CLASSFICATION FOR ENGINEERING PURPOSES:

The purpose of soil classification is to arrange various types of soils into groups according to their engineering properties or other characteristics. Soil possessing similar characteristics can be placed in the same group. However, from engineering point of view, the classification may be done with the objective of finding suitability of the soil for the construction of dams, highways or foundations etc.

For general engineering purposes, soils may be classified by the following systems:

- Particle size classification
- Textural classification
- Highway research board (HRB) classification
- Unified soil classification
- Indian standard (Bureau of Indian standard) classification (BIS)

FEATURES / REQUIREMENTS OF SOIL CLASSIFICATION :

- It should have a limited number of groups.
- It should be based on the engineering properties which are most relevant for the purpose forwhich the classification has been made.
- It should be simple and should use the terms which are easily understood.
- It should be acceptable to all engineers.
- It should have accuracy in indicating the performance of a soil under certain field conditions.

In this system, soils are arranged according to the grain size. Terms such as gravel, sand, silt, and clay are used to indicate grain sizes. It is preferable to use the word 'silt size' and 'clay size' in place of simply 'silt' or 'clay' in this system. There are various grain size classification in use, but the more commonly used systems are,

i) U.S.Bureau of soil and public road administration (PRA) system ofunited states.
ii) International soil classification

0.005mm 0.05 0.10 0.25 0.75 1.0 2.0mm

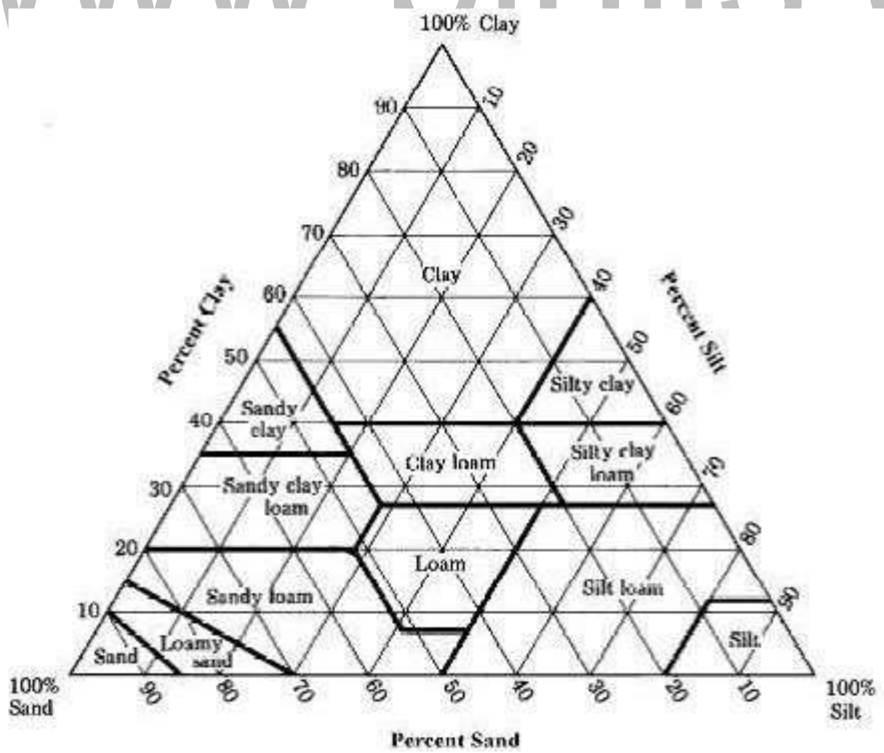
CLAY	SILT (size)	VERY FINE	FINE	MEDIUM	COARSE	FINE GRAVEL	GRAVEL

TEXTURAL CLASSIFICATION :

Soils occurring in nature are composed of different percentage of sand, silt and clay size particles. Soil classification of composite soils exclusively based on the particle size distribution is known as textural classification probably the best known of these textual classification is the triangular classification. Such a classification is more suitable for coarse grained soils than clay soils.

To use the chart, for the given percentage of the three constituents forming a soil lines are drawn parallel to the three sides of the triangle.

TEXTURAL CLASSIFICATION CHART :



HIGHWAY RESEARCH BOARD (HRB) CLASSIFICATION :

The HRB classification system also known as PRA classification system, is based on both the particle size composition as well as the plasticity characteristics. The system is mostly used for pavement construction soils are divided into 7 groups designated as A-1, A-2…… A-7 . A group index is used to describe the performance of the soils when used for pavement construction. The higher the value of the index, the poorer is the quality of the mentioned. The group index of a soil depends upon i) The amount of material passing the 75µ IS sieve. ii) The liquid limit (W_L) and

iii) The plastic limit (W_p)

UNIFIED SOIL CLASSIFICATION :

The system is based on both grain size and plasticity properties of the soil and is therefore applicable to any use. Soils are broadly divided into three divisions such on coarse soil the grainedsoil and highly organic soils.

IS (or) BIS (BUREAU OF INDIAN STANDARD CLASSIFICATION):

Soils are broadly divided into three groups.

- Coarse grained soils
- Fine grained soils
- Highly organic soils

Coarse grained soils:

In these soils, more than half the total material by mass is larger than 75µ IS sieve size.

Fine grained soils:

In these soils, more than half the total material by mass is smaller than 75µ IS sieve size.

Highly organic soils:

These soils certain large percentages of fibrous organic matter, such as peat and the particle ofdecomposed vegetation.

COARSE GRAINED SOILS :

Coarse grained soils are further divided into two sub divisions.

a) Gravel (G) : In these soils, more than half the course fraction is larger than 4.75 mm Is sievesize. This sub division includes gravels and gravelly soil and is designated by symbol G.
b) Sands (S): In these soils, more than half the course fraction is smaller than 4.75 mm Is sievesize. Each sub division is divided into four groups.
W : Well graded

P : Poorly graded

C : Well graded with excellent clay binder

M : containing fine materials

FINE GRAINED SOILS :

Fine grained soils are further divided into three sub-divisions.

a. Inorganic silts and very fine sands : M
b. Inorganic clays : C
c. Organic silts and clays and organic matter : O

Fine grained soils are further divided into following groups:

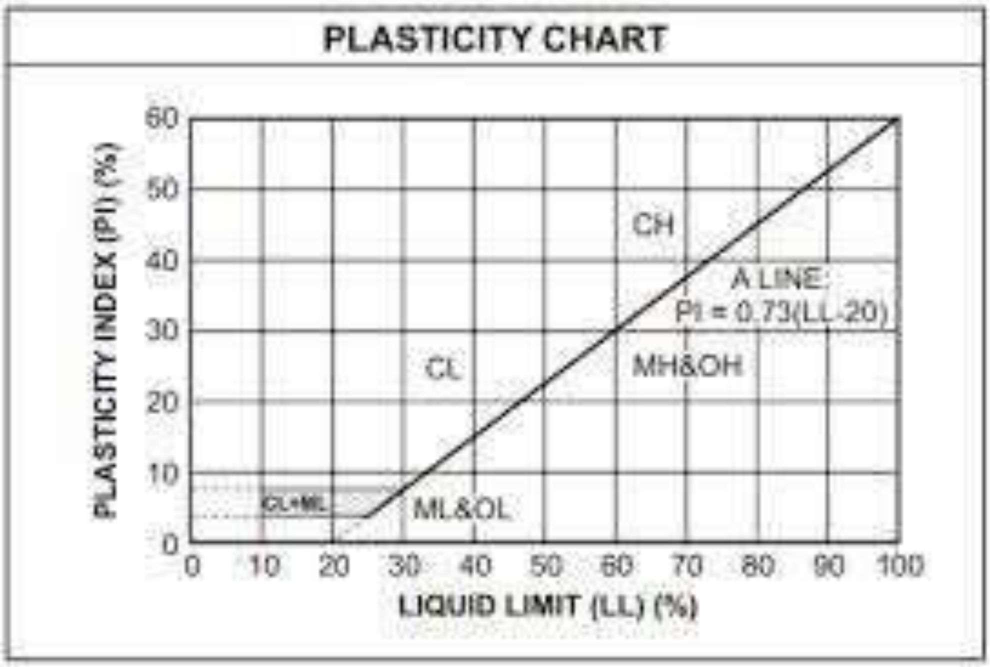
i) Silts and clays of low compressibilityLL < 35, represented by ‘L’
ii) Silts and clays of medium compressibilityLL < 35 | < 50, represented by ‘I’
iii) Silts and clays of high compressibilityLL > 50, represented by ‘H’

Example :

ML – Inorganic silt with low to medium compressibility laboratory classification of fine grained soils is done with the help of plasticity chart shown below. The A –line dividing in organic clay from silt and organic soil has the following equation.

$$I_P = 0.73 [W_L-20]$$

PLASTICITY CHART (IS SOIL CLASSIFICATION SYSTEM) :.



IS CLASSIFICATION:

TABLE 4.6. BASIC SOIL COMPONENTS (IS CLASSIFICATION) (IS : 1498-1970)

Soil	*Soil Component*	*Symbol*	*Particle size range and description*
Coarse-grained components	Boulder	None	Round to angular, bulky hard, rock particle. average diameter more than 30 cm
	Cobble	None	Round to angular, bulky hard, rock particle, average diameter smaller than 30 cm but retained on 80 mm sieve
	Gravel	G	Rounded to angular, bulk, hard, rock particle, passing 80 mm sieve but retained on 4.75 mm sieve *Coarse* : 80 mm to 20 mm sieve *Fine* : 20 mm to 4.75 mm sieve
	Sand	S	Rounded to angular bulky, hard, rocky particle, passing 4.75 mm sieve retained on 75micron sieve *Coarse* : 4.75 mm to 2.0 mm sieve *Medium* : 2.0 mm to 425 micron sieve *Fine* : 425 micron to 75 micron sieve
Fine-grained Components	Silt	M	Particles smaller than 75-micron sieve indentified by behaviour, that it is slightly plastic or non-plastic regardless of moisture and exhibits little or no strength when air dried
	Clay	C	Particles smaller than 75-micron sieve identified by behaviour, that is, it can be made to exhibit plastic properties within a certain range of moisture and exhibits considera ble strength when air dried
	Organic matter	O	Organic matter in various sizes and stages of decomposition.

TABLE 4.8. IS SOIL CLASSIFICATION : LABORATORY CLASSIFICATION CRITERIA FOR COARSE GRAINED SOILS (IS : 1498 –1970)

<table>
<tr><th>Groups Symbols</th><th colspan="3">Laboratory Classification Criteria</th></tr>
<tr><td>GW</td><td colspan="2">C_U Greater than 4
C_C Between 1 and 3</td><td rowspan="8">Determine percentages of gravel and sand from grain sizes curve depending on percentage of fines (fraction smaller than No. 75 micron sieve size) ; coarse-grained soils are classified as follows :

Less than 5% : GW, GP, SW, SP

More than 12% : GM, GC, SM, SC

5% to 12% : Border line cases requiring use of dual symbols

$C_U = \frac{D_{60}}{D_{10}}$ (uniformity coefficient)

$C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ (coefficient of curvature)</td></tr>
<tr><td>GP</td><td colspan="2">Not meeting all gradation requirements for GW</td></tr>
<tr><td>GM</td><td>Atterberg limits below "A" line or I_P less than 4</td><td rowspan="2">Above "A" line with I_P between 4 and 7 are border line cases requiring use of dual symbols.</td></tr>
<tr><td>GC</td><td>Atterberg limit above "A" line with I_P greater than 7</td></tr>
<tr><td>SW</td><td colspan="2">C_U Greater than 6
C_C Between 1 and 3</td></tr>
<tr><td>SP</td><td colspan="2">Not meeting all gradation requirements for SW</td></tr>
<tr><td>SM</td><td>Atterberg limits below "A" line or I_P less than 4</td><td rowspan="2">Limits plotting above "A" line with I_P between 4 and 7 are border line cases requiring use of dual symbols</td></tr>
<tr><td>SC</td><td>Atterberg limit above "A" line with I_P greater than 7</td></tr>
</table>

Procedure for IS soil Classification:

Step :1 check whether it is coarse grained soil or fine grained soil

- coarse grained soil – more than 50% of soil retained above 75 µ
- fine grained soil - less than 50% of soil retained above 75 µ

Step :2 Case 1: If it is coarse grained soil

a) check whether it is gravel or sand

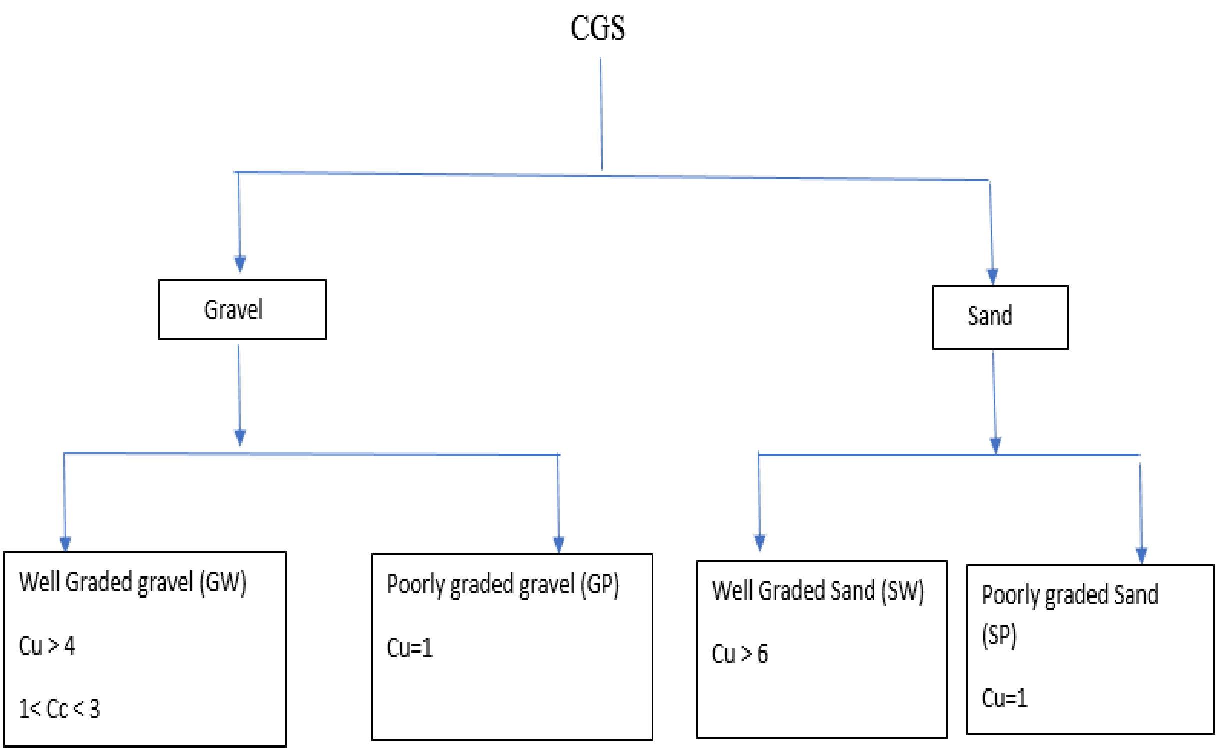
- Gravel – more than 50% of soil retained in 4.75mm IS sieve
- Sand - more than 50% of soil retained in between 4.75mm to 75 µ IS sieve

b) Check the percentage of finer

i) finer <5%

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

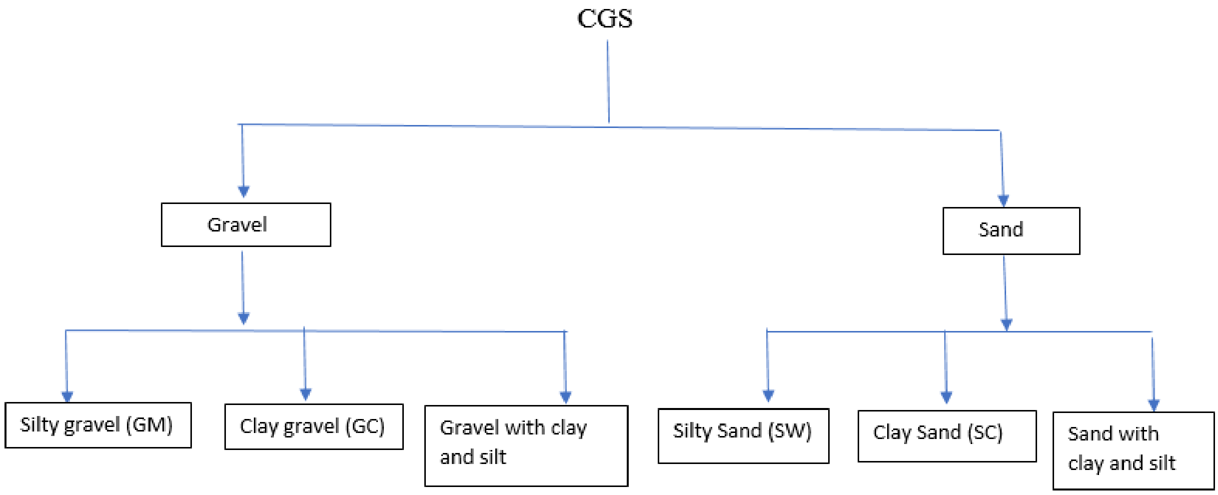
$$C_c = \frac{D_{30}^2}{D_{60} X D_{10}}$$



