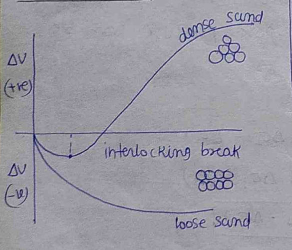
Lecture 12
16/11/19

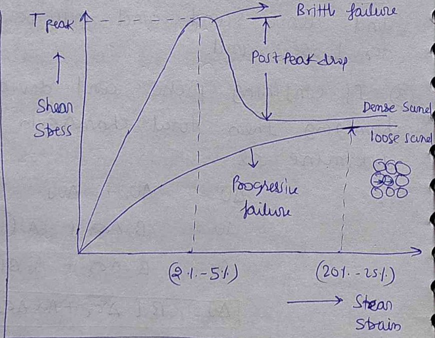
- **Shear characteristic of sand**

① In dense sand the shear strength is due to interlocking and friction both. Interlocking resistance may be (20% - 30%) of total ~~settlement~~ and at (2% to 5%) of strain interlocking fails and sudden decrease in shear strength is recorded this type of failure is termed as brittle failure. It is also found in undisturbed sensitive clay and overconsolidated clay.

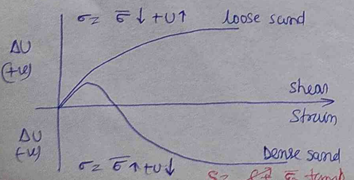
② In loose sand vol^m continues to decrease and clear cut failure point does not obtain. Hence, failure is assumed when strain is reached (20% to 25%) this type of failure is termed as Progressive failure it is also found in Remoulded clay and N.C.C.

① Volume change Vs Shear strain



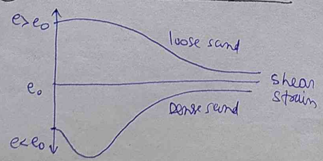


② change in Pore pressure v/s shear strain



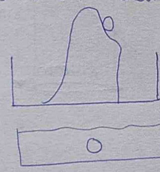
S = ℓ↑ कम temp

③ void Ratio v/s shear strain



$e_0 \rightarrow$ critical void Ratio

It is the void ratio at which negligible vol^m change occurs due to shearing.



① In dense saturated sand

due to seismic disturbance volume ↑ after interlocking break as a result pore pressure change becomes negative Hence effective stress ↑ consequently shear streght ↑ and failure does not occure.

Liquifaction

In loose saturated sand due to seismic disturbance (earth quick) or dynamic loading vol^m ↓ Hence Pore pressure change become (+ve). due to build up of high pore pressure sudden decrease in effective stress and decrease in shear strength is recorded. (sometimes the effective stress may reduce to zero also) consequently large settlement of foundation suddenly occurs along with vertical upward flow of muddy water. such Phenomenon is called liquifaction of sand. It is generally observed near the rivers and sea during seismic activity.

W.B Ques. ⑰

Ans- (37) d = 37.5 mm , length = 80 mm

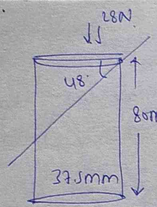
load failure = 28 N

axial deformation$_{failure}$ = 13 mm

S = ?

$$UCS = (\sigma_d)_f = \frac{P}{A_f} = \frac{28N}{\frac{\pi}{4}(37.5)^2} = \frac{28N \times 4}{\pi(37.5)^2} = 0.0253$$

$= 25.36 \ \frac{N}{m^2}$



$UCS = (\sigma_d)_f = (\sigma_1)_f = \frac{P}{A_f}$

$A_f = \frac{A_0}{1-\varepsilon_v} = \frac{\frac{\pi}{4}D^2}{1-\frac{\Delta L}{L_0}} = \frac{\frac{\pi}{4}(37.5\times10^{-3})^2}{1-\frac{13}{80}}$

$= 1318.7\times10^{-6}\ m^2$

$UCS = \frac{P}{A_f} = \frac{28N}{1318.7\times10^{-6}} = 21.33\ KN/m^2$

Undrained shear strength $S = C + \sigma_n \tan\phi$

$= C = \frac{UCS}{2} = \frac{21.23}{2}$

$= 10.62\ KN/m^2$

Solution (30)

$\sigma_3 = 200\ KN/m^2$

$\sigma_d = 342N$ = $\sigma_d = \frac{P}{A_f} = \frac{342}{\frac{A_0}{1-\varepsilon_0}} = \frac{342}{\frac{\frac{\pi}{4}(3.8\times10^{-3})^2}{(1-\frac{5.1}{76})}} = 281.6$

length = 76mm and dia = 38mm

vertical deformation = 5.1mm

S = ? of sand at 6m from ground surface

$\sigma_n = 100\ KN/m^2$

$\sigma_3 + \sigma_d = \sigma_1$

$\sigma_1 = 481.6\ KN/m^2$

$\sigma_1 = \sigma_3 \tan^2(45+\frac{\phi}{2}) + 2c\tan(45+\frac{\phi}{2})$

$481.6 = 200 \tan^2(45+\frac{\phi}{2})$

$\tan^{-1}(\sqrt{2.4}) = 45 + \frac{\phi}{2}$

$\phi = 24.36°$

$S = c^0 + \sigma_n \tan\phi$

$= 100 \times \tan 24.36 = 45.31\ KN/m^2$

Solution (41) Undrained and drain Parameter

Page (51)

