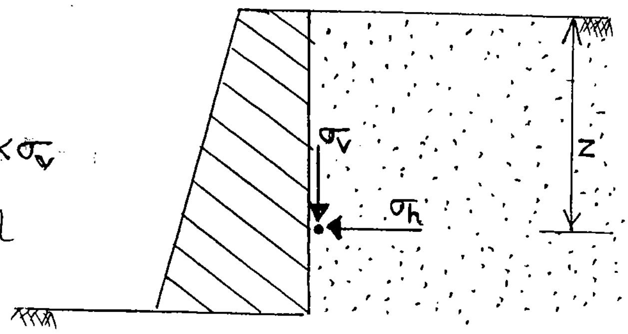
14. EARTH PRESSURE



$\sigma_v = \gamma z$

Lateral Earth Pressure, $\sigma_h = k\sigma_v$

where $K \rightarrow$ coeffecient of lateral earth pressure.

$$\boxed{K = \frac{\sigma_h}{\sigma_v}}$$

→ Types of Lateral Earth Pressures.

1. At-rest Earth pressure (P_o)
2. Active Earth pressure. (P_a)
3. Passive Earth pressure. (P_p)

* At-rest Earth Pressure :-

- It arises when there is no movement of wall.
- No yielding of soil.
- elastic equilibrium.; theory of elasticity is used to find σ_h.

At rest earth pressure, $P_o = k_o \cdot \sigma_v$

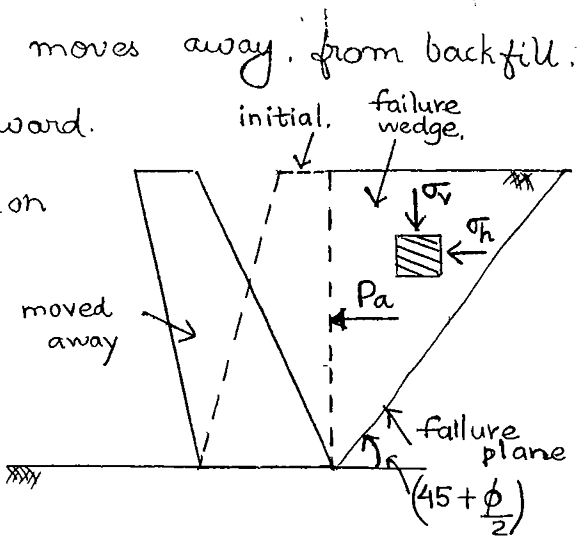
where $K_o \rightarrow$ coeffecient of at-rest earth pressure.

$$K_o = \frac{\mu}{1-\mu} \; ; \; \mu \rightarrow \text{poisson's ratio of soil.}$$

also $K_o = 1 - \sin\phi$; for cohesionless soils.

* Active Earth Pressure.

- It arises when the wall moves away from backfill.
- Failure wedge moves downward.
- It is a plastic eqbm condition

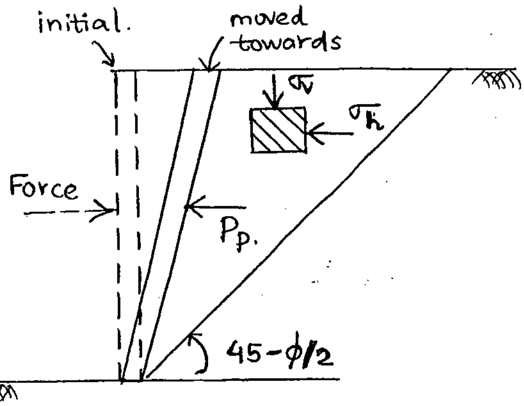


Here $\sigma_v = \sigma_1$ & $\sigma_h = \sigma_3$

Failure plane makes an angle of $(45+\phi/2)$ with Major principal plane.

