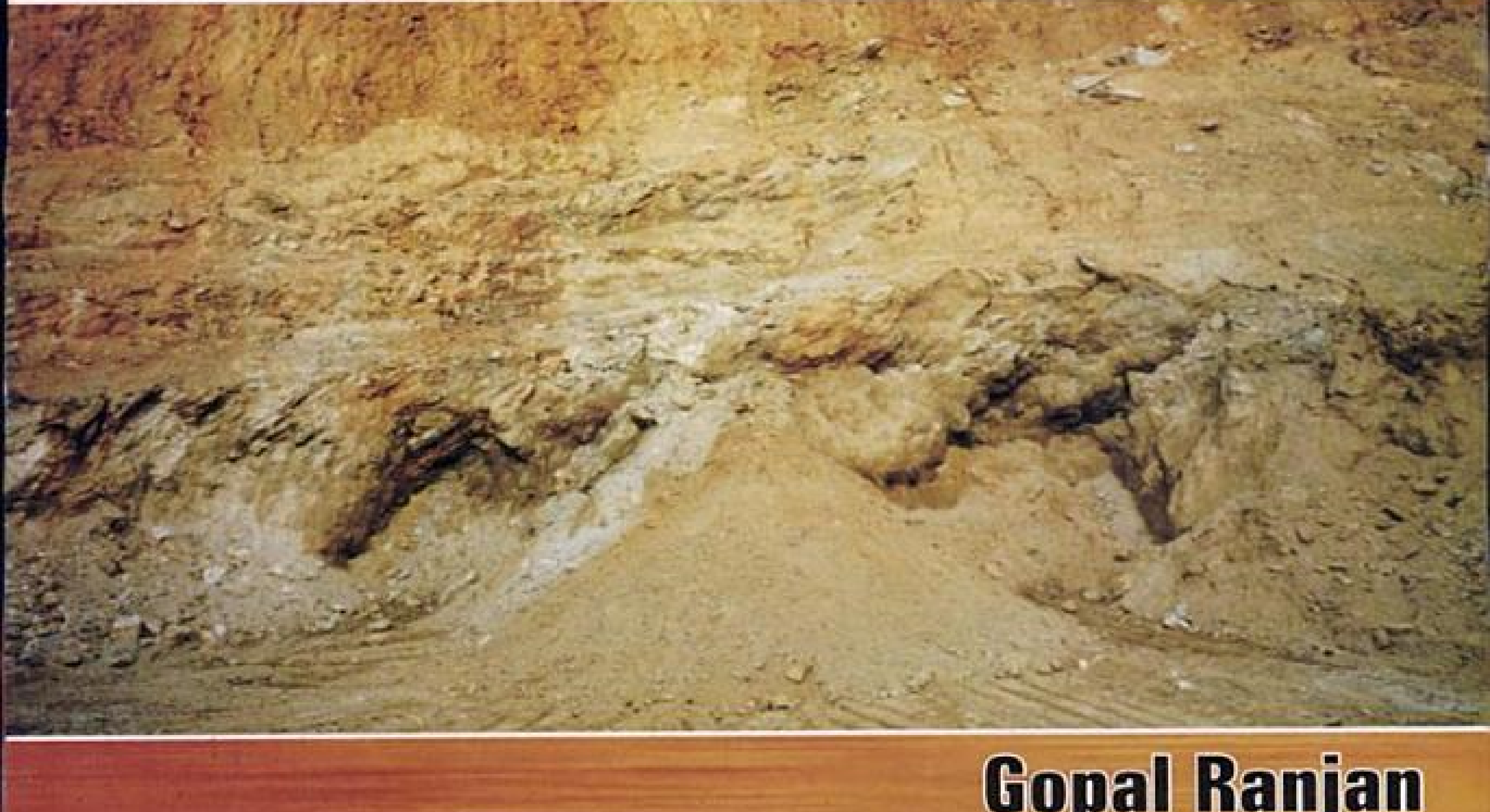
Revised Second Edition

Basic and Applied Soil Mechanics



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Introduction

1.1 SOIL, SOIL MECHANICS AND SOIL ENGINEERING

The term *'soil'* has different connotations for scientists belonging to different disciplines. The definition given to a soil by an agriculturist or a geologist is different from the one used by a civil engineer. To an agriculturist, soil merely means the top layer of the earth which is responsible for supporting plant life. Even to a geologist, soil is the thin outer layer of loose sediments within which plant roots are present. A geologist refers to the rest of the earth's crust as rock, irrespective of how strong or weak the bonding forces of the sediments are. For a civil engineer, *soils* mean all naturally occurring, relatively unconsolidated earth material—organic or inorganic in character—that lies above the bedrock. According to Terzaghi, soils can be broken down into their constituent particles relatively easily, such as by agitation in water. On the other hand, rocks are an agglomeration of mineral particles which are bonded together by strong molecular forces. Often, this distinction between soils and rocks is not clear-cut. Many a hard soil can also be termed as soft rock or *vice versa*. Rocks can be the massive bedrock or large fragments of gravel, pebbles, etc., within a soil.

Soil Mechanics is the branch of civil engineering that concerns the application of the principles of mechanics, hydraulics and to a smaller extent, chemistry, to engineering problems related to soils. The study of the science of soil mechanics equips a civil engineer with the basis scientific tools needed to understand soil behaviour. This is by no means sufficient to provide satisfactory solutions to soil problems. The reasons for this will be clear later.

Rock Mechanics is defined as the science dealing with the application of the principles of mechanics to the understanding of the behaviour of rock masses.

Soil Engineering is a broader term which eniompasses not only soil mechanics but also geology, structural engineering, soil dynamics and many other disciplines which are often essential to obtain practical solutions to problems of soil. *Geotechnical Engineering* is relatively a new term and includes soil mechanics, rock mechanics, soil engineering and rock engineering. Geotechnical engineers need to have a proper understanding of engineering geology, since there is a significant overlap between the two disciplines.

The present volume is strictly restricted to the study of the engineering behaviour of the soil mass.

1.2 CIVIL ENGINEERING PROBLEMS RELATED TO SOILS

A civil engineer has to deal with soils in their diverse roles. Every civil engineering structure, whether it be a building, a bridge, a tower, an embankment, a road pavement, a railway line, a tunnel or a dam, has to be founded on the soil (assuming that a rock stratum is not available) and thus shall transmit the dead and live loads to the soil stratum. Soil is, therefore, the ultimate foundation material which supports the structure. The proper functioning of the structure will, therefore, depend critically on the success of the *foundation* element resting on the subsoil. Here the term foundation is used in the conventional sense, namely, a substructure that distributes the load to the ultimate foundation, namely, the soil.

Soil is also the most abundantly available construction material. From ancient times, man has used soil for the construction of tombs, monuments, dwellings, and barrages for storing water. In modern times, the use of earth for building dams and for constructing pavements for highways and airfields is an important aspect of civil engineering. Many questions concerning the design and construction of these structures need to be answered before satisfactory solutions are obtained.

In the design and construction of underground structures such as tunnels, conduits, power houses, bracings for excavations and earth retaining structures, the role of soil is again very crucial. Since the soil is in direct contact with the structure, it acts as a medium of load transfer and hence for any analysis of forces acting on such structures, one has to consider the aspect of stress distribution through the soil. This, however, cannot be done by considering the behaviour of the structure in isolation of the soil or by treating the soil independently of the structure. The structure, too, causes stresses and strains in the soil, while the stability of the structure itself is affected by soil behaviour. The class of problems where the structure and soil mutually interact, are known as *soil-structure interaction* problems.

There are a host of other civil engineering problems related to soils. For designing foundations for machines such as turbines, compressors, forges etc., which transmit vibrations to the foundation soil, one has to understand the behaviour of soil under vibratory loads. The effect of quarry blasts, earthquakes and nuclear explosions on structures is greatly influenced by the soil medium through which the shock waves traverse. In those parts of the world which experience freezing temperatures, problems arise because the soils expand upon freezing and exert a force on the structures in contact with them. Thawing (due to melting ice) of the soil results in a loss of strength in the soil. Structures resting on these soils will perform satisfactorily only if measures are taken to prevent frost-heave or designed to withstand the effects of freezing and thawing.

1.3 COMPLEXITY OF SOIL NATURE

A natural soil deposit is quite unlike any other material of construction known to man. Most of the commonly used materials of construction such as wood, steel, concrete or reinforced concrete are capable of proper structural analysis once a few simple and well chosen physical and mechanical parameters like the modulus of elasticity, yield stress, Poisson's ratio, etc., are known. One can select the material which best meets the prevailing conditions and then determine the allowable stress for that material. The material can be expected to behave in a reasonably predictable manner. On the other hand, no choice of soil is normally available to an engineer. Most of the suitable sites for construction have already been used up and often one has to make do with a site having unsatisfactory subsoil conditions. Occasionally, it may be possible to improve the soil by some suitable treatment, but more often than not, the soil has to be accepted in its natural state. Unlike a manufactured material like steel, soil deposits have been placed by nature under a variety of conditions which have rendered the deposits anything but homogeneous. The wide range of characteristics that different *soil* deposits from peats to compact gravels exhibit, is simply amazing. There is more. Even at a given site, samples of soil taken from two locations not too far apart, from the same stratum, may show widely varying properties. It requires great ingenuity, therefore, to select representative soil parameters for a natural soil deposit. Unexpected changes take place in soils when certain environmental changes occur. For instance, vibrations can alter the state of a sand deposit from loose to dense. Some clay soils, which are extremely hard when dry, can turn into slush having very little shearing strength, when their water content becomes high. Indeed, water is by far the most important variable controlling the behaviour of fine-grained soils.

Natural soil deposits are complex to deal with, because

(a) The stress-strain relationship for a soil deposit is non-linear; hence the difficulty in using easily determinable parameters to describe its behaviour.

(b) Soil deposits have a memory for stresses they have undergone in their geological history. Their behaviour is vastly influenced by their stress history; time and environment are other factors which may alter their behaviour.

(c) Soil deposits being far from homogenous, exhibit properties which vary from location to location.

(d) As soil layers are buried and hidden from view, one has to rely on tests carried out on small samples obtained from selected depths and locations. Since there is a constraint on the number of samples that can be taken, there is no guarantee that the soil parameters are truly representative of the field strata.

(e) No sample is truly undisturbed. In a soil, which is sensitive to disturbance, the behaviour surmised from the laboratory tests may not reflect the likely behaviour of the field stratum.

It must now be clear why it is not sufficient enough for a soil engineer in search of practical solutions, to possess the knowledge of the principles of soil mechanics. He needs to have geology as his ally. Knowledge of the various processes that have gone into the composition of the natural soil mass is important since these have a direct bearing on soil behaviour. He must also draw upon the experience of others. In this way, he can avoid the common pitfalls which have already been identified and profit from the knowledge of workable precedents. He must also continually evaluate his own designs in the light of data obtained from field measurements. The basis of the *observational method* in soil mechanics is the comparison of the predicted performance with the measured performance. One then tries to fill the gaps in the original reasoning. Many a time, the design may need suitable modification upon reevaluation of additional data. With all this a soil engineer is a practitioner of an art, rather than a science. At best, soil engineering is an intuitive science. The intuition—also known as engineering judgement—comes from a combination of the knowledge of theoretical soil mechanics, experience and skill.

1.4 HISTORICAL DEVELOPMENT

Although our predecessors did not know soil mechanics in the sense we understand it today, they surely knew the art of using earth as a construction material. In the prehistoric times, man built earthen mounds for burial, for protection against floods, for religious purposes and even for building caves to live in. In ancient times and in the Roman period, the use of soil was appreciated in the construction of roads, canals and bridges. The Pyramids of Egypt, the famous Hanging Gardens built by the Babylonian King Nebuchadnezzar, the Wall of Babylon, the Great Wall of China and the roads and aqueducts built by the Romans are some of the outstanding constructions that must have required great knowledge, skill and ingenuity on the part of their builders. The code of Hammurabi (2250 B.C.) and the writings of Vitruvius (first century B.C.) were pioneering attempts to codify building laws. In the Middle Ages, European engineers understood the problems associated with settlement by their experience of construction of massive cathedrals. The settlement of the Leaning Tower of Pisa and the Archbishop's Cathedral in Riga are well known. It was also during this period that Scandinavians used timber piles to support structures on soft clay soils. During Renaissance, the great genius, Leonardo da Vinci, recorded some of his innovative ideas on retaining-wall fortification, road construction and canal making in his detailed sketches and drawings.

It was only in the 18th century that the first attempts to develop some theories pertaining to design of foundations and other constructions, were made. Coulomb (1776), a French physicist, formulated his classical theory on earth pressure. In this, Coulomb also introduced the concepts of frictional resistance and cohesive resistance for solid bodies, which he assumed to be applicable to granular bodies including soils. That is why the famous shear strength theory is named after him. Darcy's law of permeability of soil and the Stokes's law of velocity of solid particles through liquids, were both evolved in 1856. Rankine, a Scottish civil engineer, published in 1857 his theory on earth pressure and equilibrium of earth masses. Mohr's rupture theory and Mohr's stress circles, developed for solid materials in 1871, are extensively used in the analysis of shear strength of soils. Again, Boussinesq's theory (1885) of stress distribution in a semi-infinite, homogeneous, isotropic medium induced by a surface point load, has been extended to soils by ascribing these properties to the soil.

The beginning of the 20th century saw some important developments related to the physical properties of soils. In 1911, Atterberg, a Swedish scientist, defined the consistency limits for cohesive soils, which are still in use for soil identification. Pioneering work in practical application of soil mechanics was done by the Swedish Geotechnical Commission of the State Railways of Sweden. After a number of landslides had occurred on the railways, canals and harbour structures, a committee, headed by Fellenius, was commissioned to investigate the causes of failure and suggest remedial measures. The committee gave its final report in 1922. It was, indeed, a remarkable report in many respects. It contained new concepts on soil sampling and testing. A method for calculating the stability of slopes (now known as the Swedish circle method) made of cohesive soils, was developed. For the first time, concepts of sensitivity of clays and consolidation were understood.

Remarkable though were the developments in soil mechanics between 1900 and 1925, it was from 1925 onwards that the fledgling discipline got a tremendous fillip, thanks to the great genius of an Austrian, Karl Terzaghi. It is now accepted that the modern discipline of soil mechanics began in 1925 with the publication of the first textbook on soil mechanics, *Erdbaumechanik*, by Terzaghi. In fact, the name 'Soil Mechanics' is English translation of the German word *Erdbaumechanik.* Terzaghi's contributions to soil mechanics are too numerous to mention. Yet, his enunciation of the principle of effective stress and the one-dimensional consolidation theory cannot but be hailed as landmarks in the understanding of soil behaviour. Rightly has he been called the 'the father of soil mechanics'.

Other prominent contributors to the growth of soil mechanics in the modern era include Proctor, whose theory of compaction (1933) is well known, Arthur Casagrande, Taylor, Peck, Skempton, Bjerrum and Seed. It can be said that most of the pioneering work in soil mechanics is now complete and research in frontier areas is receiving attention.

The growth of literature in soil mechanics after the Second World War has been phenomenal. This is reflected in the proceedings of a series of International Conferences on Soil Mechanics and Foundation Engineering held at Harvard, Rotterdam, Zurich, London, Paris, Montreal, Mexico city, Moscow, Tokyo, Stockholm, San Francisco, Rio de Janeiro, New Delhi and Hamburg in 1936, 1948, 1953, 1957, 1961, 1965, 1969, 1973, 1977, 1981, 1985, 1989, 1994 and 1997 respectively. Thanks to the foresightedness of A. N. Khosla, the Indian Geotechnical Society (originally known as the Indian National Society of Soil Mechanics and Foundation Engineering) was established in 1948. The Society coordinates activities in the field of geotechnical engineering in India. Jai Krishna, formerly Vice Chancellor, University of Roorkee, was the first to teach soil mechanics as a formal course in the undergraduate curriculum of Roorkee University in 1948.

1.5 SOIL FORMATION AND SOIL TYPES

On the basis of the geological origin of their constituent sediments, soil can be divided into two main groups—those which owe their origin to the physical and chemical weathering of the parent rocks, and those which are chiefly of organic origin. The latter type are extremely compressible and their use as foundation material is best avoided. Of the former group, soils which are a product of physical weathering or mechanical disintegration, retain the minerals that were present in the parent rocks and are coarse grained. Gravels and sands fall into this category. The agencies responsible for physical weathering are the impact and grinding action of flowing water, ice, wind and splitting actions of ice, plants and animals. As against this, chemical weathering or decomposition of rocks is caused mainly by oxidation, hydration, carbonation and leaching by organic acids and water. Clays and, to some extent, silts are formed by chemical weathering.

If the products of rock weathering are still located at the place where they originated, they are called *residual* soils. Any soil that has been transported from its place of origin by wind, water, ice or any other agency and has been redeposited, is called a *transported* soil. Residual soils are not as common as transported soils.

Characteristics of soil such as the size of the particles, their shape and roundness, surface texture and the degree of sorting that takes place in a soil deposit are influenced by the agency of transportation. Table 1.1 summarises these effects. Transported soils are further classified according to the transporting agency and method of deposition:

(a) Alluvial deposit—soils that have been deposited from suspension in running water.

(b) Lacustrine deposit—soils that have been deposited from suspension in still, fresh water of lakes.

(c) Marine deposit—soils that have been deposited from suspension in sea water.

(d) Aeolian deposit—soils that have been transported by wind.

(e) Glacial deposit—deposits that have been transported by ice.

Table 1.1 Effects of Transportation on Sediments

	Water	*Air*	*Ice*	*Gravity*	*Organisms*
Size	Reduction through solution, little abrasion in suspended load, some abrasion and impact in traction load	Considerable reduction	Considerable grinding and impact	Considerable impact	Minor abrasion effects from direct organic transportation
Shape and roundness	Rounding of sand and gravel	High degree of rounding	Angular, soled particles	Angular, non-spherical	
Surface texture	Sand: smooth, polished, shiny	Impact produces frosted surfaces	Striated surfaces	Striated surfaces	
Sorting	Considerable sorting	Very considerable sorting (progressive)	Very little sorting	No sorting	Limited sorting.

(After Lambe and Whitman, 1969)

Names of some of the soils that have been formed by these methods of transportation and deposition are given and explained below:

(i) Loess — A loose deposit of wind-blown silt that has been weakly cemented with calcium carbonate and montmorillonite. Loess is formed in arid and semi-arid regions and stands in nearly vertical banks.

(ii) Tuff — A small-grained slightly cemented volcanic ash that has been transported by wind or water.

(iii) Bentonite — A chemically weathered volcanic ash.

(iv) Glacial till (boulder clay) — Typically, a mixture of boulders, gravel, sand, silt and clay, deposited by glaciers and not transported or segregated by water.

(v) Varved clay — Alternate thin layers of silt and clay deposited in fresh water glacial lakes by outwash from glaciers. The silt is deposited in warm weather during heavy run off and clay is deposited in cold weather during small run off. Generally, one band of silt and clay is deposited each year.

(vi) Marl — A very fine-grained calcium-carbonated soil of marine origin.

(vii) Gumbo — A sticky, plastic, dark coloured clay.

(viii) Peat — A highly organic soil, consisting almost entirely of vegetative matter in varying states of decomposition, brown to black in colour, possessing an organic odour. Peat is fibrous and highly compressible.

(ix) Muck — A mixture of fine particled, inorganic soil and black, decomposed organic matter. It is usually found accumulated in conditions of imperfect drainage as in swamps or is deposited by overflowing rivers.
Peat and muck are also called *cumulose* soils.

(x) Humus — A dark brown, organic, amorphous earth of top soil, consisting of partly decomposed vegetative matter.

(xi) Hard pan — A layer of extremely hard, cohesive soil that can hardly be drilled with ordinary, earth boring tools.

(xii) Colluvial soil — The accumulation of rock debris or talus at the base of a steep cliff or a rock escarpment. Its position results mainly from the effect of the force of gravity acting on the rock fragments broken from the rocks above.

(xiii) Mine tailings — These are silt-sized materials resulting as waste after extraction of minerals from natural rock and are usually deposited by hydraulic fill.

1.6 REGIONAL SOIL DEPOSITS OF INDIA

The soils of India can be broadly divided into the following groups, based on the climatic conditions, topography and geology of their formation:

(a) Marine deposits

(b) Laterites and lateritic soils

(c) Black cotton soils

(d) Alluvial soils

(e) Desert soils

(f) Boulder deposits

Figure 1.1 shows the general areas of occurrence of these soils.

Marine deposits: These deposits are found all along the coast in narrow tidal plains. The marine clays are very soft and may contain organic matter. They possess low shear strength and high compressibility and hence pose problems as a foundation material or as a material of construction.

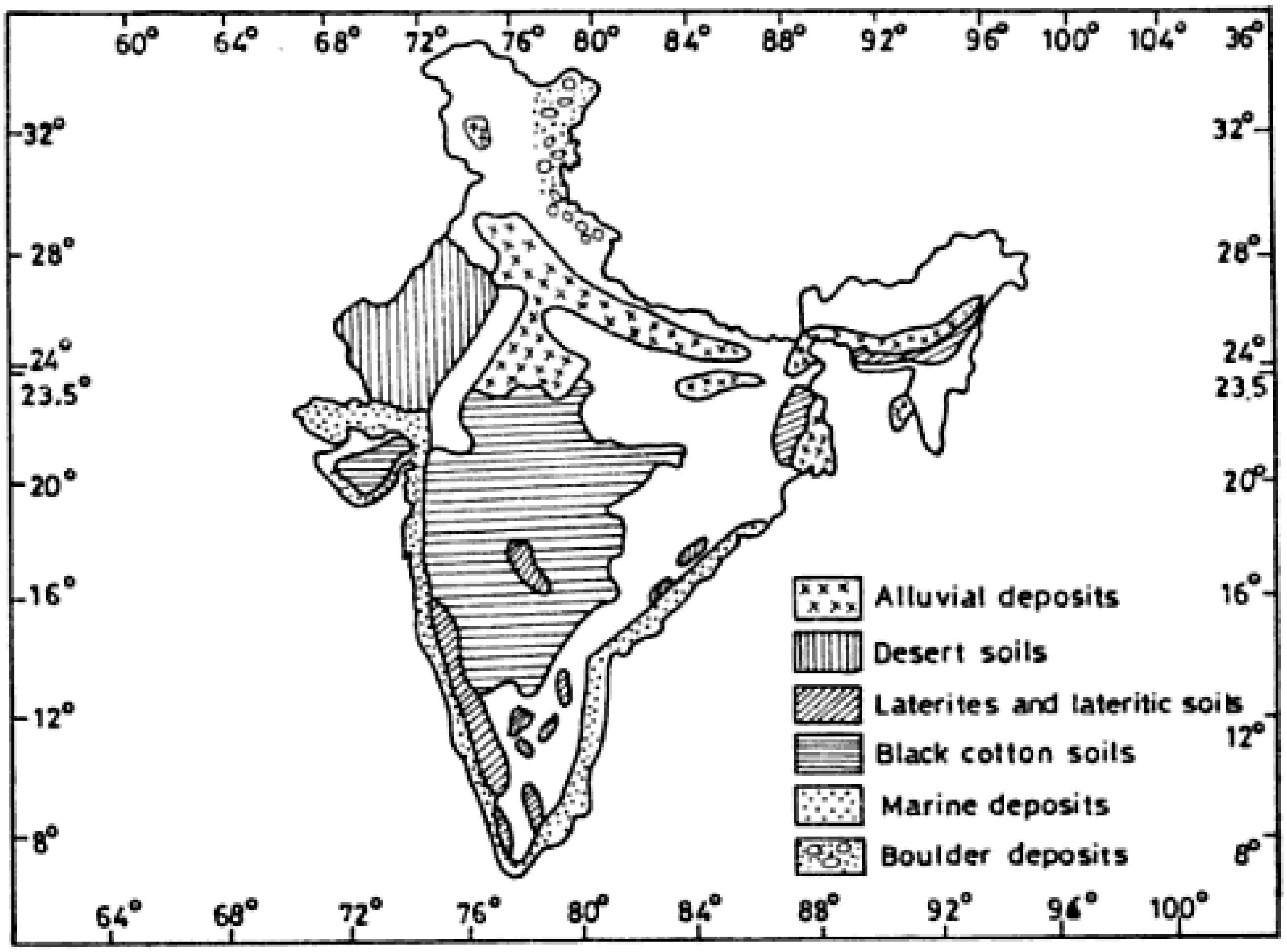

Fig. 1.1 Regional soil deposits of India

Laterites and lateritic soils: Lateritic soils cover an area of about 100,000 sq.km and extend over Kerala, Karnataka, Maharashtra, Orissa, and West Bengal. Laterites are formed by the decomposition of rock, removal of the bases and silica and formation of oxides of iron and aluminium at the top of the soil profile. In Kerala, the laterites are soft when wet but harden with age.

There are two types of laterites, namely, the primary and the secondary. Primary laterite is found *in situ.* The original rock structures, joints and quartz material are intact and the laterite deposit overlies the bedrock. Primary laterite is found at high altitudes near hills. Secondary laterites are found in the coastal belt. These are formed from sedimentary deposits such as gravels and pebbles by sesquioxide impregnation and cementation. They are pellet type and are quite different from the underlying soil or bed rock. This may create problems in foundations, if a thin laterite layer overlies a soft material. Laterites are reddish in colour and are hard in the dry state. But not all soils which show these characteristics are laterites. If the grain size increases upon alternate wetting and drying cycles, the soil is a *laterite. Lateritic* soils do not show this characteristic. Generally, laterites pose no difficulties as foundation material and retain their slopes well. However, there is a continuous softening effect with depth and in some cases, the presence of worm holes in laterites need to be examined carefully. Like all residual soils, laterites show variability in their properties, depending on the stage of weathering.

Black cotton soils: This is the Indian name given to the expansive soil deposits in the central part of the country. They cover an area of approximately 3,00,000 sq km which extends over the states of Maharashtra, Madhya Pradesh, Karnataka, Andhra Pradesh, Tamil Nadu and Uttar Pradesh. These soils have been formed from basalt or trap and contain the clay mineral montmorillonite, which is responsible for the excessive swelling and shrinkage characteristics of the soil. Lightly loaded structures are most susceptible to damage as a result of the volume changes in the soil. Under-reamed piles are considered most suitable as foundations for houses and other light structures. These piles are taken to depths below the zone of seasonal variation in moisture content.

Alluvial soils: Large parts of northern India lying north of Vindhya-Satpura range in the Indo-Gangetic and Brahmaputra flood plains are covered by the alluvial soils. The thickness of these soil deposits is sometimes over 100 m. The deposits have alternating layers of sand, silt and clay. There is a great deal of variation in the thickness of these layers and their horizontal development. The alluvial deposits extend from Assam in the east to Punjab in the west.

The fine silty sand deposits in this area are loose and prone to liquefaction under earthquake shocks.

Desert soils: Large parts of Rajasthan, covering about 5,00,000 sq km, consist of desert soils which are wind-blown deposits of sand. The sand dunes have an average height of about 15 m, but can at times be considerably higher. They are formed under highly arid conditions. Dune sand is nonplastic uniformly graded, fine sand. Some of the problems associated with this soil are of soil stabilisation for roads and runways, reducing settlement under static and dynamic loads and reducing its perviousness to make it suitable for storage and transport of water.

Boulder deposits: Rivers flowing in hilly terrains and near foot-hills carry large boulders downstream. The deposits that such flows make may contain large quantities of boulders. Such deposits are often found in the sub-Himalayan regions of Himachal Pradesh and Uttar Pradesh. The properties of these deposits depend on the relative proportions of the boulders and the soil matrix. The boulder-to-boulder contact may result in large friction resulting in higher angles of shearing resistance.

The shear strength of such deposits cannot be investigated in the laboratory, since the soil samples can never be representative of the natural deposit, in view of the limitation of their sizes. In the field, it is possible to conduct large size shear box test *in situ* (Prakash and Ranjan, 1975). Such a test is very useful to obtain inputs required for the design of foundations on boulder deposits and in the stability analysis of slopes in bouldery soils.

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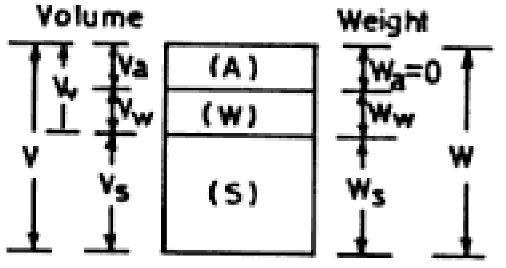
2

Solids-Water-Air Relationships and Index Properties of Soils

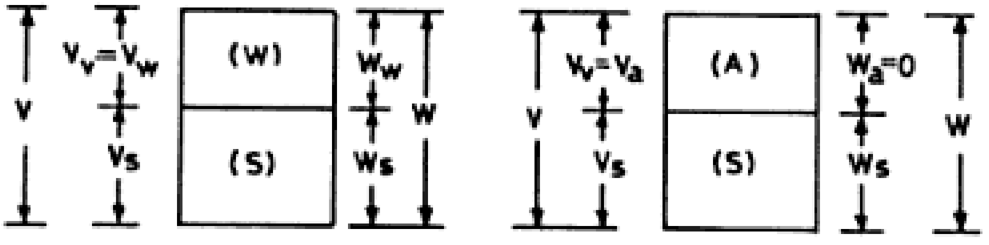
2.1 PHASE DIAGRAM

The soil mass is, in general, a *three-phase system* composed of solid, liquid and gaseous matter. The solid phase is comprised of mineral or organic matter or both. The mineral portion consists of particles of different sizes and shapes. The organic fraction is the plant and animal residue which may be present in various stages of decomposition. The solids enclose the open spaces termed *voids*. The liquid phase which is generally water, fills partly or wholly the voids. The gaseous phase (usually air) occupies the voids not filled by water.

The relative volumetric and gravimetric proportions of solids, water and air in a soil mass are important factors influencing its physical properties. It is, therefore, necessary to study them in some detail. Though the different phases present in the soil mass cannot be separated, for a better understanding of the soil behaviour, it is helpful to think of these constituents as occupying separate spaces, as shown in Fig. 2.1.



(a) 3-Phase system (partially saturated)



(Saturated) (Dry)

(b) 2-Phase system

Fig. 2.1. Phase diagrams

The diagrammatic representation of the different phases in a soil mass is called the *phase diagram*. Figure 2.1(a) shows a three-phase system which is applicable for a partially saturated soil. When all the voids are filled with water, the sample becomes saturated and thus the gaseous phase is absent whereas in a dry soil, the liquid phase is missing [Fig. 2.1(b)]. Thus, both the saturated and dry states of soil have two phases only.

In Fig. 2.1, the volumes and weights of various constituents of the soil mass are designated by V and W, respectively. The subscripts a, w, s and v stand for air, water, solids and voids respectively.

The weight of air, for all practical purposes, may be assumed to be zero. The weight of the voids is thus equal to the weight of water W_w. The weight of solids is represented by W_s, which is evidently the weight of a dry soil. The total weight of a moist sample is thus $W = W_s + W_w$. The total volume V consists of three parts, namely, (i) volume of solids, V_s (ii) volume of water, V_w and (iii) volume of air, V_a. As the voids may be filled by water or air or both, the volume of voids $V_v = V_w + V_a$. Thus $V = V_s+V_v= V_s+V_w+V_a$. It is important to note that the term 'voids' is used to include both the filled and the unfilled portions of the pore spaces enclosed by the soil grains.

2.2 SIMPLE DEFINITIONS

As mentioned earlier, the relative proportions of solids, water and air present in a soil mass influence its physical properties. Certain terms related to volumetric and gravimetric compositions of soils are frequently used in soil engineering. These terms are described in the following sections.

Water Content

Water content, w, also called the *moisture content*, is defined as the ratio of the weight of water to the weight of solids in a given mass of soil. It is usually expressed in percentage.

$$w = \frac{W_w}{W_s} \times 100 \tag{2.1}$$

In nature, fine-grained soils have higher values of natural moisture content compared to the coarse-grained soils. In general, it can be written that $w \geq 0$, since there can be no upper limit to water content.

Void Ratio

Void ratio, e of a soil sample is defined as the ratio of the volume of voids to the volume of solids. Thus, symbolically,

$$e = \frac{V_V}{V_s} \tag{2.2}$$

In nature, even though the individual void sizes are larger in coarse-grained soils, the void ratios of fine-grained soils are generally, higher than those of coarse-grained soils. In general, it can be written that $e > 0$, since a soil has to contain some voids but there cannot be an upper limit to the void volume.

Porosity

Porosity, n of a soil sample is defined as the ratio of the volume of voids in the sample to the total volume of the soil, expressed as a percentage. Thus, symbolically,

$$n = \frac{V_v}{V_s} \times 100 \tag{2.3}$$

The porosity of a soil cannot exceed 100 per cent. Thus, it can be written that $0 < n < 100$.

Both void ratio and porosity express the proportion of volume of voids-the former with respect to the volume of solids, and the latter with respect to the total volume of soil. However, in soil engineering, void ratio is more favoured for use. This is due to the fact that any change in the volume of a soil mass is a direct consequence of a similar change in the volume of voids while the volume of solids remain the same. Hence, it is more convenient to use void ratio in which only the numerator (V_v) changes when the volume of a soil changes while in porosity, both the numerator (V_v) and the denominator (V) change.

Degree of Saturation

Degree of saturation, S or S_r, is defined as the ratio of the volume of water to the volume of voids. It is usually expressed as a percentage. Thus,

$$S = \frac{V_w}{V_v} \times 100 \tag{2.4}$$

Since the degree of saturation indicates the portion of volume of voids which is filled with water, for a fully saturated soil, $V_w = V_v$ and hence $S = 1$ or 100 per cent. Similarly, for a perfectly dry soil, $V_w = 0$; thus $S = 0$ or 0 per cent. For a partially saturated soil, the degree of saturation varies between 0 per cent and 100 per cent. That is, $0 \leq S \leq 100$.

Air Content

Air content, a_c is the ratio of the volume of air voids to the volume of voids.

$$a_c = \frac{V_a}{V_v} = 1 - S \tag{2.5}$$

Percentage Air Voids

Percentage air voids, n_a is the ratio of the volume of air voids expressed as a percentage of the total volume of the soil mass. Symbolically,

$$n_a = \frac{V_a}{V} \times 100 \tag{2.6}$$

It can be seen that $n_a = n\,a_c$ (2.7)

Unit Weight

Unit weight of a soil is its weight per unit of volume. The unit weight must be expressed with due regard to the state of the soil. The following definitions of unit weight are important in relation to soils:

Bulk unit weight, γ_t is the total weight of a soil mass, W per unit of total volume, V. It is also known as the *total unit weight*. Symbolically,

$$\gamma_t = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_w + V_a} \tag{2.8}$$

In SI units, it is convenient to express it in kN/m^3. In cgs/MKS units, it is expressed in gmf/cc, kgf/m^3 or t/m^3. In this text, the suffix 'f' in the units gmf and kgf is dropped and the units g and kg are used to express weights in cgs and MKS system.

Dry unit weight, γ_d is the weight of solids, W_s per unit of total volume. Symbolically,

$$\gamma_d = \frac{W_s}{V} \tag{2.9}$$

The dry unit weight is used as a measure of the denseness of a soil. A high value of dry unit weight indicates that more solids are packed in a unit volume of the soil and hence a more compact soil.

Saturated unit weight, γ_{sat} is the ratio of the total weight of a fully saturated soil sample, W_{sat} to its total volume V. In other words, it is the bulk unit weight of a soil when it is completely saturated.

$$\gamma_{sat} = \frac{W_{sat}}{V} \tag{2.10}$$

Submerged unit weight or Buoyant unit weight γ' or γ_b *is the submerged weight of soil solids per unit volume. When a soil mass is submerged below the ground water table, a buoyant force acts on the soil solids which is equal in magnitude to the weight of water displaced by the solids. The net weight of the solids is reduced; the reduced weight is known as the submerged* or the *buoyant* weight.

Submerged unit weight is equal in magnitude to the saturated unit weight, γ_{sat} minus the unit weight of water, γ_w

$$\gamma' = \frac{W_{(sub)}}{V} = \gamma_{sat} - \gamma_w \tag{2.11}$$

The unit weight of water, γ_w depends on its temperature. However, for all practical purposes, the unit weight of water is taken to be constant at 1.0 g/cc or 9.8 kN/m^3.

The submerged unit weight is roughly one half of the saturated unit weight.

Unit weight of solids, γ_s is the ratio of the weight of solids, W_s to the volume of solids, V_s

$$\gamma_s = \frac{W_s}{V_s} \tag{2.12}$$

Specific Gravity

Specific gravity of solids, G_s (sometimes written as G) is defined as the ratio of the weight of a given volume of solids to the weight of an equivalent volume of water at 4°C.

$$G_s = \frac{W_s}{V_s \gamma_w} \tag{2.13}$$

Specific gravity of solids can also be defined as the ratio of the unit weight of solids to that of water. Thus,

$$G_s = \frac{\gamma_s}{\gamma_w} \tag{2.14}$$

At 4°C, $\gamma_w = 1$ g/cc or 9.8 kN/m^3

The value of specific gravity of soil grains is required in the determination of unit weight of solids ($\gamma_s = G_s \gamma_w$), unit weight of soil, void ratio, degree of saturation, water content by the pycnometer method and in several soil tests.

The value of G_s for a majority of soils lies between 2.65 and 2.80. Lower values are for coarse-grained soils. The presence of organic matter leads to very low values. Soils high in iron or mica exhibit high values. Table 2.1 gives typical values of G_s for different soils.

Table 2.1 Typical Values of G_s

Soil type	*Specific gravity*
Clean sands and gravel	2.65 – 2.68
Silt and silty sands	2.66 – 2.70
Inorganic clays	2.70 – 2.80
Soils high in mica, iron	2.75 – 2.85
Organic soils	Quite variable: may fall below 2.0

Apparent or Mass specific gravity, G_m is also sometimes used in soil engineering. Mass specific gravity is the specific gravity of the the soil mass and is defined as the ratio of the total weight of a given mass of soil to the weight of an equivalent volume of water. It is also the ratio of the bulk unit weight of the soil to the unit weight of water.

$$G_m = \frac{W}{V\gamma_w} = \frac{\gamma}{\gamma_w} \tag{2.15}$$

The authors. however, do not favour the use of this term. The use of '*unit weight*' is more logical in relation to a soil mass.

2.3. SOME IMPORTANT RELATIONSHIPS

A few important and often used relationships between various quantities defined in Section 2.2 can now be established. This can done by starting with the basic definitions of one of the parameters involved. Alternatively, a phase diagram can be used in which one of the volumes or weights is taken as unity (1) and the volumes and weights of all other quantities are then worked out in terms of this assumed value by using one or the other ratio defined earlier. Most commonly, the volume of solids is assumed as unity in this method. Both methods are demonstrated here.

Relation between W_s, W and w:

$$W = W_s + W_w = W_s(1 + W_w/W_s)$$

or

$$W_s = \frac{W}{1+w} \tag{2.16}$$

Relation between e and n

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

i.e.

$$n = \frac{e}{1+e} \tag{2.17}$$

$$\text{or} \quad e = \frac{n}{1-n} \tag{2.18}$$

Fig. 2.2 (a) shows the three-phase diagram in which the volume of solids is shown as unity.

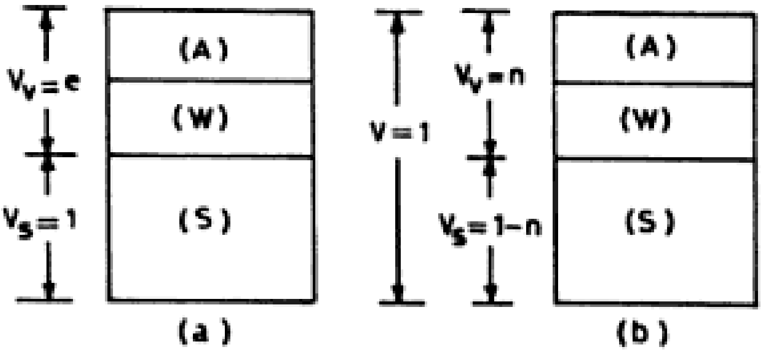


Fig. 2.2 Phase diagram.

$$V_s = 1; \text{ hence } V_v = e \text{ and } V = V_s + V_v = 1 + e$$

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

Fig. 2.2 (b) shows the phase diagram in which the total volume is taken as unity.

$$V = 1; \text{ hence } V_v = n \text{ and } V_s = V - V_v = 1 - n$$

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

It can be readily seen from Fig. 2.2 (a) that in a total volume of soil equal to (1 + e), the volume of solids is 1 and the volume of voids is e. Thus, if the total volume of a soil is V,

$$V_s = \frac{V}{1+e} \text{ and } V_v = \frac{e}{1+e} \cdot V$$

Relation between e, w, G_s and S

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

$$= \frac{V_v}{V_w} \times \frac{W_w/\gamma_w}{W_s/\gamma_s}$$

$$= \frac{V_v}{V_w} \cdot \frac{W_w}{W_s} \cdot \frac{G_s \gamma_w}{\gamma_w}$$

$$= \frac{1}{S} \cdot w\, G_s$$

$$\text{or} \quad e = \frac{wG_s}{S} \tag{2.19}$$

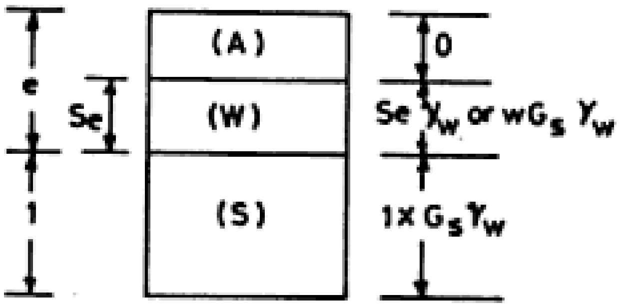


Fig. 2.3 Phase diagram.

Alternatively, from the phase diagram of Fig. 2.3,

$$V_s = 1;\ V_v = e;\ V_w = Se \text{ and}$$

$$W_s = V_s \times \gamma_s = 1 \times G_s \gamma_w;\ W_w = V_w \times \gamma_w = Se\,\gamma_w \text{ or } W_w = w \times W_s = w\,G_s\,\gamma_w$$

$$w = \frac{W_w}{W_s} = \frac{Se\,\gamma_w}{G_s\,\gamma_w} = \frac{Se}{G_s}$$

$$\text{or} \quad e = \frac{wG_s}{S}$$

For a completely saturated soil, $S = 1$; $e = w\,G_s$ (2.20)

Eq. 2.20 is frequently used to calculate the void ratio of a completely saturated soil.

Bulk Unit Weight, γ_t in terms of G_s, e, w and γ_w

$$\gamma_t = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_v} = \frac{W_s\,(1 + W_w/W_s)}{V_s\,(1 + V_v/V_s)}$$

Since $\frac{W_w}{W_s} = w,\ \frac{V_v}{V_s} = e \text{ and } \frac{W_s}{V_s} = \gamma_s = G_s\,\gamma_w,$

$$\gamma_t = \frac{G_s\,\gamma_w\,(1 + w)}{1 + e} \tag{2.21}$$

Since $w = \frac{Se}{G}$

$$\gamma_t = \left(\frac{G_s + Se}{1 + e}\right)\gamma_w \tag{2.22}$$

Alternatively, from the phase diagram of Fig. 2.3,

$$\gamma_t = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_v} = \frac{G_s\,\gamma_w + Se\,\gamma_w}{1 + e} \text{ or } \frac{G_s\,\gamma_w + w\,G_s\,\gamma_w}{1 + e}$$

or

$$\gamma_t = \left(\frac{G_s + Se}{1 + e}\right)\gamma_w = \frac{G_s\,\gamma_w\,(1 + w)}{1 + e}$$

Saturated Unit Weight, γ_{sat} in terms of G_s, e and γ_w

Fig. 2.4 shows the phase diagram for a saturated soil in which only solids and water are present.

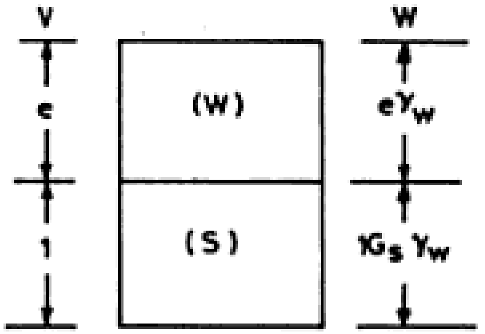


Fig. 2.4 Phase diagram (saturated soil)

$$\gamma_{sat} = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_v} = \frac{G_s\gamma_w + e\gamma_w}{1+e}$$

$$\text{or} \quad \gamma_{sat} = \left[\frac{G_s + e}{1+e}\right]\gamma_w \tag{2.23}$$

The same equation would result by substituting $S = 1$ in Eq. 2.22.

Dry Unit Weight, γ_d in terms of G_s, e and γ_w.

Fig. 2.5 shows the phase diagram for a dry soil in which only solids and air are present.

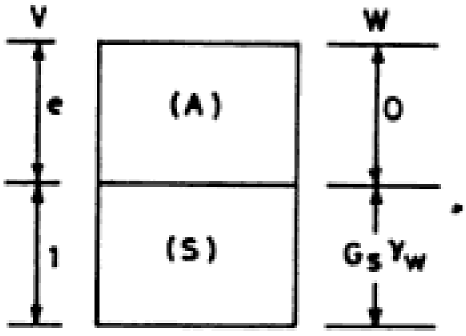


Fig. 2.5 Phase diagram (dry soil)

$$\gamma_d = \frac{W}{V} = \frac{W_s}{V} = \frac{W_s}{V_s + V_v}$$

$$\gamma_d = \frac{G_s\gamma_w}{1+e} \tag{2.24}$$

The same equation would result by substituting $S = 0$ in Eq. 2.22.

γ_d in terms of G_s, w, S and γ_w

From Eq. 2.19, $e = \frac{wG_s}{S}$

Substituting for e in Eq. 2.24,

$$\gamma_d = \frac{G_s\gamma_w}{1} + \frac{wG_s}{S} \qquad (2.25)$$

When the soil becomes fully saturated ($S = 1$) at a given water content, the dry unit weight for such a condition is given by the equation

$$\gamma_d = \frac{G_s\gamma_w}{1 + wG_s} \qquad (2.26)$$

'Zero air voids' unit weight given by Eq. 2.26 is found useful in the study of compaction behaviour of soils.

e in terms of γ_d, G_s and γ_w

From Eq. 2.24,

$$(1 + e)\,\gamma_d = G_s\gamma_w$$

$$\text{or} \quad e = \frac{G_s\gamma_w}{\gamma_d} - 1 \qquad (2.27)$$

Eq. 2.27 is often used to determine void ratio of an *insitu* soil deposit.

Submerged Unit Weight, γ' in terms of G_s, e and γ_w

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

$$= \left(\frac{G_s + e}{1 + e}\right)\gamma_w - \gamma_w$$

$$\text{or} \quad \gamma' = \left(\frac{G_s - 1}{1 + e}\right)\gamma_w \qquad (2.28)$$

Relation between γ_t, γ_d and w

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

$$\text{or} \quad \gamma_d = \frac{\gamma_t}{1 + w} \qquad (2.29)$$

Relation between γ_d, G_s, w and n_a

$$V = V_s + V_w + V_a$$

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

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

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

$$= \frac{W_s/G_s \gamma_w}{V} + \frac{W_w/\gamma_w}{V}$$

$$= \frac{\gamma_d}{G_s \gamma_w} + \frac{wW_s/\gamma_w}{V}$$

$$= \frac{\gamma_d}{G_s \gamma_w} + \frac{w\gamma_d}{\gamma_w}$$

$$= \frac{\gamma_d}{\gamma_w}\left(w + \frac{1}{G_s}\right)$$

or

$$\gamma_d = \frac{(1 - n_a)\, G_s \gamma_w}{1 + wG} \tag{2.30}$$

Eq. 2.30 expresses the relationship between dry unit weight and the percentage air voids. This will be useful in the study of compaction behaviour in soils.

γ_d when $n_a = 0$, that is, when the soil becomes fully saturated at a given water content, is given by the equation

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

which is the same as Eq. 2.26. In other words, γ_d when $n_a = 0$ and γ_d when $S = 1$ represent the same condition. Some of the useful relationships which were derived above are listed below for ready reference,

(i) $W_s = \dfrac{W}{1 + w}$

(ii) $V_s = \dfrac{V}{1 + e}$

(iii) $n = \dfrac{e}{1 + e}$

(iv) $e = \dfrac{wG_s}{S}$

(v) $$\gamma_t = \left(\frac{G_s + Se}{1+e}\right)\gamma = \frac{G_s(1+w)\gamma_w}{1+e}$$

(vi) $$\gamma_{sat} = \left(\frac{G_s + e}{1+e}\right)\gamma_w$$

(vii) $$\gamma_d = \left(\frac{G_s}{1+e}\right)\gamma_w$$

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

(ix) $$e = \left(\frac{G_s\gamma_w}{\gamma_d}\right) - 1$$

(x) $$\gamma_d = \left(\frac{\gamma_t}{1+w}\right)$$

(xi) $$\gamma_d = \left(\frac{G_s}{1+wG_s/S}\right)\gamma_w = \frac{(1-n_a)\,G_s\gamma_w}{1+wG_s}$$

2.4 WATER CONTENT DETERMINATION

This is a routine test as part of Atterberg limits, compaction test, consolidation tests, shear tests, etc. Water content of soil is an important soil parameter which influences the behaviour, particularly of cohesive soils, significantly. Water content of a sample can be determined by several methods. These are discussed in the following sections.

Oven-drying Method

This is the commonly adopted and the simplest method for the determination of water content of a soil sample in the laboratory. The method basically consists of drying a weighed moist sample of a soil in an oven at a controlled temperature (105 -110°C) for a period of 24 hours after which the dry weight of the sample is taken. The drying of soil is recommended at a temperature of 105 -110°C as temperatures higher than 110°C may break the crystalline structure of clay particles and result in the loss of chemically-bound water of crystallisation. Lower temperature (about 60°C) is recommended for organic soils to avoid oxidation of organic matter present in the samples. In the case of sandy soils, complete drying can be achieved in four to six hours, whereas clay samples require a longer duration (16 to 20 hours). However, in routine laboratory practice, samples are dried for 24 hours in the oven at temperatures 105 -110°C.

A clean, non-corrodible container is weighed accurately to 0.01 g. About 30-40 g of moist sample of soil is taken and weighed accurately after being placed in the container. The weighed container with the soil sample in it is placed in the oven for drying. After oven-drying, the container is placed in the desiccator for cooling. The container with the dried sample is then weighed. The water content is then calculated as shown below:

Let W_1 = weight of container

W_2 = weight of container with moist sample

W_3 = weight of container with dried sample

Weight of water, $W_w = W_2 - W_3$

Weight of solids, $W_s = W_3 - W_1$

$$\text{Water content, } w = \frac{W_2 - W_3}{W_3 - W_1} \times 100\ (\%) \tag{2.33}$$

Pycnometer Method

The oven-drying method in the laboratory takes a long time to determine the water content. A quick laboratory method is the pycnometer method. Pycnometer is an approximately 900 ml capacity glass bottle provided with a conical top. A conical cap provided with a 6 mm diameter hole at the top can be screwed on to the glass bottle. To avoid leakage of water, a rubber washer is provided between the conical cap and the rim of the bottle.

The method consists of weighing the empty dried pycnometer bottle complete with conical cap and washer. Let its weight be W_1 (Fig. 2.6). Next, 200 to 400 g of moist soil is taken in the pycnometer. The pycnometer with cap, washer and soil is weighed again (W_2). The pycnometer bottle is filled to about half with water and the contents are mixed thoroughly with a glass rod. A little more water is added to it and the washer and the cap are fixed. The pycnometer bottle is now filled with water upto the conical top. Sometimes, a vacuum pump is used to expel air entrapped in the soil, before filling the pycnometer with water. The pycnometer is dried from outside and weighed (W_3). Next, the pycnometer is emptied, cleaned thoroughly and weighed after filling it with water upto the top (W_4). The different weighed quantities have been shown in Fig 2.6.

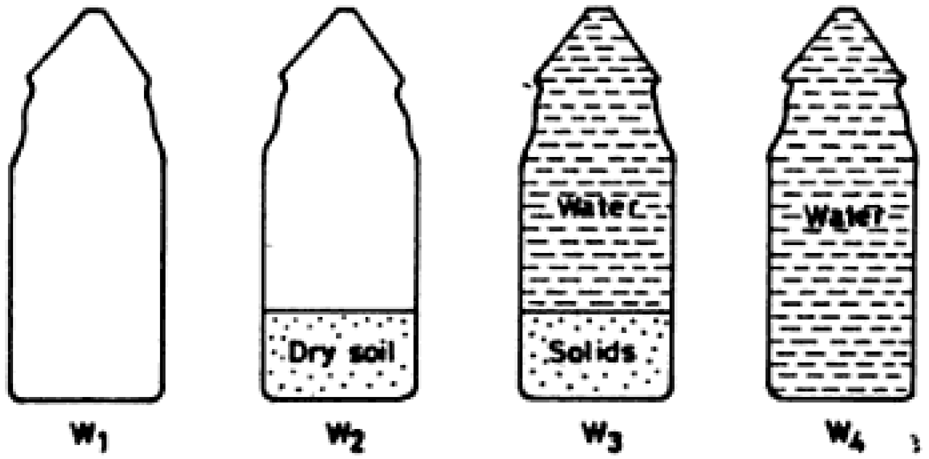


Fig. 2.6 Water content determination by pycnometer

$$\text{Water content, } w = \frac{W_w}{W_s} \times 100$$

Weight of water $= (W_2 - W_1) - W_s$

It can be seen that if from W_3 the weight of solids W_s could be removed and replaced by the weight of an equivalent volume of water, the weight W_4 would be obtained:

$$W_3 - W_s + \frac{W_s}{G_s \gamma_w} \cdot \gamma_w = W_4 \text{ and hence}$$

$$\text{Weight of solids, } W_s = (W_3 - W_4)\,\frac{G_s}{G_s - 1}$$

$$\text{Water content, } w = \left[\frac{(W_2 - W_1)}{(W_3 - W_4)}\left(\frac{G_s - 1}{G_s}\right) - 1\right] \times 100\,\% \tag{2.34}$$

In view of the difficulty in removing entrapped air from the soil sample, this method is more suited for cohesionless soils where this can be achieved easily.

