RRB JE

RAILWAY RECRUITMENT BOARD

CIVIL ENGINEERING

GEOTECHNICAL ENGINEERING

SELF STUDY MATERIAL

CIVIL ENGINEERING FOR ALL

GEOTECHNICAL ENGINEERING

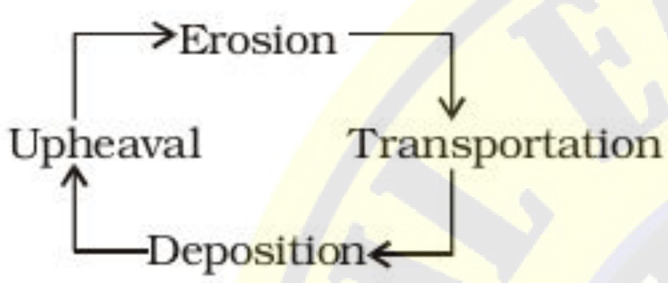
Soil is the unaggregated or uncemented deposits of mineral and/or organic particles or fragments covering large portion of the earth's crust.

Do you know?

Terzaghi is father of soil mechanics.

GEOLOGICAL CYCLE : The process of formation of soil is termed as pedogenesis. Soil is generally formed due to the erosion of rocks which may be carried out either physically or chemically.

The geological cycle of soil formation consists of four steps.



Physical agencies involved in erosion of rocks are flowing water, wind & ice. e.g. Gravel and sand.

Chemical Agencies involved in the erosion are oxidation, carbonation & reduction. e.g. Silts and clays.

If the eroded soil material remains over the parent rock, it is termed as residual soil & if the eroded soil material is transported (by any of the above mentioned agencies), the soil is termed as transported soil.

Due to physical weathering of the rocks, mineral constituent of the soil does not differ from the parent rock & due to chemical weathering mineral, constituent differs from the parent rock.

According to the Transporting Agency Transported soils are further classified as :

1. **ALLUVIAL SOIL :**
   It is the type of soil that is deposited from suspension in flowing water. It is a physical agency. generally found along the banks of the rivers. It is normally deposited at the bed of the river.
2. **LACUSTRINE SOIL (Physical Agency) :**
   It is the type of soil, that is deposited from suspension in still & fresh water of the lakes.
3. **MARINE SOIL (Physical Agency) :**
   It is the type of soil formed from suspension in marine water, It is formed at the bed of sea.
4. **AEOLINE SOIL (Physical Agency) :**
   It is the type of soil transported by wind.
5. **GLACIAL SOIL :** Here (Physical weathering is not that prominent as flow is not continuous so, graded. It is the type of soil which is transported by ice.

Some Special/Typical Soil :-

1. **LOESS SOIL :**
   It is uniformly graded silt that is transported by wind & is slightly cemented due to the presence of calcium carbonate & clay mineral like Montmorelonite. It is found in deserts. and is uniformly (Poorly) graded soil.
2. **MARL SOIL (Organic soil) :**
   It is finely graded silty cemented soil of marine origin. It is generally formed due to the decomposition of dead cell & bones of aquatic life (fishes & plants).
3. **BENTONITE SOIL :**
   It is formed due to chemical weathering of volcanic ash which is generally used as a lubricant in drilling.
4. **LATERITE SOIL :**
   This type of soil is formed due to the leaching of washing out of silicions Matter from parent rock (i.e. Silica will not be present in this soil). It is found in hilly areas, (Western Ghats, Eastern Ghats etc.) having humid condition.
5. **COLLOVIAL SOIL (Tallus) :**
   It is also known as Tallus soil. It is formed due to transportation by gravitational force in hilly areas generally in the valleys.
6. **GUMBO SOIL :**
   It is sticky, highly plastic dark coloured soil.
7. **PEAT SOIL :**
   It is highly organic almost consisting entirely of vegetative matters in different stages of decomposition.
   Its colour varies from black to dark brown & possess organic odour.
   It is highly fibrous & compressive in nature.
8. **MUCK SOIL :**
   It is the mixture of fine particulated inorganic soil and dark brown or black decomposed organic matter.
   It is generally formed, where indeficient sewerage facilities are found or due to the over flooding of the river.

   Note : Peat & Muck soil when taken together are termed as Cumulose soil.

   Some Common soils in which Engineering Problems Encountered with them are :

   1. **Marine deposits :** Marine deposits are very soft and may contain organic matter.
      These possess low shear strength and high compressibility hence, posses problems as foundation material.

2. **Black Cotton soils :** These soils have been formed from basalt or trap and contain clay mineral montmorillonite, which is responsible for the excessive swelling and shrinkage characteristics of the soil.

3. **Desert soils :** These are wind blown deposits of sand. Dune sand is non-plastic uniformly graded fine sand.

   Problems associated with these soils are of soil stablization for roads and runways, reducing settlement under static and dynamic loads and reducing its perviousness to make it suitable for storage and transportation.

Do you know?

Under-reamed piles should be used in foundations in black cotton soils.

2. PROPERTIES OF SOIL :

Soil mass is generally of 3 phase system i.e. solid, water & air. They do not occupy separate spaces but are blended together to form a complex mixture.

(a) Three phase System (Applicable for partially saturated soil)

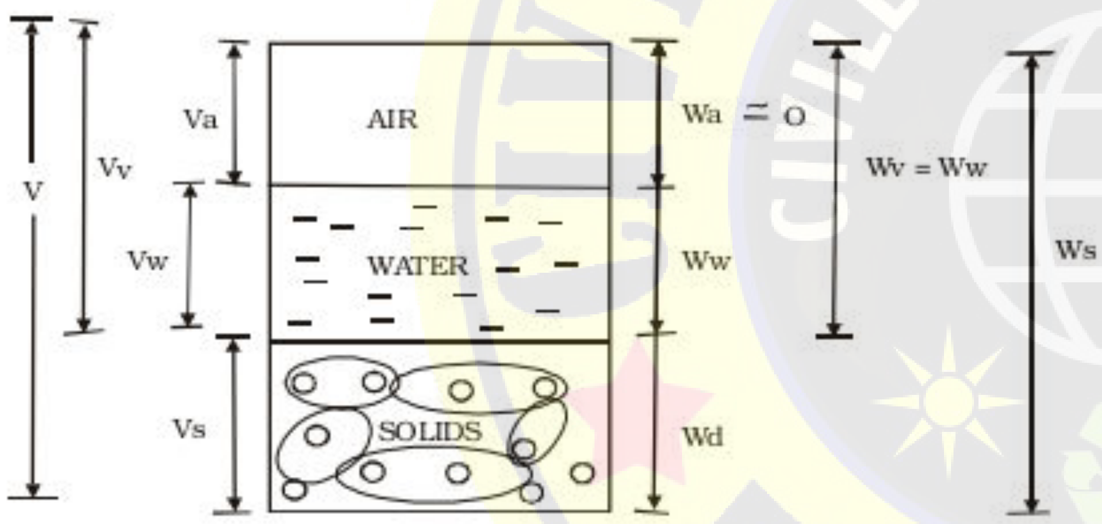
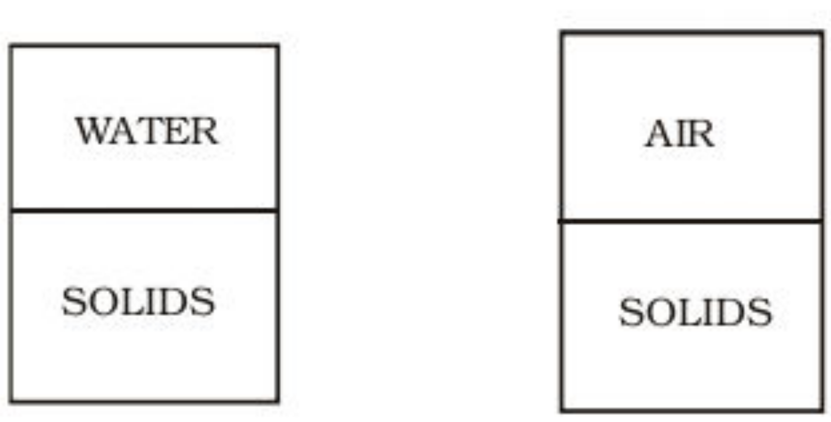


Fig 1- BLOCK DIAGRAM OF SOIL

V =Volume; W = Weight

(b) Two phase system (Applicable for fully saturated soil & Dry Soil).

The properties of the soil are dependent upon the relative proportion of solids, water & air present in it



(Solids + water) **fully saturated soil**

(Solids + Air) **Dry soil**

3. IMPORTANT PROPERTIES OF SOIL

1. Water Content : (Moisture Content or % moisture)

It is defined as the ratio of weight of water present in the soil to the weight of solids present in it.

$$W = \frac{\text{weight of water present in soil}}{\text{weight of solids present in soil}} \times 100\%$$

$$W = \frac{W_w}{W_d} \times 100$$

Water content of fine grained soil is generally more than coarse grained soil.

It is greater than or equal to zero.

i.e W 0%

2. Void Ratio (e) : It is defined as the ratio of volume of voids to the volume of solids present in soil.

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

$$e > 0$$

Individual size of voids is more in coarse grained soil than in fine grained soil but the voids ratio of fine grained soil is comparatively more than coarse grained soil.

3. Porosity (*n*) : It is also termed as percentage voids as it represents the water storage capacity of soil.

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

$$n = \frac{V_v}{V} (\%\text{voids})$$

$$0 < n < 1$$

$$0 < n < 100\%$$

Note : Significance of voids ratio & porosity is same as both represents volume of voids either in terms of volume of solids or total volume of soil, but relevance of void ratio is comparatively more then porosity as it represents volume voids in terms of stable parameter (volume of solid).

4. Degree of saturation {% saturation} (S_r) :

It is defined as the ratio of volume of water present in the soil to volume of voids presents in it.

$$S_r = \frac{\text{Vol. of water}}{\text{Vol. of voids}} \times 100$$

$$\Rightarrow S_r = \frac{V_w}{V_v} \times 100$$

0 S_r 1 ; 0 S_r 100%

On basis of degree of saturation, the soil may be classified as ; dry, humid, damp, moist, wet & saturated.

5. Air content :

It is defined as the ratio of volume of air present in the soil to the volume of voids present in it.

$$a_c = \frac{\text{Vol. of air}}{\text{Vol. of void}} \Rightarrow a_c = \frac{V_a}{V_v}$$

6. Percentage Air Voids (η_a) :

It is defined as the ratio of volume of air present in the soil to the total volume of soil.

$$\eta_a = \frac{\text{vol. of air}}{\text{vol. of soil}} ; \eta_a = \frac{V_a}{V} ; \boxed{a \quad a_c.}$$

7. Unit weight & Density :

(a) Bulk Unit weighted or Bulk density :

It is defined as the ratio of weight of soil in existing condition to the volume of the soil.

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

$$V = V_s + V_w + V_a$$

(b) Dry Unit weight or Dry density :

It is defined as the ratio of dry weight of soil to the volume of soil.

$$\gamma_d = \frac{W_d}{V} ; \quad _d = \frac{M_d}{V}$$

Note : Dry unit weight is taken as index of denseness of the soil. For the given volume of soil of dry unit weight is more, solids present in the soil are compactly packed.

(c) Saturated unit weight or Saturated density :

It is defined as ratio of saturated weight of soil to the volume of soil.

$$\gamma_{sat} = \frac{W_{sat}}{V} ; \rho_{sat} = \frac{M_{sat}}{V}$$

Note : Under dry condition, dry unit weight of the soil = Bulk unit weight & under saturated condition saturated unit of soil = bulk unit weight (for density also).

Saturated Unit weight ≥ Bulk unit weight ≥ Dry unit weight

(d) Submerged unit weight or submerged density :

When the soil is submerged below GWT (Ground water table), solid are being acted upon by force of bouyancy which is equal in magnitude to the weight of water displaced by the solids.

This force of bouyancy results a decrease in weight of the solids & the effecting weight is termed as submerged unit weight or Bouyant unit weight.

$$\gamma_{sub} = \gamma' = \frac{W_{sat} - F_B}{V}$$

$$= \frac{W_{sat}}{V} - \frac{V_W \quad w}{V}$$

Where, F_B = Bouyancy force

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

$[V_w \simeq V]$ $[V_s <<< V_w]$

$\gamma_{sat} 2 \simeq \gamma_{sat}$; hence; $\gamma'_{sat} \simeq \frac{1}{2}\gamma'_{sat}$

[When exact information is not known.]

(e) Unit weight of solid or density of solids :

It is defined as the ratio of mass of solids to the volume of solids in the given soil sample.

$$\gamma_s = \frac{W_d}{Vs} ; \rho_s = \frac{M_d}{Vs}$$

value of $\gamma_s > \gamma_d$; also ; γ_s is a stable parameter as γ_s is stable parameter

$\therefore \gamma_s > \gamma_{sat} > \gamma_{Bulk} > \gamma_{dry} > \gamma_{submerged}$

Unit weight of solids is maximum at different unit weight considered.

Relevance of unit weight of solids is comparatively more than dry unit weight as it represents weight of solids in terms of a stable parameter.

i.e. Volume of solids.

8. Density Index/ Degree of Density OR Relative Density :

Also known as Relative Density, express the relative compactness of the natural **cohensionless soil.**

It is defined as the void ratio of difference of the soil in its loosest state and void ratio of soil in its natural state to the difference of void ratio of the soil in its loosest state and its void ratio in the densest state.

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

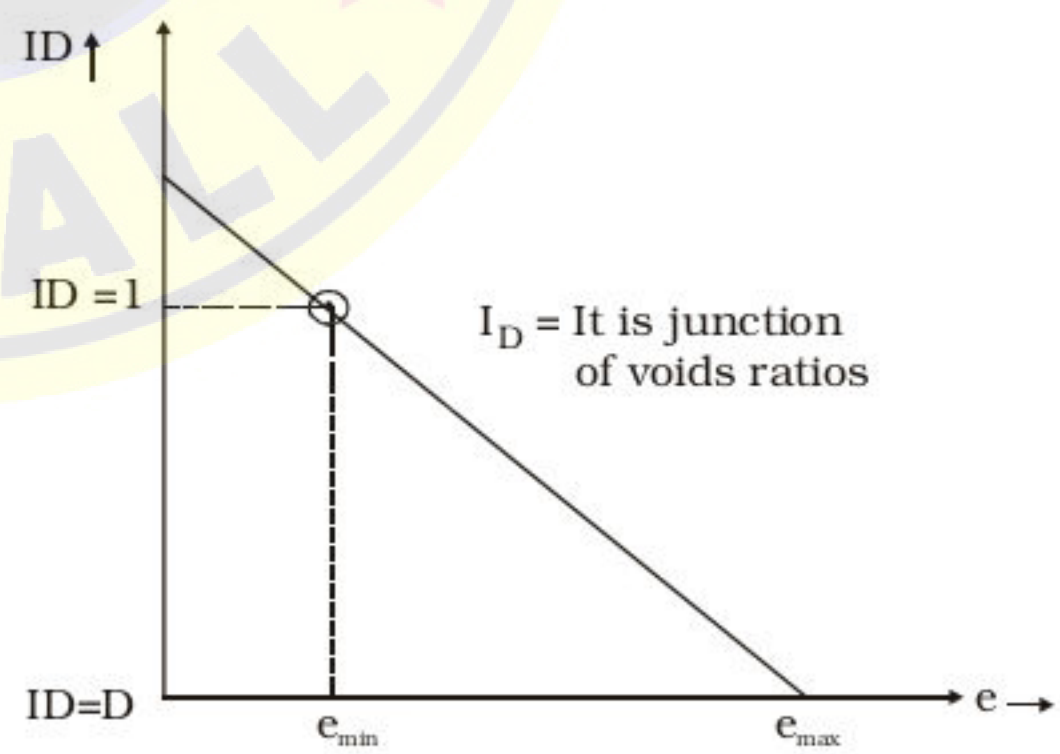
e_{max} = Void ratio of soil in its loosest state

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

e = Void ratio of soil in natural state

$e = e_{max} \Rightarrow I_D = 0$ [loosest State]

$e = e_{min} \Rightarrow I_D = 100\%$ or 1 [densest state]



Density Index is a better parameter than voids ratio to indicate the denseness of the soil.

9. Relative Compaction (RC) :

It represents the degree of denseness of both cohesive & cohensionless soil.

It is defined as the ratio of dry unit weight of the soil in its natural state to the maximum dry unit weight of soil (dry unit weight of soil in its densest state).

$$R_c = \frac{d}{d(\max)} = \frac{1 \quad e_{min}}{1 \quad e}$$

Dry density is inversely proportional to void ratio;

$$\therefore \ I_D = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{\frac{1}{d\min} - \frac{1}{d}}{\frac{1}{d\min} - \frac{1}{d\max}}$$

Imperically;

$R_c = 80 + 0.2ID.$

$\therefore$ If $I_D = 100\%$; $I_D = 0.$

$R_c = 100\%$ $\therefore R_c = 80\%$

$\Rightarrow 80\ \% \le Rc \le 100\%$

- **Description of soil on the basis of density Index is as follows :**

I_D	Discription of soil
0-15	Very loose soil
15-30	Loose soil
30-65	Moderate soil
65-85	Dense soil
85-100	Very dense soil

10. Specific Gravity :

(a) True Specific Gravity (G_s) :

$$G_s = \frac{\text{Unit weight of solid of given volume}}{\text{Unit weight of standard fluid}}$$

(water at 4°C of same volume)

$$\frac{W_d}{W_w} \quad \frac{W_d}{V_s} \cdot \frac{V_s}{W_w} \quad \frac{W_d}{V_s} \cdot \frac{V_w}{W_w} \quad \frac{s}{w}$$

The lower value of specific gravity is for fine grain soil. The higher value of specific gravity is for coarse grain soil. With increase in organic content, specific gravity decreases. With increase in mineral content, like iron & mica, specific gravity of soil decreases. In India, specific gravity is generally reported at 27°C. Specific gravity is to be computed at any other temperature. Then, corresponding change in density of water should also be considered. It is also known as absolute specific gravity.

$$G_{27°C} \cdot \gamma_{w27°C} = G_{T°C} \cdot \gamma_{wT°S}$$

(b) Mass Specific Gravity (G_m) OR Apparent Specific gravity :

(Bulk Specific Gravity)

Defined as;

$$G_m = \frac{\text{Unit weight of soil of a given volume}}{\text{Unit weight of water of same volume}}$$

$$G_m = \frac{\quad}{w}$$

True specific gravity is more than mass specific Gravity as $\gamma_s > \gamma$.

Generally G_s is used but not G_m because γ is relatively stable as compared to γ.

Some Important Relationships

1. $W_s \ \frac{W}{1 \quad W}$ or $r_d = \frac{r}{1 \quad W}$
2. $n = \frac{e}{1 \quad e}$ or $e = \frac{n}{1 \quad n}$
3. S.e = W.G
4. $\gamma = \frac{G.r_w(1 \quad W)}{(1 \quad e)} = \left(\frac{G \quad S.e}{1 \quad e}\right).r_w$
5. $r_{sat} = \left[\frac{G \quad e}{1 \quad e}\right].r_w$
6. $r_d = \frac{G.r_w}{1 \quad e}$
7. $\gamma' = \left(\frac{G \quad 1}{1 \quad e}\right).r_w$
8. $r_w = \frac{(1 \quad a)G.r_w}{1 \quad W.G}$

Note : For uniformly graded soil in which soil grains are assumed to be spherical,

→ Maximum possible void ratio is ≈ 91% & corresponding.

→ Maximum possible void porosity is ≈ 47%

→ Minimum possible void ratio is ≈ 35% & corresponding.

→ Minimum possible void porosity is ≈ 26%

- **INDEX PROPERTIES OF SOIL**

These are the properties of the soil which helps in classification & identification of the soil.

Example : (i) Water Content (ii) Unit weight of solids (iii) Specific Gravity (iv) Particle size distribution, (v) Consistency (vi) Activity (vii) Thixotropy (viii) Sensitivity (ix) Collapsibility.

(i) Water Content

It can be determined using following methods :

(a) Oven dry Method :

→ It is one of the most accurate method generally used in Lab to find the water content of the soil deposits.

→ For inorganic soils, temperature is maintained between 105°C-110°C for sufficient duration.

→ For sand, drying is required for 4 hrs; For clay & silt, it is required for 12-16 hrs.

But in general, drying is done for 24 hrs to ensure complete removal of the moisture.

→ If organic content of the soil is comparatively more, temperature is not increased beyond, 60°C in order to prevent (avoid) oxidation of organic matter.

→ If gypsum is also present in the soil, temperature is not increased beyond 80°C in order to avoid the loss of water of crystallisation.

→ Water content is calculated as

Case 1: $\therefore W = \dfrac{W_2 - W_3}{W_3 - W_1}$

Where,

W_1 = weight of container
$W_2 = W_1$ + weight of moist soil
$W_3 = W_1$ + weight of dry Soil

(b) Sand bath Method :

It is a quick field method, which is generally adopted when facility of oven is not available.

In this method, container with soil is placed over the sand bath which is further heated on a stove.

Since, there is no control over the temperature in this case, this method is not used for organic solid soils having gypsum.

Water content is determined using same as that of oven dry method.

(3) Alcohol method :

It is also quick field method which gives the approximate value of water content.

In this method, the sample is mixed with methylated spirit (alcohol) to increase the rate of evaporation.

In this method also, there is no control over the temperature hence is generally avoided for organic soil & soils having gypsum.

(4) Calcium carbide method (Rapid moisture method) :

It is a very quick method which gives the water content within 5-7minutes (field method).

It is generally used to final approximate moisture content present in the soil.

The reaction Involved is

$CaC_2 + H_2O \rightarrow C_2H_2 \uparrow + Ca(OH)_2$

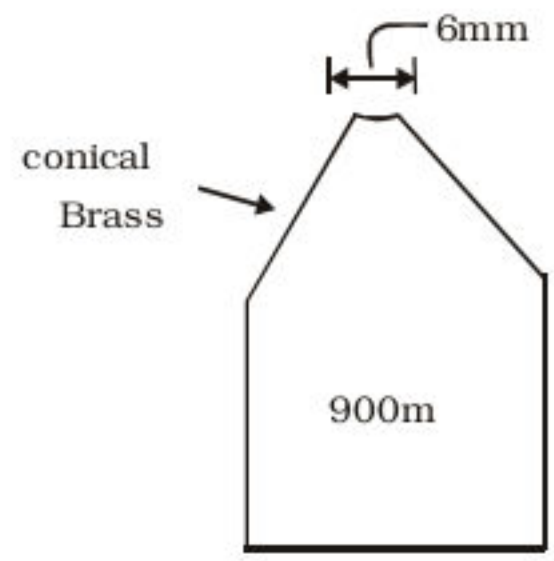
$W' \Rightarrow W = \dfrac{W'}{1 - W'}$

W = water content corresponding to weight of solid.

(5) Pycnometer Method :

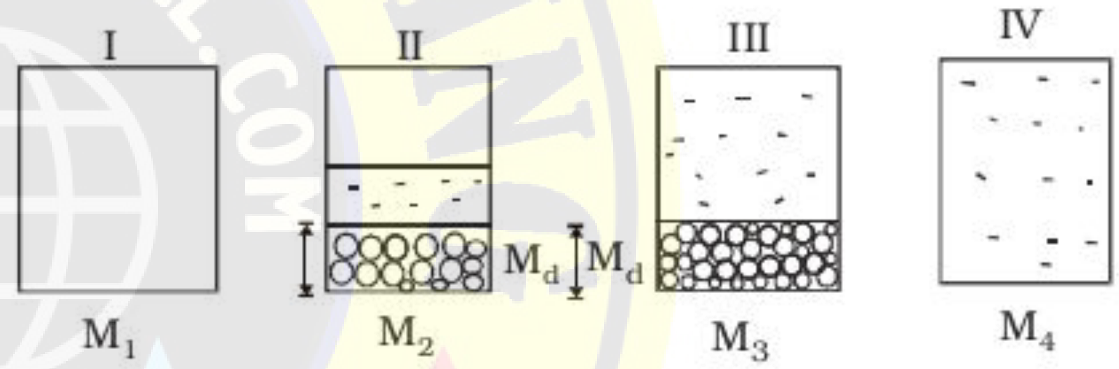
It is also a quick method, used for determination of moisture content which gives result within 10 to 20 min.

It is utilized for soil only whose specific gravity known. Pycnometer is a 900ml density bottle having conical brass top with 6 mm diameter circular hole in it.



Steps involved in the measurement of pyconometer method are as follows :

1. Empty weight of the pyconometer is being measured initially (M_1, W_1).
2. Moist sample of soil is placed in the pycnometer & its weight (mass) is again being measured (M_2, W_2).
3. The left volume of the pycnometer is being filled by the water in multiple stage & removing the entrapped air present in simultaneous by (M_3, W_3).
4. The pycnometer is completely emptied & after cleaning, it is again being filled with the water (M_4, W_4).



$(M_3 > M_4)$ as specific gravity of soil > specific gravity of water.

Mass of solids in III = M_d

Volume of solids in III = $\dfrac{M_d}{G.\,_w}$

Volume of equivalent water in IV = $\dfrac{M_d}{G\,_w}$

Mass of equivalent water in IV = $\dfrac{M_d}{G.\,_w} \cdot \rho_w = \dfrac{M_d}{G_W}$

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

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

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

$$W = \frac{M_w}{M_d} = \frac{M_2 - M_1 - M_d}{M_d} = \left[\left(\frac{M_2 - M_1}{M_3 - M_4}\right)\frac{G}{G-1} - 1\right]$$

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

Do you know?

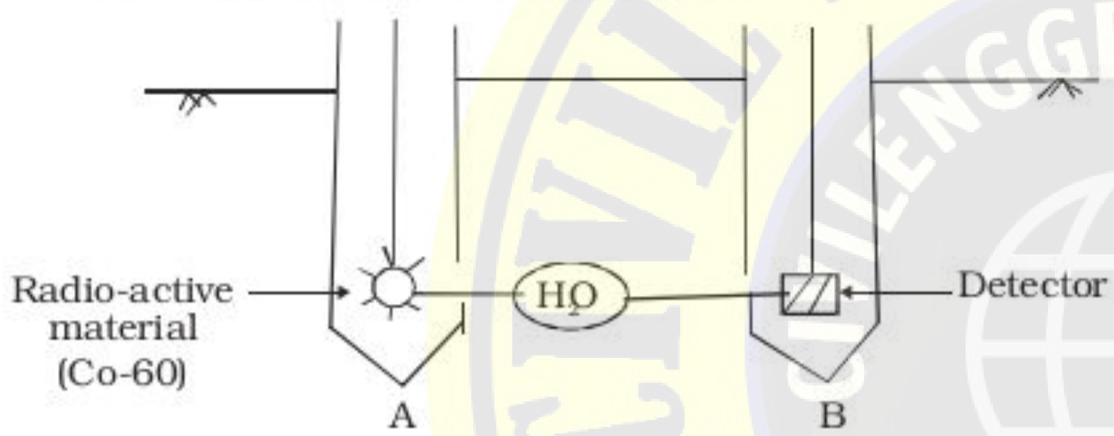
Pyconometer method is suitable for coarse grained soil but if it is used for fine grained soil than instead of water, Kerosene should be used because kerosene has good wetting properties.

(6) Radiation Method :

It is the field method in which two steel casings are boarded at the location where water content is to be found.

Radioactive material is being lowered in one of the steel casing (cobalt 60) & detector in the 2nd casing.

When the Radioactive material is being activated it emitts neutron which stricks with the molecules of the water present in the medium resulting in loss of energy of neutrons that is being detected by the detector & is being calibrated to give moisture content of the soil at that location.



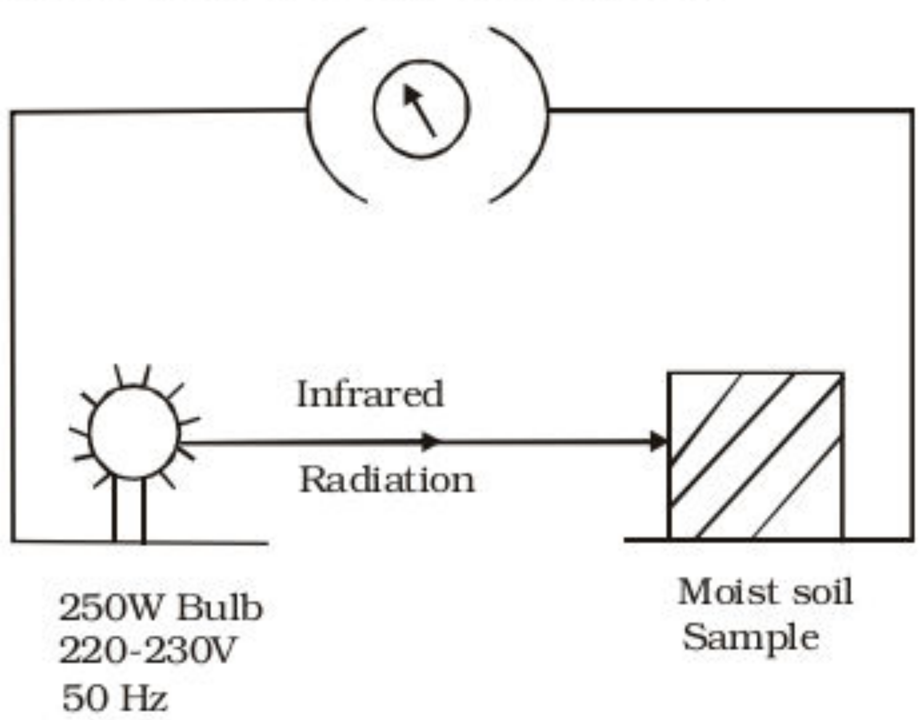
The neutron collides with hydrogen atom of water that will lead to loss of energy of neutron.

(7) Torsional Balance Method :

In this method, Infra Red Radiations are used for drying the moist soil sample.

Infra Red Radiations are generated by the use of 250 watts bulb. Operating at 220 to 230 V (50Hz) single phase power supply.

In this method drying & weighing of the sample is being carried out simultaneously Hence, it is generally suitable for those soils which quickly regain the moisture from the atmosphere.



- Costly
- Torque is calibrated to given moisture content.

Any ways oven drying is the most accurate.

Oven drying is the best method as in the process both drying and weighing is done simultaneously.

(i) Specific gravity :

It can be determined using 50ml density bottle and 500ml flask or pycnometer 900ml.

Density bottle is used for testing all types of soil whereas as flask and pyconometer are generally used for cohesionless soil.

If cohesive soil is to be tested using pyconometer (kerosene) non polarising agent is used instead of water polarising agent as it has better wetting properties.

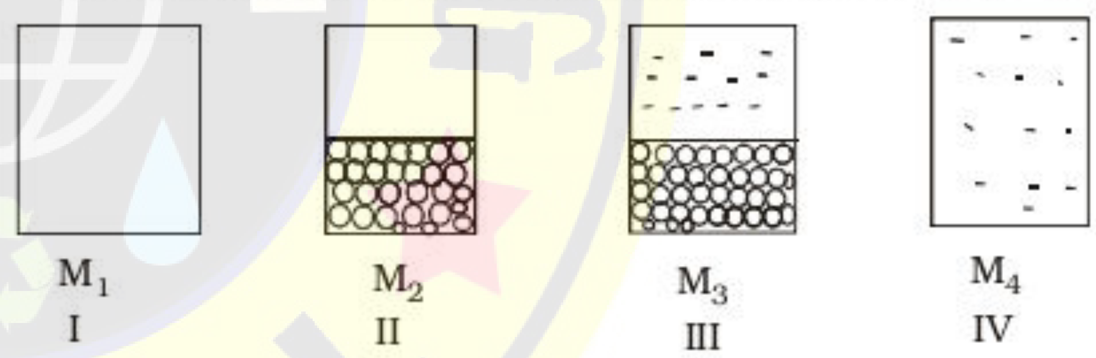
Steps of observations involved in density bottle, flask or pycnometer are same.

Step 1 : Empty weight/mass of the pycnometer is being noted initially.

Step 2 : Oven dry sample of soil is placed in pycnometer & is again weighed.

Step 3 : Empty volume of the pycnometer is filled with water/ kerosene in multiple stage along with the removal of air present in the voids of the sample by constant stirring or by the use of volume. pycnometer is again being weighted (M_3/W_3).

Step 4 : Pycnometer is completely emptied & is filled with water after its proper cleaning (M_4/W_4).



We know :

$G = \frac{M_d}{M_w}$; Mass of solid in stage II ; $M_d = M_2 - M_1$

Mass of water in stage III ; $M_3 - M_2$

Mass of water in stage IV ; $M_4 - M_1$

Mass of water having same volume in IV – Mass of water IV same as that of solids.

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

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

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

Also $M_4 = M_3 - Md + \frac{M_d}{G_w}\ \rho_w$ (mass of water of same volume as that of solid).

