

GEOLOGICAL FORMATION OF SOILS AND TYPES

1.1 DEFINITIONS OF SOILS AND SOIL MECHANICS

1.1.1 Soils

The word soil is used in many professional fields and depending upon the context in which it is used, it has different meanings. For engineering purposes, soil is defined as a natural aggregate of mineral grains, loose or moderately cohesive, inorganic or organic in nature, that have the capacity of being separated by means of simple mechanical processes e.g. by agitation in water. This definition is not quite the same as adopted by the agriculturist or the geologist. To the agriculturist, soil is the top thin layer of earth within which organic forces are predominant and which is responsible for the support of plant life, and similarly to the geologist, soil is the material found in the relatively thin surface zone within which roots occur.

1.1.2 Soil Mechanics

Soil mechanics is one of the youngest disciplines of Civil Engineering involving the study of soil, its behavior and application as an engineering material. According to Terzaghi (1948): “Soil mechanics is the application of the 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 constituents”.

1.2 GEOLOGICAL FORMATION OF SOILS

The materials that constitute the earth's crust are rather arbitrarily divided into two categories, soil and rock. Soil is a natural aggregate of mineral grains, with or without organic constituents that can be separated by gentle mechanical means. Whereas rocks is considered to be a natural aggregate of mineral grains connected by strong and permanent cohesive forces. Rocks are classified according to their mode of formation as follows:

- i. **Igneous Rocks:** - are formed by cooling and solidification of magma within or on the surface of the earth's crust e.g. granite, basalt, dolerite, gabbro, andesite, rhyolite, pegmatites, peridotite, syenite.
- ii. **Sedimentary Rocks:** - are formed by consolidation and cementation of sediments deposited under water e.g. limestone, sandstone, shale, dolomite, mudstone, conglomerate.
- iii. **Metamorphic Rocks:** - are formed from older rocks when they are subjected to increased temperature, pressure and shearing stresses at considerable depth in the earth's crust e.g. slate, schist, marble, quartzite, gneiss.

All soils originate, directly or indirectly, from solid rocks by the process of weathering. The process of weathering of the rock decreases the cohesive forces binding the mineral grains and leads to the disintegration of bigger masses to smaller and smaller particles. The weathering of the rocks might be mechanical (disintegration) and/or chemical (decomposition).

Mechanical weathering is the fragmentation of rock by physical forces. Among the many physical forces responsible for the disintegration of rock, the following may be cited.

- a. **Temperature Changes;** - Temperature changes of sufficient amplitude and frequency bring about changes in the volume of the rocks in the superficial layers of the earth's crust in terms of expansion and contraction. Such a volume change sets up tensile and shear stresses in the rock and ultimately leading to the fracture of even large rocks.
- b. **Freezing action of water:** - Water that enters the pores and small cracks of rocks during the rainy season freezes during the cold season. As the water freezes it increases in volume thereby exerting pressure against the sides of the cracks. This enlarges the cracks and loosens particles of rock.

- c. **Spreading of roots of plants:** - As the roots of trees and shrubs grow in the cracks and fissures of the rocks, forces act on the rock. The segments of the rock are forced apart and disintegration of rocks occurs.
- d. **Abrasion:** - The movement of glaciers over exposed rock surfaces, rainstorms and the resulting torrents of flood water carrying vast quantities of rock debris, the action of the sea repeatedly pounding the coast line- all these contribute in time to the physical break-up of the rock mass and extensive erosion and abrasion of the land surface.

In all the processes of the mechanical weathering, there is no change in the chemical composition. The soil formed has the properties of the parent rock. On the other hand when chemical weathering of rocks takes place, original rock minerals are transformed into new minerals by chemical reaction. The soils formed do not have the properties of the parent rock. The following chemical processes generally occur in nature.

- a) **Oxidation:** - Oxidation occurs frequently in rocks containing iron, which decomposes in a manner similar to the rusting of steel when in contact with moist-air.
- b) **Carbonation:** - The mineral containing iron, calcium, magnesium, sodium or potassium can be decomposed by carbonic acid, which is formed by carbon dioxide with water.
- c) **Hydration:** - Hydration is a common process of rock decay by which water is combined with some other soil substances thus producing certain new minerals.
- d) **Leaching:** - Leaching is the process whereby water-soluble parts are dissolved and washed out from the soil by rainfall, percolating water, subsurface flow or other water.

Soils, which are formed by mechanical or chemical weathering, may be classified as residual or transported soils. Residual soils are soils, which are still located in the place of their origin. Transported soils are soils which have been transported by water, wind, ice and deposited in an area different from their place of origin. Residual soils are usually homogeneous and stiff while transported soils are loose, soft and non-homogeneous.

Transported soils may be classified according to the mode of their transportation and deposition such as

- i. **Alluvial soils** are those soils that have been transported by running water and deposited along a stream.
- ii. **Aeolian soils** are those soils that have been transported and deposited by wind.
- iii. **Lacustrine** soils are those soils that have been deposited from suspension in quite fresh water lakes.

- iv. **Colluvial soils** are those soils that have been deposited by movement of soil by gravity such as landslides.
- v. **Marine soils** are those soils that have been deposited from suspension in seawater.
- vi. **Glacial soils** are those soils that have been deposited as a result of glacial activities

1.3 COMMON SOIL TYPES

Depending upon the weathered rocks and the different methods of transportation and deposition, different soils are formed. These soils have been differently named. Some of the important soils, so found in general practice, are given below.

1. Sand, gravel, cobbles and boulders are coarse-grained cohesionless soils. Grain-size ranges are used to distinguish between them. Particles of size from 0.06 to 2mm are referred to as sand and those with a size from 2 to 60mm as gravel. Fragments with diameters more than 60mm and less than 200mm are known as cobbles and those with a size greater than 200mm are categorized as boulders.
2. Silt is a fine grained soil, having particle size between 0.06mm to 0.002mm. Inorganic silt has little or no plasticity and is cohesionless. Organic silt contains an admixture of organic matter. It is somewhat plastic, highly compressible, cohesive and relatively impervious. It is a very poor foundation material because of compressibility.
3. Clay is composed of microscopic particles of weathered rock within a wide range of water content, clay exhibits plasticity. It contains a large quantity of clay minerals. Clay is a cohesive fine-grained soil and the particle size is less than 0.002mm. Organic clay contains some finely divided organic matter and is usually dark grey or black in color. Organic clays are highly compressible when saturated and their dry strength is very high.
4. Black cotton soil is a residual soil containing a high percentage of the clay mineral montmorillonite. The soil has high shrinkage and expansive characteristics. Its color varies from dark grey to black. Great care is required when structures are to be built on black cotton soil.
5. Tuff is a fine grained soil composed of very small particles ejected from volcanoes during its explosion and deposited by wind or water.
6. Hardpan is a term often used to describe any hard cemented layers, which are not softening when wet.

7. Peat is composed of fibrous particles of decayed vegetable matter. It is so compressible that it is entirely unsuitable to support any type of foundation.
8. Top soil is surface soil that supports plants. It contains a large quantity of organic matter and is not suitable for foundation.
9. Lateritic soil is residual soil formed in tropical region. It is formed by decomposition of rock, removal of bases and silica, and accumulation of iron oxide and aluminum oxide. The presence of iron oxide gives this soil the characteristic red or pink color. The lateritic soil is soft and can be cut with a chisel when wet. However, it becomes hard after long exposure. Hardness is due to cementing action of iron oxide and aluminum oxide. A hard crust of gravel size particles, known as laterite, exists near the ground surface.
10. Loam is a mixture of sand, silt and clay.

SIMPLE SOIL PROPERTIES

2.1 WEIGHT – VOLUME RELATIONSHIPS

Soil mass is generally a three phase system. It consists of solid particles, water and air. The inter-relationships of the weights and volumes of the different phases are important since they help to define the condition or the physical make-up of a soil.

Figure 2.1a shows an element of soil of volume V and weight W , as it would exist in a natural state. In order to develop the weight volume relationships, the three phases (solids, water and air) are separated as shown in Fig. 2.1b.

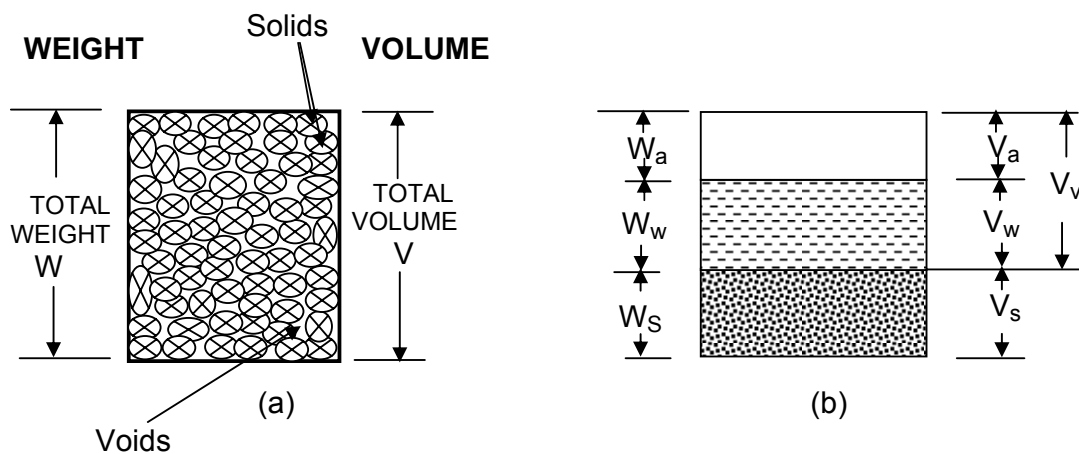


Fig.2.1 (a) Soil element in Natural state, (b) Block diagram

Thus the total volume V of a given soil sample can be expressed as

$$V = V_s + V_v = V_s + V_w + V_a \dots \dots \dots (2.1)$$

where

V_s = Volume of soil solids

V_v = Volume of voids

V_w = Volume of water in the voids

V_a = Volume of air in the voids

And the total weight W of the sample (assuming the weight of the air to be negligible) can be given by

where W_s = Weight of Solids
 W_w = Weight of water

The following relationships can be established

2.1.1 Void Ratio (e)

The void ratio of the soil mass is defined as the ratio of the volume of voids to the volume of solids, or

$$e = \frac{V_v}{V_s} \dots\dots\dots (2.3)$$

The void ratio expressed as a decimal.

2.1.2 Porosity (n)

The porosity of the soil mass is defined as the ratio of the volume of voids to the total volume, or

$$n = \frac{V_v}{V} \times 100 \dots\dots\dots (2.4)$$

It is always expressed as a percent.

2.1.3 Degree of Saturation (S)

The degree of saturation, S, is expressed as a ratio of the volume of water, V_w , in the voids to the volume of voids, V_v .

$$S = \frac{V_w}{V_v} \times 100 \dots\dots\dots (2.5)$$

It is always expressed as a percent.