Sand-bath Method

The sand-bath method of determining water-content is a quick, field method when the facility of an electric oven is not available. Wet soil sample is put in a container and dried by placing it on a sand-bath. The sand bath is heated over a kerosene stove. The wet soil sample dries quickly. The water content is then determined by using Eq. 2.33.

Rapid Moisture Meter Method

Rapid moisture meter is a portable equipment which can be conveniently used in the field as well, for the determination of moisture content. The instrument operates on the principle that calcium carbide, introduced in the weighed quantity of a sample (5 g) reacts with the free moisture in the sample and releases acetylene gas. The amount of gas released depends on the amount of free moisture in contact with the reagent. By confining the resultant gas in a sealed chamber, the gas pressure can be measured. The instrument is calibrated to interpret this pressure as the percentage of moisture in the sample on the basis of *total weight* of soil. The moisture content based on the dry weight can then be determined.

Torsion Balance Moisture Meter Method

Torsion balance moisture meter is another device for a quick determination of moisture content of soil samples in the laboratory. The balance works on a 220 volt 50 cycles single phase main supply. Infrared radiation which provides greater penetration of heat is used for drying the sample. The principle of operation is that the torsion wire is prestressed accurately to an extent equal to 100 per cent of the scale reading. Then the sample is evenly distributed upon the balance pan to counteract the prestressed torsion and the scale is brought back to zero. As the sample dries, the loss in weight is continuously balanced by the rotation of a drum calibrated directly to read moisture percentage on the *wet-weight* basis. The value is then modified to obtain moisture content on the dry weight basis.

2.5 SPECIFIC GRAVITY OF SOLIDS DETERMINATION

The specific gravity of solids (defined earlier) is frequently required for computation of several quantities such as void ratio, degree of saturation, unit weight of solids, and unit weights of soil in various states. It is determined using a pycnometer. The procedure involves weighing first an empty, dried pycnometer bottle, say, of weight W_1 (Fig. 2.7). Next, about 300 g of soil dried in oven and cooled in a desiccator, is placed in the pycnometer which is weighed again (W_2). The soil in the pycnometer is covered with water and stirred with a glass rod. The pycnometer is gradually filled with water carefully removing the entrapped air. Vacuum pump is also sometimes used to expel the entrapped air. The pycnometer with the soil is filled upto the top with water and weighed, (W_3). Finally, the pycnometer is emptied completely, cleaned, dried and weighed after filling it with water upto the top, W_4 (Fig. 2.7).

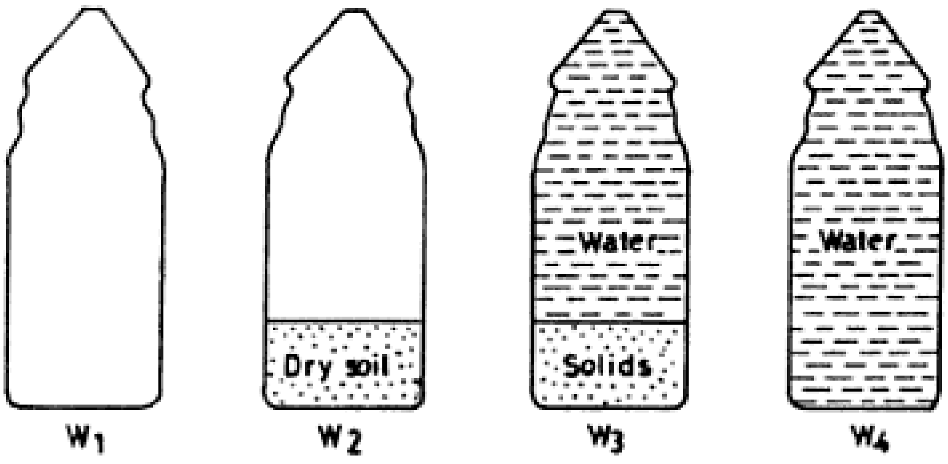


Fig. 2.7 Specific gravity determination

Weight of dry soil, $W_s = W_2 - W_1$

Weight of water in observation (iii) $= W_3 - W_2$

Weight of water in observation (iv) $= W_4 - W_1$

Weight of water having the same volume as that of solids $= (W_4 - W_1) - (W_3 - W_2)$

$= (W_2 - W_1) - (W_3 - W_4)$

$$\text{Specific gravity of solids, } G_s = \frac{W_2 - W_1}{(W_2 - W_1) - (W_3 - W_4)} = \frac{W_s}{W_s - W_3 + W_4} \tag{2.35}$$

Specific gravity values are usually reported at 27°C. If T°C is the test temperature, the specific gravity at 27°C is given by:

$$G_{s\,(27^\circ C)} = G_{s\,(T^\circ C)} \times \frac{\text{Specific gravity of water at T}^\circ\text{C}}{\text{Specific gravity of water at 27}^\circ\text{C}} \tag{2.36}$$

2.6 *IN SITU* UNIT WEIGHT DETERMINATION

Unit weight is necessary to compute overburden pressure at any depth within a soil.
The methods commonly used for the determination of *in situ* unit weight of a natural soil deposit or a compacted earth fill are:

(a) Core-cutter method

(b) Sand replacement method

(c) Water displacement method

Core-cutter Method

The core-cutter method consists of driving a core-cutter of known volume (1000 cc) into the soil after placing it on a cleaned soil surface. The core-cutter is usually provided with a 25 mm high dolly. The driving of the core-cutter is usually done by hitting the dolly mounted on top of the core-cutter with a suitable hammer. The cutter filled with soil is removed and the excess soil trimmed off. The cutter with the soil is weighed. The volume of the cutter is calculated from the dimensions of the cutter and the *in situ* unit weight is determined

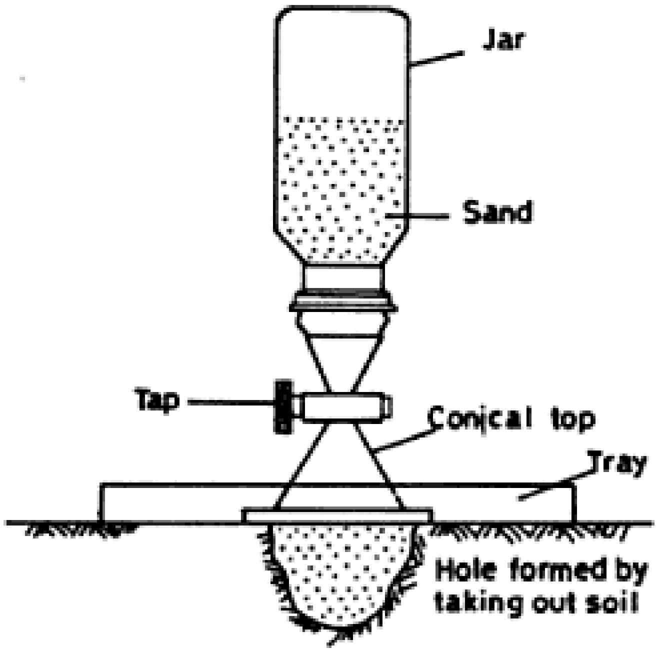


Fig. 2.8 Sand replacement method

by dividing the weight of the soil in the cutter by the volume of the cutter. If the water content of the soil in the cutter is determined in the laboratory, the dry unit weight of the soil can also be computed.

Sand Replacement Method

The core-cutter method cannot be used in the case of hard or gravelly soils. Under such situations, the sand replacement method is better suited. The sand replacement method consists of making a hole into the ground. The excavated soil is weighed. The volume of the hole is determined by replacement with sand. Knowing the weight of excavated soil and the volume of the hole, the *in situ* unit weight can be easily determined.

The site is cleaned and a square tray with a central hole in it is placed on the cleaned surface. A hole of diameter equal to the diameter of the hole in the tray and depth about 10-15 cm, is made in the ground. The excavated soil is collected in the tray and weighed. Next, a sand bottle (Fig. 2.8) about two-thirds full of clean, dry sand is weighed and placed upside down centrally over the hole. The tap is opened and the sand allowed to run to fill the excavated hole and the conical end. When no further flow of sand takes place, the tap is closed and the bottle with remaining sand is weighed. The bottle is then placed over a level surface and the weight of sand filling the cone of the sand bottle is noted. Thus, the weight of sand filling the excavated hole is computed. The unit weight of sand in the bottle is determined by pouring sand in a calibrating can of known dimensions and weighing the sand in the calibration can. Having computed the unit weight of sand in the bottle, and the weight of sand required to fill the excavated hole, the volume of the hole is determined. By dividing the weight of the excavated soil by its volume, the *in situ* unit weight of the soil is determined.

The water content of the excavated soil is also determined and the dry unit weight is worked out, using the relation $\gamma_d = \gamma_t/1 + w$.

In the case of bouldery soils, a large hole about 30 cm to 1 m diameter and about 30 cm to 1 m deep is excavated using a circular ring. The excavated soil is carefully collected and weighed. The volume of the hole is determined by pushing a thin polythene sheet bag inside the hole and filling it with water. The volume of water can be measured to give the volume of the hole which is used to calculate the unit weight.

Water Displacement Method

This method is suitable for cohesive soil only, where it is possible to have a lump sample. A small sample is trimmed to a regular shape from a larger sample brought from the field. The sample is weighed. Let the weight be W_1. The sample is then coated with a thin layer of paraffin wax. The sample coated with paraffin wax is weighed again (weight, W_2). A metal container with an overflow arrangement is taken. It is filled with water upto the brim and excess water is allowed to flow out through the overflow arrangement. The coated sample is then gradually lowered into the metal container and the overflow water is collected in a measuring jar. The measured volume of water, V_w is the volume of the displaced water. The volume of the uncoated soil specimen, V is calculated.

$$V = V_w - \left(\frac{W_2 - W_1}{\gamma_p}\right) \tag{2.37}$$

where γ_p = unit weight of paraffin wax

Thus, bulk unit weight of the soil, $\gamma_t = \dfrac{W_1}{V}$

Dry unit weight of the soil, $\gamma_d = \dfrac{\gamma_t}{1 + w}$

2.7 INDEX PROPERTIES OF SOILS

In nature, soils occur in a large variety. However, soils exhibiting approximately similar behaviour can be put together to form a particular group. The soils can also be put into major and minor groups. Various classification systems in practice place these soils in different categories based on certain properties of soils. The tests carried out in order to classify a soil are termed *classification tests.* The numerical results obtained on the basis of such tests are termed *index properties* of soils. The index properties of soils can be divided into two categories, namely, (i) soil grain properties, and (ii) soil aggregate properties. *Soil grain properties* are those properties which are dependent on the individual grains of the soil and are independent of the manner of soil formation. The properties in this category are mineralogical composition, specific gravity of solids, size and shape of grains. *Soil aggregate properties* are those properties which are dependent on the soil mass as a whole and, thus, represent the collective behaviour of a soil. Soil aggregate properties are influenced by soil stress history, mode of soil formation and the soil structure. Soil aggregate properties are of greater significance in engineering practice, since engineering structures are founded on undisturbed, natural soil deposits. The index properties are discussed in the following sections.

2.8 GRAIN SHAPE

The shape of soil grains is a useful soil grain property in the case of coarse-grained soils where it is important in influencing the engineering behaviour of these soils. The shape of grains in a coarse-grained soil can be examined with naked eyes, whereas fine-grained soils require microscopic examination. Though there is no universally accepted grain shape classification, the following general classification can be considered:

Bulky grains are soil grains where all the dimensions of a grain are more or less the same. These are characteristic of sand and gravel soils. These soils are formed by a mechanical break-down of parent rocks. At this stage, these bulky grains are angular. However, during their transportation by wind or water, the sharp edges of the grains may get worn out and the grains may become rounded. In between the two extremes of angular and rounded shapes are subangular and subrounded shapes. River gravels and wind blown sands are usually rounded whereas alluvial sands are subangular to subrounded. Fig. 2.9 shows the different shapes of bulky grains with varying angularity. Soils containing particles with high angularity tend to resist displacement and hence possess higher shearing strength compared to those with less angular particles.

The shape of the bulky grains is sometimes described in terms of their sphericity. Sphericity is defined as

$$S = \frac{D_e}{L} \tag{2.38}$$

in which D_e is the equivalent diameter of the particle (assuming it to be a sphere) and L the length of the particle. $D_e = \left(\frac{6V}{\pi}\right)^{1/3}$, where V is the volume of the particle.

Flaky grains or plate-shaped grains are the ones in which one dimension of grain, namely its thickness, bears no relationship with the other two lateral dimensions which are much bigger. As the name suggests, these resemble a sheet of paper, a leaf or a platelet. The behaviour of such grains is different from that of bulky grains; for example, during sedimentation, these particles do not settle vertically downwards but fall more like leaves drifting from a tree. Submicroscopic crystals of clay minerals usually exhibit this grain shape.

Needle-shaped grains are the grains in which one dimension of the grain is fully developed and is much larger than the other two. As these resemble needles, they are termed needle shaped grains. Such grains are characteristic of the clay mineral kaolinite.

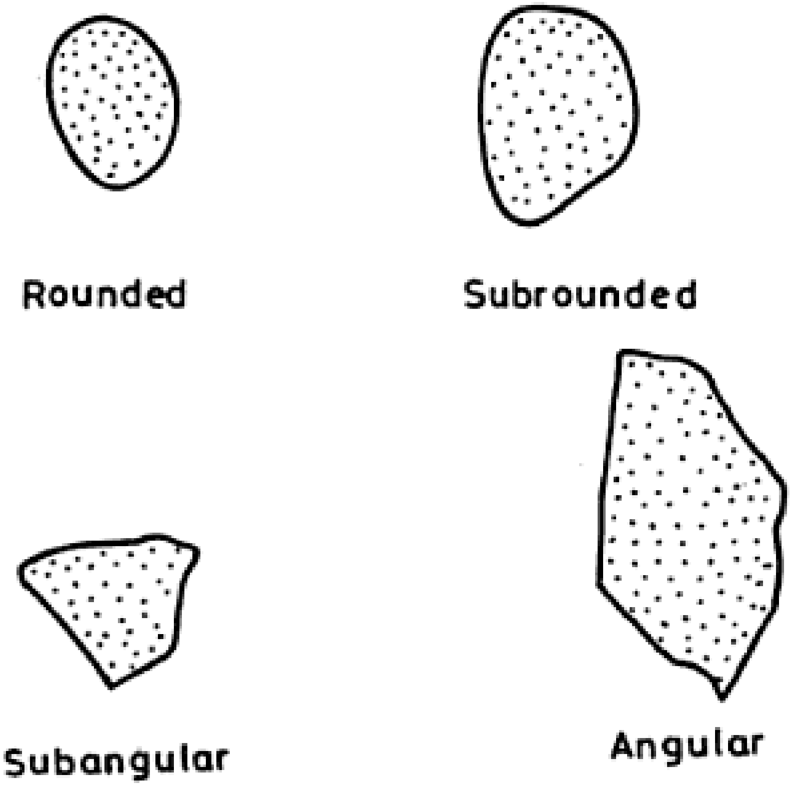


Fig. 2.9 Typical shapes of bulky particles

2.9 GRAIN-SIZE DISTRIBUTION

Grain-size distribution or the percentage of various sizes of soil grains present in a given dry soil sample, is an important soil grain property. Grain size analysis of coarse-grained soils is carried out by *sieve analysis*, whereas fine-grained soils are analysed by the *hydrometer method* or the *pipette method*. In general, as most soils contain both coarse and fine-grained constituents, a combined analysis is usually carried out. In the combined grain size analysis, a soil sample in the dry state is first subjected to sieve analysis and then the finer fraction is analysed by the hydrometer or pipette method.

Sieve Analysis

Sieves are wire screens having square openings. Different standards such as Indian, British and US, designate the sieves differently. According to the Indian Standard Code IS: 460-1962 (Revised), the sieve number is the mesh width expressed in mm for large sizes and in microns for small sizes; that is, a sieve with a mesh opening of 4.75 mm is designated as a 4.75 mm sieve and a 500 microns sieve refers to a sieve with a mesh opening of 0.500 mm. Sieves vary in size from 80 mm to 75 μ (Greek alphabet is used to represent micron). The representative soil sample is separated into two fractions by sieving through the 4.75 mm I.S. sieve. The fraction retained on this sieve (+ 4.75 mm) is called the *gravel fraction* which is subjected to coarse sieve analysis. A set of sieves of size 80 mm, 20 mm, 10 mm and 4.75 mm is required for further fractioning of gravel fraction. The material passing 4.75 mm sieve (– 4.75 mm) is further subjected to fine sieve analysis if it is sand or to a combined sieve and sedimentation analysis if silt and clay sizes are also present. The set of I.S. sieves for fine sieve analysis consist of 2 mm, 1 mm, 600 μ, 425 μ, 212 μ, 150 μ and 75 μ sieves. For a particular soil, all the sieves may not be required. Only such sieves are selected as would give a good grain size distribution curve. It must be mentioned here that when we say grain size or diameter of a particle which is far from being a sphere in shape, what we mean is an *equivalent diameter* of the particle as determined by the sieve analysis.

In the dry sieve analysis, a suitable quantity of pulverised dry soil of known weight (about 500 g) is taken and is sieved through a selected set of sieves arranged according to their sizes, with the largest apertured sieve at the top and the smallest apertured sieve at the bottom. A receiver is kept at the bottom and a lid is placed on the topmost sieve of the stack. The amount of shaking depends upon the shape and number of particles. However, ten minutes of shaking by a mechanical shaker is usually sufficient. The amount of soil retained on each sieve is weighed to the nearest 0.1 g. On the basis of the total weight of sample taken and the weight of soil retained on each sieve, the percentage of the total weight of soil passing through each sieve (also termed as per cent finer than) can be calculated as below:

$$\%\ \text{retained on a particular sieve} = \frac{\text{weight of soil retained on that sieve}}{\text{total weight of soil taken}} \times 100$$

Cumulative % retained = sum of % retained on all sieves of larger sizes and the % retained on that particular sieve

Percentage finer than the sieve under reference = 100 % - cumulative % retained

The finest sieve size used in the sieve analysis is 75 μ or 0.075 mm. If the portion passing 75 μ sieve is substantial, a wet analysis is carried out. The sample is first washed over the 75 μ sieve to remove the fine particles sticking to the sand particles. The wet sand retained on the 75 μ sieve is dried in an oven and the dry sieve analysis carried out in the usual manner while the finer fraction is utilised for the hydrometer or pipette analysis.

Sedimentation Analysis

The sedimentation analysis is most convenient for determining the grain size distribution of the soil fraction finer than 75 μ in size. The analysis is based on *Stokes's law*, according to which the velocities of free fall of spherical, fine particles, through a liquid are different for different sizes. In the case of soils, though the grains are of different shapes, it is assumed that they are spherical and have the same specific gravity (average specific gravity). If a single sphere is allowed to fall freely through a liquid of infinite extent, its velocity will first increase rapidly under the action of gravity, but a constant velocity called the *terminal velocity* is reached within a short time. According to Stokes's law, the terminal velocity, v is given by

$$v = \frac{\gamma_s - \gamma_l}{18\eta} D^2 \tag{2.32}$$

where γ_s = unit weight of soil grains (g/cm^3)

γ_l = unit weight of the liquid (g/cm^3)

η = viscosity of the liquid (g-s /cm^2)

$= \dfrac{\mu}{g}$

μ = viscosity in absolute units of dyne s/cm^2 or poise

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

D = diameter of grain (cm)

Usually, water is the medium of suspension. Thus, γ_w can be taken as 1 g/cc. Also, in cgs units, $\gamma_s = G_s\,\gamma_w = G_s$. Eq. 2.32 is, thus, modified to Eq. 2.33 when the grain diameter is expressed in mm.

$$v = \frac{G_s - 1}{1800\,\eta} D^2 \tag{2.33}$$

At 20°C, the viscosity (μ) of water is approximately 0.01 poise. Taking an average value of specific gravity G_s as 2.67, and substituting the values in Eq. 2.33,

$$v = \frac{(2.67 - 1)}{1800} \times \frac{980.7}{0.01} D^2$$

i.e.,

$$v = 90.98\, D^2 \tag{2.34}$$

where D is in mm and velocity v is in cm/s.

Eq. 2.34 is convenient to remember and may be considered as a simplified version of Stokes's law. Thus, for a particle of diameter 0.06 mm, $v = 0.3275$ cm/s and the time required for fall through a height of 10 cm is 30.5 s. The viscosity of water in absolute cgs units (poise) is given in Table 2.2.

Table 2.2 Viscosity of Water (Values are in millipoises)

°C	*0*	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
0	17.94	17.32	16.74	16.19	15.68	15.19	14.73	14.29	13.87	13.48
10	13.10	12.74	12.39	12.06	11.75	11.45	11.16	10.88	10.60	10.34
20	10.09	9.84	9.61	9.38	9.16	8.95	8.75	8.55	8.36	8.18
30	8.00	7.83	7.67	7.51	7.36	7.31	7.06	6.92	6.79	6.66
40	6.54	6.42	6.30	6.18	6.08	5.97	5.87	5.77	5.68	5.58

1 dyne sec per sq cm = 1 poise; 1 gram sec per sq. cm = 980.7 poises; 1 poise = 1000 millipoises

For a material of density approximating that the soil grains, Stokes's law is applicable for spheres of diameter between 0.2 mm and 0.0002 mm (Taylor, 1948). It may be noted that spheres of diameter larger than 0.2 mm falling through water cause turbulence whereas for spheres with diameter less than 0.0002 mm, Brownian movement takes place and the velocity of settlement is too small for accurate measurement. In both cases, Stokes's law is not applicable.

Further, the application of Stokes's law to grain size analysis has the following limitations:

(a) The analysis is based on the assumption that the falling grain is spherical. In soils, the finer particles are never truly spherical. They are usually flaky or needle shaped. The grain size thus measured is the *equivalent diameter* of a sphere which will have the same rate of fall as the actual soil grain.

(b) Stokes's law considers the velocity of free fall of a single sphere in a suspension of infinite extent, whereas the grain size analysis is usually carried out in a glass jar in which the extent of liquid is limited. Also, the rate of fall is influenced by the presence of other particles in the suspension. However, it has been shown that for concentrations of 50 g/ litre or less, the influence of particles on each other is not appreciable.

(c) An average value of specific gravity of grains is used, though the specific gravity of some of the grains may be different from the average value. This, is however, not of significance in routine soil testing.

(d) The finer grains of the soil carry charge on their surface and have a tendency for *floc* formation. A floc is an accumulation of small particles. The flocs then settle. Thus, if the tendency to floc formation is not prevented, the diameter measured will be the diameter of the floc and not of the individual grain. In the test, therefore, the soil is treated with some deflocculating agent such as sodium hexametaphosphate or sodium oxalate in order to prevent the formation of flocs.

Hydrometer Analysis

Fine-grain analysis is carried out by the hydrometer method or the pipette method. The principle of the test in both the methods is the same. The difference lies only in the method of making the observations. In the pipette method, the weight of solids per cc of suspension is determined directly by collecting 10 cc of soil suspension from a specified sampling depth whereas in the hydrometer method, the weight of solids present at any time is calculated indirectly by reading the density of soil suspension. For the analysis of results, it is first essential to calibrate the hydrometer, that is, to establish a relation between the hydrometer reading R_H on the stem and the *effective depth* H_e for a given hydrometer. The effective depth is the distance from the surface of the soil suspension to the level at which the density of soil suspension is being measured. The effective depth of the hydrometer keeps on increasing as the particles settle with time.

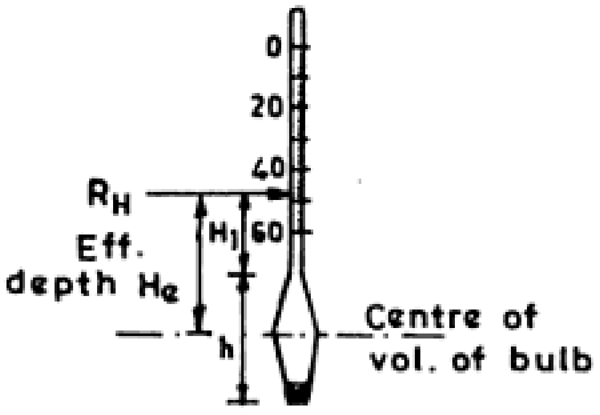


Fig. 2.10(a) Hydrometer

A stream lined hydrometer [Fig. 2.10 (a) and (b)] is used in the analysis. The reading on the graduated stem of the hydrometer provides the density of the soil suspension at the centre of the bulb (*BB*) at any instant of time. As the specific gravity of the soil suspension is close to unity, it is the usual practice to subtract 1.0 from the specific gravity and multiply the balance by 1000. For example, a reading of 25 on the hydrometer stem means a specific gravity of 1.025. The hydrometer stem is graduated with increasing values towards the bulb [Fig. 2.10(a)].

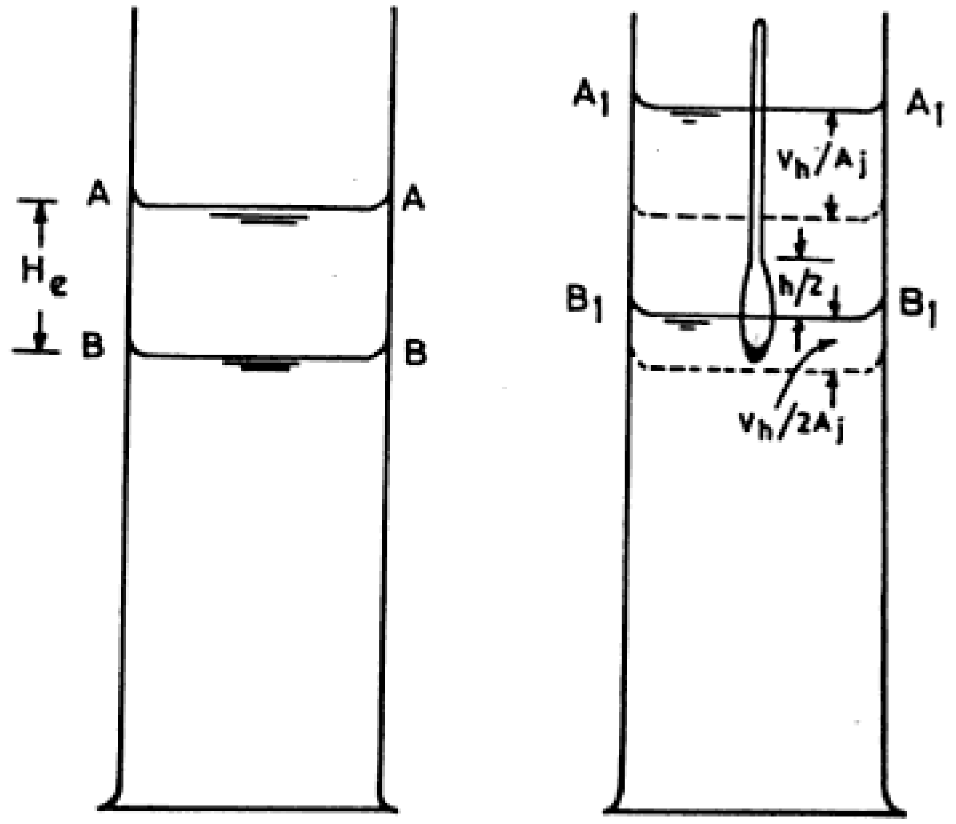


Fig. 2.10(b) Immersion correction

Let h be the length and V_H the volume of the hydrometer bulb and H_1 the distance in cm between any hydrometer reading R_H and the neck. As the hydrometer is immersed in the jar, the water level in the jar rises. If A_j is the area of cross-section of the jar, the surface of soil suspension rises by V_H/A_j, whereas the centre of the hydrometer bulb (*BB*), the rise in level is $V_H/2A_j$ (Fig. 2.10 b), if the volume of the hydrometer below the centre of the bulb is taken as approximately one-half of the total volume. Hence, the effective depth H_e below the original surface *AA* is given by equation.

$$H_e = \left(H_1 + \frac{h}{2} + \frac{V_H}{2A_j}\right) - \frac{V_H}{A_j}$$

$$= H_1 + \frac{1}{2}\left(h - \frac{V_H}{A_j}\right) \qquad (2.35)$$

For a particular hydrometer, knowing the length and volume of hydrometer bulb, the values of effective depth can be obtained for different values of hydrometer reading R_H. A graph is plotted between the hydrometer reading R_H and the effective depth, H_e. This is called the calibration plot. Such a calibration plot would take care of the effect of immersion of hydrometer.

In the hydrometer analysis, 50 g of the soil sample passing through the 75 micron sieve is taken. The sample is put in an evaporating dish and is covered with 100 cc of distilled water. About 100 cc of deflocculating agent (*e.g.*, sodium oxalate) is added and the sample allowed to soak for five minutes. The sample is then transferred into a dispersion cup. The cup is filled three-fourths using distilled water. After the suspension is stirred for 10 minutes, the suspension is transferred to a 1000 cc jar of uniform cross-section. More water is added and the total volume brought to 1000 cc. The suspension in the jar is mixed thoroughly by firmly placing the palm of the hand on the open end and turning the jar upside down and back. When the suspension is well mixed, the jar is placed on the table and the hydrometer is inserted. A stop watch is started immediately and the reading of the hydrometer taken after 1/4,1/2,1 and 2 minutes. The hydrometer is then taken out of the suspension. The soil suspension is again mixed thoroughly and the stop watch is restarted. Readings of hydrometer are then taken after 5,10,15,30 min, 1,2 and 4 hrs, etc. About 30 seconds before each hydrometer reading, the hydrometer is slowly inserted in the suspension so that it is stable by the time the reading is due. The hydrometer is taken out as soon as the reading is taken, with as little disturbance as possible.

A few corrections are also to be applied to the hydrometers reading. As the suspension is opaque, the hydrometer reading corresponds to the upper level of the meniscus. A correction due to meniscus ($+C_m$) is then applied to the hydrometer reading. The hydrometers are generally calibrated at 27°C. In case the temperature of the suspension is other than 27°C, a correction due to temperature ($+C_t$) is to be applied to the observed hydrometer readings. If the test temperature is above the standard, the correction is added and, if below, it is subtracted. Temperature correction chart is usually supplied by the manufacturer. The correction due to rise in specific gravity of the suspension on account of the addition of the deflocculating agent is called the dispersing agent correction (C_d). C_d is always negative. Thus, the corrected hydrometer reading R_c is given by:

$$R_c = R_H + C_m \pm C_t - C_d$$

where R_H is the observed hydrometer reading. R_c is to be used in the calibration graph to obtain the effective depth H_e.

According to Stokes's law,

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

or

$$D = \sqrt{\frac{18\eta v}{\gamma_s - \gamma_w}} \qquad (2.36)$$

If a soil particle of diameter D falls through a height H_e (cm) in time t (min)

$$v = H_e/60\,t \text{ (cm/s)} \tag{2.37}$$

or

$$D \text{ (cm)} = \sqrt{\frac{18\eta}{\gamma_s - \gamma_w} \times \frac{H_e \text{ (cm)}}{60\,t \text{ (min)}}} \tag{2.38}$$

At the start of the sedimentation, if the soil suspension is thoroughly shaken, the soil particles are uniformly dispersed and the concentration of particles of different sizes is the same at all depths. After a certain time elapse, at a given depth, only those particles remain which have not settled during this time. Since all particles of the same size have the same velocity, particles of a given size, if they are present at all at any depth, will be in the same concentration as at the start of the sedimentation. This means that all particles smaller than a particular size D will be present at a depth H_e in the same degree of concentration as at the beginning. All particles larger than size D would have settled below that depth. The greater the time elapse, the finer the particle sizes retained at that depth. Hence, sampling at different time intervals would give the content of particles of different sizes. Thus, the percentage of particles finer than D is equal to

$$\frac{\text{Weight of solids per cc at depth } H_e \text{ after time } t}{\text{Weight of solids per cc in the original suspension}} \times 100$$

$$= \frac{W_d}{W} \times 100 \tag{2.39}$$

If R_c is the corrected hydrometer reading, the specific gravity of soil suspension is equal to $1 + \frac{R_c}{100}$. It is also equal to the right hand side of the equation below:

$$1 + \frac{R_c}{1000} = \frac{\text{wt of water/cc} + \text{immersed wt of solids/cc}}{\text{wt of water/cc}}$$

$$= 1 + \text{immersed wt of solids / cc}$$

$$= 1 + W_d \left(\frac{G_s - 1}{G_s}\right)$$

$$\therefore\ W_d = \left(\frac{G_s}{G_s - 1}\right) \frac{R_c}{1000} \tag{2.40}$$

The grain size D corresponding to elapsed time t and effective depth H_e can be determined from Eq. 2.38. The percentage of soil particles finer than D, denoted by N is determined by substituting W_d obtained from Eq. 2.40 in Eq. 2.39.

Thus,

$$N = \left(\frac{G_s}{G_s - 1}\right) \frac{R_c}{10W} \ (\%)$$

If weight of soil grains is W_s (g) in a volume of soil suspension of 1000 cc, $W = \frac{W_s}{1000}$. Hence,

$$N = \left(\frac{G_s}{G_s - 1}\right) \frac{R_c}{W_s} \times 100\ \% \tag{2.41}$$

Several types of nomographic charts for the solution of Stokes's equation have been developed. One such chart is due to Casagrande and is reported by Lambe (1951).

In routine soil testing, about 50 g of dry soil is usually taken for suspension of 1000 cc in jars of constant cross-sectional area. A hydrometer can also be calibrated directly in terms of grams of soil particles finer than a given grain size in one litre of soil suspension. This kind of a direct reading hydrometer is called a *Bouyucos hydrometer.*

Grain-Size Distribution Curves

The result of a grain-size analysis are usually represented in the form of a graph as shown in Fig. 2.11. The aggregate or cumulative weight, as a percentage of the total weight, of all grains smaller than any given diameter is plotted on the ordinate using an arithmetic scale; the size of the soil particle, in mm, is plotted on the abscissa using a logarithmic scale. In view of the very large range of particle sizes, a logarithmic scale becomes necessary. Further, a semilogarithmic plot has the merit that soils of equal uniformity exhibit the same shape of grain size curves, irrespective of the particle size fractions present in the soil.

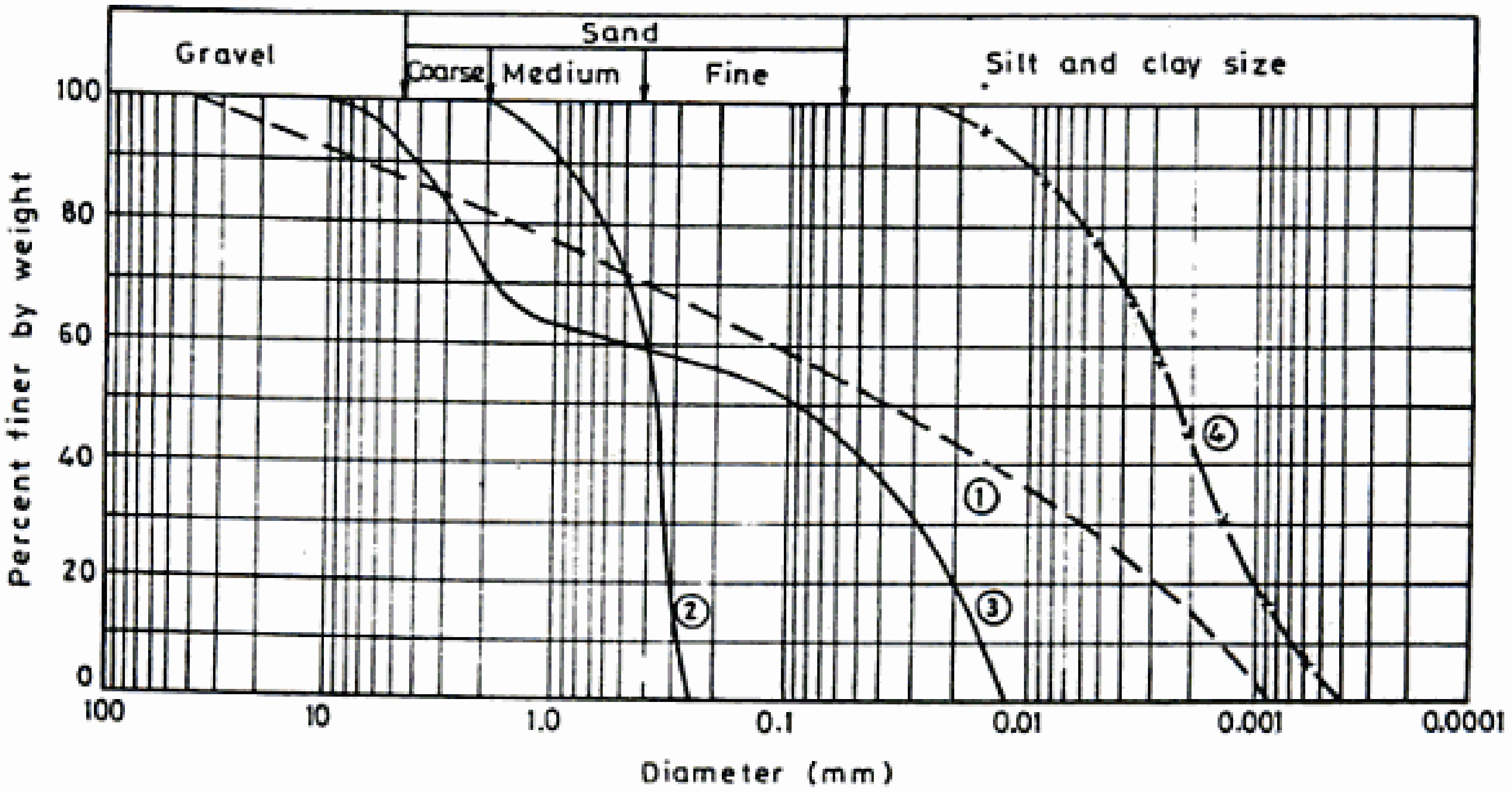


Fig. 2.11 Grain-size distribution curves

Typical results of a mechanical analysis are shown in Fig. 2.11. The *position* and the *shape* and *slope* of a curve indicate the *type* and *gradation* of the soil. Though the smaller grain sizes go toward the right in Fig. 2.11, it could just as well be plotted the other way also. The range of particle sizes for gravel, sand and silt-clay sizes (fines) are fitted on the abscissa with sand size being further divided into coarse (C), medium (M) and fine (F) fractions on the basis of I.S. grain-size classification.

A *well-graded* soil has a good representation of grain sizes over a wide range and its gradation curve is smooth. Curve 1 represents a well-graded soil with particles ranging from gravel to fines. On the other hand, a *poorly-graded* soil has either an excess or a deficiency of certain particle sizes or has most of the particles about the same size. In the latter case, the soil is also called *uniformly graded.* Curve 2 represents a poorly-graded sand of *uniform gradation.* A *gap-graded* soil is the one in which some of the particle sizes are missing. Curve 3 is an example of such a soil in which the proportion of grain sizes between 0.1 and 1 mm is relatively low.

For the coarse-grained soils, the range of particle diameters present in the sample is of interest. In addition, certain grain diameters D which correspond to a certain per cent finer than on the grain-size distribution curve (GSD) are determined.

The diameter D_{10} corresponds to 10% of the sample finer in weight on the GSD curve. The diameter D_{10} is called the *effective size.*

The *coefficient of uniformity* C_u is a shape parameter and is defined as

$$C_u = \frac{D_{60}}{D_{10}} \tag{2.41}$$

where D_{60} = grain diameter (mm) corresponding to 60% finer than. Another shape parameter that is often used is the *coefficient of curvature* C_c defined as

$$C_c = \frac{D_{30}^2}{D_{10} \times D_{60}} \tag{2.42}$$

where D_{30} = grain diameter (mm) corresponding to 30% finer than. For a soil to be well-graded, C_c must lie between 1 and 3 and in addition to this, C_u must be greater than 4 for gravels and greater than 6 for sands.

2.10 CONSISTENCY OF CLAYS : ATTERBERG LIMITS

Consistency is a term which is used to describe the degree of firmness of a soil in a qualitative manner by using descriptions such as soft medium, firm, stiff or hard. It indicates the *relative ease with which a soil can be deformed.* In practice, the property of consistency is associated only with fine-grained soils, especially clays.

The physical properties of clays are considerably influenced by the amount of water present in them. Depending upon the water content, the following four stages or states of consistency are used to describe the consistency of a clay soil : (i) the liquid state; (ii) the plastic state; (iii) the semi-solid state, and (iv) the solid state. The boundary water contents at which the soil undergoes a change from one state to another are called "*consistency limits*". In 1911, a Swedish soil scientist, Atterberg, first demonstrated the significance of these limits. Hence, they are also known as the *Atterberg Limits.* These limits of water content, though empirical in nature, are of great significance in understanding the behaviour of clays.

When a fine-grained soil is mixed thoroughly with a large quantity of water, the resulting suspension is in a *liquid state,* and offers practically *no resistance to flow.* In other words, the soil has virtually no shear strength. If the water content of the suspension is gradually reduced keeping the consistency of the sample uniform, a stage comes when it just begins offering resistance to flow. This is the stage when the sample changes from possessing no shearing strength to having an infinitesimal shear strength and changes from the liquid to the plastic state. The boundary water content between the liquid state and the plastic state is called the *liquid limit* (Fig. 2.12). In the *plastic state, the soil can be moulded to different shapes without rupturing* it, due to its *plasticity.*

If the water content is further reduced, the clay sample changes from the plastic state to the semi-solid state at a boundary water content which is called the *plastic limit.* In the *semi-solid state, the soil does not have plasticity; it becomes brittle.* When pressure is applied, the soil simply crumbles.

Upto the semi-solid state, the soil remains fully saturated and any reduction in the volume of water will result in an almost equal reduction in the volume of the soil mass (Fig. 2.12). A further reduction in the water content, however, brings about a state when with a decrease in moisture, the volume of the soil mass does not decrease any further but remains the same; the sample changes from the semi-solid to the solid state. The boundary water content is called the *shrinkage limit.* Below this limit, the sample begins to dry up at the surface and the soil is no longer fully saturated. The colour of the sample also begins to change (Fig. 2.12).

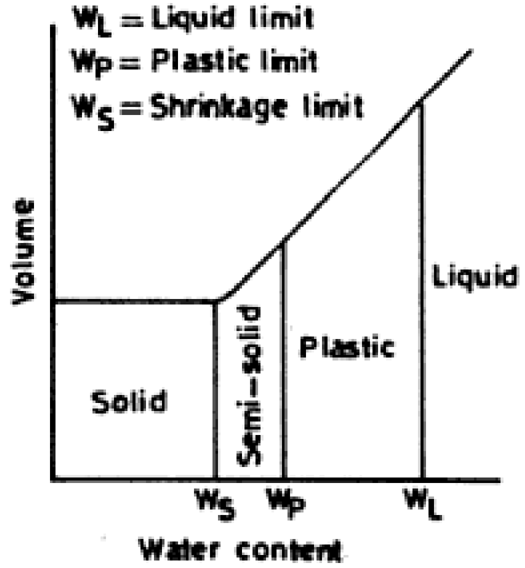


Fig. 2.12 Atterberg limits

The limits described above are all expressed in percentages of water content. The different states of consistency through which a soil mass passes through, with the decrease in water content, is depicted in Fig. 2.13.

State	liquid	plastic	semi solid	solid (brittle)
Water content w_1 %		w_L (LL)	w_P (PL)	w_S (SL) 0
Consistency	liquid	very soft to stiff	very stiff to very hard	
Saturation	S = 100 %			100 — 0 %
Volume	Volume-decreases			Constant-volume

Fig. 2.13 Different states, consistencies and Atterberg limits

In 1932, A. Casagrande devised standardised testing procedures to determine the Atterberg limits.

Liquid limit, w_L *is the* water content at which a soil is practically in a liquid state, but *has infinitesimal resistance against* flow which can be measured by any standardised procedure. With reference to one such standard procedure, the *liquid limit* is defined as the water content at which a groove cut in a pat of soil by a grooving tool of standard dimensions will flow together for a distance of 13 mm under the impact of 25 blows in a standard liquid limit device.

Plastic limit, w_P is defined as the water content at which a soil would just begin to crumble when rolled into a thread of approximately 3 mm diameter.

Shrinkage limit, w_S is the *maximum water content at which a decrease in moisture content does not cause any decrease in the volume of the soil mass. It is also the smallest value of water content at complete saturation of the soil mass.*

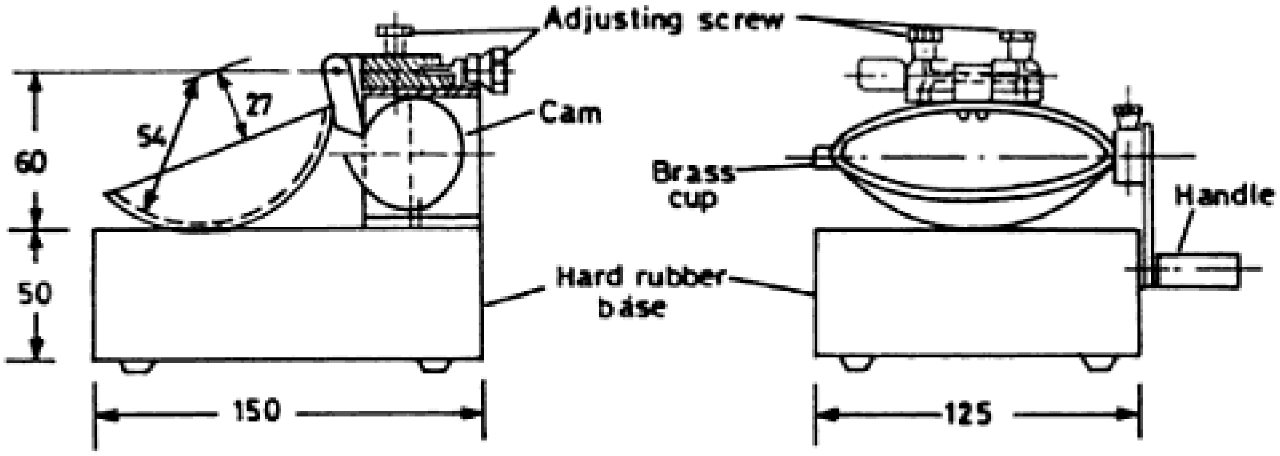
Determination of Liquid Limit

In the laboratory, the liquid limit of a soil is determined with the standard liquid limit apparatus [Fig. 2.14 (a)] designed by A. Casagrande. The apparatus consists of a vulcanised rubber compound base with a brass cup suitably mounted. The brass cup can be raised and made to fall on the rubber base through a cam arrangement operated by a handle. The height of fall of the brass cup can be adjusted with the help of an adjusting screw. Before the start of the test, the height of the fall of the cup is adjusted to 10 mm. A grooving tool is used to cut a groove in the pat of soil placed in the cup. Two types of grooving tools are in use [Fig. 2.14 (b)]. These are: (a) Casagrande tool which cuts a groove of 2 mm width at the bottom, 11 mm width at the top and 8 mm deep, and (b) ASTM tool which cuts a groove 2 mm wide at the bottom, 13.6 mm wide at the top and 10 mm deep. The selection of the grooving tool depends upon the type of soil. For a soil of low plasticity, the ASTM grooving tool is preferred.

About 120 g of air-dried soil passing through IS sieve 425 micron is taken and mixed with water such that the soil attains a putty-like consistency. A portion of the paste is placed in the cup and is levelled so as to have a maximum depth of about 10 mm. A groove is cut in the soil placed in the cup, using the grooving tool. In cutting the groove, the grooving tool is drawn through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup. The handle is rotated at the rate of 2 revolutions per second and the number of blows necessary to close the groove for a distance of 13 mm is noted. The groove should close by flow and not by slippage of soil. Fig. 2.14 (c) shows the soil pat just after the groove is made and the soil pat when the groove is closed after the test. Then, about 10 g of soil near the closed groove is taken to determine its water content. By altering the water content of the soil and repeating the operations mentioned above, four to five readings of water content in the range of 10 to 40 blows are obtained. A graph is then plotted between number of blows, N on a logarithmic scale and the water content, w on a natural scale (Fig. 2.15). It will be seen that the semi-logarithmic plot is a straight line called the *flow curve.*

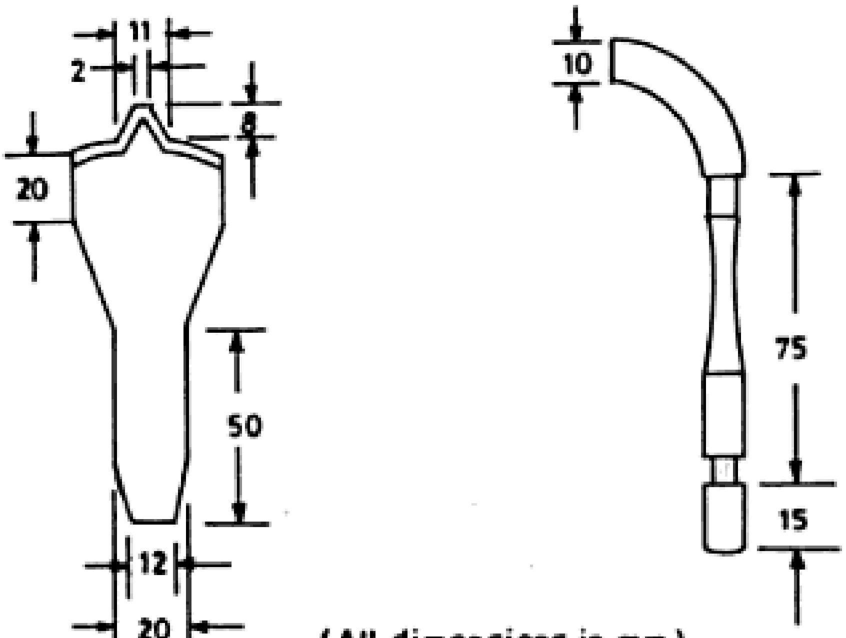
The liquid limit is determined by reading the water content corresponding to 25 blows on the flow curve. From the definition of liquid limit, *all soils possess the same value of shearing strength at liquid limit.* It is about 27 g/cm^2.

Attempts have also been made to develop empirical relations which enable the determination of liquid limit from only one observation (IS:2720 Part V-1970). In this method, the following equation is used:



(All dimensions in mm)

(a) Liquid limit apparatus



(All dimensions in mm)

(i) Casagrande tool (ii) ASTM tool

(b) Grooving tools

(i) Soil pat before test (ii) Soil pat after test

(c) Soil pat in cup

Fig. 2.14 Liquid limit apparatus and grooving tools

$$w_L = w_N \left(\frac{N}{25}\right)^{e} \tag{2.43}$$

where w_N is the water content at which N blows are required for the groove to close and e an index which varies from 0.068 to 0.121.

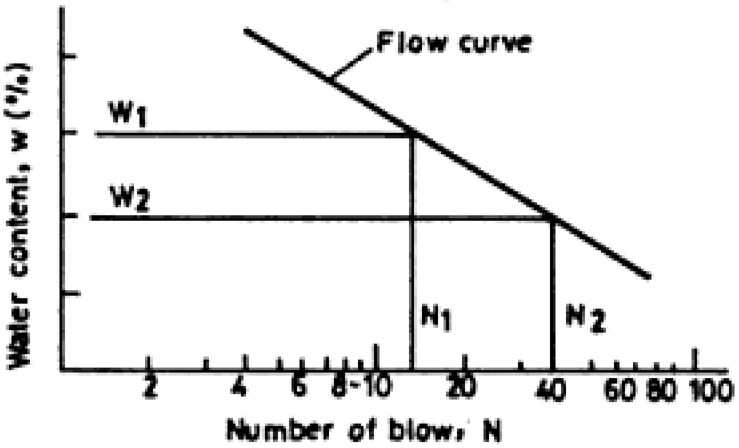


Fig. 2.15 Flow curve

A median value of 0.10 can be assumed for e when the number of blows N is in the range of 20-30.

The one-point method can be used only for an approximate estimate of the liquid limit. Standard small-sized laboratory cones have also been used for the determination of liquid limit. Indian Standard IS:2720 (Part V) -1970 describes the cone penetrometer method for liquid limit determination.

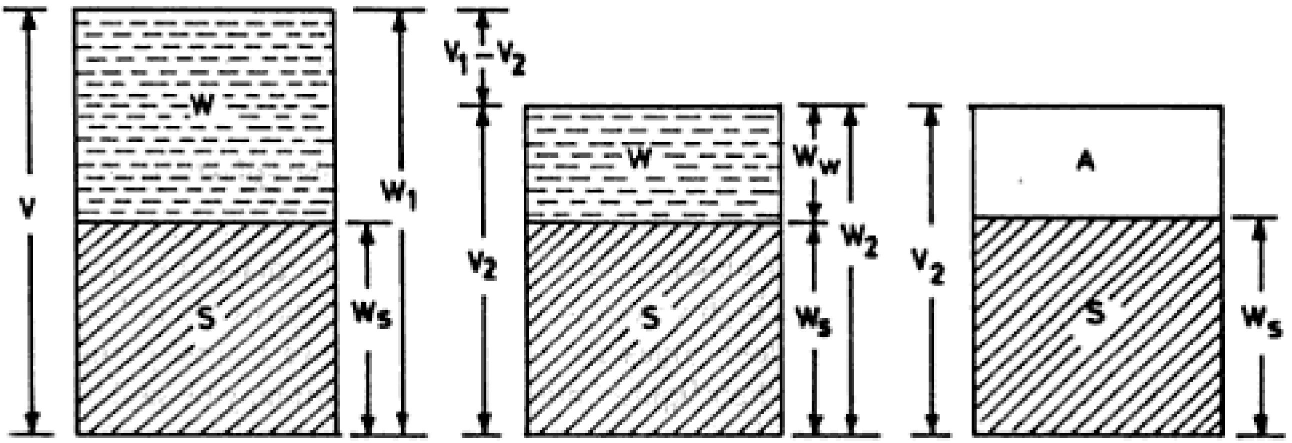
Plastic Limit Determination

About 15 g of air-dried soil passing through IS sieve 425 micron is taken for plastic limit determination and is mixed with a sufficient quantity of water which would enable the soil mass to become plastic enough to be easily shaped into a ball. A portion of the ball is taken and rolled on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length. When a diameter of 3 mm is reached, the soil is remoulded into a ball. The process of making the thread and remoulding is continued till the thread at a diameter of 3 mm, *just* starts crumbling. Some of the crumbled portion of thread is kept in the oven for water content determination. The test is repeated twice with fresh samples. The average of the three values of water contents is taken as the plastic limit.