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

$$\therefore\ G = \frac{M_d}{M_4 - M_3 \quad M_d}$$

If kerosene is used instead of water, then

$$G = \frac{M_d}{M_4 - M_3 \quad M_d}.G_k$$

Note : If density of kerosene is not mentioned, it is taken as $\simeq 0.908$ gm/cm^3.

(ii) Unit weight of Soil :

Following method to find unit weight of soil :

(a) Core Cutter method :

Core cutter comprises of cylindrical vessel of specific volume (1000 ml) [having 10 cm diameter & 13 cm height] open at top & bottom.

This steel cutter is driven into the ground upto an extent. such that Dolly is produced 1 cm above the ground.

The cutter is taken out of the ground & mass of soil retained in it is noted.

Depth of penetration is 14.5 cm

$$\rho = \frac{M}{V}\ ;\ \gamma = \frac{W}{V}$$

This method is used in case of non-cohesive soil (or sand etc.). It can not be used in case of hard and gravelly soils.

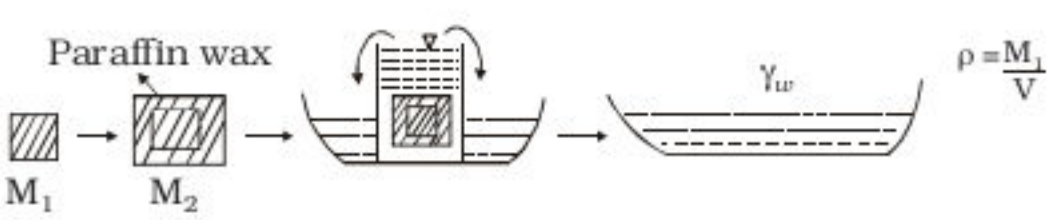
(b) Water displacement method :

This method is generally used for highly cohesive soils which are highly sticky in nature.

In this method, the soil sample is trimmed off in more or less uniform shape & is being weighed (M_1).

The sample is then coated with layer of paraffin wax & is again being (M_2).

The coated specimen is immersed in the container which is completely filled with water & the volume of the water displaced by specimen is noted.



Volume of water displacement V_w = Volume of weighted specimen

V_w = Volume of soil sample + Volume of paraffin wax

$V_w = V + V_p$

Mass of parafin wax = $M_2 - M_1$

$$\text{Volume of parafin wax } V_p = \frac{M_2 - M_1}{wax}$$

$$V = V_w - V_p = V_w - \frac{M_2 - M_1}{wax}$$

$$V = V_w - \frac{M_2 - M_1}{wax}$$

$$\rho = \frac{M_1}{V}$$

→ Used in case of hard and gravel Soil.

→ A hole in ground is made. The excavated soil is weighed. The volume of hole is determined by replacing it with sand. In-situ weight is obtained by dividing weight of excavated soil with volume of hole.

Particle size Analysis OR Grain Size distribution :

Percentage of different sizes of particle present in given dry sample of soil is being determined by particle size analysis or mechanical analysis.

It is carried out in two stages :

(a) Sieve analysis : It is also known as dry mechanical analysis.

It is not dependent on temperature.

It is for coarse grained particles.

(b) Sedimentation analysis : It is also known as wet mechanical analysis.

It is dependent on temperature and viscosity.

It is for fine grained particles.

In general, soil consists of both fine and coarse particles, hence both stages of analysis are being performed.

But in actual sieve analysis is the true representation of particle size distribution as it is not dependent upon the temperature.

(a) SIEVE ANALYSIS :

1. It is being performed for the soil fraction that are retained over 75 micron or 0.075mm sieve.

   As per IS 460-1962, sieves size are being designated by the size of the apperture in mm.

2. It is of two types ;

   - Coarse sieving (Gravel) 4.75 mm < d < 80 mm
   - Fine sieving (Sand) 0.075 mm < d < 4.75 mm
     - Dry
     - Wet (Done only if clay & silt is present)

3. In sieve analysis, different sieves are arranged one over the other in vertical frame with the sieve having maximum size of apperture at the top and minimum size at the bottom.

4. The sample to be tested is placed over the top most sieve and sieving is done for at least 10 minutes either manually or in sieve shakers.

5. Percentage of soil retained over each sieve is noted to compute the percentage finer corresponding to the given size of sieve.
6. Coarse sieving is done for soil of fraction retained over 4.75 mm sieve.
7. The sieves used in this case are ;
   80mm, 40mm, 20mm, 10mm, 4.75mm.
8. Fine sieving is being done for the soil fraction passing through 4.75 mm sieve but are retained over 0.075 mm sieve or 75 μ.
9. It is generally advised to wash the soil fraction passing through 4.75 mm sieve in order to dislodge clay and silt particles present over sand (then, its a wet sieving).
10. Sieves used in this case are :-
    2mm, 1mm, 600 μ, 425 μ, 300 μ, 212 μ, 150 μ, 75 μ.
11. The result of sieve analysis is being used to analyse the variation between percent finer & corresponding size of particle.

Concept of "Percentage finer"

% retained on a Particular sieve

$$= \frac{\text{Weight of soil retained on sieve}}{\text{Total weight of soil taken}}$$

Cumulative % retained = Sum of % retained on all sieves of larger size and % retained on particular sieve.

"Percentage finer" than the sieve under reference = 100% – cumulative retained.

(b) Sedimentation Analysis :

1. It is done for the soil fraction that pass through 0.075 mm sieve.
2. The entire sedimentation analysis is based upon Stoke's law, which may either be done by pippette method or by hydrometer method.

If a single spherical grain is allowed to fall freely through a liquid of infinite extent, its vertical velocity first increases rapidly under the action of gravity, but a constant velocity called "Terminal velocity" is reached out with in a short time. According to Stokes law, the terminal velocity is given by :

$$V = \frac{g}{18}\,\frac{\rho_s - \rho_w}{\eta}\,.\,D^2$$

Where,

ρ_s = Density of grains (g/cm^3)

ρ_w = Density of water (g/cm^3)

η = Viscosity of water

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

D = Diameter of grain (cm)

By putting the values at 20°C, we get,

$$V = 91D^2 \text{ – at } 20°C$$

where, V is in cm/s and D is in mm.

$\upsilon = 107D^2$ at 27°C.

LIMITATIONS OF STOKE'S LAW :

1. The soil particle is assumed to be spherical but generally fine grained particle (clay and silt) are flaky in nature.
2. Each particle is assumed to be setting in the medium as discrete particles but in actual, setting of the particle is hindered due to the setting of neighbouring particles.
3. In most of the cases, these particles settles together due to the formation of flocs.
4. The medium is assumed to be infinite, in order to neglect the resistance of the boundary over the settlement of the particles.
5. Stoke's law in this case is generally applicable for the particles, having the size range in between 0.2 mm and 0.2 μ.

Note : (1) If the size of the particle is > 0.2 mm flow no longer remains laminar and due to turbulent motion, particle assumes gravity acceleration during it entire. settlement and hence does not attains constant velocity.

(2) If the size of the particle is < 0.2μ Brownian motion takes place during the settlement. This does not allow particles to settle in the medium.

◆ **PREPARATION OF SOIL SUSPENSION :**

In order to carry out the removal of organic matter and calcium compounds present in the soil sample **pre-treatment** is being carried out.

For removal of organic matter oxidizing agent i.e. hydrogen peroxide (H_2O_2) that leads to oxidation of organic matter & for removal of calcium compound 0.2 N HCl is added in the sample.

Both Organic Matter & Calcium compounds if not removed leads to the aggregation of soil solids during the sedimentation (flocculant settlement occurs) (passing through 0.075 mm sieve) sample of soil.

Suitable amount of oven dried sample of soil is added in water to form the soil sample.

Note : [In pippete method 12-30 gms soil is addded in water to form 500 ml (oven dried soil + water) soil suspension].

[In hydrometer method, volume of soil suspension is double and so is the mass of soil sample].

In order to avoid the flocculation of the solids during sedimentation, deflocculating (dispersing) agent like sodium silicate, sodium oxylate, Sodium hexametaphosphate is added in the water.

This process of addition of deflocculating agent is termed as **post-treatment.**

PIPETTE METHOD :

1. 10 ml of sample is being collected from the soil sample from the fixed sampling depth of 10 cm at different time interval.
2. At any given time interval (*t*) only those solids remains in the suspension that do not settle in the suspension.

3. The collected sample is further tested for dry mass of the solids present in the suspension using oven drying method.
   [Let M_d be the mass of the solid collected from the sample at any time (t)]
4. The size of the particles collected from the sample at any given time 't' is computed using Stoke's law according to which.

$$V_S = \frac{(G-1)\left(\;_w . d^2\right)}{18} = \frac{He}{t}$$

$$\therefore\ d = K.\sqrt{\frac{He}{t}}$$

5) Percentage finer corresponding to the size of the particles (d) collected from the sample at any time (t) is ;

$$\text{percent finer N} = \frac{M_d / V_p}{M_D / V}$$

Where,

V_P = Volume of pipette (10ml)
V = total volume of soil sample (500ml)
M_D = total mass of solids added initially for preparation of the sample (12-30gm)
m_d = Mass of particle size (d)

If effect of dispersing agent is also considered then;

$$\text{percent finer N} = \frac{\frac{M_d}{V_p} - \frac{M}{V}}{M_D / V}$$

Here, m = mass of dispersing agent added during post treatment.

6) AS per IS - 2720;

33 gm of Sodium Hexametaphosphate & 7 gms of Sodium Carbonate are added in 1 litre of water & 25 ml of this water is added in soil suspension during post treatment.

$$\therefore\ m = \frac{30gm \quad 7gm}{1000\,ml} \times 25\,ml$$

$$= \frac{40}{1000\,ml} \quad 25\,ml = 1\ gm$$

$\therefore\ m = 1$ gm

Hydrometer Method :

It differs from pipette method with respect to procedure of taking the observations.

In pipette method mass of particle size 'd' is computed directly by collecting the sample from suspension at different time intervals whereas in hydrometer (M_d) is computed indirectly by reading the density value from the hydrometer.

→ In pipette method, sampling depth is maintained to be constant (10cm) but in hydrometer method sampling depth is increased with increase in time, that requires calibration of the hydrometer before each test.

→ Hydrometer is a device used to measure the density of suspension (suspension may be any).

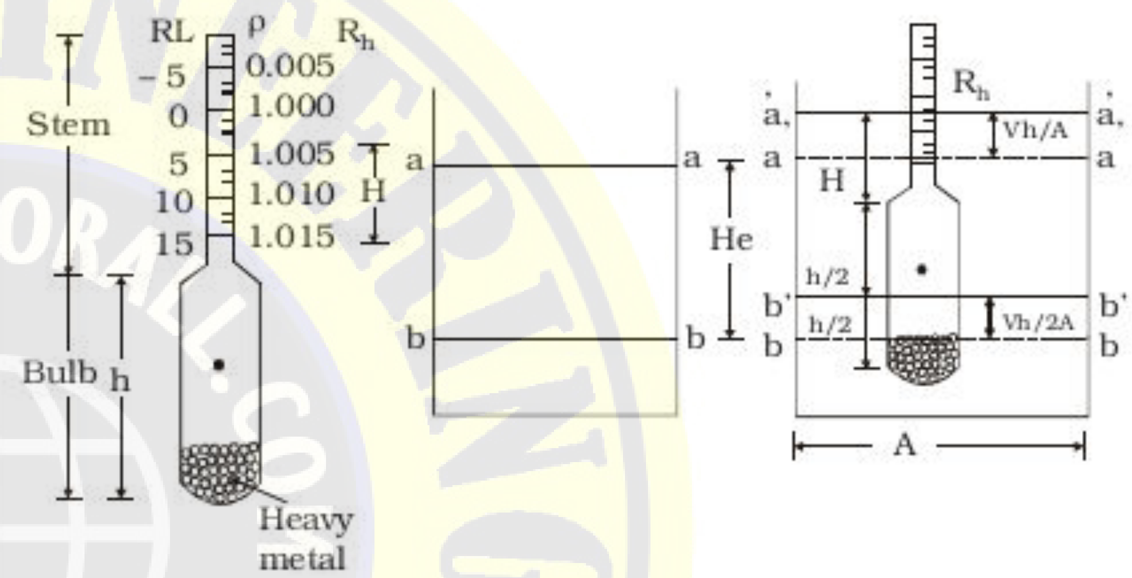
→ Volume of Hydrometer in millilitres is approximately as its mass in grams.

→ Reading on the hydrometer are being marked by substracting 1 and multiplying the remainder with 1000.

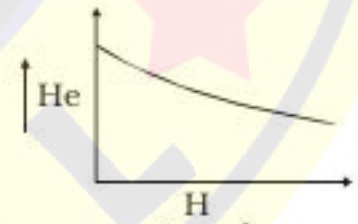
→ Reading on the hydrometer, unlike general scale increases downwards.

Calibration of the hydrometer : (He → sampling depth).

$R_n = (\rho - 1) \times 1000$ (Reduced hydrometer reading).



If He is constant, density calculated (observed) will be nearly same at that depth of particle.



Where,

V_n = Volume of hydrometer bulb
A = Area of cross section of Jar.

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

The size of particle present at any time interval (t) at sampling depth of He is computed using Stoke's law:-

$$d = k.\sqrt{\frac{He}{t}}$$

$$\therefore\ \rho = \frac{R_n}{1000} + 1 \qquad \text{...(1)}$$

Thus a reading of R_n = 25 means, G = 1.025 and a reading R_n = – 25 means, G = 0.975.

% finer is given as

$$N = \frac{G}{G-1} r_w \frac{V}{W} \cdot \frac{R_n}{10} \%$$

Where,

G = specific gravity of soil solids

R_n = final corrected value of hydrometer

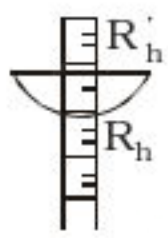
V = Total volume of soil suspension

W = weight of soil mass dissolved

CORRECTIONS TO Hydrometer reading

(1) Meniscus correction (cm) (additive).

• Due to the presence of turbidity in soil suspension, meniscus is being read at the upper level instead of lower level.



• Since reading in the hydrometer increases down the scale, observed hydrometer reading is less than actual, hence correction for meniscus is positive.

$\therefore R_h = R_h' + C_m$

(2) Temperature correction (C_T) (additive or negative)

The hydrometer is calibrated at 27°C.

If T > 27°C ; i.e. density decreases and volume increases. Therefore, level increases and reading observed is less.

Hence ; C_T is positive.

If T < 27°C ; i.e. density increases and volume decreases. Therefore level decreases and reading observed is more.

Hence , C_T is negative.

$\therefore \; R_h \quad R_h \quad C_T$

(3) Dispersing Agent correction (C_D) (negative)

Addition of dispersing agent in soil suspension apparently increases the density so the volume decreases.

Hence, dispersing correction is (–Ve)

$\therefore R_h = R_h' - C_D$

$\therefore$ From above correction factor, The composite correction factor = $C_m \pm C_T - C_D$

$\therefore R_h = R_h + C_C$

$R_h = R_h + C_m \pm C_T - C_D$

(4) Particle size distribution :

Result of particle size analysis is being referred in terms of particle size distribution curve i.e. in between percent finer & corresponding size of particle.

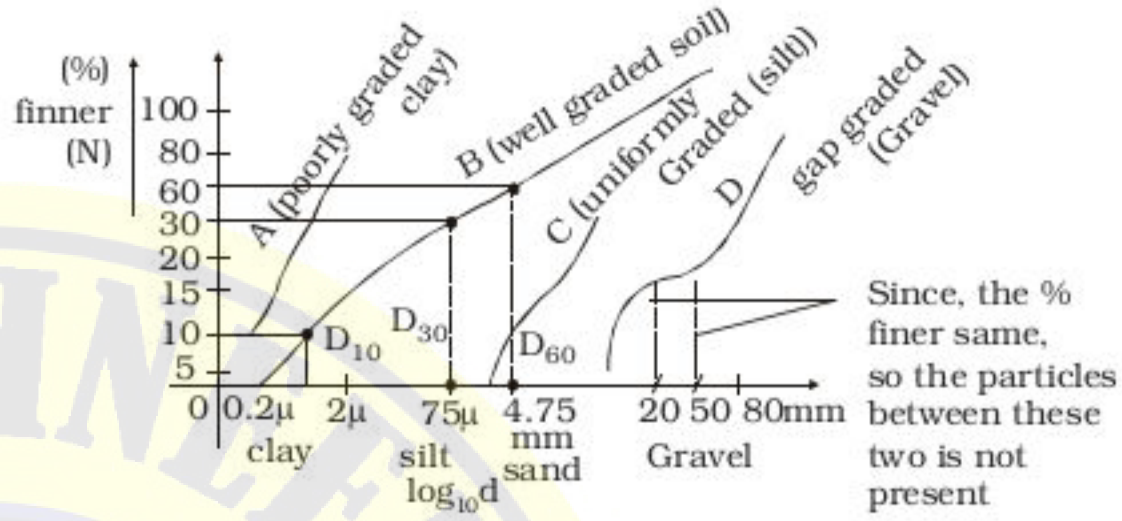
Particle size distribution curve helps in analysing the type and gradation of soil.

The soil may be termed as well graded soil or poorly (uniformly) graded soil.

A well graded soil is one which has good representation of all size of particles whereas poorly graded soil is that which either has excess of 1 size of particle deficiency of another size of particles or it contains same size of particles.

In the later case, the soil is termed as uniformly graded soil.

A soil which has deficiency of certain size of particles, then such type of soil is termed as Gap Graded soil.



For coarse soils, certain size of particles like D_{10}, D_{30}, D_{60} assumes higher significance than other size of particles as they help in representing the parameters of particle size distribution.

Note : If D_{10} represents the size in mm such that 10% of particles are finer than this size. It is also known as effective size.

These sizes of particles help in analysing the shape parameter of the curve.

(a) Co-efficient of uniformity (C_u) :

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

(B) Co-efficient of curvature; (C_c).

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

Note : Poorly graded soil is formed due to running water or wind, whereas well graded soil is formed due to ice.

For a soil to be well graded:

[$1 < C_C < 3$] and [$C_u > 4$] for gravels

[$1 < C_C < 3$] and [$C_u > 6$] for sands

$C_u \approx 1$ for uniform soils/poorly graded soils.

(5) Consistency of soil (Atterberg limits) :

It is meant as for the relative ease with which soil can be deformed because it denotes the degree of firmness of the soil.

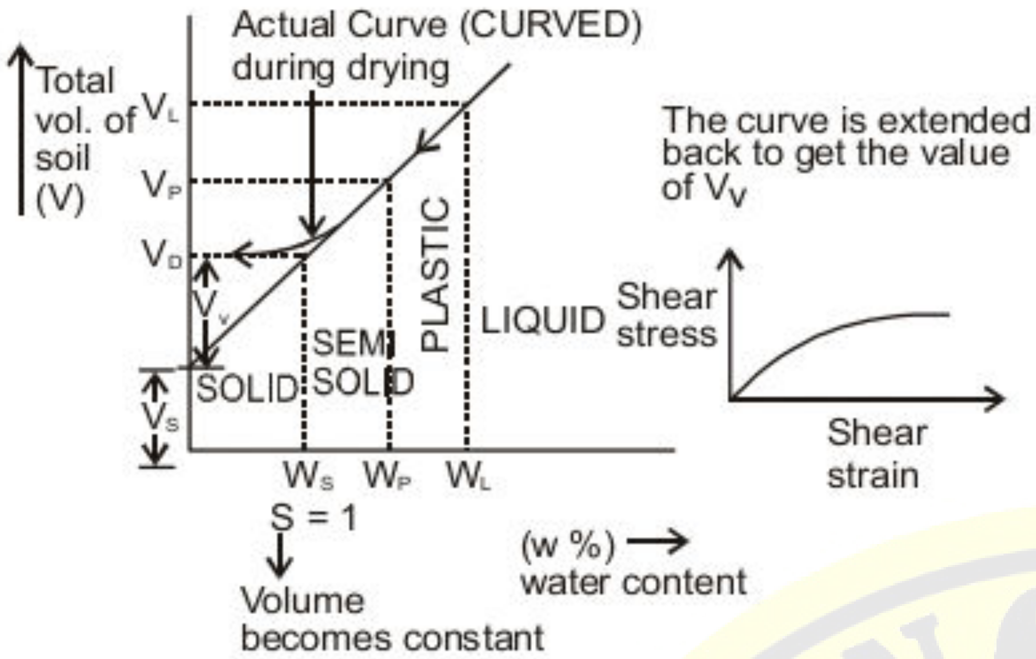
Consistency is generally used for cohesive soil which may vary between soft, stiff & hard.

Atterberg classified, the consistency of the soil in four stages.

(i) **SOLID** (ii) **SEMI-SOLID** (iii) **PLASTIC** (iv) **LIQ-**

UID

The water content at which soil passes from 1 stage of consistency to another is termed as consistency limit.



(a) Liquid limit (W_L) : It is defined as the minimum water content at which soil is still in liquid stage of consistency or it may also be defined as minimum water content at which soil has tendency to flow.

All the soils at liquid limit possess negligible shear strength that can be nearly measured.

TYPES OF SOIL	LIQUID LIMIT (%)
Gravel	Non Plastic
Sand	Non Plastic
Silt	30-40
Clay (Alluvial Soil Clay)	40-150
Clay (Black Soil)	400-500
Clay (Bentonite Soil)	400-800

If liquid limit (w_L) is higher then compressibility is more.

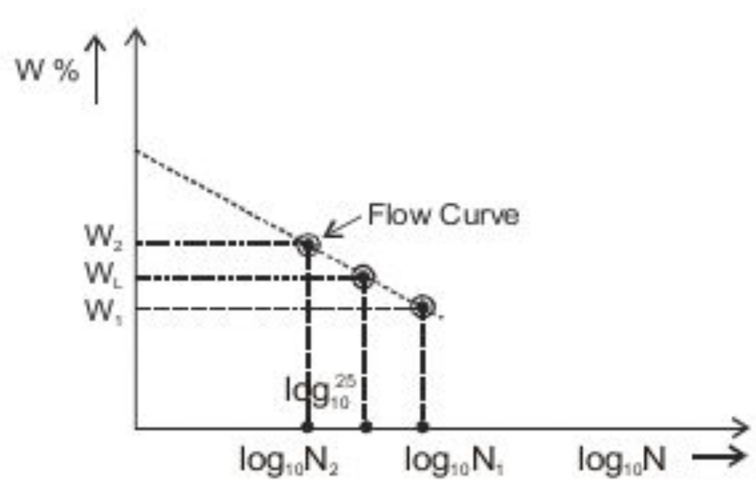
Note :

High plastic soils have high liquid limit hence posses higher compressibility.

Liquid limit is also defined as the water content at which a groove cut in a part of soil by a grooving tool of standard dimensions will flow together for a distance of 13 mm under the impact of 25 blows in a standard liquid limit device.

Flow Index (I_F) : It is slope of flow curves obtained between the number of blows and the water content in determination of liquid limit.

It should be after heading at liquidity Index.



$\tan\theta = I_f$ = flow Index

$$I_f = \frac{W_2 - W_1}{\log_{10} N_1 - \log_{10} N_2}$$

$$I_f = \frac{W_2 - W_1}{\log_{10} \dfrac{N_1}{N_2}}$$

Flow Index represents the rate of loss of shear strength of the soil with increase in water content (Rate of loss of shear strength of soil is comparatively more than the rate of increasing water content, hence the soil is not used (low quality soil).

Liquid limit can also be computed by :

Static cone penetration method

(b)Plastic Limit :

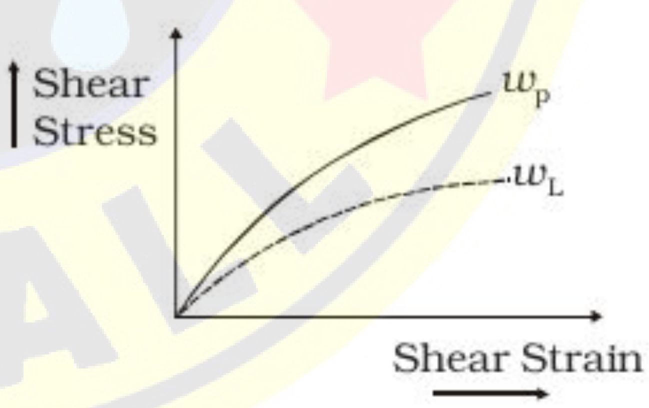
It is defined as minimum water content at which soil just begins to crumble when rolled into a thread of 3 mm size (diameter).

It is the range of water content at which soil passes from semi-solid state of consistency to plastic state of consistency.

For coarse grained soil i.e. gravel & sand liquid limit and plastic limit are very close to each other & are of no significance as these are non plastic soils.

But for clay & silt, liquid limit >>> plastic limit.

Note : When sand is added in clay, liquid limit & plastic limit both reduces, but reduction in plastic limit is comparatively less than the reduction in liquid limit.



(C) Shrinkage limit :

It is defined as maximum water content at which further reduction in water content of the soil does not lead to reduction in volume of soil.

It implies that below shrinkage limit, water is being replaced by air on drying.

Shrinkage limit may also be defined as minimum water content at which soil can remain in completely saturated state.

At shrinkage limit, S =1

Above shrinkage limit, S > 1

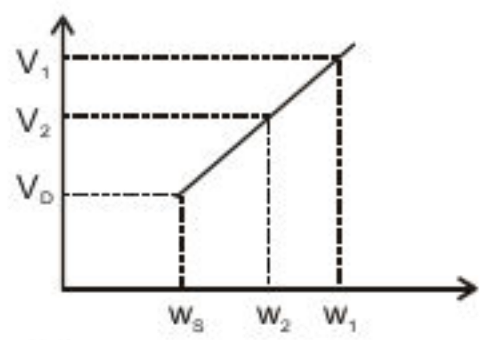
Where, S = Saturation Degree

Shrinkage limit denotes the water content at which soil passes from solid state of consistency to semi-solid state of consistency or either way.

- **SHRINKAGE RATIOS :** It is defined as the ratio of change in volume of the soil expressed as the percentage of dry volume to the corresponding change in water content above the shrinkage limit Hence, also considered as soil mass specific gravity in its dry state.

Shrinkage ratio,

$$(R) = \frac{\frac{V_1 - V_2}{V_d}}{W_1 - W_2} \times 100$$



where,

$$W_1 - W_2 = \frac{(V_1 - V_2)\ w}{M_d}$$

$$\therefore\ SR = \frac{\cancel{V_1 - V_2}}{V_d} \times 100 = \frac{\rho_d}{\rho_w} \times 100 \Rightarrow \frac{\rho_d}{\rho_w}$$

$$\Rightarrow\ R = \frac{d}{w} \times 100\ \% = \frac{d}{w} = G_d$$

If $V_2 = V_d \Rightarrow W_2 = W_s$

$$\therefore\ R = \frac{\frac{V_1 - V_d}{V_d}}{W_1 - W_s} \times 100 \quad \text{upto Shrinkage limit}$$

- **VOLUMETRIC SHRINKAGE :** It is defined as decrease in volume of soil expressed in term of its dry volume when water content is reduced from its given value upto shrinkage limit.

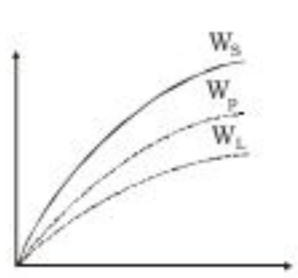
$$V_S = \frac{V_1 - V_d}{V_d} \times 100$$

$$\therefore \quad V_S = SR\ (W_1 - W_S)$$

- **LINEAR SHRINKAGE :** It is defined as decrease in one dimension of the soil expressed in terms of original dimension.

$$L_S = 100\left[1 - \left(\frac{100}{100 \quad V_s}\right)^{1/3}\right]$$

degree of firmness increases as we move from liquid to solid.



STATE OF CONSISTENCY	LIQUID	PLASTIC	SEMI SOLID	SOLID
Water content	W_L	W_P	W_s	O
Consistency	LIQUID ↔	Very soft to stiff ↔	Very stiff to very hard ↔	
Degree of sat(S)	↔ S = 1	↔	O < S < 1 ↔	
Volume	↔ Decreases	↔	Constant ↔	

Degree of Shrinkage (DOS): It is defined as decrease in its orginal volume when water is reduced from its given stage upto its shrinkage limit.

$$D.O.S. = \frac{V_1 \quad V_d}{V_1} \quad 100$$

$$D.O.S. \propto \frac{1}{\text{Plasticity}}$$

D.O.S.	Plasticity
< 5	Good
5-15	Medium good
15-20	Poor
> 20	Very poor

- **PLASTICITY INDEX :** It is the range of consistency in which soil behaves as a plastic material or it exhibits plastic properties. It is numerically equal to difference of liquid limit and plastic limit

$I_P = W_L - W_P$

Plasticity is the property of the soil due to which it undergoes deformation without cracking, fracturing or rupturing.

I_P	Plasticity	Type of Soil	I_P(%)
0	None-Plastic	Gravel	Non Plastic
< 7	Low-Plastic	Sand	Non Plastic
7 - 17	Medium-Plastic	Silt	10-15
> 17	High-Plastic	Clay [Alluvial soil]	15-100
		Clay [Black soil]	200-250

For coarse grained, there is no plastic state and their LL coincides with PL i.e. $I_P = 0$.

If PL ≥ LL. I_P is reported as zero.

- **SHRINKAGE INDEX** (I_S) **:** It is the range of consistency in which soil shows semi-soild behaviour Semi-Solid properties.

$I_S = W_p - W_s$

- **CONSISTENCY INDEX (I_C) (Or Relative Cosistancy)**

It is the ratio of difference of liquid and limit natural moisture content of soil to the plasticity Index.

$$I_C = \frac{W_L - W}{I_P} \quad \text{where ; } I_P = W_L - W_P$$

$$I_C = \frac{W_L - W}{W_L - W_P}$$

The behaviour of in-situ saturated fine grained soil is analysed with the help of this consistency index.

If $W > W_L$; $I_C < 0 \rightarrow$ Soil behaves as liquid but only upon disturbance.

$W_p < W < W_L$; $0 < I_C < 1 \rightarrow$ Behaves as Plastic material

$W < W_P$; $I_C > 1 \rightarrow$ Behaves as solid or semi solid and will be very hard or stiff.

◆ **LIQUIDITY INDEX :** It is defined as the ratio of difference of natural water content & plastic limit to the plasticity Index of the soil. It is also termed as water plasticity.

$$I_L = \frac{w - w_p}{I_p} \quad \frac{w - w_p}{w_L - w_p}$$

If :- $w > w_P$; $I_L > 1$; Soil is in Liquid state

$w_P < w < w_L$; $O < I_L < 1$; Soil is in plastic state

$w < w_L$; $I_L < O$; Soil is in solid or semi solid state

Note : Both I_C & I_L indicates the same behaviour of in-situ fine grained saturated soil. Significance of liquid Index is same as that of consistency index.

Consistency	Description	I_c	I_L	UCS (kg/cm²)
LIQUID	Liquid	< O	> 1	
PLASTIC	Very soft	0.25-0.5	1 - 0.75	0 - 0.25
	Soft	0.25-0.5	0.75-0.5	0.25 - 0.5
	Medium stiff	0.5-0.75	0.5-0.25	0.5 - 1.0
	Stiff	0.75-1.0	0.25-0	1 - 2
SEMI SOLID	Very stiff to hard	> 1	< 0	2 - 4
SOLID	Hard to very hard	> 1	< 0	> 4

Toughness Index (I_T) : It is defined as the ratio of plastic Index to the flow Index.

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

I_T gives an idea about the shear strength of a soil at plastic limit. For the same value of plasticity Index, two soils exhibit different toughness based on flow index.

For most of the soils, $0 < I_T < 3$, when $I_T < 1$ the soil is friable (easily crushed) at the plastic limit.

◆ **Unconfined compressive strength (Q_u) :**

It is defined as load per unit area at which unconfined specimen of soil fails in simple compression test in undrained conditions.

For clay : UCS = 2 × (the shear strength in undrained conditions).

q_u is related to consistency of clays as:

Consistency	q_u (kN/m²)	(kg/cm²)
Very soft	< 25	< 0.25
Soft	25-50	0.25 - 0.50
Medium	50-100	0.50-1.0
Stiff	100-200	1.0-2.0
Very stiff	200-400	2.0-4.0
Hard	> 400	> 4.0

Relationship between the Atterborg limit and Engineering Properties.

Properties	Comparing Soils at Same W_L & Increasing I_P	Comparing soils at same I_P & Increasing W_L
1. Dry strength	Increases	Decreases
2. Toughness near W_P	Increases	Decreases
3. Permeability	Decreases	Increases
4. Compressibility	Constant	Increases
5. Rate of volume change	Decreases	Increases

Note : Organic soils have high liquid of greater than 50% but also has high plastic limit, hence plasticity index of these soil is comparatively less than their liquid limit.

