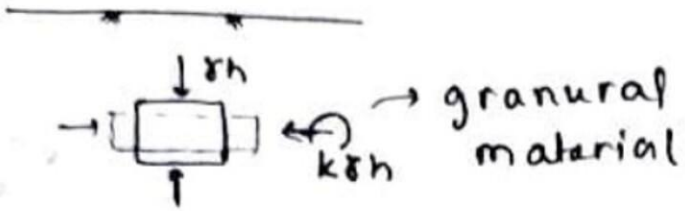
EWCE 343 — SA

LO1

24.11

lateral earth pressure

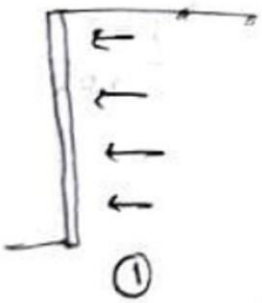


depend on :

- physical chars. of soil

cohesionless , cohesive , c-ϕ soil

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

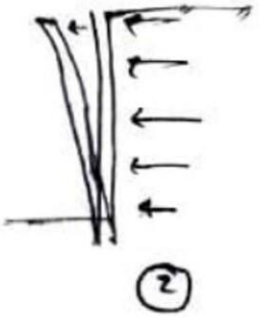


No deformation

stiff material

more resistance

earth pressure at rest

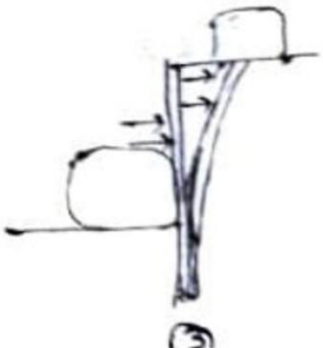


lateral expansion

active earth pressure

soil active

soil is giving ~~lot~~ load



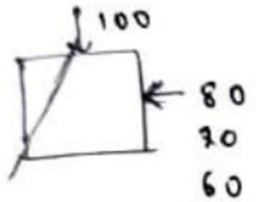
lateral contraction

passive earth pressure

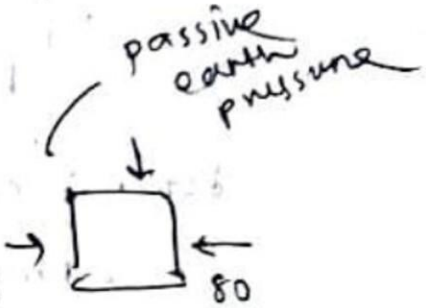
define coeff. of earth pressure at rest — k_o

$$k_o = \frac{k_o \gamma h}{\gamma h}$$

active. ka







80
70
60
50
40
(30) — er kom e pore jase
20

80
100
200
(300)
400
500

$$\frac{30}{100} = 0.3 = ka$$

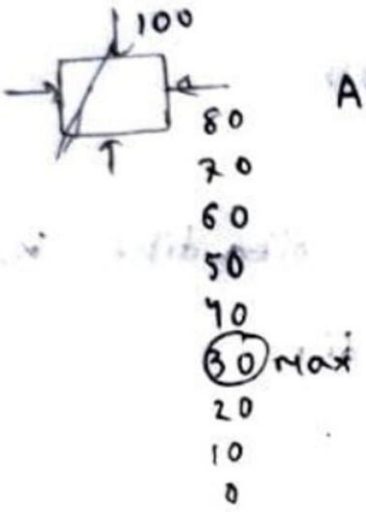
$$\frac{300}{100} = 3.0\ kp$$

L02

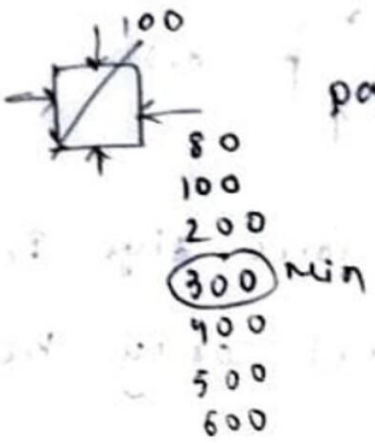
25.11

Define to earth pressure at rest

coeffi of active or passive earth pressure



Active



passive

we consider failure condition

exhane 30 ar 300 e fail kre

passive e deformation / displacement bestii lage

As k increases

2m $\gamma = 16$ kN/m³

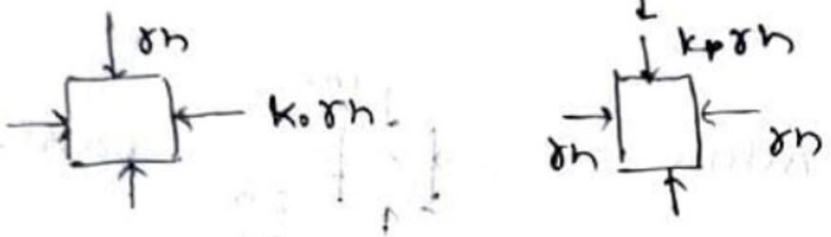
$\sigma = 2 \times 16 + 4 \times 18 = 104$

$\sigma' = 104 - 4 \times 18$

$= 64.8$ kN/m²

4m $\gamma_{sat} = 18$ kN/m³

Lateral earth pressure increase, hoile ki ki hoto pare



Jei dike force diye fail korabo oi dike Ko/Kp/Ka σh hobe not only horizontal dir.

Soil – granular material

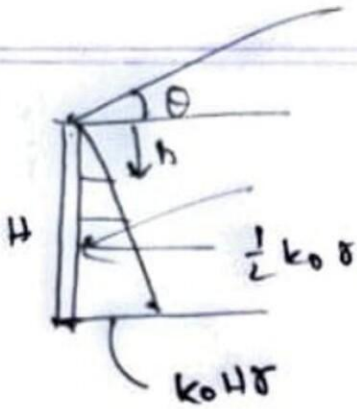
$$\epsilon_h = E\left[\sigma_h - \mu(\sigma_v + \sigma_h)\right]$$

$$0 = \ldots\ldots$$

$$\sigma_h = \frac{\mu}{1-\mu}\sigma_v = K_0 \sigma_v$$

Sand e Id dile – drained cond pabo

$$K_0 = 1 - \sin\phi'$$



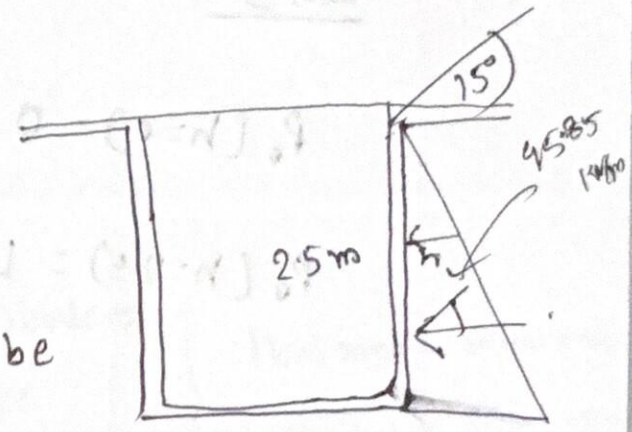
$\frac{1}{2} k_0 \gamma H . H = \frac{1}{2} k_0 \gamma H^2$

