



**University of Nairobi**

**Department of Civil and Construction Engineering**

**FCE 311**

**GEOTECHNICAL ENGINEERING 1**

**LECTURE NOTES**

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## 1 OVERVIEW

### 1.1 COURSE DESCRIPTION

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This course is an introductory part of Soil Mechanics, which focuses on soil formation, soil structures, physical properties of soils, soil classifications, soil compaction and permeability.

### 1.2 PREREQUISITE

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None

### 1.3 STUDENT LEARNING OUTCOME

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Upon successful completion of this course, the students should acquire the following knowledge:

- a) Developed competence in the principles of soil mechanics and application in engineering practice.
- b) Ability to list the relevant engineering properties of soils and their characteristics and describe the factors which control these properties.
- c) Apply laboratory methods of determining the properties of soils.
- d) Ability to identify common situations when the soil becomes a factor in an engineering or environmental problem.
- e) Ability to apply basic analytical procedures to obtain the engineering quantity desired and understand their limitations.

### 1.4 TEACHING METHODOLOGY AND TECHNIQUES

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This course relies on lectures and Power Point presentation by the lecturer. Worked examples will be offered. Students will then be required to contribute to discussions based on the explanations and will need to read the corresponding section in the assigned textbook.

### 1.5 REQUIRED TEXT BOOKS

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- a) Modern Geotechnical Engineering, CBS Publishers & Distributors, New Dheli
- b) Geotechnical Engineering (Basics of Soil Mechanics), S. Chand & Company Ltd, New Dheli
- c) Foundation Engineering Handbook, CBS Publishers & Distributors, New Dheli.

## 2 INTRODUCTION TO SOIL MECHANICS

### 2.1 DEFINITION OF SOIL

Soil is the relatively loose mass of mineral and organic materials and sediments found above the bedrock, which can be relatively easily broken down into its constituent mineral or organic particles.

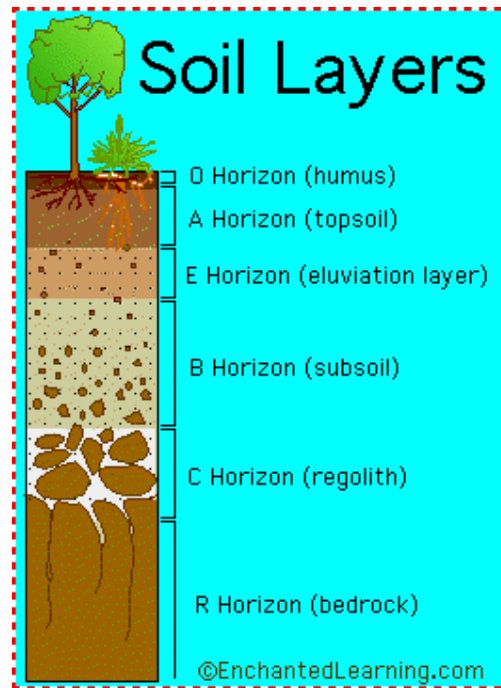


Fig. 2-1: Soil layers

Soil consists of layers of minerals constituents of variable thickness, which differ from the parent materials in the morphological, physical, chemical and mineralogical characteristics, as shown in Fig. 2-1. It is thus a natural product of weathering of rocks and decomposition of organic matter. It is an accumulation of individual particles that are bonded together by mechanical or attractive means, the strength of the bonds being a small fraction of the mineral particles. The particles may range from colloidal size to small boulders.

Soil can also be referred to as regolith, or loose rock material.

### 2.2 SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

Soil mechanics is a branch of engineering mechanics that describes the behaviour of soils. Soil mechanics provide the theoretical basis for analysis in geotechnical engineering.

Geotechnical Engineering is the branch of civil engineering concerned with the engineering behaviour of earth materials. It uses principles of soil mechanics, rock mechanics and engineering geology to investigate subsurface conditions and materials, determine the relevant physical/mechanical and chemical properties of the materials, evaluate stability of natural slopes and man-made soil deposits, assess risks posed by site conditions, design earthworks and structure foundations and monitor site conditions, earthwork and foundation construction.

A typical geotechnical engineering project begins with a review of project needs to define the required material properties. Then follows a site investigation of soil, rock, fault distribution and bedrock properties on and below an area of interest to determine their engineering properties.

Site investigations are needed to gain an understanding of the area in or on which the engineering will take place. Investigations can include the assessment of the risk to humans, property and the environment from natural hazards such as earthquakes, landslides, soil liquefaction, debris flows and rock falls.

A geotechnical engineer then determines and designs the type of foundations, earthworks and pavement subgrades required for the intended man-made structures to be built. Foundations are designed and constructed for structures of various sizes such as high-rise buildings, bridges, medium to large commercial buildings, and smaller structures where the soil conditions do not allow code-based design.

Foundations built for above-ground structures include shallow and deep foundations. Retaining structures include earth-filled dams and retaining walls. Earthworks include embankments, tunnels and sanitary landfills.

Geotechnical engineering is also related to coastal and ocean engineering. Coastal engineering can involve the design and construction of wharves (structures on the shore of harbour where ships may dock to load and unload cargo or passengers) and jetties (structures that project into a body of water to influence the current or tide or to protect a harbour or shoreline from storms or erosion).

## 3 SOIL FORMATION

### 3.1 DEFINITION

Soil formation is the process by which soil is created. The formation of soil happens over a very long period of time. Soil is formed from the weathering of rocks and minerals.

### 3.2 WEATHERING

#### 3.2.1 Introduction

Weathering is the process of breaking down rocks. Weathering occurs *in situ* or “with no movement”, and thus should not be confused with erosion, which involves the movement of rocks and minerals by agents such as water, ice, wind, and gravity.

Two important classifications of weathering processes exist – Physical and Chemical Weathering

#### 3.2.2 Physical weathering

Involves the breakdown of rocks and soils through direct contact with atmospheric conditions, such as heat, water, ice and pressure, without any change in chemical condition. The soil formed due to physical weathering will be cohesionless (sand and gravel).

In summary, the physical agencies causing mechanical weathering of rocks are;

- (i) Daily and seasonal temperature changes.
- (ii) Flowing water, glaciers and wind, which produce impact and abrasive action on rock.
- (iii) Splitting action of ice.
- (iv) Growth of roots of plants in rock fissures and to a minor degree burrowing activities of small animals like earthworms.

#### 3.2.3 Chemical weathering

Chemical weathering changes the composition of rocks by decomposing the parent minerals, transforming them into new compounds such as clay silica particles, carbonates and iron oxides.

The decomposition of rock is the result of the following reactions;

- (i) Oxidation
- (ii) Carbonation
- (iii) Hydration
- (iv) Leaching

##### i) Oxidation

Within the weathering environment, oxidation of a variety of metals occurs. The most commonly observed is the oxidation of  $\text{Fe}^{2+}$  (iron) and combination with oxygen and water to form  $\text{Fe}^{3+}$  hydroxides and oxides such as goethite, limonite and hematite. This gives the affected rocks a reddish-brown coloration on the



surface which crumbles easily and weakens the rock. This process is better known as 'rusting'.

### ii) Carbonation

Carbonation of rock material is caused by carbon dioxide in the presence of water. Limestones are very much affected by carbonation.

### iii) Hydration

Mineral hydration is a form of chemical weathering that involves the rigid attachment of  $H^+$  and  $OH^-$  ions to the atoms and molecules of a mineral. When rock minerals take up water, the increased volume creates physical stresses within the rock. For example iron oxides are converted to iron hydroxides and the hydration of anhydrite forms gypsum. Another example of hydration is the chemical decomposition of mineral feldspar in granite to form kaolite.

### iv) Leaching

Leaching is the process in which percolating water washes out water-soluble salts from the soil.

Soil produced by chemical weathering of rocks will be cohesive (silt and clay).

## 3.3 RESIDUAL AND ALLUVIAL SOILS

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### 3.3.1 Residual soils

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Residual soils are those which have remained over the parent rock from which they have been formed. They are relatively shallow in depth. They are characterized by a gradual transition from soil through partially weathered rocks, fractured and fissured rock, to bedrock.

### 3.3.2 Alluvial soils

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Alluvial soils are the soils which have been transported and subsequently deposited by flowing water. An alluvial fan is formed when the velocity of a soil-laden stream suddenly decreases due to abrupt decrease in gradient. Flood-plains are formed on the sides of a stream due to overflowing of flood water. A delta is formed just before a stream reaches the standing water of the sea. Alluvial soil deposits are usually stratified because of fluctuations in velocity of flowing water. The average particle size of alluvial deposits decreases with increasing distance from the source of stream. The delta soils are soil deposits farthest from the source of a stream and usually consist of silt and clay. **Marine deposits** are formed when fine-grained soils are carried beyond deltas into the sea. **Lacustrine soils** are soils deposited at the bed of lakes.

## 4 CLAY MINERALOGY

### 4.1 INTRODUCTION

A 'mineral' is an inorganic chemical compound formed in nature. As a solid, it may occur in an amorphous state or in a crystalline state. A 'crystal' is a homogenous body bounded by smooth plane surfaces. Soil particles are largely composed of mineral crystals. Molecules of minerals are composed of atoms of chemical elements. The atoms in a crystal are arranged in a definite orderly manner to form a three dimensional net-work, called a "***lattice***." An atom consists of a small nucleus having a positive electromagnetic charge around which a definite number of negatively charged electrons rotate. The electrons rotate in orbits of different radii forming the so-called electron shells.

Many compounds lose their identity, in solution, by separating into "ions." The ions consist of only one element of the compound or of two or more elements which are not electrically balanced. Atoms get transformed into ions by the gain or loss of electrons. The positively charged ions are called "***cations***" and the negatively charged ions are called "***anions***". On removal from solution the cations and anions unite to form the original solid compound. Many elements do not form ions, yet they unite to form compounds. Solutions of non-ion forming elements or compounds in water are poor conductors of electric current.

### 4.2 ATOMIC AND MOLECULAR BONDS

#### 4.2.1 Introduction

Forces which bind atoms and molecules to build up the structure of substances are primarily of electrical nature. They may be broadly classified into "***primary bonds***" and "***secondary bonds***." Primary bonds combine the atoms into molecules. Secondary bonds link atoms in one molecule to atoms in another. They are much weaker than the primary bonds. Primary bonds are the ionic bond and the covalent bond. Secondary bonds are the hydrogen bond and the Van der Waals bond.

#### 4.2.2 Ionic bond

The ionic bond is the simplest and strongest of the bonds which hold atoms together. This bond is formed between oppositely charged ions by the exchange of electrons. Atoms held together by ionic bonds form "***ionic compounds***", e.g. common salt (sodium chloride), and a majority of clay mineral crystals fall into this group.

Ionic bonding causes a separation between centres of positive and negative charge in a molecule, which tends the molecule to orient in an electric field forming a "***dipole***". Dipole is the arrangement of two equal electro-static charges of opposite sign. A dipolar molecule (Fig. 4-1) is one which is neutral but in which the centres of positive and negative charges are separated such that the molecule behaves like a short bar magnet with positive and negative poles.

+ \_\_\_\_\_ -

Fig. 4-1: Soil layers

**4.2.3 Covalent bond**

The covalent bond (Fig. 4-2) is formed when one or more bonding electrons are shared by two atoms so that they serve to complete the outer shell for each atom.

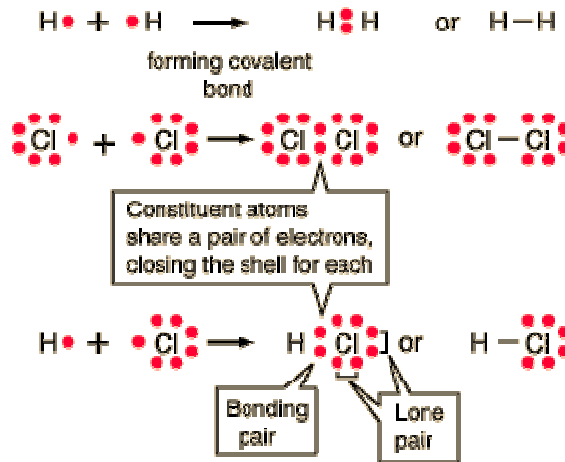
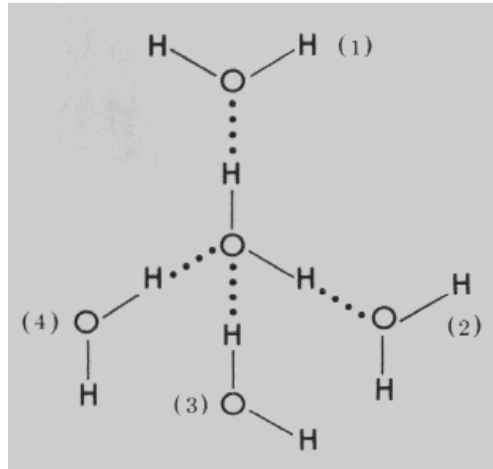


Fig. 4-2: Covalent bonding

**4.2.4 Hydrogen bond**

A hydrogen bond is the attractive interaction of a hydrogen atom with an electronegative atom, such as nitrogen, oxygen, or fluorine, that comes from another molecule.

Thus when water molecules are close together, their positive and negative regions are attracted to the oppositely-charged regions of nearby molecules. The force of attraction, shown in Fig. 4-3 below here as dotted line, is called a hydrogen bond. Each water molecule is hydrogen bonded to four others.



**Fig. 4-3:** *Covalent bonding*

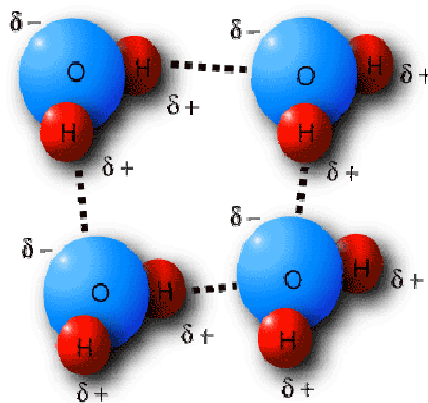
Hydrogen bond can link the oxygen from a water molecule to the oxygen on the clay particles surface. Hydrogen bonding between two oxygen atoms is responsible for some of the weaker bonds between crystal layers for holding water at the clay surface and for bonding organic molecules to the clay surface.

**4.2.5 Van der Waals bonds**

It is the sum of the attractive or repulsive forces between molecules (or between parts of the same molecule) other than those due to covalent bonds or ionic bond.

The covalent bonds within the molecules are very strong and rupture only under extreme conditions. The bonds between the molecules that allow siding and rupture to occur are called Van der Waals forces.

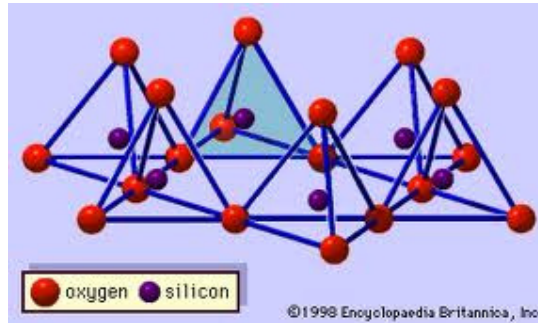
When ionic and covalent bonds are present, there is some imbalance in the electrical charge of the molecule. The angle hydrogen atoms are bonded to oxygen atom in water produces a positive polarity at the hydrogen-rich end of the molecule and a negative polarity at the other end. As a result of this charge imbalance the water molecules are attracted to each other. This is the force that holds the molecules together in a drop of water, shown in Fig. 4-4.



**Fig. 4-4:** *Van der Waals Bonding*



In the silica sheet, the bases of the tetrahedrals are all in the same plane and the tips all point in the same direction. Each of the three oxygens at the base is shared by two silicons of adjacent units.

