empirically related to the square of some representative grain size from its grain-size distribution. For filter sands, Allen Hazen in 1911 found that $k \approx 100 (D_{10})^2$ cm/s where D_{10} = effective grain size in cm.

Different relationships have been attempted relating void ratio and permeability, such as $k \propto e^3/(1+e)$, and $k \propto e^2$. They have been obtained from the Kozeny-Carman equation for laminar flow in saturated soils.

$$k = \frac{1}{k_0 k_T S_s^2} \cdot \frac{e^3}{1+e} \cdot \frac{\gamma_w}{\eta}$$

where k_0 and k_T are factors depending on the shape and tortuosity of the pores respectively, S_s is the surface area of the solid particles per unit volume of solid material, and γ_w and η are unit weight and viscosity of the pore water. The equation can be reduced to a simpler form as

$$k = C \cdot \frac{e^3}{1+e} \approx C \cdot e^2$$

For Silts and Clays

For silts and clays, the Kozeny-Carman equation does not work well, and **log k** versus **e** plot has been found to indicate a linear relationship.

For clays, it is typically found that

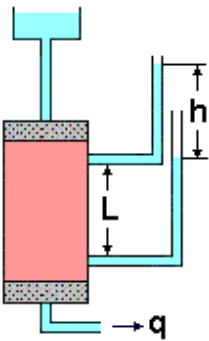
$$\log_{10} k = \frac{e - e_k}{C_k}$$

where C_k is the permeability change index and e_k is a reference void ratio.

Laboratory Measurement of Permeability

Constant Head Flow

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**.

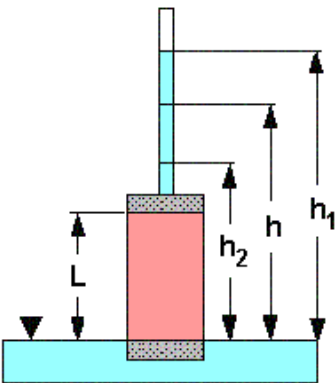


Permeability **k** is obtained from:

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

Falling Head Flow

Falling head permeameter is recommended for fine-grained soils.



Total head **h** in standpipe of area **a** is allowed to fall. Hydraulic gradient varies with time. Heads **h₁** and **h₂** are measured at times **t₁** and **t₂**. At any time **t**, flow through the soil sample of cross-sectional area **A** is

$$q = k \cdot h \cdot \frac{A}{L} \text{----- (1)}$$

Flow in unit time through the standpipe of cross-sectional area **a** is

$$= a \times \left(-\frac{dh}{dt} \right) \text{-----} (2)$$

Equating (1) and (2) ,

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

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

Integrating between the limits,

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

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

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

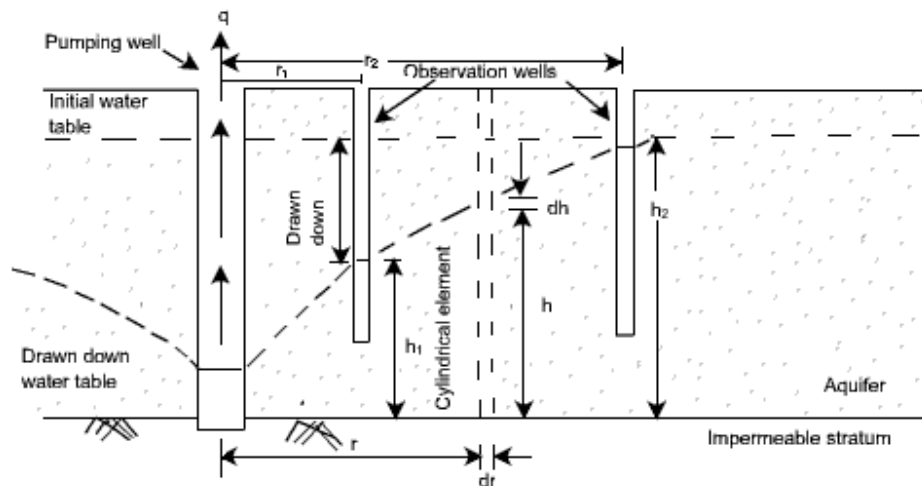
Field Tests for Permeability

Field or *in-situ* measurement of permeability avoids the difficulties involved in obtaining and setting up undisturbed samples in a permeameter. It also provides information about bulk permeability, rather than merely the permeability of a small sample.

A field permeability test consists of pumping out water from a main well and observing the resulting drawdown surface of the original horizontal water table from at least two observation wells. When a steady state of flow is reached, the flow quantity and the levels in the observation wells are noted.

Two important field tests for determining permeability are: Unconfined flow pumping test, and confined flow pumping test.

Unconfined Flow Pumping Test



In this test, the pumping causes a drawdown in an unconfined (i.e. open surface) soil stratum, and generates a radial flow of water towards the pumping well. The steady-state heads h_1 and h_2 in observation wells at radii r_1 and r_2 are monitored till the flow rate q becomes steady.

The rate of radial flow through any **cylindrical surface** around the pumping well is equal to the amount of water pumped out. Consider such a surface having radius r , thickness dr and height h . The hydraulic gradient is

$$i = \frac{dh}{dr}$$

$$\text{Area of flow, } A = 2\pi rh$$

From Darcy's Law,

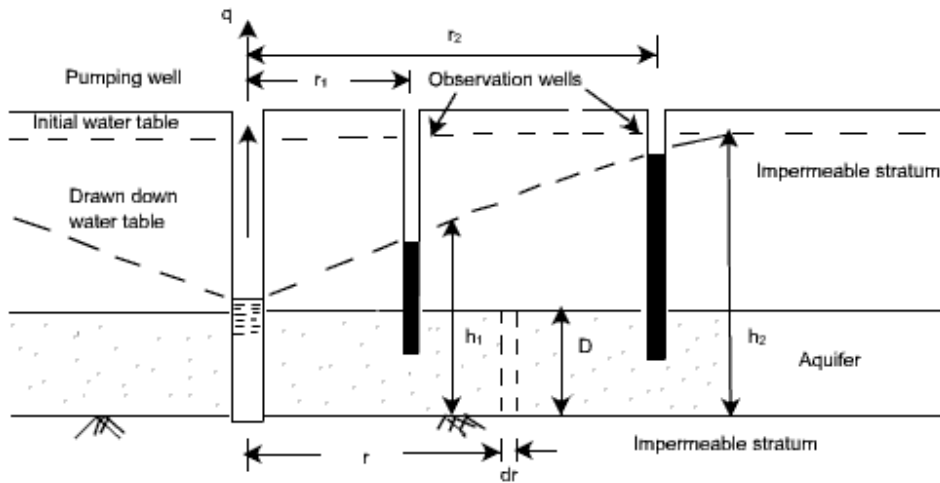
$$\begin{aligned} q &= k \cdot i \cdot A \\ &= k \cdot \frac{dh}{dr} \cdot 2\pi rh \end{aligned}$$

Arranging and integrating,

$$\begin{aligned} \int_{r_1}^{r_2} \frac{dr}{r} &= \int_{h_1}^{h_2} \frac{2r}{q} \cdot K \cdot h \cdot dh \\ k &= \frac{q \cdot \log_e \left(\frac{r_2}{r_1} \right)}{r(h_2^2 - h_1^2)} \end{aligned}$$

Confined Flow Pumping Test

Artesian conditions can exist in a aquifer of thickness D confined both above and below by impermeable strata. In this, the drawdown water table is above the upper surface of the aquifer.



For a **cylindrical surface** at radius r ,

$$q = k \cdot \frac{dh}{dr} \cdot 2\pi r D$$

$$\int_{r_1}^{r_2} \frac{dr}{r} = \int_{h_1}^{h_2} \frac{2\pi D k}{q} dh$$

Integrating,

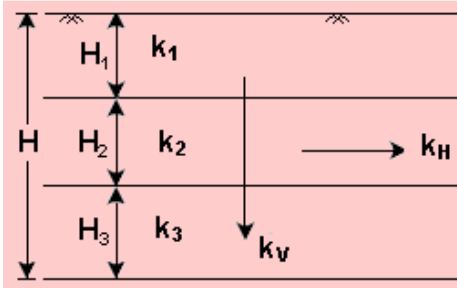
$$\log_e \left(\frac{r_2}{r_1} \right) = \frac{2\pi D k}{q} (h_2 - h_1)$$

$$k = \frac{q \cdot \log_e \left(\frac{r_2}{r_1} \right)}{2\pi r D (h_2 - h_1)}$$

Permeability of Stratified Deposits

When a soil deposit consists of a number of horizontal layers having different permeabilities, the average value of permeability can be obtained separately for both vertical flow and horizontal flow, as k_v and k_H respectively.

Consider a stratified soil having horizontal layers of thickness H_1, H_2, H_3 , etc. with coefficients of permeability k_1, k_2, k_3 , etc.