$(1 + 0.5 \tan\beta)^2$

Example

P22/

Step 1



For basement the lateral earth pressure induced will be earth pressured at rest

coeff. = K_0

$c = 0$

$\phi = 35°$

$\gamma = 18\ KN/m^3$

$OCR = 2$

$K_0 = (1 - \sin\phi)\ OCR^{\sin\phi}$

$= (1 - \sin 35°)\ 2^{\sin 35°}$

$= 0.635$

Step 2

$P_0(h) = K_0 \gamma h$

$= 0.635 \times 18 \times h$

$= 11.43\ h$

Step 3

$P_0 (h=0) = 0$

$P_0 (h=2.5) = 11.43 \times 2.5 = 28.58$ kPa

Step 4

$P_0 = \frac{1}{2} 28.52 \times 2.5 = 35.65$ KN/m

$\bar{y} = \frac{2.5}{3} = 0.83$ m

∴ correction factor for inclination

$(1 + 0.5 \tan\beta)^2 = (1 + 0.5 \tan 15^\circ)^3$

$= 1.29$

Point Ld $= 35.65 \times 1.29$

$= 45.84$ KN/m

~~On xm, if asked for~~

On xm, if asked for pressure dist, then draw else can use direct formula

$\frac{1}{2} \times 0.635 \times 18 \times 2.5^2$

$= 35.7$ KN/m

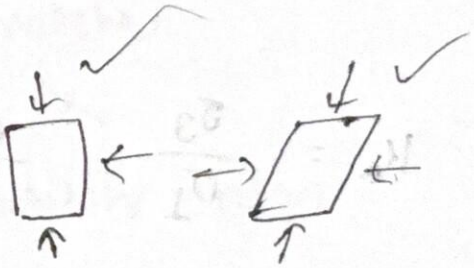
Earth Pressure Theories

Active earth pressure

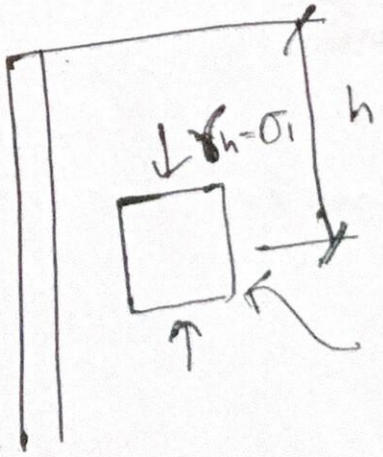
Rankine's Theory
Coulomb's Theory
} cohesionless, cohesive, C-ϕ : Horizontal Ground Surface, Inclined "

Rankine's Theory

Conjugate test →



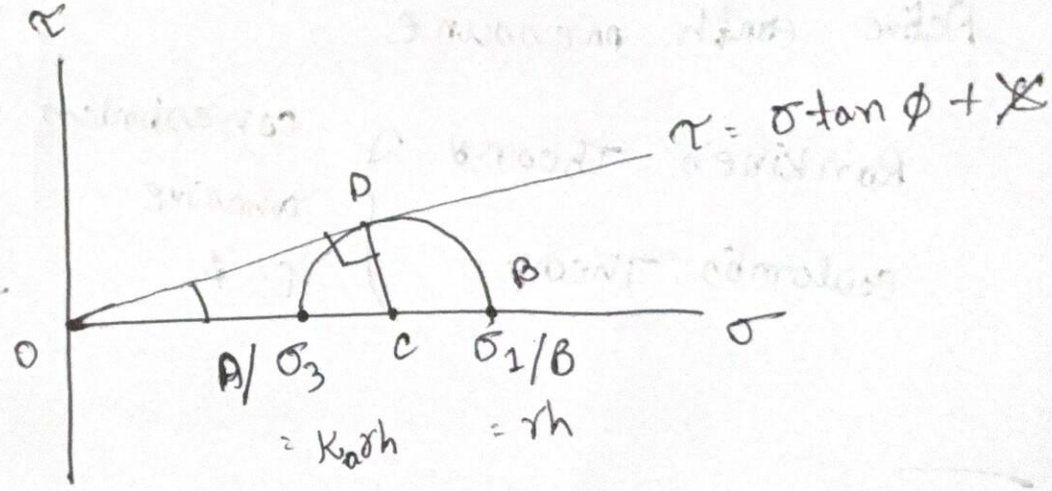
Assumptions
cohesionless
2. most impo



$\sigma_h > k_a \gamma h$

$(\sigma_1, 0)$:

$(\sigma_3, 0)$



$$K_a = \frac{\sigma_3}{\sigma_1} = \frac{OA}{OB} = \frac{OC - AC}{OC + BC}$$

$$= \frac{OC - CD}{OC + CD}$$

$$= \frac{1 - \frac{CD}{OC}}{1 + \frac{CD}{OC}}$$

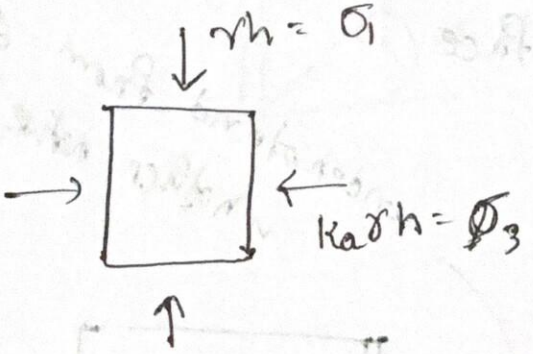
$$= \frac{1 - \sin\phi}{1 + \sin\phi}$$

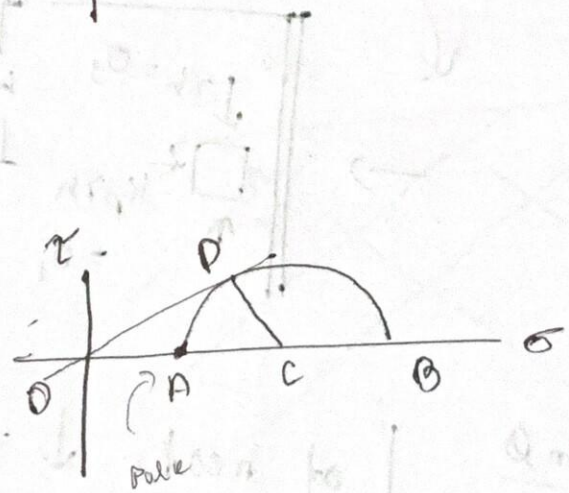
$$= \tan^2\left(45 - \phi/2\right)$$

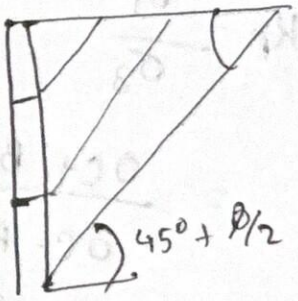
$$K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