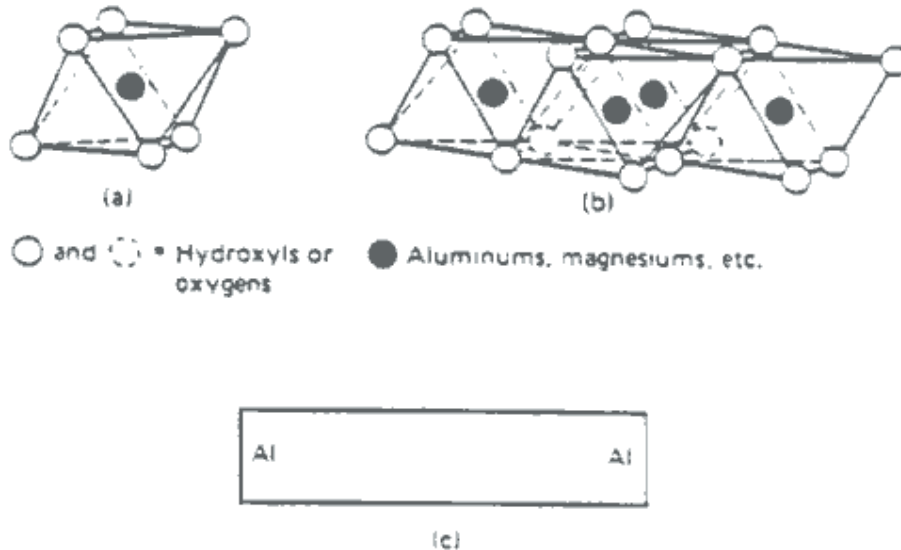


**Fig. 4-6: Silicon Tetrahedral Sheet**

For simplicity, the silica tetrahedral sheet is represented by the symbol below in Fig. 4.5(c).

**4.3.3 Aluminium (or Magnesium) Octahedral Unit**

The octahedral unit (Fig. 4-7) has an aluminium ion or a magnesium ion endorsed by six hydroxyl radicals or oxygens arranged in the form of an octahedron. In some cases, other cations (e.g. Fe) are present in place of Al and Mg.



- (a) Octahedron.
- (b) Octahedral sheet.
- (c) Schematic of gibbsite octahedral sheet. See Table 2-1.

Source: Lambe, 1958; Grim, 1963

Figure 2-2. Clay mineral octahedral sheet structure.

**Fig. 4-7: Aluminium or Magnesium Octahedral**

Combination of octahedral units forms an Octahedral sheet, which is called a **'gibbsite'** sheet if the central action of the unit is aluminum or a **"brucite"** sheet if the central cation is magnesium.

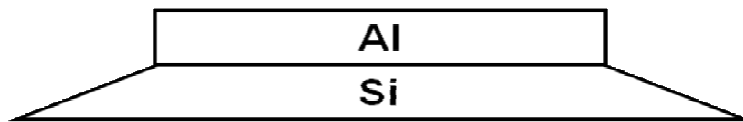
**4.4 TYPES OF CLAY MINERALS**

**4.4.1 Introduction**

The variation in the stacking of the two basic sheet structures and nature of bonding has given rise to over dozen clay minerals which have been identified. From an engineering point of view, three clay minerals of interest are kaolinite, montmorillonite and illite.

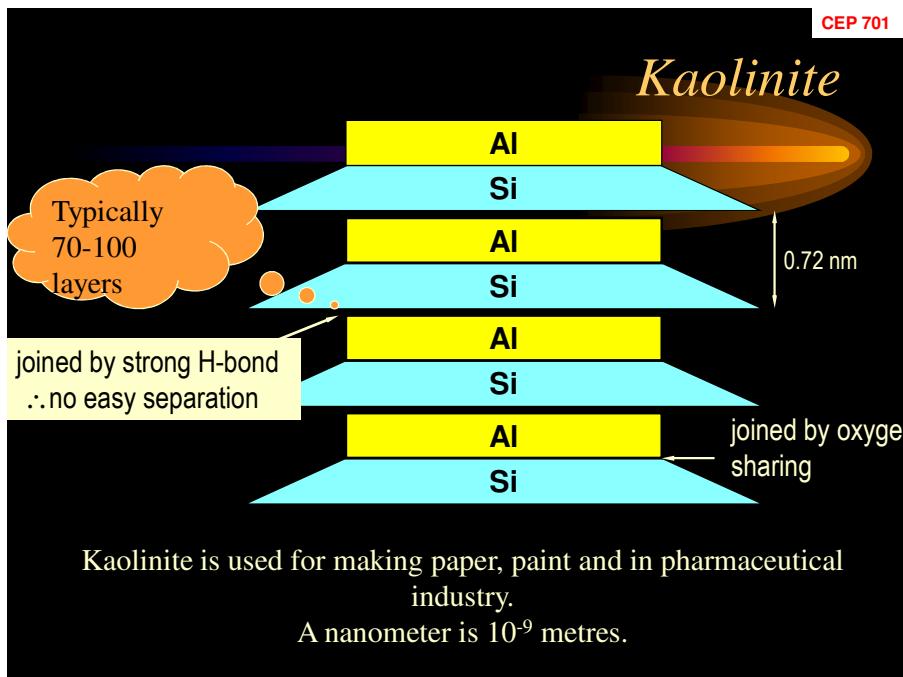
**4.4.2 Kaolinite**

This is the most common of the Kaolin group. Each structural unit of Kaolinite is a combination of two layers with apices of a silica layer joined to one of a gibbsite layer. The structural unit is represented by the symbol as shown in Fig. 4-8.



**Fig. 4-8: Kaolinite symbol**

Successive layers of structural units are held together to form kaolinite particles which occur as platelets as shown in Fig. 4-9.



**Fig. 4-9: Kaolinite particles**

### 4.4.3 Montmorillonite

The montmorillonite mineral is a stacking of basic sheet like structural units, with each unit made up of gibbsite sheet sandwiched between two silica sheets and is represented by the symbol in Fig. 4-10 and the particles in Fig. 4-11.

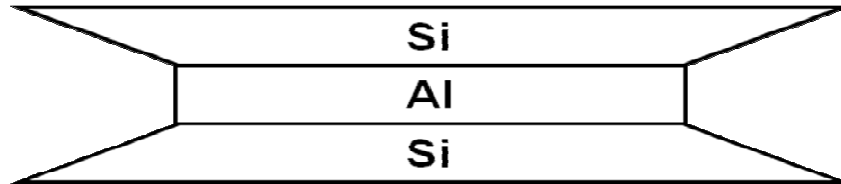


Fig. 4-10: Montmorillonite symbol

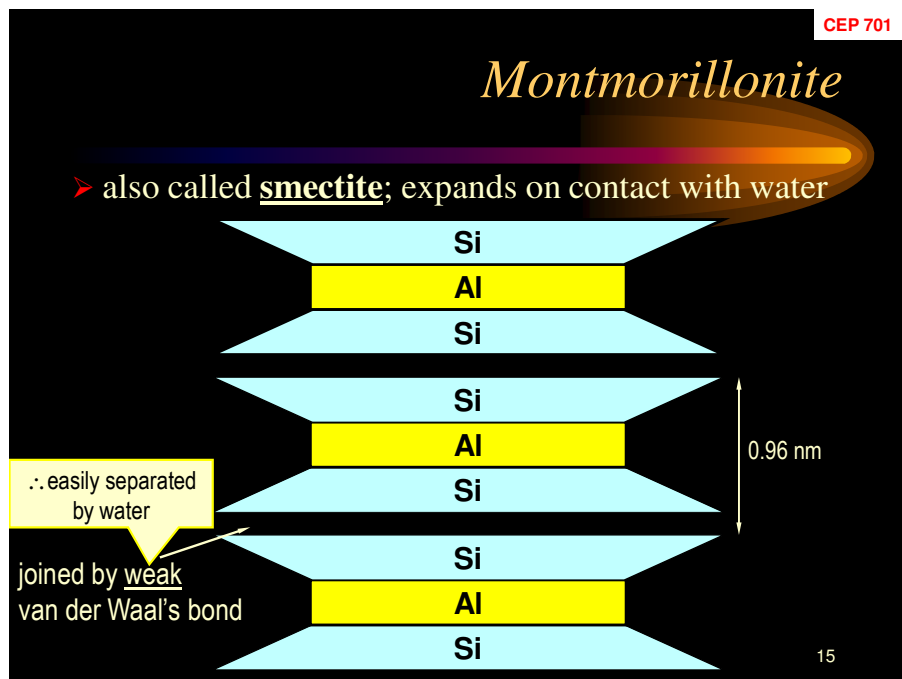


Fig. 4-11: Montmorillonite particles

Because of the fact that bonding by Van der Waals forces between silica sheet of adjacent structural units is weak and there is a net negative charge deficiency in octahedral sheet, water and exchangeable cations can enter and separate the layers. Thus soil containing montmorillonite mineral exhibits high swelling and shrinkage characteristics.

### 4.4.4 Illite

The basic structural unit of illite (Fig. 4-12) is the same as that of montmorillonite except for the fact that there is some substitution of aluminium for silicon in the silica sheet and the resultant charge deficiency is balanced by potassium ions,



which bond the layers in the stack. The bond with the non-exchangeable  $K^+$  ions are weaker than the hydrogen bond in the Kaolinite but is stronger than the water bond of montmorillonite. The illite crystal does not swell so much in the presence of water as does in montmorillonite particles.

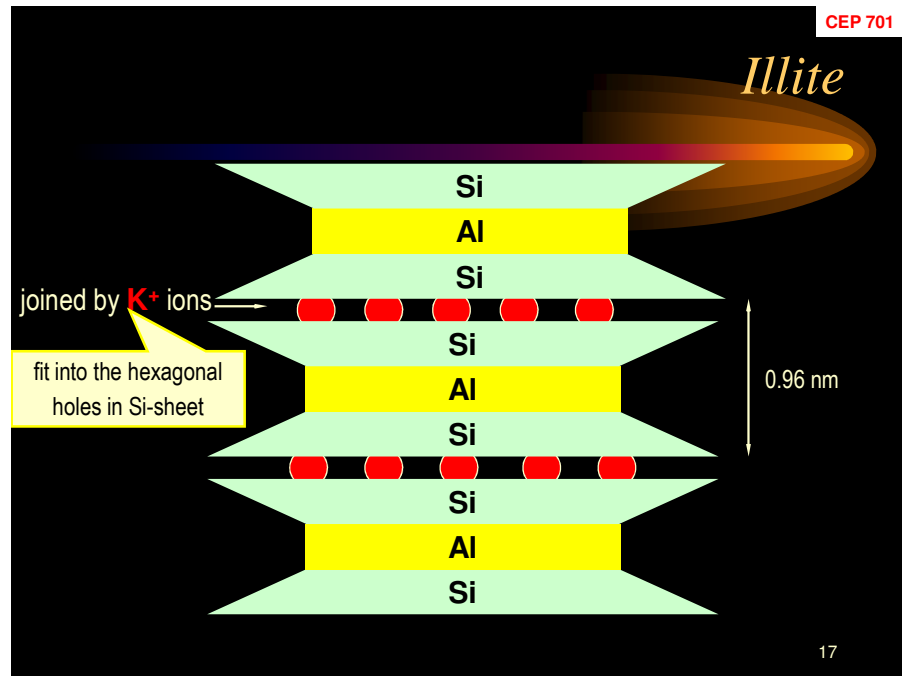


Fig. 4-12: Illite particles

## 4.5 SOIL STRUCTURES

### 4.5.1 Introduction

The term soil structure in general, refers to the arrangement or state of aggregation of particles in a soil mass. The engineering behaviour of soils is influenced by soil structure to varying degrees.

### 4.5.2 Types of Soil Structures

Following are the types of soil structures which have been recognized in various soil deposits;

- (i) Single grained structure.
- (ii) Honeycomb structure.
- (iii) Flocculated structure and dispersed structure – in the case of clay deposits.
- (iv) Course-grained skeleton structure and matrix structure – in the case of composite soils.

(i) **Single grained structure**

This type of structure (Fig. 4-13) will be found in the case of coarse-grained soil deposits. When such soils settle out of suspension in water, the particles settle independently of each other. The major force causing their deposition is gravitational and the surface forces are too small to produce any effect. There will be particle-to-particle contact in the deposit. The void ratio attained depends on the relative size of grains.



**Fig. 4-13:** *Single grained structure*

(ii) **Honeycomb structure**

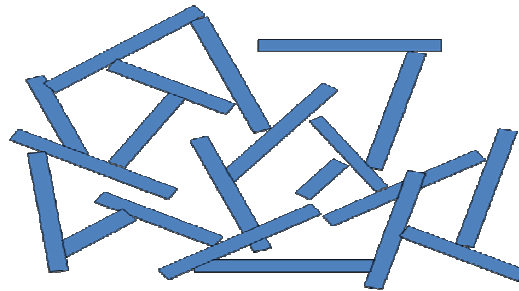
This type of structure (Fig. 4-14) is associated with silt deposits. When silt particles settle out of suspension, in addition to gravitational forces, the surface forces also play a significant role. When particles approach the lower region of suspension they will be attracted by particles already deposited as well as the neighbouring particles leading to formation of arches. The combination of a number of arches leads to the honey comb structure.

As the deposit has high void ratio, when disturbed, there will be large reduction in volume due to breakdown of structure.

**Fig. 4-14:** *Honeycomb structure*

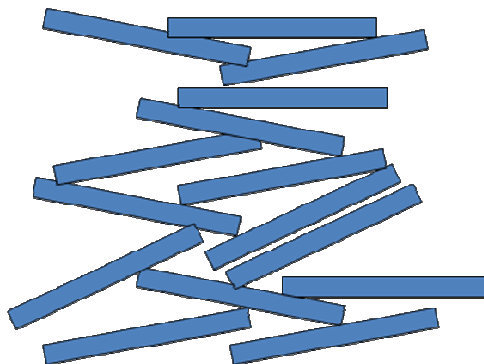
**(iii) Flocculated structure and Dispersed structure**

These are the two types of structures found in clay deposits. In the case of flocculated structure, there will be edge-to-edge and edge-to-face contact between particles. This type of formation is due to the net electrical forces between the adjacent particles at the time of disposition being attractive in nature. The concentration of dissolved minerals in water leads to formation of flocculated structure with very high void ratio as in the case of marine deposits as shown in Fig. 4-15.



**Fig. 4-15:** *Flocculated structure*

In the case of dispersed or oriented structure, the particles will have face to face contact as shown in Fig. 4-16. This type for formation is due to net electrical forces between adjacent soil particles at the time of deposition being repulsive in nature. This type of structure is common in fresh water deposits. Clays with flocculated structure will have relatively high void ratio. Remoulding of such soils or application of pressure as in compaction leads to slippage of particles resulting in dispersed structure with decrease in void ratio. Consolidation also tends to reorient the particles to form dispersed structure with decrease in volume.



**Fig. 4-16:** *Dispersed structure*

(iv) **Course-grained skeleton and Cohesive matrix structure (composite soil)**

The course-grained skeleton structure can be found in the case of composite soils in which the course-grained fraction is greater in proportion compared to fine-grained fraction as shown in Fig. 4-17. The course-grained particles form the skeleton with particle to particle contact and the voids between these particles will be occupied by the fine-grained particles.

**Fig. 4-17:** *Course-grained skeleton structure*

The cohesive matrix structure (Fig. 4-18) can be found in composite soils in which the fine-grained fraction is more in proportion compared to course grained fraction. In this case the course-grained particles will be embedded in fine-grained fraction and will be prevented from having particle-to-particle contact. This type of structure is relatively more compressible compared to the more stable course grained structure.

