GEOTECNICAL ENGINEERING

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CE 15, 1500045

"Are those who have knowledge and those who have no knowledge alike? Only the men of understanding are mindful. " (Quran, 39:9)

Special THANKS to-

My friend

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CE 15, 1500119

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THEORY

PROBLEMS

Lateral Earth Pressure

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 # Lateral earth pressure: Lateral earth pressure is the pressure that soil exerts in the horizontal direction.

Application of Lateral earth pressure:

1. To design retaining wall.
2. To design sheet pile.
3. To design abutment of bridge.
4. To Design tunnels.
5. To design deep foundation.
6. To design braced excavations.

Factors on which lateral earth pressure depends:

The magnitude and distribution of lateral earth pressure depends on many factor, such as:

1. shear strength parameters of the soil being retained.
2. inclination of the surface of the backfill.
3. height and inclination of the retaining wall at wall-backfill interface.
4. nature of wall movement under lateral pressure.
5. adhesion and friction angle at the wall-backfill interface.

Types of Lateral earth pressure: 15, 09

1. Earth pressure at rest.
2. Active earth pressure.
3. Passive earth pressure.

16,08

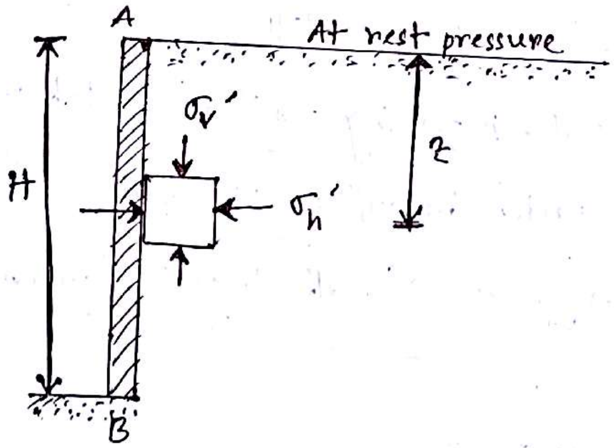
Earth pressure at rest: under conditions where,

1. wall is rigid.
2. wall is at rest.
3. does not move with lateral pressure
4. zero horizontal strain.

the value of the lateral soil pressure is commonly called lateral earth pressure at rest.

co-efficient of earth pressure at rest: The ratio of the lateral earth pressure to the vertical earth pressure at rest condition is termed as co-efficient of earth pressure at rest (K_o)

consider a mass of soil as shown in figure:



$\sigma_v' = \gamma z$

$\sigma_h' = K_o \gamma z$

The mass is bounded by a frictionless wall of height AB. A soil element located at a depth z is subjected to a vertical effective pressure, σ_v' and a horizontal effective pressure, σ_h'. There is no shear stress on the vertical and horizontal planes of soil element.

$$\therefore K_o = \frac{\sigma_h'}{\sigma_v'}$$

For normally consolidated coarse-grained soil (sand):

[Jaky, (1944)] $K_0 = 1 - \sin\phi'$. where, ϕ' = drained friction angle.

For normally consolidated fine-grained soil (clay/silt):

[Massarch, (1979)] $K_0 = 0.44 + 0.42 \times \left[\frac{PI(\%)}{100} \right]$

For normally consolidated clay:

(Brooker and Ireland, 1965) $K_0 = 0.95 - \sin\phi'$

For over consolidated clay:

$$K_{0(OC)} = K_{0(NC)} \times \sqrt{OCR}$$

where, $OCR = \frac{P_0}{P}$

P_0 = Pre consolidation pressure

P = Present effective overburden pressure.

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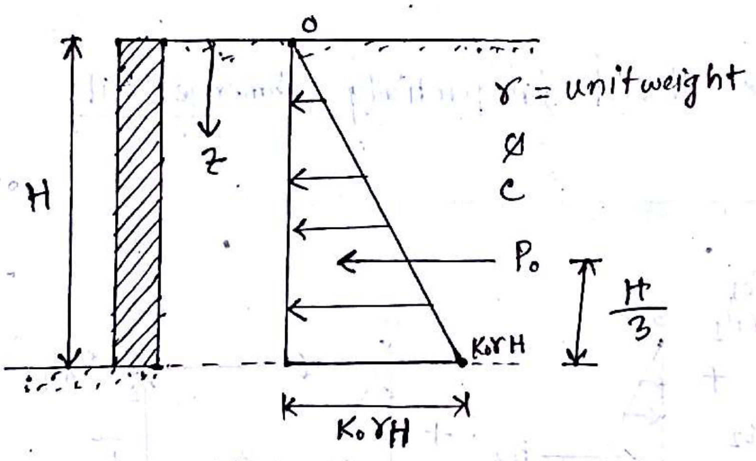
Distribution of lateral earth pressure at rest:



Total force per unit length of wall, $P_0 = \frac{1}{2} \times K_0\gamma H \times H$

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

Location of total force from bottom,

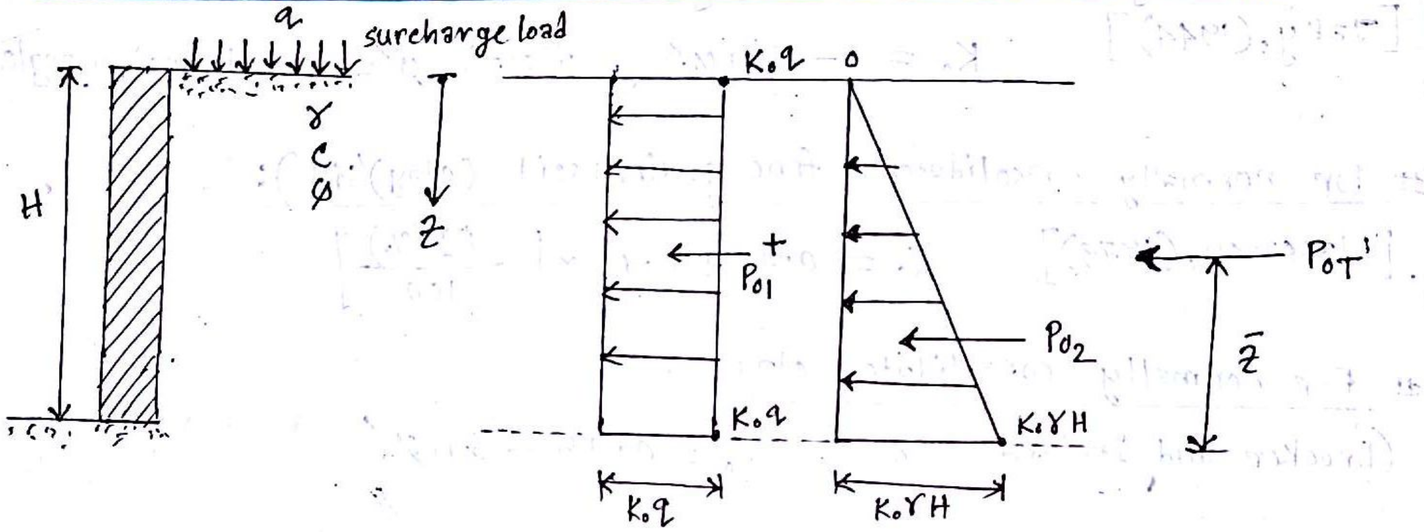
$$\bar{z} = \frac{H}{3}$$

when $z = 0$, $\sigma_h = 0$

$z = \frac{H}{2}$, $\sigma_h = K_0 \gamma \frac{H}{2}$

$z = H$, $\sigma_h = K_0 \gamma H$

Distribution of Lateral earth pressure at rest with surcharge:



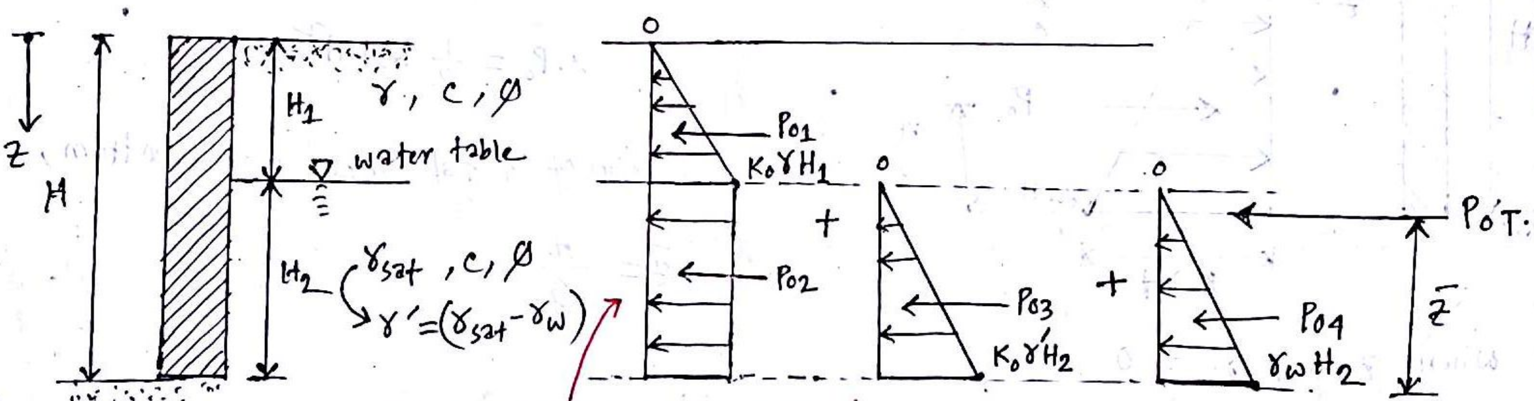
Total force per unit length of the wall, $P_{0T} = P_{01} + P_{02}$

$$\therefore P_{0T} = K_0 q H + \frac{1}{2} \times K_0 \gamma H^2$$

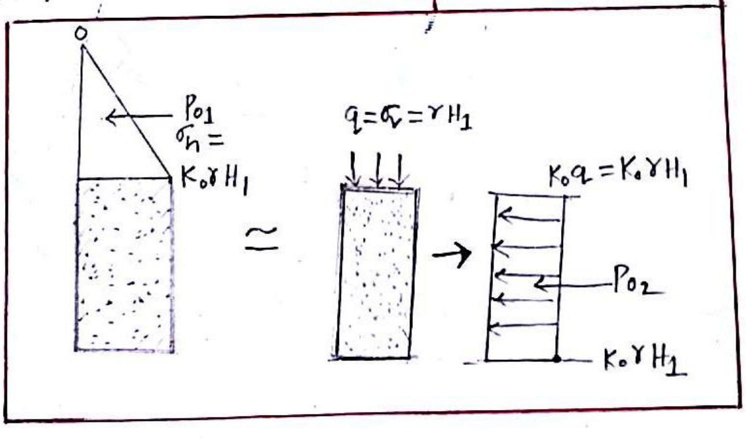
location of the resultant force from bottom,

$$\bar{z} = \frac{P_{01} \times \frac{H}{2} + P_{02} \times \frac{H}{3}}{P_{0T}}$$

Distribution of lateral earth pressure at rest for partially submerged soil:



help:



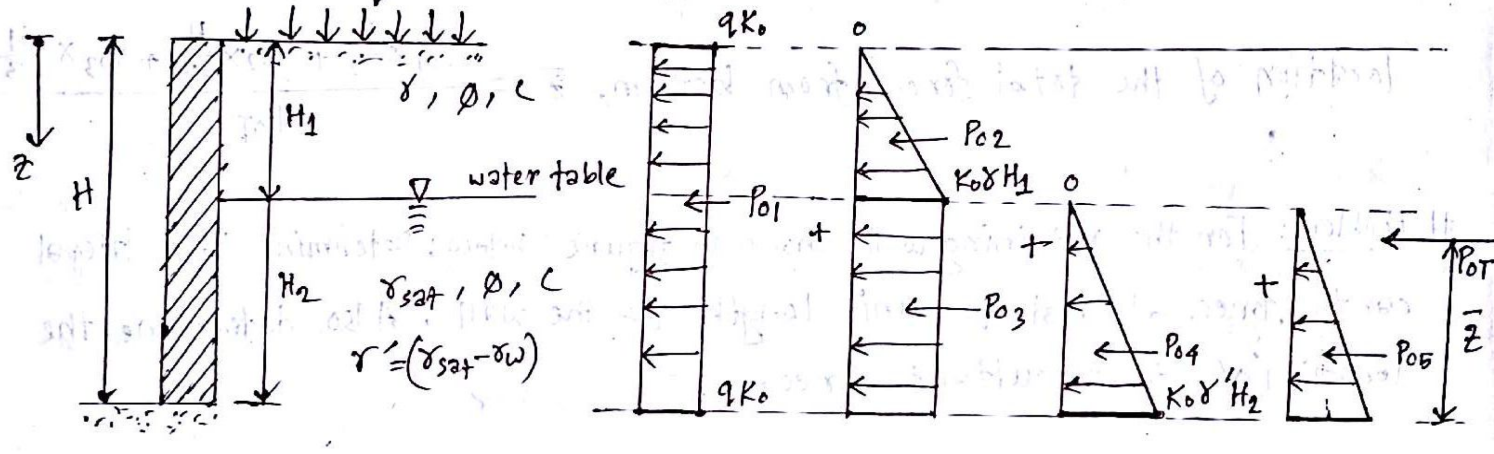
Total force per unit length of the wall, $P_{OT} = P_{01} + P_{02} + P_{03} + P_{04}$

$$\therefore P_{OT} = \frac{1}{2} \times K_0 \gamma H_1^2 + K_0 \gamma H_1 H_2 + \frac{1}{2} \times K_0 \gamma' H_2^2 + \frac{1}{2} \gamma_w H_2^2$$

location of the total force from bottom,

$$\bar{z} = \frac{P_{01} \times \left(H_2 + \frac{H_1}{3}\right) + P_{02} \times \frac{H_2}{2} + P_{03} \times \frac{H_2}{3} + P_{04} \times \frac{H_2}{3}}{P_{OT}}$$

Ditribution of lateral earth pressure at rest with partially submerged backfill and surcharge:



Total force per unit length of the wall, $\therefore P_{OT} = P_{01} + P_{02} + P_{03} + P_{04} + P_{05}$

$$\therefore P_{OT} = K_0 q H + \frac{1}{2} \times K_0 \gamma H_1^2 + K_0 \gamma H_1 H_2 + \frac{1}{2} \times K_0 \gamma' H_2 + \frac{1}{2} \gamma_w H_2^2$$

location of the total force from bottom,

$$\bar{z} = \frac{P_{01} \times \frac{H}{2} + P_{02} \times \left(H_2 + \frac{H_1}{3}\right) + P_{03} \times \frac{H_2}{2} + P_{04} \times \frac{H_2}{3} + P_{05} \times \frac{H_2}{3}}{P_{OT}}$$

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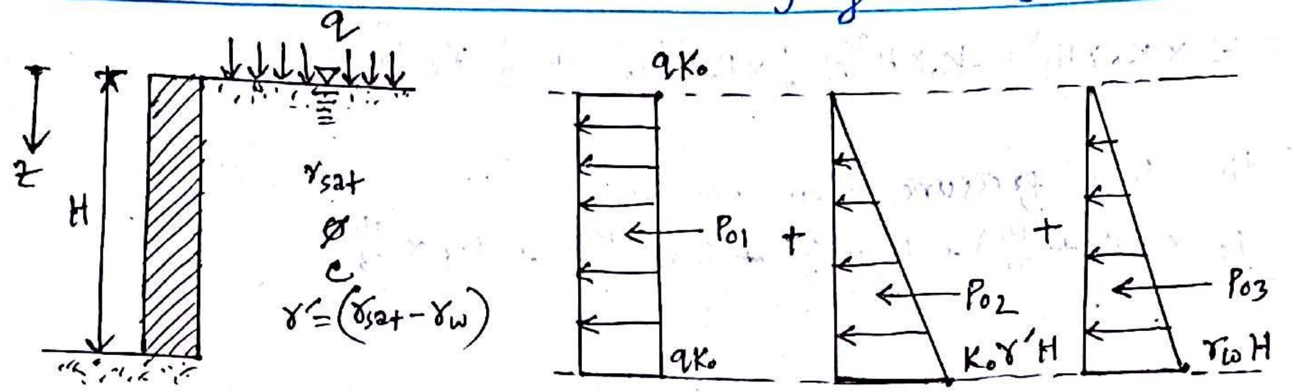
Distribution of Lateral earth pressure at rest with fully submerged backfill and surcharge:

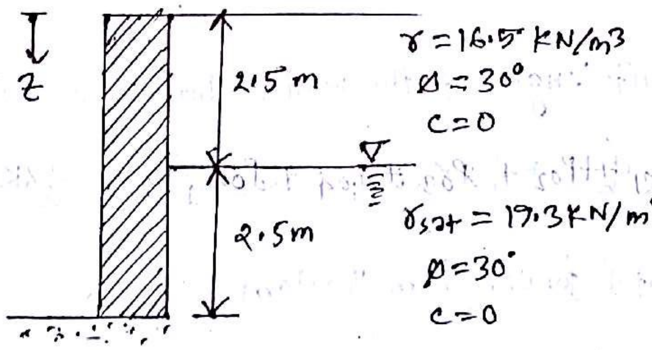


Total force per unit length of the wall, $P_{oT} = P_{o1} + P_{o2} + P_{o3}$

$$\therefore P_{oT} = K_o q H + \frac{1}{2} \times K_o \gamma' H^2 + \frac{1}{2} \gamma_w H^2$$

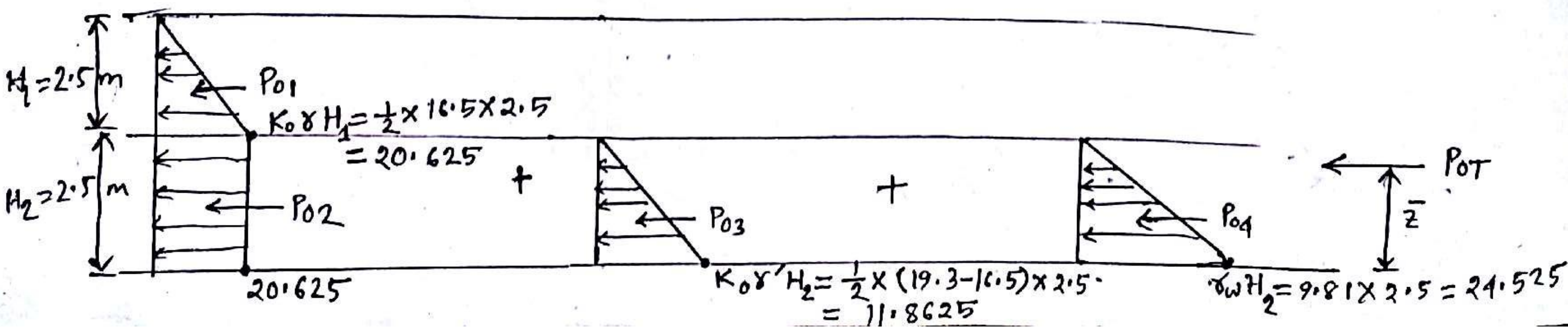
location of the total force from bottom, $\bar{z} = \dfrac{P_{o1} \times \frac{H}{2} + P_{o2} \times \frac{H}{3} + P_{o3} \times \frac{H}{3}}{P_{oT}}$

Problem: For the retaining wall shown in figure below; Determine the lateral earth force at rest per unit length of the wall. Also determine the location of the resultant force.



Solution: Here, c = 0, Hence it is normally consolidated coase-grained soil.

$$\therefore K = 1 - \sin\phi = 1 - \sin 30° = \frac{1}{2}$$



$\therefore P_{OT} = P_{O1} + P_{O2} + P_{O3} + P_{O4}$

$\Rightarrow P_{OT} = \left(\frac{1}{2} \times 20.625 \times 2.5\right) + (20.625 \times 2.5) + \left(\frac{1}{2} \times 11.8625 \times 2.5\right) + \left(\frac{1}{2} \times 24.525 \times 2.5\right)$

$\therefore P_{OT} = 122.83 \ KN/m$

$\therefore$ Total force per unit length of wall $= 122.83 \ KN/m$

Now,

Location of the resultant force, $\bar{z} = \dfrac{P_{O1} \times \left(H_2 + \frac{H_1}{3}\right) + P_{O2} \times \left(\frac{H_2}{2}\right) + P_{O3} \times \frac{H_2}{3} + P_{O4} \times \frac{H_2}{3}}{P_{OT}}$

$$\Rightarrow \bar{z} = \frac{25.78125 \times \left(2.5 + \frac{2.5}{3}\right) + 51.5625 \times 1.25 + 14.828125 \times \frac{2.5}{3} + 30.65625 \times \frac{2.5}{3}}{122.83}$$

$\therefore \bar{z} = 1.533 \ m$

(Ans.)

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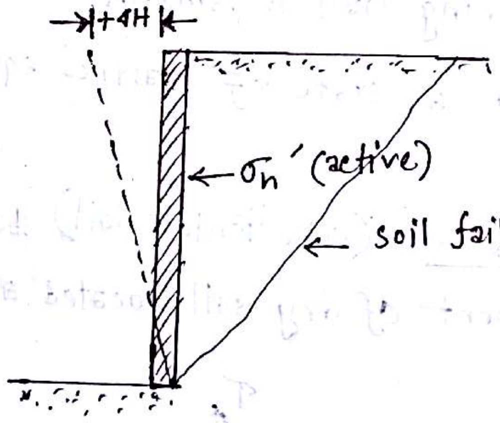
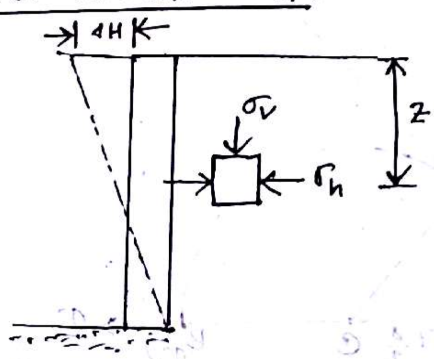
Active earth pressure:
If the wall moves away or tilt away from the soil retained, a triangular soil wedge behind the wall is fail. The lateral pressure at this condition is known as active earth pressure.



For Granular soil:



Here, $\sigma_v = \gamma z$

Initially, there is no lateral movement.

$\therefore \sigma_h = K_o \sigma_v = K_o \gamma z$

As the wall moves away from the soil.
σ_v remains the same. But,
σ_h decreases till failure occurs, which is called Active state.

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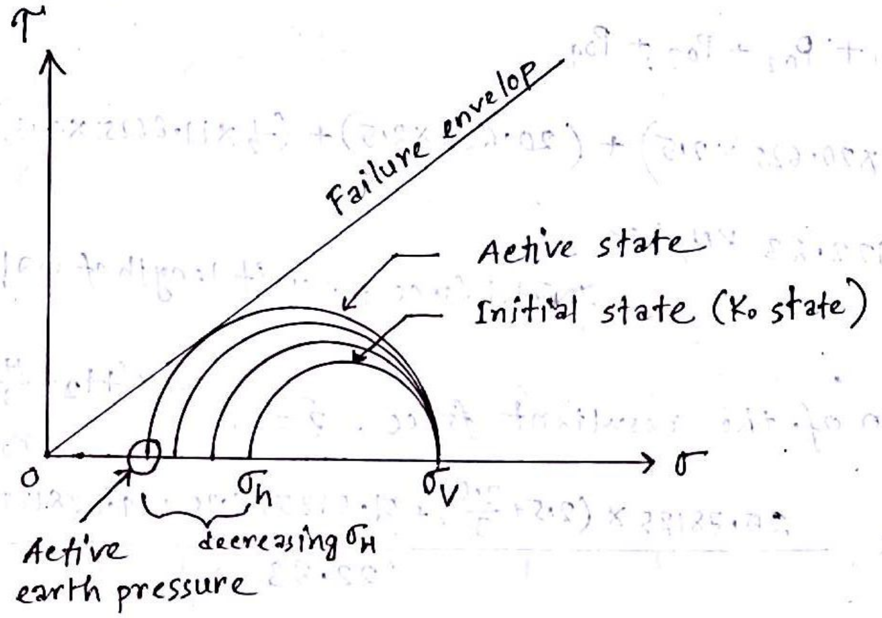


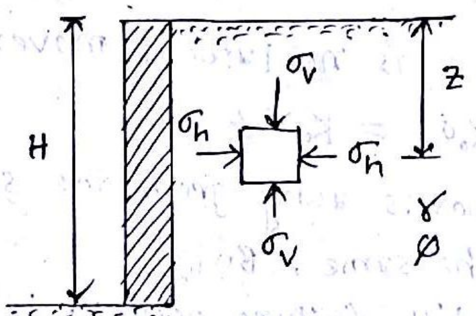
Assumptions of Rankine's Active earth pressure: 17, 11, 13, 06

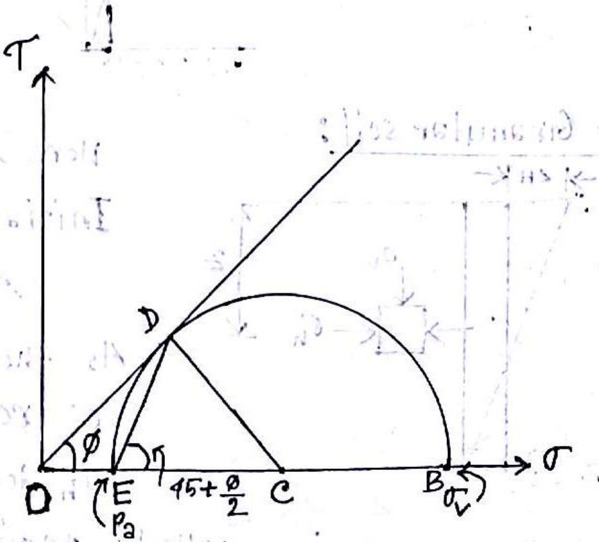
Rankine (1857) made the following assumptions:

1. The soil mass is homogeneous and semi-infinite.
2. The soil mass is dry and cohensionless.
3. The ground surface is plane.
4. The back of the retaining wall is smooth and vertical.
5. The soil element is in a state of plastic equilibrium.

Rankine's Active earth pressure: (Cohesionless Soil) 10

Let us, consider an element of dry soil located at a depth z as shown in figure below:





point E represents the active condition ∴ P_a = Active earth pressure

Here, $P_a = OE = OC - CE = OC - CD = OC - OC\sin\phi = OC(1-\sin\phi)$

and, $\sigma_v = OB = OC + BC = OC + OC\sin\phi = OC(1+\sin\phi)$

$$\frac{P_a}{\sigma_v} = \frac{1-\sin\phi}{1+\sin\phi} \Rightarrow P_a = \left[\frac{1-\sin\phi}{1+\sin\phi}\right] \times \sigma_v \Rightarrow P_a = K_a\sigma_v$$

$\therefore P_a = K_a \gamma z$ where,

K_a = co-efficient of active earth pressure $= \dfrac{1-\sin\phi}{1+\sin\phi}$

co-efficient of earth pressure at rest is larger than at active:

We know, $K_0 = 1-\sin\phi$ and $K_a = \dfrac{1-\sin\phi}{1+\sin\phi}$

Let, $\phi = 30°$

$\therefore K_0 = 1-\sin 30° = \frac{1}{2}$

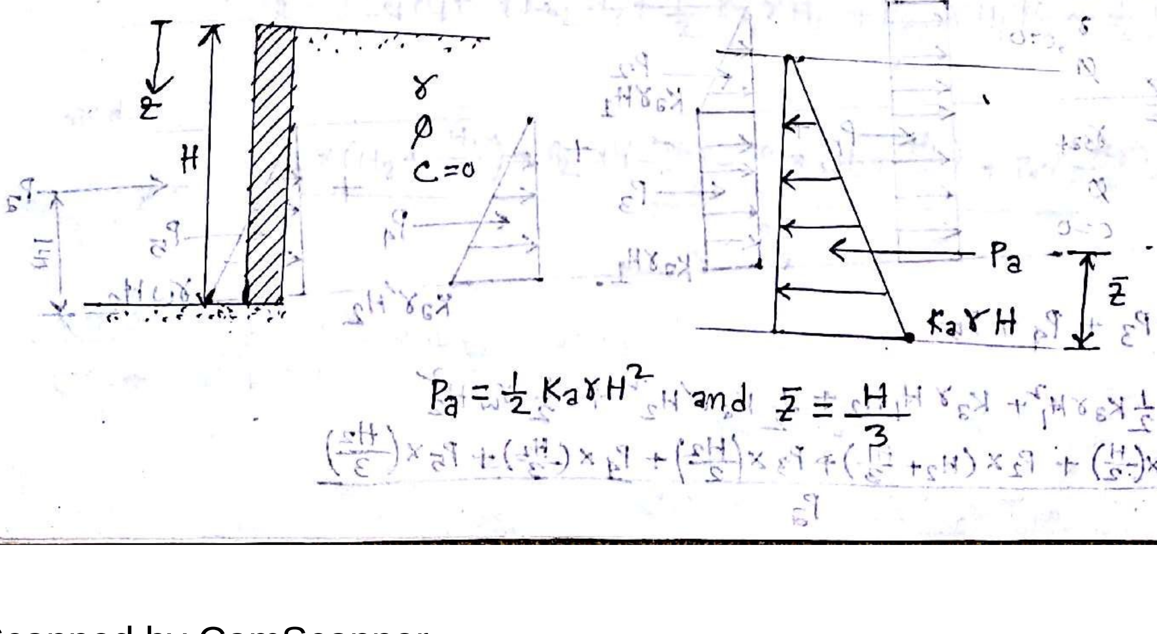
and $K_a = \dfrac{1-\sin 30°}{1+\sin 30°} = \dfrac{1}{3}$

Now,

$$\frac{K_0}{K_a} = \frac{1/2}{1/3} = \frac{3}{2} \Rightarrow K_0 = 1.5K_a \quad \therefore K_0 > K_a$$

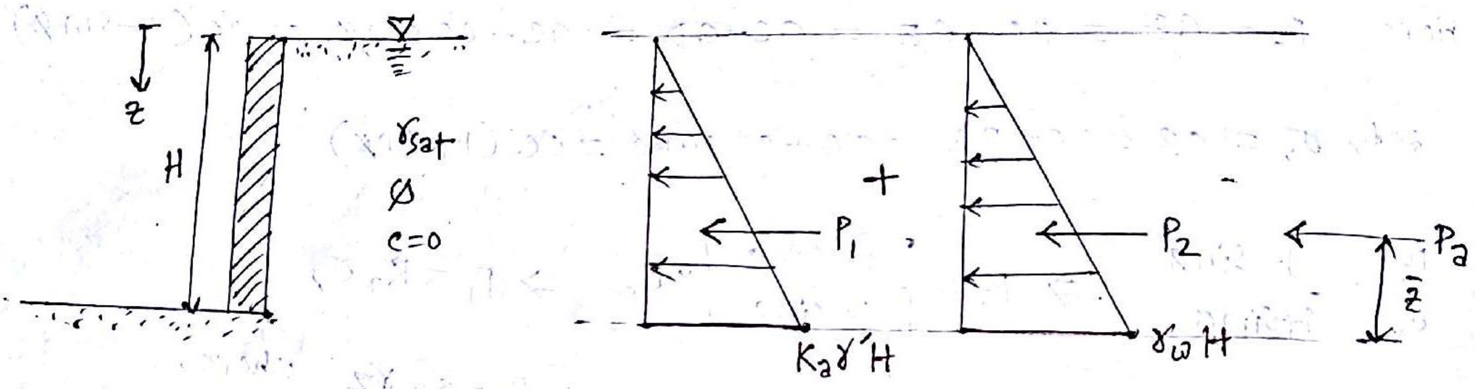
Several cases of Rankine's Active eath pressure: (Pressure Distribution)

Case-01: Dry backfill



$P_a = \frac{1}{2}K_a\gamma H^2$ and $\bar{z} = \dfrac{H}{3}$

Case-02: Submerge backfill 07, 15

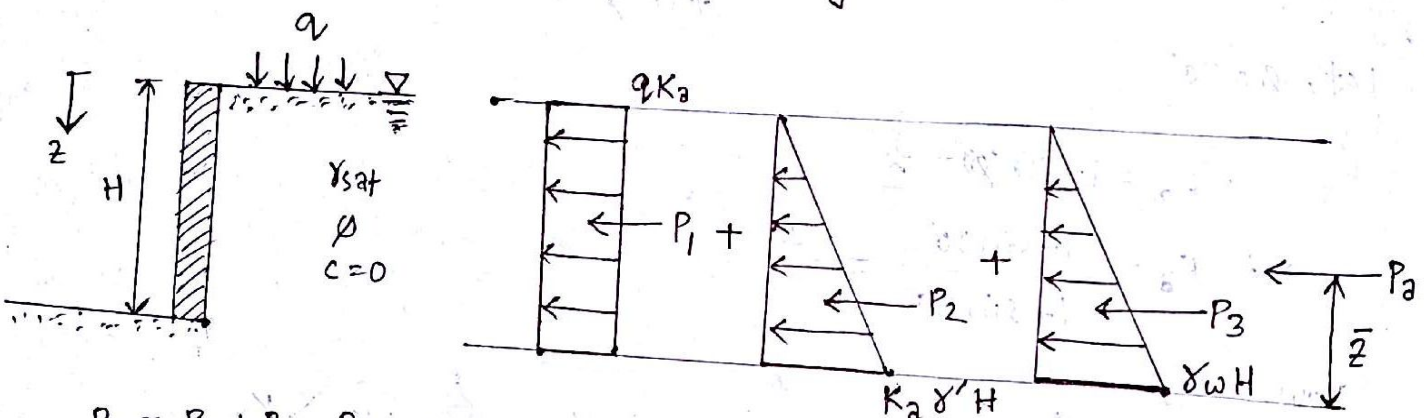


$$P_a = P_1 + P_2$$

$$\therefore P_a = \frac{1}{2} K_a \gamma' H^2 + \frac{1}{2}\gamma_w H^2$$

and, $\bar{z} = \frac{H}{3}$

Case-03: Submerged backfill with surcharge

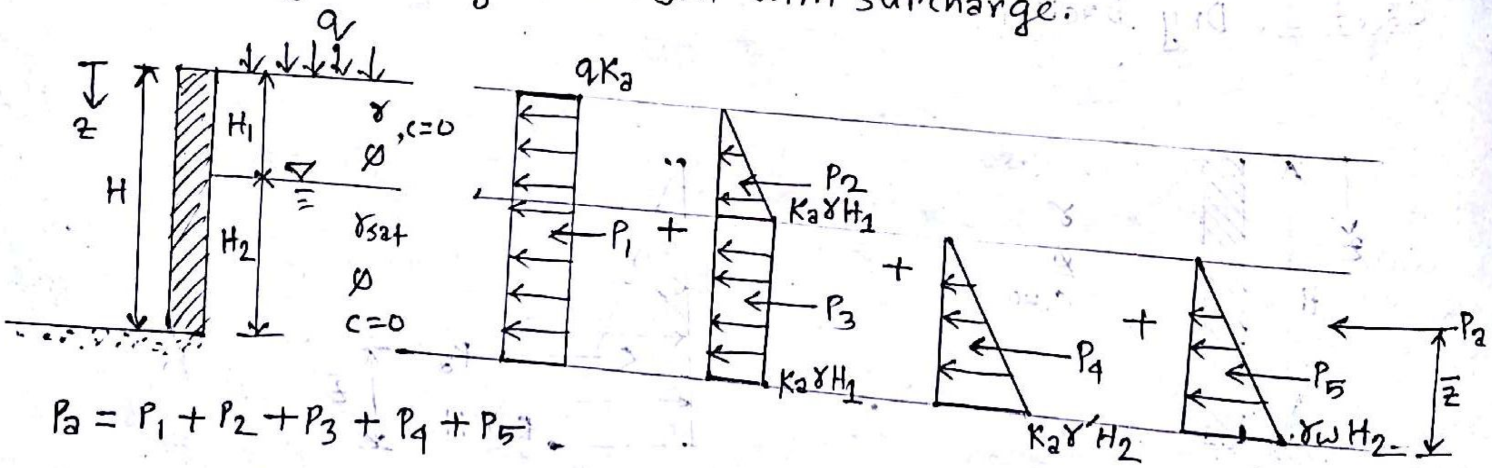


$$P_a = P_1 + P_2 + P_3$$

$$\therefore P_a = Hq K_a + \frac{1}{2} K_a \gamma' H^2 + \frac{1}{2}\gamma_w H^2$$

and, $\bar{z} = \dfrac{P_1 \times \left(\frac{H}{2}\right) + P_2 \times \left(\frac{H}{3}\right) + P_3 \times \left(\frac{H}{3}\right)}{P_a}$

Case-04: Partially submerged backfill with surcharge.

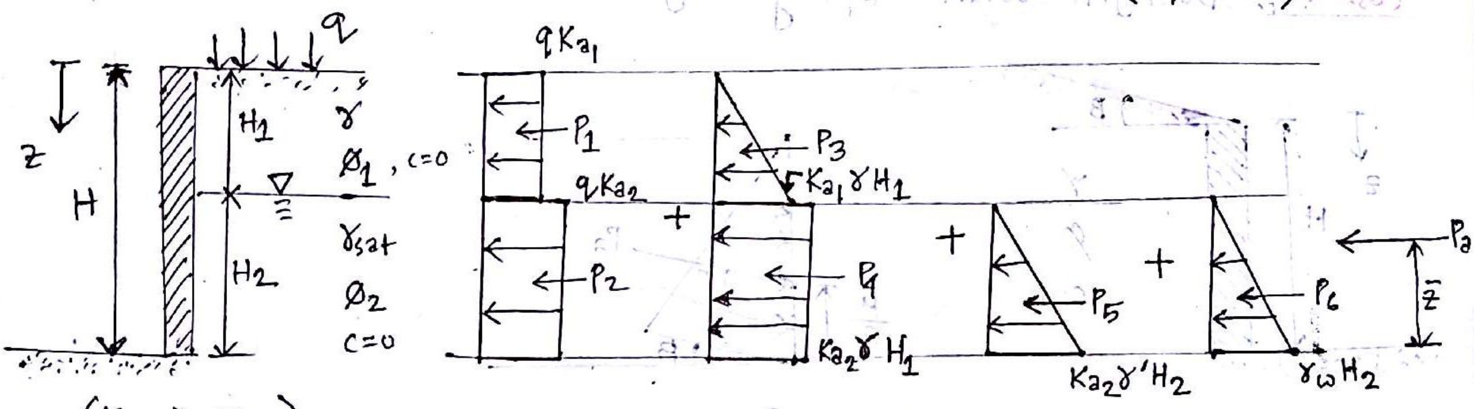


$$P_a = P_1 + P_2 + P_3 + P_4 + P_5$$

$$\therefore P_a = qK_a H + \frac{1}{2} K_a \gamma H_1^2 + K_a \gamma H_1 H_2 + \frac{1}{2} K_a \gamma' H_2^2 + \frac{1}{2}\gamma_w H_2^2$$

and, $\bar{z} = \dfrac{P_1 \times \left(\frac{H}{2}\right) + P_2 \times \left(H_2 + \frac{H_1}{3}\right) + P_3 \times \left(\frac{H_2}{2}\right) + P_4 \times \left(\frac{H_2}{3}\right) + P_5 \times \left(\frac{H_2}{3}\right)}{P_a}$

Case-05: Partially submerged backfill with surcharge ($\phi_1 > \phi_2$)



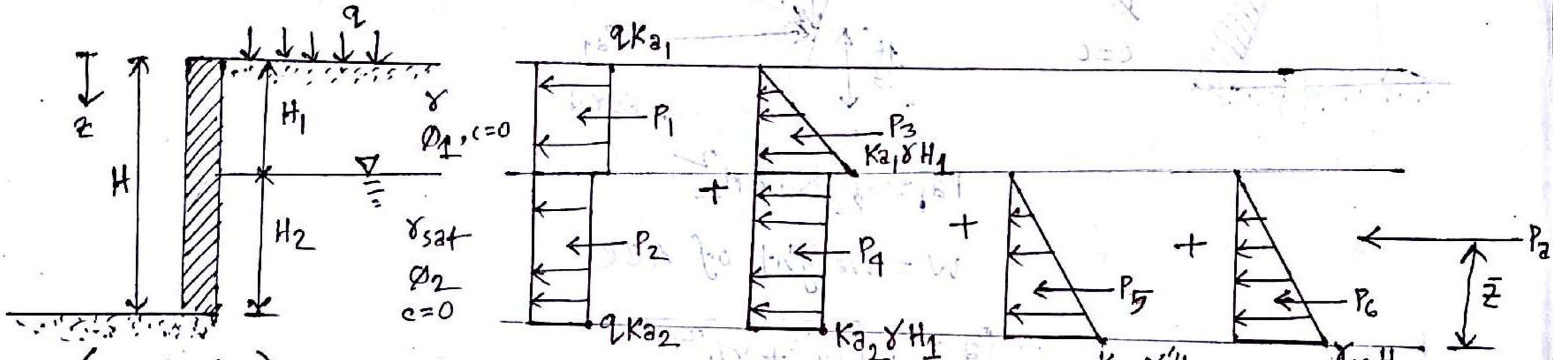
$(K_{a_2} > K_{a_1})$

$$P_a = P_1 + P_2 + P_3 + P_4 + P_5 + P_6$$

$$\therefore P_a = qK_{a_1}H_1 + qK_{a_2}H_2 + \frac{1}{2}K_{a_1}\gamma H_1^2 + K_{a_2}\gamma H_1 H_2 + \frac{1}{2}K_{a_2}\gamma' H_2^2 + \frac{1}{2}\gamma_w H_2^2$$

and,
$$\bar{z} = \frac{P_1 \times \left(H_2 + \frac{H_1}{2}\right) + P_2 \times \left(\frac{H_2}{2}\right) + P_3 \times \left(H_2 + \frac{H_1}{3}\right) + P_4 \times \left(\frac{H_2}{2}\right) + P_5 \times \left(\frac{H_2}{3}\right) + P_6 \times \left(\frac{H_2}{3}\right)}{P_a}$$

Case-06: Partially submerged backfill with surcharge ($\phi_1 < \phi_2$)



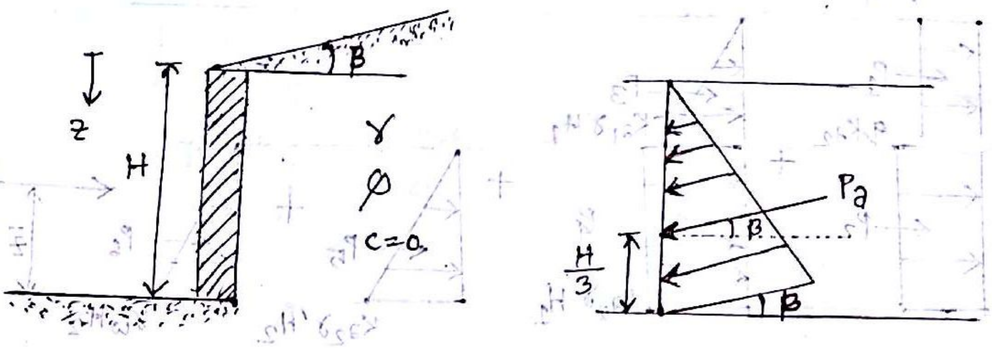
$(K_{a_1} > K_{a_2})$

$$P_a = P_1 + P_2 + P_3 + P_4 + P_5 + P_6$$

$$\therefore P_a = qK_{a_1}H_1 + qK_{a_2}H_2 + \frac{1}{2}K_{a_1}\gamma H_1^2 + K_{a_2}\gamma H_1 H_2 + \frac{1}{2}K_{a_2}\gamma' H_2^2 + \frac{1}{2}\gamma_w H_2^2$$

and,
$$\bar{z} = \frac{P_1 \times \left(H_2 + \frac{H_1}{2}\right) + P_2 \times \left(\frac{H_2}{2}\right) + P_3 \times \left(H_2 + \frac{H_1}{3}\right) + P_4 \times \left(\frac{H_2}{2}\right) + P_5 \times \left(\frac{H_2}{3}\right) + P_6 \times \left(\frac{H_2}{3}\right)}{P_a}$$

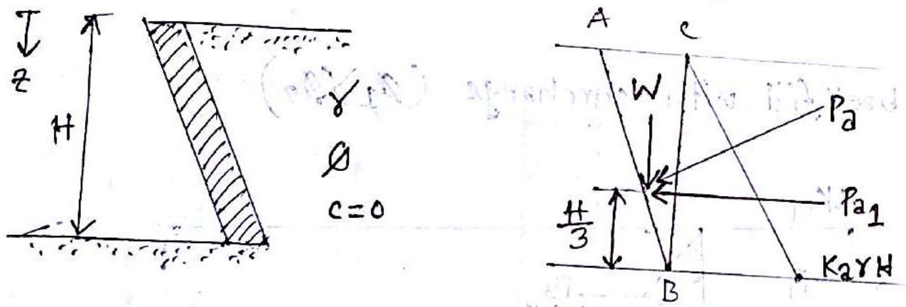
Case-07: Backfill with sloping surface 06, 16, 13



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

where, $K_a = \cos\beta \times \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$

Case-08: Inclined backfill

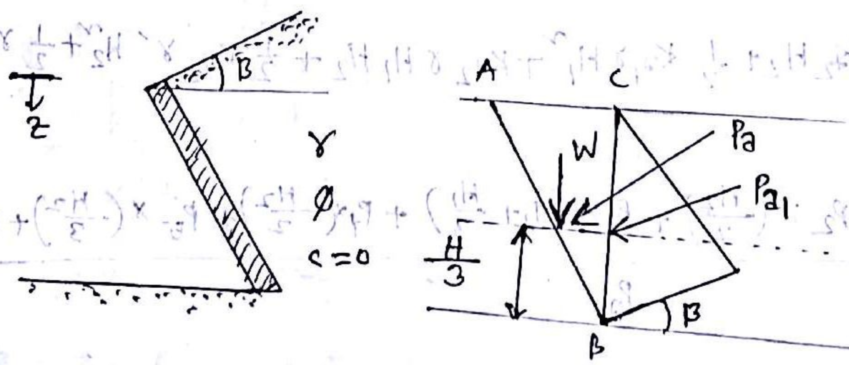


$$P_{a_1} = \frac{1}{2} K_a \gamma H^2$$

W = Weight of ABC

$$\therefore P_a = \sqrt{P_{a_1}^2 + W^2}$$

Case-09: Inclined back and surcharge with sloping surface.



$$P_{a_1} = \frac{1}{2} K_a \gamma H$$

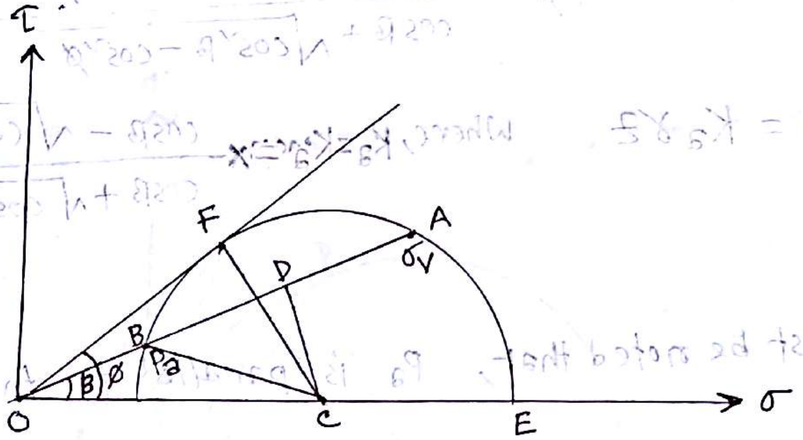
where, $K_a = \cos\beta \times \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$

W = weight of ABC

$$\therefore P_a = \sqrt{P_{a_1H}^2 + (W + P_{a_1V})^2}$$

Derive the expression for Rankine's active earth pressure co-efficient for inclined ground surface:

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



From figure,

$$\frac{P_a}{\sigma_v} = \frac{OB}{OA} = \frac{OD-DB}{OD+DA} \quad \text{.......(1)}$$

Now, $\cos\beta = \frac{OD}{OC} \Rightarrow OD = OC\cos\beta$

$$DA = DB = \sqrt{BC^2 - DC^2}$$

But, $BC = FC = OC\sin\phi$ and, $DC = OC\sin\beta$

$$\therefore DA = DB = \sqrt{(OC\sin\phi)^2 - (OC\sin\beta)^2} = OC\sqrt{\sin^2\phi - \sin^2\beta}$$

Now, from equation (1) we obtain,

$$\frac{P_a}{\sigma_v} = \frac{OC\cos\beta - OC\sqrt{\sin^2\phi - \sin^2\beta}}{OC\cos\beta + OC\sqrt{\sin^2\phi - \sin^2\beta}}$$

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

$$= \frac{\cos\beta - \sqrt{(1-\cos^2\phi) - (1-\cos^2\beta)}}{\cos\beta + \sqrt{(1-\cos^2\phi) - (1-\cos^2\beta)}}$$

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

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$$\Rightarrow P_a = \sigma_y \times \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$\Rightarrow P_a = (\gamma z \cos\beta) \times \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$\Rightarrow P_a = K_a \gamma z \qquad \text{Where, } K_a = \cos\beta \times \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

Here,

It must be noted that, P_a is parallel to the inclined surface.

When, $\beta = 0$, $\cos\beta = 1$

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

17, 16

Active pressure is the minimum pressure which develops when the wall moves away from the backfill.

If at-rest lateral earth pressures cause a retaining wall to move, the horizontal pressure behind the wall will reduce. With enough displacement, the soil behind the wall will develop a failure plane. The failure plane on which soil particles move with respect to each other, will limit horizontal soil pressure to a minimum value. This minimum pressure is active pressure.

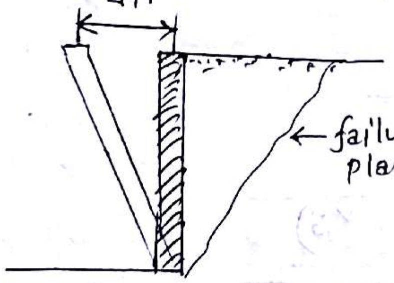


Fig. Active earth pressure

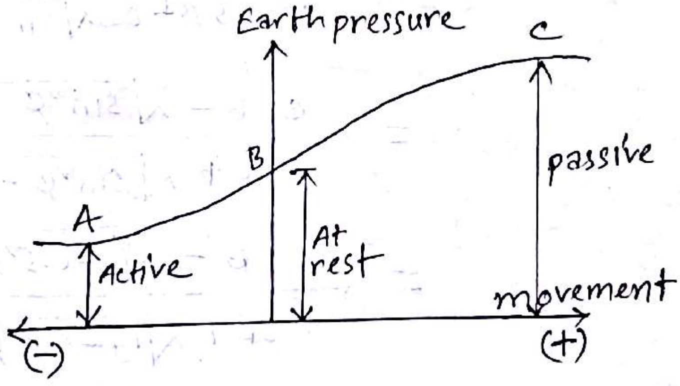
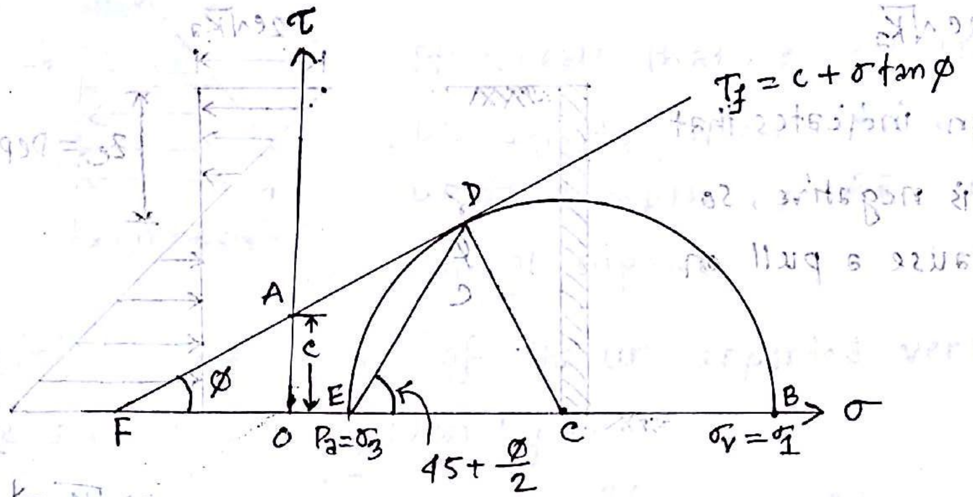


Fig. Variation of Pressure

Rankine's Active Earth Pressure: (Cohesive Soil) 07, 05, 04

Rankine's original theory was for cohesiveless soils. It was extended by Resel (1910) and Bell (1915) for cohesive soil.



From Triangle ΔFCD, $\sin\phi = \frac{CD}{FC} = \frac{CD}{FO + OC} = \frac{CE}{FO + OC}$ .... ①

Here, $CE = \frac{\sigma_1 - \sigma_3}{2}$

$\cot\phi = \frac{FO}{OA} = \frac{FO}{c} \Rightarrow FO = c\cot\phi$

$OC = \sigma_3 + \frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_1 + \sigma_3}{2}$

Now, From eq^n ①,

$$\sin\phi = \frac{\frac{\sigma_1 - \sigma_3}{2}}{c\cot\phi + \frac{\sigma_1 + \sigma_3}{2}}$$

$$\Rightarrow 2c\cot\phi \cdot \sin\phi + (\sigma_1 + \sigma_3)\sin\phi = \sigma_1 - \sigma_3$$

$$\Rightarrow \sigma_3 = \sigma_1\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) - 2c\left(\frac{\cos\phi}{1 + \sin\phi}\right)$$

$$\Rightarrow \sigma_3 = \sigma_1 \tan^2\left(45° - \frac{\phi}{2}\right) - 2c\tan\left(45° - \frac{\phi}{2}\right) \quad \text{......} \; ⑪$$

As, $\sigma_h = \sigma_3 = P_a$ and $\sigma_1 = \gamma z$,

$$P_a = \gamma z \tan^2\left(45° - \frac{\phi}{2}\right) - 2c\tan\left(45° - \frac{\phi}{2}\right)$$

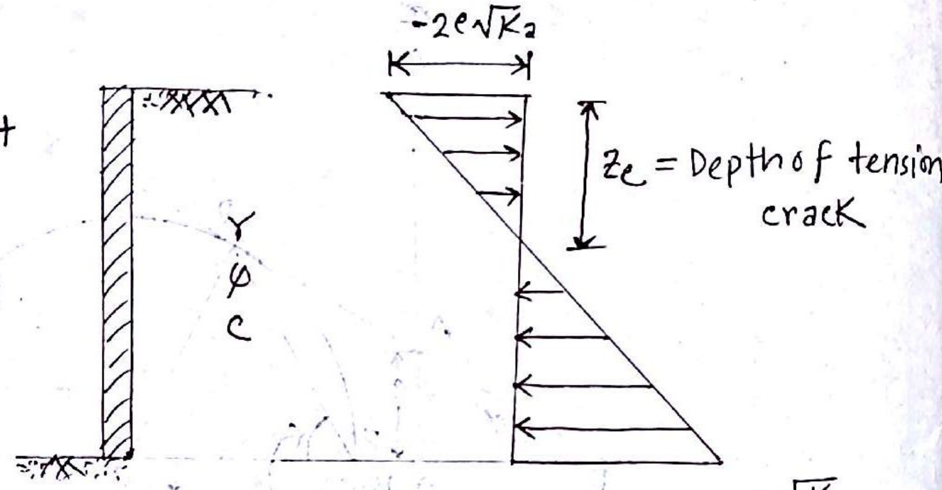
$\therefore P_a = \gamma z K_a - 2c\sqrt{K_a}$ ....(iii) where, $K_a = \tan^2(45° - \frac{\phi}{2}) = \frac{1-\sin\phi}{1+\sin\phi}$

$$\sqrt{K_a} = \frac{\cos\phi}{1+\sin\phi}$$

If $z = 0$,

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

The negative sign indicates that the pressure is negative, so it tries to cause a pull on the wall.

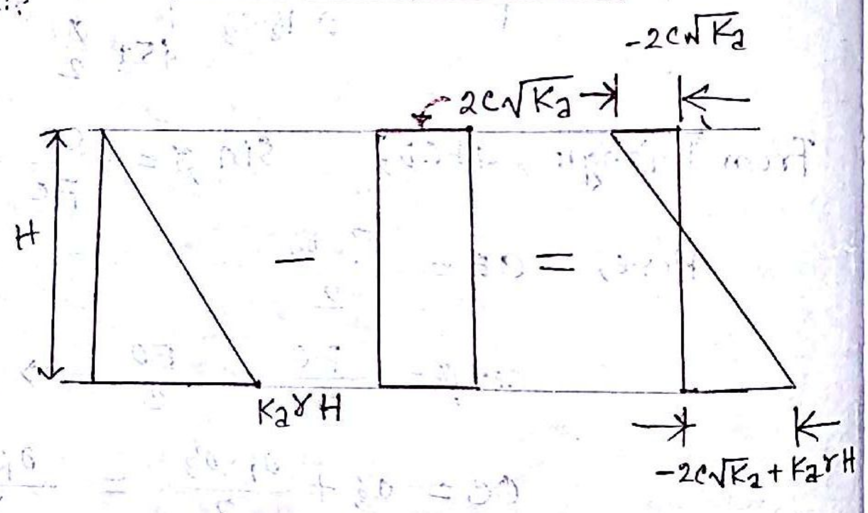


If $P_a = 0$,

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

$$\Rightarrow \gamma z K_a = 2c\sqrt{K_a}$$

$$\Rightarrow z = z_c = \frac{2c}{\gamma\sqrt{K_a}}$$



If $z = H$,

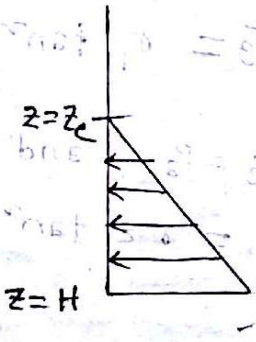
$$P_a = \gamma H K_a - 2c\sqrt{K_a}$$

Total Pressure is given by, $P_a = \int_0^H (K_a\gamma H - 2c\sqrt{K_a}) = \frac{1}{2}K_a\gamma H^2 - 2c\sqrt{K_a}H$

Note that, this pressure is applicable before the formation of crack.

After the formation of crack, the force on the wall is caused only by the pressure from $z = z_c$ to $z = H$.

Thus, $P_a = \frac{1}{2}(H - z_c)(K_a\gamma H - 2c\sqrt{K_a})$



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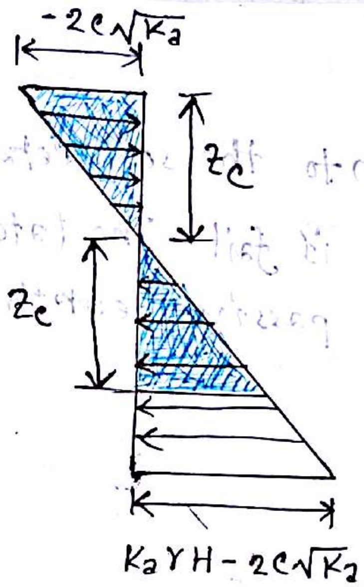


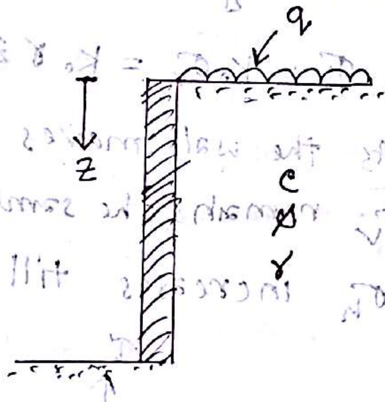
From figure, it is clear that the total net pressure upon depth of $2z_c$ is zero.

It reveals that, a cohesive soil should be able to stand with a vertical face up to a depth of $2z_c$ without any lateral support.

The critical height H_c of the unsupported vertical cut in cohesive soil can be given by —

$$H_c = 2z_c = 2 \times \frac{2c}{\gamma\sqrt{K_a}} = \frac{4c}{\gamma\sqrt{K_a}}$$

Backfill with surcharge: 11



If the backfill carries a surcharge q,

$$P_a = K_a\gamma z - 2c\sqrt{K_a} + qK_a$$

At $z=0$,

$$P_a = -2c\sqrt{K_a} + qK_a$$

When $P_a = 0$,

$$0 = K_a\gamma z - 2c\sqrt{K_a} + qK_a$$

$$\Rightarrow \sqrt{K_a}\,\gamma z = 2c - q\sqrt{K_a}$$

$$\Rightarrow z = z_c = \frac{2c}{\gamma\sqrt{K_a}} - \frac{q}{\gamma}$$

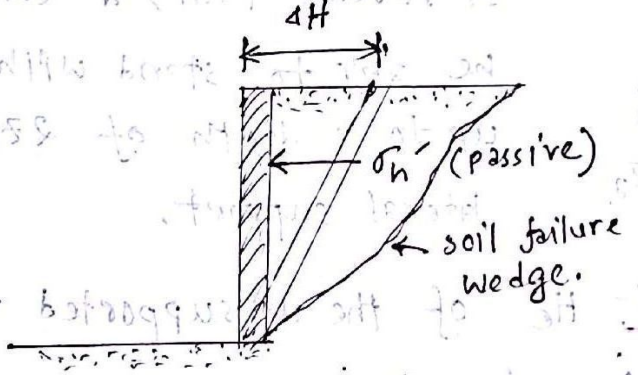
When $z=0$ & $P_a = 0$

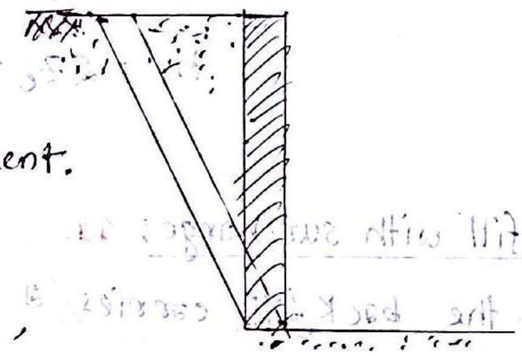
$$\frac{q}{\gamma} = \frac{2c}{\gamma\sqrt{K_a}} \Rightarrow \boxed{q = \frac{2c}{\sqrt{K_a}}}$$

surcharge value for which the soil is critically stable without any support

Passive active Pressure:

If the wall moves towards or pushed into the soil retained, a triangular soil wedge behind the wall is fail. The lateral pressure at this condition is known as passive earth pressure.





For Granular Soil:

Here, $\sigma_v = \gamma z$

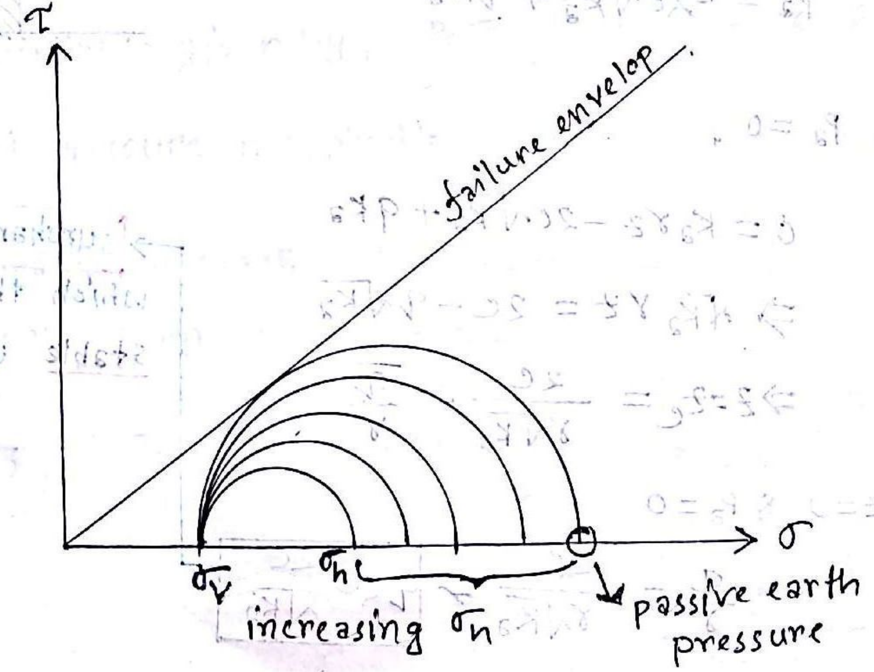
Initially, there is no lateral movement.

$\sigma_h = K_0 \sigma_v = K_0 \gamma z$

As the wall moves towards the soil,

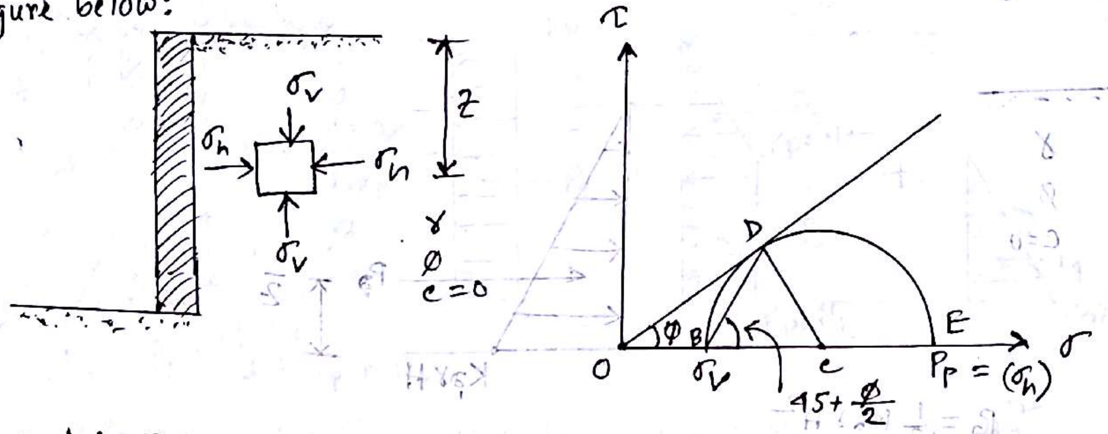
σ_v remains the same. But,

σ_h increases till failure occurs, which is called passive state.



Rankine's Passive Earth Pressure: (Cohesionless soil) 17/10

Let us, consider an element of dry soil located at a depth z as shown in figure below:



point E represents the passive condition.