For vertical flow

The flow rate q through each layer per unit area is the same.

$$q = q_1 = q_2 = \dots$$

Let i be the equivalent hydraulic gradient over the total thickness H and let the hydraulic gradients in the layers be i_1, i_2, i_3 , etc. respectively.

$$k_v \cdot i = k_1 i_1 = k_2 i_2 = \dots \text{ where } k_v = \text{Average vertical permeability}$$

$$k_v \cdot \frac{h}{H} = k_1 \cdot \frac{h_1}{H_1} = k_2 \cdot \frac{h_2}{H_2} = \dots$$

The total head drop h across the layers is

$$h = h_1 + h_2 + \dots$$

$$h = \frac{k_v h}{H} \cdot \frac{H_1}{k_1} + \frac{k_v h}{H} \cdot \frac{H_2}{k_2} + \dots$$

$$k_v = \frac{H}{\frac{H_1}{k_1} + \frac{H_2}{k_2} + \dots}$$

Horizontal flow

When the flow is horizontal, the hydraulic gradient is the same in each layer, but the quantity of flow is different in each layer.

$$i = i_1 = i_2 = i_3 = \dots$$

The total flow is

$$q = q_1 + q_2 + q_3 \dots\dots$$

Considering unit width normal to the cross-section plane,

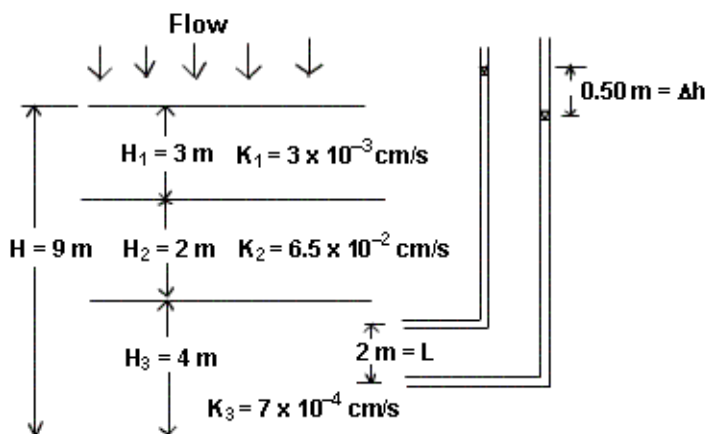
$$k_H i . H = k_1 i_1 . H_1 + k_2 i_2 . H_2 + \dots\dots\dots$$

$$k_H = \frac{1}{H} (k_1 H_1 + k_2 H_2 + \dots\dots\dots)$$

Worked Examples

Example 1: Determine the following:

- (a) Equivalent coefficient of vertical permeability of the three layers
- (b) The rate of flow per m² of plan area
- (c) The total head loss in the three layers



Solution:

$$(a) \quad K_v = \frac{H}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3}} = \frac{9}{\frac{3}{3 \times 10^{-3}} + \frac{2}{6.5 \times 10^{-2}} + \frac{4}{7 \times 10^{-4}}} = 1.33 \times 10^{-3} \text{ cm/s}$$

(b) Considering an area $A = 1 \text{ m}^2 = 1 \times 10^4 \text{ cm}^2$

$$q = k_v i A = k_v \frac{\Delta h}{L} A = 7 \times 10^{-4} \times 0.25/2 \times (1 \times 10^4) = 0.875 \text{ cm}^3/\text{s per m}^2 \text{ of plan area}$$

(c) For continuity of flow, velocity is the same.

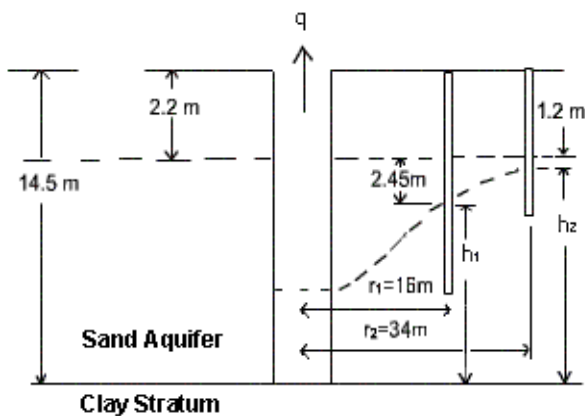
$$k_3 \frac{\Delta h}{L} = k_v \frac{\Delta h_{total}}{H}, \text{ where } \Delta h_{total} = \text{total head loss in three layers}$$

$$\therefore \Delta h_{\text{total}} = k_3 \cdot \frac{\Delta h}{L} \cdot \frac{H}{k_V} = 7 \times 10^{-4} \times \frac{0.50}{2} \times \frac{9}{1.33 \times 10^{-3}} = 1.184 \text{ m}$$

Example 2: For a field pumping test, a well was sunk through a horizontal stratum of sand 14.5 thick and underlain by a clay stratum. Two observation wells were sunk at horizontal distances of 16 m and 34 m respectively from the pumping well. The initial position of the water table was 2.2 m below ground level.

At a steady-state pumping rate of 1850 litres/min, the drawdowns in the observation wells were found to be 2.45 m and 1.20 m respectively. Calculate the coefficient of permeability of the sand.

Solution:



$$k = \frac{q \cdot \log_e \left(\frac{r_2}{r_1} \right)}{r(h_2^2 - h_1^2)}$$

$$q = 1850 \text{ litres/min} = \frac{1850 \times 10^{-3}}{60} \text{ m}^3/\text{s}$$

$$r_1 = 16 \text{ m}$$

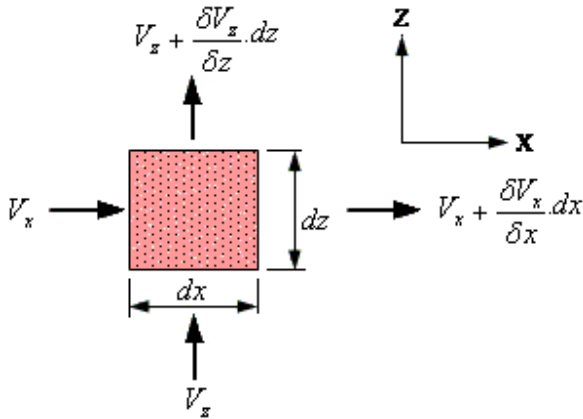
$$r_2 = 34 \text{ m}$$

$$h_1 = 14.5 - 2.2 - 2.45 = 9.85 \text{ m}$$

$$h_2 = 14.5 - 2.2 - 1.2 = 11.1 \text{ m}$$

$$k = \frac{\frac{1850 \times 10^{-3}}{60} \times \log_e \left(\frac{34}{16} \right)}{r[(11.1)^2 - (9.85)^2]} = 2.82 \times 10^{-4} \text{ m/s} = 1.41 \times 10^{-2} \text{ cm/s}$$

Seepage in Soils



A rectangular soil element is shown with dimensions **dx** and **dz** in the plane, and thickness **dy** perpendicular to this plane. Consider planar flow into the rectangular soil element.

In the **x-direction**, the net amount of the water entering and leaving the element is

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

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

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

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, $V_x = k_x \cdot \frac{\delta h}{\delta x}$, $V_z = k_z \cdot \frac{\delta h}{\delta z}$, where **h** is the head causing flow.

When the continuity equation is combined with Darcy's law, the equation for flow is expressed as:

$$k_x \cdot \frac{\delta^2 h}{\delta x^2} + k_z \cdot \frac{\delta^2 h}{\delta z^2} = 0$$

For an isotropic material in which the permeability is the same in all directions (i.e. $k_x = k_z$), the **flow equation** is

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta z^2} = 0$$

This is the **Laplace equation** governing two-dimensional steady state flow. It can be solved **graphically, analytically, numerically, or analogically**.