◆ **SENSITIVITY & THIXOTROPY :**

Consistency of undisturbed soil (clay) is altered even at same water content upon grained soil its remoulding.

This change in consistency (loss in strength) of the soil takes places due to :

i) Permanent destruction of soil solids.

ii) Reorientation of water molecules in the adsorbed layer of soil solids.

This loss in strength of soil is measured in terms of its sensitivity.

Hence, this sensitivity indicates the degree of disturbance of the undisturbed soil sample upon its remoulding.

Sensitivity is defined as the ratio of unconfined compressive strength of the soil in its undisturbed state to the unconfined compressive strength of soil in its remoulding state.

$$S_t = \frac{\text{(UCS) undisturbed state}}{\text{(UCS) Remoulded state}}$$

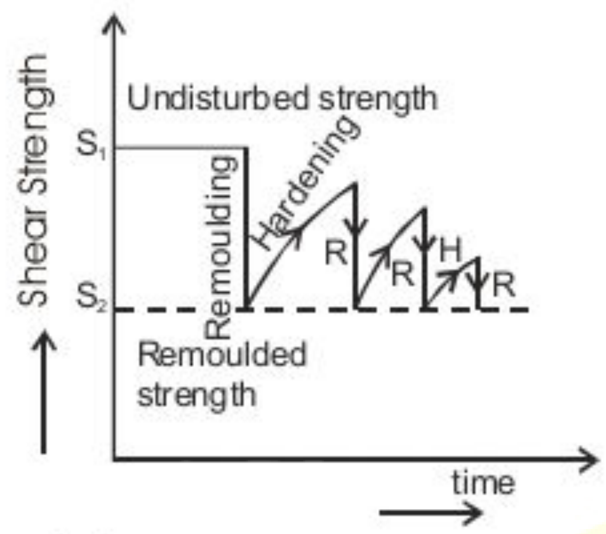
Also compressive strength = 2 × undrained cohesion strength.

S_t	Sensitivity
1	Insensitive soil
2-4	Normal (Less Sensitive soil) (Honey combed Structure)
4.8	Sensitive Soil (Honey comb/ Flocculant Structure)
8-16	Extra sensitive soil (Flocculant Structure)
> 16	Quick Soil (Unstable soil)

◆ This loss in strength of the soil is regained over a period of time and the property of the soil by the virtue of which it regains its lost strength at constant water content and no change in volume over a period of time is termed as **Thixotropy.**

Soil only regains the part of its lost strength due to rehabilitation of the water molecules in the absorbed layer.

The entire lost strength of the soil cannot be regained.
It does not regains the loss on the account of permanent destruction.
High sensitive soil shows high thixotropic hardening (regaining of strength).



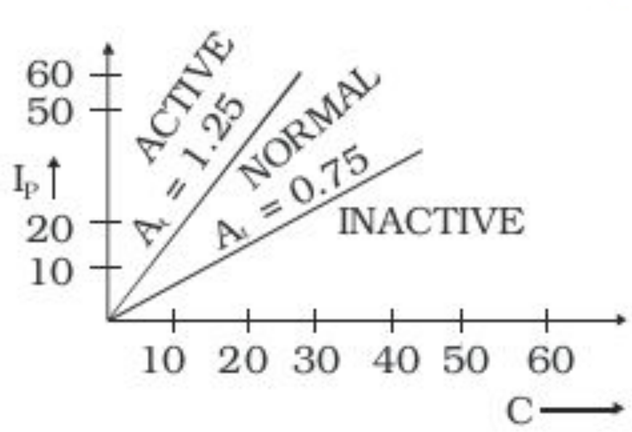
- **ACTIVITY (A_t)**

The behaviour or plasticity of soil depends upon type of clay mineral, amount of clay mineral & amount of water absorbed by the soil.
Skempton defined the parameter termed as activity of the soil which represents the compressibility (Volume change) in its soil with increase in water content of soil.
It is defined as the ratio of plasticity Index of the soil to the percentage of the particle (finer than clay size) 2μ present in it.

$$\therefore \qquad A_t = \frac{I_P}{C}$$

C = Percentage of particle finer than clay (2μ) size.
Higher is the activity of the soil, higher is the compressibility or volume changes in soil with increase in water content. Hence, soils having higher activity are not preferred for engineering works.

A_t	Description
< 0.75	Inactive
0.75-1.25	Normal Active
> 1.25	Active



TYPE OF CLAY MINERAL	A_t
Kaolinite	0.4-0.5
I llite	0.5-1
Montomorillonite	1-7
Na Montmorillonite	4-7
Ca Montmorillonite	1.5

Note : Black cotton soil consists of monomorillonite & hence highly active i.e. shows large volume change with increase in water content.

- **COLLAPSIBITY :**

The soils which shows large decrease in volume with increase in their water content without increase in the load subjected upon them are termed as collapsible soil.
Ex : Loss of soil
It is measured in terms of collapse potential which can be determined by the simple plate load test in the field.

$$\text{C.P.} = \frac{H}{H_O} = \frac{e}{1 \quad e_O} \ ; \ \text{C.P} = \frac{V}{V_O}$$

$V = V_S (1 + e)$
Δe = change in void ratio causes change in volume

C.P. (%)	DESCRIPTION
1	No trouble
1-5	Moderate/Low trouble
5-10	Trouble
10-20	Severe trouble
> 20	Very severe trouble

DIFFERENCE BETWEEN ORGANIC & INORGANIC SOIL :

1.FOR ORGANIC CLAYS ; LIQUID LIMIT (W_L) of oven dry sample is less than 70% of W_L or air dried (moist sample).
As on oven drying both W_L and W_p reduces but reduction in W_p is comparatively less than in W_L.

DIFFERENCE BETWEEN CLAY & SILT IN FIELD :

a) Dispersion test;
Take a spoon of oven dried sample of soil & mix it in the glass of water.
If the solid particles settles in 10-15 mins, soil is silt and if it forms the suspension, soil is clay.

b) Dilatency test :
Take a suitable sample of saturated soil & place it over the palm. If moisture appears over the surface the soil on remoulding, soil is silt & if the moisture does not repair, it is clay.
Silt looses its adsorbed water on remoulding.
Clay does not looses adsorbed water on remoulding Therefore, clay is used for making of earthen pots.

c) Dry strength test :
Prepare 3 mm diameter ball of soil and subject it to become dry. After drying place the ball between the finger & if the soil balls are crushed upon the application of pressure then it is silt & vice-versa. Dissicated strength of clay is more than that of silt.

d) Toughness test :
Roll the sample of soil in 3 mm diameter thread.
If the soil begins to crumble, it is silt & if the threads can be prepared easily, then it is clay.

CLASSIFICATION OF SOILS

It is done to arrange the soil into various groups on the basis of its Engineering properties.

It can be done by any of the following method ;

1. Particle size classification
2. Textural classification
3. Highway Research Board (HRB)
4. Unified soil classification system (USCS) or Indian soil classification system (ISCS)

1. Particle size classification system :

In this system, classification is done on the basis of particle size composition.

SOIL	SIEVE SIZE	SOIL	SIEVE SIZE
- Clay	< 0.002mm	Boulder	> 300 mm
- Silt	$2\mu < d < 75\mu$		
- Fine Silt	$2\mu < d < 10\mu$		
- Medium Silt	$10\mu < d < 20\mu$		
- Coarse Silt	$20\mu < d < 72\mu$		
Sand	**$75\mu < d < 4.75$ mm**		
- Fine Sand	$75\mu < d < 0.425$ mm		
- Medium sand	0.425mm $< d < 2$ mm		
- Coarse Sand	2mm $< d < 4.75$ mm		
Gravel	**$4.75 < d < 80$ mm**		
- Fine Gravel	4.75mm $< d < 20$ mm		
- Coarse Gravel	20mm $< d < 80$ mm		
- Cobble	80mm $< d < 30$ mm		

TEXTURAL CLASSIFICATION :

Highway Research Board classification (HRB) Or 'Aashto Soil classification system' :

Group Index = 0.2 a + 0.005 ac + 0.01 bd.

a = It is the proportion of percent passing through 75μ. Sieve greater than 35 and less than 75, expressed as a whole number from 0 to 40.

If percent = 55.5

then a = 55.5 – 35

= 20.5 = 21

If,	% = 27 < 35	% = 35
	a = 0	a = 0
	% = 90 > 75	% = 75
	a = 0	a = 40

b = It is the proportion of the percent passing through 75μ sieve, greater than 15 and less than 55, expressed as a whole number between 0 to 40.

c = It is the portion of liquid limit > 40 but not exceeding 60 expressed as a whole number from 0 to 20.

d = It is the portion of plasticity Index > 10 but not exceeding 30 expressed as a whole number between 0 to 20.

$\therefore$ Maximum G.I $= (0.2 \times 40) + (0.005 \times 40 \times 20) + (0.01 \times 20 \times 4)$

According to AASHTO system, the soils are classified into eight groups A-1 through A-7 with an additional group A-8 for peat or muck.

The system includes several subgroups.

Greater the G.I. value, the less desirable a soil is for highway construction with in that subgroup.

4. Indian soil classification system :

ISCS classifies the soil on the basis of particle size composition, plasticity characteristic and compressibility of the soil.

In this system, soils are broadly classified into Coarse grained soil and Fine grained soil.

(1) Coarse Grained Soil :

A soil is termed as coarse grained soil, if more than 50% of its fraction are retained on 0.075 mm sieve. It is further classified into two; Gravel & Sand.

Coarse Grained soil is termed as **gravel** if 50% or more coarse grained fraction (fraction retained on 0.075mm sieve) is more than 4.75 mm (or) is retained on 4.75 mm sieve) or otherwise coarse grained soil is termed as sand.

In this system, coarse grained soil is further divided on the basis of percentage fineness (percent of particle finer than 0.075mm).

(a) If percent fineness is < 5% :

COARSE GRAINED SOIL

GRAVEL (G)

GW
* Well Graded
* Gravel > Sand

$C_u > 4$

$1 \leq C_c \, 3 \leq$

GP
* Poorly Graded Gravel
* Gravel > Sand
* Condition of C_u & C_c are not mentioned

SAND (S)

SW
* Well Graded Sand
* Sand > Gravel

$C_u > 6$

$1 \leq C_c \, 3 \leq$

SP
* Poorly Graded Sand
* Sand > Gravel
* Condition of C_u & C_c are not mentioned

(b) If % fineness is > 12% ;

COARSE GRAINED SOIL

GRAVEL (G)		SAND (S)	
GM	GC	SM	SC
* Silty Gravel	* Clayey Gravel	* Silty Sand	* Clayey Sand
* Gravel > Sand	* Gravel > Sand	* Sand > Gravel	* Sand > Gravel
* Silt > Clay	* Clay > Silt	* Silt > Clay	* Clay > Silt
* $I_P > 4\%$	* $I_P > 7\%$	$I_P > 4\%$	$I_P > 7\%$

M = Silt, C = Clay, (Silt is less plastic than clay)

(2) Fine Grained Soil :

A soil is termed as fine grain if more than 50% of its fraction are finer than 0.075 mm. [i.e. it passes through 0.075 mm sieve]. It is further classified into 3.

(a)Inorganic clay (b) Inorganic silt (c) Organic clay & silt

In this system, fine grained soil are also classified on the basis of compressibility as low compressible soil, intermediate compressible soil and high compressible soil.

FINE GRAINED SOIL

Clay (C)	Silt (M)	Organic clay & silt or Soil having organic matter (O)

COMPRESSIBILITY

Low compressible soil (L) WL < 35%	Intermediate compressible soil (I)	High compressible soil (H)

In order to separate clay from silt and organic soil, A. Casagrande defined a curve termed as A. line which represents the relationship between plasticity index & liquid limit of the soil.

(3) If percent fineness is in between 5-12% [Dual symbol representation]

COARSE GRAINED SOIL

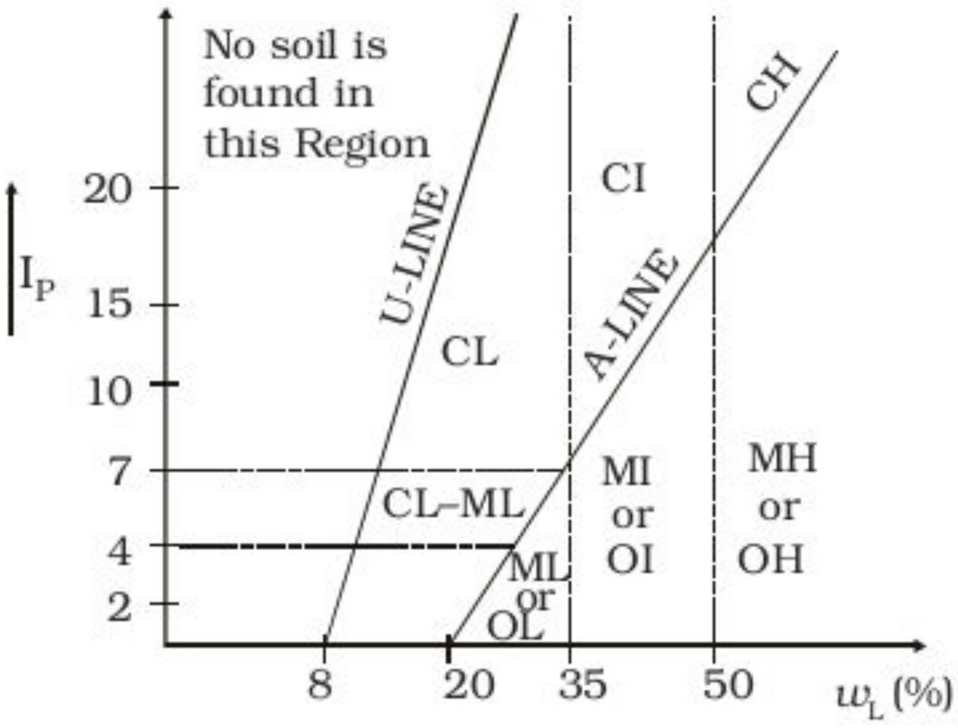
Gravel (G)				Sand (S)			
GW-GM	GW-GC	GP-GM	GP-GC	SW- SM	SW-SC	SP-SM	SP-SC
* Well graded gravel with silt	* Well graded gravel with clay	* Poorly graded Gravel with silt	* Poorly graded Gravel with clay	* Well graded sand with silt	* Well graded sand with clay	* Poorly sand Gravel with silt	* Poorly graded sand with clay
* Gravel > Sand	* Gravel > Sand	* Gravel > Sand	* Gravel > Sand	* Sand > Gravel	* Sand > Gravel	* Sand > Gravel	* Sand > Gravel
* Silt > Clay	* Clay > Silt	* Silt > Clay	* Caly > Silt	* Silt > Clay	* Clay > Silt	* Silt > Clay	* Caly > Silt
* Cu > 4	* Cu > 4	* $I_P > 4\%$	* $I_P > 7\%$	* Cu > 6	* Cu > 6	* $I_P > 7\%$	
* $1 \le C_c \le 3$	* $1 \le C_c \le 3$	* Condition of Cu & C_c are not met	* Condition of Cu C_c & are not met	* $1 \le C_c \le 3$	* $1 \le C_c \le 3$	* Condition of Cu C_c & are not met	* Condition of Cu C_c & are not met
				* $I_P < 4\%$	* $I_P > 7\%$	* $I_P < 4\%$	* $I_P < 77\%$

Clay as present above the A-line & silt, organic are found to exist below the A-line.

Equation of A-line ; $I_P = 0.73\ (W_L - 2a)$

U line is used to mark the upper limit of the existence of the soils.

No soil is found to exist above the U-line ($I_P = 0.9\ (W_L - 8)$).



◆ **SOIL (GROUND) WATER :**

Water present in the soil in any form is termed as soil water. It is of following types :

1. **Gravity Water :**

It is the water that fills the voids of the soil continuously upto ground water level or free surface water level.

This water is being subjected to no other force than gravity but obeys all the laws of hydraulics. It is capable of moving under hydro-dynamic forces.

2. **ABSORBED WATER :**

It is generally of two types,

(i) **Hygroscopic water** is that which is being absorbed by the soil solids from the atmosphere by the physical forces of attraction. It is held over the surface of the solids due to adhesion.

Coarse solid absorbs comparatively smaller quantity of water than fine solids.

Water absorbed by sand ≃ 1%

Water absorbed by silt ≃ 6 to 7%

Water absorbed by clay ≃ 16 to 17%

(ii) **Film water** is also absorbed by the soil solids but is present over the hygroscopic water and is formed due to the condensation of aqueous vapour over the solids.

All the properties of film water are comparatively linient than hygroscopic water.

3) **STRUCTURAL WATER :** It is the water that is chemically combined to the crystal, structure of the soil minerals.

POROUS WATER

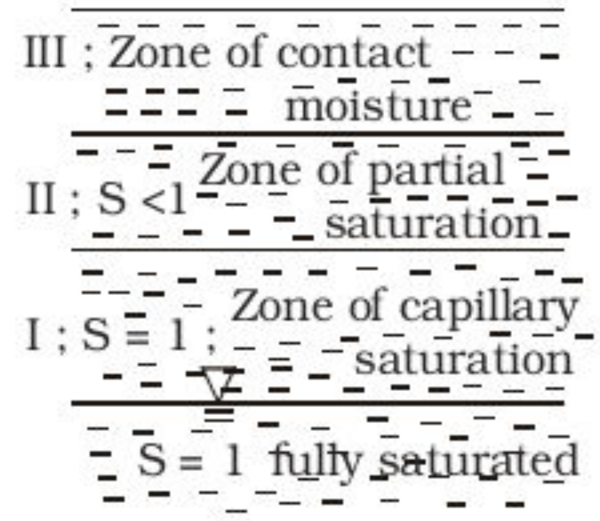
Gravity water — Capillary water

4) **CAPILLARY WATER :** It is the water which is being lifted by surface tension forces above the free ground water surface. It is held in suspension between the voids of the solids.

This water fills the voids upto a certain distance above the Ground Water level (free).

The distance of this zone is termed as zone of capillary saturation.

(Adhesive force are dominant near the Water Table and voids are fully saturated but further with increase in height, the adhesive force decreases).



$d/2 = R \cos\alpha$

$d = 2R \cos\alpha$

$\therefore h_c = \dfrac{2T_S}{.R}$;

h_{cmax} is observed, at $\alpha = 0$

If we increase the water table the $R \to \alpha$ (increases)

$R \to \alpha$; h_c (capillary rise) is zero

Capillary pressure : It is the pressure in soil solids which is constant throughout the height of capillary tube.

Note : At 20°C , Surface tension (T_S) is 72.8×10^{-8} kN/cm

$$\therefore h_c = \frac{2.94 \text{ cm}}{d \text{ mm}} \quad \frac{0.294 \text{ cm}}{d \text{ cm}}$$

The water present in the capillary tube is held in tension unlike water present below surface water table which is in state of compression.

The maximum tensile stress is at the level of the meniscus and it gradually reduces to free water surface level.

This tension stress in water is termed as capillary tension, capillary potential (or) soil suction.

In case of soil, interconnecting voids are present in both vertical and horizontal direction, hence theoretical application is not valid in case of soil.

The capillary rise in case of soil can be computed by the following method.

i) The effective size of capillary is taken to be 20% of effective size of soil solids

$\therefore d = 20\% \ D_{10}$

ii) Maximum rise of capillary in soils can be computed by following [HAZEN"S Equation]:

$$hc_{max} = \frac{C}{eD_{10}} \ [D_{10}(\text{cm})] \quad c = \text{constant} = 0.1 \text{ to } 0.5 \text{ cm}^2$$

e = void Ratio

iii) The size of capillary is computed assuming voids to be spherical and volume of solids to be directly proportional to the effective size of solid.

$d = e^{1/3} . D_{10}$

Rise of capillary water in types of soil

Types of soil	h_c (cm)
Gravel	2-10
Sand	10-100
Silt	100-1000
Clay	1000-3000

In clay if voids ratio is more then rise is less and vice-versa but in sand, comparison on this basis of void ratio is not possible.

- **SLAKING IN CLAY :**

When the sample of clay is dried below its shrinkage limit and then emerged into the water, it leads to its disintegration into soft wet mass termed as slaking of clay.

When soil is dried below its shrinkage limit, air is present in its void & when it is suddenly immersed into the water, water enter into these voids leading to development of the meniscus with the subsequent destruction of the voids of the sand.

- **FROST ACTION :**

Freezing & thawing (melting) of water present in the voids of the soils affect the properties of the soil. These properties are considered as frost action.

Frost action is generally of two types;

- **FROST HEAVE & FROST BOIL,**

FROST HEAVE : If the temperature falls below the freezing point of the water, water present in the voids of the soil freezes, resulting an increase in volume of the soil as the result of which, heaving (swelling) takes place [Volume of ice is 9% more than that of water].

Calculation ; Let volume of voids = 50%

∴ Increase in volume of soil;

= 50% of 9%

= 4.5%

∴ Eg ; 4.5cm

1m

- **FROST BOIL :** With increase in temperature frozen water present in soil metals, that causes an increase in water content of the soil also increases its consistency hence, it results in loss of its strength.

Bearing Capacity : The load carrying capacity of foundation soil or rock which enables it to bear and transmit loads from a structure.

Gross Pressure Intensity : It is the total pressure at the base of the footing due to the weight of the superstructure, self weight of the footing and weight of the earth fill.

Net Pressure Intensity : It is defined as excess of gross pressure to overburden pressure.

$$q_{net} = q_g - \bar{\ }$$

Where, q_{net} = Net Pressure Intensity

q_g = Gross Pressure

$\bar{\ }$ = Effective Stress = $\gamma' D_F$.

Net Ultimate Bearing Capacity : It is the minimum net pressure causing shear failure of soil.

$$q_{nu} = q_u - \bar{\ }$$

Where, q_{nu} = Net Ultimate bearing capacity

q_u = Ultimate bearing capacity

Net Safe Bearing Capacity :

$$q_{ns} = \frac{q_{nu}}{F_s}$$

Where, q_{ns} = Net safe bearing capacity

F_s = Factor of safety

Safe Bearing Capacity :

$$q_s = q_{ns} + \bar{\ }$$

Where, q_s = Safe bearing capacity

	Parameter	General Shear Failure	Local Shear Failure
1.	Friction angle (ϕ)	> 36°	< 28°
2.	Strain of failure	≤ 5°	≥ 15°
3.	S.P.T. number	> 30	< 5
4.	Relative density	> 17%	< 20%
5.	Void ratio	< 0.55	> 0.75
6.	Unconfined	> 100 kN/m²	< 80 kN/m²

Method to determine bearing capacity :

Terzaghi Method (C – ϕ) :

Assumptions : S-Strip footing, S-Shallow foundation, G-General Shear, Failure H-Horizontal ground, R - Rough base.

For Strip Footing :

$$q_u = CN_C + \gamma D_F N_q + \frac{1}{2}\gamma B N_\gamma$$

For Square Footing :

$$q_u = 1.3\ CN_C + \gamma D_F N_q + 0.4\gamma B N_\gamma$$

For Rectangular Footing :

$$q_u = \left(1 \quad 0.3\frac{B}{L}\right) CN_C \qquad D_F N_q$$

$$\frac{1}{2}\left(1 \quad \frac{0.2B}{L}\right) \quad BN$$

For Circular Footing :

$$q_u = 1.3\ CN_C + \gamma D_F N_q + 0.3\gamma D N_\gamma$$

Where,

D = Diameter of circular footing

CN_C = Contribution due to constant component of shear strength of Soil.

$\gamma D_1 N_q$ = Contribution due to surcharge above the footing.

$\frac{1}{2}\gamma BN_\gamma$ = Contribution due to bearing capacity due to self weight of soil.

Bearing Capacity Factors :

$$N_q = N_e \times e^{\pi \tan\phi}$$

Where, N_ϕ = Influence factor

$$N_\phi = \tan^2\left(45 \quad \frac{}{2}\right)$$

$$N_\gamma = 1.8 \tan\phi\,(N_q - 1)$$

$$N_C = \cot\phi(N_q - 1)$$

For C-Soil : $\boxed{N_c \quad 5.7}$, $\boxed{N_q \quad 1}$, $\boxed{N \quad 0}$

Terzaghi has considered general shear failure but if soil is loose and failure is local shear failure then modified values of C and ϕ expected to be used are :

$$C' = \frac{2}{3}C,\ \phi' = \tan^{-1}\left(\frac{2}{3}\tan\right)$$

IS Code :

$$q_{nu} = CN_C S_C i_C d_C + \gamma D_F (N_q - 1)$$

$$S_q i_a d_c + \frac{1}{2} B\gamma N_\gamma S_\gamma d_\gamma$$

Plate Load Test :

1. Significant only for cohesionless soils.
2. Short duration test hence, only results in immediate settlement.

(i) For ϕ – Soil, $\frac{q_{UF}}{q_{UP}} \quad \frac{B_F}{B_P}$

(ii) For C-Soil $q_{UF} = q_{UP}$

If plate load test carried at foundation level then

$$\frac{S_F}{S_P} \quad \left[\frac{B_F(B_P \quad 0.3)}{B_P(B_T \quad 0.3)}\right]^2$$

When foundation is located at deeper depth say by amount D_2

$$S_F \text{ corrected} = S_F \times \left[\frac{1}{\left(1 \quad \frac{D_2}{B_F}\right)}\right]^{0.5}$$

(iii) For dense sand, $\frac{S_F}{S_P} \quad \left[\frac{B_F(B_P \quad 0.3)}{B_P(B_F \quad 0.3)}\right]^2$

(iv) For Clays, $\frac{S_F}{S_P} \quad \frac{B_F}{B_P}$

(v) For Silts, $\frac{S_F}{S_P} \quad \left(\frac{B_F}{B_P}\right)^{n \quad 1}$

Where,

q_{UF} = Ultimate bearing capacity of foundation

q_{UP} = Ultimate bearing capacity of plate

S_F = Settlement of foundations

S_P = Settlement of plate

B_F = Width of foundation in meter

B_P = Width of plate in meter

Standard Penetration Test :

Significant for Granular Soil

(i) $N_1 = N_0 = \frac{350}{(\bar{\ } \quad 70)}$

and $\bar{\ } \times 280$

Where,

N_1 = Overburden Pressure Correction

N_0 = Observed value of S.P.T. number

$\bar{\ }$ = Effective overburden pressure at the level of test in kN/m^2.

(iii) For saturated $\bar{\ }$ fine sand and silt, where $N_1 > 15$

$$N_2 = \frac{1}{2}(N_1 - 15) + 15$$

Where,

N_2 = Dilatency correction or water table correction

$N_q + N_\gamma$ related to N value using Peck Hanson curve or (code method).

Teng's formula relate N value with reading capacity of granular soil.

Soil Exploration

(i) Inside Clearance :

$$C_i = \frac{D_2 \quad D_1}{D_1} \times 100,\ 1 < C_i < 3$$

(ii) Outside Clearance :

$$C_o = \frac{D_2 \quad D_4}{D_4} \times 100,\ 0 < C_0 < 2$$

(iii) Area Ratio (A_r) :

$$A_r = \frac{D_2^2 \quad D_1^2}{D_1^2} \times 100$$

$A_r < 20\%$ for stiff formation. $A_r < 10\%$ for soft clays.

(iv) Recovery Ratio (L_r) :

$$L_r = \frac{\text{Recovered length of sample}}{\text{Penetrations length of sample}}$$

$L_r = 1$ = Good recovery
$L_r < 1$ = Compressed
$L_r > 1$ = Swelled

Where,
D_3 = Inner diameter of sampling tube
D_4 = Outer diameter of sampling tube
D_1 = Inner diameter of cutting edge
D_2 = Outer diameter of cutting edge

Specific Yield : The specific yield of an unconfined aquifer is the ratio of volume of water which will flow under saturated condition due to gravity effect to the total volume of aquifer (V).

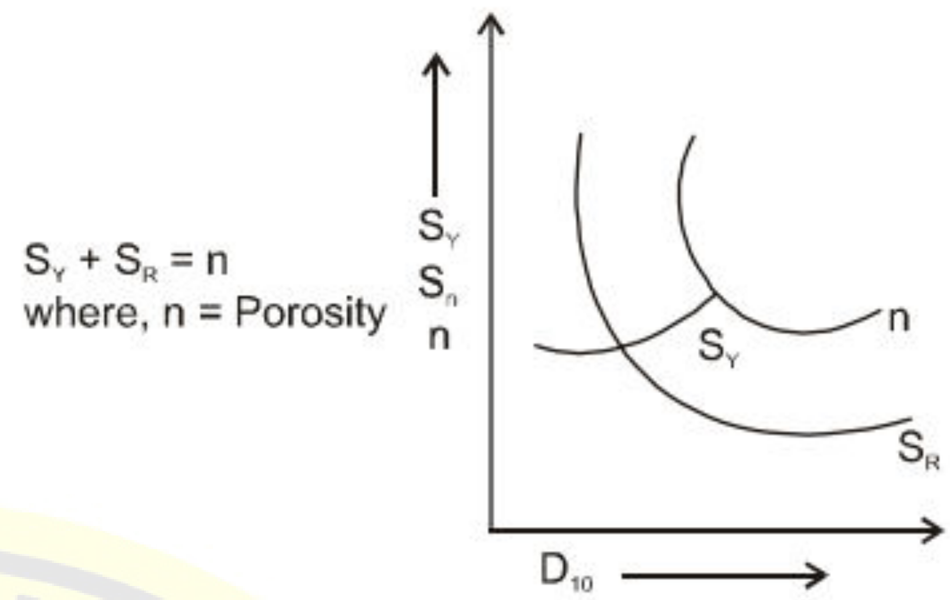
$$S_Y = \frac{V_{wy}}{V}$$

Where, V_{wy} = Volume of water yielded under gravity effect and when V equals total volume of water

Specific Retention : The specific retention of an unconfined aquifer is the ratio of volume of water retained against gravity effect to the total volume of aquifer(V).

$$S_R = \frac{V_{WR}}{V}$$

Where, V_{WR} = Volume of water retained under gravity effect.

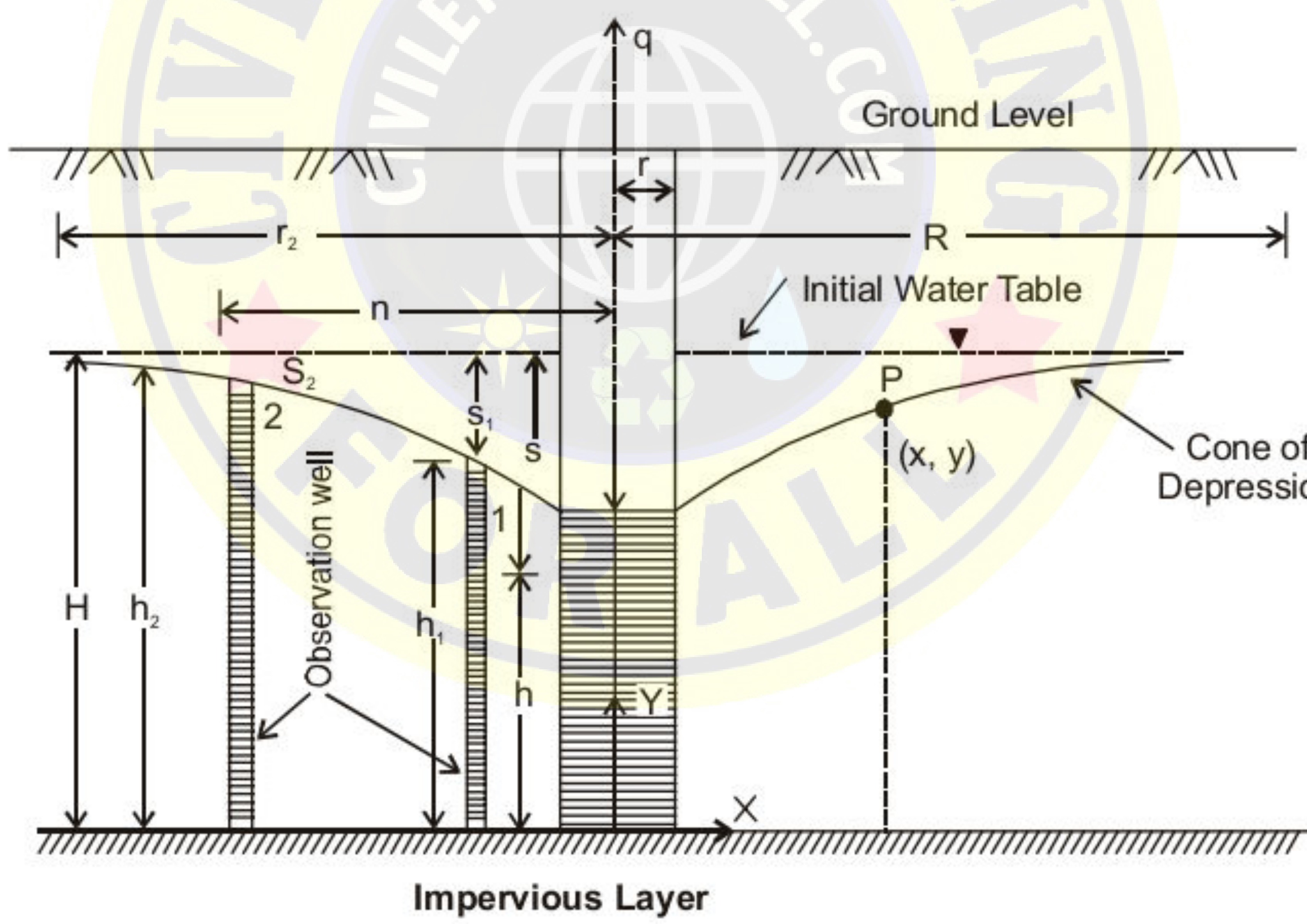


Coefficient of transmissibility :

T = KH

Where, H = Thickness
K = Coefficient of Permeability

Unconfined Aquifer



(a) **Themis Theory**

$$q = \frac{K \;\; (h_2^2 \quad h_1^2)}{2.303 \log_{10} (r_2/r_1)}$$

Where,
q = Rate of flow in m^3/s
h_1 = Height of water table of 1st observation well
$h_1 + s_1 = \quad h_2 + s_2$
h_2 = Height of water table of 2nd observation well
s_1 = Drawdown of 1st test well
s_2 = Drawdown of 2nd test well

r_1 and r_2 are radius of 1st and 2nd observation well respectively.