Here, $P_p = OE = OC + CE = OC + CD = OC + OC\sin\phi = OC(1+\sin\phi)$

and $\sigma_v = OB = OC - CB = OC - CD = OC - OC\sin\phi = OC(1-\sin\phi)$

$$\therefore \frac{P_p}{\sigma_v} = \frac{1+\sin\phi}{1-\sin\phi} \Rightarrow P_p = \left[\frac{1+\sin\phi}{1-\sin\phi}\right] \times \sigma_v$$

$$\Rightarrow P_p = K_p \sigma_v = K_p \gamma z$$

where, K_p = co-efficient of passive earth pressure $= \frac{1+\sin\phi}{1-\sin\phi}$

co-efficient of passive earth pressure is large than at active:

Let us, Assume, $\phi = 30^\circ$

$$\therefore K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = \frac{1}{3}$$

$$\text{and } K_p = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1+\sin 30^\circ}{1-\sin 30^\circ} = 3$$

Now $\frac{K_p}{K_a} = \frac{3}{1/3} = 9 \quad \therefore K_p = 9K_a$

Hence, $K_p > K_a$

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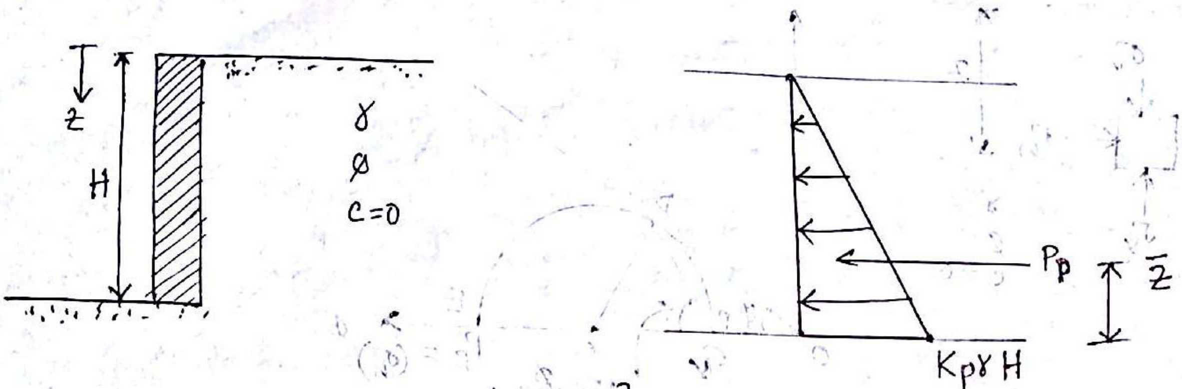
Several cases of Rankine's passive earth pressure: (Pressure distribution)

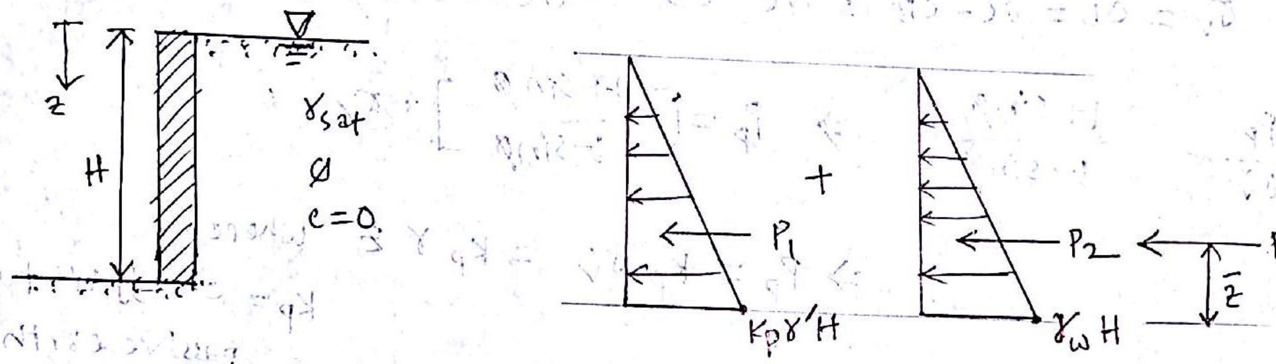
Case-01: Dry Back fill



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

and, $\bar{z} = \frac{H}{3}$

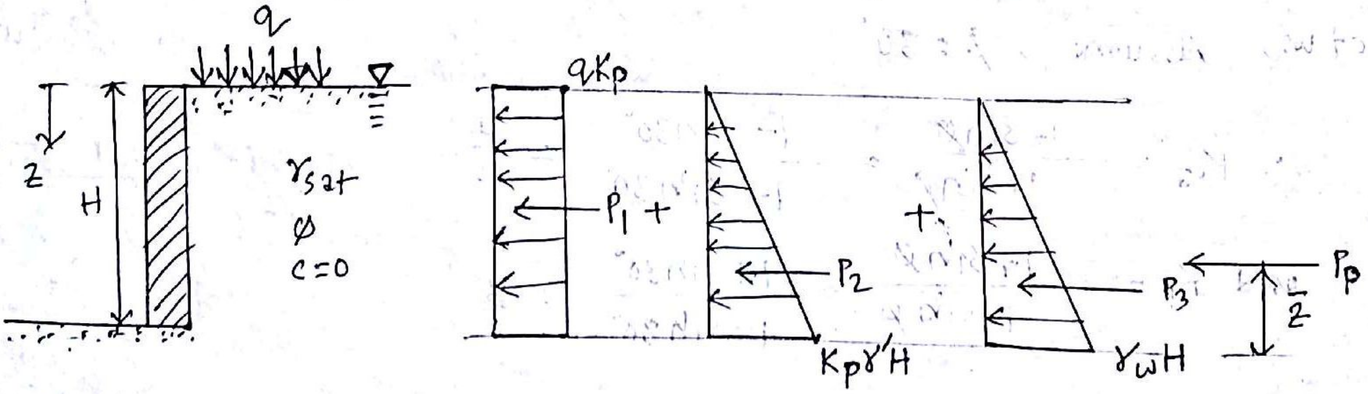
Case-02: Submerged back fill



$P_p = P_1 + P_2$

$\therefore P_p = \frac{1}{2} K_p \gamma' H^2 + \frac{1}{2} \gamma_w H^2$ and $\bar{z} = \frac{H}{3}$

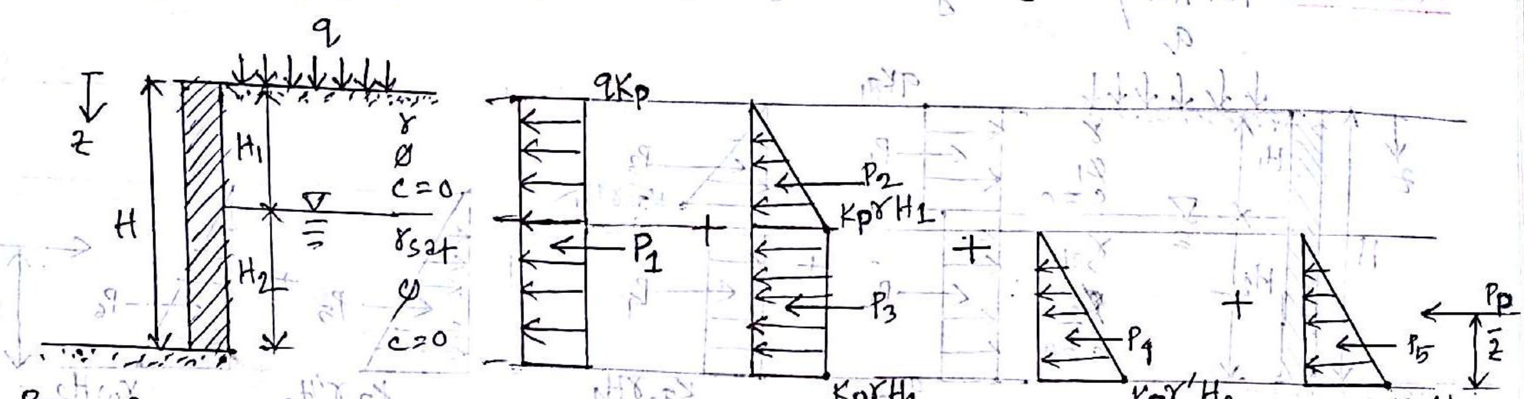
Case-03: Submerged backfill with surcharge



$P_p = P_1 + P_2 + P_3$

$\therefore P_p = qK_pH + \frac{1}{2} K_p \gamma' H^2 + \frac{1}{2} \gamma_w H^2$ and $\bar{z} = \frac{P_1 \times \left(\frac{H}{2}\right) + P_2 \times \left(\frac{H}{3}\right) + P_3 \times \left(\frac{H}{3}\right)}{P_p}$

Case-04: Partially submerged backfill with surcharge

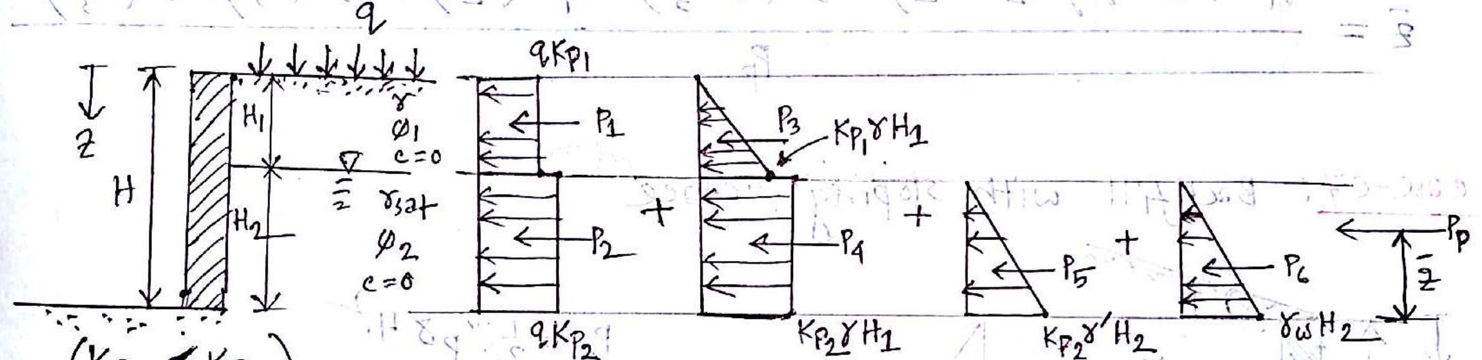


$$P_p = P_1 + P_2 + P_3 + P_4 + P_5$$

$$\therefore P_p = qK_pH + \frac{1}{2}K_p\gamma H_1^2 + K_p\gamma H_1 H_2 + \frac{1}{2}K_p\gamma' H_2^2 + \frac{1}{2}\gamma_w H_2^2$$

and, $$\bar{z} = \frac{P_1\times\left(\frac{H}{2}\right) + P_2\times\left(H_2+\frac{H_1}{3}\right) + P_3\times\left(\frac{H_2}{2}\right) + P_4\times\left(\frac{H_2}{3}\right) + P_5\times\left(\frac{H_2}{3}\right)}{P_p}$$

Case-05: Partially submerged backfill with surcharge ($\phi_1 < \phi_2$)



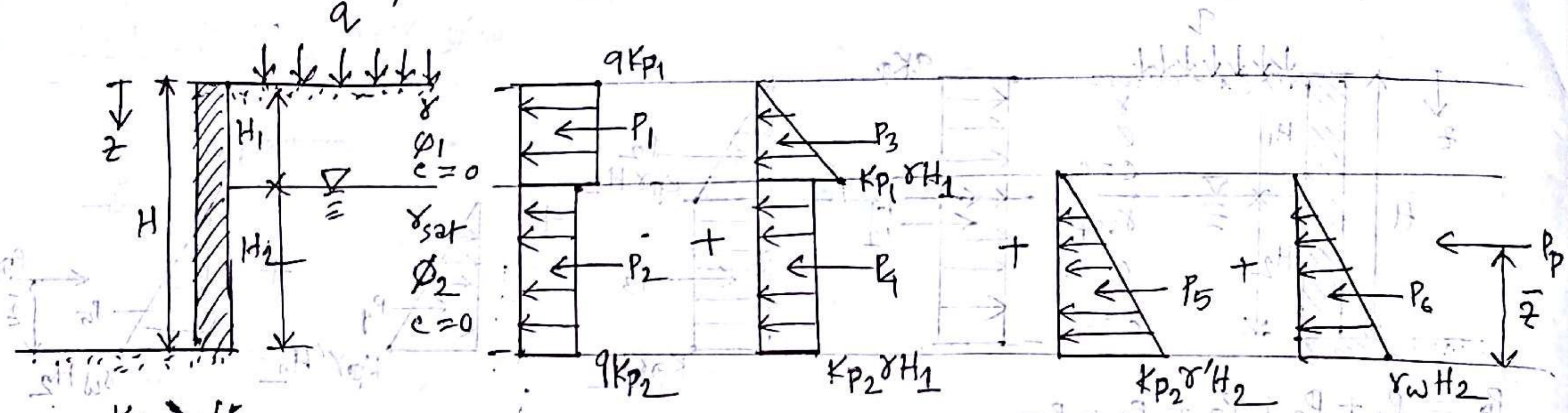
$(K_{p_1} < K_{p_2})$

$$P_p = P_1 + P_2 + P_3 + P_4 + P_5 + P_6$$

$$\therefore P_p = qK_{p_1}H_1 + qK_{p_2}H_2 + \frac{1}{2}K_{p_1}\gamma H_1^2 + K_{p_2}\gamma H_1 H_2 + \frac{1}{2}K_{p_2}\gamma' H_2^2 + \frac{1}{2}\gamma_w H_2^2$$

and, $$\bar{z} = \frac{P_1\times\left(H_2+\frac{H_1}{2}\right) + P_2\times\left(\frac{H_2}{2}\right) + P_3\times\left(H_2+\frac{H_1}{3}\right) + P_4\times\left(\frac{H_2}{2}\right) + P_5\times\left(\frac{H_2}{3}\right) + P_6\times\left(\frac{H_2}{3}\right)}{P_p}$$

case-06: Partially submerged backfill with surcharge ($\phi_1 > \phi_2$)



$K_{p_1} > K_{p_2}$

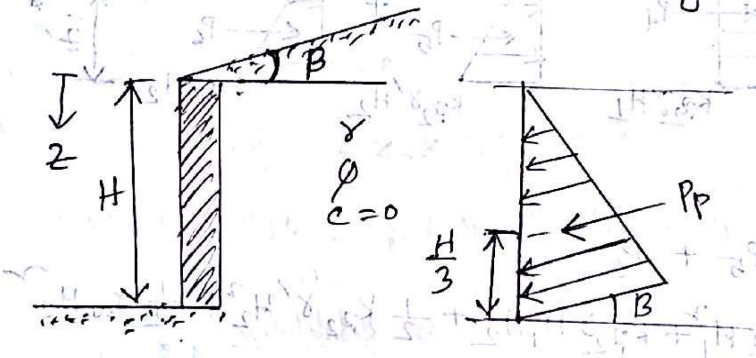
$$P_p = P_1 + P_2 + P_3 + P_4 + P_5 + P_6$$

$$\therefore P_p = qK_{p_1}H_1 + qK_{p_2}H_2 + \frac{1}{2}K_{p_1}\gamma H_1^2 + K_{p_2}\gamma H_1 H_2 + \frac{1}{2}K_{p_2}\gamma' H_2^2 + \frac{1}{2}\gamma_w H_2^2$$

and,

$$\bar{z} = \frac{P_1 \times \left(H_2 + \frac{H_1}{2}\right) + P_2 \times \left(\frac{H_2}{2}\right) + P_3 \times \left(H_2 + \frac{H_1}{3}\right) + P_4 \times \left(\frac{H_2}{2}\right) + P_5 \times \left(\frac{H_2}{3}\right) + P_6 \times \left(\frac{H_2}{3}\right)}{P_p}$$

case-07: Backfill with sloping surface

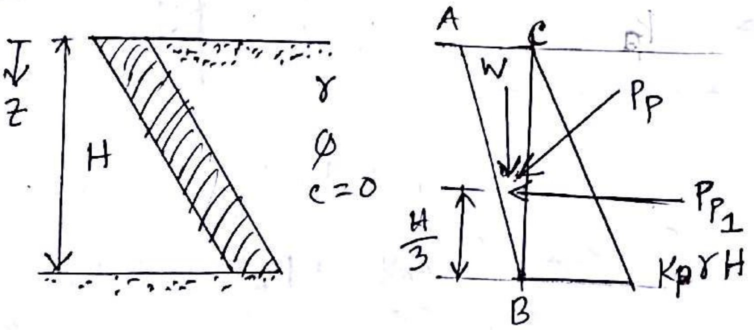


$$P_p = \frac{1}{2}K_p \gamma H^2$$

where,

$$K_p = \cos\beta \times \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

case-08: Inclined Backfill

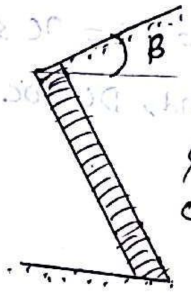


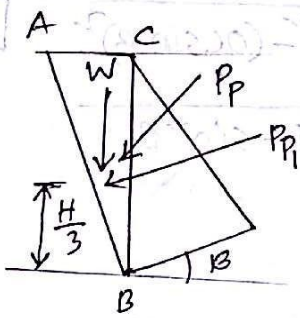
$$P_{p_1} = \frac{1}{2}K_p \gamma H^2$$

W = weight of ABC

$$\therefore P_p = \sqrt{P_{p_1}^2 + W^2}$$

Case-09: Inclined back and surcharge with sloping surface





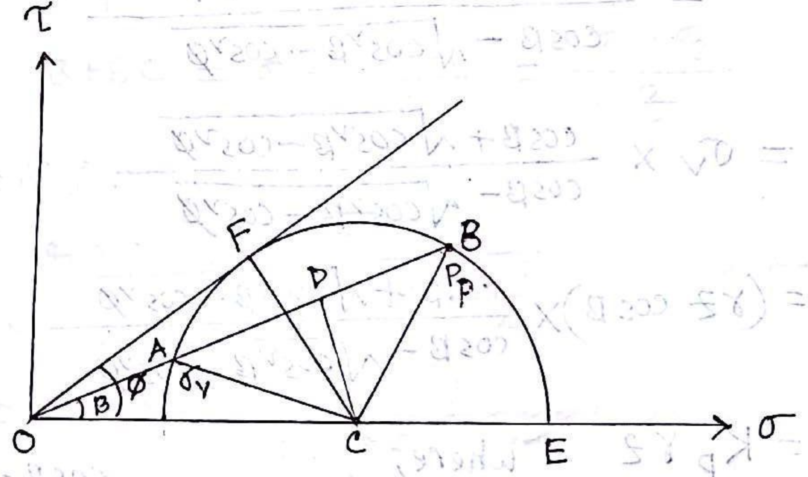
$P_{P_1} = \frac{1}{2} K_P \gamma H$ where, $K_P = \cos\beta \times \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$

W = Weight of ABC

$\therefore P_P = \sqrt{P_{P_{1H}}^2 + (W + P_{P_{1V}})^2}$

Derive the expression for Rankine's passive earth pressure co-efficient for inclined ground surface:

$K_p = \cos\beta \times \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$



From figure,

$$\frac{P_P}{\sigma_v} = \frac{OB}{OA} = \frac{OD + DB}{OD - DA} \quad \text{.......(1)}$$

Now, $\cos\beta = \frac{OD}{OC} \Rightarrow OD = OC \cos\beta$

and, $DB = DA = \sqrt{AC^2 - DC^2} = \sqrt{FC^2 - DC^2}$

$$= \sqrt{(OC\sin\phi)^2 - (OC\sin\beta)^2} \quad \left[\because BC = FC = OC\sin\phi \text{ and, } DC = OC\sin\beta\right]$$

$$= OC\sqrt{\sin^2\phi - \sin^2\beta}$$

Now, from equation ①,

$$\frac{P_p}{\sigma_v} = \frac{OC\cos\beta + OC\sqrt{\sin^2\phi - \sin^2\beta}}{OC\cos\beta - OC\sqrt{\sin^2\phi - \sin^2\beta}}$$

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

$$= \frac{\cos\beta + \sqrt{(1-\cos^2\phi) - (1-\cos^2\beta)}}{\cos\beta - \sqrt{(1-\cos^2\phi) - (1-\cos^2\beta)}}$$

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

$$\Rightarrow P_a = \sigma_v \times \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$\Rightarrow P_a = (\gamma z\cos\beta) \times \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

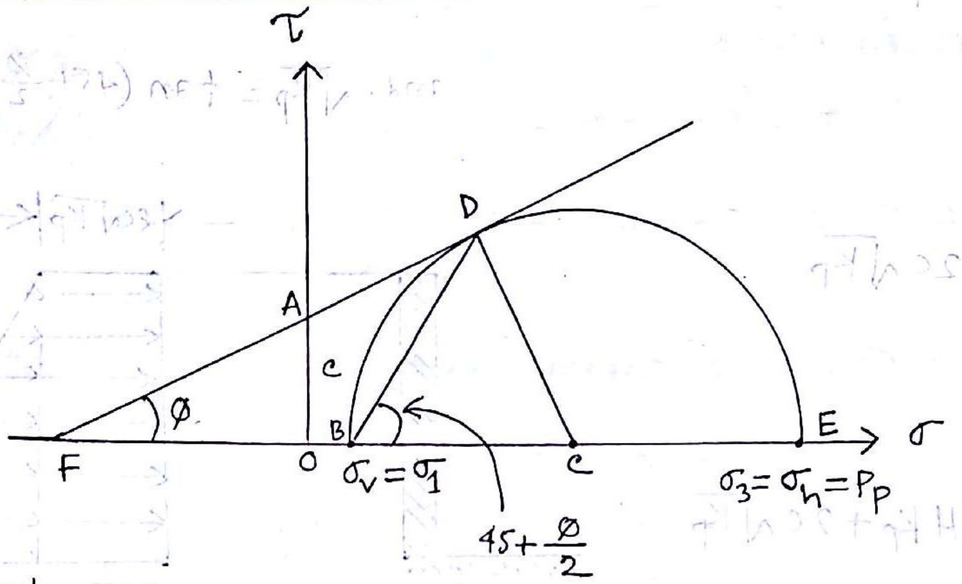
$$\Rightarrow P_a = K_p \gamma z$$ where, $$K_p = \cos\beta \times \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

It must be noted that, P_p is parallel to the inclined surface.

When $\beta = 0$, $\cos\beta = 1$

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

Rankine's Passive Earth Pressure: (Cohesive soil)



From triangle FCD,

$$\sin\phi = \frac{CD}{FC} = \frac{BC}{FO + OC} \quad \cdots\cdots\cdots (1)$$

Here, $BC = \frac{\sigma_3 - \sigma_1}{2}$

$$\cot\phi = \frac{FO}{OA} \Rightarrow FO = c\cot\phi$$

$$OC = OB + BC = \sigma_1 + \frac{\sigma_3 - \sigma_1}{2} = \frac{\sigma_1 + \sigma_3}{2}$$

Now, From eqn (1),

$$\sin\phi = \frac{\frac{\sigma_3 - \sigma_1}{2}}{c\cot\phi + \frac{\sigma_3 + \sigma_1}{2}}$$

$$\Rightarrow \sin\phi = \frac{(\sigma_3 - \sigma_1)}{2c\cot\phi + (\sigma_3 + \sigma_1)}$$

$$\Rightarrow (\sigma_3 + \sigma_1)\sin\phi + 2c\cot\phi \cdot \sin\phi = (\sigma_3 - \sigma_1)$$

$$\Rightarrow \sigma_3 = \sigma_1\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right) + 2c\left(\frac{\cos\phi}{1 - \sin\phi}\right)$$

$$\Rightarrow \sigma_h = \sigma_v \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2c\tan\left(45^\circ + \frac{\phi}{2}\right)$$

$$\Rightarrow P_p = \gamma z \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2c\tan\left(45^\circ + \frac{\phi}{2}\right)$$

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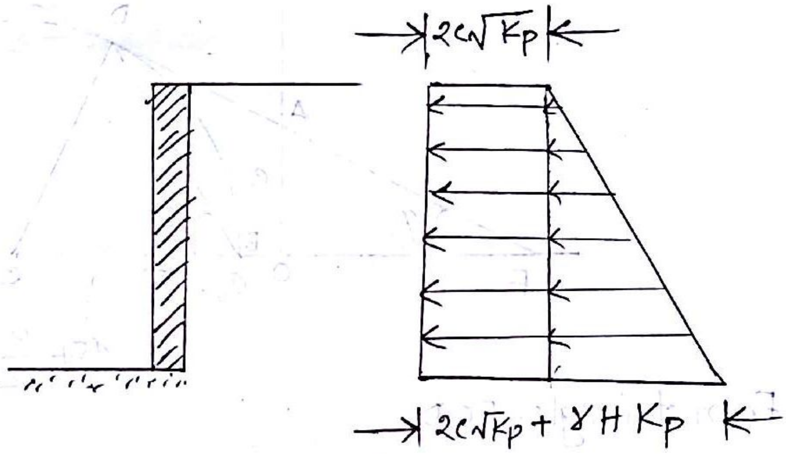
$\therefore P_p = \gamma z K_p + 2c\sqrt{K_p}$ where, $K_p = \tan^2(45+\frac{\phi}{2}) = \frac{1+\sin\phi}{1-\sin\phi}$

and, $\sqrt{K_p} = \tan(45+\frac{\phi}{2}) = \frac{\cos\phi}{1-\sin\phi}$

if $z=0$,

$$P_p = 2c\sqrt{K_p}$$



If $z=H$,

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

Total pressure is given by,

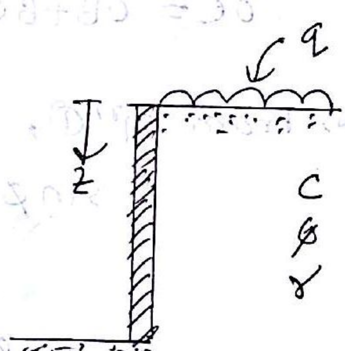
$$P_p = \int_0^H (\gamma z K_p + 2c\sqrt{K_p})\,dz$$

$$\therefore P_p = \frac{1}{2}\gamma K_p H^2 + 2c\sqrt{K_p}\,H$$

Backfill with surcharge:

If the backfill carries a surcharge q,

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



Total pressure is given by,

$$P_p = 2c\sqrt{K_p}\,H + \frac{1}{2}K_p\gamma H^2 + qK_pH$$

Coulomb's Wedge Theory: 14, 13, 07, 08

Coulomb (1776) proposed a theory for the determination of the earth pressure. The following assumptions were made:

(i) The backfill is dry; cohesionless, homogeneous, isotropic and plastic material.

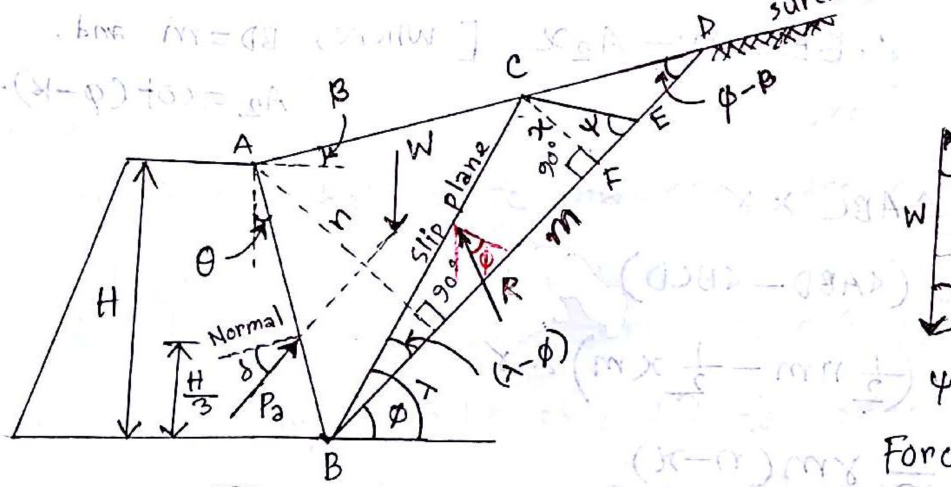
(ii) The slip surface is a plane surface passing through the heel of the wall.

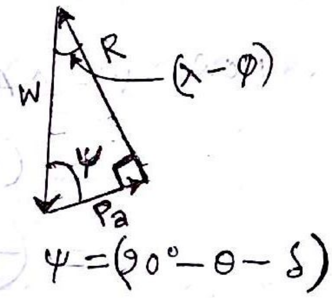
(iii) The wall surface is rough.

(iv) The sliding wedge itself acts as a rigid body.

Coulomb's Active Pressure in Cohesionless Soils: 14, 13, 12, 17

A retaining wall with an inclined back face and a sloping dry granular backfill as shown in figure below:





$\psi = (90^\circ - \theta - \delta)$

Force Triangle

The sliding wedge ABC is in equilibrium under three forces:

(1) Weight of the wedge (W)

(2) Reaction R on the slip surface **BC**

(3) Reaction Pa from the wall

From geometry, $\triangle BCE$ and force triangle are similar

Hence, $\frac{P_a}{W} = \frac{CE}{BE} \Rightarrow P_a = \frac{CE}{BE} \times W$ ...... (1)

Now, from $\triangle CEF$,

$$\operatorname{cosec} \psi = \frac{CE}{CF}$$

$$\Rightarrow CE = CF \operatorname{cosec} \psi \quad [\because CF = x]$$

$$\Rightarrow CE = x \operatorname{cosec} \psi$$

$$\Rightarrow CE = x A_1 \quad [\text{Let, } A_1 = \operatorname{cosec} \psi]$$

Again, from $\triangle CDF$ & $\triangle CEF$,

$$DF = x \cdot \cot(\phi - \beta)$$

$$\text{and, } FE = x \cot \psi$$

Now,

$$BE = BD - DE = BD - (DF - FE) = m - x[\cot(\phi-\beta) - \cot\psi]$$

$$\therefore BE = m - A_2 x \quad [\text{where, } BD = m \text{ and, } A_2 = \cot(\phi-\beta) - \cot\psi]$$

Now, $W = \triangle ABC \times \gamma$

$$= (\triangle ABD - \triangle BCD) \times \gamma$$

$$= \left(\frac{1}{2} nm - \frac{1}{2} xm\right) \times \gamma$$

$$\therefore W = \frac{1}{2} \gamma m (n - x)$$

putting the values of CE, BE and W in equation (1) we obtain,

$$P_a = W \times \frac{CE}{BE}$$

$$\therefore P_a = \frac{1}{2} \gamma m (n-x) \times \frac{A_1 x}{(m - A_2 x)}$$

For maximum active earth pressure, 13

$$\frac{dP_a}{dx} = 0$$

$$\frac{d}{dx}\left[\frac{1}{2}\gamma m(n-x) \times \frac{A_1 x}{(m-A_2 x)}\right] = 0$$

$$\Rightarrow \frac{d}{dx}\left[\frac{1}{2}\gamma m A_1 \times \frac{nx-x^2}{m-A_2 x}\right] = 0$$

$$\Rightarrow (n-2x)(m-A_2 x) = -A_2(nx-x^2)$$

$$\Rightarrow mn - mx = x(m-A_2 x)$$

$$\Rightarrow \frac{mn}{2} - \frac{mx}{2} = \frac{x \cdot BE}{2}$$

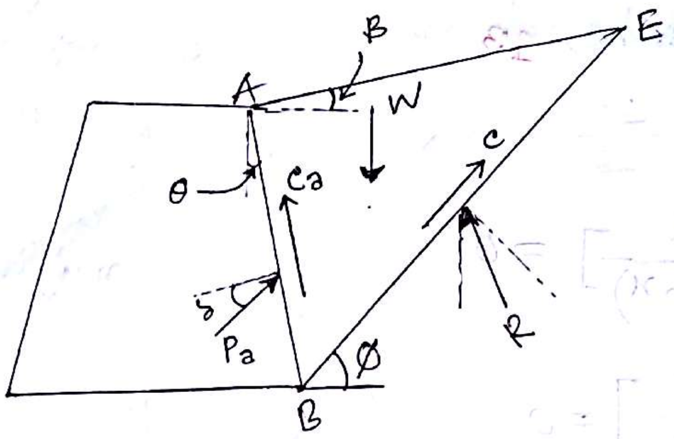
$$\Rightarrow \triangle ABD - \triangle BCD = \triangle BCE$$

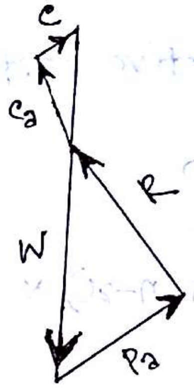
$\therefore$ $\boxed{\triangle ABC = \triangle BCE}$ → which is the criteria for maximum active earth pressure.*

The criteria for maximum active earth pressure is that the slip plane is so chosen that $\triangle ABC$ and $\triangle BCE$ are equal in area.

Coulomb's Active earth pressure for cohesive soils:

The coulombs theory can be extended to cohesive soils. In addition to the three forces (W, P_a and R) considered for the cohesionless soils in the preceding sections, there are two more forces, namely, (1) the cohesive force C acting on the failure plane BE, (2) the adhesive force C_a acting on the back of the wall AB as shown in figure:





The force polygon

In all there are five forces which keep the wedge in equilibrium.

Here, W = weight of the wedge ABE

C = c × BE where, c = unit cohesion

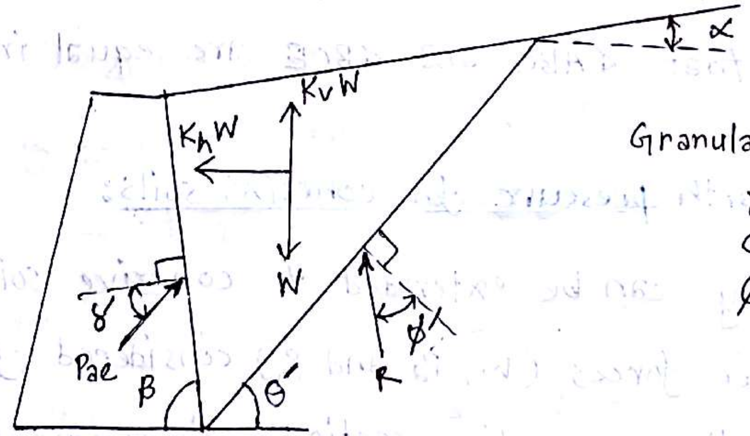
$C_a = c_a \times AB$ where, c_a = unit adhesion

R = Reaction on the slip surface BE.

P_a = Reaction from the wall.

13,10,09

Incorporation of earthquake with the coulomb's active earth pressure:



Granular backfill

γ

c' = 0

ø'

considering the above figure, The active force per unit length of the wall due to the earthquake can be given as follows:

$$P_{ae} = \frac{1}{2} K_{ae} \gamma H^2 (1 - K_v) \quad \text{------} \quad (1)$$

Where, K_{ae} = active pressure co-efficient due to earthquake

$$K_{ae} = \frac{\sin^2(\phi' + \beta - \theta')}{\cos\theta' \cdot \sin^2\beta \cdot \sin(\beta - \theta' - \delta')\left[1 + \sqrt{\frac{\sin(\phi'+\delta')\cdot\sin(\phi'-\theta'-\alpha)}{\sin(\beta-\delta'-\theta')\cdot\sin(\alpha+\beta)}}\right]^2}$$

and, $\theta' = \tan^{-1}\left[\frac{K_h}{(1-K_v)}\right]$

Here, $K_h = \frac{\text{horizontal earthquake acceleration component}}{\text{acceleration due to gravity, } g}$

$K_v = \frac{\text{vertical earthquake acceleration component}}{\text{acceleration due to gravity, } g}$

Note that, for no earthquake condition,

$K_h = 0$, $K_v = 0$ and $\theta' = 0$

Hence,

$$K_{ae} = K_a = \frac{\sin^2(\beta + \phi')}{\sin^2\beta \sin(\beta - \delta')\left[1 + \sqrt{\frac{\sin(\phi'+\delta')\cdot\sin(\phi'-\alpha)}{\sin(\beta-\delta')\cdot\sin(\alpha+\beta)}}\right]^2}$$

14,12,07

Difference between Rankine's theory and coulomb's wedge theory:

Rankine's Theory	Coulomb's wedge theory
1. The back of the wall is vertical and smooth.	1. The surface of wall is rough.
2. Wall friction is neglected.	2. Wall friction is considered.
3. The soil mass is semi-infinite.	3. The soil mass is isotropic.
4. The ground surface is plane.	4. The ground surface is irregular.
5. Elementary soil mass is considered.	5. A sliding wedge is considered.

Difference between active and passive earth pressure:

Active pressure	Passive pressure
1. If the wall moves away or tilt away from the soil retained, a triangular soil wedge behind the wall is fail. The lateral pressure at this condition is known as active earth pressure.	1. If the wall moves towards or pushed in to the soil retained, a triangular soil wedge behind the wall is fail. The lateral pressure at this condition is known as passive earth pressure.
2. The wall moves away from the earthfill in active case.	2. The wall moves towards the earth fill in passive case.
3. Soil gives pressure to the wall.	3. Wall gives pressure to the soil.
4. There is a decrease in the pressure on the wall.	4. There is an increase in the pressure on the wall.
5. The co-efficient of active earth pressure is smaller than passive.	5. The co-efficient of passive earth pressure is larger than active.

Stress Distribution

[Class Test] '15 series

Geo static stress: At a certain point with in the soil mass, the stresses are caused due to both surface loadings as well as due to self-weight of soil above it.

The stresses due to self weight is known as Geostatic stresses.

The stresses due to surface loadings is known as Excess stresses.

[classtest] '15 series

Calculation of subsurface stress increase:

(i) under point / concentrated loading.

(ii) under uniformly loaded circular area.

(iii) under line load.

(iv) Under strip load.

(v) under uniformly loaded rectangular area.

Practical Examples:

(i) Under point loading: Wheel Load

(ii) Under uniformly loaded circular area: Oil Tank, Boiler.

(iii) Under line load.

(iv) Under strip load: Brick Foundation.

(v) Uniformly distributed rectangular area: spread footing, isolated footing.

(I) concentrated Loading:

06, 09, 10

Boussinesq's Assumptions: The following assumptions are made by the theory of elasticity:

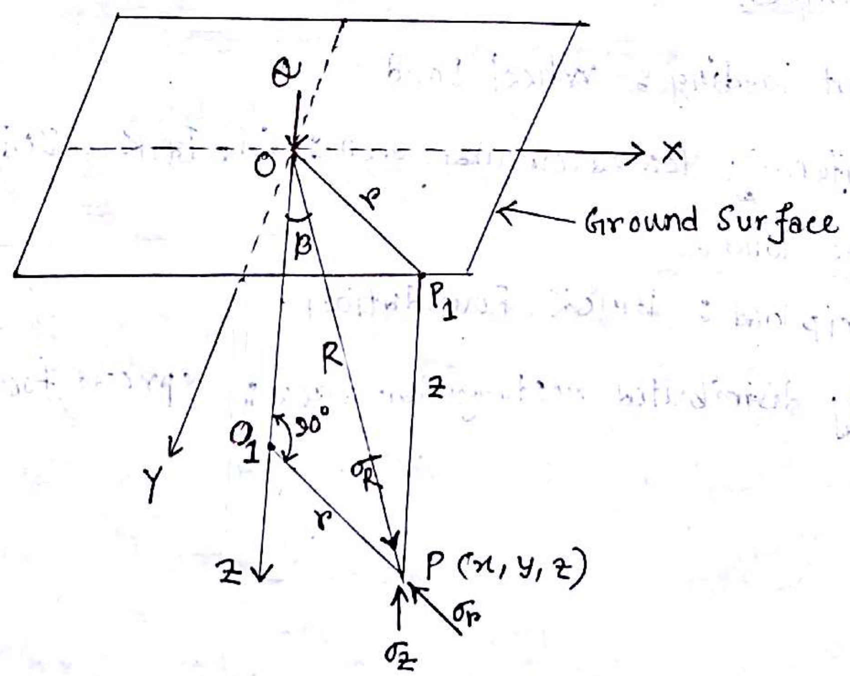
(i) The soil mass is an elastic medium, for which the modulus of elasticity E is constant.

(ii) The soil mass is homogeneous, that is: All its constituent parts or elements are similar and it has identical properties at every point in it in identical directions.

(iii) The soil mass is isotropic, that is: it has identical elastic properties in all directions through any point of it.

(iv) The soil mass is semi-infinite, that is: it extends infinitely in all directions below a level surface.

Derivation of Boussinesq's Equation:



Let a point load Q act at the ground surface, at a point O. Let us, find the stress components at a point P in the soil mass having a radial horizontal distance r and vertical distance z from the point O.

Vertical stress:

Using logarithmic stress function, the polar radial stress may be expressed as: $\sigma_R = \frac{3}{2} \times \frac{Q}{\pi} \times \frac{\cos\beta}{R^2}$

where, R = polar co-ordinate at point P $= \sqrt{r^2+z^2}$

$\Rightarrow R = \sqrt{x^2+y^2+z^2}$ and, $\cos\beta = \frac{z}{R}$

In the cylindrical co-ordinates the corresponding vertical stress, σ_z is given by,

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

$$= \frac{3}{2}\times\frac{Q}{\pi}\times\frac{\cos\beta}{R^2}\times\cos^2\beta \quad \left[\because \sigma_R = \frac{3Q}{2\pi}\times\frac{\cos\beta}{R^2}\right]$$

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

$$= \frac{3Q}{2\pi}\times\frac{z^3}{R^3}\times\frac{1}{R^2} \quad \left[\because \cos\beta = \frac{z}{R}\right]$$

$$= \frac{3Q}{2\pi}\times\frac{z^3}{R^5}$$

$$= \frac{3Q}{2\pi}\times\frac{z^3}{(x^2+y^2+z^2)^{5/2}} \quad \left[\because R = \sqrt{x^2+y^2+z^2}\right]$$

$$= \frac{3Q}{2\pi}\times\frac{z^3}{(r^2+z^2)^{5/2}} \quad \left[\because r^2 = x^2+y^2\right]$$

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

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

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

= Boussinesq's influence factor

It shout be emphasized that, vertical normal stress does not depend on modulus of elasticity (E) and poisson's ratio (μ).

Tangential stress:

In the cylindrical co-ordinates the corresponding tangential stress is given by,

$$\tau_{rz} = \frac{1}{2} \sigma_R \sin 2\beta$$

$$= \frac{1}{2} \times \frac{3}{2} \times \frac{Q}{\pi} \times \frac{\cos\beta}{R^2} \times 2\sin\beta \cdot \cos\beta$$

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

$$= \frac{3Q}{2\pi} \times \left(\frac{z}{R}\right)^2 \times \left(\frac{r}{R}\right) \times \frac{1}{R^2} \quad \left[\because \cos\beta = \frac{z}{R} \ \& \ \sin\beta = \frac{r}{R}\right]$$

$$= \frac{3Q}{2\pi} \times \frac{z^2 r}{R^5}$$

$$= \frac{3Qr}{2\pi} \times \frac{z^2}{(r^2+z^2)^{5/2}} \quad \left[\because R = \sqrt{r^2+z^2}\right]$$

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

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

$$\boxed{\tau_{rz} = \frac{Qr}{z^3} \times K_B} \quad \text{where, } \boxed{K_B = \frac{3}{2\pi} \times \left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}}$$

= Boussinesq's influence factor

It should be emphasized that, tangential stress (shearing stress) does not depend on modulus of elasticity (E) and poisson's ratio (μ)

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Relation between vertical stress and tangential stress:

We know, $\sigma_z = \frac{Q}{z^2} \times K_B$ ......(I)

and $\tau_{rz} = \frac{Qr}{z^3} \times K_B$ ......(II)

From (II) $\div$ (I) we obtain, $\tau_{rz} = \frac{Qr}{z^3} \times K_B \times \frac{z^2}{Q} \times \frac{1}{K_B}$

$$\boxed{\therefore \tau_{rz} = \left(\frac{r}{z}\right) \sigma_z}$$

Hence, Tangential stress is related with vertical stress by a non-dimensional factor $\left(\frac{r}{z}\right)$.

Vertical stress directly below the point load:

The intensities of vertical pressure, directly below the point load (where $r=0$), on its axis of loading :-

$$\text{if } r=0, \quad K_B = \frac{3}{2\pi} \times \left[\frac{1}{1+\left(\frac{0}{z}\right)^2}\right]^{5/2} = \frac{3}{2\pi} = 0.4775$$

We know, $\sigma_z = \frac{Q}{z^2} K_B$

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

Pressure Distribution Diagram:

By means of Boussinesq's stress distribution theory, the following vertical pressure distribution diagrams can be prepared:

1. Stress isobar or isobar diagram.
2. Vertical pressure distribution on a horizontal plane.
3. Vertical pressure distribution on a vertical line.

Isobar Diagram:

07, 09, 10, 14

Isobar: An isobar is a curve or contour connecting all points below the ground surface of equal vertical pressure. An isobar is a spatial, curved surface of the shape of a bulb, because the vertical pressure on a given horizontal plane is the same in all directions at points located at equal radial distance around the axis of loading.

09, 10

Pressure bulb: The zone in a loaded soil mass bounded by an isobar of given vertical pressure intensity is called a pressure bulb. The vertical pressure at every point on the surface of pressure bulb is same.

Drawing an isobar diagram:

Suppose an isobar of $\sigma_z = 0.25Q$ per unit area is to be plotted.

We know, $$\sigma_z = \frac{Q}{z^2} \times K_B$$

$$\Rightarrow 0.25Q = \frac{Q}{z^2} K_B$$

$$\therefore K_B = 0.25 z^2$$

For maximum z, $$\sigma_z = \frac{0.4775Q}{z^2}$$

$$\Rightarrow 0.25Q = \frac{0.4775Q}{z^2}$$

$$z = 1.38$$

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

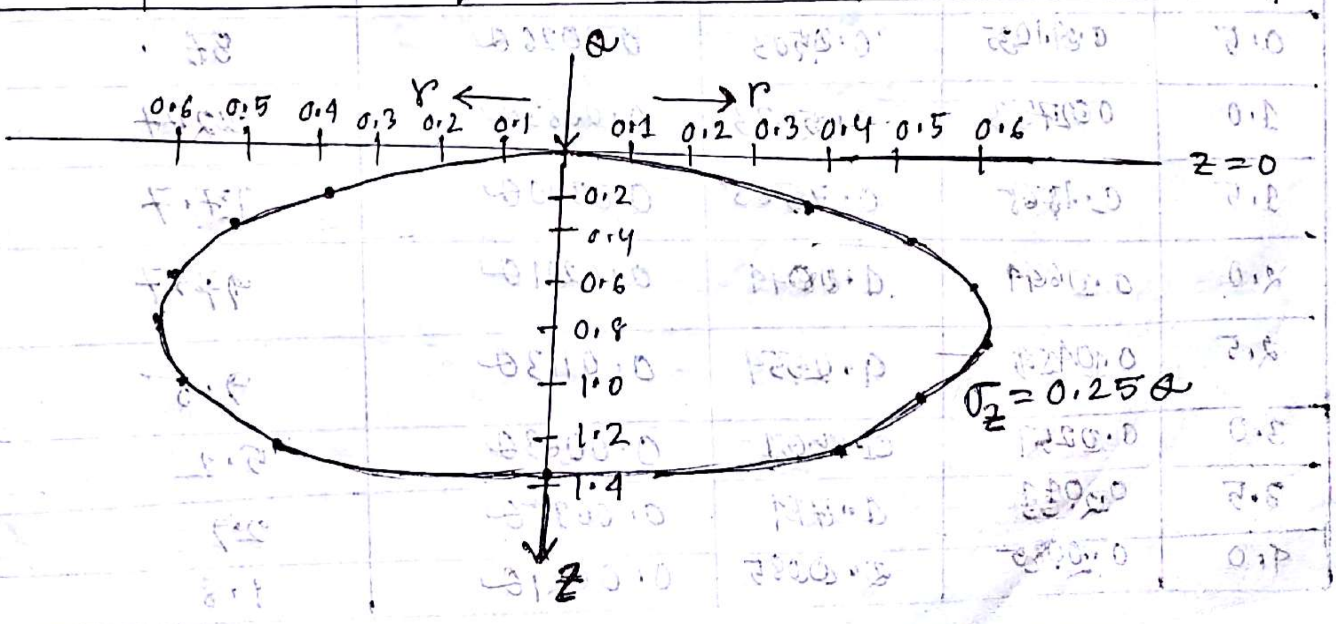
$$\Rightarrow \left[1+(\frac{r}{z})^2\right]^{5/2} = \frac{3}{2\pi K_B}$$

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

$$\Rightarrow \left(\frac{r}{z}\right)^2 = \left(\frac{3}{2\pi K_B}\right)^{2/5} - 1$$

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

z	K_B	r/z	r (units)
0.2	0.0100	1.92	0.38
0.4	0.0400	1.30	0.52
0.6	0.0900	0.97	0.58
0.8	0.1600	0.74	0.59
1.0	0.2500	0.54	0.54
1.2	0.3600	0.34	0.41
1.38	0.4775	0.00	0.00



2011, 2014

Significance of pressure bulb:

1. The vertical pressure at every point on the surface of the pressure bulb is same.

2. Pressure at points inside the bulb are greater than that a point on the surface of the bulb.

3. Presure at points outside the bulb are smaller than that a point on the surface of the bulb.

4. The smaller value of σ_z, higher the area of pressure bulb.

Vertical pressure distribution on a horizontal plane: 2015, 2019

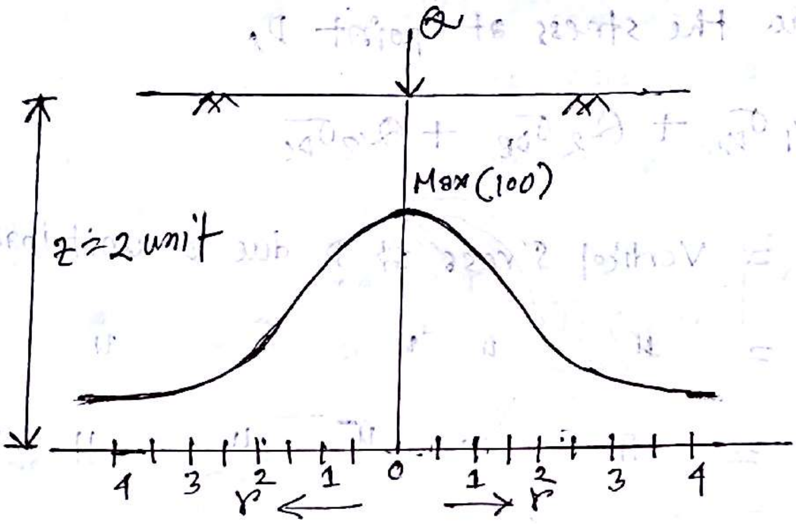
Let us, Determine the stresses at a depth $z = 2$ unit.

Therefore, $\sigma_z = \frac{Q}{z^2} \times K_B$

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

$\therefore \sigma_z = 0.25 K_B Q$ where, $K_B = \frac{3}{2\pi} \times \left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$

r	r/z	K_B	σ_z	%
0.0	0.0	0.4775	0.1194Q	100
0.5	0.25	0.4103	0.1026 Q	86
1.0	0.5	0.2733	0.0683Q	57
1.5	0.75	0.1565	0.0390Q	32.7
2.0	1.0	0.0844	0.0211Q	17.7
2.5	1.25	0.0454	0.0113Q	9.5
3.0	1.5	0.0251	0.0063Q	5.2
3.5	1.75	0.014	0.0035Q	2.9
4.0	2.0	0.0085	0.0021Q	1.8



Note:

When, The horizontal distance = 2 × Depth, the vertical pressure due to point load is negligible.

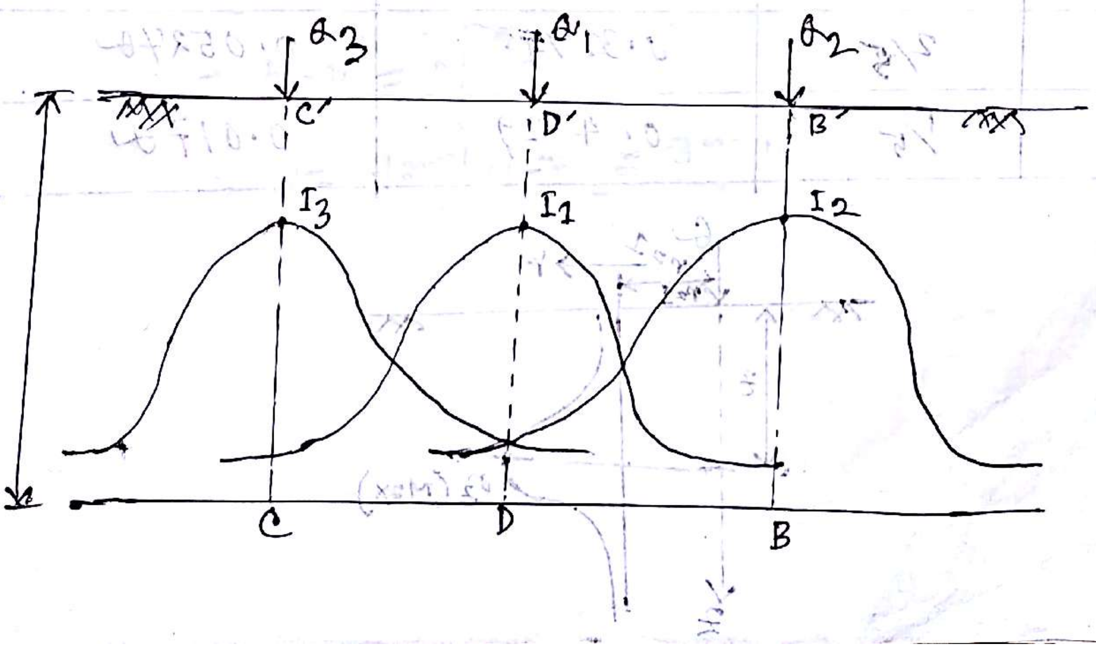
2013, 2015

Influence Diagram:

An influence line diagram is the vertical stress distribution diagram on a horizontal plane at a given depth due to a unit concentrated load.

Application of influence diagram: 2013, 2015

It is helpful to determine vertical stress at any point on the horizontal plane due to number of concentrated loads.



Let us, Determine the stress at point D,

$$(\sigma_z)_D = Q_1 \sigma_{DD} + Q_2 \sigma_{DB} + Q_3 \sigma_{DC}$$

Here, σ_{DD} = Vertical Stress at D due to unit load at D'

σ_{DB} = " " " " " at B'

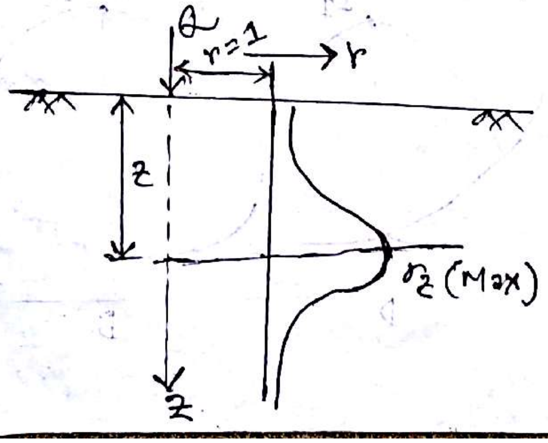
σ_{DC} = " " " " " at C'

Vertical stress distribution on a vertical line:

Let us, Determine the stresses at a radial distance $r = 1$ unit.

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

z	r/z	K_B	σ_z
0.25	4	0.0004	0.0064 Q
0.5	2	0.0085	0.0340 Q
1.00	1	0.0084	0.0844 Q
2.00	0.5	0.2733	0.0683 Q
2.5	2/5	0.3294	0.0527 Q
5.0	1/5	0.4329	0.017 Q



Prove that the maximum vertical stress on a vertical line at a constant radial distance r from the axis of a vertical load is induced at the point of intersection of the vertical line with a radial line at $\beta = 39°15'$

<u>proof:</u> We know, $\sigma_z = \frac{3Q}{2\pi z^2}\left[\frac{1}{1+(\frac{r}{z})^2}\right]^{5/2}$

For maxima; $\frac{d\sigma_z}{dz} = 0$

$$\boxed{\frac{d}{dz}\left[\frac{3Q}{2\pi z^2}\left[\frac{1}{1+(\frac{r}{z})^2}\right]^{5/2}\right] = 0} \Rightarrow \frac{3Q}{2\pi}\frac{d}{dz}\left[\frac{z^3}{(r^2+z^2)^{5/2}}\right] = 0$$

$$\Rightarrow \frac{3Q}{2\pi} \cdot \frac{3z^2(r^2+z^2)^{5/2} - z^3 \cdot \frac{5}{2}(r^2+z^2)^{3/2} \cdot 2z}{(r^2+z^2)^5} = 0$$

$$\Rightarrow 3z^2(r^2+z^2)^{5/2} = z^3 \cdot \frac{5}{2}(r^2+z^2)^{3/2} \cdot 2z$$

$$\Rightarrow (r^2+z^2) = \frac{5}{3}z^2$$

$$\Rightarrow 3r^2 = 2z^2$$

$$\Rightarrow z = \sqrt{\frac{3}{2}}\, r$$

$$\Rightarrow \frac{r}{z} = \sqrt{\frac{2}{3}}$$

$$\Rightarrow \tan\beta = \sqrt{\frac{2}{3}}$$

$$\therefore \beta = \tan^{-1}\sqrt{\frac{2}{3}} = 39°15'$$

2006, 2013

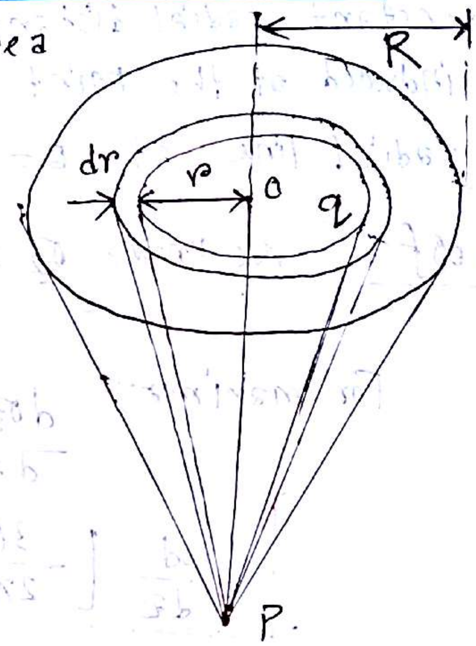
Vertical stress under uniformly loaded circular area:

Let, q = intensity of load per unit area

R = Radius of loaded area

r = Radius of elementary ring

dr = width of elementary ring



The load on elementary ring $= q \times 2\pi r\,dr$

we know,

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

Following this eqn,

$$\Delta\sigma_z = \frac{3\times(q\times 2\pi r\,dr)}{2\pi} \times \frac{1}{z^2} \times \frac{1}{\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}$$

$$\Rightarrow \Delta\sigma_z = \frac{3qr\,dr}{z^2} \times \frac{z^5}{(r^2+z^2)^{5/2}}$$

$$\Rightarrow \Delta\sigma_z = 3qz^3 \times \frac{r\,dr}{(r^2+z^2)^{5/2}}$$

vertical stress due to entire load is given by,

$$\sigma_z = 3qz^3 \int_0^R \frac{r\,dr}{[r^2+z^2]^{5/2}} \quad \ldots\ldots (1)$$

Let, $r^2+z^2 = u$

$$\Rightarrow 2r\,dr = du$$

$$\therefore r\,dr = \frac{du}{2}$$

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When $r=0$, $u=z^2$

and $r=R$, $u=R^2+z^2$

Therefore equation (i) becomes,

$$\sigma_z = 3qz^3 \int_{z^2}^{(R^2+z^2)} \frac{du}{2[u]^{5/2}}$$

$$= \frac{3qz^3}{2}\left(-\frac{2}{3}\right)\left[u^{-3/2}\right]_{z^2}^{R^2+z^2}$$

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

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

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

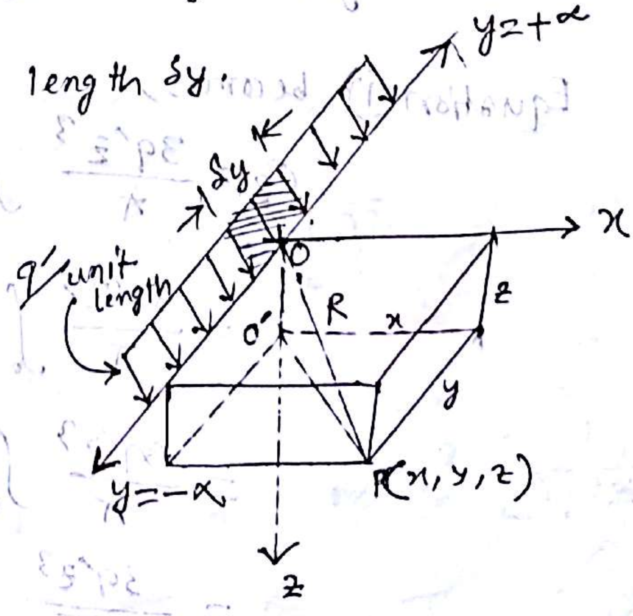
$$\boxed{\therefore \sigma_z = q \times K_c}$$ where, $K_c = \left[1 - \left\{\frac{1}{1+(\frac{R}{z})^2}\right\}^{3/2}\right]$

Vertical stress under a line load: 2015, 2012

Let, q' = intensity of line load per unit length.

consider, the load acting on a small length δy.

$\therefore$ point load $= q'\delta y$



We know,

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

Following this equation,

$$\Delta\sigma_z = \frac{3q'\delta y}{2\pi z^2}\left[\frac{z^5}{(r^2+z^2)^{5/2}}\right]$$

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$$\Rightarrow \Delta\sigma_z = \frac{3q'\,\delta y}{2\pi} \times \frac{z^3}{(r^2+z^2)^{5/2}}$$

By Integration, we obtain,

$$\sigma_z = \frac{3q'z^3}{2\pi}\int_{-\alpha}^{\alpha}\frac{dy}{(r^2+z^2)^{5/2}}$$

$$= \frac{3q'z^3}{2\pi}\times 2\int_0^{\alpha}\frac{dy}{(r^2+z^2)^{5/2}}$$

$$= \frac{3q'z^3}{\pi}\int_0^{\alpha}\frac{dy}{(x^2+y^2+z^2)^{5/2}}$$

Let, $x^2+z^2=u^2$

Hence, $\sigma_z = \dfrac{3q'z^3}{\pi}\displaystyle\int_0^{\alpha}\frac{dy}{(u^2+y^2)^{5/2}}$ ...... (1)

Let, $y = u\tan\theta$

$\Rightarrow dy = u\sec^2\theta\, d\theta$

when, $y=0$, $\theta=0$

$y=\alpha$, $\theta = \frac{\pi}{2}$

Equation (1) becomes,

$$\sigma_z = \frac{3q'z^3}{\pi}\int_0^{\pi/2}\frac{u\sec^2\theta\,d\theta}{(u^2+u^2\tan^2\theta)^{5/2}}$$

$$= \frac{3q'z^3}{\pi}\int_0^{\pi/2}\frac{u\sec^2\theta\,d\theta}{u^5(1+\tan^2\theta)^{5/2}}$$

$$= \frac{3q'z^3}{\pi}\int_0^{\pi/2}\frac{\sec^2\theta\,d\theta}{u^4(\sec^2\theta)^{5/2}}$$

$$= \frac{3q'z^3}{\pi}\int_0^{\pi/2}\frac{d\theta}{u^4\sec^3\theta}$$

$$\Rightarrow \sigma_z = \frac{3q'z^3}{\pi u^4}\int_0^{\pi/2} \cos^3\theta \, d\theta$$

$$= \frac{3q'z^3}{\pi u^4}\int_0^{\frac{\pi}{2}} \cos^2\theta \cdot \cos\theta \, d\theta$$

$$= \frac{3q'z^3}{\pi u^4}\int_0^{\pi/2} (1-\sin^2\theta) \cdot \cos\theta \, d\theta \text{ ------ (ii)}$$

Let, $t = \sin\theta$

$\therefore dt = \cos\theta \, d\theta$

when $\theta = 0$, $t = 0$

$\theta = \frac{\pi}{2}$, $t = 1$

Equation (ii) becomes,

$$\sigma_z = \frac{3q'z^3}{\pi u^4} \times \int_0^1 (1-t^2)\, dt$$

$$= \frac{3q'z^3}{\pi u^4} \times \left[t - \frac{t^3}{3}\right]_0^1$$

$$= \frac{3q'z^3}{\pi u^4} \times \left(1 - \frac{1}{3}\right)$$

$$= \frac{3q'z^3}{\pi (x^2+z^2)^2} \times \frac{2}{3}$$

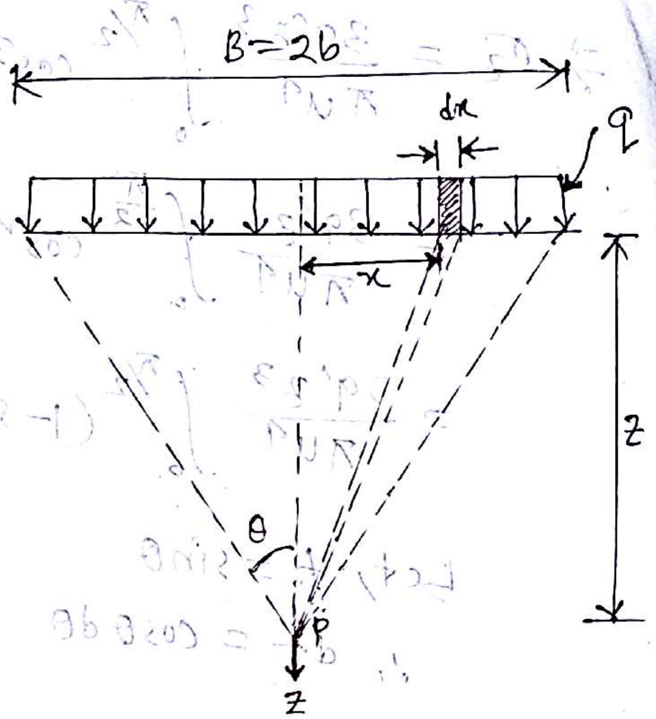
$$= \frac{2q'z^3}{\pi} \times \frac{1}{z^4\left\{1+\left(\frac{x}{z}\right)^2\right\}^2}$$

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

$$\boxed{\therefore \sigma_z = \frac{q'}{z} \times K_p}$$

where, $K_p = \frac{2}{\pi} \times \left[\frac{1}{1+\left(\frac{x}{z}\right)^2}\right]^2$

Vertical stress under a strip Load:



Here, $B = 2b$ = width of strip load.

q = intensity of load.