For the more general situation involving **three-dimensional** steady flow, Laplace equation becomes:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

One-dimensional Flow

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

Integrating twice, a general solution is obtained.

$$\frac{\partial h}{\partial x} = c_1$$

$$h = c_2 + c_1 x$$

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

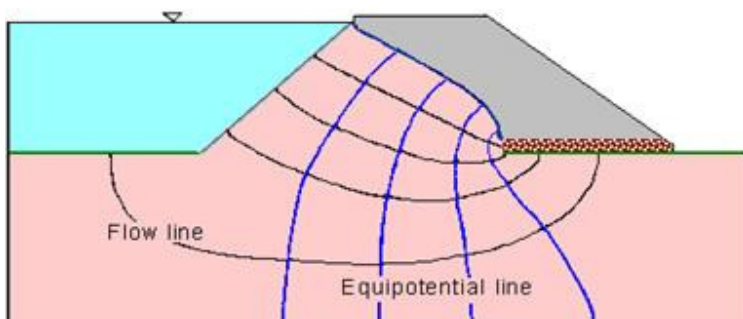
Two-dimensional Flow

Flow Nets

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

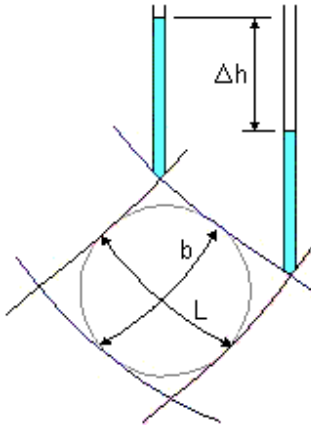
- **Equipotential lines** connecting points of equal total head **h**
- **Flow lines** indicating the direction of seepage down a hydraulic gradient

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



Calculation of flow in a channel

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



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

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

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

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

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

In such a square, $b = L$, and the flow rate is obtained as $\Delta q = k \cdot \Delta h$

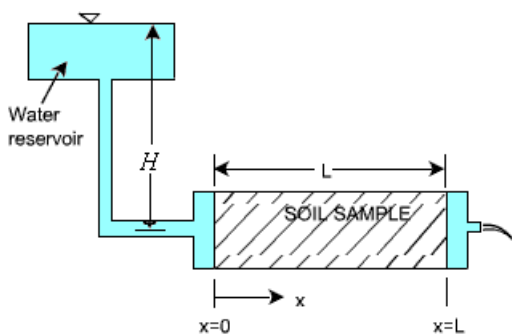
Thus the flow rate through such a flow channel is the permeability k multiplied by the uniform interval Δh between adjacent equipotential lines.

Calculation of total flow

For a complete problem, the flow net can be drawn with the overall head drop h divided into N_d so that $\Delta h = h / N_d$.

If N_f is the no. of flow channels, then the total flow rate is

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



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

Substituting and solving,

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

The specific solution for flow in the above permeameter is

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

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

Procedure for Drawing Flow Nets

At every **point (x,z)** where there is flow, there will be a value of head **h(x,z)**. In order to represent these values, contours of equal head are drawn.

A flow net is to be drawn by trial and error. For a given set of boundary conditions, the flow net will remain the same even if the direction of flow is reversed. Flow nets are constructed such that the head lost between successive **equipotential lines** is the same, say Δh . It is useful in visualising the flow in a soil to plot the flow lines, as these are lines that are tangential to the flow at any given point. The steps of construction are:

1. Mark all boundary conditions, and draw the flow cross section to some convenient scale.
2. Draw a coarse net which is consistent with the boundary conditions and which has orthogonal equipotential and flow lines. As it is usually easier to visualise the pattern of flow, start by drawing the flow lines first.
3. Modify the mesh such that it meets the conditions outlined above and the fields between adjacent flow lines and equipotential lines are 'square'.
4. Refine the flow net by repeating **step 3**.

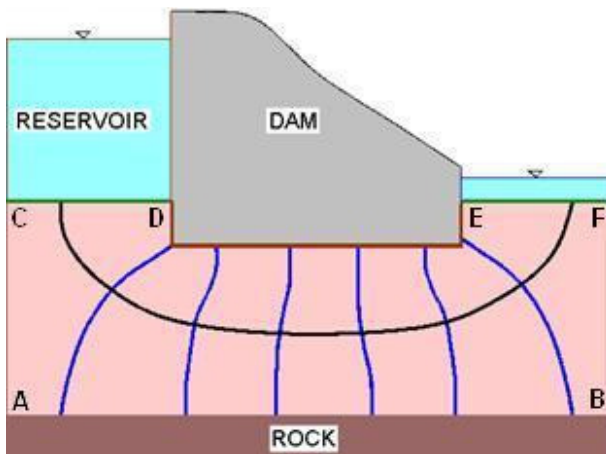
The most common **boundary conditions** are:

(a) A submerged permeable soil boundary is an equipotential line. This could have been determined by considering imaginary standpipes placed at the soil boundary, as for every point the water level in the standpipe would be the same as the water level. (Such a boundary is marked as CD and EF in the following figure.)

(b) The boundary between permeable and impermeable soil materials is a flow line (This is marked as AB in the same figure).

(c) Equipotential lines intersecting a phreatic surface do so at equal vertical intervals.

Uses of Flow Nets



The graphical properties of a flow net can be used in obtaining solutions for many seepage problems such as:

- 1. Estimation of seepage losses from reservoirs:** It is possible to use the flow net in the transformed space to calculate the flow underneath the dam.
- 2. Determination of uplift pressures below dams:** From the flow net, the pressure head at any point at the base of the dam can be determined. The uplift pressure distribution along the base can be drawn and then summed up.
- 3. Checking the possibility of piping beneath dams:** At the toe of a dam when the upward exit hydraulic gradient approaches unity, boiling condition can occur leading to erosion in soil and consequent piping. Many dams on soil foundations have failed because of a sudden formation of a piped shaped discharge channel. As the stored water rushes out, the channel widens and catastrophic failure results. This is also often referred to as piping failure.

Compression and Consolidation of Soils

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 undrained, 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.

Components of Total Settlement

The total settlement of a loaded soil has three components: Elastic settlement, primary consolidation, and secondary compression.

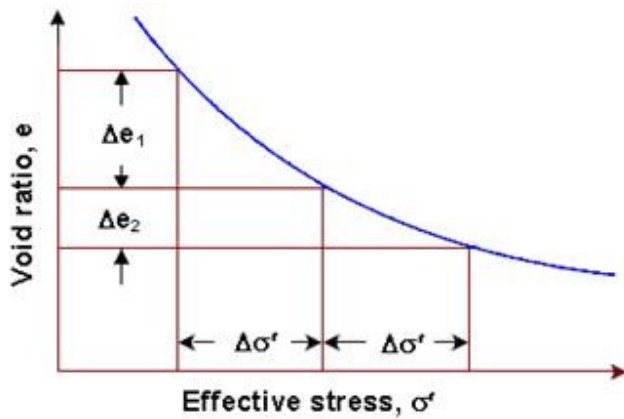
Elastic settlement is on account of change in shape at constant volume, i.e. due to vertical compression and lateral expansion. **Primary consolidation** (or simply **consolidation**) is on account of flow of water from the voids, and is a function of the permeability and compressibility of soil. **Secondary compression** is on account of creep-like behaviour.

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

Compressibility Characteristics

Soils are often subjected to uniform loading over large areas, such as from wide foundations, fills or embankments. Under such conditions, the soil which is remote from the edges of the loaded area undergoes vertical strain, but no horizontal strain. Thus, the settlement occurs only in one-dimension.

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



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

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

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

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

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

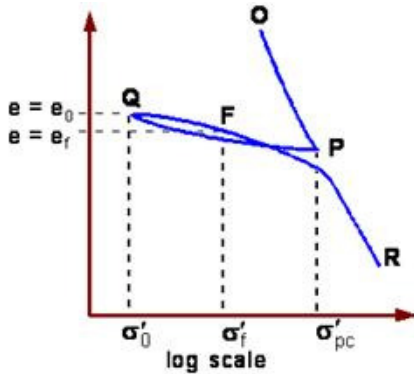
If e_0 is the initial void ratio of the consolidating layer, another useful parameter is the **coefficient of volume compressibility, m_v** , which is expressed as

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

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

Normally Consolidated and Over-Consolidated Clays

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



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

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

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

It may be seen that for the same increase in effective stress, the change in void ratio is much less for an overconsolidated soil (from e_0 to e_1), than it would have been for a normally consolidated soil as in path **OP**. In unloading, the soil swells but the increase in volume is much less than the initial decrease in volume for the same stress difference.

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

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

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

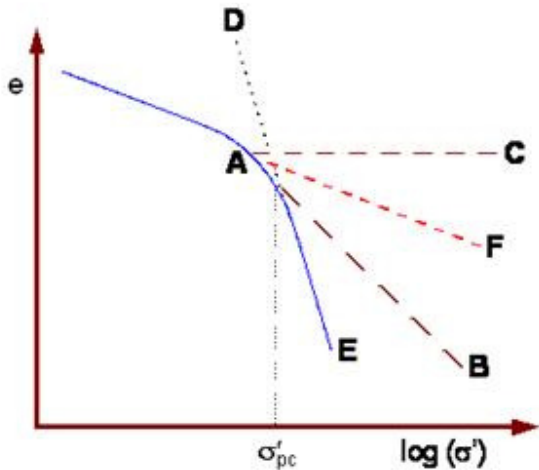
Settlements will generally be much smaller for structures built on overconsolidated soils. Most soils are overconsolidated to some degree. This can be due to shrinking and swelling of the soil on drying and rewetting, changes in ground water levels, and unloading due to erosion of overlying strata.

