



# PIE Tech

**POLLACHI INSTITUTE OF ENGINEERING AND TECHNOLOGY**

(Approved by **AICTE** and Affiliated to **Anna University**)

*sky is the limit*

**Department of Civil Engineering**

**Regulation 2021**

**II Year – IV Semester**

**CE3404-Soil Mechanics**

## UNIT - I

### SOIL CLASSIFICATION AND COMPACTION

Soil:

The term soil has various meaning depending upon the general Professional field in which it is being considered.

Soil Includes different materials:

- a) Boulders
- b) Sands
- c) Gravels
- d) Clays
- e) Silts

[Soil ~~the~~ is the unaggregated (or) unconsolidated deposits of mineral (or) organic particles ~~are~~ covering large portion of the earth's crust.]

Soil Mechanics: [Karl von Terzaghi in 1948]

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

Foundation Engineering [Taylor 1948]

It is a branch of Civil Engineering, which is associated with the design, construction, maintenance, and renovation of footings, foundation walls, pile foundations, caissons, and all other structural members which form the foundations of buildings and other engineering structures.

## History of development of soil Mechanics:

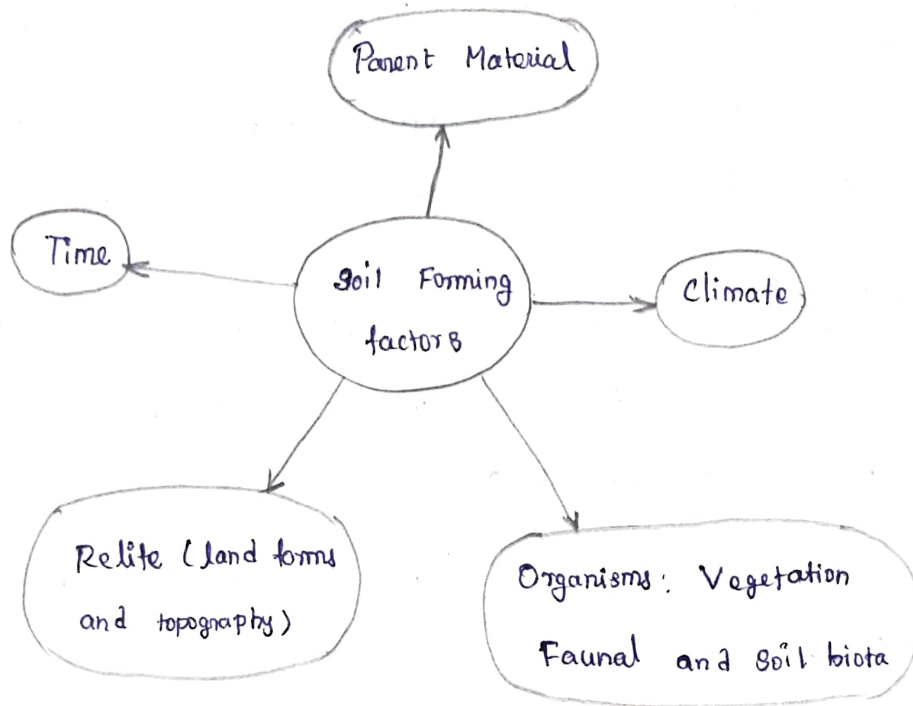
- ✓ The Hanging Gardens of Babylon: were supported by huge retaining walls, the construction of which should have required some knowledge through empirical of earth pressures.
- ✓ The Roman Engineer in the first Century B.C. Manasari and Viswakarma in India, wrote books on Construction Science during the medieval period.
- ✓ The Leaning Tower of Pisa, built between 1174 and 1350 AD is a glaring example of a lack of sufficient knowledge of the behaviour of compressible in those days.
- ✓ Coulomb a French engineer published his wedge theory of earth pressure in 1776. Which is the first major contribution to the scientific study of soil behaviour.
- ✓ 'Poncelet, Culmann' and 'Rebhann' were the other men who extended the work of Coulomb.
- ✓ 'Darcy and Stokes' were notable for their laws for the flow of water through soil and settlement of a solid particle in liquid medium respectively.
- ✓ 'Rankine' gave his history theory of earth pressure in 1857. He did not consider cohesion although he knew of its existence.

- ✓ 'Boussinesq' in 1885 gave his theory of stress distribution in an elastic medium under a point load on the surface.
- ✓ Mohr in 1871 gave a graphical representation of the state of stress of a point called as Mohr's Circle of stress.
- ✓ 'Atterberg' a Swedish soil scientist gave in 1911, the concept of Consistency limits for a soil. This made possible the understanding of the physical properties of soil.
- ✓ The Swedish method of slices for slope stability analysis was developed by Fellenius in 1926.
- ✓ 'Prandtl' gave his theory of plastic equilibrium in 1920.
- ✓ Later on R.R. Proctor and A. Casagrande and a host of others were responsible for the development of the subject as a full fledged discipline.

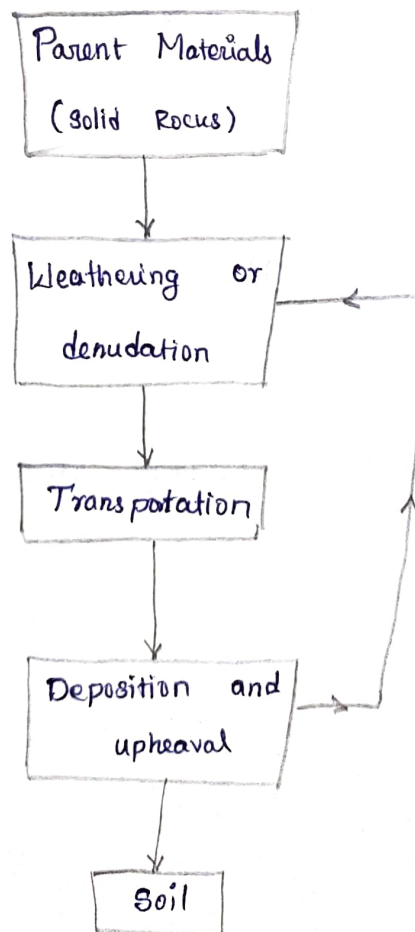
#### Applications of Soil Mechanics:

- i) Foundations
- ii) Underground and Earth-Retaining Structures
- iii) Pavement Design
- iv) Excavations
- v) Embankments
- vi) Dams etc.

## Formations of Soil.



## Soil Forming Process:



## Types of Soil:

### a) Based on Grain Size:

- i) Cobbles - size 64mm to 256mm
- ii) Gravels - size 4.75 to 76.2mm
- iii) Clay - less than 0.002mm
- iv) Silt - size 0.075 to 0.002mm
- v) Sand - less than 4.75mm

### b) Based on Origin:

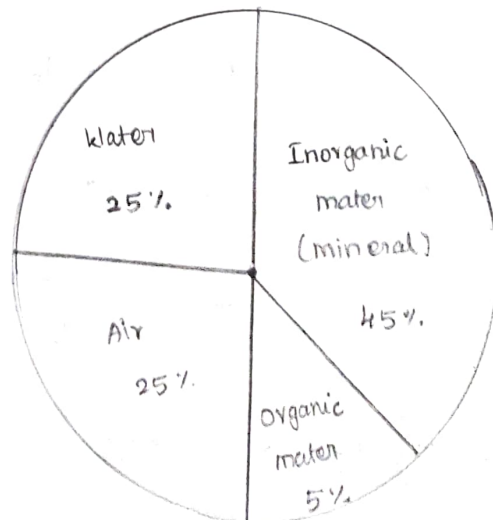
- i) Residual soils
- ii) Transported soils

### c) Based on Used in Practice:

- i) Bentonite
- ii) Kaolin
- iii) Boulder clay
- iv) Varved clays
- v) Shale
- vi) Marl
- vii) Peat
- viii) Loam.

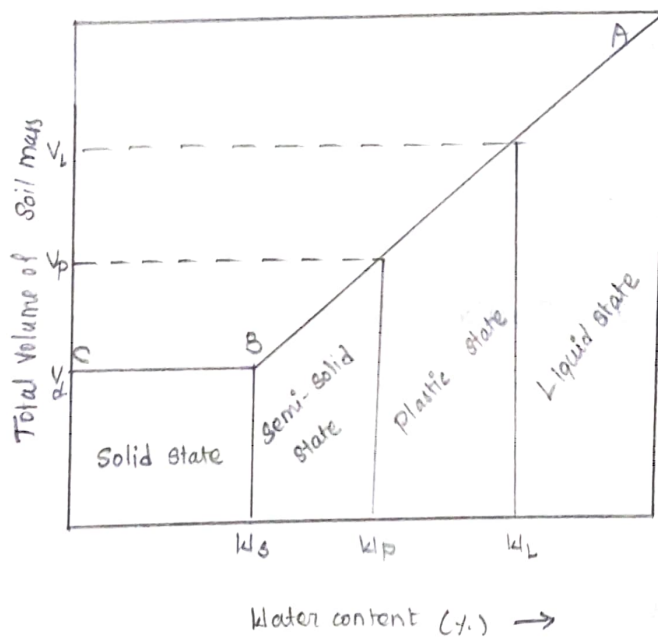
## Composition of Soil.

- i) Inorganic Matter - 45%
- ii) Organic Matter - 5%
- iii) Soil Water - 25%
- iv) Soil Air - 25%



## Index Properties:

- i) Water content
  - ii) Specific gravity
  - iii) Particle size distribution
  - iv) Consistency limits
  - v) In-situ density and density Index (Field density)
- Laboratory Experiments



a) plasticity Index ( $I_p$ )

$$I_p = w_L - w_P$$

b) Consistency Index ( $I_c$ )

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

c) Liquidity Index ( $I_L$ )

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

## Clay Mineralogy:

Clay mineralogy is the science dealing with the structure of clay minerals on microscopic molecular and atomic scale. It also includes the study of the mineralogical composition and electrical properties of the clay particles.

- i) Kaolin - Serpentine [kaolinite, halloysite, lizardite, chrysotile]
- ii) Pyrophyllite - Talc
- iii) Mica [Illite, Glauconite, Celadonite]
- iv) Vermiculite
- v) Smectite
- vi) Chlorite
- vii) Sepiolite
- viii) Interstratified clay minerals
- ix) Allophane and Imogolite.

## Classifications of Soils:

For general engineering purposes soils may be classified by the following systems.

1. Particle size classifications
2. Textural classifications
3. Highway Research Board (HRB) classification
4. Indian Standard (IS) classification
5. Unified Soil (US) classification

- i) U.S. Bureau of Soil and public road administration (PRA)  
System of United States

|      |         |           |      |        |        |             |               |
|------|---------|-----------|------|--------|--------|-------------|---------------|
|      | 0.005mm | 0.05      | 0.40 | 0.25   | 0.50   | 1.0         | 2.0mm         |
| Clay |         | Very Fine | Fine | Medium | Coarse |             |               |
| Silt |         | Sand      |      |        |        | Fine Gravel | Coarse Gravel |

- ii) International Soil Classification, proposed at the International Soil Congress at Washington D.C. in 1927.

|            |        |        |       |       |              |      |      |     |     |     |        |
|------------|--------|--------|-------|-------|--------------|------|------|-----|-----|-----|--------|
|            | 0.0002 | 0.0006 | 0.002 | 0.006 | 0.02         | 0.05 | 0.1  | 0.2 | 0.5 | 1.0 | 2.0mm  |
| Ultra clay | F      | C      | F     | C     | F            | C    | F    | M   | C   | Ve  |        |
|            | Clay   |        | Silt  |       | Mo (magilla) |      | Sand |     |     |     | Gravel |

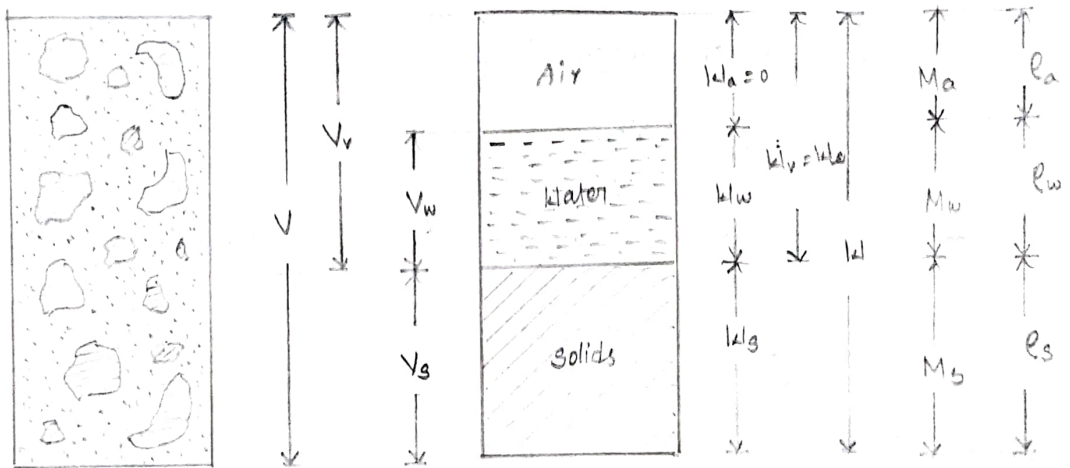
- iii) The M.I.T. classifications proposed by Prof. Wilbooy of Massachusetts Institute of Technology as a Specifications of the Bureau of Soil Classifications.

|      |        |       |      |      |     |     |        |
|------|--------|-------|------|------|-----|-----|--------|
|      | 0.0002 | 0.006 | 0.02 | 0.06 | 0.2 | 0.6 | 2.0mm  |
| Clay | F      | M     | C    | F    | M   | C   |        |
|      | Silt   |       |      | Sand |     |     | Gravel |

iv)

|         |       |       |   |        |    |    |        |          |
|---------|-------|-------|---|--------|----|----|--------|----------|
| Clay    | Silt  | F     | M | C      | F  | C  | Cobble | Boulders |
| 0.002mm | 0.075 | 0.425 | 2 | 4.75   | 20 | 80 | 300mm  |          |
| Sand    |       |       |   | Gravel |    |    |        |          |

### Phase Relationships:



## Basic Definitions and Relationships:

i) Water Content : (w)

$$W = \frac{W_w}{W_d} \times 100 \quad \text{or} \quad \frac{M_w}{M_d} \times 100$$

ii) Density of Soil:

$$\rho = \frac{M}{V} \quad \text{in } \frac{\text{kg}}{\text{m}^3} \text{ or } \frac{\text{g}}{\text{cm}^3}$$

a) Bulk density (or) Moist density ( $\rho$ )

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

b) Dry density ( $\rho_d$ )

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

iii) Unit weight of solids ( $\gamma_s$ )

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

a) Dry unit weight ( $\gamma_d$ )

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

b) Saturated Unit weight ( $\gamma_{sat}$ )

$$\gamma_{sat} = \frac{W_{sat}}{V}$$

c) Submerged Unit weight (or) Buoyant Unit weight ( $\gamma_{sub}$ )

$$\gamma_{sub} = \frac{W_{sub}}{V}$$

iv) Specific gravity :

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

v) Voids ratio

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

vii) Porosity ( $n$ )

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

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

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

vii) Degree of Saturation (or) % of Saturation ( $S$ )

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

Common formula's:

$$e \cdot s = w \cdot \eta \quad (\text{or}) \quad e = \frac{w \cdot \eta}{s}$$

For fully saturated  $s = 1$  ;  $w = w_{\text{sat}}$

$$e = w_{\text{sat}} \cdot \eta$$

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

$$n_a = n \cdot a_c$$

$$\checkmark \quad \gamma_d = \frac{\eta \cdot \gamma_w}{1+e}$$

$$\checkmark \quad \gamma_d = \frac{\gamma}{1+w}$$

$$\checkmark \quad \gamma_d = (1-n) \eta \cdot \gamma_w$$

$$\gamma_d = \frac{\eta \cdot \gamma_w}{1 + w_{\text{sat}} \cdot \eta}$$

$$\checkmark \quad \gamma_{\text{sat}} = \frac{(\eta + e) \gamma_w}{1+e}$$

$$\gamma_d = \frac{(1-n_a) \eta \gamma_w}{1 + w \eta}$$

$$\gamma_{\text{sat}} = \eta \cdot \gamma_w (1-n) + \gamma_w \cdot n$$

$$\checkmark \quad \gamma = \frac{(\eta + es) \gamma_w}{1+e}$$

$$\gamma = \gamma_d + s(\gamma_{\text{sat}} - \gamma_d)$$

$$\checkmark \quad \gamma' = \frac{(\eta - 1) \gamma_w}{1+e}$$

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

Densities:

$$\rho_d = \frac{\eta \cdot \rho_w}{1+e}$$

$$\rho' = \rho_d - (1-n) \rho_w$$

$$\rho_{\text{sat}} = \frac{(\eta + e) \gamma_w}{1+e}$$

$$\rho_d = (1-n) \eta \cdot \rho_w$$

$$\rho_d = \frac{\eta \rho_w}{1 + \frac{w \eta}{s}}$$

$$\rho_{\text{sat}} = \eta \cdot \rho_w (1-n) + \rho_w \cdot n$$

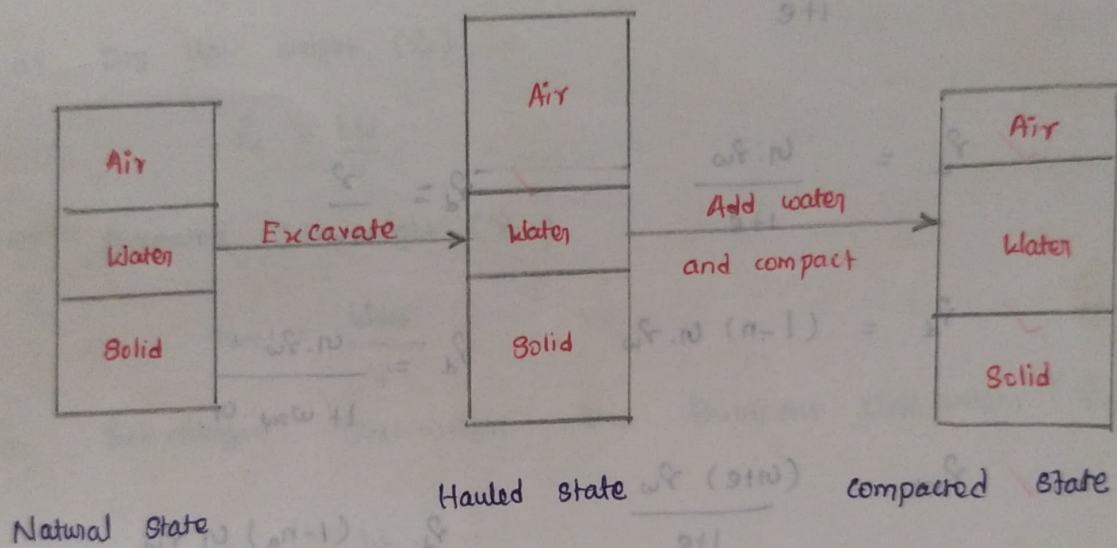
$$\rho = \frac{(\eta + es) \rho_w}{1+e}$$

$$\rho' = \frac{(\eta - 1) \rho_w}{1+e}$$

$$\rho_d = \frac{\rho}{1+w}$$

## Compaction :

Compaction is a process by which the soil particles are artificially arranged and packed together into a closer state of contact of contact by mechanical means in order to decrease the porosity (or voids ratio) of the soil and thus increase its dry density.



objectives of Compaction :

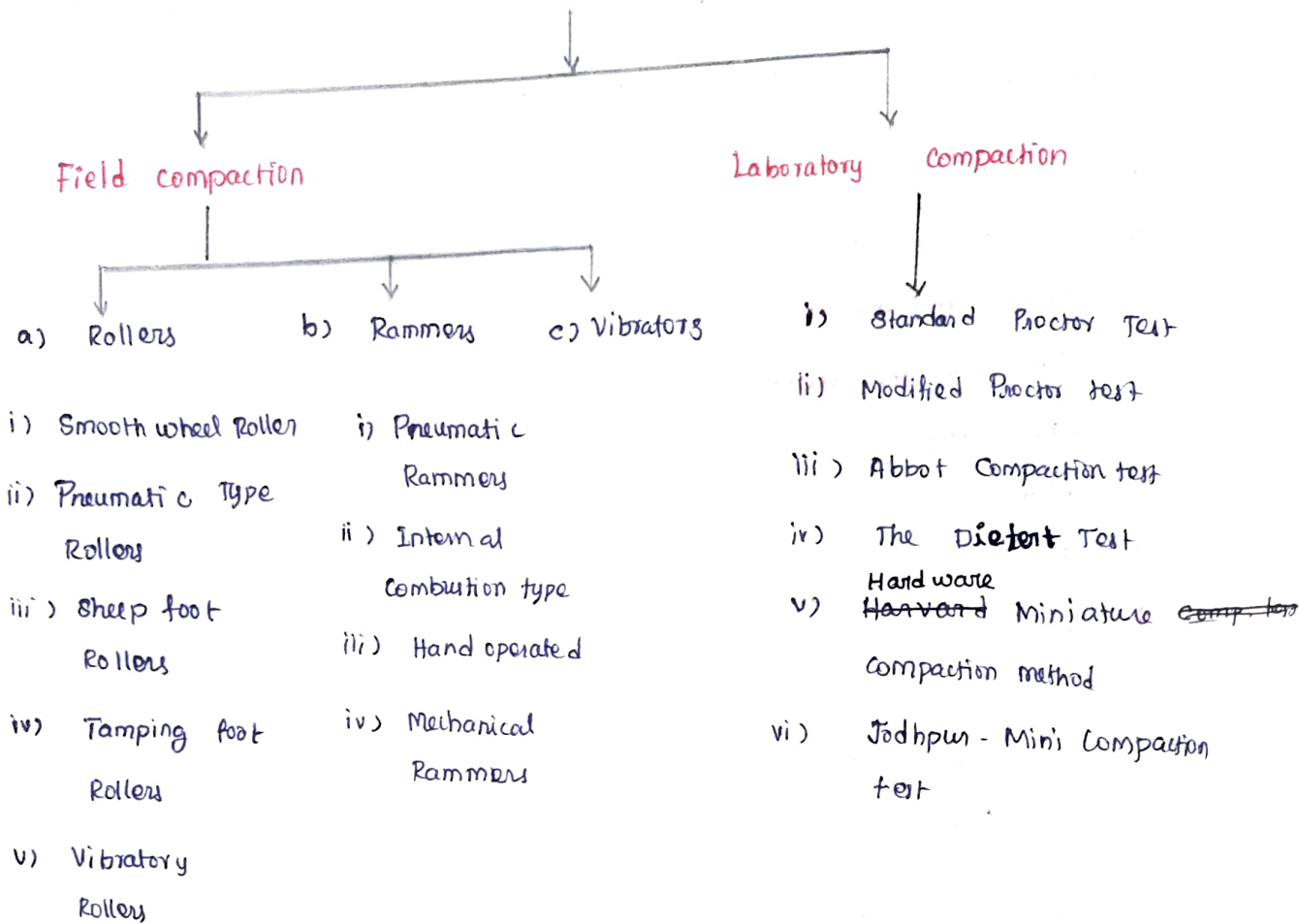
i) Increase shear strength and therefore bearing capacity

ii) Increase stiffness and therefore reduced future Settlement

iii) Decrease voids ratio, compressibility and permeability thus reducing potential frost heave.

iv) Change in swelling and shrinkage characteristics.