Let us, consider the load acting on a small elementary width 'dx' at a distance 'x' from the centre.

$q' =$ line load $= q\,dx$

We know,

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

Following this equation,

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

The stress due to entire strip,

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

$$\sigma_z = 2 \times \frac{2q}{\pi z}\int_{0}^{b}\left[\frac{1}{1+\left(\frac{x}{z}\right)^2}\right]^2 dx \quad \ldots\ldots (1)$$

Let, $\frac{x}{z} = \tan u$; $\therefore dx = z \sec^2 u\, du$

Therefore, when, $x = 0$, $u = 0$

and $x = b$, $u = \theta$

Equation (i) becomes,

$$\sigma_z = 2\times\frac{2q}{\pi z}\int_0^\theta \frac{z\sec^2 u}{(1+\tan^2 u)^2}\,du$$

$$= \frac{4q}{\pi}\int_0^\theta \frac{\sec^2 u}{\sec^4 u}\,du$$

$$= \frac{4q}{\pi}\int_0^\theta \cos^2 u\,du$$

$$= \frac{4q}{\pi}\int_0^\theta \frac{1}{2}\times(1+\cos 2u)\,du$$

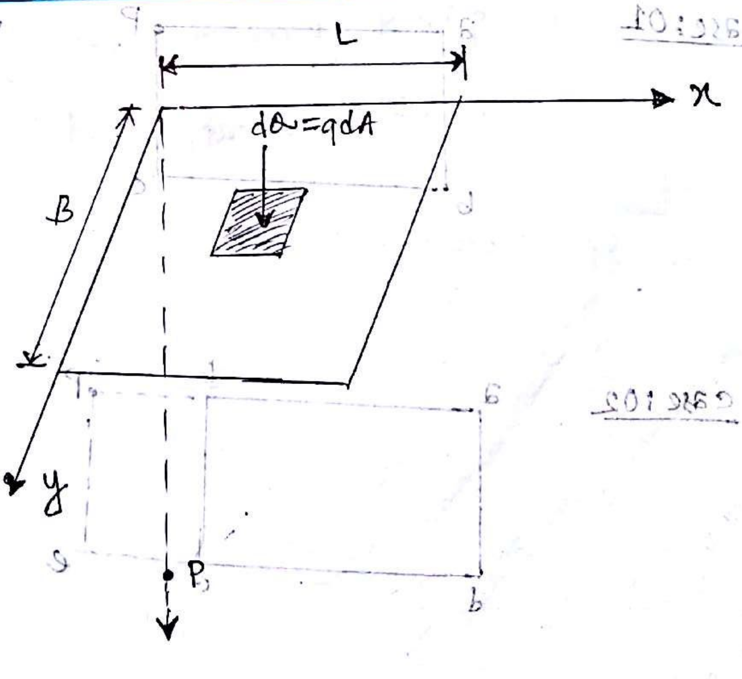
$$= \frac{2q}{\pi}\left[u+\frac{\sin 2u}{2}\right]_0^\theta$$

$$= \frac{2q}{\pi}\left[\theta+\frac{\sin 2\theta}{2}\right]$$

$$= \frac{2q}{\pi}\times\frac{2\theta+\sin 2\theta}{2}$$

$$\boxed{\sigma_z = \frac{q}{\pi}\times(2\theta+\sin 2\theta)}$$

vertical Stress at a corner of a uniformly loaded Rectangular Area:



q = intensity of load

The stress at depth z is given by taking $dQ = q\,dA$

$\therefore dQ = q\,dx\,dy$

$$d\sigma_z = \frac{3(q\,dx\,dy)z^3}{2\pi}\times\frac{1}{(x^2+y^2+z^2)^{5/2}}$$

Integrating,

$$\sigma_z = \frac{3qz^3}{2\pi}\int_0^L\int_0^B \frac{dx\,dy}{(x^2+y^2+z^2)^{5/2}}$$

Newmark's solution of the above integration is as follows:

$$\sigma_z = \frac{q}{2\pi}\left[\frac{mn}{\sqrt{m^2+n^2+1}} \times \frac{m^2+n^2+2}{m^2+n^2+m^2n^2+1} + \sin^{-1}\left(\frac{mn}{m^2+n^2+m^2n^2+1}\right)\right]$$

$$= \frac{q}{2\pi}\left[\frac{mn}{\sqrt{f}} \times \frac{f+1}{f+m^2n^2} + \sin^{-1}\left(\frac{mn}{f+m^2n^2}\right)\right]$$

$\therefore \sigma_z = K_N q$ where, $f = m^2+n^2+1$, and $m = \frac{B}{z}$, $n = \frac{L}{z}$

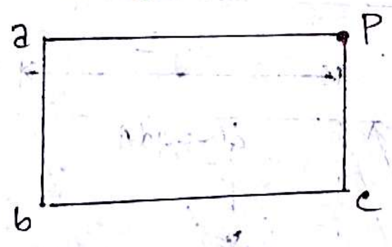
$$K_N = \frac{1}{2\pi}\times\left[\frac{mn}{\sqrt{f}} \times \frac{f+1}{f+m^2n^2} + \sin^{-1}\left(\frac{mn}{f+m^2n^2}\right)\right]$$

Alternatively,

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

Vertical stress at any point under a Rectangular area:

Case: 01

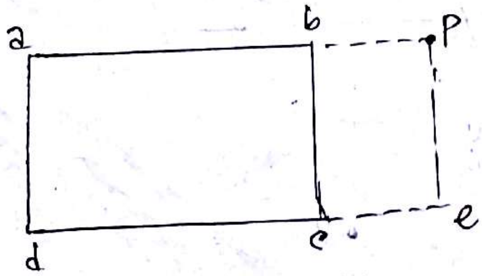


vertical stress at depth z is,

$$\sigma_z = K_N q$$

K_N = Newmark's influence factor for area abcp

case: 02

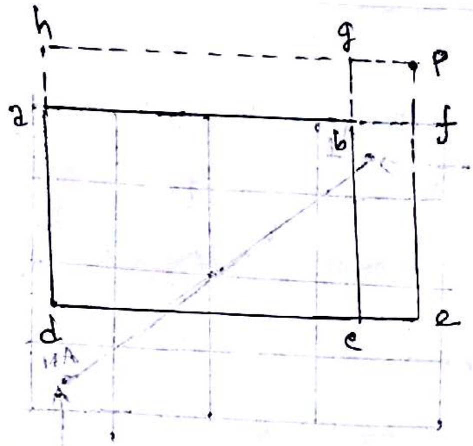


Vertical stress at depth z is,

$$\sigma_z = q(K_{N_1} - K_{N_2})$$

K_{N_1} = Newmark's influence factor for area adep.

K_{N_2} = Newmark's influence factor for area bcep case: 03



vertical stress at depth z is,

$$\sigma_z = q\,(K_{N_1} - K_{N_2} - K_{N_3} + K_{N_4})$$

where,

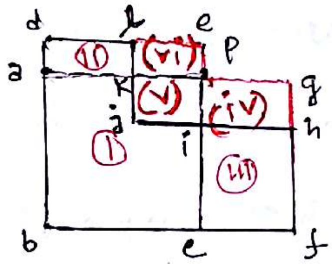
K_{N_1} = Newmark's influence factor for area hdep

K_{N_2} = " " " " hafp

K_{N_3} = " " " " gcep

K_{N_4} = " " " " gbfp

#



vertical stress at depth z is,

$$\sigma_z = q\,(K_{N_1} + K_{N_2} + K_{N_3} - K_{N_4} - K_{N_5} - K_{N_6})$$

where,

K_{N_1} = for area abcp

K_{N_2} = for area edap

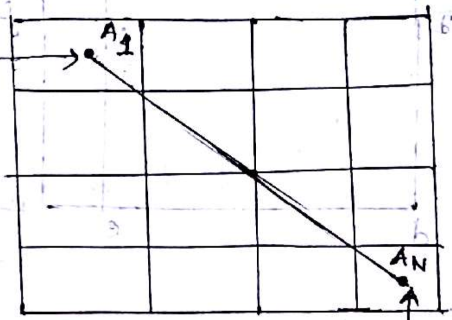
K_{N_3} = for area cfgp

K_{N_4} = for area ihgp

K_{N_5} = for area kaip

K_{N_6} = for area elkp

Equivalent point load Method:



(i) The entire area is divided in to a number of small area units

(ii) The distributed load over the entire area is replaced by a point load of the same magnitude acting at the centroid of the area.

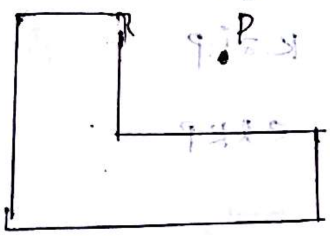
$$\sigma_{z_1} = \frac{Q_1}{z^2} \times K_{B_1} \longrightarrow f(r_1, z)$$

$$\sigma_{z_2} = \frac{Q_2}{z^2} \times K_{B_2} \longrightarrow f(r_2, z)$$

$$\sigma_{z_n} = \frac{Q_n}{z^2} \times K_{B_n}$$

$$\Sigma \sigma_z = \frac{1}{z^2} \times \left(Q_1 K_{B_1} + Q_2 K_{B_2} + \cdots\cdots + Q_n K_{B_n}\right)$$

#



(see in Mathematical problem)

* **Limitations:**

1. The accuracy of the result will depend upon the size of the area unit chosen.

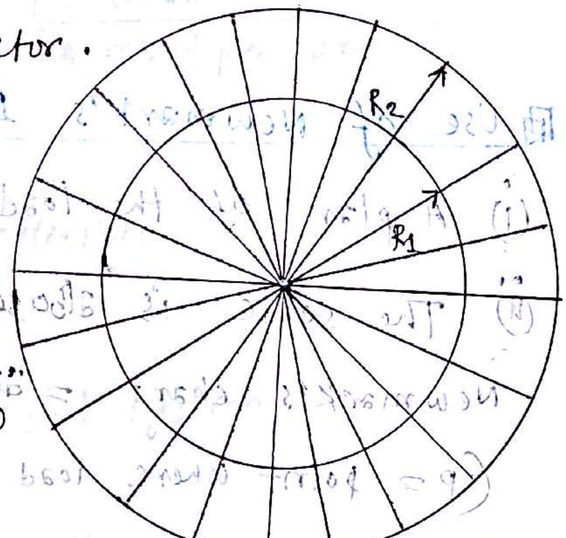
2. The smaller the size of the area, the higher the accuracy.

Newmark's Influence Chart:

Sometimes vertical stress σ_z for other shapes of foundation, the rectangular shape (circular shape) is required. In such cases Newmark's influence chart is extremly used. It is a more accurate method of determining vertical stress at a point under a uniformly loaded area of any shape.

Drawing of Newmark's influence chart:

Let us consider, concentric circle of radius $= R_1$. Let the circle be divided into 20 equal sector.



Therefore, load on each sector $= \frac{q}{20}$

Now vertical stress at the center is given by, $\sigma_z = \frac{q}{20} \times \left[1 - \left[\frac{1}{1+(\frac{R_1}{z})^2}\right]^{3/2}\right]$ ...... (I)

Let, $\sigma_z = 0.005q$

$\therefore$ Equation (I) Becomes,

$$0.005q = \frac{q}{20} \times \left[1 - \left(\frac{1}{1+(\frac{R_1}{z})^2}\right)^{3/2}\right] \text{ ...... (II)}$$

From (II),

$$R_1 = 0.27z \text{ ...... (III)}$$

Let us consider again, a concentric circle radius $= R_2$. Let the circle be divided into 20 equal sectors.

Load on each sector (for top portion) $= \frac{q}{20}$

Let, $\sigma_z = 0.005q$

Thus, $2 \times \sigma_z = \frac{q}{20} \times \left[1 - \left(\frac{1}{1+\left(\frac{R_2}{z}\right)^2}\right)^{3/2}\right]$

$\Rightarrow 2 \times 0.005q = \frac{q}{20} \times \left[1 - \left(\frac{1}{1+\left(\frac{R_2}{z}\right)^2}\right)^{3/2}\right]$

$\therefore R_2 = 0.4z$

Likewise the radius for 3rd to 9th circle can be drawn

However radius for 10th circle, we obtain $R_{10} = \infty$

Use of Newmark's Influence Chart: 2015, 2014

(i) A plan of the loaded area is drawn on a tracing paper

(ii) The scale is chosen such that the unit length in Newmark's chart = depth at point P below the surface. (P = point where load is taken)

(iii) The traced plan of the loaded area is placed over the newmark chart such that point P coincides with the center of the chart.

(iv) count the number of small unit area "n" covered by the traced plan. Fraction of all unit area should also be counted. then, $\sigma_z = I \times n \times q$

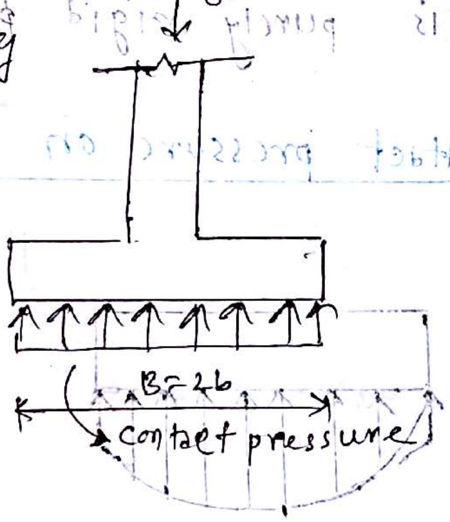
where, I = influence co-efficient = 0.005

⊞ Contact pressure distribution:

2015

\# **Contact pressure:** Upward pressure due to soil on the underside of the footing is termed as contact pressure.

So far we assumed that the footing is flexible and the contact pressure distribution is uniform. However actual footings are not flexible.



\# **Dependence of contact pressure:**

(i) Elastic properties of the footing materials.

(ii) Thickness of footing.

(iii) Relative rigity (K_r) of footing soil system.

K_r is given by,

$$K_r = \frac{1}{6} \times \frac{1-\nu_s^2}{1-\nu_f^2} \times \frac{E_f}{E_s} \times \frac{t}{b}$$

where,

ν_s = poision's ratio of soil

ν_f = poision's ratio of footing materials

E_s = Elastic modulus of soil.

E_f = Elastic modulus of footing materials

t = thickness of footing.

b = half width of footing. $= \frac{B}{2}$

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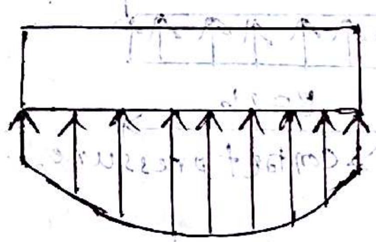
$K_r = 0$; (if $\nu_s = 1$)

It is purely flexible footing.

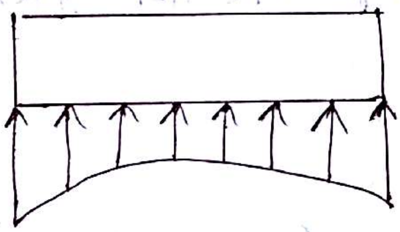
$K_r = \infty$; (if $\nu_f = 1$)

It is purely rigid footing.

Contact pressure on saturated clay: (spread footing)

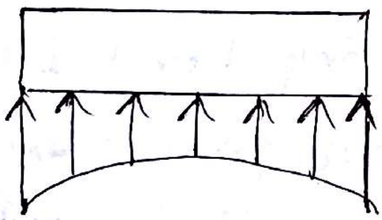


Flexible footing

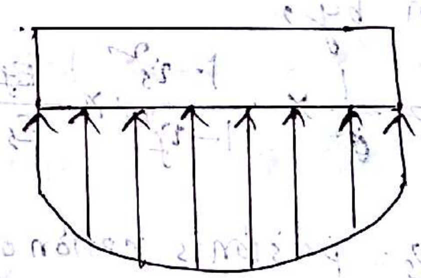


Rigid footing

Contact pressure distribution on sand: (spread footing)

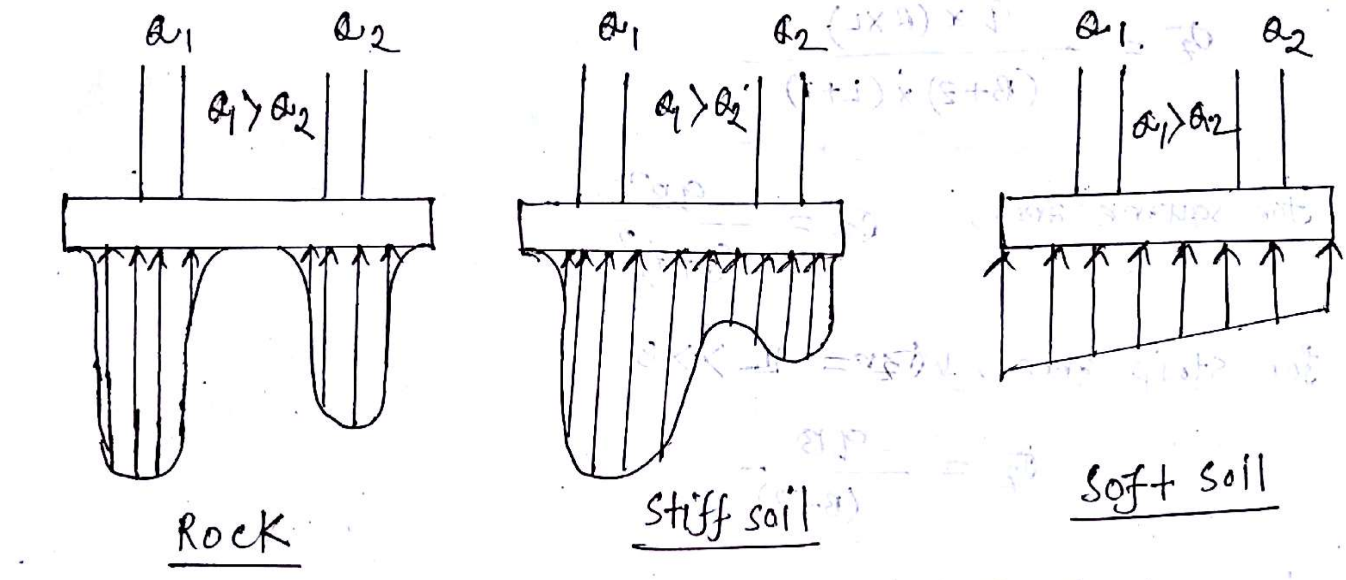


Flexible footing



Rigid footing

Contact pressure distribution on Mat foundation: 2015



(i) contact pressure distribution of mat footing is different from spread footing.

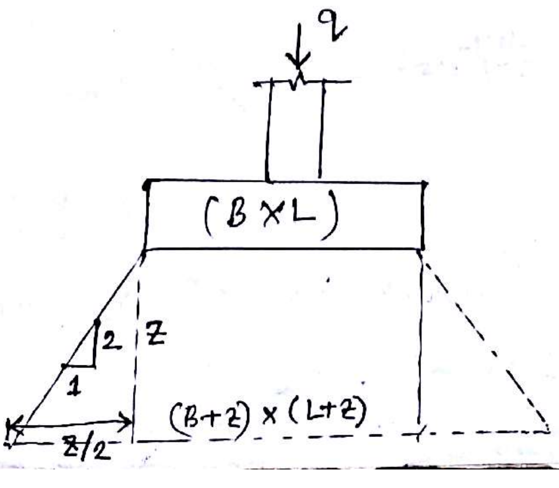
(ii) Usually a mat foundation have a much smaller thickness to width ratio and thus more flexible than spread footing.

(iii) Therefore the assumptions of rigidity is no longer valid.

(iv) Also the assumptions of linear contact pressure distribution is Erroneous.

Concept of Linear Dispersion:

Two to one load distribution Method:



The vertical stress at depth z for a footing of size (B×L) is given by,

$$\sigma_z = \frac{q \times (B \times L)}{(B+z) \times (L+z)}$$

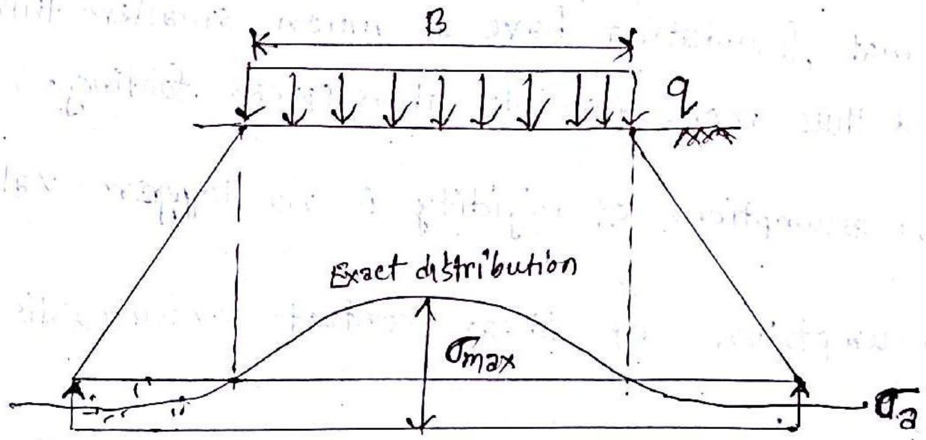
for square area, $\sigma_z = \dfrac{qB^2}{(B+z)^2}$

for strip area, where $L >> B$

$$\sigma_z = \frac{qB}{(B+z)}$$

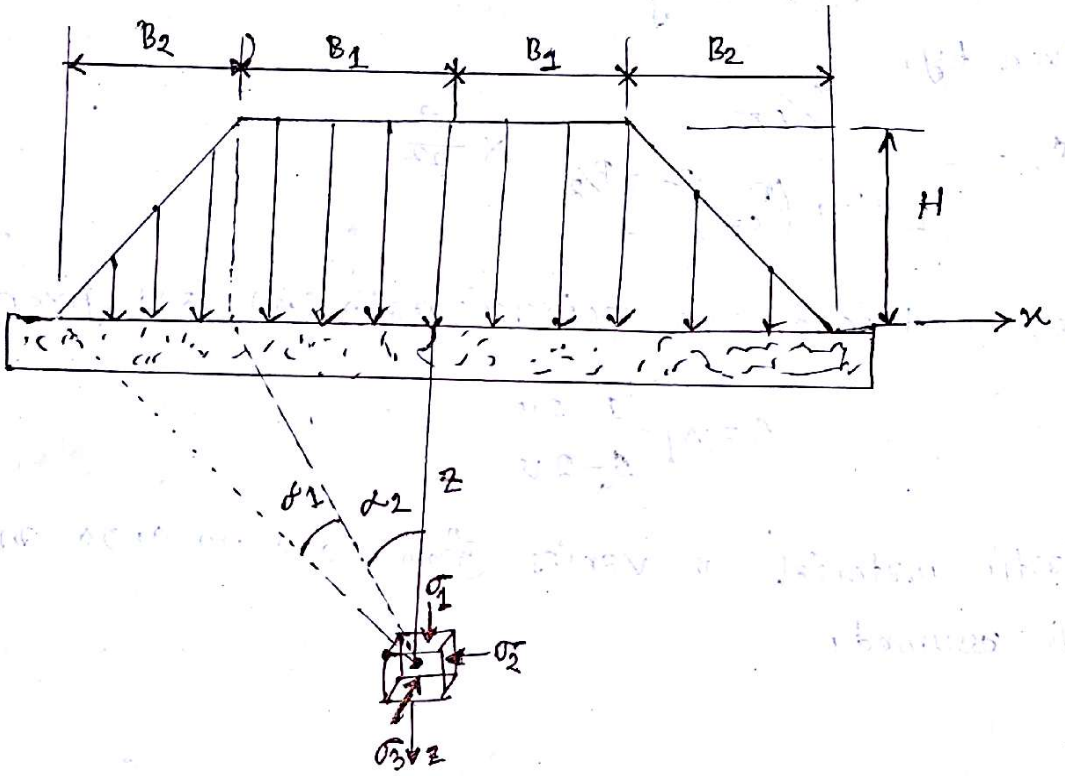
for circular area,

$$\sigma_z = \frac{qD^2}{(D+z)^2}$$



This method gives fairly accurate values of the average vertical stress if the depth z is less than 2.5 times the width of the loaded area.

vertical stress due to embankment loading



This figure shows the cross section of an embankment of height H. For this two dimensional loading condition the vertical stress increase may expressed as –

$$\Delta\sigma_z = \frac{q_0}{\pi}\left[\left(\frac{B_1+B_2}{B_2}\right)(\alpha_1+\alpha_2) - \frac{B_1}{B_2}\times(\alpha_2)\right] \quad \text{----- ①}$$

where, $q_0 = \gamma H$

γ = unit weight of the embankment

H = height of the embankment.

$$\alpha_1 \text{ (radian)} = \tan^{-1}\left(\frac{B_1+B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right)$$

$$\alpha_2 \text{ (radian)} = \tan^{-1}\left(\frac{B_1}{z}\right)$$

A simplified form of the equation ① is –

$\Delta\sigma_z = q_0 I_2$ where I_2 = a function of B_1/z and B_2/z

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Westerguard's Solution:

The vertical stress ~~may~~ at depth z below concentrated load Q is given by,

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

where, c depends on poission's ratio (υ) and given by –

$$c = \sqrt{\frac{1-2\upsilon}{2-2\upsilon}}$$

For elastic material υ varies from 0.0 to 0.50 and usually 0.3 is assumed.

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Subsurface Exploration

17, 16, 15, 14, 12, 10, 09, 07, 06

Subsoil Exploration:
The process of identifying the layers of deposits that underlie a proposed structure and their physical charactereristics is generally referred to as subsurface exploration.

Purpose of Subsurface Exploration: 17, 16, 15, 14, 12, 10, 09, 07, 06

The purpose of subsurface exploration is to obtain information that will aid the geotechnical engineer in –

1. Determining nature of soil at site and its stratification.
2. Obtaining disturbed and undisturbed soil samples for visual identification and appropriate laboratory tests.
3. determining the depth and nature of bed rock if and when encountered.
4. Selecting the type and depth of foundation suitable for a given structure.
5. Evaluating the load bearing capacity of the foundation.
6. Estimating the probable settlement of a structure.
7. Determining potential foundation problems (e.g., expansive soil, collapsible soil, sanitary landfill and so on)
8. Determining the location of the water table.
9. Predicting the lateral earth pressure for structures such as retaining walls, sheet pile bulkheads and braced cuts.
10. Establishing construction methods for changing subsoil conditions.

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Planning for Soil Exploration:

1. Compilation of the existing information regarding the structure:

This phase includes information such as,

(i) Type of structure to be constructed and its future use.

(ii) Requirment of local building codes.

2. Collection of existing information for the subsoil condition:

This type of information can be obtained from the following source:

(i) Geological survey map

(ii) Country soil survey maps

(iii) Soil manuals

(iv) Existing soil exploration reports

3. Reconnaissance of the proposed construction site:

The engineer should always make a visual inspection of the site to obtain information about -

(i) General topography of the site.

(ii) soil stratification.

(iii) Type of vegetation at the site.

(iv) Ground water level.

(v) Types of construction nearby.

4. Site investigation:

This phase consists of making several test borings at the site and collecting disturbed and undisturbed soil sample from various depth for visual observation and for laboratory test.

Approximate minimum depth of boring: 17, 16, 15, 13, 14

To determine the approximate minimum depth of boring, engineer's may use the rules established by the American Society of civil Engineers (1972):

1. Determine the net increase in the effective stress, $\Delta\sigma'$, under a foundation with depth as shown in figure:

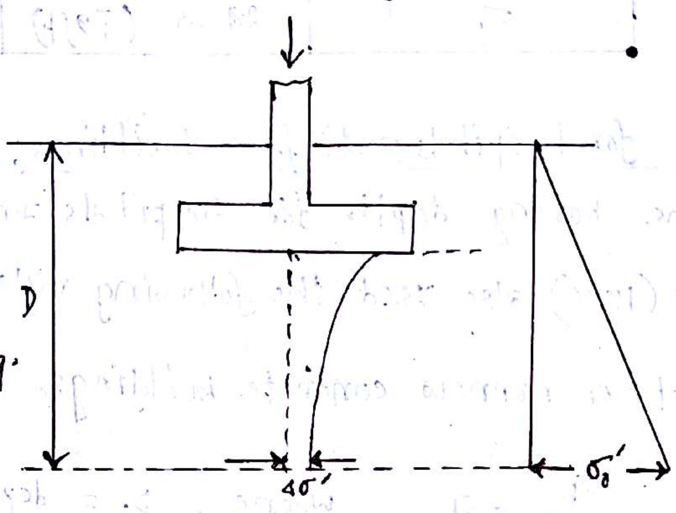

Fig. Determination of minimum depth of boring.

2. Estimate the variation of the verticle effective stress, σ_0', with depth.

3. Determine the depth $D = D_1$, at which the effective stress increase $\Delta\sigma'$ is equal to $\left(\frac{1}{10}\right)q$. where q = estimated net stress on the foundation.

4. Determine the depth $D = D_2$, at which $\Delta\sigma'/\sigma_0' = 0.05$

5. choose the smaller of two depths, D_1 and D_2, just determined as the approximate minimum depth of boring required, unless bed rock is encountered.

Depth of boring for a building:

If ASCE Rules are used, the depths of boring for a building with a depth of 30 m (100 ft) will be approximately the following:

According to Sowers and Sowers (1970):

No of Stories	Boring Depth
1	3.5 m (11ft)
2	6 m (20ft)
3	10m (33ft)
4	16m (53ft)
5	24 m (79ft)

Depth of boring for hospitals and office buildings:

To determine the boring depth for hospitals and office buildings, Sowers and Sowers (1970) also used the following rules:

1. For light steel or narrow concrete buildings,

$$\frac{D_b}{S^{0.7}} = a \qquad \text{where, } D_b = \text{depth of boring}$$

S = number of stories

$$a = \begin{cases} \approx 3 \text{ if } D_b \text{ is in meters} \\ \approx 10 \text{ if } D_b \text{ is in feet} \end{cases}$$

2. For heavy steel or wide concrete buildings,

$$\frac{D_b}{S^{0.7}} = b \qquad \text{where, } b = \begin{cases} \approx 6 \text{ if } D_b \text{ is in meters.} \\ \approx 20 \text{ if } D_b \text{ is in feet.} \end{cases}$$

When deep excavation are anticipated, the depth of boring should be at least 1.5 times the depth of excavation.

Sometimes, subsoil conditions requires that the foundation load be transmitted to bedrock. The minimum depth of core boring into the bedrock is about 3m (10ft). If bedrock is irregular or weathered, core borings may have to be deeper.

Guide rules to decide the depth of soil exploration: (B.C. Punmia)

The depth of exploration at the start of the work may be decided according to the following guide rules, which may need modification as exploration proceeds:

Type of foundation	Depth of exploration
1. Isolated spread footing or raft	one and a half times the width.
2. Adjacent footing with clear spacing less than twice the width	one and a half times the length of the footing.
3. Pile and well foundations	10 to 30 meters, or more, or at least one and a half times the width of the structure.
4. Base of retaining wall	one and a half times the base width or one and a half times the exposed hieght of face of wall, whichever is greater.
5. Dams	(i) one-half the bottom width of earthdams, twice the height from stream bed to crest for concrete dam, for dams less than 30m high. (ii) up to bed rock, or else, through all soft, ustable and permeable strata of overburden.
6. Floating basement	depth of the construction.
7. Borrow areas	convenience of excavation and thickness of avaiable material.
8. From the consideration of weathering	(i) 1.5 meters in general (ii) 3.5 meters in black cotton areas.

9. Roads, cuts and fills: (i) one meter while little cut or fill is required;

(ii) in cut sections, one meter below formation level;

(iii) in deep cuts, equal to bottom of width or equal to the height of the fill whichever is smaller.

(iv) in fill sections, two meters below the ground level, or equal to the height of the fill, which ever is greater.

Lateral extent of exploration and spacing of boring: 17, 16, 15, 13, 14

* Lateral extent of exploration and spacing of boreholes depends on horizontal variation of the strata.

* For smaller and less important buildings, even one bore hole or test pit may suffice. But for compact building sites covering an area about 0.4 hectare, one bore hole or pit in each corner and one in center may be adequate.

* There are no hard and fast rules for bore hole spacing. Spacing can be increased or decreased, depending on the condition of subsoil.

* If various soil strata are more or less uniform and predictable, fewer bore holes are needed than in non homogeneous soil strata.

* Following table gives some general guide lines for bore hole spacing:

Type of project	Spacing (m)
Multistory Building	10-30
One Story Industrial plants	20-60
Highways	250-500
Residential subdivision	250-1500
Dams and dikes	40-80

Methods of Boring:

1. Auger Boring:

* Auger boring is the simplest method of making exploratory boreholes.
* Hand auger is of two types: (i) posthole auger (ii) helical auger.
* Hand auger can not be used for advancing holes to depths exceeding 3 to 5 m. (10-16 ft)
* portable power driven helical augers (76 mm to 305 mm India) are avaiable for making deeper boreholes.
* soil samples obtained from such borings are highly disturbed.
* In some non cohesive soils or soils having low cohesion, the walls of bore holes will not stand unsupported.
* In such cases, a metal pipe is used as a casing to prevent the soil from caving in.

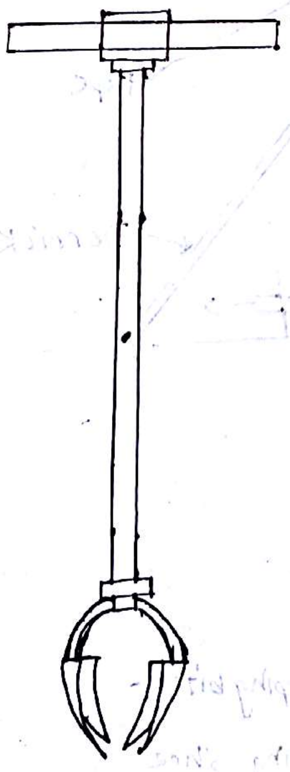

Fig. posthole auger.

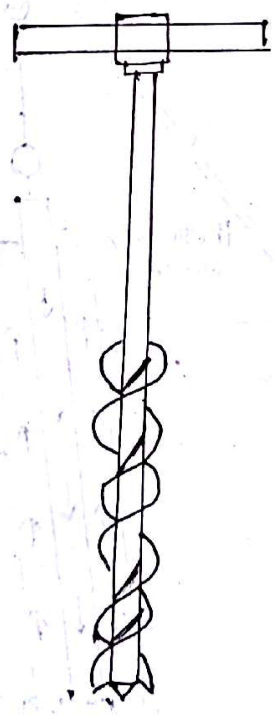

Fig: helical auger.

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2. Wash Boring:

* Wash boring is another method of advancing bore holes.
* In this method, a casing about 2 to 3 m (6 ft to 10ft.) long is driven in to the ground.
* The soil inside casing is then removed by means of a chopping bit attached to a drilling rod.
* Water is forced through the drilling rod and exits at a very high velocity through the holes at the bottom of the chopping bit.
* The water and chopped soil particles rise in the drill hole and overflow at the top of the casing through a T connection.
* The wash water is collected in a container.
* The casing can be extended with additional pieces as the bore hole progresses.

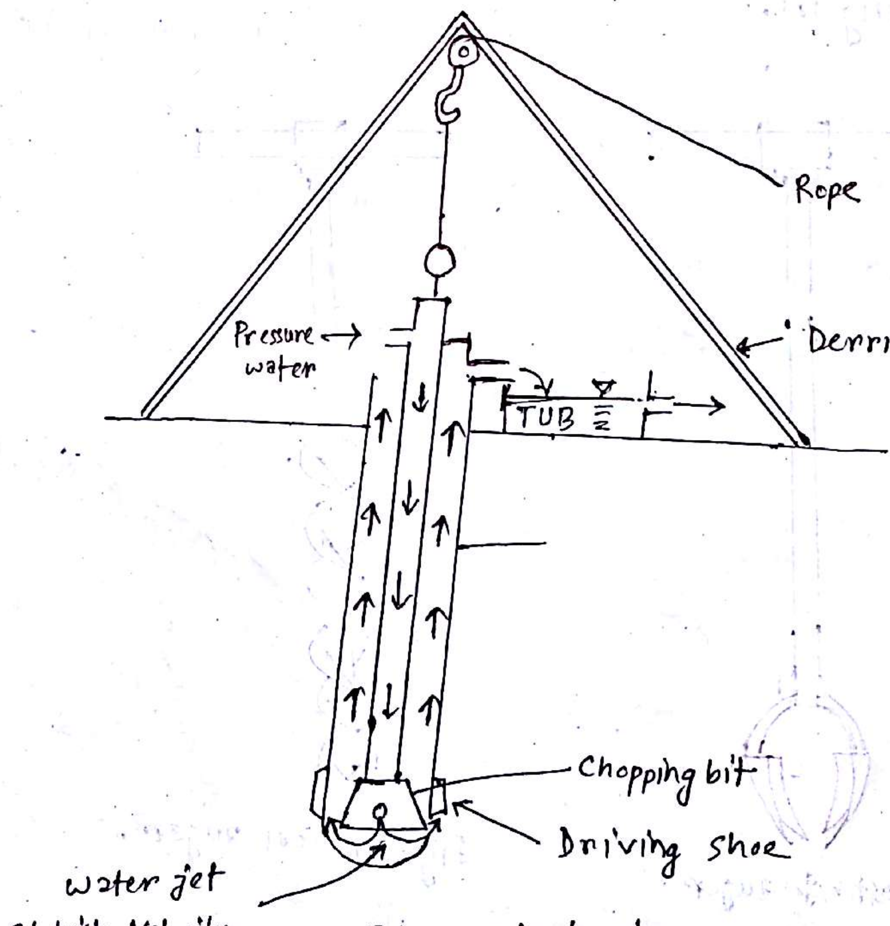


Fig. wash boring

3. Rotary Drilling:

* Rotary drilling is a procedure by which rapidly rotating drilling bits attached to the bottom of drilling rods, cut and grind the soil and advance the borehole.

* Rotary drilling can be used in sand, clay and rocks.

* water or drilling mud is forced down the drilling rods to the bits and the return flow forces the cuttings to the surface.

* Bore holes with diameter of 50 to 203 mm (2 to 8 in) can easily be made by this technique.

* Drilling mud is a slury of water and bentonite.

* Generally it is used, when the soil that is encountered is likely to cave in.

* when soil samples are needed, the drilling rod is raised and drilling bit is replaced by a sampler.

* with the environmental drilling applications, rotary drilling with air is becoming more common.

4. Percussion drilling:

* Percussion drilling is an alternative method of advancing a borehole, particularly through hard soil and rock.

* A heavy drilling bit is raised and lowered to chop the hard soil.

* The chopped soil particles are brought up by the circulation of water.

* percussion drilling may require casing.

Procedure for Sampling Soil:

Two types of soil samples can be obtained during sub-surface exploration: Disturbed and undisturbed.

* Disturbed, but representative, samples can generally be used for the following types of laboratory test:

(i) Grain size analysis.
(ii) Determination of liquid and plastic limit.
(iii) Specific gravity of soil solids.
(iv) Determination of organic content
(v) Classification of soil.

* Undisturbed soil samples must be obtained for these types of laboratory test:

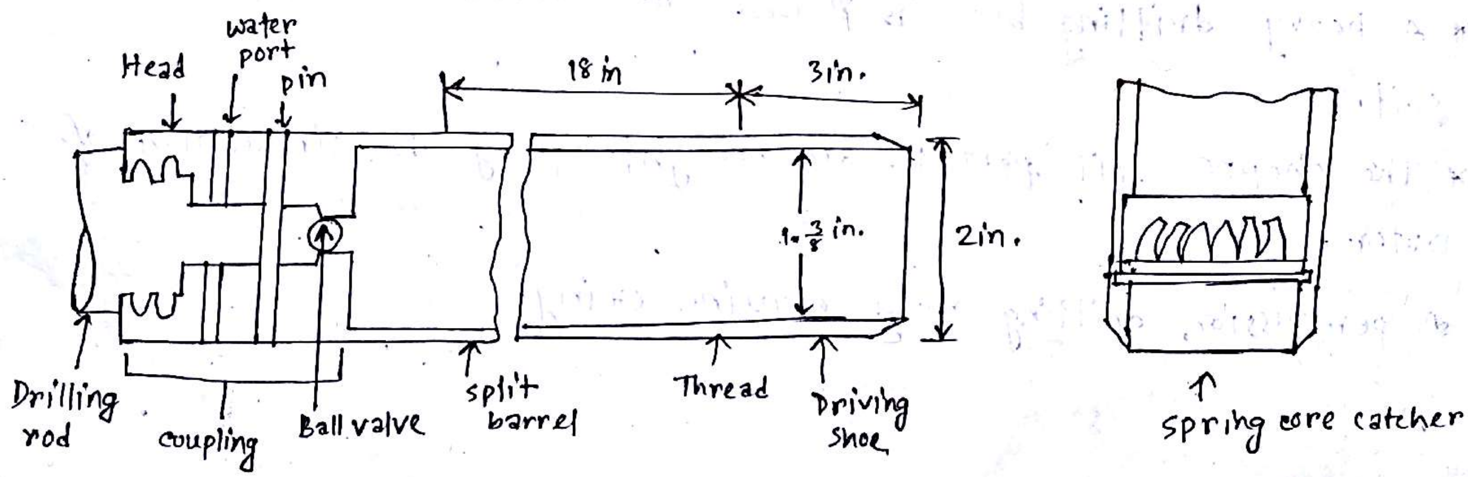
(i) consolidation test.
(ii) Hydraulic conductivity test.
(iii) Shear strength test.

1. Collection of Disturbed sample:

Split-spoon sampling:

* Split-spoon samplers can be used in the field to obtain soil samples that are generally disturbed, but still representative.

* A section of a standard split-spoon sampler is shown in figure:



* The tool consists of a steel driving shoe, a steel tube that is split longitudinally in half, and a coupling at the top.

* The coupling connects the sampler to the drill rod.
* The standard split tube has an inside diameter of $1\frac{3}{8}$ inch and outside diameter of 2 in.
* However, samplers having inside and outside diameters up to $2\frac{1}{2}$ in and 3 in. respectively, are also avaiable.

Standard penetration number ('N-value):

* When a bore hole is extended to a predetermine depth, the drill tools are removed and sampler is lowered to the bottom of the hole.

* The sampler is driven into the soil by hammer blows to the top of the drill rod.

* The standard weight of the hammer is 622.72 N (140 lb) and for each blow, the hammer drops a distance of 0.762 m (30 in.)

* The number of blows required for a spoon penetration of three 152.4 mm (6 in.) intervals are recorded.

* The number of blows required for the last two intervals are added to give the standard penetration number, N, at that depth.

* This number is generally referred to as N value.

* The sampler is then drawn, and the shoe and coupling are removed. Finally, the soil sample recovered from the tube is placed in a glass bottle and transported to the laboratory. This field test is called standard penetration test (SPT).

N-value: In split-spoon sampling, the sum of the number of blows required for spoon penetration of the last two 6 inch intervals is reffered to as standard penetration number of that depth. This number is generally reffered to as "N-value."

Importance of N-value: (v.v. Imp)

1. N-value is required to correlate the several physical parameters of soil.

2. The unconfined compression strength q_u is related to N-value.

3. Shear strength of soil is affected by the N-value.

4. In case of granular soil, N-value is highly dependent on effective overburden pressure.

5. N-value is useful guideline in subsoil exploration.

6. N-value is used in various calculations of soil mechanics problem.

7. The consistancy of clayey soil can often be estimated from N-value.

8. Net allowable bearing capacity of soil can be calculated from N-value. (Standard Penetration Number). Bearing capacity of soil is $0.15N$ ton/ft^2 (TSF) by Meyerhof.

N_{60} Value: 11

* several factors contribute to the variation of the standard penetration number 'N' at a given depth for similar soil profiles.

* Among these factors are:

  (i) SPT hammer efficiency.

  (ii) bore hole diameter.

  (iii) sampling method.

  (iv) Rod length.

* Two most common types of SPT hammer used in field are safety hammer and donut hammer.

* They are commonly dropped by a rope with two wraps around a pulley.

* The SPT hammer efficieny can be expressed as,

$$E_r\,(\%) = \frac{\text{Actual hammer energy to the sampler}}{\text{Input energy}} \times 100$$

Theoretical input energy $= Wh = (0.623\ KN \times 0.76\ m) = 0.474\ KN\text{-}m$

* In field, magnitude of E_r can vary from (30–90%)

* The standard practice now in U.S is to express the N-value to an average energy ratio of 60%.

* Thus, correcting for field procedures and on the basis of field observations,

$$N_{60} = \frac{N\eta_H\,\eta_B\,\eta_S\,\eta_R}{60}$$

where, N = measured penetration number.

η_H = hammer efficiency (%)

η_B = correction for bore hole dia.

η_S = sampler correction.

η_R = correction for rod length.

2. collection of undisturbed sample: 17, 16, 15, 13, 12, 11, 10

There are two ways to collect undisturbed soil:

(i) sampling by thin wall tube.

(ii) sampling by Piston sampler.

Sampling by thin wall tube:

* Thin wall tube is used for obtaining fairly undisturbed soil samples.
* It is made of seamless steel and commonly are referred to as shelby tubes.
* To collect samples at a given depth in a bore hole, one first must remove drilling tools.
* The sampler is attached to a drilling rod and lowered to the bottom of bore hole.
* After this, it is pushed hydraulically into the soil.
* Then it is spun to shear off the base and is pulled out.
* The sampler with soil inside is sealed and taken to laboratory for testing.
* Most commonly used thin-wall tube samplers have outside diameter of 76.2 mm (3 in.)

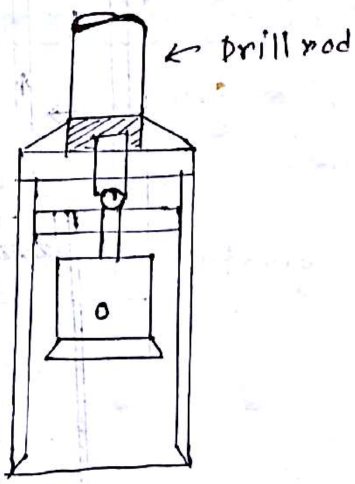


Fig. Thin walled Tube

sampling by piston sampler:

* sampling by piston sampler is particularly useful when highly undisturbed samples are required.

* It is consists of a thin wall tube with a piston.

* Initially, the piston closes the end of the tube.

* The sampler is first lowered to bottom of the bore hole.

* The thin wall tube is pushed in to the soil hydraulically, past the piston.

* After this, pressure is released through a hole in a piston rod.

* To a large extent, the presence of piston prevents distortion in the sample by not letting the soil squeeze into the sampling tube very fast and by not admitting excess soil.

* consequently, samples obtained in this manner are less disturbed than those obtained by shelby tubes.

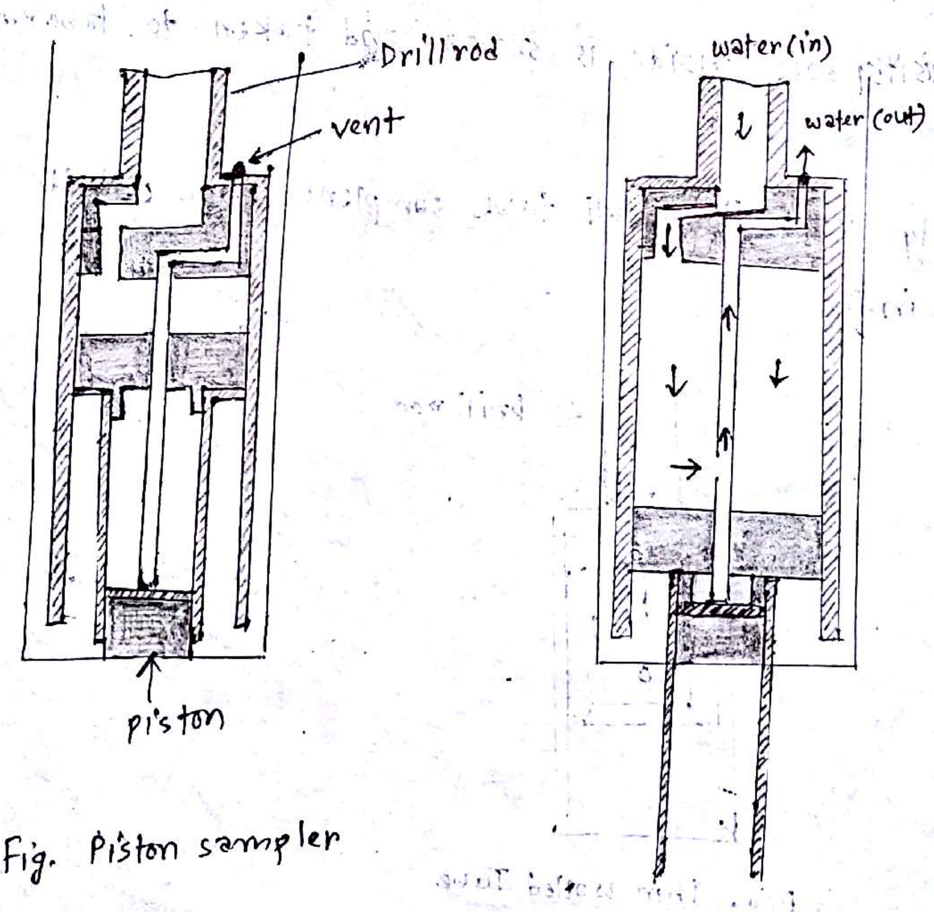


Fig. Piston sampler

Area Ratio: The degree of disturbance for a soil sample that is collected by various methods can be expressed by a term called area ratio, which is given by,

$$A_R(\%) = \frac{D_o^2 - D_i^2}{D_i^2} \times 100$$

where, A_R = area ratio (ratio of disturbed area to total area of soil)

D_o = outside diameter of the sampling tube

D_i = inside diameter of the sampling tube

A soil sample generally is considered to be undisturbed, when the area ratio is less than or equal to 10%.

* For the standard split spoon sampler, $D_i = 1.38$ in. and $D_o = 2$ in.

$$\therefore A_R(\%) = \frac{(2)^2 - (1.38)^2}{(1.38)^2} \times 100 = 110\%$$

Hence, these samples are highly disturbed.

* For the shelby tube sampler, $D_i = 1.875$ in. and $D_o = 2$ in.

$$\therefore A_R(\%) = \frac{(2)^2 - (1.875)^2}{(1.875)^2} \times 100 = 13.8\%$$

Hence, these samples are slightly disturbed or fairly undisturbed.

co-relation for Standard penetration Test in cohesive soil:

* The consistency of clay soils can be estimated from the standard penetration number N_{60}. In order to achieve that, Szechy and Vargi (1978) calculated the consistency index (CI) as,

$$CI = \frac{LL - W}{LL - PL}$$

where, W = Natural moisture content

LL = Liquid Limit

PL = Plastic Limit

The approximate correlation between CI, N_{60} and the unconfined Compression strength, q_u is given in the following Table:

Standard Penetration Number, N_{60}	Consistency	CI	Unconfined compression strength, q_u (kN/m^2)	(lb/ft^2)
<2	Very soft	<0.5	<25	900
2–8	Soft to Medium	0.5–0.75	25–80	500–1700
8–15	Stiff	0.75–1.0	80–150	1700–3100
15–30	Very stiff	1.0–1.5	150–400	3100–8400
>30	Hard	>1.5	>400	8400

- Hara, et al. (1971) also suggested the following correlation between N_{60} and the unconfined compression strength of clay (c_u),

$$\frac{c_u}{p_a} = 0.29 N_{60}^{0.72}$$

where, p_a = atmospheric pressure ($\approx 100\ kN/m^2$; $\approx 2000\ lb/ft^2$)

- The over consolidation ratio, OCR of a natural clay deposit can also be correlated with the standard penetration number. On the basis of regression analysis of 110 data points, Mayne and Kemper (1988) obtained the relationship:

$$OCR = 0.193 \left(\frac{N_{60}}{\sigma_0'}\right)^{0.689}$$

where,

σ_0' = effective verticle stress in MN/m^2

Correction for N_{60} in Granular soil:

Standard penetration number, N_{60} obtained from field needs to be corrected for following cases:

(i) Correction for overburden pressure.

(ii) Correction for submergence.

(i) Correction for overburden pressure:

In granular soils, the value of N is affected by the effective over burden pressure, σ_0'. For that reason, the value of N_{60} obtained from field exploration under different effective overburden pressure should be changed to correspond to a standard value of σ_0'. That is,

$$(N_1)_{60} = C_N N_{60}$$

where,
$(N_1)_{60}$ = value of N_{60} corrected to a standard value of σ_0' [100 kN/m²; (2000 lb/ft²)]

C_N = correction factor.

N_{60} = value of N obtained from field exploration.

A number of emperical relationships were proposed for C_N.

Liao and Whitman's relationship (1986):

$$C_N = \left[\frac{1}{\left(\frac{\sigma_0'}{P_a}\right)}\right]^{0.5} \quad \text{(SI Units)}$$

$$C_N = \sqrt{\frac{1}{\sigma_0'}} \quad \text{(English Units)}$$

Skempton's relationship: (1986)

For SI Units,

$$C_N = \frac{2}{1+\left(\frac{\sigma_o'}{p_a}\right)}$$ [Normally consolidated fine sand]

$$C_N = \frac{3}{2+\left(\frac{\sigma_o'}{p_a}\right)}$$ [Normally consolidated coarse sand]

$$C_N = \frac{1.7}{0.7+\left(\frac{\sigma_o'}{p_a}\right)}$$ [Over consolidated sand]

For English Units,

$$C_N = \frac{2}{1+\sigma_o'}$$ [Normally consolidated fine sand]

$$C_N = \frac{3}{2+\sigma_o'}$$ [Normally consolidated coarse sand]

$$C_N = \frac{1.7}{0.7+\sigma_o'}$$ [Over consolidated sand]

(ii) Correction for submergence:

In very fine, silty, saturated sand, an apparent increase in resistance occurs.

Terzaghi and Peck have recommended use of an equivalent penetration resistance $(N_1)_{60}$ in place of actually observed value of N_{60}. When N_{60} is greater than 15,

$$(N_1)_{60} = 15 + \frac{1}{2}(N_{60} - 15)$$

correlation between N_{60} and Relative density: (Granular soil)

Kulhawy and Mayne (1990) modified an emperical relationship for relative density that was given by Marcuson and Bieganousky (1977), which can be expressed as,

$$D_r\,(\%) = 12.2 + 0.75\left[222 N_{60} + 2311 - 711\,(OCR) - 779\left(\frac{\sigma_o'}{p_a}\right) - 50\,C_u^{2}\right]^{0.5}$$

where, C_u = Uniformity co-efficient of sand.

Meyerhof (1957) developed a correlation between D_r and N_{60} as:

$$N_{60} = \left[17 + 24\left(\frac{\sigma_o'}{p_a}\right)\right] D_r^{2}$$

$$\Rightarrow D_r = \left[\frac{N_{60}}{\left[17 + 24 \times \left(\frac{\sigma_o'}{p_a}\right)\right]}\right]^{0.5}$$

Cubrinovski and Ishihara (1999) also proposed a correlation between N_{60} and D_r that can be expressed as,

$$D_r\,(\%) = \left[\frac{N_{60}\left(0.23 + \frac{0.06}{D_{50}}\right)^{1.7}}{9} \times \left(\frac{1}{\frac{\sigma_o}{p_a}}\right)\right]^{0.5} \times 100$$

where, D_{50} = sieve size through which 50% of the soil will pass (mm)

An approximate relationship between the corrected standard penetration number and the relative density of sand is given in the following table:

standard Penetration number, $(N_1)_{60}$	Approximate relative density, D_r (%)
0–5	0–5
5–10	5–30
10–30	30–60
30–50	60–95

Kulhawy and Hayne (1990) correlated the corrected standard penetration member and relative density of sand penetration number and the relative density of sand in the form

$$Dr(\%) = \left[\frac{(N_1)_{60}}{C_p C_A C_{OCR}}\right]^{0.5} \times 100$$

where, C_p = grain size correlation factor $= 60 + 25 \log D_{50}$

C_A = Correlation factor for aging $= 1.2 + 0.05 \log\left(\frac{t}{100}\right)$

C_{OCR} = correlation factor for overconsolidation $= OCR^{0.18}$

D_{50} = diameter through which 50% soil will pass through (mm)

t = age of soil since deposition (years)

OCR = over consolidation ratio.

Correlation between angle of friction and Standard Penetration Number:

1. Peck, Hanson, and Thornburn (1974) give a correlation between N_{60} and ϕ' in a graphical form, which can be approximated as (Wolff, 1989)

$$\phi'(\text{deg}) = 27.1 + 0.3 N_{60} - 0.00054 [N_{60}]^2$$

2. [Next page]

Sources of error in Standard penetration test:

Although approximate, with correct interpretation standard penetration test provides a good evaluation of soil properties. Primary sources of error in standard penetration tests are:

(i) Inadequate cleaning of bore hole.

(ii) careless measurement of blow count.

(iii) Eccentric hammer strikes and drill rod.

(iv) Inadequate maintenance of water head in bore hole.

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co-relation between angle of friction and standard penetration number:

2. Schmertmann (1975) provided the correction between N_{60}, σ_o' and ϕ'.
Mathematically, the correlation can be approximated as (Kulhawy and Mayne, 1990)

$$\phi' = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma_o'}{P_a}\right)}\right]^{0.34}$$

3. Hatanaka and uchida (1996) provided a simple correlation between ϕ' and $(N_1)_{60}$ that can be expressed as

$$\phi' = \sqrt{20(N_1)_{60}} + 20$$

The following qualifications should be noted when standard penetration resistance values are used in the proceding correlations to estimate soil parameters:

1. The equations are approximate
2. Because the soil is not homogeneous, the values of N_{60} obtained from a given bore hole vary widly.
3. In soil deposits that contain large boulders and gravels, standard penetrations numbers may be erratic and unreliable.

Other in SITU Test:

Vane Shear Test:

* The vane shear test may be used during the drilling operation to determine the in situ undrained shear strength (cu) of clay soils - particularly soft clays.
* The vane shear apparatus consist of four blades on the end of a rod.

* The height, H of the vane is twice the diameter, D.

* The vane can be either rectangular or tapered.

* The vanes of the apparatus are pushed into the soil at the bottom of the bore hole without disturbing the soil appreciably.

* Torque is applied at the top of the rod to rotate the vanes at a standard rate of $0.1°/sec$.

* This rotation will induce failure in a soil of cylindrical shape sorrounding the vanes.

* The maximum torque, T applied to cause failure is measured.

* Note that, $T = f(c_u, H \text{ and } D)$

or, $c_u = \dfrac{T}{K}$

where, T is in N.m

c_u is in KN/m^2

K = a constant with a magnitude depending on the dimension and shape of the vane.

The constant,

<u>In SI Units:</u> $K = \left(\dfrac{\pi}{10^6}\right) \times \left(\dfrac{D^2 H}{2}\right) \times \left(1 + \dfrac{D}{3H}\right)$ where,

D = diameter of vane (cm)

H = measured height of vane (cm)

if $\dfrac{H}{D} = 2$ then,

$K = 366 \times 10^{-8} D^3$

<u>In English Unit:</u> $K = \left(\dfrac{\pi}{1728}\right) \times \left(\dfrac{D^2 H}{2}\right) \times \left(1 + \dfrac{D}{3H}\right)$ where,

D & H is in inch.

c_u is in lb/ft^2.

T is in lb-ft.

if $\dfrac{H}{D} = 2$ then,

$K = 0.0021 D^3$

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advantages:

(i) Field vane shear test are moderately rapid and economical

(ii) Vane shear test are used extensively used in field soil-exploration programs.

(iii) This test gives good results in soft and medium stiff clays.

(iv) It gives excellent results in determining the properties of sensitive clays.

sources of error:

(i) sources of error in the field vane shear test are poor calibration of torque measurement and damaged vanes.

(ii) other errors may be induced if the rate of rotation of the vane is not properly controlled.

* For actual design purposes, The values of $[c_{u(VST)}]$ are too high and it is recommended that they be corrected according to the equation,

$c_{u\,(corrected)} = \lambda\, c_{u\,(VST)}$ where, λ = correction factor

* several corelations for 'λ':

(i) Bjerrum (1972): $\lambda = 1.7 - 0.54 \log [PI\,(\%)]$ (most commoly used)

(ii) Morris and Williams (1994):

$$\lambda = 1.18\, e^{-0.08\,(PI)} + 0.57 \quad (\text{for } PI > 5)$$

$$\lambda = 7.01\, e^{-0.08\,(LL)} + 0.57 \quad (\text{where LL is in } \%)$$

* The field vane shear strength can be correlated with the pre consolidation pressure and over consolidation of the clay.

* Using 343 data points, Mayne and Mitchell (1988) derived the following emperical relationship for estimating the preconsolidation pressure of a natural clay deposit:

$$\sigma_c' = 7.04\,[C_u(\text{field})]^{0.83}$$

where, σ_c' = Pre consolidation pressure (KN/m^2)

$C_u(\text{field})$ = field vane shearstrength (KN/m^2)

* The over consolidation ratio OCR, also can be correlated to $C_u(\text{field})$ according to the equation,

$$OCR = \beta \frac{C_u(\text{field})}{\sigma_0'}$$ where, σ_0' = effective over-burden pressure

The magnitudes of β developed by various investigators are given below:

— Mayne and Mitchell (1988): $\beta = 22\,[PI\,(\%)]^{-0.48}$

— Hansbo (1957): $\beta = \dfrac{222}{\omega\,(\%)}$

— Larsson (1980): $\beta = \dfrac{1}{0.08 + 0.0055\,(PI)}$

Cone penetration Test: 17

* The cone penetration test (CPT), originally known as the Dutch cone penetration test, is a versatile sounding method that can be used to determine the materials in a soil profile and estimate their engineering properties.

* This test is also called the static penetration test and no boreholes are necessary to perform it.

* In the original version, a 60° cone with a base area of $10\ cm^2$ ($1.55\ in^2$) was pushed in to the ground at a steady rate of about 20 mm/sec (≈ 0.8 in./sec) and the resistance to penetration (called the point resistance) was measured.

* The cone penetrometers in use at present measure:
  (a) the cone resistance (q_c) to penetration developed by the cone, which is equal to the vertical force applied to the cone, divided by its horizontally projected area, and

  (b) the frictional resistance (f_c), which is the resistance measured by a sleeve located above the cone with the local soil surrounding it. The frictional resistance is equal to the vertical force applied to the sleeve, divided by its surface area, actually the sum of friction and adhesion.

* Generally, two types of penetrometers are used to measure q_c and f_c.
  (i) Mechanical friction cone penetrometer.
  (ii) Electric friction cone penetrometer.

Pressure meter Test : (PMT)

* The pressure meter test is an in situ test conducted in a bore hole.

* It is originally develoned by Menard (1965) to measure the strength and deformability of soil.

* It has also been adopted by ASTM as Test designation 4719.

* The menard type PMT consist of a probe with three cells.

* The top and bottom ones are gaurd cells and the middle one is measuring cell, as shown schematicall in Figure:

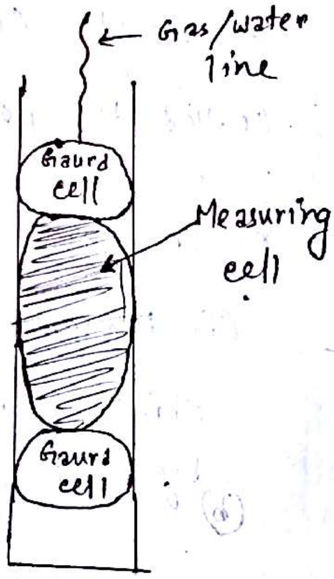

Fig. Pressuremeter

* The test is conducted in a pre bored hole with a diameter that is between 1.03 and 1.2 times the nominal diameter of the probe.

* The probe that is most commonly used has a diameter of 58 mm and a length of 420 mm.

* The probe cells can be expanded by either liquid or gas.

* The gaurd cells are expanded to reduce the end-condition effect on the measuring cell, which has a volume (V_o) of 535 cm^3,

* In order to conduct a test, the measuring cell volume, V_o, is measured and the probe is inserted into the borehole.

* The pressure is applied in increaments and the new volume of cell is measured.

* The process is continued until the soil fails or until the pressure limit of the device is reached.

* The soil is considered to have failed when the total volume of the expanded cavity (v) is about twice the volume of the original cavity.

* After the completion of the test, the probe is deflated and advanced for testing at another depth.

Boring Log:

The detailed information gathered from each bore hole is presented in graphical form which is called boring log.

prepation of boring log: As a borehole is advanced downward, the driller generally should record the following information in a standard log:

1. Name and adress of the drilling company.
2. Driller's name.
3. Job description and number.
4. Number, type and location of boring.
5. Date of boring.
6. Sub surface stratification.
7. Elevation of water table
8. Standard penetration resistance and the depth of SPT
9. Number, type and depth of soil sample collected
10. Type of core barrel used (In case of rock coring)

This information should never be left to memory, because doing so often results in erroneous boring logs.

After completion of the necessary laboratory test, the geotech-nical engineer prepares a finished log that includes notes from the driller's field log and the results of test conducted in Laboratory.

Subsoil Exploration Report: 17, 15, 14, 13

At the end of all soil exploration programs, the soil and rock specimens collected in the field are subject to visual and appropriate laboratory testing. After all the required information has been compiled, a soil exploration report is prepared for future construction work. Each report should include the following items:

(i) A description of scope of investigation.

(ii) A description of the proposed structure.

(iii) A description of the location of site.

(iv) A description of the geological setting of the site.

(v) Details of the field exploration.

(vi) A general description of subsoil conditions.

(vii) A description of water table condition.

(viii) conclutions and limitations of the investigation.

The following graphical presentations should be attached to the report:

(1.) A site location map (2) A plan view of the location of boring

(3) Boring logs (4.) Laboratory test results etc.