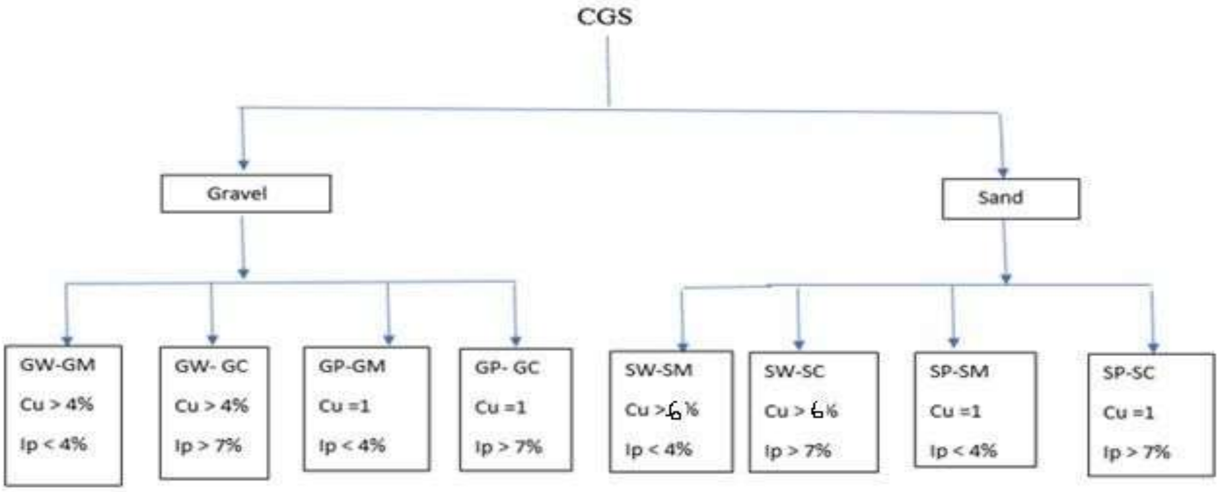
ii) finer >12%

Ip < 4% - Silt

Ip>7% - Clay

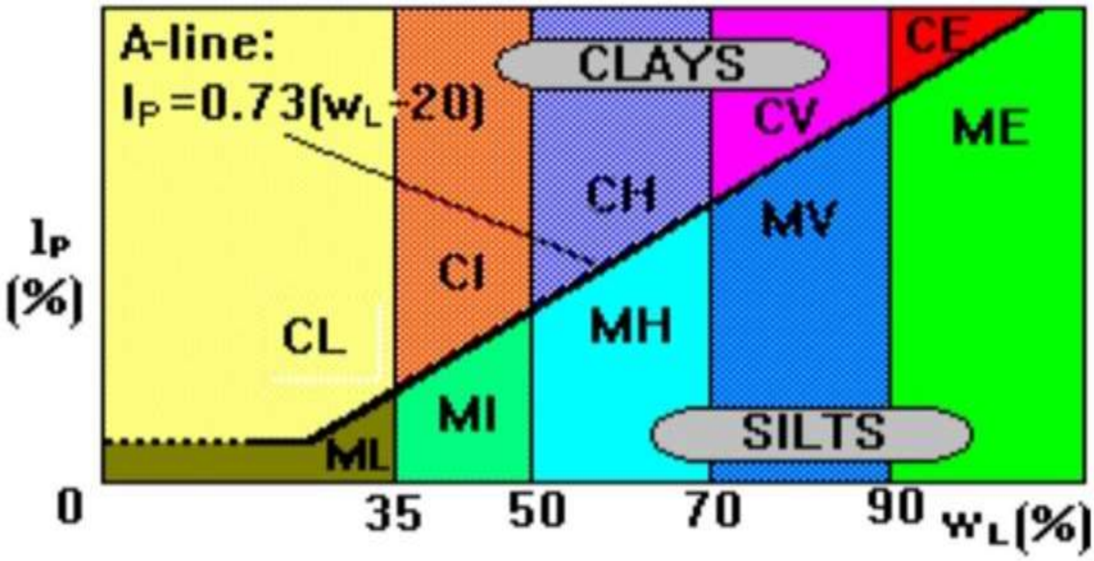


iv) finer 5-12%



Case b)If it is fine grained Soil

- Clay(c) – above A line
- Silt(M) or Organic(o) – Below A line



$I_p > I_{pc}$ ------ clay

$I_p < I_{pc}$ ------ silt or organic

0-35----- low plasticity

35-50 ----medium plasticity

>50 ----- high plasticity

1) The following results are obtained from a soil sample. Percentage passing 4.75mm sieve=70, percentage passing 75-micron sieve =10, uniformity coefficient=8 plasticity index 3.5%. Classify the soil as per IS soil classification

100%

Coarse grained soil(CGS)	=100-(70+10)=20 (4.75mm IS sieve)	Gravel(G)
90%	70 (75micron IS sieve)	Sand(S)
Fine grained soil(FGS) 10%	10	% of finer

Step :1 check whether it is coarse grained soil or fine grained soil

coarse grained soil –here more than 50% of soil retained above 75 µ

Step :2 Case 1: If it is coarse grained soil

a) check whether it is gravel or sand

Sand - more than 50% of soil retained in between 4.75mm to 75 µ IS sieve

Step3)Check the percentage of finer

Here percentage of finer is between 5-12

Given C_u=8 and Ip=3.5

Refer the flow chart

Therefore, it is SW-SM type

That is well graded sand and silty Sand

2.The following results are obtained from a soil sample. Percentage passing 4.75mm sieve=35, percentage passing 75 micron sieve =8, size corresponding to 10% finer =0.8mm; 30% finer =3mm; 60% finer =6mm;liquid limit=25%;plastic limit=17%. Classify the soil as per IS soil classification

100%

Coarse grained soil(CGS) 92%	=100-(35+8)=57 (4.75mm IS sieve)	Gravel(G)
	35 (75micron IS sieve)	Sand(S)
Fine grained soil(FGS) 8%	8	% of finer

Step :1 check whether it is coarse grained soil or fine grained soil

- coarse grained soil – Here more than 50% of soil retained above 75 μ

Step :2 Case 1: If it is coarse grained soil

a) check whether it is gravel or sand

- Gravel – more than 50% of soil retained in 4.75mm IS sieve IS sieve

Step3)Check the percentage of finer

Here percentage of finer is between 5-12

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

$$= \frac{6}{0.8} = 7.5$$

Ip=Liquid limit – Plastic limit

=25-17=8

Refer the flow chart

Therefore, it is GW-GC type

That is well graded gravel and claygravel

3. The following results are obtained from a soil sample. Percentage passing 4.75mm sieve=20, percentage passing 75 micron sieve =70, size corresponding to 10% finer =0.5mm; 30% finer =2.5mm ; 60% finer =6mm;liquid limit=30%;plastic limit=20%. Classify the soil as per IS soil classification

100%

Coarse grained soil(CGS)	=100-(20+70)=10 (4.75mm IS sieve)	Gravel(G)
30%	20 (75micron IS sieve)	Sand(S)
Fine grained soil(FGS)	70	% of finer
7s0%		

Step :1 check whether it is coarse grained soil or fine grained soil

fine grained soil –Here less than 50% of soil retained above 75 µ

Step :2 Case 2: If it is Fine grained soil

$I_p = W_L - W_P$

$= 30 - 20 = 10$

Refer the graph

Therefore, it is ML

That is Low silt

CLAY MINERALOGY

Clay mineralogy is the science dealing with the structure of clay minerals on microscopic, molecular, and atomic scale. It also includes the study of the mineralogical composition and electrical properties of the clay particles. The behavior of fine grained soils, on the other hand, depends to a large extent on the nature and characteristics of the minerals present. The most significant properties of clay depend upon the type of mineral.

CLAY MINERALS

To understand the various unique engineering behavior of clay, it is most beneficial to study microstructures of clay particles first.

There are three types of clay minerals:

- Kaolinite clay
- Montomorillonite clay
- Illite clay

All of these clay minerals have two basic atomic sheets

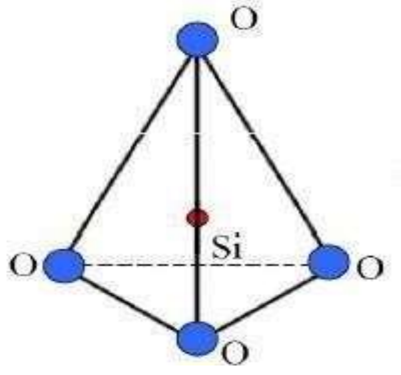
1. **Silica tetrahedral sheet**
2. **Aluminum octahedron sheet**

1.Silica tetrahedral sheet

In *silica tetrahedral sheet*, silica (Si) occupies the center positions and oxygen ions (O) are strongly bonded to the core atoms. Silica tetrahedral sheet is symbolized with a trapezoid, of which the shorter face holds electrically unsatisfied oxygen atoms and the longer face holds electrically satisfied oxygen atoms.

2.Aluminum octahedron sheet

In aluminum octahedron sheet, aluminum (Al) ion positioned at the center and hydroxyl ion (OH^-) bonded to the core atoms. Aluminum octahedron sheet is symbolized with a rectangle with top and bottom faces having the same characteristics of exposed hydroxyl ions. Figure 3.1 gives the atomic structure of the above two basic sheets.



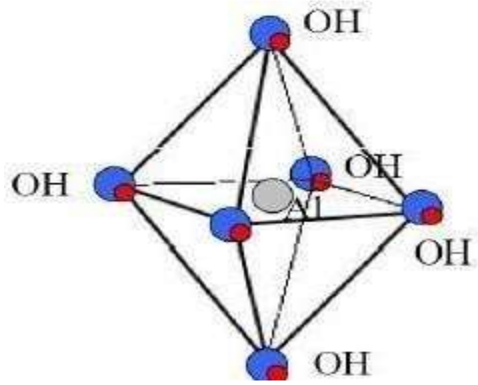


Figure 3.1: Silica tetrahedral and aluminum octahedron

KAOLINITE CLAY

The basic unit of this type of clay is formed by atomic bond of the unsatisfied face of silica sheet and either face of aluminum sheet as seen in Figure 3.2 . The bond between two sheets is strong and is primary bond. However, the stack of two sheets is not a form of clay yet. Many layers of basic kaolinite unit make a kaolinite clay. The thickness of one unit is about 7.2 angstrom. SEM (Scanning Electron Microscope) image of kaolinite clay is illustrated in Figure 3.3.

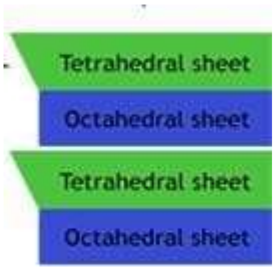


Figure 3.2: Kaolinite clay

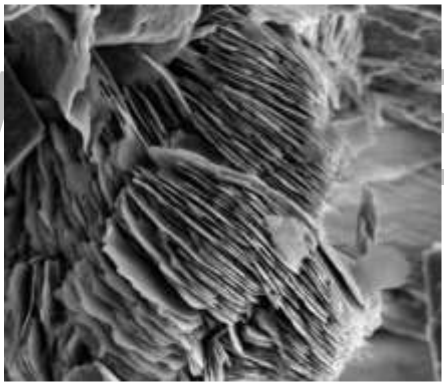


Figure 3.3: SEM image of Kaolinite clay

MONTMORILLONITE CLAY

The unused OH^- face of aluminum sheet of the silica and aluminum sheet unit in the Kaolinite clay structure may attract the unsatisfied face of another silica sheet to make a three layer stack and shown in Figure 3.4 and SEM image is illustrated in Figure

. This makes the basic unit of Montmorillonite clay structure with the thickness of about 10 angstrom. The link is due to natural attraction for the cations in the intervening space and due to Vander Waal forces. The negatively charged surfaces of the silica sheet attract water in the space between two structural units. This results in an expansion of the mineral. The soil containing a large amount of the mineral montmorillonite exhibits high shrinkage and swelling characteristics.

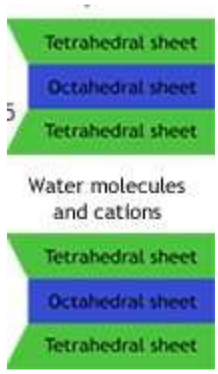


Figure 3.4: Montmorillonite clay

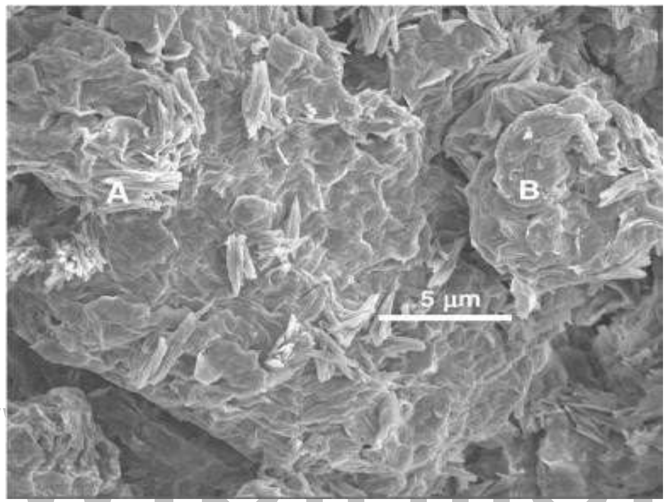


Figure 3.5: SEM image of Montmorillonite clay

ILLITE CLAY

Basic structure of this clay is the same as the one of montmorillonite. However, potassium ion (K^+) are filled in between facing O^{2-} and O^{2-}surfaces of silica sheets as seen in Figure 3.6. The characteristics of this clay are classified as in between those of kaolinite and montmorillonite. SEM image is shown in Figure 3.7.

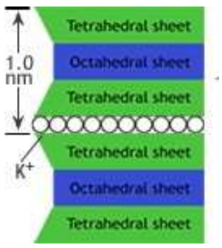


Figure 3.6: Illite clay

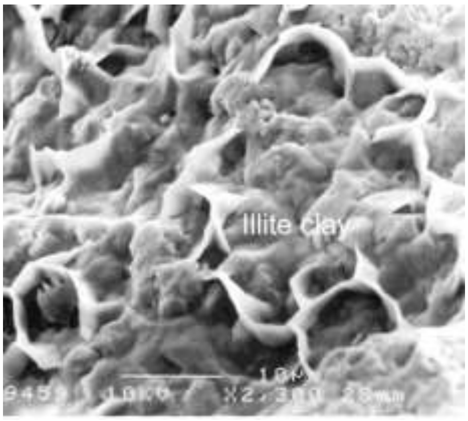


Figure 3.7: SEM image of Illite clay

CLAY SHAPES AND SURFACE AREAS

Clays are formed in stack of several layers of basic sheet units. These are generally flat and smaller in size, and thus, their surface areas per weight are very large. Table 3.1 compares the types of clay with their general shapes, general dimensions, and surface areas. The specific surface is defined as the surface area of clay per 1 gm of dry clay particles.

Table 3.1: Comparison of shape and surface areas of clay particles

Clay type	Typical length (µm)	Typical thickness (µm)	Typical dimensional ratio (L x L x T)	Specific surface (m^2/g)
Kaolinite	0.3-3	0.05-1	10 x 10 x 1	10-20
Montmorillonite	0.1-1.0	0.001-0.01	100 x 100 x 1	800
Illite	0.1-2	0.01-0.2	20 x 20 x 1	80-100

CLAY WATER SYSTEM

In the natural environment, clays are often formed under water. In water, positively charged ions (cations) and positive edges of dipoles (water molecules) are attracted to the clay surface. Several layers of water molecules are attracted on the clay surface in a very ordinary way. The water layer with thickness of 10 angstrom is called adsorbed water layer. The layer is very rigidly, electrically attracted to the clay surface and behaves as if a part of the particle itself.

In the outer part of water, there are distributions of mobile cations and anions. Those cations and anions are from resolved minerals and other matters in natural water. Cations further attract dipoles around them. This type of water is called electrostricted water, and they move together when a cation moves. The rest of the space is filled with regular water, which is called free water. There is a boundary within which a clay particle has an influence electricity. The layer extending from the clay particle surface to the limit of attraction is known as Diffuse Double Layer. The above description is outlined in Figure 3.8.