When the degree of saturation S of a soil is zero percent, the soil is in a dry state and when the degree of saturation is 100 percent the soil is called fully saturated. In both these states, it represents a two-phase system. When S lies between zero percent and 100 percent, the soil is moist or partially saturated and represents a three-phase system.

2.1.4 Water Content (ω)

The water content, ω , of a soil mass is defined as the ratio of the weight of water, W_w , in a given mass to the weight mass of soil of solids, W_s , in the same mass.

$$\omega = \frac{W_w}{W_s} \times 100 \dots\dots\dots (2.6)$$

It is always expressed as a percent

In the laboratory the moisture (water) content is determined first by weighing a representative sample of soil in its natural or wet state; drying the sample to constant weight in an oven at a temperature of 105⁰C; and then weighing the dried sample. The difference

between the weights of the sample before and after drying represents the amount of water in the sample. This weight, computed as a percentage of the dried sample, is the moisture content of the soil.

$$\begin{aligned}
 \text{Weight of the container, empty} &= W_c \\
 \text{Weight of the container + Sample, } W_1 &= W_c + (W_s + W_w) \\
 \text{Weight of the container + dried sample, } W_2 &= W_c + W_s \\
 \text{Weight of water, } W_w = W_1 - W_2 &= W_c + (W_s + W_w) - (W_c + W_s) \\
 \text{Weight of dried sample, } W_s &= W_2 - W_c = W_c + W_s - W_c
 \end{aligned}$$

$$\text{Moisture content, } \omega = \frac{W_1 - W_2}{W_2 - W_c} \times 100 = \frac{W_w}{W_s} \times 100$$

2.1.5 Unit Weights

2.1.5.1 Unit Weight of Water (γ_w)

The unit weight of water at a given temperature is expressed as the ratio of the weight of water to the volume of water at the same temperature. The notation γ_w is generally used for unit weight of water at reference temperature 4°C.

2.1.5.2 Total Unit Weight of Soil Mass (γ_t)

Total unit weight of soil mass is the ratio of the weight of the mass, W , to the volume of the mass, V .

$$\gamma_t = \frac{W}{V} \dots\dots\dots (2.7)$$

This is also known as Bulk unit weight of soil

2.1.5.3 Dry Unit Weight (γ_{dry})

It is expressed as the ratio of the weight of solids, W_s , to the total volume, V .

$$\gamma_{dry} = \frac{W_s}{V} \dots\dots\dots (2.8)$$

2.1.5.4 Saturated Unit Weight (γ_{sat})

The saturated unit weight, γ_{sat} of a soil mass is the ratio of the saturated weight of the mass (when $S = 100$ percent) to the total volume.

$$\gamma_{sat} = \frac{W}{V} \dots\dots\dots (2.9)$$

2.1.5.5 Unit Weight of Solids (γ_s)

The unit weight of solids, γ_s , is defined as the ratio of the weight of solids, W_s , to the volume of the solids, V_s .

$$\gamma_s = \frac{W_s}{V_s} \dots\dots\dots (2.10)$$

2.1.5.6 Submerged Unit Weight (γ_b)

This is defined as the unit weight of saturated soil mass minus the unit weight of water.

$$\gamma_b = \gamma_{sat} - \gamma_w \dots\dots\dots (2.11)$$

2.1.6 Specific Gravity

2.1.6.1 Absolute Specific Gravity (G_s)

The specific gravity of solid matter in a soil particle may be defined as the ratio of the unit weight of solid matter to the unit weight of water. It is a measure of and a means of expressing the heaviness of material. The specific gravity of the solids is expressed by

$$G_s = \frac{\gamma_s}{\gamma_w} \dots\dots\dots (2.12)$$

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

2.1.6.2 Bulk Specific Gravity or Apparent Specific Gravity (G_b)

This is defined as the ratio of the dry weight of a unit volume of soil (volume of solids plus volume of voids) to the unit weight of water.

$$G_b = \frac{W_s}{(V_s + V_v) \gamma_w} \dots\dots\dots (2.13)$$

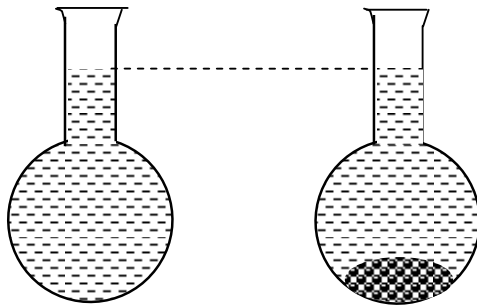
$$= \frac{W_s}{V \gamma_w}$$

Determination of specific Gravity

The specific gravity determination of a sample of soil is made by displacement in water using pycnometer (volumetric bottle). The specific gravity obtained by this method is the absolute specific gravity. In this test a known weight of oven dried soil sample is carefully put in a pycnometer, which is, then half filled with distilled water. The air entrapped in the soil

sample is removed by heating or by means of vacuum pumps. The bottle is then topped up with distilled water up to a calibration mark and brought up to a constant temperature. After carefully wiped and dried, it is weighed.

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



Wt. of pycnometer + water = W_2 ; Wt. of pycnometer + water + solids = W_1

Fig. 2.2. Determination of specific gravity

Let:

Weight of pycnometer bottle + water	= W_2
Weight of pycnometer bottle + water + soil	= W_1
Weight of dry soil	= W_s
Let the weight of displaced water	= X

Then

$$W_2 + W_s = W_1 + X$$

$$X = W_s + W_2 - W_1$$

$$\text{Volume of displaced water} = \frac{W_s + W_2 - W_1}{\gamma_{wT}}$$

$$G_T = \frac{\gamma_{wT}}{\gamma_w}$$

$$\gamma_{wT} = G_T \gamma_w$$

Where G_T = Specific gravity of water at temperature T

γ_{wT} = Unit weight of water at temperature T

$$\text{Volume of displaced water} = \frac{W_s + W_2 - W_1}{G_T \gamma_w}$$

This is equal to the volume of solids

$$\text{Therefore Volume of solids (V}_s) = \frac{W_s + W_2 - W_1}{G_T \gamma_w}$$

$$G_s = \frac{W_s / \left(\frac{W_s + W_2 - W_1}{G_T \gamma_w} \right)}{\gamma_w}$$

$$G_s = \frac{W_s G_T \gamma_w}{W_s + W_2 - W_1} \times \frac{1}{\gamma_w}$$

$$G_s = \frac{W_s G_T}{W_s + W_2 - W_1} \dots\dots\dots (2.14)$$

Interrelationships of Different parameters

Relationship between e and n

Referring Fig .2.1 and from Eqn. (2.3)

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

$$e (V - V_v) = V_v$$

$$eV = V_v (1 + e)$$

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

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

Therefore $n = \frac{e}{1 + e} \dots\dots\dots (2.15)$

$$n = (1 + e) = e$$

$$n + en = e$$

$$e(1 - n) = n$$

$$e = \frac{n}{1-n} \dots\dots\dots (2.16)$$

The Relationships Between e , G_s , S and ω

Case 1. When Partially Saturated ($0 < S < 100\%$)

From Eqn. (2.5)

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

substitute $V_w = \frac{W_w}{\gamma_w}$ and $V_v = eV_s$

then $S = \frac{W_w / \gamma_w}{eV_s}$

but $W_w = \omega W_s$

Therefore $S = \frac{\omega W_s}{\gamma_w e V_s}$

substitute $\frac{W_s}{V_s} = \gamma_s$

then $S = \frac{\omega \gamma_s}{\gamma_w e}$

but $\frac{\gamma_s}{\gamma_w} = G_s$

Therefore $S = \frac{\omega G_s}{e}$ or $e = \frac{\omega G_s}{S} \dots\dots\dots (2.17)$

Case 2. When Fully Saturated (i.e. $S = 100\%$)

From Equation (2.17)

$$e = \frac{\omega G_s}{S}$$

but $S = 1$

Therefore $e = \omega G_s \dots\dots\dots (2.18)$

Unit Weight of Soil Expressed In terms of Other Parameters

Case 1. When Partially Saturated (i.e. $0 < S < 100\%$)

From Equation (2.7)

$$\gamma_t = \frac{W}{V}$$

but

$$W = W_s + W_w$$

and

$$V = V_s + V_v$$

$$\begin{aligned} \gamma_t &= \frac{W_s + W_w}{V_s + V_v} \\ &= \frac{W_s(1 + W_w/W_s)}{V_s(1 + V_v/V_s)} \end{aligned}$$

but $\frac{W_s}{W_w} = \omega$

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

and $\frac{W_s}{V_s} = \gamma_s$

Therefore $\gamma_t = \gamma_s \frac{(1 + \omega)}{1 + e}$

But $\gamma_s = G_s \gamma_w$

Therefore $\gamma_t = \frac{G_s \gamma_w (1 + \omega)}{1 + e}$ (2.19a)

substituting $\omega = \frac{eS}{G_s}$

$$\gamma_t = \gamma_w \frac{G_s + eS}{1 + e}$$
 (2.19b)

Case 2. When Fully Saturated (i.e. $S = 100\%$)

From Equation (2.19b)

$$\gamma_t = \frac{\gamma_w (G_s + eS)}{1 + e}$$

when

$$S = 100\%$$

$$\gamma = \gamma_{\text{sat}} = \frac{\gamma_w (G_s + e)}{1 + e}$$
 (2.20)

Case 3. When the Soil is Dry (i.e. S = 0%)

From Equation (2.19)

$$\gamma_t = \frac{\gamma_w (G_s + eS)}{1 + e}$$

when

$$S = 0\%$$

$$\gamma_t = \gamma_{dry} = \frac{\gamma_w G_s}{1 + e} \dots\dots\dots (2.21)$$

Case 4. When the Soil is Submerged

From Equation (2.11)

$$\gamma_b = \gamma_{sat} - \gamma_w$$

but

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

Therefore

$$\gamma_b = \frac{\gamma_w (G_s + eS)}{1 + e} - \gamma_w$$

$$\gamma_b = \frac{\gamma_w (G_s - 1)}{1 + e} \dots\dots\dots (2.22)$$

2.1.7 Relative Density (D_r)

The looseness or denseness of sandy soils can be expressed numerically by the relative density (D_r), defined by the equation.

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 \dots\dots\dots (2.23)$$

In which

e_{max} = Void ratio of the soil in its loosest state

e_{min} = Void ratio of the soil in its densest state

e = Void ratio of the soil in the field

The loosest state for a granular material can usually be created by allowing the dry material to fall into a container from a funnel held in such a way that the free fall is about 1 cm. If the soil is silty a looser state can sometimes be established by mixing a sample with enough water to transform it into a thick suspension that is then allowed to settle. The value of e_{max} is equal to the final void ratio of the sediment. The densest state can usually be achieved by packing the soil into a container by means of a combination of static pressure and vibration.