What is N-value? 17, 16, 15, 14, 12, 11, 10

In split spoon sampling, the number of blows required for a spoon penetration of the last two intervals (12 inch in total) is generally referred to as N-value.

Significance of N-value: 12, 11, 10

1. The consistancy of clay soil can be estimated from the N-Value.

N-Value	consistancy
<2	very soft
2-8	soft to Medium
8-15	stiff
15-30	very stiff
>30	Hard

2. The unconfined compression strength of clay is correlated with N value.

$$\frac{c_u}{P_a} = 0.29 N_{60}^{0.72}$$ where, $P_a = 100 \, kN/m^2$ or $2000 \, lb/ft^2$

3. The over consolidation ratio of a natural clay deposit can also be correlated with the N-value.

$$OCR = 0.193 \left(\frac{N_{60}}{\sigma_0'}\right)^{0.689}$$

4. Net allowable bearing capacity and peak angle of friction can be calculated from the N-value.

5. The relative density of sand is correlated with the N-value.

6. The modulus of elasticity can be estimated from N value.

$$\frac{E_s}{P_a} = \alpha N_{60}$$

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Why N value need to be corrected: 17, 16, 15, 14, 13, 11

1. In granular soils, The value of N is affected by the effective over burden pressure, σ_o'. For that reason, the value of N_{60} obtained from field exploration under different effective over burden pressures should be changed to correspond to a standard value of σ_o'. That is.

$$(N_1)_{60} = C_N N_{60}$$

2. In very fine, silty, saturated sand, an apparent increase in resistance occurs.

Terzaghi and Peck have recommended.

When $N_{60} > 15$,

$$(N_1)_{60} = 15 + \frac{1}{2}(N_{60} - 15)$$

Factors Affecting the N-Value:

1. Homoginity of soil.
2. Effective over burden pressure.
3. Type of soil.
4. In adequate cleaning of bore hole.
5. Drilling method.
6. If Driven hammer does not strike the drive cap concentrically. etc..

17, 16, 15, 13, 12, 11, 10

What do you understand by disturbed and undisturbed sample?

Disturbed sample:

When there any parameter of soil such as its specific gravity, grain size, texture, density or the stress conditions are disturbed due to some reasons, then the soil sample is called Disturbed sample. / Disturbed sample is that in which natural structure and properties of soil get fully or partially modified and destroyed.

Undisturbed Sample:

If there any parameter of soil such as its specific gravity, grain size, texture, density or the stress conditions are not disturbed due to any reason, then the sample is called Undisturbed sample. /

Undisturbed sample is that in which natural structure and properties of soil remain preserved.

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Bearing Capacity of Shallow Foundation

Foundation:
The lowest part of a structure generally is referred to as foundation.
Its function is to transfer load of the structure to the soil on which it is resting.

Footing:
A foundation unit that constructed in brick work masonry or concrete under base of a wall or a column for purpose of distributing load over a large area of soil on which it resist on is called footing.

* In simple words foundation means legs and ~~footing~~ means foot of legs.

Types of foundation:
Depending on structure and soil encountered, two types of foundations are used: (i) Shallow foundation (ii) Deep foundation.

- Foundation
  - Shallow foundation
    - Spread Footing / Isolated footing
      - Rectangular footing
      - Square footing
      - Circular footing
      - Strap footing
      - Combined footing.
    - Mat foundation
  - Deep foundation
    - Pile foundation
    - Drilled shaft
    - Well foundation or caisson

Spread footing:

A spread footing is simply an enlargement of a load-bearing wall or column that makes it possible to spread load of structure over a large area of soil.

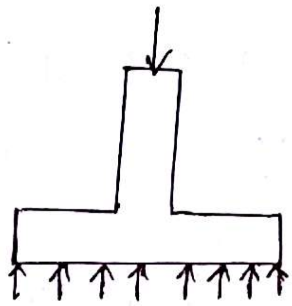

Fig. Spread footing

Mat foundation:

In soil with low load bearing capacity, size of spread footings required is impractically large.

In that case, it is more economical to construct entire structure over a concrete pad called a mat foundation.

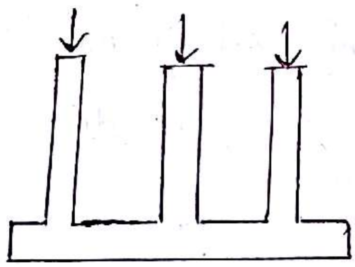

Fig. Mat foundation

Pile foundation:

Piles are structure members made of timber, concrete or steel that transmit load of superstructure to lower layers of soil.

According to how they transmit their load into subsoil, pile can be divided into two types:

① Friction piles: superstructure load is resisted by shear stress generated along structure of pile.

(ii) End-bearing piles: load carried by pile is transmitted at its tip to a film stratum.

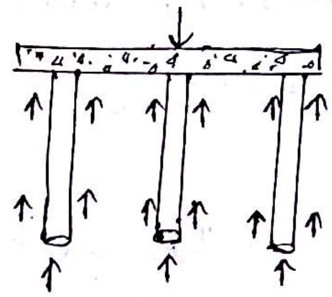

Fig. Pile foundation.

Drilled Shaft foundation:

* In this case; a shaft is drilled into sub-soil and then filled with concrete.
* A metal casing may be used while shaft is being drilled.
* casing may be left in place or may be withdrawn during placing of concrete.
* Generally diameter of a drilled shaft is much larger than that of a pile.
* Distribution between piles and drilled shaft becomes hazy at an approximate diameter of 1m (3ft)

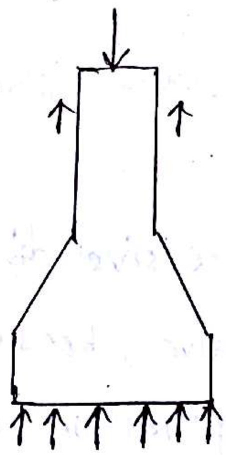

Fig. Drilled shaft foundation.

Well Foundation / Caisson:

* A large water tight retaining structure is used to work on foundations of a bridge pier or for construction of a concrete dam.

* These are constructed such that water can be pumped out, keeping the working environment dry and in which construction work may be carried out under water.

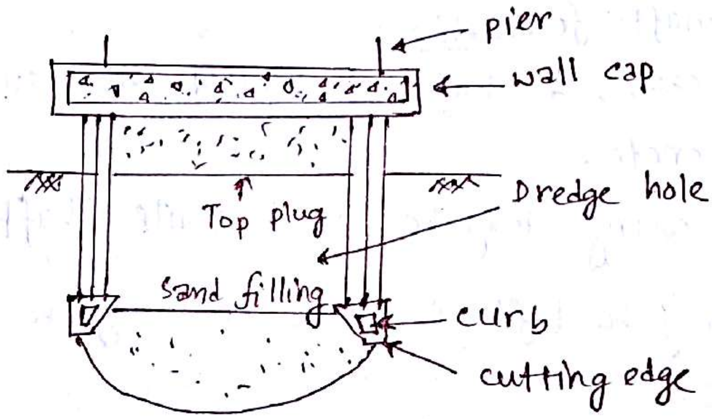

Fig. well foundation.

Characterestics of shallow foundation:

To perform satisfactory, shallow foundations must have two main characterestics:

1. They have to safe against overall shear failure in the soil that supports them.

2. They can not undergo excessive displacement or settlement (The term 'exessive' is relative, because the degree of settlement allowed for a structure depends on several conditions)

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Ultimate bearing capacity of soil:

Ultimate bearing capacity is the theoretical maximum pressure which can be supported by soil without failure.

or, The maximum pressure of the foundation at which shear failure in soil occurs is called ultimate bearing capacity of that soil.

Modes of Failure of soil: 17, 16, 15, 14, 11, 10

There are three types of shear failure. They are :-

(i) General shear failure (ii) Local shear failure (iii) Punching shear failure

General Shear failure:

* This type of shear failure occurs in dense sand, stiff soil and over-consolidated clay.
* If the uniformly distributed load (q) is increased, the settlement of the footing gradually increases. When the value of q = Ultimate bearing capacity of the soil (q_u) failure occurs, the footing undergoes a very large settlement without any increase of q.

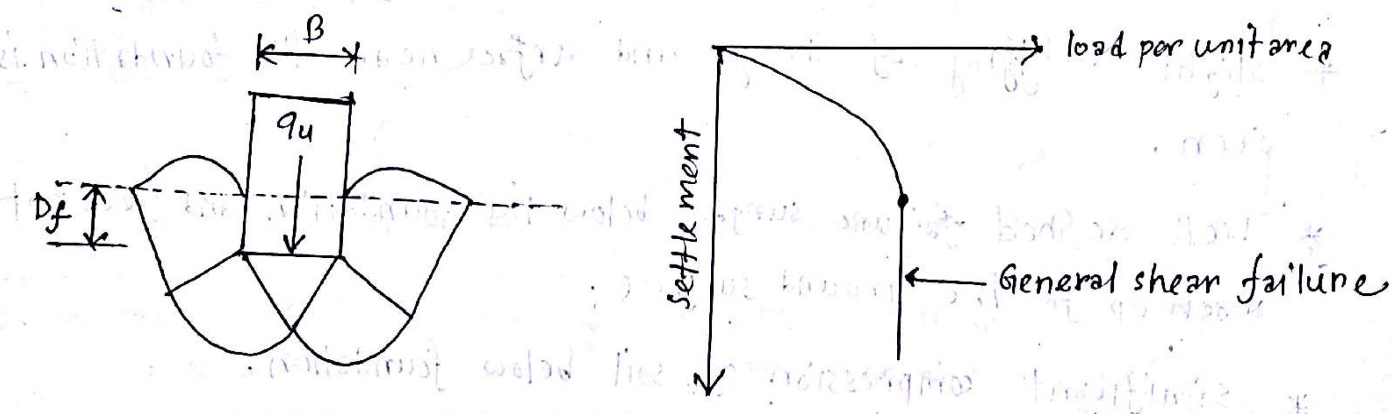
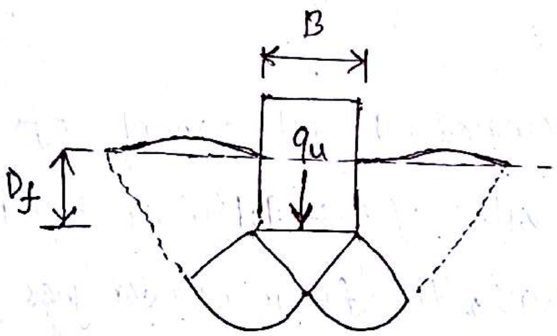


Fig. General shear failure.

* Failure surface is well defined.
* Failure is catastrophic and sudden, the foundation tilts
* The soil on one or both sides of the foundation bulges, and the slip surface extends to the ground surface.

Local shear failure:

* This type of shear failure occurs in a loose to medium dense sand.
* Beyond a certain value of q is equal to the ultimate bearing capacity of the soil (q_u'), the load-settlement relation becomes a steep, inclined straight line.



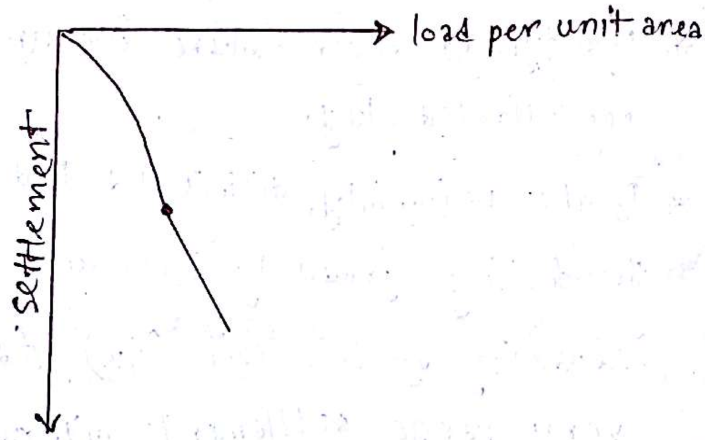


Fig. Local shear failure.

* Slight bulging of the ground surface near the foundation is seen.
* Well defined failure surface below the foundation and does not reach up to the ground surface.
* Significant compression of soil below foundation.
* The load settlement curve does not indicate the ultimate load clearly.

punching shear failure:

* This type of shear failure occurs in loose soil.
* shear plane is not properly defined.

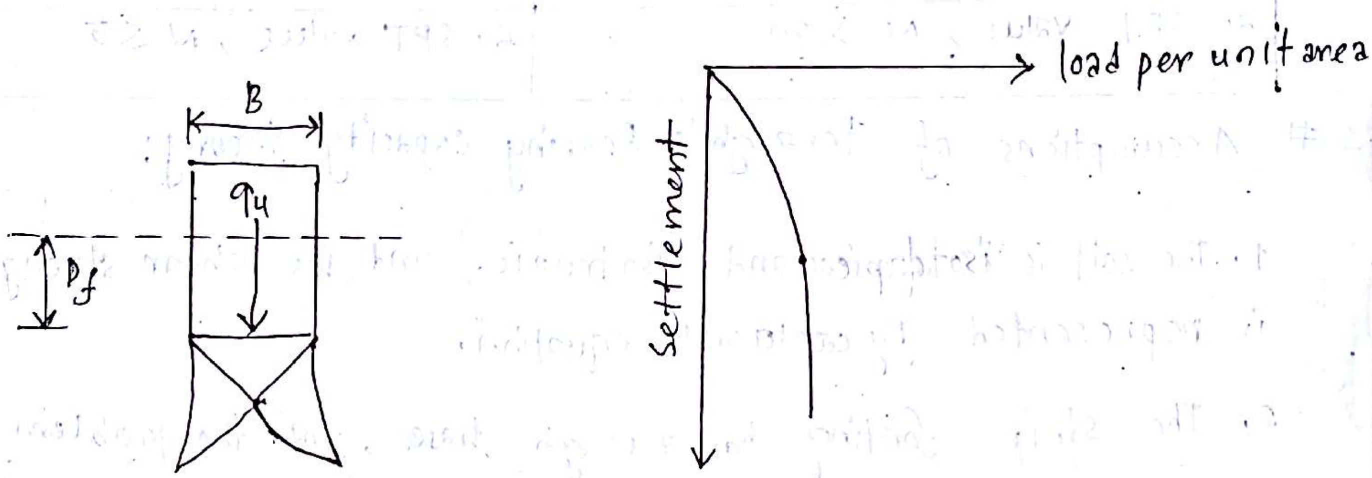


Fig. Punching shear failure

* Soil beyond the loaded area is very little affected.
* There is a significant penetration of foundation and there is vertical shear beneath the edges of foundation.
* Ultimate load can not be easily located from the load settlement curve.

Difference between General shear failure and local shear failure.

General shear failure	Local shear failure.
1. It occurs in dense soil	1. It occurs in medium dense soil.
2. Penetration of foundation is slight.	2. Penetration of foundation is more than general shear failure.
3. Angle of friction, $\phi > 36$	3. Angle of friction, $\phi < 29$
4. Relative density, $D_r > 70\%$	4. Relative density, $30\% < D_r < 70\%$

General shear failure	Local Shear Failure
5. bulging of the ground surface is significant near the foundation	5. Bulging of ground surface is slight
6. SPT Value, $N \geq 30$	6. SPT value, $N \leq 5$

Assumptions of Terzaghi's Bearing capacity theory:

1. The soil is homogenious and isotropic and its shear strength is represented by coulomb's equation.

2. The strip footing has a rough base, and the problem is essentially two dimensional.

3. The elastic zone has straight boundaries inclined at $\psi = \phi$ to the horizontal, and the plastic zones fully develop.

4. Passive pressure consists of three components which can be calculated seperately and added, Although the critical surface for these components are not identical.

5. Failure zones do not extend above the horizontal plane through the base of the footing.

17, 14

Limitation of Terzaghi's Bearing capacity theory: (B.C. Punmia)

1. As the soil compresses, ϕ changes; slight downward movement of footing may not develop fully the plastic zones.

2. Error due to passive pressure assumption is small and on the safe side.

3. Error due to failure zone assumption increases with depth of foundation and hence the theory is suitable for shallow foundation only.

Terzaghi's bearing capacity analysis:

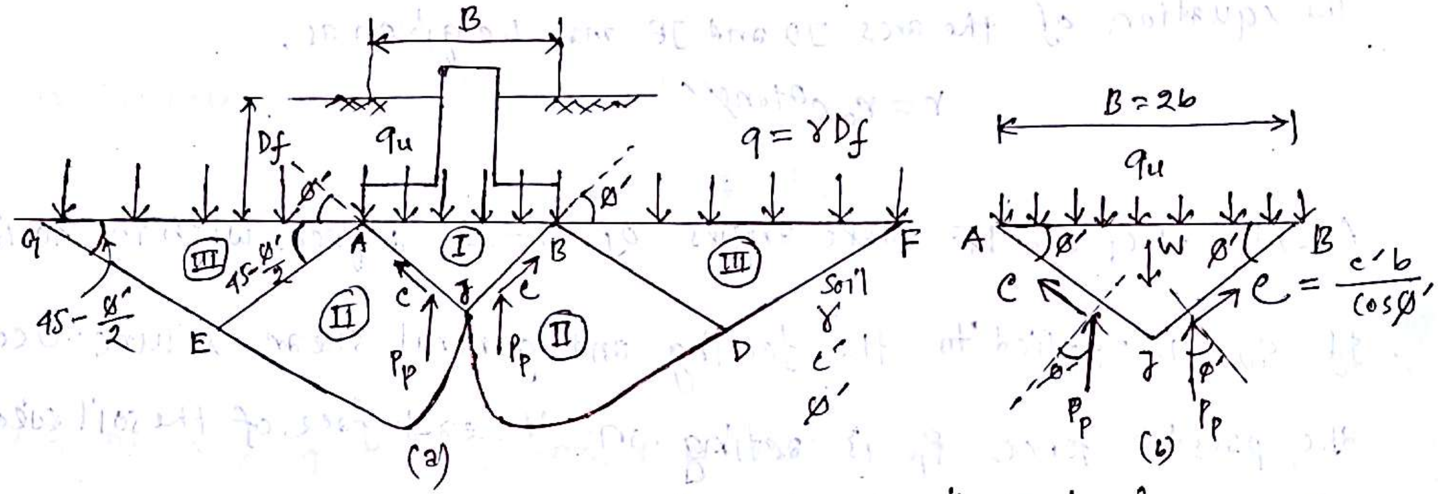


Fig. Terzaghi's bearing capacity analysis.

Terzaghi suggested that for a continuous or strip foundation, failure surface in soil at ultimate load may be assumed to be similar to that shown in figure (a)

The effect of soil above the bottom of the foundation may assume to be replaced by an equivalent surcharge, $q = \gamma D_f$

The failure zone under the foundation can be separated into three parts:

(i) The soil wedge ABJ (ZONE-I) is an elastic zone. Both AJ and BJ make an angle ϕ' with the horizontal.

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(ii) AJE and BJD (ZONE-II) are the radial shear zones.

(iii) AEG and BDF (ZONE-III) are the Rankine passive zones.

The rupture lines JD and JE are arcs of a logarithmic spiral and DF and EG are straight lines.

The equation of the arcs JD and JE may be given as,

$$r = r_0 e^{\theta \tan\phi'}$$

AE, BD, EG and DF make angles of $45 - \frac{\phi'}{2}$ degrees with the horizontal.

If q_u is applied to the footing and general shear failure occurs, the passive force P_p is acting on the each face of the soil wedge ABJ.

AJ and BJ are pushing the soil wedges AJEG and BJDF respectively to cause passive failure. P_p should be inclined at angle ϕ'.

Now let us consider the free body diagram of the wedge ABJ:

For equilibrium, (consider the unit length of footing)

$$q_u \times 2b \times 1 = -W + 2C\sin\phi' + 2P_p$$

$$\Rightarrow 2bq_u = 2P_p + 2 \times \frac{c'b}{\cos\phi'} \times \sin\phi' - \gamma b^2 \tan\phi'$$

$$\Rightarrow q_u = \frac{P_p}{b} + c'\tan\phi' - \frac{\gamma b}{2}\tan\phi' \quad \text{......(1)}$$

Here,

$$C = \frac{c'b}{\cos\phi'}$$

and,

$$W = \gamma b^2 \tan\phi'$$

The passive pressure is the sum of the contribution of the weight of soil γ, cohesion c' and surcharge q and can be expressed as:

$$P_p = \frac{1}{2}\gamma(b\tan\phi')^2 K_\gamma + c'(b\tan\phi')K_c + q(b\tan\phi')K_q \text{ ---- } (11)$$

where, K_γ, K_c and K_q are earth pressure co-efficient that are the functions of the soil friction angle ϕ'

From equation (1) and (11) we obtain,

$$q = c'N_c + qN_q + \frac{1}{2}\gamma BN_\gamma$$

where,

$N_c = \tan\phi'(K_c+1)$

$N_q = K_q \tan\phi'$

$N_\gamma = \frac{1}{2}\tan\phi'(K_\gamma \tan\phi' - 1)$

The terms N_c, N_q and N_γ are, respectively the contributions of cohesion, surcharge, and unit weight of soil to the ultimate load bearing capacity.

It is extremly tedious to evaluate K_c, K_q and K_γ.

For this reason, Terzaghi used an approximate method to calculate the ultimate bearing capacity, q_u. The principles of this approximations are the following:

1. If $c' = 0$ and surcharge $(q) = 0$ (that is, $D_f = 0$).

   Then, $q_\gamma = \frac{1}{2}\gamma BN_\gamma$

2. If $\gamma = 0$ (that is weightless soil) and $q = 0$ (that is $D_f = 0$)

   Then, $q_u = q_c = c'N_c$

3. If $\gamma = 0$ (weightless soil) and $c' = 0$,

   Then, $q_u = q_q = qN_q$

By the method of super position, we have,

$$q_u = q_c + q_q + q_\gamma = c' N_c + q N_q + \frac{1}{2} \gamma B N_\gamma$$

this equation is referred to as Terzaghi's bearing capacity equation.

For strip footing: $q_u = c' N_c + q N_q + \frac{1}{2} \gamma B N_\gamma$

For square footing: $q_u = 1.3c' N_c + q N_q + 0.4 \gamma B N_\gamma$

For circular footing: $q_u = 1.3 c' N_c + q N_q + 0.3 \gamma B N_\gamma$

<u>For undrained condition:</u>

$\phi = 0$ and $\tau_f = c_u$

$N_\gamma = 0$, $N_q = 1$ and $N_c = 5.7$

Hence, for strip footing, $q_u = 5.7 c_u + q$

for circular and square footing, $q_u = 1.3 \times 5.7 \times c_u + q$

i.e. $q_u = 7.41 c_u + q$

$$\boxed{\begin{gathered} N_c = \cot\phi' \left[\frac{e^{2\left(\frac{3\pi}{4} - \frac{\phi'}{2}\right)\tan\phi'}}{2\cos^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)} - 1 \right] = \cot\phi' (N_q - 1) \\ N_q = \frac{e^{2\left(\frac{3\pi}{4} - \frac{\phi'}{2}\right)\tan\phi'}}{2\cos^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)} \\ N_\gamma = \frac{1}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi'} - 1\right)\tan\phi' \quad \text{where, } K_{p\gamma} = \text{passive pressure co-efficient} \end{gathered}}$$

where, N_c, N_q, N_γ = Bearing capacity factors.

In case of Local shear failure:

$\bar{c'} = \frac{2}{3} c'$ and $\tan \bar{\phi'} = \frac{2}{3} \tan \phi'$

for strip footing, $q_u' = \bar{c'} N_c' + q N_q' + \frac{1}{2} \gamma B N_\gamma'$

for square footing, $q_u' = 1.3 \bar{c'} N_c' + q N_q' + 0.4 \gamma B N_\gamma'$

for circular footing, $q_u' = 1.3 \bar{c'} N_c' + q N_q' + 0.3 \gamma B N_\gamma'$

Substituting $\bar{\phi'} = \tan^{-1}\left(\frac{2}{3} \tan \phi'\right)$ for ϕ'

17, 14

Limitations of Terzaghi's analysis: (Akhtar Sir - Slide)

1. $D_f \leq B$.
2. No sliding resistance between footing and soil.
3. Soil is a homogeneous, semi infinite mass.
4. Failure plane angle is equal to ϕ'
5. Not applicable for inclined load & Rectangular foundation.
6. No resistance of soil above the level of base foundation.

General Bearing capacity Equation: (Meyerhof) 13

Meyerhof (1963) suggested following form of general bearing capacity equation,

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

where, $F_{cs}, F_{qs}, F_{\gamma s}$ = Shape factors

$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors.

F_{ci}, F_{qi}, F_{qi} = Load inclination factor

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Bearing capacity factors:

$N_q = \tan^2\left(45° + \frac{\phi'}{2}\right) e^{\pi \tan\phi'}$ (Reissner, 1924)

$N_c = (N_q - 1)\cot\phi'$ (Prandtl, 1921)

$N_\gamma = 2(N_q + 1)\tan\phi'$ (Vesic, 1973)

Shape factors:

$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$ [for circular, B=L]

$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi'$ (De Beer 1970)

$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right)$

Depth factors:

(for $\frac{D_f}{B} \leq 1$, $\phi' = 0$)

$F_{cd} = 1 + 0.4\left(\frac{D_f}{L}\right)$ (Hansen, 1970)

$F_{qd} = 1$

$F_{\gamma d} = 1$

(for $\frac{D_f}{B} \leq 1$, $\phi' > 0$) (Hansen, 1970)

$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'}$

$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2 \times \left(\frac{D_f}{B}\right)$

$F_{\gamma d} = 1$

(for $\frac{D_f}{B} > 1$, $\phi' = 0$)

$F_{cd} = 1 + \underbrace{0.4\tan^{-1}\left(\frac{D_f}{L}\right)}_{\text{radian}}$

$F_{qd} = 1$ (Hansen)

$F_{\gamma d} = 1$

(for $\frac{D_f}{B} > 1$, $\phi' > 0$)

$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'}$ (Hansen)

$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2 \underbrace{\tan^{-1}\left(\frac{D_f}{B}\right)}_{\text{radian}}$

$F_{\gamma d} = 1$

Influence factor:

$F_{ci} = F_{qi} = \left(1 - \frac{\beta}{90}\right)^2$; β = inclination of load with respect to vertical.

$F_{\gamma i} = \left(1 - \frac{\beta}{\phi'}\right)^2$

(Meyerhof, 1963)

Effect of Ground water Table on the bearing capacity of soil: 11, 14, 13, 12, 17, 16, 15

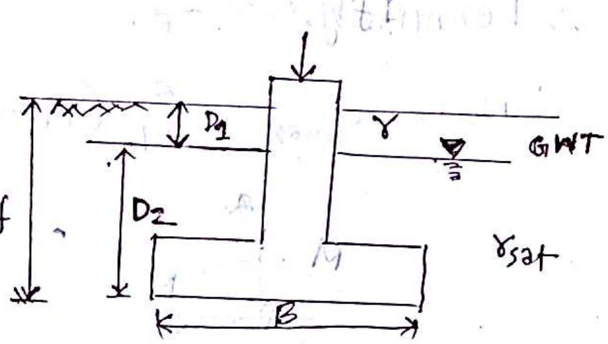
In developing the bearing capacity equations it is assumed that the ground water table is located at a depth much greater than the width B, of the footing.

But, if the ground water table is close to the footing, some changes are required.

Case-I: If the GWT is located at a distance D_2 above the bottom of the foundation $(0 \le D_1 \le D_f)$

$$q = D_1\gamma + D_2\gamma'$$

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



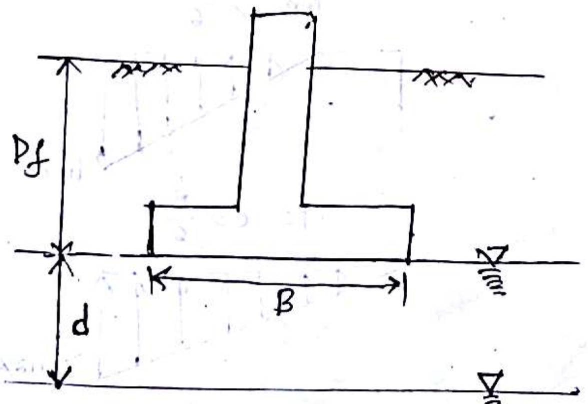
Also, the unit weight of soil γ, that appears in third term of bearing capacity equation. Should be replaced by γ'

Case-II: If the GWT coincides with the bottom of the foundation, $(0 \leq d \leq B)$

$$q = \gamma D_f$$

In this case, γ in third term of bearing capacity equation should replaced by the factor, $\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$

Case III:
When GWT is located so that $d > B$, water will have no effect on ultimate bearing capacity.



Eccentrically loaded foundation: (Footing with one-way eccentricity)

In several instances, foundations are subjected to moments in addition to vertical load. In such cases distribution upon soil is not uniform. The normal distribution of pressure is,

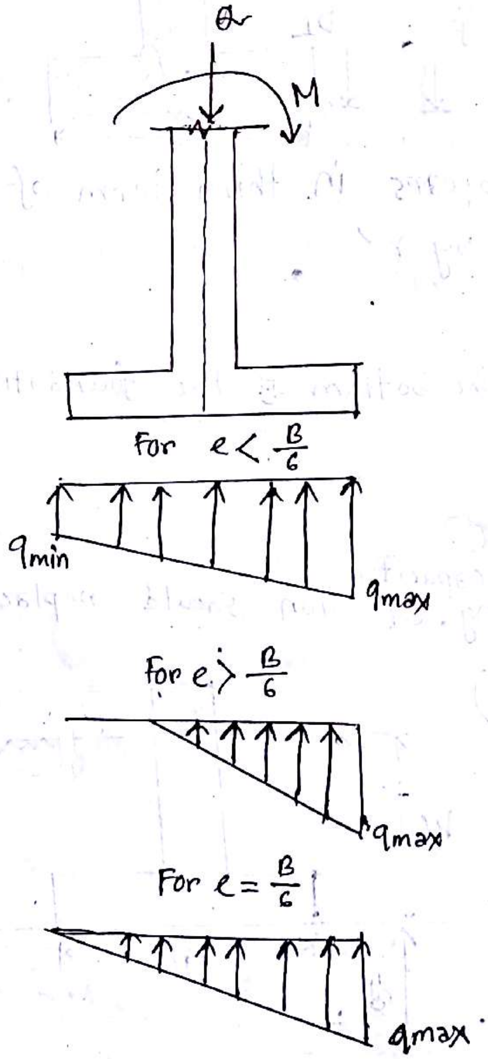
$$q_{max} = \frac{Q}{BL} + \frac{6M}{B^2L} \; ; \quad q_{min} = \frac{Q}{BL} - \frac{6M}{B^2L}$$

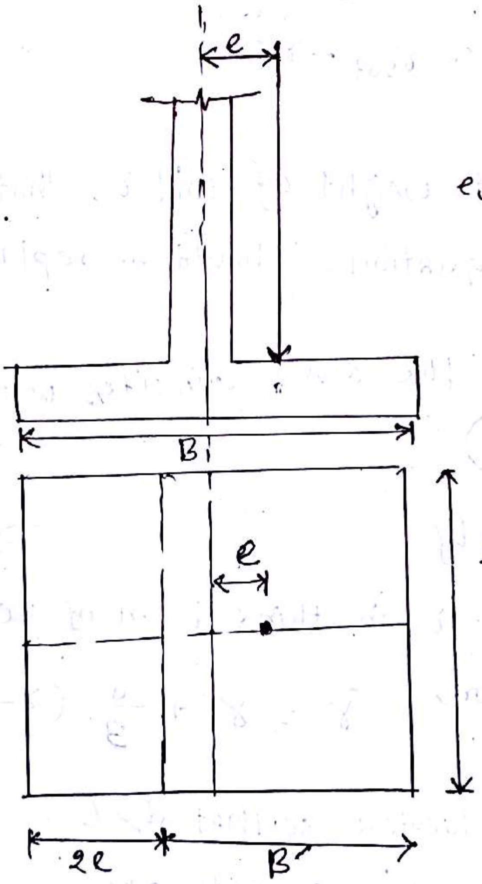
Where, Q = Total vertical load,

M = Moment of the foundation

$\therefore$ Eccentricity, $e = \frac{M}{Q}$

Now, $q_{max} = \frac{Q}{BL}\left(1 + \frac{6e}{B}\right)$; $q_{min} = \frac{Q}{BL}\left(1 - \frac{6e}{B}\right)$





effective Width is now

$B' = B - 2e$

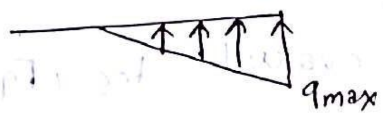
where as, The effective length is still

$L' = L$

∴ Note that,

When $e = \frac{B}{6}$; q_{min} is zero

For $e > \frac{B}{6}$; q_{min} is negative which means that tension will develop. Since soil can not take any tension, there will be a separation between foundation and soil underlying it.



The value of q_{max} is then,

$$q_{max} = \frac{4Q}{3L(B-2e)}$$

Factor of safety, $FS = \frac{Q_{ult}}{Q}$ where Q_{ult} = Ultimate load carrying capacity
Q = Applied load.

Ultimate Bearing Capacity under Eccentric Loading: (one-way eccentricity)

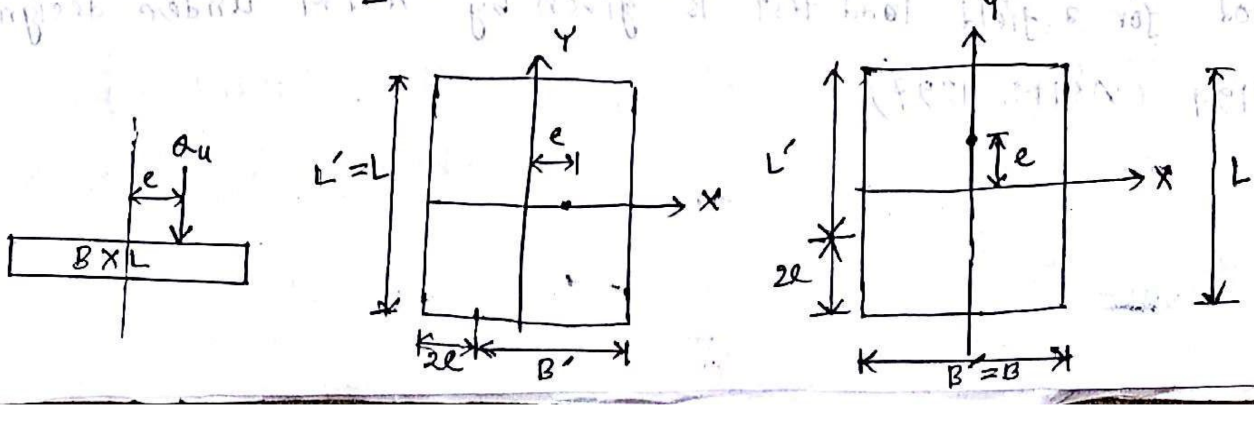
To calculate the bearing capacity of shallow foundations with eccentric loading, Meyerhof (1953) introduced the concept of effective area.

Step-1: Calculate the effective dimension of the foundation. If the eccentricity (e) is in the X direction, the effective dimensions are,

$$B' = B - 2e \quad \text{and} \quad L' = L$$

However, if the eccentricity (e) is in the Y direction, the effective dimensions are,

$$L' = L - 2e \quad \text{and} \quad B' = B$$



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The smaller of two dimensions (i.e. B' and L') is the effective width of the foundation, other is the effective length of the foundation.

Step: 2 Use ultimate bearing capacity equation.

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B' N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

To evaluate F_{cs}, F_{qs} and $F_{\gamma s}$ use relationships given earlier with B' and L' instead of B and L respectively

To determine F_{cd}, F_{qd}, $F_{\gamma d}$ use relationships given earlier without replacing B with B'.

Step: 3 Calculate the total ultimate load that the foundation can sustain is,

$$Q_{ult} = q_u (B' L') = q_u A' \quad \text{where } A' = \text{effective area.}$$

Step: 4 The factor of safety against bearing capacity failure is,

$$FS = \frac{Q_{ult}}{Q} \quad \text{where, } Q = \text{Total vertical load.}$$

Plate-Load Test: 17

In some cases, conducting field load tests to determine the soil-bearing capacity of foundations is desirable. The standard method for a field load test is given by ASTM under designation D-1194 (ASTM-1997)

⇒ circular steel bearing plates 162 to 760 mm in diameter and 305 mm × 305 mm square plates are used for this type of test

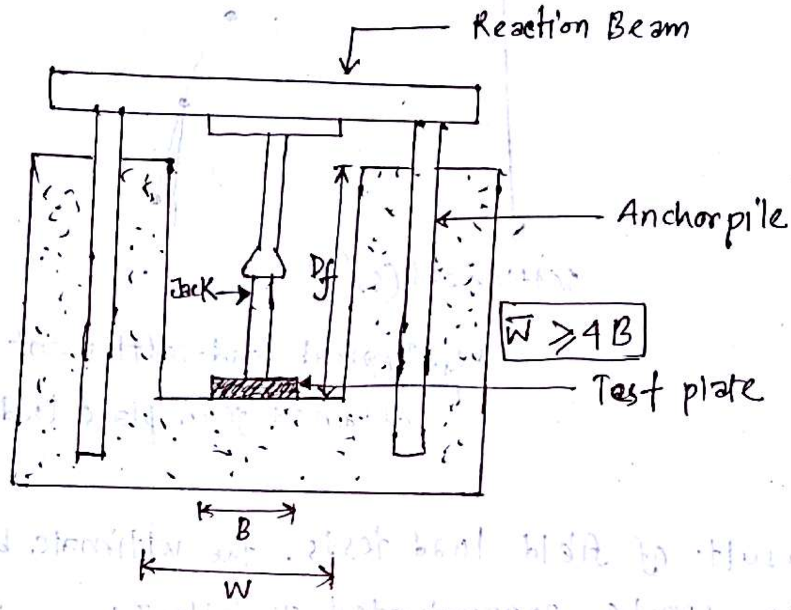


Fig. Diagram of plate-load test

⇒ To conduct the test, one must have a pit of depth, D_f excavated.

⇒ The width of the test pit should be at least four times the width of the bearing plate.

⇒ The bearing plate is placed on the soil at the bottom of the pit, and an incremental load on the bearing plate is applied.

⇒ After the application of incremental load, enough time is allowed for settlement to occur.

⇒ When the settlement of the bearing plate becomes negligible, another incremental load is applied.

⇒ In this manner, a load settlement plot can be obtained as shown in figure:

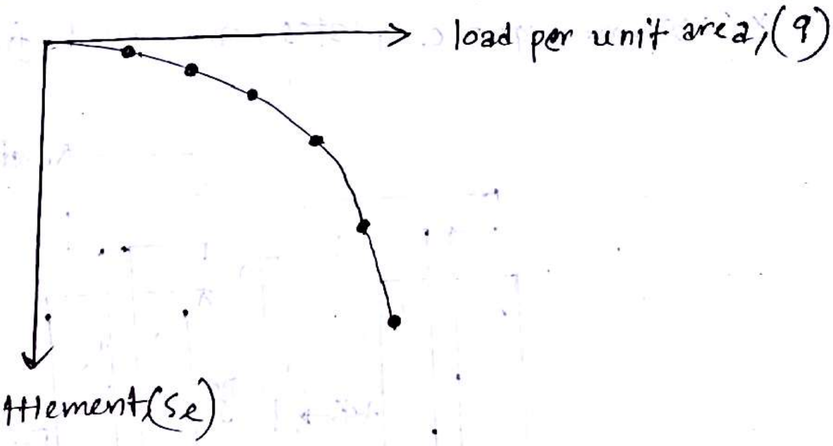


Fig: Typical load-settlement curve obtained from plate load test

⇒ From the result of field load tests, the ultimate bearing capacity of actual footing can be approximated as follows:

For clays, $q_{u(\text{footing})} = q_{u(\text{plate})}$

For sandy soils, $q_{u(\text{footing})} = q_{u(\text{plate})} \times \dfrac{B_{(\text{footing})}}{B_{(\text{plate})}}$

⇒ For given intensity of load q, the settlement of the actual footing also can be approximated as follows:

In clay, $S_{e(\text{footing})} = S_{e(\text{plate})} \times \dfrac{B_{(\text{footing})}}{B_{(\text{plate})}}$

In sandy soil,

$$S_{e(\text{footing})} = S_{e(\text{plate})} \times \left[\frac{2B_{(\text{footing})}}{B_{(\text{footing})} + B_{(\text{plate})}}\right]$$

Floating foundation: 17

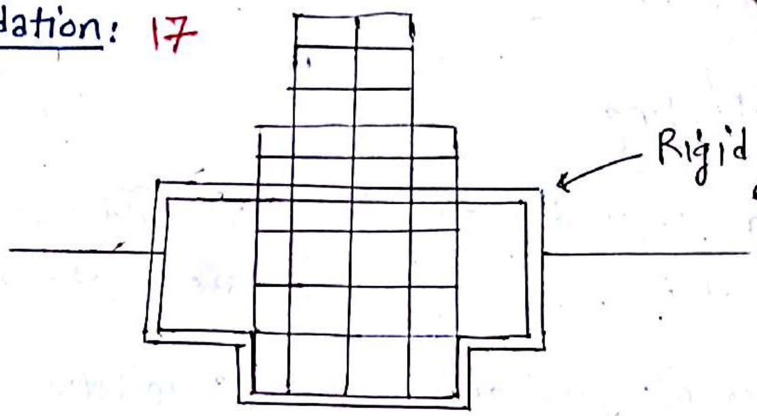


Fig. Floating foundation.

Where deep deposits of compressible cohesive are present and piles are impractical, Building substructure is a combination of mat and cassion to creat a rigid box.

$$q_{u(net)} = q_u - q \quad \text{where, } q = \gamma D \text{ (surcharge)}$$

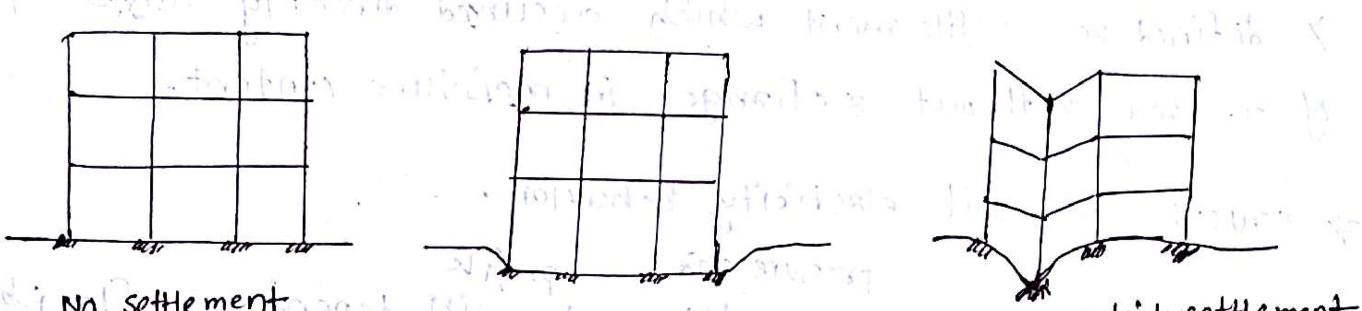
weight of earth displaced by foundation is equal to the total weight of structure, thereby minimizing settlement from consolidation.

Differentiate between Foundation and footing: 16

Footing	Foundation
1. Footing can be analogized with the feet of the leg.	1. Foundation can be compared with legs.
2. Footing is a type of shallow foundation.	2. Foundation can be shallow and deep.
3. Footing reinforces support to an individual column.	3. Foundation is an extensive support because it gives support to a group of footings as an entire building.

Footing	Foundation
4. A footing is under the foundation wall.	4. Foundations are the basement walls.
5. A number of footings repose on a foundation	5. Foundation is the support that bears all kinds of loadings
6. All footings are foundation.	6. Not all foundations are footings.

Settlement of A Foundation



uniform settlement is usually of little consequence in a building, but differential settlement can cause severe structural damage.

Settlement:

Total vertical deformation at surface resulting from External load and dewatering (lowering of water table) is called settlement.

The settlement of a foundation can be divided into two major categories:

(a) Elastic or Immediate settlement, S_e

— Due to pore air (after application of load)

(b) consolidation settlement

(i) primary consolidation settlement, S_c

— Due to pore water dissipation

(ii) Secondary consolidation settlement, S_s

— (creep)

$\therefore$ Total settlement, $S = S_e + S_c + S_s$

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Immediate settlement : 2012

⇒ defined as settlement which occurs directly after application of a load without a change in moisture content.

⇒ caused by soil elasticity behavior.

⇒ Magnitude of contact pressure and settlement profile will depend on flexibility of foundation and type of material on which it is resting.

⇒ For clay, immediate settlement generally very small comparing to consolidation settlement, therefore this immediate settlement mostly ignored.

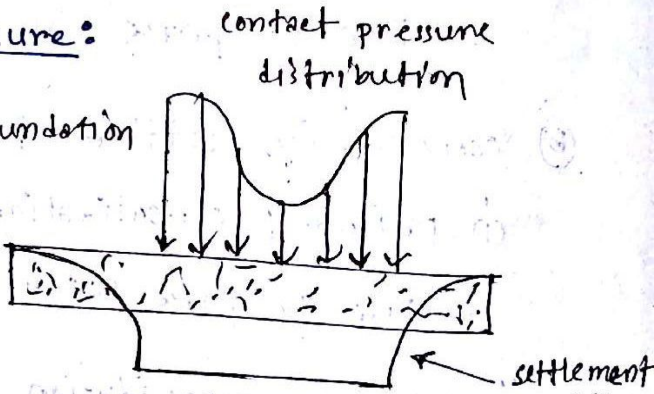
⇒ Usually considered at sand or sandy soil

⇒ Elastic settlement calculations generally are based on equations derived from theory of elasticity.

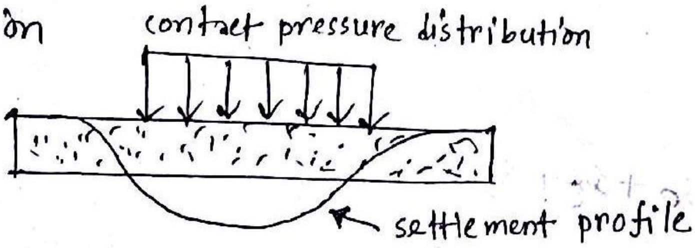
Elastic settlement profile and contact pressure:

In Clay:

(a) Rigid foundation

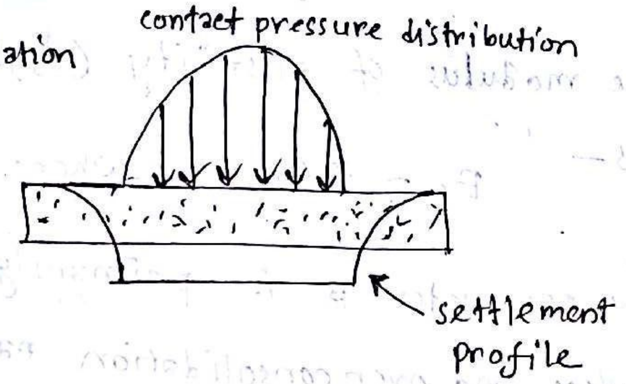


(b) flexible foundation

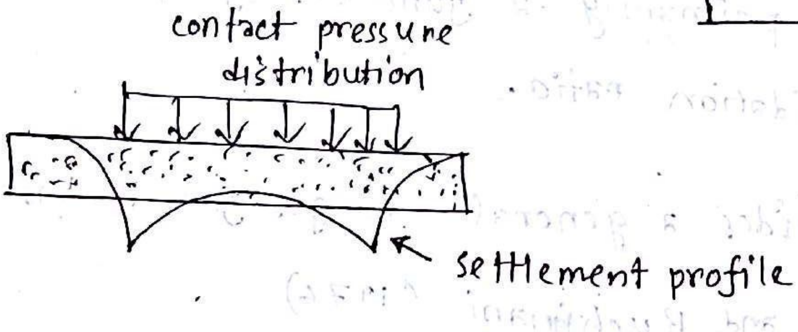


In Sand:

(a) Rigid foundation



(b) Flexible foundation



⊞ Elastic settlement of foundation on saturated clay ($\mu_s = 0.5$)

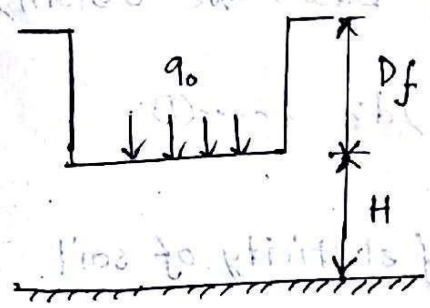
Janbu et. al. (1956) proposed an equation for evaluating average settlement of the flexible foundations on saturated clay soils (poisson's ratio, $\mu_s = 0.5$)

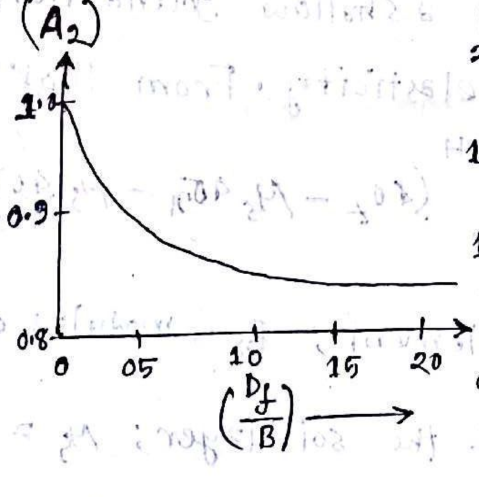
$$S_e = A_1 A_2 \frac{q_0 B}{E_s}$$

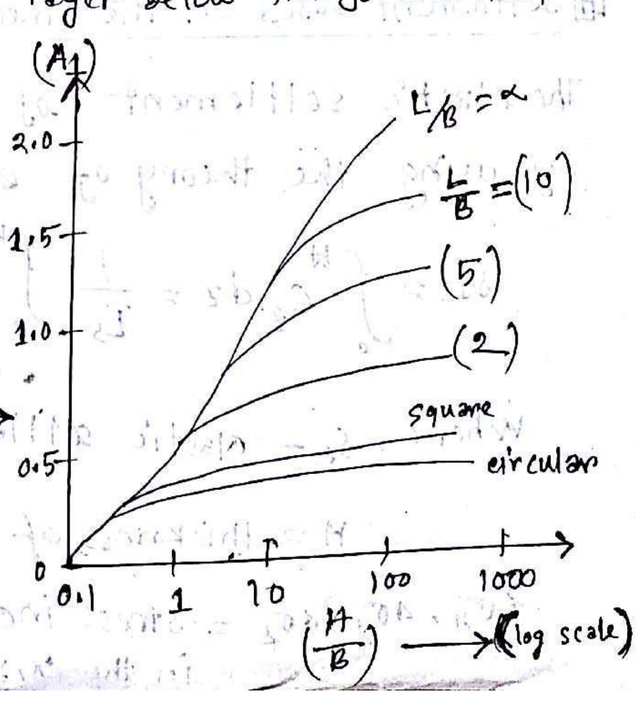
where A_1 is a function of $\frac{H}{B}$ and $\frac{L}{B}$

And, A_2 is a function of $\frac{D_f}{B}$

H = thickness of compressible layer below the foundation







The modulus of elasticity (E_s) for clay's can, in general, be given as —

$E_s = \beta C_u$ where, C_u = Undrained shear strength

The parameter β is primarily a function of the plasticity index and over consolidation ratio.

Following table provides a general range for β based on that proposed by Duncan and Buchignani (1976)

Table : Range of β for clay

Plasticity Index	β	β	β	β	β
—	OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
< 30	1500–600	1380–500	1200–580	950–380	730–300
30–50	600–300	550–270	580–220	380–180	300–150
> 50	300–150	270–120	220–100	180–90	150–75

Settlement based on the theory of Elasticity:

The elastic settlement of a shallow foundation can be estimated by using the theory of elasticity. From Hook's Law, we obtain,

$$S_e = \int_0^H \epsilon_z \, dz = \frac{1}{E_s} \int_0^H (\Delta\sigma_z - \mu_s \Delta\sigma_x - \mu_s \Delta\sigma_y) \, dz \quad \ldots\ldots (1)$$

Where, S_e = elastic settlement; E_s = modulus of elasticity of soil

H = thickness of the soil layer; μ_s = poisson's ratio of the soil.

$\Delta\sigma_x, \Delta\sigma_y, \Delta\sigma_z$ = stress increase due to net applied foundation load in the x, y, z direction respectively.

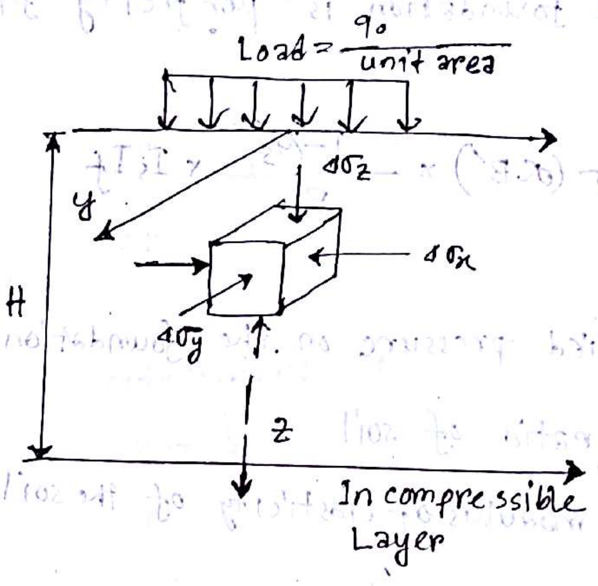


Fig. Elastic settlement of shallow foundation.

Relation for Elastic settlement Calculation:

Following figure shows a shallow foundation subjected to a net force per unit area equal to $\Delta\sigma$.

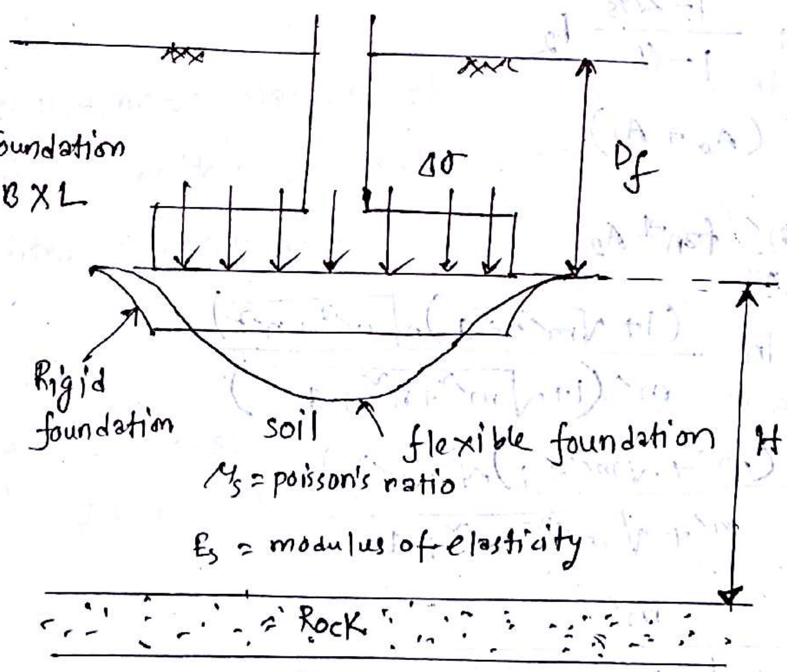


Fig. Elastic settlement of flexible and rigid foundation.

If, theoretically, the foundation is perfectly flexible, the settlement may be expressed as,

$$S_e = \Delta\sigma\,(\alpha B') \times \frac{1-\mu_s^2}{E_s} \times I_s I_f$$

where,

$\Delta\sigma$ = net applied pressure on the foundation

μ_s = poisson's ratio of soil

E_s = average modulus of elasticity of the soil ($z=0$ to $z=5B$)

$B' = \frac{B}{2}$ for center of foundation

$= B$ for corner of foundation

I_s = shape factor (Steinbrenner, 1934)

$$I_s = F_1 + \frac{1-2\mu_s}{1-\mu_s} F_2$$

$$F_1 = \frac{1}{\pi}(A_0 + A_1)$$

$$F_2 = \frac{n'}{2\pi} \tan^{-1} A_2$$

$$A_0 = m' \ln \frac{\left(1+\sqrt{m'^2+1}\right)\sqrt{m'^2+n'^2}}{m'\left(1+\sqrt{m'^2+n'^2+1}\right)}$$

$$A_1 = \ln \frac{\left(m'+\sqrt{m'^2+1}\right)\sqrt{1+n'^2}}{m'+\sqrt{m'^2+n'^2+1}}$$

$$A_2 = \frac{m'}{n'\sqrt{m'^2+n'^2+1}}$$

I_f = depth factor

$I_f = f\left(\frac{D_f}{B}, \mu_s \text{ and } \frac{L}{B}\right)$

α = factor that depends on the location on the foundation where settlement is being calculated.

* For calculation of settlement at the center of the foundation:

$$\alpha = 4$$

$$m' = \frac{L}{B}$$

$$n' = \frac{H}{(B/2)}$$

* For calculation of settlement at a corner of the foundation:

$$\alpha = 1$$

$$m' = \frac{L}{B}$$

$$n' = \frac{H}{B}$$

The elastic settlement of a rigid foundation can be estimated as:

$$S_e(\text{rigid}) \simeq 0.93\, S_e(\text{flexible, center})$$

Due to non-homogeneous nature of soil deposits, the magnitude of E_s may vary with depth. For that reason, Bowles (1987) recommended using a weighted average value of E_s:

$$E_s = \frac{\sum E_s(i)\,\Delta z}{\bar{z}}$$

where, $E_s(i)$ = soil modulus of elasticity within a depth Δz

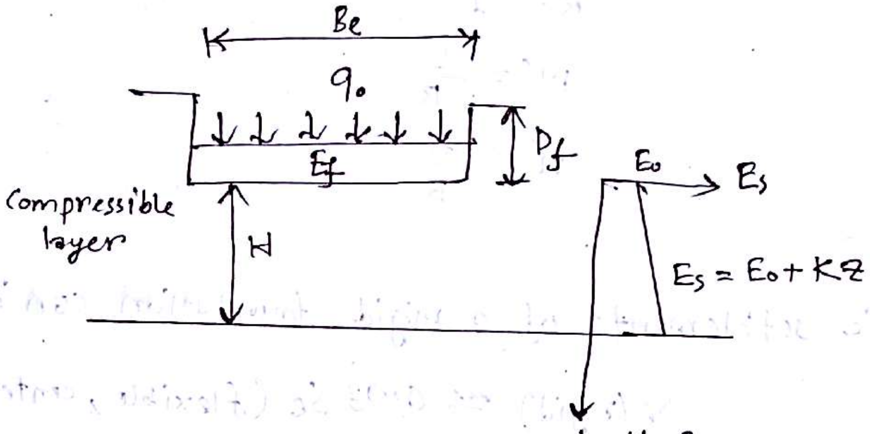
$\bar{z}$ = H or $5B$, whichever is smaller

Improved Equation for Elastic Settlement:

To use Mayne and Poulos's equation, the equivalent diameter (B_e) of a rectangular foundation,

$$B_e = \sqrt{\frac{4BL}{\pi}}$$

For circular foundation, $B_e = B$



depth, z

The elastic settlement below the center of the foundation is,

$$S_e = \frac{q_0 B_e I_G I_F I_E}{E_0} (1-\mu_s^2)$$

Where, I_G = Influence factor for variation of E_s with depth

$$= f\left(\beta = \frac{E_0}{KB_e}, \frac{H}{B_e}\right)$$

I_F = foundation rigidity correction factor

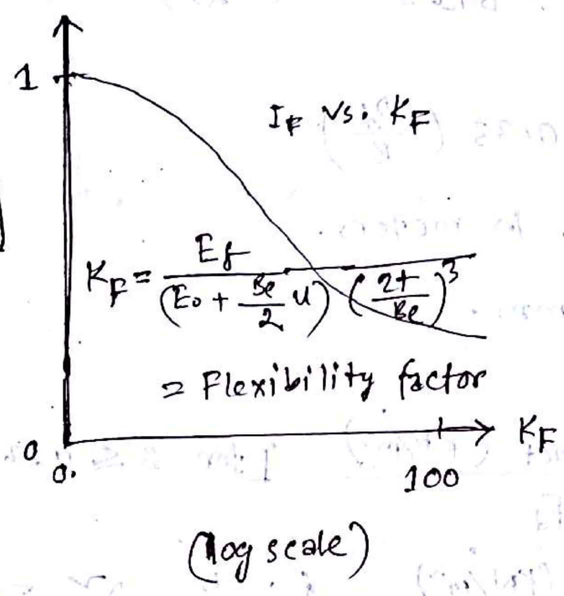
I_E = foundation embedment correction factor

The foundation rigidity correction factor,

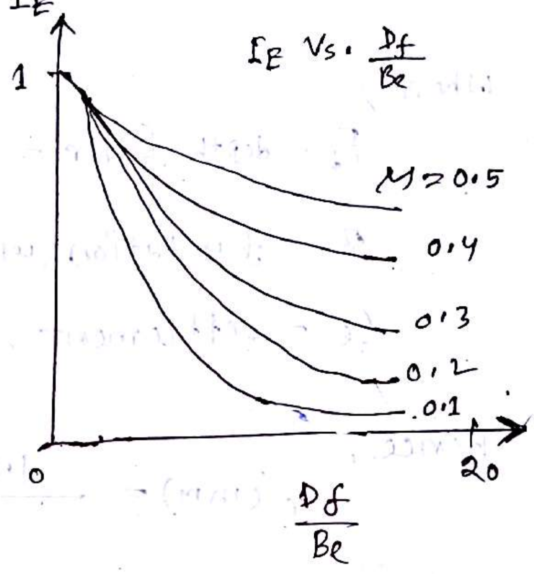
$$I_F = \frac{\pi}{4} + \frac{1}{4.6 + 10\left(\frac{E_f}{E_0 + \frac{B_e}{2}K}\right)\left(\frac{2t}{B_e}\right)^3}$$

- Foundation embankment correction factor –

$$I_E = 1 - \frac{1}{3.5\exp(1.22\mu_s - 0.4)\left(\frac{B_e}{D_f} + 1.6\right)}$$



Semi-log



Settlement of foundation on sand due to standard penetration Resistance:

Meyerhof's Method:

The net pressure, $q_{net} = \bar{q} - \gamma D_f$. where, $\bar{q}$ = stress at the level of foundation.

For 25 mm (1 inch), of estimated maximum settlement,

$$q_{net}\ (kN/m^2) = \frac{N_{60}}{0.08} \quad [\text{for } B \leq 1.22\ m]$$

and,

$$q_{net}\ (kN/m^2) = \frac{N_{60}}{0.125}\left(\frac{B + 0.3}{B}\right)^2 \quad [\text{for } B > 1.22\ m]$$

Bowle's modified form:

$$q_{net}\,(KN/m^2) = \frac{N_{60}}{0.05} F_d \left(\frac{S_e}{25}\right) \quad [\text{for } B \le 1.22\,m]$$

$$q_{net}\,(KN/m^2) = \frac{N_{60}}{0.08} \left(\frac{B+0.3}{B}\right)^2 F_d \left(\frac{S_e}{25}\right) \quad [\text{for } B > 1.22\,m]$$

where,

F_d = depth factor $= 1 + 0.33 \left(\frac{D_f}{B}\right)$

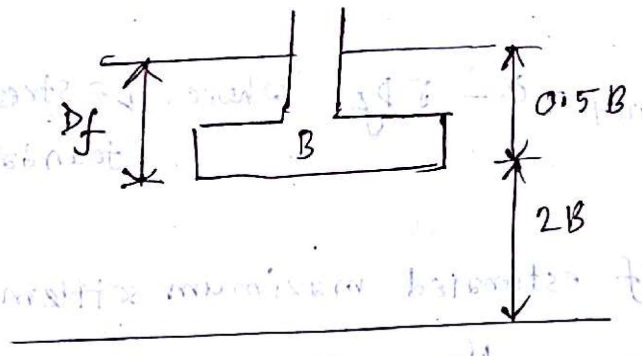
B = foundation width, in meters.

S_e = settlement, in mm.