**Fig. 4-18:** *Cohesive matrix structure structure*

## 5 BASIC PROPERTIES

### 5.1 SOME USEFUL TERMS AND ASSOCIATED SYMBOLS

Table 5-1 is a list of some useful terms and associated symbols for phase relationships.

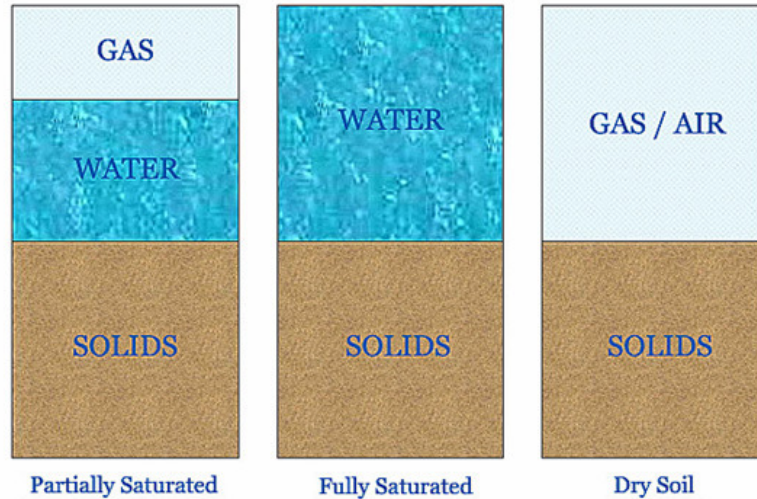
**Table 5-1:** *List of useful terms*

SYMBOL	DEFINITION	DIMENSION
$e$	Void Ratio	-
LL or $W_L$	Liquid Limit	-
$M_s$	Mass of Solids	M
$M_w$	Mass of Water	M
$n$	Porosity	-
PI or $I_p$	Plasticity Index	-
PL or $W_p$	Plastic Limit	-
$S_r$	Degree Of Saturation	-
SL or $W_s$	Shrinkage Limit	-
$V_a$	Volume of Air	$L^3$
$V_s$	Volume of Solids	$L^3$
$V_v$	Volume of Voids	$L^3$
$W$	Water Content	-
$\rho_d$	Dry Density	$M/L^3$
$\rho_s$	Density of Solids	$M L^3$
$\rho_{sat}$	Saturated Density	$M/L^3$
$\rho_w$	Density of Water	$M/L^3$
$\rho$	Bulk Density	$M/L^3$

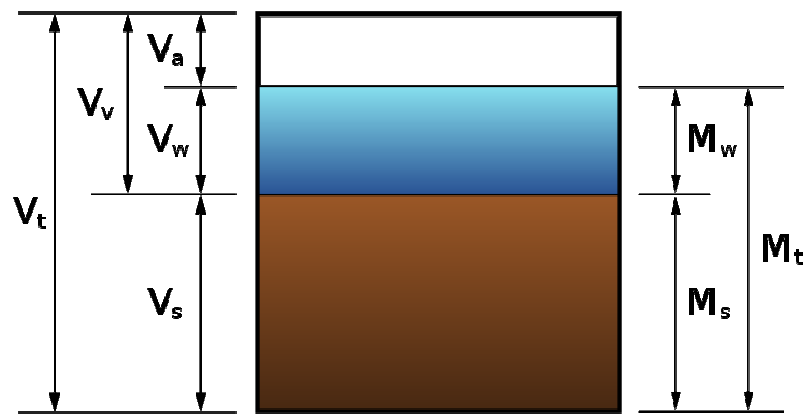
## 5.2 PHASE RELATIONSHIP

### 5.2.1 Phase diagram

Soils are generally composed of three distinct phases. These are solids, water and air. The space occupied by water and air is defined as the void of the soil. The void may be partially or wholly filled by water or air. A completely dry or completely saturated soil will have only two phases. The components parts may be illustrated as in Fig. 5-1 and Fig. 5-2 by a phase diagram.



**Fig. 5-1:** Three Typical Phase Diagrams of Soil



**Fig. 5-2:** Phase diagram of soil mass

The relationships shown derived here in Section 5.2 are commonly used in geotechnical engineering. They are derived with reference to Fig. 4-16.

### 5.2.2 Void ratio (e)

This is the ratio of the volume of voids to the volume of the solids;

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

### 5.2.3 Moisture content (Water content) (w)

Is the ratio of mass of water to mass of solids;

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

### 5.2.4 Porosity (n)

This is the ratio of the volume of voids to the total volume;

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

### 5.2.5 Specific Volume (v)

Is the total volume of the soil which contains a unit volume of solid particles. Fig. 5.3 shows a soil model with unit volume of solids.

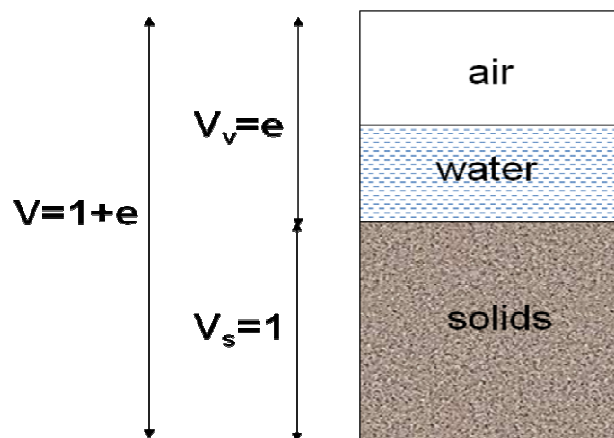


Fig. 5-3: Soil model with unit volume of solids

### 5.2.6 Relative Density (Specific Gravity) ( $G_s$ )

The relative density (specific gravity) of soil particles is the ratio of the density of solids to the density of water. It is a measure of heaviness of material.

$$G_s = \frac{\rho_s}{\rho_w}$$

### 5.2.7 Degree of Saturation ( $S_r$ )

Is the ratio of the volume of water to the volume of voids usually expressed as a percentage.

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

The voids may be filled with air or water or both. If only air is present, then the soil is dry and the degree of saturation is zero. If on the other hand the voids are filled with water the degree of saturation is 100%.

The degree of saturation can be expressed in terms of  $G_s$  and  $e$  and derived below;

$$S_r = \frac{V_w}{V_v} \times \frac{M_s}{M_s} \times \frac{M_w}{M_w} \times \frac{V_s}{V_s}$$

Where;

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

$$G_s = \frac{\rho_s}{\rho_w} = \frac{M_s}{V_s} \times \frac{V_w}{M_w}$$

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

$$\therefore S_r = \frac{w \times G_s}{e} = \frac{wG_s}{e}$$

### 5.2.8 Air Content ( $A$ )

Is the ratio of the volume of air to the total volume of the soil

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



The air content can be expressed in terms of  $e$ ,  $w$  and  $G_s$  as derived below;

$$A = \frac{V_a}{V} = \frac{V_v - V_w}{V_v + V_s} = \frac{(V_v - V_w)/V_s}{(V_v + V_s)/V_s}$$

$$= \frac{\frac{V_v}{V_s} - \frac{V_w}{V_s}}{\frac{V_v}{V_s} + \frac{V_s}{V_s}}$$

$$\text{Where, } G_s = \frac{M_s}{V_s} \times \frac{V_w}{M_w}$$

$$\text{And, } \frac{V_w}{V_s} = e$$

$$\therefore A = \frac{e - wG_s}{1 + e} = \frac{e - S_r e}{1 + e} = \frac{e}{1 + e} (1 - S_r) = n(1 - S_r)$$

### 5.2.9 Density of Solids ( $\rho_s$ )

Is the ratio of the mass of the solids to the volume of the solids.

$$\rho_s = \frac{M_s}{V_s}$$

### 5.2.10 Bulk Density ( $\rho$ )

Is the ratio of the total mass of soil to the total volume of the soil.

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

It is noted that the weight considered is the weight of the solids and that of water while the volume is the volume of the solids and that of the voids. This can be expressed in terms of  $W$ ,  $G_s$ ,  $e$  and  $S_r$ .

$$\rho = \frac{M}{V} = \frac{M_s + M_w}{V_s + V_v}$$

$$\text{But, } G_s = \frac{\rho_s}{\rho_w} \therefore \rho_s = G_s \rho_w$$

$$\frac{M_s}{V_s} = G_s \rho_w \therefore M_s = G_s V_s \rho_w$$

Substituting,

$$\rho = \frac{(G_s \cdot V_s \cdot \rho_w + M_w) / V_s}{(V_s + V_v) / V_s}$$

$$\text{Now, } \frac{M_w}{V_s} = \frac{\rho_w \cdot V_w}{V_s} \cdot \frac{V_v}{V_v} = \rho_w \cdot S_r \cdot e$$

$$\therefore \rho = \frac{(G_s \cdot V_s \cdot \rho_w + V_v \cdot \rho_w \cdot S_r) / V_s}{(V_s + V_v) / V_s}$$

$$\therefore \rho = \frac{\rho_w (G_s + e S_r)}{1 + e}$$

### 5.2.11 Saturated Density ( $\rho_{sat}$ )

The saturated density  $\rho_{sat}$  is the bulk density of a soil mass when fully saturated. For saturated soil,  $S_r = 1$ ,

$$\therefore \rho_{sat} = \frac{\rho_w (G_s + e)}{1 + e}$$

### 5.2.12 Dry Density ( $\rho_d$ )

Is the mass of soil solids per unit of total volume of a dry soil mass. For completely dry soil,  $S_r = 0$ ,

$$\therefore \rho_{sat} = \frac{\rho_w \cdot G_s}{1 + e}$$

The above bulk density expressions can be converted into force units where the unit weight of soil is the total weight in force units to the total volume of water.

The unit weight,  $\gamma = \frac{W}{V} = \frac{M \cdot g}{V}$ , where  $g$  is acceleration due to gravity (9.8 kN/m<sup>3</sup>).

$$\gamma = \frac{W}{V} = \frac{\gamma_w (G_s + e \cdot S_r)}{1 + e}$$

Where the *in situ* soil is fully saturated, the solid particles are subjected to a thrust by the water, which acts in all directions.

### 5.2.13 Dry Unit Weight ( $\gamma_{dry}$ )

Is defined as the weight of solids per unit weight of the solids. This is the same as the bulk unit of a dry soil with force units.

$$\gamma_d = \frac{W_s}{V} = \frac{W_s / V_s}{(V_s + V_v) V_s} = \gamma_s \frac{1}{1 + e}$$

$$\gamma_d = \frac{G_s \cdot \gamma_w}{1 + e}, \text{ since, } G_s = \frac{\rho_s}{\rho_w} = \frac{\gamma_s}{\gamma_w}$$

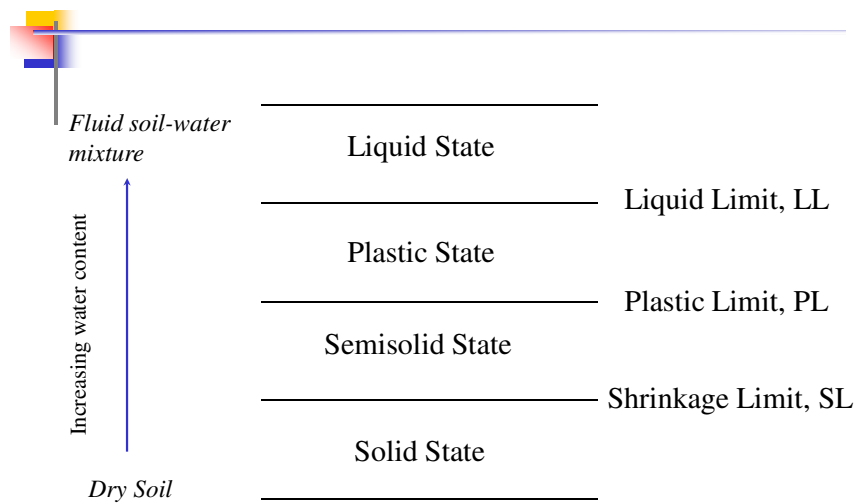
### 5.2.14 Submerged Unit Weight ( $\gamma_{sub}$ )

Where the *in situ* soil is saturated with water, the particles are subjected to an all round thrust. The submerged unit weight is given by difference in the unit weight of a saturated weight of soil and the unit weight of water.

## 6 CONSISTENCY AND PLASTICITY OF SOILS

### 6.1 STATE OF CONSISTENCY

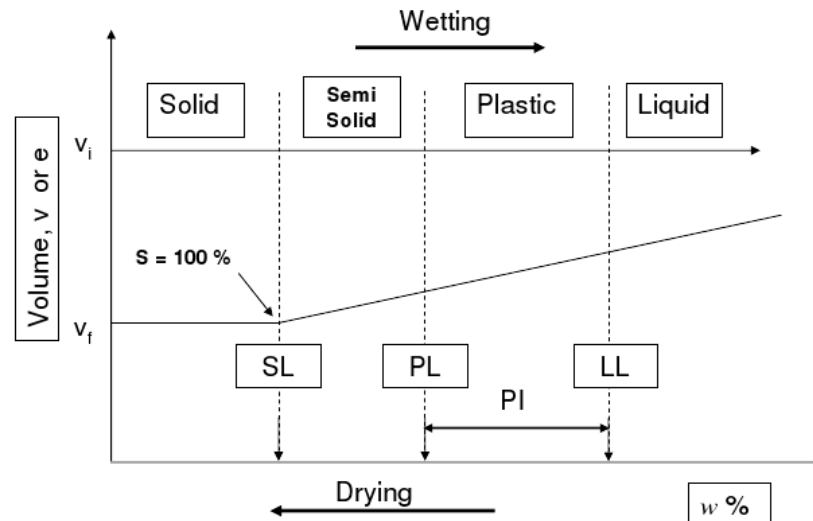
The term consistency refers to the relative ease with which a soil mass can be deformed and is used to describe the degree of firmness of fine-grained soils for which consistency relates to a large extent to water content. The four states of consistency suggested by Atterberg are indicated below in Fig. 6-1 and 6-2.



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**Fig. 6-1:** *The four states of consistency*

## Atterberg Limits (cont.)



**Fig. 6-2:** The four states of consistency in a chart form

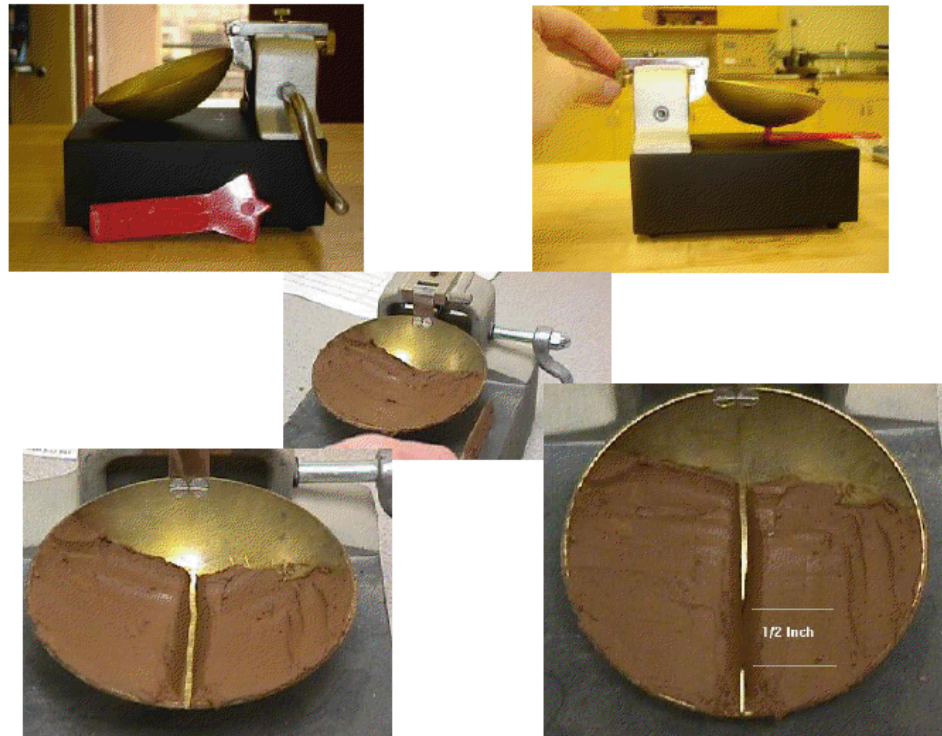
In the solid state there will be no change in volume of soil mass accompanying change in water content. In the remaining the three states increase in water content is accompanied by increase in volume of soil mass and decrease of water content by reduction in volume of soil mass. In the liquid state the soil mass behaves like a liquid possessing very less shear strength. In the plastic state the soil mass can be deformed without cracking. In the semi-solid state the soil mass cannot be deformed without cracking.