There are certain practical difficulties in determining void ratios, one of which is the problem involved in measuring solid volumes. In order to overcome this difficulty relative density is sometimes expressed in terms of unit dry weight of soils as given in Eqn. (2.23) which is also reproduced in Eqn. (2.24).

$$D_r = \frac{\gamma_{Md}}{\gamma_d} \left[\frac{(\gamma_d - \gamma_{md})}{(\gamma_{Md} - \gamma_{md})} \right] \times 100 \dots\dots \quad (2.24)$$

where γ_{Md} = Maximum unit dry weight of soil

γ_{md} = Minimum unit dry weight of soil

γ_d = Dry unit of soil in place

Table 2.1 Qualitative Descriptions of Granular Soil Deposits (Terzaghi and Peck)

Relative Density (%)	Description of Soil Deposit
0 to 15	Very loose
15 to 35	Loose
35 to 65	Medium
65 to 85	Dense
85 to 100	Very dense

Example 2.1

A sample of soil had a mass of 132.2 gm. and a volume of 62.3 cc in the natural state. The dry mass is 118.2 gm. The specific gravity of soil grains is 2.67. Determine.

- Water content
- Void ratio
- Porosity
- Degree of saturation

Solution

a) Water content, ω

$$\begin{aligned} \omega &= \frac{W_w}{W_s} \times 100 \\ &= \frac{W - W_s}{W_s} \times 100 = \frac{132.2 - 118.2}{118.2} \times 100 = 11.8\% \end{aligned}$$

b) Void ratio, e

$$e = \frac{V_v}{V_s} \times 100$$

but
$$V_s = \frac{W_w}{\gamma_s} = \frac{W_s}{G_s \gamma_w} = \frac{118.2 \text{ gm}}{(2.67)(1 \text{ gm/cc})} = 44.3 \text{ cc}$$

and
$$V_v = V - V_s = 62.3 - 44.3 = 18 \text{ cc}$$

Therefore
$$e = \frac{18}{44.3} = 0.406$$

c) Porosity

$$n = \frac{V_v}{V} \times 100 = \frac{18}{62.3} \times 100 = 28.9\%$$

d) Degree of saturation

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

but
$$V_w = \frac{W_w}{\gamma_w} = \frac{(132.2 - 118.2) \text{ gm}}{1 \text{ gm/cc}} = 14 \text{ cc}$$

and
$$V_v = 18 \text{ c.c}$$

Then
$$S = \frac{14}{18} \times 100 = 77.8\%$$

Alternatively
$$S = \frac{\omega G_s}{e} = \frac{11.8 \times 2.67}{0.406} = 77.8\%$$

Example 2.2

A fully saturated sample of clay has a void ratio of 0.825 and a moisture content of 30% in its natural state. Assuming the specific gravity of the solids as 2.75, determine its dry unit weight, bulk unit weight and the submerged unit weight of the clay.

Solution

Dry unit weight, γ_{dry}

$$\gamma_{\text{dry}} = \frac{G_s \gamma_w}{1 + e} = \frac{(2.75)(10 \text{ kN/m}^3)}{1 + 0.825} = 15.07 \text{ kN/m}^3$$

Bulk unit weight, γ_t

$$\gamma_t = \frac{G_s \gamma_w (1 + \omega)}{1 + e} = \frac{(2.75)(10)(1 + 0.30)}{1 + 0.825}$$

Submerged unit weight, γ_b

$$\gamma_b = \frac{\gamma_w (G_s - 1)}{1 + e} = \frac{10(2.75 - 1)}{1 + 0.825} = 9.59 \text{ kN/m}^3$$

Example 2.3

An earthen embankment under construction has a bulk unit weight of 16 kN/m^3 and a moisture content of 10 percent. Compute the quantity of water in liters required to be added per cubic meter of earth to raise its moisture content to 14 percent at the same void ratio.

Solution

Consider a unit volume of soil (i.e. $V = 1 \text{ m}^3$)

$$\gamma_t = \frac{W}{V} = 16 \text{ kN/m}^3$$

Therefore

$$W = 16 \text{ kN/m}^3$$

$$W_s = \frac{W}{1 + w_1} = \frac{16 \text{ kN/m}^3}{1 + 0.10} = 14.55 \text{ kN/m}^3$$

$$W_{w1} = \omega_1 W_s = (0.10)(14.55) = 1.455 \text{ kN/m}^3$$

$$V_{w1} =$$

$$W_{w2} = \omega_2 W_s = (0.14)(14.55) = 2.037 \text{ kN/m}^3$$

$$V_{w2} = \frac{W_{w2}}{\gamma_w} = \frac{2.037 \text{ kN/m}^3}{10 \text{ kN/m}^3} = 0.2037 \text{ m}^3 / \text{m}^3$$

$$\begin{aligned} \text{Volume of water to be added} &= V_{w2} - V_{w1} \\ &= 0.2037 - 0.1455 \\ &= 0.0582 \text{ m}^3 / \text{m}^3 \\ &= 58.2 \text{ liters /m}^3 \end{aligned}$$

Alternatively

$$\begin{aligned} \gamma_{\text{dry}} &= \frac{\gamma_t}{1 + \omega_1} \\ &= \frac{16}{1 + 0.10} = 14.545 / \text{m}^3 \end{aligned}$$

$$\begin{aligned}\gamma_{t2} &= \gamma_{\text{dry}} (1 + \omega_2) \\ &= 14.545 (1 + 0.14) = 16.582 \text{ kN/m}^3 \\ \Delta\gamma_t &= \Delta W_w = \gamma_{t2} - \gamma_{t1} = 16.582 - 16 = 0.582 \text{ kN/m}^2 \\ \Delta V_w &= \frac{\Delta W_w}{\gamma_w} = \frac{0.582}{10} = 0.0582 \text{ m}^3 / \text{m}^3 \\ \text{Volume of water to be added} &= 58.2 \text{ liters/m}^3\end{aligned}$$

Example 2.4

A soil in the borrow pit is at a unit weight of 19.2 kN/m^3 and a moisture content of 20 percent. If a soil of 10,000 cubic meter volume is excavated from it and compacted in an embankment with a dry unit weight of 18 kN/m^3 . Calculate the volume of the embankment, which can be constructed out of this material.

Solution

$$\begin{aligned}\gamma_t &= \frac{W}{V} \\ W &= \gamma_t V = (19.2)(10,000) = 192000 \text{ kN} \\ W_s &= \frac{W}{1 + \omega} = \frac{192000}{1 + 0.20} = 160000 \text{ kN} \\ \gamma_d &= \frac{W_s}{V} \\ V &= \frac{W_s}{\gamma_d} = \frac{160000 \text{ kN}}{18 \text{ kN/m}^3} = 8889 \text{ m}^3\end{aligned}$$

Example 2.5

For a sand, the maximum and minimum possible void ratios were determined in the laboratory to be 0.94 and 0.33 respectively. Find the moist unit weight of sand in kN/m^3 compacted in the field at a relative density of 60% and moisture content of 10%. Given $G_s = 2.65$

Solution

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

$$0.60 = \frac{0.94 - e}{0.94 - 0.33}$$

$$e = 0.57$$

$$\gamma_t = \frac{G_s \gamma_w (1 + \omega)}{1 + e}$$

$$= \frac{(2.65)(10)(1 + 0.10)}{1 + 0.57}$$

$$\gamma_t = 18.57 \text{ kN/m}^3$$

2.2 GRAIN SIZE DISTRIBUTION

In this system soils are split into coarse-grained non-cohesive, fine-grained cohesive and organic soils. Most systems of soil classification depend to some extent upon the distribution of various sized particles in the soil. For coarse-grained material this distribution may be determined by sieving, and for finer particles a method of measuring the rate of settlement in water is used. The determination of particle size distribution by these methods is known as mechanical analysis.

Mechanical analysis can be divided into

1. Sieve analysis
2. Hydrometer analysis

2.2.1 Sieve Analysis

The sieve analysis is carried out by sieving a known dry weight of sample through the set of sieves placed one below the other, so that the openings decrease in size from the top sieve down, with a pan at the bottom of the stock. The whole set of sieves is given a horizontal shaking for about ten minutes until the weight of soil remaining on each sieve reaches a constant value. By determining the weight of soil sample left on each sieve, the following calculations can be made.

$$\text{Percentage retained on any sieve} = \frac{\text{Wt. of soil retained}}{\text{Total soil weight}} \times 100$$

Cumulative percentage retained on any sieve = Sum of percentage retained on all any coarser sieves

Percentage finer than any sieve size = 100 percent minus cumulative percentage retained

The results may be plotted in the form of a graph on a semi- log paper with the percentage finer on the arithmetic scale and the particle diameter on the log scale.

2.2.2 Hydrometer Analysis

Soil particle sizes smaller than 0.075 mm (passing 200 mesh sieve) are determined by the so-called hydrometer method and it is based on the process of sedimentation of soil particles in water by gravity. Sedimentation by gravity is the separation of particles of various sizes by their velocity. Velocity depends upon its specific gravity, weight, diameter, density and viscosity. Coarse soil suspension settles out more rapidly than the finer ones of the same specific gravity.

At the beginning of the hydrometer (i.e. at $t = 0$) the soil suspension is thoroughly mixed and uniform. With time the coarser particles gradually settle down. Thus, during sedimentation the density or unit weight of the suspension decreased and the hydrometer floats deeper in the suspension. It is assumed that a hydrometer actually indicated the density of the suspension at the center of bulb.

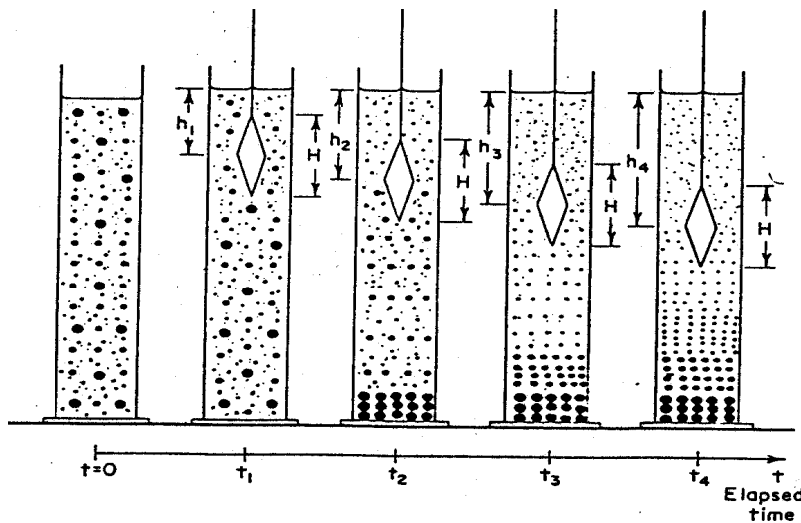


Fig. 2.3 Process of sedimentation of soil particles with time.

The hydrometer method of analysis is based on Stoke's law of settlement i.e. small spheres in a liquid settle at different rate according to the size of the sphere. The terminal velocity of a spherical soil particle settling in water is expressed by Stoke's law as

$$V = \frac{\gamma_s - \gamma_w}{18\mu} D^2 \dots\dots\dots (2.25)$$

where γ_s = Unit weight of spherical grains
 γ_w = Unit weight of water

μ = Viscosity of water

D = Diameter of the spherical particles

In practice, soil particles are never truly spherical. To overcome this, particle size is defined in terms of equivalent diameter, where equivalent diameter of a particle is the diameter of an imaginary sphere of the same material, which would sink in water with the same velocity as the irregular particle in question. It is this equivalent diameter, therefore, which is finally determined.

