Module - 1

Geotechnical Engineering - I

Introduction

History of Soil Mechanics

In ancient times and in the Roman period the use of soil was appreciated in construction of roads, canals & bridges.
In middle ages European engineers understood the problem related with settlement, then use of timber piles to support structure on soft clay was done. During Renaissance Leonardo da Vinci gave some of the innovative ideas on retaining wall, road and canal construction.

In 18th century some of theories pertaining to design of foundation were made. coulomb formulated his classical theory on Earth pressure. He also introduced the concept of frictional resistance & cohesive resistance.

Darcys law for permeability of soil and stroke's law of velocity were developed in 1856.
Earth pressure theories by Rankine's and Culmann's rupture theory on stresses by Mohrs stress distribution concept by boussinesq's and Westergaards has been extended the soil Mechanics in 1885.

In 20th century development of soil by Atterberg defining consistency limit. "Terzaghi's contribution to the soil mechanics by introducing principle of effective stress and one dimensional consolidation theory which makes him to consider as Father of Soil Mechanics.

The other contributors to the growth of soil mechanics include proctors compaction concepts Taylor pecks, Skempton, Bjerrum and casagrande.

* Definition of Soil

The used soil has different meanings for different professions.

To an agriculturist soil is the top thin layer of the earth within which organic forces are predominant and which supports plants life.

To an geologist soil is the material in the top thin zone within which roots occur.

To an engineer, Soil includes all Earth materials organic & inorganic occurring in the zone overlying the rock crust.

Terzaghi defined Soil Mechanic as

Soil Mechanics is the application of the laws of mechanics and hydraulics to engineering problem dealing with sediments and other unconsolidated accumulation of solid particles produced by mechani and chemical disintegration of rocks regardless of weather or not they contain any admixture of organic constituents.

Geo technical Engg includes soil mechanics, rock mechanics, soil engg and rock engg.

Origin & Formation of Soil

Soil is defined as a natural aggregate of mineral grains with or without organic constituents that can be seperated by agitation of water.
The soils are formed by the process of weathering or disintegration of the parent rock.

Formation of soils are grouped into two :-

1. Physical Disintegration
2. chemical Decomposition

1. Physical Disintegration

Physical disintegration or mechanical weathering of rocks occur due to the following physical processes.

a) Temperature changes :- Different minerals of rocks have different co-efficient of thermal expansion, unequal expansion and contraction of these minerals occur due to temperature changes. When the stresses induced due to such changes are repeated many times, the particle gets detached from the rocks and the soil is formed.

b) Wedging action of Ice :- Water in the pores and minute cracks of rocks get frozen in very cold climates. As the volume of ice formed is more than that of water, expansion occurs. Rocks get broken into pieces when large stresses develop in the cracks due to wedging action of the ice formed.

c. Spreading of roots of plants:-

As the roots of trees & shrubs grow in the cracks and fissures of the rocks, forces act on the rock. The segment of the rock are forced apart and disintegration of rocks occur.

d. Abrasion:-

As water, wind & glaciers move over the surface of rock, abrasion and scouring takes place. It result in the formation of soil.

Soil which is a product of physical weathering retain the minerals that were present in the parent rock and are coarse grained soil.

2) Chemical Decomposition:-

chemical decomposition or chemical weathering of rocks takes place when original rock minerals are transformed into new mineral by chemical reactions. The soils formed do not have the properties of the parent rock.

The following are the chemical processes generally occuring in nature.

a) Hydration:- In hydration, water combines with the rock minerals and results in the formation of a new chemical compound. The chemical reaction causes a change in volume and decomposition of rock into small particles.

b) Carbonation:- It is a type of chemical decomposition in which carbon dioxide in the atmosphere combines with water to form carbonic acid. The carbonic acid reacts chemically with rocks and causes their decomposition.

c) oxidation :-

Oxidation occurs when oxygen ions combines with minerals in rock. Oxidation results in decomposition of rocks. Oxidation of rocks is somewhat similar to rusting of steel.

d) Solution :-

Some of the rock minerals form solution with water when they get dissolved in water. Chemical reaction takes place in solution and soil is formed.

e) Hydrolysis :-

It is a chemical process in which the water gets dissociated into H^+ and OH^- ions. The Hydrogen cations replace the metallic ions such as calcium and potassium in rock minerals and soil is formed with a new chemical decomposition.

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If the products of rock weathering are still located at the place where they originated, they are called Residual soil.

Any soil that has been transported from its place of origin by the effects of wind, water ice or any other agency and has redeposited at some other place is called a transported soil.

Residual soils are not as common as transported soils.

Transported soils are further classified according to the transporting agency and method of deposition.

a) Alluvial deposit :- Soils that have been deposited from suspension in running water.

b) Lacustrine deposit :- Soils that have been deposited from suspension in still, fresh water of lakes.

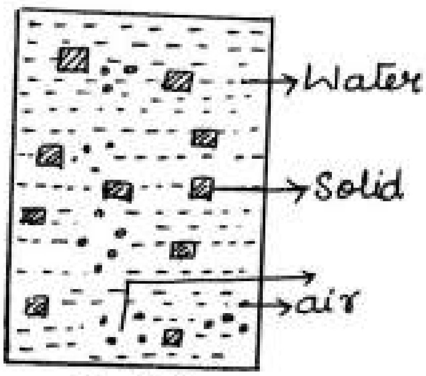
c) Marine deposit :- Soils that have been deposited from suspension in sea water.

d) Aeolian deposit :- Soils that have been ~~deposited~~ transported by wind.

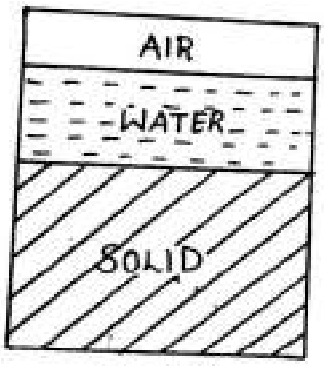
e) Glacial deposit :- Soils that have been transported by ice.

* Soil mass, a three phase system:-

Soil mass comprises of Solid particles and void space. The Voids in the soil mass is filled with air, with water or partly with air and partly with water.
The three constituents of soil mass are blended together to form a complex material as shown in this fig(a). However for convenience, all the solid particles are segregated & placed in the lower layer of the three phase diagram as shown in the fig(b)
The 3-phase diagram is also known as block diagram.



fig(a)

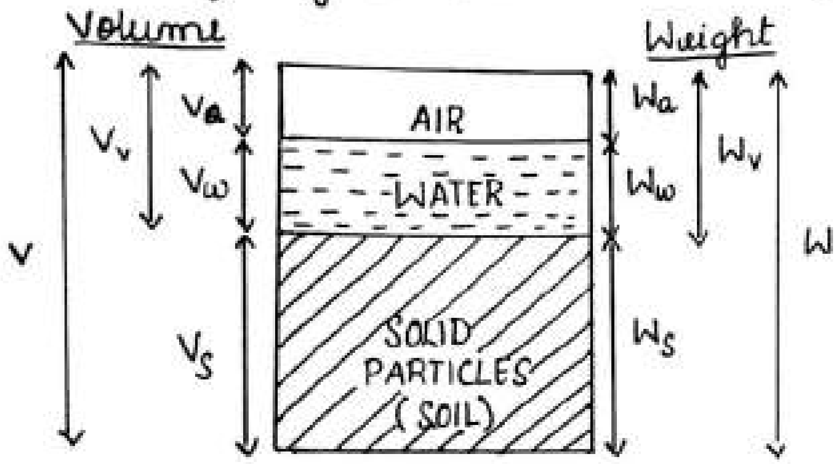


fig(b)

- When the soil voids are completely filled with water, the gaseous phase being absent, it is said to be fully saturated or saturated.
- When there is no water in the voids, the voids will be full of air, the liquid phase being absent, the soil is said to be dry.

In a three phase diagram, it is conventional to write Volumes on the left side and weight on the right side as shown in the fig.

The Volume of a given soil mass is designated as 'V.'



V_a = Volume of air

V_w = Volume of Water

V_v = Volume of voids

V_s = Volume of Solids

V = Total Volume of Soil mass

$V_v = V_a + V_w$

W_a = Weight of Air

W_w = Weight of Water

W_v = Weight of Void material occupying void spaces.

W = Total weight of Soil mass.

$W_v = W_a + W_w$

* Definitions:-

1. Water content: It is defined as the ratio of weight of water (W_w) to the weight of Solids (W_s or W_d) in a given mass of soil.

$$w = \frac{W_w}{W_d} \times 100 \, (\%)$$

- It is also called moisture content. It is expressed in percentage.
- It indicates the amount of water present in the voids in comparision with weight of solids.
- Water content of soil mass changes with season. being close to zero in summer & maximum during raining season.

2. Void Ratio:- (e) - It is defined as the ratio of volume of voids to volume of solids in the given soil mass. Thus, $e = \frac{V_v}{V_s}$

- It indicates the amount of voids present in the soil mass in comparision with the amount of solids.
- The more the void ratio, more loose will be the soil mass & hence, less strong & less stiff.
- It is not possible to determine void ratio in the laboratory. Hence it is computed from other properties.

3. **Density** :- The density of soil is defined as the mass of the soil per unit volume.

4. **Bulk Density (ρ)** :-

It is defined as the ratio of total mass of the soil per unit of its total volume.

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

- It is expressed in terms of kN/m^3, g/cm^3 or kg/m^3
- It includes the weight of air, water & solids as a function of total volume.

5. **Dry density (ρ_d)** :-

It is defined as the ratio of mass of soil solids per unit of total volume of the soil mass.

$$\rho_d = \frac{M_d}{V}$$

- Dry density will always be less than or equal to bulk density of soil mass.

6. **Density of Soil Solids (ρ_s)** :-

It is defined as the ratio of mass of soil solids per unit of volume of solids.

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

- It is always greater than dry density of soil.

7. Saturated Density (ρ_{sat}):

It is defined as the ratio of total mass of saturated sample to its total volume.

8. Density of water:-

It is defined as the ratio of mass of water to Volume of water.

- Can be taken as 9.8 kN/m³.

9. Submerged density (ρ'):-

It is defined as the ratio of submerged mass of soil solids per unit of total volume 'V' of the soil mass.

- It is also called buoyant density.
- The submerged density is expressed as

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

10. Unit weight of soil mass:- is defined as its weight per unit volume.

11. Bulk unit weight:-

The bulk unit weight or moist unit weight is the total Weight 'W' of a soil mass per unit of its total Volume 'V'.

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

12. Dry unit weight (γ_d):- is the weight of solids per unit of its total Volume 'V'.

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

13. **Unit weight of Solids:-**

The unit weight of Solids is the weight of soil solids per unit Volume of Solids.

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

14. **Saturated unit weight (γ_{sat}) :-**

When the soil mass is saturated, its bulk unit wt is called saturated unit weight.

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

15. **Submerged unit weight (γ')**

It is the submerged weight of soil solids (W_d) Per unit of total volume.

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

The Submerged unit weight is therefore, equal to the weight of soil solids in air minus the weight of water displaced by solids.

Submerged unit weight is expressed as

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

16. **Specific Gravity :-**

Specific Gravity is defined as the ratio of weight of a given Volume of soil solids at a given temperature to the weight of water of an equal volume at that temperature.

In other words, it is the ratio of unit wt of soil solids to that of water.

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

Appearant | mass/ bulk Specific Gravity (G_m) is given by

$$G_m = \frac{\gamma}{\gamma_w}$$

17. **Porosity :-**

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

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

18. **Degree of Saturation :-**

The Degree of saturation is defined as the ratio of the volume of water present in a given soil mass to the total volume of voids in it.

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

- It represents the amount of water present in the void space of soil mass.

- In dry soil, S=0 and in fully saturated soil S=100%.

19] <u>Air Content</u> :- (a_c)
It is defined as the ratio of volume of air voids to the volume of voids.

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

20. <u>Percentage of air voids (η_a)</u>:-
It is defined as the ratio of the volume of air voids to the total volume of the soil mass.

Thus $\eta_a = \frac{V_a}{V} \times 100 (\%)$

21. <u>Density Index</u> or <u>Relative Density</u> (I_D)
The density index is defined as the ratio of the difference b/w the void ratio of the soil in its loosest state (e_{max}) and its natural void ratio (e). to the difference between the void ratio in the loosest and densest state.

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

where,
e_{max} = Void ratio in the loosest sta
e_{min} = " " " " densest sta
e = Void ratio in the deposit

In terms of dry density, relative density is given as follows.

$$I_D = \frac{\gamma_{d\,max}}{\gamma_d}\left[\frac{\gamma_d - \gamma_{min}}{\gamma_{d\,max} - \gamma_{d\,min}}\right]$$

where, $\gamma_{d\,max}$ = dry density corresponding to most compact state

$\gamma_{d\,min}$ = dry density corresponding to most loosest state

γ_d = dry density.

* Interrelations:-

1. Relationship b/w e and n:-

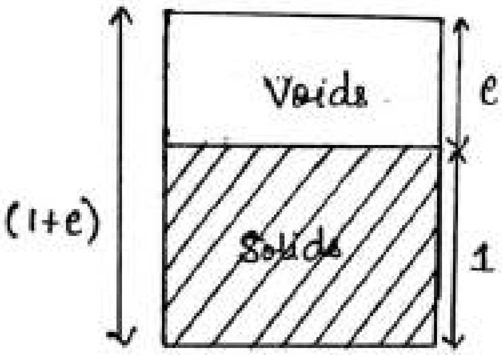


Fig (a)

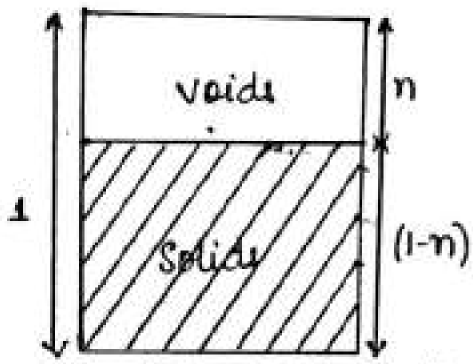


fig. Soil element in terms of n

Fig (a) shows the soil element in terms of void ratio e. If the volume of voids is taken equal to e. & Volume of solids is taken as 1.

∴ The total volume = (1+e)

Similarly from fig (b) - If the volume of void is taken equal to n & the total volume of element is taken as 1

∴ hence, the volume of Solids = (1-n)

From the definition of porosity

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

From the definition of void ratio

$$e = \frac{V_v}{V_s} = \frac{n}{1-n} \quad — ②$$

combining ① & ②

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

$$\boxed{(1-n) = \frac{1}{1+e}}$$ - is the relation b/w e & n.

2 Relation between e, G, w & S_r / with usual notation, Prove $\boxed{S_r e = wG}$

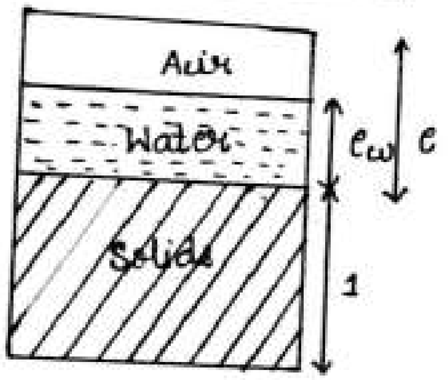


Fig. Soil element in terms of e.

e_w = Represents volume of water
e = volume of voids
volume of Solids = 1

Degree of Saturation $S_r = \frac{V_w}{V_v} = \frac{e_w}{e}$

$$e_w = S_r e \quad — ①$$

For a fully saturated sample, $e_\omega = e$

Now water content $(\omega) = \frac{W_\omega}{W_d} = \frac{e_\omega . \gamma_\omega}{1 . \gamma_s}$ $\left[\text{But } G = \frac{\gamma_s}{\gamma_\omega} \right]$

or $\gamma_s = G\gamma_\omega$

$$\omega = \frac{e_\omega . \gamma_\omega}{G\gamma_\omega} = \frac{e_\omega}{G}$$

$$e_\omega = G\omega \quad \text{—— (2)}$$

Equating 1 & 2

$$\boxed{S_r e = G\omega}$$

For fully saturated soil $S_r = 1$, $\omega = \omega_{sat}$

$$\boxed{e = \omega_{sat} G}$$

3. <u>Relation between γ, γ_d & ω</u> / Prove $\boxed{\gamma_d = \frac{\gamma}{1+\omega}}$

we have, $\omega = \frac{W_\omega}{W_d}$

Adding 1 on both sides

$$1 + \omega = \frac{W_\omega}{W_d} + 1$$

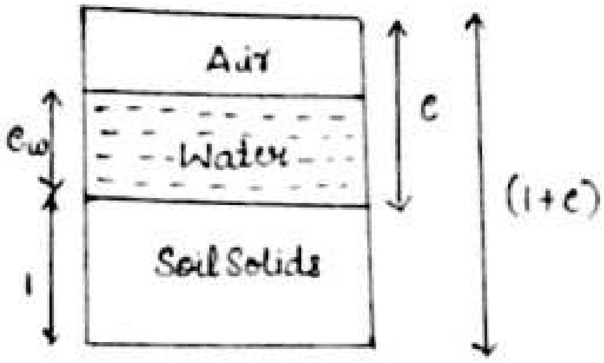
$$1 + \omega = \frac{W_\omega + W_d}{W_d}$$

$$1 + \omega = \frac{W}{W_d} \Rightarrow W_d = \frac{W}{1+\omega}$$

Dividing both the sides by V

$$\frac{W_d}{V} = \frac{W/V}{1+\omega} \Rightarrow \boxed{\gamma_d = \frac{\gamma}{1+\omega}}$$

4 Expression for $\gamma, \gamma_d, \gamma_{sat}$ & γ'



From the fig we have

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V}$$

$$\gamma = \frac{V_s\gamma_s + V_w\gamma_w}{V} = \frac{1.\gamma_s + V_w\gamma_w}{V}$$

$$\gamma = \frac{G\gamma_w + eS_r\gamma_w}{V} = \frac{(G + eS_r)\gamma_w}{V}$$

$$\gamma = \frac{(G+eS_r)\gamma_w}{1+e} \quad \text{—— (1)}$$

For dry soil mass, $\gamma = \gamma_d$ and $S_r = 0$

$$\boxed{\gamma = \frac{G\gamma_w}{1+e}} \quad \text{—— (2)}$$

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

Sub in equa (1)

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

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

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

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

5. Relationship b/w dry unit weight (γ_d), Specific Gravity (G), Void ratio (e), & porosity (n)

w.r.t, $\gamma_d = \frac{W_d}{V}$

w.r.s $\gamma_s = \frac{W_d}{V_s} \Rightarrow W_d = \gamma_s V_s$

$\therefore \gamma_d = \frac{\gamma_s V_s}{V}$ $\left[\begin{matrix} V_s = 1 \\ V = 1+e \end{matrix} \right]$

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

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

Also. $\gamma_d = \frac{\gamma_s V_s}{V}$ [Fig b]

$V_s = 1-n$ & $V = 1$

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

$\gamma_d = \gamma_s(1-n)$

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

6. Relationship b/w γ_{sat}, G, e & n

w.r.t $\gamma_{sat} = \frac{W_{sat}}{V}$

where $W_{sat} = W_w + W_d$

$= \frac{W_w + W_d}{V} = \frac{\gamma_w V_w + \gamma_s V_s}{V}$

From Fig (a) - For fully saturated soil, $e_w = e$

i] In terms of Void ratio $\rightarrow V_w = e$ & $V_s = 1$, $V = 1+e$

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

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

From fig (b)

ii] In terms of Porosity $\rightarrow V_w = n$, $V_s = (1-n)$, $V = 1$

$$\gamma_{sat} = \frac{\gamma_w n + \gamma_s(1-n)}{1} = \gamma_w n + G\gamma_w(1-n)$$

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

7. Relationship between bulk unit weight (γ), Specific Gravity (G), Void ratio (e) & degree of saturation (S_r)

w.r.t $\gamma = \frac{W}{V}$

$W = W_w + W_d$

$= \gamma_w V_w + \gamma_s V_s$

$\gamma = \frac{\gamma_w V_w + \gamma_s V_s}{V}$ $\left[V_s = 1, V_w = e_w \quad V = 1+e \right]$

$\gamma = \frac{\gamma_w e_w + \gamma_s 1}{1+e}$

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

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

8. Relationship b/w void ratio (e), degree of saturation (S_r) & percentage of air void (n_a)

we have, $n_a = \frac{V_a}{V} = \frac{V_v - V_w}{V} = \frac{V_v - V_w}{V_a + V_w + V_s}$

$$= \frac{V_v - V_w}{V_v + V_s} = \frac{V_v - V_w}{1+e}$$

$$= \frac{e - eS_r}{1+e}$$

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

9. Relationship b/w percentage air void (n_a), air content (a_c) & porosity (n):-

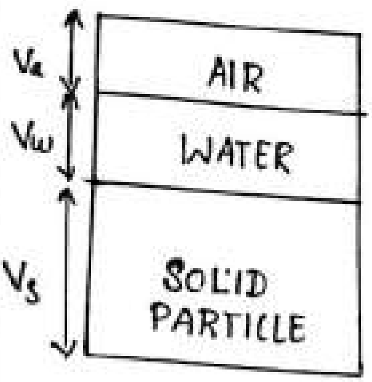
we have,

$$a_c = 1 - S_r$$

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

$$\boxed{n_a = a_c \times n}$$

10] Relationship b/w Dry unit weight (γ_d), Specific gravity (G), water content (w), degree of saturation (S_r) and percentage air void



From the fig.
in terms of void ratio

$$V = V_a + V_w + V_s$$

Here, $V_w = \frac{W_w}{\gamma_w}$ & $V_s = \frac{W_d}{\gamma_s}$

$$V = V_a + \frac{W_w}{\gamma_w} + \frac{W_d}{\gamma_s}$$ – Dividing both the sides by V.

$$1 = \frac{V_a}{V} + \frac{W_w}{V\gamma_w} + \frac{W_d}{V.\gamma_s}$$

$$1 = n_a + \frac{w.W_d}{V.\gamma_w} + \frac{W_d}{V.\gamma_s} \quad \left[w = \frac{W_w}{W_d} \right]$$

$$1 = n_a + \frac{\gamma_d w}{\gamma_w} + \frac{\gamma_d}{\gamma_s} = n_a + \frac{\gamma_d w}{\gamma_w} + \frac{\gamma_d}{G\gamma_w}$$

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

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

$$\boxed{\gamma_d = \frac{G\gamma_w(1-n_a)}{S_r e + 1}}$$

Relationship b/w submerged unit weight (γ'), dry unit weight (γ_d) and porosity

Fig in terms of e.

w.r.t

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

$$(W_d)_{sub} = W_d - W_w$$

$$= \gamma_s . V_s - \gamma_w V_w$$

Volume of the soil displaced by soil sample = volume of soil solids

$$V_s = V_w$$

$$(W_d)_{sub} = \gamma_s V_s - \gamma_w V_s$$

$$V_s = 1$$

$$(W_d)_{sub} = G\gamma_w - \gamma_w = \gamma_w (G-1)$$

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

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

$$\gamma' = \gamma_d - (1-n)\gamma_w \qquad (1-n) = \frac{1}{1+e}$$

$$\boxed{\gamma' = \gamma_d - (1-n)\gamma_w}$$

Module-1

Problems:-

* Some of the important relationship which are derived are listed below:-

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

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

3. unit weight, $\gamma = \frac{(G + S_r e)\gamma_w}{1+e}$

4. Dry unit weight, $\gamma_d = \frac{G\gamma_w}{1+e}$

5. Saturated unit weight, $\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e}$

6. Submerged unit weight, $\gamma' = \frac{(G-1)\gamma_w}{1+e}$

7. $e = \frac{wG}{S_r}$

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

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

10. $G_r = \frac{\gamma_s}{\gamma_w}$

11. $\gamma = W/V$ where $W = W_w + W_s$

12] $G_m = \frac{\gamma}{\gamma_w}$

where G_m = Bulk specific gravity

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

* **Problems:-**

1. A Soil Sample weighs $16.5 KN/m^3$ as the water content of 28%, the specific Gravity of Soil particle is 2.7. Determine the dry unit wt, Void ratio, porosity & degree of saturation.

Solu: Bulk unit weight of soil = $16.5 KN/m^3$

Water content (w) = 28%

$G = 2.7$

i] Dry unit wt. $\gamma_d = \frac{\gamma}{1+w} = \frac{16.5}{1+0.28} = 12.89 KN/m^3$

$\boxed{\gamma_d = 12.89 KN/m^3}$

ii] Void ratio (e) = $\frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{12.89} - 1$

$\boxed{e = 1.05}$

iii] porosity (n)

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

$\frac{12.89}{9.81 \times 2.7} = (1-n)$

$(1-n) = 0.48$

$n = 0.513$

$\boxed{n = 51.33\%}$

iv] Degree of Saturation (S_r) = $\frac{wG}{e} = \frac{0.28 \times 2.7}{1.05} \times 100$

$\boxed{S_r = 72\%}$

2. A Sample of soil weighing 1.36N, the following data was obtained from the laboratory test on the

unit wt = 26 KN/m³

Specific Gravity = 2.67, w = 16%.

Determine the dry unit wt, void ratio, porosity & degree of saturation.

What is the unit wt, if the soil is fully saturated.

Solu: Wt of soil sample(W_s) = 1.36N

(γ) unit wt = 26 KN/m³

$G = 2.67$, $w = 16\%$.

i] Dry unit wt = $(\gamma_d) = \frac{\gamma}{1+w} = \frac{26}{1+0.16} = \underline{22.41 \, KN/m^3}$.

ii] void ratio(e) = $\frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.67 \times 9.81}{22.41} - 1 = 0.168$

$\boxed{e = 0.168}$

iii) Porosity (n) = $\frac{e}{1+e} = \frac{0.168}{1+0.168} = \underline{14.38\%}$

iv) Degree of Saturation (S_r) = $\frac{wG}{e} = \frac{0.16 \times 2.67}{0.168}$

$S_r = 254.28\%$

If the Soil is fully saturated

$$\gamma = \frac{(G + S_r e)}{1+e}\gamma_w = \frac{(2.67 + 1(0.168))}{1+0.168} \times 9.81$$

$\gamma = \underline{23.83 \, KN/m^3}$

A Soil Sample has a porosity of 40%, $G = 2.7$. Calculate i) Void ratio ii) Dry unit weight iii) Unit weight of the soil if degree of Saturation is 50% iv) Unit weight of the soil if the soil is completely saturated.

Solution:- $n = 0.4$, $G = 2.7$

i] Void Ratio (e) $= \frac{n}{1-n} = 0.667$

ii. Dry unit weight (γ_d) $= G\gamma_w(1-n)$

$= 2.7(9.81)(1-0.4)$

$\boxed{\gamma_d = 15.89 KN/m^3}$

iii] $S_r = 0.5$

$$\gamma = \frac{\gamma_w(eS_r + G)}{1+e} = \frac{9.81(0.667(0.5)+2.7)}{1+0.667}$$

$\boxed{\gamma = 17.85 KN/m^3}$

iv) $S_r = 1$

$$\gamma = \frac{9.81(0.667(1)+2.7)}{1+0.667} = 19.81 KN/m^3$$

$\boxed{\gamma = 19.81 KN/m^3}$

4. A Soil having bulk unit weight of $20.1 KN/m^3$ and water content of 15%. Calculate

i] the water content if the soil particle is dry to a unit weight of $19.4 KN/m^3$ and void ratio remains unchanged.

Solution:- $\gamma_b = 20.1 KN/m^3$, $\gamma_d = 19.4 kN/m^3$

$w = 15\%$

w.r.t

$$\gamma_d = \frac{\gamma}{1+w} = \frac{20.1}{1+0.15} = 17.47 \, KN/m^3$$

Since after drying. Void ratio doesn't change we have

$$\gamma_d = \frac{\gamma}{1+w} \Rightarrow 1+w = \frac{19.4}{17.48}$$

$$\boxed{w = 11\%}$$

5. A dry soil has a void ratio(e) = 0.65 and its specific gravity (G) = 2.8. What is its unit weight.
ii] Water is added to the sample so that its degree of saturation is 60% without any change in the void ratio. Determine the water content & unit weight.
iii] The Sample is next placed below water. determine the true unit weight, if the degree of saturation Sr is 95% and 100% respectively. (not considering the buoyancy)

Solution:- $e = 0.65$, $G = 2.8$

i] $$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.8(9.81)}{1+0.65} = 16.64 \, KN/m^3$$

$$\boxed{\gamma_d = 16.64 \, KN/m^3}$$

ii] $S_r = 0.6$, $e = 0.65$

$$S_r = \frac{wG}{e} \Rightarrow w = \frac{S_r . e}{G} = \frac{0.6 \times 0.65}{2.8} = 0.139$$

$$\boxed{w = 13.92\%}$$

$$\gamma_d = \frac{\gamma}{1+w} \Rightarrow \gamma = \gamma_d(1+w)$$

$$= 16.64(1+0.1392)$$

$$\boxed{\gamma = 18.96 \, KN/m^3}$$

iii] Sample below water, S = 95%

$$\gamma = \frac{(G+Se)}{(1+e)}\gamma_w = \frac{2.80+0.95(0.65)}{(1+0.65)} \times 9.81$$

$$\boxed{\gamma = 20.32 KN/m^3}$$

$S_r = 100\%$

$$\gamma = \frac{(2.80+1(0.65))}{1+0.65} \times 9.81 = 20.51 KN/m^3$$

$$\boxed{\gamma = 20.51 KN/m^3}$$

6) A Sample of clay volume $1\times10^{-3} m^3$ and weighing 0.0176 KN. After drying in the oven it has wt of 0.01368 KN, if G = 2.69. Find the following from the available data.

a) Water content b) void ratio c) Air voids

d) Saturated unit wt & dry unit wt.

Solu:

$V = 1\times10^{-3} m^3$

Weight of clay $(W_s + W_w) = 0.0176 KN (W)$

Weight of drying $(W_s) = 0.01368 N$

$G = 2.69$

a) $$w = \frac{W_w}{W_d} = \frac{W - W_d}{W_d} \times 100 = \frac{0.0176-0.01368}{0.01368} = 28.65\%$$

$$\boxed{w = 28.65\%}$$

b) $$\gamma = \frac{W}{V} = \frac{0.0176}{1\times10^{-3}} = \underline{17.6 KN/m^3}$$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{17.6}{1+0.288} = 13.68\ kN/m^3$$

$$\boxed{\gamma_d = 13.68\ kN/m^3}$$

$$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.69 \times 9.81}{13.68} - 1 = 0.92$$

$$\boxed{e = 0.92}$$

$$\gamma_{sat} = \frac{(G+e)}{(1+e)}\gamma_w = \left(\frac{2.69 + 0.92}{1+0.92}\right) \times 9.81$$

$$\boxed{\gamma_{sat} = 18.44\ kN/m^3}$$

iii] Air voids percentage (n_a)

$$S_r = \frac{wG}{e} \times 100 = \frac{0.286 \times 2.69}{0.92} \times 100$$

$$S_r = 83.62\%$$

$$a_c = (1 - S_r) = (1 - 0.836) = 0.164$$

$$a_c = 0.164$$

$$n_a = \frac{e(1-S_r)}{(1+e)} = \frac{0.92(1-0.8362)}{1+0.92} = 7.84\%$$

$$\boxed{n_a = 7.84\%}$$

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A Sample of soil has volume of 1000cc & weight of 17.5N, G = 2.52, if dry unit wt is 15.8 KN/m³ Determine the water content, Void ratio, Submerged unit weight & degree of saturation.

Solution:- $V = 1000 =$

$W = 17.5N = 17.5 \times 10^{-3} kN$

$G = 2.52$

$\gamma_d = 15.8 \, KN/m^3$

i] $\gamma = \frac{W}{V} = \frac{17.5 \times 10^{-3}}{1000 \times 10^{-6}} = 17.5 KN/m^3$

Water content, (w)

$$\gamma_d = \frac{\gamma}{1+w} \Rightarrow 15.8 = \frac{17.5}{1+w}$$

$$\boxed{w = 10.75\%}$$

Void ratio (e)

$$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.52(9.81)}{15.8} - 1 = 0.56$$

$$\boxed{e = 0.56}$$

Saturated unit weight

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w = \left(\frac{2.52+0.56}{1+0.56}\right) \times 9.81$$

$$\gamma_{sat} = 19.36 \, KN/m^3$$

Submerged unit wt $(\gamma') = \gamma_{sat} - \gamma_w$

$$= 19.36 - 9.81 = 9.55 KN/m^3$$

$$\boxed{\gamma' = 9.55 KN/m^3}$$

Degree of Saturation, $S_r = \frac{wG}{e} = \frac{0.1075 \times 2.52}{0.56}$

$$\boxed{S_r = 48.37\%}$$

7. A natural soil has a bulk unit wt of 18 KN/m³ & water content of 8%. calculate the amount of water required to be added in 1m³ of soil to raise the water content to 18%. What will be the degree of saturation at this water content. Assume void ratio remains constant. Take $G_s = 2.7$

Solution:- $\gamma = 18 KN/m^3$, $\omega = 8\%$

$$\gamma_d = \frac{\gamma}{1+\omega} = \frac{18}{1+0.08} = 16.67 KN/m^3$$

$$\boxed{\gamma_d = 16.67 KN/m^3}$$

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

$W_d = 16.67 KN$

For 8% water content $= \frac{W_w}{W_d}$

$W_\omega = 1.33 KN$

For 18% water content $= \frac{W_w}{W_d}$

$W_\omega = 3 KN$

Thus, amount of water required to be added $= 3 - 1.33 = 1.67 KN$

$$\boxed{\omega = 1.67 KN}$$

iii] Degree of saturation at 18% water content

$$S_r = \frac{\omega G}{e} = \frac{0.18 \times 2.7}{0.588}$$

$$\boxed{S_r = 82.65\%}$$

$$e = \frac{G\gamma_\omega}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{16.67} - 1$$

$$\boxed{e = 0.588}$$

A dry soil has a void ratio of 0.65 and specific gravity is 2.8. Find its unit weight.
Water is added to the sample so that its degree of saturation is 55% without any change in void ratio. Determine the unit weight when the degree of saturation is 90% and 100%.

Solution: $e = 0.65$, $G = 2.8$

$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.8 \times 9.8}{1+0.65} = \underline{16.63 \text{ KN/m}^3}$$

When $S = 55\%$

$$S_r = \frac{wG}{e} \Rightarrow w = \frac{S_r . e}{G} = \frac{0.55 \times 0.65}{2.8} = 12.76\%$$

$w = 12.76\%$, $\underline{\gamma_b = 18.75 \text{ KN/m}^3}$

When $S_r = 90\%$

$$S_r = \frac{wG}{e} \Rightarrow w = 20.89\%$$

$$\gamma_b = \gamma_d(1+w) = 16.63(1+0.2089)$$

$$\boxed{\gamma_b = 20.10 \text{ KN/m}^3}$$

When $S_r = 100\%$

$w = 23.21\%$

$$\gamma_b = \gamma_d(1+w) = 16.63(1+0.2321)$$

$$\boxed{\gamma_b = 20.48 \text{ KN/m}^3}$$

9. In an earth dam under construction, the bulk unit weight is 16.5 KN/m³ at water content 11%. If the water content has to be increased to 15%, compute the quantity of water to be added per cu.m of the soil. Assume no change in void ratio. Determine the degree of saturation at this water content. Take G = 2.7

Solution:- $\gamma_b = 16.5\ KN/m^3$

$w = 11\%$

$G = 2.7$

$$\gamma_d = \frac{\gamma_b}{1+w} = \frac{16.5}{1+0.11} = \underline{14.86\ KN/m^3}$$

$W_d = \underline{14.86\ KN}$ per unit volume

For water content 11%

$W_d = \underline{14.86\ KN}$

$W_w = 0.11 \times 14.86 = 1.63\ KN$

$$V_w = \frac{W_w}{\gamma_w} = \frac{1.63}{9.81} = \underline{0.166 m^3}$$

For water content 15%

$W_d = 14.86 KN$

$W_w = 0.15 \times 14.86 = 2.22\ KN$

$$V_w = \frac{2.22}{9.81} = \underline{0.227 m^3}$$

Hence water required to raise the water content from 11% to 15% = 0.227 − 0.166 = 0.061 m³ ≈ 61 litres

$$\text{Void ratio, } e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{14.86} - 1 = \underline{0.78}$$

$$S_r = \frac{wG}{e} = \frac{0.15 \times 2.7}{0.78} = \underline{51.92\%}$$

14] An undisturbed specimen of clay was tested in the laboratory and the following results were obtained Weight = 2.1N, oven dry weight = 1.75N. Specific gravity of Soil solids = 2.7. What was the total volume of original undisturbed specimen assuming that the specimen was 50% saturated.