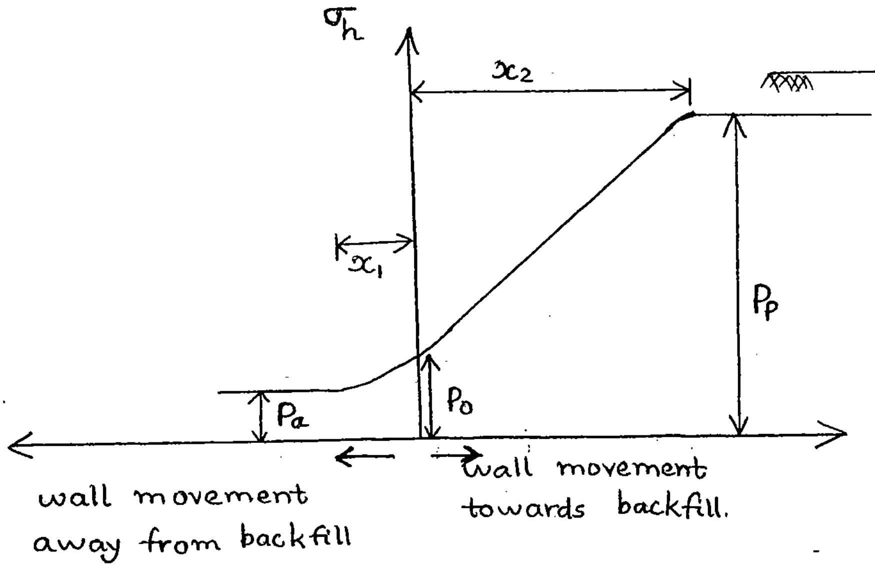
* Passive Earth Pressure

- It arises when the wall moves towards the backfill.
- Failure wedge moves upwards.
- It is a plastic eqbm condition.



Here $\sigma_v = \sigma_3$ $\sigma_h = \sigma_1$

22nd Sept,
MONDAY

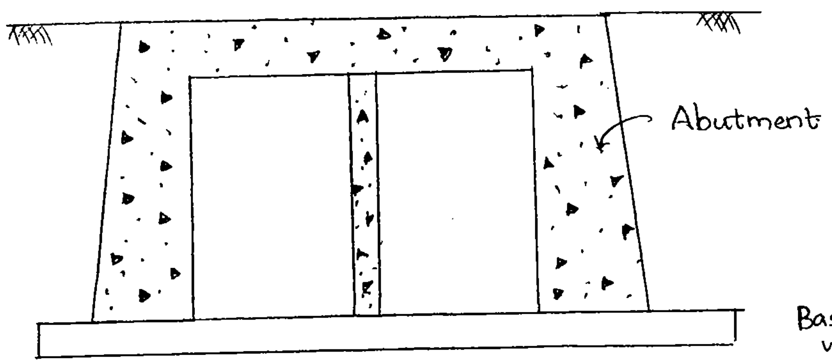


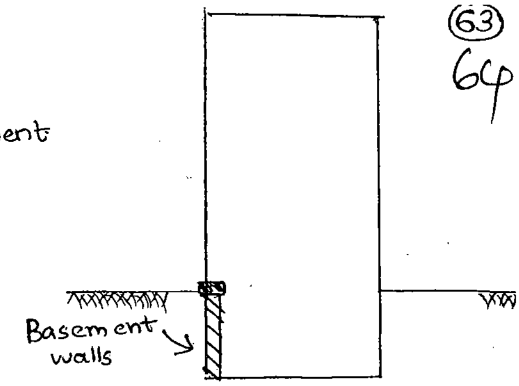
- ⊙ $P_p > P_o > P_a$
- ⊙ $x_1 < x_2$

→ Practical Applications:

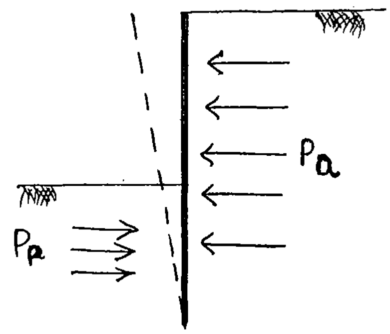
(i) For design of ordinary retaining wall. – active pressure is used

(ii) For design of bridge abutments and basement walls – at rest earth pressure is used





(iii) For design of sheet piles – both active & passive pre. used.



→ Rankine's Theory :

– to find P_a & P_p.

* Assumptions :-

(i) Soil is dry and cohesionless.

(ii) The back of the wall is vertical and smooth.

(iii) Plastic equilibrium.

