WHAT IS SOIL ?

- For a Civil Engineer, soil means all natural occurring, relatively unconsolidated earth material- organic or inorganic in character that lies above the bed rock

SOIL MECHANICS

- **Soil Mechanics is the branch of science that deals with the application of principles of mechanics, hydraulics and some chemistry to engineering problems related to soils**

Soil Engineering/ Geotechnical Engineering

- Soil Engineering includes soil mechanics, geology, structural engineering, soil dynamics and disciplines related to obtain solutions of practical soil problems.
- Geotechnical Engineering includes Soil Mechanics, Rock mechanics, soil engineering, rock engineering

Role of Soil Mechanics in Civil Engineering

- It is ultimate foundation material that supports the structure
- It is the most abundant building material
- Soil structure interaction needs to be studied for excavation, Earth retaining structures, etc
- Soil behaviour with respect to vibrations

Is it simple to study SOIL?

History of Soil Mechanics

- Modern discipline of soil mechanics began in 1925 by Karl Terzaghi
- Karl Terzaghi is known as FATHER OF SOIL MECHANICS
- He wrote a book Erdbaumechanik

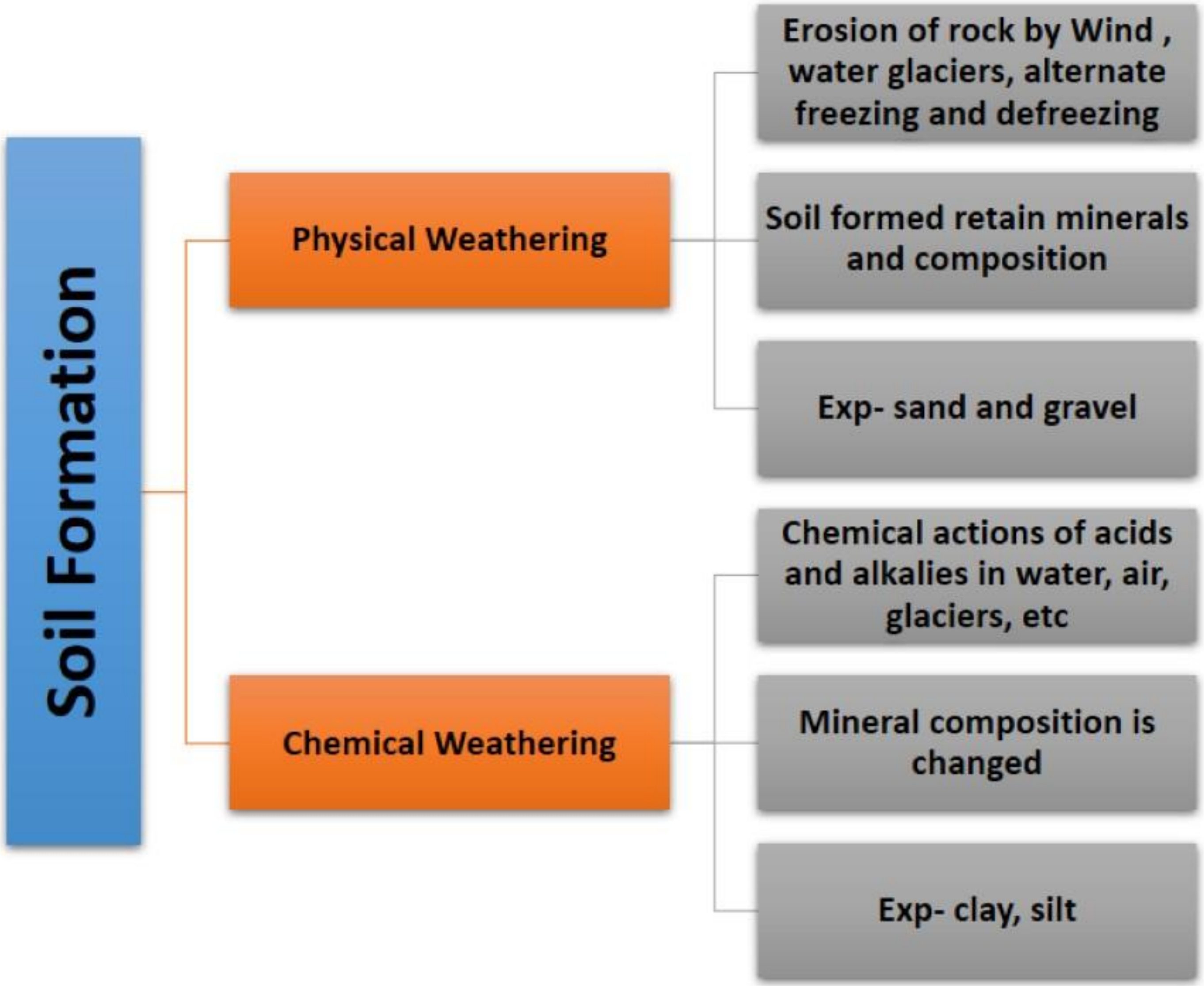
Origin of Soil and soil water relationships

- **Soil is composed of particles found from the disintegration of rocks.**

Soil = Rock + Organic Matter

- **Formation of Soil takes place by two methods:**

1. **Physical Weathering**
   - **The agencies responsible for physical weathering are the impact and grinding action of flowing water, ice, wind and splitting actions of ice, plants and animals**
2. **Chemical Weathering**
   - **Chemical weathering or decomposition of rocks is caused mainly by oxidation, hydration, carbonation and leaching by organic_ acids and water**


Soil Formation
Physical Weathering
Erosion of rock by Wind , water glaciers, alternate freezing and defreezing
Soil formed retain minerals and composition
Exp- sand and gravel
Chemical Weathering
Chemical actions of acids and alkalies in water, air, glaciers, etc
Mineral composition is changed
Exp- clay, silt

Soil Deposits

- ***Residual Soils***: Located at location of formation
- ***Transported Soil:*** Transported from parent location to a new location
  - **Alluvial deposits:** deposited by river
  - **Lacustrine deposit:** deposited by still water of lakes
  - **Marine deposit:** deposited by sea Water, exp Marl Clay
  - **Aeolian deposit:** deposited by wind , example: Loess
  - **Glacial deposit:** deposited by glaciers , example: drift, till
  - **Colluvial deposit:** transported by Gravity exp: Talus

Some important Soils

- **Bentonite clay:**
  - Has high percentage of montmorillonite
  - Highly plastic, high swelling and shrinkage
  - Formed due to volcanic ash, used as drilling mud
- **Black cotton soil:**
  - Contains high percentage of montmorillonite
  - Has high swelling and shrinkage potential
  - Has very low bearing capacity
  - Formed from chemical weathering of basalt

Some important Soils

- **Loam:** mixture of sand silt and clay, known as Garden soil
- **Indurated clay:** hardening of clay due to heat and pressure
- **Organic clays:** soil gets mixed with decomposed vegetation and dead and decayed matter
  - **Muck:** inorganic + organic matter
  - **Peat:** fully decomposed organic matter, highly compressible
  - **Humus:** Top soil, it contains partly decomposed organic material

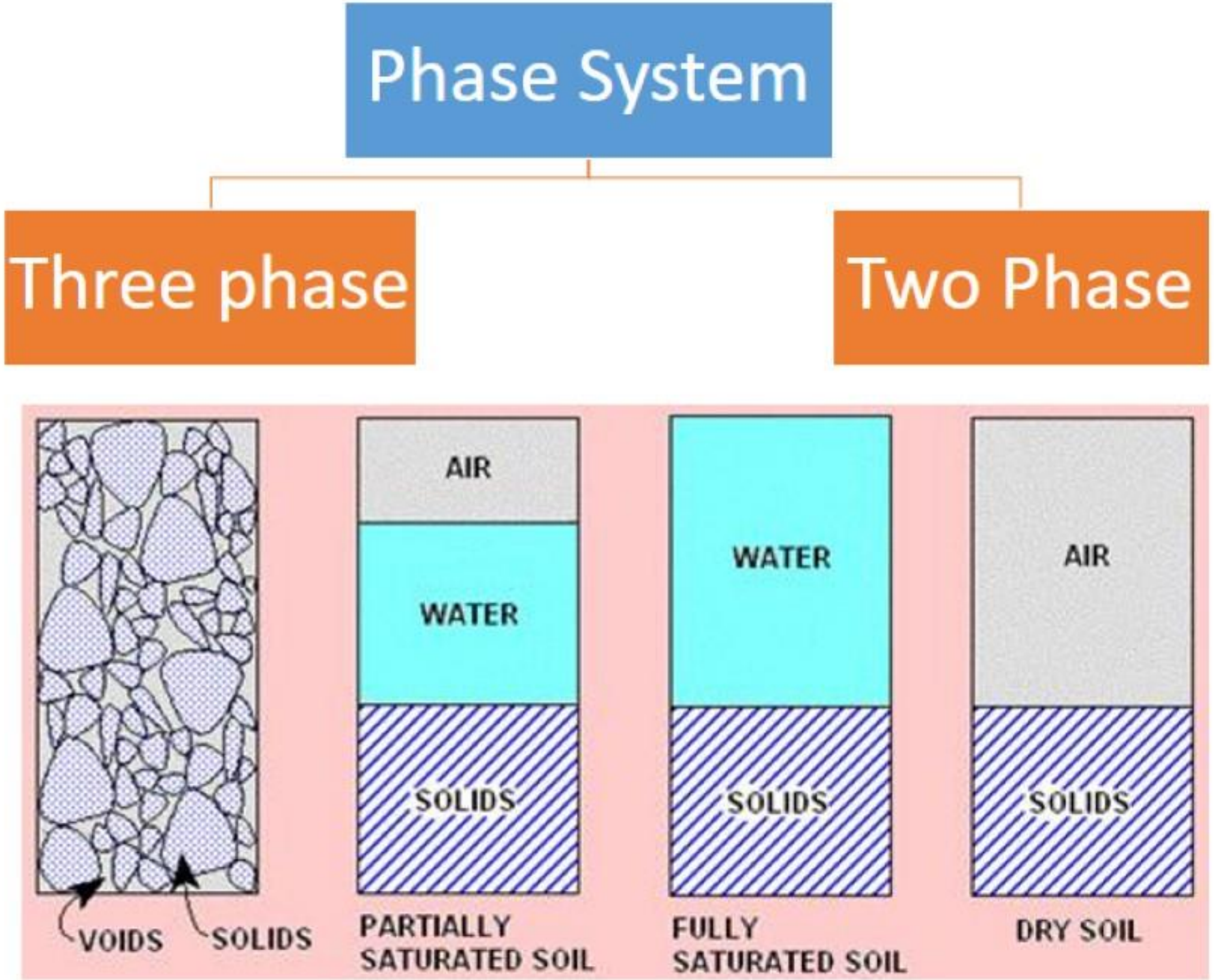
Some important Soils

- **Loess:** A loose deposit of wind-blown silt that has been weakly cemented with calcium carbonate and montmorillonite. Loess is formed in arid and semi-arid regions and stands in nearly vertical banks.
- **Tuff:** A small-grained, slightly cemented volcanic ash that has been transported by wind ,or water
- **Glacial till (boulder clay):** a mixture of boulders, gravel, sand, silt and clay, deposited by glaciers and not transported or segregated by water.
- **Varved clay:** Alternate thin layers of silt and clay deposited in fresh water glacial lakes by outwash from glaciers. The silt is· deposited in warm weather during heavy run off and clay is deposited in cold weather during small run off. Generally, one band of silt and clay is deposited each year.

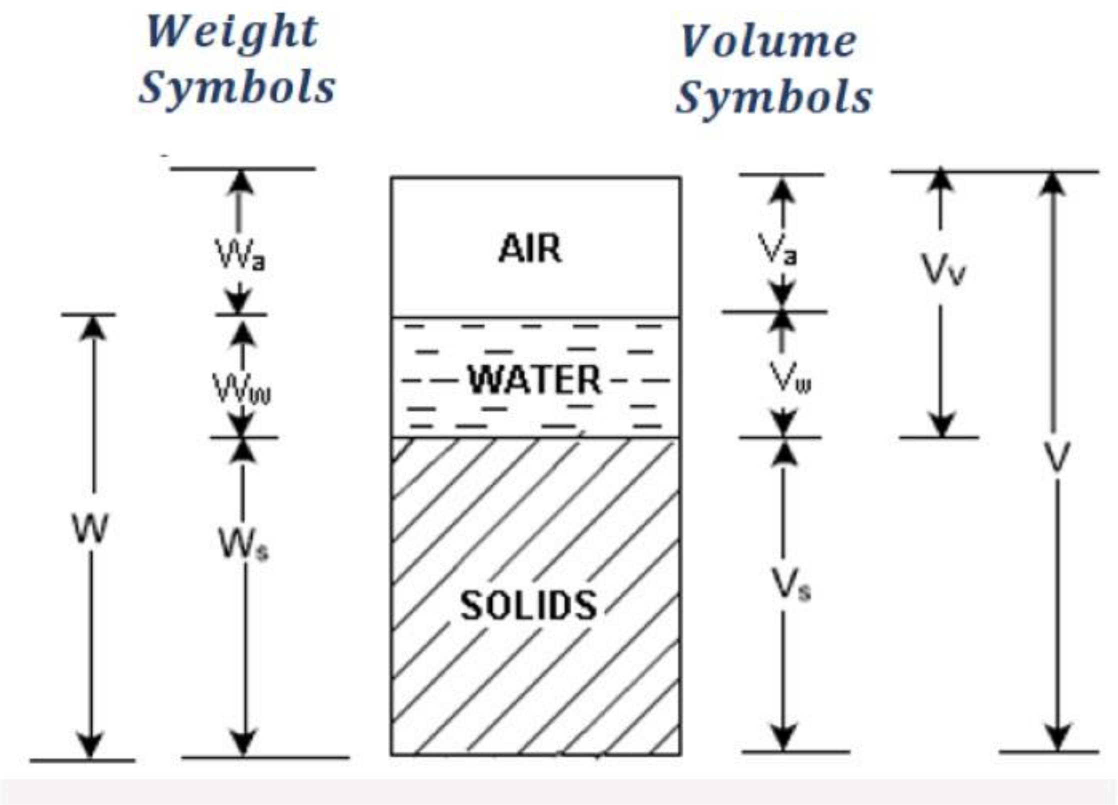
Some important Soils

- **Gumbo:** A sticky, plastic, dark coloured clay
- **Hard Pan:** A layer of extremely hard, cohesive soil that can hardly be drilled with ordinary, earth boring tools.
- **Colluvial Soil:** The accumulation of rock debris or talus at the base of a steep cliff or a rock escarpment. Its position results mainly from the effect of the force of gravity acting on the rock fragments broken from the rocks above.
- **Mine Tailings:** These are silt-sized materials resulting as waste after extraction of minerals from natural rock.

Soil Water Relationships



Soil Water Relationships



Soil Water Relationships

1. *Void ratio (e)* is the ratio of the volume of voids (V_v) to the volume of soil solids (V_s)

$$\boxed{e = \frac{V_V}{V_S}} \qquad \boxed{e = \frac{n}{1-n}}$$

2. *Porosity (n)* is the ratio of the volume of voids to the total volume of soil (V)

$$\boxed{n = \frac{V_V}{V}} \qquad \boxed{n = \frac{e}{1+e}}$$

3. *Degree of saturation (S)* The volume of water (Vw) in a soil can vary between zero (i.e. a dry soil) and the volume of voids. This can be expressed as the degree of saturation (S) in percentage.

$$\boxed{s = \frac{V_w}{V_V}}$$

Soil Water Relationships

4. ***Air content (a_c)*** is the ratio of the volume of air (V_a) to the volume of voids.

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

5. ***Percentage air voids (n_a)*** is the ratio of the volume of air to the total volume.

$$\boxed{n_a = \frac{V_a}{V}} \qquad \boxed{n_a = n \times a_c}$$

6. ***Water content (w):*** The ratio of the mass of water present to the mass of solid particles is called the water content (w), or sometimes the moisture content. Its value is 0% for dry soil and its magnitude can exceed 100%

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

Soil Water Relationships

7. True Specific Gravity (Gs):

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

8. Mass Specific Gravity (Gm):

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

- $G_s = \begin{cases} 2.6 - 2.7\ for\ Inorganic\ soils \\ 1.2 - 1.4\ for\ Organic\ Soil \end{cases}$

Soil Water Relationships

10. Relative Density:

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

Relative Density	Classification
<15	Very loose
15-35	Loose
35-65	Medium
65-85	Dense
>85	Very Dense

10. Relative Compaction

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

$$= \frac{\frac{G\gamma_w}{1+e}}{\frac{G\gamma_w}{1+e_{min}}}$$

$$\Rightarrow R_c = \frac{1+e_{min}}{1+e}$$

And $R_c = 80 + 0.2\,RD$

Unit Weight of Soil

a) ***Bulk Unit Weight (γ_t):*** **It is total weight by total volume of soil**

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

b) ***Dry Unit Weight of soil*** **(γ_d): It is weight of solids divided by total volume of soil**

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

c) ***Saturated Unit Weight*** **(γ_{sat}) of soil:**

$$\gamma_{sat} = \frac{W_w + W_s}{V}$$

$$\gamma_{sat} = \frac{\text{Wt of saturated soil}}{\text{Volume of soil}}$$

Unit Weight of Soil

d) *Submerged unit Weight of soil (γ_{sub}) :*

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

$$\gamma_{sat} = \frac{\text{Wt of saturated soil}}{\text{Volume of soil}}$$

e) *Unit Weight of Solids(γ_s):* It is weight of solids per unit volume of solids

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

f) *Unit weight of water(γ_w):* It is weight of water per unit volume of water

$$\gamma_w = \frac{W_W}{V_W}$$

Variation of Volume With respect to addition of water

Relation between different terms

1. $V_s = \frac{V}{1+e}$
2. $n = \frac{e}{1+e}$
3. $W_s = \frac{W}{1+w}$
4. $se = wG$
5. $\gamma_t = \frac{(G+es)}{1+e}\gamma_w$
6. $\gamma_d = \frac{G\gamma_w}{1+e}$
7. $\gamma_d = \frac{(1-n_a)G\gamma_w}{1+wG}$
8. $\gamma_d = \frac{\gamma}{1+w}$
9. $\gamma_{sub} = \gamma_{sat} - \gamma_w$
10. $n_a = a_c \times n$
11. $a_c = 1 - s$
12. $\gamma_{sat} = \frac{(G+e)}{1+e}\gamma_w$

Que 1. A sampler with volume of 45 cm^3 is filled with soil. Soil displaces water of 25cm^3 volume. What is the porosity and void ratio?

Que 2. The void ratio and specific gravity of a sample of clay are 0.73 and 2.7. If the voids are 92% saturated, find the bulk density, dry density and water content. What would be the water content for s=100% ?

Que 3. A soil sample has bulk density $19kN/m^3$ with a moisture content of 15%. Determine

a) **Degree of saturation of sample if $G_s = 2.7$**

b) **Additional moisture content required for saturating the soil sample**

c) **Find the void ratio at 18% and 25% water content.**

Que 4. A compacted cylindrical specimen 50mm dia and 100mm length is to be pulled from dry soil. If the specimen is required to have a water content of 15% and percentage of air voids is 20, find the weight of soil and weight of water required in the preparation of soil sample if $G_s = 2.69$

Que 5. A sampler has a porosity of 40%. The specific gravity of solids is 2.7. Find

a) **Void ratio**

b) **Dry Density**

c) **Unit weight of soil if it is 50% saturated**

d) **Unit weight of soil if it is completely saturated**

Que 6. In a proctor compaction test, the soil specimen of one of the observation has bulk density of $19\ kN/m^3$ with a moisture content of 15%. Find:

a) 's' of specimen if G=2.7.

b) Additional moisture required for saturating the soil specimen.

Que 7. A sample of sand above the water table is found to have natural moisture content of 0.15 or 15% and a unit weight 18.84 kN/m^3. Lab test on dried sample indicate value of 0.5 and 0.85 for minimum and maximum void ratio for dense and loose state.

If G=2.65, Find

a) s

b) Relative density

Que 8. A soil has a void ratio of 0.67, G=2.67, w =12%. Find out the volume of water required to be added to $100\,m^3$ of soil for full saturation.

Que 9. An Earthen dam will have volume $5 \times 10^6\ m^3$ of compacted soil. The soil is to be taken from a borrow pit and will be compacted to a void ratio of 0.80. The void ratio of soil in the borrow pit is 1.15. Estimate the volume of soil that must be excavated from the borrow pit for construction of above dam.

Que 9. An Earthen dam will have volume $5 \times 10^6\ m^3$ of compacted soil. The soil is to be taken from a borrow pit and will be compacted to a void ratio of 0.80. The void ratio of soil in the borrow pit is 1.15. Estimate the volume of soil that must be excavated from the borrow pit for construction of above dam.

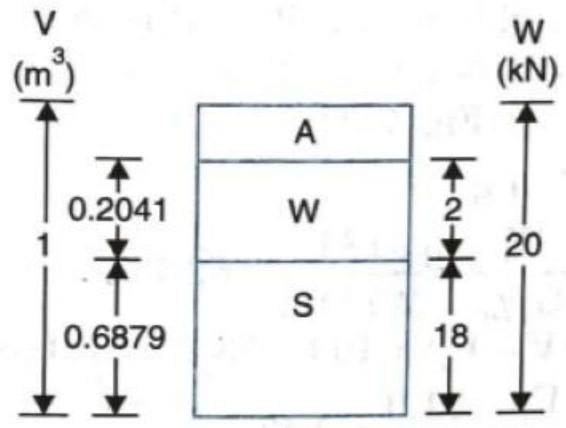
Que 10. A large earthen dam project that has a volume of $5 \times 10^6\ m^3$ is selected for soil fill is required to be compacted such that the final void ratio of earthen dam is 0.80. Earth fill from which one of the given 3 suppliers A,B,C will be most economical?

Supplier A → Soil fill @ Rs 16/ m^3

Supplier B → Soil fill @ Rs 13/ m^3

Supplier C → Soil fill @ Rs 15/ m^3

Que 1 cum of wet soil weighs 20 kN. Its dry weight is 18 kN/m^3 . Sp gravity of solids is 2.67. Determine the water content, porosity, void ratio and the degree of saturation.



Que 2. A soil sample with specific gravity of solids 2.70 has a mass specific gravity of 1.84. Assuming the soil to be perfectly dry, determine the void ratio.

Determination of Water content (w)

- Routine test used in Atterberg Limits, compaction test, consolidation tests, shear test, etc.
- Water content influences of soil

Methods of determination of Water content (w)

1. Oven Drying method
2. Pycnometer Method
3. Sand Bath method
4. Calcium Carbide Method
5. Torsion Balance Method
6. Radiation Method
7. Alcohol Method

Methods of determination of Water content (w)

1. Oven Drying method

- 105-110 °C (inorganic)
- 60°C (organic)
- 24 hours

$$\textit{Moist Soil}\ (W_1)$$

$$\Delta\ \textit{Heat} \quad \Downarrow \quad 24\ \textit{Hours},\ 105-110°\text{C}$$

$$\textit{Dry Soil}\ (W_2)$$

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

Methods of determination of Water content (w)

2. Pycnometer Method

w_1
Empty Pycnometer

w_2
Pycnometer +Wet Soil

w_3
Pycnometer + Wet Soil + water

w_4
Pycnometer + water

Methods of determination of Water content (w)

2. Pycnometer Method

Pycnometer- 900 ml glass bottle with a conical top

w_1 Empty Pycnometer | w_2 Pycnometer +Wet Soil | w_3 Pycnometer + Wet Soil + water | w_4 Pycnometer + water

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

$$Weight\ of\ water = (W_2 - W_1) - W_s$$

If we remove weight of solids from W_3 and replace by equal volume of water, we can obtain the weight W_4

$$\Rightarrow (W_3 - W_s) + \frac{W_s}{G_s\gamma_w}\gamma_w = W_4$$

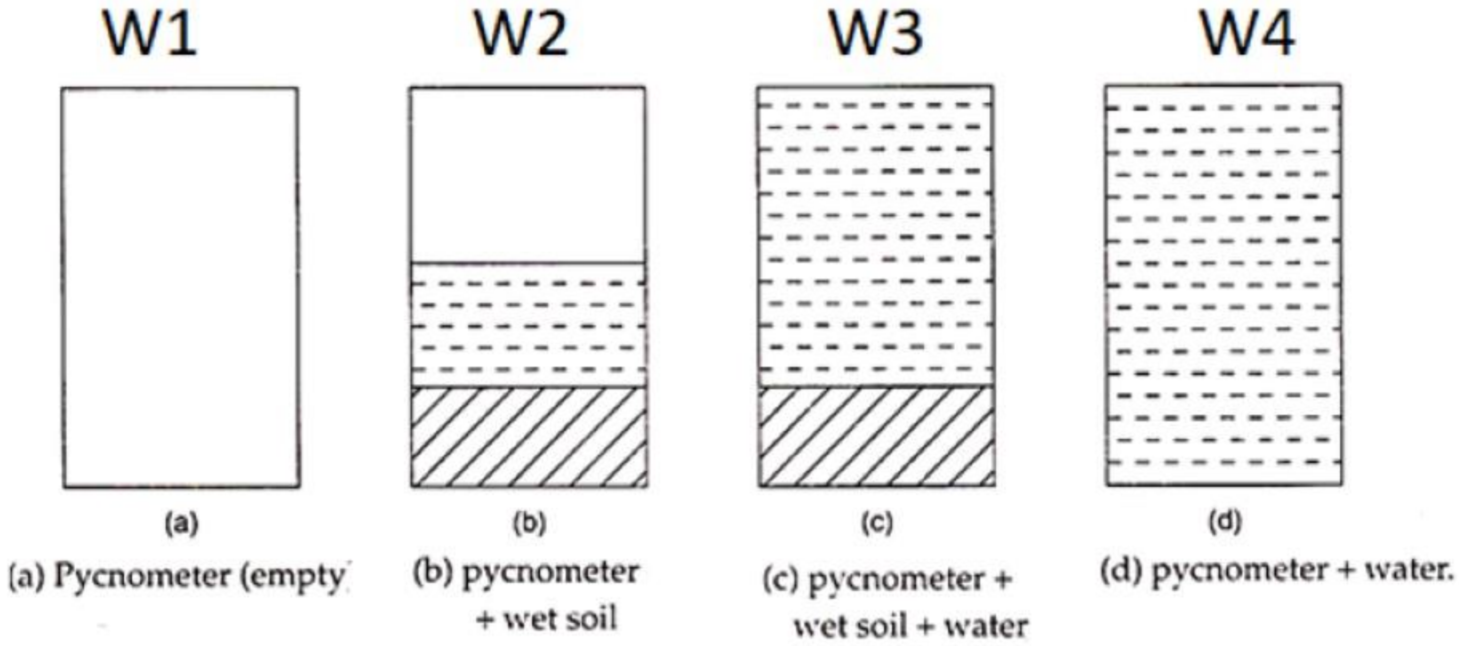
$$\Rightarrow W_s = (W_3 - W_4) - \frac{W_s}{G_s}$$

$$\Rightarrow W_s = (W_3 - W_4) - \frac{G_s}{G_s - 1}$$

$$\Rightarrow w = \left[\frac{W_2 - W_1}{W_3 - W_4}\left(1 - \frac{1}{G_s}\right) - 1\right]$$

Methods of determination of Water content (w)

2. Pycnometer Method



$$w = \left(\frac{W2 - W1}{W3 - W4}\right)\left(1 - \frac{1}{G}\right) - 1$$

$$G_s = \frac{(W2 - W1)}{(W2 - W1) - (W3 - W4)}$$

Methods of determination of Water content (w)

3. Sand Bath method

- It is quick Field Method when facility of electric oven is not available
- The sand bath is heated over a kerosene stove
- Sand bath container maintains temp of 105-110 ℃

Methods of determination of Water content (w)

4. Calcium Carbide Method/Rapid moisture meter method

- Water in soil reacts with calcium carbide and forms acetylene
- Pressure is co related with w/c

Calcium Carbide method

$CaC_2 + \text{Soil } (2H_2O) \Rightarrow Ca(OH)_2 + C_2H_2 \text{ (gas)}$

$$\boxed{CaC_2 + \text{Soil } (2H_2O) \Rightarrow Ca(OH)_2 + C_2H_2 \text{ (gas)}}$$

Methods of determination of Water content (w)

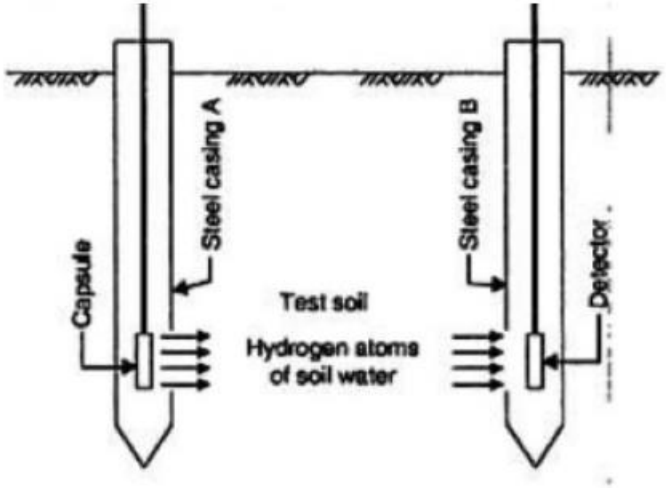
5. Torsion Balance Method

- Lab method
- Infrared radiations that has greater penetration of heat is used to dry the sample
- Used for soil which quickly absorbs moisture
- Equipment is costly

6. Radiation Method

- Radioactive isotope Cobalt 60 is used

Torsion Balance Method



Radiation Method

Cobalt 60 is used

Specific Gravity of Solids Determination

W_1
Empty Pycnometer

W_2
Pycnometer +Dry Soil

W_3
Pycnometer + Dry Soil + water

W_4
Pycnometer + water

Specific Gravity of Solids Determination

w_1 Empty Pycnometer

w_2 Pycnometer +Dry Soil

w_3 Pycnometer + Dry Soil + water

w_4 Pycnometer + water

$$Weight\ of\ dry\ soil\ W_s = (W_2 - W_1)$$

$$Weight\ of\ water\ in\ third\ case = (W_3 - W_2)$$

$$Weight\ of\ water\ in\ fourth\ case = (W_4 - W_1)$$

Weight of water having the same volume as that of solids

$$= (W_4 - W_1) - (W_3 - W_2)$$

$$= (W_2 - W_1) - (W_3 - W_4)$$

Specific Gravity of Soilds

$$\boxed{G_s = \frac{(W_2 - W_1)}{(W_2 - W_1) - (W_3 - W_4)}}$$

$$\boxed{G_{s\,27^\circ C} = G_{s\,T^\circ C}\frac{specific\ gravity\ of\ water\ at\ T^\circ C}{specific\ gravity\ of\ water\ at\ 27^\circ C}}$$

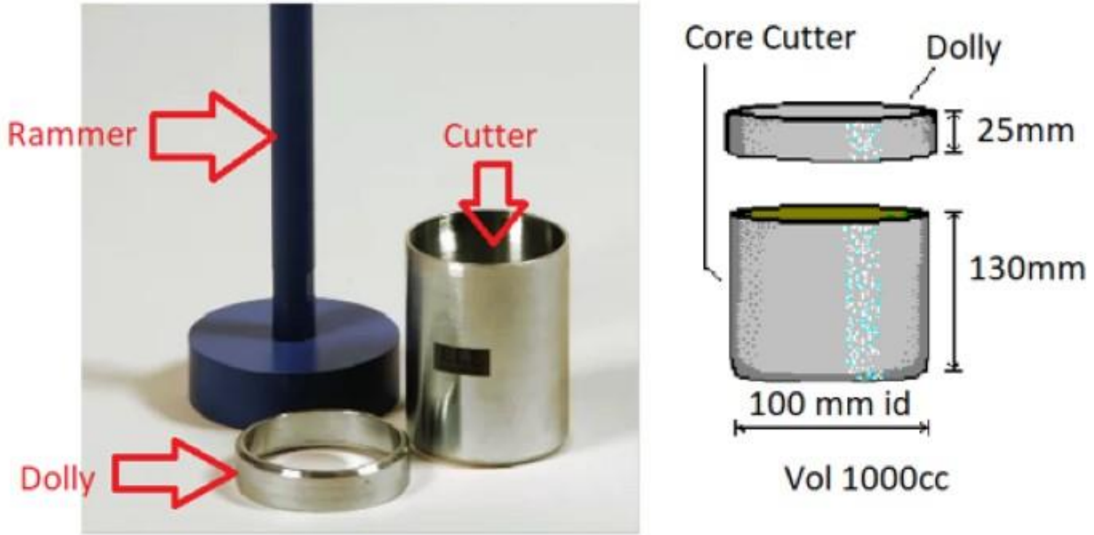
Methods of determination of Unit Weight (ɣ):

1. Core cutter method
2. Water Displacement Method
3. Sand Replacement Method
4. Water balloon method
5. Radiation method

Methods of determination of Unit Weight (γ):

1. Core cutter method

- Wt of empty cutter = W1
- Wt of Cutter+ soil= W2
- Wt of soil= W2-W1
- Vol of soil =1000cc
- Not suitable for hard or gravelly soils



Core Cutter Method

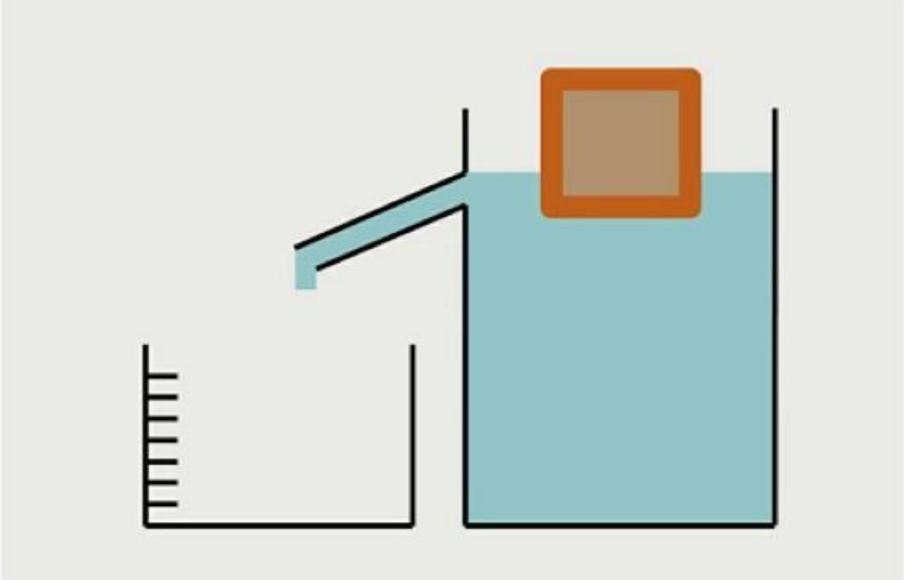
$$\boxed{\gamma_t = \frac{W}{V}}$$

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

Methods of determination of Unit Weight (γ):

2. Water Displacement Method

- This method is suitable for cohesive soil only where lump can be obtained
- Wt of sample = W1
- Wt of sample + wax = W2
- We need to find volume of sample
- Wax coating is done so as to put it in water and we can find the volume of Wax+ sample
- Then using 'G' of wax we find volume of Wax which is then subtracted from Total water displaced



Water Displacement Method

Que. A sample of clay coated with paraffin wax , its mass including the wax was found to be 697.5g. The sample was immersed in water and the volume of water displaced was 355ml. Mass of sample without wax is 690g. Water content of the specimen was 18%, find out

a) Bulk density

b) Dry density

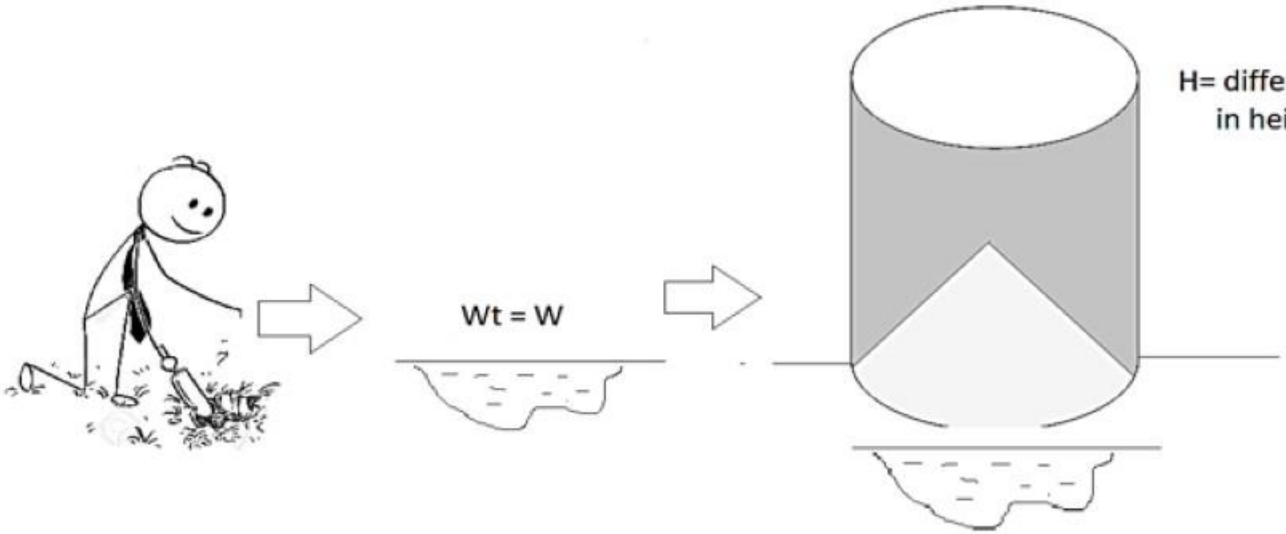
c) Void ratio

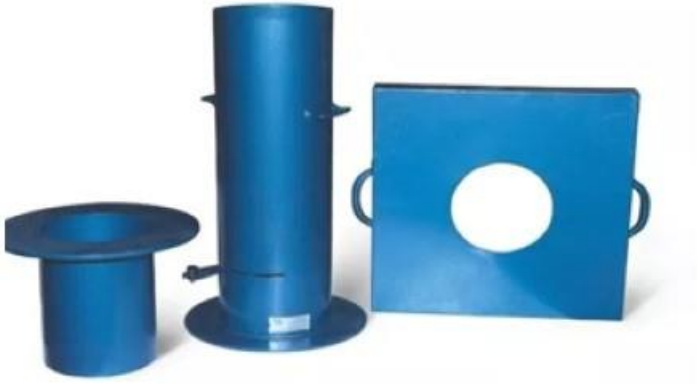
d) Degree of saturation

G_s of soil is 2.7 and G_{wax} is 0.89

Methods of determination of Unit Weight (γ):

3. Sand Replacement Method





Sand Replacement Method

Volume of soil displaced= vol of cone+ vol of Pit

Here we can find volume of Pit i.e volume of soil sample

- **Suitable for hard or gravelly soils**

Que. Calculate unit weight of soil if During the soil investigation, following observations were taken:

weight of excavated soil = 761.25 g

Weight of sand + weight of cylinder = 10500 g

Weight of sand + weight of cylinder after pouring = 9005 g

Weight of sand + weight of cylinder after pouring for cone only = 9450 g

Volume of calibrating can is 1000cc

Weight of calibrating can after pouring from cylinder =1550 g

Index Properties of Soil

Soil index properties are properties which help in identification and classification of soils for engineering purposes.

1. **Soil Aggregate Properties**
   - **Depends on soil mass as a whole**
2. **Soil Grain Properties**
   - **Depends upon soil grain size, shape, etc**

Grain Size Distribution

1. Helps to find gradation and uniformity
2. Separates out soil into different fractions based on particle size

Grain Size Distribution

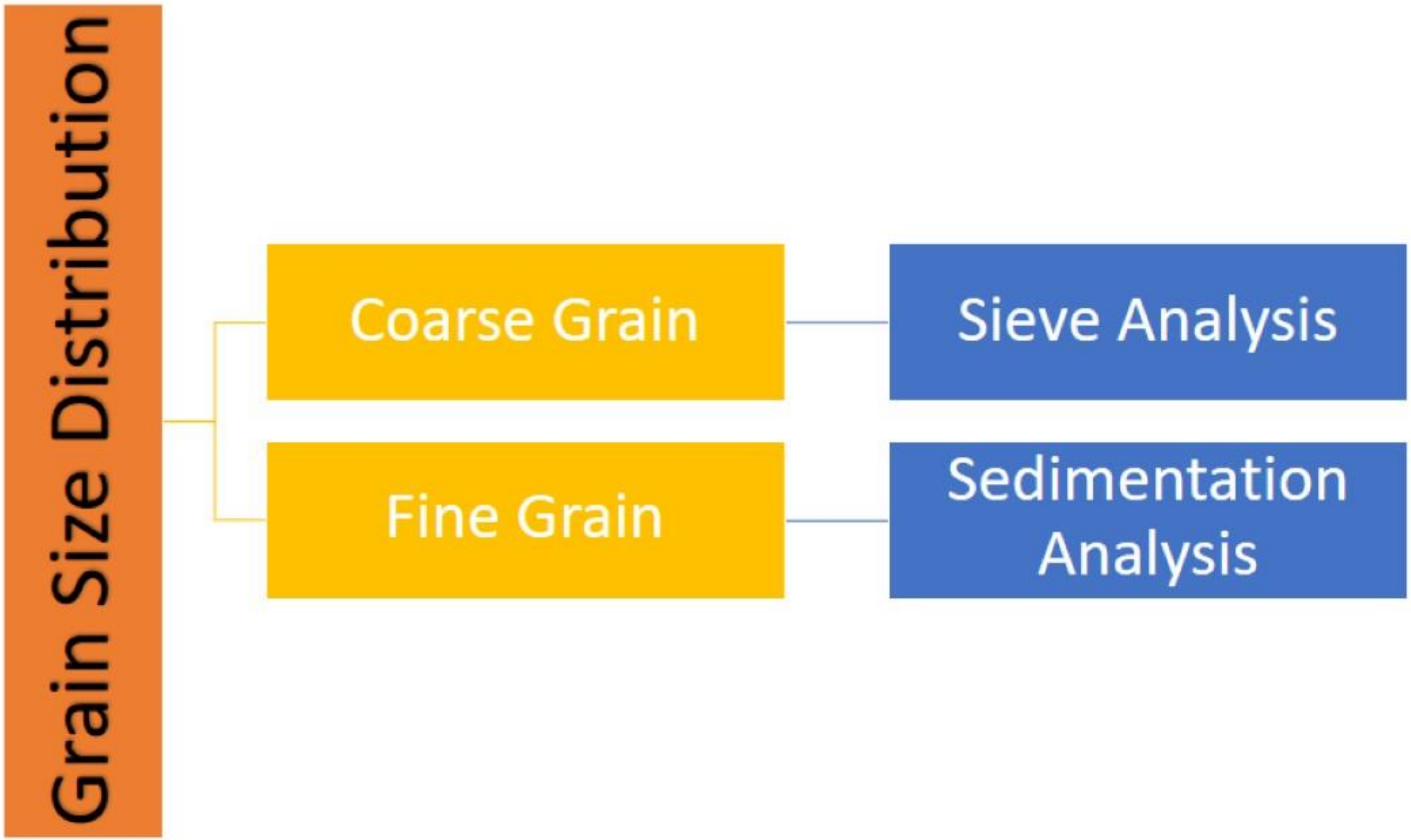
1. Helps to find gradation and uniformity
2. Separates out soil into different fractions based on particle size

Size	Soil
	Boulders
	Cobbles
	Gravel
	Sand
	Silt
	Clay

Grain Size Distribution

1. **Helps to find gradation and uniformity**
2. **Separates out soil into different fractions based on particle size**

Size	Soil
>300 mm	**B**oulders
80-300mm	**Co**bbles
4.75mm-80mm	**G**ravel
75µ - 4.75mm	**S**and
2µ -75µ	**S**ilt
less than 2µ	**C**lay



Grain Size Distribution
Coarse Grain
Fine Grain
Sieve Analysis
Sedimentation Analysis

Grain Size Distribution

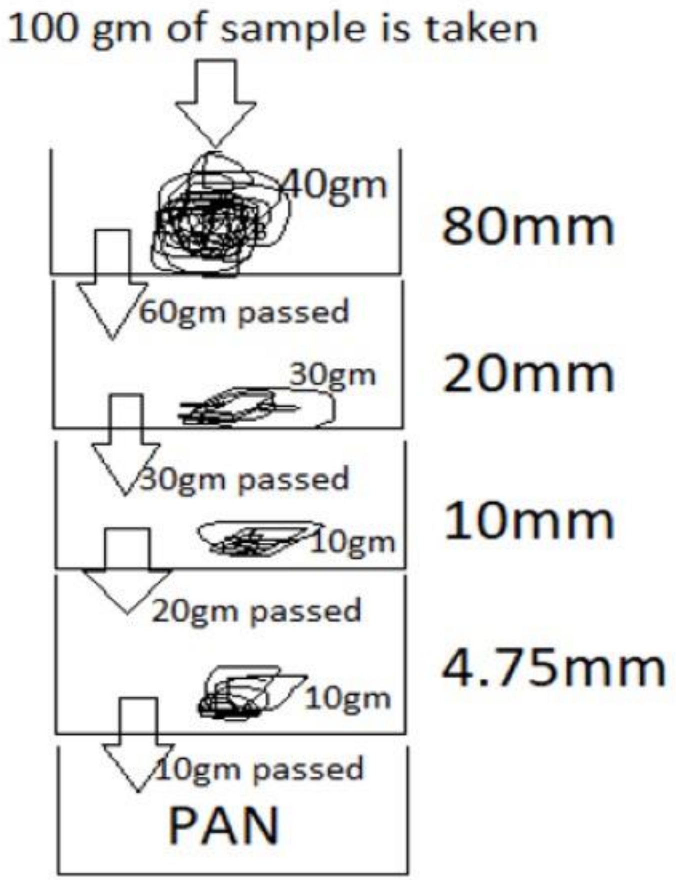
1. Sieve Analysis

- Sieves vary in size from ***80mm to 75μ***
- A set of sieves of size 80mm, 20mm, 10mm and 4.75 mm is required for further fractioning of gravel fraction
- The set of IS sieves for fine sieve analysis consists of 2mm, 1mm, 600 μ, 425μ, 212μ, 150μ and 75μ sieves

Grain Size Distribution

1. Sieve Analysis

Wt of sample = 100gm
Wt passed from 80mm dia sieve= 60gm
Wt of particles FINER than the size 80mm
= 60gm or 60% finer

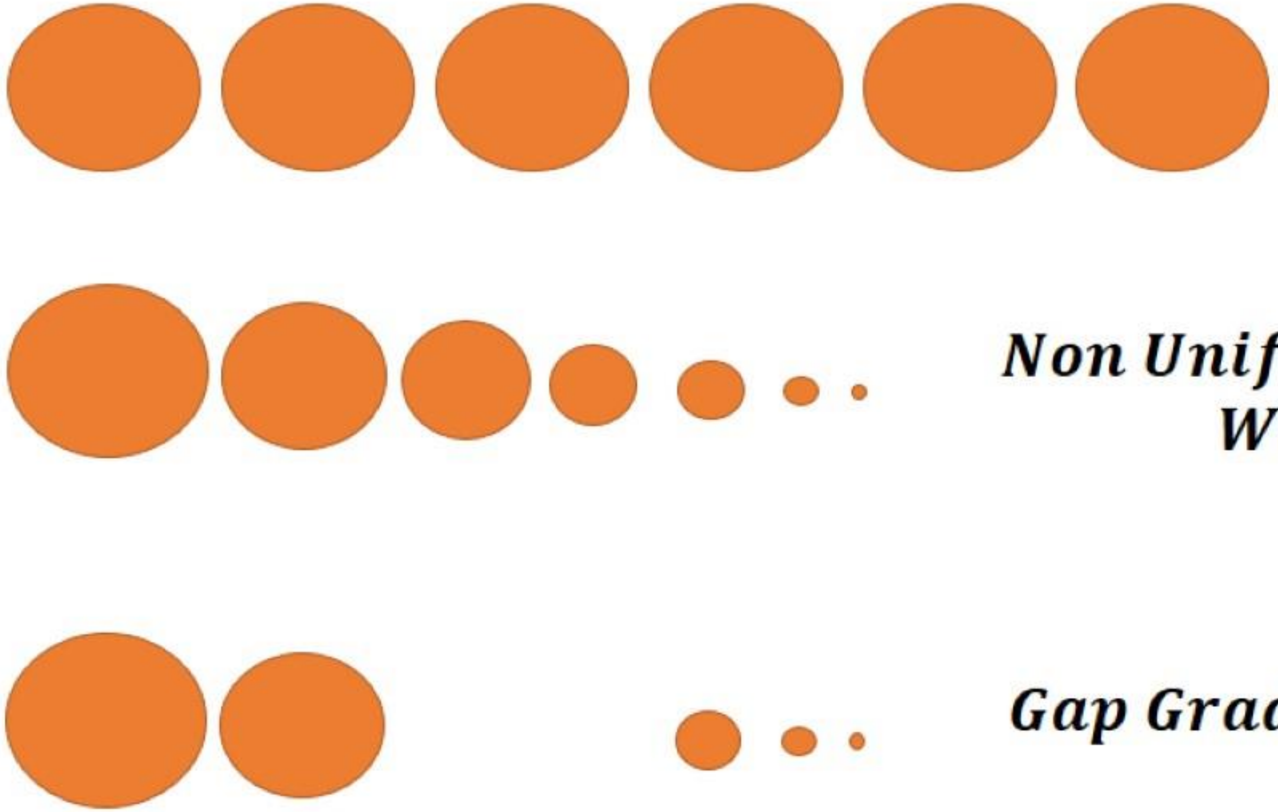


d_{60} = 60 % of the particles are finer than the size d = 80mm

d_{10} = 10 % of the particles are finer than the size d= 4.75mm

d_{10} = effective size

Grain Size Distribution



Uniformly Graded or Poorly Graded

Non Uniformly Graded or Well Graded

Gap Graded Soil

Note

- Effective size $= d_{10}$
- Uniformity coefficient $C_u = \frac{d_{60}}{d_{10}}$
- Curvature coefficient $C_u = \frac{d_{30}^2}{d_{60} \times d_{10}}$

Que. Total mass of sample is 900gm. Find coefficient of curvature and coefficient of uniformity.

IS sieve (mm)	Mass of soil retained (g)
20	35
10	40
4.75	80
2	150
1	150
0.6	140
0.425	115
0.212	55
0.15	35
0.075	25
Pan	75

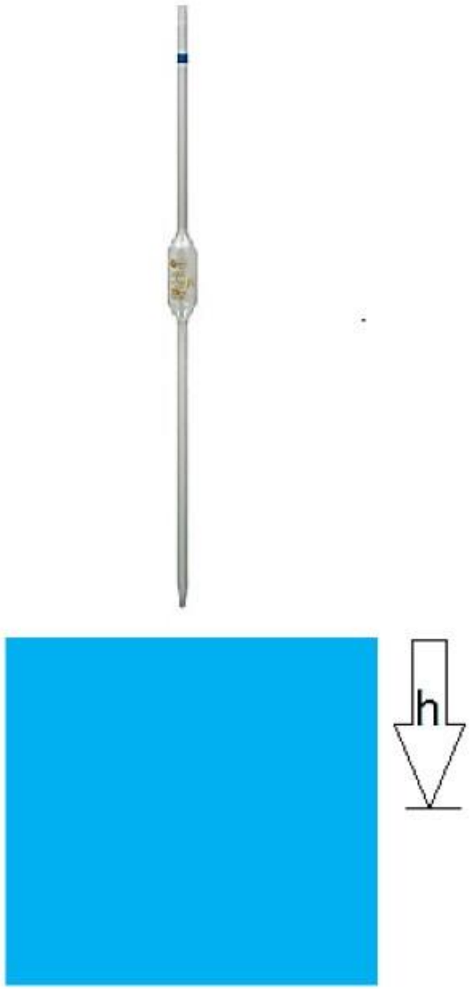
Que. Out of 500 g soil, 200g was retained on 600mm sieve, 250g was retained on 500mm sieve and remaining was retained on 425mm sieve. Find the coefficient of uniformity.

Sedimentation Analysis (Pipette Method)

- For particles less than size 75micron (clay and silt)

Based on Stokes' law which have following assumptions:

- Stoke's law is applicable for discrete particle settling (***sodium oxalate*** is added as deflocculating agent), so concentration kept is 50g per litre or less
- The particles are assumed to be of same spherical shape and have same specific gravity
- Particle size is from 0.0002mm to 0.2mm
- Valid only for Reynold's number less than 1



$$\boxed{\frac{H}{t} = V_t = \frac{(\gamma_s - \gamma_w)d^2}{18\mu}}$$

Sedimentation Analysis (Pipette Method)

- We suck out 10 or 20ml sample with pippete and collect in a sampler, from this we can calculate the dry weight of the sample , hence

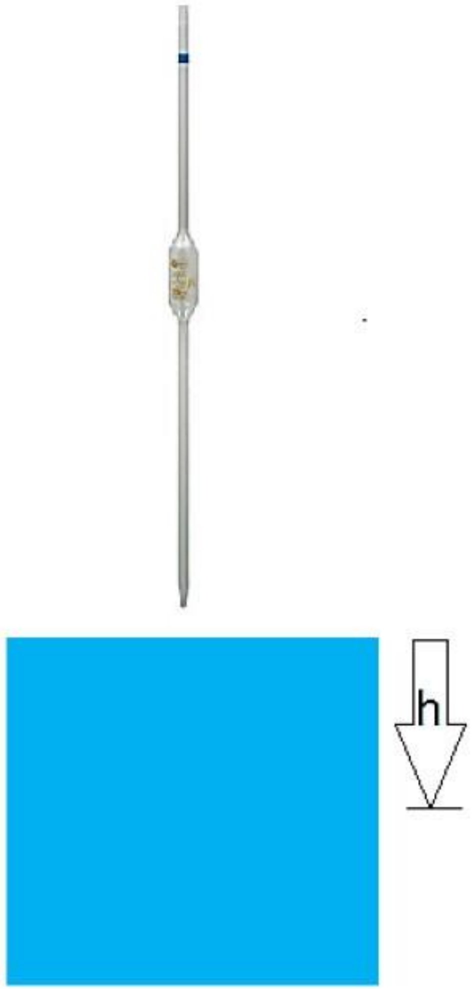
$$Concentration = \frac{dry\ weight}{pipette\ volume}$$

$$\boxed{\%\ \text{finer than 'd'} = \frac{\text{concentration at height h after time t}}{\text{Original concentration}} \times 100}$$

$$Where\ original\ concentration = \frac{original\ weight\ taken\ of\ soil}{Volume\ of\ solution\ prepared}$$

$$\boxed{\frac{H}{t} = V_t = \frac{(\gamma_s - \gamma_w)d^2}{18\mu}}$$

From here we can calculate the size of particle



Sedimentation Analysis (Pipette Method)

- <u>**Limitations of stokes law:**</u>
  - The analysis particles are assumed to be spherical but FINER particles are usually flaky or needly shaped
  - Stoke's law considers the velocity of free fall of single sphere in a suspension of infinite extent, whereas the grain size analysis is usually carried out in a glass jar in which the extent of liquid is limited
  - Average value of specific gravity of grains is used but it might be different for some grains
  - The grains may have charge on them and their behavior might not be discrete

Que. A sample of dry soil Gs= 2.68, weight 125g is uniformly dispersed in water to form one litre of suspension. Determine

a) Unit weight of soil suspension

b) In the above problem, if 10cm3 of the suspension is removed at a height of 20cm after a time of 2.5 minute.

Dry weight of sample withdrawn is 0.389g

Determine a single point of grain size distribution. $\mu = 8.36\ milli\ poise$

Sedimentation Analysis (Hydrometer Method)

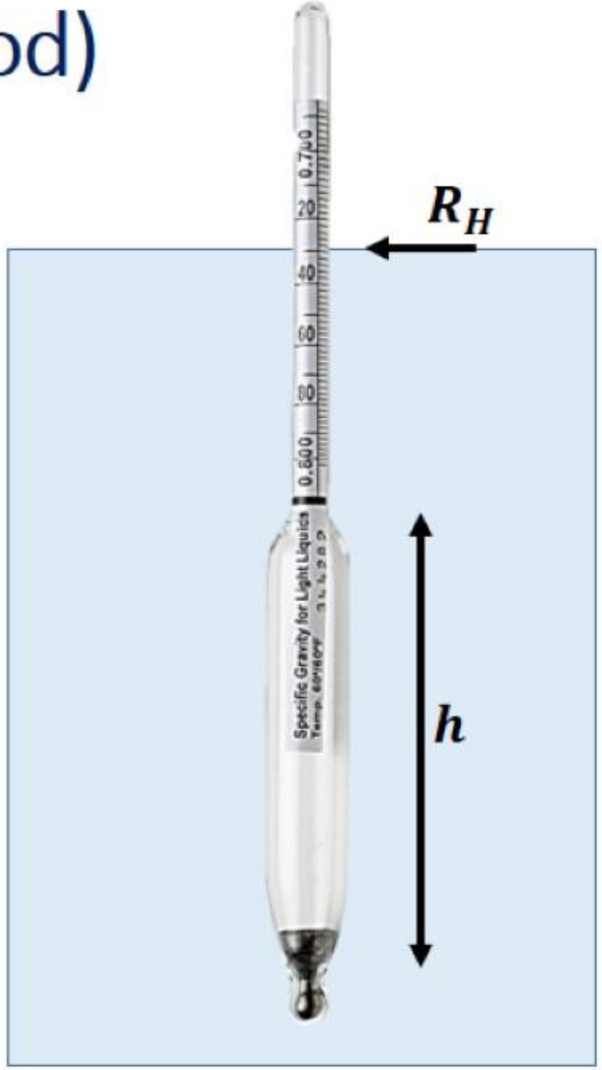
- In the **pipette method**, the **weight of solids per cc of suspension is determined** directly by collecting 10 cc of soil suspension from a specified sampling depth

whereas

- In ***Hydrometer method***, the weight of solids present at any time is calculated indirectly ***by reading the density of soil suspension***

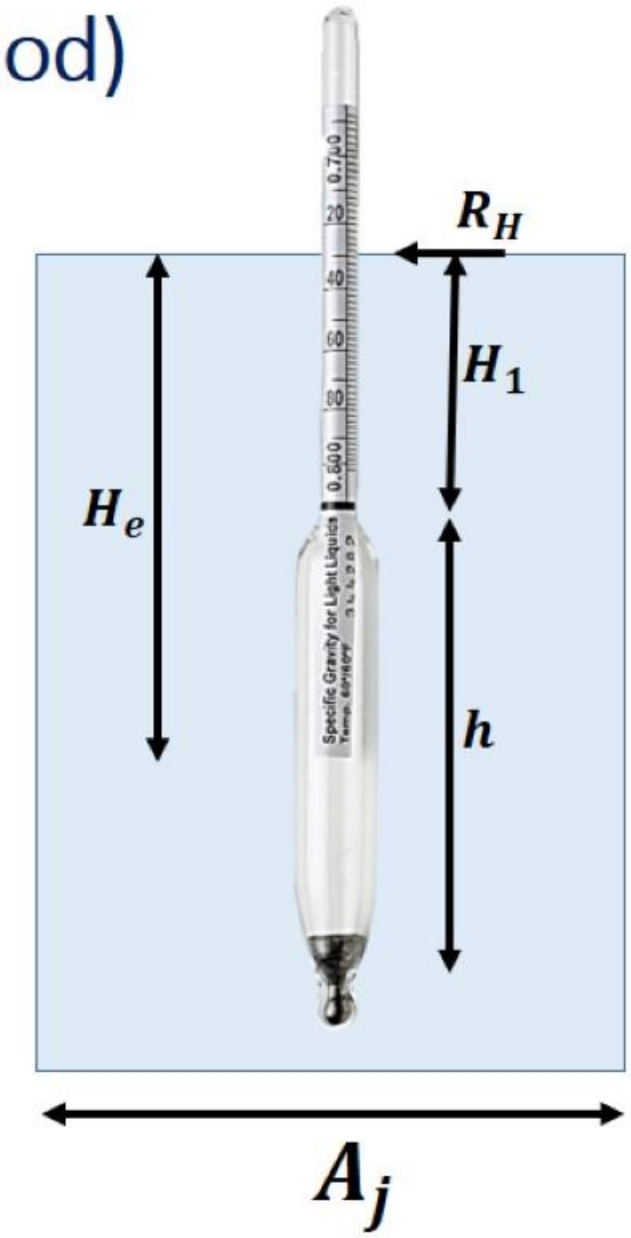
Sedimentation Analysis (Hydrometer Method)

- Effective depth is the distance from the surface of soil suspension to the level of at which the density of soil suspension is being measured
- The effective depth keeps on increasing as the particles settle with time
- The reading on the graduated stem of the hydrometer provides the density of the soil suspension at the centre of the bulb at any instant of time
- Hydrometer is calibrated in such a way that..
- $1 + \frac{R_H}{1000} = specific\ gravity\ of\ soil\ suspension$



Sedimentation Analysis (Hydrometer Method)

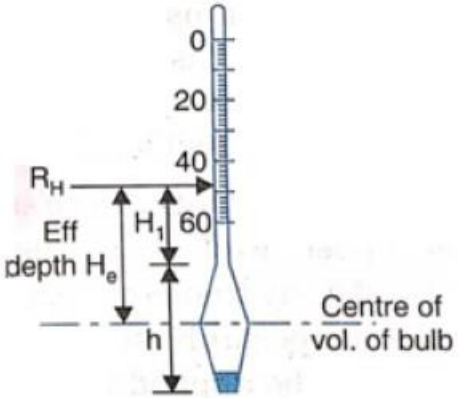
- Let h be the length and $\boldsymbol{V_H}$ the volume of hydrometer bulb and $\boldsymbol{H_1}$ be the distance in centimetre between any hydrometer reading $\boldsymbol{R_H}$ and the neck
- **If** $\boldsymbol{A_j}$ is area of cross section of jar/container, when hydrometer is immersed in jar, the surface of soil suspension rises by $\frac{V_H}{A_j}$
- Whereas the centre of hydrometer bulb is $\frac{V_H}{2A_j}$
- If the volume of hydrometer below the centre of the bulb is taken as approximately ½ of total volume….