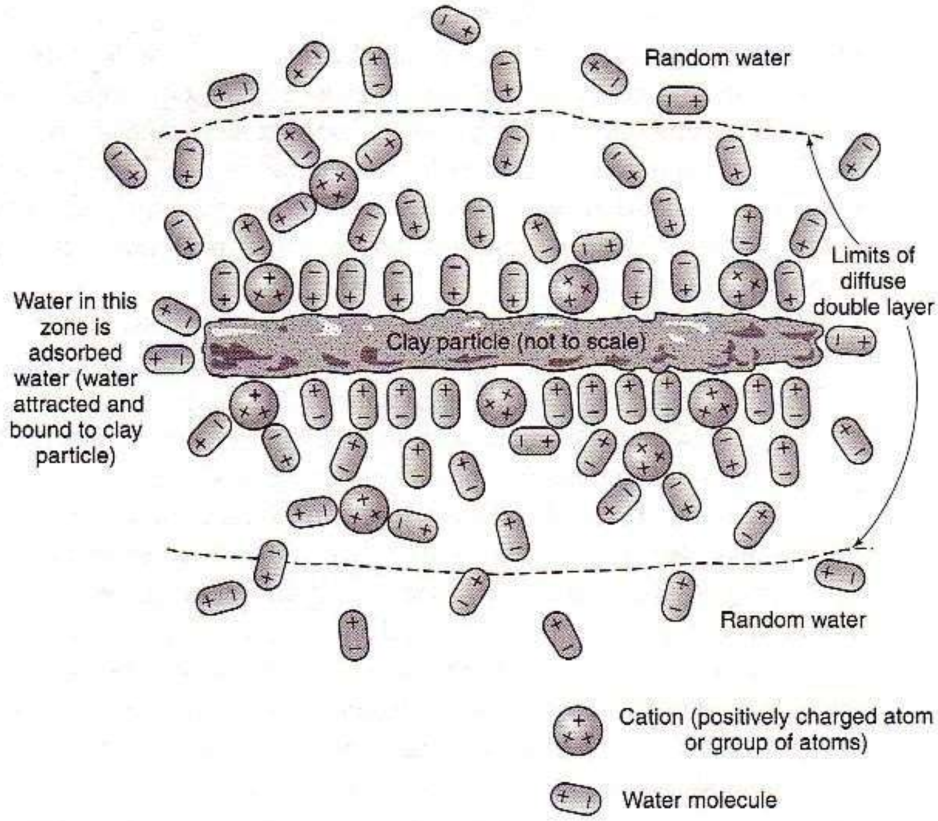


Figure 3.8: Clay water system

SOIL STRUCTURE

The geometrical arrangement of soil particles with respect to one another is known as soil structure. The soils in nature have different structures depending upon the particle size and the mode of formation.

Soil Structure for Coarse Grained Soil

Single grained structure

Cohesion less soils, such as gravel and sand, are composed of bulky grains in which the gravitational forces are more predominant than surface forces. When deposition of these soil occurs, the particles settle under gravitational forces and take an equilibrium position as shown in Figure 3.9. Each particle is in contact with those surrounding it. The arrangement is somewhat similar to the stacking of oranges on a grocer's counter. Depending upon the relative position of soil particles, the soil may be a loose structure or dense structure.

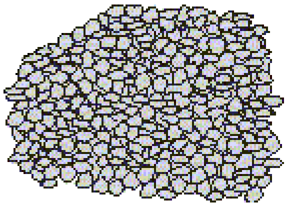
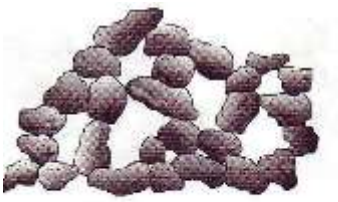


Figure 3.9: Single grained structure

Honey comb structure

It is possible for fine sands or silts to be deposited such that the particles when setting develop a particle-to-particle contact that bridges over large voids in the soil mass (Figure 3.10). The honeycomb structure usually develops when the particle size is between 0.002 mm and 0.02 mm.

Figure 3.10: Honey comb structure



Soil Structure for Clay

Dispersed clay

The final structures of clay are established from the balance of interactive forces and external forces applied to the clay assemblage. If the final inter particle forces are repulsive, the particles want to separate from each other when the boundary confinements are removed. This is a situation of dispersed clay. The soils in dispersed structure generally have a low shear strength, high compressibility, and low permeability.

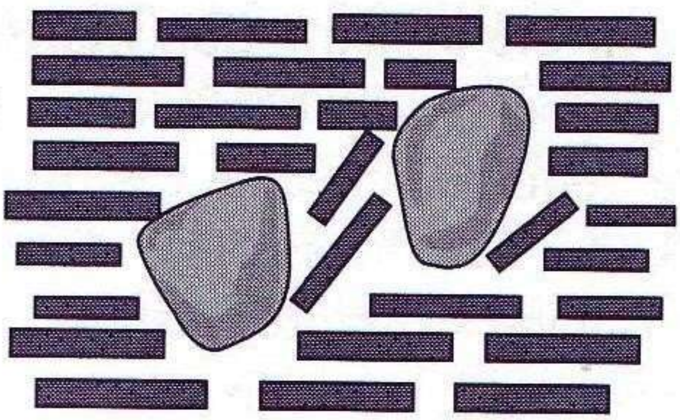


Figure 3.11: Dispersed clay

Flocculated structures

If the inter particle force are attractive, then particles want to come together, making flocculated clay. If the edge charges are positive, most likely, the edges are attracted to the flat surface of other clay particles. This makes a card house structure of flocculated clay, most commonly in salt water environment. In freshwater environments, more face-to-face flocculated structures are formed due to negative charges at the edges.

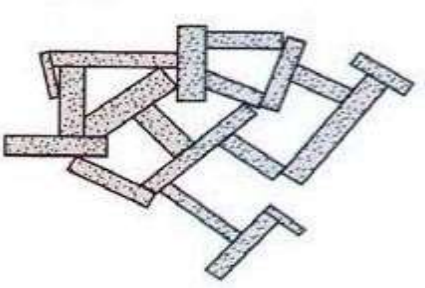


Figure 3.12: Flocculated clay

QUICK CLAY

Quick clay is also known as Leda clay and Champlain sea clay , is a unique form of highly sensitive marine clay, with the tendency to change from a relatively stiff condition to a liquid mass when it is disturbed. Undisturbed quick clay resembles a water saturated gel. When a mass of quick clay undergoes sufficient stress, however, it instantly turns into flowing ooze.

CLAY VERSUS SAND

Clays are quite different from sand in their characteristics and behaviors. Those are summarized in Table 3.2.

Table 3.2: Comparison between clay and sand

Properties and behavior	Clay	Sand
Particle size	Small (< 0.005 or 0.002 mm)	Large (>0.075 mm)
Structure	Clay structures	Crystal formations
Shape	Flat	Angular to round
Surface charge	Negative and sometimes positive at edge	Negligible
Specific charge	Large	Small
Interactive forces	Strong	Negligible
Plasticity	Plastic	Non plastic
Shear resistance	By cohesion	By friction
Volume change	Large, time dependent	Small, instantaneous

Soil and Soil Engineering:

The term *'Soil* 'has various meanings, depending upon the general professional field in which it is being considered. To an agriculturist, soil is the substance existing on the earth's surface, which grows and develops plant life. To the geologist also, soil is the material in the relatively thin surface zone within which roots occur, and all the rest of the crust is grouped under the term *rock* irrespective of its hardness. To an engineer, soil is the unaggregated or uncemented deposits of mineral and/or organic particles or fragments covering large portion of the earth's crust. It includes widely different materials like boulders, sands, gravels, clays and silts, and the range in the particle sizes in a soil may extend from grains only a fraction of a micron (10^{-4} cm) in diameter up to large size boulders.

Soil engineering, *Soil Mechanics* or Geotechnique is one of the youngest disciplines of civil engineering involving the study of soil, its behavior and application as an engineering material. According to Terazaghi (1948): 'Soil Mechanics is the application of laws of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulations of solid particles produced by the mechanical and chemical disintegration of rocks regardless of whether or not they contain an admixture of organic constituent'. The term *Soil Engineering* is currently used to cover a much wider scope implying that it is a practical science rather than a purely fundamental or mathematical one. The term *Foundation Engineering* is a branch of civil engineering, which is associated with the design, construction, maintenance, and renovation of footings, foundation walls, pile foundations, caissons, and all other structural members which form the foundations of buildings and other engineering structures (Taylor, 1948).

Soil is considered by the engineer as a complex material produced by the weathering of the solid rock. The formation of soil is as a result of the geologic cycle continually taking place on the face of the earth. The cycle consists of weathering or denudation, transportation, deposition and upheaval, again followed by weathering, and so on. *Weathering* is caused by the physical agencies such as periodical temperature changes, impact and splitting action of flowing water, ice and wind, and splitting actions of ice, plants and animals. Cohesionless soils are formed due to physical disintegration of rocks. Chemical weathering maybe caused due to oxidation, hydration, carbonation and leaching by organic acids and water. Clay minerals are produced by chemical weathering. Soil obtained due to weathering may be residual or transported. Residual soils, which remain in place directly over the parent rock, are relatively shallow in depth. The deposits of the transported soils may be considerable in depth and their homogeneity or heterogeneity depends upon the manner of their transportation and deposition. The various agencies of transporting and redepositing soils are : water, ice, wind and gravity. Water-formed transported soils arc termed as alluvial, marine or lacustrine. All the material, picked up, mixed, disintegrated, transported and redeposited by glaciers either by ice or by water issuing from melting of glaciers, is termed glacial drill or simple drift. The glacial deposits in general consists of a heterogeneous mixture of rock fragments and soils of varying sizes and proportions and except the stratified drift deposited by glacial streams, are without any normal stratification. Dune sand and loss are the wind-blown (aeoline) deposits. Loess is the wind-blown silt or silty clay having little or no stratification. Soils transported by gravitational forces are termed colluvial soils, such as talus. The accumulation of decaying and chemically deposited vegetable matter under conditions of excessive moisture results in the formation of cumulous soils, such as peat and muck

1.2 HISTORY OF DEVELOPMENT OF SOIL MECHANICS

The knowledge of the use of soil extends into prehistoric times, when man started constructing dwellings for living and roads for transportation. In the more primitive civilizations, soil was used by man as a construction material for foundations of structure and for the structures themselves. The knowledge of soils for the foundations, bunds and roads was gained by trial and error experiences. Through ancient times and even within the last few generations practically all improvement was the result of a continuously broadening by empirical knowledge. The use of both timber and stone caissons of soft-ground shaft construction was known in Egypt in 2000 BC. The cutting edge was made of a round limestone block with a vertical hole bored into its middle. The outside surface of the caisson was made smooth for reasons of reducing sinking resistance caused by friction. One of the greatest structures in ancient times was the famous 'hanging garden' built by the Babylonian King Nebuchadnezzar. The big retaining walls to support the terraces of the garden required some knowledge of earth pressures, even if the knowledge was empirical (Jumikis, 1962). The technical literature of the time during the Roman Empire supplies ample evidence that the Romans paid much attention to some properties of soils, and to the stability of foundations. The Romans built notable engineering structures, such as : harbors, moles, break-waters, aqueducts, bridges, large public buildings, sewage lines and a vast network of durable and excellent roads, requiring solutions of earth work and foundation design. The Roman engineer Vitruvius wrote his Ten books on Architecture sometimes in the first century tic discussing the stability of buildings, Vitruvius writes that " greatest care must be taken in the substructure, because, in these, immense damage is caused by the earth piled against them. For it cannot remain of the same weight as it usually has in the summer it swells in the winter by absorbing water from the rains. Consequently, by its weight and expansion it bursts and thrusts out the retaining wall" —(Jumikis, 1962). For foundations in loose or marshy land, he recommends the use of 'piles to be driven close together by machinery, and the intervals between them to be filled with charcoal.' In India also, Mansar, Mayamata, Visvakarma, Agastya, Santakumara, Mandana, Srikumara, etc. wrote books laying down rules of construction. Among these, Mansar's 'Silpa Sastre,' written sometimes in sixth or seventh century, became very popular. Mansar recommended compaction of soil by cows and oxen, and dewatering of foundations.

Many structures were built during the medieval period (about 400 to 1400 no). One of the main problems they had was about the compression of soil and the consequent settlement of buildings. During the past centuries, the compressible soil upon which heavy structures such as cathedrals etc. were built had enough time to consolidate, causing large settlements. The Leaning Tower of Pisa, constructed between 1174 to 1350 AD, is one such example. In India, Taj Mahal was constructed between 1632 to 1650 AD. It had unique foundation problems because of its proximity of the river Yamuna. The terrace and the mausoleum building, as well as the minarets, rest on one firm, compact bed of masonry, supported on masonry cylindrical wells sunk at close intervals. In the field of earth dams, the most notable example reported is that of Mudduk Masur dam in South India, of 33 m height, and built in 1500 no (On, 1969). In 1661, France undertook an extensive public works programme in improving the highways, and the building of canals. In the later part of the 17th century, French Military engineers contributed some empirical and analytical data pertaining to earth pressure on retaining walls for the design of revetments of fortifications. France established a Department of Roads and Bridges in 1715, and in 1747, the Famous Ecole des ponts et chausses was started. The first major contribution to the present scientific study of soil behaviour may be traced back towards the end of the eighteenth century, when Coulomb

(1776), a Frenchman, published his wedge theory of earth pressure. Coulomb was the first to introduce the concept that shearing resistance of soil is composed of two components, namely, cohesion and friction. Poncelet (1788-1867), a famous geometer, extended Coulomb's theory, giving a graphical method of finding the magnitude of earth pressure on the wall, vertical as well as for inclined wall surface on the back fill side, and for arbitrary broken polygonal surfaces. K.Culmann (1866) gave the Coulomb-Poncelet theory a geometrical formulation. The earth pressure theory was elaborated by graphical analysis also by Rebhann (1871) and Weyrauch (1878). Two important laws—Darcy's law for flow of water through soils and Stoke's law for settlement of solid particles in liquid—were put forward in 1856. Even today, these laws play an important role in soil engineering. In 1857, Rankine presented his theory for calculating earth pressure and safe bearing capacity of foundation. Rankine and other workers of his time did not take cohesion of clay soil into calculations, although they knew its existence. Another important contribution in the nineteenth century was made by Boussinesq (1885) who gave his analysis for stress distribution in a semi-infinite, elastic medium under surface point loads. To test the earth pressure theories, Muller-Breslau (1906) performed some relatively extensive and elaborate experiments with a large scale model retaining wall. In 1871, 0. Mohr gave a graphical representation of stress at a point, popularly known as Mohr's stress circle.

In soil mechanics, Mohr's stress circles are extensively used in the analysis of the shearing strength of soils. It is only in the beginning of the twentieth century that the basic physical properties of soil in general were understood, and the work of Atterberg, a Swedish soil scientist, and that of the Geotechnical Commission of the Swedish Government under the chairmanship of Dr. Fellenius, in this direction arc remarkable. Atterberg was the first to propose in 1911 the different stages of consistency in which a clay soil may exist, depending upon its water content. To measure the shear strength of sand, shear box was probably first developed in France, by Leygue in about 1885. Later, it was improved by Krey (1918) in Germany, and Terzaghi and Casagrande in U.S.A. Resat (1910) and Bell (1915) are credited to have extended Rankine's analysis of earth pressure so as to include soil with both friction and cohesion. Bell also suggested a method of calculating the bearing capacity of cohesive soils. In 1916, Petterson and Hultin used the circular sliding theory with the so-called friction circle in stability calculations. This method was further developed by Fellenius in 1926, and is now known as the Swedish method of slope analysis. In 1913, the Swedish Geotechnical Commission was appointed, with Fellenius as its chairman. In 1920, L. Praridtl gave his theory of plastic equilibrium, which forms the basis of various bearing capacity theories developed later. Dr. Terzaghi published his theory of consolidation in 1923 and the term Soil Mechanics was coined by him in 1925 when his book under the equivalent German title Erdbaumechanik was published. Dr. Teaaghi's contributions in the field of soil engineering have been immense and he is fittingly called the 'Father of Soil Mechanics'. Another important contribution made recently (1933) is that of Proctor on the principles of soil compaction. In 1922-23 Pavlovsky in Russia solved the complex problems of seepage below the hydraulic structures, and gave the electrical analogy method for the seepage computations. However, since his work was in Russian language, it remained unknown to the English literature till 1933, Weaver (1934) and Khosla (1936) solved some of the seepage problems independently. During World War II (1939-45) and after, a great impetus to the development of soil engineering has been made by various scientists and engineers of different countries of the World, and today it is recognised as a well-established branch of engineering. Several International conferences on Soil Mechanics and Foundation Engineering have been held till now under the auspices of International Society of Soil Mechanics and Foundation Engineering such as at Harvard (Massachusetts, U.S.A.) 1936, Rotterdam (Netherlands) 1948, Zurich (Switzerland) 1953, London (U.K.) 1957, Paris (France) 1961, Montreal

(Canada) 1965, Mexico city (Mexico) 1969, Moscow (U.S.S.R.) 1973, Tokyo (Japan) 1977, Stockholm (Sweden) 1981, San Fransisco (U.SA) 1985, Riode Jeneiro (Brazil) 1989, New Delhi (India) 1994, etc.