Plastic equilibrium equation :-

$$\sigma_1 = \sigma_3 \tan^2\alpha_f + 2c\tan\alpha_f.$$

for cohesionless soil,

$$\sigma_1 = \sigma_3 \tan^2\alpha_f.$$

In active case, $\sigma_1 = \sigma_v$ & $\sigma_3 = \sigma_h$.

$$\therefore \sigma_v = \sigma_h \tan^2\alpha_f.$$

$$\sigma_h = \frac{\sigma_v}{\tan^2\alpha_f}$$

or $\boxed{P_a = K_a \sigma_v}$

where $K_a \rightarrow$ coeffecient of active earth pressure. $\left(= \frac{1}{\tan^2\alpha_f}\right)$

$$K_a = \frac{1}{\tan^2\alpha_f} = \frac{1}{\tan^2(45+\frac{\phi}{2})} = \tan^2(45-\phi/2)$$

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

Similarly,

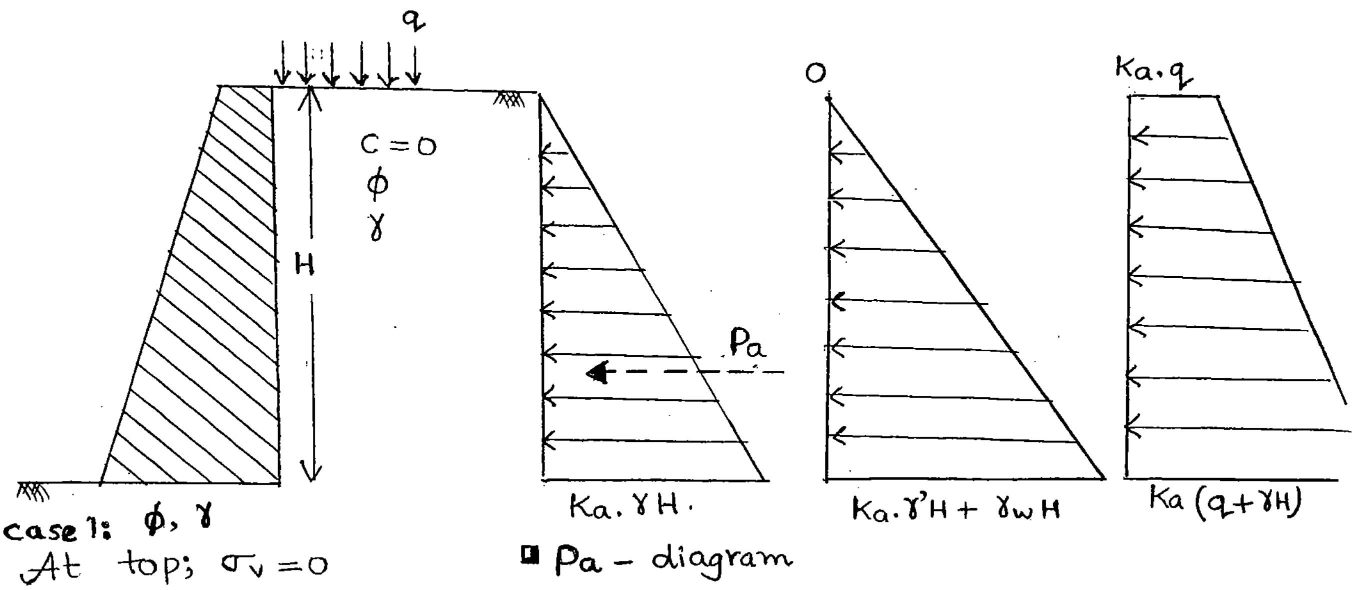
$$\boxed{P_p = K_p \cdot \sigma_v}$$

$K_p \rightarrow$ coeffecient of passive earth pressure.

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

$P_o = k_o \, \sigma_v \rightarrow$ for both cohesive & cohesionless soils

$$\left.\begin{aligned} P_a &= k_a \, \sigma_v \\ P_p &= K_p \, \sigma_v \end{aligned}\right\} \rightarrow \text{cohesionless soils}$$



case 1: ϕ, γ

At top; $\sigma_v = 0$

$P_a = K_a \sigma_v = 0.$

At bottom, $\sigma_v = \gamma H.$

$P_a = K_a \gamma H.$

Let P_a = total active force.

= area of pressure diagram.

$P_a = \frac{K_a \cdot \gamma H^2}{2}$; at $\frac{H}{3}$ from base

case 2: ϕ, γ_{sat}, <u>WT on ground</u>.