Hence,

$$S_e\,(mm) = \frac{1.25\, q_{net}\,(KN/m^2)}{N_{60}\, F_d} \quad [\text{for } B \le 1.22\,m]$$

$$S_e\,(mm) = \frac{2\, q_{net}\,(KN/m^2)}{N_{60}\, F_d} \times \left(\frac{B}{B+0.3}\right)^2 \quad [\text{for } B > 1.22\,m]$$



Primary consolidation settlement Relationships:

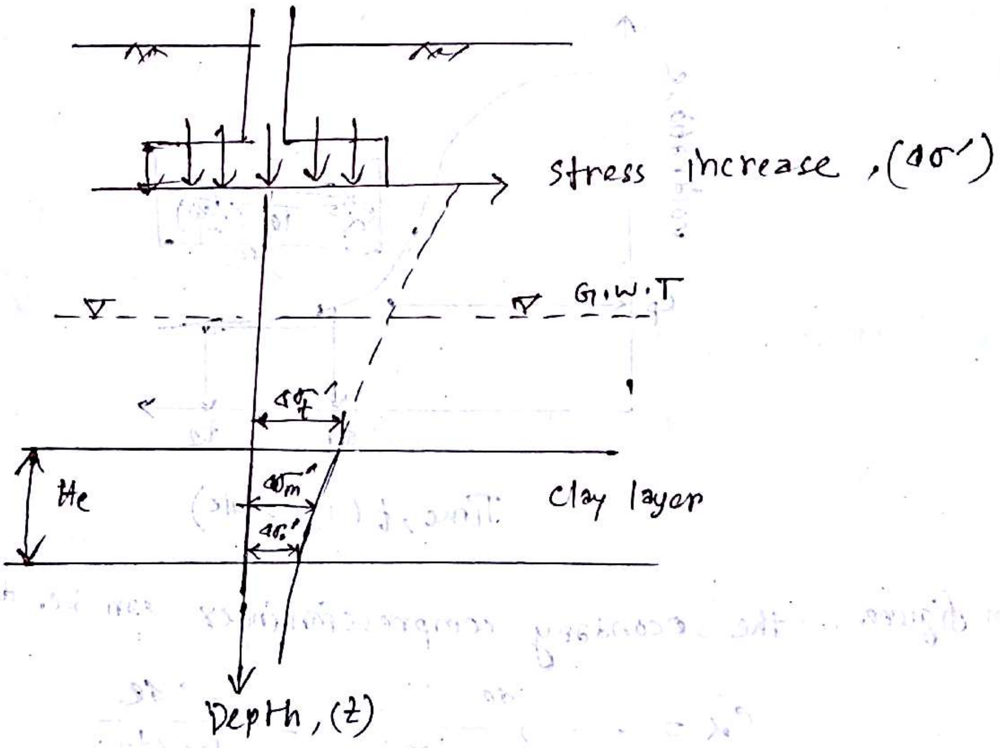
on the basis of the one dimensional consolidation settlement equation,

$S_{c(p)} = \int \epsilon_z \, dz$ where, ϵ_s = vertical strain $= \frac{\Delta e}{1+e_o}$

Δe = change of void ratio

$= f(\sigma_o', \sigma_c' \text{ and } \Delta\sigma')$

So,
$$S_{c(p)} = \frac{C_c H_c}{1+e_o} \times \log \frac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_o'}$$ [for normally consolidate clays]



$$S_{c(p)} = \frac{C_s H_c}{1+e_o} \log \frac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_o'}$$ [for overconsolidated clays, with $\sigma_o' + \Delta\sigma_{av}' < \sigma_c'$]

$$S_{c(p)} = \frac{C_s H_c}{1+e_o} \log \frac{\sigma_c'}{\sigma_o'} + \frac{C_c H_c}{1+e_o} \log \frac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_c'}$$ [for overconsolidated with $\sigma_o' < \sigma_c' < \sigma_o' + \Delta\sigma_{av}'$]

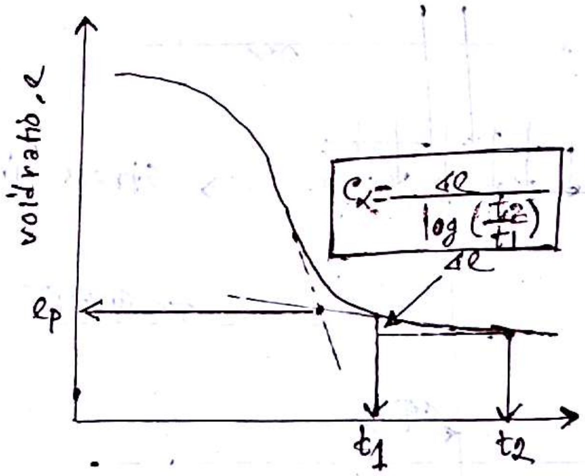
The average increase in effective pressure may be approximately

$$\Delta\sigma_{av}' = \frac{1}{6}\left(\Delta\sigma_t' + 4\Delta\sigma_m' + \Delta\sigma_b'\right)$$

Where, $\Delta\sigma_t'$, $\Delta\sigma_m'$ and $\Delta\sigma_b'$ are respectively the effective pressure increases at the top, middle and bottom of the clay layer.

Settlement due to secondary consolidation:

A plot of deformation against the logarithm of time during secondary consolidation is practically linear.



Time, t (log scale)

From figure, the secondary compression index can be defined as,

$$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} = \frac{\Delta e}{\log\left(\frac{t_2}{t_1}\right)}$$

Where, C_α = secondary compression index

Δe = change of void ratio

t_1, t_2 = time.

The magnitude of the secondary consolidation can be calculated as,

$$S_s = C_\alpha' H \log\left(\frac{t_2}{t_1}\right) \quad \text{and} \quad C_\alpha' = \frac{C_\alpha}{1+e_p}$$

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Where, e_p = void ratio at the end of primary consolidation

H = thickness of clay layer

The Magnitudes of C_α :

for over consolidated clays = 0.001 or less

for normally consolidated clays = 0.005 to 0.003

for organic soil = 0.04 or more

Differential Settlement: 2016, 15, 14, 12

Unequal settling of building foundation is called differential settlement.

Causes of Differential settlement: 2016, 15, 14, 12

(i) When the soil beneath the structure expands, contracts or shifts away. The main reasons for shifting are: (i) Highly expansive soil (ii) Frost action in soil (iii) poor drainage (iv) poorly compacted soil system.

(ii) change in water table

(iii) uneven drying of soil surface.

Measures that should be taken to avoid differential settlement: 2016, 15, 14, 12

(i) proper drainage.

(ii) proper geotechnical investigation of the soil.

(iii) The site soil must not be expansive in nature.

(iv) proper compaction of soil.

(v) proper soil treatment.

(vi) proper tie beam system.

(viii) No uneven drying of soil surface

(viii) No difference between the size and shape of the footing.

Assumptions for determining the increase in stress: 10

1. The load is applied at the ground surface.
2. The loaded area is flexible.
3. The soil medium is homogeneous, elastic, isotropic and extends to a great depth.

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Slope Stability

Slope:

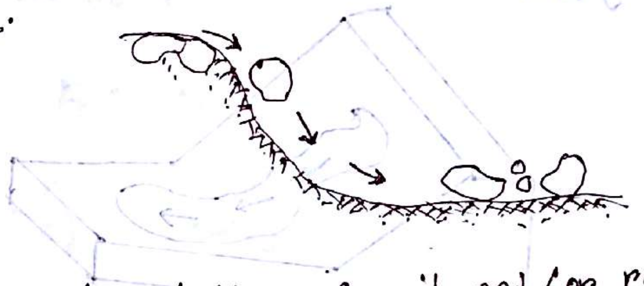
An exposed ground surface that stands at an angle with the horizontal is called an unrestrained slope.

The slope can be natural or man-made.

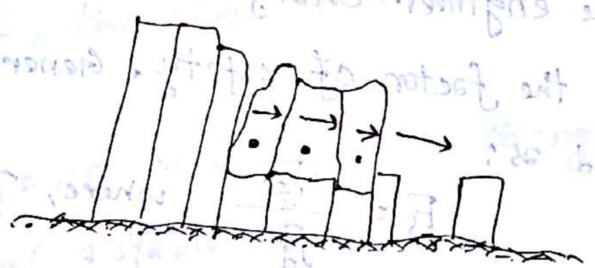
Classification of slope failure: 2013

Slope can fail in various modes. Cruden and Varnes (1996) classified the slope failures into the following five major categories. They are:

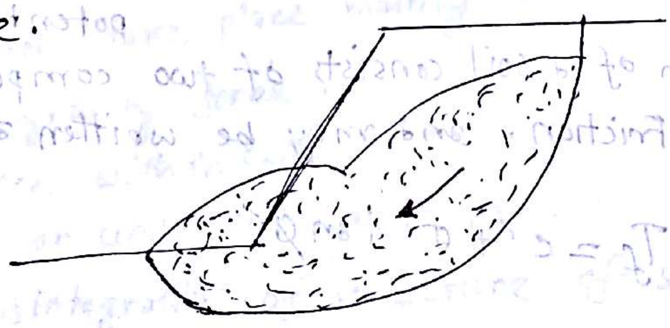
1. **Fall:** This is the detachment of soil and/or rock fragments that fall down a slope.



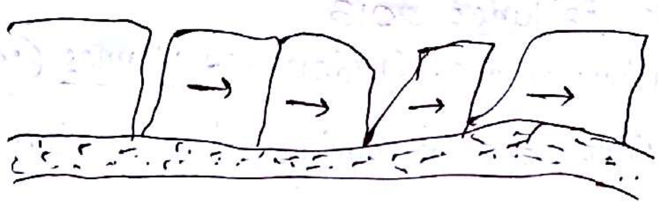
2. **Topple:** This is a forward rotation of soil and/or rock mass about an axis below the center of gravity of mass being displaced.



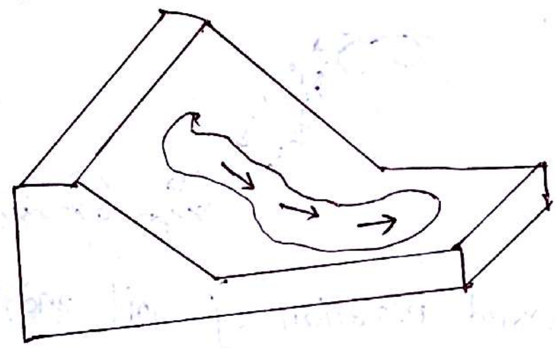
3. **Slide:** This is downward movement of a soil mass occuring on a surface of rupture.



4. Spread: This is a form of slide by transition. It occurs by "sudden movement of water bearing seams of sands or silts overlain by clays or loaded by fills."



5. Flow: This is a downward movement of soil mass similar to a viscous fluid.



Factor of Safety:

The task of the engineer charged with analysing slope stability is to determine the factor of safety. Generlly, The factor of safety is defined as:

$$F_s = \frac{\tau_f}{\tau_d}$$

where, τ_f = average shear strength of soil.

τ_d = average shear stress developed along the potential failure surface.

the shear strength of a soil consists of two components: 1. cohesion and 2. Friction. and may be written as:

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

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In similar manner, we can write,

$$\tau_d = c'_d + \sigma' \tan\phi'_d$$

Hence,

$$F_s = \frac{c' + \sigma' \tan\phi'}{c'_d + \sigma' \tan\phi'_d}$$

Factor of safety with respect to friction, $F_{\phi'}$ and the factor of safety with respect to cohesion, $F_{c'}$ are defined as:

$$F_{c'} = \frac{c'}{c'_d} \quad \text{and} \quad F_{\phi'} = \frac{\tan\phi'}{\tan\phi'_d}$$

comparing equations, we obtain,

$$F_{c'} = F_{\phi'}$$

$$\Rightarrow \frac{c'}{c'_d} = \frac{\tan\phi'}{\tan\phi'_d}$$

Then, we can write,

$$F_s = F_{c'} = F_{\phi'}$$

When, $F_s = 1$, the slope is in a state of impending failure.

Generally, $F_s = 1.5$ with respect to strength is acceptable for the design of a stable slope.

Causes of slope failure:

Failure of slopes take place mainly due to —

(i) Action of gravity force.

(ii) seepage force within soil.

(iii) Excavation or under cutting of foot.

(iv) Gradual disintegration of structure of soil.

Types of Slope:

There are two types of slope:

(i) Finite Slope

(ii) Infinite Slope

Differences between Infinite slope and Finite slope: 2012, 2011

Infinite Slope	Finite slope
1. If a slope represents the boundary surface of a semi-infinite soil mass and soil properties for all identical depth below surface area are constant, It is called Infinite slope.	1. If a slope is of limited extent, It is called Finite slope.
2. Slopes extending to infinity do not exist in nature.	2. Inclined forces of dam, embankments and cuts are finite slope.

Stability of Infinite Slopes:

The soil strength of the soil may be given by:

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

Assuming that, the pore water pressure is zero, we will evaluate the factor of safety against a possible slope failure along a plane AB located at a depth H below the ground surface.

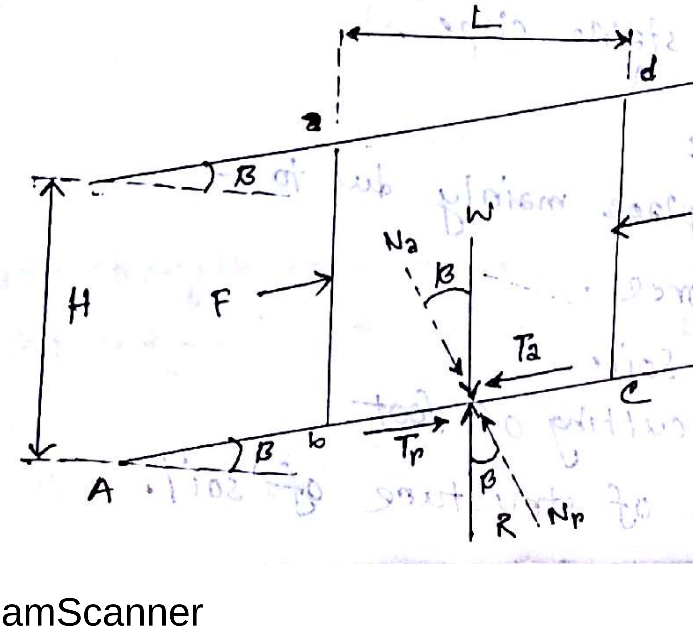

Fig. Analysis of Infinite slope

The slope failure can occure by the movement of soil above the plane AB from right to left.

Let us, consider a slope element abcd that has a unit length perpendicular to the plane of the section shown in figure.

The forces F_1 that act on the faces ab and cd are equal and opposite, and may be ignored.

The weight of soil element is,

$$W = \text{volume of soil element} \times \text{unit weight of soil}$$

$$\therefore W = \gamma L H$$

Force perpendicular to the plane AB, $N_a = W \cos\beta = \gamma L H \cos\beta$

Force parallel to the plane AB, $T_a = W \sin\beta = \gamma L H \sin\beta$

(This is the force that tends to cause the slip along surface AB)

Thus,

the effective normal stress, $\sigma' = \dfrac{N_a}{\text{Area of base}} = \dfrac{\gamma L H \cos\beta}{\left(\dfrac{L}{\cos\beta}\right)}$

$$\therefore \sigma' = \gamma H \cos^2\beta$$

and, the shear stress, $\tau = \dfrac{T_a}{\text{Area of base}} = \dfrac{\gamma L H \sin\beta}{\left(\dfrac{L}{\cos\beta}\right)}$

$$\therefore \tau = \gamma H \sin\beta \cdot \cos\beta$$

Here, $W = R$

$$\therefore N_a = N_r = R\cos\beta = W\cos\beta$$

and, $T_a = T_r = R\sin\beta = W\sin\beta$

For equilibrium, the resistive shear stress $= \gamma H \sin\beta \cdot \cos\beta$

It also be written as,

The resistive shear force, $\tau_d = c_d' + \sigma' \tan\phi_d'$

$$\Rightarrow \tau_d = c_d' + \gamma H \cos^2\beta \tan\phi_d'$$

$$\Rightarrow \gamma H \sin\beta \cdot \cos\beta = c_d' + \gamma H \cos^2\beta \tan\phi_d'$$

$$\Rightarrow \frac{c_d'}{\gamma H} = \sin\beta \cdot \cos\beta - \cos^2\beta \cdot \tan\phi_d'$$

$$\Rightarrow \frac{c_d'}{\gamma H} = \cos^2\beta(\tan\beta - \tan\phi_d')$$

we know, $\tan\phi_d' = \frac{\tan\phi'}{F_s}$ and $c_d' = \frac{c'}{F_s}$

substituting these values, we obtain,

$$\frac{\frac{c'}{F_s}}{\gamma H} = \cos^2\beta\left(\tan\beta - \frac{\tan\phi'}{F_s}\right)$$

$$\Rightarrow F_s = \frac{c'}{\gamma H \cos^2\beta \tan\beta} + \frac{\tan\phi'}{\tan\beta}$$

For granular soils, $c' = 0$ $\therefore F_s = \frac{\tan\phi'}{\tan\beta}$

This indicates that, in an infinite slope in sand, the value of F_s is independent of the height H.

And, the slope is stable as long as $\beta < \phi'$

If a soil possesses cohesion and friction, the depth of the plane along which critical equilibrium occurs, substituting $F_s = 1$ and $H = H_{cr}$. Thus,

$$H_{cr} = \frac{c'}{\gamma} \times \frac{1}{\gamma \cos^2\beta(\tan\beta - \tan\phi')}$$

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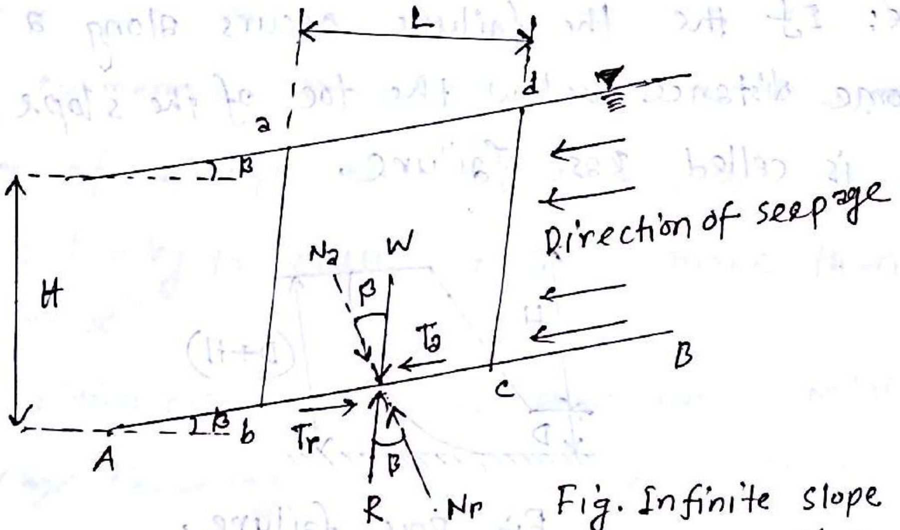


Fig. Infinite slope with steady-state seepage.

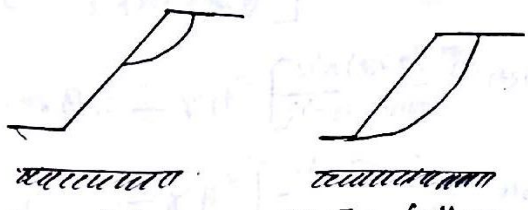
If there is steady-state seepage through the soil and ground water coinsides with the ground surface, as shown in figure,

$$F_s = \frac{c'}{\gamma_{sat} H \cos^2\beta \tan\beta} + \frac{\gamma' \tan\phi'}{\gamma_{sat} \tan\beta}$$

Types of failure of finite slopes: 2013

Failure of finite slopes occurs along a surface which is a curve. Two basic types of failure of a finite slope may occur:
(i) Slope failure (ii) Base failure.

(i) Slope failure: If the failure occurs along a surface of sliding that intersects the slope at or above its toe, the slide is known as slope failure.
Slope failure is called a face failure, if the arc passes above the toe.
Or, Toe failure if the arc passes through the toe.



(a) Face failure (b) Toe failure

Fig. Slope Failure

(ii) Base failure: If the the failure occurs along a surface that passes at some distance below the toe of the slope, such type of failure is called Base failure.

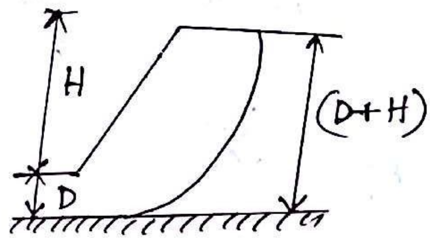


Fig. Base failure.

Types of slip surfaces or failure surfaces:

The rupture of a finite slope may take place along:

(i) Planer failure surface.

(ii) circular failure surface.

(iii) Non-circular failure surface.

Method of Analysis of the stability of finite slope:

1. culmann's method of planer surface
2. The swedish circle method (slip circle method)
3. The friction circle method.
4. Bishop's method. etc.

Analysis of Finite slopes with Plane Failure Surface (Culmann's Method) 2017

Assumptions: Culmann's analysis is based on the assumptions that –

(i) The failure of a slope occurs along a plane when average shearing stress tending to cause the slip is more than the shear strength of the soil.

(ii) The most critical plane is the one that has a minimum ratio of the average shearing stress to the shear strength of soil.

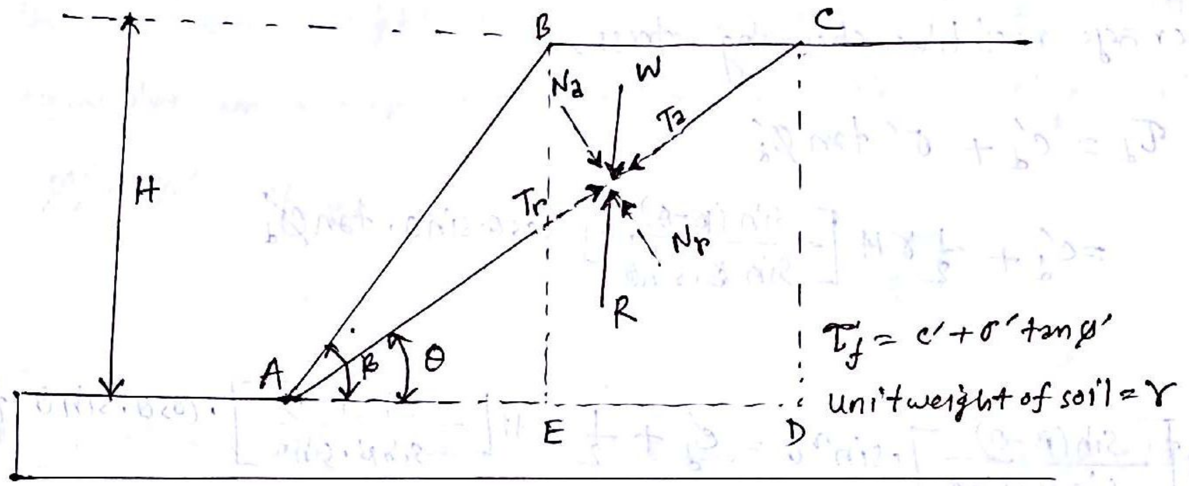


Fig. Finite slope analysis – Culmann's method

This Figure shows a slope of height H. The slope rises at an angle β with the horizontal. AC is a trial failure plane.

The weight of the wedge ABC, $W = \frac{1}{2} \times (H) \times (\overline{BC}) \times (1) \times (\gamma)$

$$= \frac{1}{2} H (H\cot\theta - H\cot\beta)\gamma$$

$$= \frac{1}{2}\gamma H^2 \left[\frac{\sin(\beta-\theta)}{\sin\beta \cdot \sin\theta}\right]$$

$$\boxed{\begin{aligned} BC &= AD - AE \\ &= H\cot\theta - H\cot\beta \\ &= H\left(\frac{\cos\theta}{\sin\theta} - \frac{\cos\beta}{\sin\beta}\right) \\ &= H \cdot \frac{\sin(\beta-\theta)}{\sin\theta \cdot \sin\beta} \end{aligned}}$$

The normal component of W, $N_a = W\cos\theta = \frac{1}{2}\gamma H^2 \left[\frac{\sin(\beta-\theta)}{\sin\beta \cdot \sin\theta}\right] \times \cos\theta$

The tangential component of W, $T_a = W\sin\theta = \frac{1}{2}\gamma H^2 \left[\frac{\sin(\beta-\theta)}{\sin\beta \cdot \sin\theta}\right] \times \sin\theta$

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The average effective normal stress,

$$\sigma' = \frac{N_a}{(\overline{AC}) \times (1)} = \frac{N_a}{\left(\frac{H}{\sin\theta}\right)} = \frac{1}{2}\gamma H \left[\frac{\sin(\beta-\theta)}{\sin\beta \cdot \sin\theta}\right] \cos\theta \cdot \sin\theta$$

and, The average shear stress,

$$\tau = \frac{T_a}{(\overline{AC})\,(1)} = \frac{T_a}{\left(\frac{H}{\sin\theta}\right)} = \frac{1}{2}\gamma H \left[\frac{\sin(\beta-\theta)}{\sin\beta \cdot \sin\theta}\right] \sin^2\theta$$

The average resistive shearing stress,

$$\tau_d = c'_d + \sigma' \tan\phi'_d$$

$$= c'_d + \frac{1}{2}\gamma H \left[\frac{\sin(\beta-\theta)}{\sin\beta \cdot \sin\theta}\right] \cos\theta \cdot \sin\theta \cdot \tan\phi'_d$$

Hence,

$$\frac{1}{2}\gamma H \left[\frac{\sin(\beta-\theta)}{\sin\beta \cdot \sin\theta}\right] \cdot \sin^2\theta = c'_d + \frac{1}{2}\gamma H \left[\frac{\sin(\beta-\theta)}{\sin\beta \cdot \sin\theta}\right] \cdot \cos\theta \cdot \sin\theta \cdot \tan\phi'_d$$

$$\Rightarrow c'_d = \frac{1}{2}\gamma H \left[\frac{\sin(\beta-\theta)(\sin\theta - \cos\theta \cdot \tan\phi'_d)}{\sin\beta}\right]$$

For the critical failure plane,

$$\frac{\partial c'_d}{\partial\theta} = 0$$

$$\Rightarrow \frac{\partial}{\partial\theta}\left[\sin(\beta-\theta)(\sin\theta - \cos\theta \cdot \tan\phi'_d)\right] = 0$$

$$\Rightarrow \theta_{cr} = \frac{\beta + \phi'_d}{2}$$

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substituting the value of $\theta = \theta_{cr}$, we obtain,

$$c'_d = \frac{\gamma H}{4}\left[\frac{1-\cos(\beta-\phi'_d)}{\sin\beta\cos\phi'_d}\right]$$

$$\Rightarrow \frac{c'_d}{\gamma H} = \frac{1-\cos(\beta-\phi'_d)}{4\sin\beta\cos\phi'_d}$$

$$\Rightarrow m = \frac{1-\cos(\beta-\phi'_d)}{4\sin\beta \cdot \cos\phi'_d}$$ where, m = stability number

The maximum height of the slope for which ~~equ~~ critical equilibrium occurs can obtained by substituting $c'_d = c'$ and $\phi'_d = \phi'$. Thus,

$$H_{cr} \doteq \frac{4c'}{\gamma}\left[\frac{\sin\beta \cdot \cos\phi'}{1-\cos(\beta-\phi')}\right]$$

* Culmann's method is suitable for very steep slope.

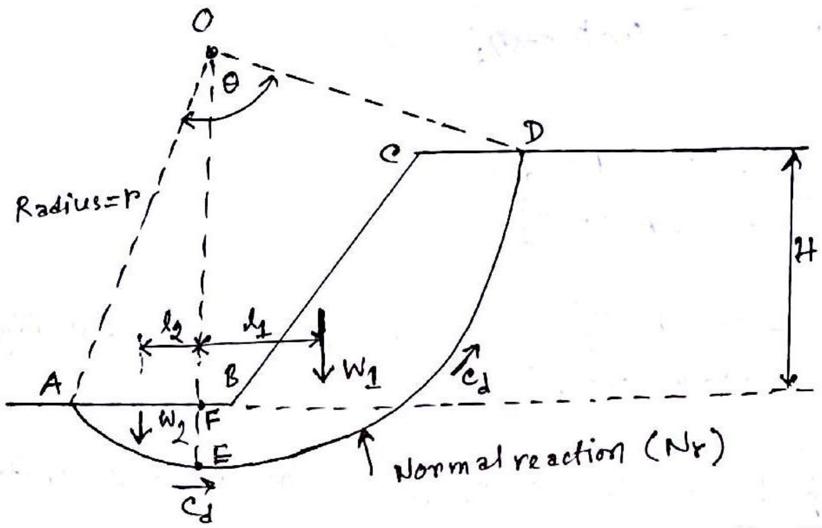
Types of stability Analysis Procedure: Two major classes are:

1. Mass procedure. 2. Method of slices. [for circular failure surface]

1. **Mass procedure:** In this case, the mass of the soil above the surface of sliding is taken as a unit. This procedure is useful when the soil that forms the slope is assumed to be homogeneous.

2. **Method of Slices:** In this procedure, soil above the surface of sliding is divided in to a number of vertical parallel slices. The stability of each slice is calculated seperately. This is a versatile Technique. In this procedure, Nonhomogeneity of the soils and pore water pressure can be taken into consideration.

Mass Procedure – Slopes in Homogeneous clay soil with $\phi = 0$



unit weight of soil $= \gamma$

$\tau_f = c_u$

This figure shows a slope in a homogeneous soil. The undrained shear strength of the soil is assumed to be constant with depth and may be given by $\tau_f = c_u$.

To perform the stability analysis, we choose a trial potential curve of sliding, AED, which is an arc of a circle that has a radius r. The center of the circle is located at O. The weight of the soil above the curve AED, $W = W_1 + W_2$

where, $W_1 = (\text{Area of FBCDEF}) \times (\gamma) \times (1)$

and, $W_2 = (\text{Area of AFEA}) \times (\gamma) \times (1)$

Failure of the slope may occur by sliding of the soil mass. The moment of the driving force about O to cause slope instability is, $M_d = W_1 l_1 - W_2 l_2$ where, l_1 & l_2 = Moment Arm

If c_d is the cohesion that needs to be developed, the moment of the resisting forces about O is,

$$M_R = c_d(\widehat{AED})(1).(r) = c_d r^2 \theta$$

For equilibrium, $M_R = M_d$

$$\Rightarrow c_d r^2 \theta = W_1 l_1 - W_2 l_2$$

$$\Rightarrow c_d = \frac{W_1 l_1 - W_2 l_2}{r^2 \theta}$$

The factor of safety against sliding, $F_s = \frac{\tau_f}{c_d} = \frac{c_u}{c_d}$

For the case of critical circles,

The developed cohesion, $c_d = \gamma H m$

$$\Rightarrow \frac{c_d}{\gamma H} = m$$

Substituting $H = H_{cr}$ and $c_d = c_u$ (full mobilization of undrained shear strength)

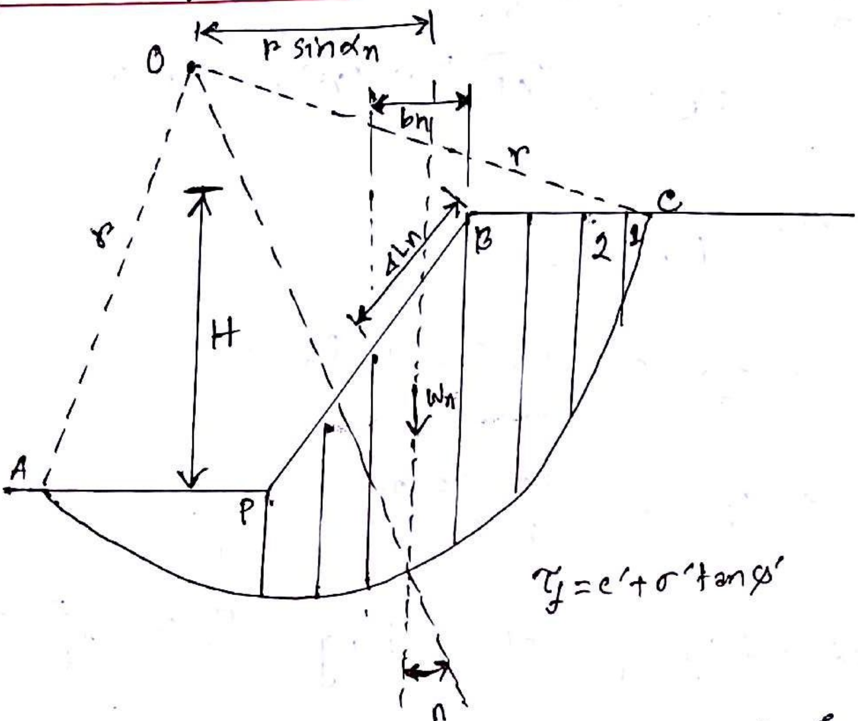
$$H_{cr} = \frac{c_u}{\gamma m}$$

Mass procedure - slopes in Homogeneous c-φ soil: (Book)

B.M. Das → Geotechnical Engineering

Chapter 15 : Slope Stability [Page : 535]

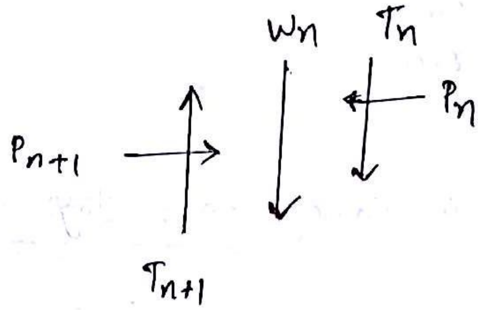
Ordinary method of slices:

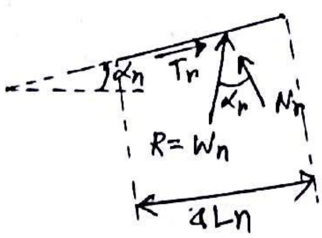


$\tau_f = c' + \sigma' \tan\phi'$

In this method, The soil above the failure surface is divided into several vertical slices. The width of each slices need not be the same. In the figure, AC is an arc of a circle representing the trial failure surface.

considering a unit length perpendicular to the cross section, the forces that act on a typical slice (nth slice) are shown in figure below:





For equilibrium consideration, $N_r = W_n \cos\alpha_n$

The resisting shear force,

$$T_r = \tau_d (\Delta L_n) = \frac{\tau_f (\Delta L_n)}{F_s} = \frac{1}{F_s}[c' + \sigma' \tan\phi'] \Delta L_n$$

The normal stress, $\sigma' = \frac{N_r}{\Delta L_n} = \frac{W_n \cos\alpha_n}{\Delta L_n}$

For equilibrium of the trial wedge ABC,

The moment of the driving force about O equals the moment of the resisting force about O.

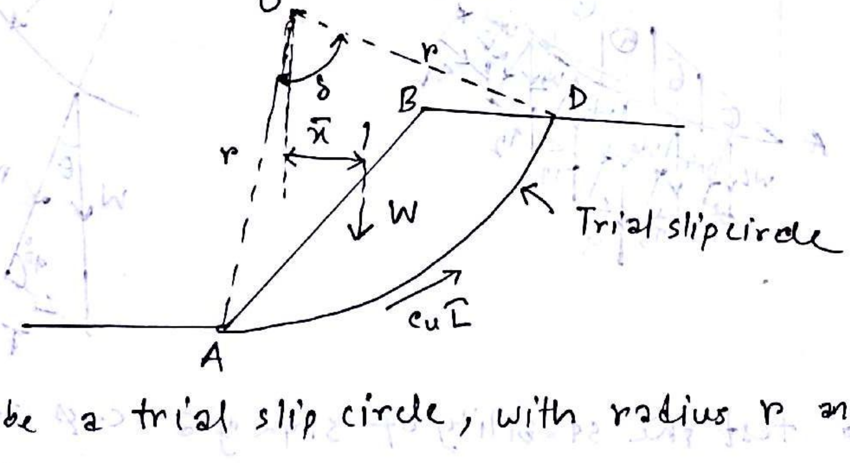
$$\therefore \sum_{n=1}^{n=p} W_n r \sin\alpha_n = \sum_{n=1}^{n=p} \frac{1}{F_s}\left(c' + \frac{W_n \cos\alpha_n}{\Delta L_n} \tan\phi'\right)(\Delta L_n)(r)$$

$$\Rightarrow F_s = \frac{\sum_{n=1}^{n=p} (c' \Delta L_n + W_n \cos\alpha_n \tan\phi')}{\sum_{n=1}^{n=p} W_n \sin\alpha_n}$$

where, $\Delta L_n = \frac{b_n}{\cos\alpha_n}$

The swedish slip circle method: . 2012, 10

(i) Analysis of purely cohesive soil ($\phi_u = 0$):



Let, AD be a trial slip circle, with radius r and center of rotation 'O'.

Let, W = weight of the soil of the wedge ABDA of unit thickness.

The driving moment, $M_D = W\bar{x}$

If c_u = unit cohesion and $\hat{L}$ = length of slip arc $AD = \frac{2\pi r\delta}{360^\circ}$

The shear resistance developed along the slip surface, $M_R = c_u \hat{L} \cdot r$

$\therefore$ The factor of safety, $F_s = \frac{M_R}{M_D} = \frac{c_u \hat{L} r}{W\bar{x}}$

Alternatively,

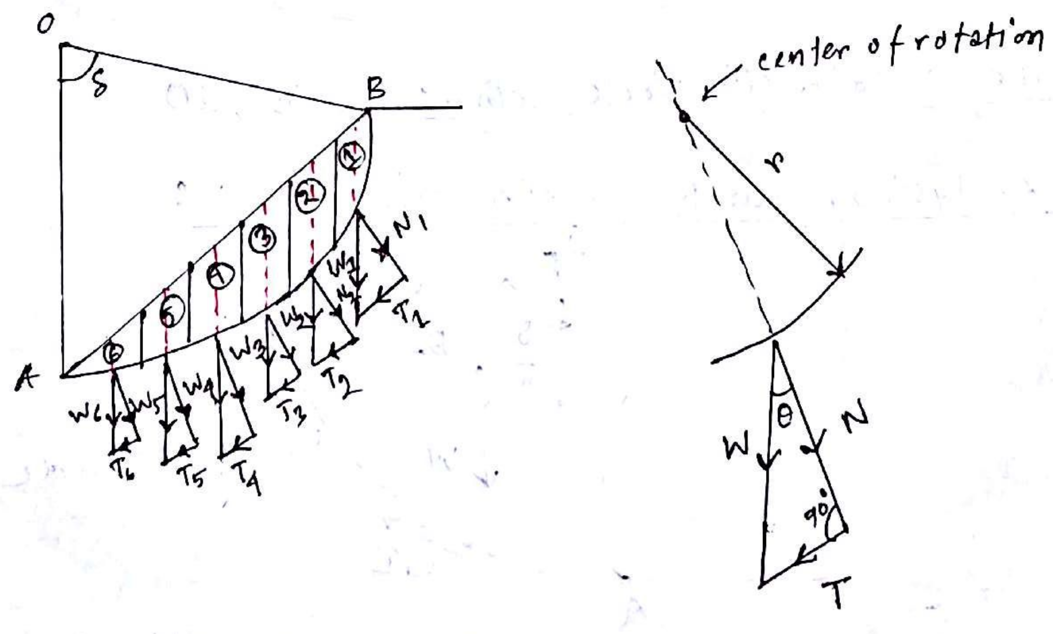
Let c_m = mobilised shear resistance of soil ($\phi = 0$)

For equilibrium, $W\bar{x} = c_m \hat{L} \cdot r$

$$\Rightarrow c_m = \frac{W\bar{x}}{\hat{L}} \cdot r$$

Hence, $F = \frac{c_u}{c_m} = \frac{c_u \hat{L} r}{W\bar{x}}$

(ii) Analysis of a $c-\phi$ soil:



In order to test the stability of slope of a $c-\phi$ soil, trial slip circle is drawn, and the material above assumed slip surface is divided into a convenient number of vertical slices.

The forces between the slices are neglected and each slice is assumed to act independently as a column of soil of unit thickness and of width b.

The tangential component T causes a driving moment,

$$M_D = T \times r.$$

If c = unit cohesion and ΔL = curved length of each slice,

the resisting Moment, $M_R = (c\,\Delta L + N \tan\phi)r$ (from coulomb's equation)

For entire slip surface AB,

$$M_D = r\Sigma T \quad \text{and} \quad M_R = r\left[c\Sigma\Delta L + \tan\phi\,\Sigma N\right]$$

where, ΣT = algebric sum of all tangential components

ΣN = " " of all normal components

$$\Sigma\Delta L = \hat{L} = \frac{2\pi r\delta}{360^\circ}$$

Hence, Factor of safety against sliding, $F_s = \dfrac{M_R}{M_D}$

$$F_s = \frac{c\hat{L} + \tan\phi\,\Sigma N}{\Sigma T}$$

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Bishop's simplified method of slices: 2013

In this method, the effect of forces on the sides of each slice are accounted for to some degree.

T

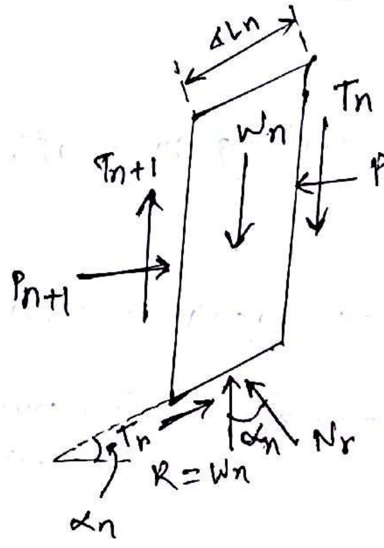


Fig. n th slice

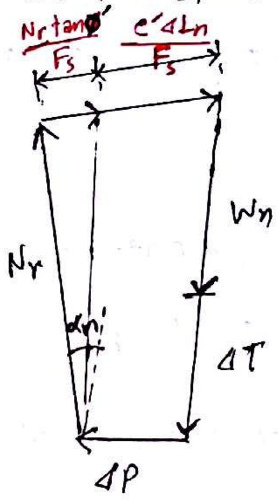


Fig. Force Polygon

Here,
$$\Delta T = T_n - T_{n+1}$$

Here

We can write,

$$T_r = N_r(\tan\phi_d') + c_d'\Delta L_n = N_r\left(\frac{\tan\phi'}{F_s}\right) + \frac{c'\Delta L_n}{F_s}$$

From Force Polygon diagram, summing forces in vertical direction,

$$W_n + \Delta T = N_r \cos\alpha_n + \left[\frac{N_r \tan\phi'}{F_s} + \frac{c'\Delta L_n}{F_s}\right]\sin\alpha_n$$

$$\Rightarrow N_r = \frac{W_n + \Delta T - \frac{c'\Delta L_n}{F_s}\sin\alpha_n}{\cos\alpha_n + \frac{\tan\phi'\sin\alpha_n}{F_s}}$$

For equilibrium of wedge ABC, Taking moment about O gives,

$$\sum_{n=1}^{n=p} W_n r \sin\alpha_n = \sum_{n=1}^{n=p} T_r \cdot r$$

Substituting the values of T_r and N_r we obtain,

$$F_s = \frac{\sum_{n=1}^{n=p}(c'b_n + W_n\tan\phi' + \Delta T\tan\phi')\frac{1}{m_{\alpha(n)}}}{\sum_{n=1}^{n=p} W_n \sin\alpha_n}$$

Where,

$$m_{\alpha(n)} = \cos\alpha_n + \frac{\tan\phi' \sin\alpha_n}{F_s}$$

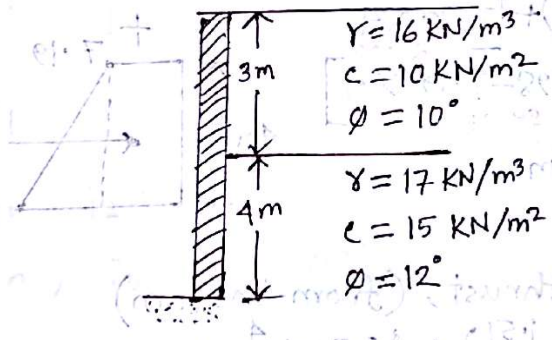
For simplicity, if we let, $\Delta T = 0$

Then,

$$F_s = \frac{\sum_{n=1}^{n=P} (c' b_n + W_n \tan\phi') \frac{1}{m_\alpha(n)}}{\sum_{n=1}^{n=P} W_n \sin\alpha_n}$$

Lateral Earth Pressure

A retaining wall is shown in figure below. Compute (i) lateral thrust on the wall and (ii) location of lateral thrust.



Solution:

For Upper layer, $K_{a_1} = \dfrac{1-\sin\phi}{1+\sin\phi} = \dfrac{1-\sin 10}{1+\sin 10} = 0.704$

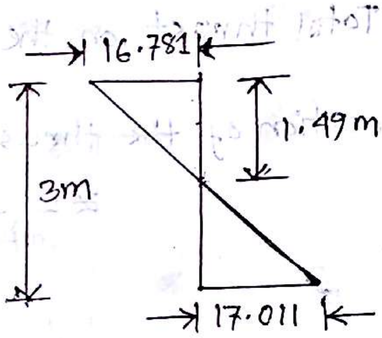
$$P_a = K_{a_1}\gamma_1 z_1 - 2c_1\sqrt{K_{a_1}} = 0.704 \times 16 \times z_1 - 2 \times 10 \times \sqrt{0.704}$$

$$\therefore P_a = 11.264 z_1 - 16.781$$

When, $z_1 = 0\,m$, $P_a = -16.781\ KN/m^2$

$z_1 = 3m$, $P_a = 17.011\ KN/m^2$

(and) $P_a = 0$, $z_1 = 1.49\ m$



For layer 2, $K_{a_2} = \dfrac{1-\sin 12}{1+\sin 12} = 0.656$

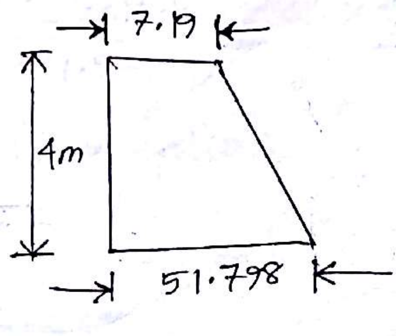
$$P_a = K_{a_2}\gamma_2 z_2 - 2c_2\sqrt{K_{a_2}} + q\,K_{a_2} \qquad q = \gamma_1 H_1$$

$$= 0.656 \times 17 \times z_2 - 2 \times 15 \times \sqrt{0.656} + \overbrace{16 \times 3}^{} \times 0.656$$

$$\therefore P_a = 11.152 z_2 + 7.19$$

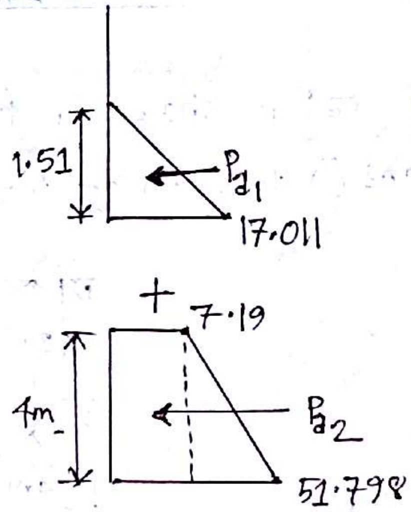
When, $z_2 = 0\,m$, $P_a = 7.19$

$z_2 = 4m$, $P_a = 51.798$



If Tension crack develops:

Total lateral thrust on the wall,



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

$$= \left(\underbrace{\tfrac{1}{2}\times 17.011\times 1.51}_{12.843}\right) + \left[\underbrace{7.19\times 4}_{28.76} + \underbrace{\tfrac{1}{2}\times(51.798-7.19)\times 4}_{89.216}\right]$$

$$\therefore P_{aT} = 130.82 \text{ KN/m}$$

location of the lateral thrust, (from bottom)

$$\bar{z} = \frac{12.843\times\left(4+\frac{1.51}{3}\right) + 28.76\times\frac{4}{2} + 89.216\times\frac{4}{3}}{130.82} = \frac{234.31}{130.82}$$

$$= 1.79 \text{ m}$$

If tension crack does not develop:

Total thrust on the wall, $P_{aT} = 130.82 - \underbrace{\tfrac{1}{2}\times 16.781\times 1.49}_{12.50} = 118.32 \text{ KN/m}$

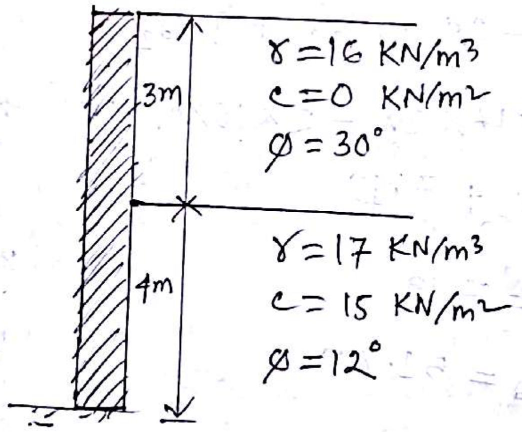
location of the thrust from bottom,

$$\bar{z} = \frac{234.31 - 12.50\times\left(5.51+\frac{2}{3}\times 1.49\right)}{118.32} = 1.293 \text{ m}$$

(Ans.)

\# A retaining wall is shown in figure below. Determine the design lateral force.

(শুধু if tension crack develops (ধরে করলেই হবে)



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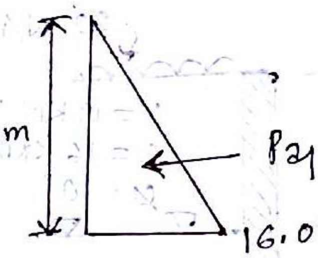
Solution:

For upper layer, $Ka_1 = \dfrac{1-\sin 30}{1+\sin 30} = \dfrac{1}{3}$

$P_a = Ka_1 \gamma_1 z_1$

$= \frac{1}{3} \times 16 \times z_1$

$\therefore P_a = \frac{16}{3} z_1$



When, $z = 0$ m, $P_a = 0$

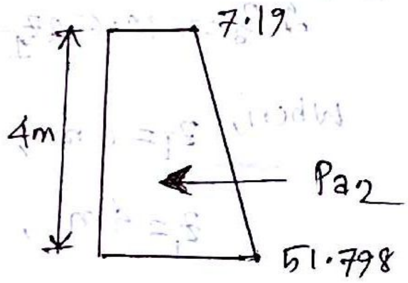
$z = 3$ m, $P_a = 16$ KN/m^2

For layer 2, $Ka_2 = \dfrac{1-\sin 12}{1+\sin 12} = 0.656$

$P_a = Ka_2 \gamma_2 z_2 - 2c_2\sqrt{Ka_2} + q\,Ka_2$

$= 0.656 \times 17 \times z_2 - 2 \times 15 \times \sqrt{0.656} + 16 \times 3 \times 0.656$

$\therefore P_a = 11.152 z_2 + 7.19$



When, $z = 0$ m, $P_a = 7.19$ KN/m^2

$z = 4$ m, $P_a = 51.798$ KN/m^2

Total lateral thrust on the wall,

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

$= \frac{1}{2} \times 16 \times 3 + 7.19 \times 4 + \frac{1}{2} \times (51.798 - 7.19) \times 4$

$\therefore P_{aT} = (24 + 28.76 + 89.216) = 141.976$ KN/m.

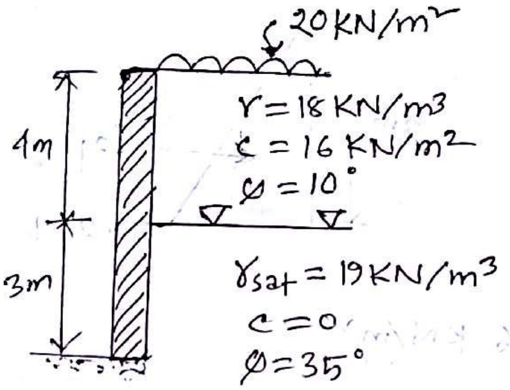
location of the lateral thrust, $\bar{z} = \dfrac{24 \times (4 + \frac{3}{3}) + 28.76 \times \frac{4}{2} + 89.216 \times \frac{4}{3}}{141.976}$

$= 2.09$ m (from bottom)

(Ans.)

2017

\# A retaining wall is shown in figure below. compute the active force on the wall. Also compute the location of resultant force.



<u>Solution:</u>

<u>For upper layer</u>, $Ka_1 = \frac{1-\sin 10°}{1+\sin 10} = 0.704$

$$P_a = qKa_1 + Ka_1 \gamma_1 Z_1 - 2c_1\sqrt{Ka_1}$$

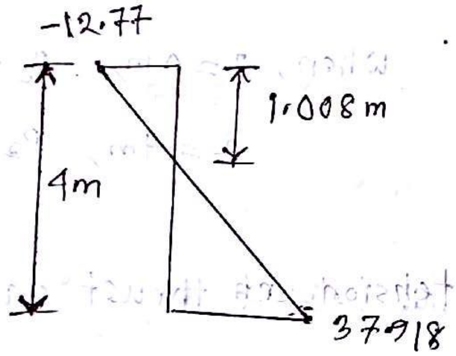
$$= 20\times0.704 + 0.704\times18\times Z_1 - 2\times16\times\sqrt{0.704}$$

$\therefore P_a = 12.672Z_1 - 12.77$

When, $Z_1 = 0\,m$, $P_a = -12.77\ KN/m^2$

$Z_1 = 4m$, $P_a = 37.918\ KN/m^2$

and, $P_a = 0$, $Z_1 = 1.008\ m$



<u>for layer 2</u>, $Ka_2 = \frac{1-\sin 35}{1+\sin 35} = 0.271$, $\gamma' = (19-9.81) = 9.19\ KN/$

$$P_a = Ka_2\gamma' Z_2 + \gamma_W Z_2 + \underbrace{qKa_2 + \gamma_1 Z_1 Ka_2}_{\text{surcharge from upper layer}}$$

$$= 0.271\times9.19\times Z_2 + 9.81\times Z_2$$

$$+ 20\times0.271 + 18\times4\times0.271$$

$\therefore P_a = 12.3Z_2 + 24.932$

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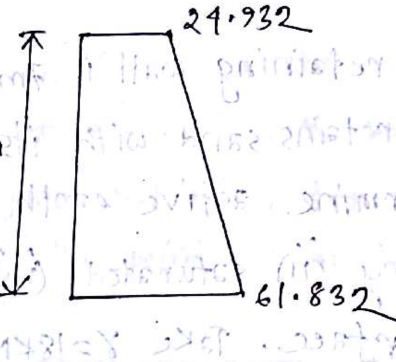
When, $z_2 = 0$ m, $P_a = 24.932$

$z_2 = 3$m, $P_a = 61.832$



if tension crack develops:

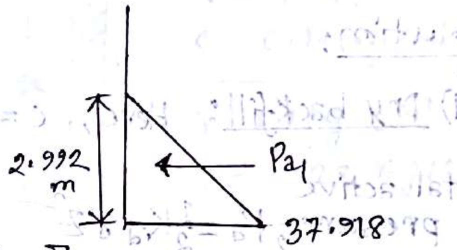
The total active force on the wall,

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

$$= \left(\tfrac{1}{2} \times 37.918 \times 2.992\right) + \left[24.932 \times 3 + \tfrac{1}{2} \times (61.832 - 24.932) \times 3\right]$$

$$= 56.725 + 74.796 + 55.35$$

$$\therefore P_{aT} = 186.871 \text{ KN/m}$$



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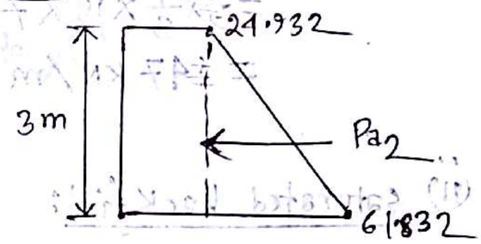


location of the resultant force, (from bottom)

$$\bar{z} = \frac{56.725 \times \left(3 + \frac{2.992}{3}\right) + 74.796 \times \frac{3}{2} + 55.35 \times \frac{3}{3}}{186.871}$$

$$\therefore \bar{z} = \frac{394.293}{186.871} = 2.11 \text{ m}$$

if tension crack does not develop:

The total active force on the wall,

$$P_{aT} = 186.871 - \left(\underbrace{\tfrac{1}{2} \times 12.77 \times 1.008}_{6.43608}\right) = 180.435 \text{ KN/m}$$

and, $$\bar{z} = \frac{394.293 - 6.43608 \times \left(3 + 2.992 + \frac{1.008 \times 2}{3}\right)}{180.435}$$

$$\therefore \bar{z} = 1.9475 \text{ m}$$

(Ans.)

2017

A retaining wall is 7m high with its back face smooth and vertical. It retains sand with its surface-horizontal. Using Rankine's theory, determine active earth pressure at the base when the backfill is (i) dry (ii) saturated (iii) submerged with ~~with~~ water table at the surface. Take $\gamma = 18 KN/m^3$, $\phi = 30°$ and $\gamma_{sat} = 21 KN/m^3$.

Solution:

(i) Dry backfill: Here, $c = 0$, $\gamma = 18 KN/m^3$, and $\phi = 30°$

$\therefore K_a = \frac{1 - \sin 30}{1 + \sin 30} = \frac{1}{3}$

$\therefore$ Total active pressure, $P_a = \frac{1}{2} K_a \gamma z^2$

$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 7^2$

$= 147 KN/m$

(ii) Saturated backfill: $\gamma_{sat} = 21 KN/m^3$

$\therefore \gamma' = (21 - 9.81) = 11.19 KN/m^3$

$\therefore$ Total active pressure, $P_a = \frac{1}{2} K_a \gamma' z^2 + \frac{1}{2} \gamma_w z^2$

$= \frac{1}{2} \times \frac{1}{3} \times 11.19 \times 7^2 + \frac{1}{2} \times 9.81 \times 7^2$

$= 331.73 KN/m$

(iii) if the backfill is submerged with water table at the surface the sand will be saturated.

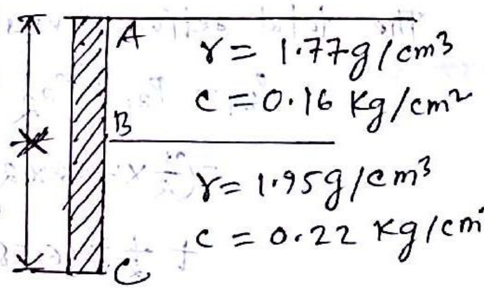
Hence, the total active pressure, $P_a = 331.73 KN/m$.

2016, 2013

A retaining wall of 6 m height is shown in figure below.

(i) Determine total active thrust on the wall if tension crack develop

(ii) Draw the pressure distribution diagram before and after tension crack develops. and (iii) Find the location of the point of application of the resultant lateral thrust



Solution: Here, $\phi = 0$

$$\therefore K_a = \frac{1-\sin 0}{1+\sin 0} = 1$$

For Upper layer, AB: $\gamma_1 = 1.77 g/cm^3 = 1.77 \text{ ton}/m^3$ (1 ton = 1000 Kg)

$c_1 = 0.16 Kg/cm^2 = 1.6 \text{ ton}/m^2$

$$P_a = K_a \gamma_1 z_1 - 2c_1\sqrt{K_a}$$

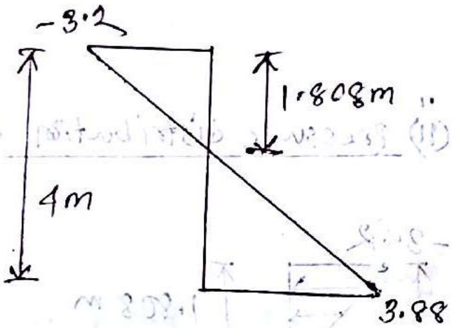
$$= 1 \times 1.77 \times z_1 - 2 \times 1.6 \times \sqrt{1}$$

$$\therefore P_a = 1.77 z_1 - 3.2$$

When, $z_1 = 0$ m, $P_a = -3.2 \text{ ton}/m^2$

$z_1 = 4$ m, $P_a = 3.88 \text{ ton}/m^2$

and, $P_a = 0$, $z_1 = 1.808$ m



For bottom layer, BC: $\gamma_2 = 1.95 g/cm^3 = 1.95 \text{ ton}/m^3$

$c_2 = 0.22 Kg/cm^2 = 2.2 \text{ ton}/m^2$

$$P_a = K_a \gamma_2 z_2 - 2c_2\sqrt{K_a} + \gamma_1 z_1 K_a$$

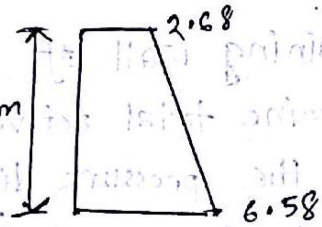
↑ surcharge from upper layer

$$= 1 \times 1.95 \times z_2 - 2 \times 2.2 \times \sqrt{1} + 1.77 \times 4 \times 1$$

$$\therefore P_a = 1.95 z_2 + 2.68$$

$\therefore$ when, $z_2 = 0$ m, $\quad P_a = 2.68$ ton/m^2

$z_2 = 2$ m, $\quad P_a = 6.58$ ton/m^2



(i) if tension crack develops,

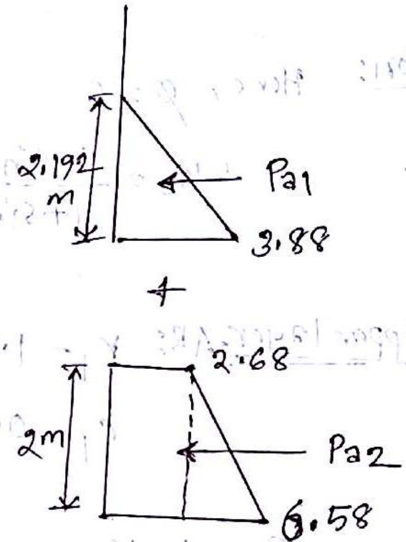
The total active thrust on the wall,

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

$= \left(\frac{1}{2} \times 3.88 \times 2.192\right) + \left[(2.68 \times 2) + \frac{1}{2} \times (6.58 - 2.68) \times 2\right]$

$= 4.25248 + 5.36 + 3.9$

$\therefore P_{aT} = 13.51248$ ton/m



(iii) location of the thrust from bottom,

$$\bar{z} = \frac{4.25248 \times \left(2 + \frac{2.192}{3}\right) + 5.36 \times \frac{2}{2} + 3.9 \times \frac{2}{3}}{13.51248} = 1.525 \text{ m}$$

(ii) Pressure distribution diagram:

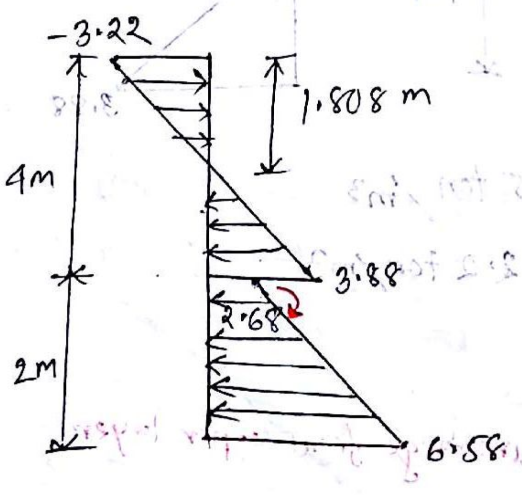


Fig.(a): before crack

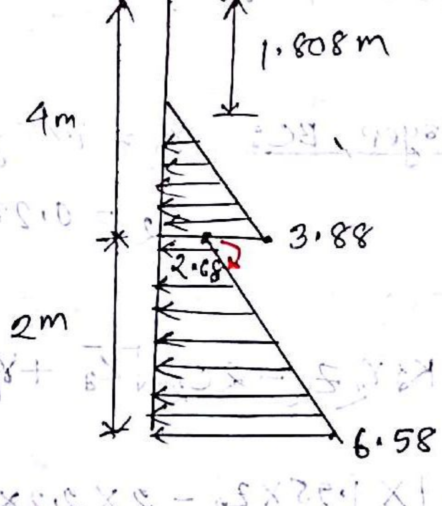


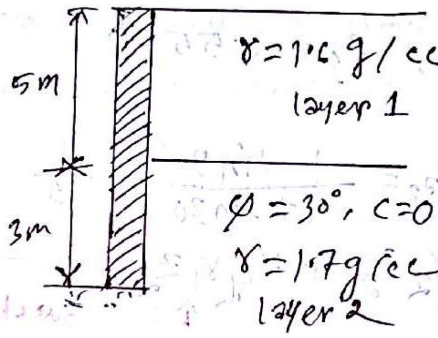
Fig.(b) after crack

2016

The consolidated drained tests on the soil of 1st layer in figure below yields the following data:

σ_3 (N/mm²)	σ_1 (N/mm²)
0.2	0.46
0.4	0.88

calculate the active earth force and its location after the formation of tension cracks.



Solution:

For layer 1:

we know,

$$\boxed{\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha}$$

$\therefore$ $0.46 = 0.2\tan^2\alpha + 2c\tan\alpha$ ..... (I)

and, $0.88 = 0.4\tan^2\alpha + 2c\tan\alpha$ ..... (II)

From (I) & (II) we obtain,

$$\tan^2\alpha = 2.1$$

and, from (I), $0.46 = .2\times 2.1 + 2c\times\sqrt{2.1}$

$\therefore c = 0.0138\ N/mm^2$

$= 13.8\ KN/m^2$

Given, $\gamma_1 = 1.6\ g/cc$

$= 1.6\ ton/m^3$

$= (1.6\times 9.81)\ KN/m^3 = 15.696\ KN/m^3$

we know, $K_a = \cot^2\alpha$ $\quad\therefore K_{a1} = \frac{1}{2.1} = 0.476$

and, $c_1 = 13.8\ KN/m^2$

Now, $P_a = K_{a_1} \gamma_1 z_1 - 2c_1 \sqrt{K_a}$

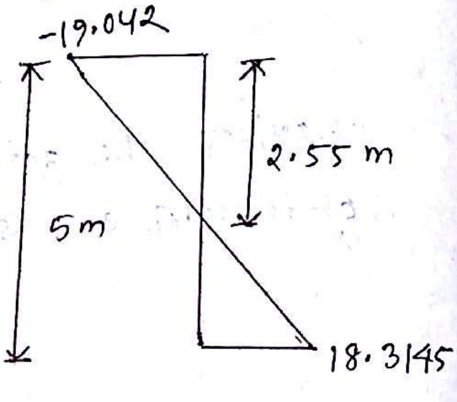
$= 0.476 \times 15.696 \times z_1 - 2 \times 13.8 \times \sqrt{0.476}$

$\therefore P_a = 7.4713\, z_1 - 19.042$

When, $z_1 = 0\,m$, $P_a = -19.042\ KN/m^2$

$z_1 = 5\,m$, $P_a = 18.3145\ KN/m^2$

and, $P_a = 0$, $z_1 = 2.55\,m$



For layer 2: $K_{a_2} = \frac{1 - \sin 30^\circ}{1 + \sin 30} = \frac{1}{3}$, $\gamma_2 = 1.7\ g/cc = 1.7\ ton/m^3$

$= (1.7 \times 9.81) = 16.677\ KN/m^3$

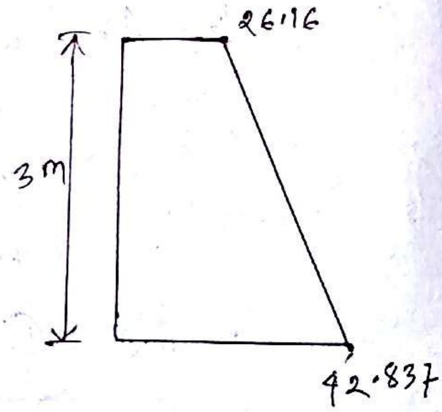
$\therefore P_a = K_{a_2} \gamma_2 z_2 + K_{a_2} \gamma_1 z_1$ ← surcharge from upper layer

$\Rightarrow P_a = \frac{1}{3} \times 16.677 \times z_2 + \frac{1}{3} \times 15.696 \times 5$

$\Rightarrow P_a = 5.559\, z_2 + 26.16$

When, $z_2 = 0\,m$, $P_a = 26.16\ KN/m^2$

$z_2 = 3\,m$, $P_a = 42.837\ KN/m^2$



if tension crack develops,

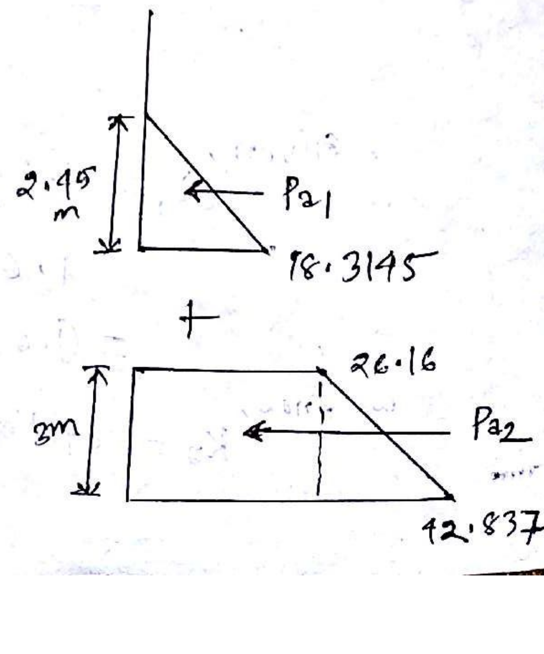
The total active earth pressure,

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

$= \left(\frac{1}{2} \times 18.3145 \times 2.45\right) + \left[(26.16 \times 3) + \frac{1}{2} \times (42.837 - 26.16) \times 3\right]$

$= 22.435 + 78.48 + 25.0155$

$= 125.9305\ KN/m$



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location of the pressure from bottom,

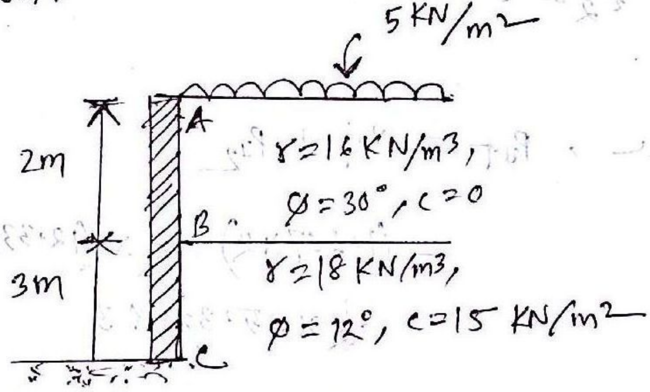
$$\bar{z} = \frac{22.435 \times \left(3 + \frac{2.45}{3}\right) + 78.48 \times \frac{3}{2} + 25.0155 \times \frac{3}{3}}{125.9305}$$

$= 1.8134$ m

(Ans.)