1.3 FIELD OF SOIL MECHANICS

The field of soil mechanics is very vast. The civil engineer has many diverse and important encounters with soil. Apart from the testing and classification of various types of soils in order to determine its physical properties, the knowledge of soil mechanics is particularly helpful in the following problems in civil engineering.

I. **Foundation design and construction.** Foundation is an important element of all civil engineering structures. Every structure — building, bridge, highway, tunnel, canal or dam — is founded in or on the surface of the earth. It is, therefore, necessary to know the bearing capacity of the soil, the pattern of stress distribution in the soil beneath the loaded area, the probable settlement of the foundation, effect of groundwater and the effect of vibrations, etc. The suitability of various types of foundations — i.e.. spread foundation, pile foundation, well foundation, etc. — depend upon the type of soil strata, the magnitude of loads and groundwater conditions. A knowledge of shrinkage and swelling characteristics of soil beneath the foundation is also very essential.

2. **Pavement design**. A pavement can either be flexible or rigid, and its performance depends upon the subsoil on which it rests. The thickness of a pavement and its component parts, depends upon some certain characteristics of the subsoil, which should be determined before the design is made. On busy pavements, where the intensity of traffic is very high, the effect of repetition of loading and the consequent fatigue failure has to be taken into account. Apart from these, other problems of pavement design are : frost, heave and thaw with their associated problems of frost damage to pavements ; frost penetration depth ; remedial measures to prevent frost damage ; problems of 'pumping' of clay subsoils and suitability of a soil as a construction material for building highways or railways, earth fills or cuts, etc. A knowledge of the techniques for the improvement of the soil properties such as strength and stability is very much helpful in constructing pavements on poor soils by stabilising them.

3. **Design of underground structures and earth retaining structures**. The design and construction of underground (subterranean) and earth retaining structures constitute an important phase of engineering. The examples of underground structures include tunnels, underground buildings, drainage structures and pipelines. The examples of earth retaining structures are : gravity retaining wall, anchored bulk heads and cofferdams. A knowledge of soil structure interaction is essential to design properly such structures subjected to soil loadings.

4. **Design of embankments and excavations.** When the surface of the soil structure is not horizontal, the component of gravity tends to move the soil downward, and may disturb the stability of the earth structure. A thorough knowledge of shear-strength and related properties of soil is essential to design the slope and height (or depth) of the embankment (or excavation). The possibility of the seeping groundwater reducing the soil strength while excavating must also be taken into account. It may sometimes be essential to drain the subsoil water, to increase the soil strength and to reduce the seepage forces. Deep excavations require lateral braces and sheet walls to prevent caving in.

5. **Design of earth dams**. The construction of an earth dam requires a very thorough knowledge of whole of the Soil Mechanics. Since soil is used as the only construction material in an earth dam, which

may either be homogeneous or of composite section, its design involves the determination of the following physical properties of soil : index properties such as density, plasticity characteristics and specific gravity, particle size distribution and gradation of the soil; permeability, consolidation and compaction characteristics, and shear strength parameters under various drainage conditions. Since huge earth mass is involved in its construction, suitable soil survey to the nearby area may be essential for the borrow-pit area. The determination of the optimum water content at which maximum density will be obtained on compaction, is probably the most essential aspect of the design. Apart from the seepage, characteristics of the dam section must be thoroughly investigated since these have the greatest impact on the stability of the slopes as well as the foundations of the dam. The consolidation characteristics help in predicting the long range behaviour of the dam toward settlement and the consequent reduction in the pore pressure. Lastly, the possible effect of vibrations during an earthquake should also be taken into account while designing.

The performance of the soil in the designs cited above depends upon the characteristics of soil. Therefore, the testing of soil with relation to the determination of its physical properties, and the evaluation of effects of certain other factors such as seepage conditions, etc. forms the most essential part of the development of soil engineering. It is through research only that design and construction methods are modified to give maximum safety and/or economy, and new methods are evolved. The knowledge of theoretical soil mechanics, assuming the soil to be an ideal elastic isotropic and homogeneous material, helps in predicting the behaviour of the soil in the field.

SOIL:

The term 'Soil' has various meaning, depending meanings, depending upon the generalprofessional field him which it is being considered.

To an Agriculturist: Soil is the substance existing on the ethics surface, which grows and developsplant life.

To the Geologist: Soil is the material in the relatively than surface zone within which rootsoccur, and all the rest of the cruet is grouped under the term Rock irrespective of its hardness.

To an Engineer:- Soil is the unaggregated or uncommented deposits of the of mineral andorganic particles or fragments convening Large portion of the earth's crust.

Different materials are

Boulders Sands Gravels	-	Coarse grained
soilsClays, Silts	-	Fine grained Soils

Range :- Particle sizes from micron (10^{-4} cm) in diameter up to layer size

bouldersSoil Engineering :-

Soil Mechanics (or) Geotechnique

It is of the youngest discipline of civil engineering involving the study of soil, its behaviour and application as an engineering material.

According to Terzaghi (1948):-

Soil Mechanics is the application of lawn of Mechanics and hydraulics to engineering problems dealing with sediment and other unconsolidated accumulations of solid particles produced by the mechanical and chemical disintegration of rock which may or not they contain organicconstituent.

Foundation Engineering:-

It is a branch of civil Engineering Which is associated with the design, construction, maintenance and renovation of toothily. Foundation walls, file foundation caimans and all other structures members which from the foundations of buildings and other engineering structures

INDEX PROPERTIES OF SOILS

1. Engineering Properties
   * Permeability
   * Compressibility
   * Shear Strength

2. Index properties –Identification &Classification of soils

   * Specific gravity of soil particles

   * Particle size distribution

   * Consistency limits and indices

   * Density Index

Categories:

i)Properties of individual particles

ii)Properties of the soil mass(or)aggregate properties

PARTICLESIZEANALYSIS

SIEVEANALYSIS:

The particle size analysis (or) mechanical analysis is meant the separation of a soil in to its different size fraction. The analysis invades into two stages.

(a) Sieve analysis -coarse grained soils(above 75µ)

(b) Sedimentation analysis -fine grained soils (below75µ)

(a) Sieve Analysis:

The sieves are designated by the of the opening in‘mm’ It earn be divided into two parts.

a) Coarse analysis(>4.75mm)

b) b) Fine analysis(4.75 mm to 75µ)

An oven – dried sample of soil is separated into two fraction by sieving it though a 4.75mm Is sieve The portion returned on it is termed as the gravel frication and is kept for the coarse analysis, while the portion passion thought it is subjected to fire sieve analysis.

Sieve sets:	Coarse analysis	Fine analysis
	IS: 100,63,20,10 &4.75mm	IS: 2mm, 1mm,600, 425,300,212,150and75µ

Procedure:-

* Sieving is performed by arranging the various sieves one over the other.
* The largest opening size being kept at the top and the smallest size is at the bottom.
* A receiver is kept at the bottom and a cover is kept at the top.
* The soil sample (1000g) is put of the top sieve and the whole assembly is filled on a sieve shaking machine.
* At least 10minof shaking is desirable for soil with small particles.
* The portion of the soil sample retained on each sieve is weighed.
* The percentage of soil retained on each sieve is calculated on the basic of the total mass of soil.
* The percentage passing through each sieve is calculated.

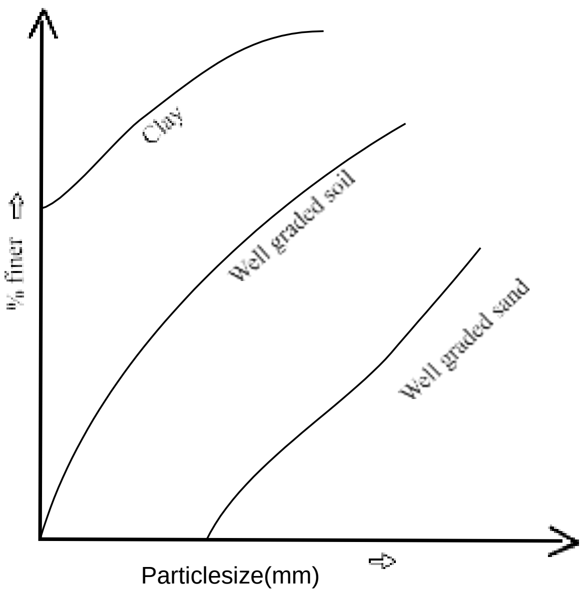
S.No.	IS Sieve	Particle size'D' (mm)	Wt.of sample retained(g)	%retained= $\frac{wt.of retained}{TotalWeight}$ x100%	Cumulative % retained(%)	% finer (N)(100-cumulative %)
1.	4.75	4.75	75	$\frac{75}{1000}$x100=7.5	7.5%	92.5
2.	2.0	2.0				
3.	1.0	1.0				
4.	600 µ	0.6				
5.	425 µ	0.425				
6.	300 µ	0.3				
7.	150µ	0.15				
8.	75 µ	0.075				
9.	Pan	-				

Then the graph is plotted between % fine in y- axis particle size 'D' (mm) in X –axis on log –scale, to get a particle size distribution curve. The curve given an idea about the type of the soil.

Well graded --- It has good representation of particles of all sizes.

Poorly graded --- It has an excess of certain particles and (uniformly graded) deficiency of other.

PARTICLESIZE DISTRIBUTIONCURVE



Two parameters are calculated from these curve, in order to classify the type of soil,

i) Uniformity co-efficient $C_u = \frac{D_{30}}{D_{60}}$

ii) Co-efficient of curvature $C_c = \frac{D_{30}^2}{D_{60} D_{30}}$

D10-Effectivesizein'mm'

D10,D30,D60–Particle size corresponding to 10%, 30%, 60% finer than this size in a soil mass.

for uniformly graded soil, C_u is nearly unity,

for well graded soil C_c = 1 to3

C_u> 4 for gravels6for sands

Uses of particle size Distribution curve:

- Used in the clarification of soils
- Determination of co-efficient of permeability
- Useful in soil stabilization
- Indirectly know the strength of soil

SEDIMENTATIONANALYSIS:

In the wet mechanical analysis or sedimentation analysis the soil frication finer than 75µ sieve is kept in suspension in a water the analysis is based on stokes law, according to which the velocity ofthesettledgrainsdependsuponthesizeandweightofthegrainthesettledgrainsdepen duponthesizeand weight of the grain the normally assumption is

*Soil particles are spherical and have same specific gravity. (G)

*The coarser particles settle more quickly than the finer ones.

Velocity $V = \frac{D^2 \gamma_w [G-1)}{18x10^6}$

D– Dia of particle G–Specific gravity

γ_w– unit wt of water ɳ –Viscosity of water.

It is done either with the help of a hydrometer (or) a pipette. In both the methods, a suitable amount of oven dried soil sample, finer than 75-micron size is mixed with a given volume (V) of water. The mixture is shaken thoroughly and the fest is started by keeping the jar, containing soil water mixed vertical. After any time, interval ‘t’ if a sample of soil suspension is taken from a height H_e with the help of formulae, die of the particle is obtained Hence sampling at different time intervals,atthissamplingdepthH_e,givetheparticlesofdifferentsizes.Usingthefollowingformulae,percentagefineris obtained.

$$N = \frac{100GXR}{(G-1)M_d}$$

Limitation of sedimentation analysis:

The analysis is based on the assumption that

* Soil particles are spherical.
* Particles settle independent of other particles.
* The walls of jar in which the suspension is kept, also do not affect the settlement.
* The upper of particle size for the validity of the law is about lower limit =0.0002mm.
* The soil has an average specifies gravity, the volume of which is used in compute the diameter ‘D’

PIPETTEMETHOD:

- In the sedimentation analysis only there particles which are finer than 75μ—size areincluded.
- About 12 to 30g of oven dried sample is accurate weighed and mixed with water to make smooth thin paste. To have proper dispersion of soil, a dispersing agent (sodium hexa-meta phosphate and sodium carbonate) is added to the soil 33g of sodium hexa-meta phosphate and 7g of sodium carbonate is distilled water to make one litre of solution.
- 25ml of this solution is added to the dish, containing the soil and water.
- The soil suspension is starved well of 15 minutes. Then it is transferred to the sedimentation tube. The tube in a constant

temperature water bath.

- The stop watch is started, and soil samples are collected at various time intervals, with thehelp of pipette.
- Those soil, which contain organic matter and calcium compounds are pretreated beforethe diversify agent is mixed.
- Sine these content act as contently agents and cause aggregations of parties. The process of removal of organic matter and calcium compounds is known as pretreatment.
- Hydrogen peroxide Solution is wired to remove the organic matter.
- To remove the calcium compound, the cooled mixture of soil is treated with hydrochloricacid.

HYDROMETER METHOD:

The volume of suspension is 1000 ml, double the quantify of dry soil and dispersing agent is taken. The sedimentation jar shaken vigorously and kept vertical.

The stop water is simultaneously started. The hydrometer is slowly inserted in the jar and readings aretaken at ½, 1, 2min trine intervals.

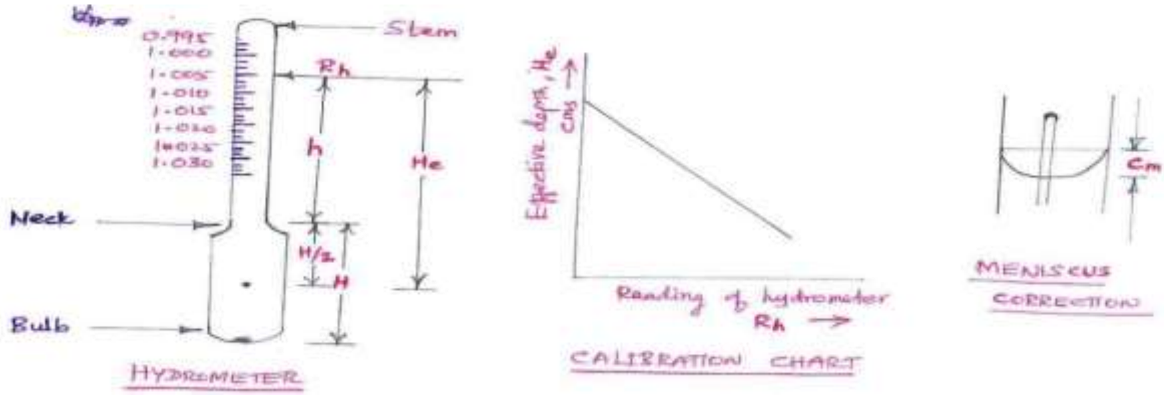
The hydrometer is then taken out. More readies are then taken at the following time intervals: 4, 8, 15,30 minutes and 1, 2, 4 hours ate.

To take the reading The hydrometer is inserted about 30 seconds before the green time interval, sothat it is stable at the time when the reading is be taken.

Effective depth $H_e = h + \frac{H}{2}$

h=distance from readily R_h on stem to the neck, in cm

H= Height of bulb, in cms.



a) Corrections to the hydrometer readings: Temperature Correction:(C_t)

Thehydrometerisgenerallydeignedat27°cIfthepremateureofsoilsuspensionis not27°c,a temperature correction C_t should be applied to the obeyed hydrometer reading. If the test temperature is more than 27°c, temperature correction is position. If the test temperate is less than27°c temperature correction is negative. So C_t+ Observed reading.

b) Meniscus Correction: (C_m)

Since the soil suspension is opaque, the hydrometer reading is taken at the top of meniscus. Actual reading to be taken at water level, will be more since the readings increase increase in the downward. Hence meniscus correction C_m is positive. Its magnitude can be found by immersing hydrometer in a jar containing clear water and finding the difference between the top and bottom of the meniscus.

C) Dispersing agent correction: (C_d)

The addition of dispersing agent in water increase its density, and hence the diversity agentcorrection C_d is always negative.

Correction hydro meter reading, $R = R_h + C_m \pm C_t - C_d C$

$= C_m \pm C_t - C_d$ may be negative(or)positive.

R_h=observed hydrometer reading. C_t=Temperature correction.

C_m=meniscus correction. C_d=Dispersing agent correction.

