# EARTH PRESSURE

→ In case of fluids, hydrostatic pressure acts equally in all directions at any given depth below the surface, thus lateral fluid pressure is equal to vertical pressure only.

→ However in the case of soils or any such materials eg food grains, coal, which posses shear strength, the lateral pressure is not equal to vertical pressure but is only related to it & depends upon magnitude of vertical pressure & lateral ~~pressure~~ strain conditions & nature of soil.

→ Structures which are used to hold back a soil mass are called RETAINING STRUCTURES

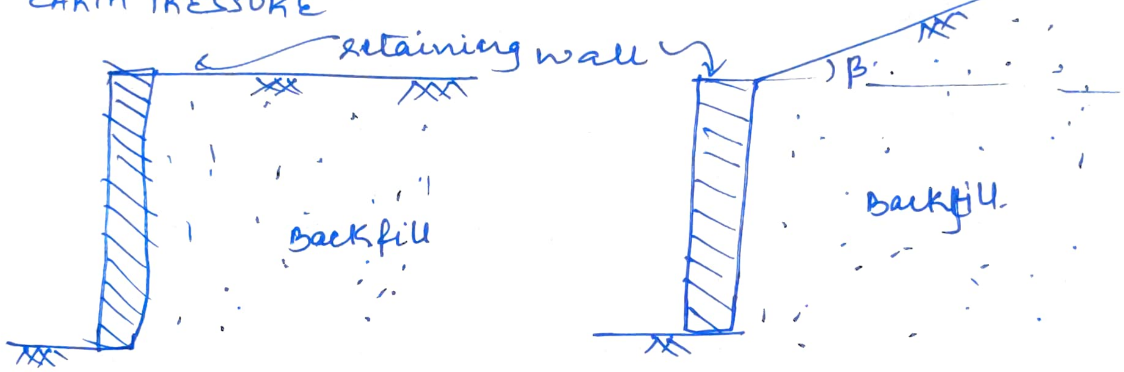
eg: • Retaining wall
- Sheet pile wall
- Sheeting in excavation
- basement wall

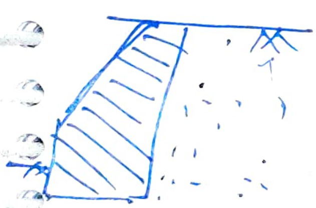
→ The retaining $st^r$ help in maintaining the ground surface at different elevations on either side of it, without such a $st^r$, the soil at higher elevation would ~~be~~ to tend to move down till it acquires it stable neutral configuration.

→ The materials that is retained by these walls / $st^r$ is termed as BACKFILL which have it top surface horizontal or inclined.

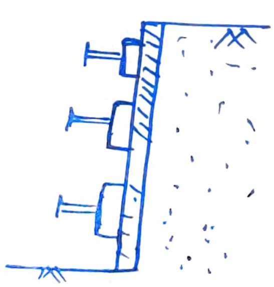
→ The portion of backfill lying above the horizontal plane at the elevation of the top of wall is called SURCHARGE and it inclination to the horizontal is called "SURCHARGE ANGLE" ($\beta$)

→ Force exerted by backfill over these retaining wall is termed as "EARTH PRESSURE"



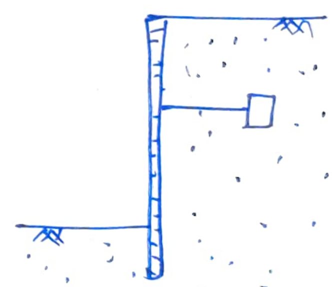



Retaining wall

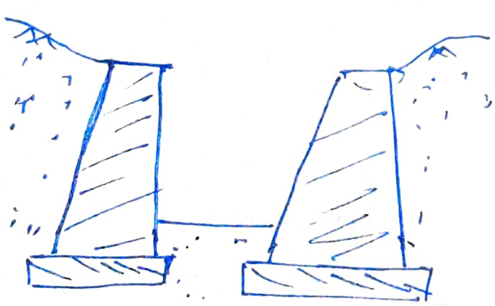


Banking in cuts

Abutment of Bride



Anchored sheet pile



Twin Retaining wall provided in cuts

# Effect of wall movement on Earth Pressure

→ When the ~~soil~~ wall is rigid & unyielding, the soil mass is in a state of rest and there is no deformation & displacement.

→ The earth pressure corresponding to this state is termed as earth pressure at rest.

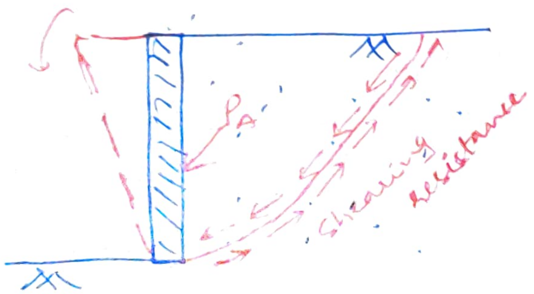
→ If walls rotates about the toe, thus moving away from backfill the soil mass expands, resulting in a decrease of earth pressure, that take place due to mobilisation of shearing resistance in direction opposite to movement the ~~ea~~ earth mass.

→ When the wall moves away from the backfill, a portion of the backfill located next to wall tends to break away & move along with wall in downward & outward direction ~~do~~ direction relative to the wall.

→ Since shearing resistance is ~~modified~~ mobilised in direction away from wall, there is a resultant decrease in earth pressure which continues, until at certain amount of displacement / failure occurs in backfill & slit surface is developed.

→ At this ~~termed~~ stage entire shearing resistance has been mobilised developed, resulting in application of least force on wall termed as ACTIVE EARTH PRESSURE.

→ This force does not decreases any more beyond this even with further wall movement.




→ On the other hand if the wall is pushed towards the backfill, the soil is compressed & the soil offers resistance to this ~~move~~movement by virtue of its shearing resistance.

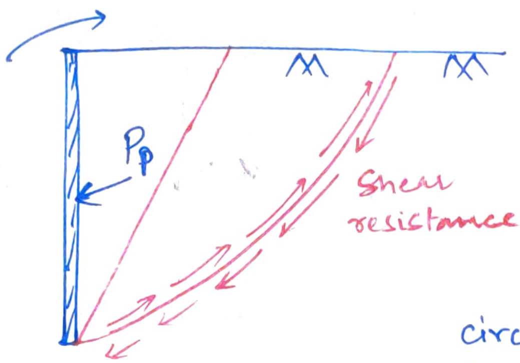
→ Since shearing resistance builds up in direction towards the wall, the earth pressure increases gradually

→ If this force reaches a value, at which ~~sand~~ backfill cannot withstand it & slip failure surface develops; the pressure at this stage reaches its max^m value termed as " PASSIVE EARTH PRESSURE" at which shear resistance is also mobilized at its max^m extent.

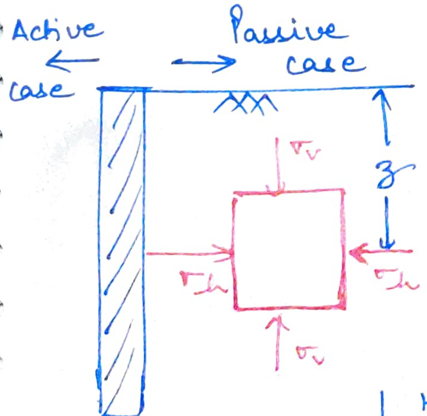
Note :→ The active earth pressure & passive earth pressure develop corresponding to the limiting states of equilibrium.

→ The soil mass is said to be in a state of plastic equilibrium at these two stages.

→ A small increase in stress at this stage will cause a continuous strain (This condition is termed as plastic flow)




→ If a soil element at a depth "z" is subjected to vertical stress "$\sigma_v$". with no shear stress acting on it on horizontal & vertical planes, The Mohr circle representing the state of stress in elastic condⁿ can be drawn as follows.



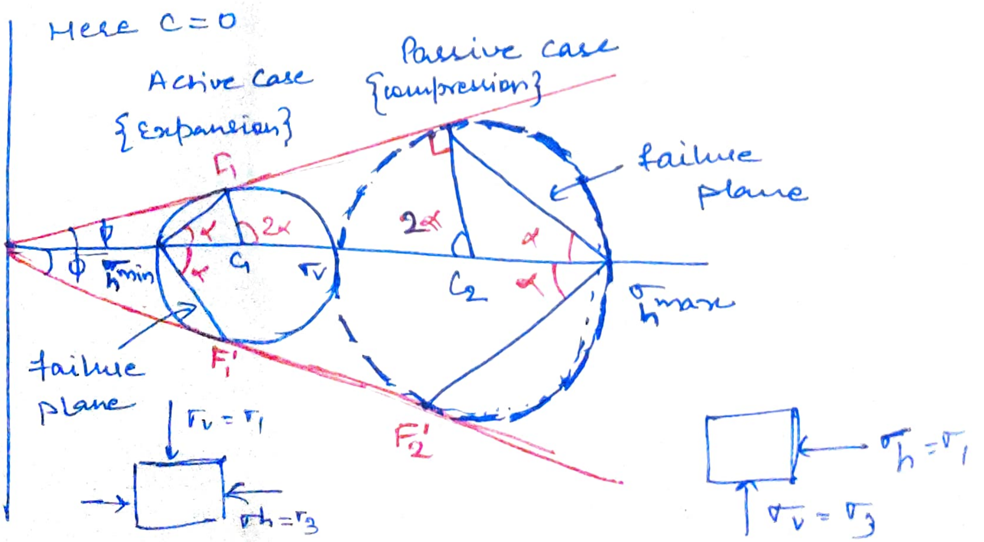

∴ For active cases

$\sigma_1 = \sigma_v$ & $\sigma_3 = \sigma_h$

∴ For passive cases

$\sigma_1 = \sigma_h$ & $\sigma_3 = \sigma_v$

In either case $\sigma_v = \sigma_z$



## Active Case

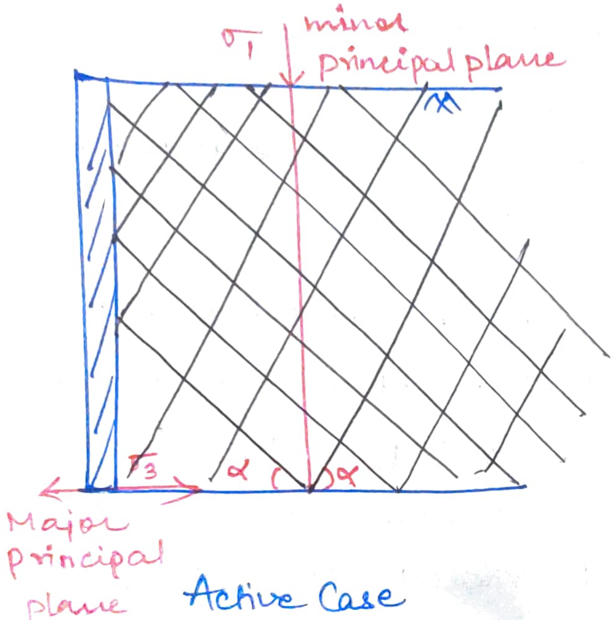
$\alpha = 45^\circ + \phi/2$

- i.e failure plane makes an angle of $\alpha$ with horizontal plane, which in this case is major principal plane

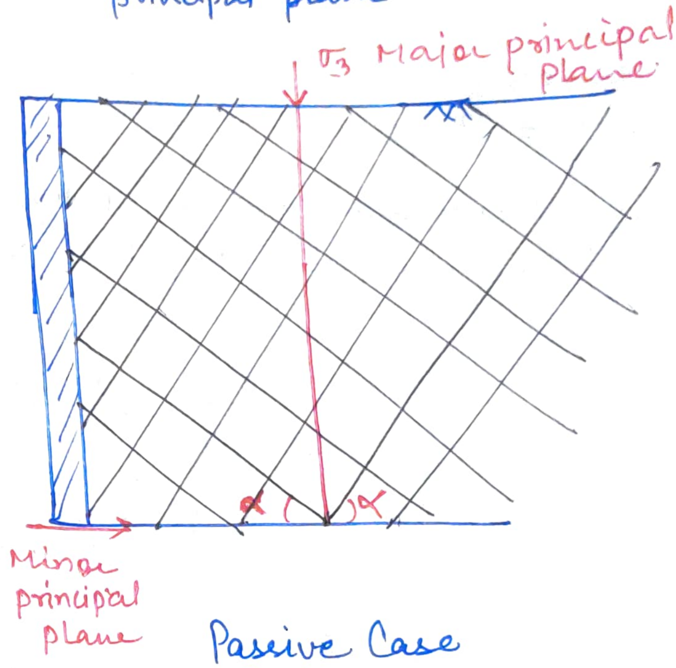
## Passive Case

$\alpha = 45^\circ - \phi/2$

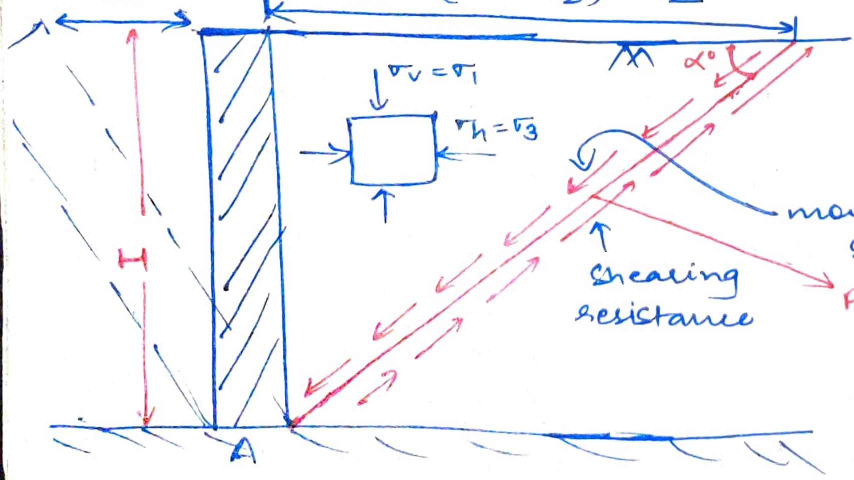
- i.e failure plane makes an angle of $\alpha$ with horizontal which in this case is minor principal plane.




Active Case
$\alpha = 45^\circ + \phi/2$

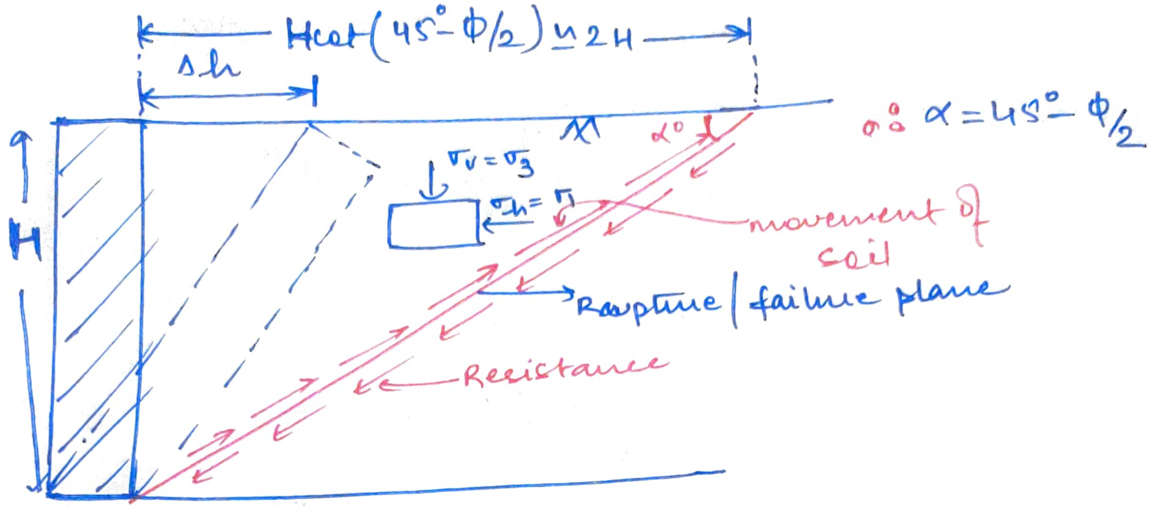



Passive Case
$\alpha = 45^\circ - \phi/2$




$\therefore \alpha = 45^\circ + \phi/2$

"Active case"




"Passive ~~Force~~ Case."

→ for active ~~stage~~ stage strain required is in order of 0.2–0.5%.

$\frac{\Delta H}{H} = 0.2\%$ for dense sand.

$\frac{\Delta H}{H} = 0.5\%$ for loose sand

→ for passive strain required is in order of 2 = 15%.

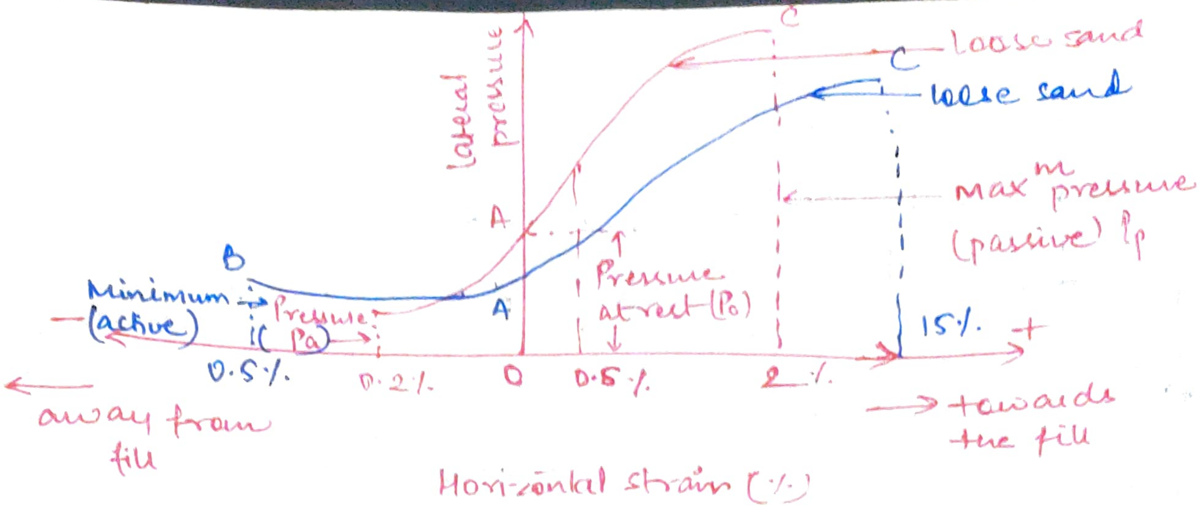
$\frac{\Delta H}{H} = 2\%$ for dense sand

$\frac{\Delta H}{H} = 15\%$ for loose sand

Note → for dense sand very little horizontal strain of less than −0.2% is required to reach the active state.

→ little horizontal compression of about 0.5% is required to reach one half of the maximum passive resistance.

→ Much more horizontal compression of about 2% is required to reach full max^m passive resistance.




<u>From stress relationship</u>

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

<u>From cohesionless soil</u>

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

$\sigma_1 = \sigma_3 \tan^2(45^0 + \phi/2)$

⇒ <u>For active stage</u>

$\sigma_1 = \sigma_v$

$\sigma_3 = \sigma_h$ ⇒ $\sigma_v = \sigma_h \tan^2(45^0 + \phi/2)$

Note :→ Here ratio of lateral to vertical earth pressure termed co-efficient earth pressure, $\frac{\sigma_h}{\sigma_v} = K$ = coeff of earth pressure.

⇒ $\frac{\sigma_h}{\sigma_v} = \frac{1}{\tan^2(45^0+\phi/2)} = \cot^2(45^0 + \phi/2)$

⇒ $\frac{\sigma_h}{\sigma_v} = \frac{1-\sin\phi}{1+\sin\phi} = K_a$ = coefficient of active earth pressure

$$\boxed{p_a = \sigma_h = K_a \sigma_v}$$

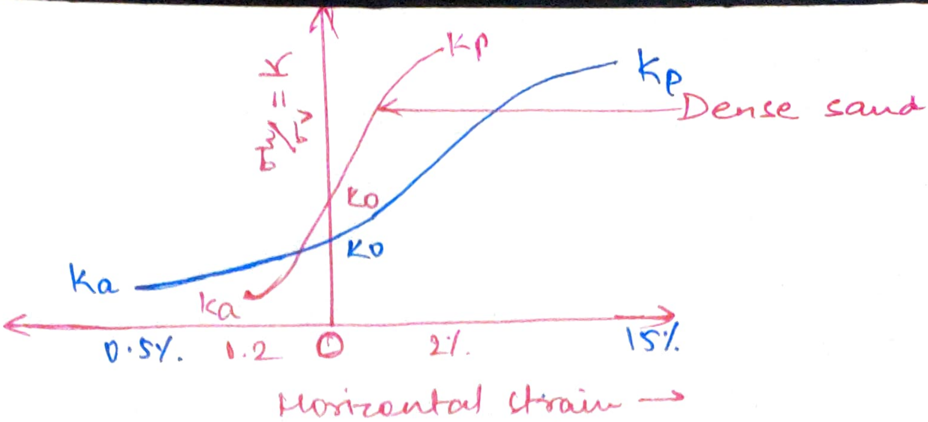
⇒ <u>For passive case</u>

$\sigma_1 = \sigma_h$, $\sigma_3 = \sigma_v$ ⇒ $\sigma_h = \sigma_v \tan^2\alpha$

$\sigma_h = \sigma_v \tan^2(45^0 + \phi/2)$

$\frac{\sigma_h}{\sigma_v} = \frac{1+\sin\phi}{1-\sin\phi} = K_p$ = coefficient of passive earth pressure

$$\boxed{p_p = \sigma_h = K_p \sigma_v}$$

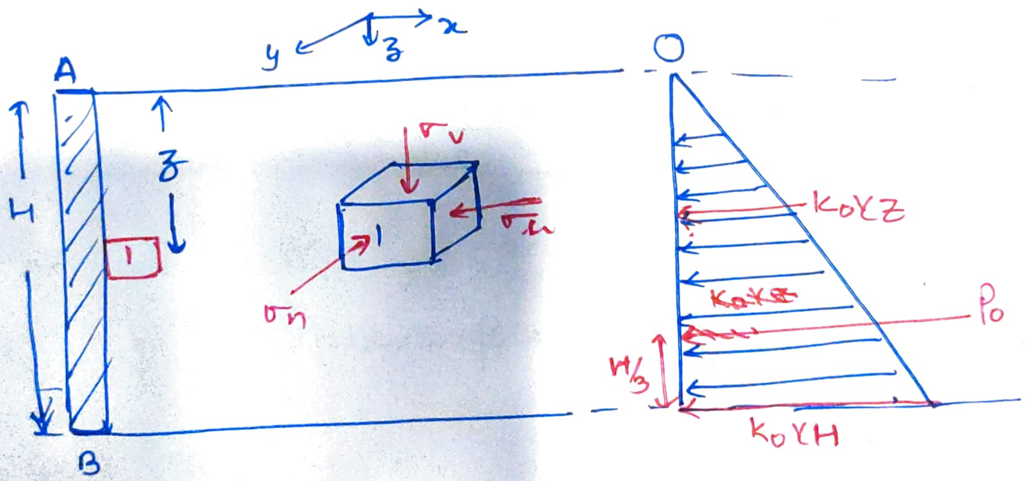



# Earth Pressure At rest

→ When the wall earth system is rigid & unyielding the element of soil at a given depth ($z$) below the ground surface is not subjected to any strain.

→ The element in this state is termed to be in condition known as "at rest" condn.

→ The corresponding lateral pressure, is called "earth pressure at rest".




At rest condition

$\varepsilon_h = 0$

From theory of elasticity

$$\frac{\sigma_h}{E} - \mu\frac{\sigma_h}{E} - \mu\frac{\sigma_v}{E} = 0 \Rightarrow \frac{\sigma_h(1-\mu)}{E} = \mu\frac{\sigma_v}{E}$$

$$\frac{\sigma_h}{\sigma_v} = \frac{\mu}{1-\mu} = K_0$$ = coefficient of earth pressure at rest

→ $p_0 = \sigma_h = \sigma_v K_0$ "Lateral earth pressure at rest"

→ At pt 'A', "$z=0$"

$\sigma_v = \sigma_z = 0 \Rightarrow p_o = k_o \sigma_v = 0$

→ At pt "B", $z=H$

$\sigma_v = \gamma z = \gamma H$

$p_o = k_o \gamma H$

Considering the unit length of wall

→ Total earth pressure / Earth pressure force / Earth pressure thrust. is given by.

(I) $P_o = \int p_o dA = \int k_o \gamma z (dz \times 1)$

$P_o = k_o \gamma \int_{z=0}^{z=H} z\,dz = k_o \gamma \left[ \frac{z^2}{2} \right]_0^H$

$$\boxed{P_o = \frac{k_o \gamma H^2}{2}}$$

(II) $P_o$ = (avg Earth pressure) x Area

$= \left( \frac{0 + k_o \gamma H}{2} \right)(H \times 1) = \boxed{\frac{k_o \gamma H^2}{2}}$

or

(III) $P_o$ = (Area of Pressure distn diagram) x length of wall

$= \frac{1}{2}(k_o \gamma H)(H)(1)$

$= \frac{k_o \gamma H^2}{2}$

Line of action of Total earth pressure would be the CG of pressure distribution diagram.

∴ $P_o$ with $z = H/3$ from Base "B"

or $z = 2H/3$ from top "A"

→ The value of $K_0$ can be evaluated if the Poisson's ratio '$\mu$' is known, However behaviour of soil is not perfectly elastic hence soils don not have well defined value of $\mu$

→ There $K_0$ is found on the basis observation

| TYPE OF SOIL | $K_0$ |
|---|---|
| • Dense sand | 0.4 - 0.45 |
| • loose sand | 0.45 - 0.5 |
| • Mechanically compacted sand | 0.8 - 1.5 |
| • Normally consolidated clay | 0.5 - 0.6 |
| • over consolidated clay | 1 - 4 |

→ For sand & normally consolidated clay $K_0$ can also found empirically as follow :—

$$K_0 = 1 - \sin\phi' \quad \text{or} \quad K_0 = 0.95 - \sin\phi' + 0.15$$

→ For normally consolidated clay

$$K_0 = 0.19 + 0.233 \log_{10} I_p$$

→ For over consolidated clay

$$K_0 = K_0(NCC)\sqrt{OCR}$$

Q. A 5m high rigid retaining wall has to retain soil having the following properties

$G = 2.68$, $\mu = 0.36$, $e = 0.74$, $\phi = 30°$

(i) Plot the distribution of lateral earth pressure for wall

(ii) Determine the magnitude & point of resultant thrust

(iii) compute the % change in lateral thrust if water table rises from a great depth to the top of backfill.

$$\gamma_d = \frac{G\gamma_w}{1+e} = \frac{2.68 \times 10}{1+0.74} = 15.4\ kN/m^3$$

As wall is rigid lateral pressure exerted by backfill is earth pressure at rest

$K_0 = \frac{\mu}{1-\mu} = \frac{0.36}{1-0.36} = 0.5625$

(i) At pt 'A', $z = 0$

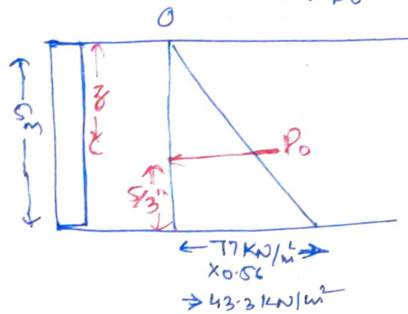
$\sigma_v = \gamma z = 0$

$p_0 = K_0 \sigma_v = 0$

At pt 'B', $z = 5m$

$\sigma_v = \gamma z = 15.4 \times 5 = 77 \, kN/m^2$

$\hookrightarrow p_0 = k_0 \sigma_v = 0.5625 \times 77 = 43.3 \, kN/m^2$




(ii) Total thrust

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

$= \frac{1}{2} \times 0.5625 \times 15.4 \times 5^2$

$P = 108.23 \, kN/m$

$\bar{z} = H/3$ from base

$\bar{z} = 5/3 \, m$ Base.

(iii) If water table rises to top of the backfill

→ At point C, $z = z$ ⇒ $\sigma_v = \sigma_{sat} z = (\gamma' + \gamma_w) z$

⇒ $\gamma' z + \gamma_w z$

$p_0 = k_0 \sigma_v = k_0(\gamma' z + \gamma_w z)$ × ⇒ $\boxed{p_0 = k_0 \gamma' z + \gamma_w z}$

→ At point A $z = 0$

$\sigma_v = 0$

$p_0 = k_0 \sigma_v = 0$

→ At point B, $z = 5m$

$\sigma_v = \gamma' z + \gamma_w z$

$= 9.65 \times 5 + 10 \times 5 = 98.25 \, kN/m^2$

$p_0 = p_{01} + p_{02}$

$= k_0 \gamma' z + \gamma_w z$

$= 0.5625 \times 48.25 + 1 \times 50$

$= 27.14 + 50$

⇒ $77.14 \, kN/m^2$ continued

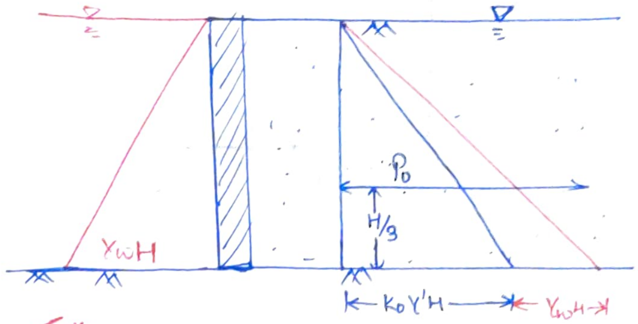
$\gamma' = \frac{(G-1)\gamma_w}{1+e}$

$= 9.65 \, kN/m^3$

Note :→ water will exert equal pressure in all directions at particular depth ($\gamma_w z$), hence water pressure will not be considered along with coeff of earth pressu at rest or active or passive coeffu.

→ Impact of water on lateral pressure increases the total thrust, which makes the designing of retaining structure uneconomical, thus to avoid its impact weep holes are provided in retaining structure.

→ If water is present on both sides, its impact would be cancelled out.




* continued

Total thrust

$P_0 = P_{01} + P_{02}$

$= \frac{1}{2} k_0 \gamma' H^2 + \frac{\gamma_w H^2}{2}$

$= 190.47 \, kN/m$

$\bar{z}_1 = H/3$, $\bar{z}_2 = H/3$

$\bar{z} = \frac{P_{01} \cdot \bar{z}_1 + P_{02} \cdot \bar{z}_2}{P_{01} + P_{02}} = H/3 = 5/3 \, m$ from B

% change in thrust $= \frac{190.47 - 108.18}{108.18} \times 100$

$= 77\%$ (increase)

(i) Hydrostatic Compression

$\sigma_v = \sigma_h \quad (\sigma_1 = \sigma_3)$

$p = \frac{\sigma_v + \sigma_h}{2} = \frac{\sigma_1 + \sigma_3}{2}$

$q = \frac{\sigma_v - \sigma_h}{2} = \frac{\sigma_1 - \sigma_3}{2}$

$p_0 = \frac{\sigma_v + \sigma_3}{2} = \frac{2\sigma_v}{2} = \sigma_v = \sigma_h$

$q_0 = \frac{\sigma_v - \sigma_h}{2} = 0$

Initial condition

$$p_f = \frac{(\sigma_v + \Delta\sigma_v) + (\sigma_h + \Delta\sigma_h)}{2}$$

$$q_f = \frac{(\sigma_v + \Delta\sigma_v) - (\sigma_h + \Delta\sigma_h)}{2}$$

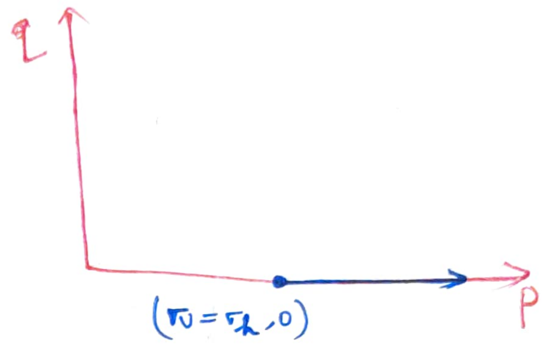
} final condition

$\therefore$ → Here $\Delta\sigma_h = \Delta\sigma_v$

$p_f = \sigma_v + \Delta\sigma_v$

$\Delta p = p_f - p_0 = \Delta\sigma_v$

$q_f = 0$




## (ii) Non hydrostatic Pressure

$\sigma_v \neq \sigma_h \neq 0$

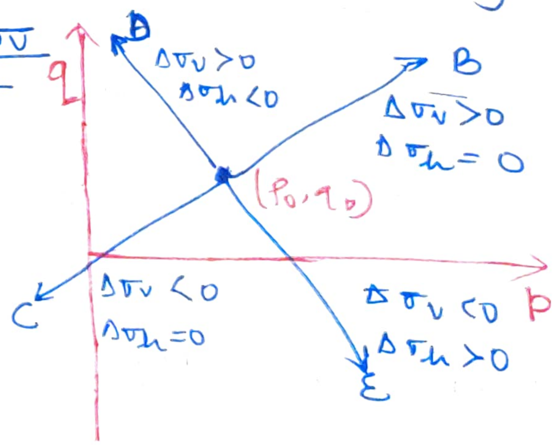
$p_0 = \frac{\sigma_v + \sigma_h}{2}$, $q_0 = \frac{\sigma_v - \sigma_h}{2}$

$q_f = \frac{(\sigma_v + \Delta\sigma_v) - (\sigma_h + \Delta\sigma_h)}{2}$ $\quad$ $p_f = \frac{(\sigma_v + \Delta\sigma_v) + (\sigma_h + \Delta\sigma_h)}{2}$

(a) $\Delta\sigma_v$ is increases ($\Delta\sigma_v > 0$), $\Delta\sigma_h = 0$ [Eg: Embankment constn]

$p_f = p_0 + \frac{\Delta\sigma_v}{2}$ $\Rightarrow$ $\Delta p = p_f - p_0 = \frac{\Delta\sigma_v}{2}$

$\Delta q = q_f - q_0 = \frac{\Delta\sigma_v}{2}$




(b) $\Delta\sigma_v$ decreases ($\Delta\sigma_v < 0$), $\Delta\sigma_h = 0$ (Excavation in soil mass)

$\Delta p = \frac{\Delta\sigma_v}{2}$ (−ve)

$\Delta q = \frac{\Delta\sigma_v}{2}$ (−ve)

(c) $\Delta\sigma_v$ is increasing, $\Delta\sigma_h$ decreases $(\Delta\sigma_v > 0)$ $(\Delta\sigma_h < 0)$

$$p_f = \frac{\sigma_v + \Delta\sigma_v + \sigma_h + \Delta\sigma_h}{2} = p_o + \frac{\Delta\sigma_v}{2} + \frac{\Delta\sigma_h}{2}$$

$$\Delta p = p_f - p_o = \frac{\Delta\sigma_v}{2} + \frac{\Delta\sigma_h}{2} \quad (\text{let } \Delta\sigma_h > \Delta\sigma_v)$$

$$q_f = \frac{(\sigma_v + \Delta\sigma_v) - (\sigma_h + \Delta\sigma_h)}{2} = q_o + \frac{\Delta\sigma_v}{2} - \frac{\Delta\sigma_h}{2}$$

$$\Delta q = q_f - q_o = \frac{\Delta\sigma_v}{2} - \frac{\Delta\sigma_h}{2} \quad (\text{let } \Delta\sigma_h > \Delta\sigma_v)$$

(d) $\Delta\sigma_v$ decreases $(\Delta\sigma_v < 0)$, $\Delta\sigma_h$ increases $(\Delta\sigma_h > 0)$

$$\Delta p = \frac{\Delta\sigma_v}{2} + \frac{\Delta\sigma_h}{2} \quad (\text{let } \Delta\sigma_h > \Delta\sigma_v)$$

$$\Delta q = \frac{\Delta\sigma_v}{2} - \frac{\Delta\sigma_h}{2} \quad (\text{let } \Delta\sigma_h > \Delta\sigma_v)$$

# RANKINE's EARTH PRESSURE THEORY

→ This theory of lateral earth pressure is applied to initially cohesionless soil only. However it was extended to include cohesive soils also later on.

→ This theory is also been extended to stratified, submerged soil.

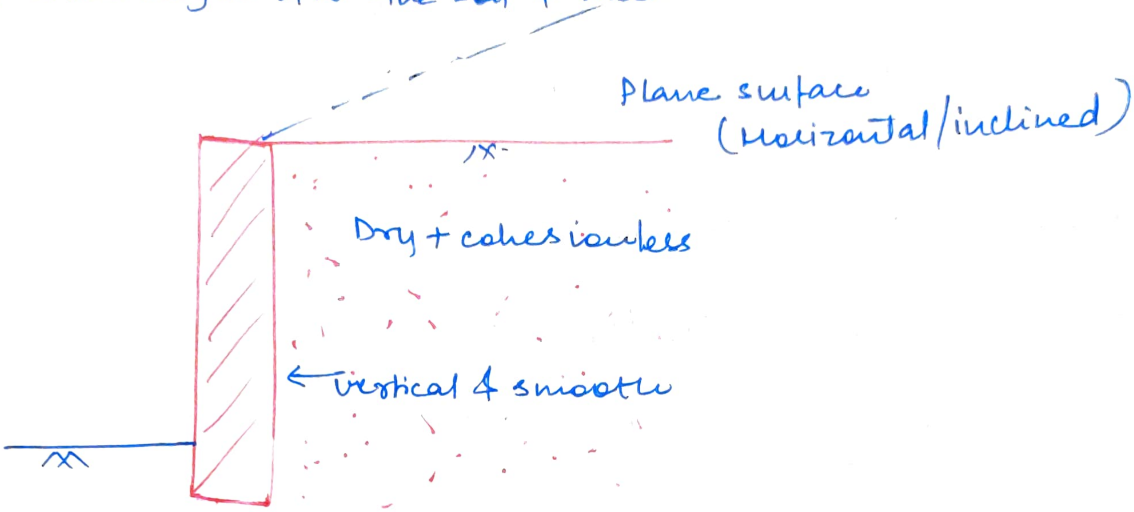
→ This theory works on following assumptions:—

- The soil mass is semi-infinite, homogeneous dry & cohesionless.
- The ground surface is plane which may be horizontal or inclined.
- The back of the wall is vertical & smooth (There is no shear stress between the wall & the stress relationship for any element adjacent to the wall is the same as for any other element far away from wall).
- The wall yields about the base & thus satisfies the deformation condition of plastic equilibrium.

Note :→ However the retaining walls are constructed of masonry or concrete & hence back of wall is never smooth.

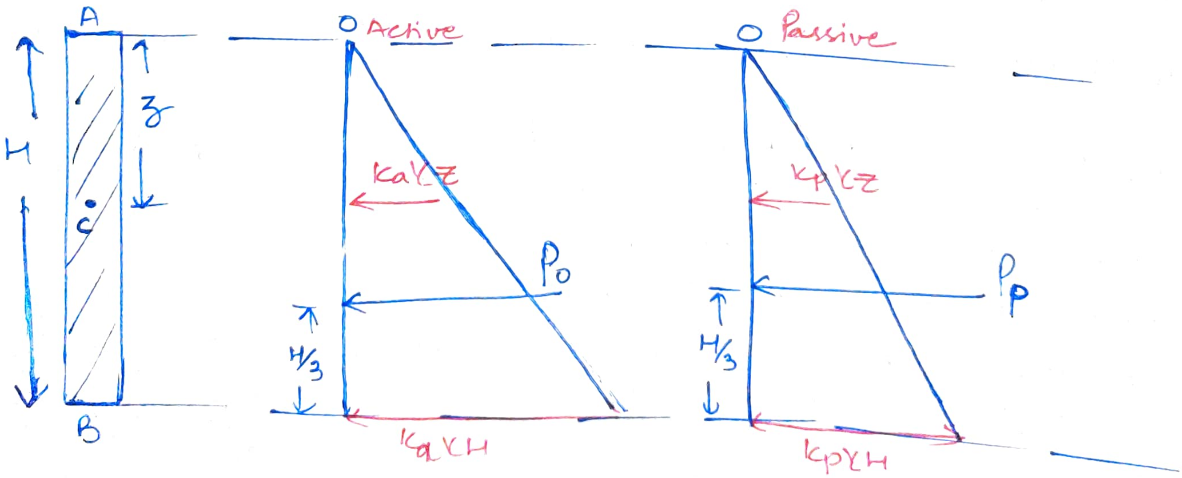
→ Due to this frictional forces develop; as a consequence of Rankine's assumption of non-existence of frictionless forces at the wall face, the resultant pressure must be parallel to the surface of backfill.

→ The existence of friction makes the resultant pressure inclined to the normal to the wall at an angle the approaches friction angle b/w^n the soil & wall.




# The following cases of cohesionless backfill are considered

Case (i) DRY/MOIST BACKFILL




(i) $\sigma_v = \gamma z$

(ii) $\sigma_h = k\sigma_v$

Active

(iii) $p_a = K_a\gamma z$

Passive

$p_p = K_p\gamma z$

At A, $z = 0$

$p_a = 0$

$p_p = 0$

At pt B, $z = H$

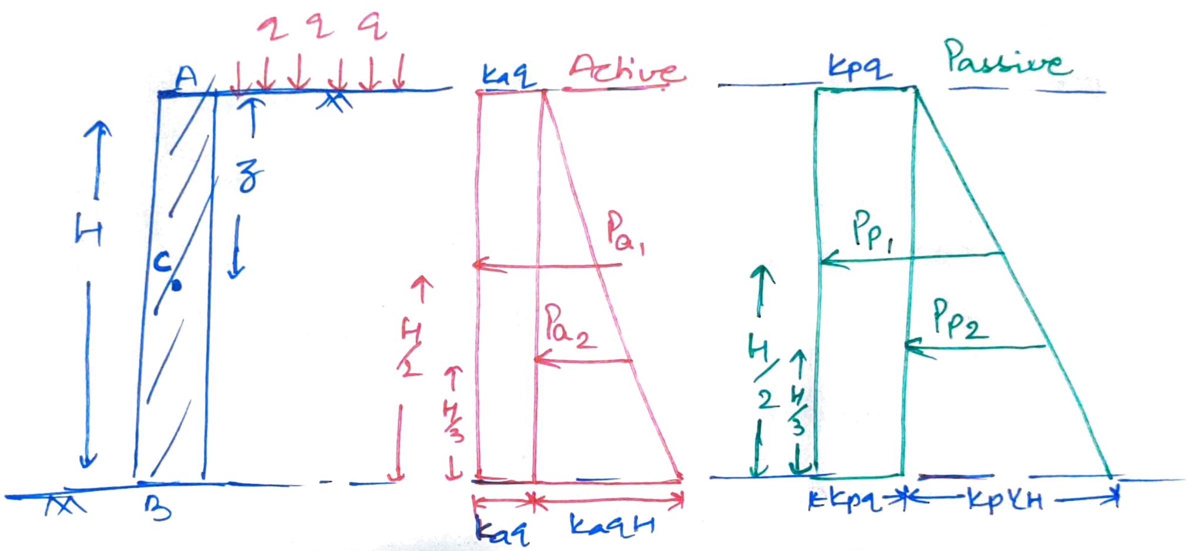
$p_a = k_a \gamma H$ | $p_p = k_p \gamma H$

considering unit length of wall

$P_a = \frac{1}{2} k_a \gamma H^2$ | $P_p = \frac{1}{2} k_p \gamma H^2$

$\bar{z} = H/3$ from Base.

## case(ii): COHESIONLESS BACKFILL WITH UNIFORM SURCHARGE




(i) $\sigma_v = q + \gamma z$

(ii) $\sigma_h = K \sigma_v$

$$\boxed{\bar{z} = \frac{\Sigma P_i z_i}{\Sigma P_i}}$$

**Active** | **Passive**

$p_a = k_a(q + \gamma z)$ | $p_p = k_p(q + \gamma z)$

$= k_a q + k_a \gamma z$ | $= k_p q + k_p \gamma z$

At point A, $z = 0$

$p_a = k_a q$ | $p_p = k_p q$

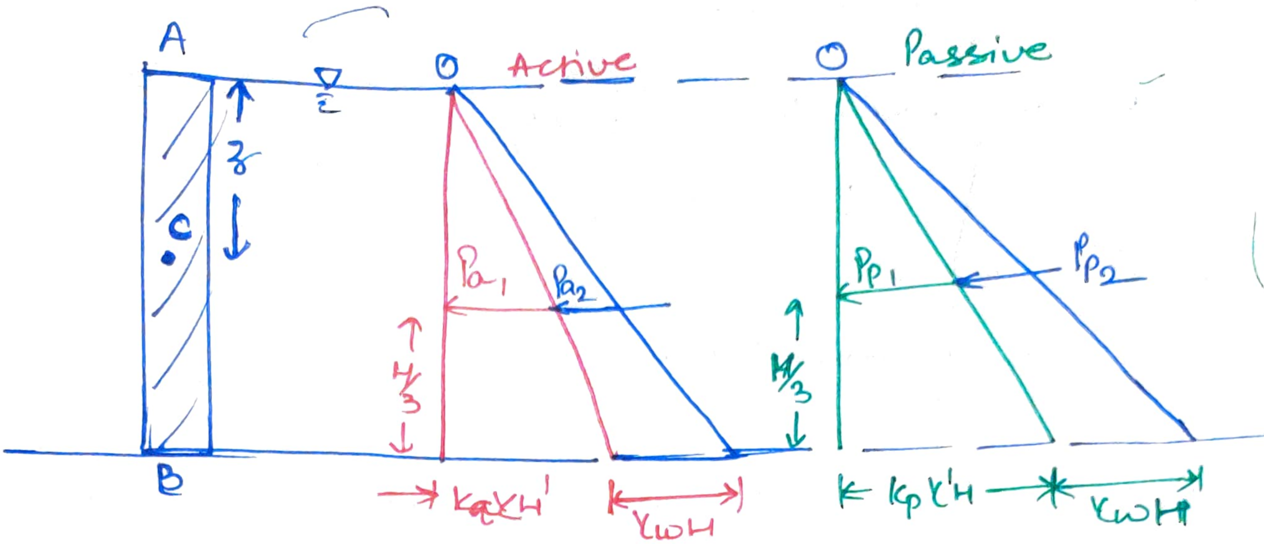
At pt 'B', $z = H$

$p_a = k_a q + k_a \gamma H$ | $p_p = k_p q + k_p \gamma H$

$P_a = k_a q H + \frac{1}{2} k_a \gamma H^2$ | $P_p = k_p q H + \frac{1}{2} k_p \gamma H^2$

- $\bar{z}_1 = H/2$ from B
- $\bar{z}_2 = H/3$ from B

## Case (iii) : SUBMERGED BACKFILL




(i) $\sigma_v = \gamma' z + \gamma_w z$

(ii) $\sigma_H = K\sigma_v$

| Active | Passive |
|---|---|
| (iii) $p_a = p_{a_1} + p_{a_2}$ <br> $= K_a\gamma' z + \gamma_w z$ | $p_p = p_{p_1} + p_{p_2}$ <br> $= K_p\gamma' z + \gamma_w z$ |

At point 'A' $z = 0$

| | |
|---|---|
| $p_a = 0$ | $p_p = 0$ |

At point 'B', $z = H$

| | |
|---|---|
| $p_a = K_a\gamma' H + \gamma_w H$ | $p_p = K_p\gamma' H + \gamma_w H$ |
| $P_a = P_{a_1} + P_{a_2}$ <br> $= \frac{K_a\gamma' H^2}{2} + \frac{\gamma_w H^2}{2}$ | $P_p = P_{p_1} + P_{p_2}$ <br> $= \frac{K_p\gamma' H^2}{2} + \frac{\gamma_w H^2}{2}$ |

$z_1 = z_2 = H/3$ from Base

$$z = \frac{\Sigma P_i z_i}{\Sigma P_i} = H/3 \text{ from Base.}$$

Q. A counterfort wall of 10m height retain non-cohesive backfill. The void ratio & angle of internal friction is 0.7 & 30° in loose state & are 0.4 & 40° in dense state. Calculate compare the active & passive earth pressure in both cases, $G = 2.7$, Also comment on result obtained.

Sol. $\gamma = \dfrac{G\gamma_w}{1+e} = 15.88 \text{ kN/m}^2$

LOOSE SAND

$k_a = \dfrac{1-\sin\phi}{1+\sin\phi} = 0.33$

DENSE SAND

$k_a = \dfrac{1-\sin\phi}{1+\sin\phi} = 0.271, \quad k_p = 4.6$

$P_a = \frac{1}{2} k_a \gamma H^2$

$P_p = \frac{1}{2} k_p \gamma H^2$

LOOSE ~~SAND~~ soil

$P_a = 262.06 \text{ kN/m}$

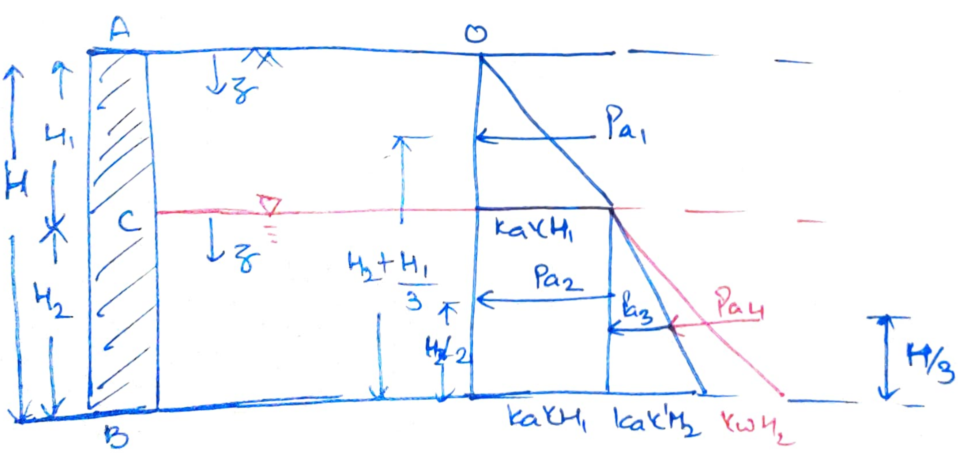
$P_p = 2382 \text{ kN/m}$.

DENSE SOIL

$P_a = 209.18 \text{ kN/m}$

$P_p = 4436.8 \text{ kN/m}$.

(iv) SUBMERGED BACKFILL, WITH WATER TABLE AT DEPTH OF $H_1$ FROM GROUND




For portion AC

i) $\sigma_v = \gamma z$

(ii) $\sigma_h = k\sigma_v$

| Active | Passive |
|---|---|
| (iii) $p_a = k_a \gamma z$ | $p_p = k_p \gamma z$ |

At point A
$z = 0$

| | |
|---|---|
| $p_a = 0$ | $p_p = 0$ |

At point C, $z = H_1$

| | |
|---|---|
| $p_a = k_a \gamma H_1$ | $p_p = k_p \gamma H_1$ |

For portion CB

$\sigma_v = \gamma H_1 + \gamma' z + \gamma_w z$ ~~At point~~

$\sigma_h = k \sigma_v$

| Active | Passive |
|---|---|
| (iii) $p_a = k_a \gamma H_1 + k_a \gamma' z + \gamma_w z$ | $p_p = k_p \gamma H_1 + k_p \gamma' z + \gamma_w z$ |

At point C, $z = 0$

| | |
|---|---|
| $p_a = k_a \gamma H_1$ | $p_p = k_p \gamma H_1$ |

At point B, $z = H_2$

| | |
|---|---|
| $p_a = k_a \gamma H_1 + k_a \gamma' H_2 + \gamma_w H_2$ | $p_p = k_p \gamma H_1 + k_p \gamma' H_2 + \gamma_w H_2$ |

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

$P_{a_1} = \frac{1}{2} k_a \gamma H_1^2, \quad z_1 = H_2 + H_1/3$

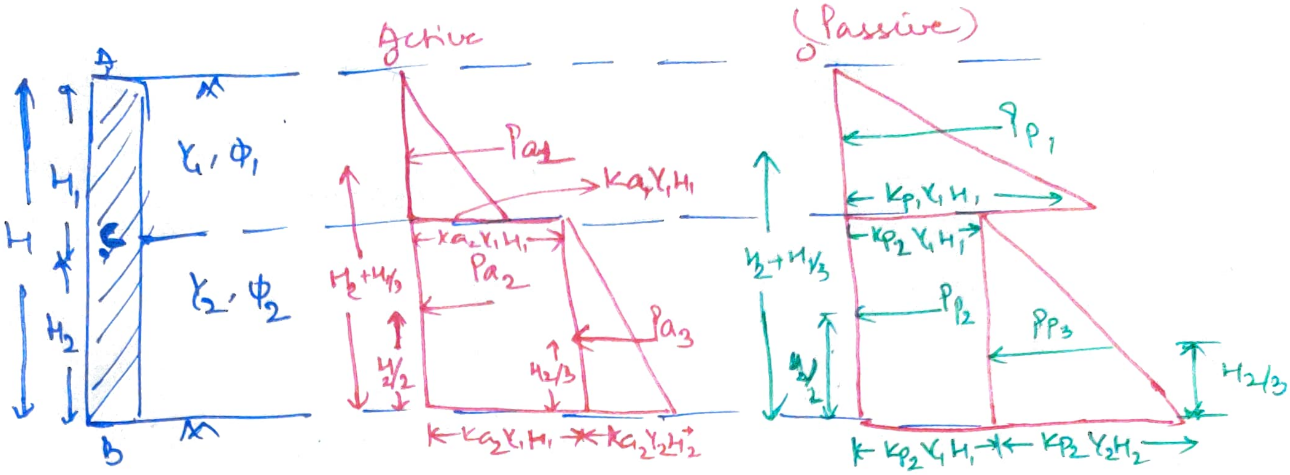
$P_{a_2} = k_a \gamma H_1 H_2, \quad z_2 = H_2/2$

$P_{a_3} = \frac{k_a \gamma' H_2^2}{2}, \quad z_3 = \frac{H_2}{3}$

$P_{a_4} = \frac{\gamma_w H_2^2}{2}, \quad z_4 = H_2/3$

$$\boxed{\bar{z} = \frac{\sum P_{a_i} z_i}{\sum P_{a_i}}}$$

## (V) BACKFILL WITH DIFFERENT SOIL HAVING DIFFERENT FRICTION ANGLE




Case (i) If $\phi_1 > \phi_2$ $\quad k_a = \dfrac{1-\sin\phi}{1-\sin\phi}$ , $k_p = \dfrac{1+\sin\phi}{1-\sin\phi}$

$\Rightarrow k_{a1} > k_{a2}$ but $k_{p1} > k_{p2}$

**For portion AC**

(i) $\sigma_v = \gamma_1 z$

(ii) $\sigma_h = K\sigma_v$

| | Active | Passive |
|---|---|---|
| (iii) | $p_a = k_{a1}\gamma_1 z$ | $p_p = k_{p1}\gamma_1 z$ |

At pt 'A', $z=0$

$p_a = 0$ $\qquad$ $p_p = 0$

At pt 'C', $z = H_1$

$p_a = k_{a1}\gamma_1 H_1$ $\qquad$ $p_p = k_{p1}\gamma_1 H_1$

**For portion CB**

(i) $\sigma_v = \gamma_1 H_1 + \gamma_2 z$

(ii) $\sigma_h = K\sigma_v$

| | Active | Passive |
|---|---|---|
| (iii) | $p_a = k_{a2}\gamma_1 H_1 + k_{a2}\gamma_2 z$ | $p_p = k_{p2}\gamma_1 H_1 + k_{p2}\gamma_2 z$ |

At point 'C', $Z = 0$

$p_a = k_{a_2} \gamma_1 H_1$ | $p_p = k_{p_2} \gamma_1 H_1$

At Pt 'B', $Z_2 = H_2$

$p_a = k_{a_2} \gamma_1 H_1 + k_{a_2} \gamma_2 H_2$ | $p_p = k_{p_2} \gamma_1 H_1 + k_{p_2} \gamma_2 H_2$

$$\therefore \boxed{P_a = P_{a_1} + P_{a_2} + P_{a_3}}$$

$P_{a_1} = \frac{1}{2} k_{a_1} \gamma_1 H_1^2$ , $Z_1 = H_2 + H_1$

$P_{a_2} = k_{a_2} \gamma_1 H_1 H_2$ , $Z_2 = H_2/2$

$P_{a_3} = \frac{1}{2} k_{a_2} \gamma_2 H_2^2$ , $Z_3 = H_2/3$

$$\boxed{Z = \frac{\Sigma P_{a_i} Z_i}{\Sigma P_{a_i}}}$$

Case (ii) If $\phi_1 < \phi_2 \Rightarrow k_{a_1} > k_{a_2}$, $k_{p_1} < k_{p_2}$. In this case active and passive pressure dist^n diag. will get interchanged

Notes : → In reality, there cannot be a sudden change in lateral pressure since shear stresses which develop along the interface have not been considered.

→ But this does not introduce any serious error in magnitude & direction of resultant forces, hence are not being considered, for simplicity in calculation.

## (vi) BACKFILL WITH SLOPING SURCHARGE

→ Let the sloping surface behind the wall be inclined at an angle $\beta$ with horizontal (surcharge angle)

→ In this case to calculate active earth pressure one more assumption is being taken that the vertical & lateral stresses are ~~conditions~~ " CONJUGATE STRESS ".

→ If the stress on a given plane at a given point is ~~flat~~ parallel to another plane, the stress on the lateral plane at same point is parallel to the first plane.

→ These planes are called ~~parallel planes~~ conjugate plane & stress acting on these planes are termed as conjugate stress.

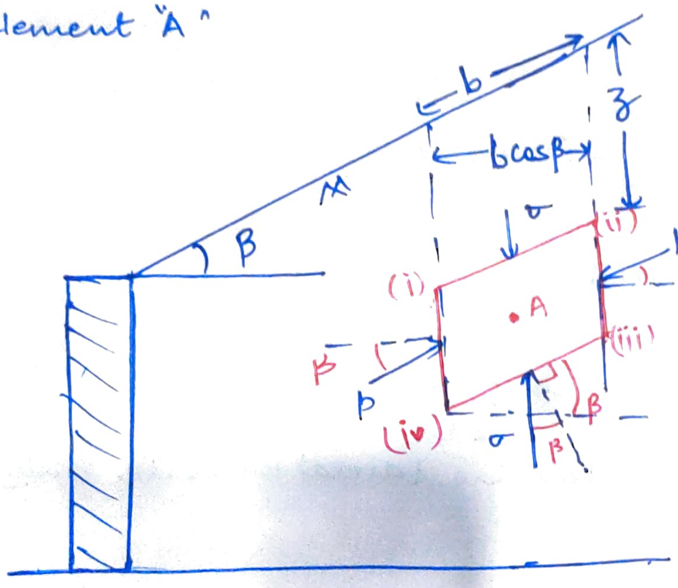
→ Consider an element of soil at a point 'A' at depth 'z' within a backfill with a ~~soil~~ sloping surface.

→ The top plane of this element is '∥' to ground surface or plane & other plane conjugate plane to this plane is vertical.

→ Let $\sigma$ & $p$ are conjugate stresses acting on these plane at same angle of obliquity "$\beta$".

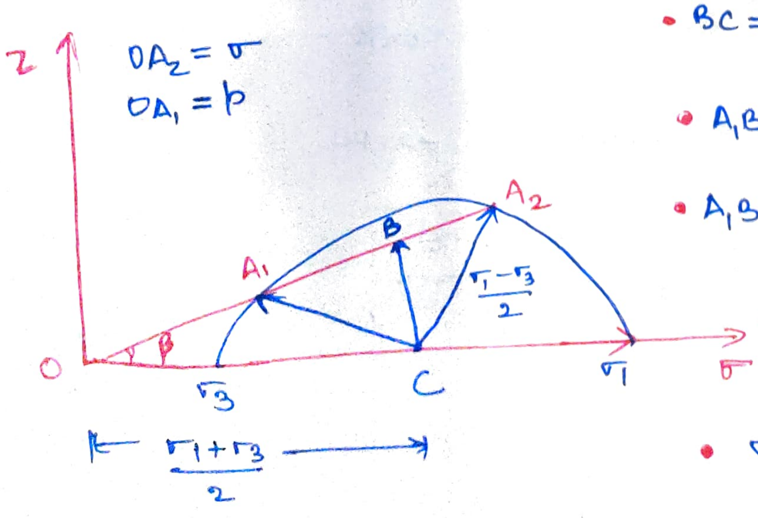
Note :→ $\sigma$, $p$ are resultant stresses on the two conjugate plane & are not principal stresses

→ Let $\sigma_1$ & $\sigma_3$ be the major & minor principal stress on the soil element "A"




In $\Delta CBO$

- $BC = OC \sin\beta$ , $OB = OC \cos\beta$
- $BC = \frac{\sigma_1+\sigma_3}{2} \sin\beta$ , $OB = \frac{\sigma_1+\sigma_3}{2} \cos\beta$




- $A_1B = BA_2 = \sqrt{A_1C^2 - BC^2}$
- $A_1B = BA_2 = \sqrt{\left(\frac{\sigma_1-\sigma_3}{2}\right)^2 - \left(\frac{\sigma_1+\sigma_3}{2}\right)^2 \sin^2\beta}$ —(i)

• From stress relationship of soil.

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

$c = 0$

$$\sigma_1 = \sigma_3 \tan^2\left(45^\circ + \phi/2\right) = \sigma_3\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$$

$$\sigma_1 - \sigma_3 = (\sigma_1+\sigma_3)\sin\phi \quad \text{—(ii)}$$

From (i) & (ii)

$$A_1B = BA_2 = \sqrt{\left(\frac{\sigma_1+\sigma_3}{2}\right)^2 \sin^2\phi - \left(\frac{\sigma_1-\sigma_3}{2}\right)^2 \sin^2\beta}$$

$$= \boxed{\frac{\sigma_1+\sigma_3}{2}\sqrt{\sin^2\phi - \sin^2\beta}}$$

Now $\sigma = OA_2 = OB + BA_2$

$$\Rightarrow \quad \sigma = \frac{\sigma_1+\sigma_3}{2}\cos\beta + \frac{\sigma_1+\sigma_3}{2}\sqrt{\sin^2\phi - \sin^2\beta}$$

$$\Rightarrow \sigma = \frac{\sigma_1+\sigma_3}{2}\left[\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}\right]$$

$p = OA_1 = OB - A_1B$

$$= \frac{\sigma_1+\sigma_3}{2}\cos\beta - \frac{\sigma_1+\sigma_3}{2}\sqrt{\sin^2\phi - \sin^2\beta}$$

$$\Rightarrow p = \frac{\sigma_1+\sigma_3}{2}\left[\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}\right]$$

$$\boxed{\frac{p}{\sigma} = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}} = K =$$ conjugate ratio or Rankine lateral pressure ratio

## Vertical Pressure

For given case $\sigma = \dfrac{\gamma \cdot b\cos\beta \cdot z \cdot 1}{b \times 1} = \gamma z \cos\beta$

$p$ = Lateral pressure on earth

$$p_a = K\sigma = \gamma z\cos\beta\left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\theta}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\theta}}\right]$$

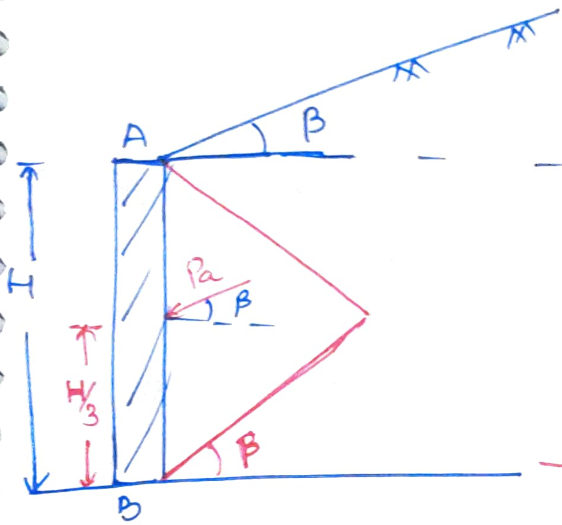
$$p_a = K_a \gamma z$$

Here $K_a = \cos\beta\left[\dfrac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\theta}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\theta}}\right]$

If $\beta = 0$  $K_a = \frac{1-\sin\phi}{1+\sin\phi}$ [same as horizontal surface by backfill]

Considering unit length of wall

Total thrust $P_a = \frac{1}{2} K_a \gamma H^2$  ∴ $z = H/3$ above the base in drtn ∥ to the surface.



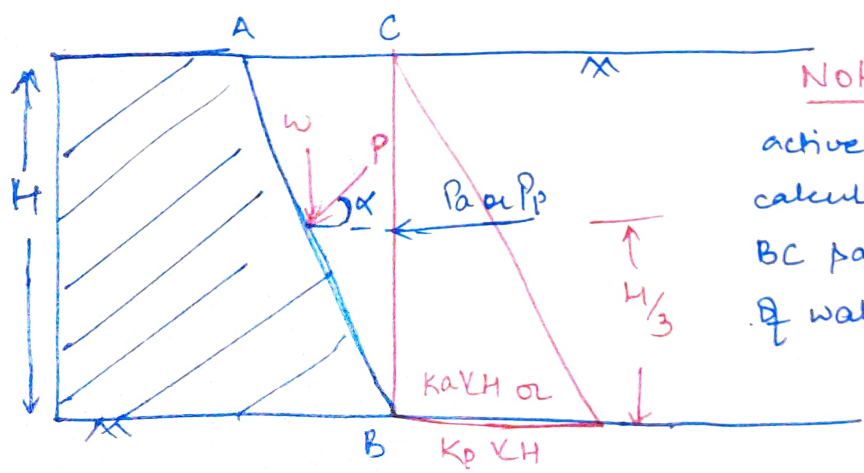

Note :→ For passive case

$$P_p = K_p \gamma Z$$

$$K_p = \cos\beta \left[ \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\theta}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\theta}} \right]$$

→ If the backfill is submerged, the lateral pressure due to submerged weight of the soil will act at 'β' with horizontal with lateral pressure due to water will act normal to the wall.

## (vii) INCLINED BACK WITH HORIZONTAL BACKFILL




Note :→ In this case total active or passive thrust is first calculated on a vertical plane BC passing through the level of wall 'B'

→ The total pressure 'P' is resultant of horizontal pressure (Pa/Pp) & weight "w" of soil weight 'ABC'

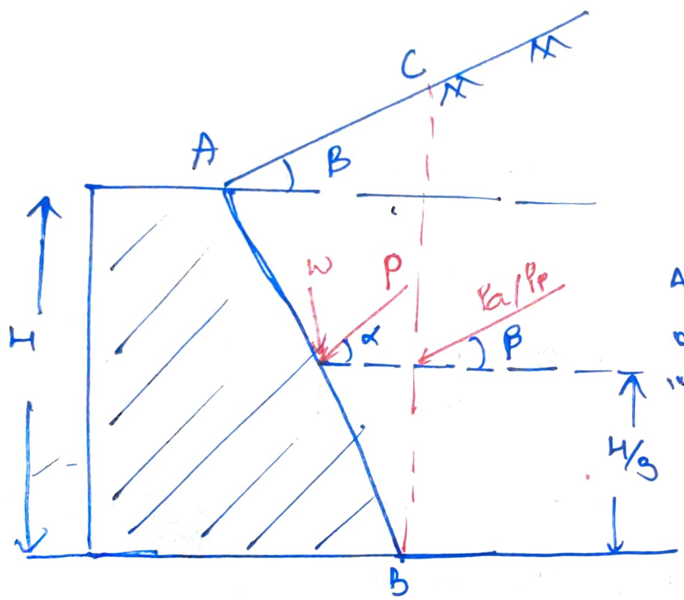
$$P = \sqrt{(P_a/P_p)^2 + w^2}$$

$Z = H/3$ from base acting at an angle of $\alpha$

$$\alpha = \tan^{-1}\left(\frac{W}{P_a}\right) \text{ or } \tan^{-1}\left(\frac{W}{P_p}\right)$$

Here $P_a = \frac{1}{2} K_a \gamma H^2$ $\quad P_p = \frac{1}{2} K_p \gamma H^2$

## (viii) INCLINED BACKFILL WITH SLOPING SURFACE




→ Here $P_a = \frac{1}{2} K_a \gamma H_{BC}^2$ & $P_p = \frac{1}{2} K_p \gamma H_{BC}^2$

Active & Passive earth pr. is calculated on imaginary vertical plane of height "$H_{BC}$" passing through heel ~~by~~ of wall B.

$$P = \sqrt{(W + P_a \sin\beta)^2 + (P_a \cos\beta)^2}$$

or

$$P = \sqrt{(W + P_p \sin\beta)^2 + (P_p \cos\beta)^2}$$

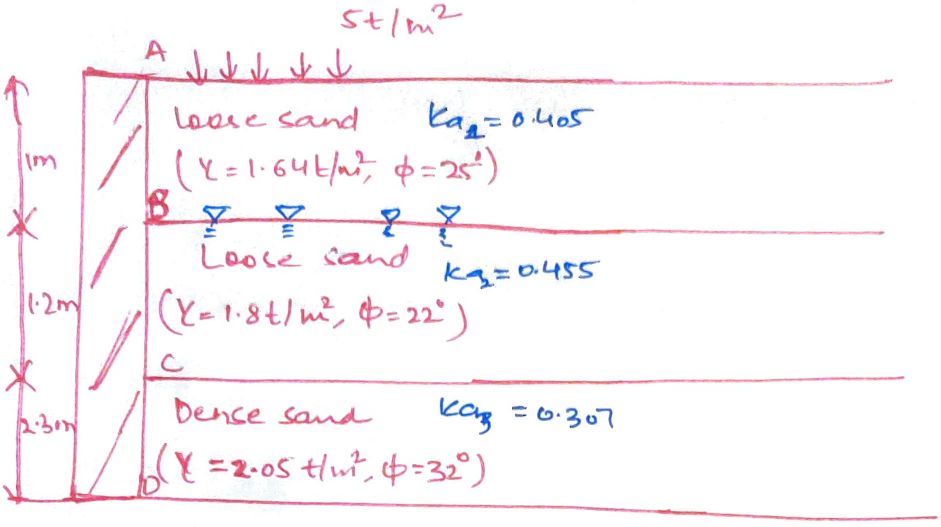
W = weight of wedge of soil (ABC

$z = \frac{H}{3}$ acting at an angle of $\alpha = \tan^{-1}\left[\frac{W + P_a \sin\phi}{P_a \cos\beta}\right]$

or

$$\alpha = \tan^{-1}\left[\frac{W + P_p \sin b}{P_p \cos\beta}\right]$$

Q. For the retaining wall as shown compute the resultant active thrust and point of its application.




$$Ka_1 = \frac{1-\sin\phi}{1+\sin\phi} = 0.405 \qquad Ka_3 = \frac{1-\sin\phi}{1+\sin\phi} = 0.307$$

$$Ka_2 = \frac{1-\sin\phi}{1+\sin\phi} = 0.455$$

* For soil ①

(i) $\sigma_v = q + \gamma z$

(ii) $\sigma_h = K\sigma_v$

(iii) $p_a = Ka(q + \gamma z)$

$= 0.405(5 + 1.65z)$

At pt 'A' $z = 0$

$p_{a_1} = 2.025 \ t/m^2$

At pt 'B' $z = 1m$

$p_a = 2.69 \ t/m^2$

For soil (2)

(i) $\sigma_v = (q + \gamma_1 H_1) + \gamma_2' z + \gamma_w z$

(ii) $\sigma_h = K\sigma_v$

(iii) $p_a = Ka_2\{(q + \gamma_1 H_1) + \gamma_2' z_2\} + \gamma_w z$

At pt "B" $z = 0$

$p_a = 3.02 \ t/m^2$

At pt 'C' $z = 1.2m$

$p_a = 0.455\{(5 + 1.64 \times 1) + (1.8 - 1)1.2\} + 1 \times 1.2$

$p_a = 4.65 \ t/m^2$

For soil (3)

(i) $\sigma_v = (q + \gamma_1 H_1 + \gamma_2' H_2) + \gamma_3' z + \gamma_w z + \gamma_w H_2$

(ii) $\sigma_h = K \sigma_v$

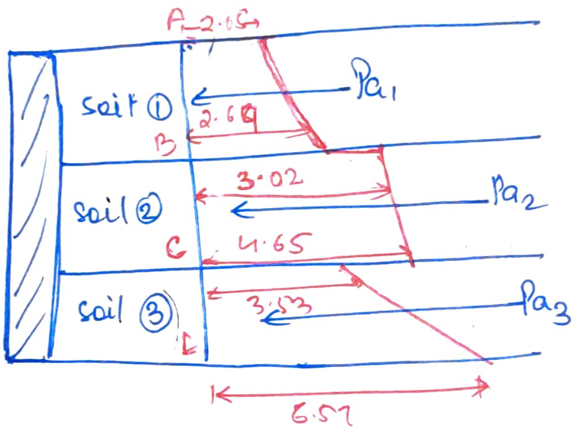
(iii) $p_a = k_{a_3} \{ (q + \gamma_1 H_1 + \gamma_2' H_2) + \gamma_3' z \} + \gamma_w z + \gamma_w H_2$

at point 'C' $z = 0$

$P_a = 3.53\ t/m^2$

at point 'D' $z = 2.3\ m$

$P_a = 6.57\ t/m^2$




$P_{a_1} = \frac{1}{2}(2.05 + 2.69) \times 1 = 2.37\ t/m$, $z_1 = 2.3 + 1.2 + \left(\frac{2.69 + 2 \times 2.05}{2.69 + 2.05}\right)\frac{1}{3}$

$= 3.972\ m$

$P_{a_2} = \frac{1}{2}(3.02 + 4.65) \times 1.2 = 4.602\ t/m$, $z_2 = 2.3 + \left(\frac{4.65 \times 2 \times 3.02}{4.65 + 3.02}\right) 1.2$

$= 2.857\ m$

$P_{a_3} = \frac{1}{2}(3.53 + 6.57)\, 2.3 = 11.615$

$z_3 = \left(\frac{6.57 + 2 \times 3.53}{6.57 + 3.53}\right)\frac{2.3}{3} = 1.05\ m$

$P_a = \Sigma P_{ai} = 18.5\ t/m$

$z = \frac{\Sigma P_{ai} z_i}{\Sigma P_{ai}} = 1.86\ m$

Q. A 5m high masonry retaining wall has to retain a backfill of sandy soil having a $\gamma = 1.82$ gm/cc and an angle of internal friction of 32°. The surface of the backfill is inclined at an angle 10° to the horizontal. Determine the magnitude & point of application of active thrust on the wall if backfill is saturated.