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

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

Estimation of Preconsolidation Stress

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

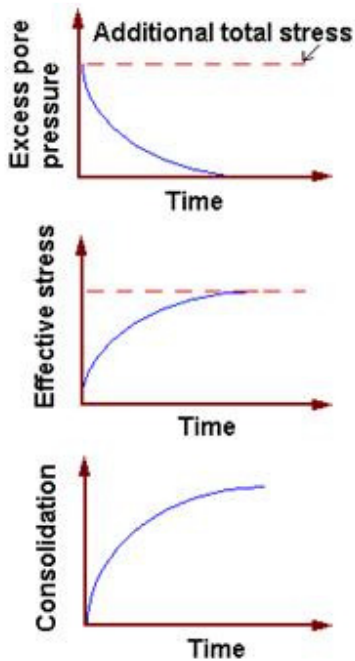


The steps in the construction are:

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

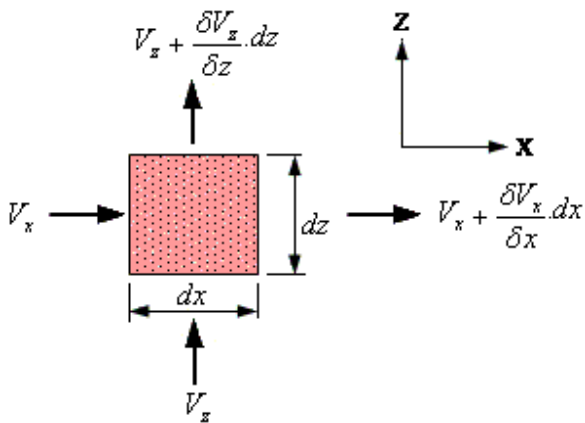
Analysis of Consolidation - Terzaghi's Theory

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



On the assumption that the excess pore water drains only along vertical lines, an analytical procedure can be developed for computing the rate of consolidation.

Consider a saturated soil element of sides dx , dy and dz .



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

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

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

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

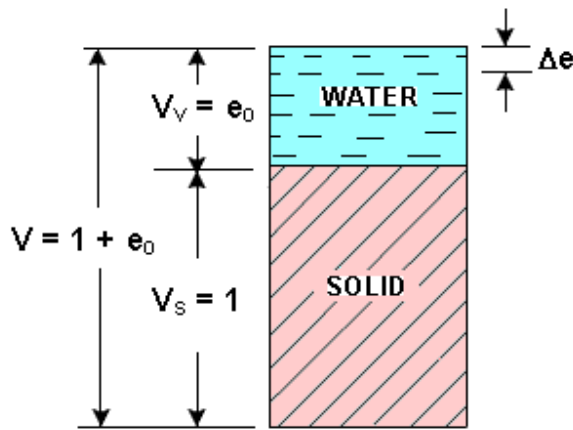
Only the **excess head(h)** causes consolidation, and it is related to the **excess pore water pressure (u)** by $h = u/\gamma_w$. The Darcy equation can be written as

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

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

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

The soil element can be represented schematically as



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

The flow eqn. can then be written as

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

or

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

This is the hydrodynamic equation of one-dimensional consolidation.

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

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

The flow eqn. can then be expressed as

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

or

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

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

By introducing a parameter called the coefficient of consolidation, then becomes

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

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

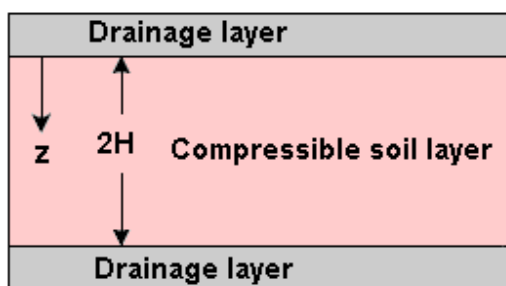
Solution of Terzaghi's Theory

During the consolidation process, the following are assumed to be constant:

1. The total additional stress on the compressible soil layer is assumed to remain constant.
2. The coefficient of volume compressibility (**m_v**) of the soil is assumed to be constant.
3. The coefficient of permeability (**k**) for vertical flow is assumed to be constant.

There are three variables in the consolidation equation:

1. the depth of the soil element in the layer (**z**)
2. the excess pore water pressure (**u**)
3. the time elapsed since application of the loading (**t**)



To take care of these three variables, three non-dimensional parameters are provided:

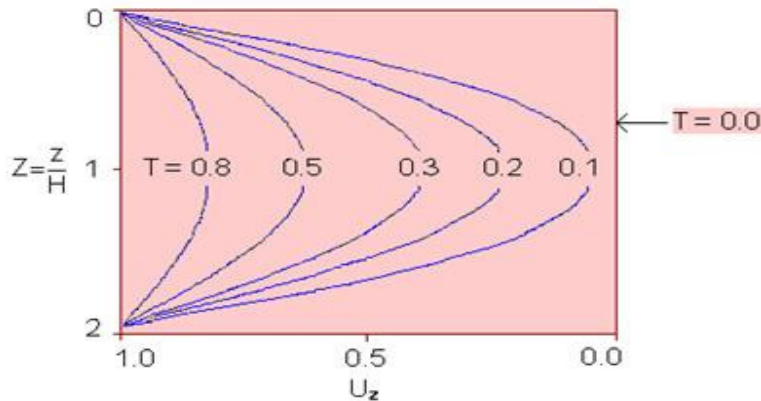
$$Z = \frac{z}{H}$$

1. Drainage path ratio, $\frac{z}{H}$, where **H = drainage path** which is the longest path taken by the pore water to reach a permeable sub-surface layer above or below.

2. Consolidation ratio at depth z , U_z , which is the ratio of dissipated pore pressure to the initial excess pore pressure. This represents the stage of consolidation at a certain location in the compressible layer.

3. Time factor, $T = \frac{c_v \cdot t}{H^2}$

The graphical solution of **Terzaghi's one-dimensional consolidation equation** using the non-dimensional parameters is shown.

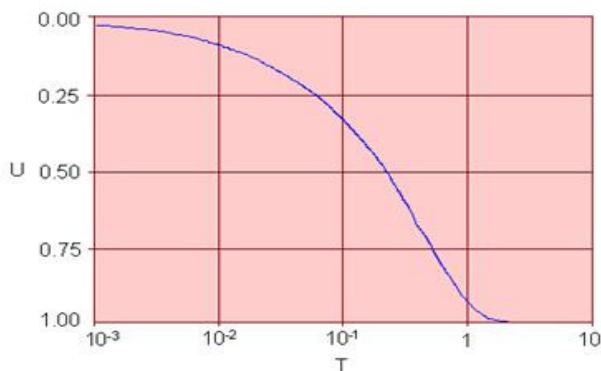


The figure is symmetrical about the horizontal line at $Z = \frac{z}{H} = 1$.

For **double drainage** conditions, pore water above this location flows upwards whereas water below this location flows downwards. Thus, the horizontal line at $Z = 1$ is equivalent to an impervious boundary. For **single drainage** conditions, only either the top half or bottom half of the figure is to be used, and the drainage path is equal to the thickness of the compressible layer.

The above graphical solution shows how consolidation proceeds with time at different locations for a particular set of boundary conditions, but it does not describe how much consolidation occurs as a whole in the entire compressible layer.

The variation of total consolidation with time is most conveniently plotted in the form of the average degree of consolidation (U) for the entire stratum versus dimensionless time T , and this is illustrated below.



There are useful approximations relating the degree of consolidation and the time factor, viz:

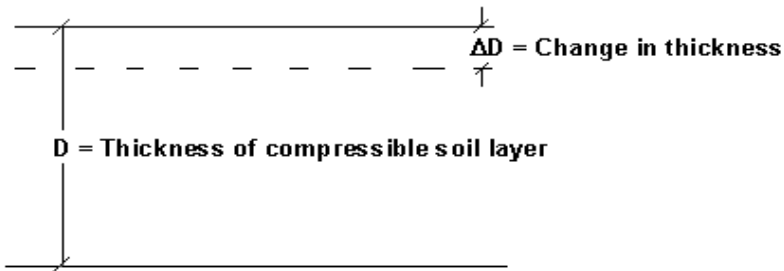
For $U \leq 0.60$, $T = (\pi/4) \cdot U^2$

For $U > 0.60$, $T = 1.781 - 0.933 \log_{10}(100 - U\%)$

Consolidation Settlement and Time

To estimate the amount of consolidation which would occur and the time it would take to occur, it is necessary to know:

1. The boundary and drainage conditions
2. The loading conditions
3. The relevant parameters of the soil, including initial void ratio, coefficient of compressibility, coefficient of volume compressibility, compression index, and coefficient of consolidation. They are obtained from consolidation tests on representative undisturbed samples of the compressible soil stratum.