(b) **Dupuits Theory**

$$q = \frac{K \quad (H^2 \quad h^2)}{2.303 \log_{10}(R/r)}$$

Where,

S = Drawdown in the well

K = Permeability coefficient m/s

R = Radius of influence in metre

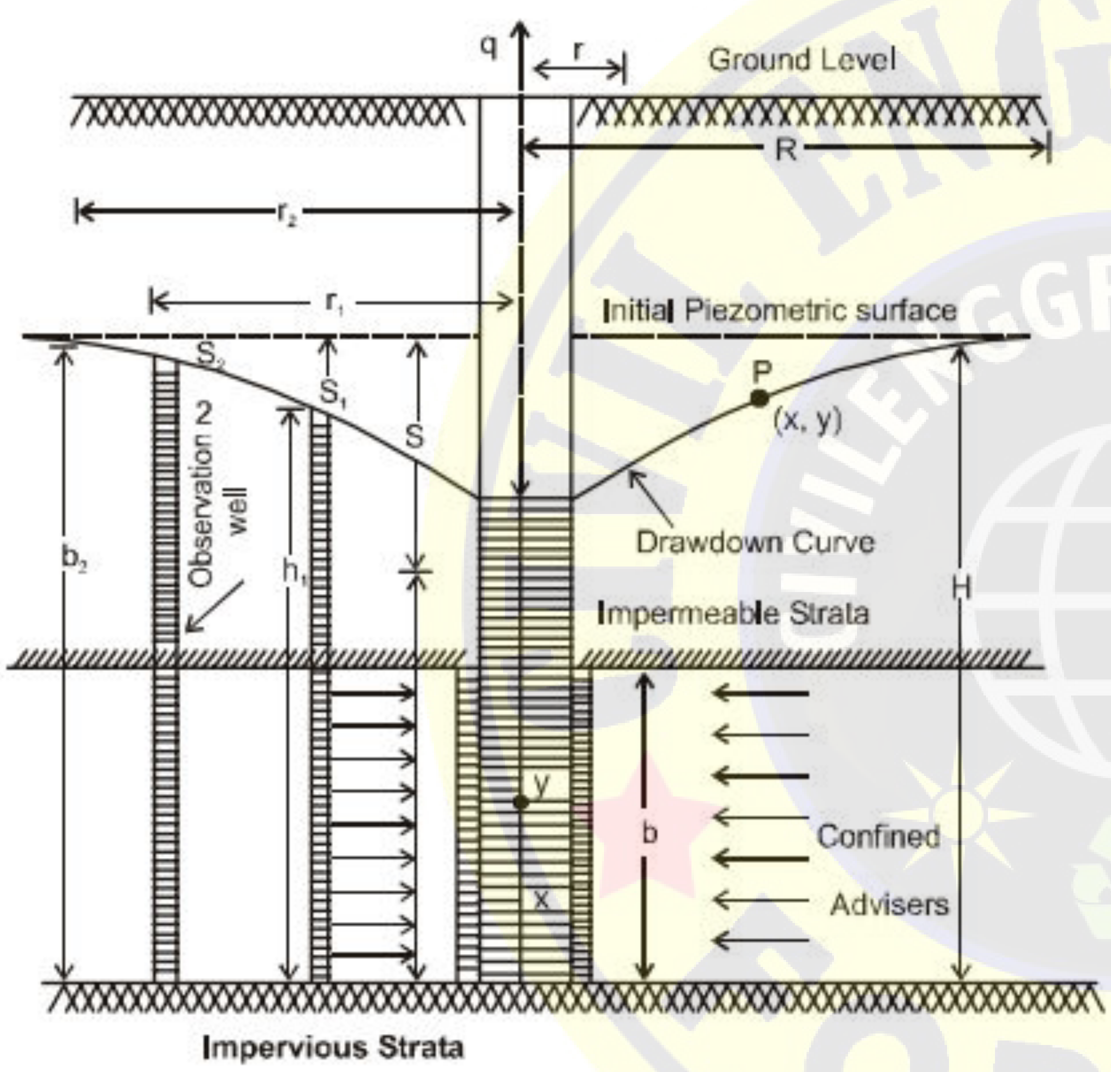
$150 \text{ m} \leq R \leq 300$ metre

r = Radius of test well in metre

$R = 3000.\ S\sqrt{K}$ and $S = H - n$

Result of dupuits theory are not accurate because 'R' is based on empirical relation.

Confined Aquifer



(a) Themis Theory

$$q = \frac{2 \quad bK(h_2 \quad h_1)}{2.303 \log_{10}\left(\frac{r_2}{r_1}\right)}$$

Where, b = width of aquiter.

(b) Dupuits theory

$$q = \frac{2 \quad bK(H \quad h)}{2.303 \log_{10}\left(\frac{R}{r}\right)}$$

EARTH PRESSURE

When designing the retaining wall sheet pile walls & other earth retaining structures, pressure exerted by the retained material on the structures is considered.

The retaining wall or structures is used to maintain the ground surface at different elevations on either side of it.

The material retained by the structures is termed as Backfill, whose surface may be either horizontal or inclined.

The position of the backfill lying above the horizontal plane at the top elevation of the wall is termed as surcharge & the angle made by it with the horizontal is termed as surcharge angle.

σ_V = Vertical stress/vertical earth pressure

σ_h = Lateral earth pressure/lateral earth pressure

$$\frac{h}{v} = K = \text{Co-efficient of earth pressure}$$

The Ratio of horizontal stress to the vertical stress is termed as co-efficient of earth pressure [It can be ; K >1, K < 1, or K = 1]

If the wall is rigid and unyielding, the soil is in the state of rest and there is no deformation and displacement. The earth pressure corresponding to the state is termed as earth pressure at rest. Therefore at rest condition ;

$\epsilon_h = 0$

$$\Rightarrow \epsilon_h = \frac{n}{E} - \mu \frac{n}{E} - \mu \frac{v}{E} = 0 \Rightarrow \sigma_n = \mu\sigma_n - \mu\sigma_V = 0$$

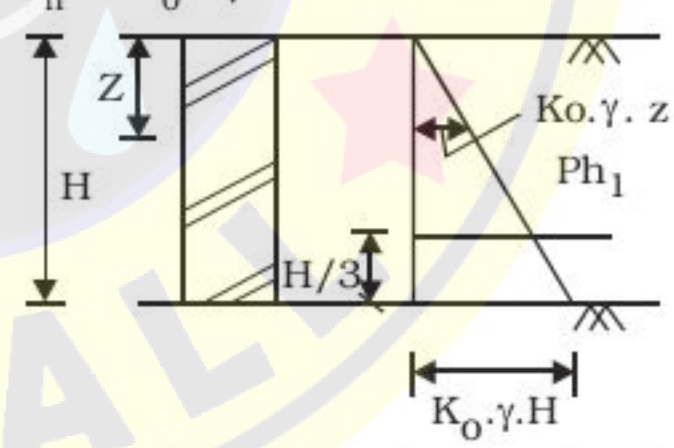
$\sigma_n (1 - \mu) = \mu\, \sigma_V$

$\sigma_n = \sigma_h$ = Co-efficient of Earth pressure at rest.

Therefore At any depth Z,

$\sigma_V = \gamma \,.\, Z \Rightarrow \sigma_n = K_0 \,.\, \sigma_V$

$\sigma_n = K_0 \,.\, \gamma \,.Z$



Considering unit length of wall

Total lateral pressure acting on wall = angular stress × area at which it is acting

$$P_h = [\frac{1}{2}(0 + K_0\,\gamma\,.Z)\,(H \times 1)]$$

$$\frac{1}{2}(K_0\,\gamma\,.\,H)\,(H) = (\frac{1}{2}K_0\,\gamma\,.\,H^2)$$

$$\therefore P_h = \frac{H^2.K_0}{2}$$

$$\text{or, } P_h = \int_O^H K_0\,\gamma z\,dz = K_0\,\gamma\left[\frac{Z^2}{2}\right]_o^H$$

$$\therefore P_h = \frac{K_0 \quad .H^2}{2} \quad \text{[at H/3 from base]}$$

If the soil mass is submerged in water table at ground level then the weight of soil considered is submerged weight.

Hence, when soil mass is submerged ;

At any depth Z ;

$\sigma_n = K_0 \gamma' Z + \gamma_w Z$

$$P_h = K_0 \gamma' \frac{H^2}{2} + \gamma_w \frac{H^2}{2}$$

For pure sand :

$K_0 = (1 - \sin\phi)$ ϕ (angle of friction) increases which implies the soil is more dense and K_0 hence decreases.

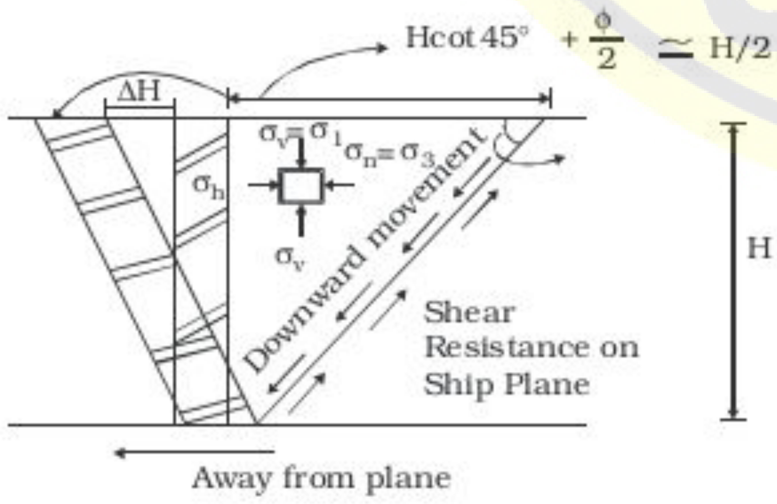
For over consolidated soil :

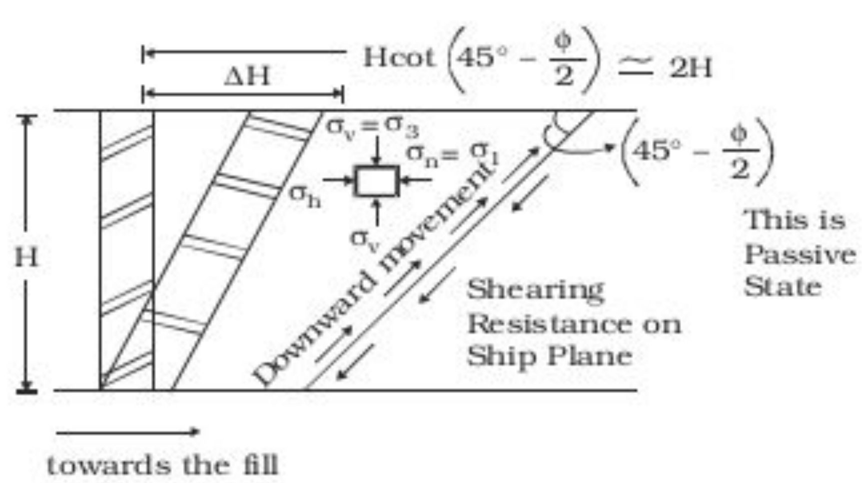
K_0 (O.C) = K_0 (NC) $\sqrt{OCR}$

Type of soil	**K_0**
Dense sand	0.4 - 0.45
Loose sand	0.45 - 0.50
Normally consolidated soil	0.5 – 0.6
Over consolidated soil	1 – 4

◆ **Active & Passive Earth pressure :**

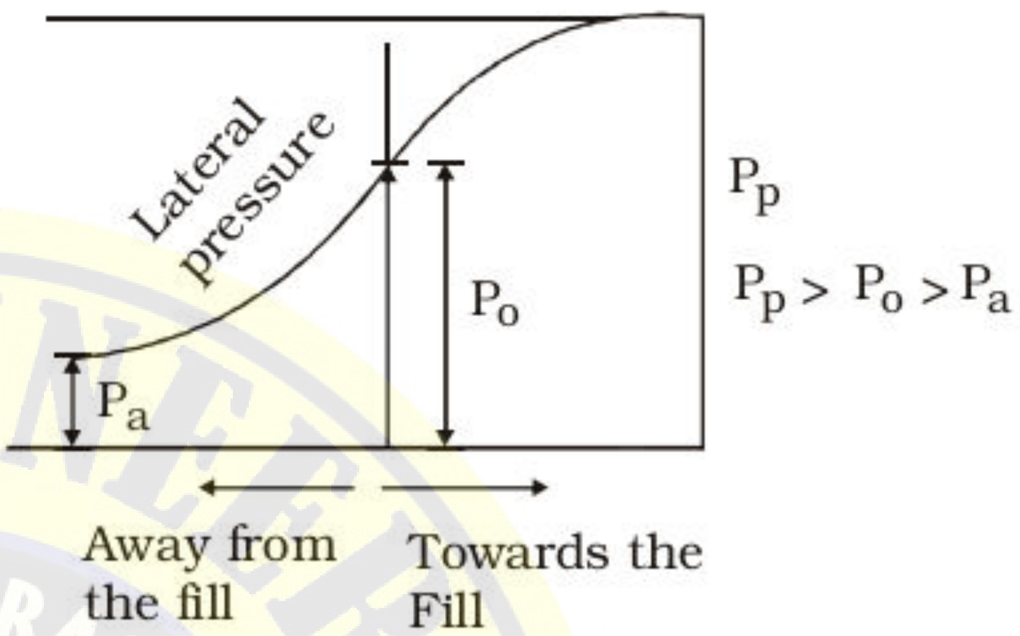
During the active state, wall tends to move away from the backfill and the portion of the backfill just behind the wall tends to move along with the backfill, leaving the rest of the soil mass.This portion of the backfill which moves along with the wall is termed as failure wedge. The resistance offered by the soil due to its shear strength acts in upward direction away from the wall along the failure plane [slip plane] which tends to reduce the earth pressure. When the entire shear resistance of the soil is being mobilized, no further reduction in the earth pressure takes place on any further movement of the wall. This minimum pressure is termed as Active Earth pressure. For active state strain required is in the range of 0.2% to 0.5% [0.5%- loose sand and 0.2% dense sand].





During the passive state the wall tends to move towards the backfill and the resistive force applied by the soil acts downward towards the wall on slip plane resulting an increase of earth pressure. When the entire shear resistance of the soil is mobilized no further increase in the earth pressure takes place with the movement of the wall. This maximum earth pressure is termed as passive earth pressure.

For passive state, the strain required is in the range of 2 to 15%.



[15% → loose sand ; 2% → dense sand]

Acive and Passive Earth Pressure

Active	**Passive**
1. Very little movement is required to mobilise, the Active pressure (about 0.5% horizontal strain).	Much higher movement is required to mobilise, the Pressure (about 2% of horizontal strain).
2. Failure plane is inclined at 45° + ϕ/2.	Failure plane is inclined 45 – ϕ/2.
3. Width of sliding wedge at the top of wall is H.cot (45° + ϕ/2).	width of sliding wedge at the top of wall is H.cot (45° – ϕ/2).

◆ **Rankine Earth Pressure theory :**

Assumptions :

1) Soil is assumed to be homogeneous, isotropic, semi infinite, dry & cohesionless.
2) The ground surface is planar which may be horizontal or inclined.
3) The face of the wall which is in contact with the backfill is vertically smooth.
4) The soil is in plastic equilibrium in active & passive earth pressure condition.
5) The rupture surface is planar which may be obtained by considering the soil to be in plastic stage equilibrium.

Note :

1. Rankine considered elemental failure whereas Coulomb considered wedge failure.
2. In Rankine theory, the face of the wall is vertical & smooth whereas in coulomb theory, the wall may be vertical or inclined and is rough.

From stress Relationship of soil :

$$\therefore \ \frac{h}{v} = \frac{1}{\tan^2\left(45 \quad \frac{}{2}\right)} = \frac{1\sin}{1 \quad \sin} = K_a$$

Here ; K_a = coefficient of active earth pressure

$\sigma_n = p_a$ = active earth pressure

$= K_a.\ \sigma_V = K_a\ \gamma_Z$

$\therefore\ \mathbf{p_a = K_a\ \gamma_Z}$

For Passive State :

$\sigma_1 = \sigma_n;\ \sigma_3 = \sigma_V$

$\sigma_1 = \sigma_V\ \tan^2\ (45° + \frac{\phi}{2})$

$$\therefore \ \frac{h}{v} = \frac{1}{\tan^2\left(45 \quad \frac{}{2}\right)} = \frac{1-\sin}{1 \quad \sin} = K_a$$

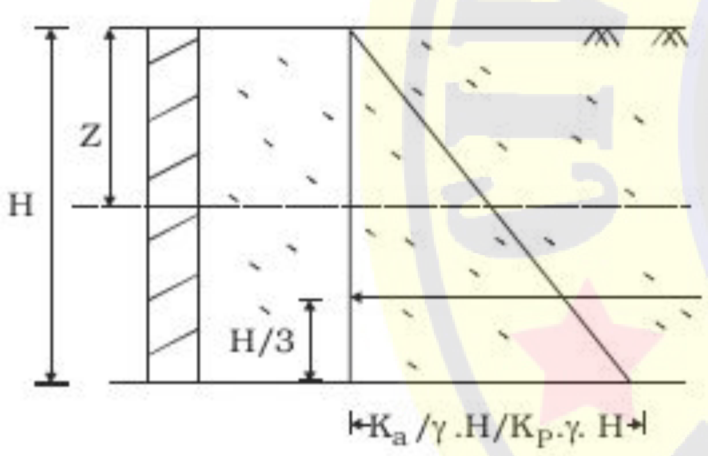
Here, K_p = coefficient of passive earth pressure

$\sigma_h = p_p = K_p\ \sigma_V$ = passive earth pressure

$\therefore\ p_p = K_p.\ \gamma\ .\ Z$

◆ **Rankine Active & passive Earth pressure of soil :**

Case-1: Dry or Moist Backfill with no surcharge :



At Z = H

$P_a = K_a\ .\ \gamma\ .\ Z = K_a\ .\ \gamma\ .\ H$

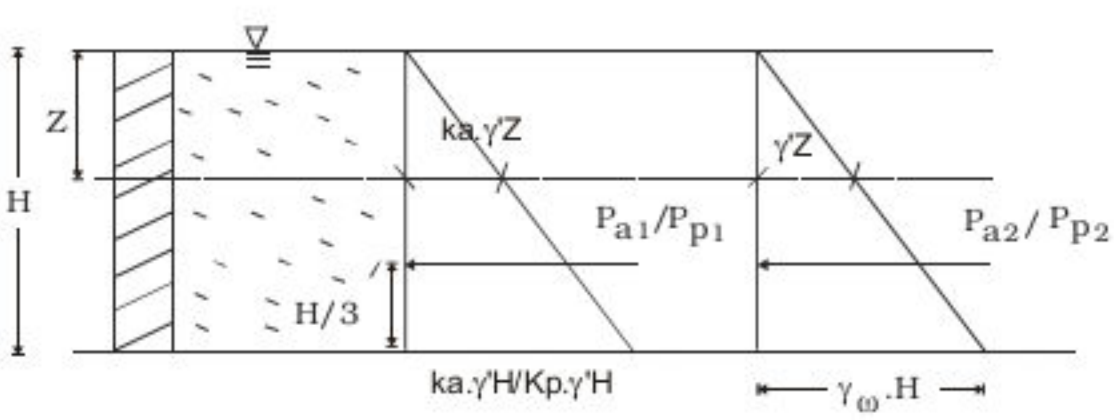
$p_p = K_p\ .\ \gamma\ .\ \cancel{Z} = K_a\ .\ \gamma\ .\ H$

Total earth pressure on unit length of wall :

$$P_a = \frac{K_a\ .H^2}{2}\ ;\ P_p = \frac{K_p\ .H^2}{2}$$

[Acting at $\frac{H}{3}$ from base of wall]

Case-2: Submerged Backfill



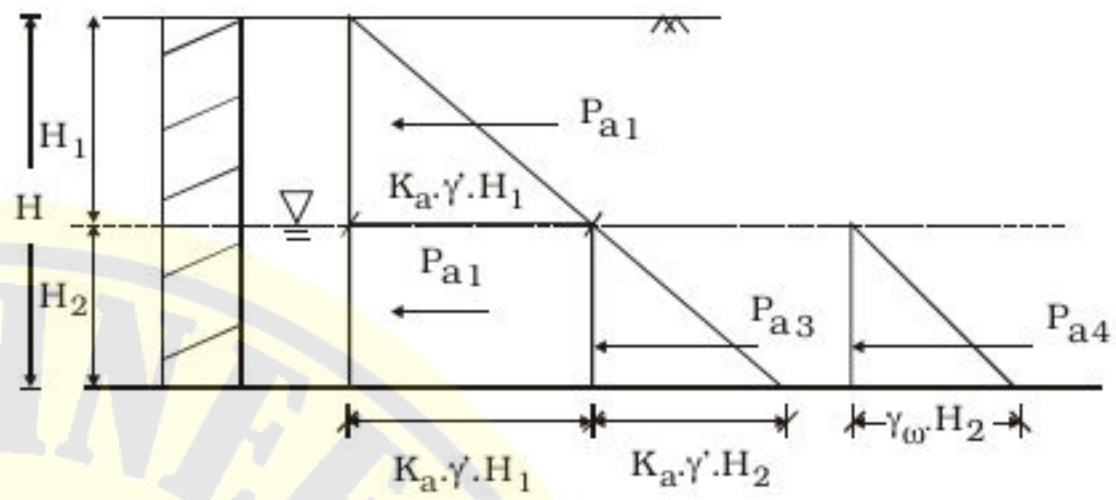
Total earth pressure on unit length of wall :-

$$P_a = P_{a1} + P_{a2} = K_a\ \gamma'\ .\ \frac{H^2}{2} + \frac{\omega H^2}{2}$$

$$P_p = P_{P1} \quad P_{P2} = K_p\ .\ \gamma'\ \frac{H^2}{2} + \frac{\omega H^2}{2}$$

[Acting at $\frac{H}{3}$ from base of the wall]

Case-3 : Water table is present at a depth H_1 from the ground level :



Total pressure is ;

$$P_a = P_{a1} + P_{a2} + P_{a3} + P_{a4}$$

$$P_a = K_a\ .\ \gamma\ \frac{H_1^2}{2} + K_a\ .\ \gamma\ .\ H_1\ .\ H_2 +$$

$$K_a\ .\ \gamma'\ .\ \frac{H_2^2}{2} + \gamma_\omega\ .\ \frac{H_2^2}{2}$$

Acting at ; $\overline{Z}$ [from base] ;

$$\overline{Z} = \frac{P_{a1}\ .\ Z_1 + P_{a2}\ .\ Z_2 + P_{a3}\ .\ Z_3 + P_{a4}\ .\ Z_4}{P_a}$$

Backfill consist of differents soils with different friction angle :

(a) $\phi_1 > \phi_2 \rightarrow K_{a1} < K_{a2} \rightarrow K_{P1} > K_{P2}$

$P_{a1} + P_{a2} + P_{a3} = P_a$

$$\therefore\ P_a = K_{a1}\ .\ \gamma\ .\ \frac{H_1^2}{2} + K_{a2}\ .\ \gamma\ .\ H_1\ .\ H_2 + K_{a2}\ .\ \gamma\ .\ \frac{H_2^2}{2}$$

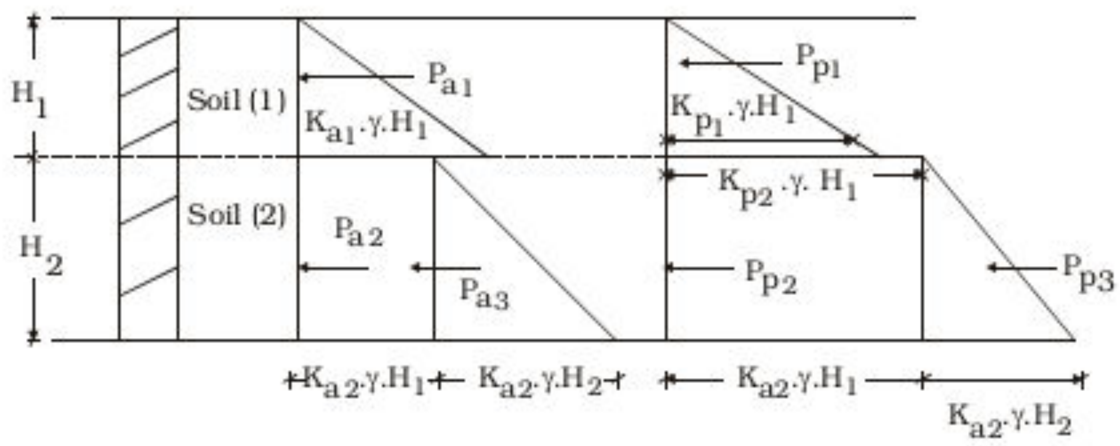
$P_{P1}\ /\ P_{a1}$ acts at $\left[H_2 \quad \frac{H_1}{3}\right]$ from the base

$P_{P2}\ /\ P_{a2}$ acts at $\left[\frac{H_2}{2}\right]$ from the base

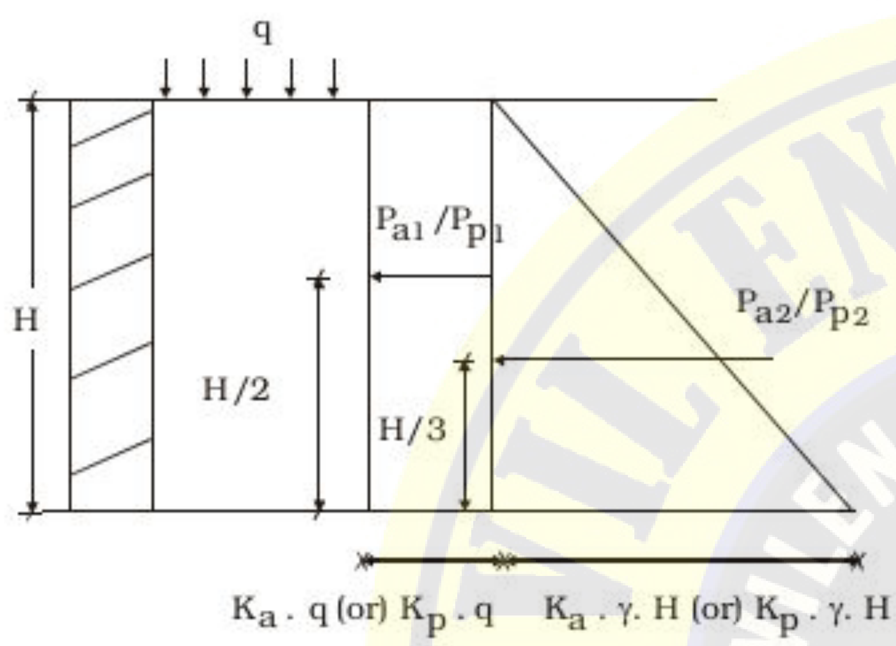
$P_{P3}\ /\ P_{a3}$ acts at $\left[\frac{H_2}{3}\right]$ from the base

$$P_p = K_{P1}\ .\ \gamma\ .\ \frac{H_1^2}{2} + K_{P2}\ .\ \gamma\ .\ H_1\ .\ H_2 + K_{P2}\ .\ \gamma\ .\ \frac{H_2^2}{2}$$

(b) $\phi_1 < \phi_2 \rightarrow K_{a1} > K_{a2} \rightarrow K_{P2} > K_{P1}$



Case–4 : Backfill with uniform surcharge :



$$P_a = P_{a1} + P_{a2} = K_a \cdot q \cdot H + K_a \cdot \gamma \cdot \frac{H}{2}$$

$$P_P = P_{P1} + P_{P2} = K_p \cdot q \cdot H + K_P \cdot \gamma \cdot \frac{H^2}{2}$$

$$\bar{Z} = \frac{P_{a1} \cdot \bar{Z}_1 + P_{a2} \cdot \bar{Z}_2}{P_a}$$

STRESSES IN SOIL

At any plane section of the soil, (total stress i.e. load per unit area) is comprised of following.

(i) Self weight of the soil

(ii) Overburden Pressure

The total stresses further consists of two distinct components.

a) Intergrannular pressure or effective stress (σ')

b) Pore water pressure or Neutral pressure (u)

Total stress (σ) = Effective stress (σ') + pore water pressure (u)

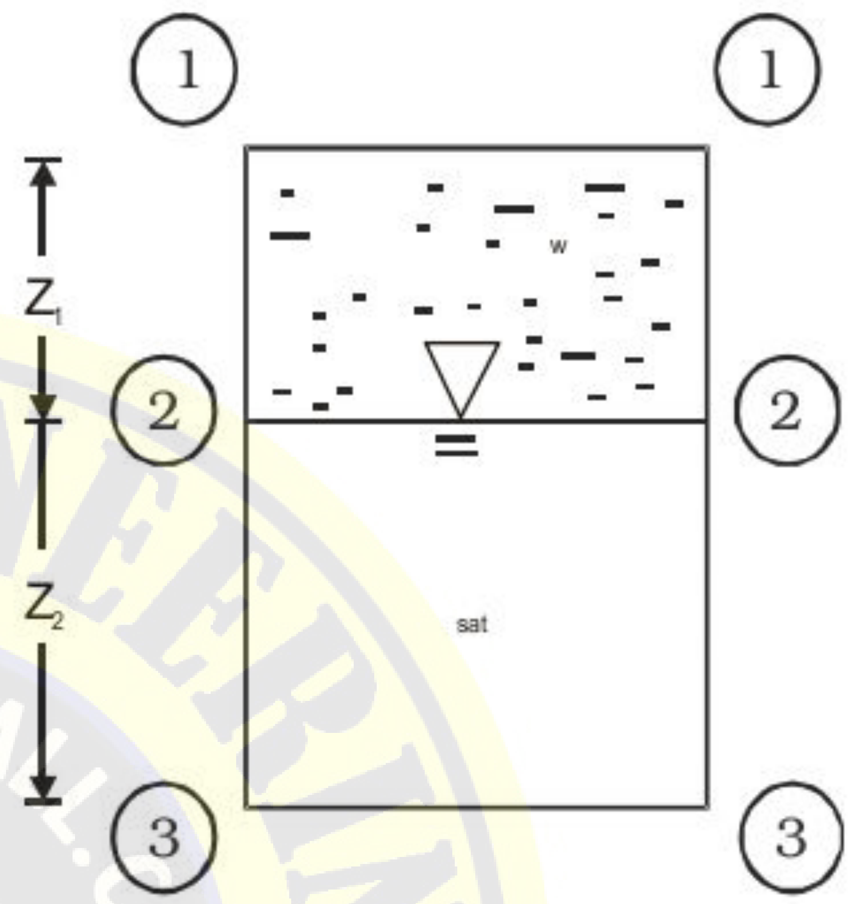
Effective stress/Intergrannular pressure is transmitted from particle to particle by contact hence it tends to force the particles to come in contact with each other resulting in the increase of their denseness and mobilisation of shear strength.

Pore water pressure is the pressure, transmitted through the pore fluid (water) and is equal to the weight of water per unit area above the section.