At top; $P_a = 0$

At bottom; $\sigma_v' = \gamma' H$

$$\boxed{P_a = K_a \sigma_v' + \gamma_w H}$$

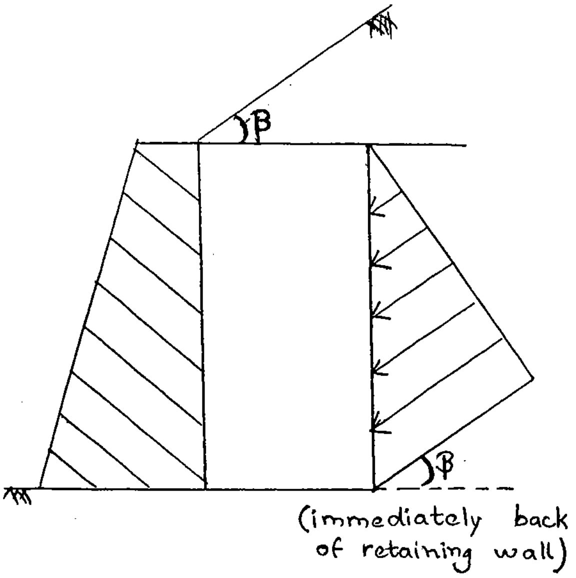
Case 3: Surcharge loading, q

At top; $\sigma_v = q$.

$$P_a = K_a \sigma_v = K_a \cdot q.$$

At bottom; $\sigma_v = q + \gamma H$

$$P_a = K_a \sigma_v = K_a (q + \gamma H).$$



$$P_a = k_a \cdot \sigma_v$$

$$P_p = k_p \cdot \sigma_v.$$

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

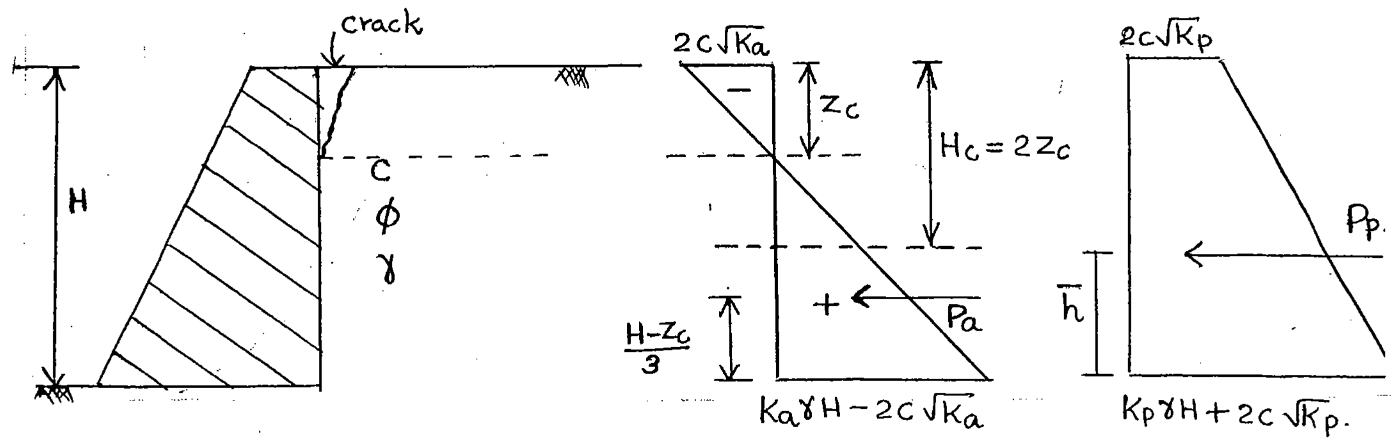
$$\boxed{K_p = \cos\beta \cdot \left(\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \right)}$$

Here $K_p \neq \dfrac{1}{K_a}$.

→ C-ϕ Soils

$$\boxed{\begin{aligned} P_a &= K_a \cdot \sigma_v - 2c\sqrt{K_a} \\ P_p &= K_p \cdot \sigma_v + 2c\sqrt{K_p} \end{aligned}} \Rightarrow \text{BELL'S EQUATIONS}$$

⊙ Cohesion decreases active pressure but increases passive pressure



At top; $\sigma_v = 0$

$$P_a = K_a \sigma_v - 2c\sqrt{K_a}.$$
$$= -2c\sqrt{K_a} \text{ (tension)}$$

At bottom; $\sigma_v = \gamma H$.