CONSISTENCY LIMITS (OR) ATTERBERG LIMITS:

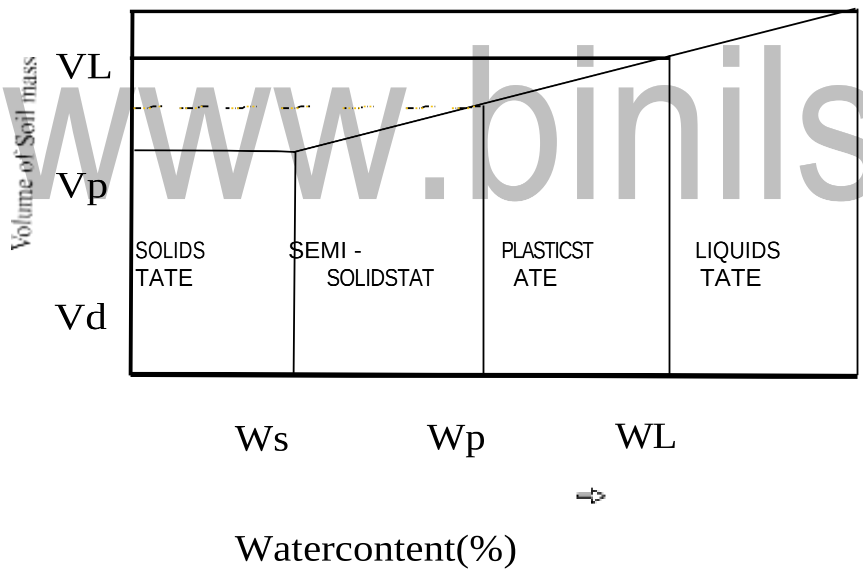
It is the water contents at which the soil mass passes from one state to the next. They ore mostuseful for engineering purposes are

* Liquid limit * Plastic limit * Shrinkage

limitExpressed as percentage of water content.

Consistency: It refers to the relative ease with which a soil mass can be deformed and used to describe the degree of firmness of fine grained soils for which Consistency relates to a large extent to water content.

STATESOFCONSISTENCY



LIQUID LIMIT (WL)

It is the water content corresponding to the limit between liquid and plastic state of the soil. It is defined as the minimum water content at which the soil is still in the liquid state.

ABOUT APPARATUS :

TEST:

The Liquid limit (WL) is determined in the Lab with the help of Casagrande's apparatus. The apparatus consist of rubber base ball over which a brass cup can be raised and lowered to fall on the rubber base with the help of rotating the handle. The height of fall of the cup can be adjusted with the help of screws.

PROCEDURE

1. About 120g of a soil passing through 425µ sieve is thoroughly mixed with water in the dish to form a uniform paste.
2. A portion of paste placed in the cup, spread in position and groove the soil specimen.
3. The handle is rotated and the number of blows are counted until the two parts of the soil sample come into contact.
4. After recording the numbe r of blows approximately 10 g of soil from near the closed groove is taken for water content determination.
5. Since it is difficult to adjust the water content preciously equal to the liquid limit when the groove should close in 25 blows, the liquid limit is determined by plotting a graph between number of blows in x - axis on a log scale and the corresponding water contentin y – axis.
6. Such a graph known as the flow curve. Slope of the flow curve is called flow index.

Flow curve

W2

W
L
W
1

Watercont(%)

N2 25N1

No.ofblows(logscale)

PLASTICLIMIT:

It is water content corresponding to an limit between the plastic and the semi – solid state.

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

PROCEDURE:

1) To defer nine the plastic limit, the soil specimen, passing 425 micron sieve, is mixed thoroughly with distilled water until the soil mass becomes plastic enough to be easily mould with fingers
2) A ball is formed with about 10g of this plastic soil mass and rolled between the fingers and a glass plate with just sufficient pressure to roll the mass into a uniform diameter throughout its length
3) When a diameter of 3mm is reached, the soil become to crumbled threads are kept for water content determination.
4) The test is repeated twice with fresh samples the plastic limit (WP) is then taken as the average of three water contents.

SHRINKAGE LIMIT :

It is defined as the maximum water content at which a reduction in water content will not cause decrease in the volume of a soil mass. It is lower water content at which a soil can still be completely saturated.

Procedure :

1. The volume V_1 of the shrinkage dish is first determined by filling to overflow with mercury, removing the excess by pressing a flat glass plate over its top and then taking the mass of the dish filled with mercury. The mass of the mercury contained in the dish divided by its density (13.6 g/cc) gives the volume of the dish.
2. About 50 g of soil passing 425µ IS sieve is mixed with distilled water and make the soil paste.

3. Soil paste is places in the shrinkage dish by three layer inorder to fill the voids, tapping ofshrinkage dish is taking place.
4. Before going to fill the soil paste, the dish is coated with oil. Excess soil paste on the top of the shrinkage dish has to be removed and leveled.
5. The dish filled with soil is then immediately weighted. The mass "M_1" of the wet soil pat is obtained be subtracting the mass of the soil with dish and the weight (or) mass of dish. The dish with soil placed in the oven.
6. The mass "M_d" of the day soil pat is found by similar manner.
7. To find the volume 'V2' of the dry soil pat, the mercury is filled in dish, the dry soil pat is placed on the surface of the mercury in the dish and is carefully forced down by means of three prong glass plate.
8. The mass of the mercury displaced from the dish divided by its density gives the volume 'V2' of the dry soil pat.

The shrinkage limit is calculated by the following relation,

$$W_S = \frac{(M_1 - M_d) - (V_1 - V_2)\gamma_w}{M_d}$$

FOLLOWING ARE THE DIFFERENT PARAMETERS OBTAINED FROM ATTERBERG LIMITS(ATTERBERGINDICES)

1. Plasticity Index(I_P):

   It is defined as liquid limit minus plastic limit

   (difference)$I_P = W_L - W_P$

2. Flow Index(I_F):

   It is the slope of flow curve obtained by plotting water content as ordinate on natural scale against number of blows as abscissa on log scale.

   $I_F = \frac{W_1 - W_2}{Log_{10}(\frac{N_1}{N_2})}$

   W1 – W/C corresponding to no. of blows N1

   W2– W/C corresponding to no. ofblowsN2

3. Toughness Index(I_T)

   It is defined as the ratio of plasticity index to flow index.

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

4. Consistency Index(I_C):

It is defined as the ratio of liquid limit minus natural water content to the plasticityindex.

$I_C = \frac{w_l - w}{I_p}$ If $W=W_L, I_c = 0$

If $W=W_p, I_c = 0$

.:I_C variesfrom'0'to'1' (100%) Plasticity range

I_C=negative->liquid state(<0)

I_C=positive ->but greaterthan1(100%)->semi-solid(or)solid state

(>1)

Liquidity Index(I_L):

It is defined as the ratio of natural water content minus plastic li of plasticity index.

When the soil mass is at liquid limit $W=W_L$ and $I_L=1$
When the soil mass is at plastic limit $W=W_P$ and $I_L=0$

Plasticity -> 0 to 1 (100%)I_L>1->liquid state

I_L>0-> semi solid(or)solid state Activity number,

$$A = \frac{I_p}{percent\ filter\ more\ than\ 2\mu}$$

A	Soiltype
< 0.75	Inactive
0.75 –1.40	Normal
>1.40	Active

Plasticityindex(%)	Plasticity
0	Non-plastic
<7	Low-plastic
7-17	Medium
>17	high

SHRINKAGE RATIO(SR):-

When a wet soil mass with its water content above shrinkage limit is dried to a water content greater than or equal to shrinkage limit, then whatever reduction in volume of soil mass takes place will be equal to the volume of water evaporated.

It is defined as the ratio of reduction in volume of soil mass expressed as % of its dry volume to the corresponding reduction in water content.

$$SR = \frac{\frac{W_1-W_2}{W_D}X100}{W_1-W_2}$$

V_1 = vol. of soil mass @ water content

W_1V_2 = vol. of soil mass @ water content

W_2V_d= volume of dry soil mass

W_1,W_2 = water contents in %

VOLUMETRIC SHRINKAGE(VS):

It is defined as the reduction in volume of soil mass expressed as a percentage of its dry volume when the soil mass is dried from a water content above shrinkage limit to shrinkage limit.

$$V_s=\frac{V1-Vd}{V_d}x100\%$$

V_1 – volume of soil mass at any water content

W1>Ws V_d–volume of dry soil mass

$$SR = \frac{\frac{W_1 - W_2}{W_D} X100}{W_1 - W_2} - \frac{V_S}{W_1 - W_d} \quad ; \quad Vs = SR(W1-Wd)$$

PROBLEM

1) The mass and volume of a saturated clay specimen were 29.8g and 17.7cm ^{3} respectively on oven drying the mass got reduced to 19g and the volume to 8.9 cm ^{3} .Calculate shrinkage limit, shrinkage ratio and volumetric shrinkage. Also compute ‘G’ of soil.

Given:

Mass of wet soil specimen	M=	29.8 g
Volume of wet soil specimen	V=	17.7cm^3
Mass of dry soil specimen	M_d=	19.0 g
Volume of dry soil specimen	V_d=	8.9cm^3

Solution: i)Shrinkage limit

$$W_s = \frac{(M - M_d) - (V - V_d)\rho_w}{M_d}$$

$$= \frac{(29.8 - 19.0) - (19.0 - 8.9)x1}{19.0}$$

=0.1053=>10.53 %

Volumetric Shrinkage(Vs) $= \frac{V - V_d}{V_d}$

$$= \frac{17.7 - 8.9}{8.9} = 98.8\%$$

ii. Shrinkage ratio (SR) $W_s = \frac{V_d \gamma_w}{M_d} - \frac{1}{G}$

$$SR = \frac{M_d}{V_d \cdot \gamma_w}$$

$$\frac{1}{G} = \frac{V_d \gamma_w}{M_d} - W_s$$

$$\frac{1}{G} = \frac{8.9}{19} - 0.01053$$

$$\frac{1}{G} = 0.3631$$

G=2.75

2) The following data on consistency limits are available for two soils A and B.

	SoilA	SoilB
Plastic limit	16 %	19 %
Liquid limit	30 %	52 %
Flow index	11	6
Natural water content	32 %	40 %

Find which soil is a)more plastic b)better foundation material on remoulding c)better shear strength as a function of water content d) better shear strength at plastic limit. Classify the soil as per ISCS. Do these soils have organic matter?

Given:

For Soil A for SoilB

Plasticity index $I_P = W_L - W_P$ plasticity index $I_P = W_L - W_P$

$= 30 - 16 \Rightarrow 14\%$ $= 52 - 19 \Rightarrow 33\%$

Since plasticity index of soil B is greater, then soil B is more plastic.

1. *For Soil A* *for Soil B*

consistency index

plasticity index $I_P = W_L - W_P$

$$I_c = \frac{W_l - w}{I_p}$$

$= \frac{30-32}{14} => -0.143\%$

$= \frac{52-40}{33} => 0.36\%$

I_C for soil A is negative, hence it will turn into slurry when remoulded. It is notsuitable for foundations. However soil B will be suitable. Because I_C = 0.25 to 0.50 (soft)

2. *For Soil A* *for Soil B*

Flow index $I_F = 11$ flow index $I_F = 6$

Since, the flow index of soil B is lesser than of soil A. Thus soil B has better shearstrength as a function of water content.

3. *For SoilA* *for SoilB*

Toughness index $I_T = \frac{I_P}{I_F}$ Toughness index $I_T = \frac{I_P}{I_F}$

Since, the toughness index for soil B is greater than that of soil A. Thus soil B hasbetter shear strength at plastic limit.

4. Classification of the soil as per ISCS

When I_P *and* w_L are named on the plasticity chart

forSoilA *forSoilB*

plasticityindexI_P=14%

plasticityindexI_P=33%W_L

=30 % W_L=52%

Hence, soil A and soil B fall in the zones of 'CL' and 'CH' respectively.

Thus, soil A is inorganic clay for low plasticity. While soil B is inorganic clay for high plasticity.

Hence, these soils do not have organic matter.

4)An undisturbed saturated specimen of clay has a volume of 18.9cm and a mass of 30.2g. On oven drying, the mass reduces to18.0g.The volume of dry specimen as determined by displacement of mercury is 9. 9cm^3.Determine shrinkage limit, specific gravity, shrinkage ratio and volumetric shrinkage.

Given:

M_1=30.2 g

M_d =18.0 g

ρ_w =1g/cm^3

V_1=18.9 cm^3

V_2=9.9 cm^3

$$w_s = \frac{(M_1 - M_d) - (V_1 - V_2)\rho_w}{M_d}$$

$$w_s = \frac{(30.2 - 18) - (18.9 - 9.9)1}{18.0} X100$$

=17.8%

ii) $$G = \frac{M_d}{V_1 - (M_1 - M_d)}$$

$$G = \frac{18.0}{18.9 - (30.2 - 18)} = 2.69$$

iii) Shrinkage ratio; $SR = \frac{\gamma_d}{\gamma_w} = \frac{\rho_d}{\rho_w}$

$$= \frac{1.818}{1} = 1.82$$

iv) Volumetric shrinkage, $VS = \frac{(V_1 - V_d)100}{V_d}$

$$VS = \frac{(18.9 - 9.9)100}{9.9} = 91\%$$

5) The mass specific gravity of a fully saturated specimen of clay having a water content of 36% is 1.86. On oven drying, the mass specific gravity drops to 1.72. Calculate the specific gravity of clay and its shrinkage limit.

Solution:

$e = w_{sat.}G = 0.36G$

mass specific gravity, $G_m = \left[\frac{(G+e)\gamma_w}{1+e}\right]\frac{1}{\gamma_w}$

$$1.86 = \left[\frac{G + 0.36G}{1 + 0.36G}\right]$$

$$1.86 = \left[\frac{1.36G}{1 + 0.36G}\right]$$

G=2.69

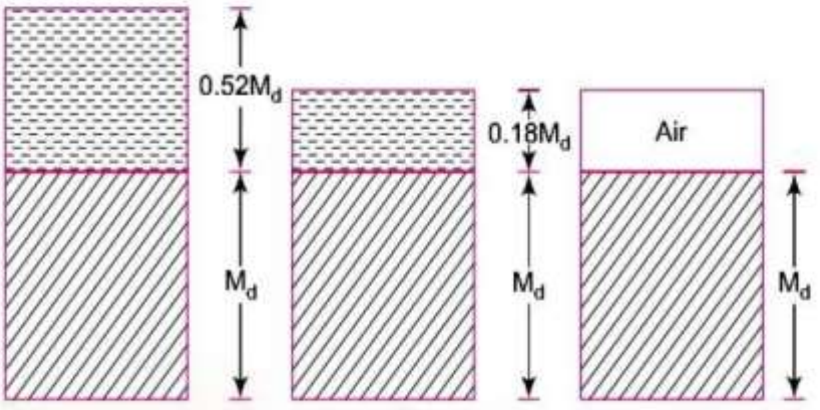
$$w_s = \frac{\gamma_w}{\gamma_d} - \frac{1}{G}$$

$$\frac{\gamma_w}{\gamma_d} = \frac{1}{1.72}$$

$$w_s = \frac{1}{1.72} - \frac{1}{2.69} = 0.21 = 21\%$$

6) The Atterberg limits of a clay soil are: liquid limit 52%, plastic lin1it 30%andshrinkage lin1it 180%. If the specimen of this s oil shrinks from a volume of 39.5 cm3 at the liquid limit to a volume of 24.2 cm^3 at the shrinkage limit, calculate the true specific gravity.

Solution. Figure (a,b,c) shows that the states of the specimen at liquid limit, shrinkage limit and dry condition,respectively .



(a) At liquid limit (b) At shrinkage limit (c) Dry sample

Difference of volume of water in (a) and (b)=39.5-24.2=15.3cm^3

Difference of mass of water in (a) and (b)=15.3g.