Since this pressure acts all around the particle hence does not tends to force the particle to come in contact with each other [It does not has any shear component] thereby, pore water pressure is also termed as neutral pressure.

EXAMPLES :

(i) Submerge soil mass :



At (1) - (1)

$\sigma = 0$

$u = 0$

$\sigma' = \sigma - u = 0$

At (2) - (2)

$\sigma = Z_1\,\gamma_w$

$u = Z_1\,\gamma_w$

$\sigma' = \sigma - u = 0$

At (3) - (3)

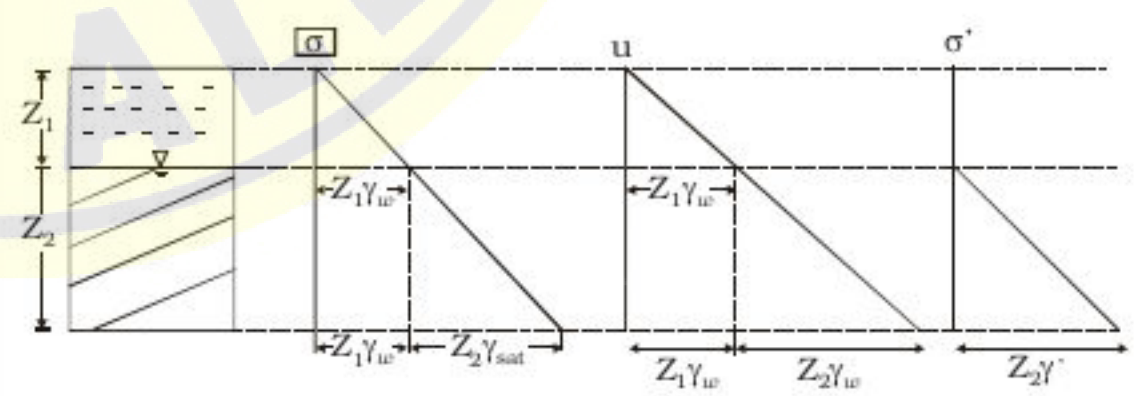
$\sigma = Z_1\,\gamma_w + Z_2\,\gamma_{sat}$

$u = (Z_1 + Z_2)\,\gamma_w$

$\therefore\ \sigma' = \sigma - u$

$= Z_2\,(\gamma_{sat} - \gamma_w)$

$\sigma' = Z_2\,\gamma'$



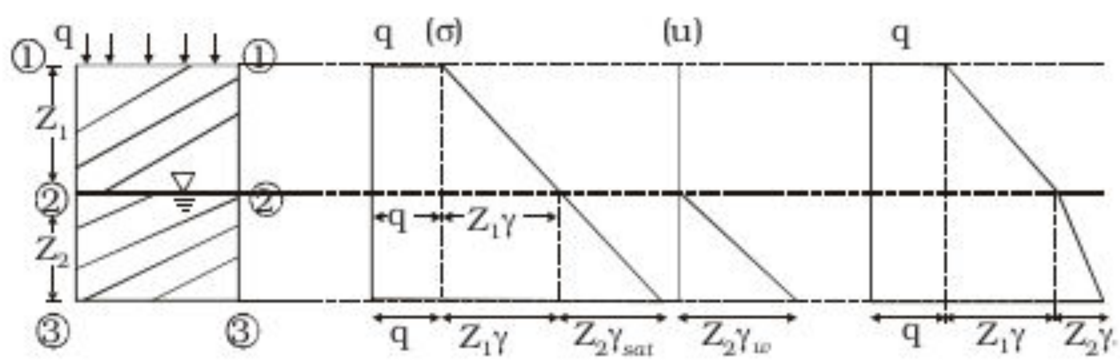
Slope of stress diagram is inversely proportional unit weight.

Note :

1. If weight is above the ground level as in the case of lake, with increase in the level of water table there is no' change in effective stress as total stress and pore water pressure change by same extent with change in level of weight.
2. It weight is below the GL, with change in weight level, effective stress also changes.

[If W.L. is increased, effective stress (σ') will decrease or vice- versa]

(ii) Soil mass with surcharge :



At (1) – (1)

$\sigma = q \Rightarrow \sigma' = \sigma' - u = q$

$u = 0$

At (2) – (2)

$\sigma = Z_1\,\gamma + q$

$u = 0$

$\sigma' = \sigma - u = Z_1\,\gamma + q$

At (3) – (3)

$\sigma = q + Z_1\,\gamma + Z_2\,\gamma_{sat}$

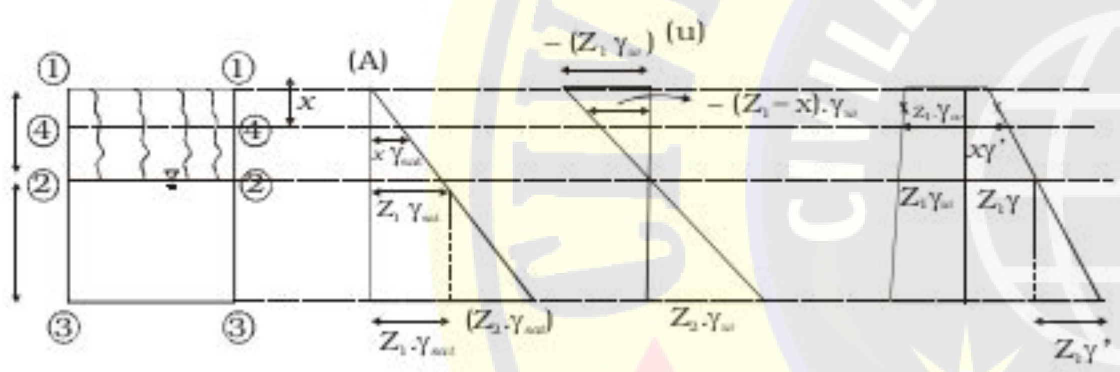
$u = Z_2\,\gamma_w$

$\sigma' = \sigma - u = q + Z_1\,\gamma + Z_2\,\gamma_{sat} - Z_2\,\gamma_w$

$= u + Z_1\gamma + Z_2\,(\gamma_{sat} - \gamma_w)$

$\therefore\ \sigma' = q + Z_1\gamma + Z_2\,\gamma'$

(iii) Soil mass with capillary fringe :



At (1) – (1)

$\sigma = q$

$u = -Z_1\,\gamma_w$

$\sigma' = \sigma - u = Z_1\,\gamma_w$

At (2) – (2)

$\sigma = Z_1\,\gamma_{sat}$

$u = 0$

$\sigma' = \sigma - u = Z_1\,\gamma_{sat}$

$\sigma' = Z_1\,(\gamma' + \gamma_w)$

$= Z_1\,\gamma_w + Z_1\,\gamma'$

At (3) – (3)

$\sigma' = (Z_1 + Z_2)\,\gamma_{sat}$

$u = Z_2\,\gamma_w$

$\sigma' = \sigma - u$

$= (Z_1 + Z_2)\,\gamma_{sat} - Z_2\,\gamma_w$

$\sigma' = Z_1\,\gamma_{sat} + Z_2\,\gamma' + Z_2\,\gamma_w - Z_2\,\gamma_w$

$\sigma' = Z_1\,\gamma_{sat} + Z_2\,\gamma'$

$\sigma' = Z_1\,.\,\gamma_w + (Z_1 + Z_2)\,\gamma'$

At (4) – (4)

$\sigma = x.\,\gamma_{sat}$

$u = -(\tau - x)\,\gamma_w$

$\sigma' = \sigma - u = x\,\gamma_{sat}\ (Z_1 + x)\,\gamma_w = (Z_1\gamma_w + x\,\gamma')$

Due to capillary rise effective stress of soil ↑↑ , it is beneficial (B.C ↑, shear strength ↑, compressibility ↓).

The increase in these parameter is equal to weight of the water/ unit weight in capillary.

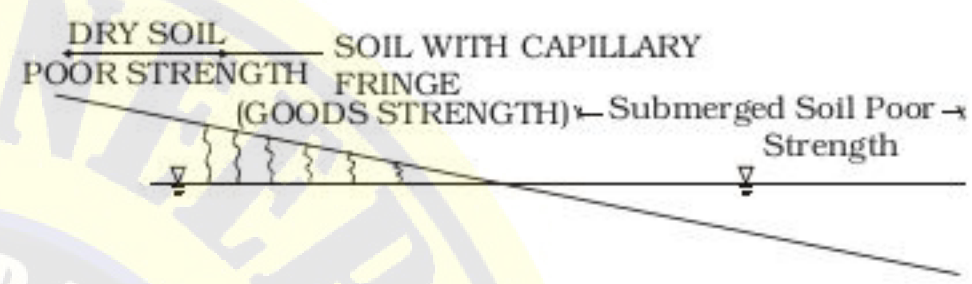
The negative sign is due to capillary tension. Capillary tension is maximum at the capillary height meniscus & decrease linearly & become zero at free water limit.

Note :

The effect of capillary is same as that of surcharge (surcharge = $h_c\,\gamma_w$) Which results an increase in effective stress in soil.

The increase in effective stress is equal to weight. per unit area of the water ($h_c.\gamma_w$) in capillary fringe.

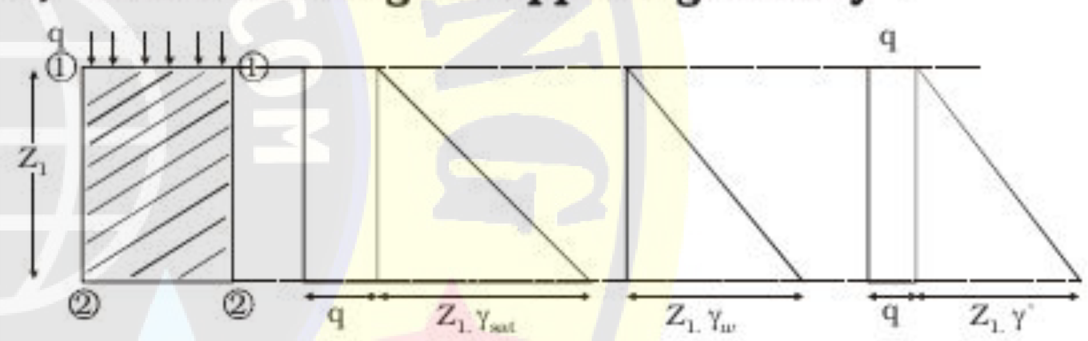
Most of the Engineering properties of soil (B.C, shear strength, consolidation, compressibility etc,are function of effective stresses.



(iv) Saturated Soil

WITH WATER TABLE AT GROUND LEVEL :

a) When surcharge is applied gradually :



At section (1) - (1)

$\sigma = q$

$u = 0$

$\sigma' = q$

At section (2) - (2)

$\sigma = q + Z_1.\,\gamma_{sat}$

$u = Z_1\,\gamma_w$

$\sigma' = \sigma - u = q + \gamma' \times Z_1$

b) When surcharge is applied immediately/Instantly

At sec (1) – (1)

$\sigma = q$

$u = q$

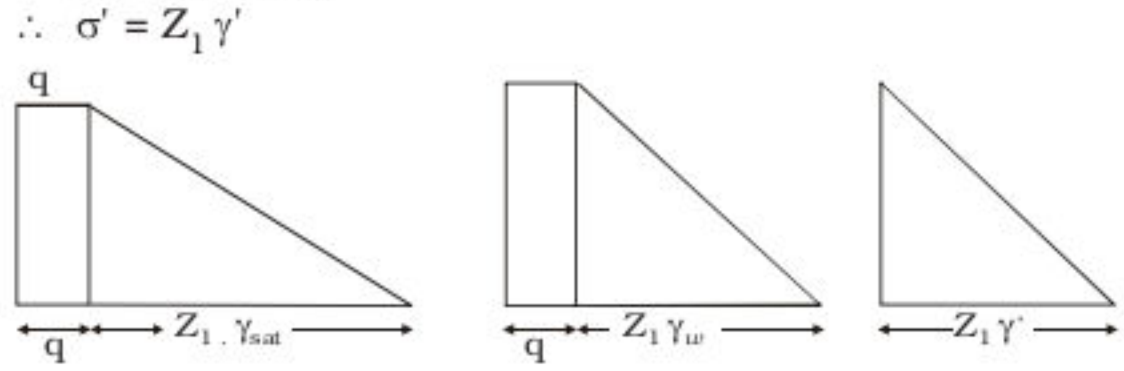
$\sigma' = 0$

At section (2) – (2)

$\sigma = q + Z_1\,.\,\gamma_{sat}$

$u = q + Z_1.\gamma_w$

$\therefore\ \sigma' = Z_1\,\gamma'$



Note :

If W.T at the G.L & surcharge (q) is applied gradually, it is being carried by the soil solid & transfered through grain to grain contact i.e. in this case, effective stress (σ') in soil increases.

But if (q) is applied instantly, it is being carried by the water present in the voids of soil i.e. sudden increase in pore water pressure will takes place i.e. in this case effective stress will remain unchanged.

Over a period of time, with the dessipation of pore water pressure due to the seepage of water through soil, effective stresses starts increasing as overburden pressure is being transferred by the water to the soil solids.

(In this case total stress remains same)

PERMEABILITY

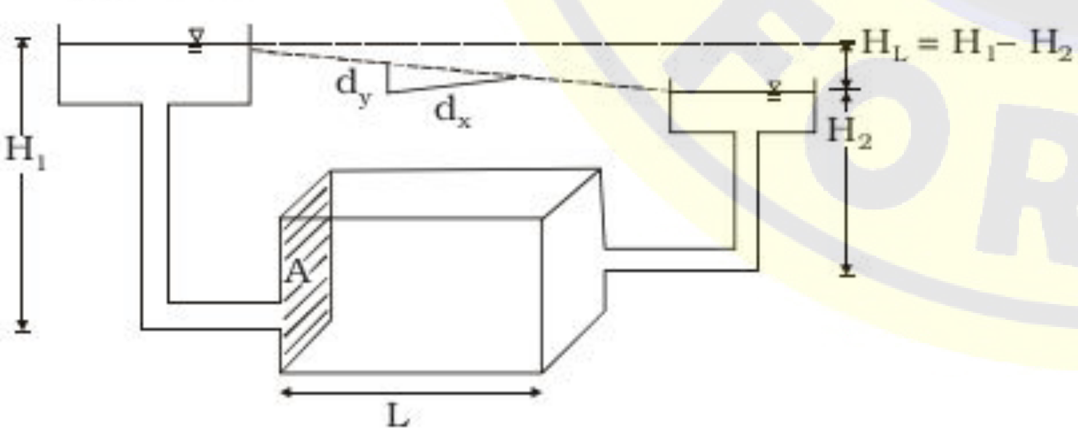
Permeability also termed as hydraulic conductivity is the ability of porous medium to permit the flow of fluid through its interconnected voids.

Generally permeability of coarse grained soil is more than permeability of fine grained soil.

TYPE OF SOIL	PERMEABILITY (K) (cm/sec)
Gravel	>1
Sand	$1–10^{-3}$
Silt	$10^{-3} – 10^{-7}$
Clay	$< 10^{-7}$

◆ **DARCY'S LAW :**

According to Darcy's law for laminar flow condition in saturated soil, mass velocity of flow through the medium is directly proportional to hydraulic gradient.



$V \propto i$

$V = Ki$

If $i = 1$; $V = K$

where ; $i = \dfrac{\text{Head loss of hydraulic head}}{\text{length of flow}}$

$$i = \frac{H_1 - H_2}{L} = \frac{H_L}{L} ;$$

$Q = AV$

$Q = KiA$

Permeability is also defined as ; Volume of flow through medium under unit hydraulic gradient.

For laminar flow in soil, Reynold's Number < 1 i.e size of particle should be less than 3 mm.

Flow in gravel is turbulent flow, hence Darcy's law is not valid in gravel in which particle size is > 3mm.

Velocity of flow considered above is average or discharge velocity as area of flow considered is total area of flow. But in actual flow takes place through the voids in between solids having area much less than total area of flow. Hence, true or seepage velocity is more than discharge velocity.

$$= \boxed{K \quad \frac{K_P}{\;}}$$

◆ **FACTORS AFFECTING PERMEABILITY :**

In general; K = f(medium properties, fluid properties)

$$K = d^2 . \left[\left(\frac{e^3}{1 \quad e}\right).\left(\frac{\;\;w}{\;}\right).(K_1).\left(\frac{1}{K_2}\right).\left(\frac{1}{K_3}\right).\left(\frac{1}{K_4}\right)\right]$$

Where, μ = Viscosity is friction so K $\propto \dfrac{1}{\;}$.

(1) $d, e, \gamma_w, \mu \rightarrow$ Clear as per above equation

(2) Degree of saturation (K_1) : Higher is the degree of saturation of the medium, higher is its permeability as because of the presence of air in voids, availability of area for fluid to flow reduces.

(3) Entrapped Gases (K_2) : Higher is the presence of entrapped gases in the medium, lower is the availability of area for fluid to flow. Hence, lower is the permeability.

(4) Entrapped Impurities (K_3) : Higher the concentration of entrapped impurities, higher is the assistance available to the flow (lower is the availability of area for fluid to flow). Hence, lower is the permeability.

(5) Absorbed water : Clay has more adsorbed water and also less K . Specific area of gravel is very less so 'K' value is also less.

Higher is the presence of absorbed water in the medium, lower is its permeability. It's merely because lower is the availability of area for pore water to flow through the medium as it is retained due to adhesion over soil solids.

(6) K also depends on type of cation absorbed over the mineral complex.

In clays, K increases in following order :

For montmorillonite ; K < Na < H < Ca

For Kaolinite Na < K < Ca < H

Na-montmorillonite has less permeability than Ca-montmorillonite.

Note : Na-clay : used to construct impervious core. The permeability of Na - clay is comparatively low. It is used for the construction of central impervious core in composite dams i.e. it is being treated with salt water during its construction.

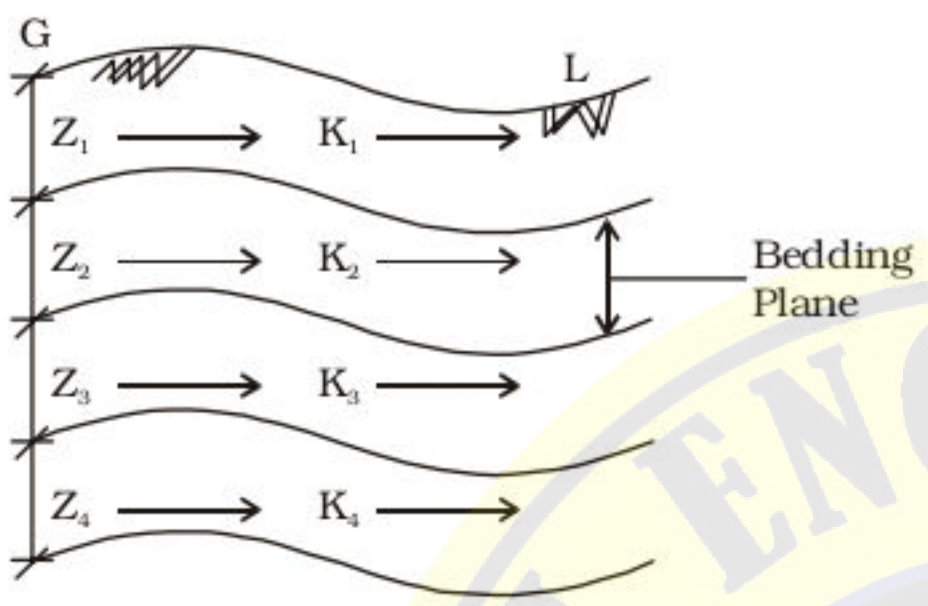
(7) Structure of Soil :

Effect of structural variation on permeability is more pronounced on fine grained soil.

For stratified soil 'K' parallel to the bedding planes is different from 'K' perpendicular the bedding planes.

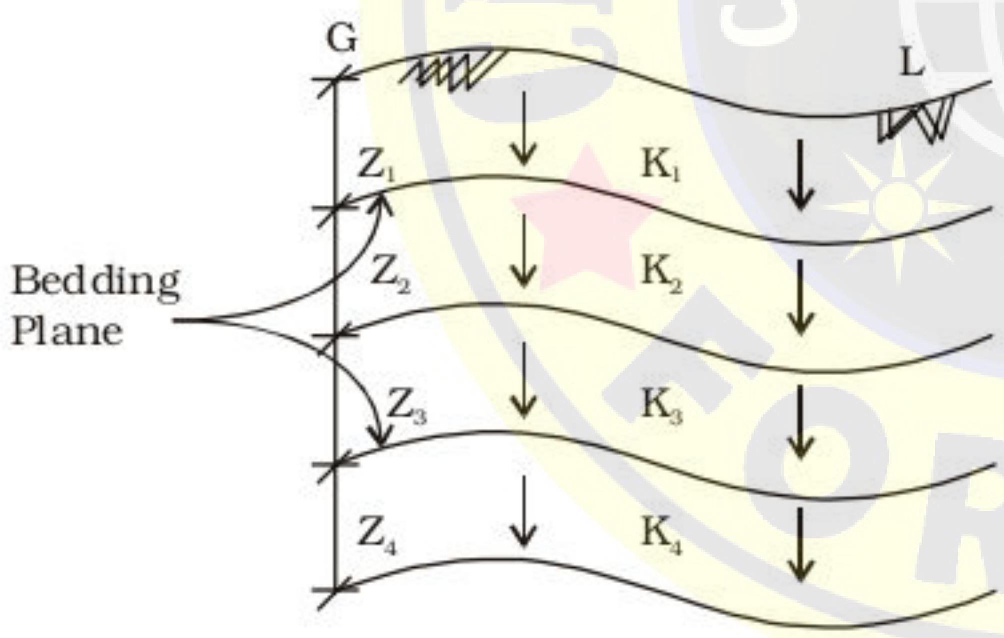
Permeability of Stratified Soils

(i) Equivalent permeability parallel to he bedding planes.



$$K_h = K_h \frac{K_1Z_1 \quad K_2Z_2 \quad ..... \quad K_nZ_n}{Z_1 \quad Z_2 \quad Z_3 \quad .... \quad Z_n}$$

(ii) Equivalent permeability perpendicular to the bedding planes.



$$K_v \quad \frac{Z}{\frac{Z_1}{K_1} \quad \frac{Z_2}{K_2} \quad .... \quad \frac{Z_n}{K_n}}$$

- In al ayered soil, the average coefficient of permeability in the horizontal direction is greater than average coefficient of permeability in vertical direction.

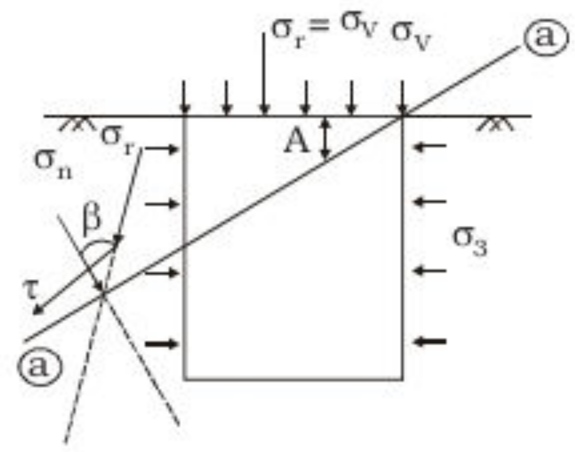
SHEAR STRENGTH OF SOIL

Definition :

Shear strength of a soil is resistance offered by a soil against shear deformation. It is equal to shear stress developed on critical plane or failure plane.

Critical plane is that plane at which angle of resultant stress with the normal of that plane is maximum.

Such a plane is also called, plane of maximum obliquity.



If $\beta = \beta_{max}$

$\theta = \theta_C$

Then, plane (a) – (a) is critical/failure plane

θ_C = angle of critical plane with plane of (σ_1)

Shear strength may or may not be the 'maximum shear stress.

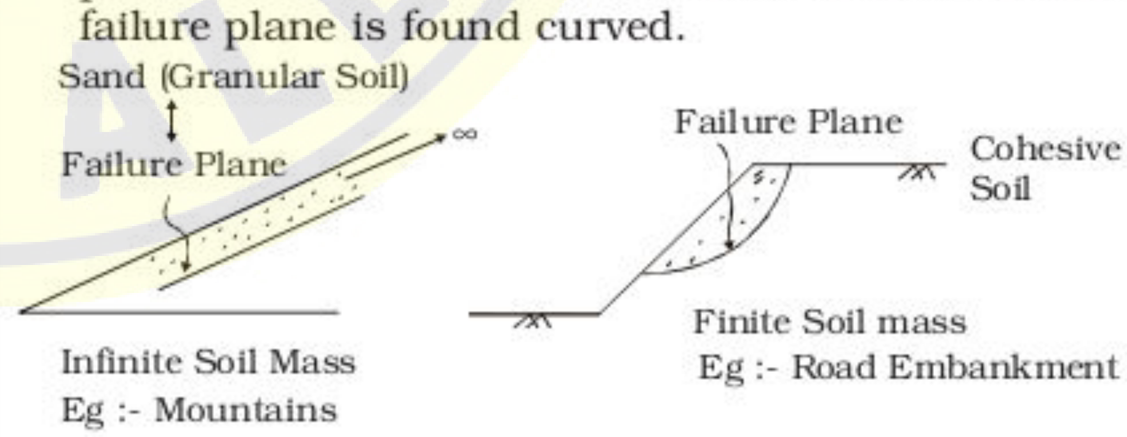
Factors determining shear strength of soil

i) Interlocking between molecules (ii) frictional resistance [Rolling and sliding friction] (iii) Attraction between molecules [cohesion & adhesion].

Stress: Stress is an internal force acting per unit area of a surface. It is a vector quantity. It has normally, two components one, that acts normal to the sectional plane, and other that acts along the plane. The Normal Component which acts along the plane is called tangential stress or shearing stress.

Note : Granular soils [Sands and Gravel] derive their strength from interlocking and friction whereas fine cohesive soils [clays] derive their strength from cohesion and adhesion, however medium grain soil [silt] derive their strength both from friction & cohesion.

The shear failure plane in granular soil [Sand] is usually linear whereas in cohesive soil is curved [arc of circle]. In case of infinite soil mass failure plane is found linear but in case of finite soil mass, failure plane is found curved.



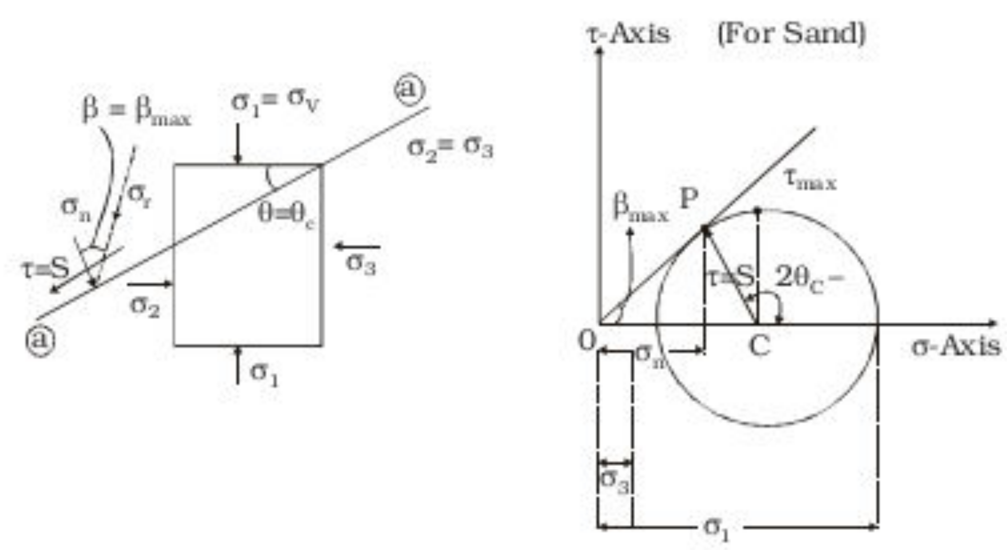


Fig :-1

$$OP = \sqrt{\sigma_n^2 + \tau^2} = \sigma_r$$

$$\tan\beta = \frac{\quad}{m}$$

$$\tan\beta_{max} = \left(\frac{\quad}{n}\right)_{max}$$

Shear strength of a soil is the shear stress developed in critical plane. On critical plane, angle of resultant stress (β) is most inclined (β_{max}) therefore failure envelop can be found by drawing a tangent to the mohr circle as shown in figure 1.

Notice that shear strength ($S = \tau \leq \tau_{max}$).

Let θ_c is the angle of critical plane from plane of σ_1 (major principal stress/vertical stress).

$2\theta_c = 90° + \beta_{max}$ [By property of triangle]

$$\therefore \qquad \theta_c = 45° + \frac{max}{2}$$

The value of β_{max} can be equal to frictional angle of soil (ϕ).

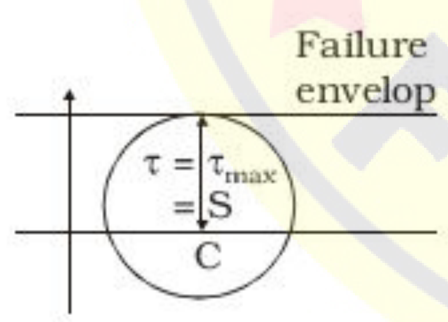
$$\therefore \beta_{max} \cong \phi \Rightarrow \therefore \theta_c = 45° + \frac{\quad}{2}$$

The normal stress on the critical plane [σ_n] is given by; $\sigma_n = \sigma_1 (1 - \sin\beta_{max})$

or, $\sigma_n = \sigma_3 (1 + \sin\beta_{max})$

$$\therefore \frac{1}{3} = \frac{(1 \quad \sin \quad max)}{1 - \sin \quad max} \text{ or, } \sin\beta_{max} = \frac{1 - 3}{1 \quad 3}$$

In case of sand, failure does not occur on the plane of τ_{max}, but in case of clays τ_{max} plane and failure planes are same.



The angle of failure plane, with the plane of σ_1 [major principal plane (θ_C)] is given by;

$$\theta_c = 45° + \frac{max}{2}$$

$$\theta_c = 45° + \frac{\quad}{2}$$

The relation between σ_1 and σ_3 is given as ;

$\sigma_1 = \sigma_3 \tan^2\theta_c + 2c\tan\theta_c$

where, also

$$\sigma_1 = \sigma_3 \tan^2\left(45 \quad \frac{\quad}{2}\right) + 2C\tan\left(45 \quad \frac{\quad}{2}\right)$$

where,

ϕ = friction angle of soil

C = cohesion of soil (kN/m²).

σ_1 and σ_3 = major and minor principal stress (kN/m²)

Note :

For purely cohesive soil [clay] → ϕ = 0

For purely frictional soil [sand] → C = 0 and [gravel]

◆ **Mohr–coloumb theory of shear strength :**

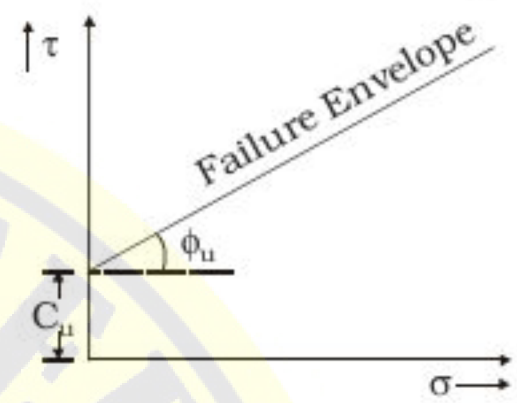
(a) Theory of undrained condition : [without considering dissipation of Water table effect]

Undrained shear strength =

Where, $(S_u) = Cu + \sigma_n . \tan\phi_u$

C_u = undrained cohesion (total cohesion)

ϕ_u = undrained/total frictional angle



σ_u = total normal stress on failure plane

[It means under undrained condition, total stress parameters are used].

Note :

The above results are found erroneous under drained condition in which pore pressure dissipates during loading.