## Compaction methods



## Factors Influencing Compaction:

- i) Water content
- ii) Amount of compaction
- iii) Type of soil compaction
- iv) Method of soil compaction
- v) No. of Roller passes
- vi) Contact pressure
- vii) Speed of Rolling
- viii) Length of foot
- ix) shape of foot
- x) Spacing of foot
- xi) Addition of Admixtures

1. In an undisturbed soil sampling 903 g of wet soil occupies a volume 425 cc. On oven drying the weight of soil reduced to 802 g. If the Specific gravity of soil is 2.64. Determine.

- Bulk Unit weight
- Dry unit weight
- Saturated unit weight
- Submerged unit weight
- Water content
- Voids ratio
- Porosity
- Degree of saturation

$$M_w = 903 - 802 = 101 \text{ g}$$

$$M_d = 802 \text{ g}$$

i) Water content ( $w$ )

$$w = \frac{M_w}{M_d} = \frac{101}{802} \approx 0.13$$

$$\therefore w = 13\%$$

ii) Bulk Unit weight ( $\gamma$ )

$$\gamma = \gamma_w \times e$$

$$= 9.81 \times 2.12$$

$$\therefore \gamma = 20.84 \text{ kN/m}^3$$

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

$$= \frac{903}{425}$$

$$\therefore e = 2.12 \text{ g/cm}^3$$

iii) Dry Unit weight ( $\gamma_d$ )

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

$$= \frac{20.84}{1+0.13}$$

$$\gamma_d = 18.45 \text{ kN/m}^3$$

iv) Voids Ratio (e)

$$e = \frac{w \cdot \gamma_w}{\gamma_d} - 1 = \frac{2.64 \times 9.81}{18.45} - 1$$

$$e = 0.40$$

v) Saturated Unit Weight ( $\gamma_{sat}$ )

$$\gamma_{sat} = \left( \frac{w+e}{1+e} \right) \times \gamma_w$$
$$= \left( \frac{2.64 + 0.40}{1 + 0.40} \right) \times 9.81$$

$$\gamma_{sat} = 21.80 \text{ kN/m}^3$$

vi) Submerged Unit Weight ( $\gamma'$ )

$$\gamma' = \gamma_{sat} - \gamma_w$$
$$= 21.80 - 9.81$$

$$\gamma' = 11.49 \text{ kN/m}^3$$

vii) Porosity (n):

$$n = \frac{e}{1+e}$$
$$= \frac{0.40}{1+0.40}$$

$$n = 0.29$$

viii) Degree of Saturation (S):

$$S = \frac{w \cdot \gamma_w}{e}$$

$$= \frac{0.13 \times 9.81}{0.40} = 0.86 \times 100$$

$$S = 86\%$$

2. A Soil mass in it's natural state is partially Saturated having a water content of 17.5% and voids ratio of 0.87. Determine the degree of Saturation total unit weight, dry unit weight. What is the weight of water required to make a mass of  $10 \text{ m}^3$  volume to get Saturated assume  $G_s = 2.69$ .

$$w = 0.175$$

$$e = 0.87$$

$$G_s = 2.69$$

i) Degree of Saturation ( $S$ )

$$S = \frac{w \cdot G_s}{e} = \frac{0.175 \times 2.69}{0.87} \Rightarrow 0.541$$

$$S = 54.10 \%$$

ii) Total unit weight ( $\gamma_t$ ):

$$\gamma = \frac{G_s (1+w) \gamma_w}{1+e} = \frac{2.69 (1+0.175) 9.81}{1+0.87}$$

$$\gamma = 16.58 \text{ kN/m}^3$$

iii) Dry unit weight ( $\gamma_d$ ):

$$\gamma_d = \frac{G_s \cdot \gamma_w}{1+e} = \frac{2.69 \times 9.81}{1+0.87}$$

$$\gamma_d = 14.11 \text{ kN/m}^3$$

$$W = \gamma \times V = 16.58 \times 10 \Rightarrow 165.8 \text{ kN}$$

$$W_s = \gamma_d \times V = 14.11 \times 10 \Rightarrow 141.1 \text{ kN}$$

$$\begin{aligned} \text{Weight of water} &= W - W_s \\ &= 165.8 - 141.1 \end{aligned}$$

$$W_w = 24.7 \text{ kN}$$

3. A partially Saturated Soil from an earth fill has a natural water content of 22% and a bulk unit weight of  $19 \text{ kN/m}^3$ . Assuming the  $G_r$  of soil is 2.65. Compute the degree of saturation and voids ratio. If subsequently the soil gets saturated, determine the dry density, buoyant unit weight and saturated unit weight.

$$w = 0.22$$

$$\gamma = 19 \text{ kN/m}^3$$

$$G_r = 2.65$$

(i) Dry density ( $\gamma_d$ )

$$\gamma_d = \frac{\gamma}{1+w} = \frac{19}{1+0.22} \Rightarrow 15.57 \text{ kN/m}^3$$

(ii) Voids ratio ( $e$ ):

$$e = \frac{G_r \cdot \gamma_w}{\gamma_d} - 1 = \frac{2.65 \times 9.81}{15.57} - 1$$

$$e = 0.67$$

(iii) Degree of Saturation ( $S$ ):

$$S = \frac{G_r \cdot w}{e} = \frac{2.65 \times 0.22}{0.67} \Rightarrow 0.87$$

$$S = 87\%$$

(iv) Saturated Unit weight  $\gamma_{sat}$ :

$$\gamma_{sat} = \frac{G_r + e}{1+e} \gamma_w = \frac{2.65 + 0.67}{1+0.67} \times 9.81$$

$$\gamma_{sat} = 19.50 \text{ kN/m}^3$$

v) Buoyant Unit weight ( $\gamma'$ ):

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

$$= 19.50 - 9.81$$

$$\gamma' = 9.69 \text{ kN/m}^3$$

$$S_1 = \frac{w \cdot \gamma_s}{e_1 \gamma_w}$$

$$= \frac{0.11 \times 26.3}{0.78 \times 9.81}$$

$$S_1 = 0.378$$

$$S_2 = \frac{w \cdot \gamma_s}{e_2 \gamma_w}$$

$$= \frac{0.11 \times 26.3}{0.429 \times 9.81}$$

$$S_2 = 0.687$$

4. A sample of clay soil has liquid limit 62% and its plasticity index of 32%. What is the consistency of the soil if the soil in its natural state has a water content of 34%. Calculate the shrinkage limit and degree of saturation of the voids of the sample at the shrinkage limit is 0.70. Assume  $G = 2.70$ .

$$W_L = 0.62$$

$$I_P = 0.32$$

$$w = 0.34$$

$$e = 0.70$$

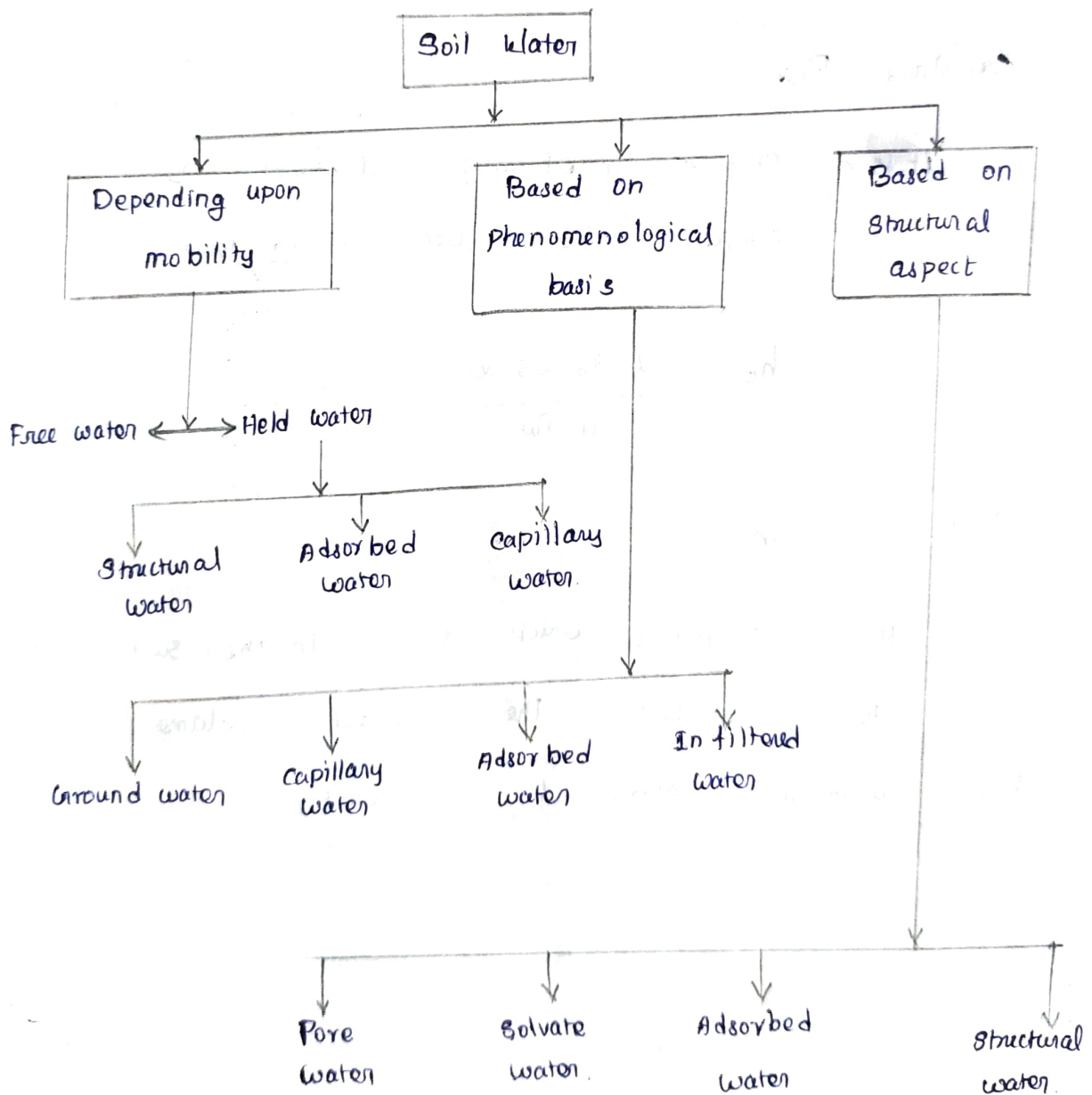
$$G = 2.70$$

## EFFECTIVE STRESS AND PERMEABILITY

Soil water:

The water present in the voids of soil is called 'Soil water'

Classification of Soil water:



## Capillary Water:

Capillary water is the water which is held in a soil mass due to capillary forces.

Factors influencing capillary forces:

- i) Surface tension of water
- ii) Pressure in water in relation to atmosphere pressure
- iii) Configuration of soil pores voids.

## Capillary Rise:

It is a phenomenon of raising of liquid surface in a small tube compared to the adjacent liquid level.

$$h_c = \frac{4 T_s \cos \alpha}{d \cdot \rho_w}$$

## Surface Tension:

It is the property which exists in the surface film of water tending to contract the contained volume into a form having a minimum superficial area possible.

## Static Pressure in Water:

It is defined as the pressure exerted a static fluid dependent on the depth of the fluid. the density of the fluid and the acceleration due to gravity.

- i) The total mass
- ii) The surface area
- iii) The shape of the liquid

Uses (or) Importance:

- ✓ Fire hoses
- ✓ Showers
- ✓ Hydraulic pumps
- ✓ Pressure washers
- ✓ wells
- ✓ Dams etc.

Calculation of Water Pressure:

$$P = \rho \times g \times h$$

$P$  - Pressure in  $N/m^2$

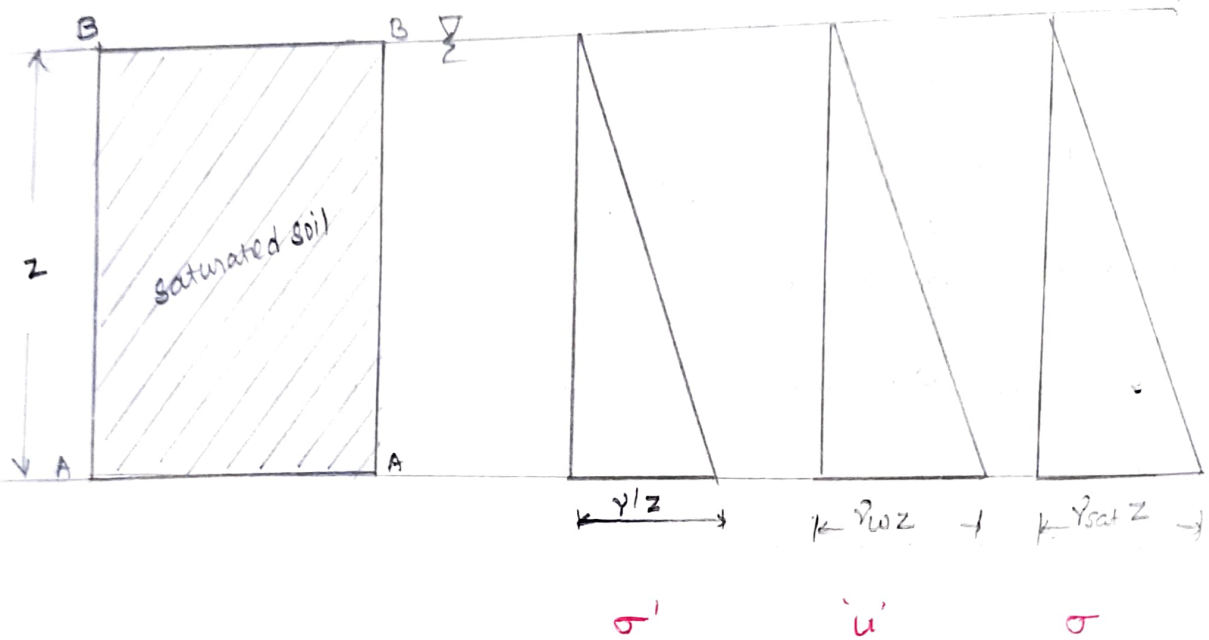
$\rho$  - density  $kg/m^3$

$g$  - sp. gravity

$h$  - Height of fluid

# Effective stress concepts in Soils:

a) Submerged soil mass with water Table at the surface:



At 'A' A

$$\text{Total stress } \sigma = \gamma_{sat} \cdot z$$

$$\text{Pore pressure } u = \gamma_w \cdot z$$

$$\text{Effective stress } \sigma' = \sigma - u$$

$$= (\gamma_{sat} - \gamma_w) z = \gamma' \cdot z$$

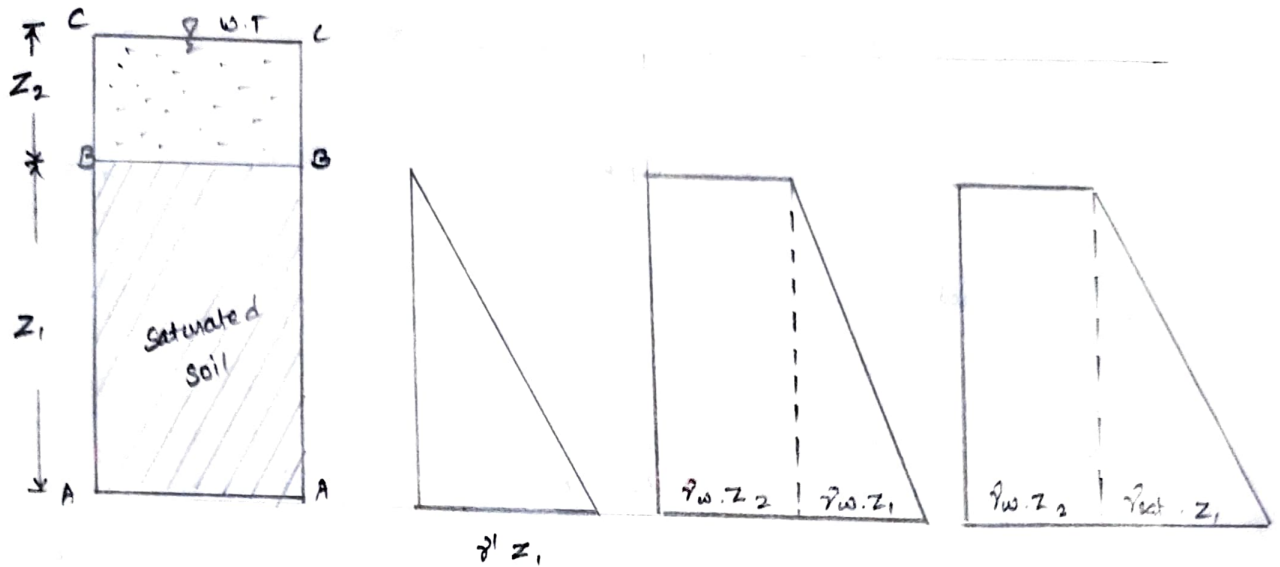
At 'B' B

$$\sigma = 0$$

$$u = 0$$

$$\sigma' = 0$$

b) Submerged Soil mass with Water Table above the surface:



At 'A'

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

$$u = \gamma_w Z_2 + \gamma_w Z_1$$

$$\sigma' = \sigma - u = (\gamma_w Z_2 + \gamma_{sat} Z_1) - \gamma_w Z_2 - \gamma_w Z_1$$

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

$$\sigma' = \gamma' Z_1$$

At 'B'

$$\sigma = \gamma_w Z_2$$

$$u = \gamma_w Z_2$$

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

At 'C-C'

$$\sigma = 0$$

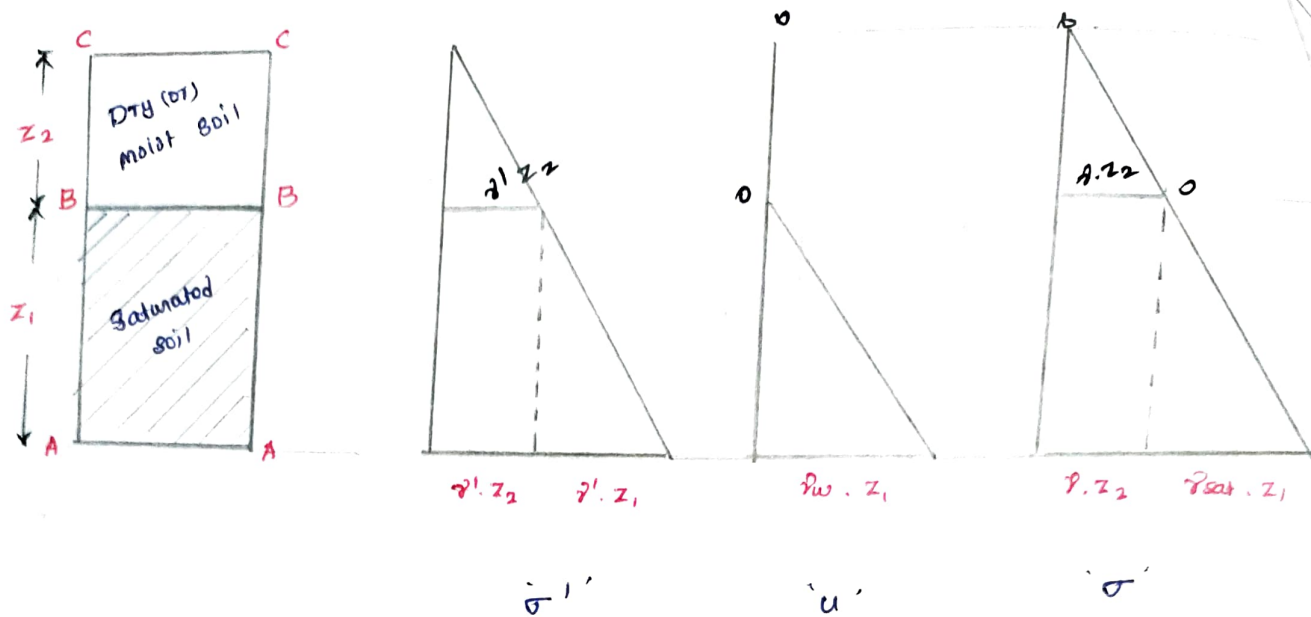
$$u = 0$$

$$\sigma' = 0$$

$$\gamma_w Z_2 + \gamma_{sat} Z_1 - \gamma_w Z_2 - \gamma_w Z_1$$

$$\gamma_w Z_2 + \gamma_w Z_1$$

c) Partially Submerged Soil mass :