2.2.2.1 Laboratory Procedure for Hydrometer Analysis

50 gm of soil passing sieve No. 200 are agitated with water and dispersing agent in 1000 cc jar. The density of the suspension is then measured with stream lined hydrometer at given intervals of time. The hydrometer reading, Z_r , is observed at the surface of the fluid on a scale of the stem and this indicates the density of the suspension.

Readings are usually taken at intervals of $\frac{1}{4}$, $\frac{1}{2}$, 1 and 2 minutes with the hydrometer remaining in the suspension all the time. For later intervals, that is 5, 10, 20 etc. minutes the hydrometer is put in the suspension just before reading and removed after each reading.

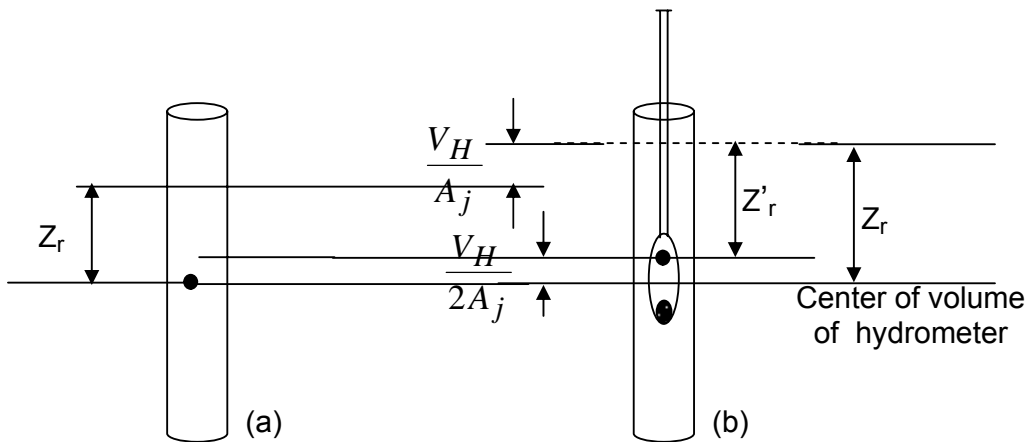
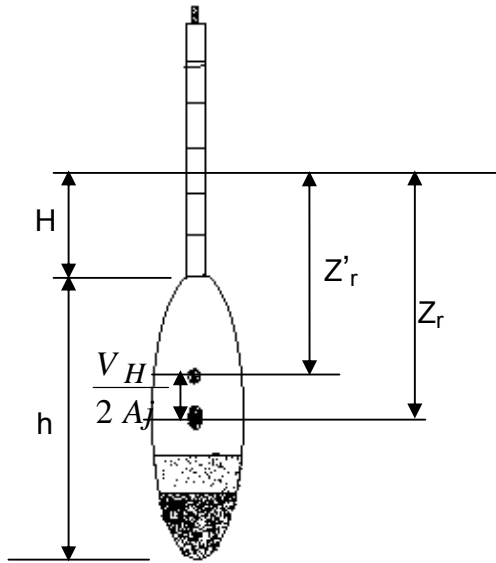


Fig. 2.4 a) Before the immersion of hydrometer, b) After the immersion of Hydrometer



$$Z_r = H + \frac{1}{2} h$$

$$Z'_r = Z_r - \frac{V_H}{2A_j}$$

$$= H + \frac{1}{2} \left\{ h - \frac{V_H}{A_j} \right\}$$

2.4 (c) Hydrometer

If a soil particle of size “D” falls through a distance Z_r in time t , its velocity will be given as

$$v = \frac{Z_r}{t}$$

Using Stoke’s law

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

$$D = \sqrt{\frac{v 18\mu}{\gamma_s - \gamma_w}} = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w} \times \frac{Z_r}{t}} \dots\dots\dots (2.26)$$

After a time t , all particles of diameter equal to and larger than D have settled through a depth Z_r . All the remaining particles finer than D are still in suspension and their concentration need to be determined.

Equation (2.26) applies to early readings taken between $\frac{1}{4}$ and 2 minutes intervals. For the readings taken at intervals of 5, 10, 20 etc. minutes, emersion correction is applied to equation (2.26).

When the hydrometer is placed in the jar, it displaces its own volume as shown in Fig. 2.4b. As a result, the surface of the suspension rises. If the hydrometer has a volume = V_H and the jar has a cross-sectional area = A_j , then the surface of the suspension as indicated on the above figure will rise by $\frac{V_H}{A_j}$. Since the displacement below the center of the bulb is

due to half of its volume, the suspension now at the center must previously be lower by

$$\frac{1}{2} \frac{V_H}{A_j}$$

Therefore, actual distance of settlement

$$Z'_r = Z_r - \frac{V_H}{2A_j}$$

$$D = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w}} \sqrt{\frac{z_r - V_H / 2A_j}{t}} \dots\dots\dots (2.27)$$

Z_r can be obtained from the calibration curve, which is a plot of Z_r against hydrometer reading.

The percentage finer, N, can be obtained from

$$N = \frac{G_s}{G_s - 1} \left(\frac{V}{W_s} \right) (\gamma_w (r - r_w)) \times 100 \dots\dots\dots (2.28a)$$

In which

G_s = Specific gravity of solids

V = Volume of suspension

γ_w = Unit weight of water at temperature (usually 20⁰c) of hydrometer calibration

r = Hydrometer reading in suspension

r_w = Hydrometer reading in water at the temperature as suspension

For combined analysis (i.e. sieve analysis + hydrometer analysis) the percentage finer, N, can be calculated from the following equation.

$$N' = \frac{N \times W_1}{W} \dots\dots\dots (2.28b)$$

where

W_1 = Weight of soil finer than No. 200 sieve

W = Total soil weight used in combined analysis

N' = Percentage finer based on entire soil sample, W

2.2.3 Grain Size Distribution Curves

Typical sets of grain size distribution curves are given in Fig. 2.5 with the grain size, D as abscissa on the logarithmic scale and the percent finer N as ordinate on the arithmetic scale. On the curve I the section AB represents the portion obtained by sieve analysis and the section BC by hydrometer analysis.

The shapes of the curves indicate the nature of the soil tested. On the basis of the shapes one can classify soils as

1. Uniformly graded or poorly graded
2. Well graded
3. Gap graded

Uniformly graded soils are represented by nearly vertical lines as shown by curve II in Fig. 2.5. Such soils will easily be displaced under load and have less supporting power.

A well-graded soil, represented by curve I, possesses a wide range of particles sizes ranging from gravel to clay size particles. Such soil is relatively stable, resistant to erosion, can readily be compacted to a very dense condition, and will develop high shearing resistance and bearing capacity.

A gap-graded soil, as shown by curve III has some of the sizes of particles missing. On this curve the soil particles falling the range XY are missing.

Uniformity Coefficient. The above discussion indicates that the soil gradation is reflected in the shape and slope of the grain size or gradation curve. A steep slope indicates poor gradation for most engineering purposes, a gentle slope, even slope, good graduation. To determine whether a material is uniformly graded or well graded, Hazen proposed the following equation

$$C_u = \frac{D_{60}}{D_{10}} \dots\dots\dots (2.29)$$

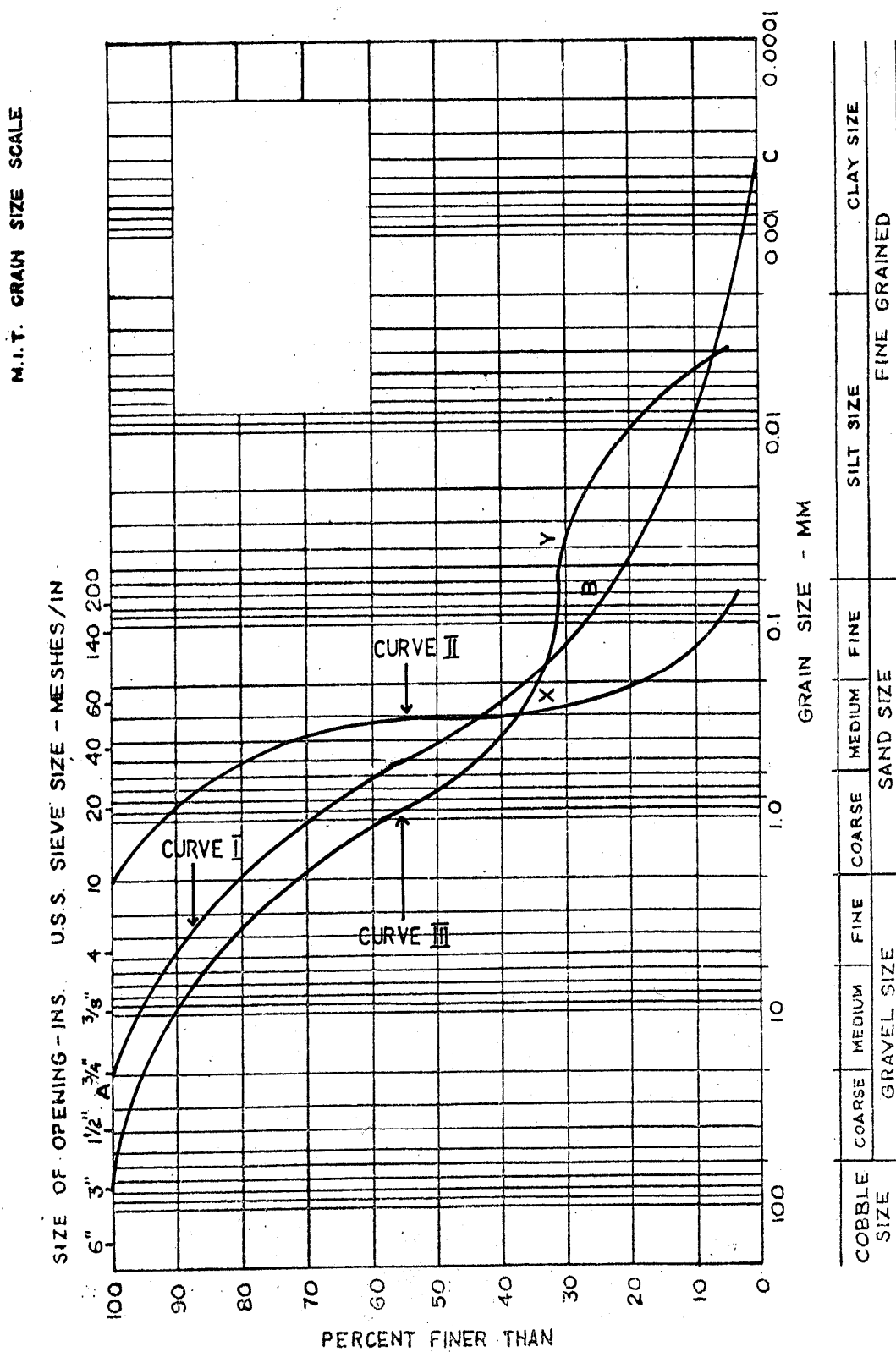
- where
- C_u = Uniformity coefficient
 - D_{60} = 60% size
 - D_{10} = Effective size, or 10% size

The uniformity coefficient C_u , is about one, usually indicates a soil in which the grains are practically of the same size (uniform soil). A large coefficient represents a well-graded soil.