Solution:- $W = 2.1N$

$W_s = 1.75N$

$G = 2.7$, $S_r = 50\%$

$W = W_w + W_s =$

$W_w = 2.1 - 1.75 = 0.35N$ $\qquad w = \frac{W_w}{W_s} = 20\%$

$$e = \frac{wG}{S_r} = \frac{0.2 \times 2.7}{0.5} = 1.08$$

$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.7 \times 9.81}{1+ .108} = 12.73 kN/m^3$$

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

$$V = 0.1376 \times 10^{-3} m^3$$

15] A Sample of Sand above water table was found to have a natural moisture content of 15% and a unit weight of 18.84 KN/m³. Laboratory tests on a dried sample indicated values of 0.5 & 0.85 for minimum & maximum void ratio respectively for densest and loosest state. Calculate the degree of saturation and the Relative density. Assume G = 2.65

Solu:

$w = 15\%$

$\gamma = 18.84\ KN/m^2$

$e_{min} = 0.5$

$e_{max} = 0.85$

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

i] $$e = \frac{2.65 \times 9.81 \times (1 + 0.15)}{18.84} - 1$$

$$\boxed{e = 0.58}$$

ii] $$S_r = \frac{wG}{e} = \frac{0.15 \times 2.65}{0.58} = 68.53\%$$

$$\boxed{S_r = 68.53\%}$$

iii] $$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{0.85 - 0.58}{0.85 - 0.5} = 0.77$$

$$\boxed{I_D = 77.14\%}$$

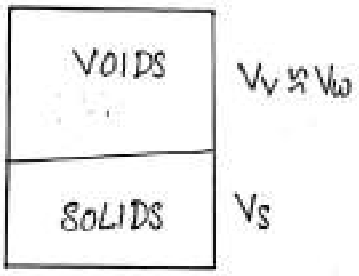
16] Saturated specimen of undisturbed clay has a volume of $19.2\,cm^3$ and weighs 32.5gms. After oven drying, the weight reduces to 20.2gm. Determine water content, specific gravity, void ratio, dry unit weight and saturated unit weight of clay.

Solution:-

$V = 19.2\,cm^3$

$W = 32.5\,gm$

$W_d = 20.2\,gm$



Weight of water

$W_w = W - W_d = \underline{12.3\,gm}$

i] Water content

$$w = \frac{W_w}{W_d} \times 100 = \frac{12.3}{20.2} \times 100$$

$$\boxed{w = 60.89\%}$$

Unit wt of water ≈ $9.81\,kN/m^3$ or $1\,gm/cc$

$$V_w = \frac{W_w}{\gamma_w} = \frac{12.3}{1} = \underline{12.3\,cc}$$

$$V_s = V - V_w = 19.2 - 12.3 = \underline{6.9\,cc}$$

ii] Void ratio (e) $= \dfrac{V_v}{V_s} = \dfrac{V_w}{V_s} = \dfrac{12.3}{6.9} = \underline{1.87}$

iii] Dry unit weight $(\gamma_d) = \dfrac{W_d}{V} = \dfrac{20.2}{19.2} = \underline{1.05\,gm/cc}$

iv) Saturated unit weight (γ_{sat})

$$\gamma_{sat} = \frac{W_{sat}}{V} = \frac{32.5}{19.2} = \underline{1.69\,gm/cc}$$

A Sampling tube of 38mm internal diameter was used to extract a sample of cohesive soil from a test pit. The length of the extracted sample was 102mm and the mass was 220gm and water content 18%. Determine the void ratio (e), porosity (n), degree of saturation (S_r) and percentage of air voids (n_a)

Solution:- Given: $G = 2.7$

$w = 18\%$

$$V = \frac{\pi d^2}{4} \times l = \frac{\pi (3.8)^2}{4} \times 10.2$$

$$V = 115.67 cm^3 \simeq \boxed{1.156 \times 10^{-4} m^3}$$

$$W = W_a^{\,0} + W_w + W_s$$

$$W = W_w + W_s$$

$$= 220 gm$$

i] Unit Weight $= (\gamma) = \frac{W}{V}$

$$\gamma = \frac{0.22 \times 9.81 \times 10^{-3}}{1.156 \times 10^{-4}} = \underline{18.66 KN/m^3}$$

ii] $\gamma_d = \frac{\gamma}{1+w} = \frac{18.66}{1+0.18} = \underline{15.81 KN/m^3}$

iii] $e = \frac{G\gamma_w}{\gamma_d} - 1 \Rightarrow \boxed{e = 0.675}$

iv] $S_r = \frac{wG}{e} = \frac{0.18 \times 2.7}{0.675} \times 100 = \underline{72\%}$

v] $n = \frac{e}{1+e} = \frac{0.675}{1+0.675} \times 100 = \underline{40.29\%}$

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

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$$15.81 = \frac{(1-n_a)2.7\times 9.81}{1+0.18\times 2.7}$$

$$(1-n_a) = 0.886$$

$$n_a = 0.113\times 100$$

$$\boxed{n_a = 11.30\%}$$

18] $1m^3$ of wet soil weights $19.8 KN/m^3$. if the specific gravity of soil particles is 2.7 and water content is 11%. Find void ratio, dry unit weight and degree of saturation.

Solution:- $V = 1m^3$

$e = ?$

$\gamma_d = ?$

$\gamma_b = 19.8 KN/m^3$

i] $\gamma_d = \frac{\gamma_b}{1+w} = \frac{19.8}{1+0.11} = \boxed{17.83 KN/m^3}$

ii] $e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.7\times 9.81}{17.83} - 1$

$$\boxed{e = 0.485}$$

iii] $S_r = \frac{wG}{e}\times 100 = \frac{0.11\times 2.7}{0.485}\times 100$

$$\boxed{S_r = 61.23\%}$$

2. Index Properties of Soil and their Determination

The Various properties of soils are grouped under two heads.

1. Index properties
2. Engineering properties

Index properties of Soils are those soil properties which are mainly used in the identification and classification of soils and help the geotechnical Engineering or Engineer in predicting the suitability of soils as foundation or construction material. Following is the Index properties of soils such as

1. Water content
2. Specific gravity of Soil particles
3. Particle size distribution
4. Consistency limit and Indices
5. Density Index
6. Insitu density

The Engineering properties of Soils are

1. Permeability
2. Compressibility
3. Shear strength.

Those properties which help to assess the Engineering behaviour such as strength or load bearing capacity swelling, shrinkage and settlement.
These properties may be relating to individual soil grains or to the aggregate soil mass.

Some of the important physical properties, which relate to the state of the soil or the type of the soil.

* **Water Content :-**

Water content (w) of a soil mass is defined as the ratio of Weight of water (W_w) present in the soil mass to the Weight of Soil solids (W_s).
It is usually expressed as percentage.

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

Water content or moisture content of a soil has a direct bearing on its strength and stability.
The water content of a soil in its natural state is termed as Natural moisture content which characterises its performance under the action of load and temperature.
The water content may range from a trace quantity to that of sufficient to saturate the soil or fill all the voids in it.
If the trace moisture has been acquired by the soil by absorption from the atmosphere there it is said to be hydroscopic moisture.

The knowledge of water content is necessary in soil compaction control in determining consistency limits of soil and for the calculation of stability of all kinds of earth work and foundations.

* Procedure

1. Weigh a clean & dry empty container with lid, (W_1) gm.
2. Take a representative sample of wet soil in the cup, replace the lid and weigh it (W_2) gms.
3. Keep it in the oven with lid removed and allow it to dry for about 16 to 24 hrs.
4. Take out the container, replace the lid & cool it. Weigh the container with lid & dried soil sample. (W_3) gm.

$$w = \left(\frac{W_2 - W_3}{W_3 - W_1}\right) \times 100 = \frac{W_w}{W_d} \times 100$$

where w = water content (%)

$(W_2 - W_3)$ = Weight of water

$(W_3 - W_1)$ = Weight of dry soil

Temperature of oven - 105°C - 110°C

b) Water content by Rapid Moisture method - calcium carbide method

1. This test is very quick, the results can be obtained within 5 to 10 min.

2.

Procedure

- It consist of air tight container with a diaphragm and a calibrated meter.
- About 6gm of Soil is mixed with fresh calcium carbide.

* Laboratory methods of determination of Index properties of Soil.

1. Water content -

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

a) Oven drying method
b) Rapid moisture method

* Water content - is defined as the ratio of the weight of water to the weight of dry soil in a given soil mass.

$$w = \frac{W_w}{W_d} \times 100\%$$

Scope :-

1. In all most all tests or experiments natural water content of the soil is to be determined.
2. The knowledge of water content is essential in all studies of Soil mechanics.
3. It is useful in determining the bearing capacity and settlement.
4. The natural moisture content will give an idea of the state of soil in the field.

- The mixture is vigorously shaken. Water in the soil reacts with calcium carbide to release acetylene gas.
- The amount of gas produced depends on available water.
- The gas creates a pressure on sensitive diaphragm and the water content is directly recorded on the calibrated meter.
- The method is not very accurate, but is extremely rapid. However, the reading gives the moisture expressed as a percentage of the wet weight of the soil. It may be converted to the moisture content expressed as percentage of the dry weight by the following relationship.

$$w = \left(\frac{w_r}{1 - w_r}\right) \times 100$$ where w_r = moisture content obtained (decimal)

2. <u>Specific Gravity of Soil Solids</u>:-

Specific gravity of Soil is defined as the ratio of the weight of a given volume of soil Particles to the weight of an equivalent volume of water at a stated temperature.

Specific gravity of Soil solids is useful in the determination of Void ratio, degree of Saturation etc.

This Index property of soil is indicative of the durability of the materials with low specific gravity likely to breakdown, where as high specific gravity materials do not break down directly. This test is primarily applicable for the evaluation of coarse grained soil.

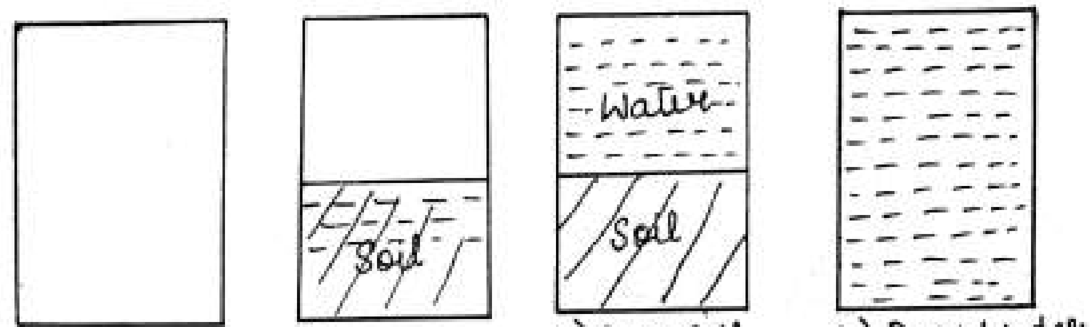
The specific Gravity of Soil particles can be determined by following methods i] Pycnometer &
ii] Density bottle method

* Pycnometer method

Apparatus consist of i) Pycnometer of about 900ml capacity with conical brass cap screwed at its top.
ii) Glass rod.
The brass cap has a 6mm diameter hole at its top.

* Procedure :-

1. Take a pycnometer which is about 900ml capacity & weigh the pycnometer with brass cap, M_1gm.
2. Fill the pycnometer with oven dry soil about $\frac{1}{3}$rd of the bottle and determine its weight along with brass cap, M_2gm.
3. Fill the pycnometer with distilled water to half its height and mix it thoroughly with the glass rod. Finally replace the screw top and fill the pycnometer with the hole in the brass cup. Dry the pycnometer from outside and find its weight (M_3) gm.
4. Empty the pycnometer, clean it and refill with clean water, find its weight, (M_4)gm.



i) Empty bottle (M_1) ii) Pyc + Dry Soil (M_2) iii) Pyc + Soil + Water (M_3) iv) Pyc + Water (M_4)

..fer ing from fig

Specific gravity = $\frac{\text{Mass of Dry soil}}{\text{Mass of equivalent Volume of water}}$

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

This method is used only for coarse grained soils.

b) Density bottle method :-

This method is more accurate method as compared to pycnometer method.

- It is suitable for all types of Soil.
- The density bottle method is the standard method used in the laboratory.
However the sequence of observations is the same as pycnometer method.

- A Density bottle of 50ml or 100ml capacity is used. The mass M_1 of the empty dry bottle is taken.
- A Sample of oven dried soil is put in the bottle and mass M_2 is taken.
- The bottle is then filled with distilled water gradually removing the entrapped air by shaking the bottle. The mass (M_3) of the bottle, Soil & water is taken.

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- Finally, the bottle is emptied completely and thoroughly washed and clean water is filled to the top, & the mass M_4 is taken.
- Based on the observations, the specific gravity can be computed.

3. Particle Size Distribution:-

The particle size distributions are made by using two methods are

a) Sieve Analysis
b) Hydrometer Analysis.

a) Sieve Analysis: Sieve analysis is mainly used for gravel & sand.
In the Indian standard, the sieves are designed by the size of the aperture in mm.

Soil in nature exists in different sizes, shapes and appearance. Depending on these attributes, the Soil at a site can be packed either densely or loosely. Hence, it is important to determine the percentage of various sized soil particles in a soil mass.
This process is called particle size distribution analysis.
For this purpose, a particle size distribution curve is plotted. Packing of Soil, ~~mass~~ amount of voids, Present influence the strength & stability of soil mass.

The distribution of grain size influences packing. Good distribution of all sizes reduces voids if compacted well

Types of Soil & their average grain sizes & shapes

Soil	Soil Component	Description	Average grain Size
Coarse grained Soil	1. Boulders	Same as below	B - more than 30cm
	2. Cobble		C - less than 30cm
	3. Gravel	Round and or angular bulky hard rock	Coarse: 80mm to 20mm Fine: 20mm to 4.75
	4. Sand	Round and or angular bulky hard rock	Coarse: 4.75mm to 2mm Medium: 2mm to 0.425mm Fine: 0.425mm to 0.075mm
Fine grained soil	1. Silt	Particle smaller than 0.075mm, exhibit little or no strength when dried	0.075mm to 0.002mm
	2. Clay	Particle smaller than 0.002mm, exhibit significant strength when dried, water reduces strength	< 0.002mm

* **Importance of Particle Size distribution :-**

1. Used for the soil classification
2. Used to select fill materials of embankments, earth dam, road sub-base materials.

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Sedimentation analysis or Hydrometer method

The Sedimentation analysis is the most convenient method for determination of grain size distribution of soil fraction finer than 75μ sieve size.

The analysis is based on Stroke's law according to which the velocities of free fall of spherical particles, fine particles settle out of suspension, all other factors, being equal is dependent on shape, weight and size of the grain.

However, in the usual analysis its assumed that soil particles are spherical and have the same specific Gravity. With this assumption, the coarser particles settle more quickly than the finer ones.

If V = is the terminal velocity of Sinking of a spherical particle, it is given by

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

where D = Diameter of the spherical particle. (m)

V = terminal velocity (m/s)

γ_s = Unit wt of particles (g/cc)

γ_w = Unit wt of water (g/cc)

η = Viscosity of water (kN-s/m²) $= \frac{\mu}{g}$

where μ = Viscosity in absolute unit

g = acceleration due to gravity.

Usually, water is the medium of suspension

$\gamma_w = 9.81 KN/m^3$ or $1 g/cc$

$\gamma_s = G\gamma_w$

$$v = \frac{1}{18} \frac{D^2 (G-1)\gamma_w}{\eta}$$

The above formula should be expressed in the consistent units of m, sec and KN. But the diameter of the particles is in mm.

$$v = \frac{1}{18} \left(\frac{D}{1000}\right)^2 \frac{(G-1)\gamma_w}{\eta} = \frac{D^2 (G-1)\gamma_w}{(18 \times 10^6)\eta}$$

Taking $\gamma_w = 9.81 KN/m^3$

we get, $v = \dfrac{D^2 (G-1)}{(1.83 \times 10^6)\eta}$ $\qquad \dfrac{18}{9.81} = 1.83$

$$D = \sqrt{\frac{18 \times 10^6 \eta . v}{(G-1)\gamma_w}}$$

$$D = 1355 \sqrt{\frac{\eta . v}{(G-1)}}$$

If a particle of diameter 'D' mm falls through a height of H_e cm in t min.

$$v = \frac{H_e}{60t} \text{ cm/s} = \frac{H_e}{6000t} \text{ m/s}$$

$$D = \sqrt{\frac{\eta(18 \times 10^6)}{(G-1)}\left(\frac{H_e}{6000t}\right)}.$$

$$\boxed{D = \sqrt{\frac{3000\eta}{(G-1)\gamma_w}} \cdot \sqrt{\frac{H_e}{t}}}$$

$$\boxed{D = K\sqrt{\frac{H_e}{t}}} \quad \text{where } K = \sqrt{\frac{3000\eta}{(G-1)\gamma_w}}$$

1) Sieve Analysis: (IS 2720 - Part 4 - 1975)

- A Set of IS Sieves are arranged in order in which one having largest aperture at the top & that with small aperture at the bottom
- A known weight of representative sample (say 1000gm) is placed in the top sieve.
- The assembly is vibrated on a sieve shaker for atleast 10min.
- Depending on the particle size, soil is collected in different sieves. Weight of soil in each sieve is measured.
-

Details of Calulation

IS Sieve No	Sieve Size (mm)	Wt of Soil on each Sieve	Cummulative weight retained	% Cummula-tive wt Retained	% Finer
4.75					
2.36					
1.18					
600					
425					
300					
212					
150					
75					

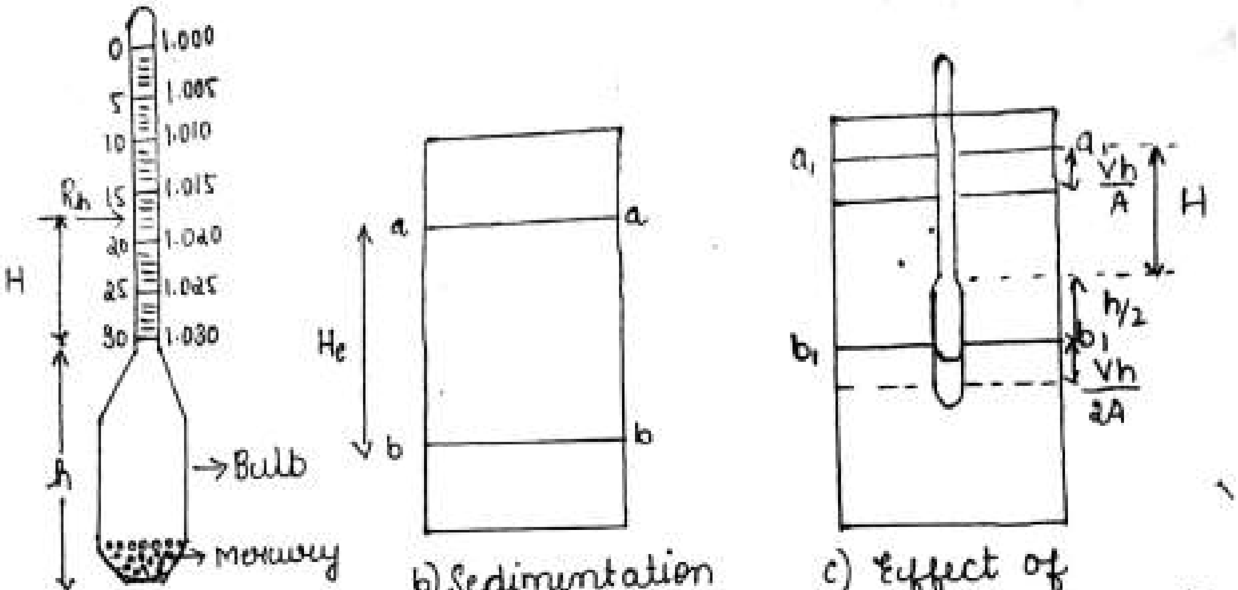
- The particle size distribution curve is obtained by Plotting % finer (N) as ordinate on natural scale against particle size 'D' mm as abscissa on logarithmic scale.

* Cummulative wt Retained = $\frac{\text{Wt of Soil on Sieve}}{\text{Total wt of soil}} \times 100$
* % Cummulative Retained = Sum of cummulative wt
* % Finer = 100 − % Cummulative Retained.

b) Hydrometer Analysis

A Hydrometer is a device made of glass, consisting of a bulb with calibrated weight A stem with calibrated readings such that when placed in pure water it floats at a level giving reading.

* Calibration of Hydrometer



a) Hydrometer

b) Sedimentation jar before immersion of Hydrometer

c) Effect of Hydrometer after immersion

- The readings on the hydrometer stem gives the density of soil suspension situated at the centre of the bulb.
- Hydrometer reading are recorded after subtracting 1 and multiplying the remaining digits by 1000. It is represented by 1000.

... example if the density reading at the intersection of horizontal surface of soil suspension with the stem, is

$R_h = 10$

Similarly a density of reading of 0.995 is -5

- Hydrometer reading R_h increases in the downward direction towards the Hydrometer bulb.
- Let H = Height (cm) between Hydrometer reading R_h & neck

  h = Height of the bulb

When Hydrometer is immersed in the jar the water level a-a rises to $a_1 - a_1$.

Rise being equal to $\left(\frac{V_h}{A}\right)$ Similarly the level b-b rises to $b_1 - b_1 = \frac{V_h}{2A}$

$$\therefore H_e = \left(H + \frac{h}{2} + \frac{V_h}{2A}\right) - \frac{V_h}{A}$$

$$H_e = H + \frac{1}{2}\left(h - \frac{V_h}{A}\right)$$

where,

H_e = Effective depth of Hydrometer

A = Internal c/s area

V_h = Volume of hydrometer

h = height of hydrometer bulb

H = Height from bulb neck to R_h

Corru

*

Procedure:-

1. About 50gms of dry soil sample passing through 75μ sieve is weighed accurately & is taken in porcelain dish.

2. 50cm³ of dispersing agent solution is added.

3. Some quantity of distilled water is also added to form a soil slurry which is gently mixed using a glass rod.

4. The contents of the dish are transferred to a cup of high speed stirrer, care should be taken to see that no particles are left behind in the dish.

5. The soil water mixture in the cup is stirred for 7 to 10min to ensure through dispersion of soil particles.

6. The contents of the cup is transferred into a 1000cc sedimentation jar using jet of water, care is taken to see that all soil particles are transferred to jar.

7. Required quantity of distilled water is added to the soil suspension in the jar to make up 1000cc.

8. The mouth of the jar is closed tightly with the palm of hand and the jar is inverted several times to ensure uniform distribution of soil particles in the soil suspension.

9. The jar is then kept on a level surface and the stopwatch is started simultaneously.

10. At the end of different elapsed time intervals. (usually 1min, 2min, 4, 8, 15, 30, 1hr, 2hr, 4hr, 8hr, 16hr, 1day the hydrometer reading is noted.

* Corrections to be applied for Hydrometer Reading

1. Meniscus Correction (C_m)

As the soil suspension is opaque, the hydrometer reading is taken corresponding to the top meniscus instead of the bottom.
To find the meniscus correction, the hydrometer is inserted into a sedimentation jar containing distilled water, when it is in Equilibrium condition, the readings corresponding to both top and bottom of the meniscus are taken.
The difference of the readings gives the magnitude of meniscus correction, which should be added to the observed hydrometer reading.

2. Correction for Temperature changes:- (C_t)

The hydrometer is calibrated at a standard temperature of 27°c.
If the temperature during the test differs from this, then corrections has to be applied.
If the temperature is more than 27°c, hydrometer readings will be less than actual value & hence correction is +ve.
If the temperature is less than 27°c, the correction will be -ve.
Thus the final correction $\pm C_t$

3. Dispersing Agent correction (C_d):-

Addition of dispersing agent to the soil causes increase in density of the suspensions. Therefore, the dispersing agent increases density the hydrometer reading will also increase & hence the correction is always -ve.

Thus, the corrected Hydrometer Reading

$$\boxed{R = R_h + C_m \pm C_t - C_d}$$

* Calculation of particle size and Percentage finer :-

$$D_{mm} = \sqrt{\frac{3000\eta}{(G-1)\gamma_w}} \times \sqrt{\frac{H_e}{t}}$$

$$N_{(\%)} = \frac{(\gamma_z - \gamma_w) \times \left(\frac{G}{G-1}\right)}{\frac{W_D}{V}} \times 100$$

where γ_z = Density indicated by hydrometer

γ_w = unit wt of water

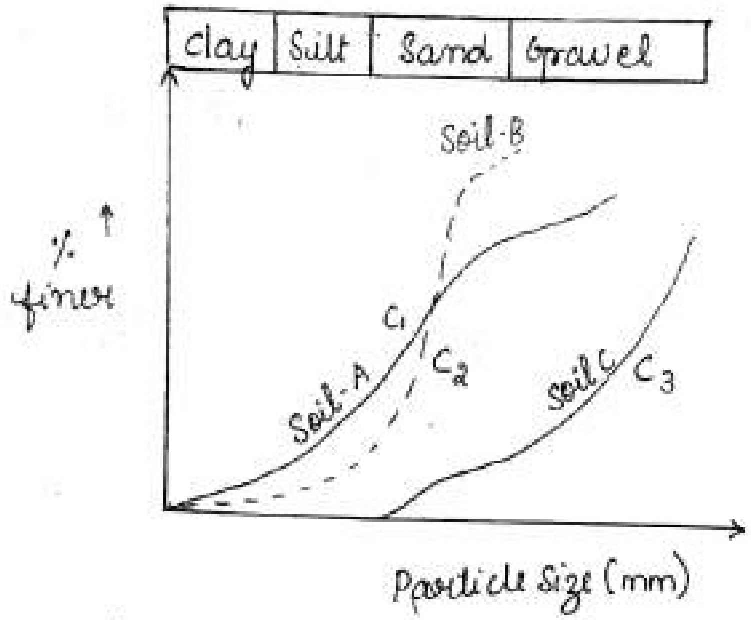
G = specific gravity of soil

Particle Size distribution Curve [Grain Size analysis / Mechanical analysis]

On the basis of shape we can classify soil as

1. **Uniformly graded or poorly graded soil**

Uniformly graded soils are those soil particles of almost same diameter



Uniformly graded soils are represented by nearly Vertical lines as shown by the curve C_2

2. **Well graded soil**: represented by curve C_1 possess a wide range of particle sizes ranging from gravel to clay size particle.

3. **Gap graded / Skip graded soil**: represented by curve C with a flat portion represent a soil in which some intermediate size particles are missing.
Gap graded Samples possess different proportions of same sized particles in increasing sizes.

* Co-efficient of uniformity :- The co-efficient of uniformity is a measure of particle size range & is given by the ratio of D_{60} to D_{10} size as

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

where D_{10} = represents size in mm such that 10% of the particles are finer than this size.

D_{30} = 30% of the particles are finer than this size

D_{60} = 60% of the particles are finer than this size.

* Co-efficient of curvature :- Represents the shape of the particle size curve & is given by

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

For well graded soil, C_c should be between 1 and 3 & C_u should be greater than 4 (if gravel) or 6 (if sand). Even if either of the two conditions is not satisfied, the sample is poorly graded.

Particle size distribution is obtained by sieve analysis for coarse grained soil fractions and Hydrometer analysis.

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Insitu Unit weight or Field Density Test

The insitu unit weight refers to the unit weight of a soil in the undisturbed conditions or of a compacted soil in place.

Determination of insitu unit weight is made on burrow pit soils so as to estimate the quantity of soil required for placing and compacting a certain fill or embankment. During the construction of compacted fills, it is a standard practise to make in-situ determination of a unit weight of the soil after it is placed to ensure that the compaction effort have been adequate.

Two important methods for the determination of the insitu unit weight are being given

1. Sand Replacement method
2. Core Cutter method.

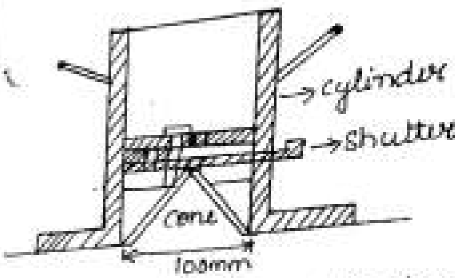
1. Sand Replacement method

The Principle of the Sand Replacement method consists in obtaining the volume of the soil excavated by filling in the hole insitu from which it is excavated with sand, previously calibrated for its unit weight and thereafter determining the weight of the sand required to fill the hole.

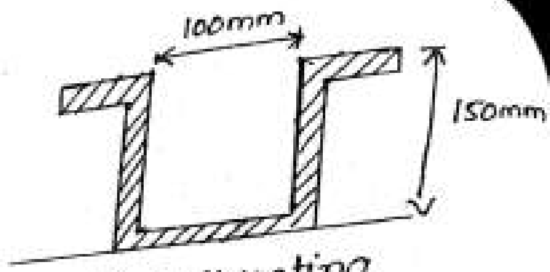
The Apparatus consists of Sand pouring cylinder, tray with a central circular hole, container for calibration of 10cm dia & 15cm height.

The Procedure consists of calibration of the cylinder and later the measurement of the unit weight of the soil.

a) Calibration of the Cylinder and sand



a) Sand Pouring Cylinder



b) Calibrating container

This consists in obtaining the weight of sand required to fill the pouring cone of the cylinder and the bulk unit weight of the sand.
Uniformly graded, dry clean sand is used.

- The cylinder is filled with sand almost to be top and the weight of the cylinder with the sand is taken as W_1 gm.
- cylinder is placed on the glass plate, the sand is run out the cylinder into the conical portion by pulling out the shutter. when no further sand runs out, the shutter is closed. The weight of the cylinder with the remaining sand is taken as W_2 gm.
The weight of the sand collected in the conical portion may also be found seperately which is equal to $(W_2 - W_1)$.
- The cylinder is placed centrally above the calibration container such that the bottom of the conical portion coincides with the top of the container.

There Sand is allowed to run into the container. If there is no further sand movement, then the Shutter is closed. The Weight of the cylinder with the remaining sand is found. (W_3).

- The weight of the Sand filling the calibrating container (W_{cc}) may be found by deducting the weight of sand filling the conical portion (W_c) from the weight of sand filling this and the container ($W_2 - W_3$).

* <u>Observation and calculations</u>

→ Calibration of the cylinder

- Initial Weight of Cylinder + Sand (W_1) gm
- Weight of cylinder + Sand after removing Sand from conical portion (W_2) gm
- Weight of Sand Occupying conical portion
  $(W_c) = W_1 - W_2$
- Weight of Sand + cylinder, Sand after removing into conical portion and calibrating container (W_3) gm
- Weight of Sand required to fill the calibrating container, $(W_{cc}) = (W_2 - W_3) - W_c$
- Volume of the calibrating container (V_{cc})

Unit weight of Sand $\gamma_s = \dfrac{W_{cc}}{V_{cc}}$

* Measurement of Unit Weight of Soil

The Site at which the insitu weight is to be determined is cleaned & levelled.

- A test hole about 10cm diameter and for the depth of calibrating container (15cm) is made at the site with the help of tray and the excavated soil is collected in it and its weight is found ie (W_5).
- The Sand is refilled into the container and take its weight ie (W_4)gm.
- The Cylinder is centrally placed over the hole, and the sand is allowed to run into it. The Value is closed when no further of movement of sand takes place. Take its Weight ie (W_5)gm.
- The Weight of sand occupying the test hole (W) will be equal to $(W_4 - W_5) - (W_1)$
- The insitu unit weight of the soil (V) is then obtained by dividing Weight of the soil by Volume.

Moisture content of the excavated soil is determined by oven drying method.

Observation & calculation

- i] Weight of cylinder + Sand (W_4)gm
- ii] Weight of cylinder + sand after running into the test hole. (W_5) gm
- iii] Weight of Sand occupying the test hole $W = (W_4 - W_5) - W_1$
- iv] Weight of Excavated Soil (W_5)gm.
- v] Volume of test hole $(V) = \frac{W}{\gamma_s}$
- vi] Insitu unit wt of Soil $= \frac{W_5}{V}$

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Core Cutter method

Apparatus:

i] Cylindrical core cutter, of 10cm dia & 12.5cm to 15cm length

ii] Steel Rammer having mass of 9Kg.

iii] Straight edge, container etc

dolly - 2.5cm in length

Procedure:-

1. Measure the height & internal diameter of the core cutter & calculate its weight.
2. Determine the empty weight of core cutter without dolly. (W_1) gm
3. Expose the small area about 30cm² to be tested & level it.
4. Place the core cutter on level surface, keep the dolly on the cutter & advance the cutter into subsoil using rammer until about 15mm of the dolly protrudes above the surfaces.
5. Dig out the core cutter from the surrounding soil, allow some to protect from lower end of the cutter with the help of straight edges or palate knife trim the cutter.
6. Weight the core cutter with soil & without dolly. W_2 gm
7. Remove the soil from the core cutter & determine the water content of soil.

Wt of core cutter (W_1) gm =

Wt of core cutter + Soil (W_2) gm =

Wt of wet soil $(W = W_2 - W_1)$ gm =

Insitu density $\gamma = \frac{W}{V}$

Consistency of Soil

The term consistency mostly used for fine grained soils for which the consistency is related to a larger extent to water.

- Definition: Consistency limits are the water contents at which the soil mass passes from one state to the other.
- consistency denotes degree of firmness of the soil which may termed as soft, firm, stiff or hard.
- The addition of water ~~reduces~~ increases the cohesion making the soil still easier to mould. Further addition of water reduces the cohesion until the material no longer retains its shape under its own weight.
- In 1911, the swedish Agricultural Atterburg observed four states of consistency, namely
  - i) Liquid state
  - ii) Plastic state
  - iii) Semi Solid state
  - iv) Solid state

  These consistency limit are also called as Atterburg limits.
- The Atterburg limits which are most useful for engineering purposes are:-

1. Liquid Limit :- (W_L)

Liquid limit is the water content corresponding to arbitary limit b/w liquid and plastic state of consistency of Soils.

- It is defined as the minimum water content at which the soil is still in liquid state but has a small shearing strength and exhibits some resistance to flow.

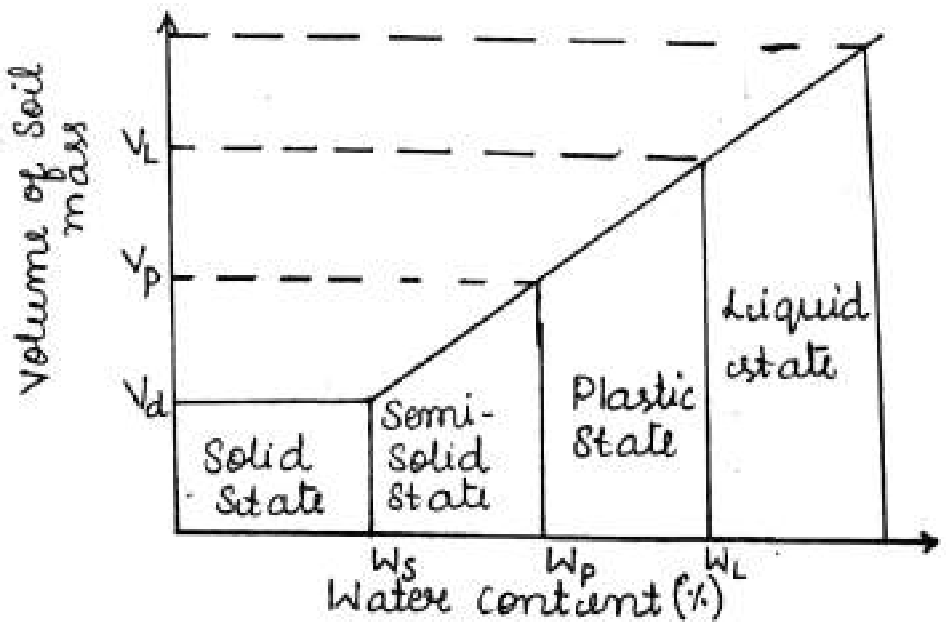
2] Plastic limit (W_p):

It is the water content corresponding to an arbitary limit b/w plastic & the semi-solid states of consistency of soils

- 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.

3] 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 soil mass.



Consistency Indices:-

The following are the consistency Indices

1. **Plasticity Index:- (I_p)**

The plasticity Index is defined as the numerical difference between Liquid limit & the plastic limit of a Soil

$$I_p = W_L - W_p$$

In case of sandy soils, plastic limit should be determined first.

- When plastic limit is not determined, the plasticity index is reported as NP (Non plastic)
- When the plastic limit is equal or greater than the liquid limit, the plasticity Index is reported as zero.

[Plasticity : Plasticity is defined as the property of soil which allows it to be deformed rapidly without rupture, without elastic rebound and without volume changes]

2] **Consistency Index (I_c):-**

The consistency index or the relative consistency is defined as the ratio of liquid limit minus the water content to the Plastic Index.

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

Thus, if consistency Index of Soil is equal to unity, it is at the plastic limit.

- A Soil with I_c equal to zero is at its liquid limit.
- If I_c exceeds unity, the soil is in a semi-solid state and will be stiff.
- A -ve I_c indicates that the soil has water content greater than liquid limit & hence behaves just like a liquid.

3] Liquidity Index :- (I_L)

Liquidity Index is the ratio expressed as %age of the natural water content minus its plastic limit to its plasticity Index.