$B = \frac{\Delta U_c}{\Delta \sigma_c} = \frac{\Delta U_c}{\Delta \sigma_3}$

$B = \frac{145}{350} = 0.414$ --- (1)

$\bar{A} = \frac{145}{242} = 0.6$ --- (2)

$\bar{A} = A.B$ $(\sigma_3 + \sigma_d = \sigma_1)$

$\bar{A} = A \times 0.414$ $(350+242)$

$\frac{0.6}{0.414} = A$ (592)

$1.44 = A$

$\Delta U = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$

$\Delta U = 0.414(350 + 1.44(592 - 350))$

$\Delta U = 289.17$ --- (1)

	σ_3	σ_d	U	$\sigma_1 = \sigma_3+\sigma_d$	$\bar{\sigma}_1 = \sigma_1 - U$	$\bar{\sigma}_3 = \sigma_3 - U$
A	250	179	101	429	328	149
B	350	242	145	592	447	205

For undrained shear parameter UK total stress

$\sigma_1 = \sigma_3 \tan^2(45+\frac{\phi}{2}) + 2c\tan(45+\frac{\phi}{2})$

$429 = 250 \times x^2 + 2cx$ --- (1)

$592 = 350 x^2 + 2cx$ --- (2)

$163 = 100 \tan^2(45+\frac{\phi}{2})$

$\phi = 13.86°$

$C = 8.42\ KPa$

② For Drained Shear Parameter use eff. stress

$\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2(45 + \frac{\phi}{2}) + 2c' \tan(45 + \frac{\phi}{2})$

$328 = 149 \times 2 + 2c' x$

$447 = 205 x^2 \pm 2c' x'$

$119 = 56 \tan^2(45 + \frac{\phi'}{2})$

$\phi' = 21.10°$
$c' = 3.9$ kPa

2 observation are given → 2 eqn → 2 unknown

1 observation → 1 eqn → 2 unknowns ?

DEMAND → assume an unknown as zero

SAND → C=0

CLAY
- → UU test $\phi = 0$
- → CD test C=0
- → CU test C=0

Solution (44) CD test

$\sigma_3 = 250$ KPas

back pressure = 100 KPas

$\sigma_d = 300$ KPa.

① $c' = 0$

② $\theta_c = ?$

③ S and σ_n

$\sigma_1 = 550$ KPas) $\bar{\sigma}_1 = \sigma_1 - U = 450 - 100 = 450$

① $\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2(45 + \frac{\phi}{2}) + 2c' \tan(45 + \frac{\phi}{2})$

① $450 = 150 \tan^2(45 + \frac{\phi}{2})$

$\tan^{-1}(\sqrt{2.3}) = 45 + \frac{\phi}{2}$

$\phi = 22.9$

$\theta_c = 45 + \frac{\phi}{2}$

$\theta_c = 45 + 56.48°$

① $\phi = 30$

② $\theta_c = 45 + \frac{\phi}{2} = 60$ [with major axis]

③ $\bar{\sigma}_n = \left(\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2}\right) + \left(\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}\right) \cos 2\theta_c$

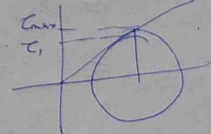
$\sigma_n = \left(\frac{450+150}{2}\right) + \left(\frac{450-150}{2}\right) \times \cos(2\times 60) = 225$ KPas

$\tau_f = \left(\frac{\sigma_1 - \sigma_3}{2}\right) \sin 2\theta_c = \left(\frac{450-150}{2}\right) \sin(2\times 60)$

$= 129.9$ kPas.

$\tau_f = \bar{\sigma}_n \tan\phi = 225 \tan 30$

$\tau_{max} = \left(\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}\right) = \left(\frac{450 - 150}{2}\right) = 150$ Kpas.



Solution (40)

$\sigma_H = 170$ kPa and $\sigma_V = 300$ kPa

hydrostatic Pressure = 30 kPa

U) at failure = 94.50

Shear stress at vertical = 0

$c' = 0$

$\phi' = 36$

Shear stress (S) = ?

$\bar{\sigma}_H = \sigma - U$

$= 75.5$

$\bar{\sigma}_V = 205.5$

$\bar{\sigma}_n = \left(\frac{\bar{\sigma}_V + \bar{\sigma}_H}{2}\right) + \left(\frac{\bar{\sigma}_V - \bar{\sigma}_H}{2}\right)\cos 2\theta_c$

$\bar{\sigma}_n = \left(\frac{205.5 + 75.5}{2}\right) + \left(\frac{205.5 - 75.5}{2}\right)\cos 2\theta_c$

$= 140.5 + 65 \times \cos(2 \times 63)$

$= 90$

hydrostatic Pressure

$U = 30 + 94.5$

$\theta_c = 45 + \frac{\phi}{2}$

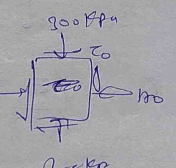
$\theta_c = 45 + 18$

63

$S = c' + \bar{\sigma}_n \tan\phi$

$= 0 + 90 \times \tan(36)$

$= 65.38$



$\bar{\sigma}_n$ = to find out

$\theta_c = 63$

$\bar{\sigma}_n = 72.3$

$S = \bar{\sigma}_n \tan\phi = 72 \tan 36 = 52.5$ kPa

$S = c^0 + \bar{\sigma}_n \tan\phi = \tau_f = \left(\frac{\sigma_1 - \sigma_3}{2}\right)\sin(2\theta_c)$

$= \left(\frac{300 - 170}{2}\right) \times \sin(2 \times 63)$

$= 52.5$

CHAPTER-9

Earth Pressure

* In the designing of retaining wall sheet pile wall and other retaining structure pressure exerted by material retain by these structures should also considered. this pressure exerted by retain material is termed as earth pressure and retaining material is termed as backfill which may be Horizontal and inclined.