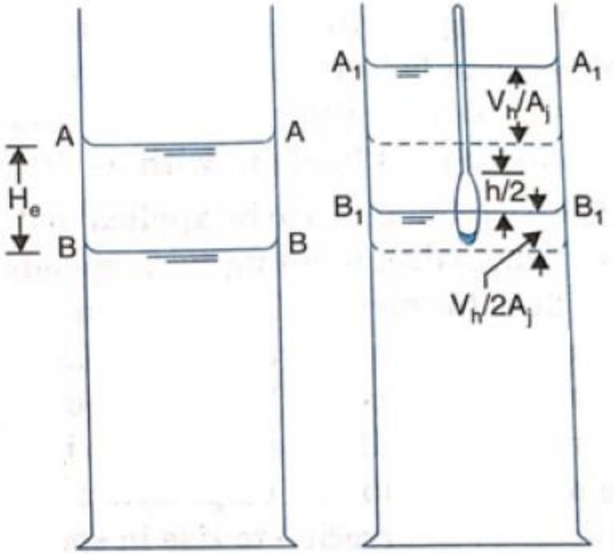


Sedimentation Analysis (Hydrometer Method)

- If the volume of hydrometer below the centre of the bulb is taken as approximately ½ of total volume, the effective depth $\boldsymbol{H_e}$ below the original surface ---

$$H_e = \left(H_1 + \frac{h}{2} + \frac{V_H}{2A_j}\right) - \frac{V_H}{A_j}$$



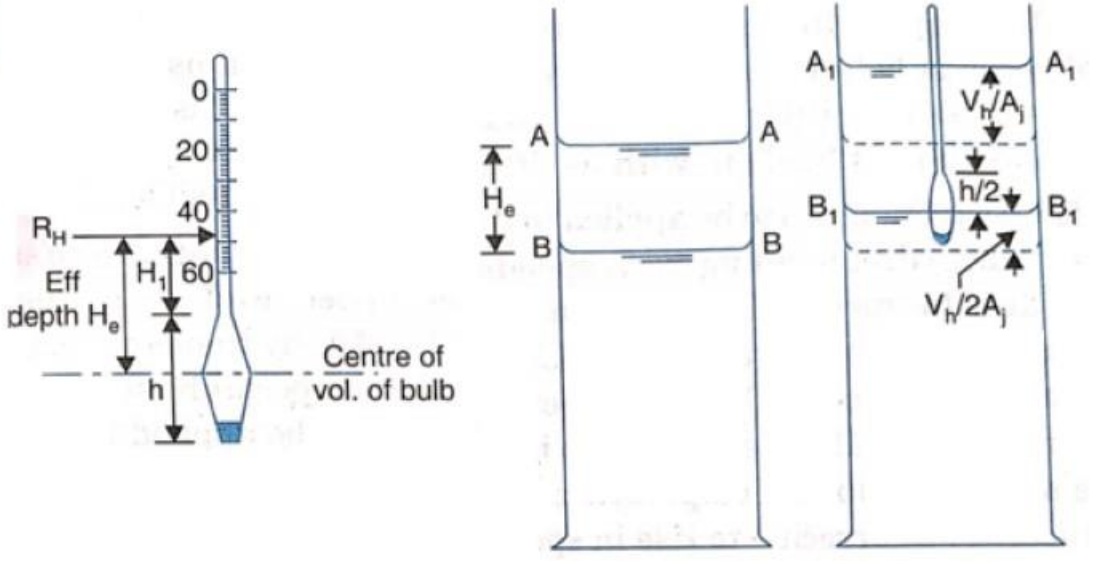


<u>Corrections required for Hydrometer:</u>

1. Meniscus correction which is always positive
2. Temperature correction (if temp is more than 27, then positive correction else negative)
3. Correction for deflocculating agent will be negative as deflocculating agent increases density

Sedimentation Analysis (Hydrometer Method)

The hydrometer reading is used for finding out the percentage finer as follows:



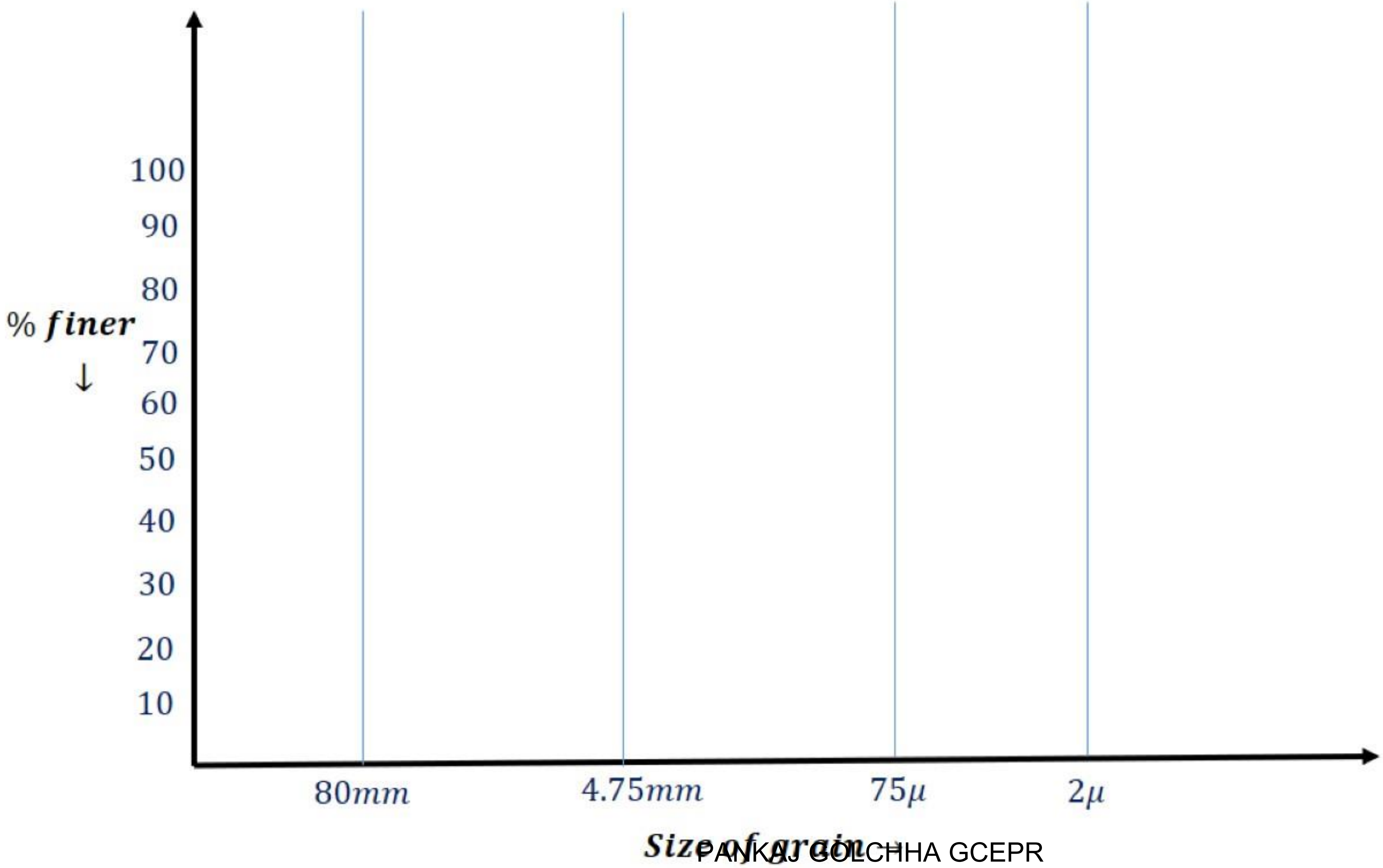
$$\% \, finer \, than \, d = \frac{concentration \, after \, time \, t}{initial \, concentration}$$

Specific Gravity of soil suspension

$$G_{suspension} = \frac{density \, of \, soil \, suspension}{density \, of \, water}$$

$$\Rightarrow G_{suspension} = \frac{density \, of \, water + immersed \;\; wt \, of \, solids \, per \, unit \, volume \, of \, solution}{density \, of \, water}$$

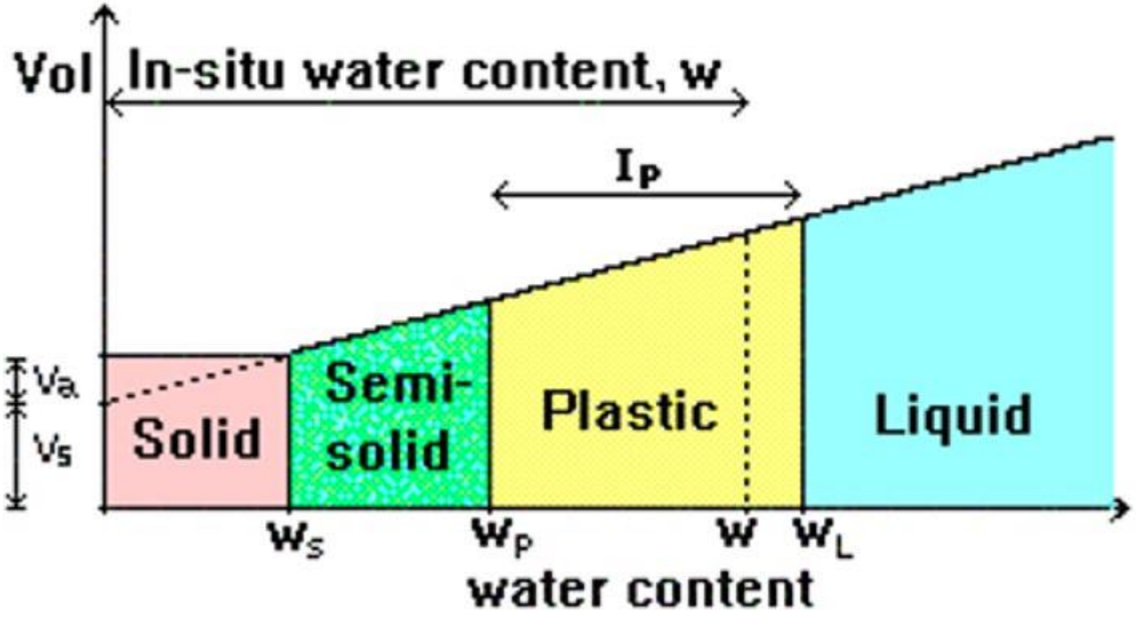
Result of GRAIN SIZE DISTRIBUTION is represented on a curve

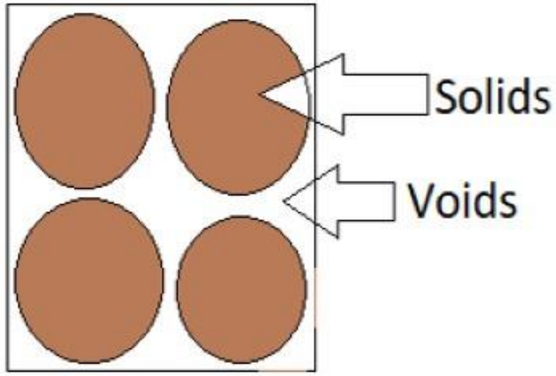


4.75mm-80mm	**G**ravel
75μ - 4.75mm	**S**and
2μ -75μ	**S**ilt
less than 2μ	**C**lay

Consistency

- The consistency of a fine-grained soil refers to its firmness, and it varies with the water content of the soil.
- Used mainly for fine soils
- Water content influences consistency of soil





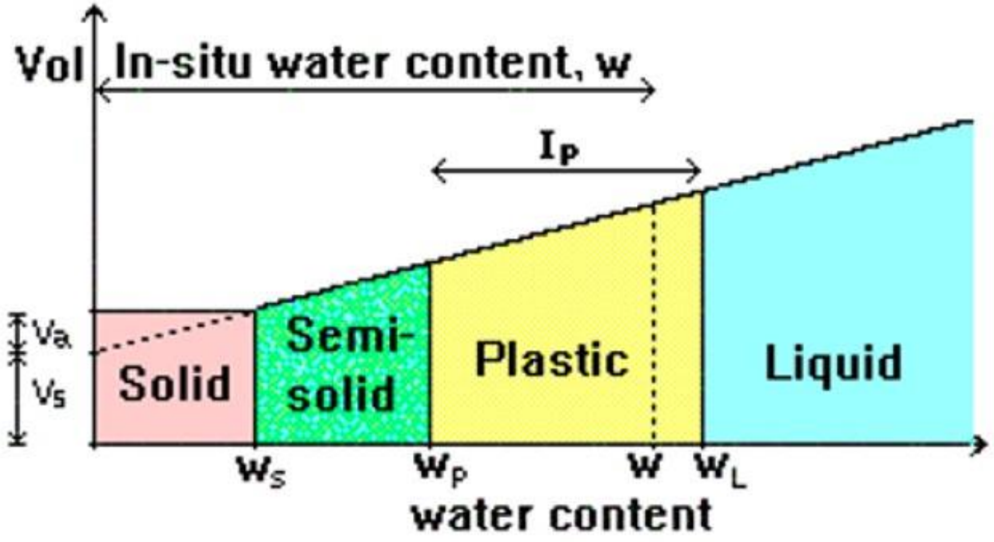
Consistency Limits/ Atterberg Limits

- **The three limits are known as the shrinkage limit (W_S), plastic limit (W_P), and liquid limit (W_L) as shown. The values of these limits can be obtained from various methods**
- ***Liquid limit (W_L)*** **- change of consistency from plastic to liquid state**
  Plastic limit (W_P) **- change of consistency from brittle/crumbly to plastic state laboratory tests.**
- **The difference between the liquid limit and the plastic limit is known as the** ***plasticity index (I_P)***

Consistency Limits

1. Liquid Limit

- It is boundary water content between plastic state and liquid state
- Shear strength is almost 0
- At liquid limit, all soils have nearly same shear strength (2.7 kN/m2)



Methods to find Liquid Limit

1. Casagrande Method
   - When Groove of 2mm width at bottom, 8mm depth and 11mm width on top is filled with 25 No. of blows, then that water content is called Liquid limit
   - Two types of grooving tools are used:
     1. Casagrande Knife for Fine soils

     2. ASTM tool for Sandy soils

Methods to find Liquid Limit

1. Casagrande Method

- Number of blows to block the groove are noted and plotted on the graph→ water content v/s logN
- The slope of the line is called as "Flow index"
- Flow index indicates loss of shear strength upon increase in water content
- $I_F \propto \frac{1}{Shear\ Strength}$

Methods to find Liquid Limit

1. Casagrande Method

- If in place of Casagrande rubber pad, harder rubber is used, then liquid limit reported will be LESS and if Softer rubber pad is used then liquid limit reported will be MORE.

- Liquid limit can also be calculated by empirical formula

$$w_L = w_N \left(\frac{N}{25}\right)^x$$

$w_N = water\ content\ at\ which\ N\ no.\ of\ blows\ are\ required$

$x\ 0.068 - 0.121$

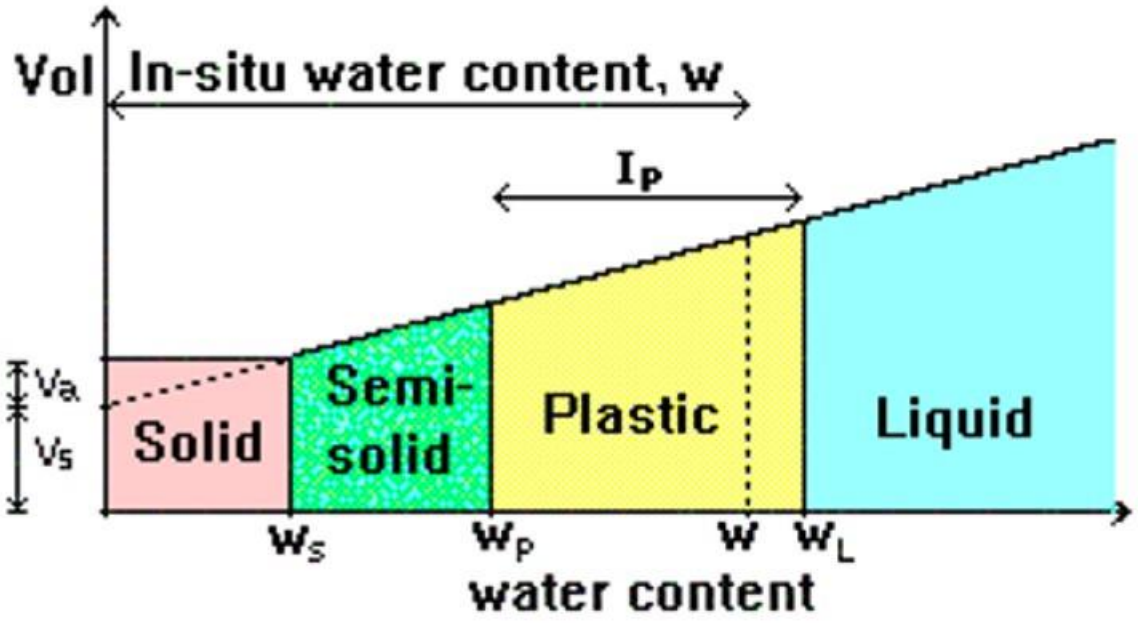
Methods to find Liquid Limit

2. Cone penetrometer Method

- 30 sec – 25 mm
- That water content is liquid limit

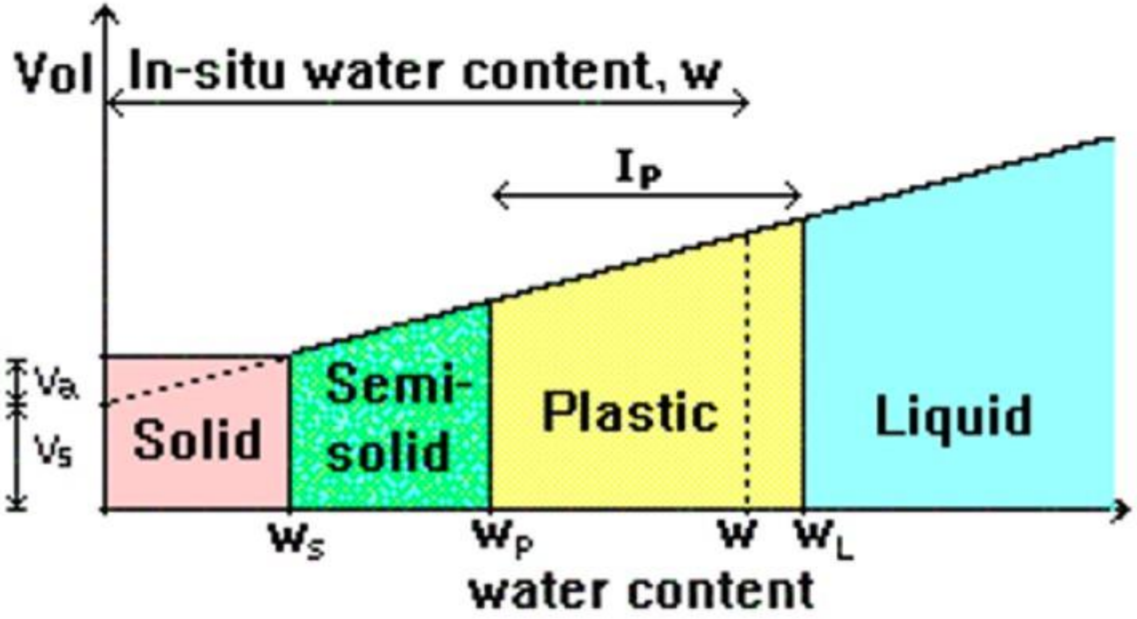
PLASTIC LIMIT

- It is boundary water content between plastic state and semi solid state
- At plastic limit water content, soil when rolled into a thread of 3mm diameter just begins to crumble (surface cracks start to appear)
- Fine soil has greater plasticity index and greater liquid limit and higher value of plasticity index
- Shear strength at plastic limit is nearly equal to 100 times that at liquid limit



SHRINKAGE LIMIT

- It is boundary water content between Solid state and semi solid state
- It is water content at which soil is saturated i.e. s=1
- If water content is reduced below shrinkage limit, volume of soil will not decrease



SHRINKAGE RATIO

Que. If mass specific gravity of a fully saturated clay having water content 0.40 is 1.88, on oven drying the mass specific gravity drops to 1.74. Calculate specific gravity of clay and its shrinkage limit.

Some Important Formulae

1. Consistency Index $I_c = \frac{w_L - w_N}{w_L - w_P}$

2. Liquidity Index $I_L = 1 - I_c$

3. Activity $A_c = \frac{I_P}{\% \; of \; clay \; sized \; particles}$

Liquidity index	Classification
> 1	Liquid
0.75 - 1.00	Very soft
0.50 - 0.75	Soft
0.25 - 0. 50	Medium stiff
0 - 0.25	Stiff
< 0	Semi-solid

Classification as per activity is:

Activity	Classification
< 0.75	Inactive
0.75 - 1.25	Normal
> 1.25	Active

Some Important Formulae

4. Flow Index $I_T = \dfrac{w_1 - w_2}{\log (N_1/N_2)}$

5. Toughness Index $I_T = \dfrac{I_P}{Flow\ Index}$

6. Shrinkage Ratio $= \dfrac{\frac{V_2 - V_1}{V_d}}{w_2 - w_1}$

$$\text{Shrinkage ratio} \quad SR = \frac{\gamma_d}{\gamma_w}$$

7. Sensitivity

$$S_t = \frac{\text{UCS of undisturbed sample}}{\text{UCS of remoulded sample}}$$

Thixotrophy

- Property by virtue of which loss of shear strength on remolding can be regained if soil is left undisturbed for sometime. The increase in shear strength is due to regain of chemical equilibrium and reorientation of water molecules.

Unconfined Compressive Strength

- Unconfined Compressive strength is defined as the load per unit area at which an unconfined prismatic or cylindrical specimen of standard dimensions of a soil fails in a simple compression test
- It is twice the value of a shear strength of a clay soil under drained condition

10. The following properties of the soil were determined by performing test on clay sample.

Natural Moisture Content=25%

Liquid Limit=32%

Plastic Limit =24%

Dia of 60% size = 0.006mm

Determine the liquidity coefficient , Uniformity coefficient and Relative Consistency.

Que 11. A silty clayey soil sample has silt particles 70%, liquid limit 50% and plastic limit as 40%. Find out activity, liquidity index, Consistency Index, Plasticity Index when $w_n = 45\%$.

Que 12. If plastic limit of soil is 25% and liquid limit of soil is 33%. When the soil is dried from plastic limit, its volume change is 25% of its volume at plastic limit. Similarly , the corresponding change in volume for liquid limit to dry state is 34% of its volume at liquid limit. Determine the shrinkage limit and shrinkage ratio.

CLASSIFICATION OF SOIL

- *USCS*-The Unified Soil Classification System
- *AASHTO* Soil Classification System (American Association of State Highway and Transport Officials)
- *ISSCS-* Indian Standard Soil Classification System

Unified Soil Classification System

- Unified Soil classification was done by
  A. Casagrande
- Based on Grain size distribution

Unified Soil Classification System

- **Coarse grained soils** (50% or more retained on 0.075mm sieve)
  a) Well graded
  b) Poorly graded
- **Fine grained soils** (more than 50% passing 0.075mm sieve)
  a) Silt (M)
  b) Clay (C)
  c) Organic soils

Unified Soil Classification System

SOIL TYPE	SUB GROUP
Gravel (G)	Well graded (W)
Sand (S)	Poorly graded (P)
Silt (M)	Silty (M)
Clay (C)	Clayey (C)
Organic (O)	Liquid limit < 50% (L)
Peat (Pt)	Liquid Limit > 50% (H)

AASHTO Soil Classification System

- Classified into 8 groups
  a) A-1 to A-7 with an additional group A-8.
  b) Evaluated according to Group Index.
  c) $G.I = 0.2a + 0.005ac + 0.01bd$
    - Where a = that part of the % passing the 75µ sieve greater than 35 and not exceeding 75 (range 1 to 40)
    - b = that part of the % passing the 75µ sieve greater than 15 and not exceeding 55 (range 1 to 40)
    - c = that part of liquid limit greater than 40 and not greater than 60 (range 1 to 20)
    - d = that part of plasticity index greater than 10 and not greater than 30 (range 1 to 20)

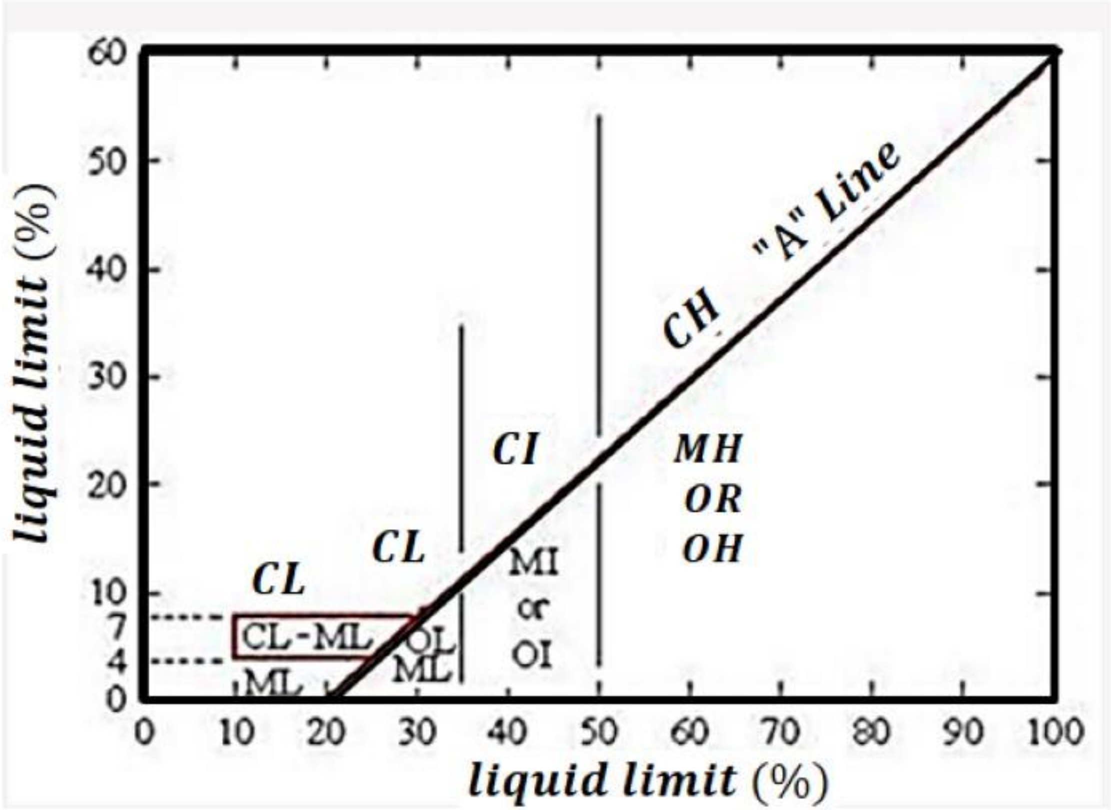
Indian Standard Soil Classification System

SOIL TYPE	SUB GROUP
Gravel (G)	Well graded (W)
Sand (S)	Poorly graded (P)
Silt (M)	Silty (M)
Clay (C)	Clayey (C)
Organic (O)	$w_L < 35$ % (L) $35 < w_L < 50$ (I)
Peat (Pt)	$w_L > 50$ %(H)

Indian Standard Soil Classification System

A Plasticity chart based on the values of liquid limit (w_L) ***and plasticity index*** (I_P)

The A-line in this chart is expressed as $I_P = 0.73(w_L - 20)$



Low plasticity	$W_L < 35\%$
Intermediate plasticity	$35\% < W_L < 50\%$
High plasticity	$W_L > 50\%$

ML = Silt with low plasticity

MI = Silt with intermediate plasticity

MH = Silt with high plasticity

CL = Clay with low plasticity

CI = Clay with medium plasticity

CH = Clay with high plasticity

CLASSIFICATION ON THE BASIS OF % FINES

%fines = percentage particles less than 75μ (silt + clay)

1. When % fines < 5%

GW	GP	SW	SP
More than 50% particles are greater than 4.75mm	More than 50% particles are greater than 4.75mm	More than 50% particles are in between 4.75mm - 75μ	More than 50% particles are in between 4.75mm - 75μ
$1 < C_c < 3$ $C_u > 4$	Otherwise	$1 < C_c < 3$ $C_u > 6$	Otherwise

%fines = percentage particles less than 75μ (silt + clay)

2. When % fines > 12%

GC	GM	SC	SM
More than 50% particles are greater than 4.75mm	More than 50% particles are greater than 4.75mm	More than 50% particles are in between 4.75mm - 75μ	More than 50% particles are in between 4.75mm - 75μ
$I_p > 7$	$I_p < 4$	$I_p > 7$	$I_p < 4$
Clayey Gravel	Silty Gravel	Clayey Sandy	Silty Sandy

%fines = percentage particles less than 75μ (silt + clay)

2. When % fines are between 5 to 12%
   a) In this case we use dual symbol
   b) For example:
      - 5% - 12%
      - GW – GC
      - GW – GM
      - SW – SM
      - SP – SC

Que. Out of 500gm of soil used in test, 200gm of soil was retained on 600mm sieve, 250gm was retained on 500mm sieve and remaining 50gm was retained on 425mm sieve.

a) find out the coefficient of uniformity

b) classify the soil as – SP, SW, GP OR GW.

Sieve	Wt of Passing	% Passing
600mm	300gm	60%
500mm	50	10%
425mm	0	0

$$\Rightarrow C_u = \frac{d_{60}}{d_{10}} \quad \Rightarrow C_u = \frac{600}{500} \quad \Rightarrow C_u = 1.2$$

$$Since\ 50\% > 4.75mm\ and\ C_u < 4$$

$$Therefore, GP$$

GROUP INDEX

- $G.I = 0.2a + 0.005ac + 0.01bd$
  a) Where a = that part of the % passing the 75μ sieve greater than 35 and not exceeding 75
     - If 50% particles < 75μ , then a = 50 – 35 = 15%
  b) b = that part of the % passing the 75μ sieve greater than 15 and not exceeding 55
  c) c = that part of liquid limit greater than 40 and not greater than 60
  d) d = that part of plasticity index greater than 10 and not greater than 30
- G.I generally varies from 0 to 20
- Greater the value of G.I., less durable the soil is for highway construction

GROUP INDEX TEST

- **Group index of soil subgrade**
  - Group index value range of different soils is given below

1. For good soil – 0 to 1
2. For fair soil – 2 to 4
3. For poor soil – 5 to 9
4. For very poor soil – 10 to 20

Que. If the proportion of soil passing through 75µ sieve is 50%, liquid limit and plastic limit are 40% and 20% respectively. Calculate the Group Index .

- **Where a = that part of the % passing the 75µ sieve greater than 35 and not exceeding 75**
  - **If 50% particles < 75µ , then a = 50 – 35 = 15%**
- **b = that part of the % passing the 75µ sieve greater than 15 and not exceeding 55**
- **c = that part of liquid limit greater than 40 and not greater than 60**
- **d = that part of plasticity index greater than 10 and not greater than 30**

$$\textit{Since}\ 50\% < 75\mu,$$

$$a = 50 - 35$$

$$a = 15\%$$

$$c = 0$$

$$d = 20 - 10 = 10$$

=>G.I = 0.2a + 0.005ac + 0.01bd

=>G.I = 0.2(15) + 0.005(15)(0) + 0.01(35)(10)

=>G.I = 3+0+3.5

=>G.I = 6.5

Que. A) calculate activity value

B) compare the engineering behavior of soil A and soil B

CONSISTENCY LIMIT	SOIL A	SOIL B
w_L (%)	60	50
w_P (%)	25	30
% finer than 0.002mm size	25	40

B) compare the engineering behavior of soil A and soil B

1. Soil A is active and soil B is inactive
2. Soil A is more likely to undergo high volume change
3. Soil A is more compressible
4. Soil B is better than soil A

Group Symbol	Classification
Coarse soils	
GW	Well-graded GRAVEL
GP	Poorly-graded GRAVEL
GM	Silty GRAVEL
GC	Clayey GRAVEL
SW	Well-graded SAND
SP	Poorly-graded SAND
SM	Silty SAND
SC	Clayey SAND
Fine soils	
ML	SILT of low plasticity
MI	SILT of intermediate plasticity
MH	SILT of high plasticity
CL	CLAY of low plasticity
CI	CLAY of intermediate plasticity
CH	CLAY of high plasticity
OL	Organic soil of low plasticity
OI	Organic soil of intermediate plasticity
OH	Organic soil of high plasticity
Pt	Peat

On the Basis of Division of Grain Size

On the Basis of Division of Grain Size

BOULDER	> 300mm		
COBBLE	80-300mm		
COARSE GRAIN SOILS	Gravel (4.75mm-80mm)	Coarse	20-80mm
		Fine	20-4.75mm
	Sand (75μ-4.75mm)	Coarse	2-4.75mm
		Medium	0.425-2mm
		Fine	0.075-0.425mm
FINE GRAIN SOIL	Silt	2μ-75μ	
	Clay	< 2μ	

Soil Structure and Clay Minerals

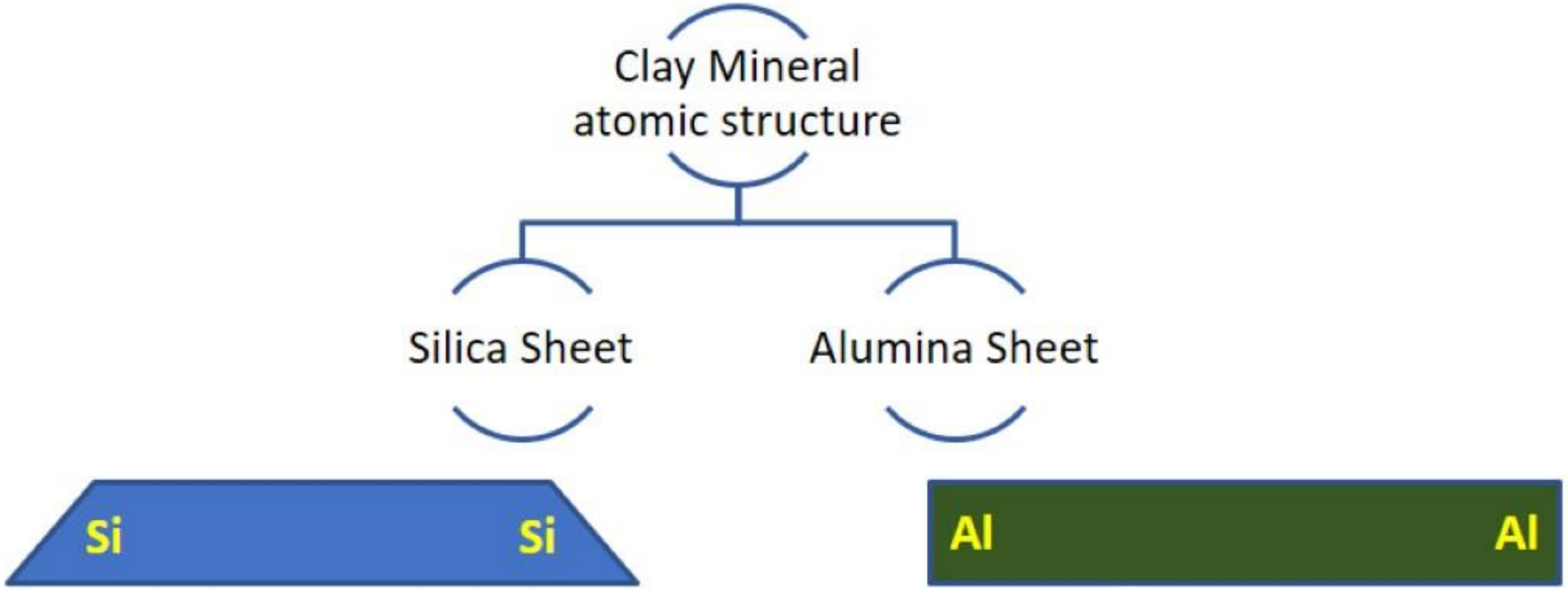
- Soil Structure means the mode of arrangement of soil particles relative to each other and the forces acting between them to hold them in position
- The types of Forces that acts is
  a) In coarse Grain Soils : Gravity Force
  b) In Fine Grain Soils : Surface Bonding Plays an Important Role

Clay Minerals

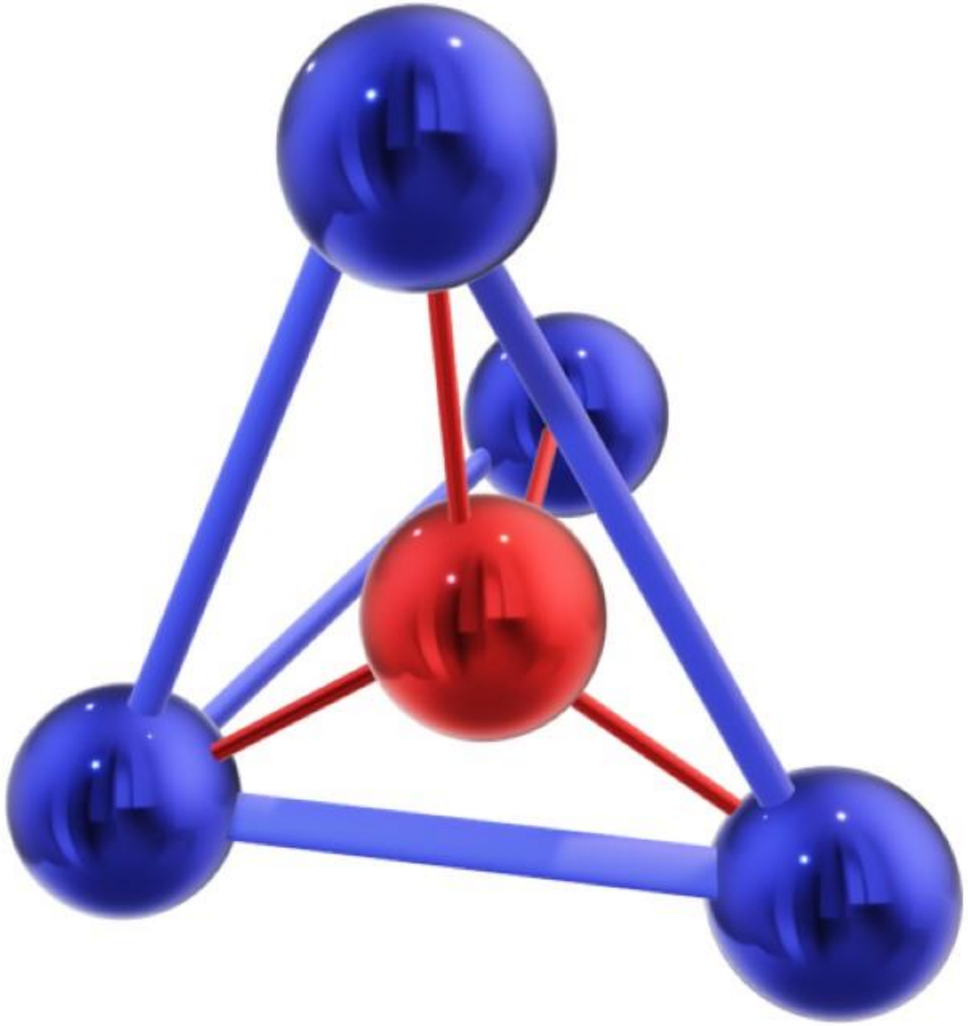
- Formed due to ***Chemical Weathering*** of Rocks
- Particles are very small in size, flaky in shape and can only be viewed with Electron Microscope
- On the Basis of Crystalline Arrangement, they can be divided into three groups:
    1) Kaoline
    2) Montmorrilonite
    3) Illite

Clay Minerals

- Atomic Structure of Clay Mineral is composed of two Fundamental Structures:
  a) Tetrahedral Sheet or Silica Sheet
  b) Octahedral Sheet or Alumina Sheet



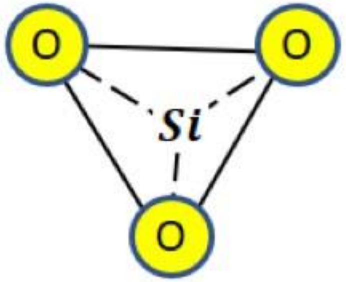
1. Tetrahedral Sheet/Units

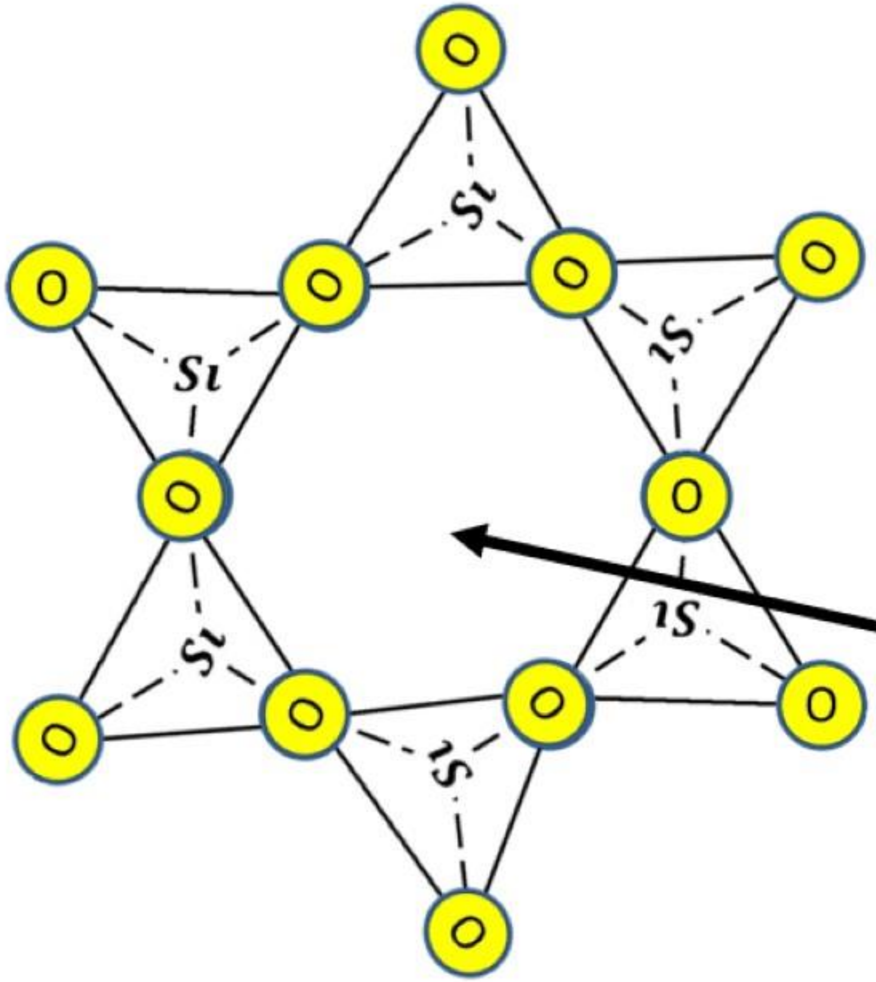


- Made up of Tetrahedron Units
- It has 3 oxygen at base and 1 at the top of tetrahedron

Clay Minerals

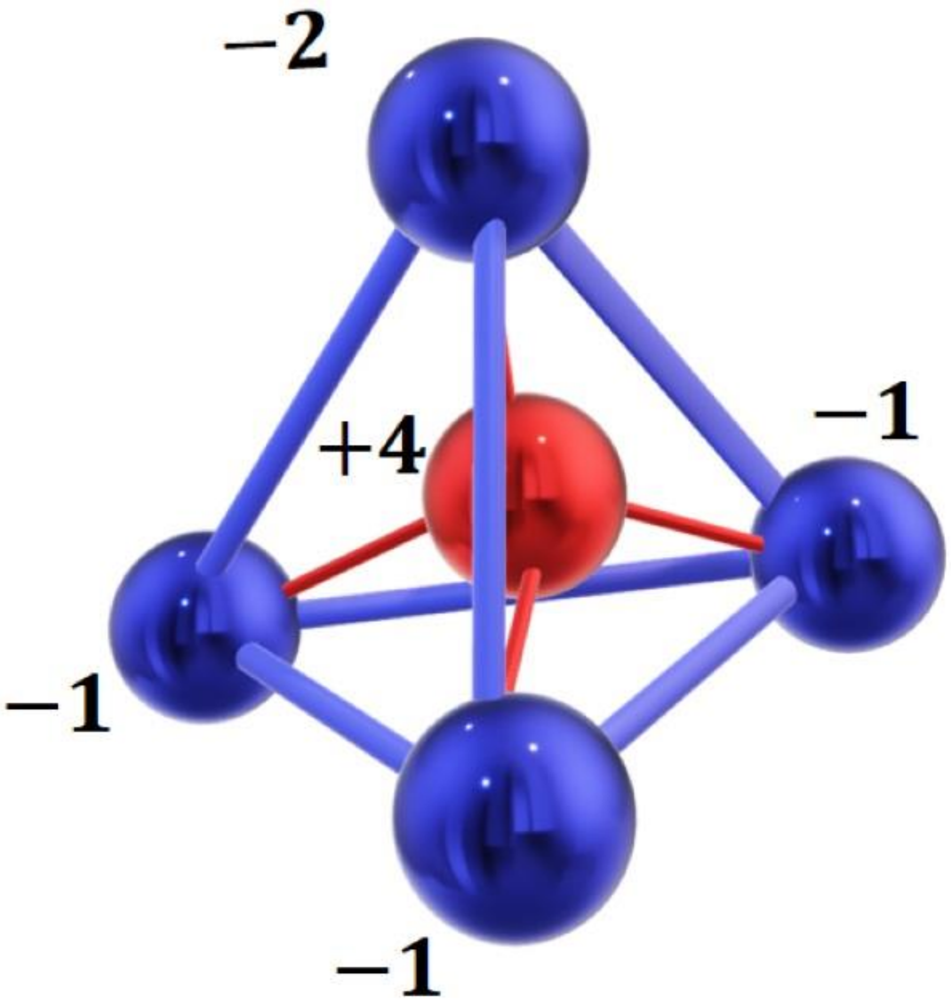
1. Tetrahedral Sheet/Units





- These units combine to form Sheet Like Structure
- Each Oxygen is shared with two units of tetrahedron
- There is a Hexagonal Opening in the sheet

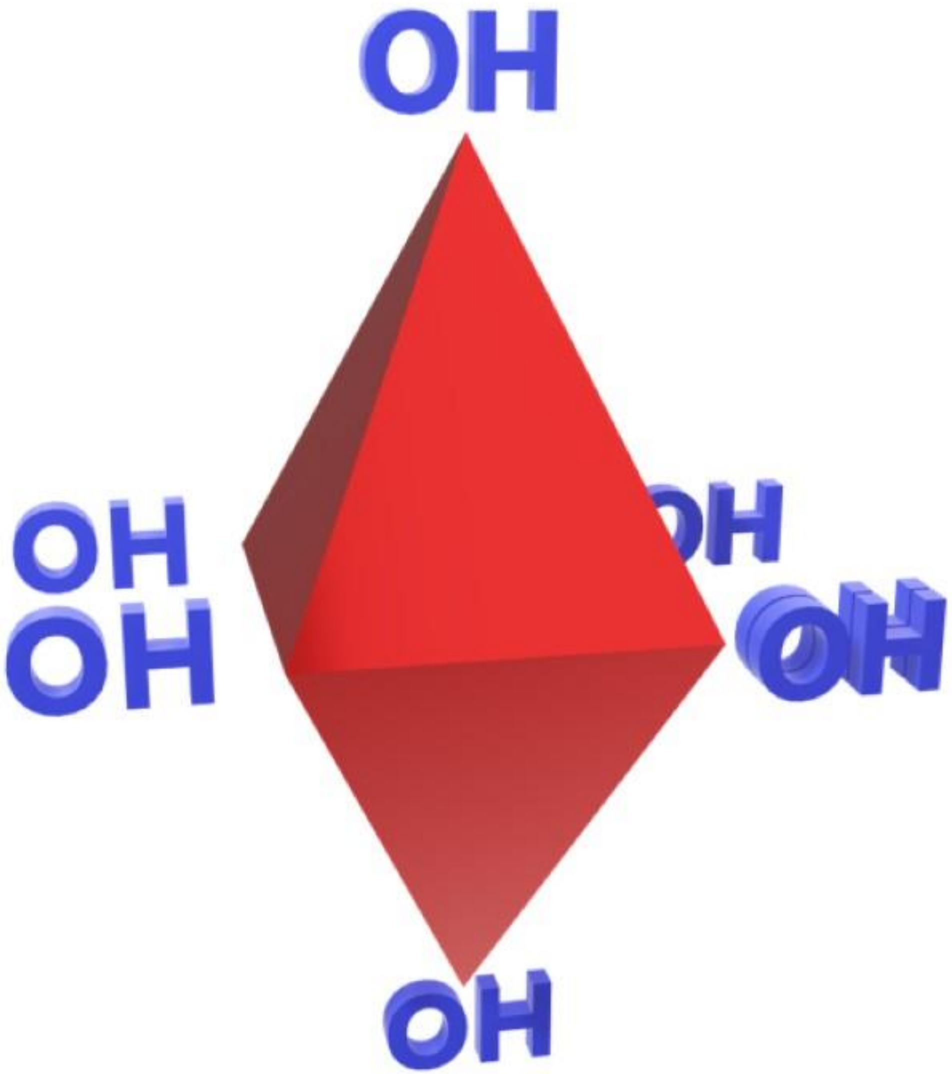
1. Tetrahedral Sheet/Units



- Each Oxygen at Base is shared in two units so carry -1 charge
- Oxygen at Top has -2 Charge
- Silicon ion has +4
- Therefore net charge on each Unit is

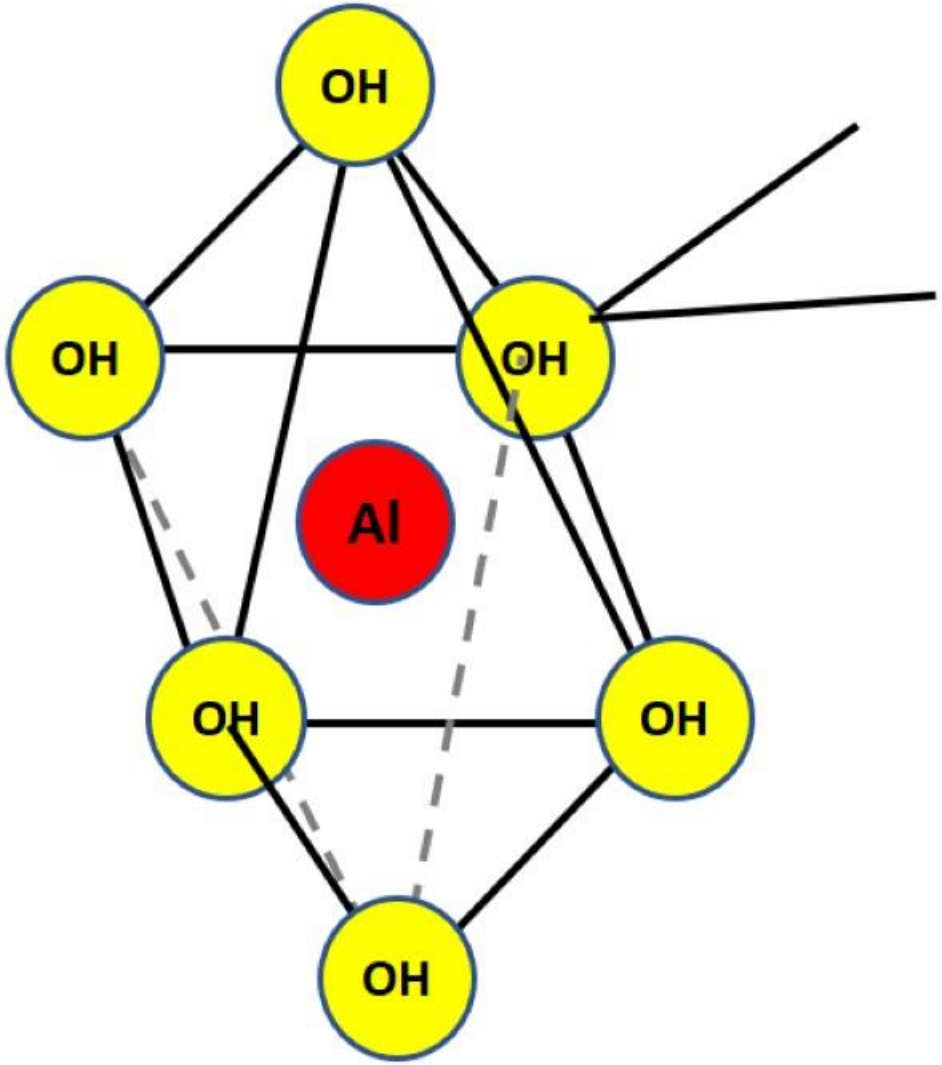
  -1-1-1-2+4 = -1

2. Octahedral Sheet/Units



- If the atom at the centre is Aluminium, resulting sheet is called as Gibbsite
- If Magnesium is at centre, the Sheet is called as Brucite Sheet

2. Octahedral Sheet/Units



- Made up of Octahedral Units
- 1 OH is shared by 3 units of Octahedral and each OH has -1 charge
- So net charge due to 6 OH

$$= 6 \times (-1) \times (\frac{1}{3})$$
$$= -2$$

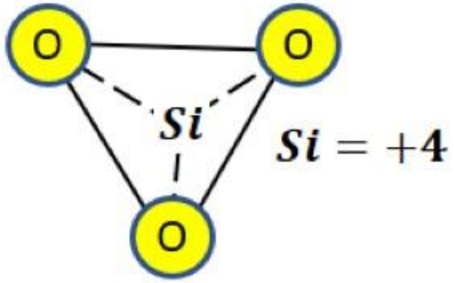
Charge on $Al = +3$

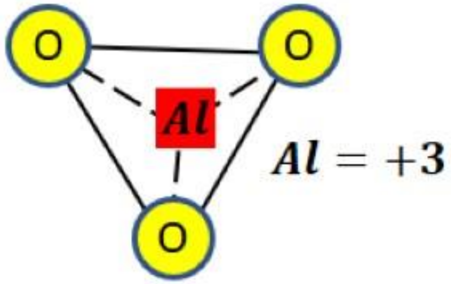
- So net charge on Each unit

$$= +3 - 2$$
$$= +1$$

Isomorphous Substitution

- In a clay Mineral lattice, metallic ions of one kind are replaced by other metallic ion of same or lower valency but of same size, such a substitution is called as Isomorphous Substitution.
- It leads to different clay minerals and different properties
- This Isomorphous substitution leads to charged surface of Soil

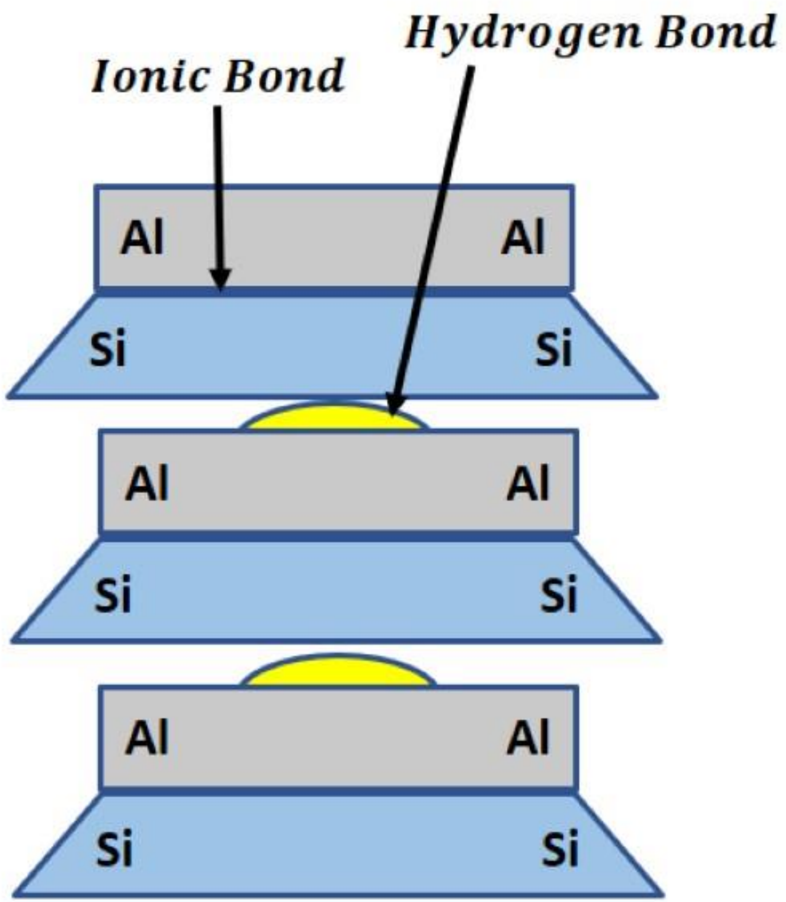




Therefore a deficiency of 1 positive charge is induced

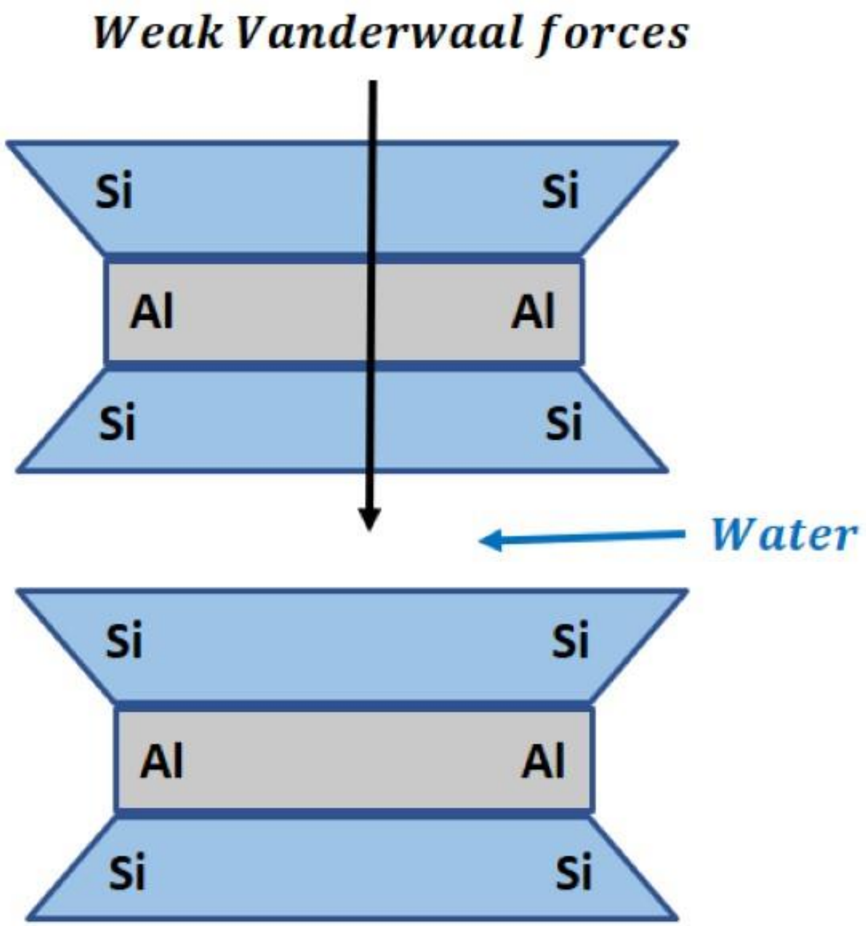
Various Clay Minerals

1. **Kaolinite (1:1)**
   a) It consists of alternative layers of Silica Tetrahedra with the tips inserted in Alumina(Gibbsite) Octahedral Unit
   b) There is H-bonding between layers
   c) There are about 70-100 of such stacked layer with each Layer having thickness of $7A^{\circ}$
   d) Total Thickness is about $500 - 1000A^{\circ}$
   e) Less swelling and shrinkage
   f) Least Active Clay Mineral
   g) Antidiarrheal medicine
   h) Another Kaolin Mineral is Halloysite, which is Randomly stacked unlike Kaolinite



Various Clay Minerals

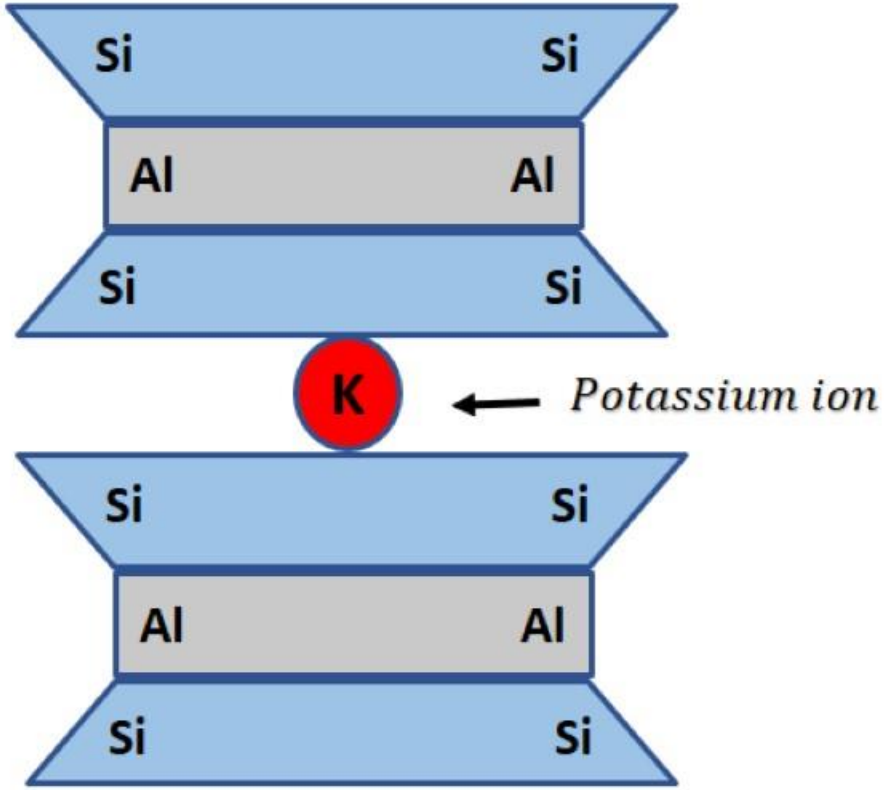
2. **Montmorillonite (2:1)**
   a) Also known as Smectide
   b) Octahedral sheet is sandwiched between two tetrahedral Units
   c) Thickness of layer is $9.6A^{\circ}$, and dimensions of other two directions are indefinite
   d) It has largest specific surface area among major clay minerals
   e) Large amount of water and exchangeable ions can easily enter between layers causing layers to be separated
   f) Weak vanderwaal forces
   g) High swelling and shrinkage
   h) Highly plastic
   i) Example- Bentonite clay (formed from Volcanic ash, used as Drilling Mud), Black cotton soil



Clay Minerals

3. Illite(2:1)

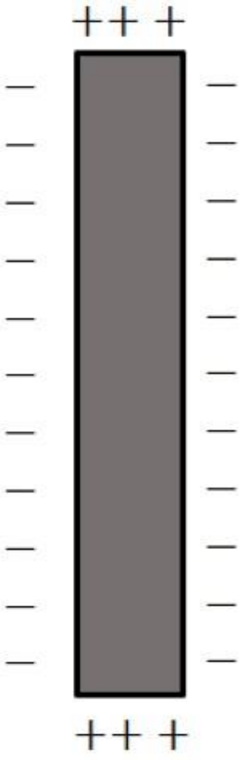
- It has Isomorphous substitution
- Potassium ion enters between two hexagonal openings of tetrahedral sheets
- consists of the basic montmorillonite units but are bonded by secondary valence forces and potassium ions
- Due to this bonding of Potassium between layers, Illite does not swell as Montmorillonite, but it does swell more than Kaolinite



Comparison of the Clay Minerals

Clay Mineral	Size	Plasticity Index	Activity	Dry strength	**Base Exchange Capacity**
Montmorillonite	Min	max	max	max	max
Illite	intermediate	intermediate	intermediate	intermediate	intermediate
Kaolinite	Max	Min	Min	Min	Min

Clay - Water Interaction



Clay Particles generally have Negative charge on them except at the edge

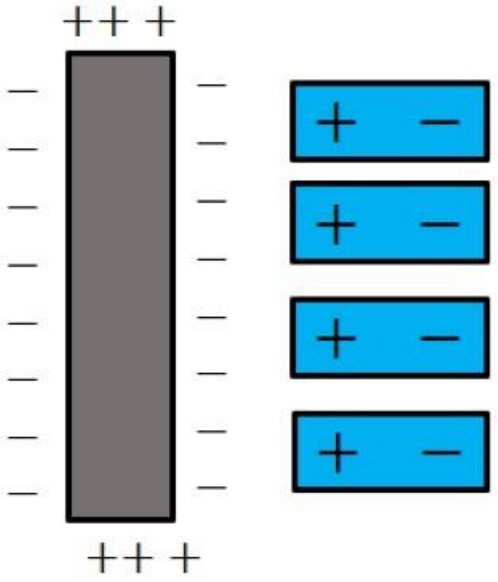
__Reasons for Charge Accumulation:__

a) Isomorphous Substitution

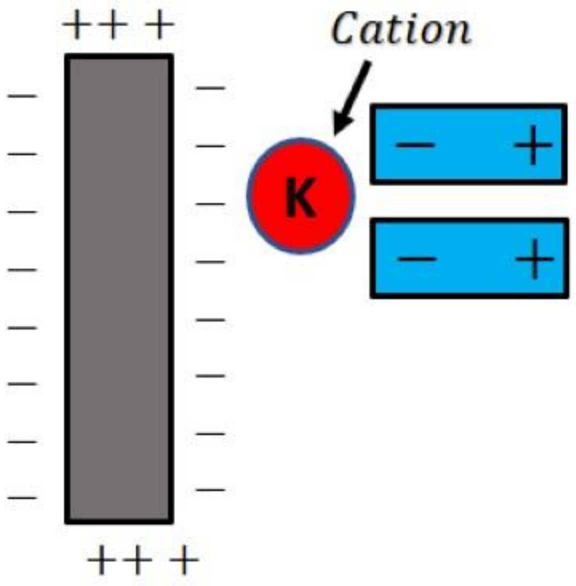
b) Breakage of Corners

Clay - Water Interaction

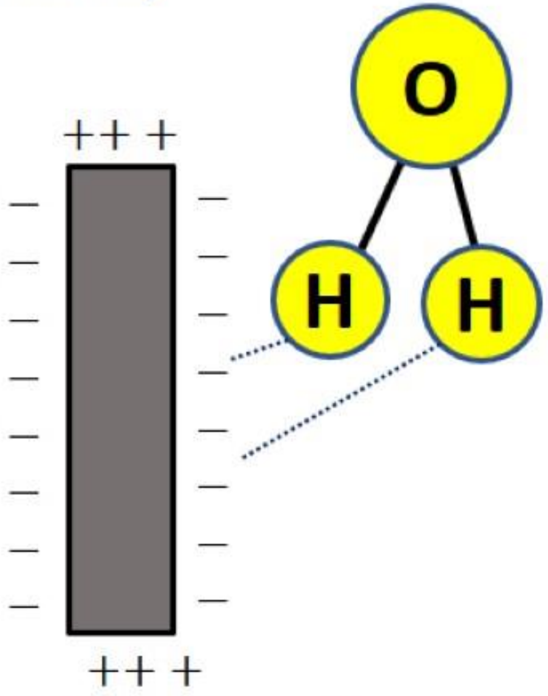
Water Molecules can interact with clay particles in the following ways:



a) Attraction due to Electrostatic Forces

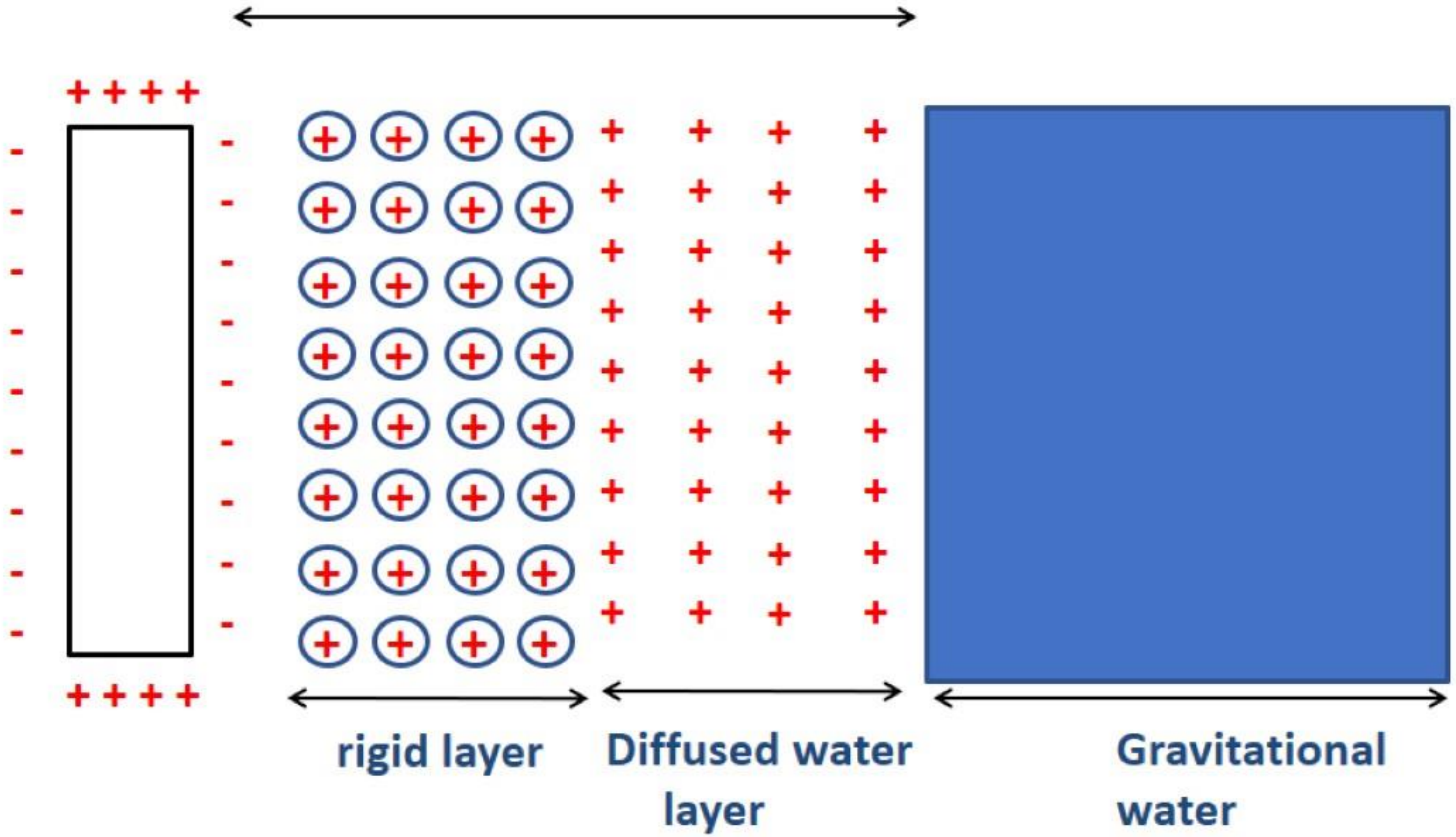


b) Attraction through Cation attachment on surface of Clay Particle



c) Interaction through H – Bonding

DOUBLE DIFFUSED LAYER



ADSORBED LAYER

- Water is held strongly due to electrostatic force of attraction and it is virtually in the solid state.
- This adsorbed layer of water is not removed if soil is heated through 105 to 110 degree Celsius.
- As we move from clay particle, density decreases.
- If some lower valency cation is replaced by higher valency cation, then thickness of double diffused layer decreases.