At 'AA'

$$\sigma = \gamma_{sat} \cdot z_1 + \gamma \cdot z_2$$

$$u = \gamma_w \cdot z_1$$

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

$$= (\gamma_{sat} - \gamma_w) z_1 + \gamma \cdot z_2$$

$$\sigma' = \gamma' z_1 + \gamma z_2$$

At 'BB'

$$\sigma = \gamma \cdot z_2$$

$$u = 0$$

$$\sigma' = \gamma \cdot z_2$$

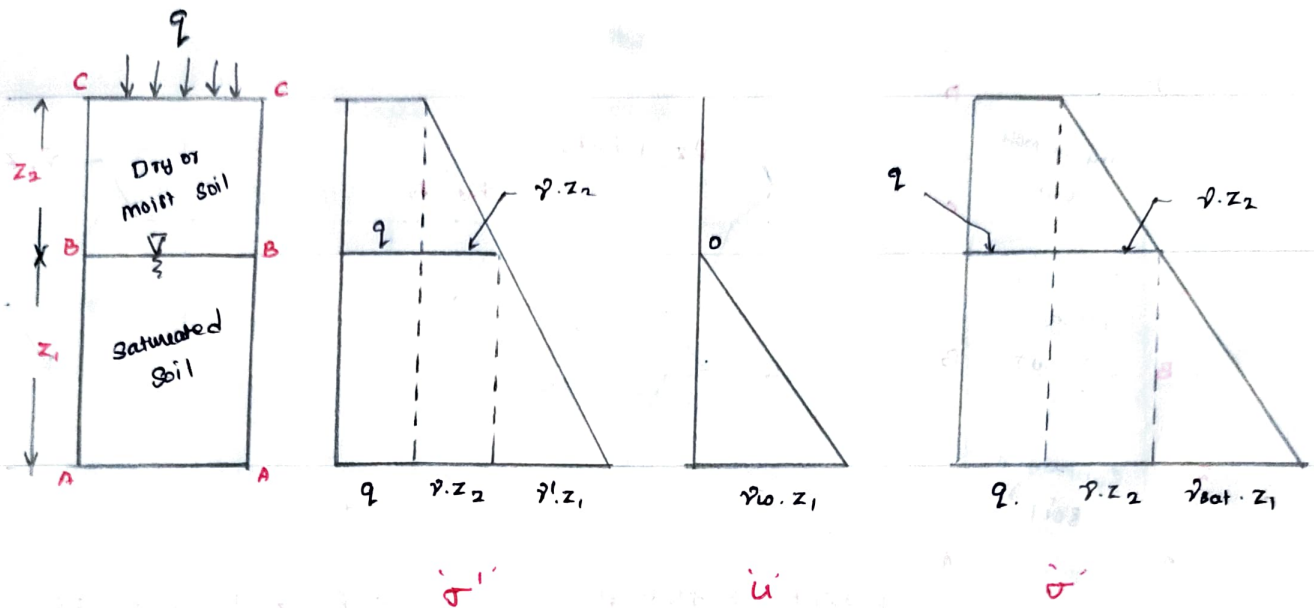
At 'CC'

$$\sigma = 0$$

$$u = 0$$

$$\sigma' = 0$$

d) Partially Submerged soil mass with Surcharge:



At 'AA'

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

$$u = \gamma_w \cdot Z_1$$

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

At 'BB'

$$\sigma = \gamma \cdot Z_2 + q$$

$$u = 0$$

$$\sigma' = \gamma \cdot Z_2 + q$$

At 'cc'

$$\sigma = q$$

$$u = 0$$

$$\sigma' = q$$

# PERMEABILITY

A material having continuous voids is called Permeable.

Permeability is defined as the property of a porous material which permits the passage or seepage of water through its inter connecting voids.

Seepage:

The slow movement of free of gravitational water through a porous medium is called seepage or percolation.

Darcy's Law:

According to Darcy's Law (1956) for a laminar flow conditions the velocity of flow ( $v$ ) is directly proportional to the hydraulic gradient ( $i$ )

$$v \propto i$$

$$v = ki$$

Where

$k$  - Co-efficient of Permeability

$i$  - Hydraulic gradient

$v$  = Velocity of flow average discharge Velocity

When

$$i = 1$$

$$\boxed{v = k}$$

| Soil Type | co. efficiency of Permeability<br>$k$ (cm/sec) |
|-----------|------------------------------------------------|
| Gravel    | 1 to 100                                       |
| Sand      | $10^{-3}$ to 1                                 |
| Silt      | $10^{-6}$ to $10^{-3}$                         |
| clay      | Less than $10^{-6}$                            |

Seepage Velocity (or) Actual Velocity:

It is defined as the rate of discharge of percolating water per unit cross-sectional area of voids perpendicular to the direction of flow.

Thus the Seepage Velocity ( $V_s$ )

$$V_s = q / A_v$$

$$q = A_v \cdot V_s$$

$$\frac{A_v}{A} = \frac{V_v}{V} = n$$

$$V_s = V \cdot \frac{A}{A_v}$$

$$= V \cdot \frac{1}{n}$$

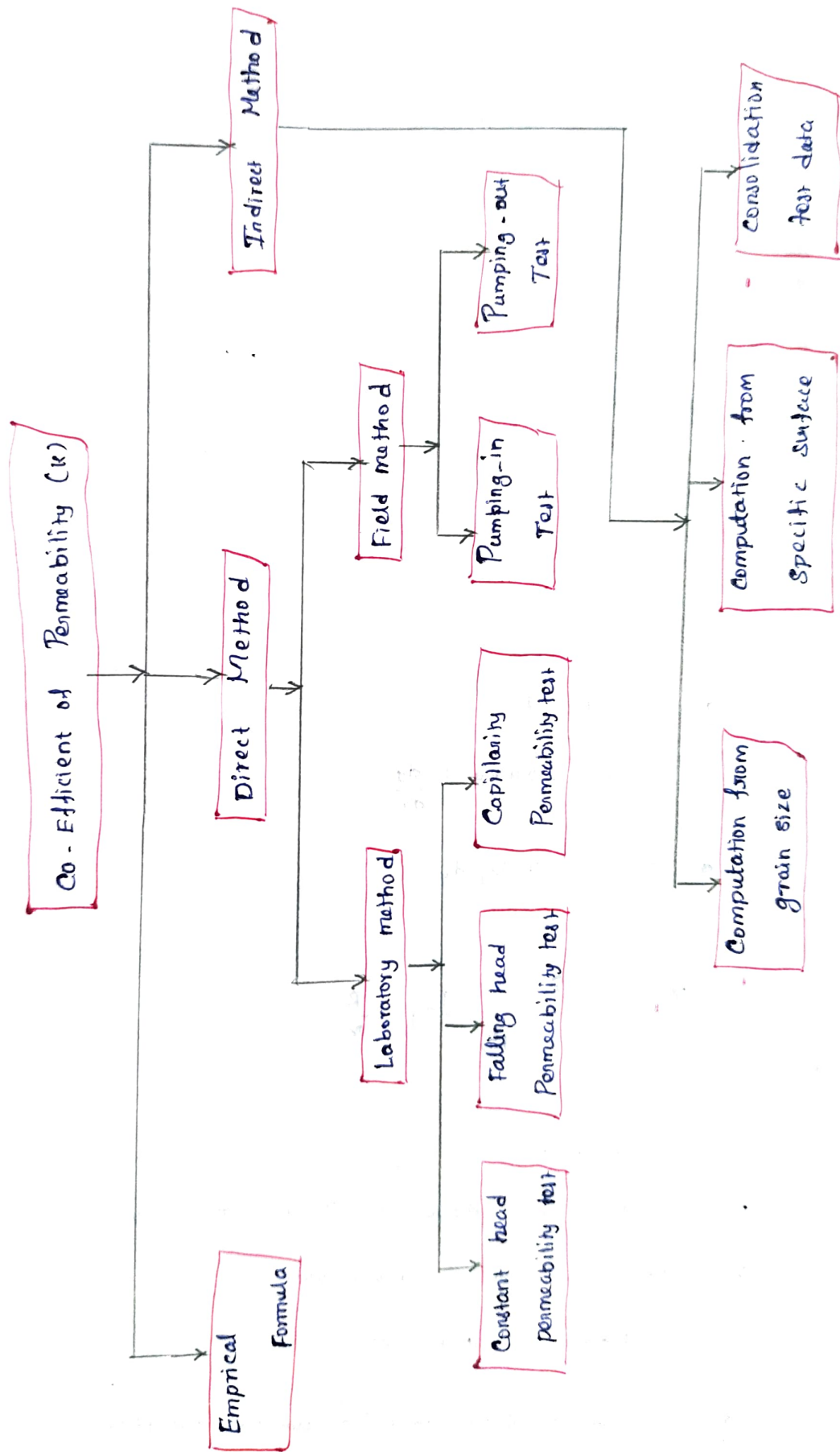
$$\therefore V_s = \frac{V}{n}$$

Where,

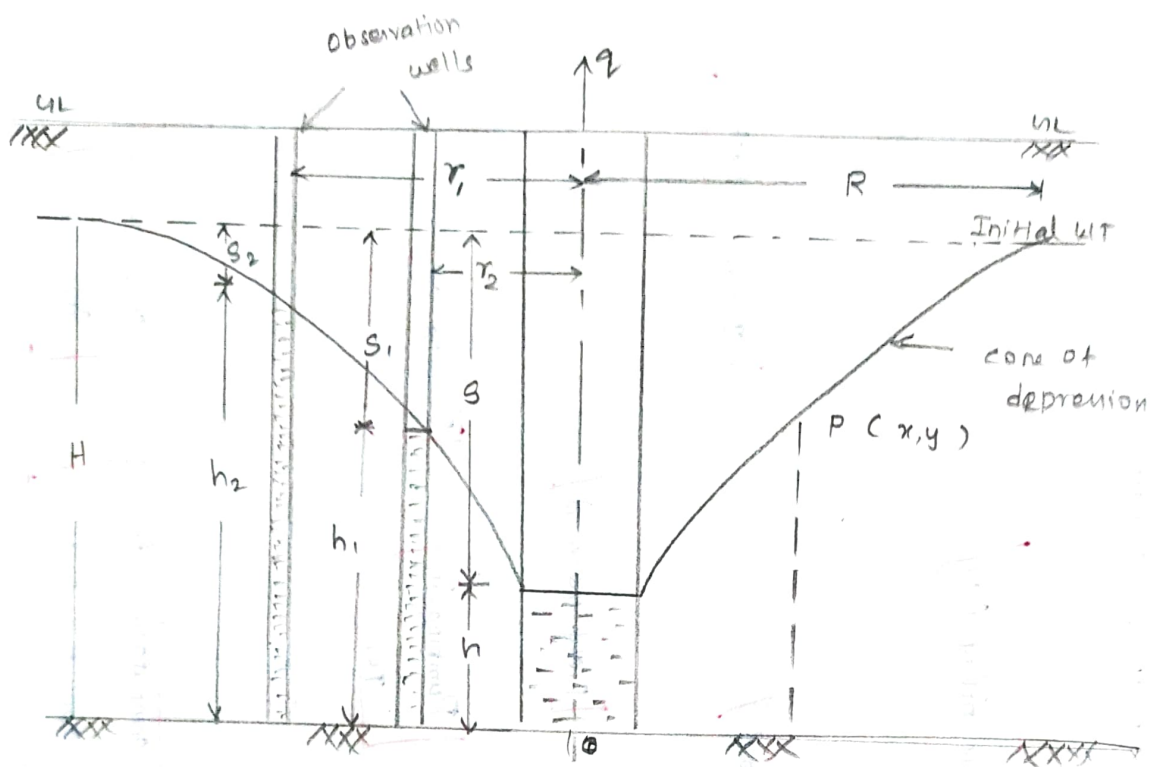
$A_v$  - Area of Voids

$n$  - Porosity of Soil mass

# DETERMINATION OF CO-EFFICIENT OF PERMEABILITY



a) Pumping-out test in unconfined aquifer



$$q = k \cdot A_n \cdot i_n$$

$$= k \cdot (2\pi xy) \frac{dy}{dx}$$

$$q \cdot \frac{dy}{dx} = 2\pi k_y dy$$

$$k = \frac{q}{\pi (H^2 - h^2)} \log_e \frac{R}{r}$$

Where

$r$  - Radius of main well

$R$  - Radius of zero drawdown known as max. drawdown

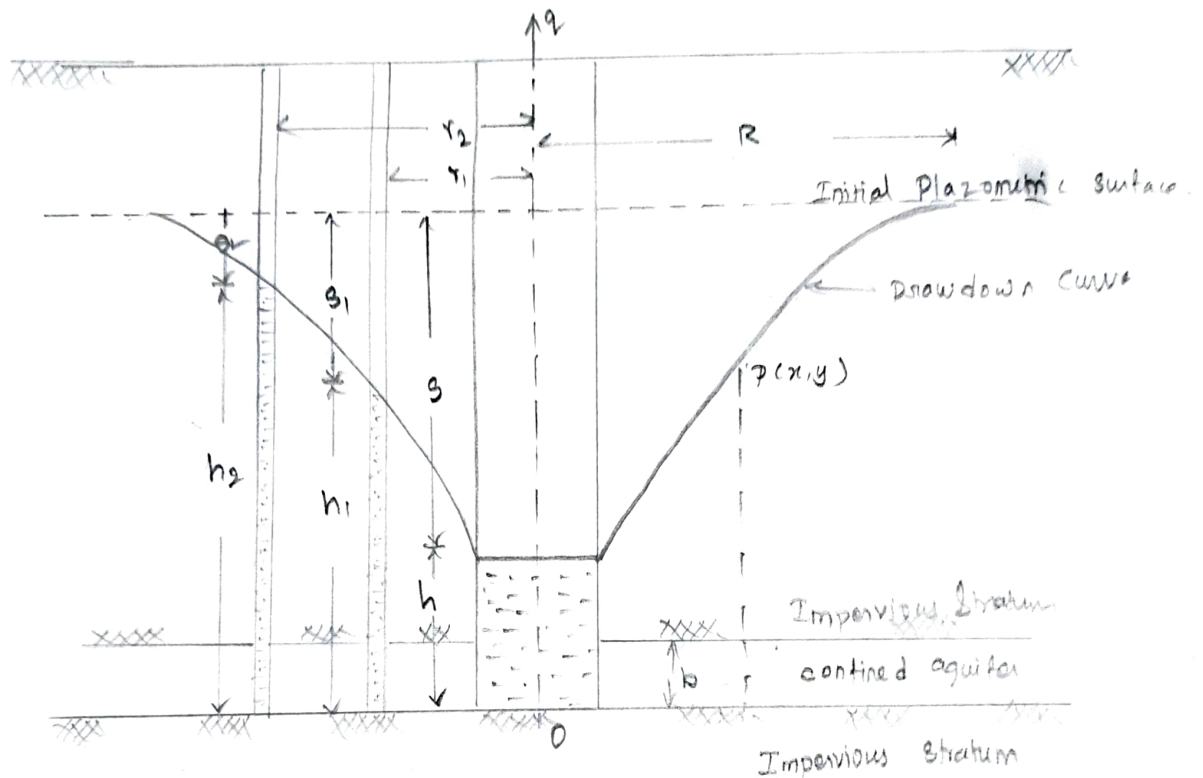
$h$  - depth of water in main well

$H$  - Height of initial water

$q$  - Rate at which water is pumped out of well.

b) Pumping-out test in confined aquifer

If the aquifer is bound by impervious stratum both at top and bottom is called Confined aquifer.

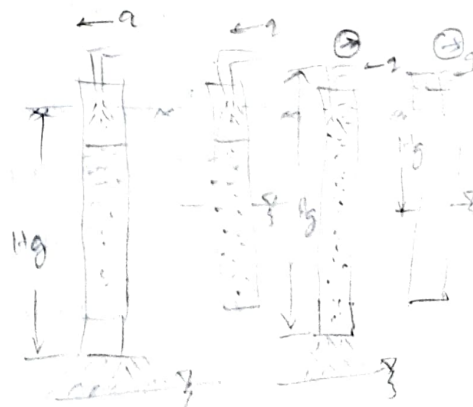


$$K = \frac{2}{2\pi b (H-h)} \log_e \frac{R}{r}$$

(b7)

$$K = \frac{q}{2\pi b (h_2 - h_1)} \log_e \frac{r_2}{r_1}$$

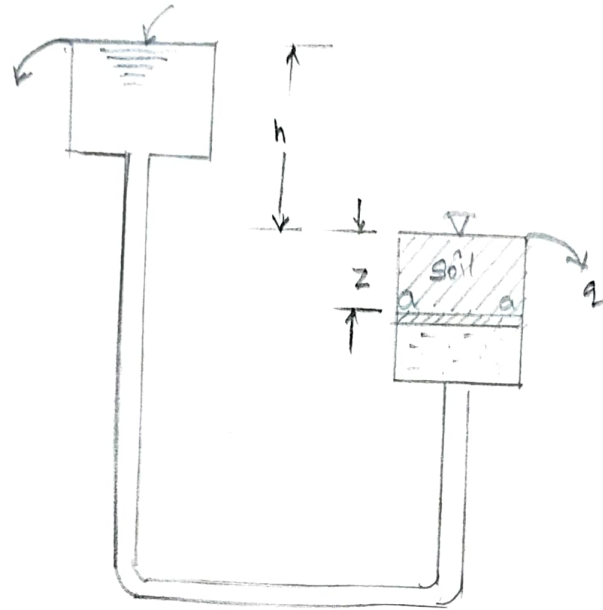
## Pumping in-Test



$$K = \frac{2}{5.574}$$

## Quick Sand Condition:

The phenomenon of lifting in the upward direction of flow of soil particles due to loss of shear strength in a cohesion less soil is called quick condition, boiling condition or quick sand.



$$i_c = \frac{\sigma - 1}{1 + e}$$

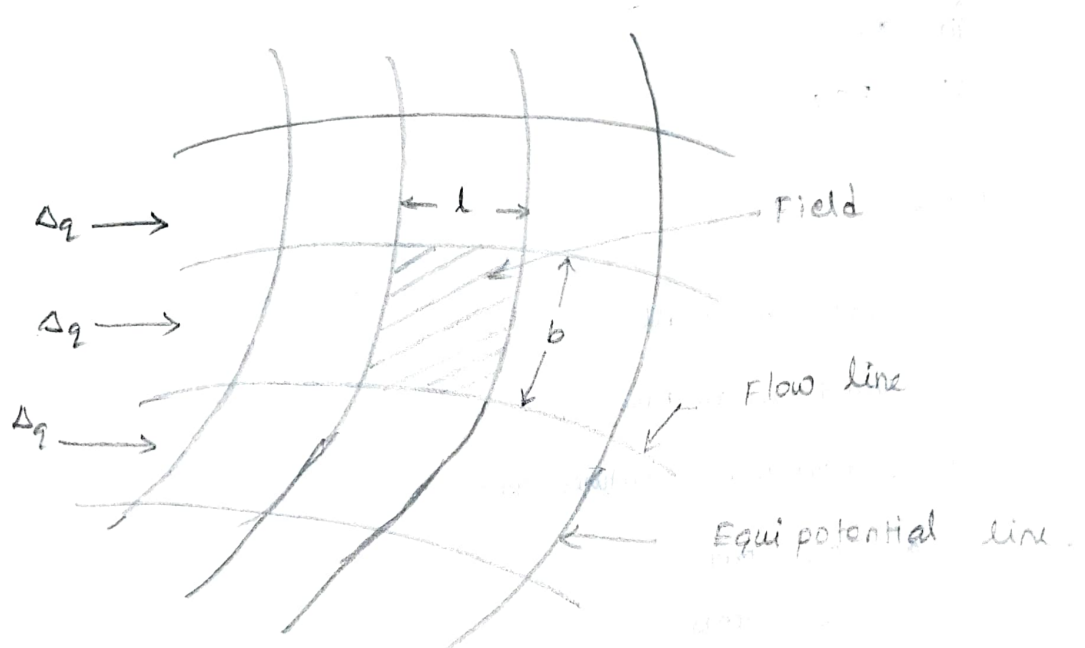
## Factors Affecting Permeability:

- i) Particle size
- ii) Properties of pore fluid
- iii) Void ratio
- iv) Soil fabric and soil stratification
- v) Degree of saturation
- vi) Presence of foreign matter
- vii) Adsorbed water

## FlowNet:

A flownet is a graphical representation of two-dimensional steady-state groundwater flow through aquifers.

- i) Flow lines
- ii) Equipotential lines



Applications (or) Uses of flownet:

- i) Quantity of seepage
- ii) Seepage pressure at a point
- iii) Hydrostatic pressure at a point
- iv) Exit gradient

## Sheet Pile:

Sheet piles are sections of sheet materials with interlocking edges that are driven into the ground to provide earth reaction and excavation support.