(b) [Modified Mohr coloumb theory] :

$S = C' + \bar{\sigma}_n \tan\phi'$ where ;

OR

C' or C_d = effective or drained cohesion

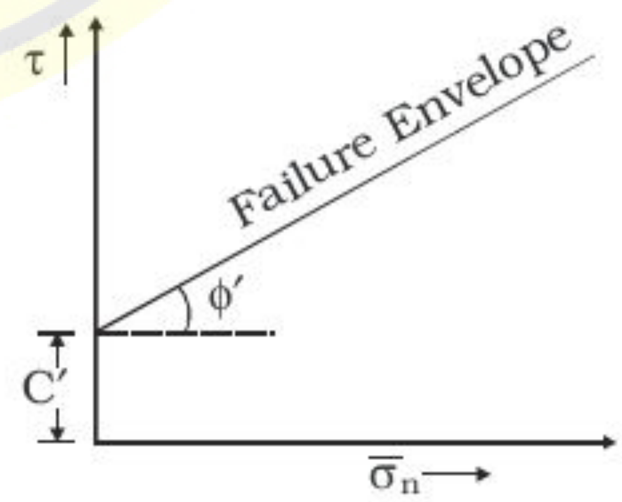
$S_d = C_d + \bar{\sigma}_n \tan\phi_d$

$\bar{\sigma}_n$ = effective normal stress on the critical plane

$\therefore \bar{\sigma}_n = \sigma_n - U$

ϕ' or ϕ_d

effective or drained friction angle



Note :

1) If there is no water table (u = 0) then $\bar{\sigma}_n = \sigma_n$;

$C' = C = Cu$; $\phi' = \phi = \phi_u$

Then drained and undrained result will be same. However if water table is present, then computations should be made using effective parameter under drained condition and total parameter under undrained condition.

2) The shear strength, consolidation settlement bearing capacity etc. are function of effective parameters. Therefore, modified Mohr-coloumb theory is more realistic.

◆ **Limitation of Mohr-Coloumb theory :**

1. The effect of major principal stress (σ_1) and minor principal stress (σ_3) is considered and analysis is made ignoring intermediate principal stress ($\sigma_2 = \sigma_3$), because in above theory, two dimensional analysis is done in which σ_2 is ignored.
2. Mohr failure envelop is approximated to a straight line which is not perfectly true in some of the case [for eg : Over consolidated soil]

◆ **TYPE OF SHEAR STRENGTH TEST :**

Type of test on the basis of drainage condition :

The choice of test depends upon the type of soil & the purpose of test for which it is required.

It also depends upon drainage facilities available at the fields.

1) Unconsolidated–Undrained test [U–U test] :

It is a quick test which takes 5 to 7 minutes. In this test, neither flow of water from the soil is permitted during cell pressure stage all round pressure and nor during back pressure stage (only 1 direction (vertical) loading). The stage is called [deviator stage]. Such a shear strength will be called undrained shear strength.

This test is suitable for saturated clays having low permeability subjected to fast rate of loading.

2) Consolidated-Undrained test [C-U test] :

In this test drainage is permitted in 1st stage [cell pressure stage], whereas during back pressure stage/ shear stage the drainage is not permitted.

It means the 2nd stage is undrained. This test is usually used for, the investigation of stability of earthern test, against the failure which may occur due to sudden drawdown. It is also called consolidated Quick Test.

3) Consolidated-drained test [C-D test] :

It is most time taken, because drainage is permitted during cell pressure stage and back pressure stage both.

4-6 weeks are required to complete the test. So, it is also called "Slow test".

In this test considerable volume change is recorded due to expulsion of pore water.

This test is suitable for saturated sands for investigation of stability under long term condition.

Tests to determine shear strength of soil :

(1) Field tests :

(a) Vane shear test.

(b) Standard penetration test.

(c) Cone penetration test.

(2) Lab test

(a) Direct shear test.

(Suitable for sands).

(b) Triaxial shear test.

(Suitable for all type of soils).

(c) Unconfined compression test.

(Suitable for saturated clay).

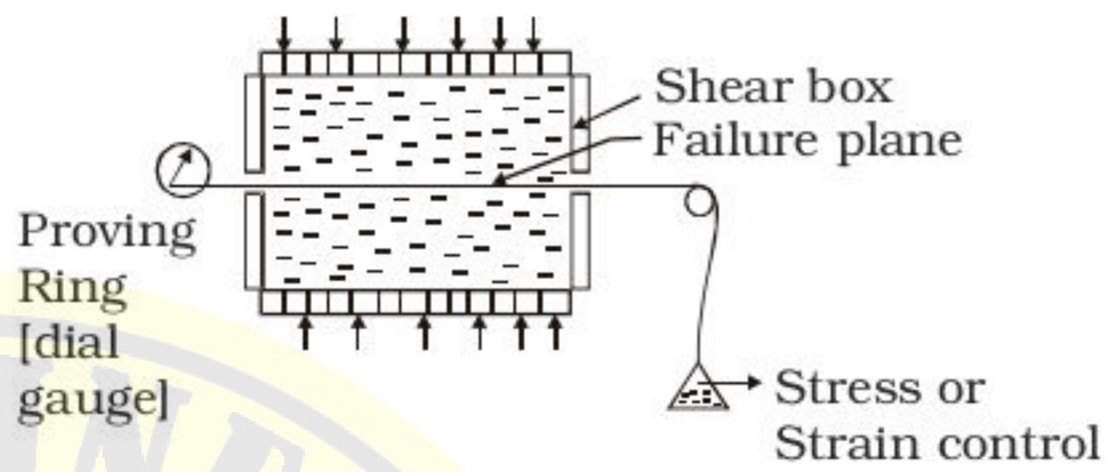
(d) Vane shear test.

(Suitable for soft saturated clay under undrained condition).

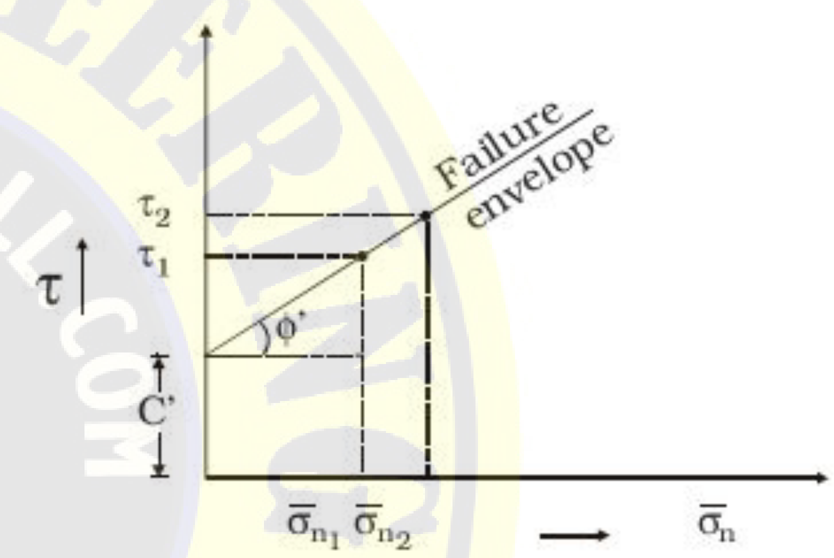
(e) Ring shear test.

(f) Torsion shear test.

Direct shear test :



[Permeable plates both at top & bottom]



Note :

1) Since, there is no mechanism to measure the pore water pressure therefore, this test is conducted under drained condition.

This makes it suitable for sandy soils.

2) Let, normal forces applied in each test are P_1, P_2 ....

P_n, then normal stresses will be $\bar{\ }_{n_1}\ \frac{P_1}{a}$,

$\bar{\ }_{n_2}\ \frac{P_2}{A}$, $\bar{\ }_{n_3}\ \frac{P_3}{A}$

Where,

A = Area of shear box =

area of soil sample $\begin{cases} \frac{\pi}{4} D^2 & \text{[linear box]} \\ B^2 & \text{[square box]} \end{cases}$

3. Let, N = proving ring dial gauge ring at failure

K = proving ring constant

Therefore, shear force at the failure is

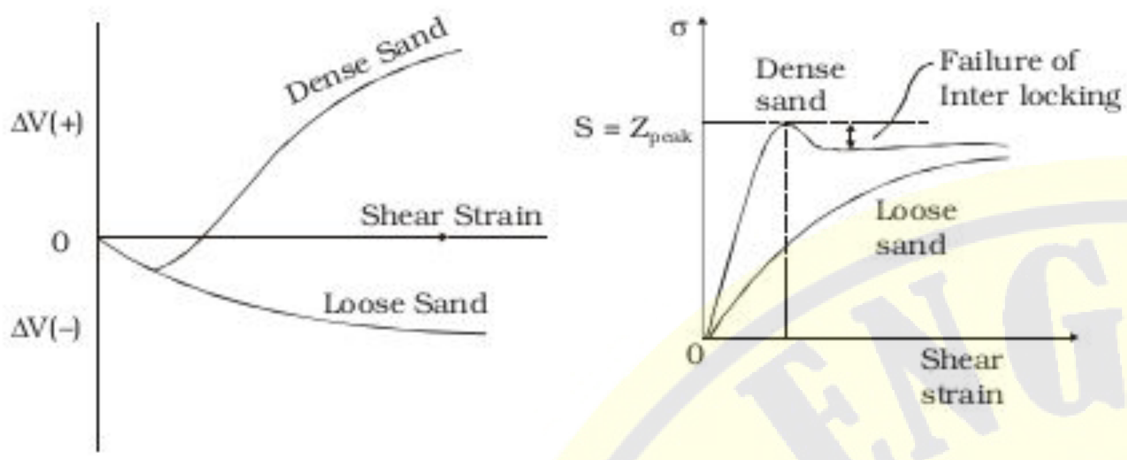
→ S.F = K.N

$\therefore\ \tau = \frac{S.F}{A} \Rightarrow \tau\ \frac{K.N}{A}$

Limitations :

1. Failure plane is predetermined which may not be the weakest plane.
2. Shear stress distribution on failure is assumed uniformed which may not be uniform.
3. The stress condition are known on only failure. Therefore, it is difficult to draw Mohr's circle.
4. There is no mechanism known to measure pore.
5. There is no control on drainage condition.

◆ **Graph between volume change & shear strain :**



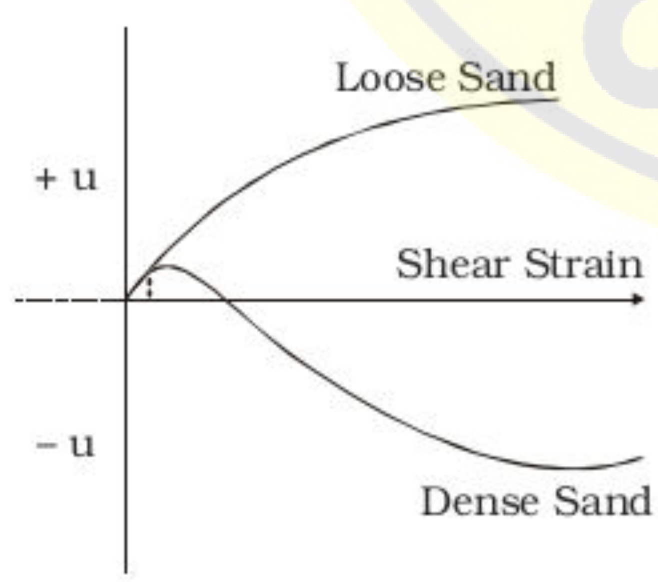
In dense sand, interlocking fails at 4-6% of strain due to which sudden loss in shear strength is recorded to be of the order of 20-25%. Before failure of interlocking, the volume of dense sand decreases but after failure of interlocking, volume increase.

Whereas in loose sand, volume continuous to decrease and large stain are recorded.

In loose sand, no clear failure point is recorded. Therefore, failure is assumed to occurs when shear strength is of the order 15-20%.

In dense sand, shear strength is due to both 'interlocking' and 'friction' whereas in loose sand Interlocking is negligible.

Pore pressure change V/S shear strain curve :



In loose saturated fine sands and silts due to seismic disturbance large settlement may occur and high pore pressure are set up, as a result sudden loss in shear strength is recorded.

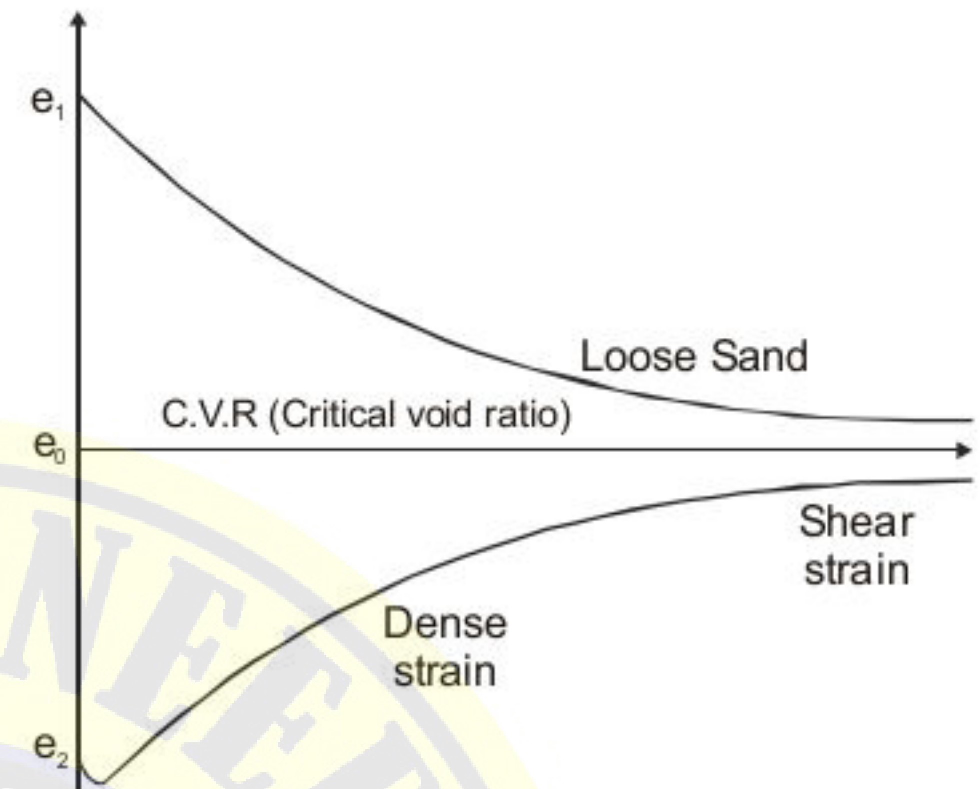
$S = C' + \bar{\sigma} \tan \phi'$

where, $\bar{\sigma} = \sigma - u$

Such a phenomena of loss of shear strength in loose saturated sand is called liquifaction.

In dense saturated sand, due to shear disturbance, negative pore pressure is set up due to which effective stress increases and shear strength increases. Therefore, the problem liquifaction process does not occur.

◆ **Graph between Voids Ratios and Shear Strain :**



e_o (Critical voids ratio is that void ratio of sand, at which there will no volume change or minimum volume change on shear disturbance).

If $e > e_o$ loose soil

If $e < e_o$ dense soil.

Mode of application of shear force

(i) Strain Controlled Tests : The rate of shearing strain is controlled manually and the shear force acting on the specimen is measured indirectly using proving ring.

(ii) Stress–Controlled test : Shear force is increased at given rate and shear displacements are obtained by means of a dial.

b) Triaxial test :

Most widely adopted and more accurate among all.

It can be adopted for both drained & undrained condition and for all types of soil.

All three test CD, Cu and *uu* can be conduction.

This test is conducted in 2 stages.

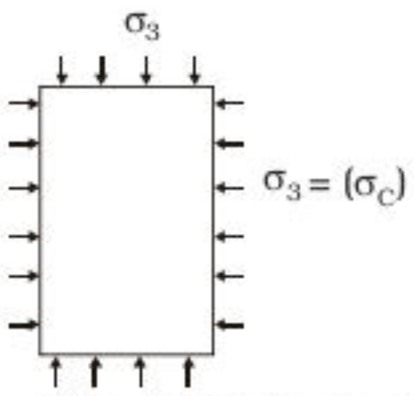
i) Cell pressure stage or consolidation stage.

ii) Back pressure stage or Deviator stage or shear stage.

i) CELL PRESSURE STAGE (Or) Confining pressure stage : Soil sample is prepared from a saturated soil mass having cylindrical shape [length of 2 times its diameter].

The sample is encased inside a impermeable rubber membrane

In 1st stage all round cell pressure is applied by a confining pressure $\sigma_c = \sigma_3$ which acts in all direction.



[Triaxial State of σ_3]

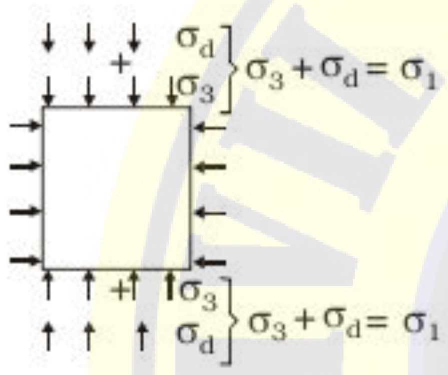
If test is undrained, then drainage value is kept closed and flow of water is not permitted. Whereas, test if is consolidated (drained) then drainage values varies and flow of water is permitted.

In consolidated test, if expulsion of pore water is stopped, under cell pressure (σ_c/σ_3) then consolidation is complete.

ii) BACK PRESSURE STAGE :

In 2nd stage, confining pressure is kept constant but axial stress/Deviator stress is increased by σ_d. Therefore, total axial stress becomes.

$(\sigma_d + \sigma_3)$



σ_d = deviator stress increase in Axial direction.

The deviator stress increases until soil fail in shear in certain plane.

The deviator stress are failure because confined compressive strength depends upon confining pressure (σ_3).

$(\sigma_d)_f = (\sigma_1 - \sigma_3)_f$ = confined compressive strength.

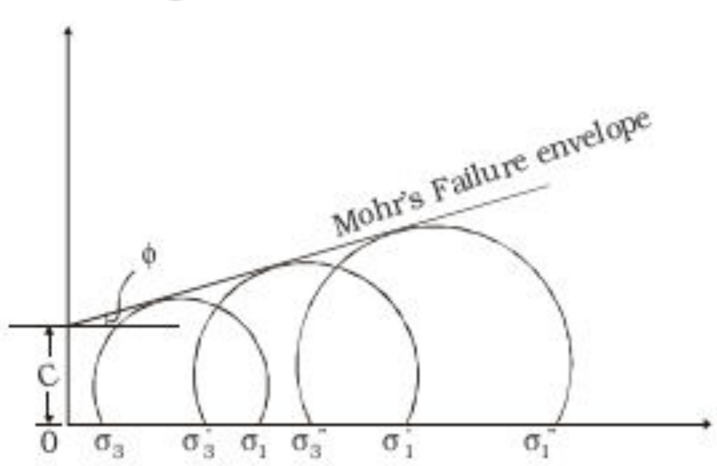
If P is increase in axial force which cause failure

then $\sigma_d = \left(\dfrac{P}{A_f}\right)$

where, A_f = area of soil at failure of cross-section

In 2nd stage, the test may be under drained or undrained condition, accordingly values will be kept open or close.

To find the shear parameters (C & ϕ) test is repeated for different values of σ_3 & σ_1. For every test, a Mohr circle is plotted.



Let in 1st specimen, cell pressure is σ_3 and total axial stress at failure = σ_1

In 2_{nd} specimen = cell pressure = $_3$ and total Axial stress at failure = $_1$

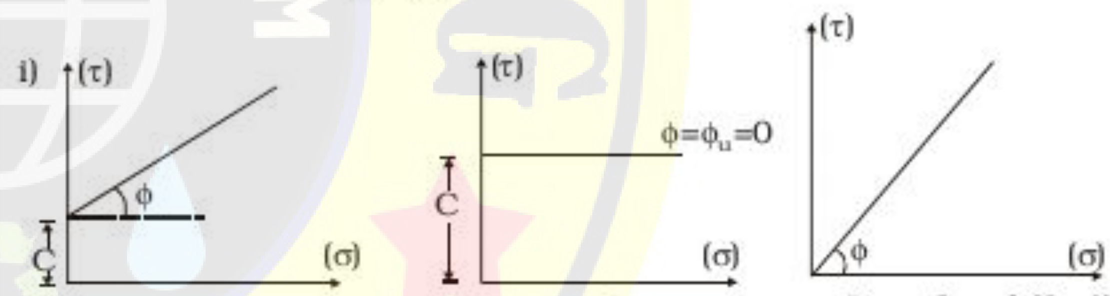
Mohr circles are drawn for each specimens of same soil and a common tangent is drawn to all mohr Circles which is called Mohr's failure envelope.

From Mohr's failure envelop, C and ϕ can be obtained.

◆ **ADVANTAGES OF TRIAXIAL TEST :**

1. Failure plane is not pre-determined and it is the weakest plane.
2. Stress distribution on failure planes are much more uniform than direct shear stress.
3. There is complete control on drainage condition. Therefore, test can be conducted under, both drained or undrained condition.
4) There is mechanisms to measure pore pressure hence using undrained test, drained shear strength can be computed.
5) It is suitable for all type of soil and can be conducted under all the drainage condition.
6) Most accurate and most versatile.

★ Depending on type of soil, the failure envelops can be of following types ;

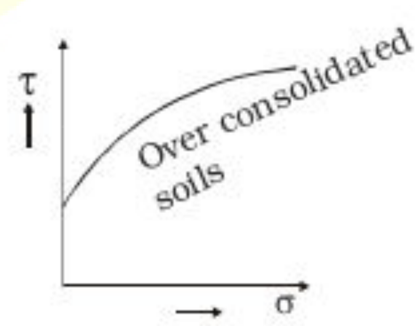


For C – ϕ Soil

Pure cohesive soils [under undrained condition]

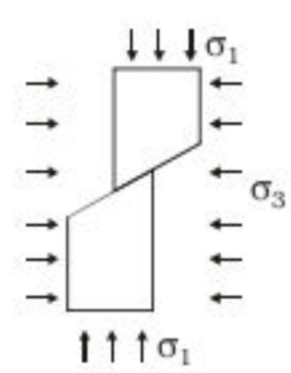
Pure Sand (C=0) or [cohesive soils under undrained condition]

Cohesive soils under drained condition behaves like sand.

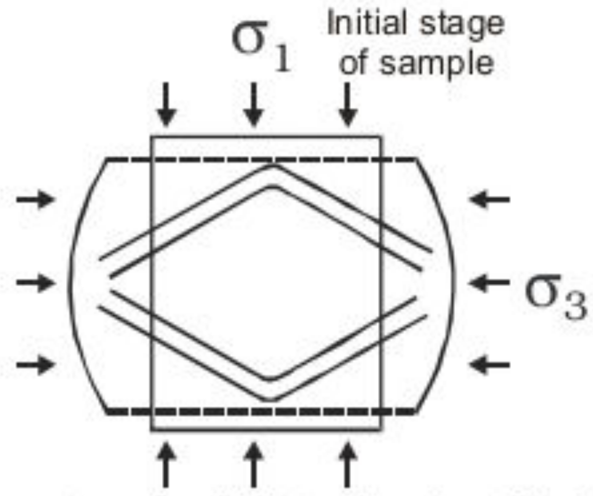


Type of failure during triaxial test :

1. **Brittle failure :** It is the case of dense sand and over consolidated clays in which during shear test, a clear crack is seen on the inclined planed.



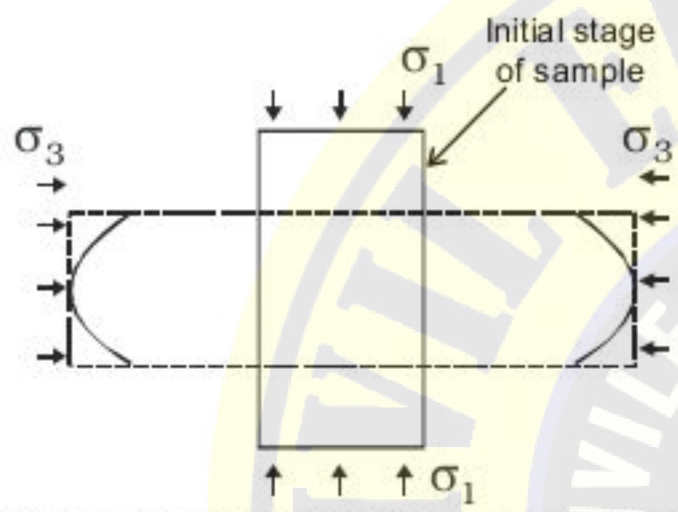
2. Semi brittle or semi plastic failure :



This failure is recorded in C - ϕ soil [silt], such soil represents considerable axial strain and lateral bulging.

3. Plastic failure :

This failure is recorded in soft clay and loose sand. In such soil large axial strain are recorded and failure plane is placed perpendicular to axis of loading.



If triaxial is under undrained condition, then volume change in soil mass is negligible, but if test is under drained condition then significant volume change occur due to expulsion of pore water, the expelled pore water is stored, Hence, (ΔV) volume change in soil can be found.

$\therefore \in_V = \frac{V}{V_o}$; V_o = initial volume before loading of the soil specimen.

$$V_0 = \frac{\ }{4} D^2$$

The area of soil mass change will the loading and area at shear failure is given by

$$A_f = \frac{A_O(1 \quad)}{1- \quad_L} \quad \text{[for drained test]}$$

where,

$\in_L$ = axial strain or longitudinal strain at failure

Ao = initial area before loading

If test is undrained then $\in_V$ is negligible ($\in_V \simeq 0$)

then $A_f = \frac{A_O}{1- \quad_L}$ (for undrained test)

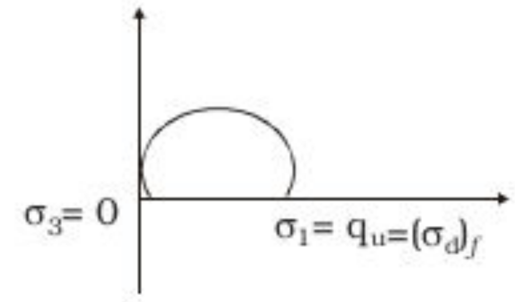
◆ **Unconfined Compression test :**

It is an special case of triaxial test in which confining pressure is zero ($\sigma_3 = 0$). No rubber member is required, because cell pressure is not applied.

The load applied is only axial, hence, axial stress and deviator stress are same.

The deviator stress at failure is called, unconfined compressive strength.

This soils are applicable for saturated clays and silts and can not be conducted in dry and sandy soils.



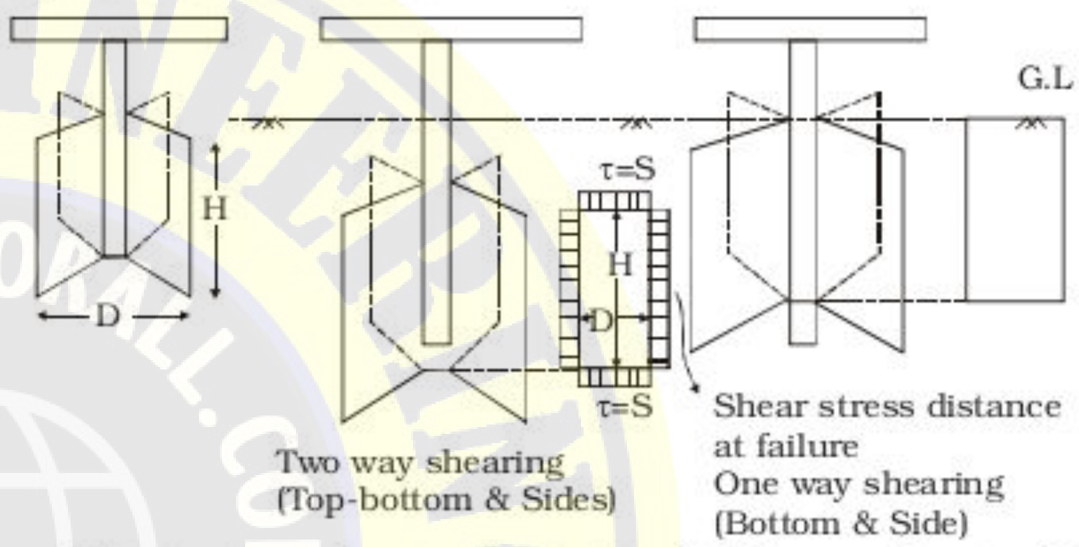
Note : In this test, there is a unique Mohr circle which passes through origin for the soil.

For pure clay ; $\phi = 0$

$\therefore \quad q_u = 2C \quad \therefore \quad C = \frac{q_u}{2}$ [for sandy soil]

$q_u = 0$

d) Vane shear test :



This test can be conducted in field as well as in lab.

The size of vanes are different used in field & lab.

This test is essentially undrained and there is no mechanism to measure pore pressure therefore it is suitable to determine undrained samples.

Undrained shear strength of soft saturated clays and highly plastic clays.

Lab Vane size	**Field Vane size**
H = 20 mm	H = 10 –20 cm
D = 12 mm	D = 5 to 10 cm
Vane thickness	Vane thickness
= 0.5 to 1 mm	= 2 to 3 mm

There are two types of shearing :

i) **2-way shearing :** When vane is punched fully inside the soil mass.

The top surface of vane is at certain depth below the surface of soil.

In this case shearing will takes place at the sides and at top and bottom both.

The shear strength of soil in Two-Way shearing is given by ;

$$S = \frac{T}{D^6\left(\frac{H}{2} \quad \frac{D}{6}\right)} \qquad \text{Where, } T = K\,.\,\theta$$

ii) **One-way shearing :** When vane is punched into the soil such that top surface of vane is in contact of air, i.e., at the surface of soil mass. In this case, shearing will takes at sides and bottom.

The shear strength of the soil is given by ;

$$S = \frac{T}{D^2\left(\frac{H}{2} \quad \frac{D}{12}\right)}$$

For the case of pure clays, S = C ; $C = \frac{q_u}{2}$

$\therefore q_u = 2C = 2\ S$

This test can be used to find sensitivity of clay.

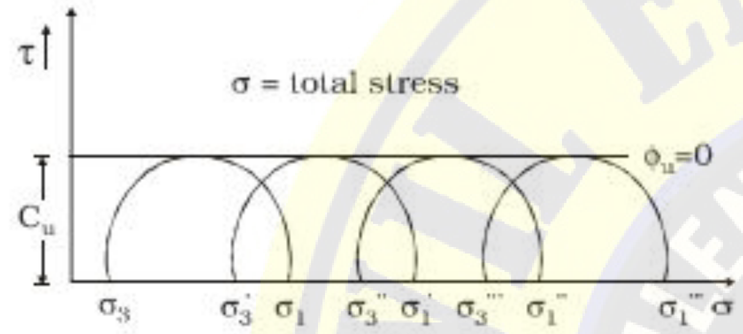
$$\therefore S_t = \frac{\text{UCS of undisturbed soil}}{\text{UCS of remoulded soil}} = \frac{(q_u \text{ undisturbed})}{(q_u)\text{ remoulded}}$$

$$= \frac{(2S)\text{ undisturbed}}{(2S)\text{ remoulded}} = \frac{(S)\text{ undisturbed}}{(S)\text{ remoulded}} = S_t$$

◆ **Shear characteristic of clays :**

1. **Saturated clay under undrained condition :**

If U-U test is performed on a saturated clay, then following results are found by triaxial test.



In this case, each Mohr circle has same radius, therefore there is unique Mohr circle.

Under undrained condition. Normally consolidated Clay and over consolidated clay have similar behaviour, but unit cohesion is different

For normally consolidated clay ;

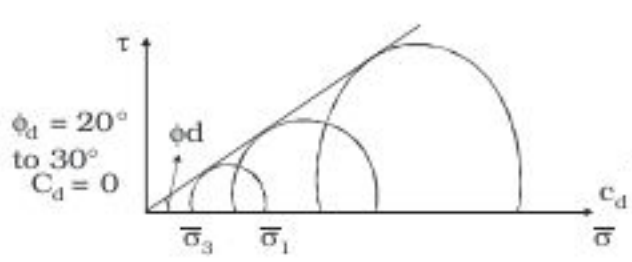
C_u = 25 to 30 kN/m^2 ; $\phi_u = 0$

For over consolidated clay ;

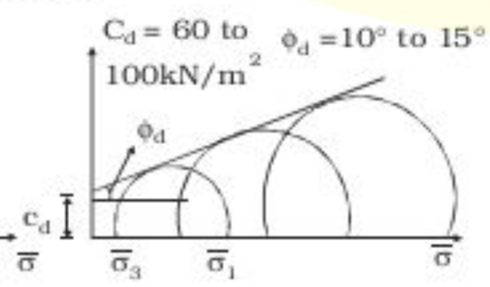
C_u = 100 to 200 kN/m^2 ; $\phi_u = 0$

2. **Saturated clays under drained condition :**

If C-D test is conducted on a saturated clay then results of Normally Consolidated Clay are similar to that of sand and results of Over Consolidated Clay are similar to that of silt.



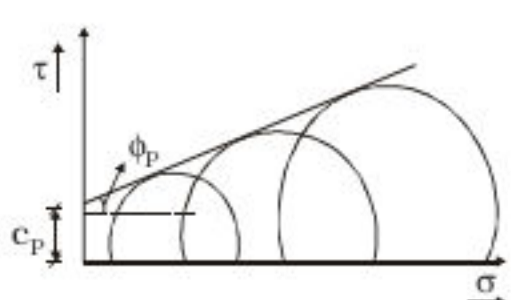
For NCC



For OCC

3) **Partial saturated clays :**

Under drained and undrained condition both the results are similar to that of silts.