Shrinkage Limit Determination

About 30 g of soil passing through IS: 425 micron sieve is taken in an evaporating dish. The soil is mixed with sufficient quantity of water to bring the soil to a consistency that it may flow. The soil mixture is placed in the shrinkage dish in three equal quantities so as to fill the dish. The excess soil is removed and the dish is weighed with soil. The soil pat is allowed to dry in the air till the colour of the pat changes from dark to light. The dish is then placed in the oven at 110°C till its weight becomes constant. The shrinkage dish is weighed with the dry sample and the dry weight of soil is determined. The volume of dry pat of soil is measured by the *mercury displacement method.*

The shrinkage limit is calculated as shown below. Figure 2.16 (a) represents the soil sample in plastic state which fills the container of known volume V_1 and has a weight W_1. As the sample is gradually dried, the water content at a certain stage becomes equal to the shrinkage limit [Fig. 2.16 (b)]. At this point, the volume decreases to V_2, the corresponding weight being W_2. The sample is still in a saturated state. Beyond the shrinkage limit, the sample continues to dry with no further volume decrease, until it reaches the



(a) Original soil pat

(b) Soil pat at shrinkage limit

(c) Dry soil pat

Fig. 2.16 Shrinkage limit determination

dry state represented by Fig. 2.16 (c). At this stage, the dry weight of the soil sample W_s and its volume V_2 are measured.

The shrinkage limit, w_S is the water content at Fig. 2.16 (b)

i.e. $$w_S = W_w / W_s.$$

Weight of water in Fig. 2.16 (b) = $(W_1 - W_s) - (V_1 - V_2)\gamma_w$

$$w_S = \frac{(W_1 - W_s) - (V_1 - V_2)\,\gamma_w}{W_s} \times 100 \tag{2.44}$$

The observations from shrinkage limit test can also be used to determine the specific gravity of solids, though the method is only approximate.

Unit weight of solids, $$\gamma_s = G_s\gamma_w = \frac{W_s}{V_s}$$

From Fig. 2.16 (a),

$$V_s = V_1 - \left(\frac{W_1 - W_s}{\gamma_w}\right)$$

Thus, $$G_s = \frac{W_s}{V_1\gamma_w - (W_1 - W_s)}$$

$$= \frac{W_s}{V_1\gamma_w - W_1 + W_s} \tag{2.45}$$

Once the specific gravity of solids is known, the shrinkage limit of the soil can also be determined from known values of W_s and V_2 (Fig. 2.16)

Shrinkage limit, $w_S = \dfrac{W_w}{W_s}$, in which

$$W_w = (V_2 - V_s)\gamma_w \text{ (Fig. 2.16 (b))}$$

$$= \left(V_2 - \frac{W_s}{G_s \gamma_w}\right)$$

Plasticity index, I_P is the range of moisture content over which a soil exhibits plasticity. It is the numerical difference between the liquid limit and the plastic limit.

Plasticity index indicates the degree of plasticity of a soil. Greater the plasticity index, greater the plasticity of the soil. Clay soils possesssing high values of liquid limit and plasticity index are referred to as *highly plastic* or *fat clays* and those with low values described as *lean clays.* Coarse grained soils cannot achieve the plastic state of consistency and their liquid limit and plastic limit may be said to coincide; that is, $I_P = 0$. When the liquid limit or the plastic limit cannot be determined, the plasticity index is reported as NP (*non-plastic*). When the plastic limit is equal to or greater than the liquid limit, I_P is reported as zero.

Soils are classified according to their plasticity index values as shown in Table 2.3.

Table 2.3 Soil Classification Related to Plasticity Index

Plasticity Index, I_P	*Soil description*
0	Non-plastic
< 7	Low plastic
7 – 17	Medium plastic
> 17	Highly plastic

$$I_C = \frac{w_L - w_N}{I_P} \tag{2.46}$$

Relative consistency, I_c or *consistency index,* is defined as the ratio of the difference between the liquid limit and the natural water content of a soil to its plasticity index.

The *in situ* behaviour of a saturated, fine-grained soil deposit at its natural water content may be studied by their consistency index or relative consistency. Relative consistency of a clay soil is a very important soil aggregate property.

When $w_N = w_L$, $I_C = 0$; when $w_N = w_P$, $I_C = 1$. If I_C is negative, the natural water content of the soil is greater than the liquid limit and the soil mass behaves like a liquid, but *only upon disturbance.* If I_C is greater than 1, the soil is in a semi-solid or solid state and will be very stiff or hard.

Liquidity index, I_L is defined as the ratio of the difference between the natural water content of a soil and its plastic limit to its plasticity index.

$$I_L = \frac{w_N - w_P}{I_P} \tag{2.47}$$

The liquidity index is used for the same purpose as the relatve consistency. In fact, $I_L = 1 - I_C$. For a soil in the plastic state, I_L varies from 0 to 1.

Table 2.4 provides guidelines to describe the consistency of cohesive (clay-like) soils using I_C and I_L.

Table 2.4 Consistency of Cohesive Soils Using Relative Consistency and Liquidity Index

Consistency	*Description*	I_C	I_L
Liquid	Liquid	<0	>1
Plastic	Very soft	0 – 0.25	0.75 – 1.00
	Soft	0.25 – 0.50	0.50 – 0.75
	Medium stiff	0.50 – 0.75	0.25 – 0.50
	Stiff	0.75 – 1.00	0 – 0.25
Semi-solid	Very stiff or hard	>1	<0
Solid	Hard or very hard	>1	<0

Flow index, I_F is the slope of the flow curve (a straight line, really) obtained between the number of blows and the water content in the A Casagrande test for the determination of liquid limit. From Fig. 2.15,

$$I_F = \frac{w_1 - w_2}{\log_{10}(N_2/N_1)} \tag{2.48}$$

The flow index indicates the *rate of loss in shearing strength upon increase in water content.* A soil with a higher value of flow index (that is, a steeper flow curve) possesses lower shear strength when compared to a soil with a lower value of flow index (that is, a flatter flow curve).

Toughness index, I_T is defined as the ratio of the plasticity index to the flow index.

$$I_T = \frac{I_P}{I_F} \tag{2.49}$$

Toughness index gives us an idea about the *shear strength of a soil at plastic limit.* Two soils, having the same plasticity index value possess toughness which vary in inverse propertion to their flow indices. It is, therefore, quite clear that *different soils have different shear strengths at plastic limit.* I_T generally lies in the range of 0 to 3 for most soils. When I_T is less than 1, the soil is *friable* (easily crushed) at the plastic limit.

Shrinkage ratio, SR is defined as the ratio of a given volume change in a soil, expressed as a percentage of the *dry volume*, to the corresponding change in water content above the shrinkage limit.

$$SR = \frac{\dfrac{V_1 - V_2}{V_d} \times 100}{w_1 - w_2} \tag{2.50}$$

where V_1 = volume of soil mass at water content w_1 (%)

V_2 = volume of soil mass at water content w_2 (%)

V_d = volume of dry soil mass

At shrinkage limit, $V_2 = V_d$ and $w_2 = w_S$; hence

$$SR = \frac{\dfrac{V_i - V_d}{V_d} \times 100}{w_1 - w_S} \qquad (2.50a)$$

If the water contents corresponding to V_1 and V_2 are expressed as ratios, Eq. 2.50 becomes

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

The change in water content $(w_1 - w_2)$ equals $\dfrac{(V_1 - V_2)/\gamma_w}{W_s}$; hence

$$SR = \frac{W_s}{V_d}\,\frac{1}{\gamma_w}$$

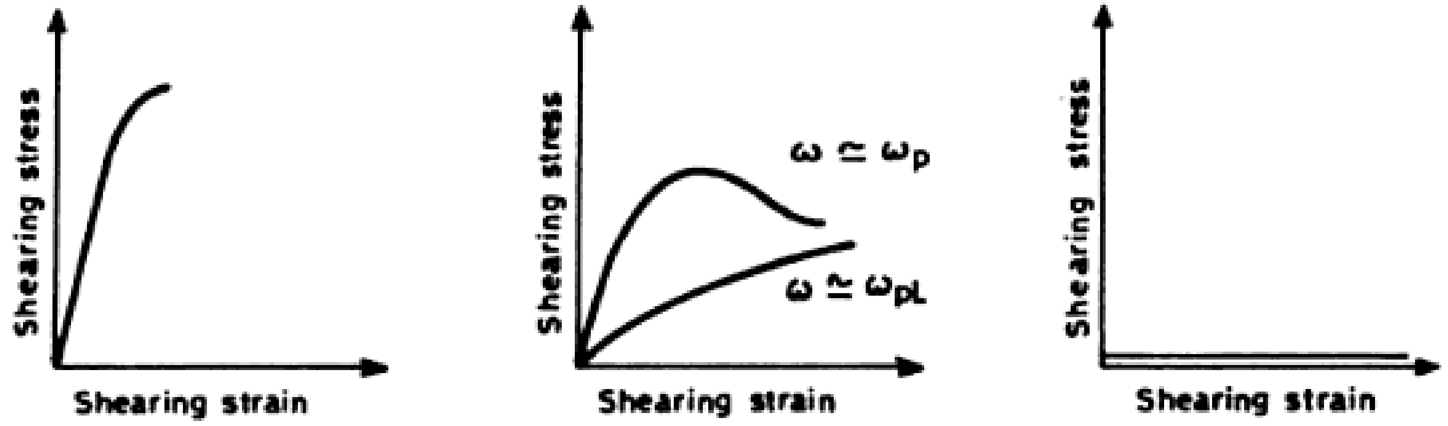
or $$SR = \gamma_d/\gamma_w \qquad (2.50b)$$

Significance of Consistency Limits and Indices

It is well known that the natural structure of a clay soil has a marked influence on its engineering behaviour. However, except for shrinkage limit which can be determined for both undisturbed and remoulded soil conditions, the consistency limits are determined on thoroughly remoulded soils. And yet, the Atterberg limits are found to correlate well with the engineering properties of soils because both the Atterberg limits and the engineering properties are found to be influenced by the same set of factors such as the clay minerals, the ions in pore water, the stress history of the soil deposit, etc. Hence, the consistency limits and indices, arbitrary and empirical though they may be, are used for classifying fine-grained soils for engineering purposes.

The liquid limit and the plastic limit indicate the plasticity of a soil and both are dependent on the amount and type of clay in a soil while the plasticity index is seen to be dependent mainly on the amount of clay present in a soil. Thus, a study of plasticity index in combination with the liquid limit forms the basis for identifying both the type and nature of clay. The shrinkage limit is a useful parameter for the study of expansive and shrinkage behaviour of a clay soil.

Liquidity index and relative consistency are good indicators of the consistency of a soil, as explained earlier. If the liquidity index is less than zero, the soil is likely to behave like a brittle solid, that is, a brittle fracture will ensue when sheared [Fig. 2.17 (a)]



(a) $I_L < 0$ (brittle) (b) $0 < I_L < 1$ (plastic) (c) $I_L > 1$ (liquid)

Fig. 2.17 Stress-strain curves for different consistency states

When I_L is between 0 and 1, the soil will behave like a plastic and its stress-strain response may vary between the shapes of the two curves shown in Fig. 2.17 (b), depending on where the natural water content scales in the plasticity range. If $I_L > 1$, the stress-strain behaviour is similar to that of a viscous liquid [Fig. 2.17(c)]. Such a soil may virtually flow like a liquid, but only upon disturbance. If left undisturbed, it is quite capable of withstanding significant loads, owing to the structural bonds that are intact. Once these bonds are broken on account of any disturbance, the soil is incapable of sustaining any load.

Table 2.5 gives an idea of the relationship between the Atterberg limits and the engineering properties of soils.

Table 2.5 General Relationships between Atterberg Limits and Engineering Properties

Characteristics	*Comparing soils at equal liquid limit with plasticity index increasing*	*Comparing soils with equal plasticity index with liquid limit increasing*
Dry strength	Increases	Decreases
Toughness near plastic limit	Increases	Decreases
Compressibility	About the same	Increases
Permeability	Decreases	Increases
Rate of volume change	Decreases	Increases

An important indication of the compressibility of a soil can be had from the liquid limit of the soil. Greater the liquid limit, greater the compressibility of a soil. In fact, reliable correlations between the liquid limit and the compressibility parameter are in practical usage.

For soils with the same value of liquid limit, the greater the plasticity index, the greater the dry strength and toughness at plastic limit, but the permeability and rate of volume change decrease while compressibility remains the same.

Comparing soils with equal values of plasticity index, it is seen that with increase in liquid limit, the dry strength and toughness decrease while the permeability and compressibility increase.

A liquid limit value of more than 100 is improbable for an inorganic soil which is not of volcanic origin. But for soils containing organic matter and for soils of volcanic origin, very high liquid limit values are possible. Bentonite, a commercial name given to a clay of volcanic ash and containing the mineral montmorillonite, has liquid limit values ranging from 400 to 600 %. Organic soils have liquid limit values greater than 50 %, but their plastic limit values are also high and therefore, their plasticity index values are not high relative to their liquid limits.

2.11 SIGNIFICANCE OF OTHER SOIL AGGREGATE PROPERTIES

In addition to the Atterberg limits, there are other soil aggregate properties which are of significance. These are defined briefly in this section and will be discussed later in detail in appropriate sections.

Permeability

The permeability of a soil is an important soil aggregate property and is useful in several engineering problems, *e.g.*, seepage through soils, drainage, rate of settlement of compressible layers, etc.

The property of soil which permits water to percolate through it, is called *permeability*.

Any material having continuous voids is permeable. The permeability of different soils varies widely. Gravels and sands are more permeable as compared to silts, whereas clays are the least permeable. The permeability of soil is specified in terms of the coefficient of permeability, k and is defined as the velocity of flow under a hydraulic gradient of unity. The coefficient of permeability of soils can be determined from various laboratory and field tests.

Unconfined Compressive Strength

The unconfined compressive strength of a cohesive soil is related to the consistency of clays.

Unconfined compressive strength is defined as the load per unit area at which an unconfined prismatic or cylindrical specimen of standard dimensions of a soil fails in a simple compression test. It is twice the value of shear strength of a clay soil under undrained condition.

Table 2.6 gives the unconfined compressive strength of natural deposits of cohesive soils in terms of their consistency.

Table 2.6 Consistency of Clays

Consistency	*Unconfined compressive strength*, q_u	
	(kg /cm^2)	(kN /m^2)
Very soft	< 0.25	< 25
Soft	0.25 – 0.50	25 – 50
Medium	0.50 – 1.0	50 – 100
Stiff	1.0 – 2.0	100 – 200
Very stiff	2.0 – 4.0	200 – 400
Hard	> 4.0	> 400

Table 2.6 can, therefore, provide a rough estimate of the undrained shear strength of a clay soil if the consistency of the soil has been assessed on the basis of its liquidity index or relative consistency.

Sensitivity and Thixotropy

It is observed that cohesive soils, upon remoulding, lose a part of their strength. The loss in strength upon remoulding is attributed partly to the breaking down of the original structure of the soil and partly to the disturbance caused to water molecules in the absorbed layer. *Sensitivity* is a measure of the loss in strength of soils as a result of remoulding and is, thus, indicative of the effect of remoulding on the consistency of a cohesive soil.

Sensitivity, S_t is defined as the ratio of the unconfined compressive strength of an undisturbed specimen of the soil to the unconfined compressive strength of a specimen of the same soil after remoulding at *unaltered water content.*

$$S_t = \frac{(q_u)\ undisturbed}{(q_u)\ remoulded} \tag{2.51}$$

Based on the value of sensitivity, natural clay deposits are grouped into four different categories (Table 2.7).

Table 2.7 Sensitivity of Clays

Sensitivity	*Classification*
1 – 4	Normal
4 – 8	Sensitive
8 – 15	Extra-sensitive
> 15	Quick

As indicated in Table 2.7, soils generally have a sensitivity value of 1 or more than 1. However, stiff clays having fissures and cracks sometimes exhibit a sensitivity value of less than 1, since these weaknesses, present in an undisturbed sample, will be removed upon remoulding.

Thixotropy is the property of certain clays by virtue of which they regain, if left alone for a time, a part of the strength lost due to remoulding, at unaltered moisture content. The increase in strength with passage of time is attributed to the tendency of cohesive soils to regain their chemical equilibrium, with the reorientation of water molecules in the adsorbed water layer. The higher the sensitivity, larger the thixotropic hardening.

Thixotropy of soils has practical implications. For example, a pile which is driven into a clay soil has the effect of disturbing the soil structure, leading to a loss of shear strength. Due to the thixotropy of the soil, however, a part of the shear strength (how much depends on the thixotropy value) will be regained if the driven pile is left alone for sometime-say, one month. Hence, the frictional resistance of a driven pile in a clay soil immediately upon installation will be considerably smaller than its value after a month or so.

Void Ratio, Porosity, Unit Weight

Void ratio, *porosity* and *unit weight* are other important soil aggregate properties. These have been discussed earlier.However, it may be noted that as the void ratio increases, the unit weight of soil decreases and the strength of the soil also decreases.

Relative Density or Density Index

The degree of denseness or looseness of natural deposits of coarse-grained soils can be measured in terms of their *relative density*.

Relative density is, for a coarse-grained soil, the equivalent of relative consistency for a clay soil. It is the most important soil aggregate property of a coarse-grained soil.

Relative density, D_r (I_D) is defined as the ratio of the difference between the void ratio of a cohesionless soil in the loosest state and void ratio in its natural state to the difference between its void ratios in the loosest and densest states.

$$D_r \text{ or } I_D = \frac{e_{\max} - e_{\text{nat}}}{e_{\max} - e_{\min}} \times 100\ \% \tag{2.52}$$

where e_{max} = void ratio in the loosest state

e_{nat} = void ratio obtained in the field in the natural state

e_{min} = void ratio in the densest state

Relative density is a better indicator of the denseness of an *in situ* coarse-grained soil deposit than the void ratio. Two sand deposits possessing similar grain shape and size characteristics can, however, exhibit quite dissimilar engineering behaviour if they happen to have different relative density values. If the relative density is high, the granular soil will be dense and will have high shearing strength and low compressibility. A loose (small D_r) granular soil is rather unstable, especially if subjected to vibratory loads.

In the loosest state, $D_r = 0$ and in the densest, $D_r = 100$ %.

On the basis of relative density, granular soil deposits can generally be grouped qualitatively into different categories (Table 2.8).

Table 2.8 Relative Density of Granular Soils

Relative density, %	*Classification*
< 15	Very loose
15 – 35	Loose
35 – 65	Medium
65 – 85	Dense
> 85	Very dense

Using the relation, $e = \dfrac{G_s\gamma_w}{\gamma_d} - 1$ in Eq. 2.52, the relative density can also be expressed in the form

$$D_r = \frac{(\gamma_d)_{max}}{\gamma_d} \times \frac{\gamma_d - (\gamma_d)_{min}}{(\gamma_d)_{max} - (\gamma_d)_{min}} \times 100 \qquad (2.53)$$

in which $(\gamma_d)_{max}$ = maximum dry unit weight

$(\gamma_d)_{min}$ = minimum dry unit weight and

γ_d = dry unit weight in the natural state of the deposit

For the determination of relative density (density index) of a granular soil deposit, Eq. 2.53 is used. For the procedure, the reader is referred to IS: 2720 - Part XIV.

Example 2.1 provides the value of e_{max} and e_{min} for a granular soil assuming that the soil grains are spherical and of uniform size.

Activity

The amount of adsorbed water in a soil is greatly influenced by the amount of clay that is present in the soil. Plasticity index of a soil is a function of both the type of clay mineral and the amount of clay mineral present in the soil. Skempton (1953) observed that I_P is directly proportional to the percentage of clay-size particles by weight.

On the basis of this, he defined a soil parameter which he called *activity:*

$$\text{Activity of a clay} = \frac{\text{Plasticity index}}{\text{\% by weight finer than } 2\,\mu} \qquad (2.54)$$

Fig. 2.18 shows some results obtained on typical clay minerals mixed with quartz sand.

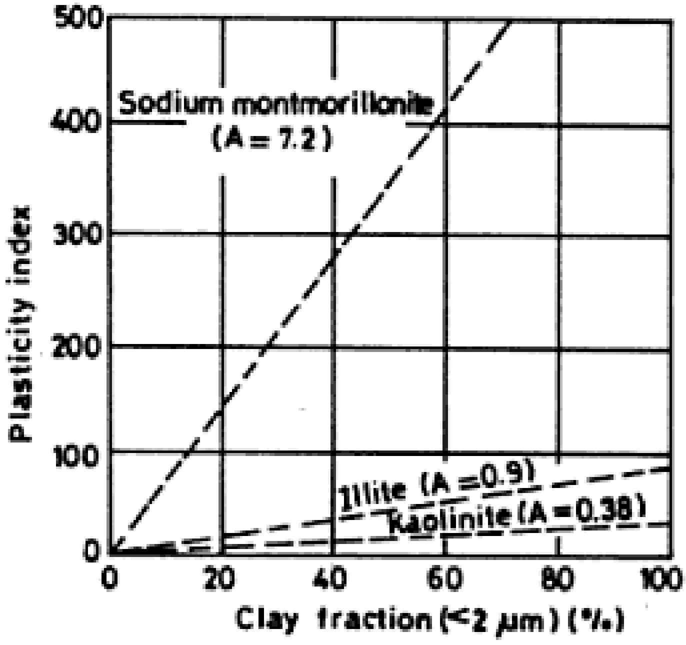


Fig. 2.18 Relation between plasticity index and clay fraction (Skempton, 1953)

One can expect pronounced clay-like behaviour such as higher compressibility and higher swelling and shrinkage characteristics from soils with higher values of activity.

Clays can be grouped into three qualitative categories (Table 2.9) depending upon their activity.

Table 2.9 Activity of Clays

Activity	*Classification*
< 0.75	Inactive
0.75 – 1.25	Normal
> 1.25	Active

EXAMPLES

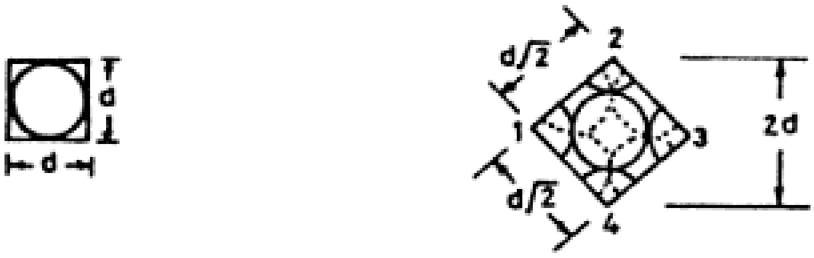
Example 2.1 For a sand soil with soil grains spherical in shape and uniform in size, determine the maximum and minimum void ratios.

Solution:

The loosest possible arrangement for round particles of uniform diameter d is shown in Fig. 2.19 (a), which is its plan view. This is known as simple packing. Each sphere makes contact with six adjacent spheres–four from sides, one from bottom and one from top. Each of the spheres may be assumed to fit within a cube of side dimension d, as shown in Fig. 2.19 (c). Thus, the volume V of the cube is d^3 and the volume of the sphere V_s is $\pi d^3/6$. Thus, the maximum void ratio (loosest state) is $e_{max} = \frac{V_v}{V_s} = \frac{d^3 - \pi d^3/6}{\pi d^3/6} = 0.91$

(a) Simple packing

(b) Face-centred cubic packing



(c) Cubic element from simple cubic

(d) Cubic element from face-centered cubic

Fig. 2.19 Example 2.1

To obtain the densest arrangement, assume that the total column is permitted to translate horizontally along a 45° line with respect to the x axis. This results in a face-centred cubic packing in which each sphere lies within the space formed by four adjacent spheres in the layer below [Fig.2.19(b).] A typical cube can be extracted from this, as shown in [Fig.2.19(d)]. This is the top view which shows a hemisphere on the top plane and four octants. The same pattern is replicated at the bottom and four vertical sides of the cube. Thus, we have a total of six hemispheres and eight octants, or a total of four complete spheres. The volume of the cube is, therefore, equal to $[d\sqrt{2}]^3 = 2d^3\sqrt{2}$ [Fig. 2.19 (d)] and the volume of the spheres within this is equal to $4(\pi d^3/6) = 2/3\,\pi d^3$. Thus the minimum void ratio (densest state) is

$$e_{\min} = \frac{2d^3\sqrt{2} - 2/3\pi d^3}{2/3\pi d^3} = 0.35$$

Example 2.2: 1 cum of wet soil weights 20 kN. Its dry weight is 18 kN. Sp. gravity of solids is 2.67. Determine the water content, porosity, void ratio and the degree of saturation. Draw a phase diagram.

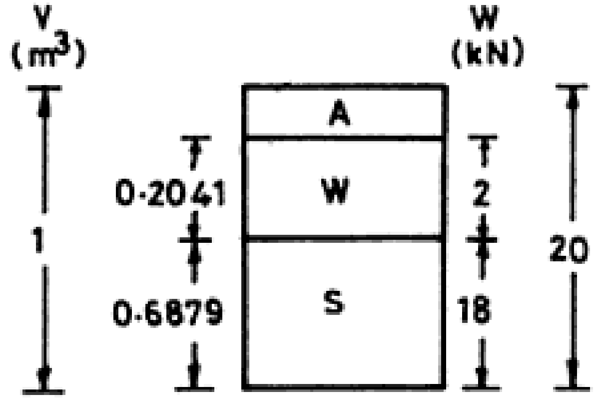


Fig. 2.20 Example 2.2

Solution:

On the phase diagram, the volume of the soil (V) and its wet weight (W) and dry weight (W_s) which are given, are first shown.

The volumes and weights of the constituents are now calculated:

$$W_w = W - W_s = 20 - 18 = 2\text{kN}$$

$$V_s = \frac{W_s}{G_s\gamma_w} = \frac{18}{2.67 \times 9.8} = 0.6879 \text{ m}^3$$

$$V_w = \frac{W_w}{\gamma_w} = \frac{2}{9.8} = 0.2041 \text{ m}^3$$

$$V_v = V - V_s = 1 - 0.6879 = 0.3121 \text{ m}^3$$

Values of W_w, V_s, V_w and V_v are shown on the phase diagram. The required ratios can now be written:

$$\text{Water content, } w = \frac{W_w}{W_s} = \frac{2}{18} = 0.1111 \text{ or } 11.1\ \%$$

$$\text{Porosity, } n = \frac{V_v}{V} = \frac{0.3121}{1} = 0.3121 \text{ or } 31.2\ \%$$

$$\text{Void ratio, } e = \frac{V_v}{V_s} = \frac{0.3121}{0.6879} = 0.45$$

$$\text{Degree of saturation, } S = \frac{V_w}{V_v} = \frac{0.2041}{0.3121} = 0.6539 \text{ or } 65.4\ \%$$

Example 2.3 A soil sample with specific gravity of solids 2.70 has a mass specific gravity of 1.84. Assuming the soil to be perfectly dry, determine the void ratio.

Solution:

$$\text{Mass specific gravity, } G_m = \frac{\gamma_d}{\gamma_w} = 1.84$$

Hence, $\gamma_d = 1.84 \times 1.0 = 1.84$ g/cc

$$\text{Void ratio, } e = \frac{G_s\gamma_w}{\gamma_d} - 1 = \frac{2.70 \times 1.0}{1.84} - 1 = 0.47$$

Alternatively, one can use the phase diagram method. Since only the ratios are given, any suitable quantity can be assumed. If the total volume of the soil V is assumed to be 100 cc, the weight of the soil (in this case, the weight of solids as the soil is dry) will be equal to $G_m V \gamma_w = 1.84 \times 100 \times 1 = 184$ g (Fig. 2.21)

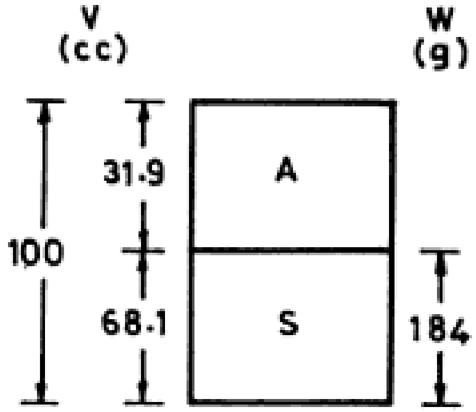


Fig. 2.21 Example 2.3

$$W_s = 184 \text{ g}$$

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{184}{2.70 \times 1} = 68.1 \text{ cc}$$

$$V_v = (V - V_s) = 100 - 68.1 = 31.9 \text{ cc}$$

$$e = \frac{V_v}{V_v} = \frac{31.9}{68.1} = 0.47$$

Example 2.4 A partially saturated sample from a borrow pit has a natural moisture content of 15 per cent and bulk unit weight of 1.9 g/cc. The specific gravity of solids is 2.70. Determine the degree of saturation and void ratio. What will be the unit weight of the sample on saturation?

Solution:

$$\gamma_d = \frac{\gamma_t}{1+w} = \frac{1.90}{1+0.15} = 1.65 \text{ g/cc}$$

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

$$= \frac{2.70 \times 1.0}{1.65} - 1$$

$$= 0.64$$

$$S = \frac{wG_s}{e} = \frac{0.15 \times 2.70}{0.64} = 0.6328 \text{ or } 63.3\ \%$$

$$\gamma_{sat} = \frac{G_s + e}{1+e} \gamma_w = \frac{2.70 + 0.64}{1 + 0.64} \times 1.0$$

$$= 2.04 \text{ g/cc}$$

Alternatively, the problem can also be solved using a phase diagram. Since only the ratios are known, any suitable quantity can be assumed. In this problem, it is convenient to take the weight of solids as unity [Fig. 2.22(a)].

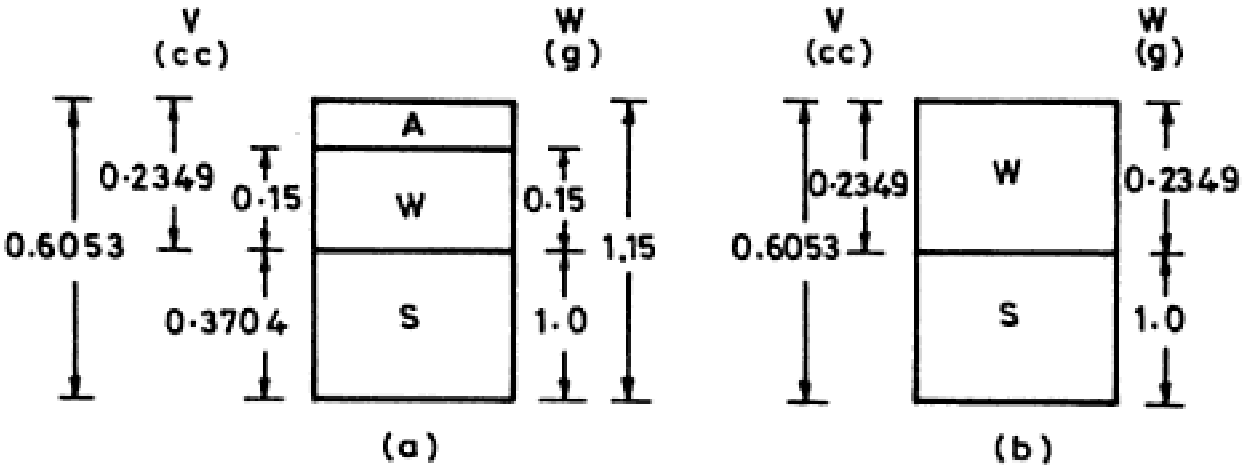


Fig. 2.22 Example 2.4

$$W_s = 1.0 \text{ g}$$

$$W_w = w \times W_s = 0.15 \times 1.0 = 0.15 \text{ g}$$

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{1.0}{2.70 \times 1.0} = 0.3704 \text{ cc}$$

$$V_w = \frac{0.15}{1.0} = 0.15 \text{ cc}$$

$$W = W_s + W_w = 1.0 + 0.15 = 1.15 \text{ g}$$

$$V = \frac{W}{\gamma} = \frac{1.15}{1.9} = 0.6053 \text{ cc}$$

$$V_v = 0.6053 - 0.3704 = 0.2349 \text{ cc}$$

$$S = \frac{V_w}{V_v} = \frac{0.15}{0.2349} = 0.6328 \text{ or } 63.3\ \%$$

$$e = \frac{V_v}{V_s} = \frac{0.2349}{0.3704} = 0.64$$

If the soil becomes fully saturated, $V_w = V_v = 0.2349$ cc

W_w will then be equal to 0.2349 g (since $W_w = V_w.\gamma_w$)

$$W = W_s + W_w = 1.0 + 0.2349 = 1.2349 \text{ g and}$$

$$\gamma_{sat} = \frac{1.0 + 0.2349}{0.6053} = 2.04 \text{ g/cc}$$

Example 2.5 An oven dry soil sample of volume 250 cc weighs 430 g. If the specific gravity of solids is 2.70, what is the water content when the soil becomes fully saturated without any change in its volume? What will be the water content which will fully saturate the sample and also cause an increase in volume equal to 10 % of the original dry volume? Use phase diagrams.

Solution:

Given: $V = 250$ cc ; $W_s = 430$ g
These values are indicated on the phase diagram (Fig. 2.23)

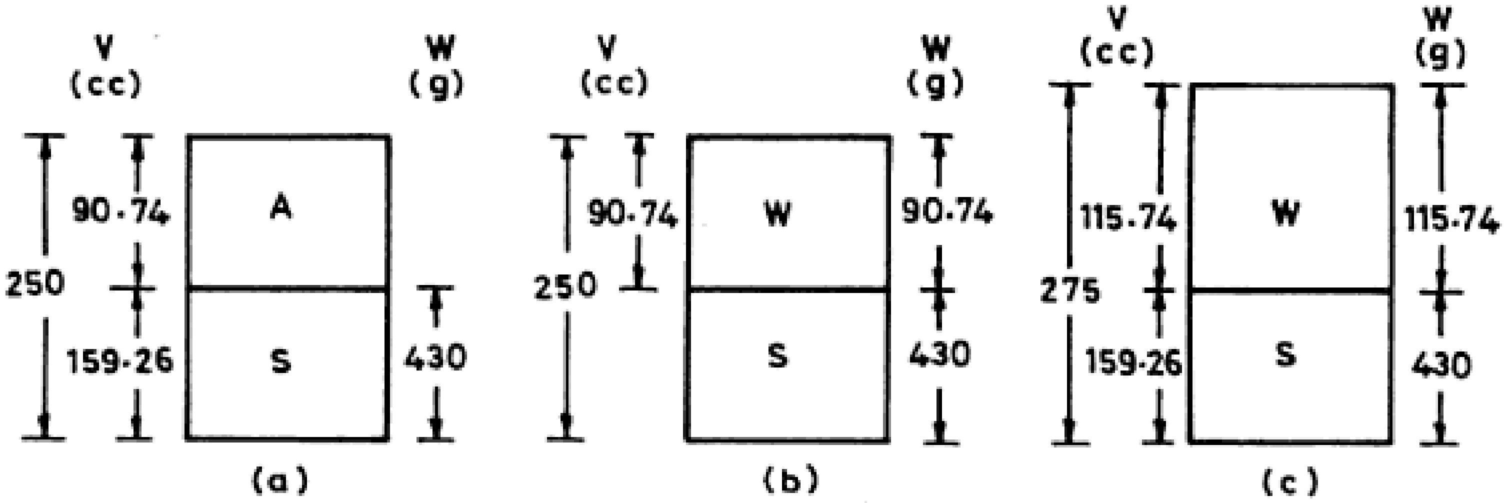


Fig. 2.23 Example 2.5

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{430}{2.7 \times 1} = 159.26 \text{ cc}$$

For the dry soil,

$$V_v = V_a = V - V_s = 250 - 159.26 = 90.74 \text{ cc}$$

These values are shown in Fig. 2.23(a).

The phase diagram for a saturated soil with no change in its volume is shown in Fig. 2.23(b).

$$V_w = V_v = 90.74 \text{ cc}$$

$$W_w = V_w \gamma_w = 90.74 \times 1 = 90.74 \text{ g}$$

Water content $$w = \frac{W_w}{W_s} = \frac{90.74}{430} = 0.2110 \text{ or } 21.1\ \%$$

This is, indeed, the shrinkage limit of the soil since this is the minimum content for complete saturation.

Fig. 2.23 (c) shows the phase diagram when there is an increase in volume of the soil following saturation.

New volume of the soil $= 1.1 \times 250 = 275$ cc

The increase in volume is due only to the increased void volume.

Increased $V_v = V - V_s = 275 - 159.26 = 115.74$ cc

$$V_w = V_v = 115.74 \text{ cc}$$

$$W_w = V_w \gamma_w = 115.74 \times 1 = 115.74 \text{ g}$$

$$\text{Water content, } w = \frac{W_w}{W_s} = \frac{115.74}{430} = 0.2692 \text{ or } 26.9\ \%$$

Example 2.6 A clay sample originally 25 mm thick and at a void ratio of 1.120, was subjected to a compressive load. After the clay sample was completely consolidated, its thickness was measured to be 24 mm. Compute the final void ratio.

Solution:

Under a compressive load, the soil particles are rearranged into a closer packing, resulting in a decrease in void volume and a consequent decrease in void ratio. For a soil with an initial void ratio of e_0 and a decreased void ratio of e, the phase diagrams are shown in Fig. 2.24. The volume of solids is assumed as unity.

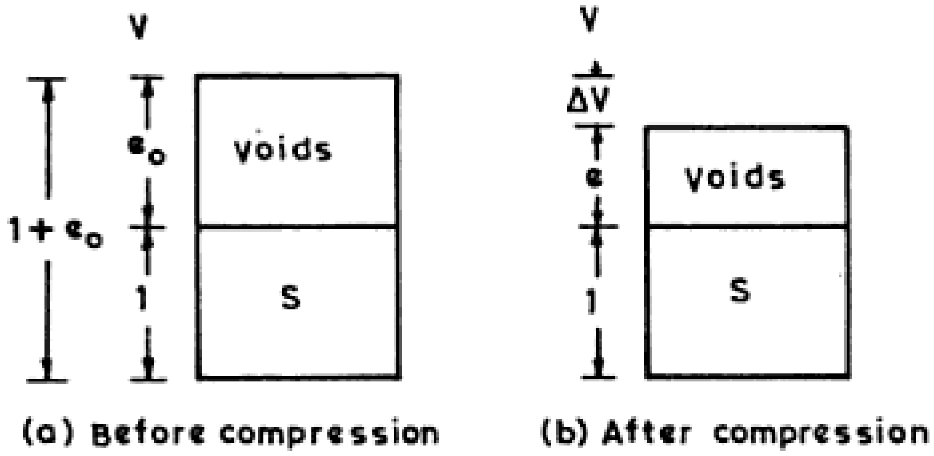


Fig. 2.24 Example 2.6

For a certain constant sectional area A of the soil specimen, change in thickness H is synonymous with the change in volume V that is,

$$\frac{\Delta H}{H} = \frac{\Delta V}{V} \tag{2.58}$$

in which H and V represent thickness and volume, respectively.

From Fig. 2.24, it can be seen that

$$\frac{\Delta V}{V} = \frac{e_0 - e}{1 + e_0} = \frac{\Delta e}{1 + e_0} \tag{2.59}$$

From Eq. 2.58 and 2.59,

$$\frac{\Delta H}{H} = \frac{\Delta V}{V} = \frac{\Delta e}{1 + e_0} \tag{2.60}$$

Given that $H = 25$ mm, $\Delta H = 25 - 24 = 1$ mm and $e_0 = 1.120$,

$$\Delta e = \frac{\Delta H}{H}(1+e_0) = \frac{1}{25} \times 2.12 = 0.0848$$

Final void ratio $e = e_0 - \Delta e = 1.120 - 0.0848 = 1.0352$

Example 2.7 The undisturbed soil at a borrow pit has a water content of 15 per cent, void ratio of 0.60 and specific gravity of solids 2.70. The soil from the borrow pit is to be used for the construction of an embankment with a finished volume of 40,000 cu m. The specifications for the embankment require a water content of 18 per cent and a dry unit weight of 1.76 g/cc. Calculate the quantity of soil required to be excavated from the borrow pit for the construction of the embankment. What would be its weight? What is the weight of the embankment soil?

Solution:

It is best to solve such problems by determining the volume of solids or the weight of solids that will be present in the finished embankment at the specified void ratio. This volume/weight of solids will have to come from the soil mass in the borrow area. Naturally, the volume of soil needed will be a function of the void ratio of the soil in the borrow pit.

$$\text{Dry unit weight of the embankment soil} = \frac{W_s}{V}$$

$$= 1.76 \text{ g/cc or } 1.76 \text{ t/m}^3\text{; that is , if } V=1 \text{ cu m,}$$

$$W_s = 1.76 \text{ t}$$

Hence for $V = 40000$ cu m, W_s required is $40000 \times 1.76 = 70400$ t.

Dry unit weight of the soil in the borrow pit,

$$\gamma_d = \frac{G_s}{1+e}\gamma_w = \frac{2.7}{1+0.6}1.0 = 1.6875 \text{ t/m}^3$$

$$\gamma_d = \frac{W_s}{V} = 1.6875 \text{ t/m}^3$$

For $W_s = 70400$ t, the volume of soil that has to be taken out from the borrow pit

$$V = \frac{W_s}{\gamma_d} = \frac{70400}{1.6875} = 41718 \text{ cu m}$$

$W_s = 70400$ t, both in the borrow pit and the embankment.

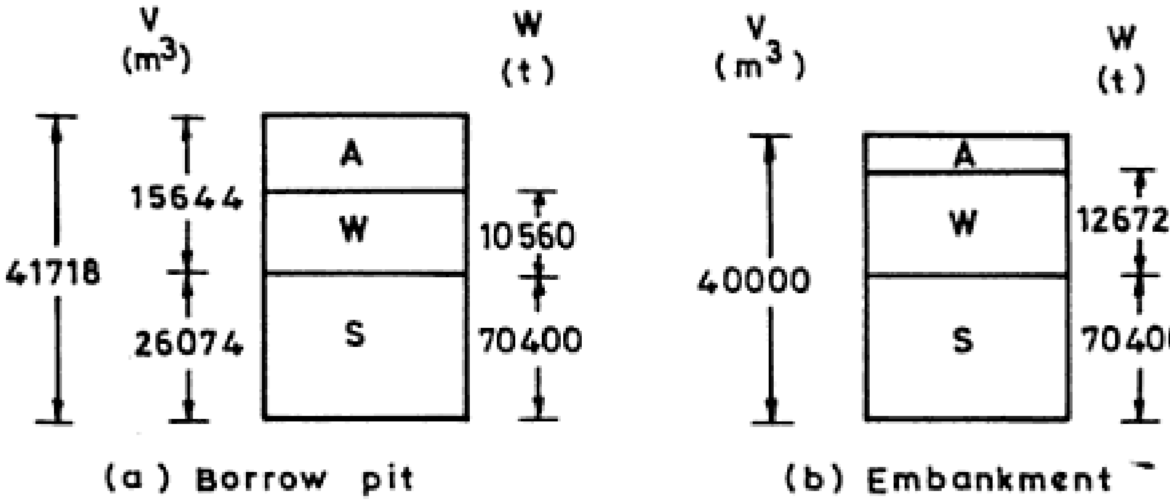
The weight of soil (W) that has to be taken out from the borrow pit which has a natural moisture content (w) of 15% given by

$$W = W_s(1+w) = 70400(1+0.15) = 80960 \text{ t}$$

The weight of soil in the embankment which has a moisture content of 18 % will be equal to

$$W = 70400(1+0.18) = 83072 \text{ t}$$

Fig. 2.25 shows the phase diagrams for the conditions at borrow area and the embankment. Since the total volume of the finished embankment is given, it is shown as such in Fig. 2.25 (b).



(a) Borrow pit (b) Embankment

Fig. 2.25 Example 2.7

In Fig. 2.25 (b), for $V = 40000 \text{ m}^3$, $W_s = 40000 \times 1.76 = 70400 \text{ t}$, since $\gamma_s = \dfrac{W_s}{V}$

In Fig. 2.25(a), for $W_s = 70400 \text{ t}$, $V_s = \dfrac{70400}{2.7 \times 1} = 26074 \text{ m}^3$, since $V_s = \dfrac{W_s}{G_s \gamma_w}$

$$V_v = e\, V_s = 26074 \times 0.6 = 15644 \text{ m}^3 \text{ and } V = V_s + V_v = 26074 + 15644 = 41718 \text{ m}^3$$

For $W_s = 70400$ t, W_w (borrow pit) = $w\, W_s = 0.15 \times 70400 = 10560$ t

and W_w (embankment) $= 0.18 \times 70400 = 12672$ t

Hence, the weight of soil in the borrow area $= W_s + W_w = 70400 + 10560 = 80960$ t

Weight of soil in embankment = 70400 + 12672 = 83072 t

Example 2.8 The pavement of a road on a level ground is to be laid on a base course 400 mm thick, consisting of a coarse-grained gravel-sand mixture with good draining properties, placed evenly on an impervious subgrade. The porosity of the gravel-sand is 40% and the degree of saturation, 60%. There is a sudden downpour during construction work. Assuming that all water immediately infiltrates into the ground, calculate the rainfall in mm that would saturate the base course to its full thickness?

Solution:

Consider a prism of gravel-sand soil 400 mm thick with a base area of 1 m^2.

$$V = 0.4 \text{ m}^3$$

$$V_v = n\, V = 0.4 \times 0.4 = 0.16 \text{ m}^3$$

$$V_w = S\, V_v = 0.6 \times 0.16 = 0.096 \text{ m}^3$$

$$V_a = V_v - V_w = 0.16 - 0.096 = 0.064 \text{ m}^3$$

Water from rainfall has to fill this volume of air in order to saturate the soil. The required amount of rain fall is, therefore, equal to

$$\frac{0.064 \text{ m}^3}{1} \quad \text{i.e., 64 mm}$$

Example 2.9 An airport runway fill needs 600,000 m^3 of soil compacted to a void ratio of 0.75. There are two borrow pits A and B from where the required soil can be taken and transported to the site.

Borrow pit	*In situ void ratio*	*Transportation cost*
A	0.80	Rs.10/m^3
B	1.70	Rs.5/m^3

Which of the two borrow pits would be more economical?

Solution:

Let us find out the volume of solids present in the finished volume of embankment.

$$V_s = \frac{V}{1+e} = \frac{600000}{1+0.75} = 342857 \text{ m}^3$$

We can now determine the volume of soil required to be taken out from borrow pits A and B such that it may contain the above volume of solids.

Borrow pit A: $V = V_s(1+e) = 348257\ (1.8) = 617143 \text{ m}^3$

Borrow pit B: $V = V_s(1+e) = 342857\ (2.7) = 9\ 25714 \text{ m}^3$

Transportation cost if borrow pit A is used

$$= 617143 \times 10 = \text{Rs. } 61,71,430$$

Transportation cost if borrow pit B is used

$$= 925714 \times 5 = \text{Rs. } 46,28,570$$

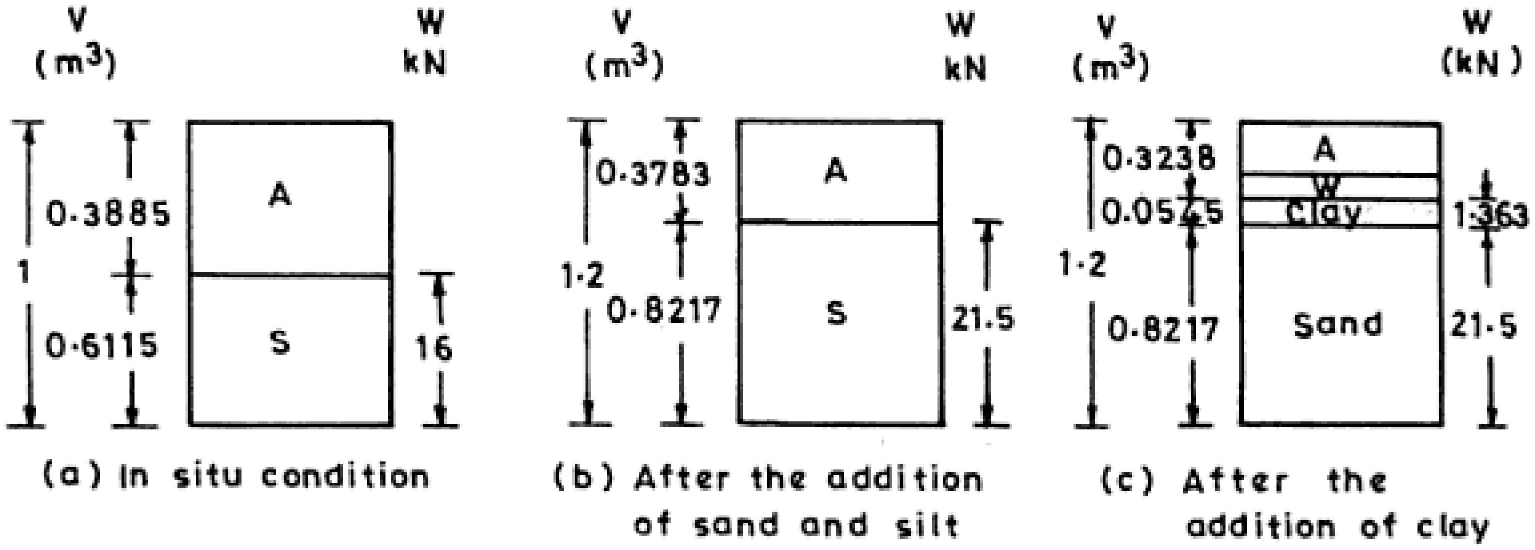
Therefore, borrow pit B is more economical.

Example 2.10 The *in situ* unit weight of a medium to coarse sand used as subgrade for a highway, was 16 kN/m^3. It was decided to improve that soil by mechanical stabilization. When 5.5 kN of a mixture of dry sand and silt was added to 1 m^3 of this subgrade, the volume was increased by 20 per cent. How much reduction in porosity of the soil was achieved? Assume average G_s as 2.67.

Further, 1.5 kN of clay, at a moisture content of 10 per cent was added to the above mixture such that no further increase in the volume of the subgrade resulted. Determine the further reduction in porosity that this addition of clay brought about. Assume G_s of clay particles as 2.55 .

Solution:

Figure 2.26 shows the phase diagrams corresponding to (a) the *in situ* condition; (b) condition after the addition of sand and silt, and (c) condition after the addition of clay.



(a) In situ condition (b) After the addition of sand and silt (c) After the addition of clay

Fig. 2.26 Example 2.10

In Fig. 2.26 (a), $V_s = \dfrac{W_s}{G_s \gamma_w} = \dfrac{16}{2.67 \times 9.8} = 0.6115 \text{ m}^3$

$$V_v = 1 - 0.6115 = 0.3885 \text{ m}^3$$

Porosity, $n = \dfrac{V_v}{V} = \dfrac{0.3885}{1}$ or 38.9 %

In Fig. 2.26 (b), $V_s = \dfrac{21.5}{2.67 \times 9.8} = 0.8217 \text{ m}^3$

$$V_v = 1.2 - 0.8217 = 0.3783 \text{ m}^3$$

Porosity $n = \dfrac{V_v}{V} = 0.3153$ or 31.5 %

Reduction in porosity achieved = 38.85 – 31.53 = 7.3 %

In Fig. 2.26(c), W_s (clay), $\dfrac{W_{\text{clay}}}{1 + w_{\text{clay}}} = \dfrac{1.5}{1.1} = 1.363$ kN

$$V_s \text{ (clay)} = \frac{W_s}{G_s \gamma_w} = \frac{1.363}{2.55 \times 9.8} = 0.0545 \text{ m}^3$$

$$V_v = 1.2 - 0.8217 - 0.0545 = 0.3238 \text{ m}^3$$

Porosity $n = \dfrac{V_v}{V} = \dfrac{0.3238}{1.2} = 0.2698$ or 26.98 %

Further reduction in porosity form (b) = 31.53 – 26.98 = 4.6 %

Example 2.11: A natural deposit of loose, dry sand, 5 m thick, with an *in situ* unit weight of 14 kN/m³ was compacted by vibroflotation technique and the surface settled by 0.5 m. The relative density of the sand after compaction was found to be 85 %. If the dry unit weight of the sand in the loosest state is 10.8 kN/m^3, calculate

(a) the unit weight of the soil after vibroflotation;

(b) the dry unit weight in the densest state, and

(c) the maximum possible settlement of the surface under ideal compaction

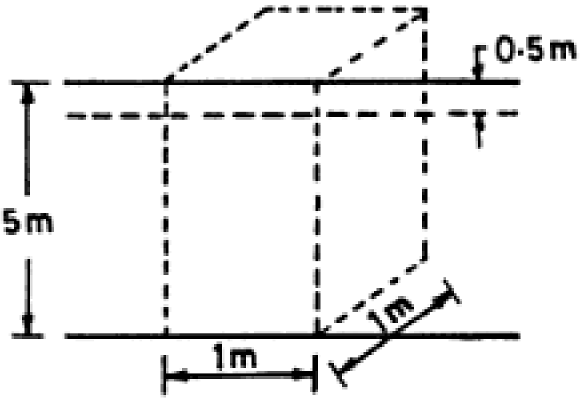


Fig. 2.27 Example 2.11

Solution:

Given: $\gamma_{d\,(in\ situ)} = 14$ kN/m^3

$\gamma_{d\,(\min)} = 10.8$ kN/m^3

$D_r = 0.85$ %. or 0.85

(a) Consider a prism of soil of 5 m thickness (*in situ* state), 1 m width and 1 m length in a direction perpendicular to the plane of paper (Fig. 2.27)

$$\gamma_d = \frac{W_s}{V} = 14 \text{ kN/m}^3$$

$$V = 5 \times 1 \times 1 = 5 \text{ m}^3$$

$$W_s = 70 \text{ kN}$$

After compaction, $V = 4.5 \times 1 \times 1 = 4.5$ m^3

$$\gamma_d \text{ after compaction} = \frac{W_s}{V} = \frac{70}{4.5}\ 15.55 \text{ kN/m}^3$$

(b) From Eq. 2.56. $D_r = \dfrac{(\gamma_d)_{\max}}{\gamma_d} \times \dfrac{\gamma_d - (\gamma_d)_{\min}}{\gamma_{d\max} - \gamma_{d\min}}$

$$0.85 = \frac{(\gamma_d)_{\max}}{15.55} \times \frac{(15.55 - 10.8)}{\gamma_{d\max} - 10.8}$$

Solving, $(\gamma_d)_{\max} = 16.85$ kN/m^3

(c) An ideal compaction would produce a dry unit weight equal to $(\gamma_d)_{\max}$, i.e., 16.85 kN/m^3

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

$$W_s = 70 \text{ kN}$$

$$V = \frac{70}{16.85} = 4.154 \text{ m}^3, \text{ corresponding to a thickness of } 4.154 \text{ m}$$

Hence maximum possible settlement of surface = 5 – 4.154 = 0.846 m

Example 2.12 The specific gravity of the grains of a soil was determined in the laboratory using a pycnometer and the following measurements were made:

$$\text{Weight of solids, } W_s = W_2 - W_1 = 100 \text{ g}$$

$$W_3 = 613 \text{ g}$$

$$W_4 = 550 \text{ g}$$

Symbols W_1, W_2, W_3 and W_4 have the meanings given in Section 2.5. What is the specific gravity of the soil grains? In order to confirm the value, a second observation of weight W_3 was made, after repeatedly evacuating the pycnometer using a vacuum pump. The value of specific gravity thus obtained was 2.73. Determine the volume of air entrapped in the soil suspension when the first measurement was made. Neglect the effect of temperature.