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

where w = water content of the Soil

Determination of Liquid limit :-

Liquid limit can be determined by two methods

1. Casagrande's method
2. Cone penetration method

1. Casagrande's method :-

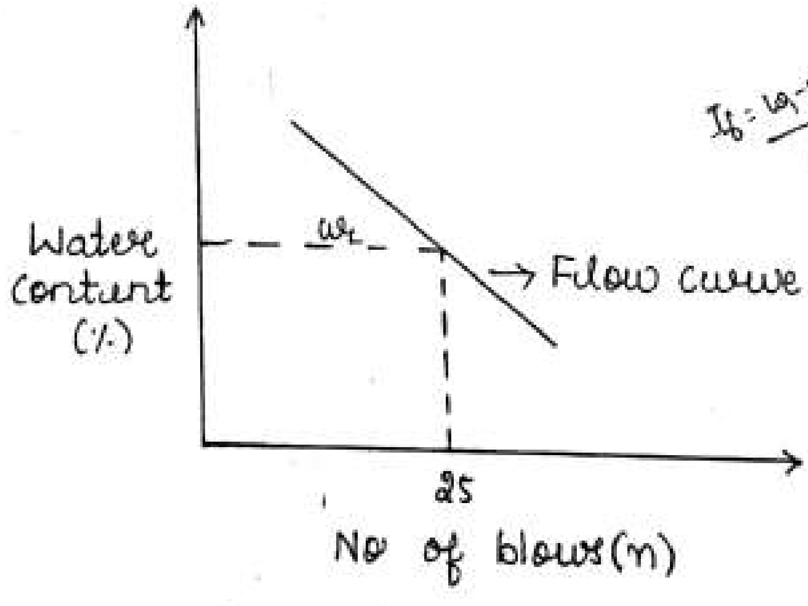
* Apparatus :- The apparatus consists of a hard rubber base over which a brass cup drops through a desired height.
- The brass cup can be raised & dropped to fall on the rubber base with the help of a cam operated by a handle.
- Two types of grooving tools are used :- namely - casagrande tool & ASTM Tool.
- Before starting the test, the height of fall is adjusted to 1cm with the help of adjusting screws.

* Procedure :-

1. Take about 120gms of soil passing through 425μ sieve. Some quantity of water is added to it & thoroughly mixed to form soil paste.
2. A portion of the soil paste is placed in the cup of liquid limit device and levelled by means of spatula.

Using Standard grooving tool a groove is cut in soil pat.

- The cup is given blows by manual operation of handle. The handle is rotated at a rate of about 2 revolutions/second & the no of blows are counted until the two parts of soil come in contact at the bottom of groove along a distance of 10mm.
- Some quantity of soil, where the groove has closed is taken for water content determination.
- Atleast 4 to 5 trials are carried out in the range of 10 to 50 blows.
- The graph plotted of water content against the no of blows as abscissa on logarithmic scale to obtain flow curve & the water content corresponding to 25 blows is termed as liquid limit



$I_f = \frac{w_1 - w_2}{\log\left(\frac{n_2}{n_1}\right)}$

Flow Index $(I_f) = \frac{W_1 - W_2}{\log\left(\frac{n_2}{n_1}\right)}$

Where

w_1 = water content corresponding to 10 blows (n_1)
w_2 = " " " to 100 blows (n_2)

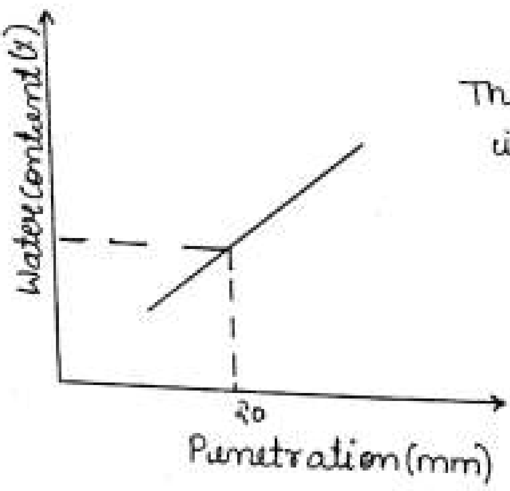
2] Cone Penetration method :-

* Procedure :-

1. Take about 150 grams of dry soil passing through 425μ sieve and mixed with sufficient distilled water until uniform consistency is achieved.
2. A portion of paste is filled in the cylindrical mould of 50mm dia and 50mm height and levelled upto top.
3. The penetrometer is so adjusted that the cone point just touches the soil surface.
4. The cone has a central angle of 31° and total weight of 148gm is released so that cone penetrates into the soil paste under its own weight.
5. The penetration is noted on the graduated scale after 30 sec from the release of the cone.
6. Take some quantity of Soil paste from the mould after test & find the water content.

The liquid limit corresponding to the moisture content of paste which gives 20mm of penetration of the cone can be easily determined by using the formula.

The no of ~~blows~~ trials are conducted and the graph is plotted for water content Versus Penetration and the water content corresponds to 20mm penetration gives liquid limit of the soil.



The liquid limit is obtained by

$$\omega_L = \frac{\omega}{0.77 \log_{10} x}$$

where x = Penetration
ω = water content corresponding to penetration 'x'.

Determination of Plastic limit :-

Procedure :-

1. About 30gms of soil sample passing through 425μ IS seive is taken & some quantity of water is added & thoroughly mixed to form soil paste which can be rolled into ball between palms of hands.
2. A Small ball is taken & rolled b/w the fingers and a glass plate with sufficient pressure to roll the mass into thread of uniform diameter throughout the length.
3. When a diameter of 3mm is reached, the thread is looked for sign of cracking.
4. If no cracks are seen, the soil is remoulded again into a ball.

- This process of rolling and remoulding is repeated until the thread just starts crumbling at a diameter of 3mm.
- The crumbled threads are taken for water content determination. which is the plastic limit.
- The test is repeated twice more with fresh samples.
- The Plastic limit (w_p) is then taken as average of three water content.

Toughness Index (I_T) is defined as the ratio of the plasticity Index to the flow Index.

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

where I_P = plasticity Index

I_F = Flow Index

* Determination of Shrinkage Index :-

Shrinkage limit is denoted by W_s and is the boundary between the solid and semi solid state of consistency.

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

Experimental Procedure

Take about 50gms of oven dried soil sample passing through IS 425μ sieve & is mixed thoroughly with distilled water to form a soil paste of slightly flowing consistency.

The Equipment or apparatus consist of a porcelain Evaporating dish of about 12cm dia, a Shrinkage dish of stainless steel with flat bottom, two plates each 75mm x 75mm - 3mm thick one plain glass and other with three prongs, straight edge, oven, mercury.

Step 1- The Shrinkage dish (non corrodible cup of 45mm dia and 15mm height) is weighed after coating inner side of the cup with a thin layer of grease & oil.

- The Shrinkage cup is filled with the soil paste in three layers, the cup being gently tapped on a cushioned surface after filling with each layer to ensure explusion of air. The Surface of the soil is levelled and outside of cup is cleaned.
- The mass of Shrinkage cup with wet soil pat is found and this is deducted from mass of shrinkage cup to get the mass of Wet soil pat (W_i)
- The Wet soil pat is allowed to dry in air for sometimes, then kept in thermostatically controlled oven and dried for 24hrs at 105°C - 110°C
- After oven drying, the mass of dry soil pat (W_d) is found.

Observations:-

Weight of Empty Shrinkage dish (W_1) gm

Weight of Shrinkage dish + Wet Soil (W_2) gm

Weight of Shrinkage dish + dry Soil (W_3) gm

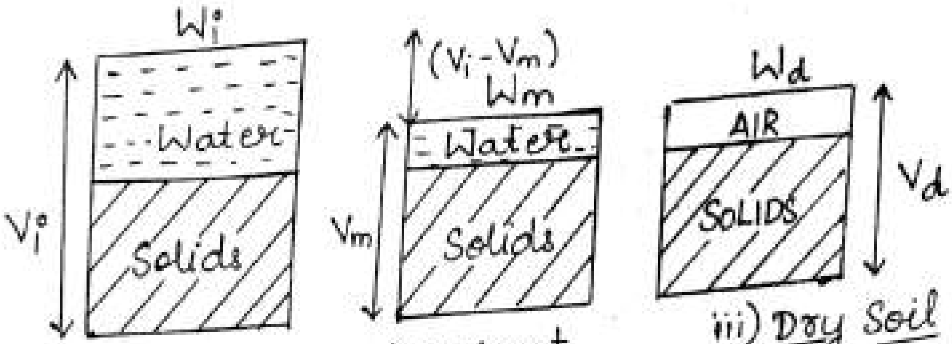
Weight of Wet Soil pat $(W_i) = W_2 - W_i$

Weight of dry soil pad $(W_d) = W_3 - W_1$

Step 2: Determination of Volume

Volume of dry soil pad (V_d) is found by mercury displacement method.

- Take the Weight of empty porcelain dish
- Fill the Glass cup with mercury and the excess mercury is removed by pressing the glass plate over the top of the cup.
- The Glass cup is then placed in the porcelain dish. The dry soil pat is placed on the surface of the mercury filled cup and is pressed down by means of excess mercury.
- Take the Weight of the mercury displaced by dry soil pat and porcelain dish.
- The mass of the mercury so displaced divided by the density of mercury gives the Volume (V_d) of the dry soil pat.



i) Initial Stage of soil pat

ii) Soil pat at Shrinkage limit

iii) Dry Soil pat

Determination of Shrinkage limit

Weight of water initially = $(W_i - W_d)$

Loss of water from the initial stage to the stage of shrinkage limit = $(V_i - V_m)\gamma_w$

∴ Weight of water at Shrinkage limit

$$= (W_i - W_d) - (V_i - V_m)\gamma_w$$

∴ Shrinkage limit (W_s) = $$\left[\frac{(W_i - W_d) - (V_i - V_m)\gamma_w}{W_d}\right] \times 100\%$$

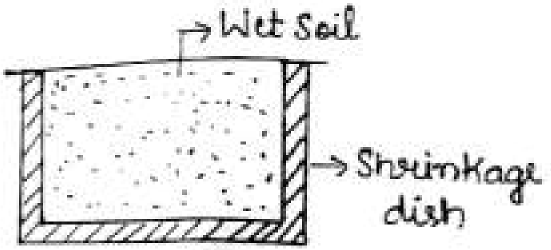
$$\boxed{W_s = \left[\frac{W_i - (V_i - V_m)\gamma_w}{W_d}\right] \times 100\%}$$

where W_i = Initial water content

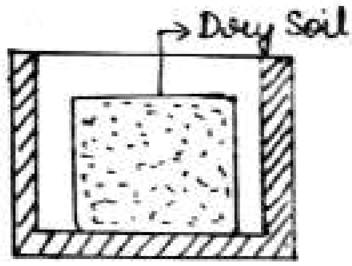
$V_d = V_m$ = dry volume of the soil pat

V_i = Initial Volume of the soil pat

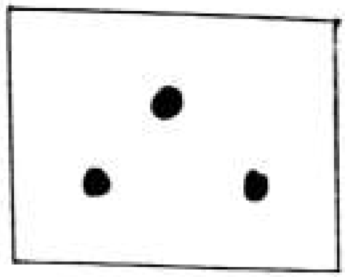
W_d = Dry weight of the Soil Sample



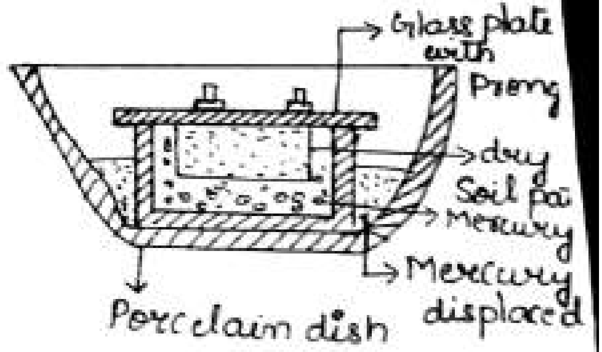
Before Shrinkage



After Shrinkage



Glassplate (75mm x 75mm) with three prongs



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

Apparatus for determining volume change in the Shrinkage limit test

* Shrinkage Factors:-

1. Shrinkage ratio :- It is defined as the ratio of volume change expressed as percentage of the dry volume to the corresponding change in moisture content from the initial value to the Shrinkage limit.

$$\boxed{R = \frac{(V_i - V_d)}{V_d} \times 100 \, (W_i - W_s)}$$

or

$$R = \frac{\left(\frac{(V_i - V_d)}{V_d}\right)}{W_i - W_s} \times 100$$

$$(W_i - W_s) = \frac{(V_i - V_d)\gamma_w}{W_d} \times 100$$

2] Volumetric Shrinkage :- (V_s)

The Volumetric Shrinkage or volumetric change (V_s) is defined as the decrease in the volume of a soil mass expressed as a percentage of the dry volume of soil mass, when the water content is reduced from an intial value to the Shrinkage limit.

$$V_s = \frac{(V_i - V_d)}{V_d} \times 100$$

3] Degree of Shrinkage (S_r) :-

The degree of shrinkage is defined as the ratio of the difference between initial volume and final volume of the soil sample to its initial volume.

$$S_r = \frac{(V_i - V_d)}{V_i} \times 100$$

4. Linear Shrinkage (L_s) :-

Linear Shrinkage is defined as the decrease in one dimension of the soil mass expressed as a percentage of initial dimension, when the water content is reduced from a initial value to the Shrinkage limit.

$$L_s = \left[1 - \sqrt[3]{\frac{100}{V_s + 100}} \right] \times 100$$

Sedimentation Analysis is based on Stoke's law & it is defined as follows

Stoke's law:-

Stoke's law states that "the velocity at which grains settle out of suspensions, all other factors being equal & is dependent upon the shape, weight & size of the grain.

Assumptions of Stoke's law:-

1. The Stoke's law assumes soil particles are spherical falling in liquid of infinite extent & all the particles have the same unit weight.
2. Particles settle independent of other particles.
3. The neighbouring particles do not have any effect on its velocity of settlement.
4. The walls of jar, in which suspension is kept also do not affect the settlement.
5. Coarse particles settle more quickly than the finer ones.
6. The soil has an average specific gravity the value of which is used in computing diameter (D).

If V = is the terminal velocity of sinking at a spherical particle, it is given by

$$V = \frac{2}{9} r^2 \left[\frac{\gamma_s - \gamma_w}{\eta} \right]$$

where r = radius of Spherical particle

η = Viscosity

γ_s = unit weight of particle

γ_w = unit weight of water

Limitation of Stoke's law

1. However the above assumptions are not strictly valid. Since the particles used in analysis are not truly spherical.
2. There will be influence of one particles over the other.
3. Different materials will have different specific gravity depending upon their mineral constitution.
4. The particles falling near the wall of the jar are also affected.
5. For particles smaller than 0.0002mm equivalent diameter, Brownian movement affects their settlement & stoke's law no longer remains valid.
   ↳ (Zig zag / Irregular motion)

* Relative Density / Density Index (I_D)

It is defined as the ratio of the difference between the void ratio of the soil in its loosest state 'e_{max}' and its natural void ratio 'e' to the difference between the void ratio in the loosest & densest state.

It can be written as

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

where e_{max} = void ratio in the loosest state
e_{min} = void ratio in the densest state
e = natural void ratio

The term Relative density is used for cohesionless soil only.

1. When the natural state of the cohesionless soil is in its loosest state, $e = e_{max}$ & hence $I_D = 0$.
2. When the natural deposit is in the densest state, $e = e_{min}$ & hence $I_D = 1$
3. For any Intermediate state, the relative index varies between zero & one.

* <u>Activity</u> of <u>clay</u>.

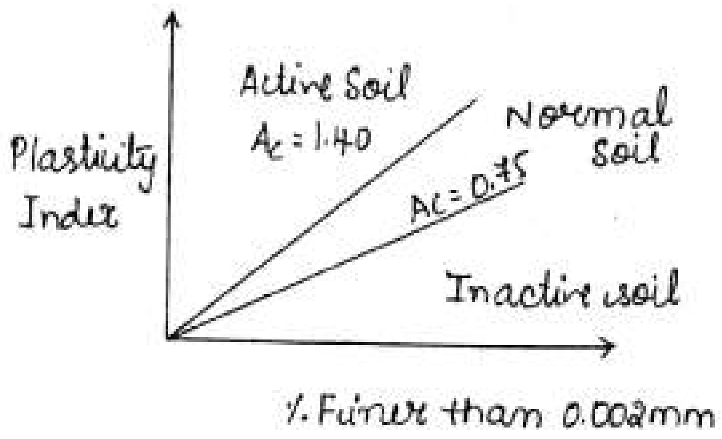
The Activity of clay is mainly influenced by the clay minerals which are present in the clay & its behaviour.

- The plasticity of clay depends upon the
  i) Nature of clay mineral present
  ii) Amount of clay mineral present
- According to skempton (1953), plasticity Index of soil is directly proportional to the percent of clay size particles.

He defined Activity (A_c) as the ratio of plasticity index to the percent weight of soil particles of diameter smaller than two microns present in the soil.

Thus,
$$A_c = \frac{I_p}{C_w}$$

where I_p = plasticity Index,
C_w = percentage by weight of clay sizes i.e. of particle less than 2μ



Activity less than 0.75 - Inactive
0.75 - 1.40 - Normal
>1.40 - Active.

- Activity of clay depends on type of mineral. For kaonite, the activity of clay is relatively low while those having montmorillonite will have high active.

Problems:-

1. In a Specific Gravity test with pycnometer, the following observations were made:-

Wt of empty pycnometer (W_1) = 750gm
Wt of Pycnometer + dry soil (W_2) = 1730gm
Wt of pycnometer + Soil + Water (W_3) = 2245gm
Wt of pycnometer + Water (W_4) = 1630gm.

Determine the Specific Gravity of Soil solids, ignoring the temp.

Soln:

$$G = \frac{W_2 - W_1}{(W_2 - W_1) - (W_3 - W_4)} = \frac{1730 - 750}{(1730 - 750) - (2245 - 1630)}$$

$$\boxed{G = 2.68}$$

2] In a Specific Gravity, wt of dry soil is taken as 66gms, wt of pycnometer filled with soil & water = 675.6gm
Wt of Pycnometer + water = 633.95gm. Temp of the test = 38°c. Determine the grain specific gravity taking specific gravity of water at 30° = 0.99568.
ii) Applying the necessary temp correction report the value of G should be obtained if test was conducted at 4°c and also at 27°c. The Specific Gravity of water at 4°c & 27°c are 1.0 & 0.99654 respectively.

Soln:

Wt of dry soil ($W_2 - W_1$) = 66gms
Wt of Pync. + Soil + Water = 675.6gm
Wt of Pync + Water = 633.95gm

$$G = \frac{W_2 - W_1}{(W_2 - W_1) - (W_3 - W_4)} = \frac{66}{66 - (675.6 - 633.95)}$$

$$G = 2.71$$

At 4°C,

Specific gravity at 4° $= G \times (\gamma_w)_4 = 2.71 \times 1$
$= 2.71$

At 27°C,

Specific gravity at 27° $= G \times (\gamma_w)_{27} = 2.71 \times 0.99654$
$= 2.70$

3. In a specific gravity test, the following observations were made.
Wt of dry soil = 104 gm, Wt of bottle + Soil + Water = 538 gm
Wt of bottle + Water = 475.6 gm
Determine the specific Gravity of Soil Solids.
While obtaining the wt 538 gm, 3 ml of air was entrapped in the suspension. Will the computed value of 'G' will be higher or lower than the corrected values. Also, calculate % error. Neglect temp effect.

Soln: $(W_2 - W_1) = 104\,gm$
$W_3 = 538\,gm$
$W_4 = 475.6\,gm$

$$G = \frac{W_2 - W_1}{(W_2 - W_1) - (W_3 - W_4)} = \frac{104}{104 - (538 - 475.6)} = 2.5$$

$$\boxed{G = 2.5}$$

If 3ml of air entrapped then

$W_3 = 538 + 3 = 541 gm$

then $G = \frac{104}{104 - (541 - 475.6)} = 2.70$

$Error(\%) = \frac{2.7 - 2.5}{2.7} \times 100 = 7.4\%$

4.) Pycnometer was used to determine the water content of a Sandy soil. The following observation were obtained. Compute Water content of Soil Sample.

Wt of empty pycnometer (W_1) = 8N
Wt of Pycnometer + Dry soil (W_2) = 11.6N
Wt of Pycnometer + Dry soil + Water = (W_3) = 20N
Wt of Pycnometer + Water (W_4) = 18N
Specific Gravity of Soil solids = 2.66

$$w = \left[\frac{(W_2 - W_1)}{(W_3 - W_4)} \left(\frac{G - 1}{G} \right) - 1 \right] \times 100$$

$$= \left[\frac{(11.6 - 8)}{(20 - 18)} \left(\frac{2.66 - 1}{2.66} \right) - 1 \right] \times 100$$

$$\boxed{w = 12.33\%}$$

5. 1000cc core cutter weighs 950grams was used to find out in-situ unit weight of embankment. The wt of corecutter filled with soil is 2770gms. Lab test on sample will be conducted a water content of 10.45% & G=2.6. Determine the Bulk density, dry unit weight, void ratio & degree of Saturation.

ii) If the embankment is saturated due to rain, calculate the water content & saturated unit weight Assume there is no change in volume of the sample on saturation.

Solu:

Volume of corecutter = 1000cc &

$W_1 = 950gm$, $W_2 = 2770\ gm$

$\omega = 10.45\%$ & $G = 2.6$

i] $\gamma = \frac{W_2 - W_1}{V} = \ldots \approx 17.85\ KN/m^3$

ii] $\gamma_d = \frac{\gamma_b}{1+\omega} = \frac{17.85}{1+0.1045} = 16.16\ KN/m^3$

iii] $e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.6(9.81)}{16.16} - 1$

$e = 0.578$

iv) $S_r = \frac{\omega G}{e} = \frac{0.1045 \times 2.6}{0.578} \approx 47.0\%$

ii] Fully saturated $S_r = 100\%$

$w = \frac{1 \times 0.578}{2.6} \times 100 \approx 22.23\%$

$$\gamma_{sat} = \frac{\gamma_w (G+e)}{1+e} = \frac{9.81(2.6+0.578)}{1+0.578}$$

$$\boxed{\gamma_{sat} = 19.745\ KN/m^3}$$

6. A density test was conducted by core cutter method. The following data was obtained.
Determine γ_d, e, Sr.

i] Wt of empty core cutter (W_1) = 2280 gms
ii] Wt of Soil + core Cutter (W_2) = 5005 gms
iii] Inside diameter of core cutter (d_1) = 90mm = 0.09m
iv] height of core cutter = 180mm = 0.18 m
v] Weight of Wet sample for moisture content determination (W_2) = 54.05 gm
Weight of oven dry Sample = 51.12 gm (W_3)
Specific Gravity of Soil grains (G) = 2.72.

Solu :

$W = 5005 - 2280 = 2725\ gm$

$$V = \frac{\pi d^2}{4} \times h = \frac{\pi \times (0.90)^2}{4} \times 0.180 = 1.145 \times 10^{-3}\ m^3$$

$$\text{Water content } (w) = \frac{0.0293}{0.5112} \times 100 =$$

$w = 5.73\%$

i] $$\gamma = \frac{W}{V} = \frac{2725 \times 9.81}{1.145 \times 10^{-3}} = 23346.9\ N/m^3 \simeq 23.34\ KN/m^3$$

$$\gamma_d = \frac{\gamma}{1+w} = \frac{23.79}{1+0.0573} = 22.50\ kN/m^3$$

$$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.72 \times 9.81}{22.50} - 1$$

$$\boxed{e = 0.185}$$

Degree of Saturation $(S_r) = \frac{wG}{e} = \frac{0.0573 \times 2.72}{0.185}$

$$\boxed{S_r = 84.24\%}$$

7. The maximum & minimum dry unit weight of Sand is determined in the laboratory are 20 kN/m³ & 15 kN/m³ respectively. If the relative density of Sand is 74%. Determine the porosity of Sand deposit. Assume $G = 2.6$.

Soln: Given: $(\gamma_d)_{max} = 20\ kN/m^3$

$(\gamma_d)_{min} = 15\ kN/m^3$

$I = 74\%$ & $G = 2.6$

we have

i] $(\gamma_d)_{max} = \frac{G\gamma_w}{1+e_m} = \frac{2.6 \times 9.81}{1+e_{min}}$

$\gamma_d(1+e_{min}) = 25.50$

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

$e_m = \frac{25.50}{20} - 1 = 0.2753$

$e_{min} = 0.2753$

$e_{max} = 0.7004$

ii] $I = \frac{e_{max} - e}{e_{max} - e_{min}}$

$$0.74(0.7004-0.2753) = 0.7004-e$$

$$e = -0.3145+0.7004$$

$$\boxed{e = 0.386}$$

$$\text{porosity}(\eta) = \frac{e}{1+e} = \frac{0.386}{1+0.386} = 0.278 \times 100$$

$$\boxed{\eta = 27.84\%}$$

8] An undisturbed Specimen of clay was tested in laboratory and the following results were obtained.

i] Wet weight = 2.1 gm
ii] Wet of Oven dried Sample = 1.75 gm
iii] Specific gravity (G) = 2.7

What was the total volume of original undisturbed specimen. Assuming that the specimen was 50% Saturated.

Solu : i] Water Content $(w) = \frac{2.1-1.75}{1.75} \times 100 = 20\%$

$$\boxed{w = 20\%}$$

ii] $wG = Sre \qquad e = \frac{0.20 \times 2.7}{0.5} = 1.08$

$$\boxed{e = 1.08}$$

iii] $\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.7 \times 9.81}{1+1.08} = 12.73 \text{kN/m}^3$

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

$\gamma = 12.73(1+0.20)$

$\gamma = 15.27 KN/m^3$

v] $\gamma = \frac{W}{V}$

$15.27\times10^{3} = \frac{2.1}{V}$

$V = 1.37\times10^{-4} m^3$

9] The Bulk unit weight of Soil is $19.10 KN/m^3$, $w = 12.5\%$ & $G = 2.67$. Determine γ_d, e, η, S_r

Solu :

$\gamma_b = 19.10 KN/m^3$

$w = 12.5\%$

$G = 2.67$

i] $\gamma_d = \frac{\gamma}{1+w} = \frac{19.10}{1+0.125} = 16.97 KN/m^3$

$\gamma_d = 16.97 KN/m^3$

ii] $e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.67\times9.81}{16.97} - 1 = 0.543$

$e = 0.543$

iii] $e+1 = \frac{1}{1-\eta}$ or $\eta = \frac{e}{1+e} = \frac{0.543}{1+0.543}$

$\eta = 0.352$

iv] $S_r = \frac{wG}{e} = \frac{0.125\times2.67}{0.543} = 0.614$

$S_r = 61.46\%$

10] The liquid limit of clay soil is 56% and its plasticity Index is 15%.

1. In what state of consistency is this material at water content of 45%
2. What is the plastic limit of the soil
3. Void ratio of the soil at minimum value reached 1.88. What is the Shrinkage limit if its specific gravity is 2.71

Solu: $W_L = 56\%$, $I_P = 15\%$

$w = 0.45$

i] $I_C = \frac{W_L - w}{I_P} = \frac{0.56 - 0.45}{0.15} = 0.733 \Rightarrow$ Plastic with medium stiff

ii] $I_L = \frac{W - W_P}{I_P} \Rightarrow \frac{0.45 - 0.41}{0.15} = 0.266 \Rightarrow$ medium Stiff

$I_P = W_L - W_P$

$0.15 = 0.56 - W_P$

$\boxed{W_P = 0.41 \simeq 41\%}$

11] The liquid limit test on sample gives the following res The plastic limit of the soil is 40%.

No of Blows (N)	12	18	22	34
Water content (w)	56	52	50	45

Plot a flow curve & obtain i] W_L ii] Flow index iii] Plasticity Index IV] Toughness Index

Solu: From graph, $W_1 = 58\%$, $W_2 = 35\%$

$$I_f = \frac{W_1 - W_2}{\log\left(\frac{n_2}{n_1}\right)} = \frac{58 - 35}{\log\left(\frac{100}{10}\right)} = 23$$

$\boxed{\therefore I_f = 23}$

ii] Liquid limit (W_L) = 48%

iii] Plasticity Index (I_p) = $W_L - W_p$ = 48-40

I_p = 8%

iv] Toughness Index

(I_T) = $\frac{I_p}{I_f}$ = $\frac{8}{23}$ = 0.35

13] An undisturbed Saturated specimen has a volume of $18.9 \times 10^3 mm^3$ and weight of 0.294N on oven drying, the above specimen of clay weighs = 0.176N. The volume of dry specimen as determined by the displacement of mercury is $9.9 \times 10^3 mm^3$. Determine the Shrinkage limit & Specific Gravity

Solu:

$M_1 = 0.294 N$; $V_1 = 18.9 \times 10^3 mm^3 \Rightarrow 18.9 \times 10^{-6} m^3$

$M_d = 0.176 N$; $V_d = 9.9 \times 10^3 mm^3 \Rightarrow 9.9 \times 10^{-6} m^3$

$$W_s = \left[\frac{(M_1 - M_d)}{M_d} - \frac{(V_1 - V_d)\gamma_w}{M_d}\right] \times 100$$

$$W_s = \left[\frac{(0.294 - 0.176)}{0.176}\right] - \left[\frac{(18.9 \times 10^{-6} - 9.9 \times 10^{-6}) 9.81 \times 10^3}{0.176}\right] \times 100$$

$$W_s = (0.670 - 0.5016) \times 100$$

$$\boxed{W_s = 16.83\%}$$

$$G = \frac{1}{\left[\frac{\gamma_w}{\gamma_s} - \frac{w_s}{100}\right]}$$

Where

$$\gamma_s = \frac{M_d}{V_d} = \frac{0.176 \times 10^{-3}}{9.9 \times 10^{-6}}$$

$$= 17.77 KN/m^3$$

$$= \frac{1}{\left[\frac{9.81}{17.77} - 0.1683\right]} = 2.608$$

$$\boxed{G = 2.608}$$

A Soil has a plastic limit of 25% and a plasticity Index of 30%. If the natural water content of the soil is 34%.
What is the liquidity Index and what is the consistency Index & how do you describe the consistency.

Solution:- $W_p = 25\%$
$I_p = 30\%$
$w = 34\%$

Liquid limit (W_L)

$$I_p = W_L - W_p$$

$$W_L = W_p + I_p = 25 + 30$$

$$\boxed{W_L = 55\%}$$

i] Liquidity Index (I_L) = $\dfrac{w - W_p}{I_p} = \dfrac{34 - 25}{30}$

$$\boxed{I_L = 0.30}$$

ii] Consistency Index (I_c) = $\dfrac{W_L - w}{I_p} = \dfrac{55 - 34}{30}$

$$\boxed{I_c = 0.7}$$

The consistency of soil may be described as <u>medium soft</u> or <u>medium stiff</u>.

20] A fine grained soil is found to have a liquid limit of 90% and a plasticity Index of 50. The natural water content is 28%. Determine the liquidity Index and indicate the probable consistency of the natural soil.

Solu:

Liquid limit (W_L) = 90%

Plasticity Index (I_p) = 50%

w = 28%

$$\text{Liquidity Index } (I_L) = \frac{w - w_p}{I_p}$$

$$I_p = W_L - W_p$$

$$\boxed{W_p = 40\%}$$

$$\text{Liquidity Index } (I_L) = \frac{28-40}{50} = \underline{-0.24 \text{ (-ve)}}$$

Since the liquidity Index is -ve, the soil is in semi solid state of consistency and is stiff. This fact can be informed directly from the observation that the natural moisture content is less than that of plastic limit.

Following are the observations ~~are~~ made from seive analysis on soil of 50N.
Find uniformity co-efficient and co-efficient of curvature. Also find percentage of gravel, sand, silt and clay.

I.S Sieve	4.75	2	1	600µ	425µ	150µ	75µ
Wt of Soil (N) retained	0.38	3.22	5.28	3.87	12.25	16	7.6

Solution :-

I.S Sieve	Sieve Size (mm)	Wt of Soil retained (N)	% Retained	Cummulative % Retained	% finer
4.75	4.75	0.38	0.76	0.76	99.24
2	2	3.22	6.44	7.2	92.8
1	1	5.28	10.56	17.76	82.24
600µ	0.6	3.87	7.74	25.5	74.5
425µ	0.425	12.25	24.5	50	50
150µ	0.150	16	32	82	18
75µ	0.075	7.6	15.2	87.2	12.8

$$\% \text{ Retained} = \frac{0.38}{50} \times 100 = 0.76$$

Cummulative % retained = Sum of % Retained

% finer = 100 − Cummulative % Retained

From Graph, we have

$D_{10} = 0.069mm$

$D_{30} = 0.26mm$

$D_{60} = 0.45mm$

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.45}{0.069} = 6.52$$

$$C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} = \frac{(0.26)^2}{0.45 \times 0.069} = 2.177$$

% percentage of gravel = 2%

sand = 87%

silt & clay = 11%

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Module-2

Clay Mineralogy and Soil Structure

Formation of a soil aggregate is the result of deposition of soil solids in suspension with air or water

The geometrical arrangement of soil particles with respect to one another in a soil mass is known as Soil structure.

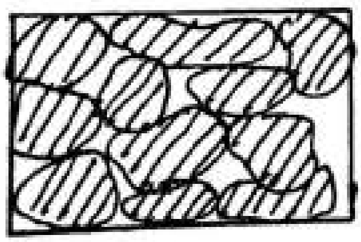
The structure is dependent on the size, shape of the grains and the minerals of which grains are formed.

Engineering properties and the behavior of both coarse grained and fine grained soils depend upon the soil structure.

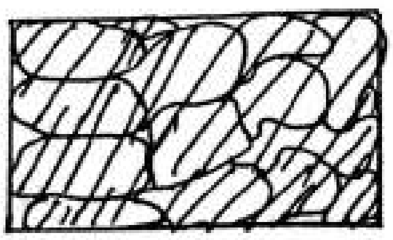
The following types of structures are usually found.

1. Single grained structure
2. Honey Comb Structure
3. Flocculated Structure
4. Dispersed Structure

1. Single Grained Structure :-



Loose

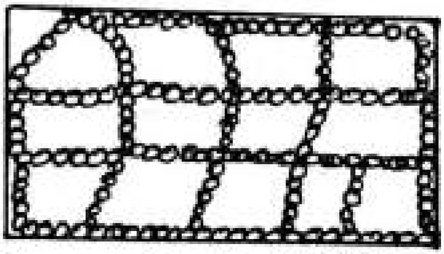


Dense

- coarse grained soils larger than settle out of suspension in water as individual grains independent of other grains.
- The weight of the grain causes them to settle and role to positions of Equilibrium, practically independent of other grains. This arrangement of single grains is called single grain structure.
- The major force causing their deposition is gravitational force. & the surface forces are small
- The particles have no tendency to adhere to one another. Any disturbance of single grain structure tends to form a similar stable structure again.
- They may be deposited in a loose state having a high void ratio or in a dense state having a low voids ratio.

2. Honey Comb Structure :-
- Such a structure exists in grains of silt or rock flour. This structure usually develops when the particle size is b/w 0.0002mm and 0.02mm.
- when such particles settle out of suspension, more or less has single grains, but are so small molecular forces at the contact surfaces as the grains come in contact at bottom are large enough compared to submerged weight which prevents the grains from rolling immediately into positions of equilibrium among the grains already deposited.
- The grains coming in contact are held until miniature arches are formed, bridging over relatively large void spacing and forms honey comb structure.

- Each cell in honey comb structure is supposed to made up of numerous single material grain.
- The structure so formed has high void ratio & is capable of carrying relatively high heavy loads without excessive volume change.



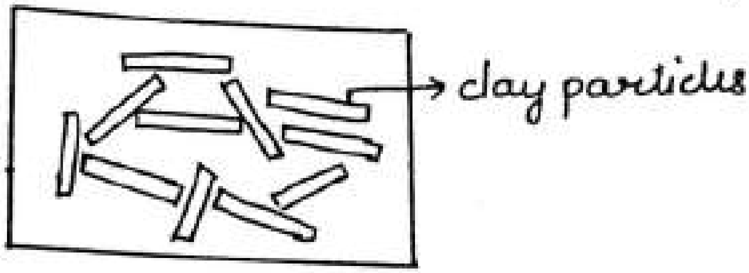
Honey Combed Structure

3) Flocculent Structure :-

Flocculated structure occurs in clay.

- The clay particles have large surface area and therefore, the electrical forces are important in such soils.
- The clay particles have a -ve charge on the surface and +ve charge on the edges.
- Inter particle contact develops between the +vely charged edges & the negatively charged faces. This results in a flocullated structure.
- The degree of flocculation of a clay deposit depends upon the type & concentration of clay particles, and the presence of salt in water.
- clay settling out in a salt water solution have more flocculent structure than clay settling out in a fresh water solution.
- Salt water acts as an electrolyte and reduces the repulsive forces between the particles

In general, the soils in a flocculated structure have a low compressibility, a high permeability and high shear strength.



Flocculated Structure

4) **Dispersed Structure :-**

Dispersed structure develops in clay that have been remoulded.

- The particles develop more or less a parallel orientation.
- clay deposits with flocculated structure when transported to other places by nature or man gets remoulded.
- Remoulding converts the edge to face orientation to face to face orientation.
- The dispersed structure is formed in nature when there is a net replusive force between particles.
- The soil in dispersed structure generally have a low shear strength, high compressibilit and low permeability
- A clay having flocculent structure have high Void ratio and when pressure is applied it transforms to a dispersed structure.