The water contents, which arbitrarily define the boundary between the four states of consistency, are referred to as consistency limits or Atterbag limits.

The three consistency limits are Liquid Limit, Plastic Limit and Shrinkage Limit.

Liquid limit is denoted by  $W_L$  and is the boundary between plastic and liquid states of consistency. It is the minimum water content at which the soil mass still flows like a liquid.

Liquid limit is defined as the water content at which a groove, cut with a standard grooving tool, in soil pat taken in the cup of a standard liquid limit device (Fig. 6-3) closes for a distance of 13mm when the cup is imparted 25 blows.



**Fig. 6-3:** *The standard liquid limit device*

Plastic limit is denoted by  $W_p$  and is the boundary between semi-solid and plastic states of consistency. It is the minimum water content at which the soil mass can still be deformed without cracking.

Plastic limit is defined as the water content at which the soil mass can be rolled into a thread of 3mm diameter and the thread first shows signs of cracking.

Shrinkage limit is denoted by  $W_s$  and is the boundary between solid and semi-solid state of consistency. It is defined as the maximum water content at which there is no reduction in volume of soil mass accompanying reduction in water content.

The laboratory determination of these states is discussed in Chapter 7.

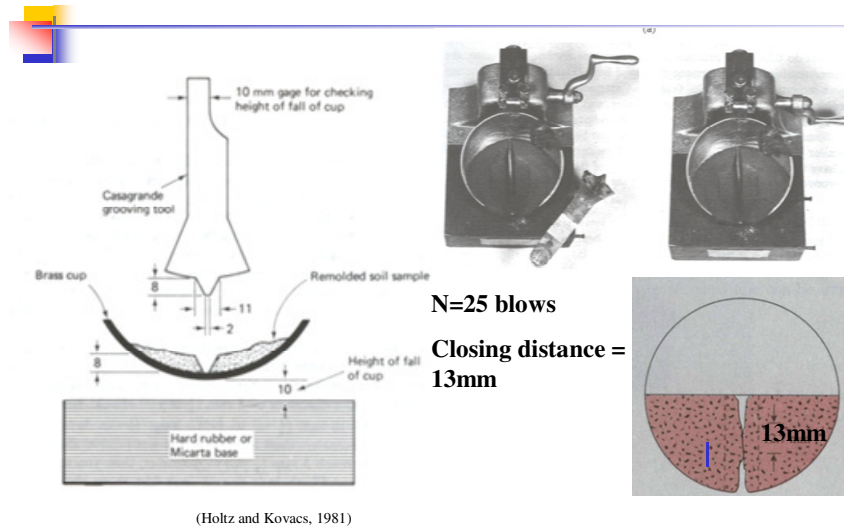
## 7 DETERMINATION OF SOIL PROPERTIES BY LABORATORY TESTING

### 7.1 DETERMINATION OF LIQUID LIMIT

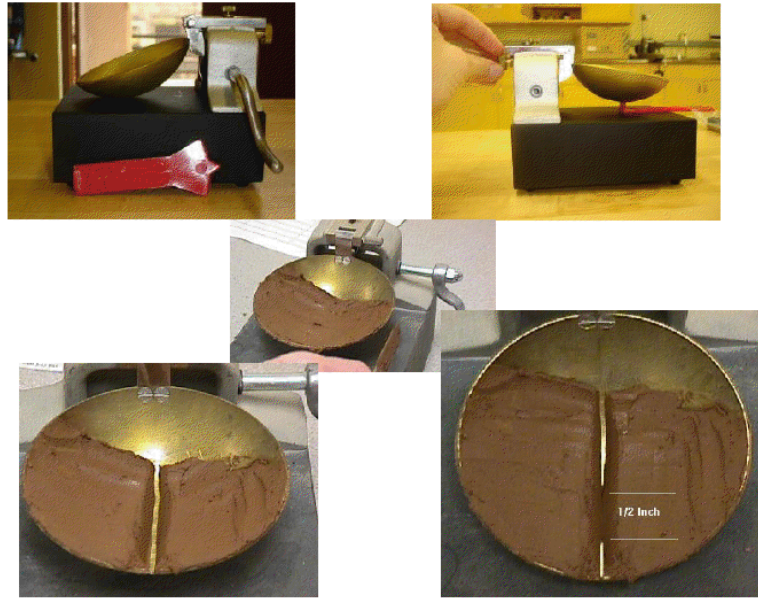
#### 7.1.1 Casagrande Apparatus

The apparatus consists of a mechanical device consisting of a cup mounted on an edge pivot. The cup rests on a hard rubber base. A mechanism enables the cup to be lifted by 10mm and dropped on the base. The soil is put in cup and levelled off horizontally. The soil is divided by a standard grooving tool through the pivot of the cup. The two halves of the soil flow together as the cup is repeatedly dropped onto the base. The number of drops at the rate of two revolutions per second required to close the groove over a distance of 13mm is recorded. The test is repeated over four times and the water content is determined each time.

The water content is plotted against log of blows. The best straight line fitting the points is drawn. The moisture content at twenty five (25) blow is the Liquid Limit of the soil. Fig. 7-1 and 7-2 show Casagrande Equipment.



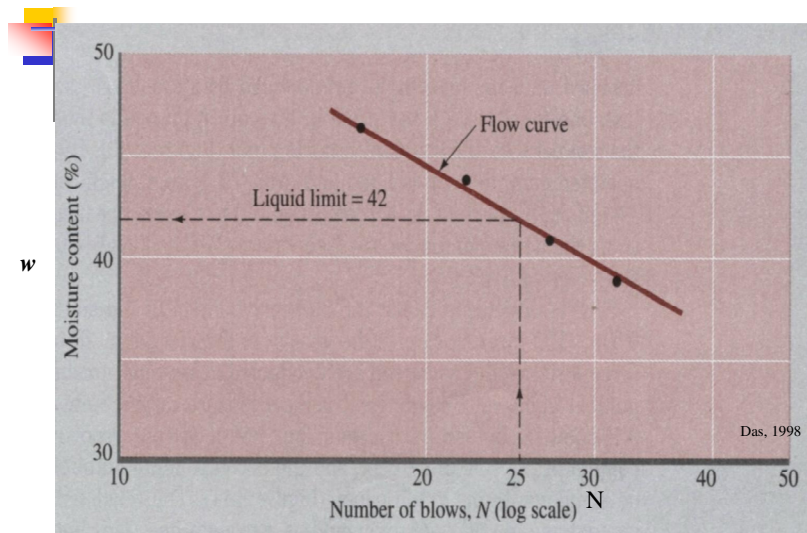
**Fig. 7-1:** Schematic representation of the Casagrande Apparatus



**Fig. 7-2:** A photograph of the Casagrande Apparatus

It is convenient to increase the water content in successive steps and obtain counts near about 40, 30, 20 and 10.

The resulting curve drawn as the best fitting straight line is called the “**flow curve**” (Fig. 7-3).



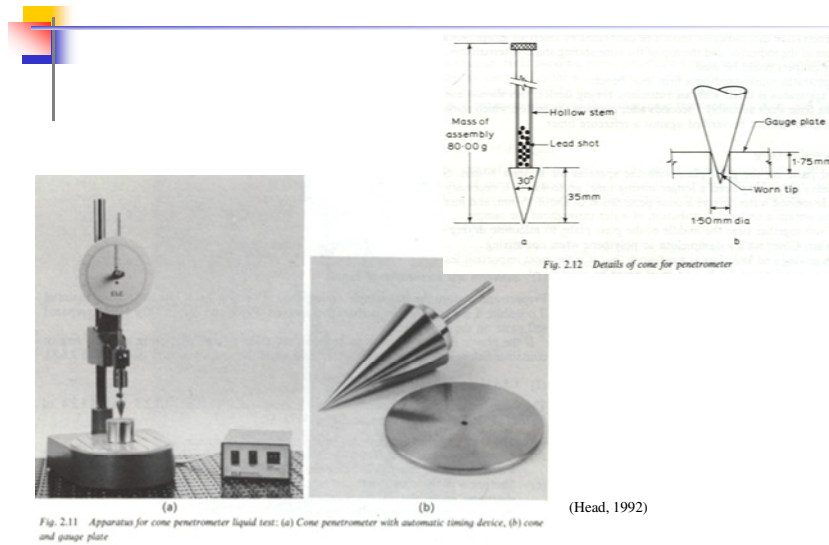
**Fig. 7-3:** Flow Curve



### 7.1.2 Cone Penetrometer Apparatus

The Penetrometer is fitted with a stainless steel cone having a smooth, polished surface. The cone is approximately 35 mm long and has an angle of  $30^\circ \pm 1^\circ$ . The mass of the cone together with its sliding shaft is 80g.

A metal cup, about 55mm in diameter and 40mm deep is used to contain the test sample. Fig. 7-4 shows the Cone Penetrometer Apparatus Schematic.



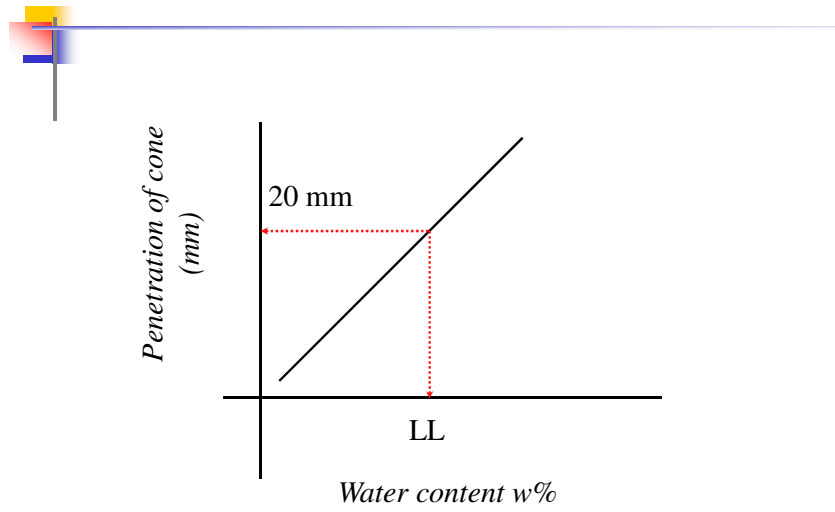
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**Fig. 7-4:** *The Cone Penetrometer Apparatus*

An air dry soil sample (about 250g) passing the 425 micron sieve is mixed with distilled water. The soil paste is filled in the metal cup and the surface struck off level. The cone is lowered to just touch the surface of soil and then released for a period of 5 seconds. The penetration is recorded. The cone is lifted and cleaned. The test is repeated over four different moisture contents. The moisture contents used in the tests should be such that the penetrations obtained lie within a range of 15 to 35 mm.

To obtain the liquid limit, cone penetration is plotted against moisture content both on normal scales to give the best fitting straight line. The moisture content corresponding to a cone penetration of 20mm is taken as the liquid limit of the soil (Fig. 7-5), which for all practical purposes is the same as the LL determined by the Casagrande apparatus.

The Cone Penetrometer method gives a more consistent estimate of the LL than the Casagrande apparatus, with greater repeatability and less operator susceptibility.

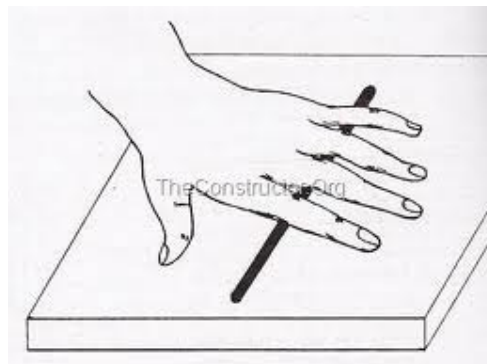


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**Fig. 7-5:** Cone penetration against moisture content

**7.2 DETERMINATION OF PLASTIC LIMIT**

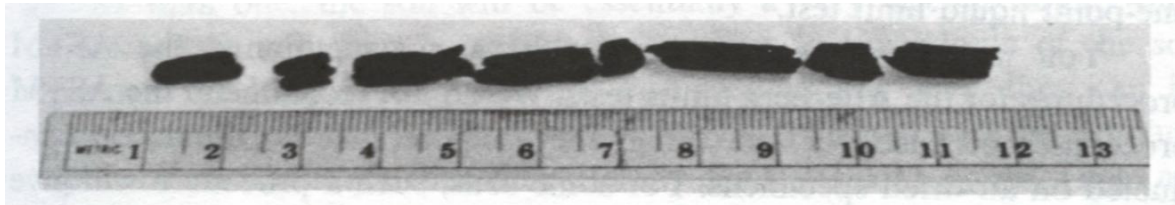
The sample of soil is mixed with distilled water until it is sufficiently plastic to be rolled into a ball between palms of hands. A small portion of the ball is then rolled on a smooth plate into a thread of 3mm diameter, and the thread is looked for signs of cracking. If no cracks are seen, the thread is picked up and again rolled into a ball between palms. The water content is reduced by the heat of the fingers. The ball is then rolled on smooth plate into a thread of 3mm diameter. The steps are repeated until a 3mm diameter thread first shows signs of cracking. A portion of the thread is taken for water content determination which gives the plastic limit.



**Fig. 7-6:** Rolling on a smooth plate



**Fig. 7-7:** Rolling on between palms of hands



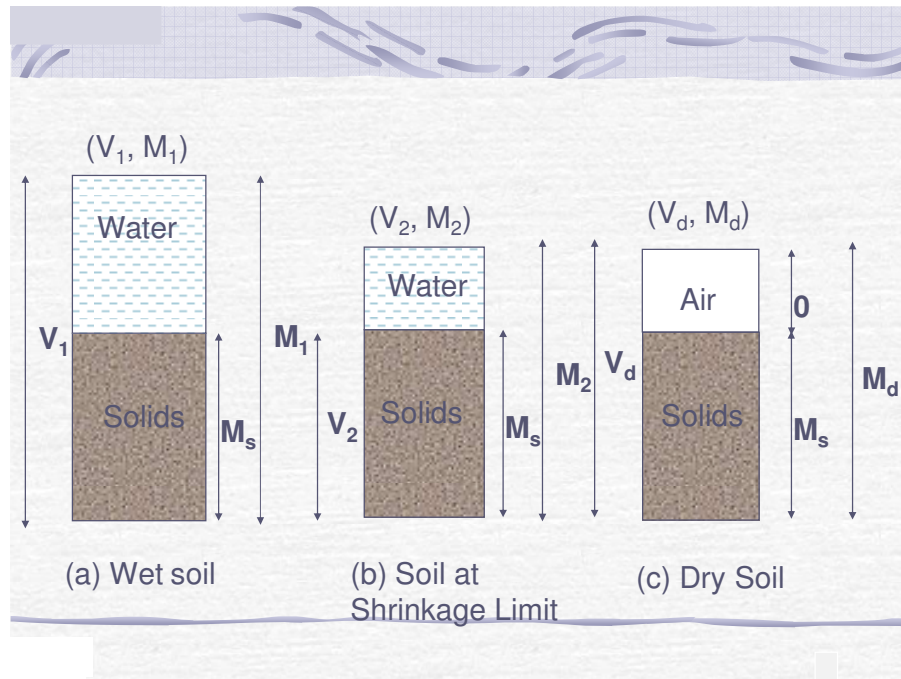
**Fig. 7-8:** Soil thread 3mm diameter just crumbles

### 7.3 DETERMINATION OF SHRINKAGE LIMIT

It is defined as the maximum water content at which there is no reduction in volume of soil mass accompanying reduction in water content.