ADSORBED LAYER

Due to this decrement in double diffused layer, voids present between clay particles will increase.

Hence seepage (permeability) through soil will increase.

- Relative density will decrease.
- Thickness of double diffused layer depends on
  a) Concentration of the cations in water.
  b) Valency of cations
  c) Temperature

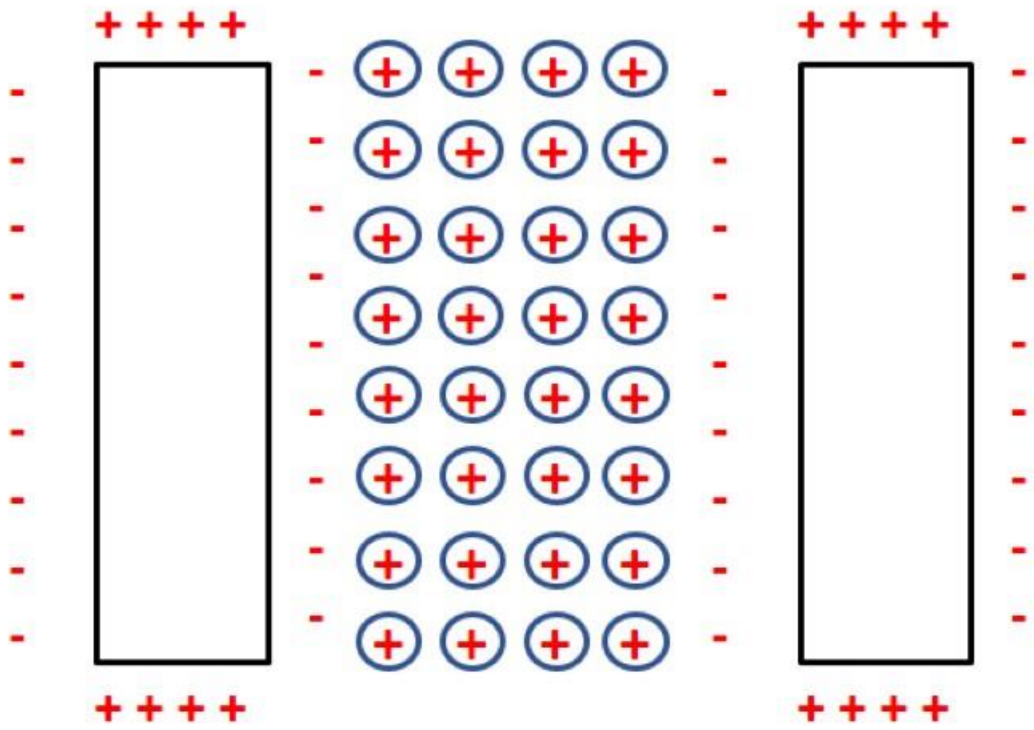
CATION EXCHANGE CAPACITY

- Also called as Base Exchange Capacity
- The ability of clay particles to adsorb ions on its surface is called Cation Exchange Capacity.
- Higher valency cation can replace lower valency cation.
- Replacement capacity order =

  $Al^{3+} > Ca^{2+} > Mg^{2+} > K^{+} > H^{+} > Na^{+} > Li^{+}$

This can be helpful in many practical situations such as treatment of sodium clay soil, where Calcium(Lime) would replace Sodium ions and decreases the swelling of Sodium Montmorrilonite

CLAY PARTICLE INTERACTION

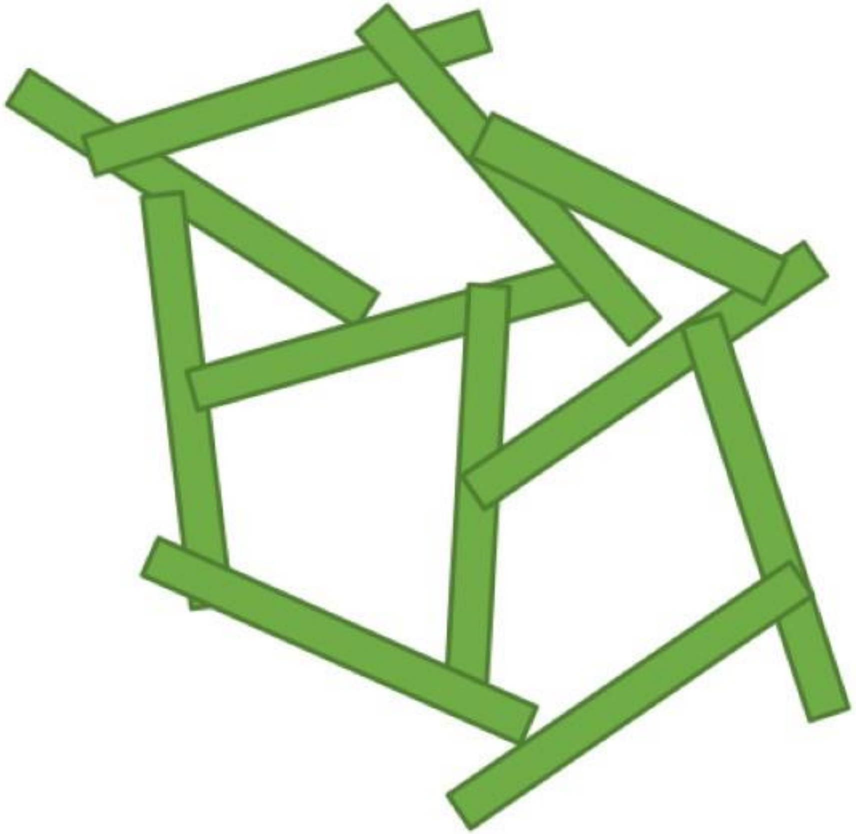


Due to less double dispersed layer,
Negative charge on one attracts
Positive charge of another

Flocculated structure

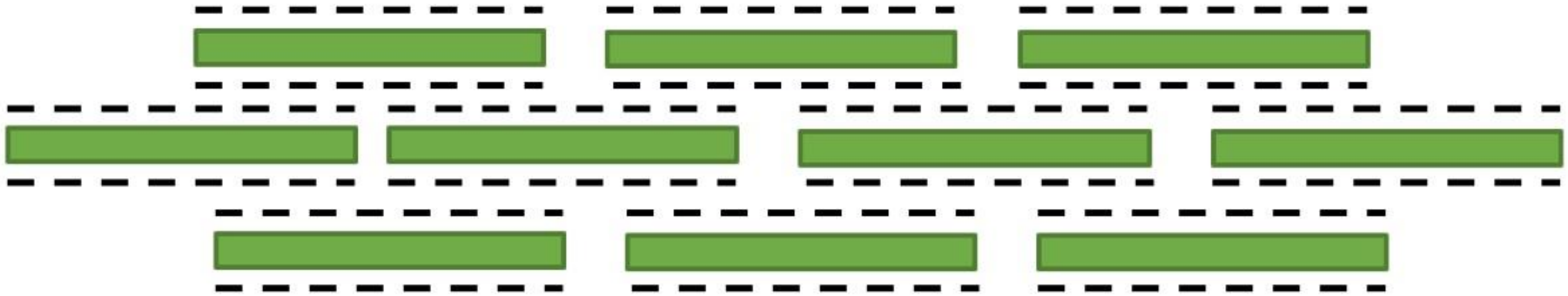
CLAY PARTICLE INTERACTION

- **Flocculated structure :**
  a) Flocculated structure is formed when net force between clay particles is attractive.
  b) This is edge to face interaction.
  c) It has high quantity of voids.
  d) The type of structure has high seepage velocity.
  e) It has larger strength due to high attraction but has large void ratio



NOTE - Marine clay have flocculated structure due to presence of salts.

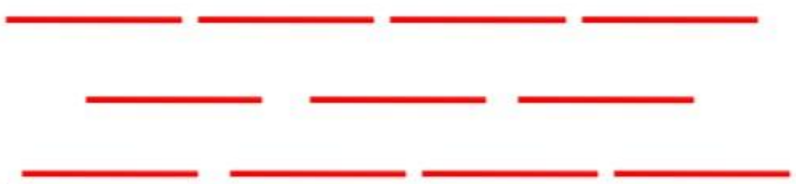
CLAY PARTICLE INTERACTION



Dispersed structure

CLAY PARTICLE INTERACTION

- **Dispersed structure :**
  a) This type of structure is formed when net force between clay particles is repulsive.
  b) It has face to face interaction
  c) Generally, one dimensional seepage velocity is high.
  d) It has low voids as compared to flocculated structure.



NOTE – Lacustrine soil has dispersed structure.

Effective Stress, Capillarity and Permeability

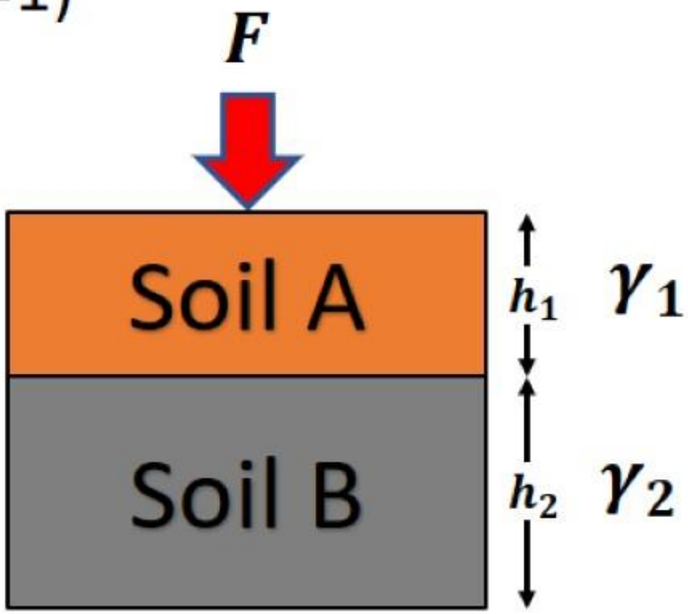
Effective Stress

Consituents of Pore Phase (Water) influence the properties of soil very much

In Effectice Stress, capillarity and Permeability, saturated Soil is considered

Principle of Effective Stress

- Terzaghi expressed the Effective Stress Principle
- This principle is applied to fully saturated Soil (s=1)



Total Stress **in a Soil mass is force per unit area of soil mass transmitted in normal Direction**

$$\Rightarrow \sigma = \frac{W}{A}$$

$$\Rightarrow \sigma = \frac{F + (Ah_1)\gamma_1 + (Ah_2)\gamma_2}{A}$$

$$\Rightarrow \sigma = \frac{F}{A} + \gamma_1 h_1 + h_2\gamma_2$$

Stress in Soil Mass

$$Total\ Stress = Effective\ Stress + Pore\ water\ pressure$$

$$\boxed{\sigma = \bar{\sigma} + u}$$

$$and$$

$$\boxed{u = u_h + u_e}$$

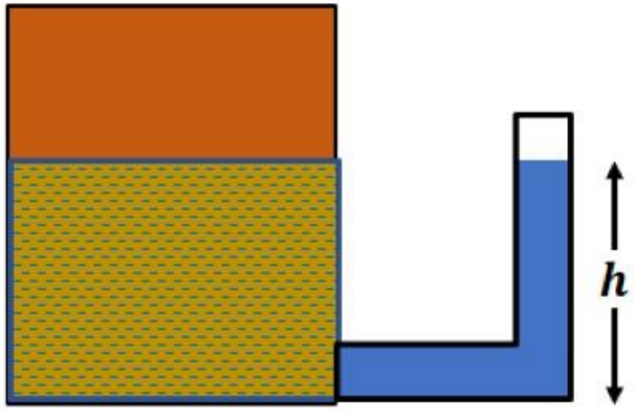
1. Pore Water Pressure (u)

- Pore Water Pressure is made up of two parts:
  a) ***Stress due to pore water known as Neutral Stress or Pore Water Pressure*** $(\boldsymbol{u})$

The pressure of water filling between the void space is known as Pore Water Pressure

If Piezometer is inserted and water rises to height 'h', then

$$\boxed{\boldsymbol{u_h = h\gamma_w}}$$



1. Pore Water Pressure (u)

- **Pore Water Pressure** is made up of two parts:

  b) Excess Pore Water Pressure (u_e)

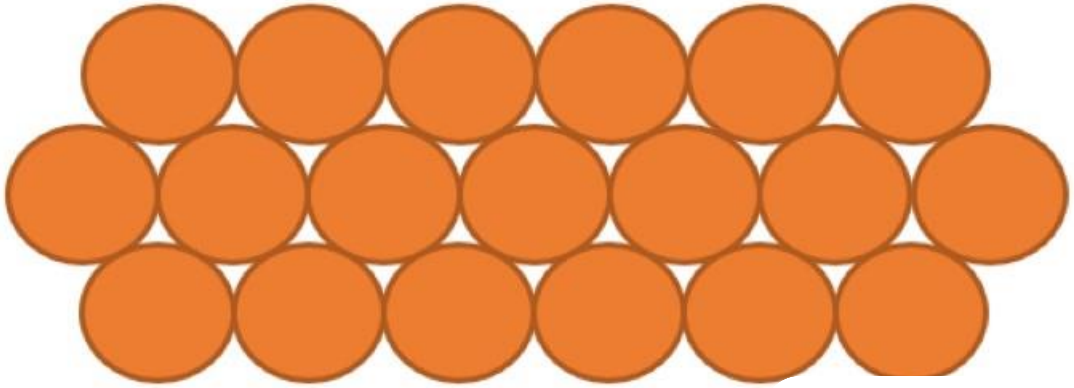
  - When external load is applied, pressure of water in voids suddenly increases and which is equal to applied external pressure.
  - As the time passes excess pore water pressure dissipates in lateral direction.
  - After infinite time, steady state condition is reached and excess pore water pressure is completely dissipated to zero.

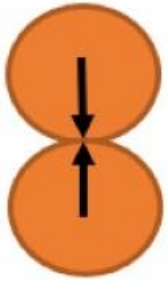
2. Effective Stress

- Total Stress and Neutral Stress are measurable but effective stress is not a physical parameter and can not be measured
- It can be computed by

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

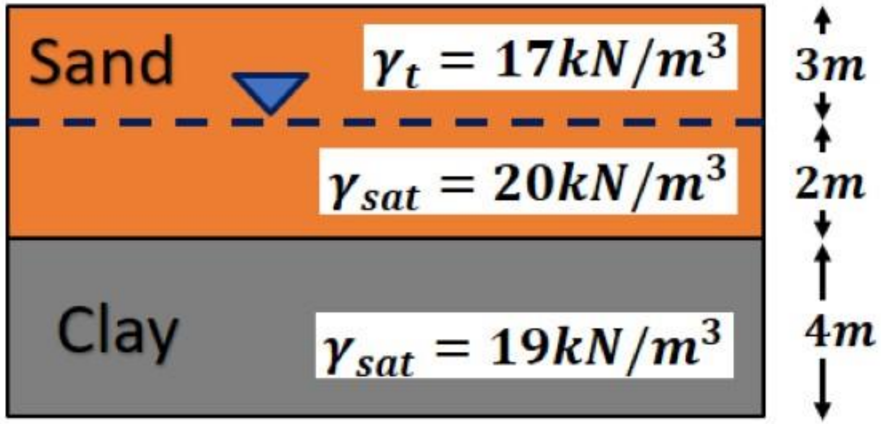
- Effective Stress is used to determine volume changes and shearing resistance
- Physical Meaning of Effective Stress :
  - Effective Stress is the sum of contact forces divided by the gross area



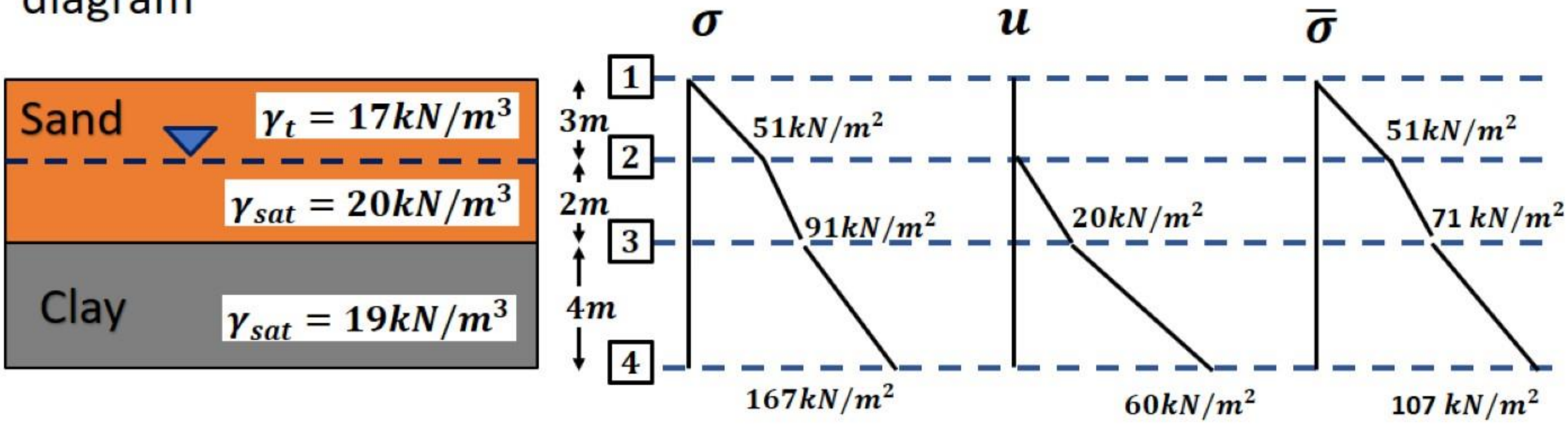


Que. A sand deposit is 10m thick and overlaid on a bed of soft clay, the ground water table is 3m below the surface. If the sand layer above the ground water table has degree of saturation as 45%, draw the diagram of total stress, pore water pressure and effective stress. Given the void ratio of sand is 0.70 and specific gravity is 2.65

Que. Draw Total Stress, effective stress and pore water stress diagram



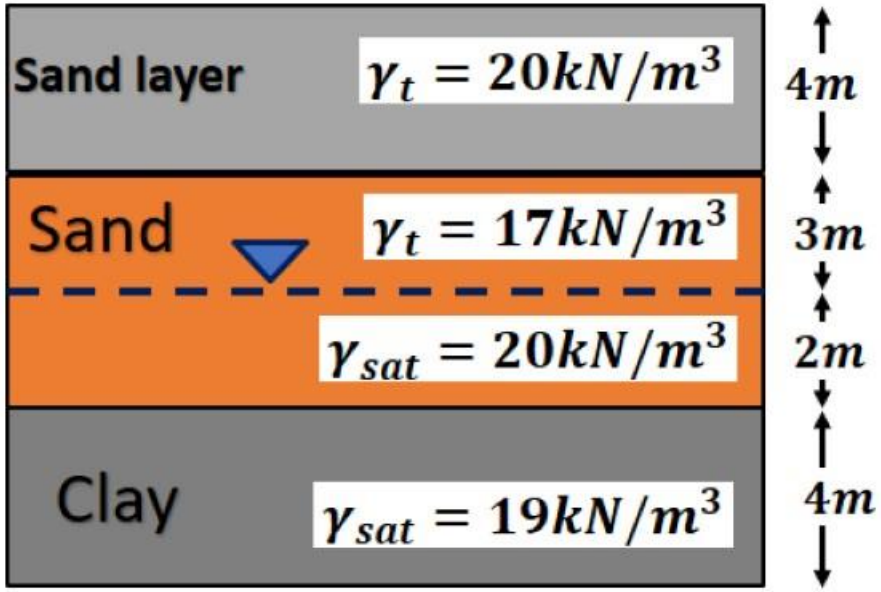
Que. Draw Total Stress, effective stress and pore water stress diagram



Que. If 4m deep layer of sand of $\gamma_t = 20kN/m^3$ is placed over the surface, find effective vertical stress

a) Immediately after the fill

b) Many years after the fill



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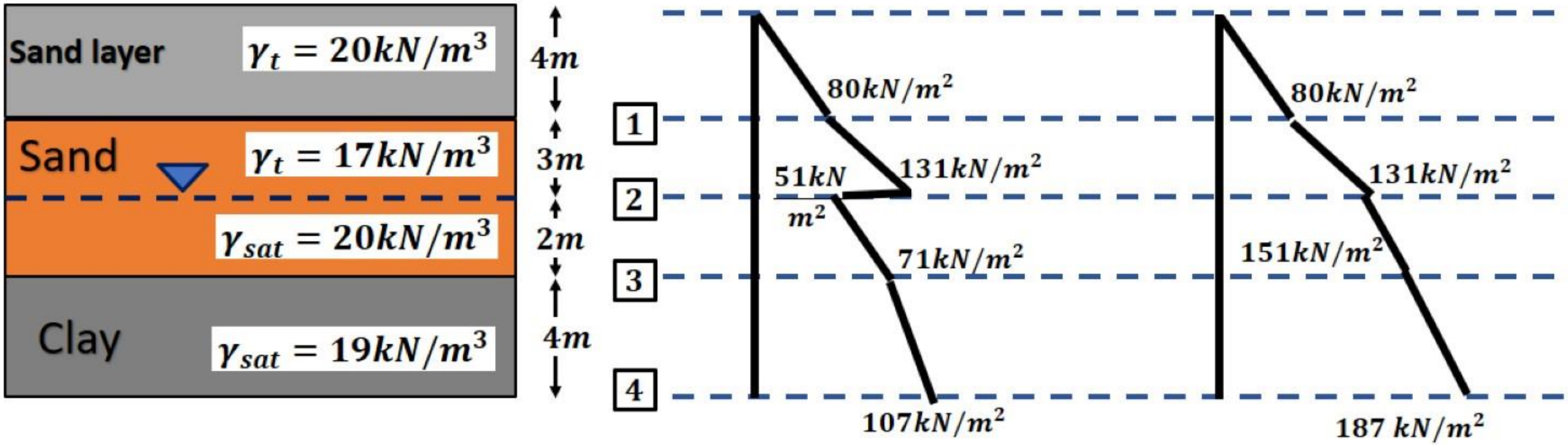


Expression for Capillary Rise and Capillary Fall

- For equilibrium condition,

$$\Rightarrow F_s \cos\theta = wt\ (weight\ of\ fluid\ column)$$

$$\Rightarrow \sigma \times perimeter \times \cos\theta = \rho \times A \times h \times g$$

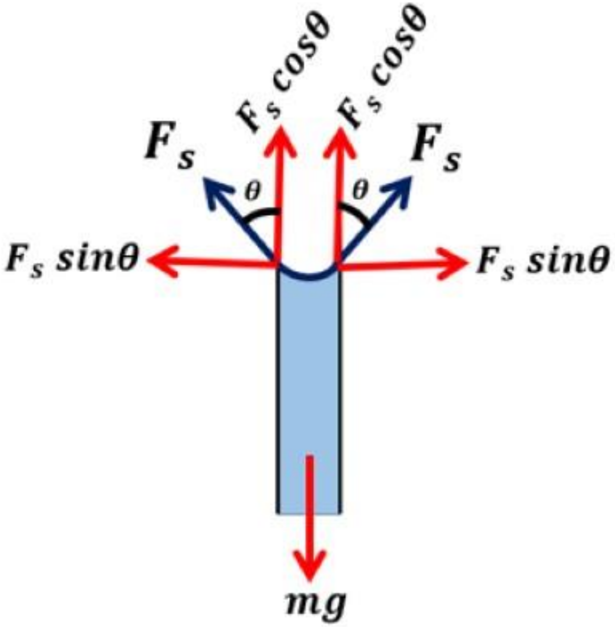
$$\Rightarrow \sigma \times \pi d \times \cos\theta = \rho \times \frac{\pi}{4} d^2 \times h \times g$$

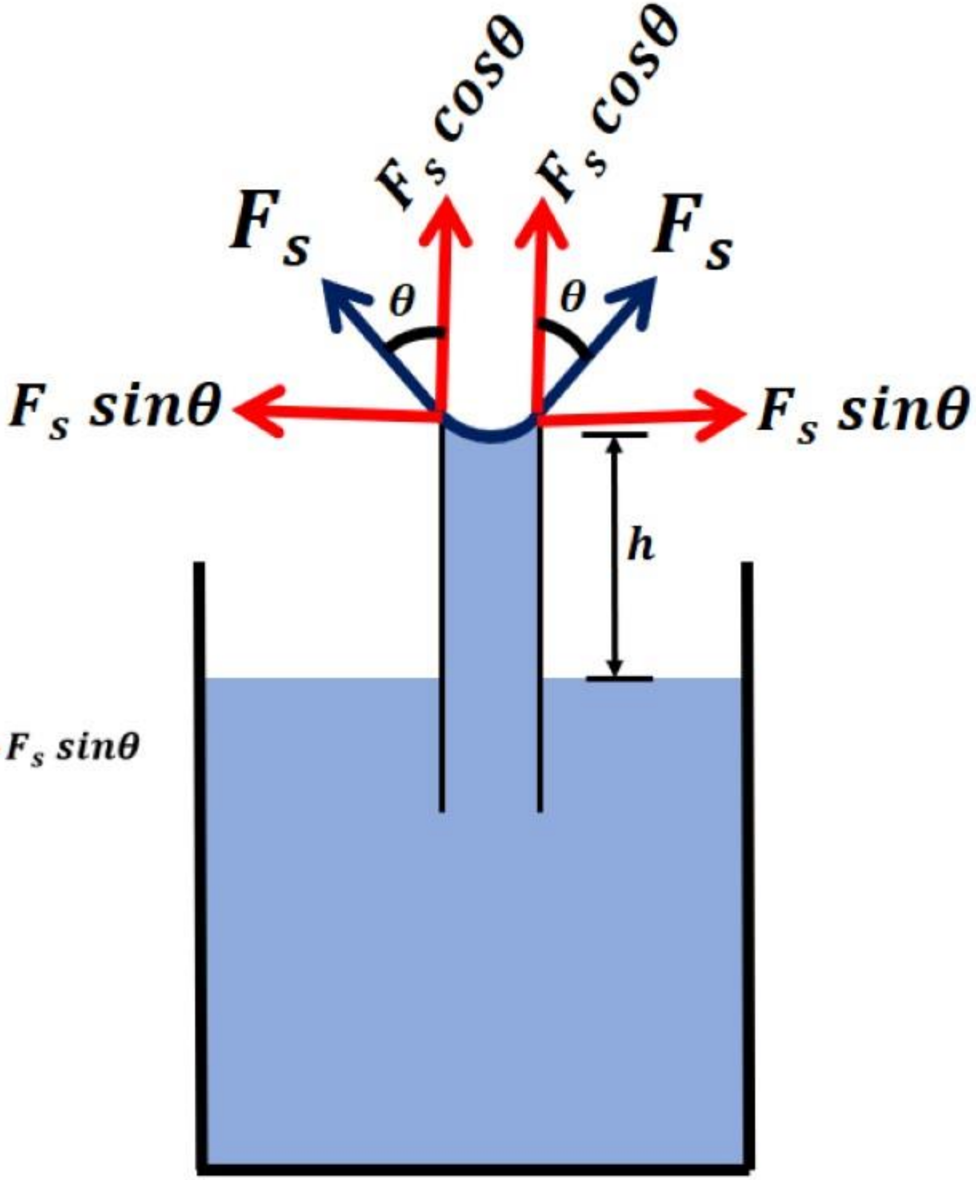
$$\boxed{\Rightarrow h = \frac{4\sigma\cos\theta}{\rho g d}}$$

Since specific weight, $w = \rho g$

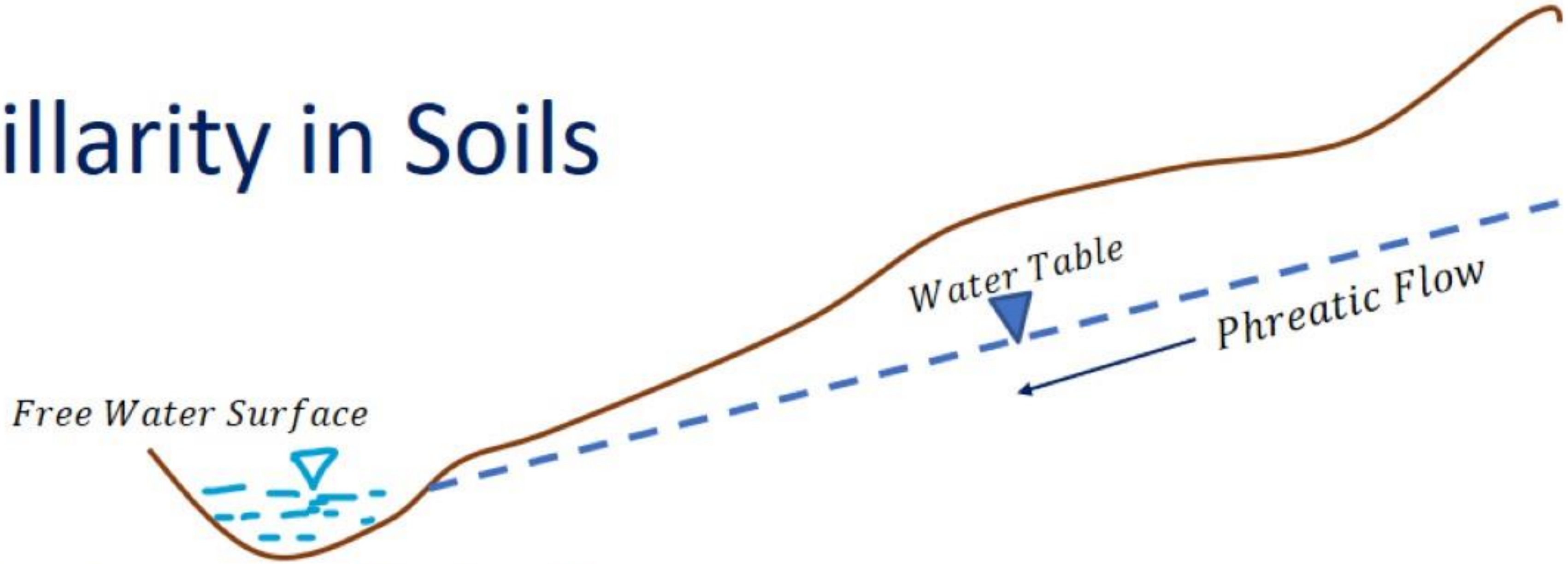
- For capillary rise,

$$h = \frac{4\sigma\cos\theta}{wd}$$



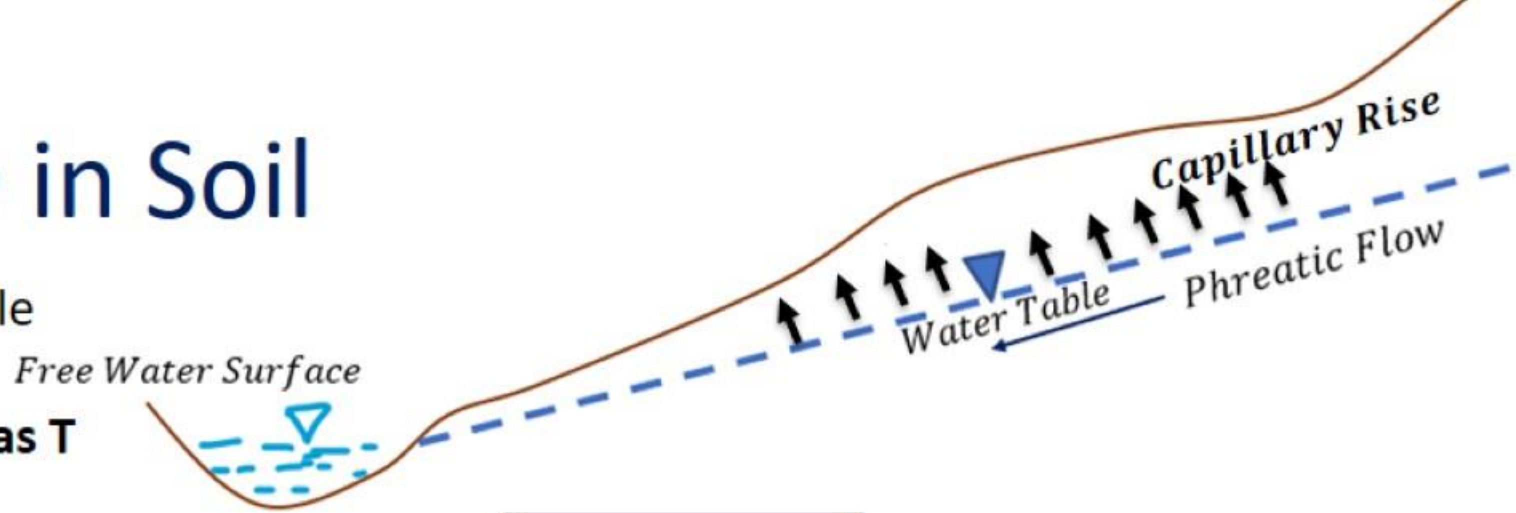


Capillarity in Soils



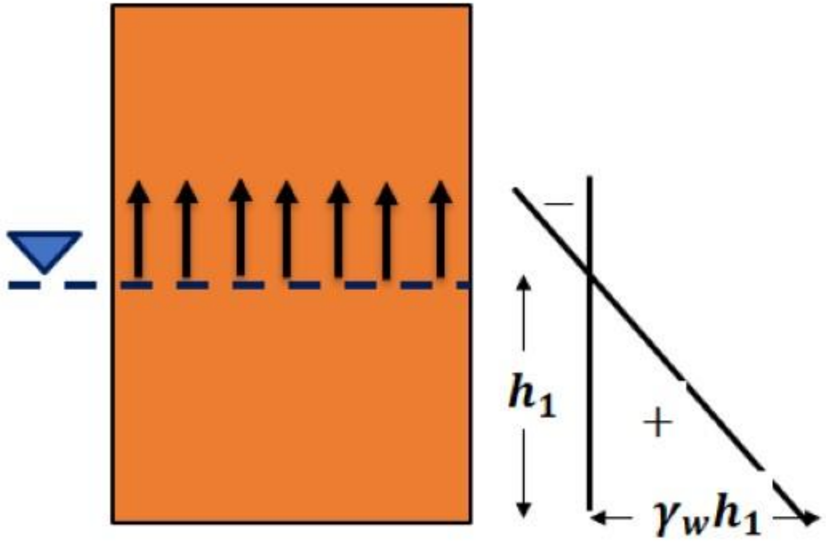
- Ground water can exist in two forms:
  a) Phreatic / Gravitational Water
  b) Capillary Water
- The upper surface of the zone of full saturation is called the ***Water Table or Phreatic Surface***
- **At the water table, ground water is subjected to atmospheric pressure so the *pore water pressure is zero***

Capillary Rise in Soil



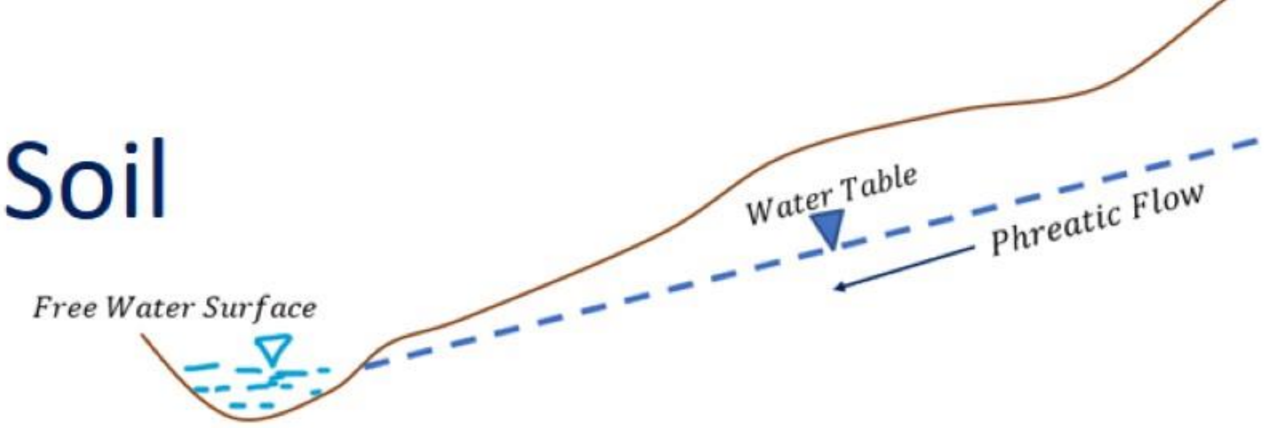
- Capillary Water is held above water table by **Surface Tension**
- **If $\theta = 0$ and we write surface tension as T**
- **Capillary Rise** is given as $h = \frac{4T}{\gamma_w d}$
- Pressure at the water table is atmospheric pressure, below water level pressure is Positive, above water level pressure is negative, Hence pressure in capillary saturation zone. Pressure is negative
- Pressure Variation is linear from water table upto Capillary Saturation Zone
- This negative Pressure in capillary zone leads to increase in effective Stress $\bar{\sigma}$

$$\boxed{\bar{\sigma} = \sigma - (-u)}$$
$$\boxed{\Rightarrow \bar{\sigma} = \sigma + u}$$



$\gamma_t = 17kN/m^3$

Capillary Rise in Soil



Note:

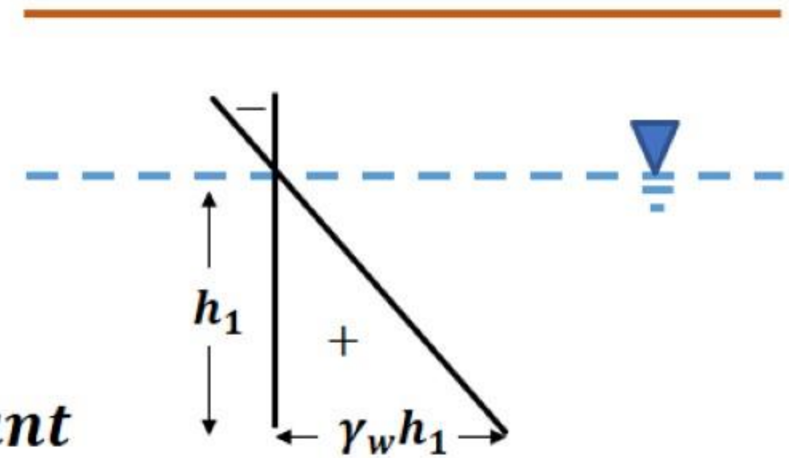
- In Capillary Saturation Zone, unsupported Excavation can be done
- Amount of capillary saturation zone height is difficult to calculate because voids are irregular, hence we use empirical formula to calculate Height of Capillary Rise

$$\bar{h} = \frac{c}{ed_{10}}$$

$c = 0.1 - 0.5\ cm^2 = emperical\ constant$

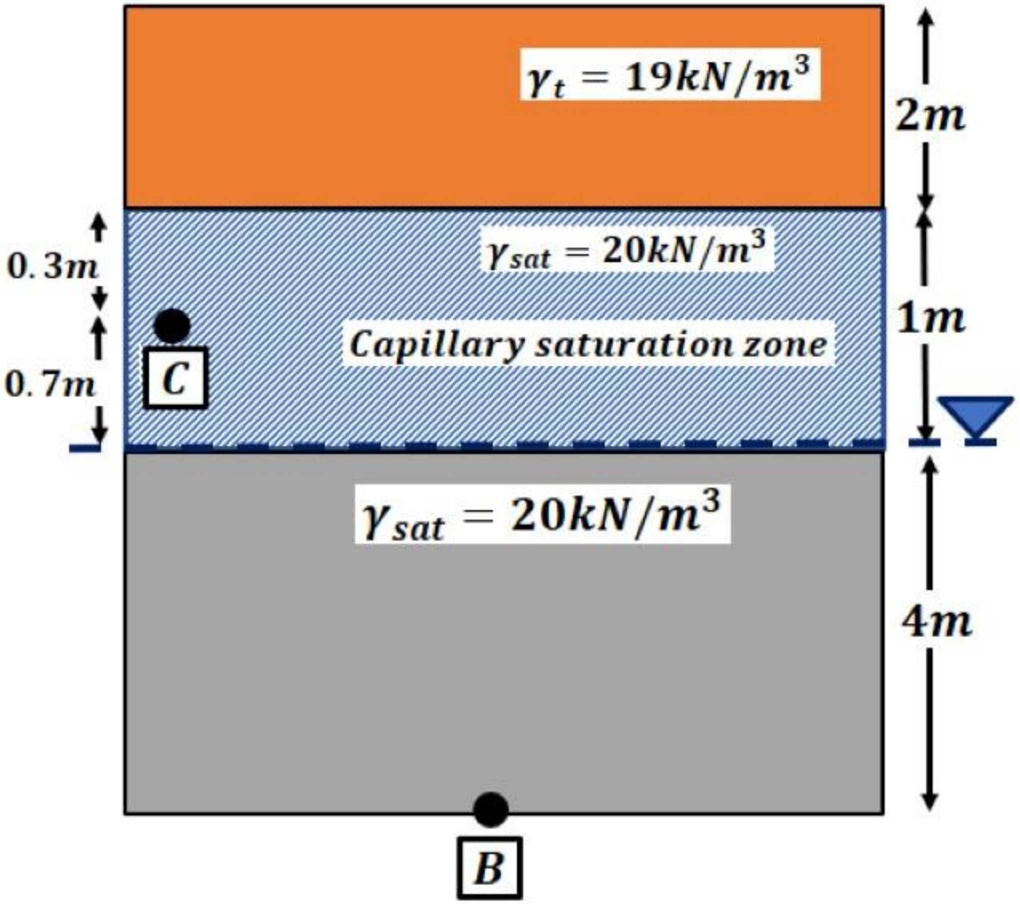
$e = void\ ratio$

$d_{10} = effective\ size$ in cm, $\bar{h} = m$

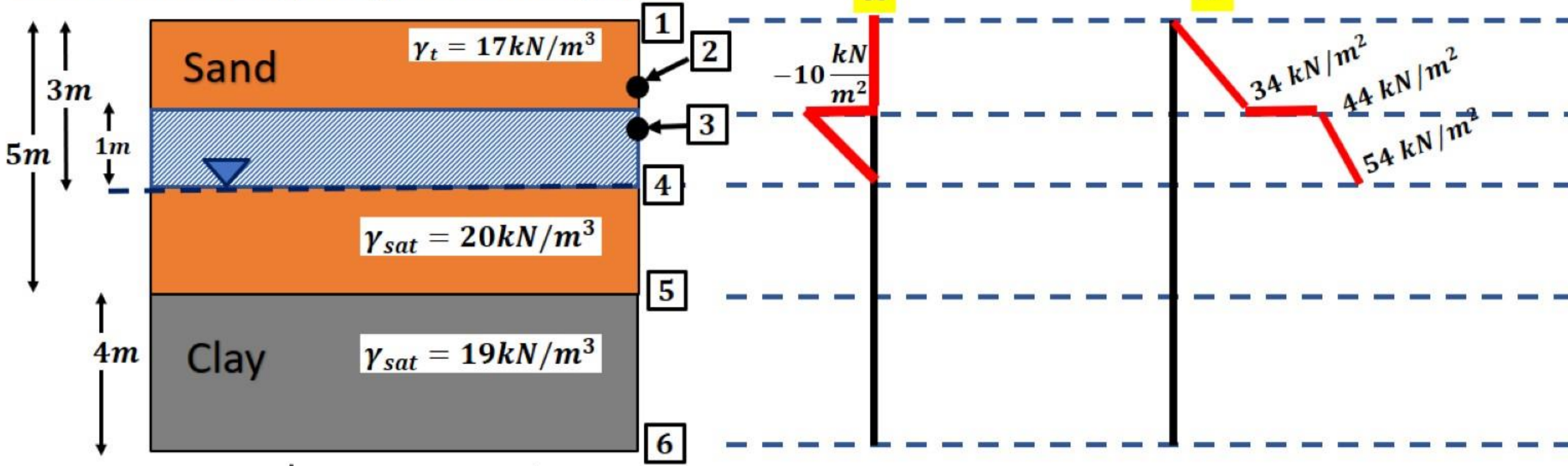


Que. Find out effective stress at B and C.

$$\gamma_w = 10kN/m^3$$



Que. A layer of Saturated Clay 4m thick and overlaid by a sand of 5m thickness and water level is 3m below ground level. Saturated Unit Weight of of clay and sand are 19 kN/m^3 and 20 kN/m^3 respectively. Unit weight above water table is 17 kN/m^3. If height of Capillary Saturation zone is 1m above water table, plot *effective stress graph*.



$\overline{\sigma_1} = \sigma - u$

$\Rightarrow \overline{\sigma_1} = 0$

$\overline{\sigma_2} = \sigma_2 - u_2$

$\Rightarrow \overline{\sigma_2} = 34 - 0$

$\Rightarrow \overline{\sigma_2} = 34\frac{kN}{m^2}$

$\overline{\sigma_3} = \sigma_3 - u_3$

$\Rightarrow \overline{\sigma_3} = 34 - (-10)$

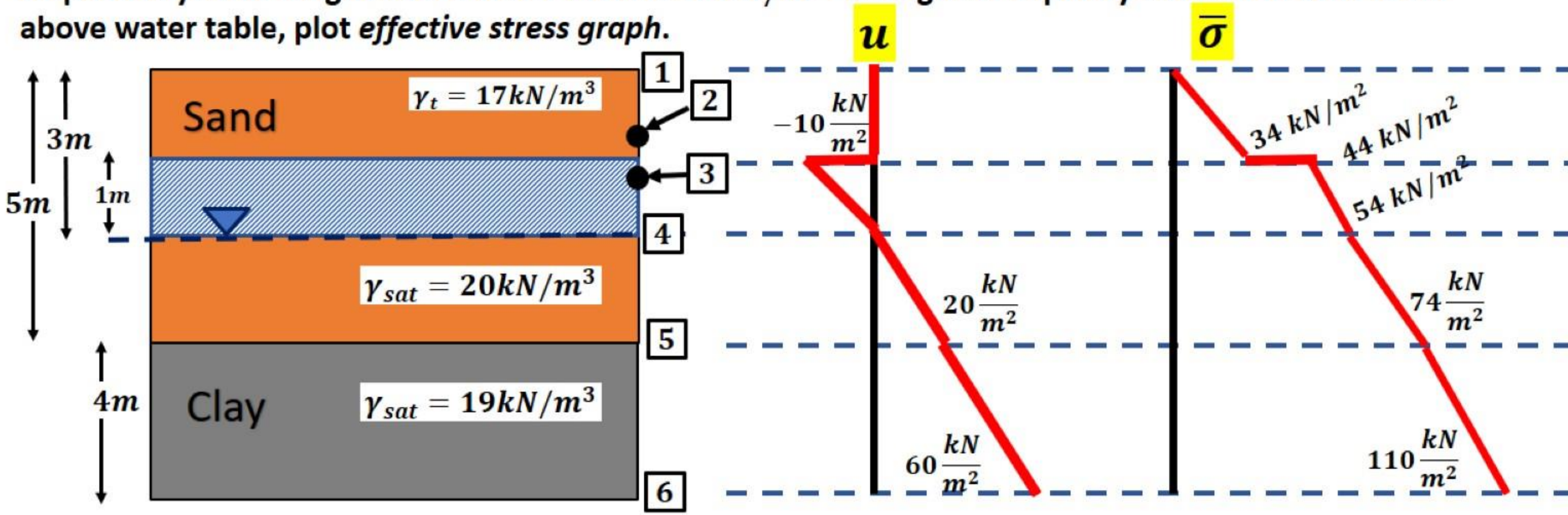
$\Rightarrow \overline{\sigma_3} = 44\frac{kN}{m^2}$

$\overline{\sigma_4} = \sigma_4 - u_4$

$\Rightarrow \overline{\sigma_4} = 34 + 20 \times 1 - 0$

$\Rightarrow \overline{\sigma_4} = 54\frac{kN}{m^2}$

Que. A layer of Saturated Clay 4m thick and overlaid by a sand of 5m thickness and water level is 3m below ground level. Saturated Unit Weight of of clay and sand are 19 kN/m^3 and 20 kN/m^3 respectively. Unit weight above water table is 17 kN/m^3. If height of Capillary Saturation zone is 1m above water table, plot *effective stress graph*.



$$\overline{\sigma_5} = \sigma_5 - u_5$$

$$\Rightarrow \overline{\sigma_5} = (34 + 20 \times 1 + 20 \times 2) - (20)$$

$$\Rightarrow \overline{\sigma_5} = 74\frac{kN}{m^2}$$

$$\overline{\sigma_6} = \sigma_6 - u_6$$

$$\Rightarrow \overline{\sigma_6} = (34 + 20 \times 1 + 20 \times 2 + 4 \times 19) - (60)$$

$$\Rightarrow \overline{\sigma_6} = 110\frac{kN}{m^2}$$

Que. A layer of sand 6m thick lies above a layer of clay soil. The water table is at a depth of 2m below the ground surface. The void ratio of the sand layer is 0.6 and degree of saturation of the sand above the water table is 40%. The void ratio of the clay layer is 0.7. Determine the total stress, pore water pressure and effective stress at a point 10m below the ground surface. Assume specific gravity of sand and clay soil respectively as 2.65 and 2.7

Que. A layer of sand 8m thick lies above a layer of clay. The water table is at a depth of 1m below the ground surface. The saturated unit weight of sand is 20kN/m3 and its dry unit weight is 17kN/m3.

a) Plot the total stress, pore water pressure and effective stress diagram with depth upto 8m.

b) If sand above water table gets saturated due to capillary moisture, what will be the changes in effective stress diagram?

Que. A deposit of sand layer has porosity 35%, G=2.7. The soil is dry in top 1.5m depth. It has 15% moisture content in next 1.8m depth and it is submerged below it.

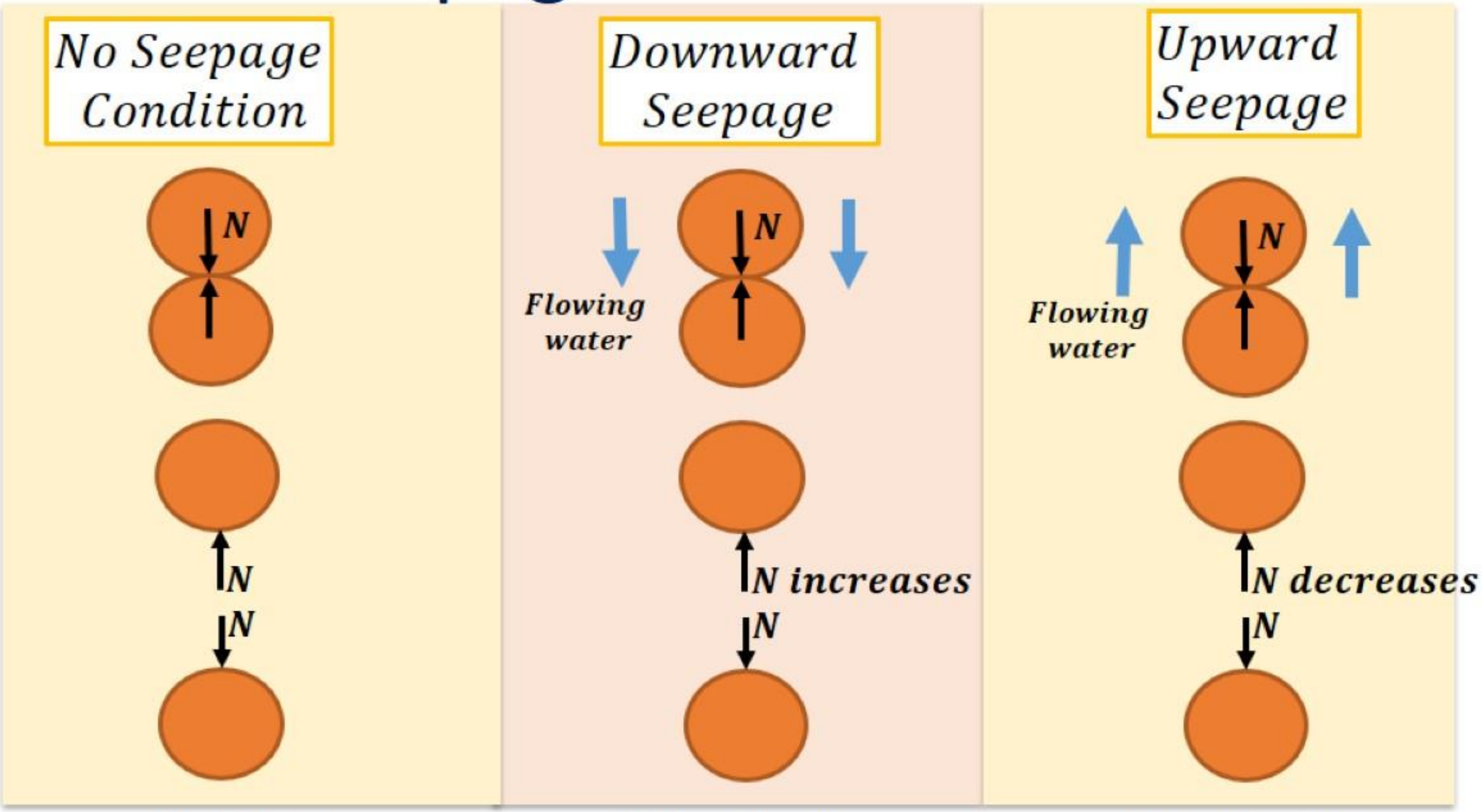
a) Find effective stress at a depth of 8m below the ground level

b) Find change in effective pressure if water table suddenly drops to a level of 6m below ground level

c) Find Shear strength of soil on horizontal plane at 8m depth from both position of ground water table if $\emptyset = 30°$

Que. A layer of saturated clay 5m thick is overlaid by sand 4m deep. The water table is 3m below the top surface. The saturated weight of clay and sand are 18kN/m3 and 20kN/m3 respectively. Above the water table, the unit weight of sand is 18kN/m3. If the soil gets saturated by capillary upto the height of 1m above the water table, then increase in effective stress is...?