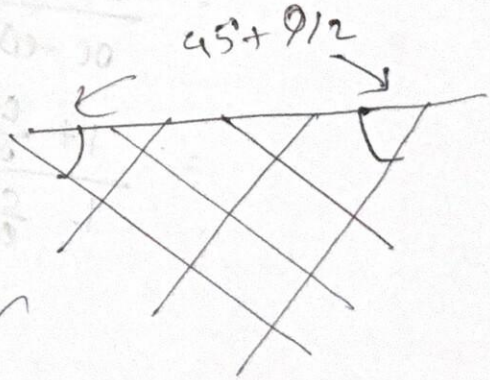
$$= \tan^2(45 - \phi/2)$$

Q8 / Draw the qualitative failure line for active cond









Rankine's Theory

Passive Earth Pressure

Cohesionless soil

Horizontal Ground surface

understand from dir whether active/passive

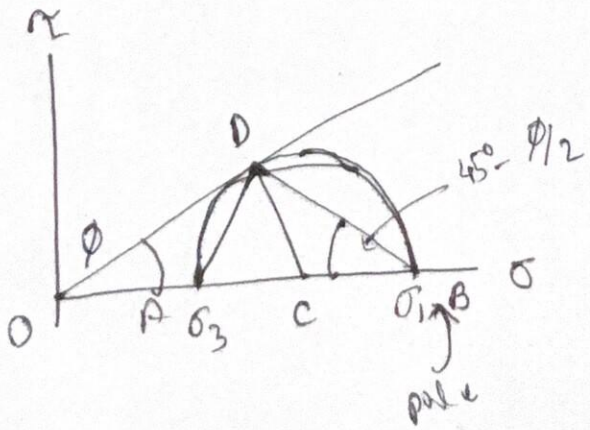
$$K_p = \frac{\sigma_1}{\sigma_3}$$

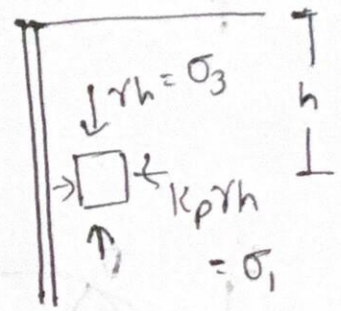
$$= \frac{OC + BC}{OC - AC}$$

$$= \frac{OC + CD}{OC - CD}$$

$$= \frac{1 + \frac{CD}{OC}}{1 - \frac{CD}{OC}} = \frac{1+\sin\phi}{1-\sin\phi}$$

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



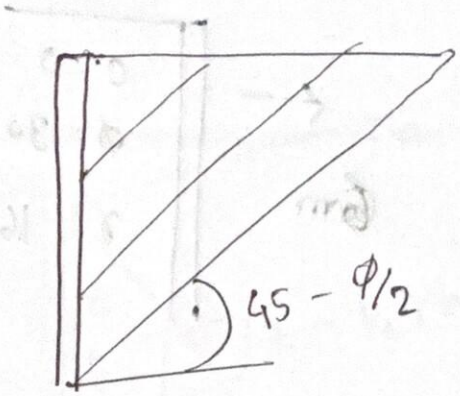


at rest, ↓ = →

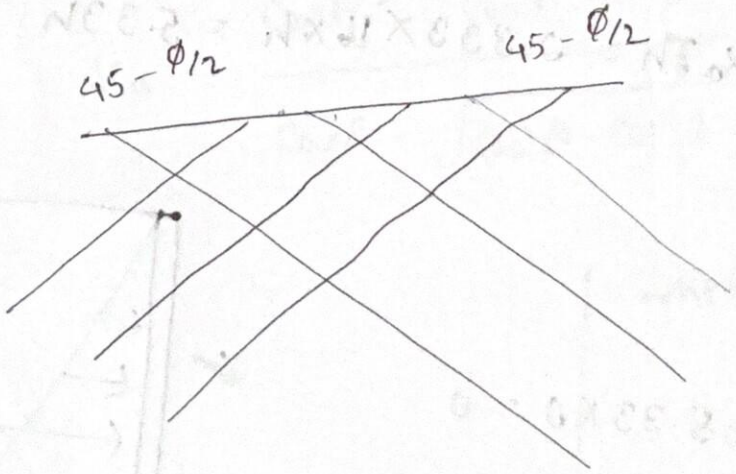
upon contraction (wall moves right → increases)

$\therefore$ σ $K_p \gamma h > \gamma h$

$\sigma_1 \quad \sigma_3$



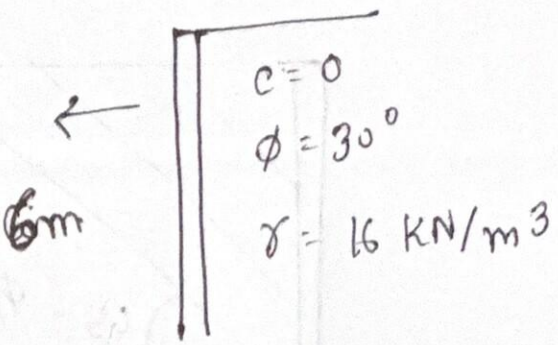
qualitative line (passive)



Step 1

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

$$= \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.333$$



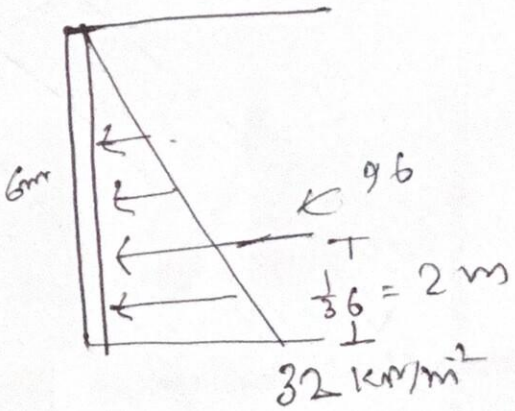
Step 2

$$P_a(h) = K_a \gamma h = 0.333 \times 16 \times h = 5.33h$$

Step 3

$P_a(h=0) = 5.33 \times 0 = 0$

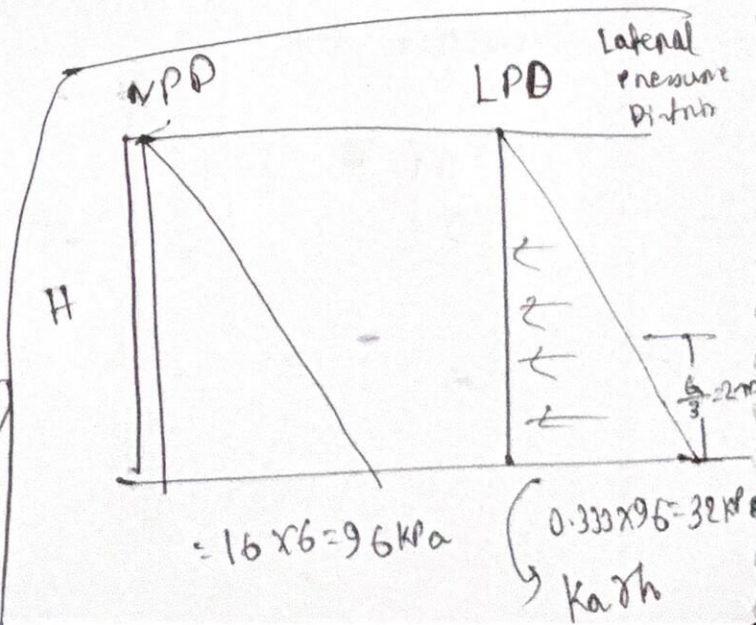
$P_a(h=6m) = 5.33 \times 6 = 32\ KN/m^2$