Solution:

The specific gravity of solids is given by:

$$G_s = \frac{W_s}{W_s - W_3 + W_4}$$

From the first measurement,

$$G_s = \frac{100}{100 - 613 + 550} = \frac{100}{37} = 2.70$$

This is incorrect, since proper removal of entrapped air was not carried out. Presence of entrapped air would mean that the value of W_3 would be smaller than the actual value since water failed to get into space which the entrapped air occupied.

The correct weight of water of volume equivalent to that of the soil grains is equal to

$$\frac{W_s}{G_s \gamma_w} = \frac{100}{2.73} = 36.63 \text{ g}$$

The difference between 37 g and 36.63 g, *i.e.*, 0.37 g represents the weight of water which would have filled the entrapped air space. Therefore, the volume of entrapped air = 0.37 cc

Example 2.13 A 1000 cc core cutter weighing 946.80 g was used to find out the *in situ* unit weight of an embankment., The weight of core cutter filled with soil was noted to be 2770.60 g. Laboratory tests on the sample indicated a water content of 10.45 per cent and specific gravity of solids of 2.65. Determine the bulk unit weight, dry unit weight, void ratio and degree of saturation of the sample.

If the embankment becomes saturated due to rains, calculate the water content and the saturated unit weight (assume there is no volume change in sample on saturation).

Solution:

Weight of soil in core cutter = 2770.60 − 946.80
= 1823.8 g

Bulk unit weight, $\gamma_t = \dfrac{W}{V} = \dfrac{1823.8}{1000} = 1.82$ g/cc

Dry unit weight, $\gamma_d = \dfrac{\gamma_t}{1+w}$

$= \dfrac{1.82}{1+0.1045} = 1.65$ g/cc

void ratio, $e = \dfrac{G_s\gamma_w}{\gamma_d} - 1$

$= \dfrac{2.65 \times 1.0}{1.65} - 1 = 0.61$

Degree of saturation, $S = \dfrac{wG_s}{e} = \dfrac{0.1045 \times 2.65}{0.61} = 0.4540 = 45.4\%$

When the embankment gets saturated, *i.e.*, for $S = 1$,

Water content at saturation, $w = \dfrac{e}{G_s} = 0.2302 = 23\%$

Saturated unit weight, $\gamma_{sat} = \dfrac{G_s + e}{1+e}\gamma_w$

$= \dfrac{2.65 + 0.61}{1+0.61} \times 1$

$= 2.02$ g/cc

Let us now solve this problem using phase diagrams. Fig. 2.28 shows the phase diagram for the partially saturated soil, with the given values of volume V and weight W shown on it at the appropriate places.

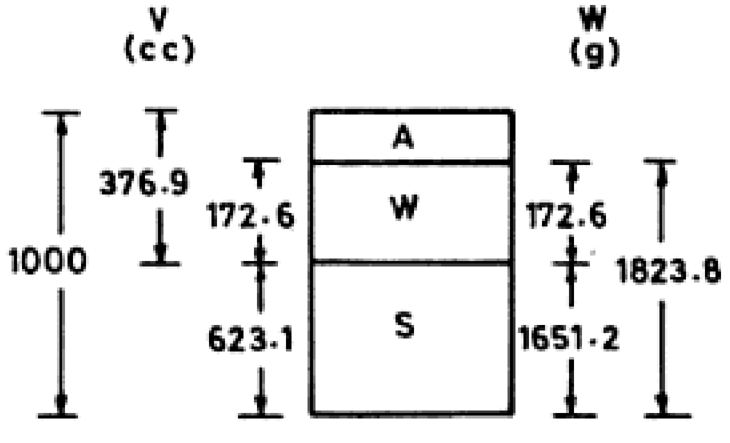


Fig. 2.28 Example 2.13

Now using the other ratios given in the example, we can calculate the volumes and weights of individual constituents of the soil mass thus:

$$w = 0.1045;\ W_s = \frac{W}{1+w} = \frac{1823.8}{1.1045} = 1651.2 \text{ g}$$

$$W_w = W - W_s = 1823.8 - 1651.2 = 172.6 \text{ g}$$

$$V_s = \frac{W_s}{\gamma_s} = \frac{W_s}{G_s\gamma_w} = \frac{1651.2}{2.65 \times 1} = 623.1 \text{ g}$$

$$V_w = \frac{W_w}{\gamma_w} = \frac{172.6}{1} = 172.6 \text{ g}$$

$$V_v = V - V_s = 1000 - 623.1 = 376.9 \text{ cc}$$

All these values are shown on the phase diagram. The required ratios can now be calculated by their basic definitions.

$$\gamma_t = \frac{W}{V} = \frac{1823.8}{1000} = 1.82 \text{ g/cc}$$

$$\gamma_d = \frac{W_s}{V} = \frac{1651.2}{1000} = 1.65 \text{ g/cc}$$

$$e = \frac{V_v}{V_s} = \frac{376.9}{623.1} = 0.61$$

$$S = \frac{V_w}{V_v} = \frac{172.6}{376.9} = 0.458 \text{ or } 45.8\%$$

Fig. 2.29 shows the phase digram for a completely saturated soil, with the volumes and weights as shown thereon.

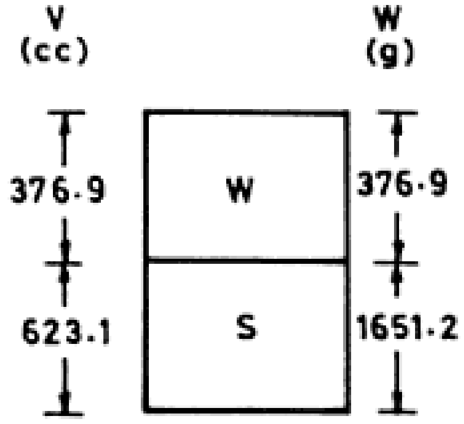


Fig. 2.29 Example 2.13

$$w = \frac{W_w}{W_s} = \frac{376.9}{1651.2} = 0.2282 \text{ or } 22.8\%$$

$$\gamma_{sat} = \frac{W_{(sat)}}{V} = \frac{1651.2 + 376.9}{1000} = 2.02 \text{ g/cc}$$

Example 2.14 During soil investigation for a residential complex site at Roorkee, the following observations were taken for the *in situ* unit weight measurement by sand replacement method:

Weight of excavated soil = 761.25 g

Weight of sand + cylinder (W_1) = 10500 g

Weight of sand + cylinder after pouring in the excavated hole and cone (W_2) = 9450 g

Weight of sand + cylinder after pouring for the cone only (W_3) = 9005 g

Volume of calibrating can = 1000 cc

Weight of sand in calibrating can after pouring from cylinder = 1500 g

Calculate the *in situ* unit weight of the soil.

Solution:

Weight of sand filling the excavated hole and the cone $= (W_1 - W_2)$

$= (10500–9450)$

$= 1050$ g

Weight of sand filling cone only $= (W_2 - W_3)$

$= 9450–9005$

$= 445$ g

Weight of sand filling hole $= 1050–445$

$= 605$ g

Unit weight of sand $= \dfrac{1550}{1000} = 1.55$ g/cc

Volume of the hole $\dfrac{605}{1.55} = 390.32$ cc.

In situ unit weight $= \dfrac{761.25}{390.32} = 1.95$ g/cc

Example 2.15 500 g of dry soil was subjected to a sieve analysis. The weight of soil retained on each sieve is as follows:

I.S. sieve size	*Wt. of soil, g*	*I.S. sieve size*	*Wt. of soil, g*
4.75 mm	10	425 µ	85
2.00 mm	165	212 µ	40
1.00 mm	100	150 µ	30
		75 µ	50

Plot the grain size distribution curve and determine the following:

(a) Percentages of gravel, coarse sand, medium sand, fine sand and silt-clay fraction in the soil as per IS: 1498-1970.

(b) Effective size

(c) Uniformity coefficient and

(d) Coefficient of curvature

(e) The gradation of the soil

Solution:

The diameter of soil grain and the corresponding per cent finer than, are calculated as below:

I.S. sieve size	*Dia of grain, mm*	*Wt. retained, g*	*Cumulative wt. retained, g*	*% retained*	*% finer than*
4.75	4.75	10	10	2.0	98.0
2.00	2.00	165	175	35.0	65.0
1.00	1.00	100	275	55.0	45.0
425	0.425	85	360	72.0	28.0
212	0.212	60	420	84.0	16.0
150	0.150	20	440	88.0	12.0
75	0.075	40	480	96.0	4.0

The grain size distribution curve is plotted between the diameter of soil grain and the per cent finer than, as shown in Fig. 2.30

(a) I.S. grain size classification scale is fitted on the abscissa. From Fig. 2.30,

(i) Percentage of gravel =100 – 98 = 2%
>4.75 mm

(ii) Percentage of coarse sand = 98 – 65 = 33%
(4.75 mm – 2.00 mm)

(iii) Percentage of medium sand = 65 – 28 = 37%
(2.00 mm – 0.425 mm)

(iv) Percentage of fine sand = 28 – 4 = 24%
(0.425 mm – 0.075 mm)

(v) Percentage of silt-clay fraction = 4%
(<0.075 mm)

(b) Effective size, $D_{10} = 0.13$ mm

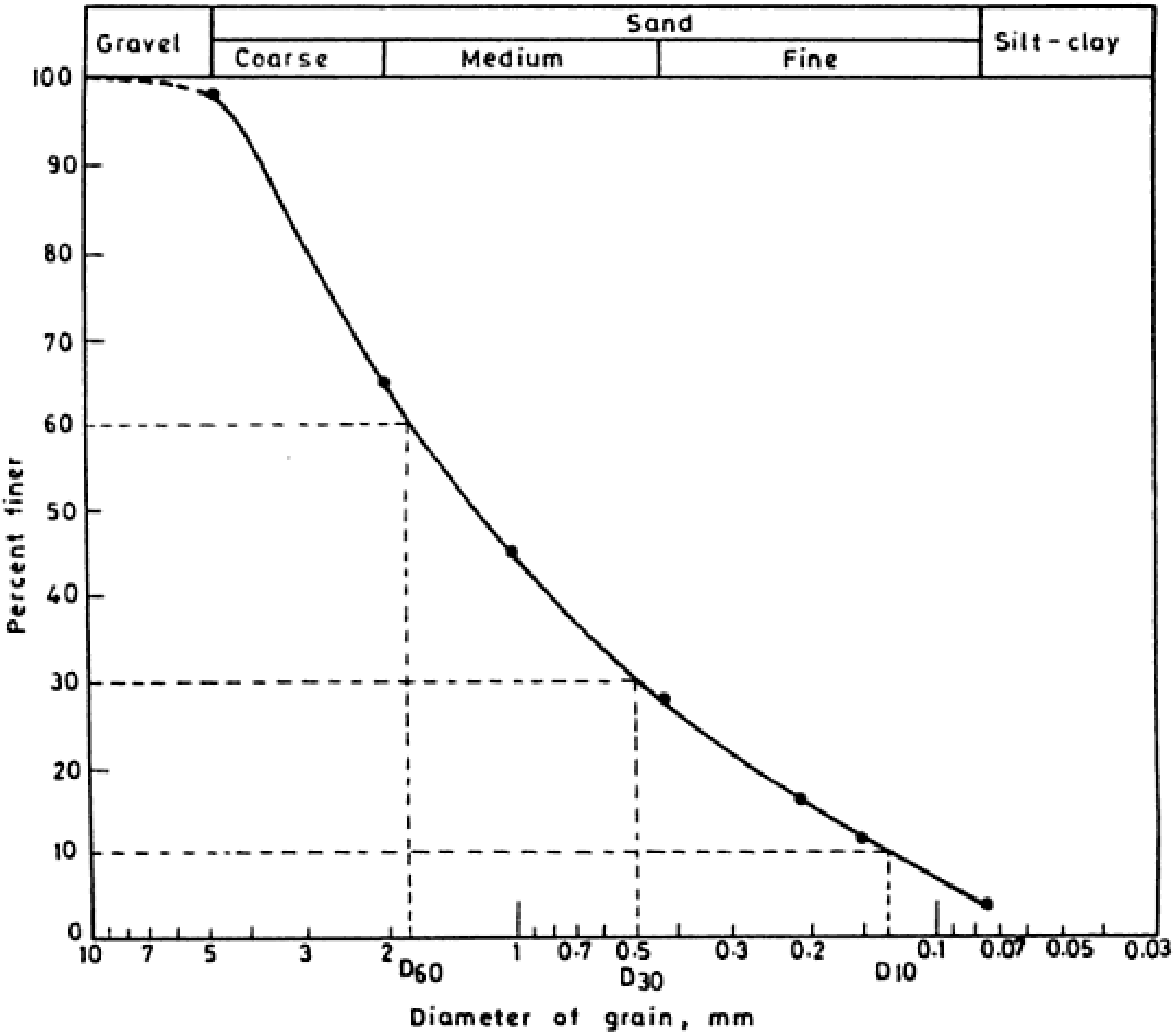


Fig. 2.30 Example 2.15

(c) Uniformity coefficient $C_u = \frac{D_{60}}{D_{10}} = \frac{1.8}{0.13} = 13.8$

(d) Coefficient of curvature, $C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} = \frac{0.5^2}{1.8 \times 0.13} = 1.1$

(e) The sand soil is classed as well-graded, since $C_u > 6$ and C_c is between 1 and 3.

Example 2.16 50 g of minus 75 micron, oven dried soil was used in a hydrometer analysis. The corrected hydrometer reading after 2 min in a 1000 cc soil suspension was 25.0. The effective depth, H_e for R_H of 25 is 12.13 cm. Taking specific gravity of solids as 2.75 and viscosity of water as 0.01 poise, calculate the coordinates of the point on the grain-size curve.

Solution:

$$R_c = 25\ ;\ \eta = \frac{0.01}{980.7}\ \text{gs/cm}^2;\ G_s = 2.75;\ H_e = 12.13\ \text{cm};\ t = 2\ \text{min}$$

$$\gamma_s = G_s\,\gamma_w = 2.75\ \text{g/cc},\ W_s = 50\ \text{g}$$

Substituting the values in Eq. 2.36,

$$D = \sqrt{\frac{18\,\eta}{(\gamma_s - \gamma_w)} \cdot \frac{H_e}{60\,t}}$$

$$= \sqrt{\frac{18 \times 0.01}{(2.75 - 1) \times 980.7} \times \frac{12.13}{60 \times 2}} = 0.0033\ \text{cm or } 0.033\ \text{mm}$$

The percentage of particles finer than 0.033 mm can be calculated from Eq. 2.39

$$N = \left(\frac{G_s}{G_s - 1}\right)\frac{R_c}{W_s} \times 100$$

$$N = \left(\frac{2.75}{2.75 - 1}\right)\frac{25}{50} \times 100$$

$$N = 78.6\%$$

Example 2.17 Five different particle sizes are mixed in proportions shown below and water is added to make the volume of soil suspension exactly equal to 1000 cc.

Particle size, mm	*Weight, g*
0.060	5
0.020	15
0.010	20
0.005	4
0.001	6

The particles have a specific gravity of 2.65 and the temperature of the soil suspension was 25°C. γ_w can be taken as 1.0 g / cc.

The soil suspension was thoroughly shaken and sedimentation allowed.

(a) What is the largest particle size present at a depth of 10 cm after 8 minutes of the start of sedimentation?

(b) What is the specific gravity of the soil suspension at a depth of 10 cm after 8 minutes of the start of sedimentation?

(c) How long after the start of sedimentation will all particles have settled down below a depth of 10 cm?

Solution:

(a) From Table 2.2, η at 25°C = 8.95 millipoises

$$\frac{8.95 \times 10^{-3}}{980.7} \text{ gs/cm}^2$$

$$\gamma_s = 2.65 \text{ g/cc}$$

$$\gamma_w = 1.0 \text{ g/cc}$$

$$H_e = 10 \text{ cm}$$

$$t = 8 \text{ min.}$$

From Eq. 2.36,

$$D = \sqrt{\frac{18\eta}{\gamma_s - \gamma_w} \cdot \frac{H_e}{60\,t}}$$

$$= \sqrt{\frac{18}{(2.65-1)}\left(\frac{0.00895}{980.7}\right)\frac{10}{60 \times 8}}$$

$$= 0.00144 \text{ cm}$$

$$= 0.0144 \text{ mm}$$

This is the largest size of the particle that can be present, theoretically, at a depth of 10 cm after 8 min. of sedimentation. However, as per the sizes of particles actually present in the soil suspension, the largest size of particle will be 0.01 mm.

(b) Since particles of equal size have equal velocities of fall, the concentration of soil suspension at 10 cm depth after 8 min. will be of all those particles equal to or smaller in size than 0.01 mm.

Weight of soil particles present at 10 cm depth

$$= 20 + 4 + 6 = 30 \text{ g}$$

Volume of soil particles $= \dfrac{30}{2.65 \times 1} = 11.3$ cc

Volume of water $= 1000 - 11.3 = 988.7$ cc

Weight of water $= 988.7 \times 1 = 988.7$ g

Weight of soil suspension $= 988.7 + 30 = 1018.7$ g

Unit weight of soil suspension $= \dfrac{1018.7}{1000} = 1.019$ g/cc

Hence, specific gravity of the soil suspension $= 1.019$

(c) $$v = \frac{\gamma_s - \gamma_w}{18\eta} D^2$$

$$\frac{H_e}{t} = \frac{\gamma_s - \gamma_w}{18\eta} D^2$$

For all particles to settle below 10 cm depth, D would have to be the smallest particle size present, that is, 0.001 mm or 0.0001 cm

$$\frac{10}{t} = \frac{2.65 - 1.0}{18 \times 0.00895} \times 980.7 \times 0.0001^2$$

or $$t = \frac{10 \times 18 \times 0.00895}{1.65 \times 980.7 \times 0.0001^2} = 99558 \text{ s}$$

or $$t = 27 \text{ h } 39 \text{ m}$$

Example 2.18 Two soils were tested for their consistency limits in the laboratory. The following data were obtained:

Soil A		*Soil B*	
No. of blows N	*w%*	*No. of blows N*	*w%*
8	43	5	65
20	39	15	61
30	37	30	59
45	35	40	58
Plastic limit = 25%		Plastic limit = 30%	

The natural moisture contents of soils A and B were measured in the field and were found to be 40% and 50 %, respectively.

(a) Which soil has greater plasticity?

(b) Which soil will be a better foundation material upon remoulding?

(c) Which soil is more compressible?

(d) Which soil shows a higher rate of loss in shear strength upon increase in water content?

(e) Which soil has a higher strength at plastic limit?

(f) Is there a likelihood of organic matter being present in these soils?

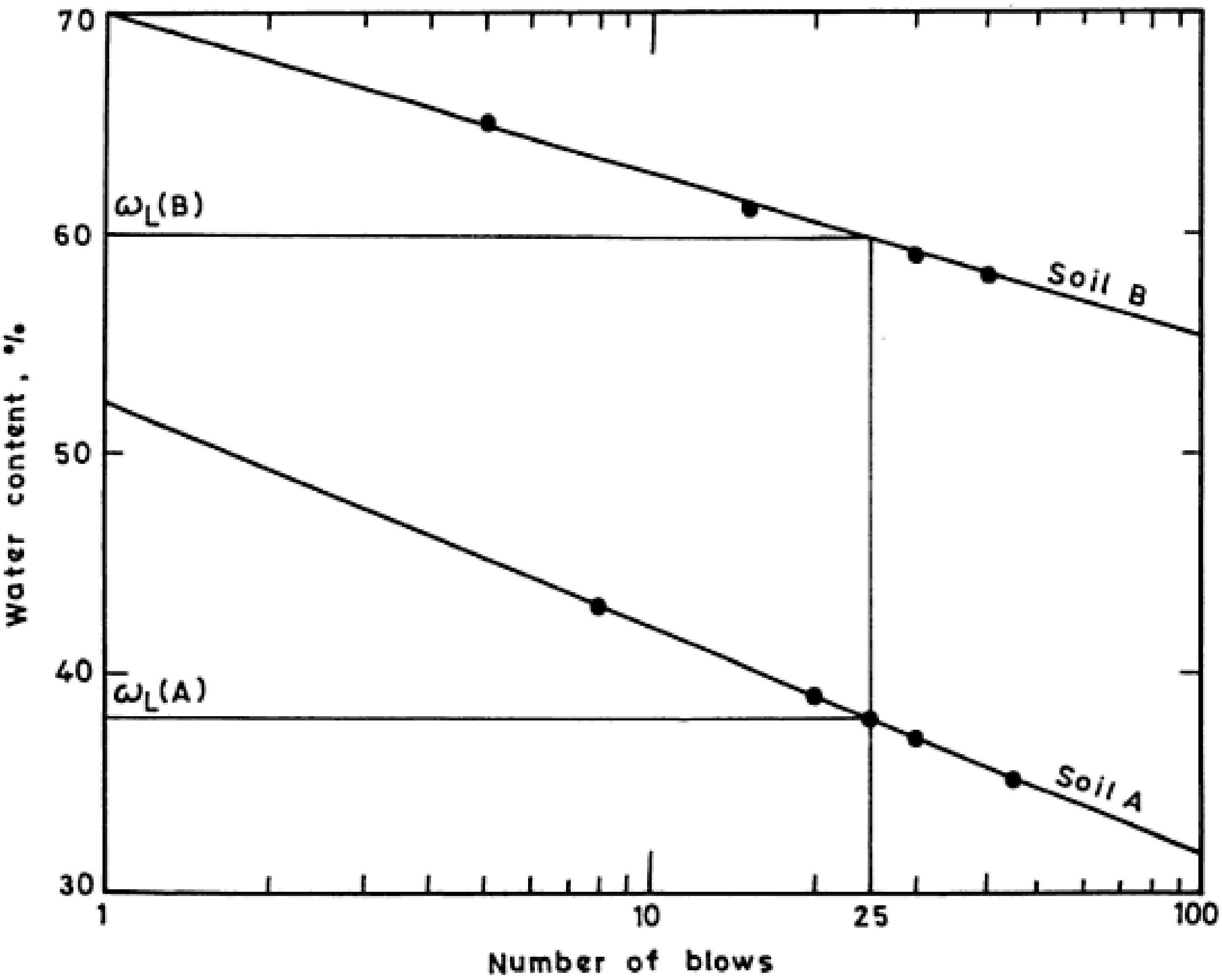


Fig. 2.31 Example 2.18

Solution:

Fig. 2.31 shows the flow curves for soils A and soils B.

From the figure, $(w_L)_A = 38\%$

$(w_L)_B = 60\%$

(a) $(I_P)_A = 38 - 25 = 13$

$(I_P)_B = 60 - 30 = 30$

Soil B has a higher degree of plasticity. Soils with $I_p > 17$ are highly plastic and soils with I_P between 7 and 17 are 'medium plastic', according to one empirical classification system.

(b) $$(I_L)_A = \frac{w_N - w_P}{I_P} = \frac{40-25}{13} = 1.15$$

$$(I_L)_B = \frac{50-30}{30} = 0.67$$

Soil A with $I_L > 1$ is in the liquid state of consistency and will, therefore, 'flow' like a viscous slurry *upon remoulding*. Soil B with $I_L = 0.67$ is in the plastic state and will be a better foundation material *upon remoulding*.

(c) Soil with $w_L = 60\ \%$ is more compressible than soil A with $w_L = 38\%$. Compressibility is a direct function of the liquid limit.

(d) From Fig. 2.31, Flow index, I_f of soil A = 42.0 – 32.0 = 10 (considering water contents at 10 and 100 blows)
I_f for soil B = 62.6 – 55.5 = 7.1

Since $(I_f)_A > (I_f)_B$, soil A shows greater loss of shear strength with increase in water content.

(e) Toughness index, I_t for soil $A = \dfrac{I_P}{I_f} = \dfrac{13}{10} = 1.3$

$$I_t \text{ for soil } B = \frac{I_P}{I_f} = \frac{30}{7.1} = 4.22$$

Soil B has a higher shear strength at plastic limit since a soil with higher toughness index possesses higher shear strength at plastic limit.

(f) Soils containing organic matter may have high liquid limit values but are also found to posses high plastic limit values and therefore low plasticity index values relative to their liquid limit values. Since both soil A and soil B have plastic limit values which are not very high, they are unlikely to contain any organic matter.

Example 2.19 An undisturbed sample of clay brought from the field was noted to have a volume of 18.0 cc and weight of 30.8 g. On oven drying, the weight of the sample was reduced to 20.5 g. The volume of dried sample as obtained by displacement of mercury was 12.5 cc. Calculate the shrinkage limit and the specific gravity of solids. What is the shrinkage ratio?

Solution:

From the data,

$W_1 = 30.8$ g; $V_1 = 18.0$ cc, $W_s = 20.5$ g, and $V_2 = 12.5$ cc (Fig. 2.16)

Substituting the values in Eq. 2.44,

$$w_s = \frac{(W_1 - W_s) - (V_1 - V_2)\,\gamma_w}{W_s} \times 100$$

$$w_s = \frac{(30.8 - 20.5) - (18.0 - 12.5)1.0}{20.5}$$

$$= 23.41\%$$

From Eq. 2.45, $$G_s = \frac{W_s}{V_1\gamma_w - W_1 + W_s}$$

$$= \frac{20.5}{18.0 \times 1.0 - 30.8 + 20.5}$$

$$= 2.66$$

$$\text{Shrinkage ratio } = \frac{\gamma_d}{\gamma_w} = \frac{20.5/125}{1} = 1.64$$

Alternatively, the problem can be solved using the phase diagrams. Phase diagram for the saturated and dry soil samples are shown in Fig. 2.32(a) and (b), respectively.

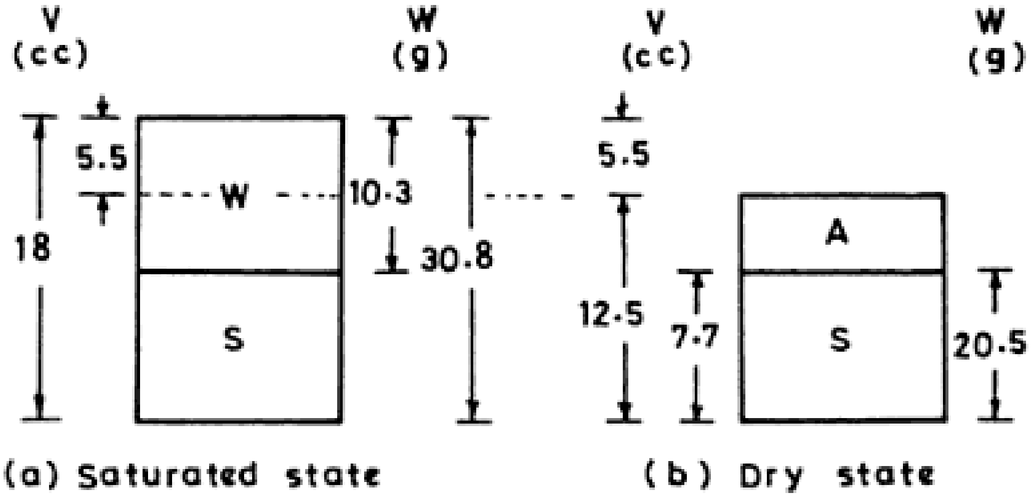


Fig. 2.32 Example 2.19

The weights and volumes as given in the problem are first shown on the phase diagrams. To determine the shrinkage limit, the weight of water that would fill in the void volume of Fig. 2.32 (b) has to be found out.

Weight of water at stage (a) $= 30.8 - 20.5$

$= 10.3$ g

Weight of water in Fig. 2.32(b) $= 10.3 - (18 - 12.5)\,\gamma_w$

$= 10.3 - 5.5 \times 1$

$= 4.8$ g

Shrinkage limit, w_S is the water content corresponding to Fig. 2.32(b)

$$W_S = \frac{4.8}{20.5} \times 100 = 23.41\ \%$$

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

$$V_s = 18 - \frac{10.3}{1.0}$$

$$= 7.7 \text{ cc}$$

$$G_s = \frac{20.5}{7.7 \times 1} = 2.66$$

Shrinkage ratio, $SR = \gamma_d/\gamma_w = \frac{20.5/12.5}{1} = 1.64$

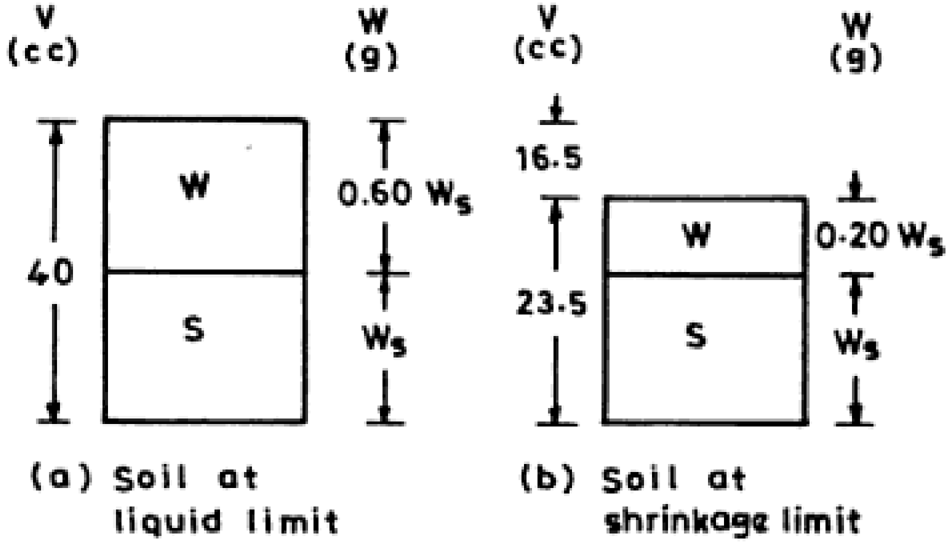
Example 2.20 The values of liquid limit, plastic limit and shrinkage limit of a soil were reported as follows:

$w_L = 60\%;\ w_P = 30\%;\ w_S = 20\%$

If a sample of this soil at liquid limit has a volume of 40 cc and its volume measured at shrinkage limit was 23.5 cc, determine the specific gravity of the solids. What is its shrinkage ratio?

Solution:

The phase diagrams for the soil at liquid limit and at shrinkage limit are shown below in Fig. 2.33 by assuming the weight of solids as W_s. Data about plastic limit is superfluous.



(a) Soil at liquid limit

(b) Soil at shrinkage limit

Fig. 2.33 Example 2.20

Since the soil continues to remain saturated till it reaches the state of consistency at shrinkage limit, the decrease in the volume of the soil mass will be equal to the decrease in the volume of water. Thus, from Fig. 2.33,

$$\frac{0.60\ W_s - 0.20\ W_s}{\gamma_w} = \text{Volume at LL - Volume at SL}$$

i.e., $0.40.\ W_s = (40 - 23.5) \times 1$

$W_s = 41.25\ g$

At shrinkage limit, $W_w = 0.20 \times 41.25 = 8.25$ g

$$V_w = \frac{8.25}{1} = 8.25 \text{ cc and}$$

$$V_s = 23.5 - 8.25 = 15.25 \text{ cc}$$

Specific gravity of solids $$G_s = \frac{W_s}{V_s \gamma_w} = \frac{41.25}{15.25 \times 1} = 2.70$$

Shrinkage ratio, $$SR = \frac{\gamma_d}{\gamma_w} = \frac{41.25}{23.5 \times 1} = 1.755$$

Example 2.21 The liquid limit and plastic limit of a soil are 50 % and 25 %, respectively. When the soil was dried from its state at liquid limit, the decrease in volume was 40 per cent of the volume at liquid limit. When it was dried from its state at plastic limit, the volume decrease was 20 per cent of the volume at plastic limit. Determine the shrinkage limit and shrinkage ratio.

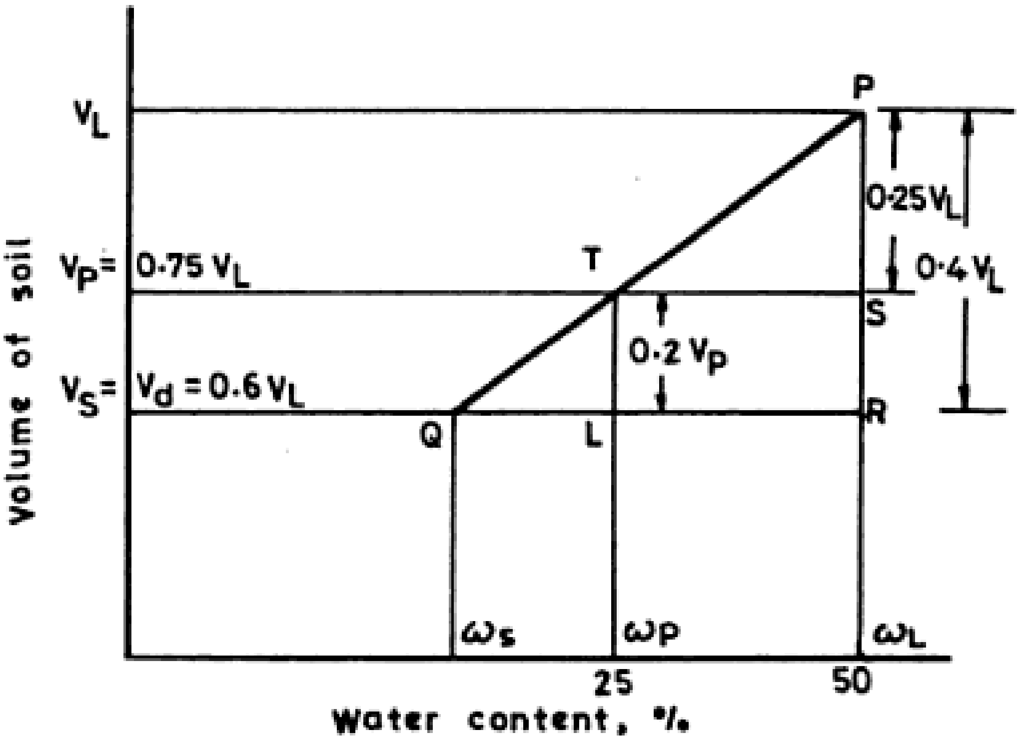


Fig. 2.34 Example 2.21

Solution:

Fig. 2.34 shows the volume changes that take place as the water content changes from the liquid limit upto the shrinkage limit. V_L, V_P and V_S denote the volumes of the soil mass at w_L, w_P and w_S respectively.

V_s is the same as the dry volume of the soil, V_d

$$V_s = V_d = V_L - 0.4\ V_L = 0.6\ V_L \text{ and}$$

$$V_d = V_P - 0.2\ V_P = 0.8\ V_P$$

$$V_P = \frac{0.6V_L}{0.8} = 0.75\ V_L$$

Triangles PQR and PTS are similar (Fig. 2.34);

$$\frac{QR}{PR} = \frac{TS}{PS} = \frac{0.50-0.25}{0.25V_L}$$

Hence, $$QR = \frac{0.4V_L}{0.25V_L}(0.25) = 0.4$$

Shrinkage limit, $$w_s = w_L - QR$$

$$= 0.5 - 0.4 = 0.1 \text{ or } 10\ \%$$

Shrinkage Ratio, $$SR = \frac{\frac{V_L - V_d}{V_d} \times 100}{w_L - w_S} = \frac{\frac{0.4V_L}{0.6V_L} \times 100}{50-10} = 1.66$$

PROBLEMS

1. Establish the following relationships:
   (a) $\gamma_d = (1-n)\,G_s\gamma_w$

   (b) $S = \dfrac{w}{\gamma_w/\gamma(1+w) - \dfrac{1}{G_s}}$

   (c) $n_a = \dfrac{e(1-S)}{1+e}$

   (d) $w = \dfrac{1 - G_m/G_s}{G_m/S - 1}$

   where γ = unit weight of soil,
   γ_d = dry unit weight of soil,
   γ_w = unit weight of water,
   S = degree of saturation
   e = void ratio
   G_s = specific gravity of solids
   G_m = mass specific gravity
   w = water content,
   n_a = percentage air voids
   n = porosity

2. Determine the unit weights in the loosest and densest packings of 2 mm dia. spheres which have a specific gravity of 2.70. (1.415 g / cc; 1.945 g/cc)

3. The natural water content of a sand sample is 20 per cent, the bulk unit weight being 1.8 g/cc. Assuming specific gravity of solids as 2.65 and the sand sample to be partially saturated, calculate the degree of saturation and void ratio of the sample. (69.1%; 0.767)

4. A sample of saturated clay weighs 15.45 g and its moisture content is 38.0 per cent. If the particle specific gravity is 2.70, find the void ratio, porosity, dry and bulk unit weights of the soil. (1.026;50.6%;1.33g/cc;1.84g/cc)

5. The weight of an undried specimen of clay was 34.62 g. The oven-dry weight of the same specimen was 20.35 g. Before drying, the specimen was immersed in mercury and its volume found to be 24.26 cc. Calculate the water content void ratio and the degree of saturation of the soil. Assume specific gravity of solids as 2.68. (70%;2.19;85.6%)

6. A sample of sand with the specific gravity of solids of 2.65 has a porosity of 40 per cent. Find out the dry unit weight, unit weight of the sample when fully saturated, submerged unit weight and bulk unit weight when the degree of saturation is 50 per cent. (1.59g/cc;1.99g/cc;0.99g/cc;1.79g/cc)

7. A natural soil deposit has a bulk unit weight of 18 kN/m^3 and a water content of 5%. Calculate the amount of water in litres required to be added to 1 cu m of soil to raise the water content to 15%. Assume the void ratio to remain constant. What will be the degree of saturation? Take $G_s = 2.60$. (174 litres ; $S = 79.6$ %)

8. The dry unit weight of a 4 m thick stratum of coarse sand is 15 t/m^3. Calculate the quantity of cement grout slurry required per meter run for providing a 2 m thick grout curtain through this stratum. G_s of sand grains= 2.68. (3.43 m^3)

9. A pycnometer test for the determination of water content of a soil sample having the $G_s = 2.70$, yielded following data:

   Weight of moist soil = 230.75 g

   Weight of pycnometer full of water = 2965.20 g

   Weight of pycnometer + soil+ water = 3092.85 g

   Calculate the water content of the soil. Derive the relationship used. (13.82)

10. A uniformly mixed heavy viscous suspension in water which has a total volume of 1000 cc has the following proportions of clay and sand particles and iron filings:
    79.5 g of clay with specific gravity of solids, G_s =2.70,
    82.5 g of sand with specific gravity of solids, G_s =2.65, and
    724.0 g of iron filings with specific gravity, G_s =7.23.
    Calculate the unit weight of the suspension. (1.725 g / cc)

11. During a sedimentation test for grain size analysis, the corrected hydrometer reading in a 1000 cc uniformly mixed soil suspension at the instant of starting of sedimentation (t = 0) was 1.030. After 30 min, the corrected hydrometer reading at an effective depth of 10 cm was noted to be 1.015. If the specific gravity of solids is 2.65 and viscosity of water is 0.01 dyne-sec/cm^2, find (i) the total weight of soil solids placed in the 1000 cc suspension, and (ii) the diameter corresponding to the 30 min reading and the percentage finer. (48.2 g; 0.00786 mm; 50%)

12. A soil sample with particle sizes ranging from 0.2 to 0.02 mm is placed on the surface of a still water pond 3 m deep. Determine the time in which the first particle reaches the bottom of the pond. How long will it take for the whole of the sample to settle? Assume $G_S = 2.67$, $\eta = 10$ millipoises. (82 s; 2h 17 m 22s)

13. In a liquid limit test, specimens of a certain sample of clay at water contents of 31.93, 27.62, 25.51 and 23.30 % required 5,16,23 and 42 blows, respectively to close the standard groove. The plastic limit of the clay is 13%. Natural water content is 18 %. Determine the liquid limit, plasticity index, liquidity index relative consistency, flow index and toughness index of the soil.

14. Two clays A and B have the following properties:

	Clay A	Clay B
Liquid limit, w_L%	44	55
Plastic limit, w_P %	20	35
Natural water content, w_N%	30	50

Which of the clays, A or B, would experience larger settlement under identical loads? Which of the soil is more plastic? Which of them is softer in consistency? (B; A; B)

15. A sample of soil with a liquid limit of 72.8% was found to have a liquidity index of 1.21 and water content of 81.3 %. What are its plastic limit and plasticity index? Comment on the consistency of the soil. (32.3%, 40.5%)

16. In a shrinkage limit test, a container of volume 9.6 cc was filled with soil slurry. The weight of the saturated soil was 17.46 g. The slurry was then gradually dried, first in atmosphere and then in an oven at a constant temperature of 110°C. The weight and volume of the dried soil were 11.58 g and 5.22 cc, respectively. Determine the shrinkage limit of the soil and the shrinkage ratio. (12.95%, 2.22)

17. A soil has a liquid limit of 50 % and plasticity index of 20. When the soil at its liquid limit was dried, the percentage decrease in volume was 40 % of its dry volume. When it was dried from its plastic limit, the percentage decrease in volume was 20 % of its dry volume. Determine the shrinkage limit of the soil and its shrinkage ratio. (10%, 1.0)

18. A sample of sand was found to have a water content of 20 % and bulk unit weight of 1.93 g /cc. Laboratory tests on the sand sample indicated the void ratios in the loosest and the densest possible states as 0.90 and 0.50, respectively. Calculate the relative density and the degree of saturation of the sample (Assume $G_s = 2.65$). (63.1%; 81.8%)

19. A sample of sand from a natural deposit has a porosity of 35 %. For a volume of 495 cc, the dry weight in the densest and loosest states are 950 and 700 g, respectively. Compute the relative density of sand assuming the specific gravity of solids to be 2.65. (67.9%)

20. The minimum and maximum dry unit weight of a sand were found to be 14.71 and 16.68 kN/m^3, respectively. Calculate the dry unit weight corresponding to a relative density of 50%. (15.65 KN/m^3)

21. Two clay soils have the following characteristics. Calculate their activity values. Compare their engineering behaviour:

	Clay A	Clay B
w_L %	60	50
w_P %	25	30
I_P %	35	20
% of minus 0.002 mm size	25	40

(Activity of clay A = 1.4; clay B = 0.5; clay A is more likely to undergo high volume change, its compressibility is higher but its permeability and rate of volume change are smaller than for clay B).

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3

Classification of Soils

3.1 INTRODUCTION

Soils, in general, may be classified as cohesionless and cohesive or as coarse-grained and fine-grained. These terms, however, are too general and include a wide range of engineering properties. Hence, additional means of categorisation are necessary to make the terms more meaningful in engineering practice. These terms are compiled to form *soil classification systems*.

A soil classification system is meant essentially to provide a language of communication between engineers. It may, thus, be considered a system of 'labelling', soils, which would convey the same meaning universally. Sorting of soils into groups each of which would show similar behaviour, is the object of soil classification. Any soil classification system must provide us with information about the probable engineering behaviour of a soil. This means that once a soil is classified, it should be possible to grade the soil for its suitability for a specific engineering project. A classification system must be simple to use, and the number of soil groups not too many in number. Most soil classification methods use simple index properties for placing a soil in a certain group. The most commonly used properties are the grain-size distribution and plasticity. However, a soil classification system does not eliminate the need for detailed soil investigations and for testing for engineering properties. It can at best give some fair idea about its engineering behaviour.

Several classification systems were evolved by different organisations having a specific purpose as the object. A. Casagrande (1948) describes the systems developed and used in highway engineering, airfield construction, etc. The two classification systems, which are adopted by the US engineering agencies and the State Departments, are the Unified Soil Classification System (USCS) and the American Association of State Highway and Transport Officials (AASHTO) system. Other countries, including India, have mostly the USCS with minor modifications.

In the following sections, the salient features of the USCS, AASHTO and the Indian Standard Soil Classification System (ISSCS) are discussed.

3.2 THE UNIFIED SOIL CLASSIFICATION SYSTEM

The Unified Soil Classification System (USCS), originally developed by Casagrande (1948), was intended for use in airfield construction during World War II. Later, in 1952, it was slightly modified to make it applicable to foundations, dams and other constructions. According to the USCS, the coarse-grained soils are classified on the basis of their grain-size distribution and the fine-grained soils (whose behaviour is controlled by plasticity) on the basis of their plasticity characteristics.

Table 3.1 shows the salient features of USCS. All the soils are classified into four major groups, nam coarse grained, fine grained, organic soils and peat. Group symbols consisting of a prefix and a suffix are us for various groups (Table 3.2).

Coarse-grained soils are those having 50 per cent or more retained on the No. 200 (0.075 mm) sieve. They are further subdivided into gravels and sands. The coarse-grained soils are designated as gravel (*G*) if 50 per cent or more of the coarse fraction (+75 µ) is retained on the No. 4 (4.75 mm) sieve; otherwise, they are designated as sand (*S*). Both gravel and sand are divided into four subgroups (Table 3.2), namely, *GW* and *SW*, *GP* and *SP*, *GM* and *SM*, *GC* and *SC* depending on the grain-size distribution and nature of fines in the soils.

A well-graded (*W*) soil has a wide distribution of grain sizes present, whereas a poorly-graded (*P*) soil is either uniform or gap graded. Whether a soil is well-graded or poorly-graded can be determined by plotting the grain size distribution curve and computing the coefficient of uniformity, C_u and the coefficient of curvature, C_c. These coefficients are defined in Chapter 2.

Table 3.2 Prefix and Suffix of USCS

Soil type	*Prefix*	*Subgroup*	*Suffix*
Gravel	*G*	Well graded	*W*
Sand	*S*	Poorly graded	*P*
Silt	*M*	Silty	*M*
Clay	*C*	Clayey	*C*
Organic	*O*	$w_L > 50$ per cent	*L*
Peat	*Pt*	$w_L > 50$ per cent	*H*

The gradation criteria for gravelly and sandy soils are shown in Table 3.1. *GW* and *SW* groups are well-graded gravels and sands with less than 5 per cent passing 75µ sieve whereas the *GP* and *SP* groups are poorly-graded gravels and sands with little or no fines.

Fine-grained soils having more than 50 per cent material passing the no 200 (0.075 mm sieve). These are subdivided into silt (*M*) and clay (*C*), based on their liquid limit and plasticity index. As shown in Table 3.1, organic soils are also included in this fraction. The *A*-line on the Casagrande's (1948) plasticity chart (Table 3.1) generally separates the more clay like materials from those that are silty and also the organic from the inorganic. Silt, clay and organic fractions are further subdivided into soils possessing low (*L*) or high (*H*) plasticity when the liquid limit is less than 50 per cent and more than 50 per cent, respectively.

Highly organic soils, fibrous in nature, usually peat and swampy soils having high compressibility are not subdivided. These are put into one group only with group symbol, P_t.

Soils possessing the characteristics of more than one group can be termed as boundary soils and, hence, are assigned dual group symbols, *e.g.*, *GW-GC*, meaning a well-graded gravel with some clay fines. The system has also the field identification procedure incorporated into it.

3.3 AASHTO SOIL CLASSIFICATION SYSTEM

The US Bureau of Public Roads (now the Federal Highway Administration) developed, in the late twenties, a classification system called the Public Roads Administration classification system which was specifically meant for use in road construction. The original system has been revised several times. The present AASHTO (1978) soil classification system is essentially the system revised in 1945. The system is based on both the particle size and the plasticity characteristics. The applicability of the system has also been extended considerably.

According to the AASHTO system, the soils are classified into eight groups; *A*-1 through *A*-7 with an additional group *A*-8 for peat or muck. The system includes several sub-groups (Table 3.3). Soils within each group are evaluated according to the *group index* calculated from the empirical formula,

$$\text{Group index, } GI = 0.2\,a + 0.005\,ac + 0.01\,bd$$

where a = that part of the percent passing the 75 μ sieve (–75 μ) greater than 35 and not exceeding 75, expressed as a positive whole number (range 1 to 40)

b = that part of the percent passing the 75 μ sieve greater than 15 and not exceeding 55, expressed as a positive whole number (range 1 to 40)

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

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

While calculating *GI* from Eq. 3.1, if any term becomes negative, it is dropped. The group index should be rounded off to the nearest whole number. If the computed value is negative, it is reported as zero.

To classify a soil, its grain-size distribution, liquid limit and plasticity index values are determined. Once these data are available, the classification is carried out by proceeding from left to right in the chart of Table 3.3 and selecting the first group that fits the test data. A complete classification includes the group index in parentheses. Thus, A-2-6 (3) indicates a soil type A-2-6, having a group index of 3.

In general, the greater the group index value, the less desirable a soil is for highway construction within that subgroup. A group index of 0 indicates a good subgrade material, while a group index of 20 or more indicates a very poor subgrade material.

3.4 INDIAN STANDARD SOIL CLASSIFICATION SYSTEM

The Indian Standard Soil Classification System (ISSCS) for classification of soils for general engineering purpose, first published in 1959, was revised in 1970. The revision is based on the USCS with the modification that the fine-grained soils have been subdivided into three subgroups of low, medium and high compressibility as against only two in the USCS. The plasticity chart as per ISSCS is shown in Fig. 3.1. The major soil groups and subgroups, along with the corresponding prefix and suffix that are used in the ISSCS are shown in Table 3.4. The laboratory classification criteria for the coarse-grained soils is the same as in the USCS and shown in Table 3.1.

The basic soil components in the ISSCS are given in Table 3.5. According to Table 3.5, the division of soil fractions purely on the basis of grain size (in mm) is as shown the Table below. The maximum possible range of grain size in soils is indeed tremendous – of the order of 10^8.

Table 3.3 AASHTO Soil Classification System

General classification	*Granular materials (35 per cent or less passing 0.075 mm)*							*Silt-clay materials (More than 35 percent passing 0.075 mm)*			
Group classification	*A-1*		*A-3*	*A-2*				*A-4*	*A-5*	*A-6*	*A-7*
	A-1a	*A-1b*		*A-2-4*	*A-2-5*	*A-2-6*	*A-2-7*				*A-7-5;* *A-7-6*
Sieve analysis per cent passing											
No. 10 (2 mm)	50 max										
No. 40 (0.425 mm)	30 max	50 max	51 max								
No. 200 (0.075 mm)	15 max	25 max	10 max	35 max	35 max	35 min	35 min	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40 (0.425 mm)											
Liquid limit				40 max	41 max	40 max	41 max	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P	10 max	10 max	11 max	11 max	10 max	10 max	11 max	11 min
Group index	0		0	0		4 max		8 max	12 max	16 max	20 max
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

A-8, peat or muck is by visual classification and is not shown in the Table.