Sheet piles are commonly made of

- i) Timber
- ii) Steel
- iii) Rcc

## Uses:

- i) Retaining walls
- ii) Land reclamation
- iii) Underground structures such as
  - a) car parks
  - b) Basements
- iv) Marine location for
  - a) River bank protection
  - b) Sea walls
  - c) Cofferdams etc.

## Weir :

A weir or low head dam is a barrier across the horizontal width of a river that alters the flow characteristics of water and usually results in a change in the height of the river level.

1. The water table in a deposit of sand 8m thick is at a depth of 3m below the ground surface. Above the water table, the sand is saturated with the capillary water. The bulk density of sand is  $19.62 \text{ kN/m}^3$ . Calculate the effective pressure at 1, 3, and 8m below the ground surface. Hence plot the variation of total pressure, natural pressure and effective pressure over the depth of 8m.

i) Stresses at B, 1m below the ground level:

$$\sigma = \gamma_{\text{sat}} \cdot z$$

$$= 1 \times 19.62$$

$$\therefore \sigma = 19.62 \text{ kN/m}^2$$

$$u = -2 \times \gamma_w = -2 \times 9.81$$

$$\therefore u = -19.62 \text{ kN/m}^2$$

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

$$= 19.62 - (-19.62)$$

$$\therefore \sigma' = 39.24 \text{ kN/m}^2$$

ii) Stresses at C, 3m below the G.L:

$$\sigma = 3 \times 19.62 \Rightarrow 58.86 \text{ kN/m}^2$$

$$u = 0$$

$$\sigma' = \sigma - u = 58.86 \text{ kN/m}^2$$

iii) Stresses at 8m, below the G.L:

$$\sigma = (3+5) \times 19.62 = 156.96 \text{ kN/m}^2$$

$$u = h \gamma_w = 5 \times 9.81 \Rightarrow 49.05 \text{ kN/m}^2$$

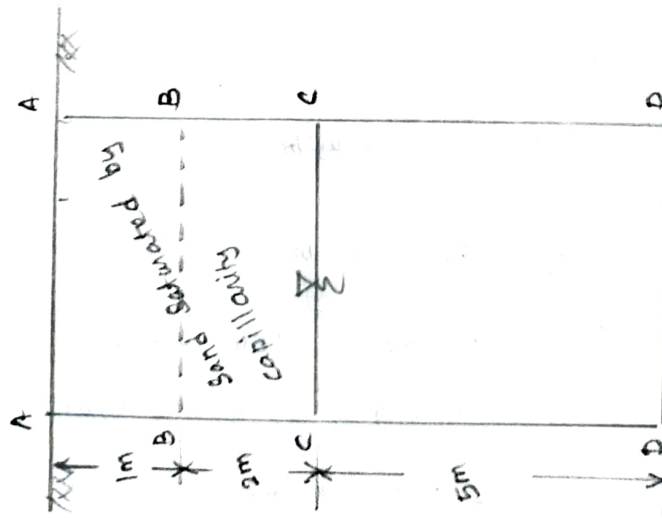
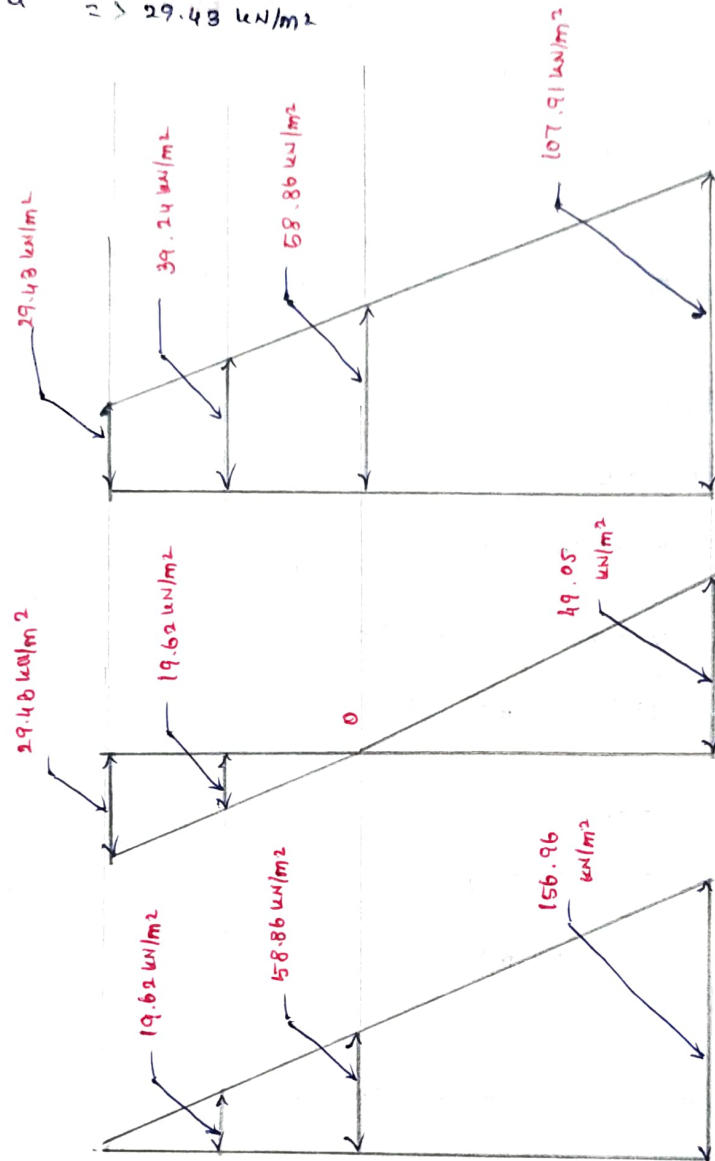
$$\sigma' = \sigma - u = 107.91 \text{ kN/m}^2$$

iv) stresses at A at ground level:

$$\sigma = 0$$

$$u = -h \cdot \gamma_w = -3 \times 9.81 \Rightarrow -29.43 \text{ kN/m}^2$$

$$\sigma' = \sigma - u \Rightarrow 29.43 \text{ kN/m}^2$$

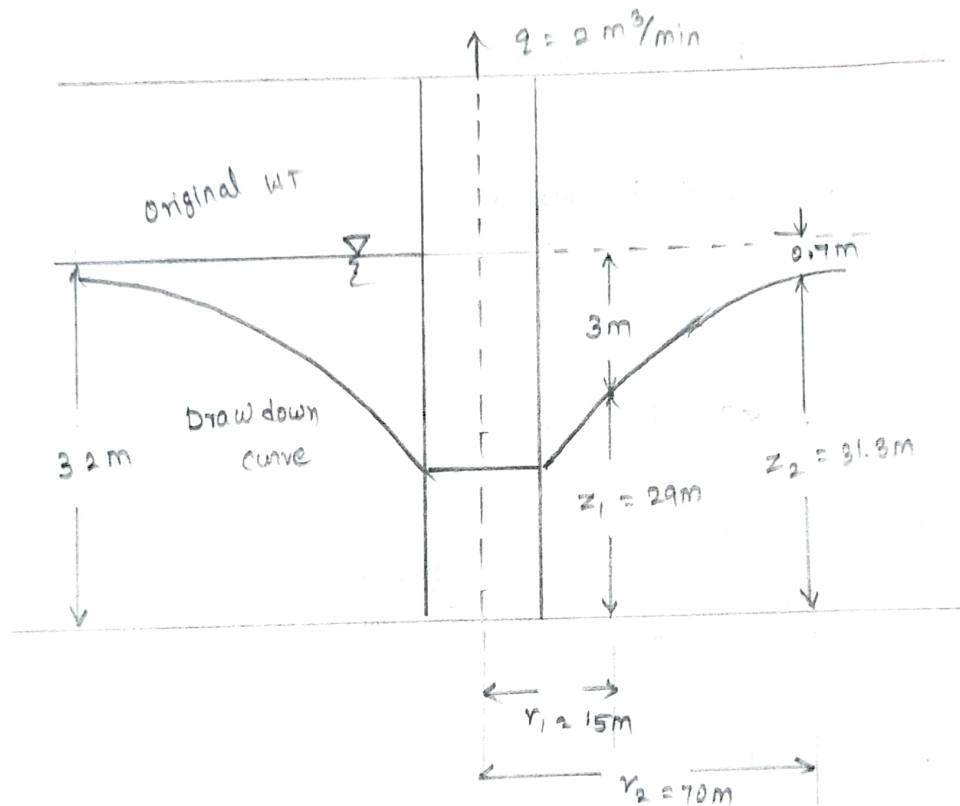


Effective stress

Neutral pressure

Total pressure

An unconfined aquifer is known to be 32m thick below the water table. A constant discharge  $2 \text{ m}^3$  per minute is pumped out of the aquifer through a tube well till the water level in the tube well becomes steady. Two observation wells at distances of 15m and 70m from the tube well show falls of 3m and 0.7m respectively from their static water levels. Find the permeability of the aquifer.



$$K = \frac{2.303 q \log_{10} \left( \frac{r_2}{r_1} \right)}{\pi \times (z_2^2 - z_1^2)}$$

$$= \frac{0.051}{871.415}$$

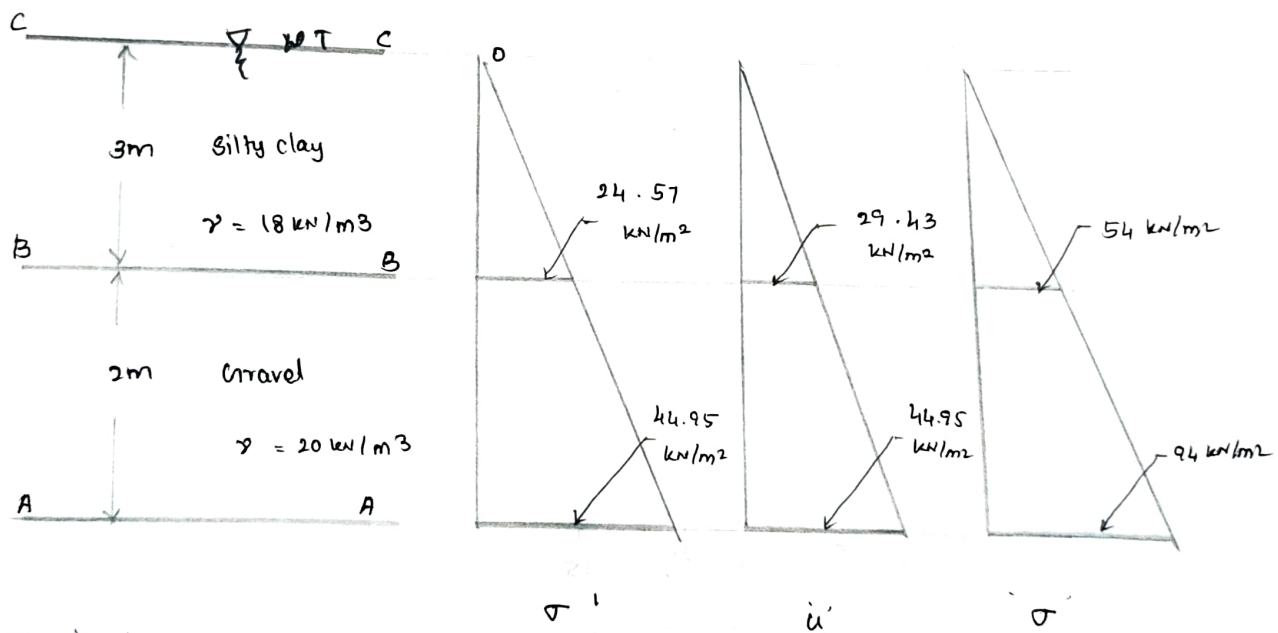
$$= 0.00005826 \text{ m/sec}$$

$$\therefore K = 0.118 \text{ mm/sec}$$

In a site reclamation project 2.5m of graded fill ( $\gamma = 22 \text{ kN/m}^3$ ) were laid in compacted layers over an existing layer of silty clay ( $\gamma = 18 \text{ kN/m}^3$ ) which has 3m thick. This was underlain by a 2m thick layer of gravel ( $\gamma = 20 \text{ kN/m}^3$ ). Assuming that the water table remains at the surface of the silty clay draw the effective stress profiles for cases.

- Before the fill is placed
- After the fill has been placed

case (i) Before the fill is placed:



At A-A

$$\sigma = 2 \times 20 + 3 \times 18$$

$$= 94 \text{ kN/m}^2$$

$$u = 3 \times 9.81$$

$$= 29.43 \text{ kN/m}^2$$

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

$$= 64.57 \text{ kN/m}^2$$

At B-B

$$\sigma = 3 \times 18 = 54 \text{ kN/m}^2$$

$$u = 3 \times 9.81 = 29.43 \text{ kN/m}^2$$

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

$$= 24.57 \text{ kN/m}^2$$

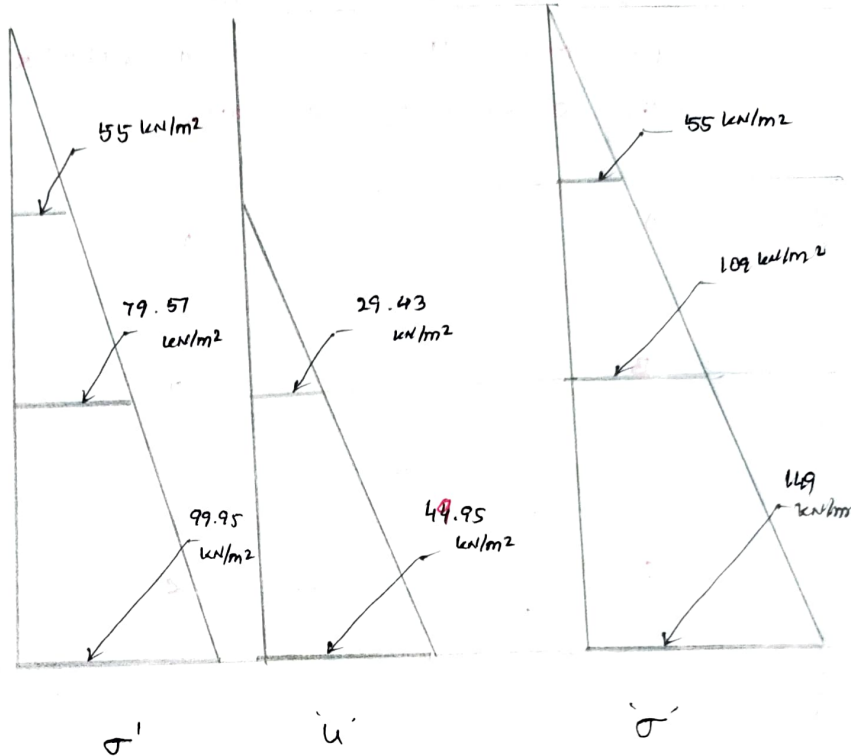
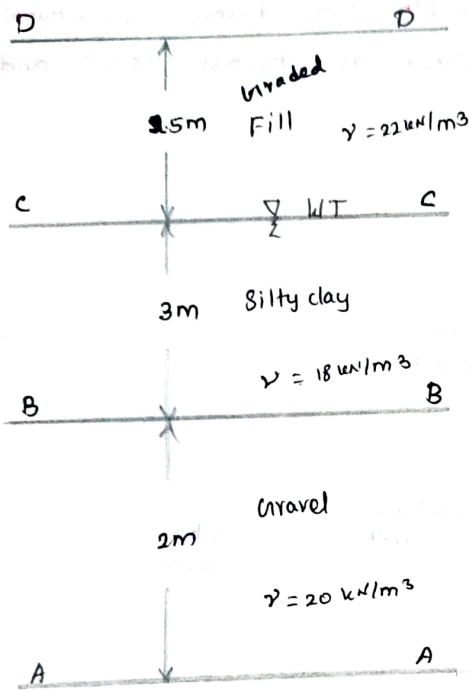
At C-C

$$\sigma = 0$$

$$u = 0$$

$$\sigma' = 0$$

Case (ii) After the fill has been placed:



At A-A

$$\sigma = 2 \times 20 + 3 \times 18 + 2.5 \times 22$$

$$= 149 \text{ kN/m}^2$$

$$u = 5 \times 9.81 \Rightarrow 49.05 \text{ kN/m}^2$$

$$\sigma' = \sigma - u \Rightarrow 99.95 \text{ kN/m}^2$$

At B-B

$$\sigma = 3 \times 18 + 2.5 \times 22 \Rightarrow 109 \text{ kN/m}^2$$

$$u = 3 \times 9.81 \Rightarrow 29.43 \text{ kN/m}^2$$

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

$$= 79.57 \text{ kN/m}^2$$

At C-C

$$\sigma = 2.5 \times 22$$

$$= 55 \text{ kN/m}^2$$

$$u = 0$$

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

$$= 55 \text{ kN/m}^2$$

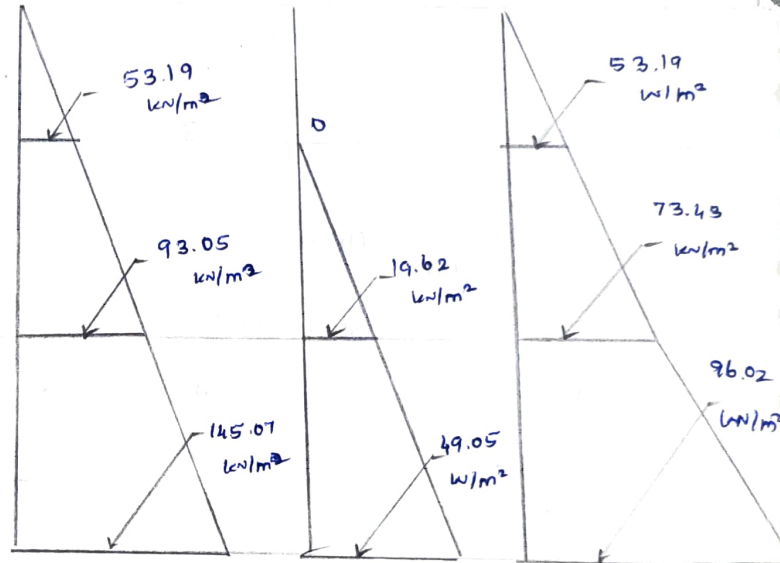
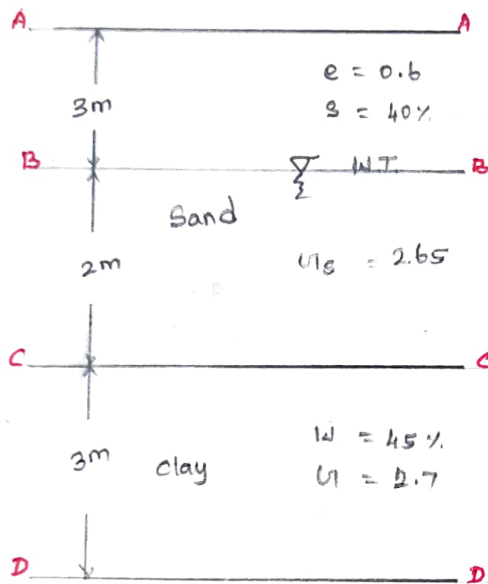
At D-D

$$\sigma = 0$$

$$u = 0$$

$$\sigma' = 0$$

A clay layer 3m thick is having water content 45% and specific gravity of solids 2.7. This clay layer is lying below another layer which is 5m thick sand layer. The sand layer lying at the top is having voids ratio 0.6 and with degree of saturation, 40%. and  $G_s = 2.65$ . The water table is at a depth 3m below. Determination the total stress, pore pressure and effective stress at various levels and draw the corresponding diagrams.



for sand:

$$\gamma_d = \frac{G_s \cdot \gamma_w}{1 + e}$$

$$\gamma_d = 16.25 \text{ kN/m}^3$$

40%.

$$w = \frac{e S}{G_s} = \frac{0.6 \times 0.4}{2.65}$$

$$w = 0.091$$

$$\gamma_{wet} = \gamma_d (1 + w)$$

$$= 16.25 (1 + 0.091)$$

$$\gamma_{wet} = 17.73 \text{ kN/m}^3$$

100%.

$$w = \frac{e S}{G_s} = \frac{0.6 \times 1}{2.65} = 0.226$$

$$\gamma_{sat} = \gamma_d (1 + w) = 19.93 \text{ kN/m}^3$$

For clay:

$$e = \frac{w \cdot G_s}{S} = \frac{0.45 \times 2.70}{100\% \text{ Sat}} = 1.215$$

$$\gamma_{sat} = \frac{G_s \cdot \gamma_w}{1 + e} (1 + w) = \frac{2.70 \times 9.81}{1 + 1.215} (1 + 0.45)$$

$$\gamma_{sat} = 17.34 \text{ kN/m}^3$$

## UNIT - III

### STRESS DISTRIBUTION AND SETTLEMENT

Stress distribution:

Stresses are induced in a soil mass due to self weight of the soil and structural loads applied at or below the surface.

Stresses due to self weights are sometimes known as geostatic stresses.

Let us take the soil mass to be bounded by the horizontal plane (ground surface)  $x$ ,  $y$ , and  $z$ -axis be directed downwards. Under this condition, the soil mass is said to be semi-infinite, where there is no external loading, the ground plane becomes a principal plane since it is devoid of any shear loading.

The point is considered at a depth of ' $z$ ' below the ground surface.