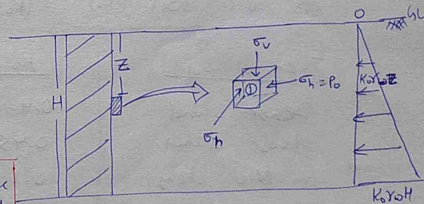
① Earth Pressure at rest cond

If the wall is rigid and unyielding, soil retained by the wall is in the rest cond and there is no displacement or deformation then pressure exerted by soil on the wall is termed as earth pressure at rest cond

σ_h = lateral earth pressure

σ_v = vertical earth pressure

$\frac{\sigma_h}{\sigma_v}$ = K = earth pressure coefficient



① At Rest cond — lateral strain $\epsilon_h = 0$

$\epsilon_h = \frac{\sigma_h}{E} - \mu\frac{\sigma_h}{E} - \mu\frac{\sigma_v}{E} = 0$

(μ = Poisson's ratio)

$\sigma_h(1 - \mu) = \mu\sigma_v$

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

Earth Pressure coeffi at Rest cond $K_0 = \frac{\mu}{1-\mu}$

$\because \frac{\sigma_h}{\sigma_v} = K_0 \Rightarrow \sigma_h = K_0 \sigma_v = P_0$

Earth Pressure at Rest cond $P_0 = K_0 \sigma_v$ ****

on the wall 1) at any depth Z, $\sigma_v = \gamma z$

② Earth pressure $P_0 = K_0 \sigma_v = K_0 \gamma z$

③ at $z = 0$ $P_0 = 0$

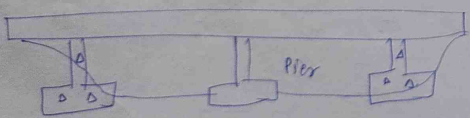
at $z = H$ $P_0 = K_0 \gamma H$

Note For pure sand Jacky $K_0 = (1 - \sin\phi)$ ***

for o.c soil $(K_0) = (K_0)_{N.C\ soil} \sqrt{OCR}$

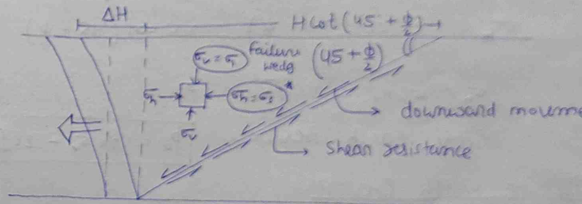
Type of soil	K_0
Dense sand	0.4-0.45
loose sand	0.45-0.5
N.C clay	0.5-0.6
Compacted soil	0.7-0.8

Note Basement wall and bridge abutments are designed at Rest cond



Active earth Pressure

If the wall moves away from the backfill then a portion of soil will also try to move along with the wall this portion is called failure wedge. Due to this movement resistance of the soil mobilized at upward and outward direction on the rupture surface which decreases the pressure on the wall this pressure decreases upto an extent when entire shear resistance is mobilized. the minm pressure acting on the wall in this stage is termed as active earth pressure.



As per Rankin's theory for cohesionless soil

At plastic equilibrium

$\sigma_1 = \sigma_3 \tan^2(45 + \frac{\phi}{2}) + 2c \tan(45 + \frac{\phi}{2})$

$\sigma_v = \sigma_H \tan^2(45 + \frac{\phi}{2}) + \cancel{2c \tan(45 + \frac{\phi}{2})}$

$\frac{\sigma_v}{\sigma_H} = \tan^2(45 + \frac{\phi}{2})$

$\frac{\sigma_H}{\sigma_v} = \cot^2(45 + \frac{\phi}{2})$ or $\frac{1}{\tan^2(45 + \frac{\phi}{2})} = K_a$

Active earth Pressure coefficient

$K_a = \cot^2(45 + \frac{\phi}{2}) = \frac{1 - \sin\phi}{1 + \sin\phi}$ ****

$\because \frac{\sigma_h}{\sigma_v} = K_a \Rightarrow \sigma_h = K_a \cdot \sigma_v = p_a$

"Bell" taken forward the Rankin's theory for cohesive soil

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

$$\div \tan\left(45^\circ + \frac{\phi}{2}\right)$$

$$\sigma_v \cot^2\left(45 + \frac{\phi}{2}\right) \pm 2c\tan\left(45 + \frac{\phi}{2}\right) + \sigma_h$$

$$\sigma_h = \sigma_v \cot^2\left(45 + \frac{\phi}{2}\right) - 2c\cot\left(45 + \frac{\phi}{2}\right)$$

$$\sigma_h = \sigma_v K_a - 2c\sqrt{K_a} = p_a$$ ★★★★

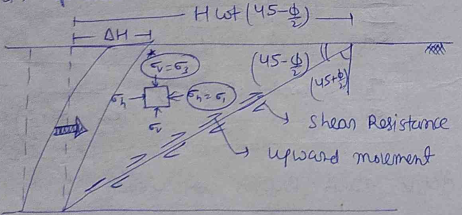
Active earth pressure = $\boxed{p_a = K_a \sigma_v - 2c\sqrt{K_a}}$