Clay Minerals in Soil ...

* Formation of clay minerals

The combination of two sheets of silica and Gibbsite in different arrangement and conditions leads to the formation of different clay minerals such as

1. Kaolinite minerals
2. Montmorillonite mineral
3. Illite mineral

1. Kaolinite mineral :-

Kaolinite is the most common mineral of the Kaolinite group of minerals.

- Its basic structural unit consists of alumina sheet (G) combined with a silica sheet (S).
- This consist of building blocks of alumina sheet / Gibbsite and silica sheet arranged as shown in fig.
- Tips of the silica sheet and one base of alumina sheet form a common interface.
- The total thickness of the structural unit is about 7A° (1A° = 10^{-10} m or 10^{-7} mm)
- The Kaolinite mineral is formed by stacking one over the other.
- The structural units join together by hydrogen bond. which develops between the oxygen of silicon sheet and hydroxyles of alumina sheet. As the bond is fairly strong. the mineral is stable.

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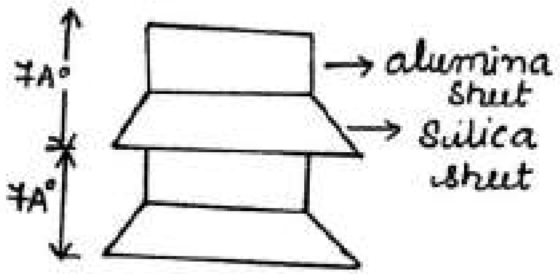
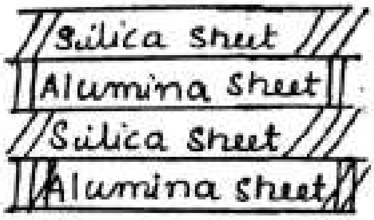
- Moreover, water cannot easily enter between the structural units. So, expansion or swelling will not take place.
- The thickness of Kaolinite mineral is about 100 to 1000A°





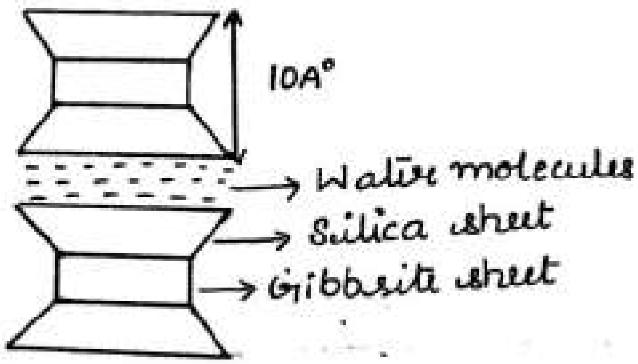
Structure of Kaolinite mineral

2) Montmorillonite mineral

This is the most common of all clay minerals in expansive clay soil.

- The basic structural unit consists of an alumina sheet sandwiched between two silica sheets.
- The thickness of each unit is about 10A°.
- Successive structural units are stacked one over another, like leaves of a book.
- It composes of unit made of two silica tetrahedral sheet with a central alumina sheet.
- There is a very weak bonding between the successive sheets and water may enter between the sheets causing the minerals to swell.
- The spacing between silica-alumina-silica sheets depend upon the amount of available water to occupy the space.

- The soil containing a large amount of mineral montmorillonite exhibits high shrinkage and high swelling characteristics
- The gibbsite sheet in montmorillonite mineral may contain Iron or magnesium instead of aluminium. In addition, the silicon atoms of tetrahedra may interchange with aluminium ions. These structural changes are called amorphous changes. This results in giving the mineral a residual negative charge. It attracts water to form an adsorbed layer.



Structure of Montmorillonite

3) Illite Mineral :-

The basic structural unit is similar to that of montmorillonite mineral.
However, the mineral has properties different from montmorillonite due to following reasons.

a) There is always a substantial amount of amorphous changes of silicon by aluminium in silica sheet. Consequently, the mineral has a larger -ve charge than that of montmorillonite.

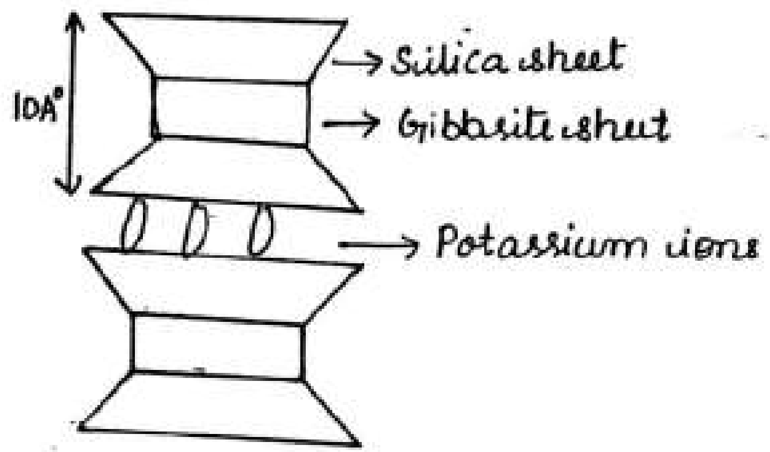
b) The link between different structural units is through non exchangeable potassium (K^+) and not through water.
This bonds the units more firmly than in montmorillonite.

c) The bond with potassium is stronger than water bond of the montmorillonite.

d) Illite swells less than montmorillonite. However, swelling is more than Kaolinite.

e) The Space between different structural unit is much smaller than montmorillonite, as the potassium ions just fit between the silica sheet surfaces.

f) The thickness of Illite mineral is 50-500A°.



Structure of Illite Mineral

* **Structure of clay mineral**

The two building blocks involved in the formation of clay minerals are

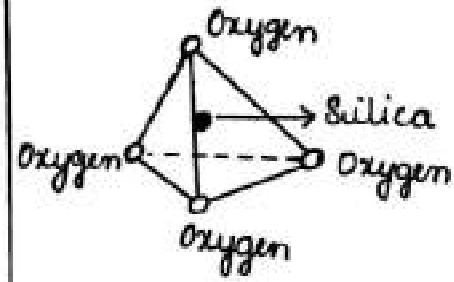
1. Tetrahedral unit
2. Octahedral unit.

1. **Tetrahedral unit** (Silica tetrahedral Sheet)

Tetrahedral unit consist of a silicon atom (Si^{4+}) surrounded by four oxygen atoms (O^{-}) forming a shape of tetrahedron.

- Oxygen atoms are at the tips of the tetrahedron. whereas the silicon atom is at the centre. A number of tetrahedron unit combine to form a sheet.
- The oxygen at the bases of all the units lie in the common plane.
- The O_2 atoms are negatively charged with two negative charge each and silicon with four positive charge.
- Each of the three oxygen atoms at the base shares its charges with the adjacent tetrahedral unit.
- The sharing of the charges leaves three -ve charges at the base per tetrahedral unit along with two negative charge at the tip altogether five negative charges. to balance the four positive charges of the silicon ions.
- The process of sharing of oxygen ions at the base with neighbouring unit leaves a net charge of -1 per unit.

Symbolic Representation of silica sheet



Silica tetrahedral unit

2) Octahedral unit :-

- The octahedral unit consists of six hydroxyl ions (OH^-) forming a configuration of an octahedron and having one aluminium or mg or Fe. atom at the centre.
- These units are bound together into a sheet like structure.
-

Scanned by CamScanner

* Adsorbed water

The water held by electrochemical forces existing on the soil surface is known as adsorbed water.

- As the adsorbed water is under the influence of electrical forces, its properties are different from normal water.
- A soil particle carries a net negative charge on its surface and water molecules is a permanent dipole. Therefore water molecules adjacent to soil particle gets attracted by it.
- Because of net negative charge on its surface a soil particles can also attract a no of other exchangeable cations like those of sodium, calcium, magnesium, potassium etc and these in turn attract nearby dipolar water molecules.
- Thickness of adsorbed layer depends upon mineralogy, composition of soil particle, specific surface of soil.
- Adsorbed water has high boiling point and greater viscosity.
-

* **Base Exchange Capacity :-**

Electrolytes dissociate when dissolved in water into positively charged cations and negatively charged anions. The cations attracted to the negatively charged surface of the soil particles are not strongly attracted. These cations can be replaced by other ions and are therefore known as exchangeable ions. The phenomenon of replacment of cations is called cation exchange or base exchange.

- The net negative charge on the mineral which can be satisfied by exchangeable cations is termed as cation exchange capacity or base exchange capacity. In other words, base exchange capacity is the capacity of the clay particles to change the cations adsorbed on the surface.

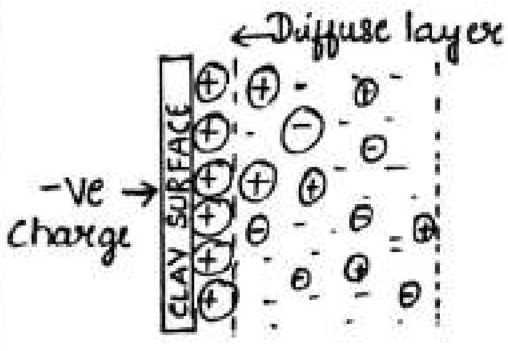
- Base Exchange capacity is expressed in terms of the total no of positive charges adsorbed per 100gm of soil.

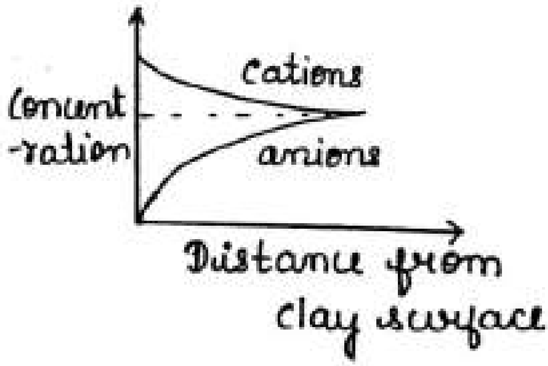
- The Base exchange capacity of clay depends upon the pH value of the water in the Environment. If the water is acidic, the base exchange capacity is reduced.

* Electrical Diffuse double layer

Water molecules consist of both +ve & -ve charge & hence they behave like dipoles, thus water molecules may be adsorbed on the surface of -vely charged clay particles.

- These water molecules are strongly oriented at the surface due to high electrical forces attracting them to the clay surface.
- More than one layer of water molecules can be adsorbed in this way but as the molecule distance from the clay surface increases the attractive force decreases and degree of orientation also decreases.
- Adsorbed water (water molecules strongly held by electrical force) affects the behaviour of the clay particles when subjected to external load.
- Since it occurs between the particle surfaces to drive off this water the clay particle must be heated to about 200°C which indicates that the bond between the water molecule and the clay surface is greater. This layer of water molecules which surrounds the clay particles constitutes Electrical diffuse double layer.





* Indian Standard Soil classification of Soil

Plasticity chart

Soils are divided into three broad divisions.

1. Coarse grained soil - where 50% or more of the total material by wt retained on 75μ IS sieve.

The coarse grained soil include Gravels (G) & Sand (S)

* Gravel :- More than half the coarse fraction are retained on 4.75 IS sieve

* Sand :- more than half the fraction are passing through 4.75mm IS Sieve.

The Gravel and Sand are further subdivided as

i) W = Well graded with fairly clean materials
ii) C = Well graded with Excellent clay binder
iii) P = Poorly graded with fairly clean materials
iv) F = coarse materials with fines

2) Fine grained Soil :-

More than half the materials by weight are smaller than 75μ.

The fine grained soil are further divided into three sub divisions

M = Inorganic silt and fine Sand
C = Inorganic clay
O = organic Silt & clay

The above groups of the fine grained soil are further divided into three subdivisions depending upon the values of the Liquid limit.

a) Silts and clay of low compressibility (L)
These soils have a liquid limit less than 35%.

b) Silt and clay of high compressibility (H)
If the liquid limit is between 35% - 50%.

c) Silt & clay of medium compressibility (I)
If the liquid limit is more than 50%.

Combination of various group symbols indicate the type of soil as follows.

GW ⇒ Well graded Gravel
GC ⇒ clayey gravel
SC ⇒ clayey Sand
ML ⇒ Inorganic silt of low compressibility
CH ⇒ Inorganic clay of high compressibility.

* Plasticity chart

Casagrande derived a plasticity chart which is useful in identifying and classifying fine grained soil.

In this chart, the ordinate indicates the plasticity Index and abscissa indicates the liquid limit.

The Equation of the line represented as A-line Given by the equa $I_P = 0.73(W_L - 20)$

- If the points lie below the A-line, the soil may be classified as inorganic silt (M), organic silts or Organic clay (c).
  These soil may be again of low, medium or high compressibility depending on their liquid limit values as $W_L < 35\%$, or between 35-50% and greater than 50%.

- If the points lie above the A-line, they are representatives of Inorganic clay again there may be of low, medium or high compressibility depending on the liquid limit values such as $W_L < 35\%$, or between 35-50% and greater than 50%.

Compaction Of Soil

Introduction

The process of compaction involves expulsion of air from soil. It is a relatively low cost for using soil as a construction material.
If properly placed & compacted, the resulting soil mass has better strength than many natural soil foundations.
For the purpose of supporting highways or buildings or for retaining water as in earth dam, the soil material must be compacted properly to support the structure.
Properly compacted clay soils will develop relatively high strength and low permeability which may be desirable feature for earth dams.
An example of compaction is the reduction in voids produced in a layer of the subgrade by a steel tyred roller during construction.

Definition:- compaction may be defined as the process by which the soil particles are artificially rearranged & packed together into a state of closer contact by mechanical means in order to decrease its porosity & there by increases its dry density. This is usually achieved by dynamic means such as tamping, rolling or vibration.

* Principle of Compaction :-

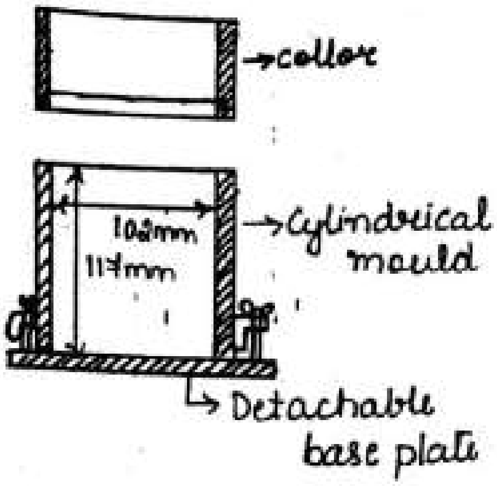
1. In case of medium cohesion soils, the compaction is to be done by means of rolling.
2. For cohesionless soil, the compaction is to be done by means of vibration.
3. The degree of compaction of a soil is characterized by its dry density.
4. The degree of compaction depends upon the moisture content, amount of compactive efforts, & the nature of soil.
5. A change in moisture content or compactive effort brings about a change in density.
6. Compaction decreases the tendency for settlement of soil compaction brings about low permeability of the soil.

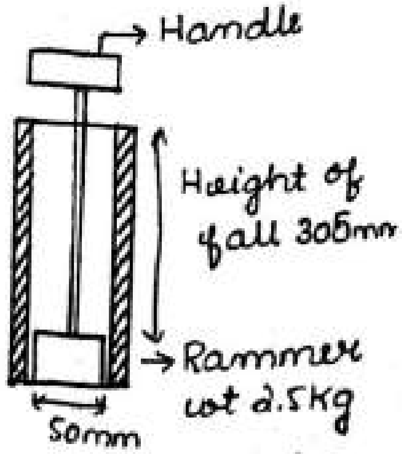
* Standard & Modified Proctors compaction test

This test was developed by B.R. Proctor (1933) in connection with the construction of earth dams in california (U.S.A).

The Apparatus consist of

i) a cylindrical mould of internal dia 108mm & an effective height of 117mm, with a volume of 0.945 litres.
ii) A detachable collar of 50mm effective height.
iii) A detachable base plate
iv) Rammer of weight 2.5kg with a height of fall of 30.5cm.





* Procedure :-

1. About 3Kg of dry soil, with all lumps pulverized and passing through 4.75mm IS sieve is taken

2. The quantity of water to be added in the first trial depends upon the probable optimum water content for the soil. The initial water content may takes as 4% for coarse grained soil & 10% for fine grained soil.

3. The empty mould with base plate & without collor is weighed (W1).

4. The inner surface of mould, base plate and collor are greased. The soil mixed with water thoroughly.

5. The Soil is placed in mould & compacted in three uniform layers, with 25 blows in each layer Blows are maintained uniform & vertical, height of drop is controlled.

6. After compacting each layer, top surface is scratched to maintain integrity between layers.

7. The height of top layer is so controlled that after compaction, soil slightly protrudes into collar. Excess soil is trimmed.

8. The weight of the mould, base plate & the compacted soil is taken (W_2) gm.

9. A Representative sample from the middle portion is taken for the determination of water content. The Procedure is repeated with increasing water content.

10. The number of trials shall be atleast 6 with a few after the decreasing trend of bulk density.

The bulk unit weight (γ) of the soil is

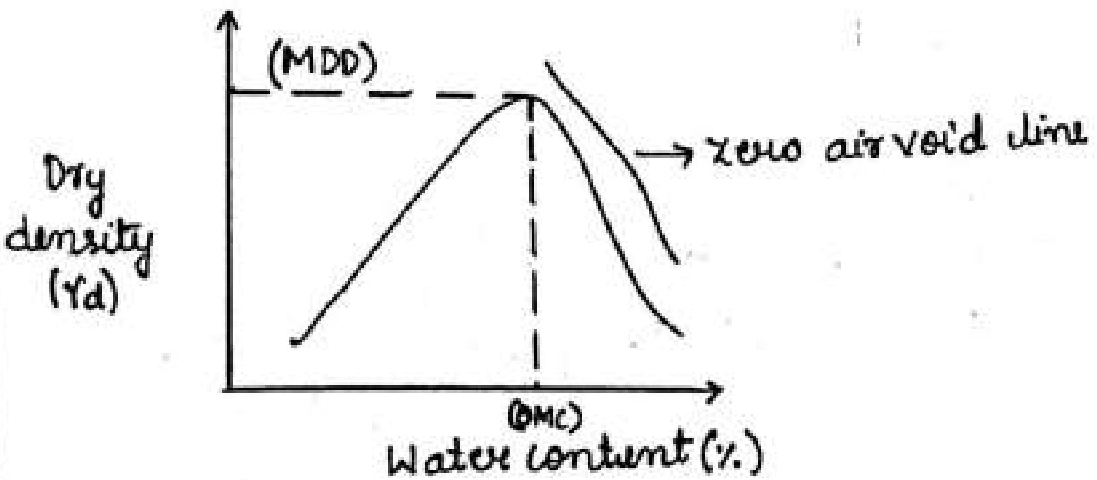
$$\gamma = \frac{W}{V} \qquad [W = W_2 - W_1]$$

The dry unit (γ_d) is

$$\gamma_d = \frac{\gamma}{1+w} \qquad \text{where } w = \text{water content}$$

Draw a compaction curve, moisture content vs dry density as shown in fig.

The optimum moisture content (OMC) & the corresponding maximum dry density is found from the graph.



* Modified Proctor's compaction test

In early days, compaction achieved in field was relatively less. With improvement in knowledge & technology, higher compaction became a necessity in field. Hence modified compaction became relevant.

Apparatus :-

The test equipments consist of

i] Cylindrical metal mould, having an internal diameter of 101.5mm & internal effective ht = 117mm

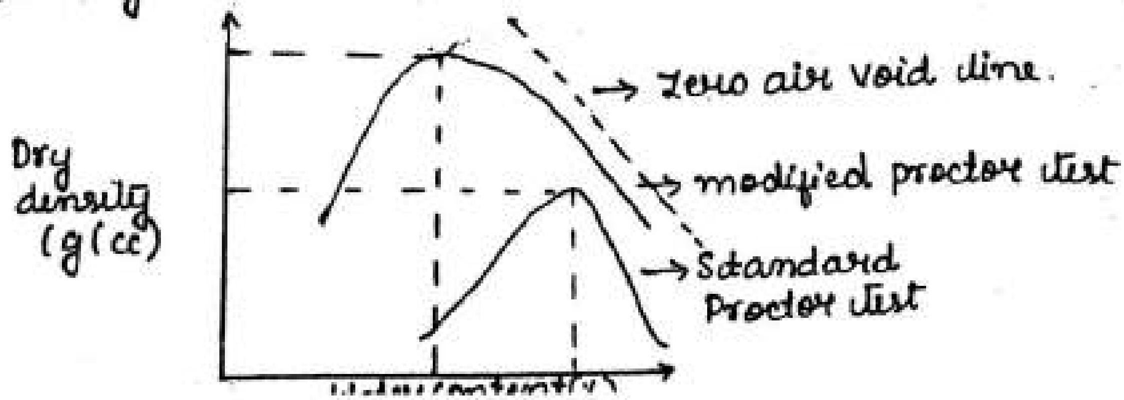
ii] Detachable base plate

iii] Collar of 50mm effective ht

IV] Rammer of wt 4.5Kg with a fall of 450mm.

- In this test, the soil is compacted in the standard proctor mould, but in 5 layers, each layer is given 25 blows of 4.5kg rammer dropped from a height of 450mm. All the procedure remains same as standard proctor test.

- In the modified proctor test, the water content, dry density curve lies above the standard proctor test curve & has its peak relatively placed towards left. Thus for a same soil, the effect of heavier compaction is to increase in the maximum dry dry density & to decrease the optimum water content.



* Distinguish b/w Standard & modified compaction test

	Standard proctor test	Modified proctor test
1.	Height of drop = 305mm	Height of drop = 450mm
2.	Weight of rammer = 2.5Kg	Weight of rammer = 4.5Kg
3.	25 blows/layer	25 blows/layer
4.	No of layers = 3	No of layers = 5
5.	Compactive energy = 605160 N-mm/m³	compactive energy = 2726000 N-mm/m³
6.	Mould size = 945ml	Mould size = 945ml

* Compactive Energy in Standard Proctor's Test

No of blows = 25
Height of drop = 0.305m
Weight of rammer = 2.5Kg ≈ 24.5 ≈ 25N
No of layers = 3
Volume of mould = 945ml ≈ $945 \times 10^{-6} m^3$

$$\text{Compactive energy} = \frac{\text{No of blows} \times \text{No of layers} \times \text{weight of rammer} \times \text{Height of drop}}{\text{Volume of mould}}$$

= 605158.73 N-mm/1000ml (m^3)

* Compactive Energy in Modified Proctor's Test

No of blows = 5
No of layers = 25
Weight of Rammer = 4.5 kg ≈ 45 N
Height of drop = 450 mm
Volume of mould = $945 \times 10^{-6} m^3$

Compactive Energy = 2726000 N-mm | 1000 ml (m^3)

Compactive Energy in Modified Proctor's test is 4.5 times bigger than Standard Proctor's test.

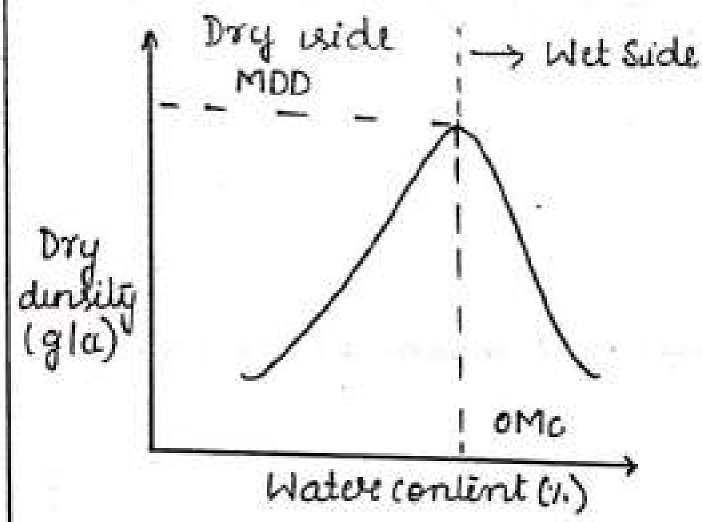
* Factors Affecting Compaction

1. Water content
2. Effect of amount of compaction
3. Effect of method of compaction
4. Effect of type of Soil
5. Effect of addition of admixtures

1. Effect of Water Content :-

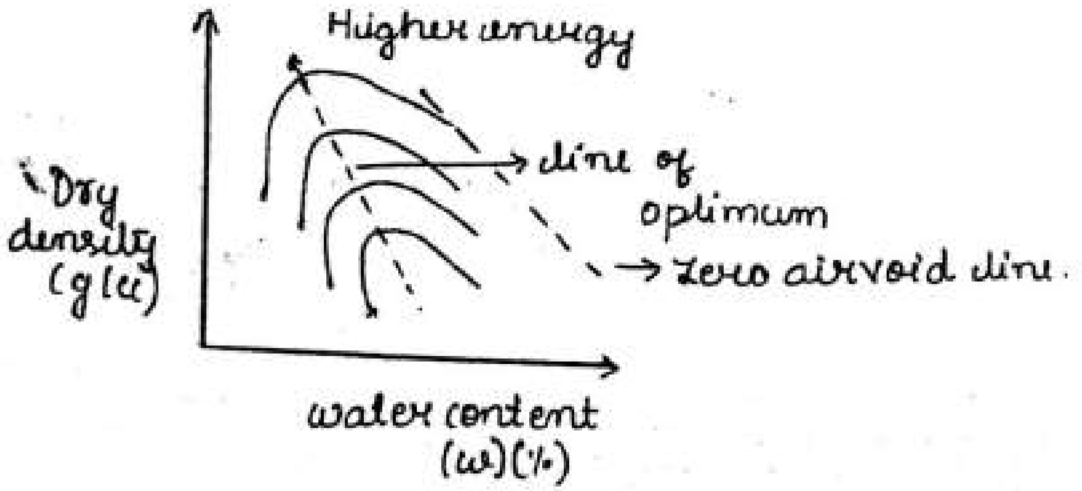
- It has been observed that as the water content increases, compacted density goes on increasing till a max density is reached and further addition of water content decreases the density.
- The maximum dry density achieved is called MDD & the corresponding water content is called OMC.

- At lower water contents than OMC, soil particles are held by electrical forces that prevents the development of diffused double layer leading to low inter particle repulsion.
- Increase in water content results in expansion of double layer and reduction in net attractive force between particles. Water replaces air in void space.
- After OMC is reached, air voids remains constant. Further increase in water content increases the void space & further decreases dry density.



2] <u>Amount of Compaction</u>:-

- Amount of compaction greatly affects the maximum dry density & OMC of the given soils.
- The effect of increasing Compactive effort is to increase MDD & reduce OMC.
- However there is no linear relationship b/w compactive effort & MDD.



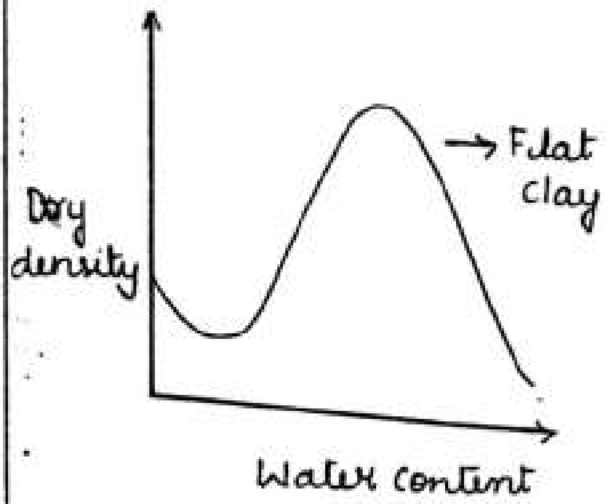
3] **Method of compaction :-**

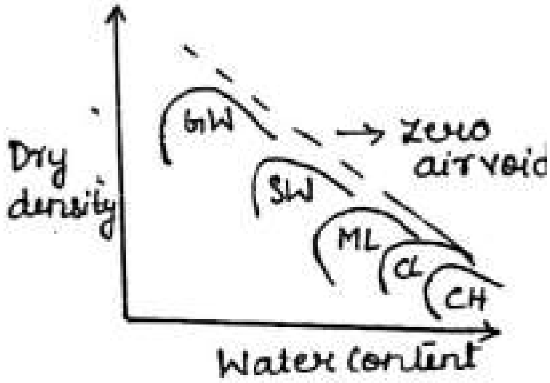
The Dry density achieved by the soil depends on the following characteristics of compacting method.

i] Weight of compacting equipment
ii] Type of compaction
iii] Area of contact of compacting equipment with Soil
iv] Time of exposure

4] **Types of Soil :-**

- Maximum density achieved depends on type of soil.
- Coarse grained soil achieves higher density at lower water content & fine grained soil achieves lesser density, but at higher water content.





5] Addition of Admixtures :-

The effect of adding admixture is to stabilise the soil.
In many cases, they accelerate the process of densification.

* Effect of Compaction on Soil properties :

1. change in structure of Soil
2. Permeability
3. Shrinkage
4. Swelling
5. Pore pressure
6. compressibility
7. Stress - Strain characteristic
8. Shear strength.

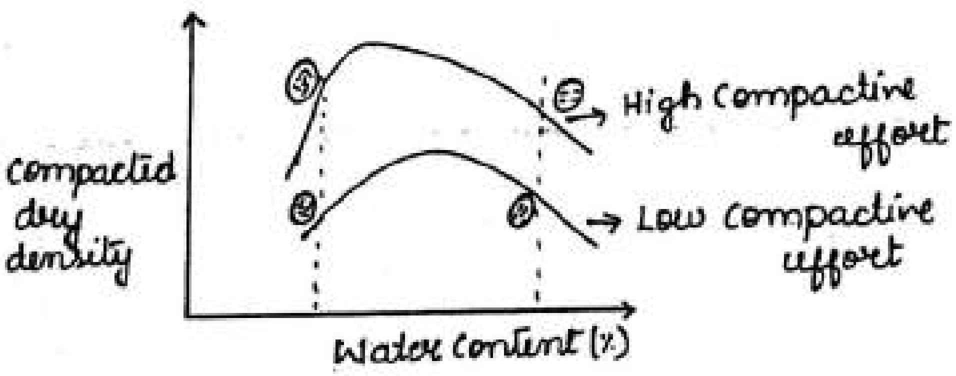
1. Change in Structure of Soil

The structure of a soil during compaction depends upon i) type of Soil
ii) moulding water content
iii) Type & amount of compaction.

a) Coarse grained Soil - maintains a single grained structure at any possible Void ratio.

b) Composite soil - The structure of composite soil depends upon the relative proportion of coarse particles & fines i.e there can either be coarse grained or cohesive structure.

c) Fine grained Soils: In fine grained soil on dry side optimum, the structure is flocculated. The particles repel i.e density is less. Addition of water increases lubrication & transforms the structure in to dispersed structure.



2. Permeability :-

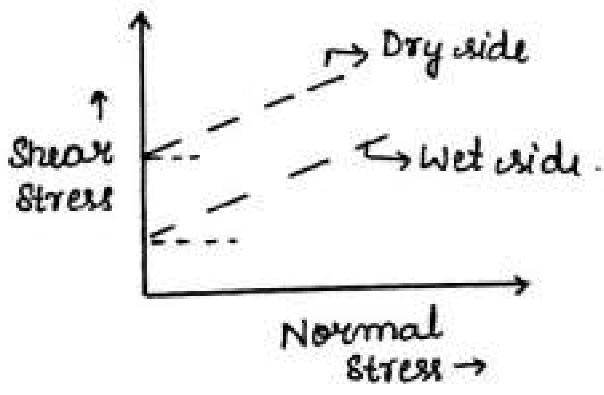
- As the dry density increases due to compaction, the voids go on reducing and hence the permeability goes on decreasing.
- At same density, fine grained samples compacted dry of optimum are more permeable than those compacted wet of optimum. This is so

because these soils will have flocculated structure when compacted dry of optimum and have dispersed structure when compacted wet of optimum.

- For a given void ratio, greater the size of individual pores, greater is the permeability
- As the compactive effort is increased, the permeability of soil decreases because of the increased dry density & better orientation of particles.

3) <u>Shear strength</u>:

- In general, effect of compaction is to increase the the number of contacts resulting in increased shear strength, especially in granualar soil
- In clay, shear strength depends on dry density, moulding water content, soil structure, method of compaction, strain level, drainage condition etc
- Shear strength of cohesive soil compacted dry of optimum will be higher than those compacted wet of optimum (dispersed structure)



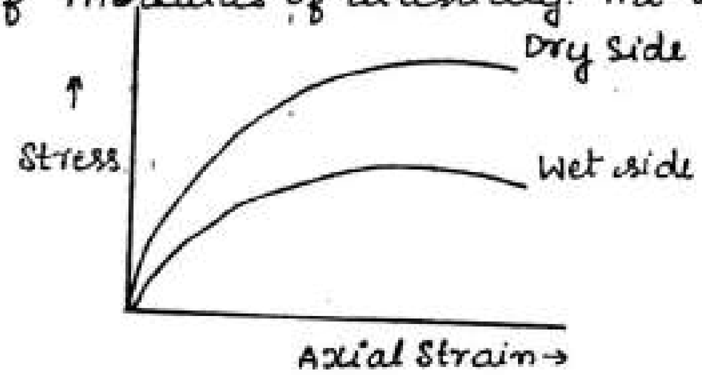
4. <u>Pore Pressure</u>:- Soil compacted wet of optimum has greater pore water pressure than the soil compacted dry side of optimum.

5. <u>Density</u>:- Effect of compaction is to reduce the voids by expelling out air. This results in increasing the dry density of soil mass.

5. **Stress - Strain characteristics :-**

For a given soil, sample compacted dry side of optimum has a steeper stress-strain curve & hence has a higher modulus of elasticity than one which is compacted wet of optimum at same density.

- Soil compacted dry of optimum show brittle failure. like dense sand. The soil compacted wet of optimum will have flatter curve and correspondingly lower value of modulus of elasticity. The failure in this type is a plastic one.



6. **Shrinkage & swelling :-**

For the same density, soil sample compacted dry of optimum shrinks appreciably less than the sample compacted wet side of optimum.

Soil compacted dry of optimum has greater swell & swell pressure than the soil compacted wet of optimum. becoz of random orientation & deficiency of water.

7. **Compressibility :-**

At low pressure, soil compacted wet of optimum show more compressibilty than that of dry side. But at higher pressure, behaviour is similar.

- However, the compressibility of the soil depends upon a no of factors such as degree of saturation method of compaction etc.

* Field Compaction Control

It is extremely important to understand the factors affecting compaction in field & to estimate the correlation between laboratory & field compaction.

Field compaction control depends on

i. Placement water content
ii. Types of equipment for compaction
iii. Lift Thickness
iv. No. of passes based on soil types & degree of compaction desired.

Placement water content is the water content at which the ground is compacted in the field. It is desirable to compact at or close to optimum moisture content achieved in laboratory so as to increase the efficiency of compaction. However, in certain jobs the compaction is done at lower than or higher than OMC (1 to 2%) as depending on the desired function as detailed in the table.

	Dry Side	Wet side
1.	Highway embankment compacted dry of optimum to achieve high strength, low volume compressibility & good resistance to deformation	cohesive subgrades under pavement compacted wet optimu to eliminate swelling: swelling pressure upo submergence.
2.	High earth dams compacted 1 to 2.5% less than OMC to reduce Pore water pressure development	Impervious core of earth dam compact on wet side to achie low permeability, gre

* Compaction control in field

There are many variables which control the Vibratory compaction or densification of soils.

- Characteristics of ...

1. Mass, size
2. Operating frequency & frequency range.

- Characteristics of the Soil

1. Initial density
2. Grain size & shape
3. Water content

- Construction Procedure

1. Number of passes of the roller
2. Lift thickness
3. Frequency of Operation vibrator
4. Towing speed.

* Degree of Compaction

Relative compaction | Degree of compaction

$$= \frac{\gamma_{d\text{-field}}}{\gamma_{d(max\text{-}laboratory)}} \times 100$$

Typhical required R.C >= 95%

* Methods of compaction used in the field.

1. Tampers | Rammers
2. Rollers
3. Vibratory compactors
4. Dynamic compaction

Types of field compaction Equipment

There are different types of field compaction equipments.

1. Smooth wheeled steel drum roller
2. Pneumatic Tyred roller
3. Sheepfoot Rollers
4. Impact Rollers
5. Vibrating Rollers
6. Hand operated Vibrating plate & rammer compactor

1 Smooth Wheeled Steel drum rollers

a. capacity 20KN to 200KN
b. Self propelled or towed
c. Suitable for well graded sand, gravel, silt of low plasticity.
d. Unsuitable for uniform sand, silt, sand & soft clay.

- These are widely used in road construction and construction of different types of embankment.

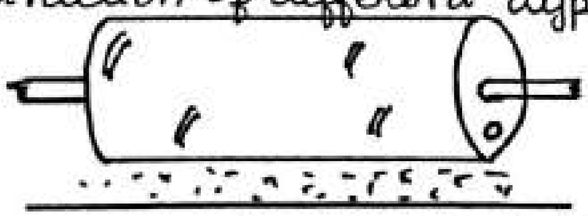


fig: Smooth wheeled Roller

2] Pneumatic Tyred Roller:-

- Usually two axled carrying rubber tyred wheel for full width of track.
- Dead load is added to give a weight of 10 to 40k

- Suitable for most coarse & fine soils.
- Unsuitable for very soft clay & high variable soil.
- widely used for compacting subgrade, bases & even in landfill area.