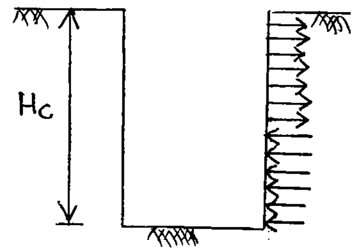
$$P_a = K_a \gamma H - 2c\sqrt{K_a}.$$

Z_c : depth of tension zone (or) depth of tension crack

* To find Z_c :-

At a depth of Z_c,

$$P_a = K_a \gamma Z_c - 2c\sqrt{K_a}.$$
$$0 = K_a \gamma Z_c - 2c\sqrt{K_a}$$



$$\therefore \quad Z_c = \frac{2c}{\gamma\sqrt{K_a}}.$$
$$= \frac{2c}{\gamma} \tan(45 + \phi/2).$$
$$= \frac{2c}{\gamma} \text{ ; for pure clay } (\phi = 0)$$

H_c : critical height or depth of unsupported vertical trench.

$$* \quad H_c = 2Z_c = \frac{4c}{\gamma\sqrt{K_a}} = \frac{4c}{\gamma}\tan\left(45 + \frac{\phi}{2}\right)$$
$$= \frac{4c}{\gamma} \text{ ; for pure clay}$$

* To find total active force, P_a

ⓐ. Before formation of crack.

$$P_a = \int_0^H p_a \cdot dz = \text{total algebraic sum of area of pressure diagram.}$$

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

ⓑ After formation of crack.

$$P_a = \int_{Z_c}^{H} p_a \cdot dz = \text{area of +ve portion only.}$$

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

* To find total Passive fore, P_p.

$P_p = K_p \sigma_v + 2c\sqrt{K_p}$.

At top; $\sigma_v = 0$. $\therefore P_p = 2c\sqrt{K_p}$

At bottom; $\sigma_v = \gamma H$. $\therefore P_p = K_p \gamma H + 2c\sqrt{K_p}$

Total passive pressure force, P_p = area of pressure diagram.

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

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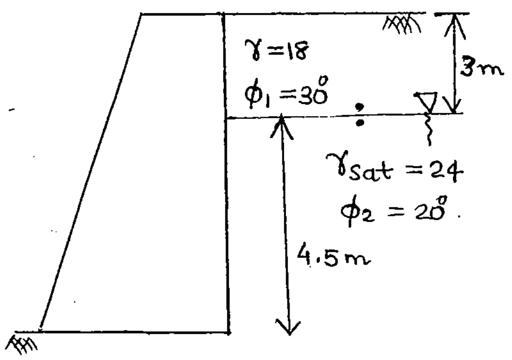
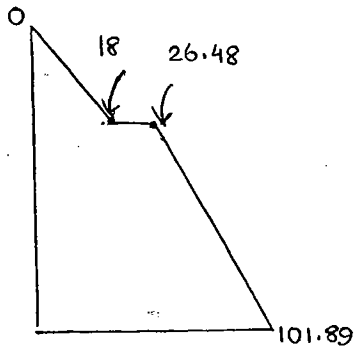
Q1. $H = 7.5\,m$;





$K_{a_1} = \frac{1 - \sin\phi_1}{1 + \sin\phi_1} = \underline{\underline{0.33}}$

$K_{a_2} = \frac{1 - \sin 20}{1 + \sin 20} = \underline{\underline{0.49}}$

At top; $\sigma_v = 0$

$P_a = K_{a_1} \cdot \sigma_v = 0$.

