# GEOTECHNICAL ENGINEERING

PREPARED BY - Prof. NIYATI NAIK

Ch-1

## Introduction

Geotechnical engineering is the systematic application of techniques which allow construction on, in or with geomaterials i.e. soil and rock. Every civil engineering structure and construction is related to soil in some way and subsequently, its design will depend on properties of the soil or rock.

## Soil:

The soil can have different meaning, depending upon the field in which it is considered.

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

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

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

→ built on: foundation of buildings, bridges
→ built in: basements, culverts, tunnels,
→ built with: embankments, roads, dams.
→ supported: retaining walls

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

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

Soil consist of a ~~multipurpose~~ aggregation of solid particles water and air. This fundamental composition gives rise to unique engineering properties and the description of its mechanical behaviour requires some of the most classic principles of engineering mechanics.

## Soil engineering:

Soil engineering is a stream of geotechnical engineering which exclusively deals with understanding the characteristic and mechanics of soil. Soil engineering helps in analyzing the structure and composition of the soil of the proposed construction sites, thus helping in deciding whether the soil of the proposed construction site or building is worth exploiting. Apart from this soil engineering also deals with providing optimized design concepts and construction techniques according to the composition and physical properties of the soil.

## Scope of soil mechanics

### 1. Foundation:

Soil engineering helps us to decide which type of foundations are required to sustain the structures

### 2. Retaining structures

Soil engineering helps us to determine which type of retaining structures are suitable for the hold Earth material or water.

### 3. Stability of slope:

Soil engineering provides us various methods for checking the stability of slopes.

### 4. Underground structures

Soil engineering helps us to determine the strength of the forces exerted by the soil on the underground structure

### 5. Earth Dam:

Deep knowledge of the properties of the soils are required while constructing the earth dam.

### 6. Pavement Design

Behaviour of the soil under the different loading condition is studied soil engineering.

## Origin of soil:

Soils are formed by → 1) weathering of rocks
2) decomposition of organic matter

Weathering of rocks are Gravel, Sand, Silt, clay etc.

Decomposition of organic matter:- Humus, peat, muck organic matter (organic soil or cumulose soil)

The products of erosion are picked up and transported to some other place by wind water etc.

The shifting of material disturbs the equilibrium of forces on the earth and causes large scale movements and upheav-

## Formation of soil:

Soil minerals from the basis of soil. They are produced from rock (parent material) through the processes of weathering and natural erosion. Water, wind, temperature change, gravity, chemical interaction, living organisms and pressure difference all help break down parent material.

Soil evolve under the action of biological, climatic geologic and topographic influences. The evolution of soils and their properties is called soil formation and pedologists have identified five fundamental soil formation processes that influence soil properties. These five state factors are parent material, topography climate, organisms and time.

Soils are formed from materials that have resulted from the disintegration of rocks by various processes of physical and chemical weathering. The nature and structure of a given soil depends on the processes and conditions that formed it.

→ Breakdown of parent rock, weathering, decomposition, erosion

→ Transportation to site of final deposition: gravity, flowing water ice, wind.

→ Environment of final deposition: flood plain, river terrace, glacial moraine, lacustrine or marine

→ Subsequent conditions of loading and drainage: little or no surcharge heavy surcharge due to ice or overlying deposits, change from saline to freshwater, leaching, contamination.

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

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

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

ii. chemical weathering:

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

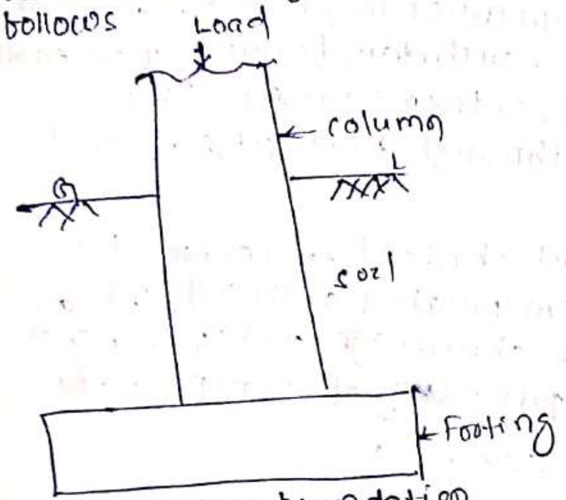
Rain water that comes in contact with the rock surface react to form hydrates oxides, carbonates and sulphates. If there is a volume increase, the disintegration continues. Due to leaching water soluble materials are washed away and rocks lose their cementing properties

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

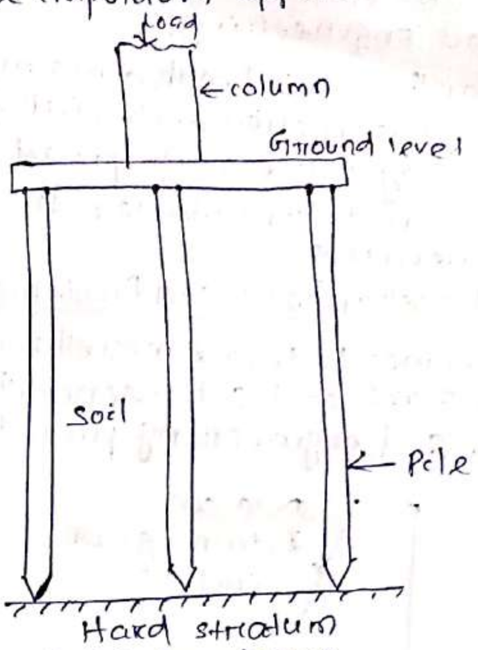
The effects of weathering and transportation mainly determine the basic nature of the soil (size, shape, composition

## Scope of soil engineering

Soil engineering has vast application in the construction of various civil engineering works. Some of the important applications are as follows




(a) shallow foundation




(b) Pile foundation

### (1) Foundations

Every civil engineering structures, whether it is a building a bridge, or a dam is founded on or below the surface of the earth. Foundations are required to transmit the load of the structure to soil safely and efficiently.

A foundation is termed shallow foundation when it transmits the load to upper strata of earth. A foundation is called deep foundation when the load is transmitted to strata at considerable depth below the ground surface (Fig 1.2). Pile foundation is a type of deep foundation. Foundation engineering is an important branch of soil engineering

### (2) Retaining structures

When sufficient space is not available for a mass of soil to spread and form a safe slope, a structure is required to retain the soil. An earth retaining structure is also required to keep the soil at differents level on its either side. The retaining structure may be a rigid retaining wall on a sheet pile bulkhead which is relatively flexible soil engineering gives the theory of earth pressure on retaining structure

(3) **stability of slopes**: If soil surface is not horizontal, there is a component of weight of the soil which tends to move it downward and thus causes instability of slope. The slopes may be natural or man-made Fig. 1.4 shows the slopes in filling and cutting. Soil engineering provides the methods for checking the stability of slopes.

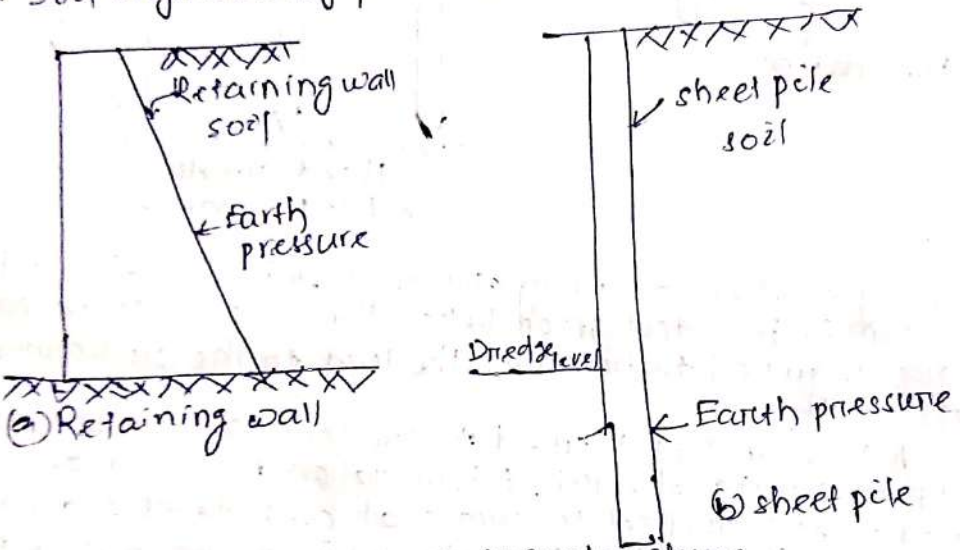
(4) **Underground structures**: The design and construction of underground structures such as tunnels, shafts and conduits require evalution of forces exerted by the soil on these structure. These forces are discussed in soil engineering Fig 1.5 shows a tunnel constructed below the ground surface and a conduit laid below the ground surface.

(5) **Pavement Design**: A pavement is a hard crust placed on soil (subgrade) for the purpose of providing a smooth and strong surface on which vehicles can move. The pavement consists of surfacing, such as a bitumen

layer, base and subgrade (Fig 1.6). The behaviour of subgrade under various conditions of loading and environmental changes is studied in soil engineering.

(6) Earth Dam: Earth dams are huge structures in which soil is used as a construction material (Fig 1.7). The earth dam failure of an earth dam may cause widespread catastrope, extreme care is taken its design and construction. It requires a through knowledge of soil engineering

(7) Miscellaneous soil Problems :- The geotechnical engineers has sometimes to tackle miscellaneous problem related with soil, such as soil have soil subsidence, frost heave shrinkage and swelling of soil. Soil engineering provides an indepth study of such problems.




Retaining structure

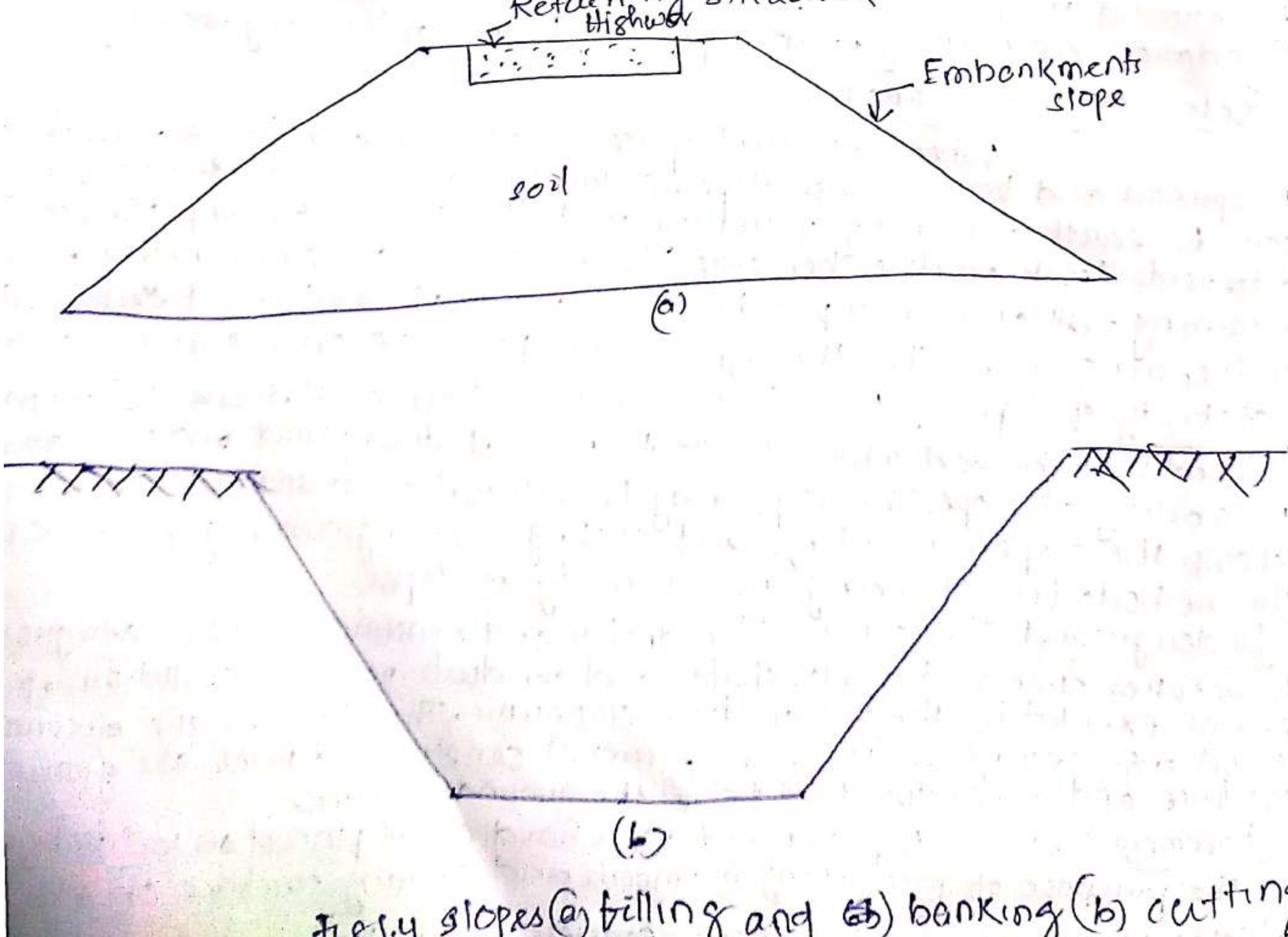
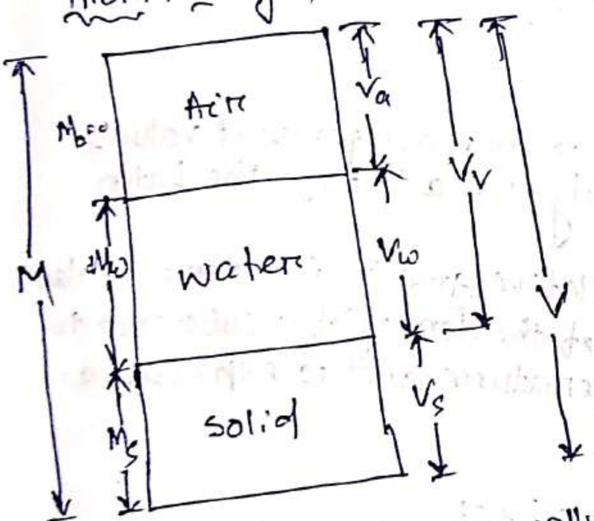



Fig 1.4 slopes (a) filling and (a) banking (b) cutting

Parent material is the initial state of the solid matter making up a soil. It can consist of consolidated rocks and it can also include unconsolidated deposits such as river alluvium, lake or marine sediments glacial tills, loess (silt-sized, wind-deposited particles), volcanic ash, and organic matter (such as accumulations in swamps or bogs). Parent material influence soil formation through their mineralogical composition, their texture, and their stratification. Dark-coloured ferromagnesian (iron- and magnesium-containing) rock, for example can produce soils with a high content of iron compounds and clay minerals in the Kaolin or smectite groups, whereas light coloured siliceous (silica-containing) rocks tend to produce soils that are low in iron compounds soils that ~~are~~ contain clay minerals in the illite or vermiculite groups. ——x——

CH-2

## Preliminary Definitions and Relationship




$V$ = Total Volume - $M$ = Total Mass
$V_a$ = volume of air
$V_w$ = volume of water
$V_s$ = volume of solids
$V_v$ = volume of void = $V_a + V_w$
$M_a$ = mass of air
$M_w$ = mass of water
$M_s$ = mass of solid

Soil mass is generally referred to as three-phase system because it consists of solid particles, liquid and gas. For many civil engineering purposes, the liquid may be considered to be water and the gas as air (Fig 1 and 2) with the exception geoenvironmental, and oil and gas application.

A saturated soil may be assumed to have only two of these phases as voids are completely saturated with water (Fig 3)

Fig.1

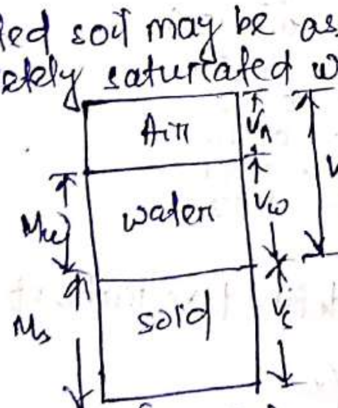



Fig 2. 3 phase diagram

Fig 3 Fully saturated soil

## Soil as a three phase system

A soil as a three phase system consisting of solid particles (called soil grains), water and air. The void space between the soil grains is filled partly with water and partly with air. However, if we take a dry soil mass, the voids are filled with air only. In case of a Perfectly saturated soil, the voids are filled completely with water.

In general, the soil mass has three constituents which done occupy separate spaces but are blended together forming a complex material. the properties of which depend upon the relative % of these constituents, their arrangement and a variety of other factors. For calculation purposes, it is always more convenient to show these constituents occupying separate spaces, as shown in fig (a) and Fig (b).

Moist sample = ($M_w + M_d$)

Water Content (W): also known as natural water content or natural moisture content, is the ratio of the weight of the water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage. When voids are completely filled with air, water content is equal to zero

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

Density (ρ): The density of a substance is its mass per unit volume. The symbol most often used for density is ρ although the Latin letter D can also used.

Specific Gravity: Specific gravity or relative gravity is a dimensionless quantity that is defined as the ratio of the density of a substance to the density of water at a specified temperature and is expressed as

$$SG = \frac{\rho_{substance}}{\rho_{H_2O}}$$

$$\text{specific gravity} = \frac{\text{Density of the object}}{\text{Density of water}}$$

| Material | Specific Gravity |
|---|---|
| Dry Air | 0.0013 |
| Alcohol | 0.82 |
| Carbon dioxide | 0.00126 |
| cast iron | 7.20 |
| petrol | 0.72 |
| Rubber | 0.96 |
| wood oak | 0.77 |
| Nylon | 1.12 |

Void Ratio (e)

Void ratio (e) is defined as ratio of the volume of voids to the volume of solids

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

where, $V_v$ is volume of void and
$V_s$ is volume of solids

$$e > 0$$

Porosity (n)

Porosity is defined as the ratio of volume of voids to the total volume of soil mass.

$$\eta = \frac{V_v}{V} \quad \text{where } V_v = \text{volume of void}, \; V = \text{total volume}$$

Percentage of air voids:-

% air voids is the ratio of volume of air to the total volume.

$$\% \eta_a = \frac{\text{volume of air}}{\text{Total volume}} \times 100 = \frac{V_a}{V} \times 100$$

Air Content:

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

$$a_c = \frac{V_a}{V_v} = 1 - S \qquad V_a \text{ - volume of air}$$

Degree of Saturation (S)

The degree of saturation is the ratio of volume of water to the volume of voids.

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

where, $V_w$ = volume of water

$V_v$ = volume of voids

$$0 \le S \le 100$$

for perfectly dry soil $S = 0$

for fully saturated soil $S = 100\%$

Density Index:-

Relative density or density index is the ratio of the difference between the void ratios of a cohesionless soil in its loosest state and existing natural state to the difference between its void ratio in the loosest and densest states.

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

It is the Unit less.

$e_{max}$ = void ratio of soil in loosest state

$e_{min}$ = void ratio of soil in densest state

$e$ = void ratio of soil in natural state

If soil naturally is in the loosest state

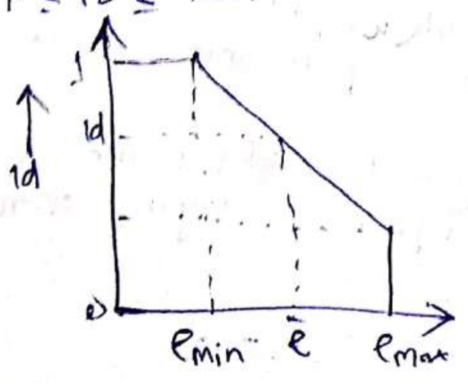
$e = e_{max} \Rightarrow I_D = 0\%$

If soil naturally is in the densest state

$e = e_{min} \Rightarrow I_D = 100\%$

$0\% \le I_D \le 100\%$

$$\Rightarrow e \propto (1/\gamma_d)$$




$$I_d = \frac{\frac{1}{\gamma_{dmin}} - \frac{1}{\gamma_d}}{\frac{1}{\gamma_{dmin}} - \frac{1}{\gamma_{max}}} \times 100$$

Scanned with OKEN Scanner

## Degree of Denseness According to Degree of Density

| Id (%) | Degree of Denseness |
|---|---|
| 0-15 $\longrightarrow$ | very Loose |
| 15-35 $\longrightarrow$ | Loose |
| 35-65 $\longrightarrow$ | Medium Dense |
| 65-85 $\longrightarrow$ | Dense |
| 85-100 $\longrightarrow$ | very Dense |

**Density:**

The density or unit weight of a soil mass is defined as its weight per unit volume.

**Bulk Density ($\gamma$)**

The bulk density or moist density $\gamma$ is defined as the total weight (W) as a soil mass per unit of its total volume (V).

Thus, $\gamma = W/V$

**Submerged Density ($\gamma_{sub}$)**

The submerged density or buoyant unit weight is defined as the submerged weight (Wd) sub of the soil solids per unit of total volume (V) of the soil mass

$$\gamma_{sub} = \frac{(W_d)_{sub}}{V}$$

The submerged density or buoyant density is also expressed as

$$\rho' = \rho_{sat} - \rho_w$$

where, $\rho_w$ is the density of water which may be taken as 1g/cm³ for calculation purposes.

**Saturated density ($\rho_{sat}$):**

When the soil mass is saturated, its bulk density is called saturated density. Thus, saturated density is the ratio of the total soil mass of saturated sample to its total volume.

**Unit weight of soil mass**

The unit weight of a soil mass is defined as its weight per unit volume.

**Bulk unit weight ($\gamma$):** The bulk weight or moist unit weight is the total weight W of a soil mass per unit of its total volume V.

Thus $\gamma = \frac{W}{V}$

**Dry unit weight ($\gamma_d$):** The dry unit weight is the weight of solids per unit of its total volume (prior to drying) of the soil mass.

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

## Unit weight of solids ($\gamma_s$):-

The unit weight of soil solids is the weight of soil solids $W_d$ per unit volume of solids ($V_s$):

$$\gamma_s = \frac{W_d}{V_s}$$

## Saturated unit weight ($\gamma_{sat}$):

When the soil mass is saturated, its bulk unit weight is called the saturated unit weight. Thus, saturated unit weight is the ratio of the total weight of a saturated soil sample to its total volume.

## Submerged unit weight ($\gamma'$):

The submerged unit weight $\gamma'$ is the submerged weight of soil solids $(W_d)_{sub}$ per unit of total volume $V$ of the soil mass.

$$\gamma' = \frac{(W_d)_{sub}}{V}$$

## Relative compaction ($R_c$):

Degree of compaction is also some times expressed in terms of an index called relative compaction ($R_c$) defined as follows.

$$R_c = \frac{\gamma_d}{\gamma_{d.max}}$$

where, $\gamma_{d.max}$ is the maximum dry dencity from compaction test

Relative compaction ($R_c$) can also be expressed in terms of relative density ($I_D$) as follows:

$$R_c = \frac{R_0}{1 - I_D(1 - R_0)}$$

where, $R_0 = \gamma_{d.min}/\gamma_{d.max}$ and

$R_c$ and $I_D$ are in fraction form.

## Functional Relationships

(i) Relation between $e$, $G$, $w$ and $S$

In Fig 2.5, $e_w$ represents volume of water, $e$ represents the volume of voids, and volume of solids is equal to unity.

Now, $S = \frac{V_w}{V_v} = \frac{e_w}{e}$

Hence $e_w = eS$ ...... 1.a.

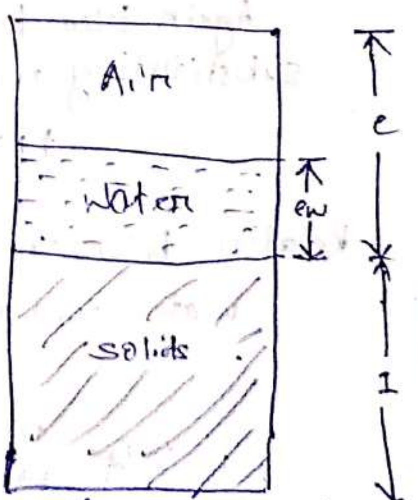
The term $e_w$ is known as the water voids ratio

For a fully saturated sample, $e_w = e$

Now $w = \frac{W_w}{W_d} = \frac{e_w \gamma_w}{\gamma_s \cdot 1}$

But $G = \frac{\gamma_s}{\gamma_w}$ or $\gamma_s = G\gamma_w$

$$w = \frac{e_w \gamma_w}{G\gamma_w} = \frac{e_w}{G}$$




soil element in terms of $e_w$ and $e$

$\Rightarrow \quad e_w = wG$ . . . . (1.b)

Equating Eqs. 1.(a) and 1.(b), we get

$$e = \frac{wG}{S} \quad . . . . (2.)$$

For a fully saturated soil, $S = 1$ and $w = w_{sat}$

$$e = w_{sat} G \quad . . . . . 2.(1)$$

(ii) <u>Relation between $e$, $S$ and $n_a$</u>

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

From Fig 2.5, $V_a = V_v - V_w = e - e_w$ and

$$V = V_s + V_v = 1+e$$

$\therefore n_a = \frac{e - e_w}{1+e}$ But $e_w = eS$ (Eq. 2.21)

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

(iii) <u>Relation between $n_a$, $a_c$ and $n$</u>

$$a_c = \frac{V_a}{V_v}, \quad n = \frac{V_v}{V} \quad . - - 2.$$

$$\therefore n_a = \frac{V_a}{V} = n.a_c$$

(iv) <u>Relation between $\gamma_d$, $G$ and $e$ (or $n$)</u>

$$\gamma_d = \frac{W_d}{V} \qquad \gamma_d = \frac{\gamma_s . V_s}{V}$$

Now from Fig 2.(a)

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

$$\gamma_d = \frac{\gamma_s . 1}{1+e} \quad \text{But } \gamma_s = G\gamma_w$$

$$\therefore \gamma_d = \frac{G\gamma_w}{1+e} \quad . . . . 2.3$$

A convenient expression for calculating the voids ratio of soil mass is obtained from Eq. 2.23 as

$$e = \frac{G\gamma_w}{\gamma_d} - 1 \quad . . . 2.24$$

Again, from fig. 2.2(b), $V_s = (1-n)$ and $V = 1$

substituting in (1) and taking $\gamma_s = G\gamma_w$ we get

$$\gamma_d = \frac{G\gamma_w(1-n)}{1} = (1-n)G\gamma_w$$

<u>Relation between $\gamma_d$, $\gamma$ and $w$</u>

Water content $w = \frac{W_w}{W_d}$ Hence $1+w = \frac{W_w + W_d}{W_d}$

$$= \frac{W}{W_d}$$

$$\therefore W_d = \frac{W}{1+w}$$

$$\therefore \gamma_d = \frac{W_d}{V} = \frac{W}{(1+w)V} \qquad \therefore \gamma_d = \frac{\gamma}{1+w}$$

Ex.1 A soil sample has a porosity of 40%. The specific gravity of solids is 2.70. Calculate (a) void ratio, (b) dry densities, (c) unit weight if the soil is 50% saturated and (d) unit weight if the soil is completely saturated.

Ans Given data

$n = 40\% = 0.4$, $G = 2.70$

(a) We have $e = \frac{n}{1-n} = \frac{0.4}{1-0.4} = 0.667$

(b) $\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.7 \times 9.81}{1+0.667} = 15.89\ kN/m^3$ (Taking $\gamma_w = 9.81\ kN/m^3$)

(c) $e = \frac{wG}{S}$ or $w = \frac{eS}{G} = \frac{0.667 \times 0.5}{2.70}$

$= 0.124$

$\gamma_d = 15.89\ kN/m^3$ (as before)

$\therefore \gamma = \gamma_d(1+w) = 15.89 \times 1.124 = 17.85\ kN/m^3$

check: $\gamma = \frac{\gamma_w(G+es)}{1+e} = \frac{9.81 \times (2.70 + 0.667 \times 0.5)}{1+0.667} = 17.85\ kN/m^3$

(d) When the soil is fully saturated, $e = w_{sat} G$

$\therefore w_{sat} = \frac{e}{G} = \frac{0.667}{2.70} = 0.247$

$\therefore \gamma_{sat} = \gamma_d(1+w_{sat}) = 15.89 \times 1.247 = 19.81\ kN/m^3$

Ex.2.2 An undisturbed sample of soil has a volume of $100\ cm^3$ and mass of 190 g. On oven drying for 24 hours, the mass is reduced to 160 g. If the specific gravity of grains is 2.68, determine the water content, void ratio and degree of saturation of the soil.

Ans ~~$M_w = 1980$~~ $M_w = 190 - 160 = 30\ g$

$M_d = 160\ g$

$\therefore w = \frac{M_w}{M_d} = \frac{30}{160} = 0.188 = 18.8\%$

Mass of moist soil $= M = 190\ g$

$\therefore$ Bulk density $\rho = \frac{M}{V} = \frac{190}{100} = 1.9\ g/cm^3$

Hence $\gamma = 9.81 \times \rho = 9.81 \times 1.9 = 18.64\ kN/m^3$

(since $1\ g/cm^3 = 9.81\ kN/m^3$)

$\gamma_d = \frac{\gamma}{1+w} = \frac{18.64}{1+0.188} = 15.69\ kN/m^3$

$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.68 \times 9.81}{15.69} - 1 = 0.67$

$S = \frac{wG}{e} = \frac{0.188 \times 2.68}{0.67} = 0.744 = 74.4\%$

Q3 The in-situ density of an embankment, compacted at a water content of 12% was determined with the help of a core cutter. The empty mass of the cutter was 1286 g and the cutter full of soil has a mass of 3195 g, the volume of the cutter being 1000 cm³. 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 change in soil on saturation. Take the specific gravity of the soil as 2.70.

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

$\therefore$ Bulk unit weight $\gamma = 9.81\rho = 9.81 \times 1.909 = 18.73$ kN/m³

$$\gamma_d = \frac{\gamma}{1+w} = \frac{18.73}{1+0.12} = 16.72 \text{ kN/m}^3$$

$$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{16.72} - 1 = 0.584$$

and $$S = \frac{wG}{e} = \frac{0.12 \times 2.70}{0.584} = 0.555 = 55.5\%$$

At saturation, since the volume remains the same, the void ratio also remains unchanged. Now $e = w_{sat} \cdot G$

$$\therefore \; w_{sat} = \frac{e}{G} = \frac{0.584}{2.70} = 0.216 = 21.6\%$$

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w = \left(\frac{2.7+0.58}{1+0.584}\right) \times 9.81$$

$$= 20.34 \text{ kN/m}^3$$

Q4 The in-situ % voids of a sand deposit is 34%. For determining the density index, dried sand from the stratum was first filled loosely in a 1000 cm³ 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 1980 gm. Determine the density index if the specific gravity of the sand particles is 2.67.

Ans $n = 34\% = 0.34$

$$e = \frac{n}{1-n} = \frac{0.34}{1-0.34} = \frac{0.34}{0.66} = 0.515$$

$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.67 \times 9.81}{1+0.515} = 17.289 \text{ kN/m}^3$$

$$(\gamma_d)_{max} = \frac{1980}{1000} \times 9.81 = 19.42 \text{ kN/m}^3$$

$$(\gamma_d)_{min} = \frac{1610}{1000} \times 9.81 = 15.79 \text{ kN/m}^3$$

$$e_{min} = \frac{G\gamma_w}{(\gamma_d)_{max}} - 1 = \frac{2.67 \times 9.81}{19.42} - 1$$

$$= 0.349$$

Scanned with OKEN Scanner

$$e_{max} = \frac{G\gamma_w}{(\gamma_d)_{min}} - 1 = \frac{2.67 \times 9.81}{15.79} - 1 = 0.659$$

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{0.659 - 0.515}{0.659 - 0.349} = 0.465 = 46.5\%$$

Q. 5 The mass specific gravity (apparent specific gravity) of a soil equals 1.67. The specific gravity of solids is 2.70. Determine the void ratio under the assumption that the soil is perfectly dry. What would be the void ratio, if the sample is assumed to have a water content of 8%?

Ans When the sample is dry, $G_m = \frac{\gamma_d}{\gamma_w} = 1.64$ (given)

$\therefore \gamma_d = 1.64\gamma_w = 1.64 \times 9.81 = 16.09\ kN/m^3$

Now $e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{16.09} - 1 = 0.646$

When the sample has the water content $w = 8\%$, $G_m = \frac{\gamma}{\gamma_w} = 1.64$

$\therefore \gamma = 1.64\gamma_w = 1.64 \times 9.81 = 16.09\ kN/m^3$

$\therefore \gamma_d = \frac{\gamma}{1+w} = \frac{16.09}{1+0.08} = 14.9\ kN/m^3$

$$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{14.9} - 1 = 0.78$$

6 A natural soil deposits has a bulk unit weight of 18.44 kN/m³ and water content of 5%. Calculate the amount of water required to be added to 1 cubic metre 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.

Ans Given

$\gamma = 18.44\ kN/m^3$ and $w = 5\%$

$\therefore \gamma_d = \frac{\gamma}{1+w} = \frac{18.44}{1+0.05} = 17.56\ kN/m^3$

Earlier, when $w = 5\%$, $w = 0.05 = \frac{W_w}{W_d}$

For one cubic metre of soil $V = 1 m^3$

$\therefore W_d = \gamma_d . V = 17.56 \times 1 = 17.56\ kN$

$\therefore W_w = 0.05 \times W_d = 0.05 \times 17.56 = 0.878\ kN$

$\therefore V_w = \frac{W_w}{\gamma_w} = \frac{0.878}{9.81} = 0.0895\ m^3$

Later, when $w = 15\%$ $W_w = wW_d = 0.15 \times 17.56 = 2.634\ kN$

$V_w = \frac{W_w}{\gamma_w} = \frac{2.634}{9.81} = 0.2685\ m^3$

Hence, additional water required to raise the water content from 5% to 15% = 0.2685 - 0.0895 = 0.179 $m^3$ = 179 litres.

Void ratio, $e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.67 \times 9.81}{17.56} - 1 = 0.49$

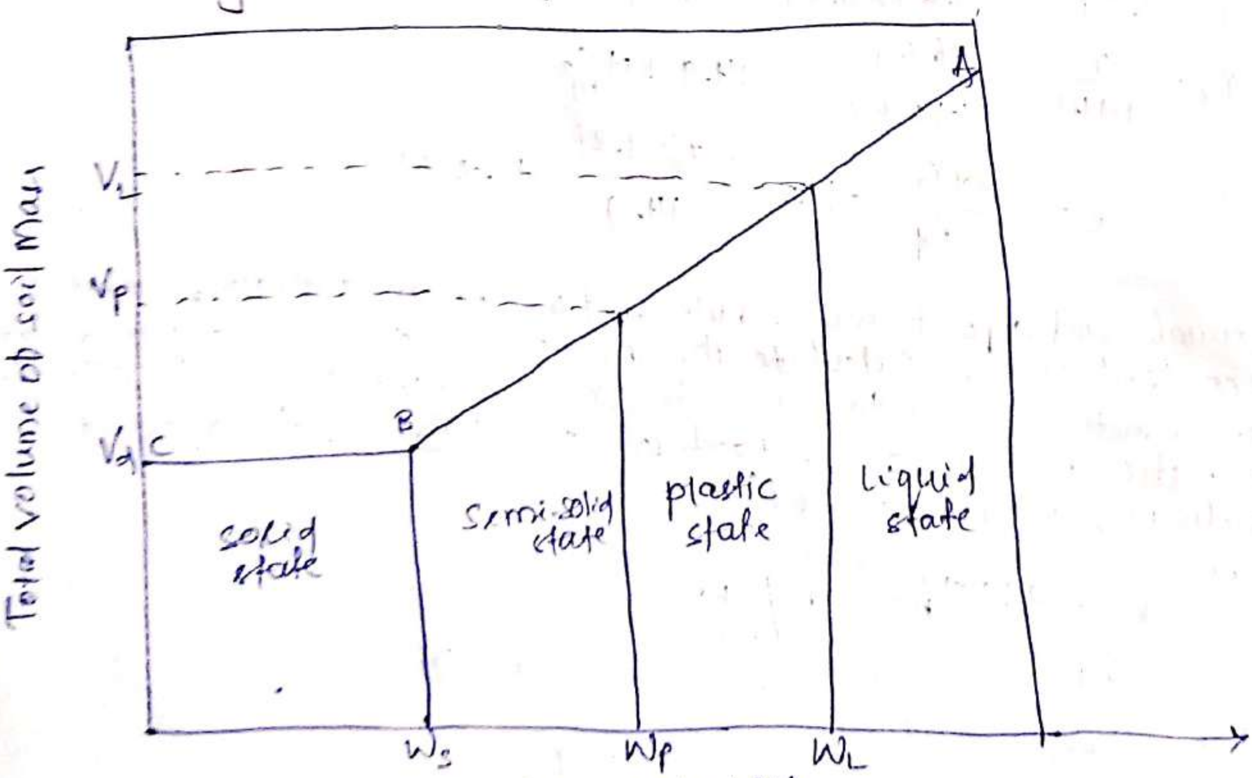
After the water has been added e remains the same.

$$S = \frac{wG}{e} = \frac{0.15 \times 2.67}{0.49} = 0.817 = 81.7\%$$

## Consistency of soil

By consistency is meant the relative ease with which soil can be deformed. This term is mostly used for fine grained soils for which the consistency is related to a large extent to water content. Consistency denotes degree of firmness of the soil which may be term as soft, firm, stiff or hard.