But from Fig.(a),(b),this difference is equal to (0.52-0.18)M_d

(0.52-0.18) M_d =15.3 or $M_d = \frac{15.3}{0.34} = 45g$

Mass of water in (b)=0.18 M_d

=0.18X45=8.1g

Volume of water in (b)=8.1 cm^3

Volume of solis V_s in(b)=24.2-8.1=16.1 cm^3

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

$$= \frac{45}{16.1} 2.8 g/cm^3$$

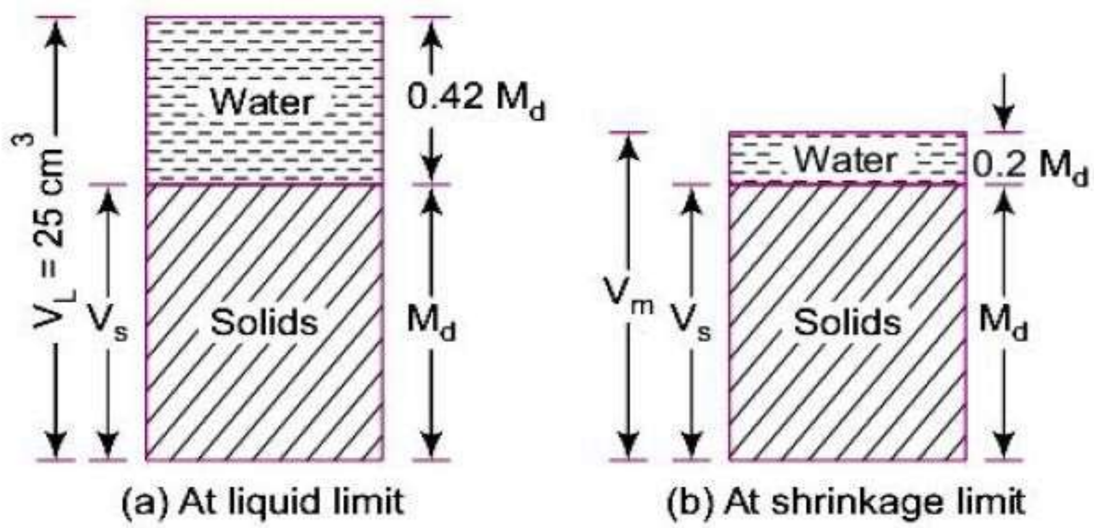
$$G_s = \frac{\gamma_s}{\gamma_w} = \frac{\rho_s}{\rho_w} = \frac{2.8}{1} = 2.8$$

7)A saturated soil sample has a volume of 25cm^3 at the liquid limit. If the soil has liquid limit and shrinkage limit of 42% and 20%, respectively, determine the minimum volume which can be attained by the soil specimen. Take G=2.72.

Solution: The soil specimen will attain minimum volume at shrinkage limit .Figure(a)and(b)Show the states of the specimen at liquid limit and shrinkage limit respectively.

If M_d is them as of solid, volume of water at liquid limit is

$$V_L = 0.42 M_d \text{cm}^3$$



Volume of solids,

$$V_s = \frac{M_d}{G\rho_w} = \frac{M_d}{2.71 X 1} = \frac{M_d}{2.71} cm^3$$

$$=0.368 M_d\, cm^3$$

$$\text{Total volume} = 0.42\, M_d + 0.368\, M_d = 25$$

$$M_d = 31.74\text{g}$$

At the shrinkage limit ,soil attains its minimum volume V_m

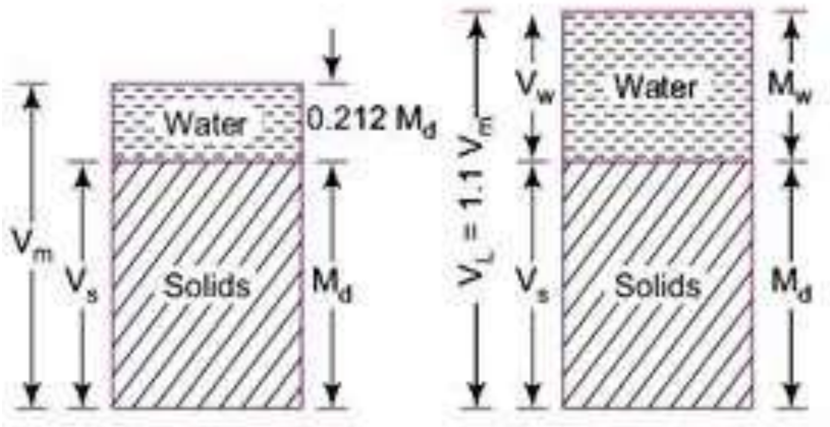
$$V_m = V_s + 0.2\, M_d$$

$$=0.368\, M_d + 0.2\, M_d$$

$=0.568X31.74s$

$=18.03\ cm^3$

8) An oven dried sample of soil has a volume of 265 cm3 and a mass of 456 g. Taking G = 2.71. determine the voids ratio and shrinkage limit. what will be the, water content which will fully saturate the soil sample and also cause an increase in volume equal to 10% of the original dry volume?



(a) At shrinkage limit (b) At desired wet state

Dry Density, $\rho_d = \frac{M_d}{V} = \frac{456}{265} = 1.721 g/cm^3$

$$\rho_d = \frac{G\rho_w}{1+e}$$

$$1.721 = \frac{2.7X1}{1+e}$$

$$e = 0.575$$

$$\text{shrinkage limit} = \frac{e}{G} = \frac{0.575}{2.71} = 0.212 = 21.2\%$$

Figures (a) and (b)show the states of the specimen at shrinkage limit and desired final state, respectively.

Now $V_m = 265\ cm^3$,

$$V_s = \frac{M_d}{G} = \frac{456}{2.71} = 168.27\ cm^3$$

At the final desired state $V = 1.1V_m = 1.1X265 = 291.5\ cm^3$-------- (1)

But from fig (a), $V = V_w + V_s = V_w + 168.27$ ------------------ (2)

From 1 an 2 we get $V_W = 291.5 - 168.27 = 123.23\ cm^3$

Mass of water in final state, $M_w = 123.23g$

Henc water content in final state $= \frac{M_w}{M_d} = \frac{123.23}{456} = 0.27 = 27\%$

Note:

Activity	Classification
< 0.75	Inactive
0.75 – 1.40	Normal
> 1.40	Active

Sensitivity	Classification	Structure
1	Insensitive	–
2 to 4	Normal or less sensitive	Honeycomb structure
4 to 8	Sensitive	Honeycomb or Flocculent structure
8 to 16	Extra sensitive	Flocculent structure
> 16	Quick	Unstable

9) A clay sample has liquid limit and plastic limit of 96% and 24% respectively. Sedimentation analysis reveals that the clay soil has 50% of the particles smaller than 0.002mm. Indicate the activity classification of the clay soil and the probable type of clay mineral.

Solution. We have $w_L = 96\%$ and $w_P = 24\%$

Hence plasticity index, $I_P = w_L - w_P = 96 - 24 = 72\%$

$$\text{activity} A_c = \frac{I_p}{C_w} = \frac{72}{50} = 17.44$$

Since the activity No .is greater than1.4, clay may be classified as being active. Also, the probable clay mineral is montmorillonite.

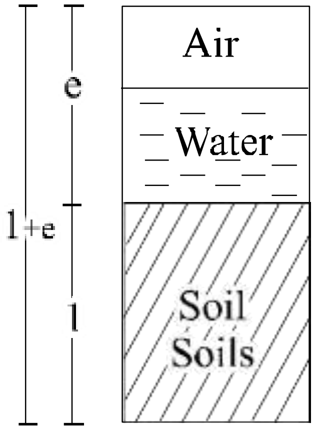
10) A clay specimen has unconfined compressive strength of 240kN/m^2 in undisturbed state .later, on remolding, the unconfined compressive strength is found to be 54kN/m^2 Classify the clay soil on the basis of sensitivity and indicate the probable structure of clay soil.

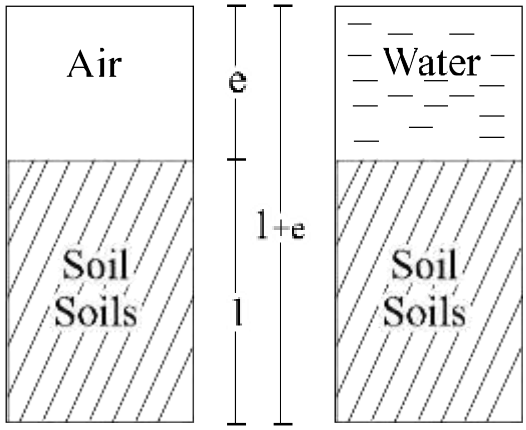
Solution:

$$\text{Sensitivity}, S_t = \frac{q_u(\text{undisturbed})}{q_u(\text{disturbed})} = \frac{240}{54} = 4.44$$

Since S, is between 4 and 8, the given clay is classified as sensitive. The possible structure of clay soil may be honey-comb or flocculent.

INTER RELATIONS:-



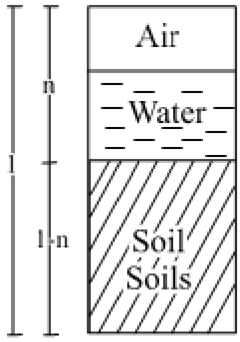


Soil Element interms of 'e' **Soil Element interms of 'e'**

FIG – 2

1) *Relation between 'e' and 'n'*

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



$$V = V_v + V_s$$

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

$$e = \frac{V_v}{V - V_v}$$

Divide V_v on top and bottom

$$e = \frac{1}{\frac{V}{V_v} - 1}$$

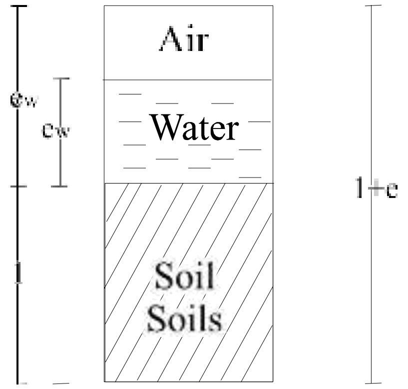
$$= \frac{1}{\frac{1}{n} - 1}$$

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

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

2) *Relation between ' e , s, w ' and ' G '*

Refer fig 4 e - water voids ratio



$$G = \frac{\frac{W_s}{V_s}}{\frac{W_w}{V_w}}$$

$$= \frac{W_s}{V_s} x \frac{V_w}{W_w}$$

$$= \frac{W_s}{W_w} X \frac{V_w}{V_s}$$

$$= \frac{1}{w} X \frac{V_w}{V_s}$$

Multiply and divide by V_v

$$G = \frac{1}{w} X \frac{V_w}{V_v} X \frac{V_v}{V_s}$$

$$G = \frac{1}{w} x S_r X e$$

W x G = S_r X e

For fully saturated S_r=1

e=w.G

3) *Relation between 'γ , γ_d' and ' w '*

We have

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

$$1 + w = \frac{W_w}{W_s} + 1 \quad = \frac{W_w + W_s}{W_s} = \frac{W}{W_s}$$

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

Dividing both sides by 'V' then we get,

$$\frac{W_s}{V} = \frac{W}{V(1 + w)}$$

Finally,

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

4) *Relation between 'γ , γ_d, γ_{sat}' and 'γ'*

Refer fig 4,

We have $\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{V_s\gamma_s + V_w\gamma_w}{V}$

Therefore $\gamma = \frac{1.\gamma_s + e_w\gamma_w}{1+e} = \frac{G\gamma_w + es\gamma}{1+e} = \frac{(G+es)\gamma_w}{1+e}$

For dry soil mass, s = 0

$$\gamma = \frac{G.\gamma_w}{1+e}$$

For fully saturated soil mass, s = 1 , $\gamma = \gamma_{sat}$

So that,

$$\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e}$$

For submerged condition, $\gamma' = \gamma_{sat} - \gamma_w$

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

$$= \frac{(G+e)\gamma_w - \gamma_w(1+e)}{1+e}$$

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

finally,

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

4) Relation between ‘ e , s ’ and ‘ n_a’

$$n_a = \frac{e(1-s)}{1+e}$$

5) *Relation between ‘ n_a , a_c ’ and ‘n’*

$$n_a = n \, . a_c$$

6) *Relation between ‘ γ_d , G ’ and ‘n’*

$$\gamma_d = (1-n)\, G\, \gamma_w$$

7) *Relation between ‘ γ_{sat} , G ’ and ‘n’*

$$\gamma_{sat} = (1-n)\, G\, \gamma_w + n.\gamma_w$$

8) *Relation between ‘ γ' , γ_d ’ and ‘n’*

$$\gamma' = \gamma_d - (1-n)\,\gamma_w$$

9) *Relation between ‘ γ_{sat} , γ , γ_d ’ and ‘ s ’*

$$\gamma = \gamma_d + s\,(\gamma_{sat} - \gamma_d)$$

10) *Relation between 'γ_d , G, w ' and 's'*

$$\gamma_d = \frac{G.\gamma_w}{1+\frac{w}{s}} \text{ if } S=1 \; \gamma_d = \frac{G.\gamma_w}{1+w_{sat}.G}$$

11) *Relation between 'γ_d , G , w ' and 'n_a'*

$$\gamma_d = \frac{(1-n_a)G.\gamma_w}{1+w.G}$$

13) Density Index and Relative compaction

It is defined as the ratio of the difference between the voids ratio of the soil in its *loosest* state 'e_{max}' and its natural voids ratio 'e' to the difference between the void ratio in the *loosest and densest* states. (e_{min})

$$\text{ie,} I_D = \frac{e_{max}-e}{e_{max}-e_{min}}$$

It is also referred as Density Index (or) Relative density (or) Degree of density. It is used to express the relation the compactness or degree of compaction of a natural cohesion less soil deposit. Degree of compaction is also sometimes expressed interims of an index called relation compaction (R_c).

$$\text{ie,} R_c = \frac{\gamma_d}{\gamma_{dmax}} \; or = \frac{1+e_{min}}{1+e}$$

γ_d max – maximum dry density from compaction test.

PROBLEM

1) A soil sample in its undisturbed state was found to have volume of 105cm^3 and massof 201 g. After oven drying the mass got reduced to 168g. Compute (i) water content (ii) void ratio (iii) porosity (iv) degree of saturation (v) air content. Take G = 2.7

Give:

Volume of soil mass, V = 105 cm^3
Mass of soil mass, M = 201 g
Dry mass of soil, M_d = 168 gSpecific gravity,
G = 2.7

Solution :

(i) Water content,

$$w = \frac{W_w}{W_s or\ W_d} = \frac{201 - 168}{168}$$

$$= 0.196 \Rightarrow 19.6\%$$

(ii) Void ratio,

$$e = \frac{G.\gamma_w}{\gamma_d} - 1$$

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

ie, Dry density, $\gamma_d = \frac{W_s or\ W_d}{V}$

$$= \frac{168}{105} = 1.6\ g/cm^3$$

void ratio,

$$e = \frac{G.\gamma_w}{\gamma_d} - 1$$

$$= \frac{2.7x1}{\gamma_d} - 1 = 0.69$$

(iii) Porosity (n)

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

$$= \frac{0.69}{1+0.69}$$

$$= 0.408 \Rightarrow 40.8\%$$

(iv) Degree of saturation (s)

$$\text{Wkt,} \quad e.s = w.G$$

$$S = \frac{wG}{e}$$

$$= \frac{0.190 X 2.7}{0.69}$$

$$= 0.767 \Rightarrow 76.7\%$$

(v) Air Content (a_c)

Wkt, $a_c = 1 - s \Rightarrow 1 - 0.767 \Rightarrow 0.233 \Rightarrow 23.3\%$

2) A natural soil deposit has a bulk unit weight of 18.44 KN/m^3 and water content of 5%. Calculate the amount of water required to be added to 1m^3 of soil to raise the water content to 15% Assume the voids ratio to remain constant. What will then be the degree of saturation? Assume G = 2.67

Given :

γ = 18.44 KN/m^3, V = 1 m^3, w_1 = 5%, w_2 = 15%, e = constant, G = 2.67

Solution :--

$$\gamma_d = \frac{\gamma}{1+w} = \frac{18.44}{1+0.05}$$

$=17.56 KN/m^3$

$$\gamma_d = \frac{W_d}{V} = 17.56 = \frac{W_d}{1}$$

$W_d = 17.56$ KN

If W=0.05(5%)

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

$$0.05 = \frac{W_w}{17.56}$$

$W_w = 0.88$ KN

$$\gamma_w = \frac{w_w}{V_w}$$

$$9.81 = \frac{0.88}{V_w}$$

$V_w = 0.089\ m^3$

If W=0.15(15%)

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

$$0.15 = \frac{W_w}{17.56}$$

$W_w = 2.634$ KN

$$\gamma_w = \frac{w_w}{V_d}$$

$$9.81 = \frac{2.634}{V_w}$$

$V_w = 0.27\ m^3$

The amount of water required to raise the water content = 0.270 – 0.089 = 0.181 m^3

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

$$17.56 = \frac{2.67X9.81}{1+e}$$

e=0.49

$$e = \frac{w.G}{S_r}$$

$$0.49 = \frac{0.15X2.67}{S_r}$$

$$S = 0.81 = 81\%$$

3) A soil specimen has a water content of 10% and a wet unit weight of 20kN/m3.If the specific gravity of solids is 2.70, determine the dry unit weight, void ratio, and the degree of saturation. Take γ_w=10kN/m^3.

Give data:

w=10%

γ=20kN/m^3

G=2.7

γ_w=10kN/m^3.

To Find:

γd=?

e =?

Sr=?