Comparing the compressible soil layer with a soil element of this layer,

$$\frac{\text{Change in thickness}}{\text{Total thickness}} = \frac{\text{Change in volume}}{\text{Total volume}}$$

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

$$\therefore \Delta D = \frac{\Delta e}{1 + e_0} D$$

Δe can be expressed in terms of a_v or C_c .

$$\Delta e = a_v \cdot \Delta \sigma'$$

or

$$\Delta e = C_c \log(\sigma'_0 + \Delta \sigma' / \sigma'_0)$$

The magnitude of consolidation settlement is

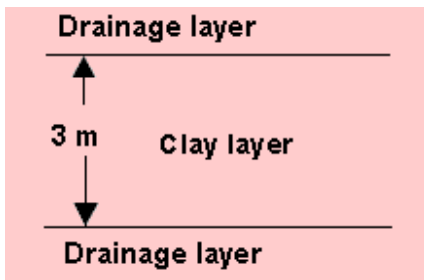
$$\Delta D = \frac{a_v \cdot \Delta \sigma'}{1 + e_0} \cdot D = m_v \cdot \Delta \sigma' \cdot D$$

or

$$\Delta D = \frac{C_c \log \left(\frac{\sigma'_0 + \Delta \sigma'}{\sigma'_0} \right) \cdot D}{1 + e_0}$$

Worked Examples

Example 1: A 3 m thick layer of saturated clay in the field under a surcharge loading will achieve 90% consolidation in 75 days in double drainage conditions. Find the coefficient of consolidation of the clay.



Solution:

As the clay layer has two-way drainage, $H = 1.5 \text{ m} = 150 \text{ cm}$

$t_{90} = 75 \text{ days} = 75 \times 24 \times 60 \times 60 \text{ seconds}$

For 90% consolidation ($U = 90\%$)

$$\begin{aligned}
 T_{90} &= \frac{c_v \cdot t_{90}}{H^2} = 0.848 \\
 \therefore c_v &= \frac{T_{90} \cdot H^2}{t_{90}} \\
 &= \frac{0.848 \times (150)^2}{75 \times 24 \times 60 \times 60} \\
 &= 2.94 \times 10^{-3} \text{ cm}^2 / \text{s}
 \end{aligned}$$

Example 2: A 3 m thick clay layer in the field under a given surcharge will undergo 7 cm of total primary consolidation. If the first 4 cm of settlement takes 90 days, calculate the time required for the first 2 cm of settlement.

Solution:

Total consolidation = 7 cm

For 4 cm settlement, $U_1 = 4/7 \times 100 = 57.14\%$

For 2 cm settlement, $U_2 = 2/7 \times 100 = 28.57\%$

$t_1 = 90 \text{ days}$.

For,

$$U \leq 60\%,$$

$$\frac{c_v t}{H^2} = T \propto U^2$$

$$\therefore \frac{t_1}{t_2} = \frac{U_1^2}{U_2^2}$$

$$\begin{aligned}\therefore t_2 &= \frac{U_2^2}{U_1^2} \times t_1 \\ &= \frac{(28.57)^2}{(57.14)^2} \times 90 \\ &= 22.5 \text{ days}\end{aligned}$$

Example 3: For a laboratory consolidation test on a soil specimen that is drained on both sides, the following were obtained:

Thickness of the clay specimen = 25 mm

$P_1 = 50 \text{ kN/m}^2$; $e_1 = 0.92$

$P_2 = 120 \text{ kN/m}^2$; $e_2 = 0.78$

Time for 50% consolidation = 2.5 min

Determine the soil permeability for the loading range.

Solution:

$$a_v = \frac{\Delta e}{\Delta \sigma'} = \frac{0.92 - 0.78}{120 - 50} = 0.002 \text{ m}^2 / \text{kN}$$

$$H = \frac{25 \text{ mm}}{2} = \frac{0.025 \text{ m}}{2} = 0.0125 \text{ m}$$

$$t_{50} = 2.5 \text{ min}$$

$$\begin{aligned}c_v &= \frac{T_{50} \cdot H^2}{t_{50}} \\ &= \frac{0.197 \times (0.0125)^2}{2.5} \\ &= 1.23 \times 10^{-5} \text{ m}^2 / \text{min}\end{aligned}$$

$$\begin{aligned}k &= \frac{c_v \cdot a_v \cdot \gamma_w}{1 + e_0} \\ &= \frac{1.23 \times 10^{-5} \times 0.002 \times 9.81}{1 + 0.92} \\ &= 1.26 \times 10^{-7} \text{ m} / \text{min}\end{aligned}$$

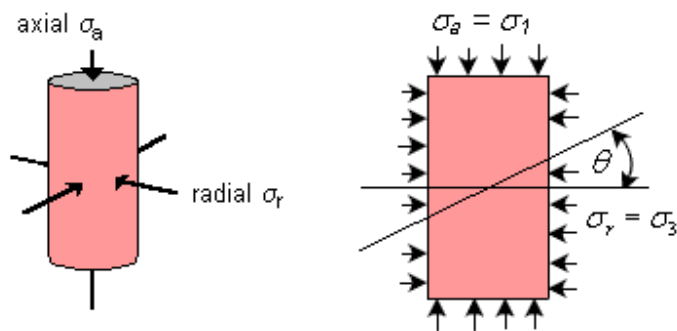
Shear Strength of Soils

Soils consist of individual particles that can slide and roll relative to one another. 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.

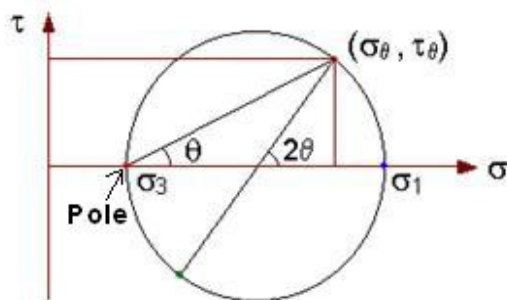
The shear strength of a soil is a function of the stresses applied to it as well as the manner in which these stresses are applied. A knowledge of shear strength of soils is necessary to determine the bearing capacity of foundations, the lateral pressure exerted on retaining walls, and the stability of slopes.

Mohr Circle of Stresses

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).



To visualise 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.



Draw a line inclined at angle θ with the horizontal through the pole of the Mohr circle so as 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 angle θ within the soil sample.

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

$$\text{Shear stress } \tau_\theta = \frac{(\sigma_1 - \sigma_3)}{2} \sin 2\theta$$

The plane inclined at an angle of 45° to the horizontal has acting on it the maximum shear stress equal to $\frac{\sigma_1 - \sigma_3}{2}$, and the normal stress on this plane is equal to $\frac{\sigma_1 + \sigma_3}{2}$.

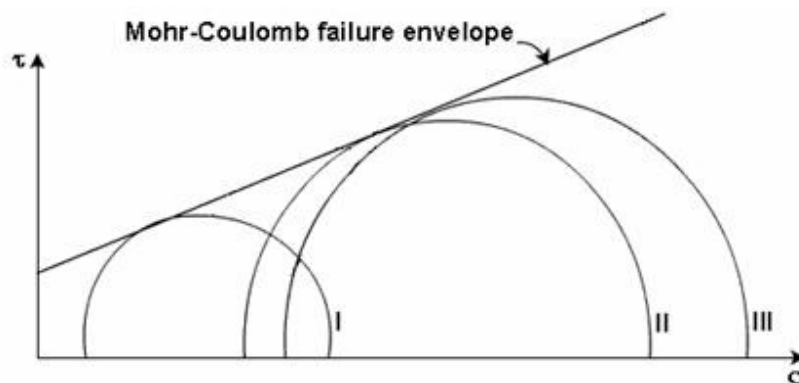
The plane with the maximum ratio of shear stress to normal stress is inclined at an angle of $45^\circ + \frac{\alpha}{2}$ to the horizontal, where α is the slope of the line tangent to the Mohr circle and passing through the origin.

Mohr-Coulomb Failure Criterion

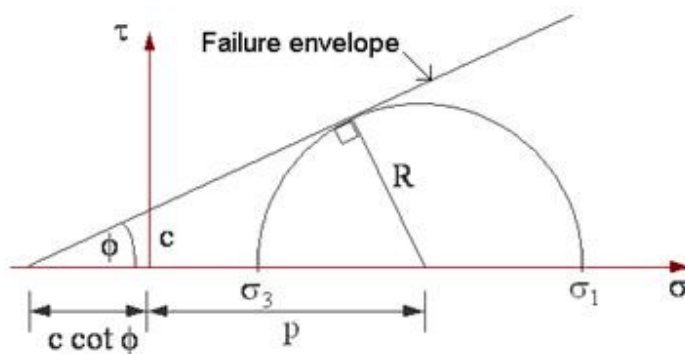
When the soil sample has failed, the shear stress on the failure plane defines the shear strength of the soil. Thus, it is necessary to identify the failure plane. Is it the plane on which the maximum shear stress acts, or is it the plane where the ratio of shear stress to normal stress is the maximum?

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 upto 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.



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 a 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.