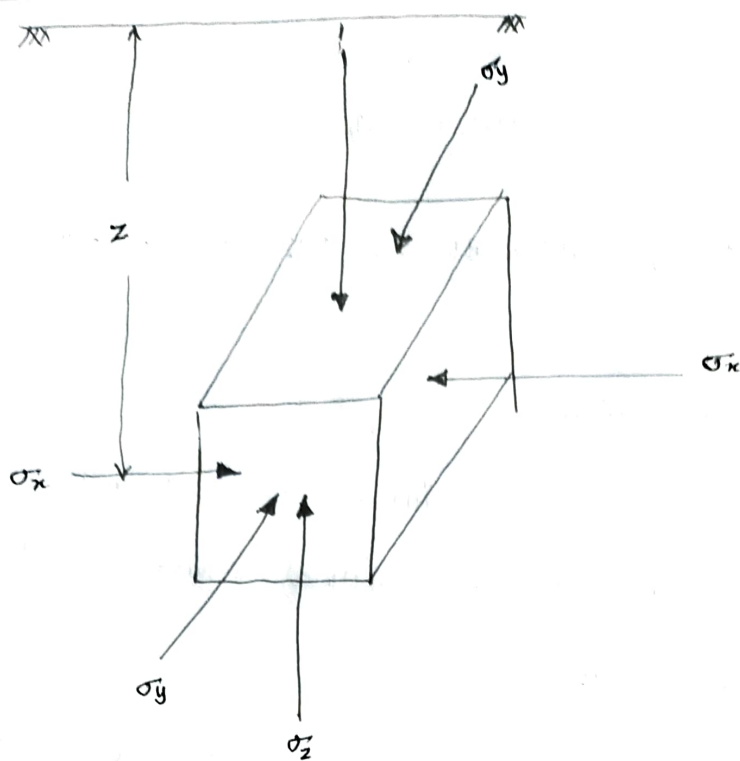
$$\sigma_z = \gamma z$$

where,

$\sigma_z$  - Vertical stress at a point within and soil mass

$\gamma$  - Unit weight of soil

$z$  - depth below the ground surface.



Soil Element and stress components

$$\sigma_x = \sigma_y = \frac{\mu}{1-\mu} \gamma \cdot z$$

$$\therefore \sigma_x = \sigma_y = K_0 \gamma \cdot z$$

$$\therefore K_0 = \frac{\mu}{1-\mu}$$

$\mu$  - Poisson's ratio

$K_0$  - Coefficient of lateral pressure at rest.

The stress components due to self weight and surface loads can be found separately and then added algebraically to get the final stresses at the point. The total stress field at a point within a soil mass loaded at its boundary consists of nine stress components i.e.

$$\begin{bmatrix} \sigma_x & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_y & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_z \end{bmatrix}$$

These nine stress components as given by this group of square matrix of stresses are the components of a mathematically entity called the "stress tensor" with a symmetrical matrix.

The mostly used theories regarding distribution of stress in soil are those of Boussinesq and Klestergaard.

### Boussinesq Equation (Concentrated force)

Boussinesq (1885) solved the problem of stress distribution in soil due to a concentrated load acting at the ground surface by assuming a suitable stress function. The following assumptions are made in the solutions by the theory of elasticity.

1. The soil mass is an elastic medium for which the modulus of elasticity ( $E$ ) is constant.
2. The soil mass is homogeneous, all its elements are similar and it has identical properties at every point in all identical directions.
3. The soil mass is isotropic, that is, it has identical properties in all directions through any point of it.
4. The soil mass is semi-infinite, that is, it extends infinitely in all directions below a level surface.
5. The self weight of the soil is ignored.

6. The soil is initially unstressed.
7. The change in Volume of the soil upon application of the load on it is neglected.
8. The top surface of the medium is free of shear stress and is subjected to only the point load at a specified location.
9. The continuity of stress is considered to exist in the medium.
10. The stress are distributed symmetrically with respect to  $z$ -axis.

Normal Stress at any point in a Soil Mass  
Subjected to point load.

Let a point load  $Q$  (single concentrated vertical load) act at the ground surface, at a point  $O$  which may be taken as origin of the  $x$ ,  $y$  and  $z$  axes as shown in fig.

Let us find the stress components at a point  $P$  in the soil mass, having coordinates  $x, y$  and  $z$  or having a radial horizontal distance  $r$  and vertical distance  $z$  from the point  $O$ .

Using the logarithmic stress function, the polar radial stress ( $\sigma_r$ ) may be expressed as



$$\text{Polar radial stress } (\sigma_R) = \frac{3}{2} \frac{Q}{\pi} \frac{\cos \beta}{R^2}$$

$R$  - Polar distance between the origin  $O$  and point  $P$

$\beta$  - angle which the line  $OP$  makes with Vertical.

$$R = \sqrt{r^2 + z^2}$$

$$= (x^2 + y^2 + z^2)^{1/2}$$

$$r = \sqrt{x^2 + y^2}$$

$$R = \sqrt{r^2 + z^2}$$

$$\cos \beta = \frac{z}{R}$$

$$\sin \beta = \frac{r}{R}$$

The Vertical normal stress  $\sigma_z$  at point  $P$  is given by.

$$\sigma_z = \sigma_R \cos^2 \beta$$

$$\therefore \sigma_z = \frac{3Q}{2\pi} \cdot \frac{\cos \beta}{R^2} \times \cos^2 \beta$$

$$= \frac{3Q}{2\pi R^2} \cos^3 \beta$$

$$= \frac{3Q}{2\pi R^2} \left[ \frac{z}{R} \right]^3$$

$$= \frac{3Qz^3}{2\pi R^5}$$

$$= \frac{3Qz^3}{2\pi [r^2 + z^2]^{5/2}}$$

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

$$\frac{z^2}{2\pi} \times \frac{z^2}{z^2} = \frac{z^2}{2\pi}$$

$$\frac{z^2}{2\pi} \times \frac{z^2}{z^2} = \frac{z^2}{2\pi}$$

This equation may be written as.

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

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

$K_B$  is known as Boussinesq influence factor.

- 1) A concentrated load of 2000 kN is applied @ a ground surface. Determine the vertical stress @ a point 'P' which is 6m below the load. Also calculate the vertical stress at a point R which is at a depth of 6m and a horizontal distance of 5m from the axis of the load

Given data:

$$Q = 2000 \text{ kN} ; z_1 = 6 \text{ m} ; z_2 = 6 \text{ m} ; r = 5 \text{ m} ;$$

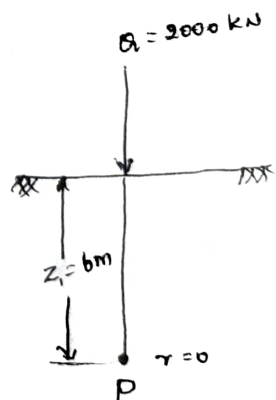
To find:

Vertical stress ( $\sigma_z$ )

Sol:

Case (i) vertical stress at point 'P'

$$\begin{aligned} \sigma_z &= \frac{3Q}{2\pi z^2} \left[ \frac{1}{1 + \left(\frac{r}{z}\right)^2} \right]^{5/2} \\ &= \left( \frac{3 \times 2000}{2 \times \pi \times 6^2} \right) \left[ \frac{1}{1 + \left(\frac{0}{6}\right)^2} \right]^{5/2} \end{aligned}$$



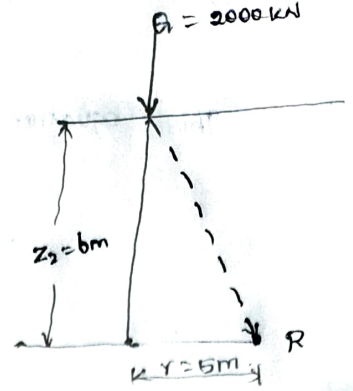
$$\therefore \boxed{\sigma_z = 26.5 \text{ kN/m}^2}$$

Case (ii) Vertical stress at 'R'

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

$$= \frac{3 \times 2000}{2 \times \pi \times 6^2} \left[ \frac{1}{1 + (5/6)^2} \right]^{5/2}$$

$$\sigma_z = 7.09 \text{ kN/m}^2$$



2. A Concentrated point load of 200kN acts at the ground surface. Find the intensity of vertical pressure at a depth of 10m below the ground surface, and what will be vertical pressure at a point at a depth of 5m and radial distance of 2m from the axis of loading? Use Boussinesq analysis.

$$Q = 200 \text{ kN}$$

$$d = 10 \text{ m}$$

$$z_1 = 10 \text{ m}$$

$$z_2 = 5 \text{ m}$$

$$r = 2 \text{ m}$$

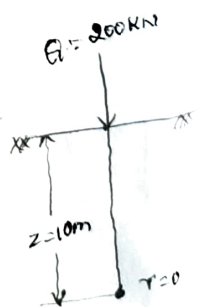
Sol

i) Vertical pressure at 10m below the G.S

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

$$= \frac{3 \times 200}{2 \times \pi \times 10^2} \left[ \frac{1}{1 + (0/10)^2} \right]^{5/2}$$

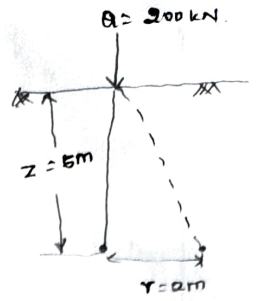
$$\therefore \sigma_z = 0.954 \text{ kN/m}^2$$



ii) Vertical pressure at 5m below the G.S and  $r = 2m$

$$\sigma_z = \frac{3 \times 200}{2 \times \pi \times 5^2} \left[ \frac{1}{1 + \left(\frac{2}{5}\right)^2} \right]^{5/2}$$

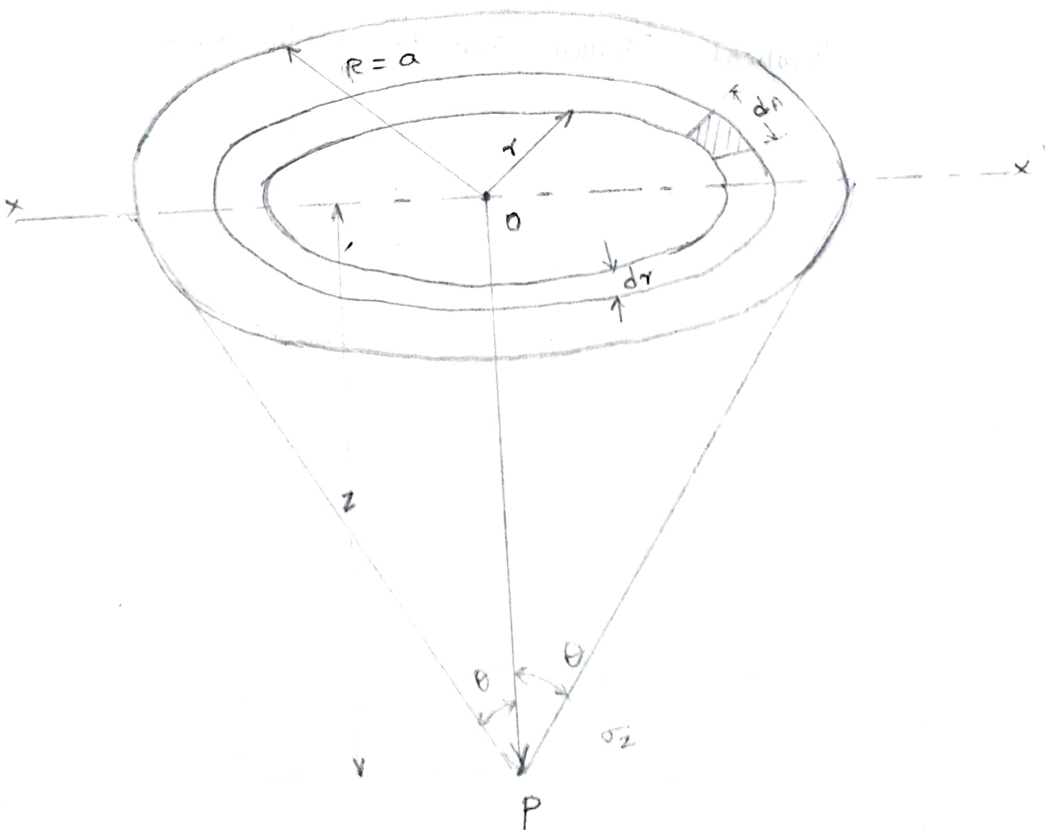
$$\therefore \sigma_z = 2.63 \text{ kN/m}^2$$



Case (ii)

Vertical Pressure Under A Uniformly loaded Circular Area:

The vertical pressure in a homogeneous soil mass under a uniformly loaded circular area can be found using Boussinesq Analysis.



$$\sigma_z = K_B \cdot q$$

$$K_B = 1 - \left[ \frac{1}{1 + (R/2)^2} \right]^{3/2} \quad (R=a)$$

$K_B$  is Boussinesq Influence factor

$\theta$  is the angle which the line joining the outer edge of the circular loaded area makes with the central axis.

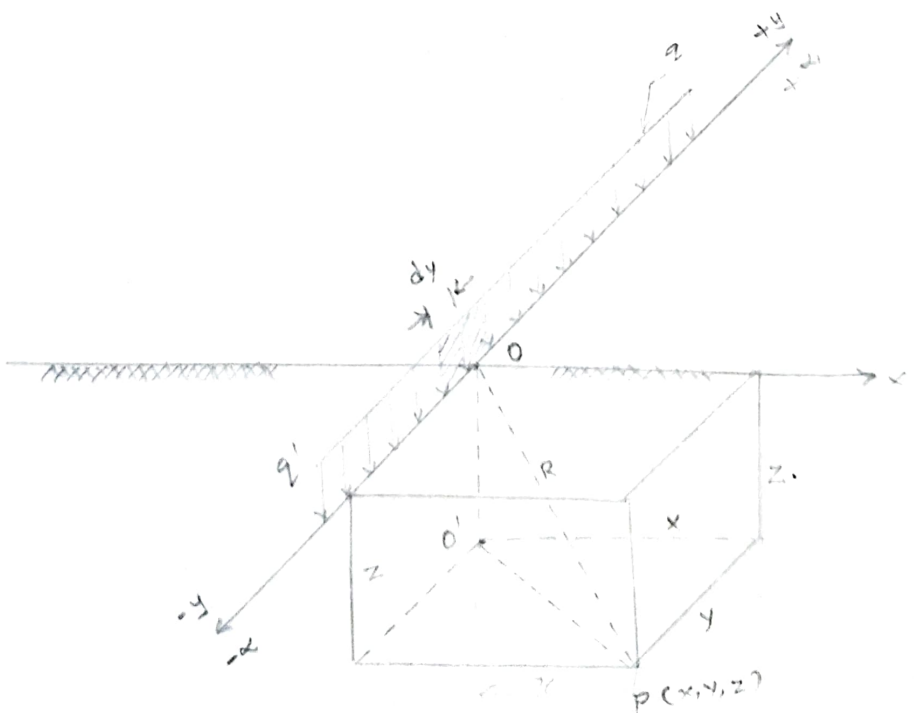
$$\sigma_z = q \left[ 1 - \left( \frac{1}{\sec^2 \theta} \right)^{3/2} \right]$$

(or)

$$\sigma_z = q [1 - \cos^3 \theta]$$

Case (iii)

Vertical Pressure Due to a Line Load.



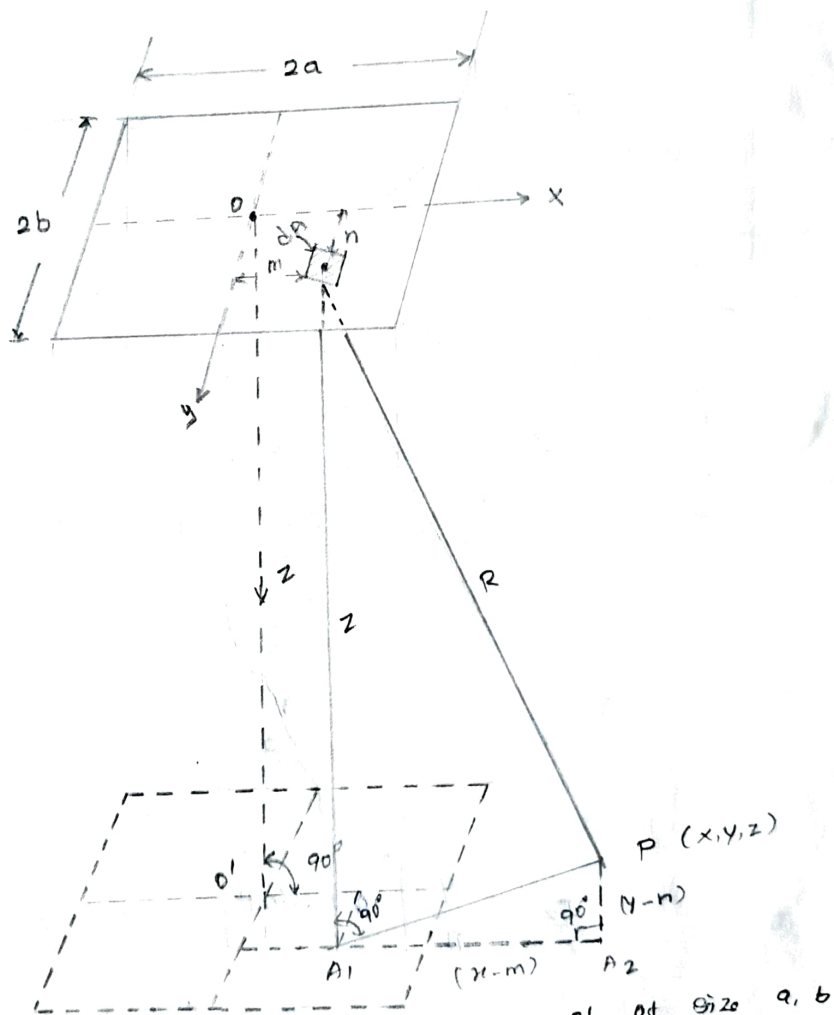
LINE LOAD

The vertical pressure at 'P' due to entire Line load is,

$$\sigma_z = \frac{2q}{\pi z} \left[ \frac{1}{1 + (x/z)^2} \right]^2$$

If the point load P is situated directly below the line load  $x=0$ .

Case (iv) Vertical Pressure Under a Uniformly loaded Rectangular Area.



Vertical Pressure under the corner of a rectangular area of size  $a, b$

$$(\sigma_z)_c = \frac{q}{\pi} \left[ \frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+m^2n^2+1} \cdot \frac{m^2+n^2+1}{m^2+n^2+1} + \tan^{-1} \frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2-m^2n^2+1} \right]$$

$$\therefore (\sigma_z)_c = K \cdot q$$

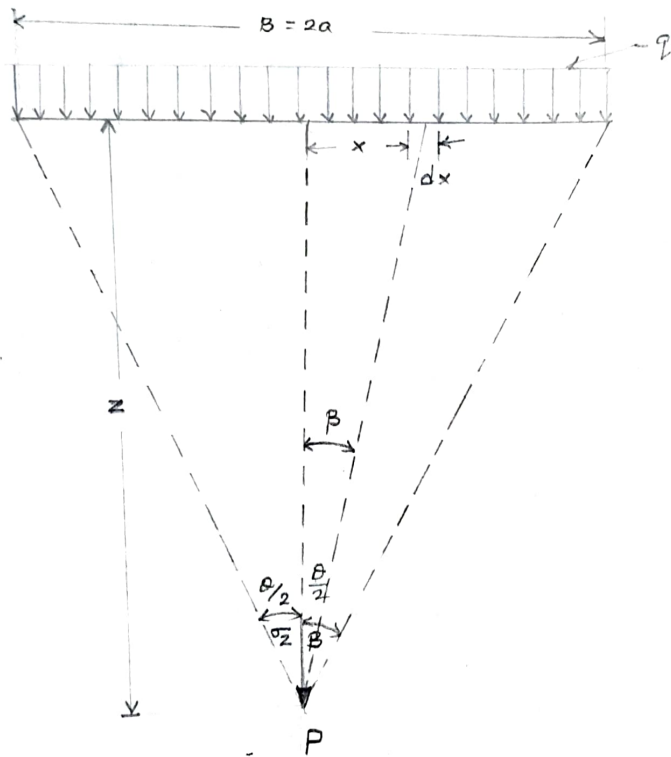
where  $m = a/z$  ;  $n = b/z$

$K$  - Influence factor.

Vertical Stress Under the Centre of the rectangle  
( $x=y=0$ )

$$(\sigma_z)_0 = \frac{2q}{\pi} \left[ \frac{abz(a^2+b^2+2z^2)}{(a^2+z^2)(b^2+z^2)\sqrt{a^2+b^2+z^2}} + \sin^{-1} \frac{ab}{\sqrt{a^2+z^2}\sqrt{b^2+z^2}} \right]$$

Case (v) Vertical Pressure Under Strip Load.



$$\sigma_z = \frac{q}{\pi} [\theta + \sin \theta]$$

✓ 1. An angular ring of external & internal diameters are 8m and 4m respectively. Transmits a pressure of  $100 \text{ kN/m}^2$ . Compute the vertical stress at a depth of 2m. below the centre, use Boussinesq Analysis.