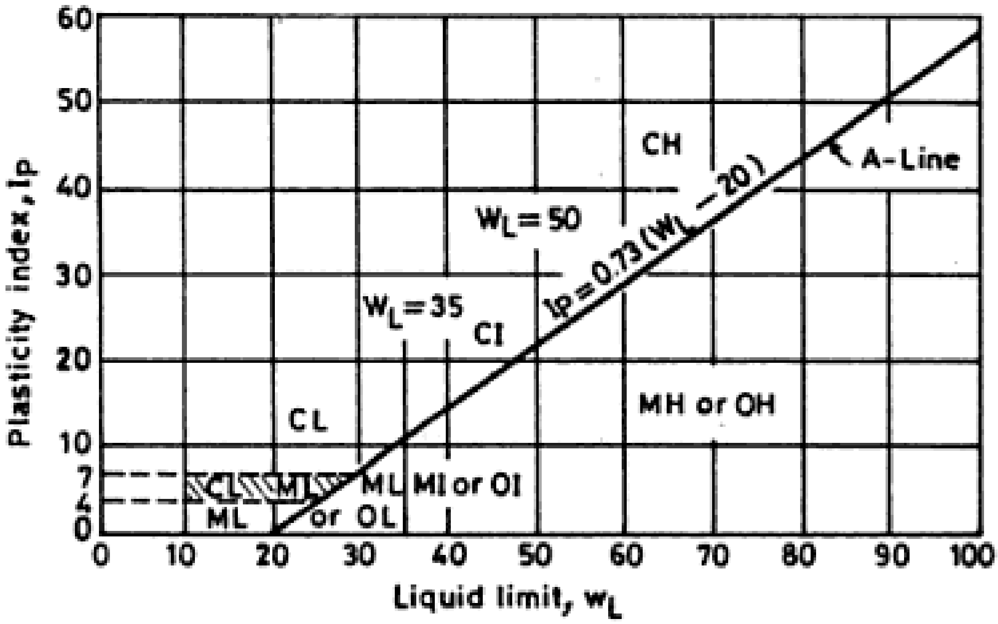

Fig. 3.1 Plasticity chart as per Indian Standard Soil Classification System

Table 3.4 Prefixes and Suffixes of ISSCS

Soil type	*Prefix*	*Subgroup*	*Suffix*
Gravel	*G*	Well graded	*W*
Sand	*S*	Poorly graded	*P*
Silt	*M*	Silty	*M*
Clay	*C*	Clayey	*C*
Organic	*O*	$w_L < 35$ percent	*L*
		$35 < w_L < 50$	*I*
Peat	*Pt*	$50 < w_L$	*H*

The classification of coarse-grained soils is done on the basis of their grain and gradation characteristics, as illustrated in Table 3.1 of USCS, when the fines (< 75 μ) present in them are less than 5% by weight.

The fine-grained soils are classified on the basis of their plasticity characteristics using the I.S. Plasticity chart shown in Fig. 3.1.

Coarse-grained soils which contain more than 12% fines (< 75 μ) are classified as *GM* or *SM* if the fines are silty in character (meaning, the limits plot below the *A*-line on the plasticity chart); they are classified as *GC* or *SC* if the fines are clayey in character (meaning the limits plot above the *A*-line on the plasticity chart).

Coarse-grained soils having 5% to 12% fines are borderline cases and given a dual symbol. The first part of the dual symbol is indicative of the gradation, while the second part indicates the nature of fines. For example, a soil with the dual symbol *SW-SC* is a well-graded sand with 'clayey' fines that plot above *A*-line.

Table 3.5 Basic Soil Components

Sl. No.	*Soil component*	*Symbol*	*Particle-size range and descripition*
	Boulders	None	Rounded to angular, bulky, hard, rock particle; average diameter mor than 300 mm
	Cobbles	None	Rounded to angular, bulky, hard, rock particle; average diamete smaller than 300 mm but retained on 80 mm IS sieve
(1)	Coarse-grained soils		
	Gravel	*G*	Rounded to angular, bulky, hard, rock particle; passing 80 mm IS sieve but retained on 4.75 mm IS sieve Coarse: 80 mm to 20 mm Fine : 20 mm to 4.75 mm
	Sand	*S*	Rounded to angular, bulky, hard, rock particle; passing 4.75 mm IS sieve but retained on 75 micron Coarse: 4.75 mm to 2.0 mm Medium: 2.0 mm to 425 micron Fine: 425 micron to 75 micron
(2)	Fine-grained soils		
	Silt	*M*	Particles smaller than 75 micron; identified by behaviour, that is slightly plastic or non-plastic regardless of moisture and exhibits littl or no strength when air dried.
	Clay	*C*	Particles smaller than 75 micron; identified by behaviour, that is, i can be made to exhibit plastic properties within a certain range o moisture and exhibits considerable strength when air dried.
(3)	Organic matter	*O*	Organic matter in various sizes and stage of decomposition (n specific grain size)
(4)	Peat	*Pt*	Fibrous, spongy (no specific grain size)

Division of Soil Fractions on the Basis of Grain Sizes

Boulder	Cobble	Coarse-grained soils					Fine-grained soils
		Gravel		Sand			Silt and Clay size
		Coarse	Fine	Coarse	Medium	Fine	

>300mm 300 80 20 4.75 2.0 0.425 0.075 < 0.075 mm

Fine-grained soils also can have dual symbols. If the limits plot in the hatched zone on Fig. 3.1 (I_P betwee 4 and 7), the soil has a group symbol *CL–ML*. If the w_L and I_P values fall close to the *A*-line, dual symbols ar used. Again, if the liquid limit is very close to 35 % or 50 %, dual symbols are used. Possible dual symbol are: *CL-ML, CI-MI, CH-MH, ML-MI, MI–MH, CL-CI, CI-CH, OL-OI, OI-OH.*

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

A step-by-step procedure for classifying the soils as per IS: 1498–1970 is presented in Fig. 3.2 in th form of a flow-chart. The classification is done using this chart and Table 3.1 and Fig. 3.1. The procedure i illustrated below:

3.5 APPLICATION OF SOIL CLASSIFICATION

Certain important engineering properties of soils belonging to different soil groups in the ISSCS and their relative suitability for various engineering projects are given in Table 3.7. The relative desirability for various uses is given numerical ratings - no. 1 being the best, no. 2 second best and so on. This kind of rating is intended as a guide in comparing soils for a specific purpose. The soil engineer must however, be cautious in the use of soil classification. Designs should never be based on soil classification alone. Fundamental soil behaviour, like flow phenomenon, compressibility and shearing strength, can be predicted only through direct measurement in the laboratory. Index tests which are used for classifying soils are conducted on remoulded soils and as such, may not be truly representative of the behaviour of the *in situ* undisturbed soil. Yet, soil classification plays an important role in decision-making and indicates the direction in which detailed investigation must be carried out.

Table 3.7 Comparative Engineering Properties of Soil Groups

	Important Properties				*Relative grading*					
					Rolled Earth Dams			*Foundations*		*Roadways Surfacing*
Group Symbol	*Permeability when Compacted*	*Shearing Strength when Compacted and Saturated*	*Compressibility when Compacted and Saturated*	*Workability as Construction Material*	*Homogeneous Embankment*	*Core*	*Shell*	*Seepage Important*	*Seepage not Important*	
GW	pervious	excellent	negligible	excellent	—	—	1	—	1	3
GP	very pervious	good	negligible	good	—	—	2	—	3	—
GM	semi pervious to impervious	good	negligible	good	2	4	—	1	4	5
GC	impervious	good to fair	very low	good	1	1	—	2	6	1
SW	pervious	excellent	negligible	excellent	—	—	3	—	2	4
SP	pervious	good	very low	fair	—	—	4	—	5	—
SM	semi pervious to impervious	good	low	fair	4	5	—	3	7	6
SC	impervious	fair	low	good	3	2	—	4	8	2
ML	semi pervious to impervious	fair	medium	fair	6	6	—	6	9	—
CL	impervious	fair	medium	good to fair	5	3	—	5	10	7
OL	semi pervious to impervious	poor	medium	fair	8	8	—	7	11	—
MH	semi pervious to impervious	fair to poor	high	poor	9	9	—	8	12	—
CH	impervious	poor	high	poor	7	7	—	9	13	—
OH	impervious	poor	high	poor	10	10	—	10	14	—
Pt	—	—	—	—	—	—	—	—	—	—

3.6 TEXTURAL CLASSIFICATION OF SOILS

Texture of a soil describes how a soil 'feels' and is influenced by the size, shape and gradation of soil particles in a soil. Coarse-grained soils such as sands and gravels have a coarse texture, while fine-grained soils like silts and clays which are composed of mineral grains tiny and flaky in character, are fine-textured.

Soil texture has been used as the basis for soil classification, but mainly in agronomy. In soil engineering, however, it is only in the case of coarse-grained soils that soil texture has a correlation with their engineering behaviour. Plasticity and consistency become all important when it comes to fine-grained soils, in which the presence of water affects mineral grain interaction.

Texture classification terms, namely, gravels, sands, silts and clay are commonly used in engineering practice and serve a useful purpose inasmuch as these terms are associated with certain characteristics which are typical of these soils. Table 3.8 shows some of the general engineering characteristics associated with gravels, sands, silts and clays. Silts fall between gravels and sands on one hand and clays on the other and must be seen as "in between" material.

Table 3.8 General Characteristics of Soils

Soil characteristics	*Gravels, Sands*	*Silts*	*Clays*
Grain size	Granular, coarse-grained, particles can be seen by naked eyes	Fine-grained, cannot see individual particles	Fine-grained, cannot see individual particles
Plasticity and cohesion	Nonplastic, cohesionless	Slight or no plasticity, and cohesion	Plastic, cohesive
Effect of grain size distribution	Important	Less important	Unimportant
Effect of water	Unimportant (except for loose, saturated soils under dynamic loadings)	Important	Very important
Permeability and Drainage	Pervious Freely draining	Less pervious	Almost impervious
Compressibility	Low	Medium	High
Shear strength	Depends on relative density (generally high)	Intermediate	Depends on consistency (generally poor)

From Table 3.8, it is clear that gravels and sands possess engineering characteristics which are to be preferred for most of the engineering projects. Clays are the least preferred soils while silts are better than clays but not as good as the granular soils.

EXAMPLES

Example 3.1 The following data refers to the classification tests carried out on three samples from the clinker grinding unit of a cement factory at Bhatinda.

Sieve size	*Per cent finer*		
	Sample 1	*Sample 2*	*Sample 3*
4.75 mm	100.00	100.00	100.00
2.00 mm	100.00	99.25	100.00
1.00 mm	99.80	98.75	99.90
450 micron	99.65	98.00	99.70
212 micron	98.95	54.15	90.00
150 micron	97.55	7.60	81.50
75 micron	96.85	6.00	63.00
Liquid limit	23.00	—	—
Plastic limit	16.77	—	—
Plasticity index	6.23	Non-plastic	Non-plastic

Classify the soils as per Indian Standard Soil Classification System.

Solution:

The grain-size distribution curves for the three samples are shown in Fig. 3.3

Sample 1: We see that more than 50 percent passes 75 micron IS sieve. Thus, the soil is fine-grained and plasticity characteristics are required to further classify the soil. With a liquid limit of 23.0 percent and plasticity index of 6.23, the soil plots in the hatched zone on the plasticity chart. The soil is, therefore, classified as *CL-ML* as per ISSCS.

Sample 2: it is a coarse-grained soil since only 6 percent passes through the 75 micron sieve. As a greater percentage of the coarse fraction passes through 4.75 mm sieve, the soil is sand. The amount of soil passing through 0.075 mm sieve is between 5 percent and 12 percent, hence the soil will have a dual symbol such as *SP–SM* or *SW–SM* depending upon the values of C_u and C_c. From the grain-size curve, $D_{60} = 0.22$ mm, $D_{30} = 0.19$ mm and $D_{10} = 0.16$ mm.

Hence,
$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.22}{0.16} = 1.38 < 6$$

and
$$C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} = \frac{(0.19)^2}{0.22 \times 0.16} = 1.03$$

For a soil to be considered well graded, $C_u > 6$ and $1 < C_c < 3$. These criteria are not satisfied; so the soil is poorly graded and is classified as *SP–SM* (NP). *SM* is because the fines are non-plastic silt.

Sample 3: The soil is fine grained as more than 50 percent passes through the 75 micron IS sieve. Further, the soil is non-plastic. The soil is, thus, classified as *ML* (NP).

Example 3.2 The classification tests on a sample of soil from a highway project site reveal that 60 percent of the soil passes US sieve no. 200. The liquid limit of soil mis 38 percent and the plastic limit 25 percent. Calculate the group index of the soil and classify the soil as per AASHTO system.

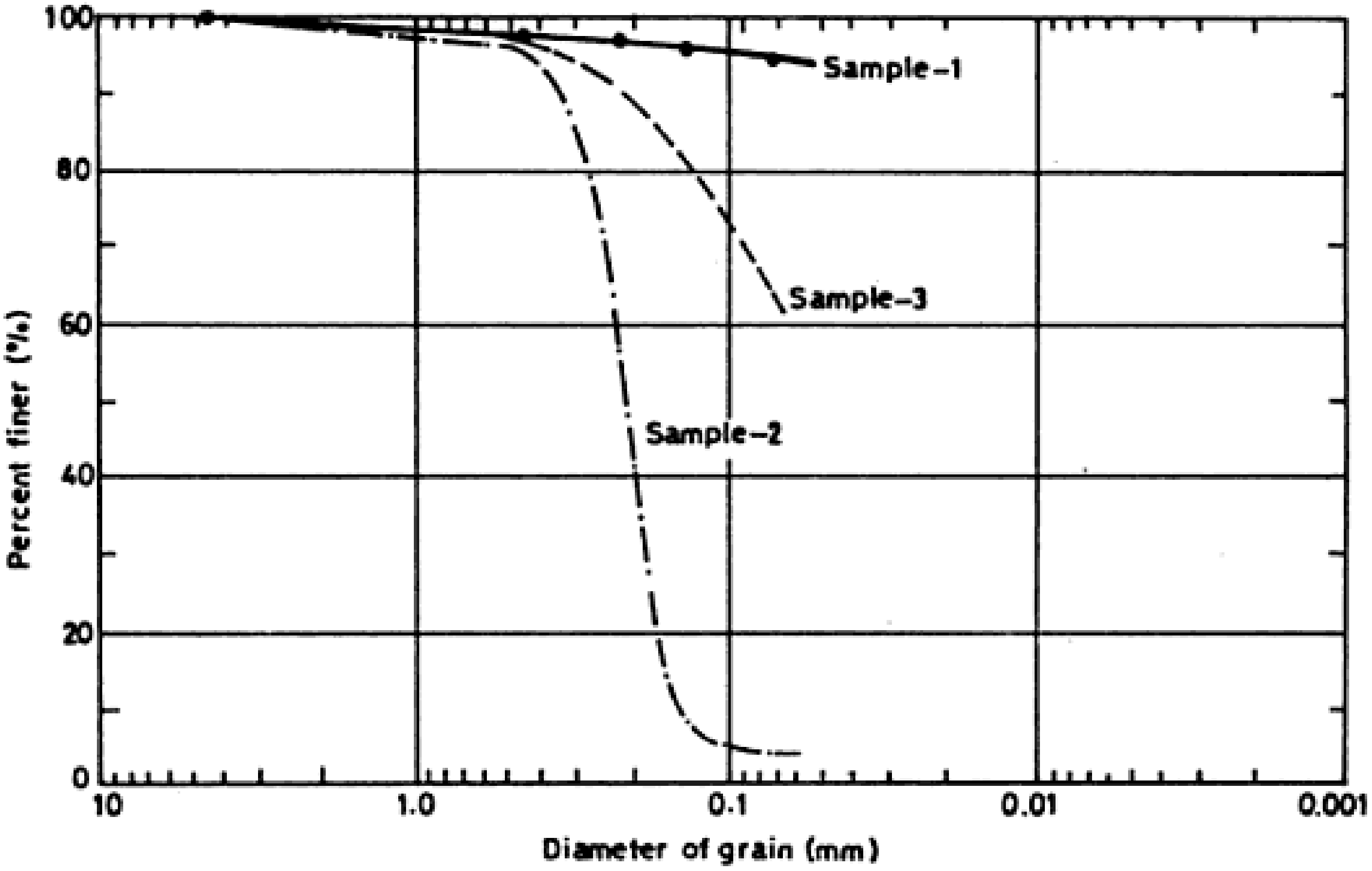

Fig. 3.3 Example 3.1

Solution:

From Table 3.3, as more then 35 percent of soil passes through sieve no. 200, the soil is in the group *A-4* or higher. Since the liquid limit is 38 percent, the soil is either *A-4* or *A-6*. With plasticity index of 13 percent, the soil is in *A-6* group.

Substituting the values in Eq. 3.1,

Group index = 0.2(25) +0.005(25)(0) + 0.01(40)(3)

= 5.0 + 1.20 = 6.20

Hence, the group of the soil is *A-6(6)*

Example 3.3: Classify soils on the basis of the data provided, as per IS: 1498-1970. Where additional information is required, say what data is needed.

Soil	w_L	w_P	*% passing through 75 μ sieve*	*% gravel > 4.75 mm*	*% sand (4.75 mm) – (0.075 mm)*
A	450	50	100	0	0
B	34	20	80	0	20
C	60	30	90	0	10
D	—	Non-plastic	100	0	0
E	35	20	20	60	20
F	—	Non-plastic	10	20	70

Solution:

Soil A: 75 μ fraction > 50 %. Hence, it is a fine-grained soil, $w_L = 450\%$, $I_P = 400$. The coordinates plot above the A line on the plasticity chart, in the *CH* region. It is, thus, an inorganic clay of high compressibility. Values of liquid limit and plastic limit are unusually high. These are typical data from the Mexico city clay.

Soil B: 75 μ fraction > 50 %. It is a fine-grained soil. $w_L = 34\%$; $I_P = 14$. The point lies above the A line but very close to the $w_L = 35$ % line. Hence, the soil can be assigned the dual symbol *CL–CI*. It is an inorganic clay of low to intermediate compressibility.

Soil C: It is a fine-grained soil with $w_L = 60$ % and $I_P = 30$. On the plasticity chart, the point plots too close to the A line. Hence, the soil can be assigned the dual symbol *CH-MH*. It is an inorganic clay to silt of high compressibility.

Soil D: 75 μ fraction is 100%. Hence, it is a fine-grained soil. However, the soil is non-plastic and the plasticity chart cannot be used. It can, thus, classified as *ML* (NP) *i.e.*, non-plastic silt of low compressibility.

Soil E: % of soil retained on the 75 μ sieve = 80, *i.e.*, > 50%. Hence it is a coarse-grained soil. Since a larger proportion of the coarse-grained soil is gravel, the soil is gravel. Since -75 μ fraction is 20% (*i.e.*, more than 12%), we have to use the plasticity chart to complete the classification.

Here $w_L = 35$ % and $I_P = 15$. The point plots above the A-line. Hence, the soil is classified as *GC*, *i.e.*, clayey gravel.

Soil F: It is a coarse-grained soil and the soil is sand. The –75 μ fraction is 10% that is, greater than 5% but less than 12%. Hence, a dual symbol has to be used – the first reflecting the gradation and the second related to its plasticity. Since grain-size distribution is not given, the first symbol can be either *SP* or *SW*. The group symbol of the soil can be either *SP-SM* (NP) or *SW-SM* (NP), because the soil is non-plastic and the plasticity chart cannot be used. For the correct classification, values of C_u and C_c have to be obtained from the GSD curve.

PROBLEMS

1. Given the data of Fig. 2.11, classify the soils using the AASHTO system.

Soil	w_L	w_P	I_P
1	35	21	14
2	—	—	—
4	42	26	16

2. Figure 3.4 shows the grain-size curves for five different soils. The Atterberg limits of these soils are given below:

Soil	w_L	w_P	I_P
1	—	—	—
2	—	—	—
3	20	15	5
4	45	31	14
5	90	52	38

Classify the soils as per (a) USCS (b) ISSCS and (c) AASHTO

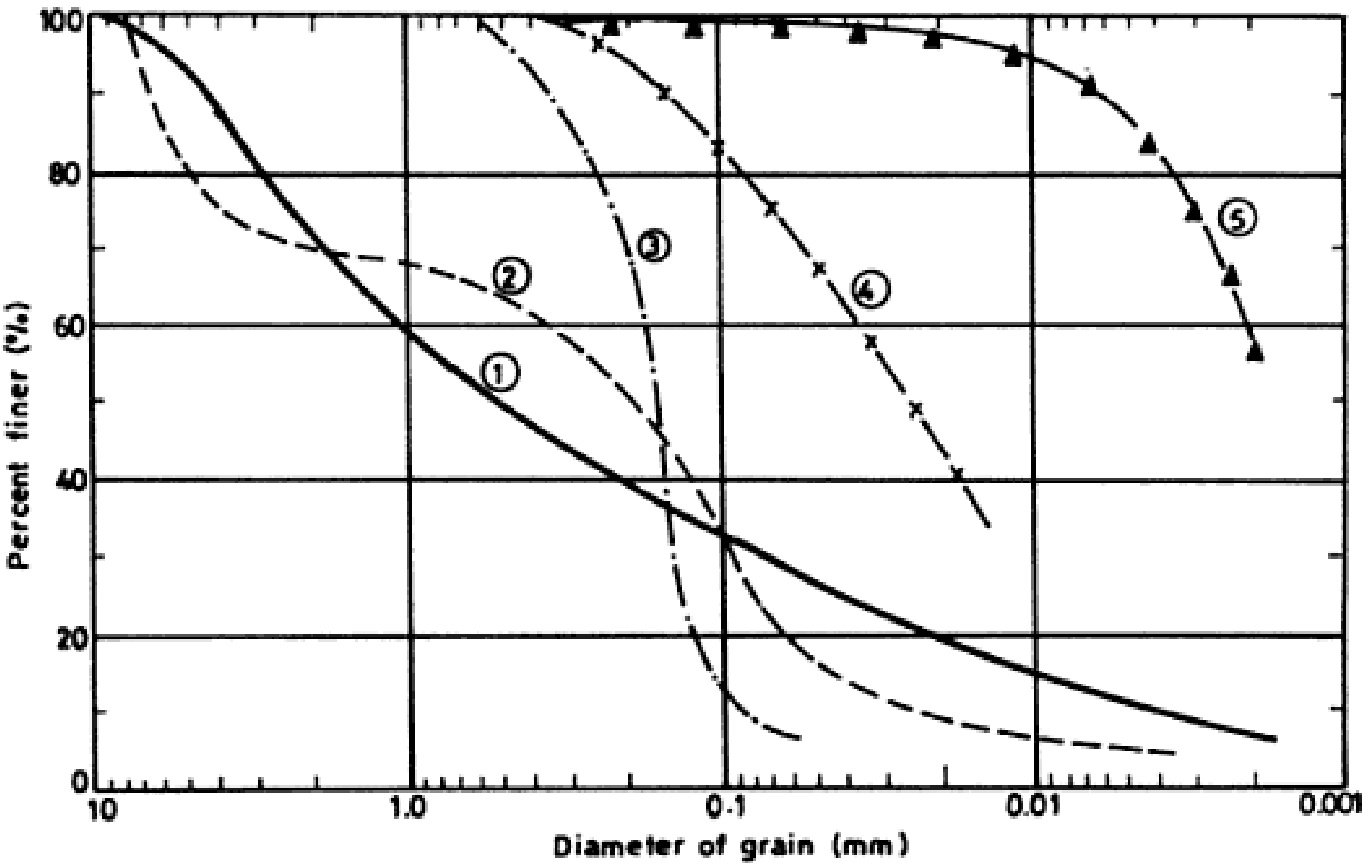

Fig. 3.4 Problem 2

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4

Soil Structure and Clay Minerals

4.1 INTRODUCTION

One would normally expect the engineering behaviour of a soil mass to be greatly influenced by the mineral composition of the soil grains forming the soil mass. This, however, is only partly true. In case of coarse-grained soils, the mineralogical composition of the grains hardly affects the engineering properties of the soils. Perhaps the grain to grain friction is influenced to a degree. In such soils, interparticle forces other than those due to gravity are of no consequence. But the finer the particles, the more significant become the forces associated with the surface area of the grains. The chemical character of the individual grain assumes importance especially when the surface area is large relative to the size of the grain—a condition which is associated with fine-grained soils. Thus, interparticle attraction holding the grains together becomes increasingly important as the size decreases.

Soil structure means the mode of arrangement of the soil particles relative to each other and the forces acting between them to hold them in their positions. The concept is further extended to include the mineralogical composition of the grains, the electrical properties of the particle surface, the physical characteristics, ionic composition of the pore water, the interactions among the solid particles, pore water and the adsorption complex.

The formation of soil structure is governed by several factors. In coarse-grained soils, the force of gravity is the main factor, while in fine-grained soils, the surface bonding forces become predominant. The *specific surface* (the ratio of the surface area of a material to its mass or volume) is a parameter which is often used to decide the importance of surface bonding forces relative to forces of gravity. Smaller particles have much larger specific surface than the larger particles. Though this value for soil particles is not computed as they are far too irregular in shape, it is clear that a soil mass made up of many small particles will have a much larger specific surface than the same material made up of large particles. Another fact which emerges from the concept of specific surface is that for the same void ratio, water contents are more for fine grained soils than for coarse grained soils. Also, the quantity of water needed to just wet the surface of smaller particles in a given volume of soil is many times more than what is needed in the case of larger particles.

'Clay' is normally understood to mean a clay soil whose grains are predominantly composed of clay minerals and which has plasticity and cohesion. Though the clay soils are fine-grained, not all fine-grained soils possess plasticity and cohesion. The presence of water, which is relatively unimportant in coarse-grained soils, plays a decisive role in the engineering behaviour of a clay soil. On the other hand, grain-size distribution and grain shape influence the engineering behaviour of granular soils and hardly affect the behaviour of clay soils. Silts, which can be both granular and fine-grained, are somewhere in between. They are noncohesive and non-plastic, but water does affect their behaviour in the sense that they are dilatant even though their strength is not very much affected by water content.

The ingredients necessary to give a soil deposit cohesion are *clay minerals*. Clay minerals are very active electrochemically and the presence of even a small amount of clay minerals can appreciably alter the engineering properties of a soil mass. Generally, when over 50 per cent of the soil deposit consists of particles whose diameter is 0.002 mm or less, the deposit is termed *clay*. When the percentage of clay content is about 50, the larger soil particles are suspended in a clay matrix, and have little effect on the properties of the soil mass.

Some of the important clay minerals, their identification, their interaction with water and the nature of atomic structure of these minerals are briefly described in the following sections.

4.2 CLAY MINERALS

The clay materials are basically of composed of tiny crystalline substances of one or more members of a small group of minerals—commonly known as clay minerals. These clay minerals are evolved mainly from the chemical weathering of certain rock-forming minerals. Chemically, these minerals are hydrous aluminosilicates with other metallic ions. Their particles are very small in size, very flaky in shape and thus have considerable surface area. They can only be viewed with an electron microscope.

The clay minerals, on the basis of their crystalline arrangement, can be divided into three main groups. It is observed that more or less similar engineering properties are exhibited by all clay minerals belonging to a particular group. Table 4.1 gives the group of minerals and the most important clay minerals in each of these groups.

Table 4.1 Clay Minerals

Name of mineral and group	*Structural formula*
Kaolin group	
(a) Kaolinite	$(OH)_4.Al_4Si_4O_{10}$
(b) Halloysite	$(OH)_4.Al_4Si_4O_{10}.4H_2O$
Montmorillonite group	
Montmorillonite	$(OH)_4.Al_4Si_gO_{20}.nH_2O$
Illite group	
Illite	$(OH)_4K_y(Si_{g-y}Al_y)(Al_4.Mg_6.Fe_4.Fe_6)O_{20}$

Atomic Bonds

The chemical elements of the rock forming minerals are composed of atoms, the ultimate units with discrete chemical identity. The basic concepts concerning atomic structure are fairly well known. Atoms unite together to form molecules, and molecules with other molecules to build up the structure of substances. The forces binding them together are primarily of electrical nature and it is necessary to know about them before the behaviour of the clay particles and of the aggregate of these particles can be discussed.

There are two main groups of atomic bonds, primary—of high energy bonds which hold the atoms together, and secondary—of weak bonds which link molecules.

Primary Valence Bond is due to a chemical combination of atoms of two elements which lack a complete complement of electrons in their outermost or valence shells. One atom joins with another atom by adding some electrons to its outer shell or by losing them to arrive at a stable configuration. The process usually would mean the loss or addition of the fewest number of electrons. The actual number of electrons an atom gains or loses will be dependent on the valency of the element involved. Atoms which have lost or gained electrons in this manner are called *ions*, and the forces binding them together are called *ionic bonds*. Atoms may give or receive several electrons and the resulting ion is given the notation Na^+, Al^{3+}, Si^{4+}, Cl^-, O^{2-}, etc. In the formation of a molecule, such as Al_2O_3, there are 2 ions of Al and 3 ion of O because each Al ion has an excess of 3 electrons in its outer shell, while O has a deficit of two electrons in its outer shell. Al ion gives excess electrons to O ion to form a molecule of Al_2O_3. The ionic bonds are very strong and are not normally broken up in engineering practice.

Secondary Valence Bonds—While the primary valence bonds combine the atoms into molecule, the forces that link atoms in one molecule to atoms in another molecule produce secondary valence bonds. These are much weaker than the primary valence bonds. The secondary valence bonds are of two kinds—van der Waals forces and hydrogen bond.

The *van der Waals forces* are the attractive forces between the surfaces of two parallel clay mineral particles separated by water. These forces depend essentially on the crystal structure of the minerals and on the distance separating the particles. Since the force varies inversely as a power of the distance of separation, the force between two non-parallel crystal surfaces is not merely a function of the average distance between them but is also dependent on the orientation of the surfaces with respect to each other.

In some electrically neutral molecules, such as water molecules, the centres of positive and negative polarity do not coincide, and thus the molecules behave like permanent magnets or dipoles. The force of attraction between the oppositely charged ends of such dipoles is one component of the van der Waals forces and is termed the *orientation effect* (Fig. 4.1). Secondly, the attractive force between permanent dipoles and dipoles induced by these in adjacent originally non-polar molecules is termed *induction effect*, and thirdly, the interaction between instantaneous, fluctuating dipoles due to the constant oscillation of the electrons is termed *dispersion effect*. The contributions of these effects to the van der Waals forces are: Orientation 77 per cent, Dispersion 19 per cent and Induction 4 per cent.

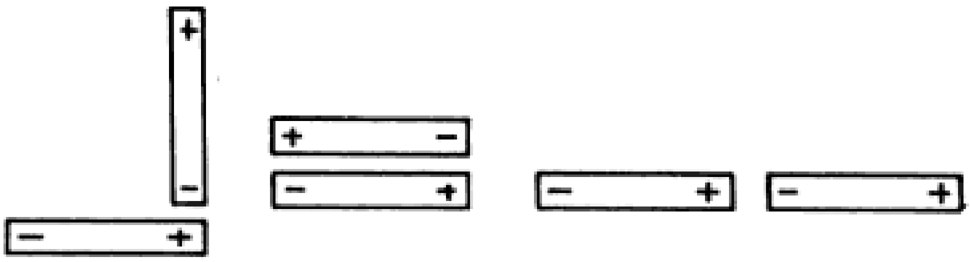


Fig. 4.1 Water molecules as oriented dipoles

The hydrogen atom is capable of forming but one primary valence bond. When this is formed between hydrogen and oxygen, the hydrogen atom with a deficiency of electrons, appears to have a positive charge. The positively charged hydrogen is capable of attracting an oxygen atom of another molecule, thus forming a *hydrogen bond*. Hydrogen bond is weaker than the primary valence bond, but much stronger than the van der Waals forces and is not easily broken under the stresses normally applied on a soil system. The linkage between water molecules (Fig. 4.2) is the best example of a hydrogen bond. In the water molecule, the electrons of the hydrogen atoms are shared by the oxygen atom. The positively charged H-atom now sticks out from the H_2O molecule and has an attraction for the negatively charged electrons of a neighbouring water molecule. Each

H atom belongs, therefore, to an O atom, forming a H_2O molecule and is linked to another O atom by the relatively weak hydrogen bond. The relative strengths of the various kinds of bonds are approximately the following:

van der Waals forces	1 to 10
Hydrogen bond	10 to 20
Ionic and covalent bonds	40 to 400

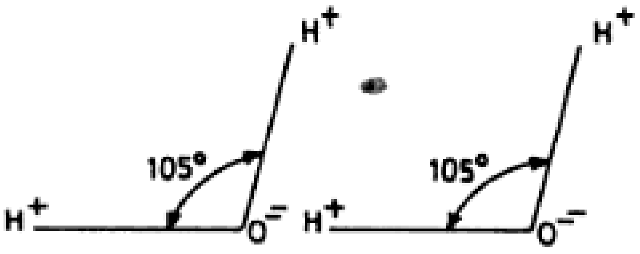


Fig. 4.2 Hydrogen bond

Structure of Clay minerals

As mentioned earlier, a large majority of clays that are present in nature are made up of only a few minerals, which again can conveniently be grouped into three categories, according to their crystalline arrangement. Table 4.1 shows the groups and the major minerals falling in each group. Since the engineering properties of the minerals in a group are roughly the same, an examination of the crystal structure of typical clay minerals in these groups will provide a useful insight into the basic engineering behaviour of the clays themselves.

The flakes or platelets of a clay soil consist of many crystal sheets which have a repeating atomic structure. The atomic structures of clay minerals are built of two fundamental crystal sheets, the *tetrahedral* or *silica sheet* and the *octahedral* or *alumina sheet.* It is only the mode of stacking of these sheets, the nature of bonding forces and the different metallic ions in the crystal lattice that go to make different clay minerals.

Tetrahedral Sheet

The tetrahedral sheet is a result of combining silica tetrahedral units which consist of four oxygen atoms placed at the tips of a tetrahedron enclosing a silicon atom. Figure 4.3(a) shows a single-silica tetrahedron, and Fig. 4.3(b) illustrates how the oxygen atoms at the base of each tetrahedron combine to form a sheet structure with all the tips of the bases lying in a common plane. Note that each of the O ions at the base is common to two adjacent units. The sharing of charges leaves three negative charges at the base per tetrahedral unit. This, along with the two negative charges at the apex, makes a total of five negative charges to four positive charges of the silicon ion. Thus, there is a net charge of −1 per unit. A symbolic representation of the silica sheet is shown in Fig. 4.3(c). A top view of the silica sheet, shown in Fig. 4.3(d), illustrates the typical hexagonal openings in the silica sheet structure.

Octahedral Sheet

The octahedral sheet is a combination of octahedral units. An octahedral unit has six hydroxyl ions at the tips of an octahedron enclosing an aluminium or magnesium or some other metallic atom. Figure 4.4(a) shows a single octahedron. Figure 4.4(b) shows how the octahedral units are bound together in a sheet structure. The rows of hydroxyl ions in the sheet are in two planes with each hydroxyl ion common to three octahedral units. Each hydroxyl ion divides its -1 charge with two other units, leaving a total of 2 negative charges $\frac{1}{3} \times 6$ per unit. Thus, the net charge of an octahedral unit with aluminium ion (+3 charge)

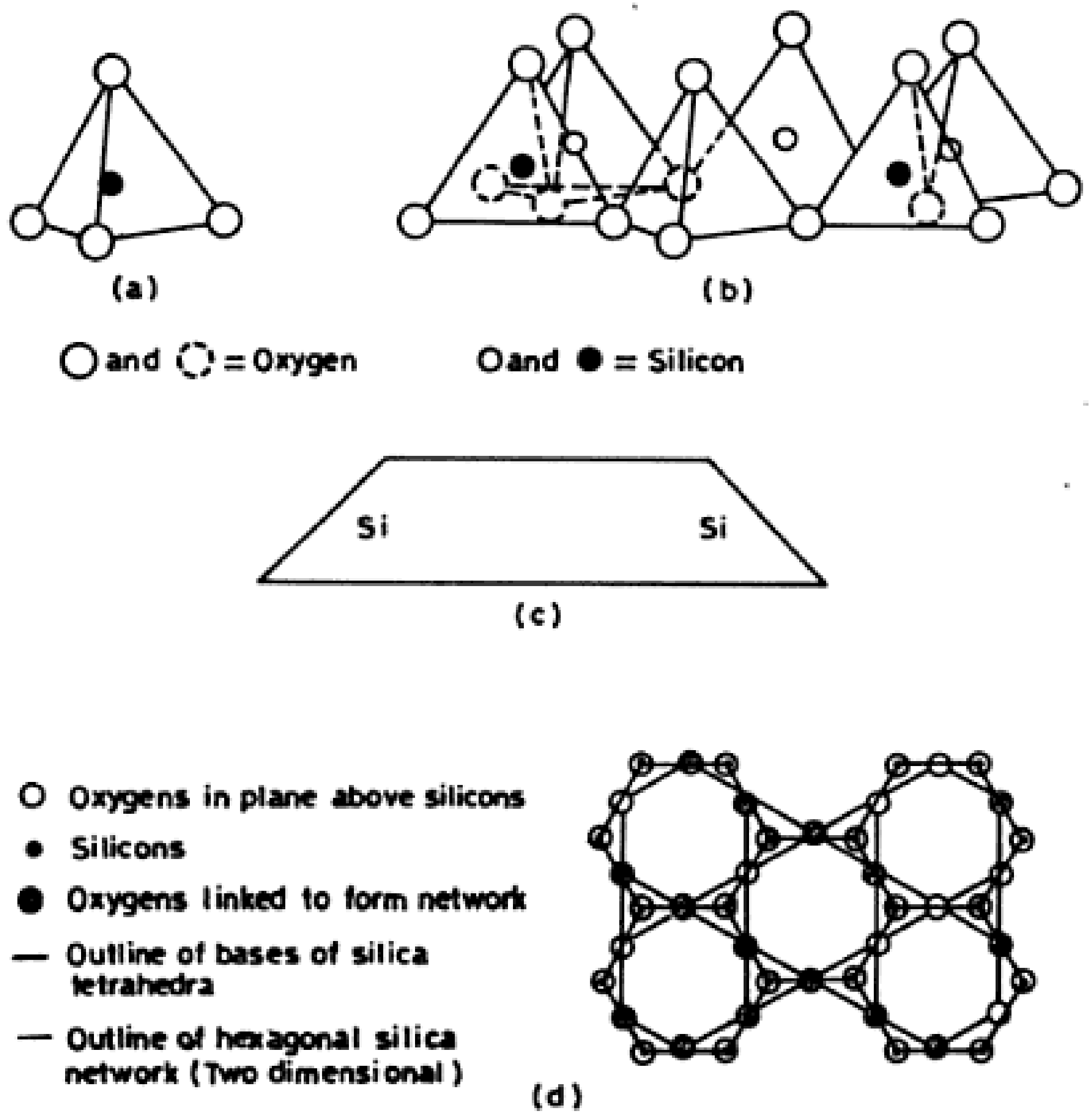


Fig. 4.3 (a) Single silica tetrahedron
(b) Isometric view of the tetrahedral or silica sheet
(c) Symbolic representation of the silica sheet
(d) Top view of the silica sheet (after Warshaw and Roy, 1961)

at the centre is +1. Figure 4.4(c) is a symbolic representation of the octahedral sheet. A top view of the octahedral sheet showing the sharing and bonding of different atoms is shown in Fig. 4.4(d). If the atom at the centre is aluminium, the resulting sheet is called the *gibbsite sheet*, and if magnesium is the central atom, the sheet is called the *brucite sheet.*

Isomorphous Substitution

Frequently, in a clay mineral lattice, metallic ions of one kind may be substituted by other metallic ions of a lower valence, but of the same physical size. Such a substitution is called *isomorphous substitution* and may lead to different clay minerals with different physical properties. For example, one silicon ion in a tetrahedral unit may be substituted by an aluminium ion, which could happen when aluminium ions are more readily available in water. As an aluminium ion has 3 positive charges and a silicon 4 positive charges, there would be a net unit charge deficiency of positive charge per substitution. This would mean an increase in the net negative residual charge on the particle.

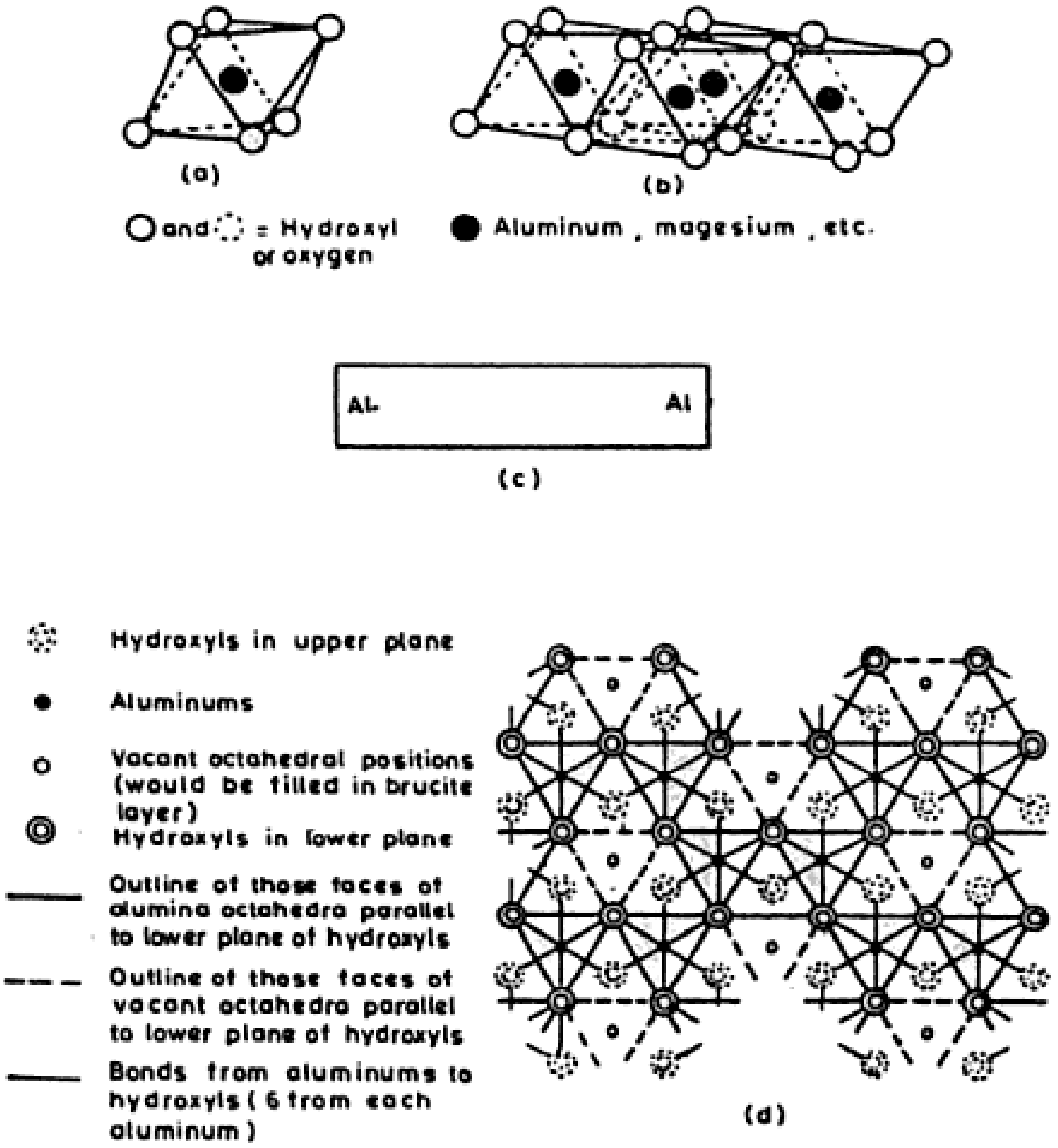


Fig. 4.4 (a) Single aluminum (or magnesium) octahedron
(b) Isometric view of the octahedral sheet
(c) Symbolic representation of the octahedral or alumina (or magenesia) sheet
(d) Top view of the octahedral sheet (after Warshaw and Roy, 1961)

Different clay minerals simply consist of the two basic sheets-the silica sheet and the octahedral sheet-stacked together in certain unique fashions with certain metallic ions present in these sheets. The structures of a few important clay minerals are described below.

Kaolinite—The kaolinite structural unit consists of alternating layers of silica tetrahedra with the tips embedded in an alumina (gibbsite) octahedral unit, as in Fig. 4.5. This stacking of one layer of each of the two basic sheets produces what is sometimes called a 1:1 clay mineral. The resulting layer is about 7 A° thick (1 A° = 10^{-10}m) and extends indefinitely in the other two dimensions. A kaolinite crystal is a stacking of 70 to 100 or more of these basic 7A° thick layers. The structural units are held together by hydrogen bonds between the hydroxyls of the octahedral sheet and the oxygens of the tetrahedral sheet. The bonding with hydrogen bond results in considerable strength and stability with little tendency in the interlayers

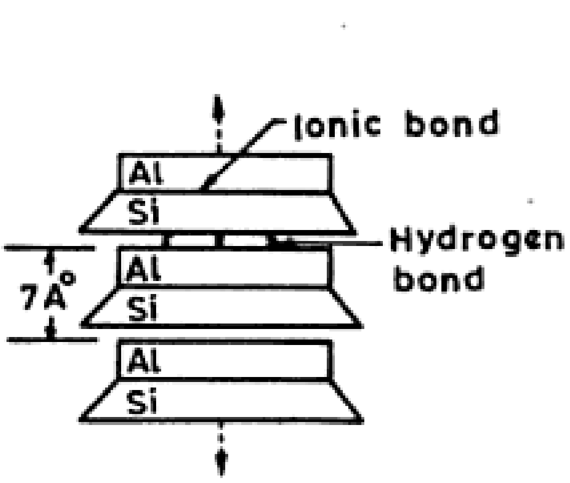


Fig. 4.5 Schematic diagram of kaolinite structure

to allow water and to swell. Kaolinite is, thus, the least 'active' of the clay minerals and has a thickness between 500 A° and 1000 A°.

Another 1:1 mineral of the Kaolin group is *halloysite.* It is more randomly stacked than kaolinite and its 7 A° layers are separated by water molecules having the equation $(OH)_8Al_4Si_4O_{10}.4H_2O$. In contrast to most other clays which are flaky, halloysite particles are tubular and rod-like. Dehydration by heat at a temperature of 60 - 70°C and even air drying will often permanently alter halloysite, reducing it to 2 H_2O or completely removing water to produce approximately kaolinite. The engineering properties of kaolinite and halloysite are quite different; so care has to be exercised in carrying out laboratory tests such as classification and compaction tests. If these tests are made on air-dried samples instead of on samples at their natural water contents, the results can be misleading. Kaolinite and halloysite clays are used for making chinaware, since they do not discolour as joining is free from iron. Kaolin clay is also widely used as an intestinal absorbent in antidiarrhoeal medicines.

Montmorillonite—It is also called *Smectite*. The structural unit of the mineral is composed of two silica sheets and one alumina (gibbsite) sheet (Fig. 4.6), a 2:1 mineral. The octahedral sheet is sandwiched between two silica sheets with the tips of the tetrahedra combining with the hydroxyls of the octahedral sheet to form a single layer. The thickness of this layer is about 9.6 A° and the dimensions in the other two directions are indefinite. The interlayer bonding between the tops of silica sheets is mainly due to van der Waals forces and is, thus, very weak compared to hydrogen or other ion bonding. Partial isomorphic substitution such as Al^{3+} for Si^{4+} in the tetrahedral sheet and Mg^{2+}, Fe^{2+}, Li^{+} or Zn^{+} for Al^{+3} in the octahedral sheet also takes place, resulting in a relatively large net negative charge deficiency. Further, montmorillonite has the largest specific surface among major clay minerals (Fig. 4.8). All these factors mean that a large amount of water and other exchangeable ions can easily enter between the layers causing the layers to be separated. Because of this affinity for water, clay soils containing montmorillonite mineral are susceptible to substantial volume changes. They swell as the water gains entry into the lattice structure and shrink if the water is removed because of some reason. In a moist state, montmorillonite is highly plastic and has little internal friction. Its excessive swelling capacity may seriously endanger the stability of overlying structures and road pavements.

Bentonite is a montmorillonite clay derived from volcanic ash. It is used in drilling oil wells and in soil exploration as a 'drilling mud' and as a clay grout. In India, clay soils containing montmorillonite mineral are commonly known as 'black cotton soils' and cover about 20 per cent of the country's area in the states of Maharashtra, Karnataka, Andhra Pradesh, Madhya Pradesh and Uttar Pradesh.

Illite— The Illite mineral is also a 2 : 1 mineral similar to montmorillonite. Illite too has a substantial amount of isomorphous substitution of silicon ions by aluminium in the silica sheet. But in illite, potassium ions occupy positions between the adjacent O base planes (Fig. 4.7). Hexagonal holes in the silica sheet [Fig. 4.3(d)] have just the right diameter for the potassium ions to fill the gap, and thus the potassium ion bonds the two layers together more firmly than is the case in montmorillonite, with the result that the lattice is much less susceptible to cleavage. Illite, therefore, does not swell as much in the presence of water as montmorillonite, but it does much more than kaolinite.

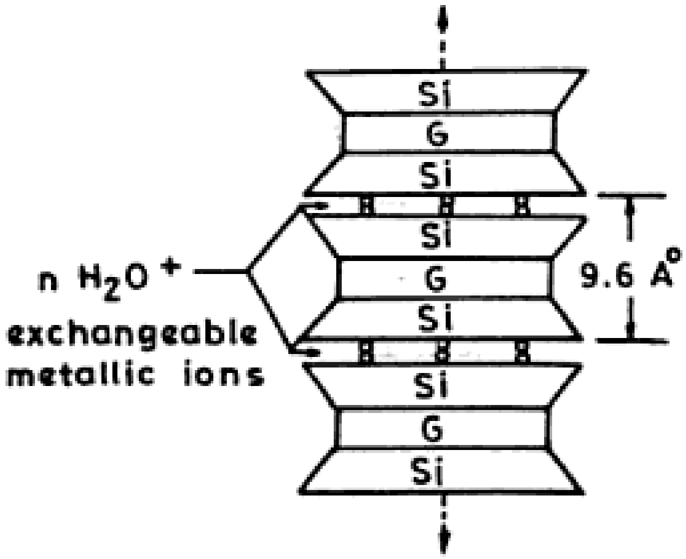


Fig. 4.6 Schematic diagram of montmorillonite structure

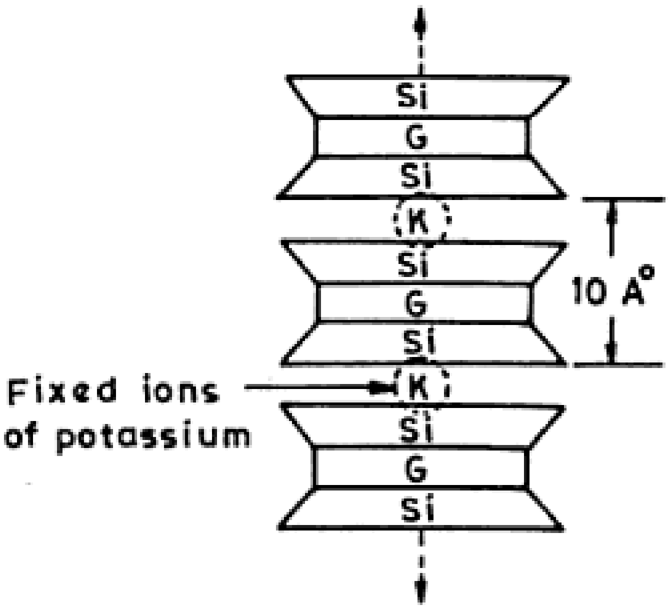


Fig. 4.7 Schematic diagram of illite structure

4.3 CLAY-WATER RELATIONS

Clay soils are normally associated with water and their properties are greatly influenced by the presence of water. This is in marked contrast to the granular soils which, generally, are not sensitive to the amount of water present. A high value of specific surface in a clay soil means that a large proportion of the total number of atoms making up a particle form the surface of the particle. The particles in a clay soil consist of thin platelets whose diameter may, in some cases, be several hundreds of times the plate thickness. Figure 4.8 shows the relative sizes of some clay minerals and their specific surface values. It can be seen from Fig. 4.8 that montmorillonite, the smallest clay mineral, has an average thickness of about 3 nm, while kaolinite, the largest, has an average thickness of, may be, 1000 nm. The average diameter of the crystal of any clay mineral is, however, of the same order; that is how the specific surfaces are so unequal and why montmorillonite is more active than any other clay mineral. The activity of a clay was earlier defined in Eq. 2. 54. A reasonable correlation exists between the type of mineral and the activity value (Table 4.2)

Adsorbed Water—The layers of water which surround the clay crystal are called 'adsorbed water'. Consideration as to how and why water is adsorbed on the surface of clay particle is useful in understanding the behaviour of clay soils. *First*, water molecule being a dipole, is electrostatically attracted to the negatively charged surface of the crystal. *Secondly*, water is adsorbed by hydrogen bonding; that is, hydrogen atom of the

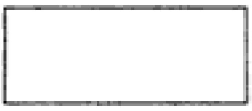
Edge view	*Typical thickness (nm)*	*Typical diameter (nm)*	*Specific surface* (km^2/kg)
Montmorillonite 	3	100-1000	0.8
Illite 	30	10000	0.08
Chlorite 	30	10000	0.08
Kaolinite 	50-2000	300-400	0.015

Fig. 4.8 Average values of sizes and specific surfaces of common clay minerals (after Yong and Warkentin, 1975)

Table 4.2 Activity Values of Minerals

Mineral	*Activity Value*
Na-montmorillonite	4 - 7
Ca-montmorillonite	1.5
Illite	0.5 - 1.3
Kaolinite	0.3 - 0.5
Halloysite (hydrated)	0.1
Calcite	0.2
Quartz	0

(After Skempton, 1953)

water molecule is attracted to the oxygens or hydroxyls on the surface of the clay crystal. The third reason is that the cations present in water get attracted to the negatively charged surface of the clay particles and water dipoles are, in turn, attracted to the cations. It seems natural that the attraction of water to the particle surface is very strong near the surface and tapers off with distance. Immediately next to the particle surface, there is a very strongly adsorbed thin layer of water about 10 A° thick. Beyond this, there is an outer layer which is attracted to a lesser degree and is more mobile. This layer which extends upto the limit of attraction is known as the *diffuse-double layer*. The water which is outside the diffuse double layer is nonoriented "normal" water.

The oriented water is considered to affect the behaviour of clay particles when subjected to stresses, since the adsorbed film of water comes between the particle surfaces. The three modes of attraction are illustrated in Fig. 4.9.