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

$$\tau_f = c + \sigma_f \cdot \tan \phi$$

Where τ_f = shear stress on the failure plane
 c = apparent cohesion

σ_f = normal stress on the failure plane
 ϕ = 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,

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

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

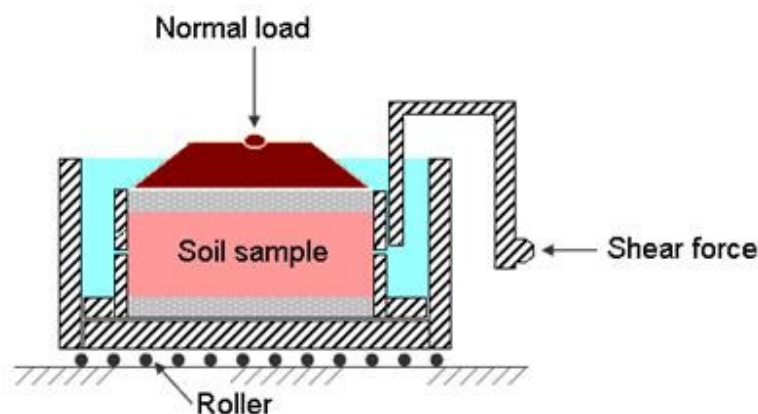
Rearranging,

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

Methods of Shear Strength Determination

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 plan of size 60 mm x 60 mm. A typical shear box is shown.



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 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**:

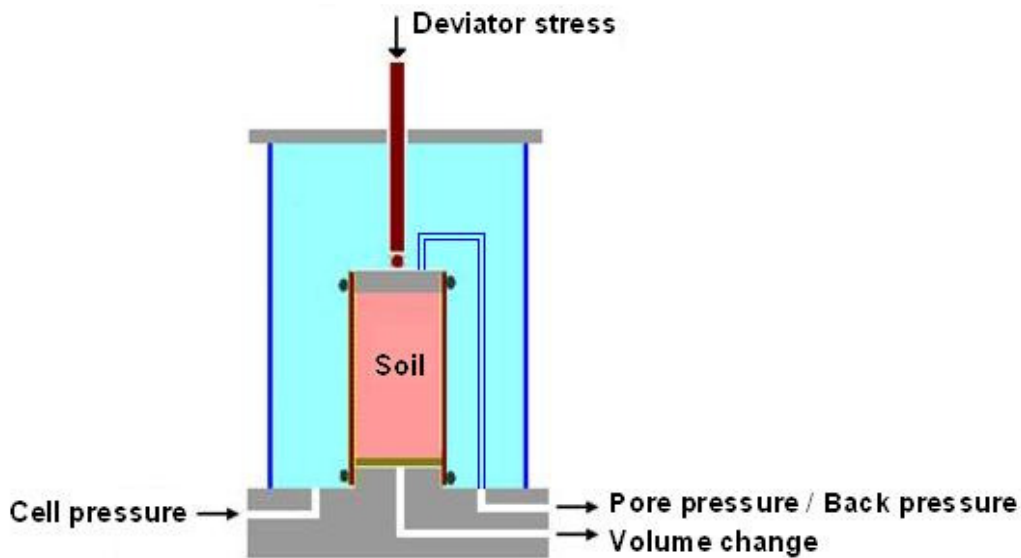
- It is easy to test sands and gravels.
- Large samples can be tested in large shear boxes, as small samples can give misleading results due to imperfections such as fractures and fissures, or may not be truly representative.
- Samples can be sheared along predetermined planes, when the shear strength along fissures or other selected planes are needed.

The **disadvantages** of the test include:

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

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.



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 in order to speed up the consolidation process. Pore pressure generated inside the specimen during testing can be measured by means of 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 the stages, the applied stresses, axial strain, and pore water pressure or change in sample volume can be measured.

Test Types

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 **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 \overline{UU} .

In the CU test, if pore water pressure is measured in the second stage, the test is symbolized as \overline{CU} .

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.

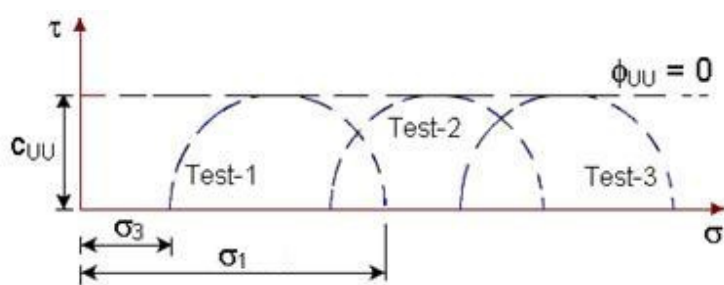
During the shearing process, the soil sample experiences axial strain, and either volume change or development of pore water pressure occurs. The magnitude of shear stress acting on different planes in the soil sample is different. When at some strain the sample fails, this limiting shear stress on the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

Total Stress Parameters

UU Tests:



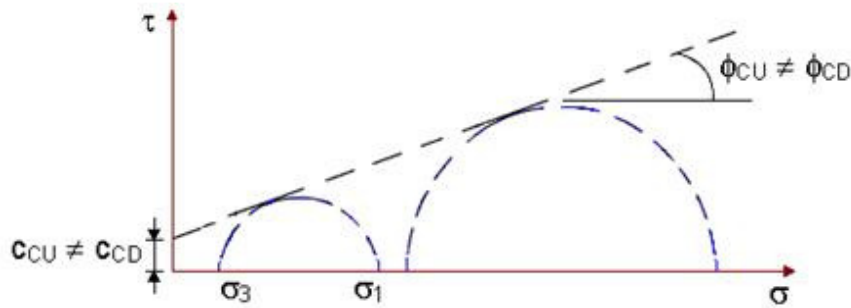
All Mohr circles for UU test plotted in terms of total stresses have the same diameter.

The failure envelope is a horizontal straight line and hence $\phi_{UU} = 0$

It can be represented by the equation:

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

CU & CD Tests:



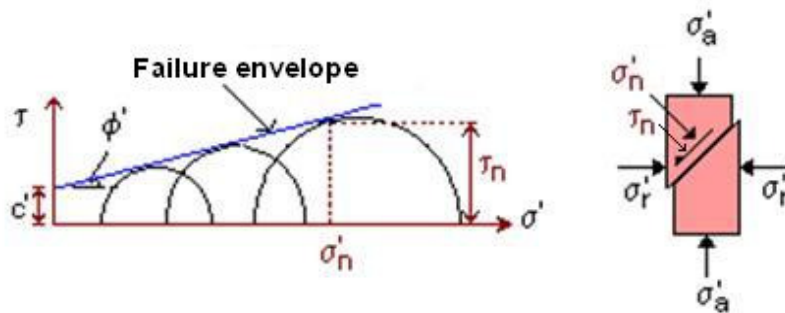
For tests involving drainage in the first stage, when Mohr circles are plotted in terms of total stresses, the diameter increases with the confining pressure. The resulting failure envelope is an inclined line with an intercept on the vertical axis.

It is also observed that $c_{CU} \neq c_{CD}$ and $\phi_{CU} \neq \phi_{CD}$

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.

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.



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

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

where c' = cohesion intercept in terms of effective stress

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

If σ'_n is the effective stress acting on the rupture plane at failure, τ_n 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

$$\sigma'_1 = \sigma'_3 \left(\frac{1 + \sin \phi'}{1 - \sin \phi'} \right) + 2c' \sqrt{\frac{1 + \sin \phi'}{1 - \sin \phi'}}$$

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 the **σ - 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 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:

$$\Delta u = \Delta u_1 + \Delta u_2 = B \cdot \Delta \sigma_3 + B \cdot A \cdot (\Delta \sigma_1 - \Delta \sigma_3) = B [\Delta \sigma_3 + A (\Delta \sigma_1 - \Delta \sigma_3)]$$

where Δu_1 = pore water pressure developed in the first stage during application of confining stress $\Delta \sigma_3$, Δu_2 = pore water pressure developed in the second stage during application of deviator stress $(\Delta \sigma_1 - \Delta \sigma_3)$, and

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 in 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.

For **UU tests** on saturated soils, pore water pressure is not dissipated in both the stages (i.e., $\Delta u = \Delta u_1 + \Delta u_2$).

For **CU tests** on saturated soils, pore water pressure is not dissipated in the second stage only (i.e., $\Delta u = \Delta u_2$).