~~P = Pa₁ + Pa₂~~

$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}\right]$$

$p_a = K_a \gamma' z$

$$= 0.321$$

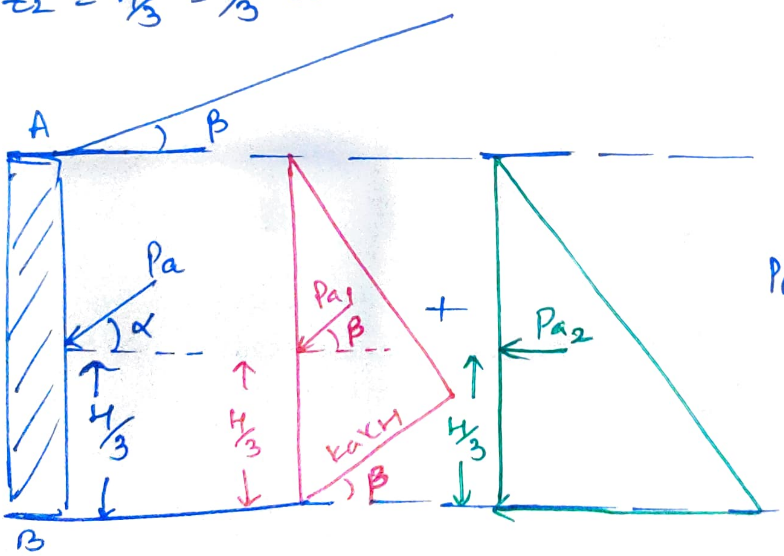
$$P_{a_1} = \frac{K_a \gamma' H^2}{2}$$

$$P_{a_1} = \frac{0.321 \times (18.2 - 10) \times 5^2}{2} = 32.9 \text{ kN/m}$$

$z_1 = H/3 = 5/3$ m at an angle 10° from horizontal.

$p_{a_2} = \gamma_w z$ , $P_{a_2} = \frac{\gamma_w H^2}{2} = \frac{10 \times 5^2}{2} = 125$ kN/m

$z_2 = H/3 = 5/3$ m




$$P_a = \sqrt{(P_{a_1}\cos\beta + P_{a_2})^2 + (P_{a_1}\sin\beta)^2}$$

$P_a = 157.53$ kN/m

$$\alpha = \tan^{-1}\left(\frac{P_{a_1}\sin\beta}{P_{a_1}\cos\beta + P_{a_2}}\right)$$

$\alpha = 2.07°$

Note :→ Rankine theory of earth pressure, overestimate active earth pressure & under estimate passive earth pressure.

# ACTIVE & PASSIVE EARTH PRESSURE FOR COHESIVE SOIL

→ Rankine's theory was assumed to be applicable for cohesionless soil, however it was taken forward by "BELL" to be applied cohesive soil.

→ As cohesive soil are partially self supporting soils (becoz of cohesion present in them), active pressure exerted by them is comparitively less than cohesionless soil, but passive pressure exerted by them is comparitively more than cohesionless soil.

Case (1) For Active State

From stress relationship of soil

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

for active state $\sigma_1 = \sigma_v$ , $\sigma_3 = \sigma_h$

$$\sigma_v = \sigma_h \tan^2\alpha + 2c\tan\alpha$$

$$\sigma_h = \sigma_v \cot^2\alpha - 2c\cot\alpha$$

$$\sigma_h = \sigma_v \cot^2(45° + \phi/2) - 2c(\cot 45° + \phi/2)$$

$$\cot^2(45° + \phi/2) = \frac{1-\sin\phi}{1+\sin\phi} = k_a$$

$$\Rightarrow \sigma_h = k_a\sigma_v - 2c\sqrt{k_a}$$

$$\boxed{p_a = k_a\sigma_v - 2c\sqrt{k_a}}$$

{ for cohesionless soil, it was $p_a = k_a\sigma_v$ }

(a) For dry or moist backfill

At pt 'A', $z = 0$

$p_a = -2c\sqrt{k_a}$ let at $z = z_0$ $p_a = 0$

$$p_a = k_a \gamma z - 2c\sqrt{k_a} = 0$$

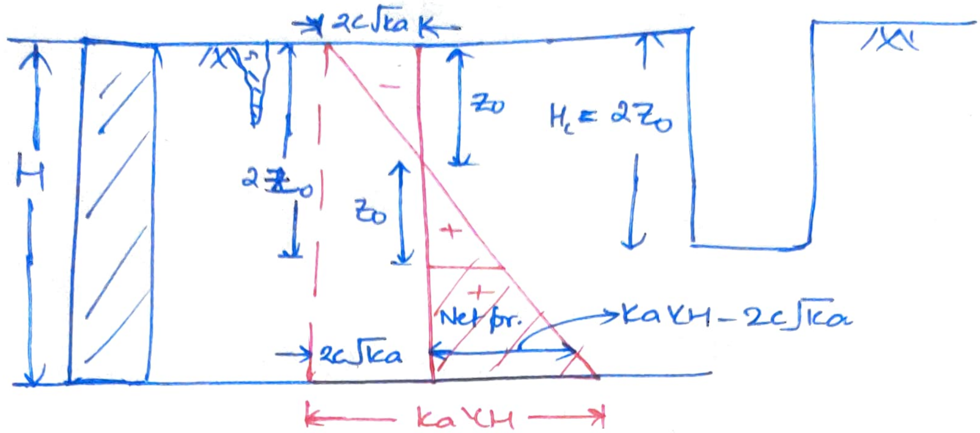
$$z_0 = \frac{2c\sqrt{k_a}}{\gamma k_a}$$

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

∴ This shows the negative pressure (i.e. tension) is developed at top level of retaining wall & it dec. to zero at depth $z_0 = \frac{2c}{\gamma\sqrt{k_a}}$

→ For depth $z > z_0$, at $z = H$

$$p_a = k_a \gamma H - 2c\sqrt{k_a}$$




Note :→ It can be reffered, that due to cohesion in soil, active pressure is reduced by $2c\sqrt{ka}$ throughout the ht. of wall.

→ Because of negative pr. a tension crack is usually developed is the soil near the top of wall, upto a depth of "$z_0$" hence soil does not neccearily remains adhered to the top portion of wall upto height "$z_0$", It is usual to neglect the "-ve" pressure & consider whole of the positive pressure below $z_0$

→ The net pressure upto a depth $2z_0$ is zero ie it means that a cohesive soil should be able to ~~attend~~ stand with a vertical face upto the depth of "$2z_0$" without any lateral support.

The critical height Hc of an unsupported vertical cut in cohesive soil is.

$$H_c = 2z_0 = \frac{4c\cancel{\sqrt{ka}}}{\gamma\sqrt{ka}} \quad \text{or} \quad \frac{4c}{\gamma}\tan\alpha$$

→ Practically this value of heigh (Hc) is less than theoritical value calculated above.

Consider the unit length of wall

Case (i) : when tension cracks are not developed

Total thrust on the wall

$$P_a = \int_0^H p_a (dz \times 1) = \int (k_a \gamma z - 2c\sqrt{k_a})\, dz$$

$$= k_a \gamma \left(\frac{z^2}{2}\right)_0^H - 2c\sqrt{k_a}\, |z|_0^H$$

$$P_a = \frac{k_a \gamma H^2}{2} - 2c\sqrt{k_a} H$$

or

$$P_a = \frac{1}{2}\left[k_a \gamma H - 2c\sqrt{k_a} + 2c\sqrt{k_a}\right]\left[H - 2z_0\right] \times 1$$

$$P_a = \frac{1}{2}\left[k_a \gamma H\right]\left[H - \frac{2 \cdot 2c}{\gamma\sqrt{k_a}}\right]$$

$$\boxed{P_a = \frac{k_a \gamma H^2}{2} - 2c\sqrt{k_a} H}$$

Line of action : $\bar{z} = \left(\frac{b+2a}{b+a}\right)\frac{H}{3}$ from the base

$$\bar{z} = \left(\frac{k_a \gamma H - 2c\sqrt{k_a} + 2 \cdot 2c\sqrt{k_a}}{k_a \gamma H - 2c\sqrt{k_a} + 2c\sqrt{k_a}}\right)\left(\frac{H - 2z_0}{3}\right)$$

case (ii) : when tension cracks are developed.

$$P_a = \int_{z_0}^H p_a (dz \times 1)$$

$$P_a = \int (k_a \gamma z - 2c\sqrt{k_a})\, dz \Rightarrow P_a = k_a \gamma \left|\frac{z^2}{2}\right|_{z_0}^H - 2c\sqrt{k_a}\, |z|_{z_0}^H$$

$$P_a = \frac{k_a \gamma H^2}{2} - 2c\sqrt{k_a} H + \frac{2c^2}{\gamma} \quad \text{or} \quad P_a = \frac{1}{2}(k_a \gamma H - 2c\sqrt{k_a})(H - z_0)$$

$$\boxed{P_a = \frac{k_a \gamma H^2}{2} - 2c\sqrt{k_a} H + \frac{2c^2}{\gamma}}$$

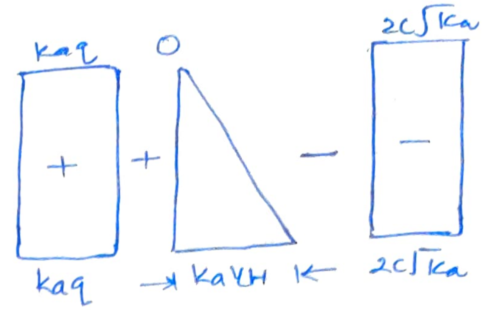
$$\bar{z} = \left(\frac{H - z_0}{3}\right) \text{ from base}$$

## II BACKFILL WITH UNIFORM SURCHARGE

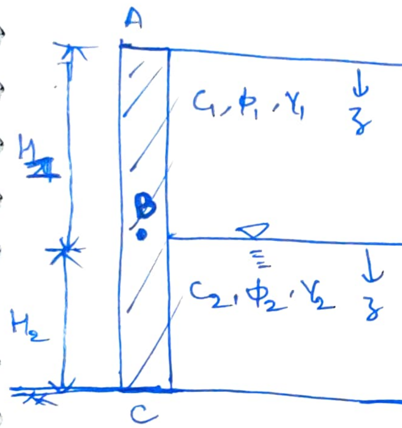
$$\sigma_v = q + \gamma z$$

$$\sigma_h = k_a \sigma_v - 2c\sqrt{k_a}$$

$$p_a = k_a q + k_a \gamma z - 2c\sqrt{k_a}$$




## III SUBMERGED BACKFILL WITH WATER TABLE AT DEPTH $H_1$




$$\sigma_v = \gamma z$$

$$\sigma_h = k_{a1} \sigma_v - 2c_1\sqrt{k_{a1}}$$

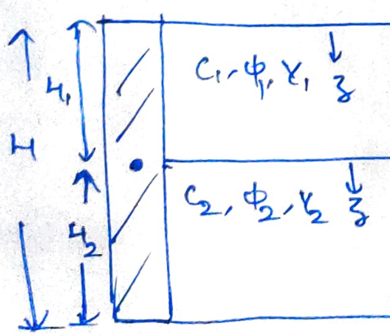
$$p_a = k_{a1} \gamma_1 z - 2c_1\sqrt{k_{a1}}$$

$$\sigma_v = \gamma_1 H_1 + \gamma_2 z$$

$$\sigma_h = k_{a2} \sigma_v - 2c_2\sqrt{k_{a1}}$$

$$p_a = k_{a2} \gamma_1 H_1 + k_{a2} \gamma_2 z - 2c_2\sqrt{k_{a2}}$$

## IV BACKFILL WITH DIFFERENT SOILS




$$\sigma_v = \sigma_1 z$$

$$\gamma_h = k_{a1} \sigma_v - 2c_1\sqrt{k_{a1}}$$

$$p_a = k_{a1} \gamma_1 z - 2c_1\sqrt{k_{a1}}$$

$$\sigma_v = \gamma_1 H_1 + \gamma_2 z$$

$$\sigma_h = k_{a2} \sigma_v - 2c\sqrt{k_{a2}}$$

$$p_a = k_{a2} \gamma_1 H_1 + k_{a2} \gamma_2 z - 2c_2\sqrt{k_{a2}}$$

## V BACKFILL OF INTACT SATURATED CLAY

The active (or passive) lateral pressure of intact saturated clay for temporary work or immediately after construction of retaining wall is calculated by considering $\phi = \phi_u = 0$

$\Rightarrow$ If $\phi = 0 \Rightarrow k_a = k_p = 1$

$$\sigma_v = \gamma' z + \gamma_w z$$

$$p_a = K_a \gamma' z - 2c\sqrt{K_a} + \gamma_w z$$
$$= \gamma' z - 2c + \gamma_w z$$

$$\boxed{p_a = \gamma_{sat} z - 2c}$$

## Case II : PASSIVE STATE

From stress relationship of soil

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

$\sigma_1 = \sigma_h$, $\sigma_3 = \sigma_v$ $\quad \therefore \sigma_h = \sigma_v \tan^2\alpha + 2c\tan\alpha$

$$\therefore K_p = \tan^2\alpha = \tan^2\left(45^\circ + \frac{\phi}{2}\right) = \frac{1+\sin\phi}{1-\sin\phi}$$

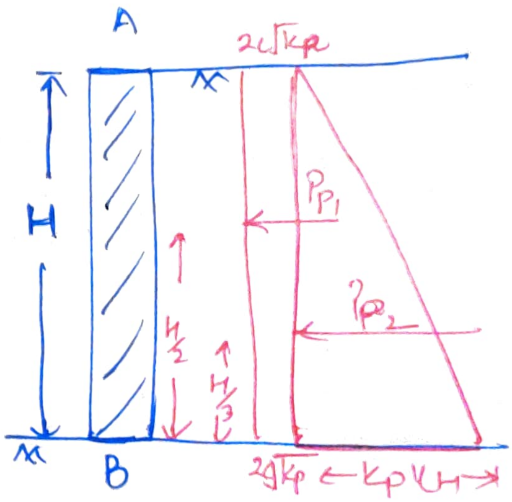
$$\boxed{p_p = \sigma_h = K_p \sigma_v + 2c\sqrt{K_p}}$$

If $\sigma_v = \gamma z$

$$p_p = K_p \gamma z + 2c\sqrt{K_p}$$

at pt "A", $z = 0$

$\therefore p_p = 2c\sqrt{K_p}$




At pt "B" $z = H$

$$p_p = K_p \gamma H + 2c\sqrt{K_p}$$

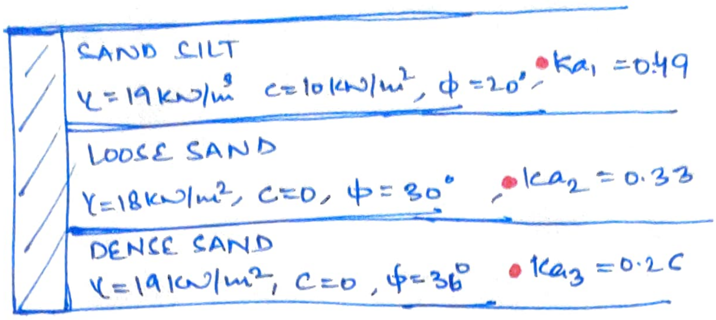
Note : $\longrightarrow$ effect of cohesion is increase passive earth pressure

$P_p = P_{p_1} + P_{p_2}$ $\qquad P_{p_1} = 2c\sqrt{K_p} \cdot H$, $z_1 = H/2$

$\qquad\qquad\qquad\qquad P_{p_2} = \frac{1}{2} K_p \gamma H^2$, $z_2 = H/3$

$$\boxed{\bar{z} = \frac{\Sigma P_{pi} z_i}{\Sigma P_{pi}}}$$

Q. A retaining wall of 4.5m height has to retain a stratified backfill as given. Find out the magnitude of total active thrust on the wall & locate the point of application.




Sol:

For soil ①

(i) $\sigma_v = \gamma_1 z$

(ii) $\sigma_H = Ka_1 \sigma_v - 2c_1\sqrt{Ka_1}$

(iii) $p_a = Ka_1 \gamma_1 z - 2c_1\sqrt{Ka_1}$

$p_a = 9.31z - 14$

- At pt. "A", $z = 0 \Rightarrow p_a = -14\ kN/m^2$
- At pt "B", $z = 2 \Rightarrow p_a = 4.62\ kN/m^2$
- At $z = z_0 \Rightarrow p_a = 0$ $\quad z = \frac{2c}{\gamma_1\sqrt{Ka_1}} = 1.5m$

For soil ②

(i) $\sigma_v = \sigma_1 H_1 + \gamma_2' z + \gamma_w z$

(ii) $p_a = Ka_2 \gamma_1 H_1 + Ka_2 \gamma_2' z + \gamma_w z$

At pt "B", $z = 0 \Rightarrow p_a = 12.54\ kN/m^2$

At pt "C", $z = 1 \Rightarrow p_a = 25.18\ kN/m^2$.

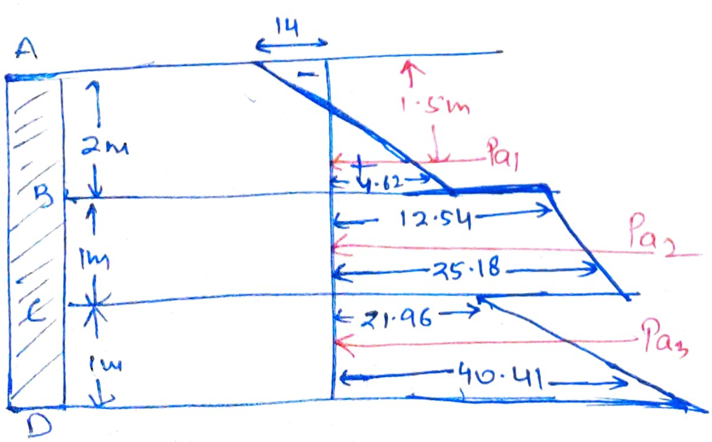
For soil ③

(i) $\sigma_v = (\gamma_1 H_1 + \gamma_2' H_2) + \gamma_3' z + \gamma_w z + \gamma_w H_2$

$p_a = Ka_3(\gamma_1 H_1 + \gamma_2' H_2) + Ka_3 \gamma_3' z + \gamma_w z + \gamma_w H_2$

At pt "C", $z = 0 \Rightarrow p_a = 21.98\ kN/m^2$

At pt "D", $z = 1.5m \Rightarrow p_a = 40.41\ kN/m^2$




$P_a = Pa_1 + Pa_2 + Pa_3 = 66.83\ kN/m$

$\bar{z} = \frac{\Sigma P_{ai} z_i}{\Sigma P_{ai}} = 1.018m$

Q A retaining wall, with a smooth horizontal backface has to retain a backfill, c–ϕ soil upto 5m above GL. The surface of the backfill is horizontal & it has following properties

$\gamma = 20\,kN/m^3$, $c = 15\,kN/m^2$, $\phi = 12^\circ$

(i) Plot the distribution of active earth pressure on the wall

(ii) Determine the magnitude & point of application of active thrust.

(iii) Determine the depth of zone of tension cracks.

(iv) Determine the intensity of fictitious uniform surcharge which if placed over the backfill can prevent the formation of tension cracks.

(v) Compute the resultant active thrust after placing the surcharge

Sol: (i) $\sigma_v = \gamma z$

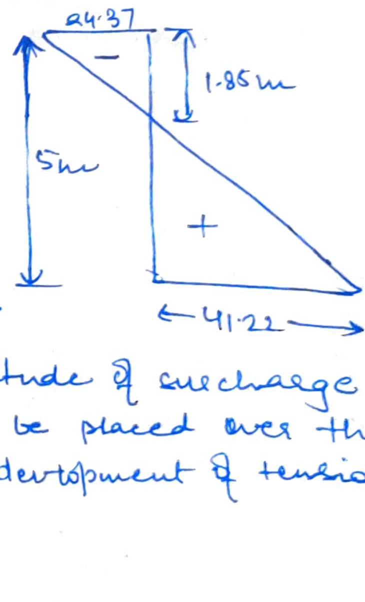
$\sigma_h = k_a \sigma_v - 2c\sqrt{k_a}$ $\qquad p_a = k_a \gamma z - 2c\sqrt{k_a}$

$p_a = 13.11\,z - 24.37$.

At pt "A", $z = 0 \Rightarrow p_a = -24.37\,kN/m^2$

(iii) At $z = z_0$, $\Rightarrow p_a = 0$, $z_0 = \dfrac{2c}{\gamma\sqrt{k_a}} = 1.85m$

At pt "B", $z = 5m \Rightarrow p_a = 41.22\,kN/m^2$




(ii) $P_a = \frac{1}{2} \times 41.22 \times (5 - 1.85)$

$P_a = 64.92\,kN/m$

$z = \left(\dfrac{5 - 1.85}{3}\right)$ $\qquad z = 1.05m$ ~~for~~ from Base.

(iv) ~~At pt "B", $z = 5m$~~

~~$p_a = k_a\left(\frac{2c}{\sqrt{k_a}} + \gamma z\right) - 2c$~~

(iv) let the magnitude of surcharge be $q\,kN/m^2$ to be placed over the backfill for no devlopment of tension crack.

$\sigma_v = q + \gamma z$

$\sigma_h = k_a \sigma_v - 2c\sqrt{k_a}$ $\Rightarrow$ $p_a = k_a(q + \gamma z) - 2c\sqrt{k_a}$

For no tension crack to be devloped

$\Rightarrow z = 0$, $p_a = 0$ $\Rightarrow$ $0 = k_a(q + 0) - 2c\sqrt{k_a}$

$q = \dfrac{2c}{\sqrt{k_a}} = 37.04\,kN/m^2$

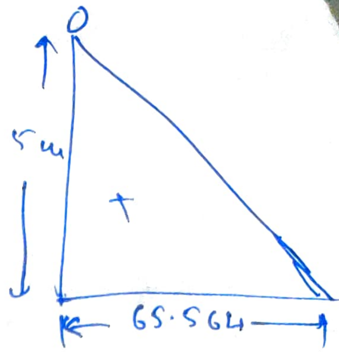
(v) At pt 'B', $z = 5m$

$p_a = K_a\left(\frac{2c}{\sqrt{K_a}} + \gamma z\right) - 2c\sqrt{K_a}$

$p_a = 65.564\ kN/m^2$ · $P_a = \frac{1}{2} \times 65.564\ kN/m^2$

$= 164\ kN/m$

$\bar{z} = 5/3\ m$ from Base




# Coulomb's Wedge Theory

→ Instead of considering equilibrium of an element within the mass of the soil material this theory considered the equilibrium of whole of soil material supported by a retaining wall. when the wall is on the point of moving slightly away from fill.

→ The wedge theory of earth pressure is based on concept of sliding wedge which is torn off from the rest of the backfill.

→ Assumptions of wedge theory

→ soil is homogeneous, isotropic, elastic, dry semi-infinite & cohesionless.

→ The face of wall in contact with backfill ie vertical or inclined & is rough.

→ The failure wedge acts a rigid body & stress over it are uniformly distributed.

→ The failure is essentially two dimensional & rupture surface is planar & passes through the heel of the wall.

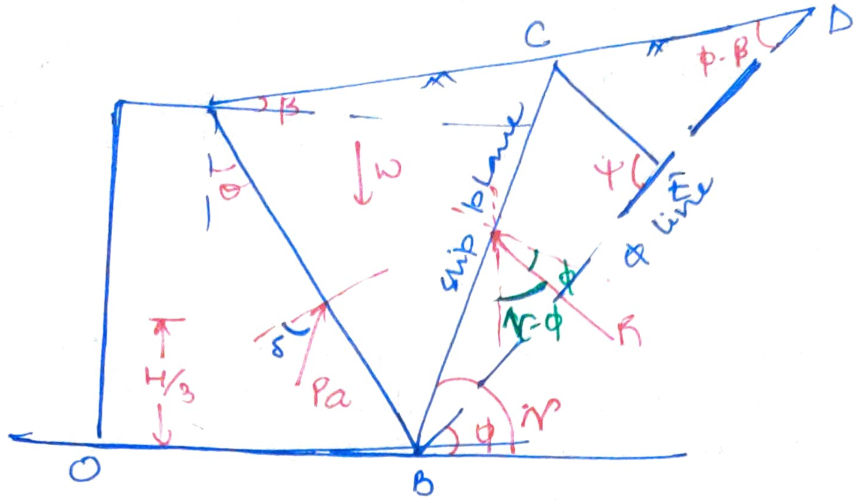
→ The location & distn of resultant thrust b/wn wall & fill is known. The point of application is taken at the lower third pt of the wall by assuming triangular distn of earth pressure distribution of earth pressure.

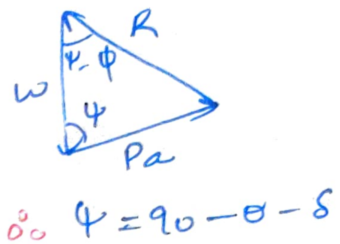
→ The forces acting on wedge of soil are

(i) its weight (W) [soil wedge] → vertically downwards.

(ii) Resultant soil reactn "R" → acts at downward angle $\phi$, with normal to soil plane.

(iii) Resultant thrust '$P_a$' btw wall & soil, act as downward angle [$\delta$] with normal to inclined face of wall




$\therefore \psi = 90 - \theta - \delta$

→ For the condⁿ of yield of wall from the backfill the most dangerous / critical slip surface is that for which the wall reaction is maximum. (for active case) [for passive case it must be min$^m$] i.e wall must resist the max$^m$ lateral pressure before it ~~wo~~ moves away from the fill.

→ Condition for max$^m$ pressure from a sliding wedge

→ BD shows an inclined, at an angle of $\phi$ to the horizontal at which soil is expected to stay in the absence of any lateral support i.e this line BD is called natural slope line / repose line / $\phi$ line

→ The value of Pa depends upon angle of slip "$\gamma$"

→ Pa is zero when $\gamma = \phi$, As $\gamma$ increases beyond $\phi$, Pa also increase reaches it max value & again starts decreasingly & becomes zero when $\gamma = 90 + \theta$.

→ Thus the critical slip plane lies b/w $\phi$ line & back of wall

→ It can shown that the criteria for max$^m$ active pressure is that the slip plane is so chosen that — area of $\Delta$ ABC is equal to area of $\Delta$BCE

Area of $\Delta$ABC = Ar. of $\Delta$BCE

→ And active thrust corresponding to this condn is given by

$$P_a = \frac{1}{2}\gamma H^2 \left[ \frac{\sec\theta \cdot \cos(\phi-\theta)}{\sqrt{\cos(\theta+\delta)} + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\beta-\theta)}}} \right]^2$$

• $P_a = \frac{1}{2} K_a \gamma H^2$, $\bar{z} = H/3$ from base, at an upward angle of $\delta$ with normal to face.

$$K_a = \left[ \frac{\sec\theta \cos(\phi-\theta)}{\sqrt{\cos(\theta+\delta)} + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\beta-\theta)}}} \right]^2$$

$\beta$ = surcharge angle

$\theta$ = angle made by face of wall with vertical

$\phi$ = frictional angle of soil

$\alpha$ = angle made by rupture / slip plane with horizontal.

$\delta$ = angle of friction b/wn wall & soil

→ The value of "$\delta$" is different to be calculated hence is assumed as follows –

(i) for smooth wall $\delta = \phi/3$

(ii) for ordinary retaining wall $\delta = 2/3\,\phi$

(iii) for rough wall with well drained backfill $\delta = 3/4\,\phi$

(iv) for backfill subjected to vibration $\delta = 0$

CASE I $\theta = \beta = 0$, $\delta = \phi$

$$K_a = \left( \frac{\cos\phi}{\sqrt{\cos\phi} + \sqrt{\sin 2\phi \sin\phi}} \right)^2$$

$$K_a = \frac{\cos\phi}{(1+\sqrt{2}\sin\phi)^2}$$

CASE II $\theta = \beta = \delta = 0$

$$K_a = \frac{\cos^2\phi}{(1+\sin\phi)^2} = \frac{1-\sin^2\phi}{(1+\sin\phi)^2}$$

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

Hence it can be concluded that "Rankine's theory" is simplified form of coulombs theory.

It can also be concluded that Rankine theory overestimate active earth pressure.

→ In coulomb's failure theory, failure wedge is assumed to be planar, but in actual it is spiral, specially in passive case, hence this theory is not preferred for passive earth pressure computation.

Note → Earth pressure can also be computed using ~~Rankine's~~ Rebhann's theory & Culmann's theory, which are graphical approach.

→ Culmann's theory is based on coulomb's theory only as it assume failure surface to be planner.

→ Rebhann's theory is also termed as "Poncelet's Mtd."

Q. A wall of height 5m retains a backfill of dry granular soil that has a level surface & weight 20 kN/m³. When there is no surcharge above the fill, the overturning moment caused by total active pressure at a base of the wall is 125 kN/m length of the wall. The specifications permits certain amount of uniformly distributed surcharge but state that surcharge must not increase the overturning moment by more than 50%. What surcharge can be allowed if angle of wall friction is 20°.

(i) $\delta = 20°$, $\beta = 0$, $\theta = 0°$

$$M_0 = P_a \cdot \frac{H}{3} = \frac{1}{2} K_a \gamma H^2 \cdot \frac{H}{3} = \frac{K_a \gamma H^3}{6}$$

$$\frac{K_a \gamma H^3}{6} = 125 \Rightarrow \frac{K_a \times 20 \times 5^3}{6} = 125 \Rightarrow K_a = 0.3$$

$$K_a = \left[\frac{\sec\theta \cos(\phi - \theta)}{\sqrt{\cos(\theta + 20°)} + \sqrt{\frac{\sin(\phi + 20°)\sin(\phi - \theta)}{\cos(\theta - \theta)}}}\right]^2$$

~~O.S~~ $\phi = 29.76°$

In second case $M_0' = 1.5 \times 125 = 187.5$ kN/m

$K_a' = 1.5 K_a = 1.5 \times 0.3 = 0.45$

$$K_a' = 0.45 \left[ \frac{\sec\theta \cos(29.76 - 0)}{\sqrt{\cos(0+20^\circ)} + \sqrt{\frac{\sin(29.76+20)\,(\sin(29.76-\beta))}{\cos(\beta - 0)}}} \right]^2$$

$$\boxed{\beta = 22.4^\circ}$$

# SHEET PILE WALL

→ Sheet pile wall consist of no. of sheet piles driven side by side to form continous vertical wall into the medium, which is used to retain earth mass or water.

→ Sheet piles walls are generally used in water front structures temporary constn, river training works to prevent piping & failure below the dam & to prevent the wall under excavation from failure.

→ Material used for construction of sheet pile should be strong, light in weight and thin in section.

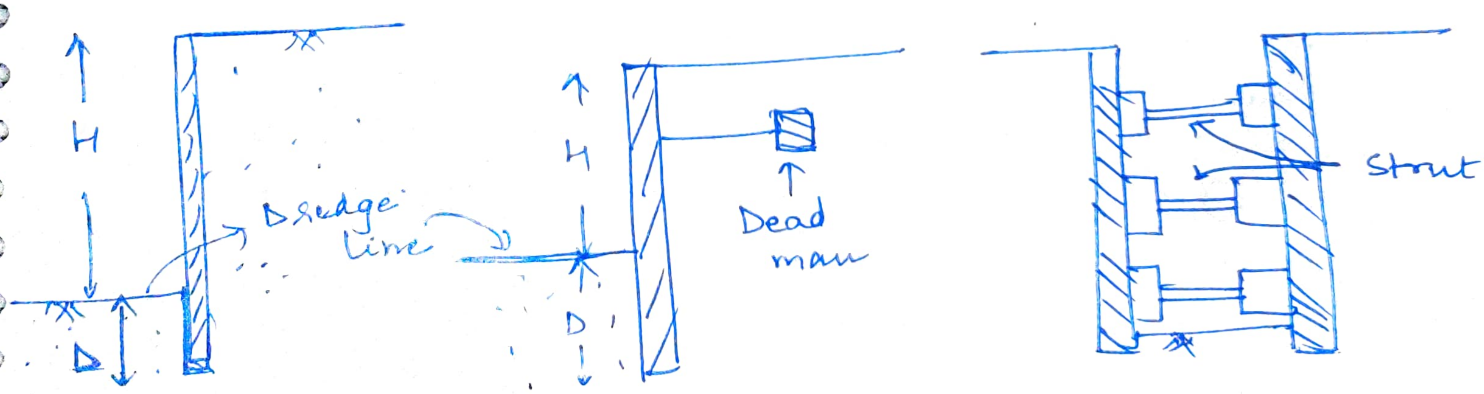
→ Sheet pile wall must have sufficient depth of embankment (D) to prevent over ~~thining~~ turning.

→ Sheet piles are generally of 3 types —

(i) Cantilever Sheet Pile

(ii) Anchored Sheet Pile

(iii) Braced Sheeting.



→ Certain configuration of sheet piling are termed as "BULKHEADS OR COFFER DAMS."

→ A bulkheads is a sheet pile retaining wall of water front & bracket up ~~the~~ by ground.

→ A cofferdam is a seasonable water-tight enclosure made of sheet pile wall, usually tempeary, built around a working area for the purpose of excluding water during construction.

→ Sheet pile walls are employed as bulkheads in piers, docks, harbours, sea walls, break waters & other shore protection works.

# CANTILEVER SHEET PILE WALL

→ A cantilever sheet pile wall or bulkhead derives its stability entirely from the lateral ~~pressure~~ resistance of the soil into which it is driven.

→ The bulk is adequetly embedded into the soil below the dredge line so that a driven line of sheeting act as wide cantilever beam in resisting the lateral earth pressure developed above the dredge line.

→ A cantilever bulkhead is used for ~~a~~ moderate height only.

## (i) Cantilever sheet pile in granular soil

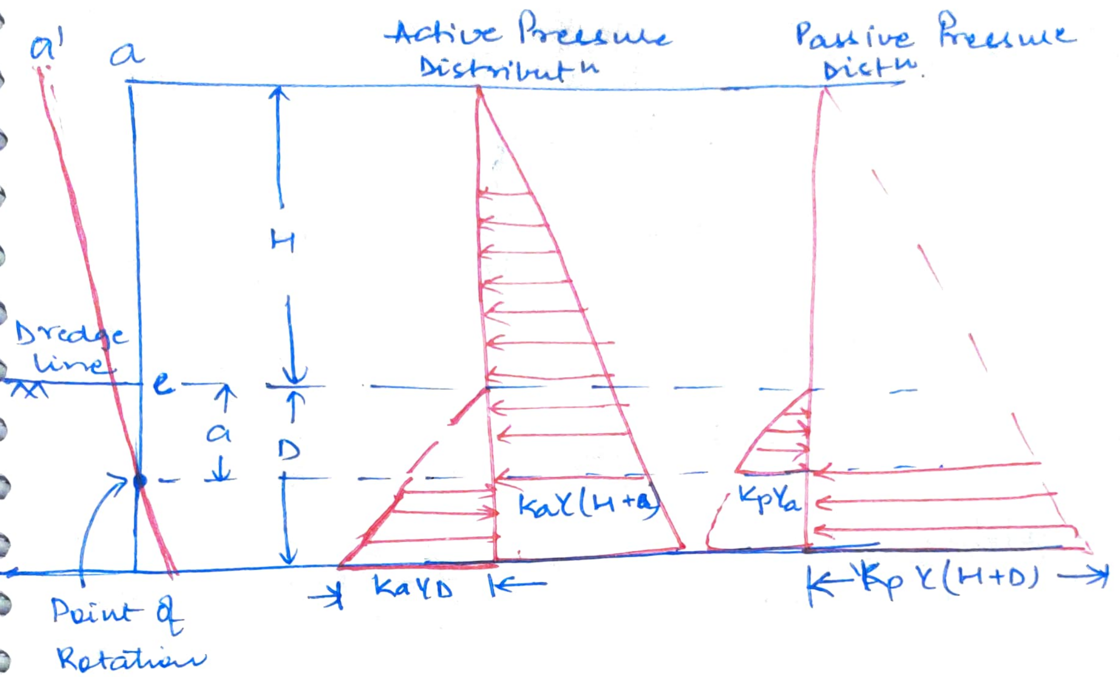
→ The cantilever sheet pile is assumed to be rigid. The pile tends to rotate about a point "O", deflecting away from the backfill.

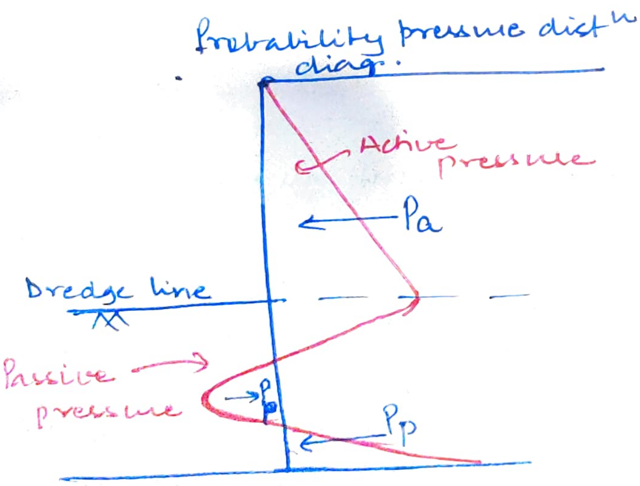
→ Above the point of rotation, the ~~rotation~~ sheet pile deflects away from the backfill, thus generating active conditions on the back of the wall. At the same time, b/w^n the dredge line (point e) & the point of rotation (point O), at the wall tends to moves towards the soil in front of the wall passive cond^n are generated on this side.

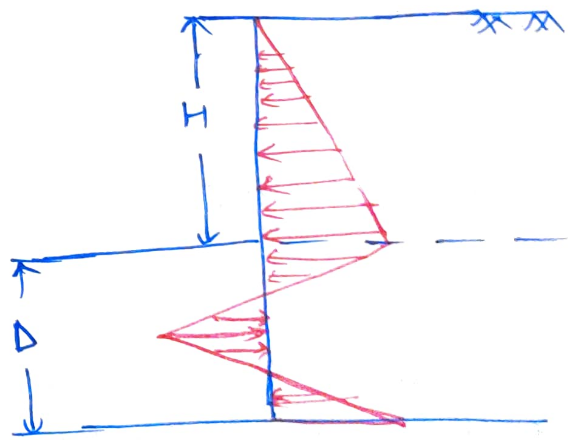
→ However, below the point of rotation the active & passive cond^n generated on the two side are reversed.

→ Stability ~~B~~ the sheet pile wall is provided by passive pressure over the embedded depth of sheet pile

→ Probable earth pressure diagram is comparitively difficult to analyse, hence simplified earth pressure distribution dia is used for theoritical analysis.



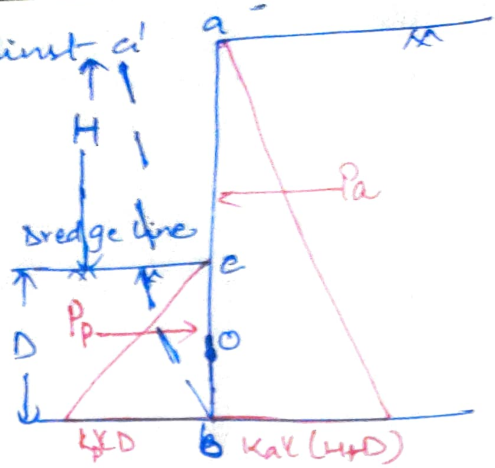




→ For simplicity in computation it is ~~measured~~ assumed that sheet pile overturns about the base point "b" instead of "O".

→ Hence it is subjected to active pressure on its right face ab & passive pressure on its left face cb below dredge line.

→ In order to ensure safety against overturning.




$$\Sigma M_b = 0$$

$$P_a\left(\frac{H+D}{3}\right) - P_p\left(D/3\right) = 0$$

$$\frac{1}{2} k_a \gamma (H+D)^2 \left(\frac{H+D}{3}\right)$$

$$-\frac{1}{2} K_p \gamma D^2 \frac{D}{3} = 0$$

from above eq "D" can be found

→ Factor of safety against overturning

$$F.O.S = \frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{P_p \times D/3}{P_a\left(H+D/3\right)}$$

$$FOS = \frac{\frac{1}{2} K_p \gamma D^2 \cdot D/3}{\frac{1}{2} k_a \gamma (H+D)^2 \left(\frac{H+D}{3}\right)}$$

Note: → If FOS is not given, depth of embankment is increased by 20-40%.

(ii) Cantilever sheet pile in cohesive soil

→ The analysis of cantilever sheet pile in cohesive soil is carried out in almost same manner to that in granular soil.

→ However certain phenomena such as consolidation of clay in passive pressure zone, formation of tension cracks in active zone may need to considered seperately.

→ Further clay may shrink & loose contact with the wall, to account for this benefit of wall ~~cohesion~~ adhesion is neglected in design.

→ If $\phi_u = 0$

$K_a = K_p = 1$

→ Active earth pressure on right face of wall

$$p_a = k_a \sigma_v - 2c\sqrt{k_a} = \gamma z - 2c$$

At point "a", $z = 0 \Rightarrow p_a = -2c$

At point "d", $z = z_0 \Rightarrow p_a = 0, \; z_0 = \frac{2c}{\gamma}$

At point "e", $z = H \Rightarrow p_a = \gamma H - 2c$

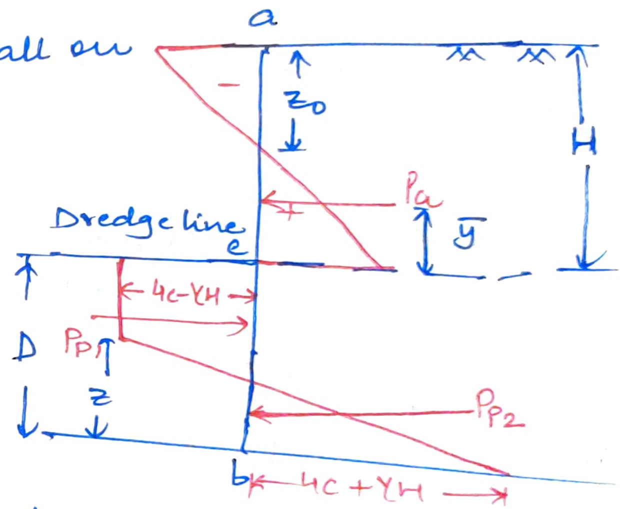
→ passive pressure on left face of wall below dredge.

$$p_{pnet} = (p_p - p_a)_e = k_p \gamma z + 2c\sqrt{k_p} - (\gamma H - 2c)$$

$$p_{pnet} = 2c \cdot 1 - \gamma H + 2c$$

$$p_{pnet} = 4c - \gamma H$$

→ passive pressure at the base of wall on right face




$$P_{pnet} = (p_p - p_a)_b$$

$$= 1 \cdot \gamma (H+D) + 2c - (1 \cdot \gamma D - 2c)$$

$$= \gamma H + \gamma D + 2c - \gamma D + 2c$$

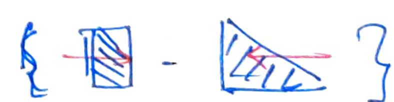
$$P_{pnet} = 4c + \gamma H$$

→ For calculation of D & z $\quad \{ \overset{+}{\leftarrow} \; \rightarrow \}$

$$\Sigma F_H = 0$$

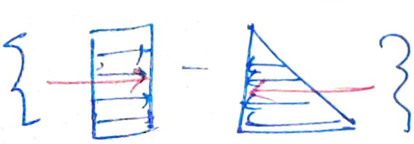
$$P_a + \frac{1}{2}\left(4c + \gamma H + 4c - \gamma H\right) z - (4c - \gamma H) D = 0$$

$$P_a + \frac{z}{2}(8c) - (4c - \gamma H) D = 0 \quad \text{—(i)}$$



$$\Sigma M_b = 0$$

$$P_a(\bar{y} + D) - (4c - \gamma H) D \cdot D/2 + z/2 (8c) z/3 = 0 \quad \text{—(ii)}$$



# ANCHORED BULKHEAD

→ The stability of an anchored sheet pile depends not only on the passive earth resistance but also on the anchor ~~head~~ rod.

→ The driving ~~head~~ depth that is required in an anchored sheet pile is thus, comparitively smaller than in a cantilever sheet pile.

→ The total length of sheet pile is reduced and will lead to the economy where the height of sheet pile above the dredge line is not small.

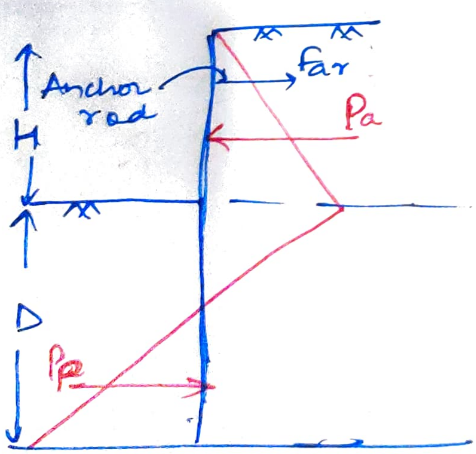
→ For analysis of anchored sheet generally "FREE-EARTH SUPPORT MTD" & "FIXED EARTH SUPPORT MTD" is used.

## (i) Free Earth Support Method

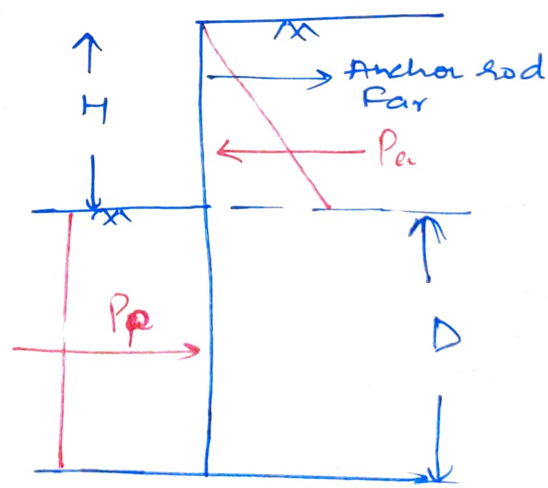
It is based on the assumption that.

(a) sheet pile is rigid as compared to surrounding soil and may rotate at the level of anchor rod.

(b) passive earth pressure devlops in the soil in front of the piling & active pressure devlops in the soil at the back of piling.




(a) Granular soil

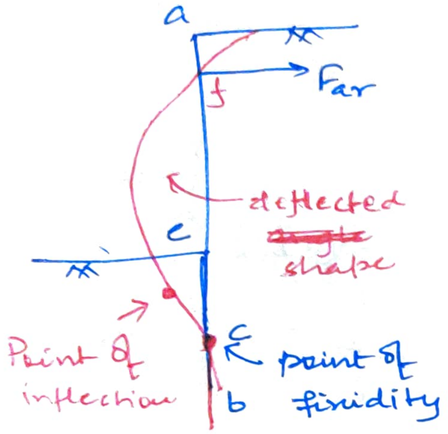


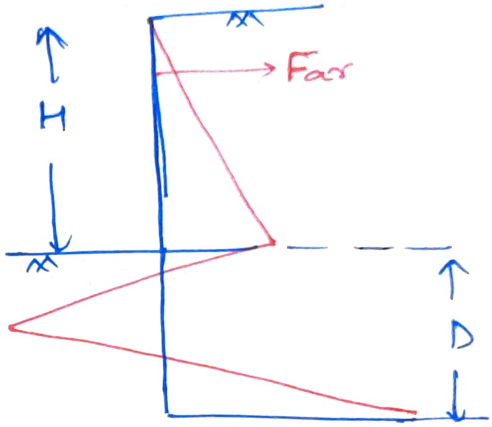

(b) Cohesive soil

## (ii) FIXED EARTH MTD

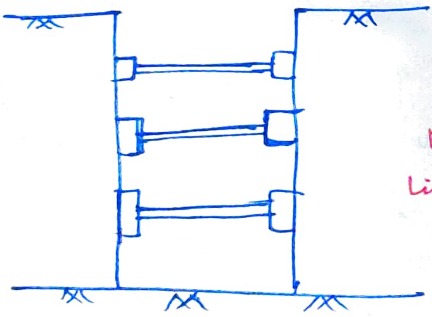
→ When the anchored bulkhead penetrates to a considerable depth so that the lower end of the wall is perfectly fixed and the wall acts as a vertical propped cantilever, Here the wall is ~~soil~~ said to be fixed earth support types.

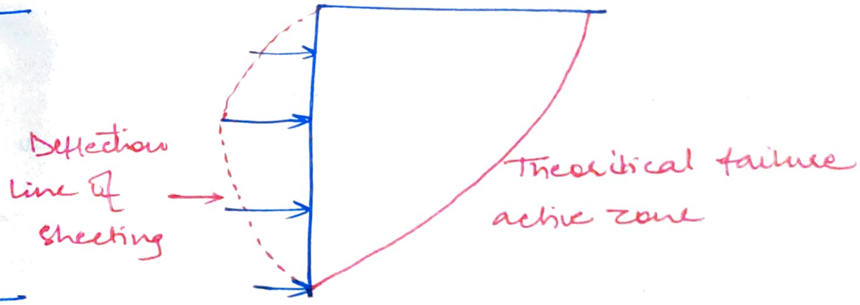
→ In this case a certain point above the base "c" is the point of fixidity, ie below which the sheet pile remains vertically straight.



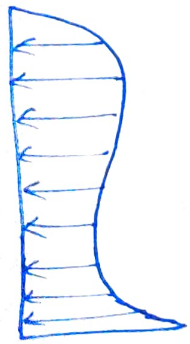



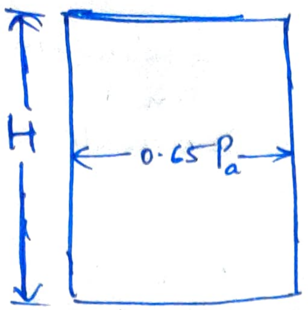

## iii) BRACED COFFER DAMS OR STRUTTED EXCAVATION



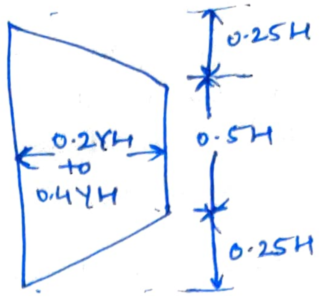



### Actual pressure distribution diagram

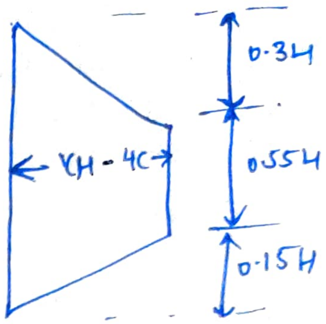





moist & dense sand

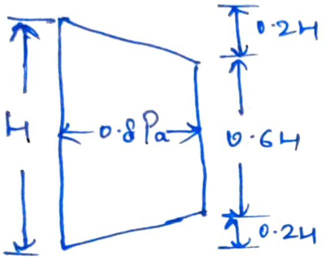



clay ($\gamma H / c_u < 4$)
(Peck, Hanson, Thornburn)

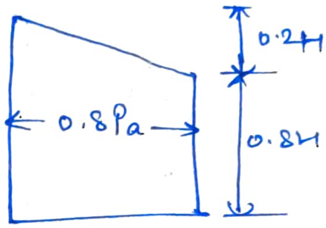



Clay ($\gamma H / c_u > 4$)

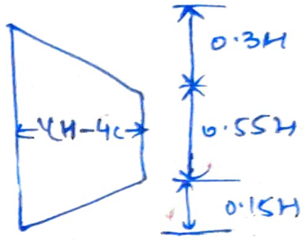
Apparent earth Pressure dia for cuts




Terzaghi & Peck
(Dense sand)

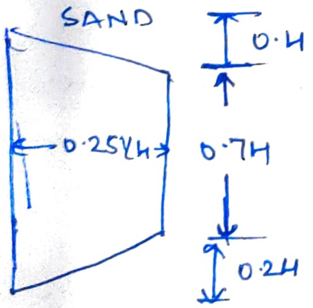


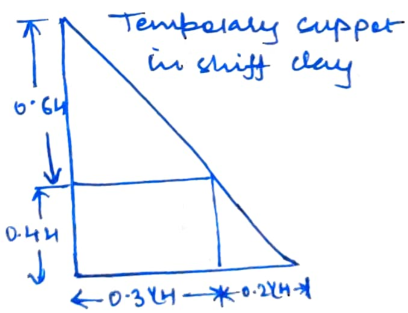

Terzaghi & Peck
(loose Sand)

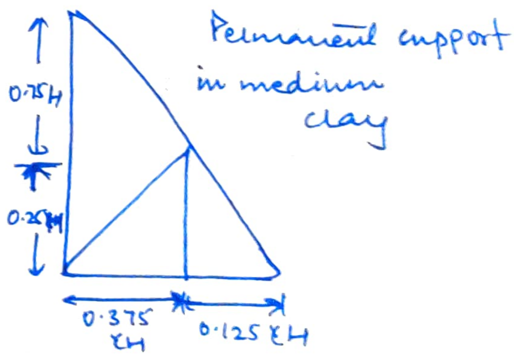



Peck
(clay)

Earth pressure on strutted excavation




"Tschebotarioff's"

→ Note :→ The bracing for a cut are composed of several units, as compared to rigid retaining wall which is a single unit.

→ Hence in view of this, it is necessary that the stability of each unit/member be examined.

→ The strut being the major unit, the estimation of strut load is of prime importance.

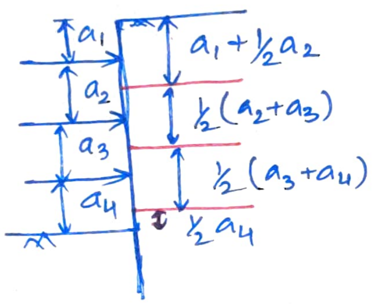
→ The Terzaghi & peck proposed following simplified procedure for the same.

~~(i) Hence in view of this, it is necessary that the stability of each unit/member be examined.~~

(i) It is assumed that the load on each strut is equal to the total earth pressure acting on the sheeting over a rectangular area extending horizontally half the distance to next vertical row on either side.

(ii) The earth pressure is assumed to be uniformly distributed over rectangular area.

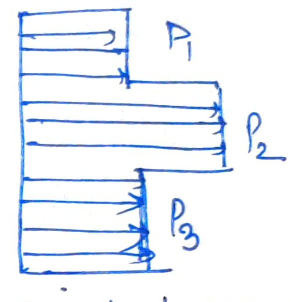
(iii) For purpose of calculating, the bottom of cut is assumed as strut.




$$P_1 = \frac{Q_1/b}{(a_1 + a_2/2)\cdot 1}$$

$$P_2 = \frac{Q_2/b}{\frac{1}{2}(a_2+a_3)\cdot 1}$$

$$P_3 = \frac{Q_3/b}{\frac{1}{2}(a_3+a_4)\cdot 1}$$




Apparent pressure distn diag.

Q: A 6m deep excavation in sand is supported by smooth wall vertical wall. The backfill is horizontal & supports a surcharge of 100 kN/m² on it ~~surface~~ surface. Properties of sand are sand are, $c' = 0$, $\phi = 30°$, $\gamma = 16\,kN/m^3$ - what should be the embedment depth if the wall is cantilever sheet pile wall in clay when $c' = 35\,kN/m^2$, $\gamma = 18\,kN/m^3$

For portion ae, active on right face

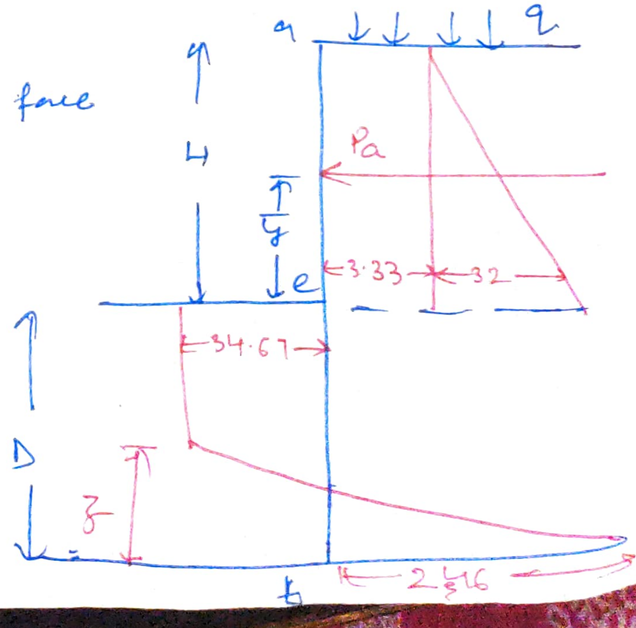
$p_{a_R} = p_{a_1} + p_{a_2}$

$p_{a_1} = K_a q H = 20\,kN/m$

$p_{a_2} = \frac{1}{2} K_a \gamma_1 H^2 = 96\,kN/m$

$z_1 = D + H/2 = D + 3\,m$

$z_2 = D + \frac{H}{3} = D + 2\,m$



$$\bar{z} = \frac{\Sigma P_{ai} z_i}{\Sigma P_{ai}} = 2.17 + D\,m, \quad P_a = 1.76\ kN/m$$

For portion eb, passive pressure at pt "e" on left face

$$p_p = (P_p - P_a)_e = 2c' - (k_a q + K_a \gamma H)$$

$$= 34.67\ kN/m^2$$

passive pressure at pt "b" on right ~~please~~ face

$$p_p = (P_p - P_a)_b = (q + \gamma_1 H_1 + \gamma_2 D) K_p + 2c - (\gamma_2 D - 2c)$$

$$P_p = 246\ kN/m^2$$

For equilibrium of wall, $\Sigma F_H = 0$

$$P_a - 34.67 D + \frac{1}{2}(246 + 34.67) z = 0$$

$$116 - 34.67 D + 140.33 z = 0 \quad \text{— (i)}$$

$$\Sigma M_b = 0$$

$$P_a \bar{z} - 34.67 D \cdot \frac{D}{2} + \frac{1}{2}(246 + 34.67) z \cdot \frac{z}{3} = 0$$

$$1.6 \times (2.17 + D) - 17.33 D^2 + 46.77 z^2 = 0 \quad \text{— (ii)}$$

$$z = 1.37\ m$$

$$D = 8.9\ m$$

Increase the depth of embankment by 25%.

$$D' = 1.25 D = 1.25 \times 8.9$$

$$D' = 11.125\ m$$

# Stability of Slopes

→ Slopes may be artificial man made as in cuttings, embankment for highways, railway earth dams, temporary excavation, tips & spoil heaps, landscaping operations etc.

→ Slopes may be natural as in hillside and valleys, coastal areas, rivers cliff etc.

→ In all above cases, a force exists which tends to cause the soil to move from top point to low i.e there exist a force or inherent tendency in slopes to assume more stable configuration

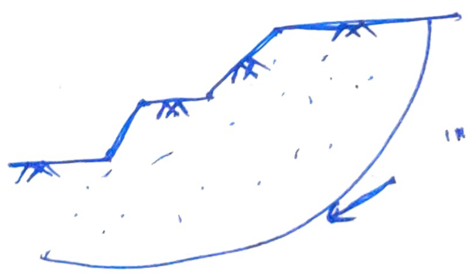
→ If there is only tendency to move it can be considered instability & if actual movement of soil takes → "slope failure".

→ Failure of the slope takes place due to following forces —

- Gravity force
- Seepage force
- Earthquake
- Sudden drawdown of water table
- Erosion of soil
- Excavation near the slope

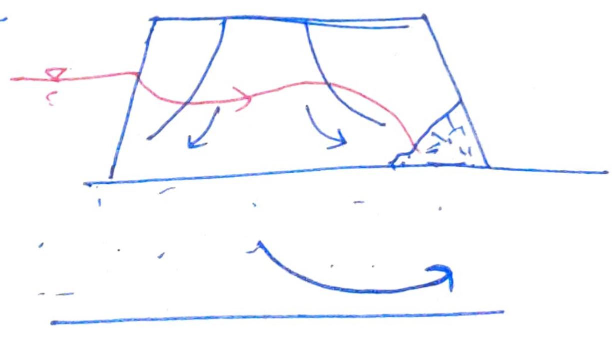
→ Slope stability problems may be found in following cases

"Natural slope"



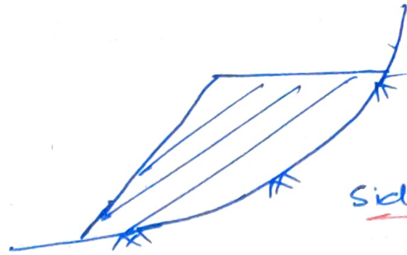
"Cut slope"

Open excavation

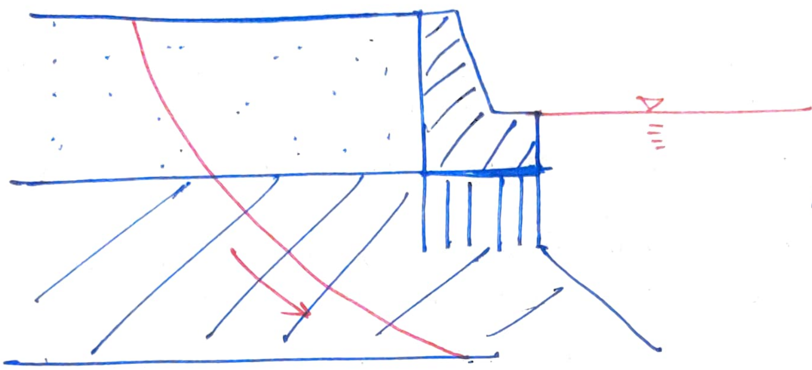


Eartherm Dam

Embankment over Poor soil



Side hill fill



water front structure

→ stability of the slope can be enhanced by suitable geometric configuration / modification such as flatter slope & by effectively improving internal & external drainage or sometimes by reinforcement of soil

→ An analysis of stability of slopes consists of two parts

i) The determination of the most severly stressed internal surface & the magnitude of the shearing stress to which it is subjected

ii) The determination of shearing strength along the surface

→ The shearing stress to which any slope can be subjected depends upon the unit wt. of the material & geometry of the slope.

→ whereas the shearing which can be mobilized to resist the shearing stress depends on the type of soil, its density & drainage.

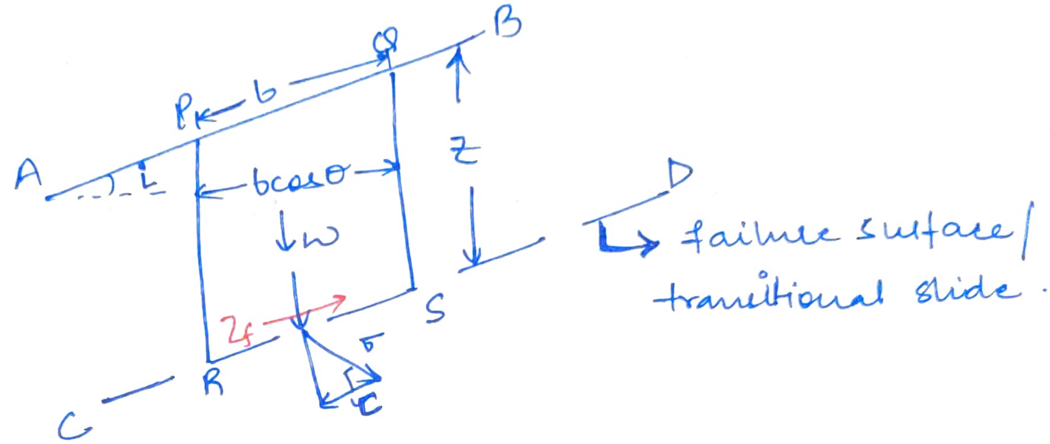
→ Slopes may be of two types —

(a) infinite slope

(b) finite slope

(I) Infinite Slope :→ If a slope represented the boundary surface of a semi-infinite soil mass, & soil properties of all identical depth below the surface are constant, it is called an "infinite slope".

# STABILITY ANALYSIS OF INFINITE SLOPE




Note :→ Infite slope practically do not exist.

- Let AB be an infinite slope with slope angle "i" to horizontal & For infinite slope soil properties & soil stresses on any plane ∥ to slope surface are identical hence the failure of slope usually involves sliding of soil mass along a plane parallel to the slope at some depth.

→ Let CD be that plane at depth of "z" below the surface — Consider a prism of soil PQRS of inclined length "b" along the slope & depth "z" from the surface.

weight of prism $W = \gamma\,(b\cos i \cdot z \cdot 1)$

{considering unit length of slope in plane of screen/copy}

Vertical stress on $\sigma_z = \dfrac{W}{b\times 1} = \dfrac{\gamma b\cos i \cdot z}{b\times 1}$

$$\sigma_z = \gamma z\cos i$$

normal to the surface CD $\sigma = \sigma_z\cos i = \gamma z\cos^2 i$

Tangent stress to the surface CD $\tau = \sigma_z\sin i = \gamma z\cos i\sin i$

→ The tangential component "$\tau$" called as shear stress induces failure ~~stag~~ along surface "CD"

→ while it is resisted by shear strength "$\tau_f$" of soil.

→ Hence FOS of slope against sliding is

$$FOS(F) = \frac{\text{Resisting force}}{\text{Deforming force}} \Rightarrow \boxed{F = \frac{\tau_f}{\tau}}$$

→ Now shear strength $\tau_f$ consists of both cohesion & internal friction

(i) CASE (i) : COHESIONLESS SOIL ($c=0$)

(a) Dry moist slope

$$\tau_f = c + \sigma \tan\phi \qquad (c=0)$$

$$\tau_f = \sigma \tan\phi \quad , \quad \sigma = \gamma z \cos^2 i \, , \; \tau = \gamma z \cos i \sin i$$

$$\tau_f = \gamma z \cos^2 i \tan\phi \, , \Rightarrow F = \frac{\tau_f}{\tau_f} = \frac{\gamma z \cos^2 i \tan\phi}{\gamma z \cos i \sin i}$$

$$\boxed{F = \frac{\tan\phi}{\tan i}}$$

→ If $i < \phi \Rightarrow \tan i < \tan\phi \Rightarrow F > 1$

"SLOPE IS STABLE"

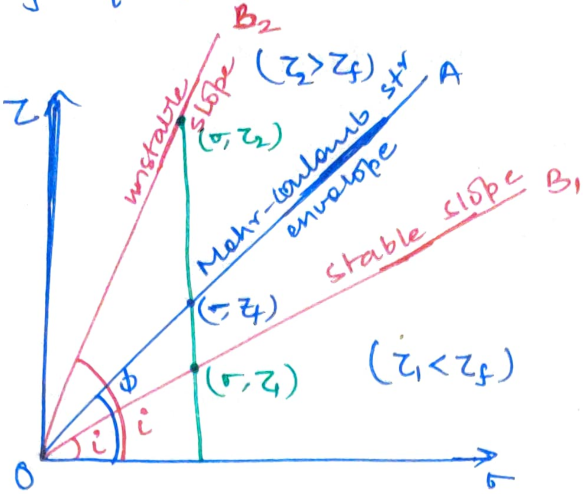
→ If $i > \phi \Rightarrow \tan i > \tan\phi \Rightarrow F < 1$

"SLOPE IS UNSTABLE"

→ If $i = \phi \Rightarrow \tan i = \tan\phi \Rightarrow F = 1$

Note : → For a given value of normal stress "$\sigma$" failure will not occur as long as "$\tau$" is smaller than "$\tau_f$" or as long as $i < \phi$

→ In limiting case of stability, the angle of slope "$i$" is referred as angle of repose.




b) SUBMERGED SLOPE :→ If slope is submerged stability is governed as —

$\sigma_z = \gamma' z \cos i$ , $\sigma = \gamma' z \cos^2 i$ , $\tau = \gamma' z \cos i \sin i$

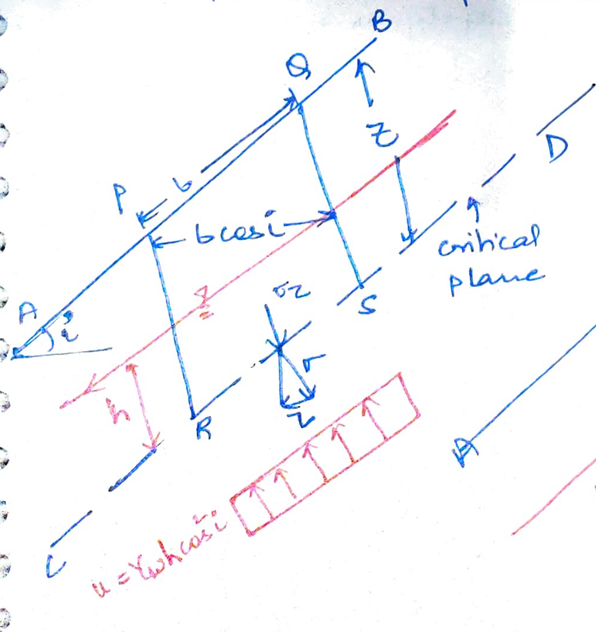
$\tau_f = c + \sigma \tan\phi$ [$c = 0$] $\Rightarrow$ $\tau_f = \gamma' z \cos^2 i \tan\phi$

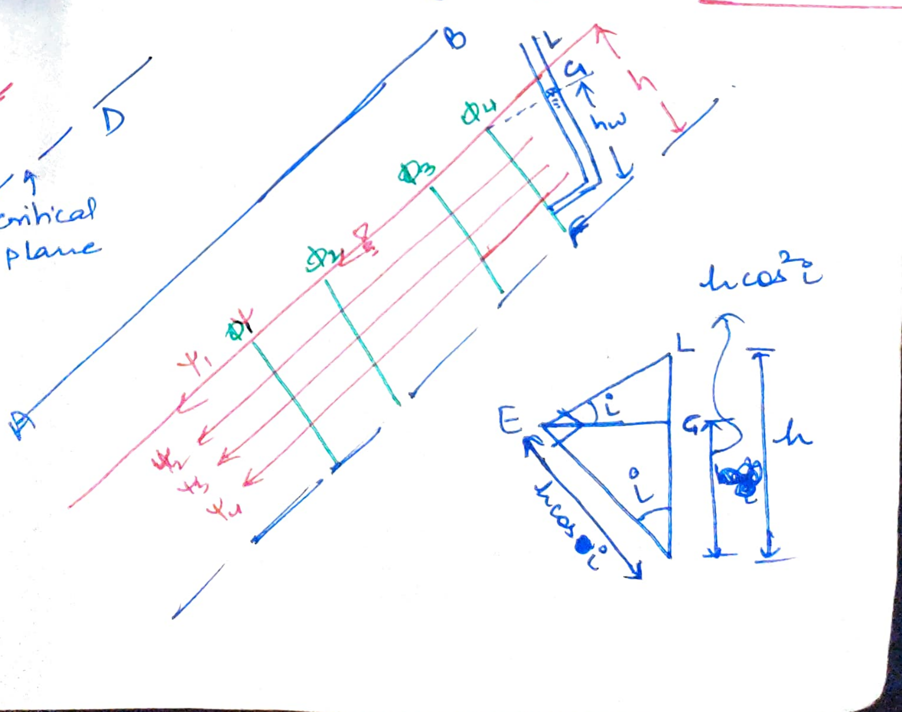
$$F = \frac{\tau_f}{\tau} = \frac{\gamma' z \cos^2 i \tan\phi}{\gamma' z \cos i \sin i} = \boxed{\frac{\tan\phi}{\tan i}}$$

Note :→ FOS of a submerged slope is same as that dry state.