In 1911, the swedish agriculturist Atterberg divided the entire range from liquid to solid state into four stages (i) the liquid state, (ii) the plastic state, (iii) the semi-solid state and (iv) the solid state. The water content at which the soil changes from one state to another state. That is known as consistency limits or Atterberg's limit.




### Liquid Limit ($W_L$):-

Liquid limit is the water content corresponding to the arbitrary limit between liquid and plastic state of consistency of a soil. It is defined as the minimum water content at which the soil is still in the liquid state, but has a small shearing strength against flowing which can be measured by standard available means. With reference to the standard are liquid limit device. it is defined as the minimum water content at which a point of soil cut by a groove of standard dimension will flow together for a distance of 12mm ($\frac{1}{2}$ inch) under an impact of 25 blows in the device.

Plastic Limit ($W_p$):-

plastic limit is the water content corresponding to an arbitrary limit between the plastic and the semi-solid state of consistency of a soil. 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.

Shrinkage limit ($W_s$):-

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

Plasticity Index ($I_p$)

The range of consistency within which a soil exhibit plastic properties is called plastic range and is indicated by plasticity index. The plasticity index is defined as the numerical difference between the liquid limit and the plastic limit of a soil.

$$I_p = W_L - W_p$$

plasticity:-

plasticity is defined as that property of a soil which allows it to be deformed rapidly, without rupture, without elastic rebound and without volume change. According to Goldschmidt theory, the plasticity is due to the pressure of thin scale like particles which carry on their surfaces electro-magnetic charges.

Consistency Index ($I_c$):-

The consistency index or the relative consistency is defined as the ratio of the liquid limit minus the natural water content to the plasticity index of a soil.

$$I_c = \frac{W_L - W}{I_p}$$ where $W$ is the natural water content of the soil.

Consistency index is useful in the study of the field behaviour of saturated fine grained soils. Thus, if the consistency index of a soil is equal to unity. It is at the plastic limit

Liquidity Index ($I_L$):-

The liquidity index or water-plasticity ratio is the ratio, expressed as a percentage, of the natural water content of a soil minus its plastic limit, to its plasticity index.

$$I_L = \frac{W - W_p}{I_p}$$ where $W$ is natural water content of soil.

## Toughness Index ($I_T$):

The toughness index is defined as the ratio of the plasticity index to the flow index.

$$I_T = I_P / I_f$$

**Shrinkage Ratio (SR)**: It is defined as the ratio of a given volume change expressed as a percentage of dry volume, to the corresponding change in water content above the shrinkage limit expressed as a percentage of the weight of the oven dried soil.

shrinkage limit is known.

$$G = \frac{1}{\frac{\gamma_w}{\gamma_d} - \frac{w_s}{100}} = \frac{1}{\frac{\rho_w}{\rho_d} - \frac{w_s}{100}}$$

$\rho_d$ = dry density of soil (g/cm³);
$\rho_w$ = density of water (= 1 g/cm³)

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

where $V_1$ = volume of soil mass at water content $w_1$
$V_2$ = volume of soil mass at water content $w_2$
$V_d$ = volume of dry soil mass
$w_1, w_2$ = water content, expressed as percentage

At the shrinkage limit $V_2 = V_d$ and $w_2 = w_s$: hence

$$SR = \frac{\left(\frac{V_1 - V_d}{V_d}\right) 100}{w_1 - w_s}$$

The change in the water contents $(w_1 - w_2)$ is given by

$$w_1 - w_2 = \frac{(V_1 - V_2)\rho_w}{M_d} \times 100$$

Hence, $$SR = \frac{M_d}{V_d \rho_w} = \frac{\rho_d}{\rho_w} = \frac{\gamma_d}{\gamma_w}$$

**Volumetric shrinkage (VS)**: The volumetric shrinkage or volumetric change is defined as the decrease in the volume of a soil mass, expressed as a percentage of the dry volume of the soil mass, when the water content is reduced from a given percentage to the shrinkage limit.

$$VS = \frac{V_1 - V_d}{V_d} \times 100$$

But $\frac{V_1 - V_d}{V_d} \times 100 = (w_1 - w_s) SR$

$$VS = (w_1 - w_s) SR$$

where $V_1$ is the volume of soil mass at any water content $w_1$

Linear shrinkage ($L_s$):- It is defined as the decrease in one dimension of a soil mass expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit. It is calculated from the following formula.

$$L_s = 100\left[1-\left(\frac{100}{VS+100}\right)^{\frac{1}{3}}\right]$$

AKS.S.G

Ex.1 An undisturbed saturated specimen of clay has a volume of 18.9 cm³ and a mass of 30.2g. On oven drying, the mass reduces to 18.0g. The volume of dry specimen as determined by displacement of mercury is 9.9 cm³. Determine shrinkage limit, specific gravity, shrinkage ratio and volumetric shrinkage.

Ans Given data

$M_1 = 30.2\,g$, $M_d = 18.0\,g$ $\rho_w = 1\,g/cm^3$

$V_1 = 18.9\,cm^3$, $V_2 = 9.9\,cm^3$

(i) $$W_s = \left[\frac{M_1-M_d}{M_d} - \frac{(V_1-V_2)\rho_w}{M_d}\right]\times 100$$

$$= \left[\frac{30.2-18.0}{18.0} - \frac{(18.9-9.9)1}{18.0}\right]\times 100$$

$= 17.8\%$

(ii) $$G = \frac{M_d}{V_1-(M_1-M_d)} = \frac{18.0}{18.9-(30.2-18.0)} = 2.69$$

~~where $\rho_w$~~ $$G = \frac{1}{\left(\frac{\rho_w}{\rho_d} - \frac{W_s}{100}\right)}$$ given equation

where $\rho_w$ = density of water = $1\,g/cm^2$

$\rho_d$ = dry density of soil specimen $= \frac{18.0}{9.9} = 1.818\,g/cm^3$

$$\therefore \quad G = \frac{1}{\left(\frac{1}{1.818} - \frac{17.8}{100}\right)} = 2.69$$

(iii) shrinkage ratio, $SR = \frac{V_d}{V_o} = \frac{\rho_d}{\rho_w} = \frac{1.818}{1} = 1.82$

(iv) Volumetric shrinkage $VS = \frac{(V_1-V_d)100}{V_d} = \frac{18.9-9.9}{9.9}\times 100 = 91\%$

Ex.2 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.

Ans Data

$W_{sat} = 36\% = 0.36$

$e = W_{sat}\,G = 0.36\,G$

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

$$1.86 = \frac{G+0.36G}{1+0.36G} = \frac{1.36G}{1+0.36G}$$

From which, $G = 2.69$

Now $w_s = \frac{\gamma_w}{\gamma_d} - \frac{1}{G}$ where $\frac{\gamma_d}{\gamma_w}$ = mass specific gravity of dry soil $= 1.72$

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

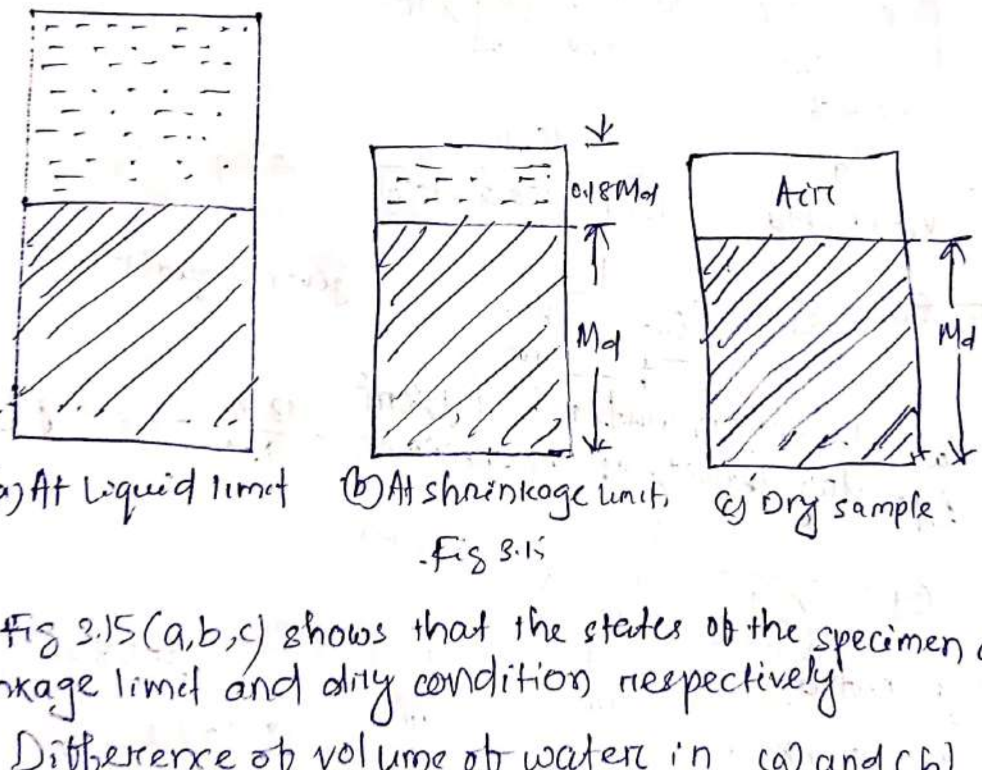
Alternate

$$(e)_{dry} = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{G\rho_w}{\rho_d} - 1 = \frac{2.69 \times 1}{1.72} - 1$$
$$= 0.565$$

$$w_s = \frac{e}{G} = \frac{0.565}{2.69} = 0.21 = 21\%$$

EX2 The Atterberg limits of a clay soil are: liquid limit 52%, plastic limit 30% and shrinkage limit 18%. If the specimen of this soil shrinks from a volume of 39.5 cm$^3$ at the liquid limit to a volume of 24.2 cm$^3$ at the shrinkage limit, calculate the true specific gravity.

Ans




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

Fig 3.15

Ans Fig 3.15(a,b,c) shows that the states of the specimen at Liquid limit shrinkage limit and dry condition respectively.

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

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

But from Fig 3.15 (a),(b), this difference is equal to
$(0.52 - 0.18) M_d$

$(0.52 - 0.18) M_d = 15.3$ or

$$\Rightarrow M_d = \frac{15.3}{(0.52 - 0.18)} = 45 \text{ g}$$

Mass of water in (b) = $0.18 M_d = 0.18 \times 45 = 8.1 g$

Volume of water in (b) = $24.2 - 8.1 = 16.1 \ cm^3$

Hence $\rho_s$ = density of solids = $\frac{M_d}{V_s} = \frac{45}{16.1} = 2.8 \ g/cm^3$

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

Q.3 The plastic limit of a soil is 25% and its plasticity index is 8%. When the soil is dried from its state at plastic limit, the volume change is 25% of its volume at plastic limit. Similarly, the corresponding volume change from the liquid limit to the dry state is 34% of its volume at liquid limit. Determine the shrinkage limit and the shrinkage ratio.

Ans

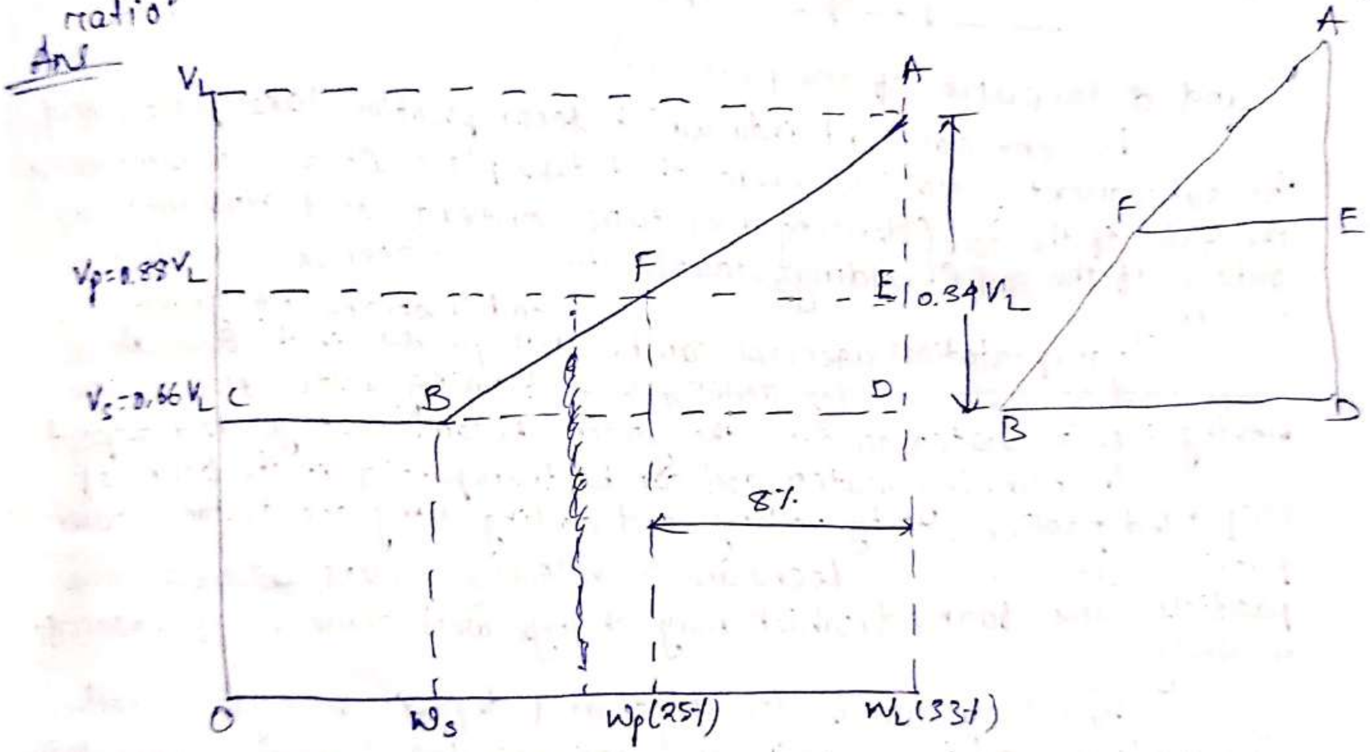



Fig 3.16

$W_p = 25\%$, $I_p = 8\%$

$\therefore W_L = 25 + 8 = 33\%$

Volume change at liquid limit = 34%

$\therefore$ Dry volume $V_d = V_L - 0.34 V_L = 0.66 V_L$ .... (i)

where $V_L$ = volume at liquid limit

Similarly, volume change at plastic limit = 25%

$\therefore V_d = V_p - 0.25 V_p = 0.75 V_p$ .... (ii)

where $V_p$ = volume at plastic limit

Equating (i) and (ii), we get $V_p = \frac{0.66}{0.75} V_L = 0.88 V_L$

Fig 3.16 shows the consistency limits. From the diagram, it's clear that when the soil passes from liquid limit to plastic limit, there is a change of $(1 - 0.88) V_L$ in volume and 8% change in water content.

$$\therefore \frac{BD}{AD} = \frac{FE}{AE} = \frac{8}{0.12V_L}$$

$$\therefore BD = \frac{8}{0.12V_L} \times 0.34\,V_L = 22.6\%$$

$$\therefore W_s = W_L - 22.6\%$$

$$= 33 - 22.6 = 10.4\%$$

$$SR = \frac{\frac{V_1 - V_2}{V}}{W_1 - W_2} = \frac{(V_L - V_p)\times 100}{V_L(W_L - W_p)}$$

$$= \frac{V_L(1-0.88)100}{0.66V_L(33-25)} = \frac{0.12\times 100}{0.66\times 8} = 2.27$$

## Index Properties of soil

In this chapter of determining those properties of soils which are used in their identification and classification. These include the determination of (i) water content, (ii) specific gravity (iii) particle size distribution, (iv) consistency limits, (v) in-situ density and (vi) density index.

These properties are known as index properties

## Water Content :-

The water content of a soil sample can be determined by the following methods.

(i) Oven drying method (ii) Sand bath method
(iii) Alcohol method (iv) Calcium Carbide method
(v) Pycnometer method (vi) Radiation method
(vii) Torsion balance method

**1 Oven drying method:** This is the most accurate method of determining the water content, and is therefore, used in laboratory. A specimen of soil sample is kept in a clean container and put in thermostatically controlled oven with interior of non-corroding soils take about four hours and fat clays take about 14 to 16 hours. Usually the sample is kept for about 24 hours in the oven so that complete drying is assured. A temperature higher than 110°C may break the crystalline structure of clay particles resulting in the loss of chemically bound structural water. For highly organic soils, such as a peat, a lower temperature of about 60°C is preferable to prevent the oxidation of the organic matter. Certain soils contain gypsum which on heating looses its water of crystallisation. If it is suspected that gypsum is present on the soil, the sample is dried at not more than 80°C but for a longer time (IS: 2720 part II-1969)

A clean non-corrodable container is taken and its mass is found with its lid, on a balance accurate to 0.01g. A specimen of the moist soil is placed in the container and the lid is replaced. The mass of the container and the contents is determined. With the lid removed, the container is then placed in the oven for drying. After drying, the container is removed from the oven and allowed to cool in a desiccator. The lid is then replaced, and the mass of container and the dry soil is found. The water content is calculated from the following expression

$$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100 \text{ (percent)}$$

Where, $M_1$ = mass of container with lid

$M_2$ = mass of container with lid and wet soil
$M_3$ = mass of container with lid and dry soil.

2 Sand bath Method :- This is a field method of determining rough value of the water content, where the facility of an oven is not available. The container with the soil is placed on a sand bath. The sand bath is heated over a kerosene stove. The soil becomes dry within $\frac{1}{2}$ to 1 hr. The water content is then determined from Eq. 2.1. However, higher temperature may break the crystalline structure of soils. This method should not be used for organic soils or for soils having higher percentage of gypsum (B.S. 1377: 1961)

3 Alcohol method :- This is also a crude field method. The wet soil sample is kept in a evaporating dish and mixed with sufficient quantity of methylated spirit. The dish is then properly covered and the mixture is ignited. The mixture is kept stirred by ~~dish is then~~ a wire during ignition. Since there is no control over the temperature it should not be used for soils containing large percentage of organic matter or gypsum. The water content is determined from the expression:

$$W = \frac{M_2 - M_3}{M_3 - M_1} \times 100$$ where $M_1$ = mass of empty dish
$M_2$ = mass of dish + wet soil
$M_3$ = mass of dish + dry soil

4 Calcium Carbide Method :- In this method, 6 gm of wet soil sample is placed in an air-tight container (called moisture tester) and is mixed with sufficient quantity of fresh calcium carbide powder. The mixture is shaken vigorously. The acetylene gas produced by the reaction of the moisture of the soil and the calcium carbide, exerts pressure on a sensitive diaphragm placed at the end of the container. The dial gauge located at the diaphragm reads the water content directly. However, the calibration of the dial gauge is such that it gives the water content (w') based on the wet weight of the sample. Knowing the water content w' based on wet weight, the water content (w) based on dry weight can be found from Eq. 2.1 (d)

$$W = \frac{w'}{1 \cdot w'}$$

The method is very quick - the result can be obtained in 5 to 10 minutes. The field kit contains the moisture tester, a small single-pan weighing balance, a bottle containing calcium carbide and a brush. A big container permits the use of 26 gm sample (Blyfeden: 1961). In order that balls, having wet soil inside and dry soil outside, may not form during the reaction with calcium carbide, the soil may be mixed with perfectly dry sand.

In the larger container, two 20 mm diameter balls are placed for proper pulverisation of clay soils. This method is specially suited to a circumstance where water content is to be quickly determined for the purpose of proper field control. Such as in the compaction of an embankment.

5 Pycnometer Method: This is also a quick method of determining the water content of those soils whose specific gravity G is accurately known. Pycnometer (Fig 3.1a) is a large size density bottle of about 900ml capacity. A conical brass cap, having a 6mm diameter hole at its top is screwed to the open end of the pycnometer. A rubber washer is placed between conical cap and the rim of the bottle so that there is no leakage of water.

Test Procedure:-

1. Take a clean, dry pycnometer and find its mass with its cap and washer ($M_1$)
2. Put about 200 g to 400g of wet soil sample in the pycnometer and find its mass with its cap and washer ($M_2$)
3. Fill the pycnometer to half its height and mix it throughly with the glass rod. Add more water, and stir it. Replace the screw top and fill the pycnometer flush with the hole in the conical cap. Dry the pycnometer from outside and find its mass ($M_3$)
4. Empty the pycnometer, clean it throughly and filled it with clean water to the hole of the conical cap and find its mass ($M_4$).

The water content is then calculated from the following expression:

$$w = \left\{ \left( \frac{M_2 - M_1}{M_3 - M_4} \right) \left( \frac{G-1}{G} \right) - 1 \right\} \times 100 \quad \ldots (3.2)$$

The above expression can be derived with reference to Fig 3.2. In Fig 3.1a (iii), if $M_d$ is the mass of soil particles, the volume of solid particles will be equal to $M_d/G$. Thus, if the solids from (iii) are replaced with water of mass $M_d/G$, we get the mass $M_4$ indicated in (iv). Thus,

$$M_4 = M_3 - M_d + \frac{M_d}{G}$$

$$\Rightarrow M_d \left( \frac{G-1}{G} \right) = M_3 - M_4$$

From which, $M_d = (M_3 - M_4) \frac{G}{G-1}$

Now mass of water $M_w$ in the wet soil sample

$$= (M_2 - M_1) - M_d$$

$$\therefore\ w = \frac{M_w}{M_d} \times 100$$

$$= \frac{M_2 - M_1 - M_d}{M_d} \times 100$$

$$= \left(\frac{M_2 - M_1}{M_d} - 1\right) \times 100 = \left[\left(\frac{M_2 - M_1}{M_3 - M_1}\right)\left(\frac{G-1}{G}\right)\right] \times 100$$

This method is suitable for coarse grained soils only

6 **Radiation Method**:- This method is extremely useful for the determination of water content of soil deposit in the in-situ condition. It uses two steel casing - casing A and casing B which are placed in two bore holes at some distance apart, in the soil deposit the field moisture content of which is to be determined. A device containing some radioactive isotope material (such as cobalt-60) is placed in a capsule which in turn is lowered into casing A.

Similarly, a detector unit is lowered in steel casing B. Small openings are made in both casing A and B, facing each other. When the radio-active device is activated, it emits neutrons. When these neutrons strike with the hydrogen atoms of water in the sub-soil, they loose energy. The loss of energy is evidently equal to water content in the soil. The detector device is calibrated to given directly the water content of the subsoil, at that level of emission. However, proper shielding precautions should be taken to avoid radiation probleme.

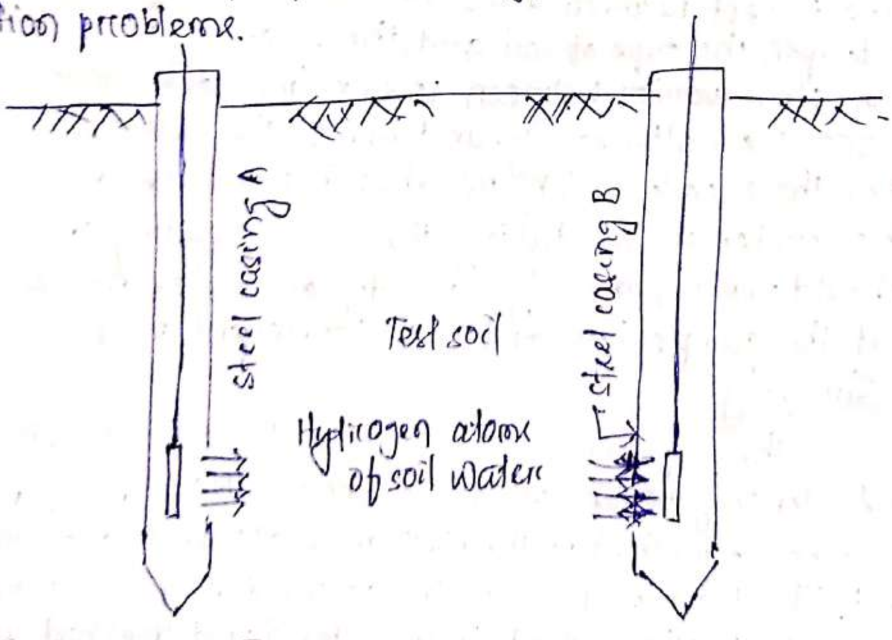



Fig 3.2. Radiation Method

7 **Torsional Balance Method**: (IS: 2720, part II-1973) The equipment has two main parts: i) infra-red lamp and ii) torsional balance. The infra-red radiation is provided by 250 watt lamp built in the balance for use with alternating current 220-230 V, 50 cycle

cycles, single phase main supply. The weighing machanism, a torsion-balance, has a built-in magnetic damper to reduce pan vibrations during quick drying. The balance scale is divided in terms of water percentages from 1 to 100 water content in 0.2% division. The moisture meter is generally calibrated to use 25g of soil, and hence the maximum size of particle present in the specimen should be less than 2mm.

The test specimen is kept in a suitable container so that the water content to be determined is not affected by ambient conditions. Torque is applied to one end of the torsion wire by means of a calibrated drum to balance the loss of weight of the sample as it dries out under infrared lamp. To determine the percentage reduction of mass at any instant, rotate the drum scale by turning the drum drive knob until the point returns to the index. The percent is read directly from the scale. However, this percent ($w'$) is the percent of water based upon the initial mass (i.e. wet mass) of the sample. The water content ($w$) based on the dry mass is computed from Eq. 2.1 (d)

$$w = \frac{w'}{1-w'}$$

Provision is made to adjust the input voltage to the infrared lamp to control the heat for drying the specimen. A suitable thermometer graduated from 40°C to 150°C is provided for ascertaining the temperature of drying in the pan housing. Normally, the temperature is kept between 110° ± 5°C. The time required for the test depends upon the type of soil and the quantity of water present the normal time varies between 15 to 30 minutes. Drying and weighing occur simultaneously, and hence the method is specially suitable for those soils which re-absorb moisture quickly on drying. The criterion for taking the final reading is that the pointer should remain ~~quickly~~ steady on index mark which shows that the sample has dried to constant mass.

Specific Gravity:-

The specific gravity of soil solids is determined by: i) a 50ml density bottle, or (ii) a 500 ml flask, or (iii) a pycnometer. The density bottle method is the most accurate, and is suitable for all types of soils. The flask or pycnometer is used only for coarse grained soils. The density bottle method is the standard method used in the laboratory. However, in all the three methods, the sequence of observation is the same. The mass $M_1$ of the empty, dry, bottle (or flask or pycnometer) is first taken. A sample of oven-dried soil, cooled in a desiccator is put in the bottle, and the mass $M_2$ is taken. The bottle is then filled with distilled water (or kerosene) gradually, removing the entrapped air either by applying vacuum or by shaking the bottle. The mass $M_3$ of the bottle, soil and water (fill up to the top) is taken. Finally the bottle is emptied completely and throughly washed, and clean water (or kerosene) is filled to the top, and the mass $M_4$ is taken. Based on these four observations, the specific gravity can be computed as follows (Fig 3.3)

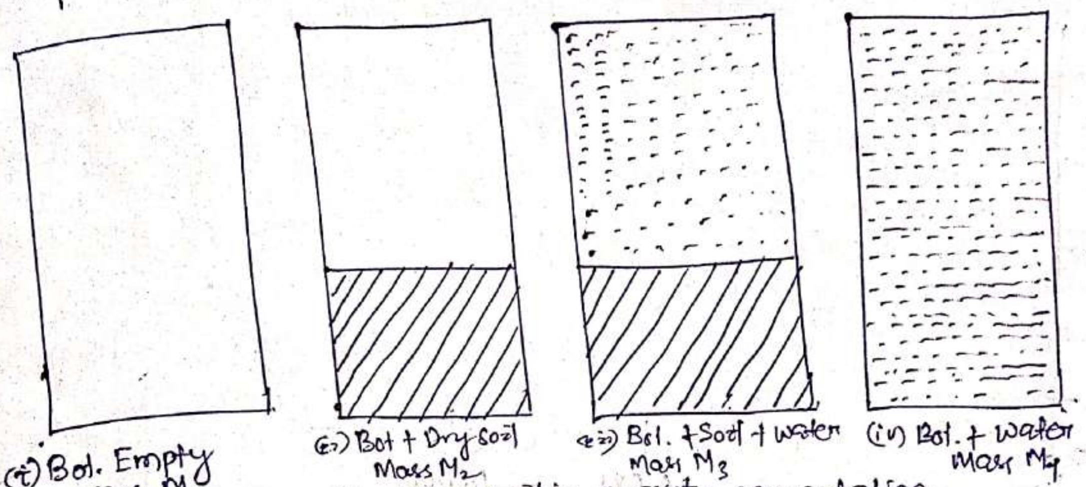



Fig 3.3. specific gravity computation

From Fig 3.3 (i) and (ii), dry mass $M_d$ of the soil is $M_d = M_2 - M_1$ .. (a)

Mass of water in (iii) $= M_3 - M_2$; Mass of water in (iv) $= M_4 - M_1$

Hence, mass of water having the same volume as that of soil solids is

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

$$\text{Now } G = \frac{\text{Dry mass of soil}}{\text{Mass of water of equal volume}} = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$$

$$\Rightarrow G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)} = \frac{M_d}{M_d - (M_3 - M_4)} \quad \text{-- (3.3)}$$

When a 500 ml flask is used, mass is taken to an accuracy of 0.1 g. In case of pycnometer, mass measurements are taken to 1 g accuracy. In both the cases, the entrapped air is expelled by stirring and distilled water is used. However, in the density bottle method, mass measurements are taken to an accuracy of 0.001 to 0.005 g and kerosene is used since it is a better wetting agent. In this case, specific gravity of soil solids is given by

$$G = \frac{M_d \cdot G_K}{M_d - (M_3 - M_4)}$$

where $G_K$ is the specific gravity of kerosene at the test temperature $T°C$.

**Ex. 3.1** In order to determine the water content, 370 g of a wet sandy sample was placed in a pycnometer. The mass of the pycnometer, sand and water full to the top of the conical cap was found to be 2148 g. The mass of pycnometer full of clean water was 1932 g. Taking $G = 2.65$, determine the water content of the sample.

**Ans**

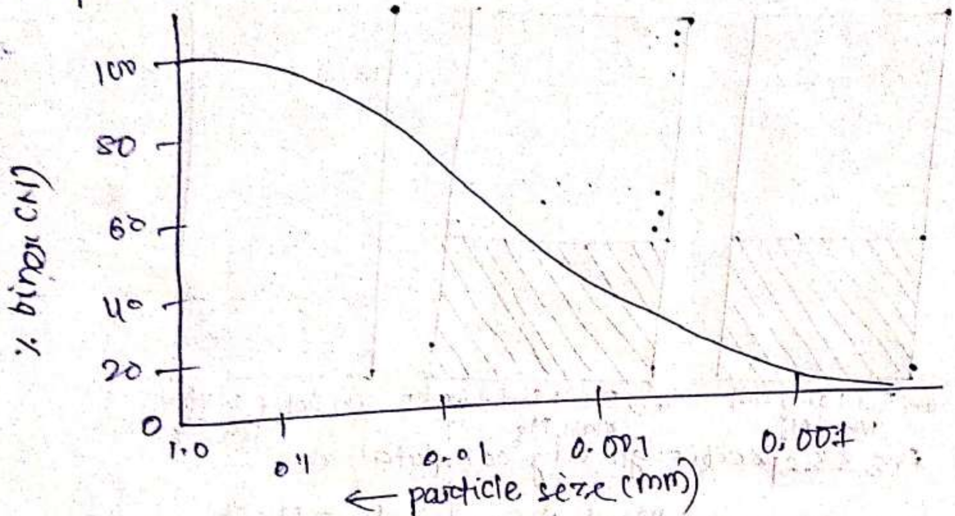
$$W = \left[\frac{M_2 - M_1}{M_3 - M_4} \cdot \frac{G-1}{G} - 1\right] \times 100 = \left[\frac{M}{M_3 - M_4} \cdot \frac{G-1}{G} - 1\right] \times 100$$

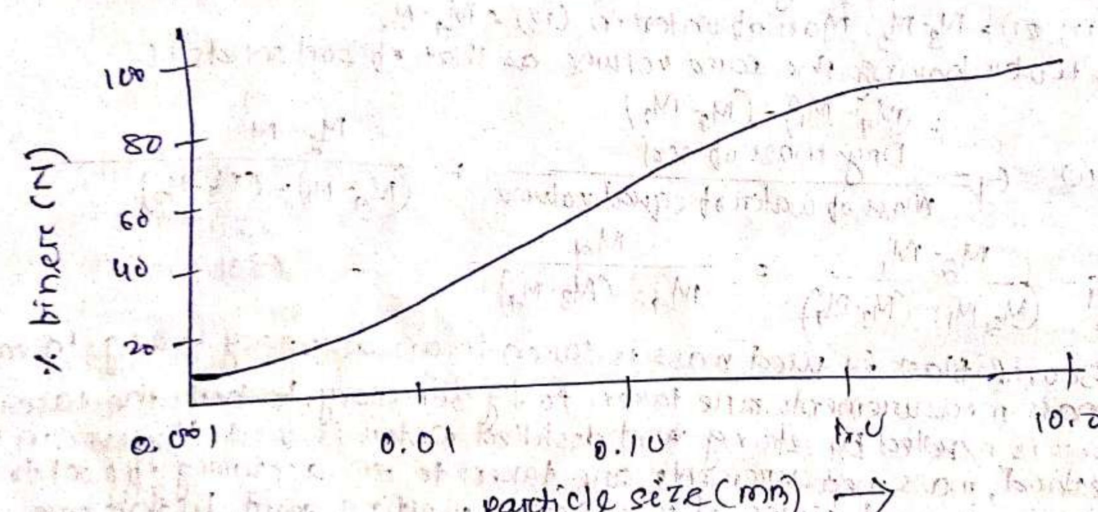
where $M$ = wet mass of soil $= 370$ g, $M_3 = 2148$ g, $M_4 = 1932$ g

$$W = \left[\frac{370}{2148 - 1932} \times \frac{2.65 - 1}{2.65} - 1\right] \times 100 = 6.5\%$$

## Particle Size Distribution Curve

The particle size distribution curve, also known as a gradation curve, represents the distribution of particles of different sizes in the soil mass. The percentage finer N than a given size is plotted as ordinate (on natural scale) and the particle size as abscissa (on log scale). In Fig 3.7(a), the particle size decreases from left to right, whereas in fig 3.7(b) the particle size increases from left to right.



## Sieve Analysis

The soil is sieved through a set of sieves. Sieves are generally made of spun brass and phosphor bronze (or stainless steel) sieve cloth. According to IS: 1498-1970, the sieve are designated by the size of square opening in mm or microns (1 micron = $10^{-6}$ m = $10^{-3}$ mm). Sieves of various size ranging from 80mm to 75 μ are available. The diameter of the sieve is generally between 15 to 20cm.

As mentioned before, the sieve analysis is done for coarse-grained soils. The coarse-grained soils can be further sub-divided into gravel fraction (size > 4.75mm) and sand fraction (75μ < size < 4.75 mm), where Greek letter μ is used to represent micron. A set of sieve (coarse) consisting of the sieves of size 2mm, 1mm, 600μ, 425μ fraction. However all the sieves may not be required for a particular soil. The selection of the required number of sieves is done to obtain a good particle size distribution curve. The sieves are stacked one over the other with decreasing size from the top of the bottom. Thus the sieve of the largest opening is kept at the top. A lid or cover is placed at the top of the largest sieve. A receiver known as pan which has no opening is placed at the bottom of the smallest sieve.

### (a) Dry Sieve Analysis :

The soil sample is taken in suitable quantity as given in Table 3.1. The largest the particle size, the greater is the quantity of soil required.

The soil should be oven-dry. It should not contain any lump. If necessary, it should be pulverized. If the soil contain organic matter, it can be taken air-dry instead of oven dry.

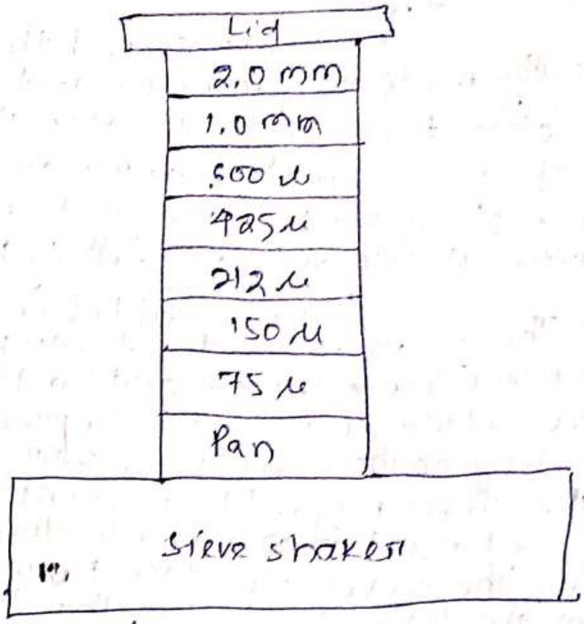
The sample is sieved through a 4.75mm IS sieve. The portion retained on the sieve i.e. the ~~grave~~ gravel fraction or plus 4.75mm material. The gravel fraction is sieved through the set of coarser sieves manually or using a mechanical shaker. Hand sieving is normally done. The weight of soil retained on each sieve is obtained.