Under Undrained condition

ϕ_p = 0° to 50° cm

C_p = 80 to 150 kN/m^2

Under drained condition

ϕ_p = 28° to 30°

C_p = 60 to 70 kN/m^2

Note: In loose saturated sand on disturbance soil molecules comes closer & volume decreases, hence possitive pore pressure is set up as a result, effective stress reduces and sudden loss in shear strength is obtained, which results in failure. Such phenomenon is called liquification of sand.

COMPACTION OF SOIL

Compaction is the process in which soil particles are artificially rearranged and packed in closer state of contact by mechanical means, in order to reduce its void ratio permeability, compressibility and increase its denseness, stability and bearing compaction.

[In order to modify its Engineering properties].

Compaction can be brought about either by rolling, vibrating, ramming or tampering.

Compaction is different from consolidation as it is instantenous process of volume reduction of soil that is carried out mechanically in unsaturated soils by decreasing the volume of air voids, whereas consolidation is a gradual natural process in which volume reduction of saturated soil mass is brought about by expulsion of pore water.

Compaction of soil is measured in terms of dry unit weight (easy to calculate and hence used in field method) with Voids Ratio fo relative density to Relative compactness.

Compaction characteristics of the soil is first measured in lab, by performing following test and is further replicated in the field.

1. **Standard Proctor test :**

This test comprises of standard cylindrical bowl of volume 1/30 cubic feet ($\simeq$ 942 ml) in which given sample of the soil is filled in 3 number of layers where each layer is compacted by subjected it to 25 number of blows with the help of the hammer having the total weight of 5.5 pound which is approximately (2.495 kg) and having free fall height of 12 inches ($\simeq$ 304.8 mm).

The mass of soil sample filling the mould is its bulk unit weight (m).

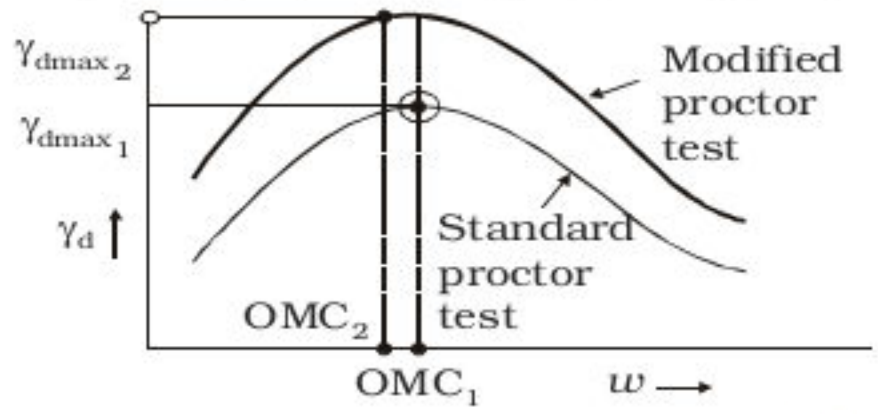
The sample of soil filling the mould is further tested for its water content using oven dry method in order to compute dry density of the soil.

$$\rho = \frac{m}{v} \Rightarrow \rho_d = \frac{\quad}{1+w}$$

V = volume of mould

The test is repeated on sample of soil at different water contents and corresponding dry density of the soil is noted.

The result of the test is further represented in the form of a curve termed as compaction curve, in which water content is expressed on *x*-axis and corresponding dry unit weight (density) on *y* – axis.



Note : The compactive energy used in standard Proctor test is approximately 595 kJm^{-3}.

◆ **IS 2720; recommends equivalent standard proctor test [light proctor test].**

In this test standard cylindrical mould of volume (1000ml) is used in which the sample of soil is filled in 3 number of layers. Each layer is compacted by subjecting it to 25 number of blows with the help of the hammer having total weight of 2.6 kg and face fall height of 310 mm.

In order to stimulate heavy compaction required for military aircrafts and heavy transports modified proctor test is used which consists of standard cylindrical mould of volume of 1/30 cubic feet in which soil is filled in 5 number of layers, and each layer is compacted by subjecting it to 25 number of blows with the help of the hammer having total weight of 10 pounds (4.54 kg) & free fall height of 18 inches (457.2mm).

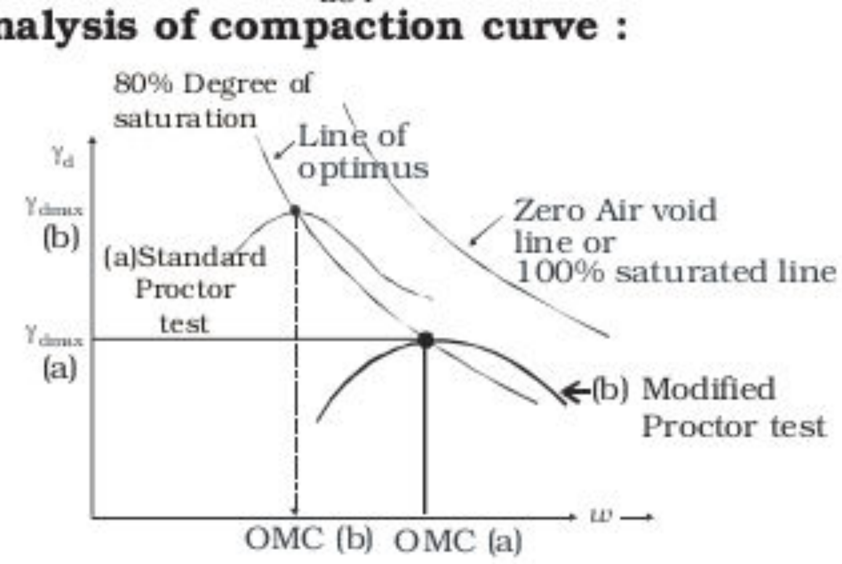
[Compaction energy provided in modified proctor test is approx 2674 kg].

◆ **IS 2720 recommends Equivalent modified proctor test (heavy compaction test) :** In this test standard cylindrical mould of 1000 ml is used and soil is filled in it in five layers, where in each is subjected to 25 no.of blows with the help of the hammer having total weight of 4.9 kg & free fall height of 450mm.

$$\frac{E, HCT}{E, LCT} = \frac{5 \quad 4.9 \quad 450}{3 \quad 2.6 \quad 310} \Rightarrow 4.559$$

E, HCT = 4.56 × E_{LCT}

Analysis of compaction curve :



(1) Optimum moisture content is the water content at which maximum dry unit weight is obtained, (maximum densesness in soil is achieved).

(2) A curve showing the water content & dry unit weight relationship for the compacted soil at a constant percentage of air voids is known as air void line. Therefore Equation of Air void line;

$$\gamma_d = \frac{(1 - \ a).G.\ \ w}{1 \ \ wG}$$

3) A curve showing the water content & dry unit weight relationship for the compacted soil at a constant degree of saturation is termed as saturation line.
∴ Equation of saturation line;

$$\gamma_d = \frac{G.\ \ w}{1 \ \ \frac{wG}{S}} = \frac{G\ \ w}{1\ \ e}$$

4) Theoretical maximum dry unit of the soil is achieved at a particular water content when soil is completely saturated or no air is present in it [S = 1 ; η_a = 0]. In such case, zero air void line corresponds to 100% saturation line.

For, γ_{dmax}

$$\eta_a = 0;\ \gamma_{dmax} = \frac{G.\ \ w}{1 \ \ wG}$$

For, γ_{dmax}

$$S = 1;\ \gamma_{dmax} = \frac{G.\ \ w}{1 \ \ wG}$$

1) Though zero % air void line corresponds to 100% saturation line but 5% or 10% air void line is not equivalent to 95% and 90% of saturation line.

2) Practically in case of silt and clay, maximum dry unit weight is attained at 85% or 95% degree of saturation as air voids can not be reduced to zero even under heaviest state of compaction.
[as soon as a hammer is lifted after the compaction some amount of air again enters into the soil].
However in case of sand, maximum dry density is achieved at 100% saturation as it does not behave similar to cohesive soil under compaction.
In field compaction characteristic of soil analysed by noting the water content & corresponding dry unit weight.
In order to reduce the test time methods adopted to compute the water content is (i) Rapid moisture meter method (calcium carbonate method) or (ii) Proctor needle method.

- The dynamic method gives better results in coarse-grained soils and the static compaction is suitable for less permeable fine-grained soil.
- Typical optimum moisture content values ranges from 10 to 20% with a maximum range of 5 to 30%.

- Under heavier compaction, the moisture density curve is shifted upwards and simultaneously moves to the left, resulting in a lower OMC but a greater $\gamma_{d(max)}$.
- Zero air voids line or saturation line is always a steadily descending line.

Mechanical devices used for compacting the soils :

Compaction can be achieved by the use of any of the following equipment as listed in the table given below.

Compaction Equipments		
Type of Equipment	**Suitability for soil type**	**Nature at Project**
1. Rammers or Tampers	All soils	In confined areas as fills behinds retaining walls, basement walls etc. Trench fills.
2. Smooth wheeled roller	Crushed rocks gravels sands	Road construction.
3. Pneumatic tyred rollers	Sands, gravels, silts, clayey soils	Base, Sub-base and embankment compaction for highway, air fields etc. Earth dam.
4. Sheep foot Rollers	Clayey soils	Core of earth dam.
5. Vibratory Rollers	Sands	Embankments for oil storage tanks etc.

◆ **Factors affecting compaction**

1. Water content :

Initially with increase in water content, compactness of the soil increases. It reaches its maximum value at a particular water content termed as optimum moisture content beyond which increase in water content reduces the compactness of soil.

2. Degree of compaction :

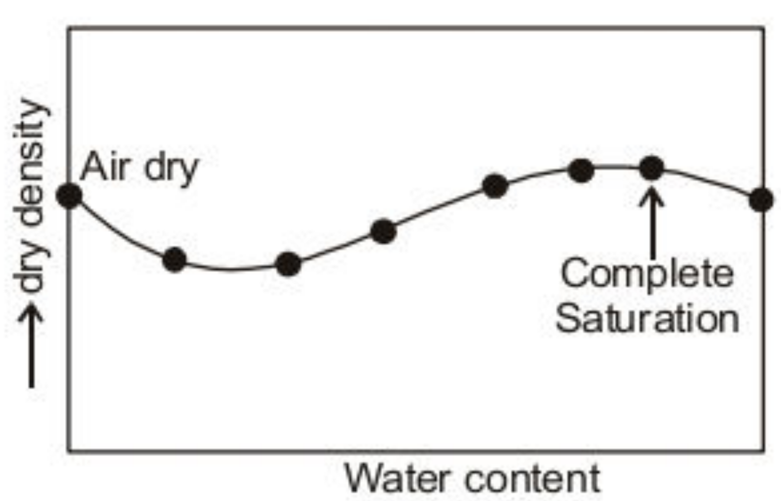
With increase in compactive effort, on the soil, its γ_d increases corresponding to a particular w.c, hence, it can be stated that with increase in degree of compaction (γ_{dmax}) corresponding optimum moisture content reduces.

There in no direct relationship between increase in degree of compaction and increase of maximum dry unit weight.

3. Method of compaction :

It also affects the degree of compaction being achieved in the given sample of soil as it is dependent upon type of compacting equipment used. Time and area of contact between the compacting element & soil.

Compaction Curve for Sand



Initially there is decrease in γ_d with increase in water content. This is due to Bulking of sand (capillary tension in pore water prevents soil particles coming closer). The maximum bulking occurs at 4-5% water content.

- The maximum dry unit weight results when soil is fully saturated.
- When water content is increased further, there is fall in dry unit weight again.
- Thus, max. dry unit weight occurs only when the soil is dry or completely saturated.

4. Type of soil :

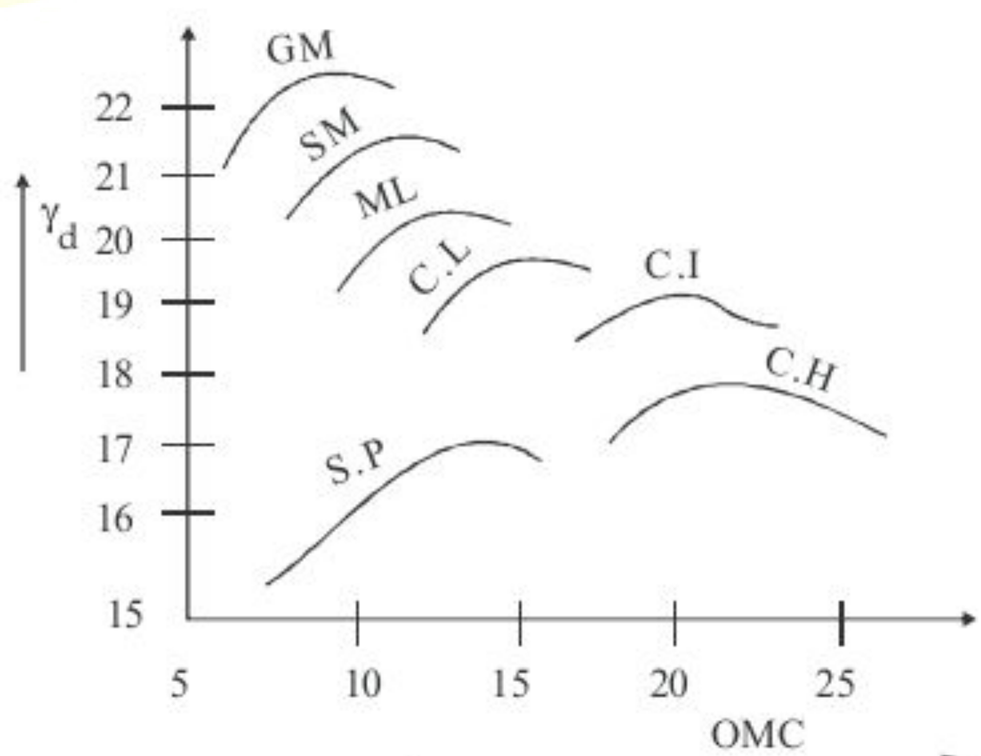
Compaction of the soil is also dependent upon the type of the soil being compacted.

Well graded coarse grained soil having fines (having fine grained soil) is found to have higher value of maximum dry unit weight and low optimum moisture content.

With increase in percentage fineness maximum dry unit cut tends to decrease and optimum moisture content tends to increase.

Maximum dry unit weight of cohesive soil is generally found to be less and their optimum moisture content is generally found to be more with increase in plasticity of these cohesive soils, their maximum dry unit weight tends to decrease.

Poorly graded coarse grained soil is found to have minimum (maximum dry unit weight).



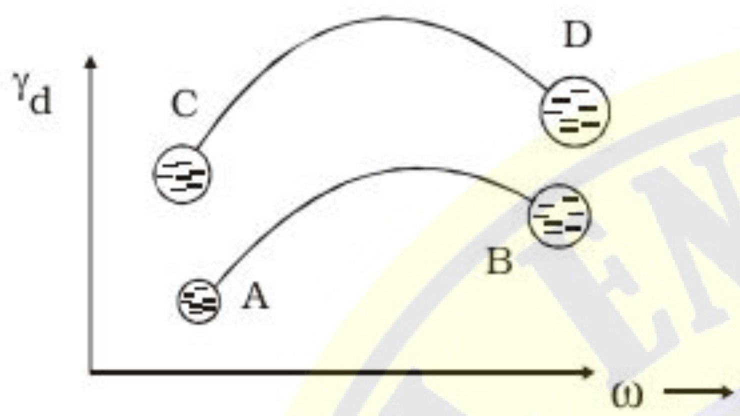
Effect of compaction on Engineering properties of the soil:

i) Structure of the soil :

Compaction affect the structure of fine grained soil to a large extent than the structure of coarse grained soil.

At same compactive effort with increase in water content of the soil orientation of the soil particles improves as the result of which floculated structure is formed on the dry side of the optimum and dispersed structure is found on the wet side of optimum.

With increase in compactive effect, orientation of the soil particles also improves.



So, soil particles at B and D are more oriented than soil particles at A, soil particles at C are more oriented than soil particles at A.

ii) Permeability of the soil :

With increase in water content the permeability of the soil decrease on drying of optimum side and it slightly increases on wet optimum side [However, it remains less than the permeability on dry optimum side].

On wet optimum side, slight increase in permeability is observed due to decrease of dry unit weight whose effect is more than orientation of soil particles.

At same dry unit weight permeability on dry optimum is found to be more than the permeability on wet optimum side (face).

With increase in compactive effort on the soil, the permeability reduces at a particular water content.

iii) Shrinkage : It is less on dry optimum than on wet optimum due to stronger interparticle bonds found in the soil on dry optimum side.

iv) Water Deficiency : The deficiency of the water in soil is found to be more in dry optimum side in the soil due to which Pore water pressure is found to be more during the compaction on wet optimum. [Initially on dry optimum, the pore water pressure is negative].

v) Swelling: As deficiency of water is more on dry optimum, high swelling pressure, resulting in high swelling is found on dry optimum.

vi) Compressibility : At lower stress compressibility on dry optimum is found to be less than the compressibility on wet optimum due to strength interparticle bonds between the solids.

But at higher stress, interparticle bonds are broken. [Flocculated structure are broken] compressibility on dry side optimum is found to be more.

vii) Strength of the soil :

Strength of the soil on dry optimum is found to be more than the strength of the soil on wet optimum due to stronger interparticle bonds found in the soil on dry optimum.

	Engg. properties of soil	Dry optimum	Wet optimum
1)	Soil strength	Brittle failure/ High peak/ High E	Ductile failure/ No peak/ low E
2)	Soil structure	Flocculated	Dispersed
3)	Soil permeability	More	Less
4)	Shrinkage	Less	More
5)	Water Deficiency	More	Less
6)	Pore water pressure	Less	More
7)	Swelling	More	Less
8)	Compressibility	Low stress → Less High stress → More	More Less

Note : Core of earth Dam → Wet of optimum → Permeability required is less.

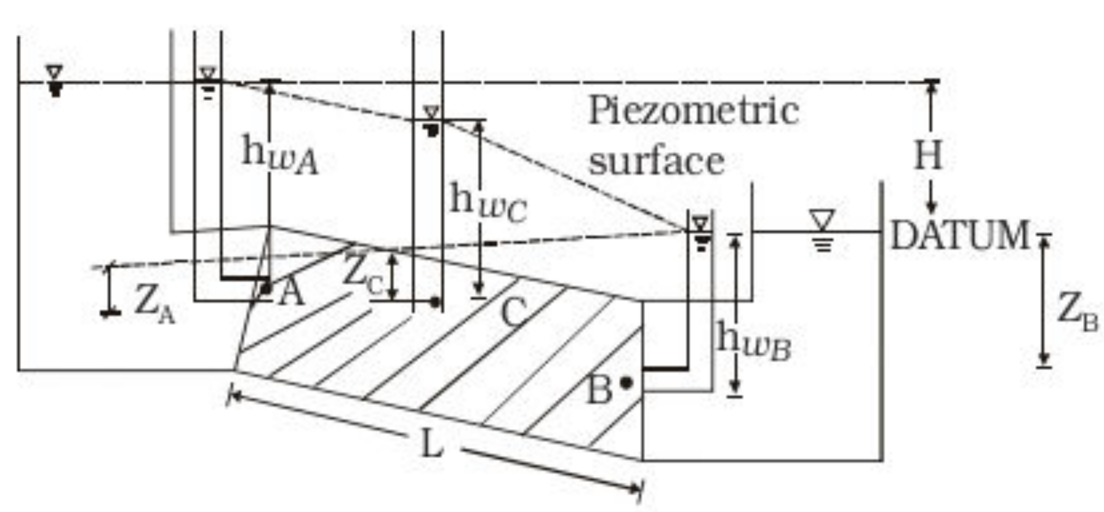
Subgrade of the pavement → Wet of optimum → Permeability required is less → Swelling required is less

Composite Dam → Dry of optimum → Strength of soil required is more.

SEEPAGE ANALYSIS

When water flows through the soil medium i.e., completely saturated, total head at any point. consists of pressure head (piezometer head), velocity head, datum head (position head). In case of flow through soil, velocity is comparatively less hence velocity head can be neglected, therefore total head comprises of pressure head & datum head. This total head is termed as hydraulic head.

Pieozometric head is the pressure head of but pressure head with respect to a reference of datum is total head.



AT Point A	At Point B	AT Point C
Pressure head (h_{wA})	h_{wB}	h_{wC}
Pressure head ($-Z_A$)	$-Z_B$	$-Z_C$
Pressure head $= h_{wa} - Z_A$	$h_{wB} - Z_B = 0$ ($h_{wB} - Z_B$)	$h_{wC} - Z_C$

(datum head can be positive or negative depending upon datum considered position).

Since our soil sample is saturated, capillarity tension does not exist, pressure head is always positive. It can be negative also in other cases.

$h_A = h_{wA} - Z_A$; $h_B = 0$

Therefore, hydraulic head $= h_w \pm z$ (Overall hydraulic head)

$H = h_A - h_B = h_{WA} - Z_A$

Head under which flow is occuring from A to C

$H' = h_A - h_c = h_{WA} - Z_A - (h_{wc} - Z_c)$

$= h_{WA} - Z_A - h_{wc} + Z_c$

When water flows through the soil medium, it exerts pressure over the soil solids due to friction i.e. termed as the **seepage pressure.** Hence, seepage pressure may be defined as pressure exerted by the water over the soil skeleton through which it percolates.

Seepage pressure $(P_S) = h.\ \gamma_w$

$\therefore\ P_S = \dfrac{h}{L} . L . \gamma_w = i . l . \gamma_w$

h = hydraulic head at a point $= h_w \pm z$

hydraulic gradient $= i = \dfrac{h}{L}$ = hydraulic head loss per unit length

P_f (seepage force) $= P_S . A = i . L . \gamma_w . A$

$P_f = i . L. \gamma_w .A = i . \gamma_w . V$

Specific seepage force $= \dfrac{\text{Seepage force}}{\text{Volume}}$

$\therefore\ SP_f = \dfrac{i\ \ w.V}{V} = i\gamma_w$

$SP_f = i\gamma_w$

Seepage pressure always acts in the direction of flow, hence, vertical pressure may be increased or decreased due to seepage pressure depending upon the direction of flow.

At any general section, total vertical pressure comprises of submerged weight of the soil and seepage pressure.

Net effective vertical pressure

$\Rightarrow$ **$\sigma' = L . \gamma' \pm P_S$**

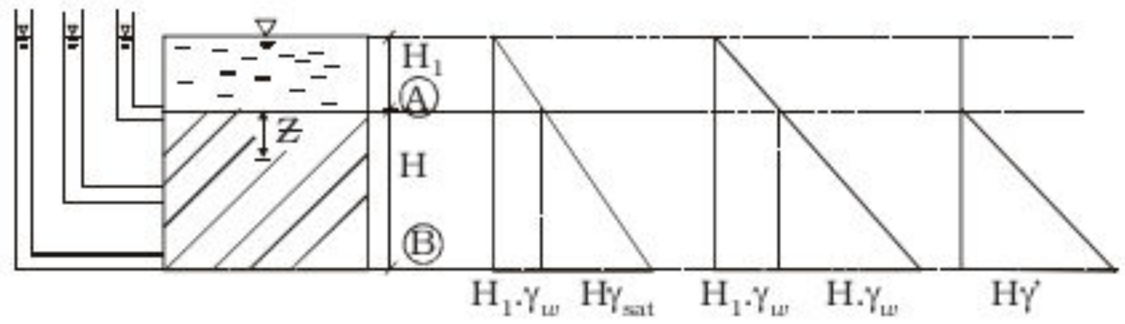
If flow takes place in vertically downward direction, seepage pressure also acts in the direction of flow, hence, it increases the effective stresses.

$\sigma' = L . \gamma' + P_S \downarrow$

If flow takes place in vertically upward directions. it reduces the effective stresses as in this case. P_S also acts in vertically upward directions.

$\sigma' = L . \gamma' - P_S \uparrow$

No flow condition



At A

$\sigma = H_1 . \gamma_w$

$u = H_1\ \gamma_w$

$\sigma' = 0$

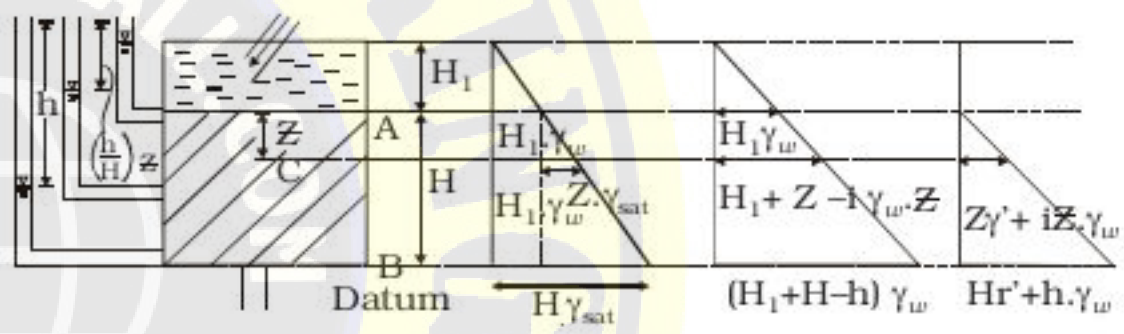
At B

$\sigma = H_1 . \gamma_w + H_1\ \gamma_{sat}$

$u = (H_1 + H)\ \gamma_w$

$\sigma' = H\ \gamma'$

b) FLOW IN VERTICAL DOWNWARD DIRECTION



At Point A

Pressure head $= H_1$

Datum head $= H$

Total head $= H_1 + H$

At Point B

Pressure head $= H_1 + H - h$

Datum head = 0

Total head $= H_1 + H - h$

Head through which flow takes i.e. from A to B A to B $= h_A - h_B = h$

$\dfrac{\text{Hydraulic head lost}}{\text{Length of flow}}\ \ i\ \ \dfrac{h}{H}$

$\therefore$ **At point A**

$\sigma = H_1\ \gamma_w$

$u = H_1\ \gamma_w$

$\sigma' = 0$

At point B

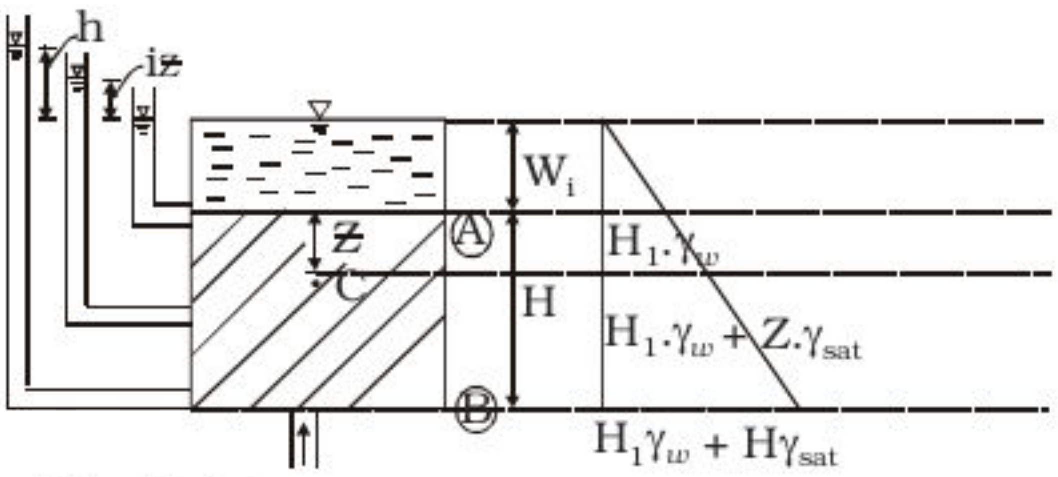
$\sigma = H_1 . \gamma_w + H . \gamma_{sat}$

$u = (H_1 + H - h)\ \gamma_w$

$\sigma' = \sigma - u = H_1\ \gamma_w + H\ \gamma_{sat} - H_1\ \gamma_w - H\gamma_w + H\gamma_w$

$\sigma' = H\gamma' + h\ \gamma_w$

Flow in vertically upward Direction :



At point A

Pressure head = H_1

Datum head = H

Total head = H_1 + H

At point B

Pressure head = H_1 + H + h

Datum head = 0

Total head = H_1 + H + h

Head through which flow takes from B to A = $h_B - h_A$

$= \cancel{H_1} + \cancel{H} + h - \cancel{H_1} - \cancel{H} = h$

$$i = \frac{\text{Hydraulic head lost}}{\text{Length of flow}} \quad \frac{h}{H}$$

At point A

$\sigma = H_1\,\gamma_w$

$u = H_1\,\gamma_w$

$\sigma' = 0$

At point B

$\sigma = H_1.\gamma_w + H.\gamma_{sat}$

$u = (H_1 + H + h).\gamma_w$

$\sigma' = H\gamma' - h.\gamma_w$

When flow takes place in upward direction, seepage pressure also acts in upward direction and effective pressure is reduced, if seepage pressure is such that, it becomes equal to the submerged weight of the soil mass then total vertical pressure is reduced to zero, leading to loss of shear strength of the soil solids in cohesion less soils, that results in tendency of the particles to move up in direction of the flow. This process in which soil particles are lifted and leaves the soil mass is termed as Quick sand condition, Quick condition or piping or sand boiling.

Quick condition :

When flow is upward P_s is also upward and effective total pressure reduces (seepage pressure = submerged weight of soil).

$\therefore\ \sigma' = H\gamma' - h \times \gamma_w = 0$

$H\gamma' - i\,H\,\gamma_w = 0$

$i.\,H.\,\gamma_w = H\gamma'$

$i.\,\gamma_w = \gamma_{sat} - \gamma_w$

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

The hydraulic gradient at which such a condition exists is termed as critical Hydraulic Gradient, Piping (HG), Bursting (HG).

For fine sands, critical HG = 1 as in such case $G \simeq 2.65$ and $e \simeq 0.65$

$$\therefore i_c = \frac{2.65-1}{1+0.65} = 1$$

In order to avoid quick sand condition or piping, hydraulic should be less than critical hydraulic gradient.

$\therefore i < i_c \ \therefore\ \ \text{F.O.S} = \frac{i_c}{i}$

Quick sand condition is generally found in fine sand and silt and is not found in the case of gravel, coarse sand and clay.

In case of cohesive soil like clay, shear strength is not reduced to zero, even if (σ') Net are reduced to zero, as they posses inherent cohesion which present soil particles to leave the soil mass.

For cohesionless soil :

$\tau = \sigma' \tan\phi$

if $\sigma' = 0$; $\tau = 0$

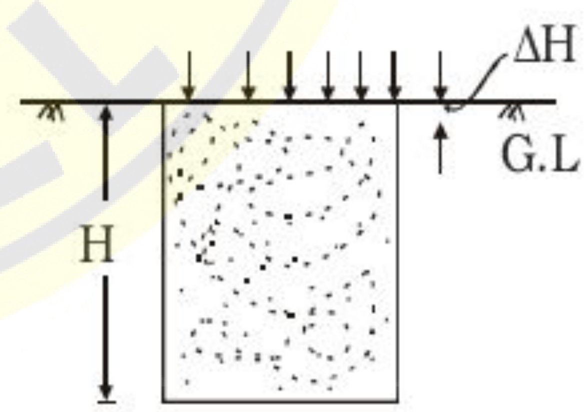
- Quick sand is not a type of sand but only a condition.

Use of Flow Net

1. For seepage calculation
2. Flow net can be used in calculating uplift pressure under masonry dams.
3. For exit gradient and piping.

- A factor of safety of 6 to 7 is recommended for safety against piping.
- Piping can be reduced by making vertical cut off walls at downstream ends or by making a graded filter.

COMPRESSIBILITY OF SOIL



Compressibility refers to the decrease in volume of soil due to application of load.

If load continues to act then volume of soil will decrease, which is primarily due to decrease in depth [Area is infinitely large hence considered constant]

The volume change may be due to :

i) Compression of pore air with expulsion of trapped air. (Initial compression/compaction)

ii) Expulsion of pore water (Primary consolidation because molecules are considered incompressible)

iii) Compression of pore water (Negligible)

iv) Compression of solids (Negligible)

v) Readjustment of molecules into more stable position (Secondary consolidation)

Compressibility
- Compaction
  * Immediately on loading
- Consolidation
  * time taking
  - Primary consolidation
  - Secondary consolidation
    — Insignificant in coarse soil but significant in clay soil
    — (acts a creep effect)

◆ DIFFERENCE BETWEEN COMPACTION & CONSOLIDATION

Compaction	Consolidation
1. Immediate effect	1. Time dependent and slow process
2. Effect of expulsion and compression of air	2. Effect of expulsion of pore water
3. Soil is dry or partially saturated	3. Soil remains fully saturated during entire process of consolidation (S = 1)
4. Degree of compaction depends on magnitude of load applied (compactive effort)	4. Its degree depends upon magnitude of load applied (effective stress) permeability of soil, drainage condition and soil properties. (eg: Voids Ratio; clayey has) greater voids ratio and so, consolidation is more

PRIMARY CONSOLIDATION :

Due to increase in effective stress, expulsion of pore water takes place which is time dependent phenomena and is called primary consolidation, at a given increase in effective stress [$\Delta \bar{\sigma}$], expulsion of pore water will stop after a certain time than primary consolidation is considered to be complete. The change in volume of soil is equal to volume of expelled water. It means the degree of saturation will remain 100% throughout the process of primary consolidation.