(C) SLOPE SUBJECTED TO STEADY SEEPAGE & WATER TABLE IS AT DEPTH "h" ABOVE CRITICAL SECTION

→ when steady seepage takes place parallel to the slope it in also been subjected to pore water pressure normal to the critical plane CD in upward direction having magnitude $\boxed{u = \gamma_w h \cos^2 i}$



$$W = (\gamma(z-h) + \gamma_{sat} h)\, b \cos i \cdot 1$$

$$\sigma_z = [\gamma(z-h) + \gamma_{sat} h] \cos i \quad \text{or} \quad \gamma_{avg}\, z \cdot \cos i$$

$$\left\{ \gamma_{avg} = \frac{\gamma(z-h) + \gamma_{sat} h}{z} \right\}$$

$$\sigma = \gamma_{avg}\, z \cos^2 i \quad , \quad \sigma' = \sigma - u$$

$$\tau' = \gamma_{avg}\, z \cos i \sin i \qquad \tau_f = c' + \sigma' \tan\phi' \; (c' = 0)$$

$$\Rightarrow \tau_f = \sigma \tan\phi'$$

$$\Rightarrow \tau_f = (\gamma_{avg}\, z \cos^2 i - \gamma_w h \cos^2 i) \tan\phi$$

$$F = \frac{\tau_f}{\tau} = \frac{(\gamma_{avg} \cdot z \cos^2 i - \gamma_w h \cos^2 i) \tan\phi}{\gamma_{avg}\, z \cos i \cdot \sin i}$$

$$\boxed{F = \left(1 - \frac{\gamma_w h}{\gamma_{avg}\, z}\right) \frac{\tan\phi}{\tan i}}$$

Note :→ If seepage is taking place ‖ to the ~~plane~~ slope FOS decreases with increases in water table level ($h$) above the critical plane (CD).

Note :→ Critical Case i.e least factor of safety is achieved when table coincides with ground surface. ($h = z$)

If $h = z$ , $\Rightarrow \gamma_{avg} = \gamma_{sat}$

$$\sigma_z = \gamma_{sat}\, z \cos i \qquad \sigma = \gamma_{sat}\, z \cos^2 i$$

$$\sigma' = \gamma_{sat}\, z \cos^2 i - \gamma_w z \cos^2 i \;\Rightarrow\; \sigma' = \gamma' z \cos^2 i$$

$$\tau' = \gamma_{sat}\, z \cos i \sin i \quad , \quad \tau_f = \sigma' \tan\phi'$$

$$= \gamma' z \cos^2 i \tan\phi$$

$$F = \frac{\tau_f}{\tau} = \frac{\gamma' z \cos^2 i \tan\phi}{\gamma_{sat}\, z \cos i \cdot \sin i}$$

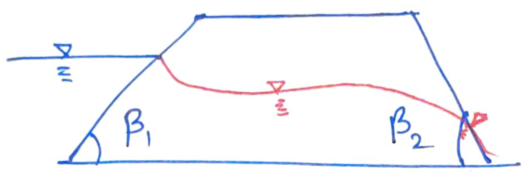
$\boxed{F = \dfrac{\gamma'}{\gamma_{sat}} \cdot \dfrac{\tan\phi}{\tan i}}$ , $\gamma' \approx \dfrac{\gamma_{sat}}{2}$ , $\boxed{F = \dfrac{1}{2}\dfrac{\tan\phi}{\tan i}}$

Note :→ If slope is completely submerged & steady seepage takes place at ground surface then, FOS is reduced to $\frac{1}{2}$ in ~~Comparision~~ comparison to FOS of dry & submerged slope.

→ Hence FOS to be provided for dry & submerged slope must be $\boxed{\geq 2}$

Note :→ If it is mentioned slope is submerged no seepage is considered but if slope is ~~considered~~ saturated ~~the~~ seepage is considered. & water table is assumed to be parallel to ground surface.

Note :→




$\beta_1 > \beta_2$

Q. A granular soil has $\gamma_{sat} = 19\ kN/m^3$, $\phi' = 35°$. A slope has to be made of this material. If FOS = 1.3. Determine the safe angle of slope +

(i) when slope is dry or submerged without seepage.

(ii) If seepage occurs at and parallel to the surface of the slope.

(iii) If seepage occurs ∥ to the slope with the water table at depth of 1.5m,

what is the FOS available on a slip plane parallel to the ground surface at the depth of 4m? Assume $\beta = 28°$

Sol: (i) Dry or submerged slope

$$FOS = \frac{\tan\phi'}{\tan i} = 1.3 = \frac{\tan 35°}{\tan i}$$

$$\tan i = \frac{\tan 35°}{1.3} \Rightarrow \boxed{i = 28.3°}$$

(ii) when the flow occurs at & ∥ to ground surface

$$FOS = \frac{\gamma'}{\gamma_{sat}} \frac{\tan\phi}{\tan i}$$

$$1.3 = \frac{1}{2}\frac{\tan 35^\circ}{\tan i} \Rightarrow \tan i = \frac{\tan 35^\circ}{2\times 1.3}$$

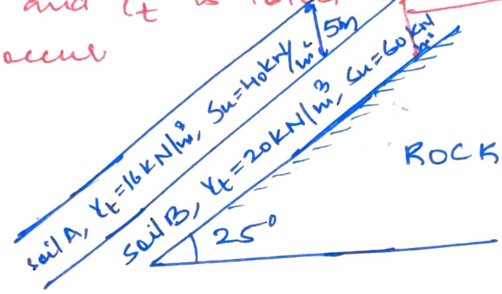
$$\boxed{i = 15.07^\circ}$$

(iii) $FOS = \left(1 - \frac{\gamma_w}{\gamma_{avg}}\frac{h}{z}\right)\frac{\tan\phi}{\tan i} \Rightarrow h = 4 - 1.5 = 2.5\,m$

$\gamma_{avg} = \gamma_{sat}$

$$FOS = \left(1 - \frac{10\times 2.5}{19\quad 4}\right)\frac{\tan 35^\circ}{\tan 28^\circ} \qquad \boxed{FOS = 0.88}$$

Q. The soil profile above the rock surface for a $25^\circ$ infinite slope is as follows. Where $S_u$ is the undrained shear strength and $\gamma_t$ is total unit wt. Find at what depth slip will occur




Sol: case (i) let slip occur in soil A at depth of "x m" from top.

$$FOS = \frac{\tau_f}{\tau} = \frac{c + \sigma\tan\phi}{\tau} = \frac{40}{\gamma_t\, x\cos i\sin i}$$

$$FOS = \frac{40}{x\, 16\sin 25^\circ\cos 25^\circ} = \frac{5}{x\sin 50^\circ}$$

For slip to occur $FOS = 1$

$$FOS = \frac{5}{x\sin 50^\circ} = 1 \Rightarrow x = 6.52\,m$$

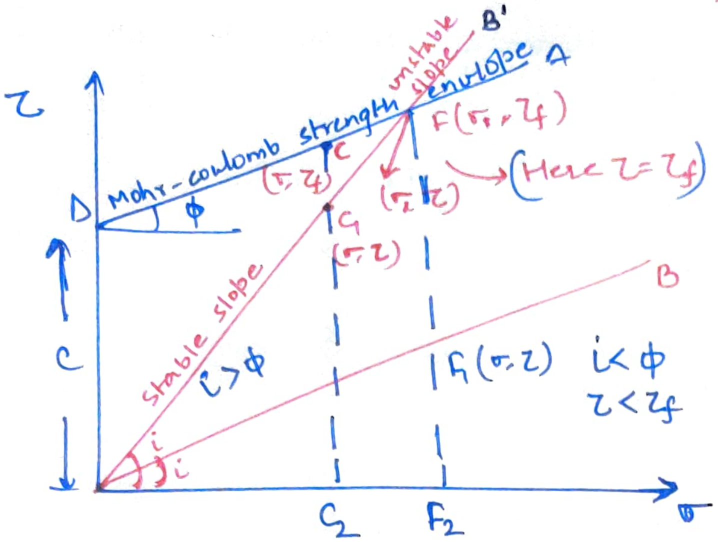
case (ii) Each slip occur in soil B at depth of x m from top

$$FOS = \frac{\tau_f}{\tau} = \frac{\cancel{40}\; 60}{[\gamma_{tA}5 + \gamma_{tB}(x-5)]\cos i\sin i} = 1$$

$$\frac{60}{16\times 5 + 20(x-5)\frac{\sin 50^\circ}{2}} = 1$$

$$\boxed{x = 8.83\ m}$$

Case (iii) : COHESIVE SOIL (c – φ soil)




→ If DA is strength envelope given by —

$$\tau_f = c + \sigma \tan\phi \text{ , then}$$

→ If the slope angle is equal ~~to~~ or less than, represented by line OB, no critical state of stress is reached and slope will be stable

→ If a line OB, at a slope $i > \phi$ is drawn, it cuts the strength envelope at some point F, and a state failure is reached, because shear stress corresponding to the depth represented by "F" equal the shear strength.

→ For any depth 'z' less than that represented by point F, the shear stress "τ" is less than the shear strength $\tau_f$ & slope remains stable.

→ For any depth "z" more than that represented by point F, the shear ~~strength~~ stress "τ" is more the shear strength $\tau_f$ & slope becomes unstable.

$$F = \frac{\tau_f}{\tau} = \frac{c + \sigma \tan\phi}{\tau}$$

Case (i) : DRY / MOIST SLOPE

$$\sigma = \gamma z \cos^2 i \quad , \quad \tau = \gamma z \cos i \sin i$$

$$F = \frac{c + \gamma z \cos^2 i \cdot \tan\phi}{\gamma z \cos i \sin i}$$

$$\boxed{F = \frac{c}{\gamma z \cos i \sin i} + \frac{\tan\phi}{\tan i}}$$

Note :→ for all the conditions to be same FOS for cohesive soil is more than cohesionless soil.

For the critical ~~soil~~ depth, $z = H_c$, $z = z_f$, FOS = 1

$$F = \frac{c}{\gamma H_c \cos i \sin i} + \frac{\tan\phi}{\tan i} = 1$$

$$\Rightarrow \frac{c + \gamma H_c \cos^2 i \tan\phi}{\gamma H_c \cos i \sin i} = 1$$

$$\Rightarrow H_c \gamma (\tan i - \tan\phi) \cos^2 i = c$$

$$\boxed{H_c = \frac{c}{\gamma} \frac{1}{(\tan i - \tan\phi)\cos^2 i}}$$

Note :→ $\frac{c}{\gamma H_c} = (\tan i - \tan\phi)\cos^2 i = S_n$

Here $S_n$ is termed as "TAYLOR'S STABILITY No" that is used analyse the stability of slope.

$$\Rightarrow \boxed{S_n = \frac{c}{\gamma H_c} = \frac{C_m}{\gamma H}}$$

$C_m$ = mobilised cohesion, at depth H.

→ Let "H" be the ~~highest~~ height of the slope at which cohesion mobilised be "$C_m$".

Factor of safety wrt Height $F_H = H_c/H$

" " " " cohesion $F_c = c/C_m$

Here both factor of safety wrt. height "$F_H$" and wrt cohesion "$F_c$" are same

$$S_n = \frac{c}{\gamma H_c} = \frac{C_m}{\gamma H}$$

$$S_n = \frac{c}{\gamma H_c} = \frac{c}{\gamma F_H \cdot H} = \frac{F_c \cdot C_m}{\gamma F_H \cdot H} = \frac{C_m F_c}{\gamma H F_H}$$

$$S_n = \frac{S_n F_c}{F_H} \Rightarrow \boxed{F_c = F_H}$$

Note : → It is based upon the assumption that frictional resistance of the soil is fully mobilized / developed.

Note : → Factor of safety wrt. Friction :

It is the ratio of the available frictional strength to the mobilised frictional resistance.

$$\boxed{F_\phi = \frac{\tan\phi}{\tan\phi_m}}$$

→ However the factor of safety is expressed wrt to shear strength.

$$F = \frac{\tau_f}{\tau} = \frac{c + \sigma\tan\phi}{\tau}$$

→ The shear strength mobilised at every point on slip surface may be give as —

$$\boxed{\tau = \frac{c + \sigma\tan\phi}{F}} \Rightarrow \tau = \frac{c}{F} + \frac{\sigma\tan\phi}{F}$$

$$\tau = C_m + \sigma\tan\phi_m$$

Note : → In reality, the shear strength is not mobilised to the same degree at all points on the slip surface.

→ Hence the previous analysis may be said to represent only the average condⁿ over the slip plane

(ii) SUBMERGED SLOPE :

If slope is submerged, FOS is given by

$$F = \frac{\tau_f}{\tau} = \frac{c + \gamma' z \cos^2 i \tan\phi}{\gamma' z \cos i \sin i}$$

For critical Height $z = H_c$ , $F = 1$

$$1 = \frac{c + \gamma' H_c \cos^2 i \tan\phi}{\gamma' H_c \cos i \sin i}$$

$$H_c = \frac{c}{\gamma'} \frac{1}{(\tan i - \tan\phi)\cos^2 i}$$

(iii) STEADY SEEPAGE ALONG THE SLOPE :

$$F = \frac{\tau_f}{\tau} = \frac{c + (\gamma_{avg} z - \gamma_w h)\cos^2 i \tan\phi}{\gamma_{avg} z \cos i \sin i}$$

$$\gamma_{avg} = \frac{\gamma(z-h) + \gamma_{sat} h}{z}$$

→ If $h = z$ , $\gamma_{avg} = \gamma_{sat}$

$$F = \frac{c + \gamma' z \cos^2 i \tan\phi}{\gamma_{sat} z \cos i \sin i}$$

→ For critical height $z = H_c$ , $F = 1$

$$1 = \frac{c + \gamma' z \cos^2 i \tan\phi}{\gamma_{sat} H_c \cos i \sin i}$$

$$H_c = \frac{c}{(\gamma_{sat} \tan i - \gamma' \tan\phi)\cos^2 i}$$

$$H_c = \frac{c}{\gamma_{sat}\left(\tan i - \frac{\gamma'}{\gamma_{sat}} \tan\phi\right)\cos^2 i}$$

Note : → It can be seen that the effect of shearing resistance & $\phi$ is reduced in this case.

## (ii) STABILITY ANAYSIS OF FINITE SLOPE

→ If the slope is of finite extent bounded by top & bottom surface then it is ~~teermed~~ termed as finite slope.

→ ~~Finite~~ Failure of finite slope takes place due to rotation in most of the cases & the failure plane is either planner, circular or spiral.

→ Finite slope may ~~be~~ have any of the following modes of failure -

(i) SLOPE FAILURE (ii) BASE FAILURE

It is further of two ~~slopes~~ types :—

→ FACE FAILURE

→ TOE FAILURE

(i) Face Failure : → If the failure surface passes through slope above the toe it is termed as face failure.

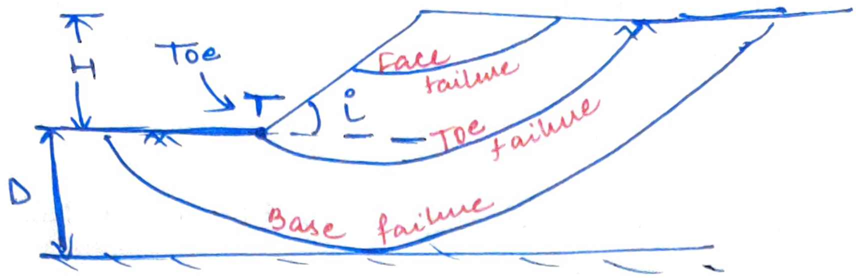
→ The type of failure takes place in the case of steep slope or when the soil mass ~~near~~ the toe is strong & rigid in comparison to soil mass above the toe.

(ii) Toe failure : → It is the most common mode of failure in finite slopes in which failure surface passes through toe.

→ This failure occurs in case of steep slope when soil mass is homogeneous above & below the toe.

Note : → For slope angle $(i) > 53°$, the toe failure occurs, irrespective of any angle of friction $(\phi)$

(iii) Base Failure : → when failure surface takes place in flat slope, when soil mass below the toe is soft weak in comparison to soil mass. above the toe.




Note :→ If "H+D" is to the total depth of failure surface
→ H is the height of slope.
→ Then ratio of total depth of failure surface to the height of slope is termed as Depth factor "$D_f$"

$$D_f = \frac{H+D}{H}$$

∴ For Base failure ($D>0$) , $D_f > 1$

∴ For Toe failure ($D=0$) , $D_f = 1$

→ The stability of finite slope is analysed in 2-stages.

(i) Immediately after construction [short term]

(ii) Long time after construction [long term]

Note :→ For sand, total effective stress analysis (drained) is preferred for both short term & long term stability.

→ For clays, total stress analysis (undrained) is transferred for short term stability & effective stress analysis (drained) is preferred for long term stability.

→ For OC clays effective stress analysis (drained) is preferred for both the stages for short term & long term stability.

→ Methods based on total stress analysis are ——

(i) PLANNER FAILURE SURFACE MTD : CULMANN'S MTD.

(ii) SWEDISH SLIP CIRCLE MTD

(iii) FRICTION CIRCLE MTD

→ Methods based upon effective stress analysis –

(i) TAYLOR'S STABILITY NO MTD.

(ii) BISHOP'S MTD.

→ The rupture of the finite slope may take place along any one the following failure surface.

(i) PLANAR FAILURE SURFACE

(ii) CIRCULAR " "

(iii) NON " " "

→ Planner failure surface may commonly occur in soil deposits or embankment with a specific plane of weakness.

→ Excavation in stratified deposits leads to a planner failure surface along a plane ∥ to the strata.

→ Generally failure surfaces have arcs somewhat flatter at the ends & sharper at the centre.

→ To idealise the problem failure surface may be assumed to be circular i.e homogeneous & isotropic soil.

→ non-circular or composite failure surface may arise in homogeneous dams in following cases –

- Foundation of infinite depth
- Rigid boundary plane of max or zero shear.
- Presence of relatively stronger or weather layer.

→ Similarly in non-homogeneous dam, non circular failure surface is observed due to following.

- Presence of soft ~~dense~~ layer in foundation.
- Use of different type of soil or rock.
- Use of drainage blanket.

(i) CULMANN'S MTD"

→ In this mtd a simple failure mechanism of slope of homogeneous soil ~~which~~ with plane failure surface passing through the toe of the slope is considered.

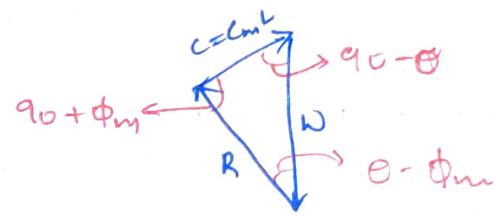
→ Let AB be any probable slip plane.

The wedge ADB is in equilibrium under the action of three forces.

(i) weight of wedge "W" = $\frac{1}{2}\gamma AB\,h \cdot 1$

(ii) The cohesive force "c" along the surface AB, resisting motion $= C_m L$

(iii) The reaction R, inclined at downward angle of $\phi_u$ to the normal of slip plane.


In Δ ADE

$$\frac{H/\sin i}{\sin 90^\circ} = \frac{h}{\sin(i-\theta)} \Rightarrow h = \frac{H\sin(i-\theta)}{\sin i}$$

$$\Rightarrow W = \frac{1}{2}\frac{\gamma L H \sin(i-\theta)}{\sin i} \Rightarrow \tau_f = c + \sigma\tan\phi$$

$$\tau_f = C_m L + W\cos\theta\tan\phi$$

$$\tau = W\sin\theta$$

$$F = \frac{\tau_f}{\tau} = \frac{cL + W\cos\theta\tan\phi}{W\sin\theta}$$

$$F = \frac{cL + \frac{1}{2}\gamma LH\sin(i-\theta)\cos\theta\tan\phi}{\frac{1}{2}\gamma LH\frac{\sin(i-\theta)}{\sin i}\sin\theta}$$

$$F = \frac{c + \frac{1}{2}\gamma H\left(\frac{\sin(i-\theta)}{\sin i}\right)\cos\theta\tan\phi}{\frac{1}{2}\gamma H(\sin(i-\theta)\sin i)\sin i}$$

using sine rule;

$$\frac{C_m L}{\sin(\theta-\phi_m)} = \frac{W}{\sin(90+\phi_m)}$$

$$\frac{C_m L}{\sin(\theta-\phi_m)} = \frac{\frac{1}{2}\gamma LH\sin(i-\theta)}{\sin i\cos\phi_m}$$

$$\frac{C_m}{\gamma H} = \frac{1}{2}\frac{\sin(i-\theta)\sin(\theta-\phi_m)}{\sin i\cos\phi_m}$$

Here $\frac{C_m}{\gamma H} = S_n$ = Taylor's stability number.

For failure to occur, $S_n$ has be maximum for which $\theta = \theta_c$

$$\frac{d(S_n)}{d(\theta)} = 0 \Rightarrow \frac{d}{d\theta}\left[\sin(i-\theta)\sin(\theta-\phi_m)\right] = 0$$

$$\sin(i-\theta)\cos(\theta-\phi_m) + \sin(\theta-\phi_m)\cos(i-\theta)(-1) = 0$$

$$\sin(i-\theta)\cos(\theta-\theta_m) = \sin(\theta-\phi_m)\cos(i-\theta)$$

$$\tan(i-\theta) = \tan(\theta-\phi_m)$$

$$i-\theta = \theta-\phi_m$$

$$\theta_c = \frac{1}{2}(i+\phi_m)$$

$$\left(\frac{C_m}{\gamma H}\right)_{max} = \frac{1}{2} \operatorname{cosec} i \sec \phi_m \left[\sin i - \left(\frac{i+\phi_m}{2}\right)\right] \sin\left[\left(\frac{i+\phi_m}{2}\right) - \phi_m\right]$$

$$\left(\frac{C_m}{\gamma H}\right)_{max} = \frac{1}{2} \operatorname{cosec} i \sec \phi_m \sin\left(\frac{i-\phi_m}{2}\right)^2$$

$$\boxed{H_{safe} = \frac{4 c_m \sin i \cos \phi_m}{\gamma\left[1-\cos(i-\phi_m)\right]}}$$

Here $C_m$, $\phi_m$ are corresponding to FOS 'F'.

Note :→ Culmans mtd is suitable for very steep slopes

## II SWEDISH SLIP CIRCLE MTD

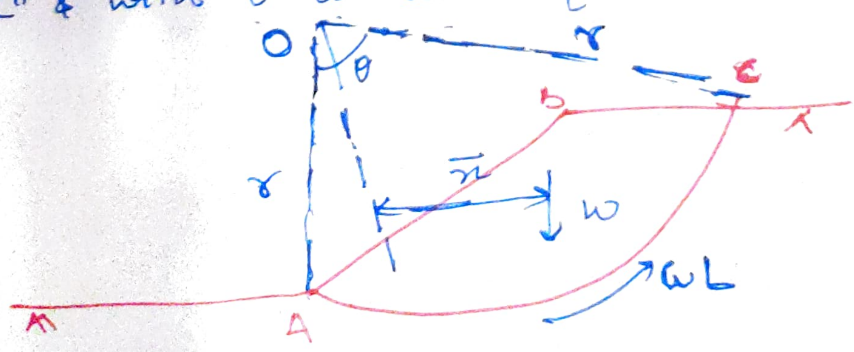
In this mtd surface of sliding is assumed to be an arc of a circle

case (i) PURE COHESIVE SOIL ($\phi_u = 0$)

→ Let slope AB be analysed for its stability by considering a number of trial slip circles & finding the factor of safety for each.

→ The circle corresponding to minimum FoS is the critical slip circle.

→ Let AC be trial slip circle / rotational slide " with radius "r" & with "O" as centre of rotation



$$F = \frac{M_R}{M_D} = \frac{C_u L \cdot r}{W \bar{x}}$$

Here $L$ = length of arc AC $= r\theta$

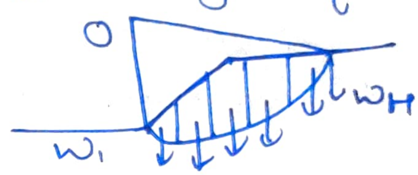
or

$$F_c = \frac{C_u}{C_m}, \quad \text{Here } W\bar{x} = C_m L r, \quad C_m = \frac{W\bar{x}}{Lr}$$

$$\boxed{F_c = \frac{C_u L r}{W \bar{x}} = F}$$

Note : → The distance $\bar{x}$ of centroid of the slip circle from the centre of rotation "O" can be found by dividing the wedge into number of vertical slices & dividing the algebric sum of moment of weight of each slice by total weight of wedge.

$$\bar{x} = \frac{\Sigma W_i \bar{x}_i}{\Sigma W_i} = \frac{\Sigma W_i \bar{x}_i}{W}$$

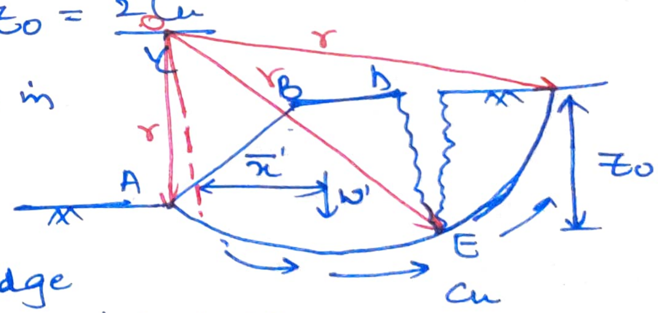



Note : → Effect of tension circles

→ In cohesive soils, tension cracks tends to open up near the top of the slip circle.

→ The max depth of tension crack $z_0 = \frac{2C_u}{\gamma}$

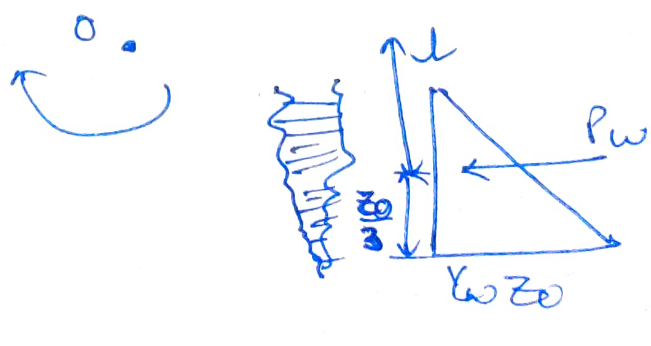
→ The length of slip circle to be taken in the computation of resisting force is only AE.



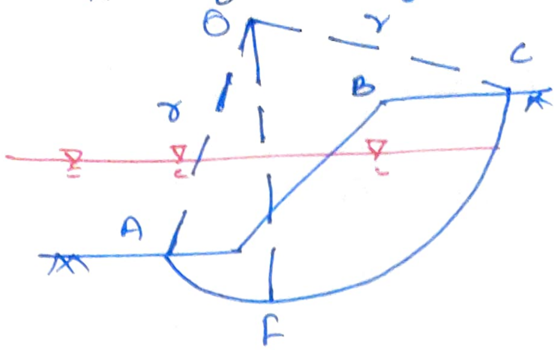

→ And the weight of the sliding wedge is weight of area bounded by ground surface slip circle arc AE & tension crack i.e Area AEDB

→ If the cracks are filled with water an additional driving moment due to horizontal hydrostatic pressure for depth $z_0$ is also considered.

$$F = \frac{M_R}{M_D} = \frac{C_u L_{AE}\, r}{W\bar{x} + P_w l}$$



Note :→ Effect of submergence



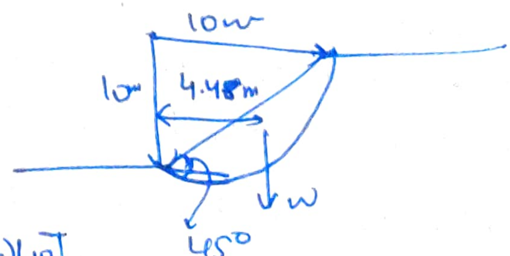

→ Slopes of embankment dams, canals banks etc, may be submerged partly or fully at different times.

→ This ~~different~~ effect of body of water balances act on either side of line of symmetry "OF"

→ The net driving moment can be obtained by using the bulk unit weight soil "γ" above the level of water surface & submerged unit wt "γ'" of the soil below the water surface.

→ If the slope is fully submerged, then "γ'" is used for entire soil section, In addition to this moment due to the water resting on the slope will be an additional resistance moment & increases FOS.

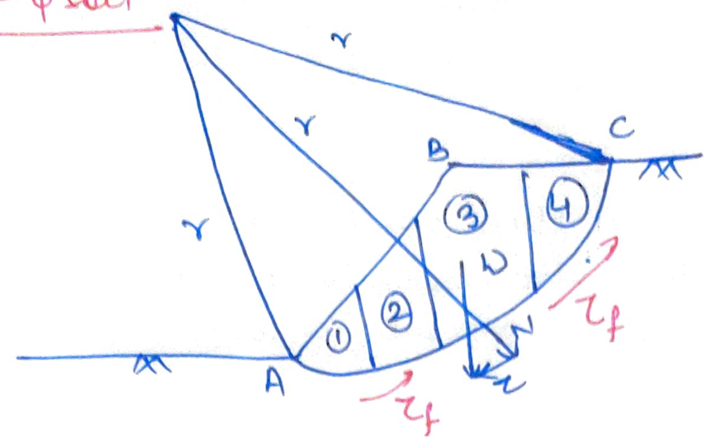
Q. A 10m high slope of dry soil ($\gamma = 20N/m^3$) with soil angle of 45° & circular slip surface, is shown in fig. The wt. of the slip wedge is "W". The undrained unit cohesion $C_u = 60kPa$. Compute FOS against slip failure




$$F = \frac{M_R}{M_D} = \frac{C_u L D}{W\bar{x}} = \frac{60 \times \frac{\pi 10}{2} \times 10 \times 10}{20\left(\frac{\pi(10)^2}{4} - \frac{1}{2}(10)(10)\right) \times 1 \times 4.48}$$

$F = 3.685$

Case (ii) "c – ϕ soil"




→ In c – ϕ let AC be the trial slip surface.

→ For c – ϕ soil, shear strength at different points on the slip surface varies acc. to the value of effective normal stress at these points.

→ The analysis of stability in such case is carried out by method of slices developed by Fellenius termed as swedish slip circle mtd.

→ In this method, the soil mass above the assumed slip circle is ~~obtained~~ ~~assumed~~ divided into no. of vertical slices of equal width (generally 4–6 (12) slices may be considered.

→ The forces btw<sup></sup>n the slices are neglected and each slice is considered to be an independent col<sup></sup>m of soil of unit thickness.

→ weight of each slice is computed & is resolved into normal & tangential components.

→ As normal component passes through the centre of rotation it does not produce any moment but contributes into mobilisation of frictional resistance along the slip surface.

→ To compute the FOS, then total moment of the slices is considered.

Driving force = $\Sigma T$

Resisting forces = $\Sigma c'L + \Sigma N \tan\phi'$

$$F = \frac{M_R}{M_D} = \frac{(\Sigma c'L + \Sigma N \tan\phi)r}{\Sigma T r}$$

$$\boxed{F = \frac{c'L + \Sigma w \cos i \tan\phi}{\Sigma w \sin i}}$$

Note :→ The method of slices can be used for homogeneous or stratified soils & can also can be used where seepage is taking place & pore pressure are present in soil.

# Location of Critical Slip Circle

→ Since the determination of min FOS for a slope is required for its designing, the most critical slip circle is to be located by hit & trial mtd which consumes time [as there are no. of variables involved i.e centre of the circle, radius of circle & distance of intercept in front of toe or δ]

→ Fellinus proposed an empirical procedure to find the centre of this most critical circle for pure cohesive soil ($\phi_u = 0$)

→ The centre of this circle lies on the "XY" in which the point Y has its ~~condition~~ coordinates "H" downwards from toe & 4.5 H horizontally away from toe.

→ The ~~po~~ other point "X" is located with the help of direction angle $\alpha$ & $\beta$

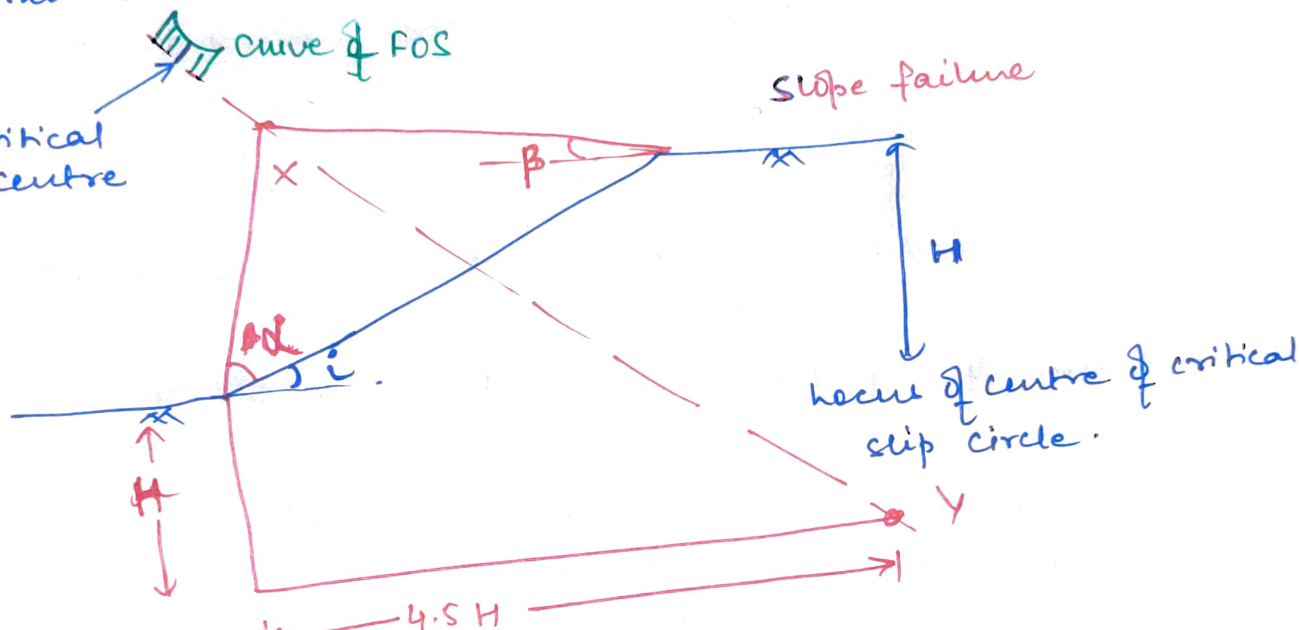
→ when the line XY is obtained, trial centres are obtained on it & FOS corr. to each centre is calculated.

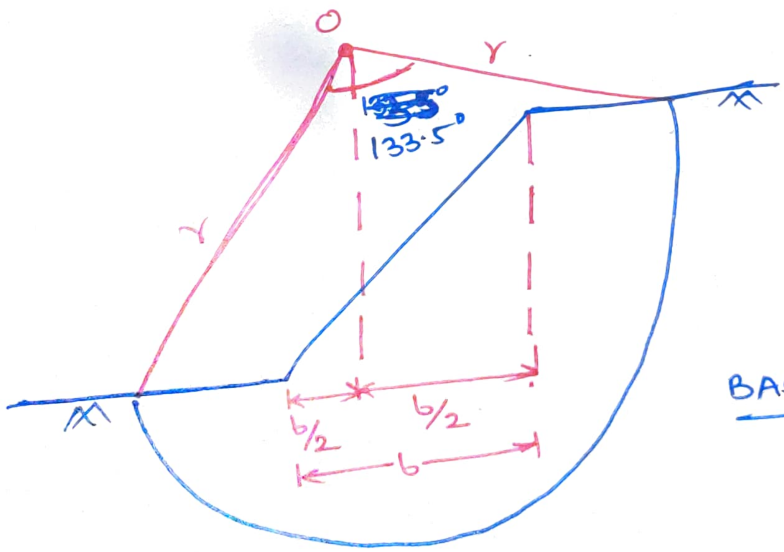
→ These various FOS corresponding to each centre is then analysed to find the centre for min FOS.

Note : → For flat slopes or when soil ~~holds~~ below the toe is softer than the slope material, the critical slip circle will not be a toe circle but will reach much below the toe, resulting in base failure.

→ Fellinius should ~~be~~ that in homogeneous, pure cohesive soil with slope < 53°, the centre of rotation for a deep seated base failure lies on a vertical drawn through the mid point of the slope & the central angle of 133.5°.

→ When a stiffer layer of soil or rock lies beneath the slope, the most critical circle tends to be tangential to this stratum.




BASE FAILURE

# STABILITY OF SLOPES OF EARTH DAMS

→ The stability of slopes of an earth dam is tested under following conditions.

(i) Stability of downstream slope during steady seepage.

(ii) " " upstream slope during sudden drawdown.

(iii) " " upstream & downstream slope during & immediately after construction.

(i) Stability of downstream slope during steady seepage:—

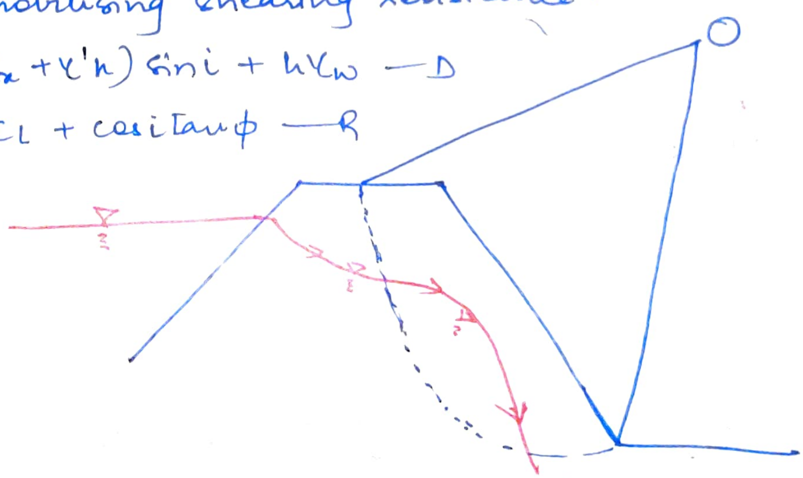
→ Critical condition of d/s slope occurs when the reservoir is full & percolation is at max rate.

→ The direction of seepage force tends to decrease the stability of slope.

→ or the pore water pressure acting on the soil below the saturation line reduces the effective stresses responsible for mobilising shearing resistance.

$(\gamma z + \gamma' h) \sin i + h\gamma_w$ — D

$cL + c \cos i \tan\phi$ — R




$$FOS = \frac{\tau_f}{\tau} = \frac{c'L + \Sigma(N-u)\tan\phi'}{\Sigma T}$$

Note :— Here pore water pressure at any point is determined with help of flow net but in case data of flow net is not available, then FOS is given

$$FOS = \frac{\tau_f}{\tau} = \frac{c'L + \Sigma N'\tan\phi}{\Sigma T}$$

Here normal ~~condition~~ component N are to be calculated on the basis difference of $\gamma_{avg}$ & pore water pressure & T component are to be soil.

$$\gamma_{avg} = \frac{\gamma(z-h) + \gamma_{sat} h}{z}$$

(ii) Sudden drawdown stability.

→ It is assumed that the reservoir has been maintained at high level for a sufficient long-time so that the fill material of the dam is fully saturated & steady seepage is established.

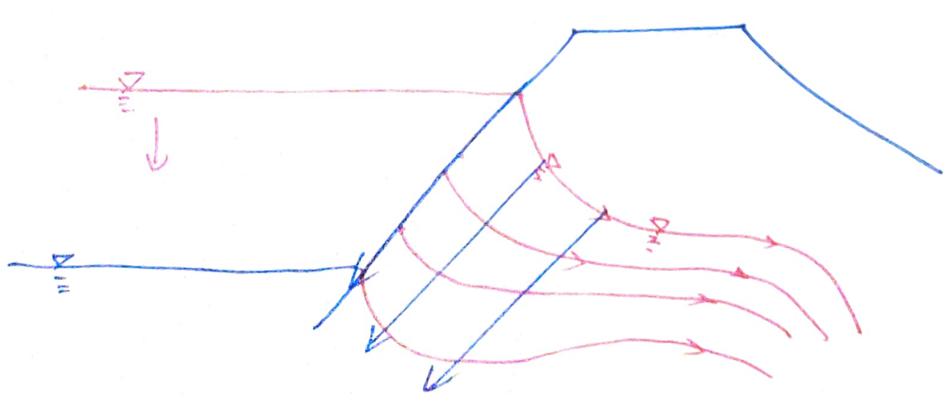
→ If the reservoir is drawdown at this stage the direction of flow is reversed, causing instability in the upstream slope of the dam.

→ The instantaneous drawdown pratically does not take place & pore & pore pressure along the sliding surface are determined with the help of flow net.

→ The most critical condition of sudden drawdown means that while the water pressure acting on the u/s slope at full flow condition is removed, there is no change in the water content of soil in embankment.

→ The saturated slope wt. produces the shearing resistance is reduced due to development of pore water pressure, which do not dissipate quickly.

→ FoS is calculated same as previous case.



(iii) Stability during & at the end of construction

→ An embankment dam is normally compacted at 80–90%. saturation ie 80–90% of voids are filled by water & rest by air.

→ The compression of this water air-pore fluid under increasing load of embankment causes build up of pore pressure in fine grained soils, magnitude of which can be computed empirically as —

$$u = \frac{p_o \Delta}{V_a + HV_w - \Delta}$$

$p_a$ = pressure of pore air before consolidation

$\Delta$ = embankment compression fraction

$V_a$ = vol. of free air in soil pore pressure in unit vol. of embankment soil before consolidation.

$V_w$ = " " water present in unit vol. of embankment.

$HV_w$ = vol. of dissolved air in vol. $V_w$ of water.

$H$ = Henry's const (0.02)

For no drainage $\Delta = V_a$ ⇒ $u_{sat} = \dfrac{p_o V_a}{V_a + HV_w - V_a}$

$$\Rightarrow u_{sat} = \frac{p_o V_a}{H V_w}$$

Note: → FOS for

| | |
|---|---|
| End of construction | 1.3 |
| Steady seepage condition | 1.25 |
| Sudden draw down cond | 1.2 |

Q: In order to find the FOS of all slope of an earth dam during steady seepage the section of dam was drawn to scale of 1cm = 4cm & the following results obtained on critical slip circle.

Area of N rectangle = $14.4\ cm^2$

" " T " = $6.4\ cm^2$

" " U " = $6.9\ cm^2$

Length of arc = 12.6 cm

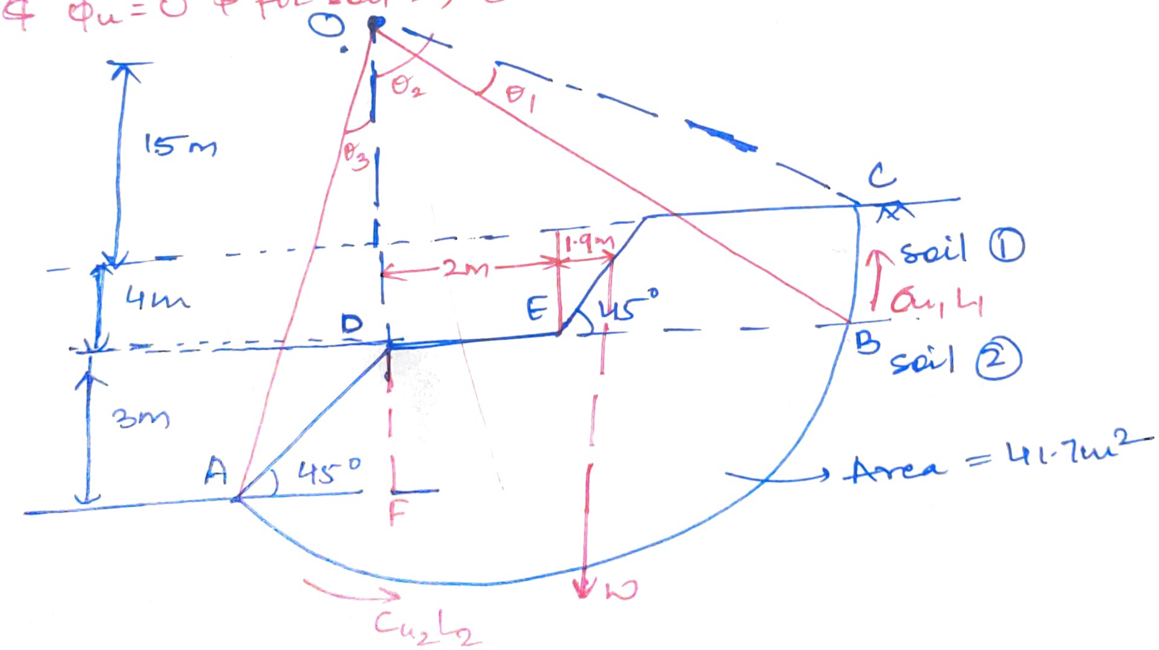
Lab test indicates 26° for effective angle of shear resistance & 19.5 kN/m² for cohesion. Determine the FOS of the slope $\gamma = 19$ kN/m³

sol:
$$F = \frac{\tau_f}{\tau} = \frac{c'L + \Sigma(N-u)\tan\phi'}{\Sigma T}$$

$$F = \frac{19.5 \times 12.6 \times 4 + 19(14.4 - 6.9)\times 4^2 \tan 26^\circ}{19 \times 6.4 \times 4^2}$$

$$\boxed{F = 1.076}$$

Q. For the given failure surface, determine the factor of safety is terms of total stress for the slope as shown. $\gamma$ of both sides ① & ② 19 kN/m³. For soil, the shear parameters are $C_u = 20$ kN/m² & $\phi_u = 0$ & for soil 2, $C_u = 26$ kN/m² & $\phi_u = 0$




$$F = \frac{M_R}{M_D}, \quad M_D = W\bar{x}$$

$\bar{x} = 3.9$ m

$W = \gamma \cdot A \cdot 1 = 19 \times 41.7 \times 1 = 792.3$ kN/m

$M_D = 792.3 \times 3.9 = 3089.97$ kN-m/m

$M_R = C_{u2} L_{AB} R + C_{u1} L_{BC} R$

$R = \sqrt{8.5^2 + 3^2} = 9.01$ m

$\theta_3 = \tan^{-1}\left(\frac{3}{8.5}\right) = 19.44^\circ \qquad \theta_2 = \cos^{-1}\left(\frac{1.5}{9.01}\right) = 80.41^\circ$

$$\theta_2 - \theta_1 = \cos^{-1}\left(\frac{5.5}{9.01}\right) = 52.38^\circ$$

$$\theta_1 = 80.41^\circ - 52.30^\circ = 28.03^\circ$$

$$L_{CB} = R\theta_1 = 9.01 \times 28.03 \times \frac{\pi}{180} = 4.4\,m$$

$$L_{AB} = R(\theta_2 + \theta_3 - \theta_1) = 9.01(19.44^\circ + 52.38^\circ)\frac{\pi}{180} = 11.3\,m$$

$$M_R = (25 \times 4.4 + 20 \times 11.3)\,9.01$$

$$= 3339.6 \text{ KN-m/m}$$

$$F = \frac{3339.6}{3089.97} = 1.08$$

## (iii) FRICTION CIRCLE MTD

→ This method is based on total stress analysis, but it enables the angle of shearing resistance to be taken into account.

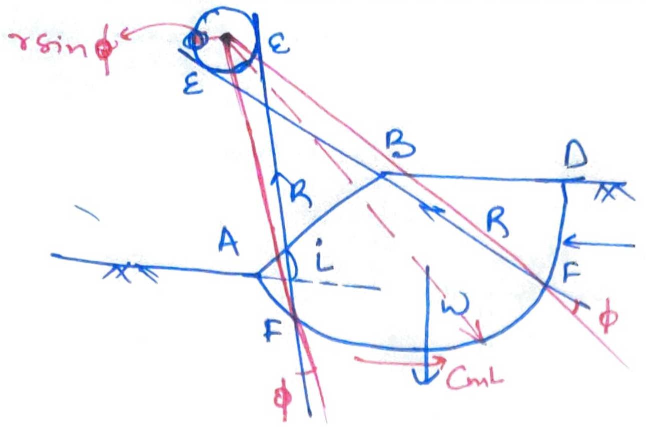
→ It should be noted that some soils, such as saturated silts & unsaturated clays, do exhibite a $\phi$ value under undrained condition.

→ The friction circle method also assumes the failure surface as the arc of the circle.

→ If a small concentric circle is drawn with "O" as centre & $r\sin\phi$ as the radius, any line "EF" tangential to this smaller circle will cut the failure arc "AD" at an obliquity angle of $\phi$.

→ conversely, any line (vector) reaction R at and obliquity angle of $\phi$ ~~too~~ at an ~~angle~~ element of the failure arc
" " will be tangential to the small circle.

→ The small circle of radius "$r\sin\phi$" is thus called "FRICTION CIRCLE" or "$\phi$-CIRCLE"




- $C_m$ = mobilized unit cohesion assumed along the arc

∴ Mobilized cohesion on elementary arc of length

$\Delta L = C_m \Delta L$

∴ Total cohesive resistance –

$$\boxed{C_m \bar{L} = C_m \Sigma \Delta L}$$

→ If the total cohesive resistance $C_m\bar{L}$ is assumed to consist of elementary resistance $C_m \Delta L$, the arc "AD" divided into no. elementary arc of length $\Delta L$, represent a force polygon.

→ As the chord AD, representing the closing side of polygon, represente the magnitude as well as direction of the resultant of all the elementary cohesive force

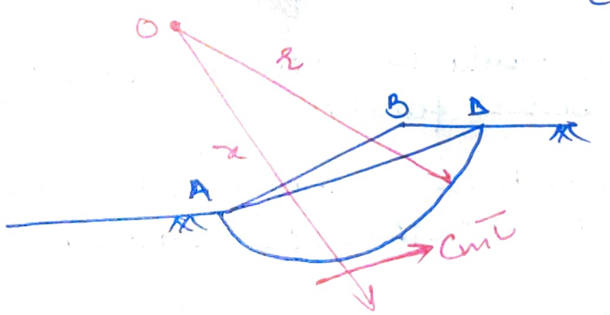
$\bar{L}$ = length of chord AD

Total cohesive force represented by AD = $C_m\bar{L}$

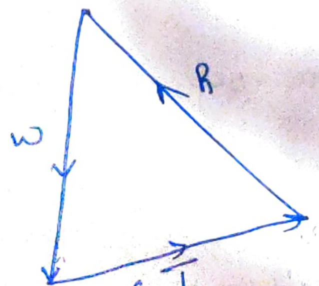
→ The position of this resultant cohesive force $C_m\bar{L}$, acting ∥ to the chord AD is such that its moment about the centre of rotation is equal to sum of moment of elementary cohesive forces.

→ Thus if "x" is the perpendicular distance of direction of $C_m\bar{L}$ from O, then.

$$(C_m\bar{L})x = (C_m \Sigma \Delta L)r \Rightarrow x = \frac{rL}{\bar{L}}$$




→ Thus the direction & location of resultant cohesive force $C_m\bar{L}$ is known



→ In previous force Δ, all the angle are known along with magnitude of W.

→ Hence using sine law R & $C_m \bar{L}$ can be calculated from which $C_m$ can be found.

→ The factory of safety wrt cohesion is given by —

$$F_c = \frac{C}{C_m}$$

→ A no. of slip circles are taken & FOS for each is found. The circle giving minimum FOS is ~~found~~ the critical slip circle.

## (iv) TAYLOR'S STABILITY NUMBER

As gravity force (due to γ) are cause of instability, while the cohesive force ($C_u$) contribute to the stability in the soil mass.

→ These two forces are uniformly distributed throughout the soil mass & is valid for every point within the sliding mass.

→ The max height $H_c$ of the slope that can be built without failure is thus proportional to following —

$$H_c = \frac{C_u}{\gamma} f(\phi_u, i)$$

where $f(\phi_u, i)$, means a function of both $\phi_u$ & $i$.

→ The above relationship would be dimensionally correct if $f(\phi_u, i)$ is dimensionless function.

→ The function is expressed as a reciprocal of dimensionless number by Taylor termed as "STABILITY NUMBER" ($S_n$)

$$H_c = \frac{C_u}{\gamma S_n} \Rightarrow \boxed{S_n = \frac{C_u}{\gamma H_c}}$$

→ If $F_c$ & $F_h$ are FOS with respect of cohesion & height resp. then —

$$S_n = \frac{F_c\, C_m}{\gamma H_c} = \frac{C_u}{\gamma F_h \cdot H} \Rightarrow \boxed{S_n = \frac{C_m}{\gamma H}}$$

→ Here it is assumed that friction is full mobilised, while computing $F_c$ & $F_H$.

→ Taylor utilised the friction circle analysis along with an analytical procedure to determine the values of $S_n$ as a function of $\phi_u$ & $i$ & represented in form of table & curve.

→ These solutions are strictly valid only for simple, homogeneous finite slopes & for cases involving no seepage, but they can be used for approximate & preliminary solution.

→ Theoritical max value of $S_n = 0.5$ but practically it comes out to be 0.261.

→ The stability chart or table provided by Taylor can be used for the following situations —

(i) For slope of height "H" at any angle "i" in a soil for which $\gamma$, c & $\phi$ are known, to find FOS in this case.

(ii) For a required FOS of a slope in a soil whose characteristics are known, to find the steepest angle at which a slope of a certain height can be allowed or what is max height upto which a slope of certain angle can be allowed.

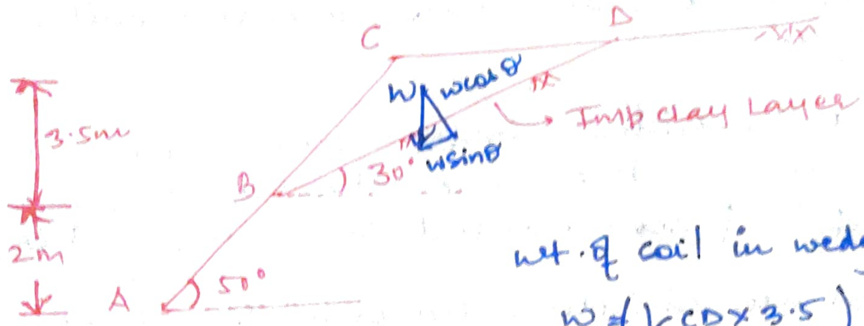
Note :- → In case of sudden drawdown the angle of $\phi$ is empirically reduced to $\phi_w$ ie weighted friction angle given by —

$$\boxed{\phi_w = \frac{\gamma'}{\gamma_{sat}} \phi_u}$$

→ $\phi_w$ is used in the stability chart to obtain stability number "$S_n$", however the saturated unit wt "$\gamma_{sat}$" is in expression for $S_n$.

(The above recommendation are empirical & approximate, however error involved is insignificant)

Q A soil mass "BLD" having $c = 8\ kN/m^2$ $\phi = 20°$, & $\gamma = 19\ kN/m^3$ is resting on an inclined impermeable clay layer. Determine the FOS against wedge failure along the interface "BD".




wt. of soil in wedge BCD —

$W = \left(\frac{1}{2} CD \times 3.5\right) \times 1 \cdot \gamma$

$CD = \frac{3.5}{\tan 30^\circ} - \frac{3.5}{\tan 50^\circ} = 3.12\,m$

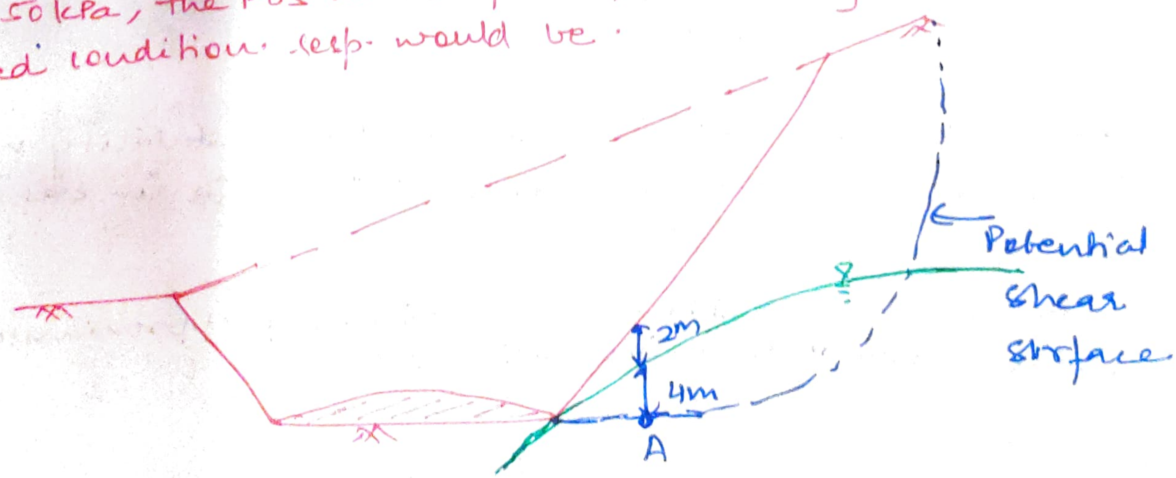
$BD = L = \frac{3.5}{\sin 30^\circ} = 7m$

$W = \frac{1}{2} \times 3.12 \times 3.5 \times 19 = 103.74\ kN/m$

$F = \frac{\tau_f}{\tau} = \frac{CL + N\tan\phi}{T} = \frac{CL + W\cos 30^\circ \tan\phi}{W\sin 30^\circ}$

$F = \frac{8 \times 7 + 103.74 \cos 30^\circ \tan 30^\circ}{103.74 \sin 30^\circ} = 1.7\%$

Q. For the constitution of a highway, a cut is to be made as shown.
The soil exhibits $c' = 20kPa$, $\phi' = 18^\circ$ & undrained shear strength $= 80kPa$, $\gamma_w = 9.81\ kN/m^3$. The unit wt. of soil above & below ground water table are 18 & $20 kN/m^3$. If the shear stress at pt A = 50kPa, the FOS at this point, considering the undrained & drained condition resp. would be.




**Sol:** (i) For undrained condition

$F = \frac{\tau_f}{\tau} = \frac{80}{50} = 1.6$

(ii) For ~~lateral~~ drained condition

$$F = \frac{\tau_f}{\tau} = \frac{c' + \sigma' \tan\phi'}{\tau}$$

$$= \frac{20 + [18\times2 + 4(20-9.81)]\tan 18^\circ}{50}.$$

$$\boxed{F = 0.9}$$

Q. An infinitely long slope is made up of c-φ soil having c = 20kPa & γ = 16 kN/m³. The angle of inclination of ht of slope 40° & 5m resp. To maintain the limiting equilibrium the angle of internal friction of soil is "?"

Sol. For limiting equilibrium FOS = 1

$$F = \frac{\tau_f}{\tau} = \frac{c + \gamma H \cos^2 i \tan\phi}{\gamma H \cos i \sin i}$$

$$H = \frac{c}{\gamma(\tan i - \tan\phi)\cos^2 i} \rightarrow 5 = \frac{20}{16(\tan 40^\circ - \tan\phi)\cos^2 40^\circ}$$

$$\boxed{\phi = 22.44^\circ}$$

Q. A canal having side slopes 1:1 is proposed to be constructed in a cohesive soil to a depth of 10m below the ground surface. The soil properties are —

$\phi_u = 15^\circ$, $C_u = 15$kPa, $e = 1.0$, $G = 2.65$

(i) If $S_n = 0.08$ & if the canal flows full, the FOS with respect to cohesion against failure of canal bank would be?

(ii) If there is sudden drawdown of water in the canal & if $S_n$ for the reduced value of $\phi_w$ is 0.126, the FOS wrt. to cohesion against the failure of bank would be?

Sol. (i) When canal runs full, slope is submerged

$$\gamma' = \frac{(G-1)\gamma_w}{1+e} = 8.09\ \text{kN/m}^3$$

$$S_n = \frac{C_u}{\gamma' H_c} = \frac{C_u}{\gamma' F_{c} \cdot H} = \frac{C_u}{\gamma' F_c H} \quad (F_c = F_H)$$

$$F_c = \frac{C_u}{H \gamma' S_n} = \frac{12}{10 \times 8.09 \times 0.08} = 1.85$$

(ii) In case of sudden drawdown, slope is $sat^n$

$$\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e} = 17.9 \, kN/m^3$$

$$S_n = \frac{C_u}{\gamma_{sat} H_c} = \frac{C_u}{\gamma F_c H} \Rightarrow F_c = \frac{C_u}{S_n \gamma_{sat} H} = \frac{12}{0.126 \times 17.9 \times 10}$$

$$\boxed{F_c = 0.53}$$

# Foundation Engineering

→ Structure foundations are the substructure elements which transmit the structural load to the earth in such a way that ~~ste~~ the supporting soil is not overst-ressed & not undergo deformation that would cause excessive settlement of the structure.

→ Here the ~~structure~~ properties of the supporting soil must be expected to affect the ~~circle~~ of choice of type of structural foundation suitable for the structure.

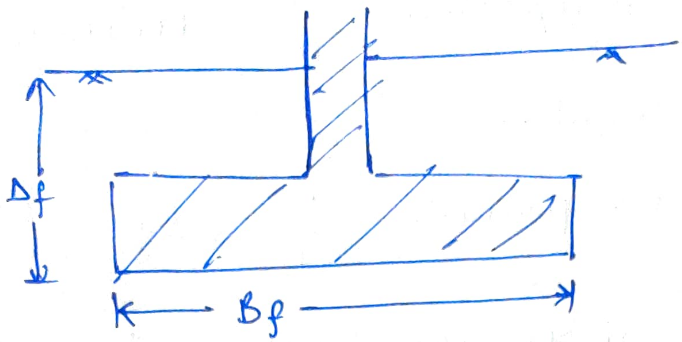
→ The failure of foundation may be due to

(i) settlement of soil / foundation (It is called as settlement failure)

(ii) sliding / slipping of soil foundation (It is called as shear failure)

→ The foundation should be safe in shear criteria as well as settlement criteria.

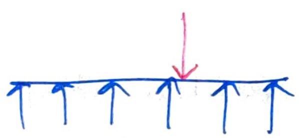
# TYPES OF FOUNDATIONS




**Shallow foundation**

(a) Terzaghi $\frac{D_f}{B} \leq 1$

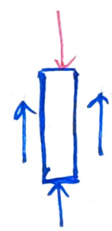
(b) carries the load due to Base resistance.



**Deep foundation**

(a) Terzaghi $\frac{D_f}{B} > 1$

(b) carries the load due to Base & side resistance.



**Shallow Foundation**

- Raft / Mat
- Footing
  - Isolated
    - sq
    - Circular
    - Rectⁿ
  - Combined
    - Rectangular
    - Trapezoidal
  - Strip
  - strap
  - spread

## Deep foundation

- PILE
- Well/CAISSON
- FOOTING

Note :→ As per Skempton

∴ For shallow foundation $\frac{D_f}{B} \leq 2.5$

→ The various type of ~~standard~~ structural foundation can be ~~bounding~~ broadly grouped into two:—

(a) Shallow foundation

→ A shallow foundation transmit structural load to the soil strata at a relatively small depth by the action of end Bearing.

→ As per Terzaghi if $\frac{D_f}{B} \leq 1$ it is termed as shallow found$^n$

Note 3 → $\frac{D_f}{B} = 1-15$ it is termed as moderated deep foundation.

→ It is further classified into Raft/Mat & footing.

(b) Deep foundation

→ If $\frac{D_f}{B} > 1$ it is termed as deep foundation.

→ If $\frac{D_f}{B} > 15$ it is termed as very deep foundation.

→ In deep foundation load is supported partly by frictional resistance around the surface and rest by bearing at the base of foundation.

→ While constructing shallow foundation in open excavation the disturbance soil is minimal however in case of deep foundation disturbance of soil extends to a larger zone along the length of deep foundation.

Note → For reasons of economy, shallow foundation are always preffered over deep foundation.

# Guidelines for Selective of Foundation

| Type of soil & loading condition | Suitable foundation |
| --- | --- |
| → If structural load is less & soil is medium to dense. | shallow foundation |
| → If structural load is heavy & soil is loose/weak. | Raft/deep/ foundation |
| → If swelling pressure is high & differential free ~~swall~~ value is more than 35%. | Raft foundation / Deep foundation |
| → If footing area is more than 40% of plinth area. | Raft foundation/ Combine footing. |
| → If structural load is heavy and foundation is to be placed in running water (river, sea, canal). | Compaction pile. |
| → If soil is loose saturated sand & is prone to liquifaction. | Compaction pile. |
| → If structural load is less (2-3 storey building). | Isolated footing. |
| → If soil is expensive & high swelling & shrinkage characteristics is found. | Floating or balancing foundation/ under-reamed pile. |

# General Requirements of foundations

→ For a satisfactory performance, a foundation must satisfy the following three conditions —

(i) Location & Depth criterion

(ii) Shear failure criterion (Bearing capacity criterion)

(iii) Settlement criterion

→ A foundation must be properly located and founded at such a depth that its performance is not affected by factors such as lateral expulsion of soil beneath the foundation, seasonal volm changes caused by freezing thawing & presence of moisture & adjoining structure.

→ A foundation must be safe against shear failure or soil rupture.

→ The settlement of the foundation, especially differential settlement, must be within the permissible limit.

→ Excessive settlement may effect the utility of the structure & may also cause its damage & decreases its asthetic value.

Note: → The three requirement are independent of each other and must be satisfied seperately.

(i) Location & Depth Criterion of foundation

→ As a general rule, any foundation should be placed at a depth where soil strata is adequate from the ~~soil~~ point view of ~~history~~ bearing capacity of settlement criterion.

→ However it must be placed at a minimum depth of 5m below natural ground surface.

→ further foundation must be placed below the zone of volume changes, where volume change is expected.

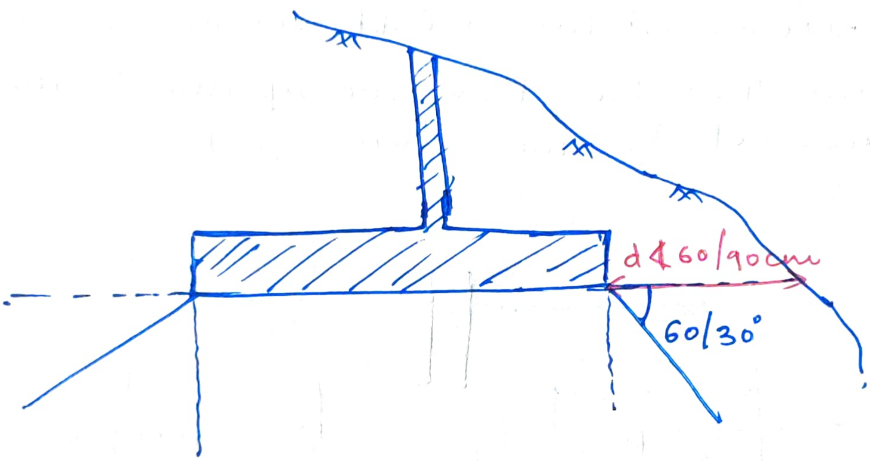
→ In ~~expression~~ expansive soil depth of foundation must be placed below the depth frost zone.

→ The zone of seasonal variation in water content varies in thickness from 1.5 – 3.5m (for black cotton soil)

→ Foundations for structure in river have to be ~~placed~~ protected from ~~from~~ the scouring action of water. The depth of foundation in such case must be below the deepest ~~of foundation in such case in~~ scour level.

→ When footing are adjacent to sloping ~~level~~ ground or where the base of footing are near by at different levels or at level different from these of footing of adjoining structure.
Following recommendations are adopted :—

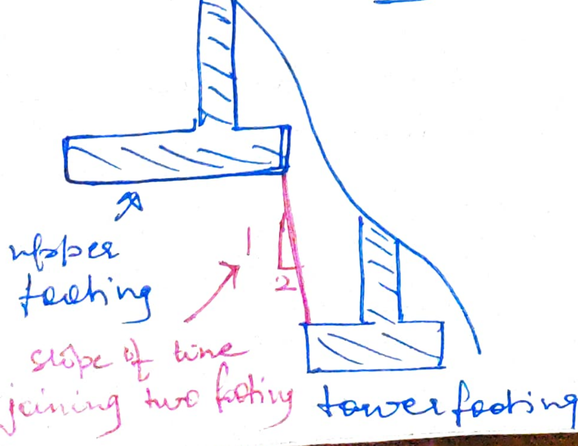
(a) When the ground surface slopes downward adjacent to a footing, the sloping surface should not encroach upon a formation of bearing material under the footing having sides which make an angle of 60° with horizontal for rock & 30° for soil & the horizontal distance from the lower edge of the footing to the sloping surface shall be of atleast 60 cm for rock & 90 cm for soil.



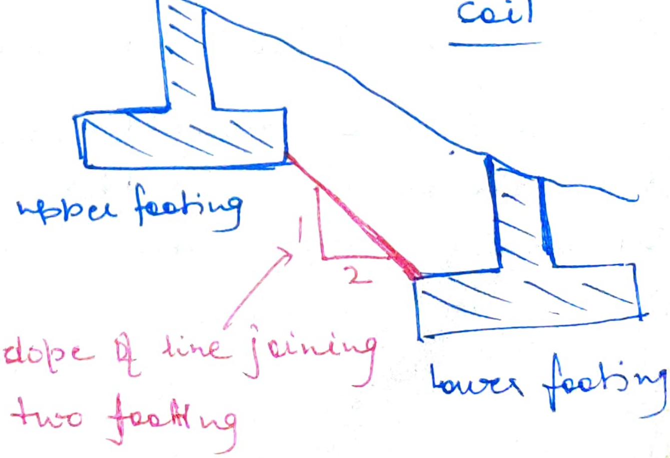

(b) for footing in granular soils, the line joining the lower adjacent edges of the adjacent footing should not have a slope steeper than 2H : 1V.

(c) In clayey soil, slope of line joining the lower ~~sur~~ adjacent edge of the upper footing & the upper adjacent edge of lower footing should not be steeper than 2H : 1V.

(a) Granular soil




(b) Clayey soil



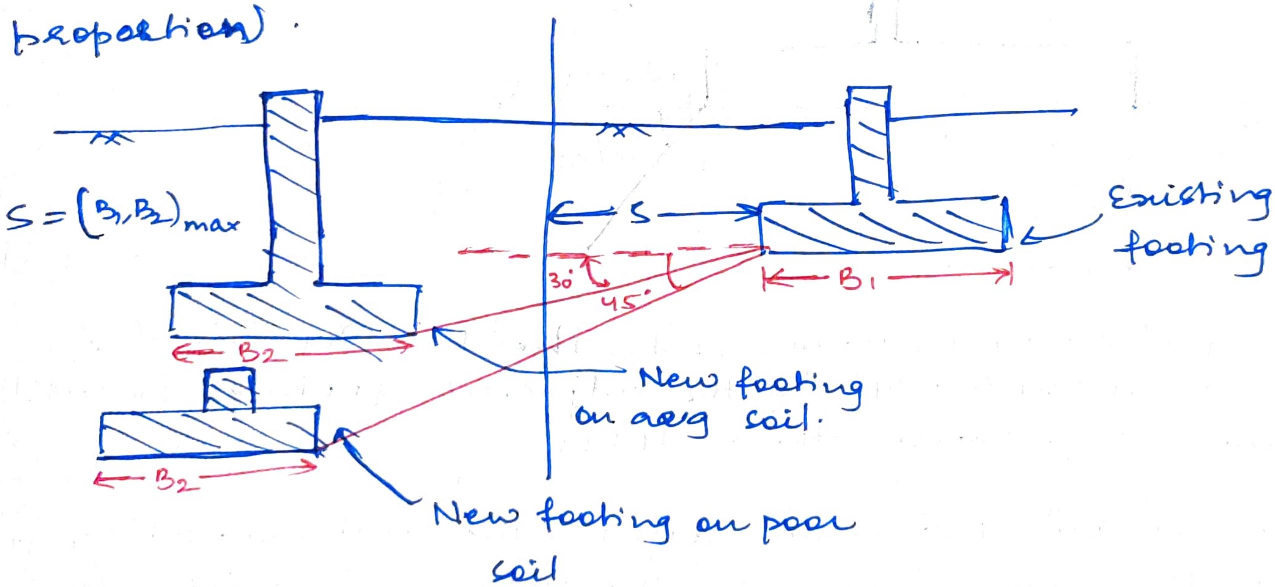
(d) To avoid damage to an existing structure, the foundation for a new structure at an adjacent site should be located suitable distance such that

→ The adjacent edge of the new footing must be at least at a distance "s" from the edge of the existing footing where "s" is the width of larger footing.