The minus 4.75mm fraction is sieved through the set of fine sieves. The sample is placed in the top sieve and the set of sieves is kept on a mechanical shaker (Fig 3.1) and the machine is started. Normally, 10 minutes of shaking is sufficient for most soils. The mass of soil retained on each sieve and on pan is obtained to the nearest 0.1 gm. The mass of the retained soil is checked against the original mass.

Dry sieve analysis is suitable for cohesionless soil with little or no fines. If the sand is sieved in wet conditions the surface tension may cause a slight increase in the size of the particles and the particles smaller than the aperture size may be retained on the sieves and the result could be erroneous.

Fig 3.1 (Quantity of soil for Sieve Analysis)

| Sl. No | Maximum size | Quantity (Kg) |
|---|---|---|
| 1. | 80mm | 6.0 |
| 2. | 20mm | 6.5 |
| 3. | 4.75mm | 0.5 |




(b) Wet Sieve Analysis :-

If the soils contains a substantial quantity (say, more than 5%) of fine particles, a wet sieve analysis is required. All lumps are broken into individual particles. A representative soil sample in the required quantity is taken, using a riffer and dried in an oven. The dried sample is taken in a tray and soaked with water. ~~The dried sample is taken in~~ If deflocculation is required, sodium hexameta phosphate at the rate of 2g per litre of water is added. The sample is stirred and left for a soaking period of at least one hour. The slurry is then sieved through a 4.75mm IS sieve and washed with a jet of water. The material retained on the sieve is the gravel fraction. It is dried on an oven and sieved through set of coarse sieves.

The material passing through 4.75mm sieve is sieved through a 75µ sieve. The material is washed until the wash water becomes clear. The material retained on the 75µ sieves is collected and dried in an oven. It is then sieved through the set of fine sieves of the particle size 2mm, 1mm, 600µ, 425µ, 212µ, 150µ collected and weighed. The material that could have been retained on pan is equal to the total mass of soil minus the sum of the masses of material retained on all sieves.

Pipette Method

In this method, 500ml of soil suspension is required. The process for preparation of 1000ml of suspension has been discussed. All the quantities required for 1000ml of suspension are halved to get 500ml of suspension. The suspension is taken in a sedimentation tube. Fig 3.3 shows a 10ml capacity pipette used for extraction of the sample. The pipette is fitted with a suction inlet.

Calibration of pipette:

For determination of the volume of pipette, it is calibrated before use. For calibration, the nozzle of the pipette is immersed in distilled water. The stop cock S is closed. The three-way stop cock T is opened and the water is sucked up into the pipette until it raises in safety bulb. The stop cock T is now turned the other way round to connect it to the wash outlet to drain the excess water from the safety bulb. The stop cock is then turned the other way round to discharge the water contained in the pipette into a glass weighing bottle. The mass of water in the bottle in grams is equal to the volume of the pipette in ml.

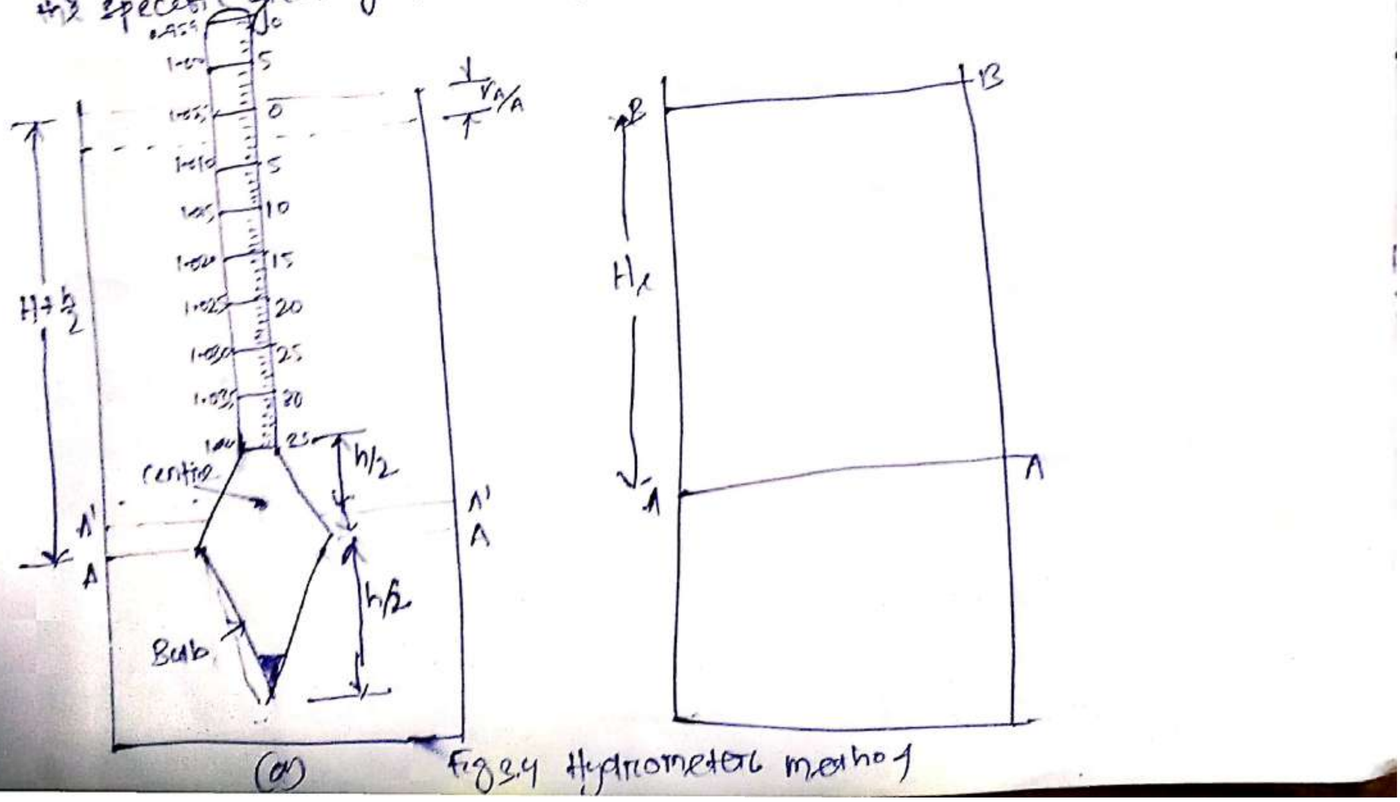
Merits and Demerits of the pipette method

The pipette method is a standard laboratory method for the particle size analysis of fine-grained soils. It is a very accurate method. However, the apparatus is quite delicate and expensive. It requires a very sensitive weighing balance. For quick particle size analysis, the hydrometer method

Hydrometer Method

A hydrometer is an instrumental used for the determination of the specific gravity of liquids. As the specific gravity of the soil suspension depends upon the particle size a hydrometer can be used for the particle size analysis. A special type of hydrometer with a long stem (neck) is used. The stem is marked from top to bottom generally in the range of 0.995 to 1.030 (Fig 3.4) At the time of commencement of top to bottom generally in the range 0.995 to 1.030. At the time of commencement of sedimentation, the specific gravity of suspension is uniform at all depth. When the sedimentation takes place, the larger particles settle more deeper than the smaller ones. This results in non-uniform specific gravity of the suspension at different depths. The lower layers of the suspension have specific gravity greater than that of the upper layers.

Casagrande has shown that the hydrometer measure the specific gravity of suspension at a point indicated by the centre of the immersed volume if the volume of the stem is neglected, the centre of the immersed volume the hydrometer is the same as the centre of the bulb. Thus, the hydrometer gives the specific gravity of the suspension at the centre of the bulb.




(a) Fig 3.4 Hydrometer method

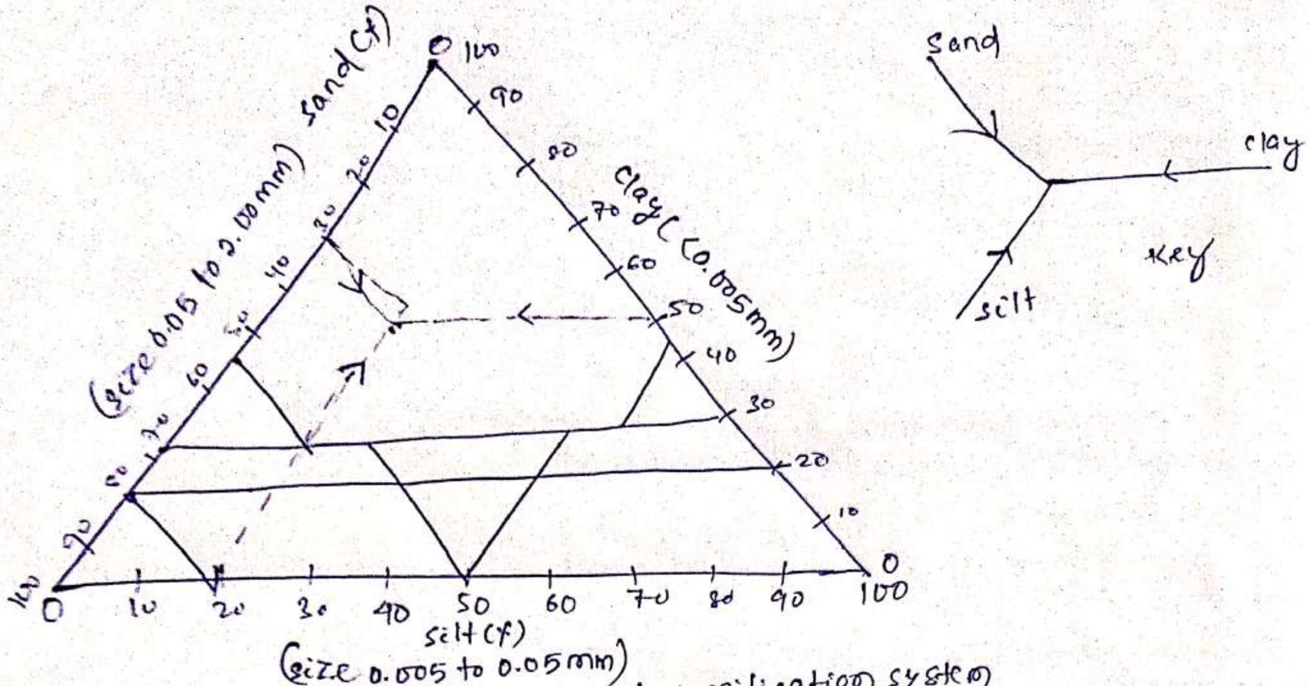
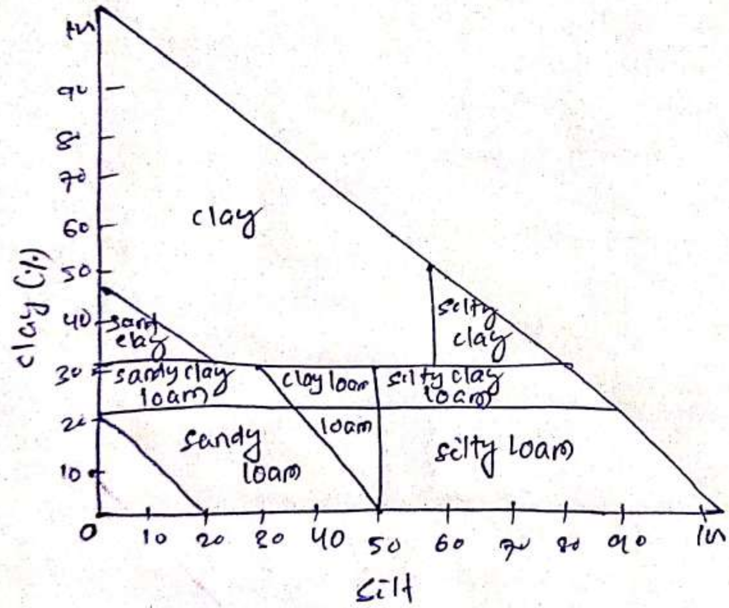



Fig 5.2 Textural classification system



# CLASSIFICATION OF SOIL



## General

The purpose of soil classification is to arrange various types of soils into groups according to their engineering or agricultural properties and various other characteristics. Soil processing similar characteristics can be placed in the same groups. Soil survey and soil classification are carried out by several agencies for different purpose. For example, the agriculture departments undertake soil investigation from the point of view, the classification may be done with the objective of finding the suitability of the soil for constructions of dams, highways or foundation, etc. For general engineering purposes, soils may be classified by the following:

1. Particle size classification
2. Textural classification
3. Highway Research Board (HRB) classification
4. Unified soil classification and IS classification system

## 1. Particle size distribution classification

The size of individual particles has an important influence on the behaviour of soils. It is not surprising that the first classification of soils was based on the particle size. It is a general practice to classify the soils into four broad groups, namely gravel, sand, silt size and clay size on the basis of particle size. While classifying the fine grained soils on the basis of particle size. It is a good practice to write silt size and clay size and not just silt and clay. In general usage, the terms silt and clay are used to denote the soils that exhibit plasticity and cohesion over a wide range of water content. The soil with clay size particles may not exhibit the properties associated with clays. For example, rock flour has the particles of the size of the clay particle but does not possess plasticity. It is classified a clay size and not just clay in the particle size classification system.

Any system of classification based only on particle size may be misleading for fine-grained soils. The behaviour of such soils depends on the plasticity characteristics and not on the particle size. However classification based on particle size in of immense value in the case of coarse grained soils, since the behaviour of such soils depends mainly on the particle size.

## 2. Textural Classification

Texture means visual appearance of the surface of a material such as fabric or cloth. The visual appearance of a soil is called its texture. The texture depends upon the particle size, shape of particles and gradation of particles. The textural classification incorporates only the particle size as it is difficult to incorporate the other two parameters.

In facts, all the classification systems based on the particle size, as discussed in section 5.2 are textural classification system. However in soil engineering, the term textural classification is used rather in a restricted sense. The triangles classification system suggested by U.S Bureau of Public Roads in commonly known as the textural classification system. The term texture is used to express the % of the three constituents of soils namely sand, silt and clay.

According to textural classification system, the percentage of sand (size 0.05 to 2.0mm), silt (0.005 to 0.05) and clay (size less than 0.005mm) are plotted along the three sides of an equilateral triangle. The equilateral triangle is divided into 10 zones, each zones indicates a type of soil. The soil can be classified by determining the zone in which it lies. A key is given that indicates the direction in which the lines are to be drawn to locate the point. For example, if a soil contains 30% sand and 20% silt and 50% clays it is shown by point (a) in the figure. The point follows the zone lebelled clay. Therefore, the soil is classified as clay.

The textural classification system is useful for classifying soils consisting of different constituents. The system assumes that the soil doesn't contain particles larger than 2.0mm, a correction is requirement in which the sum of the percentage of sand, silt and clay is increased to 100%. For example if a soil contains 20% particles of size larger than 2mm size the actual sum of the percentage of sand, silt and clay particles is 80% Let these be respectively 12, 24 and 44%. The corrected % would be obtained by multiplying with a factor of 100/80. Therefore the corrected percentage are 15,30 and 55%. The textural classification of the soil would be done based on these corrected percentage.

In this system, the term loam is used to describe a mixture of sand, silt and clay particles in various proportions. The term loam originates in agricultural engineering where the suitability of a soil is judged for crops. The term is not used in soil engineering. In order to eliminate the term loam, the Mississipe River Commission (USA) proposed a modified triangular diagram (Fig 5.2). The term loam is replaced by soil engineering terms such as silty clay. The principal components of a soil is taken as silty clay. The principal components of a soil is taken as a noun and the less prominent component as an adjactive. For example, silty clay contains mainly particles of clay, but some silt particles are also present. It must be noted that the primary soil type with respect to behaviour is not necessary the soil type that constitutes the largest part of the sample. for example, the general character of a mixed soil is determined by clay fraction of it exceeds 30%.

Right Triangle chart.

Since the sum of the percentage of sand, silt and clay size particles is 100% there is no need to plot all the three percentage. The percentage of sand particles can be found by deduction of the sum of % of silt and clay particles from 100%. It is possible to determine the texture classification by locating the point of intersection of lines represents silt and clay as shown in right triangle chart (Fig ...)

The right tangle chart is more convenient than the conventional triangular chart as it involves only orthogonal arrangement of grid lines.

AASHTO classification system

American Association of state Highway and Transport Officials (AASHTO) classification system is useful for classifying soils for highways. The particle size analysis and the plasticity characteristics are required to classify a soil. The classification system is a complete classify system which classified both coarse-grained and fine grained soils. In this system, the soils are divided into 7 types of soil designated as A-1 to A-7. The soils A-1 and A-7 are further subdivided into two categories and the soil A-2 into four categories as shown in Table 5.1

To classify a soil, its particle size analysis is done and the plasticity index and liquid limit are determined. With the values of these parameters known are examines the first, (extreme) left column of the table 5.1 and ascertains whether the known parameters satisfy the limiting values in that column. If these satisfy the requirements, the soil is classified as A-1a. If these do not satisfy, one enters the second column (from the left) and determine whether these satisfy the limiting values in that column. The procedure is repeated for the next column until the column is reached when the known parameters satisfy the requirement. The soil is classified as per nomenclature given at the top of that column.

The soil with the lowest number, A-1 is the most suitable as a highway material or subgrade. In general, the lower is the number of soil, the more suitable is the soil. For example, the soil A-4 is better than the soil A-5 in Table 5.1, the column of soil A-3 is to the left of the column for soil A-2. This arrangement is only to determine the classification of the soil. This does not indicate that soil A-3 is more suitable for highways than A-2 soil.

Unified soil classification system

The Unified Soil classification system (USC) was first developed by Casagrande in 1948 and later in 1952 was modified by the Bureau of Reclamation and the Corps of Engineers of the United states of America. The system has also been adopted by American Society of Testing Material (ASTM). The system is the most popular system for use in all types of engineering problems involving soils. The various symbols used are in Table 5-2.

Table 5.2. symbols used in USC system

| | symbols | Description |
|---|---|---|
| Primary | G | Gravel |
| | S | Sand |
| | M | silt (symbol M is obtained from the swedish word 'mo') |
| | C | clay |
| | O | Organic |
| | Pt | Peat |
| secondary | W | Well-graded |
| | P | Poorly graded |
| | M | Non-plastic fines |
| | C | plastic fines |
| | L | Low plasticity |
| | H | High plasticity |

The system uses both the particle size analysis and plasticity characteristics of soils, like AASHTO system. In this system, the soils are classified into 15 groups (Table 5.3). The soils are first classified into two categories
(1) Coarse-grained soils: If more than 50% of the soil is retained on No. 200 (0.075 mm) sieve, it is designated as coarse-grained soil. There are 8 groups of coarse-grained soils.
(2) Fine-grained soils: If more than 50% of the ~~total~~ soil phases No. 200 sieve, it is called fine-grained soil.
There are 6 groups of fine grained soils.
1. Coarse grained soils: The coarse grained soils are classified as gravel (G) if 50% or more of coarse fraction (plus 0.075 mm) is retained on No. 4 (4.75 mm) sieves, otherwise it is termed sand (S).
If the coarse-grained soil contain less than 5% fines and are well-graded (W), they are given the symbols GW and SW and if poorly graded (P), symbol GP and SP. The criteria for well-grading are given in Table 5.3. If the coarse-grained soil contain more than 12% fines, these are designated as GM, GC, SM or SC as per criteria given. If the percentage of fines is between 5 to 12% dual symbols such as GW-GM, SP-SM are used.
2. Fine grained soils - Fine grained soils are further divided into 2 type
(1) Soils of low compressibility (L) if the liquid limit is 50% or less. These are given the symbols ML, CL and OL (2) soils of high compressibility (H) if the liquid limit is more than 50%. These are given the symbols MH, CH and OH. The exact type of soil is determined from the plasticity chart (Fig 5.5). The A-line has the equation $I_p = 0.73(w_L - 20)$. It separates the clays from silts. When the plasticity index and the liquid limit plot in the hatched ~~option~~ portion of the plasticity chart, the soil is given doubles symbol CL-ML.
The inorganic soil ML and MH and the organic soils OL, OH plot in the zones of the plasticity chart. The distinction between the inorganic and organic soils is made by oven-drying. If oven drying decreases the liquid limit by 30% or more, the soil is classified organic (OL or OH). otherwise inorganic (ML or MH).
Highly Organic Soils - Highly organic soils are identified by visual inspection. These soils are termed Peat (Pt).

Indian Standard classification system:

Indian standard classification system adopted by Bureau of Indian standards is in mainly respects similar to the Unified soil classification (USC) system. However, there is one basic difference in the classification of finegrained soil. The fine-grained soils in ISC system are subdivided into three categories of low, medium and high compressibility instead of two categories of low and high compressibility in USC system. A brief outline of classification and identification of soils for general engineering purposes (IS: 1498: 1970) is given below. For complete details the reader should consult the code.
ISCS system classifies the soils into 18 groups as per Table 5.6 and 5.7. Soils are divided into 3 broad division
(1) Coarse-grained soil, when 50% or more of the total material ...

Scanned with OKEN Scanner

weight is retained on 75 micron IS sieve.

(2) Fine-grained soils, when more than 50% of the total material passes 75 micron IS sieve.

(3) If the soil is highly organic and contains a large percentage of coarse matter and particles of decomposed vegetation, it is kept in a ~~sequence~~ separate category marked as Peat (Pt).

Table 5.5 Basic soil components in ISC system

| soil | soil components | symbol | Particle size range and description |
|---|---|---|---|
| (i) coarse grained components | Boulder | None | Rounded to angular, bulky, hard, rock, particle average diameter more than 300mm. |
| | cobble | None | Rounded to angular, bulky, hard, rock particle average diameter smaller than 300mm but retained on 80mm IS sieve. |
| | Gravel | G | Rounded to angular, bulky, hard, rock particles passing 80mm IS sieve but retained on 4.75mm IS sieve<br>coarse: 80mm to 20mm IS sieve<br>Fine: 20mm to 4.75mm IS sieve |
| | sand | S | Rounded to angular bulky, hard, rock particle, passing 4.75 mm IS sieve, but retained on 75 micron sieve<br>coarse - 4.75mm to 2.0mm IS sieve<br>Medium - 2.0mm to 425 micron IS sieve<br>Fine:- 425 micron to 65 micron IS sieve |
| (ii) Fine-grained components | silt | M | Particles smaller than 75 micron IS sieve, identifies by behaviour that 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 IS sieve identified by behaviour, that is, it can be made up to exhibit plastic properties within a certain considerable strength when air dried. |
| | organic matter. | O | Organic matter in various sizes and stages of decomposition. |

Plasticity chart:-

plasticity chart, developed by Arthur Casagrande (1932) is a plot of the plasticity index (PI) versus the liquidity Index (LL) of soils. The chart is used for the classification of fine grained soils (or fine grained fraction of coarse grained soils) based on their plasticity.

The plasticity chart comprises of two important lines. A-line and U-line. A-line is an empirically choosen line which separates the chart between clays and silts, soils that fall above A-line are classified as clays and that falling below as silts. A-line is given by the equation $PI = 0.73(LL-20)$

U-line lies above the A-line and is approximately the upper limit of the relationship of PI to LL, for any currently known soil. The equation of U-line is given as $PI = 0.9(LL-8)$

There is also a vertical line in the plasticity chart which corresponds to a liquid limit of 50% and separates the high plasticity fine grained soils (LL > 50%) from low plasticity fine grained soil (LL < 50)

To classify as soil based on the plasticity chart, plot the

PI and LL of the soil on the chart : the region in which the point falls indicates the type of fine-grained soil it is.

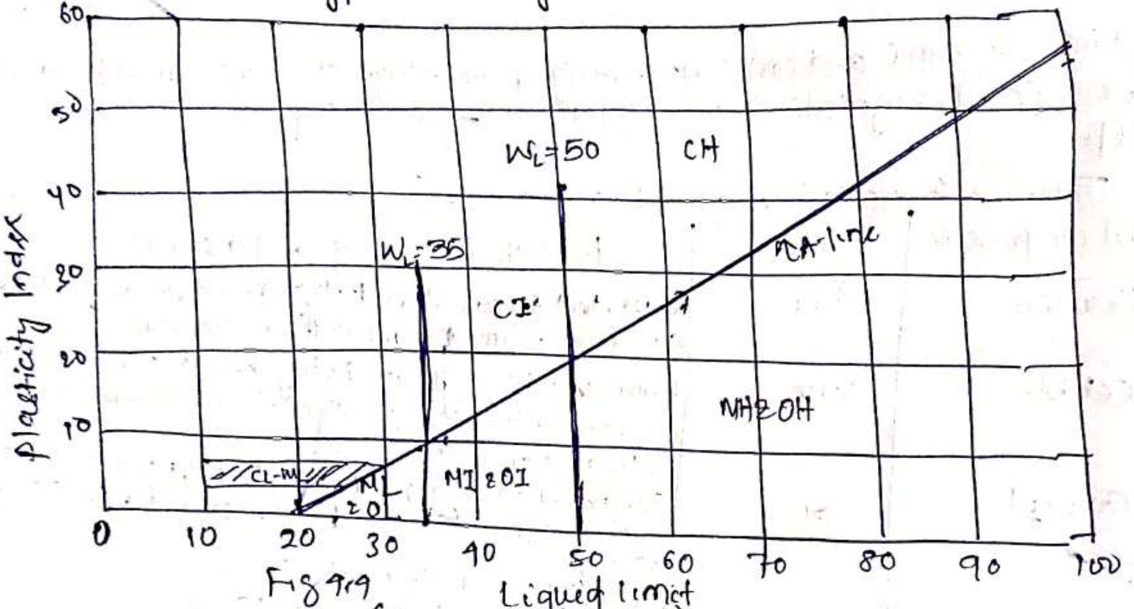



Fig 4.4
(Plasticity chart)

1. Sketch the plasticity chart used for classifying fine grained soil in the IS soil classification system.
Give the group symbols for the following soils
(i) Liquid limit = 40%, plastic limit = 22%
(ii) Liquid limit = 20% plastic limit = 14%
(iii) Passing 4.75mm sieve = 70%. Passing 75 micron sieve = 8%.
Uniformity coefficient = 7, Coefficient of curvature = 3,
plasticity index = 3

Ans Fig 4.4, shows the plasticity chart of the IS system
(i) $W_L = 40\%$, $W_P = 22\%$, $I_P = 40 - 22 = 18\%$.
plotting the point for $I_P = 18\%$ and $W_L = 40\%$ on the plasticity chart, group symbol for the soil will be CI.
(ii) $W_L = 20\%$, $W_L = 14\%$ ∴ $I_P = 6\%$
Plotting the point for $I_P = 60\%$ and $W_L = 20\%$, the soil falls in the CL-ML section
(iii) Since more than half the portion (70%) of the soil passes through 4.75mm ~~falls in the CL-ML section~~ sieve, the soil is essentially sandy (S), Referring to Table 4.8. Since $C_u = 7$ (> and $C_c = 3$, the soil is of SW group. However, since percentage passing 75-micron size is 8% (between 5 and 12%), it is a board-line case. Also, since $I_P = 3$ (less than 4). It satisfies the requirement of SM. Hence the soil may be designated as SW-SM.

# Permeability CH-5

Permeability is defined as the property of a porous material which permits the passage or seepage of water (or other fluids) through its interconnecting voids. A material having continuous voids is called Permeable. Gravels are highly permeable while ~~stiff~~ clay is the least permeable, and hence a clay may be termed impermeable for all practical purposes.

The flow of water through soils may either be a laminar flow or a turbulent flow. In laminar flow, each fluid particle travels along a definite path which never crosses ~~due~~ the path of any other particle. In turbulent flow, the paths are irregular and twisting, crossing and recrossing at random (Taylor, 1948). In most of the practical flow problems in soil mechanics, the flow is laminar.

The study of seepage of water through soil is important for the following engineering problems.

1. Determination of rate of settlement of a saturated compressible soil layer
2. Calculation of seepage through the body of earth dams and stability of slopes.
3. Calculation of uplift pressure under hydraulic structures and their safety against piping
4. Ground water flow towards wells and drainage of soil.

## Darcy's law :-

The law of flow of water through soil was first studied by Darcy (1856) who demonstrated experimentally that for laminar flow conditions in a saturated soil, the rate of flow or the discharge per unit time is proportional to the hydraulic gradient.

$$q = KiA$$

$$\text{or } V = \frac{q}{A} = Ki \quad \ldots \quad (7.2)$$

where $q$ = discharge per unit time

$A$ = total cross-sectional area of soil mass, perpendicular to the direction of flow

$i$ = hydraulic gradient

$K$ = Darcy's coefficient of permeability

$V$ = velocity of flow, or average discharge velocity

If a soil sample of length $L$ and cross-sectional area $A$, is subjected to differential head of water, $h_1 - h_2$, the hydraulic gradient $i$ will be equal to $\frac{h_1 - h_2}{L}$ and,

we have $$q = K\frac{h_1 - h_2}{L}A \quad \ldots \quad 7.1(a)$$

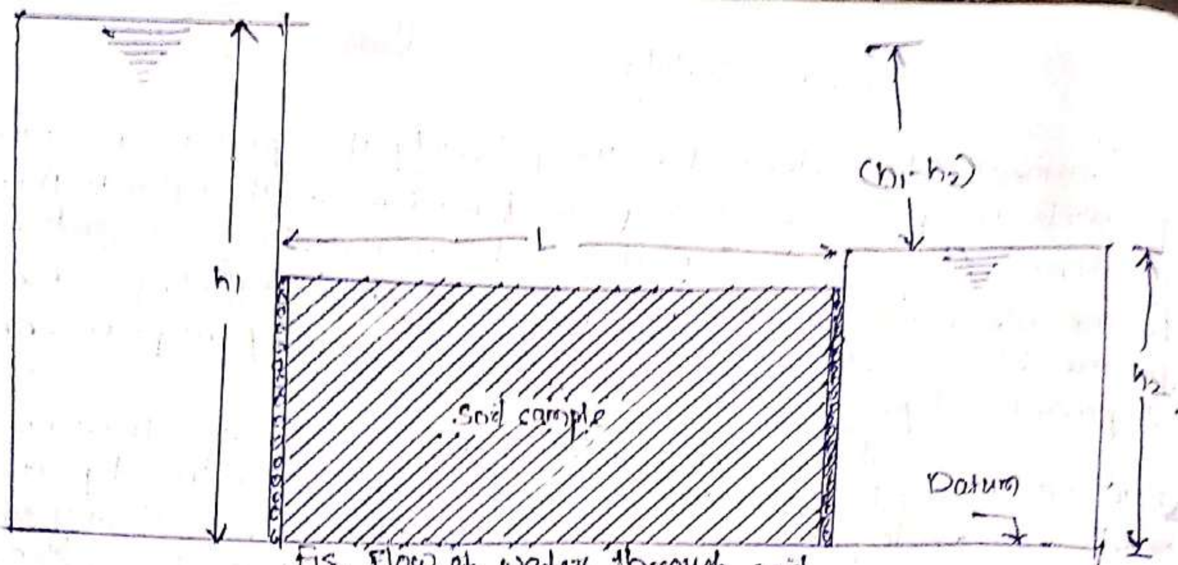



Fig. Flow of water through soil

From Eq. 7.2, when hydraulic gradient is unity, K is equal to v. Thus, the coefficient of permeability, or simply permeability is defined as the average velocity of flow that will occur through the total cross-sectional area of soil under unit hydraulic gradient. The dimensions of the coefficient of permeability K are the same as those of velocity. It is usually expressed as cm/sec or m/day or feet/day.

Table 7.1 gives some typical values of co-efficient of permeability of various soils. 7.1 Typical values of k

| Soil Type | | Co-efficient of permeability cm/sec |
|---|---|---|
| clean gravel | $\longrightarrow$ | 1.0 and greater |
| clean sand (coarse) | $\longrightarrow$ | $1.0 - 1 \times 10^{-2}$ |
| sand (mixture) | $\longrightarrow$ | $1 \times 10^{-2} - 5 \times 10^{-2}$ |
| fine sand | $\longrightarrow$ | $5 \times 10^{-2} - 1 \times 10^{-3}$ |
| silty sand | $\longrightarrow$ | $2 \times 10^{-3} - 1 \times 10^{-4}$ |
| silt | $\longrightarrow$ | $5 \times 10^{-4} - 1 \times 10^{-5}$ |
| clay | $\longrightarrow$ | $1 \times 10^{-6}$ and smaller |

## Determination of coefficient of permeability:-

The coefficient of permeability can be determined by the following:

(a) Laboratory methods
  (1) Constant head permeability test
  (2) Falling head permeability test

(b) Field methods
  (1) pumping-out tests. (2) pumping-in tests.

(c) Indirect methods
  (1) Computation from grain size or specific surface
  (2) Horizontal capillary test
  (3) Consolidation test data.

Permeability can be determined in the laboratory by direct measurement with the help of permeameter by allowing the water to flow through soil sample either under constant head or under variable head. Permeability can also be determined directly by field test. The indirect method of computing the permeability from consolid. test data

## Factors Affecting Permeability

Eq 7.17 is the Poiseuille's law adapted for the flow through the soil pores. Comparing it with the Darcy's law: $q = KiA$.

We get,
$$K = D_s^2 \cdot \frac{\gamma_w}{\eta} \cdot \frac{e^3}{1+e} C$$

Thus, the factors affecting permeability are:

1. Grain size
2. Properties of the pore fluid
3. Voids ratio of the soil.
4. Structural arrangement of the soil particles
5. Entrapped air and foreign-matter and
6. Adsorbed water in clayey soils.

1. Effect of size and shape of particles

Permeability varies approximately as the square of the grain size. Since soils consist of many different-sized grains, some specific grain size has to be used for comparision. Allen Hazen (1892) based on his experiments on filter, sands of particle size between 0.1 and 3mm, found that the permeability can be expressed as

$$K = CD_{10}^2 \quad \ldots \quad (1)$$

where, $K$ = coefficient of permeability (cm/sec)
$D_{10}$ = effective diameter (cm)
$C$ = constant, approximately to 100, when $D_{10}$ is expressed in centimeter

Attempt have been made to correlate the permeability with specific surface of the soil particles. One such relationship is given by Kozeny (1907)

$$K = \frac{1}{K_K \eta S_s^2} \times \frac{n^3}{1-n^2} \quad \ldots \quad (2)$$

where, $K$ = coefficient of permeability (cm/sec per unit hydraulic gradient)
$n$ = porosity
$S_s$ = specific surface of particles (cm²/cm³).
$\eta$ = viscosity (g·sec/cm²)
$K_K$ = constant, equal to 5 for spherical particles

On the basis of his experiments, Loudon (1952-53) developed the following empirical formula.

$$\log_{10}(KS_s^2) = a + bn \quad \ldots \quad (3)$$

where $a, b$ are constants, the values of which are 1.365 and 5.15 respectively for permeability at 10°C. As demonstrated by Loudon's experiments, the permeability of coarse grained soils is inversely proportional to the specific surface at a given porosity.

2. Effect of properties of pore fluid: Eq. 7.18 indicates that the permeability is directly proportional to the unit weight of water and inversely proportional to its viscosity. Through the unit weight of water does not change much with the change in temperature there is great variation in viscosity with temperature. Hence, when other factors remain constant, the effect of the property of water on the values of permeability can be expressed as

It is usual to convert the permeability results to a standard temperature (27°C) for comparision purposes by expression.

$$K_{27} = K\frac{\eta}{\eta_{27}}$$

However, if change in the unit weight of water due to temperature is also taken into account, we have the more general equation.

$$\frac{K_1}{K_2} = \frac{\eta_2}{\eta_1}\cdot\frac{\gamma_{w1}}{\gamma_{w2}} = \frac{\eta_2}{\eta_1}\cdot\frac{S_{w1}}{S_{w2}} \quad \text{.... (1)}$$

~~It is re~~

where, $K_{27}$ = permeability at 27°C, $\eta_{27}$ = viscosity at 27°C

K = permeability at test temperature

$\eta$ = viscosity at test temperature

Muskat (1937) pointed out that a more general ~~significant~~ coefficient of permeability, called the physical permeability $K_p$ is related to the Darcy's coefficient of permeability K as follows:

$$K_p = K\frac{\eta}{\gamma_w} \quad \text{.... (2)}$$

In any soil, $K_p$ has the same value for all fluids and all temperatures as long as the void's ratio and the structure of the soil skeleton are not changed.