Step 4

$$P_a = \frac{1}{2} \times 32 \times 6$$

$$= 96\ KN/m$$

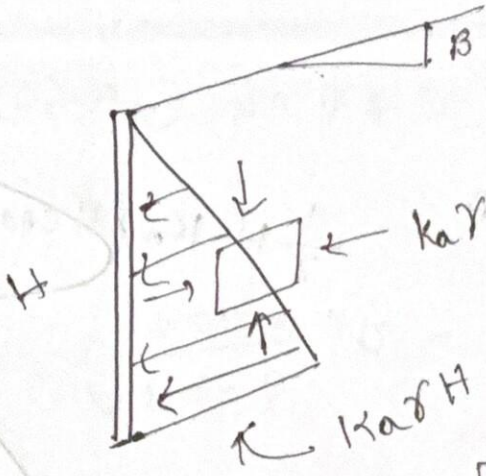


$$P_a = \frac{1}{2} K_a \gamma H \times H$$

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

If distribution curve not asked then just do like this

Inclined backfill surface ~~(Passive)~~



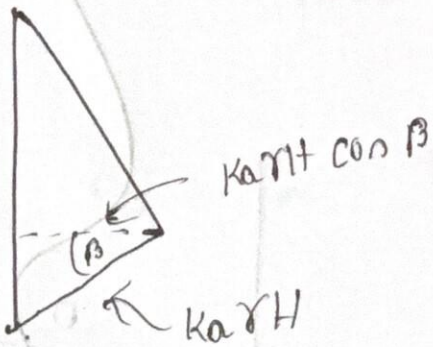
active

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

when horizontal, $\beta = 0^\circ$

$$K_a = \frac{1-\sqrt{1-\cos^2\phi}}{1+\sqrt{1-\cos^2\phi}}$$

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



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

$$= \frac{1}{2} K_a \gamma_a H^2 \cos\beta$$

calc active thrust

calc horizontal thrust comming on the wall

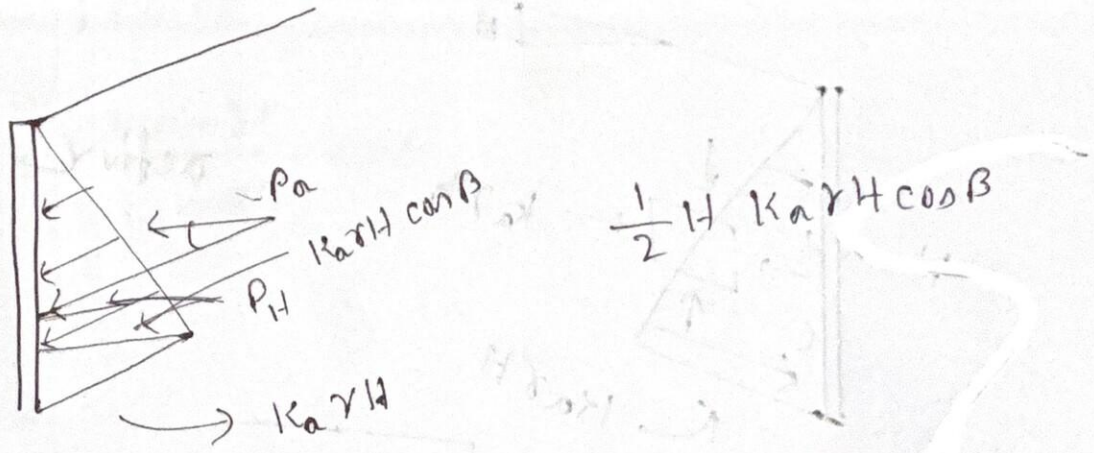
again X by cos β

$$P_h = \frac{1}{2} K_a \gamma H^2 \cos\beta \times \cos\beta$$

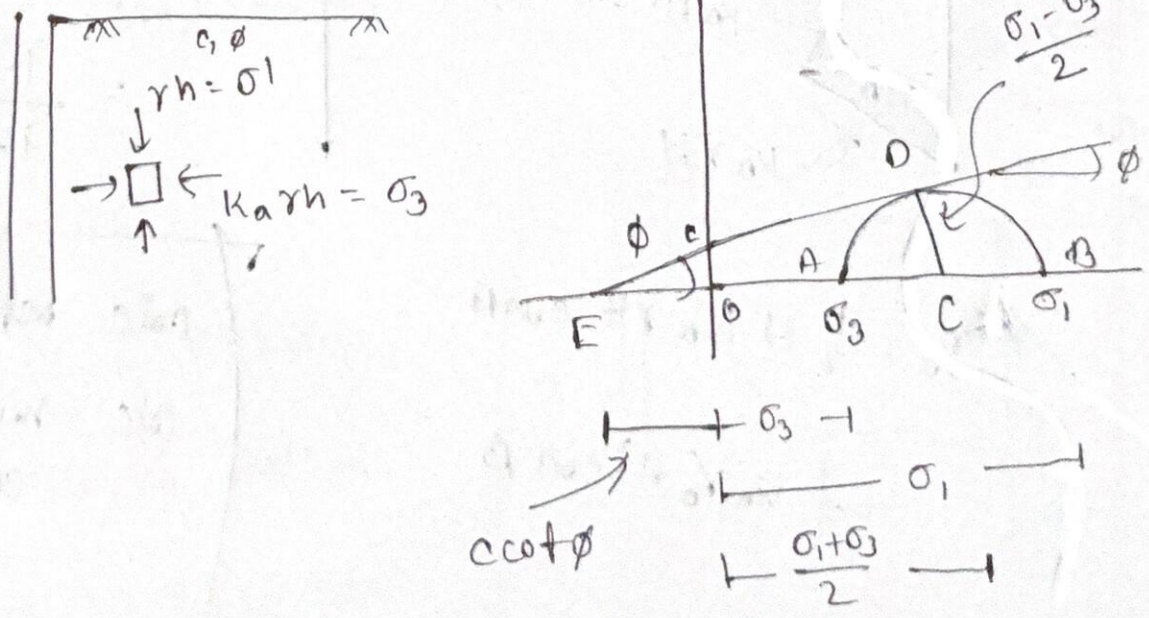
SA/LOS

08/12

Active Earth Pressure (inclined backfill surface)



Active Earth Pressure for Partially Cohesive (c-ϕ) soil : Rankine's Theory



$$\sin\phi = \frac{CD}{EC} = \frac{CD}{EO+OC} = \frac{\frac{\sigma_1+\sigma_3}{2}}{c\cot\phi + \frac{\sigma_1+\sigma_3}{2}}$$

$$\Rightarrow (\sigma_1+\sigma_2)\sin\phi + 2c\cot\phi\sin\phi = \sigma_1-\sigma_2$$

$$\Rightarrow \ldots$$

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

← connection

Now, $\frac{\cos\phi}{1+\sin\phi} = \ldots$

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

$$\Rightarrow \sigma_3 = P_{ac} = K_a\gamma h - 2c\sqrt{K_a}$$

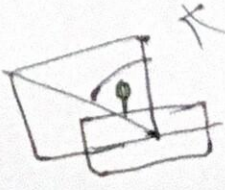
---

when $h = 0$

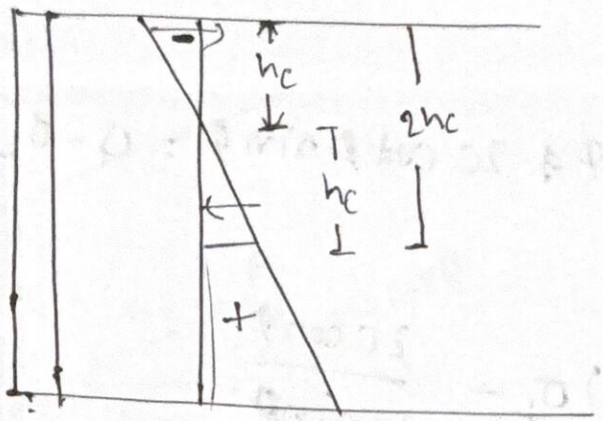
$$\sigma_3 = -2c\sqrt{K_a}$$

$$= -2c\tan\left(45-\frac{\phi}{2}\right) \quad [\phi < 90^\circ \text{ always}]$$

$\therefore$?