2015

A retaining wall with a stratified backfill and surcharge is shown in figure below. Draw the earth pressure diagram dealing the values at the critical points. Also estimate the resultant thrust on the wall and its position.



Solution:

For upper layer, AB: $K_{a_1} = \frac{1 - \sin 30}{1 + \sin 30} = \frac{1}{3}$

$P_a = q K_{a_1} + K_{a_1} \gamma_1 z_1$

$= 5 \times \frac{1}{3} + \frac{1}{3} \times 16 \times z_1$

$\therefore P_a = 1.667 + 5.333 z_1$

when, $z_1 = 0$ m, $P_a = 1.667$ KN/m²

$z_1 = 2$ m, $P_a = 12.333$ KN/m²

For bottom layer, BC: $Ka_2 = \dfrac{1-\sin 12}{1+\sin 12} = 0.656$

$$P_a = Ka_2 \gamma_2 z_2 - 2c_2\sqrt{Ka_2} + qKa_2 + \underbrace{Ka_2 \gamma_1 z_1}_{\text{surcharge from upper layer}}$$

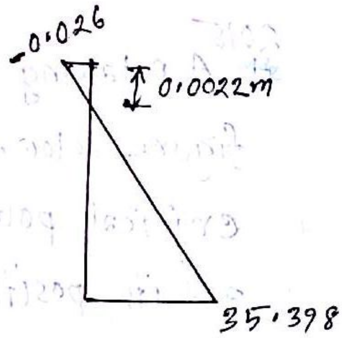
$$= 0.656 \times 18 \times z_2 - 2 \times 15 \times \sqrt{0.656} + (5 \times 0.656) + (0.656 \times 16 \times 2)$$

$$\therefore P_a = 11.808\, z_2 - 0.026$$

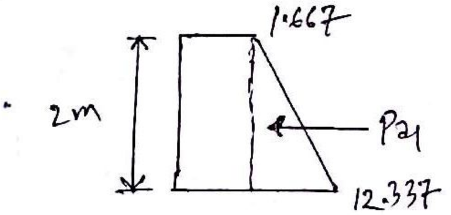
when, $z_2 = 0\,m$, $P_a = -0.026$ KN/m²

$z_2 = 3\,m$, $P_a = 35.398$ KN/m²

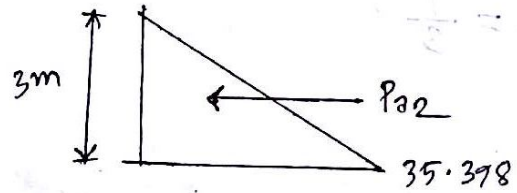
when, $P_a = 0$, $z_2 = 0.0022\,m \simeq 0\,m$



$\therefore$ The resultant force, $Pa_T = Pa_1 + Pa_2$



+



$$= (1.667 \times 2) + \tfrac{1}{2} \times (12.337 - 1.667) \times 2 + \tfrac{1}{2} \times 35.398 \times 3$$

$$= 3.334 + 10.67 + 53.097$$

$$\therefore Pa_T = 67.101$$

and,
location of the thrust,

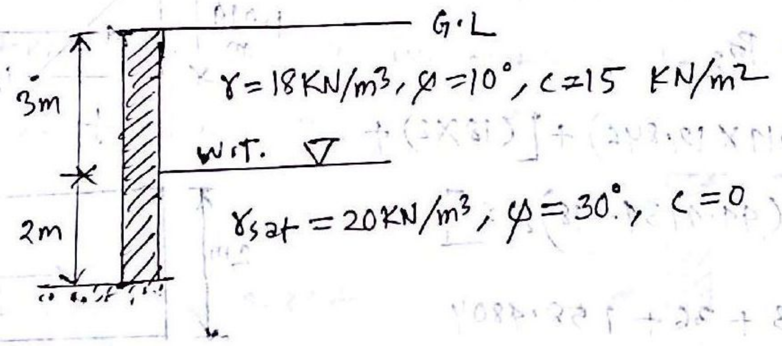
$$\bar{z} = \frac{3.334 \times (3 + \frac{2}{2}) + 10.67 \times (3 + \frac{2}{3}) + 53.097 \times (\frac{3}{3})}{67.101}$$

$$= 1.5731\ m$$

(Ans.)

2015

Calculate the total active thrust and its location from ground level following below.



Solution:

For top layer: $Ka_1 = \frac{1-\sin 10°}{1+\sin 10°} = 0.704$

$P_a = Ka_1 \gamma_1 z_1 - 2c_1\sqrt{Ka_1}$

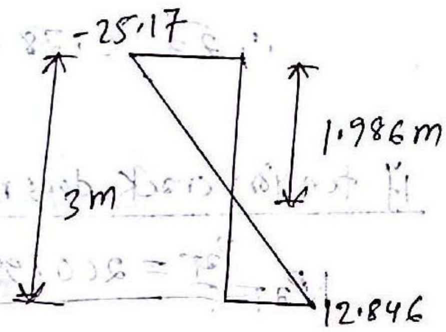
$= 0.704 \times 18 \times z_1 - 2 \times 15 \times \sqrt{0.704}$

$= 12.672 z_1 - 25.17$

When, $z_1 = 0$ m, $P_a = -25.17$ KN/m²

$z_2 = 3$ m, $P_a = 12.846$ KN/m²

and, $P_a = 0$, $z_1 = 1.986$ m



For bottom layer: $Ka_2 = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$, $\gamma' = (20-9.81) = 10.19$ KN/m³

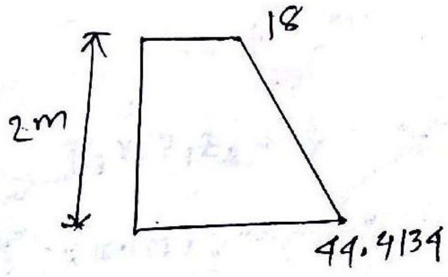
$P_a = Ka_2 \gamma_2 z_2 + \gamma_w z_2 + Ka_2 \gamma_1 z_1$

$= \frac{1}{3} \times 10.19 \times z_2 + 9.81 \times z_2 + \frac{1}{3} \times 18 \times 3$

$\therefore P_a = 13.2067 z_2 + 18$

When, $z_2 = 0$m, $P_a = 18$ KN/m²

$z_2 = 2$m, $P_a = 44.4134$ KN/m²



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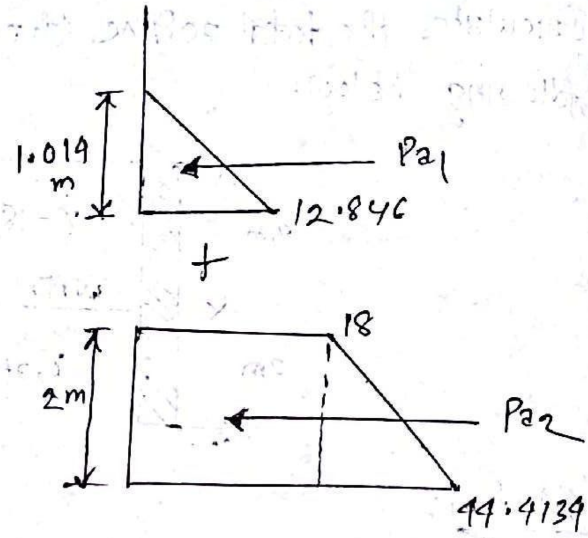
if tension crack develops,

The total active thrust,



$$P_{aT} = P_{a1} + P_{a2}$$

$$= \left(\tfrac{1}{2} \times 1.014 \times 12.846\right) + \left[(18 \times 2) + \tfrac{1}{2} \times (44.4134 - 18) \times 2\right]$$

$$= 6.513 + 36 + 158.4804$$

$$P_{aT} = 200.9934 \text{ KN/m}$$

location of the active thrust,

$$\bar{z} = \frac{6.513 \times \left(2 + \frac{1.014}{3}\right) + 36 \times \left(\frac{2}{2}\right) + 158.4804 \times \left(\frac{2}{3}\right)}{200.9934} = \frac{156.881}{200.9934}$$

$$\therefore \bar{z} = 0.78 \text{ m (from bottom)}$$

if tension crack does not develop,

$$P_{aT} = 200.9934 - \tfrac{1}{2} \times 1.986 \times 25.17 = 176 \text{ KN/m}$$

and, location, $\bar{z} = \dfrac{156.881 - \left(\frac{1}{2} \times 1.986 \times 25.17\right) \times \left(2 + 1.1014 + \frac{2}{3} \times 1.986\right)}{176}$

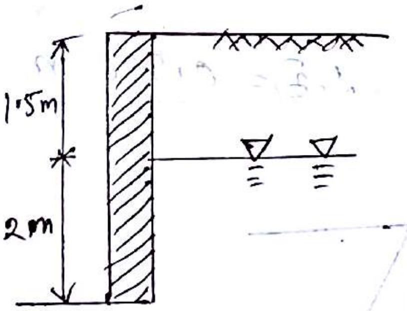
$$= 0.263 \text{ m from bottom.}$$

(Ans.)

2014, 2012

A cohesionless soil with a retaining wall is shown in figure below. Determine (i) Total vertical stress (ii) Effective vertical stress (iii) pore pressure (iv) Horizontal pressure (v) location of horizontal pressure.

Given, $G_s = 2.66$, $e = 0.6$, and $\phi = 32°$



Solution:

$$K_a = \frac{1-\sin 32°}{1+\sin 32°} = 0.307$$

$$\gamma_d = \frac{G_s \cdot \gamma_w}{1+e} = \frac{2.66 \times 9.81}{1+0.6} = 16.31 \text{ KN/m}^3$$

$$\gamma_{sat} = \frac{(G_s+e)\gamma_w}{1+e} = \frac{(2.66+0.6) \times 9.81}{1+0.6} = 19.988 \text{ KN/m}^3$$

$$\therefore \gamma' = (19.988 - 9.81) = 10.178 \text{ KN/m}^3$$

(i) Total vertical stress, $\sigma_v = \gamma_d H_1 + \gamma_{sat} H_2$

$$= (16.31 \times 1.5) + (19.988 \times 2)$$
$$= 64.441 \text{ KN/m}^2$$

(ii) Effective vertical stress, $\sigma_v' = \gamma_d . H_1 + \gamma' . H_2$

$$\Rightarrow \sigma_v' = (16.31 \times 1.5) + (10.178 \times 2) = 44.821 \text{ KN/m}^2$$

(iii) pore pressure, $\sigma_u = \gamma_w H_2$

$$= (9.81 \times 2) = 19.62 \text{ KN/m}^2$$

(iv) Horizontal pressure,

surcharge from upper layer

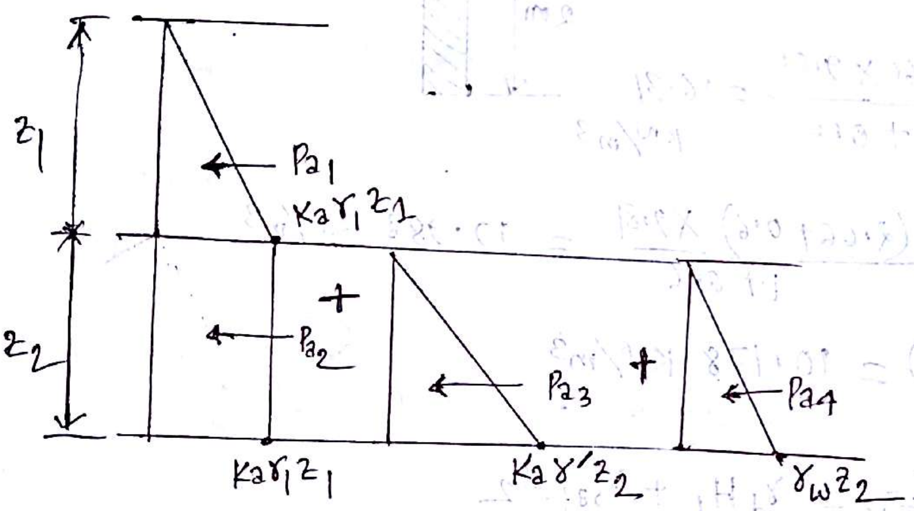
$$P_{aT} = P_{a1} + P_{a2}$$
$$= \frac{1}{2} \times K_a \gamma_1 z_1^2 + \left[\frac{1}{2} K_a \gamma' z_2^2 + K_a \gamma_1 z_1 z_2 + \frac{1}{2} \gamma_w z_2^2\right]$$
$$= \left(\frac{1}{2} \times 0.307 \times 16.31 \times 1.5^2\right) + \left(\frac{1}{2} \times 0.307 \times 10.178 \times 2^2\right) + (0.307 \times 16.31 \times 1.5 \times 2) + \left(\frac{1}{2} \times 9.81 \times 2^2\right)$$

$$P_{aT} = (5.633 + 6.249 + 15.022 + 19.62) = 46.524 \text{ KN/m}$$

Location of the horizontal pressure,

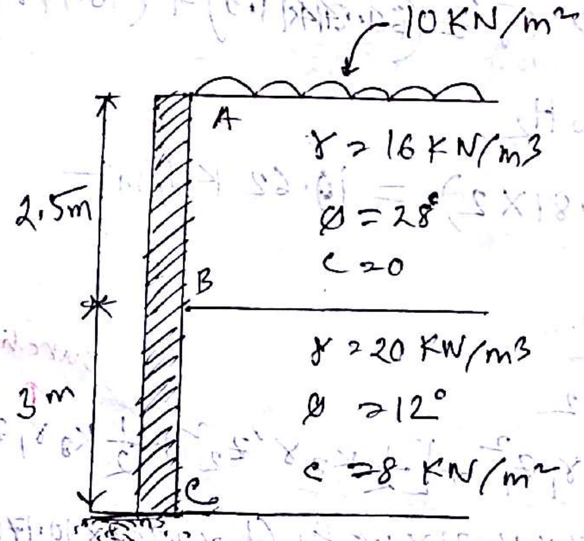
$$\bar{z} = \frac{5.633 \times \left(2 + \frac{1.5}{3}\right) + 6.249 \times \frac{2}{3} + 15.022 \times \frac{2}{2} + 19.62 \times \frac{2}{3}}{46.524}$$

$$\therefore \bar{z} = 0.996 \text{ m} \quad \text{(from bottom)}$$



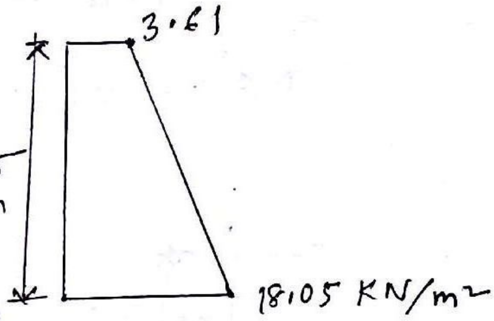
2019

A retaining wall with stratified backfill and a surcharge load is shown in figure below. (i) Draw earth pressure diagram detailing contact point (ii) Determine the resultant thrust and its location.



Solution: For layer 1:

$$Ka_1 = \frac{1 - \sin 28^\circ}{1 + \sin 28^\circ} = 0.361$$



$P_a = qKa_1 + Ka_1 \gamma_1 z_1$

$= 10 \times 0.361 + 0.361 \times 16 z_1$

$= 3.61 + 5.776 z_1$

When, $z = 0$ m, $P_a = 3.61$ KN/m²

$z = 2.5$ m, $P_a = 18.05$ KN/m²

For layer 2:

$$Ka_2 = \frac{1 - \sin 12^\circ}{1 + \sin 12^\circ} = 0.656$$

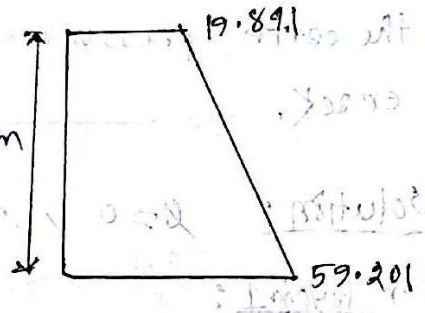
$$P_a = Ka_2 \gamma_2 z_2 - 2c_2\sqrt{Ka_2} + qKa_2 + \underbrace{Ka_2 \gamma_1 z_1}_{\text{surcharge from upper layer}}$$

$= 0.656 \times 20 \times z_2 - 2 \times 8 \times \sqrt{0.656} + 10 \times 0.656 + 0.656 \times 16 \times 2.5$

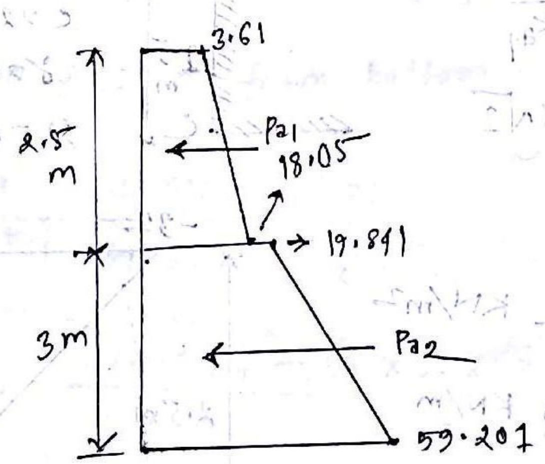
$= 13.12 z_2 + 19.841$

When, $z_2 = 0$ m, $P_a = 19.841$ KN/m²

$z_2 = 3$ m, $P_a = 59.201$ KN/m²



Pressure diagram



The total resultant thrust, $P_a = P_{a_1} + P_{a_2}$

$$= \left[(3.61 \times 2.5) + \frac{1}{2} \times (18.05 - 3.61) \times 2.5\right] + \left[(19.891 \times 3) + \frac{1}{2} \times (59.201 - 19.891) \times 3\right]$$

$$= 9.025 + 18.05 + 59.523 + 59.04$$

$$P_a = 145.638 \text{ KN/m}$$

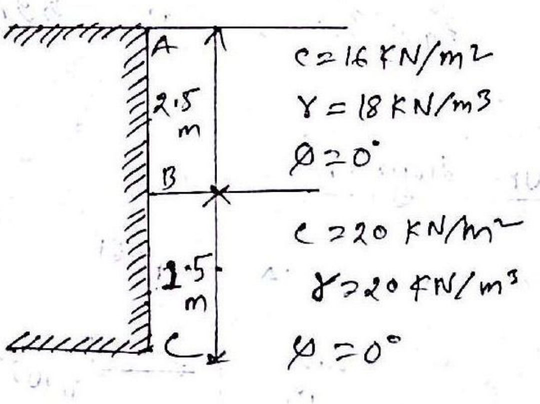
$\therefore$ Location of the thrust,

$$\bar{z} = \frac{9.025 \times (3 + \frac{2.5}{2}) + 18.05 \times (3 + \frac{2.5}{3}) + 59.523 \times \frac{3}{2} + 59.04 \times (\frac{3}{3})}{145.638}$$

$\therefore \bar{z} = 1.757$ m (from bottom)

2011

For an earth retaining structure shown in figure below. Determine the total active earth pressure on the wall. Also draw the earth pressure diagram before and after the formation of crack.



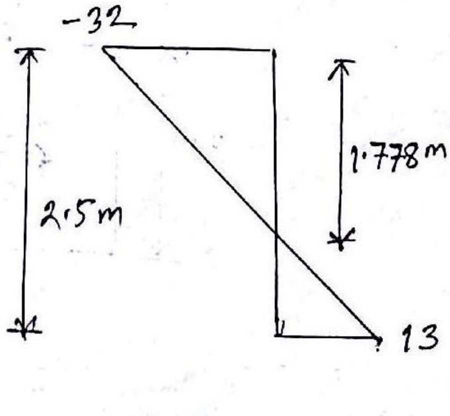
Solution: $\phi = 0$, $\therefore K_{a_1} = K_{a_2} = 1$

For layer 1:

$$P_a = K_{a_1} \gamma_1 z_1 - 2 c_1 \sqrt{K_{a_1}}$$

$$= 1 \times 18 \times z_1 - 2 \times 16 \times \sqrt{1}$$

$$= 18 z_1 - 32$$



when, $z_1 = 0$, $P_a = -32$ KN/m²

$z_1 = 2.5$ m, $P_a = 13$ KN/m

and, $P_a = 0$, $z_1 = 1.778$ m

For layer 2:

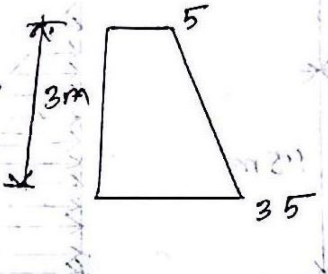
$$P_a = K_{a_2}\gamma_2 z_2 - 2c_2\sqrt{K_{a_2}} + K_{a_2}\gamma_1 z_1$$

$$= 1\times 20\times z_2 - 2\times 20\times\sqrt{1} + 1\times 18\times 2.5$$

$$= 20z_2 + 5$$

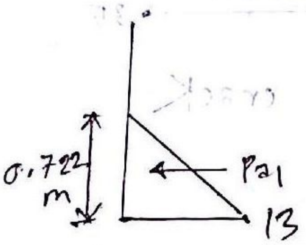
when, $z_2 = 0$ m, $P_a = 5$ KN/m2

$z_2 = 1.5$ m, $P_a = 35$ KN/m2



if tension cracks develop:

The total active pressure, $P_{aT} = P_{a_1} + P_{a_2}$

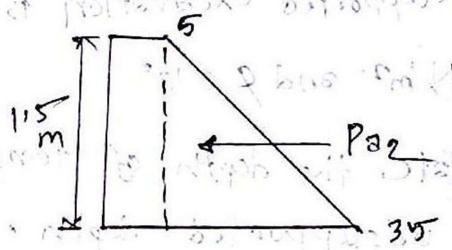


$$= \left(\frac{1}{2}\times 13\times 0.722\right) + \left[5\times 1.5 + \frac{1}{2}\times(35-5)\times 1.5\right]$$

$$= 4.693 + 7.5 + 22.5$$

$$= 34.693 \text{ KN/m}$$

+



location of the pressure,

$$\bar{z} = \frac{4.693\times\left(1.5+\frac{.722}{3}\right) + 7.5\times\frac{1.5}{2} + 22.5\times\frac{1.5}{3}}{34.693} = \frac{25.044}{34.693}$$

$$\therefore \bar{z} = 0.722 \text{ m from bottom}$$

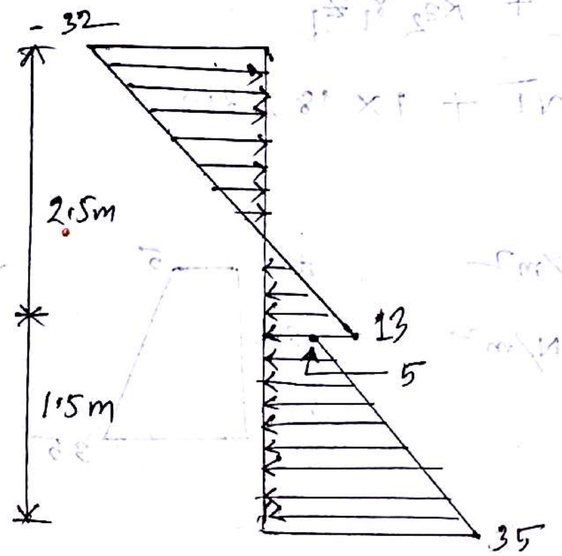
if tension cracks do not develop:

$$P_{aT} = 34.693 - \left(\frac{1}{2}\times 32\times 1.778\right) = 6.245 \text{ KN/m}$$

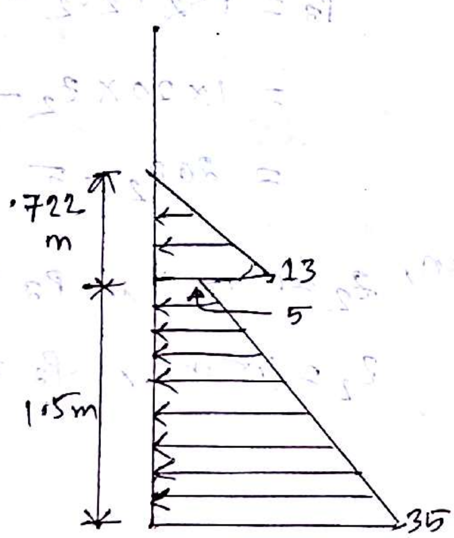
and, $$\bar{z} = \frac{25.044 - \left(\frac{1}{2}\times 32\times 1.778\right)\times\left(1.5+0.722+\frac{1.778\times 2}{3}\right)}{6.245}$$

$$= -11.51 \text{ m from bottom}$$

pressure diagram 1



(a) before crack



(b) after crack

2011

An unsupported excavation is to be made in clay layer. If $\gamma = 19\,KN/m^3$, $c = 35\,KN/m^2$ and $\phi = 12°$

(i) Calculate the depth of tension crack (ii) Calculate the maximum possible unsupported depth. (iii) Draw pressure diagram before and after tension crack developed.

Solution: $\phi = 12°$ $\therefore K_a = \frac{1-\sin 12}{1+\sin 12} = 0.656$

$P_a = K_a \gamma z - 2c\sqrt{K_a} = 0.656 \times 19 \times z - 2 \times 35 \times \sqrt{0.656}$

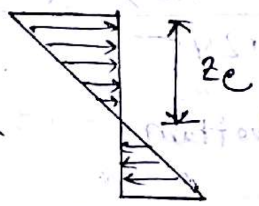
$\therefore P_a = 12.464z - 56.696$

(i) When, $P_a = 0$, $z_c = \frac{56.696}{12.464} = 4.55\,m$ (depth of tension crack)

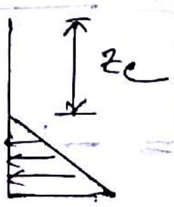
(ii) The maximum possible unsupported depth, $2z_c = (2 \times 4.55) = 9.1m$

(iii) pressure diagram:

(a) before crack



(b) after crack



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Class Test

A soil has the following properties: $\gamma = 18 KN/m^3$, $\phi = 20°$, $c = 10 KN/m^2$. Calculate the critical depth of a vertical excavation that can be made in the soil without any lateral support. Also calculate the value of surcharge for which the soil is critically stable without any support.

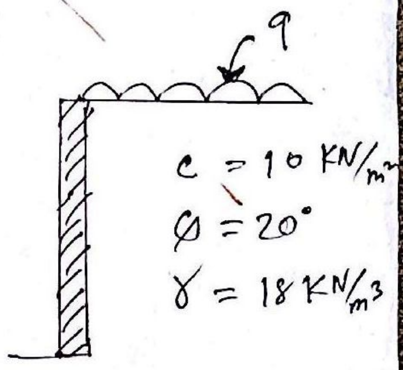
<u>Solution:</u> we know, $K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 20}{1+\sin 20} = 0.49$

the critical depth of unsupported vertical cut,

$$H_c = 2Z_c = 2\times \frac{2c}{\gamma\sqrt{K_a}} = 2\times\frac{2\times 10}{18\times\sqrt{0.49}} = 3.17$$

Now, if the back fill carries a back fill,

$$P_a = K_a\gamma.Z - 2c\sqrt{K_a} + q K_a$$



$Z = 0$, $P_a = 0$. $\therefore q = \frac{2c}{\sqrt{K_a}}$

$$= \frac{2\times 10}{\sqrt{0.49}}$$

$$\therefore q = 28.57 \ KN/m^2$$

This the value of surcharge for which the soil is critically stable without any support.

[Do practice problem from <u>B.C. Punmia</u>]

Example 20.20 — Example 20.51

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Stress Distribution

Formula:

* increased vertical stress due to $\boxed{\text{Concentrated}}$ loading,

$$\boxed{\sigma_z = \frac{Q}{z^2} \times K_B} \quad \text{where}, \quad K_B = \frac{3}{2\pi} \times \left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$$

* increased vertical stress under uniformly loaded $\boxed{\text{circular}}$ area,

$$\boxed{\sigma_z = q K_B} \quad \text{where}, K_B = \left[1 - \left\{\frac{1}{1+\left(\frac{R}{z}\right)^2}\right\}\right]^{3/2}$$

* vertical stress increase under a $\boxed{\text{line load,}}$

$$\sigma_z = \frac{q'}{z} \times K_B \quad \text{where} \quad K_B = \frac{2}{\pi}\left[\frac{1}{1+\left(\frac{x}{z}\right)^2}\right]^2$$

* vertical stress increase at a corner of a uniformly distributed $\boxed{\text{Rectangular}}$ Area,

$$\sigma_z = q K_N \quad \text{where}, \quad K_N = \frac{1}{2\pi} \times \left[\frac{mn}{\sqrt{f}} \times \frac{f+1}{f+m^2n^2} + \sin^{-1}\frac{mn}{f+m^2n^2}\right]$$

(Newmark's Solution) or,

$$K_N = \frac{1}{4\pi} \times \left[\frac{2mn\sqrt{f}}{f+m^2n^2} \times \frac{f+1}{f} + \tan^{-1}\frac{2mn\sqrt{f}}{f-m^2n^2}\right]$$

where, $m = \frac{B}{z}$; $n = \frac{L}{z}$ and $f = m^2+n^2+1$

* vertical stress increase under a $\boxed{\text{strip load}}$,

$$\sigma_z = q K_B \quad \text{where} \quad K_B = \frac{1}{\pi} \times \left[2\theta + \sin 2\theta\right]$$

* vertical stress due to embankment loading,

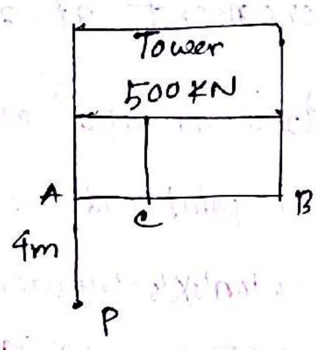
$$\Delta\sigma_z = \frac{q_0}{\pi}\left[\left(\frac{B_1+B_2}{B_2}\right)(\alpha_1+\alpha_2) - \left(\frac{B_1}{B_2}\right)\alpha_2\right]$$

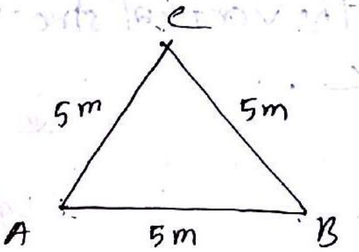
where, $\alpha_1 = \tan^{-1}\left(\frac{B_1+B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right)$ (in radian mode)

$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right)$ (in radian mode)

Stress distribution

#





compute the increase in the vertical stress at point "P".

Solution: We know, $\sigma_z = \frac{Q}{z^2} \times K_B$,

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

Given,

Total $Q = 500$ KN on three legs.

per leg $Q = \frac{500}{3} = 166.67$ KN

At leg A, $r=0$, $z=4m$; $\therefore K_B = \frac{3}{2\pi} \times \left[\frac{1}{1+\left(\frac{0}{z}\right)^2}\right]^{5/2} = \frac{3}{2\pi}$

$\therefore (\sigma_z)_{pA} = \frac{166.67}{4^2} \times \frac{3}{2\pi} = 4.97$ KN/m^2

At leg B, $r = 5m$, $z = 4m$ $\therefore$ $K_B = 0.0454$

$\therefore (\sigma_z)_{pB} = \frac{166.67}{4^2} \times 0.0454 = 0.473$ KN/m^2

At leg 3, $r = 5m$, $z = 4m$ $\therefore K_B = 0.0454$

$\therefore (\sigma_z)_{pc} = 0.473$ KN/m^2

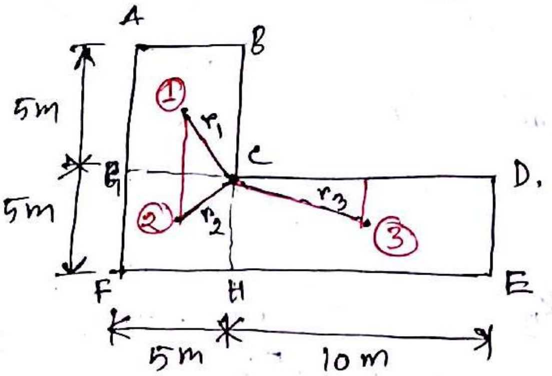
$\therefore$ vertical stress at point P below A,

$\sigma_z = \left[(\sigma_z)_{pA} + (\sigma_z)_{pB} + (\sigma_z)_{pc}\right] = (4.97 + 0.473 + 0.473)$

$= 5.916$ KN/m^2

(Ans)

A L-shaped building in plan exerts a pressure of 75 kN/m^2 on the soil. Determine the vertical stress increment at a depth 5 m below the point C.



* যদি টেবিল না থাকে তাহলে Equivalent point load method অথবা Newmark's Influence chart যেকোনো এক method -এ করতেই হবে। তবে Newmark's Influence chart use করা ভালো

Solution: using equivalent point load method,

$$\sigma_z = \frac{1}{z^2}\left(Q_1 K_{B_1} + Q_2 K_{B_2} + Q_3 K_{B_3}\right)$$

Given, $z = 5$ m ~~$Q = 75 \times (10\times5 + 10\times5) = 750$ kN~~

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

Here,

Area	r (m)		$\frac{r}{z}$	K_B	Q (kN)
ABCG	$\sqrt{2.5^2+2.5^2}$	2.5, 2.5, r_1	0.707	0.1733	$75\times(5\times5) = 1875$
CGFH	$\sqrt{2.5^2+2.5^2}$	2.5, 2.5, r_2	0.707	0.1733	$75\times(5\times5) = 1875$
CHED	$\sqrt{5^2+2.5^2}$	5, 2.5	1.118	0.063	$75\times(10\times5) = 3750$

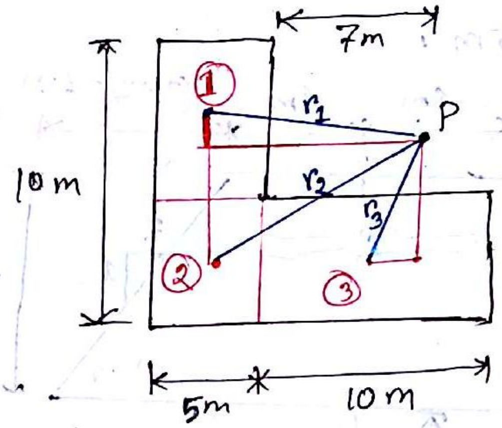
$\therefore$ vertical stress at point C,

$$\sigma_z = \frac{1}{5^2}\times\left(1875\times .1733 + 1875\times 0.1733 + 3750\times .063\right)$$

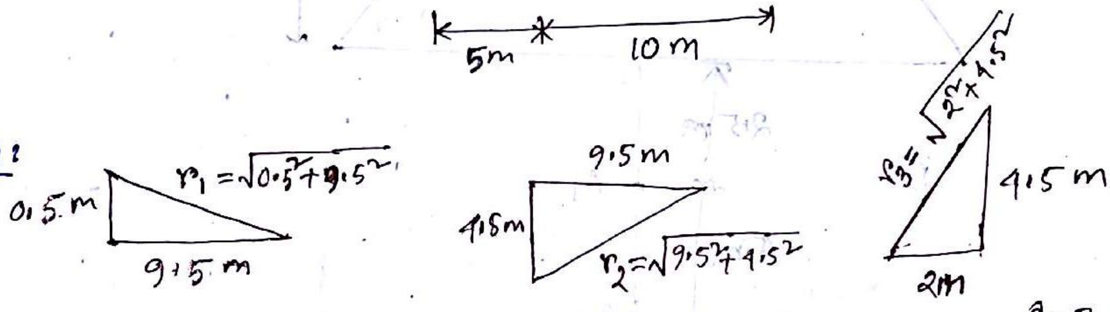
$$= 35.445 \ kN/m^2$$

(Ans)

A L-shapped building in plan exerts a pressure of 80 kN/m^2. Determine the vertical stress increment at a depth 9 m below the point 'P'. (Use equivalent point load method)



Solution:



$Q_1 = 80 \times (5 \times 5)$ KN
$= 2000$ KN

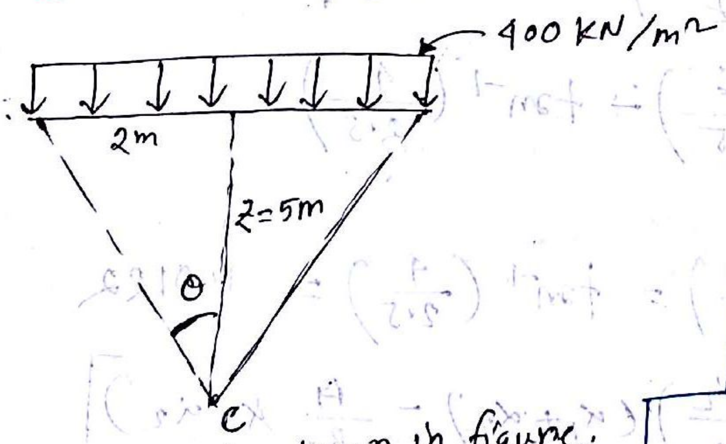
$Q_2 = 80 \times (5 \times 5)$ KN
$= 2000$ KN

$Q_3 = 80 \times (10 \times 5)$
$= 4000$ KN

We know, $\sigma_z = \frac{1}{z^2} \times (Q_1 K_{B_1} + Q_2 K_{B_2} + Q_3 K_{B_3})$

DO yourself (same as previous problem)

#



A strip footing is shown in figure. Determine the vertical stress increment at point e.

Solution: We know,

$\sigma_z = \frac{q}{\pi} \times [2\theta + \sin 2\theta]$

Here, $\theta = \tan^{-1}\left(\frac{2}{5}\right) = 21.8°$

$\therefore \sigma_z = \frac{400}{3.1416} \times \left[2 \times 21.8 \times \frac{\pi}{180} + \sin(2 \times 21.8)\right]$

$= 184.7$ KN/m²

(Ans.)

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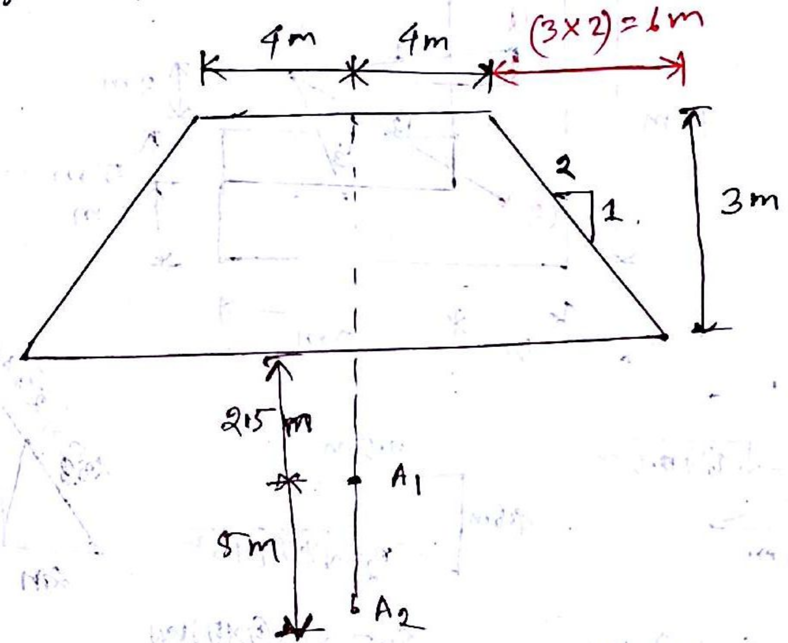
A railway embankment is shown in Figure below. Assume the unit weight of the soil to be 20 KN/m3. Compute the ~~the~~ increase in vertical stress under the center line at depths of 2.5m and 5m.



<u>Solution:</u> Here, $B_1 = 4m$

$B_2 = 6m$

Here, $q_0 = \gamma H = (20 \times 3) = 60\ KN/m^2$

For 2.5m depth.

$$\alpha_1 = \tan^{-1}\left(\frac{B_1+B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right) \quad \text{(radian mode)}$$

$$= \tan^{-1}\left(\frac{4+6}{2.5}\right) - \tan^{-1}\left(\frac{4}{2.5}\right)$$

$$\therefore \alpha_1 = 0.314$$

and $\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right) = \tan^{-1}\left(\frac{4}{2.5}\right) = 1.0122$

we know, $\sigma_z = \frac{q_0}{\pi}\left[\left(\frac{B_1+B_2}{B_2}\right)(\alpha_1+\alpha_2) - \frac{B_1}{B_2} \times (\alpha_2)\right]$

$$= \frac{60}{3.1416} \times \left[\left(\frac{4+6}{6}\right) \times (0.314 + 1.0122) - \frac{4}{6} \times (1.0122)\right]$$

$\sigma_{2.5} = 29.33 \ KN/m^2$

For 5m depth,

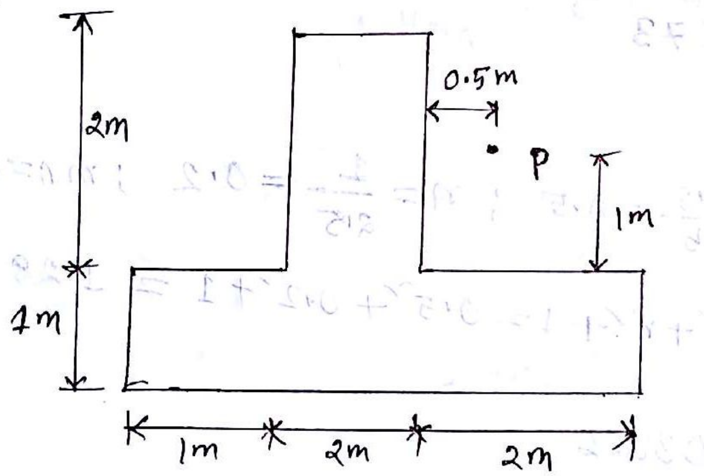
$$\alpha_1 = \tan^{-1}\left(\frac{4+6}{5}\right) - \tan^{-1}\left(\frac{4}{5}\right) = 0.4324 = 0.4324$$

and, $\alpha_2 = \tan^{-1}\left(\frac{4}{5}\right) = 0.67474$

$$\therefore \sigma_5 = \frac{60}{3.1416} \times \left[\left(\frac{4+6}{6}\right) \times (0.4324 + 0.67474) - \frac{4}{6} \times (0.67474)\right]$$

$$= 26.65 \ KN/m^2$$

#



Given that,

$q = 200 \ KN/m^2$

$z = 5m$

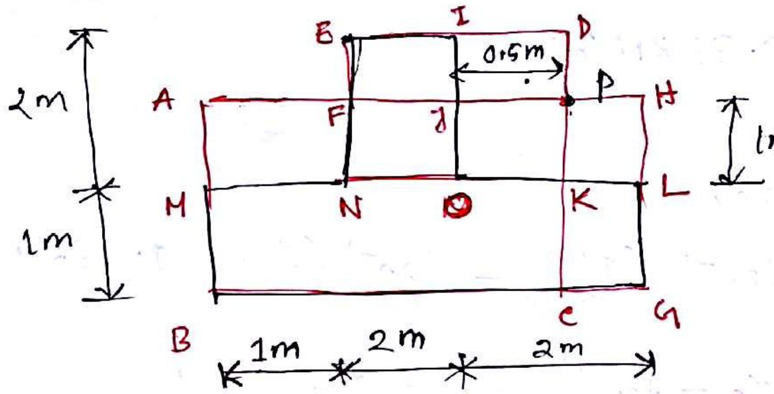
Calculate the vertical stress increment at point P

(i) using Newmark's Solution.

(ii) using equivalent point load method.

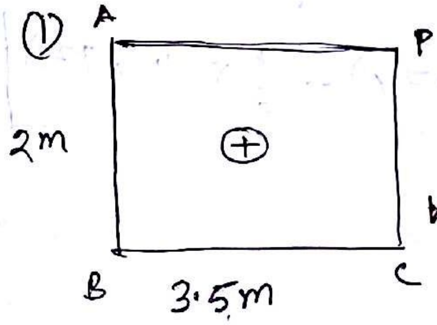
(iii) Also calculate the percentage error in the equivalent point load method compare to newmark's solution.

solution:



using Newmark's solution,

(i)



$m = \frac{B}{z} = \frac{3.5}{5} = 0.7$; $n = \frac{L}{z} = 0.4$

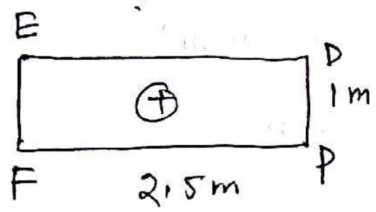
$f = m^2 + n^2 + 1 = 0.7^2 + 0.4^2 + 1 = 1.65$

We know, (KN এর যে কোনো একটি formula- use করলেই হবে)

$$K_N = \frac{1}{4\pi} \times \left[\frac{2mn\sqrt{f}}{f + m^2n^2} \times \frac{f+1}{f} + \tan^{-1}\left(\frac{2mn\sqrt{f}}{f - m^2n^2}\right) \right]$$

$\therefore K_{N_1} = 0.0873$

(ii)

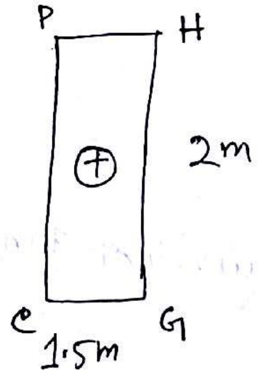


$m = \frac{2.5}{5} = 0.5$; $n = \frac{1}{5} = 0.2$; $mn = 0.1$

$f = m^2 + n^2 + 1 = 0.5^2 + 0.2^2 + 1 = 1.29$

$K_{N_2} = 0.03866$

(iii)



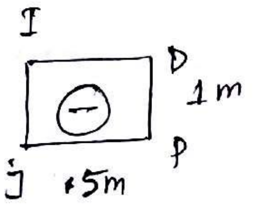
$m = \frac{1.5}{5} = 0.3$; $n = \frac{2}{5} = 0.4$; $mn = 0.12$

$f = .3^2 + .4^2 + 1 = 1.25$

$\therefore 2mn\sqrt{f} = 2 \times .12 \times \sqrt{1.25} = 0.26833$

$\therefore K_{N_3} = 0.0436$

(iv)



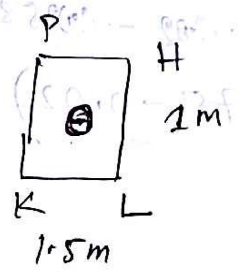
$m = \frac{.5}{5} = 0.1$; $n = \frac{1}{5} = 0.2$; $mn = .02$

$f = m^2 + n^2 + 1 = .1^2 + .2^2 + 1 = 1.05$

$\therefore 2 \times mn \times \sqrt{f} = 0.08944$

$\therefore K_{N_4} = 0.02$

(v)



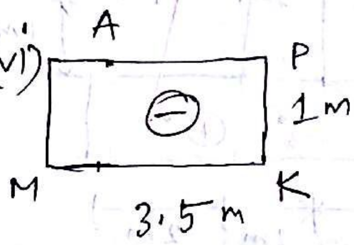
$m = \frac{1.5}{5} = 0.3$; $n = \frac{1}{5} = 0.2$; $mn = 0.06$

$f = .3^2 + .2^2 + 1 = 1.13$;

$2mn\sqrt{f} = 0.12756$

$\therefore K_{N_5} = 0.02585$

(vi)



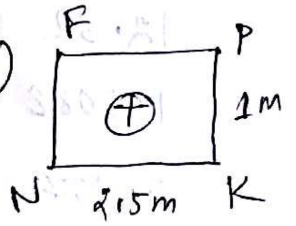
$m = \frac{3.5}{5} = 0.7$; $n = \frac{1}{5} = 0.2$; $mn = 0.14$

$f = .7^2 + .2^2 + 1 = 1.53$

$2mn\sqrt{f} = 0.34634$

$\therefore K_{N_6} = 0.04735$

(vii)



$m = \frac{2.5}{5} = 0.5$; $n = \frac{1}{5} = 0.1$; $mn = 0.05$

$f = .5^2 + .1^2 + 1 = 1.26$;

$2mn\sqrt{f} = 0.11225$

$\therefore K_{N_7} = 0.019775$

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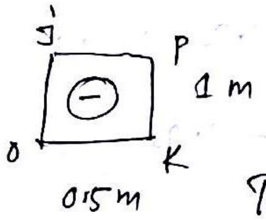
(viii) 

$K_{N8} = 0.02$

Total vertical stress at point P,

$$\therefore \quad \sigma_z = q\,(K_{N1} + K_{N2} + K_{N3} - K_{N4} - K_{N5} - K_{N6} + K_{N7} - K_{N8})$$

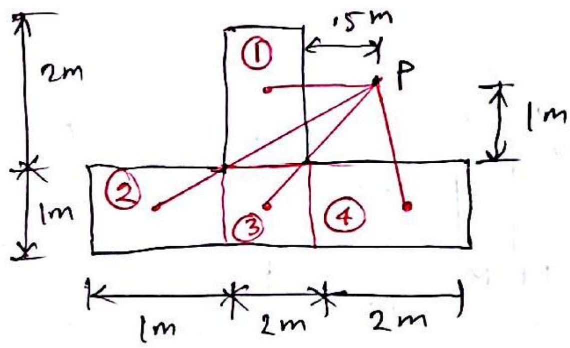
$$= 200 \times (0.0873 + 0.03866 + 0.0436 - 0.02 - 0.02585 - 0.04735 + 0.019775 - 0.02)$$

$$\sigma_z = 15.227 \ KN/m^2$$

Using Equivalent point load method:

We know,
$$\sigma_z = \frac{Q}{z^2} \times K_B$$

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



Area unit	r (m)	$\frac{r}{z}$	K_B	$Q_p = qA$ (KN)	$\sigma_z = \frac{Q}{z^2} \times K_B$ (KN/m²)
1	1.5	0.3	0.385	(200×2×2) = 800	12.32
2	$\sqrt{1.5^2+3^2} = 3.354$	0.6708	0.1886	(200×1×1) = 200	1.5088
3	$\sqrt{1.5^2+1.5^2} = 2.12$	0.424	0.316	(200×2×1) = 400	5.056
4	$\sqrt{.5^2+1.5^2} = 1.58$	0.316	0.3764	(200×2×1) = 400	6.0224

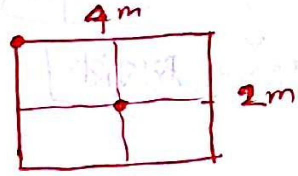
$\Sigma \sigma_z = 24.9072$

% Error $= \frac{24.9072 - 15.227}{15.227} \times 100 = 63.57\%$

Hence, It is better to solve with Newmark's Influence chart.

B.C. Punmia (Example - 13.5)

A rectangular area 2m X 4m carries a uniform load of 80 kN/m^2 at the ground surface. Find the vertical pressures at 5 m below the center and corner of the loaded area. using Newmark's influence chart.

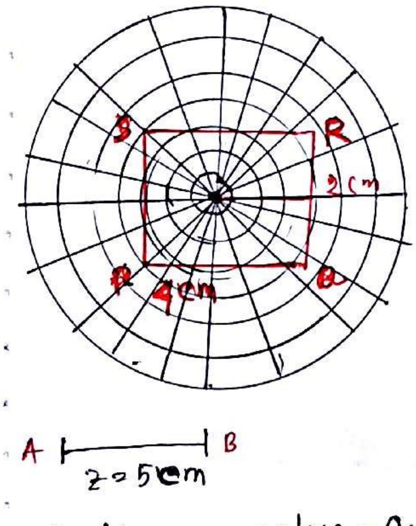


Solution: Given, $z = 5$ m

Hence, the scale of the plan will be $AB = (5\,cm) = 5m$

or, 1cm = 1m

(a) The plan of the rectangular area is drawn to the scale of 1cm = 1m and, oriented on the chart in such way that its centroid is over the center of the diagram.



Number of area units under the rectangular, $N_A = 25.5$ units.

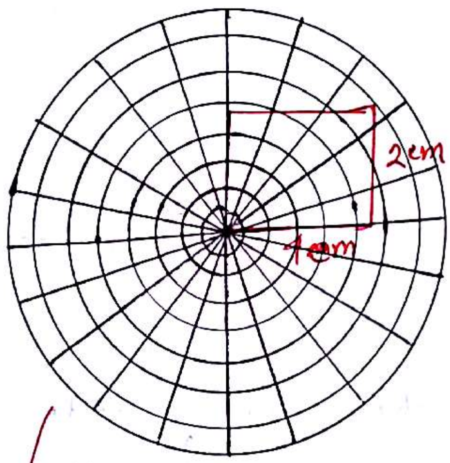
$\therefore \sigma_z$ under center of area $= \underset{I}{0.005} \times q N_A$

$= 0.005 \times 80 \times 25.5$

$= 10.2\ kN/m^2$

Influence value = 0.005

(b) The plan of the rectangular area is then oriented in such a way that one of its corner is above the center of chart.



[Chart লাগবে]

Then,

Number of area unit under the rectangle,

$N_A = 18.5$ units.

σ_z under corner of area $= 0.005 \times 80 \times 18.5$

$= 7.4 \ KN/m^2$

Do Practice from B.C Punmia

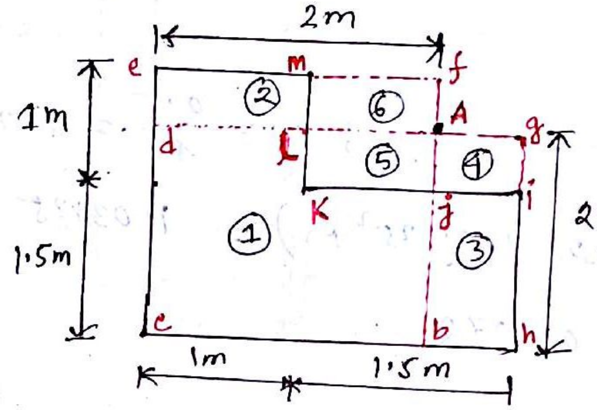
Example – 13.1, 13.2, 13.3, 13.4, 13.5, 13.7, 13.8*

Problem – 1, 2, 3, 5, 9

Stress Distribution

2011

\# Determine the vertical stress at point A below 4m of the following figure. The foundation given in the figure below carries a uniform load of 60 KPa.



\# **Solution:** Given, $z = 4m$.

$q = 60$ KPa.

radian mode

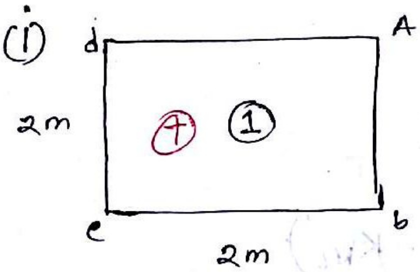
We know, $\sigma_z = q K_N$,

Here, $K_N = \frac{1}{2\pi}\left[\frac{mn}{\sqrt{f}} \times \frac{f+1}{f+m^2n^2} + \sin^{-1}\left(\frac{mn}{f+m^2n^2}\right)\right]$

$m = \frac{L}{z}$, $n = \frac{B}{z}$, $f = m^2+n^2+1$

Now,

(i)

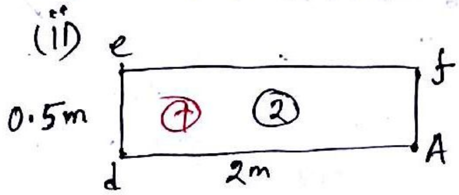


$m = \frac{2}{4} = 0.5$; $n = \frac{2}{4} = 0.5$

$\therefore f = (.5)^2 + (.5)^2 + 1 = 1.5$

$\therefore K_{N_1} = 0.0776$

(ii)

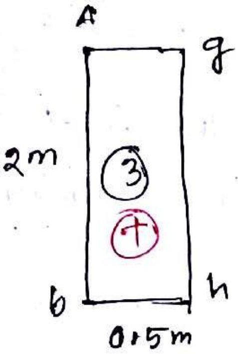


$m = \frac{2}{4} = 0.5$; $n = \frac{0.5}{4} = 0.125$

$f = (0.5)^2 + (.125)^2 + 1 = 1.2656$

$\therefore K_{N_2} = 0.0236$

(iii)

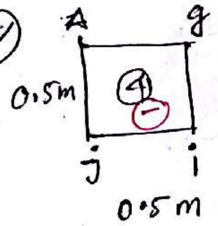


$m = \frac{0.5}{4} = 0.125$; $n = \frac{2}{4} = 0.5$

$f = (0.125)^2 + (0.5)^2 + 1 = 1.2656$

$\therefore K_{N_3} = 0.0236$

(iv)

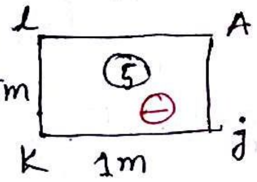


$m = \frac{0.5}{4} = 0.125$; $n = \frac{0.5}{4} = 0.125$

$f = (0.125^2 + 0.125^2 + 1) = 1.03125$

$\therefore K_{N_4} = 0.0072$

(v)

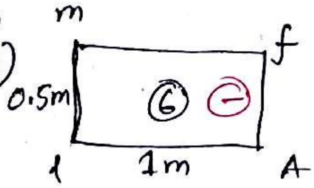


$m = \frac{1}{4} = 0.25$; $n = \frac{0.5}{4} = 0.125$

$f = (0.25^2 + 0.125^2 + 1) = 1.078$

$\therefore K_{N_5} = 0.0138$

(vi)



$m = \frac{1}{4} = 0.25$, $n = \frac{0.5}{4} = 0.125$

$f = (0.25^2 + 0.125^2 + 1) = 1.078$

$\therefore K_{N_6} = 0.0138$

$\therefore$ Total vertical stress at point A,

$\sigma_z = q\,(K_{N_1} + K_{N_2} + K_{N_3} - K_{N_4} - K_{N_5} - K_{N_6})$

$= 60 \times (0.0776 + 0.0236 + 0.0236 - 0.0072 - 0.0138 - 0.0138)$

$= 5.4$ KPa

(Ans)

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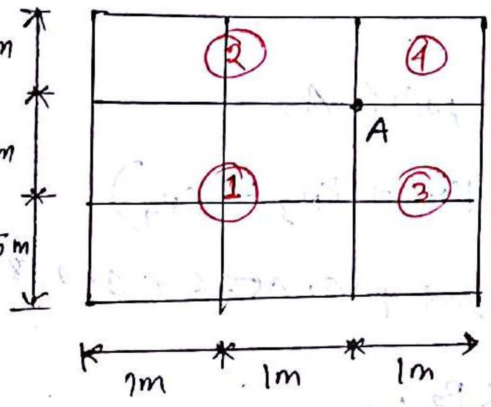
2012 (Similar — 2010)

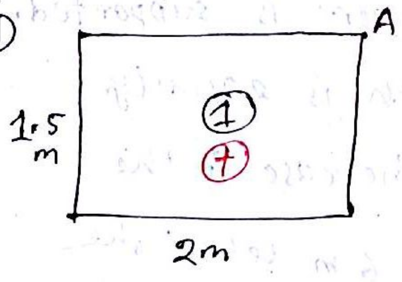
A rectangular foundation 3m × 2.25 m carries a uniform load of 50 KPa as shown in figure below. Determine the vertical stress at 'A' which is 3 m below the ground surface.



<u>Solution:</u> Given, $z = 3m$

$q = 50 \text{ KPa}$

(i)



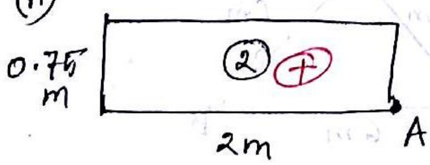
$m = \frac{L}{z} = \frac{2}{3} = 0.667$; $n = \frac{B}{z} = \frac{1.5}{3} = 0.5$

$f = m^2 + n^2 + 1 = (0.667)^2 + (0.5)^2 + 1 = 1.6944$

$K_{N_1} = \frac{1}{2\pi}\left\{\frac{mn}{\sqrt{f}} * \frac{f+1}{f+m^2n^2} + \sin^{-1}\left(\frac{mn}{f+m^2n^2}\right)\right\}$

$\therefore K_{N_1} = 0.0904$

(ii)

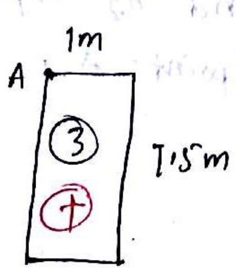


$m = \frac{02}{3} = 0.667$; $n = \frac{0.75}{3} = 0.25$

$f = (0.667)^2 + (.25)^2 + 1 = 1.507$

$\therefore K_{N_2} = 0.0526$

(iii)

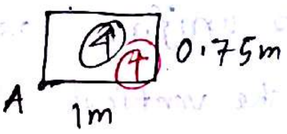


$m = \frac{1}{3} = 0.333$; $n = \frac{1.5}{3} = 0.5$

$f = (0.333)^2 + (0.5)^2 + 1 = 1.36$

$\therefore K_{N_3} = 0.058$

(iv)



$m = \frac{1}{3} = 0.333$; $n = \frac{0.75}{3} = 0.25$

$F = (0.333)^2 + (0.25)^2 + 1 = 1.1736$

$\therefore K_{N_4} = 0.0338$

$\therefore$ Total vertical stress at point A,

$$\sigma_z = q \times (K_{N_1} + K_{N_2} + K_{N_3} + K_{N_4})$$

$$= 50 \times (0.0904 + 0.0526 + 0.058 + 0.0338)$$

$$= 11.74 \text{ KPa.}$$

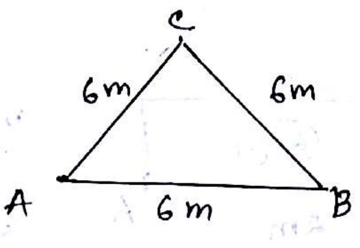
(Ans)

2017
2012 (Ex-13.8 – B.C. Punmia) 2007

\# A base of a tower consists of a equilateral triangular frame with 6m side A, on the corners of which the three legs of the tower is supported. The total weight of the tower is 800 KN, which is equally carried by all the three legs. compute the increase in the vertical stress in the soil caused at a point 6 m below ~~the~~ one of the legs.

Solution: Given, Total $Q = 800$ KN on three legs

$\therefore$ per leg $Q = \frac{800}{3} = 266.67$ KN



we know,

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

. Let us find σ_z at point P below point A :

At leg A, $r = 0$, $z = 6$ m

$$\therefore (\sigma_z)_{PA} = \frac{3 \times 266.67}{2 \times 3.1416} \times \frac{1}{6^2} \times \left[\frac{1}{1+\left(\frac{0}{6}\right)^2}\right]^{5/2} = 3.5368 \text{ KN/m}^2$$

At leg B, $r = 6$ m, $z = 6$ m

$$\therefore (\sigma_z)_{PB} = \frac{3 \times 266.67}{2 \times 3.1416} \times \frac{1}{6^2} \left[\frac{1}{1+\left(\frac{6}{6}\right)^2}\right]^{5/2} = 0.6252 \text{ KN/m}^2$$

Similarly, At leg C, $r = 6$ m, $z = 6$ m

$$\therefore (\sigma_z)_{PC} = 0.6252 \text{ KN/m}^2$$

$\therefore$ Total vertical stress at point P below A,

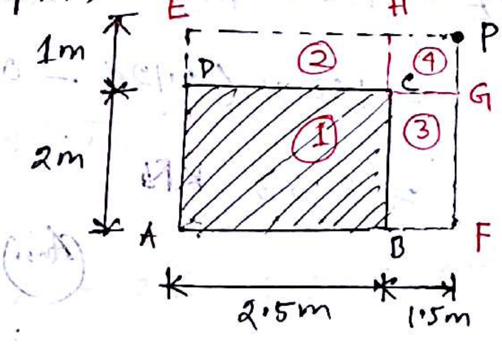
$$\sigma_z = \left[(\sigma_z)_{PA} + (\sigma_z)_{PB} + (\sigma_z)_{PC}\right] = (3.5368 + 0.6252 + 0.6252)$$

$$= 4.7872 \text{ KN/m}^2$$

(Ans)

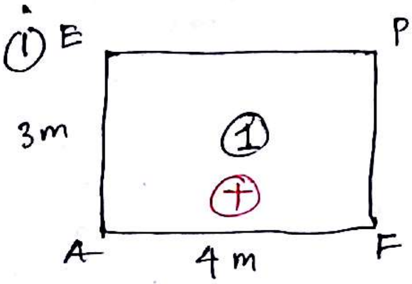
2013

A rectangular loaded area (shaded) is shown in figure, which carries a load of 100 KPa. Determine the vertical stress at point 'P' at a depth of 4 m.