A saturated sample of soil is placed in a container of known volume  $V_1$ . The weight of the soil is determined in stage (a) of the experiment. The soil is initially dried in the air and then in an oven to ensure that no cracks are formed. Further drying of the soil at some water content (b) will not result in the reduction of the volume of the soil.

At stage (c) of the drying process the soil is completely dry but the volume in the stage is the same as at stage (b). The weight of the soil is measured at stage (c) and its volume is measured by immersing the dry soil in mercury.



**Fig. 7-9: Shrinkage Limit Phase Diagram**

$$\text{Shrinkage Limit} = \frac{\text{Mass of Water in (b)}}{\text{Mass of Soil Solids}}$$

(a) Fully saturated sample of soil of volume  $V_1$  and (b) mass  $M_1$  which on reducing water content attains volume  $V_2$  and Mass  $M_2$ .

If it is assumed that any further reduction in water does not cause a decrease in the total volume of the sample, the water content is the shrinkage limit.

$$\text{Mass of water in (a)} = (M_1 - M_d)$$

$$\text{Loss in water from (a) to (b)} = (V_1 - V_2)\gamma_w$$

$$\text{Mass of water (b)} = \frac{(M_1 - M_d) - (V_1 - V_2)\gamma_w}{M_d}$$

## 7.4 ATTERBERG INDICES

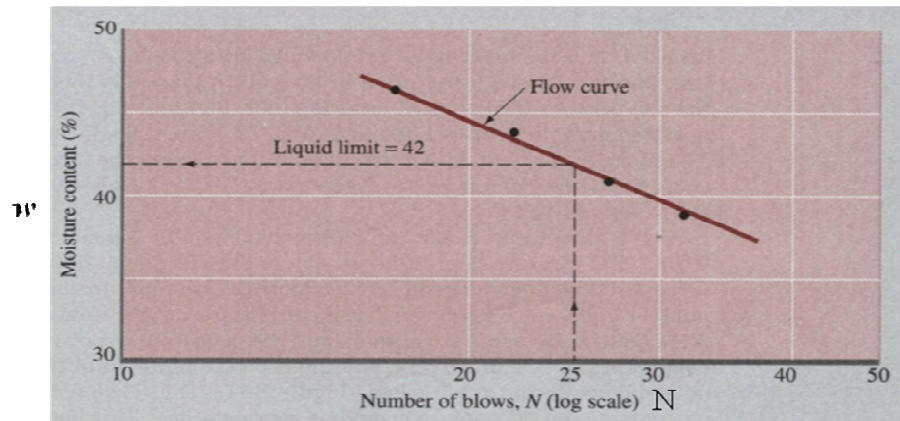
### 7.4.1 Plasticity Index

Is defined as Liquid Limit minus Plastic Limit

$$PI = LL - PL$$

### 7.4.2 Flow Index, $I_F$

Is the slope of flow curve obtained by plotting water content as ordinate or natural scale against number of blows as abscissa on logarithmic scale (Fig. 7-10).



**Fig. 7-10:** Flow curve for determination of Liquid Limit

$$I_F = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$$

Where;

$w_1$  = water content corresponding to number of blows  $N_1$ , and

$w_2$  = water content corresponding to number of blows  $N_2$ .

### 7.4.3 Toughness Index

Defined as the ratio of Plasticity Index to Flow Index.

$$I_T = \frac{I_p}{I_F}$$

### 7.4.4 Consistency Index

Defined as the ratio of Liquid Limit minus natural Water Content to the Plasticity Index.

$$I_C = \frac{LL - w}{I_p}$$

### 7.4.5 Liquidity Index

Defined as the ratio of natural Water Content minus Plastic Limit to Plasticity Index.

$$I_L = \frac{w - PL}{I_p}$$

### 7.5 USE OF CONSISTENCY LIMITS

The consistency limits and related indices are very useful for soil identification and classification. The limits are often in specification for soil compaction and in semi-empirical methods of design. The liquid limit and plastic limit depend on both the type and amount of clay.

Plasticity Index	Plasticity
0	Non-Plastic
<7	Low Plastic
7 - 17	Medium Plastic
>17	Highly Plastic

The following table shows some typical values of Atterberg Limits

**Table 10.1 Atterberg Limit Values for the Clay Minerals.**

Mineral <sup>a</sup>	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit
Montmorillonite	100–900	50–100	8.5–15
Nontronite	37–72	19–27	
Illite	60–120	35–60	15–17
Kaolinite	30–110	25–40	25–29
Hydrated Halloysite	50–70	47–60	
Dehydrated Halloysite	35–55	30–45	
Attapulgite	160–230	100–120	
Chlorite	44–47	36–40	
Allophane (undried)	200–250	130–140	

## 7.6 WATER CONTENT

The water content may be determined by a number of methods.

### 7.6.1 Oven-drying method

This is the usual standard laboratory method in which the soil is dried in an oven at 105°C to 110°C. This is the most accurate method. The temperature range of 105 to 110°C has been selected because at higher heating temperatures there may be danger of breaking down of crystalline structure of clay particles resulting in the loss of chemically bound structural water. Also at higher temperatures organic soils tend to decompose and get oxidated giving a reduced weight of the dried soil. Drying time may extend up to 24 hours or more depending upon the amount and type of soil. The water content is always reported on the basis of the dry weight of the soil sample.

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

Soils containing gypsum or significant amount of organic matter are dried at a lower temperature (60 - 80°C) for a longer time. Though very accurate, this method is time consuming.

### 7.6.2 Sand Bath method

It is a field method requiring only about one hour's drying period. A sand bath (a basin containing about 50mm thick sand layer) is heated over a gas burner or stove. Wet soil sample kept in a container, is placed on the sand bath and heated until dry. The sand is stirred with a palette knife (or a rod) during heating to assist drying. Observations are similar to those of the oven drying method and the moisture content is calculated using the formulae;

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

If  $M_1$  = Mass of container

$M_2$  = Mass of wet soil + container

$M_3$  = Mass of dry soil + container

$$w = \frac{M_w}{M_s} = \frac{M_2 - M_3}{M_3 - M_1} \times 100\%$$

However, the method suffers from the disadvantages of higher heating. The method should not be used for soils containing large proportions of gypsum, calcareous or organic matter.

**7.6.3 Other methods**

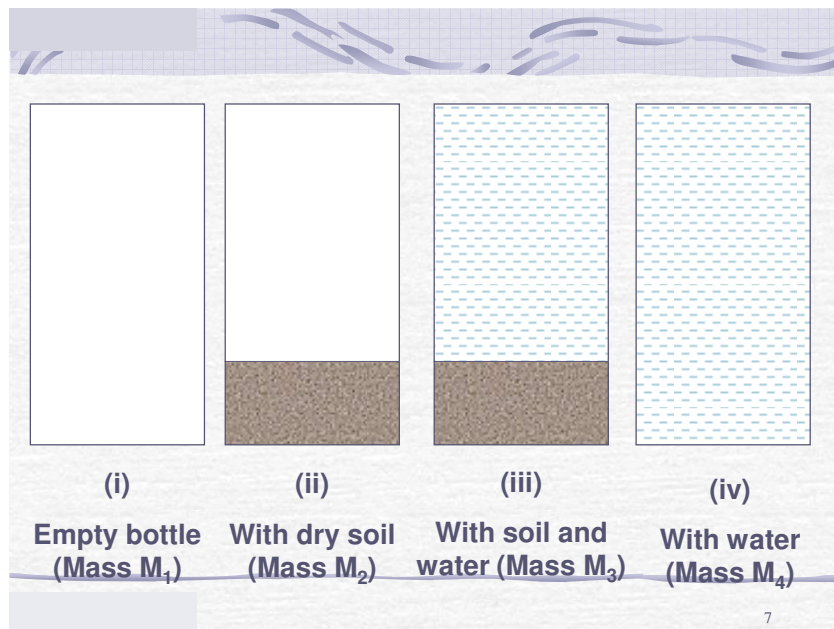
Infrared lamp and torsion balance method, calcium carbide method and nuclear method can be used for rapid determination of water content.

**7.7 SPECIFIC GRAVITY**

A knowledge of the specific gravity of soils is required for the determination of voids ratio, degree of saturation, and in the sedimentation and consolidation tests. The individual mineral particles constituting a soil have different specific gravities. The specific gravity  $G$  of a soil refers to the average value for all the particles. The specific gravity can be determined either by the use of a 50 - 100ml density bottle, a 500ml flask or a pycnometer. The density bottle method is the usual laboratory method.

The mass  $M_1$  of the clean, dry bottle is found. Suitable quantity of oven-dried soil sample, cooled in a desiccator is put in the bottle and the mass  $M_2$  of the bottle with soil is found. Distilled water is then added to the soil inside bottle until the bottle is full, care being taken to see that entrapped air is fully expelled (either by applying vacuum or by gentle heating and shaking). The mass  $M_3$  of the bottle with soil and water is found. The bottle is then emptied of its contents, cleaned and filled with distilled water only. The outer surface of the bottle is wiped dry and the mass  $M_4$  of the bottle with water is found. Fig. 3-11 shows the phase diagram for determination of Specific Gravity. Specific gravity of soil is computed as;

$$G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}$$



**Fig. 7-11:** Phase diagram for determination of Specific Gravity



$$\text{Mass of soil solids} = M_2 - M_1$$

$$\text{Mass of water in (iii)} = M_3 - M_2$$

$$\text{Mass of water in (iv)} = M_4 - M_1$$

Hence mass of water equivalent in volume of that of soil solids

$$= \text{Mass of water in (iv)} - \text{Mass of water in (iii)}$$

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

$$G = \frac{\text{Mass of soil solids}}{\text{Mass of equivalent volume of water}}$$

$$= \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$$

$$= \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)} = \frac{M_d}{M_d - (M_3 - M_4)}$$

### Example 1:

An oven-dried sample having mass of 195g was put inside a pycnometer which was then completely filled with distilled water. The mass of pycnometer with soil and water was found to be 1584g. The mass of pycnometer filled with water alone was 1465g. Calculate the Specific Gravity of soil solids.

### **Solution:**

Mass of solids,  $M_d = 195\text{g}$

Mass of pycnometer + soil + water = 1584g  $(M_3)$

Mass of pycnometer + water = 1465g  $(M_4)$

$$G = \frac{M_d}{M_d - (M_3 - M_4)} = \frac{195}{195 - (1584 - 1465)} = 2.56$$

**Example 2:**

In above example, while finding mass of pycnometer with soil and water,  $2\text{cm}^3$  of air got entrapped. Will the computed value of  $G$  be higher or lower than the correct value and what will be the percentage error?

**Solution:**

$$G = \frac{195}{195 - (1584 - 1465)}$$

If some air got entrapped while finding  $M_3$ , then the value of  $M_3$  will be less than that when water replaces the entrapped air. Since  $M_3$  occurs with negative sign in the denominator, the denominator will increase and hence computed value of  $G$  will be than the correct value.

Corrected value of  $G$  is obtained as;

$$G = \frac{195}{195 - (1586 - 1465)} = 2.63$$

$$\text{Percentage error} = \frac{2.63 - 2.57}{2.63} \times 100\% = 2.28\%$$

## 7.8 DETERMINATION OF FIELD DENSITY

The *in-situ* bulk density of a soil deposit in the field is commonly determined by;

- (i) Core-cutter method in the case of cohesive soils and
- (ii) Sand replacement method in the case of cohesionless soils

### 7.8.1 Core Cutter method

A core cutter is a steel cylinder open at both ends with one end sharpened to form the cutting edge. The usual dimensions are 10cm internal diameter and height about 12 to 15 cm. The internal diameter, height and mass of core-cutter are noted. The place where the field density is to be determined is cleared of shrubs, if any, levelled and the core-cutter is placed vertically on the ground surface. A steel ring about 2.5cm height (Steel Dolly) is placed on top of the core cutter and is gently driven into the ground by blows of a rammer, until the top of the steel ring is nearly flush with the ground surface. Sufficient soil is excavated from around the core-cutter to enable a person to put his hands and lift the core-cutter with soil inside off the ground. The core cutter with soil inside is brought to the laboratory, the ends are trimmed, levelled and weighed. The soil is removed from the core-cutter and the samples are taken from top, middle and bottom positions for water content determination. The average of the three determinations gives the *in-situ* water content. Using the experimental data, computations are as follows;

1. Internal volume of core cutter =  $V$  in cc
2. Weight of core cutter =  $W_1$  in g
3. Weight of core cutter + soil =  $W_2$  in g
4. Weight of soil =  $W_2 - W_1$  in g

5. Bulk density of soil  $\gamma$ ,

$$\rho = \frac{W_2 - W_1}{V} \text{ g / cc}$$

6. Moisture content = w in %

7. Dry density,  $\rho_d = \frac{100\rho}{100 + w} \text{ g / cc}$

**Derivation;**

Relation between  $\rho$ ,  $\rho_d$  and w

We have  $w = \frac{M_w}{M_s}$

Adding 1 to both sides;

$$1 + w = \frac{M_w}{M_s} + 1 = \frac{M_w + M_s}{M_s} = \frac{M}{M_s}$$

$$\therefore M_s = \frac{M}{1 + w}$$

Dividing both sides by V,

$$\frac{M_s}{V} = \frac{M / V}{1 + w}$$

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

S/No.	Description	Determination No.		
		I	II	III
1	Internal diameter of corecutter (mm)	100	100	100
2	Internal height corecutter in mm	129.75	129.75	129.75
3	Volume of cutter (V) in cc	1019.05	1019.05	1019.05
4	Weight of core cutter ( $W_1$ ) in g	1130	1130	1130
5	Weight of core cutter + soil ( $W_2$ ) in g	3120	3122	3119
6	Weight of soil ( $W_2 - W_1$ ) in g	1990	1992	1989
7	Bulk density of soil, $\rho = \frac{W_2 - W_1}{V} \text{ g / cm}^2$	1.95	1.95	1.95
8	Moisture content w in %	17.75	17.76	17.73
9	Dry density of soil, $\rho_d = \frac{\rho}{1 + w}$	1.66	1.66	1.66
	Average value	1.66 g/cm <sup>3</sup>		

## 7.8.2 Sand replacement method

### ***Equipments:***

- (i) Small pouring cylinder: suitable for fine and medium grained soils.
- (ii) Metal tray to excavate the hole with suitable shape and size.
- (iii) Calibration container of the small pouring cylinder (size 100 x 150mm).
- (iv) Large pouring cylinder: suitable for fine, medium and coarse grained soils.
- (v) Calibration container
- (iii) Medium pouring cylinder: suitable for fine, medium and coarse grained soils.
- (iv) Tools for levelling and excavating.
- (v) Containers.
- (vi) Sand.
- (vii) Balance

### ***Procedure:***

The unit's base plate is laid on the compacted surface and material is excavated through the hole in the plate to a depth of about 150 mm. This wet material is weighed, dried in an oven and weighed again to determine the moisture content.