→ at $h=0$, $\rightarrow \sigma(-)$



tension – (−)

depth of tension crack h_c

at $2h_c$,
resultant force = 0
Theoretical unsupported ht, $H_a = 2h_c$

derive

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

$$\Rightarrow h_c = \frac{2c\sqrt{K_a}}{\gamma K_a} = \frac{2c}{\gamma}\frac{1}{\sqrt{K_a}}$$

Depth of tension crack, $h_c = \frac{2c}{\gamma}\frac{1}{\sqrt{K_a}}$

$$= \frac{2c}{\gamma}\frac{1}{\sqrt{\tan^2(45-\phi/2)}} = \frac{2c}{\gamma}\frac{1}{\tan(45-\phi)}$$

$$= \frac{2c}{\gamma}\cot(45-\phi/2)$$

$$= \frac{2c}{\gamma}\tan(45+\phi/2)$$

$H_a = 2h_c$

$$= \frac{4c}{\gamma}\tan(45+\phi/2)$$

For purely cohesive soil, $\phi = 0$

$$h_c = \frac{2c}{\gamma}$$

$$P_a = K_a \gamma h - 2c\sqrt{K_a}$$ ← prev

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

For passive EP

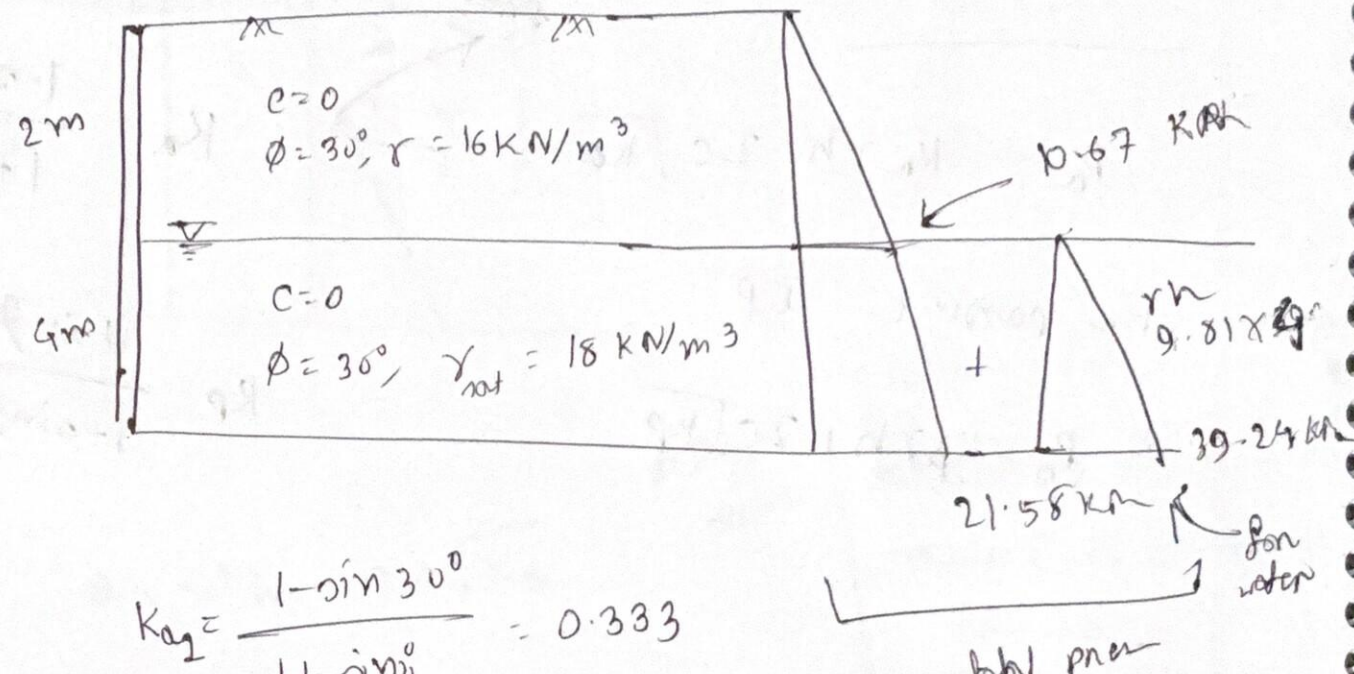
$$P_b = K_p \gamma h + 2c\sqrt{K_p}$$

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

For inclined ground surface (c-φ soil)

~~derive~~ not req for exam ← e.g. on hilly slope

Effect of water table



$$K_{a_1} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.333$$

---

hor. pressure → eff. ver. pre. × K_a

@2m → 2×16 = 32 → × K_a → 10.67

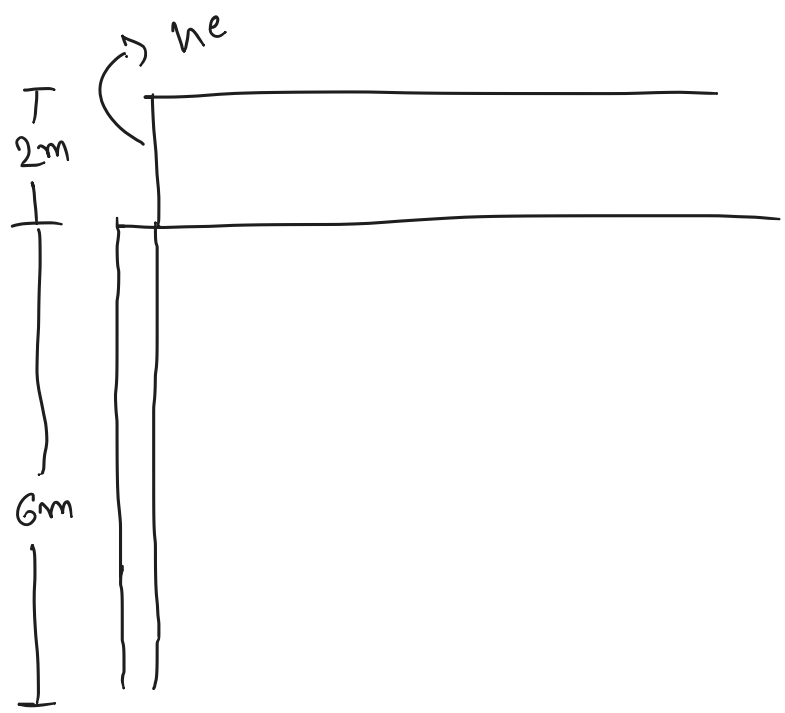
$$K_{a2} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.333$$