At 3m depth, $\sigma_v = 18 \times 3 = 54$

a) Just above 3m depth

$P_a = K_{a_1} \sigma_v = \underline{\underline{18}}$ kPa

b) Just below 3m depth,

$P_a = K_{a_2} \sigma_v = 0.49 \times 54$

$= 26.48$ kPa

At bottom; $\sigma_v = 18 \times 3 + (24 - 9.81) \times 4.5$

$= 117.855$ kPa

$P_a = K_{a_2} \sigma_v' + \gamma_w h$

$= 0.49 \times 117.855 + 9.81 \times 4.5$

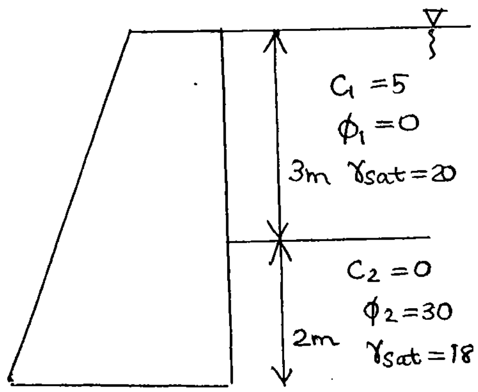
$= \underline{\underline{101.89}}$ kPa

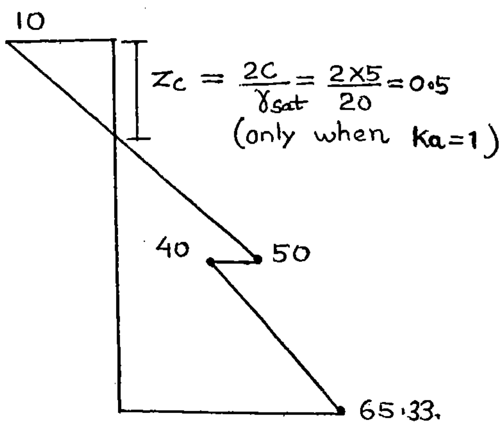
Total force, $P_a = \frac{1}{2} \times 18 \times 3 + \left(\frac{26.46 + 101.89}{2}\right) \times 4.5$

$= \underline{\underline{315.832}}$ kN/m.

⊙ Area of Pressure force diagram = Force per unit length.

02.





$Ka_1 = \frac{1 - \sin\phi}{1 + \sin\phi} = \underline{\underline{1}}$

$Ka_2 = \frac{1 - \sin 30}{1 + \sin 30} = \underline{\underline{\frac{1}{3}}}$

At top;

$\sigma_v = 0$

$P_a = Ka_1 \sigma_v - 2C_1 \sqrt{Ka_1}$.

$= 0 - 2 \times 5 \times 1 = -10$ kPa. (tension).

At bottom; $\sigma_v' = 10 \times 3 + 8 \times 2 = 46$.

$P_a = 46 \times \frac{1}{3} + 10 \times 5$.

$= \underline{\underline{65.33}}$ kPa.

At 3m depth;

$\sigma_v' = (20 - 10)3 = 30$ kPa.

a) Just above 3m depth,

$P_a = \sigma_v' Ka_1 - 2C_1\sqrt{Ka_1} + \gamma_w h$.

$= 30 \times 1 - 2 \times 5 + 10 \times 3 = 50$ kPa.

b) Just below 3m depth,

$P_a = \sigma_v' Ka_2 - 2C_2\sqrt{Ka_2} + \gamma_w h$.

$= 30 \times \frac{1}{3} - 0 + 30 = 40$ kPa.

To find Z_c :

At a depth of Z_c, $P_a = 0$.

$$P_a = K_{a1} \gamma' Z_c - 2 c_1 \sqrt{K_{a1}} + \gamma_w Z_c$$

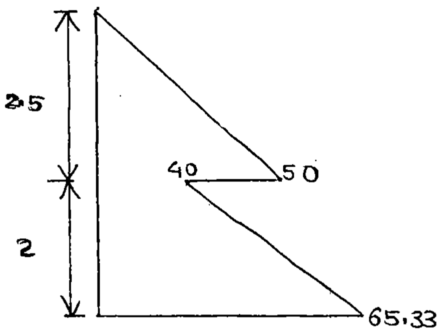
$$0 = 1 \times 10 Z_c - 2 \times 5 \times \sqrt{1} + 10 Z_c.$$

$$Z_c = \frac{10}{20} = \underline{\underline{0.5 \text{ m}}}$$

OR

From similar triangles,

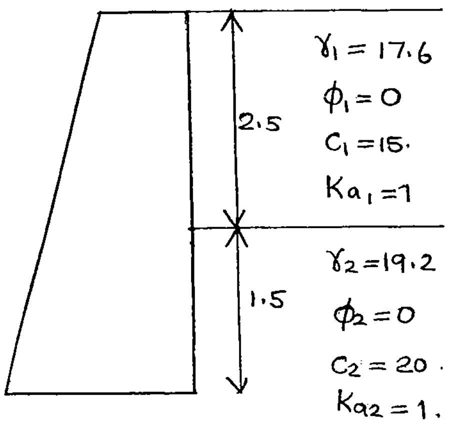
$$\frac{10}{Z_c} = \frac{50}{3 - Z_c} \Rightarrow Z_c = \underline{\underline{0.5 \text{ m}}}$$