The volume of the hole is measured by filling it with dry, free-flowing sand from a special sand-cone cylinder. Since the density of the sand is known, the volume of the hole is calculated. The density (wet unit weight) of the compacted sample is found by dividing the weight of the material by the volume of the hole. Dry unit weight is found by using the formulae;

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

## 7.9 PARTICLE-SIZE DISTRIBUTION

The soil grading or the distribution of particle size is quantitatively determined by performing the particle-size analysis, also called mechanical analysis, which is carried out in two parts: sieve analysis and sedimentation analysis. The distribution of gravel and sand particles is determined by sieve analysis and that of silt and clay by sedimentation analysis. Depending on the type of soil and the extent of particle-size distribution required, mechanical analysis may involve both sieving and sedimentation or it may be restricted to either of them. For gravel and sand, sieve analysis alone will suffice, but if silt and clay are present, a combined sieve and sedimentation analysis may be required. If soil is predominantly silty and or clayey, sedimentation alone will do.

### 7.9.1 Sieve analysis

Soil sample to be analysed is first either air-dried or oven-dried. Soil aggregates are then broken by pulverisation with a wooden mallet or in a mortar with a rubber pestle. If the sample is more than required, it is reduced in size by means of a riffle box. A representative sample is then taken for sieve analysis.

The objective of this test is to determine the relative proportions of different granular sizes as they are passing through certain sieve sizes. Thus, the percentage of sand, gravel, silt and clay can be obtained.

Sieve analysis is widely used in classification of soils. Data obtained from particle-size distribution curves is used in the design of filters for earth dams and to determine suitability of soil for road, highway, construction, embankment fill of dam, airport runway etc.

Sieve analysis of soil test is in accordance to ASTM D-422 (American Society for Testing and Materials) or BS 1377: Part 2 1990 (British Standards) as both are the most widely used technical standards in construction. The dry sieving of soil is the simplest and cheapest method among others.

#### ***Apparatus;***

- i. Stack of Sieve aperture sizes (including cover and pan) (Fig. 7-12).
- ii. Electronic balance.
- iii. Rubber pestle, mortar (for crushing the soil if lumped) and brush.
- iv. Mechanical sieve vibrator (shaker).
- v. Oven dry (thermostatically controlled temperature).



**Fig. 7-12:** Sieve Analysis Apparatus: (A) Sieve aperture sizes, (B) Dry oven, (C) Sieve shaker, (D) Mortar & Tray, (E) Rubber pestle, (F) Balance

**Procedure;**

- i. Oven-dry sample.
- ii. Crush the oven-dried sample using rubber pestle and mortar.
- iii. Determine mass of sample and label as  $W_{total}$  in (g).
- iv. Then prepare a stack of sieve aperture sizes with larger opening sizes at the top and down to the last one with smaller opening sizes.
- v. Pour the soil slowly into the stack of sieves from the top and place the cover, put the stack onto the sieve shaker (vibrator), tighten the clamps, adjust the time with 5 to 10 minutes and turn it on.
- vi. When time is out, take out and measure the mass of retained soil inside, from the top sieve until the pan.

The results are presented in a graph of percent passing versus the sieve size (Fig. 7-13). On the graph the sieve size is logarithmic. To find the percent of aggregate passing through each sieve, first find the percentage retained in each sieve.

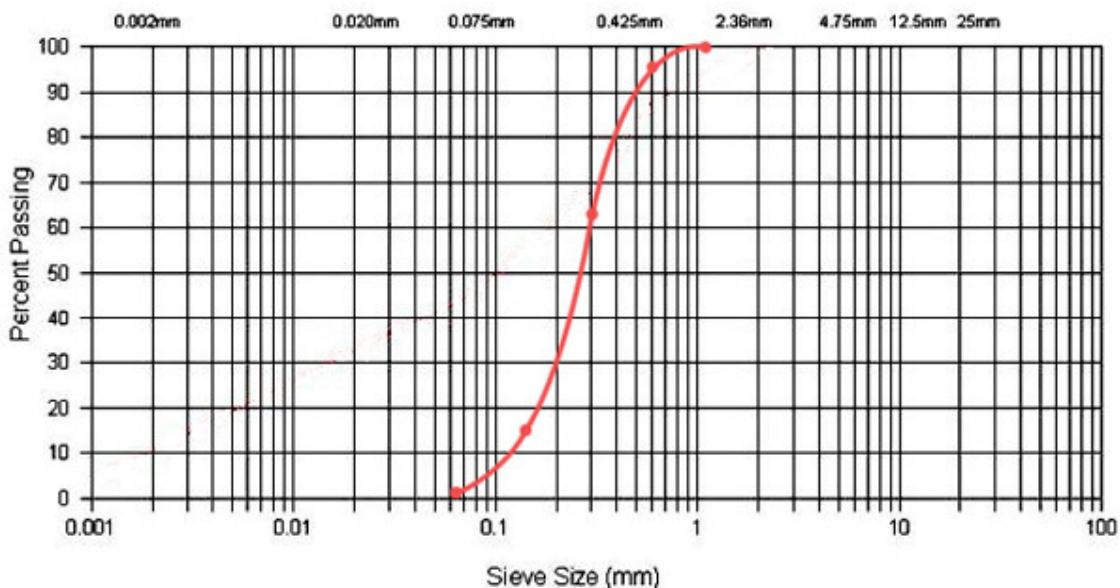
$$\% \text{ Retained} = \frac{W_{sieve}}{W_{total}} \times 100\%$$

Find cumulative percent of aggregate retained in each sieve.

Then % Cumulative Passing = 100% - % Cumulative Retained.

**7.9.2 Particle-Size Distribution Curves**

Consider Fig. 7.13 showing a typical particle-size distribution curve.



**Fig. 7-13:** Particle size distribution curve (Graph for BS 1377: Part 2 1990)

The position and general shape and slope of a curve indicate type and grading of the soil.

- A curve lying higher up or to the left represents relatively finer material.
- The gradation of soil is said to be either well graded or poorly graded.
- A soil is well graded when there is a good representation of all the particle sizes from the largest to the smallest.
- A soil is poorly graded if there is an excess or a deficiency of certain particle sizes within the limits of the minimum and maximum sizes, or if most of the particles are of about the same sizes (uniformly graded).

We can compute the Coefficient of Uniformity  $C_u$  and Coefficient of Curvature  $C_c$  by obtaining  $D_{10}$ ,  $D_{30}$  and  $D_{60}$  values from the curves.

- $D_{10}$  – 10% passing point on the curve.
- $D_{30}$  – 30% passing point on the curve.
- $D_{60}$  – 60% passing point on the curve.

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

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$$

To be well graded,  $C_c$  must lie within 1 and 3 and in addition,  $C_u$  must be greater than 4 for gravels and greater than 6 for sands. If all particles are of the same size,  $C_u$  is unity.

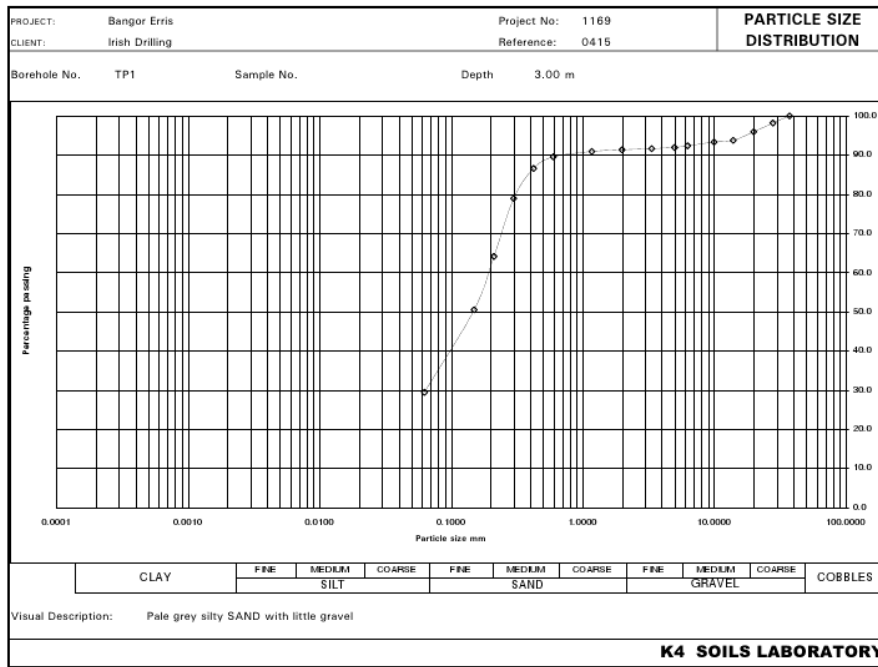
A low value of  $C_u$  indicates a uniform soil and a high value a well graded soil.

### 7.9.3 Sieve analysis test result

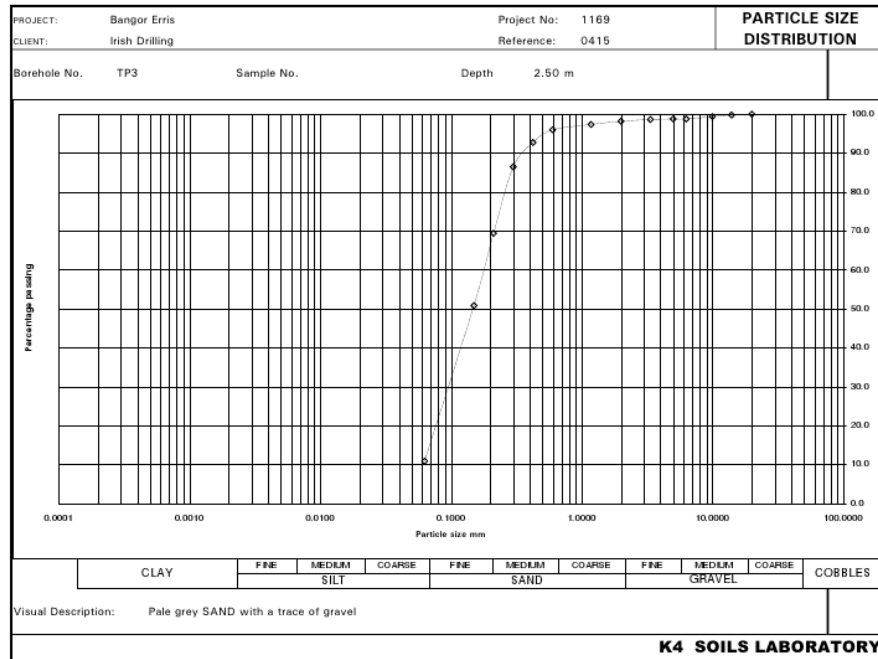
Sieve Size	Weight retained	% Retained	% Cum. Retained	% Passing
6	126	12.6	12.6	87.4
2	86	8.6	21.2	78.8
0.6	194	19.4	40.6	59.4
0.2	202	20.2	60.8	39.2
0.075	124	12.4	73.2	26.8
Base	268	26.8	100	0
<b>Total</b>	<b>1000</b>			

### 7.9.4 Example Particle Size Distribution Curves

(a) Silty SAND with little gravel

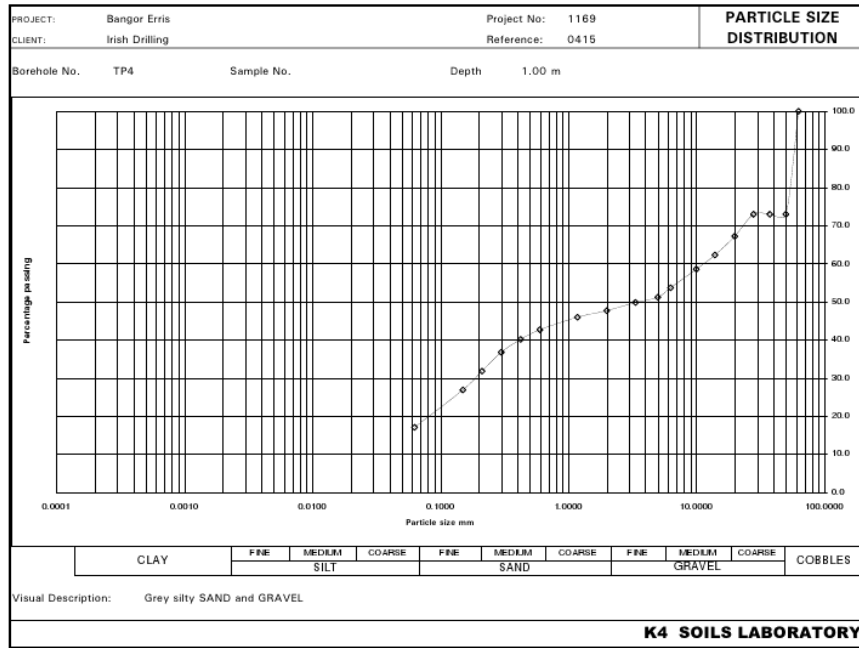


(b) Silty SAND with a trace of gravel

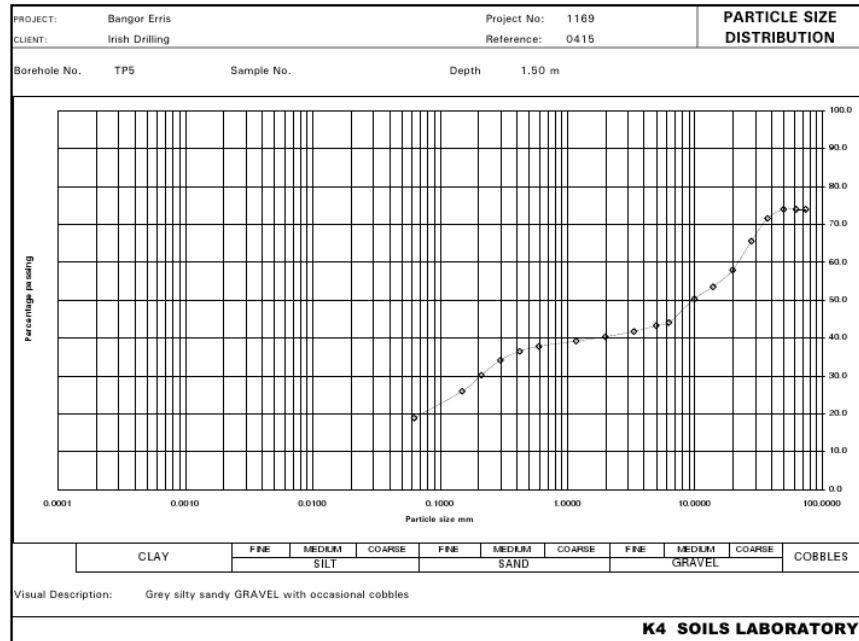




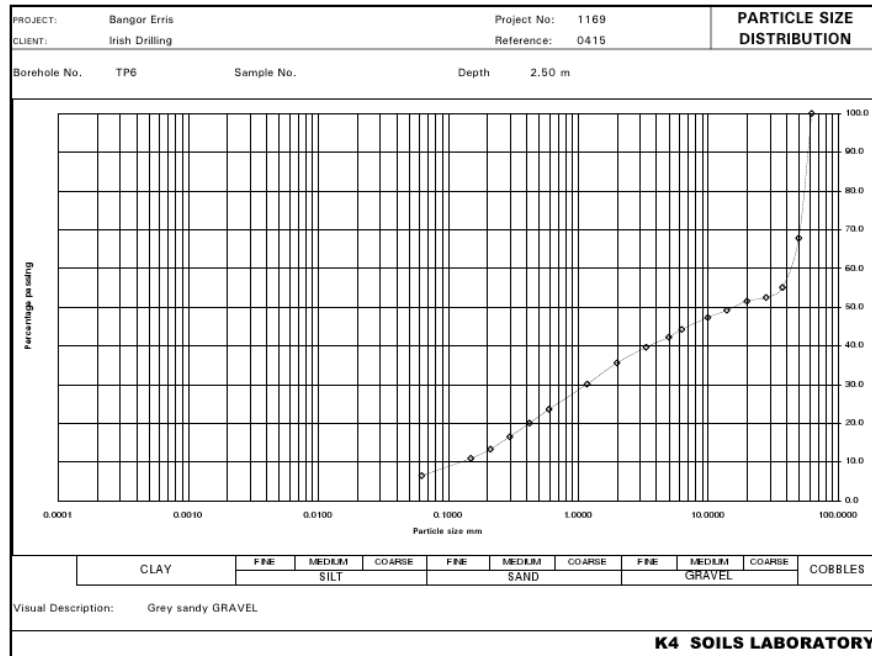
**(c) Silty SAND and GRAVEL**



**(d) Silty sandy GRAVEL with occasional cobbles**



(e) sandy GRAVEL



**7.9.5 Limitations of Sieve Analysis**

For materials finer than 150 μm, dry sieving can be significantly less accurate. This is because the mechanical energy required to make particles pass through an opening and the surface attraction effects between the particles themselves and between particles and the screen increase as the particle sizes decreases. Wet sieving analysis can be utilized where the material analyzed is not affected by the liquid – except to disperse it. Suspending the particles in a suitable liquid transports fine material through the sieve much more efficiently than shaking the dry material.