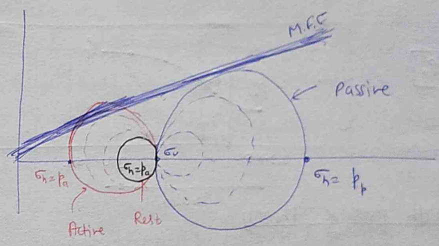
③ Passive earth Pressure

If the wall moves towards the backfill then shear resistance of soil built's up against the wall. Due to which pressure on the wall increases.

② Pressure on the wall increases upto the extent when entire shear ~~strength~~ resistance is mobilized

③ The maximum pressure acting on the wall is termed as passive earth pressure





As per Rankin's theory for cohesionless soil

At plastic equilibrium $\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)$ (→ 0)

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

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

Passive earth pressure coeff. ★★★

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

"Bell" taken forward the Rankin's theory for cohesive soil

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

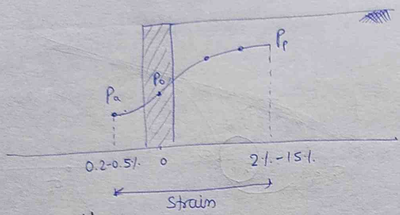
$$\sigma_h = \sigma_v \cdot K_p + 2c\sqrt{K_p} = p_p \qquad \left(\because \tan^2\left(45 + \frac{\phi}{2}\right) = K_p\right)$$

Passive earth pressure = $\boxed{p_p = K_p \sigma_v + 2c\sqrt{K_p}}$ ★★★★

$$\boxed{K_a \cdot K_p = 1}$$

Note: Strain required in active stage is in the order of 0.2% to 0.5% ......
0.2% is for dense sand and 0.5 is for loose sand
Strain reqd in passive stage is in the order of 2% to 15%. 2% is for dense sand and 15% is for loose sand

$P_a < P_o < P_p$



- Ramkin's earth pressure theory

Assumption

1) Soil is Homogeneous, isotropic, semi-infinite, elastic, dry and cohesionless.
2) the ground surface is planar which may be Horizontal and inclined.
3) The face of wall in contact with backfill is vertical and Smooth.
4) Soil is in the state of Plastic equilibrium in passive and active earth pressure cond
5) the Rupture surface is planer

Case ① Dry / Moist cohesionless backfill

Five step method

Step① At any depth Z , $\sigma_v = \gamma Z$

***

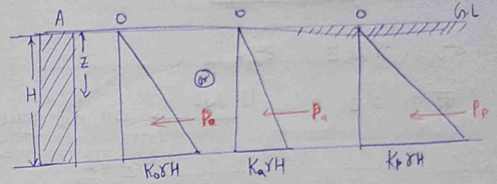
Step②

At Rest cond.	$P_o = K_o \sigma_v$		$= K_o \gamma Z$
At Active Stage	$P_a = K_a \sigma_v - 2c\sqrt{K_a}$ (c → 0)		$= K_a \gamma Z$
At Passive Stage	$P_p = K_p \sigma_v - 2c\sqrt{K_p}$ (c → 0)		$= K_p \gamma Z$

$\left(\frac{kN}{m^2}\right)$

Step ③ at Z=0 [A] , $P_o = 0$, $P_a = 0$, $P_p = 0$

at Z = H [B] , $P_o = K_o \gamma H$ $P_a = K_a \cdot \gamma H$, $P_p = K_p \cdot \gamma H$



Step④ total earth Pressure / Earth Pressure force / earth Pressure thrust | or unit length of wall $\left(\frac{kN}{m}\right)$

$P_o = \frac{1}{2} K_o \gamma H^2$, $P_a = \frac{1}{2} K_a \gamma H^2$, $P_p = \frac{1}{2} K_p \gamma H^2$

Step⑤ $\bar{Z} = \frac{H}{3}$ from Base

Case② Cohesionless backfill with surcharge

Five step Method

Step① At any depth Z or $\sigma_v = q + \gamma Z$

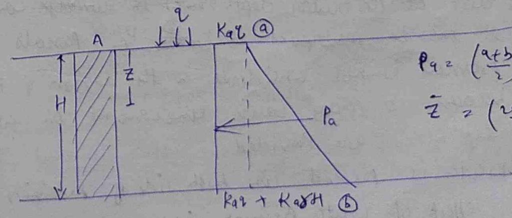
***

Step②

At Rest cond	$P_o = K_o \sigma_v$	$= K_o q + K_o \gamma Z$
Active cond	$P_a = K_a \sigma_v - 2c\sqrt{K_a}$ (c → 0)	$= K_a q + K_a \gamma Z$
Passive cond	$P_p = K_p \sigma_v + 2c\sqrt{K_p}$ (c → 0)	$= K_p q + K_p \gamma Z$