(e) The line from the edge of new footing to the edge of 45° or less with the horizontal plane, that is, the distance "s" should be greater than the difference in elevation btw^n the adjacent footing.

→ If even position of new & existing footing were to be interchanged, the recommendation would remain same.

(This provision ensure that the stress overlap due to the adjacent footing does not assume any significant proportion).




(f) When a new footing is placed lower than an old footing the existing structure may be endangered because of lateral flow of soil from beneath the existing footing.

→ The excavation must ~~be~~ not, therefore be too close to the ~~close~~ existing footing & if it is done proper provision of bracing must be provided.

# Bearing Capacity of Shallow foundation

→ Terminology used for analysis of bearing capacity.

(i) GROSS PRESSURE/GROSS LOADING INTENSITY ($q_g$)

→ For a footing constructed with its base at the depth $D_f$ below the ground surface, the total pressure at the base of footing due to wt. of the superstructure, wt. of footing & wt. of soil fill over the footing, it is termed as gross pressure/loading intensity.

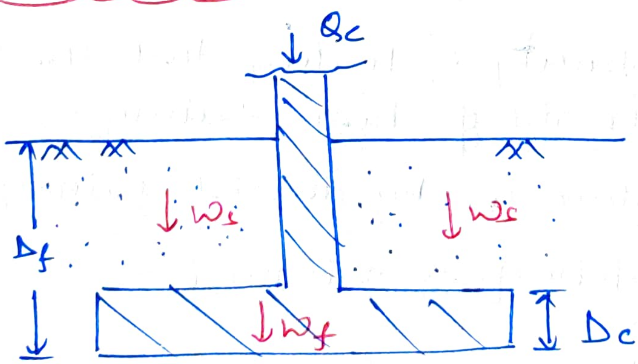
(ii) NET PRESSURE/NET LOADING INTENSITY ($q_n$)

→ The deformation of soil below the base of the footing is caused only by the pressure over & above that which existed before the construction of the footing & the super structure.

→ The difference btw^n the gross pressure & the over burden pressure at the base of footing is termed as net pressure or net loading intensity.

$$q_n = q_g - \gamma D_f$$

or $q_n = q_g - \gamma' D_f$ (If water table effect is also considered)




$$q_g = \frac{Q_c + W_f + W_s}{A_f}$$

(iii) ULTIMATE BEARING CAPACITY ($q_u$)

→ If the load at base of footing is gradually increased, a stage will be reached when the load will cause the shear failure of the supporting soil.

→ The max^m gross pressure intensity of the loading that the soil can support before it fails in shear or min gross pressure intensity of the loading at which soil fails in shear is termed ultimate bearing capacity.

→ The maximum gross pressure intensity of the loading that the soil can support before it fails in shear or min gross pressure intensity of the loading at which soil fails in shear is termed ultimate bearing capacity.

→ ultimate bearing capacity of the is not only related to the properties of soil but also to the characteristics of footing such as its size, shape, depth, mode of loading & whether loading is applied axially or eccentricity.

## (iv) NET ULTIMATE BEARING CAPACITY ($q_{nu}$)

The maximum net intensity of loading at the base of the foundation that the soil can support before failure in shear.

→ It is difference of ultimate bearing capacity & overburden pressure

$$q_{nu} = q_u - \gamma D_f$$

or

$$q_{nu} = q_u - \gamma' D_f$$ (If water table effect is also considered)

## (v) NET SAFE BEARING CAPACITY ($q_{ns}$)

→ It is the max net intensity of loading that the soil can safely support without the risk of shear failure.

→ It is obtained by dividing $q_{nu}$ by desired factor of safety F.

→ Usually, a factor of safety of 2.5–3 is used —

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

## GROSS SAFE BEARING CAPACITY ($q_s$)

→ It is the max$^m$ gross intensity of loading that the soil can safely support without failing in shear.

$$q_s = q_{ns} + \gamma D_f$$

$$\Rightarrow q_s = \frac{q_{nu}}{F} + \gamma D_f \quad \Rightarrow \boxed{q_s = \frac{q_u - \gamma D_f}{F} + \gamma D_f}$$

Note :→ Since the additional the due to self wt. of soil ($\gamma D_f$) is available in full & is present from past history, it seems logical & rational not to use factor of safety for this term.

→ It is preferred to use not only net safe bearing capacity in calculation instead of gross safe bearing capacity.

## (vii) NET SAFE BEARING PRESSURE ($q_{ns}$)

→ It is maximum net intensity of loading that can be allowed on the soil without the settlement exceeding the permissible value.

→ No factor of safety is to applied in this as it is already included in permissible value of settlement.

## (viii) ALLOWABLE BEARING PRESSURE ($q_{a-net}$)

→ It is the max$^m$ net intensity of loading that can be imposed on soil with no possibility of shear failure or settlement failure.

→ Hence it is the smaller of net safe bearing capacity (shear failure criterion) & net safe bearing pressure (settlement criterion)

$$q_{anet} = (q_{nsc}, q_{nsp})_{min}$$

Note :→ It is often confusing to come across varied terminology in different literature & even in code of practice.

→ But the terminology used here is logical & rational.

→ In IS 6403 allowable bearing pressure is named as allowable bearing capacity but bearing capacity is associated with only shear failure criterion.

# Choice of Net Allowable Bearing Pressure

→ For a safe design of a footing, the loading intensity at the base of the foundation should be less than allowable bearing pressure.

→ It can be net or gross allowable pressure.

→ It is conventional & also convinient to use net allowable bearing pressure in the proportioning of footing, due to following reason —

(i) It is logical to compare the net allowable bearing pressure with net loading intensity (i.e the loading intensity at the base of footing in excess of loading intensity to which it was orginally subjected).

→ Where the excavated soil in the trench is backfilled after construction of footing & column the net loading intensity is the gross loading intensity minus loading intensity due to backfill soil.

→ If there is no backfilling of coil after foundation of structure is incorporated in basement floor like in case raft/matt the coil is excavated upto the level of base of raft, Here the gross loading intensity is only due structural load & self wt. of raft & does not include the wt. of soil above the foundation.

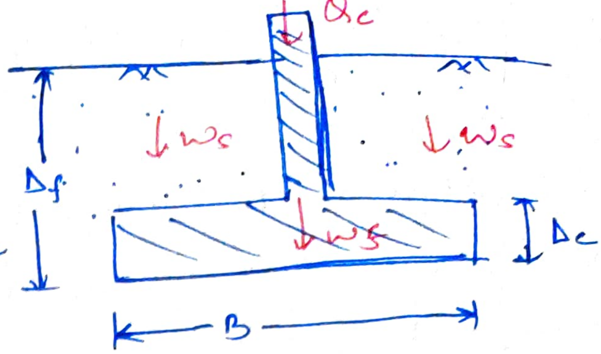
→ In such case net loading intensity would get reduced & even become zero.

(ii) Another reason for using net allow ~~the~~ able bearing pressure in design of footing subjected to superimposed load ($Q_c$) upto ground level.

[It consists colu$^n$ load in soil below the ground level equal to the wt. of soil.)

$$Q_g = Q_c + W_f + W_s$$

Area of footing $(A) = \dfrac{Q_g}{q_{c\text{-gross}}}$




Assuming the footing to be square of size "B"

$$B^2 = \frac{Q_g}{q_{a\text{-gross}}}$$

$$B = \sqrt{\frac{Q_c + W_f + W_s}{q_{a\text{-gross}}}}$$ Here $W_f = B^2 D_c \gamma_c$, $W_s = (D_f - D_c) B^2 \gamma$

$$\Rightarrow B = \sqrt{\frac{Q_c + B^2 D_c \gamma_c + B^2 (D_f - D_c)\gamma}{q_{a\text{-gross}}}}$$

→ The above equation can be used provided $W_f$ & $W_s$ are known.

→ $W_f$ & $W_s$ cannot be calculated unless depth of foundation "$D_f$" & size "B" is known.

→ Depth of foundation can selected by codal provisions but still calculation of size "B" of footing would require trial & error approach.

→ However this problem can be eliminated by use net loading intensity / net pressure.

$$Q_g = Q_c + W_f + W_s$$

$$\frac{Q_g}{B^2} = \frac{Q_c}{B^2} + \frac{W_f}{B^2} + \frac{W_s}{B^2} \Rightarrow q_g = \frac{Q_c}{B^2} + D_c\gamma_c + (D_f - D_c)\gamma$$

$$\Rightarrow q_g = \frac{Q_c}{B^2} + D_c(\gamma_c - \gamma) + D_f\gamma$$

$$\Rightarrow q_g - D_f\gamma = \frac{Q_c}{B^2} + D_c(\gamma_c - \gamma)$$

here $q_g - D_f\gamma = q_n$

$$\boxed{q_n = \frac{Q_c}{B^2} + D_c(\gamma_c - \gamma)}$$

Max$^m$ value of $\gamma_c = 24\text{-}26\ kN/m^3$ & $\gamma = 19\text{-}21\ kN/m^3$.

If $\gamma_c$ is assumed to be equal to $\gamma$, then —

$$q_n = \frac{Q_c}{B^2}$$

∴ For safe design of footing net loading intensity must be less than net allowed bearing pressure".

$$q_n \leq q_{a\text{-}net} \quad = \quad \frac{Q_c}{B^2} \leq q_{a\text{-}net}$$

$$\boxed{B = \sqrt{\frac{Q_c}{q_{anet}}}}$$

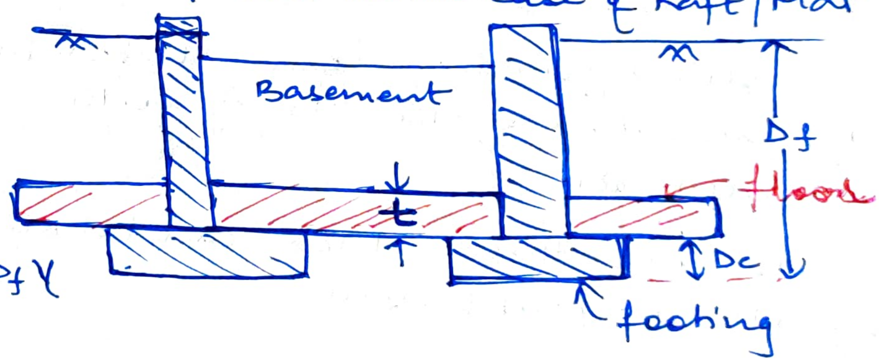
→ Since there is no ~~pa~~ unknown term in RHS, this approach is more preferable for proportioning of footing.

→ If the foundation is not backfilled as in case of Raft/Mat

$$q_g = \frac{Q_c}{B^2} + D_c\gamma_c + t\gamma_c$$

$$q_n = q_g - D_f\gamma$$

$$q_n = \frac{Q_c}{B^2} + (D_c + t)\gamma_c - D_f\gamma$$




∴ Since $D_c, t << D_f$ → $D_c$ & $t$ can be neglected

$$\boxed{q_n = \frac{Q_c}{B^2} - D_f\gamma}$$

→ It can be analysed that if backfilling is not done net loading intensity is reduced ~~that~~ by overburned pressure due to "$D_f\gamma$" & it may also reduce to zero when $\frac{Q_c}{B^2} = D_f\gamma$

(i.e when wt. of soil excavated is equal to safe structural load)

→ This principle is used behind designing of floating / Balancing / Compensated raft / footing.

Further, for safe design of footing

$\therefore q_n \leq q_{anet} \Rightarrow \frac{Q_c}{B^2} - D_f \gamma \leq q_{anet}$

$$\Rightarrow \boxed{\frac{Q_c}{B^2} \leq q_{anet} + D_f \gamma}$$

→ ~~This~~ It can be referred that load carrying capacity of foundation is considerably enhanced when foundation trench is not backfilled.

# Modes of Shear Failure

→ When a horizontal strip footing is subjected to the gradually increasing load, it undergoes characteristic settlement.

→ This settlement behaviour is found to be related to the soil properties.

→ The soil zones involved in generating resistance to foundation load, that is responsible for bearing capacity, have been identified qualitatively.

→ Three different types of failure mechanism based on pattern of shearing zones have been identified as follows —

## (I) General Shear Failure

→ It is typical of soils possessing brittle-type stress-strain behaviour (medium-dense sand).

→ This mode of failure is found in shallow foundation in very dense-medium dense sand.

→ It has well defined failure pattern

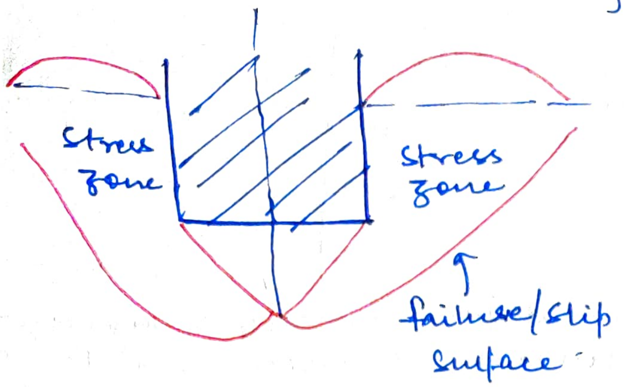
→ Stress zone in this case extend upto ground surface.

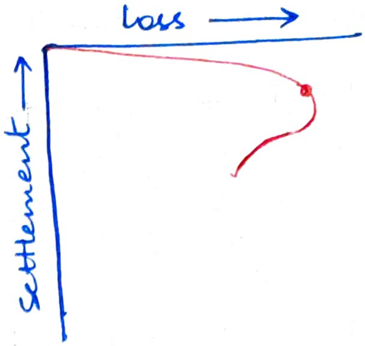
→ A sudden failure accompanied by tilting of foundation is observed in this case.

→ A bulging of ground adjacent to the foundation take place.

→ Before failure settlement would be small.

→ Load settlement curve indicates that failure is abrupt & ultimate load can be easily located.




General Shear Failure

## (II) Punching Shear Failure

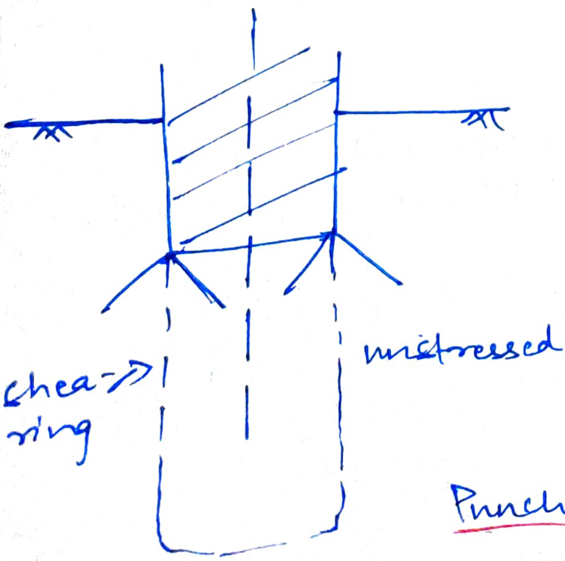
→ It occurs in soils possessing the stress-strain characteristics of very plastic soil.

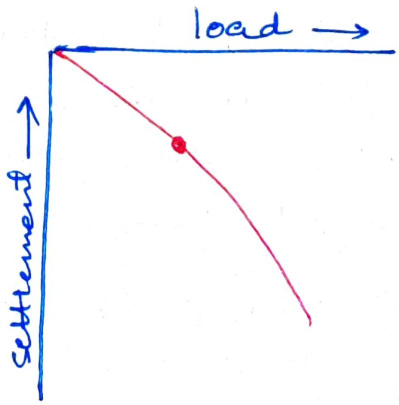
→ It is found in shallow foundation in loose sand / NCC & in deep foundations.

- It has poorly defined shear places.
- Soil zones beyond loaded area is not / little affected.
- Significant penetration / settlement of wedge shaped soil zone beneath the foundation is observed accompanied by vertical shear beneath edges of foundation.

→ The load settlement curve indicates a continuous increase in settlement with increasing load.

→ Ultimate load cannot be clearly identified.




Punching Shear Failure

## (III) Local Shear Failure

→ It's pattern has some of the characteristics of both general & punching shear failure.

→ Main failure of local shear failure is it well defined wedge & slip surface only beneath the foundation.

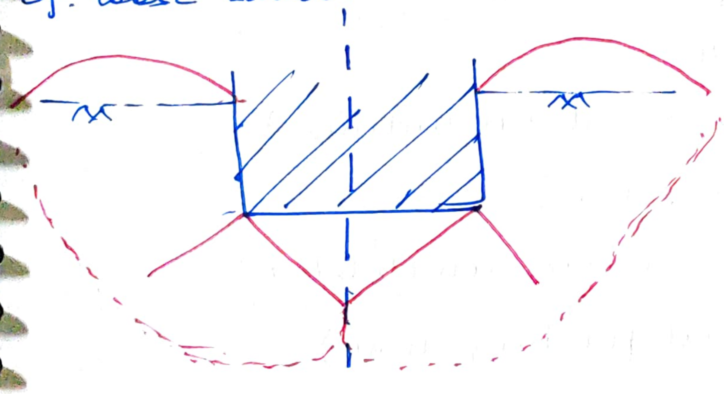
→ Slip surface not visible beyond the edges of the foundation.

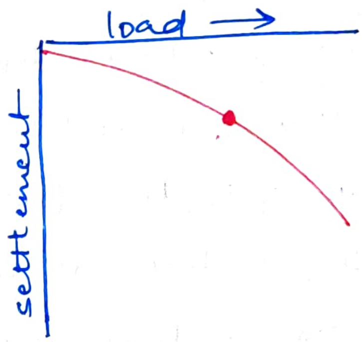
→ Slight bulging of the ground surface adjacent to the foundation may be observed.

→ Significant settlement of soil directly beneath the foundation take place.

→ The local-settlement curve does not indicate ultimate load clearly.

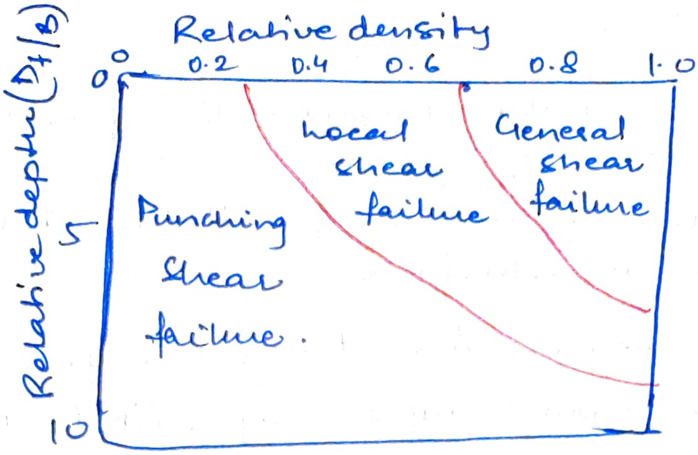
→ Soil possessing plastic stress-strain properties fail in this made eg: loose sand.






Note :→ Guidelines to identify mode of failure

| | General | Local |
|---|---|---|
| SAND | | |
| → friction angle ($\phi$) | $>36°$ | $<28°$ |
| → SPT No | $>30$ | $<5$ |
| → Relative density | $>70\%$ | $<30\%$ |
| → void ratio | $<0.55$ | $>0.75$ |
| CLAY | | |
| → UCS | $>100 kN/m^2$ | $<80 kN/m^2$ |




# Factors Affecting Bearing Capacity

(i) Position of ground water table.
(ii) Type of soil & its physical & avg properties ($\gamma$, c & $\phi$)
(iii) Type of foundation (strip, square, circular)
(iv) Size of footing (depth & width)
(v) Nature of ground surface (horizontal or inclined)
(vi) Type of loading (concentric/eccentric)
(vii) Initial stress on soil (Normal or over consolidated)
(viii) Type of shear failure (general, punching, local)

## Methods to Determine Bearing Capacity

It can be determined by any of the following —

(i) Analytical / static Mtd.
(ii) Codal provision : various building codes are published by codal agencies like BIS, NBC, IRC, CPWD etc. in which bearing capacity of zonal soil is published ~~with~~ which can be used directly for rough analysis.
(iii) Field Method
→ Plate Load test (PLT)
→ Standard Penetration test (SPT)
→ Cone Penetration test (CPT)

## (I) RANKINE'S MTD

→ It considered the equilibrium of two soil elements, one immediately below the foundation (element I) & the other just beyond the edge of footing (element II), but adjacent to element I.

→ When the load on footing increases & approaches ultimate bearing capacity ($q_u$), a state of plastic equilibrium is reached under the footing.

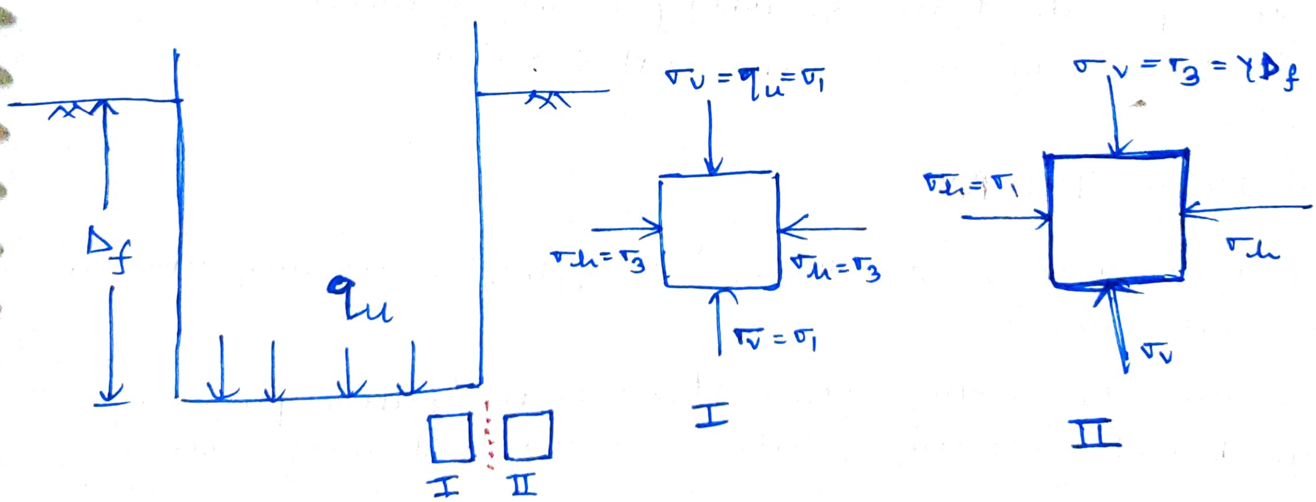
→ For the shear failure of element I, element II must ~~be~~ also fail by lateral thrust from element I

→ During the state of shear failure (plastic equilibrium), the analysis can be done as follows —

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

For cohesionless soil $c = 0$

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




For element I which is in ~~above~~ active state of eqlm.

$\sigma_1 = \sigma_v = q_u$

$\sigma_3 = \sigma_h = \gamma D_f$

$q_u = \sigma_h \tan^2\alpha$ —(ii)

for element II, which is in passive state of eqlim.

$$\sigma_1 = \sigma_h$$

$$\sigma_3 = \sigma_v = \gamma D_f$$

$$\sigma_h = \gamma D_f \tan^2\alpha \quad \text{—— (ii)}$$

from (i) & (ii)

$$q_u = \gamma D_f \tan^4\alpha \quad \text{or} \quad q_u = \gamma D_f \left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$$

Note: → In the effective stress analysis $q_u$ is given by —

$$q_u = \gamma' D_f \left(\frac{1+\sin\phi'}{1-\sin\phi'}\right)^2$$

→ This analysis can also be used to find the minimum depth of foundation as follows.

$$D_{f\,min} = \frac{q_u}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

→ This theory is only applicable of cohesionless soil.
→ Effect of width & size of footing is not considered.
→ If $D_f = 0$, then $q = 0$, which is not practical.

## II) FELLINIUS THEORY

→ It is based on plastic theory & is applicable for only pure cohesive soil (clays).
→ In this theory the rupture ~~phase~~ plane / failure plane is assumed to be an arc of circle & general shear failure is considered.

## (III) TERZAGHI'S THEORY

→ It is an improvement over Prandtl theory.

→ Prandtl considered the base of the footing to be smooth, whereas, Terzaghi considered the base to be rough.

# Assumptions in Terzaghi Theory

(i) Foundation is shallow ($D_f \leq B$)

(ii) Base of foundation is rough.

(iii) Footing is continuous (strip) ($L >> B$). It means analysis in 2D along with & depth.

(iv) At the time of failure soil reaches into plastic stage.

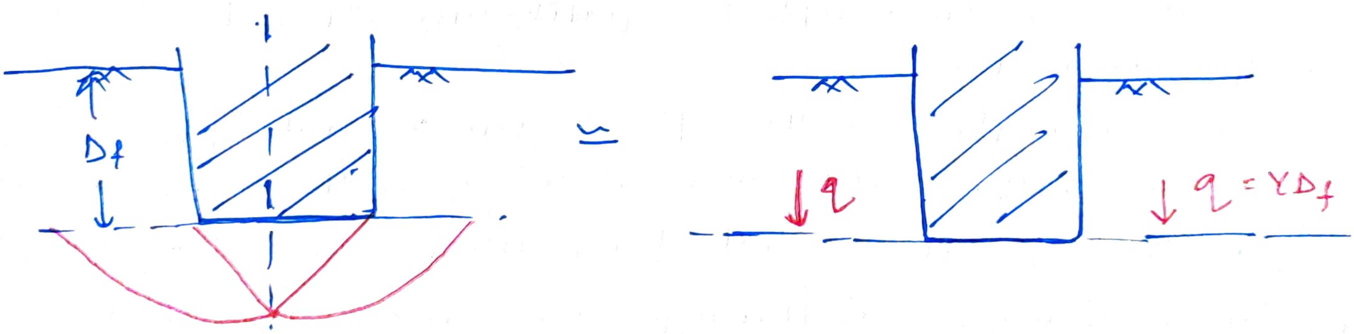
(v) Failure is general shear failure.

(vi) The stress zone of soil extends upto the foundation level only but not upto the ground level.

(vii) The shear resistance of soil above the foundation is ignored it means only base resistance is considered while side resistance is ignored.

Note :→ It is the main reason due to which this theory is not applicable for deep foundation.

(viii) The soil above the base of foundation is removed & replaced by an equivalent surcharge ($q = \gamma D_f$)

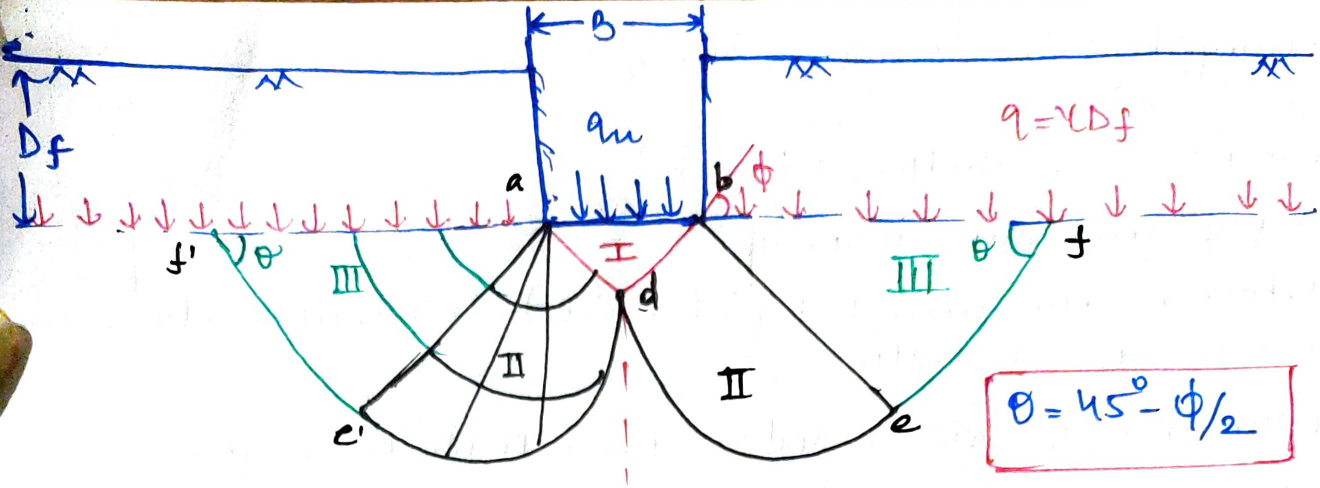



(ix) The load is vertical & concentric (Note : if load is eccentric Meyerhoff's theory).

(x) Ground surfaced is horizontal & foundation is also horizontal.

(xi) Water table is beyond the zone of influence of stress hence it does not effects the bearing capacity.

→ This theory based on the limiting equilibrium approach, wherein the forces acting on soil wedge immediately beneath the foundation are examined for static equilibrium condn of the ultimate bearing capacity is determined.




→ The base of footing being rough, when the footing sinks into the soil, a certain portion of soil, i.e. soil wedge "adb" immediately beneath the footing is prevented from undergoing any lateral movement by friction & adhesion btw^n the base of footing & the soil.

→ The wedge of soil "adb" is termed as ZONE-I and remain in a state of "ELASTIC EQUILIBRIUM" & is effectively a part of the footing itself.

→ At failure, the vertical downward movement of the footing & the intact soil wedge "adb" pushes the soil on either side of elastic wedge & transforms it into a state of "plastic equilibrium."

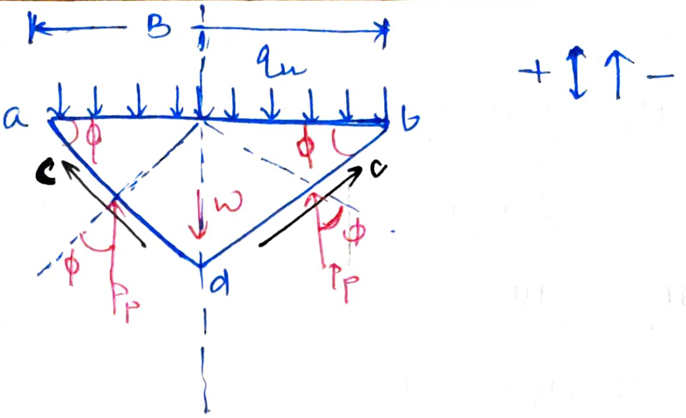
→ In zone II where soil is in plastic equilibrium it is termed as "zone of radial shear".

→ One set of shear plane in these ~~plane~~ zone radiate from the edge of footing & the curved lower boundary of these zone has shape of a circle (for cohesive soil) & logarithmic spiral (for c-$\phi$ soil)

→ Zone III is termed as Rankine passive zone of linear shear in which the two sets of shear plane are inclined at angle of ($\theta = 45° - \phi/2$) to the horizontal.

→ The lower boundary "ad" & "bd" of the elastic wedge are failure plane rising at angle of $\phi$ to the horizontal.

→ The failure surface de'f' & def are taken to be vertical at "$\phi$'




→ The footing ~~caused~~ cannot break into the soil, until the passive resistance of soil masses is overcome.

→ The inclined face 'ad' or 'bd' can be considered as a rough back of rigid wall with shear parameters $c$ & $\phi$.

→ At failure, $\Sigma f_y = 0$

$$q_u(B \cdot 1) + \gamma\left(\tfrac{1}{2} \times B \times \tfrac{B}{2}\tan\phi\right) = 2P_p + 2C\sin\phi$$

$$C = c \times (ad \times 1) = c\left(\frac{B/2}{\cos\phi}\right)$$

$$\Rightarrow \boxed{q_u B = 2P_p + Bc\tan\phi - \tfrac{1}{4}\gamma B^2\tan\phi}$$

Note :→ The total passive resistance $P_p$ on the surface ad & bd is made upto three components —

(i) $P_{p\gamma}$ produced by wt. of soil in shear zone assuming the soil to be cohesionless ($c=0$) & neglecting the surcharge.

(ii) $P_{pc}$ produced due to soil cohesion, assuming soil to be weightless ($\gamma=0$) and neglecting the surcharge.

(iii) $P_{pq}$ produced by the surcharge, assuming soil to be cohesionless & weightless ($c=0$, $\gamma=0$)

$$\Rightarrow q_u B = 2(P_{p\gamma} + P_{pc} + P_{pq}) + Bc\tan\phi - \tfrac{1}{4} \times \gamma B^2\tan\phi$$

Assume $2P_{p\gamma} - \tfrac{1}{4}\gamma B^2\tan\phi = B \times \tfrac{1}{2} \times BN_\gamma$

$$2P_{pc} + Bc\tan\phi = B \times cN_c$$

$$2P_p = B \times N_q$$

$$q_u B = BcN_c + BqN_q + \frac{1}{2}\gamma B N_\gamma$$

$$\boxed{q_u = cN_c + qN_q + \frac{1}{2}B\gamma N_\gamma}$$

I II III

$$q_u = cN_c + \gamma D_f N_q + \frac{1}{2}B\gamma N_\gamma$$

$\gamma$ in II term is for soil above the base of footing.

$\gamma$ in III term is for " below " " "

c is unit cohesion for soil below the footing.

$N_c, N_q, N_\gamma$ are bearing capacity which depends upon friction angle ($\phi$) of soil.

$$N_\phi = \tan^2(45^\circ + \phi/2) \Rightarrow N_q = N_\phi e^{\pi \tan\phi}$$

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

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

Note: → For pure clay ($\phi = 0$)

$N_\phi = 1$

$N_q = 1$

$N_\gamma = 0$

$N_c = 5.7$ (As per L'Hospital rule)

$$q_u = 5.7c + \gamma D_f \times 1 + 0.5 B\gamma (0)$$

$$q_u = 5.7c + \gamma D_f$$

$$q_{nu} = q_u - \gamma D_f = 5.7c + \cancel{\gamma D_f} - \cancel{\gamma D_f}$$

$$\boxed{q_{nu} = 5.7c}$$

→ The net ultimate bearing capacity of pure cohesive soil is independent of its size & depth.

→ The bearing capacity eq in Terzaghi & Prandtl theory is same with difference in only bearing capacity factor.

eg: for pure clay as per Prandtle theory.

$N_c = 3.14$

$N_q = 1$

$N_\gamma = 0$

(i) Modification for Diff Shape of the footing

(a) for strip footing :- $q_u = CN_c + qN_q + 0.5B\gamma N_\gamma$

(b) For square " :- $q_u = 1.3cN_c + qN_q + 0.4B\gamma N_\gamma$

(c) For circular " :- $q_u = 1.3cN_c + qN_q + 0.3B\gamma N_\gamma$

(d) For rectangular/raft footing

$$q_u = \left(1 + 0.3\,B/_2\right)cN_c + qN_q + \tfrac{1}{2}\left(1 - 0.2\,B/_2\right)B\gamma N_\gamma$$

| | Strip | square | circular | rectangular/Raft. |
|---|---|---|---|---|
| $S_c$ | 1 | 1.3 | 1.3 | $(1+0.3\,B/_2)$ |
| $S_q$ | 1 | 1 | 1 | 1 |
| $S_\gamma$ | 1 | 0.8 | 0.6 | $(1-0.2\,B/_2)$ |

(ii) Modification for Shear Failure

→ Terzaghi's theory is applicable for general shear failure.

→ No theoritical solution is available for Local & punching shear failure.

→ Punching shear failure is very uncommon, since footing are rarely place on very loose sand.

→ local shear failure is however common.

→ In case of footing on loose sand or soft clays where local shear failure can take place, shearing resistance is not mobilised along the entire length of failure surface as observed in general shear failure.

→ Terzaghi proposed emperical adjustments to shear strength parameter $(c, \phi)$ to cover the case of local shear failure.

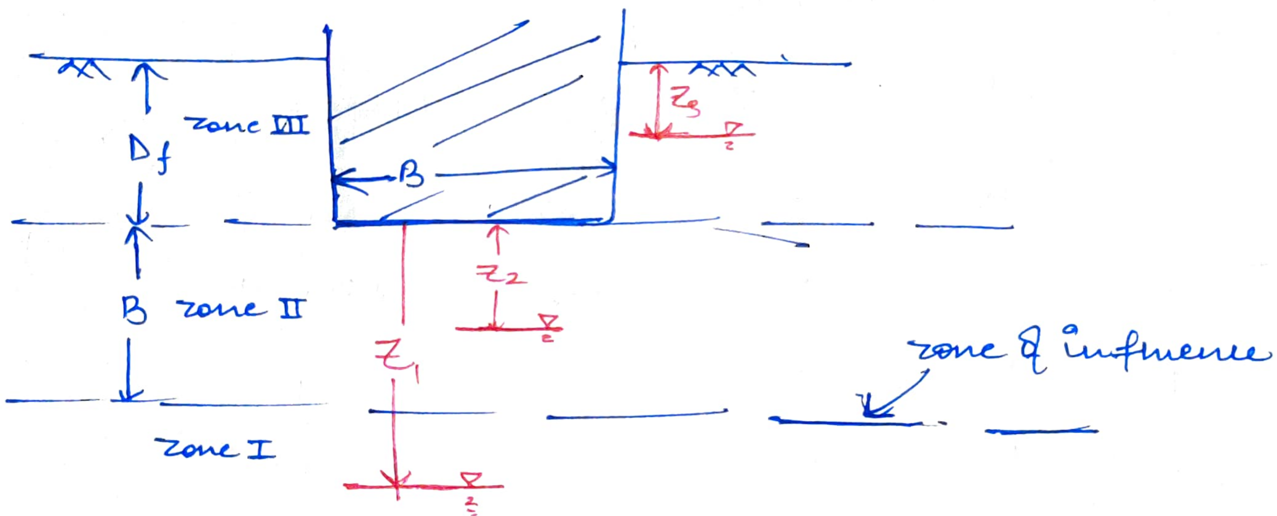
→ Shear strength parameters $C_m$ & $\phi_m$ must be used in bearing capacity equation & bearing capacity factors are determined on basis of $\phi_m$

$$C_m = \tfrac{2}{3} c \Rightarrow \tan\phi_m = \tfrac{2}{3}\tan\phi$$

Hence,

$$q_u = \tfrac{2}{3} c N_c' - q N_q' + 0.3 B \gamma N_\gamma'$$

(iii) Modification due to water table level




$$q_u = \underset{(i)}{CN_c} + \underset{(ii)}{\gamma D_f N_q} + \underset{(iii)}{0.5 B \gamma N_\gamma}$$

Case I → when water table is in Zone-I ie $(Z_1 > B)$

→ water table level below the zone of influence of stress.

→ water table in this zone has influence on the bearing capacity hence no change in bearing capacity is introduced ie terms (i), (ii) & (iii) remains unaffected.

case (ii) → when water table is in zone-II, at depth of $Z_2$ below the foundation $(0 \leq Z_2 \leq B)$

→ In this case terms (i) & (iii) of bearing capacity eq$^n$ will be affected whereas term (ii) will remains unaffected.

→ In this case terms (i) & (iii) of bearing capacity eq$^n$ will be affected whereas term (ii) will remain unaffected.

→ To account for water table effect either use effective parameters $c'$ & $\phi'$ & effective unit of the soil or use water table correction factor.

→ generally effect on $c$ & $\phi$ is neglected in absence of and day not given, but effect on unit wt. of soil is accounted, ie effect on first term is negative.

∴ In absence of data $c' \simeq c$, $\phi' \simeq \phi$

| Effective Parameters | Water table correction factor |
| --- | --- |
| $q_u = c'N_c' + \gamma D_f N_q' + 0.5 B \gamma_e N_\gamma'$ | $q_u = c'N_c' + \gamma D_f N_q' + 0.5 B R_\gamma + \gamma_a N_\gamma'$ |
| $\gamma_e = \dfrac{Z_2 \gamma + (B - Z_2)\gamma'}{B}$ | $\gamma_a = \dfrac{Z_2 \gamma + (B - Z_2)\gamma_{sat}}{B}$ |
| → $N_q' = N_q$ | $R_\gamma^* = \frac{1}{2}\left(1 + \frac{Z_2}{B}\right)$ |
| → If $Z_2 = B$, $\gamma_e = \gamma$ | → If $Z_2 = 0$, $\gamma_a = \gamma_{sat}$, $R_\gamma^* = \frac{1}{2}$ |
| $q_u = cN_c + \gamma D_f N_q + 0.5 B \gamma N_\gamma$ | $q_u = c'N_c' + \gamma D_f N_q + 0.5 B\left(\frac{1}{2}\gamma_{sat}\right)N_\gamma'$ |
| → If $Z_2 = 0$, $\gamma_e = \gamma'$ | |
| $q_u = c'N_c' + \gamma D_f N_q$ | |

case (iii) :→ when water table is in Zone-III, i.e at the depth $Z_3$ from ground level $(0 \leq Z_3 \leq D_f)$

→ In this case all three terms of bearing capacity eq$^n$ (i), (ii), (iii) will be effected, but effect on Ist term (i) is negligible.

→ Here also either use effective parameters or ~~&~~ water table correction factor.

## Effective Parameter

$$q_u = c'N_c' + \gamma_e D_f N_q' + 0.5B\gamma' N_\gamma'$$

$$\gamma_e = \frac{Z_3\gamma + (D_f - Z_3)\gamma'}{D_f}$$

→ If $Z_3 = 0$, $\gamma_e = \gamma'$

$$q_u = c'N_c' + \gamma' D_f N_q' + 0.5B\gamma' N_\gamma'$$

→ If $Z_3 = D_f$, $\gamma_e = \gamma$

$$q_u = c'N_c' + \gamma D_f N_q + 0.5B\gamma' N_\gamma'$$

## Water Table correction factor

$$q_u = c'N_c' + (R_q^* \gamma_a) D_f N_q' + 0.5B\gamma' N_\gamma'$$

$$\gamma_a = \frac{Z_3\gamma + (D_f - Z_3)\gamma_{sat}}{D_f}$$

$$R_q^* = \tfrac{1}{2}(1 + Z_3/D_f)$$

→ If $Z_3 = 0$, $\gamma_a = \gamma_{sat}$, $R_q^* = \frac{1}{2}$

$$q_u = c'N_c' + (\tfrac{1}{2}\gamma_{sat}) D_f N_q' + 0.5B\gamma' N_\gamma'$$

→ If $Z_3 = D_f$, $\gamma_a = \gamma$, $R_q^* = 1$

$$q_u = c'N_c' + \gamma D_f N_q + 0.5B\gamma' N_\gamma'$$

# SPECIAL CASE

(i) for cohesionless soil $(c=0)$ [i.e SAND]

$$q_u = cN_c + \gamma D_f N_q + 0.5B\gamma N_\gamma$$

$$q_u = \gamma D_f N_q + 0.5B\gamma N_\gamma$$

If water table rises to the ground level.

$$q_u = \gamma' D_f N_q' + 0.5B\gamma' N_\gamma' \text{ or } (\tfrac{1}{2}\gamma_{sat}) D_f N_q' + 0.5(\tfrac{1}{2}\gamma_{sat}) N_\gamma'$$

→ In cohesionless soil ultimate bearing capacity is proportional to depth $(D_f)$ & width of foundation $(B)$.

→ Due to rise in water table level upto ground level, ultimate bearing capacity approx reduced to half.

(ii) For pure cohesive soil $(\phi = 0)$ (clay)

$N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$

$$q_u = cN_c + qN_q + 0.5B\gamma N_\gamma$$

$q_u = 5.7c + \gamma D_f$

→ If water rises to the ground level.

$q_u = 5.7c' + \gamma' D_f \Rightarrow q_{nu} = 5.7c' + \gamma' D_f - \gamma' D_f = 5.7c'$

$$\boxed{q_{nu} = 5.7c'}$$

→ The ultimate bearing capacity of foundation placed on clay is independent of width/size of foundation, but is dependent on its depth. ($D_f$)

→ Net ultimate bearing capacity is nearly unaffected by rise of water table to the ground level.

## IV) SKEMPTON THEORY

→ This theory is partly based on field observation, partly on laboratory test & partly on theoritical analysis.

→ It is suitable for pure cohesive soil (clay).

→ It can be applied for ~~the~~ both shallow footing as well as deep footing.

→ In this theory base resistance and side resistance both are considered.

The net ultimate bearing capacity is given as —

$$\boxed{q_{nu} = CN_c}$$

c = unit cohesion

$N_c$ = Skempton bearing capacity factor which depends upon $\frac{D_f}{B}$ ratio.

case(i) if $D_f/B = 0$

| Footing | Strip | Square/Circular/Rect^n/Raft |
|---|---|---|
| $N_c$ | 5 | 6 |

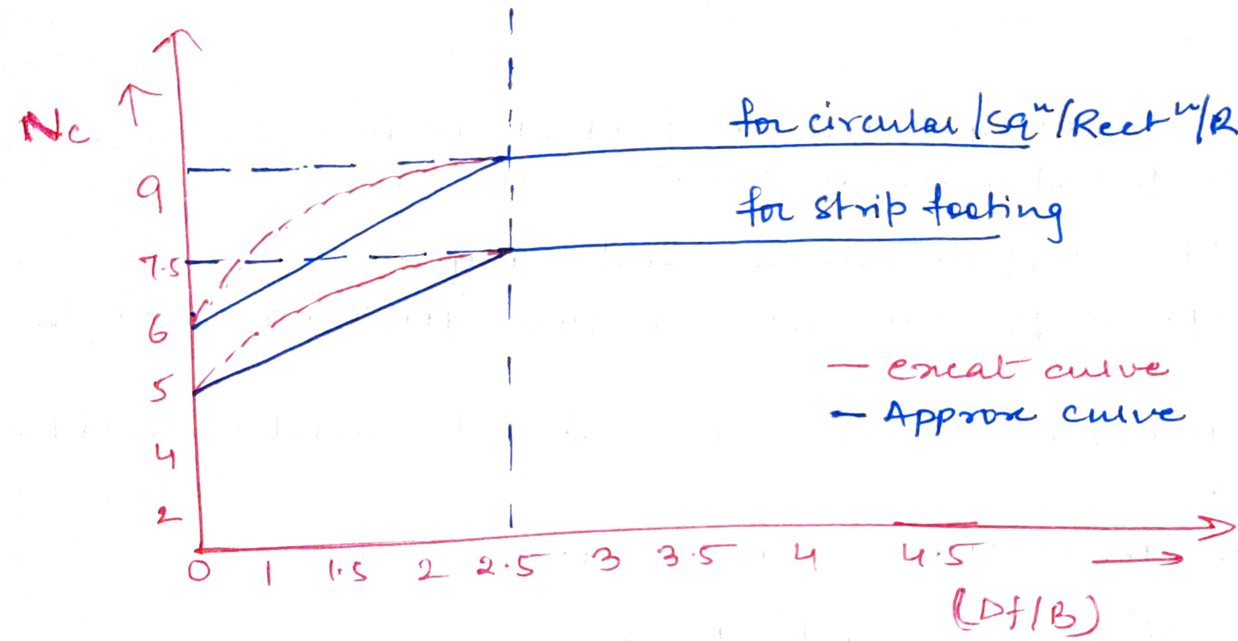
case(ii) if $D/B \geq 2.5$

| Footing | Strip | Square/circular/Rect^n/Raft |
|---|---|---|
| $N_c$ | 7.5 | 9 |

case (iii) if $0 < D_f/B < 2.5$

| footing | strip | square/strip | Rect^n/Raft |
|---|---|---|---|
| $N_c$ | $5\left(1+0.2\frac{D_f}{B}\right)$ | $6\left(1+0.2\frac{D_f}{B}\right)$ | $5\left(1+0.2\frac{D_f}{B}\right)\left(1+0.2\frac{B}{L}\right)$ |

Note :→ ultimate Terzaghi's theory Skemption are




(V) MEYERHOF'S THEORY

→ It is the most generalised theory available to find the bearing capacity as it gives comprehensive analysis for bearing parameters.

→ The failure mechanism in this is similar to that of Terzaghi but it considered failure surface that extended above the foundation level, hence shearing resistance of soil above the base of foundation is also taken into account.

→ Ultimate bearing capacity for shallow foundation in this case is given by —

$$q_u = \underbrace{cN_c s_c d_c i_c}_{(i)} + \underbrace{qN_q s_q d_q i_q}_{(ii)} + \underbrace{0.5B\gamma N_\gamma s_\gamma d_\gamma i_\gamma}_{(iii)}$$

Here $N_c$, $N_q$, $N_\gamma$ are Meyerhoff bearing capacity factor same as proposed by Prandtle.

- $N_q = e^{\pi \tan\phi} \tan^2(45^\circ + \phi/2)$
- $N_c = (N_q - 1)\cot\phi$
- $N_\gamma = (N_q - 1)\tan(1.4\phi)$

Here s, d, i are shape factor, depth factor & inclination factor for each of bearing capacity term respectively, given as –

| Factors | Expression |
|---|---|
| $s_c, s_q, s_\gamma$ | 1 for strip footing |
| $s_c$ | $1 + 0.2\,B/L \tan^2(45^\circ + \phi/2)$ |
| $s_q, s_\gamma$ | $1 + 0.1\,B/L \tan^2(45^\circ + \phi/2)$ for $\phi > 10^\circ$ and 1 for $\phi = 0$ |
| $d_c$ | $1 + 0.2\,D/B \tan(45^\circ + \phi/2)$ |
| $d_q, d_\gamma$ | $1 + 0.1\,D/B \tan(45^\circ + \phi/2)$ for $\phi > 10^\circ$ & 1 for $\phi = 0$ |
| $i_c, i_q$ | $\left(1 - \frac{\alpha}{90}\right)^2$. $\alpha$ is in degree |
| $i_\gamma$ | $\left(1 - \alpha/90\right)^2$ |

→ Here "B" refers to the width or diameter of foundation

→ The inclination factor 'i' takes into account the effect of inclination of load on bearing capacity.

→ "$\alpha$" is the inclination of the resultant force from the vertical [i.e $\alpha = \tan^{-1}(H/V)$]

→ Meyerhoff also recommends that in case of ~~value~~ strip & rectangular footing involving plain strain value of $\phi_{ps}$ is determined from the triaxial test should be converted to plain strain as follows —

$$\phi_{ps} = (1.1 - 0.1\,B/L)\,\phi_{tr}$$

→ Meyerhoff introduced the concept of useful width to compute the bearing capacity when the resultant load on footing act eccentrically with respect to centre of the footing.

→ To account for eccentricity of loading the footing dimensions are modified in such a way that becomes concentric to the reduced dimensions of footing.

→ For a strip footing, if the load has an eccentricity $e_x$ in direction of width, a modified width 'B'' is used in place of B in bearing capacity eq & in the determination of shape and depth factors.

$$B' = B - 2e_x$$

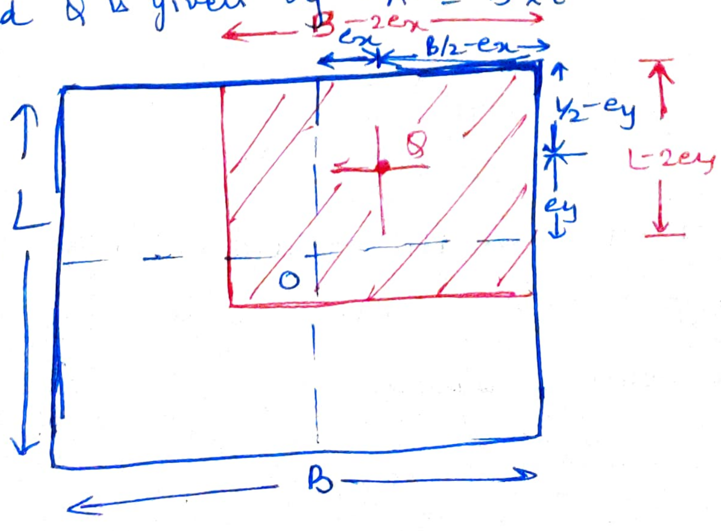
→ In a rectangular footing, there can be eccentricity of loading in direction of both width & length equal to $e_x$ & $e_y$.

→ In such case reduced dimension B' & L' are used in place of actual dimension.

$$B' = B - 2e_x$$

$$L' = L - 2e_y$$

→ The effective area 'A'' for the purpose of calculation of total vertical load Q is given by "A' = B' x L'"



## (vi) IS CODE RECOMENDATIONS FOR BEARING CAPACITY (IS 6403)

→ IS code recommends that for computation of ultimate bearing capacity c/a shallow foundation in general shear failure, following eq is used.

$$q_{nu} = cN_c s_c d_c i_c + q(N_q - 1) s_q d_q i_q + 0.5 B \gamma N_\gamma s_\gamma d_\gamma i_\gamma \cdot w'$$

Here $N_c, N_q, N_\gamma$ are bearing capacity factors recommended by Vesic.

$N_q = e^{\pi \tan\phi} \tan^2(45° + \phi/2)$ , $N_\gamma = 2(N_q + 1)\tan\phi$

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

→ Here $w'$ is water table correction factor [it is equivalent to $R_\gamma$]

$$\boxed{w' = \frac{1}{2}\left(1 + Z_2/B\right)}$$

→ $Z_2$ is measured from base of foundation.

→ $\gamma$ in third term is $\gamma_{avg}$.

→ The influence of water table is taken care in second term of bearing capacity eq. by taking 'q' as effective surcharge at the level of base of the footing.

→ The shape, depth & inclination factors are as follows —

| Factors | Expression |
|---|---|
| $s_c$ | $1 + 0.2 B/L$ for rectangle<br>1.3 for square & circle |
| $s_q$ | $1 + 0.2 B/L$ for rectangle<br>1.2 for square & circle |
| $s_\gamma$ | $1 - 0.4 B/L$ for rectangle<br>0.8 for square & 0.6 for circle |
| $d_c$ | $1 + 0.2\, D_f/B \tan(45° + \phi/2)$ |
| $d_q = d_\gamma$ | $1 + 0.1\, D_f/B \tan(45° + \phi/2)$<br>for $\phi > 10°$ & 1 for $< 10°$ |

$i_c = i_q$ | $(1-\alpha/90)^2$ $\alpha$ in degrees

$i_\gamma$ | $(1-\alpha/\phi)^2$

→ For Local failure the recommendation are same as of terzaghi theory.

∴ $C_m = \frac{2}{3}c$ ∴ $\tan\phi_m = \frac{2}{3}\tan\phi$

→ For cohesive soil, net ~~ult~~ ultimate bearing capacity of a footing immediately upon construction ($\phi_u = 0$) is given by —

$$q_{nu} = C_u N_c s_c d_c i_c$$

Here $N_c = 5.14$, & the undrained shear strength $C_u$ is obtained either from unconfined compressive strength test or from co-relation with point resistance value obtained from static cone penetration test.

→ $C_u$ varied between $\frac{q_c}{18} - \frac{q_c}{15}$ for NCC & between $\frac{q_c}{26} - \frac{q_c}{22}$ for OCC. [where $q_c$ is point resistance].

# Comparison of Bearing Capacity Factors

(i) $N_c = (N_q - 1)\cot\phi$ , $N_q = e^{\pi\tan\phi}\tan^2(45° + \phi/2)$

If $\phi = 0$, $N_c = 5.14$ (instead of 5.7 given by Terzaghi)

→ The above two equation have been adopted by —

- Terzaghi & peck
- Meyerhoff
- Hansen
- Vesic
- BIS

(ii) $N_\gamma = (N_q - 1)\tan(1.4\phi)$ Meyerhoff

$N_\gamma = 1.5(N_q - 1)\tan\phi$ Hansen

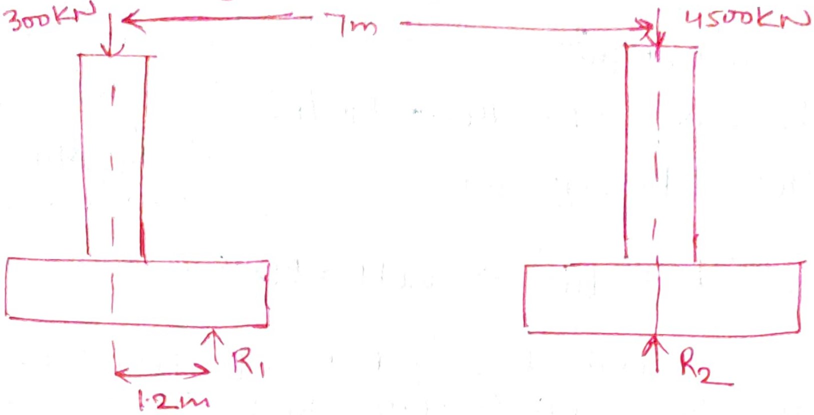
$N_\gamma = 2(N_q + 1)\tan\phi$ Vesic

$N_\gamma = 1.1(N_q - 1)\tan 1.3\phi$ Spangler & Hardy.

Note :→ $N_\gamma$ factor has the widest range of values for any N factors

→ In general $\boxed{38 \le N_\gamma \le 192}$ , for $\phi = 40°$

Q. A strap footing is to be provided for two columns as shown




Calculate the length of footing considering width of both footing as 3.5m & safe bearing capacity of soil as 350 KN/m²

(a) 2m (b) 2.5m (c) 3.5m (d) 2.95m

Note :→ The strap footing is considered to be stiff, transferring the column loads with equal & uniform soil pressure under both footing.

$\Sigma M_{R2} = 0$

$3000 \times 7 - R_1 \cdot (7 - 1.2) = 0$

$R_1 = 3620.7 \text{ kN}$

$\Sigma F_y = 0$

$3000 + 4500 - R_1 - R_2 = 0$

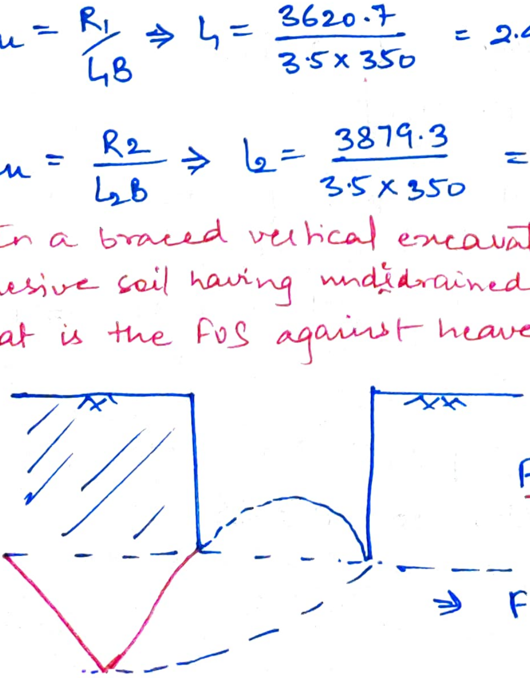
$R_2 = 3000 + 4500 - 3620.7$

$R_2 = 3879.3 \text{ kN}$

$q_u = \dfrac{R_1}{L_1 B} \Rightarrow L_1 = \dfrac{3620.7}{3.5 \times 350} = 2.95 m$

$q_u = \dfrac{R_2}{L_2 B} \Rightarrow L_2 = \dfrac{3879.3}{3.5 \times 350} = 3.167 m$

Q. In a braced vertical excavation of 5m ht & 2.5m width in a cohesive soil having undrained cohesion = 20 kN/m² & $\gamma_s = 20 \text{ kN/m}^3$ What is the FOS against heave failure at base.



$\dfrac{D_f}{B} = 5/2.5 = 2 \Rightarrow \dfrac{D_f}{B} < 2.5$

For no. failure → $q_n \ngtr q_{ns}$

$\Rightarrow \gamma H = q_{nu}/F$

$\Rightarrow F = \dfrac{CN_c}{\gamma H}$

$\Rightarrow F = \dfrac{20 \times 5}{20 \cdot 5}\left(1 + 0.2\, D_f/B\right) = \boxed{1.4}$

Q. An embankment is to be constructed with a granular soil ($\gamma = 20$ kN/m$^3$) on a saturated clayey silt deposit (undrained shear strength = 25 kPa) Assuming undrained general shear failure & Bearing capacity factor of 5.7 compute the max height of the embankment at point of failure.

$$q_n = \gamma H = 20H \text{ kN/m}^2$$

$$q_{nu} = CN_c = 25 \times 5.7 = 142.5 \text{ kN/m}^2$$

$H = 7.125$ m

At point of failure FOS (F) = 1

For safe designing $q_n \leq q_{ns} \Rightarrow 20H = 142.5/1$

Q. A building has to be supported on a RCC raft foundation of dimensions 14m x 21m. The subsoil is clay which has an avg unconfined compressive strength of 0.15 kg/cm$^2$. The pressure on the soil due to the weight of building & load that it will carry will be 14 t/m$^2$ at the base of raft. If the $\gamma$ of excavated soil is 1.9 t/m$^3$ at what depth should the bottom of raft be placed to provide a FOS of 3 against shear failure.

$$q_g = 14 \text{ t/m}^2 \qquad q_n = 14 - \gamma D_f \qquad CN_c s_c d_c i_c$$

$$q_{nu} = CN_c \Rightarrow \left[\frac{0.15 \times 10^{-3}}{2 \times 10^{-4}}\right] 5 \left(1 + 0.2 D_f/B\right)\left(1 + 0.2\, B/L\right)$$

$$\Rightarrow 0.75 \times 5 \left(1 + 0.2 D_f/14\right)\left(1 + 0.2 \times 14/21\right) \Rightarrow 4.25 + 0.06 D_f$$

$$q_{ns} = \frac{q_{nu}}{F} = \frac{4.25 + 0.06 D_f}{3} = 1.4165 + 0.02 D_f$$

For safe design of ~~following~~ footing $q_n \leq q_{ns}$

$$14 - 1.9 D_f = 1.4165 + 0.02 D_f \Rightarrow \boxed{D_f = 6.56 \text{ m}}$$

Q. A sq. footing of 2m sides rests on the surface of a homogeneous soil bed having the properties $c = 24$ kPa, $\phi = 25°$, $\gamma = 18$ kN/m$^3$.

Terzaghi bearing capacity factors for $\phi = 25°$ are $N_c = 25.1$, $N_q = 27.1$

$N_\gamma = 9.7$ $N_c' = 14.8$, $N_q' = 5.6$, $N_\gamma' = 3.2$

Compute the ultimate bearing capacity of the foundation.

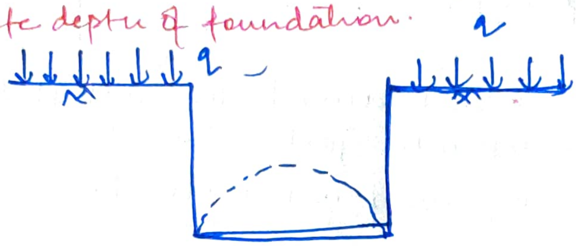
$$q_u = 1.3 c' N_c' + q N_q' + 0.4 B \gamma N_\gamma' \Rightarrow \cancel{c' = 2/3}$$

$$\Rightarrow c' = 2/3\, c$$

$q_u = 1.3 \times \left(\frac{2}{3} \times 24\right) \times 14.8 + 0 + 0.4 \times 2 \times 18 \times 3.2$

$q_u = 353.9$ kPa

Q: A raft foundation 10m wide & 12m long is to constructed in a clayey soil having shear str of 12 kN/m². γ of soil is 16 kN/m³. The ground surface carries a surcharge of 20 kN/m². FOS = 1.2 & Nc = 5.7. Find the safe depth of foundation.




$q_u = \left(1 + 0.3 \frac{B}{L}\right) c N_c + (\gamma D_f + q) N_q + \left(1 - 0.2 \frac{B}{L}\right) 0.5 B \gamma N_\gamma$

$q_u = \left(1 + 0.3\left[\frac{10}{12}\right]\right) 12 \times 5.7 + 16 \times D_f + 20 \Rightarrow q_u = 105.5 + 16 D_f$ kN/m²

$q_{nu} = q_u - \gamma D_f = 105.5$ kN/m² $\Rightarrow q_{ns} = q_{nu}/F = 105.5/1.2 = 87.91$.

for safe design $\boxed{q_n \leq q_{ns}}$

$\gamma D_f = q_{ns} \Rightarrow D_f = \frac{87.91}{16} = 5.49$ m

## III) FIELD METHOD

### (i) Plate load Test (IS:1888)

→ The test was designed to determine modulus of subgrade reaction which is used in designing of rigid pavement.

→ It is field test to determine the ultimate bearing capacity of soil, and probable settlement under a given loading.

→ The test essentially consists is loading a rigid plate at foundation level & determining the settlement corresponding to load increment.

→ The ultimate bearing is then taken as the load at which the plate starts sinking at a rapid rate.

→ This method assumes that down to the depth of influence of stresses the soil strata is reasonably uniform.

1) Bearing Plate: → The bearing is either square or circular made of mild steel of not less than 25mm thickness & size ranging from 300-750mm

→ Small size plate is used for dense & stiff soil whereas large size plate is used for loose & soft soils.

→ The size of plate shall be atleast ~~for~~ 4 times the max size of the soil particles present at test location.

ii) Test Pit :→ The test pit, usually at foundation level, must have normally width equal to five times the size of test plate.

→ The test pit should preferably have steps to go into the pit for setting & making readings.

iii) Loading Arrangement :→ The loading to the test plate may be applied with the help of a hydraulic jack. The reaction of the hydraulic jack can be taken by any of two method.

(a) Gravity loading Method (b) Reaction Truss Method.

→ When load is applied to the plate, it sinks to settle, where settlement of plate is measure with the help of atleast two dial gauges which are mounted on independent datum bar.

→ No support of loading platform should be locked within a distance of test plate from it centre.

iv) Setting of Plates :→ The test plate shall be placed over a fine sand layer of max thickness 5mm, so that the centre of plate coincides with the centre of the reaction girder/beam.

→ A minimum seating pressure of ($0.7 t/m^2$) shall be applied & removed before starting the load test.

v) Load Increments :→ Apply the load to soil in cummulative equal increment upto $10 t/m^2$ or $1/5$ of the estimated ultimate bearing capacity, whichever is less.

vi) Settlement & Observation :→ Settlement should be observed for each increment of load as interval of 1, 2.25, 4, 6.25, 9, 16, 25 mins & there after at every one hour interval. to the nearest of 0.02 mm.

→ In case of clayey soils, the time - settlement curve shall be plotted at each load stage & load shall be increased to next stage either when the curve ~~distribution~~ indicates that the settlement has exceeded 70-80% of probable ultimate settlement or at the end of 24hrs period.

→ For soils other the clay rate of settlement get appreciably reduced to a value "0.02 mm/min".

→ The next increment of load shall then be applied & observation is repeated.

→ The test shall be continued till a settlement of 25mm or (50mm extreme/special case) such as in dense gravel, gravel & sand mix is obtained or till failure, whichever is earlier.

→ When settlement does not reach 25mm, the test should be continued to atleast "two times" the estimated design pressure.

vii) Load Settlement curve & Ultimate Bearing Capacity

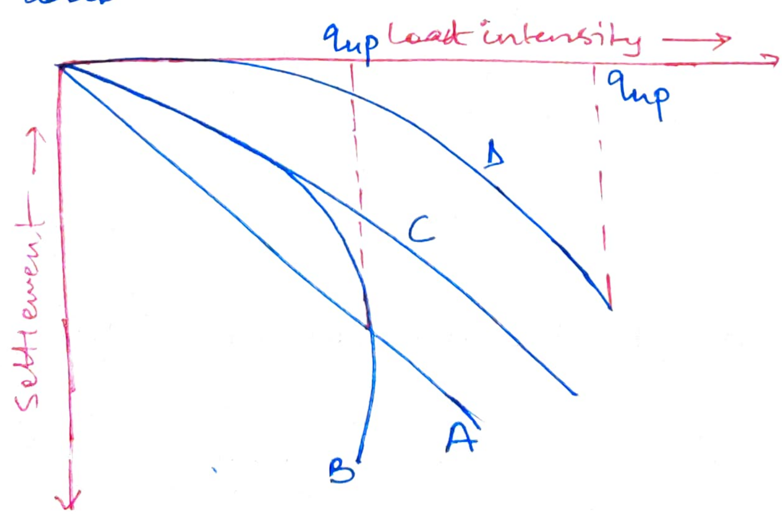
→ A load settlement curve is plotted out to airthmetic scale, std nature of which for different soils is as follows.

A: Loose to medium ~~cohesive~~ cohesionless soil.

B: cohesive soil

C: Partially cohesive soil

D: Dense cohe



→ Curve A :→ is typical for loose to medium cohesionless soil which is st line in early stages but flattens out in later stage. and there is no clear point of failure.

→ Curve B :→ is for cohesive soil which is not quite straight in early & leans towards settlement axis as the settlement increases.

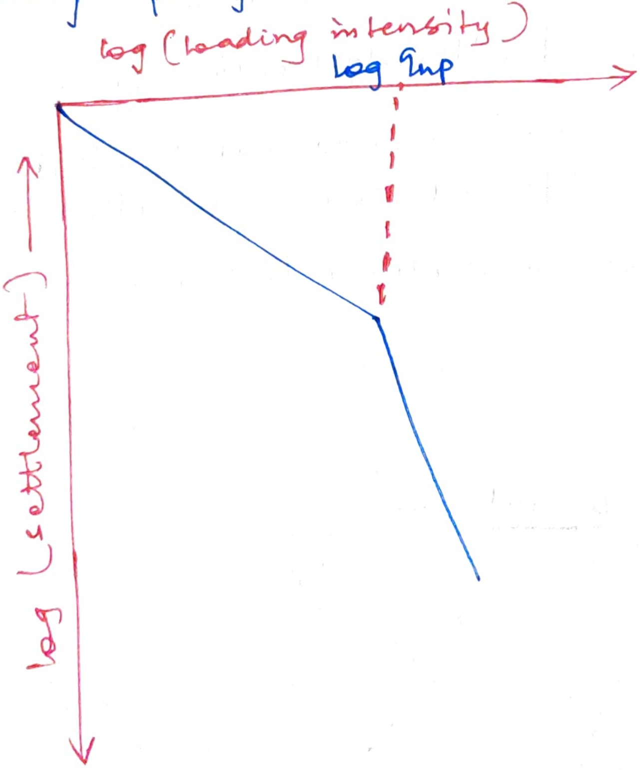
→ Curve C :→ posses characteristics of both curve A & B.

→ Curve D :→ is for pure dense cohesionless soil.

Note :→ for case B & D, no difficulty is experienced in observing ultimate bearing capacity as the failure is well defined.

→ However in case of curve A & C, where yield point is not well defined, settlement curve is plotted on semi-log scale.

→ IS Code does not specify any FOS hence in order to determine the safe bearing capacity it is taken be 2–2.5.




" Determination of ultimate Bearing Capacity as peak shear Criteria "

case A) FOR CLASS :

In clays, Bearing capacity is approx independent of width of footing, hence —

$$\boxed{q_{uf} = q_{up}}$$

$\therefore q_{uf}$ = ultimate bearing capacity for footing

$\therefore q_{up}$ = ultimate bearing capacity for plate which is determined from load intensity v/s settlement curve.

case B) For SAND :

in sandy soil, bearing capacity is proportional to the width of the footing.

$$\boxed{q_{uf} = q_{up} \cdot \frac{B_f}{B_p}} \qquad \{q_u \propto B\}$$

$B_f$ = width of footing

$B_p$ = " " plate.

Note :→ to compute safe bearing capacity FOS of 2-2.5 is applied over bearing capacity computed above.

Determination of safe ~~bearing capacity~~ settlement Pressure using PLT as per settlement criteria."