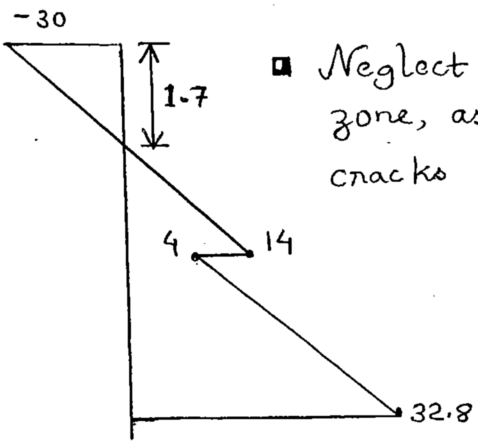


Neglecting tension zone :

$$P_a = \frac{1}{2} \times 50 \times 2.5 + 2\left(\frac{40 + 65.33}{2}\right) = \underline{\underline{167.8 \text{ kN/m}}}$$

Q3.





▪ Neglect tension zone, as tension cracks develop.

At top : $\sigma_v = -2c\sqrt{K_{a1}} = \underline{\underline{-30}}$

$$P_a = K_{a1} \sigma_v = -30 \text{ kPa}.$$

At 2.5 m depth :

$$\sigma_v^* = \gamma \times 2.5$$

$$= 17.6 \times 2.5 = 44 \text{ kPa}.$$

a) Just above,

$$P_a = K_{a1} \sigma_v^* - 2C_1\sqrt{K_{a1}}$$

$$= 44 - 2 \times 15 = 14 \text{ kPa}.$$

b) Just below,

$$P_a = 1 \times 44 - 2 \times 20 = 4 \text{ kPa}.$$

At bottom :