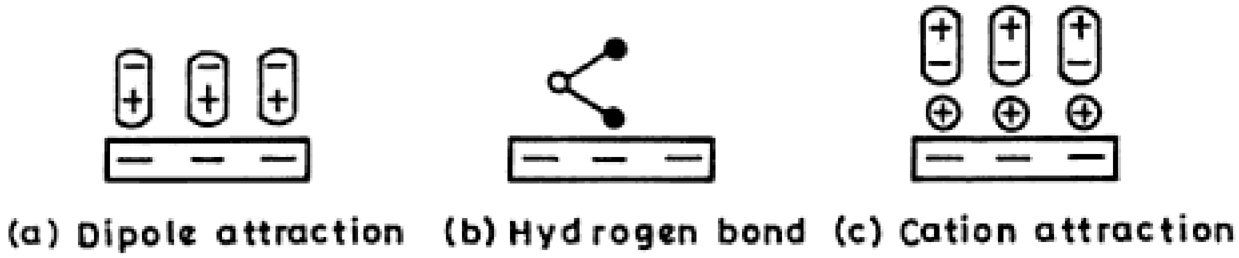


Fig. 4.9 Attraction of water molecules to particle surface

Base (Cation) Exchange Capacity—The surface atoms are predominantly negative oxygen or hydroxyl ions, whereas the metallic (Al, Si) ions occupy positions within the gibbsite or silica sheets. Thus, although the particle as a whole may be electrically neutral, its surface can be considered to be strongly negatively charged. Also, with the exception of Kaolin, isomorphous substitution, mentioned earlier, is one of the sources of negative charge at the surface of the clay crystal. Positive ions or cations in the water are, therefore, attracted to these surfaces of the particles to render the crystal electrically neutral. Different clays have different charge deficiencies and have varying ability to adsorb exchangeable cations. The ability of a clay particle to adsorb ions on its surface or edges is called its *base* or *cation exchange capacity* which is a function of the mineral structure of the clay and the size of the particles. The cations are termed exchangeable because one cation can easily be exchanged with one of the same valence or by two of the cations with one-half the valence of the original cation. The base exchange capacity is expressed in terms of the weight of a cation which may be held on the surface of 100 g of dry soil material. Conventionally, it is measured in milliequivalent (meq) per 100 of dry soil where 1 meq is 1 mg of hydrogen or the portion of any ion which will combine with or displace 1 mg of hydrogen. On the basis of earlier discussions, it can be seen that montmorillonite has a much greater base exchange capacity than kaolinite, with illite being intermediate in position, as shown in Table 4.3 below :

Table 4.3 Base Exchange Capacity

Mineral	*Base exchange capacity (meq/100 g)*
Kaolinite	3–8
Illite	40
Montmorillonite	80

Depositional environment, weathering action, leaching, etc., will govern the kind of ions present in a clay deposit. Ca and Mg are the major exchangeable cations in soils. Marine clays have predominantly Na and Mg cations. The valence of the cation is the basic factor in the process of replacement or exchange. Higher valence cations can easily replace lower valence cations. When the ions have the same valence, larger the size of the ion, greater its replacement capacity. Potassium ion, even though monovalent, has more replacement power than sodium (which is also monovalent) because of its ability to fit into the hexagonal holes in the silica sheet. Cations can be placed in approximate order of their replacement ability as follows :

$$Li^+ < Na^+ < H^+ < K^+ < NH_4^+ < Mg^{++} < Ca^{++} < Al^{+++}$$

The principle of cation exchange can be used with advantage in many practical situations, as for example, in the stabilisation of sodium clay soil by using lime. Here, calcium ions replace the sodium ions by virtue of their superior replacing power and reduces the swelling of sodium montmorillonite, because the adsorbed layer of water would be thinner when the valence of the cation is larger.

4.4 CLAY PARTICLE INTERACTION

The clay particles interact through the adsorbed water layers. Therefore, factors such as the nature of ions present, their concentration and size and other environmental conditions influence the host of soil structures found in natural clay deposits. The soil structure is the end product of the interplay of the forces of attraction and repulsion between the particles of a soil mass.

The inter-particle repulsive force is mainly due to the similar charge on the particle surfaces, and will come into play when adjacent particles come close enough for the adsorbed water layers to interact. Forces of attraction between particles may be due to the van der Waals forces, hydrogen bonding and other kinds of linkages. van der Waals bonds are universal attractive forces and may be the only attractive forces effective in many cases. The closer the particles, stronger is the van der Waals force. For a given type of clay in suspension, the net force between the adjacent particles at a given separation (distance) is the algebraic sum of the repulsive and attractive forces acting at that distance. The interparticle force or potential fields decrease with increasing distance from the mineral surface, as shown in Fig. 4.10. The shape of the curve depends, among other factors, on the valence and concentration of the ions and on the nature of the bonding forces.

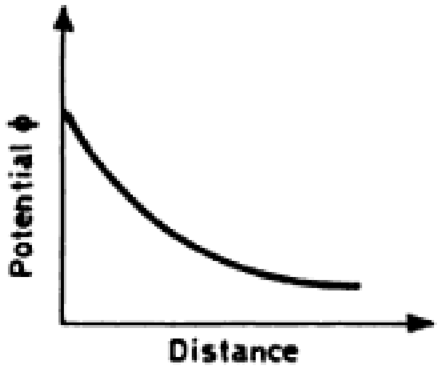


Fig. 4.10 Potential versus distance from mineral surface

Clay particles in aqueous environment may be mutually attracted or repulsed. If the total potential energy between the particles decreases as they approach each other, there is attraction between them in which case the particles approach each other and 'flocculate'. If the approaching particles increase the energy of the system, they move apart or 'disperse'.

Particles flocculate essentially in the edge-to-face configuration, although edge-to-edge and face-to-face flocculations are also possible. Dispersion of particles will be mostly in the face-to-face configuration. The tendency to flocculate will increase on increase of one or more of the following (Lambe, 1958):

(i) concentration of the electrolyte
(ii) valence of the ion
(iii) temperature

or on decrease of one or more of the following :

(i) dielectric constant of the medium
(ii) size of the ion
(iii) *p*H
(iv) anion adsorption

Essentially, all natural clay soil deposits are flocculated to a greater or smaller degree. The structures of sedimented clays arising from dispersed and flocculated suspensions are shown in Fig. 4.11. Owing to the positive charges at the edges of the platelets, and negative charges at the surface of the platelets, the bond between many of the platelets in a floc is of edge-to-face kind and a low degree of parallelism exists, Fig. 4.11(b). The dispersed clay [Fig. 4.11(a), with a basic face-to-face arrangement of platelets has a lower void ratio than the flocculated clay, which has an appreciably larger proportion of voids. A 'cardhouse' structure of flocculated clay in three dimensions is shown in Fig. 4.11(c). The structure of the clay soil formed by sedimentation depends to a large extent on the electrolytic properties of the water. Marine clays, deposited in sea water containing a large quantity of dissolved salt, may be expected to have flocculated structures similar to that of Fig. 4.11(b) and (c). Lacustrine clays, which have been deposited in fresh water lakes, may have a dispersed structure, as shown in Fig. 4.11(a).

4.5 SOIL STRUCTURE AND FABRIC

While the term soil structure is used to encompass both the geometric arrangement of particles and the inter-particle forces which may act upon them, *soil fabric* is a term which has been recently introduced to describe only the geometric arrangement of the mineral particles. In granular soils where inter-particle forces are small, both fabric and structure may essentially mean the same thing. In a fine-grained soil, a detailed study of the soil fabrics may sometimes be the only feasible method of studying the soil structure. A description of the cohesive soil fabrics is not given in the discussion here. However, it will be useful to understand the significance of *macrostructure* of a clay deposit.

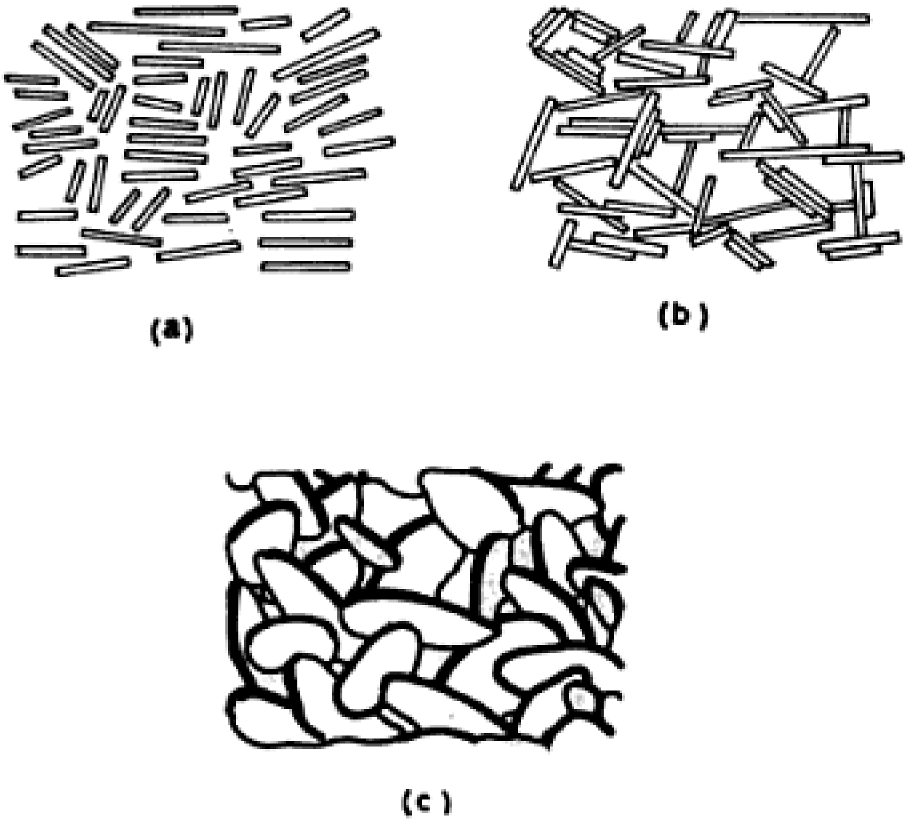


Fig. 4.11 Structure of sedimented clays : (a) Dispersed clay (in two dimensions), (b) Flocculated clay (in two dimensions), (c) Card house structure of flocculated clay (in three dimensions)

Macrostructure of a clay deposit has a very decisive influence on its engineering behaviour. Joints, fissures, rootholes, varves, silt and sand seams and lenses and other discontinuities very often control the behaviour of the entire deposit. The strength of a soil mass is appreciably less along a fissure than in a thoroughly intact material. Similarly, the drainage of a clay layer can be greatly affected by the presence of even a very thin layer of silt or sand. Therefore, in any engineering problem involving stability or settlements, the geotechnical engineer must carefully consider the clay macrostructure.

Microstructure is more important in the fundamental sense than in the engineering sense, although it aids in the general understanding of a soil behaviour. The microstructure reflects the imprint of the geologic as well as stress history of the deposit, the depositional environment, its weathering history and so on.

Thus, one can say that the engineering behaviour of clay deposits is influenced both by the macro and the microstructure and it is necessary for an engineer to properly evaluate the role of these in influencing the engineering behaviour.

4.6 GRANULAR SOIL FABRIC

In granular soils composed of bulky grains, the forces causing deposition are predominantly gravitational and surface bonding forces are not of much consequence. Under the influence of gravitational forces, the grains will assume a particle to particle contact referred to as *single-grained* structure. Although with this structure, the volume of voids seldom exceeds the volume of solids, large variation in void ratio is possible depending on the grain size distribution and the arrangement of grains. Single-grained structure may be *loose* or *dense* (Fig. 4.12).


(a) Loose (b) Dense

Fig. 4.12 Single grained soil structures

The void ratio corresponding to the loosest possible condition of a granular soil is called the *maximum void ratio*, $e_{\max}$ and is determined in the laboratory by pouring dry sand slowly and avoiding vibration inside a calibrated mould of known volume and finding the weight of the sand in the mould. The *minimum void ratio*, $e_{\min}$ at the densest possible state of the soil, is determined by vibrating a known weight of dry soil into a known volume.

The relative density D_r of a soil deposit is defined as

$$D_r = \frac{e_{\max} - e_{nat}}{e_{\max} - e_{\min}} \times 100\% \tag{2.52}$$

where e_{nat} is the void ratio of the soil in its natural state.

The relative density is an important parameter that has been invariably used to assess the engineering behaviour of coarse-grained soil deposits (Table 2.8) although it is by no means sufficient by itself to do it.

EXAMPLES

Example 3.1 Calculate the specific surface of a cube (a) 10 mm (b) 1 mm (c) 0.1 mm and (d) 0.001 mm on a side. How much more water is needed to wet the surface in case (d) compared to case (a) for an equivalent volume of the cube

Solution:

$$\text{Specific surface} = \frac{\text{Surface area}}{\text{Volume}}$$

(a) Side length : 10 mm

$$\text{Specific surface} = \frac{6 \times 10^2\ \text{mm}^2}{10^3\ \text{mm}^3} = 0.6/\text{mm}$$

(b) Side length: 1 mm

$$\text{Specific surface} = \frac{6 \times 1^2}{1^3} = 6/\text{mm}$$

(c) Side length: 0.1 mm

$$\text{Specific surface} = \frac{6 \times 0.1^2}{0.1^3} = 60/\text{mm}$$

(d) Side length : 0.001 mm

$$\text{Specific surface} = \frac{6 \times 1 \times 10^{-6}}{1 \times 10^{-9}} = 6000/\text{mm}$$

This illustrates that smaller the size, larger the specific area. It can be seen that it would take 10,000 times as much water to wet the surface when the cubes are 0.001 mm on a side than when the same volume was made up of cubes 0.001 mm on a side.

Example 3.2 The data presented below are related to three clay minerals– kaolinite, illite and montmorillonite. Place the proper name beside the properties given below:

(a) Largest grain size
Smallest grain size
Intermediate grain size

(b) Largest swelling and shrinkage behaviour
Intermediate value
Smallest value

(c) Plasticity Index = 30
Plasticity Index = 300
Plasticity Index = 15

(d) Highest dry strength
Lowest dry strength
Intermediate dry strength

Solution:

(a) Kaolinite, Montimorillonite, Illite

(b) Montmorillonite, Illite, Kaolinite

(c) Illite, Montmorillonite, Kaolinite

(d) Montmorillonite, Kaolinite, Illite

Example 3.3 Determine the base exchange capacity of a clay if 100 g of dry clay soil adsorbs 80 mg of calcium. Find the quantity of H that can be adsorbed by 5 cum of this clay if the dry unit weight of clay is 1500 kg/m^3.

Solution:

Molecular weight of H = 1
Ca^{++} has two positive charges while H^+ has one positive charge
$\therefore$ 1 meq = 1 mg of H^+ or $40 \div 2$, *i.e.*, 20 mg of Ca^{++}.

Since 100 g of clay adsorbs 80 mg of Ca, the base exchange capacity of clay $= \dfrac{80}{20} = 4$ meq/100g of clay

5 cum of clay weighs $1500 \times 5 = 7500$ kg
Since 100 g of clay can adsorb 4 mg of H,

7500 kg of clay can adsorb $\dfrac{4}{100} \times \dfrac{7500 \times 10^3}{10^6} = 0.3$ kg of H

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5

Soil Compaction

5.1 INTRODUCTION

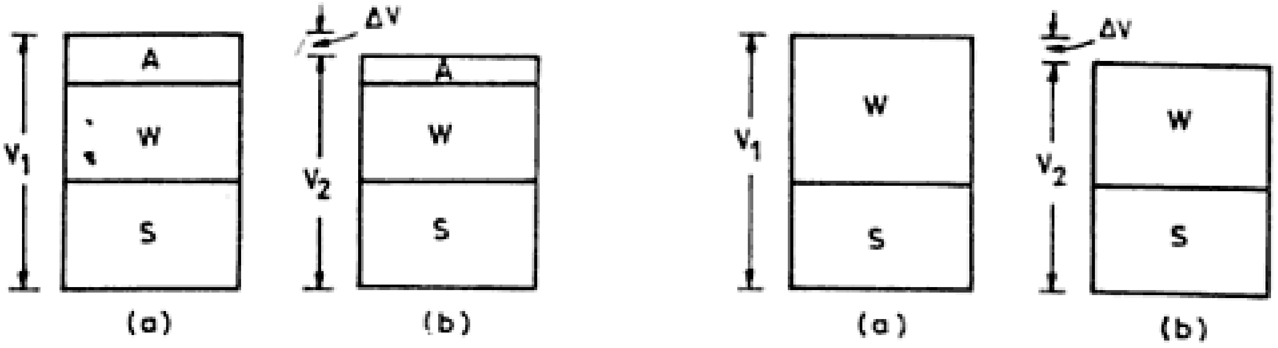
There are many situations in engineering practice when the soil itself is used as a construction material. In the construction of engineering structures such as highway embankments or earth dams, for example, loose fills are required to be compacted to increase the soil density and improve their strength characteristics. Sometimes, an existing soil deposit may need to be 'improved' in order to enhance its engineering performance. Compaction is the most common and important method of soil improvement. The densification of soil by the application of mechanical energy is known as compaction. It is a process by which the soil grains get rearranged more closely, the volume of air voids get reduced, and the density of soil increased.

Compaction should not be confused with 'consolidation', another soil phenomenon in which a volume reduction takes place. Chapter 9 describes consolidation in detail. The points of difference between compaction and consolidation are given in Table 5.1.

Table 5.1 Difference between Compaction and Consolidation

Compaction	*Consolidation*
1. It is almost an instantaneous phenomenon.	1. It is a time-dependent phenomenon.
2. Soil is always unsaturated.	2. Soil is completely saturated.
3. Densification is due to a reduction in the volume of air voids at a given water content.	3. Volume reduction is due to expulsion of pore water from voids.
4. Specified compaction techniques are used in this process.	4. Consolidation occurs on account of a load placed on the soil.

Figure 5.1 illustrates the process of volume reduction involved in compaction and consolidation.



(a) Before compaction
(b) After compaction

(a) Before consolidation
(b) After consolidation

Fig. 5.1 Compaction and consolidation

Compaction generally leads to an increase in shear strength and helps improve the stability and the bearing capacity of a soil. It also reduces the compressibility and permeability of the soil. Detrimental settlements can be prevented and undesirable volume changes through swelling and shrinkage can be controlled.

Smooth-wheel rollers, pneumatic-tyred rollers, sheepsfoot rollers and vibratory rollers are used in the field for compaction. Vibratory rollers are used mostly for densification of granular soils. The technique of vibroflotation is extensively used for compacting *in situ* deposits of granular soil upto considerable depths.

The degree of compaction of a soil is measured in terms of dry unit weight, i.e., the amount of soil solids that can be packed in a unit volume of the soil.

5.2 LABORATORY TESTS

R.R. Proctor, while building dams in the USA in the early thirties, developed the principles of compaction in a series of articles in Engineering News Record (Proctor, 1933). As a tribute to Proctor, the standard laboratory compaction test, which he devised, is called the *Standard Proctor Test.* The compaction characteristics and the degree of compaction can be obtained from the laboratory tests. In these tests, a specified amount of compactive effort is applied to a constant volume of soil mass. The compactive energy is reported in kgfm/m^3 or in S.I. unit, in J/m^3. In the laboratory, impact compaction is most commonly used; a hammer (rammer) is dropped several times on a soil sample in a mould. In the field, compactive effort is the number of passes or coverages of a roller of a certain kind and weight on a given volume of soil.

In the Standard Proctor Test, a standard volume (944 cc or 1/30 cft mould—(Fig. 5.2) is filled up with soil in three layers. Each layer is compacted by 25 blows of a standard hammer of weight 2.495 kg (5.5 lb), falling through 304.8 mm (12"). Knowing the wet weight of the compacted soil and its water content, the dry unit weight of the soil can be calculated:

$$\gamma_t = \frac{\text{Weight of the compacted soil}}{\text{Volume of the mould}} \tag{5.1}$$

$$\gamma_d = \frac{\gamma_t}{1+w} \tag{5.2}$$

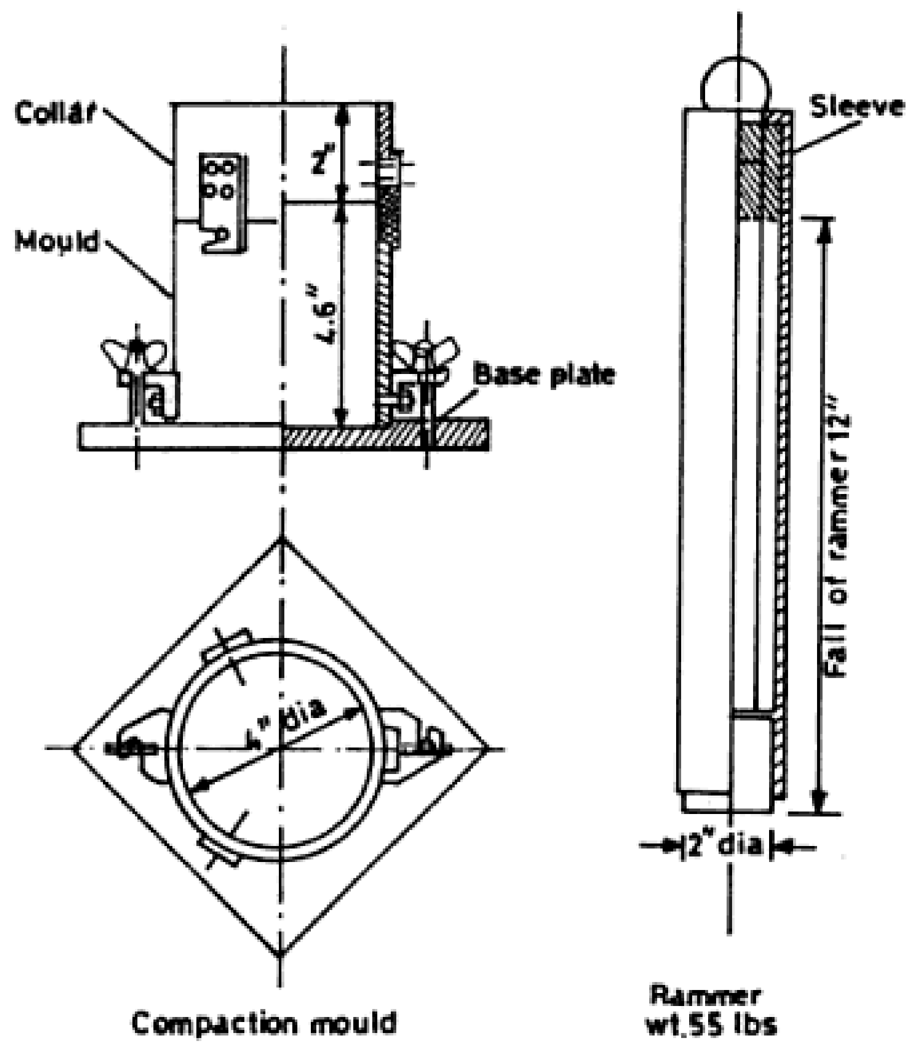


Fig. 5.2 Compaction mould and rammer of Standard Proctor test

The test is repeated at different water contents. The dry unit weight of each compacted sample is plotted against the water content and the curve called compaction curve obtained (Fig. 5.3). Each data point on the curve represents a single compaction test. Usually, four to five points are required to obtain the compaction curve. This curve is unique for a given soil type, method of compaction and compactive effort. The peak point of the compaction curve corresponds to the maximum dry unit weight, $\gamma_{d\,(\max)}$. The water content corresponding to the maximum dry unit weight is known as the *optimum moisture content* (OMC). It may, however, be noted that maximum dry unit weight so obtained is only for a given amount of compactive effort and method of compaction. It is also not necessarily the maximum dry unit weight that can be obtained in the field. The specifications for compaction of fills in the field are usually based on maximum dry unit weight but sometimes on both the maximum dry unit weight and the OMC.

Typical values of maximum dry unit weights range from 16 to 20 kN/m^3 with the widest range being 13 to 24 kN/m^3. Typical optimum moisture content values range from 10 to 20 per cent with a maximum range of 5 to 30 per cent.

The Modified Proctor Test was developed during World War II to better simulate the compaction required for airfields to support heavier aircraft. The test employs a heavier hammer (4.54 kg or 10 lb) with a height of fall of 457.2 mm (18") and 5 layers tamped 25 times into a standard Proctor mould.

The Indian Standard equivalent of the Standard Proctor Test is called the *light compaction* test (IS: 2720, Part VII-1974). The volume of the mould is 1000 cc, the weight of hammer 2.6 kg and the drop 310 mm.

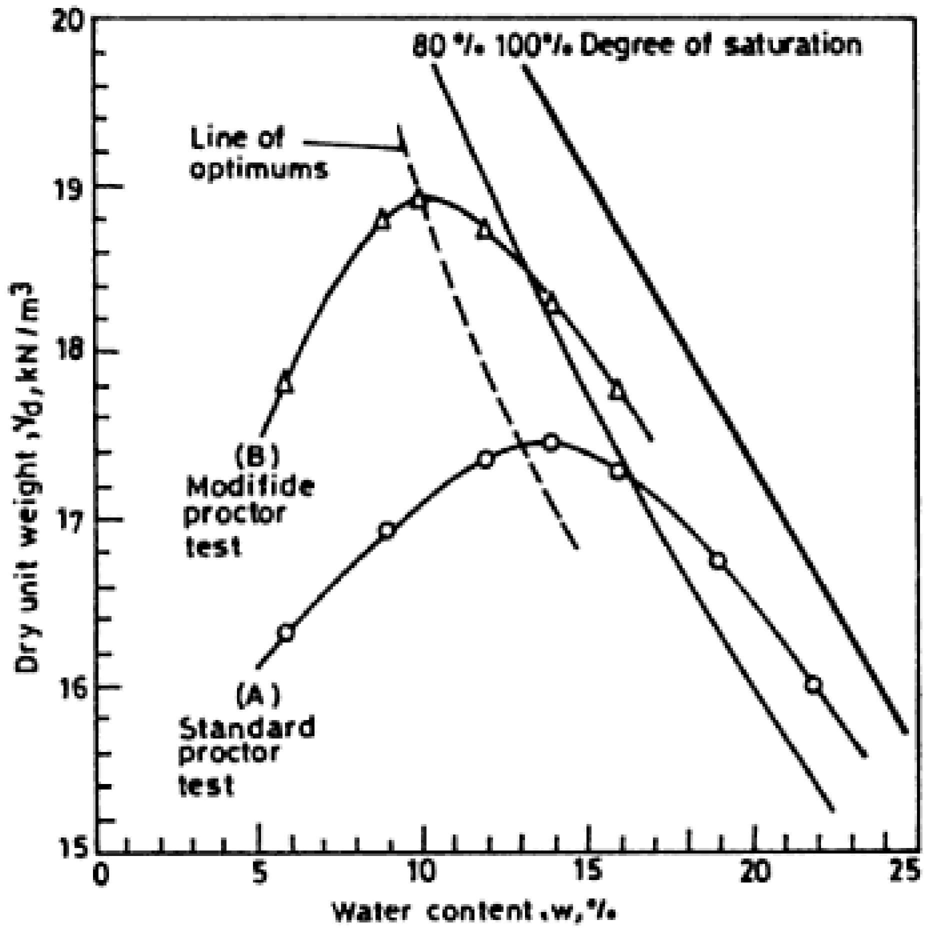


Fig. 5.3 Typical Standard and Modified Proctor test data

The Indian Standard equivalent of the Modified Proctor test is called the *heavy compaction* test (IS: 2720 Part VIII-1983). The weight of hammer is 4.9 kg and the drop 450 mm. The soil is compacted in 5 layers, each layer tamped 25 times.

It can be seen that the Indian Standard compaction tests are not significantly different from the original Proctor tests. In India, the Indian Standard specifications are followed in laboratory testing.

5.3 FACTORS AFFECTING COMPACTION

Compaction is a function of the following factors:

(i) water content *(ii)* compactive effort, *(iii)* type of soil, and *(iv)* method of compaction.

Excepting the type of soil where there is no choice, the other three factors are variables which can be controlled in the field.

Water Content: Proctor explained the typical moisture content dry unit weight relationship with the help of a 'lubrication' theory. At low water contents, the soil is stiff and the soil grains offer more resistance to compaction. As the water content increases, the particles develop larger and larger water films around them, which tend to lubricate the particles and make them easier to be worked around, to move closer into a denser configuration, resulting in a higher dry unit weight and lower air voids. The dry unit weight continues to increase till the optimum moisture content is reached, a stage when the lubrication effect is the maximum. With further increase in moisture content, however, the water starts to replace the soil particles and since $\gamma_w << \gamma_s$, the dry unit weight starts decreasing.

The above explanation of the compaction curve in terms of the lubrication phenomenon, is somewhat simplistic. Lambe (1958) uses the concepts of soil structure and the electrical double layer theory to explain the effect of water content on dry unit weight. In the case of cohesive soils, there is an attractive force, namely, the van der Waals' force which acts between two soil particles and a repulsive force which is due to the double layers of adsorbed water tending to come into contact with each other. While the attractive force essentially remains the same in magnitude, the repulsive force is directly related to the size of the double layers. If the net force between the particles is attractive, flocculated structure is the result; if it is repulsive, the particles tend to move away—'disperse'. At low water contents, the double layer is not fully developed, the repulsive force is small and the net force between particles is a strong attractive force. This makes it difficult for the particles to move about when compactive effort is applied. A low dry unit weight is the consequence. As the water content is increased, the double layer expands and the interparticle repulsive forces increase. The particles easily slide over one another and get packed more closely, resulting in a higher dry unit weight. The double layer expansion is complete at the optimum moisture content and that is why the dry unit weight is the maximum at that stage. Beyond the OMC, the addition of water does not add any further to the expansion of the double layer but the water tends to occupy space which otherwise would have been occupied by soil grains. Hence, a decrease in dry unit weight.

Lambe's theory is more rational. It also explains why the shape of the compaction curve is not the typical inverted V shape in the case of soils which are not cohesive and plastic in nature.

The dry unit weight can also be related to the water content and degree of saturation by Eq. 5.3

$$\gamma_d = \frac{G\,\gamma_w}{1+e} = \frac{G\,\gamma_w}{1+\dfrac{wG}{S}} \tag{5.3}$$

For a given water content, the theoretical maximum value of dry unit weight for a compacted soil is obtained corresponding to the situation when no air voids are left, i.e., when the degree of saturation becomes equal to 100 percent. This is not the same thing as a soil becoming saturated when its water content is increased so as to fill all the air voids. The *Zero Air Void Density ZAVD,* (density is often used in place of unit weight) is obtained for a given soil by substituting $S = 1$ in Eq. 5.3. It is defined as the maximum dry unit weight that one can ideally obtain for a soil at a given water content by applying compaction. It is only an ideal to aim for, a theoretical limit—something never reached in practice, because no matter how much compactive effort is used and in whatever manner, some air voids will always remain within the soil.

If the zero air void density is calculated for different water content values and plotted alongside the compaction curve, a zero air void curve is obtained ($S = 100\%$ curve) (Fig. 5.3). Likewise, the lines for other degrees of saturation such as for $S = 90\%$, $S = 80\%$ etc. can also be drawn, using Eq. 5.3. Sometimes, it is more convenient to draw lines corresponding to different *percentage air voids*, n_a. From Ch. 2, Eq. 1,

$$\gamma_d = \frac{(1-n_a)\,G\,\gamma_w}{1+wG} \tag{5.4}$$

The zero air void line obtained for $S = 100\%$ in Eq. 5.3 and for $n_a = 0\%$ in Eq. 5.4 are identical. However, it can be seen that 10 % air void line and 90% saturation line are not identical.

Compactive Effort: For a given type of compaction, the higher the compactive effort, the higher the maximum dry unit weight and lower the optimum moisture content. In Fig. 5.3, compaction curve B corresponds to the higher compactive effort in a modified Proctor test. Comparing it with the compaction curve A for a standard Proctor test, one can see that the compaction curve shifts to the top and to the left when the compactive effort is increased. However, as the moulding water content increases, the influence of compactive effort on dry unit weight tends to diminish. Again, the maximum dry unit weight does not go on increasing as the compactive effort is increased. The margin of increase becomes smaller and smaller even on the dry side of OMC, while on the wet side of OMC, there is hardly any increase at all. If the peaks of compaction curves for different compactive efforts are joined together, a 'line of optimums' is obtained (Fig. 5.3). The line of optimum is nearly parallel to the zero air void curve. This brings out the fact that even a higher compactive effort does not result in a higher efficiency of compaction. The degree of saturation at OMC remains almost the same in all cases.

Type of Soil: Fig. 5.4 shows typical compaction curves for different types of soils.

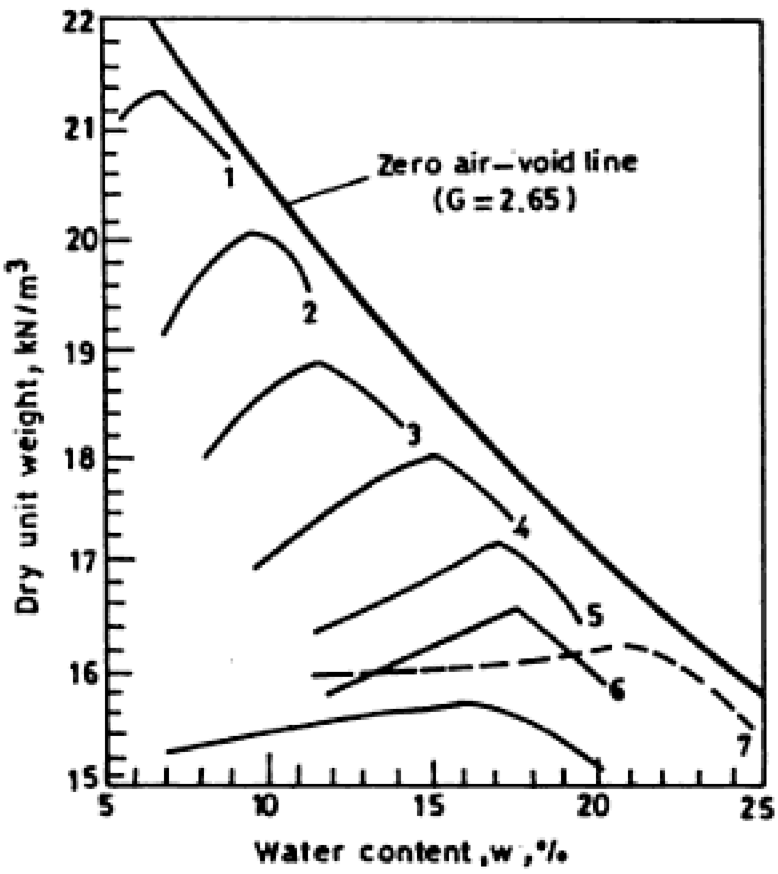


Fig. 5.4 Typical standard compaction test curves for different soils (after Johnson and Sallberg, 1960)

Compaction curve	*Description of soil*
1	Well-graded to loamy sand (*SW-SM*)
2	Well-graded sandy loam (*SM*)
3	Med-graded sandy loam (*SM*)
4	Lean sandy silty clay (*CL*)
5	Lean silty clay (*CI*)
6	Loessial silt (*ML*)
7	Heavy clay (*CH*)
8	Poorly graded sand (*SP*)

All these soils, barring soil 8, have fines present in them in varying proportions and possess plasticity. Some observations follow:

1. Coarse-grained soils, well graded, compact to high dry unit weights, especially if they contain some fines (top curve). However, if the quantity of fines is excessive, maximum dry unit weight decreases.
2. Poorly graded or uniform sands lead to the lowest dry unit weight values (bottom curve).
3. In clay soils, the maximum dry unit weight tends to decrease as plasticity increases (4 to 7).
4. Cohesive soils have generally high values of OMC.
5. Heavy clays with high plasticity have very low maximum dry unit weight and very high OMC.

Compaction curve for sands

The typical inverted *V* shape of a compaction curve applies only to soils possessing some amount of plasticity. In the case of pure sandy soils, the trend is somewhat different.

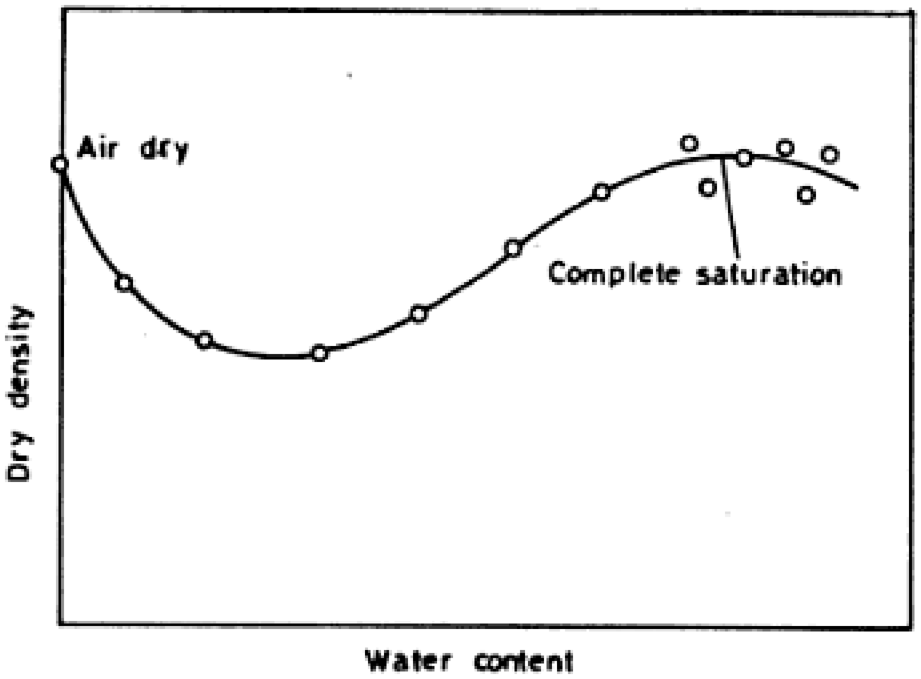


Fig. 5.5 Typical compaction curve for cohesionless sands.

Figure 5.5 shows the variation of dry unit weight of sand with moisture content. Initially, there is, in fact, a decrease in dry unit weight with increase in water content. This is due to the capillary tension in pore water which prevents soil particles coming closer. The phenomenon is known as *bulking* of sand. The maximum bulking occurs at a water content of about 4 to 5 per cent. With increase in water content, the dry unit begins to increase as the menisci are broken and the particles are able to move and adopt a closer packing. The maximum dry unit weight results when the soil is fully saturated. When the water content is increased further, there is a fall in dry unit weight again.

Coarse-grained soils do not adsorb water the way fine-grained soils do and hence the Lambe's double layer theory is not applicable to them. In fact, the compaction curve is of little practical importance to sands or gravels. Maximum dry unit weight results when the soil is either dry or completely saturated. Therefore, sands are usually compacted either in a dry state or in a saturated state by flooding with water. In any case, the difference in dry unit weight at different states is so small that compaction water content is not a very significant factor in sandy soils.

Method of Compaction: Ideally speaking, the laboratory test must reproduce a given field compaction procedure, because the mode of compaction does influence somewhat the shape and the position of the γ_d versus w plot. Figure 5.6 shows the plots for the same soil, compacted under different conditions. Since the field compaction is essentially a kneading type compaction or rolling type compaction and the laboratory tests use the dynamic-impact type compaction, one must expect some divergence in the OMC and $\gamma_{d\,(max)}$ values in the two cases. However, it does not warrant the devising of special laboratory tests that can simulate the different field compaction procedures.

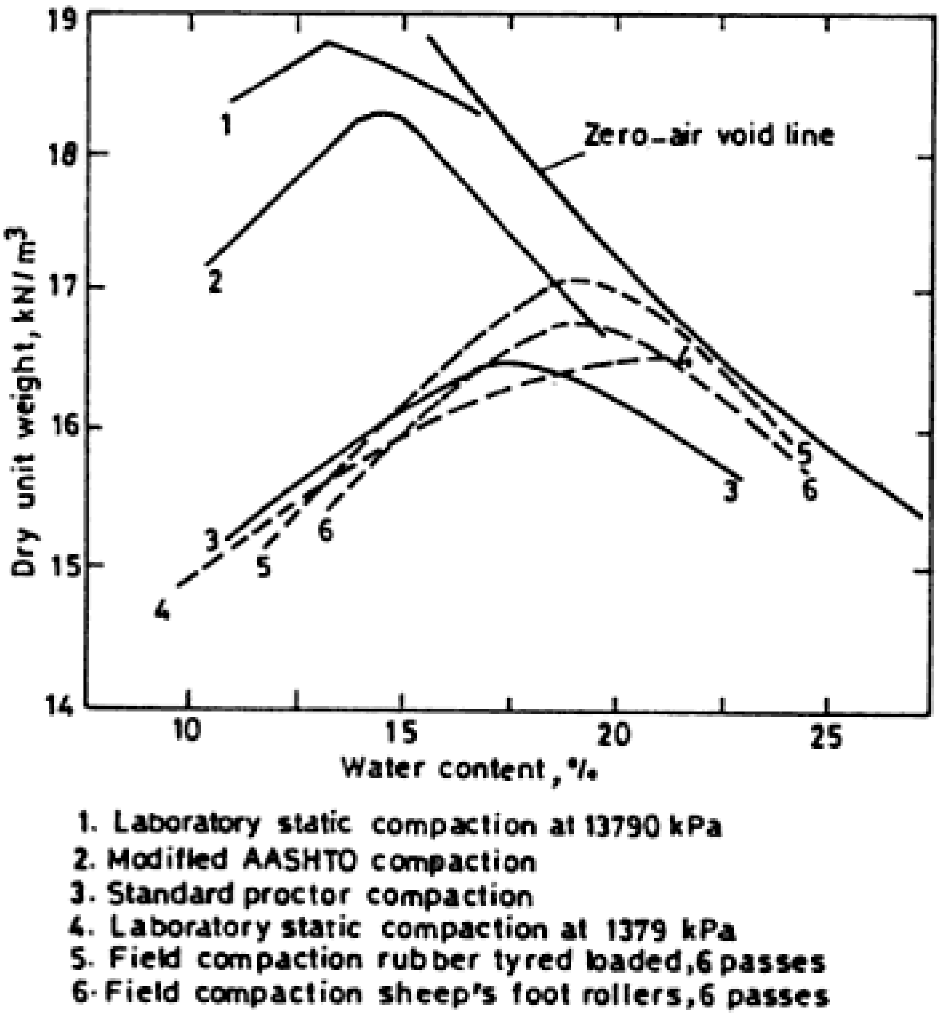


Fig. 5.6 Comparison of field and laboratory compaction (after Turnbull 1950, Lambe and Whitman 1969)

5.4 STRUCTURE AND ENGINEERING BEHAVIOUR OF COMPACTED COHESIVE SOILS

That an increase in density would produce a soil which is more strong, less compressible and less permeable is intuitively visualised. Where the compaction does not lead to significantly different structures in a soil, this logic is true to a large extent. But when structural changes become important, the logic fails. In such soils, therefore, the effect of compaction on structure and consequently on the engineering behaviour has to be properly understood. Usually, the water content of a compacted soil is referenced to the OMC. Thus, soils are said to be compacted *dry of optimum* or *wet of optimum.*

Structure

Figure 5.7 illustrates the effect of compaction on soil structure. For a given compactive effort, the soil tends to have a flocculated structure on the dry of optimum, whereas wet of optimum, the structure tends to be more dispersed. The structure of soil at point *A* is more flocculated than at point *C* (γ_d is the same in both cases). For a given moulding water content, if the compactive effort is increased, the soil tends to become more oriented (dispersed), especially on the dry side of optimum. In Fig. 5.7, the structure at *E* is more oriented than at point *A*. Wet of optimum, the fabric at point *D* is more oriented than at *C*, but the effect is much less significant than at dry of optimum. Among the different methods of compaction, the kneading compaction (such as the one induced by sheepsfoot rollers) produces a more oriented structure as compared to either static or impact type compaction. Thus, one can summarise the influence of compaction on soil structure as below:

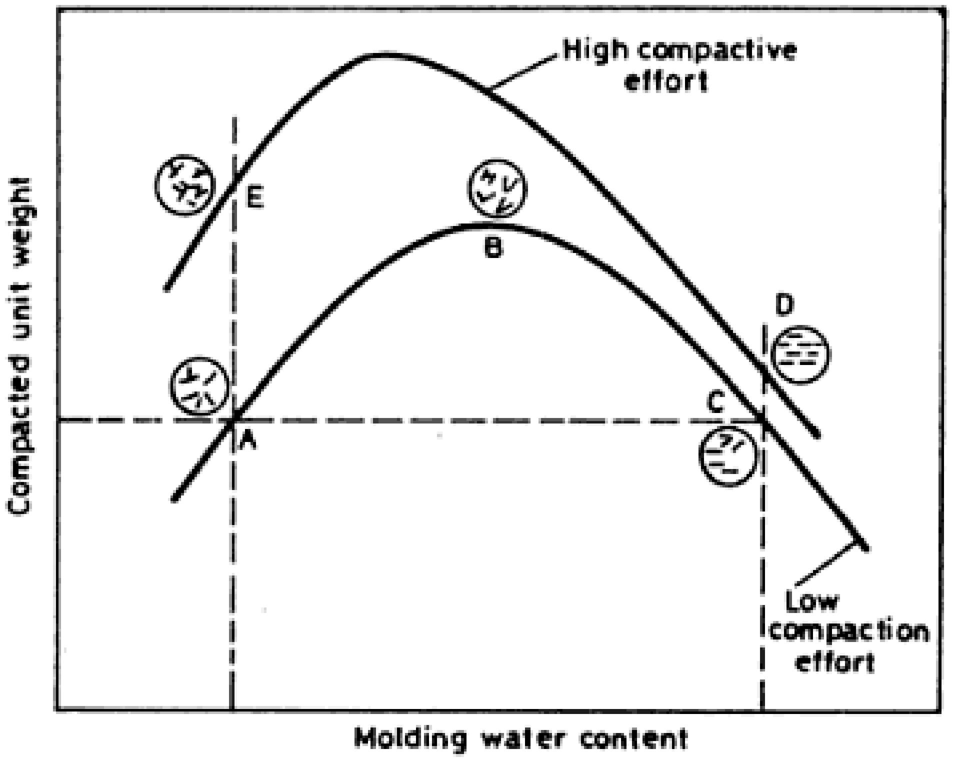


Fig. 5.7 Effect of compaction on structure (after Lambe, 1958)

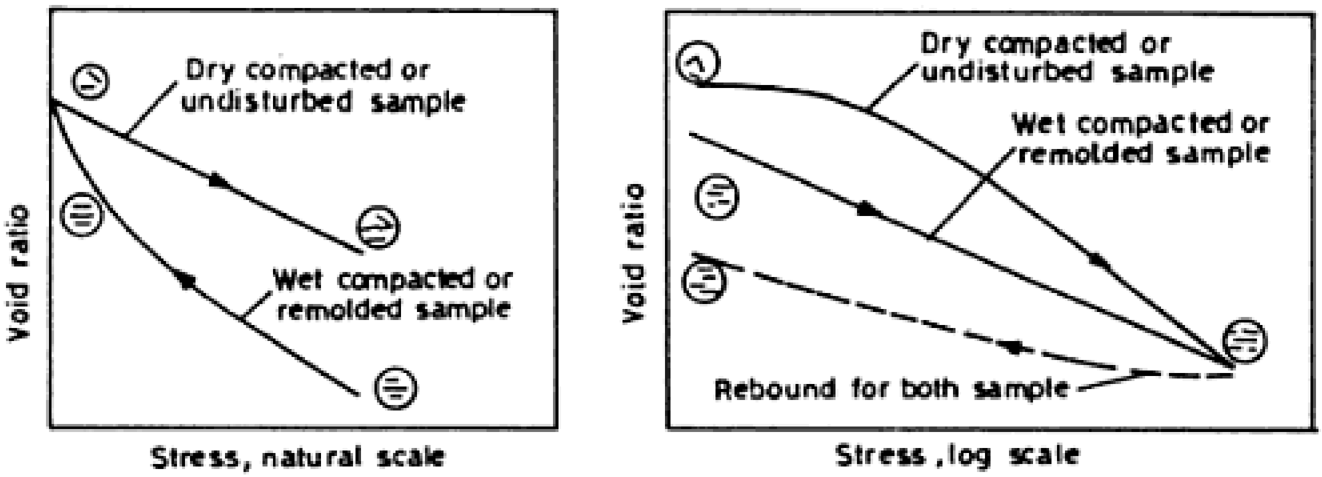
Soil fabric becomes more dispersed as the compaction water content is increased, as the compactive energy is increased and as one uses a method of compaction that induces more shearing strains in the soil such as the kneading compaction.

Permeability

For a given compactive effort, the permeability decreases sharply with increase in water content on the dry side of optimum. The minimum permeability occurs at or slightly above the OMC. Beyond OMC, the permeability may show a slight increase, but it always remains much smaller than on the dry side of optimum. The sharp decrease in permeability on the dry side of optimum is on account of the improved orientation of particles which results in a decrease in the size of voids. The slight increase in permeability on the wet side of optimum is due to the effect of a decrease in the dry unit weight which is more pronounced than the effect of improved orientation.

Compressibility

Figure 5.8 shows the effect of compaction on the compressibility of a cohesive soil. At relatively low stress levels, a soil compacted wet of optimum is more compressible than the flocculated soil on the dry side of optimum, because the strong interparticle bonds in a flocculated structure do not allow the particles to be displaced. At high stress levels, the opposite is true because once the bonds are broken, the flocculated structure with its larger void volume, can undergo a larger volume decrease.



(a) Low-stress consolidation (b) High-stress consolidation

Fig. 5.8 Effect of compaction on compressibility (after Lambe, 1958)

Swelling

A soil on the dry side of optimum (point *A* in Fig. 5.7) has a higher water deficiency and a more random particle arrangement. It can, therefore, imbibe more water than a soil on the wet of optimum (point *C*) and in the process, swells more.

Shrinkage

Soils compacted on the wet of optimum tend to exhibit more shrinkage upon drying than those compacted dry of optimum. The more orderly, nearly parallel orientation of particles (points *C* and *D* in Fig. 5.7) allows the particles to pack more efficiently as compared to the randomly oriented particles on the dry side of optimum.

Stress-Strain Relationship

Soils compacted on the dry of optimum, with their strong interparticle bonds, resist deformation upon application of deviator stress; the stress-strain curve rises steeply to a peak and then falls off as the interparticle bonds are broken (Fig. 5.9). Soils compacted wet of optimum have a flatter stress-strain curve without any peak. At high strains, both kinds of samples will have similar particle arrangement and have about the same strength.

Construction Pore Water Pressure

During the compaction of a man-made fill in layers, the soil below will be subjected to normal and shear stresses. These induce pore water pressures. Soil compacted wet of optimum will have higher pore water pressure compared to soils compacted dry of optimum which have initially a negative pore water pressure.

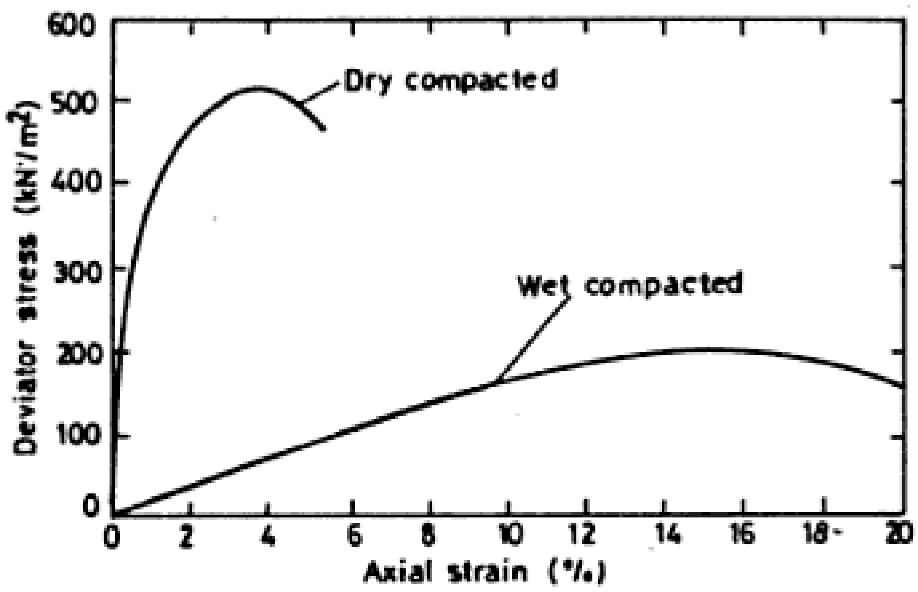


Fig.5.9 Effect of compaction on stress-strain behaviour

Table 5.2 summarises the points made above.

Table 5.2 Comparison of Dry of Optimum with Wet of Optimum Compaction

Property	*Dry of Optimum*	*Wet of optimum*
Structure after compaction	Flocculated (random)	Dispersed (oriented)
Water deficiency	More	Less
Permeability	More, isotropic	Less, anisotropic ($k_H > k_v$)
Compressibility		
at low stress	Low	Higher
at high stress	High	Lower
Swellability	High	Low
Shrinkage	Low	High
Stress-strain behaviour	Brittle; high peak higher elastic modulus	Ductile; no peak, lower elastic modulus
Construction pore water pressures	Low	High
Strength (undrained) as moulded,	High	Much lower
after saturation	somewhat higher if swelling prevented	Low

Form the above discussion, it should be clear that even though compaction at OMC produces maximum dry unit weight, it is not always desirable to do so. In a certain engineering project, limiting the volume changes in a soil might be the overriding consideration; in another, limiting the construction pore pressures might be all important. Increased shear strength might be most desirable in some others.

It is not possible to have a compacted soil having all desirable properties to an ideal extent. But by choosing a compaction water content slightly dry or slightly wet of optimum, it may be possible to secure an improvement in a set of soil properties which are critical for the performance of that particular project. Selecting compaction conditions is indeed a problem of optimisation. Table 5.3 gives some examples of this process.

Table 5.3 Selection of Compaction Water Content

Project	*Compaction water content*	*Reason*
Core of an earth dam	Wet of optimum	To reduce permeability and prevent cracking in core
Homogeneous earth dam	Dry of optimum	To have a stronger soil and to prevent build-up of high pore water pressure
Subgrade of pavement	Wet of optimum	To limit volume changes

5.5 COMPACTION IN THE FIELD

Laboratory compaction tests tests are usually utilised to specify the compacted dry unit weight to be attained in the field. Depending upon the size of the equipment to be used in the field, Indian Standard (light or heavy compaction) test may be used. Since the control in the field connot be as strict as in the laboratory, the specifications usually require attainment of 90 to 95 per cent of dry unit weight attained in the laboratory.