**3. Effect of voids ratio:** Eq. 7.18 indicates that the effect of void ratio on the values of permeability can be expressed as

$$\frac{K_1}{K_2} = \left[\frac{C_1e_1^3}{1+e_1}\right] \div \left[\frac{C_2e_2^3}{1+e_2}\right] \quad (3)$$

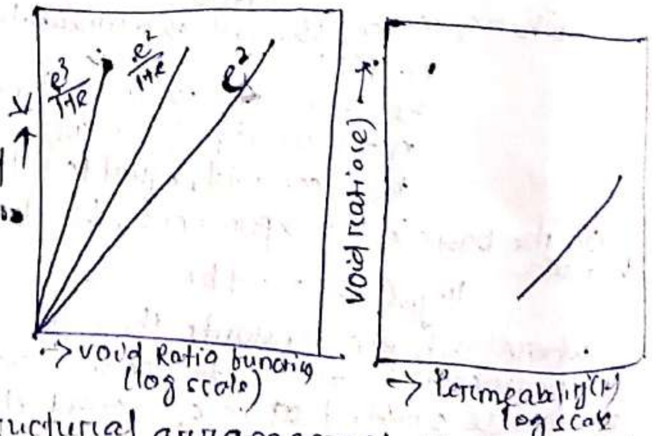
Laboratory experiments have shown that the factor C changes very little with the change in the voids ratio of un-stratified sand samples. However, for clays, it varies appreciably. Thus, for coarse ~~sand~~ grained soil. Eq. 7.29 reduces to

$$\frac{K_1}{K_2} = \frac{e_1^3}{1+e_1} \div \frac{e_2^3}{1+e_2} = \frac{e_1^3}{1+e_1}\cdot\frac{1+e_2}{e_2^3} \quad \text{... (3(a))}$$

Based on another concept of mean hydraulic radius for the soils, the following relationship is obtained

$$\frac{K_1}{K_2} = \frac{e_1^2}{e_2^2} \quad \text{--- 3(b)}$$

It has been found that a semi-logarithmic plot of void ratio versus permeability is approximately a straight line for both coarse grained as well as fine grained soil (Fig 7.4)




**4. Effect of structural arrangement of particles and stratification:** The structural arrangement of the particle may vary, at the same voids ratio, depending upon the method of deposition or compacting the soil mass. The structure may be entirely different for a disturbed sample as compared to an undisturbed sample which may possess stratification. The effect of structural disturbance on permeability is much pronounced in fine-grained soils. Stratified soil masses have marked variation in their permeabilities in direction parallel and perpendicular to stratification, the permeability parallel to the stratification being always greater ~~perpendicular to stratification, the permeability parallel to the stratification~~ (8.7.13). When flow through natural soil deposits is under consideration, permeability should be determined on undisturbed soil as its natural structural arrangement.

5) Effect of degree of saturation and other foreign matters:
The permeability is greatly reduced if air is entrapped in the voids thus reducing its degree of saturation. The dissolved air in the pore fluid (water) may get liberated, thus changing the permeability. Ideal condition of test are when air-free distilled water is used and the soil is completely saturated by vacuum saturation, for measuring the permeability. However, since the percolating water in the field may have some gas content, it may appear more realistic to use the actual field water for testing in the laboratory. Organic foreign matter also has the tendency to move towards critical flow channels and choke them up, thus decreasing the permeability.

6) Effect of adsorbed water: The adsorbed water surrounding the fine soil particles is not free to move, and reduces the effective pore space available for the passage of ~~water~~ void ratio occupied by adsorbed water and the permeability may be roughly assumed to be proportional to the square of the net void ratio $(e-0.1)$

## Constant Head Permeability Test:-

Fig 7.5 shows the diagrammatical representation of constant h test.

Water flows from the overhead tank consisting of three tubes: the inlet tube, the overflow tube and the outlet tube. The constant hydr gradient 'i' causing the flow is the head h (i.e difference in the water of the overhead and bottom tanks) divided by the length L of the sample. the length of the sample is large; the head lost over a length of specim is measured by inserting piezometric tubes, as shown in fig 7.5(b)

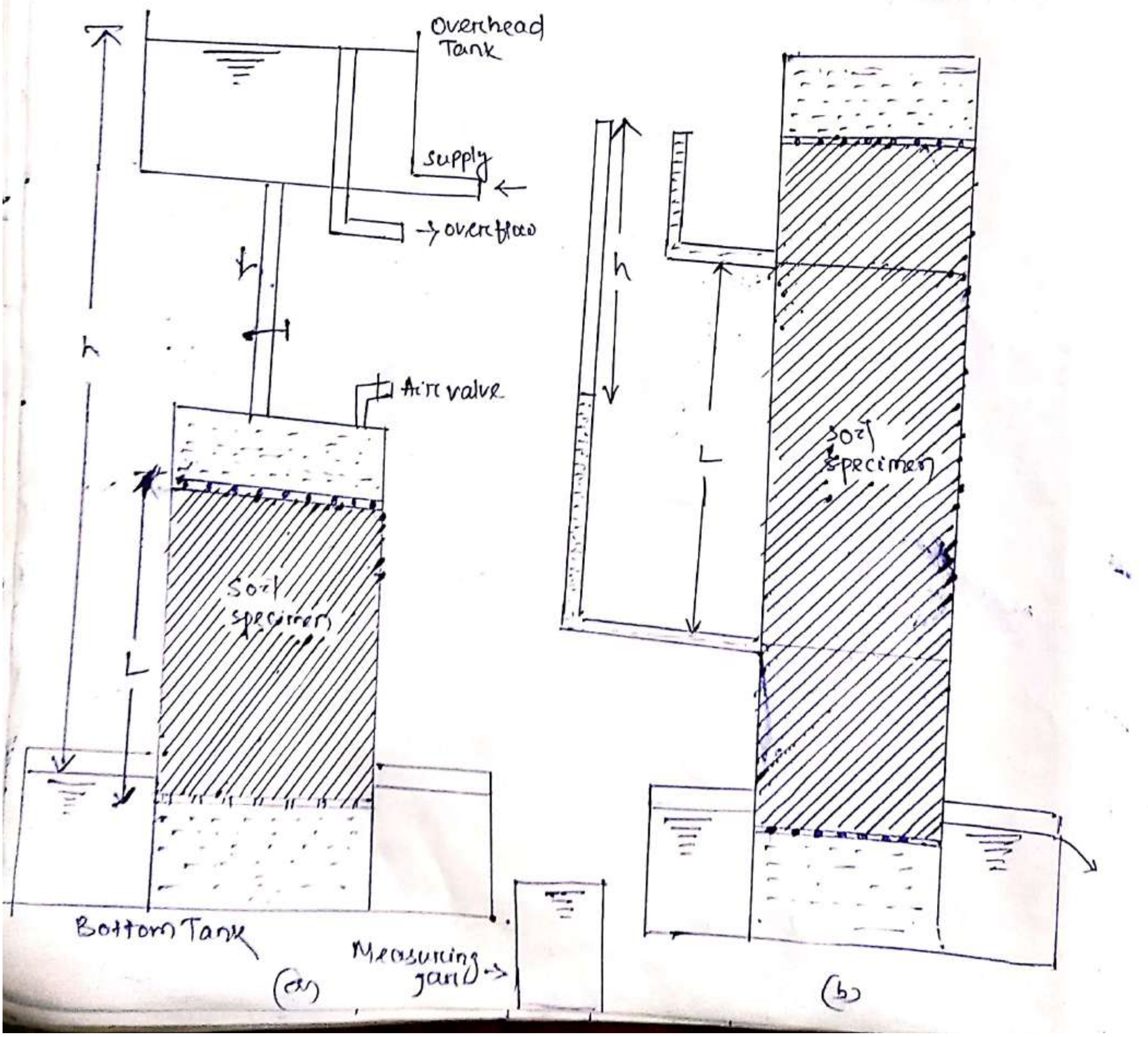
If Q is the total quantity of flow in a time interval t, we h from Darcy's law

$$q = \frac{Q}{t} = kiA$$

$$\therefore K = \frac{Q}{t} \cdot \frac{1}{iA} = \frac{Q}{t} \cdot \frac{L}{h} \cdot \frac{1}{A}$$

where A = total cross-sectional area of sample

When steady state of flow is reduced, the total quantity of water in time t collected in a measuring jar. The observation are recorded in Table 7.3



Falling head permeability test :

The constant head permeability test is used for coarse grained soil only where a reasonable discharge can be collected in a given time. However, the falling head test is used for relatively less permeable soils when the discharge is small. Fig 7.6 shows the diagrammatical representation of a falling head test arrangement.

A standpipe of known cross-sectional area $a$ is fitted over the permeameter, and water is allowed to run down. The water level in a stand pipe constantly falls as water flows. Observations are started after steady state of flow has reached. The head at any time instant $t$ is equal to the difference in the water level in the stand pipe and the bottom tank. Let $h_1$ and $h_2$ be heads at time intervals $t_1$ and $t_2$ ($t_2 > t_1$) respectively. Let $h$ be the head at any intermediate time interval $t$ and $-dh$ be the change in the head in a smaller time interval $dt$ (minus sign has been used since $h$ decreases as $t$ increases). Hence, from Darcy's law the rate of flow $q$ is given by

$$q = \frac{(dh.a)}{dt} = KiA$$

where $i$ = hydraulic gradient at time $t = \frac{h}{L}$

$$\therefore \frac{Kh}{L}A = -\frac{dh}{dt}a \quad \text{or} \quad \frac{AK}{aL}dt = -\frac{dh}{h}$$

Integrating between two time limits, we get

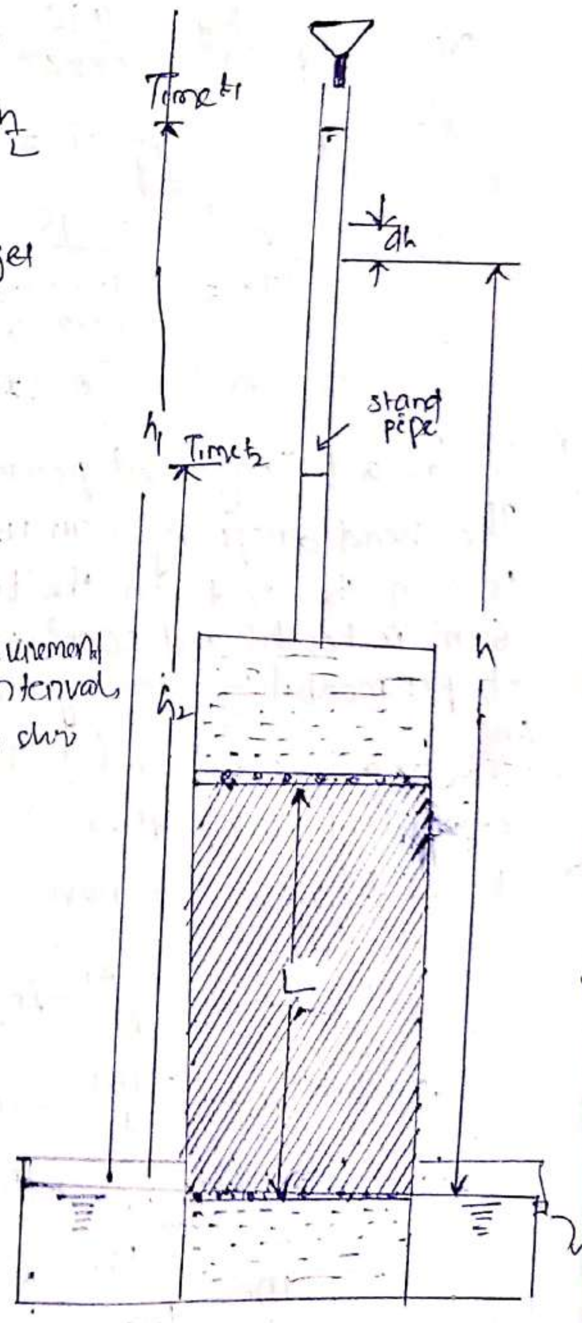
$$\frac{AK}{aL}\int_{t_1}^{t_2} dt = -\int_{h_1}^{h_2}\frac{dh}{h} = \int_{h_2}^{h_1}\frac{dh}{h}$$

$$\Rightarrow \frac{AK}{aL}(t_2 - t_1) = \log_e\frac{h_1}{h_2}$$

Denoting $t_2 - t_1 = t$, we get

$$K = \frac{aL}{At}\log_e\frac{h_1}{h_2} = 2.3\frac{aL}{At}\log_{10}\frac{h_1}{h_2} \quad ...$$

The laboratory observations consist of measurement of the heads $h_1$ and $h_2$ at two choosen time intervals $t_1$ and $t_2$. The observation are recorded as shown Table 7.4




Falling head Test

Ex.7.1 Calculate the co-efficient of permeability of a soil sample, 6cm in height and 50 $cm^2$ in cross-sectional area, if a quantity of water equal to 430 ml passed down in 10 minutes under an effective constant head of 40 cm.

On oven-drying, the test specimen has mass of 498g. Taking the specific gravity of soil solids as 2.65. Calculate the seepage velocity of water during the test.

Ans Given: $Q = 430\ ml$, $t = 10 \times 60 = 600$ second

$A = 50\ cm^2$; $L = 6\ cm$; $h = 40\ cm$

From Eq. 7.26. $K = \frac{Q}{t} \times \frac{L}{h} \times \frac{1}{A} = \frac{430}{600} \times \frac{6}{40} \times \frac{1}{50} = 2.15 \times 10^{-3}$ cm/sec

Now $v = \frac{q}{A} = \frac{430}{600 \times 50} = 1.435 \times 10^{-2}$ cm/sec

Now $\rho_d = \frac{M_d}{V} = \frac{498}{50 \times 6} = 1.66\ g/cm^3$

$\therefore e = \frac{G \rho_w}{\rho_d} - 1 = \frac{2.65 \times 1}{1.66} - 1 = 0.595$

$\therefore n = \frac{e}{1+e} = \frac{0.595}{1.595} = 0.373$

$\therefore V_s = \frac{v}{n} = \frac{1.435 \times 10^{-2}}{0.373} = 3.85 \times 10^{-2}$ cm/sec

Ex.7.2 In a falling head permeameter test, the initial head $(t=0)$ is 40cm. The head drops by 5cm in 10 minutes. Calculate the time required to run the test for the final head to be at 20cm. If the sample is 6cm in height and 50 $cm^2$ in cross-sectional area. Calculate the coefficient of permeability, taking area of stand pipe = 0.5 $cm^2$.

Ans In a time interval $t = 10$ minutes, the head drops initial value of $h_1 = 40$ to $h_2 = 40 - 5 = 35$ cm.

From Eqs 7.26, We have $K = 2.3 \frac{aL}{At} \log_{10} \frac{h_1}{h_2}$

$\Rightarrow t = \frac{2.3\ aL}{AK} \log_{10} \frac{h_1}{h_2} = m \log_{10} \frac{h_1}{h_2}$

where $m = \frac{2.3aL}{AK}$ = constant for the set up

$\therefore 10 = m \log_{10} \frac{40}{35}$

$\Rightarrow m = \frac{10}{\log_{10} \frac{40}{35}} = \frac{10}{0.058} = 172.5$ unit

Scanned with OKEN Scanner

$$t = m\log_{10}\frac{h_1}{h_2} = 172.5\log_{10}\frac{h_1}{h_2}$$

Now, let the time interval required for the head to drop from initial value of $h_1 = 40cm$ to a final value of $h_2 = 20cm$, be t minute.

$$t = 172.5\log_{10}\frac{40}{20} = 172.5\times0.301 = 51.9 \text{ minutes}$$

Again, $m = \frac{2.3aL}{AK} = 172.5$ unit.

$$\therefore K = \frac{2.3aL}{A\times172.5} \text{ cm/minute}$$

(since t used to compute m was in minutes)

$$K = \frac{2.3\times0.5\times6}{50\times172.5\times60} \text{ cm/sec}$$

$$= 1.335\times10^{-5} \text{ cm/sec.}$$

Ex. 7.4 A constant head permeability test was run on a sand sample 16cm in length and 60 cm² in cross-sectional area. Porosity was $n_1 = 40\%$. Under a constant head of 30 cm, the discharge was found to be 45 cm³ in 18 seconds. Calculate the coefficient of permeability. Also, determine the discharge velocity and seepage velocity during the test. Estimate the permeability of the sand for a porosity of $n_2 = 35\%$

Ans

From Eq. 7-26 $K = \frac{Q}{t}\times\frac{L}{h}\times\frac{1}{A} = \frac{45}{18}\times\frac{16}{30}\times\frac{1}{60} = 2.22\times10^{-2}$ cm/s

Discharge velocity, $v = Ki = K\frac{h}{L} = 2.22\times10^{-2}\times\frac{30}{16} = 4.17\times10^{-2}$ cm/s

seepage velocity, $v_s = \frac{v}{n} = \frac{4.17\times10^{-2}}{0.4} = 10.42\times10^{-2}$ cm/s

Again, from Eq. 7.23(a).

$$\frac{K_1}{K_2} = \frac{e_1^3}{1+e_1}\cdot\frac{1+e_2}{e_2^3} = \frac{n_1^3}{(1-n_1)^2} \bigg/ \frac{n_2^3}{(1-n_2)^2}$$

$$K_2 = K_1\frac{\frac{n_2^3}{(1-n_2)^2}}{\frac{n_1^3}{(1-n_1)^2}} = 2.22\times10^{-2}\times\frac{\frac{0.35^3}{(1-0.35)^2}}{\frac{0.4^3}{(1-0.4)^2}} = 1.26\times10^{-2} \text{ cm/s.}$$

## Discharge Velocity and seepage velocity

The Darcy's law (figs 7.1, 7.2) in no way describes the state of flow within individual pores. Darcy's law represents the statistical macroscopic equivalent of the Navier-stokes equations of motion for the viscous flow of ground water. The velocity of flow $v$ is the rate of discharge of water per unit of total cross-sectional area $A$ of soil. This total area of cross-section is composed of the area of solids $A_s$ and area of voids $A_v$. Since the flow takes through the voids, the actual or true velocity of flow will be more than the discharge velocity, is called the seepage velocity $V_s$, and is defined. This actual velocity is called the seepage velocity per unit cross-section as the rate of discharge of percolating water per unit cross-section area of voids perpendicular to the direction of flow.

From the definition of the discharge velocity and seepage velocity we have

$$q = vA = V_s A_v$$

$$\therefore \quad V_s = v\frac{A}{A_v} \; ; \quad \text{But } \frac{A_v}{A} = \frac{V_v}{V} = n$$

$$V_s = v\frac{A}{A_v} = v\cdot\frac{1}{n} = \frac{v}{n} = \frac{1+e}{e}v$$

The seepage velocity $V_s$ is also proportional to the hydraulic gradient

$V_s = K_p i$ (where $K_p$ = coefficient of percolation)

From Darcy's law, $v = ki$ $\quad \therefore \; \frac{V_s}{V} = \frac{K_p}{K} = \frac{1}{n}$

$$K_p = \frac{K}{n}$$

## Seepage Pressure

When water flows through soil pores a viscous friction is exerted on it which causes transfer of energy between soil and water. This water pressure applied on soil which tends it to percolate is called as seepage pressure.

It is generally represented by notation: '$p_s$'

Seepage pressure Formula ($P_s$)

$$P_s = h\gamma_w$$

$$P_s = \frac{h}{L}\times L\gamma_w = iL\gamma_w$$

where, $h$ = hydraulic head

$L$ = length over which head is lost

$i$ = hydraulic gradient

$\gamma_w$ = unit weight of water

Also seepage force ($F_s$) is given by:

$$F_s = p_s \cdot A = i L \gamma_w \cdot A$$

where
$A$ = total cross sectional area of soil mass

seepage force per unit volume is given by:

$$F_s = \frac{i L A \gamma_w}{L \cdot A} = i \gamma_w$$

This seepage pressure will generally act in the direction of flow and the effective pressure in mass of soil is given by.

$$P_e = L\gamma' \pm P_s$$

$$P_e = L\gamma' \pm i L\gamma_w$$

For downward flow it is given as

$$P_e = L\gamma' + i L\gamma_w$$

For upward flow it is given as,

$$P_e = L\gamma' - i L\gamma_w$$

Quick sand condition:

Quick sand condition is a phenomena which is caused by seepage. When the flow of water takes place in upward direction, this condition occurs.

Because of the seepage pressure acting in the upward direction the effective pressure is reduced when the flow of water takes place in upward direction.

When seepage pressure and the weight of submerged soil becomes equal the effective pressure becomes zero.

Particularly in this case the cohesionless soil looses it shear strength and soil particles tends to move in the direction of flow. This overall phenomena due to which lifting of soil occurs is known as quick sand condition or sand boiling condition.

Moreover during the period of this phenomena effective pressure becomes zero.

$$P_e = L\gamma' - P_s = 0$$

$$P_s = L\gamma'$$

$$\Rightarrow i L\gamma_w = L\gamma'$$

$$\Rightarrow i\gamma_w = \gamma'$$

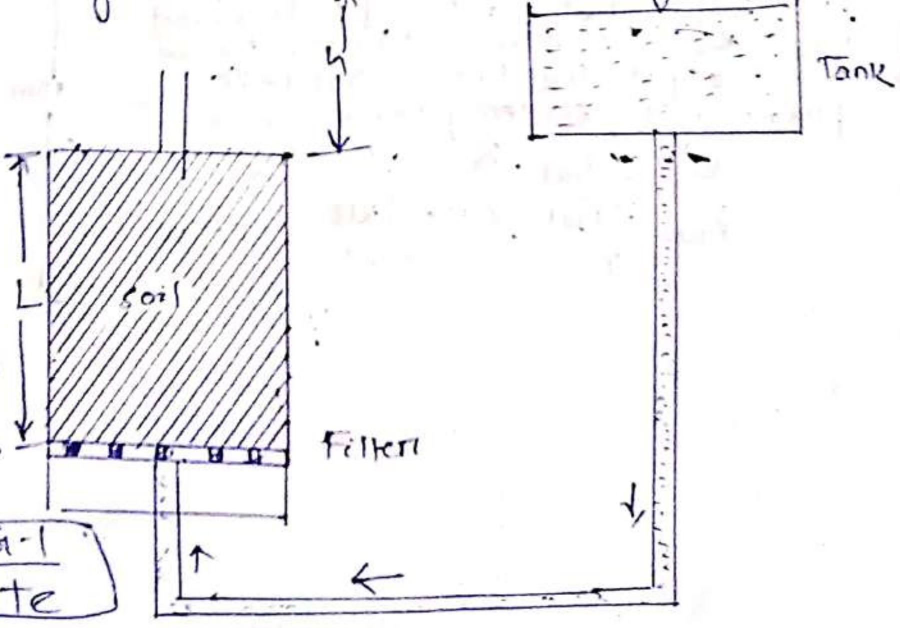
$$\Rightarrow i = \frac{\gamma'}{\gamma_w}$$

But we have

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

Thus

$$\boxed{i = i_c = \frac{G-1}{1+e}}$$



Q.1 A coarse-grained soil has a voids ratio of 0.78 and specific gravity as 2.67. Calculate the critical gradient at which quicksand condition will occur.

Ans

$$i_c = \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e} = \frac{2.67-1}{1+0.78} = 0.94$$

The Hydraulic gradient of this condition is called as critical hydraulic gradient. Thus, quick sand condition is the particular condition of flow which takes place when effective pressure reduces to zero at the time of upward flow.

Q.2 What is flowing at the rate of 0.05 m/sec in an upward direction through a fine sand sample whose co-efficient of permeability is $2\times10^{-3}$ cm/sec. The sample thickness is 12 cm and cross-sectional area is 50 cm². Find the effective pressure at the middle and bottom sections of the sample, if the saturated unit weight of sand is 19.4 kN/m³.

Soln

$q = 0.05$ cm³/sec, $K = 2\times10^{-3}$ cm/sec, $Z = 12$ cm, $A = 50$ cm²

$\gamma' = 19.4 - 9.81 = 9.59$ kN/m³

Now $i = \frac{q}{KA} = \frac{0.05}{2\times10^{-3}\times50} = 0.5$.

For upward flow of water, the effective pressure is given by $\sigma' = z\gamma' - iz\gamma_w$

For the bottom section of the sample, $z = 12$ cm $= 0.12$ m measured from the top

$\therefore\ \sigma' = (0.12\times9.59) - (0.5\times0.06\times9.81) = 0.281$ kN/m².

Q.3 A large open excavated was made in a stratum of stiff clay with a saturated unit weight of 18.6 kN/m³. When the depth of excavation reached 7 m, the bottom rose gradually, cracked and was flooded from below by a mixture of sand and underlain by a bed of sand with its surface at a depth would have risen above the stratum into a drill hole before the excavation was started.

Ans Fig 9.3). Let the water in the drill hole rise to a value hw metres above the sand strata before the excavation was started. At the point A, the effective pressure is reduced when the excavation of the top soil is started. When the soil is excavated by 7 m, the soil is lifted up. At that time, the effective pressure at A evidently becomes zero

$$\sigma' = Z\gamma_{sat} - h_w\gamma_w = 0$$

$$h_w = \frac{Z\gamma_{sat}}{\gamma_w} = \frac{(12-7)\times18.6}{9.81} = 9.48\text{ m}$$

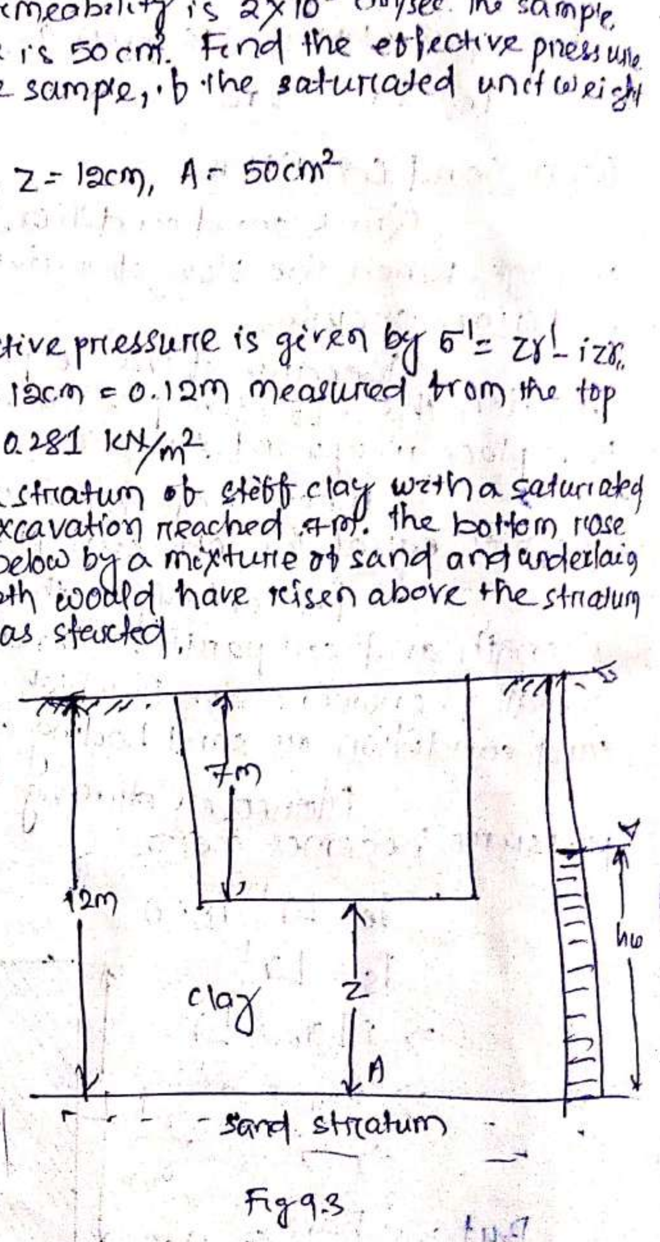



Fig 9.3

# Compaction and Consolidation

## Compaction:

Compaction is a process by which the soil particles are artificially rearranged and packed together into a closer state of contact by mechanical means in order to decrease the porosity (or voids ratio) of the soil and thus increase its dry density. The compaction process may be accomplished by rolling, tamping, or vibration. Compaction is somewhat different from consolidation. While consolidation is a gradual process of volume reduction under sustain loading, compaction refers to a more or less rapid reduction mainly in the air voids under a loading of short duration. An example of compaction is the reduction in voids produced in a layer of the sub-grade by a rubber-tyred or steel-tyred roller during construction.

| S.No. | Light compaction Test | Heavy compaction Test |
|---|---|---|
| 1. | Also known as standard proctor test | Also known as modified proctor test |
| 2. | Number of blows per layer is 25 | Number of blows per layer is 25 |
| 3. | Number of layers is 3 | Number of layers is 5 |
| 4. | Weight of rammer is 2.5kg | Weight of rammer is 4.5kg for greater impact |
| 5. | Rammer dropped from a height of 310mm | Rammer dropped from a height of 450mm. |
| 6. | Volume of mold is 1000 $cm^3$ | Volume of mold is 1000 $cm^3$ |

The mould and the amount of dry soil used in this test is same for both tests. Both the tests are laboratory tests and used to determine maximum dry density and optimum moisture content for a given compaction energy for a given soil sample. Specific gravity of the solid soil grain is 2.67.

## Optimum Moisture Content (OMC):

It is the water content at which the soil attains maximum dry density. Maximum Dry Density (MDD) is the dry density of soil corresponding to Optimum Moisture Content $w_o$.

**Zero air voids line:** A line which shows the water content dry density relation for the compacted soil containing a constant percentage air voids is known as an air voids line. and can be established from the following relation. (Eq. 2.35)

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

where, $n$ = % air voids

$\gamma_d$ = dry density corresponding to w.

$\gamma_w$ = density of water = 1 g/cm³

$w$ = water content of compacted soil

$G$ = specific gravity

The theoretical maximum compaction for any given water content corresponds to zero air voids condition ($n_a = 0$). The line showing the dry density as a function of water content for soil containing no air voids is called the zero air voids line or the saturation line, and is established by the equation:

$$\rho_d = \frac{G \rho_w}{1+wG} \quad \ldots (17.3)$$

Alternatively, a line showing the relation between water content and dry density for a constant degree of saturation S is established from eqn

$$\rho_d = \frac{G \rho_w}{1+\frac{wG}{S}} \quad (17.4)$$

For S = 100% Eq. 17.4 reduces to Eq. 17.3. The air voids line or degree of saturation lines when drawn across a compaction curve gives a direct indication of the percentage air voids or of the degree of saturation existing at different points on the curve.

Factors Affecting compaction:

The various factors which effect the compacted density are as follows: (i) water content, (ii) amount and type of compaction (iii) method of compaction, (iv) types of soil, and (v) addition of admixtures

1. Water Content: It has been seen ~~that~~ by laboratory experiments that as the water content is increased, the compacted density goes on increasing till a maximum dry density is acheived after which further addition of water decreases the density. When only a relatively small amount of water is present in soil. It is firmly held by the electrical forces at the surface of soil particles with a high concentration of electrolyte which prevents the diffuse double layer surrounding the particles ~~with a high concentration of electrolyte which prevents the diffuse double layer~~ from developing fully. The double layer depression leads to a low inter-particle repulsion and the particles do not move over one another easily when compactive energy is applied and hence high percentage air voids and low density is achieved. The increase in water content result in an expansion of double layer and a reduction in the net attractive force between particles or in an increased inter-particle repulsion which permits the particles to slide more easily past one another into a more oriented and denser state of packing together and hence higher density. After the optimum water content is reduced, the air voids approach approximately a constant value as further increase in water content does not cause any appreciable in them, even though a more orderly arrange of particles may exist at higher water content. The total voids due to water and air combination go on increasing with increase in water content beyond the optimum and hence the dry density of the soil falls.

3. Amount of compaction

The amount of compaction greatly affects the maximum dry density and optimum water content of a given soil. The effect of increasing the compactive energy results in an increase in the maximum dry density and decrease in the optimum water content as shown in Fig. 17.7. However the increase in maximum dry densities does not have a linear relationship with increase of compactive effort.

3. Method of compaction:

The density obtained during compaction, for a given soil greatly depends upon the type of compaction or the manner in which the compactive effort is applied. The various variables in this aspect are
(i) weight of the compacting equipment,
(ii) the manner of operation such as dynamic or impact, static, kneading, or rolling, and
(iii) time and area of contact between the compacting element and the soil.

4. Type of soil

The maximum dry density acheived corresponding to a given compactive energy largely depends upon the type of soil. Well graded coarse-grained soils attain a much higher density and lower optimum water contents than fine grained soils which require more water for lubrication because of the greater specific surface.

Fig 17.12 show dry density water content curves for a range of soil types. In general, coarse grained soils can be compacted to higher dry density than fine grained soils.

Ex 17.1 A laboratory compaction test on soil having specific gravity equal to 2.68 gave a maximum dry densities of 1.82 g/cm³ and a water content of 17%. Determine the degree of saturation, air content and % air voids at the maximum dry density. What would be theoretical maximum dry density corresponding to zero air voids at the optimum water content?

Ans

$$\rho_d = \frac{G\rho_w}{1+\frac{wG}{S}} \Rightarrow 1+\frac{wG}{S} = \frac{G\rho_w}{\rho_d}$$

$$\therefore\ 1+\frac{0.17\times 2.68}{S} = \frac{G\rho_w}{\rho_d} = \frac{2.68\times 1}{1.82} = 1.485$$

$$\Rightarrow \frac{0.17\times 2.68}{S} = 1.485-1 = 0.485$$

$$\Rightarrow S = \frac{0.17\times 2.68}{0.485} = 0.94 = 94\%$$

$$a_c = 1-S = 1-0.94 = 0.06 = 6\%$$

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

$$\Rightarrow (1-n_a) = \frac{1.82(1+0.17\times 2.68)}{2.68} = 0.99$$

$$\Rightarrow n_a = 1-0.99 = 0.01 = 1\%.$$

When $n_a = 0$ $(S=1)$, theoretical dry density at $w = 17\%$ is given by

$$\rho_d = \frac{G\rho_w}{1+wG} = \frac{2.68\times 1}{1+0.17\times 2.68} = 1.84 \text{ g/cm}^3$$

The corresponding dry unit weight is

$$\gamma_d = 9.81\rho_d = 9.81\times 1.84 = 18.05 \text{ kN/m}^3$$

: The following are the results of a compaction test

| | | | | | |
|---|---|---|---|---|---|
| Mass of mould + wet soil (g) | 2925 | 3095 | 3150 | 3125 | 3070 |
| water content (%) | 10.0 | 12.0 | 14.3 | 16.1 | 18.2 |

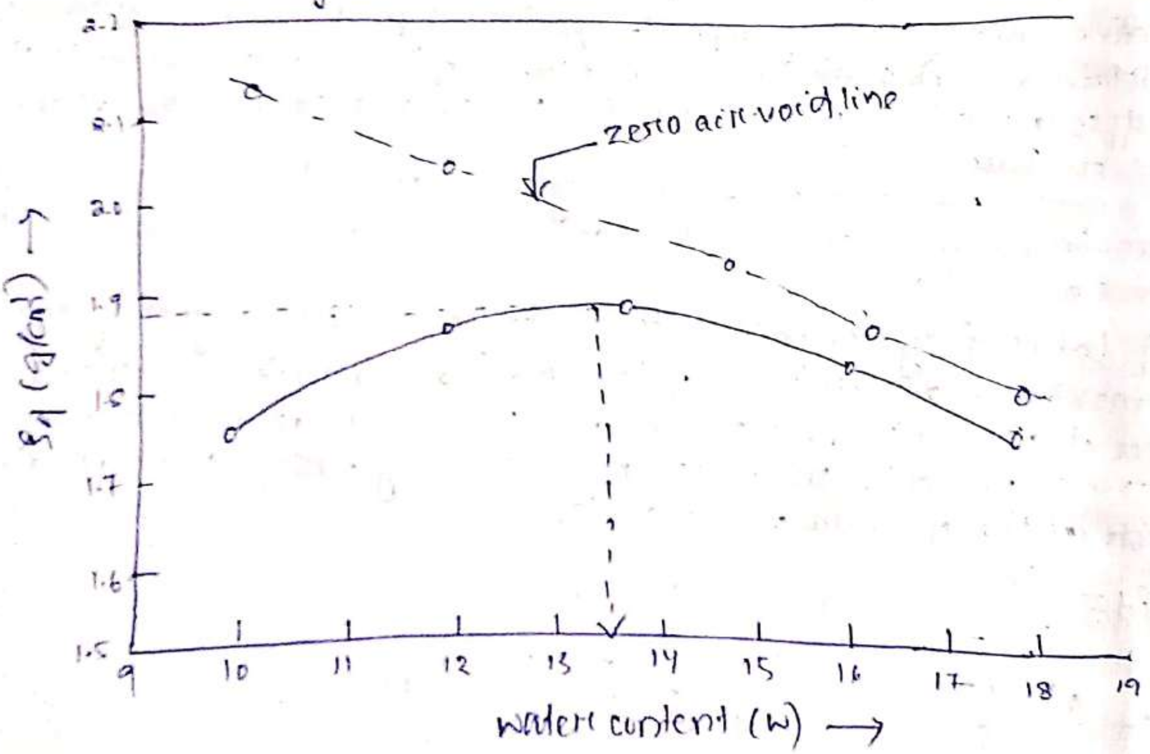
Volume of mould = 1000 ml. Mass of mould = 1000 g

specific gravity of solids = 2.70

(i) Find the compaction curve showing the optimum moisture content and maximum dry density

(ii) plot the zero air void line

(iii) Determine the degree of saturation at the maximum dry density.




The zero-air void line, corresponding to $S = 1$, is placed by the equation

$$(\rho_d)_s = \frac{G\rho_w}{1+wG} = \frac{2.7\times 1}{1+2.7w} = \frac{2.7}{1+2.7w} \text{ g/cm}^3$$

Also, $$\rho_d = \frac{\rho}{1+w} = \frac{M}{V}\times\frac{1}{1+w} = \frac{M}{1000(1+w)} \text{ g/cm}^3$$

The computations are arranged in a tabular form below

| S. No | Water content (w) | Mass of soil (M) g | $\rho_d = \frac{M}{1000(1+w)}$ g/cm³ | $(\rho_d)_0 = \frac{2.7}{1+2.7w}$ g/cm³ |
|---|---|---|---|---|
| 1. | 0.10 | 1925 | 1.75 | 2.13 |
| 2. | 0.12 | 2095 | 1.87 | 2.04 |
| 3. | 0.143 | 2150 | 1.88 | 1.95 |
| 4. | 0.161 | 2125 | 1.83 | 1.88 |
| 5. | 0.182 | 2070 | 1.75 | 1.81 |

Fig. 17.14 shows the compaction curve along with zero-air voids line. From the curve we get

$\rho_{d,max} = 1.89$ g/cm³ and $w = 0.135$.