Solution: Given, $q = 100$ KPa

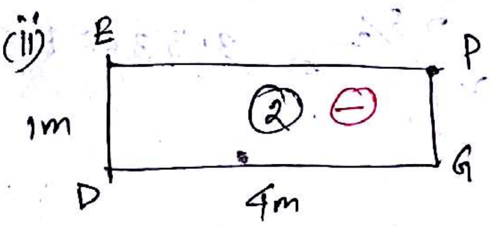
$z = 4$ m



$$m = \frac{L}{z} = \frac{4}{4} = 1 \; ; \; n = \frac{B}{z} = \frac{3}{4} = 0.75$$

$$f = m^2 + n^2 + 1 = 1^2 + (.75)^2 + 1 = 2.563$$

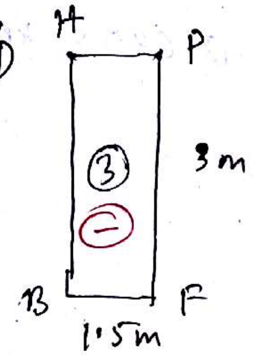
$$K_{N_1} = \frac{1}{2\pi}\left[\frac{mn}{\sqrt{f}} \times \frac{f+1}{f+m^2n^2} + \sin^{-1}\left(\frac{mn}{f+m^2n^2}\right)\right] = 0.124$$

(ii) 

$m = \frac{4}{4} = 1$; $n = \frac{1}{4} = 0.25$

$f = 1^2 + 0.25^2 + 1 = 2.0625$

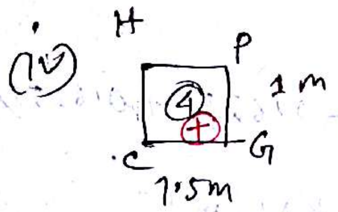
$\therefore K_{N_2} = 0.0587$

(iii) 

$m = \frac{1.5}{4} = 0.375$; $n = \frac{3}{4} = 0.75$

$f = (0.375)^2 + 0.75^2 + 1 = 1.703$

$\therefore K_{N_3} = 0.077$

(iv) 

$m = \frac{1.5}{4} = 0.375$, $n = \frac{1}{4} = 0.125$

$f = (0.375)^2 + (0.25)^2 + 1 = 1.203$

$\therefore K_{N_4} = 0.037$

$\therefore$ vertical stress at point P,

$$\sigma_z = q\,(K_{N_1} - K_{N_2} - K_{N_3} + K_{N_4})$$

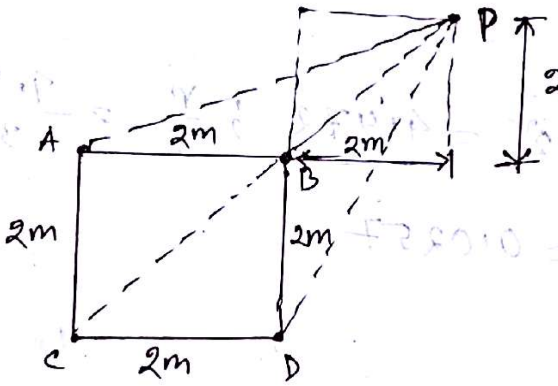
$$= 100 \times (0.124 - 0.0587 - 0.077 + 0.037)$$

$$= 2.53 \text{ KPa}$$

(Ans.)

2014

A water tank of weight 1000 KN is supported by four legs as shown in figure below. Determine the increase in the vertical stress in the soil below 3 m at point P.



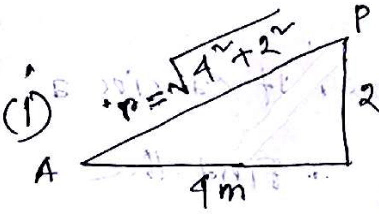
Solution: Given, Total $Q = 1000$ KN on four legs

– per leg $Q = \frac{1000}{4} = 250$ KN

and, $z = 3$ m

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

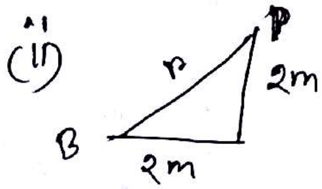
(i)



$r = \sqrt{4^2+2^2} = 4.472$; $\frac{r}{z} = \frac{4.472}{3} = 1.49$

$\therefore K_{PA} = \frac{3}{2 \times 3.1416} \times \left[\frac{1}{1+(1.49)^2}\right]^{5/2} = 0.0257$

(ii)



$r = \sqrt{2^2+2^2} = 2.83$; $\frac{r}{z} = \frac{2.83}{3} = 0.943$

$\therefore K_{PB} = \frac{3}{2 \times 3.1416} \times \left[\frac{1}{1+(0.943)^2}\right]^{5/2} = 0.0974$

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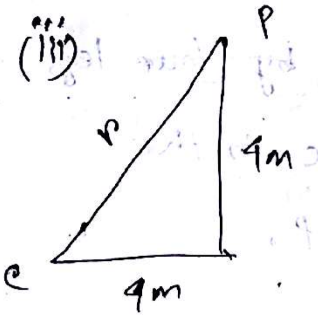
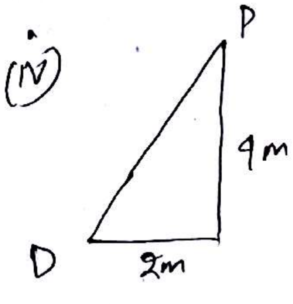
(iii)



$r = \sqrt{4^2 + 4^2} = 5.657$; $\frac{r}{z} = \frac{5.657}{3} = 1.8856$

$\therefore K_{PC} = \frac{3}{2 \times 3.1416} \times \left[\frac{1}{1 + (1.8856)^2}\right]^{5/2} = 0.0108$

(iv)



$r = \sqrt{4^2 + 2^2} = 4.472$; $\frac{r}{z} = \frac{4.472}{3} = 1.49$

$\therefore K_{PD} = 0.0257$

Now, the increase in the vertical stress at point P,

$$\sigma_z = \frac{Q}{z^2} \times (K_{PA} + K_{PB} + K_{PC} + K_{PD})$$

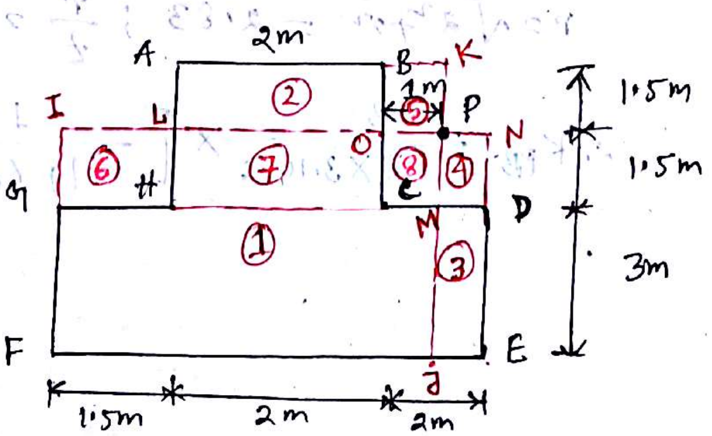
$$= \frac{250}{3^2} \times (0.0257 + 0.0974 + 0.0108 + 0.0257)$$

$$= 4.433 \ KN/m^2$$

(Ans.)

2014

A uniformly loaded area is shown in figure below. It carries a uniform load of 80 KN/m^2 at ground surface. Find the vertical pressure at point P below 5m of ground surface using Network's solution.

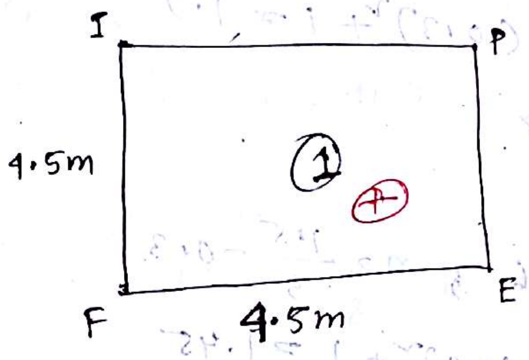


Solution:

$q = 80\ KN/m^2$ We know, $\sigma_z = q\,K_N$

$z = 5m$ where, $K_N = \frac{1}{2\pi} \times \left[\frac{mn}{\sqrt{f}} \times \frac{f+1}{f+m^2n^2} + \sin^{-1}\left(\frac{mn}{f+m^2n^2}\right)\right]$

(i)

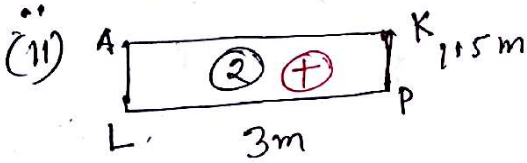


$m = \frac{L}{z} = \frac{4.5}{5} = 0.9$; $n = \frac{B}{z} = \frac{4.5}{5} = 0.9$

$f = (0.9)^2 + (0.9)^2 + 1 = 2.62$

$\therefore K_{N_1} = 0.128$

(ii)

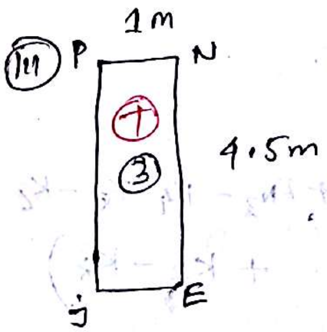


$m = \frac{3}{5} = 0.6$; $n = \frac{1.5}{5} = 0.3$

$f = (0.6)^2 + (0.3)^2 + 1 = 1.45$

$\therefore K_{N_2} = 0.059$

(iii)

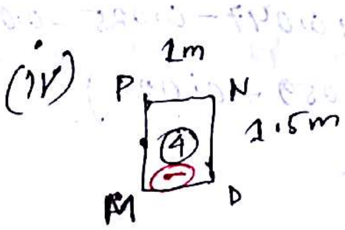


$m = \frac{1}{5} = 0.2$; $n = \frac{4.5}{5} = 0.9$

$f = (0.2)^2 + (0.9)^2 + 1 = 1.85$

$\therefore K_{N_3} = 0.047$

(iv)

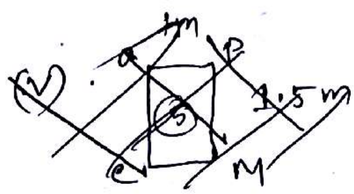


$m = \frac{1}{5} = 0.2$; $n = \frac{1.5}{5} = 0.3$

$f = (0.2)^2 + (0.3)^2 + 1 = 1.13$

$\therefore K_{N_3} = 0.025$

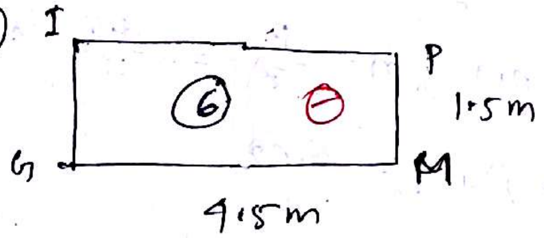
(v)



~~$\therefore K_{N_4}$ 0.025~~

(v) 

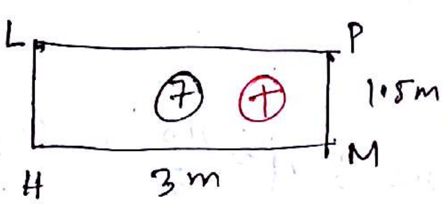
$\therefore K_{N_5} = 0.025$

(vi) 

$m = \frac{4.5}{5} = 0.9$; $n = \frac{1.5}{5} = 0.3$

$f = (0.9)^2 + (0.3)^2 + 1 = 1.9$

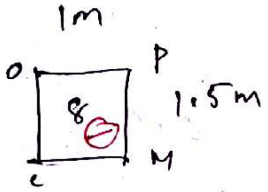
$K_{N_6} = 0.068$

(vii) 

$m = \frac{3}{5} = 0.6$; $n = \frac{1.5}{5} = 0.3$

$f = (0.6)^2 + (0.3)^2 + 1 = 1.45$

$K_{N_7} = 0.059$

(viii) 

$\therefore K_{N_8} = 0.025$

Now,

vertical stress at point P, $\sigma_z = q \times (K_{N_1} + K_{N_2} + K_{N_3} - K_4 - K_5 - K_6 + K_7 - K_8)$

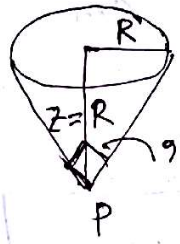
$= 80 \times (0.128 + 0.059 + 0.047 - 0.025 - 0.025 - 0.068 + 0.059 - 0.025)$

$= (80 \times 0.15)$ kN/m²

$= 12$ kN/m²

2015 (Problem-2 - Punmia)

A circular area is loaded with uniform load intensity of 80 KN/m² at ground surface. calculate the vertical pressure at a point P so suited on the vertical line through the center of the loaded area that the area subtents an angle 90° at P. Use Boussinesq equation.



Solution: We know,

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

Given, $q = 80$ KN/m²

$z = R$

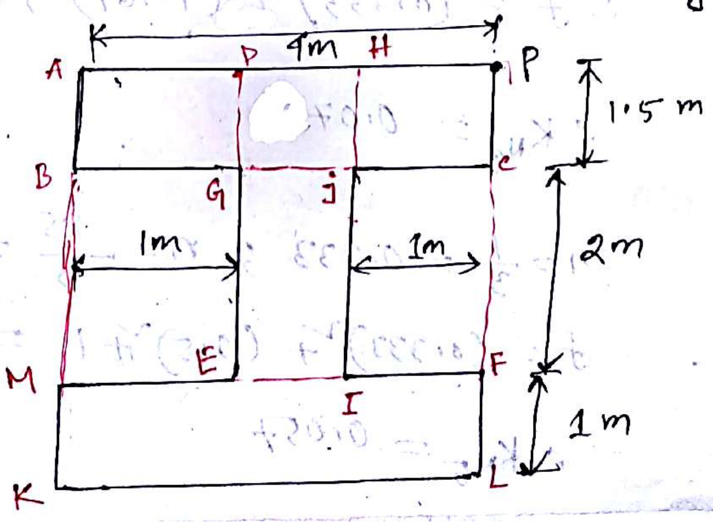
$$\therefore \sigma_z = 80 \times \left[1 - \left\{\frac{1}{1+\left(\frac{R}{R}\right)^2}\right\}^{\frac{3}{2}}\right] = 80 \times \left[1 - (.5)^{3/2}\right]$$

$\therefore \sigma_z = 51.72$ KN/m²

(Ans.)

2015

A uniformly loaded area is shown is figure. It carries a uniform load of 60KN/m² at ground surface. Find the Vertical pressure at point P below 3 m depth using Newmark solution.

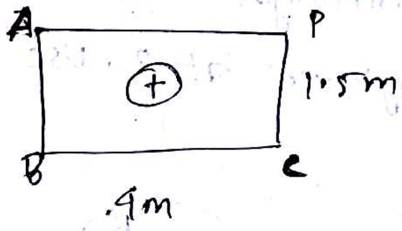


Solution: Given, $q = 60\ kN/m^2$, we know, $\sigma_z = q K_N$

$z = 3m$

where, $K_N = \frac{1}{2\pi}\left[\frac{mn}{\sqrt{f}} \times \frac{f+1}{f+m^2n^2} + \sin^{-1}\left(\frac{mn}{f+m^2n^2}\right)\right]$

(i)

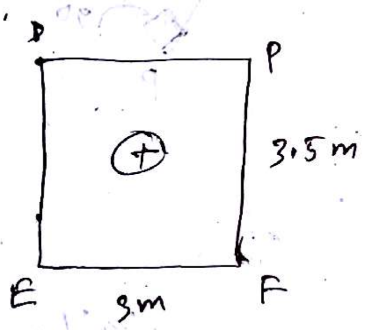


$m = \frac{L}{z} = \frac{4}{3} = 1.33$; $n = \frac{B}{z} = \frac{1.5}{3} = 0.5m$

$f = (1.33)^2 + (0.5)^2 + 1 = 3.02$

$\therefore K_{N_1} = 0.1015$

(ii)

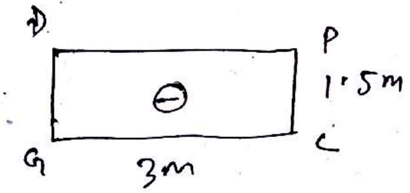


$m = \frac{L}{z} = \frac{3}{3} = 1$; $n = \frac{3.5}{3} = 1.167$

$f = 1^2 + (1.67)^2 + 1 = 3.36$

$\therefore K_{N_2} = 0.1133$

(iii)

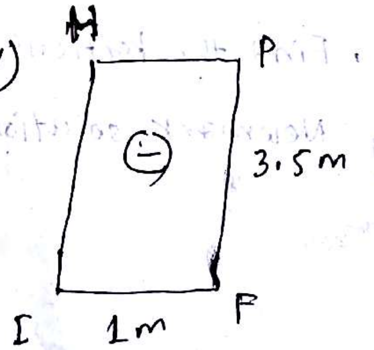


$m = \frac{3}{3} = 1$; $n = \frac{1.5}{3} = 0.5$

$\therefore f = 1^2 + .5^2 + 1 = 2.25$

$\therefore K_{N_3} = 0.101$

(iv)

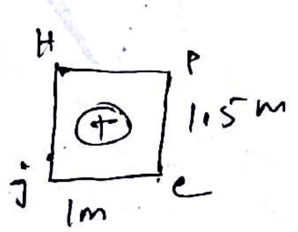


$m = \frac{1}{3} = 0.333$; $n = \frac{3.5}{3} = 1.167$

$\therefore f = (0.333)^2 + (1.167)^2 + 1 = 2.472$

$\therefore K_{N_4} = 0.075$

(v)

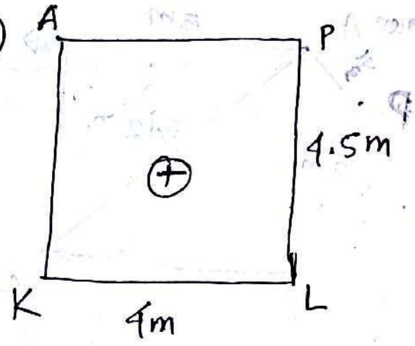


$m = \frac{1}{3} = 0.333$; $n = \frac{1.5}{3} = 0.5$

$f = (0.333)^2 + (0.5)^2 + 1 = 1.36$

$\therefore K_{N_5} = 0.057$

(vi)

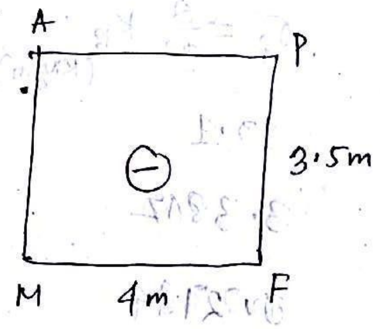


$m = \frac{5}{4} = 1.333$; $n = \frac{4.5}{3} = 1.5$

$f = (1.333)^2 + (1.5)^2 + 1 = 5.03$

$K_{N_6} = 0.13032$

(vii)



$m = \frac{4}{3} = 1.333$; $n = \frac{3.5}{3} = 1.167$

$K_{N_7} = 0.1334$

∴ vertical stress at point P,

$$\sigma_z = q\,(K_{N_1} + K_{N_2} - K_{N_3} - K_{N_4} + K_{N_5} + K_{N_6} - K_{N_7})$$

$$= 60 \times (0.1015 + 0.133 - 0.101 - 0.075 + 0.057 + 0.13032 - 0.1334)$$

$$= 60 \times (0.11242)$$

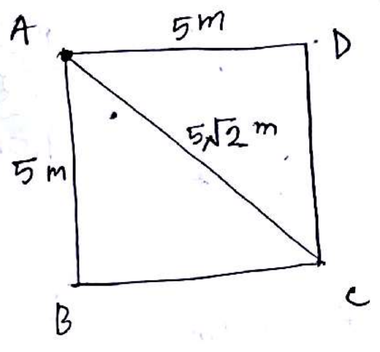
$$= 6.7452 \text{ KN/m}^2$$

(Ans.)

2016

Four vertical loads of 1000 KN each are placed at the corner of a square of side 5 m. Determine the increase of vertical stress 5 m below (i) each load (ii) Mid point between adjacent loads and (iii) center of square.

(i) below each load: Given, $Q = 1000\,KN$ on each corner



we know, $\sigma_z = \frac{Q}{z^2} \times K_B$

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

Given, $z = 5m$, let that point is P.

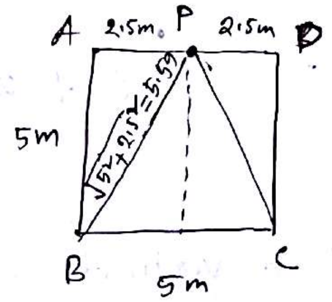
Load at point	Q (KN)	r (m)	$\frac{r}{z}$	K_B	$\sigma_z = \frac{Q}{z^2} K_B$ (KN/m²)
A	1000	0	0	0.4775	19.1
B	1000	5	1	0.0845	3.38
C	1000	$5\sqrt{2}$	$\sqrt{2}$	0.0306	1.224
D	1000	5	1	0.0845	3.38

$\Sigma\sigma_z = 27.084\ KN/m^2$

Hence,
vertical stress 5m below at each load will be same.

$(\sigma_z)_{PA} = (\sigma_z)_{PB} = (\sigma_z)_{PC} = (\sigma_z)_{PD} = 27.084\ KN/m^2$

(ii) Mid point between adjacent load:



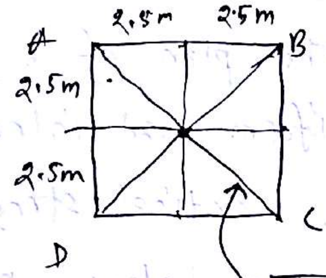
Load at point	Q (KN)	r (m)	$\frac{r}{z}$	K_B	$\sigma_z = \frac{Q}{z^2} K_B$ (KN/m²)
A	1000	2.5	0.5	0.2733	10.932
B	1000	5.59	1.118	0.063	2.52
C	1000	5.59	1.118	0.063	2.52
D	1000	2.5	0.5	0.2733	10.932

$\Sigma\sigma_z = 26.904\ KN/m^2$

vertical stress mid point between adjacent load below 5m,

$\sigma_z = 26.904\ KN/m^2$

(iii) centre of square:



$\sqrt{2.5^2+2.5^2}$

$= 3.536$

Load at point	Q (KN)	r (m)	$\frac{r}{z}$	K_B	$\sigma_z = \frac{Q}{z^2} K_B$ (KN/m²)
A	1000	3.536	0.7072	0.17323	6.9292
B	1000	3.536	0.7072	0.17323	6.9292
C	1000	3.536	0.7072	0.17323	6.9292
D	1000	3.536	0.7072	0.17323	6.9292

$\Sigma \sigma_z = 27.7168$ KN/m²

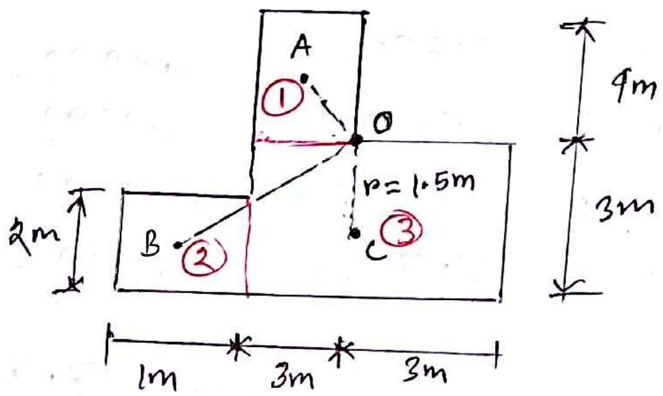
∴ At the center of square below 5m,

$\sigma_z = 27.7168$ KN/m²

(Ans.)

2016

The plan of a foundation is shown in figure below. The uniform load intensity on the soil is 50 KN/m². Determine the vertical stress due to the foundation at a depth of 5 m below point O. Use **Boussinesq influence chart**. (2)



* Equivalent point load Method

২৫ পাতা

Solution: Given, $q = 50\ KN/m^2$

(i) 2m, 1.5m; $r = \sqrt{2^2 + 1.5^2} = 2.5$; $\frac{r}{z} = \frac{2.5}{5} = 0.5$; $K_{B_1} = \frac{3}{2\pi} \times \left[\frac{1}{1+(0.5)^2}\right]^{5/2}$

$\therefore K_{B_1} = 0.2733$

$Q_1 = 50 \times (4 \times 3)\ KN$
$= 600\ KN$

(ii) 3.5 m, 2m; $r = \sqrt{3.5^2 + 2^2} = 4.03$; $\frac{r}{z} = \frac{4.03}{5} = 0.806$; $K_{B_2} = \frac{3}{2\pi} \times \left[\frac{1}{1+(.806)^2}\right]^{5/2}$

$\therefore K_{B_2} = 0.1366$

$Q_2 = 50 \times (2 \times 1)$
$= 100\ KN$

(iii) $r = 1.5$; $\frac{r}{z} = \frac{1.5}{5} = 0.3$; $K_{B_3} = \frac{3}{2\pi} \times \left[\frac{1}{1+(.3)^2}\right]^{5/2} = 0.385$

$Q_3 = 50 \times (6 \times 3) = 900\ KN$

Now, The increase in the vertical stress at a depth of 5 m below point O,

$$\sigma_z = \frac{Q_1}{z^2} \times K_{B_1} + \frac{Q_2}{z^2} \times K_{B_2} + \frac{Q_3}{z^2} \times K_{B_3}$$

$$= \frac{600}{5^2} \times 0.2733 + \frac{100}{5^2} \times .1366 + \frac{900}{5^2} \times 0.385$$

$$= 20.9656\ KN/m^2$$

(Ans.)

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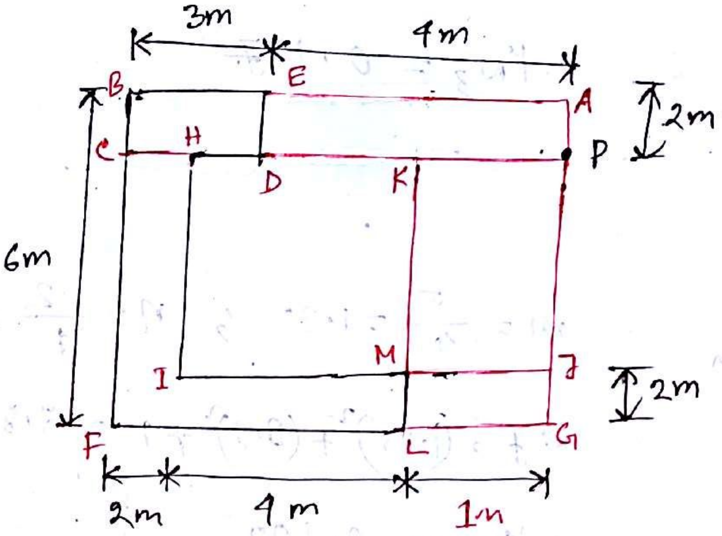
2017

Determine the vertical stress increase at point P of the figure given below a depth of 4m. Given that $q = 100\ KN/m^2$



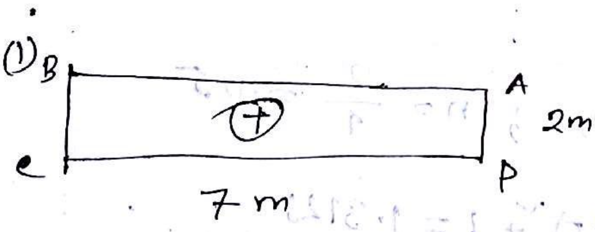
Solution:

We know, $\sigma_z = q\,K_N$

where, $K_N = \frac{1}{2\pi}\left[\frac{mn}{\sqrt{f}} \times \frac{f+1}{f+m^2n^2} + \sin^{-1}\left(\frac{mn}{f+m^2n^2}\right)\right]$

Given, $z = 4m$

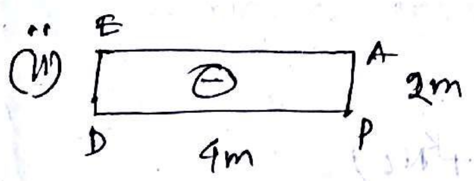
$q = 100\ KN/m^2$

(i) 

$m = \frac{7}{4} = 1.75$; $n = \frac{2}{4} = 0.5$

$\therefore f = m^2+n^2+1 = 4.3125$

$\therefore K_{N_1} = 0.0977$

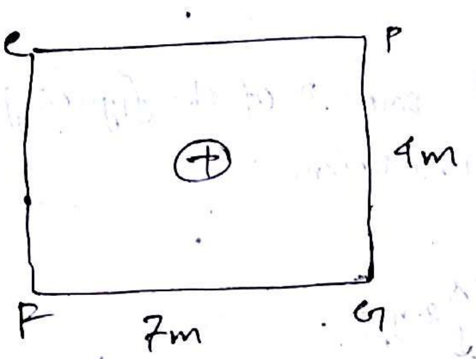
(ii) 

$m = \frac{4}{4} = 1$; $n = \frac{2}{4} = 0.5$

$\therefore f = 1^2 + 0.5^2 + 1 = 2.25$

$\therefore K_{N_2} = 0.101$

(iii)

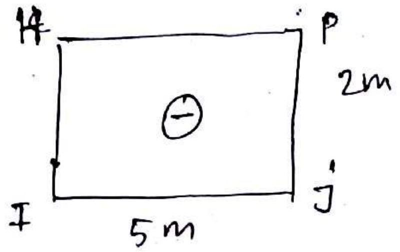


$m = \frac{7}{4} = 1.75$; $n = \frac{4}{4} = 1$

$\therefore f = (1.75)^2 + 1^2 + 1 = 5.0625$

$\therefore K_{N_3} = 0.127$

(iv)

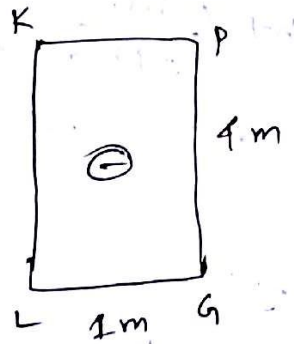


$m = \frac{5}{4} = 1.25$; $n = \frac{2}{4} = 0.5$

$\therefore f = (1.25)^2 + (0.5)^2 + 1 = 2.8125$

$\therefore K_{N_4} = 0.102$

(v)

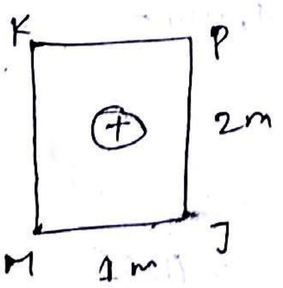


$m = \frac{1}{4} = 0.25$; $n = \frac{4}{4} = 1$

$\therefore f = (.25)^2 + 1^2 + 1 = 2.0625$

$\therefore K_{N_5} = 0.0587$

(vi)



$m = \frac{1}{4} = 0.25$; $n = \frac{2}{4} = 0.5$

$\therefore f = (.25)^2 + (.5)^2 + 1 = 1.3125$

$\therefore K_{N_6} = 0.04524$

$\therefore \sigma_z = q\,(K_{N_1} - K_{N_2} + K_{N_3} - K_{N_4} - K_{N_5} + K_{N_6})$

$= 100 \times (.0977 - 0.101 + 0.127 - 0.102 - 0.0587 + 0.04524)$

$= 0.824\ KN/m^2$

(Ans.)

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2016

A concentrated load of 40 KN acts on the surface of a soil. Determine the variation of vertical stress increaments at points directly beneath the load upto a depth of 10 m and draw a plot. Also plot the variation of vertical stress incre-ment due to load on horizontal planes at depths of 1 m and 2m upto a horizontal distance of 3 m on either side of corner.

Solution: Variation of vertical stress directly beneath the load of 40 KN:

Here, $r=0$. Hence, $\sigma_z = 0.4775 \times \frac{Q}{z^2}$

z (m)	2	4	6	8	10
σ_z (KN/m²)	4.775	1.2	0.531	0.3	0.191

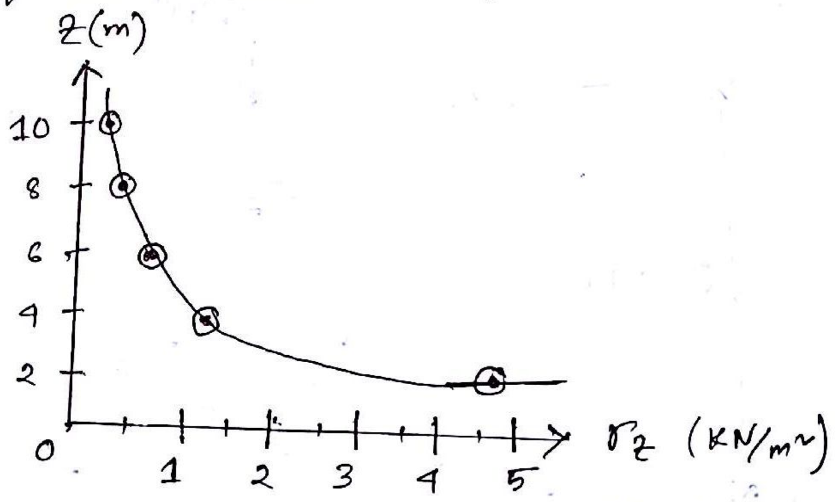


Fig. Variation of vertical stress with depth

Variation of vertical stress increment due to load on horizontal planes:

Given, $z=1$ and $z=2$; we know, $\sigma_z = \frac{Q}{z^2} \times K_B$

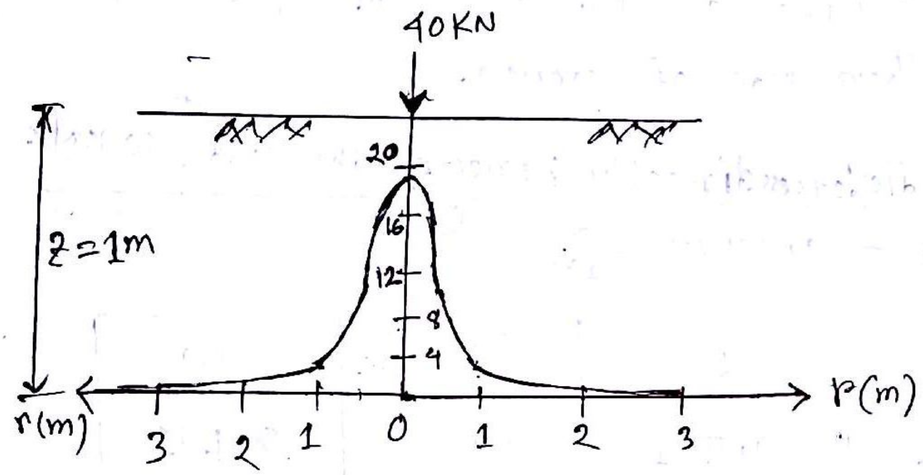
where, $K_B = \frac{3}{2\pi} \times \left[\frac{1}{1+\left(\frac{r}{z}\right)^2}\right]^{5/2}$

For $\boxed{z=1m}$

r (m)	$\frac{r}{z}$	K_B	σ_z	r	$\frac{r}{z}$	K_B	σ_z
0.0	0	0.4775	19.1	1.5	1.5	0.0251	1.004
0.5	0.5	0.2733	10.932	2.0	2.0	0.0085	0.34
1.0	1.0	0.0844	3.376	3.0	3.0	0.0015	0.06

For $\boxed{z = 2}$

r(m)	$\frac{r}{z}$	K_B	σ_z	r(m)	$\frac{r}{z}$	K_B	σ_z
0.0	0	0.4775	4.775	1.5	0.75	0.1565	1.565
0.5	0.25	0.4103	4.103	2.0	1	0.0844	0.844
1.0	0.5	0.2733	2.733	3.0	1.5	0.0251	0.251



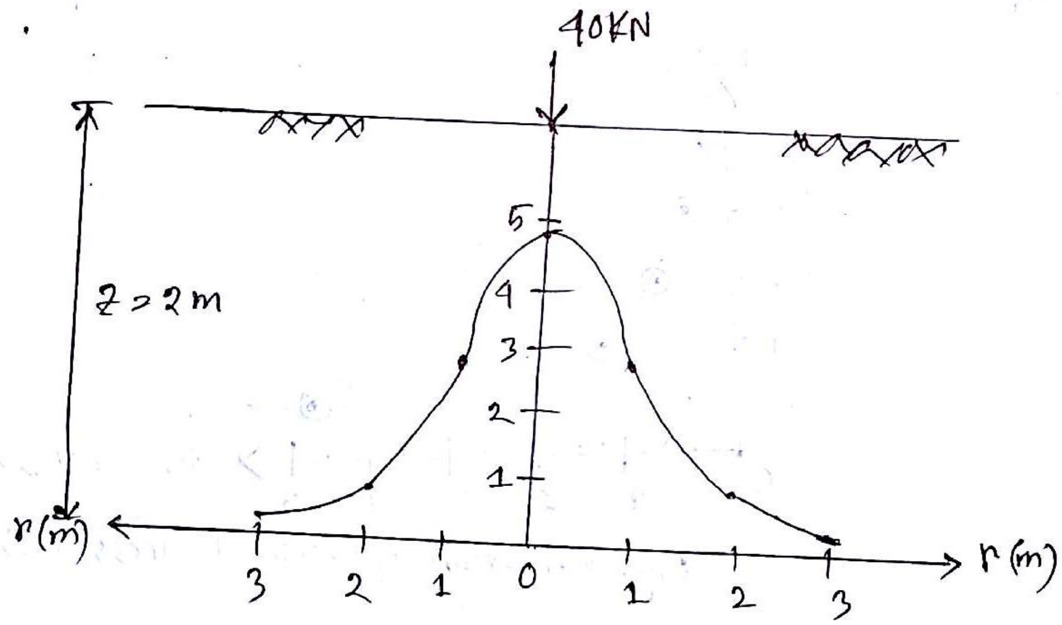


Fig. variation of vertical stress on horizontal plane.

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2017

Subsoil Exploration

Calculate the corrected SPT number using skemption's relationship. The GWT is located at a depth of 5.5 m. The dry and saturated unit weight of sand are 18 KN/m3 and 19.5 KN/m3 respectively. The sand is over consolidated sand.

Depth (m)	1.5	3.0	4.5	6.0	7.5	9.0	10.5
N_{60}	5	7	9	8	13	12	14

Solution: According to skemption's relationship,

For over consolidated sand, $C_N = \dfrac{1.7}{0.7 + \left(\dfrac{\sigma_0'}{P_a}\right)}$ Here, $P_a = 100\ KN/m^2$

$\sigma_0' = \gamma' h$

Given, $\gamma_d = 18\ KN/m^3$ and,

$\gamma_{sat} = 19.5\ KN/m^3 \quad \therefore \gamma' = (19.5 - 9.81) = 9.69\ KN/m^3$

Depth (m)	σ_0' (KN/m²)	C_N	N_{60}	$(N_1)_{60} = C_N N_{60}$
1.5	$(1.5 \times 18) = 27$	1.753	5	$8.765 \simeq 9$
3.0	$(3 \times 18) = 54$	1.371	7	$9.597 \simeq 10$
4.5	$(4.5 \times 18) = 81$	1.126	9	$10.134 \simeq 10$
6.0	$(5.5 \times 18) + (0.5 \times 9.69) = 103.845$	0.978	8	$7.824 \simeq 8$
7.5	$(5.5 \times 18) + (2 \times 9.69) = 118.38$	0.902	13	$11.726 \simeq 12$
9.0	$(5.5 \times 18) + (3.5 \times 9.69) = 132.915$	0.838	12	$10.056 \simeq 10$
10.5	$(5.5 \times 18) + (5 \times 9.69) = 147.45$	0.782	14	$10.948 \simeq 11$

Same Problem:

2016 ⇒ The sand is Normally consolidated coarse sand. Use skempton's relationship.

$\boxed{C_N = \dfrac{3}{2 + \left(\dfrac{\sigma_0'}{P_a}\right)}}$

$\boxed{C_N = \dfrac{2}{1 + \left(\dfrac{\sigma_0'}{P_a}\right)}}$

2015 ⇒ The sand is Normally consolidated fine sand. Use skempton's relationship.

2011, 2012

2014 ⇒ use Li'ao and whitman relationship.

$$\boxed{C_N = \sqrt{\frac{1}{\left(\frac{\sigma_0'}{P_a}\right)}}}$$

2013

The N_{60} value at a depth of 4.5 m for a sandy soil is 10. Determine the soil frinction angle

Solution:

We know, (Peck, Hanson and Thornburn)

(i) $\phi'(\text{deg}) = 27.1 + 0.3 N_{60} - 0.00054 [N_{60}]^2$

$= 27.1 + 0.3 \times 10 - 0.00054 \times (10)^2$

$\therefore$ friction angle, $\phi' = 30.046°$

(ii) (Hatanaka and Uchida:)

$\phi' = \sqrt{20 (N_1)_{60}} + 20 = \sqrt{20 \times 10} + 20$

$\therefore \phi' = 34.14°$ (Ans:)

2006

Following are the standard penetration numbers determined from a sand soil deposite in the field.

Depth (m)	2	3	4	5	7	9
soil density γ (KN/m^2)	16	16	16	18	18	18
N_{60}	6	8	10	15	18	20

Determine the variation of the peak soil friction angle. Estimate an average value of ϕ for the design of shallow foundation.

Solution: We know, $\sigma = \gamma h$

$$C_N = \frac{2}{1+\left(\frac{\sigma_0'}{P_a}\right)} \text{ ; where, } P_a = 100\ KN/m^2$$

$$(N_1)_{60} = C_N N_{60} \text{ and } \phi' = 27.1 + 0.3(N_1)_{60} - 0.00054\,(N_1)^2_{60}$$

Depth (m)	γ (KN/m^3)	σ_0' (KN/m^2)	C_N	N_{60}	$(N_1)_{60}$	ϕ' (deg)
2	16	32	1.515	6	9.09 ≃ 9	29.756
3	16	48	1.351	8	10.808 ≃ 11	30.33
4	16	64	1.22	10	12.2 ≃ 12	30.62
5	18	64+1×18= 82	1.099	15	16.5 ≃ 17	32.044
7	18	82+2×18= 118	0.917	18	16.506 ≃ 17	32.044
9	18	118+2×18= 154	0.787	20	15.74 ≃ 16	31.76

$\Sigma\phi' = 186.474$

$\therefore$ The average value of $\phi' = \frac{186.474}{6} = 31.08°$

(Ans)

2000

For a shelby tube outside diameter = 2" and inside diameter = 1.75"

(i) What is the area ratio of the tube?

(ii) if the outside diameter remaining same, what would be the inside diameter of the tube to give an area ratio of 10%

Solution:

(i) We know, $A_r(\%) = \frac{D_o^2 - D_i^2}{D_i^2} \times 100$

$= \frac{2^2 - 1.75^2}{1.75^2} \times 100$

$\therefore A_r(\%) = 30.6$

(ii)

Given, $A_r = 10\%$

$\therefore \frac{10}{100} = \frac{2^2 - D_i^2}{D_i^2}$

$\Rightarrow 0.1 D_i^2 = 4 - D_i^2$

$\Rightarrow D_i^2 = 3.636$

$\therefore D_i = 1.907$ in

$\therefore$ inside diameter would be 1.907 in. (Ans)

2015 Let, The sand is normally consolidated fine sand.

We Know,

Skempton's relationship for the sand,

$$C_N = \frac{2}{1+\left(\frac{\sigma_o'}{P_a}\right)}$$

Here, $P_a = 100\ KN/m^2$

$\sigma_o' = \gamma' H$

Given,

$\gamma_d = 18\ KN/m^3$ and,

$\gamma_{sat} = 19.5\ KN/m^3$ $\therefore \gamma' = (19.5 - 9.81) = 9.69\ KN/m^3$

Depth (m)	σ_o' (KN/m^2)	C_N	N_{60}	$(N_1)_{60} = C_N N_{60}$
1.5	$(18\times1.5) = 27$	1.575	5	$7.875 \simeq 8$
3.0	$(3\times18) = 54$	1.2987	7	$9.09 \simeq 9$
4.5	$(4.5\times18) = 81$	1.105	9	$9.945 \simeq 10$
6.0	$(5.5\times18) + (0.5\times9.69) = 103.845$	0.98	8	$7.84 \simeq 8$
7.5	$(5.5\times18) + (2\times9.69) = 118.38$	0.916	13	$11.908 \simeq 12$
9.0	$(5.5\times18) + (3.5\times9.69) = 132.915$	0.8586	12	$10.3 \simeq 10$
10.5	$(5.5\times18) + (5\times9.69) = 147.45$	0.808	14	$11.3 \simeq 11$

* যদি কি ধরনের sand বলে না দেয়া থাকে, Liao and whitman relationship use করেও করা যাবে।

#2015

Given, The sand is normally consolidate coarse sand.

Using skempton's relationship for the sand,

$$C_N = \frac{3}{2+\left(\frac{\sigma_o'}{P_a}\right)}$$

Here, $P_a = 100\ kN/m^2$

$\sigma_o' = \gamma' H$

Given, $\gamma_d = 18\ kN/m^3$ and,

$\gamma_{sat} = 19.5\ kN/m^3$ $\therefore \gamma' = (19.5 - 9.81) = 9.69\ kN/m^3$

Depth (m)	σ_o' (kN/m^2)	C_N	N_{60}	$(N_1)_{60} = C_N N_{60}$
1.5	$(18 \times 1.5) = 27$	1.32	5	$6.6 \simeq 7$
3	$(3 \times 18) = 54$	1.18	7	$8.26 \simeq 8$
4.5	$(4.5 \times 18) = 81$	1.07	9	$9.63 \simeq 10$
6	$(5.5 \times 18) + (0.5 \times 9.69) = 103.845$	0.987	8	$7.896 \simeq 8$
7.5	$(5.5 \times 18) + (2.0 \times 9.69) = 118.38$	0.9423	13	$12.25 \simeq 12$
9.0	$(5.5 \times 18) + (3.5 \times 9.69) = 132.915$	0.9	12	$10.8 \simeq 11$
10.5	$(5.5 \times 18) + (5 \times 9.69) = 147.45$	0.863	14	$12.082 \simeq 12$

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2014
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Using Liao and whitman relation ship,

$$C_N = \sqrt{\frac{1}{\left(\frac{\sigma_o'}{P_a}\right)}}$$

where, $P_a = 100\ KN/m^2$

$\sigma_o' = \gamma' H$

Given, $\gamma_d = 18\ KN$ and

$\gamma_{sat} = 19.5\ KN$

$\therefore \gamma' = (19.5 - 81) = 9.69\ KN/m^2$

Depth (m)	σ_o' (KN/m^2)	C_N	N_{60}	$(N_1)_{60} = C_N N_{60}$
1.5	$(18 \times 1.5) = 27$	1.92	5	$9.6 \simeq 10$
3.0	$(18 \times 3) = 54$	1.36	7	$9.52 \simeq 10$
4.5	$(18 \times 4.5) = 81$	1.11	9	$9.99 \simeq 10$
6.0	$(18 \times 5.5) + (9.69 \times 0.5) = 103.845$	0.98	8	$7.84 \simeq 8$
7.5	$(18 \times 5.5) + (9.69 \times 2) = 118.38$	0.92	13	$11.96 \simeq 12$
9.0	$(18 \times 5.5) + (9.69 \times 3.5) = 132.915$	0.867	12	$10.404 \simeq 10$
10.5	$(18 \times 5.5) + (9.69 \times 5) = 147.45$	0.824	14	$11.536 \simeq 12$

Bearing Capacity of Shallow Foundation

Terzaghi's Equation:

(i) For Strip Footing: $q_u = c' N_c + q N_q + \frac{1}{2} \gamma B N_\gamma$

(ii) For Square Footing: $q_u = 1.3\, c' N_c + q N_q + 0.4\, \gamma B N_\gamma$

(iii) For circular Footing: $q_u = 1.3\, c' N_c + q N_q + 0.3\, \gamma N_\gamma B$

Where,

$$N_q = \frac{e^{2\left(\frac{3\pi}{4} - \frac{\phi'}{2}\right)\tan\phi'}}{2\cos^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)}$$

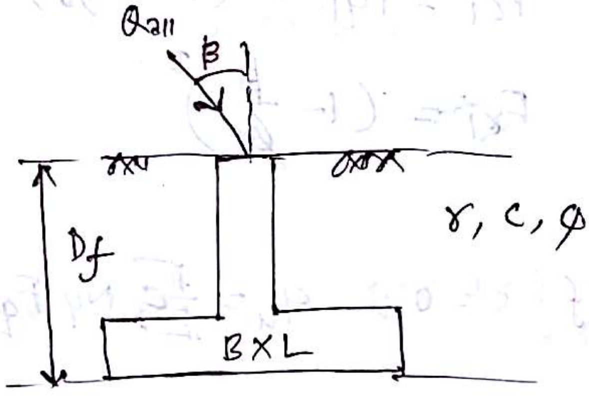
$$N_c = (N_q - 1)\cot\phi'$$

$$N_\gamma = \frac{1}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi'} - 1\right)\tan\phi'$$; $K_{p\gamma}$ = passive pressure co-efficient

Meyerhof's Equation:

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

Where,



Bearing capacity factors:

$$N_q = \tan^2\left(45 + \frac{\phi'}{2}\right) e^{\pi \tan\phi'}$$

$$N_c = (N_q - 1)\cot\phi'$$

$$N_\gamma = 2(N_q + 1)\cdot\tan\phi'$$

Shape factors:

$$F_{cs} = 1 + \left(\frac{B}{L}\right) \times \left(\frac{N_q}{N_c}\right)$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right) \times \tan\phi'$$

$$F_{\gamma s} = 1 - 0.4 \times \left(\frac{B}{L}\right)$$

* for circular, B = L

* for square, B = L

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Depth factors:

For $\frac{D_f}{B} \le 1$ & $\phi' = 0$

$$F_{cd} = 1 + 0.4 \times \frac{D_f}{L}$$

$$F_{qd} = 1$$

$$F_{\gamma d} = 1$$

For $\frac{D_f}{B} \le 1$ & $\phi' > 0$

$$F_{qd} = 1 + 2\tan\phi'(1-\sin\phi')^2 \times \left(\frac{D_f}{B}\right)$$

$$F_{cd} = F_{qd} - \frac{1-F_{qd}}{N_c \tan\phi'}$$

$$F_{\gamma d} = 1$$

For $\frac{D_f}{B} > 1$ & $\phi' = 0$

$$F_{cd} = 1 + 0.4 \times \underline{\tan^{-1}\left(\frac{D_f}{L}\right)}$$

radian mode

$$F_{qd} = 1$$

$$F_{\gamma d} = 1$$

For $\frac{D_f}{B} > 1$ & $\phi' > 0$

$$F_{qd} = 1 + 2\tan\phi'(1-\sin\phi')^2 \times \underline{\tan^{-1}\left(\frac{D_f}{B}\right)}$$

radian mode

$$F_{cd} = F_{qd} - \frac{1-F_{qd}}{N_c \tan\phi'}$$

$$F_{\gamma d} = 1$$

Influence factors:

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta}{90}\right)^2$$

$$F_{\gamma i} = \left(1 - \frac{\beta}{\phi'}\right)^2$$

* if $\beta = 0$ then,

$$F_{ci} = F_{qi} = F_{\gamma i} = 1$$

if $c = 0$, $q_u = \left[q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}\right]$

[Note: যদি কোনো কিছু উল্লেখ না থাকে তাহলে, Meyerhof equation use করতে হবে। শুধুমাত্র Terzaghi's equation -উল্লেখ থাকলেই তা use করতে হবে]

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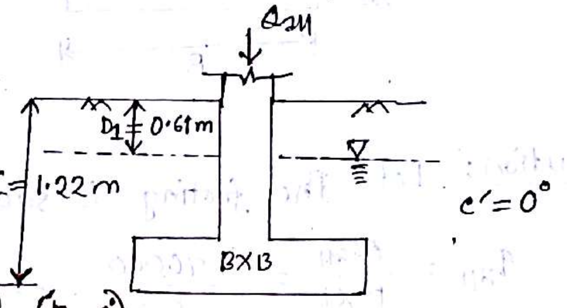
Assignment

Problem: 01 A square foundation (B×B) has constructed as shown in figure. Assume that, $\gamma = 16.5\ KN/m^3$, $\gamma_{sat} = 18.55\ KN/m^3$, $\phi = 34°$, $D_f = 1.22m$ and $D_1 = 0.61\ m$. The gross allowable load, Q_{all} with $F.S = 3$ is $667.2\ KN$. Determine the size of footing.



Solution: $q_{all} = \frac{Q_{all}}{B \times L} = \frac{667.2\ KN}{B^2}$ ....(1)

Again, $q_{all} = \frac{q_u}{F.S.} = \frac{1}{3}\left(q N_q F_{qs} F_{qd} + \frac{1}{2}\gamma' B N_\gamma F_{\gamma s} F_{\gamma d}\right)$ $(\beta = 0°)$ $\rightarrow F_{qi} = F_{ci} = F_{\gamma i} = 1$

$q = D_1\gamma + D_2\gamma'$

$\Rightarrow q = 0.61 \times 16.5 + (1.22 - 0.61) \times (18.55 - 9.81) = 15.4\ KN/m^2$

$N_q = \tan^2(45 + \frac{\phi}{2}) e^{\pi \tan\phi} = \tan^2(45 + \frac{34}{2}) e^{\pi \times \tan 34} = 29.44$

$N_\gamma = 2(N_q + 1)\tan\phi = 2 \times (29.44 + 1) \times \tan 34 = 41.064$

$F_{qs} = 1 + (\frac{B}{L})\tan\phi = 1 + 1 \times \tan 34 = 1.6745$

$F_{\gamma s} = 1 - 0.4 \times (\frac{B}{L}) = 1 - 0.4 \times 1 = 0.6$

Let, $\frac{D_f}{B} \leq 1$; $\phi = 34°$

$\therefore F_{qd} = 1 + 2\tan\phi(1 - \sin\phi)^2 \times (\frac{D_f}{B}) = 1 + 2 \times \tan 34 \times (1 - \sin 34)^2 \times \frac{1.22}{B}$

$\Rightarrow F_{qd} = 1 + \frac{0.32}{B}$

and $F_{\gamma d} = 1$

Hence, $q_{all} = \frac{1}{3}\left[15.4 \times 29.44 \times 1.6745 \times (1 + \frac{0.32}{B}) + \frac{1}{2} \times (18.55 - 9.81) \times B \times 41.064 \times 0.6 \times 1\right]$

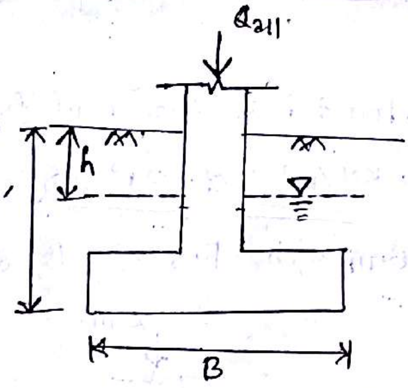
From eqn (1),

$\Rightarrow \frac{667.2}{B^2} = 253.06 + \frac{80.98}{B} + 35.89B$

$\therefore B = 1.36\ m$ $\qquad \frac{D_f}{B} = \frac{1.22}{1.36} < 1$ (OK)

$\therefore$ size of footing $= (1.36\ m \times 1.36\ m)$ **Ans.**

Problem: 02



Given, $\gamma = 100\ lb/ft^3$, $\gamma_{sat} = 120\ lb/ft^3$

$c' = 0$, $\phi = 30°$, $D_f = 5\ ft$, $Q_{all} = 40000\ lb$

and, $F.S = 3$

Determine the size of footing when:

(i) $h = 0\ ft$ (ii) $h = 2\ ft$ (iii) $h = 5\ ft$

Solution: Let, The footing is square. $\therefore$ size $= (B \times B)\ ft^2$

$$q_{all} = \frac{Q_{all}}{B \times L} = \frac{40000}{B^2} \quad \cdots\cdots (1)$$

again, $q_{all} = \frac{q_u}{F.S.} = \frac{1}{3}\left[q\, N_q \cdot F_{qs} \cdot F_{qd} + \frac{1}{2}\gamma' B N_\gamma F_{\gamma s} \cdot F_{\gamma d}\right]$

(i) When, h = 0 : $q = \gamma' D_f = (120 - 62.4) \times 5 = 288\ lb/ft^2$

$$N_q = \tan^2\left(45 + \frac{30}{2}\right) e^{\pi \times \tan 30} = 18.40$$

$$N_\gamma = 2(N_q + 1)\tan\phi = 22.40$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right) \times \tan\phi = 1 + (1 \times \tan 30) = 1.577$$

$$F_{\gamma s} = 1 - 0.4 \times \left(\frac{B}{L}\right) = 0.6$$

Assume, $\frac{D_f}{B} \leq 1$; $\phi = 30°$

$\therefore F_{qd} = 1 + 2\tan 30 \times (1 - \sin 30)^2 \times \frac{5}{B} = \left(1 + \frac{1.4434}{B}\right)$

and $F_{\gamma d} = 1$

Hence, $q_{all} = \frac{1}{3}\left[288 \times 18.40 \times 1.577 \times \left(1 + \frac{1.4434}{B}\right) + \frac{1}{2} \times (120 - 62.4) \times B \times 22.40 \times 0.6 \times 1\right]$

From eq^n (1) $\Rightarrow \frac{40000}{B^2} = 2785.6128 + \frac{4020.75}{B} + 129.024\,B$

$$\Rightarrow B = 2.974$$

$$\frac{D_f}{B} = \frac{5}{2.974} = 1.68 > 1$$

(not OK)

Hence, considering $\frac{D_f}{B} > 1$:

$$F_{qd} = 1 + 2\tan 30 \times (1-\sin 30)^2 \times \tan^{-1}\left(\frac{5}{B}\right) = 1 + 0.29\tan^{-1}\left(\frac{5}{B}\right)$$

Now,

$$\frac{40000}{B^2} = 2785.6128 + 807.83\tan^{-1}\left(\frac{5}{B}\right) + 129.024B$$

$$\Rightarrow B = 3.16\ ft$$

$$\therefore \frac{D_f}{B} = \frac{5}{3.16} = 1.58 > 1$$ (OK) $\therefore$ size of footing $= (3.16' \times 3.16')$

(ii) When $h = 2ft$:

$$q = h\gamma + (D_f - h) \times \gamma'$$

$$\Rightarrow q = 2 \times 100 + (5-2) \times (120 - 62.4) = 372.8\ lb/ft^2$$

Then, Assume, $\frac{D_f}{B} \leq 1$

$$\therefore \frac{40000}{B^2} = \frac{1}{3}\left[372.8 \times 18.40 \times 1.577 \times \left(1 + \frac{1.4434}{B}\right)\right] + 129.024B$$

$$\Rightarrow \frac{40000}{B^2} = 3605.82 + \frac{5204.64}{B} + 129.024B$$

$$\Rightarrow B = 2.5934\ ft$$

$$\therefore \frac{D_f}{B} = \frac{5}{2.5934} = 1.93 > 1$$ (not OK)

Hence, considering $\frac{D_f}{B} > 1$,

$$\therefore F_{qd} = 1 + 0.29\tan^{-1}\left(\frac{5}{B}\right)$$

Now, $$\frac{40000}{B^2} = 3605.82 + 1045.69\tan^{-1}\left(\frac{5}{B}\right) + 129.024B$$

$$\Rightarrow B = 2.81\ ft$$

$$\therefore \frac{D_f}{B} = \frac{5}{2.81} = 1.78' > 1$$ (OK) $\therefore$ size of footing $= (2.81' \times 2.81')$

(iii) When $h=5$: $q = \gamma D_f = (100\times5) = 500\ lb/ft^2$

Assume, $\frac{D_f}{B} \leq 1$

$$\therefore \frac{40000}{B^2} = \frac{1}{3}\left[500\times18.40\times1.577\times\left(1+\frac{1.4439}{B}\right)\right] + 129.024\,B$$

$$\Rightarrow \frac{40000}{B^2} = 4836.13 + \frac{6980.475}{B} + 129.024\,B$$

$$\Rightarrow B = 2.195$$

$$\therefore \frac{D_f}{B} = \frac{5}{2.195} = 2.28 > 1 \text{ (not OK)}$$

considering $\frac{D_f}{B} > 1$

$$\frac{40000}{B^2} = 4836.13 + 1402.4777\,\tan^{-1}\left(\frac{5}{B}\right) + 129.024\,B$$

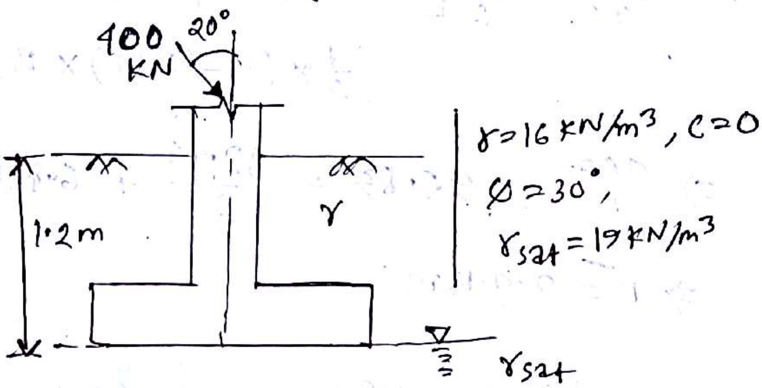
$$\Rightarrow B = 2.44\ ft$$

$$\therefore \frac{D_f}{B} = \frac{5}{2.44} = 2.05 > 1 \quad \text{(OK)}$$

$\therefore$ size of footing = $(2.44' \times 2.44')$

Problem: 03 A square footing shown in figure below: Use F.S. = 6. Determine the size of footing.

Solution:



$$q_{all} = \frac{Q_{all}}{B \times L} = \frac{400}{B^2} \quad \ldots\ldots (I)$$

again,

$$q_{all} = \frac{q_u}{F.S.}$$

$$\Rightarrow q_{all} = \frac{1}{6}\left[q \times N_q \cdot F_{qs} \cdot F_{qd} \cdot F_{qi} + \frac{1}{2} \times \gamma' \times B \times N_\gamma \cdot F_{\gamma s} \cdot F_{\gamma d} \cdot F_{\gamma i}\right] \ldots (II)$$

$$N_q = \tan^2\left(45 + \frac{30}{2}\right) \times e^{\pi \tan 30} = 18.40$$

$$N_\gamma = 2(N_q + 1)\tan\phi = 2 \times (18.40 + 1) \times \tan 30 = 22.40$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi = 1 + 1 \times \tan 30° = 1.577$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = 0.6$$

$$F_{qi} = \left(1 - \cancel{B}/90\right)^2 = \left(1 - \frac{20}{90}\right)^2 = 0.605$$

$$F_{\gamma i} = \left(1 - \frac{B}{\phi}\right)^2 = \left(1 - \frac{20}{30}\right)^2 = 0.111$$

Assume, $\frac{D_f}{B} \leq 1$.

$$F_{qd} = 1 + 2 \times \tan 30 \times (1 - \sin 30)^2 \times \frac{1.2}{B} = 1 + \frac{0.3464}{B}$$

$$F_{\gamma d} = 1$$

Here, $q = \gamma D_f = (16 \times 1.2) = 19.2 \ KN/m^2$

Now,

From eqn (i) & (ii)

$$\frac{400}{B^2} = \frac{1}{6}\left[19.2 \times 18.40 \times 1.577 \times \left(1+\frac{0.3464}{B}\right) \times 0.605 + \frac{1}{2} \times (19-9.81) \times B \times 22.40 \times 0.6 \times 1 \times 0.111\right]$$

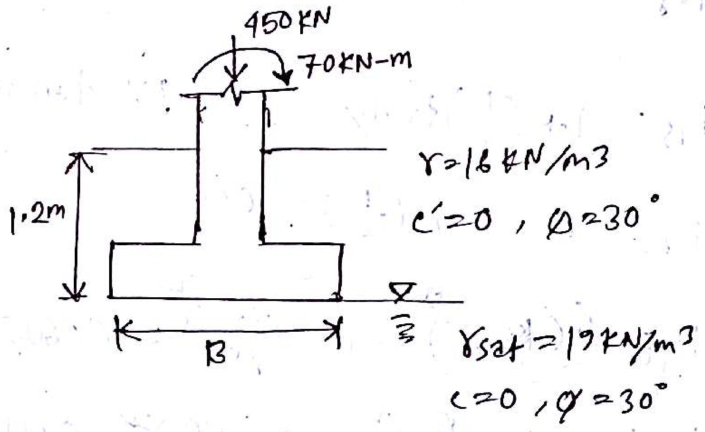
$$\Rightarrow \frac{400}{B^2} = 56.17 + \frac{19.46}{B} + 1.1425B$$

$$\Rightarrow B = 2.445m$$

$$\therefore \frac{D_f}{B} = \frac{1.22}{2.445} = 0.5 < 1 \quad \text{(OK)}$$

$\therefore$ size of footing = (2.445m × 2.445m)

Problem: 04 A square footing is shown in figure below. Use F.S. = 6 Determine the size of footing.