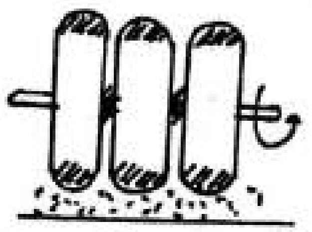


Fig: Pneumatic Tyred roller

3. Sheep Foot Roller :-

1. Self propelled or towed
2. Drum fitted with projecting club shaped feet to provide kneading action
3. Weight of 50 to 80KN
4. Suitable for fine grained soil, sand & gravel with considerable fines.
5. widely used for compacting core of earthen dams & core of canal embankments.

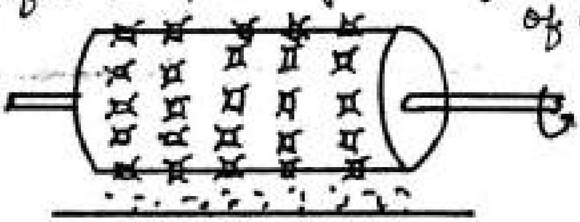
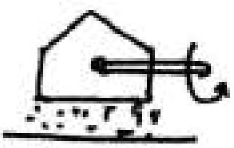


Fig: sheep foot roller

4. Impact roller

- compaction by static pressure combined with impact of pentagonal roller.
- Higher impact energy breaks soil lumps & Provides kneading action.



5 Vibrating drum

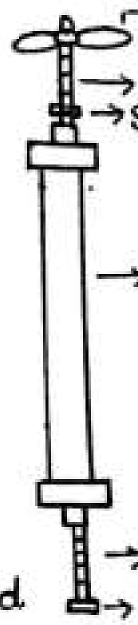
1. Roller drum fitted with Vibratory motion
2. Levels & smoothens outs
C. Weight from 300 to 400N, Suitable for Sandy soil used to compact embankment, sand beds & most effective in granular soils.

6. Plate & Rammer compactor

It is used for backfilling Trenches. Used for Smaller construction & in less accessible locations.

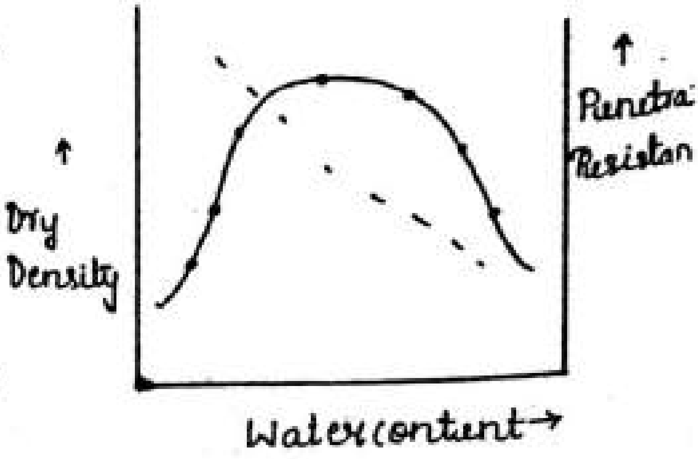
* Proctor Needle Test :-

In order to find out the degree of compaction achieved, as the work is in progress, rapid method of testing is to be adopted.



- For rapid determination of water content, the proctor needle method has been in use for long time.
- The Proctor's needle consist of a needle point attached to a needle shrank which in turn is attached to spring loaded plunger.
- Needle points of varying c/s areas are available to measure a wide range of penetration resistance. The penetration resistance is read on calibrated stem
- A calibrated curve is obtained in the laboratory by plotting penetration resistance against water content

- The penetration resistance is measured by inserting the proctor needle in the soil compacted in the proctor mould & corresponding water content at different trials are to be found & obtain the curve penetration resistance vs water content.



- The obtain calibration curve is used in the field to obtain water content of the compacted soil.
- The bulk density of the compacted soil in the field can be determined by Sand replacement or core cutter method.

Using bulk density, calculate the dry density in field $\gamma_d = \frac{\gamma}{1+w}$

The degree of compaction achieved in the field is measured by

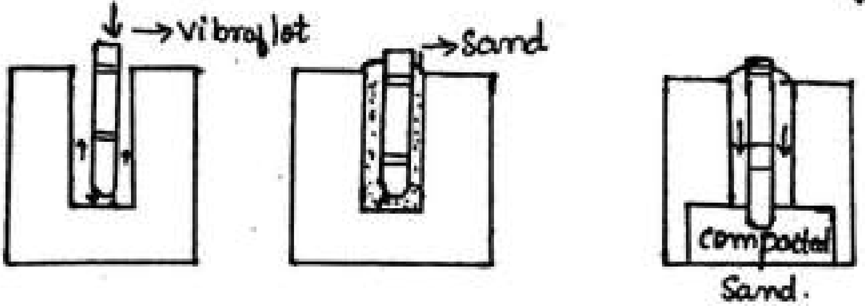
$$= \frac{(\gamma_d)\ \text{field}}{(\gamma_d\ \text{max dry density in lab})} \times 100\%$$

* **Dynamic Compaction:-**

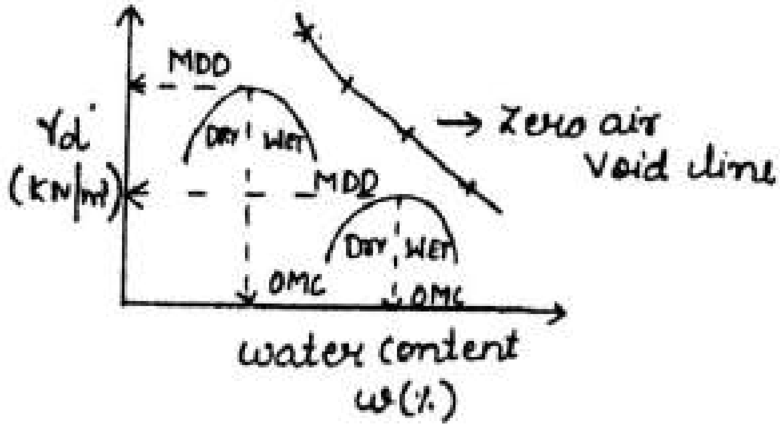
- This is used to compact both cohesive & cohesionless soil.
- In this compaction, a crane is used to lift a concrete or steel block weighing upto 500KN upto a height of 40 to 50m
- From this height, the block is allowed to fall freely on the ground surface.
- The process is repeated over the area.
- Soil at surface is disturbed. It is then refilled and levelled. Depth of compaction is upto 12m.
- This method is quick & produces uniform settlement.

* **Vibrofloatation :-**

- It is used for compacting granular soil in the field.
- It consists of cylindrical tube, eccentric weight & water jet.
- Vibrofloat is sunk into the ground and vibratory motion develops an enlarged hole.
- Hole is backfilled with suitable granular soil
- The spacing of vibroflotation depends on initial density & desired density.
- Usually 2m spacing can enhance relative density upto 70%.



Zero Air Void line :-



A line which shows the water content, dry density relation for the compacted soil containing a constant %age air voids is known as an Air Void line and can be established from the following relationship

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

$$\gamma_d = \frac{G\gamma_w}{1+\frac{wG}{S_r}} \qquad [wG = S_r.e]$$

If the values of water content and dry wt are plotted, then the curve obtained is known as air void curve.

A curve showing the dry unit wt as a function of water content for soil containing no air voids is called zero air void curve or Saturation line. can be obtained from the following equⁿ

$$\gamma_d = \frac{G\gamma_w}{1+\frac{wG}{S_r}}$$

For 100% saturation ($S_r=1$) Zero Air Void

For 90% saturation ($S_r = 0.9$

Difference between compaction & consolidation

	COMPACTION	CONSOLIDATION
1.	It is almost a instantaneous phenomenon	It is a time dependent phenomenon
2.	Soil is in partially saturated condition	Soil is in fully saturated condition
3	Densification is due to reduction in the air void at a given water content	Volume reduction is due to reduction in water voids
4.	compaction can be achieved artificially	consolidation can be achieved by the application of load or naturally.
5.	Dynamic loading is commonly applied	Static loading is commonly applied.
6.	Improves the bearing capacity & settlement characteristics	Improves bearing capacity and settlement characteristics.
7.	Relatively quick process	Relatively slow process
8.	Compaction is complex phenomenon	consolidation is a Simple phenomenon.

Flow of water through Soil

Introduction

Water strongly affects engineering behaviour of most kind of soils and water is an important factor in most geotechnical engineering problems. Hence it is essential to understand basic principles of flow of water through soil medium. Flow of water takes place through interconnected pores between soil particles is considered in one direction.

* Permeability

Flow of water in soil media takes place through void spaces which are apparently connected. Water can flow through the densest of natural soils. Water does not flow in a straight line but in a winding path.
However in soil mechanics, flow is considered to be along straight line at an effective velocity.
The velocity of drop of water at any point along its flow path depends on the size of the pore & its position inside the pore.

Since water can flow through pore spaces in the soil, hence soil medium is considered to be permeable. Thus the property of a porous medium such as soil by virtue of which water can flow through it is called its Permeability. In other words, the ease with which water can flow through a soil mass is called Permeability.

* **Darcy's Law**

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

$$q = KiA \quad \text{or} \quad V = \frac{q}{A} = Ki$$

where q = discharge per unit time
A = c/s area of soil mass
i = hydraulic gradient
K = Darcy's co-efficient of Permeability
V = Velocity of flow

If a soil sample of length 'L' & c/s area 'A' is subjected to different head of water $h_1 - h_2$, then we have

$$q = K\left(\frac{h_1 - h_2}{L}\right)A$$

* Assumptions made in Darcy's Law

1. The flow is laminar that is, flow of fluids is described as laminar if a fluid particles flow follows a definite path and does not cross the path of other particles.
2. Water & soil is incompressible that is, continuity equation is assumed to be valid.
3. The soil is saturated.
4. The flow is in steady state i.e. flow condition do not change with time.
5. In the case of fine grained soil, the velocity of flow is less & hence flow may be turbulent.

* Limitations of Darcy's Law or Validity

1. In the case of coarse grained soil where pore dimensions are larger, there will be great possibilities of flow becomes turbulent.
2. Flow through sand remains laminar and darcy's law is valid so long as Reynold no equal to less than 2000.
3. For ground water, flow occurring in nature, the darcy's law is generally within its validity limits.

* Co-efficient of Permeability - is defined as the average velocity of flow that will occur through the total c/s area of soil under unit hydraulic gradient.

It is usually expressed as cm/sec.

* Determination of Co-efficient of Permeability

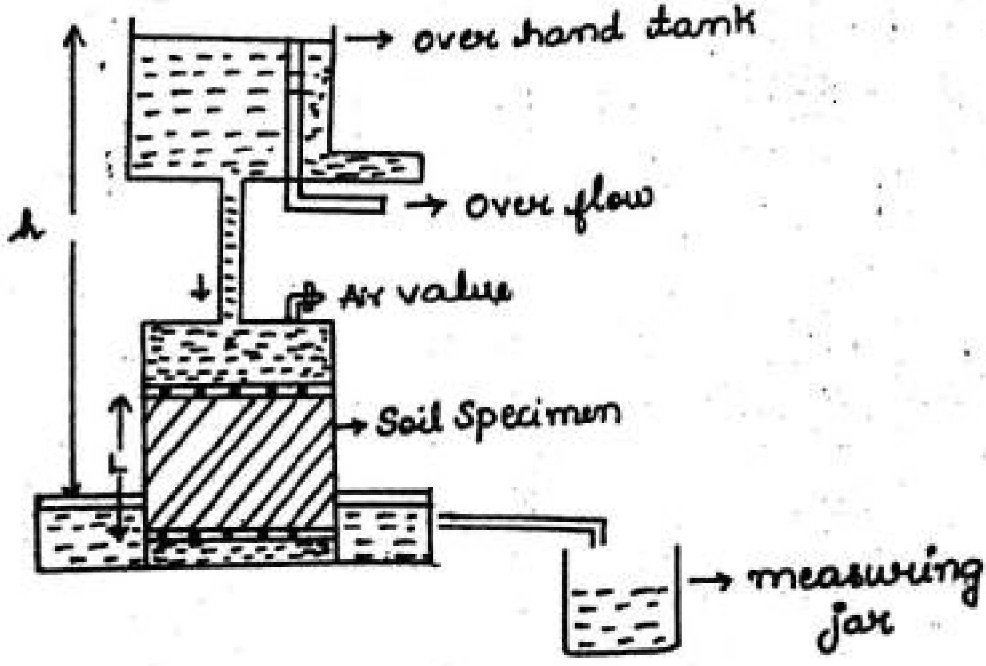
a) Laboratory method

1. Constant head permeability test
2. Falling head permeability test

b) Field test

1. Pumping out test
2. Pumping in test

- Constant Head Permeability Test



- The experimental setup for constant head test is as shown in fig.
- The constant head test is suitable for more permeable coarse grained soil.
- The air value is provided to drive out air bubbles if any observed in the transparent rubber tubing through which water flows from overhead tank to the soil specimen.
- After ensuring that the soil specimen is fully saturated & the flow has become steady, the outflow is collected in a measuring jar.

The quantity of water 'Q' flowing through the soil specimen in a known interval of time 't' is found.

Applying Darcy's Law,

$$q = KiA \qquad \left[q = \frac{Q}{t} \right]$$

$$\frac{Q}{t} = K.\frac{h}{L}.A$$

$$\boxed{K = \frac{QL}{A.ht}}$$

where,

A = Area of c/s of soil specimen
L = Length of specimen
h = constant head
t = time 'sec'
K = Co-efficient of Permeability

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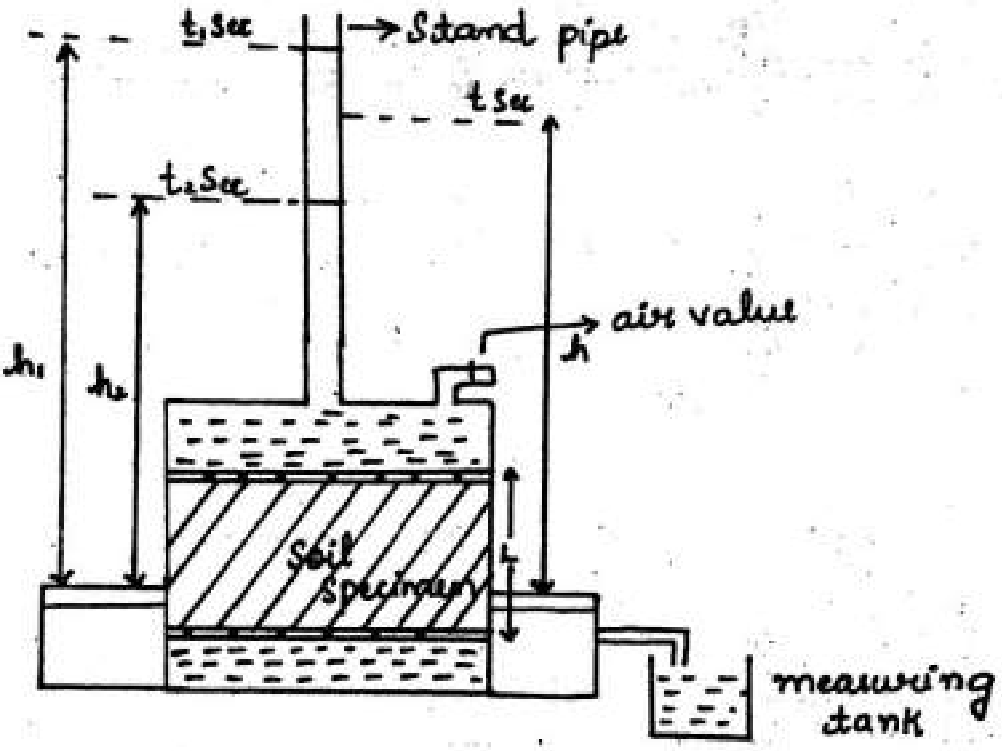
2. Falling head Permeability / variable head permeability test

The falling or variable head test is used for relatively less permeable soils where the discharge is small. The experimental setup for falling head test is as shown in the fig:-



- A standpipe of known cross-sectional area is fitted over the permeameter and water is allowed to run down.
- The water level in the standpipe constantly falls as water flows.
- observations are started after steady state of flow has reached.

Let h_1 & h_2 be head at time interval t_1 & t_2 respectively.

Also let h = head at any time interval t.

$-dh$ = change in head in smaller time interval dt

A = area of c/s of Soil specimen

a = area of c/s of Stand pipe

As per darcy's law, the rate of flow 'q' is given by q = a x velocity of flow

$$q = -a\frac{dh}{dt} \quad \text{— (i)}$$

$$q = kiA = K\frac{h}{L}A \quad \text{— (ii)}$$

From equa i & ii

$$-a\frac{dh}{dt} = K\frac{h}{L}A$$

$$\frac{-dh}{h} = \frac{AK}{aL}dt$$

Intergrating b/w suitable limit

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

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

$$K = \frac{aL}{At}\log_e\frac{h_1}{h_2} \quad \text{where } t = t_2 - t_1$$

$$\boxed{K = 2.303\frac{aL}{At}\log_{10}\frac{h_1}{h_2}}$$

2. Field methods

The field test can be conducted in both unconfined aquifer & confined aquifer.

- An aquifer is a water bearing stratum in natural ground formation. If it overlies an impervious stratum & the water table is free to fluctuate, it is called unconfined aquifer.

On the other hand, if the aquifer is bound by impervious strata both at top & bottom it is called confined aquifer.

There are two types of field tests for determining the co-efficient of permeability are

a) Pumping out tests
b) Pumping-in tests

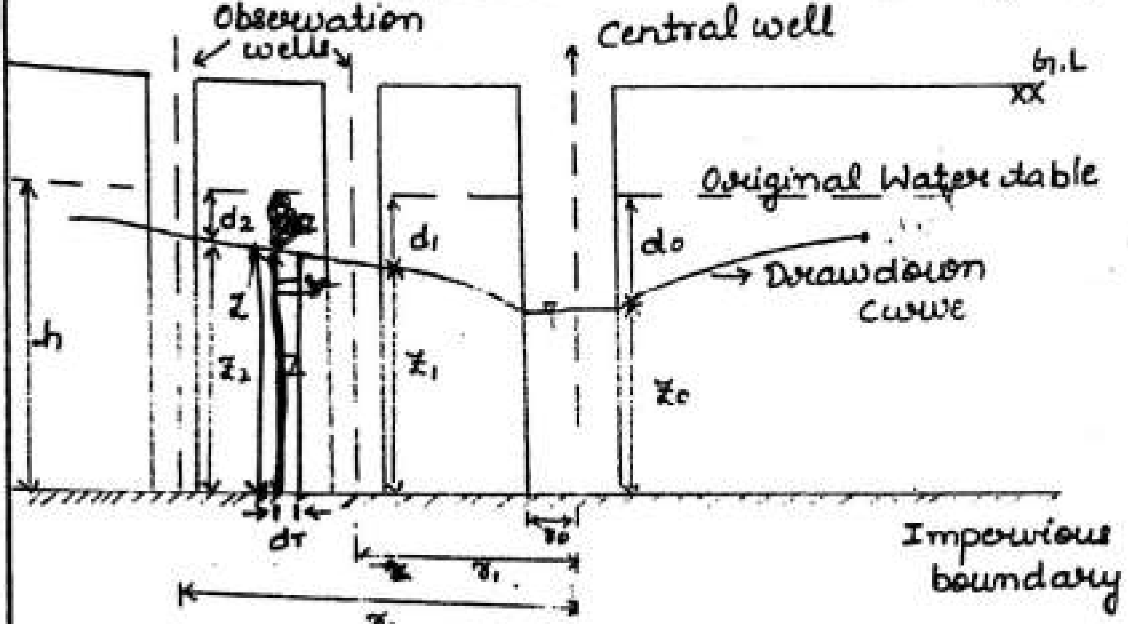
a) Pumping Out tests :-

1. Pumping out tests in unconfined aquifer

The co-efficient of permeability of Permeable layer can be determined by pumping from a well at a constant state and observing the steady state water table in near by observation wells.

- The steady state is established when the water levels in the test well & observation wells become constant.

When water is pumped out from the well, the aquifer gets depleted of water and the water table is lowered resulting in circular depression in the surface. This is referred to as the '<u>Drawdown Curve</u>' or '<u>Cone of depression</u>'



Let r_0 = radius of well. h = height of the aquifer measured from the impervious layer to the initial water table, d_0 = drawdown at the well. Consider the flow through an elementary cylinder of soil having radius 'r' thickness 'dr' and height z.

$$q = KiA = K\frac{dz}{dr}2\pi rz \qquad (\because A = 2\pi rz)$$

$$\frac{dr}{r} = \frac{2\pi K}{q}(z\,dz) \qquad \left[i = \frac{dz}{dr} \text{ from Dupuit's theory}\right]$$

Integrating both sides

$$\int_{r_1}^{r_2}\frac{dr}{r} = \frac{2\pi K}{q}\int_{z_1}^{z_2} z\,dz = \frac{2\pi K}{q}\left[\frac{z^2}{2}\right]_{z_1}^{z_2}$$

$$\ln\left[\frac{r_2}{r_1}\right] = \frac{\pi K}{q}(z_2^2 - z_1^2)$$

$$K = \frac{q}{\pi(z_2^2 - z_1^2)} \ln\left[\frac{r_2}{r_1}\right]$$

In terms of $\log_{10}$

$$\boxed{K = \frac{2.303q}{\pi(z_2^2 - z_1^2)} \log_{10}\left[\frac{r_2}{r_1}\right]}$$

$$z_1 = (h - d_1)$$
$$z_2 = (h - d_2)$$

$$K = \frac{2.303q}{\pi[(h-d_1)^2 - (h-d_2)^2]} \log_{10}\left[\frac{r_2}{r_1}\right]$$

$$K = \frac{2.303q}{\pi[(d_1 - d_2)(2h - d_1 - d_2)]} \log_{10}\left[\frac{r_2}{r_1}\right]$$

Case 2: Confined aquifer

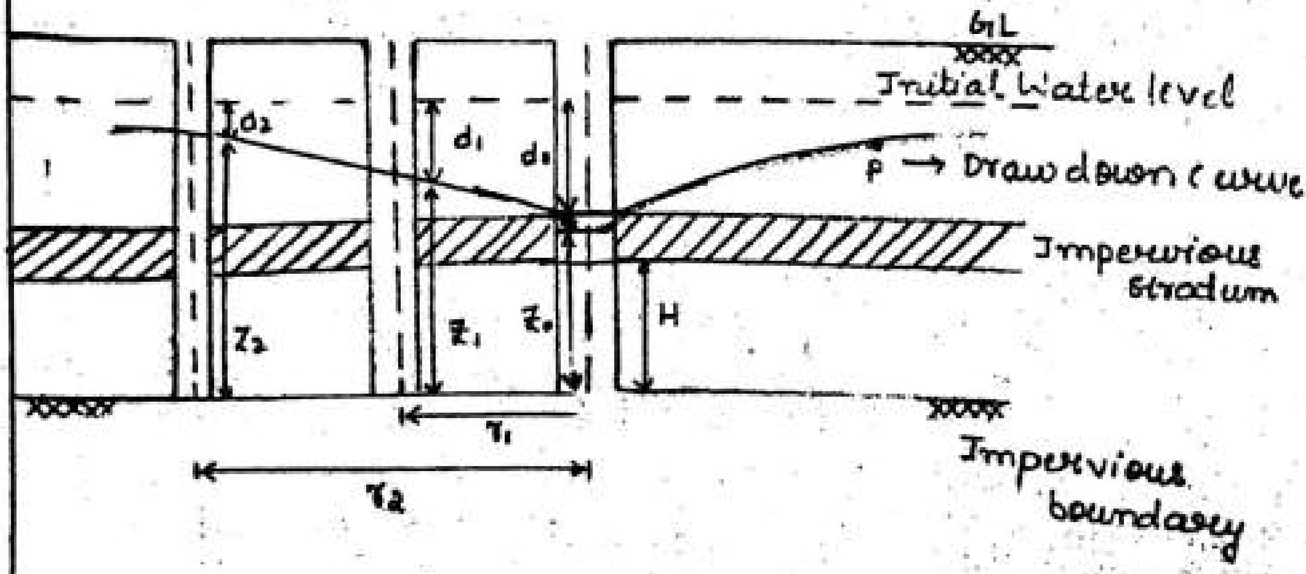


Fig shows a well fully penetrating a confined aquifer

At steady state, the rate of discharge due to pumping can be expressed as.

$$q = KiA$$

$$\therefore q = K\frac{dz}{dr}2\pi r H$$

[H = depth of confined aquifer]

$$K\,dz = \frac{q\,dr}{2\pi r H}$$

Intergrating on both sides

$$K\int_{z_1}^{z_2} dz = \frac{q}{2\pi H}\int_{r_1}^{r_2}\frac{dr}{r}$$

$$K\left[z\right]_{z_1}^{z_2} = \frac{q}{2\pi H}\left[\log_e r\right]_{r_1}^{r_2}$$

$$K\left[z_2 - z_1\right] = \frac{q}{2\pi H}\log_e\left[\frac{r_2}{r_1}\right]$$

$$\boxed{K = 2.303\,\frac{q}{2\pi H}\,\frac{\log_{10}\left[r_2/r_1\right]}{z_2 - z_1}}$$

2] Pumping In tests :-

Pumping in tests are conducted to determine the co-efficient of permeability of an individual stratum through which a hole is drilled.

These tests are more economical than the pumping out tests.

However, the pumping out tests gives more reliable values than that given by pumping in tests.

The pumping in tests gives the value of the co-efficient of permeability of stratum just close to the hole. whereas the pumping out tests gives the value for a large area around the hole.

There are basically two types of pumping in tests

1. Open end tests
2. Packer tests

1. Open end tests :- An open end pipe is sunk in the strata and the soil is taken out of the pipe just to the bottom.

- After the hole has been cleaned out, water is added to the hole through a metering system. The water ∴ added will have high temperature slightly higher than the water table.
- Water may also be allowed to enter the hole under some pressure head.

- The constant rate of flow(q) is determined at which the steady conditions are established.
- The permeability is determined by the following equation.

$$K = \frac{q}{5.5 r h}$$

where K = co-efficient of Permeability
r = radius of casing
h = difference of levels between the inlet of casing and the water table.

2. <u>Packer tests</u> :-

An uncased portion of the drill hole or a perforated portion of the casing is used for performing the test.

- The packer tests are more commonly used for testing of rocks. The tests are occasionally used for testing of soils if the borehole can stay open without any casing.
- To perform the test after completion of the hole, which can stand without casing, two packers are set on a pipe keeping the perforated portion of the pipe between the plugs.
The bottom of the pipe is plugged.
- Water is pumped into the hole.
- The value of the co-efficient of permeability is obtained from the quantity of water

that flows out through the sides of the section of a hole enclosed between packers.

The co-efficient of Permeability is determined by the following equation

$$K = \frac{q}{2\pi LH} \log_e\left[\frac{L}{r}\right] \quad \text{if } L \geq 10r$$

$$K = \frac{q}{2\pi LH} \sin^{-1}(L/2r) \quad \text{if } 10r > L \geq r$$

where r = inside radius of hole
L = Length of the hole tested
H = difference of water level

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* **Factors affecting Permeability**

The various factors affecting permeability are listed below:-

1. Grain Size
2. Properties of Pore fluid
3. Void ratio of the soil
4. Soil fabrication & stratification
5. Degree of Saturation
6. Presence of foreign matter
7. Adsorbed matter

1. **Grain Size :-**

Permeability varies approximately as the square of the grain Size.

- Smaller the grain Size, smaller voids and thus the lower the permeability.
- According to Allen Hazen (1911), the permeability of Sand can be estimated using the relation

$$K = CD_{10}^2$$

where, K - co-efficient of Permeability (cm/s)

D_{10} = Effective Size (cm)

C = Constant value

or

$$\boxed{K = 10^4 D_{10}^2}$$

This is applicable for D_{10} = 0.1mm to 3mm

2] <u>Effect</u> of <u>Properties</u> of <u>Pore</u> fluid

The permeability is directly proportional to the unit wt of water (γ_w) and inversely proportional to its viscosity.

$$K \propto \frac{\gamma_w}{\mu}$$

So, the unit wt of water remains constant with variations of temperature.
The permeability varies with the variation of viscosity.

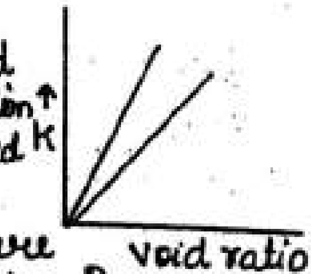
3. <u>Effect</u> of <u>Void</u> <u>ratio</u> :-

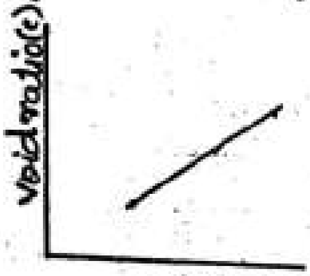
The increase in void ratio of soil sample increases with the permeability.

- It has been found that a semi logarithmic plot of void ratio versus permeability is approximately a straight line for both coarse grained & fine grained soil.

- For coarse grained soil, following relation has been established

$$\frac{K_1}{K_2} = \frac{e_1^3}{1+e_1} \Big/ \frac{e_2^3}{1+e_2}$$





Where
K_1 = Permeability at void ratio e_1.
K_2 = " " " e_2.

4. <u>Effect</u> of <u>structural</u> <u>arrangement</u> of <u>particles</u>

* <u>stratification</u>

Fine grained soil with a flocculated structure have a higher co-efficient of permeability than those with a dispersed structure.

- The effect of structural arrangement of soil particles on permeability can be found by

- determining permeability of undisturbed & disturbed soil samples.
- Stratified soil mass will have different average permeability in direction parallel & perpendicular to their bedding planes. The average permeability parallel to bedding plane will be more than that of perpendicular to bedding plane.

5. Effect of degree of Saturation & other foreign matter

- The permeability is greatly reduced if air is entrapped in the Voids thus reducing the degree of saturation.
Higher the degree of Saturation, higher the permeabili & Vice Versa.
- Organic foreign matter, if present in soil mass may be carried by flowing water & may choke flow channel causing reduction in permeability.

6. Effect of adsorbed water :-

The adsorbed water surrounding the fine soil particles is not free to move & reduces the effective pore space available for the passage of water.

* ## Permeability of Stratified Soils

In nature, soil mass may consist of several layers deposited one above the other.

- The bedding plane of soil may be horizontal, inclined or vertical.
- The layers of the soil deposited is assumed to be homogenous & isotropic, has its own value of co-efficient of permeability.
- The average permeability of the whole deposit will depend on the direction of flow to its direction of the bedding planes.

Case i] Average Permeability parallel to the Bedding plane.

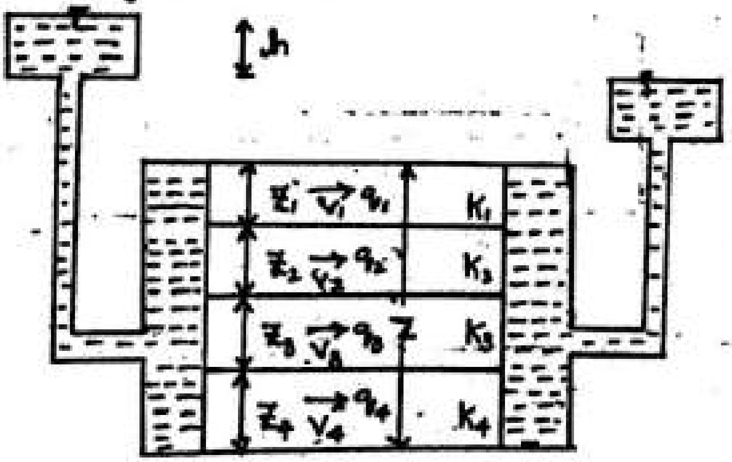


Fig: Flow parallel to the bedding plane

Let, $z_1, z_2, \ldots z_n$ be the thickness of layers
$K_1, K_2, \ldots K_n$ be the Permeability of layers.
K_x = average permeability of the soil deposit parallel to the bedding plane.

When a flow takes place parallel to the bedding plane, the hydraulic gradient, I will be same.

However $V = Ki$ & since K is different & V is also different for different layers.

Thus, we have

Total discharge of the soil deposit = Sum of discharge through individual layers.

$$q = q_1 + q_2 + q_3 + \dots + q_n$$

Apply darcy's law

$$q = K_x i A \quad \text{— considering unit width}$$

$$\therefore \quad q = K_x I Z$$

For individual layers, we have

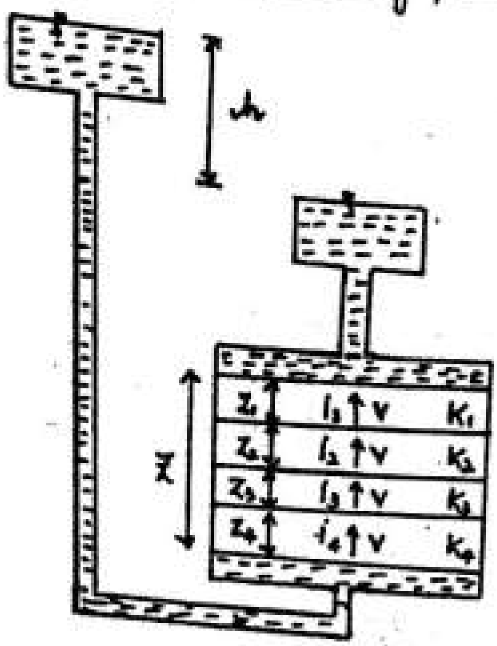
$$q = K_x i Z = K_1 i Z_1 + K_2 i Z_2 + K_3 i Z_3 + \dots + K_n i Z_n$$

$$K_x = \frac{i(K_1 Z_1 + K_2 Z_2 + K_3 Z_3 \dots + K_n Z_n)}{i Z}$$

$$\therefore \quad \boxed{K_x = \frac{K_1 Z_1 + K_2 Z_2 + \dots K_n Z_n}{Z}}$$

where $Z = Z_1, Z_2, Z_3 \dots + Z_n$

Case ii] <u>Average Permeability perpendicular to the bedding planes</u>



When the flow takes place perpendicular to bedding plane the velocity of flow becomes same, while hydraulic gradient is different for different layers.

Thus we have,

Total head loss = head loss through Individual layer

$$h = h_1 + h_2 + h_3 + \ldots$$

Apply darcy's law

$$V = K_z i$$

where K_z = Average permeability ⊥lar to bedding plane

$$i = \frac{h}{Z}$$

$$V = K_z \frac{h}{Z} \quad \text{or} \quad h = \frac{VZ}{K_z}$$

For Individual layers, we have

$$\frac{VZ}{K_z} = \frac{VZ_1}{K_1} + \frac{VZ_2}{K_2} + \frac{VZ_3}{K_3} + \dots \frac{VZ_n}{K_n}$$

$$K_z = \frac{VZ}{V\left[\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3} + \dots \frac{Z_n}{K_n}\right]}$$

$$\boxed{K_z = \frac{Z}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3} \dots \frac{Z_n}{K_n}}}$$

It can be seen that for any stratified soil mass K_x is always greater than K_z.

To show that average permeability parallel to bedding plane is greater than that perpendicular to bedding plane.

Let us assume

$Z_1 = 2$ units, $K_1 = 5$ units

$Z_2 = 6$ units, $K_2 = 3$ units

$Z_3 = 4$ units, $K_3 = 7$ units

$$K_x = \frac{K_1 Z_1 + K_2 Z_2 + K_3 Z_3}{Z_1 + Z_2 + Z_3}$$

$$= \frac{5(2) + 3(6) + 7(4)}{2 + 6 + 4}$$

$K_x = 4.67$ units

$$K_z = \frac{Z_1 + Z_2 + Z_3}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}}$$

$$K_z = \frac{2 + 6 + 4}{\frac{2}{3} + \frac{6}{3} + \frac{4}{3}}$$

$K_z = 3$ units

$$\therefore \boxed{K_x > K_z}$$

* Seepage Velocity (V_s)

It is a fictious velocity obtained by dividing the total discharge 'q' by the total cross sectional area (A).

The actual is defined as the rate of discharge of percolating water per unit cross sectional area of voids perpendicular to the direction of flow.

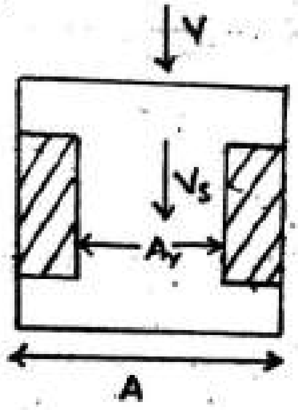


Fig (a) shows the longitudinal section through a soil sample in which the voids and solid particles are segregated.

From the continuity of flow

$$q = VA = V_s A_v \quad \text{— (1)}$$

where A_V = Area of flow through voids

V_s = Actual Seepage velocity.