The degree of saturation is given by

$$\rho_d = 1.89 = \frac{G\rho_w}{1+\frac{wG}{S}} = \frac{2.7\times1}{1+\frac{0.135\times2.7}{S}}$$

$$\Rightarrow \frac{0.135\times2.7}{S} = \frac{2.7}{1.89} - 1 = 0.428$$

From which $S = \frac{0.135\times2.7}{0.428} = 0.852 = 85.2\%$.

Q. 3 Work out theoretical maximum dry density for a soil sample having sp. gr. of 2.7 and OMC = 16%. Also ~~again~~ explain the difference in OMC value in case of Proctor test and modified proctor test for cohesive soils and granular soils.

Ans $\gamma_{d,max}$ occurs when S is maximum i.e. when S=1

$$\gamma_{d,max} = \frac{G\gamma_w}{1+\frac{wG}{S}} = \frac{G\gamma_w}{1+w.G}$$

Hence. $\rho_{d,max} = \frac{G\rho_w}{1+wG} = \frac{2.7\times1}{1+0.16\times2.7} = 1.885$ g/cm³

4. A cohesive soil yields a maximum dry densities of 1.8 g/cc at an OMC of 16% during a standard proctor test. If the values of G is 2.65. What is the degree of saturation? What is the maximum dry density it can further compacted to?

Ans Given $\rho_d = 1.8$ g/cm³, $w = 0.16$, $G = 2.65$

$$e = \frac{G\rho_w}{\rho_d} - 1 = \frac{2.65\times1}{1.8} - 1 = 0.4722$$

$$S = \frac{wG}{e} = \frac{0.16\times2.65}{0.4722} = 0.8979 = 89.79\%$$

Now $\rho_d = \frac{G\rho_w}{1+\frac{wG}{S}}$, when $S=1$, we get

$$\rho_{d,max} = \frac{G\rho_w}{1+wG} = \frac{2.65\times1}{1+0.16\times2.65} = 1.861 \text{ g/cm}^3$$

Field compaction method :

Various types of soils can be compacted in the field by three method rolling, ramming (by impact) and vibration. Corresponding to these, the various compacting equipments can be grouped under three categories : rollers, rammers and vibrators. The rolling equipments are of five types : (i) smooth wheel rollers, (ii) pneumatic tyred rollers, (iii) sheep foot rollers, (iv) lorries and pneumatic tyred construction plant, and (v) track laying vehicles. The ramming equipment consists of three types : (i) dropping weight type (including piling equipments) (ii) internal combustion type and (iii) pneumatic type. The vibrating equipment, mounted on screeds, plates or rollers are of two types. (i) dropping weight type, and (ii) pulsating hydraulic type.

The smooth wheel rollers are of three types : (i) the conventional three-wheel type with two large smooth faced wheels in the rear and one smaller smooth-faced drum in the front weighing from 20 to 150 KN. (ii) tandem rollers weighing from 10 to 140 KN and (iii) the three axle tandem rollers weighing from 120 to 180 KN. Smooth wheel rollers are usually self-propelled and are equipped with a clutch-type reversing gear so that they can be operated back and forth without turning.

The pneumatic type rollers range in size from the smaller wobble wheel rollers to very heavy rollers. A common form of pneumatic roller consists of a box or platform mounted between two axles, the rear of which has one more wheel when the front at the wheel mounted on the front axle being arranged to track between those mounted on the rear axle. The tyre pressures in the small rollers are of the order of 250 KN/m² and in the heavier rollers, the pressure ranges from 400 to 1050 KN/m². Smaller rollers are having tyres loads of about 7.5 KN per tyre. The pneumatic tyred rollers are loaded with kentledge such that when the tyres are inflated to their desired pressure, the sum of the contact widths of the tyres approximately equals 80 percent of the width of the roller. The wobble-wheel roller has wheels mounted at slight angle with respect to the axle, so as to provide a kneading action. The rollers are normally towed ~~cash~~ by either a track laying or a pneumatic tyred tractor.

The sheep foot rollers consist of hollow cylindrical steel drum on which projecting feet are mounted. The weight of the drum can be varied by filling it partly or fully with water or sand and they are mounted either singly or in pairs on a steel frame which is towed by either track-laying or pneumatic-tyred tractor. The loaded weight per drum ranges from about 15 to 120 KN and the foot pressure ranges from 500 to 3500 KN/m². Rammers for compacting the soil comprise of prismatic, and internal combustion types weighing from 300 to 1500 N. Internal combustion type jumping rammers known as frog rammers, which weight upto one tonne. The vibrators consists of a vibrating unit of either the out of balance weight type or a pulsating hydraulic type mounted on a screed, plate or roller.

Suitability of various compaction equipment: The performance of a compaction equipment depends upon the soil type, its particle size distribution and its water content. In general, smooth wheel rollers are most suited to crushed rock, hard core, mechanically stable gravel sands. They can also be used satisfactorily on moderately cohesive soils. In cohesionless sands and gravel vibrating type equipment, crawler tractors and rubber-tyred rollers are effective in producing densities upto about 100% of modified AASHO. Sheep-foot rollers are recommended for compacting cohesive soils, but are not considered effective on coarse grained cohesionless soils. The kneading action of the sheep-foot rollers results in a better bond between compacted layers compared to other types of rollers. The action of pneumatic-tyred rollers is a combination of pressure and kneading and they are suitable both on cohesionless sand and gravels and on cohesive soils. Rammers are used for compacting soils in confined places. Vibratory rollers are useful for cohesionless soils.

## Consolidation of soil

When a compressive load is applied to a soil mass, a decrease in its volume takes place. The decrease in the volume of soil mass under stress is known as compression and the property of soil mass pertaining to its susceptibility to decrease in volume under pressure is known as compressibility. A solid crystaline material, like steel, is deformed, under stress, by a relative distortion of the position of atoms in the molecular structure. However, soils are composed of small solid particles not bonded together, except by the small van der waals forces and adsorbed double layer water. When a stress is applied to it, the elastic deformation of solid particles is negligibly small compared to the deformation caused by change in relative position of the ~~discharge~~ discrete particles and the resulting decrease in the volume of voids. When the voids are filled with air alone, compression of soil occurs rapidly because air is compressible and can escape easily from the voids. In a saturated soil mass having its void filled with compressible water decrease in volume or compression can take place when water is expelled out of the voids. Such a compression resulting from a long-term ~~static~~ static load and the consequent escape of pore water is termed as consolidation.

According to Terzaghi: " every process involving a decrease in the water content of a saturated soil without replacement of the water by air is called a process of consolidation. The opposite process is called a process of swelling, which involves an increase in the water content due to an increase in the volume of voids."

Compression of soil also takes place by expulsion of air from the voids, under short duration moving or vibratory loads. Such a compression is usually known as compaction. The compression of partly saturated soils is accompanied by expulsion and compression of air and its partial dissolution in water. Depending upon degree of saturation water may also be expelled out along with air.

When a compressive load is applied to a laterally confined layer of sand, rapid vertical deformation occurs. The rate at which this deformation can take place depends upon the permeability of soil and upon

the distance the water must travel to reach a drainage surface.
The compressibility of clays may also be caused by three factors: (i) the expulsion of double layer water from between the grains (ii) slipping of the particles to new positions of greater density, and (iii) bending of particles as elastic sheets. The permeability of clay being very small, time is an important factor in the consolidation of clays.

## Consolidation process: spring Analogy:

The mechanics of consolidation was demonstrated by Terzaghi by means of the piston and spring analogy.

Fig 15.1. shows a spring with a piston on its top. Let the length of the spring be $z_0$ under a pressure of 10 units. If 12 units of pressure are added to its top, the spring will be compressed immediately to a length $z_1$. A further application of load will result in further decrease in the length of the spring. Within elastic limit, the load deflection curve may be assumed to be straight. If this spring and piston is placed in a cylinder containing water upto the bottom of the piston, and a valve at the bottom water will be free of stress since the whole load is carried by the spring alone. If the pressure on the piston is increased to 12 units, and the valve is closed, the spring cannot deform since water is incompressible. Hence the additional pressure of 2 units is entirely borne by water. If $\sigma$ denotes the total pressure, $\sigma'$ the pressure in spring and $\bar{u}$ as the pressure in water (i.e. pore water pressure), the governing equation of Fig 15.1a. is given by.

$$12 = 10 + 2 \quad \text{or} \quad \sigma = \sigma' + \bar{u} \quad \cdots (15.1)$$

Now, let the valve be opened slightly so that some water escapes & then valve is closed. Due to escape of some water, the piston moves down the spring is compressed and hence some pressure, out of pressure of 2 units entirely borne by water is now transfered to the spring. Thus, at any intermediate stage, the pressure equation becomes,

$$12 = (10 + \Delta\sigma') + (2 - \Delta\sigma') \quad \cdots \quad 15.2$$

where $\Delta\sigma'$ is the transfer of pressure from water to the spring corresponding to a given amount of expulsion of water. If the valve is fully opened, sufficient water will escape till the length of spring is reduced to a height of $Z_1$. Thus the whole of 2 units of pressure is transferred from water to the spring. The water becomes free of pressure and the spring carries the whole of pressure. The pressure equation at this stage becomes

$$12 = 12 + 0 \quad \text{or} \quad \sigma = \sigma' + u \quad \cdots \quad (5.3)$$

Thus, we see that when there is a pressure increment, the whole of pressure is first taken by water. As the water escapes out of the system, the load transfer takes place from water to the spring till the spring is determined by the full amount corresponding to the applied stress increment. This analogy can be applied to the consolidation process of a soil mass consisting of soil water system. The grain structure represents the spring while the voids filled with water represent the cylinder. The valve opening is represented by the permeability of the soil mass and the rate of load transfer from water to soil depends upon the permeability and the ~~lab~~ boundary condition (i.e. the drainage paces available). The pressure that builds up in pore water due to load increment on the soil is termed excess pore pressure or excess hydraulic pressure or hydrodynamic pressure $\bar{u}$, because it is in excess of the initial pressure in water under static condition. The excess hydrostatic pressure forces the water to drain out of the voids. As water starts escaping from the voids, the excess hydrostatic pressure in water gets gradually dissipated and the pressure increment is shifted as an increase in effective pressure on the soil solids and the soil mass decreases in volume. When the whole of the pressure increment or the consolidation pressure is carried as an increase in the effective pressure on the solids, no more water escapes from the voids and a condition of equilibrium is attained. Under different applied pressures, soil attains different equilibrium or final voids ratio, and under each equilibrium condition the whole of the applied pressure is carried by the solids as an effective pressure. The delay caused in consolidation by the series slow drainage of water out of a saturated soil mass is called hydrodynamic lag. The reduction in volume of soil which is due principally to a squeezing out of water from the voids is termed primary consolidation, primary compression or primary time effect. Even after the reduction of all excess hydrostatic pressure to zero, some compression of soil takes place at a very slow rate. This is known as secondary consolidation, secondary compression or secondary time effect. During the secondary compression, some of the highly viscous water between the points of contact is forced out from between the particles.

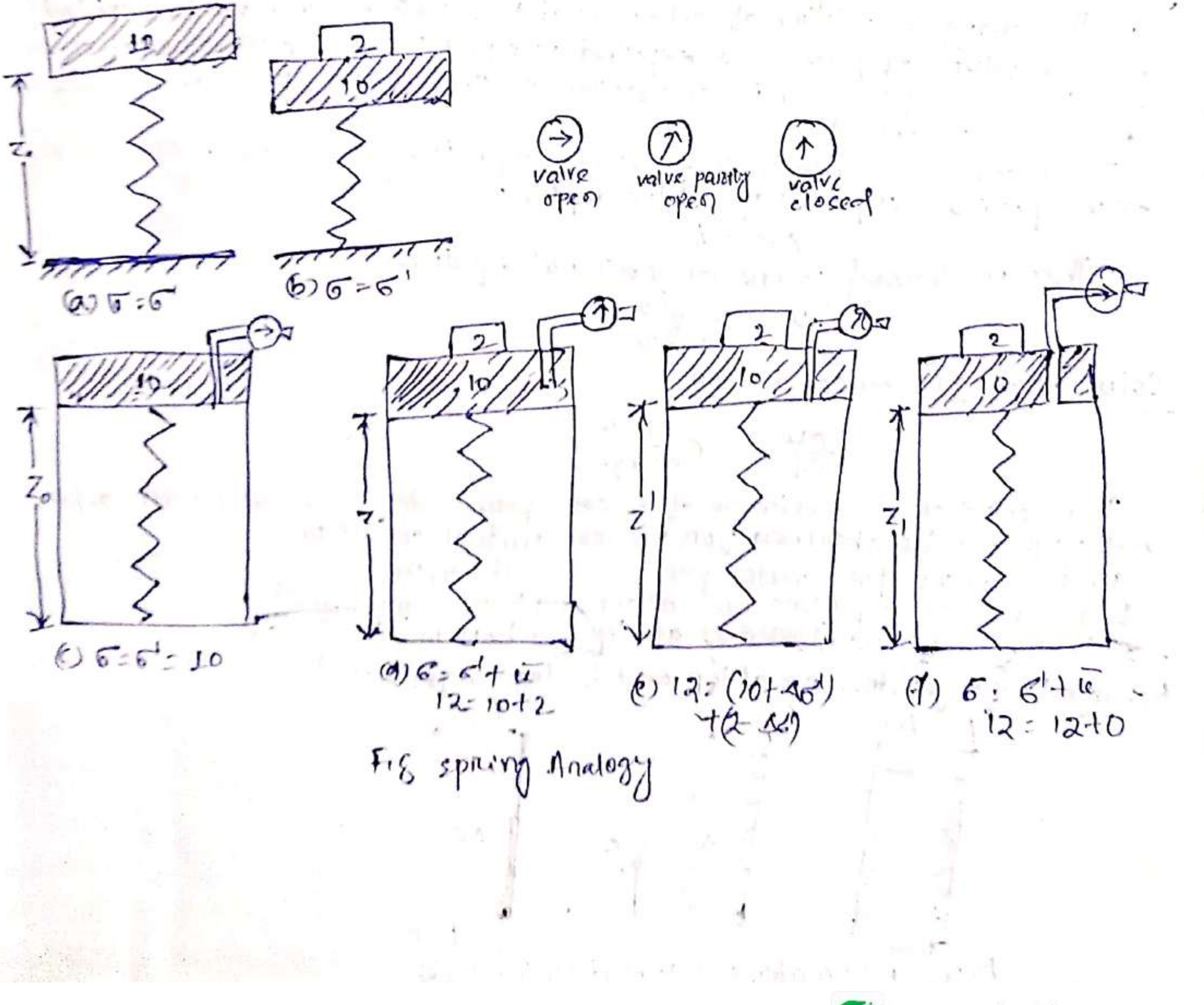



Fig spring Analogy

Terzaghi's one dimensional consolidation Equation

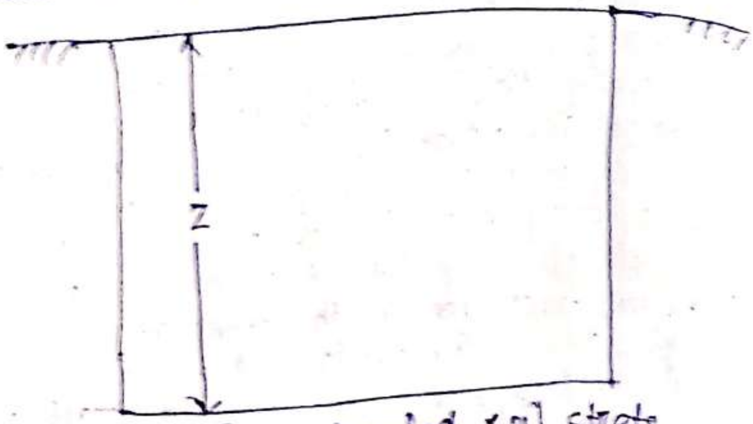



Fig. saturated soil strata

Assumption:

→ The soil medium is completely saturated
→ The soil medium is isotropic and homogeneous
→ Darcy's law is valid for flow of water
→ Flow is one dimensional in the vertical direction
→ The coefficient of permeability is constant
→ The coefficient of volume compressibility is constant
→ The increase in stress on the compressible soil deposit is constant
→ Soil particles and water are incompressible

One dimensional theory is based on the following hypothesis

→ The change in volume of soil is equal to volume of pore water expelled
→ The volume of pore water expelled is equal to change in volume of ...
→ Since compression in one direction the change in volume is equal to change in height.

The increase in vertical stress at any depth is equal to the decrease in excess pore water pressure at the depth.

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

This is Terzaghi's one dimensional equation.

$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2}$$

Solution of 1D consolidation

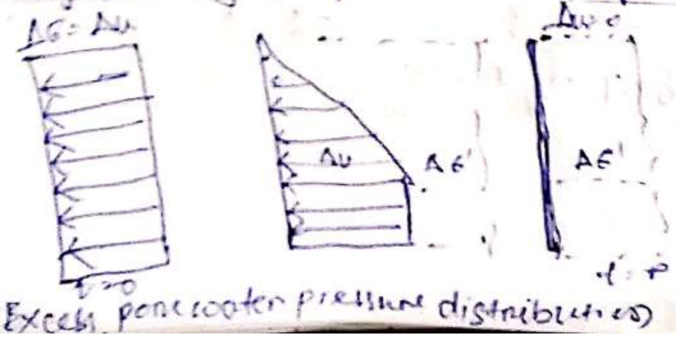
$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2} \quad (1)$$

The solution of variation of excess pore water pressure with depth and time can be obtained for various initial conditions.

Uniform excess pore water pressure with depth.

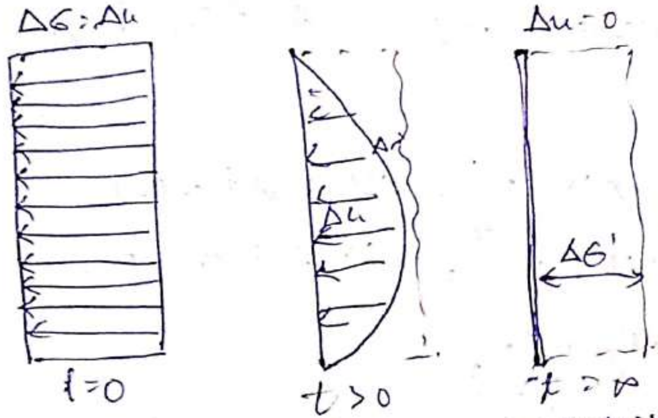
1. Single Drainage (Drainage at top and bottom impervious).
2. Double Drainage (Drainage at top and bottom).

1. Single Drainage (drainage at top and bottom impervious)




Excess pore water pressure distribution

## Double Drainage




Excess pore water pressure distribution

Excess porewater pressure distribution of double

Boundary conditions are

(i) At $t=0$ $\Delta u = \Delta\sigma$ and $\Delta\sigma' = 0$

(ii) At the top $Z=0$, $\Delta u = 0$ $\Delta\sigma = \Delta\sigma'$

(iii) At the bottom $Z = 2H_{dr}$ $\Delta u = 0$ $\Delta\sigma = \Delta\sigma'$

A solution of equation (1) for the above boundary conditions using Fourier series is given by

$$\Delta u(z,t) = \sum_{m=0}^{\infty} \frac{2\Delta u_0}{M} \sin\left(\frac{MZ}{H_{dr}}\right) e^{-M^2 T_v}$$

$M = \frac{\pi}{2}(2m+1)$ where $m$ = +ve integer with values from 0 to $\infty$.

$T_v = \frac{C_v t}{H_{dr}^2}$ where $T_v$ = Time factor (dimensionless)

5.16 A compressible layer is expected to have total settlement of 15 cm under a given loading. It settles by 3 cm at the end of two months after the application of load increment? How many months will be required to reach a settlement of 7.5 cm? What is the settlement in 18 months? The layer has double drainage.

Ans Given $\delta_f = 15$ cm

At $t_1 = 60$ days, $U_1 = \frac{\delta_1}{\delta_f} \times 100 = \frac{3}{15} \times 100 = 20\% = 0.2$

At $t_2 = ?$ $U_2 = \frac{\delta_2}{\delta_f} \times 100 = \frac{7.5}{15} \times 100 = 50\% = 0.5$

At $t_3 = 18 \times 30 = 540$ days, $\delta_3 = ?$

For $U < 60\%$, $T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$

$\therefore T_{v_1} = \frac{\pi}{4}\left(\frac{20}{100}\right)^2 = 0.03142$, $T_{v_2} = \frac{\pi}{4}\left(\frac{50}{100}\right)^2 = 0.1963$

Now $\frac{t_2}{t_1} = \frac{T_{v_2}}{T_{v_1}}$ or $t_2 = 60 \times \frac{0.1963}{0.03142} = 374.95$ days

Also $T_{v_3} = \frac{t_3}{t_1} \times T_{v_1} = \frac{540}{60} \times 0.03142 = 0.2827$

$\therefore U_3 = \left(\frac{4}{\pi} T_{v_3}\right)^{1/2} = \left(\frac{4}{\pi} \times 0.2827\right)^{1/2} = 0.6$

Hence the approximate expression, given by (1) is just valid

$\therefore \delta_3 = U_3 \delta_f = 0.6 \times 15 = 9$ cm

2 Two clay specimens A and B, of thickness 2 cm and 3 cm, has equilibrium void ratio 0.65 and 0.70 respectively under a pressure of 200 kN/m². If the equilibrium voids ratio of the two soils reduced to 0.48 and 0.60 respectively when the pressure was increased to 400 kN/m², find the ratio of coefficients of permeability of the two specimens. The time required by the specimen A to reach 40% of consolidation is one fourth of that required by specimen B for reaching 40% degree of consolidation.

Ans This problem is practically the same as ex 3. Key 3

Now $\frac{K_A}{K_B} = \frac{(C_v)_A}{(C_v)_B} \times \frac{(m_v)_A}{(m_v)_B}$

For soil A, $e_0 = 0.65$, $e = .48$, $\Delta e = e - e_0 = 0.48 - 0.65 = -0.17$

For soil B, $e_0 = 0.70$, $e = 0.60$, $\Delta e = e - e_0 = 0.6 - 0.7 = -0.10$

For both soils, $\Delta \sigma' = 400 - 200 = 200$ kN/m²

$(m_v)_A = \frac{\Delta e}{1+e_0} \times \frac{1}{\Delta\sigma'} = \frac{0.17}{1+0.65} \times \frac{1}{200} = 5.1515 \times 10^{-4}$

$(m_v)_A = -\frac{\Delta e}{1+e_0} \times \frac{1}{\Delta\sigma'} = \frac{0.10}{1+0.70} \times \frac{1}{200} = 2.9412 \times 10^{-4}$

Also, $\frac{(C_v)_A}{(C_v)_B} = \frac{(d_A)^2}{(d_B)^2} \times \frac{t_B}{t_A} = \left(\frac{2/2}{3/2}\right)^2 \times \frac{4}{1} = 1.778$

$\frac{K_A}{K_B} = 1.7778 \times \frac{5.1515}{2.9412} = 3.114$

Scanned with OKEN Scanner

1. If a material has identical properties in all direction, it is said to be isotropic.
2. A body having similar properties throughout its volume, is said to be homogeneous.
3. The variation in volume of a liquid with the variation of pressure is called compressibility.

## Concept of shear strength

In engineering, shear strength is the strength of a material or component against the type of yield or structural failure when the material or component fails in shear. A shear load is a force that tends to produce a sliding failure on a material along a plane that is parallel to the direction of the force.

## Mohr coulomb failure theory:

Of the many theories of failure that have been proposed, only that formulated by Mohr (1900) has been useful in case of soils. The following are essential points of Mohr's strength theory.

1. Material fails essentially by shear. The critical shear stress causing failure depends upon the properties of the material as well as on normal stress on the failure plane.
2. The ultimate strength of the material is determined by the stresses on the potential failure plane (or plane of shear)
3. When the material is subjected to three dimensional principal stress (i.e. $\sigma_1, \sigma_2, \sigma_3$) the intermediate principal stress does not have any influence on the strength of material. In other words, the failure criterion is independent of the intermediate principal stress.

The theory was first expressed by Coulomb (1776) and later generalised by Mohr. The theory can be expressed algebraically by the equation $\tau_f = s = F(\sigma)$ . . . (18.5)

where $\tau_f = s$ = shear stress on failure plane, at failure = shear resistance of material

$F(\sigma)$ = function of normal stress

If the normal and shear stress corresponding to failure are plotted, then a curve is obtained. The plot or the curve is called the strength envelope. Coulomb defined the function $F(\sigma)$ as a linear function of $\sigma$ and gave the following strength equation:

$$s = c + \sigma \tan\phi \quad (2)$$

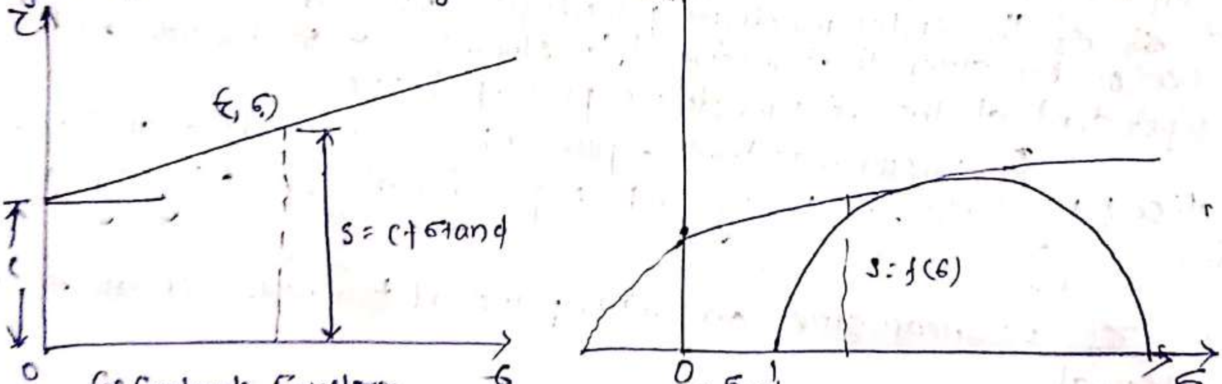
where the empirical constants c and $\phi$ represent respectively the intercepts on the shear axis, and the slope of the straight line of Eq 18.7 [Fig 18.3 (a)]. These parameters are usually termed as cohesion and angle of internal friction or shearing resistance respectively.

Fig. 18.3(b) shows the Mohr's envelope ~~respectively, the intercepts on the shear axis, and the slope~~ which is the graphical representation of Eq. 18.6. Coulomb considered that the relationship between shear strength and normal stress could be adequately representeby the straight line of Eq. 18.7 [Fig 18.3(a)]. These parameters are usually termed as cohesion and angle of internal friction or shearing resistance respectively.

Fig 18.3(b) shows the Mohr's envelope, which is the graphical representation of Eq. 18.6. Coulomb considered that the relationship between shear strength and normal stress could be adequately represented by the straight line. The generalised Mohr theory also recognises that the shear strength depends on the normal stress, but indicates that the relation is not linear. The strength theory upon which the Coulomb and Mohr strength linear. The strength theory upon which the Coulomb and Mohr strength lines are based indicates that definite relationship exists among the principal stresses, the angle of internal friction and the inclination of the failure plane. The curved failure envelope of Mohr is often referred to as a straight line for most of the calculations regarding the stability of soil mass. For an ideal pure friction material, such a straight line passes through the origin [Fig 18.4(a)]

However, dense sands exhibit a slightly curved straight line, indicated by dashed line. Fig 18.4(b) represents purely cohesive (plastic) material, for which the straight line is parallel to the $\sigma$-axis. The strength of such a material is independent of the normal stress acting on the plane of failure. The way in which a straight line is fitted to a Mohr envelope will depend on the range of $\sigma$ which is of interest.

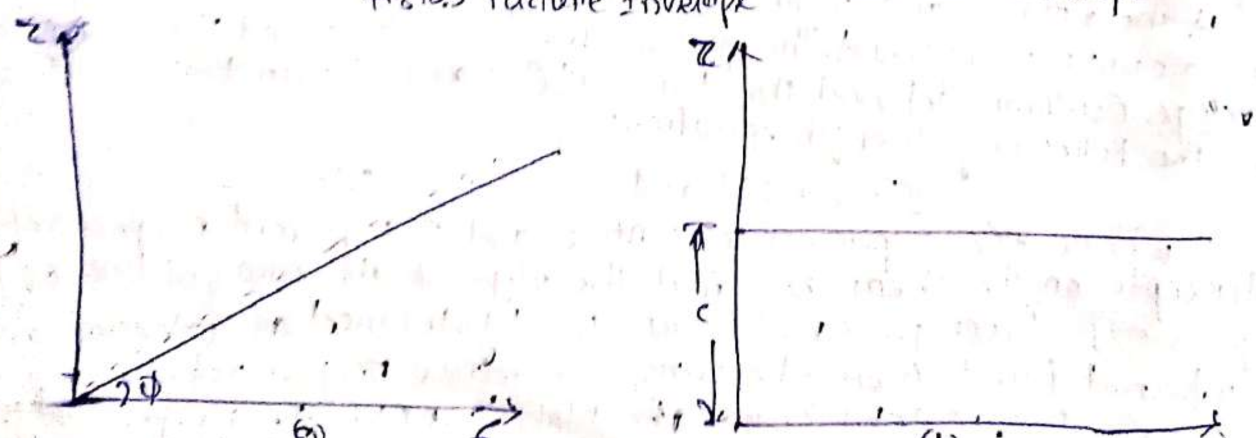
It can, therefore be concluded that the Mohr envelope can be considered to be straight if the angle of internal friction $\phi$ is assumed to be constant. Depending upon the properties of a material the failure envelope may be straight or curved and it may pass through the origin of stress or it may intersect the shear stress axis.




(a) Coulomb Envelope

(b) Mohr's Envelope

Fig 18.3 Failure Envelope




(a)

(b)

Fig 18.4 Straight Failure envelope

$$e = e_0 - C_c \log_{10}\frac{\sigma'}{\sigma_0'}$$

where, $e_0$ = initial void ratio corresponding to the initial pressure $\sigma_0'$

$e$ = void ratio at increased pressure $\sigma'$

$C_c$ = compression index (dimensionless)

$$C_c = \frac{e_0 - e}{\log_{10}\frac{\sigma'}{\sigma_0'}} = \frac{\Delta e}{\Delta \log_{10}\sigma'}$$

$$\Delta e = C_c \log_{10}\frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}$$

## coefficient of compressibility $a_v$.

The coefficient of compressibility is defined as the decrease in voids ratio per unit increase of pressure.

$$a_v = \frac{-\Delta e}{\Delta\sigma'} = \frac{e_0 - e}{\sigma' - \sigma_0'}$$

## coefficient of volume change $m_v$

The coefficient of volume changes or the coefficient of volume compressibility is defined as the change in volume of a soil per unit of initial volume due to a given unit increase in the pressure.

$$m_v = -\frac{\Delta e}{1+e_0}\frac{1}{\Delta\sigma'}$$

substituting, $-\frac{\Delta e}{\Delta\sigma'} = a_v$, we get $m_v = \frac{a_v}{1+e_0}$

When the soil is laterally confined, the change in the volume is proportional to change in the thickness $\Delta H$ and the initial volume is proportional to the initial thickness $H_0$

Hence $m_v = -\frac{\Delta H}{H_0}\frac{1}{\Delta\sigma'}$

Thus the change in the thickness, $\Delta H$ due to pressure increment is given by

$$\Delta H = -m_v H_0 \Delta\sigma'$$

## Final settlement by void ratio:

The final settlement $S_f$ can also be computed from the following relation.

$$\frac{\Delta H}{H} = \frac{e_0 - e}{1+e_0} \quad \text{or} \quad S_f = \Delta H = \frac{e_0 - e}{1+e_0} H$$

## Normally consolidated soils:

Compression index for normally consolidated soil is constant. Hence substituting the value of $e_0 - e$ in terms of $C_c$

$$S_f = H\frac{C_c}{1+e_0}\log_{10}\frac{\sigma'}{\sigma_0'}$$

$$\sigma' = \sigma_0' + \Delta\sigma'$$

1. A clay layer, whose total settlement under a given loading is expected to be 12 cm settles 3 cm at the end of 1 month after the application of load increment. How many months will be required to reach a settlement of 6cm? How much settlement will occur in 10 months? Assume, the layer to have double drainage.

Ans when $t_1 = 30$ days, $U_1 = \frac{s}{s_f} \times 100 = \frac{3 \times 100}{12} = 25\%$.

$t_2 = ?$ $\quad U_2 = \frac{6 \times 100}{12} = 50\%$.

The corresponding values of time factors can either be known from Table 15.1 or calculated from the approximate expression for $U < 60\%$

$$T_V = \frac{\pi}{4}\left(\frac{U}{100}\right)^2$$

when $U_1 = 25\%$ $\quad (T_V)_1 = \frac{\pi}{4}(0.25)^2 = 0.0492$

when $U_2 = 50\%$, $(T_V)_2 = \frac{\pi}{4}(0.5)^2 = 0.1965$

$$\frac{t_2}{t_1} = \frac{(T_v)_2}{(T_v)_1} \Rightarrow t_2 = \frac{0.1965}{0.0492} \times 30 = 120 \text{ days} = 4 \text{ months}$$

Also, when $t = 10$ months, $T_v = \frac{t}{t_1}(T_v)_1 = \frac{10}{1} \times 0.0492 = 0.492$.

When $U = 60\%$, $T_v = 0.287$ and hence the approximate expression between $T_v$ and $U$ is valid for a maximum value of $T_v = 0.287$

In the present case, $T_v = 0.492$ and hence $U$ cannot be found out from the approximate expression of Eq. 15.33. However, from approx. Eq 15.34 we have

$$T_v = 0.492 = 1.7813 - 0.9332 \log_{10}(100 - U\%)$$

$\Rightarrow \log_{10}(100 - U\%) = 1.3816$, from which $U \simeq 76\%$.

Also from the table 15.1 when $T_v = 0.492$.

we get $U \simeq 76\%$

$$s = U \cdot s_f = 0.76 \times 12 = 9.12 \text{ cm}$$

## Measurement of shear strength

The measurement of shear strength of soil involves certain test observations at failure with the help of which the failure envelope or strength can be plotted corresponding to a given set of conditions (specially the draining conditions). Shearing resistance can be determined in the laboratory by the following four methods.

(1) Direct shear test (2) Triaxial shear test
(3) Unconfined compression test (4) Vane shear test

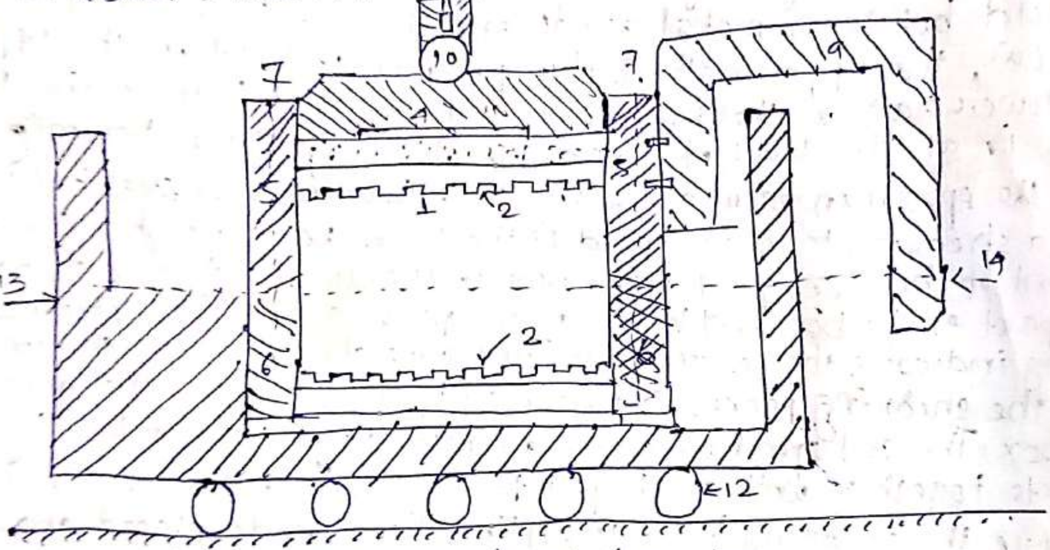
## Direct Shear Test :-

This is a simple and commonly used test and is performed in a shear box apparatus (Fig 18.6). The apparatus consists of a two piece shear box of square or circular cross section. The lower half of the box is rigidly held in position in a container which rest over slides or rollers and which can be pushed forward at a constant rate by geared jack, driven either by electric motor or by hand. The upper half of the box butts against a proving ring. The soil sample is compacted in the shear box and is held between metal grids and porous stones (or plates). As shown in Fig 18.6(a), the upper half of the specimen is held in the upper box and the lower half in the lower box and the joint between the two parts of the box is at the level of the centre of the specimen. Normally load is applied on the specimen from a loading yoke Bearing upon steel ball of pressure pad. When a shearing force is applied to the lower box through the geared jack the movement of the lower part of the box is transmitted through the specimen to the upper part of the box and hence on the proving ring. The deformation of proving ring indicates the shear force. The volume change during the consolidation and during the shearing process is measured by mounting a dial gauge at the top of the box. The soil specimen can be compacted in the shear box by clamping both the parts together with the help of two screws. These screws are, however, removed before the shearing force is applied. Metal grids, placed above the top and below the bottom of the specimen may be perforated if drained test is required, or plain if undrained test is required. The metal grids have linear slots or serrations to have proper grip with the soil specimen, and are so oriented that the serrations are perpendicular to the direction of the shearing force.