Effect of Seepage on Effective Stress



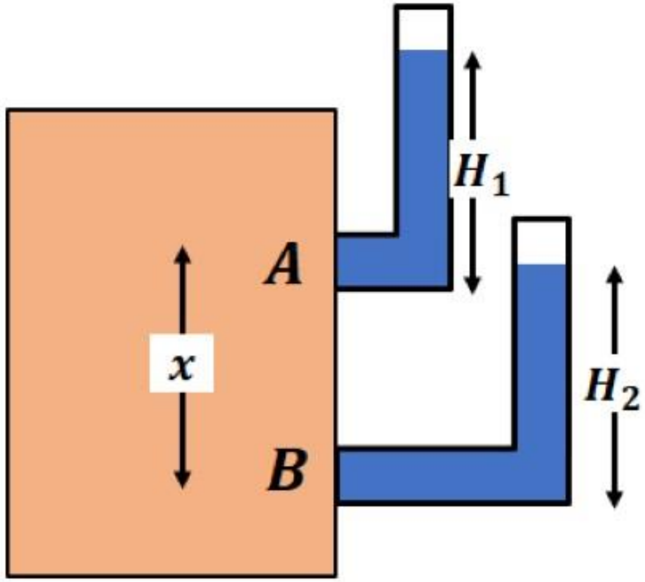
No Effect

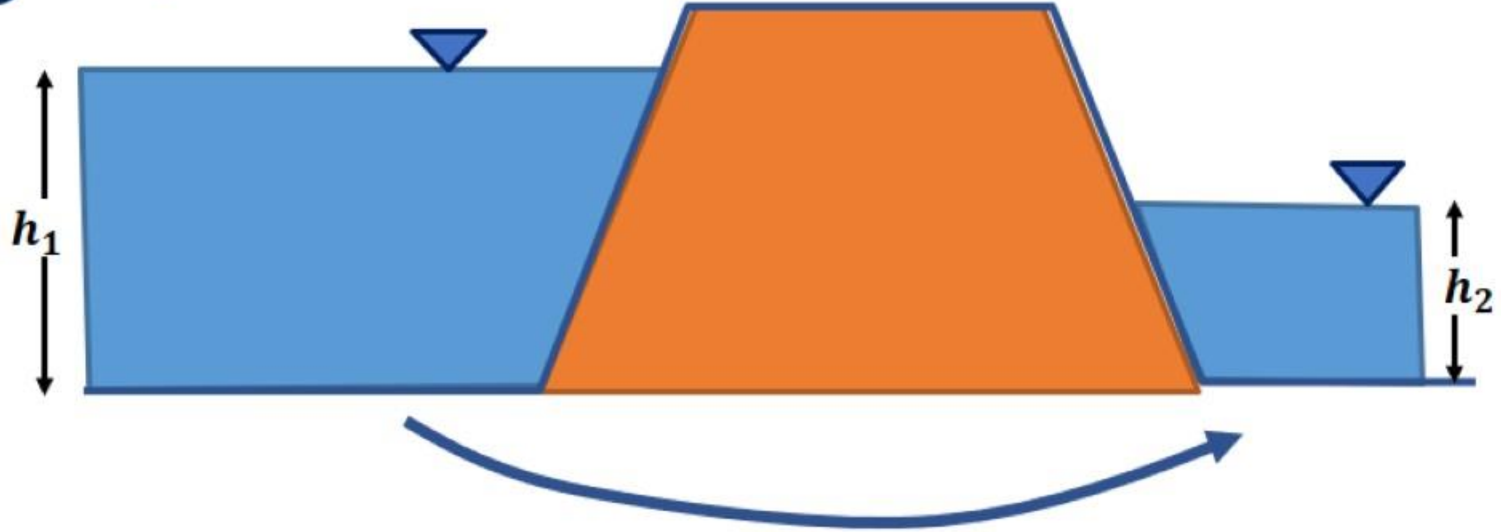
Effective Stress Increases

Effective Stress Decreases

Head Causing Flow

$$H = h_1 - h_2 = Head\ Causing\ Flow$$





$$Head\ Causing\ Flow\ between\ A\ and\ B = (x + H_1 - H_2)$$

$$Length\ of\ Resisting\ Media = x$$

$$\Rightarrow Hydraulic\ Gradient\ (i) = \frac{Head\ Causing\ Flow}{Length\ of\ Resisting\ Media}$$

$$\Rightarrow (i) = \frac{x + H_1 - H_2}{x}$$

Head Loss occurs only in resisting media, here soil is resisting media

Quick Sand Condition

- $(\overline{\boldsymbol{\sigma}})\ \boldsymbol{Effective\ Stress} = \frac{\boldsymbol{Seepage\ force + Buoyant\ Force}}{\boldsymbol{A}}$
- In Case of Upward flow, seepage flow is negative and buoyant weight is positive
- If $\overline{\boldsymbol{\sigma}} = \mathbf{0}$, then shear strength of the soil

$$Shear\ Strength = c + \bar{\sigma}\ tan\emptyset$$

$$\Rightarrow \tau = c + 0 \times tan\emptyset$$

$$for\ cohesionless\ soil\ (sand), c = 0$$

$$\Rightarrow \tau = 0$$

Quick Sand Condition

- Quick Sand Condition occurs when effective stress becomes 0.
- Effective stress 0 means the contact force within soil grain is zero.
- This condition is called Quick Sand.
- We can avoid quick sand condition by lowering the water table by
  a) Lowering the Water table
  b) Increasing the depth of sand
  c) Adding surcharge on Downstream End

Critical Hydraulic Gradient

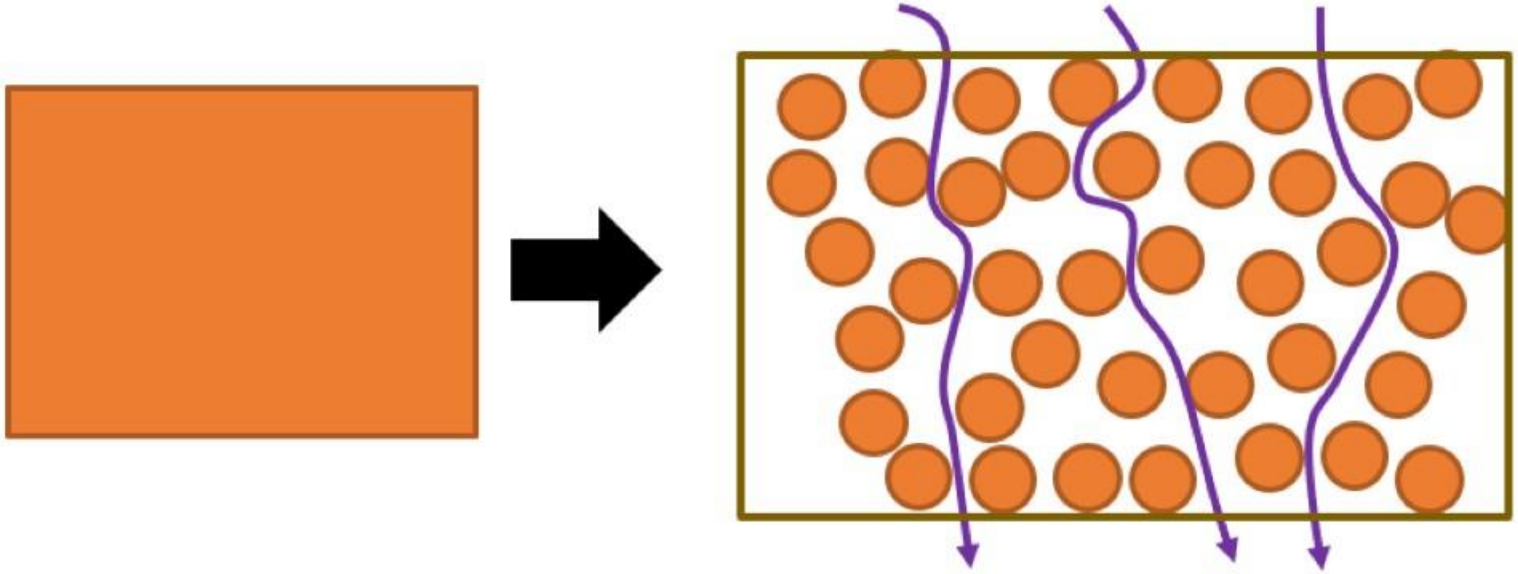
$$\boxed{i_{cr} = \frac{\gamma_{sub}}{\gamma_w}}$$

$$\boxed{i_{cr} = \frac{\gamma_{sat}}{\gamma_w} - 1}$$

$$\Rightarrow i_{cr} = \frac{G + e}{1 + e} - 1$$

$$\boxed{\Rightarrow i_{cr} = \frac{G - 1}{1 + e}}$$

Permeability of Soils



Property of soil described *quantitatively*, the ease with which water flows through the connected voids of the Soil sample is called as Permeability.

Permeability of Soils

In Permeability, one dimensional flow, through a fully saturated soil, Discharge, as mentioned by Darcy, is

$$\Rightarrow Q \propto A \quad (Area\ of\ cross\ section)$$

$$Q \propto H \quad (Head\ causing\ flow)$$

$$Q \propto \frac{1}{L} \quad (Length\ of\ resisting\ soil)$$

$$\Rightarrow Q \propto i \quad (Hydraulic\ Gradient)$$

Hence,

$$Q \propto iA$$

$$Q = KiA$$

$$where\ K = permeability\ coefficient$$

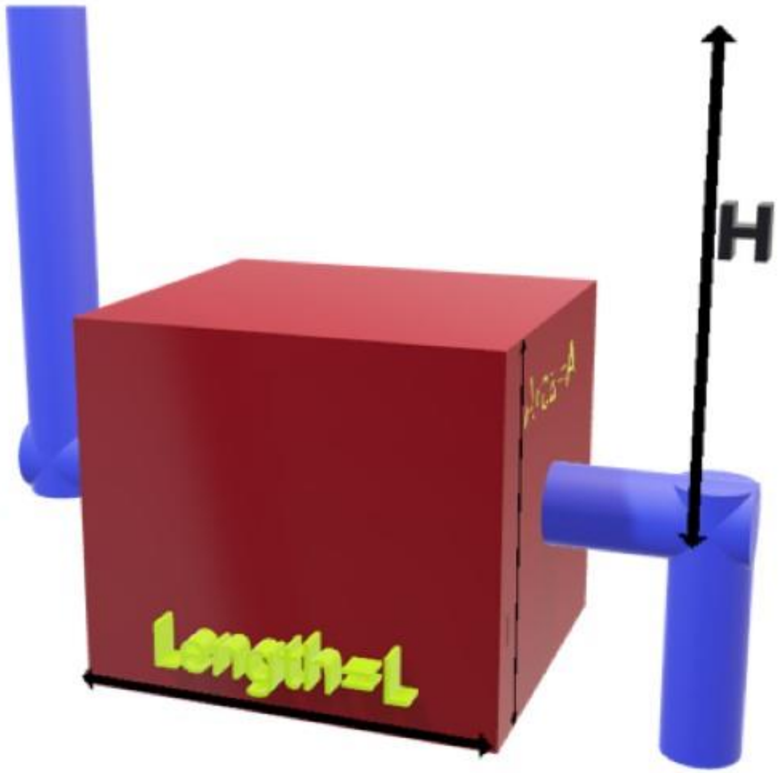
Permeability of Soils

$$Q = KiA$$

$$where\ K = permeability\ coefficient$$

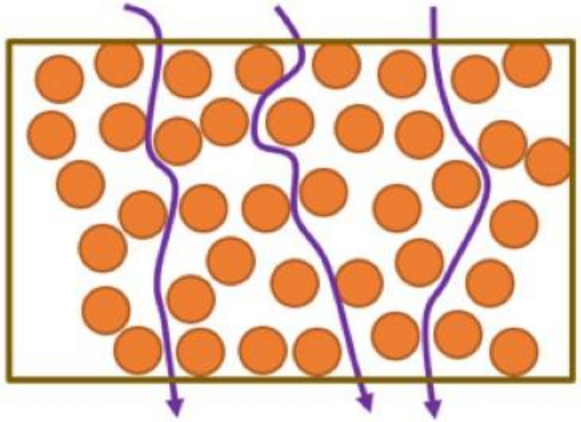
$\Rightarrow V = Ki$ *(discharge velocity)*

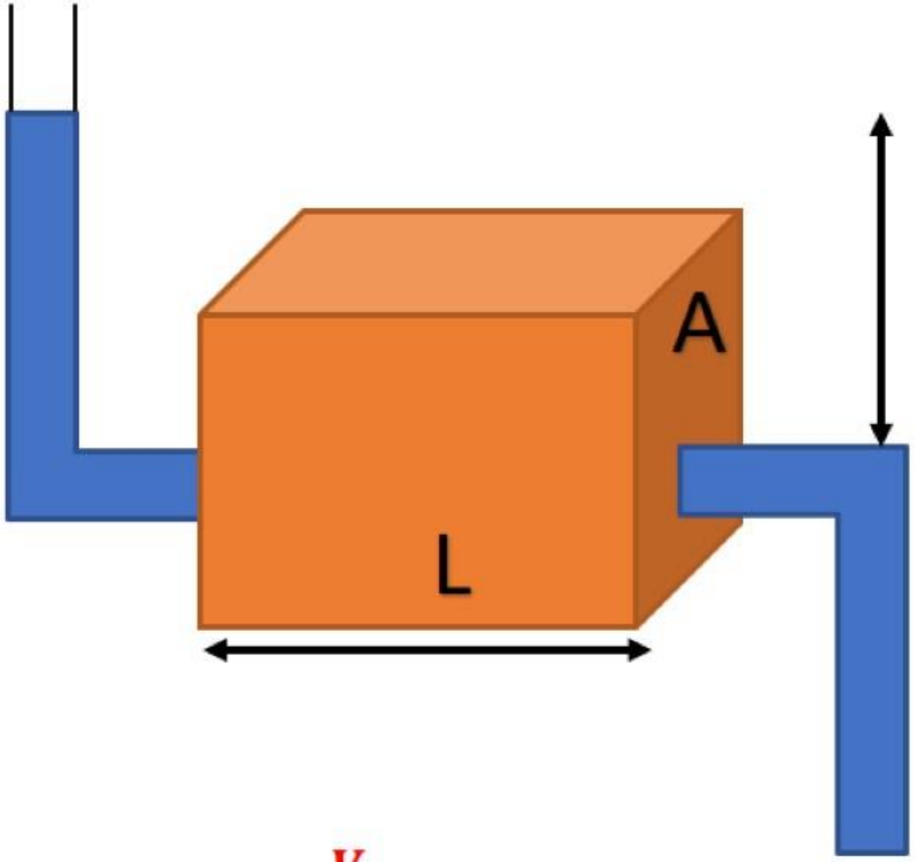
But this velocity is not the actual velocity as water seeps through area of pores, not through the total cross sectional area



Permeability

Actual Velocity





$$\Rightarrow V_a = \frac{Q}{A_v}$$

$$\Rightarrow \boldsymbol{V_a} = \frac{\boldsymbol{Q}}{\frac{\boldsymbol{A_v}}{\boldsymbol{A}}\boldsymbol{A}} \qquad \Rightarrow \boldsymbol{V_a} = \frac{\boldsymbol{V}}{\frac{\boldsymbol{A_v L}}{\boldsymbol{A L}}} \qquad \Rightarrow \boldsymbol{V_a} = \frac{\boldsymbol{V}}{\frac{\textbf{Volume of voids}}{\textbf{Total Volume}}} \qquad \boxed{\Rightarrow \boldsymbol{V_a} = \frac{\boldsymbol{V}}{\boldsymbol{n}}}$$

$$\frac{\boldsymbol{Q}}{\boldsymbol{A}} = \boldsymbol{V} = \textbf{discharge velocity}$$

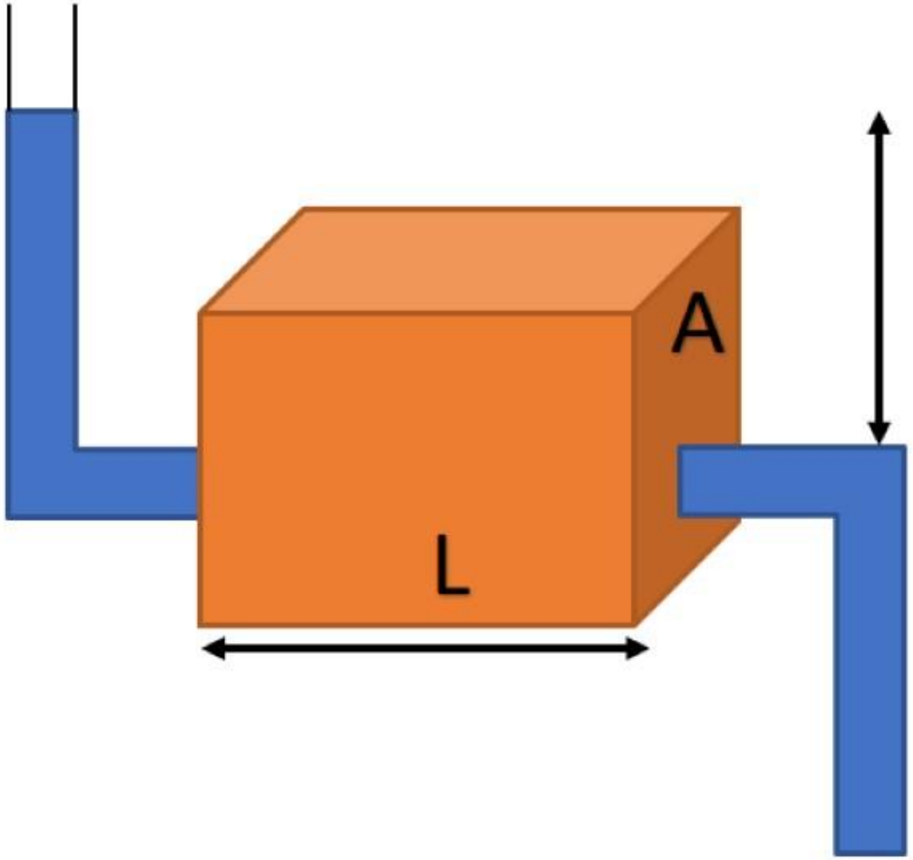
Permeability

<u>***Actual Velocity***</u>

$$\boxed{\Rightarrow V_a = \frac{V}{n}}$$

<u>***Discharge Velocity***</u>

$$\boxed{\Rightarrow V = \frac{Q}{A}}$$



Permeability

- Coefficient of Permeability depends upon:
  a) Type of Soil
  b) Particle Size
  c) Voids

effective size

Unit wt of fluid

void ratio

$$\boxed{K \ \alpha \ \ c \times d_e^2 \left(\frac{\gamma_w}{\mu}\right)\left(\frac{e^3}{1+e}\right)}$$

Composite shape factor μ = viscosity

Permeability

Soil	k (cm/sec)
Gravel	10^{0}
Coarse sand	10^{0} to 10^{-1}
Medium sand	10^{-1} to 10^{-2}
Fine sand	10^{-2} to 10^{-3}
Silty sand	10^{-3} to 10^{-4}
Silt	1×10^{-5}
Clay	10^{-7} to 10^{-9}

Effect of grain Size on Permeability

- It is given by Allen Hazen Formula

$$K = CD_{10}^2$$

$K = cm/sec$

$C = 100$

$D_{10} = cm$

The above formula is applicable to clean sands with percent fines less than 5%

Effect of Cation on Permeability

- In case of lower valency cation, permeability will be low as compared to higher valency cation, thickness of ***double diffused layer*** is high, so voids are less, hence low permeability
- Sodium Clays have smaller permeability than Calcium clays, so it is used in core of Earthen Dams

Effective Stress

- Increase in Effective stress causes decrease in Permability

Other Factors

- Presence of organic matter in soil decreases the permeability as it has tendency to flow with water and it will choke the path
- Entrapped air decreases the permeability

Methods to determine Permeability

- Permeability
  - LAB TESTING
    - Constant Head Permeability test
    - Falling Head Permeability Head
  - FIELD TESTING
    - Pumping Out Test
    - Pumping in Test

Lab Methods to determine Permeability

1. Constant Head Permeability Test

$H = \textbf{Head causing Flow, remains constant}$

$L = \textbf{Length of Resisting Media}$

$\textit{Therefore, Hydraulic gradient}$

$$\Rightarrow i = \frac{\textit{Head Causing Flow}}{\textit{Length of Resisting Media}}$$

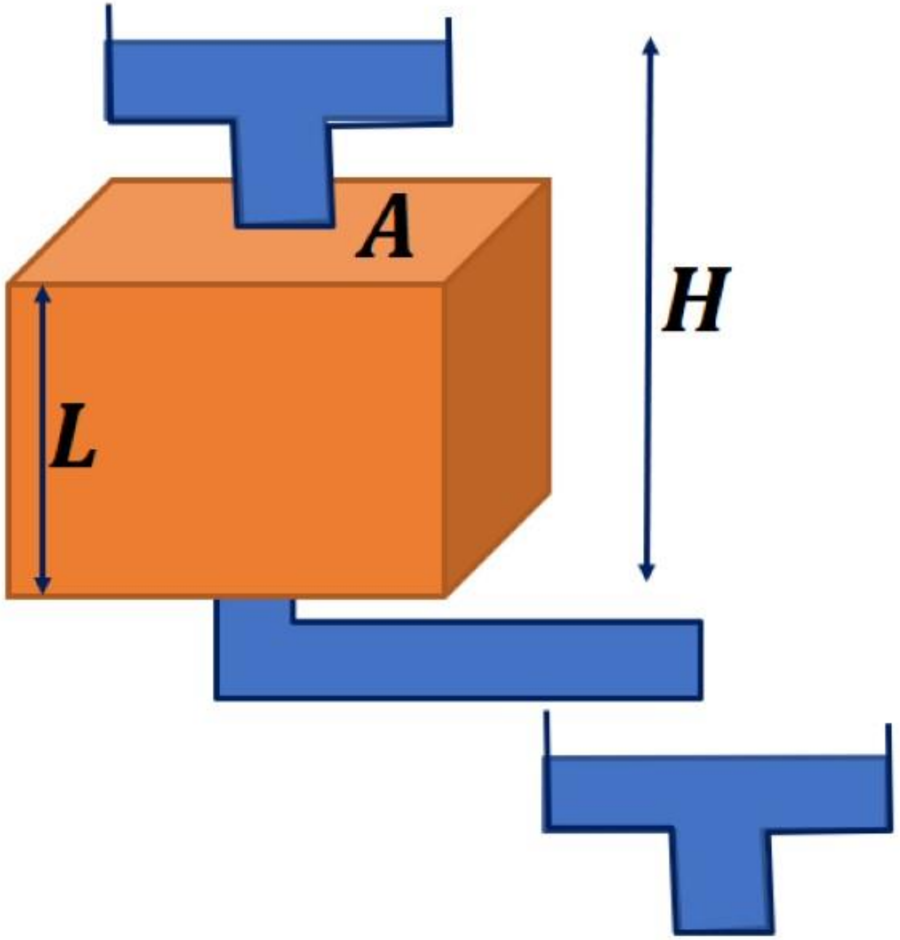
$$\Rightarrow i = \frac{H}{L}$$

$\textit{Applying Darcy's Law,}$

$$Q = KiA$$

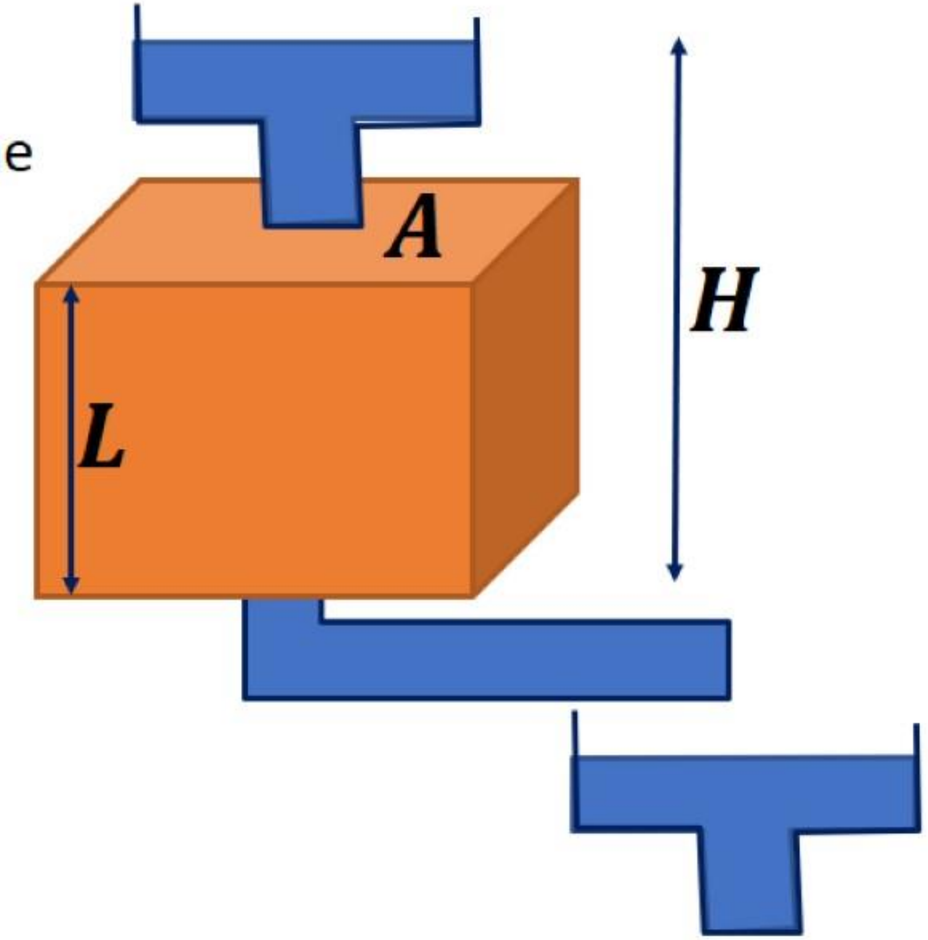
$$\Rightarrow Q = K\frac{H}{L}A$$

$$\boxed{\Rightarrow K = \frac{QL}{HA}}$$



Lab Methods to determine Permeability

1. <u>Constant Head Permeability Test</u>
   a) Constant head permeability test is done for coarse grain soils only (Sand and Gravel)
   b) This test is not used for fine grain soil because it is difficult to measure discharge volume



Methods to determine Permeability

2. Variable Head Permeability Test

$$a = area\ of\ stand\ pipe$$

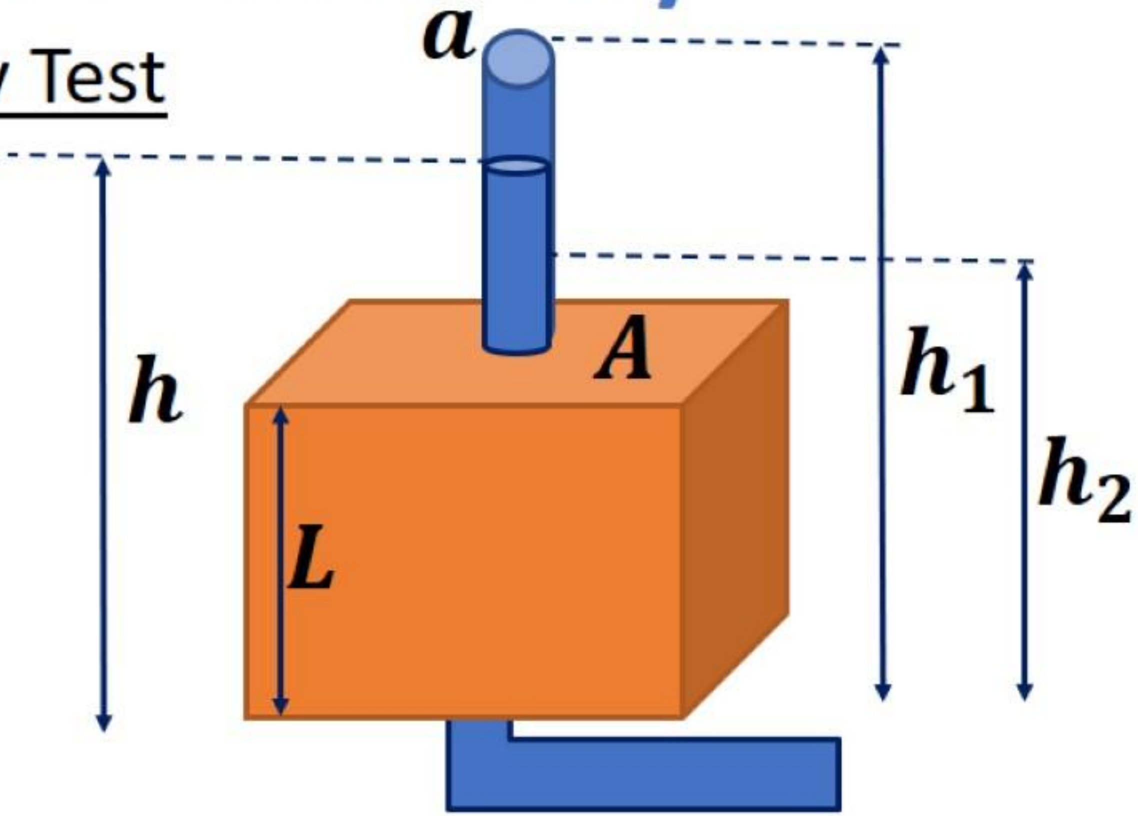
$$-\frac{dh}{dt} = V$$

$$Acc\ to\ Darcy, \qquad Q = KiA$$

$$Q = Va \qquad = -\frac{dh}{dt} \times a$$

$$\Rightarrow -\frac{dh}{dt} \times a = K\frac{h}{L}A$$

$$\Rightarrow -\frac{dh}{h} = \frac{KA}{La}dt$$



Lab Methods to determine Permeability

2. <u>Variable Head Permeability Test</u>

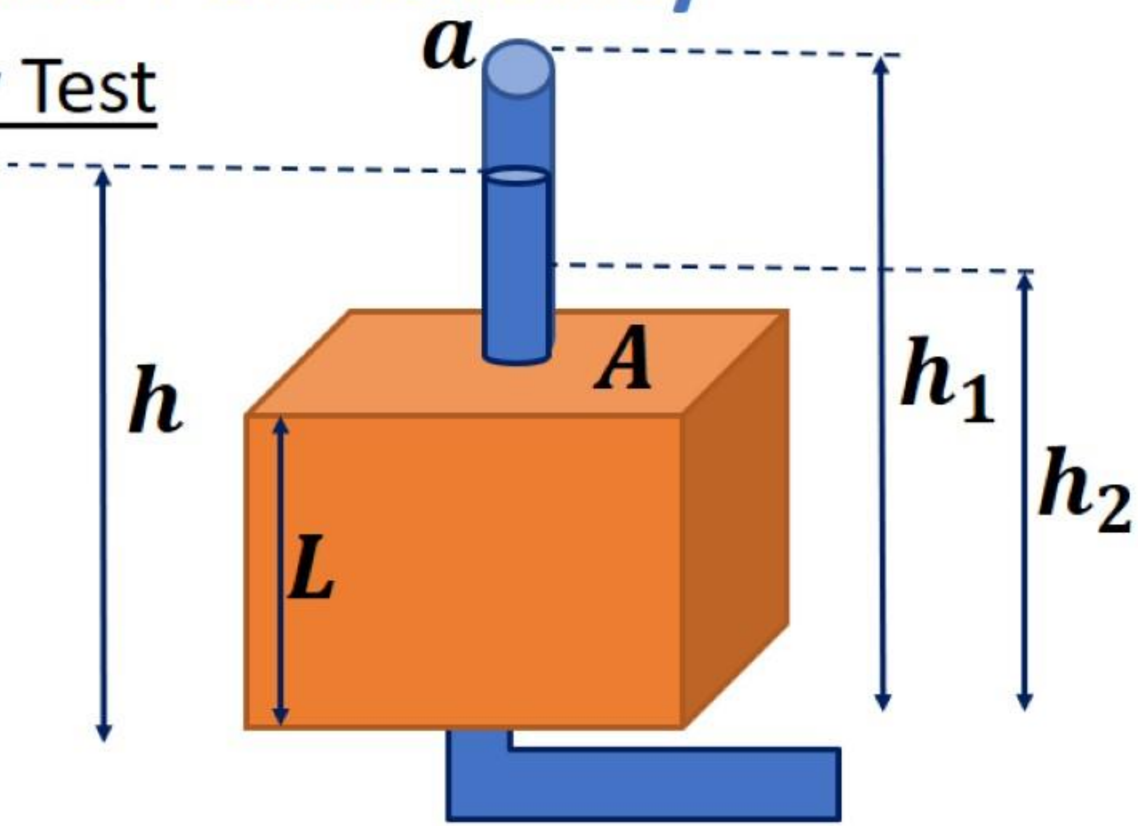
Integrating both sides

$$\Rightarrow -\int_{h_1}^{h_2} \frac{dh}{h} = \frac{KA}{La}\int_0^t dt$$

$$\Rightarrow -ln\left(\frac{h_2}{h_1}\right) = \frac{KA}{La}t$$

$$\Rightarrow +ln\left(\frac{h_1}{h_2}\right) = \frac{KA}{La}t$$

$$\Rightarrow K = \frac{La}{At}ln\left(\frac{h_1}{h_2}\right)$$



$$\boxed{\Rightarrow K = 2.303\,\frac{La}{At}\,log_{10}\left(\frac{h_1}{h_2}\right)}$$

Lab Methods to determine Permeability

2. Variable Head Permeability Test

a) In this case, sample used is undisturbed sample

b) Variable Head Permeability Test is done for fine grain soil (clay, silt)

c) This test is not suitable for coarse grain soil because it would discharge in very less time

Aquifer

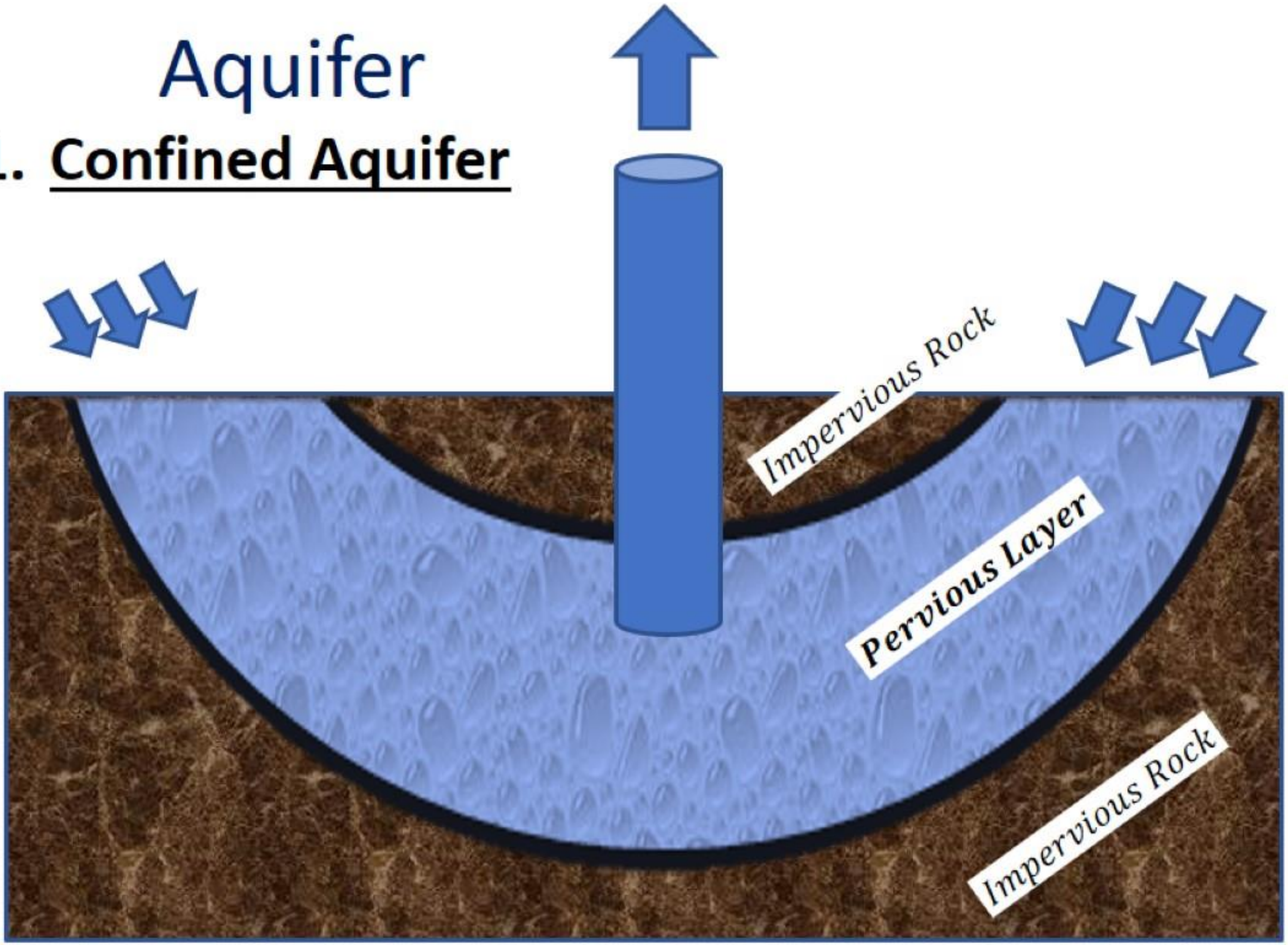
- A soil deposit which is pervious in nature and allows extraction of water.
- These are mainly of two types:
  a) Confined aquifer
  b) Unconfined aquifer

✓ **High Porosity**

✓ **High Permeability**

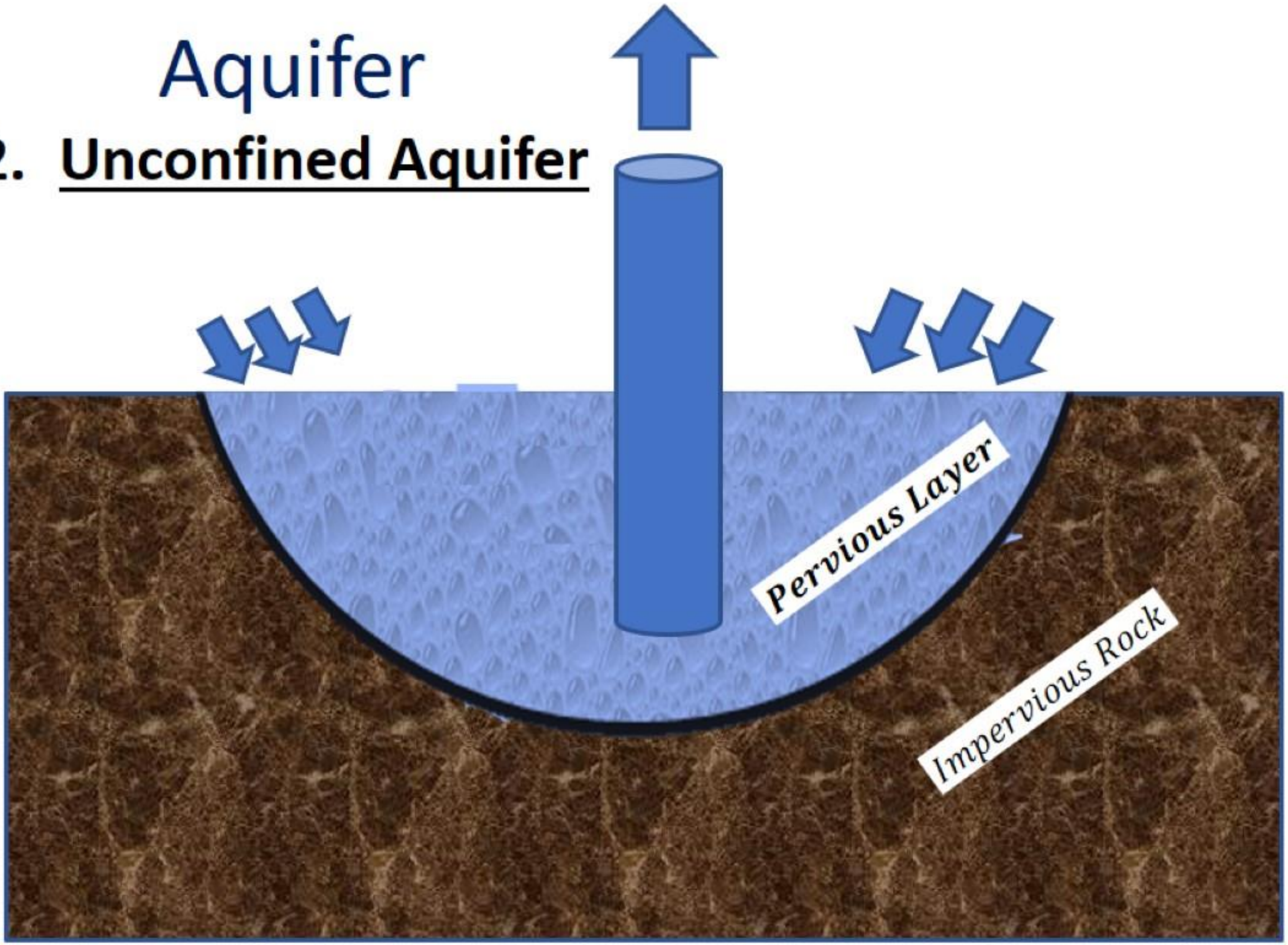
Aquifer

1. Confined Aquifer



Aquifer

2. Unconfined Aquifer



ACQUITARD

- It is the formation through which only seepage is possible and thus the ***yield is insignificant*** as compared to an acquifer.
- It is partly permeable.
- Example – clay lenses interbedded with sand.

AQUICLUDE

- A soil deposit that contains water but which is not capable of transmitting adequate amount of water.
- Generally it is clay.
- They have high porosity and low permeability.

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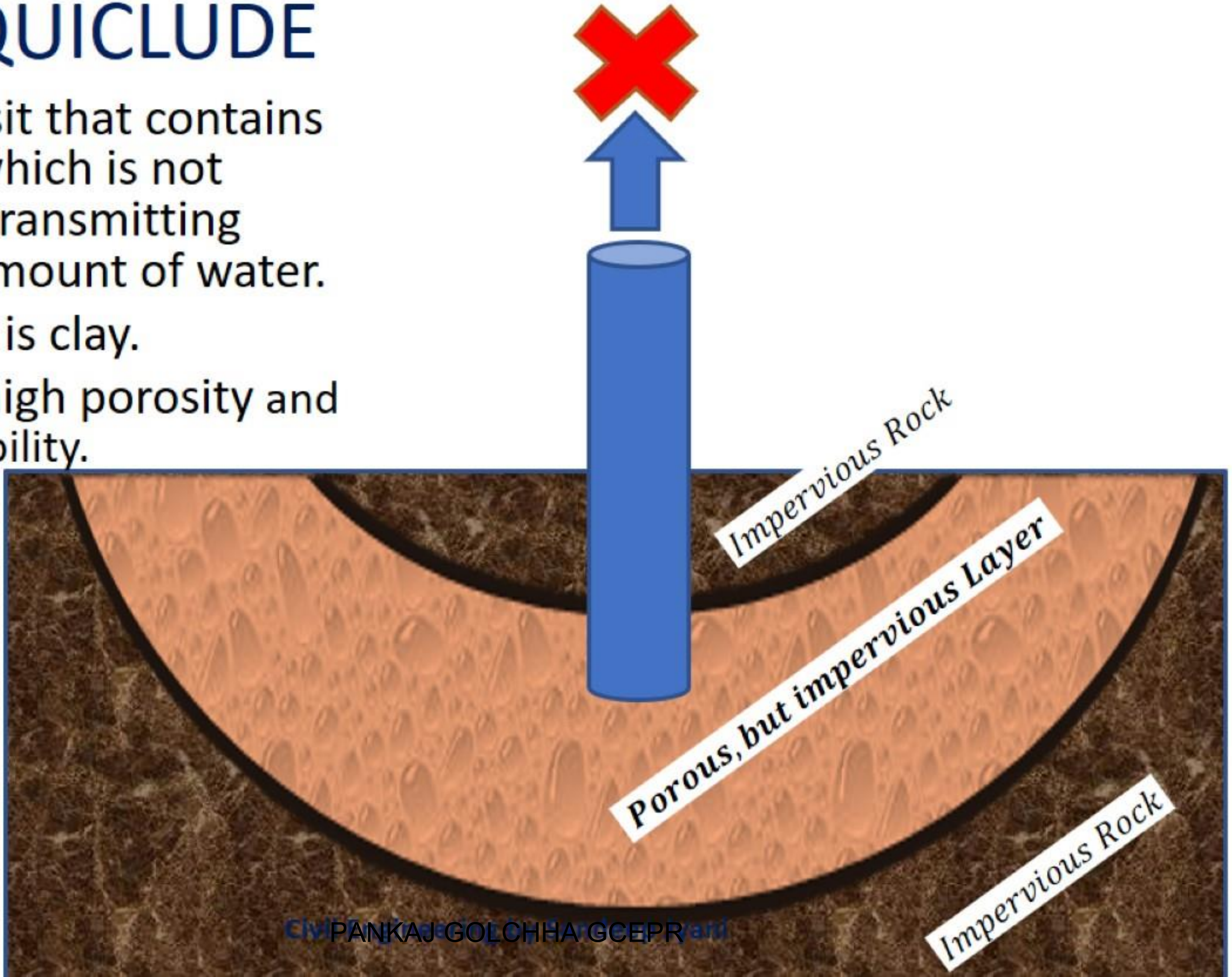


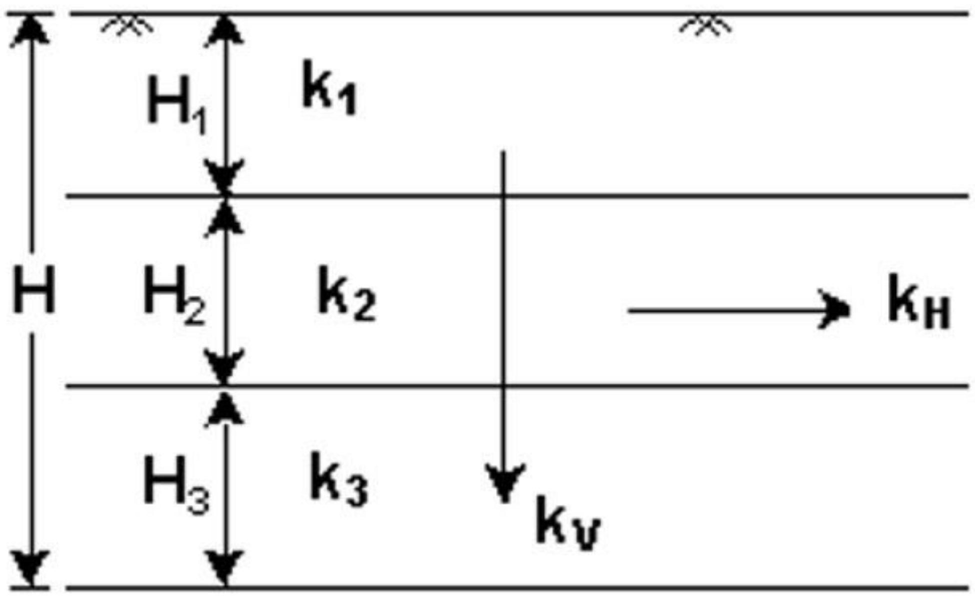
ACQUIFUSE

- It is a rocky mass which neither store not transmit the water
  a) Non porous
  b) Non permeable

Permeability of Stratified Deposits

$$K_{Horizontal} = \frac{K_1 H_1 + K_2 H_2 + K_3 H_3}{H_1 + H_2 + H_3}$$

$$K_{Vertical} = \frac{H_1 + H_2 + H_3}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3}}$$



Que. A stratified soil deposit has 3 layers of thickness 4m, 1m, 2m and corresponding permeability 2m/sec, 1m/sec, 4 m/sec respectively. Find the ratio of average permeability perpendicular to the bedding plane and parallel to the bedding plane

$$K_{Horizontal} = \frac{K_1 H_1 + K_2 H_2 + K_3 H_3}{H_1 + H_2 + H_3}$$

$$K_{Vertical} = \frac{H_1 + H_2 + H_3}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3}}$$

Que. A horizontal stratified soil deposit consists of 3 layers each uniform in itself. The permeability of the layer are

$$8 \times 10^{-4}\ cm/sec$$
$$50 \times 10^{-4}\ cm/sec$$
$$15 \times 10^{-4}\ cm/sec$$

And their thickness are 6m, 3m and 12m respectively.

Find the effective average permeability of the deposit in horizontal and vertical direction.

$$\boxed{K_{Horizontal} = \frac{K_1 H_1 + K_2 H_2 + K_3 H_3}{H_1 + H_2 + H_3}}$$

$$\boxed{K_{Vertical} = \frac{H_1 + H_2 + H_3}{\frac{H_1}{K_1} + \frac{H_2}{K_2} + \frac{H_3}{K_3}}}$$

SOIL MECHANICS

Civil Engineering by Sandeep Jyani

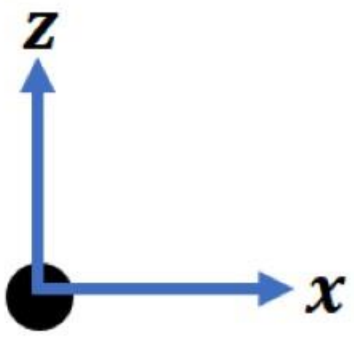
Seepage

Seepage through Soil in 2-D

- In 2-D conditions, following assumptions are made:
  a) Soil is fully saturated
  b) Darcy's law is applicable in both directions

$$\Rightarrow V_x = K_x i$$

$$\Rightarrow V_z = K_z i$$



"i" is same at this point for both direction

  c) Soil Homogenous and Isotropic

$$\Rightarrow K_x = K_z$$

$$\Rightarrow V_x = K_x i_x$$

$$\Rightarrow V_z = K_z i_z \qquad i_x = \frac{\partial h}{\partial x} \qquad i_z = \frac{\partial h}{\partial z}$$

Seepage through Soil in 2-D

- In 2-D conditions, following assumptions are made:
  d) Soil grain and water are incompressible and steady state condition is reached
  e) Continuity Equation is valid for 2-D flow

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \quad for\ 3D$$

$$\frac{\partial u}{\partial x} + \frac{\partial w}{\partial z} = 0 \quad for\ 2D$$

$$\Rightarrow \frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z} = 0$$

$$\Rightarrow \frac{\partial (K_x i_x)}{\partial x} + \frac{\partial (K_z i_z)}{\partial z} = 0$$

$$\Rightarrow \frac{\partial (K_x \frac{\partial h}{\partial x})}{\partial x} + \frac{\partial (K_z \frac{\partial h}{\partial z})}{\partial z} = 0$$

$$Since\ K_x = K_z$$

$$\Rightarrow \frac{\partial (\frac{\partial h}{\partial x})}{\partial x} + \frac{\partial (\frac{\partial h}{\partial z})}{\partial z} = 0$$

$$\boxed{\Rightarrow \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0}$$

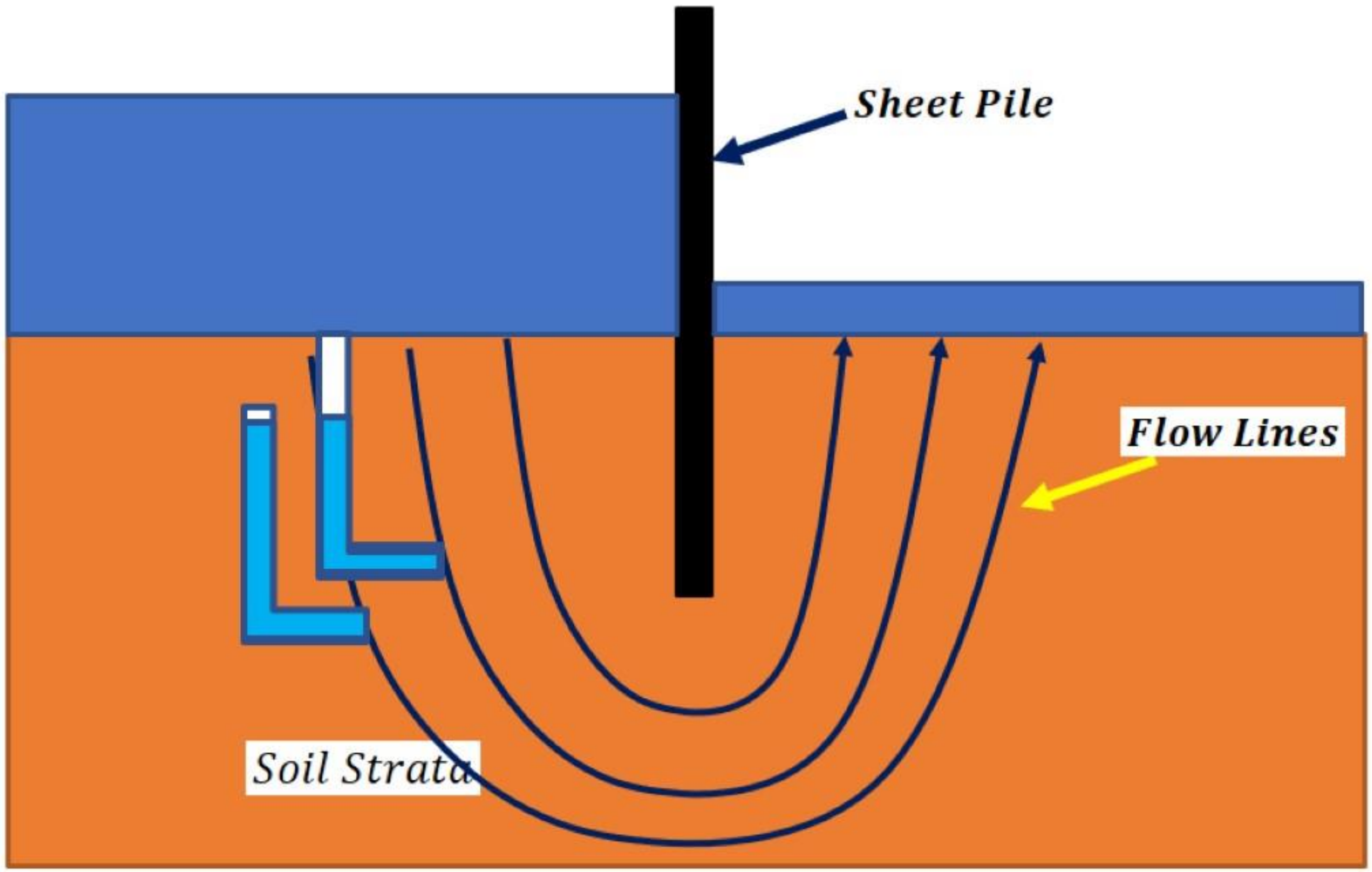
$$Which\ is\ laplace\ equation$$

Seepage through Soil in 2-D

$$\boxed{\Rightarrow \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0}$$

Which is laplace equation

- General Solution of Laplace Equation gives two sets of orthogonal curve known as Flow Line and other set is called as Equipotential Line


Sheet Pile
Flow Lines
Soil Strata

$$\frac{\alpha}{\beta} = 1 \ (\textit{Elementary square})$$

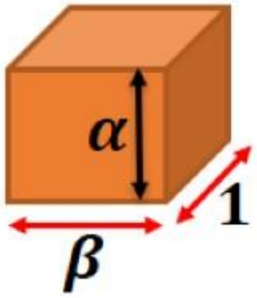
$$n_f = \textit{no. of flow channels}$$

$$n_d = \textit{no. of equipotential drops}$$

$$\Rightarrow i = \frac{\Delta h}{\beta}$$

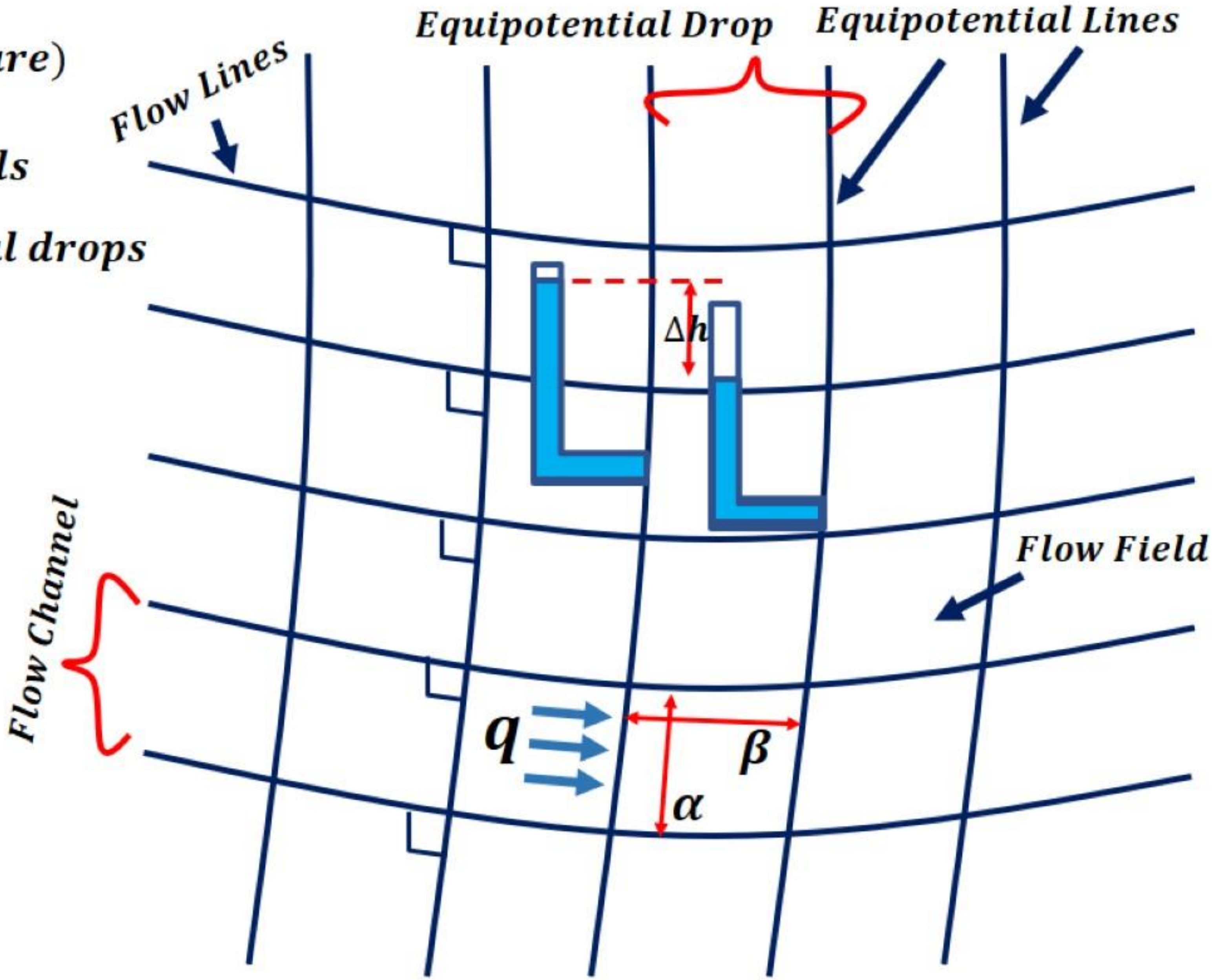
$$\Rightarrow q = KiA$$

$$A = \alpha \times 1$$



$$\Rightarrow \Delta h = \frac{H}{n_d}$$

$$\Rightarrow q = K\left(\frac{\frac{H}{n_d}}{\beta}\right)(\alpha \times 1)$$



$$\Rightarrow q = K\left(\frac{\frac{H}{n_d}}{\beta}\right)(\alpha \times 1)$$

$$\Rightarrow q = K\left(\frac{H}{n_d}\right)\left(\frac{\alpha}{\beta}\right)$$

$$\Rightarrow q = K\left(\frac{H}{n_d}\right) \quad \textit{This discharge is for one channel}$$

For Total discharge, multiply with number of flow channels

$$\boxed{\Rightarrow Q = \frac{KHn_f}{n_d}}$$

$n_f = no.\,of\ flow\ channels = no.\,of\ flow\ lines - 1$

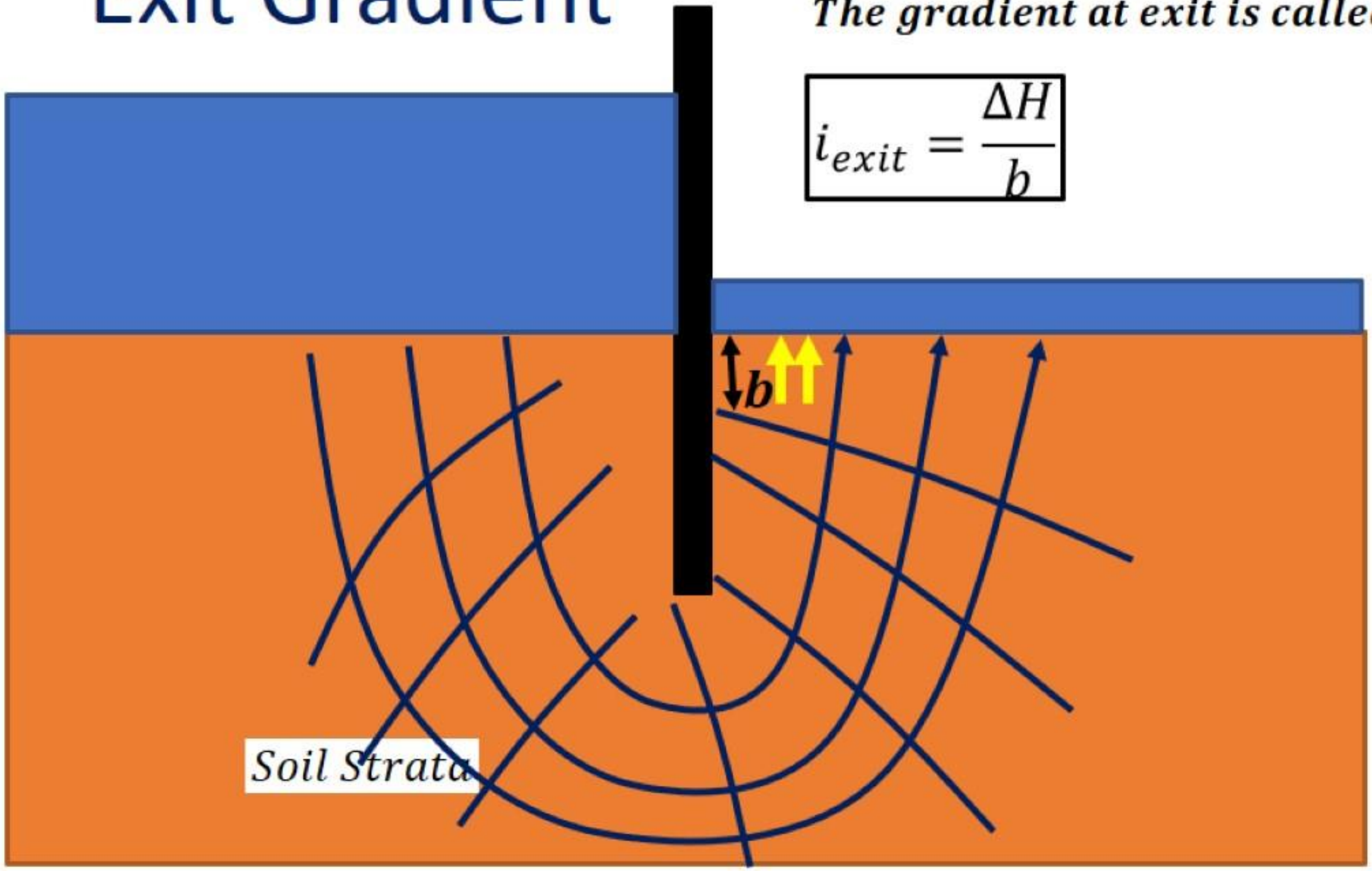
$n_d = no.\,of\ equipotential\ drops = no.\,of\ equipotential\ lines - 1$

$$\frac{n_f}{n_d} = Shape\ Factor$$

Exit Gradient

The gradient at exit is called Exit Gradient

$$\boxed{i_{exit} = \frac{\Delta H}{b}}$$



The exit gradient must never come close to critical hydraulic Gradient

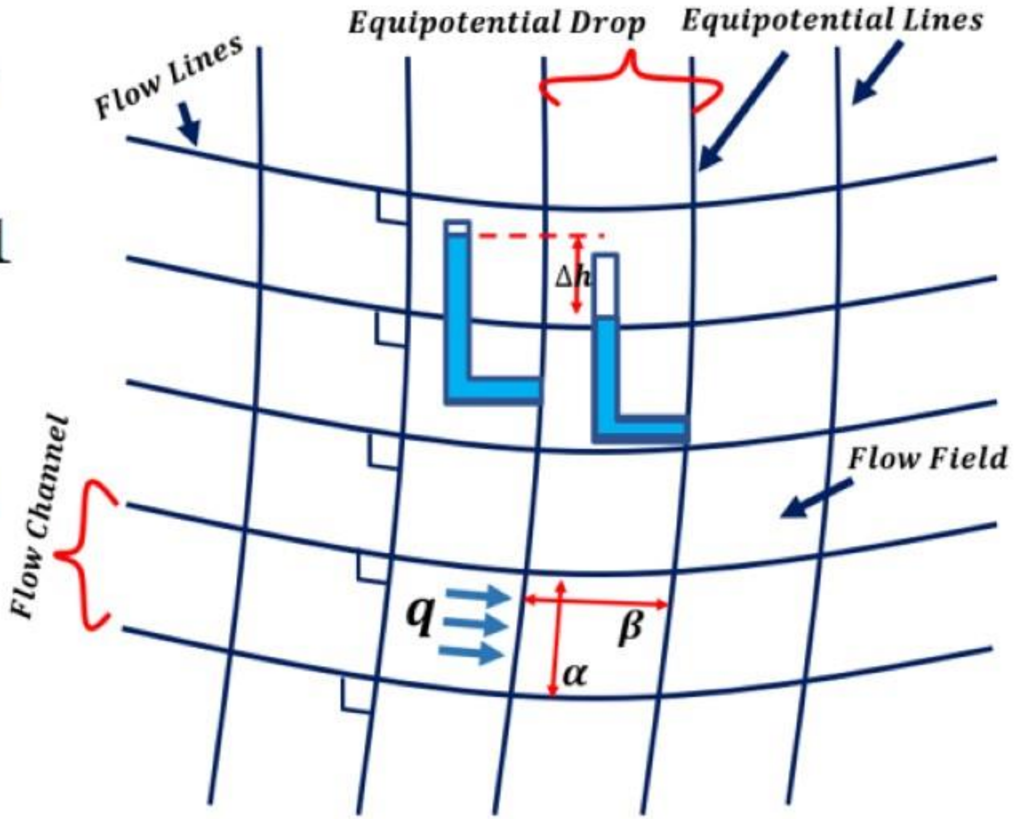
Exit Gradient

- $Factor\ of\ Safety = \frac{i_{cr}}{i_{exit}}$
- FOS of at least 6 is recommended for safety against piping

- Note: Flow net will not change if permeability of soil changes, if head loss to the flow is different or direction of flow changes
- Flow net changes only when Flow space (boundary) is changed

Properties of Flow Net

1. Flow line and Equipotential line are orthogonal to each other (Valid only for homogenous and isotropic soil)
2. Flow field are elementary Squares $\frac{\alpha}{\beta} = 1$
3. Head loss through each successive equipotential line are same
4. Discharge through each Flow channel is same
5. Shape factor $\frac{n_f}{n_d}$ is a function of boundary condition



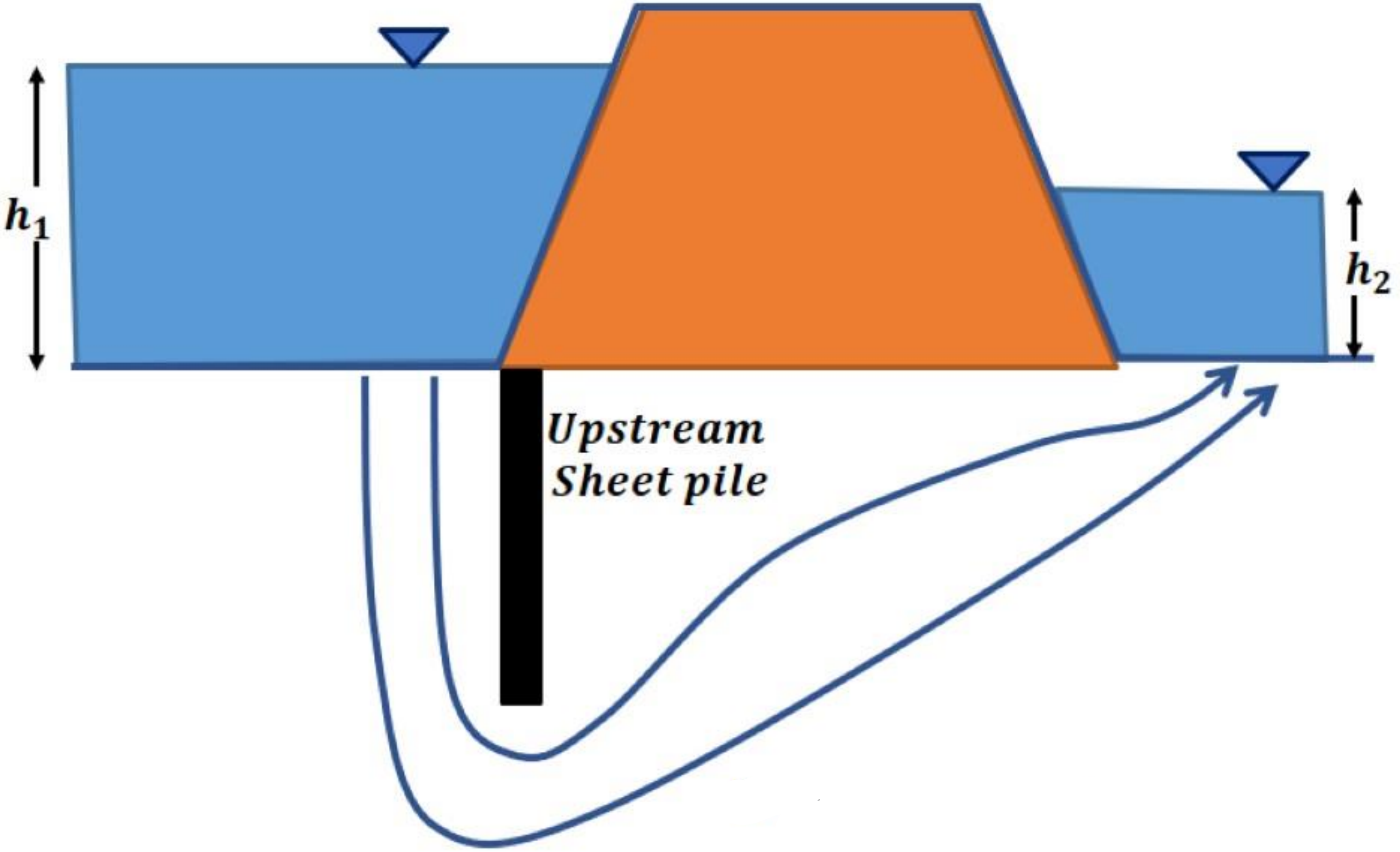
Uses of Flow Net

1. To calculate amount of Seepage
2. To calculate coefficient of Permeability
3. To calculate Effective Stress
4. To calculate Uplift Pressure

1. To Reduce Uplift Pressure

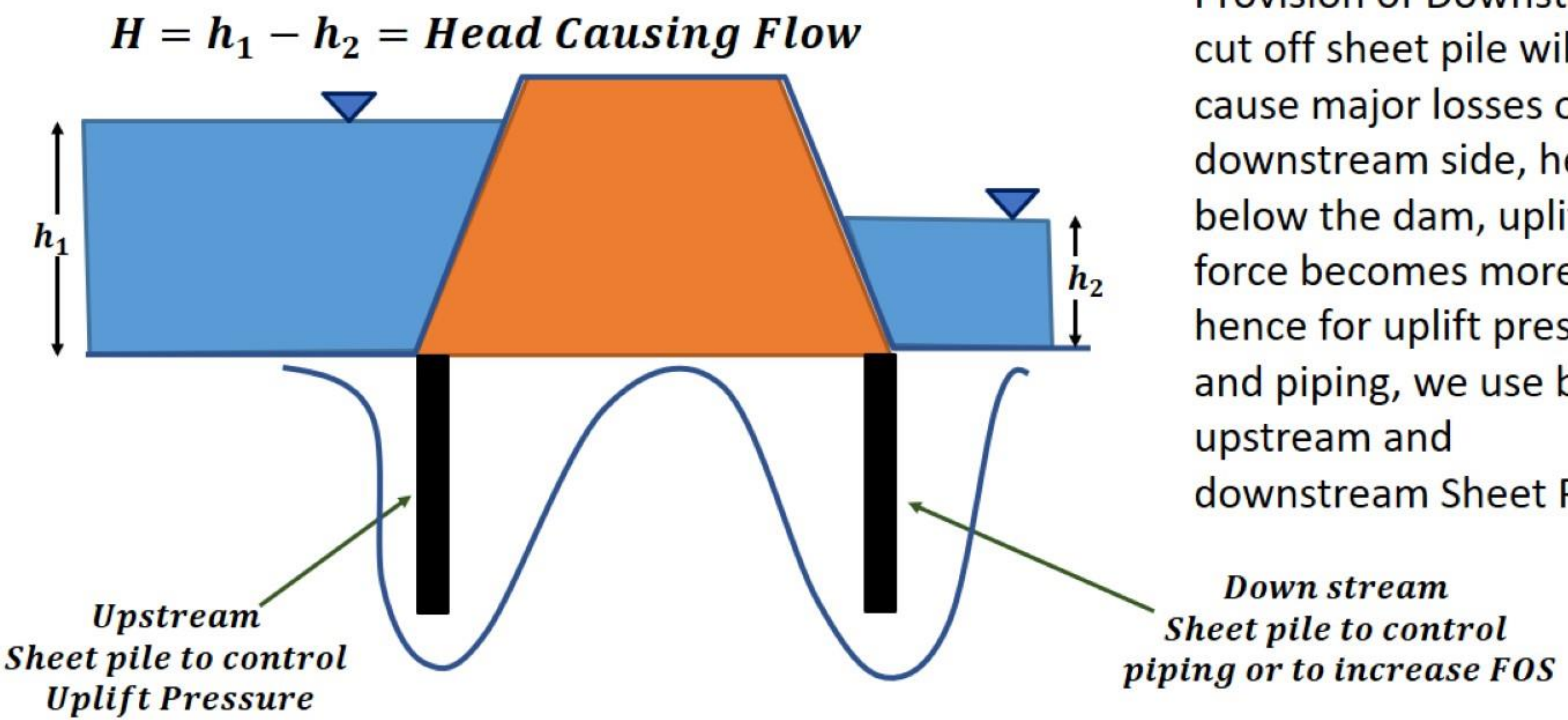
- Uplift Pressure along the dam can be reduced by providing vertical sheet pile or cut off pile at the up stream end of the dam

$$H = h_1 - h_2 = Head\ Causing\ Flow$$



Length of Flow Path increases and by time, water come up to the base of dam much of the head is already lost and pressure head remaining would be less

2. For Piping and Exit Gradient



Provision of Downstream cut off sheet pile will cause major losses on downstream side, hence below the dam, uplift force becomes more, hence for uplift pressure and piping, we use both upstream and downstream Sheet Pile.