$$\sigma_v = 17.6 \times 2.5 + 19.2 \times 1.5.$$

$$= 72.8.$$

$$P_a = 72.8 - 2 \times 20$$

$$= \underline{\underline{32.8}} \text{ kPa}.$$

Total active force, P_a

$$= (2.5 - 1.7) \times 14 \times 0.5 +$$

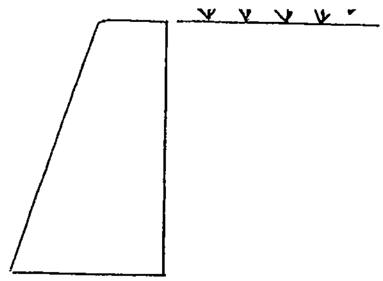
$$0.5(4 + 32.8) \times 1.5$$

$$= \underline{\underline{33.168}} \text{ kPa}$$

04. $P_a = K_a \sigma_v - 2c\sqrt{K_a}.$

$$0 = K_a \cdot q - 2c\sqrt{K_a}.$$

$$q = \frac{2c}{\sqrt{K_a}} = 2c \tan \alpha_f.$$

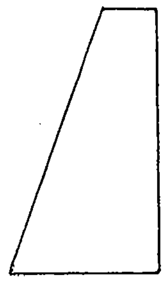


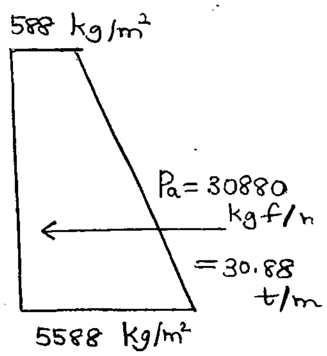
05. When there is no surcharge,

at bottom, $P_a = K_a \gamma H.$

$$5000 = K_a \cdot 1700 \times 10.$$

$$K_a = 0.294.$$





If there is surcharge,

at ~~bottom~~ top, $P_a = K_a \cdot \sigma_v.$

$$= K_a \cdot q = 0.294 \times 2000$$

$$= \underline{588} \text{ kg/m}^2.$$

at bottom, $\sigma_v = q + \gamma H.$

$$P_a = K_a \cdot \sigma_v.$$

$$= K_a (q + \gamma H) = \underline{\underline{5588}} \text{ kg/m}^2.$$

Maximum earth pressure $= \underline{\underline{5588}}$ kg/m²

Resultant force on the wall $= \frac{1}{2}(5588 + 588) \times 10$

$$= 30880 \text{ kgf/m}$$

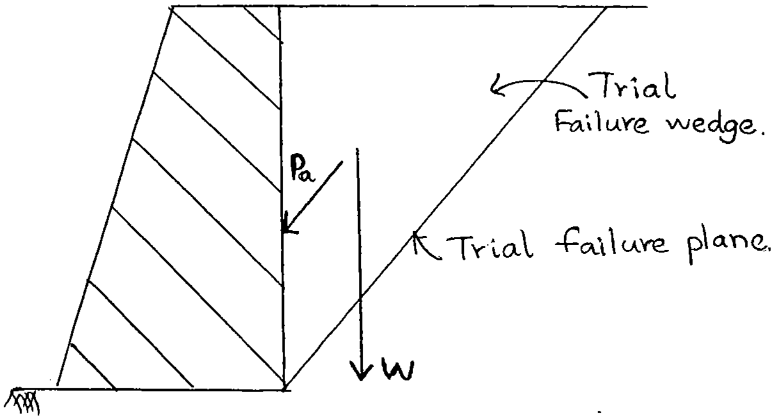
$$= \underline{\underline{30.880}} \text{ t/m}$$

→ Coulomb's Theory

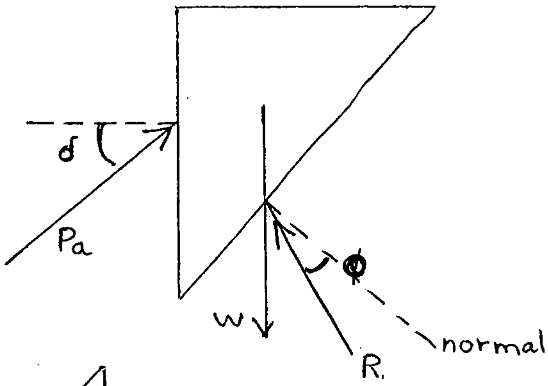
* Assumptions:

(i) Soil is dry and cohesionless.

(ii) Back of the wall is <u>rough</u>

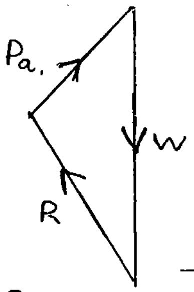


▪ FBD of Failure wedge.

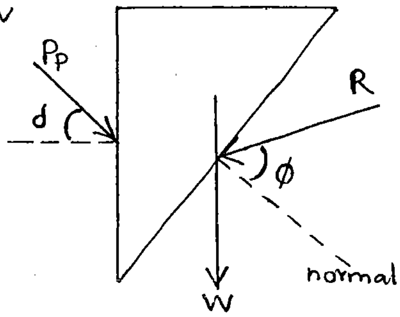


$\delta \rightarrow$ angle of wall friction.

Coulomb's theory is a graphical trial & error method of computing lateral earth pressures (P_a & P_p).



▪ force triangle



NOTE: Effect of Wall friction :-

⊙ The wall friction reduces active pressure but increases passive pressure; both are advantageous.

⊙ For RCC retaining walls, Rankine's Theory can be used. For stone masonry retaining walls, Coulomb's Theory is used.

* Rebhan's Method & Culman's Method – graphical method of computing P_a & P_p using Coulomb's theory.

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16. When soil is compacted, $\phi \uparrow \Rightarrow K_a \downarrow$

$$P_a = K_a \gamma H \quad (\downarrow).$$

$K_a \downarrow > \gamma \uparrow$

26. Cohesive soils are poor for backfilling as they cause more lateral pressure. due to following reasons:

(i) for clays ϕ is less, Hence K_a is more.

(ii). Swelling of clays.

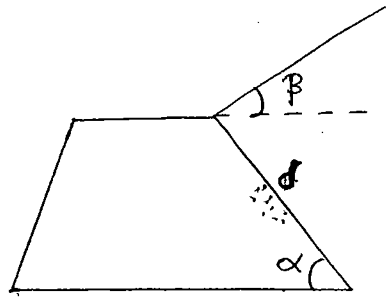
(iii) Clays have poor drainage properties.

(iv) Compaction of clays behind the wall is difficult.

For backfilling behind the walls, cohesionless soils like gravel and sand are best.

$\uparrow K_a$ due to above factors is more than the $\downarrow K_a$ due to cohesion. $(P_a = K_a \sigma_v - 2c\sqrt{K_a})$.

→ Solution for Coulomb's Theory.



$$\text{Force}, \; P_a = K_a \frac{\gamma H^2}{2}.$$

where K_a depends on $\alpha, \beta, \delta, \phi$

⊙ For vertical wall and horizontal backfill and if $\delta = \phi$,

then, $K_a = \dfrac{\cos\phi}{(1+\sqrt{2}\sin\phi)^2}$; $K_p = \dfrac{\cos\phi}{(1-\sqrt{2}\sin\phi)^2}$