Given data:

External dia.  $D = 8\text{m}$  ;  $R = 4\text{m}$

Internal dia  $d = 4\text{m}$  ;  $r = 2\text{m}$

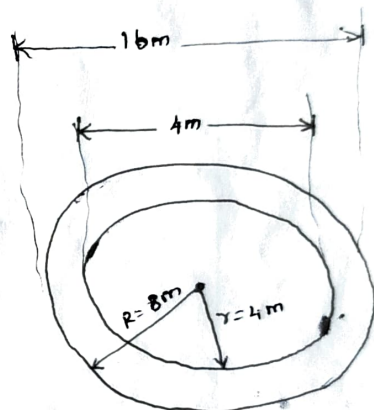
Pressure  $q = 100 \text{ kN/m}^2$

$Z = 2\text{m}$

Sol:

From Boussinesq equation:

$$\sigma_z = q \left\{ 1 - \left[ \frac{1}{1 + (r/z)^2} \right]^{3/2} \right\}$$



Vertical stress at a depth of 2m

$$\sigma_z = K_{B1} q - K_{B2} q$$

$$\begin{aligned} \sigma_z &= q \left\{ 1 - \left[ \frac{1}{1 + (R/z)^2} \right]^{3/2} \right\} - q \left\{ 1 - \left[ \frac{1}{1 + (r/z)^2} \right]^{3/2} \right\} \\ &= 100 \left\{ 1 - \left[ \frac{1}{1 + (4/2)^2} \right]^{3/2} \right\} - 100 \left\{ 1 - \left[ \frac{1}{1 + (2/2)^2} \right]^{3/2} \right\} \end{aligned}$$

$$\therefore \sigma_z = 100 (0.986) - 100 (0.911)$$

$$= 98.6 - 91.1$$

$$\therefore \sigma_z = 7.5 \text{ kN/m}^2$$

2.

A circular area on the surface of an elastic soil mass of great extent carries a uniformly distributed load of  $15 \text{ kN/m}^2$ . The radius of the circular area is  $4 \text{ m}$ . Compute the vertical normal stress at a point  $6 \text{ m}$  beneath the centre of the circle using Boussinesq method.

$$q = 15 \text{ kN/m}^2$$

$$r = 4 \text{ m}$$

$$z = 6 \text{ m}$$

The Boussinesq equation for circular area is,

$$\sigma_z = q \left\{ 1 - \left[ \frac{1}{1 + \left(\frac{r}{z}\right)^2} \right]^{3/2} \right\}$$

$$= 15 \left\{ 1 - \left[ \frac{1}{1 + (4/6)^2} \right]^{3/2} \right\}$$

$$\therefore \sigma_z = 6.86 \text{ kN/m}^2$$

3.

(H.W)

A water tank is supported by a ring foundation having outer diameter  $10 \text{ m}$  and inner diameter  $7.5 \text{ m}$ . The ring foundation transmits uniform load intensity of  $160 \text{ kN/m}^2$ . Compute the vertical stress induced at a depth of  $4 \text{ m}$  below the centre of ring foundation by Boussinesq analysis.

$$D = 10 \text{ m} ; R = 5 \text{ m}$$

$$d = 7.5 \text{ m} ; r = 3.75 \text{ m}$$

$$q = 160 \text{ kN/m}^2$$

$$z = 4 \text{ m}$$

$$\therefore \sigma_z = 23.11 \text{ kN/m}^2$$

4.

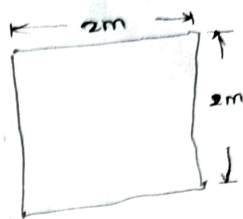
Calculate the intensity of Vertical stress induced at the point 2m vertically below the corner of a square footing of 2m side transmitting a bearing pressure of  $12 \text{ t/m}^2$  to the surface of the soil deposit. Use Bouwinesq equation:

Datas:

$$\text{Size of Footing} = 2\text{m} \times 2\text{m}$$

$$q = 12 \text{ t/m}^2$$

$$z = 2\text{m}$$



Bouwinesq equation for Rectangular or Square area is, the intensity of load acting at the corner is.

$$(\sigma_z)_c = \frac{q}{4\pi} \left[ \frac{2mn(m^2+n^2+1)^{\frac{1}{2}}}{m^2+n^2+m^2n^2+1} \cdot \frac{m^2+n^2+1}{m^2+n^2+1} + \tan^{-1} \frac{2mn(m^2+n^2+1)^{\frac{1}{2}}}{m^2+n^2-m^2n^2+1} \right]$$

$$m = a/z; \quad n = b/z$$

$$= 2/2 \quad = 2/2$$

$$\therefore m = 1 \quad \therefore n = 1$$

$$(\sigma_z)_c = \frac{12}{4\pi} \left[ \frac{(2 \times 1 \times 1) (1^2 + 1^2 + 1)^{\frac{1}{2}}}{1^2 + 1^2 + (1 \times 1) + 1} \cdot \frac{1^2 + 1^2 + 1}{1^2 + 1^2 + 1} + \tan^{-1} \left( \frac{(2 \times 1 \times 1) (1^2 + 1^2 + 1)^{\frac{1}{2}}}{1^2 + 1^2 - 1 + 1} \right) \right]$$

60.264

$$= \frac{12}{4\pi} \left[ (0.866 \times 1.333) + \tan^{-1} (1.732) \right]$$

116

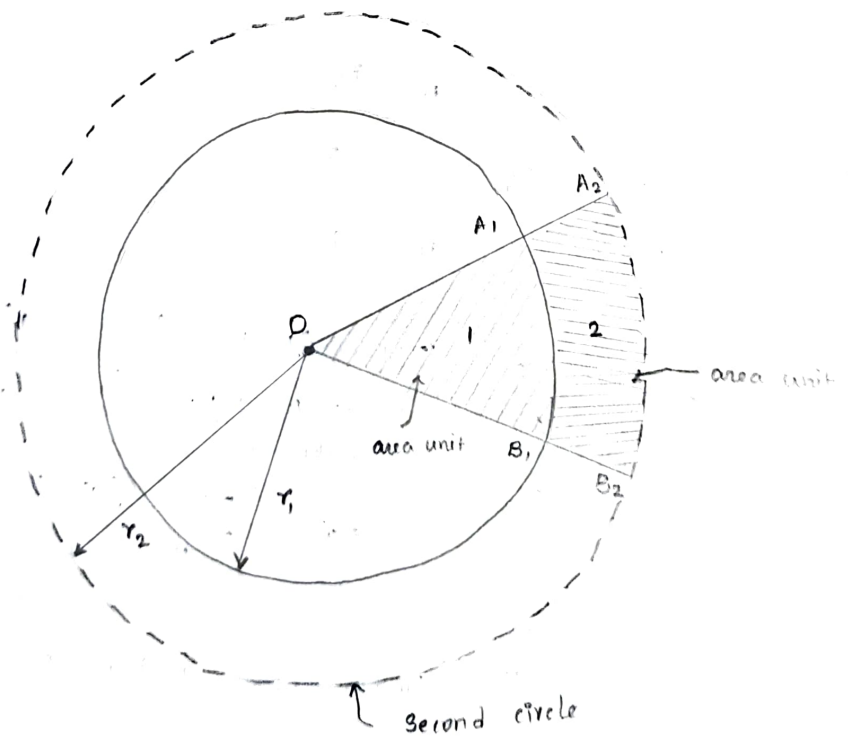
$$= 0.955 [1.154376 + 59.99]$$

$$\therefore (\sigma_z)_c = 58.39 \text{ t/m}^2$$

# NEWMARK'S INFLUENCE CHART

A more accurate method of determining the vertical stress at any point under a uniformly loaded area of any shape is with the help of influence chart or influence diagram originally suggested by Newmark in 1942.

A chart consist of no of circles and radiating lines, is so prepared that the influence of each area unit is the same at the centre of the circles, i.e. each area unit causes the equal vertical stress at the centre of the diagram.



Preparation of Influence Chart:

Let a uniformly loaded circular area of radius  $r$ , cm be divided into 20 sectors (area units) as shown in fig. If  $q$  is the intensity of loading and  $\sigma_z$  is the vertical pressure at a depth  $z$  below the centre of the area each unit such as  $OA_1B_1$  exerts a pressure equal to  $\sigma_z/20$  at the centre.

from equation:

$$\sigma_z = q \left\{ 1 - \left[ \frac{1}{1 + (a/z)^2} \right]^{3/2} \right\}$$

$$\frac{\sigma_z}{20} = \frac{q}{20} \left\{ 1 - \left[ \frac{1}{1 + \left( \frac{r}{z} \right)^2} \right]^{3/2} \right\}$$

$$= i_f q$$

where  $i_f = \text{Influence value} = \frac{1}{20} \left\{ 1 - \frac{1}{1 + \left( \frac{r}{z} \right)^2} \right\}^{3/2}$

$i_f$  be made equal to an arbitrarily fixed value

Say 0.005 we have

$$\frac{q}{20} \left\{ 1 - \left[ \frac{1}{1 + \left( \frac{r}{z} \right)^2} \right]^{3/2} \right\} = 0.005q$$

Selecting the value of  $z = 5$  cm (say), the value of  $r$ , solved from above equation will be 1.35 cm

# SETTLEMENT

Settlement of a structure is its vertical downward movement due to a volume decrease of the soil on which it is built, the settlement is the gradual sinking of a structure due to compression of the soil below.

Components of Settlement:

1. Immediate settlement ( $S_i$ )
2. Primary Consolidation Settlement ( $S_c$ )
3. Secondary consolidation Settlement ( $S_{sc}$ )

$$\text{Total settlement } S_T = S_i + S_c + S_{sc}$$

1. Immediate Settlement ( $S_i$ )

This settlement occurs immediately after the load is applied. This is due to change in shape at constant volume.

2. Primary consolidation Settlement ( $S_c$ )

It occurs due to expulsion of pore water from the voids of a saturated soil. In case of saturated fine grained soils.

3. Secondary consolidation Settlement:

This is also called Secondary compression. It is the change in volume of a fine grained soil due to rearrangement of soil particles at constant effective stress.

## Consolidation:

The compression of a saturated soil under a steady static pressure is known as consolidation.

Normal consolidation, <sup>clay</sup> and Over consolidation; ( $N_c$  &  $O_c$ )

Normally consolidated clay:

A soil deposit that has been subjected to an effective pressure greater than the existing overburden pressure.

Over consolidated clay:

Over consolidated clay is defined as the ratio between preconsolidation pressure to present effective overburden pressure.

## Terzaghi's Theory of One dimensional Consolidation.

The theory concept of the consolidation process was developed by Terzaghi (1923) in the development of the mathematical statement of the consolidation process. the following assumptions are made.

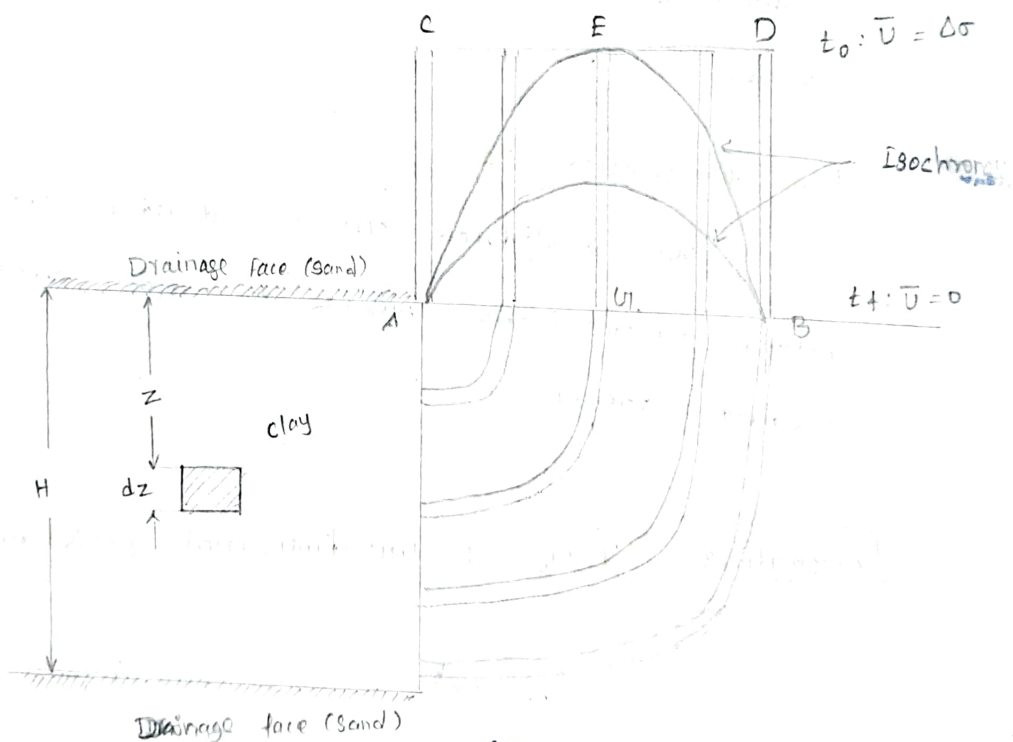
- i) The soil is homogeneous and fully saturated
- ii) Soil water and water are incompressible.
- iii) The deformation of the soil is entirely change in volume.
- iv) Darcy's law for the velocity of flow of water through the soil is valid.

v) coefficient of Permeability is constant during consolidation.

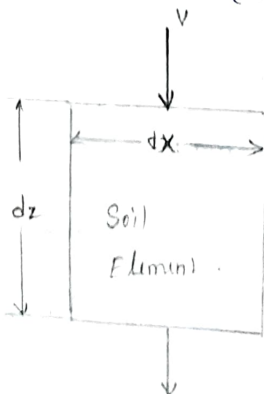
vi) Load is applied in one direction only and deformation occurs only in the direction of the load application.

vii) Excess pore water drains out only in the vertical direction.

viii) The change in thickness of the layer during Consolidation is insignificant.



(a)



$$V + \frac{\partial V}{\partial z} dz$$

(b)

\* Fig (a) shows a clay layer of thickness  $H$ , sandwiched between two layers of sand which serves as drainage faces.

\* When the layer is subjected to a pressure increment  $\Delta\sigma$ , excess hydrostatic pressure is set up in the clay layer. At the time  $t_0$ , the instant of pressure application, whole of the consolidating pressure  $\Delta\sigma$  is carried by pore water so that the initial excess hydrostatic pressure  $\bar{u}_0$  is equal to  $\Delta\sigma$ , and is represented by a straight line  $\bar{u} = \Delta\sigma$  on the pressure distribution diagram.

\* The straight line CED joining the water levels in the piezometric tubes represent this distribution. As water starts escaping into the sand, the excess hydrostatic pressure at the pervious boundaries drops to zero and remains so at all times.

\* After a very great time  $t_f$ , the whole of the excess hydrostatic pressure is dissipated so that  $\bar{u} = 0$ , represented by line AHB.

\* At the intermediate time  $t$  the consolidating pressure  $\Delta\sigma$  is partly carried by water and partly by soil, and the following relationship is obtained:  $\Delta\sigma = \Delta\sigma' + \bar{u}$ .

\* The distribution of excess hydrostatic pressure  $\bar{u}$  at any time  $t$  is indicated by the curve AFB, joining water levels in the piezometric tubes; this curve is known as

Isochrone, and number of such isochrones can be drawn at various time intervals  $t_1, t_2, t_3$  etc. The slope of isochrones at any point at a given time indicates the rate of change of  $\bar{u}$  with depth.

At any time  $t$  the hydraulic head  $h$  corresponding to the excess hydrostatic pressure is given by

$$h = \frac{\bar{u}}{\gamma_w} \rightarrow (1)$$

Hence the hydraulic gradient  $i$  is given by

$$\begin{aligned} i &= \frac{\partial h}{\partial z} \\ &= \frac{1}{\gamma_w} \frac{\partial \bar{u}}{\partial z} \rightarrow (2) \end{aligned}$$

The rate of change of  $\bar{u}$  along the depth of the layer represents the hydraulic gradient. The velocity with which the excess pore water flows at the depth  $z$  is given by Darcy's law

$$\begin{aligned} v &= k i \\ &= \frac{k}{\gamma_w} \frac{\partial \bar{u}}{\partial z} \rightarrow (3) \end{aligned}$$

The rate of change of velocity along the depth of layer is then given by

$$\frac{\partial v}{\partial z} = \frac{k}{\gamma_w} \frac{\partial^2 \bar{u}}{\partial z^2} \rightarrow (4)$$

Consider a small Soil element of size  $dx, dz$  and width  $dy$  perpendicular to the  $xz$  plane, If  $v$  is the velocity of water at the entry into the elements, the velocity at the exit will be equal to  $v + \frac{\partial v}{\partial z} dz$

The quantity of water entering the soil element =  $v dx dy$

The quantity of water leaving the soil element =  $\left(v + \frac{\partial v}{\partial z} dz\right) dx dy$

Hence the net quantity of water  $dq$  leaving out of the soil element per unit time is given by

$$\Delta q = \frac{\partial v}{\partial z} dx dy dz \rightarrow (5)$$

The decrease in the volume of soil is equal to the volume of water squeezed out however from

eqn.  $\Delta v = -m_v V_0 \Delta \sigma' \rightarrow (6)$

where

$V_0$  - Volume of soil element at time  $t_0 = dx dy dz$

Change of volume per unit time is given by

$$\frac{\partial (\Delta v)}{\partial t} = -m_v dx dy dz \frac{\partial (\Delta \sigma')}{\partial t} \rightarrow (7)$$

equating (5) & (7) we get,

$$\frac{\partial v}{\partial z} = -m_v \frac{\partial (\Delta \sigma')}{\partial t} \rightarrow (8)$$

Now  $\Delta \sigma = \Delta \sigma' + \bar{u}$

where  $\sigma$  is constant

$$\therefore \frac{\partial (\Delta \sigma')}{\partial t} = - \frac{\partial \bar{u}}{\partial t} \rightarrow (9)$$

from eqn (8) & (9),

$$\frac{\partial v}{\partial z} = m_v \frac{\partial \bar{u}}{\partial t} \rightarrow (10)$$

Combining Eqn. (9) & (10), we get

$$\frac{\partial \bar{u}}{\partial t} = \frac{k}{m_v \gamma_w} \frac{\partial^2 \bar{u}}{\partial z^2}$$

(or)

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

where

$C_v$  = Co-efficient of Consolidation

$$\therefore C_v = \frac{k}{m_v \gamma_w}$$

$$= \frac{k (1 + e_0)}{\alpha_v \gamma_w} \text{ in cm}^2/\text{sec}$$

Following data were obtained from a consolidation test on a clay sample with double drainage conditions:

$$\text{Voids ratio at } 100 \text{ kPa} = 1.37$$

$$\text{Voids ratio at } 200 \text{ kPa} = 1.25$$

$$\text{Thickness of the Soil Sample at } 100 \text{ kPa} = 20 \text{ mm}$$

$$\text{Co-efficient of Permeability} = 5 \times 10^{-7} \text{ mm/sec}$$

Calculate i) Compression Index

ii) Co-efficient of Volume Change

iii) Co-efficient of consolidation in  $\text{mm}^2/\text{year}$

Sol

$$\sigma'_0 = 100 \text{ kPa} = 100 \text{ kN/m}^2 ; \quad e = 1.37$$

$$\sigma'_1 = 200 \text{ kPa} = 200 \text{ kN/m}^2 ; \quad e = 1.25$$

$$\therefore \Delta \sigma' = \frac{200 - 100}{100} = 100 \text{ kPa} = 100 \text{ kN/m}^2 \quad \Delta e = -0.12$$

$(1.25 - 1.37)$

i) Compression Index ( $C_c$ )

$$C_c = \frac{e_0 - e}{\log_{10} \left( \frac{\sigma'_1}{\sigma'_0} \right)} = \frac{\Delta e}{\Delta \sigma' \log_{10}}$$

$$C_c = \frac{0.12}{\log_{10}(100)}$$

$$C_c = 0.06$$

Co-efficient of Volume Change ( $m_v$ )

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

$$\frac{1}{1 + e_0} \times \frac{\Delta e}{\Delta \sigma'}$$

$$= \frac{0.12}{1 + 1.37} \cdot \frac{1}{100}$$

$$\therefore m_v = 5.06 \times 10^{-4} \text{ m}^2/\text{kN}$$

Co-efficient of Consolidation ( $C_v$ )

$$C_v = \frac{k}{m_v \times \gamma_w}$$

$$= \frac{5.0 \times 10^{-10}}{5.06 \times 10^{-4} \times 9.81}$$

$$\therefore C_v = 1.01 \times 10^{-7} \text{ m}^2/\text{sec}$$

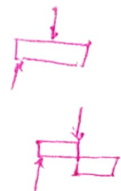
- e) In a Consolidation test Voids ratio decreased from 0.70 to 0.60 when the load was changed from 50 kN/m<sup>2</sup> to 100 kN/m<sup>2</sup>. Compute co-efficient of Compressibility and co-efficient of Volume change.

Sol

$$\Delta e = e_0 - e$$

$$= 0.70 - 0.60$$

$$\therefore \Delta e = 0.10$$



$$\Delta \bar{\sigma} = 100 - 50$$

$$= 50 \text{ kN/m}^2$$

Co-efficient of compressibility ( $a_v$ )

$$a_v = \frac{\Delta e}{\Delta \bar{\sigma}} \Rightarrow \frac{0.10}{50}$$

$$\therefore a_v = 0.002 \text{ m}^2/\text{kN} \quad (2 \times 10^{-3})$$

Co-efficient of Volume Change ( $m_v$ )

$$m_v = \frac{a_v}{1+e_0} \Rightarrow \frac{0.002}{1+0.70}$$

$$\therefore m_v = 0.0012 \text{ m}^2/\text{kN} \quad (1.176 \times 10^{-3})$$

8)

In a consolidation test the Voids ratio of Soil Sample decreases from 1.2 to 1.10 when the pressure is increased from 160 to 320 kN/m<sup>2</sup>. Calculate the co-efficient of Consolidation if the co-efficient of Permeability is  $8.0 \times 10^{-7} \text{ mm/sec}$ .