Analysis of Seepage Force: ***a) Under Hydrostatic Condition***

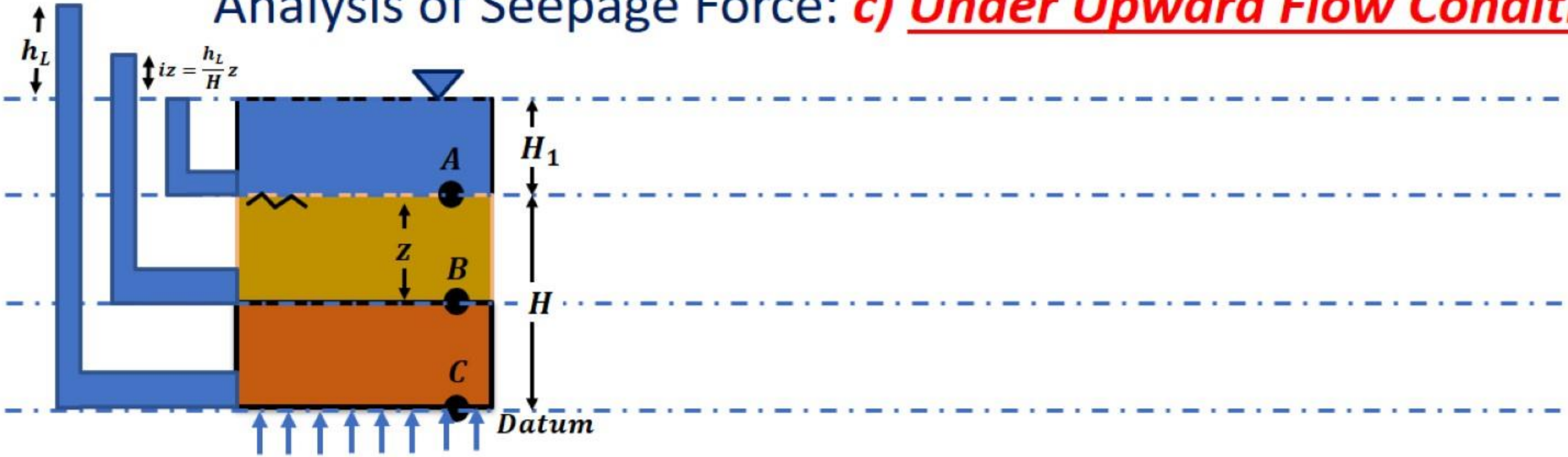
Points	Pressure Head (Piezometer reading)	Elevation Head (above datum)	Total Head
A			
B			
C			

Analysis of Seepage Force: ***b) Under Downward Flow Condition***

Points	Pressure Head (Piezometer reading)	Elevation Head (above datum)	Total Head
A			
B			
C			

Analysis of Seepage Force: ***b) <u>Under Downward Flow Condition</u>***

Analysis of Seepage Force: *c) Under Upward Flow Condition*



Points	Pressure Head (Piezometer reading)	Elevation Head (above datum)	Total Head
A			
B			
C			

Analysis of Seepage Force: ***c) Under Upward Flow Condition***

SOIL MECHANICS

Civil Engineering by Sandeep Jyani

Compaction

Compaction

Compaction of Soil is the process of increasing the unit weight of Soil by Forcing the Soil Solid into a dense state by Removal of Air Void.

- Compaction Leads to increase in Shear Strength of and thus helps in ensuring the stability and sufficient bearing capacity of Soil
- It also reduces the Permeability and Compressibility
- It reduces subsequent settlement under working loads.
- It is measured Quantitatively in terms of Dry Unit Weight of Soil (γ_d)

Compaction

- **Soil is Partially Saturated**
- **Compaction is instantaneous process/immediate**
- **In this process, impact loading is used**
- **Densification of Soil occurs due to reduction in air voids**

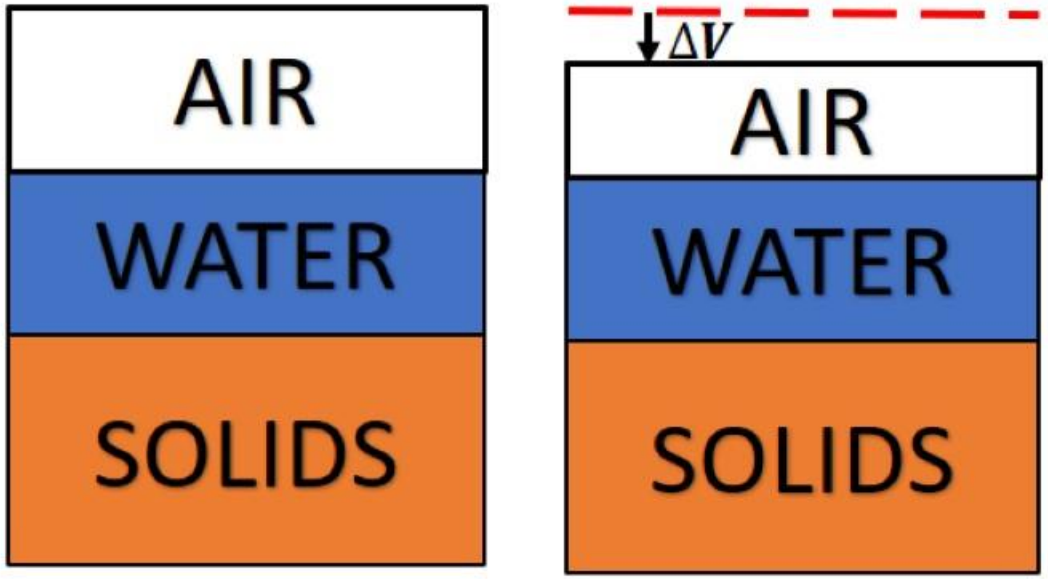
Compaction Vs Consolidation

Sr No	Compaction	Consolidation

Compaction Vs Consolidation

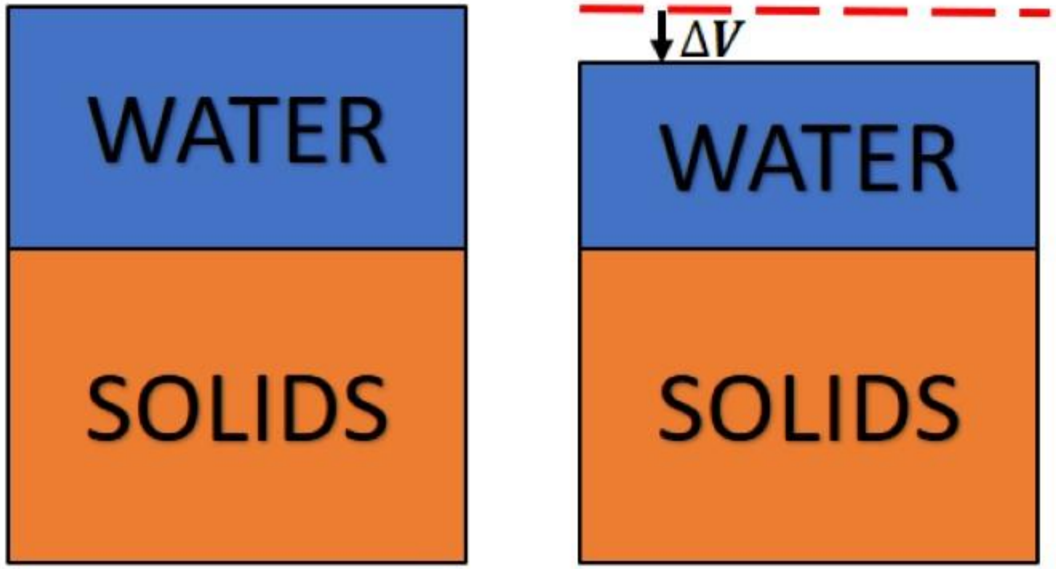
Sr No	Compaction	Consolidation
1.	It is an instantaneous process	It is time dependent process
2.	Soil is always UNSATURATED	Soil is competely saturated
3.	Densification is due to reduction in the volume of air voids at a given water content	Volume Reduction is due to Expulsion of pore water from the voids
4.	Compaction techniquies are used in compaction	Consolidation occurs on account of a Load Placed on the soil

Compaction



Before Compaction　　*After Compaction*

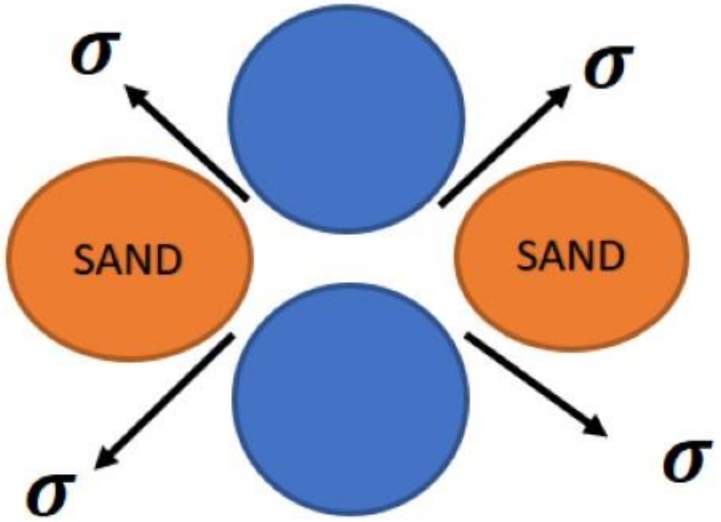
Consolidation



Before Consolidation　　*After Consolidation*

Compaction of Sand

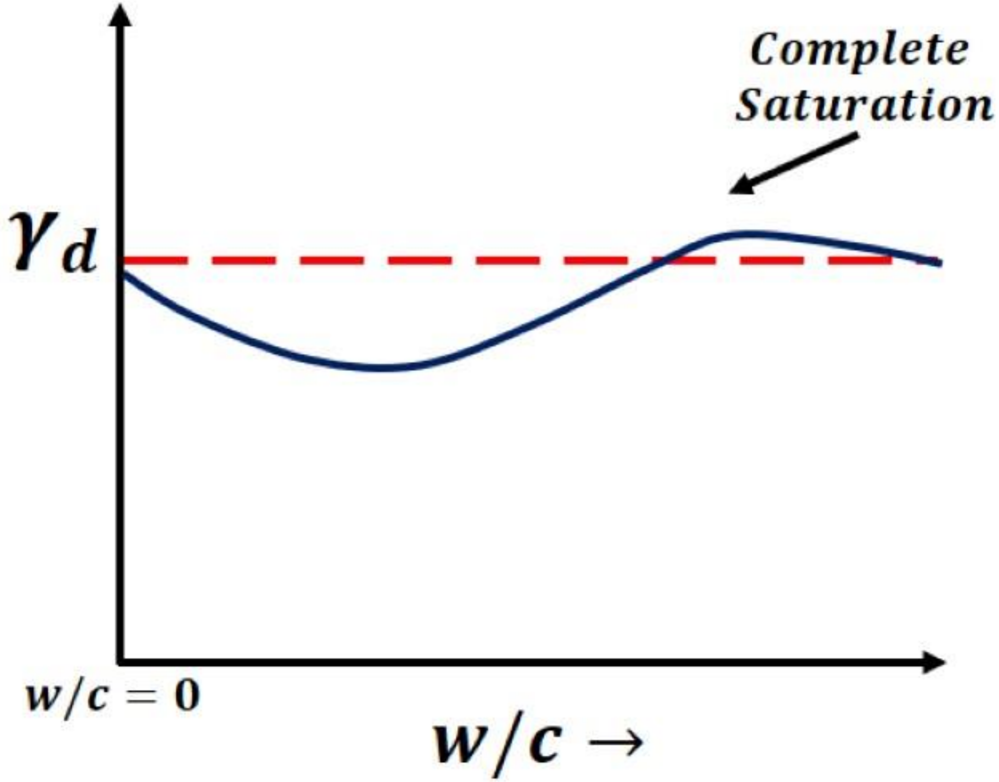
- In Case of Sand / coarse grain soil at around 4 to 5% water content, surface tension or capillary tension will develop and volume of sand increases as Capillary tension prevent soil particles coming closer



- This Phenomenon is known as ***Bulking of Sand***

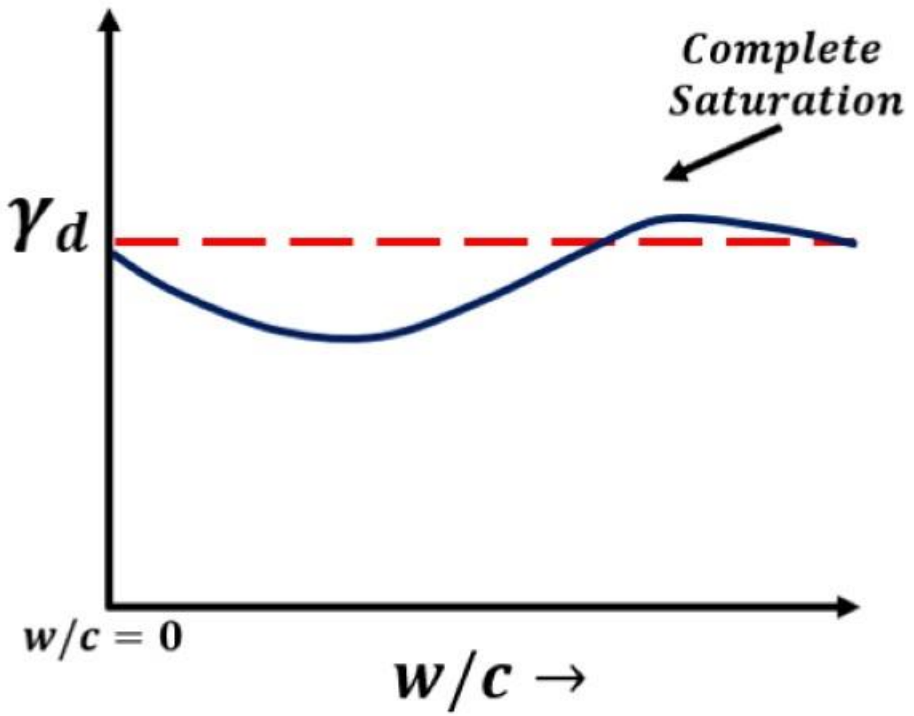
Compaction of Sand

- In Case of Sand / coarse grain soil at around 4 to 5% water content, surface tension or capillary tension will develop and volume of sand increases

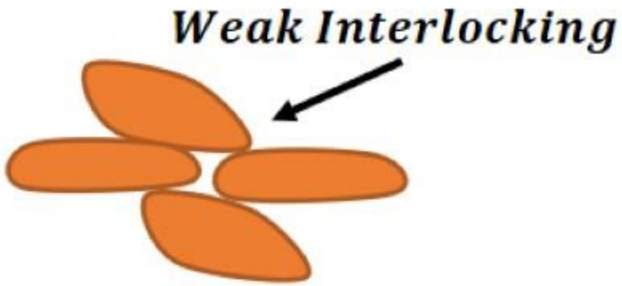


Compaction of Sand

- **In Case of Sand / coarse grain soil at around 4 to 5% water content, surface tension or capillary tension will develop and volume of sand increases**
- **As the moisture content increases, capillary tension decreases and soil goes into denser packing and thus γ_d increases**
- **As the water content increases further, capillary tension is removed, hence maximum density in sandy soil can be achieved either completely dry or Completely Saturated Condition**
- ***Sandy Soil has maximum Dry density at dry condition and Completely Saturated condition, nearly the same.***



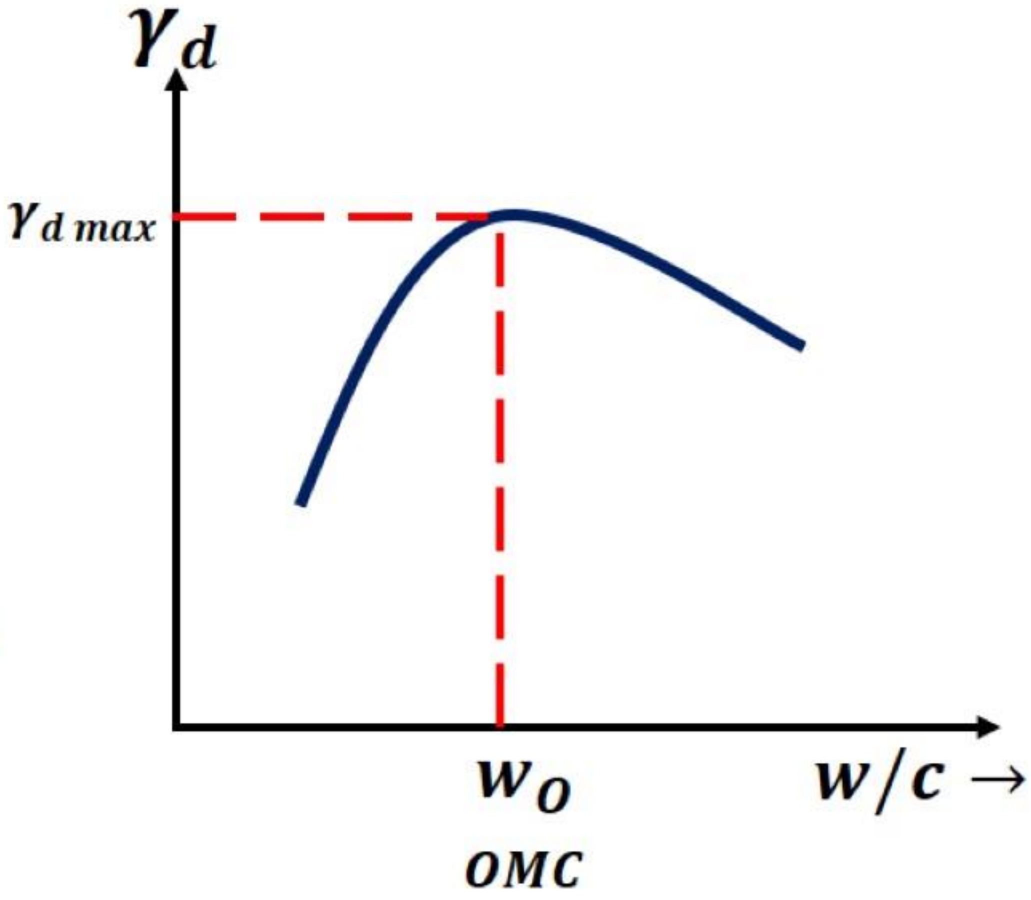
Compaction of Cohesive Soil



- Cohesive Soil has weak Interlocking Forces
- This Interlocking can not be broken by Vibration(as in clay, charges are there)
- Compaction of cohesive Soil is done by Sheep Foot Roller
- Sheep foot Roller in case of clay which exert high contact pressure forcing particles to Slide

Proctor Compaction Test

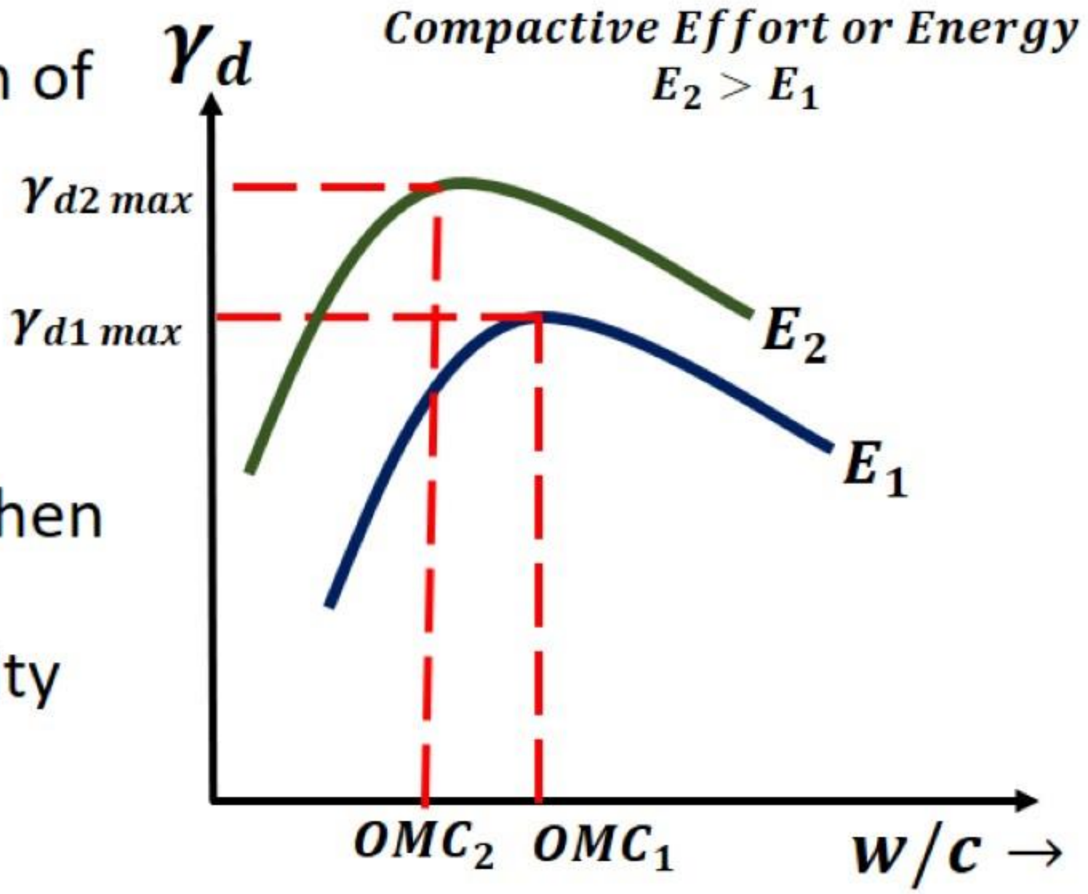
- As per Proctor, a definite relationship exist between the soil moisture content and dry density to which soil can be compacted.
- For a specific amount of Energy applied to soil, there is one moisture content found out as **Optimum Moisture Content** at which a particular soil attains maximum Dry Density



Maximum Dry Density ($\gamma_{d\,max}$)

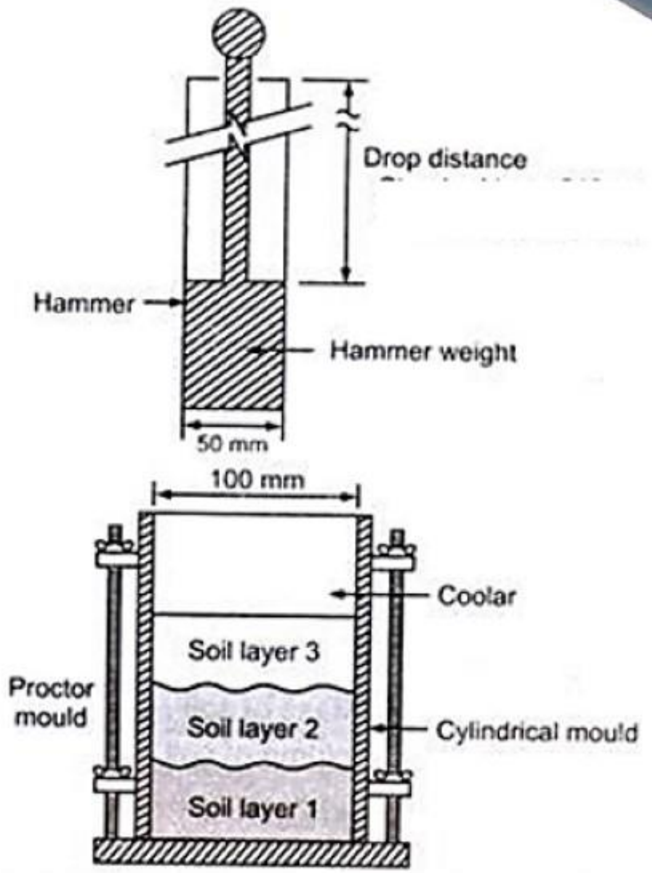
- Maximum dry density is a function of
  a) Compactive Effort
  b) Type of Soil
  c) Moisture Content
  d) Method of Compaction
- If compactive Effort is increased, then Optimum Moisture Content will decrease and Maximum Dry Density will increase i.e. If $E_2 > E_1$, so

$$OMC_2 < OMC_1$$
$$\gamma_{d2\,max} > \gamma_{d1\,max}$$



Tests for Compaction

1. Standard Proctor test:
    a) Wt of hammer = 2.5 kg
    b) Height of fall= 305mm
    c) Volume of mould= 944cc
    d) 3layers->25 no. Of blows



Energy Imparted in Standard Proctor Test

$$\Rightarrow \frac{Energy}{Volume} = \frac{(mgh) \times (no. of\ blows) \times (no. of\ layers)}{944}$$

$$\Rightarrow \frac{Energy}{Volume} = \frac{(2.5 \times 9.81 \times 0.305) \times (25) \times (3)}{944}$$

$$\boxed{\Rightarrow E_1 = 594.24\ \frac{kJ}{m^3}}$$

Tests for Compaction

2. Modified Proctor test
   a) Wt of hammer = **4.5** kg
   b) Height of fall= **457.2mm**
   c) Volume of mould= 944cc
   d) **5layers**->25 no. Of blowS

$$\boldsymbol{Energy\ Imparted\ in\ Modified\ Proctor\ Test}$$

$$\Rightarrow \frac{\boldsymbol{Energy}}{\boldsymbol{Volume}} = \frac{(\boldsymbol{mgh}) \times (\boldsymbol{no.of\ blows}) \times (\boldsymbol{no.of\ layers})}{\mathbf{944}}$$

$$\Rightarrow \frac{\boldsymbol{Energy}}{\boldsymbol{Volume}} = \frac{(\mathbf{4.54} \times \mathbf{9.81} \times \mathbf{0.457}) \times (\mathbf{25}) \times (\mathbf{5})}{\mathbf{944}}$$

$$\boxed{\Rightarrow \boldsymbol{E_2} = \mathbf{2695.10}\ \frac{\boldsymbol{kJ}}{\boldsymbol{m^3}}}$$

$$\boxed{\Rightarrow \boldsymbol{Therefore\ E_2} = \mathbf{4.51}\boldsymbol{E_1}}$$

Tests for Compaction

3. *Indian Standard Light Compaction Test* :

a) Wt of hammer = 2.6 kg
b) Height of fall= 310mm
c) Volume of mould= 1000 cc
d) 3layers->25 no. Of blows

Energy Imparted in Indian Standard Proctor Test

$$\Rightarrow \frac{Energy}{Volume} = \frac{(mgh) \times (no.of\ blows) \times (no.of\ layers)}{1000cc}$$

$$\Rightarrow \frac{Energy}{Volume} = \frac{(2.6 \times 9.81 \times 0.310) \times (25) \times (3)}{1000cc}$$

$$\boxed{\Rightarrow E_1' = 593.01 \frac{kJ}{m^3}}$$

Tests for Compaction

4. *Indian Standard Heavy Compaction Test*

a) Wt of hammer = **4.9** kg

b) Height of fall= **450mm**

c) Volume of mould= 1000cc

d) **5layers-**>25 no. Of blows

$\boldsymbol{Energy\ Imparted\ in\ Indian\ Standard\ Proctor\ Test}$

$$\Rightarrow \frac{Energy}{Volume} = \frac{(mgh) \times (no.of\ blows) \times (no.of\ layers)}{1000cc}$$

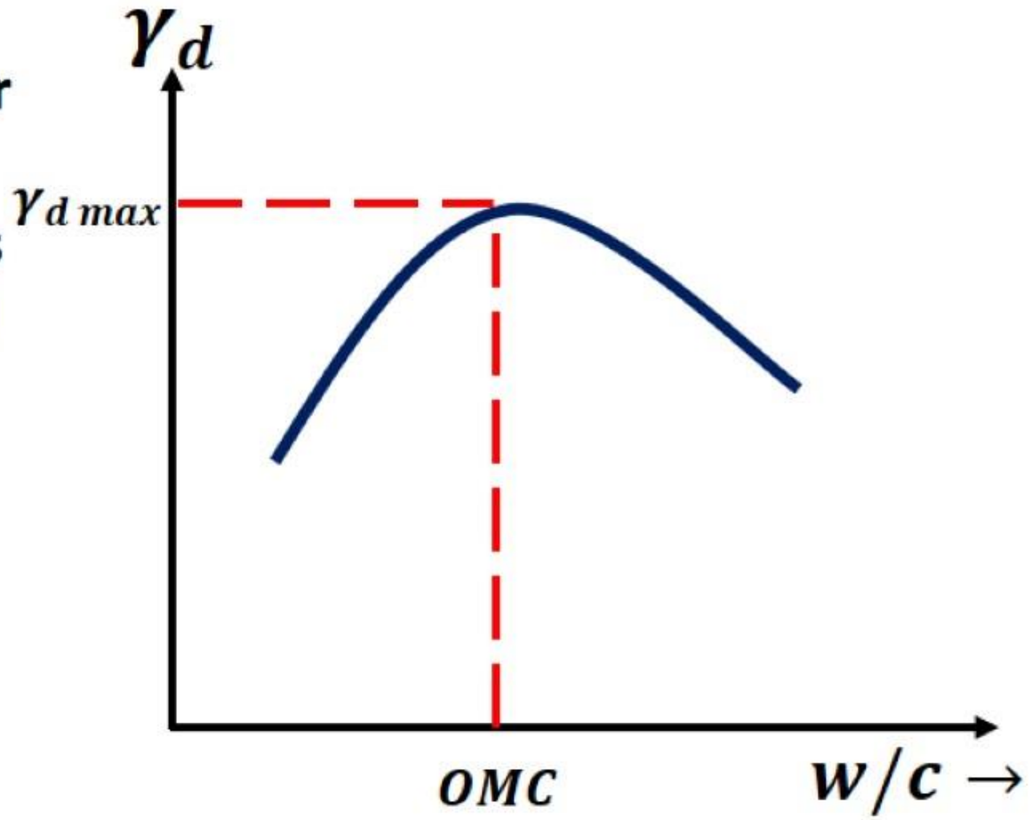
$$\Rightarrow \frac{Energy}{Volume} = \frac{(4.9 \times 9.81 \times 0.0.450) \times (25) \times (5)}{1000cc}$$

$$\boxed{\Rightarrow E_2' = 2703.8\ \frac{kJ}{m^3}}$$

$$\boxed{\Rightarrow Therefore\ E_2{}' = 4.55E_1{}'}$$

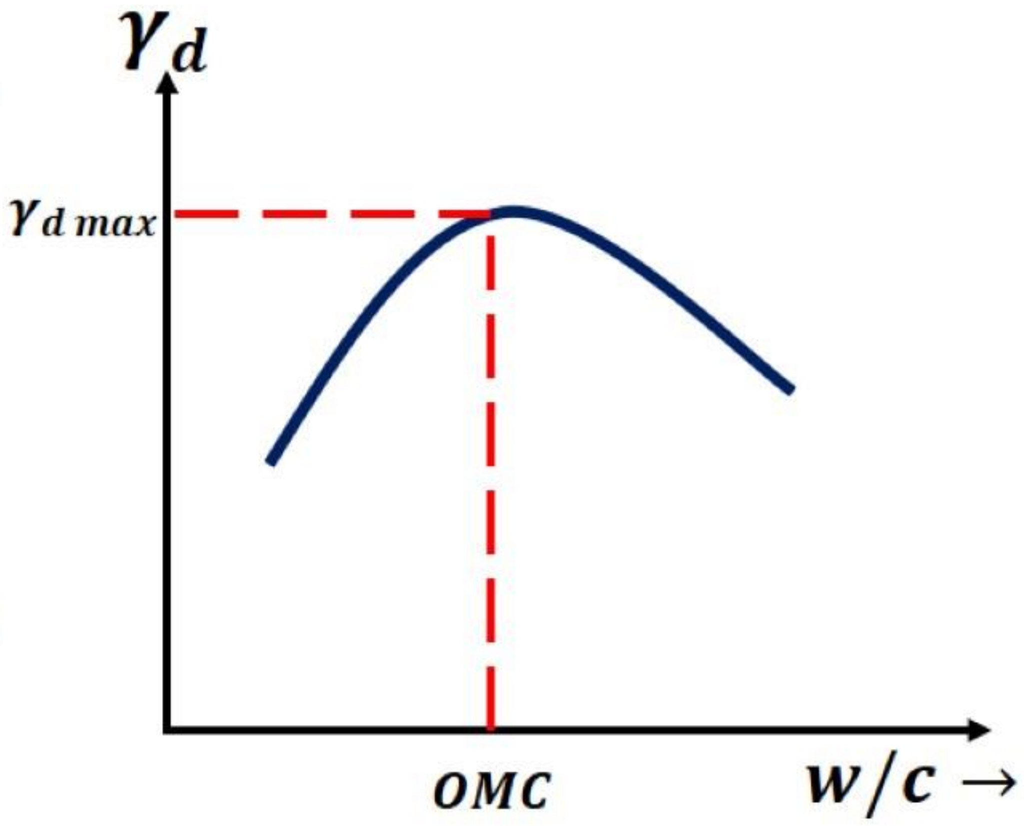
EXPLANATION FOR NATURE OF GRAPH

- **LUBRICATION THEORY:**
  - At low water content, soil is stiff and offer resistance to compaction; as moisture content increases, lubrication effect takes place and soil can easily move (slide) and go into closer packing.
  - This process goes upto OMC (Optimum Moisture Content). Beyond OMC, water starts occupying the space which could have been occupied by the soil particles. Thus dry density decreases.

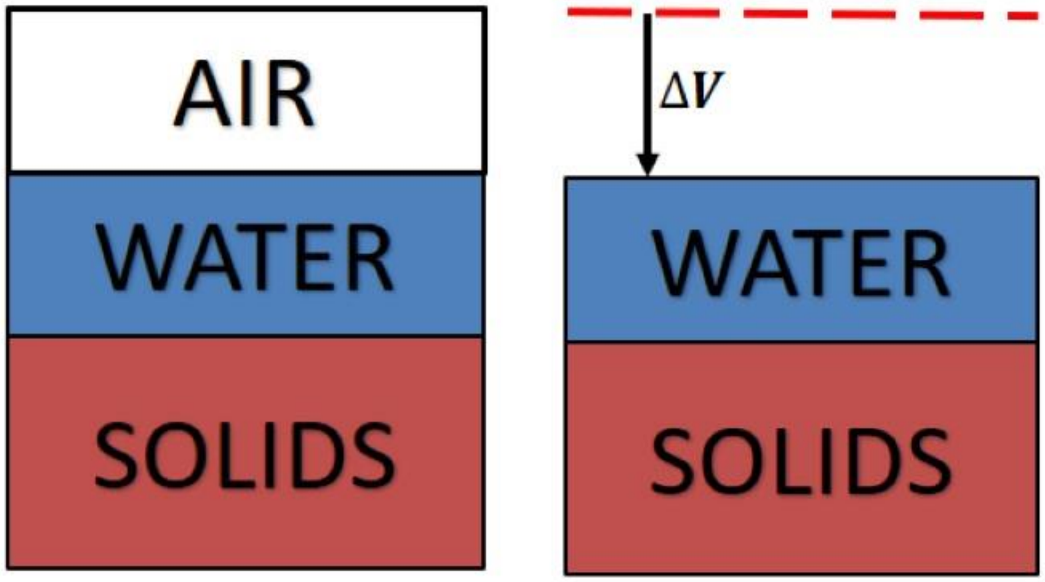


EXPLANATION FOR NATURE OF GRAPH

- **LAMBES THEORY:**
  - At low water content, double diffused layer is not completely developed and hence there is net attractive force between the particles that causes difficulty in the movement.
  - As water content increases, the double diffused layer develops and hence, repulsion increases.
  - Thus particles can move into closer packing leading to increase in dry density.
  - This process continues upto OMC and double diffused layer develop completely at OMC.
  - Beyond OMC, water starts occupying the space which could have been occupied by soil particles, thus dry density decreases.

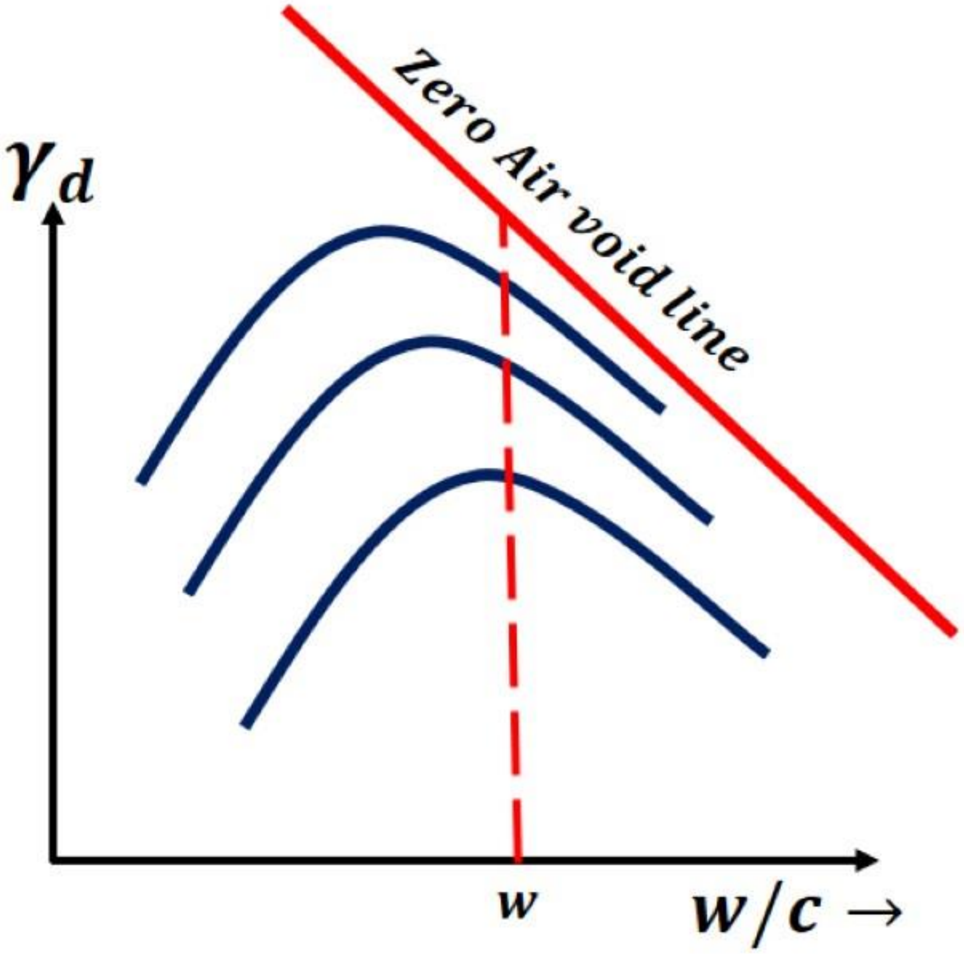


THEORETICAL MAXIMUM DRY DENSITY (ZERO AIR VOID)



Before Compaction
Soil was partially Saturated

After Compaction
Soil got Saturated due to removal of all air



THEORETICAL MAXIMUM DRY DENSITY (ZERO AIR VOID)

- For a given water content, Theoretical Maximum Dry Density is said to be achieved in a situation when NO AIR VOIDS are left.
- This is not same to the situation when water content is increased to make soil saturated
- It is Maximum Dry Unit Weight that can be obtained from given soil at a given water content BY APPLYING COMPACTION.
- It is called as THEORETICAL, because practically it is not possible to make air voids equal to 0.

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

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1+\left(\frac{wG}{s}\right)}$$

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

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1+wG}$$

a) **100% Saturation (s) and 0% air content (a_c)**

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + wG/s}$$

$If\ s = 1,$

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + wG}$$

$If\ a_c = 0,$

$$\Rightarrow s = 1 - a_c$$

$$\Rightarrow s = 1$$

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + wG}$$

✓ *100% saturation line and 0% air content are same.*

b) **100% Saturation (s) and 0% air Void (n_a)**

$If\ s = 1,$

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + wG/s}$$

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + wG}$$

$If\ n_a = 0,$

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

$$\Rightarrow \gamma_d = \frac{(1 - 0)G\gamma_w}{1 + wG}$$

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + wG}$$

✓ *So, 100% saturation line and 0% air VOID are same.*

c) 90% Saturation (s) and 10% air content (a_c)

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + wG/s}$$

$If\ s = 0.9,$

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + \frac{wG}{0.9}}$$

$If\ a_c = 0.1,$

$$\Rightarrow s = 1 - 0.1$$

$$\Rightarrow s = 0.9$$

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + \frac{wG}{0.9}}$$

90% saturation line and 10% air content are same.

d) 90% Saturation (s) and 10% air Void (n_a)

$If\ s = 0.9,$

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + wG/s}$$

$$\Rightarrow \gamma_d = \frac{G\gamma_w}{1 + \frac{wG}{0.9}}$$

$If\ n_a = 0.1,$

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

$$\Rightarrow \gamma_d = \frac{(1 - 0.1)G\gamma_w}{1 + wG}$$

$$\Rightarrow \gamma_d = \frac{0.9\ G\gamma_w}{1 + wG}$$

So, 90% saturation line and 10% air VOID are NOT same.

THEORETICAL MAXIMUM DRY DENSITY (ZERO AIR VOID)

- **CONCLUSION**
  - 100% saturation line and 0% air content are same.
  - 100% saturation line and 0% air void lines are also same.
  - 90% saturation line and 10% air content line are same.
  - 90% saturation line and 10% air void lines are NOT same.

Que. In a Standard Proctor Test, 1.8kg of moist soil was filled in mould of 944cc. After compaction, a soil sample weighing 23gms was taken from mould and on oven drying the weight of dry sample is 20gms. Specific gravity is 2.7.

Find the theoretical maximum dry density of the sample at above water content.

$$water\ content \Rightarrow w = \frac{3}{20}$$

$$\Rightarrow w = 0.15$$

$$\Rightarrow (\gamma_{d\,max})_{Theoretical} = \frac{G\gamma_w}{1 + wG/s}$$

$$\Rightarrow (\gamma_{d\,max})_{Theoretical} = \frac{2.7 \times 9.81}{1 + 0.15 \times 2.7/1}$$

$$\Rightarrow (\gamma_{d\,max})_{Theoretical} = 18.95\ kN/m^3$$

Que. OMC of a soil is 16.5% and its maximum dry density is 1.57gm/cc. Specific gravity is 2.65.

Determine –

a) Degree of saturation and % air content at OMC

b) Theoretical dry density at OMC corresponding to zero air content.

c) Calculate % air voids at OMC.

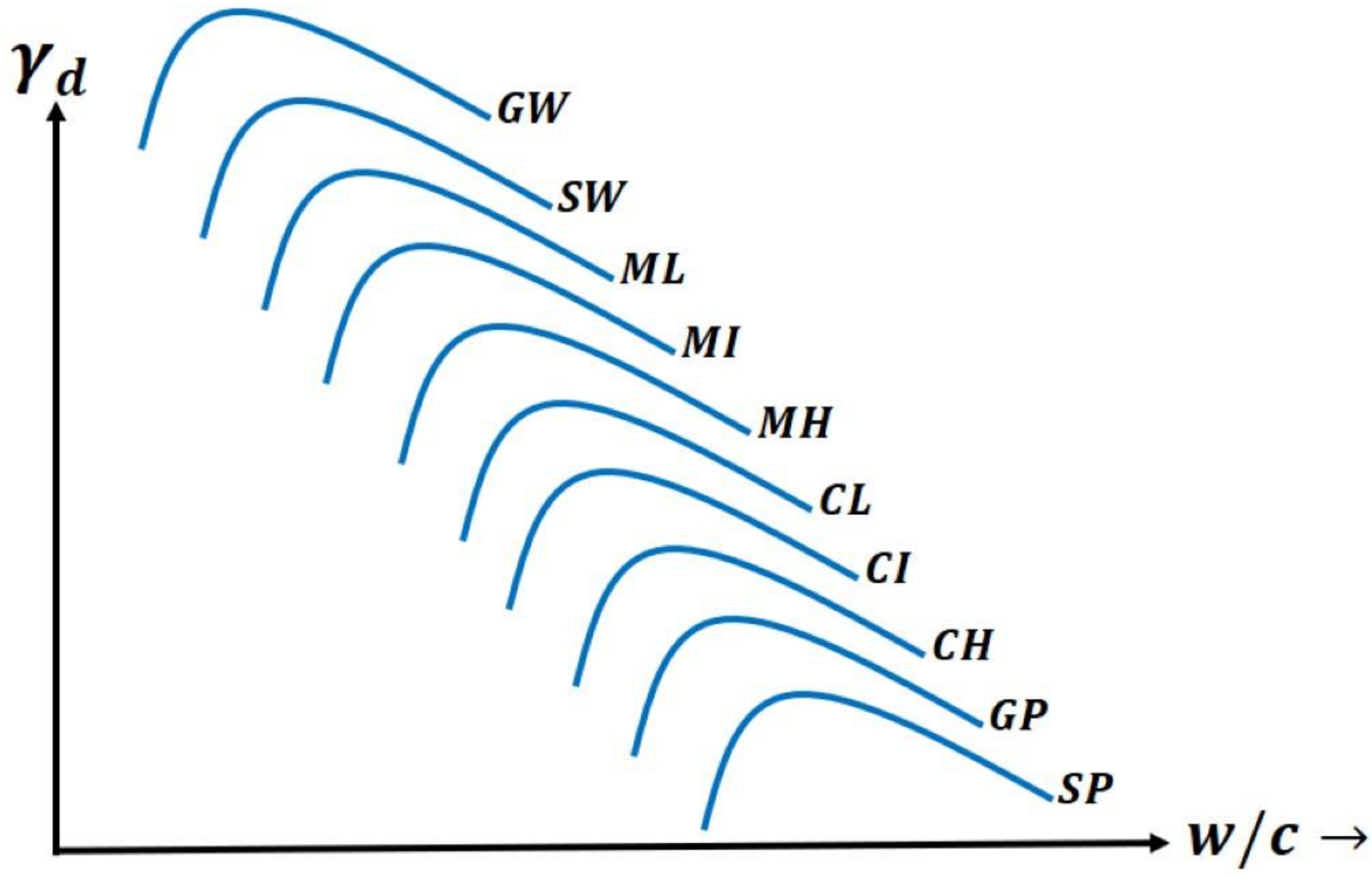
Que. The maximum dry density of a sample is 1.78gm/ml at an optimum w/c of 15%. Specific gravity is 2.67.

Determine –

a) Degree of saturation

b) Calculate % air voids

TYPES OF SOIL



TYPES OF SOIL

- Coarse grain well graded soil can be compacted to high dry density because all size particles are available.
- Poorly graded or uniformly graded soil can be compacted to lowest dry unit weight.
- As the plasticity of soil increases, dry density decreases.

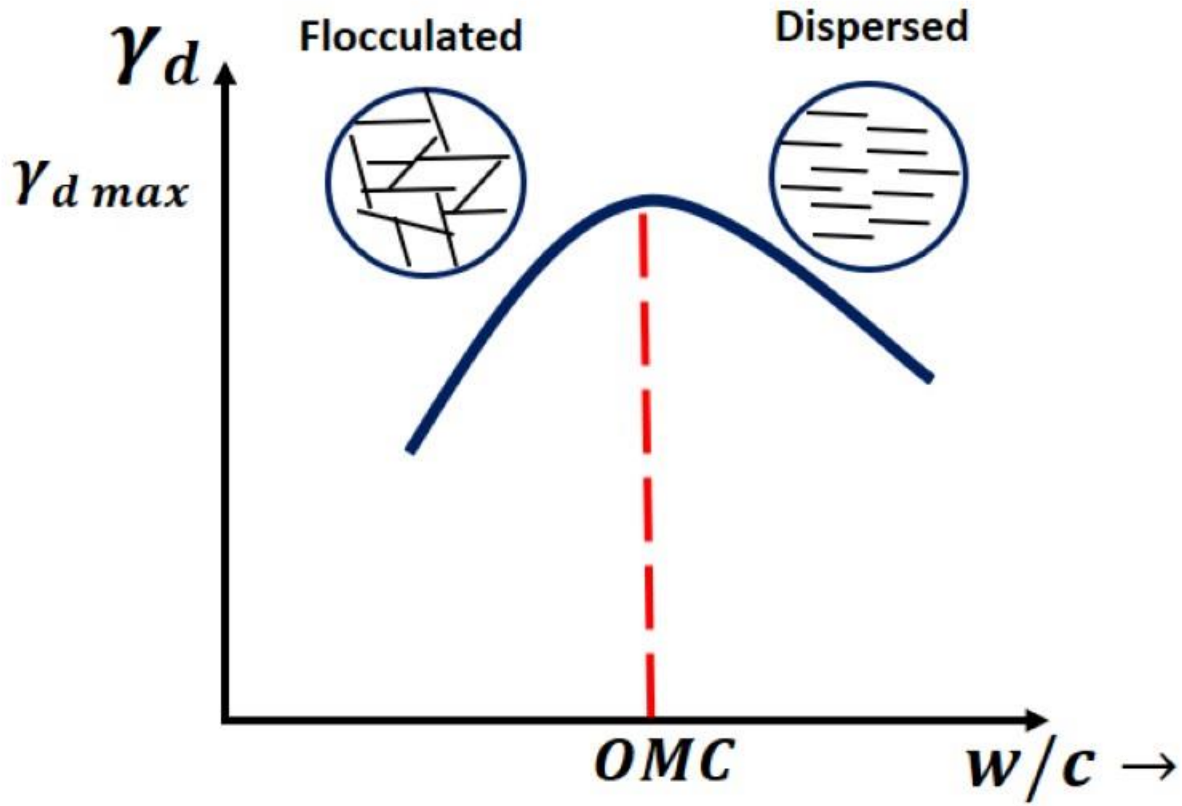
For example,

If soil A is SW-SM and soil B is SM

Then soil A can be compacted to a greater extent.