$2\times16 + 4\times(18-9.81) = 64.76$ → 21.58 kPa

09/12

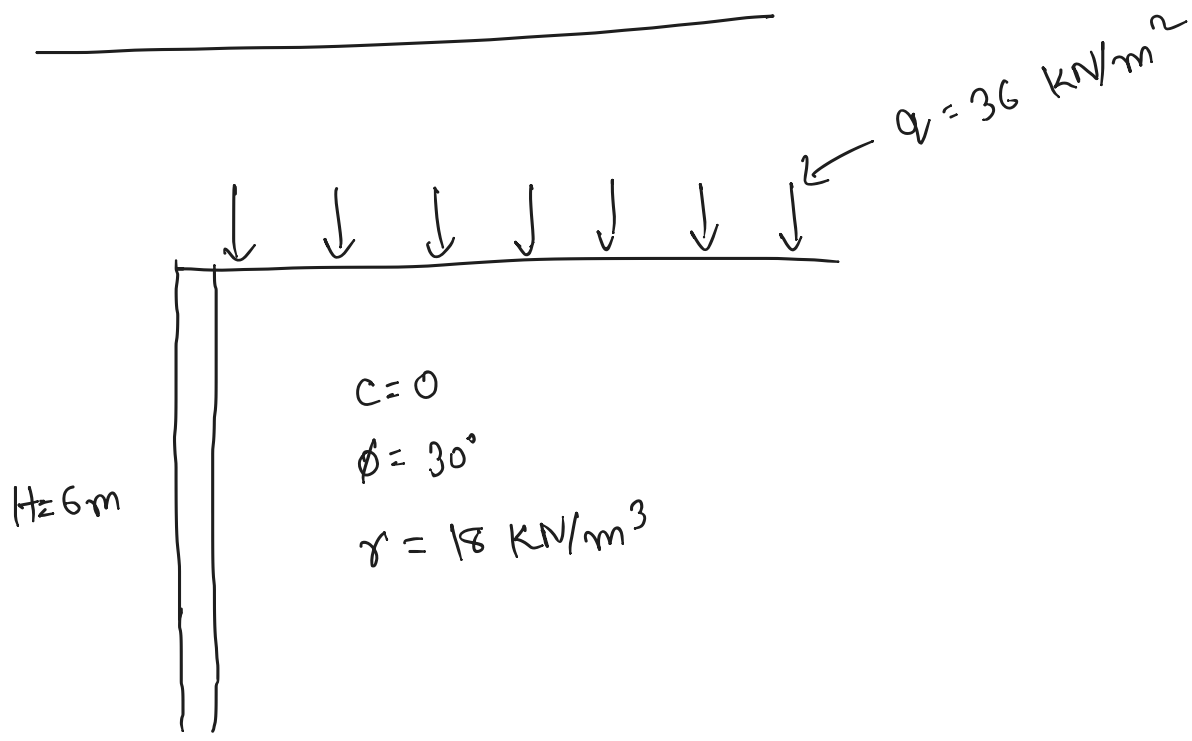
LO6

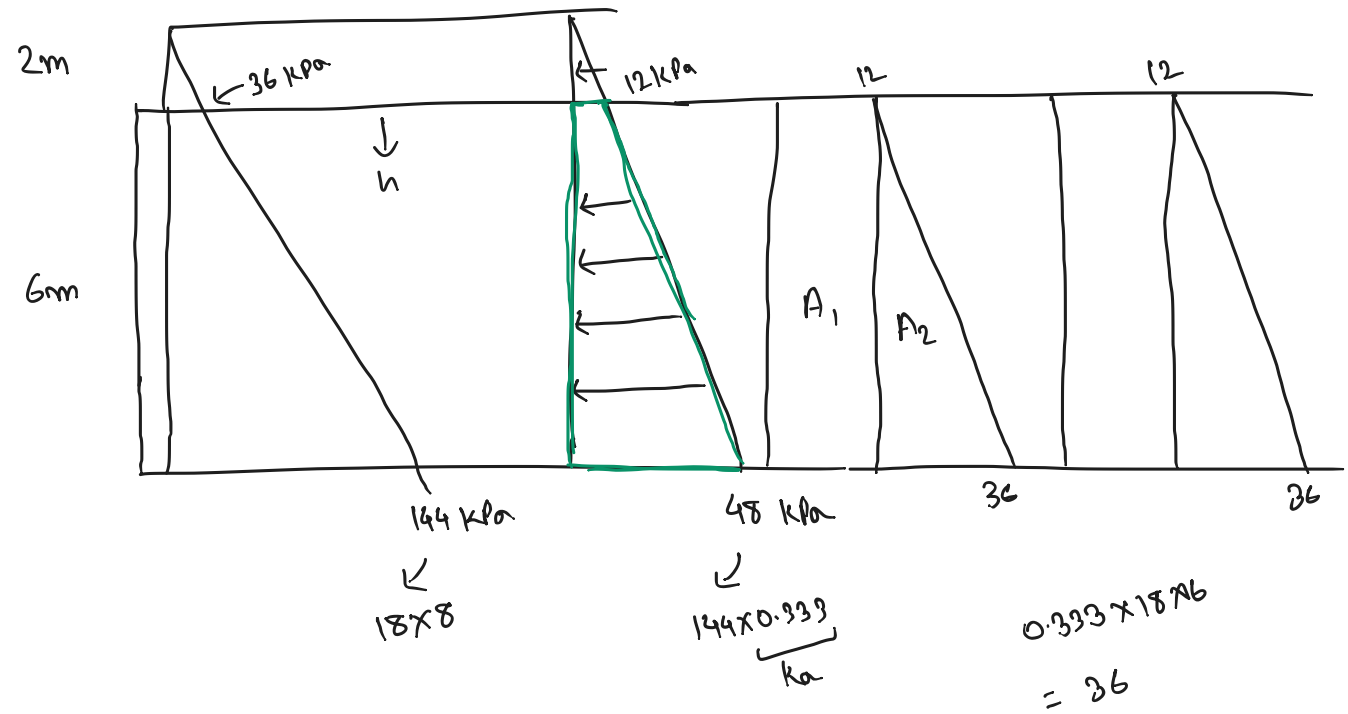
Effect of uniform surcharge



Active (or passive) pressure — cohesionless soil

Effect of stratification





Step 1

Equivalent height, $H_e = \frac{a}{\gamma}$

$= \frac{36}{18} = 2.0\ m$

Step 2

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.333$$

Step 3

$$P_a(h) = K_a \gamma (H_e + h)$$

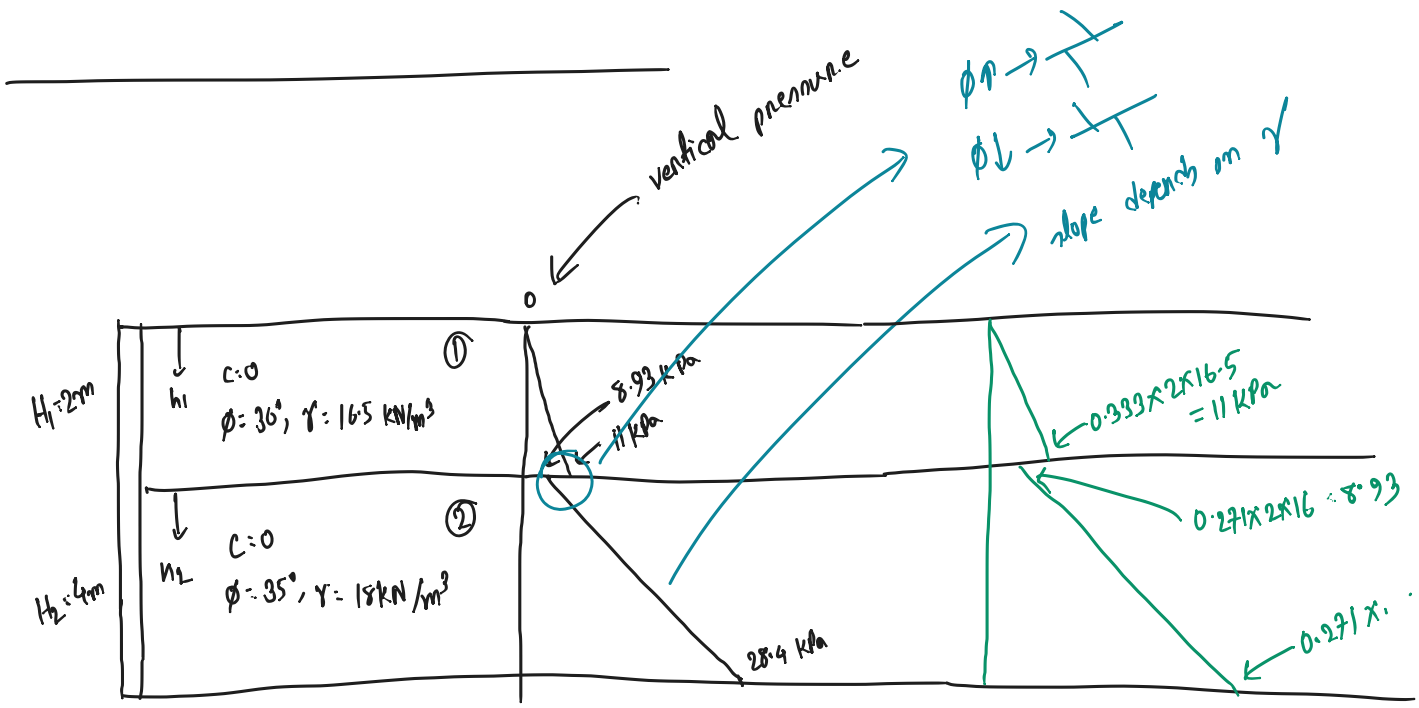
$$= 0.333 \times 18 (2+h)$$

$$= 12 + 6h$$

Step 4

$Pa(h=0) = 12$ KPa

$Pa(h=6m) = 48$ KPa



on find vertical pressure
then multiply by ka

For layer 1

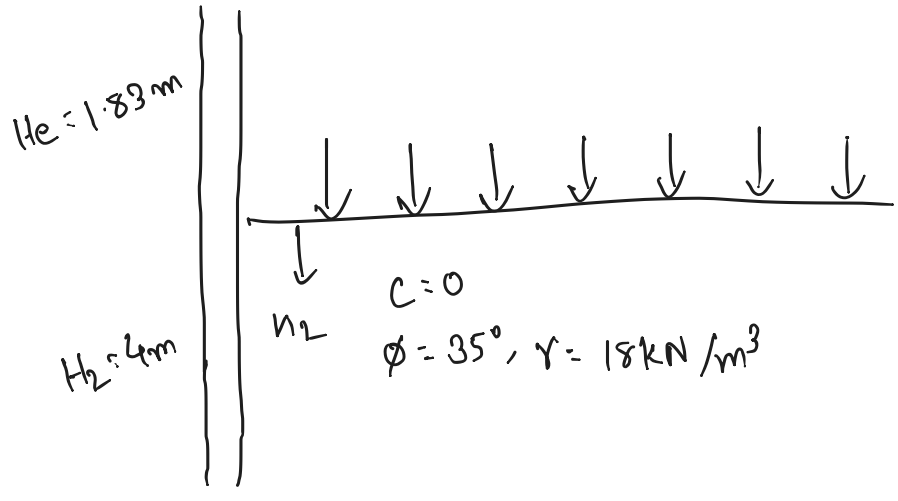
$$k_{a1} = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.333$$