Given

$$e_0 = 1.20$$

$$\sigma'_0 = 160 \text{ kN/m}^2$$

$$e = 1.10$$

$$\sigma' = 320 \text{ kN/m}^2$$

$$\Delta e = e_0 - e$$

$$\Delta \bar{\sigma} = \sigma'_1 - \sigma'_0$$

$$m_v = \frac{a_v}{1+e_0} \text{ in } \text{m}^2/\text{kN}$$

$$C_v = \frac{k}{m_v \gamma_w}$$

$k$  in  $\text{m}^2/\text{sec}$

4. Find the time required for 50% consolidation in a soil stratum, 9 m thick with a pervious strata on top and bottom. Also determine the coefficient of consolidation given that  $k = 10^{-9} \text{ m/s}$ ,  $e_0 = 1.5$ ,  $a_v = 0.003 \text{ m}^2/\text{kN}$ , Time factor = 0.2

Sol

For 50% consolidation

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

Hence  $T_v$  for  $U = 50\%$  is given by

$$(T_v)_{50} = \left( \frac{50}{100} \right)^2$$

$$= 0.1956$$

$$= 0.196$$

$$t_v = \frac{(T_v)_{50} d^2}{C_v}$$

$$d = \frac{1}{2} \times \text{thickness of soil strata}$$

$$= \frac{1}{2} \times 9$$

$$d = 4.5 \text{ m}$$

$$t_v = \frac{0.196 \times 4.5^2}{C_v}$$

$$C_v = \frac{k}{m_v \times \gamma_w}$$

Co-efficient of Volume Change (mv)

$$M_v = \frac{a_v}{1 + e_0} = \frac{0.003}{1 + 1.50}$$

$$m_v = 1.2 \times 10^{-3} \text{ m}^2/\text{kN}$$

Co-efficient of Consolidation (Cv)

$$C_v = \frac{1.0 \times 10^{-9}}{1.2 \times 10^{-3} \times 9.81}$$

$$\therefore C_v = 8.49 \times 10^{-8} \text{ m}^2/\text{sec}$$

time for 50% of consolidation

$$t_{50} = \frac{(T_v)_{50} d^2}{C_v}$$

$$= \frac{0.196 \times 4.5^2}{8.49 \times 10^{-8}} \Rightarrow 46.98 \times 10^6 \text{ sec}$$

$$46.98 \times 10^6 \text{ sec} = 544 \text{ days}$$

$$\therefore t_{50} = 544 \text{ days}$$

5. ✓ Two clay specimens A and B of thickness 20mm and 30mm have voids ratios of  $e_1$  0.68 and  $e_0$  0.72 respectively under a pressure of 200 kPa. If the voids ratios of the specimens are 0.55 and 0.65 respectively when the pressure was increased to 350 kPa. Find the ratio of co-efficient of Permeability of two specimen if the time required by the specimen B to reach the given degree of consolidation is three times that required by specimen A.

Sol

Given data:

Clay Specimen A

Clay Specimen B

$$H = 20 \text{ mm}$$

$$H = 30 \text{ mm}$$

$$e_0 = 0.68$$

$$e_0 = 0.72$$

$$\bar{\sigma}_0 = 200 \text{ kPa}$$

$$\bar{\sigma}_0 = 200 \text{ kPa}$$

$$e_1 = 0.55$$

$$e_1 = 0.65$$

$$\bar{\sigma}_1 = 350 \text{ kPa}$$

$$\bar{\sigma}_1 = 350 \text{ kPa}$$

$$t_B = 3 t_A$$

To determine:

$$\frac{K_A}{K_B}$$

Sol

$$C_v = \frac{k}{m_v \times \gamma_w} \checkmark$$

For Specimen A

$$C_{vA} = \frac{K_A}{m_{vA} \times \gamma_w} \checkmark$$

$$K_A = C_{vA} \times m_{vA} \times \gamma_w$$

For Specimen B

$$K_B = C_{vB} \times m_{vB} \times \gamma_w$$

$$\frac{K_A}{K_B} = \frac{C_{vA} \times m_{vA} \times \gamma_w}{C_{vB} \times m_{vB} \times \gamma_w}$$
$$= \frac{C_{vA} \times m_{vA}}{C_{vB} \times m_{vB}}$$

$$M_{vA} = \frac{a_v}{1 + e_0} \checkmark$$

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

$$= \frac{(0.68 - 0.55)}{(350 - 200)}$$

$$\therefore a_v = 8.6 \times 10^{-4} \text{ m}^2/\text{kN}$$

$$M_{vA} = \frac{8.6 \times 10^{-4}}{1 + 0.68}$$

$$\therefore M_{vA} = 5.11 \times 10^{-4} \text{ m}^2/\text{kN}$$

$$C_{vA} = ?$$

$$\left( T_A = \frac{t_A \times C_{vA}}{H_A^2} \right)$$

$$C_{vA} = \frac{T_A \times H_A^2}{t_A} \checkmark$$

$$\therefore C_{vA} = \frac{1 \times (0.020)^2}{t_A} \rightarrow \textcircled{1}$$

$$= \frac{4 \times 10^{-4}}{t_A}$$

$$(T_A = T_B = 100\% \text{ Consolidation} = 1)$$

For Specimen B

$$C_{vB} = \frac{T_B \times H_B^2}{t_B}$$

$$= \frac{1 \times (0.030)^2}{3 t_B} = \frac{9 \times 10^{-4}}{3 t_B} \rightarrow \textcircled{2}$$

$$\text{Eqn } \frac{\textcircled{1}}{\textcircled{2}} \Rightarrow$$

$$\frac{C_{vA}}{C_{vB}} = \frac{4 \times 10^{-4}}{t_A} \times \frac{3 t_B}{9 \times 10^{-4}}$$

$$= \frac{12}{9}$$

$$\frac{M_{vA}}{M_{vB}}$$

$$= \frac{5.11 \times 10^{-4}}{2.71 \times 10^{-4}} = 1.885$$

$$\frac{k_A}{k_B} = \frac{12}{9} \times \frac{5.11}{2.71} = 2.51$$

# SHEAR STRENGTH

## Shear Strength

- i) The stress of a material is the greatest stress it can sustain
- ii) The safety of any geotechnical structure is dependent on the strength of the soil.
- iii) If the soil fails the structure founded on it can collapse.

✓ Thus Shear Strength is the capacity of a material to resist the internal and external forces which slide past each other.

## Shear Strength of a Soil

- \* Shear Strength of a soil is its resistance to shearing stress
- ✓ \* It is measure of the soil resistance to deformation by continuous displacement of its individual soil particles.
- \* Shear failure occurs when the stresses between the particles are such that they slide or roll on each other
- \* Shear Strength of a soil is its maximum resistance to shear stresses just before the failure.

## UNIT - IV

### SHEAR STRENGTH

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✓ \* It is measure of the soil resistance to deformation by continuous displacement of its individual soil particles.

\* Shear failure occurs when the stresses between the particles are such that they slide or roll on each other.

\* Shear strength of a soil is its maximum resistance to shear stresses just before the failure.

## Components of Shear Strength:

- i) Cohesion between particles (Stress independent component)
- ii) Frictional resistance and interlocking between particles (Stress dependent component)

## Mohr Strength Theory:

The important points in Mohr strength Theory:

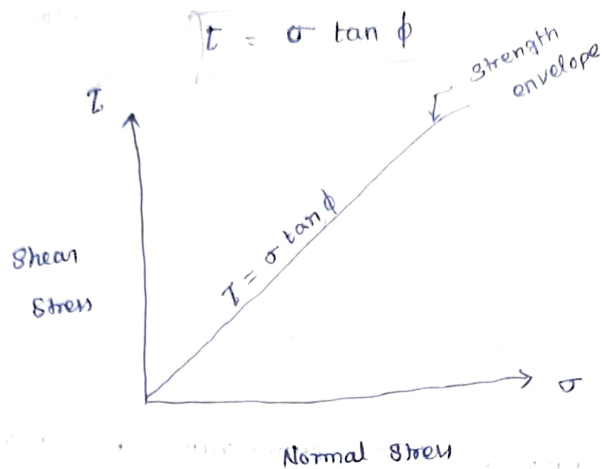
- i) Materials fails essentially by Shear
- ii) The ultimate shear stress depends on the normal stress on the potential failure plane and the properties of material.
- iii) In a three dimensional stress system  $(\sigma_1, \sigma_2, \sigma_3)$  the failure criterion is independent of intermediate principal stress  $\sigma_2$

Normal stress ( $\sigma$ ) acts normal to the plane and tends to compresses soil grains towards each other. Shear stress ( $\tau$ ) acts tangential to the plane and tends to slide grains relative to each other.

If  $\sigma$  &  $\tau$  are the normal and shear stress on any plane and  $\beta$  the angle of obliquity is the angle made by the resultant of  $\sigma$  &  $\tau$  normal to the plane

$$\tau = \sigma \tan \beta$$

On the potential failure plane 'P' will have limiting value  $\phi$  and then shear strength will be represented by



We note that Mohr did not consider cohesion of soil and hence this theory is applicable only for cohesionless soil.  $\phi$  is called angle of internal friction.

### Mohr - Coulomb Failure Theory ✓

The theory was first expressed by Coulomb (1776) and later generated by Mohr (1900)

Mohr (1900) failure theory is based on the hypothesis that material fail when "the Shear Stress" on the failure plane at failure reaches a value which is a unique function of the normal stress on that plane that is

$$\tau_f = F(\sigma)$$

Coulomb assumed the relation between  $\tau_f$  &  $\sigma$  to be linear and gave

$$\tau_f = c + \sigma \tan \phi$$

Coulomb observed that one component of the shearing strength called the intrinsic cohesion (Sometimes called the apparent cohesion) is constant for a given soil and is independent of the applied stress.

The other component namely the frictional resistance somewhat similar to sliding friction in solids, varies directly as the magnitude of the normal stress on the plane of rupture.

where

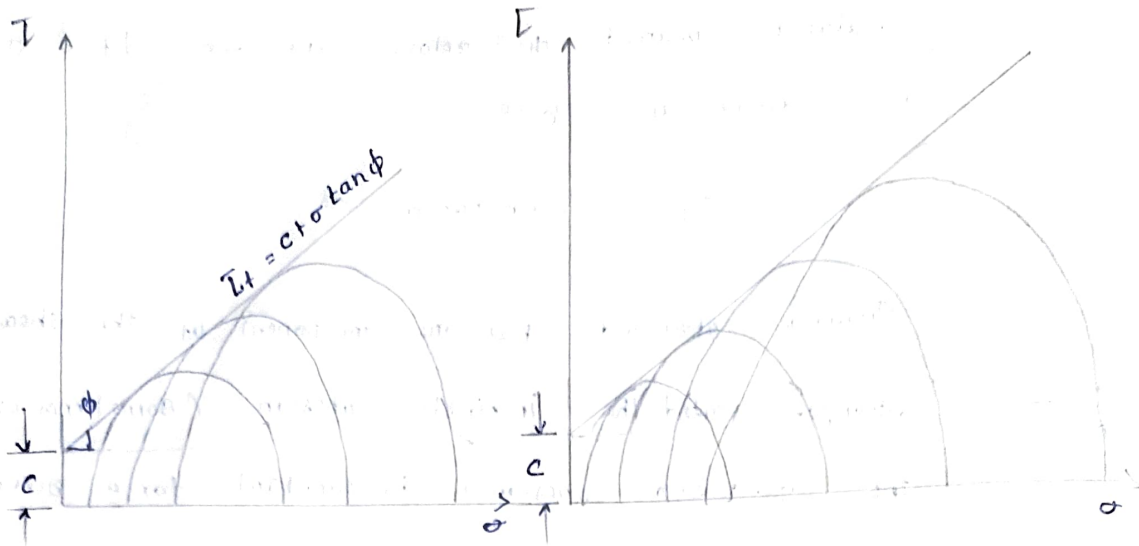
$\tau_f$  - Shear strength of soil

$c$  - Apparent Cohesion

$\sigma$  - Normal stress on the plane of rupture

$\phi$  - angle of internal friction.

where  $c$  &  $\phi$  are the shear strength parameters Mohr generalized the strength envelop also known as failure envelope as a curve which becomes flatter with increasing normal stress as shown in figure.

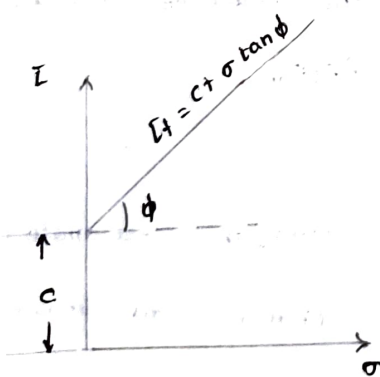


a) Coulomb Envelope

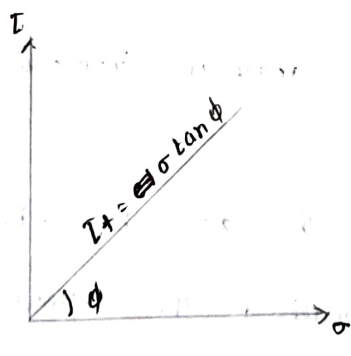
b) Mohr Envelope

Based on the values of shear strength parameters soils can be described as

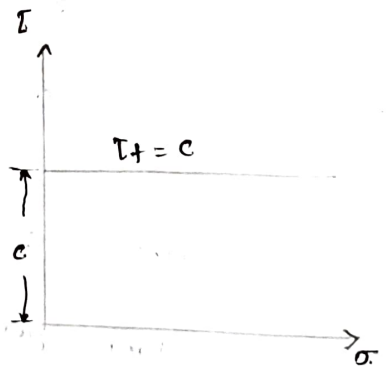
- i) Cohesive soil ( $c - \phi$ )
- ii) Cohesionless soil ( $c = 0$ )
- iii) Purely cohesive soil ( $\phi = 0$ )



a) ( $c - \phi$ ) soil



b)  $c = 0$



c)  $\phi = 0$

Strength envelope for the three types of soils.

## Determination of Mohr Shear Strength Parameters:

Shear strength parameters are determined in the laboratory by conducting shear tests and using the results we obtain failure envelope. The tests are conducted on undisturbed soil samples obtained from the field and taking care to simulate the field drainage conditions.

Based on the method of application of loads the shear tests commonly used in the laboratory.

- i) Direct shear test
- ii) Triaxial shear test
- iii) Unconfined compression test (UCC test)
- iv) Vane shear test

Based on drainage condition the shear tests are classified as

- i) Undrained test - UU test
- ii) Consolidated Undrained test - CU test
- iii) Drained test - CD test

1. A direct shear test was carried out on a cohesive soil sample and the following results were obtained

Normal stress ( $\text{kN/m}^2$ )      160      240

Shear stress ( $\text{kN/m}^2$ )      110      120

What would be the deviator stress at failure of a triaxial test carried out on the same soil with cell pressure  $150 \text{ kN/m}^2$

Sol

i)

$$\tau_f = c + \sigma \tan \phi$$

$$110 = c + 160 \tan \phi$$

$$120 = c + 240 \tan \phi$$

$$\tau_{f1} = 110 \text{ kN/m}^2$$

$$\tau_{f2} = 120 \text{ kN/m}^2$$

$$\sigma_1 = 160 \text{ kN/m}^2$$

$$\sigma_2 = 240 \text{ kN/m}^2$$

$$\sigma_3 = 150 \text{ kN/m}^2$$

Solving the above equations, eqn (2) - (1)  $\Rightarrow$

$$c = 90 \text{ kN/m}^2$$

$$\phi = 7^\circ 7' 30''$$

$$10 = 0 + 80 \tan \phi$$

$$10 = 80 \tan \phi$$

$$\tan \phi = \frac{10}{80}$$

ii) Deviator stress

$$\phi = \tan^{-1} \left( \frac{10}{80} \right)$$

$$\phi = 7^\circ 7' 30''$$

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

$$\sigma_3 = 150 \text{ kN/m}^2$$

$$\sigma_1 = \sigma_3 + \sigma_d$$

$$\alpha = 45^\circ + \frac{\phi}{2}$$

$$= 48^\circ 31' 45''$$

$$\sigma_3 + \sigma_2 = 150 \tan^2 48^\circ 31' 45'' + 2 \times 90 \tan 48^\circ 31' 45''$$

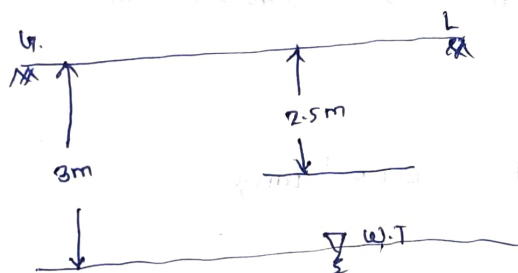
$$150 + \sigma_2 = 392.83$$

$$\sigma_2 = 242.83 \text{ kN/m}^2$$

2. In a formation of cohesionless soil the water Table is at a depth of 3m. The degree of Saturation may be taken as 0.5 and average void ratio 0.5,  $G_s = 2.7$ ,  $\phi = 30^\circ$ . Calculate the potential <sup>co</sup> effective Shear Strength on a horizontal plane at a depth of 2.5m below the Surface. Calculate the modified Value of Shear Strength if the water table reaches the ground Surface.

Sol

17



water Table is @ 3m below the Surface. Shear Strength at a depth of 2.5m below the Surface.

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

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

$$\sigma = 2 \times \sigma_d$$

$$\sigma_d = \left( \frac{G_s}{1+e} \cdot \gamma_w \right) 1 + u$$

con

$$\gamma = \frac{G_s}{1+e} \cdot \gamma_w$$

$$w = \frac{e.s}{u1} \Rightarrow \frac{0.5 \times 0.5}{2.7}$$

$$w = 0.0926$$

$$\gamma = \left( \frac{2.7 \times 9.81}{1+0.5} \right) (1 + 0.926)$$

$$\gamma = 19.293 \text{ kN/m}^3$$

$$\sigma = 2.5 \times 19.293$$

$$\sigma = 48.23 \text{ kN/m}^2$$

$$u = 0$$

$$\sigma' = \sigma \Rightarrow 48.23 \text{ kN/m}^2$$

$$\tau_f = c + \sigma' \tan \phi$$

$$= 0 + 48.23 \tan 30^\circ$$

$$\therefore \tau_f = 27.85 \text{ kN/m}^2$$

ii) When the water Table reaches Ground Surface :

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

$$= \left( \frac{2.7 + 0.5}{1 + 0.5} \right) \times 9.81$$

$$\therefore \gamma_{sat} = 20.93 \text{ kN/m}^3$$

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

$$= 2.6 (20.93 - 9.81)$$

$$\therefore \sigma' = 27.8 \text{ kN/m}^2$$

$$\tau_f = c + \sigma' \tan \phi$$

$$= 0 + 27.8 \tan 30^\circ$$

$$\therefore \tau_f = 16.05 \text{ kN/m}^2$$

3.