Step ③ at Z=0 [A] $P_a = K_a \cdot q$

at Z = H [B] $P_a = K_a q + K_a \gamma H$



$P_a = \left(\frac{a+b}{2}\right) H$ $\left(\frac{kN}{m}\right)$

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

Case - ③ <u>Submerged Backfill</u>

Step ① $\sigma_v = \gamma_{sat} Z = \bar{\sigma_v} + U$

$\bar{\sigma_v} = \gamma' Z$, $U = \gamma_w Z$ → will not multiple by K_a or K_p

Step ②

At Rest cond	$P_o = K_o \bar{\sigma_v} + U$	$= K_o \gamma' Z + \gamma_w Z$
At Active stage	$P_a = K_a \bar{\sigma_v} - 2c\sqrt{K_a} + U$	$= K_a \gamma' Z + \gamma_w Z$
Passive stage	$P_p = K_p \bar{\sigma_v} + 2c\sqrt{K_p} + U$	$= K_p \gamma' Z + \gamma_w Z$

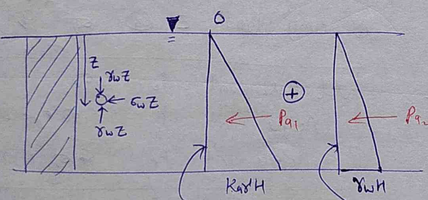
Step ③ at $Z = 0$ [A] $P_a = 0$

at $Z = H$ [B] $P_a = K_a \gamma' . H + \gamma_w H$

Step ④ $P_a = P_{a_1} + P_{a_2} = \frac{1}{2} K_a \gamma' H^2 + \frac{1}{2} \gamma_w H^2$

Step ⑤ $\bar{Z} = \frac{H}{3}$ from base

Case- ③



Note

Water will exert equal Pressure in all dirn at particular depth ($\gamma_w Z$) in submerge cond which will not be multiplied by K_a, K_p (pascals law)

Water pressure is not considered in the design of Retaining wall because it may cause uneconomic in construction thus weep hole's are Provided.

→ If water is present in both side of retaining wall then effect of water pressure is cancelled out

Case ④ <u>Stratified backfill</u>

Case Ⓐ $\phi_1 > \phi_2$ (Assume) $K_{a_1} < K_{a_2}$ $K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$

$K_{p_1} > K_{p_2}$ $K_p = \frac{1 + \sin\phi}{1 - \sin\phi}$

<u>In soil ① at depth Z below Point A</u>

1) $\sigma_v = \gamma_1 Z$

2) $P_a = K_{a_1} \sigma_v - 2c\sqrt{K_{a_1}} = K_{a_1} \gamma_1 Z$

3) at $Z = 0$ [A] $P_a = 0$

at $Z = H$ [A] $P_a = K_a \gamma_1 H_1$

$P_p = K_{p_1} \sigma_v + 2c\sqrt{K_{p_1}} = K_{p_1} \gamma_1 Z$

$P_p = 0$

$P_p = K_{p_1} \gamma_1 H_1$

<u>In soil ② at depth Z below Point 'B'</u>

$U = \gamma_w Z$

1) $\bar{\sigma_v} = \gamma_1 H_1 + \gamma_2' Z$

2) $P_a = K_{a_2} \bar{\sigma_v} - 2c\sqrt{K_{a_2}} + U$

$P_a = K_{a_2} \gamma_1 H_1 + K_{a_2} \gamma_2' Z + \gamma_w Z$

3) at $Z = 0$ [B] $P_a = K_{a_2} \gamma_1 H_1$

at $Z = H_2$ [C] $P_a = K_{a_2} \gamma_1 H_1 + K_{a_2} \gamma_2' H_2 + \gamma_w H_2$

$P_p = K_{p_2} \bar{\sigma_v} + 2c\sqrt{K_p} + U$

$P_p = K_{p_2} \gamma_1 H_1 + K_{p_2} \gamma_2' Z + \gamma_w Z$

$P_p = K_{p_2} \gamma_1 H_1$

$P_p = K_{p_2} \gamma_1 H_1$

$P_p = K_{p_2} \gamma_1 H_1 + K_{p_2} \gamma_2' H_2 + \gamma_w H_2$

4) <u>total earth Pressure & unit length of wall ($\frac{kN}{m}$)</u>

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

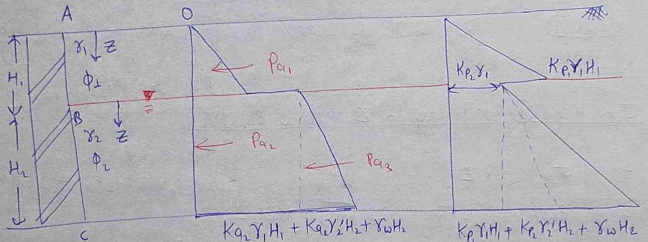
$P_{a_1} = \frac{1}{2} . K_{a_1} \gamma_1 H_1 . H_1$ will act at $\bar{Z_1} = H_2 + \frac{H_1}{3}$

$P_{a_2} = K_{a_2} \gamma_1 H_1 H_2$ $\bar{Z_2} = \frac{H_2}{2}$

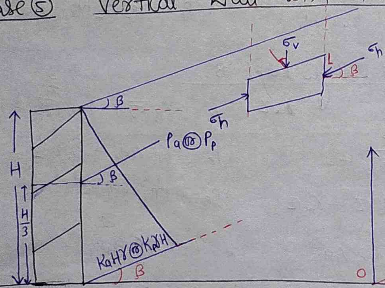
$P_{a_3} = \frac{1}{2} (K_{a_2} \gamma_2' H_2 + \gamma_w H_2) H_2$ $\bar{Z_3} = \frac{H_2}{3}$