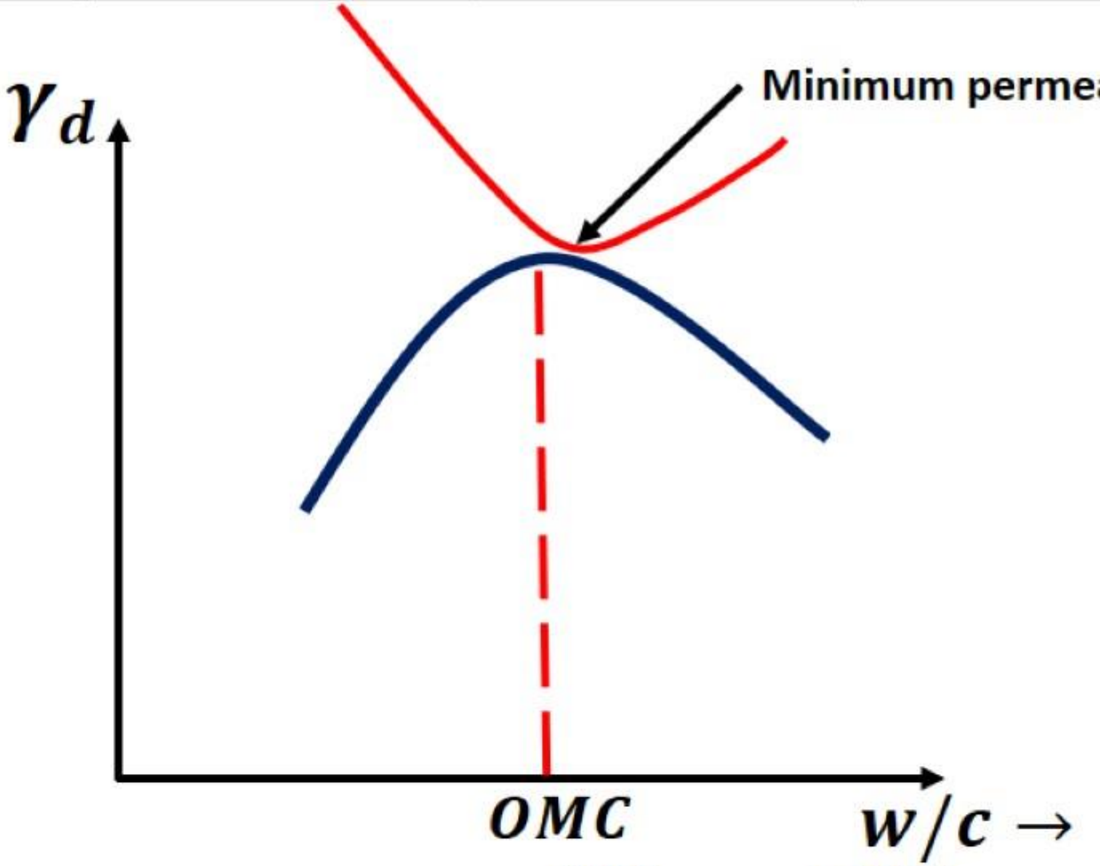
EFFECT OF COMPACTION ON SOIL PROPERTIES

Sr. No	PROPERTIES	DRY OF OPTIMUM (Left of Optimum)	WET OF OPTIMUM (Right of Optimum)	EXPLANATION
1	Structure	Flocculated	Dispersed	



EFFECT OF COMPACTION ON SOIL PROPERTIES

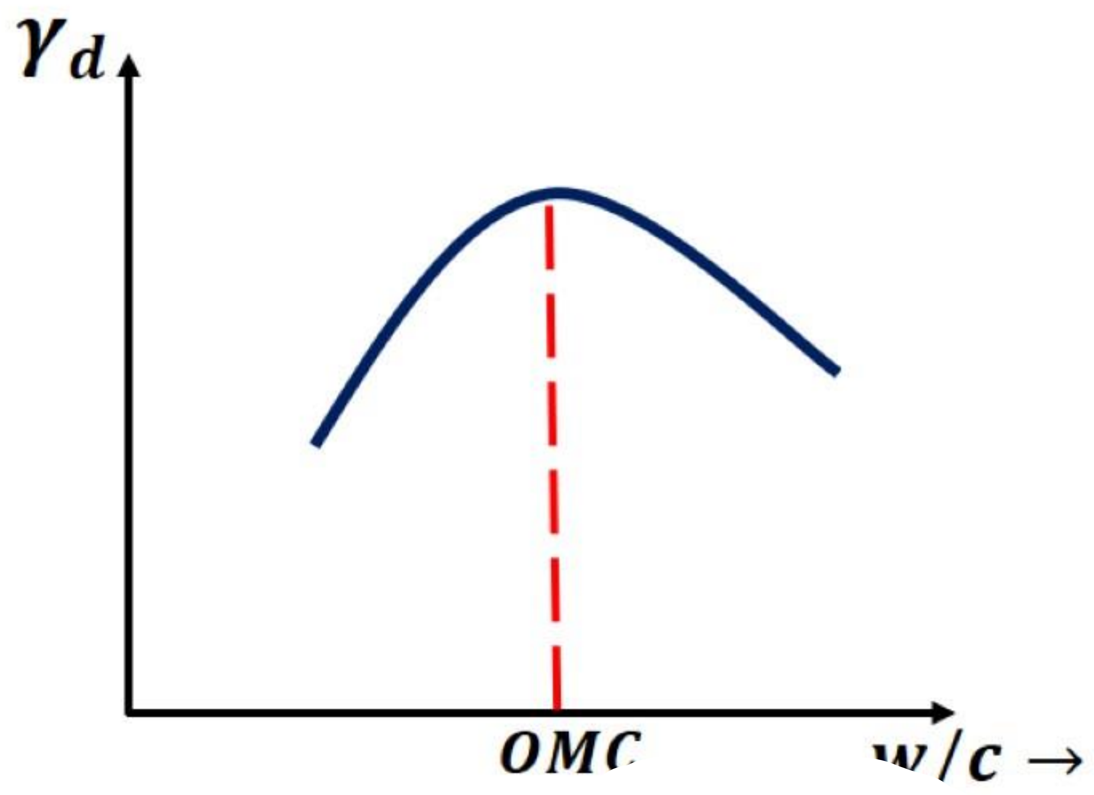
Sr. No	PROPERTIES	DRY OF OPTIMUM (Left of Optimum)	WET OF OPTIMUM (Right of Optimum)	EXPLANATION
2	Permeability	More (isotropic)	Less (anisotropic) $K_h > K_v$	Minimum permeability is at just wet side of optimum



Left of OMC, it sharply decreases and to the right it slightly increases

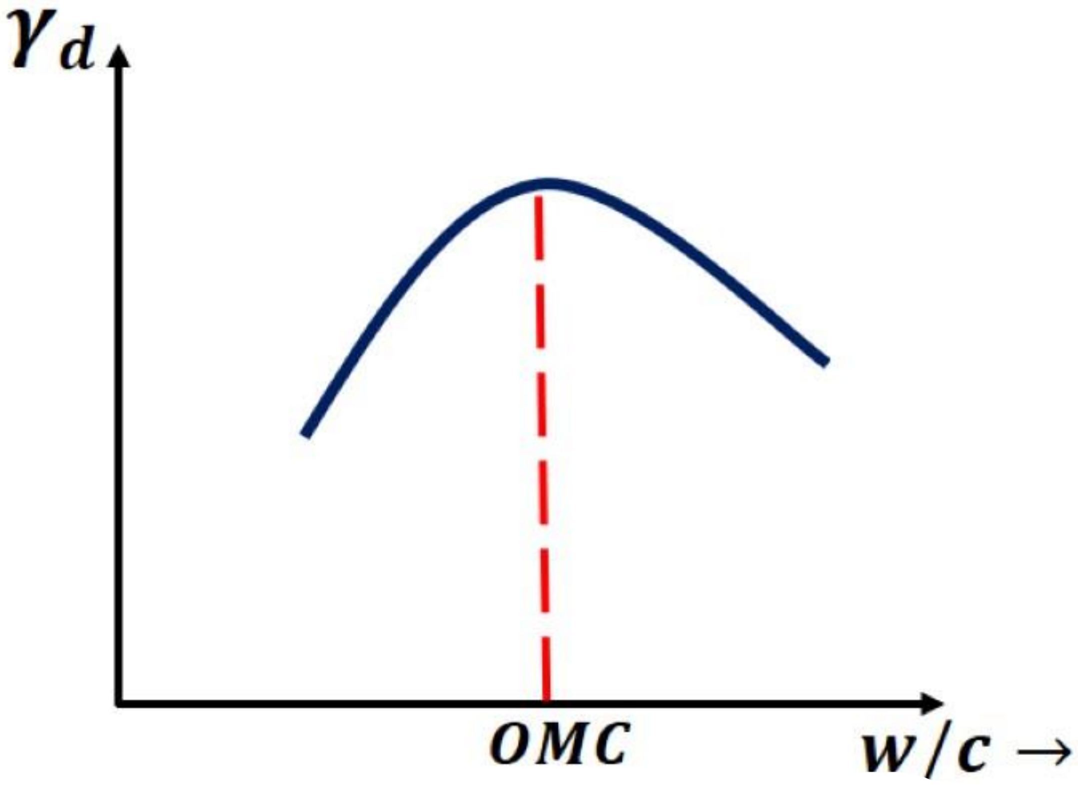
EFFECT OF COMPACTION ON SOIL PROPERTIES

Sr. No	PROPERTIES	DRY OF OPTIMUM (Left of Optimum)	WET OF OPTIMUM (Right of Optimum)	EXPLANATION
3	Compressibility 1. Low stress 2. High stress	Dry of optimum 1. Less 2. High	Wet of optimum 1. High 2. Less	At low stress, bonds in flocculated structure cannot be broken



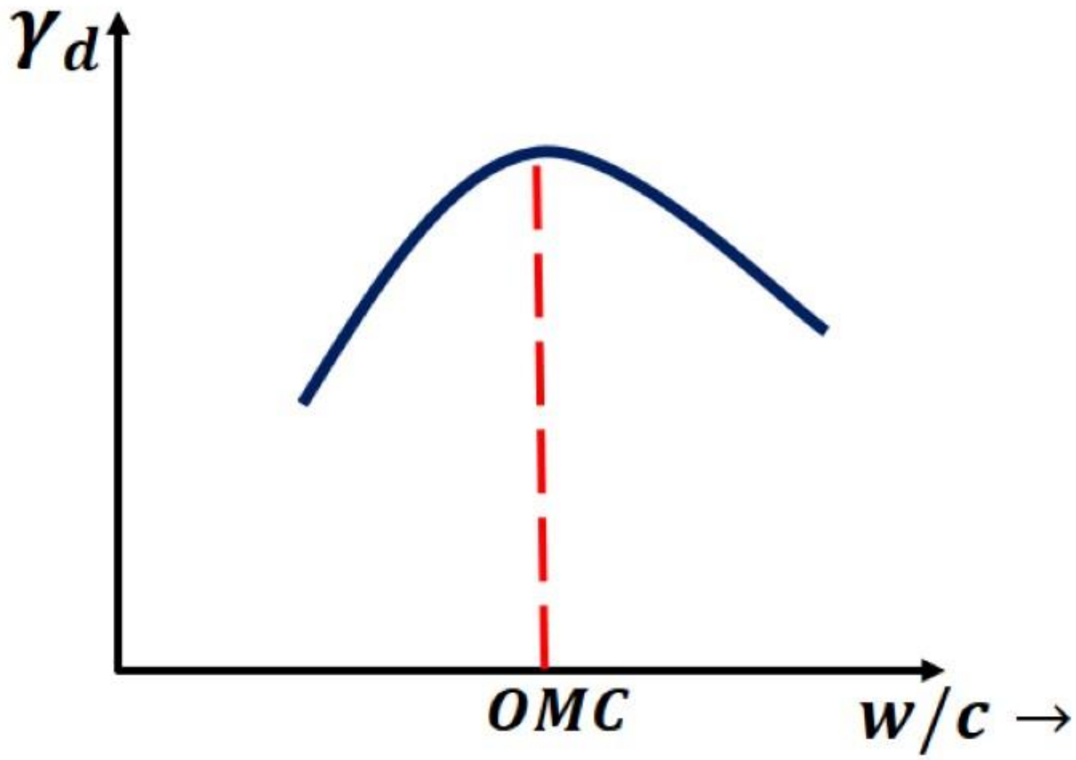
EFFECT OF COMPACTION ON SOIL PROPERTIES

Sr. No	PROPERTIES	DRY OF OPTIMUM (Left of Optimum)	WET OF OPTIMUM (Right of Optimum)	EXPLANATION
4	Swelling	High	Low	Due to large voids in flocculated structure, soil can absorb more amount of water



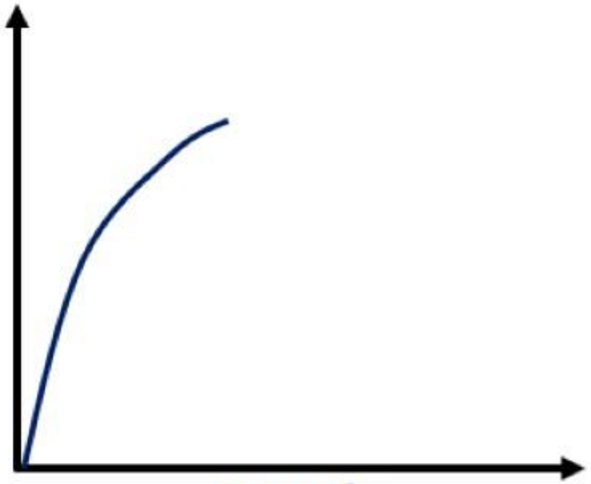
EFFECT OF COMPACTION ON SOIL PROPERTIES

Sr. No	PROPERTIES	DRY OF OPTIMUM (Left of Optimum)	WET OF OPTIMUM (Right of Optimum)	EXPLANATION
5	Shrinkage	Low	High	In right of optimum, high quantity of water is present.

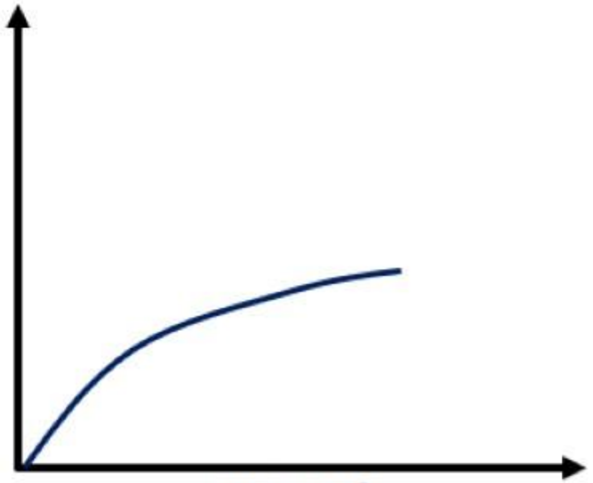


EFFECT OF COMPACTION ON SOIL PROPERTIES

Sr. No	PROPERTIES	DRY OF OPTIMUM (Left of Optimum)	WET OF OPTIMUM (Right of Optimum)	EXPLANATION
6	Stress strain curve			Flocculated structure can take more amount of stress



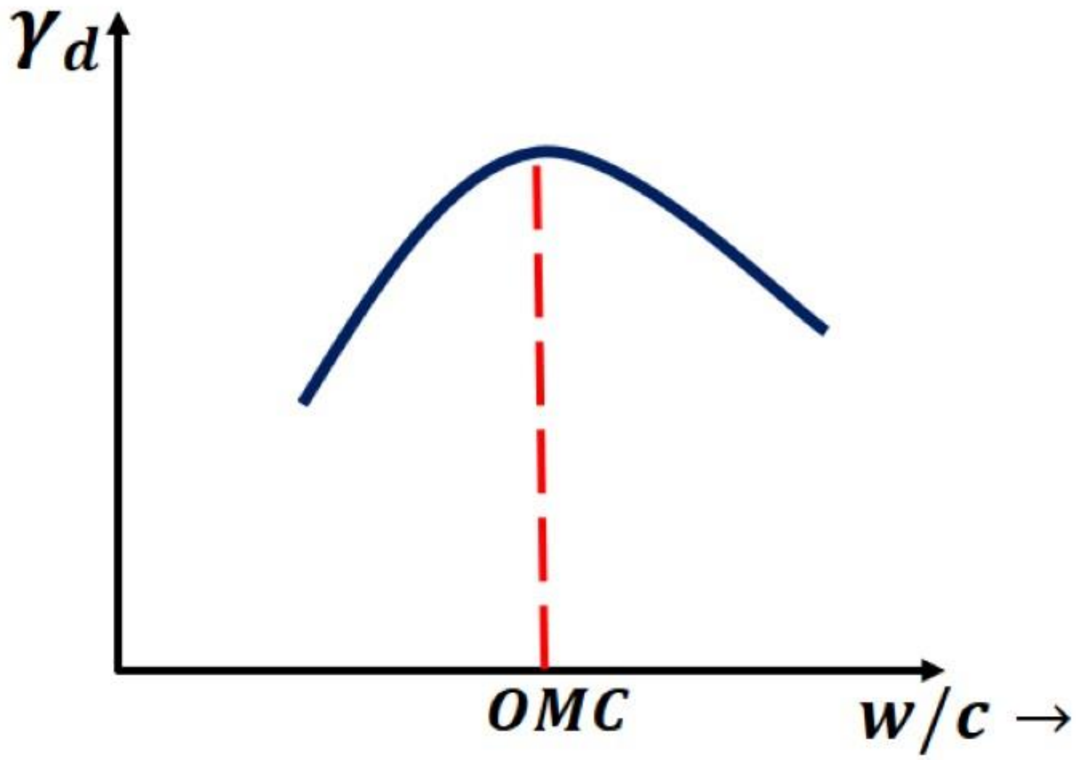
Brittle



Ductile

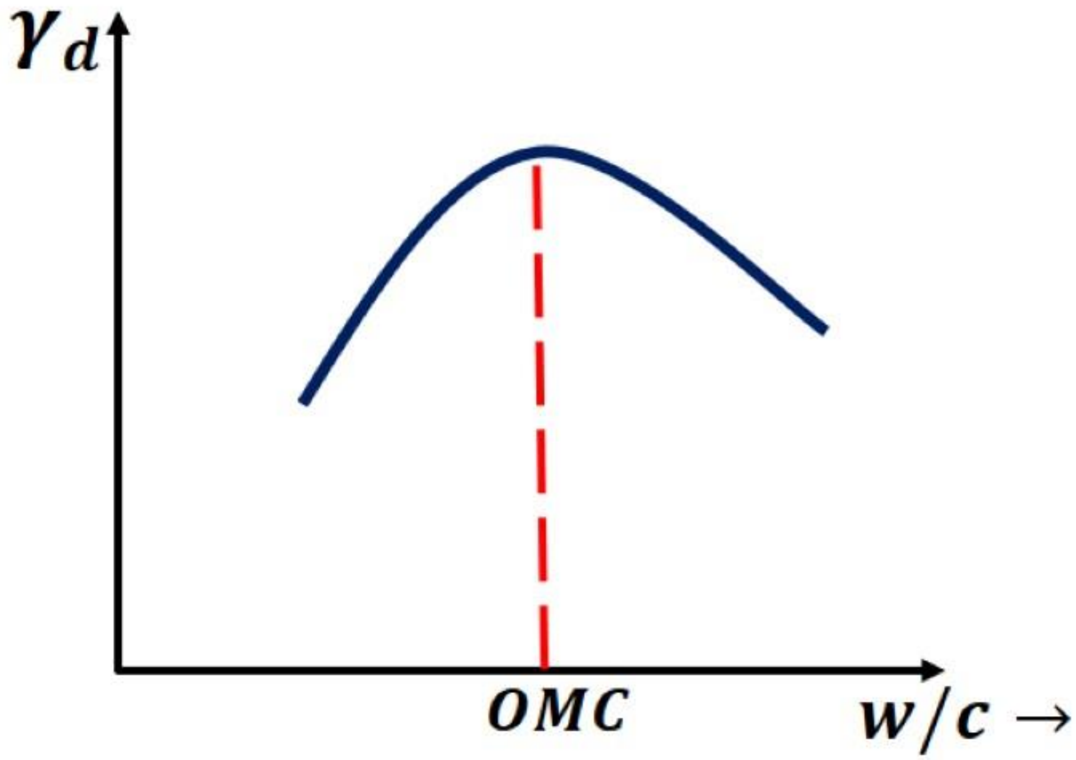
EFFECT OF COMPACTION ON SOIL PROPERTIES

Sr. No	PROPERTIES	DRY OF OPTIMUM (Left of Optimum)	WET OF OPTIMUM (Right of Optimum)	EXPLANATION
7	Sensitivity	More	Less	Flocculated is more sensitive as it looses more amount of strength in remoulding



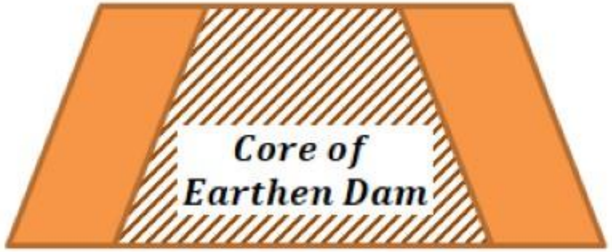
EFFECT OF COMPACTION ON SOIL PROPERTIES

Sr. No	PROPERTIES	DRY OF OPTIMUM (Left of Optimum)	WET OF OPTIMUM (Right of Optimum)	EXPLANATION
8	Construction of pore water pressure	Low	High	



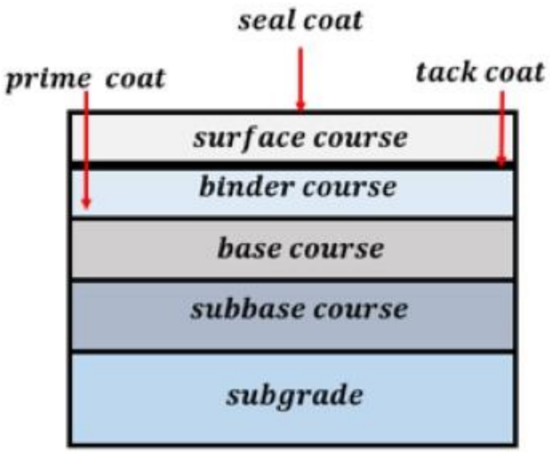
IMPORTANT RESULTS

1. ***Construction of Core of Earthen Dam***
   - **In the core of earthen dam, compaction is done at the just wet side of optimum to reduce permeability.**



2. ***Homogeneous Earth dam***
   - ***It* is compacted at dry side of optimum to achieve higher strength.**
   - **By compacting on dry side of optimum pore, water pressure built up is less, hence strength is more.**

3. ***For Subgrade of pavements***
   - **We adopt compaction on wet side of optimum so that volume changes are limited.**



Compaction Equipment

1. **Sheep foot roller**
   - Suitable for fine grain soils
   - Not suitable for use on cohesionless granular materials
2. **Tamping rollers**
   - Tamping compactors are high speed non vibratory rollers
   - It is effective on all soils except sand and soft clays
3. **Vibrating rollers**
   - Amplitude of vibration controls the effective depth to which vibration has been transmitted

Compaction Equipment

4. Smooth Drum Vibratory Compactor

- They create three types of compactive forces
  a) Pressure
  b) Impact
  c) Vibration
- They are effective on granular Materials with particle sizes ranging from large rocks to fine sand

5. Pneumatic tyred Rollers

- Suitable for Coarse and fine soils
- Not suitable for very soft clay
- They apply kneading effect to compact the soil
- Used to proof roll roadway subgrades and air field bases and on earth fill dams

Compaction Equipment

6. **Manually operated Vibrating Plate Compactors**
   - Used where large units are not feasible
   - Suitable for Coarse Soils

7. **Manually Operated Rammer Compactors**
   - These are self propelled rammers
   - Used where Lift thickness is small (3-4 inch)

Compaction Equipment

8. Tampers and Rammers
   - All soil types

SOIL MECHANICS

Civil Engineering by Sandeep Jyani

Compressibility

PANKAJ GOLCHHA GCEPR

COMPRESSIBILTY AND CONSOLIDATION

- **Consolidation is a gradual process in which reduction in the volume of fully saturated soil of low permeability occurs due to dissipation of excess pore water pressure set up due to increase in stress, or otherwise (due to sudden lowering of water table).**

DIFFERENCE BETWEEN COMPACTION AND CONSOLIDATION

COMPACTION	CONSOLIDATION
Compaction process is instantaneous process.	Consolidation is a time dependent process.
Soil is partially saturated.	In consolidation, we assume fully saturated soil.
Thickness of layer decreases due to expulsion of air from the voids.	Thickness of layer decreases due to dissipation of pore water pressure.
Loading is impact loading.	Loading is gradual loading.
Void ratio decreases.	Void ratio decreases.

DIFFERENCE BETWEEN COMPACTION AND CONSOLIDATION

COMPACTION	CONSOLIDATION

SETTLEMENT

$$S_{\mathrm{Total}} = S_{\mathrm{i}} + S_{\mathrm{p}} + S_{\mathrm{s}}$$

- **Total settlement of soil is expressed in 3 components:**
  1. **Immediate Settlement**
  2. **Primary Settlement**
  3. **Secondary Settlement**

SETTLEMENT

1. Immediate Settlement

a) If the soil is having air, then due to expulsion of air as well as compression of air may take place, which leads to immediate settlement.

b) The other cause of immediate settlement would be lateral strain generation when soil is not perfectly confined.

$$S_{Total} = S_i + S_p + S_s$$

c) Here S_i denotes immediate settlement.

SETTLEMENT

2. Primary Settlement

a) It occurs due to dissipation of excess pore water pressure generated due to increase in stress or otherwise.

b) It is a time dependant process which depends upon-

- Coefficient of permeability
- Thickness of layer
- Magnitude of stress increase

$$S_{\mathbf{Total}} = S_i + S_p + S_s$$

a) Here S_p denotes primary settlement.

SETTLEMENT

3. Secondary Settlement

a) Compaction of soil doesn't stop when excess pore water pressure dissipated to zero but continue at a gradual decreasing rate under constant effective stress

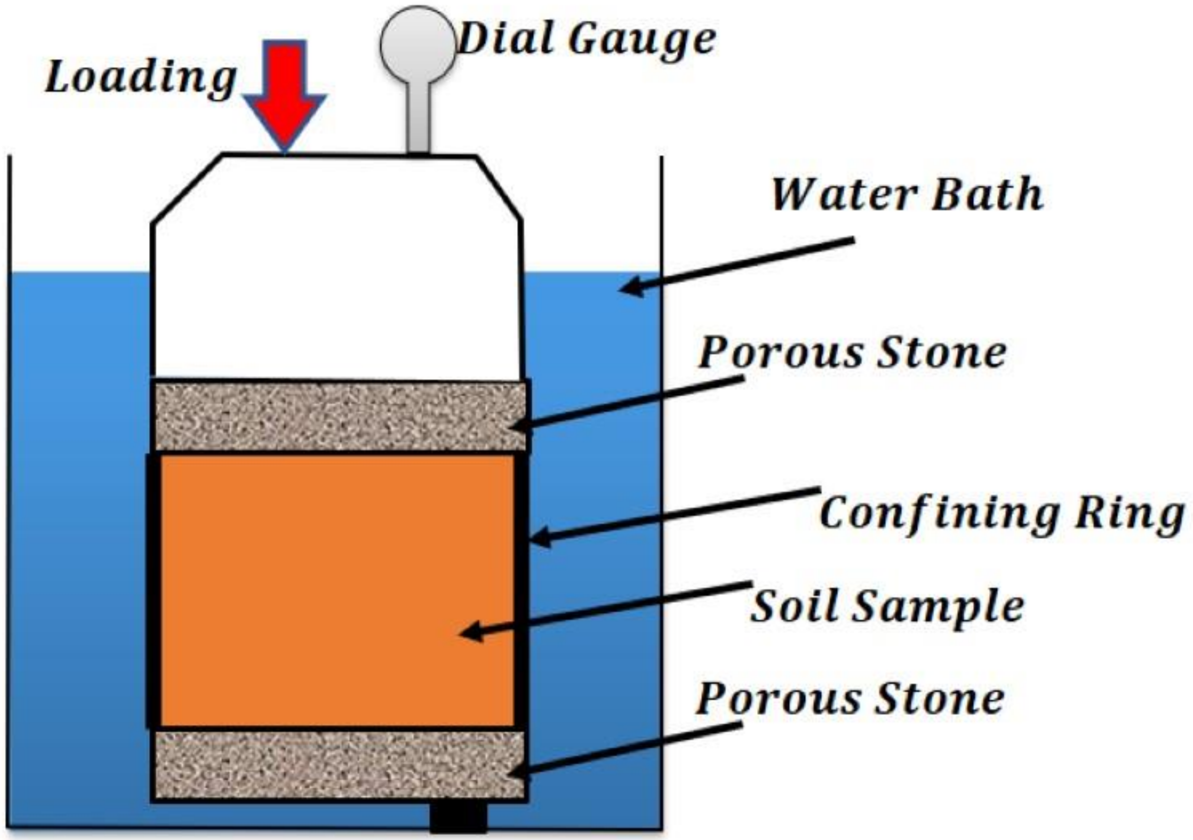
b) Secondary consolidation is due to gradual readjustment of clay particles into more stable configuration.

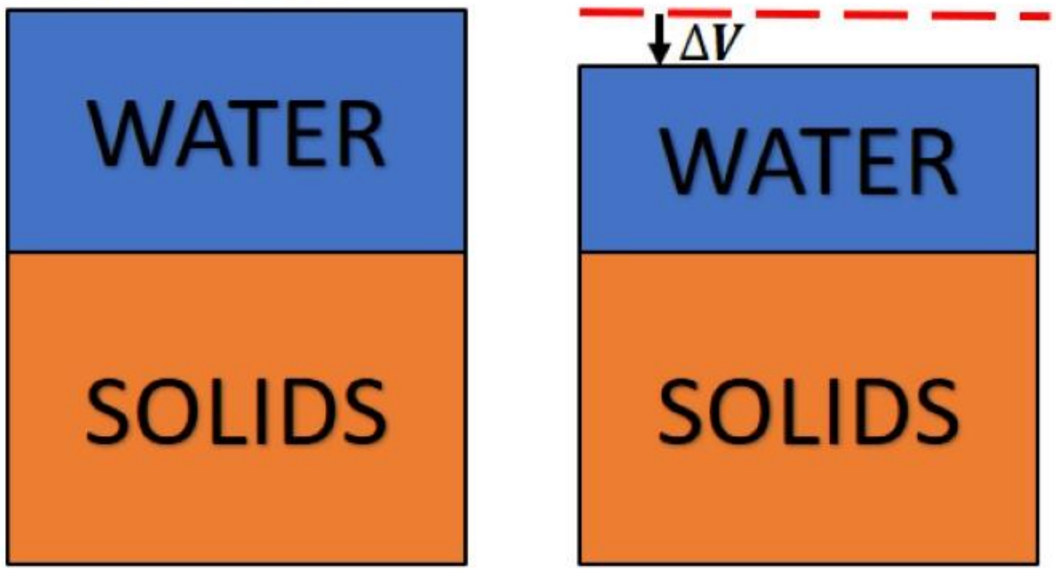
c) Secondary consolidation starts after primary consolidation.

$$S_{Total} = S_i + S_p + S_s$$

a) Here S_s denotes secondary settlement.

ONE DIMENSIONAL CONSOLIDATION





$$\Rightarrow \frac{\Delta V}{V_0} = \frac{\Delta e}{1 + e_0}$$

$$\Rightarrow \frac{A\Delta H}{AH_0} = \frac{\Delta e}{1 + e_0}$$

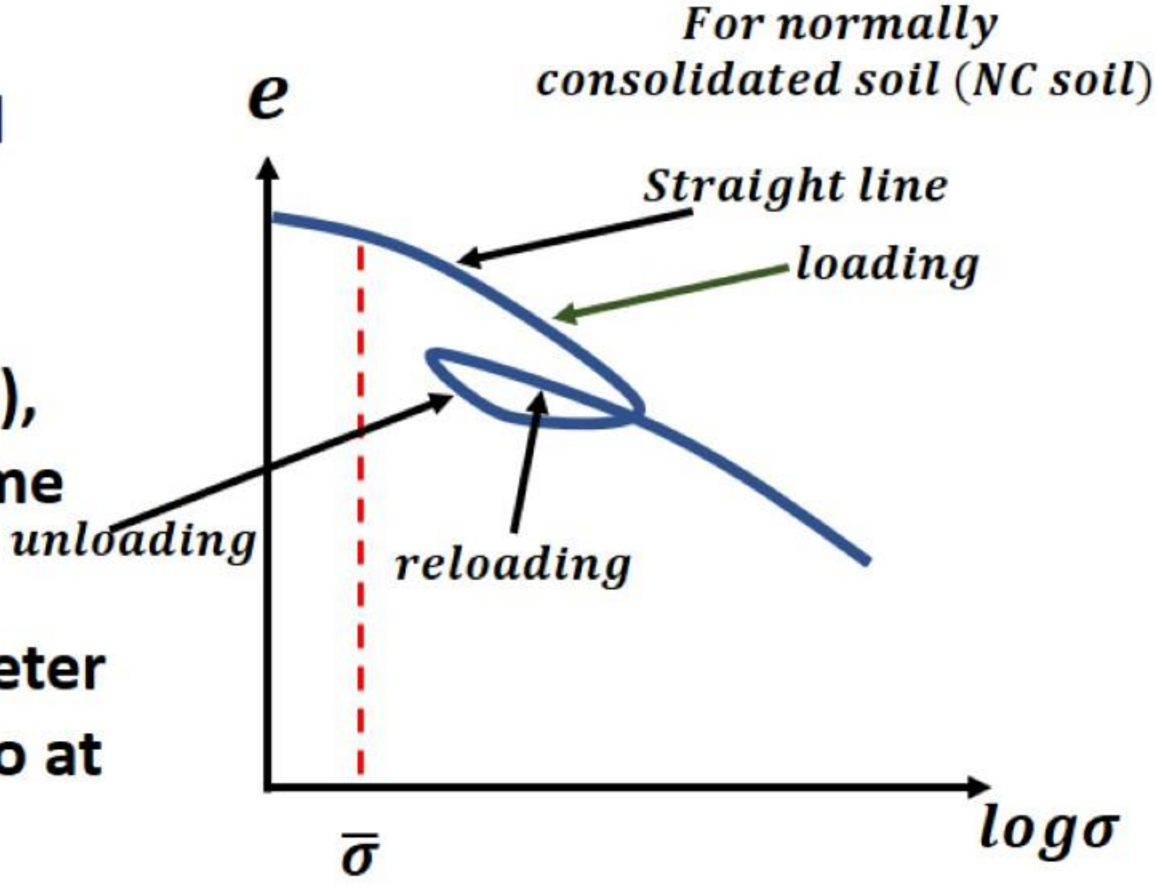
$$\Rightarrow \frac{\Delta H}{H_0} = \frac{\Delta e}{1 + e_0}$$

ONE DIMENSIONAL CONSOLIDATION

- **Soil is loaded in increment of vertical stress and under each increment, soil is allowed to consolidate till complete dissipation of excess pore water pressure, (usually kept for 24 hours)**
- **The compression reading is noted at suitable internal during this test (this gives time rate of consolidation)**
- **After the end of increment of period, when excess pore water pressure is completely dissipated, applied pressure becomes equal to effective stress in the specimen.**

ONE DIMENSIONAL CONSOLIDATION

- The 24 hours reading give the final compression under each stress increment.
- After the first increment (24 hours), next investment is applied and same procedure is repeated.
- The results obtained from Odeometer test is plotted in terms of void ratio at the end of each stress increment against corresponding effective stress.



$\bar{\sigma}$ = Preconsolidated stress

PRE CONSOLIDATION STRESS

$$\overline{\sigma_c} = preconsolidation\ stress$$

- **NC soil (Normally Consolidated Soil)**
  a) **A soil is said to be normally consolidated when existing effective stress is the maximum that it has ever experienced in the history.**
  b) **The soil is said to be over consolidated when existing effective stress is less than the stress that it has experienced.**
  c) **The maximum value of stress that the soil has ever experienced is called Pre-consolidated stress.**

PRE CONSOLIDATION STRESS

$$\overline{\sigma_c} = preconsolidation\ stress$$

- **Over Consolidation is due to -**
  a) **Erosion of over burden.**
  b) **Permanent rise of ground water table.**
  c) **Melting of ice sheet.**

NOTE :

a) **e vs log $(\bar{\sigma})$graph for NC soil is always a straight line.**
b) **e vs log $(\bar{\sigma})$graph for coarse soil is convex upward.**

OVER CONSOLIDATION RATIO

$\overline{\sigma_c} = preconsolidation\ stress$

$$OCR = \frac{\overline{\sigma_c}}{\overline{\sigma}}$$

Generally for Over Consolidated soil OCR>1

SOME PARAMETERS

1. Compression Index C_c

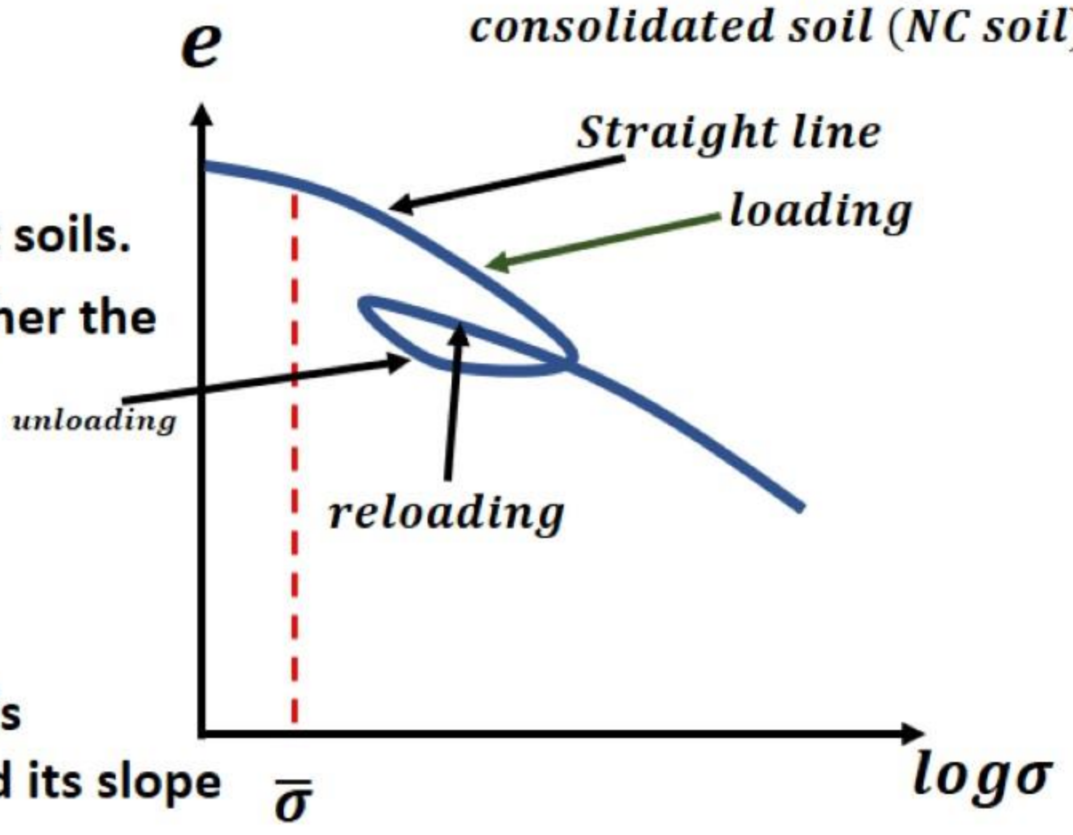
a) It doesn't depend on stress range.

b) Compression Index is different for different soils.

c) Higher the value of compression index, higher the vertical deformation of soil layer.

$$C_c = \frac{e_1 - e_2}{\log(\overline{\sigma_1}) - \log(\overline{\sigma_2})}$$

d) For unloading curve or expansion curve, it is approximately a straight line for NC soil and its slope is said to be expansion index.



SOME PARAMETERS

a) Calculation of Compression Index –

b) In case of NC soil

- *For undisturbed Clay*

$$C_c = 0.009[w_L - 10]$$

- *For disturbed/remoulded Clay*

$$C_c = 0.007[w_L - 7]$$

$$W_L = \text{liquid limit } (\%)$$

SOME PARAMETERS

2. Coefficient of Compressibility a_v

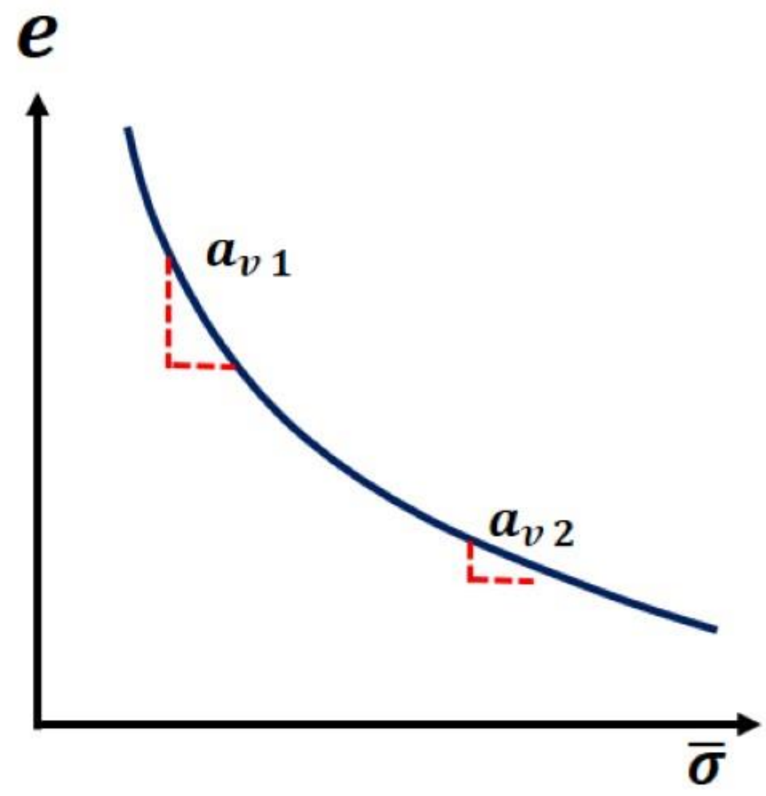
a) It depends on stress range.

b) As the stress value increases, the value of coefficient of compressibility decreases.

c) This is due to resistance to further compression.

d) It is denoted by a_v

$$a_v = \frac{\Delta e}{\Delta \bar{\sigma}}$$



SOME PARAMETERS

3. Coefficient of Volume Compressibility m_v

a) This value is not constant for a particular soil.

b) It depends upon the stress range

c) It is denoted by m_v

$$m_v = \frac{a_v}{(1+e_o)}$$

$$a_v = \frac{\Delta e}{\Delta \bar{\sigma}}$$

$$m_v = \frac{\Delta e}{(1+e_o)\Delta \bar{\sigma}}$$

4. Compression Modulus E_C

$$E_c = \frac{1}{m_v}$$

TIME RATE OF CONSOLIDATION

- **CONSOLIDATION ANALYSIS**
  a) In consolidation analysis, the rate of settlement is directly related to the rate of dissipation of excess pore water pressure.
  b) Therefore in order to predict the time rate of consolidation, a theory was proposed by Tezarghi for one dimensional condition.

TIME RATE OF CONSOLIDATION

- **ASSUMPTIONS OF TEZARGHI ONE DIMENSIONAL THEORY:**
    - **Soil is homogenous and fully saturated**
    - **Soil particles and water are incompressible.**
    - **Coefficient of permeability, coefficient of compressibility, coefficient of volume compressibility are constant.**
    - **Darcy's law is valid for all hydraulic gradient.**
    - **Compression and flow are in 1-D.**
    - **Differential equation proposed by Tezrghi**

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

u = degree of consolidation

$$C_v = \frac{k_Z(1+e)}{a_v \gamma_w}$$

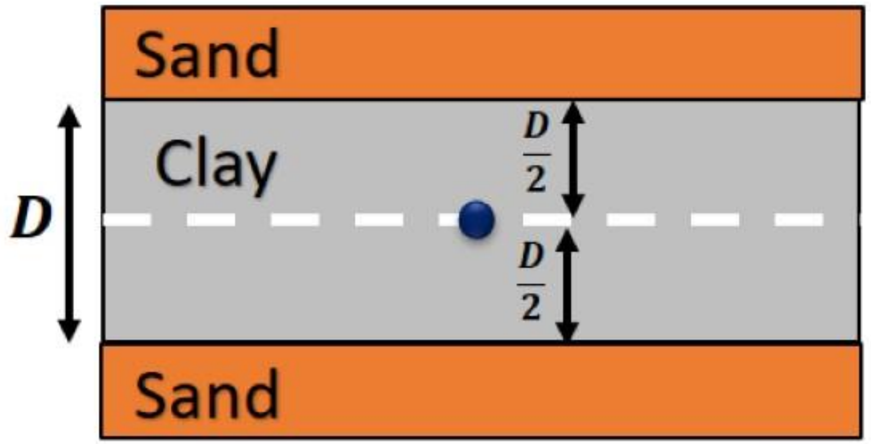
In the solution of Tezarghi one dimensional theory, following non dimensional parameters were defined:

1. Drainage path ratio
   1. Double drainage

$$Z = \frac{z}{H}$$

$z = depth\ of\ any\ point\ from\ the\ top\ of\ the\ clay\ layer$

$H = maximum\ distance\ travelled\ by\ the\ water\ molecules$



$$H = \frac{D}{2}\ for\ double\ drainage$$

$$Z = \frac{z}{H}$$

$$Z = \frac{D}{D/2} = 2$$

In the solution of Tezarghi one dimensional theory, following non dimensional parameters were defined:

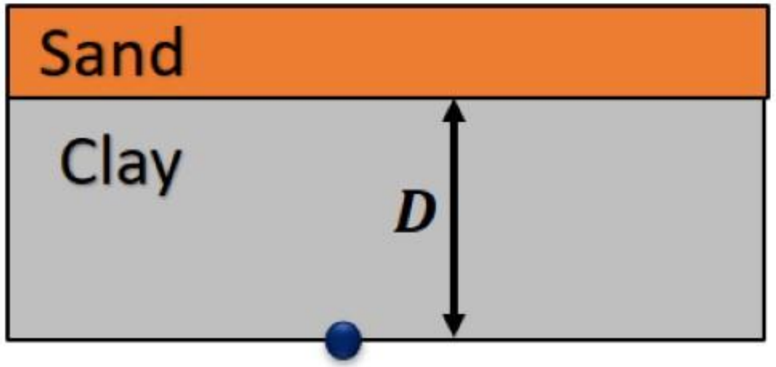
1. **Drainage path ratio**
   2. **Single drainage**

$$Z = \frac{z}{H}$$

$z = depth\ of\ any\ point\ from\ the\ top\ of\ the\ clay\ layer$

$H = maximum\ distance\ travelled\ by\ the\ water\ molecules$

$H = D\ \ for\ single\ drainage$



$$Z = \frac{z}{H}$$

$$Z = \frac{D}{D} = 1$$

In the solution of Tezarghi one dimensional theory, following non dimensional parameters were defined:

2. Time Factor T_v

$$T_v = \frac{C_v t}{H^2}$$

$C_v = Coefficient\ of\ consolidation$

$t = time$

$H = maximum\ distance\ travelled\ by\ the\ water\ molecules$

Time Factor is the function of function of degree of consolidation

- **If U ≤ 60%** $\quad T_v = \frac{\pi}{4} U^2 \ where\ U\ is\ in\ fraction$
- **If U > 60%** $\quad T_v = 1.78 - 0.933[log(100 - U)] \quad where\ U\ is\ in\ \%$

- If U ≤ 60% $T_v = \frac{\pi}{4} U^2$ *where U is in fraction*
- If U > 60% $T_v = 1.78 - 0.933[log(100 - U)]$ *where U is in %*

Time Required for 50% consolidation

$$T_{50} = \frac{\pi}{4}(0.50)^2$$

$$T_{50} = 0.196$$

Time Required for 90% consolidation

$$T_{90} = 1.78 - 0.933 log(100 - 90)$$

$$T_{90} = 0.848$$

$$\boxed{\frac{T_{50}}{T_{90}} = \frac{0.196}{0.848}}$$

Calculation of Settlement

1. Immediate Settlement (S_i):

a) Immediate Settlement in Sand

$$s_i = \frac{H_0}{c_s} log_{10}\left(\frac{\overline{\sigma}_0 + \Delta\overline{\sigma}}{\overline{\sigma}}\right)$$

$C_s = 1.5\,\frac{C_r}{\overline{\sigma}_0}$ $\quad (C_r = Static\ cone\ resistance)$

b) Immediate Settlement in Clay

$$s_i = \frac{qB(1-\mu^2)I_t}{E_s}$$

- $B = width\ of\ foundation$
- $\mu = poisson's\ ratio\ of\ soil\ (03 - 0.5)$
- $E_s = young's\ modulus\ of\ soil$

- $I_t = shape\ factor\ or\ influence\ factor\ which\ depends\ upon \frac{B}{L} ratio$
- $q = uniform\ pressure\ at\ the\ base\ of\ foundation$

$$q = \frac{Q}{A} = \frac{Q}{BL}\ \frac{kN}{m^2}$$

COMPUTATION OF SETTLEMENT (PRIMARY SETTLEMENT)

$$\frac{\Delta H}{H} = \left(\frac{\Delta e}{1 + e_o}\right)$$

$$\Delta H = \left(\frac{H}{1 + e_o}\right)\Delta e$$

$$m_v = \frac{\Delta e}{(1+e_o)\Delta\bar{\sigma}}$$

$$\Delta H = \left(\frac{H}{1 + e_o}\right) m_v \Delta\bar{\sigma}(1 + e_o)$$

$$\boxed{\Delta H = H m_v \Delta\bar{\sigma}}$$

COMPUTATION OF SETTLEMENT(PRIMARY SETTLEMENT)

$$\frac{\Delta H}{H} = \left(\frac{\Delta e}{1 + e_o}\right)$$

$$\Delta H = \left(\frac{H}{1 + e_o}\right)\Delta e$$

$$C_c = \frac{\Delta e}{\log\frac{\overline{\sigma_2}}{\overline{\sigma_1}}}$$

$$C_c = \frac{\Delta e}{\log\left(\frac{\overline{\sigma_1} + \Delta\overline{\sigma}}{\overline{\sigma_1}}\right)}$$

$$\boxed{\Delta H = \frac{C_c H}{1 + e_o}\log\left(\frac{\overline{\sigma_1} + \Delta\overline{\sigma}}{\overline{\sigma_1}}\right)}$$

COMPUTATION OF SETTLEMENT (PRIMARY SETTLEMENT)

Case 1: When change in void ratio is given

$$S_c = \Delta H = H_0 \frac{\Delta \mathrm{e}}{1 + e_0}$$

Case 2: When m_v is given

$$S_c = \Delta H = H_0 m_v (\Delta \bar{\sigma})$$

Case 3: When C_c is given

$$S_c = \boldsymbol{\Delta H = \frac{c_c H_0}{1 + e_0} \log_{10} \left(\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0} \right)}$$

Que. In a consolidation test on a soil, the void ratio of the sample decreases from 1.24 to 1.12 when the pressure is increased from 20t/sq.m to 40t/sq.m.

Find the coefficient of consolidation in m^2/year, given that the coefficient of permeability of soil during this process is $\boldsymbol{8.5 \times 10^{-3} cm/sec}$

Que. In a consolidation test, the void ratio of the specimen which was 1.068 under the effective pressure of $214 kN/m^2$, change to 0.994 when the pressure was increased to $429 kN/m^2$. Find the coefficient of compressibility, compression index and coefficient of volume compressibility. Also find the settlement of foundation resting on above type of clay. If the thickness of layer is 8m and increase in pressure is $10 kN/m^2$

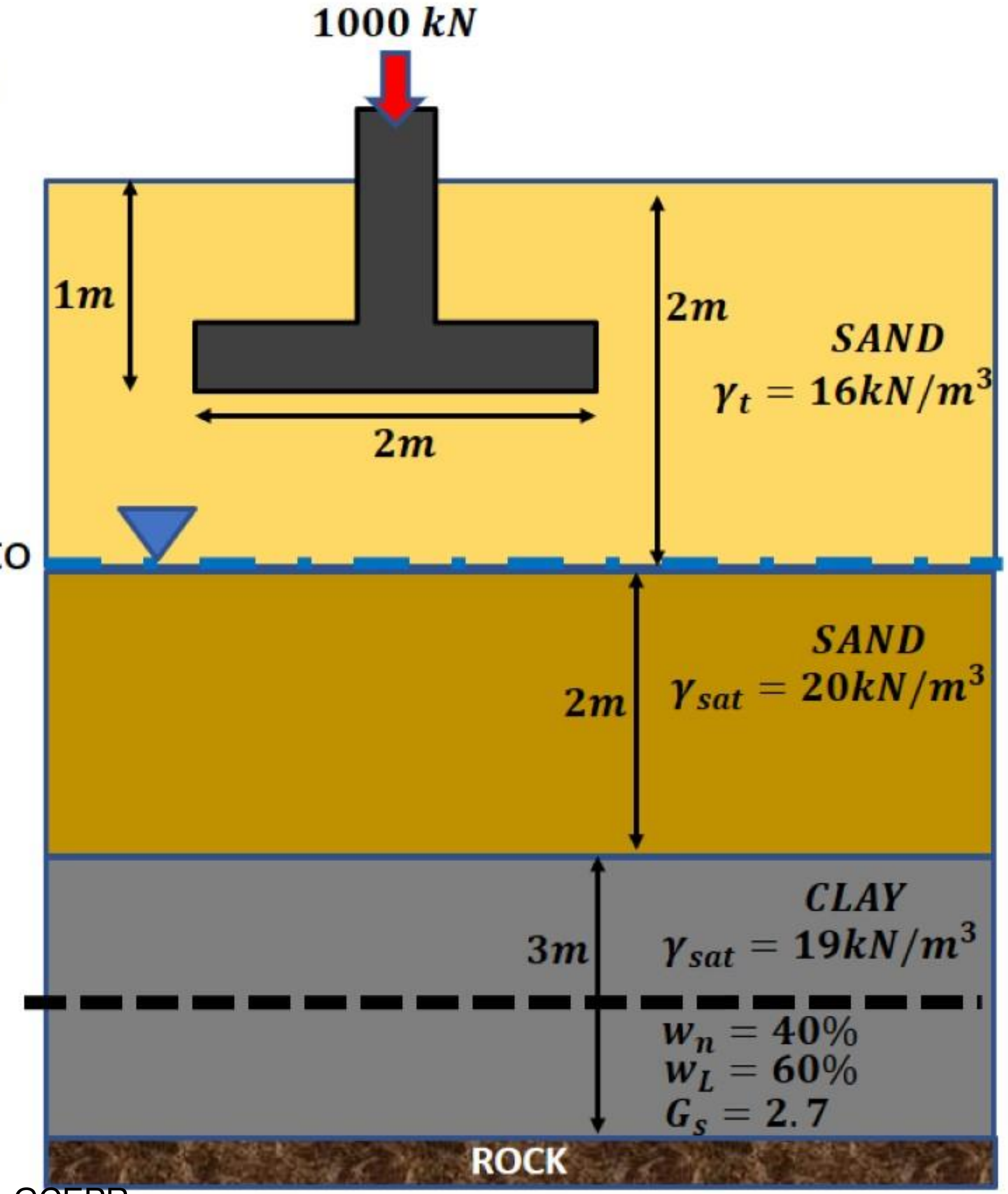
Que. Undisturbed soil sample 30mm thick got 50% consolidation in 20minutes with drainage allowed at top and bottom in the lab. If the clay layer from which the sample was obtained in 3m thick in field condition, estimate the time it will take for consolidation 50% with –

i) **Double surface drainage**

ii) **Single surface drainage if in both cases consolidation pressure is uniform.**

Que. A soil of specific gravity 2.67 has a moisture content of 20%. When fully saturated 2cm thick sample of this soil tested in a consolidometer, compression of 0.050cm when the load is increased from $45kN/m^2$ to $90kN/m^2$. Determine the compression index of soil.

Que. The subsoil profile at a site of construction is shown in figure. The footing is square footing of base size $2m \times 2m$ and carries a load of $1000kN$ and laid at a depth of 1m. Assume that after construction, settlement of sand is negligible. Determine the consolidation settlement of clay layer on account of construction. The clay is normally consolidated. Use 2V:1H load spreading to estimate the stress increase in the clay layer



$$\Delta H = \frac{c_c H_0}{1+e_0}\log_{10}\left(\frac{\overline{\sigma}_0 + \Delta\overline{\sigma}}{\overline{\sigma}_0}\right)$$

$c_c = 0.009(w_L - 10)$

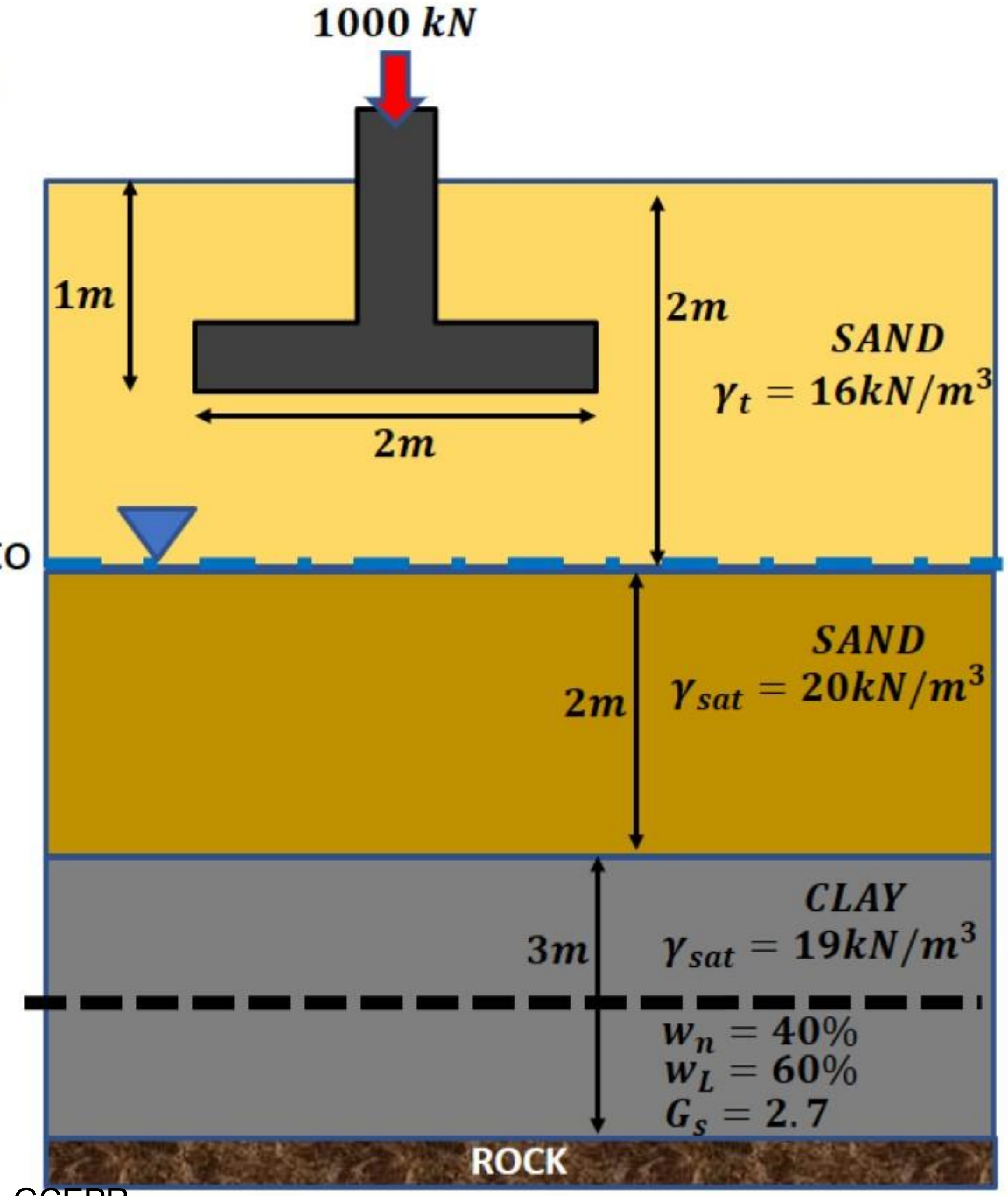
$\Rightarrow c_c = 0.009(60 - 10) = 0.45$

$e_0 = w_n G_s$

$\Rightarrow e_0 = 0.4 \times 2.7$

$\Rightarrow e_0 = 1.08$

Que. The subsoil profile at a site of construction is shown in figure. The footing is square footing of base size $2m \times 2m$ and carries a load of $1000kN$ and laid at a depth of 1m. Assume that after construction, settlement of sand is negligible. Determine the consolidation settlement of clay layer on account of construction. The clay is normally consolidated. Use 2V:1H load spreading to estimate the stress increase in the clay layer



$$\Delta H = \frac{c_c H_0}{1+e_0}\log_{10}\left(\frac{\overline{\sigma}_0 + \Delta\overline{\sigma}}{\overline{\sigma}_0}\right)$$

$$\overline{\sigma}_0 = \gamma_t \times 2 + \gamma_{sub}^{sand} \times 2 + \gamma_{sub}^{Clay} \times 1.5$$

$$\Rightarrow \overline{\sigma}_0 = 16 \times 2 + (20 - 9.81) \times 2 + (19 - 9.81) \times 1.5$$

$$\Rightarrow \overline{\sigma}_0 = 66.165\ kN/m^2$$

$\Delta\bar{\sigma} = Change\ in\ effective\ stress\ at\ the\ centre\ of\ clay\ layer$

$$\Delta\bar{\sigma} = \frac{P}{(b+z)^2}$$

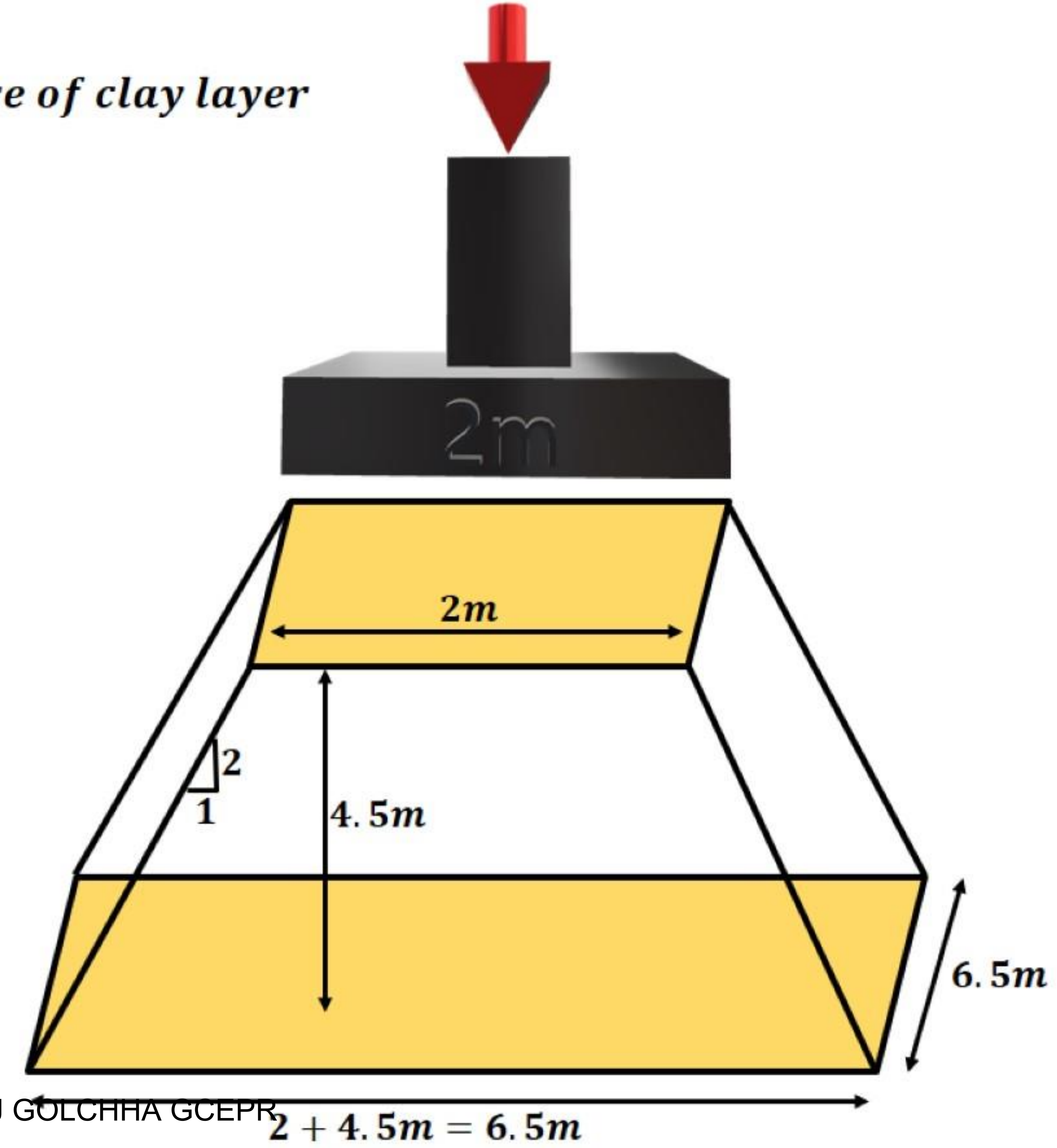
$$\Rightarrow \Delta\bar{\sigma} = \frac{1000}{(6.5)^2}$$

$$\Rightarrow \Delta\bar{\sigma} = 23.668\ kN/m^2$$

$$\Delta H = \frac{c_c H_0}{1+e_0}\log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$\Rightarrow \Delta H = \frac{0.45 \times 3}{1+1.08}\log_{10}\left(\frac{66.165 + 23.668}{66.165}\right)$$

$$\Rightarrow \Delta H = 86.2mm$$



Computation of Secondary Consolidation

$$\frac{\Delta H_S}{H_f} = \frac{\Delta e}{1 + e_f}$$

$e_f = void\ ratio\ after\ the\ end\ of\ primary\ consolidation$

$H_f = height\ of\ the\ layer\ at\ the\ end\ of\ Primary\ Consolidation$

$C_f = Secondary\ Compression\ Index$

$$C_f = \frac{\Delta e}{\Delta logt}$$

$\Delta e = decrease\ in\ void\ ratio$

$\Delta logt = \log of\ time\ corresponding\ to\ time\ \Delta t$

SOIL MECHANICS

Civil Engineering by Sandeep Jyani

Shear Strength

SHEAR STRENGTH

- Whatever be the nature of loading on Soil, failure always occur by shearing, not by Crushing of Particles
- Shearing Stresses tend to displace a part of soil mass with respect to another
- Shear Failure occurs when shear stress exceeds Shear Strength of Soil

SHEAR STRENGTH

- **Whatever be the nature of loading, soil failure always occur by shearing, not by crushing of particles.**
- **Shear failure occurs when shear stress exceeds shear strength of soil.**
- **Soils consist of individual particles that can slide and roll relative to one another. Shear strength of a soil is equal to the maximum value of shear stress that can be mobilized within a soil mass without failure taking place.**
- **The shear strength of a soil is a function of the stresses applied to it as well as the manner in which these stresses are applied.**
- **It is necessary to determine the bearing capacity of foundations, the lateral pressure exerted on retaining walls, and the stability of slopes.**

Mechanism of SHEAR STRENGTH

Shear Strength is categorized into two parts:

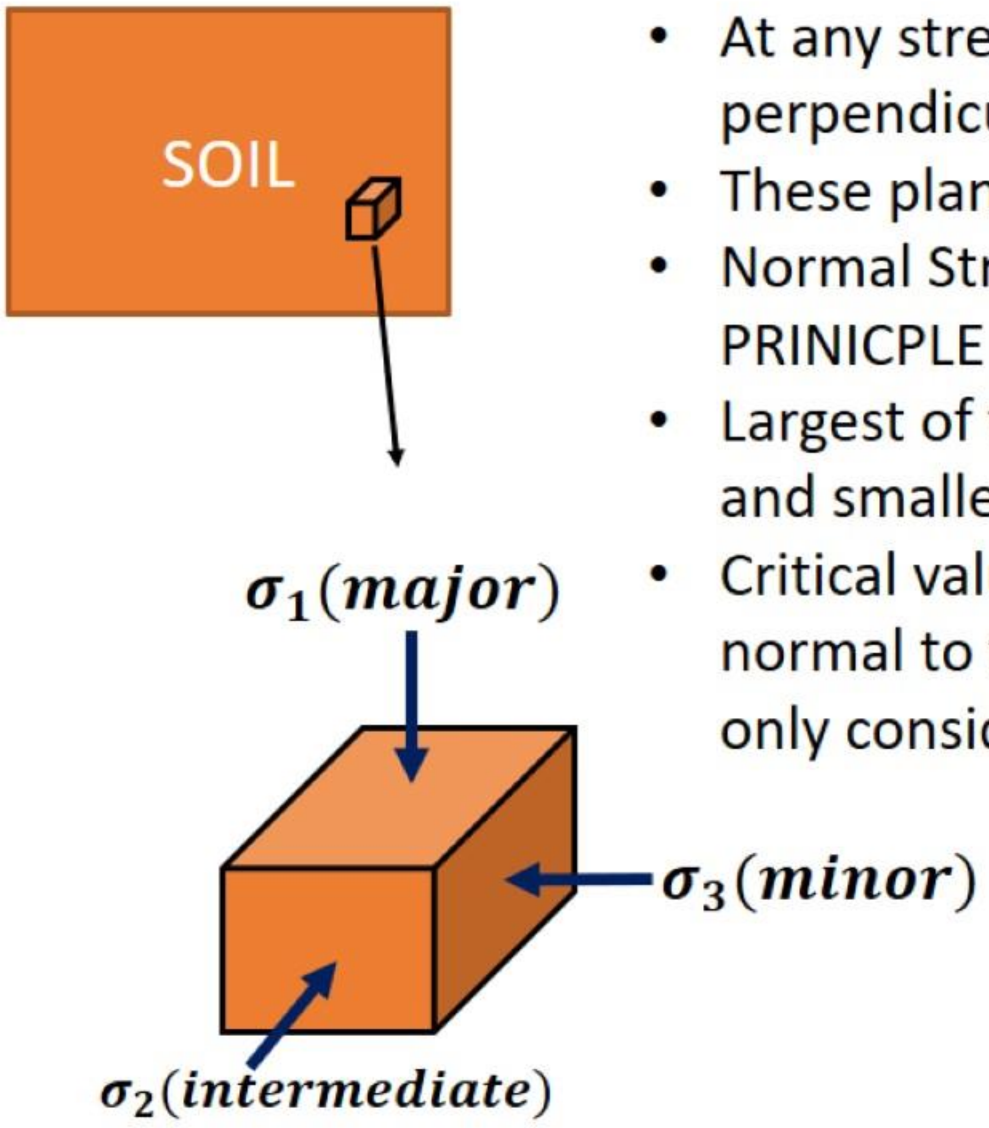
Friction + Interlocking of particles	Cohesion

Further,

Cohesion is of Two types:

1. **True Cohesion:** True Cohesion is due to electrostatic attraction between particles
2. **Apparent Cohesion**: It is due to negative pressure in the soil mass

Stress at a Point-MOHR's Circle of Stress

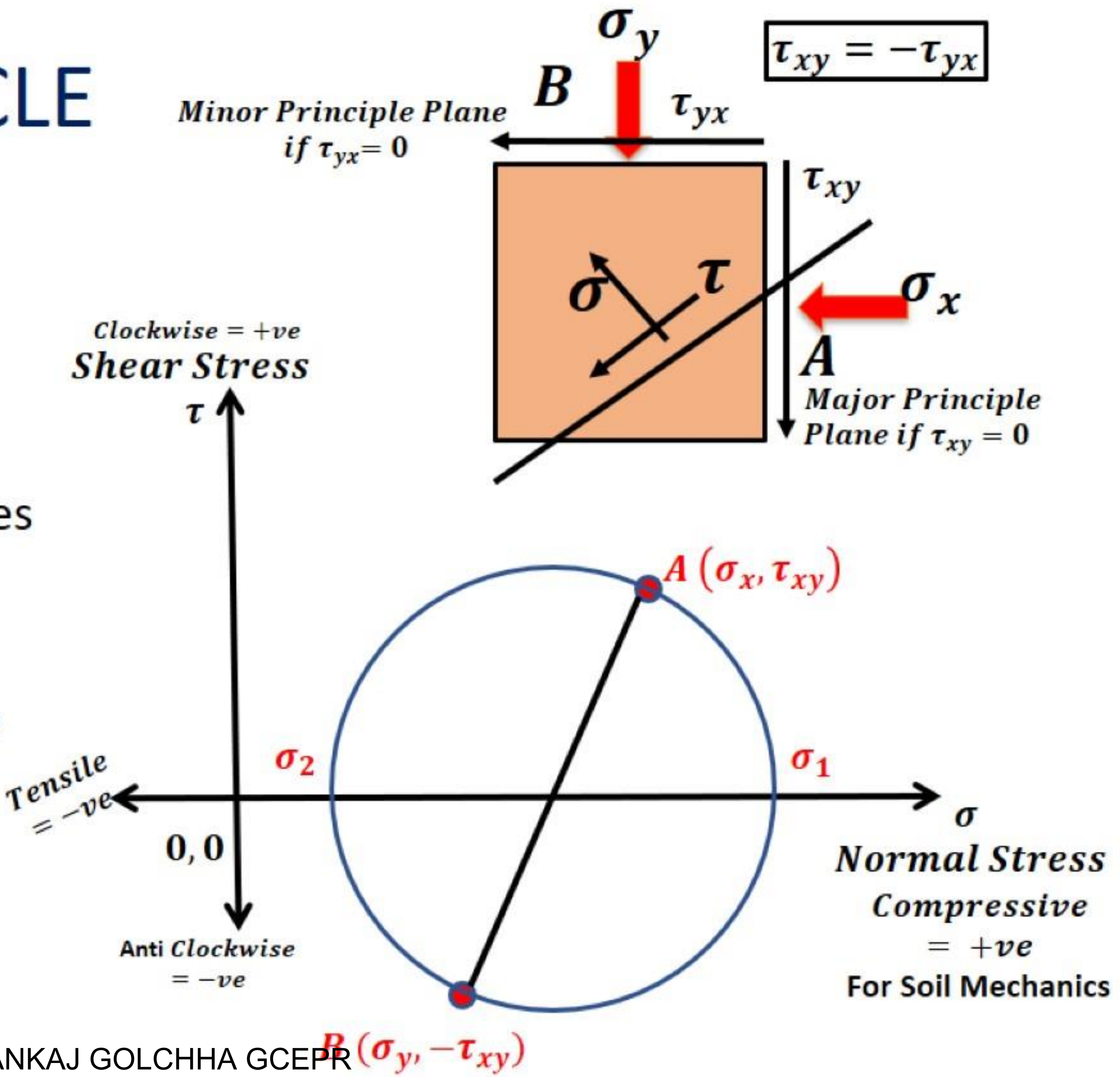


- At any stressed point in a soil, there are three mutually perpendicular planes in which no shear stress is acting.
- These planes are known as Principle Planes
- Normal Stresses acting on these planes are called as PRINICPLE STRESSES
- Largest of these is known as MAJOR PRINCIPLE STRESS and smallest as MINOR PRINCIPLE STRESS
- Critical values of stress generally occurs on plane normal to the intermediate plane, so for 2-D study, we only consider $\boldsymbol{\sigma_1 (major)}$ **and** $\boldsymbol{\sigma_3 (minor)}$ only

MOHR CIRCLE

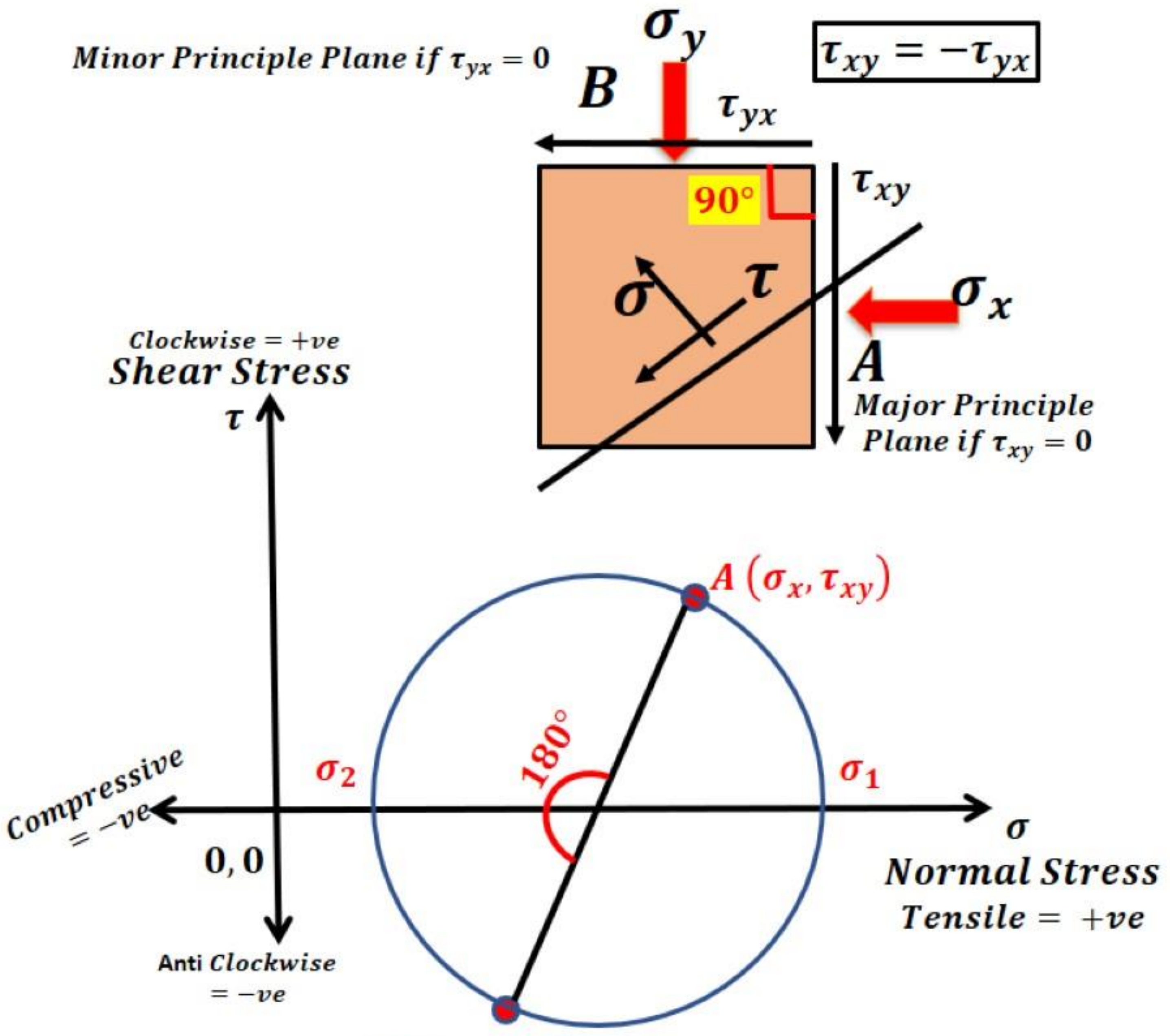
Guidelines to draw Mohr's Circle

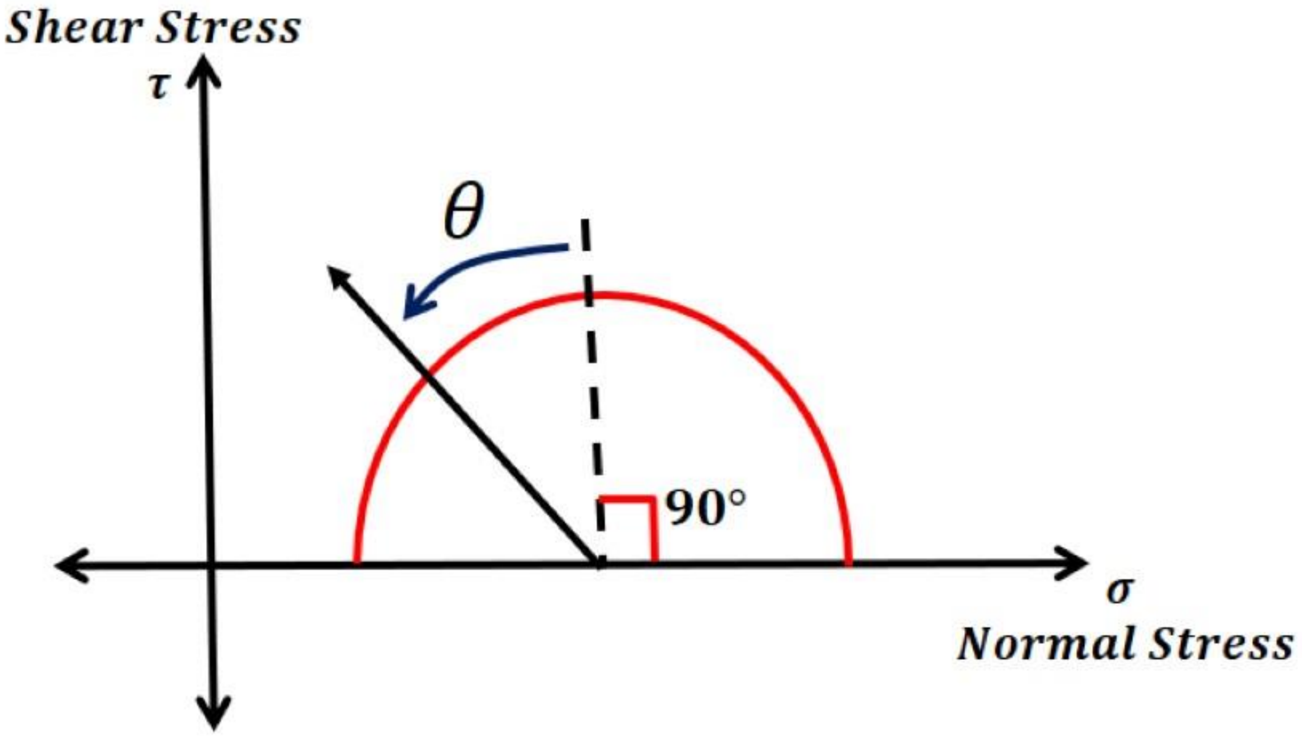
1. Draw the x axis representing Normal Stress and y axis representing Shear Stress
2. Mark the point A and B on the faces and denote the coordinates $A\,(\sigma_x, \tau_{xy}), B\,(\sigma_y, -\tau_{xy})$
3. Join the points A and B, and bisect the line AB, and draw the circle



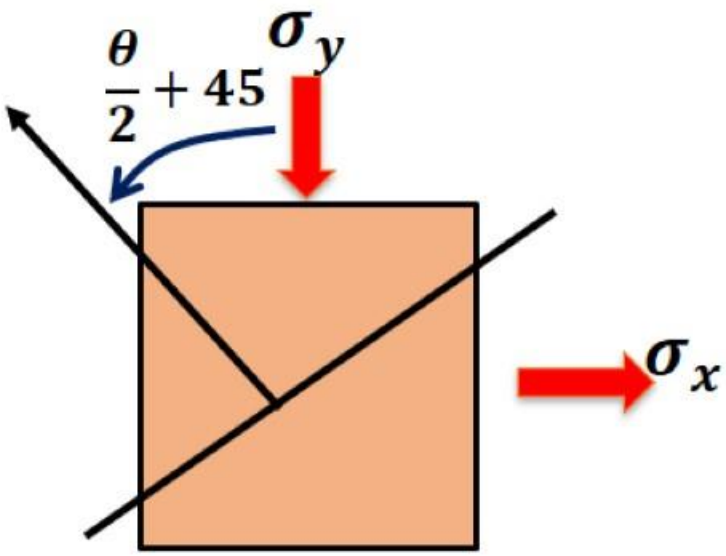
Conclusions

The angle between any two planes on Mohr's Circle is twice the actual angle





Shear Stress
τ
θ
$90°$
σ
Normal Stress



$\frac{\theta}{2} + 45$
σ_y
σ_x

MOHR's Hypothesis

- As per Mohr, shear strength on failure plane reaches the value which is a unique function of *Normal stress* on that plane

$$\tau_f = f(\sigma_f)$$

- Mohr's theory is based on logical arrangement of experimental results
- The characteristics of soil are not considered while constructing Mohr's circle
-

COULOMB Hypothesis

$$\tau = c + \sigma tan\emptyset$$

$\tau = shear\ strength$

$\sigma = total\ stress$

$\emptyset = angle\ of\ internal\ friction$

$c\ \&\ \emptyset\ are\ combinely\ called\ total\ stress, shear\ strength\ parameter$

MOHR COULOMB CRITERIA

- **After the discovery of effective stress, shear strength is written as –**

$$\tau = c' + \bar{\sigma} tan\emptyset'$$

$$\bar{\sigma} = effective\ stress$$

$$c' \,\&\, \emptyset' are\ combinely\ called\ effective\ stress, shear\ strength\ parameter$$

- **NOTE:**
  a) **For sand (cohesionless soil)**

     $c = 0\ , \emptyset\ exist$

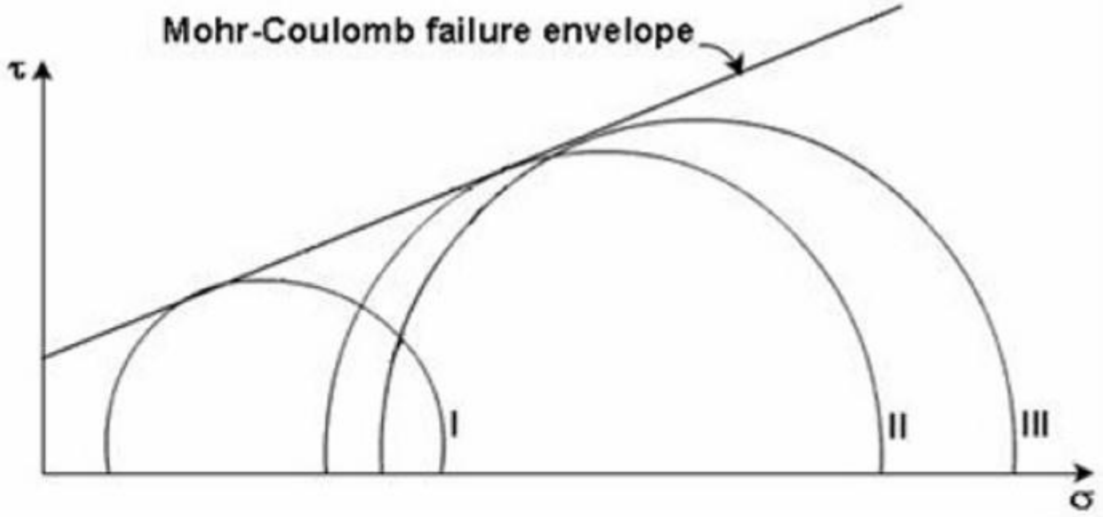
  b) **For clay (purely cohesive soil)**

     $\emptyset = 0\ , c\ exist$

$c \ and \ \emptyset$

- c and $\emptyset$ are not inherent properties of soil
- They are related to the type of test and condition to which the test is done
- Hence $c \ and \ \emptyset$ depends on
  a) Slow or rapid loading condition
  b) Drained or undrained condition
  c) Type of soil
  d) Size of soil particles
  e) Water Content

MOHR COULOMB CRITERIA



$$y = mx + c$$

$$\tau = \sigma tan\emptyset + c$$

$$\tau = shear\ strength$$

$$\sigma = total\ stress$$

$$\emptyset = angle\ of\ internal\ friction$$

c & $\emptyset$ *are combinely called total stress, shear strength parameter*

As per Mohr Coulomb criteria, if series of Mohr Circle are drawn corresponding to different tests carried out on different samples of same soil, upto failure, then a common tangent to all Mohr circle will give a failure envelope representing

$$\tau = c + \sigma tan\emptyset$$

$$dia = \sigma_{1f} - \sigma_{3f}$$

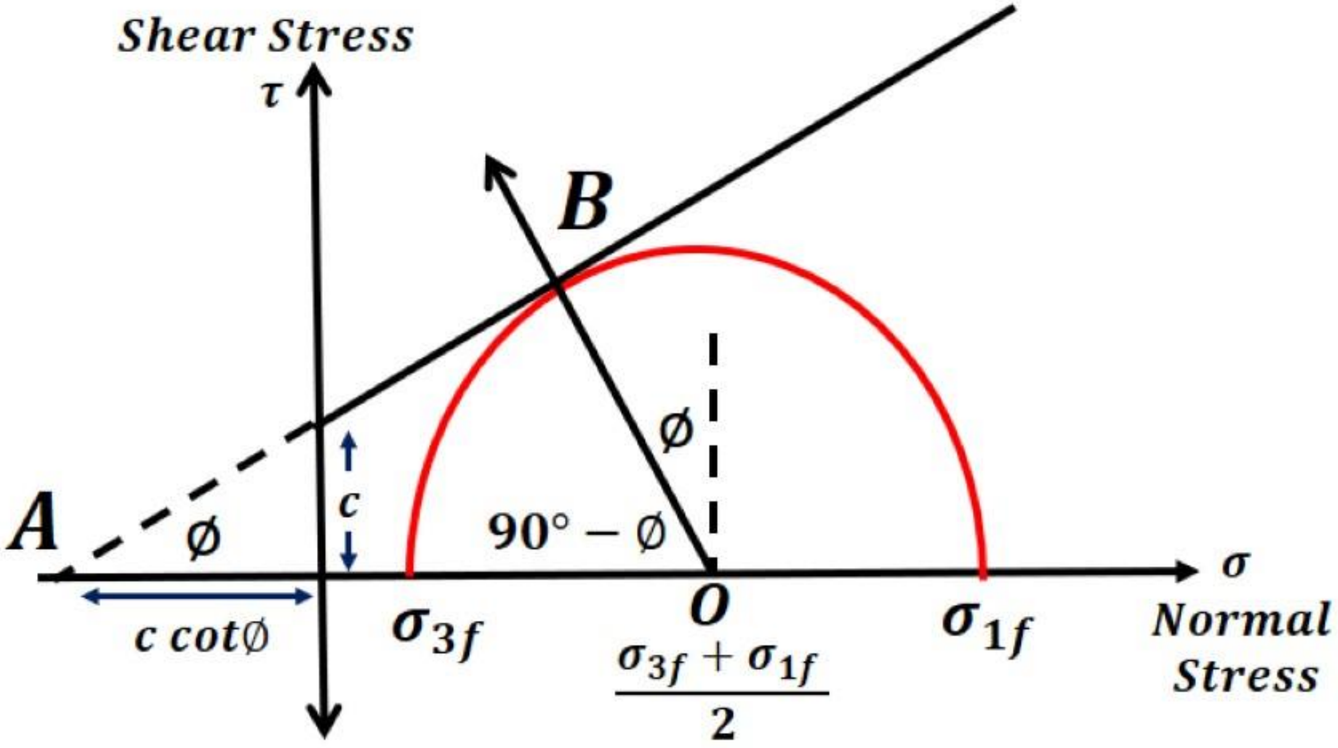
$$radius = \frac{\sigma_{1f} - \sigma_{3f}}{2}$$

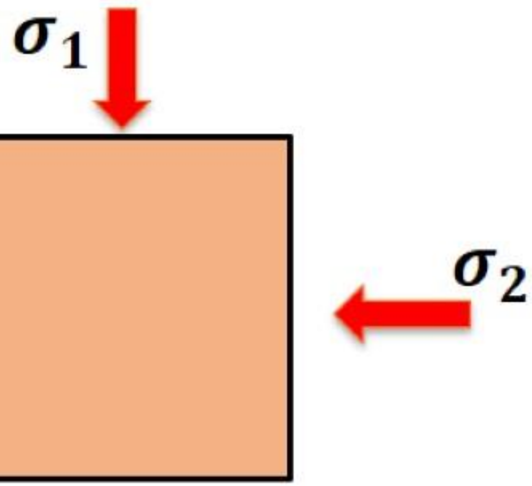
In $\Delta ABO,$

$$BO = \frac{\sigma_{1f} - \sigma_{3f}}{2}$$

$$AO = c\cot\emptyset + \frac{\sigma_{1f} + \sigma_{3f}}{2}$$

From $\Delta ABO,$ $$\sin\emptyset = \frac{OB}{OA} = \frac{\frac{\sigma_{1f} - \sigma_{3f}}{2}}{c\cot\emptyset + \frac{\sigma_{1f} + \sigma_{3f}}{2}}$$





From ΔABO,

$$\sin\phi = \frac{OB}{OA} = \frac{\dfrac{\sigma_{1f} - \sigma_{3f}}{2}}{c\cot\phi + \dfrac{\sigma_{1f} + \sigma_{3f}}{2}}$$

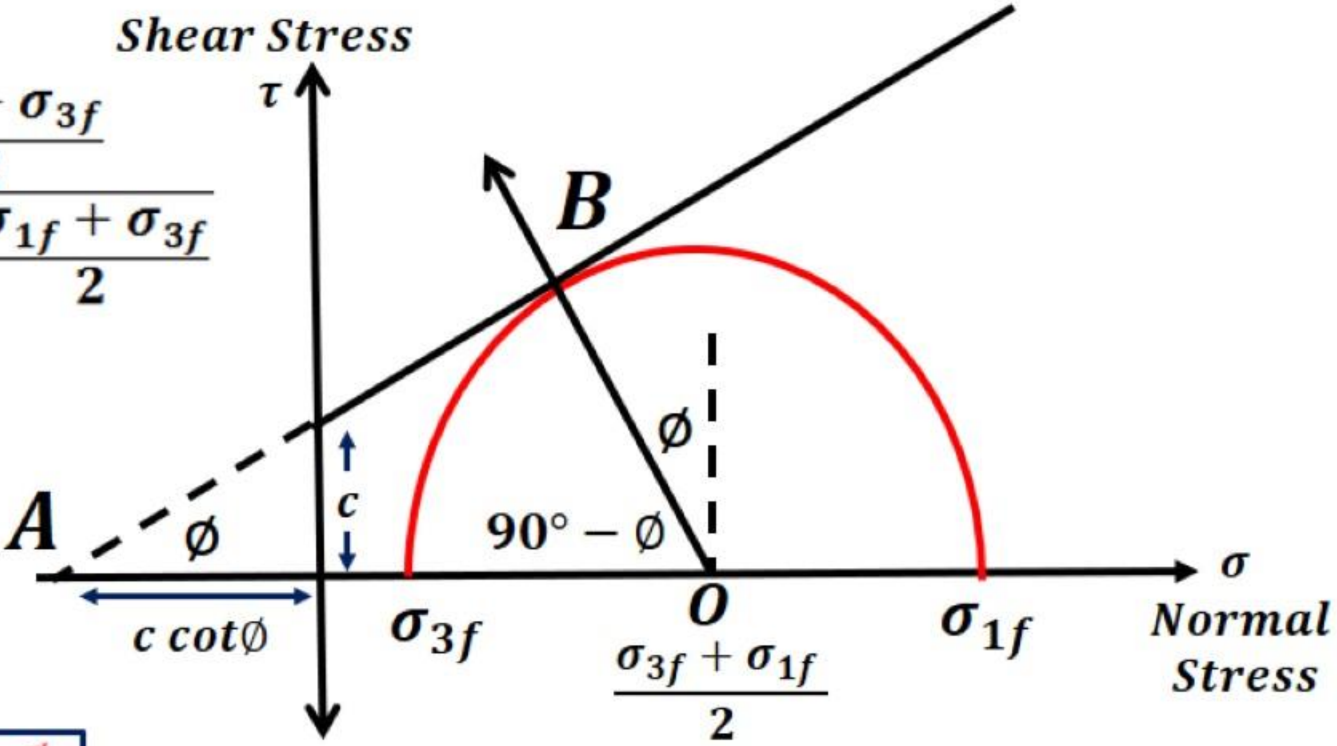
$$c\cos\phi + \sin\phi\left(\frac{\sigma_{1f} + \sigma_{3f}}{2}\right) = \frac{\sigma_{1f} - \sigma_{3f}}{2}$$

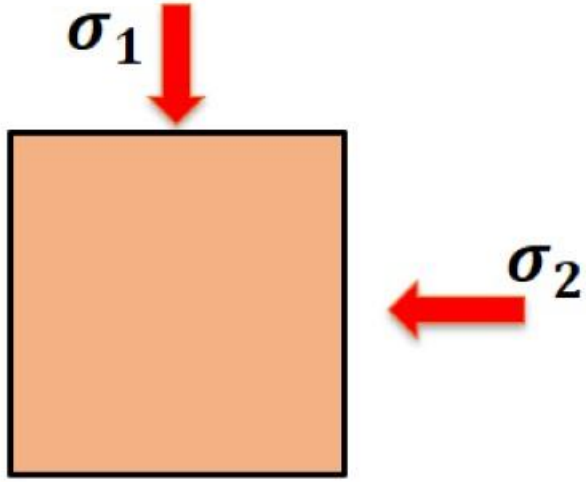
$$\sigma_{1f} = \sigma_{3f}\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right) + 2c\sqrt{\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)}$$

$$\boxed{\sigma_{1f} = \sigma_{3f}\tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)}$$

$$\boxed{\sigma_{3f} = \sigma_{1f}\tan^2\left(45 - \frac{\phi}{2}\right) - 2c\tan\left(45 - \frac{\phi}{2}\right)}$$

σ_{1f} = *major principle stress* σ_{3f} = *minor principle stress*





Que. Determine the axial stress at failure for dry dense sand in case of a triaxial loading of σ_3=300kN/m^2, If previous test load is given as σ_3=150kN/m^2 and σ_1=735kN/m^2 at failure.

$$\boxed{\sigma_1 = \sigma_3 tan^2\left(45 + \frac{\emptyset}{2}\right) + 2c\, tan\left(45 + \frac{\emptyset}{2}\right)}$$

Que. A sample of saturated cohesionless soil tested in a drain triaxial compression test at an angle of internal friction 30°, the deviator stress σ_d at failure for the sample of a confined pressure 200kN/m² will be ….?

$$\boxed{\sigma_1 = \sigma_3 tan^2\left(45 + \frac{\emptyset}{2}\right) + 2c\, tan\left(45 + \frac{\emptyset}{2}\right)}$$

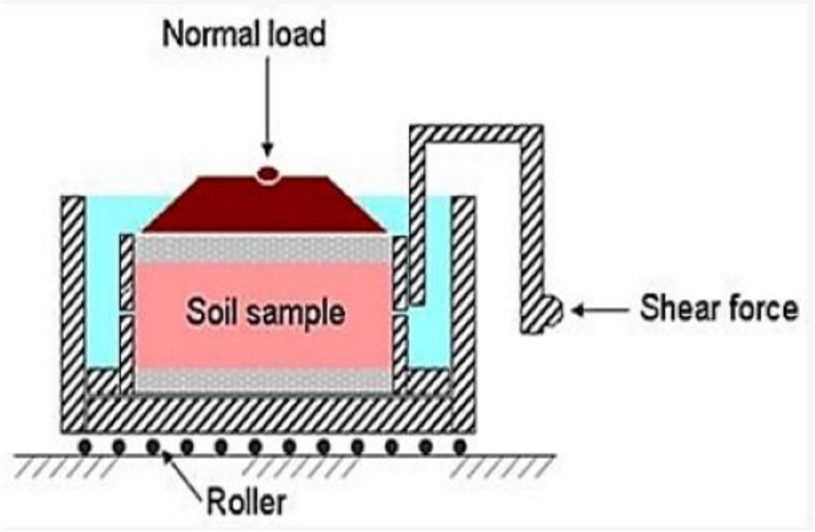
Que. In a drained triaxial compression test conducted on dry sand, failure occur when deviator stress was 218kN/m² at a confined pressure of 61kN/m². Find the angle of shearing resistance and the inclination of failure plane to major principle plane.

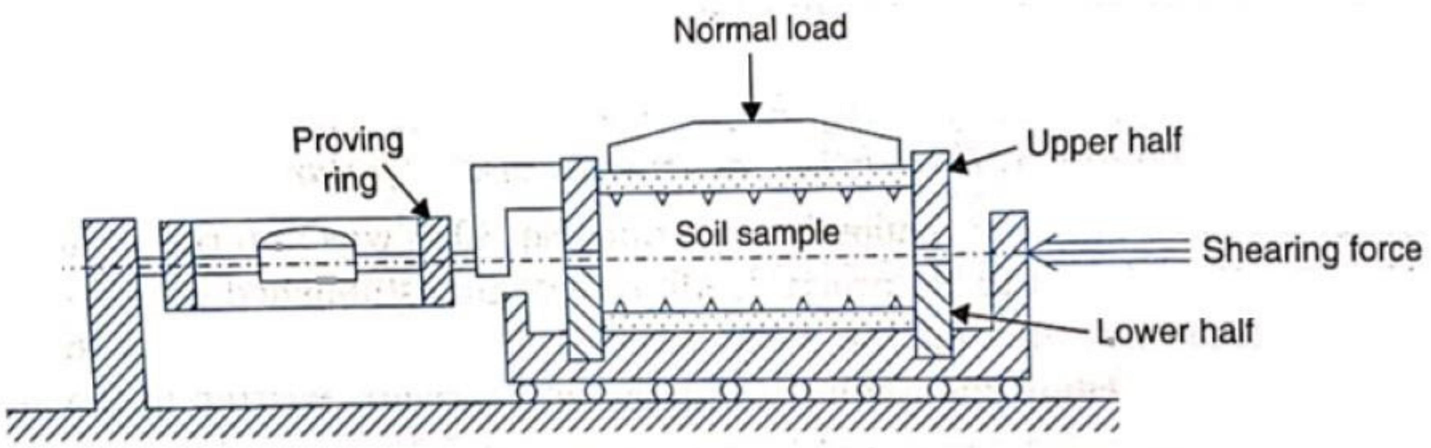
$$\boxed{\sigma_1 = \sigma_3 tan^2\left(45 + \frac{\varnothing}{2}\right) + 2c\, tan\left(45 + \frac{\varnothing}{2}\right)}$$

Shear Strength Tests

1. Direct Shear Test

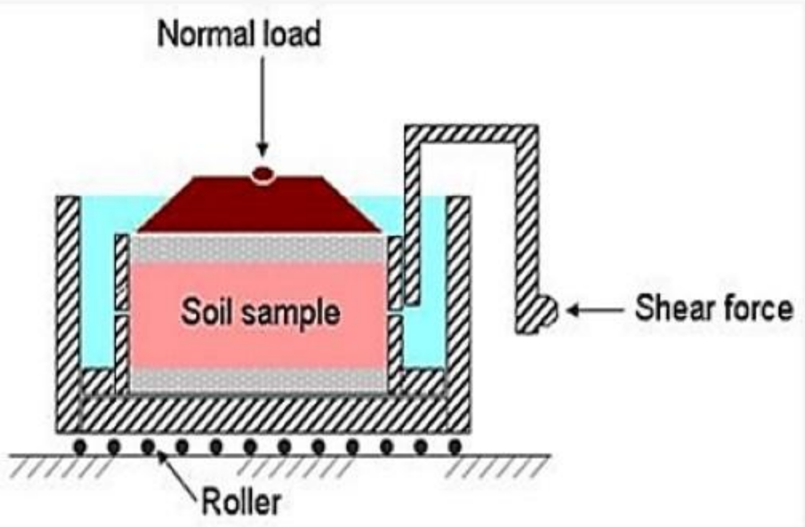
- The test is carried out on a soil sample confined in a metal box of square cross-section which is split horizontally at mid-height
- The soil is sheared along a predetermined plane by moving the top half of the box relative to the bottom half. The box is usually square in plan of size 60 mm x 60 mm.
- Magnitude of shear load is measured by proving ring, strain is measured using dial gauge
- Normal stresses and shear stresses are found by dividing normal and shear force by NOMINAL AREA
- Tests on sands and gravels can be performed quickly





Shear Strength Tests

1. Direct Shear Test



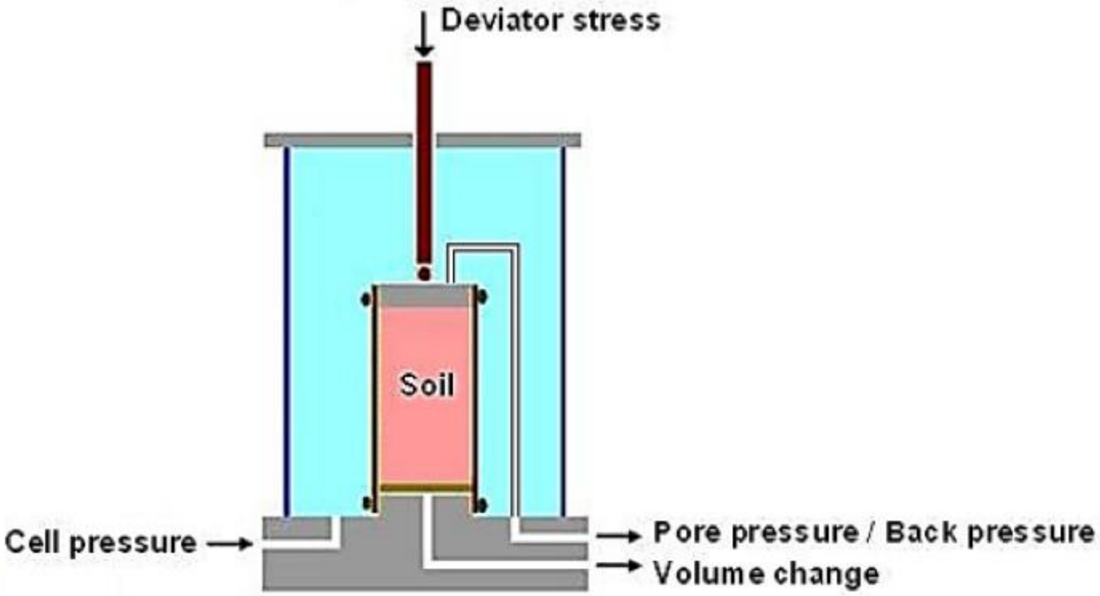
- ADVANTAGES
  - **Quick**
  - **Inexpensive**
  - **Simple**
  - **Sample preparation is easy in case of sands**
  - **Useful when failure plane is predefined**
- DISADVANTAGES
  - **Drainage conditions can not be controlled**
  - **Pore water pressure can not be measured**
  - **Which means it is not useful for fine grained soils where drainage conditions play an important role**
  - **Pre defined failure plane may not be the weakest plane**
  - **The direction of principle planes are unknown at every stage of the test**

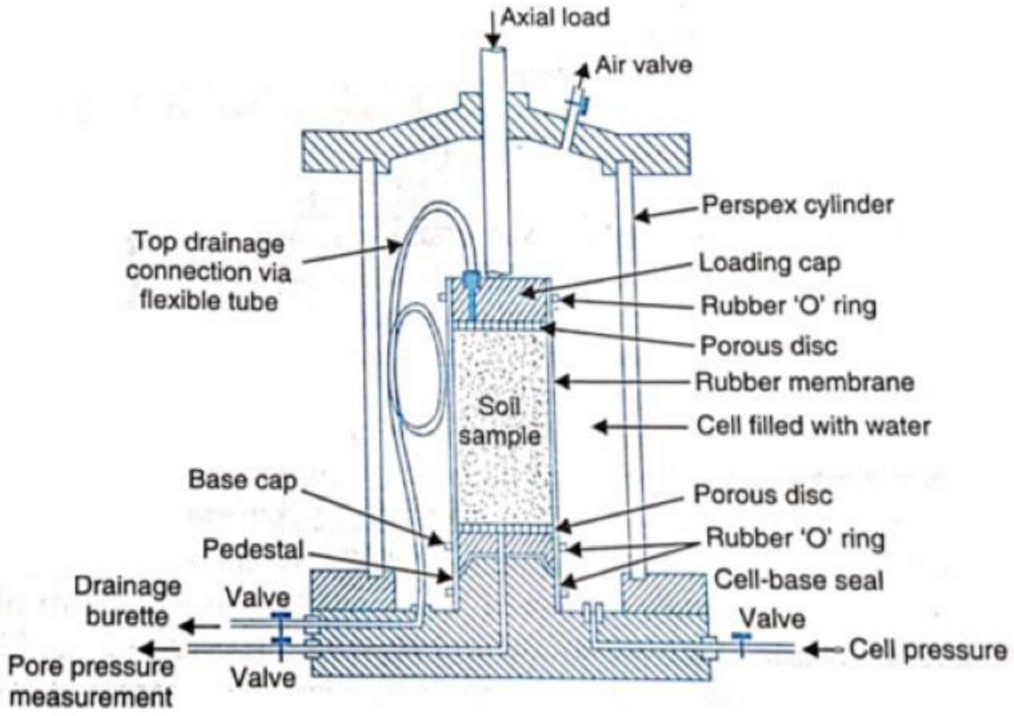
Shear Strength Tests

2. Triaxial Test

A Casagrande developed the triaxial Test

- **Drainage conditions can be controlled**
- **Pore water pressure can be measured**
- **Soils can be tested under drained(coarse grain) as well as undrained conditions (fine grain)**
- **Volume change of specimen can also be measured**

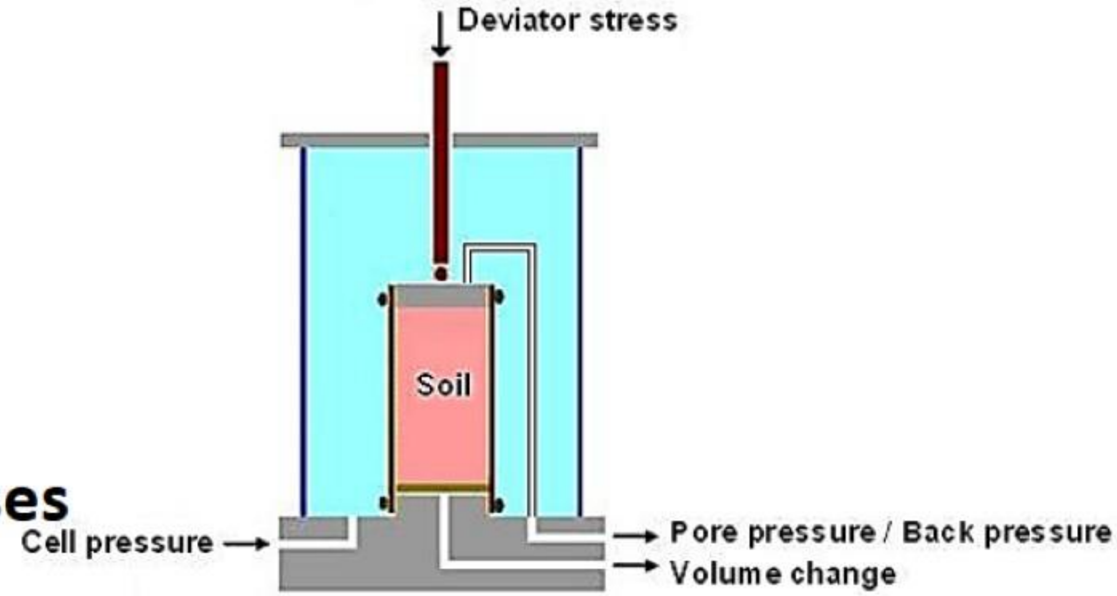


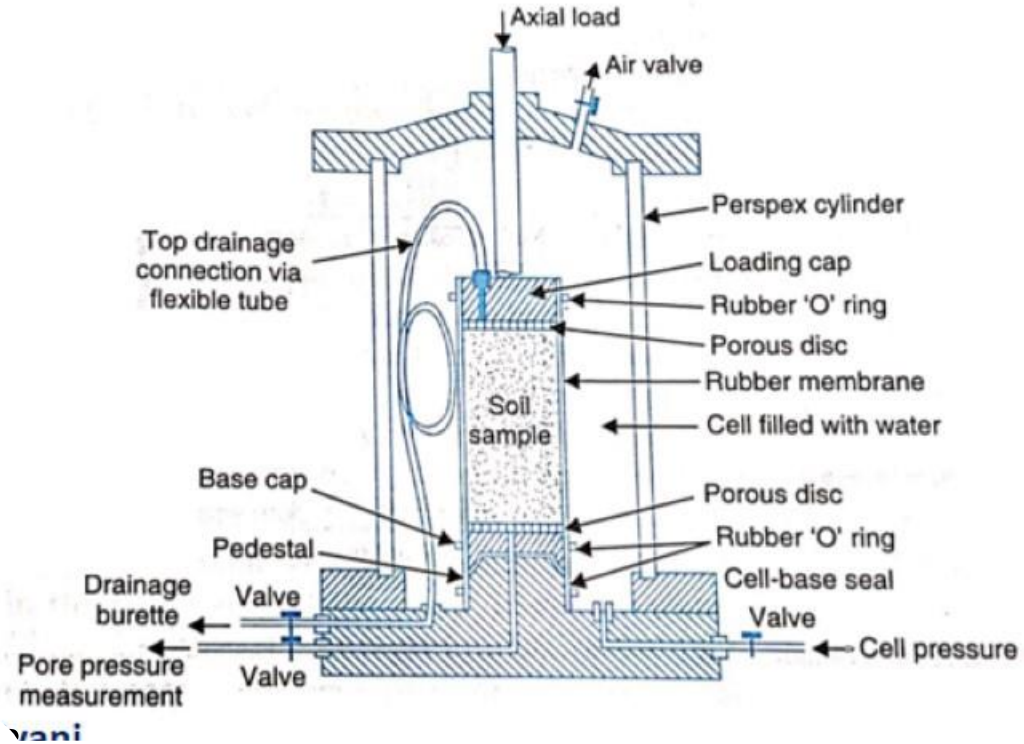


Shear Strength Tests

2. Triaxial Test

- There is no rotation of principle stresses during the test
- Failure plane is not pre determined
- Stress distribution is uniform
- Specimen being cylindrical, there is no accumulation of stresses unlike Direct Shear Test
- Specimen has the size of $76 \times 38mm$





Shear Strength Tests

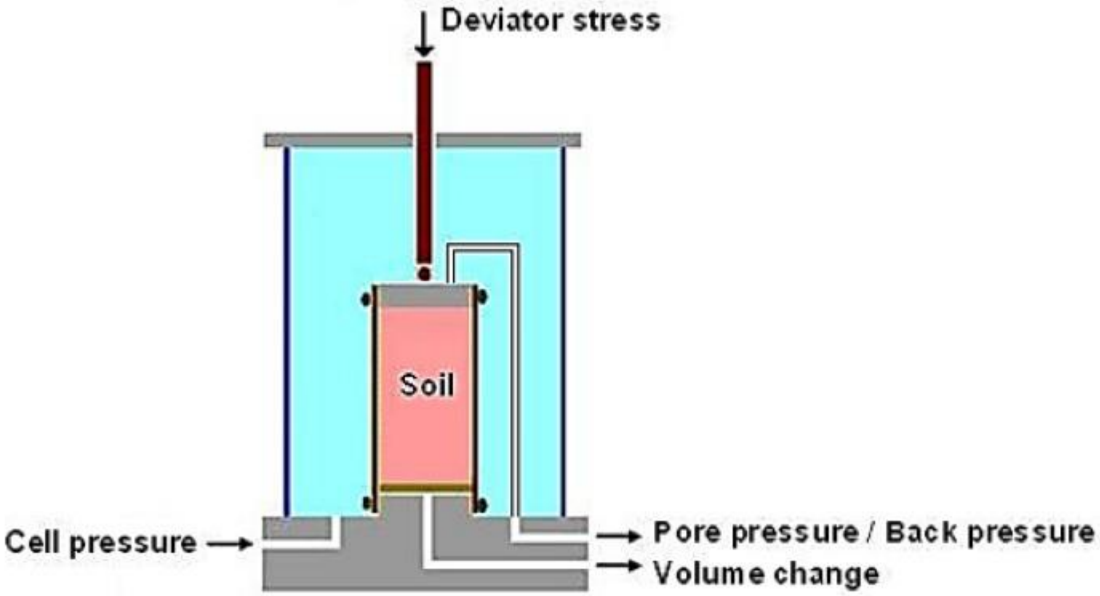
2. Triaxial Test

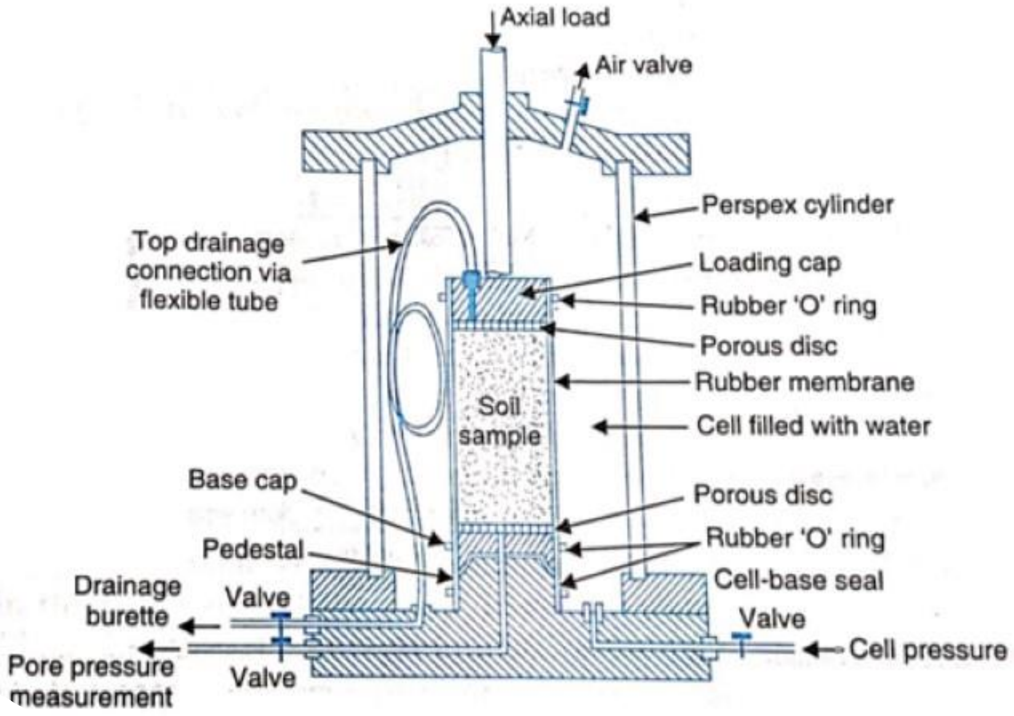
A Casagrande developed the triaxial Test

The triaxial compression test consists of two stages:

First stage: In this, a soil sample is set in the triaxial cell and confining pressure is then applied.

Second stage: In this, additional axial stress (also called deviator stress) is applied which induces shear stresses in the sample. The axial stress is continuously increased until the sample fails.

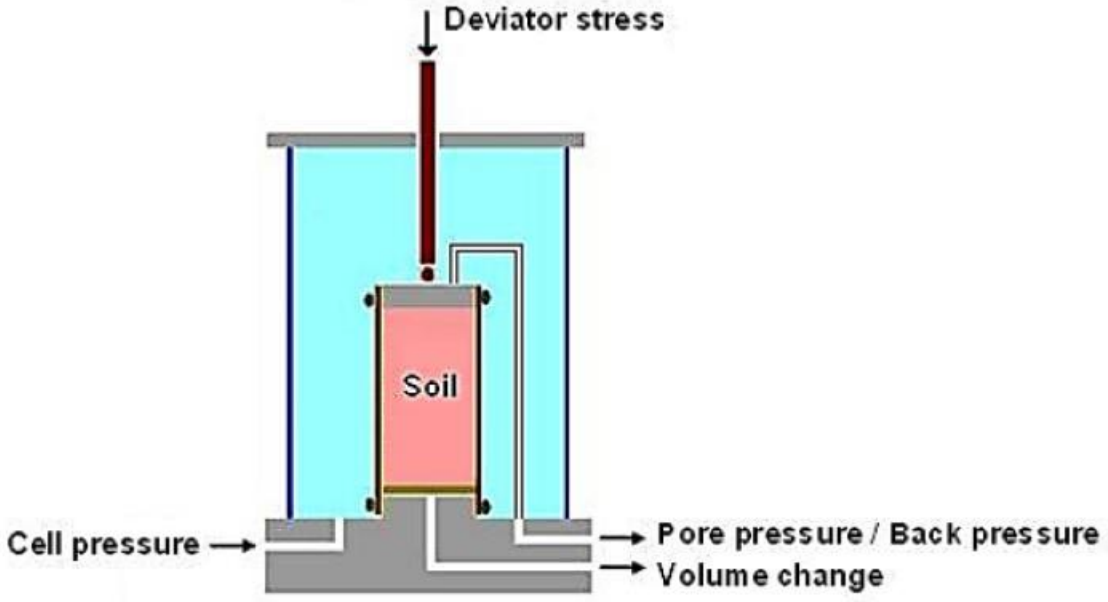




Triaxial Test Types

There are several test variations, mainly used in practice are:

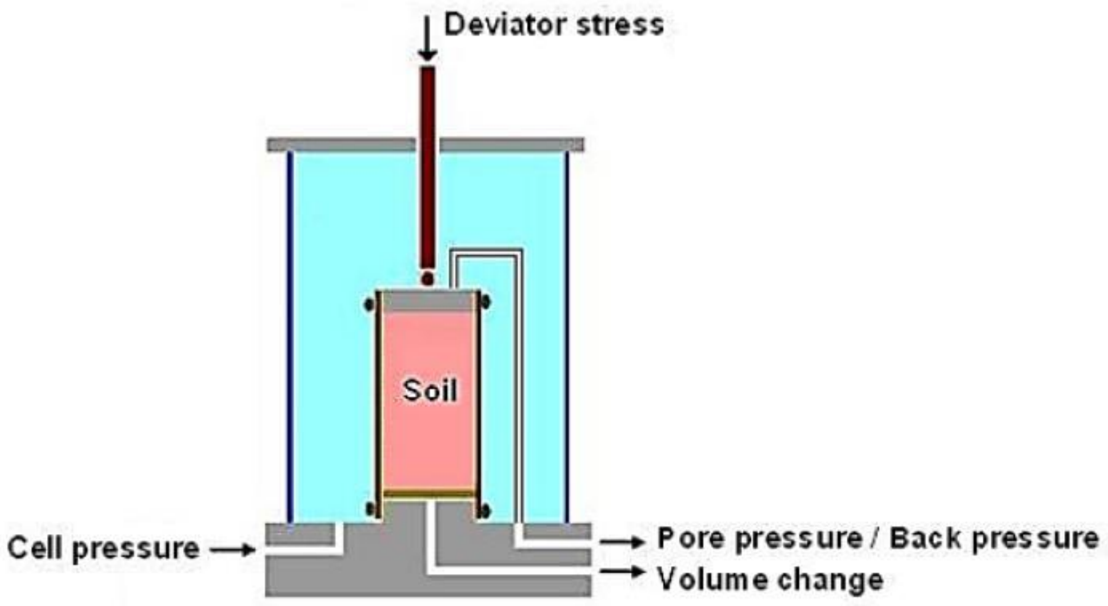
1. UU (Unconsolidated Undrained) Test
    - **In this, cell pressure is applied without allowing drainage**
    - **Then keeping cell pressure constant, deviator stress is increased till failure without drainage**
    - **It takes overall 15 minutes**
    - **It is used in case where sudden loading such as Rapid construction is there**
    - **It is used for short term stability**



Triaxial Test Types

2. CU (Consolidated Undrained) Test

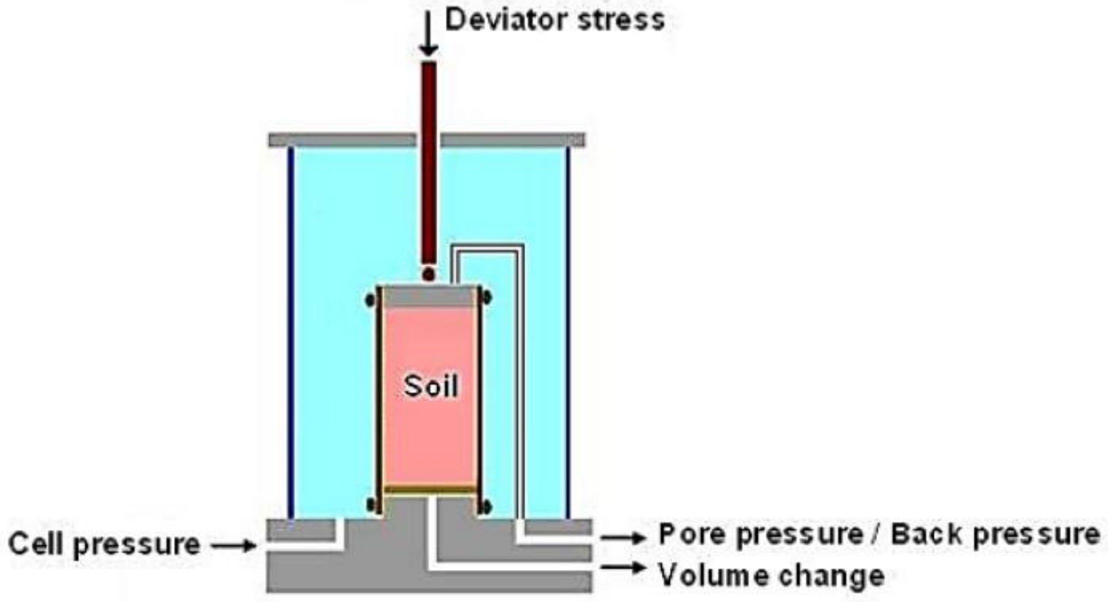
- **In this, drainage is allowed during cell pressure application (1st stage)**
- **Then without allowing further drainage (2nd stage), deviator stress is increased keeping cell pressure constant**
- **1st stage consolidation takes 24 hours**
- **2nd stage consolidation takes 2 hours**
- **In this test we find Total Stress parameters**
- **It is used where embankment has lived some of its life and now has sheared**



Test Types

There are several test variations, mainly used in practice are:

3. CD (Consolidated Drained) Test
   - **This is similar to CU test except that deviator stress is increased, drainage is permitted**
   - **Rate of loading should be slow enough to ensure that no excess pore water pressure develops**
   - **It takes longest time**
   - **This test is used to determine long term stability in embankments which have been in existence since long ago**
   - **In this test we find Effective stress parameters**

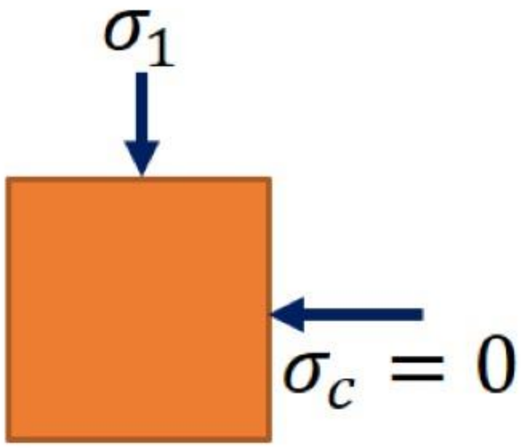


The triaxial test has many **advantages** over the direct shear test:

- The soil samples are subjected to uniform stresses and strains.
- Different combinations of confining and axial stresses can be applied.
- Drained and undrained tests can be carried out.
- Pore water pressures can be measured in undrained tests.
- The complete stress-strain behavior can be determined

Unconfined Compression Test

- It is a special case of triaxial test in which No confining pressure is applied
- It is used to test cohesive soil
- Loading rate is so fast that undrained conditions occur
- Angle of shearing resistance will not get mobilised
- This test can not be done for sand and gravel because the sample can not stand without confinement



$$c = \frac{q}{2}$$

$q = unconfined$
$compresive\ strength$

Shear Strength Tests

3. Vane Shear Test

- In plastic cohesive soil, cohesion is obtained with Vane Shear Test
- The cane is pushed into soil and twisted upto soil failure
- Failure is decided on the basis of torque required
- Maximum torque applied corresponding to Total Shear Resistance

Shear Strength Tests

3. Vane Shear Test

- Torque required for shearing on the surface of cylinder

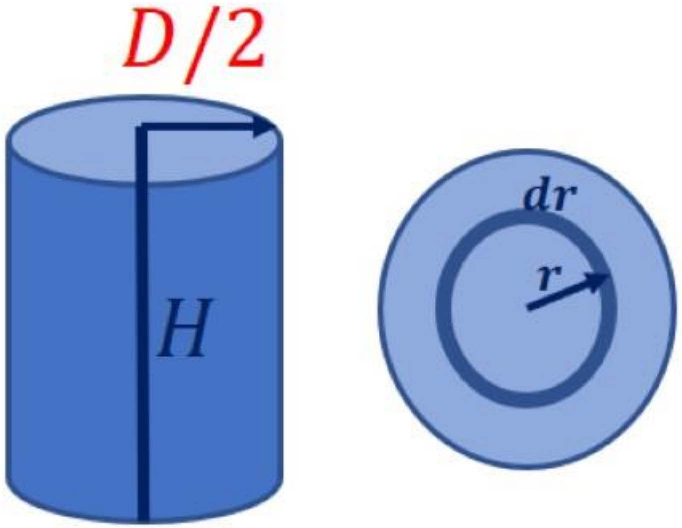
$$T_1 = \underbrace{c(\pi D)H}_{Shear\ strength \times Area} \times \underbrace{\frac{D}{2}}_{Distance}$$

- Torque required for shearing on TOP and BOTTOM

$$T_2 = \int_0^{D/2} c(2\pi r)dr \times r$$

$$= 2\pi c\left(\left(\frac{r^3}{3}\right)_0^{D/2}\right)$$

$$= \pi c\frac{D^3}{12}$$



Shear Strength Tests

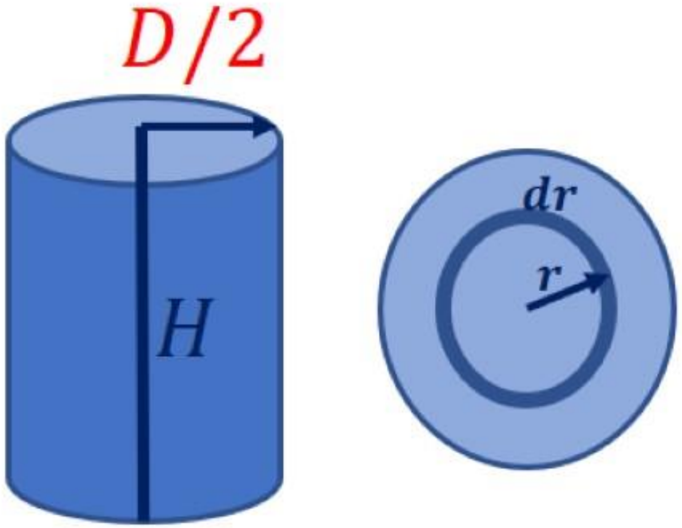
3. Vane Shear Test

- TOTAL TORQUE

$$T = T_1 + 2T_2$$

$$= c(\pi D)H \times \frac{D}{2} + 2 \times \pi c \frac{D^3}{12}$$

$$C = \frac{T}{\pi D^2 \left(\frac{h}{2} + \frac{d}{6}\right)}$$



Shear Strength Tests

3. Vane Shear Test

Used in plastic cohesive clays where obtaining undisturbed sample is difficult

$$C = \frac{T}{\pi D^2 \left(\frac{h}{2} + \frac{d}{6}\right)}$$

$$S = \frac{T}{\pi\left(\frac{D^2 H}{2} + \frac{D^3}{6}\right)}$$

Total Torque $T = \pi c D^2 \ (H/2 + D/6)$

If Vane is pushed $T = \pi c D^2 \ (h/2 + D/12)$
to depth h only

Shear Strength Tests

3. Vane shear Test

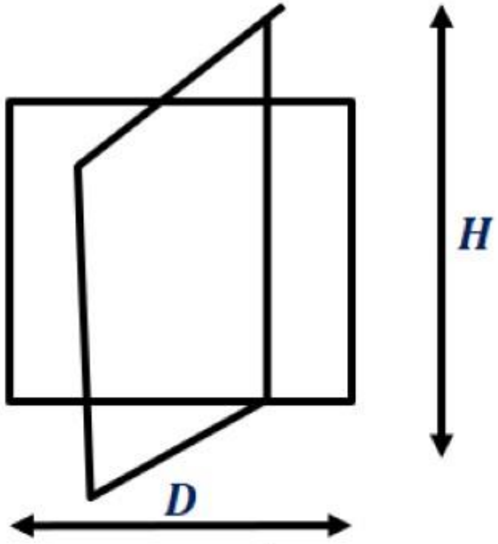
- In plastic cohesive soils, cohesion is obtained with Vane Shear Test
- Vane is pushed into the soil and twisted upto soil failure
- Failure is decided on the basis of Torque Required
- Maximum torque applied corresponds to Total Shear Resistance
- From Vane Shear Test, liquid limit can also be determined

$$C = \frac{T}{\pi D^2 \left(\frac{h}{2} + \frac{d}{6}\right)}$$

Shear Strength Tests

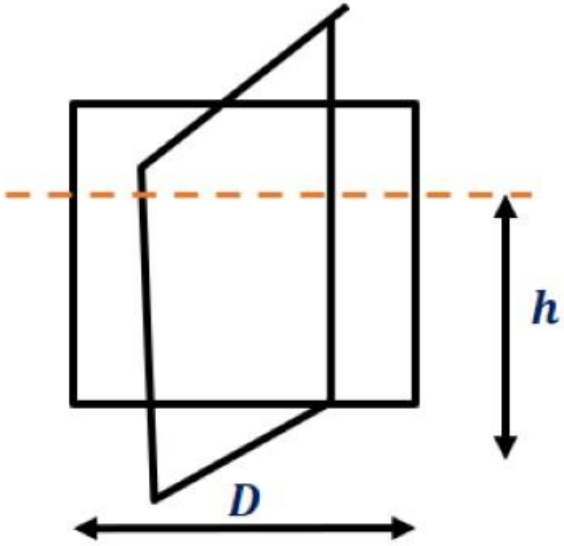
1. If vane is pushed till complete depth 'H'

$$C = \frac{T}{\pi D^2 \left(\frac{H}{2} + \frac{D}{6}\right)}$$



2. If Vane is pushed till depth 'h' only

$$C = \frac{T}{\pi D^2 \left(\frac{h}{2} + \frac{D}{12}\right)}$$



Que: A 20cm long, 10cm wide in dia was pressed into a sub marine clay at the bottom of a bore hole. The torque was applied. At failure, torque was 1000 kg-cm. The cohesion of soil will be ... ?