2.3 SOIL CONSISTENCY

Consistency is a term used to indicate the degree of firmness of fine-grained soils. The consistency of natural fine-grained soil deposits is expressed by such terms as soft, stiff and hard. The physical properties of fine-grained soils greatly differ at different water contents. Soil that is soft at a higher percentage of water content becomes hard with the decrease of water content. Consistency of a soil can be expressed in terms of Atterberg limits of soils.

Fig. 2.5 Grain size distribution curves



2.3.1 Atterberg Limits

The Swedish Scientist, Atterberg (1911), developed a method of describing quantitatively the effect of varying water content on the consistency of fine-grained soils. He established the four states of soil consistency, which are called the liquid, the plastic, the semi-solid, and the solid states. He also proposed a series of tests for determining the boundaries known as Atterberg limits between the physical states of soil. Each boundary or limit is defined by the water content that produces a specified consistency. The liquid state is produced when a fine-grained soil is mixed with a large quantity of water. In such state the soil behaves like a liquid. That is, it flows freely like a liquid and has no resistance to deformation. If, however, its water content is gradually reduced, it will begin to show a small shearing strength. The limit at which soil suspension passes from no strength to a very small strength is the liquid limit. The limit is defined by moisture content of soil at that point and is designated by ω_ℓ .

At moisture content lower than its liquid limit, the soil is in a plastic state. If the sample is subjected to a further decrease in moisture content, it will eventually lose its plasticity. The moisture content at which the sample, when it is rolled into a thread, starts to crumble rather than distort plastically, is called its plastic limit and is designated by ω_p .

After the plastic limit, the soil displays the properties of semi-solid. With a further decrease in moisture content, the soil sample will finally reach a point where it can no longer change in volume. At this point, the soil is said to have reached its shrinkage limit designated by ω_s .

The limits described above are all expressed by their percentage water contents.

States	Limits	Indexes	Volume Change
Liquid	Liquid limit, ω_ℓ	Plasticity index $P_I = \omega_\ell - \omega_p$	Decrease in Volume
Plastic			
Semi-solid	Plastic limit, ω_p	Shrinkage index $= \omega_p - \omega_s$	Constant in Volume
Solid	Shrinkage limit, ω_s		

2.3.2 Determination of Atterberg Limits

Liquid Limit (ω_ℓ):- The apparatus shown in Fig. 2.6 is the liquid limit device used for determining the liquid limits of soils. The device contains a brass cup, which could be raised and allowed to fall on a hard rubber base by turning the handle. The cup is raised by one cm.

The limits are determined on that portion of soil finer than a No. 40 sieve (ASTM). About 100 gm of soil is mixed thoroughly with distilled water into a uniform paste. A portion of the paste is placed in the cup and leveled. A groove is cut at the center of the soil pat, using the standard grooving tool (Fig.2.6b). The handle is next turned at a rate of about two revolutions per second and the number of blows necessary to close the groove along the bottom for a distance of 12 mm is counted. The water content of the soil taken near the closed groove is found out. The water content of the soil in the cup is altered and the tests repeated. At least four tests should be carried out by adjusting the water contents in such a way that the number of blows required to close the groove may fall within the range of 10 to 40 blows. A plot of water content against the log of blows is made as shown in Fig. 2.7. Within the range of 10 to 40 blows, the plotted points lie almost on a straight line. The curve so obtained is known as a “flow curve”. The water content corresponding to 25 blows is termed as liquid limit. The equation of the flow curve can be written as

$$\omega = - I_f \log N + C \quad \dots\dots\dots (2.30)$$

where

- ω = Water content
- I_f = Slope of the flow curve, termed as flow index
- N = Number of blows
- C = a constant

Plastic Limit (ω_p): - About 15 gm of soil passing through sieve No. 40 (ASTM), mixed thoroughly with water. The soil is rolled on a glass plate with the hand, until it is about 3 mm in diameter. This procedure of mixing and rolling is repeated till the soil shows signs of crumbling when the diameter is 3 mm. The water content of the crumbled portion of the thread is determined. This is called as plastic limit.

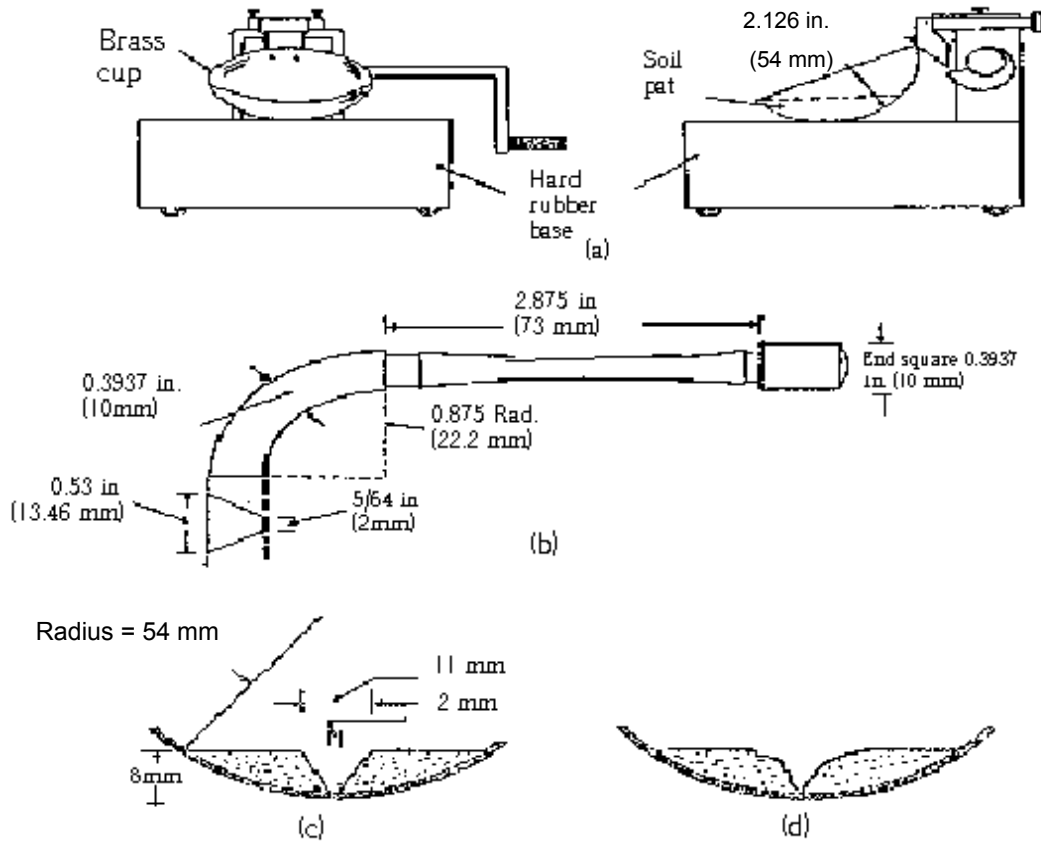


Fig. 2.6 Liquid Limit test; (a) Liquid limit device (b) ASTM Grooving tool; (c) Soil pat before test; (d) Soil pat after test

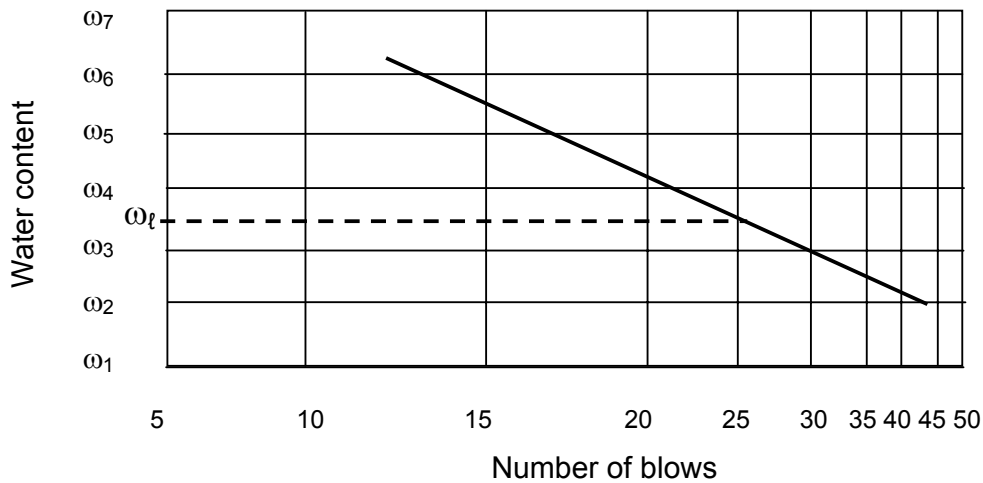


Fig. 2.7 Flow curve for liquid limit

Shrinkage Limit (ω_s): - For determination of shrinkage limit, a container of known volume V_1 is filled with plastic soil in the saturated state. The weight of the saturated soil is determined. The specimen is dried gradually first in air and then in an oven at a constant temperature of 105°C . After oven drying the specimen weighed. Following the weighing, its volume is determined by displacement in mercury.

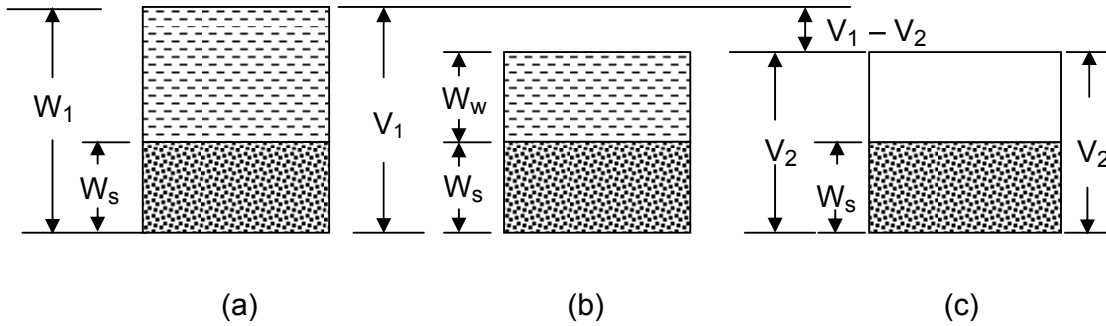


Fig. 2.8 Determination of shrinkage limit.

Fig. 2.8a represents saturated soil in a container of volume V_1

Fig. 2.8b represents the saturated soil at shrinkage limit

Fig. 2.8c represents the dry soil after oven drying.