Sieve analysis assumes that all particles will be round (spherical) or nearly so and will pass through the square openings when the particle diameter is less than the size of the square opening in the screen. For elongated and flat particles, a sieve analysis will not yield reliable mass-based results, as the particle size reported will assume that the particles size reported will assume that the particles are spherical, where in fact an elongated particle might pass through the screen end-on, but would be prevented from doing so if it presented itself side-on.

## 8 SOIL DESCRIPTION AND CLASSIFICATION

### 8.1 INTRODUCTION

Soil description includes the details of both material and mass characteristics. Soil classification on the other hand is the allocation of a soil to a limited number of groups on the basis of material characteristics only, viz: - particle size distribution and plasticity. Soil classification is therefore independent of the *in situ* mass condition of the soil mass. It is unlikely that any two soils will have the same description, while they could have the same classification.

In some engineering processes the description and classification are useful. Such engineering processes include foundations and seepage. However in other cases the classification is by the far the most important. Such processes include embankment construction where classification of material is key factor in choice and construction process. It is essential that a standard language be used in description and classification of soils. A comprehensive description should include the characteristics of both soil material and *in situ* soil mass.

**Material characteristics** can be determined from disturbed samples of soil, i.e. samples having the same particle size distribution as the *in situ* soil but whose *in situ* structure has been altered. The principal material characteristics are particle size distribution (or grading) and plasticity. Secondary material characteristics are colour of the soil, shape, texture and composition of the particles.

**Mass characteristics** are best determined in the field but can also be determined in the laboratory when undisturbed samples are available. Mass characteristics include firmness, strength, details of any bedding, discontinuities and weathering. The arrangement of minor geological details referred to as macro-fabric should be carefully described as this can influence the engineering behaviour of *in situ* soil considerably. Such macro-fabric features are thin layers of fine sand and silt in a clay strata, silt filled fissures in clay, small lenses of clay in sand, organic intrusions and root holes etc.

### 8.2 SOIL DESCRIPTION

A detailed description of the method of describing soils is contained in BS 5930. The basic soils are boulders, cobbles, gravels, sand, silt and clay. Often soils appear in mixtures and are referred to composite types.

In accordance to BS 5930, a soil is of basic type sand or gravel (coarse soil), if after removal of boulders and cobbles, over 65% of the material is in the sand and gravel range. A soil is of basic type silt or clay (fine grained soil) when over 35% of the soil is in the silt and clay range. Composite soils are named as described in Table 8-1. Mixtures containing over 50% boulders and cobbles are referred to as very coarse soils. The descriptions may be of the form COBBLES with finer material or gravelly SAND with occasional cobbly BOULDERS.

The firmness or strength of the soil in the field can be assessed by means of tests shown on Table 8-2.

**Table 8.1: Soil description terminology**

Soil group	Soil description	Indicative particle size distribution
COARSE SOILS	Slightly sandy GRAVEL	Up to 5% sand
	Sandy GRAVEL	5-20% sand
	Very sandy GRAVEL	Over 20% sand
	GRAVEL/SAND	About equal proportions
	Very gravelly SAND	Over 20% gravel
	Gravelly SAND	5-20% gravel
	Slightly gravelly SAND	Up to 5% gravel
	Slightly silty SAND (or GRAVEL)	Up to 5% silt
	Silty SAND (or GRAVEL)	5-15% silt
	Very silty SAND (or GRAVEL)	15-35% silt
	Slightly clayey SAND (or GRAVEL)	Up to 5% clay
	Clayey SAND (or GRAVEL)	5-15% Clay
	Very clayey SAND (or GRAVEL)	15-35% Clay
FINE SOILS	Sandy SILT (or CLAY)	35-65% Sand
	Gravelly SILT (or CLAY)	35-65% Gravel

**Table 8.2: Field identification tests**

Soil type	Term	Field test
Sands and gravels	Loose	Can be excavated by spade. A 50 mm wooden peg can be easily driven
	Dense	Requires a pick to excavate. 50 mm wooden peg difficult to drive
	Slightly cemented	Visual examination. Pick removes lumps
Silt	Soft or loose	Easily molded or crushed in the fingers
	Firm or dense	Can be molded or crushed by strong pressure in fingers
Clays	Very soft	Exudes between fingers when squeezed in the hand
	Soft	Molded by light finger pressure
	Firm	Can be molded by strong finger pressure
	Stiff	Cannot be molded by fingers
	Very stiff	Cannot be indented by thumb nails
Organic & Peat	Firm	Fibers already compressed together
	Spongy plastic	Very compressible and open structure. Can be molded by fingers and smears fingers

**8.1 SOIL CLASSIFICATION SYSTEMS**

**8.1.1 Purpose of Soil classification**

A soil classification system is an arrangement of different soils into soil groups having the similar properties. The purpose of soil classification is to make possible the estimation of soil properties by association with soils of the same class whose properties are known. Thus with consistent soil classification it is possible to;

- 1 Use data of others in predicting foundation performance.
- 2 Build the geotechnical engineers data base for application of design.
- 3 Maintain a permanent record which can be easily understood by others should problems develop later.

There are several methods of soil classification. Two such methods in use by engineers in Kenya are described below

**(A) BRITISH SOIL CLASSIFICATION**

The British soil classification system is based on the particle size distribution and the plasticity as plotted on a plasticity chart. The plasticity chart is a plot of the soil PI against LL.

In the British soil classification any cobbles and boulders retained on 63mm BS Sieve Size are removed from the soil before the classification. The percentage of this very coarse portion is determined and mentioned in the report. The soil groups in the classification are noted by the group symbols composed of main and qualifying descriptive letters having the meaning shown on Table 8-3.

**Table 8-3: Soils descriptive terms**

Main terms	Qualifying terms		
GRAVEL	G	Well graded	W
SAND	S	Poorly graded	P
		Uniform	Pu
		Gap graded	Pg
FINE SOIL, FINES SILT (M SOIL) CLAY	F	Of low plasticity (LL<35)	L
	M	Of intermediate plasticity (LL 35-50)	I
	C	Of high plasticity (LL 50-70)	H
		Of very high plasticity (LL 70-90)	V
		Of extremely high plasticity	E
		Of upper plasticity range (LL<35)	U
	Organic (may be a suffix to any group)	O	
Peat	Pt		

The letter describing the dominant group is placed first in the group symbol. When the group has significant organic content the suffix O is added as the last letter. The above is illustrated below in the description of sand, clay and silt samples.

SW – well graded SAND

SCL – very clayey SAND (the clay in the sample is of low plasticity)

CIS – sandy CLAY of intermediate plasticity

MHSO – organic sandy SILT of high plasticity

The fine grained soils are represented by a point on the plasticity chart. The plasticity chart is divided into a low plasticity (LL<35) and upper (U) plasticity zones (LL>35). The upper plasticity zones are subdivided into Intermediate plasticity (LL 35-50), High plasticity (LL 50-70), very high plasticity (LL 70-90) and extremely high plasticity (LL >90). Fig. 8-1 shows the plasticity chart as used in the British soil classification.

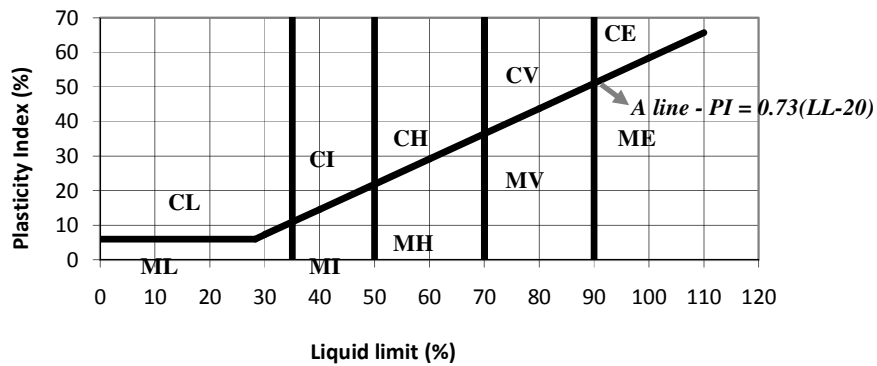


Fig. 8-1: Plasticity Chart (BS 5930: 1981)

The recommended standard for soil classification is the **British Soil Classification System** (Table 8-4), and this is detailed in BS 5930 Site Investigation.

**Table 8-4: British soil classification system**

Soil Group	Group Symbol	Subgroup Symbol		Recommended name
<b>Coarse Soils</b>			<b>Fines %</b>	
GRAVEL	G	GW	0-5	Well-graded GRAVEL
		GPu/ GPg	0-5	Uniform/poorly-graded GRAVEL
	G-F	GWM/GWC	5-15	Well-graded silty/clayey GRAVEL
		GPM/GPC	5-15	Poorly graded silty/clayey GRAVEL
	GF	GML	15-35	Very silty GRAVEL
		GCL	15-35	Very clayey GRAVEL
SAND	S	SW	0-5	Well-Graded SAND
		SPu/ GPg	0-5	Uniform / poorly-graded SAND
	S-F	SWM/SWC	5-15	Well-graded silty/clayey SAND
		SPM/SPC	5-15	Poorly graded silty/clayey SAND
	SF	SML	15-35	Very silty SAND
		SCL	15-35	Very clayey SAND
<b>Fine soils</b>		<b>&gt;35% Fines</b>	<b>Liquid Limit %</b>	
SILT	M	MG		Gravelly SILT
		MS		Sand SILT
CLAY	C	CG		Gravelly SILT
		CS		Sandy SILT
		CL	<35	CLAY of low plasticity
		CI	35 – 50	CLAY of intermediate plasticity
		CH	50 – 70	CLAY of high plasticity
		CV	70 - 90	CLAY of very high plasticity
		CE	>90	CLAY of extremely high plasticity
Organic soils	O			[Add letter 'O' to group symbol when organic matter is present]
Peat	Pt			[Soil predominantly fibrous and organic]

## (B) UNIFIED SOIL CLASSIFICATION

Like the British soil classification system, the Unified Soil Classification System is based on the particle size distribution and the plasticity as plotted on a plasticity chart. The plasticity chart is a plot of the soil PI against LL. Table 8-5 shows the unified soil classification. The main difference with the British soil classification is that the detail in the unified classification is reduced. In effect the classification is simplified. Thus the separation of the coarse and the fine grained soils is basically determined on the 50% percentage fraction instead of the 35 and 65% used in the British classification system. In addition the division of the plasticity chart is limited to the 50% value for the lower and the higher plasticity for the purposes of classifying the fine grained soils.

The system is based on both grain size and plasticity characteristics of soils. In this system soils are broadly divided into three divisions;

1. Course-grained soils – if more than 50% by weight is retained on No. 200 ASTM sieves (American Society for Testing Materials)
2. Fine-grained soils – if more than 50% by weight passes through No. 200 ASTM sieve
3. Organic soils

The soil components are assigned group symbols as indicated below;

### *Course-grained soils:*

Gravel: G

Sand: S

### *Fine grained soils:*

Silt: M (Symbol M is derived from the Swedish word 'Mo' for Silt)

Clay: C

Organic soil: O

No. 200 sieve is of aperture size 0.074 mm

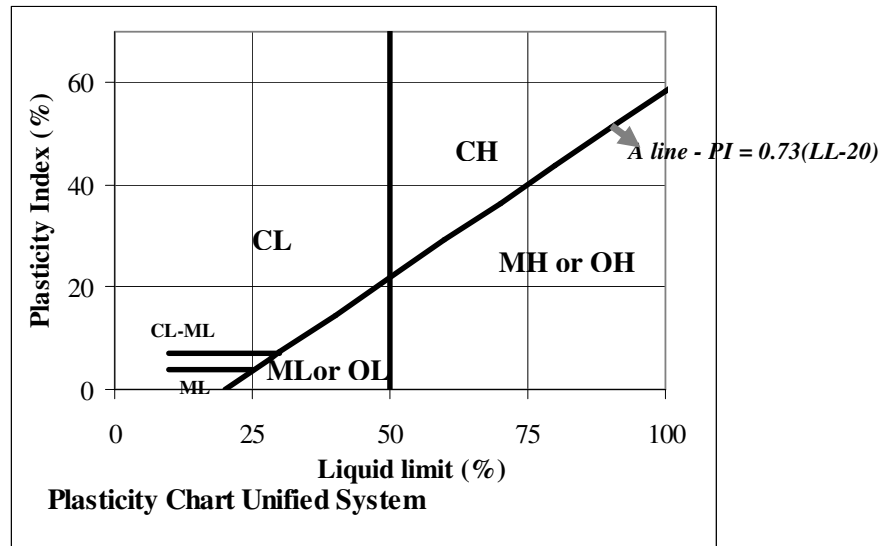
No. 4 sieve is of aperture size 4.76 mm



**Table 8-5: Unified Soil Classification System**

Major Divisions		Typical names	Group Symbol	Fines %
Coarse grained soils (more than 50% larger than 63µBS or No 200US sieve size)	Gravels (more than 50% of coarse fraction of gravel size)	Well graded gravels, sandy gravels with little or no fines	GW	0-5
		Poorly graded gravels, sandy gravels with little or no fines	GP	0-5
		Silty gravels, silty sandy gravels	GM	>12
		Clayey gravels, clayey sandy gravels	GC	>12
	Sands (more than 50% of coarse fraction of sand size)	Well graded sands, gravelly sands with little or no fines	SW	0-5
		Poorly graded sands, gravelly sands with little or no fines	SP	0-5
		Silty sands,	SM	>12
		Clayey sands	SC	>12
Fine grained soils (more than 50% smaller than 63µBS or No 200US sieve size)	Silts and clays (liquid limit less than 50%)	Inorganic silts, silty or clayey fine sands with slight plasticity	ML	
		Inorganic silts, silty or clayey fine sands of low plasticity	CL	
		Organic silts, silty or clayey fine sands of low plasticity	OL	
	Silts and clays (liquid limit greater than 50%)	Inorganic silts of high plasticity	MH	
		Inorganic clays of high plasticity	CH	
		Organic clays of high plasticity	OH	
Highly organic soils	Peat and highly organic soils	Pt		

The original Casagrande plasticity chart used for classifying fine-grained soils is given in Fig. 8-2.



**Fig. 8-2: Unified classification system plasticity chart**

### 8.1.2 Shortcomings of the classification systems

The classification systems are based on the properties of the grains and their remolded properties. They do not consider the properties of the materials *in situ*. Yet it is the intact nature of soils in the field that determine the behaviour of the soil during and after construction. The foundation engineer should therefore take recognition of the description of soil described above. It is also important to have basic understanding of the geology of the soil to have a complete description and understanding of the soil under study.