From equ (1)

$$V_s = V\left(\frac{A}{A_v}\right)$$ — x'ly numerator & denominator by 'L' length of the specimen

$$V_s = V\left(\frac{A \times L}{A_v \times L}\right)$$

The product $(A \times L)$ = Total volume &

$(A_v \times L)$ = Volume of voids.

$$V_s = V \times \frac{V}{V_v} \quad \left[\frac{V_v}{V} = \eta\right]$$

$$\boxed{V_s = V/\eta}$$

* **Superficial Velocity :**

Is defined as the rate of discharge of water per unit of total cross sectional area 'A' of the soil.

$$V = Ki \quad / \quad V = \frac{Q}{At}$$

where V = referred as the superficial velocity of flow.

* **Co-efficient of Percolation :-**

The co-efficient of percolation is defined as the ratio of co-efficient of permeability to porosity.

Also, $V_s = K_p i$

where K_p = co-efficient of percolation

$$\frac{V}{\eta} = K_p i$$

$$\frac{Ki}{\eta} = K_p i$$

$$\boxed{K_p = \frac{K}{\eta}}$$

Thus, K_p - can defined as the ratio of co-efficient of permeability to porosity

* Quick Sand Phenomena

When flow takes place in an upward direction the seepage pressure also acts in the upward direction & the effective stress is reduced.

- If the seepage pressure becomes equal to the pressure due to submerged wt of the soil, the effective stress is zero.
- In such a case, a cohesionless soil loses all the shear strength & the soil particles have a tendency to move up in the direction of flow.
- This phenomena at lifting of soil particles is called quick sand phenomena or boiling condition.
- The hydraulic gradient at such a critical state is called critical hydraulic gradient.
- It should be noted that quick sand is not a type of sand but a flow condition occuring within a cohesionless soil when its effective stress is reduced to zero due to upward flow of water.

Demonstration of Quick Sand phenomena

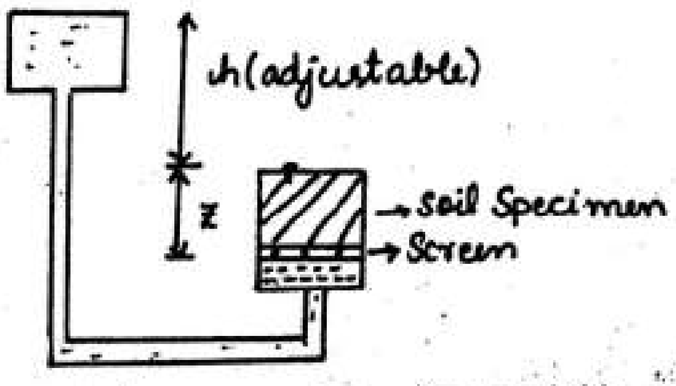


fig: quick Sand condition

The figure shows a setup to demonstrate the phenomenon of quick sand.

The water flows in an upward direction through the saturated soil of thickness 'z' under a hydraulic head 'h'.
When the soil particles are in state of critical equilibrium we have

upward force = downward force

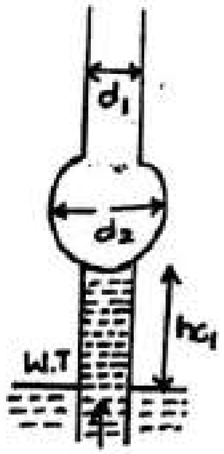
$$(h+z)\gamma_w A = z\gamma_{sat} A$$

$$h\gamma_w = z(\gamma_{sat} - \gamma_w)$$

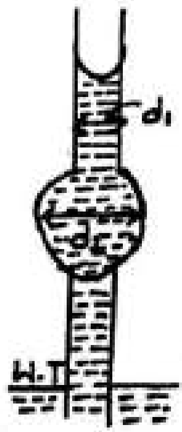
$$h\gamma_w = z\gamma'$$

$$\frac{h}{z} = \frac{\gamma'}{\gamma_w} = \frac{G-1}{1+e} \Rightarrow i_{critical} = \left(\frac{G-1}{1+e}\right)$$

* Capillary Phenomena



a) upward flow



b) downward flow

The height of capillary rise will depend upon the direction of flow of water in the tube.

when the tube is dipped in water, water lifted in the tube upto a height of hc, only. The water will not enter into the larger tube because water cannot maintain equilibrium of the larger diameter (d_2).

- If the water tube is filled by pouring water from the top or if the tube is lowered below the water table & then raised, equilibrium is maintained at a greater height (hc_2). Hence capillary rise in tube of non uniform diameter is more, if the flow is downward than when the flow is upward.

- Capillary action or capillary is the phenomena of movement of water in the interstices of a soil due to capillary rise.

- The minute pores of soil serves as capillary tube through which the moisture raises above the ground water table.

- The capillary forces depends upon the surface tension of water, pressure in water in relation to atmospheric pressure & size & conformation of soil pores.

The capillary rise in circular tube is given by

$$\boxed{h_c = \frac{4 \sigma_c \cos\alpha}{\gamma_w d}}$$

where σ_c = Surface Tension (N/m)

According to terzaghi peck, we have

$$\boxed{(h_c)_{max} = \frac{C}{e D_{10}}}$$

where C = constant
e = Void ratio
D_{10} = Eff. Size (mm)

Problems:-

Calculate the co-efficient of permeability of a soil sample 6cm in height and $50cm^2$ in cross sectional area, if a quantity of water is equal to 430cc passed down in 10minutes under an effective constant head of 40cm.
On oven drying, the test specimen weighed 4.98N. Taking $G = 2.65$. Calculate the seepage velocity of water during test.

Given:-

height = 6cm

c/s area = $50cm^2$

Quantity of water = 430cc

Time of flow = $10 \times 60 = 600$ sec

Constant head (H) = 40cm

$$K = \frac{QL}{Aht} = \frac{430 \times 6}{50 \times 40 \times 600} = 2.15 \times 10^{-3} cm/s$$

$$\boxed{K = 2.15 \times 10^{-3} cm/s}$$

$$\text{Velocity } (v) = \frac{q}{A} = \frac{Q}{At}$$

$$V = \frac{430}{50 \times 600} = \underline{0.0143 cm/s}$$

Dry weight of specimen, $W_d = 4.98N$

Volume of specimen $V = A \times L$

$= 50 \times 6 = \underline{300cm^2}$

$$\gamma_d = \frac{W_d}{V} = \frac{507.64}{300} = \underline{1.69\,gm/cm^3}$$

$\frac{[illegible]}{981} \times \frac{[illegible]}{1000} = 507.64\,gm$

$$e = \frac{G\gamma_w}{\gamma_d} - 1 = \frac{2.65(1)}{1.69} - 1 \qquad [\gamma_w = 1g/cc]$$

$$\boxed{e = 0.56}$$

$$\eta = \frac{e}{1+e} = \frac{0.56}{1+0.56} = 0.358$$

$$\boxed{\eta = 0.358}$$

Seepage Velocity, $V_s = \frac{V}{\eta} = \frac{0.0143}{0.358} = 0.039\,cm/s$

$$\boxed{V_s = 0.039\,cm/s}$$

2) A cylindrical mould of diameter 7.5cm contains 15.0cm long sample of sand. When water flows through the soil under constant head at a rate of 55 cc/minute, the loss of head b/w two points 8cm apart is found to be 12.5cm. Determine the co-efficient of permeability of the soil.

Solu: Given: $d = 7.5cm$ Head Loss $(h) = 12.5cm$

$L = 15.0cm$

$q = 55cc/min$

$$q = \frac{55}{60} = 0.92\,cc/sec$$

$$q = 0.92\,cc/s$$

hydraulic gradient $(i) = \frac{h}{L}$:

$$= \frac{12.5}{15} = 0.833$$

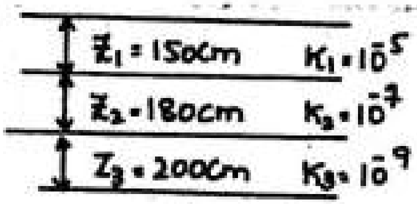
Area of c/s $= \frac{\pi}{4} \times (7.5)^2 = 44.18 cm^2$

$$K = \frac{QL}{Aht} = \frac{55 \times 15}{44.18 \times 12.5 \times 60} = 0.0248 cm/s$$

$$\boxed{K = 0.0248 cm/s}$$

3] Calculate the horizontal & vertical permeability of a soil deposit consisting of three layer 150cm. 180cm & 200cm thick with permeability $10^{-5}, 10^{-7}$ & 10^{-9} m/s respectively.

Solu :



Average permeability parallel to the Bedding plane

$$K_x = \frac{K_1 Z_1 + K_2 Z_2 + K_3 Z_3}{Z_1 + Z_2 + Z_3}$$

$$= \frac{10^{-5}(1.5) + 10^{-7}(1.8) + 10^{-9}(2)}{1.5 + 1.8 + 2}$$

$$\boxed{K_x = 2.86 \times 10^{-6} m/Sec}$$

Average permeability perpendicular to bedding plane

$$K_Z = \frac{Z_1 + Z_2 + Z_3}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}} = \frac{1.5 + 1.8 + 2}{\frac{1.5}{10^{-5}} + \frac{1.8}{10^{-7}} + \frac{2}{10^{-9}}}$$

$$\boxed{K_Z = 2.62 \times 10^{-9}\ m/s}$$

4. In a falling head permeability test the length & area of c/s of soil specimen are 0.17m & 21.8×$10^{-4}$$m^2$ respectively. calculate the time required for the head to drop from 0.25m to 0.10m. The area of c/s of stand pipe is 2×$10^{-4}$$m^2$. The sample has three layer with permeabilities 3×10^{-5} for 1st 0.06m, 4×10^{-5}m/s for second 0.06m & 6×10^{-5}m/s for the third 0.05m thickness. Assume the flow is taking place perpendicular to the loading plane.

Solu:- Given:- $L = 0.17m$, $h_1 = 0.25m$, $a = 2\times10^{-4}m^2$

$A_s = 21.8\times10^{-4}m^2$, $h_2 = 0.10m$,

$h = 0.15m$

$K_1 = 3\times10^{-5}m/s$ $Z_1 = 0.06m$

$K_2 = 4\times10^{-5}m/s$ $Z_2 = 0.06m$

$K_3 = 6\times10^{-5}m/s$ $Z_3 = 0.05m$

$$K_Z = \frac{Z_1 + Z_2 + Z_3}{\frac{Z_1}{K_1} + \frac{Z_2}{K_2} + \frac{Z_3}{K_3}} = \frac{0.06 + 0.06 + 0.05}{\frac{0.06}{3\times10^{-5}} + \frac{0.06}{4\times10^{-5}} + \frac{0.05}{6\times10^{-5}}}$$

$$\boxed{K_Z = 3.92\times10^{-5}\ m/s}$$

$$K = 2.303 \frac{aL}{At} \log_{10} \frac{h_1}{h_2}$$

$$t = 2.303 \times \frac{2\times10^{-4} \times 0.17}{21\times10^{-4} \times 3.92\times10^{-5}} \log\left(\frac{0.25}{0.10}\right)$$

$t = 364.62 \simeq 365\,sec$

$t = 6\,min\ 5\,sec$

5. A glass cylinder $50cm^2$ inside c/s area & 40cm height is provided with a screen at the bottom & is open at the top. Saturated sand is filled in the cylinder upto a height of 10cm above the screen. The cylinder is then filled with water upto its top. Determine the co-efficient of permeability if the water level drops from the top of the cylinder through a distance of 20cm in half an hour.

Solu: Given: $A = 50cm^2$, $L = 10cm$

$h_1 = 40cm$

$h_2 = 20cm$

$t = \frac{1}{2} hr = 30\times60 = 1800\,sec$

$$K = 2.303 \frac{aL}{At} \log\left[\frac{h_1}{h_2}\right] = 2.303$$

Scanned by CamScanner

6] A constant head permeameter contains a sand Sample of 20cm length, 25cm² under a head of 40cm. The discharge was found to be 180cc in 110sec. The specific gravity of the grain is 2.66. Determine the co-efficient of Permeability, Superficial velocity, seepage velocity and co-efficient of percolation if void ratio is 0.50.

Solu:

$e = 0.50$
$G = 2.66$
$L = 20cm$
$A = 25cm^2$

$Q = 180cc$
$t = 110sec$
$h = 40cm$

To find K, V_s, V, K_p

i) $K = \frac{QL}{Aht} = \frac{180 \times 20}{25 \times 40 \times 110} = \underline{0.0327\ cm/sec}$

ii) Superficial velocity

$V = \frac{Q}{At} = \frac{180}{25 \times 110} = \underline{0.0654\ cm/sec}$

iii) Seepage velocity

$V_s = \frac{V}{\eta} = \frac{0.0654}{0.33}$

$\eta = \frac{e}{1+e} = \frac{0.50}{1+0.50}$

$\underline{\eta = 0.33}$

$\boxed{V_s = 0.198 cm/s}$

iv) Co-efficient of Percolation

$K_p = \frac{K}{\eta} = \frac{0.0327}{0.33}$

$\boxed{K_p = 0.099 cm/s}$

7] A canal is located in an area & is parallel to nearby river at a distance of 160m. The elevation of water surface in canal & river respectively are 125m & 115m. A stratum of 3m thick coarse grained soil ($K = 2 \times 10^{-4}$ m/s) sandwitched b/w clay strata connects canal & the river. Calculate seepage in m^3/s per Km length of canal.

Solu :

$K = 2 \times 10^{-4}$

$h = 125 - 115 = 10m$

$L = 160m$

$d = 3m$

$$q = KiA$$

$$= 2 \times 10^{-4} \times \frac{10}{160} \times (3 \times 160)$$

$$= 6 \times 10^{-4} \ m^3/s \Rightarrow \frac{6 \times 10^{-4}}{160} = 3.75 \times 10^{-5} m^3/sec/ \text{length of canal}$$

$$q = 3.75 \times 10^{-5} \times 1000 = 0.0375 m^3/sec/Km$$

length of canal

8] In a falling head test permeability test initial head of 1.0m dropped to 0.35m in 3 hours, the diameter of the stand pipe being 5mm. The soil specimen is 200mm long and 100mm in diameter. Calculate co-efficient of permeability of the soil.

Solution: Data:- $h_1 = 1.0m \approx 1000mm$

$h_2 = 0.35m \approx 350mm$

$t = 3hr = 180\ min \approx 10800 sec$

$A = \text{Area of Soil sample} = \frac{\pi(100)^2}{4} = 7853.98 mm^2$

$d = 100m, L = 200mm$

$A = 7853.98 mm^2$

$a = \frac{\pi(5^2)}{4} = 19.634 mm^2$

$K = 2.303 \frac{aL}{At} \log_{10}\left[\frac{1000}{350}\right] = 2.303 \times \frac{19.63 \times 200}{7853.98 \times 10800} \log_{10}\left[\frac{1000}{350}\right]$

$\boxed{K = 4.85 \times 10^{-5} mm/sec}$

9] A Sample in a variable head permeameter is 8cm in diameter and 10cm high. The permeability of the samples is estimated to be $10 \times 10^{-4} cm/s$. If it is desired that the head in the standpipe should fall from 24cm to 12cm in 3min, determine the size of the stand pipe which should be used.

Solution: Soil sample dia = 8cm & L = 10cm

$K = 10 \times 10^{-4} cm/s$ $\quad A = \frac{\pi d^2}{4} = 50.26 cm^2$

$h_1 = 24cm$

$h_2 = 12cm$

$t = 3min = 180 Sec$

$$k = 2.303 \frac{al}{At} \log_{10} \frac{h_1}{h_2}$$

$$10^{-3} = \frac{2.303 \times a \times 10}{50.26 \times 180} \log_{10}\left[\frac{24}{12}\right]$$

$$a = \frac{10^{-3} \times 50.26 \times 180}{2.303 \times 10 \left(\log \frac{24}{12}\right)}$$

$$a = \frac{9.5026}{6.93}$$

$$\boxed{a = 1.30 \, cm^2}$$

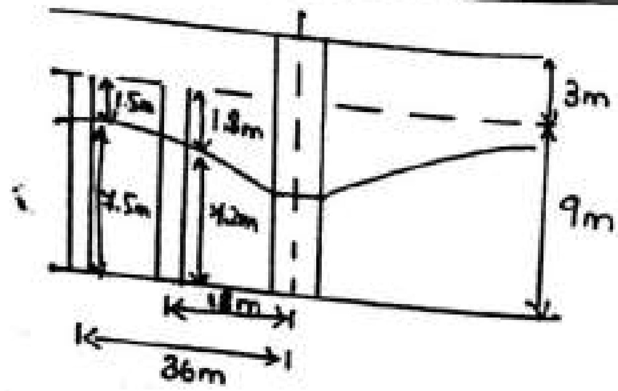
Diameter of the stand pipe

$$1.30 = \frac{\pi d^2}{4}$$

$$\boxed{d = 1.29 \, cm}$$

10] A Sand deposit of 12m thick overlies a clay layer. The water table is 3m below the ground surface. In a field permeability pump out test, the water is pumped out at a rate of 540l/min when steady state conditions are reached. Two observation well are located at 18m & 36m from centre of the test Well. The depths of the drawdown curve 1.8m & 1.5m respectively for these two wells. Determine the co-efficient of permeability.

Solution:-



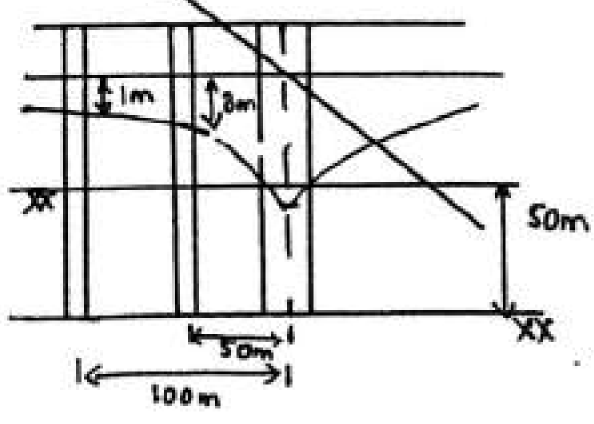
Data:- $r_1 = 18m$, $z_1 = 7.2m$
$r_2 = 36m$, $z_2 = 7.5m$
$h = 9m$
$q = 540 l/min = 0.009 m^3/s$

$$K = \frac{2.303q}{\pi(z_2^2 - z_1^2)}\left[\log\frac{r_2}{r_1}\right] = \frac{2.303 \times 0.009}{\pi(7.5^2 - 7.2^2)} \log_{10}\left[\frac{36}{18}\right]$$

$$\boxed{K = 4.504 \times 10^{-4} m/s}$$

A Pumping In test carried out in a 50m thick confined aquifer results in a flow rate of 600 l/min. Drawdown in two observation well located 50m and 100m from the well are 3m & 1m respectively. Calculate the co-efficient of permeability of the aquifer.

Solution:



11] The results of a constant head permeability test on fine sand are as follows. area of the soil specimen = $180 cm^2$, length of the specimen = 32(
constant head maintained 460mm and flow of water through the specimen = 200ml in 5min
Determine the co-efficient of permeability.

Solu:

$A = 180 cm^2$

$L = 320 mm = 32 cm$

$h = 460 mm = 46 cm$

$Q = 200 ml = 200 cm^3$

$t = 5 min = 300 Sec$

$$K = \frac{QL}{Aht} = \frac{200 \times 32}{180 \times 46 \times 300} =$$

$$\boxed{K = 2.57 \times 10^{-3} cm/s}$$

12] A falling head permeability test is to be performed on a soil sample whose permeability is estimated to be about $8 \times 10^{-5} cm/s$.
What diameter of the stand pipe should be used if the head is to be drop from 27.5cm to 20.0cm in 5min and if the c/s area and length of the sample are respectively $15 cm^2$ & 8.5cm.
Will it take the same time for the head drop from 37.7cm to 30.0cm.

Solu:-

$K = 3 \times 10^{-5} cm/s$ $\quad$ $A = 15 cm^2$

$h_1 = 27.5 cm$ $\quad$ $L = 8.5 cm$

$h_2 = 20.0 cm$

$t = 5 min = 60 Sec$

$$K = 2.303 \frac{aL}{At} \log\left[\frac{h_1}{h_2}\right]$$

$$3\times10^{-5} = 2.303 \frac{a\times 8.5}{15\times 300} \log\left[\frac{27.5}{20}\right]$$

$$\boxed{a = 0.0498 cm^2}$$

$$a = \frac{\pi d^2}{4} \Rightarrow 0.0498 = \frac{\pi d^2}{4}$$

$$\boxed{d = 0.25 cm}$$

Time Required for $h_1 = 37.7 cm$ to $h_2 = 30.0 cm$

$$K = 2.303 \frac{aL}{At} \log_{10}\left[\frac{h_1}{h_2}\right]$$

$$3\times10^{-5} = 2.303\times \frac{0.0498\times 8.5}{15\times t} \log_{10}\left[\frac{37.7}{30}\right]$$

$t = 214.9 Sec < t = 300$

∴ It requires less time.

13] In 12min 800ml passes through a soil specimen of about 1000mm height & 7500mm² C/S area under the head of 800mm.
Determine the discharge velocity and co-efficient of Permeability.

Solution:- $t = 12\text{min} \approx 720\text{sec}$

$Q = 800\text{ml} = 800\text{cm}^3 = 800\times10^3\text{mm}^3$

$L = 1000\text{mm}$

$h = 800\text{mm}$

$A = 7500\text{mm}^2$

$$K = \frac{QL}{Aht} = \frac{800\times10^3\times1000}{7500\times800\times720} = 0.185\text{mm/s}$$

$$\boxed{K = 0.185\text{mm/s}}$$

$$V = Ki = K\times\frac{h}{l} = 0.185\times\frac{800}{1000}$$

$$\boxed{V = 0.148\text{mm/s}}$$

14] Constant rate permeability test is carried out on cylindrical specimen of Sand of 100mm diameter & 150mm height. 160ml of water was collected in 1.75min under a head of 300mm. Compute the co-efficient of permeability, Seepage velocity taking porosity of soil as 0.42

Solution: $d = 100\text{mm} \Rightarrow A = \frac{\pi d^2}{4} = 7853.9\text{ mm}^2$

$L = 150\text{mm}$

$t = 1.75 min = 105 sec$

$h = 300 mm$

$Q = 160 ml = 160 cm^3 = 160 \times 10^3 mm^3$

$$K = \frac{QL}{Aht} = \frac{160 \times 10^3 \times 150}{7854 \times 300 \times 105} = 0.097 mm/s$$

$$\boxed{K = 0.097 mm/s}$$

$$V = Ki = K \times \frac{h}{l} = 0.097 \times \frac{300}{150}$$

$$\boxed{V = 0.194 mm/s}$$

$$V_s = \frac{V}{\eta} = \frac{0.194}{0.72} = 0.269 mm/s$$

$$\boxed{V_s = 0.269 mm/s}$$

15] A Soil Sample of ht 60mm & c/s area of 8000mm² was subjected to a Variable head permeability test. In a time interval of 6min, the head dropped from 750mm to 300mm if the c/s area of Stand pipe is 150mm², compute the co-efficient of permeability.

If the Sample is subjected to a constant head of 200mm, compute the total quantity of water that will discharged through a sample in a time interval of 10min.

Solu :- $L = 60 mm$, $a = 150 mm^2$

$A = 8000 mm^2$, $h_1 = 750 mm$

$t = 6 min = 360 sec$. $h_2 = 300 mm$

Case I]

$$K = 2.303 \frac{al}{At} \log\left[\frac{h_1}{h_2}\right] = 2.303 \times \frac{150 \times 60}{8000 \times 360} \log\left[\frac{750}{300}\right]$$

$$\boxed{K = 2.86 \times 10^{-3}\ mm/s}$$

Case II] h = 200mm

Q = ? t = min = 0 sec

$q = KiA$

$= 2.86 \times 10^{-3} \times \frac{200}{60} \times 8000$

$$\boxed{q = 76.26\ mm^3/s}$$

$q = \frac{Q}{t} \Rightarrow Q = q \times t$

$= 45.75 \times 10^3\ mm^3/s$

$$\boxed{Q = 4.53 \times 10^{-3}\ m^3/s}$$

16] Find the ratio of average permeability in horizontal direction to that of vertical for a soil deposit of three layers with the thickness in the ratio 1:2:3. The permeability of 2nd layer is twice that of first layer and the third is twice that of second.

$Z_1 = x$, $Z_2 = 2x$, $Z_3 = 3x$

$K_1 = K$, $K_2 = 2K$, $K_3 = 4K$

$$K_z = \frac{z_1+z_2+z_3}{\frac{z_1}{k_1}+\frac{z_2}{k_2}+\frac{z_3}{k_3}} = \frac{x+2x+3x}{\frac{x}{K}+\frac{2x}{2K}+\frac{3x}{4K}}$$

$$K_z = \frac{6x}{\frac{2.75x}{K}}$$

$$\boxed{K_z = 2.18K}$$

$$K_x = \frac{K_1 z_1 + K_2 z_2 + K_3 z_3}{z_1+z_2+z_3} = \frac{K(x)+4(Kx)+12Kx}{6x}$$

$$K_x = \frac{17Kx}{6x} = 2.83K$$

$$\therefore \frac{K_x}{K_y} = \frac{2.83K}{2.18K}$$

$$\boxed{\therefore K_x : K_y = 1 : 0.77}$$

17] It is observed in 12 min, 800ml of water passes through a soil sample 10cm height & 75cm² of c/s area under a head of 60cm. Determine the velocity, co-efficient of permeability. If on oven drying, the soil sample weighs 0.0685KN compute seepage velocity. Take G = 2.7

Solution:- $t = 12 min \approx 720 sec$

$L = 10cm = 100mm \approx$

$h = 60cm$, $A = 75cm^2$, $h = 60cm \approx 600mm$

18) A loose uniform Sand with rounded grains has an effective size = 0.3mm. Estimate the co-efficient of permeability.

Solution:- $D_{10} = 0.3mm = (0.3 \times 10^{-3})m$

From the affect of shape & Size on Permeability

Terzaghi gave an Equation

$$K = 10^4 D_{10}^2$$

$$K = 10^4 \times (0.3 \times 10^{-3})^2$$

$$\boxed{K = 9 \times 10^{-4} m/s}$$

19] Due to rise in temperature, Viscosity, and the unit wt of percolating fluid are reduced to 75% and 97% respectively.
Find the percentage change in co-efficient of permeability.

Solution:-

Let x & y be the initial value of Viscosity & unit wt of fluid respectively

Viscosity = 0.75x
unit wt = 0.97y

$$K = \frac{\gamma}{\mu} = \frac{y}{x} \qquad K' = \frac{0.97y}{0.75x}$$

$$\frac{K}{K'} = \frac{y/x}{\frac{0.97y}{0.76x}} = 1.29$$

$$K = K'(1.293)$$

Thus %age increase $= \left(\frac{1.293K_1 - K_1}{K_1}\right) \times 100$

29.73%

20] Due to rise in temperature, the viscosity & unit weight of percolating fluid are reduced by 30% & 4% respectively. If all other things be the same. Calculate % change in permeability.

Solution:-

In Initial temperature, viscosity, permeability and unit weight } $= \mu_1, K_1, \gamma_1$

After rise in temperature

$K_2, \mu_2 = 0.7\mu_1, \gamma_2 = 0.96\gamma_1$

$$K_2 = \frac{\gamma_2}{\mu_2} \qquad \left[K_1 = \frac{\gamma_1}{\mu_1}\right]$$

$$= \frac{0.96\gamma_1}{0.7\mu_1} = 1.37\,K_1$$

$$K_2 = 1.371K_1$$

% change in Permeability } $= \left(\frac{1.37K_1 - K_1}{K_1}\right) \times 100$

$= 37.14\%$

21) The Co-efficient of Permeability of the soil sample is 1×10^{-5} mm/s at a void ratio of 0.4. Estimate its value at void ratio of 0.6

Solution:- $K_1 = 1 \times 10^{-5}$ mm/s, $e_1 = 0.4$, $e_2 = 0.6$

$$\frac{K_1}{K_2} = \frac{e_1^3}{1+e_1} \times \frac{1+e_2}{e_2^3} = \frac{0.4^3}{1+0.4} \times \frac{1+0.6}{0.6^3}$$

$$\frac{K_1}{K_2} = 0.3386 \Rightarrow \boxed{K_2 = 2.95 \times 10^{-5}\ \text{mm/s}}$$

22] A Soil is having an average particle size of 4.75mm and void ratio of 0.75. Another soil is having average particle size of 1mm & void ratio of 0.5 keeping all other factors in the permeability test as same, find the ratio of permeability between them.

Solution:

[Note: An equation reflecting the influence of the characteristics of the permeability fluid and the soil on permeability was developed by Taylor and is given by

$$K = \frac{\gamma}{\mu} \times \frac{e_1^3}{1+e_1} \times D_{10}^2 \times C$$

For 1st trial,

$$D_{10} = 4.75mm = 4.75 \times 10^{-3} m, \; C = C$$

$$e_1 = 0.75$$

$$K_1 = \frac{\gamma}{\mu} \times \frac{0.75^3}{1+0.75} \times (4.75 \times 10^{-3})^2 \times C$$

$$K_1 = \frac{C\gamma}{\mu} (5.439 \times 10^{-6})$$

For 2nd trial, $D_{10} = 1mm = (1 \times 10^{-3})m$, $e_2 = 0.5$

$$K_2 = \frac{C\gamma}{\mu} \times \frac{0.5^3}{1+0.5} \times (1 \times 10^{-3})^2$$

$$K_2 = \frac{C\gamma}{\mu} [8.33 \times 10^{-8}]$$

$$\frac{K_1}{K_2} = 65.26$$

Consolidation of Soil

Consolidation is defined as the process in which gradual reduction in volume of soil mass occurs under substantial loading and is primarily due to expulsion of pore water.

- In sand almost full consolidation take place as load is being applied and after effects are much smaller, but in fine grained soils after affects are more. That is why consolidation is mainly concerned with compressibility of fine grained soil.
- In the analysis of this process both water & soil particles are assumed to be relatively incompressibity so that the decrease in volume is entirely due to the change in relative positions of soil Particles.

* **Consolidation Process :-**

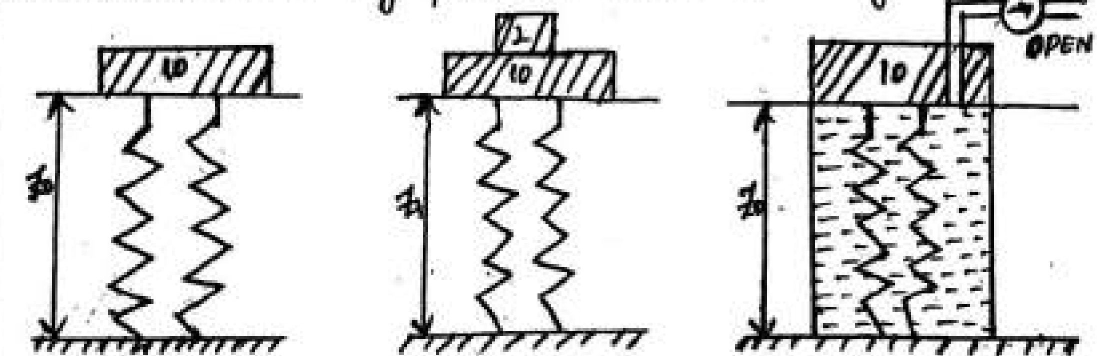
- When a clayey soil is subjected to loading, it first of all undergoes a very small compression due to explusion of air from its voids. This compression is called the initial compression.
- If the loading continues, then its volume goes on compressing gradually due to removal of water from the soil. This process is known as primary consolidation.

Once the excess pore water pressure becomes zero, that is excess pore water pressure gets fully dissipiated, the compression under the applied stress ends. It is found in practise that some compression takes place even after the primary compression has ceased. This is referred to as Secondary consolidation

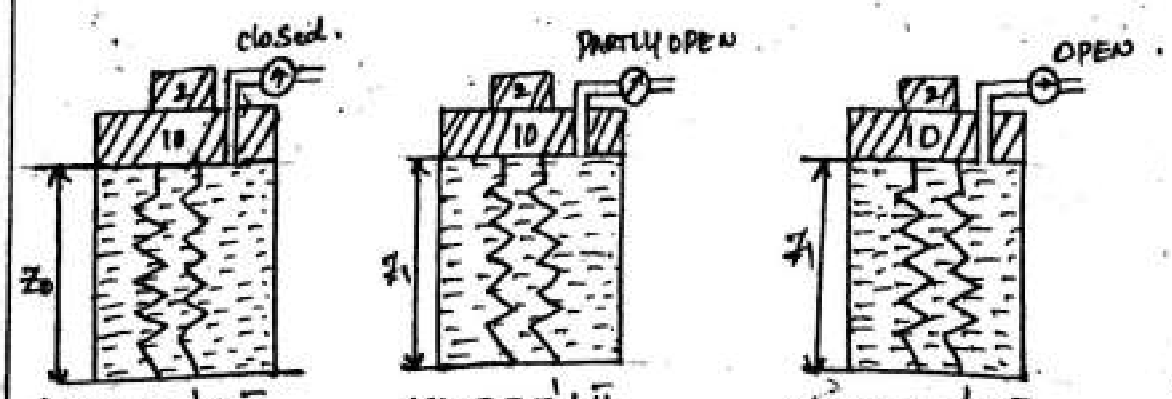
- In most soil deposits, the Secondary consolidation is very small compared to primary consolidation and is after neglected.

* Mass Spring Analogy:-

Terzaghi demonstrated the mechanics of consolidation by piston and Spring analogy.



(a) $\sigma = \sigma' = 10$ (b) $\sigma = \sigma' = 12$ (c) $\sigma = \sigma' = 10$



(d) $\sigma = \sigma' + \bar{u}$ (e) $\sigma = \sigma' + \bar{u}$ (f) $\sigma = \sigma' + \bar{u}$

$12 = (10 + \Delta\sigma')$

- In fig(d) addition pressure of 2units acts and the value is closed. Because of water is incompressible the springs are prevented from undergoing any further compression and therefore the additional pressure will have to be borne by water.

by analogy $\sigma = \sigma' + \bar{u}$

$12 = 10 + 2$

- In fig(e) the value is partly open & as the water starts flowing out transfering the additional pressure from water to spring commences and at any intermediate stage.

$\sigma = \sigma' + u'$

$12 = (10 + \nabla\sigma') + (2 - \nabla\sigma')$

where $\nabla\sigma'$ = part of the additional pressure transferred to springs at that stage.

- In fig(f) the value is shown fully open and rate of drainage of water increases and finally the drainage stops when all the additional pressure is transferred from water to spring. This is similar to the condition when excess pore water pressure has fully dissipiated in the case of soil mass.

$\sigma = \sigma' + \bar{u}$

- A Saturated soil mass is taken in a cylinder. Soil mass consists of soil particles forming the Skeleton of soil and Voids filled with water.
- The skeleton formed of soil particles can be assumed to be replaced by no of springs and the water filling
- The compressive stress is caused by load applied on piston placed on the top of the spring.
- An outlet with Value is provided to control drainage of water from out of the cylinder.
- Let Z_0 = Length of Spring under pressure of say 10 units. as shown in fig (a)

Let the length decrease to Z_1, when the pressure is increased to by say 2 units. fig (b)

In fig c, d, e, & f spring with piston is shown placed in a container filled with water.

- In fig (c) the Value is opened but no drainage takes place as the entire pressure of 10 units is borne by the springs and the pressure in water is zero.

For Soil mass, by Spring anology

$$\sigma = \sigma' + \bar{u}$$

where σ = total stress

σ' = Effective stress

$\bar{u}$ = Excess pore water pressure.

The piston & spring analogy helps a beginner in understanding the process of primary consolidation.

It is clear from the analogy model that in case of saturated soil mass subjected to an initial pressure σ & when no drainage is occurring.

$$\sigma = \sigma' + u$$

where u = pore water pressure under static condition.

* Terzaghi's One dimensional consolidation theory

Terzaghi derived the basic differential Equation of consolidation which represents the first stage in the theoretical analysis of the consolidation process.

- Assumptions

1. The Soil mass is homogenous and fully saturated.
2. The soil particles & water are Incompressible.
3. Darcy's law for flow of water through soil mass is applicable during consolidation.
4. Co-efficient of permeability is constant during consolidation.

5. Load is applied in one direction only & deformation occurs only in one direction.

6. Deformation is entirely due to decrease in volume.

7. The Drainage in pore water pressure occurs in only one direction.

8. During consolidation. the change in thickness is continous, but final value of compression is related to initial thickness only.

9.

- Limitations:-

1. The value of the co-efficient of consolidation has been assumed to be constant. In reality it changes with a change in the consolidation pressure.

2. The distance of the drainage path cannot be measured accurately in the field. The thickness of the deposit is generally variable & the average value has to be estimated.

3. There is sometimes difficult in locating the drainage path.

4. In field, the load is seldom applied instantaneously.

5. Initial consolidation & the secondary consolidation have been neglected.

* **Normally consolidated Soil**

The formation of soil in nature takes place by the deposition of disintegrated particles of rock by geological transporting agencies. As the deposition of soil is continued layer after layer the pressure on the bottom most layer is increased & gets compressed. Such soils are called normally consolidated soil.