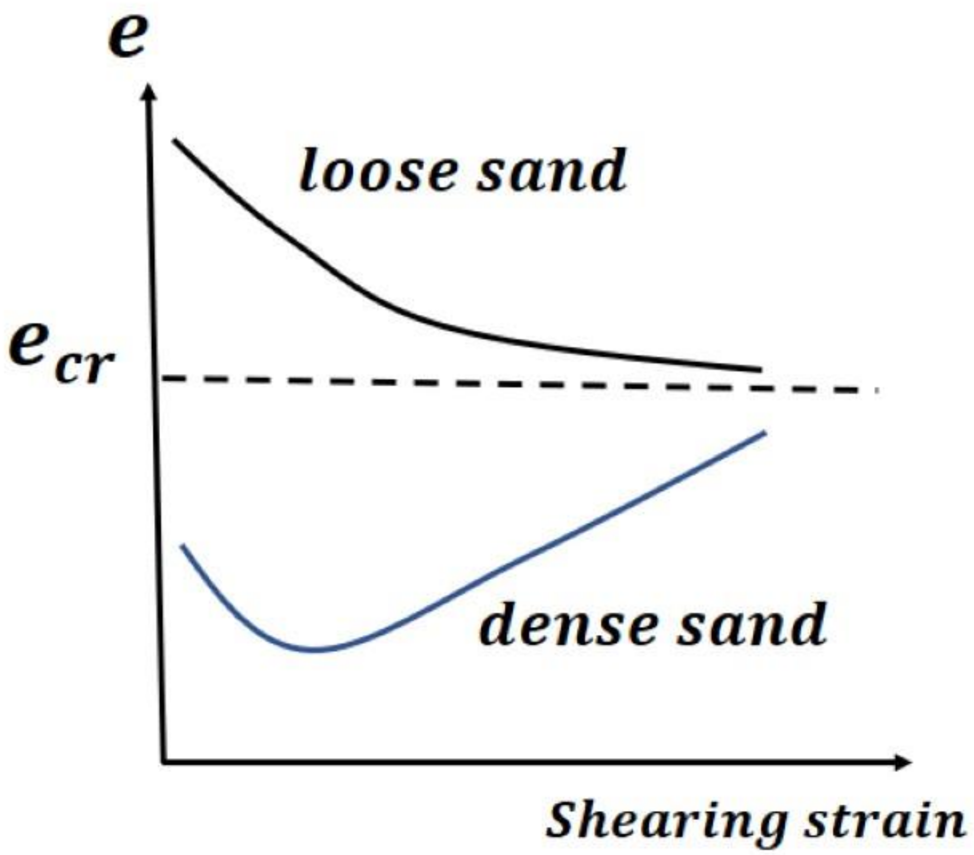
$$C = \frac{T}{\pi D^2\left(\frac{H}{2} + \frac{D}{6}\right)}$$

$$\Rightarrow C = \frac{1000}{\pi(10)^2\left(\frac{20}{2} + \frac{10}{6}\right)}$$

$$\Rightarrow C = \frac{6}{7\pi}$$

Critical Void Ratio

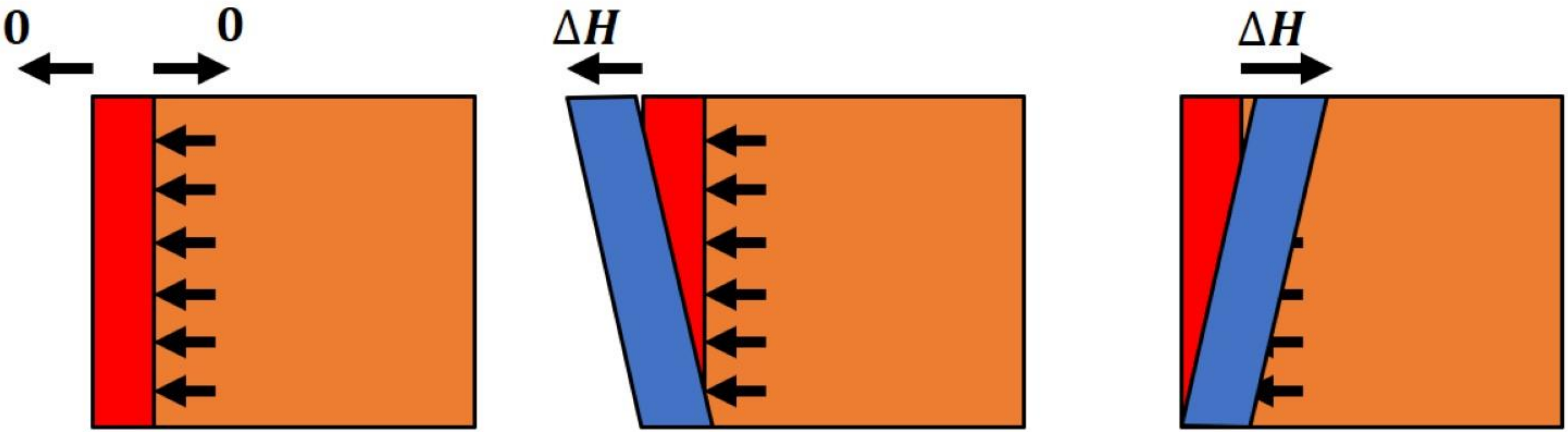
- If initial void ratio of sand is greater than critical void ratio, then sand has volume decrease tendency
- If initial void ratio of sand is less than critical void ratio, then sand initially has volume decrease tendency and further volume increase tendency
- e_{cr} depends upon effective stress
- At $\boldsymbol{e_{cr}}$ no volume change occurs



Earth Pressure

- Earth pressure is lateral force exerted by Soil on Retaining wall
- Earth Pressure is classified as
    a) Active Earth Pressure
    b) Passive Earth Pressure
    c) Earth Pressure at Rest

Earth Pressure

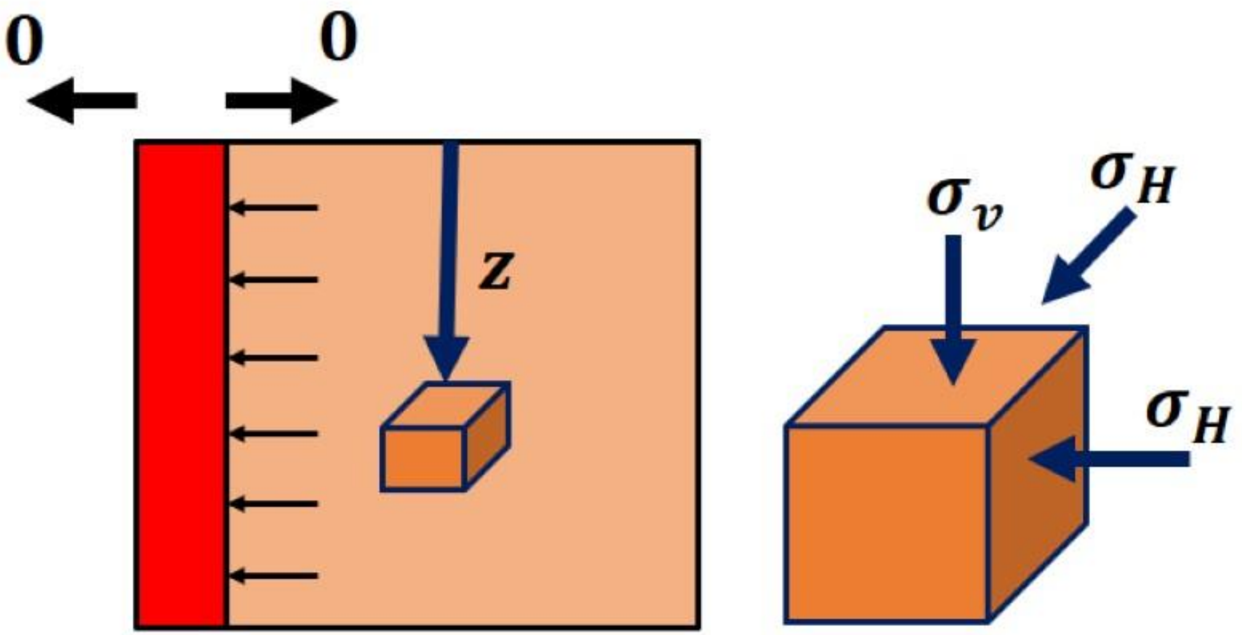


Earth Pressure at Rest

Active Earth Pressure

Passive Earth Pressure

Earth Pressure at Rest



Earth Pressure at Rest

Strain in Horizontal Direction

$$\varepsilon_h = 0$$

tension = +ve, compressive = −ve

$$\varepsilon_h = \frac{-\sigma_h}{E} - \frac{\mu(-\sigma_H)}{E} - \frac{\mu(-\sigma_v)}{E} = 0$$

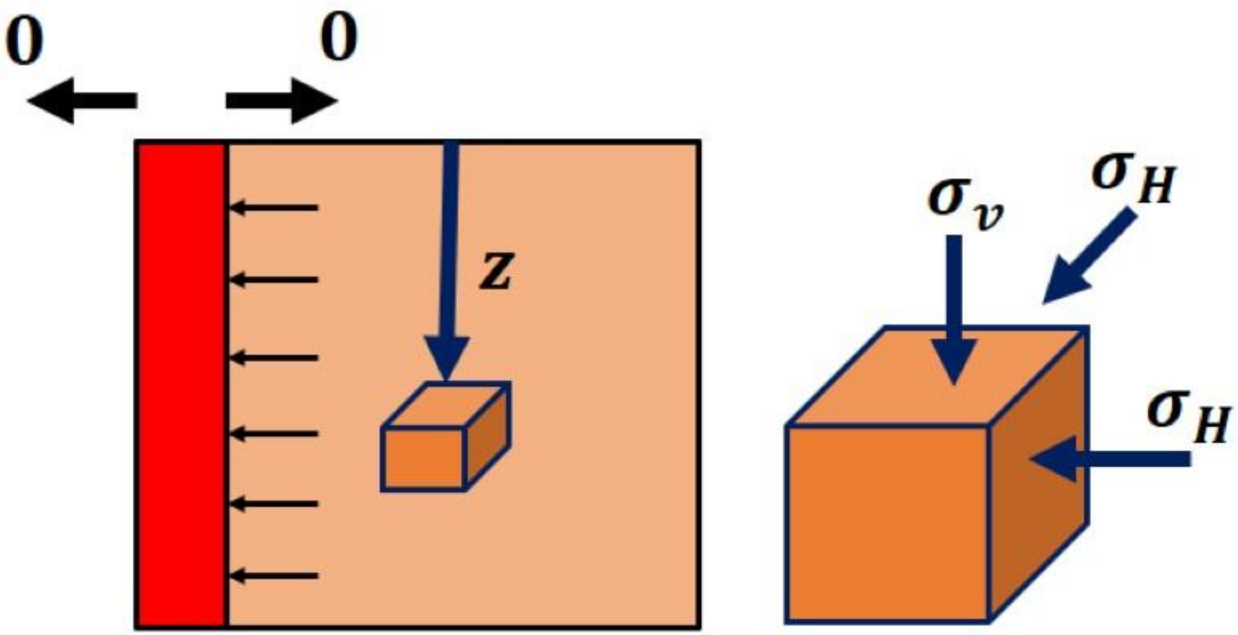
$$\Rightarrow \sigma_h = \left(\frac{\mu}{1-\mu}\right)\sigma_h$$

$$\Rightarrow \sigma_h = K_0 \sigma_h$$

$$K_0 = \left(\frac{\mu}{1-\mu}\right)$$

Above relationship is not correct because soil is not linearly elastic material

Earth Pressure at Rest



Earth Pressure at Rest

For perfectly cohesion less soil

$$\boxed{K_0 = 1 - \sin \emptyset}$$

$\emptyset = effective\ stress\ parameter$

Coefficient of Earth Pressure

- Earth Pressure Coefficient at Rest (K_o):

a. For cohesion less soil (c=0, sand)

$$K_o = 1 - \sin\phi$$

b. For Cohesive Soil/ Clay

- NC Soil

$$(K_o)_{NC} = 0.19 + 0.233 \log_{10}(I_p)$$

- For OC Clay $(K_o)_{OC} = (K_o)_{NC} \times (OCR)^{1/2}$

- Active Earth Pressure Coefficient (K_a):

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

- Passive Earth Pressure Coefficient (K_a):

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

Coefficient of Earth Pressure

- Earth Pressure at Rest

$$P_o = K_o \, \sigma_z$$

- Active Earth Pressure

$$P_a = K_a \, \sigma_z - 2c\sqrt{K_a}$$

- Passive Earth Pressure

$$P_p = K_p \, \sigma_z + 2c\sqrt{K_p}$$

$$P_a < P_o < P_p$$

- Failure plane makes an angle **45+ϕ/2** with horizontal plane in case of **Active Earth Pressure**

- Failure plane makes an angle **45-ϕ/2** with horizontal plane in case of **Passive Earth Pressure**

Earth Pressure Theory

1. Rankine Theory
2. Coulomb Theory

Rankine Theory

Assumptions of Rankine Theory

a) Soil is semi infinite

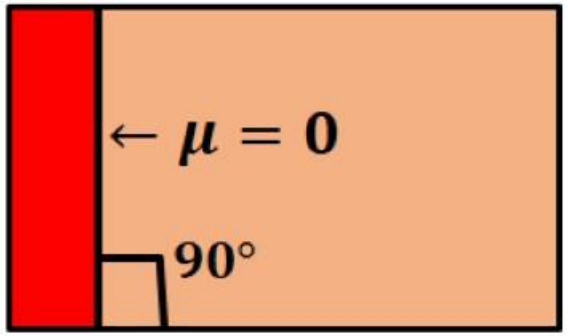
b) Soil is homogenous and isotropic

c) Backfill of soil is horizontal

d) Back of the wall is smooth and vertical

e) Soil is dry and Cohesion less (c=0)

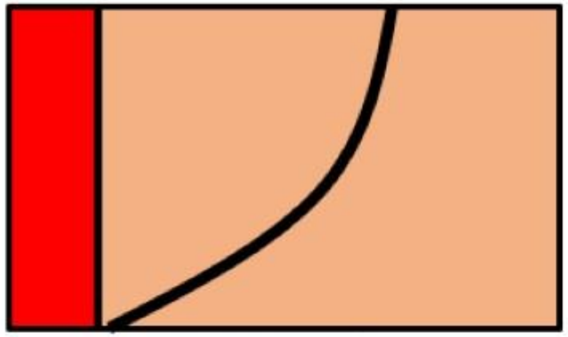
f) Soil is in the state of Plastic condition at the time of Active and Passive Earth Pressure



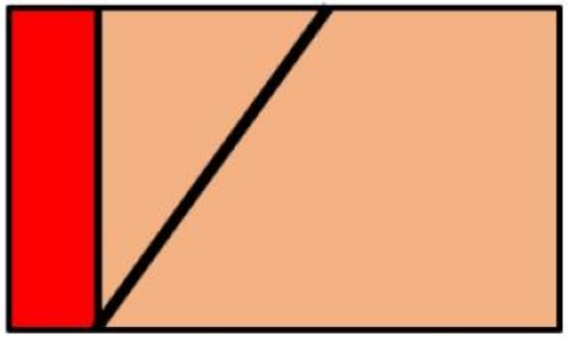
Coulomb's Theory

Assumptions of Coulomb's Theory

1. Backfill is dry, cohesionless and isotropic
2. Back of the wall can be inclined
3. Back of the wall can have friction
4. Failure plane is assumed to be a planar surface
5. Sliding Wedge is assumed to be a rigid body



Actual



Assumed

Earth Pressure

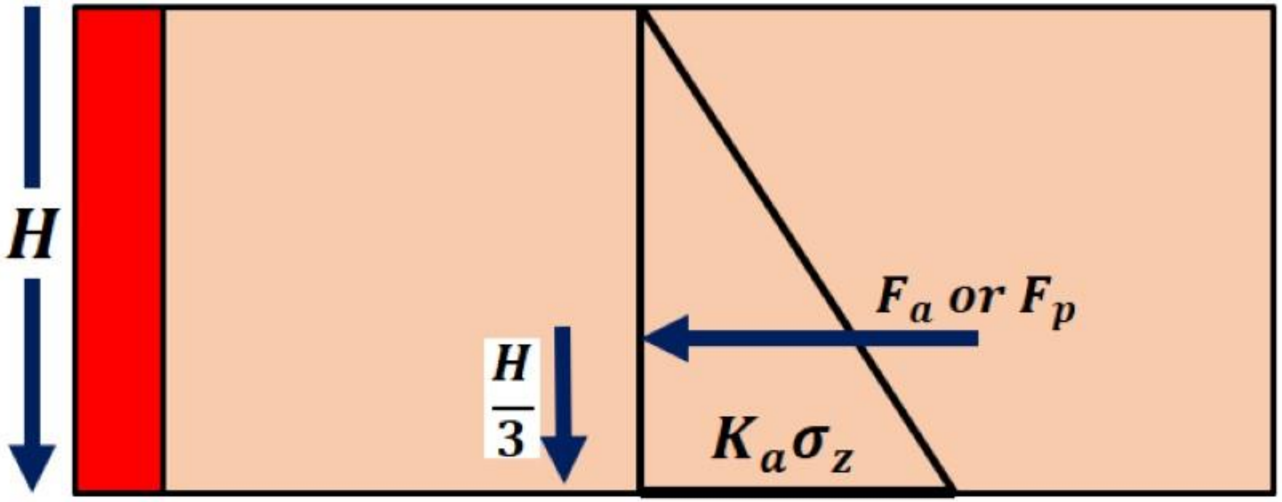
A. FOR COHESIONLESS SOIL MASS

1. Dry and Moist Soil Mass with No surcharge

$$P_a = K_a\sigma_z - 2c\sqrt{K_a}$$

$$\Rightarrow P_a = K_a\sigma_z - 0$$

$$\Rightarrow P_a = K_a\sigma_z$$



Consider unit length of Wall and find the total active pressure

Area of pressure diagram = Force

$$F_a = \frac{1}{2} \times (H) \times (K_a\sigma_z)$$

$$\Rightarrow F_a = \frac{1}{2}(K_a\gamma H^2)$$

$$\Rightarrow F_a = \frac{1}{2}(K_a\gamma H^2)$$ at H/3 from base

For Passive Earth Pressure,

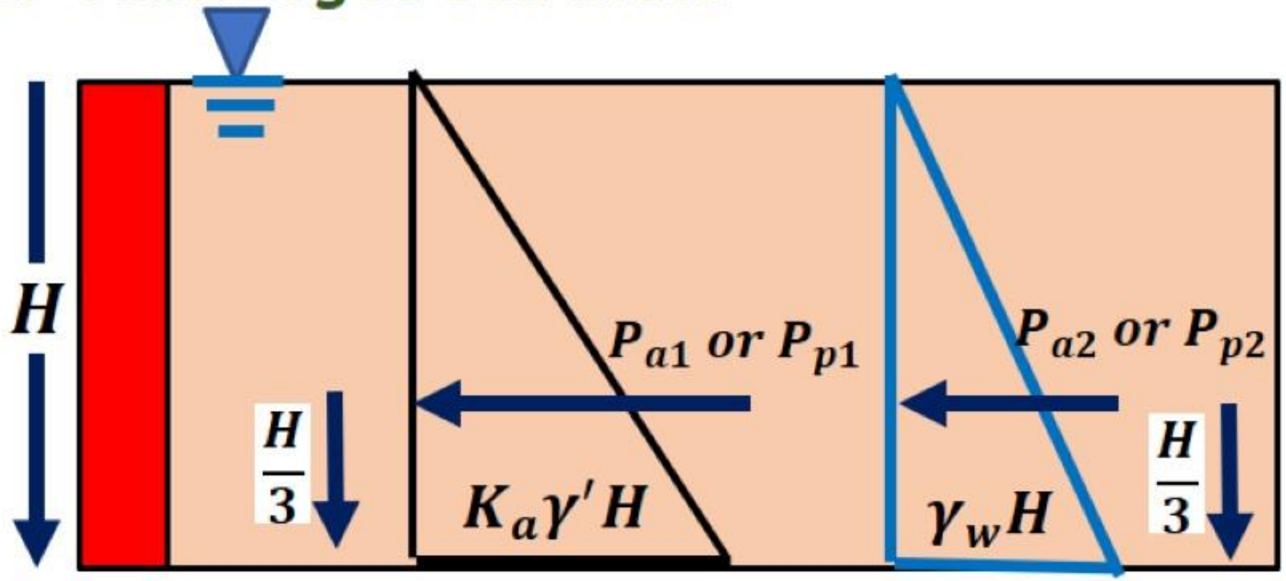
$$P_P = K_P\sigma_z + 2c\sqrt{K_P}$$

$$\Rightarrow P_P = K_P\sigma_z \Rightarrow F_p = \frac{1}{2}\left(K_p\gamma H^2\right)$$ at H/3 from base

Earth Pressure

A. FOR COHESIONLESS SOIL MASS

2. Submerged Soil mass



Total Active pressure per unit length

$$P_a = P_{a_1} + P_{a_2}$$

$$= \frac{1}{2}K_a\gamma' H \times (H \times 1) + \frac{1}{2}\gamma_w H \times H$$

$$= \frac{K_a\gamma' H^2}{2} + \frac{\gamma_w H^2}{2} @ \frac{H}{3} \textit{ from base}$$

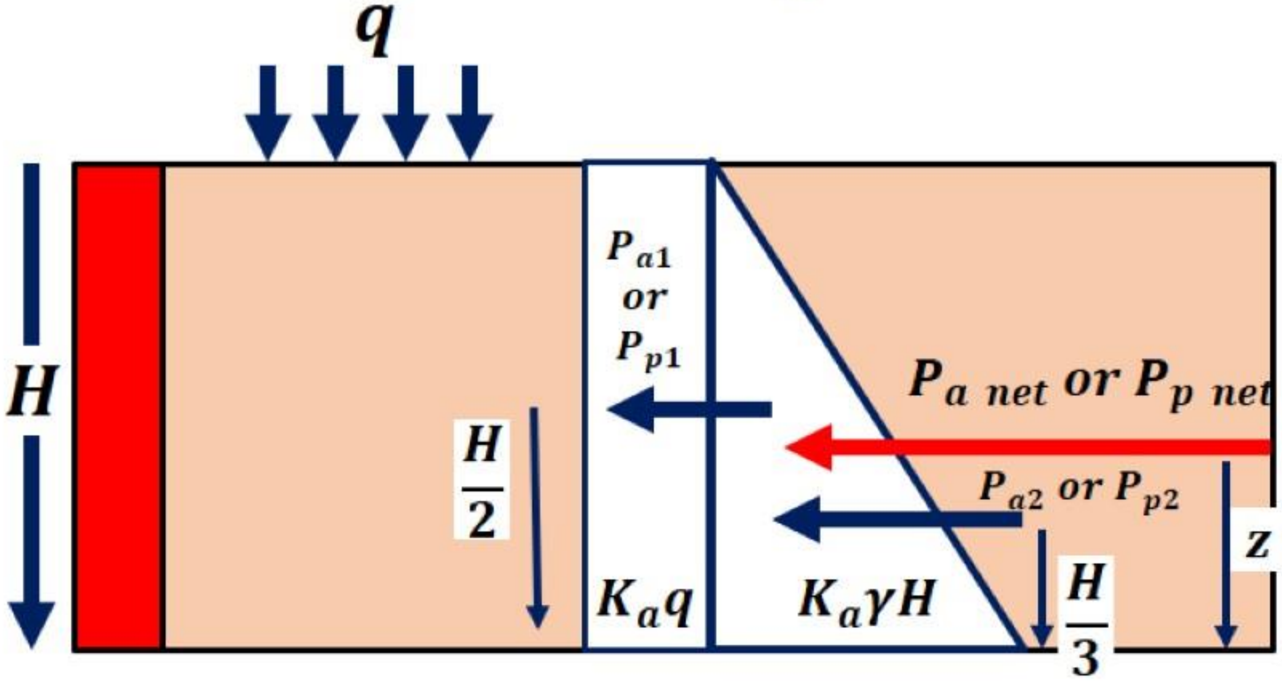
Total Passive pressure per unit length

$$= \frac{K_p\gamma' H^2}{2} + \frac{\gamma_w H^2}{2} @ \frac{H}{3} \textit{ from base}$$

Earth Pressure

A. FOR COHESIONLESS SOIL MASS

3. Soil mass with Surcharge



Total Active pressure per unit length

$$P_a = P_{a_1} + P_{a_2}$$

$$= (k_a q)(H \times 1) + \frac{k_a \gamma H^2}{2}$$

$$= k_a q H \quad + \quad \frac{k_a \gamma H^2}{2} \text{ @ z from base}$$

$\frac{H}{2}$ *from Base* $\qquad$ $\frac{H}{3}$ *from Base*

$$\Rightarrow z = \frac{P_{a_1} \times \frac{H}{2} + P_{a_2} \times \frac{H}{3}}{P_{a_1} + P_{a_2}}$$

Total Passive pressure per unit length

$$P_p = P_{p1} + P_{p_2} = k_p q H \quad + \quad \frac{k_p \gamma H^2}{2} \text{ @ z' from base}$$

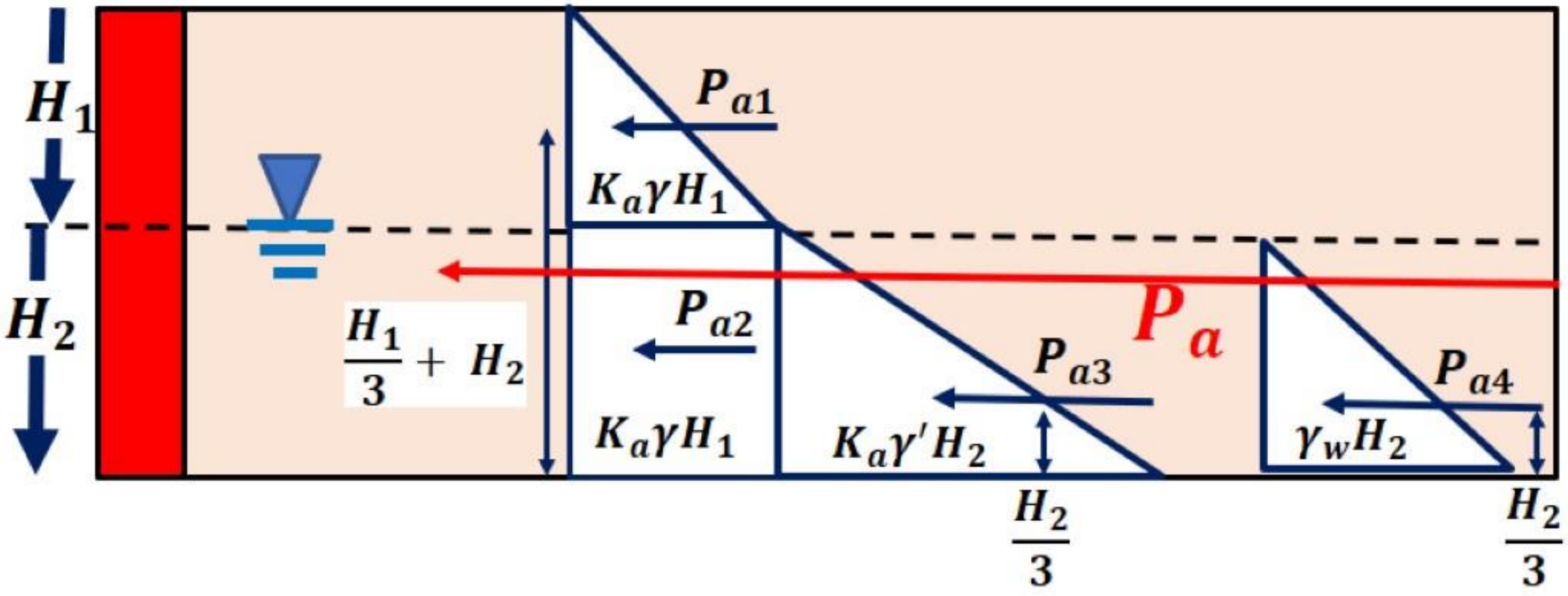
$$\Rightarrow z' = \frac{P_{p_1} \times \frac{H}{2} + P_{p_2} \times \frac{H}{3}}{P_{p_1} + P_{p_2}}$$

Earth Pressure

A. FOR COHESIONLESS SOIL MASS

$$P_a = P_{a_1} + P_{a_2} + P_{a_3} + P_{a_4}$$

4. Water Table at Depth H_1 from Ground Level



$$P_{a_1} = \frac{K_a\gamma H_1^2}{2}$$

$$P_{a_2} = K_a\gamma H_1 H_2$$

$$P_{a_3} = \frac{k_a\gamma' H_2^2}{2}$$

$$P_{a_4} = \frac{\gamma_w H_2^2}{2}$$

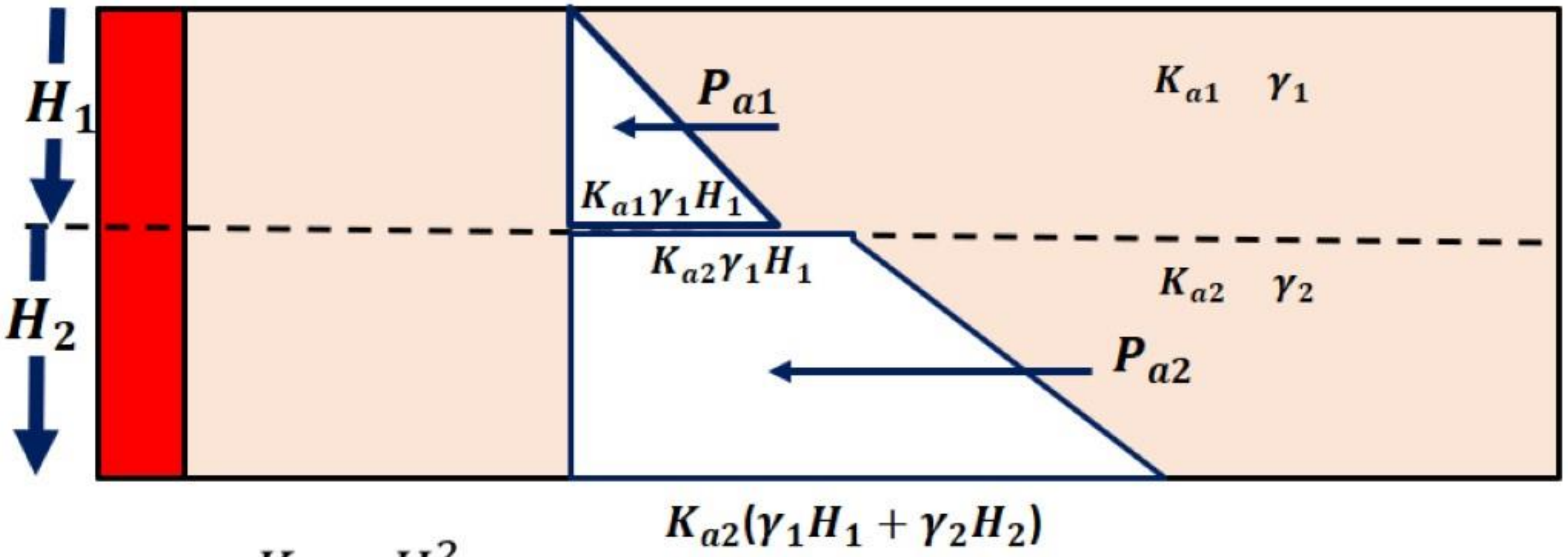
$$and\ z = \frac{P_{a_1} \times \left(H_2 + \frac{H_1}{3}\right) + P_{a_2} \times \frac{H_2}{2} + P_{a_3} \times \frac{H_3}{3} + P_{a_4} \times \frac{H_2}{3}}{P_a}$$

Earth Pressure

A. FOR COHESIONLESS SOIL MASS

5. Cohesionless soil with different type of soil

$$P_a = P_{a_1} + P_{a_2}$$



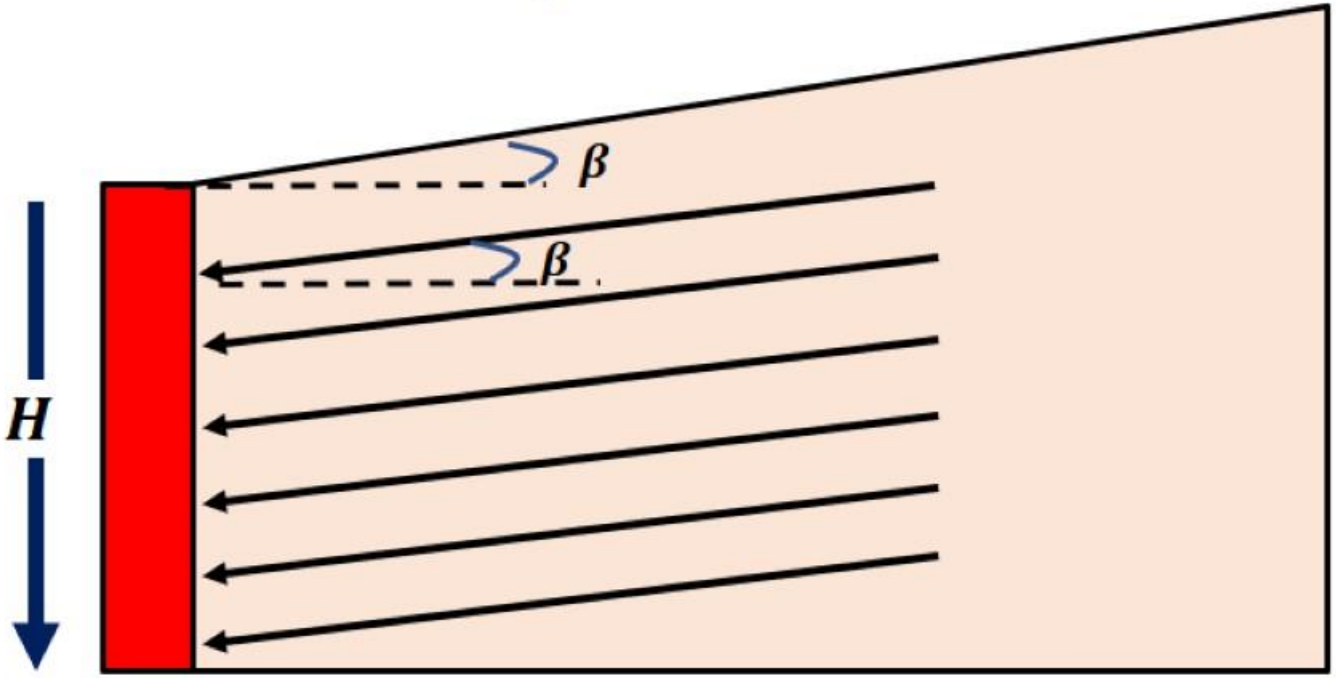
$$P_{a_1} = \frac{K_a \boldsymbol{\gamma_1} H_1^2}{2}$$

$$P_{a_2} = \frac{1}{2} H_2(\boldsymbol{K_{a2}\gamma_1 H_1} + \boldsymbol{K_{a2}(\gamma_1 H_1 + \gamma_2 H_2)})$$

Earth Pressure

A. FOR COHESIONLESS SOIL MASS

6. Inclined Backfill



Along the plane of backfill,

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

OR,

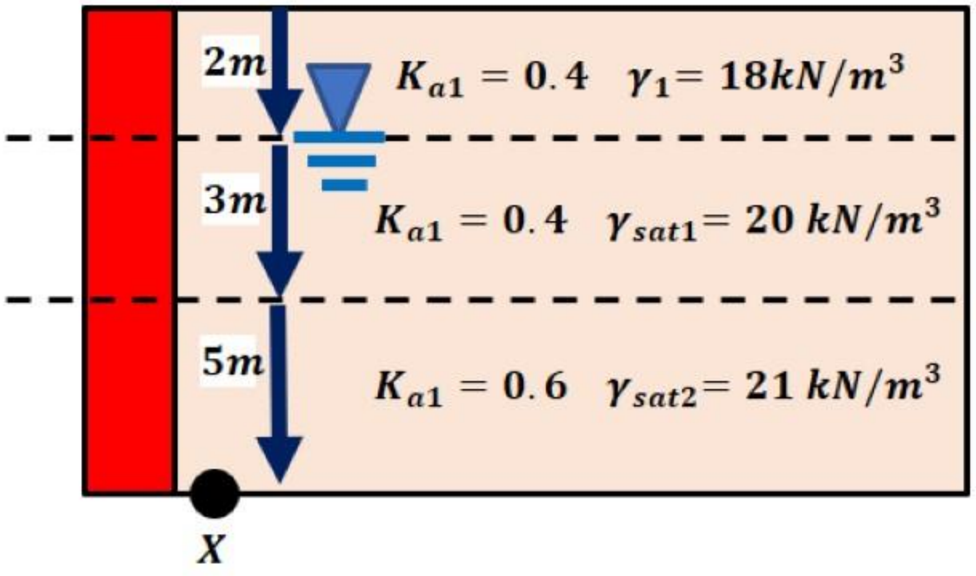
$$P_a = \frac{K_a \gamma H_1^2}{2} \cos\beta$$

$$K_a = \frac{\cos\beta - \sqrt{cos^2\beta - cos^2\emptyset}}{\cos\beta + \sqrt{cos^2\beta - cos^2\emptyset}} \times \cos\beta$$

$$K_a = \frac{\cos\beta + \sqrt{cos^2\beta - cos^2\emptyset}}{\cos\beta - \sqrt{cos^2\beta - cos^2\emptyset}} \times \cos\beta$$

It is assumed that vertical stresses and lateral pressure acting on soil element are conjugate stresses i.e. direction of one is parallel to plane on which the other one acts. So, lateral earth pressure is parallel to the backfill

Que

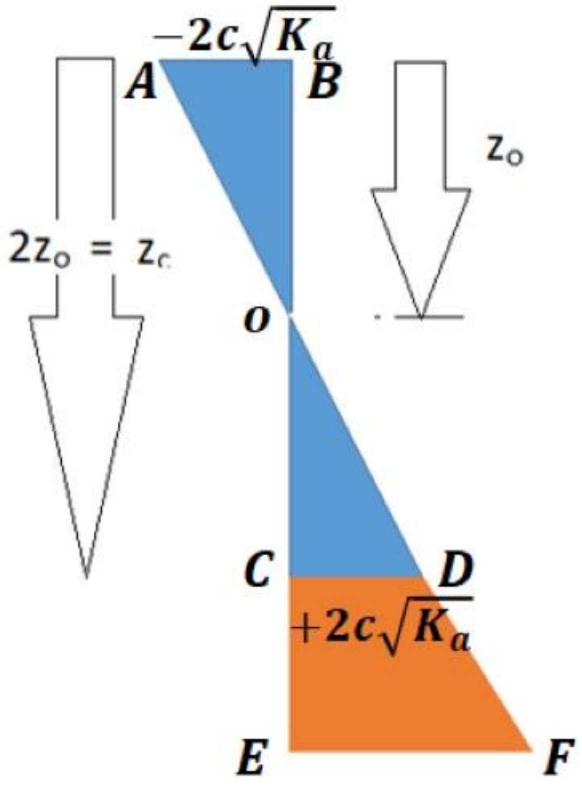


Find active earth pressure at X

Earth Pressure

B. FOR COHESIVE SOIL MASS

- In case of cohesive soil, there is a tendency of development of tension in the soil and it reaches upto a depth of z_0
- As there is no net effective pressure upto a depth of $2z_0$, we can make unbraced cut in clayey soil upto a depth of $2z_0$



$$P_a = K_a\sigma_z - 2c\sqrt{K_a}$$

$$at\ z = 0, \quad P_a = -2c\sqrt{K_a}$$

$$at\ P_a = 0, \qquad z = z_0,$$

$$\Rightarrow 0 = K_a\sigma_{z0} - 2c\sqrt{K_a}$$

$$\boxed{\Rightarrow z_0 = \frac{2c}{\gamma\sqrt{Ka}}}$$

Critical depth of unsupported excavation $\Rightarrow 2z_0 = \frac{4c}{\gamma\sqrt{Ka}}$

Earth Pressure

B. FOR COHESIVE SOIL MASS

1. *When Tension Cracks are not developed*

$$\boxed{P_a = K_a\,\sigma_z - 2c\sqrt{K_a}}$$

2. *When Tension Cracks are Developed*

$$\boxed{P_a = K_a\,\sigma_z - 2c\sqrt{K_a} + 2c^2}$$

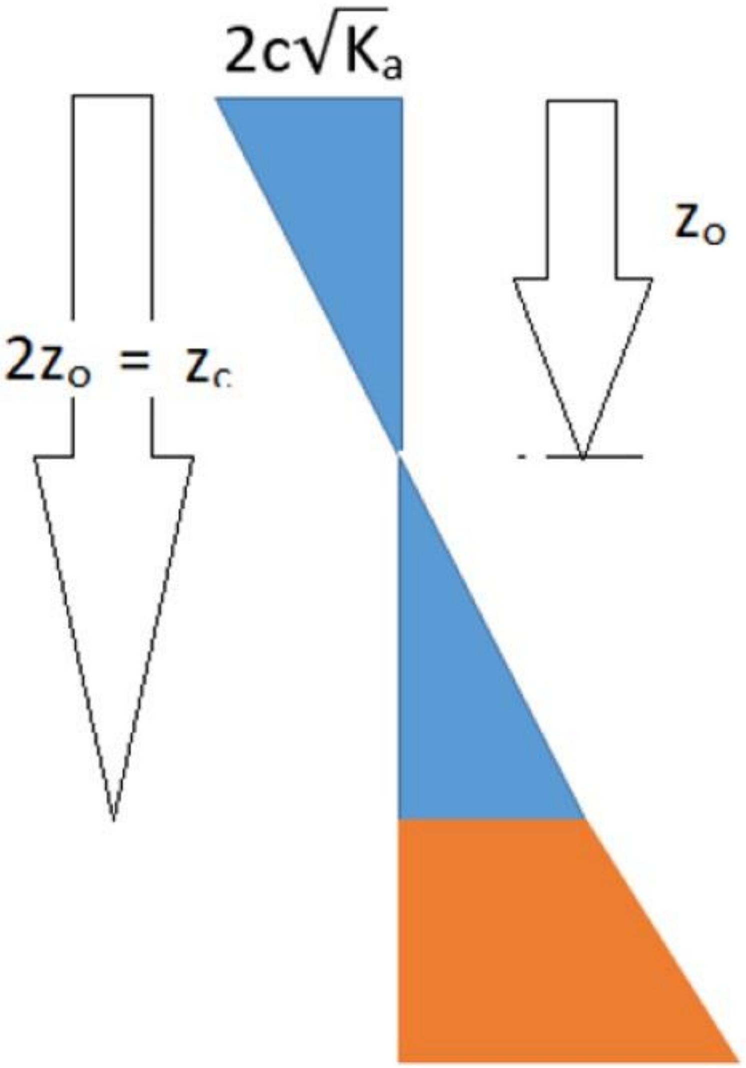
Earth Pressure

Critical Height for Unsupported Vertical cutoff:

$$z_c = 2z_o = 2 \times \frac{2c}{\gamma \sqrt{K_a}}$$

$$\boxed{z_c = \frac{4c}{\gamma\sqrt{Ka}}}$$

$$\boxed{z_c = \frac{4c \tan \alpha}{\gamma} = \frac{4c}{\gamma}\tan\left(45 + \frac{\phi}{2}\right)}$$



Que. A 6m high retaining wall is to support a soil with unit weight $\gamma = 17.4\ kN/m^3$, $\emptyset = 26°$ and $c = 14.36\ kN/m^2$. Determine the rankine active force per unit length of wall before tensile crack occurs. Also find the critical depth.

We know that

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

$$\Rightarrow K_a = \frac{1 - \sin 26°}{1 + \sin 26°}$$

$$\Rightarrow K_a = 0.39$$

$$P_a = K_a \sigma_z - 2c\sqrt{K_a}$$

$$\Rightarrow F_a = \frac{K_a \gamma H^2}{2} - 2cH\sqrt{K_a}$$

$$\Rightarrow F_a = \frac{0.39 \times 17.4 \times 6^2}{2} - 2 \times 14.36 \times 6 \times \sqrt{0.39}$$

$$\Rightarrow F_a = 14.47 kN/m$$

$$z_c = \frac{4c}{\gamma\sqrt{Ka}} \qquad \Rightarrow z_c = \frac{4 \times 14.6}{17.4 \times \sqrt{0.39}} \qquad \Rightarrow z_c = 5.28m$$

Que. Find the active Earth Pressure at a depth of 3.6m in a sandy soil with angle of internal friction as $30°$ and having a density of 1.9 g/cc.

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

$$\Rightarrow K_a = \frac{1 - \sin 30°}{1 + \sin 30°}$$

$$\Rightarrow K_a = \frac{1}{3}$$

$$\gamma = 1.9 \times \frac{10^{-3}}{10^{-6}} \times 9.81$$

$$\Rightarrow \gamma = 18.63 kN/m^3$$

$$P_a = K_a \sigma_z - 2c\sqrt{K_a}$$

$$\Rightarrow F_a = \frac{K_a \gamma H^2}{2} - 2cH\sqrt{K_a}$$

$$\Rightarrow F_a = \frac{\frac{1}{3} \times 18.63 \times 3.6^2}{2} - 0$$

$$\Rightarrow F_a = 40.24\ kN$$

Que. A 6m height retaining wall is to support a soil of unit weight 17.4 kN/m3. $\emptyset =$ 26°and c=14.36kN/m2. Determine the Rankine Active Force per unit length of wall before tension crack occurs and also the critical depth.

SOIL MECHANICS

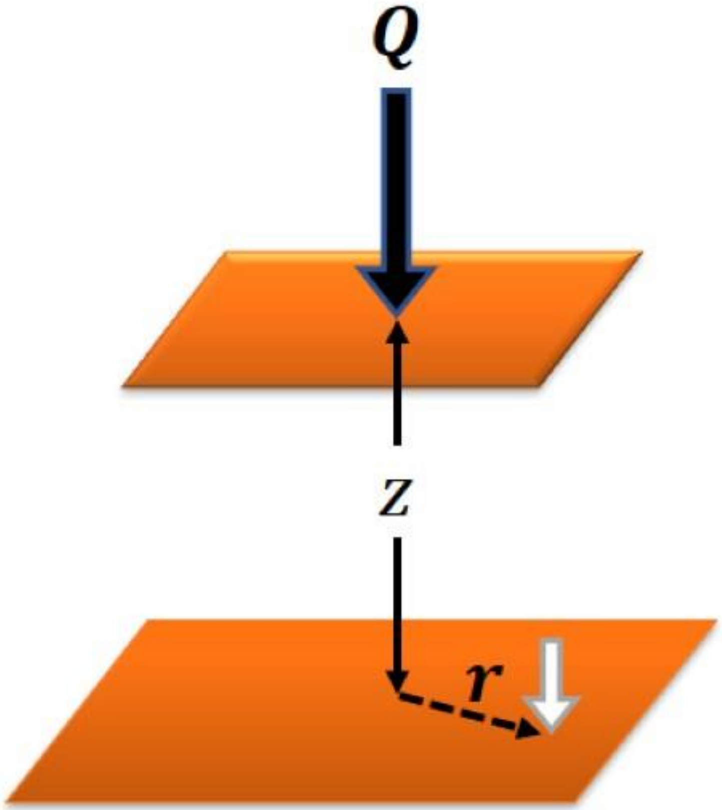
Civil Engineering by Sandeep Jyani

Vertical Stresses

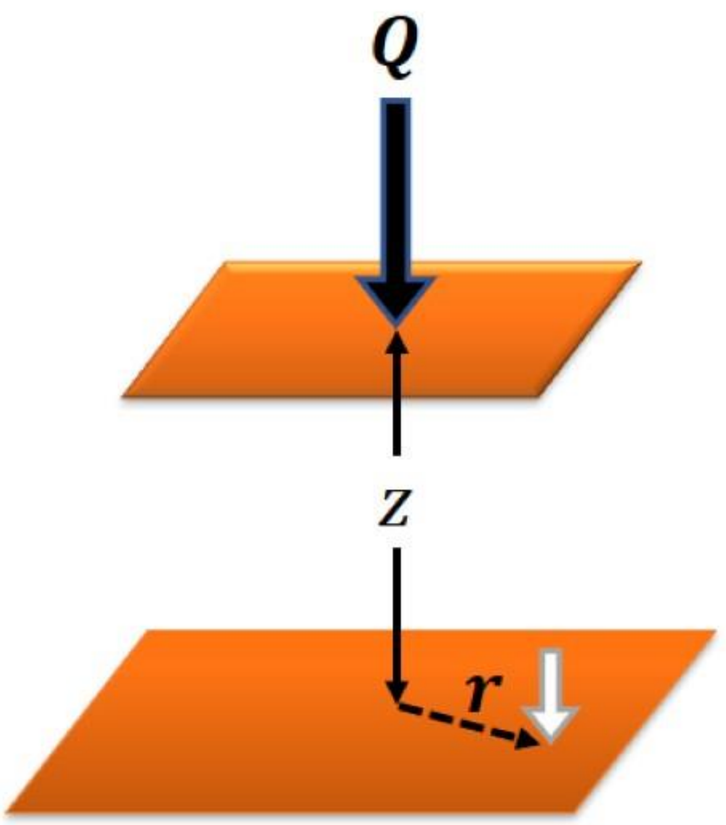
Vertical Stresses

Vertical Stresses due to Concentrated Load can be explained by

1. Boussinesq Equation
2. Westergaard's Equation



Vertical Stresses



1. **Boussinesq Equation for concentrated Load**

Boussinesq gave the theoretical solutions for the stress distribution in an elastic medium subjected to concentrated load on its surface

It is not suited for sedimentary soil

Assumptions:

a) Soil mass is elastic

b) Soil is homogenous and isotropic

c) Soil is semi infinite

d) Soil is weightless and unstressed before the application of load

$$\sigma_z = \frac{3}{2\pi}\frac{\boldsymbol{Q}}{\boldsymbol{z^2}}\left[\frac{1}{\left(1+\left(\frac{r}{z}\right)^2\right)}\right]^{\frac{5}{2}}$$

$$\Rightarrow \sigma_z = K_B\,\frac{\boldsymbol{Q}}{\boldsymbol{z^2}}$$

K_B = ***Boussinesq's Influence Factor***

$$K_B = \frac{3}{2\pi}\left[\frac{1}{\left(1+\left(\frac{r}{z}\right)^2\right)}\right]^{\frac{5}{2}}$$

Vertical Stresses

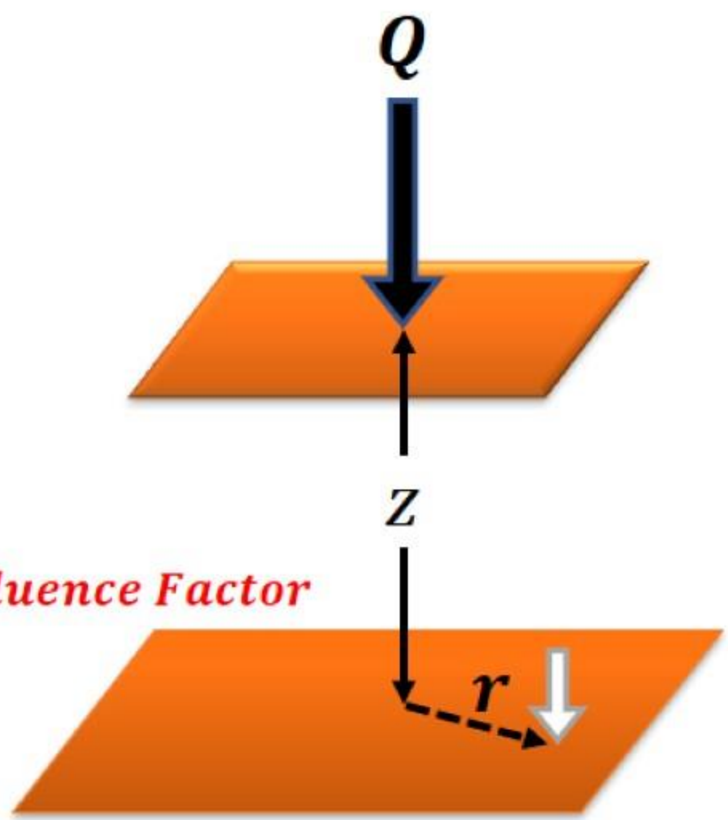
1. Boussinesq Equation for concentrated Load

$$\sigma_z = \frac{3}{2\pi}\frac{Q}{z^2}\left[\frac{1}{\left(1+\left(\frac{r}{z}\right)^2\right)}\right]^{\frac{5}{2}}$$

$$\Rightarrow \sigma_z = K_B\frac{Q}{z^2}$$

$K_B = \textit{Boussinesq's Influence Factor}$

$$K_B = \frac{3}{2\pi}\left[\frac{1}{\left(1+\left(\frac{r}{z}\right)^2\right)}\right]^{\frac{5}{2}}$$



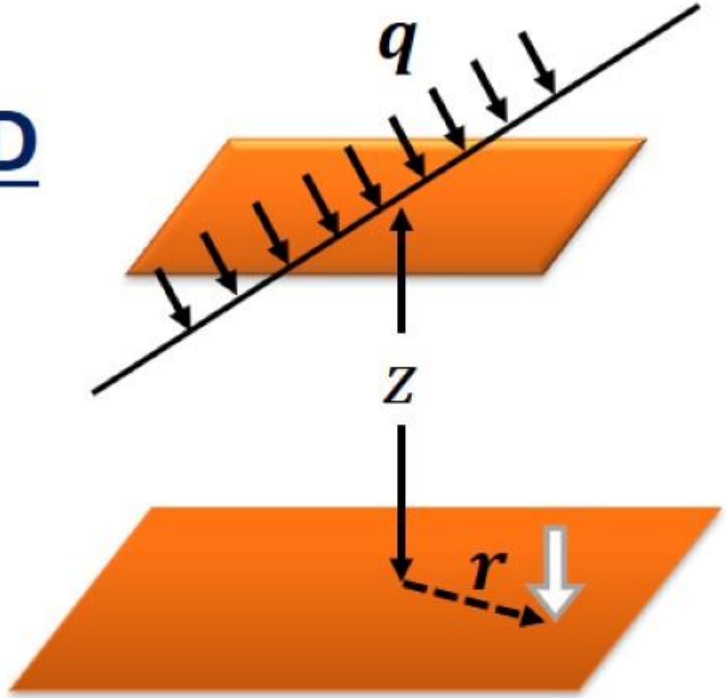
$if\ r = 0,$

$$\boxed{\sigma_z = 0.4775\frac{Q}{z^2}}$$

Vertical Stresses

1. Boussinesq Equation for LINE LOAD

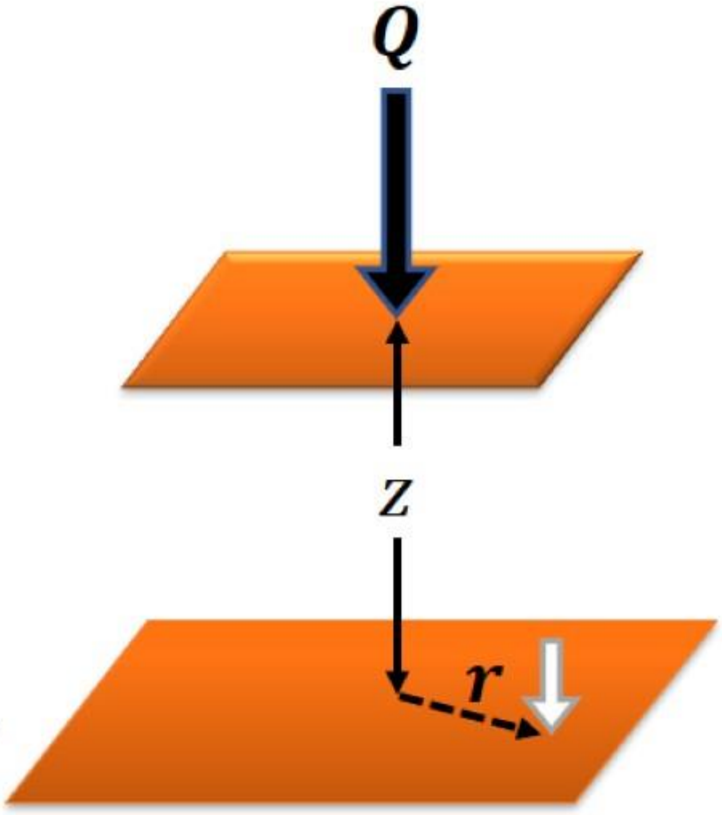
$$\sigma_z = \frac{2}{\pi}\frac{\mathbf{Q}}{\mathbf{z}}\left[\frac{1}{\left(1+\left(\frac{r}{z}\right)^2\right)}\right]^2$$



$if\ r = \mathbf{0},$

$$\boxed{\sigma_z = \frac{\mathbf{2Q}}{\pi\mathbf{z}}}$$

Vertical Stresses



2. Westergaard's Equation

Westergaard gave the theoretical solutions for the vertical stress distribution due to concentrated load on its surface

Assumptions:

a) Soil is **anisotropic**

b) Soil mass is elastic

c) Soil is semi infinite

d) Soil mass is divided into horizontal sheets of negligible thickness, closely spaced, and infinite rigidity in horizontal direction that allows only vertical movement and prevents soil mass as a whole from undergoing lateral strain $\mu = 0$.

$$\boxed{\sigma_z = \frac{1}{\pi}\frac{Q}{z^2}\left[\frac{1}{\left(1+2\left(\frac{r}{z}\right)^2\right)}\right]^{\frac{3}{2}}}$$

$$\Rightarrow \sigma_z = K_W \frac{Q}{z^2}$$

$K_B = \text{Westergaard's Influence Factor}$

$$K_B = \frac{1}{\pi}\left[\frac{1}{\left(1+2\left(\frac{r}{z}\right)^2\right)}\right]^{\frac{3}{2}}$$

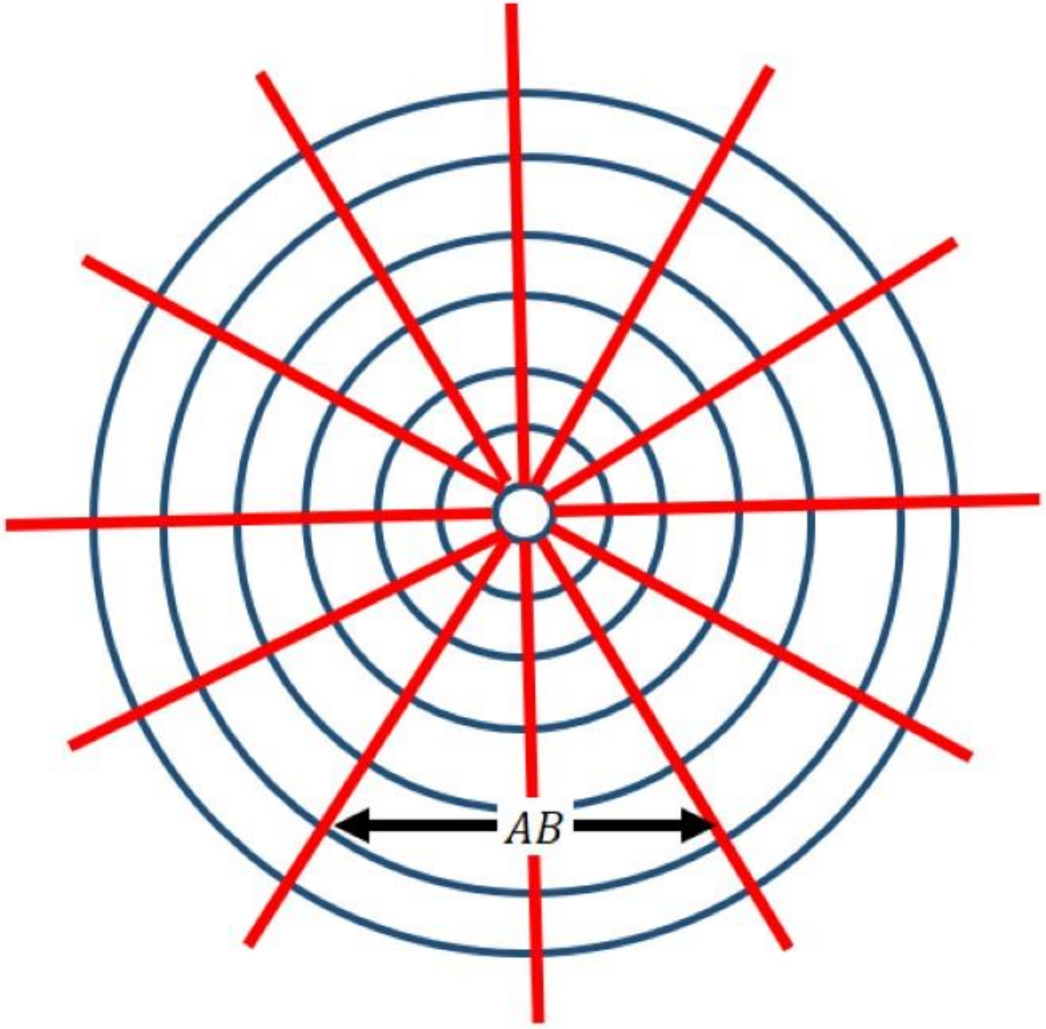
$if\ r = 0,$

$$\boxed{\sigma_z = 0.318\frac{Q}{z^2}}$$

Vertical Stresses

3. Newmark's Influence Chart

- Newmark developed Influence Chart to compute vertical stress
- This chart is based on Boussinesq Equations
- It consists of
  - m=no. of concentric circles
  - n= no of radial lines



Vertical Stresses

3. <u>Newmark's Influence Chart</u>

- To find out vertical stress at any point below or outside loaded area, plan of loaded area is drawn such that ***depth z*** at which stress is being calculated equals ***length AB*** as shown
- Further the plan is placed over the chart such that the point at which stress is to be calculated coincides the chart
- Then count the number of influenced area (N) covered by plan area

$$\sigma_z = \frac{1}{m \times n} \times q \times N$$

- m=no. of concentric circles
- n= no of radial lines
- q= intensity of load
- N= equivalent no. of areas