## 9 SOIL COMPACTION

### 9.1 INTRODUCTION

Soil compaction is the process of increasing the density of soil by packing the particles together with reduction in volume of air. The process does not involve removal of water. The process primarily results in the increase of soil unit weight (density). The reduction of the air content results in the reduction of pores which act as conduits of water and consequently reduce the permeability of the soil. In addition compaction reduces the liquefaction and increases the erosion resistance of the soil. The result is increased shear strength and less compressibility of the soil.

The purpose of the compaction is to produce a soil having the physical properties appropriate to the particular project. A good measure of compaction is needed in the construction of road embankments, improvement of road subgrade, subbase and base layers. Compaction of materials in dams is needed to ensure stability and water tightness of the dam walls

### 9.2 THEORY OF COMPACTION

#### 9.2.1 General

The degree of compaction is measured quantitatively by the dry density ( $\gamma_d$ ). The increase in dry density of soil is a function of the moisture content of the soil and the compaction effort. The variation of dry density and moisture content (M) is shown in Fig 9.1. Curve is usually plotted by obtaining the dry density from bulk density and water content measurements as follows;

$$\gamma_b = \frac{W}{V} = \frac{W_w + W_s}{V} = \frac{W_w / W_s + 1}{V / W_s} = \frac{(w + 1)}{1 / \gamma_d}$$

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

At low water content, the soil is stiff and difficult to compact. As the water content increases, the soil is workable facilitating compaction and reduction in air. The dry density increases. As the air is reduced and replaced by water, at certain water content the voids are occupied by water and prevent any appreciable decrease in the air voids. The result is increase in the void ratio and consequent decrease in the dry density. The curve has a peak which shows an optimum moisture content (OMC) at which the soil would need to be compacted to achieve the maximum dry density (MDD).

If it was possible to remove all the air at various water contents, then the dry density achieved at the zero air voids would be the maximum achievable for the various water contents. This curve can be drawn by the relationship of dry density and  $G_s$ ,  $\gamma_w$ , and  $S_r$  and assuming the degree of saturation  $S_r$  is equal to one.

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

$$A = \frac{e - w * G_s}{1 + e}$$

$$\gamma_d = \frac{G_s * \gamma_w (1 - A)}{1 + w G_s}$$

The dry unit weight can also be expressed in terms of, A, w and G<sub>s</sub>. The zero air voids curve is also obtained by assuming that A is zero.

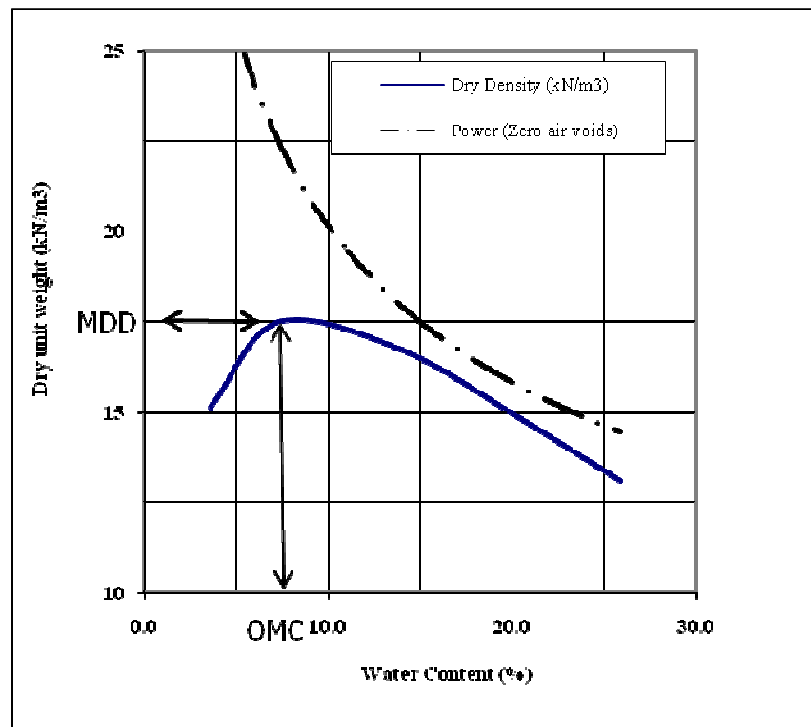


Fig. 9.1: Typical compaction curve

## 9.2.2 Variation in Compaction Curve

### A) Variation in compaction effort

Increasing the compaction effort results in the increase in the maximum dry density and a decrease in the optimum moisture content as shown in Fig. 9.2. Thus if light compaction is used more water would be needed to overcome the resistance of the soil grains to packing. With more compaction effort the soil grains need less water to occupy the available pore spaces.

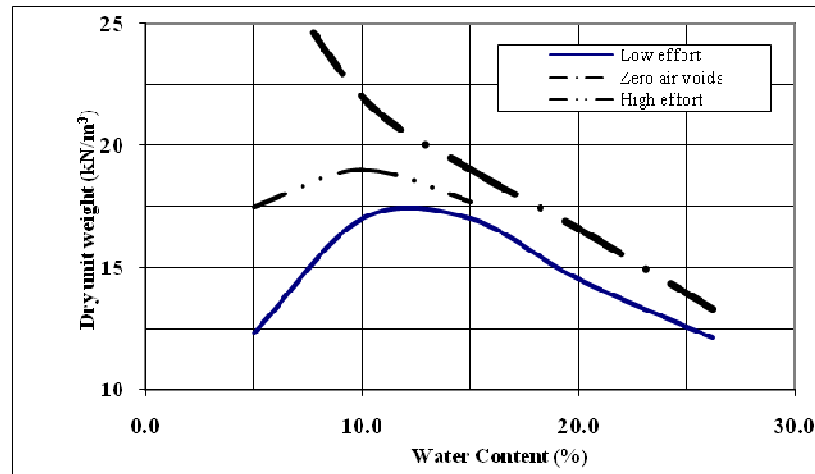


Fig. 9.2: Effect of compaction effort

### B) Variation in soil grading soil type

The particle size distribution influences the arrangement of particles in the compaction of soils. A well-graded soil will compact to lesser voids than a poorly graded soil. The effect is that the well-graded soils will have an increased density and improved properties such as bearing capacity on compaction. These enhanced strength characteristics of compacted fills are important because of the increasing need of embankments capable of supporting higher loads.

The behaviour of soils containing clay particles is different from that of granular soils. Their behaviour is related to the arrangement of the clay minerals, the inter-particle forces and the nature of soil water. Thus when two particles are pressed together the contact is initially between a few grains. These grains deform elastically to develop Van der Waal forces. The Van de Waal forces arise from electrical forces developed in atoms bonding the particles together. They hold the particles together and when they are brought closer more forces have to be overcome during shearing. However, the overall effect of the cohesive forces and the plasticity is reduction in the maximum dry density achieved upon compaction

Thus for the same compaction effort MDD and OMC increases and decreases respectively as the soil grain size increases and the plasticity decreases (Fig. 9.3).

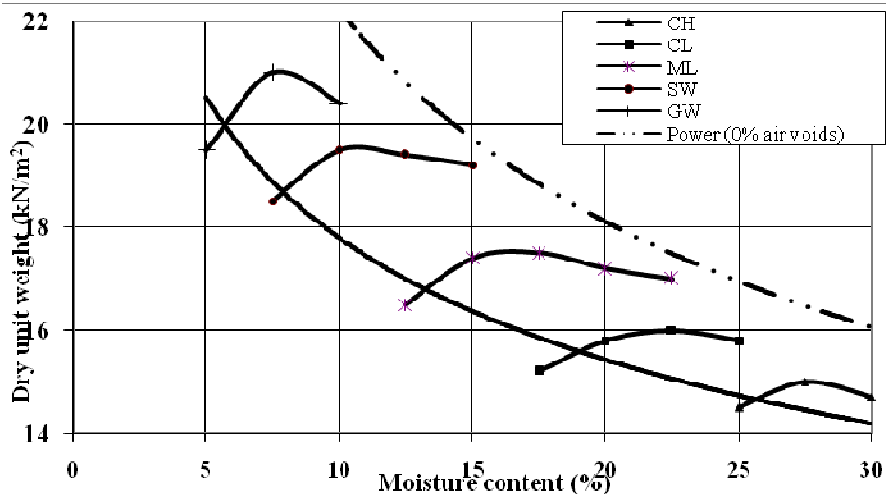


Fig. 9.3: Effect of soil type on compaction

### 9.3 LABORATORY COMPACTION TESTS

Three types of compaction tests are explained. The first is the light manual compaction test with 2.5kg hammer. The second is heavy compaction test in which much greater compaction is achieved by use of 4.5kg hammer. The third is use of a vibrating hammer and is intended mainly for granular soils passing 37.5 mm test sieve with no more than 30% retained on a 20mm test sieve. For each type of test a small variation of the test is done to take recognition of whether the soil crushes during compaction.

The mould for 2.5 kg rammer method is 1000 cm<sup>3</sup> and 100mm diameter. The test covers soil with particles finer than 20mm sieve size. The compaction is effected by free fall of the 2.5 kg rammer through 300mm in three layers. Each layer receives 27 blows

In 4.5kg method the rammer is 4.5kg. The rammer is made to freely fall through 450mm. In addition the soil in the same type of mould is compacted through 5 layers. Each layer receives 62 blows. The test is suitable for soils containing not more than 30% retained on 20mm sieve size. These particles may include particles retained on 37.5mm sieve size.

In the vibratory rammer method the mould is 2360cm<sup>3</sup> and 152 mm diameter. The compaction is effected by placing the rammer on the soil surface and vibrating the rammer at 60±2seconds. During this period a steady downward force on the rammer is applied to enable a force of between 300 and 400 N to be applied on the soil. Like the method using 4.5 kg rammer the test is suitable for soils containing not more than 30% retained on 20mm sieve size. These particles may include particles retained on 37.5mm sieve size. The test is however not suitable for cohesive soils.

After the compaction using any of the three methods the mould is trimmed at the top. The soil bulk density and moisture content are determined. The soil dry density is then computed. The procedure is repeated at different moisture contents to enable the plotting of the compaction curve. The coordinates of the peak dry density define the MDD and OMC. Typical results are shown on Fig 9.4.

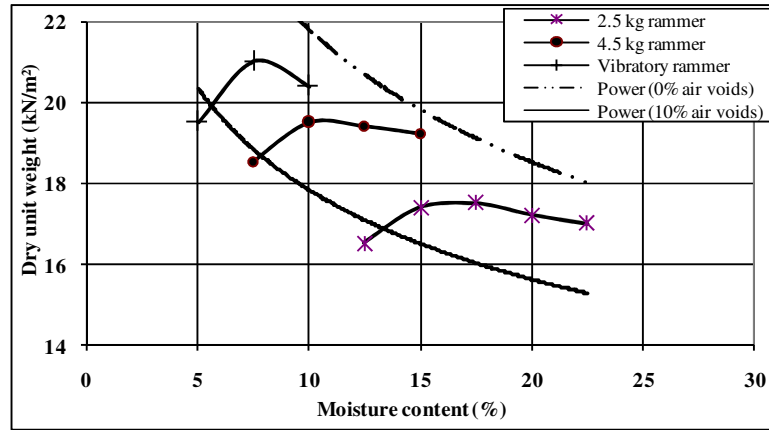


Fig. 9.4: Laboratory compaction curves

**9.4 FIELD COMPACTION**

**9.4.1 Compaction**

In the field, compaction of soil is carried out by applying energy by application of pressure, Impact or vibration or combination of two or three of the ways. The application of energy is effected by different machinery including rollers, rammers, vibrators, tampers and construction equipment. The type of compaction method to be adopted in the field depends on the kind of soils and the level of compaction required. Table 9.1 shows the different varieties of compaction equipment. A brief outline of the various compacting equipment is now presented.

**Table 9.1: Varieties of compaction equipment**

Rollers	Rammers	Vibrators	Construction equipment
Smooth wheeled rollers	Dropping wheel	Out of balance weight	Lorries
Vibratory rollers	Internal combustion	Pulsating hydraulic	Scrapers
Pneumatic tired rollers	Pneumatic type		Excavators
Sheep’s foot rollers	Piling equipment		Graders
Construction equipment			

**The smooth wheeled rollers** have their dead weight increased by water or sand. A smooth surface is generally achieved after compaction. The compacted layers are laminated. These rollers can be fitted with vibratory equipment for more effective compaction. They generally come in three tandem or three axle tandem rollers and are particularly good for finishing works on embankments and roads.

**The pneumatic – tired rollers** are good for coarse and fine grained soils. They are set on two axles. Usually the rear axle may have three wheels while the front axle has two wheels. In between the axles a platform is mounted which carries loads for providing the needed pressure on the soils under compaction. The tires are wide and increase in pressure increases the compaction effort. It is common to have the rollers towed by tractors.

**The sheep's foot rollers** consists of a roller with hollow steel drums acting as wheels fitted with numerous tapered or club shaped feet projecting from a drum surface. The feet take the shape of a sheep's foot and hence the name. Initially a soft soil is penetrated by the sheep's foot. As the soil increases in density upon rolling of the roller, the sheep's foot rises to the surface. The penetrating feet mix the soil up bonding it together. This type of roller is suitable for water retaining earthworks.

**Rammers** are usually employed when the site conditions are such that conventional rollers are not suitable. They are usually in various shapes and are used by manual labour. They are particularly useful in compacting corners and crevices. They are used for compacting backfill soil below ground floor slabs

**Vibrators** are generally used for compacting coarse grained soils. They are attached to smooth wheeled rollers and fitted with out of balance weights or pulsating type engine is fitted to the vibrator.

**Construction equipment** can be utilized to compact soils to a reasonable degree of compaction. This type of compaction has been used in reclaiming swampy fields where the quality of the final surface need not be of high quality. In the field compaction equipment employed is usually in different types depending on the complexity of the works. This construction traffic should be channelled and directed on the earthworks systematically on the loose earthworks to achieve beneficial compaction. The construction equipment comes in an array of different types such as;

- i) Lorries
- ii) Scrapers
- iii) Excavators
- iv) Graders
- v) Etc

#### 9.4.2 Field Control of Compaction

Tests for determination of bulk density and water content during earthwork construction must be carried out at every layer of the compacted earthworks. In the least at least four density tests per 8 hour shift with a minimum one test for every 400m<sup>3</sup> of earthworks are needed. Bulk density determination may be carried out by means of a core cutter, sand replacement method or nuclear radiation.

In the core cutter method undisturbed sample is obtained by pushing a thin walled cutter, 10cm in diameter and 12.5 cm high into a compacted soil. The bulk density and the moisture content are determined to enable determination of the field dry density. The sand replacement is by far the most widely used method. In this method a hole of



100x150mm (diameter x depth) is dug at the test site. The mass of soil from the hole is carefully retrieved and weighed. The volume from where the soil has been excavated is gotten by pouring loose dry sand from a fixed height through a cone shaped stand.

The nuclear radiation technique uses a nuclear gauge. In the usage of the gauge, an aluminum probe is inserted in the compacted soil. Neutrons are then released from a source. The neutrons lose energy depending on the soil density and water content as they pass through the soil. The instrument is calibrated to give water content, bulk density and the dry density of the soil. The instrument should be calibrated against the sand replacement test results.

### **9.4.3 Specification of the Field Compacted Density**

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The compacted density is usually specified as a percentage of the MDD as determined in the laboratory. In some instances the moisture content is required to be within a small band above or below the OMC. In rare instances the air content is specified. The specification usually ranges from 95-100% MDD when a 2.5 kg hammer is used the field control. A minimum number of roller passes is usually required to reach the desired compaction. The usual range is between 3 and 12 passes.

In road work embankments the specification for the lower layers is usually a minimum of 95% MDD. The compaction for the upper subgrade and the pavement layers should however be raised to between 100 and 105 MDD.