* **Over Consolidated Soil**

After some time due to various geological phenomenon, Some portion of soil layer might get eroded resulting in decrease of over burden pressure. Such soil deposits which have been subjected to more stress in the past are said to be pre consolidated soil or over consolidated soil.

over consolidated soil may be formed due to the following factors such as

- Due to the weight of an overburden of soil which has eroded.
- Due to the weight of a continental ice sheets that melts

* **Under Consolidated Soil**

The soil is said to be under consolidated when it is not fully consolidated under existing over burden pressure.

* **Pre Consolidation pressure and its determination by Casagrande's method**

The maximum pressure to which an over consolidated soil had been subjected in the past is known as the pre consolidation pressure or over consolidated pressure ($\bar{\sigma}_c$)

- When a Soil specimen is taken from a natural deposit, the weight of the overlying material is removed. This causes an expansion of soil due to reduction in pressure. Thus, the specimen is generally pre consolidated.

- **Casagrande method**

A undisturbed sample of clay is consolidated in lab and void ratio e versus log σ is Plotted.

- The initial portion of the curve is flat & resembles the recompression curve of a remoulded specimen.

- The lower portion of the curve is a straight line.

- Casagrande construction

1. Select point 'A' of maximum curvature. Draw a horizontal line AB. Draw a tangent AC at point A.
2. Bisect the angle BAC. Let AD be the bisector.
3. Produce the straight line portion of the curve to meet AD at a portion of Point E.
4. The pressure 'P' at point E give the preconsolidation pressure.
5. Draw the vertical line EF through E which cuts the log $\bar{\sigma}$ axis at F. The point F indicates preconsolidation Pressure $\bar{\sigma}$.

* Consolidation characteristics of Soil

1. Coefficient of compressibility
2. Compression Index &
   Swell Index
3. Time factor
4. Co-efficient of Compressibility

* Laboratory consolidation test

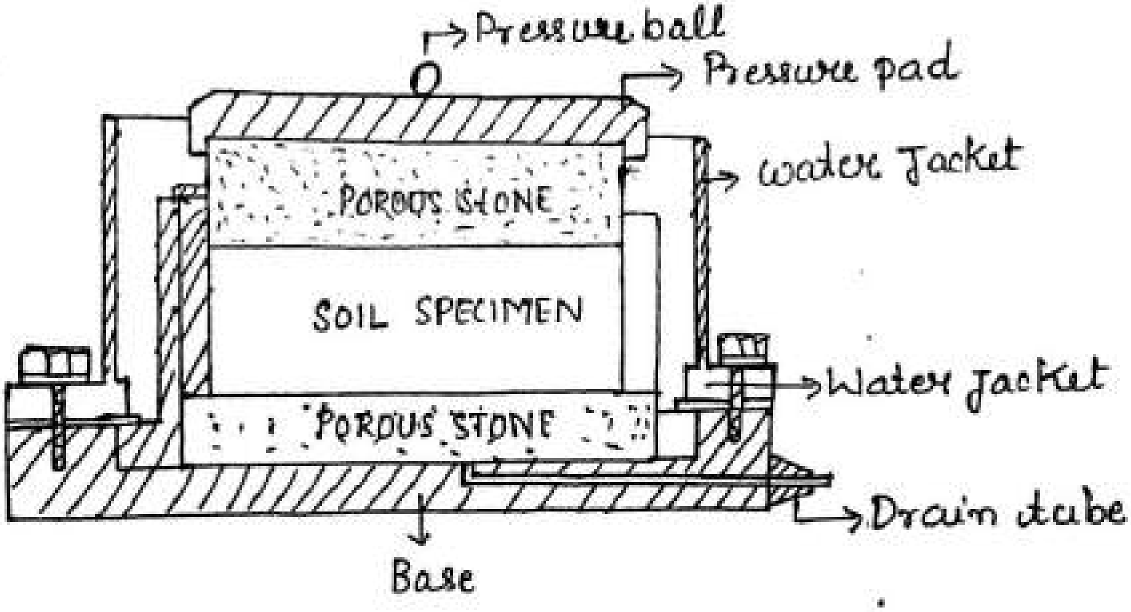
The laboratory consolidation test is conducted with an apparatus known as consolidometer consisting of a loading frame and consolidation cell in which the specimen is kept.

- Porous stone are put on the top and bottom ends of specimen.
- In the fixed ring cell, only the top porous stone is permitted to move downwards as the specimen compresses. Direct measurement of permeability of the specimen at any stage of loading can be made in the fixed ring type.
- During the test, the specimen is allowed to consolidate under a number of increments of vertical pressure, such as 10, 20, 50, 100, 200, 400, 800 & 1000 KN/m². The vertical compression of specimen is measured by means of dial guage.
- Dial guage readings are taken after the application of each pressure increment at the following total elapsed time. 0.25, 1, 2.25, 4.00, 6.25, 9.00, 12.25, 16.00, 20.25, 25, 36, 49, 60 min & 2, 4, 8 & 24 hrs.
- The dial guage readings showing the final compression under each pressure increment are also recorded. After the completion of consolidation under the desired maximum vertical load, the specimen is unloaded & allowed to swell.
- The final dial reading corresponding to the completion of swelling is recorded.

- The specimen is taken out and dried to determine its water content & weight of soil solids.

The consolidation test data are then used to determine the following

1. Void ratio & co-efficient of volume change
2. co-efficient of consolidation &
3. co-efficient of permeability.



* Fixed Ring Consolidation.

* Determination of Consolidation characteristics of Soils

Two methods by which characteristics can be determined are

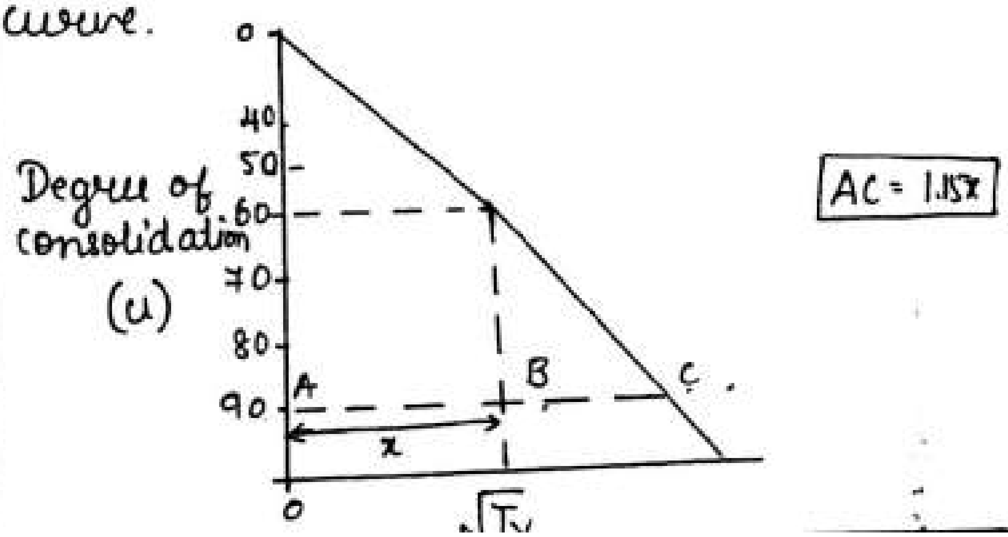
1. Square root of time fitting method
2. Logarithmic time fitting method

1. Square Root of time fitting method

In this method, given by Taylor, a theoretical consolidation curve is considered.

- An experimental consolidation curve, based on the consolidometer test data is also plotted, as a plot between $\sqrt{t}_{(min)}$ and dial guage reading (representing compression).

- It is seen that the theoretical curve is linear up to about 60% consolidation, and at 90% consolidation, the abscissa (AC) is about 1.15 times the abscissa (AB) of the extension of the linear part of the curve. This characteristics of the theoretical curve is used to determine the point of 90% consolidation of the experiment curve.



- The experimental curve usually consists of a small initial curved portion, a large linear part and finally a second curve.
- To seperate the initial compression ie to determine consolidation at t=0, in the experiment curve we extend back the straight line portion of the curve, so as to meet on y axis at point O' as against its start point O.
- It means OO' will represent the initial compression. The point O', where the extended straight line portion of the curve meets the vertical axis is called the corrected zero point.
- A new straight line is now drawn from the corrected zero point (O') such that its abscissa everywhere is 1.15 times the abscissa of the straight portion of the experimental plot, so as to cut the original curve at point E, which represents 90% consolidation corresponding to point 'E', t as t_{90} can be measured on x-axis, ie the value of C_v can be calculated. T_v for 90% U = 0.848

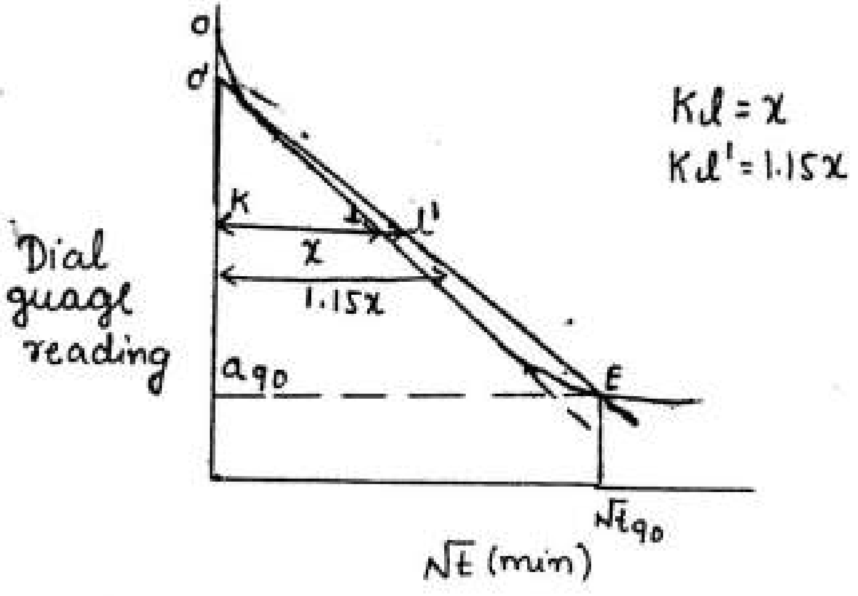
$$T_v = \frac{C_v}{d^2} \cdot t$$

$$\boxed{C_v = \frac{T_v d^2}{t}}$$

where T_v = Time factor corresponds to 90% U

C_v = co-efficient of consolidation

d = avg drainage path.



Experimental curve

2] <u>Logarithm of time fitting method</u>

This method was devised by cassagrande, uses log scale for plotting time on x-axis & dial guage reading.

The experimental & the theoretical curves obtained in this type of plotting are shown in fig(i). The charecteristics of theortical curve is to determine the 100% u on the semi log plot.

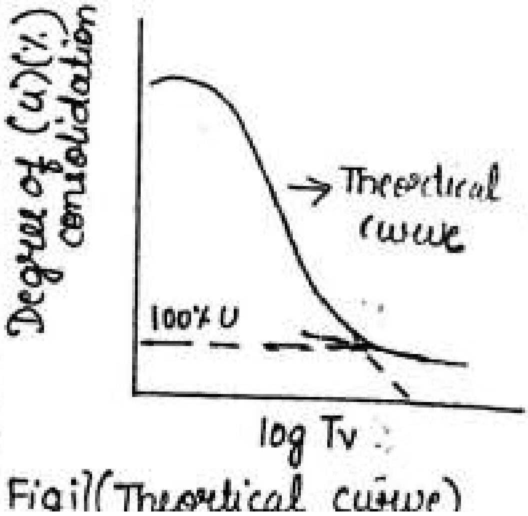
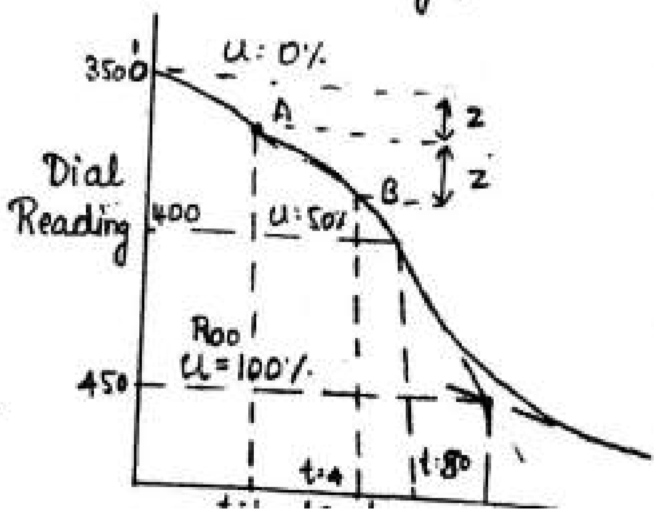


Fig i) (Theortical curve)



- The theortical curve consists of three parts ie an initial curve which approximates closely to a parabolic relationship, a linear & a final curve to which the x-axis is an asymptote
- In the experimental plot, the corresponding point to $U=0$ ie point O' can be determined by using the fact that the initial curve represents an approximately parabola.
- A time $t_1 = 1$ min is selected and its corresponding point 'A' is marked on the curve. Another point B is so selected on the curve that its corresponding time is in the ratio of 4:1 and the vertical distance between them is measured. An equal distance set off above the first point fixes the point O' corresponding to $U=0$.
- The dial reading R_{100} corresponding to $U=100\%$ is given by extending the straight portion of curves to meet the point 'P' which is the point of 100% consolidation. The point corresponding to $U=50\%$ can be located midway & the corresponding time t_{50} can be read out.

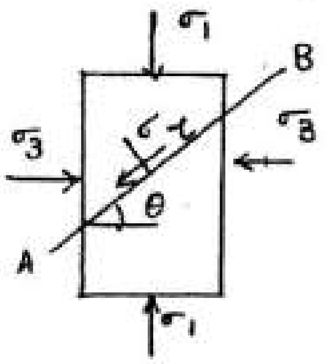
The theortical value of Tv corresponding to 50% U is 0.197 & the co-efficient of consolidation (Cv) is given by

$$C_v = \frac{0.197 d^2}{t_{50}}$$

Mohr Circle of Stress

In a stressed soil mass shear failure can occur along any plane and hence it is neccessary to study the stress condition at a point in a soil mass.

- At any stressed point, there exist three mutually perpendicular planes on which there is no shearing stress acting. These are known as the principal planes.
- The normal stresses that act on these planes are called the principal stresses, the largest of these is called the major principal stress σ_1, the smallest the minor principal stress σ_3 & the third one is called the intermediate principal stress σ_2. The corresponding planes are respectively designated the major, minor and intermediate principal planes.
- The stresses are therefore assumed to exist in two dimension rather than in three dimension. In other words, the state of stress in the plane containing only σ_1 & σ_3 will be considered.
-



- The direction of major and minor principal stress are shown in fig. These are in the horizontal and vertical directions respectively.
- If σ_1 & σ_3 are known, it can be shown analytically on the plane AB inclined at an angle θ to the direction of major principal plan

the normal & shear stress σ & τ can be given by

$$\sigma = \left(\frac{\sigma_1+\sigma_3}{2}\right) + \left(\frac{\sigma_1-\sigma_3}{2}\right)\cos 2\theta$$

$$\tau = \left(\frac{\sigma_1-\sigma_3}{2}\right)\sin 2\theta$$

Mohr demonstrated that these equations lend themselves to graphical representation. It can be shown that the locus of stress co-ordinates (σ, τ) for all planes through a point is a circle, the mohr circle of stress

* To draw Mohr circle

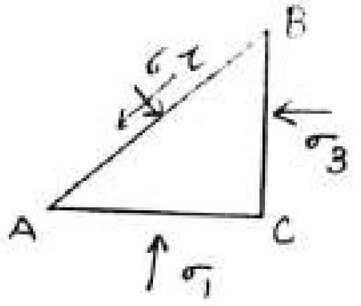
- To draw Mohr circle, the normal stress σ is plotted along the x-axis and shear stress τ is plotted along the Y axis.
- For convenience, compressive normal stresses are taken as +ve, becoz most of the normal stresses acting on soil are compressive in nature.
- Shear stresses the produce counter clockwise are considered +ve
- Given the values of principal stresses σ_1 & σ_3, a circle is constructed with its centre at $C\left(\left(\frac{\sigma_1+\sigma_3}{2}\right), 0\right)$ and radius is equal to $\frac{\sigma_1-\sigma_3}{2}$.
The circle cuts the X axis at 2 points.

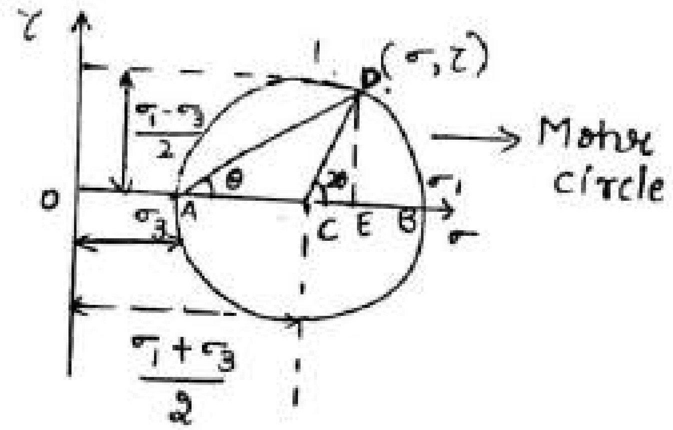
If a line is drawn through the point (σ_3, 0) and Parallel to the plane AB of fig(b), the line intersects the mohr circle at a point D whose coordinates represent the normal stress & shear stress on the Plane AB.

$\angle BCD = 2\theta$

$$\sigma = OE = OC + CE = \left(\frac{\sigma_1+\sigma_3}{2}\right) + \left(\frac{\sigma_1-\sigma_3}{2}\right)\cos 2\theta$$

& $$\tau = DE = \frac{\sigma_1-\sigma_3}{2}\sin 2\theta$$





Problems

1. In a consolidation test, a soil sample 20mm in thickness took 28min to reach 90% consolidation under two way drainage conditions, for the same soil in the field. What would be the time taken in days for 50% and 90% consolidation. If the thickness of soil layer is 4m and if there a) One way drainage b) Two way drainage.

Solu:-

$H = d = 20mm = 2cm$

$t = 28min$

$U = 90\%$ consolidation

i) $T_v = -0.9332\log_{10}\left[1-\frac{U}{100}\right] - 0.0851$

$T_v = -0.9332\log_{10}\left[1-\frac{90}{100}\right] - 0.0851$

$T_v = 0.8481$

ii] $T_v = \frac{C_v.t}{d^2}$ [For two way drainage $d = \frac{H}{2} = \frac{2}{2} = 1cm$]

$0.841 = \frac{C_v(28)}{(1)^2}$

$C_v = 0.0300\ cm^2/min$

* Field condition for 50% of U:

Thickness $d = 4m = 400cm$

U is less than 60%.

$$T_v = \frac{\pi}{4}\left[\frac{U}{100}\right]^2 = \frac{\pi}{4}\left[\frac{50}{100}\right]^2$$

$T_v = 0.196$

a) One way drainage:-

$d = 400cm$

$$Tv = \frac{Cv.t}{d^2}$$

$$0.196 = \frac{0.030 \times t}{400^2}$$

$t = 1045333.33\ min$

$t = 725.92\ days$

b) Two way drainage:- $\left(\frac{d}{2}\right)$

$$Tv = \frac{Cv.t}{(d/2)^2}$$

$$0.196 = \frac{0.030 \times t}{\left(\frac{400}{2}\right)^2}$$

$t = 261333.33\ min$

$t = 181.48\ days$

ii) Field condition for 90% U:-

$H = 4m = 400m$

$U = 90\% \geq 60\%$

$$Tv = -0.9332\log_{10}\left[1 - \frac{U}{100}\right] - 0.0851$$

$$Tv = -0.9332\log_{10}\left[1 - \frac{90}{100}\right] - 0.0851$$

$Tv = 0.848$

a) One way drainage:

$d = 400cm$

$$T_v = \frac{C_v t}{d^2}$$

$$0.848 = \frac{0.030 \times t}{(400)^2}$$

$t = 4522666.66 min$

$t = 3140.74 days$

b) Two way drainage:-

$$d/_2 = \frac{400}{2}$$

$$T_v = \frac{C_v . t}{d^2}$$

$$0.848 = \frac{0.030 \times t}{\left(\frac{400}{2}\right)^2}$$

$t = 1130666.66 min$

$t = 785.18 days$

2) The time for 40% condition of a two way drained saturation clay sample of 10mm thick in the laboratory is 40Sec. Determine the time required for 60% consolidation of the same soil 12m thick on an impervious layer subjected to same loading condition on the laboratory Sample

Solu :-

$U = 40\% \leq 60\%$, $d = 10mm$, $t = 40Sec$

$$T_v = \frac{\pi}{4}\left[\frac{U}{100}\right]^2 = \frac{\pi}{4}\left[\frac{40}{100}\right]^2 = 0.126$$

Two way drainage, $\frac{d}{2} = \frac{10}{2} = 5mm$

$$T_v = \frac{C_v . t}{(d/2)^2}$$

$$0.126 = \frac{C_v(40)}{5^2}$$

$$C_v = 0.0787 \, mm^2/Sec$$

For 60% condition

$U = 60\%$, ~~d~~ $d = 12m = 12000mm$

considering one way drainage

$$T_v = \frac{\pi}{4}\left[\frac{U}{100}\right]^2 = \frac{\pi}{4}\left[\frac{60}{100}\right]^2 = 0.282$$

$$T_v = 0.282$$

$$C_v = 0.0787 \, mm^2/Sec$$

One way drainage

$$T_v = \frac{C_v t}{d^2}$$

$$0.282 = \frac{0.0787 \times t}{(12000)^2}$$

$$t = 515984752.2 \, Sec$$

$$t = 5972.04 \, day$$

$$a_v = \frac{\Delta e}{\Delta \sigma} = \frac{e_0 - e}{\sigma' - \sigma_0'} = \frac{1.9 - 1.86}{196 - 147} = 8.16 \times 10^{-4}$$

$$a_v = 8.16 \times 10^{-4}\, m^2/KN$$

$$m_v = \frac{8.16 \times 10^{-4}}{1 + 1.9} = 2.81 \times 10^{-4}\, m^2/KN$$

iii) Coefficient of consolidation (C_v)

$$C_v = \frac{K}{m_v} \times \gamma_w$$

$$C_v = \frac{3.2 \times 10^{-10}}{2.81 \times 10^{-4}} \times 9.81$$

$$C_v = 1.17 \times 10^{-5}\, m^2/sec$$

iv) Time required for 50% consolidation

$$U = 50\% \leq 60\%$$

$$T_v = \frac{\pi}{4}\left[\frac{U}{100}\right]^2 = \frac{\pi}{4}\left[\frac{50}{100}\right]^2$$

$$T_v = 0.196$$

One way drainage

$$T_v = \frac{C_v . t}{d^2}$$

$$0.196 = \frac{1.17 \times 10^{-5} \times t}{(5)^2}$$

$$t = 418803.41\, sec$$

3. An Saturated Soil layer 5m thick lies above a impervious stratum. Below a pervious stratum it has a compression Index = 0.25, $K = 3.2 \times 10^{-10}$ m/s. Its void ratio of 147 KN/m² is 1.9. calculate

i) The change in Void ratio due to increase of stress to 196 KN/m².

ii) Coefficient of Volume compressibility

iii) co-efficient of consolidation

iv) Time taken for 50% consolidation.

Solu :-

$C_c = 0.25$

$K = 3.2 \times 10^{-10}$ m/s

$e_0 = 1.9$

$\sigma_0' = 147$ KN/m²

$d = 5$m

i) The change in void ratio due to increase in stress to 196 KN/m², $\sigma' = 196$ KN/m²

$$C_c = \frac{e_0 - e}{\log\left[\frac{196}{147}\right]} \quad \frac{\sigma'}{\sigma_0'}$$

$$0.25 = \frac{1.9 - e}{\log\left[\frac{196}{147}\right]} \qquad e = 1.86$$

ii) Coefficient of volume compressibility (m_v)

$$m_v = \frac{a_v}{1 + e_0}$$

4) In a consolidation test the void ratio decreased from 0.70 to 0.60, when pressure changed from 50KN/m² to 100KN/m². Determine
a) compression Index b) Coefficient of compressibility
c) co-efficient of volume change.

Solu :-

Given $e_0 = 0.70$

$e = 0.60$

$\sigma' = 100 KN/m^2$

$\sigma_0' = 50 KN/m^2$

a) compression Index :-

$$C_c = \frac{e_0 - e}{\log_{10}\left[\frac{\sigma'}{\sigma_0'}\right]} = \frac{0.70 - 0.60}{\log_{10}\left[\frac{100}{50}\right]}$$

$C_c = 0.332$

b) coefficient of compressibility (a_v) :-

$$a_v = \frac{e_0 - e}{\sigma' - \sigma_0'} = \frac{0.70 - 0.60}{100 - 50} = 2 \times 10^{-3} m^2/KN$$

$a_v = 2 \times 10^{-3} m^2/KN$

c) Coefficient of volume change (m_v) :-

$$m_v = \frac{a_v}{1 + e_0} = \frac{2 \times 10^{-3}}{1 + 0.7} = 1.17 \times 10^{-3} m^2/KN$$

5) In a consolidation test the void ratio of the sample decreases from 1.20 to 1.10 when the pressure is increased from 160 to 320 kN/m^2. Calculate the co-efficient of consolidation, if co-efficient of permeability is 8.0×10^{-10} m/s.

Soln :-

$e_0 = 1.20$

$e = 1.10$

$\sigma_0' = 160 \, kN/m^2$

$\sigma' = 320 \, kN/m^2$

$K = 8.0 \times 10^{-10} \, m/s$

$$a_v = \frac{e_0 - e}{\sigma' - \sigma_0'} = \frac{1.20 - 1.10}{320 - 160} = 6.25 \times 10^{-4} \, m^2/kN$$

i) Coefficient of volume change (m_v)

$$m_v = \frac{a_v}{1 + e_0} = \frac{6.25 \times 10^{-4}}{1 + 1.20}$$

$$m_v = 2.84 \times 10^{-4} \, m^2/kN$$

ii) Coefficient of consolidation (C_v)

$$C_v = \frac{K}{m_v} \times \gamma_w$$

$$= \frac{8.0 \times 10^{-10}}{2.84 \times 10^{-4}} \times 9.81 = 2.76 \times 10^{-5} \, m^2/sec$$

$$C_v = 2.76 \times 10^{-5} \, m^2/sec$$

Shear Strength of Soil

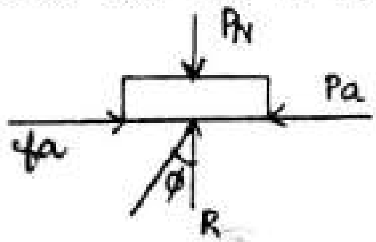
Introduction :-

The Shear strength of Soil is the maximum resistance offered by a soil to shearing stress
It is a measure of the soil resistance to deformation by continous displacement of its individual soil particles
Shear strength of soil depends primarily on iteraction between particles
Shear failure occurs when the Stresses between the particles are such that they slide or roll past each other.
Soil derives its shear strength from two sources namely as.
i] cohesion between the particles
ii] frictional resistance & interlocking between particles

* Basic Concept of Shear Strength

The basic Concept of Shear strength can be explained by understanding the basic principle of fric- tions

Consider a block resisting on a plane surface subjected to Pa & PN force as shown in fig.



where
fa = frictional force
R = normal Strain
ϕ : angle of Repose

When the force P_a is applied to block gradually due to opposition of force developed by the soil known as frictional force, the block will not move.

When Pa reaches maximum fa, the block will start sliding. Thus, the force applied is a shearing force and the developed force is friction or shearing strength. If the above expressions are repeated with higher normal value P_N, the shearing force fails proportional to the normal load P_N.

$$f_a = P_N \tan\phi$$

$$\frac{f_a}{A} = \frac{P_N}{A} \tan\phi$$

$$\boxed{\tau = \sigma \tan\phi}$$

where
τ = Shearing Streng
σ = normal stress

* <u>Mohr's Coulomb</u> Failure <u>theory</u> or <u>Mohr Strength theory</u>

Mohr (1990) presented a theory for rupture in materials which can be conveniently applied in case of soils based on the following simplified assumptions.

1. The Soil fails by shear: The critical shear stress causing failure depends on the properties of the material as well as normal stress acting on the failure plane.
2. The ultimate strength of the material is determined by the stresses acting on the potential failure plane.
3. The intermediate principal stress does not have any influence on the strength of the material.

The theory was first expressed by coulomb (1776) and later generalised by Mohr. Thus, the function relationship between normal stress and shear stress on a failure plane can be expressed mathematically in the form of

$$\boxed{\tau_f = F(\sigma)} \rightarrow (1)$$

where τ_f = Shear strength of soil at failure

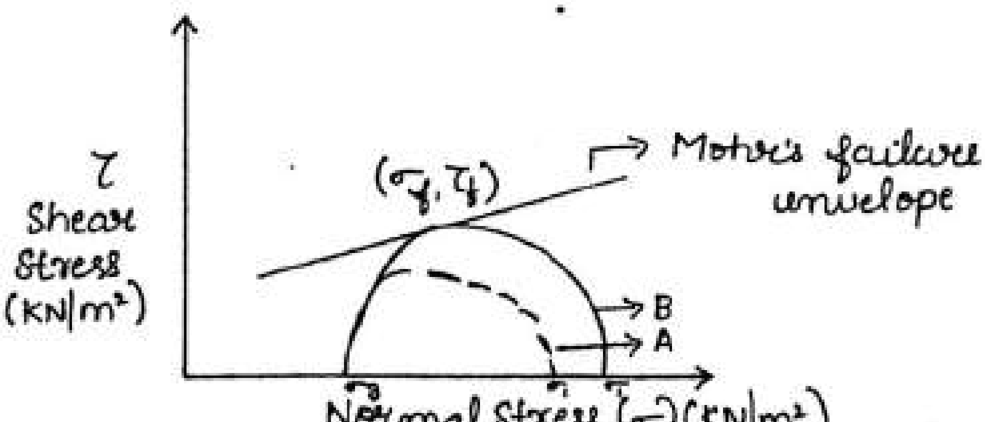
$F(\sigma)$ = function of normal stress

Mohr also expressed in terms of principal Stresses σ_1 & σ_3 as

$$(\sigma_1 - \sigma_3) = F(\sigma_1 + \sigma_3)$$

If the normal & shear stress corresponding to failure are plotted, then a curve is obtained. It may be noted that any mohr's circle such as circle 'A' lying below the failure envelope represents stable condition.

- Mohr's circle (B) which touches the failure envelope tangentially represents a failure condition.
- Circle lying above the failure envelope cannot exist as material will fail before reaching that state of stress.



a) Mohr's Strength Envelope

Coulumb defined the function $F(\sigma)$ as a linear function of τ_f and gave the following Strength Equation, which is most commonly used.

$$S = \tau_f = C + \sigma \tan\phi$$

where C = cohesion

ϕ = angle of internal friction

σ = Normal stress acting over failure plane.

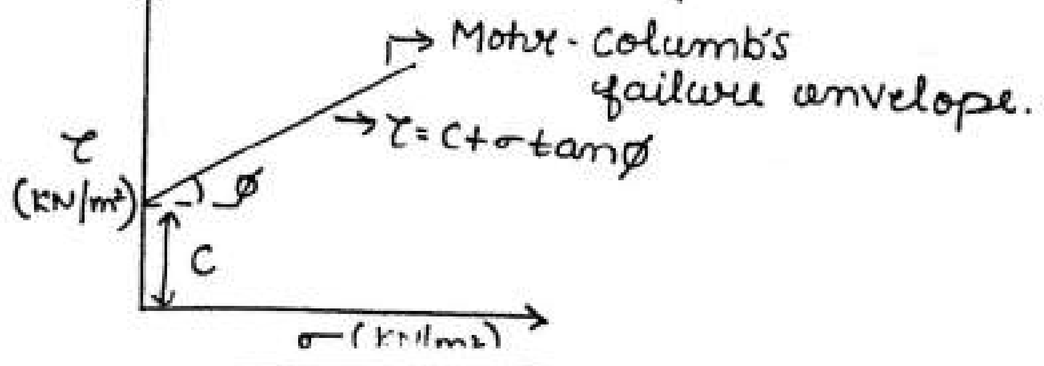
The above equation if plotted will give a failure envelope as a straight line as shown in the fig A below.

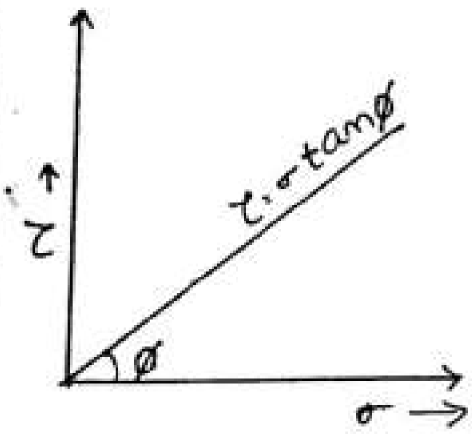
The parameters 'C' and ϕ are generally called Shear strength parameters.

For a purely cohesionless or granular soil or Sand, the failure envelope will pass through the origin '0' as shown in the fig B.

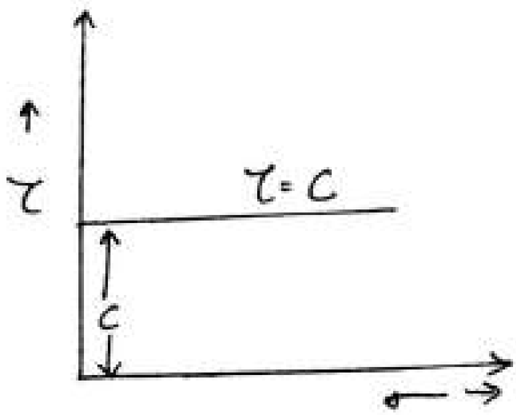
$$\tau_f = \sigma \tan\phi$$

Similarly, for a pure cohesive soil or pure clay ($\phi = 0$), the failure envelope is as shown in the fig C and represented by the Equation. $\tau_f = C$





B) Pure cohesionless soil (c = 0 or φ soil)



c) Pure clay or cohesive soil

Envelopes for φ soil & c-soil

* **Sensitivity of clay**

- The consistency of an undisturbed sample of clay is altered, even at the same water content, if it is remoulded.
- It is because the original structure of clay is altered by reworking or remoulding.
- The degree of disturbance of undisturbed clay sample due to remolding is expressed by Sensitivity.
- Sensitivity is defined as the ratio of its unconfined compressive strength in the natural state to that in the remoulded state, without change in the water content.

$$S_t = \frac{q_u\,(\text{undisturbed})}{q_u\,(\text{remoulded})}$$

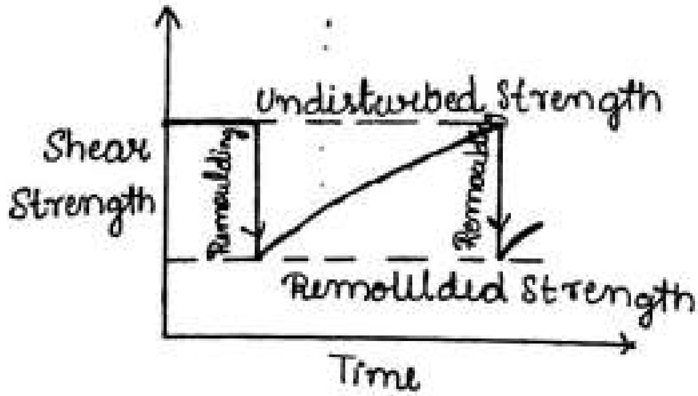
Based on Sensitivity soil can be classified as

Sensitivity	Nature of clay
1	Insensitive
2-4	Normal / Less sensitive
4-8	Sensitive
8-16	Extra sensitive
>16	Quick

* Thixotropy of clay

- When sensitive clay is used in construction, they loose strength due to remoulding during construction operation.
- However, with the passing of time, the strength again increases, though not to the same original level.
- This phenomenon of "Strength loss or strength gain" with no change in volume or water content is called thixotropy.
- Thus, thixotropy is defined as an isothermal reversible, time dependent process which occurs under constant composition & volume, thereby a material softens as a result of moulding and then gradually returns to its original strength when allowed to rest.

This phenomenon is shown in fig.



- Larger the sensitivity, larger will be the thixotropy hardening.
- The loss of Strength due to remoulding is partly due to
  1. Permanent destruction of the Structure
  2. Reorientation of the molecules in

- The gain of strength is due to
    1. Rehabilation of molecular stress or Soil &
    2. Its thixotropic property

* ## Total Shear strength

At any plane in a soil mass, total stress or pressure (σ) is the total load per unit area. This pressure may be due to

1 Self wt of the soil
2. Overburden on the soil

The total pressure consist of two distinct components

1. Intergranular pressure or effective pressure or effective stress (σ')
2. Neutral pressure or pore water Pressure or pore pressure / Pore Stress (u)

- Effective Stress (σ')

It is defined as the intergranular pressure which is transmitted by grain to grain contact. This stress effectively reduce void ratio and increases the shear strength. Thus, certain aspects of behavior of soil like volume change, shearing resistance are controlled by effective stress.