A cylindrical specimen of soil fails under an axial vertical stress  $160 \text{ kN/m}^2$ . When it is unconfined. The failure plane makes an angle of  $50^\circ$  with horizontal. Calculate the value of  $c$  and  $\phi$

Sol

$$\sigma_1 = 160 \text{ kN/m}^2$$

$$\alpha = 50^\circ$$

$$\sigma_3 = 0$$

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

$$\sigma_1 = 2c \tan \alpha \quad (\sigma_3 = 0)$$

$$c = \frac{\sigma_1}{2 \tan \alpha} \Rightarrow \frac{160}{2 \tan 50^\circ}$$

$$c = \frac{57.12}{\text{—}} \text{ kN/m}^2$$

$$\alpha = 45^\circ + \frac{\phi}{2}$$

$$\phi = 10^\circ$$

4. When an undrained triaxial compression test was conducted on specimens of clayey silt. The following results were obtained

| Specimen                        | 1   | 2   | 3   |
|---------------------------------|-----|-----|-----|
| $\sigma_3$ (kN/m <sup>2</sup> ) | 17  | 44  | 56  |
| $\sigma_1$ (kN/m <sup>2</sup> ) | 157 | 204 | 225 |
| $U$ (kN/m <sup>2</sup> )        | 12  | 20  | 22  |

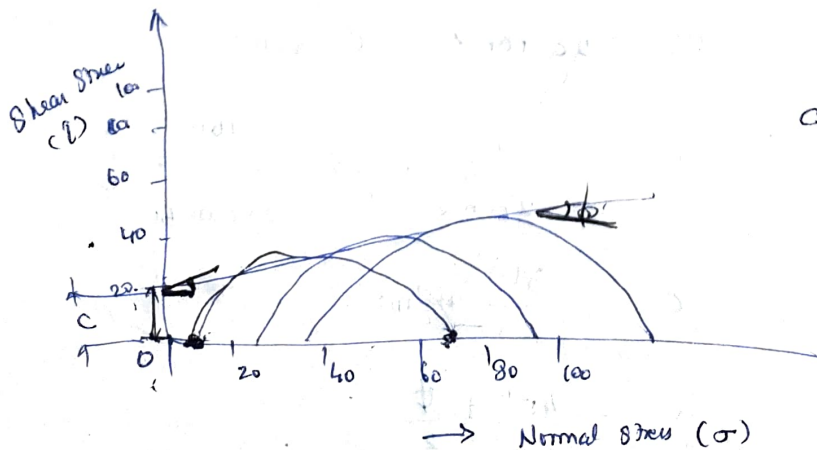
Determine the value of effective shear strength parameters

We know that

$$\sigma_1' = \sigma_1 - U$$

$$\sigma_3' = \sigma_3 - U$$

| Specimen No                      | 1   | 2   | 3   |
|----------------------------------|-----|-----|-----|
| $\sigma_3'$ (kN/m <sup>2</sup> ) | 5   | 24  | 34  |
| $\sigma_1'$ (kN/m <sup>2</sup> ) | 145 | 184 | 203 |



$$C' = 38 \text{ kN/m}^2$$

$$\phi' = 24^\circ$$

5 A sample of clay taken from a natural stratum was found to be partially saturated and when tested in the laboratory gave the following results: Compute the degree of saturation, specific gravity of soil particles = 2.6  
 wet weight of sample = 2.50 N, dry weight of sample = 2.10 N and Volume of sample = 150 cm<sup>3</sup>

Sol:

$$G = 2.60$$

$$\text{Wet weight (W)} = 2.50 \text{ N}$$

$$\text{Dry weight (W}_d\text{)} = 2.10 \text{ N}$$

$$\text{Volume } V = 150 \text{ cm}^3$$

$$\begin{aligned} \text{Water Content (w)} &= \frac{W - W_d}{W_d} \times 100 \Rightarrow \frac{2.5 - 2.1}{2.1} \times 100 \\ &= \frac{0.40}{2.10} \times 100 \end{aligned}$$

$$\boxed{w = 19.05 \%}$$

$$\text{Bulk unit weight } (\gamma) = \frac{W}{V} = \frac{2.50}{150} \Rightarrow 0.01638 \text{ N/cm}^3$$

$$\boxed{\gamma = 16.38 \text{ kN/m}^3}$$

$$\text{Dry unit weight } \gamma_d = \frac{\gamma}{1+w} = \frac{16.38}{1 + 0.1905} \Rightarrow 0.01376 \text{ N/cm}^3$$

$$\boxed{\gamma_d = 13.76 \text{ kN/m}^3}$$

(or)

$$\gamma_d = \frac{W_d}{V} = \frac{2.10}{150} \Rightarrow 0.01376 \text{ N/cm}^3$$

$$\boxed{\gamma_d = 13.76 \text{ kN/m}^3}$$

$$\begin{aligned} \text{But } \gamma_d &= \frac{G \cdot \gamma_w}{1+e} \\ 13.76 &= \frac{2.6 \times 9.81}{1+e} \end{aligned}$$

$$1+e = \frac{2.6 \times 9.81}{13.76}$$

$$\boxed{e = 0.854}$$

$$e s = w G$$

$$s = \frac{w \cdot G}{e} = \frac{0.1905 \times 2.6}{0.854} \times 100$$

$$\boxed{s = 58 \%}$$

a) Consistency Index ( $I_c$ )

$$I_c = \frac{w_L - w}{I_p} = \frac{0.62 - 0.34}{0.32}$$

$$I_c = 0.88$$

The consistency index ( $I_c$ ) of a soil is between 0 - 1 that the soil is in a stage b/w liquid and plastic limit

b) Shrinkage limit ( $w_s$ )

$$w_s = \frac{e}{\gamma}$$

$$= \frac{0.70}{2.70}$$

$$= 0.26 \times 100$$

$$w_s = 26\%$$

c) Degree of Saturation ( $S$ )

$$e = \frac{w \cdot \gamma}{S}$$

$$S = \frac{w \cdot \gamma}{e}$$

$$= \frac{0.34 \times 2.70}{0.70}$$

$$= 1.31 \times 100$$

$$\therefore S = 131\%$$

Sol:

Specimen I

$$\sigma_d = 870 \text{ kPa}$$

$$\sigma_3 = 300 \text{ kPa}$$

$$\sigma_1 = \sigma_d + \sigma_3 \Rightarrow 1170 \text{ kPa}$$

Specimen II

$$\sigma_d = 1400 \text{ kPa}$$

$$\sigma_3 = 500 \text{ kPa}$$

$$\sigma_1 = \sigma_3 + \sigma_d \Rightarrow 1900 \text{ kPa}$$

① Shear strength Parameters:

$$\sigma_1 = \sigma_3 N\phi + 2c\sqrt{N\phi}$$

$$1170 = 300 N\phi + 2c\sqrt{N\phi} \rightarrow \textcircled{1}$$

$$1900 = 500 N\phi + 2c\sqrt{N\phi} \rightarrow \textcircled{2}$$

$$\text{eqn } \textcircled{2} - \textcircled{1} \Rightarrow$$

$$730 = 200 N\phi$$

$$N\phi = 3.65$$

$$N\phi = \tan^2(45 + \phi/2)$$

from eqn ② we get

$$1900 = 500 \times 3.65 + 2c \times \sqrt{3.65}$$

$$c = 19.63 \text{ kN/m}^2 \quad \text{but } c = 37.5 \text{ kN/m}^2$$

$$1900 = 1825 + (2c \times 1.91)$$

$$\frac{75}{1.91} = 2c$$

$$39.26 = 2c$$

$$c = 19.63 \text{ kN/m}^2$$

$$3.65 = \tan^2(45 + \phi/2)$$

$$\sqrt{3.65} = \tan(45 + \phi/2)$$

$$\phi = 20^\circ$$

$$1.91 = \tan(45 + \phi/2) \Rightarrow 62.36 = 45 + \phi/2$$

$$\phi/2 = 17.36$$

$$\phi = 34.72$$

$$\phi = 34.43'$$

② Deviator stress ( $\sigma_d$ ) when  $\sigma_3 = 700 \text{ kPa}$

$$\sigma_1 = \sigma_3 N\phi + 2c\sqrt{N\phi}$$

$$\sigma_1 = 700 \times 3.65 + 2 \times 37.5 \sqrt{3.65}$$

$$\sigma_1 = 2698.29 \text{ kPa}$$

$$\sigma_d = \sigma_1 - \sigma_3 \Rightarrow 2698 - 700 \Rightarrow 1998 \text{ kPa}$$

6. The results of a series of CV tests on undisturbed samples of an oven dried consolidated clay were as below

|                                | Trial I | Trial II | Trial III | Trial IV |
|--------------------------------|---------|----------|-----------|----------|
| Cell Pressure ( $\sigma_3$ )   | 100     | 200      | 400       | 600      |
| Deviator Stress ( $\sigma_d$ ) | 300     | 400      | 610       | 850      |
| Pore water Pressure (U)        | -45     | -15      | 50        | 110      |

Determine the shear strength parameter in terms of effective stress.

Sol

|                                                     | Trial 1 | Trial 2 | Trial 3 | Trial 4 |
|-----------------------------------------------------|---------|---------|---------|---------|
| Eff. Cell Pressure ( $\sigma_3'$ ) = $\sigma_3 - U$ | 145     | 215     | 350     | 490     |
| $\sigma_1 = \sigma_3 + \sigma_d$                    | 400     | 610     | 1010    | 1450    |
| $\sigma_1' = \sigma_1 - U$                          | 445     | 625     | 960     | 1340    |

Plot graph between  $\sigma_1'$  and  $\sigma_3'$  from graph.

$$C = 40 \text{ kN/m}^2$$

$$\phi = 26^\circ 33' \text{ (or } 21^\circ 33')$$

$$\sigma_1' = \sigma_1 - U$$

$$\sigma_3' = \sigma_3 - U$$

$$\sigma_1 = \sigma_3 + \sigma_d$$

30

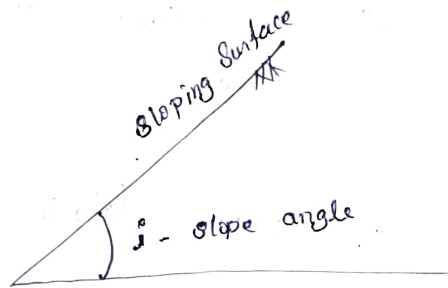
## UNIT - V

# SLOPE STABILITY

### Introduction:

A exposed ground surface at an angle ( $i$ ) with the horizontal is called a slope. An earth slope is an unsupported, inclined surface of a soil mass. Earth Slope may be found in nature or may be man made.

■



Earth slopes are formed for railway embankments, earth dams, canal banks, and river training works. It is essential to check the stability of these earth slopes since their failure may lead to loss of human life and property.

### Slope failure Mechanism:

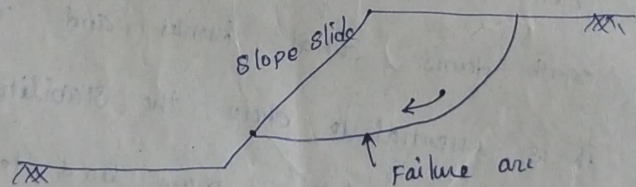
- i) The action of gravitational forces
- ii) Seepage forces within the soil mass.
- iii) Intense Rainfall
- iv) Water level change
- v) Seepage water flow
- vi) Earthquake force.
- vii) Human activity.

## Causes of Slope failure:

- i) Erosion — wind and water
- ii) Steady seepage
- iii) Sudden draw down
- iv) Rainfall
- v) Earthquakes
- vi) External loading
- viii) Constructional activities at the toe of the slope.

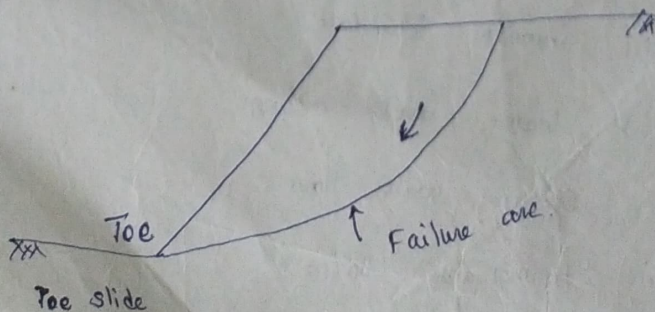
## Types of Slope failure:

### a) Face or Slope failure.



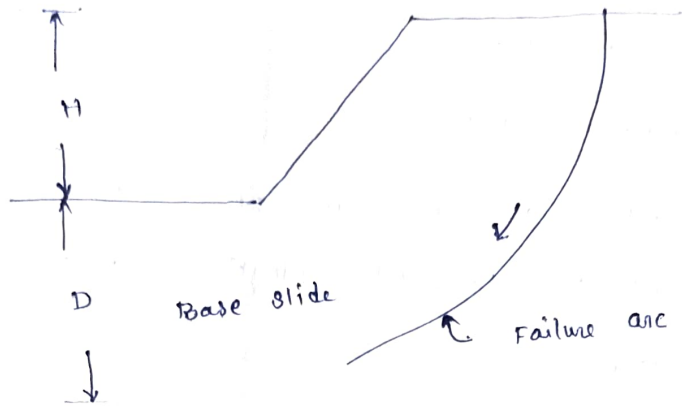
This type of failure occurs when the slope angle is large and when the soil at a toe portion is strong.

### b) Toe failure:



In this case the failure surface passes through the toe. This occurs when the slope is steep and homogeneous.

c) Base Failure:



In this case the failure surface passes below the toe. This generally occurs when the soil below the toe is relatively weak and soft.

The ratio of the total depth ( $H+D$ ) to depth ' $H$ ' denoted by  $D_f$  depth factors,

For base failure  $D_f > 1$

For toe failure  $D_f = 1$

Types of Slopes:

i) Infinite Slope

ii) Finite Slope

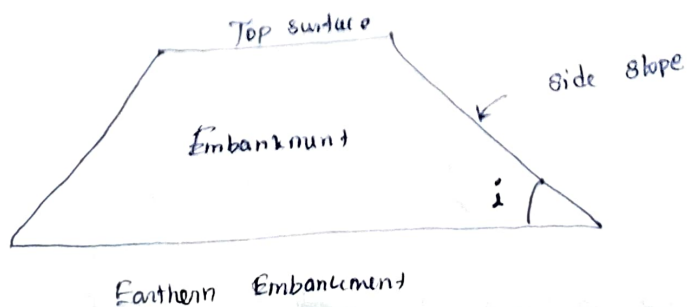
19/08/2022  
i) Infinite Slope :

They have dimensions that extend over great distances and the soil mass is inclined to the horizontal. It is the one which represents the boundary surface of a semi-infinite soil mass inclined to the horizontal and having constant soil properties for all identical depths below the surface.

Slope extending to infinity do not exist in nature. However in practice if the height of the slope is very large it may be considered as infinite slope.

19/08/2022  
ii) Finite Slope :

A slope of limited extent, bounded by a base and top surface is called finite slope. A finite slope is one which with a base and top surface, the height being limited. The examples are Embankments, earth dams cuts etc.



For this geometry, any mass movement or failure at the slope usually involves the displacement of a soil mass having a length in many times greater than its depth.

In granular soils for stability from plane failure concept, for the infinite slope we get.

$$\begin{aligned}\text{Factor of safety} &= \frac{\text{Restraining force}}{\text{Sliding force}} \\ &= \frac{kl \cos \beta \tan \phi}{kl \sin \beta}\end{aligned}$$

The normal component contribute restraining force as

$$E = \sigma \tan \phi$$

$$F = \frac{\tan \phi}{\tan \beta}$$

For limiting equilibrium  $F=1$ ,  $\beta = \phi$

From this,

1. The weight 'kl' of soil does not affect the stability of the slope.
2. The safe angle for the slope is the same the soil is dry or submerged
3. The embankment can be any height.

When  $\beta = \phi$  stability is critical:

In an infinite slope of sand subjected to steady seepage flow parallel to the ground surface, the effective stresses and pore water pressures at corresponding depth are the same and thus the factor of safety

$$F = \frac{\gamma_{sub}}{\gamma_{sat}} \frac{\tan \phi}{\tan \beta}$$

$$\therefore F = \frac{1}{2} \frac{\tan \phi}{\tan \beta}$$

In the case of clay slopes,

for no seepage condition

$$F = \frac{\tan \phi}{\tan \beta} + \frac{2c}{\gamma_{sat} H \sin 2\beta}$$

for full seepage

$$F = \frac{\gamma_{sub}}{\gamma_{sat}} \frac{\tan \phi}{\tan \beta} + \frac{2c}{\gamma_{sat} H \sin 2\beta}$$

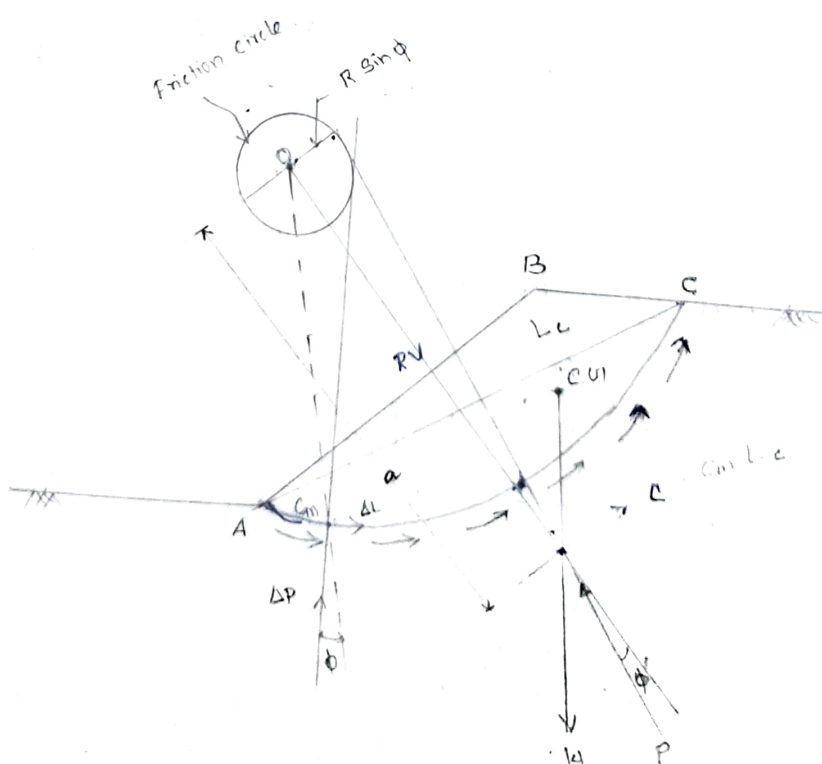
### Stability analysis of Finite Slopes:

1. Friction Circle method.
2. Swedish circle method or Fellenius circle method.
3. Taylor stability number.
4. Bishop method.

## Friction Circle Method:

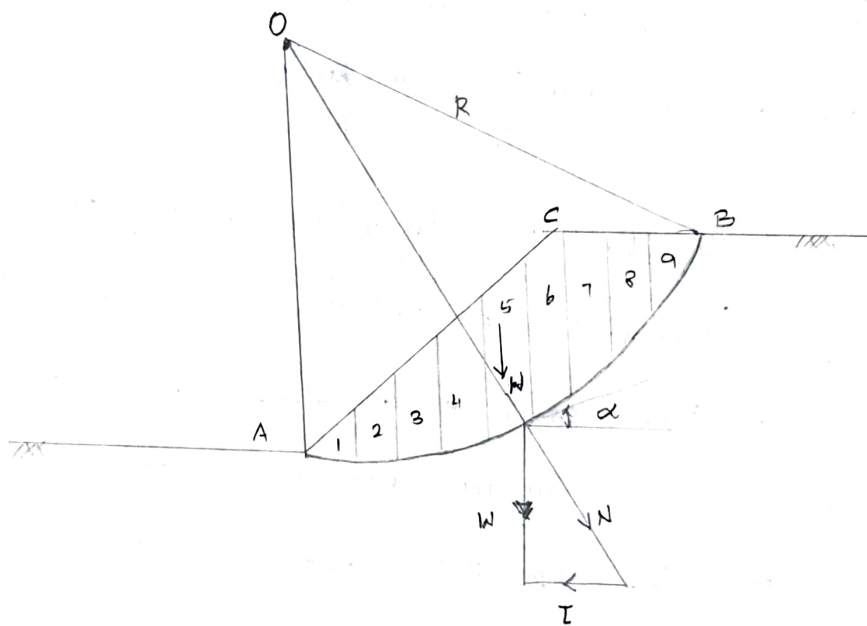
This method is based on the total stress analysis but it enables the angle of shearing resistance to be taken into account. It should be noted that some soils, such as saturated silt and unsaturated clays, do

The friction circle method also assumes a circular slip surface, from fig a slope with a trial slip surface in the form of a circular arc of radius  $R$  with its centre at  $O$ . The three forces which are considered in the equilibrium of the sliding mass  $ABCA$  are the weight  $W$  of the sliding mass, the total frictional force  $P$  on the slip surface, and the cohesive resistance  $C$  along the slip surface.

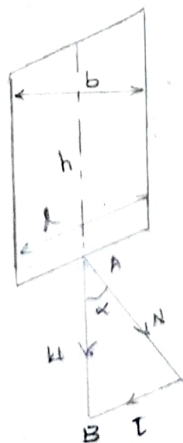


Fellenius (or) Swedish Circle method :

In this method the slip circle is divided into a number of vertical slices of equal width, as shown in fig. The number of slices may vary from 6 to 12, when hand computations are to be used.



c- $\phi$  analysis . method of slices



Let the slip circle arc AC (2) be considered to be made up of number of elementary arcs of length  $\Delta L$ . The elementary cohesive force acting along this element of length  $\Delta L$  opposing the probable movement of the soil mass is  $c_m \Delta L$ .  $c_m$  is mobilised cohesion, which is assumed to be a constant along the slip surface.

The total cohesive force is considered to be made up of elementary cohesive forces  $c_m \Delta L$  representing a force polygon. The closing line of which must represent the magnitude and direction of the resultant cohesive force.

If  $L_c$  is the length of chord AC, the magnitude of the resultant cohesive resistance is  $c_m L_c$ . The position of the resultant  $c_m L_c$  is determined by equating the sum of the moments of all the elementary cohesive forces along the slip circle about the centre of rotation  $O$ .

$$\sum c_m \Delta L R = c_m L_c \cdot a$$

$$c_m L R = c_m L_c a$$

$$\because \Delta L = L$$

$$\therefore a = \frac{L}{L_c} \cdot R$$

Where  $a$  - moment arm of the cohesive force.

$L_c$  - length of chord

$L$  - length of slip circle arc

$R$  - Radius.

The factor of safety of the slope with respect to cohesion.

$$F_c = \frac{C_u}{c_m}$$

$C_u$  - cohesive force

$c_m$  - mobilised cohesion.

In the conventional method, the forces between the slices are neglected and each slice is considered to be an independent column of soil of unit thickness.

If slice no 5 is taken, the weight of the slice  $W$  is calculated as equal to  $\gamma h b$

$$W = \gamma h b$$

$\gamma$  - bulk unit weight of soil

$h$  - average height of slice

$b$  - width

The line of action which can be taken to pass through the mid-width point of the slice, meets the base of the slice of length  $L$  at  $A$ . The weight  $W$  is plotted as a vector  $AB$  and then resolved into its normal and tangential components  $N$  and  $T$  respectively, at  $A$ . Since the normal component  $N$  passes through  $A$  and the centre of rotation  $O$ .

Considering the whole slip surface  $AB$  of length  $L$ , the total driving and resisting forces are

$$\text{Driving forces} = \sum T$$

$$\text{Resisting forces} = \sum c' + \sum N \tan \phi'$$

$$= c' L + \sum N \tan \phi'$$

$$\text{Driving moment} = \sum T R$$

$$\text{Resisting moment} = c' L R + \sum N \tan \phi' R$$

The factor of Safety against sliding  $F$  is

$$F = \frac{C'L + \sum N \tan \phi'}{\sum T}$$

Where  $N = W \cos \alpha$

$$T = W \sin \alpha$$

Now  $F$  is

$$F = \frac{C'L + \tan \phi' \sum W \cos \alpha}{\sum W \sin \alpha}$$

The method of slices can be used for homogeneous and can be used where seepage is taking place and pore pressures are present in the soil.

Stability Number:  $(N)$

The ratio of  $c/\gamma H$  is known as the stability number and is given the symbol 'N'

Where  
 $c$  - cohesion mobilised with regard to total stress  
 $\gamma$  - density of soil  
 $H$  - Vertical height of embankment

$$N = \frac{c}{\gamma H}$$