5) <u>Line of Action</u> $\bar{Z} = \frac{P_{a_1} \bar{Z_1} + P_{a_2} \bar{Z_2} + P_{a_3} \bar{Z_3}}{P_{a_1} + P_{a_2} + P_{a_3}}$

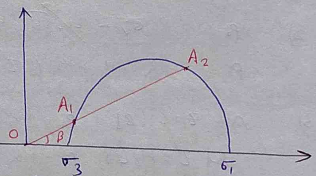
If $\phi_2 > \phi_1$ then dia. will interchange



Case ⑤ Vertical wall with inclined backfill (inclined surcharge)



assumption : σ_v and σ_H are conjugate stress



$OA_1 = \sigma_n$ and $OA_2 = \sigma_v$ Active stage

$OA_1 = \sigma_v$ $OA_2 = \sigma_h$ Passive stage

→ $P_a = \frac{1}{2} K_a \gamma H^2$ or $P_p = \frac{1}{2} K_p \gamma H^2$

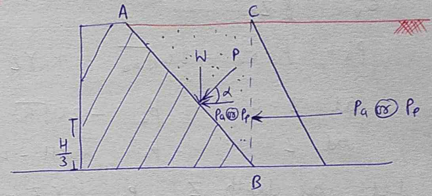
→ P_a or P_p will act at $\bar{Z} = \frac{H}{3}$ from base at an angle of β with Horizontal

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

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

$$\boxed{K_a . K_p = \cos^2\beta}$$

Case ⑥ नहीं आता है



→ $P_a = \frac{1}{2} K_a \gamma H^2$ or $P_p = \frac{1}{2} K_p . \gamma H^2$

P_a or P_p will act at $\frac{H}{3}$ from base

Resultant Earth Pressure

$P = \sqrt{W^2 + P_a^2}$ or $P = \sqrt{W^2 + P_p^2}$

W ⟶ weight of wedge ABC

P will act at $\frac{H}{3}$ at an angle of α with Horizontal

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

Active and Passive earth pressure for cohesive soil

① Rankin's theory taken forward by bell in cohesive soil as cohesive soils are partially self supporting soil. Hence, active earth pressure exerted by them on the wall is comparatively less than cohesionless soil but passive pressure exerted by them is comparatively more then cohesionless soil.

Case ① Dry / moist (cohesive soil) (Active stage)

① $\sigma_v = \gamma Z$

② $p_a = K_a \sigma_v - 2c\sqrt{K_a} = K_a \gamma Z - 2c\sqrt{K_a}$

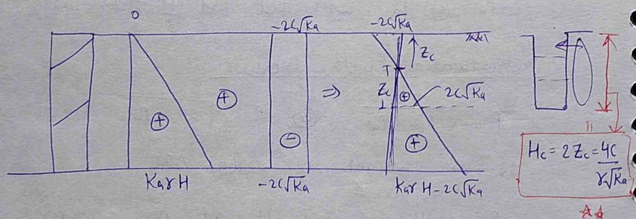
③ at $Z = 0$ $p_a = -2c\sqrt{K_a}$

at $Z = H$ $p_a = K_a \gamma H - 2c\sqrt{K_a}$

Let at depth Z_c, $p_a = K_a \gamma Z_c - 2c\sqrt{K_a} = 0$

$$Z_c = \frac{2c\sqrt{K_a}}{K_a \gamma} = \frac{2c}{\gamma\sqrt{K_a}}$$

imp



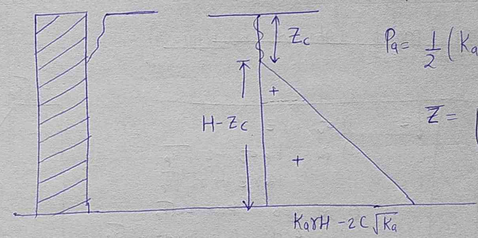
Note total Net pressure in cohesive soil in active stage upto a depth of 2 Hence these cohesive soil can be stand their vertical phase upto depth without any lateral support thereby this depth of twice (Z) is termed as critical depth of unsupported cut.

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

Note② because of presence of tention. tension crack may developed in cohesive soil as a result of which it doesn't remain bonded to the face of wall portion of the soil is considered to be inert no pressure on the wall

Step ④ total Active earth pressure or unit length of wall

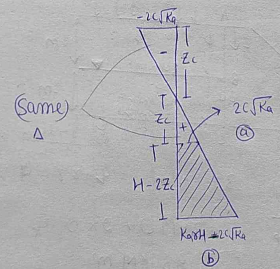
Case Ⓐ When tension cracks are developed