A fill is made by compacting successive horizontal layers. It is necessary to control the moisture content of the soil which is to be compacted. Also, the moisture content should be as near the chosen value as possible. If the soil from the borrow pit is too dry, the deficiency of water in the soil may be made up by sprinkling water and mixing it thoroughly before compacting, whereas if the soil is too wet, it may be excavated in advance and dried.

Different types of soils in the field can be compacted by various methods, e.g., rolling, ramming (by impact) and vibrations. The various types of rollers are: smooth wheel rollers, pneumatic tyred rollers and sheepsfoot rollers. Ramming equipment can be the impact type, internal combustion type or the pneumatic type. Vibrating unit can be mounted on plates or rollers. Vibration may be induced by rotating an unbalanced mass or by a pulsating hydraulic system.

The selection of the equipment and the procedure of compaction depends on the characteristics of the soil to be compacted. The compaction achieved in the field would depend on (i) thickness of lift (layer) (ii) type of roller, (iii) number of passes of the roller, and (iv) intensity of pressure on the soil.

Compaction of Cohesionless Soils

As explained earlier, cohesionless soils do not display a marked optimum moisture content. Application of vibrations is the most effective method of compacting cohesionless soils. Best results are obtained when the frequency of vibrations is close to the natural frequency of the soil. The vibrating equipment can be the dropping weight or the pulsating hydraulic type. In order of effectiveness, watering is the next method that can be used for compacting cohesionless deposits. The seepage force of water percolating through a cohesionless soil makes the soil grains occupy a more stable position. However, large quantities of water are required in this method. Rolling is the least effective method of compacting cohesionless soil deposits.

Existing loose sand deposits, if subjected to vibrations, also get densified and exhibit less settlement. It is always desirable to compact such deposits by vibroflotation before placing structures on them. Pile driving may also be considered as a useful method for compacting such deposits.

Compaction of Moderately Cohesive Soils

When compacting moderately cohesive soils, best results are obtained when the soils are compacted in layers. These soils are compacted by rollers. Depending upon the plasticity of the soil, pneumatic tyred rollers or sheepsfoot rollers can be used.

In the case of silts of low plasticity, pneumatic tyred rollers are preferred. A common form of pneumatic tyred roller consists of a box or platform mounted between two axles. The rear axle has one wheel more than at the front. The wheel mounted on the front axle is arranged to track in between those mounted on the rear axle. The pneumatic or rubber tyred roller has about 80 per cent coverage, i.e., 80 per cent of the total area is covered by tyres. The tyre pressures in small rollers are of the order of 250 kPa whereas in heavier rollers, it may range between 400 and 1050 kPa. The soils are usually compacted in about 200 mm thick layers with 8 to 10 passes of the roller.

Sheepsfoot rollers are more suitable for plastic soils of moderate plasticity. This type of roller has many round or rectangular shaped projections or feet attached to a steel drum. There is approximately one foot for about 50 to 100 sq cm of the surface area of the drum of the roller. The feet project by about 150 to 250 mm from the drum surface. The weight of the drum can be varied by filling it partly or fully with water or sand. As the coverage is about 8 to 12 per cent, very high contact pressures, ranging from 1,400 to 7,000 kPa, depending upon the drum size and whether the drum is partly or fully filled, are possible.

The sheepsfoot roller starts compacting the soil below the bottom of the foot and works its way up the layer as the passes increase in number. Eventually, the roller 'walks out' of the fill as the upper part of the fill is compacted. Sheepsfoot rollers are usually towed in tandem by crawler-tractors or are self-propelled. Sheepsfoot rollers induce shear strains in a plastic soil more than any other type of roller.

Compaction of Clays

When excavated from the borrow pits, clay comes out in the form of chunks. In case these are required for small scale work, it may be possible to get the chunks broken up by manual labour. However, for large scale work, it is not economically feasible to get these lumps broken. Further, once the clay has been excavated, it is difficult to change its water content.

In the case of a clay deposited in chunks, there are no effective means of reducing the void ratio within a chunk. All that can be achieved is a reduction in the space between adjacent chunks. This is satisfactorily achieved by sheepsfoot rollers.

Suitability of Compaction Equipment

Table 5.4 provides guidelines for the suitability of field compaction equipment, depending on the type of soil and the kind of project.

Table 5.4 Suitability of Compaction Equipment

Type of equipment	*Suitability for soil type*	*Nature of project*
Rammers or tampers	All soils	In confined areas such as fills behind retaining walls, basement walls, etc. Trench fills.
Smooth wheeled rollers	Crushed rocks, gravels, sands	Road construction, etc.
Pneumatic tyred rollers	Sands, gravels silts, clayey soils	Base, sub-base and embankment compaction for highways, airfields, etc. Earth dams
Sheepsfoot rollers	Clayey soil	Core of earth dams
Vibratory rollers	Sands	Embankments for oil storage tanks, etc.

5.6 COMPACTION SPECIFICATIONS AND FIELD CONTROL

As the aim of compaction is to stabilise soils and to improve their engineering bebaviour, it is important to keep the engineering properties of the fill in mind rather than just the dry density and water content. This aspect is usually ignored in earthwork construction control. Great emphasis is laid on attaining a particular dry unit weight and very little importance is given to the engineering properties of the compacted fill. Dry unit weight and water content, however, correlate well with the engineering properties and are thus convenient construction control parameters.

Earthwork specifications can be classified into two categories, namely, *end-product specifications* and *method specifications*. Under the first group, a certain *relative compaction* or per cent compaction is specified. *Relative compaction* is defined as the ratio of the field dry unit weight, $\gamma_{d\,(\text{field})}$ to the laboratory maximum dry unit weight, $\gamma_{(d\max)}$ as per specified standard test, namely Indian standard light or heavy compaction test.

$$\text{Relative compaction} = \frac{\gamma_{d\,(\text{field})}}{\gamma_{d\,(\max)}} \tag{5.5}$$

Relative compaction should not be confused with the term relative density, defined through Eqs 2.52 and 2.53.

Figure 5.10 shows the relationship between relative density and relative compaction. A statistical study (Lee and Singh, 1971) on 47 different granular soils indicated that the relative compaction corresponding to zero relative density is about 80 per cent. Further, it may be noted that relative compaction can never be less than about 80 per cent, since soil will always have a unit weight and simply dumping a soil would produce a unit weight of the order of 12 to 14 kN/m^3.

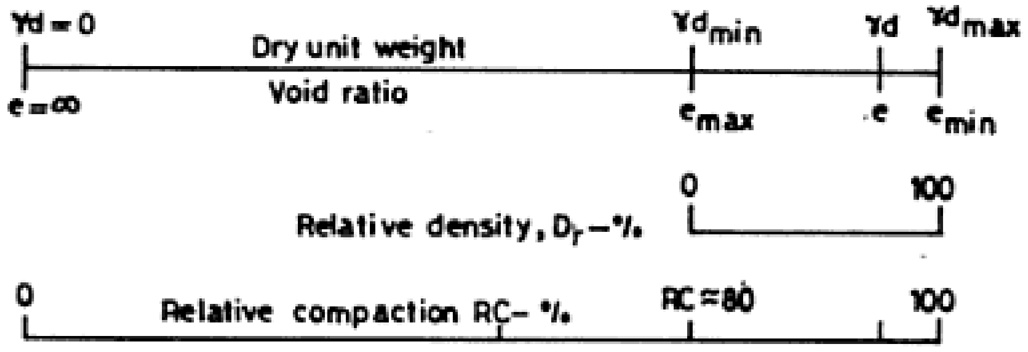


Fig. 5.10 Relative density and relative compaction concepts (after Lee and Singh, 1971)

End-product Specifications: These are used mostly for building foundations and highways. Under these specifications, it is the specified relative compaction which is important, the method (procedure and equipment used) for achieving this is not so important. The economics of any project requires that the contractor adopt the most efficient procedure. Whatever may be best from the contractor's view point, may not provide a fill with the desired engineering properties, for example, compacting a soil on the wet of optimum results in a lower shear strength than compacting the soil on the dry side of the optimum water content. Apart from this, other properties like permeability and shrinkage characteristics will also be different. Thus, a range of placement water content should be specified in addition to the percentage of relative compaction.

Method Specifications: In this method, the type and weight of the roller, the number of passes and the lift thickness are specified. Many a time, the maximum allowable size of the material is also specified. In the end- product specifications, the responsibility for proper compaction rests with the contractor whereas in the method specifications, the responsibility rests with the owner or the owner's engineer. If the compaction control test results do not meet the required standard, the contractor will have to be paid extra for extra rolling. This category of specifications calls for a prior knowledge of borrow soils. Also, test fills with different equipment, procedures, etc., may be necessary. As the test fill programmes are expensive, method specifications are useful only for large projects.

Field Control Tests

Field control tests may be *destructive or non-destructive.* Destructive tests require excavations and removal of fill material, whereas non-destructive tests involve the estimation of unit weight and water content of the fill indirectly.

Destructive field tests consist of estimating the field unit weight by excavating a hole of proper size and determining the mass of excavated material. The volume of the hole is estimated by the sand bottle method, balloon method or pouring water or oil of known unit weight into the hole (Chapter 2). The water content of the sample is determined and field dry unit weight, $\gamma_{d\,field}$ obtained and compared with $\gamma_{d\,max}$ obtained in the laboratory to calculate relative compaction. The major problem in a destructive test is the estimation of water content, which takes time. To save on the time required for water content estimation, several methods, namely, pan drying, speedy moisture meters, etc., are used. Pan drying or 'frying' the sample over an open flame is a common practice. As it is difficult to control the flame temperature. Pan drying gives poor results, especially for fat clay (CH) soils.

Water content in the field can be determined rapidly by the calcium carbide method (IS: 2720 Part II-1973) or by the *Proctor needle method.*

Proctor's compaction needle is a useful tool for the rapid determination of compaction in the field. It consists of a needle attached to a spring loaded plunger, the stem of which is graduated in kg. The needle is provided with interchangeable tips having bearing areas from 0.32 sq cm to 6.45 sq cm. The needle, fitted with a tip of known bearing area, is forced into the compacted soil in the mould in the laboratory compaction test at the rate of 1.25 cm per second to a depth of 7.5 cm and penetration resistance in kg/cm^2 is noted. A calibration chart is prepared by plotting the moulding water content against penetration resistance (Fig. 5.11). The Proctor needle is used to measure the penetration resistance of the compacted soil in the field. For a measured penetration resistance in the field, the water content can be read off from the calibration chart. Suitable corrective measures are taken if this value is different form the specified water content for compaction.

On account of the problems associated with destructive field tests, non-destructive tests for compaction control using radioactive isotopes, are becoming popular. Nuclear methods have the advantage that the test

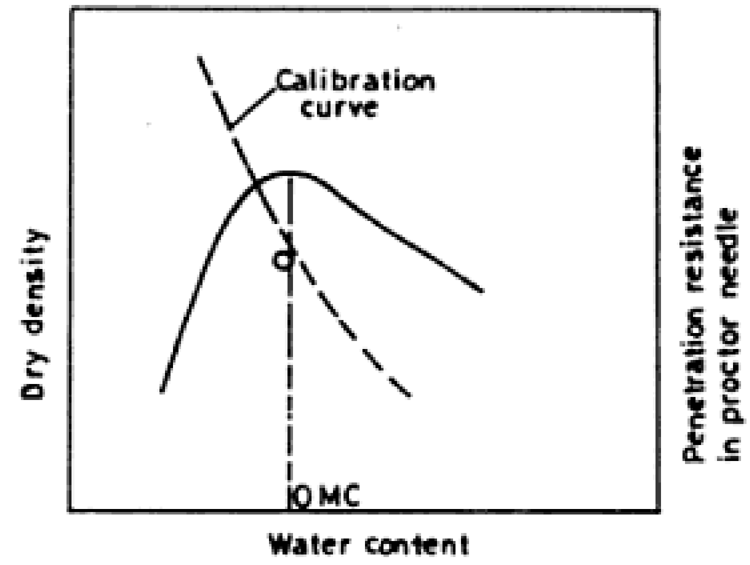


Fig. 5.11 Calibration curve for Proctor needle

can be conducted rapidly and results obtained within a short time. As a number of tests can be conducted by nuclear methods, a better statistical control of the fills is provided. However, the disadvantage of the nuclear method is the relatively high initial cost of the testing equipment and the potential danger to the field personnel from radiation.

EXAMPLES

Example 5.1 Compare the compactive energy used in the IS heavy compaction test with that of the IS light compaction test.

Solution:

Compactive energy in IS heavy compaction test

$$= \frac{4.9\ (\text{kgf}) \times 0.45 (\text{m}) \times 5\ (\text{layers}) \times 25\ (\text{blows/layer})}{10^3 \times 10^{-6}\ (\text{m}^3)}$$

$$= 27562.5\ \text{kgf m/m}^3$$

Compactive energy in IS light compaction test

$$= \frac{2.6\ (\text{kgf}) \times 0.31 (\text{m}) \times 3\ (\text{layers}) \times 25\ (\text{blows/layer})}{10^3 \times 10^{-6}\ (\text{m}^3)}$$

$$= 60450\ \text{kgf m/m}^3$$

IS heavy compaction test uses 4.56 times the compactive energy that is used in the IS light compaction test.

Example 5.2 The compaction of an embankment is carried out in 300 mm thick lifts (layers). The rammer used for compaction has the foot of area 0.05 sq m. The energy developed per drop of the rammer is 40 kg m. Assuming 50 percent more energy in each pass over the compacted area due to overlap, calculate the number of passes required to develop compactive energy equivalent to IS light compaction for each layer.

Solution:

Compactive energy as per IS light compaction test

$$= \frac{2.6\ (\text{kgf}) \times 0.31 (\text{m}) \times 3\ (\text{layers}) \times 25}{10^3 \times 10^{-6}\ (\text{m}^3)}$$

$$= 60450 \text{ kgf m/m}^3$$

Compactive energy per drop provided by the rammer per cum of the soil

$$= \frac{40}{0.05 \times 300 \times 10^{-3}}$$

$$= 2666.67 \text{ kgf m/m}^3$$

However, in each pass over a layer, the energy supplied will be 1.5 times this value on account of overlap of rammer footprints. If n is the number of passes required to develop compactive energy equivalent to IS light compaction,

$$n \times 1.5 \times 2666.67 = 60{,}450$$

$$n = 15.11 \text{ say } \mathbf{16}$$

Example 5.3. The *in situ* void ratio of a granular soil deposit is 0.50. The maximum and minimum voil ratios of the soil were determined to be 0.75 and 0.35. $Gs = 2.67$. Determine the relative density and relative compaction of the deposit.

Solution:

$$\begin{aligned}
\text{Relative density} &= \frac{e_{\max} - e_{\text{nat}}}{e_{\max} - e_{\min}} \times 100\ \% \\
&= \frac{0.75 - 0.50}{0.75 - 0.35} \times 100 \\
&= \mathbf{62.5\%} \\
\gamma_{d\,(\max)} = \frac{G_s \gamma_w}{1 + e_{\min}} &= \frac{2.67 \times 9.8}{1 + 0.35} = 19.38 \text{ kN/m}^3 \\
\gamma_{d\,(\min)} = \frac{G_s \gamma_w}{1 + e_{\max}} &= \frac{2.67 \times 9.8}{1 + 0.75} = 14.95 \text{ kN/m}^3 \\
\gamma_{d\,(in\ situ)} = \frac{G_s \gamma_w}{1 + e_{in\ situ}} &= \frac{2.67 \times 9.8}{1 + 50} = 17.44 \text{ kN/m}^3
\end{aligned}$$

$$\text{Relative compaction} \quad \frac{\gamma_{d\,(in\ situ)}}{\gamma_{d\,(max)}} = \frac{17.44}{19.38} \times 100$$

$$= \mathbf{89.9\%}$$

Example 5.4 A compacted fill is to be constructed using one of the two potential borrow areas *A* and *B*. The *in situ* properties of soil at these sites are as follows:

Borrow area *A*: $e_n = 0.80$; $w_n = 17.5\ \%$, $G_s = 2.65$

Borrow area *B*: $e_n = 0.68$; $w_n = 14.0\ \%$, $G_s = 2.65$

The compacted volume of the embankment will be 50,000 m^3, its unit weight 20 kN/m^3 at a placement water content of 20%.

Soil from the borrow area is to be excavated and transported to the site in trucks of 10 m^3 capacity. During excavation and dumping of soil in the trucks, the soil increases in volume by 10 per cent. At the site, the required additional amount of water is added to the soil and compacted to the desired extent by pneumatic rubber tyred rollers. The cost of excavation, transportation and compaction is Rs 400 per truck for borrow area *A* and Rs 500 per truck for borrow area *B*. Water charges per truck is Rs 150.

Which of the two borrow areas is more economical?

Solution:

Embankment: $\gamma_t = 20\ \text{kN/m}^3$; $V = 50{,}000\ \text{m}^3$; $w = 20\%$

Weight of soil $= \gamma_t \times V = 20 \times\ 50{,}000 = 10^6$ kN

Weight of soilds $= \dfrac{W}{1+w} = \dfrac{10^6}{1+0.2} = 8.33 \times 10^5$ kN

Weight of water $= 10^6 - 8.33 \times\ 10^5 = 1.67 \times 10^5$ kN

Volume of solids $= \dfrac{8.33 \times 10^5}{2.65 \times 9.8} = 32{,}075\ \text{m}^3$

These values are shown in the phase diagram in Fig 5.12 (a).

Borrow area *A*: $e_n = 0.80$

Volume of soil needed to have 32,075 m^3 of solids $= 32075\ (1+ 0.8) = 57{,}736\ \text{m}^3$

Number of truck trips required to be made making allowance for 10% increase in volume

$$= \frac{57{,}736 \times 1.1}{10} = 6{,}351$$

Amount of water present in 57,736 m^3 of soil

$$= w_n \times\ W_s = 0.175 \times\ 8.33 \times 10^5 = 1{,}45{,}775\ \text{kN}$$

Additional amount of water needed = 1,67,000 – 1,45,775 = 21,225 kN

Number of truck loads required to transport water $= \dfrac{21{,}225}{10 \times 9.8} = 217$

Cost of excavation, transport and compaction of soil $= 6351 \times 400 =$ Rs 25,40,400

Cost of transport of water $= 217 \times 150 =$ Rs 32,550

Total cost of using soil from borrow area A = 25,40,400 + 32,550 = Rs 25,72,950

Borrow area B: $e_n = 0.68$

Volume of soil needed to contain 32075 m^3 of solids = 32,075 (1+ 0.68) = 53,886 m^3

Number of truck trips required to be made = $\frac{53,886 \times 1.1}{10} = 5,928$

Amount of water present in 53,886 m^3 of soil = $wn \times Ws = 0.14 \times 8.33 \times 10^5 = 1,16,620$ m^3

Additional account of water needed = 1,67,000 – 1,16,620 = 50,380 kN

Number of truck trips required to transpost water = $\frac{50,380}{10 \times 9.8} = 514$

Total cost of using borrow area B = 5,928 × 500 + 514 × 150 = 29,64,000 + 77,100 = Rs 30,41,100

Using borrow area A is, therefore, more economical.

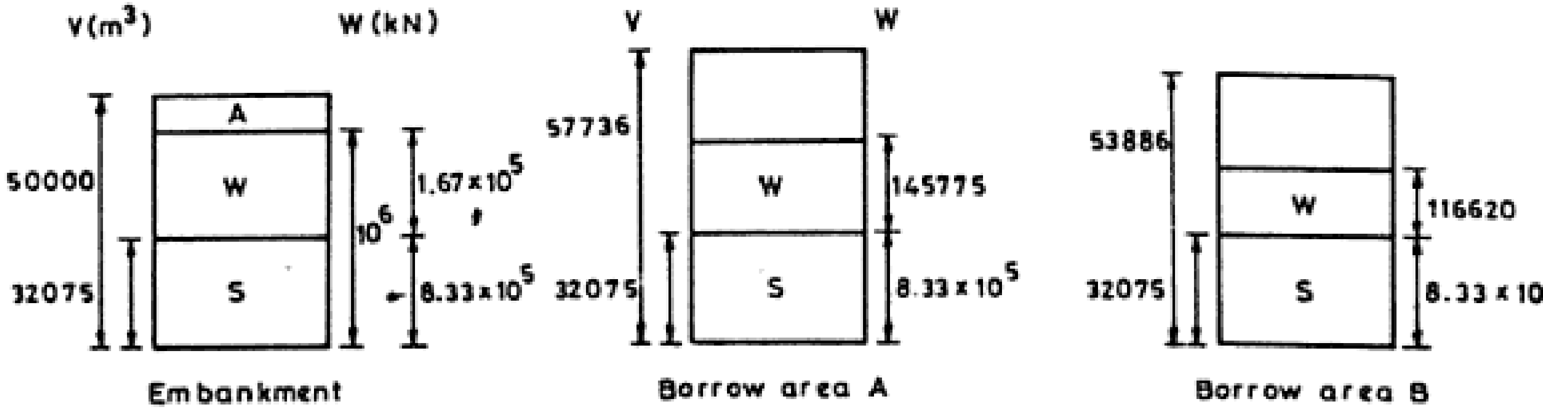


Fig. 5.12 Example 5.4

Example 5.5 The following data refers to a compaction test as per Indian Standard (light compaction):

Water content (%)	8.5	12.2	13.75	15.5	18.2	20.2
Weight of wet sample (kg)	1.80	1.94	2.00	2.05	2.03	1.98

If the specific gravity of soil grains was 2.7, (i) Plot the compaction curve and obtain the maximum dry unit weight and the optimum moisture content; (ii) Plot the 80 per cent and 100 per cent saturation lines; (iii) If it is proposed to secure a relative compaction of 95 per cent in the field, what is the range of water content that can be allowed, and (iv) Would the 20 per cent air voids curve be the same as the 80 per cent saturation curve?

Solution:

In the Indian Standard (light compaction) test, a cylindrical mould of volume 1,000 cc is used. The wet unit weight can be obtained by dividing the weight of wet sample by the volume of the mould. As the water content is known, the dry unit weight can be found by the equation.

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

The dry unit weight of a soil compacted at a certain content to achieve a certain degree of saturation can be calculated using the equation

$$\gamma_d = \frac{G\gamma_w}{1+(w\,G/S)} \text{ (given } G = 2.7)$$

The calculations can be made and tabulated as below:

Water content (%)	8.5	12.2	13.75	15.5	18.2	20.2
Dry unit weight γ_d (kN/m^3)	16.26	16.94	17.23	17.39	16.83	16.14
γ_d for $S = 80$ % (kN/m^3)	20.56	18.74	18.07	17.37	16.39	15.73
γ_d for $S = 100$ % (kN/m^3)	21.52	19.89	19.30	18.65	17.74	17.12

(i) The compaction curve is the plot between w and γ_d and is shown in Fig. 5.13.

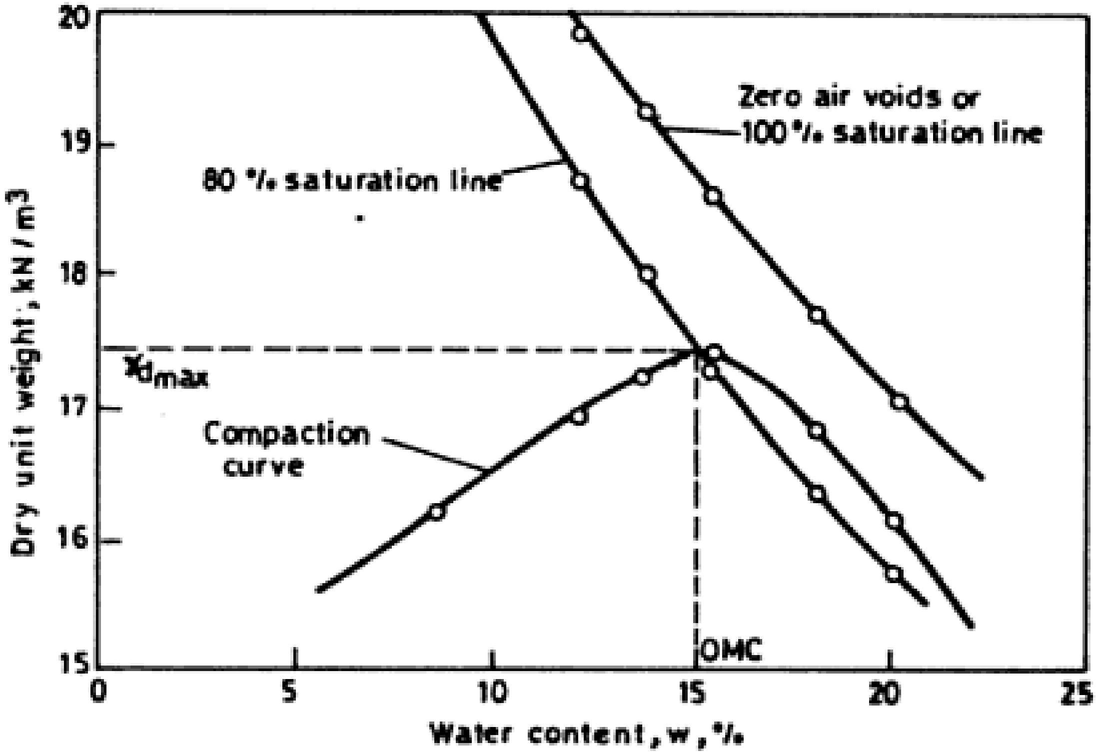


Fig. 5.13 Example 5.2

From the figure,

Maximum dry unit weight = 17.45 kN/m^3

Optimum moisture content = 15.17 %

(ii) The 80 and 100 per cent saturation lines can be drawn by using tabulated values. They are shown in Fig. 5.13.

$$\text{(iii) Relative compaction} = \frac{\gamma_{d\,\text{field}}}{\gamma_{d\,\max}} \times 100$$

$$95 = \frac{\gamma_{d\,\text{field}}}{17.45} \times 100$$

$$\Rightarrow \gamma_{d\,(\text{field})} = 16.58\ \text{kN/m}^3$$

From Fig. 5.13, the range of water content that can be allowed in the field is 10 to 17 per cent.

$$\text{(iv)}\ \gamma_d = \frac{G\gamma_w(1-n_a)}{1+wG}$$

For $w = 8.5\%$, $n_a = 0.2$, $\gamma_d = \dfrac{2.7 \times 9.8\,(1-0.2}{1+0.085\times 2.7} = 17.22\ \text{kN/m}^3$

which is different from 20.56 kN/m^3 for $S = 80\%$ and $w = 8.5\%$.

Hence the 20% air voids curve is not the same as the 80% saturation curve.

PROBLEMS

1. A borrow material has a water content of 8 per cent. Assuming that 6 kg of wet soil is required for a laboratory compaction test, estimate the amount of water required to be added to other 6 kg samples to bring their water contents upto 10, 12, 15, 18, 20 and 25 per cent.
   (0.112, 0.223, 0.389, 0.556, 0.667, 0.945 kg)

2. Following data was obtained during a field compaction control test:
   Weight of compacted soil excavated from pit = 510 g
   Bulk unit weight of sand (determined separately) = 1.46 g/cc
   Weight of sand required to fill the pit = 425 g
   Water content of soil = 10.5 per cent
   Compute the dry unit weight of the compacted fill. (1.586 g/cc)

3. An undisturbed soil in a borrow area has water content = 15 per cent, void ratio = 0.60, and specific gravity of solids = 2.7. The soil from the borrow area is used for the construction of an embankment (finished volume 50,000 cu m). the soil from the borrow area is excavated by shovels and transported through trucks (capacity 5 cu m). Whan loaded to capacity, these trucks contain, on an average, a net weight of soil equal to 65 kN.
   The construction process requires the trucks to dump their load at the site, which is then spread and broken. The water is then sprinkled to bring the water content to 18 per cent. The soil is then compacted through suitable equipment to attain a dry unit weight of 17.6 kN/m^3. Calculate:

(*a*) the degree of saturation, bulk unit weight and dry unit weight of the undisturbed borrow material.

(*b*) the number of truck loads required for construction assuming each truck to be loaded to capacity.

(*c*) the amount of water to be added per truck load, assuming that the loss of water due to evaporation during excavation, transportation is negligible.

($S = 67.5\%$; 19.4 kN/m^3; 16.9 kN/m^3; 15,570; 0.1696 m^3)

4. The soil from a borrow area having an average *in situ* unit weight of 15.5 kN/m^3 and water content of 10 per cent, was used for the construction of an embankment (total finished volume 6,000 cu m). In half of embankment, due to improper control during rolling, the dry unit weight achieved was slightly lower. If the dry unit weight in the two parts is 16.5 and 16.0 kN/m^3, find the volume of borrow area soil used in each part and the amount of soil used.

(3,512.9 m^3 and 3,406.45 m^3; 54,450 kN; 52,800 kN)

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6

Principle of Effective Stress, Capillarity & Permeability

6.1 INTRODUCTION

Soil is a multiphase system consisting of a solid phase, called the mineral skeleton and a fluid phase, called the pore fluid. The constituents of the pore phase (which is invariably water) will influence the behaviour of the soil mass in many ways. The fluid may influence the nature of the mineral surface chemically, and consequently affect the bonding forces between adjacent soil grains. This kind of interaction between the two phases, called the chemical interaction, is important mainly for fine grained soils, and has already been discussed in Chapter 4.

Another important role of the pore phase, namely, the physical interaction between the mineral skeleton and the pore fluid is discussed here. In the present discussion, a soil whose pore spaces are completely filled with water—a saturated soil—is considered. First, a *no flow* condition is considered and the nature of stresses in the soil mass is examined, and then the soil property called *permeability* is discussed. Permeability enables the flow of fluids through the soil mass. How this flow can alter the magnitude of the forces at the contact between particles and how it influences the engineering behaviour of a soil, will be discussed later.

6.2 PRINCIPLE OF EFFECTIVE STRESS

Terzaghi (1925, 1936) was the first to enunciate the effective stress principle. It can now be safely said that in doing that he opened the floodgates for the discipline of soil mechanics. Some of the basic developments on shear strength, compressibility and lateral earth pressures are a direct offshoot of the effective stress concept. According to this, the *total vertical stress* σ at a point O in a soil mass (Fig. 6.1) is given by

$$\sigma = h_1 \gamma_d + h_2 \gamma_{sat} \tag{6.1}$$

where γ_d and γ_{sat} are dry unit weight and saturated unit weight respectively. The total stress σ can be calculated by simply adding up the weights (mass × gravitational constant) of all the material (solids plus water) above the point per unit area. All that is needed to compute the total stress are the unit weights and thicknesses of the soil layers and the position of the ground water table. The total stress is thus a parameter which can be computed or even measured with suitable instruments, such as a pressure cell.

The total stress is made up of two parts. One part is due to the pore water and is called the *neutral stress* or *pore water pressure, u*. It is simply equal to the depth (h_2) below the ground water table of the point O, multiplied by the unit weight of water γ_w.

Hence $$u = h_2\,\gamma_w \tag{6.2}$$

It is called the neutral stress because it acts on all sides of the particles, but does not cause the soil particles to press against adjacent particles. It has no shear component. The soil pores can be visualised as an interconnected but intricate network of irregular tubes. Under equilibrium condition, water level in these tubes rises to the same elevation. This level can be easily determined by inserting a stand-pipe at the point in question and observing the height upto which the water rises in the stand pipe [h_2 in Fig. 6.1 (a)]. Thus the pore water pressure, like the total stress, is also a measurable parameter. A standpipe or a piezometer is used to measure the pore water pressure.

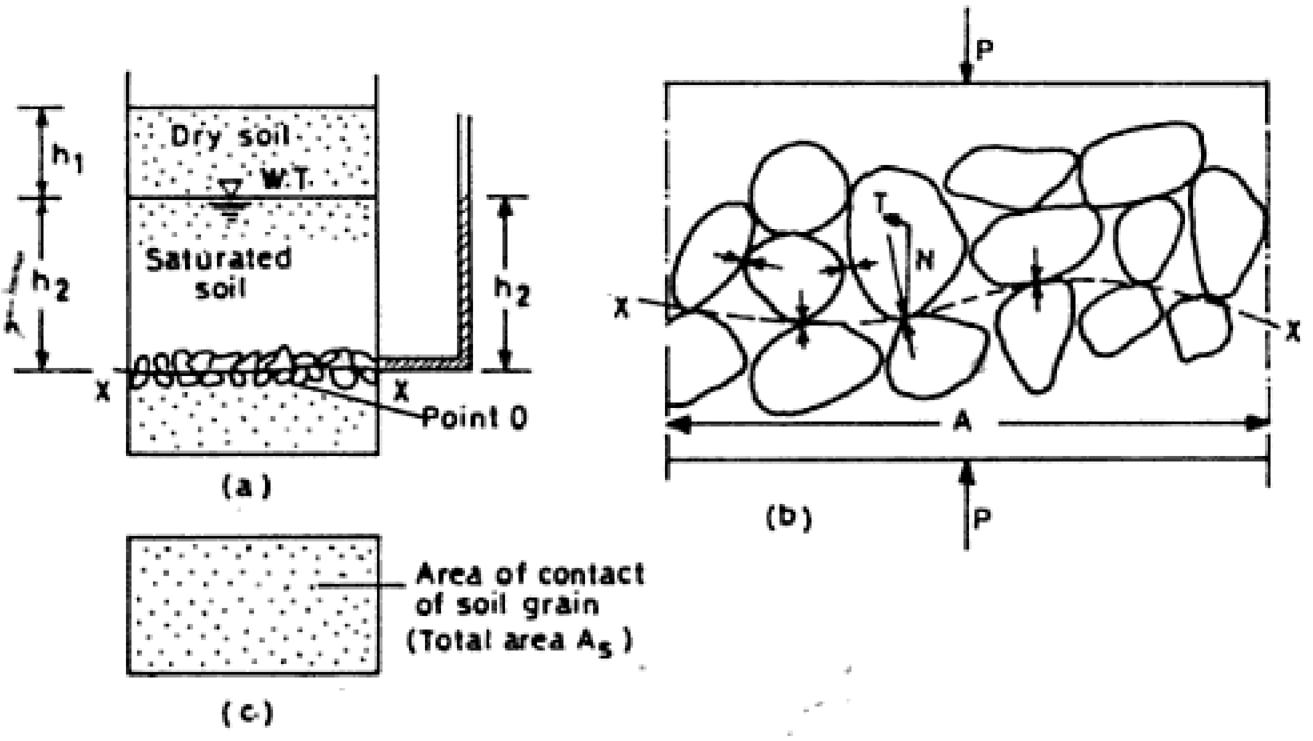


Fig. 6.1 (a) Soil mass and pore water pressure; (b) interpretation of effectives stress; and (c) sectional plan of the contact area

The other part of the total stress is due to the soil skeleton and is called the *effective stress*, $\bar{\sigma}$. Thus,

$$\sigma = \bar{\sigma} + u \quad \text{or} \quad \bar{\sigma} = \sigma - u \tag{6.3}$$

Equation 6.3 is the effective stress equation. Combining Eqs. 6.1, 6.2 and 6.3,

$$\bar{\sigma} = \sigma - u = (h_1\,\gamma_d + h_2\,\gamma_{sat}) - h_2\,\gamma_w = h_1\,\gamma_d + h_2\,\gamma' \tag{6.4}$$

where γ' is the submerged unit weight. The term $h_1\,\gamma_d$ occurs in both Eqs. 6.1 and 6.4 because for dry soils, $u = 0$ and $\sigma = \bar{\sigma}$.

Effective stress is defined as equal to the total stress σ minus the neutral stress *u*.

The total stress, σ acts on the entire area under consideration, namely, *A* (Fig. 6.1) and the pore water pressure or the neutral stress on that area, A_w where pore water is in contact with this total area, that is, the total area, *A* minus the grain contact area, A_s. The effective stress is equal, albeit approximately, to the force due to the soil skeleton divided by the total area *A* of the surface. While the total stress and the neutral stress are measurable, the effective stress is not a physical parameter and cannot be measured. It can only be computed by subtracting the neutral stress from the total stress, both of which are physical parameters.

The concept of effective stress, even if it is not a measurable parameter, is still very useful for geotechnical engineers. It can be easily realised that the mechanical behaviour of a soil mass is linked with the effective stress rather than with the total stress or neutral stress. Thus, increase in effective stress causes the particles to pack more closely, decreases the void ratio, leads to a decrease in compressibility and increases the shearing resistance of the soil. But an equal increase in the total stress and the neutral stress, which would keep the effective stress unchanged, would have little or no effect on the soil behaviour. This fact is borne out by experimental evidence as well.

The principle of effective stress can be spelt out as follows:

(*a*) The effective stress is equal to the total stress minus the pore water pressure.

(*b*) The effective stress controls certain aspects of the behaviour of soil, the most important of which are the volume changes and the shearing resistance.

6.3 PHYSICAL MEANING OF EFFECTIVE STRESS

From the principles of mechanics, it is known that stress is a fictitious parameter and is not physically meaningful. It is especially so in a material like soil, which is not continuous and contains both the soil grains and void spaces within its fabric. In this kind of material, on a microscopic scale, as the area reduces to a point in the limit, the point may lie on a void instead of a particle.

A truly horizontal surface through a soil at any depth must cut through many mineral particles. However, conditions of stress at the points of contact of particles, rather than within them, are of relevance in consideration of strains, shear resistance, etc., within a soil mass. Hence, it would be rather useful to consider a wavy plane *XX* in a fully saturated soil, passing through points of inter-particle contact only, as shown in Fig. 6.1(b). On the mass scale, the wavy plane is really indistinguishable from the true horizontal plane because of the small size of particles. As indicated in Fig. 6.1(c), the area of contact of soil grains, A_s is a very small fraction of the total surface area. The entire area of this horizontal plane can thus be taken to go through the pore water. The 'intergranular stress', the terminology which is sometimes used for 'effective stress', is really a misnomer for the reason that effective stress is really not the stress at particle contacts. The actual contact stress can be very large since the contact area between particles is very small. Effective stress is the sum of the contact forces divided by the gross area. This is the reason why effective stresss is not physically meaningful and cannot be measured.

Now consider the inter-particle forces, which are random in magnitude and direction throughout the soil mass [Fig. 6.1 (c)]. In the plane *XX*, at every point of contact, these forces can be resolved into components normal and tangential to the direction of the true plane, which the wavy plane *XX* can be said to approximate.

The effective normal stress $\overline{\sigma}$ is equal to the sum of all the components *N'* within the area *A* of the plane divided by the area *A*, or

$$\overline{\sigma} = \frac{\Sigma N'}{A} \tag{6.5}$$

The total normal stress σ is given by

$$\sigma = \frac{P}{A} \tag{6.6}$$

Since the pore water pressure can act only over the void area, the hydrostatic force is equal to the pore water pressure u multiplied by the area of pores, A_w.

For equilibrium in the direction normal to plane XX,

$$P = \Sigma N' + uA_w \tag{6.7}$$

$$\frac{P}{A} = \frac{\Sigma N'}{A} + u\frac{A_w}{A} \tag{6.8}$$

or

$$\sigma = \overline{\sigma} + u\frac{A_w}{A} \tag{6.9}$$

If A_w is taken as approximately equal to A (a condition which is almost satisfied in the case of granular soil particles where point contact can be assumed),

$$\sigma = \overline{\sigma} + u \tag{6.10}$$

Equation 6.10 is very important for saturated soils. It may be noted that $\overline{\sigma}$ here does not represent the true contact stress between particles, which would be the much higher value of $\frac{\Sigma N'}{A_c}$, where A_c is the *actual* contact area between the particles.

In the case of clay mineral particles in a soil mass, the mineral crystals are not in direct contact, since they are surrounded by adsorbed layers of water. But it is assumed that inter-granular forces can be transmitted through the adsorbed water.

In conclusion, it can be safely said on the basis of experimental evidence and practice that the effective stress is a function of the particle contact force and for saturated sands and clays, the principle of effective stress holds considerable validity.

Effective Stress in a Partially Saturated Soil

In a partially saturated soil, the water in the void spaces is not continuous—that is, the material comprises a three-phase system of solids, pore water, and pore air (Fig. 6.2). Hence, the total stress at any point in a soil consists of the effective, pore air, and pore water pressures. From laboratory test results, Bishop *et al.* (1960) gave the following equation for effective stress in partially saturated soils:

$$\overline{\sigma} = \sigma - u_a + \chi\,(u_a - u_w) \tag{6.11}$$

where σ = total stress

u_a = pore air pressure

u_w = pore water pressure

In Eq. 6.11, χ represents the fraction of a unit cross-sectional area of the soil occupied by water. For a dry soil, $\chi = 0$, and for a saturated soil, $\chi = 1$.

Bishop *et al.* (1960) have pointed out that the intermediate values of χ will depend primarily on the degree of saturation S, but will also be influenced by factors such as soil structure, the cycle of wetting and drying or stress change leading to a particular value of S.

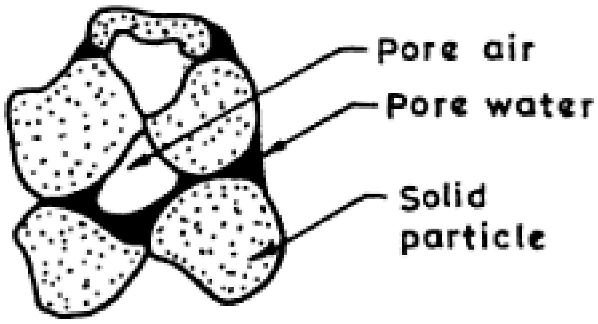


Fig. 6.2 Partially saturated soil

6.4 CAPILLARITY IN SOILS

Rainfall accounts for most of the land-based water. This water flows under the influence of gravity and results in the formation of streams, rivers, lakes etc., on the surface, but a considerable proportion of the precipitation percolates into the rocks and soil. The depth of penetration of percolating water is basically a function of the porosity of the soils or rocks.

The ground water can exist in either of the two forms, namely, *(i) phreatic* or *gravitational water*, and *(ii) capillary* water, occurring in two distinct zones demarcated by the water table of 'phreatic surface' (Fig. 6.3c).

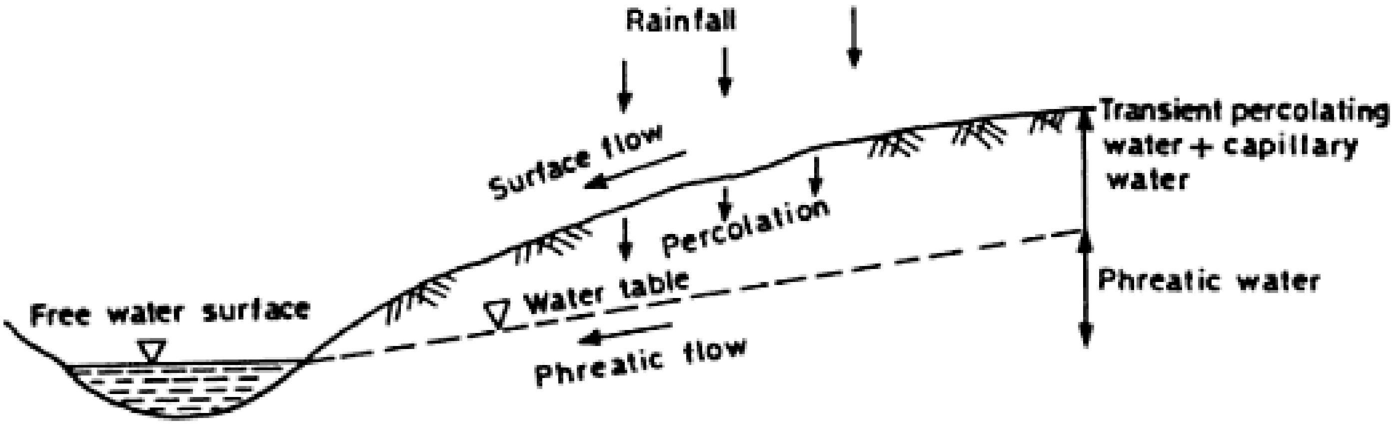


Fig. 6.3 Ground water occurrence

The gravitational water is subject to gravitational forces, saturates the voids completely, has an internal porewater pressure greater than atmospheric pressure and flows in lateral direction. Gravitational water may be removed from soils by drainage.

The upper surface of the zone of full saturation of the soil is called the *water table* or *phreatic surface.* At the water table, the ground water is subjected to atmospheric pressure. In other words, the pore water pressure is zero.

If the water contained in the pores of the soil were subject only to the gravitational force, the soil above the water table would be perfectly dry. However, in reality, every soil in the field is completely saturated upto some height above the water table and partially upto some more height. This is attributed to the phenomenon of *capillarity* in soils.

Capillary water is held above the water table by *surface tension,* a fluid property, which is the attractive force exerted at the interface between materials in different physical states, i.e., liquid/gas, solid/liquid. In the case of soils, it occurs between surfaces of water, mineral grains and air. Surface tension is attributed to the differences in forces of attraction between the molecules of the materials at the interface.

To demonstrate the capillary effects in soils, the analogy of small bore glass tubes with the voids between the soil grains, is quite handy. Consider a capillary tube of diameter *d* standing with its lower end in water. At the triple interface, the water surface is pulled up into a meniscus, with the surface tension force *T* acting along its perimeter at angle α (contact angle) to the wall of the tube (Fig. 6.4). The adhesion forces between the glass and water molecules cause the water to rise in this manner. The surface tension (force) pulls the water upwards until, at a height h_c, the weight of water in the column is in equilibrium with the magnitude of the surface tension force [Fig. 6.4 (a)]. The meniscus formed is concave upward in case of water. In the case of some materials (like mercury for instance), where the internal cohesion forces are greater than the forces of adhesion, the substance does not wet the glass; the resulting meniscus is depressed and its shape is convex upward.

From Fig. 6.4, the equation of equilibrium can be written for the forces acting at the meniscus. The force acting downward is the weight of the column of water of height h_c suspended from the meniscus. The upward force is the vertical component of the surface tension force around the circumference. Figure 6.4 (b) shows the magnified view of the meniscus at the surface.

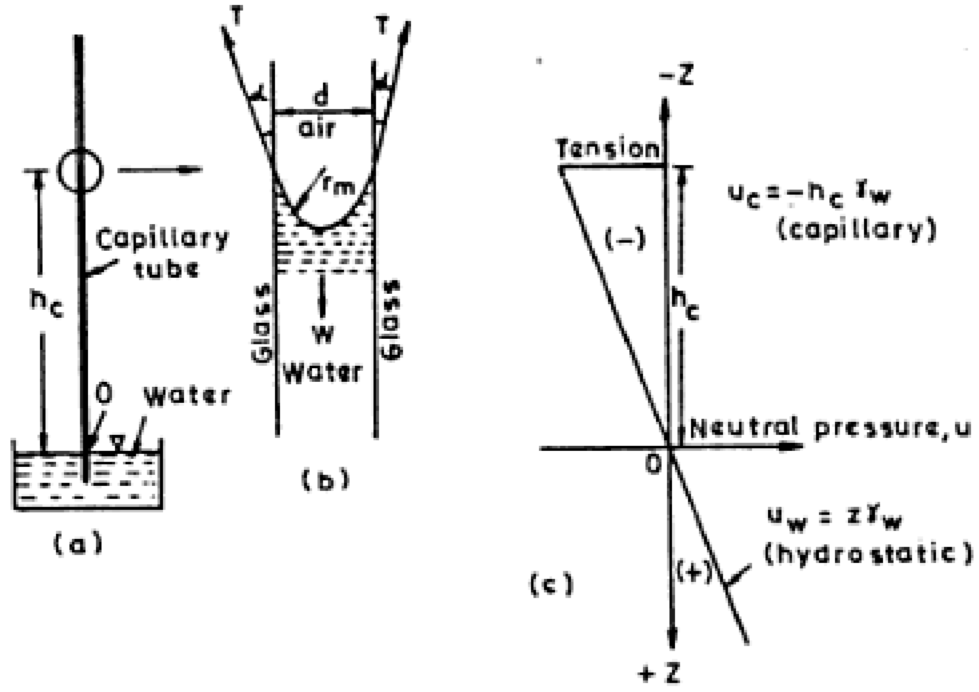


Fig. 6.4 Capillary rise: *(a)* capillary tube, *(b)* magnified at surface and *(c)* capillary and hydrostatic pressure

$$h_c \frac{\pi d^2}{4} \gamma_w = \pi \, d T \cos \alpha$$

where *T* has dimensions of force/unit length. For chemically clean glass tube and pure water, $\alpha = 0$.

Thus,

$$h_c = \frac{4\,T}{\gamma_w \, d} \tag{6.12}$$

T is a physical property of water and its value is about 73 dynes/cm or 73 mN/m. Since $\gamma_w = 9.81$ kN/m^3, fo pure water in clean glass tubes, Eq. 6.12 becomes

$$h_{c(m)} = \frac{3\,(10^{-5})\ \text{m}}{d_{(m)}} = \frac{0.03}{d\ (\text{mm})} \qquad (6.13$$

which is convenient to remember.

The height of the column of water is greater when the diameter of the tube is smaller. Figure 6.4 (c) show the stress distribution in water. At the level of the water surface in the container, the pressure is one atmospher (or the gauge pressure) inside the capillary tube, the same as outside the tube since there is no elevatio difference between them. Below the surface level, the pressure increases linearly with depth, so that at a dept below point O, its magnitude is $z\,\gamma_w$. Above this point, the pressure in water is negative, that is, less tha atmospheric pressure. Thus, the water here may be said to be in tension. Its value u_w at height h_c above O i given by

$$u_w = -h_c\,\gamma_w$$

$$= -\frac{4T}{d} \ \text{(from Eq. 6.12)} \qquad (6.14$$

When $\alpha = 0$, the radius of the meniscus r_m is equal to $d/2$. For $\alpha \neq 0$, r_m is greater than the radius of th tube.

From consideration of the relationships between absolute, gauge and vapour pressure of water, it ca be shown that in capillary tubes less than 3μ m in diameter, water cannot cavitate while under tensio The capillary rise in such small tubes is a function only of their diameters. But in tubes of larger diameter the limit for the maximum negative pressure that can be attained is governed by the vapour pressure o water, and hence cavitation will result when the ambient pressure reaches the vapour pressure. Therefor for large tubes, the maximum negative pressure depends only on the atmospheric pressure and not on th diameter of the tube.

Capillary Rise of Water in Soils and Capillary Pressures

The capillary tube analogy to explain the capillary phenomenon in soils is true only to a degree. In contrast t capillary tubes, the voids created by random assemblages of soil grains are themselves random and irregula and communicate with each other in all directions. When water rises in a soil from below, the lower pa becomes fully saturated. In the upper part, however, water fills only the narrowest voids and the larger void remain occupied by air. In Fig. 6.5, the overall regime of ground water has been divided into four zones. Whil the value of h_c given by Eq. 6.13 represents the maximum capillary rise, the soil will be completely saturate by capillary moisture only up to a height h_{cs}. A few large voids may effectively stop capillary rise in certai parts and in the smaller pores, water would rise upto h_c. Thus, the zone between the heights h_{cs} and h_c remain partially saturated. For a detailed discussion on the influence of capillary tubes of different lengths, shapes an sizes on capillary rise, the reader is advised to refer to Taylor (1948).

The negative pressure of water (*capillary pressure*) held above the water table results in attractive force between the particles and is referred to as *soil suction*. Figure 6.6 shows two sand particles connected by menis of radius r_m inducing in them an intergranular contact stress σ' ($\overline{\sigma}$).

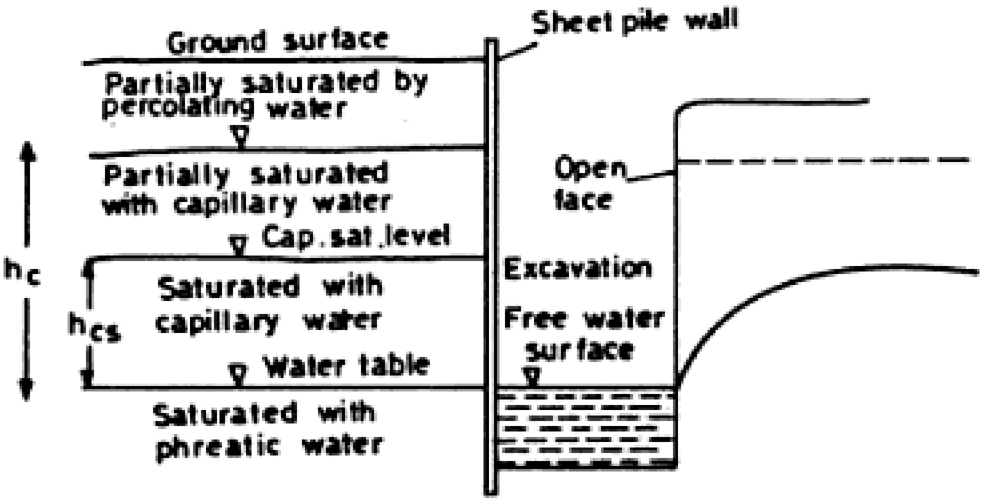


Fig. 6.5 Various zones of groundwater

Because of the complex nature of the soil voids, a theoretical prediction of capillary rise in soils is not only of dubious accuracy, but is even misleading. An empirical method, based on direct observation of capillary rise *in situ*, in the more reliable approach. One approximate relationship suggested by Terzaghi and Peck (1967) is of the form

$$h_c\,(\text{cm}) = \frac{C}{e\,D_{10}} \tag{6.15}$$

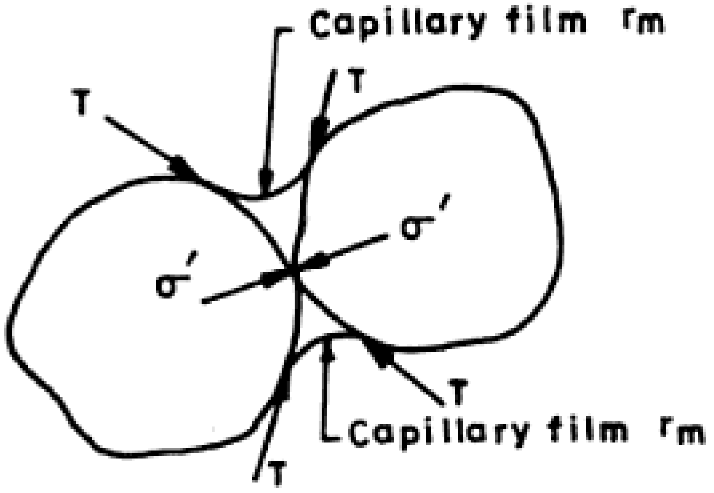


Fig. 6.6. Capillary film inducing attractive force

in which e is the void ratio, D_{10} is the effective grain size in cm and C is an empirical constant which can have a value between 0.1 and 0.5 sq cm. Another method of estimating the theoretical height of capillary rise is to use Eq. 6.13 by substituting for effective pore diameter d a value equal to 20% of the effective grain size D_{10} (a reasonable assumption in soils). The important point to note here is that capillary rise is controlled by pore size and not the grain size, and that the same soil mass with the same D_{10} can have different pore size distributions depending on soil structure and fabric, geological history, etc., leading to variable capillary pressure.