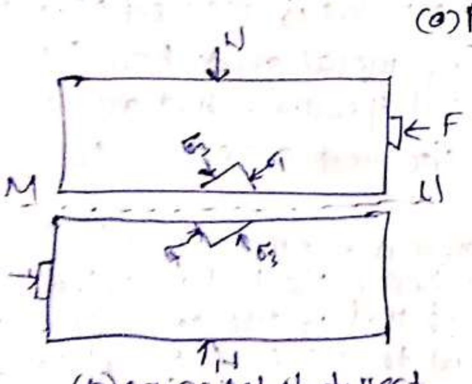
The specimen of the shear box is sheared under a normal load N. The shearing strain is made to increase at a constant rate and hence the test is called the strain controlled shear box test. The other type of test is the stress controlled shear box test, in which there is an arrangement to increase the shear stresses of a desired rate and measure the shearing strain. Fig 18.6 (a) shows the strain controlled shear box. The shear force F, at failure corresponding to the normal load N is the measured with the help of the proving ring. A number of identical specimen are tested under increasing normal loads and the required maximum shear force is recorded. A graph is plotted between the shear force F as the ordinate and the normal load, N as the abscissa. Such a plot gives the failure envelope plotted as a function of shear stress s and the normal stress $\sigma$. The scales of both s and $\sigma$ are kept equal so that the angle of shearing resistance can be measured stress in the material during failure, under a given normal stress.

In the direct shear test, the failure plane MN is predetermined, and is horizontal. Fig 18.6 (b) shows the stress conditions during failure. In order to find the direction of principal stress planes at failure, we first locate the position of the pole on the Mohr circle [Fig.18.6(c)] on the principle that the line joining any point on the circle to the pole P gives the direction of the plane on which the stresses are those given by the co-ordinates of that point. Hence, through point F a horizontal line (representing the direction of the failure plane) is drawn to intersect the circle at the point P which is the pole. Since points A and B represent respectively the major and minor principal stresses, PA and PB give the directions of major and minor principal planes.

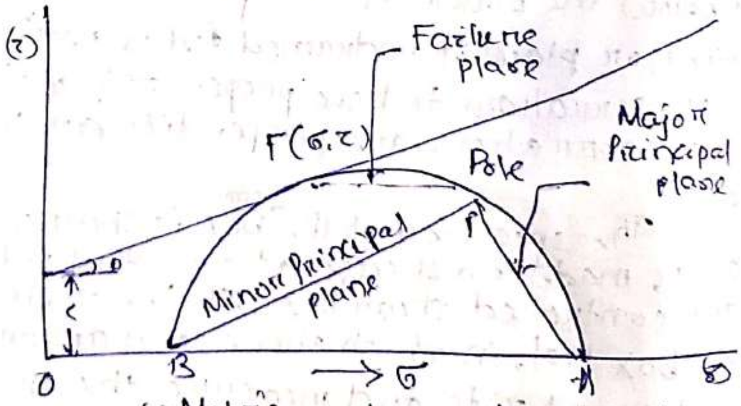
Tests can be performed under all the three conditions of drainage. To conduct undrained test, plain grids are used. For the drained test, perforated grids are used. The same is first consolidated under the normal load and then sheared sufficiently slowly so that complete dissipation of pore pressure takes place. The drained test is therefore also known as the slow test and the shearing of cohesive soil may sometimes require 2 to 5 days. Cohesionless soils are sheared in relatively less time. For the consolidated undrained test perforated grids are used. The sample is permitted to consolidate under the normal load. After the completion of consolidation, the specimen is sheared quickly in about 5 to 10 minutes.




(a) Parts of direct shear box




(b) principal of direct shear box




(c) Mohr's envelope and principal stresses during the test

1. Soil specimen
2. Metal Grids
3. Porous stones
4. Loading pads
5. Upper part
6. Lower part
7. Screws to fix two halves of shear box
8. Container for shear box
9. U-RAM
10. steel ball
11. Loading yoke
12. Rollers
13. Shear force applied by jack
14. Shear resistance measured proving ring

## Unconfined compression test

The unconfined compression test is a special case of triaxial compression test in which $\sigma_2 = \sigma_3 = 0$. The cell pressure in the triaxial cell is also called the confining pressure. Due to the absence of such a confining pressure, the uniaxial test is called the unconfined compression test. The cylindrical specimen of soil is subjected to major principal stress $\sigma_1$ till the specimen fails due to shearing along a critical plane of failure.

In its simplest form, the apparatus consists of a small load frame fitted with a proving ring to measure the vertical stress applied to the soil specimen. Fig 15.12(a) shows an unconfined compression tester (Goyal and singh 1955). The deformation of the sample is measured with the help of a separate dial gauge. The ends of the cylindrical specimen are hollowed in the form of cone. The cone seatings reduce the tendency of the specimen to become barrel shaped by reducing end-restraints. During the test, load versus deformation readings are taken and a graph is plotted. When a brittle failure occurs, the proving ring dial indicates a definite maximum load which drops rapidly with the further increase of strain. In the plastic failure, no definite maximum load is indicated. In such a case, the load corresponding to 20% strain is arbitrarily taken as the failure load.

Fig 15.12(b),(c) shows the stress conditions, at failure, in an unconfined compression test which is essentially an undrained test (It is assumed that no moisture is lost from the specimen during the test). Since $\sigma_3 = 0$, the Mohr circle passes through the origin which is also the pole.

From Eq 15.18, we get $\sigma_1 = 2C_u \tan\alpha$

$$= 2C_u \tan\left(45^\circ + \frac{\phi_u}{2}\right) \quad \ldots (15.22)$$

In the above equation, there are two unknowns $C_u$ and $\phi_u$, which can not be determined by the unconfined test since a number of tests on the identical specimens give the same value of $\sigma_1$. Therefore, the unconfined compression test is generally applicable to saturated clays for which the apparent angle of shearing resistance $\phi_u$ is zero. Hence,

$$\sigma_1 = 2C_u \quad (15.23)$$

When the Mohr circle is drawn, its radius is equal to $\sigma_1/2 = C_u$. The failure envelope is horizontal. PF is the failure plane and the stresses on the failure plane are

$$\sigma = \frac{\sigma_1}{2} = \frac{q_u}{2} \quad \ldots (15.24) \quad \text{and} \quad \tau_f = \frac{\sigma_1}{2} = \frac{q_u}{2} = C_u \quad \ldots (15.25)$$

where $q_u$ = unconfined compressive strength at failure. The compressive strength is calculated on the basis of changed cross-sectional area $A_2$ at failure which is given by

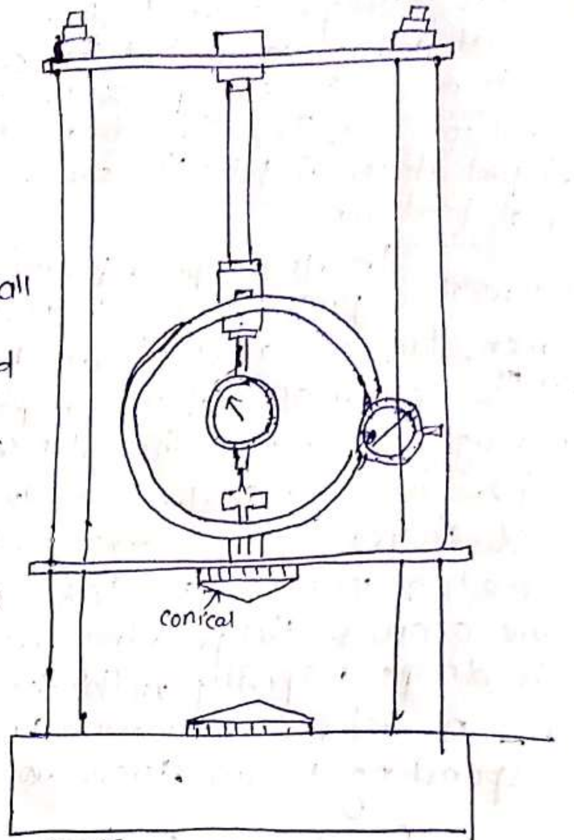
$$A_2 = \frac{V}{L_1 - \Delta L} = \frac{A_1}{1 - \frac{\Delta L}{L_1}}$$

where, $V$ = initial volume of the specimen

$L_1$ = initial length of the specimen

$\Delta L$ = change in length at failure.

Q1 A cylinder specimen of saturated clay. 4cm in diameter and a in overall length is tested in an unconfined compression tester. The specimen has · ends and its length between the apices of cones is 8cm. Find the unconfined compressive length of clay if the specimen fails under an axial load of 46.5 N. The change in length of specimen at failure is 1cm.




(a) The unconfined compression test

Sol: Original length of specimen = 9cm overall and 8cm to apieces of cones.

Length of cylinder of the same volume and diameter (average length), $L = 8.66cm$.

Initial cross-sectional area, $A_1 = \frac{\pi}{4} \times 4^2$

$= 12.57 \ cm^2$

change in length at failure, $\Delta L = 1cm$

Area of failure, $A_2 = \frac{A_1}{1 - \frac{\Delta L}{L_1}} = \frac{12.57}{1 - \frac{1}{8.66}}$

$= 14.2 \ cm^2$

∴ Unconfined compressive strength

$q_u = \frac{\text{failure load}}{A_2} = \frac{466}{14.2}$

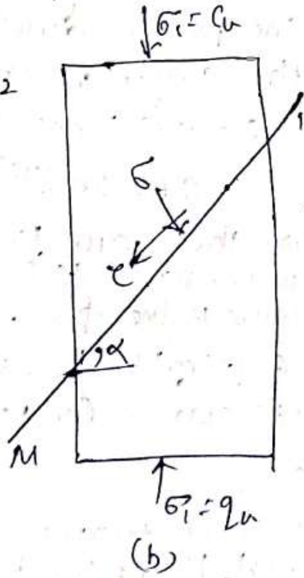
$= 32.8 \ kN/cm^2$

$= 328 \ kN/m^2 = 328 \ kPa$

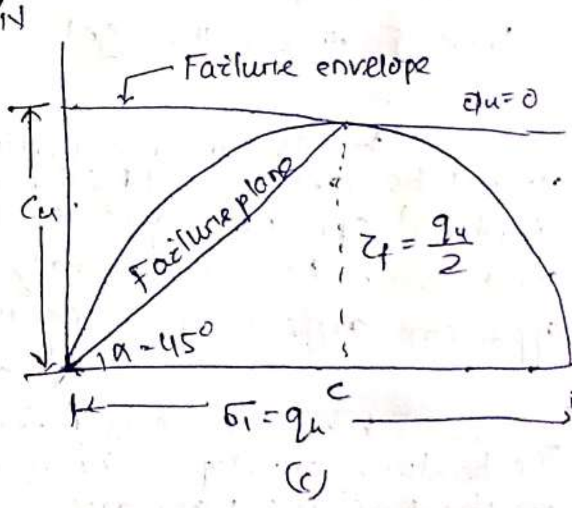
shear strength $C_u = \frac{q_u}{2} = \frac{328}{2}$

$= 165 \ kN/m^2$

$= 165 \ kPa$




(b)




(c)

Scanned with OKEN Scanner

## Vane shear test

Vane shear test is a quick test, used either in the laboratory or in the field, to determine the undrained shear strength of cohesive soil. The vane shear test consists of four thin steel plates, called vanes, welded orthogonally to a steel rod. A torque measuring arrangement, such as a calibrated torsion spring is attached to the rod which is rotated by a worm gear and worm wheel arrangement. After pushing the vanes gently into the soil, the torque rod is rotated at a uniform speed (usually at 1° per minute). The rotation of the vane shears the soil along a cylindrical surfaces. The rotation of the spring in degrees is indicated by a pointer moving on a graduated dial attached to the worm wheel ~~shaft~~ the shaft. The torque T is then calculated by multiplying the dial reading with the spring constant. A typical laboratory vane is 20mm high and 12mm in diameter with blade thickness from 0.5 and 1mm, the blades being made of high tensile steel. The field shear vane is from 10 to 20 cm in height and from 5 to 10cm in diameter, with blade thickness of about 2.5mm.

Let $\tau_f$ = unit strength of the soil

$H$ = height of the vane

$d$ = diameter of the vane

Let us assume that the top end of the vane is embedded in the soil, so that both top and bottom ends partake in the shearing of the soil. Assuming that the shear resistance of the soil is developed uniformly on the cylindrical surface, the maximum total shear resistance, at failure, developed along the cylindrical surface $= \pi d H \tau_f$ - - - (i)

To find the maximum shear resistance developed at top and bottom ends, consider a radius r of the sheared surface. The shear strength of a ring of thickness dr will be $2\pi r\, dr\, \tau_f$. Hence the total resistance of both top and bottom faces will be

$$= 2\int_0^r (2\pi r\, dr)\tau_f \quad - - - \text{(ii)}$$

The total shear strength developed will be equal to the sum of (i) and (ii). The maximum moment of the total shear resistance about the axis of torque rod equals the torque T at failure. Hence

$$T = (\pi d H \tau_f)\frac{d}{2} + 2\int_0^{d/2} (2\pi r\, dr\, \tau_f) r$$

$$= \pi \tau_f \left[\frac{d^2 H}{2} + \frac{d^3}{6}\right]$$

$$= \pi d^2 \tau_f \left[\frac{H}{2} + \frac{d}{6}\right]$$

If only the bottom end partakes in the shearing the above equation takes the form:

$$T = \pi d^2 \tau_f \left[\frac{H}{2} + \frac{d}{12}\right] \quad . \; . \; . \; \text{(iv)}$$

Knowing T, H and d, the shear strength $\tau_f$ can be determined.

Ex A vane 10 cm long and 8 cm in diameter was pressed into soft clay at the bottom of a bore hole. Torque was applied and gradually increa. to 45 N.m when failure took place. Subsequently, the vane rotated rapidly so as to completely remould the soil. The remoulded soil was sheared at a torque of 18 N.m. Calculate the cohesion of the clay in natural and remoulded states and also the value of the sensitivity.

Ans

**Natural state.**

$T = 4500\ N\cdot cm$

$H = 10\ cm$

$d = 8\ cm$

From Eq 15.27 $T = \pi d^2 \tau_f \left[\frac{H}{2} + \frac{d}{6}\right]$

$\therefore 4500 = \pi (8)^2 \tau_f \left[\frac{10}{2} + \frac{8}{6}\right]$

$\therefore \tau_f = \dfrac{4500}{64\pi (6.33)} = 3.54\ N/cm^2$

$\therefore c = \tau_f = 35.4\ kN/m^2 \quad (\because \phi = 0)$

**Remoulded state.** $T = 1800\ N\cdot cm$

$1800 = \pi (8)^2 \tau_f \left[\frac{10}{2} + \frac{8}{6}\right]$

$\therefore \tau_f = \dfrac{1800}{64 (6.33)} = 1.41\ N/cm^2 = 14.1\ kN/m^2$

$c = \tau_f = 14.1\ kN/m^2\ (kPa)$

$\therefore$ Sensitivity $= \dfrac{35.4}{14.1} = 2.5$.

# Earth pressure on Retaining structures

## Lateral earth pressure

It is the pressure that soil exerts in the horizontal direction. The lateral earth pressure is important because it affects the consolidation behaviour and strength of the soil and because it is considered in the design of geotechnical engineering structures such as retaining walls, basements, tunnels, deep foundations and braced excavation.

The earth pressure problem dates from the begining of the 18th Century, when Gautier listed five areas requiring research, one of which was the dimensions of gravity-retaining walls needed to hold back soil. However, the first major contribution to the field of earth pressures was made several decades later by Coulomb, who considered a rigid mass of soil sliding upon a shear surface. Rankine extended earth pressure theory by deriving a solution for a complete soil mass in a state of failure, as compared with Coulomb's solution which had considered a soil mass bounded by a single failure surface. Originally, the Rankine's theory considered the case of only cohesionless soils. However, this theory has subsequently been extended by Bell to cover the case of soils possessing both cohesion and friction. Caquot and Kerisel modified Muller Breslau equations to account for a nonplanar rupture surface.

## The coefficient of lateral earth pressure

The coefficient of lateral earth pressure K is defined as the ratio of the horizontal effective stress $\sigma_h'$, to the vertical effective stress $\sigma_v'$. The effective stress is the intergranular stress calculated by subtracting the pore pressure from the total stress as described in soil mechanics. K for a particular soil deposits is a function of the soil properties and the stress history. The minimum stable value of K is called the active earth pressure coefficient $K_a$, the active earth pressure is obtained, for example, when a retaining wall moves away from the soil. The maximum stable develop, for example against a vertical flow that is pushing soil horizontally. For a level ground deposit with zero lateral ~~earth~~ strain in the soil, the at-rest coefficient of lateral earth pressure $K_0$ is obtained.

There are many theories for predicting lateral earth pressure some are empirically based and some are analytically derived.

OCR - overconsolidation Ratio

$\beta$ = Angle of the backslope measured to the horizontal

$\delta$ = wall friction angle

$\theta$ = Angle of the wall measured to the vertical

$\varphi$ = soil stress friction angle

$\varphi'$ = Effective soil stress friction angle

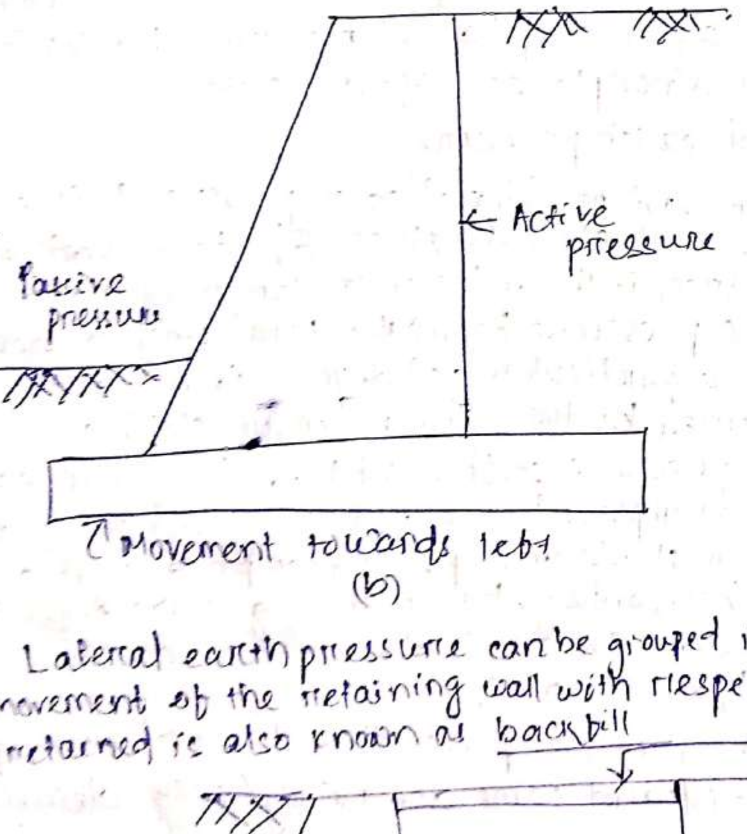
$\varphi'_{cs}$ = Effective stress friction at critical state

## Active Pressure

A state of active earth pressure occurs when the soil mass yields in such a way that it tends to stretch horizontally. It is a state of plastic equilibrium as the entire soil mass is on the verge of failure. A retaining wall when moves away from the backfill, there is a stretching of soil mass and the active state of earth pressure exists. In fig 19.1(b) the active ~~state of earth~~ pressure develops on the right hand side when the wall moves towards left.

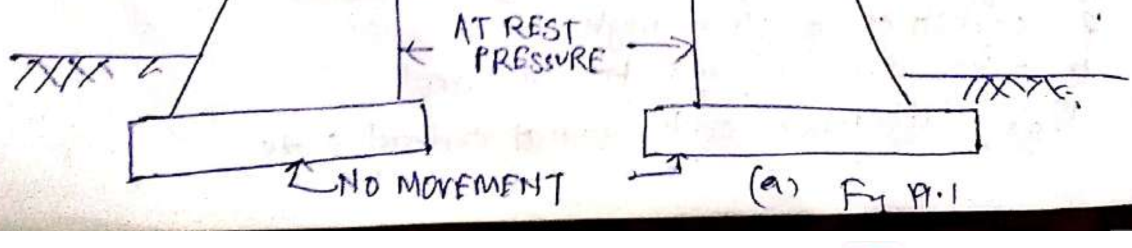
## Passive Pressure

A state of passive pressure exists when the movement of the wall is such that the soil tends to ~~hea~~ compress horizontally. It is another extreme of the limiting equilibrium condition. In fig 19.1(b) the passive pressure develops on the left side of the wall below the ground level, as the soil in this zone is compressed when the movement of the wall is towards left. Another example of the passive earth pressure is the pressure acting on ~~an~~ another block.




(b)

Lateral earth pressure can be grouped into 3 categories, depending upon the movement of the retaining wall with respect to the soil retained. The soil retained is also known as backfill




(a) Fig 19.1

## Rankine's Earth Pressure Theory

Rankine (1857) considered the equilibrium of a soil element within a soil mass bounded by a plane surface. The following assumptions were made by Rankine for the derivation of earth pressure.

→ The soil mass is homogeneous and some infinite.
→ The soil is dry and cohesionless
→ The ground surface is plane, which may be horizontal or inclined
→ The back of the retaining wall is smooth and vertical
→ The soil element is in a state of plastic equilibrium i.e. at the verge of failure.

**Active**

$$p_a = \left(\frac{1-\sin\phi'}{1+\sin\phi'}\right)\sigma_v$$

$$p_a = K_a \gamma Z$$

$$K_a = \frac{1-\sin\phi'}{1+\sin\phi} = \tan^2\left(45° - \frac{\phi}{2}\right)$$

**Passive**

$$p_p = \left(\frac{1+\sin\phi'}{1-\sin\phi'}\right)\sigma_v$$

$$p_p = K_p \gamma Z$$

$$K_p = \frac{1+\sin\phi'}{1-\sin\phi'} = \tan^2\left(45° + \frac{\phi'}{2}\right) \quad K_p = 3$$

**At-Rest Pressure:**

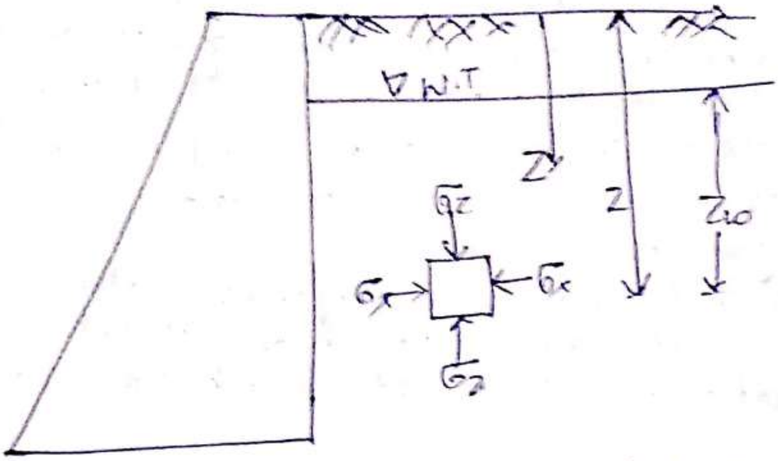
The lateral earth pressure is called at rest pressure when the soil mass is not subjected to any lateral yielding or movement. This case occurs when the retaining wall is firmly fixed at its top and is not allowed to rotate or move laterally. Fig 19.1(c) shows the basement retaining walls which are restrained against the movement by the basement slab provided at their tops. Another example of the at rest pressure is that of a bridge abutments wall which is restrained at its top by the bridge slab. The at rest condition is also known as the elastic equilibrium as no part of soil mass has failed and attached the plastic equilibrium.

**Earth pressure at rest**

The earth pressure at rest was discu.. However, the emphas. there was on the determination of the horizontal stresses on the soil mass. The expressions for earth pressure at rest would be used the determinati of the magnitude and line of action of the total forces due to earth pressure on the retaining structures. The methods for estimation of the coefficient of earth pressure at rest ($K_0$) have been discussed.

Fig 19.3 shows a retaining wall in which no movement take place. The vertical effective stress at point A at a depth Z is given by:

$$\bar{\sigma}_z = \gamma Z - \gamma_w Z_w \quad (19.)$$




The horizontal intergranular (effective) stress can be obtained using the coefficient of earth pressure at rest ($K_0$) which is equal to the ratio of the horizontal stress to the vertical stress.

Thus, $K_0 = \frac{\bar{\sigma}_x}{\bar{\sigma}_z}$

$$\Rightarrow \bar{\sigma}_x = K_0 \bar{\sigma}_z = K_0(\gamma Z - \gamma_w Z_w) \quad \cdots (19.2)$$

The stress $\bar{\sigma}_x$ is usually represented as $P_0$ indicating the lateral pressure at rest

Thus $P_0 = K_0 \bar{\sigma}_z \quad \cdots (19.3)$

It may be noted that the coefficient of lateral pressure at rest ($K_0$) relates the effective stress. The total lateral pressure ($P_h$) is equal to sum of the intergranular pressure ($P_0$) and the pore water pressure ($u$).

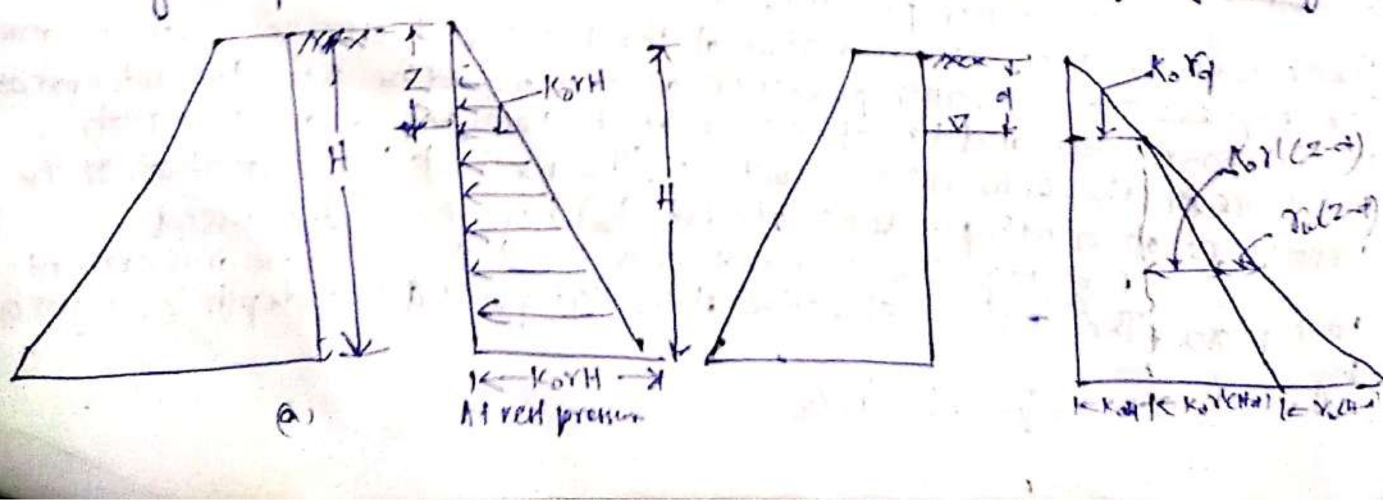
Thus, $P_h = P_0 + u \quad \cdots (19.4)$

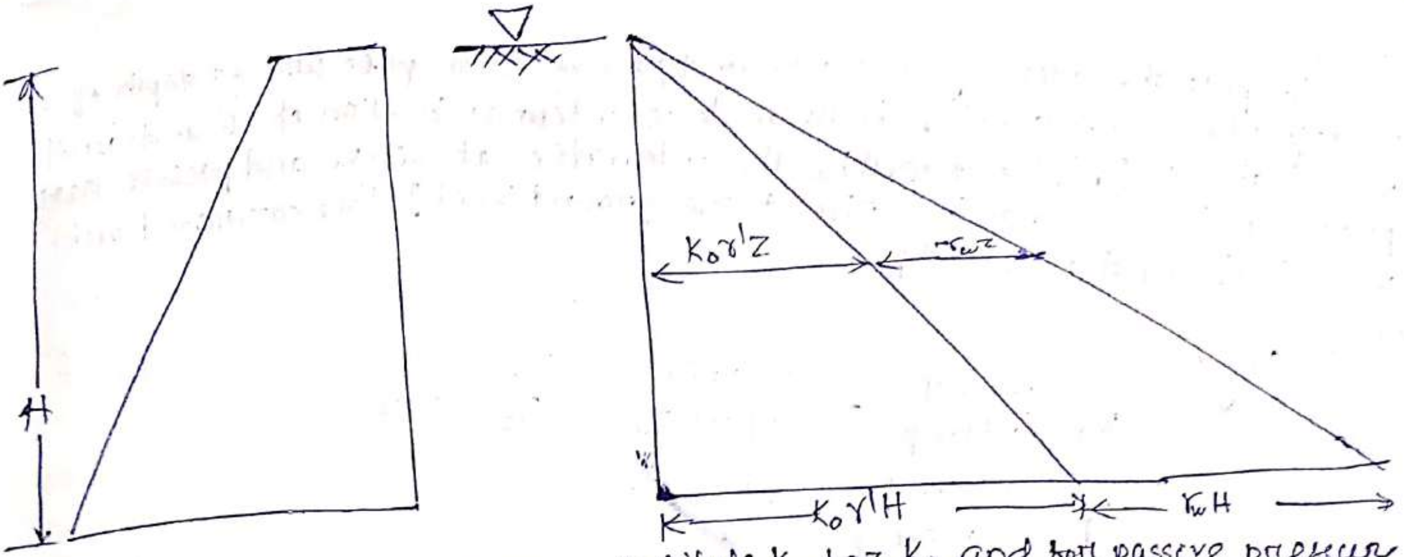
In fig. 19.3, the lateral pressure at depth z is, then

$$P_h = K_0(\gamma Z - \gamma_w Z_w) + \gamma_w Z_w \quad \cdots (19.5)$$

As Eq 19.5 indicates, the pressure distribution is triangular with zero pressure at the top (Z=0) and the maximum pressure at the bottom of the wall.

Fig 19.4(a) shows the pressure distribution when the soil is dry. The pressure at the bottom of the wall at depth H is given by






Note: For active pressure, substitute $K_a$ for $K_o$ and for passive pressure substituting $K_p$ and $K_o$

Fig 19.4

$$P_h = K_o \gamma H$$

The total pressure force per unit length of the wall given by

$$P = \int_0^H K_o \gamma Z \, dZ$$

$$\Rightarrow P = \frac{1}{2} K_o \gamma H^2 \quad \text{--- (19.6)}$$

In fig 19.4(b), the depth of water table is at depth of below the surface.
The pressure at depth $Z > d$ is given by.

$$P_h = K_o[\gamma Z - \gamma_w (Z-d)] + \gamma_w (Z-d)$$

or $P_h = K_o \gamma d + K_o \gamma' (Z-d) + \gamma_w (Z-d)$.

The pressure at the bottom ($Z=H$) of the wall is given by

$$P_h = K_o \gamma d + K_o \gamma' (H-d) + \gamma_w (H-d) \quad \text{--- (19.7)}$$

The total pressure force (P) can be determined from the pressure distribution diagram.

If the water table is at the bottom surface [Fig 19.4(c)], the pressure at the bottom of the wall is given by, taking $d=0$ in Eq 19.7.

$$P_h = K_o \gamma' H + \gamma_w H \quad \text{--- (19.8)}$$

The resultant pressure (P) acting on the wall is determined from the pressure distribution diagram.

The point of application of the resultant pressure P is determined from the pressure distribution diagram.

For triangular pressure distribution, it acts at height H/3 from the base.

The retaining wall or retaining structure is used to maintaining the ground surface at different elevations on either side of it. The material retained or supported by the surface is called backfill which may have its top surface horizontal or inclined. The position of the backfill lying above a horizontal plane at the elevation of the top of a wall is called the surcharge and its inclination is called surcharge angle B.

Ex.20.2

Compute the intensity of active and passive earth pressure at depth of 8m in dry cohesionless sand with an angle of internal friction of 30° and unit weight of 18 kN/m³. What will be the intensities of active and passive earth pressure if the water level rises to the ground level? Take saturated unit weight of sand as 22 kN/m³.

Ans

(a) Dry soil

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30°}{1+\sin 30°} = \frac{1/2}{3/2} = \frac{1}{3}$$

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1}{K_a} = 3$$

$$\therefore\ P_a = K_a \gamma H = \frac{1}{3} \times 18 \times 8 = 48\ kN/m^2$$

$$P_p = K_p \gamma H = 3 \times 18 \times 8 = 432\ kN/m^2$$

(b) submerged backfill

$$\gamma' = \gamma_{sat} - \gamma_w = 22 - 9.81 = 12.19\ kN/m^3$$

$$P_a = K_a \gamma' H + \gamma_w H = \frac{1}{3} \times 12.19 \times 8 + 9.81 \times 8$$
$$= 111\ kN/m^2$$

$$P_p = K_p \gamma' H + \gamma_w H = (3 \times 12.19 \times 8) + (9.81 \times 8)$$
$$= 371\ kN/m^2$$

Critical height of unsupported vertical cut

As shown in fig 19.21, the pressure is negative in the top region. It becomes zero at a depth $Z_e$. If the wall has a height of $2Z_e$ the total earth pressure is zero. This height is known as the critical height

$$H_e = 2Z_e$$

If the height of an unsupported vertical cut is smaller than He, it should be able to stand. However, the condition in unsupported vertical cut are different from those near a retaining wall. In the vertical cut, the lateral stress is everywhere zero, whereas in the retaining wall, it varies from $-2c'\sqrt{K_a}$ to $+2c'\sqrt{K_p}$. Because of this difference in the stress condition, the safe height of the vertical cut is slightly smaller than that given by Eq. 19.31

Substituting the value of $Z_e$ from Eq. 19.21

$$H_c = 2 \times 2c'/(\gamma\sqrt{K_a}) = 4c'/(\gamma\sqrt{K_a})$$

For $\phi = 0$, $H_c = 4c/\gamma$

20.1 A rigid retaining wall, 6m high is restrained from yielding. The backfill consists of cohesionless soil having $\phi = 26°$ and $\gamma = 19\ kN/m^3$, compute the total earth pressure per metre length of the wall?

Ans Since the wall is restrained from yielding, the wall will be subject to earth pressure ($P_0$), at rest, given by Eq. 20.13

$$P_0 = \frac{1}{2} K_0 \gamma H^2$$

Here, $K_0$ can be estimated from Eq. 20.10(a) by Jacky.

$$K_0 = 1 - \sin 26^\circ = 0.5616$$

(Note: This value corresponds to sand in nearly dense state)

$$P_0 = \frac{1}{2} \times 0.5616 \times 19 (6)^2 = 192.1 \text{ kN/m length of wall}$$

## Active Earth Pressure: Rankine Theory

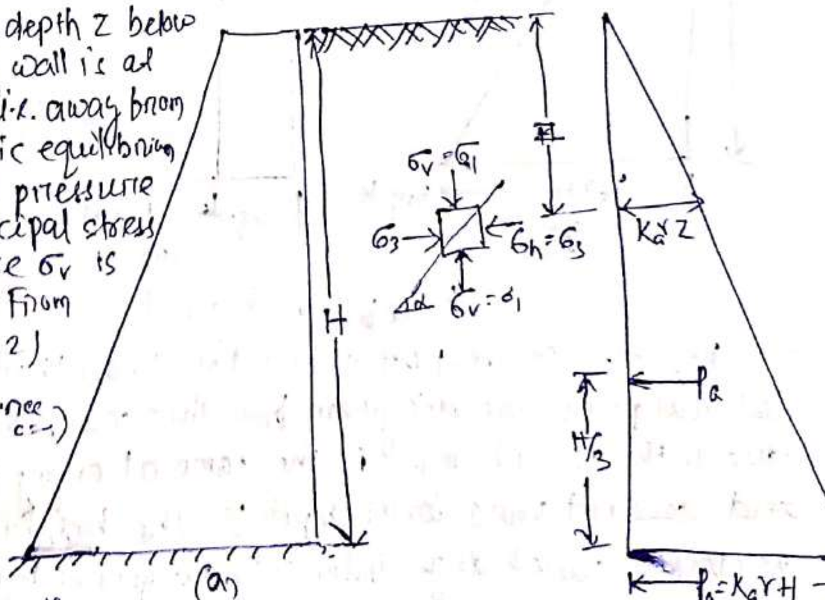
As originally proposed, Rankine's theory of lateral earth pressure is applied to uniform cohesionless soils only. Later, it was extended to include cohesive soils, by Resal (1910) and by Bell (1915). The theory has also been extended to stratified, partially immersed and submerged soils. Following are the assumptions of the Rankine theory.