$$P_a = \frac{1}{2}\left(K_a\gamma H - 2c\sqrt{K_a}\right)(H - Z_c)$$

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

Case (b) When tension cracks are not developed

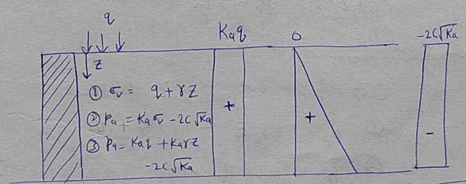


$$P_a = \left(\frac{a+b}{2}\right)(H - 2Z)$$

$$\bar{Z} = \left(\frac{2a+b}{a+b}\right)\left(\frac{H - 2Z}{3}\right)$$

Active stage

Case ② Cohesive backfill with uniform surcharge

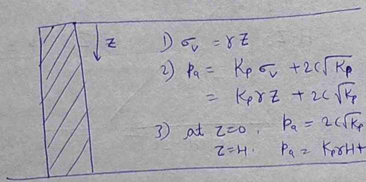


Depth of tension crack $Z_c = \dfrac{2c\sqrt{K_a} - K_a q}{K_a \gamma}$

Surcharge required for no tension crack $q = \dfrac{2c\sqrt{K_a}}{K_a} = \dfrac{2c}{\sqrt{K_a}}$

Passive earth Pressure

1) Dry / moist cohesive backfill



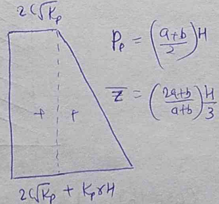
1) $\sigma_v = \gamma Z$

2) $p_a = K_p \sigma_v + 2c\sqrt{K_p}$

$= K_p \gamma Z + 2c\sqrt{K_p}$

3) at $Z=0$, $p_a = 2c\sqrt{K_p}$

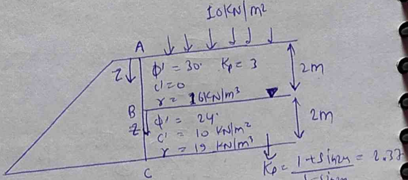
$Z=H$, $p_a = K_p\gamma H + 2c\sqrt{K_p}$



$P_p = \left(\frac{a+b}{2}\right)H$

$\bar{Z} = \left(\frac{2a+b}{a+b}\right)\frac{H}{3}$

W.B Question (28) Page (55)



$K_p = \frac{1+\sin 24}{1-\sin 24} = 2.37$

$\gamma_w = 10 kN/m^3$

Rankines Passive force per unit length

Step ① $\sigma_v = \gamma Z = 16\times(2) = 32 kN/m^2 = q + \gamma z = 42 kN/m^2$

Step ② $p_p = K_p \sigma_v + 2c\sqrt{K_p} = ~~3\times(16)+2\times 0\times\sqrt{K_p}~~$

$= ~~98 kN/m^2~~$

$= 3\times(10+16Z) =$

Step ③

at $Z=0$, $p_p = 30$

at $Z=2$, $p_p = 126$

and ② Processes

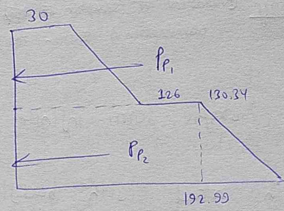
① $\sigma_v = q + \gamma_1 H_1 + \gamma_1' Z$ $(U = \gamma_w Z)$

② $p_p = K_{p_2}\sigma_v + 2c\sqrt{K_p} + (U)$

③ $p_p = 2.37[10 + 16\times2 + 9Z) + 2\times10\sqrt{2.37} + 10Z$

3) at $Z=0$, $p_p = 130.34$

at $Z=2$, $p_p = 192.99$



$P_p = P_{p_1} + P_{p_2}$

$= \left[\frac{30+126}{2}\right]\times 2$

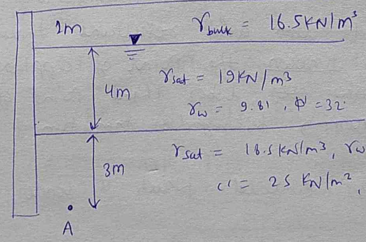
$+ \left(\frac{130.34+192.9}{2}\right)\times 2$

$= 479.2 kN/m$

Solution (33)

Step ① $P_a = \frac{1}{2}(K_a\gamma H - 2c\sqrt{K_a})(H - Z_c)$

$\bar{Z} = \left(\frac{H-Z_c}{3}\right)$ from base



$\bar{\sigma_v} = [1\times\gamma_1 + U\gamma_2 + 3\gamma_3] - 7\gamma_w$

$= 1\times16.5 + 4\times19 + 3\times18.5 - 7\times9.81$

$= 79.33$

$K_a = \frac{1-\sin 24}{1+\sin 24} = 0.421$

$p_a = K_a\bar{\sigma_v} - 2c\sqrt{K_a} + U$

$= 0.421(79.33) - 2\times25\times\sqrt{0.421} + 7\times9.81$

$= 69.67 kN/m^3$

Coulomb Wedge theory (Numerical में Question नहीं आते) (Assumption में Question आया)

Assumptions

1) Soil is Homogeneous, isotropic, semi-infinite elastic dry and cohesionless

2) The face of wall in contact with backfill is vertical or inclined and is rough

3) The failure wedge acts as a rigid body and stresses over are uniformly distributed,

4) The failure is essentially 2D and rupture surface is planer and passes through the heel of the wall

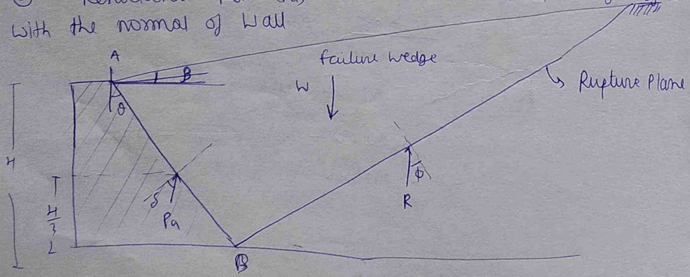
5) The location and dirn of resultant thrust b/w wall and soil is known the point of application is taken at the ~~lower~~ ~~otes~~ lower third point of the wall ~~whi~~ by assuming triangular distribution of earth pressure.