Stress-Strain Behaviour 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 (σ')**. The relative density is usually defined in percentage as

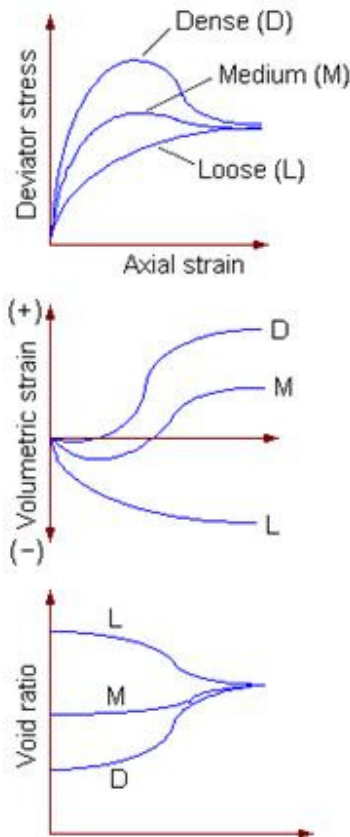
$$I_D = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100$$

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

$$I_D = \left(\frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}} \right) \times \frac{\gamma_{d\max}}{\gamma_d} \times 100$$

where $\gamma_{d\max}$ and $\gamma_{d\min}$ are the maximum and minimum dry densities, and γ_d is the current dry density. Sand is generally referred to as dense if $I_D > 65\%$ and loose if $I_D < 35\%$.

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.



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 higher peak angle of shearing resistance in between.
- Initially dense samples expand or dilate when sheared, and initially loose samples compress.

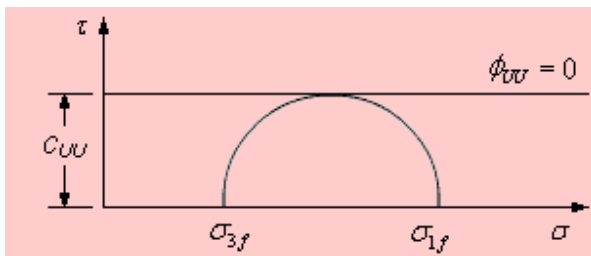
Worked Examples

Example 1: A UU test is carried out on a saturated normally consolidated clay sample at a confining pressure of 3 kg/cm^2 . The deviator stress at failure is 1 kg/cm^2 .

(a) Determine its total stress strength parameters.

(b) If another identical sample is tested at a confining pressure of 4 kg/cm^2 , what will be the vertical axial stress at failure?

Solution:



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

(a) $\sigma_{1f} - \sigma_{3f} = 1 \text{ kg/cm}^2$

From the plot, note that $\phi_{UU} = 0$ and

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

(b) $\sigma_{3f} = 4 \text{ kg/cm}^2$

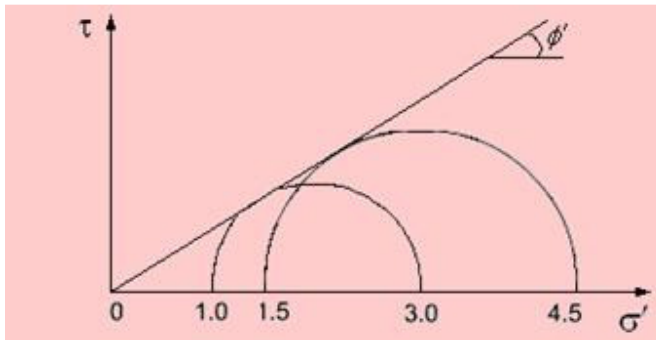
UU tests on identical samples yield the same failure deviator stress $(\sigma_{1f} - \sigma_{3f})$ at all confining pressures.

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

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

	Sample1	Sample2
Confining pressure -----	4.8 kg/cm ²	6.3 kg/cm ²
Axial stress at failure -----	6.8 kg/cm ²	9.3 kg/cm ²
Pore water pressure at failure ---	3.8 kg/cm ²	4.8 kg/cm ²

Solution:



For sample 1:

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

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

For sample 2:

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

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

From the plot, one can obtain

$$c' \approx 0$$

$$\phi' = 30^\circ$$

Compaction of Soils

Compaction is the application of mechanical energy to a soil so as to rearrange its particles and reduce the void ratio.

It is applied to improve the properties of an existing soil or in the process of placing fill such as in the construction of embankments, road bases, runways, earth dams, and reinforced earth walls. Compaction is also used to prepare a level surface during construction of buildings. There is usually no change in the water content and in the size of the individual soil particles.

The objectives of compaction are:

- To increase soil shear strength and therefore its bearing capacity.
- To reduce subsequent settlement under working loads.
- To reduce soil permeability making it more difficult for water to flow through.

Laboratory Compaction

The variation in compaction with water content and compactive effort is first determined in the laboratory. There are several tests with standard procedures such as:

- Indian Standard Light Compaction Test (similar to Standard Proctor Test)
- Indian Standard Heavy Compaction Test (similar to Modified Proctor Test)

Indian Standard Light Compaction Test

Soil is compacted into a 1000 cm³ mould in 3 equal layers, each layer receiving 25 blows of a 2.6 kg rammer dropped from a height of 310 mm above the soil. The compaction is repeated at various moisture contents.

Indian Standard Heavy Compaction Test

It was found that the Light Compaction Test (Standard Test) could not reproduce the densities measured in the field under heavier loading conditions, and this led to the development of the Heavy Compaction Test (Modified Test). The equipment and procedure are essentially the same as that used for the Standard Test except that the soil is compacted in 5 layers, each layer also receiving 25 blows. The same mould is also used. To provide the increased compactive effort, a heavier rammer of 4.9 kg and a greater drop height of 450 mm are used.

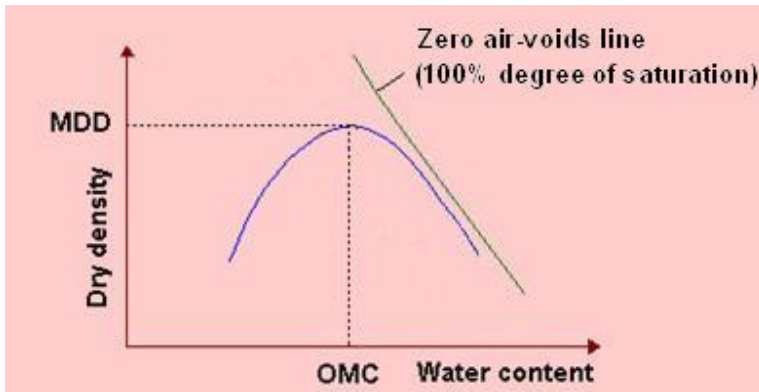
Dry Density - Water Content Relationship

To assess the degree of compaction, it is necessary to use the dry unit weight, which is an indicator of compactness of solid soil particles in a given volume. The laboratory testing is meant to establish the maximum dry density that can be attained for a given soil with a standard amount of compactive effort.

In the test, the dry density cannot be determined directly, and as such the bulk density and the moisture

content are obtained first to calculate the dry density as $\gamma_d = \frac{\gamma_t}{1 + w}$, where γ_t = bulk density, and w = water content.

A series of samples of the soil are compacted at different water contents, and a curve is drawn with axes of dry density and water content. The resulting plot usually has a distinct peak as shown. Such inverted "V" curves are obtained for **cohesive soils** (or soils with fines), and are known as compaction curves.



Dry density can be related to water content and degree of saturation (S) as

$$\gamma_d = \frac{G_s \cdot \gamma_w}{1 + e} = \frac{G_s \cdot \gamma_w}{1 + \frac{w \cdot G_s}{S}}$$

Thus, it can be visualized that an increase of dry density means a decrease of voids ratio and a more compact soil.

Similarly, dry density can be related to percentage air voids (n_a) as

$$\gamma_d = \frac{(1 - n_a) G_s \cdot \gamma_w}{1 + w G_s}$$

The relation between moisture content and dry unit weight for a saturated soil is the **zero air-voids line**. It is not feasible to expel air completely by compaction, no matter how much compactive effort is used and in whatever manner.

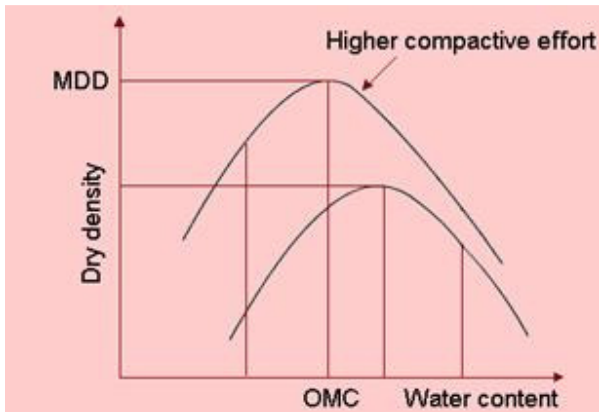
Effect of Increasing Water Content

As water is added to a soil at low moisture contents, it becomes easier for the particles to move past one another during the application of compacting force. The particles come closer, the voids are reduced and this causes the dry density to increase. As the water content increases, the soil particles develop larger water films around them.

This increase in dry density continues till a stage is reached where water starts occupying the space that could have been occupied by the soil grains. Thus the water at this stage hinders the closer packing of grains and reduces the dry unit weight. The **maximum dry density (MDD)** occurs at an **optimum water content (OMC)**, and their values can be obtained from the plot.