Solution: $M = Q \times e$

$$\Rightarrow e = \frac{M}{Q} = \frac{70}{450} = 0.16m$$

$$\therefore B' = (B-2e) = (B - 2\times0.16)$$

$$\Rightarrow B' = B - 0.32$$

and $L' = L = B$

$$q = \gamma D_f = (16 \times 1.2) = 19.2 \text{ KN/m}^2$$

$$N_q = \tan^2\left(45+\frac{30}{2}\right) \times e^{\pi \times \tan 30^\circ} = 18.40$$

$$N_\gamma = 2 \times (18.40+1) \times \tan 30^\circ = 22.40$$

$$F_{qs} = 1 + \left(\frac{B'}{L'}\right) \times \tan 30^\circ = 1 + \left(\frac{B-0.32}{B}\right) \times \frac{1}{\sqrt{3}}$$

$$F_{\gamma s} = 1 - 0.4 \times \left(\frac{B'}{L'}\right) = 1 - 0.4 \times \left(\frac{B-0.32}{B}\right)$$

Assume; $\frac{D_f}{B} \leq 1$; $F_{qd} = 1 + 2 \times \tan 30° \times (1-\sin 30°)^2 \times \frac{1.2}{B}$

no effect on depth factor

$$\Rightarrow F_{qd} = 1 + \frac{0.3464}{B} \quad \text{and } F_{\gamma d} = 1$$

Now,

$$q_{all} = \frac{Q_{all}}{B'L'} = \frac{450}{(B-0.32) \times B}$$

again, $q_{all} = \frac{q_u}{F.S} = \frac{1}{F.S} \times \left[q N_q F_{qs} F_{qd} + \frac{1}{2}\gamma' B' N_\gamma F_{\gamma s} F_{\gamma d}\right]$

$$\Rightarrow \frac{450}{(B-0.32) \times B} = \frac{1}{6} \times \left[19.2 \times 18.40 \times \left(1 + \frac{0.3464}{B}\right) \times \left\{1 + \left(\frac{B-0.32}{B}\right) \times \frac{1}{\sqrt{3}}\right\} + \frac{1}{2} \times (19-9.81) \times (B-0.32) \times 22.40 \times \left\{1 - 0.4 \times \frac{B-0.32}{B}\right\} \times 1\right]$$

$$\Rightarrow \frac{450 \times 6}{B^2 - 0.32B} = 353.28 \times \left(1 + \frac{0.3464}{B}\right) \times \left\{1 + \left(\frac{B-0.32}{\sqrt{3}B}\right)\right\} + 102.928 \times (B-0.32) \times \left\{1 - \left(\frac{0.4B - 0.128}{B}\right)\right\}$$

$$\Rightarrow \frac{2700}{B^2 - 0.32B} = \left(353.28 + \frac{122.4}{B}\right) \times \left(1 + \frac{B-0.32}{\sqrt{3}B}\right) + (102.928B - 32.94) \times \left\{1 - \frac{0.4B - 0.128}{B}\right\}$$

$$\Rightarrow B = 1.98 \text{ m}$$

$$\therefore \frac{D_f}{B} = \frac{1.2}{1.98} = 0.606 < 1 \quad \text{(OK)}$$

$\therefore$ size of the footing $= (1.98 \text{ m} \times 1.98 \text{ m})$

(Ans.)

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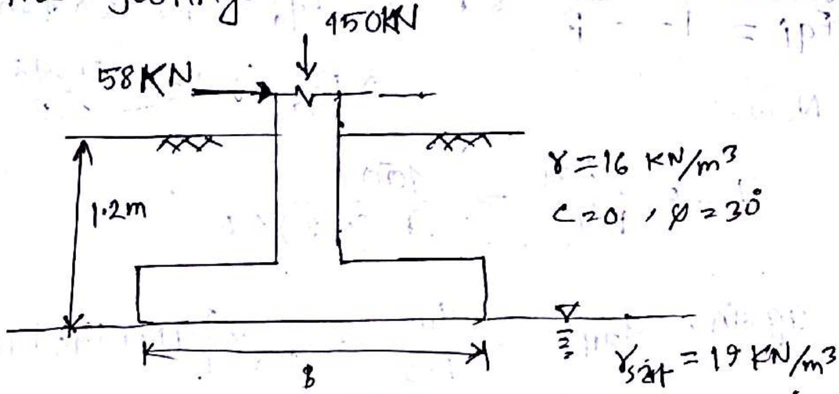
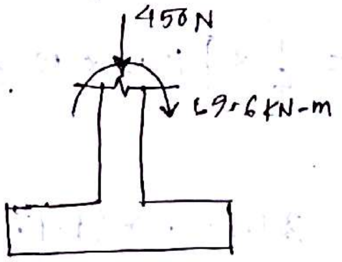
Problem: 05 A square footing is shown in figure below: Use F.S. = 6 and determine the size of the footing.



Solution:



Here, $M = (58 \times 1.2) = 69.6$

$$e = \frac{M}{Q} = \frac{69.6}{450} = 0.155$$

$$B' = B - 2e = B - 0.31$$

and, $L' = L = B$

$$q_{all} = \frac{Q_{all}}{B' \times L'} = \frac{450}{(B-0.31) \times B} \quad \ldots\ldots (i)$$

again, $$q_{all} = \frac{q_u}{F.S.} = \frac{1}{6} \times \left[q N_q F_{qs} F_{qd} + \frac{1}{2} \times \gamma' \times B' N_\gamma F_{\gamma s} F_{\gamma d} \right] \ldots\ldots (ii)$$

Here,

$$q = \gamma D_f = (16 \times 1.2) = 19.2 \text{ KN/m}^2$$

$$N_q = \tan^2\left(45 + \frac{30}{2}\right) e^{\pi \tan 30} = 18.40$$

$$N_\gamma = 2 (N_q + 1) \tan 30° = 20.40$$

$$F_{qs} = 1 + \left(\frac{B'}{L'}\right) \times \tan 30° = 1 + \left(\frac{B-0.31}{B}\right) \times \frac{1}{\sqrt{3}} = 1 + \frac{1}{\sqrt{3}} - \frac{0.31}{\sqrt{3} B}$$

$$\therefore F_{qs} = \left(1.577 - \frac{0.179}{B}\right)$$

$$F_{\gamma s} = 1 - 0.4 \left(\frac{B'}{L'}\right) = 1 - 0.4 \times \frac{(B-0.31)}{B} = 1 - 0.4 + \frac{0.31 \times 0.4}{B}$$

$$\therefore F_{\gamma s} = \left(0.6 + 0.124/B\right)$$

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Assume, $\frac{D_f}{B} \leq 1$

$\therefore F_{qd} = 1 + 2\tan 30^\circ (1-\sin 30^\circ)^2 \times \frac{1.2}{B} = 1 + \frac{0.3464}{B}$

$F_{\gamma d} = 1$

Hence, from eqn ① & (ii),

$$\frac{450}{B^2 - 0.31B} = \frac{1}{6}\Big[19.2 \times 18.40 \times \left(1.577 - \frac{0.179}{B}\right) \times \left(1 + \frac{0.3464}{B}\right) + \frac{1}{2} \times (19 - 9.81) \times (B - 0.31) \times 20.40 \times \left(0.6 + \frac{0.124}{B}\right) \times 1\Big]$$

$$\Rightarrow \frac{450}{B^2 - 0.31B} = 58.88 \times \left(1.577 - \frac{0.179}{B}\right) \times \left(1 + \frac{0.3464}{B}\right) + 15.023 \times (B - 0.31) \times \left(0.6 + \frac{0.124}{B}\right)$$

$\Rightarrow B = 2.092\ m$

$\therefore \frac{D_f}{B} = \frac{1.2}{2.092} = 0.57 < 1$ (ok)

$\therefore$ size of the footing = $(2.092\ m \times 2.092\ m)$

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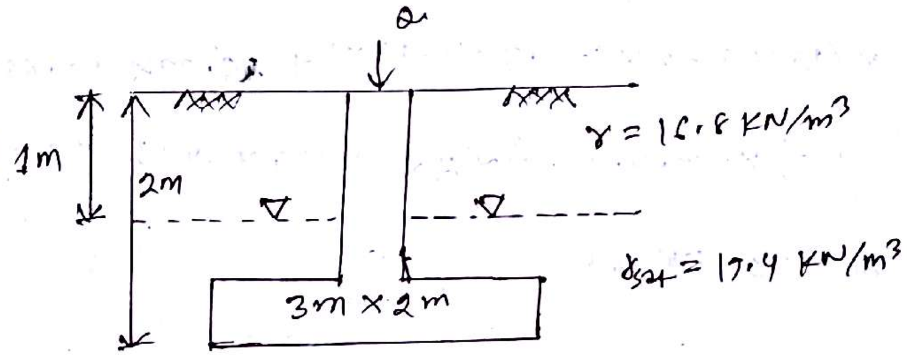
2010, 2011

A column foundation is 3m × 2m in plan. Given, $D_f = 2$ m, $\phi = 25°$ and $c = 50$ KN/m². Using F.S = 4. Determine the net allowable load the foundation could carry.



Solution: Here, $B = 2$ m, $L = 3$ m

We know,

$$q_{all} = \frac{q_u}{F.S.} = \frac{1}{4} \times \left(c N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma' B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \right) \quad \text{......} (1)$$

Here, $c = 50$ KN/m², $q = D_1 \gamma + D_2 \gamma' = 1 \times 16.8 + (2-1) \times (19.4 - 9.81) = 26.39$ KN/m²

$$N_q = \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi \tan\phi} = \tan^2\left(45 + \frac{25}{2}\right) \times e^{\pi \times \tan 25} = 10.662$$

$$N_c = (N_q - 1)\cot\phi = (10.662 - 1) \times \cot 25 = 20.72$$

$$N_\gamma = 2(N_q + 1) \cdot \tan\phi = 2 \times (10.662 + 1) \cdot \tan 25 = 9.944$$

$$F_{cs} = 1 + \left(\frac{B}{L}\right) \times \frac{N_q}{N_c} = 1 + \left(\frac{2}{3}\right) \times \frac{10.662}{20.72} = 1.343$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right) \times \tan\phi = 1 + \left(\frac{2}{3}\right) \times \tan 25 = 1.311$$

$$F_{\gamma s} = 1 - 0.4 \times \left(\frac{B}{L}\right) = 1 - 0.4 \times \frac{2}{3} = 0.733$$

Here, $\frac{D_f}{B} = \frac{2}{2} = 1$ and $\phi = 25°$

$$\therefore F_{qd} = 1 + 2\tan\phi(1 - \sin\phi)^2 \times \left(\frac{D_f}{B}\right) = 1 + 2 \times \tan 25 \times (1 - \sin 25)^2 \times \left(\frac{2}{2}\right)$$

$$\therefore F_{qd} = 1.311$$

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$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi} = 1.311 - \frac{1 - 1.311}{20.72 \times \tan 25} = 1.343$$

$F_{\gamma d} = 1$

Here,
$\beta = 0 \quad \therefore F_{qi} = F_{ci} = F_{\gamma i} = 1$

From eqn (1) we obtain,

$$\therefore q_{all} = \frac{1}{4} \times (50 \times 20.72 \times 1.343 \times 1.343 \times 1 + 26.39 \times 10.662 \times 1.311 \times 1.311 \times 1$$
$$+ \frac{1}{2} \times (19.4 - 9.81) \times 2 \times 9.944 \times 0.733 \times 1 \times 1)$$

$\therefore q_{all} = 605.52 \ KN/m^2$

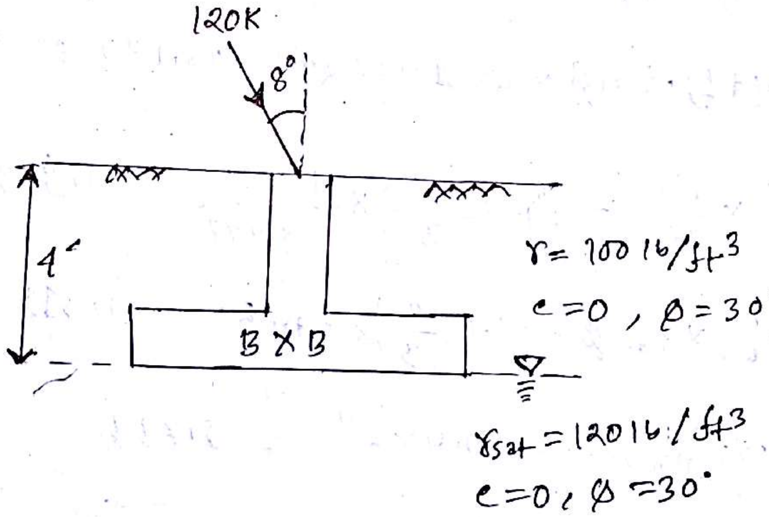
Again, we know, $q_{all} = \frac{Q_{all}}{B \times L}$

$\therefore Q_{all} = (B \times L \times q_{all}) = (2 \times 3 \times 605.52) = 3633.12 \ KN$

(Ans.)

2011

A square footing is shown in figure below. Using a F.S of 6. Determine the size of footing:



Solution:

$$q_{all} = \frac{Q_{all}}{B \times L} = \frac{120\times10^3}{B\times B} = \frac{120\times10^3}{B^2} \quad \text{---- (i)}$$

again,

$$q_{all} = \frac{q_u}{F.S.} = \frac{1}{6}\times\left(q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2}\gamma' B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}\right) \quad \text{---- (ii)}$$

$$q = \gamma D_f = (100\times4) = 400 \text{ lb/ft}^2$$

$$N_q = \tan^2\left(45+\frac{\phi}{2}\right)\times e^{\pi\tan\phi} = \tan^2\left(45+\frac{30}{2}\right)\times e^{\pi\tan30^\circ} = 18.40$$

$$N_\gamma = 2(N_q+1)\tan\phi = 2\times(18.40+1)\times\tan30^\circ = 22.40$$

$$F_{qs} = 1+\left(\frac{B}{L}\right)\tan\phi = 1+\left(\frac{B}{B}\right)\times\tan30^\circ = 1.577$$

$$F_{\gamma s} = 1-0.4\left(\frac{B}{L}\right) = 0.6$$

$$F_{qi} = \left(1-\frac{\beta}{90}\right)^2 = 0.83$$

$$F_{\gamma i} = \left(1-\frac{\beta}{\phi}\right)^2 = \left(1-\frac{8}{30}\right)^2 = 0.54$$

Assume, $\frac{D_f}{B} \le 1$

$$\therefore F_{qd} = 1+2\tan30\times(1-\sin30^\circ)^2\times\frac{4}{B} = 1+\frac{1.155}{B}$$

$$F_{\gamma d} = 1$$

Now,

From eq^n (i) & (ii)

$$\frac{120\times10^3}{B^2} = \frac{1}{6}\times\left[400\times18.40\times\left(1+\frac{1.155}{B}\right)\times0.83\times1.577+\frac{1}{2}\times(100-62.4)\times B\times22.40\times0.6\times0.54\times1\right]$$

$$\Rightarrow \frac{120\times10^3}{B^2} = 1605.6+\frac{1854.5}{B}+136.44B$$

$$\therefore B = 6.57 \text{ ft} \qquad \therefore \frac{D_f}{B} = \frac{4}{6.57} = 0.61 < 1 \text{ (ok)}$$

$\therefore$ size of footing = (6.57×6.57) (Ans)

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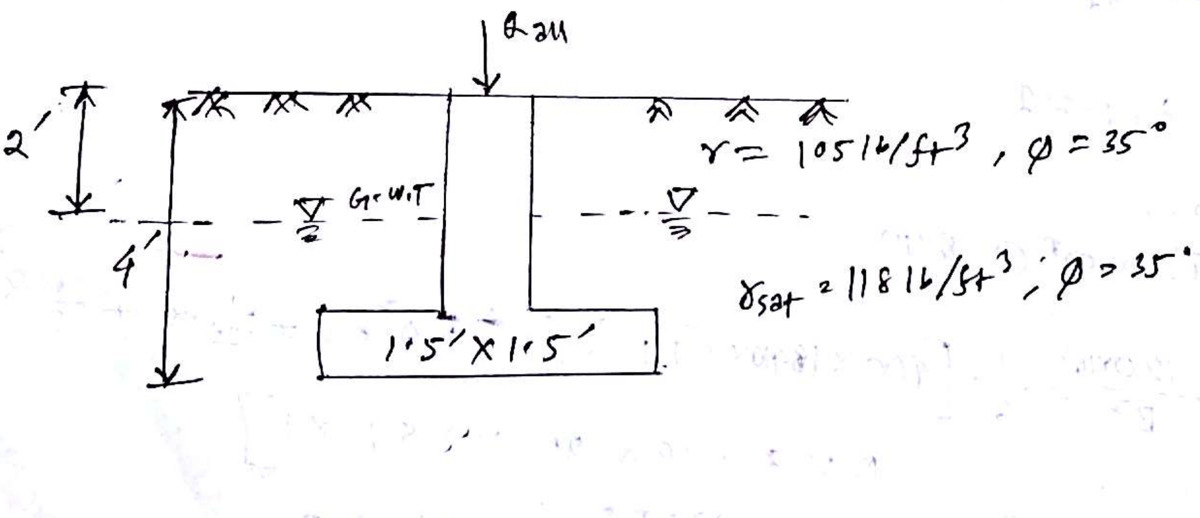
2012

A square foundation is 4.5 ft × 4.5 ft in plan. Given $D_f = 4$ ft, $\phi = 35°$ and $c = 0$, using a F.S. of 3. Determine the gross allowable load the foundation could carry.



Solution:

We know, $q_{all} = \frac{Q_{all}}{B \times L} = \frac{Q_{all}}{1.5 \times 1.5} = \frac{Q_{all}}{2.25}$ ..... ①

and, $q_{all} = \frac{q_u}{F.S.} = \frac{1}{3} \times \left(q N_q F_{qs} F_{qd} + \frac{1}{2} \gamma' N_\gamma B F_{\gamma s} F_{\gamma d} \right)$

Here,

$q = 2 \times 105 + (4-2) \times (118 - 62.4) = 321.2 \ lb/ft^2$

$N_q = \tan^2\left(45 + \frac{35}{2}\right) e^{\pi \tan 35} = 33.3$

$N_\gamma = 2(N_q + 1) \tan\phi = 2 \times (33.3 + 1) \times \tan 35 = 48.034$

$F_{qs} = 1 + \left(\frac{B}{L}\right) \times \tan\phi = 1 + \frac{1.5}{1.5} \times \tan 35^\circ = 1.7$

$F_{\gamma s} = 1 - 0.4 \times \left(\frac{B}{L}\right) = 1 - 0.4 \times \frac{1.5}{1.5} = 0.6$

Here, $\frac{D_f}{B} = \frac{4}{1.5} = 2.67 > 1$ and $\phi = 35^\circ$

$\therefore F_{qd} = 1 + 2 \tan\phi (1 - \sin\phi)^2 \tan^{-1}\left(\frac{D_f}{B}\right)$

$= 1 + 2 \tan 35 \times (1 - \sin 35)^2 \times \underbrace{\tan^{-1}\left(\frac{4}{1.5}\right)}_{\text{radien}}$

$= 1.31$

$F_{\gamma d} = 1$

$\therefore q_{all} = \frac{1}{3} \times \left[(321.2 \times 33.3 \times 1.7 \times 1.31 + \frac{1}{2} \times (118 - 62.4) \times 1.5 \times 48.034 \times 0.6 \times 1 \right]$

$= 8340.57 \ lb/ft^2$

$\therefore$ From eqn ①, $Q_{all} = (q_{all} \times 2.25) = (2.25 \times 8340.57)$

$= 18766.3 \ lb$

$\therefore Q_{all} = 18.77 K$ (Ans.)

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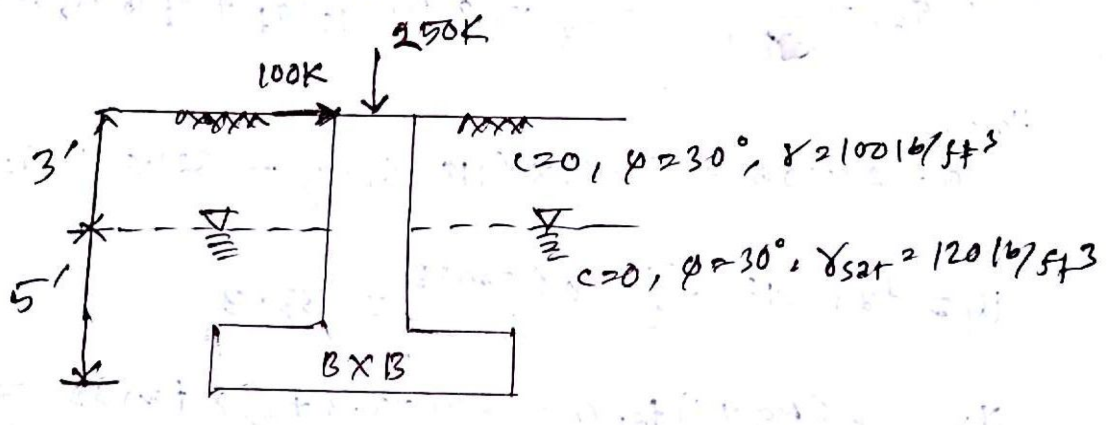
2013

\# A square footing is shown in figure below, using a FS of 4. Determine the size of the footing.



Solution:

$$M = (100 \times 8) = 800 \text{ K ft}$$

$$e = \frac{M}{Q} = \frac{800}{250} = 3.2$$

$$B' = B - 2e = (B - 2\times 3.2) = B - 6.4$$

and $L' = L = B$. Now, $q_{all} = \frac{Q_{all}}{B' \times L'} = \frac{250,000}{(B-6.4)\times B}$ ....(i)

$$q_{all} = \frac{q_u}{F.S.} = \frac{1}{4} \times \left[q N_q F_{qs} F_{qd} + \frac{1}{2} \times \gamma' \times B' \times N_\gamma F_{\gamma s} F_{\gamma d} \right] \text{....(ii)}$$

$$\therefore q = D_1\gamma + D_2\gamma' = (3\times 100) + 5\times(120 - 62.4) = 588 \text{ lb/ft}^2$$

$$N_q = \tan^2\left(45 + \frac{30}{2}\right) e^{\pi \tan 30} = 18.40$$

$$N_\gamma = 2(N_q + 1)\tan 30° = 20.40$$

$$F_{qs} = 1 + \left(\frac{B'}{L'}\right) \times \tan 30° = 1 + \left(\frac{B - 6.4}{B}\right) \times \frac{1}{\sqrt{3}} = 1 + \frac{1}{\sqrt{3}} - \frac{6.2}{\sqrt{3}B}$$

$$\therefore F_{qs} = 1.577 - \frac{3.58}{B}$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B'}{L'}\right) = 1 - 0.4 \times \left(\frac{B-6.4}{B}\right) = 0.6 + \frac{2.56}{B}$$

Assume, $\frac{D_f}{B} \leq 1$

$$F_{qd} = 1 + 2\tan 30^\circ (1-\sin 30^\circ)^2 \times \frac{8}{B} = 1 + \frac{2.31}{B}$$

$$F_{\gamma d} = 1$$

Hence,

From eq^n (i) & (ii),

$$\frac{250000}{(B-6.4)\times B} = \frac{1}{4}\times\left[588\times 18.40\times\left(1.577-\frac{3.58}{B}\right)\times\left(1+\frac{2.31}{B}\right) + \frac{1}{2}\times(120-62.4)\times(B-6.4)\times 20.40\times\left(0.6+\frac{2.56}{B}\right)\times 1\right]$$

$$\Rightarrow \frac{250000}{(B-6.4)\times B} = \left[2704.8\times\left(1.577-\frac{3.58}{B}\right)\times\left(1+\frac{2.31}{B}\right) + 587.52\times(B-6.4)\times\left(0.6+\frac{2.56}{B}\right)\right]$$

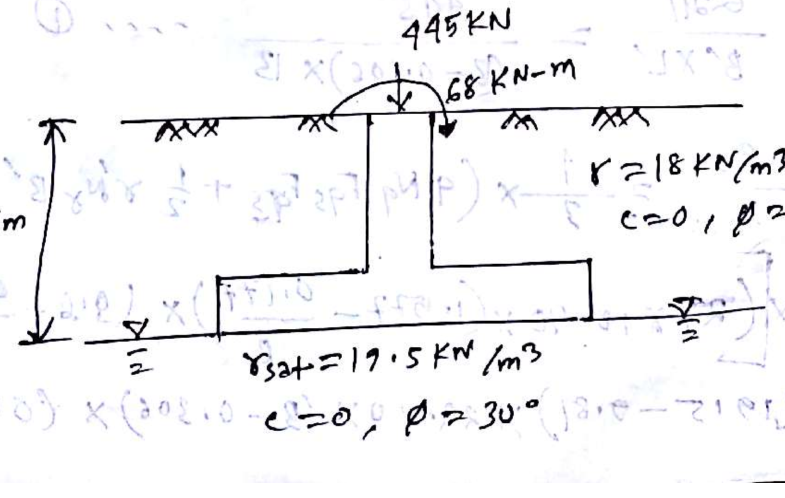
$\Rightarrow B = 10.38$ ft $\qquad \therefore \frac{D_f}{B} = \frac{8}{10.38} = 0.77 < 1$ (OK)

$\therefore$ The size of the footing = $(10.38' \times 10.38')$

(Ans.)

2014

\# A square footing is shown in figure below. Using a factor of safety of 3, determine the size of the footing.



Solution:

$$e = \frac{M}{Q} = \frac{68}{445} = 0.153$$

$$\therefore B' = B - 2e = B - (2 \times 0.153) = B - 0.306$$

and, $L' = L = B$

$$q = \gamma D_f = (18 \times 1.5) = 27 \text{ KN/m}^2$$

$$N_q = \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi \tan\phi} = \tan^2\left(45 + \frac{30}{2}\right) e^{\pi \tan 30} = 18.40$$

$$N_\gamma = 2 \times (N_q + 1) \times \tan\phi = 2 \times (18.40 + 1) \times \tan 30 = 22.40$$

$$F_{qs} = 1 + \left(\frac{B'}{L'}\right) \tan\phi = 1 + \left(\frac{B - 0.306}{B}\right) \times \tan 30° = 1 + 0.577 - \frac{0.177}{B}$$

$$\therefore F_{qs} = 1.577 - \frac{0.177}{B}$$

$$F_{\gamma s} = 1 - 0.4 \times \left(\frac{B'}{L'}\right) = 1 - 0.4 \times \frac{B - 0.306}{B} = 0.6 - \frac{0.1224}{B}$$

Assume, $\frac{D_f}{B} \leq 1$

$$\therefore F_{qd} = 1 + 2\tan\phi\,(1 - \sin\phi)^2 \times \left(\frac{D_f}{B}\right) = 1 + 2\tan 30° \times (1 - \sin 30)^2 \times \frac{1.5}{B}$$

$$\therefore F_{qd} = 1 + \frac{0.433}{B}$$

$$F_{\gamma d} = 1$$

Now, $$q_{all} = \frac{Q_{all}}{B' \times L'} = \frac{445}{(B - 0.306) \times B} \quad \cdots\cdots (1)$$

and, $$q_{all} = \frac{q_u}{F_s} = \frac{1}{3} \times \left(q N_q F_{qs} F_{qd} + \frac{1}{2} \gamma' N_\gamma B' F_{\gamma s} F_{\gamma d}\right)$$

$$\Rightarrow q_{all} = \frac{1}{3} \times \left[\left(27 \times 18.40 \times \left(1.577 - \frac{0.177}{B}\right) \times \left(1.0 + \frac{0.433}{B}\right) + \frac{1}{2} \times (19.5 - 9.81) \times 22.40 \times (B - 0.306) \times \left(0.6 - \frac{0.1224}{B}\right) \times 1\right]$$

$\cdots\cdots (11)$

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From (i) & (ii) we obtain,

$$\frac{445}{B^2 - 0.306B} = \Big[165.6 \times \left(1.577 - \frac{0.177}{B}\right) \times \left(1.0 - \frac{0.433}{B}\right) + 36.176 \times (B - 0.306) \times \left(0.6 - \frac{0.1224}{B}\right)\Big]$$

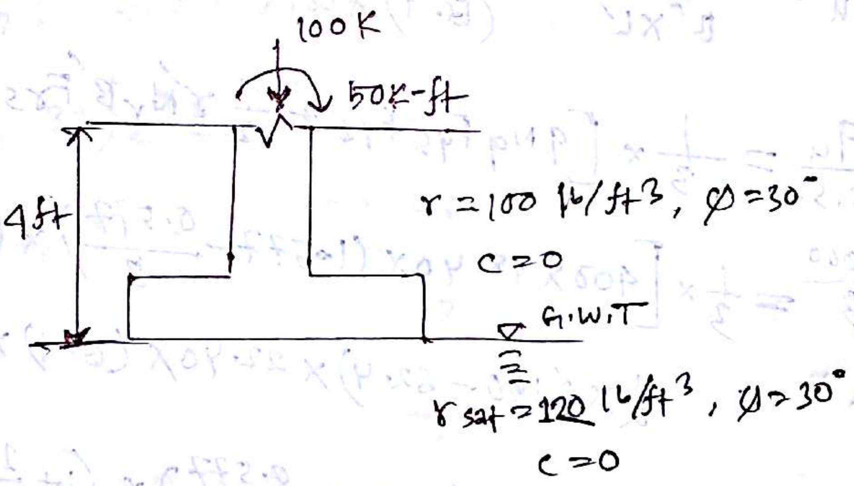
$$\Rightarrow B = 1.636 \text{ m}$$

$$\frac{D_f}{B} = \frac{1.5}{1.636} = 0.917 < 1 \quad (\text{OK})$$

$\therefore$ size of the footing $= (1.636\text{m} \times 1.636\text{ m})$ (Ans)

2015, 2017

A square footing is shown in figure below. Use an F.S of 3 and determine the size of the footing.



Solution:

$$e = \frac{M}{Q} = \frac{50}{100} = 0.5$$

$$\therefore B' = B - 2e = B - (2 \times 0.5) = (B - 1)$$

and $L' = L = B$

$$q = \gamma D_f = (100 \times 4) = 400 \text{ lb/ft}^3$$

$N_q = \tan^2\left(45 + \frac{30}{2}\right) e^{\pi \tan 30^\circ} = 18.40$

$N_\gamma = 2 \times (18.40 + 1) \times \tan 30^\circ = 22.40$

$F_{qs} = \left[1 + \left(\frac{B'}{L'}\right)\tan\phi\right] = 1 + \frac{B-1}{B} \times \tan 30^\circ = 1 + 0.577 - \frac{0.577}{B}$

$\therefore F_{qs} = 1.577 - \frac{0.577}{B}$

$F_{\gamma s} = 1 - 0.4 \times \left(\frac{B'}{L'}\right) = 1 - 0.4 \times \left(\frac{B-1}{B}\right) = 1 - 0.4 + \frac{0.4}{B} = \left(0.6 + \frac{0.4}{B}\right)$

Assume, $\frac{D_f}{B} \le 1$

$\therefore F_{qd} = 1 + 2\tan 30^\circ \times (1 - \sin 30^\circ)^2 \times \frac{4}{B} = 1 + \frac{1.155}{B}$

$F_{\gamma d} = 1$

Now, $q_{all} = \frac{Q_{all}}{B' \times L'} = \frac{100000}{(B-1) \times B}$ ——— ①

again,

$q_{all} = \frac{q_u}{F.S.} = \frac{1}{3} \times \left[q N_q F_{qs} F_{qd} + \frac{1}{2}\gamma N_\gamma B' F_{\gamma s} F_{\gamma d}\right]$

$\Rightarrow \frac{100000}{B^2 - B} = \frac{1}{3} \times \left[400 \times 18.40 \times \left(1.577 - \frac{0.577}{B}\right) \times \left(1 + \frac{1.155}{B}\right) + \frac{1}{2} \times (120 - 62.4) \times 22.40 \times (B-1) \times \left(0.6 + \frac{0.4}{B}\right) \times 1\right]$

$\Rightarrow \frac{100000}{B^2 - B} = 2453.33 \times \left(1.577 - \frac{0.577}{B}\right) \times \left(1 + \frac{1.155}{B}\right) + 215.04 \times (B-1) \times \left(0.6 + \frac{0.4}{B}\right)$

$\Rightarrow B = 5.00$ ft

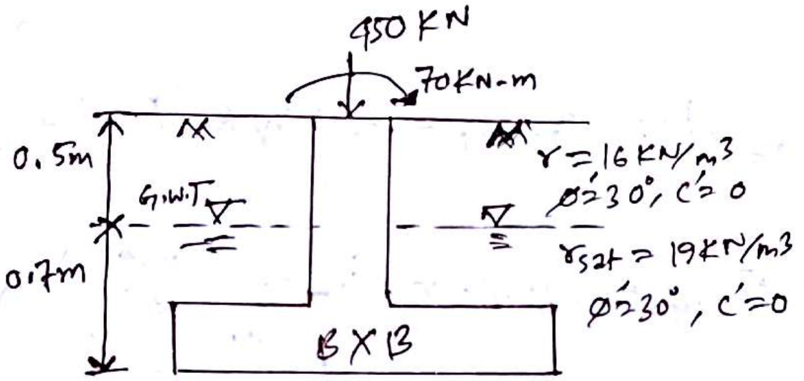
$\therefore \frac{D_f}{B} = \frac{4}{5} = 0.8 < 1$ (ok)

$\therefore$ size of the footing $= (5' \times 5')$

(Ans.)

2016

A square footing is shown in figure below. Use $FS = 3$ and determine the size of footing.



Solution: $e = \frac{M}{Q} = \frac{70}{450} = 0.156$

$\therefore B' = (B - 2e) = (B - 2 \times 0.156) = (B - 0.312)$

and, $L' = L = B$

$q = D_1 \gamma + D_2 \gamma' = 0.5 \times 16 + 0.7 \times (19 - 9.81) = 14.433$

$N_q = \tan^2(45 + \frac{30}{2}) e^{\pi \tan 30°} = 18.40$

$N_\gamma = 2 \times (18.40 + 1) \tan 30 = 22.40$

$F_{qs} = 1 + (\frac{B'}{L'}) \times \tan\phi' = 1 + \frac{B - 0.312}{B} \times \tan 30° = (1.577 - \frac{0.492}{B})$

$F_{\gamma s} = 1 - 0.4 \times (\frac{B'}{L'}) = 1 - 0.4 \times (\frac{B - 0.312}{B}) = (0.6 + \frac{0.1248}{B})$

Assume, $\frac{D_f}{B} \leq 1$

$\therefore F_{qd} = 1 + 2 \tan 30 (1 - \sin 30)^2 \times \frac{1.2}{B} = (1 + \frac{0.3464}{B})$

$F_{\gamma d} = 1$

(* Exam-এ calculation করে B এর মান বের করতে হবে, তা না হলে মার্ক কাটতে পারে)

Now,

$$q_{all} = \frac{Q_{all}}{B' \times L'} = \frac{450}{(B-0.312)\times B} \quad \text{------} \; (1)$$

again,

$$q_{all} = \frac{q_u}{F.S.} = \frac{1}{3}\times\left[q N_q F_{qs} F_{qd} + \frac{1}{2}\gamma' N_\gamma B' F_{\gamma s} F_{\gamma d}\right]$$

From (1),

$$\frac{450}{(B-0.312)\times B} = \frac{1}{3}\times\left[19.433 \times 18.40 \times \left(1.577 - \frac{0.492}{B}\right)\times\left(1+\frac{0.3464}{B}\right) + \frac{1}{2}\times(19-9.81)\times 22.40 \times (B-0.324)\times\left(0.6+\frac{0.1248}{B}\right)\times 1\right]$$

$$\Rightarrow \frac{450\times 3}{(B-0.312)B} = 265.5672\times\left(\frac{1.577B-0.492}{B}\right)\times\left(\frac{B+0.3464}{B}\right) + 102.928 \times (B-0.324)\times\left(\frac{0.6B+0.1248}{B}\right)$$

$$\Rightarrow \frac{1350}{(B-0.312)B} = \frac{265.5672(1.577B^2-0.0543B-0.17)}{B^2} + \frac{102.928}{B}\times(0.6B^2-0.0696B-0.04)$$

$$= \frac{1350}{(B-0.312)} = \frac{418.8B^2-14.42B-45.15}{B} + 61.7568B^2-7.164B-4.12$$

$$\Rightarrow 1350B = (B-0.312)(418.8B^2-14.42B-45.15+61.7568B^3-7.164B^2-4.12B)$$

$$\Rightarrow 1350B = (B-0.312)(61.7568B^3+411.636B^2-18.54B-45.15)$$

$$\Rightarrow 1350B = 61.7568B^4+411.636B^3-18.54B^2-45.15B-19.27B^3-128.43B^2+5.7845B+14.0868$$

$$\Rightarrow 61.7568B^4+392.366B^3-146.97B^2-1389.3655B+14.0868=0$$

$\therefore B = 1.8\,m$ $\quad \therefore \frac{D_f}{B} = \frac{1.2}{1.8} = 0.67 < 1$ (OK) $\quad \therefore$ size of footing $= (1.8\,m \times 1.8\,m)$

(Ans.)

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Formula:

Settlement of Shallow Foundation:

Elastic settlement, $S_e = \frac{Bq_o}{E_s}(1-\mu_s^2)\alpha$

where, $\alpha = \frac{1}{\pi}\left[\ln\frac{\sqrt{m^2+1}+m}{\sqrt{m^2+1}-m} + m\ln\frac{\sqrt{m^2+1}+1}{\sqrt{m^2+1}-1}\right]$

$m = \frac{L}{B}$

Special cases: (2010)

[redacted], $S_e = \frac{Bq_o}{E_s}(1-\mu_s^2)\left[(1-\mu_s^2)F_1 + (1-\mu_s-2\mu_s^2)F_2\right]$

Where,

$F_1 = \frac{1}{\pi}(A_o + A_1)$ Here, $A_o = m\ln\left[\frac{(1+\sqrt{m^2+1})\times\sqrt{m^2+n^2}}{m\times(1+\sqrt{1+m^2+n^2})}\right]$

$F_2 = \frac{n}{2\pi}\tan^{-1}A_2$ $\quad A_1 = \ln\left[\frac{(m+\sqrt{m^2+1})\sqrt{1+n^2}}{m+\sqrt{1+m^2+n^2}}\right]$

$A_2 = \frac{m}{n(\sqrt{m^2+n^2+1})}$

Primary consolidation settlement,

$$S_c(p) = \frac{C_c H}{1+e_o}\times\log\left(\frac{\sigma_o' + \sigma_{av}'}{\sigma_o'}\right)$$

where, $C_c = 0.009\,[LL-100]$

$$\sigma_{av}' = \frac{\sigma_t' + 4\sigma_m' + \sigma_b'}{6}$$

* $I_c = \frac{2}{\pi}\times\left[\frac{mn}{\sqrt{m^2+n^2+1}}\times\frac{m^2+2n^2+1}{(m^2+n^2)(1+n^2)} + \sin^{-1}\left(\frac{m}{\sqrt{1+n^2}\times\sqrt{m^2+n^2}}\right)\right]$

radian mode

Secondary consolidation settlement,

$S_c(s) = C_\alpha' H\log\left(\frac{t_2}{t_1}\right)$ where, $C_\alpha' = \frac{C_\alpha}{1+e_p}$

$e_p = (e_o - \Delta e)$

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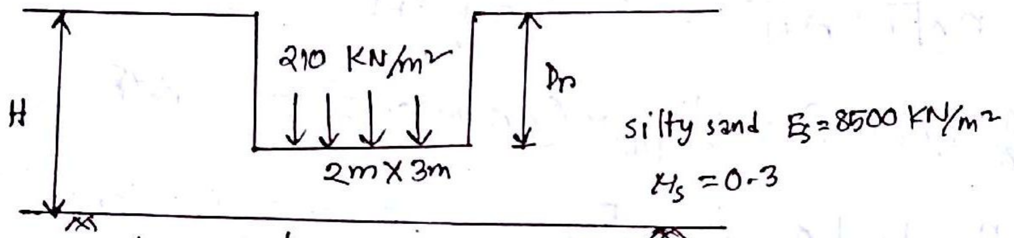
Shallow Foundation and Its Settlement

2010, 2011, 2012

A flexible area shown in figure below is 2m × 3m in plan and carries a uniformly distributed load of 210 kN/m². Estimate the elastic settlement below the center of the loaded area.



Assume (i) $H = \alpha$, $D_n = 0$

(ii) $H = 5m$, $D_n = 0$

Solution: Given, $L = 3m$, $B = 2m$, $q_o = 210$ KN/m², $E_s = 8500$ KN/m²; $\mu_s = 0.3$

(i) $H = \alpha$, $D_n = 0$: We know, $S_e = \frac{Bq_o}{E_s}(1-\mu_s^2)\,\alpha$

Here, $\alpha = \frac{1}{\pi}\left[m \ln \frac{\sqrt{m^2+1}+1}{\sqrt{m^2+1}-1} + \ln \frac{\sqrt{m^2+1}+m}{\sqrt{m^2+1}-m}\right]$

Now,

$$m = \frac{L}{B} = \frac{3}{2} = 1.5$$

$$\therefore \alpha = \frac{1}{\pi} \times \left[1.5 \times \ln \frac{\sqrt{1.5^2+1}+1}{\sqrt{1.5^2+1}-1} + \ln \frac{\sqrt{1.5^2+1}+1.5}{\sqrt{1.5^2+1}-1.5}\right]$$

$$= 1.36$$

$$\therefore S_e = \frac{2 \times 210}{8500} \times \left(1-(0.3)^2\right) \times 1.36 = 0.06115 \text{ m}$$

$$\therefore \boxed{S_e = 61.15 \text{ mm}}$$

(ii) $H = 5m$, $D_n = 0$: We know,

$$S_e = \frac{Bq_o}{E_s}(1-\mu_s^2)\left[(1-\mu_s^2)F_1 + (1-\mu_s-2\mu_s^2)F_2\right]$$

Here,

$m = \frac{L}{B} = 1.5$ and $n = \frac{H}{B/2} = \frac{5}{3/2} = 5$ (for settlement at center)

$$A_0 = m \ln\left[\frac{(1+\sqrt{m^2+1})\sqrt{m^2+n^2}}{m \times (1+\sqrt{m^2+n^2+1})}\right] = 0.652$$

$$A_1 = \ln\left[\frac{(m+\sqrt{m^2+1})\sqrt{1+n^2}}{m+\sqrt{m^2+n^2+1}}\right] = 0.90$$

$$A_2 = \frac{m}{n\sqrt{m^2+n^2+1}} = \frac{1.5}{5 \times \sqrt{1.5^2+5^2+1}} = 0.0564$$

$$F_1 = \frac{1}{\pi}(A_0 + A_1) = \frac{1}{3.1416} \times (0.652 + 0.90) = 0.494$$

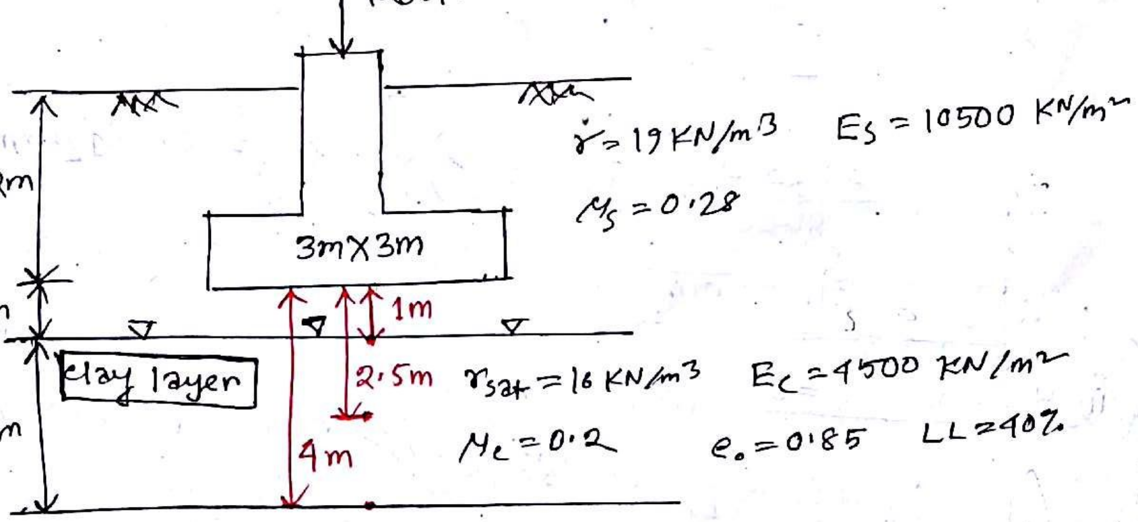
$$F_2 = \frac{n}{2\pi}\tan^{-1} A_2 = \frac{5}{2 \times 3.1416} \times \underbrace{\tan^{-1}(0.0564)}_{\text{radian mode}} = 0.045$$

Now, $$S_e = \frac{2 \times 210}{8500} \times (1 - 0.3^2) \times \left[(1 - 0.3^2) \times 0.494 + (1 - 0.3 - 0.3^2 \times 2) \times 0.045\right]$$

$$\boxed{\therefore S_e = 0.0213 \text{ m} = 21.3 \text{ mm}}$$ (Ans.)

2009 (Same type- 08, 07, 06)

A square footing 3m X 3m carries a column load of 1200KN as shown in figure below. Determine the total settlement of the footing.



Solution: (i) Elastic settlement:

Here, $m = \frac{L}{B} = \frac{3}{3} = 1$, $q_o = \frac{1200}{3\times3} = 133.33\ KN/m^2$

$$\therefore \alpha = \frac{1}{\pi}\left[\ln \frac{\sqrt{m^2+1}+m}{\sqrt{m^2+1}-m} + m \ln \frac{\sqrt{m^2+1}+1}{\sqrt{m^2+1}-1}\right] = 1.12$$

Now,

$$S_{e_1} = \frac{Bq_o}{E_s} \times (1-\mu_s^2) \times \alpha = \frac{3\times 133.33}{10500} \times (1-0.28^2) \times 1.12$$

$\therefore S_{e_1} = 0.04\ m$

$$S_{e_2} = \frac{3\times133.33}{4500} \times (1-0.20^2) \times 1.12$$

$\therefore S_{e_2} = 0.096\ m$

Total elastic settlement, $S_e = (0.04+0.096) = 0.136\ m$

(ii) consolidation settlement:

For normally consolidated clay, $S_c = \frac{C_c H}{1+e_o} \log\left(\frac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_o'}\right)$

where, $C_c = 0.009(LL-10) = 0.009(40-10) = 0.27$

$H = 3m$ (clay layer)

$e_o = 0.85$

$$\sigma_o' = 3\times19 + \frac{3}{2}\times(16-9.81) = 66.285\ KN/m^2$$

For $\Delta\sigma_{av}'$ following table to be prepared:

$$** I_c = \frac{2}{\pi} \times \left[\frac{mn}{\sqrt{1+m^2+n^2}} \times \frac{1+m^2+2n^2}{(1+n^2)(m^2+n^2)} + \sin^{-1}\left(\frac{m}{\sqrt{m^2+n^2}\times\sqrt{1+n^2}}\right)\right]$$

$m=\frac{L}{B}$	z (m)	$n=\frac{z}{B/2}$	q (KN/m²)	** I_c	$\Delta\sigma' = qI_c$	$\Delta\sigma_{av}' = \frac{\Delta\sigma_t' + 4\Delta\sigma_m' + \Delta\sigma_b'}{6}$
1	1	0.67	133.33	0.87	116.0	
1	2.5	1.67	133.33	0.4265	56.865	62.065 KN/m²
1	4	2.67	133.33	0.217	28.93	

$$S_c = \frac{0.27 \times 3}{1+0.85} \times \log\left(\frac{66.285+62.065}{66.285}\right) = 0.126\ m$$

$\therefore$ Total settlement $= (S_e + S_c) = (0.136 + 0.126) = 0.\quad m$

(Ans.)

2013

Estimate the immediate settlement of a concrete footing, 1m x 2m size, founded at a depth of 1m in a soil with $E = 10^4\ KN/m^2$, $\mu = 0.3$. The footing is subjected to pressure of $200\ KN/m^2$. Assume the footing to be rigid. Given for $L/B = 2.0$, $\alpha_r = 1.20$.

Solution: Given, $m = \frac{L}{B} = 2$

$\alpha_r = 1.20$, $\mu = 0.3$

$E_s = 10^4\ KN/m^2$

$q_o = 200\ KN/m^2$

$\therefore$ immediate settlement,

$$S_e = \frac{B q_o}{E_s} \times (1 - \mu_s^2) \times \alpha_r$$

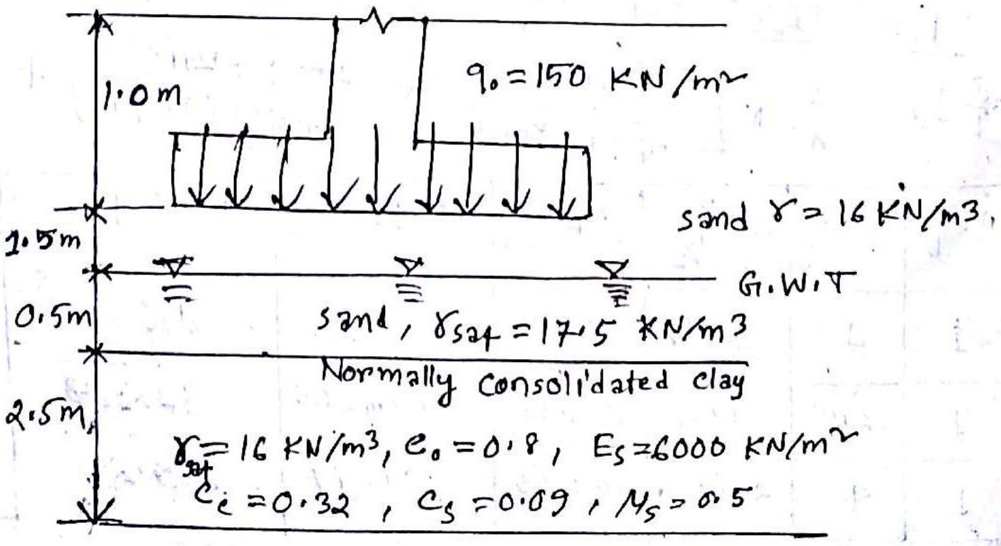
$$= \frac{1 \times 200}{10^4} \times (1 - 0.3^2) \times 1.20$$

$$= 0.022\ m$$

(Ans.)

2014, 2015, 2017

A plan of a foundation 1m x 2m is shown in figure below. Estimate the consolidation settlement of the foundation.



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* Primary consolidation:

Solution: For normally consolidated clay,

$$S_c = \frac{C_c H}{1+e_0} \log \frac{\sigma_0' + \Delta\sigma_{av}'}{\sigma_0'}$$

Here, $L = 2m$, $B = 1m$

~~$S_c = 0.009 (LL-10) = 0.009 \times (40$~~

$C_c = 0.32$ $\qquad \sigma_0' = 2.5 \times 16 + 0.5 \times (17.5 - 9.81) +$

$H = 2.5m$ $\qquad \frac{2.5}{2} \times (16 - 9.81)$

$e_0 = 0.8$ $\qquad = 51.58 \ KN/m^2$

For $\Delta\sigma_{av}'$ following table tobe prepared:

$$I_c = \frac{2}{\pi} \times \left[\frac{mn}{\sqrt{1+m^2+n^2}} \times \frac{1+m^2+2n^2}{(1+n^2)(m^2+n^2)} + \sin^{-1}\left(\frac{m}{\sqrt{m^2+n^2} \times \sqrt{1+n^2}} \right) \right]$$

and, $\sigma_{av}' = \frac{\Delta\sigma_t' + 4\Delta\sigma_m' + \Delta\sigma_b'}{6}$

$m = \frac{L}{B}$	Z (m)	$n = \frac{Z}{B/2}$	q (KN/m^2)	I_c	$\Delta\sigma' = qI_c$ (KN/m^2)	$\Delta\sigma_{av}$ (KN/m^2)
2.0	2	4	150	0.19	28.5	
2.0	3.25	6.5	150	0.082	12.3	$\frac{84.45}{6}$
2.0	4.5	9	150	0.045	6.75	$= 14.075$

Now,

$$S_c = \frac{0.32 \times 2.5}{1+0.8} \times \log \left(\frac{51.58 + 14.075}{51.58} \right)$$

$$= 0.04657 \ m$$

$$= 46.57 \ mm.$$

(Ans.)

Same question: 2015

*(i) Find primary consolidation settlement and

(ii) Find secondary consolidation settlement. (Use $C_\alpha' = 0.0005$ and $\frac{t_2}{t_1} = 10\,000$)

Solution: from previous solution,

$$S_{c(p)} = 46.57 \text{ mm}$$

For secondary consolidation settlement,

$$S_{c(s)} = C_\alpha' H \log\left(\frac{t_2}{t_1}\right)$$

$$= 0.0005 \times 2.5 \times \log(10{,}000)$$

$$= 0.005 \text{ m}$$

$$\therefore S_{c(s)} = 5 \text{ mm}$$

(Ans)

2017

change: $\begin{cases} LL = 40\% \\ C_\alpha = 0.0013 \\ e_p = 0.25 \end{cases}$

* C_c (not given)

$\boxed{C_c = 0.009\,(LL - 10)}$

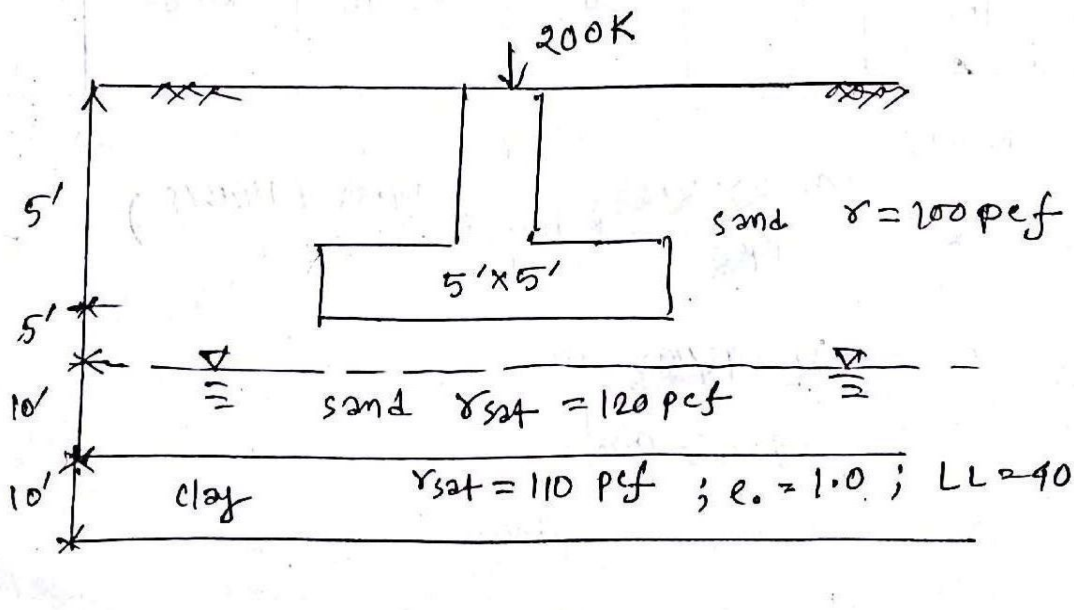
* C_α' (not given)

$\boxed{C_\alpha' = \frac{C_\alpha}{1 + e_p}}$

2016

Calculate the consolidation settlement (Primary and secondary) of the foundation as shown in figure below. The clay is normally consolidated. Use the weighed average method to calculate the average increase of effective pressure in the clay layer.

Use $C_\alpha' = 0.0005$ and $t_2/t_1 = 10000$



Solution: (i) primary consolidation settlement:

$$S_c = \frac{C_c H}{1+e_0} \log \frac{\sigma_0' + \Delta\sigma_{av}'}{\sigma_0'}$$

Here, $L = 5'$, $B = 5'$, $q_0 = \frac{200 \times 1000}{5 \times 5} = 8000 \ lb/ft^2$

$C_c = 0.009\,(LL-10) = 0.009 \times (40-10) = 0.27$

$H = 10\ ft$

$e_0 = 1.0$

$\sigma_0' = 10 \times 100 + 10 \times (120 - 62.4) + \frac{10}{2} \times (110 - 62.4)$

$= 1814 \ lb/ft^2$

For $\Delta\sigma_{av}'$ following table to be prepared:

$$I_c = \frac{2}{\pi} \times \left[\frac{mn}{\sqrt{1+m^2+n^2}} \times \frac{1+m^2+2n^2}{(1+n^2)(m^2+n^2)} + \sin^{-1}\left(\frac{m}{\sqrt{m^2+n^2} \times \sqrt{1+n^2}} \right) \right.$$

and,
$$\Delta\sigma_{av}' = \frac{\Delta\sigma_t' + 4\Delta\sigma_m' + \Delta\sigma_b'}{6}$$

$m = \frac{L}{B}$	z (m)	$n = \frac{z}{B/2}$	q (K/ft^2)	I_c	$\Delta\sigma' = q I_c$ (K/ft^2)	$\Delta\sigma_{av}$ (K/ft^2)
1	15	6	8000	0.051	408	
1	20	8	8000	0.029	232	248
1	25	10	8000	0.019	152	

$$\therefore S_c(P) = \frac{0.27 \times 10}{1+1} \times \log\left(\frac{1814 + 248}{1814}\right) = 0.075\ ft$$

$\therefore S_c(P) \doteq 0.9$ in.

(ii) secondary consolidation settlement:

$$S_c(s) = C_\alpha' H \log\left(\frac{t_2}{t_1}\right) = 0.0005 \times 10 \times \log(10000)$$

$\therefore S_c(s) = 0.02\ ft = 0.24$ in.

$\therefore$ Total settlement, $S_c = (0.9 + 0.24) = 1.14$ in. (Ans.)

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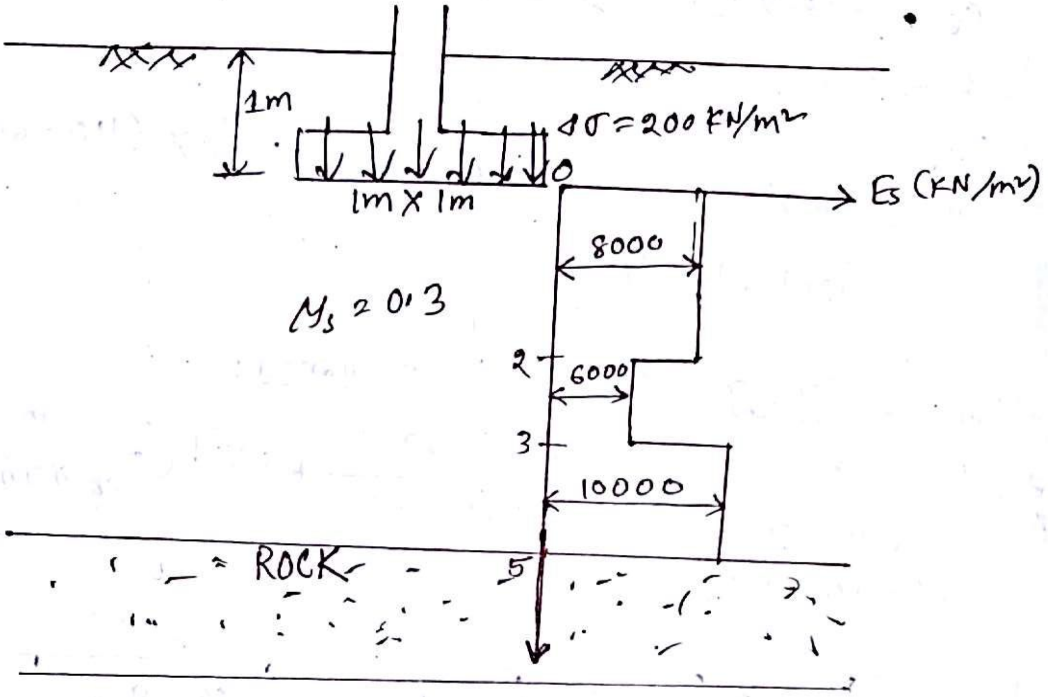
Example : 11·1 (BM Das – Geotechnical Engineering)

A rigid foundation 1m × 1m in plan is shown in figure; calculate the elastic settlement at the center of the foundation.



solution: Given, $B = 1\ m$

$L = 1m$

and $\bar{z} = 5$

Now, $E_s = \dfrac{\sum E_{s(i)}\,\Delta z}{\bar{z}}$

$= \dfrac{8000 \times 2 + 6000 \times 1 + 10000 \times 2}{5}$

$= 8400\ kN/m^2$

for center of the foundation,

$\alpha = 4$

$m' = \dfrac{L}{B} = 1$

$n' = \dfrac{H}{B/2} = \dfrac{5}{0.5} = 10$

using formula, $F_1 = 0.498$ and $F_2 = 0.016$

$\therefore I_s = F_1 + \dfrac{1 - 2\mu_s}{1 - \mu_s} F_2 = 0.498 + \dfrac{1 - 0.6}{1 - 0.3} \times (0.016)$

$\therefore I_s = 0.507$

Again, $\dfrac{D_f}{B} = 1$, $\dfrac{L}{B} = 1$ and $\mu_s = 0.3$ $\boxed{\text{using table -11.3}}$ $I_f = 0.65$

$\therefore S_e(\text{flexible}) = \Delta\sigma(\alpha B') \times \dfrac{1 - \mu_s^2}{E_s} \times I_s I_f = 200 \times (4 \times \frac{1}{2}) \times \left(\dfrac{1 - 0.3^2}{8400}\right) \times 0.507 \times 0.65$

$B' \rightarrow \dfrac{B}{2}$

$= 14.3\ mm$

$S_e(\text{rigid}) = (0.93 \times 14.3) = 13.3\ mm$ (Ans.)

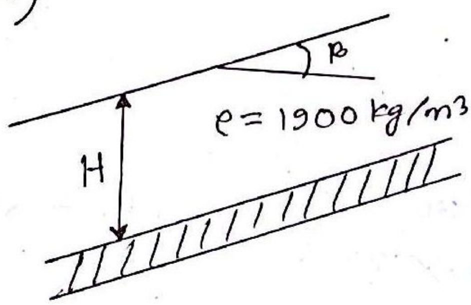
Slope Stability

2010

An infinite slope is shown in figure below. The shear strength parameter at the interface of soil and rock are as $c = 18\ KN/m^3$ and $\phi = 25°$

(i) If $H = 8\,m$ and $\beta = 20°$, Find the factor of safety against sliding on the rock surface.

(ii) If $\beta = 30°$ find the height H for $FS = 1$ (Assume pore water pressure to be zero)



Solution: We know,

$$FS = \frac{c}{\gamma H \cos^2\beta \tan\beta} + \frac{\tan\phi}{\tan\beta}$$

(i) Given,

$c = 18\ KN/m^3$

$\phi = 25°$

$H = 8m$

$\beta = 20°$

$\gamma = \rho g = (1900 \times 9.81)\ N/m^3$

$= 18639\ N/m^3 = 18.639\ KN/m^3$

$$FS = \frac{18}{18.639 \times 8 \times \cos^2 20° \times \tan 20°} + \frac{\tan 25°}{\tan 20°}$$

$$\therefore FS = 1.657$$

(Ans.)

(ii) Given, $\beta = 30°$ and $FS = 1$.

Then, $$1 = \frac{18}{18.639 \times H \times \cos^2 30 \times \tan 30°} + \frac{\tan 25°}{\tan 30°}$$

$$\Rightarrow H = 11.6\ m$$

2011, 2013

A 6m deep cut is to be made in cohesive soil with a slope of 1 : 1. The soil has $c_u = 30\ KN/m^2$, $\phi_u = 10°$ and $\gamma = 18\ KN/m^3$. Find the factor of safety with respect to cohesion. What will be the critical height of the slope in this soil?

Solution: Here, slope angle $i = \tan^{-1}(\frac{1}{1}) = 45°$

For $i = 45°$ and $\phi_u = 10°$

Stability Number, $S_n = 0.108$ [Table - 23.5] B.C. Punmia.

for $i = 26.5° \rightarrow 1:2$
and $\phi_u = 10°$.
$S_n = 0.064$

(i) We know,

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

$$\Rightarrow 0.108 = \frac{30}{F_c \times 18 \times 6} \qquad \therefore F_c = 2.572$$

(Ans)

(ii) The critical height, $H_c = F_c \times H$

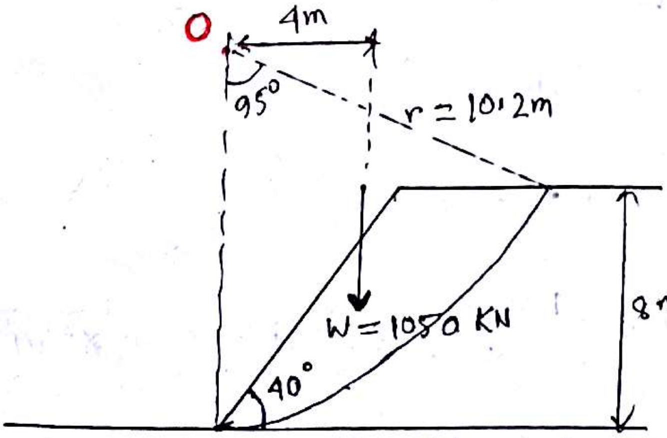
$$= (2.572 \times 6)$$

$$= 15.43\ m$$

(Ans)

2015, 2016

A 40° slope is excavated to a depth of 8m in a deep layer of saturated clay ($c = 70\ KN/m^2$ and $\phi = 0$, $\gamma = 19\ KN/m^3$). Determine the factor of safety for the trial failure surface as shown in figure below.



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Solution: Given, $W = 1050\,KN$, $r = 10.2\,m$, $\theta = 95°$

Arc length, $\hat{L} = r.\,\theta\,(rad)$

$$= 10.2 \times \left(\frac{\pi}{180} \times 95°\right)$$

$$= 16.9123\,m$$

The cohesive force developed along the arc,

$$C = c.\hat{L}$$

$$= (70 \times 16.9123) = 1183.86\ KN/m$$

Taking moment about O –

$\bar{x} \times W$

Disturbing moment, $M_D = 4 \times W = (4 \times 1050)$

$$= 4200\ KN\text{-}m/m$$

Restoring Moment, $M_R = r.C = (10.2 \times 1183.86)$

$$= 12075.372\ KN\text{-}m/m$$

Factor of safety, $F.S = \dfrac{M_R}{M_D} = \dfrac{12075.37}{4200}$

$$= 2.88$$

(Ans.)

Do practice from –

B.C. Punmia – (23.1 – 23.12, 23.14, 23.17, 23.18)

B.M. Das (Geotechnical Engineering) – [15.1, 15.2] for $i = 30^\circ$ and $\phi = 15^\circ$; $S_n = 0.046$

for $i = 45^\circ$ and $\phi = 15^\circ$; $S_n = 0.083$