6) Wedges failure is considered which is under equilibrium of three forces :-

① self wt of wedge ABC, acting vertically downward

② Resultant Reac (R) act as downward angle of ϕ with normal to the slip plane

③ Resultant Reac (P_a) acts as downward angle of δ with the normal of wall



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

P_a will act at $\bar{z} = \frac{H}{3}$ from base of an angle 'δ' with normal of the wall

Numerical नहीं आता है GATE में

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

Note ① If wall is vertical
If backfill is Horizontal
If wall is smooth
$(K_a)_{coulomb's} = (K_a)_{Rankine's}$

~~Note ② Effect of wall friction~~
~~• Active earth pressure reduce and Passive pressure ↑~~

θ = Angle of inclined face of wall with vertical
β = Angle of backfill
ϕ = friction angle of soil
δ = friction angle b/w wall and soil
Generally $\delta = \frac{2}{3}\phi$ for normal wall

Note ② Effect of Wall friction

- Active earth Pressure reduce and Passive Pressure increases

Note ③ In coulomb theory failure is assumed to be planer bt in actual it is spiral specially in passive case Hence coulomb theory is not preferred for passive earth pressure analysis.

Note ④ In Rankine's theory elemental failure is considered bt in coulomb theory wedge failure is considered.

→ earth Pressure can also be analysed by
culman's theory Rebhmann's theory
Poncelete or Thornburn theory

culman's theory is based upon coulomb's wedge theory.

Sheet Pile wall

Consist of No. of Sheet Pile driven side by side to fall a continuous vertical wall into the medium which used to retain earth mass Generally it is used in water front structures, temporary construction, river training works to prevent the piping failure below the dams and to prevent walls of excavation from failure

Type's of sheet Pile wall

① continuous sheet pile wall

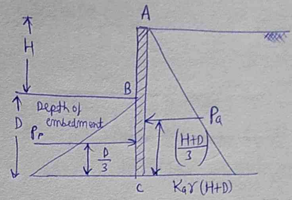
1) In actual rotation of Sheet pile wall takes Place about Pivot Point o Hence sheet Pile wall is subjected to

	Active	Passive
Right side	ABO	OC
left side	OC	OB

Case (A)

2) For cohesive soil/sand : for simplicity in calculation rotation of sheet pile was is submerged to base point (Hence sheet pile is subjected to active pressure in right side of ~~A~~ ABC and also subjected to Passive pressure in left side BC

Important Numerical ** Question आते है



Minm depth of embedment

$\Sigma M_c = 0$

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

$\frac{1}{2} K_p \gamma D^2 \cdot \frac{D}{3} - \frac{1}{2} K_a \gamma (H+D)^2 \left(\frac{H+D}{3}\right) = 0$

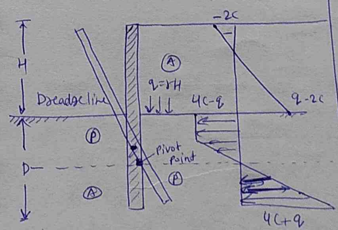
solve for Drain

If F.O.S. is given

$F.O.S = \frac{M_R}{M_O} = \frac{P_p \frac{D}{3}}{P_a\left(\frac{H+D}{3}\right)} = \frac{\frac{1}{2} K_p \gamma D^2 \cdot \frac{D}{3}}{\frac{1}{2} K_a \gamma (H+D)^2 \left(\frac{H+D}{3}\right)}$

Solve for D safe

Case b for Pure cohesive soil/ clay ($\phi = 0$), $K_a = K_p = 1$ Assume the rotation about Pivot Point O



② Anchored sheet Pile / Anchored Bulk head

③ Braced sheeting (Determination of strut load)

Important है GATE के Point of view से

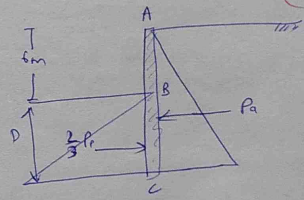
(13) Sheet Pile ht = 6m

$\frac{2}{3}$ theoretical Passive resistance

$\gamma = 19\ kN/m^3$ and $\phi = 30°$

using Approximate method

Depth = ? $K_a = \frac{1}{3}$ (Important



$\Sigma M_c = 0$

$\frac{2}{3} P_p \cdot \frac{D}{3} - P_a\left(\frac{H+D}{3}\right) = 0$

$\frac{2}{3}\left(\frac{1}{2} K_p \gamma D^2 \cdot \frac{D}{3}\right) - \frac{1}{2} K_a \gamma (H+D)^2 (\ldots = 0$

$\frac{2}{3} \times \frac{1}{2} \times 3 \times 19 \times \left(\frac{D^3}{3}\right) - \frac{1}{2} \times \frac{1}{3} \times 19 (\ldots = 0$

$D = 7.35m$