Let $S_f$ be the permissible settlement of footing prescribed by codal agencies then by using following empirical relation:

$$\frac{S_f}{S_p} = \left\{ \frac{B_f(B_p + 0.3)}{B_p(B_f + 0.3)} \right\}^2 \qquad \text{(for dense sand)} \quad (x)$$

$B_p$ & $B_p$ is in ~~cm~~ m

$$\frac{S_f}{S_p} = \frac{B_f}{B_p} \qquad \text{for clays}$$

$$\frac{S_f}{S_p} = \left(\frac{B_f}{B_p}\right)^{n+1} \qquad \text{for silt } (n = 0.5)$$

using above relationship for a particular soil find the permissible settlement of the plate ($S_p$) and then using load intensity settlement curve, read the safe bearing pressure for the plate

($q_{up}$) which is further used to find safe bearing pressure for footing ($q_{uf}$) by use of following relationship.

$$q_{af} = q_{ap} \quad \text{for clays}$$

$$q_{af} = q_{ap}' \frac{B_f}{B_p} \quad \text{for sands}$$

$q_{af}$ = allowable bearing pressure for footing
$q_{ap}$ = " " " plate

Note :→ The above computed pressure is allowable pressure or safe pressure in which further FOS is not req. because permissible settlement for footing, given by IS code is safe settlement.

Q. The load settlement curve data from a ~~data~~ plate load test ~~and~~ dry a sandy soil are as follows

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Load ($t/m^2$) | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 |
| Settlement (mm) | 4.5 | 10 | 15.5 | 22 | 29 | 38.5 | 50 | 64 |

The size of plate used was 0.3 m × 0.3 m. Determine the size of square colm footing to carry a load of 250 t with a maxm settlement of 25 mm.

Since $B_f$ is not known

let $B_f = 1.5 m$

$$q_{uf} = \frac{250}{1.5^2} = 111.11 t/m^2$$

$$\Rightarrow q_{uf} = q_{up} \times \frac{B_f}{B_p} \Rightarrow q_{up} = 22.22 t/m^2$$

Settlement from Table → for 22.22 $t/m^2$ load by interpolation $S_p = 11.1 mm$

$$\frac{S_f}{S_p} = \left\{\frac{B_f(B_p + 0.3)}{B_p(B_f + 0.3)}\right\}^2 \qquad S_f = 30.83 mm > 25 mm$$

Redesign

Increase $B_f$ in ratio of $\sqrt{\frac{30.83}{25}}$

$B_f = 1.5 \times \sqrt{\frac{30.83}{25}} = 1.67\,m \Rightarrow q_{uf} = \frac{250}{1.67^2} = 89.64\,t/m^2$

$q_{up} = 16.1\,t/m^2$, from table $S_p = 7.855\,mm$

$S_f = 22.58\,mm < 25\,mm$

# Housel Approach to find ultimate Bearing Capacity

→ Acc to Housel, the failure load is function of area and perimeter of plate / foundation & it is also influenced by soil properties.

→ In this method PLT is conducted on two rigid plates of different size at same "$D_f$".

→ Let $A_1$ & $P_1$ be area & perimeter of the Ist plate. & $Q_1$ is failure load on 1st plate. then —

$\therefore Q_1 = A_1 m + P_1 n$ —— (i)   ∴ m & n are constants which depends on type of soil.

→ Let $A_2$ & $P_2$ are area & perimeter of 2nd plate and $Q_2$ is failure/ultimate load on 2nd plate corresponding to the given settlement

$\therefore Q_2 = A_2 m + P_2 n$ —— (ii)

→ From (i) & (ii) m & n are computed.

→ It Area of footing is $A_f$ & Perimeter of footing $P_f$ then ultimate load at failure for foundation.

$\therefore Q_{uf} = A_f m + P_f n$

ultimate bearing capacity $q_{uf} = \frac{Q_{uf}}{A_f}$

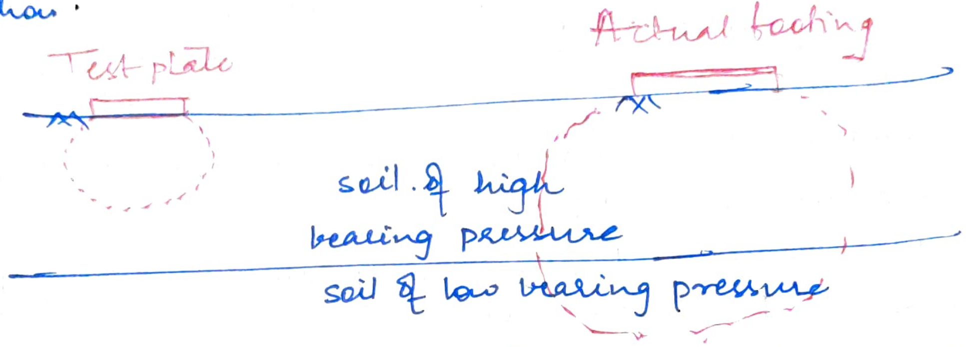
# Some of the more important considerations which need to be taken care of are as follows.

→ In no case shall a test plate smaller than 30cm can be used because load-settlement behaviour of soil is qualitatively different for smaller width of test plate.

→ It can be shown for settlement equation "x" settlement of a foundation cannot exceed about four times the settlement of plate of 30cm width, howsoever large is the width may be, hence this eq is valid only for medium to dense sand.

→ If the soil at site is not homogenous upto a large depth relative to the size of footing this test gives misleading result (It would give much smaller settlement than actual.)

Hence plate load test must be supplement by ~~approach~~ adequate soil exploration through borhole which may reveal any non-homogenity of strata, upto a depth of 1.5–2 times the width of foundation.




→ The effect of capillarity in sand bed is to increase effective stress, that would ~~be~~ give less settlement than actual, hence this test should be performed at water table level if it is within 1m below the foundation.

→ This test is short duration test hence is suitable for granular soil in which immediate settlement can be taken as total settlement for not for cohesive soil.

Note: → If the load test is carried out above water tables the settlement computed from curve will have to be corrected if there is likelihood of rise in water table in near future.

$$\text{Actual settlement} = \frac{\text{settlement from plate load test}}{\text{correction factor}}$$

# Penetration Test

→ These test involve the measurement of resistance to penetration of sampling spoon, cone or other shaped tools under dynamic or static loadings.

→ The resistance is then empirically correlated with some of the engineering properties of soil such as density index, consistency, bearing capacity etc.

→ The two commonly used test are
(a) Standard penetration Test (b) Cone penetration Test.

## (i) Standard Penetration Test (IS 2131)

→ This test is used to determine.

- Relative density / density index
- Angle of shearing resistance.
- unconfined compressive str.
- pile load capacity
- ultimate bearing capacity on the basis of shear centre.
- Allowable bearing pressure on basis of settlement criteria.

Note: → This test is suitable for medium to dense sand. In clays due to dynamic loading remoulding may occur & excess pore water pressure may set up whereas in loose saturated sand liquifaction may occur.

→ This test is performed in a clean hole, 55 to 150 mm in dia.

→ A casing or drilling mud is used to support the side of the hole.

→ A thick wall split spoon sampler, 50.8 mm outer dia & 35 mm inner dia is driven into the undisturbed soil at the bottom of the hole under the blows of 63.5 kg drive wt. with 75 cm free fall. The min open length of the sample should be 60 cm.

→ The split tube sampler commonly known as split spoon sampler resting on bottom of the bore hole is allowed to sink under it own wt.

→ ~~The~~ It is then seated 15cm with the blows of hammer falling through a ht of 75cm

→ thereafter the spoon sampler is further driven by 30cm

→ The no. of blows required to affect each 15cm penetration is recorded.

→ The first 15cm of drive may be considered to be seating drive.

→ The total no. of blows req for second & third 15cm of penetration is termed as penetration resistance "No"

→ If the ~~split~~ split spoon sampler is driven less than 45cm (total), then the penetration resistance of last 30cm is considered.

→ The entire sample may sometimes sink under its own wt. when very soft sub-soil stratum is considered / encountered, under such conditions it may not be necessary to give any blow to the sample & SPT value would be indicated be zero.

→ SPT value is not considered in fraction.

→ The test is repeated at every 2m to 5m interval or at the change of stratum.

→ The observed SPT no. (No) may be required to be corrected for the following.

(a) overburden pressure correction.

(b) water table / Dialatency / fine's correction.

# overburden pressure correction.

→ In granular soil, overburden pressure ~~depth~~ effects the penetration resistance.

→ If two soils having same relative density, but different confining pressure are tested, the one with a higher confining pressure ~~gives a higher~~ gives a higher penetration no. as the confining pressure in cohesionless soil increases with depth.

→ The penetration number for soil at shallow depth is under estimated & that at a greater depth is overestimated.

→ For uniformity, the N values obtained from field test under different effective overburden pressure are corrected to a std effective overburden pressure

$$N_1 = N_0\left(\frac{350}{\sigma_0' + 70}\right) \text{ or } \left(\frac{500}{1.42\sigma_0' + 100}\right)$$

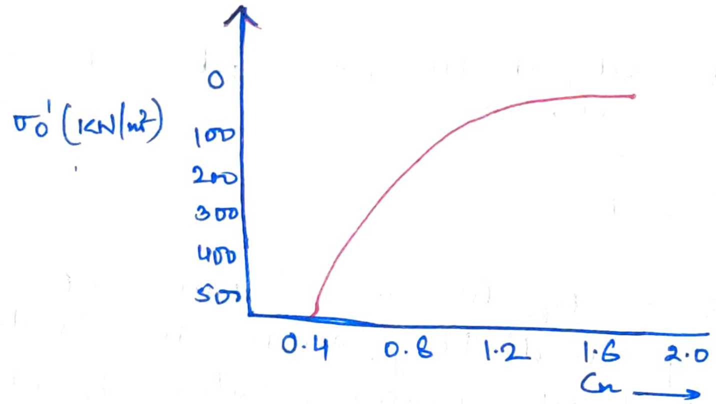
$\sigma_0'$ = effective overburden pressure ($kN/m^2$)

The above correction is applicable for

$\sigma_0' \leq 280\, kN/m^2$ & for dry or moist sand.

Note :→ The N value for cohesionless soil corrected for overburden pressure as per PECIC is given by –

$N_1 = N_0 C_n$ $C_n$ = normalising correction factor.




→ Ratio of $N_1/N_0$ should be b/w 0.45–2

→ If this ratio is more than 2, then $N_1$ should be divided by 2 to obtain the design value used in finding the bearing capacity of soil.

→ Overburden pressure correction is applied first & then dilatency correction is applied.

(ii) Dilatency correction

→ Silty fine sand & fine sands below the water table develops pore pressure which is not easily dissipated.

→ Pore pressure increases the resistance of soil, thus penetration number (N) also increases.

→ This correction is applied when the observed value of No. after first correction exceeds 15.

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

→ This correction is required only when WT is at or above the test level, i.e if WT is below the test level, this correction is not required.

→ Final corrected value of penetration number is avg of corrected value of Penetration No. at different levels.

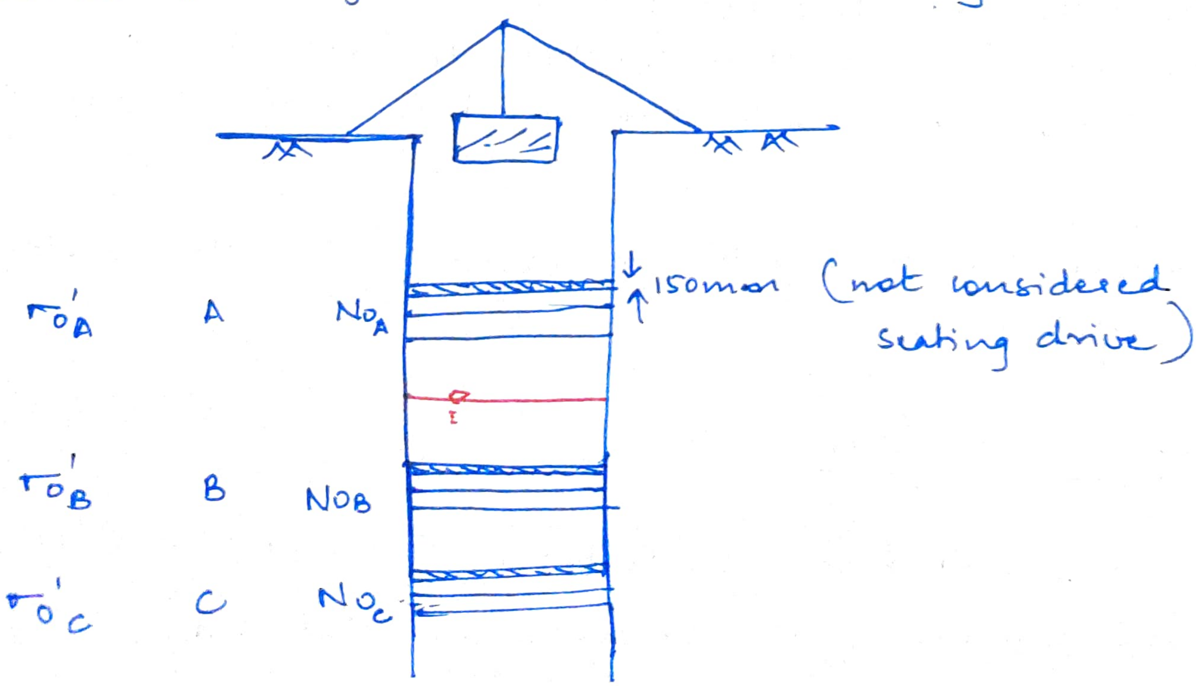
$$N_{final} = \frac{N_A + N_B + N_C \cdots N_n}{n}$$

→ If the is conducted at different levels say A, B, C, D --- N the above analysis is done.

→ At last check is applied that all the corrected SPT no. should be in the limits as follows.

$$N_{final} \pm 0.5 N_{final}$$

if any of the value $N_A, N_B$ --- $N_N$ has more than 50% variation on either side of final $N_{avg}$, that value is discarded & average is found from remaining values of SPT no.



Q: During the sub surface investigation for design of foundation a SPT is conducted at 4.5m below ground the ground surface The record of no. of blows is given below :-

| Penetration depth (cm) | No. of blows |
|---|---|
| 0 – 7.5 | 3 |
| 7.5 – 15 | 3 |
| 15 – 22.5 | 6 |
| 22.5 – 30 | 6 |
| 30 – 37.5 | 8 |
| 37.5 – 45 | 7 |

0–15: 3 + 3 } discarded

15–30: 6 + 6 } 12

30–45: 8 + 7 } 15

~~No = 27~~

Assuming the water table at ground level, soil is find sand & correction factor for overburden is 1. the corrected N value for soil is. $N_0 = 27$ $N_1 = N_0 \cdot C_n \Rightarrow N_1 = 27 \times 1 = 27$.

consider water table correction

$$N_2 = 15 + \frac{1}{2}(27-15) \Rightarrow N_2 = 21$$

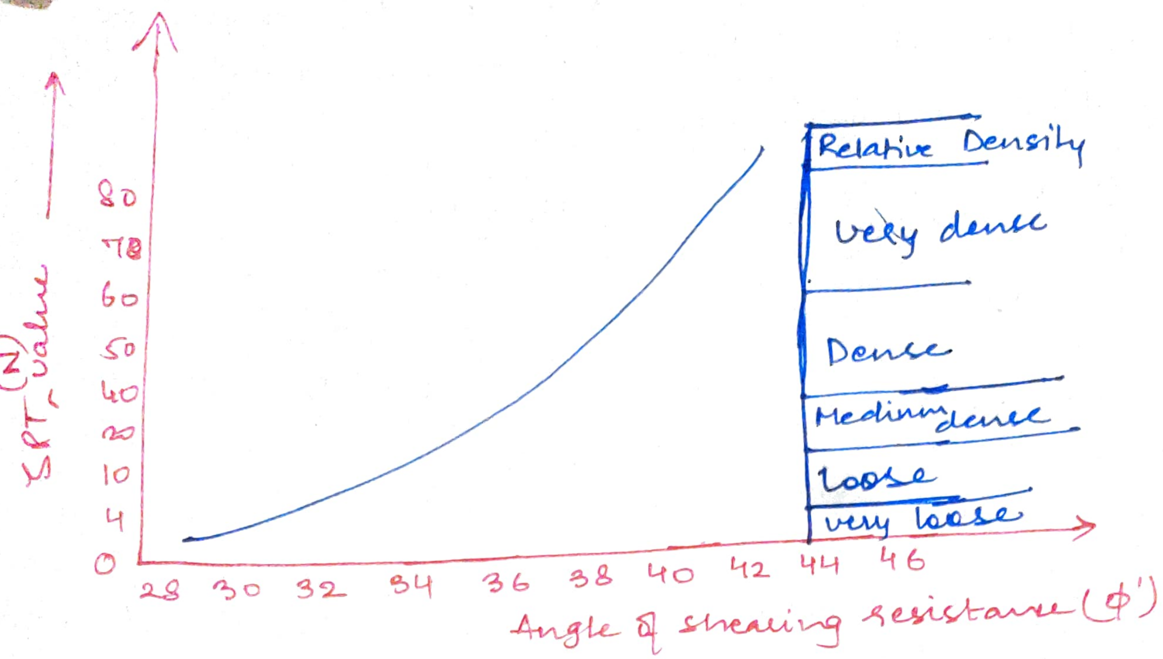
## # Ultimate Bearing Capacity As per Shear Criteria

→ The final corrected avg SPT no. is related to the friction angle & relative density, which is further ~~angle~~ expressed either graphically or in tabular form.

→ Using friction angle bearing capacity factors ($N_c$, $N_q$, $N_\gamma$) can be determined hence ultimate capacity can be found using any of analytical theory.

→ Relationship btw^n SPT value and Relative Density is as follows

| SPT (N) - Value | Relative Density |
|---|---|
| 0 – 4 | very loose |
| 4 – 10 | loose |
| 10 – 30 | Medium |
| 30 – 50 | Dense |
| > 50 | Very Dense |




# Determination of Allowable Bearing Pressure / Safe Bearing Pressure as per settlement criterion.

Emperical relationship can be used, to find allowable bearing pressure on the basis of SPT no.

## (i) Peck Hanson Equation

The net allowable bearing pressure or net safe settlement pressure is given by –

$$q_{a\,net} = 0.41\, C_w \cdot S \cdot N \quad (kN/m^2)$$

where, S = permissible settlement of foundation in mm as given by IS code.

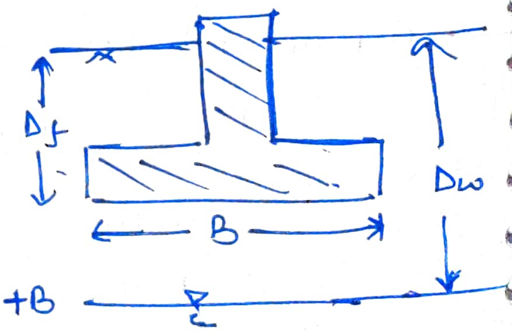
N = final corrected avg. SPT no.

$C_w$ = water table correction factor

$$C_w = \frac{1}{2}\left(1 + \frac{D_w}{D_f + B}\right)$$

$0.5 \leq C_w \leq 1$

$0 \leq D_w \leq D_f + B$




{Note: here depth of water table is measured from ground surface.}

## (ii) Teng's Equation

The net allowable bearing pressure is given by —

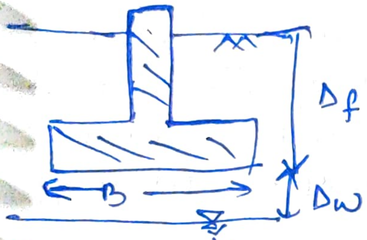
$$q_{a_{net}} = 1.4\left(\frac{B+0.3}{2B}\right)^2 \cdot S \cdot (N-3) \cdot C_w \cdot C_D \quad (kN/m^2)$$

where $S$ = permissible settlement (mm)
$B$ = width of the footing (m)
$N$ = final corrected avg. SPT no.
$C_D$ = depth correction factor

$C_D = (1 + D_f/B) \leq 2$

$C_w$ = water table correction factor

$C_w = \frac{1}{2}(1 + D_w/B)$




$0.5 \leq C_w \leq 1$
$0 \leq D_w \leq B$

{Note! Here depth of water table is measured from foundation level ∴ Teng's eqn also includes the effect of depth in combustion of safe bearing pressure}

# IS CODE METHOD

→ IS Code recommends use of Teng's eqn with some modification to find safe bearing pressure for footing.

$$q_{a_{net}} = 1.38\left(\frac{B+0.3}{2B}\right)^2 \cdot S \cdot (N-3) \cdot C_w \quad (kN/m^2)$$

→ IS Code recommends used Peck, Hanson & Thornburn eqn to find safe bearing pressure for raft.

$$\therefore q_{a_{net}} = 0.88\, C_w N S \quad (kN/m^2)$$

for permissible settlement of 25mm

$$\therefore q_{a_{net}} = 22\, C_w N \quad (kN/m^2)$$

If water is much below the foundation level

$$q_{a_{net}} = 22N \quad (kN/m^2)$$

Note :- → A raft or mat foundation covers the entire plan area of a building.

→ It is used when individual spread footing cannot be used ~~because~~ because of heavier load or due to poor soil condition or both.

→ Because of rigidity a raft tends to bridge over local soft pockets or any other heterogenity of soil & irregularities tends to get even distributed below a raft.

→ Due to these reasons, a raft foundation suffers much less differential settlement than isolated footing.

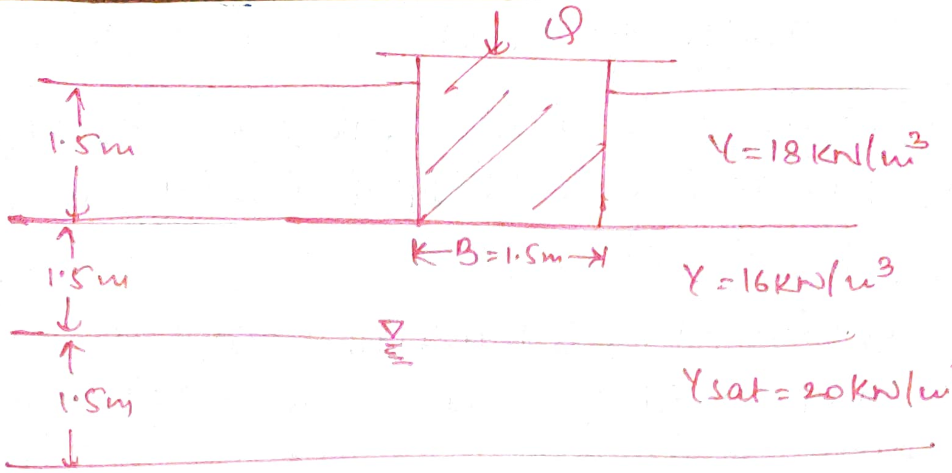
→ Hence the safe bearing pressure for a raft should be much higher than isolated footing (for same limit of differential settlement). or total permissible settlement for raft is almost 2 that of an isolated footing but still diff. settlement would be same.

→ The allowable bearing pressure of raft is always governed by settlement criteria as due to its large size raft assure a very high safe bearing capacity.

# unconfined compressive strength of cohesive soil.

| Consistency | very soft | soft | medium | stiff | very stiff | solid. |
|---|---|---|---|---|---|---|
| | $q_u$ (KN/m²) | | | | | |
| SPT(N) value | <2 | 2-4 | 4-8 | 8-15 | 15-30 | >30 |
| $q_u$ | <25 | 25-50 | 50-100 | 100-200 | 200-400 | >400 |

Q. using pen-hanson eqⁿ, find net safe bearing pressure below the foundation for the given situation test is conducted at 1.5m interval. and the observed value of SPT no. are 10, 15, 20 at depth of 1.5m, 3m & 4.5m resp. [$\gamma_w = 10 kN/m^3$]




$\rightarrow$ $q_{anet} = 0.41 C_w N S$

$C_w = \frac{1}{2}\left(1 + \frac{D_w}{D_f+B}\right) = \frac{1}{2}\left(1 + \frac{3}{1.5+1.5}\right) = 1$

$S = 40\,mm$

$N_0 A = 10$

$N_0 A_1 = N_0 A \left(\frac{350}{\sigma_0' + 70}\right)$ $\qquad \sigma_0' A = \gamma H_A = 18 \times 1.5 = 27\,kN/m^2$

$N_1 A = 10\left(\frac{350}{27+70}\right) = 36.08.$

$N_1 A / N_0 A = \frac{36.08}{10} = 3.608 > 2$

$N_2 A = N_1 A = 36.08$

$\boxed{N_1 A = 36.08/2 = 18.04}$

$N_0 B = 15 \Rightarrow \sigma_0' B = \gamma_1 H_1 + \gamma_2 H_B - 18 \times 1.5 + 16 \times 1.5 = 51\,kN/m^2$

$N_1 B = N_0 B\left(\frac{350}{\sigma_0' B + 70}\right) = 43.38$ $\qquad N_1 B / N_0 B > 2$, $N_1 B = \frac{43.38}{2} = 21.69 > 15$

$N_2 B = 15 + \frac{1}{2}(N_1 B - 15) = \boxed{18.345}$

$N_0 C = 20$ $\qquad \sigma_0' c = \gamma_1 H_A + \gamma_2 H_B + \gamma_3' H_c = 66\,kN/m^2$

$N_1 C = N_0 C\left(\frac{350}{\sigma_0' c + 70}\right) = 51.47$ $\qquad N_0 C / N_0 c > 2$ $\qquad N_1 C = \frac{51.47}{2} = 25.75 > 15$

$N_2 C = 15 + \frac{1}{2}(N_1 c - 15) = \boxed{20.365}$

$N_{final} = \frac{N_A + N_B + N_C}{3} = \frac{18.04 + 18.345 + 20.365}{3} = \boxed{18.91 \simeq 19}$

Chk $\Rightarrow$ $N_{final} \pm 0.5 N_{final} = 19 \pm 0.5 \times 19 = 9.5 - 28.5$

$q_{anet} = 0.41 \times 1 \times 19 \times 40 = 311.61\,kN/m^2.$

## (iii) Static Cone Penetration Test (CPT)

→ The static cone penetration test, simple termed as CPT in a simple test that is used in place SPT particularly for soft clays & fine to medium sand deposits.

→ "This test is also termed as "DUTCH CONE TEST."

→ The test assembly consist of a cone with an apex angle of 60° & base area of $10 cm^2$.

→ The sequence of operations of penetrometer are as follows—

(i) The cone & the friction jacket is in stationary position.

(ii) The cone is pushed into the soil by the inner rod to a depth of "a" at a steady rate of 20 mm/sec.

→ The trip resistance $q_c$ called cone or point resistance is the computed as $(q_c = Q/A)$ where $Q$ is force read on pressure gauge.

(iii) The sounding rod is pushed further to a depth 'b'. This has effect of pushing both friction jacket & cone assembly together. The total force $(Q_t)$ required for this again noted.

→ The force required to push friction jacket $(Q_f)$ is the obtained as $Q_f = Q_t - Q$.

→ The side or skin friction $f_s = Q_f/A_f$, where $A_f$ is surface area of friction jacket.

Note: → a & b are normally taken to be 40 mm however there min^m value is 35 mm.

(iv) The outside mantle tube is pushed down ~~the~~ to distance (a+b) bringing cone & friction jacket to original position.

→ The above procedure is continued till the proposed depth of sounding is reached.

→ CPT gives a continuous record of variation of both cone resistance & friction ~~resist~~ resistance of soil, however it does not give yield value of any sample.

Note :→ It is unsuitable in gravels & very dense sand owing to difficulty in pushing the cone.

→ Data from CPT is used to estimate the point bearing resistance & skin friction resistance of pile foundation.

→ In granular soil relationship btw $q_c$ & N is bearing further established.

| Type of soil | $q_c/N$, $q_c$ (kg/cm²) |
| --- | --- |
| sandy gravel or gravel | 8 – 10 |
| Coarse sand | 5 – 10 |
| fine to medium sand | 3 – 4 |
| silty sand | 2 |

→ further $q_c$ & N for fine silty medium loose to medium dense sand is given by –

$$\boxed{q_c = 4N}$$

→ $q_c$ can also be related with undrained shear strength of clay ($C_u$).

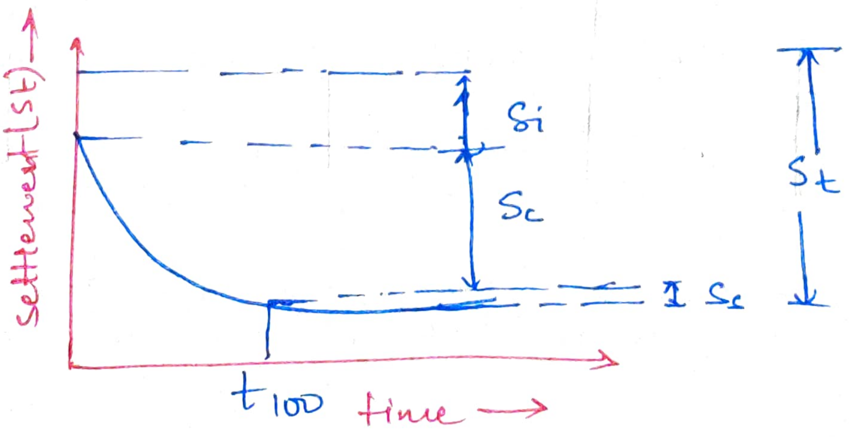
$$\boxed{q_c = N_K C_u + \sigma_0}$$

$N_K$ = cone factor   $\sigma_0$ = total overburden pressure.

# Settlement of Shallow Foundation

→ Total settlement ($S_t$) in general case for any type of soil is given by –

$$S_t = S_i + S_c + S_s$$



→ Immediate or elastic ~~different~~ settlement "Si" which takes place immediately or over a short time (approx about 7 days) after the load is place is termed as "Initial Compression".

→ In clay it is also known as "Distortion settlement." and is due to change in the shape of soil skeleton which any change water content.

→ It saturated clays immediate settlement is computed using elastic theory.

→ It is sometimes considered small compared to the longer term consolidation settlements hence neglected.

→ In granular soil immediately settlement accounts approx the entire settlement.

→ In inorganic clays primary consolidation accounts for most of settlement.

→ In organic soil secondary consolidation assumes greater significance.

→ The theory of elasticity can be used to determine settlement caused by load acting over flexible or rigid area of different geometrical ~~sto~~ shape. and is given by —

$$S_i = qB\left(\frac{1-\mu^2}{E}\right) I_f$$

$S_i$ = vertical displacement
B = width of foundation
$\mu$ = Poisson ratio.

E = Young's modulus of elasticity
$I_f$ = Influence factor for settlement which depends upon shape & rigidity of foundation.

| Shape | (If) Flexible foundation Centre | Corner | Average | If Rigid foundation |
|---|---|---|---|---|
| Circle | 1 | 0.64 | 0.85 | 0.86 |
| Square | 1.2 | 0.56 | 0.95 | 0.82 |
| Rectangle L/B = 1.5 | 1.36 | 0.68 | 1.2 | 1.06 |

→ Settlement of rigid foundation such as beam & slab raft is ~~rigid~~ approx equal to mean settlement for corresponding flexible foundation.

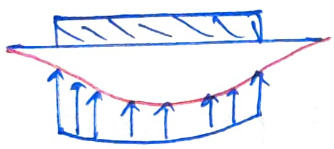
→ In a flexible footing such as beam & slab raft is approx equal to mean settlement for corresponding flexible foundation.

→ In a flexible footing, the contact pressure, ie pressure at the interface btwⁿ the footing & soil is uniformly distributed.

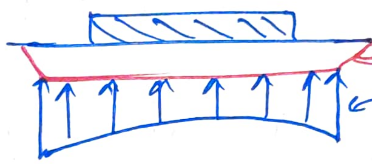
→ A uniform pressure produces a dish shaped pattern of displacement in clay soil.

→ For a rigid footing the settlement is more or less uniform over the area of contact.

→ Since a uniform contact pressure produces dish shaped settlement pattern the contact pressure must be more near the edges of loaded area. ~~of~~ & less near the centre in order to produce uniform settlement.



Flexible




Rigid

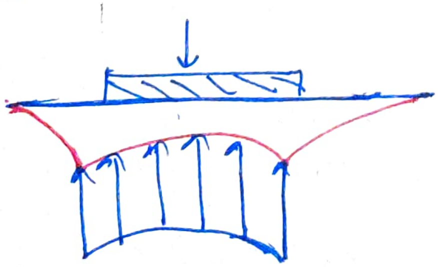
→ Because of variation in modulus of elasticity of granular soil the immediate settlement cannot be accurately found in such soils.

→ In these soils E increases with confining pressure and thus in increase with depth.

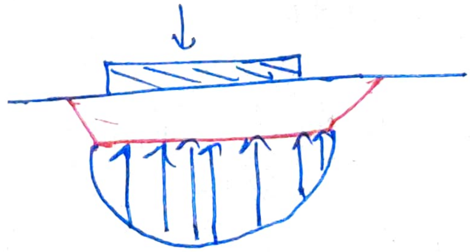
→ Further E varies across the width of the loaded area, it is more near centre, than near edges.

→ As a consequence the distribution of pressure will be uniform below flexible footing, but deformation will be inverted dish pattern.

→ For a rigid footing where settlement is uniform, the contact pressure is more near the centre than near the edges.

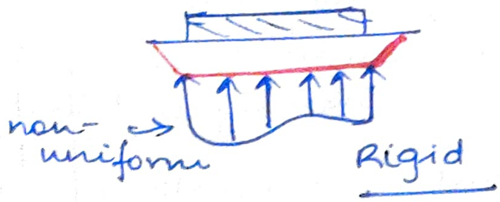


Flexible



Rigid

Note :→ for rigid footing in silt




Note :→ The elastic ~~consistency~~ constants of the soil are determined in the lab under conditions of stress range & mode of deformation which are identical with field observation

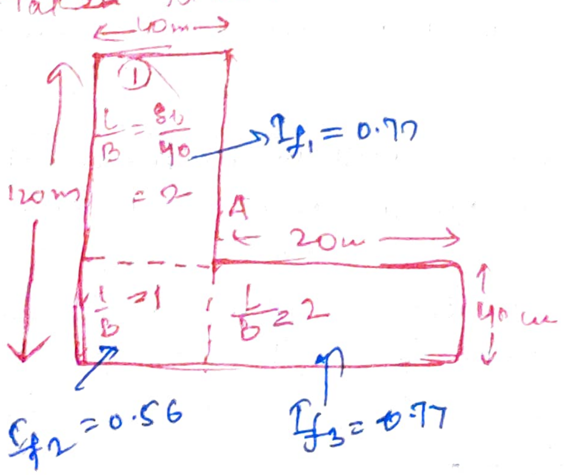
→ E & n for different types of soil can be related with SPT value and cone ~~point~~ resistance value empirically.

↔ Acc. to IS 8009, the ratio of total settlement of a rigid foundation to the total settlement at the centre of flexible foundation is called "rigidity factor."

→ A rigidity factor of 0.8 is recommended by code for computing the settlement of rigid footing.

Q. The plan of a proposed heap is shown. The heap will stand on a thick deposit of soft clay with $E = 15\,MN/m^2$. The ~~heap~~ uniform pressure on soil may be assumed as $150\,kN/m^2$. The immediate settlement ~~settlement~~ under the point A at surface of soil is ____ cm.

Taken $\mu = 0.5$




$\to I_{f_1} = 0.77$

$I_{f_2} = 0.56$ $\quad I_{f_3} = 0.77$

Value of Influence factor

| Shape of loaded area rectangle (L/B) | Flexible footing at corner ($I_f$) |
|---|---|
| 1 | 0.56 |
| 1.5 | 0.68 |
| 2 | 0.77 |
| 5 | 1.05 |
| 10 | 1.26 |
| 100 | 1.69 |

$$S_{i_A} = q\left(\frac{1-\mu^2}{E}\right)\sum_{i=1}^{3} B_i I_{f_i}$$

$$= 150\left(\frac{1-0.5^2}{15\times10^3}\right)\left[40\times0.77+40\times0.56+20\times0.77\right]$$

$$\simeq 51.45\ \text{cm}$$

# ALLOWABLE SETTLEMENT

→ Settlement can cause cracks in masonry walls and interior plaster walls of building.

→ It can cause a str. to tilt, which may become noticeable in high building or can avoid the function of str to fullfilled in many ways.

→ Settlement can of different patterns under different conditions & the effect caused on the structure will depend on the type of settlement.

→ If the structure settles uniformly it is not likely to suffer damage.

→ A structure with a very rigid raft or mat foundation will experience uniform settlement.

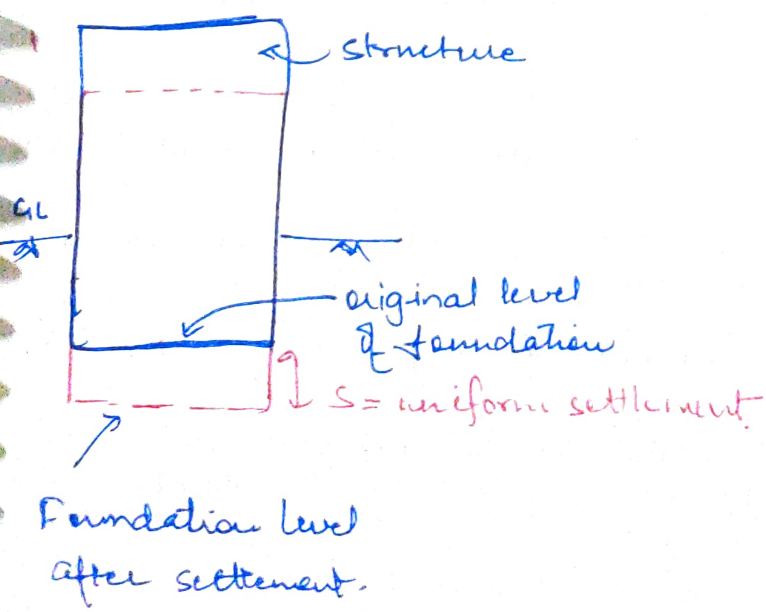
→ A structure is to said to undergo differential settlement, if one of its part settles more than the other.

→ The difference in total settlement btw^n any two points is called "differential settlement."

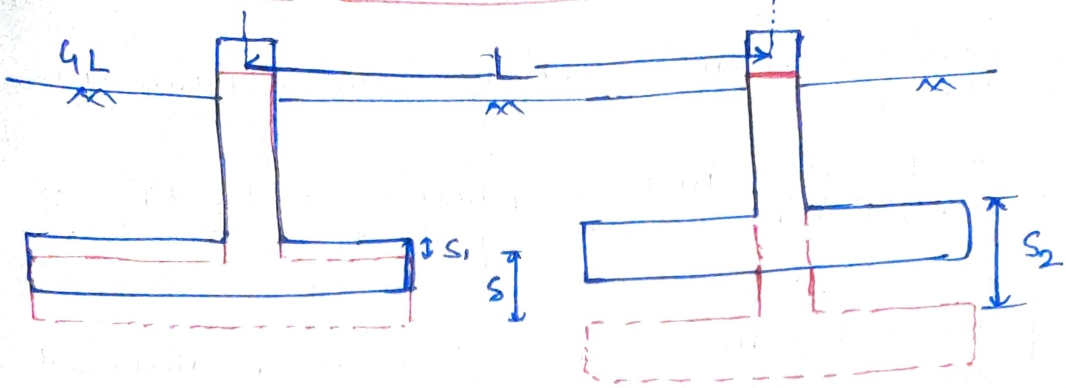
→ Angular distortion is the ratio of differential settlement b/w^n two colounms "$\delta$", to the spacing between them "$l$".

$$\therefore \text{Angular distortion} = \text{"}\frac{\delta}{l}\text{"}$$

→ The differential settlement may also lead to the uniform tilt, when the entire structure rockets as its consequences.

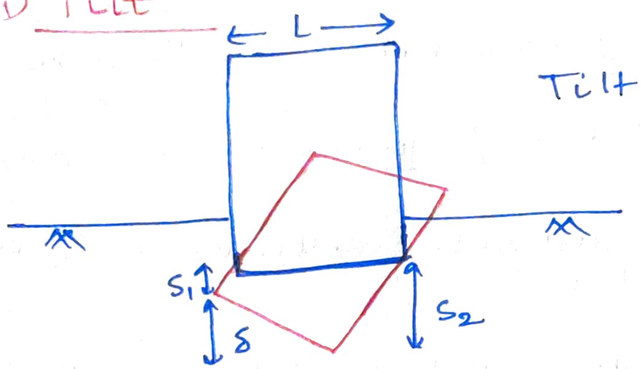


## (ii) Differential Settlement




Angular distortion $= \delta / L$

## (iii) Tilt




Tilt $= \delta / L$

→ It is relatively easy to estimate the total settlement & is very difficult to estimate differential settlement.

→ In granular soils the minimum differential settlement can, ~~is permitted~~ in some cases, be close to maximum total settlement whereas in clay the differential settlement is much less than total settlement.

→ Maximum differential settlement, generally does not exceeds 75% of the max total settlement in granular soil, while in clays it rarely exceeds 50% of maximum total settlement.

→ Higher total settlement is permissible in clay than in sand.

→ It is due to difference in rate of settlement in sand & clay.

→ In sand settlement occurs almost immediately on placement of load while in clay it takes time, thus there is time for structure resting on clay to adjust to differential settlements.

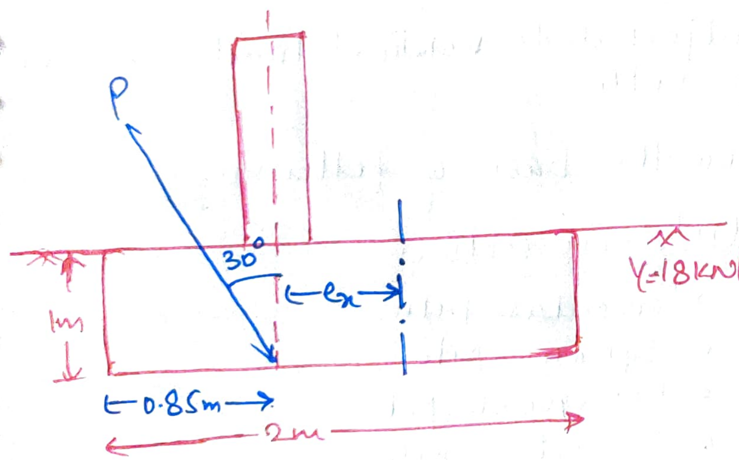
→ In sand, the differential settlement occurs as soon as total settlement, thus leaving the str no time for gradual adjustment.

→ In any settlement analysis all temporary & transient load have to be considered in determining the settlement in sand, whereas in clays permanent load than can cause consolidation settlement is to be considered.

Q. A square footing (2m×2m) is subjected to an inclined point load "P" as shown. The water table is located well below the base of footing consider only one-way eccentricity and compute the net safe load capacity of the footing for FOS = 3.

Assume :

| Capacity Factors | q | γ |
|---|---|---|
| Bearing | 33.3 | 37.16 |
| Shape | 1.314 | 1.314 |
| Depth | 1.113 | 1.113 |
| Inclination | 0.444 | 0.02 |




$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + 0.5B'\gamma N_\gamma s_\gamma d_\gamma i_\gamma$$

$$q_u = 0 + (1\times18)\times35\times1.314\times1.113\times0.444 + 0.5\times(2-2\times0.15)\times18\times37.16\times1.314\times1.113\times0.02$$

$$q_u = 405.84\ \text{kN/m}^2$$

$$q_{nu} = q_u - \gamma D_f = 405.84 - 1\times18 = \cancel{388} \ 387.84\ \text{kN/m}^2$$

$$q_{nf} = \frac{387.84}{3} = 129.3\ \text{kN/m}^2$$

$$Q_{ns} = q_{nf}\times A = 129.3\times(2\times1.7) = 439.55\ \text{kN}$$

# Deep Foundation

→ In situation where soil is poor at shallow depth, in order to transmit the load safety, the depth of foundation has to be increased till a suitable, soil stratum is met.

→ In view of increased depth, such foundations called deep foundations.

→ Pile, pier, well are examples of deep foundation

## # Pile foundation

→ A pile is a relatively small diameter shaft which is driven or installed into the ground by suitable means.

→ The piles are usually driven in groups to provide foundation for str^n.

→ The pile group may be subjected to ~~vertical~~ loads, horizontal loads, or a combination of both.

→ Piles have been classified on the basis of following –

(i) Material & construction
- Timber pile
- Steel pile
- Concrete pile.
- Sand pile.

(ii) Cross section
- circular pile
- square pile
- hexagonal pile
- I-section pile
- H section pile.

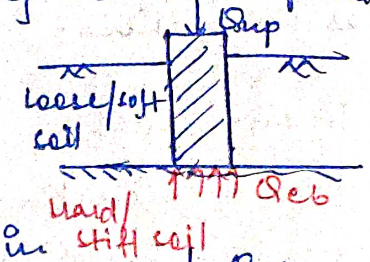
(iii) Shape
- cylindrical ~~shape~~ pile
- tapered pile
- under-reamed pile

These piles are considered useful in expansive soil where the use of shallow spread footings is not feasible due to excessive shrinkage ~~or~~ swelling behaviour of such soils.

→ A further development of these piles is multi-under reamed piles, in which when the number of bulbs is increased from one to two the carrying capacity of pile increases by about 50%

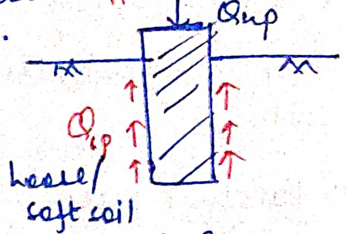
## (iv) Mode of load transfer

→ End bearing pile :→ These piles rest over stiff or hard strata & load carrying capacity is due to end bearing action or point resistance.

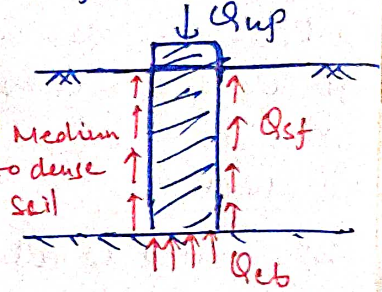



→ The length of these piles depends on position of hard strata.

→ Friction/hanging piles :→ Such piles are driven in soft clay or loose sand extending to greater depth. The load carrying capacity of these piles is due to skin friction action. The length of friction piles may be 10-20m.




→ Bearing & friction piles :→ If pile are driven in medium, to dense / stiff soil, then load carrying capacity is due to combine action of end bearing action & sking friction action.




## (v) Method of forming

- pre cast pile
- pre stressed
- Cast in situ

## vi) On the basis of functions

### i) Laterally loaded piles

Piles are also used to resist horizontal load, as in case of foundation for retaining walls, bridge abutments, etc.

As the horizontal load acts perpendicular to the pile axis, these piles ~~axis~~ are known as laterally loaded piles.

### ii) Batter Piles

In case of large lasteral loads piles are driven at an angle & are termed Batter piles, which serve better than vertical piles.

iii) Compaction piles

Short piles are sometimes used for compacting loose sand deposits when get densified by vibrations set up on driving are termed as compaction.

iv) Tension piles

→ Piles are sometimes used to resist uplift loads and are thus in tension, hence known as tension / uplift piles.

→ They are suitable to be provided in swelling soils eg. black cotton soil.

v) Anchor piles

→ Piles can also be used to provide anchorage against horizontal pull as in case of anchored bulk heads termed as anchor piles.

vi) Sheet Pile

→ It is used to retain earthfills and provided below hydraulic structures.

vii) Fender Pile

→ These are used to anchor the structure against tidal waves or floating objects in water.

viii) On the basis of method of installation

(a) Driven piles
(b) Bored piles
(c) Vibrated piles
(d) Jetted piles

(a) Driven / Displacement Piles

These piles are driven through hammer action, such piles are essentially pre cast, made of metal / wood.

→ In driven piles, end bearing resistance & skin friction resistance both are efficiency developed.

(a) Bored Piles

Bored piles may be precast or cast in situ. These piles are less efficient than driven piles.

Note :→ The best way to classify the pile is on the basis of the effect of installation of pile on the soil.

→ Based on this criterion, piles can be considered to fall into two classes—

(a) Displacement Piles (b) Non-Displacement Pile.

→ If during the installation of the pile, a large vol. of soil is displaced laterally & upwards, such a pile is termed as "DISPLACEMENT PILE".

→ In loose sand such pile densifies the sand upto a distance of about 3.5 times the ~~dilution~~ diameter of pile from centre.

→ Compaction of sand leads to increases in its angle of shearing resistance within the zone of influence, Impact of which can be computed as follows :—

$$\boxed{\phi_2 = \frac{\phi_1 + 40^\circ}{2}}$$

$\phi_1$ = initial value of angle of shearing resistance.

$\phi_2$ = final " " " " after compaction

• If $\phi_1 > 40^\circ$, no benefit will be derived from piles-driving.

→ For $\phi_1 > 40^\circ$, pile driving shall ~~be~~ in fact, have the effect of reducing the angle of shearing resistance due to dialancy effects.

→ In clays large displacement pile remould the soil to a distance of about twice the dia of pile.

→ During driving very high pore water pressure are set up around the pile & soil regains its initial str only after a period of time, when the excess pore water pressure dissipate.

→ This is why displacement piles are preferred for use in "loose to medium dense sand" & not in "dense sand or clay".

→ Driven & cast in situ or precast piles, made up of concrete or timber are examples of displacements piles.

→ A void / opening is formed in the soil by boring or ~~exc~~ excavation & is then filled with concrete.

→ These piles offer advantage of no ground heaving noise or vibration, their length can be easily varied at the site & it is possible to install very long piles with large diameter.

# SELECTION OF PILE TYPE

→ The selection of the type of pile to be used depends upon several ~~frame~~ factors.

(a) type of structure & load it carries.
(b) Location of the site.
(c) Soil condition & position of water table.
(d) Required pile length & structural capability of pile.
(e) economy.

→ The selection of material of the pile would depend upon the magnitude of structural load.

→ For light load timber pile can be used & for heavy loads only steel or RCC piles are considered.

# PILE LOAD CAPACITY

→ For satisfactory behaviour of pile foundation, it must safe in shear failure & settlement failure criterion (same as that of shallow foundation)

→ The load capacity of the pile can be estimated by several methods as follows.

(a) Static / Analytical Pile load Mtd.
(b) Dynamic / Pile Driving Mtd.
(c) Pile load test.
(d) Correlation with penetration test Data.

## (a) Static Pile Load Method

→ When a compressive load is applied at the top of pile, it tends to move vertical downward relative to the surrounding soil.

→ As a result, the applied load is distributed as friction load along a certain length of pile from the top.

→ As the load at top is increased, the friction load distribution will extend more & more towards the tip of pile. till a certain extent the entire length of pile is involved in generating friction resistance.

→ This is termed as ultimate skin friction resistance.

→ Load in excess of this, begins to be transferred to the soil at the base of pile & is termed point resistance / base / end bearing resistance at the point where pile fails by punching shear failure.

→ The max load which pile can support through combined resistance is sking friction & end bearing is termed as "ULTIMATE LOAD CAPACITY" ($Q_u$) of the pile.

$$\boxed{Q_u = Q_{eb} + Q_{sf}}$$

→ If $Q_{eb} >> Q_{sf}$ the pile is termed as end/point bearing pile & $Q_{sf} >> Q_{ef}$ the pile is termed friction pile.

→ The relative proportion of load carried by end bearing & skin friction depends on the shear strength & elasticity of soil.

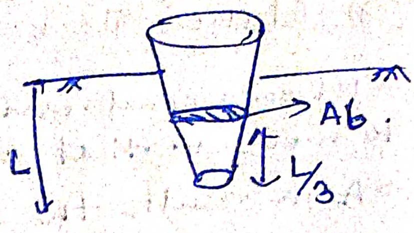
Note: → It is being observed that —

(i) when the ultimate skin friction resistance is mobilised only a fraction of ultimate point load is mobilised.

(ii) when the ultimate point load is resistance is mobilised, the skin friction resistance is decreased to a lower value than its peak.

$$Q_u = Q_{sf} + Q_{eb}$$

$$Q_{eb} = q_{eb} A_b \ , \ Q_{sf} = q_{sf} A_s$$




$$\boxed{Q_u = q_{eb} A_b + q_{sf} \cdot A_s}$$

$A_s$ = surface area of pile upon which skin friction acts.
$A_b$ = area of cross-section of pile on which bearing resistance acts.

Note :→ For tapered pile '$A_b$' may be taken as the cross-sectional area at the lower one-third of the embedded length.

$q_{eb}$ = unit point / toe / end bearing resistance
$q_{sf}$ = avg. skin friction resistance.

Case (i) COHESIVE SOIL

→ For the pile in cohesive soil, point bearing is generally neglected for individual pile action, since it is negligible as compared to frictional resistance.

→ The unit skin friction may be taken equal to the shear strength multiplied by a reduction factor $\alpha$ or $m$.

$q_{sf}$ = average skin friction along the length of pile
$= \alpha \bar{c}$ or $m\bar{c}$

$q_{eb} = CN_c$
Here $N_c = 9$ ⇒ $q_{eb} = 9c$

$$Q_u = \alpha \bar{c} A_s + 9 c A_b$$

$\alpha$ or $m$ = adhesion coeff or reduction factor, value of which depends upon type of clay & it value can be determined by pile load test.

| Consistency | N-value | Value of α | |
|---|---|---|---|
| | | Bored piles | Driven Piles |
| soft to very soft | < 4 | 0.7 | 1 |
| medium | 4-8 | 0.5 | 0.7 |
| stiff | 8-15 | 0.4 | 0.4 |
| stiff to hard | > 15 | 0.3 | 0.3 |

$\bar{c}$ = avg. undrained cohesion along the length of pile

$C$ = Average undrained cohesion of soil at tip of pile.

Note: → In absence of any depth both $\bar{c}$ & $c$ may be taken equal to "$q_u/2$"

→ The allowable load for pile is given by $\boxed{Q_a = \frac{Q_u}{F}}$

→ $F$ = FOS (2.5-3)

Note: → If difference FOS values are adopted for skin friction resistance & end bearing resistance then -

$$Q_a = \frac{\alpha \bar{c} A_s}{F_1} + \frac{9 c A_b}{F_2}$$

case (ii) FOR COHESIONLESS SOIL

$$Q_u = Q_{sf} + Q_{eb} \quad ; \quad Q_{sf} = q_{sf} A_s$$

$q_{sf} = \mu \times$ Earth pressure $\quad \therefore \mu = \tan \delta$

$\delta$ = frictional angle btwn piles & soil.

→ Earth pressure $= K\bar{\sigma}$

$K$ = earth pressure coeff. $\quad \therefore \bar{\sigma} = \left(\frac{0 + K\gamma L}{2}\right) = \frac{K\gamma L}{2}$

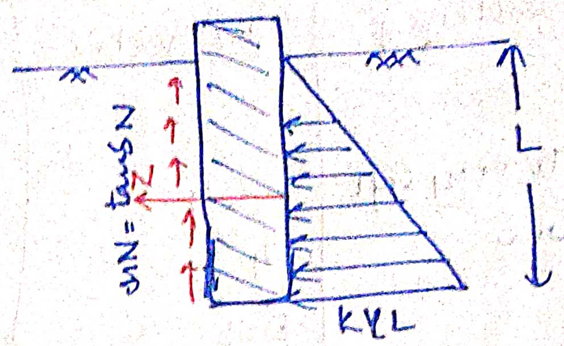
$\rightarrow q_{sf} = \frac{1}{2} K \gamma H \tan\delta$

$\rightarrow Q_{eb} = q_{eb} A_b$

$q_{eb} = q_u = \cancel{cN_c} + qN_q + 0.5 B\gamma N_\gamma$

$q_{eb} = qN_q + 0.5 B\gamma N_\gamma$

Since $qN_q \ggg 0.5 B\gamma N_\gamma \Rightarrow q_{eb} = qN_q = \gamma N_q L$




$$\boxed{Q_u = \frac{1}{2} K\gamma L \tan\delta\, A_s + \gamma N_q L \cdot A_b}$$

| Area | circular | square |
|---|---|---|
| $A_s$ | $\pi D L$ | $4BL$ |
| $A_b$ | $\pi D^2/4$ | $B^2$ |

| Pile Material | $\delta$ | Value of K: Loose sand | Dence sand |
|---|---|---|---|
| steel | 20 | 0.5 | 1 |
| concrete | $0.75\phi$ | 1 | 2 |
| timber | $0.87\phi$ | 1.5 | 4 |

## II Dynamic Load

→ when a pile hammer hits pile, the total driving energy is equal to the weight of hammer times the ht. of drop or stroke.

→ In addition to this, in case of double acting hammer, some energy is consumed by work done in penetrating the pile & by ~~work done~~ certain losses, morever in this case energy is also imparted by stream pressure.

→ following methods are available on this approach.

## (I) Engineering News Formula

As per this method allowable load on pile is given by —

$$\boxed{Q_a = \frac{WH}{F(S+C)}}$$

$Q_a$ = allowable load
$H$ = ht. of fall (cm)
$F$ = factor of safety = 6

$S$ = final set (penetration) per blow, usually taken as an avg. penetration (cm) per blow for last 5 blows of a drop hammer & 20 blows of steam hammer.

$C$ = Empirical formula
= 2.5 cm for drop hammer
= 0.25 cm for single & double acting hammers

Hence

(i) For drop hammer : $Q_a = \frac{WH}{6(S+2.5)}$

(for drop hammer is lifted manually & is allowed to fall freely)

(ii) For single acting steam hammer : $Q_a = \frac{WH}{6(S+0.75)}$

(single acting steam hammer is lifted by steam pressure & is allowed to fall freely)

(iii) For double acting steam hammer : $Q_a = \frac{(W+ap)H}{6(S+0.25)}$

$a$ = effective area of piston ($cm^2$)

$p$ = mean eff. stream pressure ($kg/cm^2$)

(double acting stream hammer is both lifted & dropped under stream pressure)

## II HILLEY'S FORMULA

→ This method is recommended by IS code also.

→

In this mtd allowable load carrying capacity of pile is given by

$$Q_u = \frac{W H \eta_H \eta_b}{S + C/2}$$

$Q_u$ = ultimate load on pile
$W$ = wt. of hammer (kg)

$H$ = height of drop on hammer (cm)
$S$ = penetration on set (cm) per blow
$C$ = total elastic compression (const) ∴ $C = C_1 + C_2 + C_3$
$C$ = temporary elastic compression of dolly & packing, pile, soil resp.

$\eta_h$ = efficiency of hammer
= 65% − 100% (65% for some double acting hammer & 100% for drop hammer)

$\eta_b$ = efficiency of hammer blow
(i.e ratio of energy after impact to the striking energy of ram)

" It signifies the loss of energy during impact."

$$\eta_b = \frac{W + e^2 P}{W + P} \quad \text{If } W > eP$$

or

$$\eta_b = \frac{W + e^2 P}{W + P} - \left\{\frac{W - eP}{W + P}\right\}^2 \quad \text{If } W < eP$$

$P$ = wt. of pile, helmet & follower
$e$ = coeff. of restitution
= It is variable from zero for a timber pile with poor condition of head to 0.5 for double action steam hammer for steel pile or RCC piles.

→ The allowable load is given by $\boxed{Q_a = \frac{Q_u}{F}}$ F = FOS (2−3)

Note: → Dynamic formula are best suited to coarse grained soil for which the shear str. is independent of rate of loading as they allow no development of excess pore ~~water~~ pressure around the pile during driving it sat or dry.

→ The great objection of any of the pile driving formula is uncertainity about the relationship the dynamic & static resistance of soil.

→ In case of submerged loose uniform fine sand impact of driving may cause liquifaction of soil, thus showing much less resistance than that which will occur under a static load.

→ On the same lines, a very dense saturated fine sand may show an increased driving resistance.

→ For clay dynamic formula are useless become the skin friction developed is very much less & remoulding also take place.

→ Dynamic formula gives no indication about propable future settlement.

→ It does not take into account the reduced bearing capacity of pile in group.

→ wt. of pile is inertia effect is neglected.

→ In Hilley's formula a number of const. are involved which are ~~digested~~ difficult to calculate.

## III Pile Load Test

→ This test can be preformed either on working pile which forms the foundation of the structure or on a test pile.

~~(Since after the test pile cannot be put into jack placed over a rigid circular or square plate which in turn is placed over the head of pile.~~

(Since after the test pile cannot be put into service/use it is a type of destructive test).

→ The test load is applied with the help of caliberated jack placed over a rigid circular or square plate which in turn is placed over the head of pile projecting above the ground level.

→ The reaction of ~~gate~~ jack is borne by a truss which may have gravity loading or truss can be anchored to the ground with the help of anchor piles.

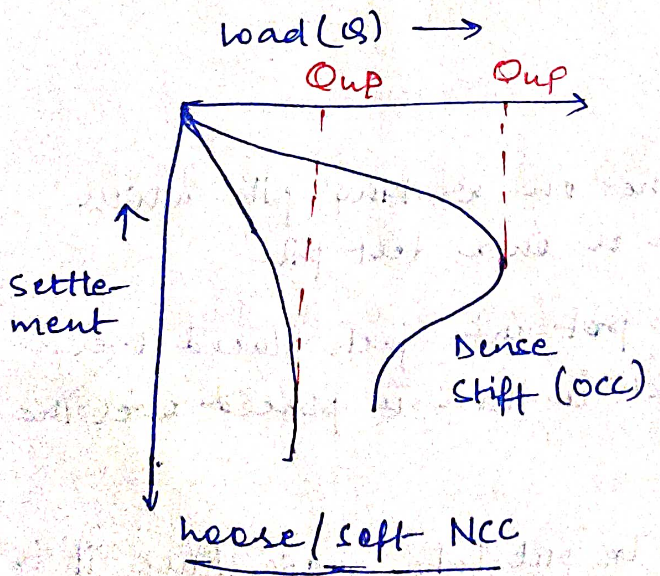
→ ~~The reaction of piles~~; The load is applied in equal increments of about $\frac{1}{5}$th of estimated allowable load.

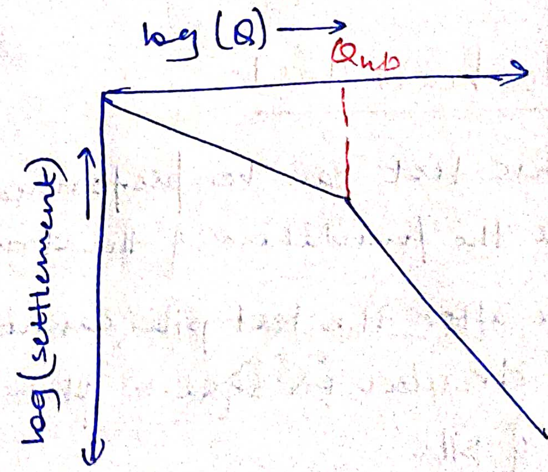
→ The settlement is recorded by the help of three dial gauges (placed at an angle of 120°)

→ each load increament is kept / applied upto an extent the rate of settlement reduces 0.02 mm / hour.

→ The test pile is load until ultimate load is reached, ordinarily the test load is increased upto 2.5 times the estimated allowable value or a load which causes the settlement equal to $\frac{1}{10}$th of pile diameter, which ever occur earlier.

→ The results are reported in terms to load-settlement curve either on Airthmetic or log scale.




→ In case ultimate local cannot be obtained from load settlement curve, the allowable load is taken as follows.

(i) $\frac{1}{2}$ to $\frac{1}{3}$ of ~~soil~~ final load which cause the settlement equal to 10% of pile diameter.

(ii) 2/3 of final load which causes a total settlement of 12mm.

(iii) 2/3 of final load which causes a net settlement of 6mm (residual settlement after the ~~settlement~~ removal of load.)

→ This method is more accurated & recommended by IS Code.

→ This method can also be used to find the allowable load on pile using settlement criteria.

→ In order to seperately find both end bearing resistance & skin friction resistance, cyclic plate load test is performed.

IV Corelation with SPT (N) value & CPT resistance value —

(a) Relation with SPT(N) value

(i) Acc. to Meyerhoff - for driven piles (Displacement ~~type~~ type)

$$Q_{up} = q_{eb}A_b + q_{sf}A_s$$

$$Q_{up} = 400NA_b + 2\bar{N}A_s$$

N = SPT no. at the base of pile

$\bar{N}$ = avg. SPT no. over the length of pile.

(ii) For driven pile (Non displacement type) (H-pile).

$$Q_{up} = Q_{eb} + \frac{1}{2}Q_{sf}$$

$$= 400NA_b + \frac{1}{2}(2\bar{N})A_s$$

(iii) For Bored piles

$$Q_{up} = \frac{1}{3}(Q_{up})_{Displacement}$$

$$Q_{up} = \frac{400}{3}NA_b + \frac{2}{3}\bar{N}A_s$$

(b) Relation with CPT resistance

(i) For driven pile (displacement type)

$$Q_{up} = Q_{eb} + Q_{sf}$$

$$Q_{up} = q_c A_b + \frac{\bar{q_c}}{2}A_s$$

(ii) For driven pile ( Non displacement type)

$$Q_{up} = Q_{cb} + \frac{1}{2} Q_{cf}$$

$$Q_{up} = q_c A_b + \frac{\bar{q_c}}{4} A_s$$

(iii) For Bored piles

$$Q_{up} = \frac{1}{3} (Q_{up})_{displacement}$$

$$Q_{up} = \frac{q_c}{3} A_b + \frac{\bar{q_c}}{6} A_s$$

$q_c$ = static cone resistance of soil base of pile (kg/cm$^2$)

$\bar{q_c}$ = Avg. static cone resistance of soil over the length of pile (kg/cm$^2$)

# Negative Skin Friction

→ Piles installed in freshly placed fills of soft compressive deposits are subjected to a downward drag, a consequence of consolidation of soil after pile are installed.

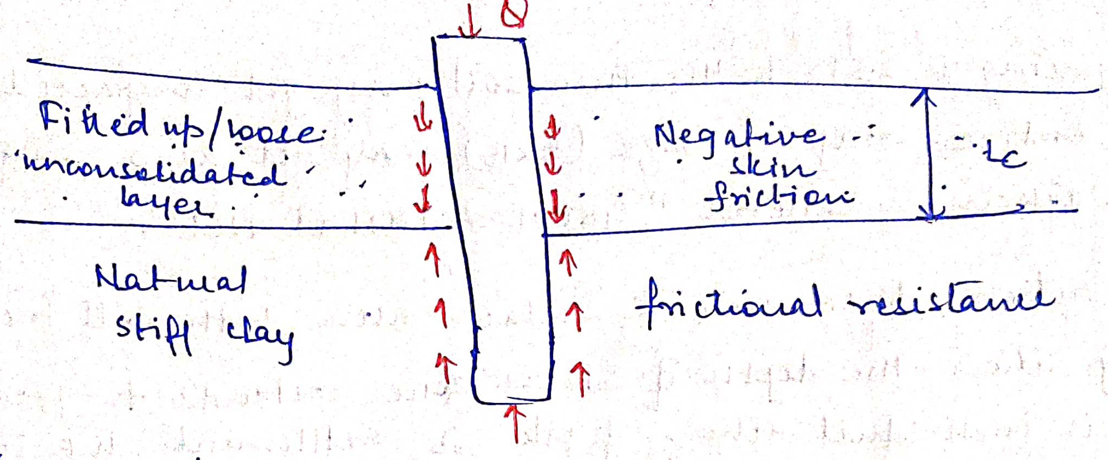
→ This downward drag on pile surface when the soil move down relative to the pile is termed as "NEGATIVE SKIN FRICTION".

→ This negative skin friction may also develop if the fill material is loose, sand deposit or due to lowering of the ground water table which increases the effective stress, thus causing consolidation of the soil.

→ It may also be observed due to placement of surcharge over the ground surface due to dynamic load on soil.

→ A small relative movement between the soil & the pile of order of about 10mm, may be sufficient for the full negative skin friction to materialistic.

→ In bearing piles where settlement of pile is negligible, negative skin friction becomes a pile capacity problem, but for piles in compressive soil where pile capacity is contributed by both point resistance & skin, the problem of negative skin friction should be considered as settlement problem.




→ The magnitude of negative skin friction for a ~~given~~ single pile in filled up soil deposit may be given as —

(a) For cohesive soil.

$$F_n = \alpha \bar{c} A_s$$

$$\boxed{F_n = \alpha \bar{c} P L_c}$$

∴ $\alpha$ = adhesion factor

$\bar{c}$ = Avg. cohesion of compressible layer

P = Perimeter of pile

$L_c$ = Length of pile in compressible layer

(b) For cohesionless soil.

$$F_n = \frac{1}{2} K \gamma L_c \tan\delta \cdot A_s \Rightarrow \frac{1}{2} K \gamma L_c \tan\delta \, P_c L_c$$

$$\boxed{F_n = \frac{1}{2} K_p \gamma L_c^2 \tan\delta}$$

K = lateral earth pressure coeff.

$\delta$ = angle of friction btw$^n$ the pile & soil $\left(\frac{1}{2}\phi - \frac{2}{3}\phi\right)$

# Group Action of Pile

→ If applied load is large & more no. of piles are used, then either piles will act individually or in the group depending upon the spacing in btw$^n$ the piles.

→ If c/c spacing is 2.5D to 4D, then soil may get compacted btw$^n$ piles & entire wedge of size (B×B) may act as a single pile such a action is called "GROUP ACTION OF PILE".

→ In group action, base area & surface area both will increase.

→ In group action the depth of stress zone extends to greater depth then is individual action of pile ∴ settlement due to consolidation in group action is more than settlement in individual action of pile.

→ The min no. of pile req. for group action is "3".

→ The pile group may be triangular, circular, rectangular or polygon (however square pile group is preferred).

→ Group action depends upon c/c spacing btw$^n$ piles.

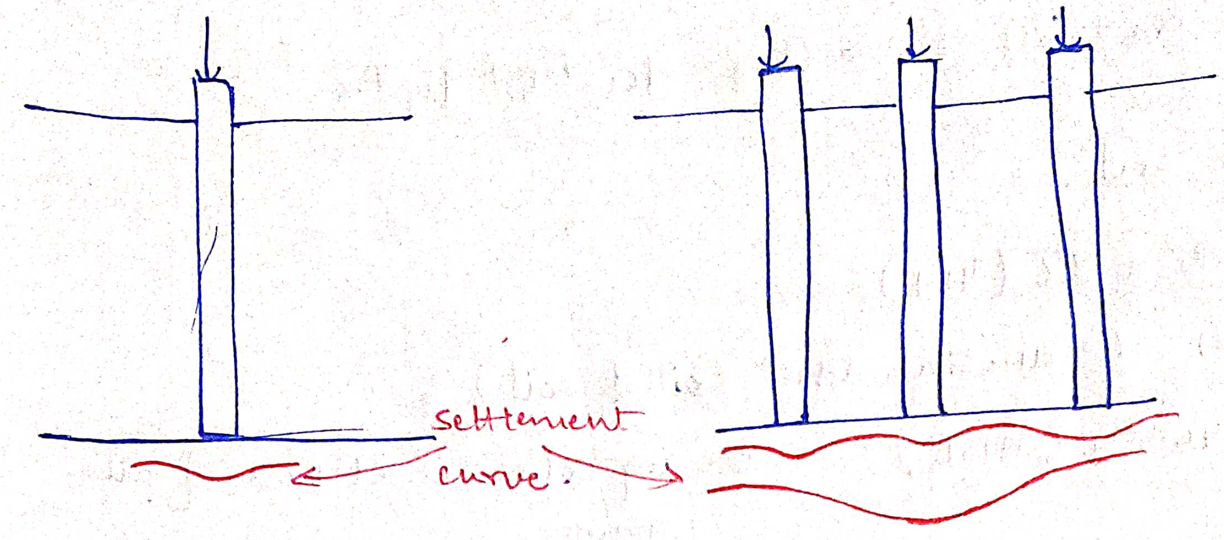
(i) for end bearing pile spacing is (2.5–3.5) D.

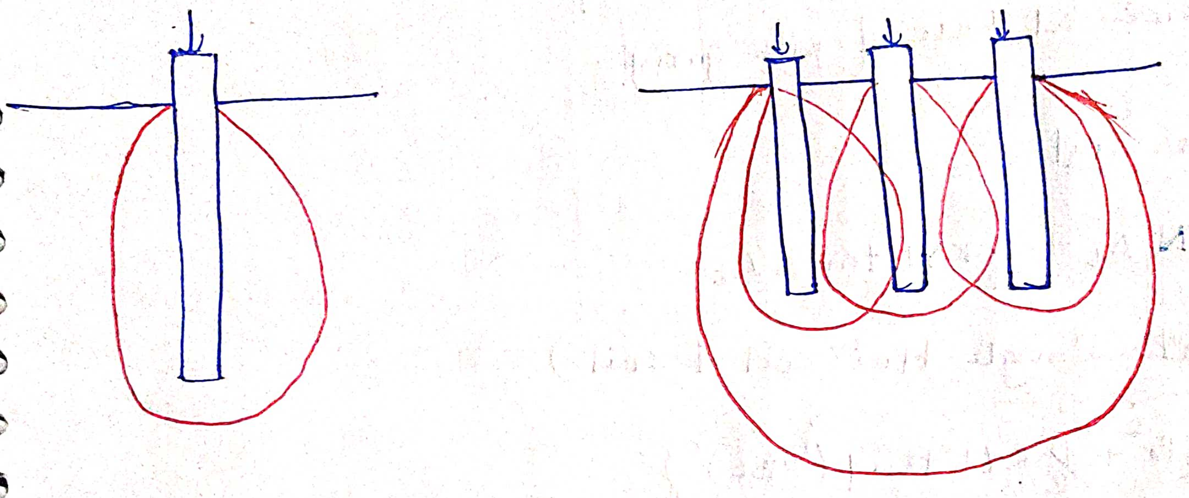
(ii) for friction pile " " (3–4) D.