Solution:

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

$$= \frac{20}{1+w} = 18.18KN/m^3$$

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

$$1+e = \frac{2.7X10}{18.18} = 1.49$$

e =1.49-1=0.49

$$S = \frac{w\,G}{e}$$

$$= \frac{0.1X2.7}{0.49} = 0.551 = 55.1\%$$

4) A soil to be excavated from a borrow pit which has a density of 1.75 g/cc and water content of 12%. The specific gravity of soil is 2.7. the soil is compacted so that the water content is 18% and dry density 1.65g/cc for 1000 m^3 of soil, in fill estimate,

a)The quantity of soil excavated from the pit in m^3

b)The amount of water to be added.Also determine the void ratio soil in borrow pit and fill.

Given:

ρ = 1.75 g/cc, w = 12%, G = 2.67, V = 1000 m^3, after compaction, γ_d = 1.65g/cc,w=18%

Solution :

Let us use suffix '1' for borrow pit and '2' for the fill

For the borrow pit $$\rho = \frac{G\rho_w(1+w)}{1+e}$$

$$e_1 = \frac{G\rho_W(1+W)}{\rho} - 1$$

$$= \frac{2.7X(1+0.12)}{1.75} - 1 = 0.728$$

For the fill

$$\rho_d = \frac{G\rho_w}{1+e}$$

$$e_2 = \frac{G\rho_w}{\rho_d} - 1$$

$$= \frac{2.7X1}{1.65} - 1 = 0.636$$

Since the volume of soilds remains constant

$$V_s = \frac{V_1}{1+e_1} = \frac{V_2}{1+e_2} = \frac{1000}{1+0.636} = 611.1m^3$$

V_1 =611.1(1+0.728)=1056m^3

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

M_d=M_s =V_sG.ρ_w= 611.1X2.7X1=1649.97

M_{w1}=M_d.w_1=1649.97X0.12=197.996m^3

M_{w2}=M_d.w_2=1649.97X0.18=296.995m^3

Water to be added= M_{w2} - M_{w1} =296.995-197.996=99m^3

5) A soil sample has a porosity of 40% The specific gravity of solids is 2.70.Calculate (a)voids ratio, (b)dry density, (c)unit weight if the soil is 50% saturated, and (d)unit weight if the soil is completely saturated.

Given:

n=40%=0.4

G=2.7

$$e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.667$$

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

$$= \frac{2.7X9.81}{1+0.667} = 15.89\ KN/m^3$$

$$e = \frac{wG}{S}$$

$$w = \frac{eS}{G} = \frac{0.667X0.5}{2.7} = 0.124$$

$$\gamma = \gamma_d(1+w) = 15.89(1+0.124) = 17.86\ KN/m^3$$

When the soil is fully saturated e=w.G

$$w = \frac{e}{G} = \frac{0.667}{2.7} = 0.247$$

$$\gamma_{sat} = \gamma_d(1+w)$$

$$=15.89(1+0.247)=19.81 KN/m^3$$

6)The insitu- density of an embankment, compacted at a water content of l2% was determined with the help of a core cutter. The empty mass of the cutter was1286g and the cutter full of soil has a mass of 3195g, the volume of the cutter being 1000cm^3 Determine the bulk density, dry density and the degree of saturation of the embankment.

If the embankment becomes fully saturated during rains, what would be its water content and saturated unit weight? Assume no volume changes in soil on saturation. Take the specific gravity of the soilas2.70.

Solution:

Mass of soil in cutter, M=3195-1286=1909g

$$\text{Bulk density}, \rho = \frac{M}{V}$$

$$= \frac{1909}{1000} = 1909\ g/cm^3$$

$$\gamma = 9.81X1.909 = 18.73 KN/m^3$$

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

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

$$= \frac{2.7x9.81}{16.72} - 1 = 0.584$$

$$S = \frac{wG}{e} = \frac{0.12X2.7}{0.584} = 0.55 = 55\%$$

At saturation since the volume remains the same ,the voids ratio also remains unchanged now,

$$e = w.G$$

$$w = \frac{e}{G} = \frac{0.584}{2.7} = 0.216 = 21.\%$$

$$\gamma_{sat} = \frac{G+e}{1+e}\gamma_w$$

$$= \frac{2.7+0.584}{1+0.584} X9.81 = 20.34 KN/m^3$$

7)The insitu- percentage voids of a sand deposit is 34%. For determining the density index, dried sand from the stratum was first filled loosely in a 1000cm^3 mould and was then vibrated to give a maximum density. The loose dry mass in the mould was 1610 g and the dense dry mass at maximum compaction was found to be 1980g. Determine the density index if the specific gravity ofthe sand particles is 2.67.

Solution:

$$n = 34\% = 0.34$$

$$e = \frac{n}{1-n} = \frac{0.34}{1-0.34} = 0.515$$

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

$$= \frac{2.67X9.81}{1 + 0.515} = 17.289$$

$$(\gamma_d)_{max} = \frac{1980}{1000} X9.81 = 19.42 KN/m^3$$

$$(\gamma_d)_{min} = \frac{1610}{1000} X9.81 = 15.79\ KN/m^3$$

$$e_{min} = \frac{G\gamma_w}{(\gamma_d)_{max}} - 1$$

$$= \frac{2.67X9.81}{19.42} - 1 = 0.349$$

$$e_{max} = \frac{G\gamma_w}{(\gamma_d)_{min}} - 1$$

$$= \frac{2.67X9.81}{15.79} - 1 = 0.659$$

$$I_d = \frac{e_{max} - e}{e_{max} - e_{min}}$$

$$= \frac{0.659 - 0.515}{0.659 - 0.349} = 0.465 = 46.5\%$$

8) Sandy soil in a borrow pit has unit weight of solids as 26.3 kN/m^3 equal to 11% and bulk unit weight equal to 16.4kN/m^3 How many cubic meter of compacted fill could be constructed of 3500m^3 of sand excavated from the borrow pit, if the required value of porosity in the compacted fill is30%?Also compute the change in degree of saturation.

Let us use suffix I for the borrow pit soil and 2 for the compacted soil. Assuming that weight and water content do not change during construction, the change in the volume can be calculated from the change in the unit weight.

$$\frac{V_1}{V_2} = \frac{W/\gamma_1}{W/\gamma_2} = \frac{\gamma_2}{\gamma_1} = \frac{1 + e_1}{1 + e_2}$$

$$V_2 = V_1 X \frac{1+e_2}{1+e_1} = V_1 X \frac{1-n_1}{1-n_2} \text{ --------(1)}$$

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

$$= \frac{\gamma_s}{\gamma/(1+w)} - 1$$

$$= \frac{\gamma_s(1+w)}{\gamma} - 1$$

$$e_1 = \frac{\gamma_s(1+w)}{\gamma_1} - 1$$

$$= \frac{26.3(1+0.11)}{16.4} - 1 = 0.780$$

$$e_2 = \frac{n_2}{1-n_2}$$

$$= \frac{0.3}{1-0.3} = 0.429$$

$$V_2 = V_1 X \frac{1+e_1}{1+e_2}$$

$$= 3500X \frac{(1+0.429)}{(1+0.780)} = 2810m^3$$

$$S_1 = \frac{w\gamma_s}{e_1\gamma_w} = \frac{0.11X26.3}{0.78X9.81} = 0.378$$

$$S_2 = \frac{w\gamma_s}{e_2\gamma_w} = \frac{0.11X26.3}{0.429X9.81} = 0.687$$

Worked Example

1) A sample of dry soil weighs 68gm.Find the volume of voids if the total volume of the sample is 40ml and the specific gravity of solids is 2.65.Also determine the void ratio.
2) A moist soil sample weighs 3.52N.After drying in an oven, its weight is reduced to 2.9N.The specific gravity of solids and the mass specific gravity are,respectively2.65 and 1.85.Determine the water content ad voidratio,porosity and the degree of saturation. Take γ_w=10kN/m^3.
3) A soil has a porosity of 40%, the specific gravify of solids of 2.65and a water content of 12%.Determine the mass of water required to be added to 100m^3 of this soil for full saturation.
4) A sample of saturated soil has a water content of 25percnt and a bulk unit Weight of 20 kN/m^3 .Determine dry density, void ratio and specific gravity of solid particles. What would be the bulk unit weight of the same soil at the same void ratio but at a degree of saturation of 80%?Take γ_w=10kN/m^3.
5) A sample of clay was coated with paraffin wax and its mass, including the1 mass of wax ,was found to be 697.5gm.The sample was immersed in water and the volume of water displaced was found to be 355ml.The mass of the sample without wax was 690.0g and the water content of the representative specimen was18%.Determie the bulk density ,dry density, void ratio and the degree of saturation. The specific gravity of the solids was 2.70 and that of the wax was 0.89.
6) In a compaction test of a soil, the mass of wet soil when compacted in the mould was 1.855kg. The water content of the soil was16%.If the volume of the mould was 0.945litres,determine the dry density,void ratio,degree of saturation and percentage air voids.TakeG=2.*68*

PHASERELATION: -

A soil may be a three Phase system consisting of solid practical, water and air, the void space between the soil grains is filled partly with water and partly with air.(Three phase)

In case of dry soil mass, the voids are filled with air only. In case of perfectly saturated soil, the voids are filled completely. (Two phase)

The total volume (V) of the soil mass consists of

(i)Volume of air(Va) (ii)Volume of water(Vw) and

(iii)Volume of solids (Vs)

iv) Volume of voids, $V_v = V_a + V_w$

The Weight of air is considered to be negligible, hence the weight of total voids is equal to the weight of water(W_w) $W_a = 0$

$$W_v = W_w + W_a W_v = W_w$$

DEFINITION: -

1) Void ratio(e): -

It is defined as the ratio of volume of voids (V_v) to the volume of soil solids (V_s) no unit.

Expressed as &number in decimal form.

$$e = \frac{V_V}{V_S}$$

*Coarse grained soils, e=0.5to 0.9 *fine grained soils, e =0.7to1.5

2) Porosity(n): -

It is defined as the ratio of volume of voids to the total volume (V) of soil mass Expensed as %.It is also referred to as percentage voids. 0 < n < 100%

$$n = \frac{V_V}{V} X100\ \%$$

3) Degree of Saturation(s):

It is defined as the ratio volume of water(Vw) present in a soil mass to the volume of voids (Vv) Expressed as % it is also referred to as percent saturation.

$$S = \frac{V_W}{V_V} X100\%$$

For dry soil mass ,s= 0

For fully saturated ,s= 100%

4) Air content:(a_c)

It is defined the ratio of volume of air (Va) present in a soil mass to the volume of voids(V_v)expressed as a number in decimal form.

$$a_c = \frac{V_a}{V_V}$$

$$a_c = \frac{V_V - V_W}{V_V} = 1 - \frac{V_W}{V_V}$$

a_c =1-S

Percentage air voids:(n_a)

It is defined as the ratio of volume air (V_a) present in a soil mass to the total volume of soil mass (v), Expressed as%

$$\eta_a = \frac{V_a}{V} X100\%$$

5) Water content:(w)

It is defined as the ratio of water (W_w) present in a soil mass to the weight of soil solids (W_s) it is usually as %. It is also referred to as moisture content.

$$w = \frac{W_w}{W_s} x100\ \%$$

Range – 0 to∞ forsand–10%to30% for clay– 5%to300%.

UNIT WEIGHTS AND DENSITIES: -

1) Unit Weight of water:(γ_w)

It is the ratio weight of a given a volume water (W_w) to the volume of water (V_w) at a stated temperature.

$$\gamma_w = \frac{W_w}{V_w}\ \text{KN/m}^3$$

Density of Water, $\rho_w = \frac{M_W}{V_w} = \frac{mass\ of\ water}{volume\ of\ water}\quad g/cc$

γ_w@4°c=19.81KN/m^3 and ρ_w @ 4°c = 1g/cc

2)Bulk Unit Weight:(γ)

It is the Weight of soil mass(w) to the volume of soil mass(V)

$$\gamma = \frac{W}{V} KN/m^3$$

For saturated–moist unit weight

<u>Bulk density :</u>(ρ) Mass per unit volume of soil mass.

$$\rho = \frac{M}{V} g/cc$$

3) <u>Dry unit weight</u>(γ_d)

It is the ratio of weight soil solids(W_s)to the volume of soil mass(V)

$$\gamma_d = \frac{W_s or W_d}{V} KN/m^3$$

Dry density(ρ_d)

It is the ratio of soil solids (M_s) to the volume soil mass(V).

$$\rho_d = \frac{M_s or M_d}{V} \quad \text{g/cc}$$

4) Saturated unit weight(γ_{sat})

It is the ratio of weight of fully saturated soil mass (W_{sat}) to the total volume of soil mass(V)

$$\gamma_{sat} = \frac{W_{sat}}{V} KN/m^3$$

Saturated Density(ρ_{sat}): -

It is ratio of mass of fully saturated soil mass (M_{sat}) to the total volume of soil mass(V)

$\rho_{sat} = \frac{M_{sat}}{V}$ g/cc for fully saturated soil mass,

$\gamma = \gamma_{sat}$(or)$\rho = \rho_{sat}$Submerged unit Weight

Buoyant(or)Effective unit Weighty): (γ_{sub})or (γ'):

It is the ratio of submerged weight of soil mass (W_{sub})to the total volume of soil mass(v).

$\Upsilon_{sub} = \Upsilon' = \Upsilon_{sat} - \Upsilon_w$

$$\gamma_{sub}\ or \gamma' = \frac{W_{sub}}{V} KN/m^3$$

Submerged density(ρ_{sub}) (ρ'):

It is the ratio of submerged mass of soil mass (M_{sub}) to the total volume of soil mass(V)

$$\rho_{sub}\, or\, \rho' = \frac{M_{sub}}{V} g/cc$$

$$\gamma' = \gamma_{sat} - \gamma_w \text{(or)} \rho' = \rho_{sat} - \rho_w$$

5) Unit weight of soil solids (γ_s): -

It is the ratio of weight of soil solids (W_s) to the volume of soil solids (V_s)in a given soil mass

$$\gamma_{sub} = \frac{W_s}{V_s} \frac{KN}{m^3}$$

Density soil solids(ρ_s): -

Mass of soil solids (M_s) per unit volume of soil solids (V_s) in a given soil mass

$$\rho_s = \frac{M_s}{V_s} g/cc$$

Conversion:

$$1 gcc = \frac{10^6 X\, 9.81}{10^3 X 10^3} \frac{m^3 XN}{Kg x KN}$$

1 g/cc = 9.81KN/m^3

Specific gravity:(G) or (G_s)Specific gravity of soil particles:

It is defined as the ratio of the weight of a given volume of soil particles (W_s) to the weight of an equivalent volume of pure water at a stated temperature. (W_w)

Ratio of unit weight of soil particles (γ_s) to the unit weight of pure water at a stated temperature.(γ_w)

$$G = \frac{\gamma_s}{\gamma_w} \text{ or } \frac{\rho_s}{\rho_w}$$

* for sand, G=2.64to2.67

* for clay, G= 2.70to 2.80

* for silt, G = 2.68 to 2.70

<u>Specific gravity of soil mass</u>: -(G_m)

It is the ratio of bulk unit weight of (γ) soil mass to the unit weight of pure water (γ_w)at a stated temperature.

$$G_m = \frac{\gamma_m}{\gamma_w} \text{ or } \frac{\rho}{\rho_w}$$

SOIL COMPACTION :

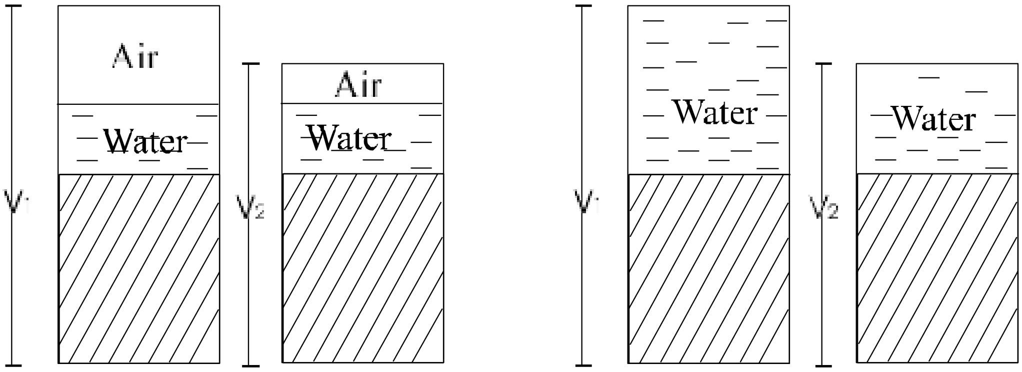
Compaction is a process by which the soil particles are artificially re-arranged and packed together into a closer of contact by mechanical means in order to decrease the void ratio of the soil and thus increase its dry density.

Three are two methods of compaction.