1. The soil mass is semi-infinite, homogeneous, dry and cohesionless.
2. The ground surface is a plane which may be horizontal or inclined.
3. The back of the wall is vertical and smooth. In other words, there are no shearing stresses between the wall and the soil and the stress relationship for any element adjacent to the wall is the same as for any other element far away from the wall.
4. The wall yields about the base and thus satisfies the deformation condition for plastic equilibrium.

However, the retaining walls are constructed of masonry or concrete and hence the back of the wall is never smooth. Due to this, frictional forces develop. As a consequence of Rankine's assumption of no-existance of frictional forces at the wall face, the resultant pressure must be parallel to the surface of the backfill. The existence of the friction makes the resultant pressure inclined to the normal to the wall at an angle that approaches the friction angle between the soil and the wall.

### 1 Dry or moist backfill with no surcharge

Consider an element at a depth Z below the ground surface. When the wall is at the point of moving outwards (i.e. away from the fill), the active state of plastic equilibrium is established. The horizontal pressure $\sigma_h$ is then the minimum principal stress $\sigma_3$ and the vertical pressure $\sigma_v$ is the major principal stress $\sigma_1$. From the stress relationship (Eq. 20.2) we have




Active earth pressure of Dry or moist cohesionless soil.

$$\sigma_1 = \sigma_3 \tan^2(45^\circ + \phi/2) \quad (\text{c = none})$$

$$\therefore \frac{\sigma_3}{\sigma_1} = \frac{\sigma_h}{\sigma_v} = \frac{1}{\tan^2(45^\circ + \frac{\phi}{2})}$$

$$= \cot^2(45^\circ + \frac{\phi}{2})$$

Now $\sigma_h$ = lateral earth pressure = $p_a$

$\sigma_v$ = vertical pressure on the element $= \gamma . Z$

$$\therefore p_a = \gamma . Z \cot^2(45^\circ + \frac{\phi}{2})$$

$$= K_a \gamma Z \quad \text{.......} (1)$$

where $K_a$ = co-efficient of active earth pressure

$$= \cot^2(45° + \phi/2) = \frac{1 - \sin\phi}{1 + \sin\phi} \quad \ldots \quad (1.a)$$

When $\phi = 30°$, $K_a = \frac{1 - \sin 30°}{1 + \sin 30°} = \frac{1}{3}$

Fig 20.7 shows the distribution of active earth pressure p over the retaining wall. At $z = H$, the earth pressure is :

$$P_a = K_a \gamma H \quad \ldots \quad (2)$$

The total active earth pressure $P_a$ or the resultant pressure per unit length of the wall is found by integrating Eq. 20.7 or from the triangular pressure distribution diagram.

$$P_a = \frac{1}{2} K_a \gamma H^2 \quad \ldots \quad (3)$$

acting at H/3 above the base of the wall.

If the soil is dry, $\gamma$ is the dry weight of the soil, and if wet, $\gamma$ is the moist weight, to be submitted in Eq (2) and (3)

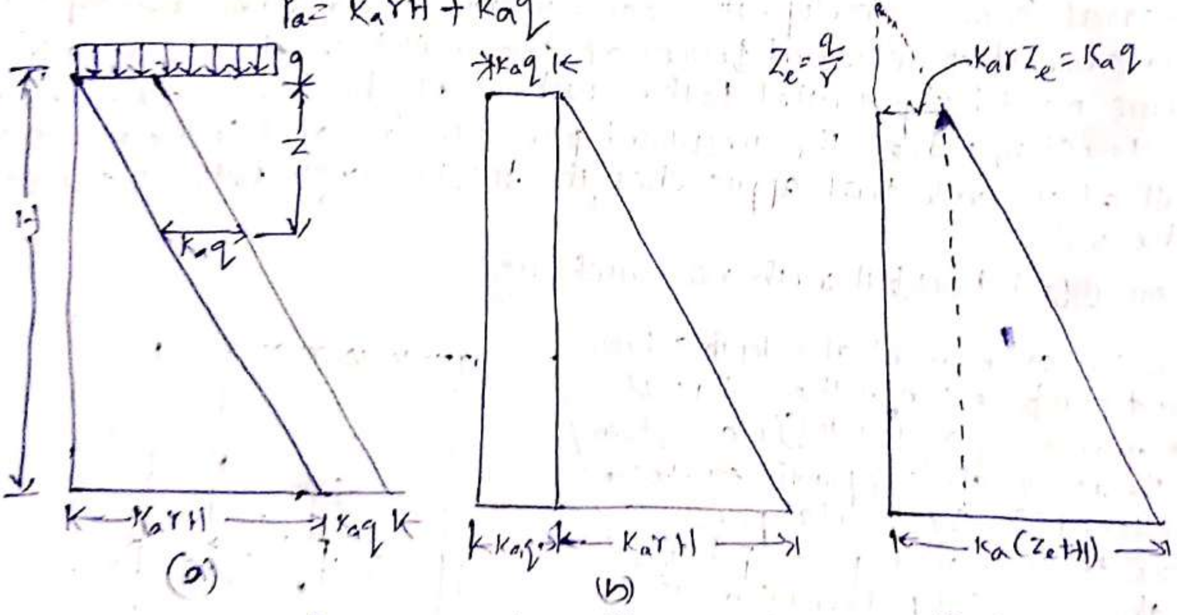
Backfill with uniform surcharge :

If the backfill is horizontal and carries a surcharge of uniform intensity q per unit area, the vertical pressure increment at any depth z, will increase by q. The increase in the lateral pressure due to this will be $K_a q$. Hence the lateral pressure at any depth z is given by

$$P_a = K_a \gamma z + K_a q \quad \ldots \quad (2.2)$$

At the base of the wall, the pressure intensity is

$$P_a = K_a \gamma H + K_a q$$




Backfill with uniform surcharge Fig 20.7

Fig 20.7 (a) and (b) shows the two alternative methods of plotting the lateral pressure diagram for this case. The lateral pressure increment due to the surcharge is the same at every point of the back of the wall, and does not vary with depth z. The height of fill $Z_e$ equivalent to the uniform surcharge intensity is given by the relation

$$K_a \gamma Z_e = K_a q \quad \text{or} \quad Z_e = \frac{q}{\gamma} \quad \ldots \quad (20.23)$$

This means that the effect of the surcharge of intensity q is the same as that of a fill of height $Z_e$ above the ground surface.

## Passive earth pressure

**(i) Cohesionless backfill :** In the case of passive state of plastic equilibrium the lateral pressure is the major principal stress while the vertical pressure is the minor principal stress. Thus

$$\sigma_h = p_p = \sigma_1 , \quad \sigma_v = \sigma_3 = \gamma z$$

Substituting this in the principal stress relationship

$$\sigma_1 = \sigma_3 \tan^2\alpha ,$$

we get

$$p_p = \gamma z \tan^2\alpha = k_p \gamma z \quad \dots (1)$$

where $p_p$ = passive earth pressure intensity

$K_p$ = Rankine's coefficient of passive earth pressure

$$K_p = \tan^2\alpha = N_\phi = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1}{K_a} \quad \dots (2)$$

Also, ratio, $$\frac{K_p}{K_a} = \frac{\tan^2(45°+\phi/2)}{\cot^2(45°+\phi/2)} = \tan^4(45°+\phi/2)$$

For example, if $\phi = 30°$

$$\frac{K_p}{K_a} = \tan^4\left(45°+\frac{30}{2}\right) = \tan^4 60° = 9$$

$$\Rightarrow K_p = 9 K_a$$

The distribution of passive earth pressure, given by Eq (1) is triangular with maximum value of $K_p \gamma H$ at the base of the retaining wall of height H. The total pressure $P_p$ for a depth H is given by

$$P_p = \int_0^H K_p \gamma H \, dz = \frac{1}{2} K_p \gamma H^2 \quad \dots (3)$$

If a uniform surcharge intensity q per unit area acts over the surface of the backfill, the increase in the passive pressure will be equal to $K_p \cdot q$. The passive pressure intensity at a depth z is then given by $p_p = K_p(\gamma z + q)$ . . . (4)

If the backfill is having its top surface inclined at an angle $\beta$, the passive pressure is given by

$$p_p = \gamma z \cos\beta \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \quad \dots (5)$$

$$\Rightarrow p_p = K_p \gamma z \quad \text{where} \quad K_p = \cos\beta \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \quad \dots (6)$$

**(ii) Cohesive backfill**

For the case of cohesive soil, the principal stress relationship at failure is given by

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$$

For the case of passive pressure,

$$\sigma_1 = \sigma_h = p_p$$
$$\sigma_3 = \sigma_v = \gamma z$$

Substituting these values of $\sigma_1$ and $\sigma_3$, we get

$$p_p = \gamma z \tan^2\alpha + 2c\tan\alpha \quad \dots (7)$$

$$\Rightarrow p_p = \gamma z N_\phi + 2c\sqrt{N_\phi}$$

At $z = 0$, $p_p = 2c\tan\alpha$

At $z = H$, $p_p = \gamma H \tan^2\alpha + 2c\tan\alpha$

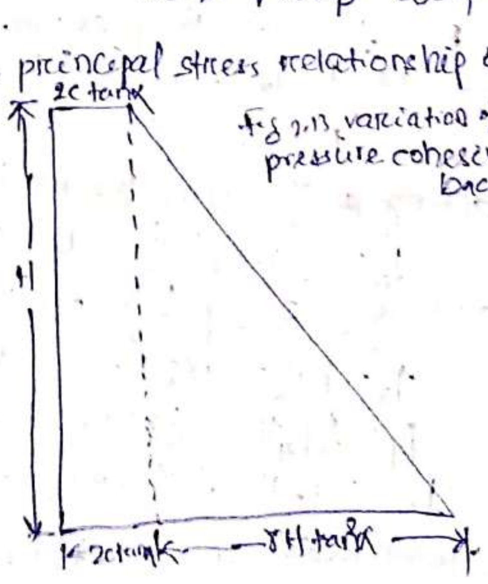



Fig 2.13 variation of passive pressure cohesive backfill

Scanned with OKEN Scanner

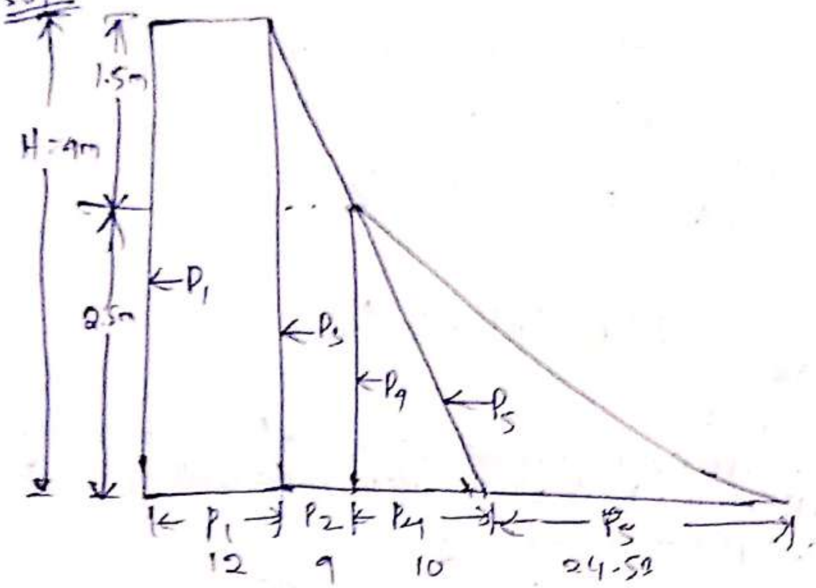
Fig 20.13 shows the pressure distribution diagram. The total pressure is given by

$$P_p = \int_0^H p_p \, dz = \frac{1}{2} \gamma H^2 \tan^2\alpha + 2cH \tan\alpha \quad \ldots$$

$$= \frac{1}{2} \gamma H^2 N_\phi + 2cH \sqrt{N_\phi} \quad \ldots \ldots (20.6)$$

Ex 20.4 In example 20.3, if the water table rises behind the wall to an elevation 1.5 m below the top, determine the total active pressure and its point of application. Take submerged weight of sand as 12 kN/m³. Assume that there is no change in the angle of shearing resistance due to submergence.

Sol^n




Let $p_1$ = lateral pressure intensity due to surcharge

$p_2$ = " " due to dry soil

$p_4$ = " " " due to submerged soil

$p_5$ = " " " due to water

$\therefore \; p_1 = K_a q = \frac{1}{3} \times 36 = 12 \text{ kN/m}^2$

$p_2 = K_a \gamma H_1 = \frac{1}{3} \times 18 \times 1.5 = 9 \text{ kN/m}^2$

$p_4 = K_a \gamma' H_2 = \frac{1}{3} \times 12(2.5) = 10 \text{ kN/m}^2$

$p_5 = \gamma_w \cdot H_2 = 9.81 \times 2.5 = 24.53 \text{ kN/m}^2$

Fig 20.15 shows the pressure distribution diagram with the resultant pressure $P_1, P_2, P_3, P_4$ and $P_5$.

$P_1 = p_1 H = 12 \times 4 = 48 \text{ kN/m}$

acting @ $\frac{4}{2} = 2$m from base

$P_2 = \frac{1}{2} p_2 H_1 = \frac{1}{2} \times 9 \times 1.5 = 6.75 \text{ kN/m}$

acting @ $2.5 + \frac{1.5}{3} = 3$, from base

$P_3 = p_2 H_2 = 9 \times 2.5 = 22.5 \text{ kN/m}$, acting @ 1.25 m from base

$P_4 = \frac{1}{2} p_4 H_2 = \frac{1}{2} \times 10 \times 2.5 = 12.5 \text{ kN/m}$ acting @ 0.833 m from base.

$P_5 = \frac{1}{2} p_5 H_2 = \frac{1}{2} \times 24.53 \times 2.5 = 30.66 \text{ kN/m}$, acting @ 0.833 m from base.

Total pressure $= P = P_1 + P_2 + P_3 + P_4 + P_5$

$= 48 + 6.75 + 22.5 + 12.5 + 30.66$

$= 120.41 \ kN/m$

The distance $\bar{z}$ of the point of application of P above the base is obtained by taking about the base.

$$\bar{z} = \frac{1}{120.41}\left[(48\times2) + (6.75\times3) + (22.5\times1.25) + (12.5\times0.833) + (30.66\times0.833)\right]$$

$= 1.50 \ m$

## Submerged backfill

In this case, the sand fill behind the retaining wall is saturated with water. The lateral pressure is made up of two components.

(i) lateral pressure due to submerged weight $\gamma'$ of the soil and

(ii) lateral pressure due to water. Thus, at any depth z below the surface

$$P_a = K_a\gamma' z + \gamma_w z$$

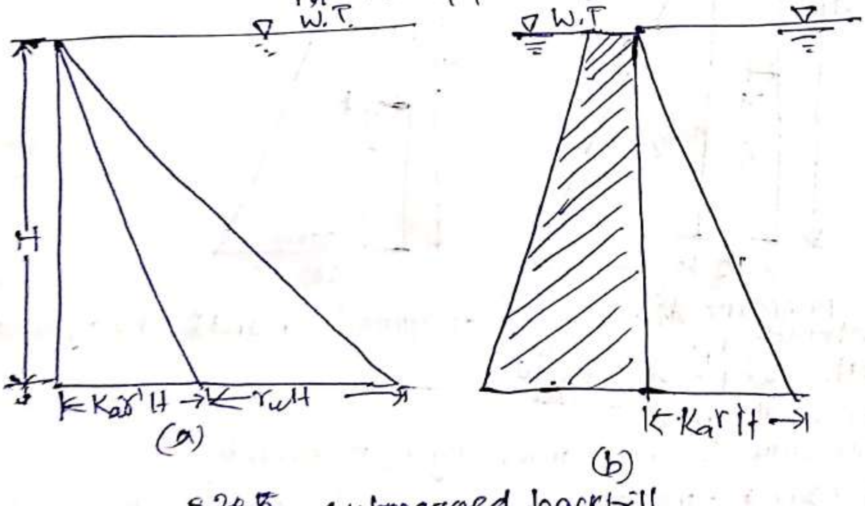
The pressure at the base of the retaining wall (z = H) is given by .. ①

If free water stands to both sides of the wall (Fig 20.5 (b)) the water pressure need not be considered, and the net lateral pressure is given by:

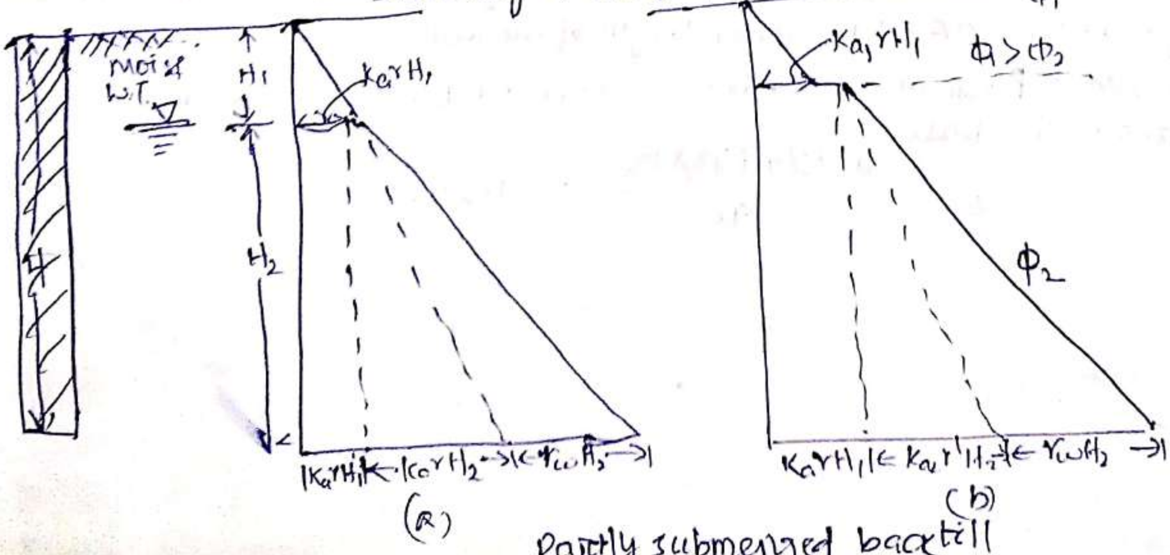
$$P_a = K_a\gamma' H \quad \cdots \cdots \ ②$$

If the backfill is partly submerged, i.e. the backfill is moist to a depth $H_1$ below the ground level, and then it is submerged, the lateral pressure intensity at the base of the wall is given by ③

$$P_a = K_a\gamma H_1 + K_a\gamma' H_2 + \gamma_w H_2$$




F.20.5 submerged backfill




Partly submerged backfill

The above expression is on the assumption that the value of $\phi$ is the same for the moist as well as submerged soil. If it is different, say $\phi_1$ and $\phi_2$ respectively, the earth pressure coefficient $K_{a_1}$ and $K_{a_2}$ for both the portions will be different. As $\phi$ decreases, $K_a$ increases. The lateral pressure intensity [Fig 20.6(c)] at the base of wall is given by:

$$P_a = K_{a_2}\gamma H_1 + K_{a_2}\gamma' H_2 + \gamma_w H_2 \quad (20.17)$$

**Ex 20.3** A retaining wall 4m high has a smooth vertical back. The backfill has a horizontal surface in level with the top of the wall. There is uniformly distributed surcharged load of 36 KN/m² intensity over the backfill. The unit weight of the backfill is 18 KN/m³, its angle of shearing resistance is 30° and cohesion is zero. Determine the magnitude and point of application of active pressure per metre length of the wall.

**Ans**

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = \frac{1}{3}$$

The lateral pressure intensity due to the surcharge is given by

$$P_1 = K_a q = \frac{1}{3}\times 36 = 12 \text{ KN/m}^2$$

The pressure intensity due to the backfill, at depth $H = 4$m is given by

$$P_2 = K_a \gamma H = \frac{1}{3}\times 18\times 4 = 24 \text{ KN/m}^2$$

The total pressure intensity at the base of the wall is given by

$$P_a = P_1 + P_2 = 12 + 24 = 36$$

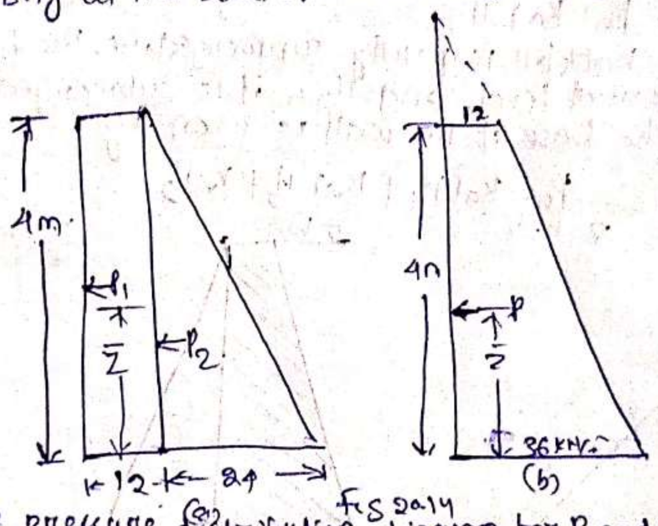



Fig 20.14 shows the pressure distribution diagram for $P_1$ and $P_2$. The resultant total pressure $P_1$ due to intensity $p_1$ is given by

$$P_1 = p_1 \times H = 12\times 4 = 48 \text{ KN/m}$$

acting at $\frac{4}{2} = 2$m from the base

The resultant total pressure $P_2$ due to intensity $p_2$ is given by

$$P_2 = \frac{1}{2}p_2 H = \frac{1}{2}\times 24\times 4 = 48 \text{ KN/m}$$ acting at $\frac{1}{3}\times 4 = 1.33$m from the base.

$\therefore P = P_1 + P_2 = 48 + 48 = 96$ KN per meter length of the wall.

The resultant $P$ acts at a distance $\bar{z}$ above the base, given by taking the moments about the base.

$$\bar{z} = \frac{(48\times 2) + (48\times 4/3)}{96} = 1.67 \text{ m}.$$

# Foundation

Foundation is the lowest part of the building or the civil structure that is in direct contact with the soil which transfer loads from the structure to the soil safely. Generally, the foundation can be classified into two, namely shallow foundation and deep foundation.

A shallow foundation transfers the load to a stratum present in a shallow depth. The deep foundation transfer the load to a deeper depth below the ground surface.

A tall building like a skyscraper or a building constructed on very weak soil requires deep foundation. If the constructed building has the plan to extend vertically in future, then a deep foundation must be suggested.

Function

Foundations provide the structure stability from the ground. To distribute the weight of the structure over a large area in order to avoid overloading the underlying soil

1. Reduction of load intensity
2. Even distribution of load
3. Provision of level surface
4. Lateral stability
5. Safety Against
6. Protection against soil movement

Footing : A footing is a portion of the foundation of a structure that transmits loads directly to the soil.
Foundation: A foundation is that part of the structure which is direct contact with and transmits loads to the ground.
Foundation soil : It is the upper part of the earth mass carrying the load of the structure.

---

The foundation of some important engineering structure require special treatment. Such structures have to be designed for heavy loads and ordinary methods of providing foundations may not be suitable for such structures.

(1) Grillage foundation
(2) Raft foundation
(3) Inverted arches

1. Grillage foundation

In this method, the depth is limited to 1m to 1.50m and the width is increased considerably to bring the pressure on the soil within permissible limit. The superstructure rests on two perpendicular tiers of R.S.J. Fig 5-11 and Fig 5-12 shows typical grillage foundations for a steel stanchion and a wall respectively. Following points should be noted.

(i) The R.S.J. work should be thoroughly embedded in concrete so as to protect it from the atmospheric actions. The bed of concrete should have minimum thickness of 150mm and at no other point, the depth of concrete should be less than 80mm,

(iii) The concrete filling does not carry any load. But it maintains the beam in proper position and prevents them from the corrosion.

2 Raft foundation :- Raft foundation is actually a thick concrete slab resting on a large area of soil reinforced with steel, supporting columns or walls and transfer loads from the structure to the soil. Usually, mat foundation is spread over the entire area of the structure it is supporting.

Raft foundation is generally used to support structures like residential or commercial buildings where soil condition is poor, storage tanks, silo foundation, for heavy industrial equipment etc.

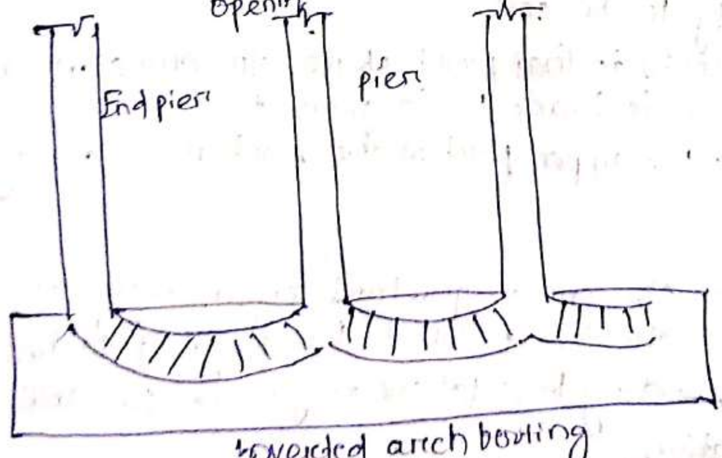
For foundation design, one of the most important aspects is choosing the right type of foundation. Raft foundation is preferred when

→ The soil has a low bearing capacity
→ Load of the structure has to be distributed over a large area
→ Individual or any other foundation area would approximately over 50% of the total ground area beneath the structure.
→ The columns or walls are placed so closely that the individual footings would overlap.
→ Stress on soil needs to be reduced.

3 Inverted arch footing foundation :- An inverted arch or invert is a civil engineering structure in the form of an inverted arch, inverted in comparision to the usual arch bridge. Like the flying arch, the inverted arch is not used to support a load, as for a bridge, but rather to resist sideways, inwards loads.

They constructed between two walls of the base. When the walls must be sufficiently thick. When it makes a strong withstand the outward horizontal thrust caused by the arch action.

In an arch making parts shown in the diagram.
→ End pier → piers → openings




inverted arch footing

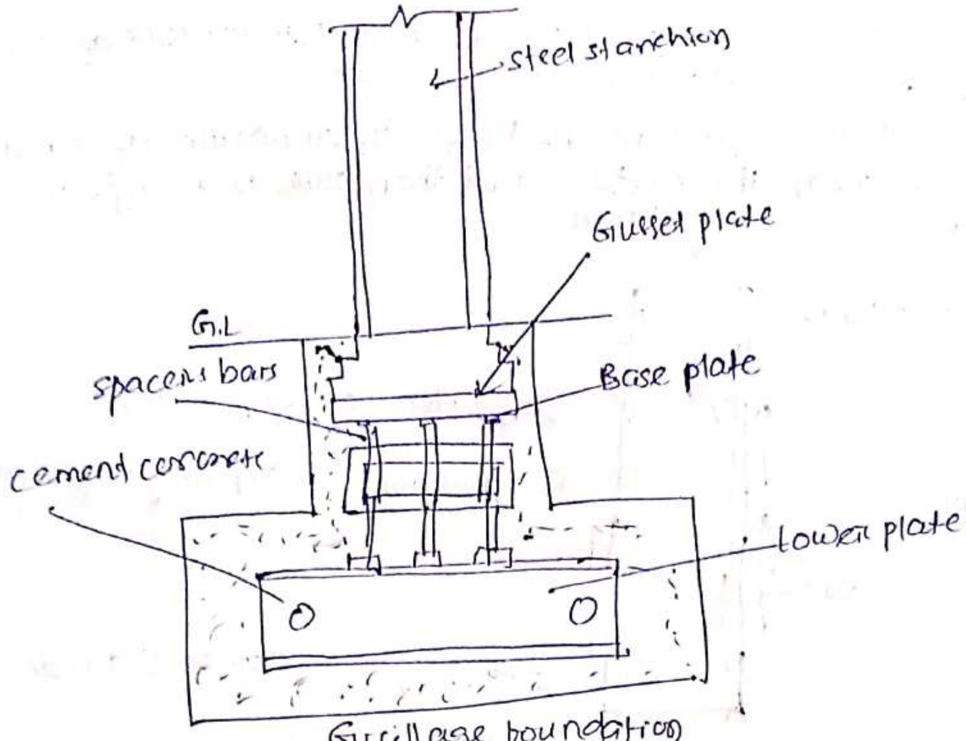
End pier: The end pier has to be especially strength end by buttress to avoid the arch trusted trading the rupture the pier junction. It is an endpoint of arch.

They are also used in places where the bearing capacity of the soil is very poor. When a load of the structure concentrate over the walls and deep excavations are not possible.

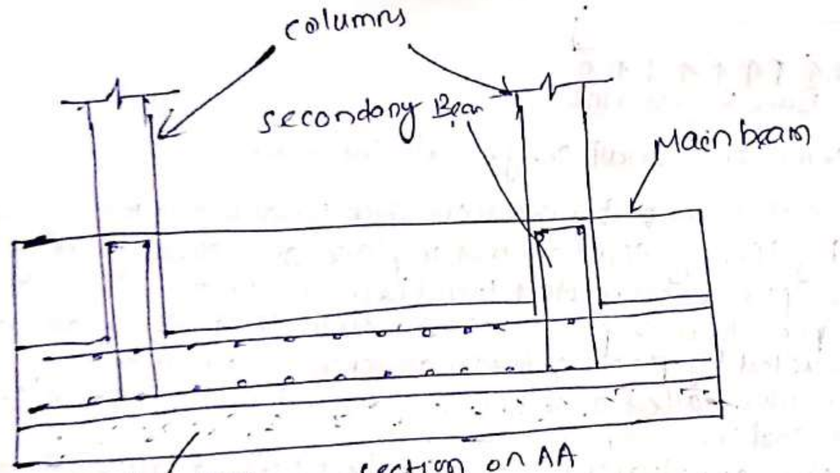
Inverted arch construction is that in soft soils the depth of foundation generally reduces.

In They inverted arch need skilled labour.

This is a costly footing.



Grillage foundation



Raft foundation

## Deep foundation

It is required to carry loads from a structure through weak compressible soils or fills on to stronger and less compressible soils or rocks at depth, or for functional reasons. Deep foundation are founded too deeply below the finished ground surface for their base bearing capacity to be affected by surface condition, thus is usually at depth >3m below finished ground level.

Deep foundation can be used to transfer the loading to a deeper more competent strata at depth if unsuitable soils are present near the surface.

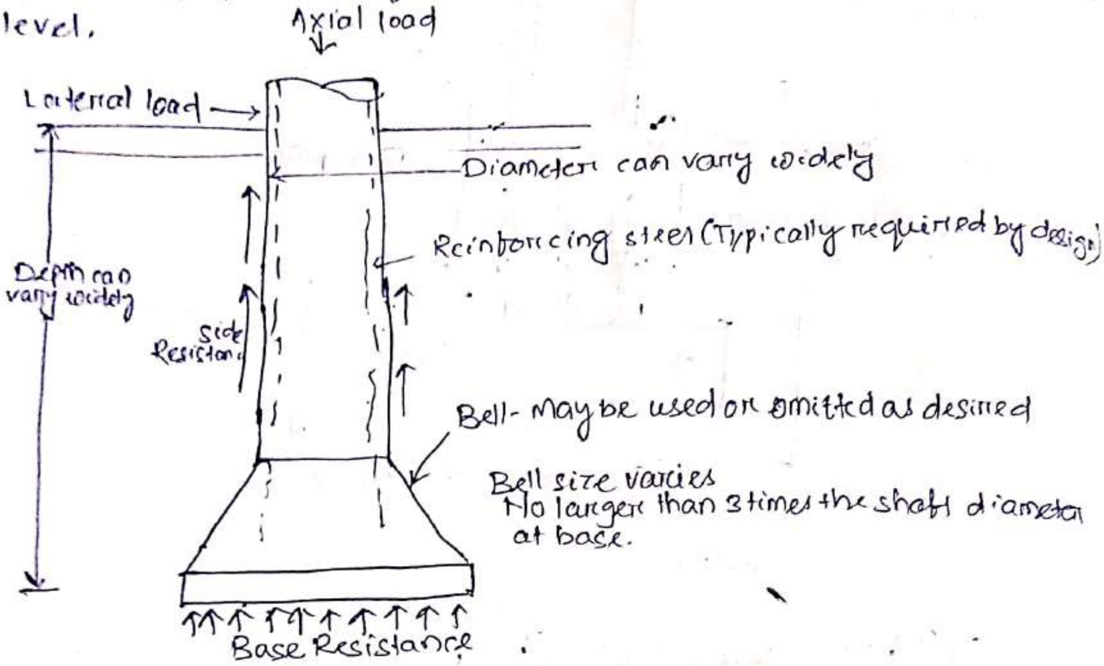
The type of deep foundation in general use are as follow.

1. Basements 2. Buoyancy rafts (hollow box foundation)
3. Caissons 4. cylinders 5. shaft foundation
6. Pile foundation

**1 Basement foundation:-** These are hollow substructures designed to provide working or storage space below ground level. The structural design is governed by their functional requirements rather than from considerations of the most efficient method of resisting external earth and hydrostatic pressure. They are constructed in place in open excavation.

**2 Buoyancy Rafts:-** Buoyancy rafts are hollow substructures designed to provide a buoyant or semi buoyant substructure beneath which the net loading on the soil is reduced to the desired to be sunk as caissons, they can also be constructed in place in open excavations

3 Caissons Foundation :- Caissons are hollow substructures designed to be constructed on or near the surface and then sunk as a single unit to their required level.



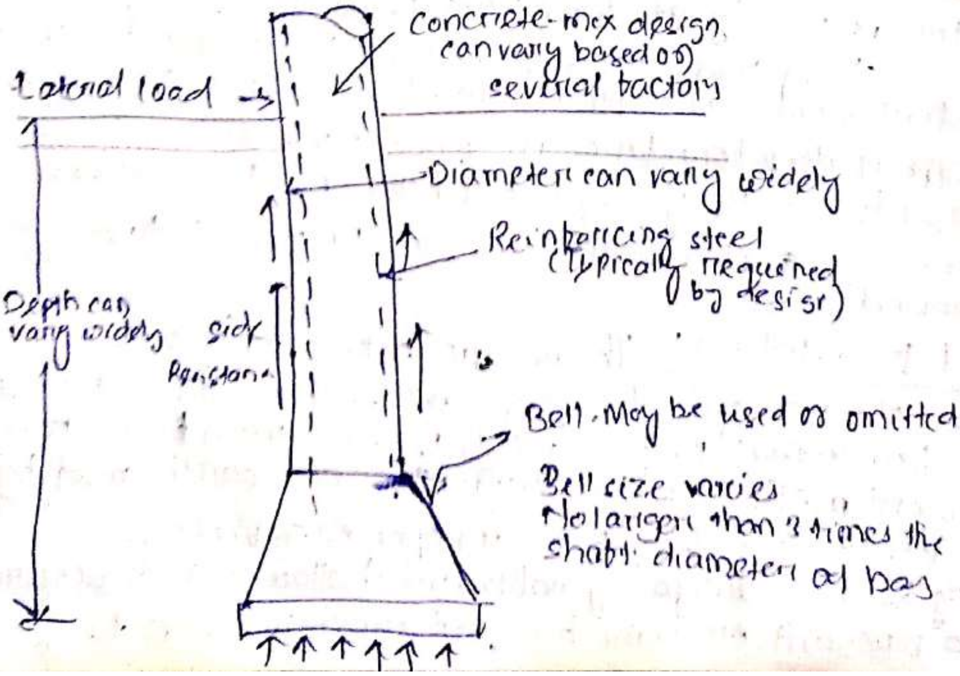

4. cylinders : Cylinders are small single-cell caissons

5 Drilled shaft foundations :- Shaft foundations are constructed within deep excavation supported by lining constructed in place and subsequently filled with concrete or other pre-fabricated load bearing units.

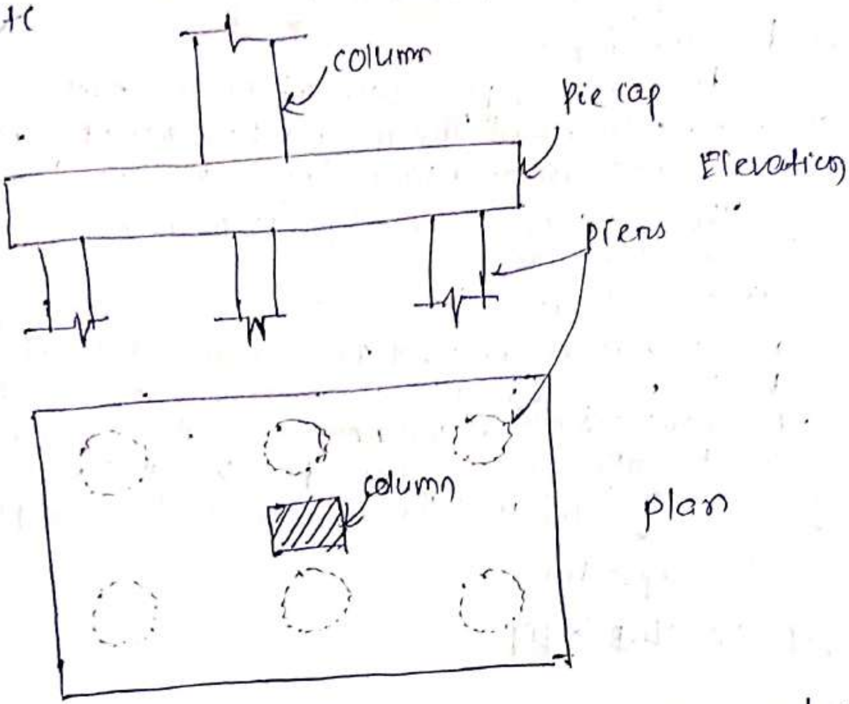
A drilled pier is a deep foundation system that uses a large diameter concrete cylinder constructed by placing fresh concrete and reinforcing steel into a drilled shaft. It is also called as a caisson, drilled shaft, cast-in-drilled hole piles (CIDH piles) or cast-in-situ piles.