In the preceding pages, it was established that the intergranular or effective stress $\bar{\sigma}$ is equal to the total stress, σ minus the pore water pressure, u, i.e., $\bar{\sigma} = \sigma - u$. When the pore water is in compression, as in the case of hydrostatic pressure, u is positive and when it is in tension, u is negative. Since u is negative in the capillary zone [Fig. 6.4 (c)], $\bar{\sigma} = \sigma - (-u_c) = \sigma + u_c$. In other words, the effective stress is increased by u_c. If the height of capillary rise is h_c,

$$u_c = -h_c \gamma_w \tag{6.16}$$

and, therefore, the capillary pressure or the intergranular pressure induced by capillary flow can be very large indeed for fine-grained soils. It is well understood that the most important consequence of the increase in $\bar{\sigma}$ is an increase in shearing resistance of the soil mass. This is the reason why one can walk comfortably on a sand beach adjacent to the sea, a patch falling within the capillary zone but not on the dry sand above the capillary zone some further distance away from the sea where the soil exhibits much less bearing power in the absence of capillary pressures (Fig. 6.7). Again, when the sea water breaks the capillary menisci during tides, the temporarily induced shearing strength is lost. Anyone walking on this surface at this time can virtually feel the ground sinking under his feet. Yet, in all these different states of the beach zone, the relative density of the soil remains more or less the same. It is the presence or absence of capillary moisture that makes all the difference.

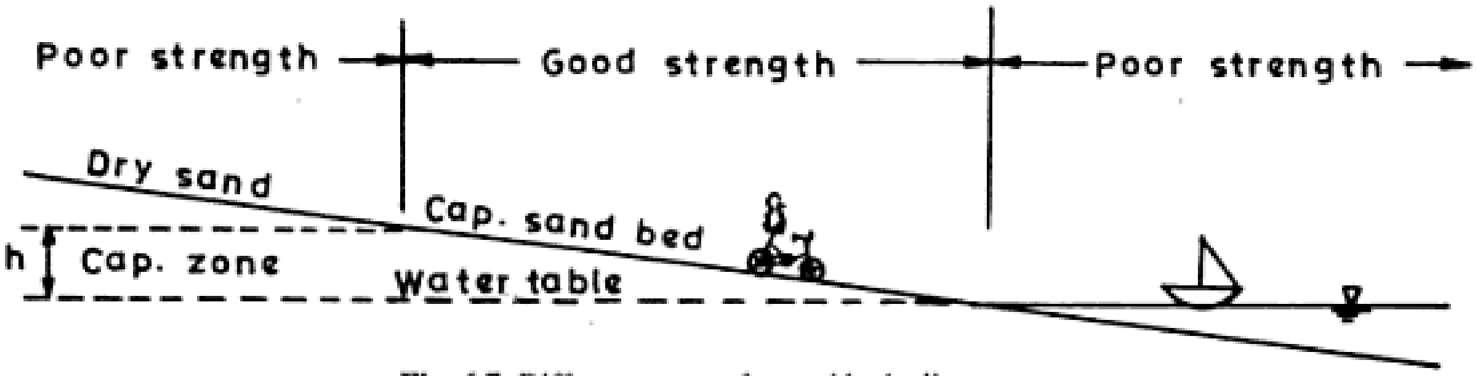


Fig. 6.7 Different states of a sand bed adjacent to sea

Capillary moisture in fine sands and silts allows excavations to be made because of the stability it provides by virtue of the induced shearing strength. Such excavations would be impossible under dry soil conditions or under water table. Capillary menisci are easily destroyed by saturation due to rainfall or by any disturbance or even by evaporation. The stability in fine sands and silts is a temporary phenomenon which lasts as long as the capillary moisture exists.

Bulking in moist sands is another manifestation of capillary moisture. The capillary menisci surrounding the soil grains produces *apparent cohesion* which holds the particles together in clusters, enclosing honeycombs, especially when the moist soil mass is loosely poured (Fig. 6.8). The increase in volume, known as bulking, occurs only in moist sands. Saturating the soil mass would result in the destruction of the menisci and an appreciable decrease in volume. It is not a good idea, therefore, to buy moist sand by volume.

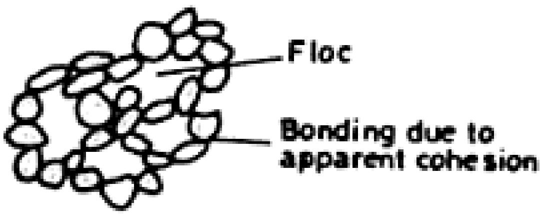


Fig. 6.8 Bulking in moist sand

Another effect of capillarity is often called capillary siphoning. Water may flow over the crest of an impermeable core in a dam even though the free surface may be lower than the crest of the core. Terzaghi and Peck (1967) report the case of the 20 km long canal between Berlin and Stettin in Germany where a loss of

2,000 litres of water per minute was observed, when the impermeable core of the dikes was 30 cm above the water level. This freeboard was apparently less than the height of capillary rise, resulting in capillary syphoning of water. The loss was reduced to 450 litres per minute when the core height was increased to 40 cm.

The above paragraphs highlight the practical significance of capillary rise in soils. The height of capillary rise can be really large in the case of fine grained soils and capillary pressures can be of significant magnitudes. Table 6.1 shows approximate heights of capillary rise of different soil types.

Table 6.1 Approximate Height of Capillary Rise in Different Soils

Soils	*Height of capillary rise**
Coarse sand	0.03 – 0.15 m
Medium sand	0.12 – 1.10 m
Fine sand	0.30 – 3.5 m
Silt	1.5 – 12 m
Clay	> 10 m

*Loose sand/soft clays show smaller rise compared to dense sand/stiff clay. (after Hansbo, 1975)

6.5 PERMEABILITY OF SOILS

Since soils consist of discrete particles, the void spaces between the particles are inter-connected and may be viewed as a highly complex and intricate network of irregular tubes. In a two-phase solid-liquid system, these voids are completely filled by the liquid, which is water in most problems of soil engineering. Water in these tubes is free to flow when a potential difference is created in a soil mass. Water flows form zones of higher potential to zones of lower potential. The resistance to flow is greater when the pores are smaller in size and pore channels irregular. On the other hand, the resistance to flow is much less when soils have larger voids, *i.e.*, more or less regular flow channels. Thus, even while all soils are permeable, the degree of perviousness is different. Gravels are more pervious than sands, sands more pervious than silts and silts more pervious than clays. Indeed, the same soil may exhibit different degrees of perviousness, depending on its structure. A loose sand is much more pervious than when it is dense. A clay soil with a flocculated structure is more pervious than the same soil with a dispersed structure. The *permeability* of a soil is a soil property which describes quantitatively, the ease with which water flows through that soil. It is a very important engineering property of a soil. Since permeability directly influences the rate of flow of water in a soil, it has a decisive effect on problems involving excavations of open cuts in sands below water table (where estimates of the rate of flow of water into the cut to keep the area free from water by pumping are required), seepage through earth embankment dams (where a clay core is sometimes used to inhibit seepage loss stability of foundations (where a quick release of pore water pressure by drainage will increase the shearing resistance), subgrade drainage, rate of consolidation of compressible soils and many other engineering aspects.

Some Basic Concepts of Fluid Flow—Darcy's Law

In 1856, Darcy, a French water works engineer, studied experimentally the flow of fluids through porous media. Before taking up Darcy's work for discussion, a brief description of the terms used in the dynamics of fluid flow would be in order.

Steady flow in fluids means that flow parameters such as pressure, head or velocity do not vary with time.

Unsteady flow refers to flow conditions which vary with time; also referred to as *transient flow.*

One-dimensional flow is the type of flow where the fluid parameters such as pressure, velocity etc., are the same at all points in a section perpendicular to the flow direction but can vary from section to section along the direction of flow, *i.e.*, with respect to only one coordinate variable, say x.

Two-dimensional flow means the fluid parameters vary in two coordinate directions, *i.e.*, $f(x, z)$ but they remain the same in parallel planes.

In *three-dimensional flow,* the fluid parameters are a function of three coordinate directions *i.e.*, $f(x, y, z)$. An assumption of one or two-dimensional flow is made in most soil engineering problems, to simplify the analysis. Even where there is a departure from the assumed flow conditions, the error involved is not appreciable.

Again, in geotechnical engineering problems, assumption of *incompressible flow* of water is made without much error.

Flow of fluids is described as *laminar* if a fluid particle follows a defined path and does not cross the path of other particles and as *turbulent* where random velocity fluctuations result in a zig zag and criss-crossing path of fluid particles. There can be transition states between laminar and turbulent flow.

The fundamental laws that determine whether in a given case, the flow is laminar or turbulent, were formulated by Reynolds. The results of his experiments with fluid flow in a tube are shown in Fig. 6.9. Flow velocity, v is plotted against the hydraulic gradient, i which is the head lost in flow due to friction per unit length of tube, *i.e.*, h/l. The flow velocity is proportional to the hydraulic gradient as long as the flow is laminar (Section I). As the velocity is increased, a critical state is reached at which eddies begin to form near the wall of the tube and the relationship between flow velocity and hydraulic gradient becomes uncertain (Section II). A further increase in velocity again leads to a definite relationship; this time, however, represented by a continuous curved line (Section III). Once in the turbulent zone, if the velocity is decreased, the flow remains turbulent well into transition zone II until the motion reverts to laminar flow. The velocity at which this change occurs is called the *lower critical velocity* v_{cr}. Below v_{cr} the flow is always laminar. Reynolds found that v_{cr} is inversely proportional to the diameter of the tube and established the following general formula applicable for any kind of fluid. It is independent of the system of unit.

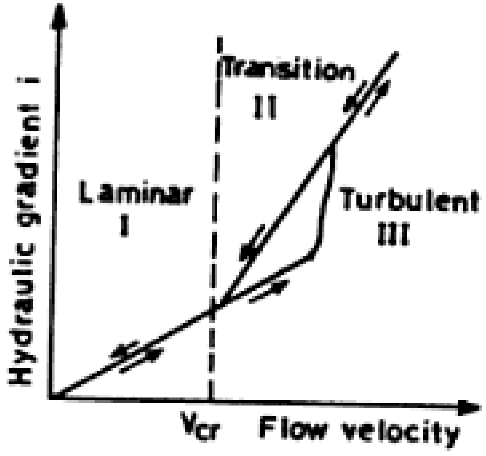


Fig. 6.9 Laminar and turbulent flow of liquids

$$\frac{v_c d\gamma_w}{\eta g} = 2000 \qquad (6.17)$$

in which d is the tube diameter, γ_w is the unit weight of water, η is the viscosity of the fluid and g is the acceleration due to gravity.

In most soils (except perhaps coarse gravel), the pores are so small that the flow of water through them is always laminar. Hence, from Fig. 6.9, the velocity of flow is proportional to the hydraulic gradient. Darcy conducted his experiments on *clean sands.* A schematic diagram, such as the one shown in Fig. 6.10 can be used to study the flow conditions and explain the development of Darcy's law. The figure shows a section through the soil sample of length L and cross-sectional area A. Connected to each end of the sample are water reservoirs in which the water levels can be maintained constant by overflows. The difference between these

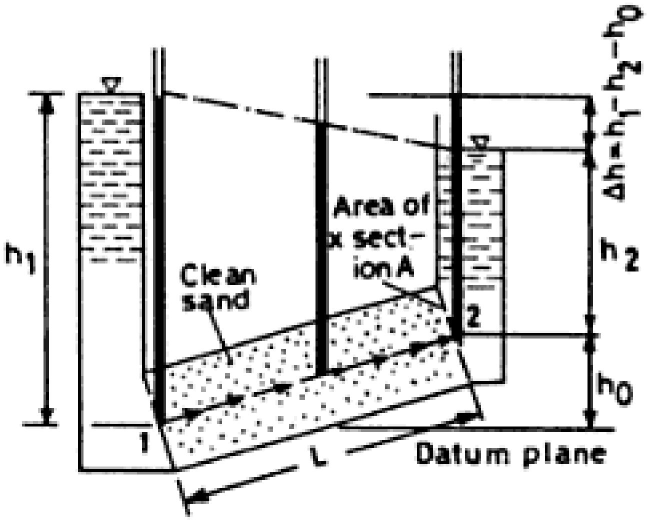


Fig. 6.10 Basic definitions in the theory of seepage

two water levels is denoted by Δh. The flow demonstrated here is laminar and also one dimensional, which implies that the flow velocity is the same at any point of a cross section perpendicular to the direction of flow. If stand-pipes are inserted at points 1 and 2 (Fig. 6.10) which mark the two ends of flow line, the heights to which water rises in the pipes indicate the pressure heads at these points. If elevation of point 1 is taken as datum, h_o represents the elevation head of point 2. From fluid dynamics, we know,that the potential difference that causes flow is the difference between *total potential* or *total head* which is the sum of the pressure head, the elevation head and the velocity head. For most soil flow problems *the velocity head is usually ignored* since it is small compared to the other two. The pressure head at 1 is h_1 and at 2 it is h_2. If water stands at the same elevation in both pipes, no flow occurs. If the elevation head h_o is zero (when 1 and 2 are at the same elevation, the difference in pressure heads that causes flow is the difference in piezometric heads between points 1 and 2. But in the set up shown in Fig. 6.10, it is equal to $\Delta h = h_1 - h_2 - h_0$. Thus, to determine the total head difference between any two points in a soil, one needs to insert two piezometers at two points and note the difference in the levels of water in the piezometers.

By conducting tests on the same soil and varying one of the three variables Δh, L and A at a time, Darcy established that the flow occurring per unit time is directly proportional to the head causing flow and the area of cross-section of the soil sample but is inversely proportional to the length of the soil sample; thus

$$q \;\alpha\; \frac{\Delta h}{L} A \tag{6.18}$$

where q is the flow and has the units of volume/time. The constant of proportionality was observed to change when the soil in the tube was changed. Thus, it is a function of the soil property, namely, the *coefficient of permeability of the soil, k*; hence

$$q = k \frac{\Delta h}{L} A \tag{6.19}$$

Since $\frac{\Delta h}{L}$ is the hydraulic gradient i,

$$q = k\,i\,A \tag{6.20}$$

Dividing both sides of Eq. 6.20 by A

$$q/A = v = k\,i \tag{6.21}$$

Darcy's law can be expressed in the form of either Eq. 6.20 or Eq. 6.21.

In Eq. 6.21, v is referred to as the *superficial velocity* of flow; superficial or fictitious because the actual flow is through the pores in the cross-section and not through the entire cross-sectional area A. The permeability of a soil can then be defined as the *superficial velocity of flow under unit hydraulic gradient.* The actual velocity of water flowing in the voids is called the *seepage velocity* v_s. Hence,

$$q = vA = v_s A_v \tag{6.22}$$

where A_v is the area of voids in the total cross sectional area A.

$$\text{Since} \quad \frac{A_v}{A} \simeq \frac{V_v}{V} \simeq n, \; v = nV_s \tag{6.23}$$

In engineering practice, for the sake of convenience, v is used instead of v_s. Since $n < 1$, the seepage velocity is always greater than the superficial or discharge velocity.

Measurement of Permeability

The coefficient of permeability can be determined in three ways—by laboratory tests, by field tests and by empirical approach.

In the laboratory, it is possible to use either the constant head or the variable head test.

Constant head test: It is based on the measurement of the quantity of water that flows under a given hydraulic gradient through a soil sample of known length and cross-sectional area in a given time. Constant head permeameters are specially suited to the testing of pervious, coarse grained soils, since adequate, measurable discharge is needed for the accurate determination of permeability by this method. A typical set-up of this kind of permeameter is shown in Fig. 6.11. The soil sample is contained in a perspex cylinder. At the side of the cylinder, a number of manometer connection points are provided to enable pairs of pressure head readings to be taken (one pair is shown in the figure). Water is allowed to flow through the sample from a reservoir designed to keep the water level constant by overflow. The quantity of water flowing out of the soil or discharge Q during a given time t is collected in a vessel and weighed.

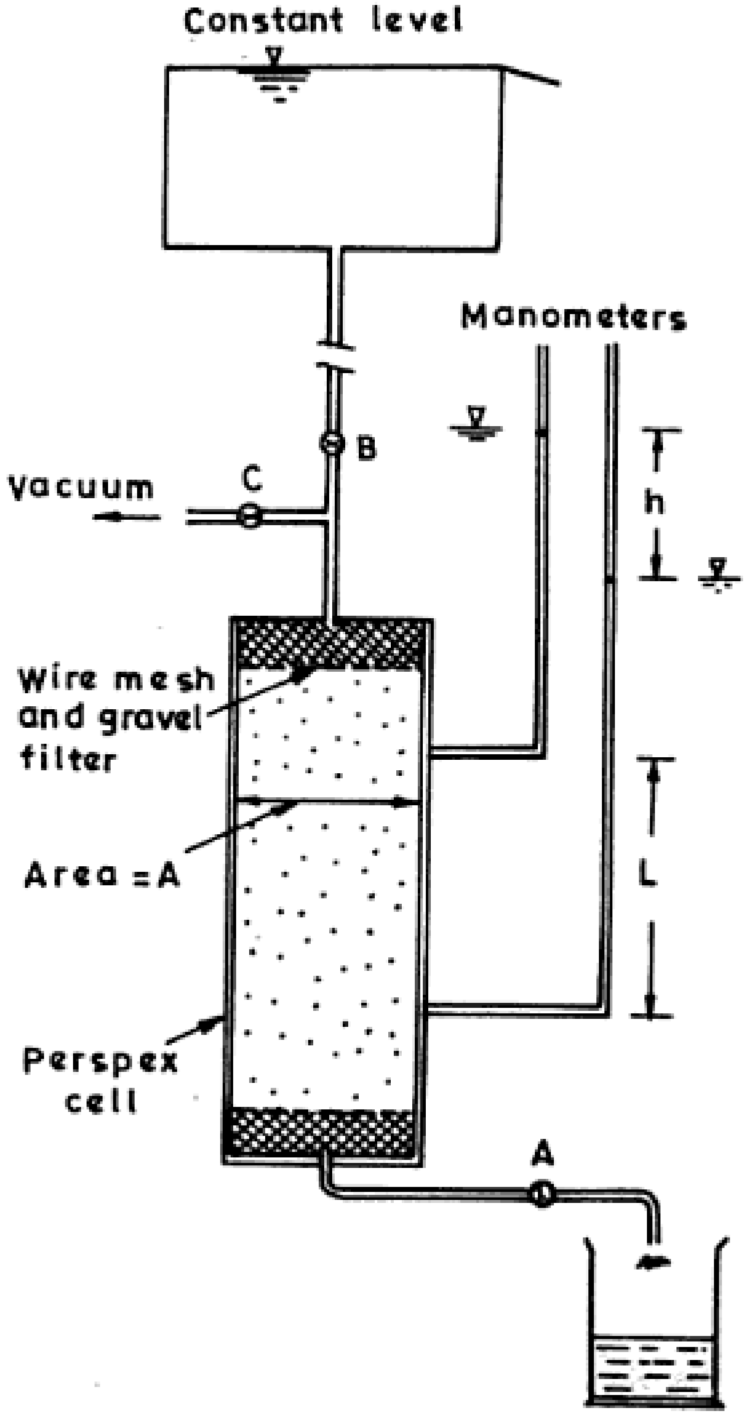


Fig. 6.11 Constant head permeameter

The presence of entrapped air in the soil can affect the results seriously. To eliminate this possibility, firstly de-aired water is supplied to the reservoir and then vacuum is applied to the soil sample before commencing the test. The test is then started by closing the valve C and keeping valves A and B open, with valve A being used to control the rate of flow. Flow is allowed to continue till a steady state of flow is established, as evidenced by constant levels in the manometer tubes. Then the discharge is measured. Several such tests at varying rates of flow can be performed and the average value of k determined.

From Eq. 6.20,

$$q = k\ i\ A$$

Hence, $$k = \frac{q}{iA} = \frac{QL}{Aht}\ \text{cm/s} \tag{6.24}$$

where k = coefficient of permeability (cm/s)

Q = discharge (cm^3) collected in time t (s)

A = cross-sectional area of sample (cm^2)

h = difference in manometer levels (cm)

L = distance between manometer tapping points (cm)

Falling head test: This method is used to determine the permeability of soils such as fine sands and silts. In such soils, the permeability is too small to enable accurate measurement of discharge using a constant head permeameter. A common arrangement of a falling head permeameter is shown in Fig 6.12. A cylinder containing the soil sample is placed on a base (perforated disc) fitted with a fine gauze. A graduated standpipe of known diameter is connected to the soil sample.

The test is conducted by filling the standpipe with de-aired water and allowing flow to take place through the sample. During the test, the water level will continuously drop and the height of water in the standpipe is recorded at several time intervals during the test. Any one pair of measurements, namely, the time taken for the head to fall from h_1 to h_2 (Fig. 6.12), will yield one value of k. The average value of k can be computed from several such readings.

From Darcy's law, the discharge in unit time

$$q = k\ \ iA = k\ \frac{h}{L}A$$

where h is the flow head at an intermediate time t between t_1 and t_2 corresponding to h_1 and h_2.

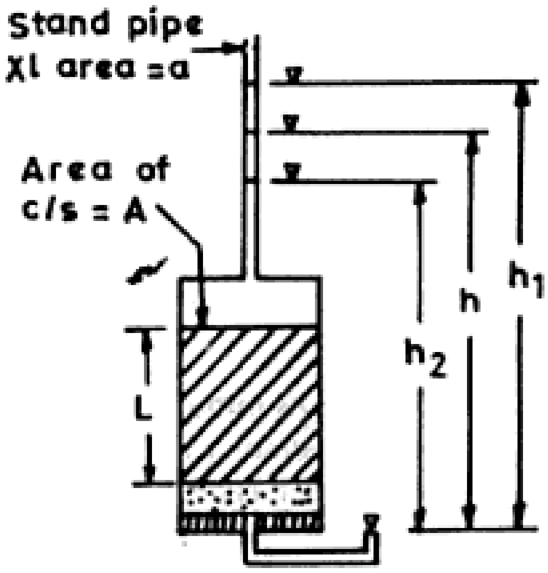


Fig. 6.12 Falling head permeameter

Referring to Fig 6.12, if the level of water in the standpipe falls by dh in time dt, the flow in unit time through the sample = q = a × velocity of fall, *i.e.* $-a\ \frac{dh}{dt}$, the negative sign indicating that the head decreases with time. Equating the two expressions

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

Rearranging and integrating,

$$-a\int_{h_1}^{h_2}\frac{dh}{h} = k\frac{A}{L}\int_{t_1}^{t_2}dt$$

Hence,

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

If $t_2 - t_1 = t$,

$$k = 2.3\,\frac{aL}{At} \log_{10} \frac{h_1}{h_2} \tag{6.25}$$

Laboratory tests must be performed, as far as possible, on undisturbed soil samples. But in the type of soils for which permeability is important (sand, silty sand, etc.) undisturbed sampling poses the greatest problem. The permeability values obtained from the laboratory tests may often be quite different from the true values. Degree of saturation being less than 100 per cent is one important reason. This happens when there is entrapped air in the sample. But by far the most important reason for discrepancy is the impossibility of simulating the *in situ* soil structure in the test sample. Permeability of natural soil deposits is greatly influenced by macrostructure factors such as heterogeneity, stratification, etc. and such effects can hardly be reproduced in the laboratory. To properly account for these factors, the geotechnical engineer may have to use the field pumping tests.

There are several ways of measuring permeability in the field. The most suitable method in a given situation involves the selection of an area which duplicates the conditions of the particular problem for which the permeability values are to be used. For example, values of k are often needed in problems involving drainage of excavations, reservoir storage losses due to seepage etc. An appropriate field test for such situations consists of pumping out water from a well and estimating the soil permeability from observations on the resulting drawdown surface. The pumping tests are no doubt expensive but are more reliable than the laboratory tests, since more of the soil macro-structure is incorporated in the *in situ* test.

Water is pumped out from the main well at a contant rate until the water levels in the observation wells (at least two in number) become steady or constant, indicating a steady state of flow. The flow quantity and the levels in the observation wells are noted. One of the two basic flow conditions will apply, namely *unconfined flow* or *confined flow.*

Unconfined flow pumping test: The test conditions are shown in Fig. 6.13. Here the aquifer is underlain by an impermeable stratum and the test well extends to the bottom of the permeable stratum and is perforated over the section which is below the water table. In its original state, it is assumed that the ground water is at rest. The pumping generates a radial flow of water towards the filter well and as a result, the water table assumes a curved surface called *drawdown water table*. The hydraulic gradient is also assumed to be constant at any given radius. Under steady-state conditions, the rate of radial flow through any cylindrical surface at radius r is constant and equal to the amount of water pumped out of the well in unit time.

Consider the flow through an elementary cylinder of soil having radius r, thickness dr and height h.
Hydraulic gradient (from outside to inside)

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

Area of flow,

$$A = 2\,\pi\,r\,h$$

From Darcy's law,

$$q = k\,i\,A$$

$$= k\frac{dh}{dr}\,2\,\pi r h$$

or

$$\frac{dr}{r} = 2\frac{\pi}{q}k\ \ h\,dh$$

Integrating,
$$\log_e \frac{r_2}{r_1} = \frac{\pi}{q} k (h_2^2 - h_1^2)$$

Thus,
$$k = 2.3 \frac{q}{\pi} \frac{\log_{10}(r_2/r_1)}{(h_2^2 - h_1^2)} \tag{6.26}$$

Though two observation wells suffice for computing k, higher accuracy will be obtained by observing water table in more than two boreholes. Values of h_1 and h_2 can be determined by measuring the drawdown at the two observation wells.

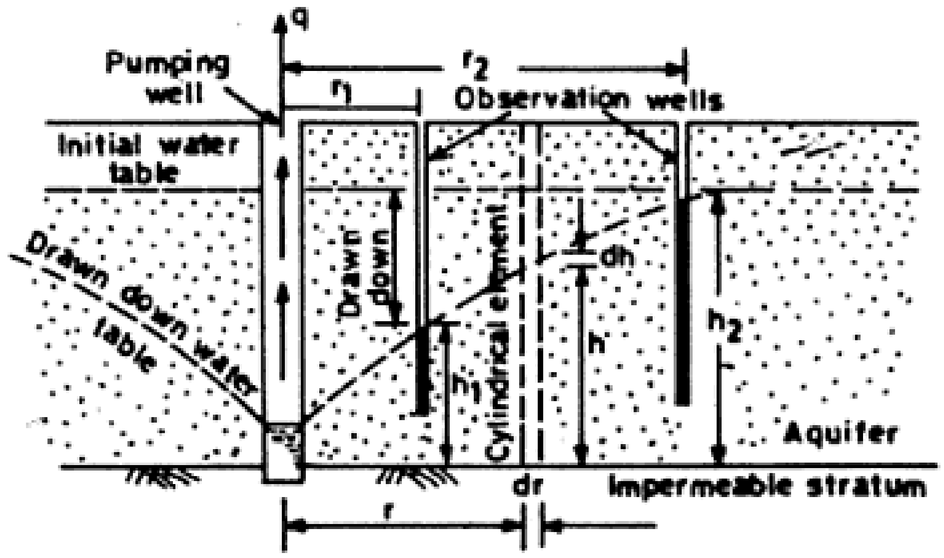


Fig. 6.13 Unconfined flow pumping test

Confined flow pumping test: A confined flow condition occurs when the aquifer is confined both above and below by impermeable strata (Fig. 6.14). Here, the drawdown surface is, for all values of r, above the upper surface of the aquifer.

Consider the flow after steady state is obtained (in some cases, pumping at a steady rate must be continued for many days before a steady flow is reached).

From Darcy's law,
$$q = k\,i\,A$$

Here,
$$A = 2\pi r D$$

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

or
$$\frac{dr}{r} = 2\frac{\pi}{q} Dkdh$$

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

Then,
$$k = 2.3 \frac{q}{2\pi D} \log_{10} \frac{(r_2/r_1)}{(h_2 - h_1)} \tag{6.27}$$

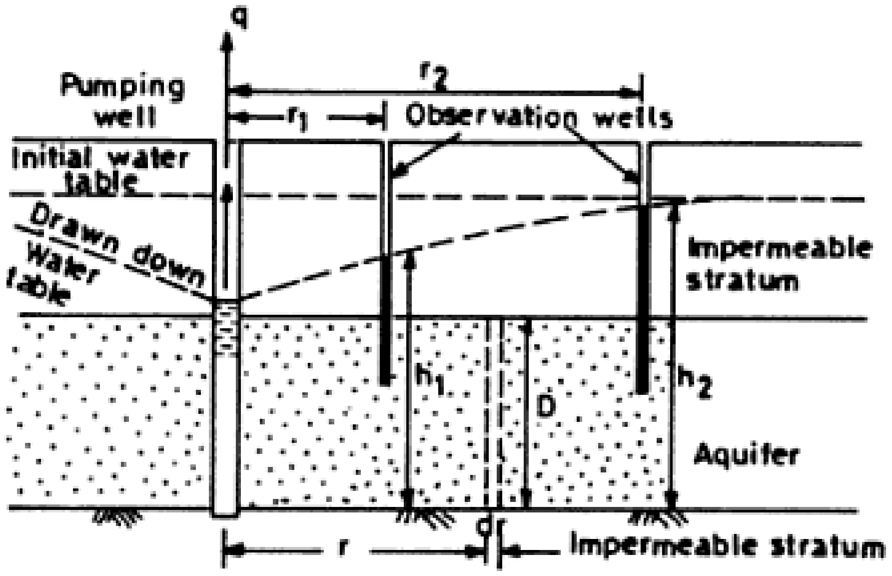


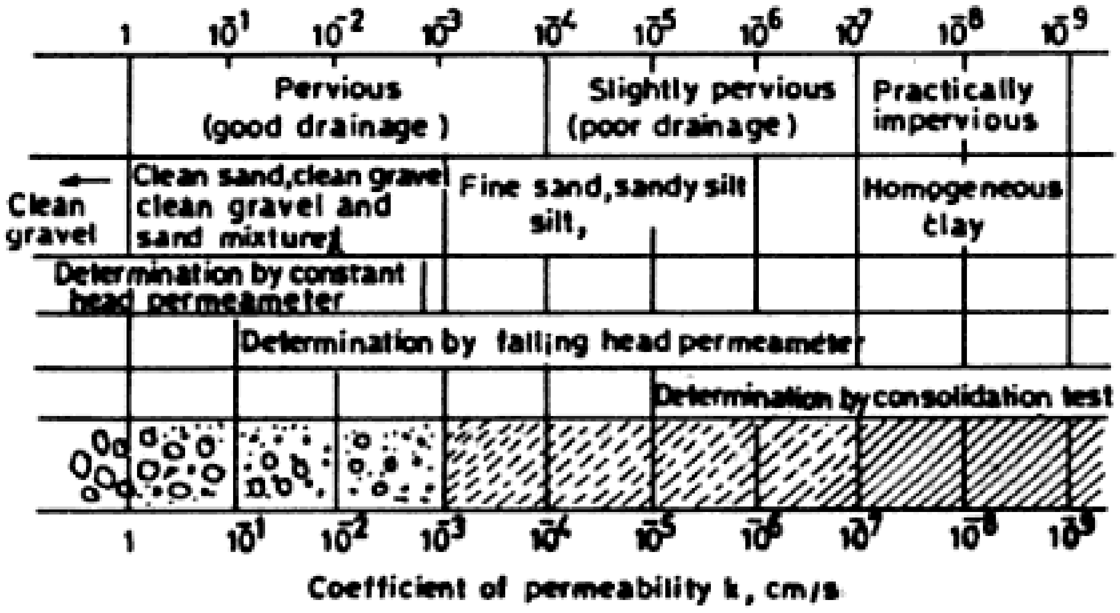
Fig. 6.14 Pumping test in a confined aquifer

Other Methods of Permeability Determination

For fine grained soils of very low permeability such as clay soils, the coefficient of permeability is determined by an indirect method—from the results of consolidation test (Chapter 9).

Useful empirical formulas and tables giving numerical values of k for different soils exist. Table 6.2 is one such Table, which provides some useful pointers. Certain boundary values of *k* are significant. For instance, *k* of 1.0 cm/s is the approximate boundary that separates clean gravels from clean sand and sandy gravels, *k* of 10^{-4} cm/s is the approximate boundary separating pervious soils and poorly-draining soils and *k* of 10^{-9} cm/s is, perhaps, the lower limit of permeability of soil, though values lower than 10^{-9} cm/s have been reported for some colloidal clays. It can be seen that the range of values for *k* is simply enormous. The permeability of a clean sand, for example, can be a million times the permeability of a plastic clay.

Table 6.2 Numerical Values of *k*, Soil Type and Methods of Permeability Determination



Factors Affecting Permeability

It has already been seen how the soil type can make such difference to the value of permeability. For example, clean gravel has a k value greater than 10° cm/s, sand between 10° and 10^{-3} cm/s, silt between 10^{-3} and 10^{-6} cm/s and clay has k values smaller than 10^{-6} cm/s. The variation in the value of permeability is so large that engineers are interested mostly in determining the power to which 10 must be raised while expressing the permeability value. For example, while writing Roorkee silt has a value of k equal to 3×10^{-4} cm/s, the power -4 is important. The number 3 can vary, it can be 2 or 4 or even 1 or 5 in the field on account of nonhomogeneity.

For a given soil, however, one can identify a few factors which influence the value of permeability.

Writing down the Poiseuille equation for the rate of flow through a tube of any geometrical cross-section and adapting it to the case of irregular cross-section somewhat similar to the flow passages between soil grains, we get the equation

$$q = d_e^2 \frac{r_w}{\eta} \frac{e^3}{1+e} CiA \tag{6.28}$$

Comparing this equation with Darcy's law gives

$$k = C\, d_e^2 \frac{\gamma_w}{\eta} \frac{e^3}{1+e} \tag{6.29}$$

where C is a composite shape factor dependent on grain shape and d_e is a representative grain size. The above equation is quite useful since it reflects the effect of factors that affect permeability. Some of the commonly used empirical formulae are being presented below.

Effect of grain size: The coefficient of permeability of a soil is proportional to the square of a representative particle size. One of the earliest and simplest empirical formulae was proposed by Allen Hazen (1911). For clean sands (with less than 5% fines) with D_{10} size between 0.1 and 3.0 mm,

$$k = CD_{10}^2 \tag{6.30}$$

where k has the unit cm/s. Here, D_{10} is in mm, constant C varies from 0.4 to 1.2 with an average value of 1. An approximate equation $k = 100\, D_{10}^2$ is sometimes used, when k and D_{10} are in cm/s and cm respectively. The equation holds good for $k \leq 10^{-3}$ cm/s.

Effect of the permeant—its viscosity and temperature: The factor $(\gamma_{w/\eta})$ depends on the kind and physical state of the pore fluid. Again from Eq. 6.29,

$$k \; \alpha \frac{\gamma_w}{\mu}$$

or

$$k_1 : k_2 = \frac{\gamma_{w_1}}{\eta_1} : \frac{\gamma_{w2}}{\eta_2} \tag{6.31}$$

Since both the viscosity and the unit weight of a permeant vary with temperature, k will be affected by changes in temperature. The variation in unit weight with change in temperature is much less important for soils where water is the common permeant. Viscosity effects are more important. Greater the viscosity, lower the permeability. It is common practice to note the temperature of water during permeability determination and reduce the computed permeability value to the value corresponding to 27°C (standard room temperature).

Let the hydraulic gradients in different layers be denoted by $i_1, i_2 \ldots i_n$ and the total head loss in flow over the total thickness H be denoted by h made up of head losses in each of the layers equal to $h_1, h_2 \ldots h_n$. The constant velocity of flow is

$$v = k_v \frac{h}{H} = k_1 i_1 = k_2 i_2 = k_n i_n$$

Loss of head in layer 1 is $h_1 = H_1 i_1 \because i_1 = \frac{h_1}{H_1}$ and similarly,

$$h_2 = H_2 i_2 \ldots\text{and } h_n = H_n i_n$$

The total head loss $h = H_1 i_1 + H_2 i_2 + \ldots + H_n i_n$

Combining the first and the last equations,

$$k_v \frac{h}{H} = k_1 i_1$$

$$k_v = \frac{H k_1 i_1}{H_1 i_1 + H_2 i_2 + \ldots + H_n i_n}$$

or

$$k_v = \frac{H}{\frac{H_1}{k_1} + \frac{H_2}{k_2} + \ldots + \frac{H_n}{k_n}} \tag{6.38}$$

For flow normal to the stratification,

$$h = h_1 + h_2 + \ldots + h_n \text{ and}$$

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

From Eqs. 6.37 and 6.38, it can be proved that for any stratified system, k_H will always be greater than k_v

6.6 TYPES OF HEAD, SEEPAGE FORCES AND QUICKSAND CONDITION

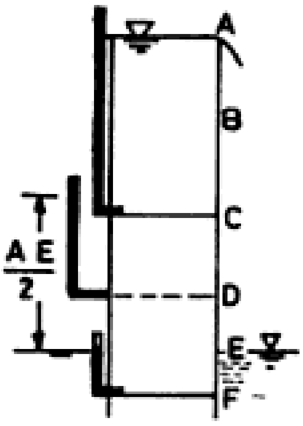


Fig. 6.16 Calculation of heads—Flow downwards

It has been mentioned earlier that of the three types of heads in fluid flow, namely, the velocity head $v^2/2g$, the pressure head p/γ_w and the elevation head z, the first one is negligible for seepage problems in soils. Henceforth, when we say total head, it means the sum of the pressure head and the elevation head only.

The elevation head at a point is the vertical distance of that point measured form an assumed datum plane which is normally taken at the tail water elevation, for convenience. The pressure head is the pressure of water at the point p divided by its unit weight γ_w. If a piezometer or an open standpipe is inserted at a point of flow, water would stand at a particular height inside the piezometer. If we measure the actual height of rise of water column in the piezometer, this represents the pressure head while the elevation of the water level referenced to the datum plane is its total head or the piezometer head. The difference in total heads at any two points in a soil through which flow is occurring represents the head loss during flow between these points. The hydraulic gradient is calculated with respect to the total head. To illustrate these concepts, Fig 6.16 is shown.

Table 6.3 Magnitudes of Head and Head Loss at Different Points

Point	*Pressure head*	*Elevation head*	*Total head*	*Head loss*
A	0	*AE*	*AE*	0
B	*AB*	*BE*	*AE*	0
C	*AC*	*CE*	*AE*	0
D	*CD*	*DE*	*CE* (=*AE*–*AE*/2)	*AE*/2
F	*EF*	– *EF*	0	*AE*

All the head loss is assumed to have occurred during flow through the soil sample. Thus, at the entry point *C*, the head loss is zero; at *F*, all the head *AE* is lost while at *D*, the midpoint of the soil sample, half of the head is lost (*AE*/2). For computing the magnitudes of heads at any point within the soil, it is necessary to first calculate the head loss upto that point in a proportionate manner (related to the length of flow upto that point as a fraction of the total length of flow through the soil). Thus, the total head at that point is estimated as the total head at entry point minus the head loss. The elevation head is reckoned from the position of the assumed datum. As a final step, the pressure head is calculated as the difference between the total head and the elevation head. Table 6.3 shows the magnitudes of heads at different points.

Hydrodynamic Case - Flow Condition

The stresses in a soil mass are computed as explained earlier (Section 6.1), provided there is no flow taking place through the soil. The condition is known as the *hydrostatic* condition. But when water flows through the soil, it exerts drag forces called seepage forces on the individual grains of the soil. The presence of the seepage forces, which act in the direction of flow, will cause changes in the pore water pressures and effective stresses in the soil. These effects can be studied with the help of illustration shown in Figs. 6.17, 6.18 and 6.19.

'No flow' Condition (Hydrostatic case)

Figure 6.17 shows a tank filled with submerged soil, with no seepage occurring, because the valve at the base is closed. Hence, the water levels in the standpipes inserted at the top, bottom and any intermediate position of the soil layer are the same. The variation of the total stress σ, pore water pressure u and effective stress $\bar{\sigma}$ with depth for the 'no flow' condition are shown in Fig. 6.17.

Downward Flow

The condition of downward seepage is shown in Fig. 6.18. The valve below the tank is opened. A constant level of water in the soil tank is maintained by adjusting the supply from the top and the outflow at the bottom. The base of the tank can be taken as the datum.

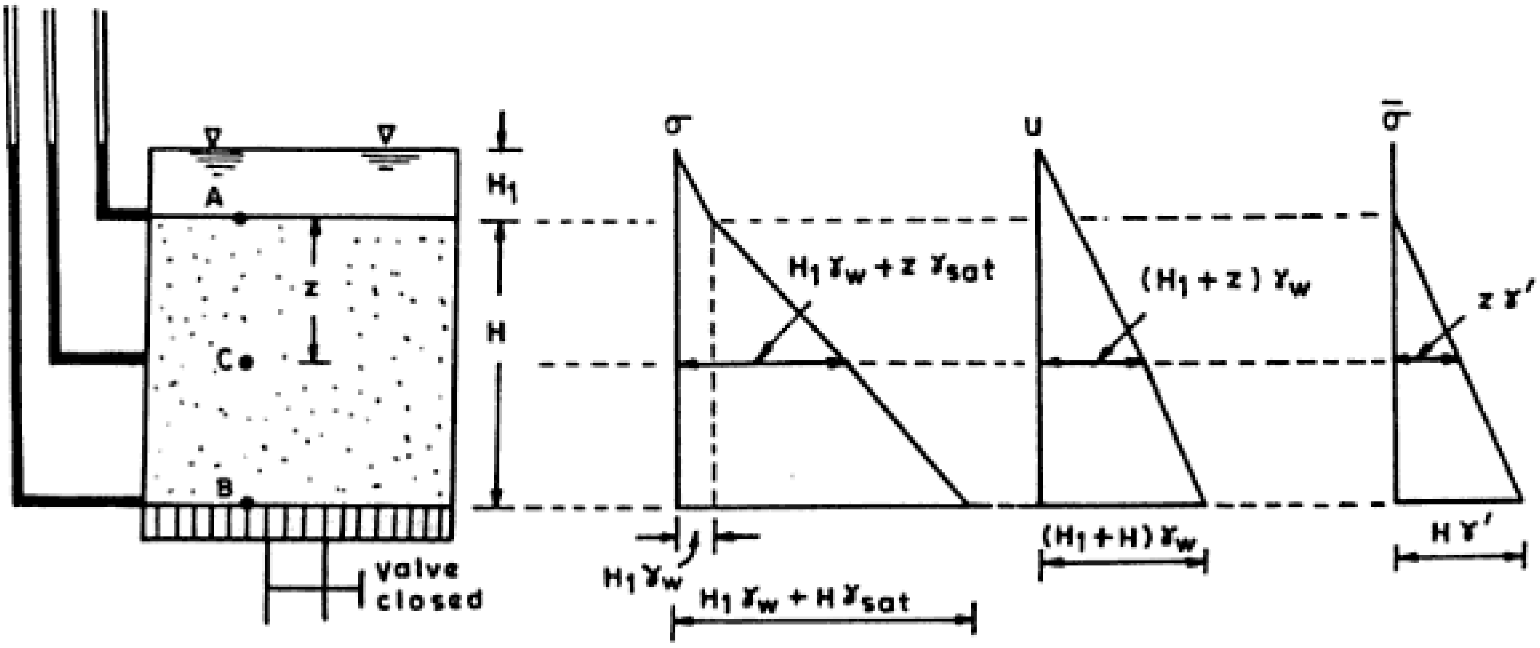


Fig. 6.17 Variation of σ, *u* and σ̄ with depth for 'no flow condition

Total head at A = Pressure head$_A$ + Elevation head$_A$ = $H_1 + H$

Total head at B = Pressure head$_B$ + Elevation head$_B$ = $(H_1 + H - h) + 0$

Since the total head at A is greater than the total head at B, flow shall take place in the downward direction under a hydraulic gradient $i = \frac{\text{head loss}}{\text{length of flow}} = \frac{h}{H}$. The difference in elevation of water level in the stand pipes at the planes of A and B is h.

The total stress at any point in the soil mass is solely due to the weight of soil and water above it. Hence, there is no change in the distribution of σ with depth when compared to the 'no flow' condition. The stress calculations are shown below:

At A: total stress $\sigma_A = H_1\,\gamma_w$

pore water pressure $u_A = H_1\,\gamma_w$

effective stress $\bar{\sigma}_A = \sigma_A - u_A = 0$

At B: $\sigma_B = H_1\,\gamma_w + H\,\gamma_{sat}$

$u_B = (H_1 + H - h)\,\gamma_w$

$$\bar{\sigma}_B = \sigma_B - u_B = H\,(\gamma\,\text{sat} - \gamma_w) + h\,\gamma_w = H\,\gamma' + h\,\gamma_w \tag{6.39}$$

The σ, u, σ̄ diagrams are shown in Fig. 6.18.

A comparison of the effective stress $\bar{\sigma}_B$ in Eq. 6.39. with the value of $\bar{\sigma}_B$ for the 'no flow' (hydrostatic) condition shows that the effective stress is increased by $h\,\gamma_w$ or $\frac{h}{H} H\gamma_w = i\,H\,\gamma_w$.

At *B:*

$$\sigma_B = H_1 \gamma_w + H\gamma_{sat}$$

$$u_B = (H_1 + H + h)\gamma_w$$

$$\bar{\sigma}_B = H(\gamma_{sat} - \gamma_w) - h\gamma_w = H\gamma' - i\,H\gamma_w \tag{6.41}$$

The σ, u $\bar{\sigma}$ plots are shown in Fig. 6.19.

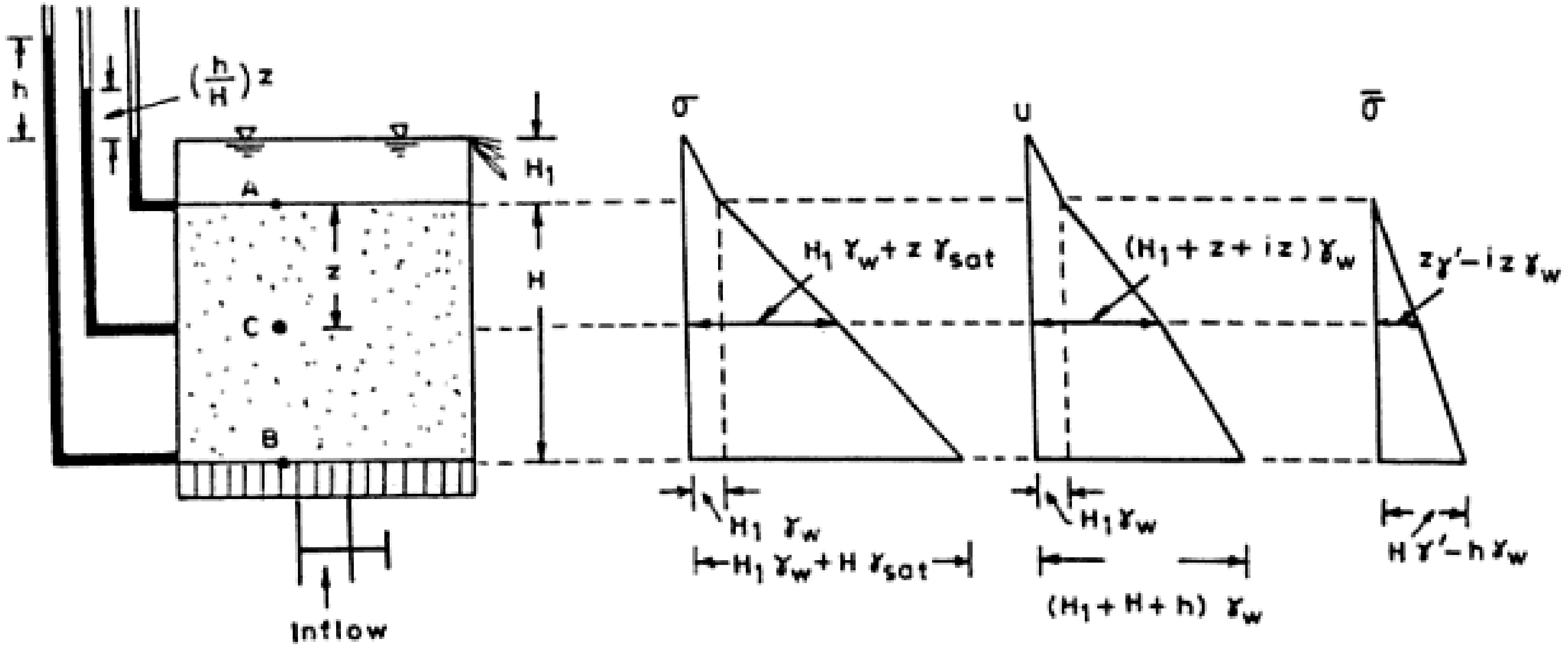


Fig. 6.19 Variation of σ, u and $\bar{\sigma}$ with depth for upward flow

An upward seepage, therefore, produces an effect which is contrary to that of downward seepage. It reduces the effective stress at the level of point *B* by $i\,H\,\gamma_w$. The seepage pressure acts in the direction of flow, which is upward in this case.

Quick Sand Condition

It is clear from Eq. 6.41 that by increasing the total head difference *h*, it is possible to reach a condition when the effective stresses in the soil become equal to zero; that is,

$$H\gamma' - i\,H\gamma_w = 0$$

This condition occurs when the hydraulic gradient

$$i = i_{cr} = \frac{\gamma'}{\gamma_w} \tag{6.42}$$

i_{cr} is called the *critical hydraulic gradient.* When upward flow takes place at the critical hydraulic gradient, a soil such as sand loses all its shearing strength and it cannot support any load. The soil is said to have become 'quick' or 'alive' and *boiling* will occur. The popular name for this phenomenon is *quicksand.* Quicksand is not a type of sand but only a hydraulic condition. What has happened under this condition is that the seepage pressure $iH\gamma_w$ has become equal to the effective pressure $H\gamma'$ so that the effective stress throughout the soil is reduced to zero. As long as $i < i_{cr}$, only a part of the total head difference h is used up in viscous friction, with the seepage pressure reducing the net effective stress but not quite completely nullifying it.

From Eq. 6.42, substituting for γ' from Eq. 2.22,

$$i_{cr} = \frac{G-1}{1+e} \tag{6.43}$$

The specific gravity of sand particles does not vary much; so if the void ratio of the natural deposit is known, i_{cr} can be computed from Eq. 6.43. Assuming $G = 2.65$, i_{cr} will vary from 1.1 for $e = 0.5$ (dense sand) to 0.83 for $e = 1.0$ (loose sand). For the usual range of void ratios in sand soils of 0.6 to 0.7, the critical gradient will be just about 1, an easy figure to remember.

Why does 'boiling' occur mostly in fine sands or silts? Seepage forces affect sands more than clays because sands do not possess cohesion, while fine silts and clays have some inherent cohesion which holds the grains together even at the critical hydraulic gradient. In sands, the shear strength τ_f is given by the equation, $\tau_f = \bar{\sigma} \tan \phi$ where ϕ is the angle of shearing resistance. Hence when $\bar{\sigma} = 0$, $\tau_f = 0$. In clays, however, $\tau_f = c + \bar{\sigma} \tan \phi$ where c is the cohesion. The cohesion component of shear strength is independent of $\bar{\sigma}$. Boiling does not occur in coarse sands and gravels either, because these soils are highly pervious; hence as per Darcy's law, large discharges are required to produce a critical gradient of unity and such flows rarely materialise in practice.

When a natural soil deposit becomes 'quick', it cannot support the weight of a person or an animal. But the common belief that human beings or beasts are sucked into a quicksand does not stand scrutiny. Quicksand is a viscous liquid with a unit weight of about twice that of water. Hence, any person can easily float in it. material being viscous, movement in it is, however, The very tiring. In trying to make desperate movements in panic, one may tire oneself and die by suffocation. But if one were to hold the head clear and make slow movements, one can grasp something and pull oneself out of the affected zone.

A second approach to obtain an expression for i_{cr} is to equate the total boundary pore water pressure and the total weight of all the material above that boundary. From Fig. 6.19, considering an area of cross section A of a container in which the soil and water are assumed to be placed,

Upward force (due to water) = $(h + H_1 + H)\gamma_w A$

Total downward force (due to soil + water) = $\gamma_{sat} HA + \gamma_w H_1 A$

For quick condition, $(h+H_1 + H)\gamma_w A = \gamma_{sat} HA + \gamma_w H_1 A$ (6.44)

Substituting for γ_{sat} and rearranging,

$$\frac{h}{H} = i_{cr} = \frac{G-1}{1+e}$$

Thus, the effective stress approach used earlier and the total approach used above, give the same results.

In practice, boiling condition may occur when excavations are made below water table and water is pumped out from the excavation pit to keep the area free from water. It can be prevented by lowering the water table at the site before excavation or alternatively, by increasing the length of upward flow. Boiling condition is also common when a pervious sand stratum underlying a clay soil is in an artesian pressure condition.