W_1 = Weight of saturated soil

W_s = Weight of dry soil

W_w = Weight of water in saturated soil at shrinkage limit

V_1 = Volume of saturated soil

V_2 = Volume of dry soil

$$\text{Shrinkage limit } \omega_s = \frac{W_w}{W_s} = \frac{W_1 - W_s - \gamma_w (V_1 - V_2)}{W_s} \times 100 \dots \quad (2.31)$$

Plasticity Index (P_I):- Is the range of water content over which the soil exhibits plasticity.

$$P_I = \omega_l - \omega_p \dots \dots \dots \quad (2.32)$$

Greater the difference between liquid limit and plastic limit is the greater the plasticity of the soil. A cohesionless soil has zero plasticity index. Such soil as termed as non-plastic. Fat clays are highly plastic and possess a high plasticity index. Soils possessing large values of liquid limit and plasticity index are said to be highly plastic or fat. Atterberg classifies the soils according to their plasticity indices as follows

Plasticity index	Plasticity
0	Non-plastic
< 7	Low plastic
7-17	Medium
> 17	Highly plastic

Liquidity Index (LI): - The Atterberg limits are found for remoulded soil samples. These limits as such do not indicate the consistency of undisturbed soils. The index that is used to indicate the consistency of undisturbed soils is called as the liquidity index or water plasticity ratio. The liquidity index is expressed as

$$LI = \frac{\omega - \omega_p}{P_L} \dots\dots\dots (2.33)$$

Where, ω is the natural moisture content of soil in the undisturbed state. The liquidity index of undisturbed soil can vary from less than zero to greater than one. The value of LI varies according to the consistency of soils as follows

Consistency	Liquidity index
Semi solid or solid state	Negative
Very stiff state ($\omega = \omega_p$)	0
Very soft state ($\omega = \omega_l$)	1
Liquid state (when disturbed)	>1

Relative Consistency (C_r): - of a soil is defined as the ratio between liquid limit minus the natural water content of a soil and its plastic index.

$$C_r = \frac{\omega_l - \omega}{P_L} \dots\dots\dots (2.34)$$

The relative consistency indicates the consistency (firmness) of a soil. It shows the nearness of its natural water content to its plastic limit. A soil with a relative consistency of zero is at the liquid limit. It is extremely soft and has negligible shear strength. On the other hand, a soil at a natural water content equal to the plastic limit has a relative consistency of one, indicating that the soil is relatively firm. A relative consistency of greater than one shows that the soil is relatively strong, as it is the semi-solid state. A negative value of relative consistency indicates that the natural water content is greater than the liquid limit.

It is worth noting that the sum total of the liquidity index and the relative consistency is always equal to one, indicating that a soil having a high value of liquidity index has a low value of relative consistency and vice-versa. From this one can deduce that liquidity index and relative consistency are just two ways of representing the same properties of a soil.

Example 2.6

In a hydrometer analysis the following observations were taken:

$$t = 4 \text{ min.}$$

$$r = 1.015$$

The weight of solids used in suspension of 1000 cc was 0.50 N.

If the specific gravity of the solid particles is 2.60, calculate the coordinates of the point on the grain-size plot. Other particulars of the hydrometer and the jar are as follows:

$$V_H = 50 \text{ cm}^3$$

$$h = 20 \text{ cm} \quad \mu = 10 \times 10^{-8} \text{ N-sec/cm}^2$$

$$H = 2 \text{ cm}$$

$$A_j = 50 \text{ cm}^2$$

Solution

Using Stoke's law

$$V = \frac{\gamma_s - \gamma_w}{18\mu} D^2 t$$

$$D = \sqrt{\frac{V 18\mu}{\gamma_s - \gamma_w}} = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w}} \sqrt{\frac{Z_r}{t}}$$

For readings taken greater than 2 minutes

$$D = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w}} \sqrt{\frac{z_r - V_H / 2A_j}{t}}$$

but

$$Z_r = H + \frac{1}{2} h = 2 + \frac{20}{2} = 12 \text{ cm}$$

Therefore

$$D = \sqrt{\frac{18 \times 10 \times 10^{-8}}{2.6 - 1}} \sqrt{\frac{12 - \frac{50}{2(50)}}{4}}$$

$$= 1.80 \times 10^{-3} \text{ cm} = 0.018 \text{ mm}$$

Percent finer,

$$N = \frac{G_s}{G_s - 1} \left(\frac{V}{W_s} \right) (\gamma_w (r - r_w)) \times 100$$

$$= \frac{2.60}{2.60 - 1} \left(\frac{1000}{50} \right) (1.015 - 1) \times 100$$

$$= 48.75 \%$$

(0.018 mm, 48.75%)

Example 2.7

A sample of clay soil has a liquid limit of 62% and its plasticity index is 32 %.

1. What is the state of consistency of the soil if the soil in its natural state has a water content of 34%?
2. Calculate the shrinkage limit if the void ratio of the sample at the shrinkage limit is 0.70. Given $G_s=2.70$

Solution

i) Plastic limit, $\omega_p = \omega_l - P_I = 62 - 32 = 30\%$

$\omega_l = 62\%$ $\omega_p = 30\%$ ω_s

Liquid State	Plastic State	Semi-solid State	Solid State
-----------------	------------------	---------------------	----------------

Since the natural water content of the given soil lies between the liquid and plastic limits, it is in the plastic state.

ii)
$$e = \frac{\omega G_s}{s} = \frac{\omega G_s}{100\%}$$

$$\omega = \frac{e}{G_s} = \frac{0.70}{2.70} = 25.9\%$$

EXERCISES

2.1 A clay sample is found to weigh 4.24 N in its natural state. It is then dried in an electric oven at 105°C . The dried weight is found to be 3.38 N. The specific gravity of the solids is 2.70 and the unit weight of the soil mass in its natural state is 17kN/m^3 . Determine the water content, degree of saturation and dry unit of the sample in its natural state.

2.2 A soil sample, taken from a depth that is located below the ground water table, has a volume of 75 cc and weight 1.2 N. It is dried cut and found to weigh 0.75 N. Compute its unit weight, moisture content, void-ratio, porosity and specific gravity of the solid particles.

2.3 A soil sample has a moist unit weight of 21kN/m^3 and the degree of saturation is 80 percent. Determine its void ratio and water content, if the specific gravity of the solids is 2.70.

2.4 A clay fill has a bulk density of 19kN/m^3 . If the moisture content is 25 percent and the specific gravity of the solid particles is 2.70, determine the degree of saturation.

2.5 A soil has a unit weight of 20.5 kN/m^3 and a water content of 15 percent. What will be the water content if the soil dries out to a unit weight of 19.5kN/m^3 and the void ratio remains unchanged?

2.6 Earth is required to be excavated from a borrow pit for building embankment. The unit weight of undisturbed soil in wet condition is 18kN/m^3 and its water content is 8 percent. In order to build a 4 m high embankment with top width 2 m and side slopes 1:1 estimate the quantity of earth required to be excavated per meter length of embankment. The dry density required in the embankment is 15 kN/m^3 with moisture content of 10 percent. Assume the specific gravity of solids as 2.67. Also determine the void ratio and the degree of saturation of the soil in both the undisturbed and remolded state.

2.7 A soil in borrow pit is at a dry density of 17 kN/m^3 and with a water content of 12 percent. If soil of 2000 cubic meter volume is excavated from it and compacted in an embankment with a void ratio of 0.32, calculate the volume of the embankment, which can be constructed out of this material. (Assume $G_s = 2.65$)

2.8 An earthen dam is going to be constructed by compacting a soil to a void ratio of 0.7. The total volume of soil in the dam is estimated to be $4 \times 10^6\text{ m}^3$. There are three alternatives borrow pits with the average void ratio of the soil in them and moving cost is shown below. Choose the economical borrow pit and calculate the cost of moving for it.

<u>Pit</u>	<u>Void Ratio</u>	<u>Cost of Moving</u>
A	0.8	3.15 Birr/m ³
B	0.9	2.88 Birr/m ³
C	1.0	2.52 Birr/m ³

2.9 A sample of sand in its natural state has a relative density of 65 percent. The dry unit weights of the sample at its densest and loosest states are 18kN/m³ and 14 kN/m³ respectively. Assuming the specific gravity of the solids as 2.64, determine

- Its dry unit weight
- Moist unit weight when fully saturated, and
- Submerged unit weight

2.10 For a sand, the maximum and minimum dry unit weights were determined in the laboratory to be 20 kN/m³ and 13.6kN/m³, respectively.

- Find the moist unit weight of sand compacted in the field at a relative density of 50 percent and a moisture content of 15 percent.
- How much a 5m stratum of this sand is settle if it is densified to a relative density of 70 percent

$$\text{Hint } \Delta H = \frac{H\Delta e}{1 + e_1}$$

2.11 The Atterberg limits of a particular soil are reported as liquid limit (ω_l) = 60%, shrinkage limit (ω_s) = 40% and plastic limit (ω_p) = 35%. Are these values reasonable? Explain.

2.12 A saturated soil with a volume of 19.65 cc has a mass of 36 gms. When the soil dried, its volume and mass were 13.5 cc and 25 gms, respectively. Determine the shrinkage limit for the soil.

2.13 A 100 cc clay sample has a natural water content of 30%. It is found that the shrinkage limit occurs when the water content is 19%. If the specific gravity of soil solids is 2.70, what will the volume of the sample when the water content is 15%?

2.14 A clay is found to have a liquid limit of 75% and a shrinkage limit of 25%. If a sample of this has a total volume of 30 cc at the liquid limit and a volume of 16.7 cc at the shrinkage limit, what is the specific gravity of soil solids?

CLASSIFICATIONS AND FIELD IDENTIFICATION OF SOILS

3.1 INTRODUCTION

If an area is studied from soil-engineering point of view, it may contain several soil deposits of different types that in general exhibit different soil characteristics. Natural soil deposits are never homogeneous in their nature. A soil is said to be homogeneous, if it has similar properties at all points in the given mass and it is impossible to find a natural deposit that has similar properties at all points within the deposit. Within a deposit, the soil varies from point to point. A deposit may constitute of a mixture of soils of many types and sizes. Soil properties may vary with the depth of the deposit as well as its extent in the horizontal direction. Such variations in soil properties in a deposit may be termed as local variations. In spite of such local variations, the deposit as a whole may have average properties that might be the same for all portions of the deposit.

Deposits that exhibit, in general, similar average properties can be grouped together, as a class. Classification of soil is essential because, by classifying a soil, a fairly accurate idea of its average properties, can be made and an estimation of these average properties just by classification is helpful in ordinary soil engineering projects. The opinion, however on the applicability of classification of soils differs among soil-engineers. The difference of opinion arises mainly due to the fact that many classification systems exist. Some systems take into consideration only the grain-sizes of the soils, while other take into consideration the plastic properties, while quite another classification exists, which defines soils in terms of their origin and general features. So, a soil classified into a group according to one system may fall into quite another group of another system. Also in the same system there is a possibility that a soil may form a borderline case, in which case, it could be classified under two groups.