(iii) for pile in loose sand is kept 2D.

→ If the spacing is governed as above ~~in sand~~ than load carrying capacity of pile group in sand comes to be greater than sum of load carrying capacity of all the piles, whereas in case of clay, it depends upon properties of soil & spacing.

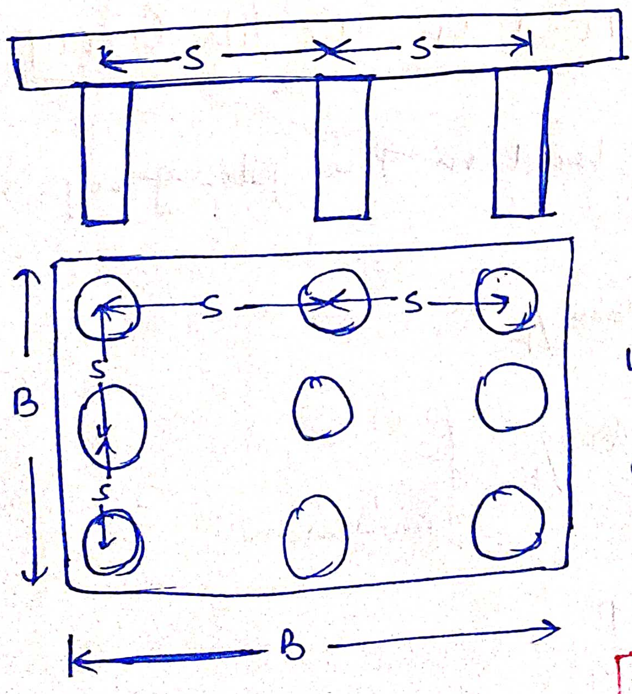
→ If piles are to be driven, then pile driving mechanism should start from centre & proceed outward i.e It means central pile is to driven first & processing piles are driven in radially outward direction as in this process resistance in pile driving will be less & hence it would be more economical.



X–X

Plan




B = size of pile group

3x3, $B = 2s + d$

4x4, $B = 3s + d$

5x5, $B = 4s + d$

$\therefore$ For mxm

$$\boxed{B = (m-1)s + d}$$

# Determination of Load Carrying Capacity of Pile Group.

$$Q_{ug} = Q_{eb} + Q_{sf} \Rightarrow Q_{ug} = q_{eb} A_b + q_{sf} A_s$$

(i) For cohesive soil

$$Q_{ug} = 9cB^2 + \alpha \bar{C}(4BL)$$

Here, $\alpha = 1$ (adhesion btw$^n$ soil & soil).

$$Q_{ug} = 9cB^2 + \bar{C}4BL$$

c = avg cohesion at base of pile group.

$\bar{c}$ = Avg. cohesion at base of pile group.

(ii) For cohesionless soil

$$Q_{ug} = qN_q A_b + \frac{1}{2} K\gamma L \tan\delta A_s$$

$\delta = \phi$ (friction angle btw$^n$ soil & soil)

$$Q_{ug} = \gamma L N_q B^2 + \frac{1}{2} K\gamma L \tan\phi (4BL)$$

# Allowable load / safe Load on the Pile Group

→ Allowable load / safety load on the pile group is given as minimum of following —

$$Q_{safe} = \left.\begin{matrix} Q_{ug}/F \\ n\,Q_{up}/F \end{matrix}\right\}_{\text{minimum}}$$

n = no. of piles in group

$Q_{up}$ = individual pile capacity

$Q_{ug}$ = Pile capacity of group

F = FOS

# Group Efficiency

→ It is defined as ratio of ultimate load carrying capacity of pile group to the sum of ultimate load carrying capacity of all the piles under individual action:

$$\eta_g = \frac{Q_{ug}}{n\,Q_{up}}$$ $n$ = no. of pile in the group

Note: → If $\eta_g > 1$, then $Q_{safe} = n\frac{Q_{up}}{F}$

∴ In design, $\eta_g$ should be $\geq 1$

→ Disturbance of soil during installation of pile & overlap of stresses btw^n adjacent piles may cause the group capacity to become less than the sum of individual capacities i.e. $\eta_g < 1$

→ Generally for smaller spacings between the piles $\eta < 1$, for larger spacing, the effect of pile interaction reduces & $\eta_g$ approaches unity.

→ In driven piles where the soil around the piles get densified, as in loose to medium sand, $\eta_g$ may be more than 1.

→ In this case group tends to behave like a block or like equivalent single pile.

→ The $\eta_g$ depends mainly on the spacing btw^n pile, type of soil in which pile is installed & manner of pile installation (ie, driven, bored).

→ $\eta_g$ can also be computed empirically as using:

(a) Converse Labarre Formula

$$\eta_g = 1 - \frac{\theta}{90}\left(\frac{(n-1)m + (m-1)n}{mn}\right)$$

$m$ = no. of rows

$n$ = no. of piles in a row

$d$ = dia of piles

$s$ = spacing of piles

∴ $\theta = \tan^{-1}(d/s)$ (degrees)

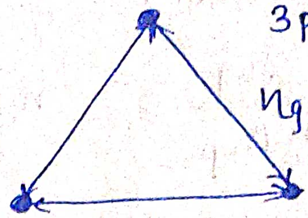
## (b) Feld's – Rule

According to this rule, the value of each pile is reduced by $1/16$th on account of effect of nearest pile in each diagonal or straight row of which the pile in question is a member.

eg:→

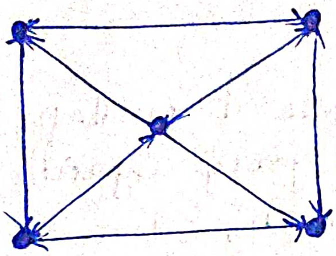

2 piles @ $15/16$

$$\eta_g = \frac{2 \times 15/16 \times 100}{2} = 93.75\%$$



3 piles @ $14/16$

$$\eta_g = \frac{3 \times 14/16 \times 100}{3} = 87.5\%$$



4 pile @ $13/16$ + 1 pile @ $12/16$

$$\eta_g = \frac{4 \times 13/16 + 1 \times 12/16 \times 100}{5} = 80\%$$

## # Group Settlement Ratio

→ For estimating the settlement of pile group in sand pile load test is used {interpolation of load settlement curve is done}.

→ Settlement of no. of pile groups, consisting of driven piles, with the settlement of individual pile in sand, for the same load per pile after being compared can be expressed in terms settlement of an individual pile & is termed as "Group Settlement Ratio" given by —

$$\frac{S_g}{S_i} = \left(\frac{4B + 2.7}{B + 3.6}\right)^2$$

B = width of pile group (m)
$S_g$ = settlement of pile group.
$S_i$ = settlement of individual pile.

Note :→ (The above ratio is for same load of per pile)

→ $S_g$ in sand varies in the range of 1–16, irrespective of width of pile.

# Determination of settlement of pile group in Clay.

→ The settlement of a pile group in clay cannot be estimated from the data of load test on a single pile, because of time effect, the effect of remoulding of soil due to pile driving & the scale effect, which are different for the single test pile & group of pile.

→ In this case approach used for settlement of pile group is "Equivalent Raft Approach".

→ Depending the sub soil conditions, several assumptions are being made to identify the location of equivalent raft over which the pile are assumed to transfer the vertical load acting on them.

Case(i) When Pile are End Bearing

→ It means the soil below the base of pile is stiff / dense where as above the base of pile is loose / soft.

→ In this case skin friction is negligible & equivalent ray may be assumed at the base of pile group.

→ In this settlement is computed as —

(i) Assume equivalent raft at base of pile group.

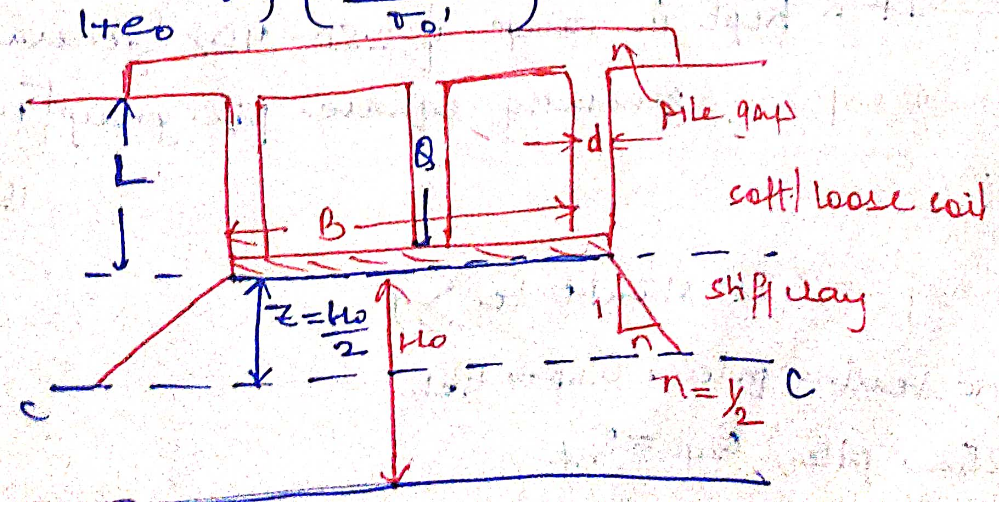
(ii) Identify thickness of compressible soil ($H_0$)

(iii) $\sigma_0'$ at cc

(iv) Increase in effective stress $\Delta\sigma'$

$$\Delta\sigma' = \frac{Q}{(B+2nZ)^2}$$

(v) $$\Delta H = \frac{H_0 C_c}{1+e_0} \log\left(\frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}\right)$$

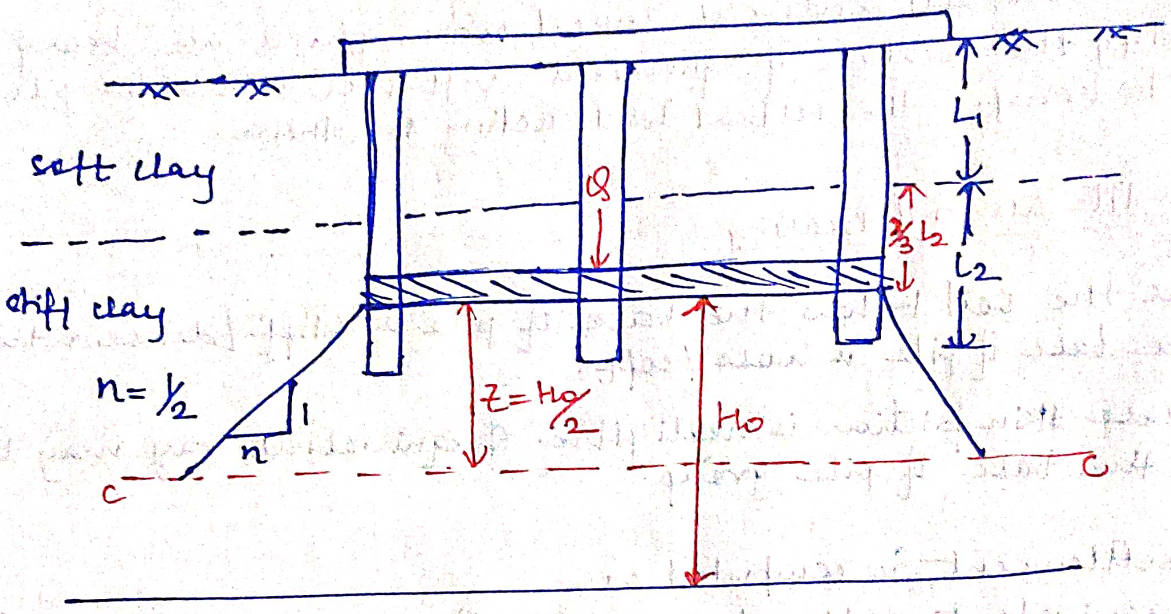


Case (ii) when piles are driven through uniform clay deposit & pile group act as a friction pile group.

→ Let the top layer is soft of length $L_1$ & bottom layer is stiff and is embedded upto the length is $L_2$.

→ In this case end bearing resistance & skin friction resistance both will be developed.

→ The equivalent raft in this is assumed to be present at $\frac{2}{3}L_2$ from base of top layer & remaining procedure is same.




# Guidelines to Design a pile Group

(i) Length :→ for friction pile group length may be 10–20 cm whereas for end bearing pile group, it is equal to depth of hard strata.

(ii) Dia :→ It is kept in range of 0.3 – 0.9 m

(iii) Spacing :→ It is kept in range of 2.5D – 4D, (generated)

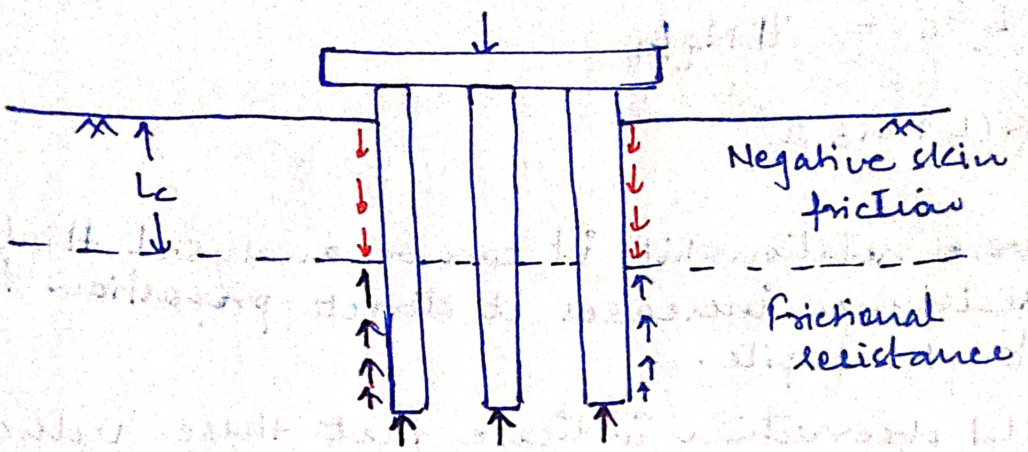
(iv) No. of piles in group :→ generally square pile group [(3x3), (4x4) & (5x5)] is preferred.

(v) group efficiency :→ It should be $\geq 1$

Note :→ In end bearing pile, $Q_{up} = Q_{eb}$

In friction pile, $Q_{up} = Q_{sf}$

# Negative Skin friction in Pile group




→ when pile group passes through a soft, unconsolidated stratum the magnitude of negative skin friction on the group $F_{ng}$ is given as higher of the value obtained through following relationship.

$$F_{ng} = n F_n \quad \text{——(i)}$$

$$F_{ng} = \alpha \bar{C}_u L_c P_g + \gamma L_c A_g \quad \therefore \alpha = 1 \text{ (adhesion b/w soil \& soil)}$$

$$F_{ng} = \bar{C}_u L_c P_g + \gamma L_c A_g \quad \text{——(ii)}$$

$n$ = no. of piles in group

$P_g$ = perimeter of group

$\gamma$ = unit wt. of soil with pile group upto depth $L_c$

$A_g$ = area of pile group within the perimeter $P_g$.

Note : → The second relationship is computed on the basis block shear failure along the perimeter of pile group which includes the vol. of soil enclosed in the group ($A L_c$).

→ The effect of negative skin friction on the FOS wrt. to ultimate load capacity of pile or a pile group can be considered as —

$$FOS = \frac{\text{ultimate load capacity of single pile/pile group}}{\text{working load + negative skin friction load.}}$$

Note: → $Q_u = Q_{eb} + Q_{sf}$ (for cohesionless soil)

$Q_{eb} = qN_qA_b = \gamma LN_qA_b$

$Q_{sf} = \frac{1}{2}K\gamma L \tan\delta A_s$

→ From the above relationship it can be analysed that the unit point resistance increases it direct proportion to the embedded length of pile.

→ However, field observation indicate that these values increases only upto a limited depth; beyond which it remain const.

→ This depth is called the "critical depth of pile".

→ This phenomenon is due to "arching action" in the granular soil.

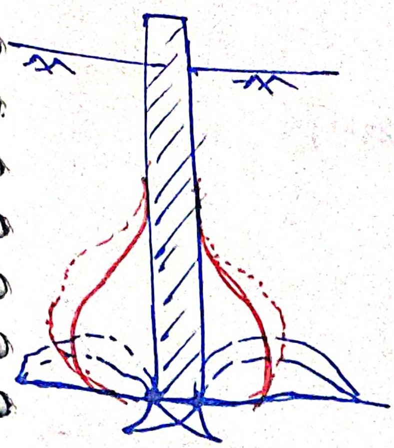
→ The critical depth depends on the ~~edge~~ angle of shearing resistance of soil & size of pile.

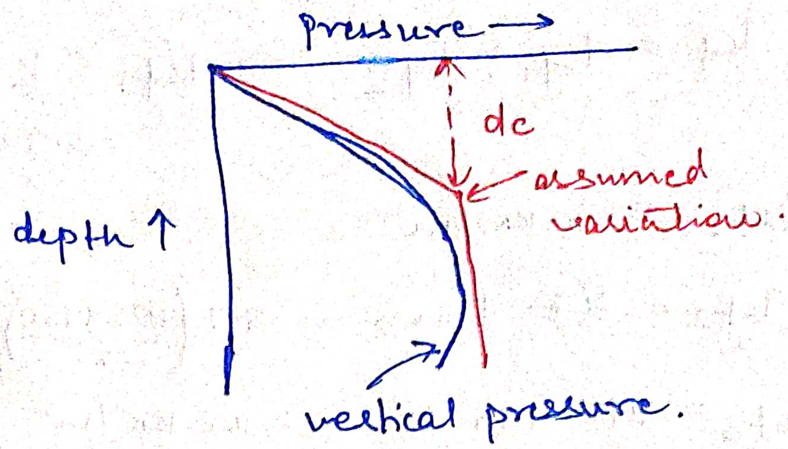
→ Its value may vary from about 15D in loose to medium sand to 20D in dense sand where D is pile diameter or width.

→ It is also recommended that max value of unit point resistance $q_{eb}$ be 11000 $KN/m^2$ in normal silicate sand or 5000 $KN/m^2$ for calcerous sand.

→ "Arching occurs when there is a difference of the stiffness btw^n the install str & the surrounding soil.

→ If the str is stiffer than the soil, then load arches onto the str otherwise, if the str. is less stiffer than soil, then load arches away from the str.






→ From the above relationship it can be analyzed that $q_{sf}$ increases with depth, but in actual it does so only upto critical depth which varies from 15-20 times the pile diameter

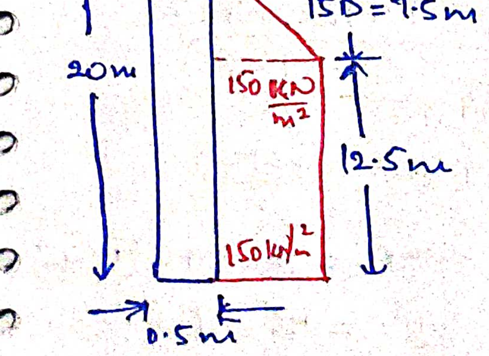
→ Below the critical depth, the value of $\bar{\sigma}$ & the value of $q_{sf}$ remains const.

→ The maximum value of $q_{sf}$ should be limited to 100 kN/m² for straight piles in normal silica sand & upto 20 kN/m² for calcerous sand.

Q. A single vertical friction pile of ~~foundation~~ diameter 500mm & length 20m is subjected to a vertical compressive load. The pile is embedded in a homogenous sandy strata where angle of internal friction $\phi = 30°$, $\gamma = 20\,kN/m^3$ & angle of wall friction $\delta = \frac{2}{3}\phi$. Considering the coeff. of lateral earth pressure $K = 2.7$, $N_q = 25$, find the ultimate bearing capacity of pile.

check for arching effect

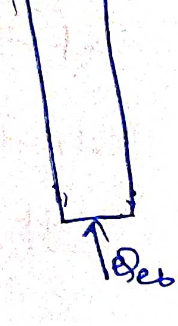
$$L/D = \frac{20}{0.5} = 40 > 15$$




$$Q_{eb} = q_{eb} A_b \Rightarrow q_{eb} = q N_q$$

$$\Rightarrow q_{eb} = \gamma(15D) N_q \Rightarrow 150 \times 25$$

$$= 3750 < 11000\ kN/m^2$$




$$Q_{eb} = q_{eb} A_b = 3750 \times \frac{\pi}{4}(0.5)^2$$

$\Rightarrow \boxed{Q_{eb} = 736\,kN}$

$q_{sf} = q_{sf_1} + q_{sf_2} \Rightarrow q_{sf_1} = K\bar{\sigma}\tan\delta$

$= 2.7 \times \frac{150}{2} \tan\left(\frac{2}{3}\times 30^\circ\right)$

$q_{sf_1} = 73.70\ kN/m^2$

$\Rightarrow q_{sf_2} = K\bar{\sigma}\tan\delta = 2.7\left(\frac{150+150}{2}\right)\tan\left(\frac{2}{3}\times 30^\circ\right)$

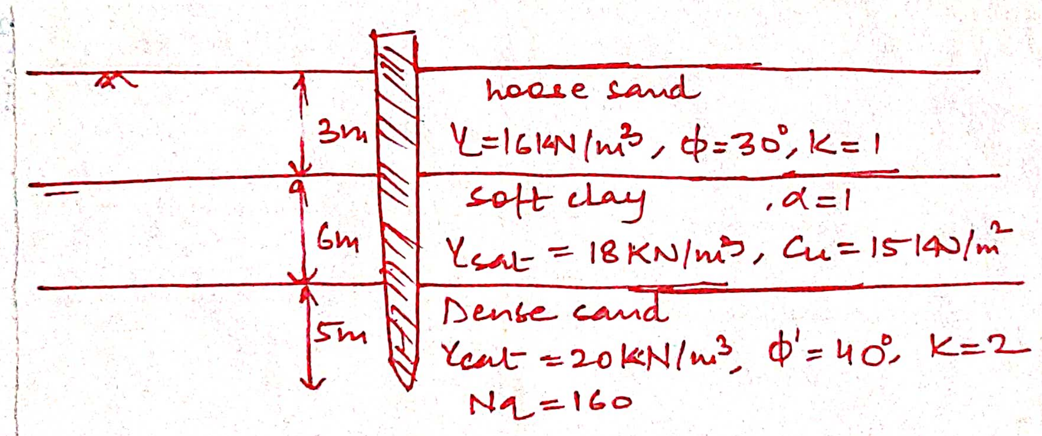
$q_{sf_2} = 147.4\ kN/m^2 > 100\ kN/m^2$

$q_{sf} = 73.70 \times \pi(0.5)\,15 \times 0.5 + 100 \times \pi(0.5) \times 7.5 \Rightarrow \boxed{q_{sf} = \cancel{2831.5}\ 2831.5\ kN}$

$Q_u = Q_{eb} + Q_{sf} = 736 + 2831.5$

$\boxed{Q_u = 3567.7\ kN.}$

Q. Determine the allowable pile load capacity of the 40cm dia, driven concrete pile, $F = 2.5$




(i) Loose Sand

$L/D = \frac{3}{0.4} = 7.5 < 15$, here no arching effect $\Rightarrow Q_{sf_1} = q_{sf_1} A_{s_1}$

$q_{sf_1} = K\bar{\sigma}\tan\delta$

$q_{sf_1} = 1 \times \frac{16\times 3}{2}\tan\left(\frac{3}{4}\times 30\right) = 9.9\ kN/m^2 < 100\ kN/m^2$

$Q_{sf_1} = 9.9 \times \pi \times 0.4 \times 3 = 37.47\ kN \Rightarrow Q_{sf_1} = 9.9\pi \times 0.4 \times 3$

$\boxed{Q_{sf_1} = 37.47\ KN}$

(ii) Soft Clay

$\hookrightarrow Q_{sf_2} = \alpha C_u A_{s_2} = 1 \times 15 \times \pi(0.4)\,6 \Rightarrow 113.09\ kN = Q_{sf_2}$

(iii) Dense Sand

$L/D = \frac{5}{0.4} = 12.5 < 20$, no arching effect $\therefore Q_{sf_3} = q_{sf_3} A_{s_3}$

$q_{sf_3} = K\bar{\sigma}\tan\delta,$

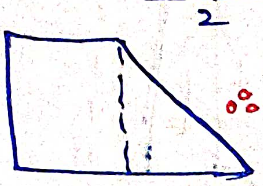
$\rightarrow \sigma'_z = 16\times 3 + 6(18-10)$

$= 96\ kN/m^2$

$\bar{\sigma}_b' = 16\times3 + 6(18-10) + 5(10)$

$= 146\ kN/m^2$

$\bar{\sigma}' = \frac{96+146}{2} = 121\ kN/m^2$



$\therefore Q_{eb} = 11000 \times \frac{\pi}{4}(0.4)^2$

$= 1382\ kN$

$q_{sf_1} = 2\times121\times\tan(\frac{3}{4}\times40°) = 139.71\ kN/m^2 > 100\ kN/m^2$

$\Rightarrow Q_{sf_3} = 100\times\pi(0.4)\times5 = 628.31\ kN$

$\boxed{Q_{eb} = q_{eb} A_b}$

$\therefore q_{eb} = q N_q = \sigma_f' N_q \Rightarrow 146\times10 = 23360\ kN/m^2 > 11000\ kN/m^2$

$\therefore Q_u = (Q_{sf_1} + Q_{sf_2} + Q_{sf_3}) + Q_{eb}$

$\boxed{Q_u = 2161\ kN}$

$\therefore Q_a = Q_u/F = \frac{2161}{2.5}$

$\boxed{Q_a = 864.5\ kN}$

# Piles Subjected to Uplift Loads

→ Str. such as tall towers, silos, chimneys, off shore platforms & dry are usually provided with pile foundation to resist the large uplift pressure (due to water) & overturning moments.

→ Piles used for this purpose are called as tension or uplift piles.

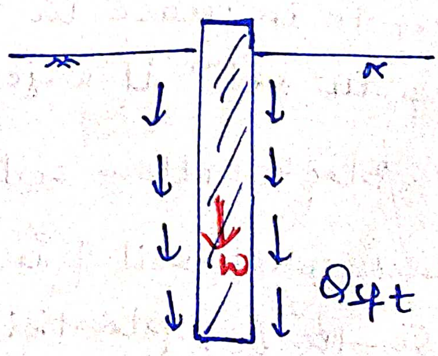
→ The uplift capacity of pile is calculated in a manner similar to pile subjected to compressive load.

→ uplift piles are invariably provided with an enlarged area at ~~that~~ the base in-form of bulb or ball.

→ Pile develop resistance to pull-out only from the skin friction develop along the embedded length & point bearing is not included but the wt. of pile is included in uplift resistance.

case (i) PILE IN CLAY

$Q_{ut} = q_{sft} A_s + w_p$



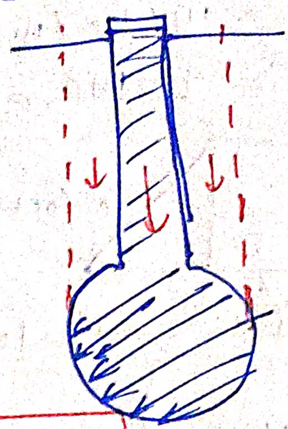

$q_{sft}$ = unit skin friction in tension

$A_s$ = embedded area of pile shaft

$w_p$ = wt. of pile.

$q_{sft}$ is taken equal to $q_{sf}$ in compression for undrained conditions.

→ when the base of pile is enlarged in form of bulb or a bell, the smaller of two values is considered given by —



$$Q_{ut} = C_u \bar{A_s} K + W_s + W_p \quad \text{—— (i)}$$

or

$$Q_{ut} = 2.25\pi (D_b^2 - D^2) C_u + W_p \quad \text{—— (ii)}$$

$\bar{A_s}$ = surface area of vertical cylinder above the base.

$D_b$ = diameter of base, $D$ = Diameter of pile shaft

$K$ = coeff.

$W_s$ = wt. of soil included in the region b/w pile shaft and cylinder above the base.

| Type of soil | k |
|---|---|
| soft clay | 1–1.25 |
| medium clay | 0.7 |
| stiff clay | 0.5 |
| fissured clay | 0.25 |

Note :→ The (i) eq is on basis of failure assumed through full mobilisation of friction resistance along the cylindrical surface above the base diameter and eq (ii) is based on bearing capacity failure of base.

# Foundation On Expansive Soil

→ Expansive soils or swelling soils are those which have the tendency to increase in volm when water is available & to decrease in volm if water is removed.

→ Eg: Black Cotton soil, Bentonite soil

→ Black cotton soil of india have liquid limit ($W_L$) ranging from 50–100%, plasticity index ($I_P$) ranging from 20–65% & shrinkage limit ($W_S$) from 9–14%.

→ The minus 2μ fraction in soil varies from 40-75%, The soil is very hard in dry state and posses high shearing strength which get reduced appreciably with addition of water.

→ In general expansive soil have $w_L \uparrow$, $w_s \downarrow$, $I_p \uparrow$, $I_c \uparrow$

→ The swelling behaviour of soil would depend largely on the type of clay mineral present in the soils and proportions of these minerals.

→ In order to determine this behaviour following techniques are available —

(i) Differential Thermal Analysis (DTA)
(ii) X-ray ~~differential Method~~ diffraction method.
(iii) Electron - Microscopy.

→ The DTA method is based on the fact that certain characteristic reaction takes place at specific temp. for different minerals when these are heated to high temp, resulting in loss or gas in heat.

→ Different minerals will diff regular patterns of crystalline str will diffract x-ray to yield diff. x-ray diffraction pattern.

→ In electron-microscopy the soil is observed under polarised light in a electron microscope to identify the characteristic strain of a mineral.

→ From engineering point of view following test are performed to find the swelling potential of soil.

(1) Free swell test

→ This is performed by poring slowly 10 $cm^3$ of dry soil passing through 425 micron sieve, into a 100 cc graduated cylinder filled with water.

→ The volm of swelled soil is read after 24 hrs from graduation of cylinder to give free swell value.

$$\text{Free swell (\%)} = \frac{\text{final vol}^m - \text{Initial Vol}^m}{\text{Initial Vol}^m} \times 100$$

→ Bentonite soil, containing montmorillonite, have free swell value of 1200 - 2000%.

→ Kaolinite has free swell value of 80% & illite have swell value of 30 - 80%

(ii) Differential Free Swell Test

→ In this test two sample of dried soil weighing 10gm each, passing through 425 μ sieve, are taken

→ One is put in a 50cc graduated glass cylinder containing kerosene oil (a non polar liquid).

→ The other sample is put in a similar cylinder containing distilled water.

→ Both the sample are left undisturbed for 24 hrs and vol^m is noted.

→ The differential free swell (DFS) is expressed

$$DFS = \frac{\text{soil volume in water} - \text{soil } \cancel{\text{water}} \text{ vol}^m \text{ in kerosene}}{\text{soil vol}^m \text{ in kerosene}} \times 100$$

→ Degree of expansiveness & possible damage on lightly loaded str. may be referred as follows —

| Degree of expansiveness | DFS (%) |
| --- | --- |
| Low | <20 |
| Moderate | 20 - 35 |
| High | 35 - 50 |
| Very high | >50 |

Note :→ Expansive/Swelling potential of soil can also be related as follows —

| Colloid content (%) | $I_p$ (%) | $W_s$ (%) | Probable expansion (%) | Degree of expansion |
|---|---|---|---|---|
| < 15 | < 18 | > 15 | < 10 | Low |
| 13–23 | 15–28 | 10–16 | 10–20 | medium |
| 20–31 | 25–41 | 7–12 | 20–30 | high |
| > 28 | > 35 | < 11 | > 30 | very high. |

| Swelling potential | $I_p$ (%) |
|---|---|
| Low | 0 – 15 |
| Medium | 10 – 35 |
| High | 20 – 35 |
| Very high | > 35 |

| $W_s$ (%) | Linear Shrinkage | Degree of expansion |
|---|---|---|
| < 10 | > 8 | critical |
| 10 – 12 | 5 – 8 | marginal |
| > 12 | 0 – 5 | non-critical |

## (iii) Swelling Pressure Test

→ When an expansive soil imbibe water from outside, pressure builds up inside the soil.

→ If free swelling of soil is restrained by placement of str. over the soil, this pressure is called "Swelling Pressure" is exerted by soil on the overlying str.

→ The force req. to prevent expansion in the soil is function of time.

→ A swelling pressure of ~~20 kN/m²~~ less than $20\ kN/m^2$ may not be regarded as of much consequence.

# Field Condition that favours Swelling

→ A soil may have high swelling potential but still may or may not swell in actual practice.

→ This would depend on many factors, the most significant of them is being the diff b/w the field moisture content & equilibrium moisture content that will be materialised after construction.

If FMC < EMC → expansion of soil results.

FMC > EMC → shrinkage of soil results.

→ A fill which has been compacted to a greater degree or an over consolidated deposit will have more tendency to swell, given access to water.

→ If load that is placed on swelling soil is more, the swelling of soil is inhibited & vice-versa.

# Design of foundation on Expansive Soil

→ The rational methods of foundation design which are being used to reduce or prevent the effect of swelling can be grouped into 3 categories.

i) isolating the str from swelling soil

ii) Designing a str to withstand the effect of swelling.

iii) Preventing the swelling.

i) In first approach either Belled piers or under-reamed piles are provided.

→ The principle of under-reamed piles is ~~placed~~ based on fact that to involve the load transfer to pile at a depth beyond the zone of seasonal variation

→ The depth of black cotton soil is approx 3.5 – 4 m in India.

→ For other expansive soil, the pile is taken down to atleast 0.6 m into the non-expansive layer under lying the swelling soil.

→ The under-reamed piles are bored cast-in situ piles with their lower portion enlarged or reamed in the form of bulb.

→ The spacing of pile is in range of 1.5–3m

→ Dia of these piles varies between 200–500 mm & ratio of diameter of enlarged base or bulb to that of shaft ($D_u/D$) is in range of 2–3.

→ In case of multi-reamed piles is having more than one bulb, the topmost bulb should be at a min depth of 2 the bulb diameter.

→ The c/c distance btw^n the bulb is (1.25–1.5) $D_u$.

→ A FOS of 2.5 – 3 is used to calculate safe load.

→ For clay soil the ultimate load carrying capacity of under-reamed pile is given by —

$$Q_u = A_p N_c C_p + A_a N_c C_a' + C_a' A_s' + \alpha C_a A_s$$

$Q_u$ = ultimate bearing capacity of pile.

$A_p$ = cross-sxn area of pile stem at toe level $(\pi D^2/4)$

$N_c$ = bearing capacity factor (9)

$A_a$ = Area of soil surrounding the stem below the bulb.

$\frac{\pi}{4}(D_u^2 - D^2)$

$C_a'$ = avg. cohesion of soil around under-reamed bulb.

$\alpha$ = reduction factor.

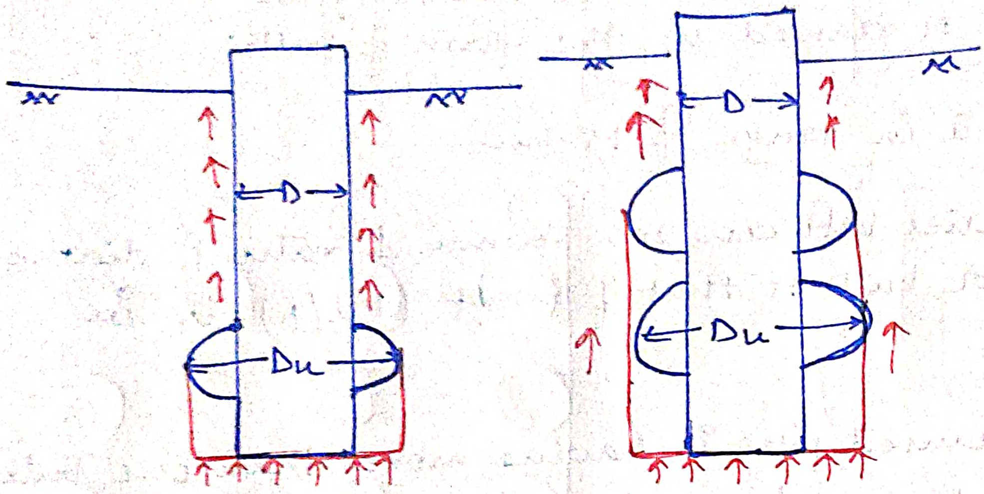
$C_p$ = avg. cohesion ~~at base~~ of soil around the toe

$C_a$ = Avg. adhesion of soil along pile stem

$A_s^*$ = Surface area of stem

$A_s'$ = Surface area of cylinder circumscribing the bulb

Note : → If pile has only one bulb, third term will not be used.




(ii) Designing a structure which is ~~strong~~ & rigid enough to withstand the effect of swelling may prove to be highly uneconomical, except ~~the~~ in case of very small str where even if the loads are supported by the central area or peripherical area much smaller than the plan area, the bearing pressure are within limits.

(iii) Swelling can often be controlled but cannot be eliminated by providing an impervious a apron around the structure.

→ By providing the apron, the moisture gradient btw<sup></sup>n the centre of the str & its edges is minimised hence the differential swelling is controlled.

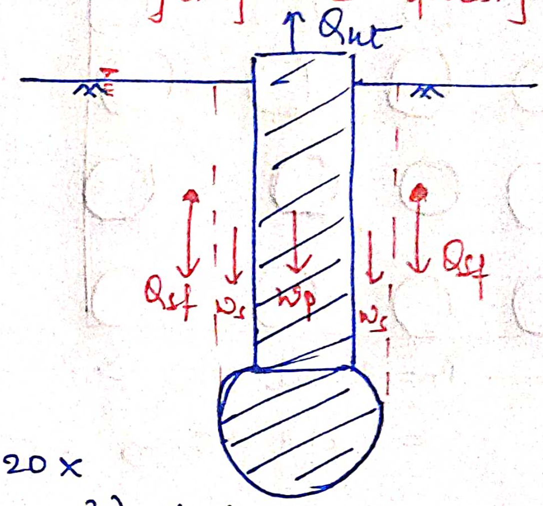
→ Elimination of possible swelling can be achieved by –

(a) pre-wetting the ground to a moisture content equal to the equilibrium moisture content.

(b) making downward load large to exceed swelling pressure.

(c) by Chemical stabilisation.

Q. A pile 45cm in dia & 20m long, is driven into a clay soil with $C_u = 40 kN/m^3$, $\phi_u = 0$, $\gamma_{sat} = 19 kN/m^3$ ground water is almost at ground level.
If pile has an enlarged base of diameter 1.25m, determine the uplift capacity of pile undrained condition. Submerged unit wt of pile is 40 kN.
Also consider skin friction resistance while analysing the capacity by bearing failure.




$$Q_{ut} = C_u A_s K + W_s + W_p$$

Assume $K = 0.75$

$$Q_{ut} = 40 \times \pi(1.25) \times 20 \times 0.75 + \overset{\gamma'}{(19-10)} \times 20 \times \frac{\pi}{4}(1.25^2 - 0.45^2) + 40$$

$$Q_{ut} = 2588.45 kN$$

$$Q_{ut} = 2.25\pi(D_b^2 - D^2) C_u + W_p + \alpha C_u A_s$$

$$= 2.25 \times 3.14 \times (1.25^2 - 0.45^2) \times 40 + 40 + 0.5 \times 40 \times \pi \times (\cancel{1.25}\ 0.45) \times 20$$

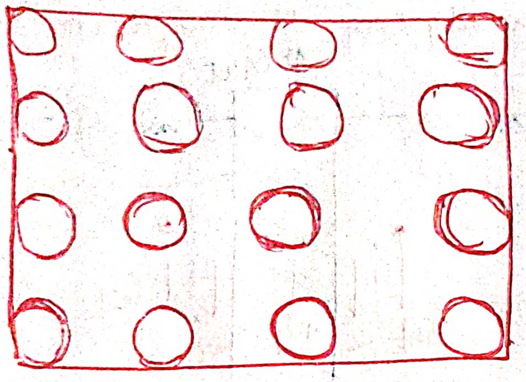
$Q_{ut} = 990$ kN

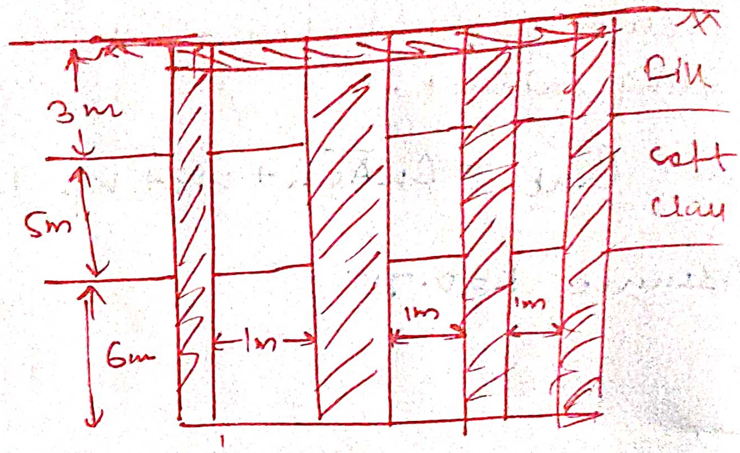
$$Q_{ut} = (2588.45, 990) kN_{min} \Rightarrow Q_{ut} = 990 kN$$

Q. A group of 16 piles of dia (500mm), length 14m, c/c spacing of 1m) arranged in square pattern passes through a recent fill (thickness 3m) overlying a soft clay deposit (thickness 5m) which is consolidating under the fill load & rest is stiff clay strata

All the strata are saturated & soil properties are given below.
Estimate ultimate load carrying capacity of pile group.

| Type of soil | unit wt (kN/m3) | Strength Parameter $C_u$ (kPa) | $\phi_u$ | Adhesion parameter |
|---|---|---|---|---|
| Fill | 16 | 50 | 0 | 0.6 |
| Soft clay | 17 | 20 | 0 | 0.4 |
| Stiff clay | 21 | 70 | 0 | 0.45 |






$B = 3s + d$

$= 3\times1 + 0.5 \Rightarrow 3.5m$

(i) Pile acting individually

$Q_{ug} = nQ_{up}$

$= n[Q_{eb} + Q_{sf}]$

$= n[q_{eb}A_b + q_{sf}A_s]$

$= 16[C_u N_c A_b + \Sigma \alpha_i C_{ui} (\pi D) L_i]$

$= 16\left[70\times9\times\frac{\pi}{4}\times(0.5)^2 + (\pi\times0.5)\times[0.6\times50\times3 + 0.4\times20\times5 + 0.45\times70\times6]\right]$

$Q_{ug} = 9996.54$ kN

(ii) Pile acting in group

$Q_{ug} = Q_{eb} + Q_{sf}$

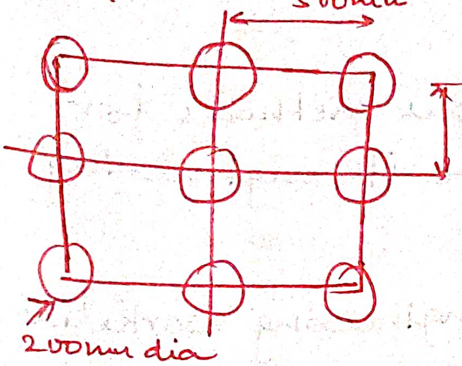
$Q_{ug} = q_{eb} A_b + q_{sf} A_s \Rightarrow C_u N_c A_b + \sum \alpha C_{ui} (4B) L_i$

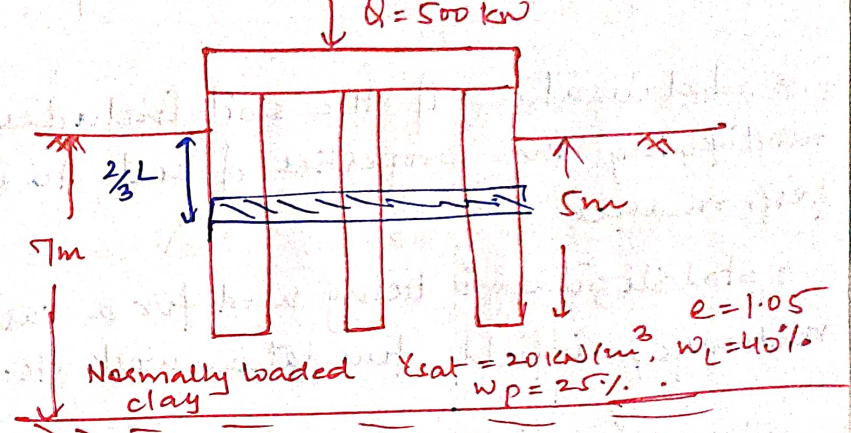
$\Rightarrow Q_{ug} = 70 \times 9 \times (3.5)^2 + 1 \times 4 \times 3.5 \left[50 \times 3 + 20 \times 5 + 70 \times 6\right]$

$Q_{ug} = 17097.5 \text{ kN}$

$Q_{ug} = (9996.54, 17097.5)_{min} \text{ kN} \Rightarrow \boxed{Q_{ug} = 9996.54 \text{ kN}}$

Q. Calculate the settlement of pile group for condition indicated as follows.




Sol: Location of equivalent raft from ground surface $= \frac{2}{3}L = \frac{2}{3} \times 5 = 3.34$ m

Thickness of compressible layer $H_0 = 7 - 3.34 = 3.67$ m

Location of centre of compressible layer from ground surface $= 3.34 + \frac{3.67}{2} = 5.17$ m

$\sigma_0' = \gamma' D = (20-10)\,5.17 = 51.7 \text{ kN/m}^2$

$B = 2S + d = 2 \times 0.5 + 0.2 = 1.2 \text{m}$

$B' = B + 2nZ = 1.2 + \cancel{2} \times \frac{1}{\cancel{2}} \times \frac{3.67}{2} = 3 \text{m}$

$\Delta\sigma = \frac{500}{3^2} = 55.5 \text{ kN/m}^2$

$\Delta H = \frac{C_c H_0}{1+e} \log_{10}\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)$

$C_c = 0.009(w_L - 10)$
$= 0.009(40 - 10)$
$= 0.27$

$$\Delta H = \frac{0.27 \times 3.67}{1+1.05} \log_{10}\left(\frac{51.7+55.5}{51.7}\right)$$

$\Delta H = 153$ mm

# Stabilisation of soils

→ Stabilisation of soil includes various methods for modifying the prop. of soil to improve its engineering performance.

→ Stabilisation is being used for the variety of engineering works like roadways, airfields, foundation works etc.

→ The main objective in this case is to increase the strength or stability of soil to make it more cost effective.

→ Methods of stabilisation is grouped into two

(a) Modification or improvement of a soil property of the existing soil (drainage compaction)

(b) Modification of property with the help of admixtures (mechanical, cement, lime, bitumen, chemical, electrical, thermal stabilisation)

## i. Mechanical stabilisation:

→ It involves 2 operations

a. changing the composition of the soil by addition or removal of certain constituents

b. Densification or compaction

→ This is been carried out largely in construction of cheap roads using locally available material.

## ii. Cement stabilisation:

→ The soil stabilised with cement is known as soil cement.

→ The cementing action is due to the result of chemical reaction of cement with silicious soil & by binding action of the individual particles through the cement (in CG soil)

→ In fine grained soil, binding action is due to cohesion.

→ Well graded soil with less than 50% fraction finer than $75\mu$ sieve & $I_P < 20$ are found to be most responsive to cement stabilisation

→ The amount of cement required is in range of 5-15% by wt of dry soil

→ For gravels, it is 5-10%

→ For sand, it is 7-15% (7-12%)

→ For silt, it is 12-15%

→ For clay, it is 12-20%

→ The amount of water to be added is decided from the consideration of good compaction & this amount must be adequate for complete hydration of cement also

Q. Calculate the no. of cement bag req. for preparing a layer of soil cement having a surface area of $5 \times 5\ m^2$ & thickness of 50cm with dry density of $18 kN/m^3$. If 10% of cement (assume) by wt of dry soil is required.

Let wt of soil be '$x$' kg

wt of cement = 10% of $x = 0.1x$ kg

wt of soil cement = $x + 0.1x = 1.1x$ kg —①

wt of soil cement stabilised = $25 \times 0.50 \times 18 \times 10^2 = 225 \times 10^2$ kg

$= 22500$ kg —②

$1.1x = 22500$

$x = 20454.54$ kg

wt of cement = 2045.5 kg

No. of cement bags = $\frac{2045.5}{50}$ = 41 bags of cement

iii. Lime stabilisation:

→ Hydrated lime is very effective in treating heavy plastic clayey soil

→ Lime may be used alone or in combination with cement, bitumen or in combination with fly ash.

→ Sandy soil can also be stabilised using this combination

→ It is mainly used for stabilising the road base, subgrades

→ On addition of lime to soil, 2 types of chemical reaction occurs

a. alteration in the nature of absorbed layer through base exchange phenomenon

b. cementing or pozzolanic action

→ Lime reduces the $I_P$ of highly plastic soils, it increases the optimum moisture content & decreases the compacted density & but strength & durability of soil increases.

→ Normally, 2-8% of lime may be required for coarse grained soil & 5-10% for plastic soil.

iv. Bitumen stabilisation:

→ Asphalt & tar are the bituminous material which are used for stabilisation of soil for pavement contruction

→ Since the viscosity of these materials is comparatively very high, it is first reduced before adding it into the soil.

→ The bituminous material when added to imparts binding action or water proofing action or both in soil.

→ Depending upon these actions & nature of soil, bituminous (bitumen) stabilisation is further classified as

a. soil bitumen

b. sand bitumen

c. Water proofed mechanical stabilisation

d. Oiled earth

a. It refers to a cohesive soil in which main function is to preserve the cohesive strength of soil by water proofing the soil

b. This term refers to the bitumen stabilised cohesionless soil, such as loose, dunes, river sand

→ Here the primary function of the bitumen is to bind the soil particles.

c. In this, small amount of bitumen 1-3% are sometimes added to mechanically stabilised soil to make them water proof

d. Slow & medium curing road oils are spread on the ground surface to make it water & abrasion resistant. The oil penetrates into the soil to the short depth & also impart cementing action

v. Chemical stabilisation:

a. Calcium chloride:

→ It is used as water retentive additive in mechanical stabilised bases & surfacing

→ Being hygroscopic & deliquescent, this absorbs a moisture from the atmosphere & retains it.

→ It act as a type of floc forming reagant & helps in compaction of the soil.

b. Sodium chloride:

→ The action of sodium chloride is same as that of calcium chloride but it is has not been widely used.

→ It attracts & retains the moisture & reduces the rate of evaporation

c. Sodium silicate: Water glass

→ It is used in combination of other chemicals such as calcium chloride, NaCl etc for stabilisation of the soil

vi. Stabilisation by heat:

→ Heating a fine grained soil to temp. of 400-600°C causes irreversible changes in clay minerals.

→ The soil becomes non-plastic, less water sensitive & non-expansive

→ This method is deployed in furnaces

vii. Electrical stabilisation:

→ The stability or shear strength of fine grained soil can be increased by draining them with passage of direct current through them.

→ This process is also termed as electroosmosis

## → Site investigation + sub-soil exploration:

→ The object of soil exploration is to provide reliable, specific + detailed information about the soil + ground + ground water conditions of the site, which may be required for a safe + economic design of engg. works.

→ The exploration must be preceded by the site reconnaissance.

→ This information is required is

a. the order of occurrence + extent of soil + rock strata

b. the nature + engg. prop. of soil + rock formation

c. the location of ground water + its variation

## → Depth of exploration:

→ Exploration in general should be carried out upto a depth at which increase in pressure due to structural loading is likely to cause settlement or shear failure.

→ Such a depth is called 'significant depth'

→ It depends upon type of structure, wt, size, shape, disposition of loaded area

→ It is generally safe to assume significant depth as depth of 10% or 20% overburden pressure residual.

→ The depth of exploration for diff. works are as follows

i. Isolated spread footing / raft : 1.5B

ii. Adjacent footing with clear spacing less than 2B : 1.5L

iii. Pile foundation : 10–30m or more at ~~level~~ least 1.5 width of structure

iv. Base of retaining wall : 1.5 the base width or 1.5 times exposed height of face of wall whichever is greater.

v. Floating basement : depth of construction

vi. Dams : 1.5 bottom width of earth dam, 2 times ht of bed to crest from concrete dams.

vii. Road, cut, fills : one meter where little fills or cut is required, can be extended upto 2m for deep cut

viii. Borrow area : convenience of excavation or thickness available

ix. From the consideration of weathering : 1.5 m in general

## → Method of site exploration:

→ Site exploration can be done by any of the following methods

i. Open excavation

ii. Boring

iii. Sub-surface soundings

iv. Geophysical method

i. Open excavation:

→ Test pits & trenches can be used for all types of soils

→ Soil can be inspected in their natural condition & samples disturbed or undisturbed can be conveniently taken

→ The cost of open excavation however increases rapidly with depth.

→ They are generally considered suitable for shallow depth upto 3m

ii. Boring:

→ The method of boring & drilling are as follows

a. Auger boring:

→ These are used in cohesive & other soft soils above water table

→ Hand augers are used upto depth of 6m & mechanical augers are used for greater depth & they can also be used in gravelly soil

→ Sample recovered from these soil brought up by augers are badly disturbed & are useful for identification purpose only

b. Auger & shell boring:

→ Cylindrical auger & shell with cutting edge on teeth at the lower end can be used for making deep borings

→ Augers are suitable for soft to stiff clays, shells for very stiff & hard clays & shell pumps for sandy soils

c. Wash boring:

→ It is a fast & simple method for advancing holes in all types of soils

→ Boulders & rocks cannot be penetrated by this method.

→ This method consists of first driving a casing through a hollow drill rod with a sharp chisel & water is forced under pressure through through the drilled rod.

→ The cuttings are forced up on the ground and are further tested

d. Percussion boring:

→ In this method, soil & rock formation are broken by repeated blows of heavy chisels by cable or drilled rod.

→ Water is added to the hole during the boring if not already present to form the slurry.

→ It is suitable for all types of soil, boulder & rocks

e. Rotatory boring:

→ It is a very fast method of advancing hole in both rocks & soil

→ A drill bit fixed to the lower end of the drill rods is rotated by a suitable mechanism

→ **Soil samples:**

→ Soil samples in general are classified into 2 categories

i. Disturbed sample

ii. Undisturbed sample

i. **Disturbed samples:**

→ These are those where the soil structure gets modified or destroyed during the sampling operation

→ These are further classified into following

a. **Representative sample:**

→ With suitable precautions, the natural moisture content & proportion of mineral constituent can be preserved in the sample termed as representative sample

b. **Non-representative sample:**

→ If during the sampling, the addition of to the alteration in the original soil structure, soil from the other layer gets mixed up or mineral constituents gets altered, the sample is termed as non-representative

→ Representative samples are useful for identification test but non-representative samples are of no use.

ii. **Undisturbed sample:**

→ These are those where original soil structure is preserved & the material prop. have not been altered.

→ These samples are practically not possible to obtain, but samples with minute/minor alterations are still suitable for lab test.

→ **Types of soil samples req. for lab test:**

| Type of test | Type of sample required |
|---|---|
| Natural water content | Undisturbed |
| Density | " |
| Specific gravity | Representative or Undisturbed |
| grain size distribution | " " " |
| Atterberg limit | " " " |
| Coeff. of permeability | Undisturbed |
| consolidation parameters | " |
| shear strength parameter | " |

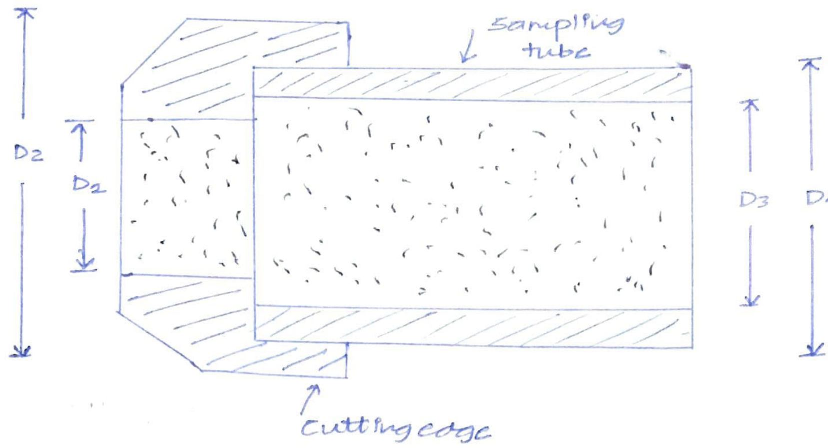
→ The extent of disturbance of the sample, due to sampler depends upon 3 features

i. cutting edge

ii. Inside wall friction

iii. Non-return wall

→ The following ratios related to the dimensions of the cutting edge & the sampler are useful




→ Inside clearance ($C_i$)

$$C_i = \frac{D_3 - D_1}{D_1} \times 100$$

→ Outside clearance ($C_o$)

$$C_o = \frac{-D_4 + D_2}{D_2} \times 100$$

$$C_o = \frac{D_2 - D_4}{D_2} \times 100$$

→ Area ratio ($A_r$),

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

$D_1$ = Inside dia. of cutting edge

$D_2$ = Outside " " " "

$D_3$ = Inside dia. of sampling tube

$D_4$ = Outside " " " "

→ The inside clearance is meant to reduce the friction b/w the soil sample & the sampler when the soil enters into the tube by allowing for elastic expansion

→ If the inside clearance is too high, there will be too much of lateral expansion.

→ Outside clearance will help reduce the friction will while the sampler is being driven & being withdrawn after the sample is collected

→ $C_i$ = 1–3%

→ $C_o$ = 0–2% (it should not be much greater than $C_i$)

→ Area ratio should be kept as low as possible, consistent with the strength requirements of the drive shoe & the sampling tube so as to reduce sample disturbance.

→ $A_r \ngtr 20\%$, for stiff formation

→ $A_r \ngtr 10\%$ for soft sensitive soil

→ Another parameter which is an index of sample disturbance is recovery ratio

$$\text{Recovery ratio } (L_r) = \frac{\text{recovered length of the sample}}{\text{penetration length of the sampler}}$$

→ If $L_r = 1$ ⇒ good recovery

→ If $L_r < 1$ ⇒ compression while sampling

→ If $L_r > 1$ ⇒ expansion/swelling while sampling

→ To reduce wall friction, the sampling tube should have smooth finish & should be properly oiled before use.

→ Types of sampler:

→ The commonly used sampler are as follows

i. Open drive sampler

→ This sampler essentially consist of a seamless open-end steel tube with a cutting edge

→ The tube is connected through a head to the drill rod.

→ The sampler head is provided with vents to permit water & air to escape during the sampling & also a check valve to retain the sample

→ Thin wall samplers are used in this case to obtain undisturbed samples

ii. Piston sampler:

→ A piston sampler consists of 2 seperate parts

a. sampler cylinder

b. Piston cylinder

→ The piston which is actuated seperately and fits tightly in the sampler cylinder

→ It is useful in sampling saturated sand & other soft & wet soil which cannot be sampled by open drive sampler.

iii. Rotatory sampler:

→ It is a double walled tube sampler with an inner removable liner.

→ The outer tube or the rotating barrel is provided with a cutting bit.

→ The bit cuts an annular ring when the barrel is rotated

→ The inner tube which is stationery slides over the cylindrical sample cut by outer rotating barrel

iv. Block/chunk sampler:

→ Block or chunk can be obtained from open excavation like test pit, shaft etc

→ For chunk sampling, it is necessary to that the soil has some cohesion

iii. Sounding & Penetration test:

→ Subsoil exploration can be done by following penetration test

a. Standard penetration test (SPT)
b. Static cone penetration test (SCPT)
c. Dynamic " " "
d. Pressure meter test
e. Plate load test

c. Dynamic cone penetration test:

→ In this test, a cone which has an apex angle of 60° & attached to the drill rod is driven into the soil by blows of the hammer of 65kg falling freely from the ht of 750mm.

→ The blow count for every 100mm penetration is noted continuously.

→ The cone is driven until refusal or upto the required depth

→ The no of blows required for 300mm penetration is noted as dynamic cone resistance.

→ This test gives a continuous record of $N_{cd}$ with depth

→ No samples however can be obtained in this test

→ Co-relation b/w dynamic cone resistance & SPT value can be referred as follows when 50mm dia. cone is used,

$N_{cd} = 1.5N$ for depth upto 3m

$N_{cd} = 1.75N$ " " 3-6m

$N_{cd} = 2N$ " " > 6m

d. Pressure meter test:

→ It is a form of load test & the load in this case being applied by a uniform radial pressure to the sides of the borehole in which a pressuremeter is placed.

→ There are two basic types of pressure meter

i. The Menard pressuremeter (MPM), which is lowered into a preformed borehole

ii. The self boring pressuremeter (SBP), which forms its own borehole & thus causes much can less disturbance to the soil prior to the testing

→ In this test, elastic constants of the soil like Menard's modulus of elastic & Menard's shear modulus is also found.

NOTE:- Relationship b/w Menard's modulus of elasticity (EM) & young's modulus of elasticity (E) is given by

$$\boxed{E = \frac{E_M}{\alpha}}$$

$\alpha$ = Rheological factor

2. In clayey soil, undrained shear strength can also be determined as follows

$$C_u = \frac{P_i}{9}$$

$P_i$ = limit pressure

## iv. Geophysical method:

→ These methods were developed in connection with prospecting for useful minerals + oils.

→ The major methods of this category are

a. Gravitational method

b. Magnetic "

c. Seasonic seismic refraction method

d. Electrical resistivity "

### a. Siesmic refraction method:

→ In this method, shock waves are created into the soil at their ground level or certain depth below it by exploding small charges.

→ These waves are then detected by geophones, where the time of travel gets recorded

→ Either a number of geophones along a line are used or shock producing device is moved away from the geophone to produce shock waves in given interval.

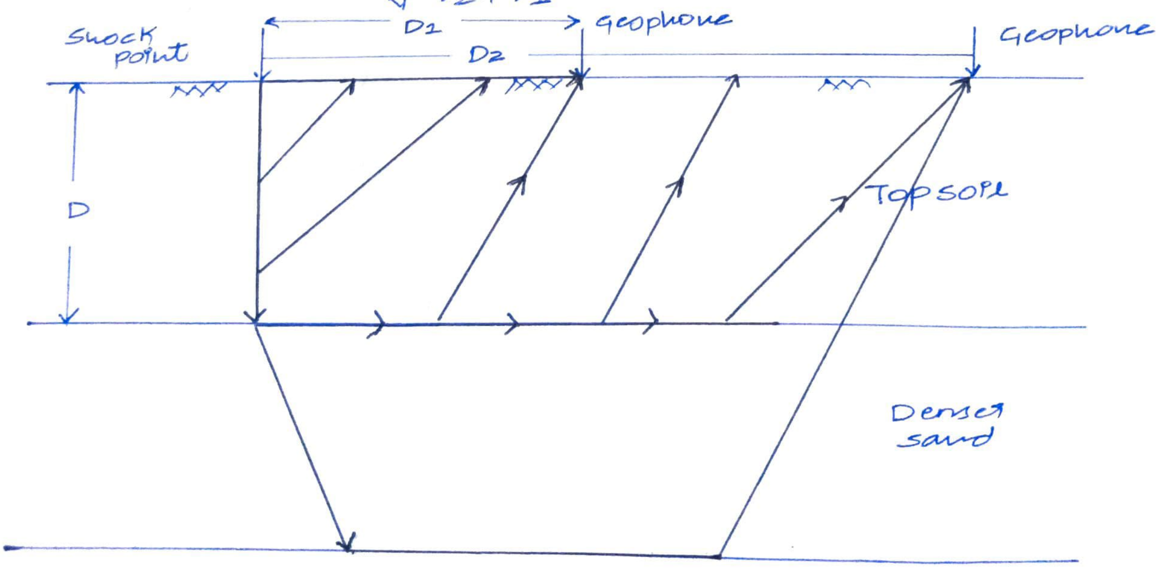
→ Some of the waves are directly reached to the geophones along the ground surface whereas some are first penetrated downwards into underlying denser medium & then reflected back to the ~~fn~~ geophone

→ Waves travel much faster in denser medium underlying the loose medium.

→ As the distance b/w the shock pt & geophone increases, the refracted waves are able to reach earlier than direct waves to the geophone

→ The depth of the boundary b/w 2 strata can be estimated as

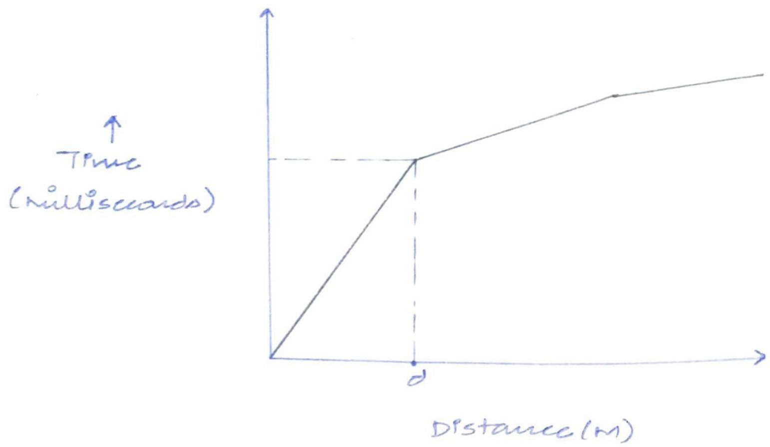
$$D = \frac{d}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}}$$




$$D = \frac{d}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}}$$

$v_2$ = velocity of wave in denser medium

$v_1$ = " " " " loose "




## 2. Electrical resistivity method:

→ This method is based upon the measurement of changes in the mean resistivity or specific resistance of various soil

→ The resistivity is defined as the resistance b/w opposite faces of a unit cube of the material

→ Each soil has its own resistivity depending upon water content, compaction & minerology (mineral structure)

→ This test is conducted by driving 4 metal electrodes into the ground along the straight line at equal distance.

→ A direct voltage is imposed b/w the 2 outer electrodes & potential drop is measured b/w the inner electrodes.

→ The mean resistivity is given by,

$$\rho = 2\pi D \frac{E}{I} = 2\pi D R$$

$\rho$ = mean resistivity

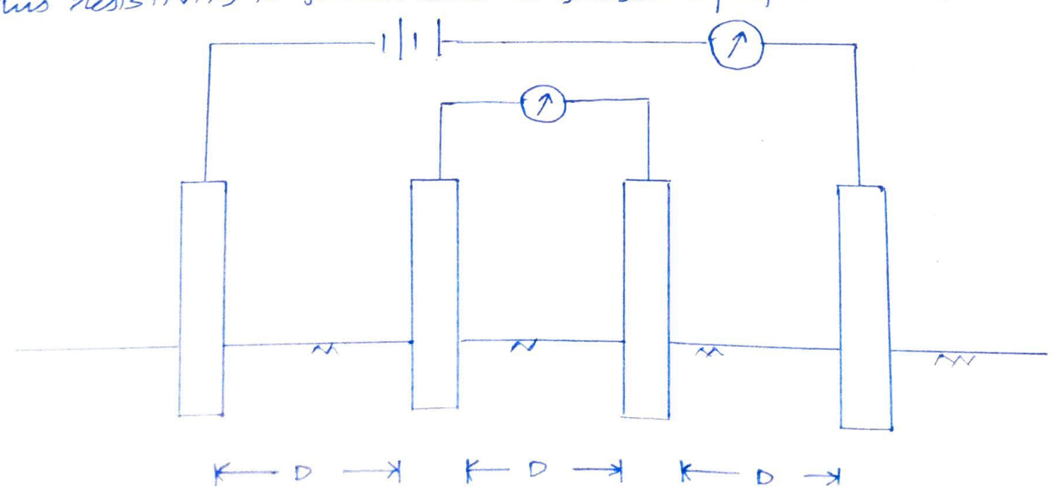
D = Dist. b/w the electrode

E = pot. drop b/w outer electrode

I = current flowing b/w outer electrode

R = Resistance

→ This resistivity is further used to gauge the prop. of the soil



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Course on Geo-Technical & Foundation Engineering - Part II

# Session Name : Soil Exploration and Soil Stabilization - Part 6

Lesson 43 • Sept 17, 2020 • Jaspal Singh

## (iii) SOUNDING & PENETRATION TEST

- Sub soil exploration can be done by following penetration test

(a) STANDARD PENETRATION TEST (SPT)

(b) STATIC CONE " " (CPT)

(c) DYNAMIC " " "

(d) PRESSURE METER TEST

(e) PLATE LOAD TEST

## (c) DYNAMIC CONE PENETRATION TEST

- In this test a cone which has an apex angle of 60° & attached to drill rods is driven into the soil by blows of a hammer of 65 kg, falling freely from the ht 1750 mm
- The blow count for every 100 mm penetration is noted continuously
- The cone is driven until refusal or upto the required depth

- The no of blows required for 300mm penetration is noted as dynamic cone resistance
- This test gives a continous record of Ncd with depth.
- No samples, however can be obtained in this test
- Co-relation b/w Ncd & N can be reffered as follows.

when 50mm dia cone is used

$N_{cd} = 1.5N$ for depth upto 3m

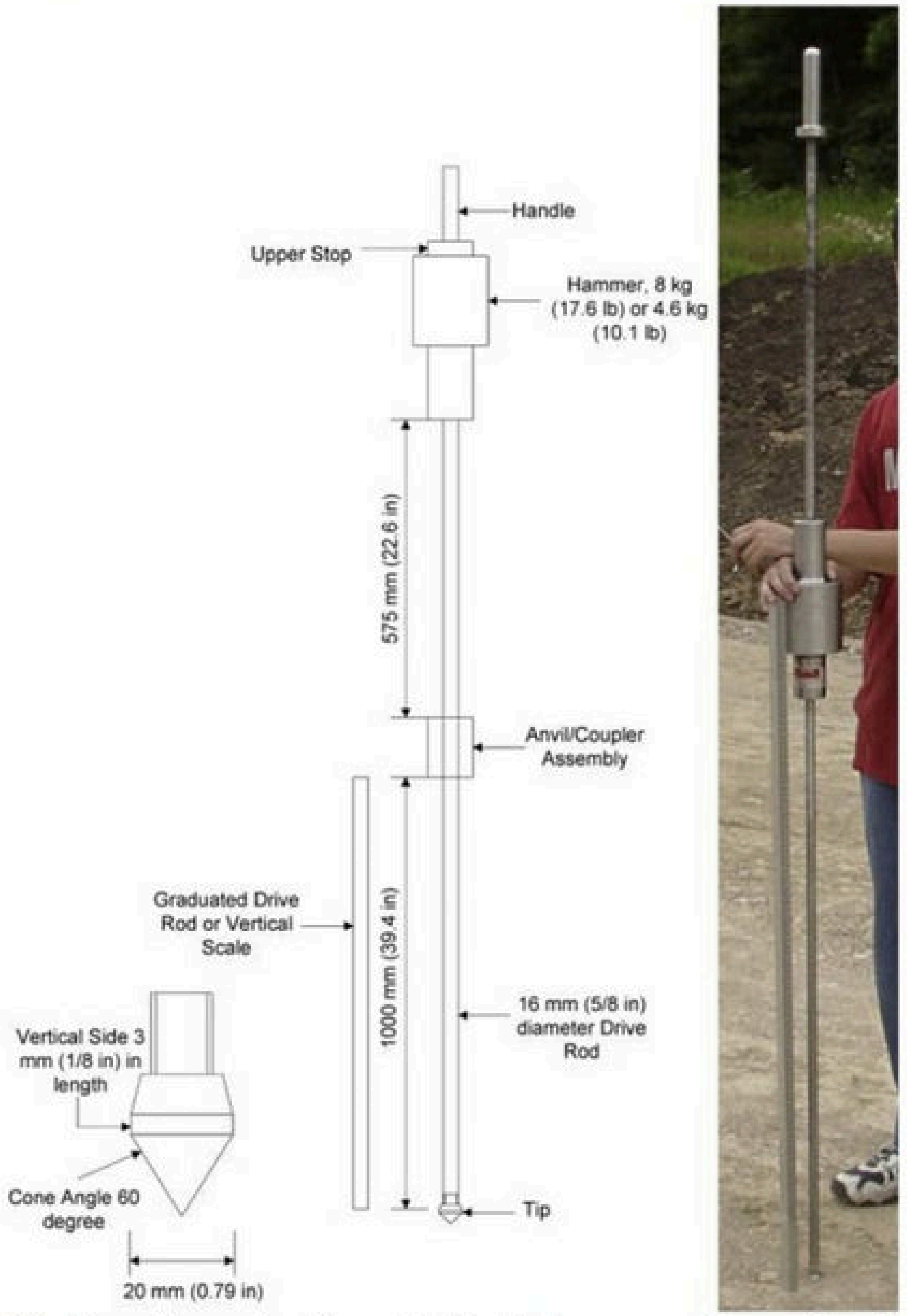
$N_{cd} = 1.75N$ " " 3 – 6m

$N_{cd} = 2N$ " " > 6m

# Question

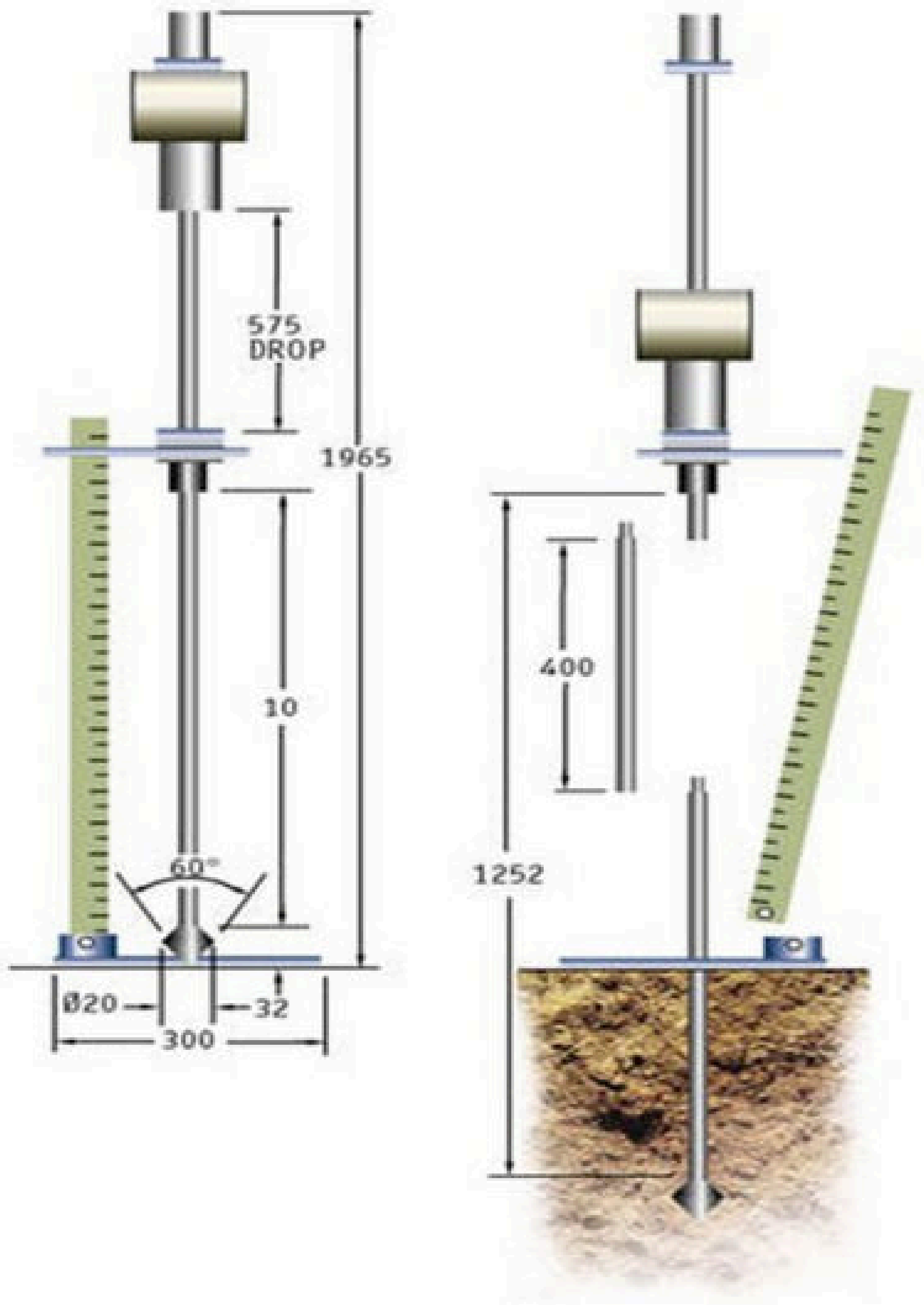
Anmol Agrawal

dynamic cone



# Question

Saurabh Singh



(d) PRESSURE METER TEST

- It is a form of load test, the load in this case being applied by uniform radial pressure to the sides of a bore hole in which a pressure-meter is placed.
- There are two basic type of pressure meter

(i) The Menard pressumeter (MPM), which is lowered into a preformed borehole

(ii) The self boring pressure meter (SBP), which.

forms its own borehole & thus causes much less disturbance to the soil prior to testing.