Note : Since, soil mass is considered semi-infinite (finite depth but infinite area), therefore, volume change is due to depth change only (If area is not infinite, then there will be rigid body because of continuous compaction.)

SECONDARY CONSOLIDATION :

After primary consolidation is complete, when expulsion of pore water is stopped, if load continues to act, then, further decrease in volume of soil is recorded due to plastic readjustment of soil molecules in order to occupy more stable position, which occurs with the time. This is known as secondary consolidation.

It may take several years. It is significant only for highly plastic clayey solis.

In granular soils, (gravels and sands), secondary consolidation is negligible, whereas in highly plastic clay, it may be 10 to 20 % of total settlement.

In this type of consolidation, expulsion of pore water does not occurs.

Note :

1. The consolidation settlement is a function of increase in effective stress but not the function of increase in total stress.
2. Due to infinite length and width of soil, the expulsion of pore water takes places in vertical direction. It results achange in depth. Therefore consolidation is mainly one dimensional, three dimensional consolidation is also possible (triaxial test) in which area is kept constant.

The cause of secondary consolidation are not fully established but it may be attributed to plastic readjustment of soil molecules.

◆ NORMALLY CONSOLIDATED AND OVER CONSOLIDATION SOILS :

Normally consolidated soils are those, which are loaded 1st time, i.e., the present applied effective stress is large enough and soil was never loaded in the past to this stress.

Over consolidation soils are those which have been loaded in the past history to the effective stress ≥ to present applied effective stress.

Over consolidation soils are also called pre-consolidated soils or pre-compressed soils.

Such soils are less compressible & represents negligible volume change on loading.

A heavily over consolidatied clay behaves similar to the dense sand [In dense sand not much settlement occurs].

Over Consolidation Ratio (OCR) :

Over Consolidation Ratio =

$$\frac{\text{Maximum applied effective stress } (\bar{\sigma}) \text{ in past history}}{\text{Present applied effective stress } (\bar{\sigma})}$$

For;

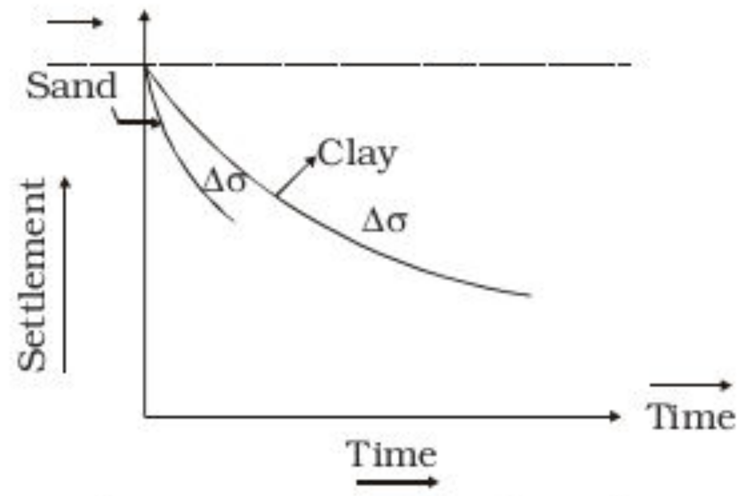
Normal Consolidation Soil; Over Consolidation Ratio ≤ 1 [maximum Over Consolidation Ratio for N.C. soil = 1]

Over Consolidation Soil Over Consolidation Ratio > 1

Also ;

if Over Consolidation Ratio >>1 → Soil is then called heavily over consolidation soil.

◆ TIME V/S SETTLEMENT CURVE : (In relation to consolidation) [consolidation effect]



Sands are less compressible than clays and rate of settlement is sands in higher than that of clays.

Sands has more permeability than clays hence, sands will require less time to complete consolidation.

Clays have greater voids ratio, therefore on application of load time change in voids ratios will be large hence, total settlement of clay will be much larger than sands.

Note :

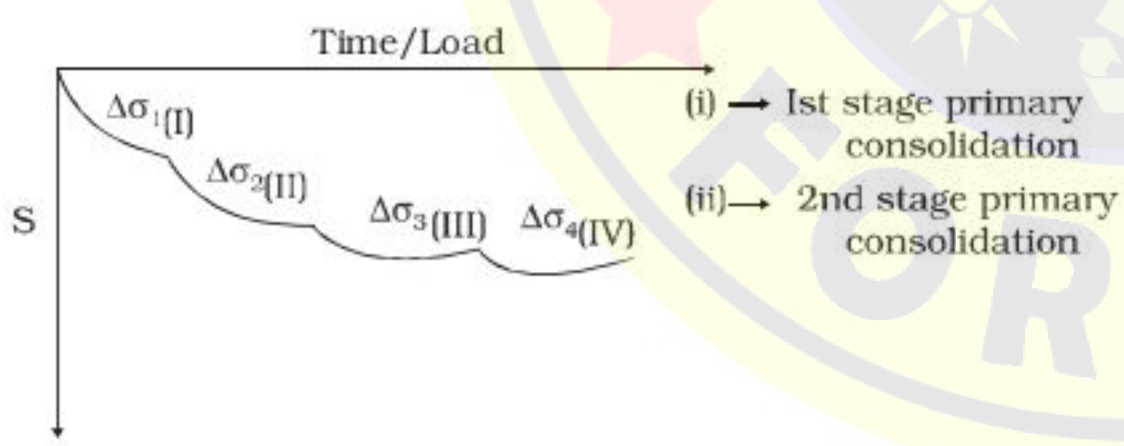
For all practical purposes, the consolidation settlement in sand is neglected due to;

i) Settlement gets completed during the period of construction of building.

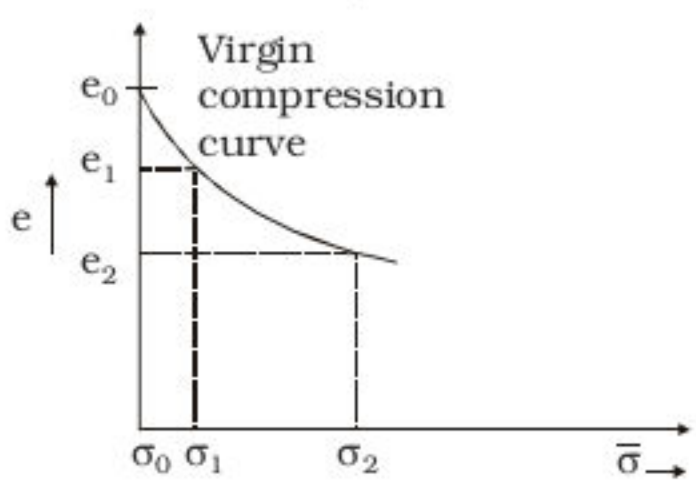
ii) The total settlement in sands is not very large.

In case of clays due to low permeability, settlement process is slow and may take from few months to few years and future problem of settlement may occur, therefore, clays are considered compressible and their compressibility is significant.

◆ **MULTI STAGE LOADING EFFECT IN CLAYS :**



EFFECTIVE STRESS V/S VOID RATIO CURVE :



a) Normal Consolidated Soil :

Always for clay. The graph between effective stress and void ratio is called virgin compression curve, the slope of this curve in Arithematic scale is called co-efficient of compressibility (a_v).

From graph ; the co-efficient of compressibility decrease with increase in effective stress.

$\therefore$ a_v = co-efficient of compressibility = $-\left(\frac{e}{-}\right)$

$$a_v = -\left(\frac{e}{-}\right)$$

where,

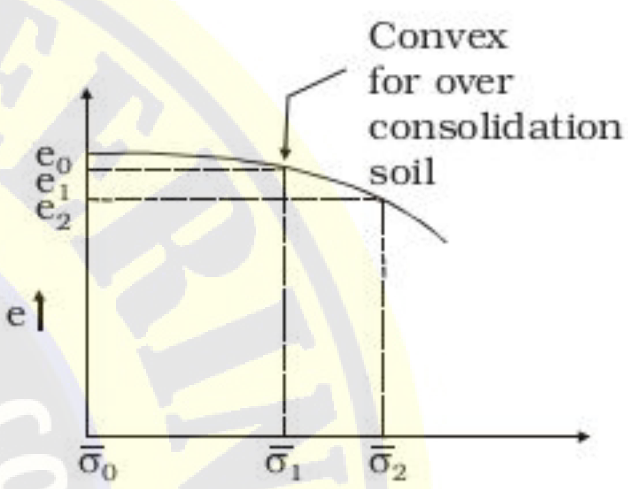
$\therefore$ $\Delta e = (e_2 - e_1)$

So, $a_v = \frac{-(e_2 - e_1)}{(\bar{}_2 - \bar{}_1)}$ or $\frac{(e_1 - e_2)}{(\bar{}_2 - \bar{}_1)}$

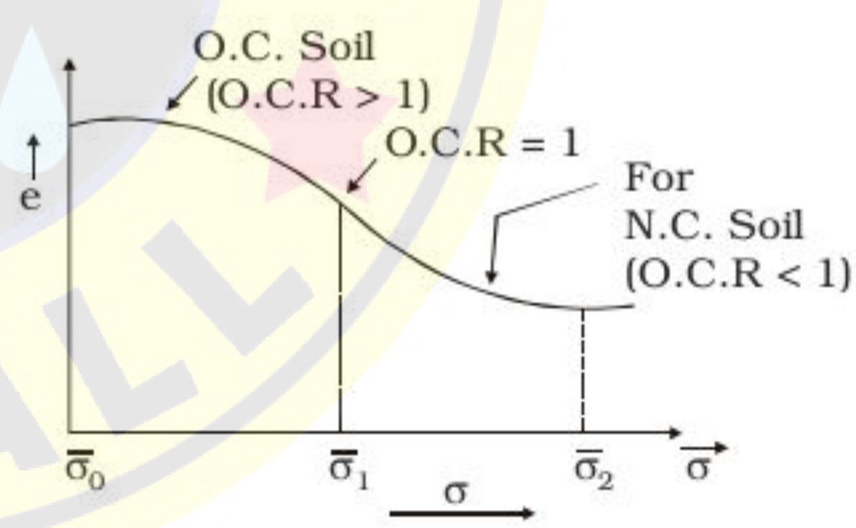
◆ This coefficient of parameter is not constant.

b) Over-consolidated soils :

→ a_v increases with increases in $\bar{\sigma}$.



C) Over Consolidatied Soil and Normal ConsolidatiedSoil :



From $\bar{\sigma}_0$ to $\bar{\sigma}_1$ soil is over consolidated and beyond $\bar{\sigma}_1$ soil is normally consolidated.

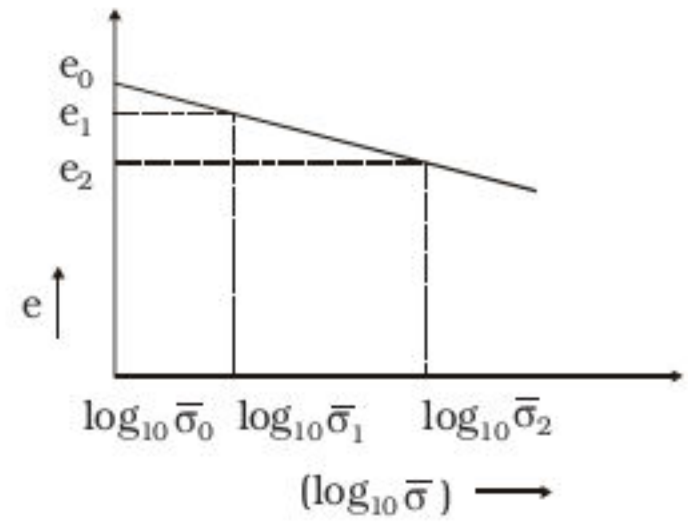
Note :

(a_v) represents change in void ratio due to unit change in effective stress since, it changes with increase in effective stress, hence its use is limited. Instead co-efficient of compression (C_C) is used for computation of settlement of soil.

◆ **Co-efficient of compression (C_C) :**

If effective stress is plotted in log scale (*x*-direction) and void ratio plotted in arithematic scale hence, curve is plotted on semi-log scale, then for N.C. soil, the curve is found straight line.

Slope of such curve is called co-efficient of compression/consolidation



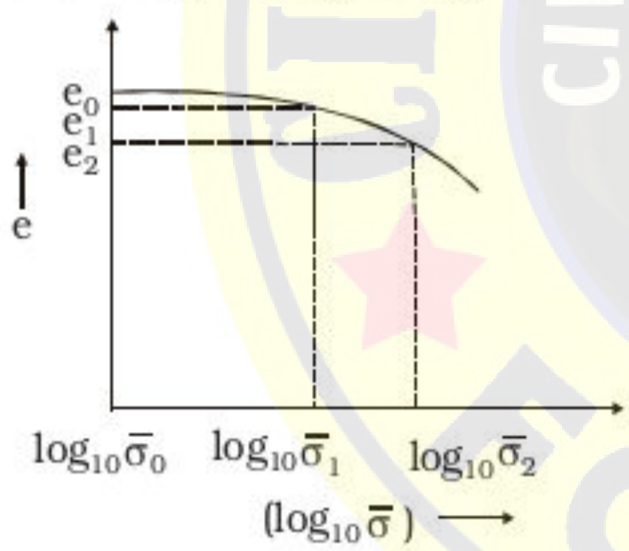
$$C_C = \frac{-[e_2 - e_1]}{\log_{10}\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)} = \frac{\Delta e}{\log_{10}(\bar{\sigma}_2/\bar{\sigma}_1)}$$

Note :

1) For Normal Consolidation clays;
C_C = 0.1 to 0.8

2) Greater the C_C/ (a_v) greater is the compressibility of soil.

3) For Over Consolidatied clays
It is called co-efficient of Recompression.
(C_r) Co-efficient of Recompression :-
→ Defined for Over Consolidatied Soil
The sloped of void Ratio V/S effective stress in log scale is known as Co-efficient of Recompression.
The curve is not perfectly straight line.



$$C_r = \frac{-\Delta e}{\log_{10}\frac{\bar{\sigma}_1}{\bar{\sigma}_0}}$$

Note :

Co-efficient of recompression is $1/5^{th}$ to $1/10^{th}$ of Co-efficient of compression of the soil, hence, Over Consolidation soils are less compressible.

◆ **Determination of C_C using empirical Relation :**

1. For undisturbed soil of medium sensitivity :
$C_C = 0.009\ (W_L - 10)$
2. For remoulded soil of medium to low sensitivity :-
$C_C = 0.007\ (W_L - 10)$
3. C_C using natural water content :-
$C_C = 1.15 \times 10^{-2}\, W_n$
Where, W_n = % water content (natural) in the soil.
4. C_C using initial void Ratio :
$C_C = 0.40\ (e_0 - 0.25)$

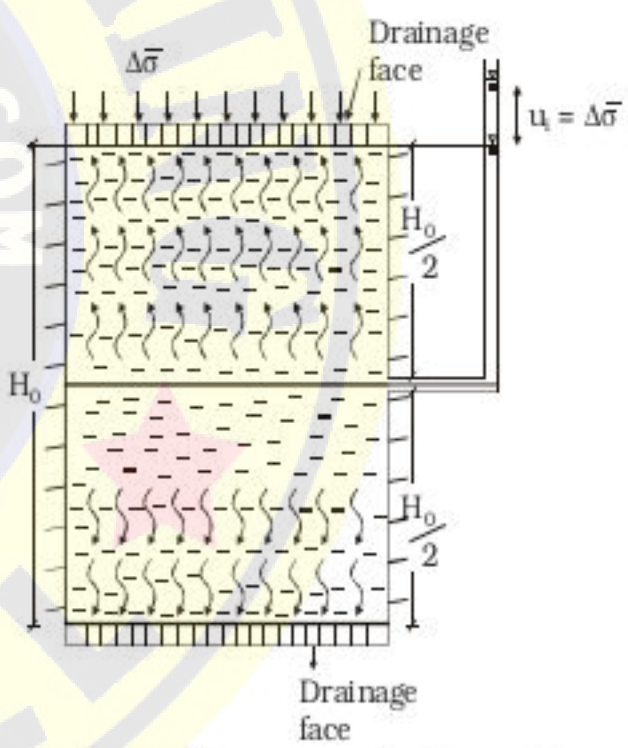
where, e_0 = initial voids Ratio in fraction or
$C_C = 1.15\ (e_0 - 0.35)$
(W_L, W_n, e_0 should be computed at the center of clay layer).

TERZAGHI THEORY OF ID-CONSOLIDATION :

Assumption (in context of permeability) :

1. The soil mass is homogeneous and isotropic.
[i.e., $K_x = K_y = K_z$ = constant]
2. Darcy's law is valid (saturated soil and laminar flow).
3. The flow is **1-Dimensional** (Vertically).
4. The strains produced are small, due to applied stress, therefore no change in soil structure is considered.
5. The soil is fully saturated and remains saturated throughout the process of consolidation (S = 1).
6. The soil mass is semi-infinite [finite depth and infinite area] in which volume change is due to expulsion of pore water that reduces the depth.
7. Only hydro-dynamic lag is considered but plastic lag is ignored, however it is known to exist.
Therefore it means this theory, is applicable only for primary consolidated and not for secondary consolidated soil.

◆ **CONSOLIDATION TEST :**



At time t = 0 but after completion of consolidation (primary) : u = 0
at t → ∞

Consolidation test is conducted in (OEDOMETER)

If in the field soil is subjected two way drainage (top and bottom) then in the lab test should also be 2-way drain.

But if in the field soil is one-way drained either (top or bottom) then test should be also one way drain.

Let 'd' is length of drainage path

$= \frac{H_O}{2}$ for 2 -way drainage

$= H_0$ for 1-way

When effective stress, $\Delta\bar{\sigma}$ is increased/applied then pore pressure increases, **$u = u_i = \Delta\bar{\sigma}$** which is called excess pore pressure.

At the beginning of test, applied effective stress/ pore pressure is taken by pore water and with the passage of time, excess pore pressure reduces & at the completion of consolidation, excess pore pressure becomes zero.

According to Terzaghi, the 1-Dimensional consolidation equation is given by ;

$$\frac{u}{t} = C_V \cdot \frac{^2u}{z^2}$$

where,

u = excess pore pressure = $u_i = \Delta\bar{\sigma}$ at t = 0

u = 0 at $t \to \infty$

$\frac{u}{t}$ = Rate of change of pore pressure,

$\frac{u}{z}$ = Rate of change of pore pressure with respect to depth

C_V = Co-efficient of consolidation which depends on type of soil and its unit is m^2/S or cm^2/S

$$C_V = \frac{K}{_w . m_V}$$

Where,

K = co-efficient of permeability or hydraulic conductivity

γ_w = unit weight of water

m_v = co-efficient of volume compressibility or Modulus of Volume change

$\frac{1}{m_V}$ = compression modulus = E_C

m_v is defined as the ratio of unit volume change to the corresponding change in effective stress.

$$m_v = \frac{-\left(\frac{V}{V}\right)}{-}$$ ΔV decreases (–ve) when $\Delta\bar{\sigma}$ increases i.e. (+ve) .... $\left(\frac{m^2}{kN}\right)$

e_o = initial void ratio at the centre of layer

$$\therefore \quad m_v = \frac{a_V}{1+e_o} \;....\; \left(\frac{m^2}{kN}\right)$$

Note :

The soil having greater C_V will consolidate faster, since $C_V \propto K$, hence permeable soil will complete consolidation quickly.

◆ **DEGREE OF CONSOLIDATION (U) :**

It is defined as the fraction/percentage of primary consolidation which is completed after the time (t).

Theoretically, infinite time is required for primary consolidation completion however practically consolidation is supposed to be completed when degree of consolidation is reached to greater than or equal to 90%.

Degree of consolidation may be determined as :

case 1 : In terms of settlement :

Let 'ΔH' is ultimate primary consolidation settlement [when U = 100% ; $t \to \infty$]

Let 'Δh' is the settlement after time 't', when Degree of consolidation is U

$$\therefore \; U = \frac{h}{H} \qquad \text{OR} \qquad U = \frac{h}{H} \times 100\%$$

Case 2 : In terms of voids ratio :

Let e_o is initial voids ratio at the beginning of test (t = 0)

Let e_{100} is voids ratio after completion of primary consolidation (t = ∞)

Let 'e' is the void ratio at any intermediate time 't' at which degree of consolidation is U.

$$U = \frac{e_o - e}{e_o - e_{100}} \quad \text{or } \%\, U = \frac{e_o - e}{e_o - e_{100}} \times 100\%$$

The void ratio (e, e_0 & e_{100}) should be determined at the centre of soil layer.

Case 3 : In terms of excess pore pressure:

Let u_i is initial excess pore pressure at the begining of test due to application of effective stress [$u_i = \Delta\bar{\sigma}$] ; (at t = 0)

Let u is excess pore pressure after time 't' when Degree of consolidation (U).

If $t \to \infty$; u = 0

$$\text{Then, } U = \frac{u_i - u}{u_i} \;;\quad \%\, U = \frac{u_i - u}{u_i} \times 100\%$$

At t = 0 ; u = u_i ⇒ U = 0

At t → ∞ ; u = 0 ⇒ U = 100%

◆ **TIME FACTOR (T_V) :**

It depends upon the distribution of initial excess hydrostatic pressure for a particular average degree of consolidation.

It is that parameter which relates to the degree of consolidation with time required to achieve that degree of consolidation.

$$T_V = \frac{C_V . t}{d^2},$$

where,

C_V = Co-efficient of consolidation

t = time required to achieve degree of consolidation U which is related to T_V

d_1 = H_o ; For 1 way drainage path

$d_2 = \frac{H_O}{2}$; For Two-way drainage path

Note : 1. Due to consolidation settlement 'd' changes with time.

∴ Average value should be taken. Let d_i is initial length of drainage path and d_f/d_{100} is final length of drainage path after completing the primary consolidation, then average drainage path is

$$d = \frac{d_i \quad d_f}{2}$$

2) Since, $\Delta H <<< H_O$,

$\therefore$ for all practical purpose 'd' may be taken = (d_i)

◆ **Relation between U & T_V :**

$T_V = \frac{\;}{4}(U)^2$ .... when $U \leq 0.6$

$T_V = -0.9332 \log_{10}(1-U) - 0.0851$ ... when $U > 0.6$

Note that :

If U = 0   If U = 0.5

$T_V = 0$ $T_V = \frac{\;}{4}(0.5)^2 \Rightarrow T_V = 0.196$

If U = 0.9

$T_V = -0.9332 \log(1-0) - 0.0857$

$T_V = 0.848$

$T_{100} \rightarrow \infty$ $T_0 = 0$

$t_{100} \rightarrow \infty$ $T_{50} = 0.196$

$T_{90} = 0.848$

◆ **Determination of C_V :**

C_V depends on type of soil, its value increases with increase in liquid limit of soil.

Though value of C_V changes with change in effective stress along the depth of soil ; however an average value is computed at the centre of layer.

For most of the soil (clays) its value is in the range of 5×10^{-4} mm²/sec to 5×10^{-2} mm²/sec.

There are two methods to find C_V : These are called, time fitting methods or curve fitting methods

i) Taylor's Methods (OR) Square root of the fitting Methods :

Let, $T_{90} \rightarrow$ time factor for 90% of U.

$t_{90} \rightarrow$ time required for 90% of U.

d = length of drainage path

then ; $C_V = \frac{T_{90} \quad d^2}{t_{90}}$

ii) A Casagrande Method (OR) Logarithmic time fitting Method

Let, T_{50} Time factor for 50% of U = 0.196

t_{50} = time required for 50% of U

then

$$C_V = \frac{T_{50} \quad d^2}{t_{50}}$$

Note : The square root of time fitting method is better for those soils which have high secondary consolidation such as high plastic clays.

COMPRESSION RATIO :

Let R_i is initial dial gauge reading before application of stress/pressure.

R_0 = dial gauge reading when all the air is expelled out/at the beginning of consolidation.

R_{100} = dial gauge reading after 100% completion of primary consolidation.

R_f = Final dial gauge reading after secondary consolidation

a) Initial compression Ratio or Immediate compression ratio :

$r_i = \frac{R_i - R_o}{R_i - R_f}$ Note that; $r_i + r_p + r_s = 1$

b) Primary compression ratio :

$$r_P = \frac{R_0 - R_{100}}{R_i - R_f}$$

c) Secondary compression ratio :

$$r_S = \frac{R_{100} - R_f}{R_i - R_f}$$

◆ **SETTLEMENT ANALYSIS :**

The total settlement in the soil is sum of the following three:

i) Immediate Settlement (Si) :

It is due to compression and expulsion of pore air or due to deformation of molecules or due to turbulent flow of water.

ii) Primary consolidation settlement (S_c) :

This settlement is due to expulsion of pore water under laminar condition.

iii) Secondary consolidation settlement (S_S) :

It is due to plastic readjustment of soil molecules.

$\therefore$ Total settlement (S) = $S_i + S_C + S_S$

I.S. Specification for permissible settlement:

i) Permissible total settlement :

Type of soil & foundation	Permissible total settlement
1. Isolated foundation on clay	65 mm
2. Isolated foundation on sand	40 mm
3. Raft foundation on clay	65-10mm
4. Raft foundation on sand	40-65 mm

ii) Permissible differential settlement :

Type of soil & foundation	Permissible differential settlement
1. Raft foundation on clay	40 mm
2. Raft foundation on sand	25 mm

◆ **Computation of immediate settlement (Si):**

a) For cohesionless soil [sand] :

The immediate settlement in sand is computed by standard penetration test data, which is co-related to the equation given below:

$$S_i = \frac{H_O}{C_S} \log_{10}\left(\frac{\bar{\;}_0 \quad \bar{\;}}{\bar{\;}_0}\right)$$

where,

$$C_S = 1.5 \frac{C_r}{\bar{\sigma}_o}$$

C_r = Static cone resistance in kN/m^2

H_o = Initial thickness/height of the sand layer

$\bar{\sigma}_0$ = Initial effective overburden pressure at the centre of sand layer

$\Delta\bar{\sigma}_0$ = Increase in effective stress at the centre of sand layer

b) For cohesive soil [clay] :

The immediate elastic settlement of the clays at the corner (below the corner) is given by ;

$$S_i = \frac{q.B(1-\mu^2)}{E_S} . I_t$$

where,

q = uniform pressure at the base of flexible foundation

B = width of flexible foundation

Computation of primary settlement (S_c) :

a) Using change in void ratio :

Let e_o = initial void ratio at the centre of soil layer (clay)

Δe = change in void ratio due to increase in effective stress by $\Delta\bar{\sigma}$ at centre of soil layer

$$S_C = \Delta H = \frac{\Delta e}{1+e_o} . H_O$$

b) Using co-efficient of volume change :

The magnitude of settlement is given by ;

$S_C = \Delta H = Ho. m_v . \Delta\bar{\sigma}$

where,

$\Delta\bar{\sigma}$ = Increase in effective stress at the centre of soil

m_v = Co-efficient of volume change at centre of soil or modulus of change in volume

c) Using co-efficient of compression (C_C) :

$$S_C = \Delta H = \frac{C_C H_o}{1+e_o} \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

C_C = Compress Index or co-efficient of compression

H_O = Initial thickness of soil layer

$\bar{\sigma}_0$ = Initial effective stress/overburden pressure at the centre of soil layer

$\Delta\bar{\sigma}$ = Increase in effective stress at the centre of soil layer

e_O = Initial void ratio at the centre of soil layer

Note :

1) If soil is over consolidated/preconsolidated, then use C_r (Co-efficient of Re-compression) in place of C_C in above equation.

If from $\bar{\sigma}_0$ to $\bar{\sigma}_1$ soil is over consolidation and then $\bar{\sigma}_1$ to $\bar{\sigma}_2$ soil is normally consolidated then S_C is $S_{C1} + S_{C2}$

S_{C1} = settlement of O.C stage $\bar{\sigma}_1 - \bar{\sigma}_0 = \Delta\bar{\sigma}_1$

S_{C2} = settlement of N.C stage $\bar{\sigma}_2 - \bar{\sigma}_1 = \Delta\bar{\sigma}_2$

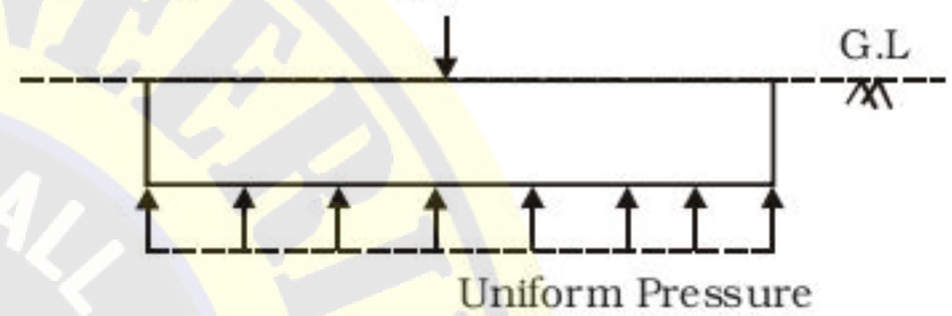
$$\therefore S_C = \frac{C_r H_O}{1+e_o} \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}_2}{\bar{\sigma}_1}\right) + \frac{C_C - H_o}{1+e_o} . \log_{10}\left(\frac{\bar{\sigma}_1 + \Delta\bar{\sigma}_2}{\bar{\sigma}_1}\right)$$

S_{C1} << S_{C2} and hence can be neglected.

Foundation :

Case 1 : Flexible foundation ;

The settlement of flexible foundation may be not uniform but pressure at the foundation will be uniform, irrespective of type of soil.

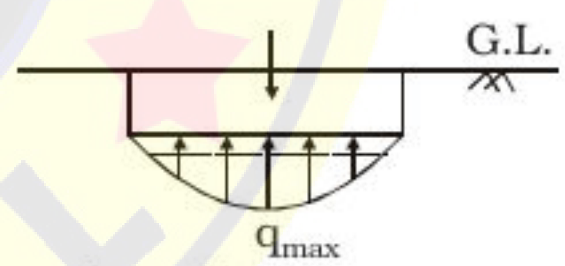


Case 2 : Rigid foundation ; [RAFT/MAT]

In this case, settlement below the foundation is uniform but the pressure developed on the foundation mill depend on the type of soil.

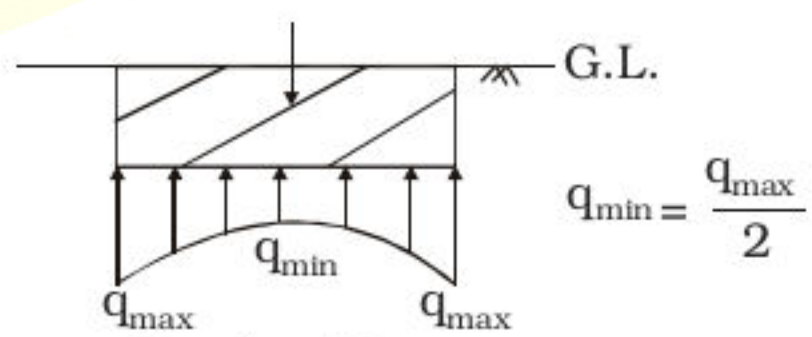
a) **FOR SAND (ϕ - Soil) :**

Settlement will be uniform

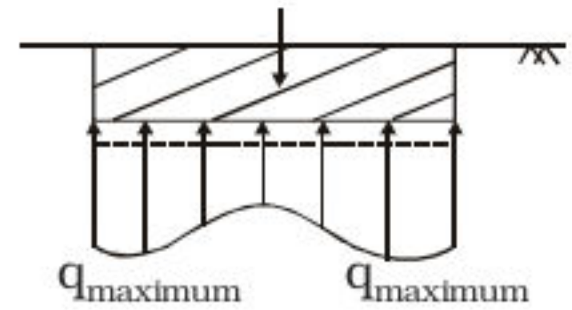


(b) FOR CLAYS (C-Soil) :

In *clayey* soil, maximum contact pressure is at the edge & minimum at the centre & settlement will be uniform.



c) **FOR SILTS [C - ϕ Soils] :**



In C - ϕ soil maximum pressure occurs between centre & edges.