Effect of Increasing Compactive Effort

The effect of increasing compactive effort is shown. Different curves are obtained for different compactive efforts. A greater compactive effort reduces the optimum moisture content and increases the maximum dry density.



An increase in compactive effort produces a very large increase in dry density for soil when it is compacted at water contents drier than the optimum moisture content. It should be noted that for moisture contents greater than the optimum, the use of heavier compaction effort will have only a small effect on increasing dry unit weights.

It can be seen that the compaction curve is not a unique soil characteristic. It depends on the compaction effort. For this reason, it is important to specify the compaction procedure (light or heavy) when giving values of MDD and OMC.

Factors Affecting Compaction

The factors that influence the achieved degree of compaction in the laboratory are:

- Plasticity of the soil
- Water content
- Compactive effort

Compaction of Cohesionless Soils

For **cohesionless soils** (or soils without any fines), the standard compaction tests are difficult to perform. For compaction, application of vibrations is the most effective method. Watering is another method. The seepage force of water percolating through a cohesionless soil makes the soil grains occupy a more stable position. However a large quantity of water is required in this method. To achieve maximum dry density, they can be compacted either in a dry state or in a saturated state.

For these soil types, it is usual to specify a magnitude of **relative density (I_D)** that must be achieved. If e is the current void ratio or γ_d is the current dry density, the relative density is usually defined in percentage as

$$I_D = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100$$

or

$$I_D = \frac{\gamma_{d\max} (\gamma_d - \gamma_{d\min})}{\gamma_d (\gamma_{d\max} - \gamma_{d\min})} \times 100$$

where e_{\max} and e_{\min} are the maximum and minimum void ratios that can be determined from standard tests in the laboratory, and $\gamma_{d\min}$ and $\gamma_{d\max}$ are the respective minimum and maximum dry densities

On the basis of relative density, sands and gravels can be grouped into different categories:

<u>Relative density (%)</u>	<u>Classification</u>
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< 15	Very loose
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15-35	Loose
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35-65	Medium
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65-85	Dense
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> 85	Very dense
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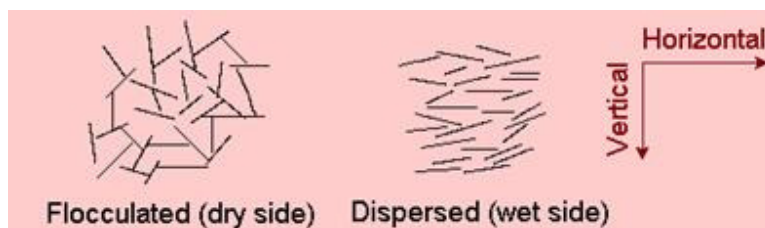
It is not possible to determine the dry density from the value of the relative density. The reason is that the values of the maximum and minimum dry densities (or void ratios) depend on the gradation and angularity of the soil grains.

Engineering Behaviour of Compacted Soils

The water content of a compacted soil is expressed with reference to the OMC. Thus, soils are said to be compacted **dry of optimum** or **wet of optimum** (i.e. on **the dry side** or **wet side** of OMC). The structure of a compacted soil is not similar on both sides even when the dry density is the same, and this difference has a strong influence on the engineering characteristics.

Soil Structure

For a given compactive effort, soils have a flocculated structure on the dry side (i.e. soil particles are oriented randomly), whereas they have a dispersed structure on the wet side (i.e. particles are more oriented in a parallel arrangement perpendicular to the direction of applied stress). This is due to the well-developed adsorbed water layer (water film) surrounding each particle on the wet side.



Swelling

Due to a higher water deficiency and partially developed water films in the dry side, when given access to water, the soil will soak in much more water and then swell more.

Shrinkage

During drying, soils compacted in the wet side tend to show more shrinkage than those compacted in the dry side. In the wet side, the more orderly orientation of particles allows them to pack more efficiently.

Construction Pore Water Pressure

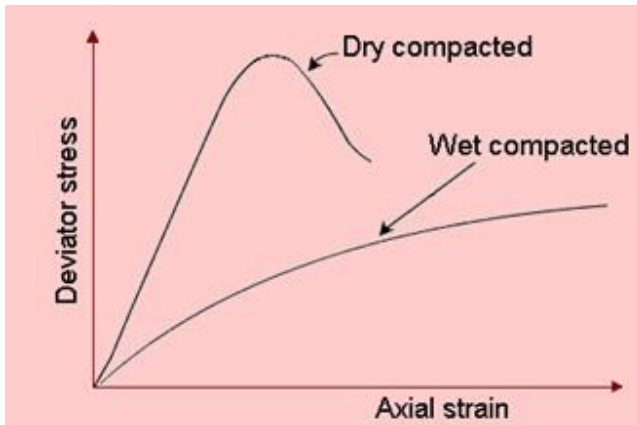
The compaction of man-made deposits proceeds layer by layer, and pore water pressures are induced in the previous layers. Soils compacted wet of optimum will have higher pore water pressures compared to soils compacted dry of optimum, which have initially negative pore water pressure.

Permeability

The randomly oriented soil in the dry side exhibits the same permeability in all directions, whereas the dispersed soil in the wet side is more permeable along particle orientation than across particle orientation.

Compressibility

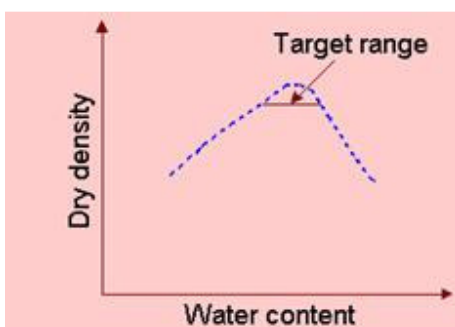
At low applied stresses, the dry compacted soil is less compressible on account of its truss-like arrangement of particles whereas the wet compacted soil is more compressible.



The stress-strain curve of the dry compacted soil rises to a peak and drops down when the flocculated structure collapses. At high applied stresses, the initially flocculated and the initially dispersed soil samples will have similar structures, and they exhibit similar compressibility and strength.

Field Compaction and Specifications

To control soil properties in the field during earthwork construction, it is usual to specify the **degree of compaction** (also known as the **relative compaction**). This specification is usually that a certain percentage of the maximum dry density, as found from a laboratory test (Light or Heavy Compaction), must be achieved. For example, it could be specified that field dry densities must be greater than 95% of the maximum dry density (MDD) as determined from a laboratory test. Target values for the range of water content near the optimum moisture content (OMC) to be adopted at the site can then be decided, as shown in the figure.



For this reason, it is important to have a good control over moisture content during compaction of soil layers in the field. It is then up to the field contractor to select the thickness of each soil lift (layer of soil added) and the type of field equipment in order to achieve the specified amount of compaction. The standard of field compaction is usually controlled through either end-product specifications or method specifications.

End-Product Specifications

In end-product specifications, the required field dry density is specified as a percentage of the laboratory maximum dry density, usually 90% to 95%. The target parameters are specified based on laboratory test results.

$$\text{Relative compaction} = \frac{\text{Achieved field dry density}}{\text{Laboratory maximum dry density}}$$

The field water content working range is usually within $\pm 2\%$ of the laboratory optimum moisture content.

It is necessary to control the moisture content so that it is near the chosen value. From the borrow pit, if the soil is dry, water is sprinkled and mixed thoroughly before compacting. If the soil is too wet, it is excavated in advance and dried.

In the field, compaction is done in successive horizontal layers. After each layer has been compacted, the water content and the in-situ density are determined at several random locations. These are then compared with the laboratory OMC and MDD using either of these two methods: the sand replacement method, or the core cutter method.

Method Specifications

A procedure for the site is specified giving:

- Type and weight of compaction equipment
- Maximum soil layer thickness
- Number of passes for each layer

They are useful for large projects. This requires a prior knowledge of working with the borrow soils to be used.

Field Compaction Equipment

There is a wide range of compaction equipment. The compaction achieved will depend on the thickness of lift (or layer), the type of roller, the no. of passes of the roller, and the intensity of pressure on the soil. The selection of equipment depends on the soil type as indicated.

Equipment	Most suitable soils	Least suitable soils
Smooth steel drum rollers (static or vibratory)	Well-graded sand-gravel, crushed rock, asphalt	Uniform sands, silty sands, soft clays
Pneumatic tyred rollers	Most coarse and fine soils	Very soft clays
Sheepsfoot rollers	Fine grained soils, sands and gravels with > 20% fines	Uniform gravels, very coarse soils
Grid rollers	Weathered rock, well-graded coarse soils	Uniform materials, silty clays, clays
Vibrating plates	Coarse soils with 4 to 8% fines	
Tampers and rammers	All soil types	

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