- In this test elastic constant of the soil like Menard's modulus of elasticity & Menard shear modulus is also found

<u>Note</u> 1) Relationship b/w Menard modulus of elasticity ($E_m$) & Youngs modulus of elasticity ($E$) is given by

$$E = \frac{E_m}{\alpha}$$

$\alpha$ = RHEOLOGICAL FACTOR

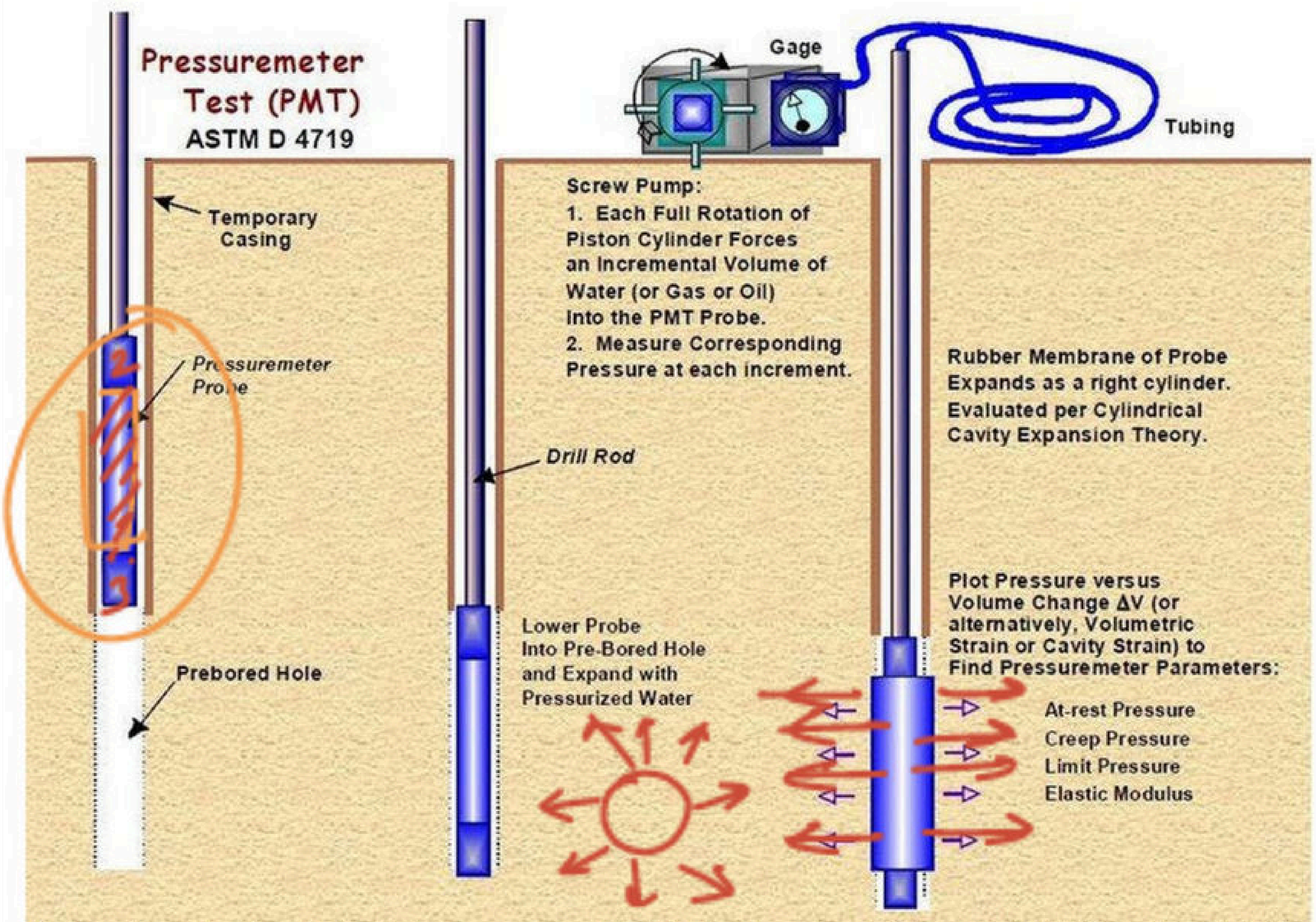
2) In clayey soil undrained shear strn can also be determined as follows

$$C_u = \frac{P_L}{9}$$

$P_L$ = Limit pressure.

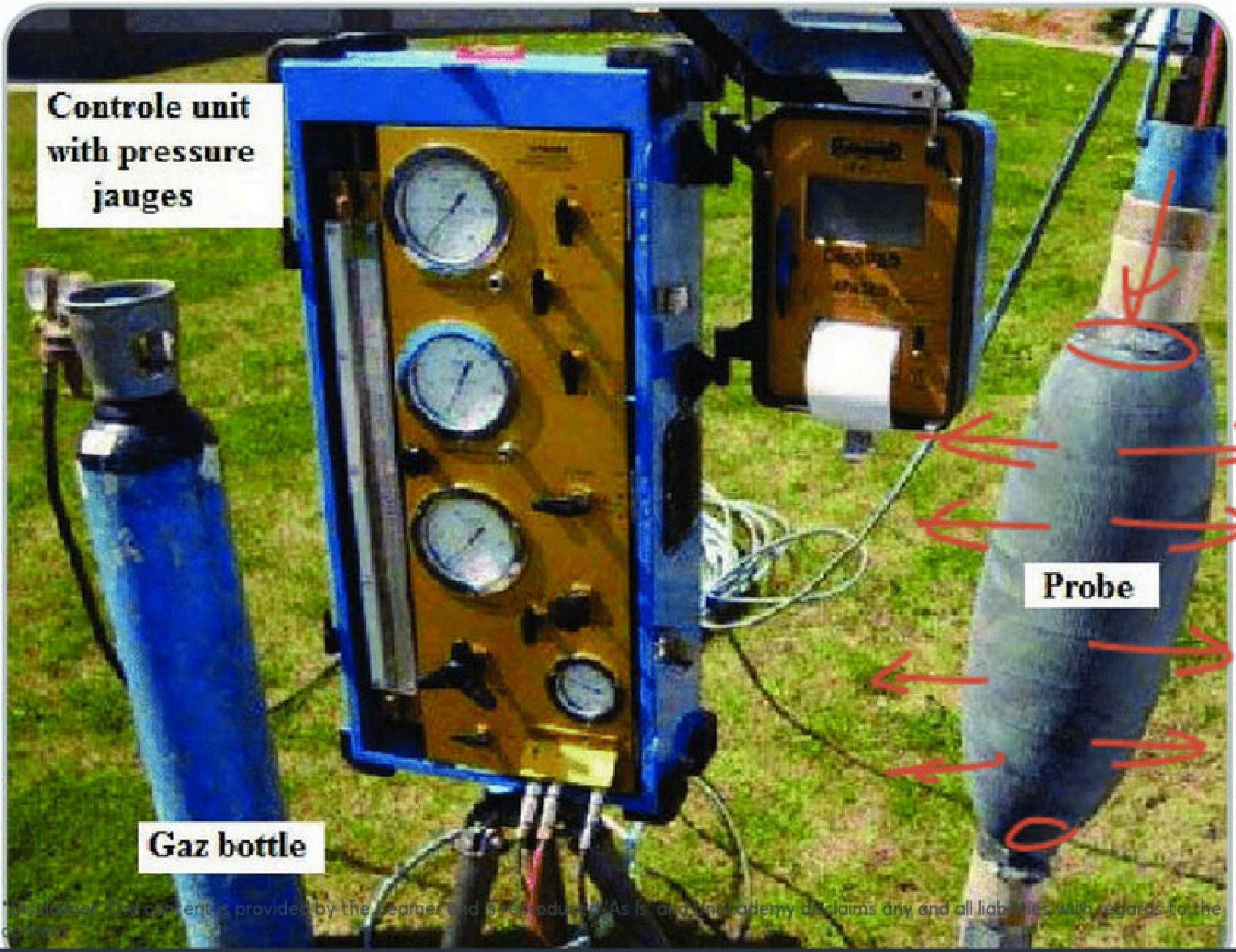
# Question

Saurabh Singh



# Question

Saurabh Singh



## (iv) GEOPHYSICAL MTHD

- These mtd were developed in connection with prospecting for useful minerals & oil.
- The major mtd of this category are

(a) gravitational mtd

(b) magnetic "

(c) seismic refraction "

(d) electrical resistivity "

(1) SEISMIC REFRACTION MTDn.

- In this mtd, shock waves are created into the soil at their ground level or certain depth below it by exploding small charges.
- These waves are then detected by geophones, where the time of travel gets recorded.
- Either a number of geophones along a line are used or shock producing device is moved away from the geophone to produce

shock waves in given interval.

- Some waves are directly reached to the geophone along the ground surface where some are first penetrated downward into underlying denser medium & then reflected back to the geophone.

- waves travel much faster in denser medium underlying the loose medium

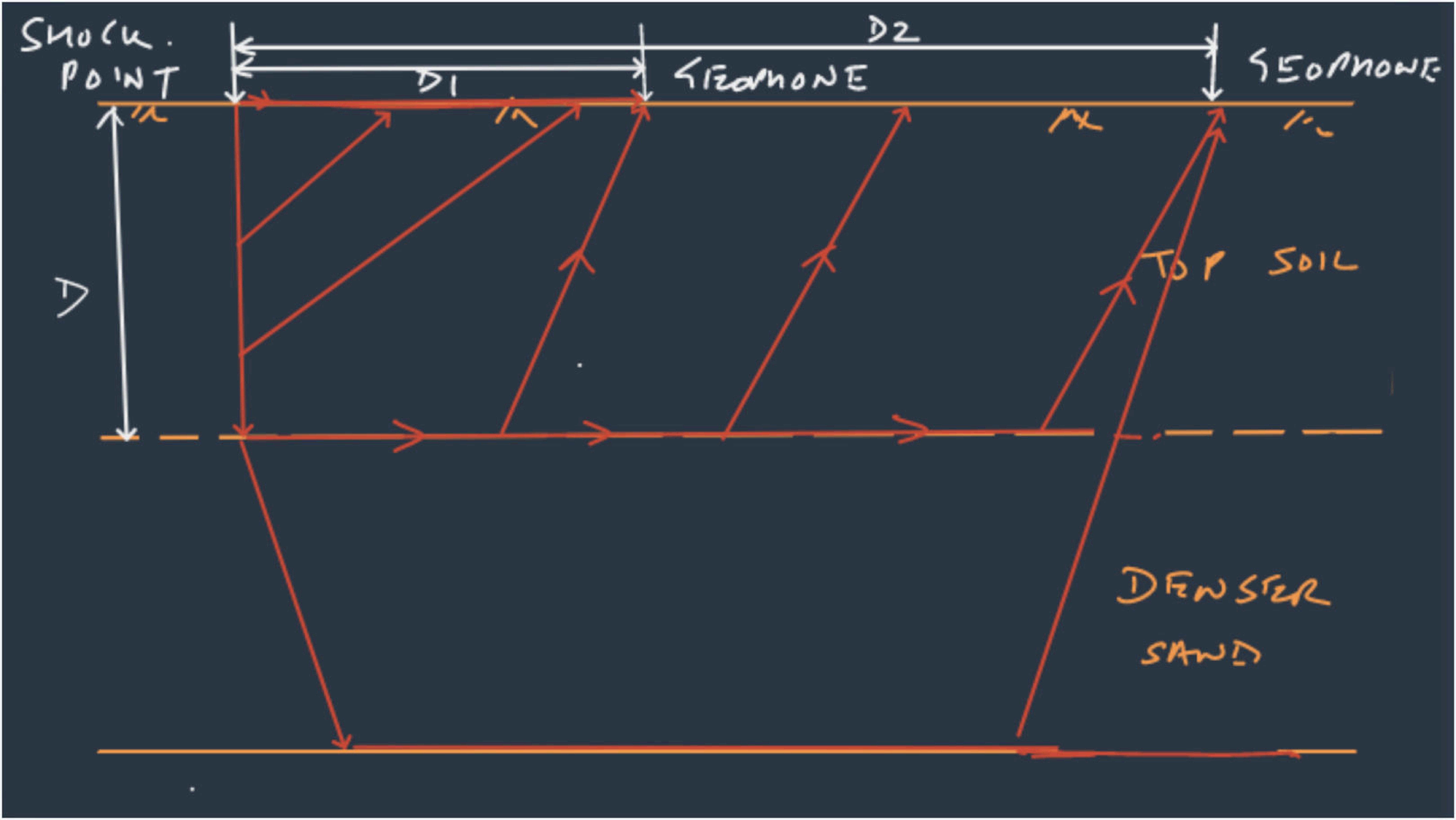
- As the distance b/w shock point & geophone increases, the refracted waves are able to reach earlier than direct waves to the geophone

- The depth of the boundry b/w two strata can be estimated as

$$D = \frac{1}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}}$$

$v_2$ = velocity of wave in denser medium

$v_1$ = velocity of wave in loose medium.



SHOCK POINT
D2
D1
GEOPHONE
GEOPHONE
D
TOP SOIL
DENSER SAND

## (iii) ELECTRICAL RESISTIVITY MTD

- This mtd is based on the measurement of changes in the mean resistivity or specific resistance of various soil
- The resistivity is defined as the resistance b/w opposite faces of a unit cube of the material
- Each soil has its own resistivity, depending upon water content, compaction & mineral structure.

- This test is conducted by driving four metal electrodes in the ground along the str line at equal distance.
- A direct voltage is imposed b/w the two outer electrodes & potential drop is measured b/w the inner electrodes.

- The mean resistivity is given by

$$\rho = 2\pi D \frac{E}{I} = 2\pi D R$$

$\rho$ = mean resistivity

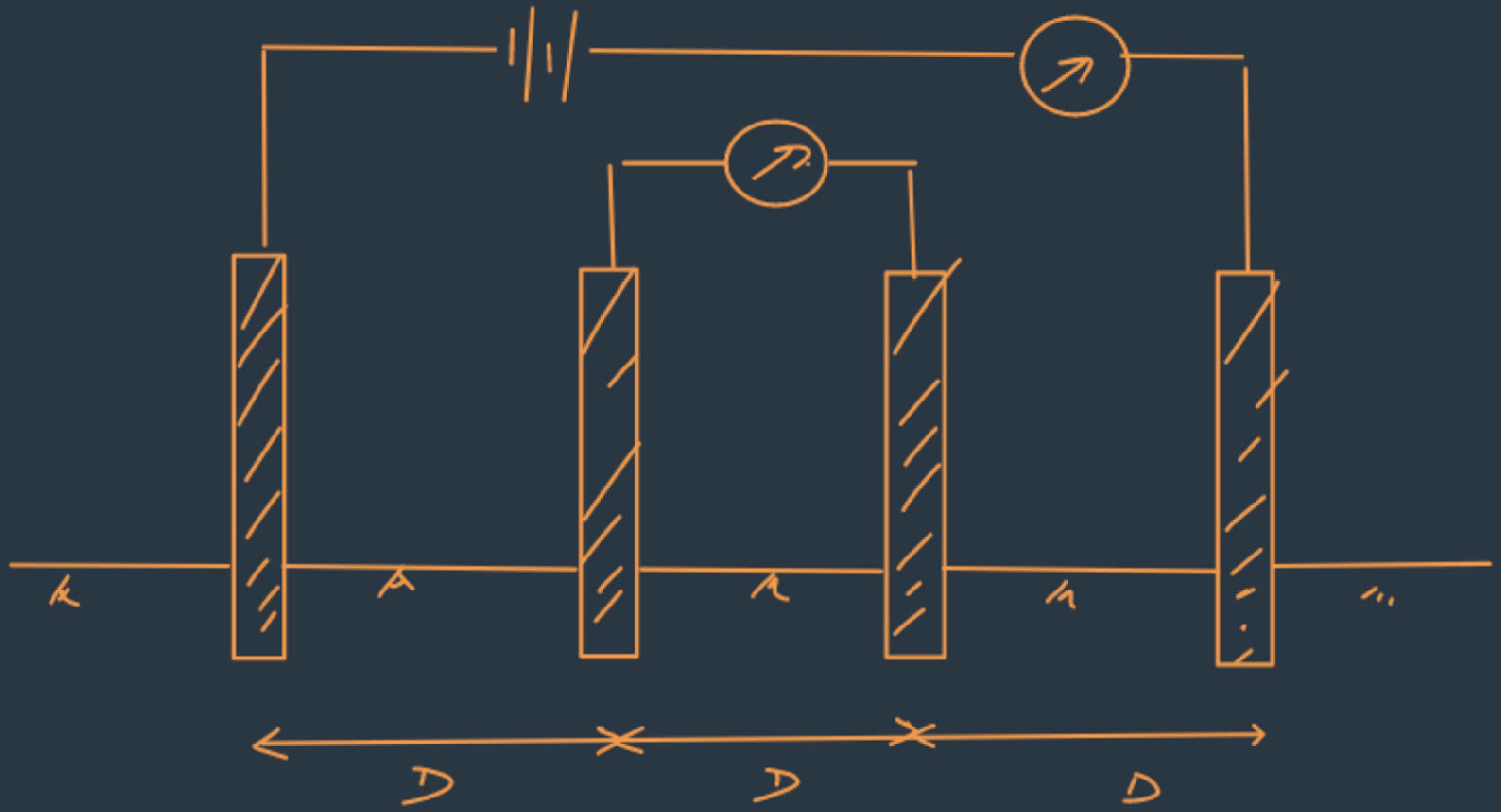
D = Distance b/w^n electrode

$r_2$ = potential drop b/w^n inner electrode

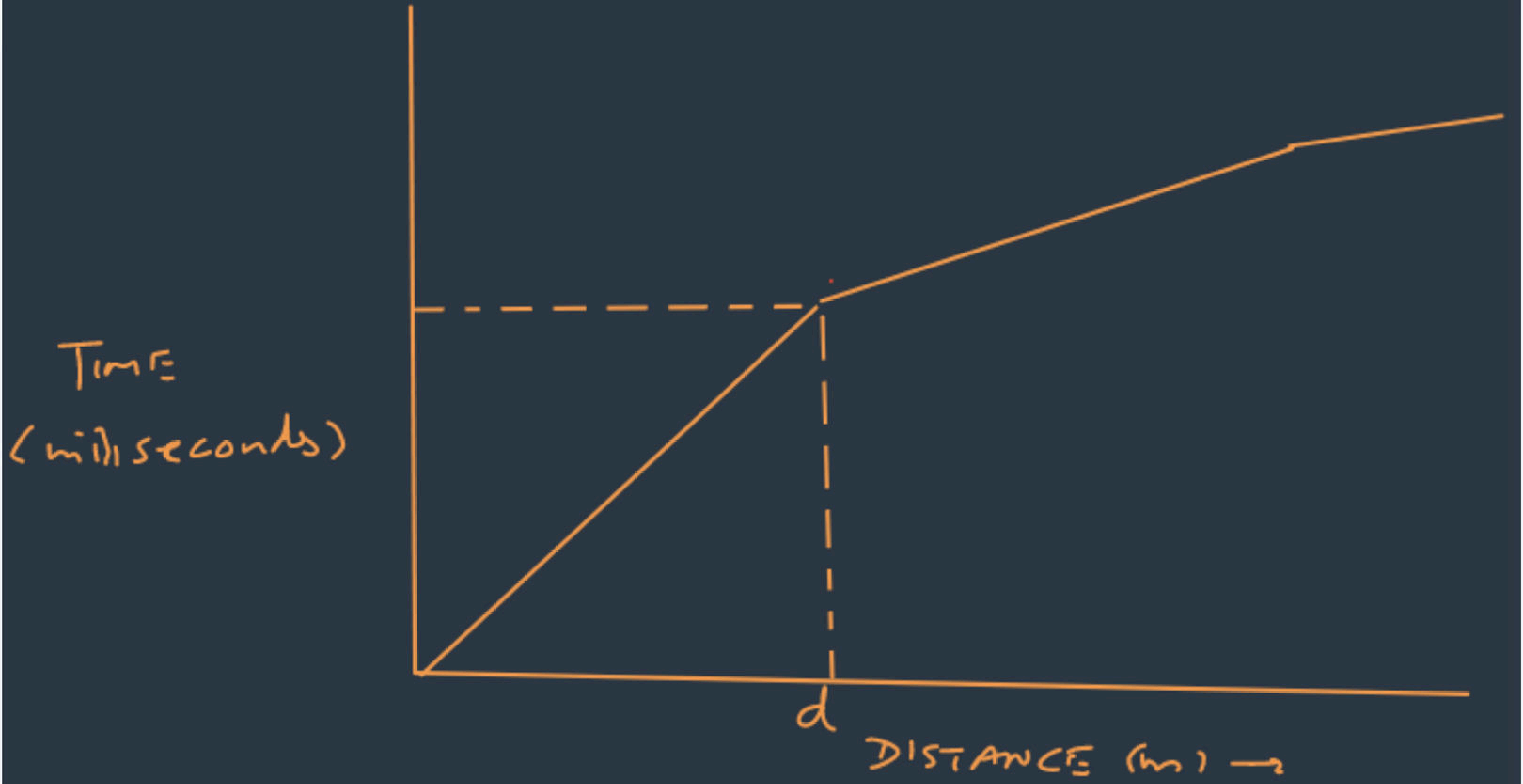
I = current flowing b/w^n outer electrode

R = Resistance

- This resistivity is further used to gauge the property of the soil.


D
D
D

# GEOTEXTILES & REINFORCED - SOIL



TIME
(milliseconds)
d
DISTANCE (m)

# What is a geosynthetic ?

- Natural or artificial product that is used along with soil in geotechnical constructions.
- Natural: coir, jute, hemp, etc.
- Artificial: polymeric or metallic

# Why geosynthetics ?

- Geosynthetics have entirely changed the way geotechnical engineering is practiced.
- Innovative solutions to solve difficult problems economically and expediently
- Enables the use of local materials – sustainable solutions
- Unskilled labour can be employed
- Installation does not require heavy machinery

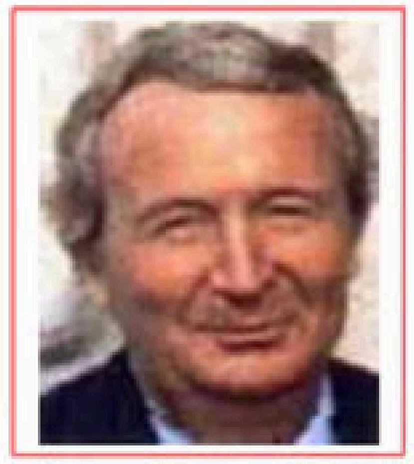
# REINFORCED SOIL

- Soil + reinforcement = reinforced soil
- Reinforcement:

  Ancient: Tree branches, grass reeds, straw, roots of vegetation, bamboo, tree trunks

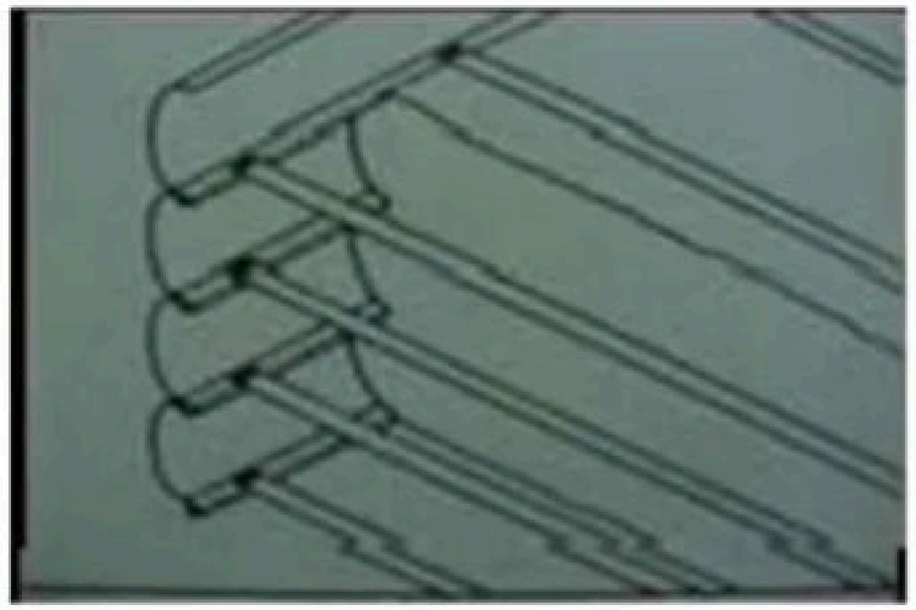
  Modern: Steel, polymeric, natural materials
- Soil is strong in compression & reinforcement is strong in tension
- Combined product has much better engineering properties than the individual constituents
- Reinforced soil concept is similar to that of reinforced concrete

# Henri Vidal (re)invents Reinforced Earth in 1963



Henri Vidal
French Engineer & Architect

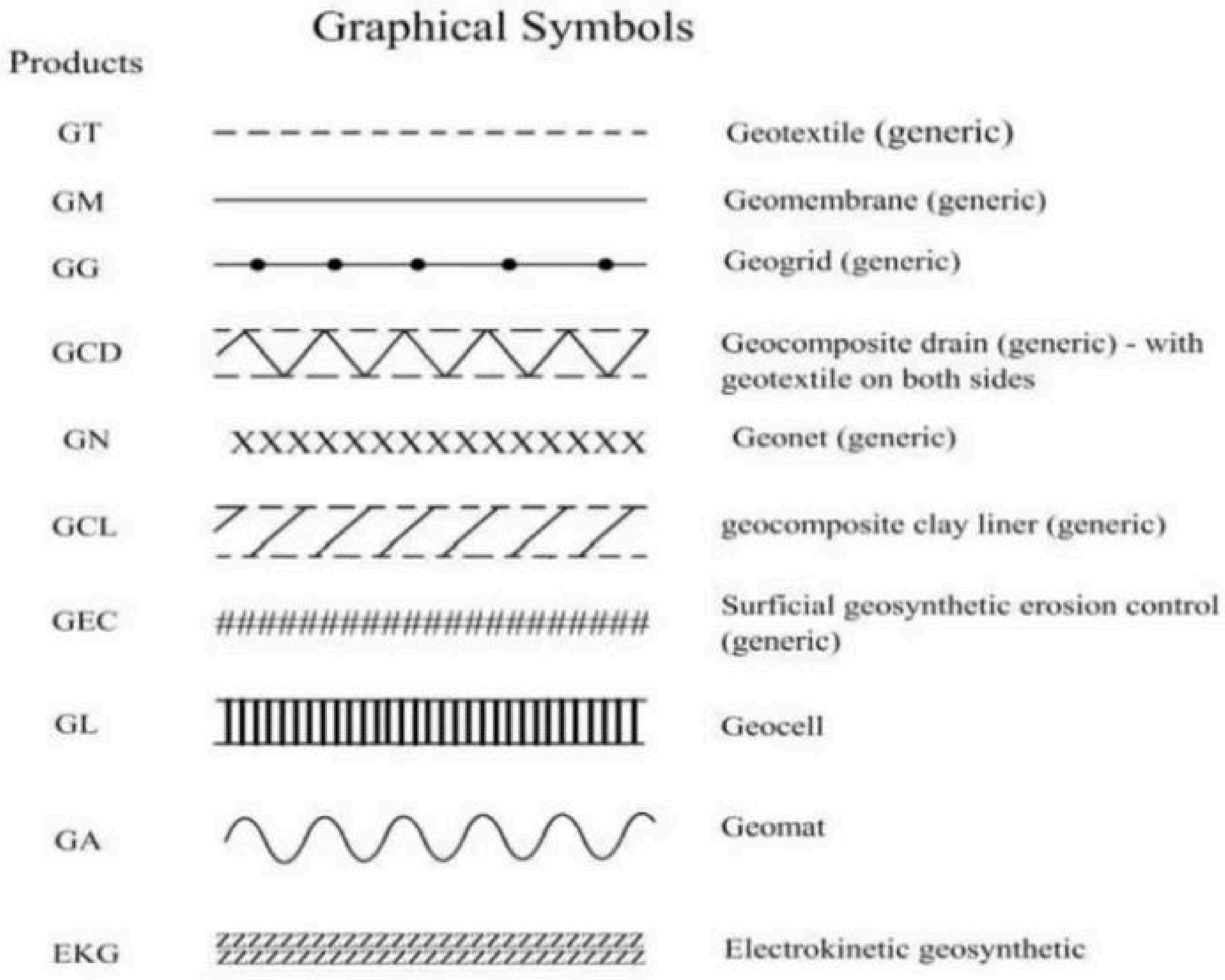

1963 : Patent filed for Reinforced Earth

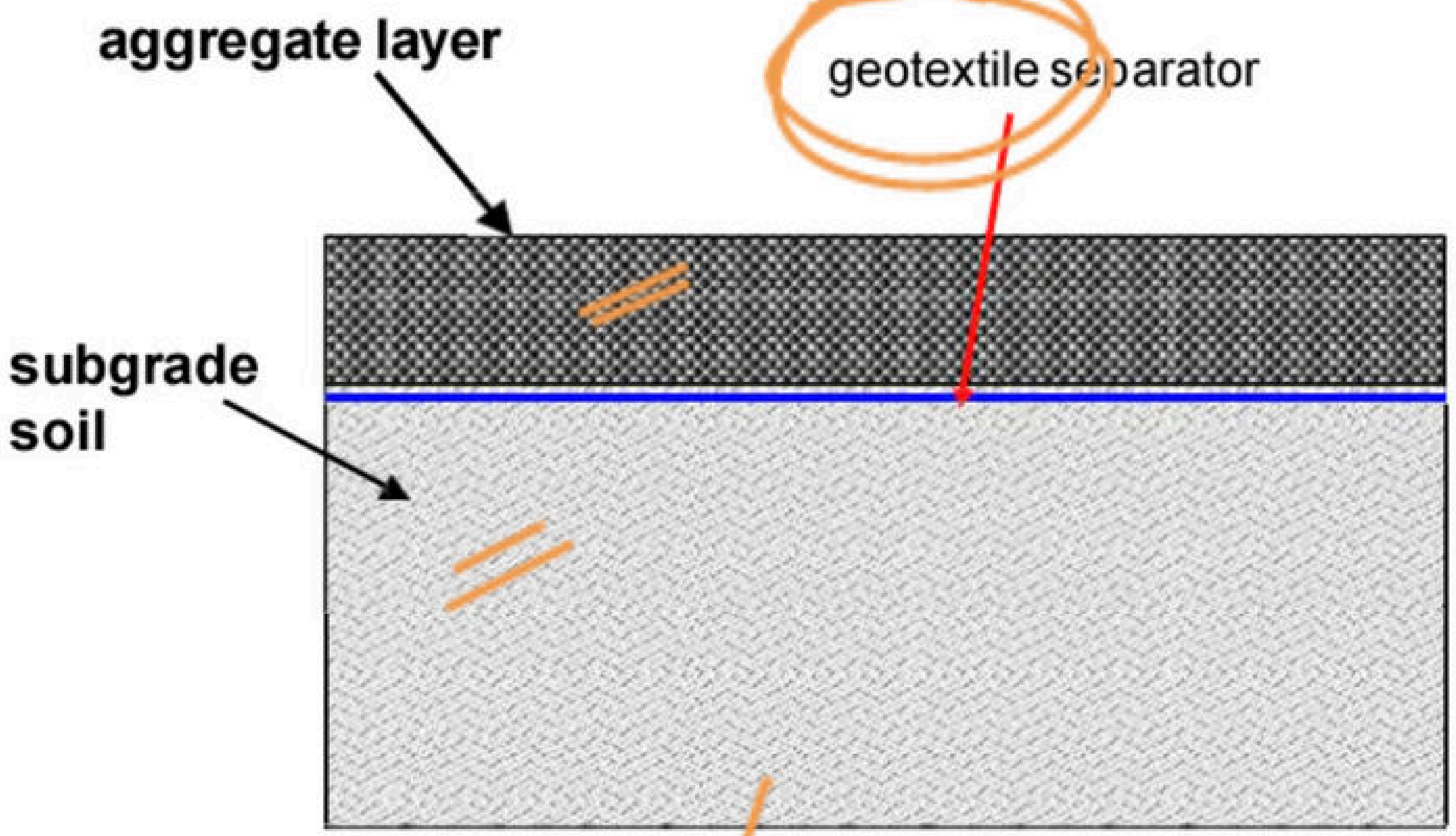


An early form of Reinforced Earth using steel strip reinforcement and steel membrane facing

The concept of reinforced soil was accidentally thought about by Mr. Vidal while playing with his children on a beach

## Graphical symbols proposed by IGS letter
## Symbol symbol product

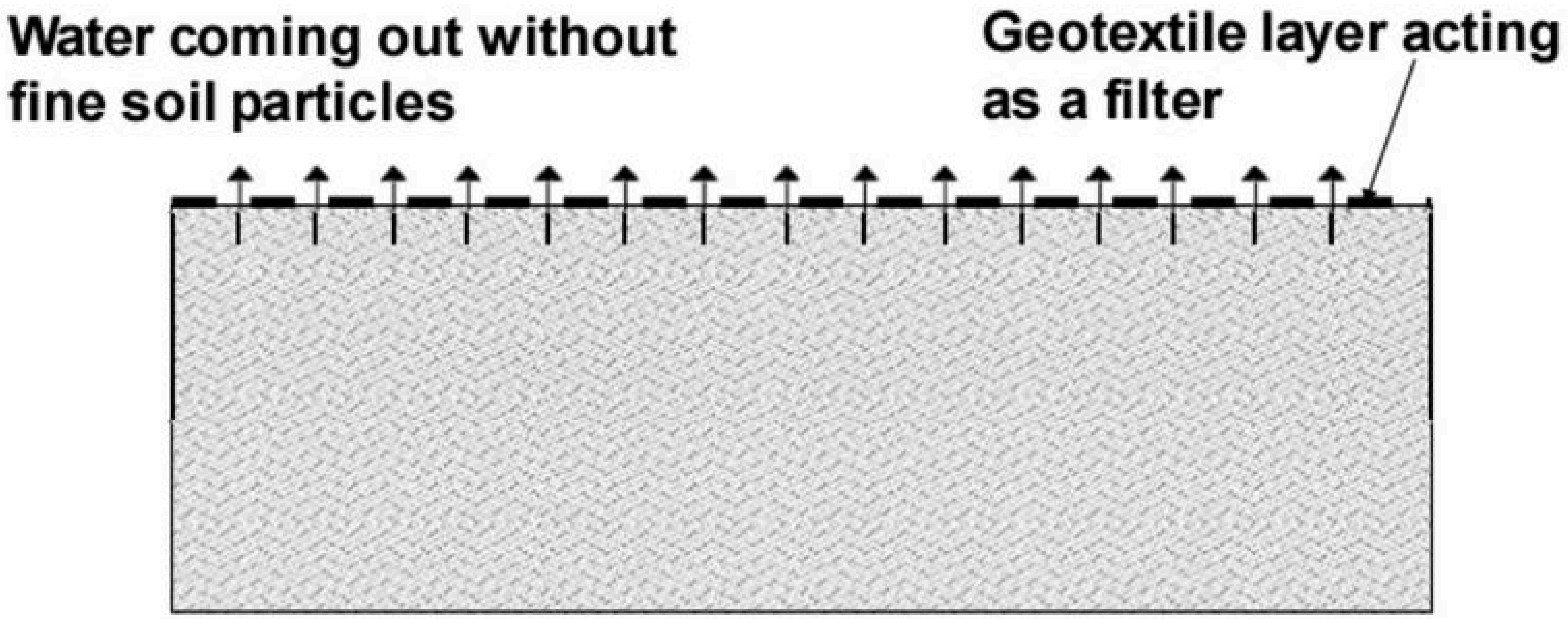




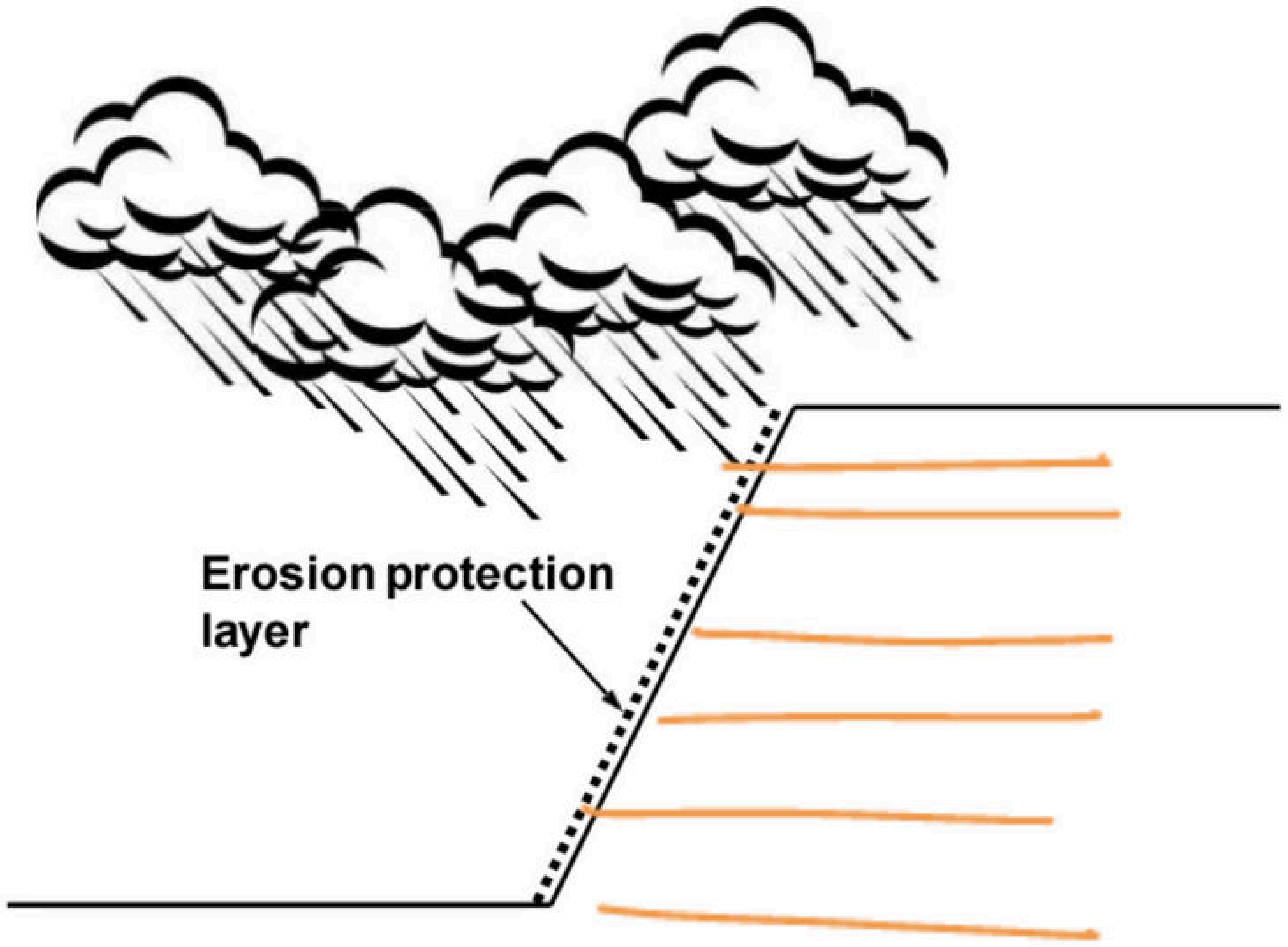

**Prevents the intermixing, prevents piping, strength of aggregate is preserved**

## Separation Function in a pavement layer

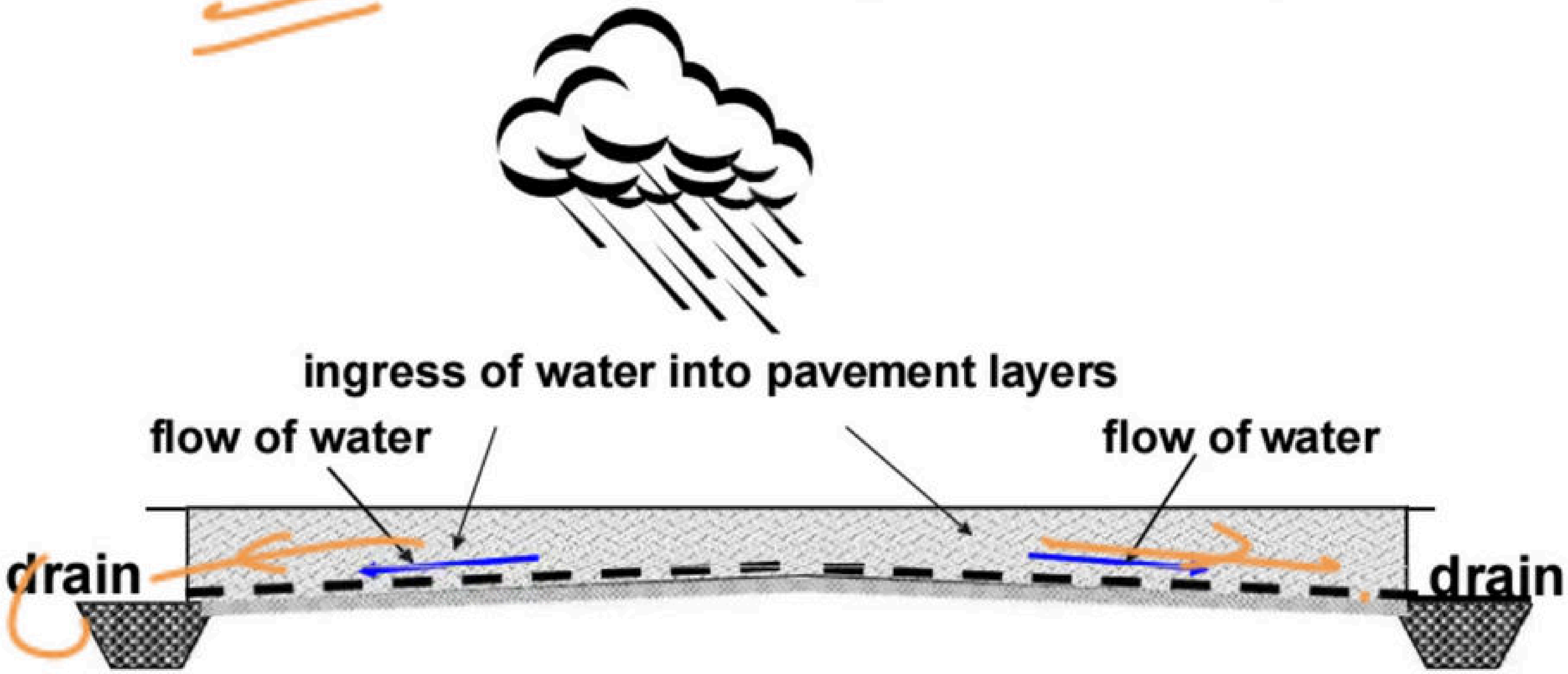
# Filtration Function



# Surface Erosion protection



# Drainage function of a geotextile layer

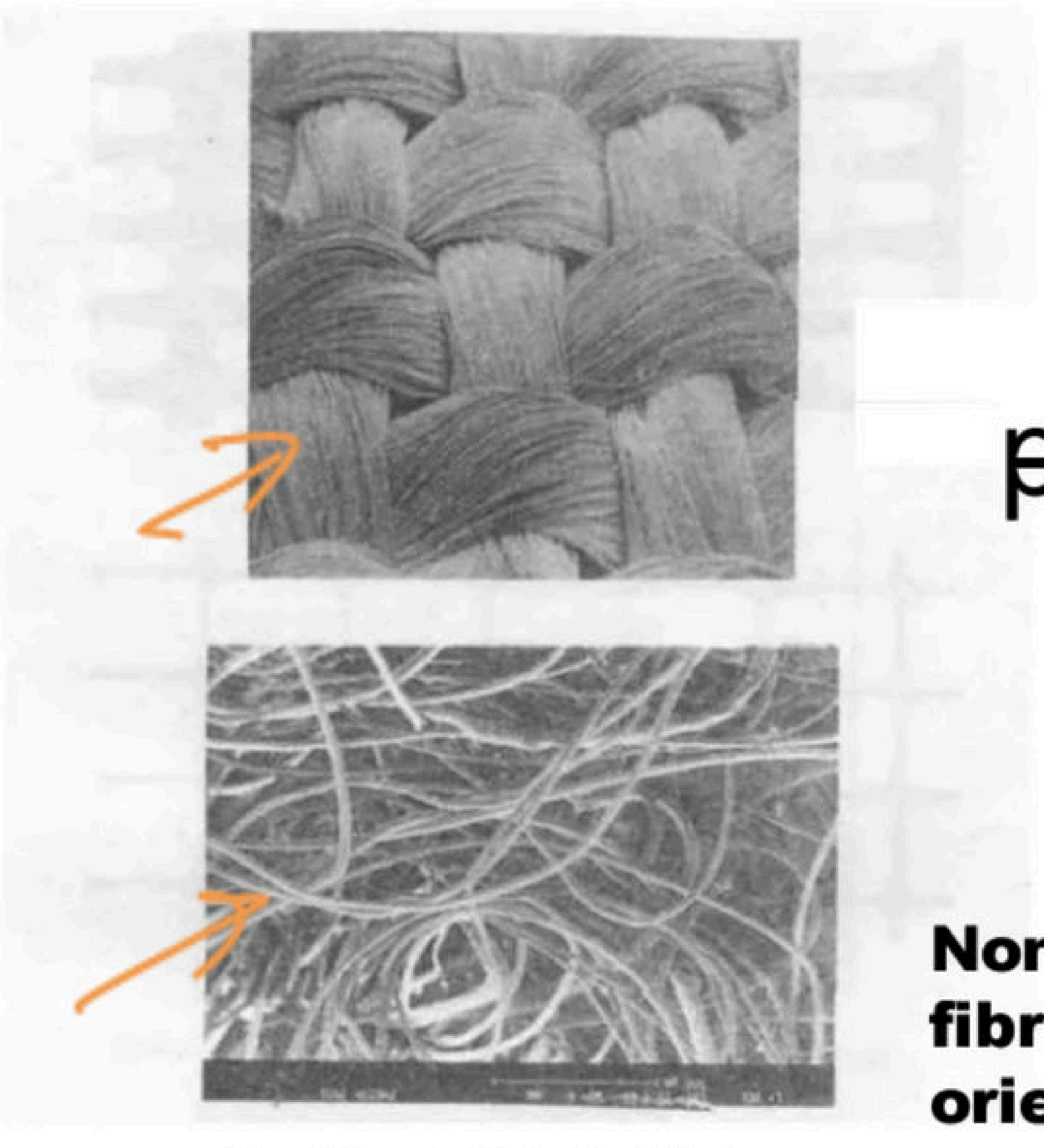


# TYPES OF GEOSYNTHETICS

- **Geotextiles**
- **Geogrids**
- **Geonets**
- **Geomembranes**
- **Pre-fabricated vertical drains (PVD)**
- **Geosynthetic Clay Liner (GCL)**
- **Geocells (3-d confinement)**
- **Geocomposites & Geo-others**

# Geotextiles

- Engineered sheet like products made of natural or synthetic materials
- Woven and non-woven types
- Used for separation, drainage, filtration, erosion control and reinforcement



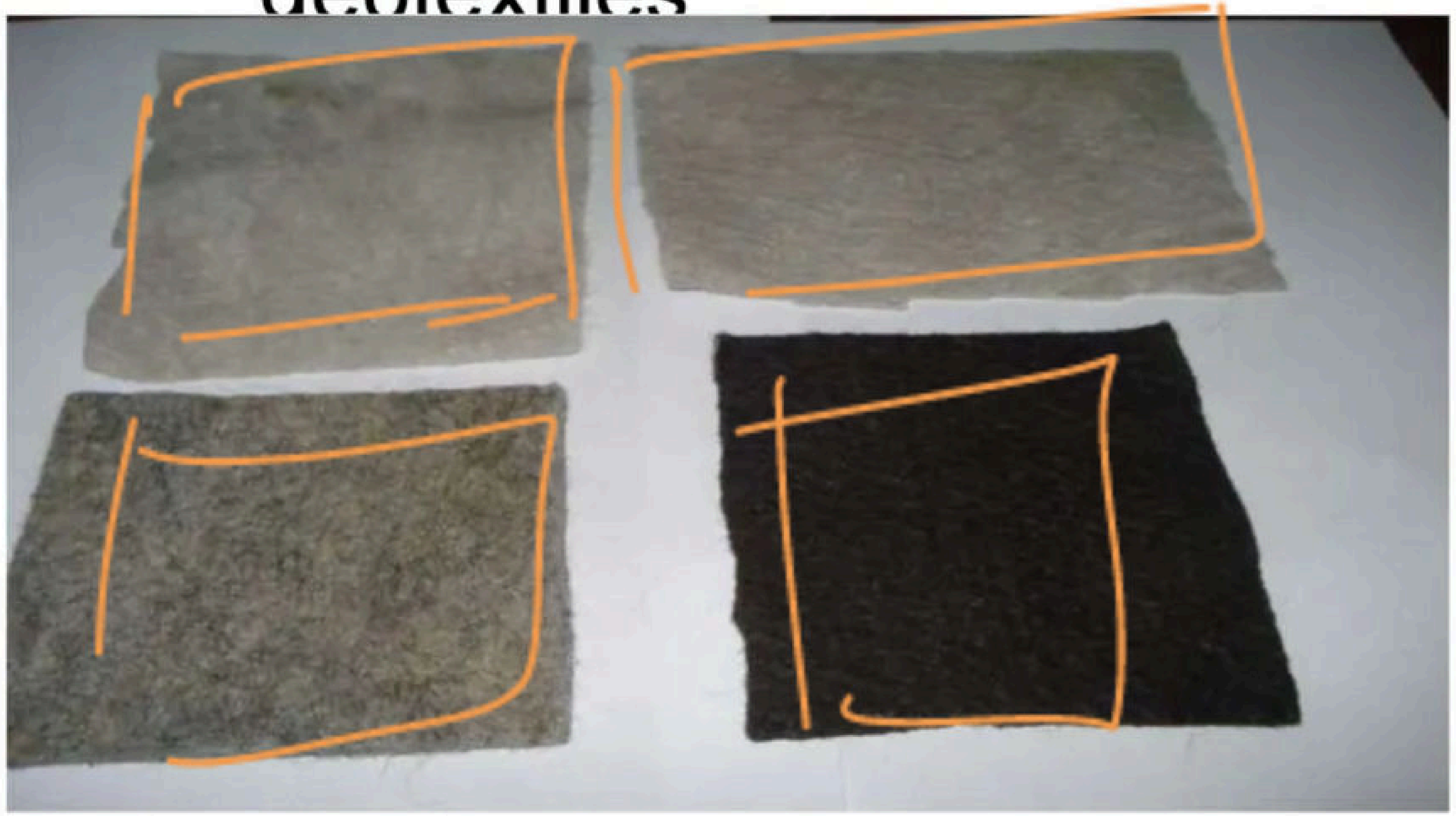
Two Types of Geotextiles.

Woven fabric weave pattern is visible

**Non-woven fabric – fibres are randomly oriented**

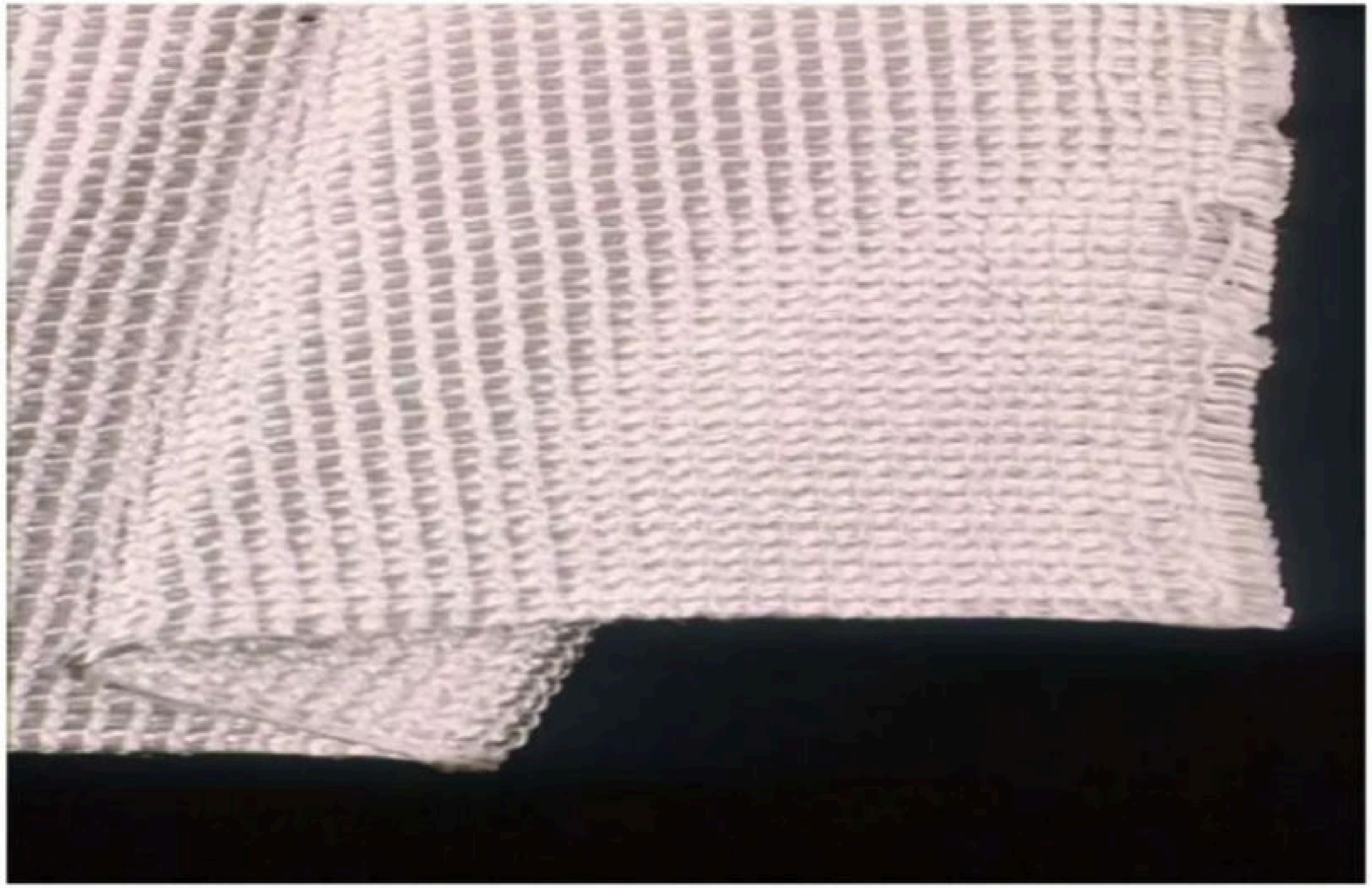
**Rao (1995)**

# Some pictures of geotextiles



woven non-woven

Natural geotextiles made of jute



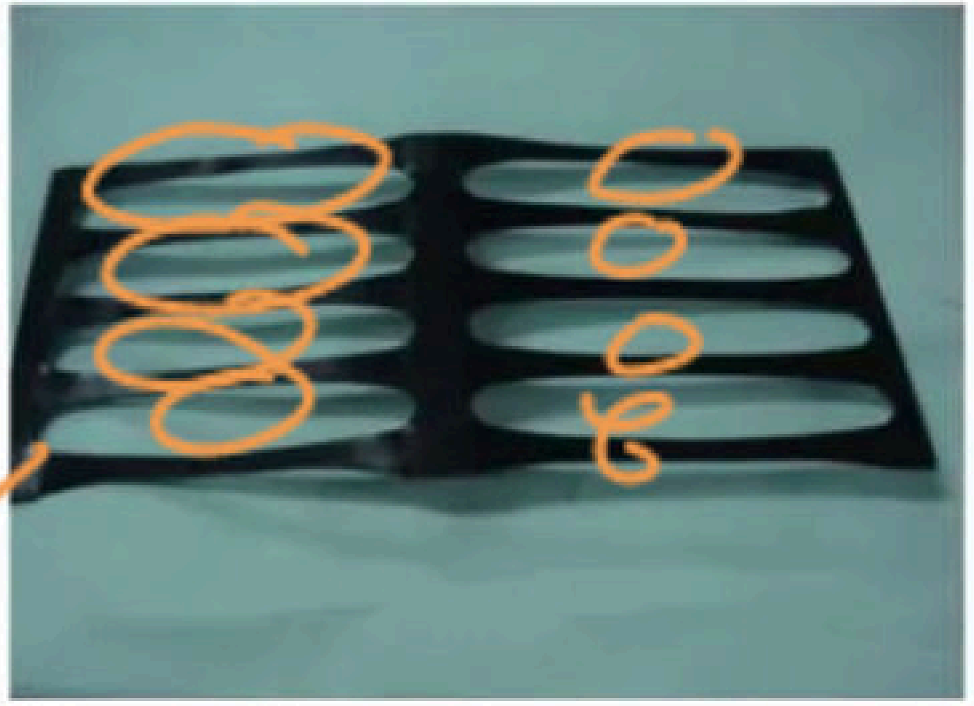
A woven geotextile fabric

Geotextile layer being applied belo

track

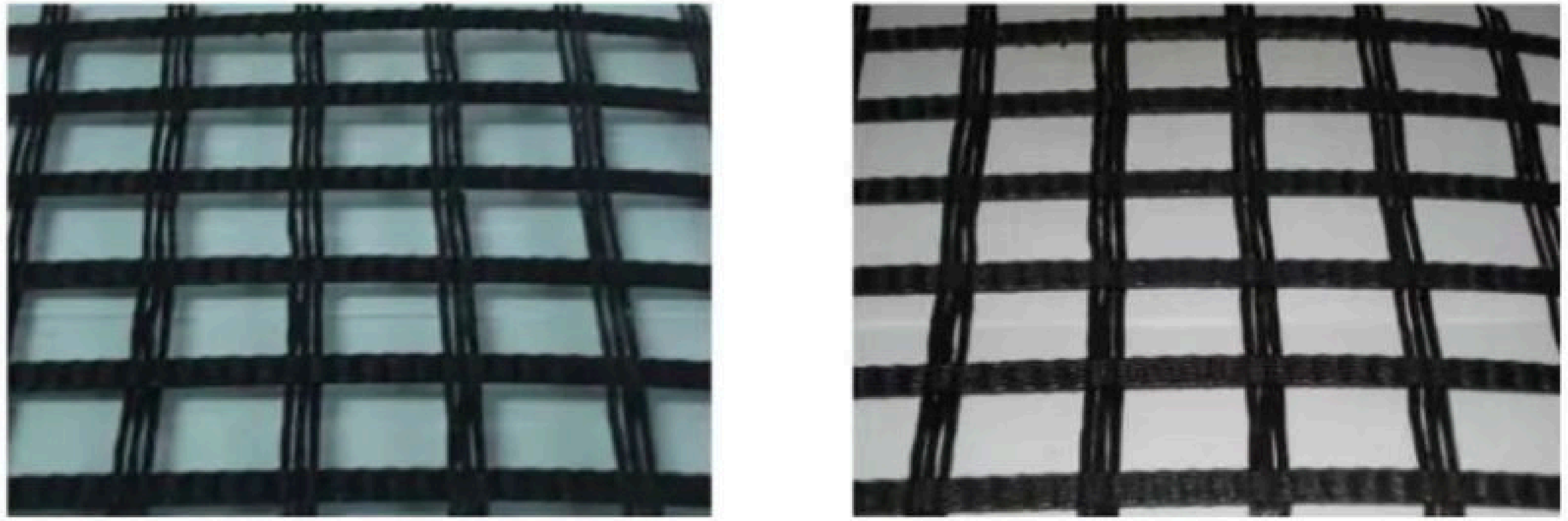
# Geogrids

- Sheet like products with open apertures. Excellent interlocking with soil. High strength products, used for reinforcement.
- The geogrids are of several varieties. The extruded grids have low strength (e.g. Netlon India products). Stretched grids (e.g. Tensar products) are made by stretching process. More recently several types are made by knitting, welding process, etc.
- Uniaxial products used as reinforcement layers in retaining walls and embankments
- Biaxial products used are used in road bases, below rail tracks, ground reinforcement