For the construction of a drilled pier, a large diameter hole is drilled in the ground and filled with concrete subsequently. The difference between a drilled pier and bored pile is basically of the size.

Generally, bored piles are of diameters less than or equal to 0.6m. The shafts of sizes larger than 0.6m are generally designated as drilled piers. A drilled pier is a type of deep foundation constructed to transfer heavy axial or lateral load to a deep stratum below the ground surface.



6. Pile foundations :- Pile foundations are relatively long and slender member constructed by driving preformed units to the desired founding level or by driving or drilling in tubes to the required depth - the tubes being filled with concrete before or during withdrawal or by drilling unlined or wholly or partly lined boreholes which are then filled with concrete




The pressure which the soil can safely withstand is known as the allowable bearing pressure.

Ultimate Bearing Capacity ($q_u$) :- The ultimate bearing capacity is the gross pressure at the base of the foundation at which the soil fails in shear.

Net ultimate Bearing Capacity ($q_{nu}$) :- It is the net increased in pressure at the base of foundation that causes shear failure of the soil. It is equal to the gross pressure minus overburden pressure . . . ①

Thus $q_{nu} = q_u - \gamma D_f$

where, $q_u$ = ultimate bearing capacity (gross)

$\gamma$ = unit weight of foundation soil and

$D_f$ = depth of foundation

The overburden pressure equal to $\gamma D_f$ existed even before the construction of foundation.

Net safe Bearing Capacity ($q_{ns}$) :-

It is the net soil pressure which can be safely applied to the soil considering only shear failure. It's obtained by dividing the net ultimate bearing capacity by a suitable factor of safety.

Thus, $q_{ns} = \frac{q_{nu}}{F}$ where F = Factor of safety, which is usually taken 3.0

Gross safe Bearing Capacity ($q_s$) :-

It is the maximum gross pressure which the soil can carry safely without shear failure. It is equal to the net safe bearing capacity plus the original overburden pressure. Thus

$$q_s = q_{ns} + \gamma D_f$$

$$q_s = \frac{q_{nu}}{F} + \gamma D_f \quad \cdots \quad ②$$

Some author define, the gross safe bearing capacity ($q_s$) as the ultimate bearing capacity divided by a factor of safety (F), that is

$$q_s = \frac{q_u}{F} = \frac{q_{nu} + \gamma D_f}{F} = \frac{q_{nu}}{F} + \frac{\gamma D_f}{F}$$

As the added strength due to $\gamma D_f$ is available in full, it does not seem logical to apply a factor of safety to this term. It is more rational to define the gross safe bearing capacity as indicated by equation (2)

Net safe settlement Pressure ($q_{np}$):- It is the net pressure which the soil can carry without exceeding the allowable settlement. The maximum allowable settlement generally varies between 25mm and 40mm for individual footing.

The net safe settlement pressure is also known as unit soil pressure or safe bearing pressure.

Net Allowable Bearing Pressure ($q_{na}$):- The net allowable bearing pressure is the net bearing pressure which can be used for the design of foundation.

As the requirements for the design of foundation are that there should be no shearing failure and more over the settlement should also be within the limits, the allowable bearing pressure is the smaller of the net safe bearing capacity ($q_{ns}$) and the net safe settlement pressure ($q_{np}$)

Thus.

$$q_{na} = q_{ns} \text{ if } q_{np} > q_{ns}$$

$$q_{na} = q_{np} \text{ if } q_{ns} > q_{np}$$

## Minimum depth of foundation: Rankine's Analysis

Rankine considered the equilibrium of two soil elements, one immediately below the foundation (element I) and the other just beyond the edge of the footing (element II) but adjacent to element I. When the load on the footing increases, and approaches a value $q_f$, a state of plastic equilibrium is reached under the footing. For the shear failure of element I, element II must also fail by lateral thrust from element I. During the state of shear failure (plastic equilibrium), the following principal stress relationship exist

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha \quad \text{--- (1) 24.5}$$

24.5 (1)

For cohesionless soil $\sigma_1 = \sigma_3 \tan^2\alpha$ ... (2)

For element II: $\sigma_3 = \sigma_v = \gamma D$

$\therefore \sigma_1 = \sigma_h = \gamma D \tan^2\alpha$ ... (3)

For element I,

$\sigma_3 = \sigma_h = \sigma_1$ of element II

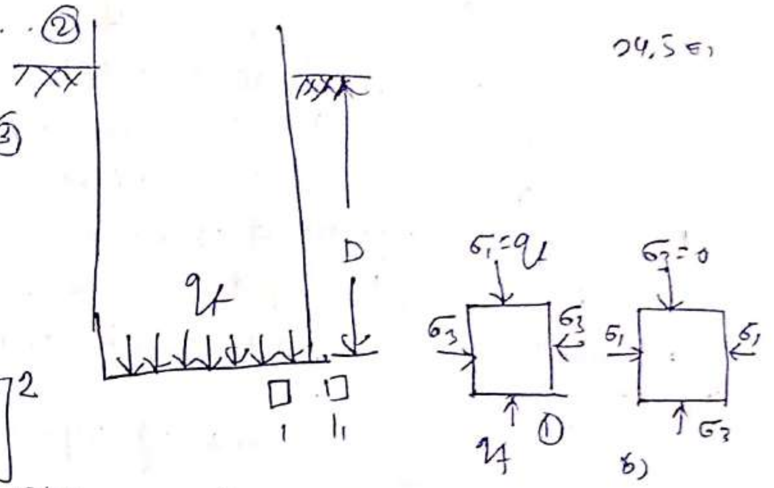
$= \gamma D \tan^2\alpha$

$\sigma_1 = \sigma_3 \tan^2\alpha = \gamma D \tan^4\alpha$

But $\sigma_1 = q_f$

$$\therefore q_f = \gamma D \tan^4\alpha = \gamma D \left[\frac{1+\sin\phi}{1-\sin\phi}\right]^2$$

- (24.6)




f 24.1 Rankine's Analysis

In effective stress analysis the above expression reduces to

$$q_f = \gamma D \left[\frac{1+\sin\phi'}{1-\sin\phi'}\right]^2 \quad \text{--- (24.7)}$$

Eq 24.7 gives the bearing capacity of cohesionless soils as zero at the ground surface. This is not consistent with the general experience. Eq 24.6 may be used in the following form to get the minimum depth of foundation

$$D_{min} = \frac{q}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2 \quad \text{--- (24.8)}$$

where $q$ = intensity of loading.

## Assumption in Terzaghi's Analysis

→ The soil is homogeneous and isotropic and its shear strength is represented by Coulomb's equation

→ The strip footing has a rough base, and the problem is essentially two dimensional.

→ The elastic zone has straight boundaries inclined at $\psi = \phi$ to the horizontal, and the plastic zones fully develope.

→ $P_f$ consists of three components which can be calculated separately and added, although the critical surface for these components are not identical.

→ Failure zones do not extend above the horizontal plane through the base of the footing i.e. the shear resistance of soil above the base is neglected and the effect of soil around the footing is considered equivalent to a surcharge $\sigma = \gamma D$.

## General Bearing Capacity Equation: Brinch Hansen's Analysis.

In recent years, many new equation have been proposed by various workers, giving the ultimate bearing capacity of foundations. These equations are based on the assumption of various values of angle $\psi$ and on the shape of failure surfaces. Notable amongst the workers are Hasen (1970)

Hu(1964), Chen and Davidson(1972) and Balla(1962). Out of these the one proposed by Hansen gives better results.

According to Hansen, the ultimate bearing capacity is given by

$$q_f = cN_c s_c d_c i_c g_c b_c + \bar{\sigma}_0 N_q s_q d_q i_q g_q b_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

where $\sigma_0 = \bar{\sigma}$ = effective overburden pressure at foundation level

$s$ = shape factor, to account for the effect of the shape of the foundation in developing a failure surface

$d$ = depth factor to account for the embedment depth and the additional shearing resistance in the top soil

$i$ = inclination factor to account for both horizontal and vertical components of foundation loads

$g$ = ground factor ; $b$ = base factor

$\gamma$ = density of soil below the foundation level

For $\phi = 0$ or undrained condition the above equation is modified to

$$q_f = 5.14c\left[1 + s_c' + d_c' - i_c' - b_c' - g_c'\right] + \bar{\sigma}_0 \quad \ldots (24.27)$$

where the various factors with dashes (primed) are for $\phi = 0$ (undrained) condition.

In Eq. 24.26, the bearing capacity factors are computed by the following equation

$$N_q = \tan^2\left(45° + \frac{\phi}{2}\right)$$

## Types of bearing capacity of failures

Experimental investigation have indicated that when a footing fails due to insufficient bearing capacity, distinct failure patterns are developed depending upon type of failure mechanism. Failure is accompanied by appearance of failure surfaces and by bulding of sheared mass of soil. Vesic (1963) observed 3 types of bearing capacity failures (Fig 24.2)

i) General shear failure, 2. Local shear failure 3. Punching shear failure

1. General shear failure: In the case of general shear failure, continuous failure surfaces develop between the edges of the footing and the ground surface, as shown in Fig 24.2(c). When the pressure approaches the value of $q_f$, the state of plastic equilibrium is reached initially in the soil around the edges of the footing and it then gradually spreads downwards and outwards. Ultimately, the state of plastic equilibrium is fully developed throughout the soil above the failure surface. The failure is accompanied by appearance of failure surfaces and by considerable bulging of sheared mass of soil. However, the final slip movement would occur only on one side, accompanied by tilting of the footing. Such a failure occurs in soil of low compressibility i.e. dense or stiff soil and the pressure. settlement curve is of the general form as shown in curve a of Fig 24.2(a). Following are the typical characteristics of general shear failure.

→ It has well defined failure surfaces, reaching upto ground surface

→ There is a considerable bulging of sheared mass of soil adjacent to the footing

→ Failure is accompanied by tilting of the footing
→ Failure is sudden, with pronounced peak resistance.
→ The ultimate bearing capacity is well defined.

2. Local shear failure: In local shear failure, there is a significant compression of the soil under the footing and only partial development of state of plastic equilibrium. Due to this reason, the failure surface do not reach the ground surface and only slight heaving occurs. The pressure settlement curve is represented by curve b of Fig 24.2(a) where the peak of the base resistance may never be reached. In such a failure, tilting of foundation is not expected. Local shear failure is associated with soils of high compressibility and in sands having relative density lying between 35 and 70%. The failure is not sudden, and it is characterised by occurrence of relatively large settlement which would not be acceptable in practice. Also, ultimate bearing capacity in such a failure is not well defined. Following are typical characteristics of local shear failure.

→ Failure pattern is clearly defined only immediately below the footing
→ The failure surfaces do not reach ground surface.
→ There is only slight bulging of soil around the footing.
→ Failure is not sudden and there is no tilting of footing.
→ Failure is defined by large settlement.
→ Ultimate bearing capacity is not well defined.

3. Punching shear failure:- Punching shear failure occurs when there is relatively high compression of soil under the footing, accompanied by shearing in the vertical direction around the edges of the footing. Punching shear may occur in relatively loose sand with relative density less than 35%. Punching shear failure may also occur in a soil of low compressibility if the foundation is located at considerable depth. The failure surface which is vertical or slightly inclined and follows the perimeter of the base never reaches the ground surface. There is no heaving of the ground surface away from the edges and no tilting of the footing. Relatively large settlements occur in this mode. The ultimate bearing capacity is not well defined. Following characteristics of punching shear failure.

→ No failure pattern is observed
→ The failure surface, which is vertical or slightly inclined, follows the perimeter of the base
→ There is no bulging of soil around the footing.
→ There is no tilting of footing
→ Failure is characteristics in terms of very large settlement
→ The ultimate bearing capacity is not well defined.

Ex 24.5 A strip footing 2m wide carries a load intensity of 400 kN/m² at a depth of 1.2m in sand. The saturated unit weight of sand is 19.5 kN/m³ and unit weight above water table is 16.8 kN/m³. The shear strength parameters are c=0 and $\phi = 35°$. Determine the factor of safety with respect to shear failure for the following cases of location of water table:

(a) Water table is 4m below G.L (b) Water table is 1.2m below G.L.
(c) Water table is 2.5m below G.L. (d) Water table is 0.5m below G.L.
(e) Water table is at G.L. itself. Use Terzaghi's equation.

Ans For a strip footing, the bearing capacity is given by

$$q_f = cN_c + \bar{\sigma} N_q + \frac{1}{2} B \gamma N_\gamma$$

Taking into account the water reduction factor, we have from E[24.3]

$$q_f = cN_c + \gamma D N_q R_{w1} + \frac{1}{2} B \gamma N_\gamma \cdot R_{w2}$$

For the present case c = 0.

$$\therefore q_f = \gamma D N_q R_{w1} + \frac{1}{2} B \gamma N_\gamma \cdot R_{w2}$$

For $\phi = 35°$ assuming general shear failure, $N_q = 41.4$ and $N_\gamma = 42.4$

$$\therefore q_f = 41.4 \times 1.2 \times \gamma R_{w1} + \frac{1}{2} \times 2 \times 42.2 \gamma R_{w2}$$

$$q_f = 49.68 \gamma R_{w1} + 42.3 \gamma R_{w2}$$

Case (a) Water table is 4m below G.L

$Z_{w2} = 4 - 1.2 = 2.8m$ $R_{w1} = 1$

since $Z_{w2} > B$, $R_{w2} = 1$

Hence there will be no effect of water table.

Also $\gamma = 16.8$.

$$\therefore q_f = 49.68 \times 16.8 \times 1 + 42.4 \times 16.8 \times 1$$
$$= 1546.9 \text{ kN/m}^2$$

Now actual footing load $= q_a = 400$ kN/m²

$$\therefore F.S. = \frac{q_f}{q_a} = \frac{1546.9}{400} = 3.87$$

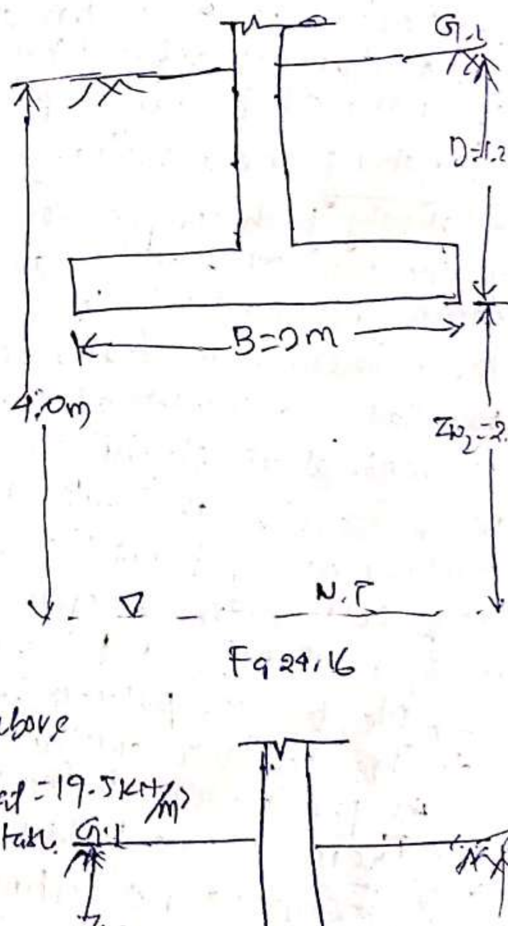



Fig 24.16

Case (b) Water table is just at base of footing.

$$\therefore R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5(1+1) = 1$$

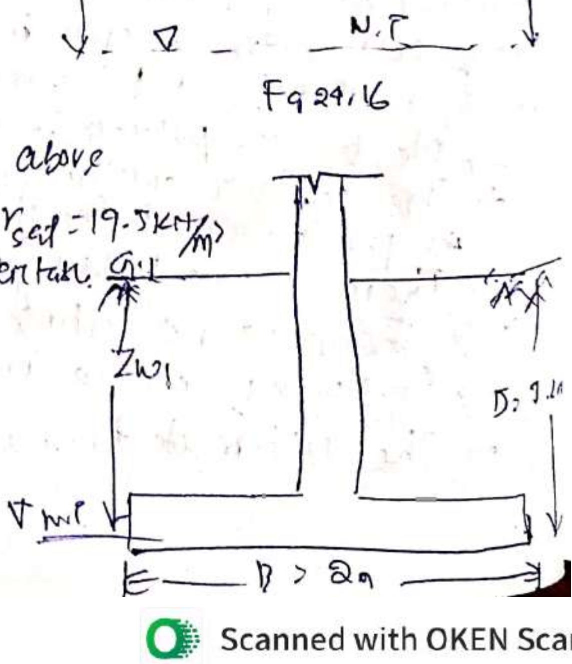
$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5(1+0) = 0.5$$

$\therefore$ For the surcharge soil is saturated above water table. For the wedge term, use $\gamma = \gamma_{sat} = 19.5$ kN/m³ since the wedge soil is situated below water table.

$$\therefore q_f = 49.68 \gamma R_{w1} + 42.4 \gamma_{sat} \cdot R_{w2}$$

$$\Rightarrow q_f = 49.68 \times 16.8 \times 1 + 42.4 \times 19.5 \times 0.5$$
$$= 1248 \text{ kN/m}^2$$

$$\therefore F.S. = \frac{q_f}{q_a} = \frac{1248}{400} = 3.12$$



Case (c) Water table at 2.5m below the G.L. (Fig 24.18 (a))

$Z_{w2} = 2.5 - 1.2 = 1.3\,m < B.$

$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{1.3}{2}\right) = 0.825$

For the surcharge term, $\gamma = 16.8\ kN/m^3$

For the wedge term, $\gamma$ will be taken as average unit weight of soil situated below the footing level, since the soil upto depth B below the footing is partly above water table and partly the water table.

$$\gamma_{av} = \frac{(16.8 \times 1.3) + (19.5 \times 0.7)}{(1.3 + 0.7)} = 17.75\ kN/m^3$$

Hence from Eq. 1, we have $q_f = 49.68\,\gamma R_{w1} + 42.4\,\gamma_{av} R_{w2}$

$= (49.68 \times 16.8 \times 1) + (42.4 \times 17.75 \times 0.825)$

$= 1455.5\ kN/m^2$

$$F.S. = \frac{q_f}{q_a} = \frac{1455.5}{400} = 3.64$$

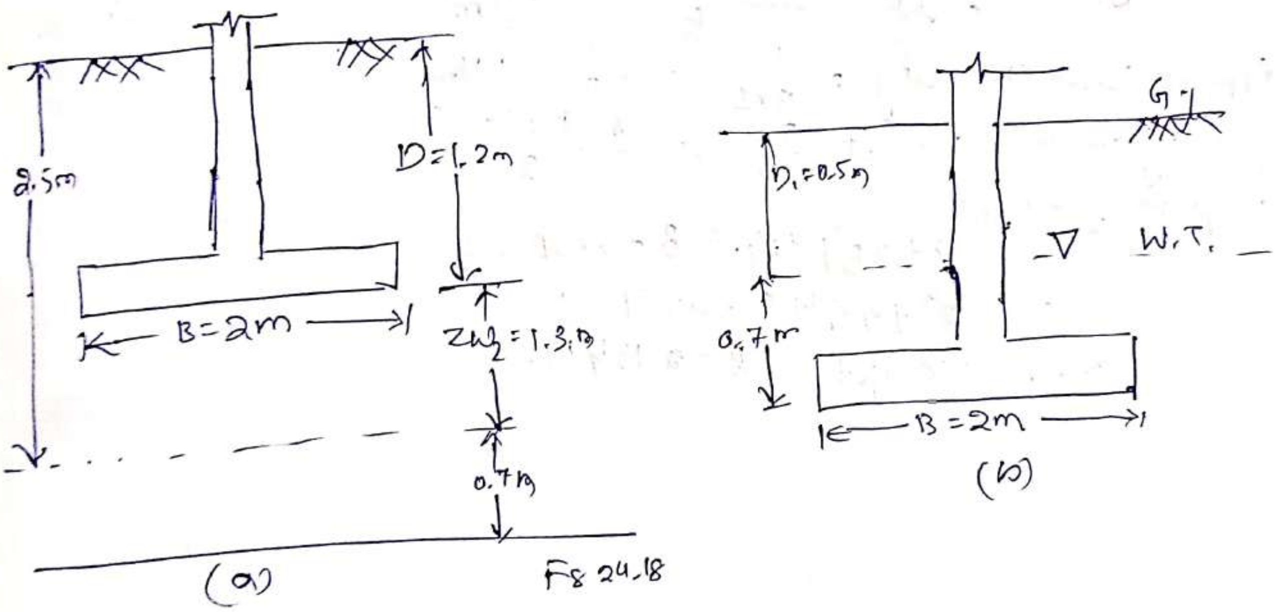



Fig 24.18

(d) Water table is 0.5m below G.L. (Fig 24.18 b)

$Z_{w1} = 0.5$

$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{0.5}{1.2}\right) = 0.708$

$R_{w2} = 0.5$. Since water table is above footing

For the wedge term, $\gamma = \gamma_{sat}\ 19.5\ kN/m^3$

For the surcharge term, take $\gamma$ = average unit weight of soil situated above the base of footing, since this soil is located partly above the water table and partly below the water table.

$$\therefore \gamma_{av} = \frac{(16.8 \times 0.5) + (19.5 \times 0.7)}{(0.5 + 0.7)} = 18.38$$

Hence from Eq. 1 $q_f = 49.68\,\gamma_{av} R_{w1} + 42.4\,\gamma_{sat} R_{w2}$

$= (49.68 \times 18.38 \times 0.708) + (42.4 \times 19.5 \times 0.5)$

$= 1060\ kN/m^2$

$$F.S. = \frac{q_f}{q_a} = \frac{1060}{400} = 2.65$$

Ex24.7 Design a strip footing to carry a load of 750 kN/m at a depth of 1.6 m in a c-ϕ soil having a unit weight of 18 kN/m³ and shear strength parameters as $c = 20$ kN/m² and $\phi = 25°$. Determine the width of footing, using a factor of safety of 3 against shear failure. Use Terzaghi's equation.

Sol$^n$ Assume general shear failure. From Eq. 24.12

$$q_f = cN_c + \bar{\sigma} N_q + 0.5\gamma B N_\gamma$$

For $\phi = 25°$

we have $N_c = 25.1$, $N_q = 12.7$ and $N_\gamma = 9.7$ from Table 24.2.

Also $\bar{\sigma} = \gamma D = 18 \times 1.6$

Substituting the values, we get

$$q_f = (20 \times 25.1) + (18 \times 1.6) \times 12.7 + 0.5 \times 18 \times B \times 9.7$$

$$q_f = 502 + 365.8 + 87.3B = 867.8 + 87.3B \quad \text{--- (1)}$$

∴ Intensity of pressure, at F.S. of 3 at footing level

$$= \frac{q_f}{3} = \frac{867.8 + 87.3}{B} \text{ kN/m}^2 \quad \text{--- (2)}$$

Applied load intensity $= \dfrac{750}{B \times 1} = \dfrac{750}{B}$ kN/m² --- (3)

Equating the two, we get $\dfrac{750}{B} = \dfrac{867.8 + 87.3B}{3}$

simplifying, we get

$$87.3B^2 + 867.8B - 2250 = 0$$

$$\Rightarrow B^2 + 9.94B - 25.7 = 0$$

From we get, $B = 2.134$ m.

Scanned with OKEN Scanner

## Plate load test

It is a field test to determine the ultimate bearing capacity of soil, and the probable settlement under a given loading. The test essentially consists in loading a rigid plate at the foundation level, and determine the settlements corresponding to each load increment. The ultimate bearing capacity is then taken as the load, at which the plate starts sinking at a rapid rate. The method assumes that down to the depth of influence of stresses, the soil strata is reasonably uniform.

1. Bearing plate: The bearing plate is either circular or square, made of mild steel of not less than 25mm in thickness and varying in size from 300 to 750mm with chequered or grooved bottom (Fig 24.20). The plate is provided with handles for convenient setting and centre marked. As an alternative, cast in situ or precast concrete blocks may be used with depth not less than thirds the width.

Except in case of road problems or circular footings, square plates may be adopted. For clayey and silty soils and for loose to medium dense sandy soils with N <15, a 450mm square plate or concrete block shall be used. In case of dense sandy or gravely soil (15<N<30) three plate of sizes 300mm to 750mm shall be used depending upon the practical considera of reaction loading and maximum grain size. The size of the plate shall be atleast four times the maximum size of the soil particles present at the test location.

2. Test pits: The test pit, usually at the foundation level, having in general normally of width equal to five times the test plate ($B_p$) or block, shall have a carefully levelled and cleaned bottom at the foundation level, protected against disturbances or changes in natural formation. The test pits should preferably have steps to conveniently go in the pit for setting and for taking observation.

3. Loading arrangement :- The loading ~~arrangement~~ to the test plate may be applied with the help of a hydraulic jack. The reaction of the hydraulic jack may be borne by either of the following 2 methods.

(a) Gravity loading platform method (Fig 24.21)

b) Reaction truss method (Fig 24.23)

## Gravity Loading Method

In the case of gravity loading method, a platform is constructed over a vertical column resting on the test plate, and the loading is done with the help of sand bags, stones or concrete blocks. The general arrangement of the test set up for this method is shown in Fig 24.22.

When load is applied to the plate, it sinks or settles. The settlement of the plate is measured with the help of sensitive dial gauges. For square plate, two dial gauges are used. The dial gauges are mounted on independently supported datum bar. As the plate settles, the ram of the dial gauges moves down and settlement is recorded. The load is indicated on the load gauge of the hydraulic jack.

Reaction truss method

Fig 24.22 shows the arrangement when the reaction of the jack is borne by a reaction truss. The truss is held to the ground through soil anchors. These anchors are firmly driven in the soil with the help of hammer. The reaction truss is usually made of mild steel reactions. Guy ropes are used for the lateral stability of the truss.

Indian standard code (IS:1888-1982) recommends that the loading of the plate should invariably be borne either by gravity loading platform (Fig 24.22) or by the reaction truss (Fig 24.33). The use of the reaction truss is more popular now-a-days since this is simple quick and less clumsy. No support of loading platform should be located within a distance of 3.5 times the size of the test plate from its centre.

4. Setting of the plate: The test plate shall be placed over a fine sand layer of maximum thickness 5mm, so that the centre of the plate coincides with the centre of the reaction girder/beam with the help of a plumb and bob and horizontally levelled by a sprit level to avoid eccentric loading. The hydraulic jack should be centrally placed over the plate with the loading column in between the jack and the reaction beam so as to transfer the load to the plate. A ball and socket arrangement shall be inserted to keep the direction of the load vertical throughout the test. A minimum seating pressure of 70 g/cm² (0.7 t/m²) shall be applied and removed before starting the load test.

5. Load increments: Apply the load to soil in cumulative equal increments upto 1 kg/cm² (10 t/m²) or one fifth of the estimated ultimate bearing capacity, whichever is less. The load is applied without any impact, fluctuation or eccentricity and in case of hydraulic jack, load is measured over the pressure gauge, attached to the pumping units kept over the pit, away from the testing plate, through extending pressure pipes.

6. Settlement and observations :- Settlement should be observed for each increment of load after an interval of 1, 2.25, 4, 6.25, 9, 16 and 25 minutes and ~~attach~~ thereafter at hourly intervals to the nearest 0.02 mm. In case of clayey soils, the time-settlement curve shall be plotted at each load stage and load shall be increased to next stage either when the rate of settlement gets appreciably reduced to a value of 0.02 mm/min. The next increment of load shall be ~~plotted at each load stage and load shall be increased to next~~ applied and the observations repeated. The test shall be continued till a settlement of 25mm under the normal circumstances or 50mm in special cases such as in dense gravel, gravel and sand mixture is obtained or till failure occurs, whichever is earlier. Alternatively, where settlement does not reach 25mm, the test should be continued to atleast two times the estimated design pressure. If needed, rebound observations may be taken while releasing the load.

7. Load settlement curve and ultimate bearing capacity: A load settlement curve is placed out to arithmetic scale. From this load settlement

curve is placed out to arithmetic scale. From this load settlement curve, zero correction which a given by the intersection of the early straight line or nearly straight line of the curve with zero load line shall be determined and subtracted from the settlement readings to allow for the perfect seating of the bearing plate and other causes. Four typical curves are shown in Fig 24.24. Curve A is typical for loose to medium cohesionless soil. It is straight line in the earlier stages but flattens out at later stage, and there is no clear point of failure. Curve B is for cohesive soil. It may not be quite straight line in the early part and leans toward set

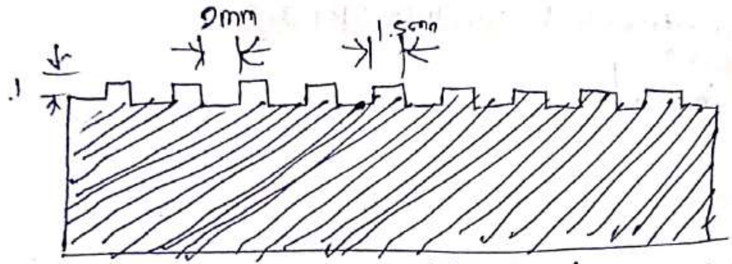
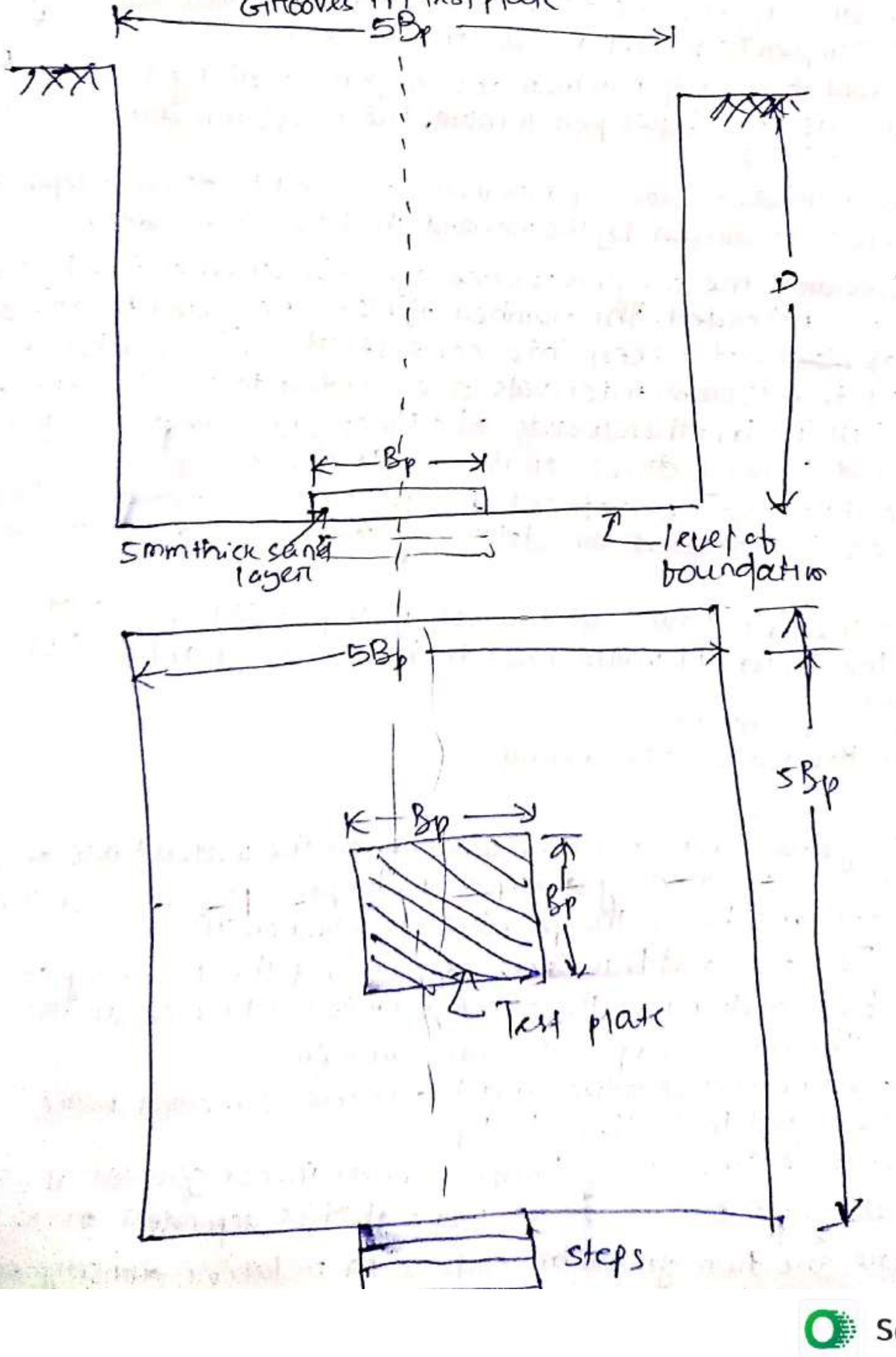



Fig 24.20 Details of cheques or Grooves in test plate



## Standard Penetration Test :-

The standard penetration test in an in-situ test that is commonly under the category of penetrometer tests. The standard penetration tests are carried out in borehole. The test will measure the resistance of the soil strata to the penetration undergone. A penetration emphirical correlation is derived between the soil properties and the penetration resistance.

The test is extremely useful for determining the relative density and the angle of shearing resistance of cohesionless soils. It can also be used to determine the unconfined compressive strength of cohesive soil.

### Tool for standard penetration test :-

The requirements to conduct SPT are:

1. Standard split spoon sampler
2. Drop Hammer weighing 63.5 kg.
3. Guiding rod
4. Drilling Rig
5. Driving head (anvil)

### Procedure:

The test is conducted in a bore hole by means of a standard split spoon sampler. Once the drilling is done to the desired depth, the drilling tool is removed and the sampler is placed inside the bore hole.

By means of a drop hammer of 63.5 kg mass falling through a height of 750mm at the rate of 30 blows per minute, the sampler is driven into the soil. This is as per IS-2131: 1963.

The number of blows of hammer required to drive a depth of 150mm is counted. Further it is driven by 150 mm and the blows are counted.

Similarly, the sampler is once again further driven by 150mm and the number of blows recorded. The number of blows recorded for the first 150mm ~~and the number of blows~~ not taken into consideration. The number of blows recorded for last two 150mm intervals are added to give the standard penetration number (N). In otherwords. N = No. of blows required for 150mm penetration beyond seating drive of 150mm. If the number of blows for 150mm drive exceeds 50, it is taken as refusal and the test is discontinued. The standard penetration number is corrected for dilatancy correction and overburden correction.

Before the SPT values are used in empirical correlations and in design charts, the field 'N' value have to be corrected as per IS 2131-1981. The corrections are:

1. Dilatancy Correction
2. Overburden Pressure Correction

### 1. Dilatancy Correction

Silty fine sands and fine sands below the water table develop pore water pressure which is not easily dissipated. The pore pressure increases the resistance of the soil and hence the penetration number (N).

Terzaghi and Peak (1967) recommend the following correction in the case of silty fine sands when the observed value is N exceeds 15.

The corrected penetration number

Where $N_R$ is the recorded value and $N_C$ is the corrected value.

If $N_R$ less than or equal to 15, then $N_C = N_R$

**2. Overburden Pressure Correction:** From several investigations, it is proved that the penetration resistance or the value of N is dependent on the overburden pressure. If there are two granular soils with relative density same, higher N value will be shown by the soil with higher confining pressure.

With the increase in the depth of the soil, the confining pressure also increases. So the value of 'N' at shallow depth and larger depths are underestimated and overestimated respectively.

Hence, to account this the value of 'N' obtained from the tests are corrected to a standard effective overburden pressure.

The corrected value of 'N' is

$$N_c = C_N N$$

Here $C_N$ is the correction factor for the overburden pressure.

## Precautions taken for Standard Penetration Test

-> Split spoon sampler must be in good condition.

-> The cutting shoe must be free from wear and tear.

-> The height of fall must be 750mm. Any change from this will affect the 'N' value.

-> The drill rods used must be in standard condition. Bent drill rods are not used.

-> Before conducting the test, the bottom of the borehole must be cleaned.

## Advantage of SPT :-

The advantages of standard penetration test are:

-> The test is simple and economical

-> The test provides representative samples for visual inspection, classification tests and for moisture content.

-> Actual soil behaviour is obtained through SPT values

-> The method helps to penetrate dense layers and fills

-> Test can be applied for variety of soil conditions

## Disadvantages of SPT :-

The limitations of SPT are:

-> The results will vary due to any mechanical or operator variability or drilling disturbances.

-> Test is costly and time consuming.

-> The samples retrieved for testing is disturbed

-> The test results from SPT cannot be reproduced.

-> The application of SPT in gravels, cobbles and cohesive soils are limited.