$$Pa_1(h_1) = k_{a1}\gamma_1 h_1 = 0.333 \times 16.5 \times h_1$$

$$= 5.5\, h_1$$

$Pa_1(h_1=0) = 0$

$Pa_1(h_1=2m) = 5.5 \times 2 = 11$ kPa



Layer 2

Layer 1 is acting as an uniform surcharge on layer 2

$q = \gamma_1 H_1 = 16.5 \times 2 = 33$ kPa

$He = \frac{q}{\gamma_2} = \frac{33}{18} = 1.83$ m

$K_{a2} = \frac{1-\sin 35^\circ}{1+\sin 35^\circ} = 0.271$

$P_{a2}(h_2) = K_{a2}\gamma_2(He + h_2)$

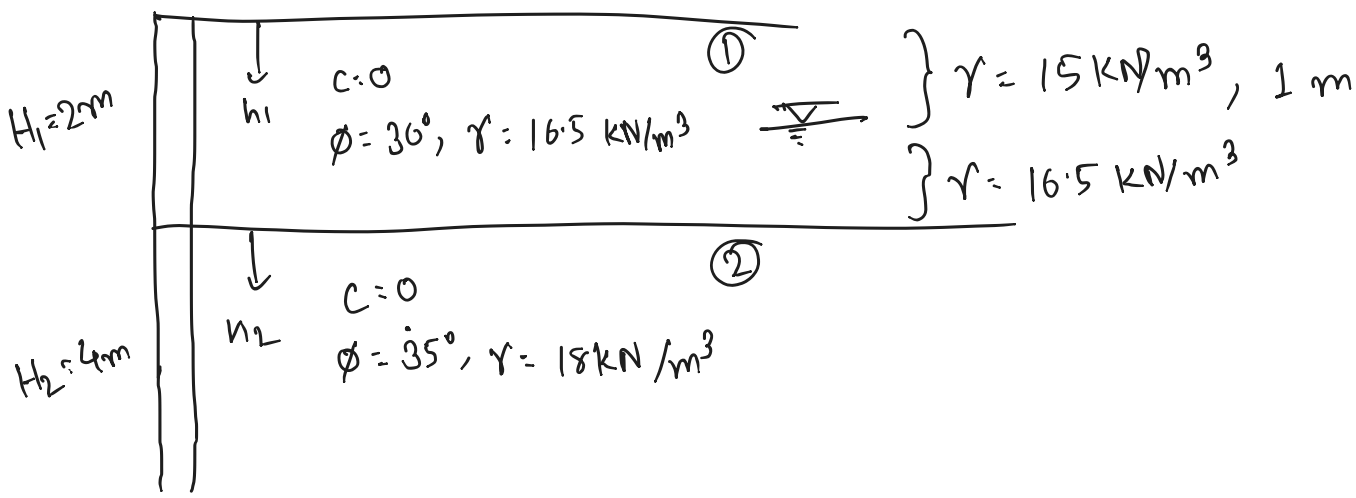
$= 0.271 \times 18(1.83 + h_2)$

$= 8.93 + 4.88 h_2$

$P_{a2}(h_2 = 0) = 8.93$ kPa

$P_{a2}(h_2 = 4m) = 8.93 + 4.88 * 4 = 28.4$ kPa

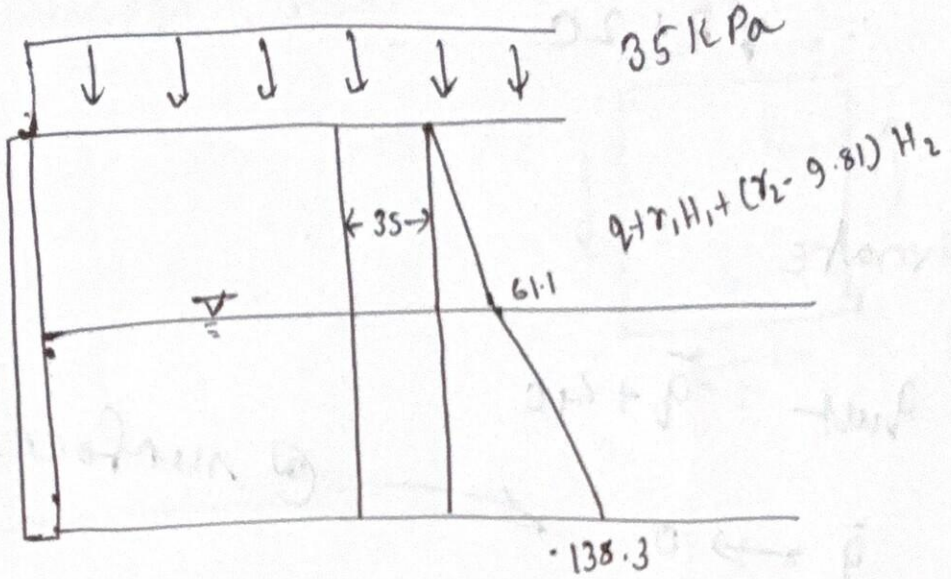
Assignment


H1=2m
h1
c=0
ϕ=30°, γ=16.5 kN/m3
①
γ = 15 KN/m3, 1 m
γ = 16.5 KN/m3
②
H2=4m
h2
c=0
ϕ=35°, γ=18kN/m3