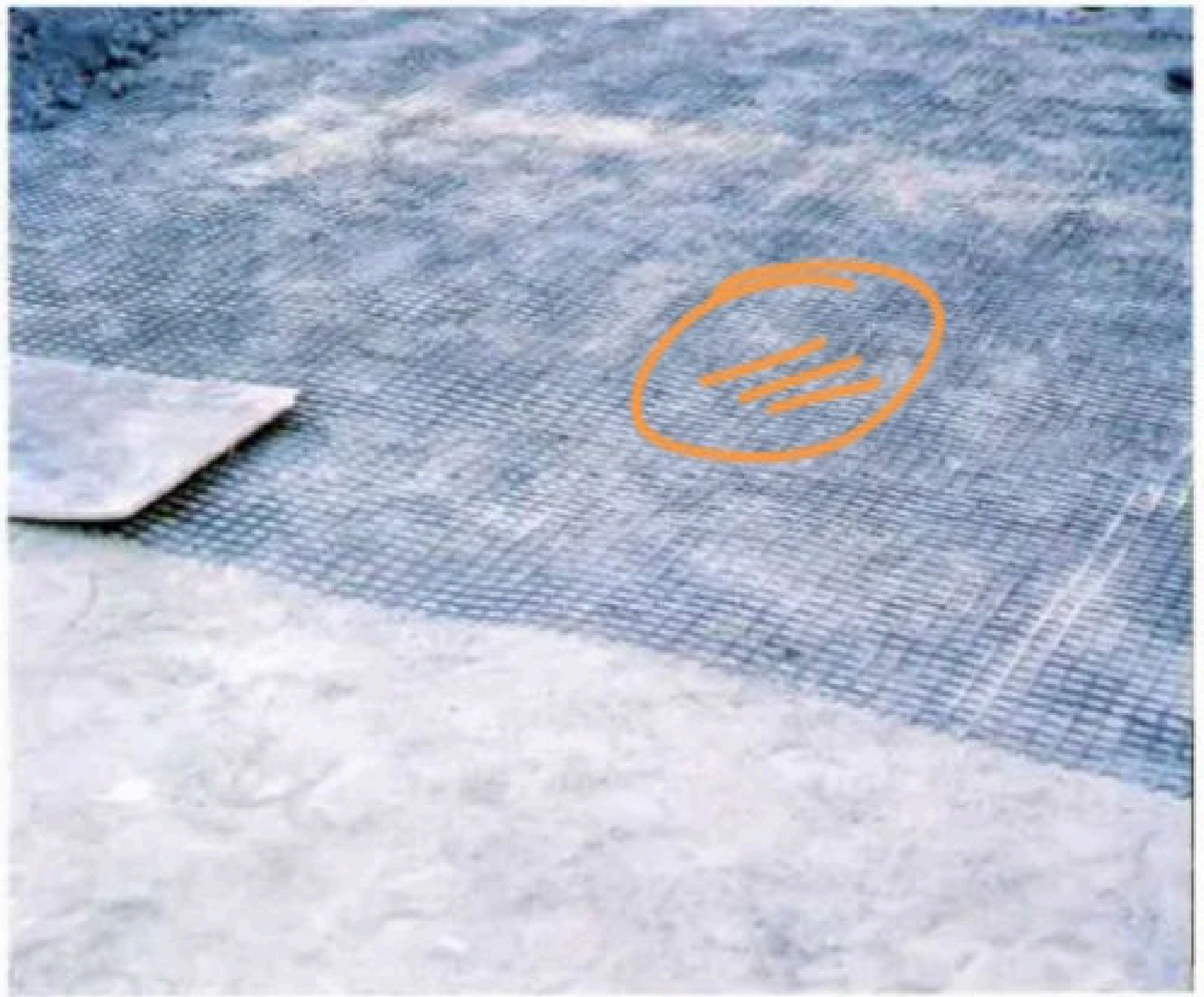


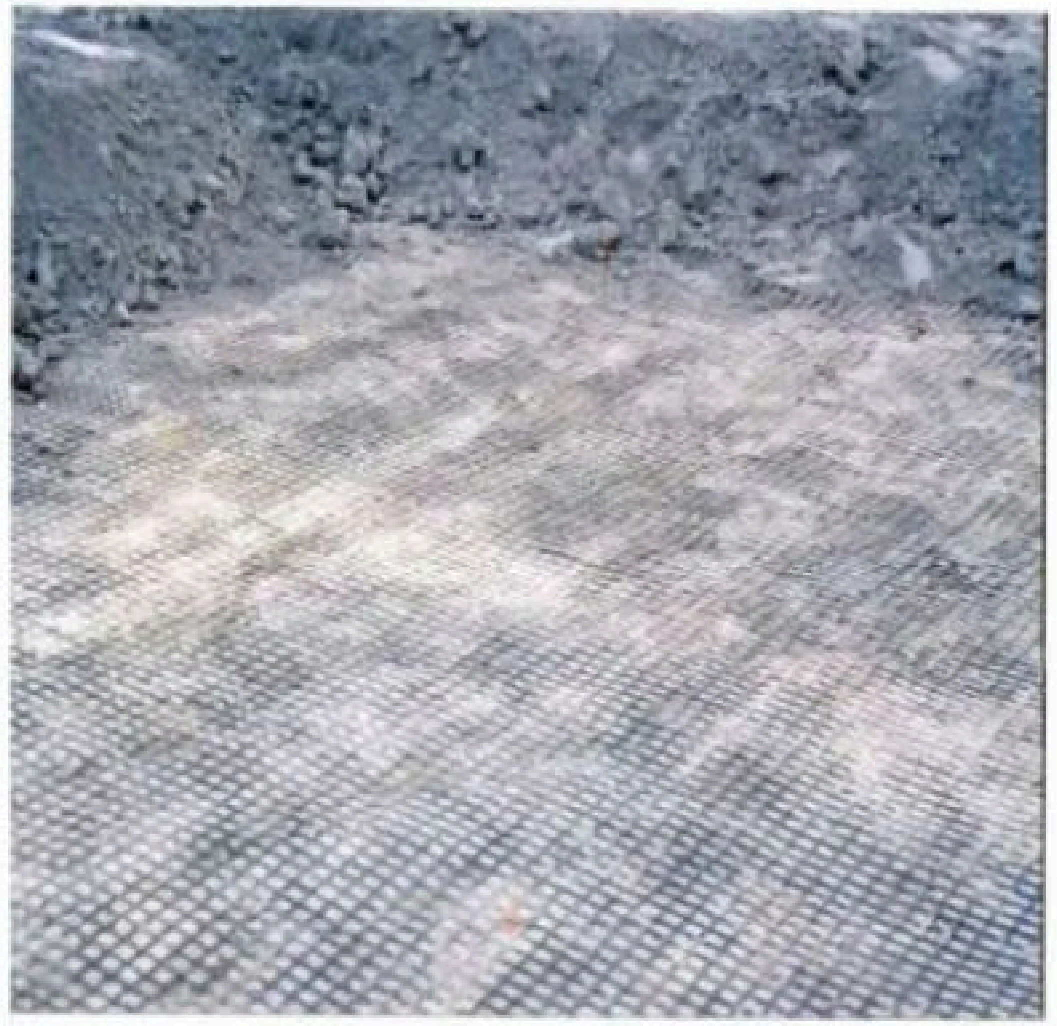
Stretched uniaxial geogrid

Stretched biaxial geogrid

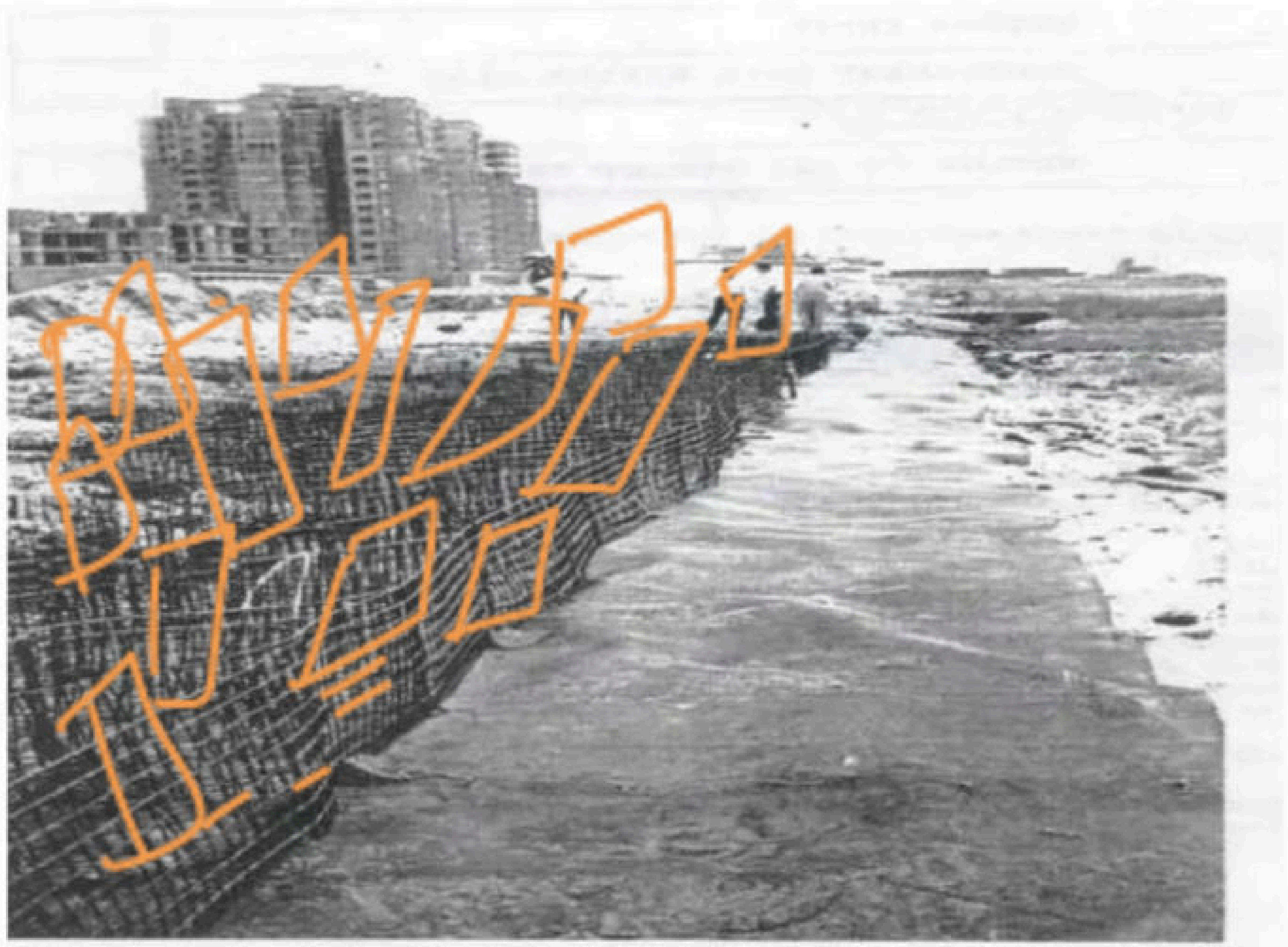


Knitted polyester geogrids





**Geogrid reinforcement in pavements**



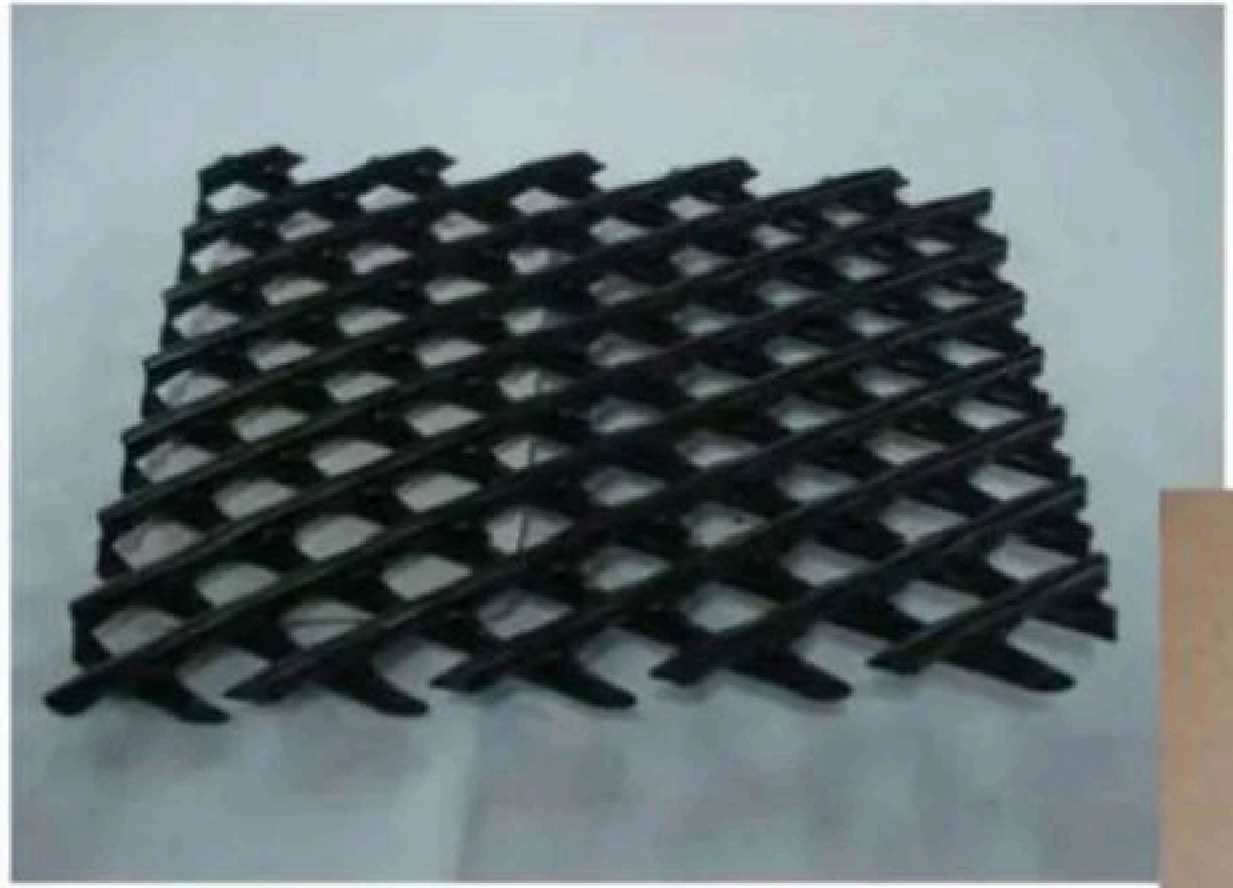
Innovative use of geogrids for shore protection at Navi Mumbai

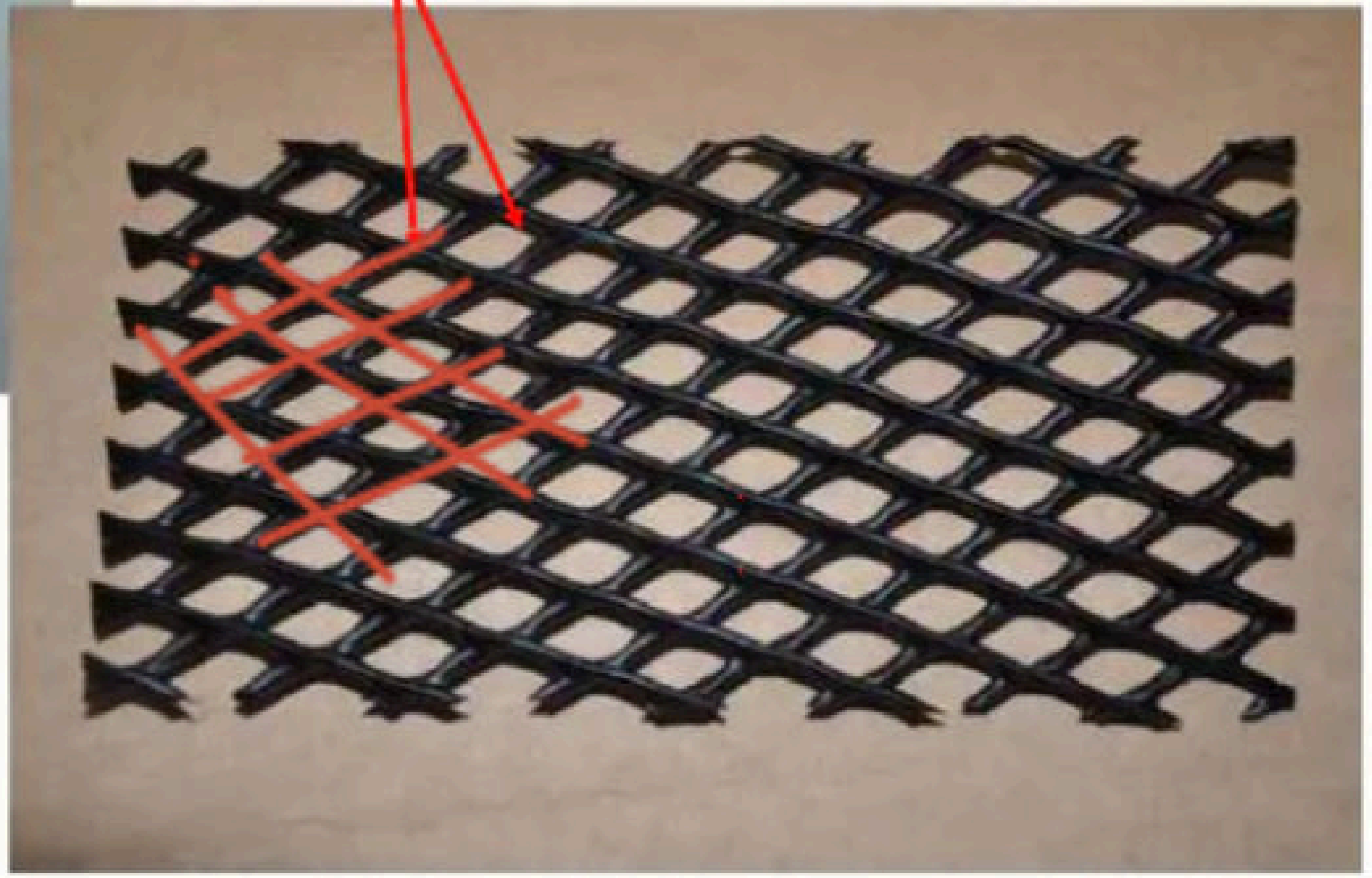
# GEONE

GEONETS

- Geonets are also planar products
- Consists of ribs in two directions
- Apertures are of diamond shape
- Ribs in the two directions are at different planes
- Thickness of geonets is larger than that of geogrids
- Geonets are also referred to as geospacers

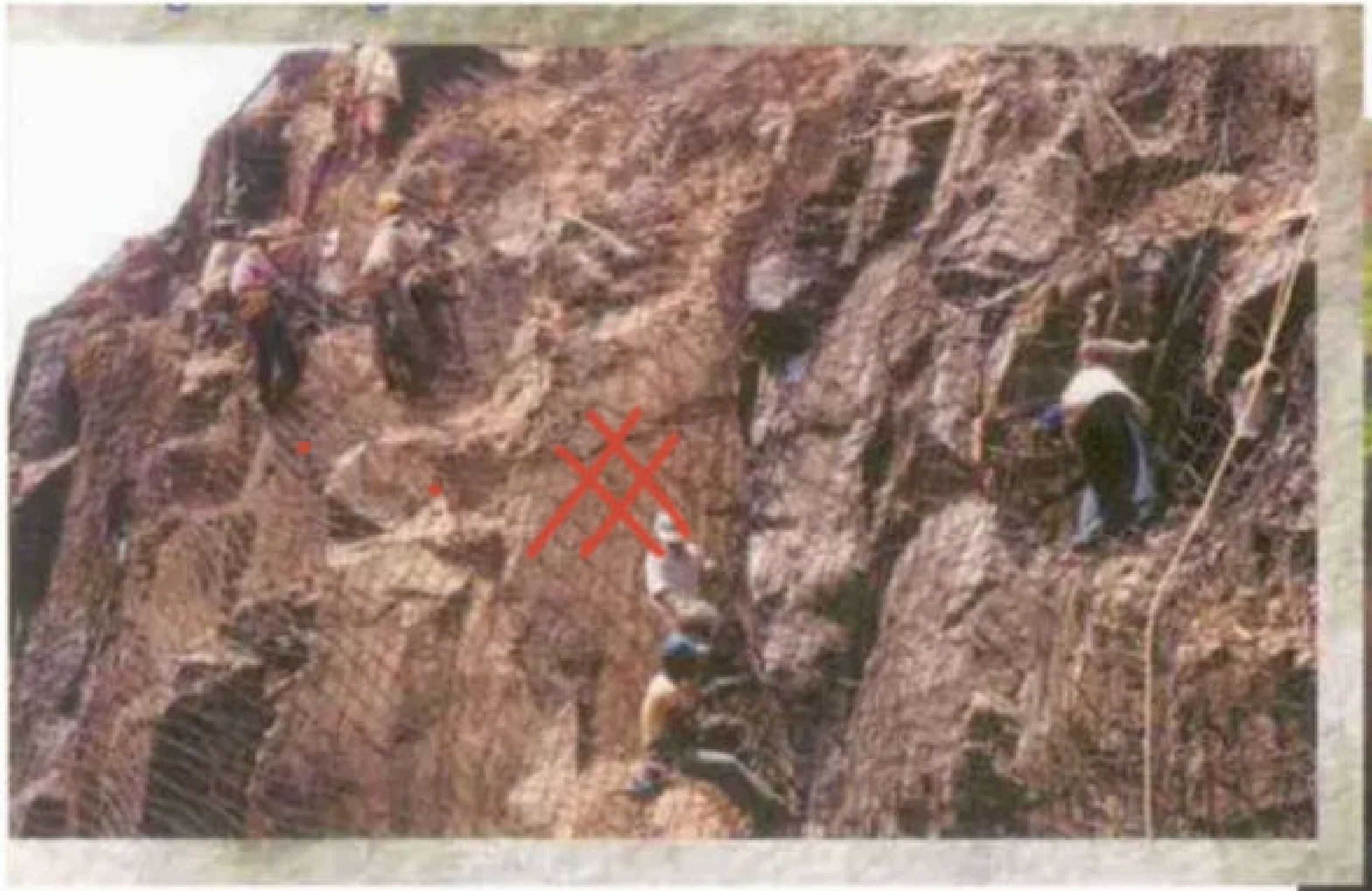
# TYPICAL



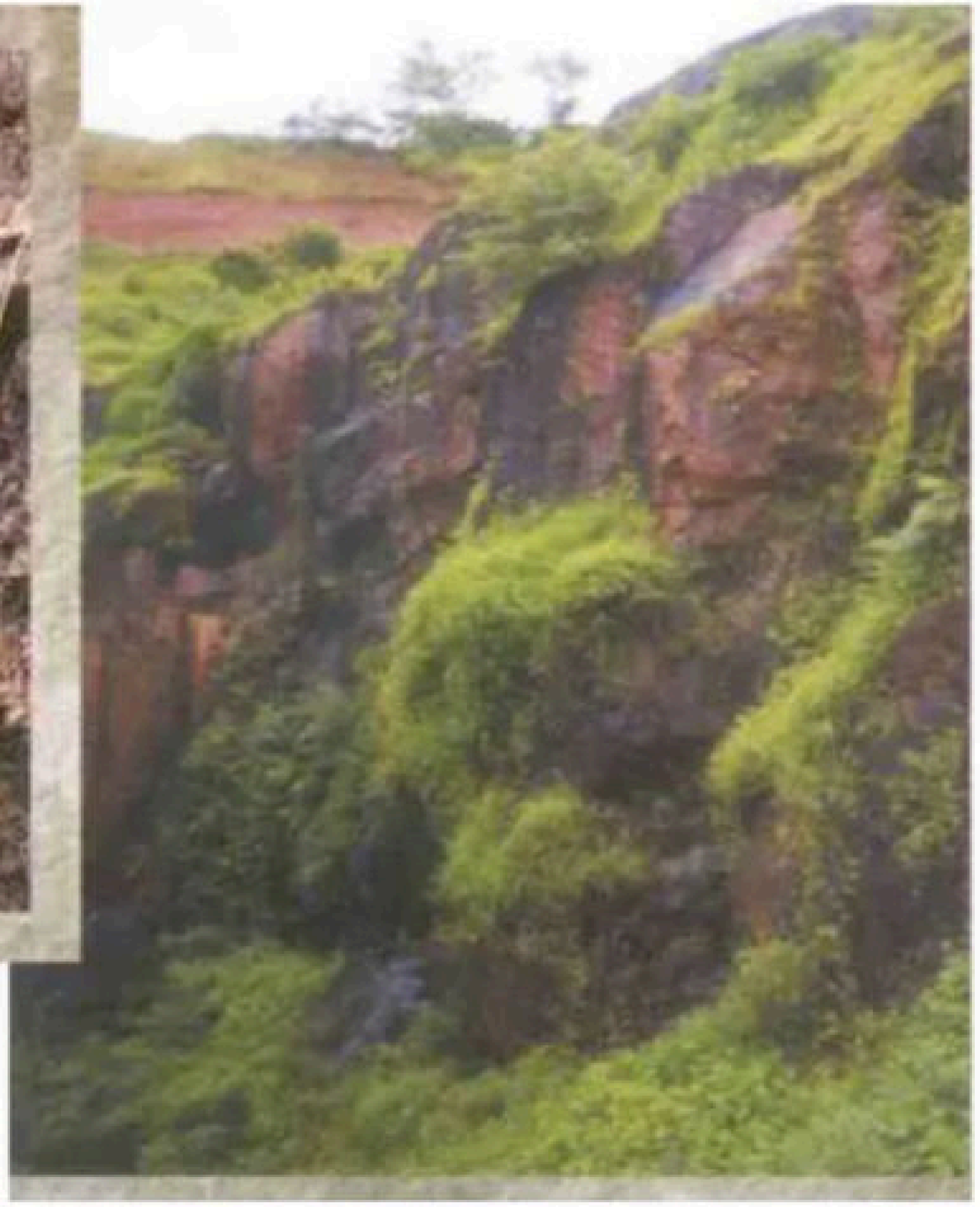


# GEONET APPLICATIONS

- Erosion control – ribs act as small check dams to slow down the surface runoff – decreases erosion potential of water
- Drainage layers – water flows along the geonet because of large thickness



Laying of boulder net

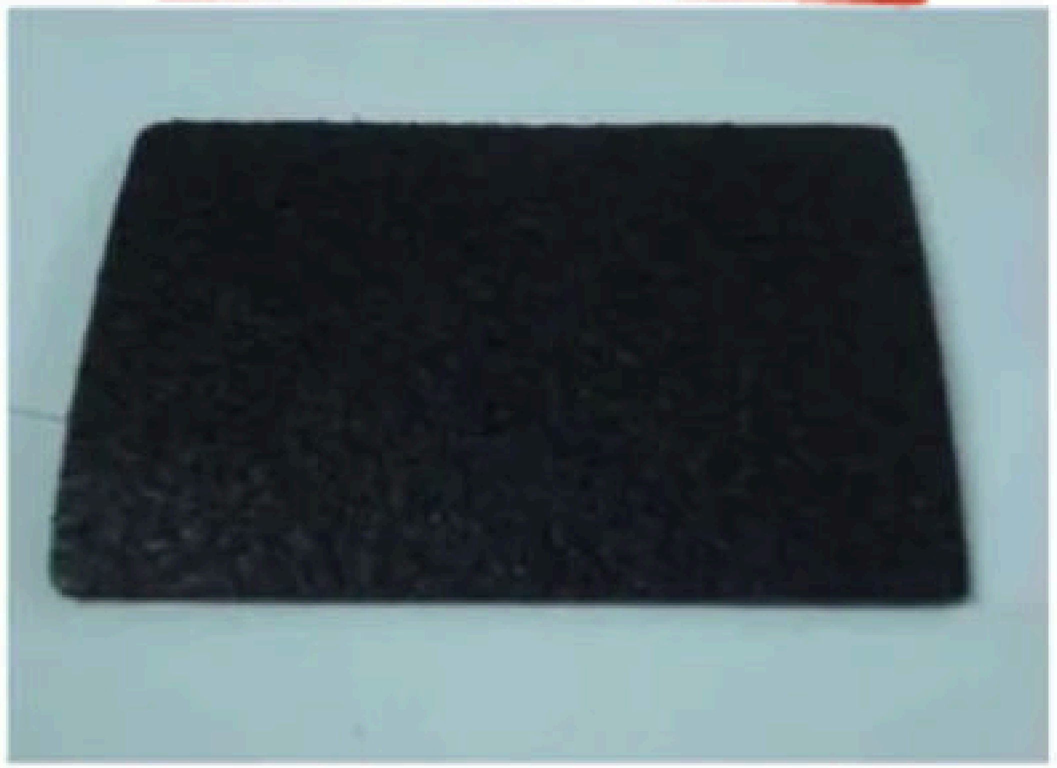


Vegetation growth after two seasons

Courtesy: M/s Garware Wall Ropes Ltd., Pune

# GEOMEMBRANES

- Thick impervious plastic sheets
- Thickness .5 mm to 3 mm approximately
- To contain liquids and gases



Rough surface texture

Smooth – double sided membrane

# APPLICATIONS OF GEOMEMBRANES

- Landfill lining
- Canal lining
- Tunnel lining

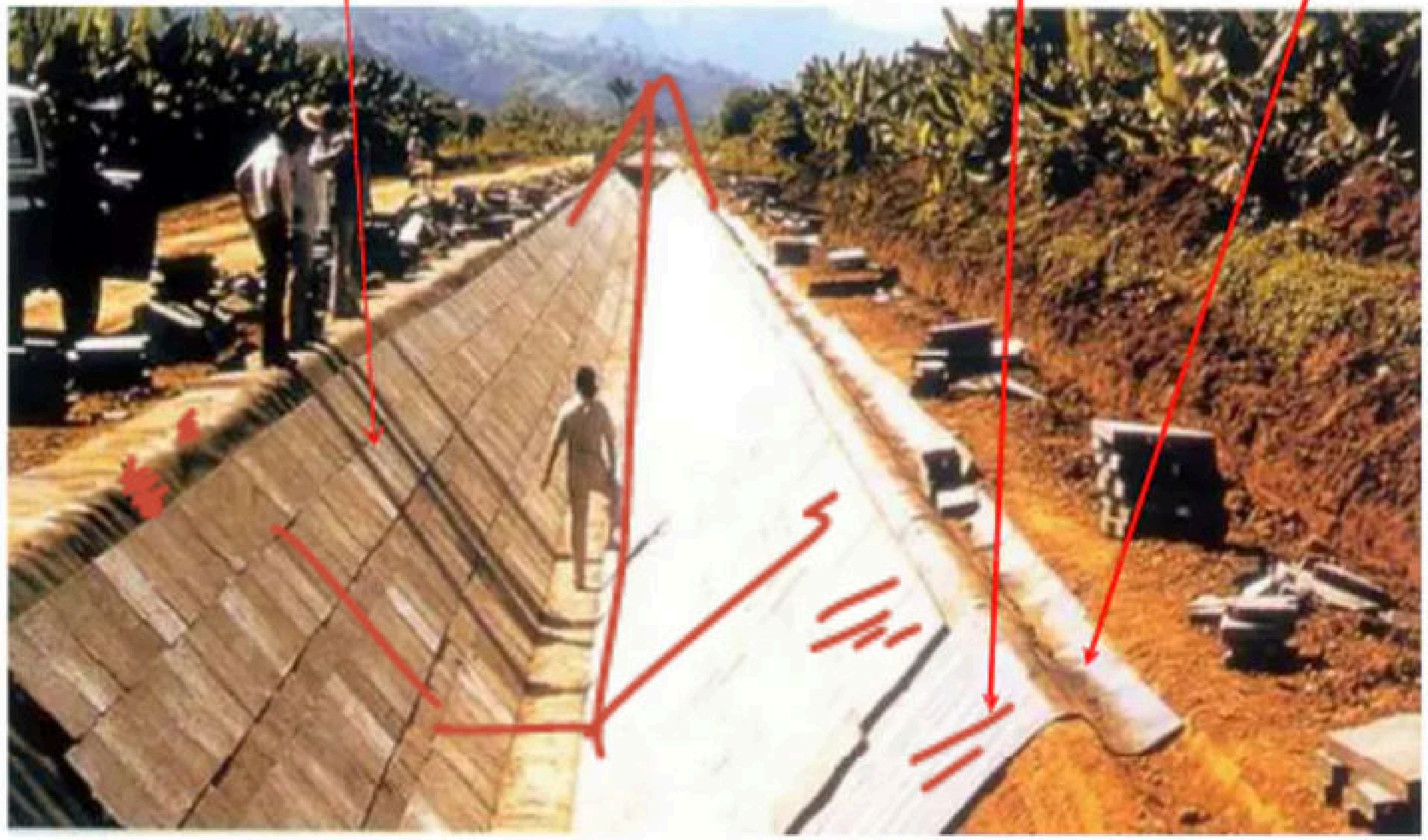
# Geomembrane in a

# Canal lining using geomembranes

Concrete lining of surface

geomembrane

Anchor trench



# Tunnel lining for moisture

protection

# Pre-fabricated vertical drains to accelerate the pre-consolidation of soft clay soils

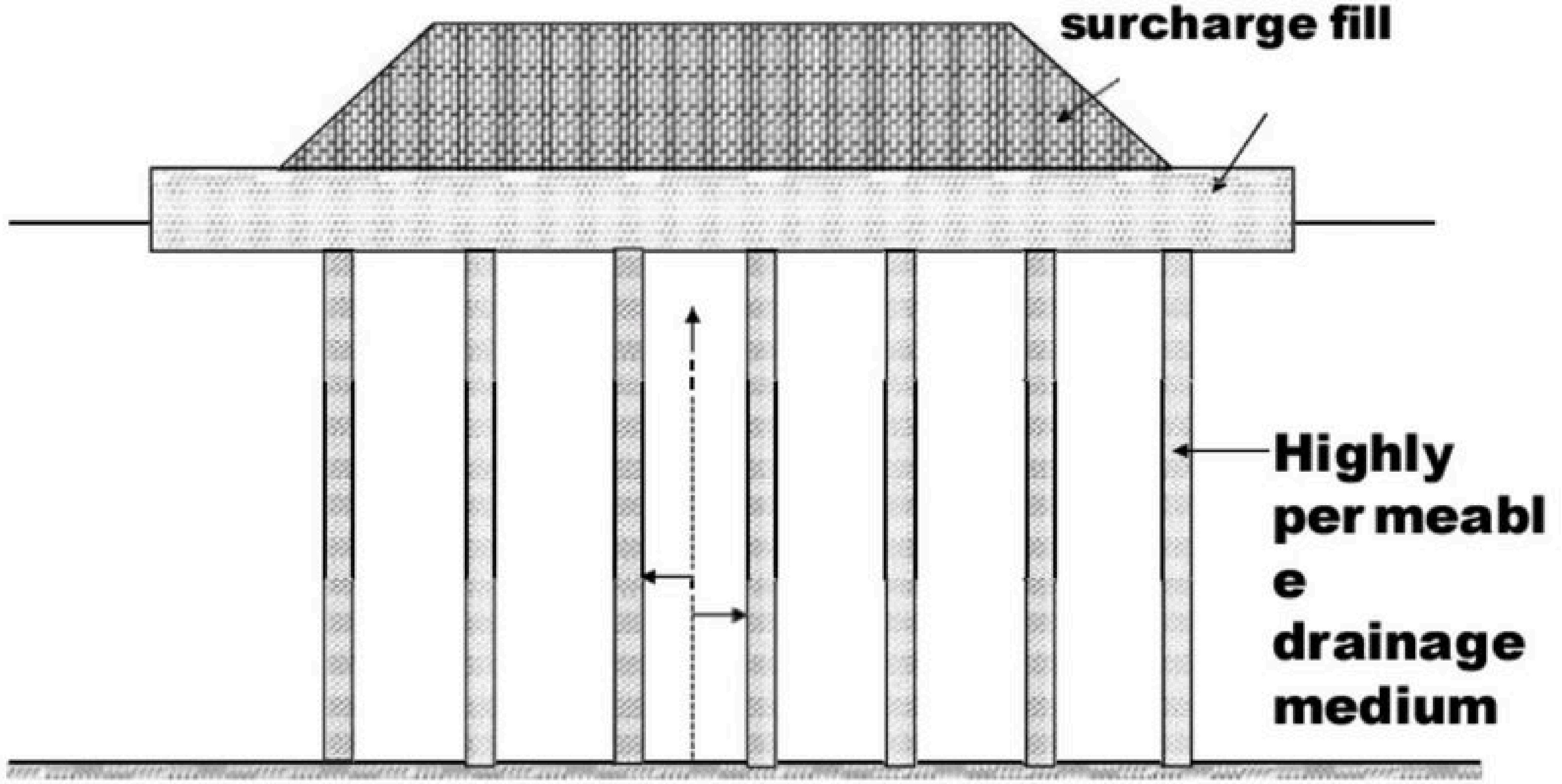
$$T_v = \frac{c_v t}{d^2}$$

$$T_v \Rightarrow f(U\%)$$

$$t = \frac{T_v d^2}{c_v}$$

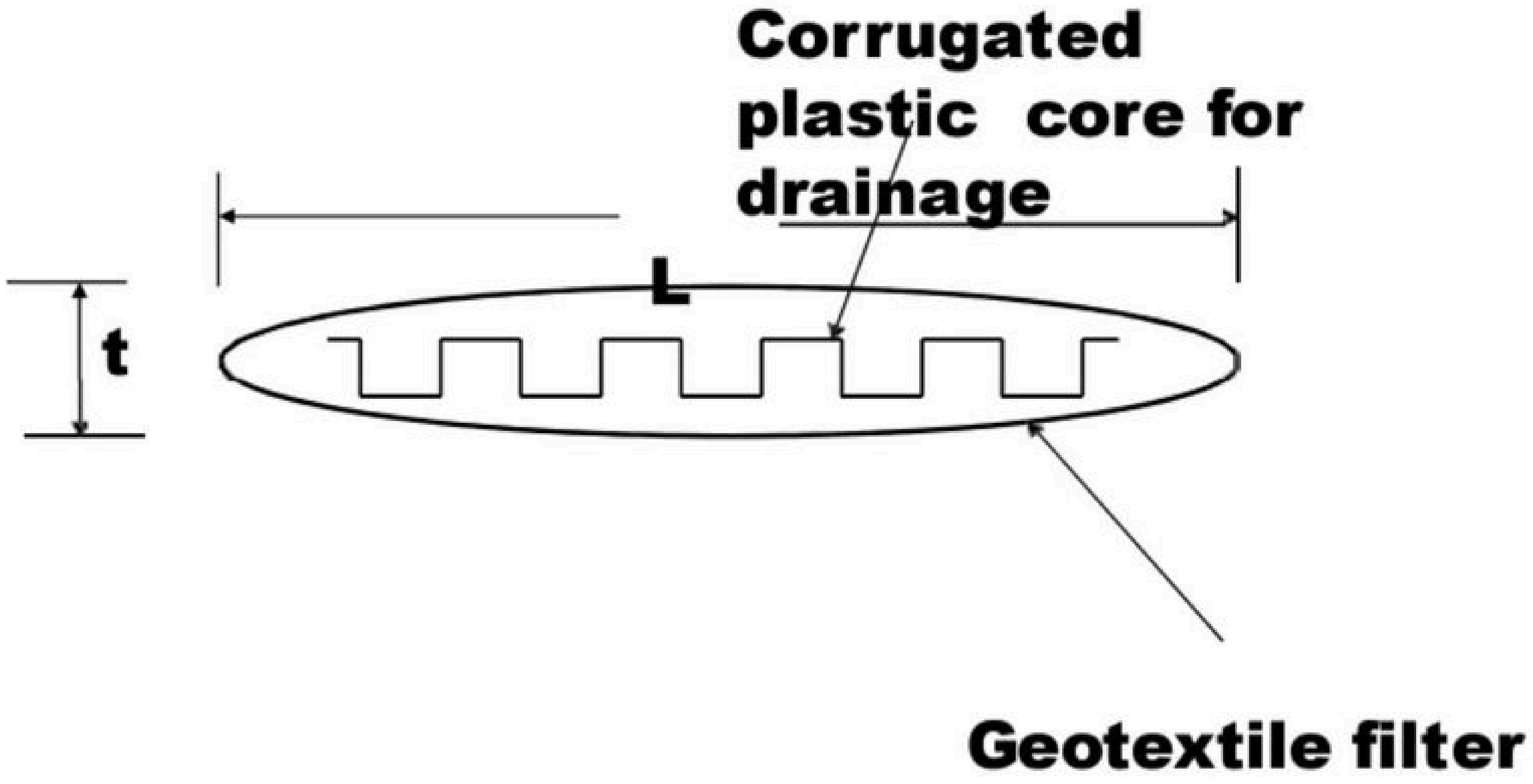
**$T_v$ = time factor**
**t = time**
**$c_v$ = coefficient of consolidation**
**d = drainage path length**
**U% = degree of consolidation**

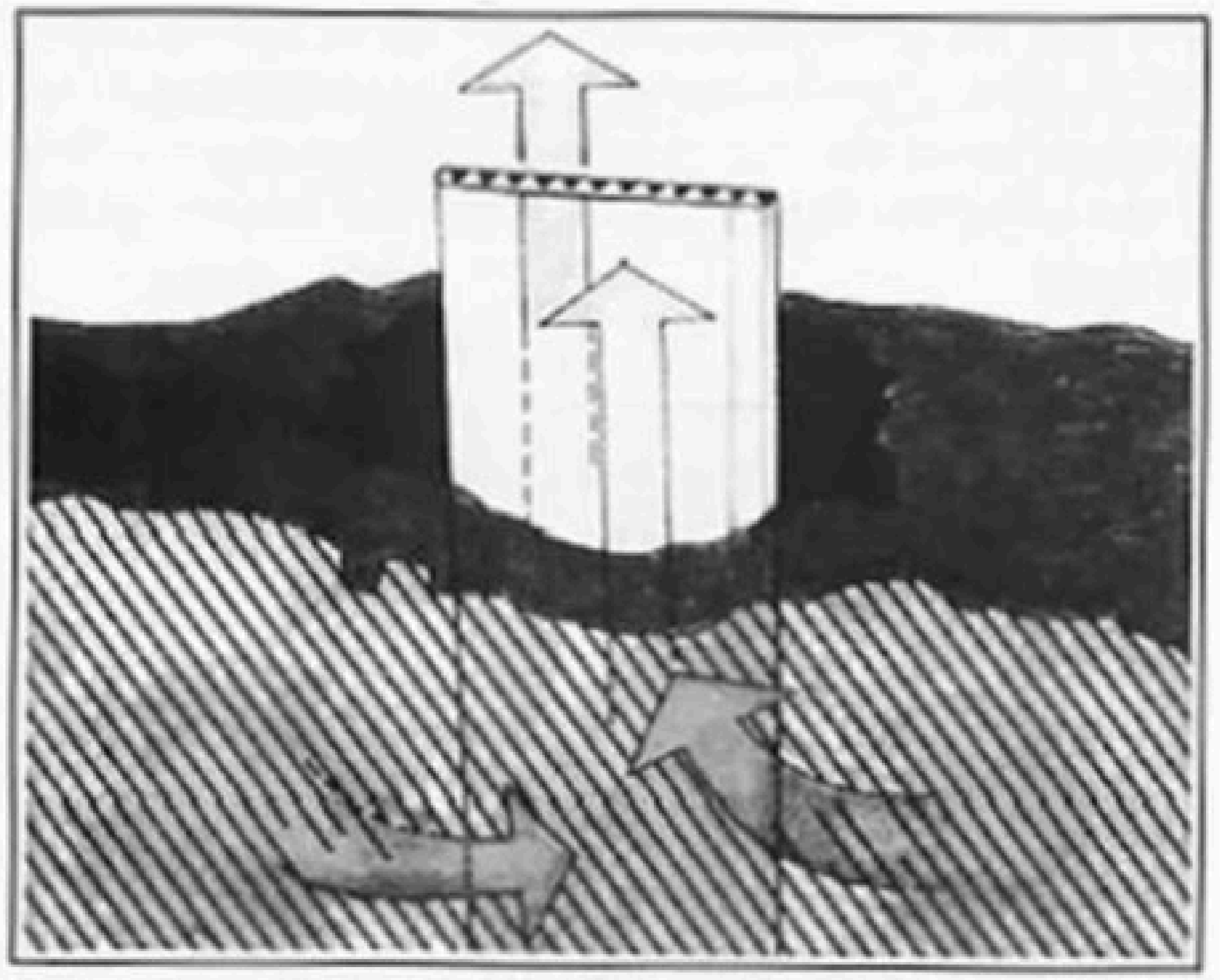
# Reducing the flow path length to accelerate rate of consolidation



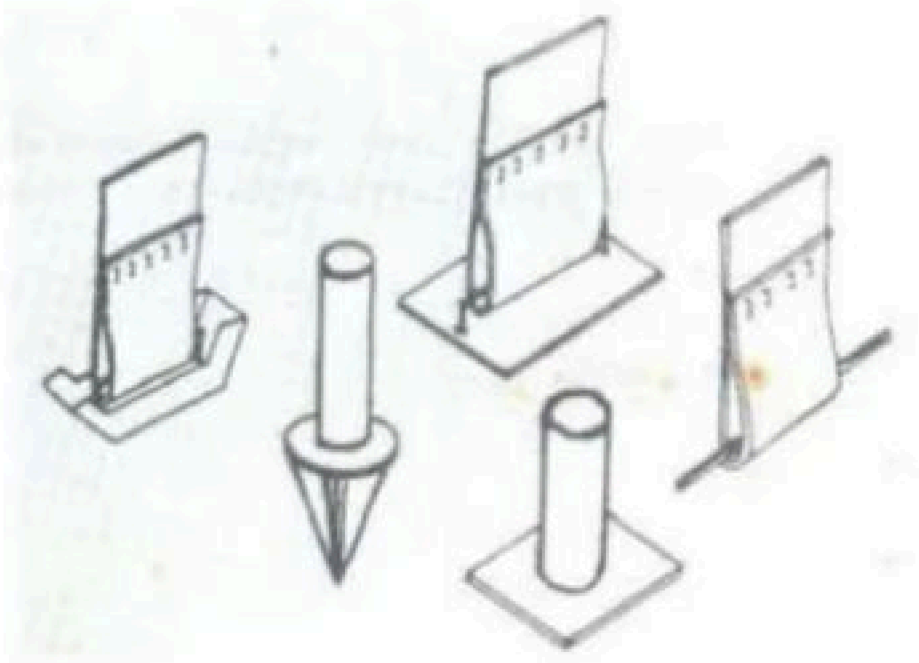

PFD

# PVDs for pre-consolidation





Pore water flows laterally to the wick drains and is carried through the core



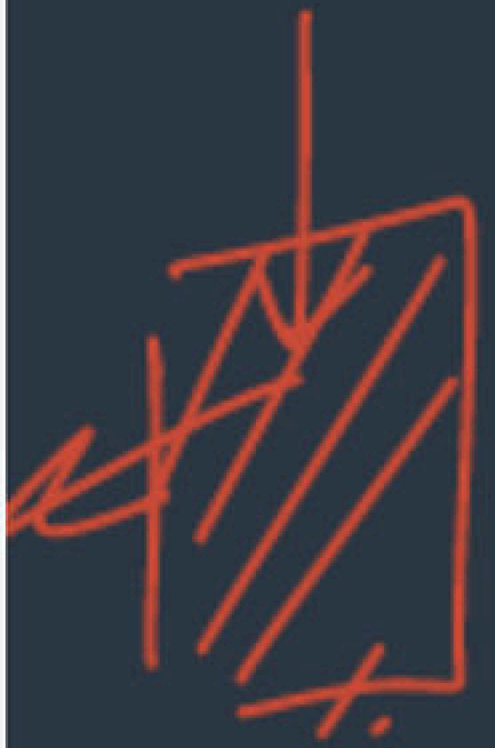
Connection arrangements for wick drain installation

Installation of PVDs at a construction site – notice the connection of PVD with the anchor plate

PVD being pushed into the ground

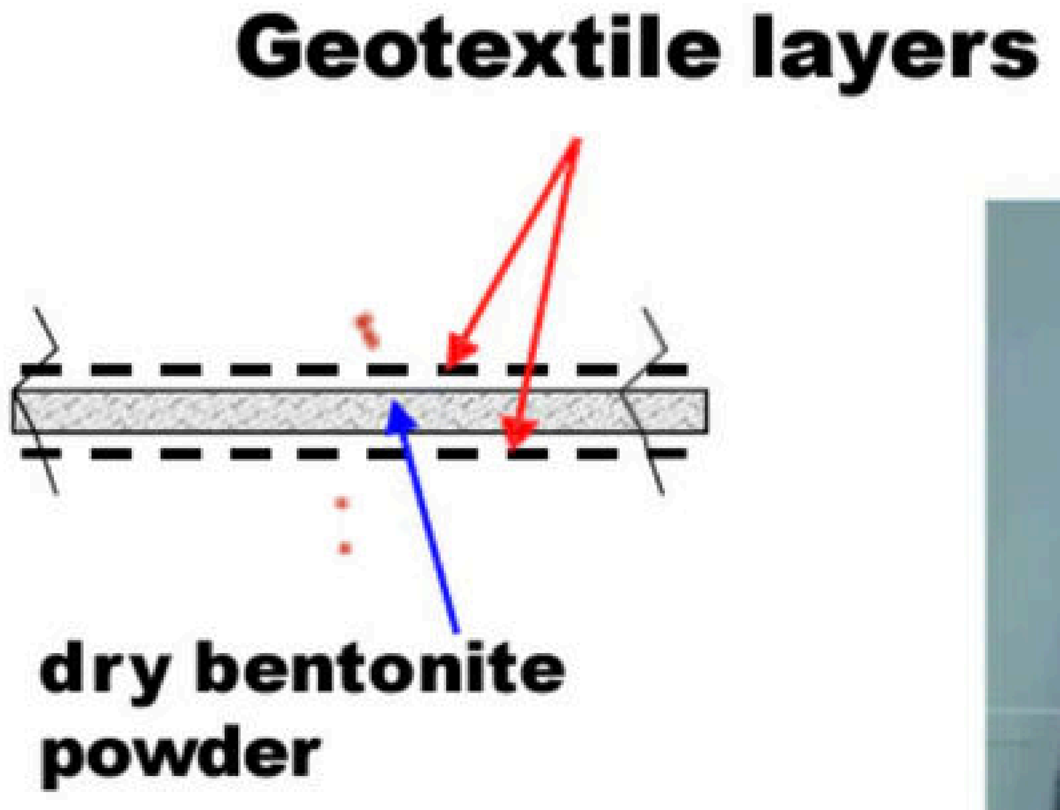
General view after installation of PVD' s at a
site

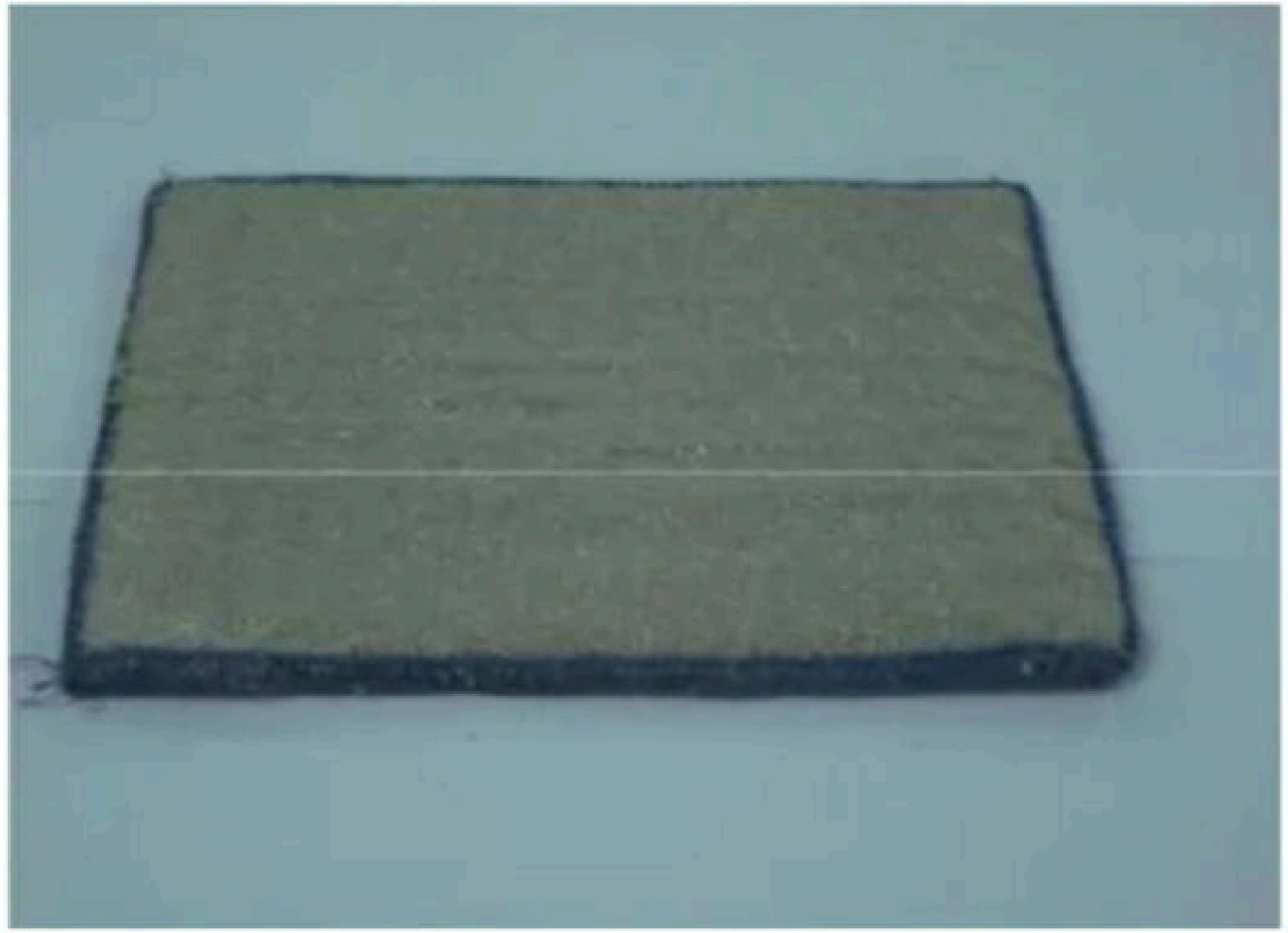
# Geosynthetic Clay Liners



- Consist of a core of bentonite clay sandwiched between layers of thick non-woven geotextile
- Applied below and above geomembrane layers in landfills
- Self-repair mechanism
- Bentonite expands when fluid leaks through punctured geomembrane – closes the gap

# Geosynthetic Clay Liner



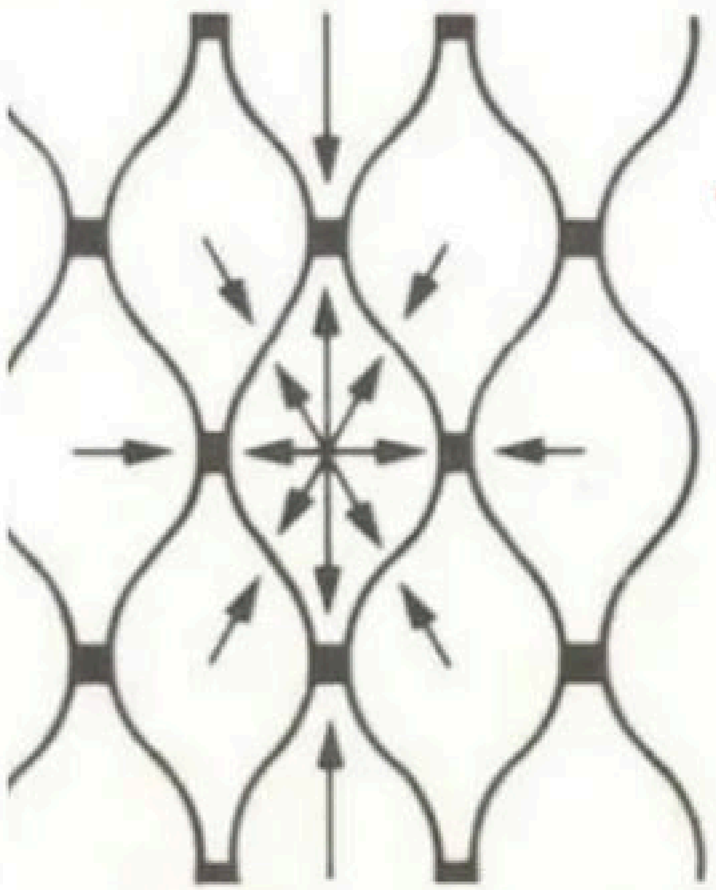



# Question

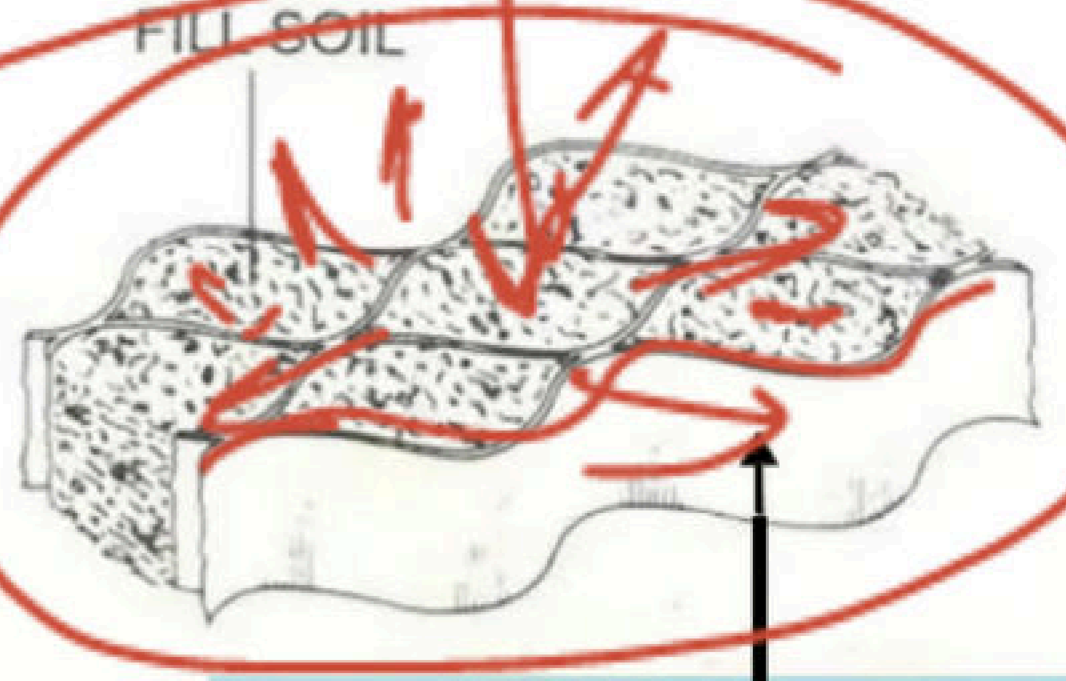
S **Sagar**

sir aap ye bol rahe ho

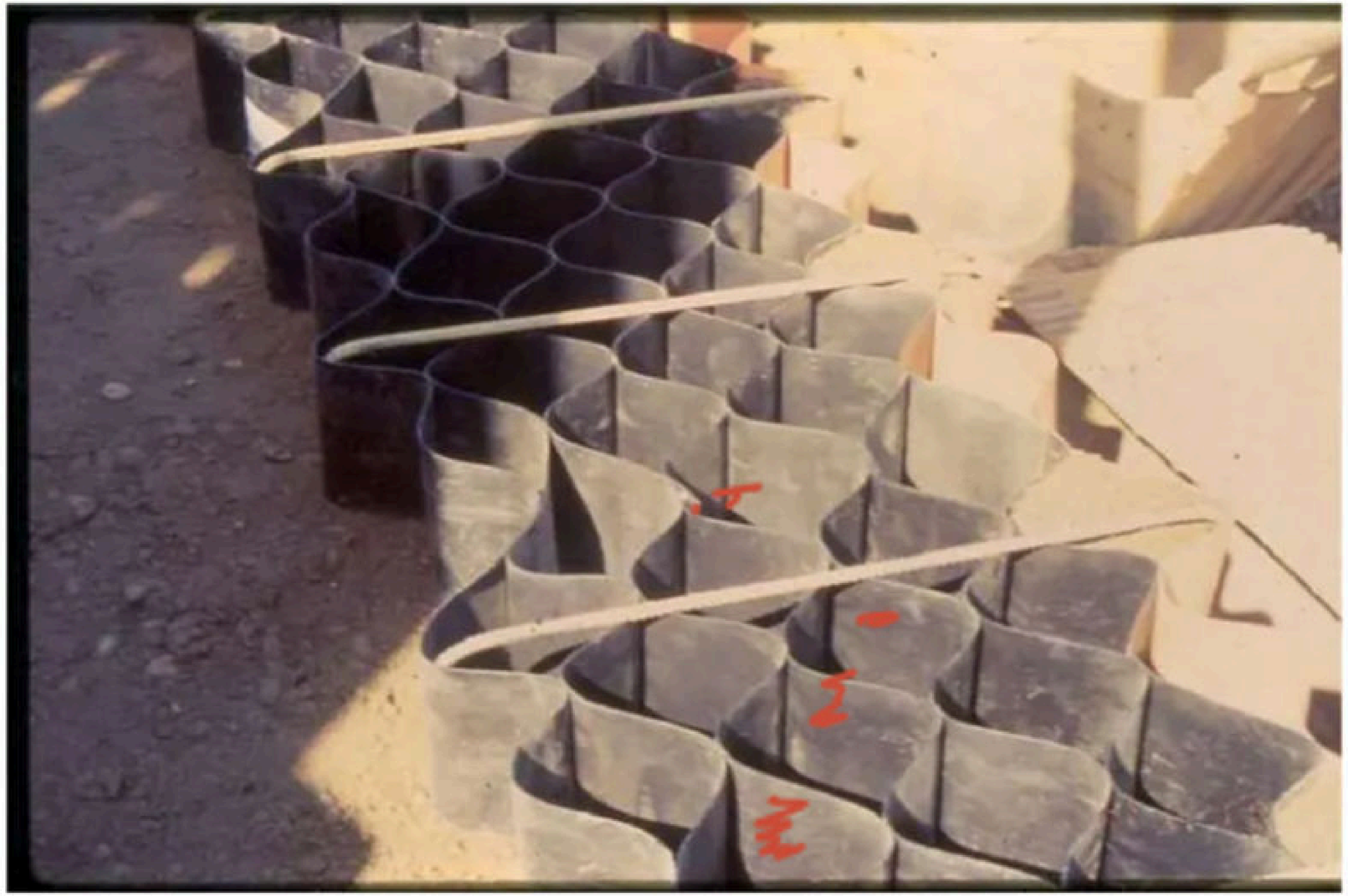
# GEOCELL – 3d



Plan view showing the mechanism of confinement




Iso-metric view of a geocell layer



Photograph of an expanded
geocell

# Advantag
es

- Easy to transport
- Any fill material can be used
- All round confinement to soil
- Semi-rigid layer (very stiff support)
- Spreads loads over a large area
- Excellent support even under cyclic loads.

# APPLICATIONS

- Erosion control
- Steep slopes and retaining walls
- Sub-base support
  - **Road bases**
  - **Railway tracks**
  - **Container yards**

## Use of geocells for construction of unpaved road Factory

Preparation of ground

Stapling to join different geocells

Stretching of the geocell layer

Stone aggregate filled in geocell pockets

Compaction by a 10 tonne roller

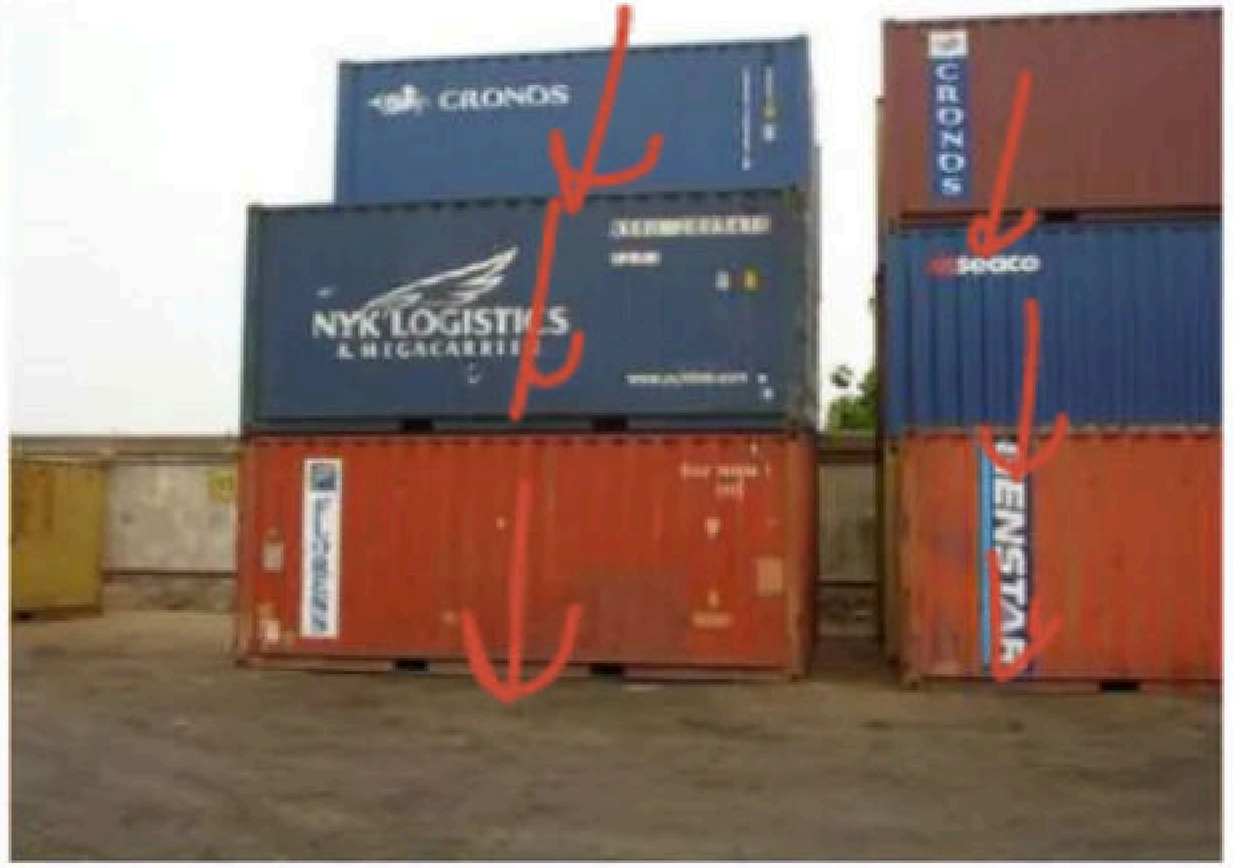
Geocells used for construction of a steep slope

Vegetation taking root through geocell pockets

Typical mud wave formation in container yards due to heavy loads and extremely soft subgrade soil

Geocell layer laid on the geotextile separator and filled with stone aggregate

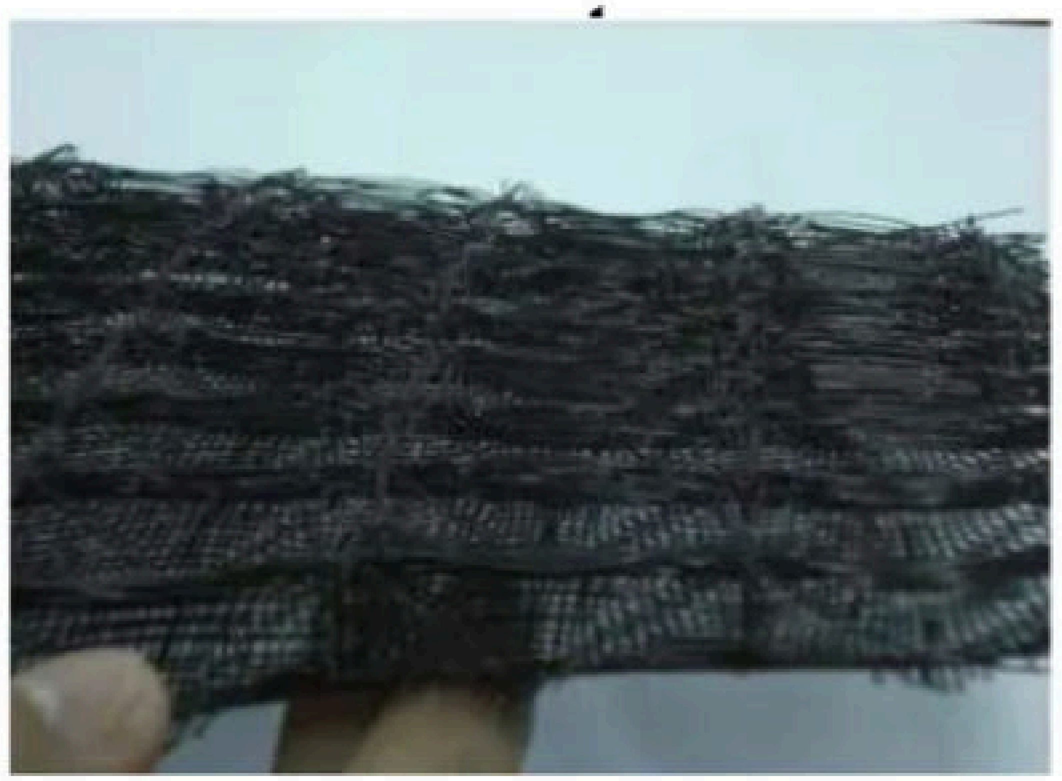
Container yard 3 years after geocell treatmen



# Some more pictures of the same yard

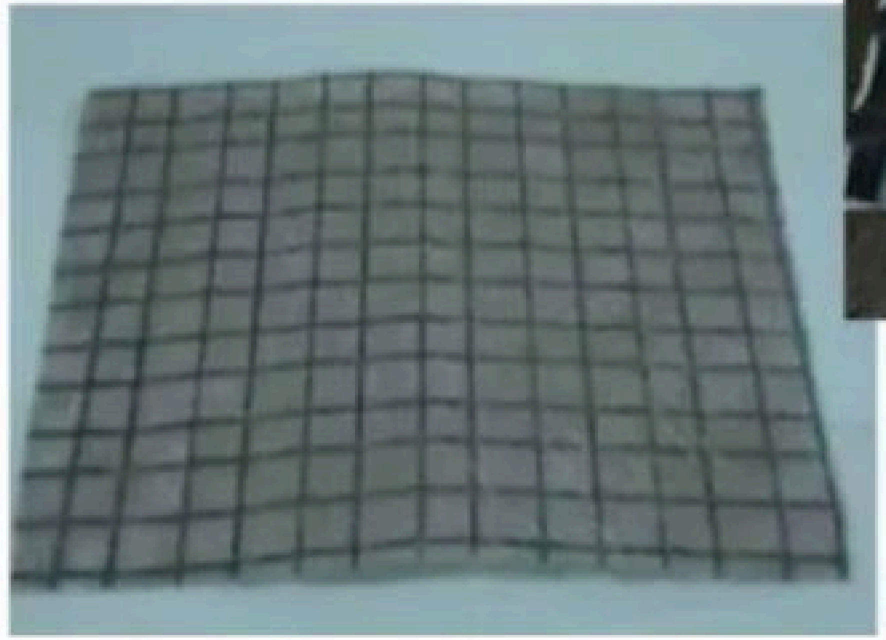
Perfectly level surface – minor damage in paver blocks

# Polymeric erosion control



# Geocomposites

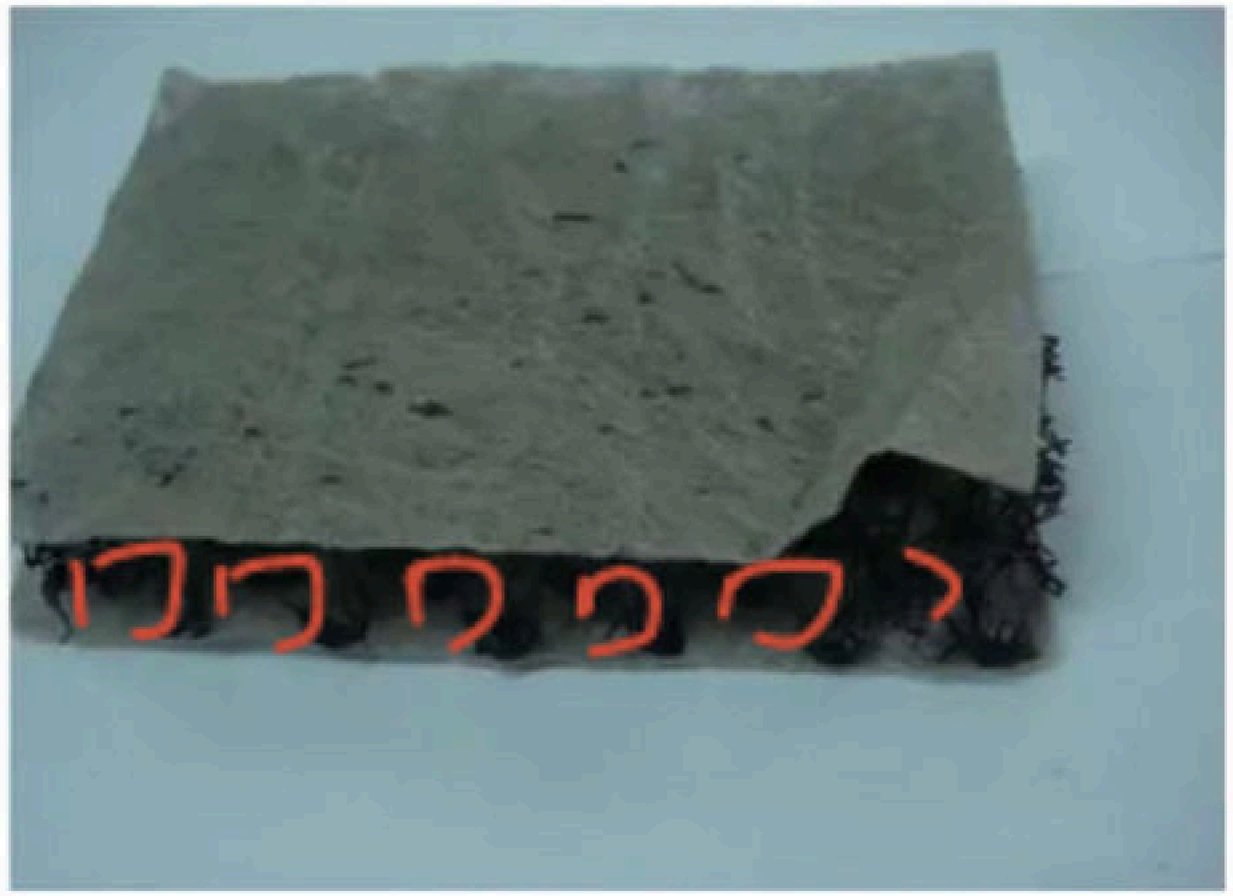
- Combination of two different types of geosynthetics to take advantage of each



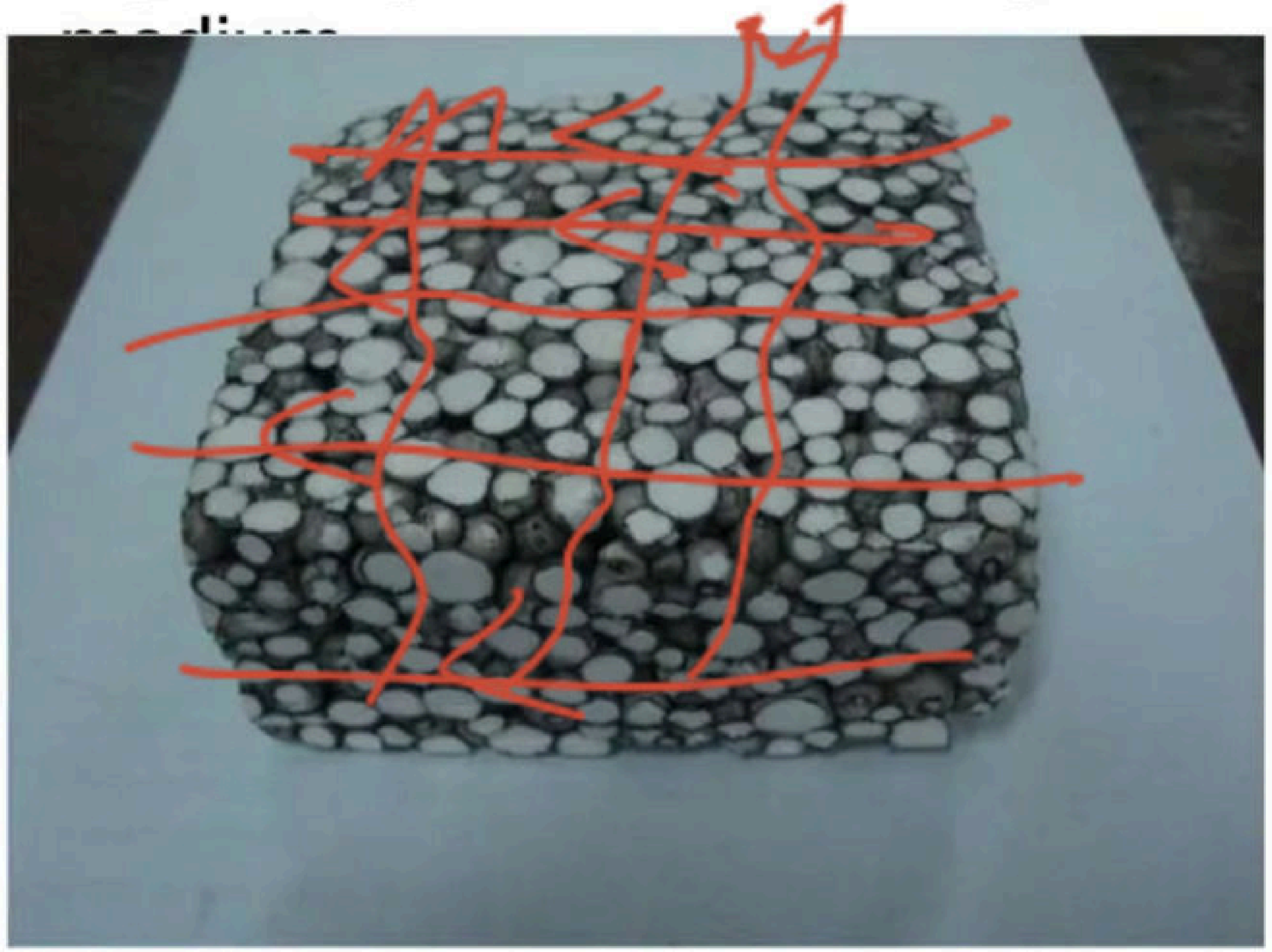
# Geo-others

- Geodrains
- Lightweight fills
- Geopipes
- Geotextile bags & soil encapsulation
- Gabions
- Geosynthetic Encased Stone Columns
- Many others – left to your imagination

# Drainage boards for use in Retaining Walls



Light-weight fill cum drainage



Thick medium made of polystyrene beads

# Gabions filled with sand bags

# Gabions filled with sand bags

SAND FILLED GEOBAGS

PLACEMENT OF GEOBAGS

TYING OF ROPE GABIONS

FINALVIEW