Classification of a soil is a mere indicator to its general properties and behavior of a soil cannot be solely predicted from its classification alone. Certain other important tests and parameters would, of course, be necessary before the conclusions about its behavior as a construction or foundation material could be deduced. In spite of the inadequacies pointed out above, it is essential that a soil for an engineering project is classified and then further tests made to predict its suitability for the project.

3.2 CLASSIFICATIONS OF SOILS

A soil classification system is the arrangement of different soils having similar properties into groups and sub-groups based on their application. It provides a common language to express briefly the general characteristics of soils. There are several methods of classifying soils. The most widely used classification systems by engineers are described hereunder.

3.2.1 Grain Size Classification

It is based on grain size of the soils and is essentially useful for classifying soils in which single grain properties are of importance. A mechanical analysis is all that is required to classify a given sample of soil. Three of the grain size classifications that are in common use are the following.

(a) U.S. Bureau of Public Roads Classification

Gravel	Sand		Silt	Clay
	Coarse	Fine		
2	0.25	0.05	0.005	

Grain size in mm

(b) M.I.T* Classification

Gravel	Sand			Silt			Clay
	Coarse	Medium	Fine	Coarse	Medium	Fine	
2	0.6	0.2	0.06	0.02	0.006	0.002	

Grain size in mm

*Massachusetts Institute of Technology

(c) Textural Classification

This classification is based on a triangular chart shown in Fig. 3.1. A soil with known percentage of sand, silt and clay sizes is represented by a given point on a triangular chart of this type.

The use of this triangular chart can best be demonstrated by an example. If the particle-size distribution of a soil shows 35% sand -, 25% silt-, and 40% clay-size particles, its textural classification can be determined by proceeding in a manner as shown by the broken lines in

Fig. 3.1. This soil falls into the zone of Sandy clay. Note that this chart is based only on the fraction of soil passing a No. 10 sieve (particles ≤ 2 mm). Hence, if the particle-size distribution of a soil shows a certain percentage of soil particles > 2 mm in diameter, a correction is necessary. For example, if a soil has particle-size distribution 10% gravel, 20% sand, 30% silt, and 40% clay, the corrected textural compositions are:

$$\text{Sand-size} = \frac{20 \times 100}{100 - 10} = 22.22\%$$

$$\text{Silt-size} = \frac{30 \times 100}{100 - 10} = 33.33\%$$

$$\text{Clay-size} = \frac{40 \times 100}{100 - 10} = 44.44\%$$

Based on the above-corrected percentages, the textural classification is silty clay.

Textural classification is more suitable for describing coarse-grained soils rather than clay soils, whose properties are less dependent on particle size distribution.

3.2.2 Soil Classification Based on both Grain-size and Atterberg limits.

Though textural classification of soil is relatively simple, it is based entirely on particle-size distribution. The physical properties of fine-grained soils are dictated to a great extent by the amounts and types of clay minerals present in them. Hence, for proper interpretation of soil characteristics, the plasticity that is the result of the presence of clay minerals needs to be considered. Since textural classification systems do not take plasticity into account and are based on particle-size distribution alone, they are not totally indicative of many important soil properties and hence are inadequate for most engineering purposes. Presently, two more elaborate classification systems that take into consideration both particle-size distribution and Atterberg limits are commonly used. They are (1) The AASHTO (American Association of State Highway and Transport Official) classification system and (2) The unified soil classification system

3.2.2.1 AASHTO Classification System

The AASHTO classification system, also called Public Roads Administration (PRA) classification, is based on the following three soil properties:

- Grain-size distribution.
- Liquid limit
- Plasticity index

This system is generally used by highway engineers, for classification of subgrade soils for the highway pavements. This classification has undergone various modifications, and the latest version one is based on the proposals of the Highway Research Board (1945).

The AASHTO classification in present use is given in Table 3.1. According to this system, soil is classified into seven major groups, A-1 through A-7. Soils classified under groups A-1, A-2, and A-3 are granular materials with 35% or less passing through a No. 200 sieve. Soils with more than 35% passing a No. 200 sieve are classified under groups A-4, A-5, A-6 and A-7. These soils are mostly silt and clay-type materials. To classify a soil using Table 3.1, one must proceed from left to right with required test data available. By the process of elimination, the first group from the left into which the test data will fit gives the correct classification.

Under this classification system, a characteristic called group index (G.I) is used to describe the performance of a soil when used as a highway subgrade material. Group index is not used to place the soil in a particular group, but is actually a means of rating the value of a soil as a subgrade material, within its own group. The group index of a soil may range from 0 to 20 and is expressed as a whole number. The approximate subgrade performance quality of a give soil is inversely proportional to its group index, and it can be expressed by the following empirical relation,

$$GI = 0.2a + 0.005ac + 0.01bd \dots\dots\dots (3.1)$$

- where
- a = that portion of percentage of particles passing No.200 sieve greater than 35% and not exceeding 75%, expressed as positive whole number (0 to 40).
 - b = that portion of percentage of particles passing No.200 sieve greater than 15% and not exceeding 55 %, expressed as positive whole number (0 to 40).
 - c = that portion of the liquid limit greater than 40 and not exceeding 60 , expressed as positive whole number (0 to 20).
 - d = that portion of the plasticity index greater than 10 and not exceeding 30, expressed as positive whole number (0 to 20).

3.2.2.2 Unified Soil Classification System

The unified system of soil classification was originally proposed by A. Casagrande in 1942 and was then revised in 1952 by the Corps of Engineers and the U.S. Bureau of Reclamation. The system has also been adopted by American Society of Testing Material (ASTM). The system is most popular for use in all types of engineering problems involving soils.

The unified classification system is based on recognition of the type and predominance of the constituents considering grain size, gradation and plasticity. It divides soil into three

major divisions; coarse-grained soils, fine-grained soils, and highly organic (peat) soils. In the field, identification is accomplished by visual examination for the coarse grains and a few simple hand tests for the fine-grained soils. In the laboratory, the grain-size curve and the Atterberg limits can be used. The peat soils are readily identified by color, odor, spongy feel and fibrous texture. The name and symbols used to distinguish between the typical and boundary soil groups are as follows:

a) Coarse-grained soils

G = Gravel

S = Sand

The gravel and sand are further divided into subgroups as;

W = Well graded

P = Poorly graded

b) Fine-grained soils

M = Inorganic silt

C = Inorganic clay

O = Organic silts and clays

P_t = Peat

The above groups of fine-grained soils (excluding peat) are further subdivided according to their liquid limits into

L = Low plasticity ($\omega_L < 50\%$)

H = High plasticity ($\omega_L > 50\%$)

The symbols indicated above are combined to form the group symbol. Table 3.2 gives the groupings of soils for coarse and fine-grained soils. The classification of fine-grained soils is easier to understand on plasticity chart, which is described hereunder.

Plasticity Chart: - The plasticity chart (shown in bottom right-hand portion of Table 3.2) is a plot of plasticity index versus liquid limit. Fine-grained soils are subdivided into soils of low, medium and high plasticity following the criteria cut-lined below

Low plasticity ($\omega_L < 30\%$)

Medium plasticity (ω_L between 30% and 50%)

High plasticity ($\omega_L > 50\%$)

The diagonal line drawn in the plasticity chart is called the "A" line and is given by the equation $P_I = 0.73 (\omega_L - 20)$. Clays fall above the A-line and silts below it.