* Laboratory methods – It includes the following
  1) Standard proctor test
  2) Modified proctor test
  3) Harvard miniature compaction test
  4) Abbot compaction test
  5) Jodhpur – mini compactor test
* Field methods – It induces the following
  1) Tampers
  2) Rollers
  3) Vibratory compactors

Compaction	Consolidation
It is a rapid process of reduction of volume bymechanical means such as rolling, tamping and vibration.	It is a gradual process of reduction of volumeunder sustained, static loader.
The volume of a partially saturated soil decreases because of expulsion of air from thevoids of the unaltered water content.	It eases a reduction in volume of a saturated soildue to squeezing out of water from the soil.

In both the process, cause a reduction in the volume of soil mass takes place.



1) STANDARD PROCTOR TEST: (IS 2720 Part –VIII)

Equipment Required

(i) Cylindrical mould (metal) of capacity 100ml with an internal diameter of 100mm and internal effective height of 127.3mm mould fixed with a detectable base of removable collar of 60mm height .

(ii) Metal rammer 50mmdia, 2.6kg mass with a free fall of 310mm.

(iii) Mixing equipments, balances, sieves etc., Procedure:

1.) About 3kg of air-dried soil passing 4.75mm sieve is taken. Water is added to the soil to bring its water content to about 4% for coarse grained and 8% for fine grained.

2.) The proctor mould is cleaned, dried and greased lightly. The mass of the empty mould with the base plate, but without collar is taken.

3.) The collar is then fitted to the mould. The mould is placed on a solid base and filled with the soil sample to about 1/3 of its height. The soil is compacted by 25 blows of the rammer, with a free fall of 310mm. The blows are evenly distributed.

4.) The soil surface is scratched before the second layer. The mould is filled toabout 2/3 height with the soil and compacted again by 25 blows

5.) Likewise, the third layer is placed and compacted. It should project above the top of the mould in to the collar by not more than 6mm.

6.) The collar is rotated to break the bond between the soil and collar. The collar is then removed, and the soil is trimmed off flush with the top of the mould.

7.) The mass of the mould, base plate and compacted soil is taken, and thus the mass of the compacted soil is determined. Calculate the volume of the mould and bulk density of the soil.

8.) Eject out the soil from the mould, cut it in the middle and keep a representative soil specimen for water content determination.

9.) Repeat the process for about five or six times by increasing the water content by 2% for each times.

Bulk density $\rho = \frac{M}{V}$ g/cc

Dry density $\rho_d = \frac{\rho}{1+w}\ g/cc$

w – water content

M_1 – mass of empty mould with base

M_2 – mass of mould + base

compacted soil

M – mass of the compacted soil

V – volume of the mould.

Compaction curve:

A compaction curve is plotted between the w/c as abscissa and the corresponding dry densityas ordinate (see fig).

Note: It is observed that the dry density initially increases with an increase in w/c till the maximumdensity (ρ_d max) is affined. With further increase in w/c ρ_d decreases.

The water content corresponding to maximum dry density is know as optimum water content(OWC) (or) optimum moisture content (OMC).

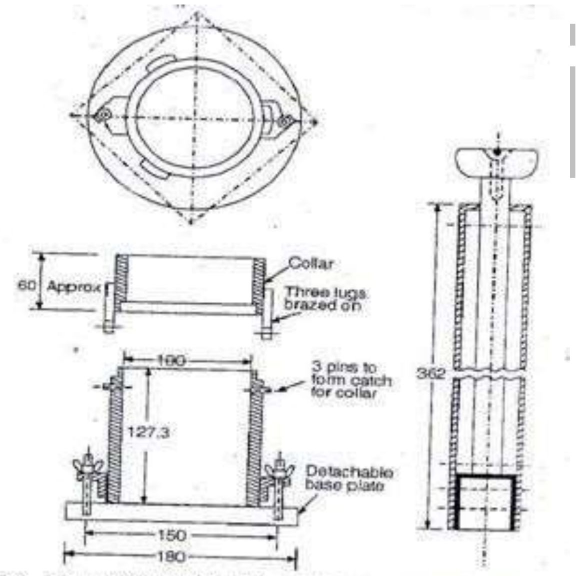
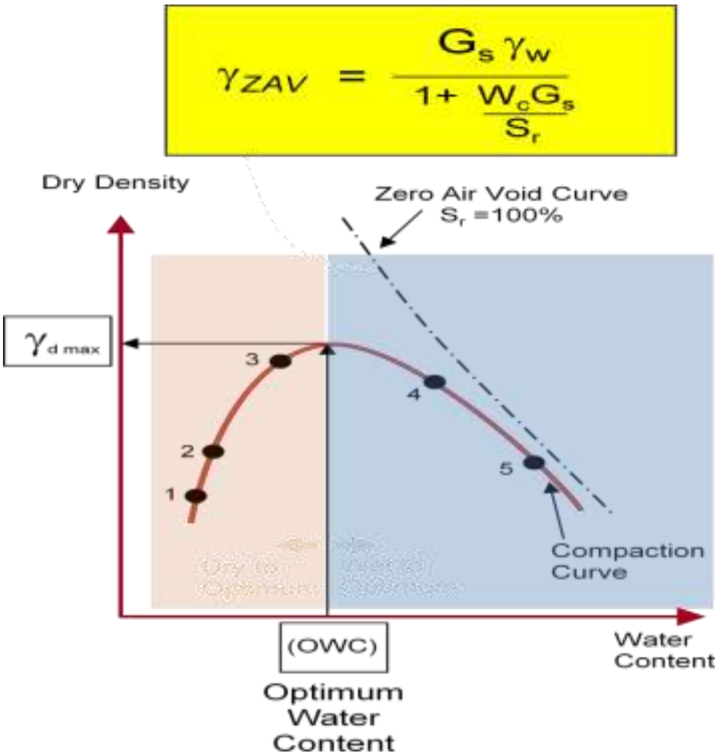


FIG. 17.1. EQUIVALENT FOR STANDARD PROCTOR TEST
[IS : 2720 (PART VII) : 1965 : LIGHT COMPACTION]



AIRVOIDS LINE:

A line which Shows the w/c and dry density relation for the compacted soil containing acontent percentage are voids known as Air – voids line and can be established from

$$\rho_d = \frac{(1 - n_a) G \rho_w}{(1 + wG)}$$

MODIFIED PROCTOR TEST:

It was developed to simulate the field conditions where heavy rollers are used. The test was standardized by the American Association of State Highway Officials and it therefore, also known as modified AASHO - test. IS – 2720 (Part –VIII) given the specification for heavy compaction bared on this test.

Equipment :

Mould – same as SPT

Man of hammer – 4.89kg. free fall – 450mm

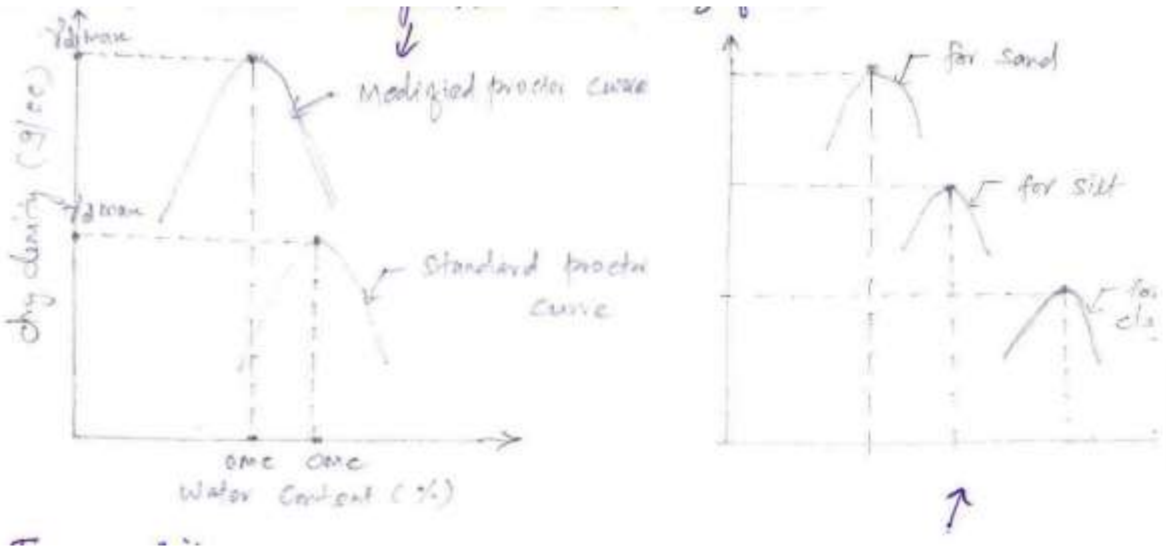
Compaction - 5 equal layers and 25 no. of blows for each.Effort – 4.56 times more than SPT.

Procedure – same as SPT.

In curve – Heavier compaction increases the maximum dry density but decreases the optimummoisture content (one)

FACTORS AFFECTING THE COMPACTION:

Water content:



When the water content is increases the compacted density goes an increasing till the maximum dry density is achieved after which further addition of water decreases the density. The increase in water content results in an reduction in the net affricative forces blow particles and hence higher density After the optimum water content is reached the air voids approach approximated a constant value as further increase in water contents does not cause any appreciable decrease in them the total voids due to water and air combination goes on increasing with increate of water content, beyond the optional, and hence the dry density of the soils falls. Amount of compaction: The effect of increasing the amount of comp active effort is to increase the maximum dry density and to decrease the OMC (see fig.) At a water content less than the optimum, the effect of increased compaction is more predominant. At a water content more than the optimum, the volume of air becomes almost and the effect of increased compaction is not significant.

2) Type of soil:

The dry density achieved depends upon the type of soil in general, carves – grained soils can be compacted to high air voids. These soils attain a relatively lower maximum dry density as compared with cohesion laws soils. Such soils regret more water for lubrication than cohesion less soils and therefore the optimum water content is high (refer fig, above)

3) Method of compaction:

The dry density achieved depends upon not only the amount of compactive effort bolt also on the method of compaction. For the same amount of comp active effort, the dry density will depends upon whether the method of compaction utilizes kneading action, dynamic action (or) static action.

4) Addition of Admixtures:

The compaction characteristics of the soils are improver by adding other materials, known as admixtures the most commonly used admixture are lance, cement and bitumen. The dry density achieved defends upon the type and amount of admixtures. For example, addition of lime, cement and solid waste will improves the compaction characteristics (ρ_d max &OMC) of expansive solids.

FIELD COMPACTION METHODS AND MONITORING:

Several methods are used for compaction of soil in the field the choice of the method will depends upon the soil type, the maximum dry density required, and economic consideration. The following are more commonly used conventional method.

1) Tampers:

A hand – operated tamper (or rammer) consists of a block of iron (or stone) about 3 to 5kg in mass, attached to a wooden rod. The tamper is lifted for about 0.3m and dropped on the soil to be compacted . A mechanical rammer is operated by compressed air (or) gasoline power.

They are used to compact soils adjacent to existing structures where other methods of compaction cannot be used they are not economical where large quantities of soils are involved It can be used for all types of soils.

2) Rollers:

Rollers of different type are used for compaction of soils the compaction depends upon the factors such as.

* Contact pressure – It increases, the compaction also increases
* Number of passes – for economy consideration, restricted blow 5 to 15
* Layer thickness – compaction increases the compaction.
* Speed of roller – compunction depends on it of the layer.

Types of Rollers:

a) Smooth – Wheel Rollers:

It generally crests of three wheels, Another type, consists of only two drums, one in the rear and one in the front, the mass of a smooth – wheel rollers generally varies blow 2 to 15mg

They are useful for finished operations after compaction of fills and for compaction granular back courses of highways.

There are not effective for compaction of deep layers of soils, as the results compaction pressure induced are low.

b) Pneumatic tyred rollers :

This type of rollers generally consists of 9 to 11 wheel fixed on two axles, with the pneumatic tires so speed that a complete coverage is obtained. With each pass of the rollers the gross mass of the roller varies between 5 to 200mg compressed air is used for operation.

These rollers are effective for compacting cohesive as well as cohesion soils light rollers are effective for soil layers of small thickness up to 15cm, where as heavy rollers are useful for layers of thickness up to 30cm.

C) Sheep – foot rollers:

In ancient time before the advent of the rollers, it was useful proactive to pass a flock a shape on the newly formed soil fill to cause its compaction. The same principle is used for the design of shape – foot rollers.

The sheep foot rollers consists of a hollow drum with a lave number of small projections on its surface the projections penetrate the soil layers during the rolling operation and cause compaction.

The drums are mounded on a steed frame the .drum eon be filled with water or ballast to increase the mass the contact pressure is generally between food to 4200 kN/m^2 the roller may since in to the soil if the contact pressure is more than the bearing capacity of the soil It is ideally suited for cohesive soils the depth of layer that can be compacted depends upon the length of the projection andweight of the roller.

d) Vibratory compactors:

In vibratory compactors, vibrations are induced in the soil during impaction the compactors, ifthe vibrator is mounted on a drum it is called a vibrators roller. It is available in a variety of forms.

Smooth – wheel type roller – compacting sandy soil unto 1m thickness
Pneumatic – typed compactor – compacting sandy soil upon 30cm thickness
Vibrating Plate – there are number of small plates Each plate is operated by a separate Vibrating unit Hand operated Vibrating placates are also available.

SUITABILITY OF ROLLERS:

For cohesive soils – sheep fort rollers
Cohesion less soils – smooth wheel rollers
For both soils – pneumatic tyred rollers.

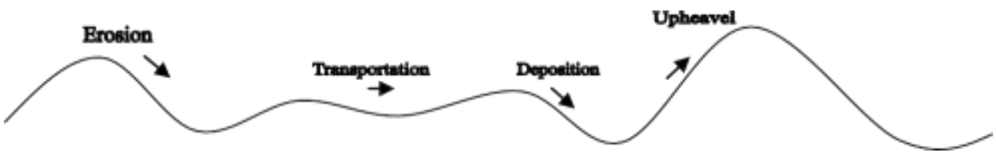
DIFFERENT FIELD COMPATION METHODS:

Bared on the principles of vibration, the compaction methods are clarified as,

* TERRAPROSE METHOD
* VIBROFLOTATION
* COMPACTION PILES
* COMPACTION BY EXPLOSIVES

Soil Formation :- It is a result of the geologic cycle continually taking place on the face of theearth.

* Weathering * Transportation * Deposition * Upheaval



GEOLOGIC CYCLE

Application (or) scope of Soil Engineering (or) Soil Mechanics :-

- Foundations - Design Considerations
- Retaining Structures - Earth Pressure determination & Analysis
- Stability of slopes - To check the stabling of slopes
- Underground Stuffiness - Design & Construction of tunnels, conduits etc.
- Pavement design - Behavior of sub grade under different loadings
- Earth Dam - Design and Construction.
- Miscellaneous Soil Problems - Soil Subsidence, Shrinkage and Swelling of soils.

Origin of Soils:-

Formed by Weathering of rocks due to Mechanical or chemical disintegration. When a rock surface gets exposed to atmosphere for an appreciable time, it dirt grate or decomposes into small particles and thus the Soils are formed.

PHYSICAL DISINTEGERATION : It is Occurred due to following physical processes.

(i) Temperature changes.

(ii) Rocks get broken into pieces when large sterns develop.

(iii) Cracks due to affine of the ice formed

(iv) Abrasion.

(v) There is no change in chemical composition.

(vi) Soil formed due to this are coarse grained soils.

CHEMICAL DISINEGRATION: It is due to following reasons:-

* Hydration	* Solution
* Carbonation	* Hydrolysis
* Oxidation	* Clay

minerals are formed.CLASSIFICATION

OF SOILS Based on its

formation :-

a) *Residual Soil:-* If the soil stays of the place of the formation just above the parent rocks.(shallow in depth)

b) *Transported soil:-* When the soil has been deposited at a place away from the place of itsorigin (considerable depth)

Based on Transportation Agent:-

a) Alluvial soils	:	Soils transported by rivers and streams.
b) Aeoline soils	:	Soils transported by wind.
c) Glacier soils	:	Soils transported by Glaciers.
d) Lancastrians soils	:	Soils deposited in Lake beds.
e) Marine soils	:	Soils deposited in sea beds.
f) Colluvial soils	:	Soils transported by gravitational forces.

Alluvial, marine and lacustrine - water transported soils.

Dune sand and loess - transported by wind.

Glacial drift - transported & re-deposited by glaciers.Glacial till (Boulder day) - transported & re-deposited by ice.

Varied clay (fine groaned) - transported by blocked mother water. Hard Pan - transported by ice presumes. (dense soil)

Talus - transported by gravitational forces.Commonly used type of soils:-

* Black cotton soils	* Cobbles	* Gravel
* Cleary	* Expansive clays	* Sand
* Boulders	* Bentonite	* Silt.