- Neutral Stress or Pore water pressure (u) :-

It is the pressure transmitted through the water voids (pores). It is also known as Pore water pressure or simply pore pressure.

As the pore pressure acts on all the sides of soil particles. it is non effective in decreasing void ratio or increasing shear strength.

The magnitude of pore pressure is equal to piezometric head (h) times unit wt of water.

ie $\boxed{u = \gamma_w h}$

Total Pressure (σ):-

In a fully saturated soil, the total pressure on any plane is the sum of effective pressure and the neutral pressure. And is given by

$$\boxed{\sigma = \sigma' + u}$$

Effective pressure can also be defined as the difference of total pressure & neutral pressure.

ie $\boxed{\sigma' = \sigma - u}$

Problems :-

1. Determine the shear strength in terms of effective stress on a plane within a saturated soil mass at a point where the total normal stress is 200KN/m² and the pore water pressure is 80KN/m². The effective stress, shear strength Parameters for the soil are $c' = 16 KN/m^2$

Soln :-

$\sigma = 200 KN/m^2$

$u = 80 KN/m^2$

$c' = 16 KN/m^2$

$\phi' = 30°$

$\tau_f = c' + \sigma' \tan\phi'$

$= 16 + 120 \tan 30°$

$\boxed{\tau_f = 85.30 KN/m^2}$

$\sigma' = \sigma - u$

$= 200 - 80$

$\sigma' = 120 KN/m^2$

2. In an insitu vane shear test on a saturated clay, a torque of 35Nm was required to shear the soil. The diameter of the vane was 50mm & length 100mm. calculate the undrained shear strength of the clay. The vane was then rotated rapidly to cause remolding of the soil. The torque required to shear the soil in the remolded state was 5Nm. Determine the sensitivity of the clay.

Solution:-

$T = 35\,Nm$, $D = 50mm$
$H = 100mm$

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

The undrained shear strength of clay is

$$35 = \tau_f \pi (0.05)^2 \left[\frac{0.05}{6} + \frac{0.1}{2}\right]$$

$$\boxed{\tau_f = 76.39\, kN/m^2}$$

The undrained shear strength in remolded state

$T = 5\,Nm$

$$5 = \tau_f \pi (0.05)^2 \left[\frac{0.05}{6} + \frac{0.1}{2}\right]$$

$$\boxed{\tau_f = 10.91\, kN/m^2}$$

Sensitivity of clay

$$S_t = \frac{q_u(\text{undisturbed})}{q_u(\text{remolded})}$$

$$= \frac{76.39}{10.9} = 6.99 \approx 7$$

∴ The soil is said to be <u>Sensitive</u>.

3) Following data refers to three triaxial tests conducted on representative undisturbed sample of Soil

Test No	Cell Pressure σ_3 (KN/m²)	Axial load in (N) dial reading at failure
1.	50	66
2.	150	106
3	250	147

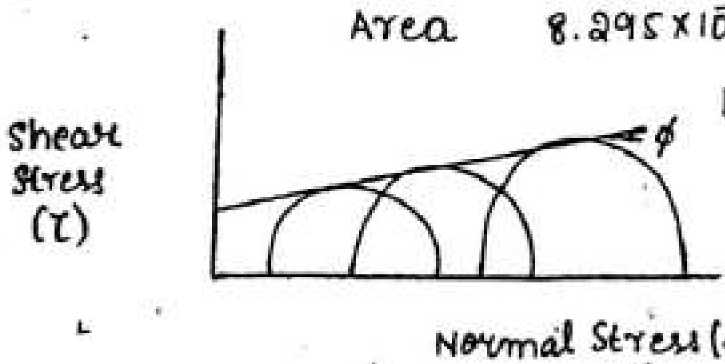
Load dial calculation factor is 1.4N/div Each sample is 75mm long and 32.5mm in diameter. Find the shear parameter

Solution:- Area = $8.295 \times 10^{-4} m^2$

Test No	Cell (σ_3) Pressure	Axial load in dial	Axial load (KN)	Deviator Stress (KN/m²)	$\sigma_1 = \sigma_d + \sigma_3$
1.	50	66	0.0924	111.38	161.38
2.	150	106	0.1484	178.90	328.9
3.	250	147	0.208	250.75	500.75

$$\text{Axial load} = \frac{66 \times 1.4}{1000} = 0.0924 \text{ kN}$$

$$\text{Deviator Stress} = \frac{\text{load}}{\text{Area}} = \frac{0.0924}{8.295 \times 10^{-4}} = 111.38 \text{ KN/m}^2$$



From Graph

$c = 35 \text{ KN/m}^2$

$\phi = 15°$

4] A Series of undrained triaxial shear test on samples of saturated soil gave the following results.

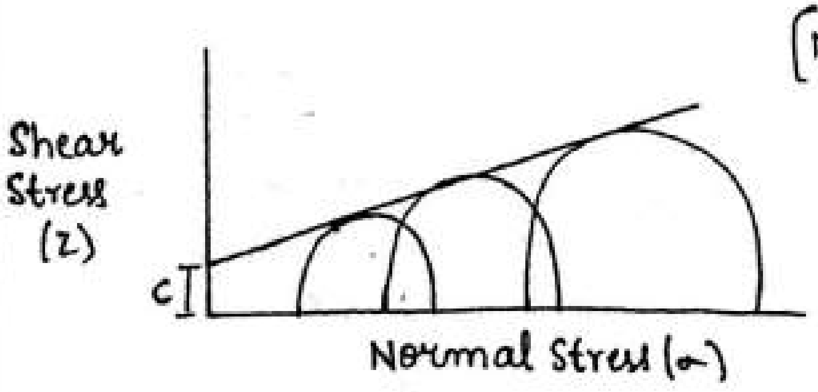
Lateral pressure (KN/m²) (σ_3)	100	200	300
Pore water pressure (KN/m²)	20	50	95
Principle stress diff at failure ($\sigma_1 - \sigma_3 = \sigma_d$)	290	400	490

Find the values of shear parameter
a) w.r.t effective stress
b) w.r.t total stress

Solution :-

w.r.t total stress

Cell pressure σ_3 (KN/m²)	Deviator Stress (σ_d)	Major Principle stress σ_1 (KN/m²) ($\sigma_1 = \sigma_d + \sigma_3$)
100	290	390
200	400	600
300	490	790



[Note: Both x axis & y axis should have Same Scale]

$C = 73 KN/m^2$

$\phi = 20°$

w.r.t Effective stress

Effective cell pressure $\sigma_3' = \sigma_3 - u$	Deviator stress σ_d (kN/m²)	Effective major Principle stress $\sigma_1' = \sigma_3' + \sigma_d$
80	290	370
150	400	550
205	490	695

From graph } $c' = 50\ kN/m^2$, $\phi' = 25°$

5] Unconfined compressive strength of a soil is found to be 150 kN/m². A sample of the same soil failed at a deviator stress of 200 kN/m² when tested in a consolidated undrained triaxial compression test with a cell pressure of 50 kN/m². Determine the shear strength of soil.

Solution :-

$\sigma_1 = 150\ kN/m^2$
$\sigma_d = 200\ kN/m^2$ } UCC test

$\sigma_3 = 50\ kN/m^2$
$\sigma_d = 200\ kN/m^2$
$\sigma_1 = 250\ kN/m^2$ } Triaxial test

$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$

UCC $150 = 2c\tan(\alpha)$ — (1)

Triaxial $250 = 50\tan^2\alpha + 2c\tan\alpha$ — (2)

Solving 1 & 2

$150 = 2c\tan\alpha$

$(-)\ 250 = 50\tan^2\alpha + 2c\tan\alpha$

$-100 = -50\tan^2\alpha$

$\tan^2\alpha = 2$

$\tan\alpha = \sqrt{2}$

$\boxed{\alpha = 54.73}$

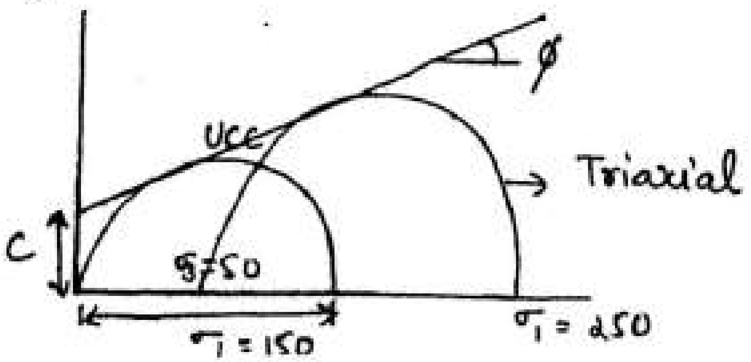
$\alpha = 45 + \frac{\phi}{2}$

$\boxed{\phi = 19.46}$

$c = \frac{150}{2\tan\alpha} = \frac{150}{2\sqrt{2}} = 53.03\ kN/m^2$

$\boxed{c = 53.3\ kN/m^2}$

Graphically:-



6] A Cylinder of soil fails under an axial vertical stress 150 KN/m², when it is laterally unconfined. The plane makes an angle of 50° with horizontal. Calculate Shear parameters.

Solu :

Analytic method

w.r.t $\sigma_1 = \sigma_3 \tan^2\alpha + 2c \tan\alpha$

$\sigma_3 = 0$

$\sigma_1 = 2C_u \tan\left(45 + \frac{\phi_u}{2}\right)$

$\alpha = 45 + \frac{\phi_u}{2}$

$50 = 45 + \frac{\phi_u}{2}$

$\boxed{\phi_u = 10^\circ}$

$150 = 2 \times C_u \tan\left(45 + \frac{10}{2}\right)$

$\boxed{C_u = 62.9 \text{ KN/m}^2}$

7] A cylindrical specimen of saturated clay 40mm in dia & 80mm in length is tested in an unconfined compression test. Find shear strength of clay, if the specimen fails under an axial load of 46.5N. The change in length of the specimen at failure is 10mm.

Solu : d = 40mm, l = 80mm

$A_o = \frac{\pi d^2}{4} = \frac{\pi (40)^2}{4} = 1256.63 \text{ mm}^2$

Solution:-

UCC $\Rightarrow \sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$ ($\sigma_3 \to 0$)

$\sigma_1 = 120 KN/m^2$

$\sigma_1 = 2c\tan\alpha \Rightarrow 120 = 2c\tan\alpha$ — (1)

Triaxial $\Rightarrow$ $\sigma_3 = 40 KN/m^2$

$\sigma_d = 160 KN/m^2$

$\sigma_1 = 200 KN/m^2$

$200 = 40\tan^2\alpha + 2c\tan\alpha$ — (2)

Solving 1 & 2

$120 = 2c\tan\alpha$

$200 = 2c\tan\alpha + 40\tan^2\alpha$

(-) (-) (-)

$-80 = -40\tan^2\alpha$

$\tan^2\alpha = 2$

$\alpha = 54.73$

$\alpha = 45 + \frac{\phi}{2}$

$\phi = 19.46$

Now, $2c\tan\alpha = 120$

$c\tan\alpha = \frac{120}{2}$

$c = \frac{120}{2\tan\alpha} = 42.42 KN/m^2$

$c = 42.42 KN/m^2$
$\phi = 19.46$

9] A Soil Specimen having $c = 86 kN/m^2$ & $\phi = 30°$ is tested in triaxial apparatus. Estimate

i) the deviator stress at which the sample fails when the cell pressure is $60 kN/m^2$.

ii] the cell pressure when the sample fails at a major principle stress of $900 kN/m^2$.

Solu:-

$c = 86 kN/m^2$ & $\phi = 30°$

i] $\sigma_d = ?$

$\sigma_3 = 60 kN/m^2$

$\sigma_1 = \sigma_3 \tan^2\alpha + 2c \tan\alpha$

$\sigma_1 = 60 \tan^2\left(45 + \frac{30}{2}\right) + 2(86)\tan\left(45 + \frac{30}{2}\right)$

$\sigma_1 = 180 + 297.91 = 477.91 kN/m^2$

$\sigma_1 = \sigma_3 + \sigma_d \Rightarrow \sigma_d = \sigma_1 - \sigma_3 = 417.91$

$\boxed{\sigma_d = 417.91 kN/m^2}$

ii] $\sigma_3 = ?$

$\sigma_1 = 900 kN/m^2$

$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$

$900 = \sigma_3 \tan^2\left(45 + \frac{30}{2}\right) + 2 \times 86 \times \tan\left(45 + \frac{30}{2}\right)$

$\boxed{\sigma_3 = 200.69 kN/m^2}$

10) A Sample of soil failure in a triaxial test under a deviator stress of 200KN/m² when the confining pressure was 100KN/m². If, for the same sample the confining pressure had been increased to 200KN/m², what would have been the deviator stress.

when

$C = 0$

$\phi = 0$

Solu:

$$\boxed{\sigma_1 = \sigma_3 \tan^2\alpha + 2C\tan\alpha}$$

i) when $C = 0$

* First condition

$$\sigma_1 = \sigma_3 + \sigma_d$$

$$\sigma_1 = 100 + 200 = 300\,KN/m^2$$

Above equa reduces to

$$\sigma_1 = \sigma_3 \tan^2\alpha$$

$$300 = 100 \tan^2\alpha$$

$$\tan^2\alpha = 3$$

* Second condition

$$\sigma_1 = \sigma_3 \tan^2\alpha$$

$$\sigma_1 = 200 \times 3 = 600\,KN/m^2$$

But $\sigma_1 = \sigma_3 + \sigma_d$

$$600 = 200 + \sigma_d$$

$$\boxed{\sigma_d = 400\,KN/m^2}$$

ii] When $\phi = 0$

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c \tan\alpha$$

$$\alpha = 45 + \frac{\phi}{2}$$

First condition → $\sigma_1 = 100 + 200 = 300 \, kN/m^2$

$$300 = 100 \tan^2 45 + 2c \tan 45$$

$$300 = 100 + 2c$$

$$2c = 200$$

$$\boxed{c = 100 \, kN/m^2}$$

Second condition

$$\sigma_1 = 200 \tan^2 45 + 2(100) \tan 45$$

$$\sigma_1 = 400 \, kN/m^2$$

$$\sigma_1 = \sigma_3 + \sigma_d \Rightarrow \sigma_d = 400 - 200 = \underline{200 \, kN/m^2}$$

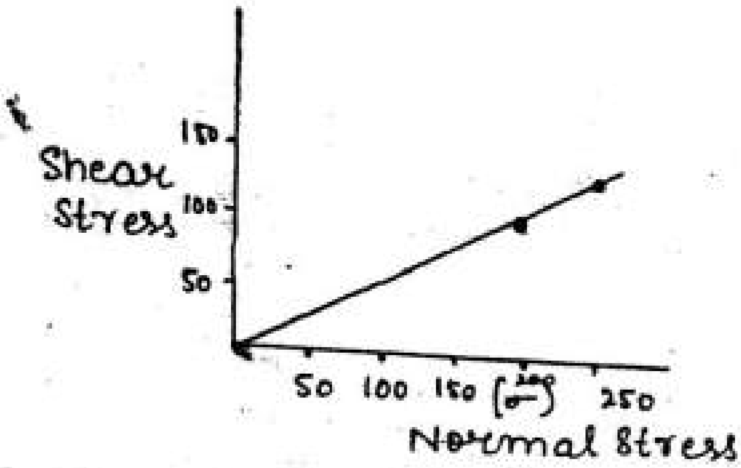
11] A shear box test is carried out and the following results were obtained.

Normal Stress (kN/m^2)	200	250
Shear Stress (kN/m^2)	100	125

i] Find shear parameters

ii] What would be the deviator stress at failure if a triaxial test is carried out from the same soil with cell pressure of $100 \, kN/m^2$

Solution :-



From Graph $c = 0$

$\phi = 26.56$

or

Solving analytically

Shear stress at failure

$\tau_f = c + \sigma \tan\phi$

1st Specimen

$\tau_f = 100$

$\sigma = 200$

$100 = c + 200 \tan\phi$ — (1)

2nd specimen

$\tau_f = 125$

$\sigma = 250$

$125 = c + 250 \tan\phi$ — (2)

Solving 1 & 2

$$100 = c + 200 \tan\phi$$
$$(-)\ 125 = c + 250 \tan\phi$$
$$-25 = -50 \tan\phi$$

$\tan\phi = 0.5$

$$\boxed{\phi = 26.56}$$

$125 = c + 250 \tan 26.56$

$$\boxed{c = 0}$$

ii) $\sigma_d = ?$

$\sigma_3 = 100 KN/m^2$

$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$ (→ 0)

$= 100 \tan^2(45 + \frac{\phi}{2})$

$= 100 \tan^2(45 + \frac{26.56}{2})$

$$\boxed{\sigma_1 = 261.75 KN/m^2}$$

$\sigma_1 = \sigma_3 + \sigma_d$

$\sigma_d = 261.75 - 100$

$$\boxed{\sigma_d = 161.75 KN/m^2}$$

①

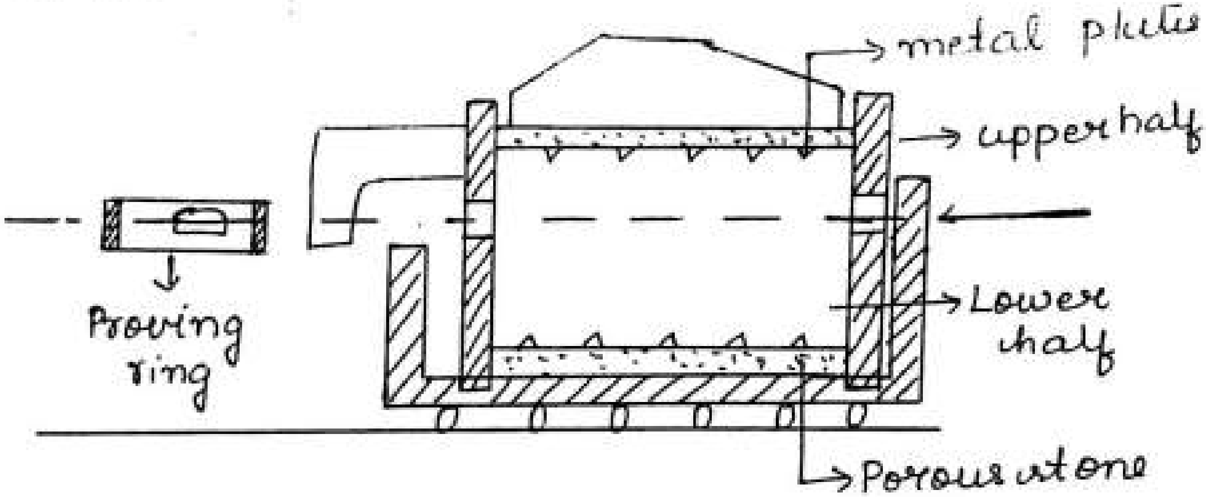
Pallavi P.

Determination of Shear Strength &

I Measurement of Shear parameters

1. Direct Shear Test

* Apparatus :-



* Procedure :-

The apparatus consists of a square box split horizontally at the level of the centre of the soil specimen, which is held between metal grilles & porous stones.

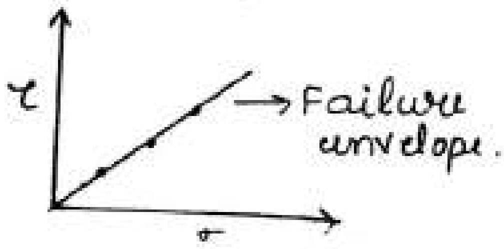
- In direct shear test, a sample of soil is placed into the shear box. The size of box normally used is 6 x 6cm & the sample is 2cm thick.

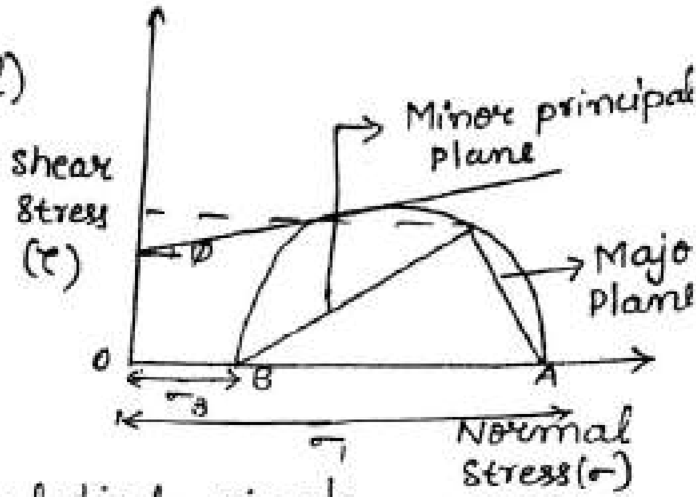
- The soil used for the test are either undisturbed samples or remolded.

- If undisturbed, the specimen has to be carefully trimmed & fitted into the box.
If remolded samples are required, the soil is placed into the box in layers at the required water content and damped to the required dry density.
- Normal load is applied on the specimen & is held constant during a test.
- A gradually increasing horizontal load (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 & hence on to the proving ring.
The deformation of proving ring indicates the shear force. A Shear is normally applied at constant rate.
- The Volume change during the consolidation & during the shearing process is measured by mounting a dial guage at the top of the box.
- The load is applied to the box until the sample fails in shear.
- The shear force at failure, corresponding to the normal load N is measured with the help of the proving ring. A number of identical specimens are tested under increasing normal load.
- A graph is plotted between the shear Stress as ordinate & the normal Stress as the abscissa. Such a plot gives the failure envelope for the soil under the given test conditions.

- The normal stress and shear stress on the failure plane are determined by dividing normal load & shear load by the nominal c/s area of the specimen

- Failure envelope for sandy soil (ø soil)





* Advantages

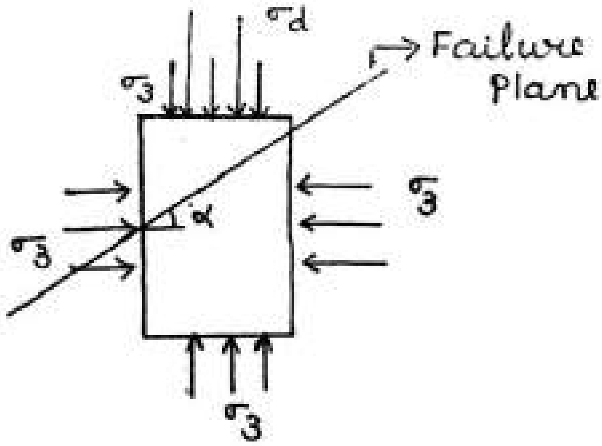
1. Direct shear test is relatively simple.
2. Quick dessipation of pore water pressure is possible since the thickness of specimen is small.
3. It is very easy to conduct test on Sandy soil.

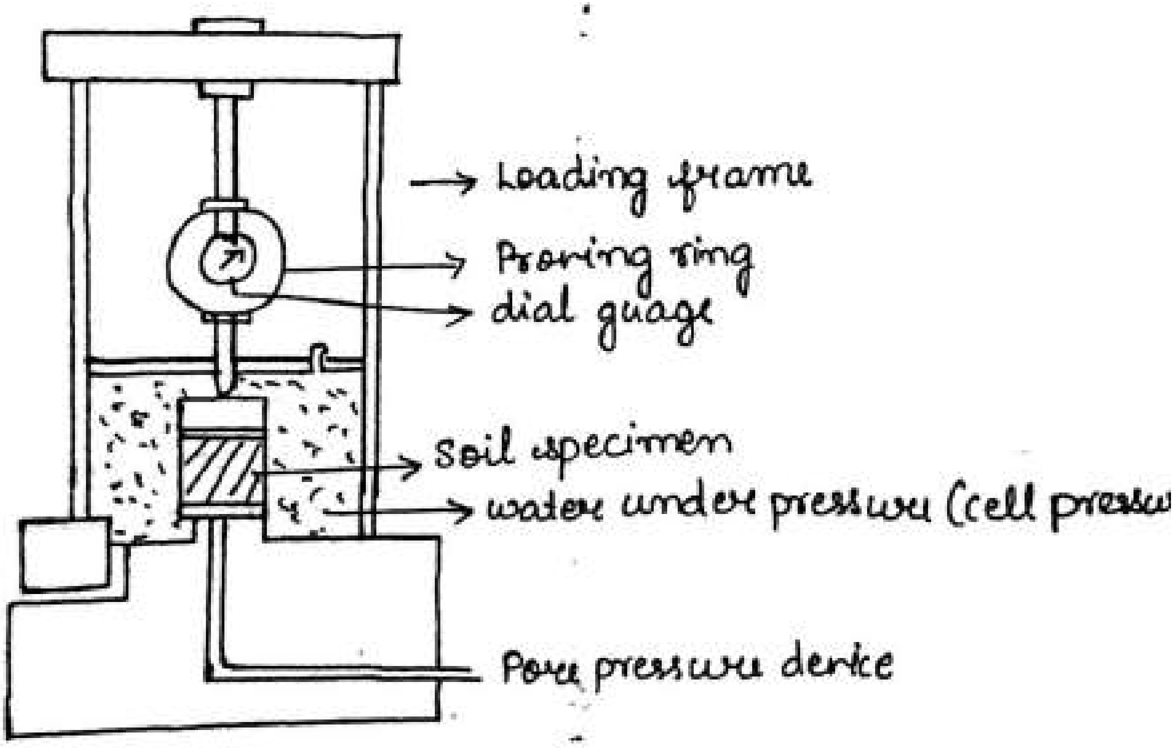
* Disadvantages (limitations):-

1. Failure planes is predetermined and this may be the weakest plane. In fact, this is the most important limitation of the direct shear test.
2. The normal stress is applied by means of lever system & hence there is uneven distribution of loads.
3. There is virtually no control of the drainage of the soil specimen as the water content of saturated soil changes rapidly with stress

- The ridges of the metal gratings embedded on the top & bottom of the specimen causes distortion of the specimen to some degree.
-

* Triaxial compression test :- ($\sigma_3+\sigma_d=\sigma_1$)



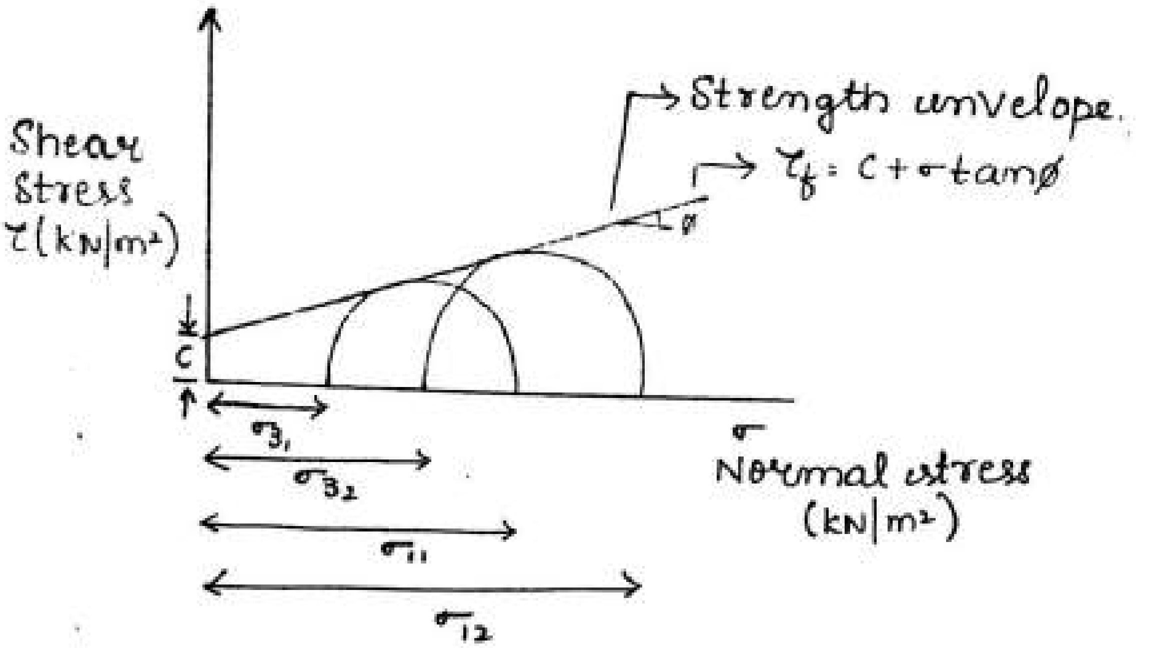


This test is carried out on a cylindrical specimen of soil. having a diameter of 38mm & length 76mm.

* **Procedure:-**

- The cylindrical specimen covered with rubber membrane is placed on a saturated porous disc resting on triaxial cell. The membrane is used to prevent the pressurized cell fluid, which is usually water from penetrating into the soil specimen.
- The triaxial cell is filled with water at the required pressure, thus the soil specimen is subjected to all round pressure. This is called cell pressure or the confining pressure (σ_3). It acts radially on the vertical surface of the specimen & axially at the top & bottom.
- In the first stage, the specimen is subjected to an all round confining pressure σ_3. on the sides, top & bottom.
- In the Second stage, additional axial stress is applied until the soil sample fails. This additional axial stress is called deviator stress, is applied on the top of the specimen.
- Thus, the total stress in axial direction at the time of shearing is $(\sigma_3 + \sigma_d) = \sigma_1$, which represents the major principal stress.

- To obtain shear parameters, Mohr's circle (atleast two number) for different cell or confining pressure are drawn. Common tangent for these circle represents the failure envelope from which the shear parameters c & ϕ can be determined.



Alternatively, the shear parameter can be determined analytically from the following relationship. ie

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$$

$$\alpha = 45 + \frac{\phi}{2}$$

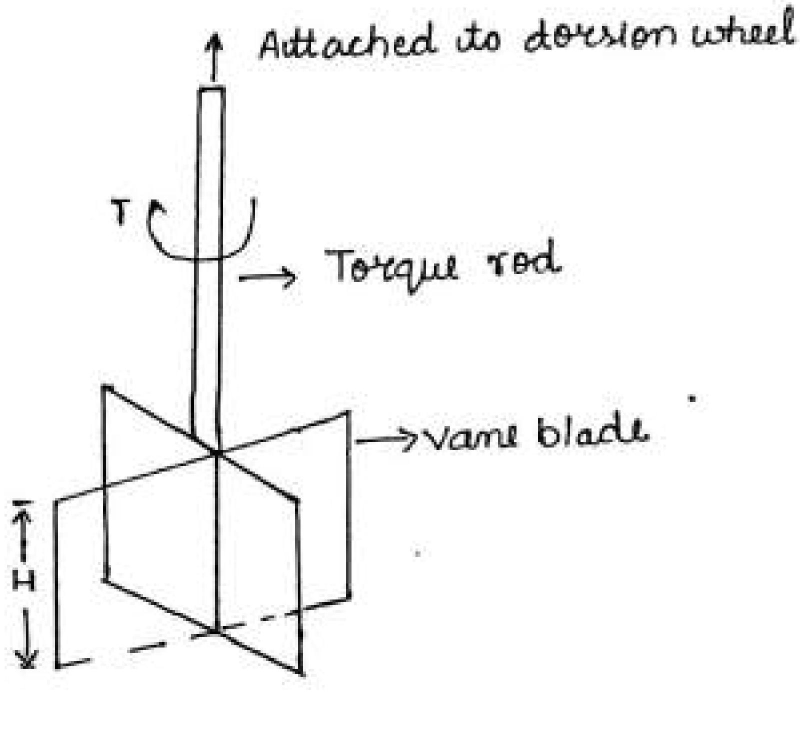
here α = inclination of failure plane w.r.t horizontal.

When two specimens are tested with different lateral pressure, σ_3 & σ_1 will be known for two specimen. Solving these equathe shear parameter c & ϕ can be obtained.

Advantages of Triaxial test

- The shear test under all the three drainage conditions can be performed with complete control.
- Precise measurement of the pore pressure & Volume change during the test are possible.
- The stress distribution on failure plane is uniform.
- The State of stress within the specimen during any stages of the test, as well as at failure is completely determinate.

* Vane shear test :-



- This is a quick test to determine the shear strength of cohesive soil.
- The shear vane consist of a pair of thin steel blades connected to vertical shaft as shown in fig. Steel blades are called Vanes.
- The torque measuring device is attached to the shaft. The Vane is pushed gently into the soil upto required depth & torque is applied on the top of the shaft.
- Rotation of Vanes inside the soil sample means shearing off a cylindrical piece of sample.
- The torque is measured by noting the angle of twist.
- The Shear strength is then calculated by the following equa

$$T = \tau_f \pi d^2 \left(\frac{d}{6} + \frac{h}{2}\right)$$

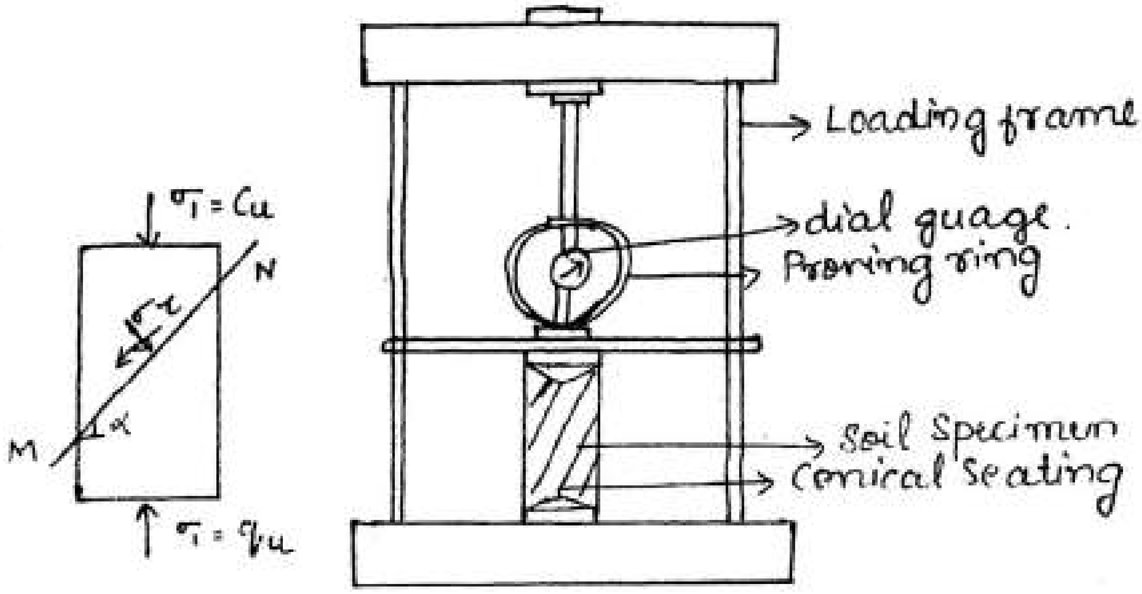
where T = torque applied in N-m

τ_f = Shear stress at failure

d = diameter of the sheared cylinder (m)

h = height of Vanes in (m)

* Unconfined compressive Test (U.C.C) Test



This test is a special case of triaxial test in which cell pressure (σ_3) is zero.
This test is limited to clayey soil & silty soil only.

- The apparatus consist of a small loading frame with proving ring to measure the vertical stress (σ_1). also Known as unconfined compressive strength (q_u) applied to the soil.

- A cylindrical soil specimen of dia 38mm & length 76mm is subjected to gradually increasing axial stress until it fails.
Since the test is quick, water is not allowed to drain out of the sample. The ends of the specimen are allowed in the form of cone.

The general equ^n for σ_1 is given by

$$\boxed{\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha}$$

where α = inclination of failure plane w.r.t horizontal.

$$\alpha = 45 + \frac{\phi}{2}$$

In unconfined compression test $\sigma_3 = 0$ & hence c is replaced by c_u & ϕ by ϕ_u

$$\sigma_1 = q_u = 2c_u \tan\left(45 + \frac{\phi_u}{2}\right) \quad \text{---} \quad (1)$$

The above equ^n contains two unknown namely c_u & ϕ_u. Since any no of tests on identical specimens gives only one value σ_1. Shear parameter can't be determined from the above equ^n. Hence of direction of failure plane ($\alpha = 45 + \phi/2$) is to be measured from the failed specimen & then the other parameters.

Due to difficulty of measurement of inclination of failure plane, unconfined compression test is generally applicable for clays for which $\phi_u = 0$.

Equ^n 1 reduces to

$$q_u = 2c_u$$

$$\therefore c_u = \frac{q_u}{2} = \frac{\sigma_1}{2}$$

Determination of shear strength:-

The shear stress on the failure plane representing shear strength of the soil is given by the following general equa

$$\tau_f = S = \left(\frac{\sigma_1 - \sigma_3}{2}\right) \sin 2\alpha$$

In unconfined compression test on clays

$$\alpha = 45 \;\&\; \sigma_3 = 0$$

$$\tau_f = S = \frac{\sigma_1}{2} = C_u$$

The area of c/s at failure (A_c) can be obtained from the following equa

$$A_c = \frac{A_0}{1-e}$$

where A_c = corrected c/s area

A_0 = original c/s area

e = Strain = $\frac{\Delta L}{L}$ = $\frac{\text{change in length}}{\text{original length}}$