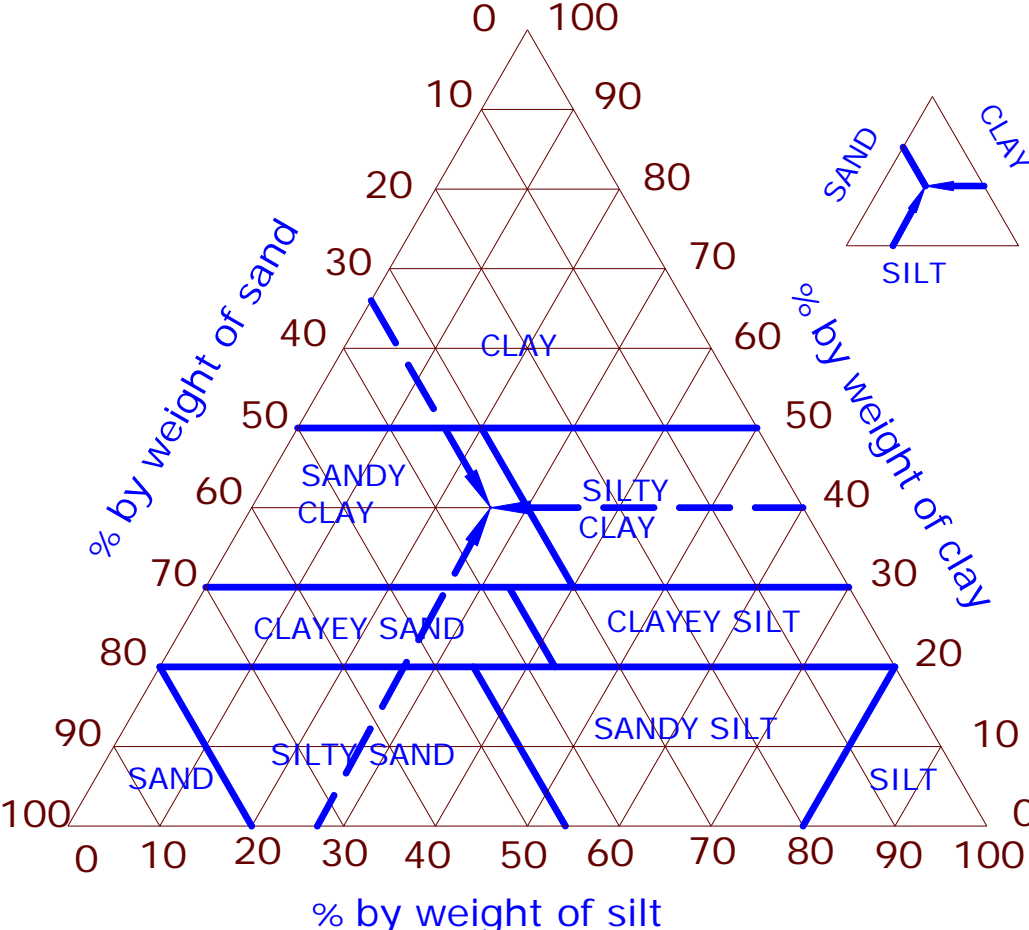


Fig. 3.1 Triangular classification chart

Table 3.1 AASHTO Soil Classification System

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

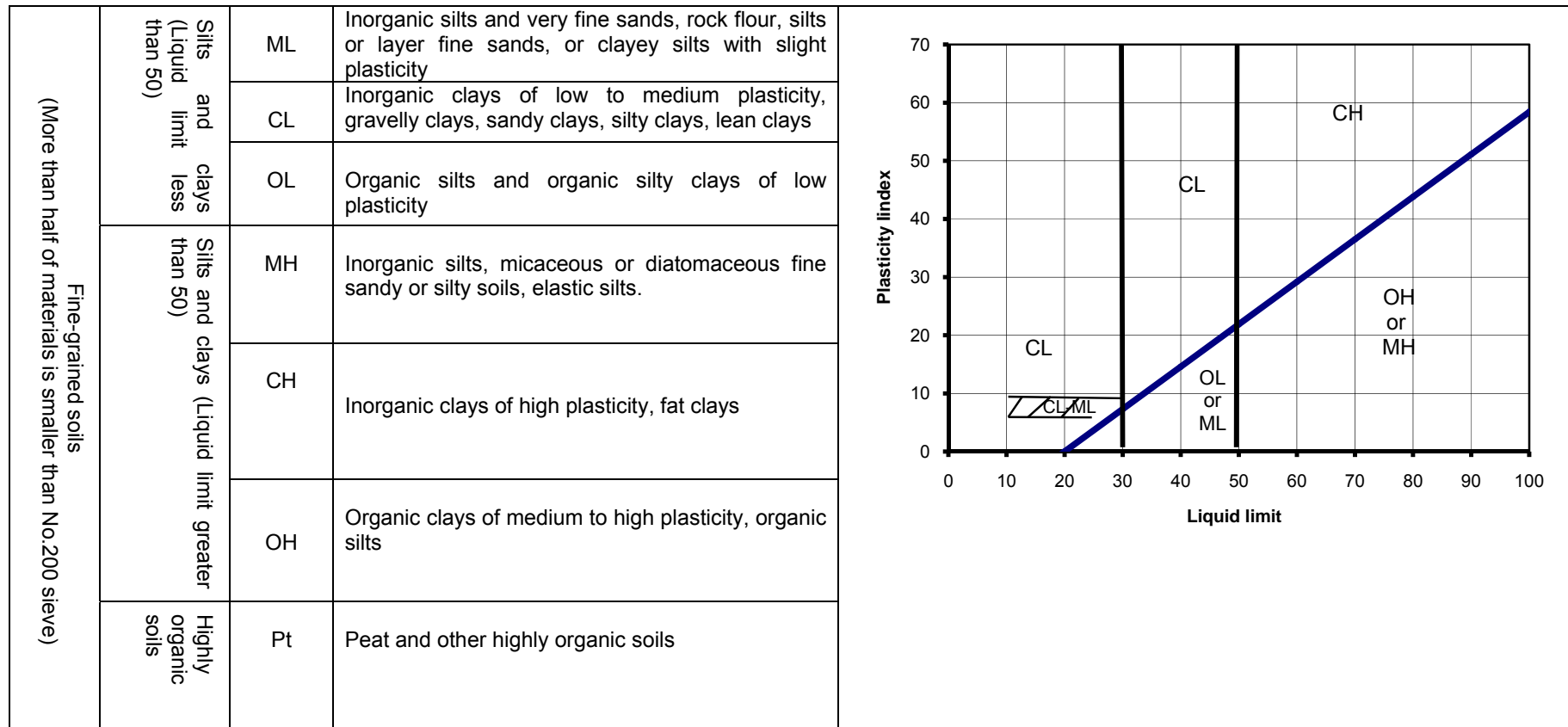
The A-7 group is subdivided into A-7-5 or A-7-6 depending on the plastic limit. For P.L.<30, the classification is A-7-6; for P.L ≥30, it is A-7-5.

Table 3.2 Unified Soil Classification System

Major divisions		Group symbols	Typical names	Laboratory classification criteria	
Coarse-grained soils (More than half of materials is larger than No.200 sieve size)	Gravels (More than half of coarse fraction is larger than No.4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixture, little or no fines.	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4, $C_c = \frac{D_{30}^2}{D_{10} \times D_{60}}$ between 1 and 3 Not meeting all gradation requirements for GW Atterberg limits below "A" line or PI less than 4 Atterberg limits above "A" line with PI greater than 7 $C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 Not meeting all gradation requirements for SW Atterberg limits below "A" line or PI less than 4 Atterberg limits above "A" line or PI greater than 7 Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	
		GM*	d	Silty gravels, gravel-sand silt mixtures	
			u		
	GC	Clayey gravels, gravel-sand-clay mixtures			
	Sands (More than half of coarse fraction is smaller than No.4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	
			SP	Poorly graded sands, gravelly sands, little or no fines	
		SM*	d	Silty sands, sand-silt mixtures	
			u		
		SC	Clayey sands, sand-clay mixtures		
Sands with fines (Appreciable amount of)					

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows,
 Less than 5 percent GW, GP, SW, SP
 More than 12 percent GM, GC, SM, SC
 Borderline case requiring dual symbols **

Table 3.2 Unified Soil Classification System Cont'd



* Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when w_L is 28 or less and the PI is 6 or less; the suffix u used with w_L is greater than 28.

** Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

3.3 FIELD IDENTIFICATION OF SOILS

Soils can be identified in the field by conducting the following simple tests. The sample is first spread on a flat surface. If more than 50% of the particles are visible to the naked eye, the soil is coarse-grained; otherwise, it is fine-grained. The fine-grained particles are smaller than 0.075mm size and are not visible to naked eye. The techniques used for further identification of soils in the field can conveniently be discussed under the headings of coarse-grained and fine-grained soils.

1. Coarse-grained Soil

Coarse-grained soils are mineral fragments that may be identified primarily on the basis of grain size. The different constituents of coarse-grained soils are sand and gravel. As described in the earlier chapters the size of sand varies from 0.075 mm to 2 mm and that of gravel from 2 mm to 60mm. The sand can further be classified as coarse, medium and fine. Engineers should have an idea of the relative sizes of the grains in order to identify the various fractions. The description of sand and gravel should include an estimate of the quantity of material in the different size ranges as well as a statement of the shape and mineralogical composition of the grains. The mineral grains can be rounded, sub-rounded, sub-angular, or angular. The presence of mica or a weak material such as shale affects the durability of compressibility of the deposit. A small magnifying glass can be used to identify the small fragments of shale or mica. The property of a coarse grained material mass depends also on the uniformity of the sizes of the grains. Well-graded sand is more stable as a foundation base as compared to a uniform or poorly graded material.

2. Fine Grained Soils

Inorganic Soils: - The constituent parts of fine-grained materials are silt and clay fractions. Since both these materials are microscopic in size, physical properties other than grain size must be used as criteria for field identification. The classification tests used in the field for preliminary identification are;

1. Dry strength test
2. Shaking test
3. Plasticity test

Dry Strength Test

The strength of a soil in a dry state is an indication of its cohesion and hence of its nature. It can be estimated by crushing a 3mm size of a dried fragment between thumb and forefinger. A clay fragment can be broken only with a great effort, whereas a silt fragment crushes easily.

Shaking Test

Shaking test is also called as dilatancy test. It helps to distinguish silt from clay since silt is more permeable than clay. In this test a part of soil mixed with water to a very soft consistency is placed in the palm of the hand. The surface of the soil is smoothed out with a knife and the soil pat is shaken by tapping the back of the hand. If the soil is silt, the water will rise quickly to the surface and give it a shiny glistening appearance. If the pat is then deformed either by squeezing or by stretching, the water will flow back into the soil and leave the surface with a dull appearance. Since clay soils contain much smaller voids than silts and are much less permeable, the appearance of the surface of pat does not change during the shaking test. An estimate of the relative proportions of silt and clay in an unknown soil mixture can be made by noting whether the reaction is rapid, slow or non-existent.

Plasticity Test

If a sample of moist soil can be manipulated between the palms of the hands and fingers and rolled into a long thread of about 3mm diameter, the soil then contains a significant amount of clay. Whereas silt cannot be rolled into a thread of 3mm diameter without sever cracking.

Organic Soils

Surface soils and many underlying formations may contain significant amounts of solid matter derived from organisms. While shell fragments and similar solid matter are found at some locations, organic material in soil is usually derived from plant or root

growth and consists of almost completely disintegrated matter, such as much or more fibrous material, such as peat. The soils with organic matter are weaker and more compressible than soils having the same mineral composition but lacking in organic matter. The presence of an appreciable quantity of organic material can usually be recognized by the dark-grey to black colour and odour of decaying vegetation, which it lends to the soil.

Organic Silt

It is a fine-grained more or less plastic soil containing mineral particles of silt size and finely divided particles of organic matter. Shells and visible fragments of partly decayed vegetative matter may also be present.

Organic Clay

It is a clay soil, which owes some of its significant physical properties to the presence of finely divided organic matter. Highly organic soil deposits such as peat or muck may be distinguished by a dark-brown to black color, by the presence of fibrous particles of vegetable matter in varying states of decay. The organic odor is a distinguishing characteristic of the soil. The organic odor can sometimes be distinguished by a slight amount of heat.

Example 3.1

A sample of inorganic soil has the following grain size characteristics:

Sieve No	Percent passing
10	100
40	80
200	40

The liquid limit is 65 percent and the plasticity index 22 percent; classify the soil by AASHTO system.

Solution

Using Table 3.1 (since 40% of the soil is passing through No. 200 sieve), it falls under the silt – clay type of material, i.e., A-4, A-5, A-6 or A-7. Proceeding from left to right, it falls under group A-7. It may be A-7-5 or A-7-6 which is decided as below:

The plastic limit (ω_p) of this soil can also be worked out as

$$\omega_p = \omega_L - P_I$$

or $\omega_p = 65 - 22$

or $\omega_p = 43$ which is greater than 30

Hence, the classification, as per note given at the bottom of Table 3.1, is A-7-5, as $\omega_p > 30$. From Eq. (3.1),

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

$$a = (40-35)=5, b = (40-15)=25$$

$$c = (65-40)=25, d = (22-10)=12$$

$$GI = 0.2(5) + 0.005(5)(25) + 0.01(25)(12) = 4.6; \text{ say } 5$$

Therefore, the soil will be classified as A-7-5 (5)

Example 3.2

The results of the grain size analysis of a soil are as follows:

$$\text{Percent passing No. 10 sieve} = 100$$

$$\text{Percent passing No. 40 sieve} = 80$$

$$\text{Percent passing No. 200 sieve} = 58$$

The liquid limit and plastic index of minus No. 40 fraction of the soil are 30% and 10%, respectively. Classify the soil by the unified classification system.

Solution

Since 58% of the soil is passing through No. 200 sieve, it is a fine-grained soil. Referring to the plasticity chart in Table 3.2 for $\omega_L = 30\%$ and $P_I = 10\%$, it can be classified as CL.

EXERCISES

3.1 The mechanical analysis of a sample of a soil gave the following information.

Sand = 35 percent

Silt = 40 percent

Clay = 25 percent

Classify the soil as per the triangular classification system.

3.2 A sample contains 15 percent gravel, 32 percent sand, 33 percent silt and 20 percent clay. Classify the material by means of the triangular classification system.

3.3 The sieve analysis of two soils and the liquid and plastic limits of the fraction passing a No. 40 sieve are given below.

Classify the soils by the unified classification system.

Soil	Sieve Analysis, Percent finer					
	No.4	No.10	No.40	No.200	ω_L	ω_P
A	100	100	99	86	55	28
B	100	100	94	45	36	22