LO7

15/12

Qn 1



$16.5 \times 3.7 = 61.1$

Effect of surchange

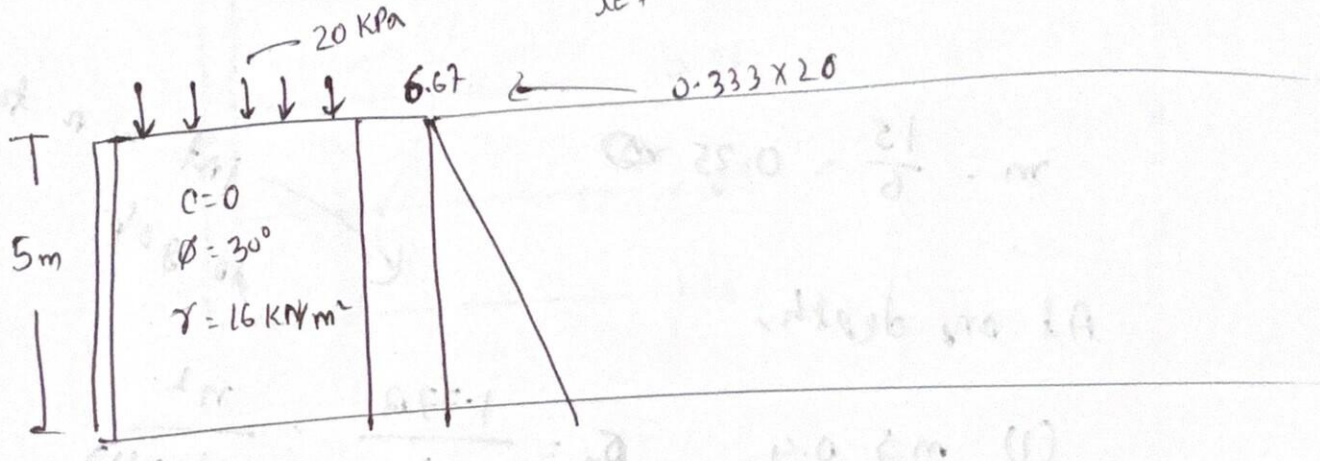
For exam → uniform ld, line ld, point ld ~~(another)~~

UD

~~Line~~ ld

Two approach:

let $K_a = 0.333$



$P_a(h) = K_a \gamma (H_e + h)$

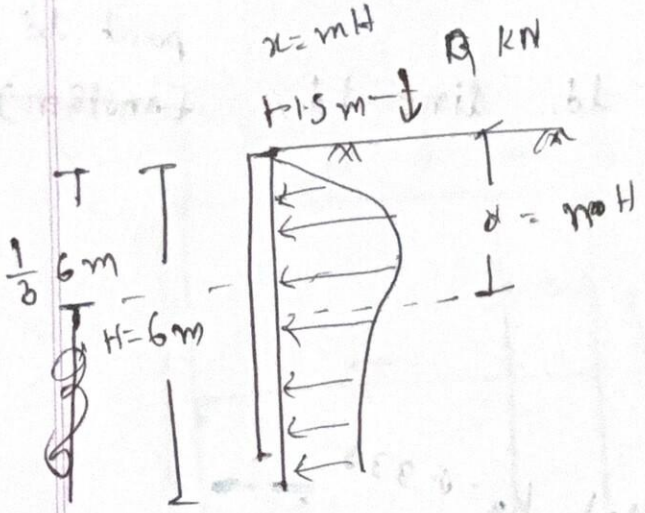
$H_e = \frac{q}{\gamma} = \frac{20}{16} = 1.25\,m$

$= 6.67 + 5.33h$

$P_a(0) = 6.67$

$P_a(h=5) = 33.33$

Point 1d :



at 2m depth

$$n = \frac{y}{H} = \frac{2}{6} = 0.333$$

$$m = \frac{1.5}{6} = 0.25$$

int. w.r.to h to get

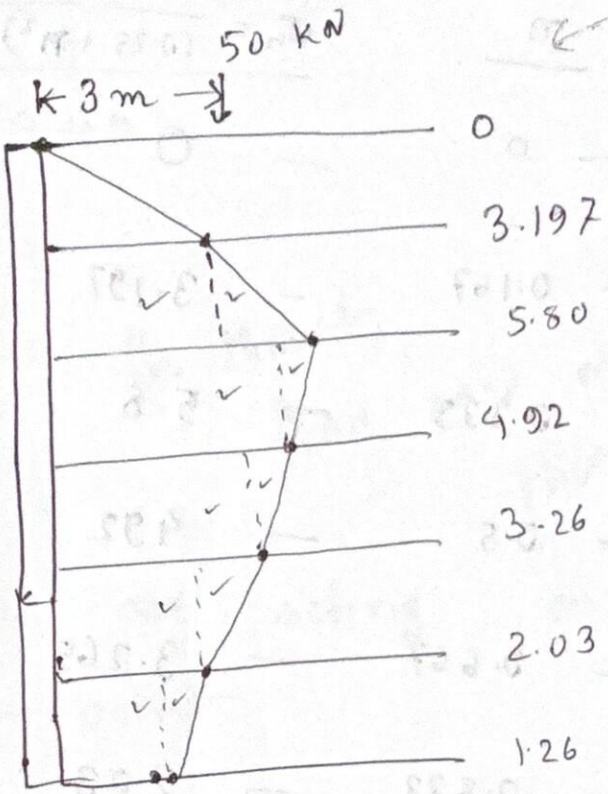
At any depth,

(i) $m > 0.4$ $\quad \sigma_h = \frac{1.77Q}{H^2} \cdot \frac{n^2}{(m^2+n^2)^3}$

(ii) $m \leq 0.4$ $\quad \sigma_h = \frac{0.28Q}{H^2} \cdot \frac{n^2}{(0.16+n^2)^3}$

Integrate using numerical method

[at least 6 section]



Step 1: $m = \frac{3}{6} = 0.5$ [> 0.4]

Step 2: $\sigma_h = \frac{1.77Q}{H^2} \cdot \frac{n^2}{(m^2+n^2)^3}$

$= \frac{1.77 \times 50}{6^2} \frac{n^2}{(0.5^2+n^2)} = 2.458 \frac{n^2}{(0.25+n^2)^3}$

3 digits ??

Step 3

Depth, h(m)	n	$\sigma_h = \frac{2.46\,n^2}{(0.25+n^2)^3}$ (kPa)
0	0	0
1	0.167	3.197
2	0.333	5.8
3	0.5	4.92
4	0.667	3.265
5	0.833	2.03
6	1	1.26

Then trapezoidal rule

~~Defn point of action~~

Detn point of action:

$A_1 = 1.59$ kN/m

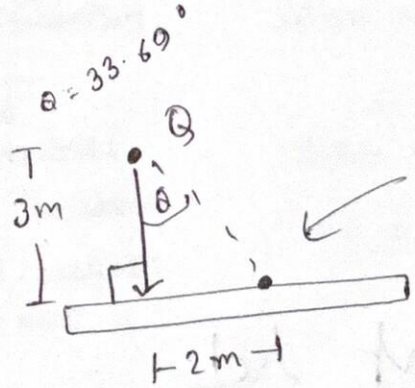
$A_2 = 4.49$

. . .

in most cases total thrust calc be asked; not point of action

for thrust, find area and then cal

Pressure are acting on perpendicular to the wall — point



find pressure distribution diagram

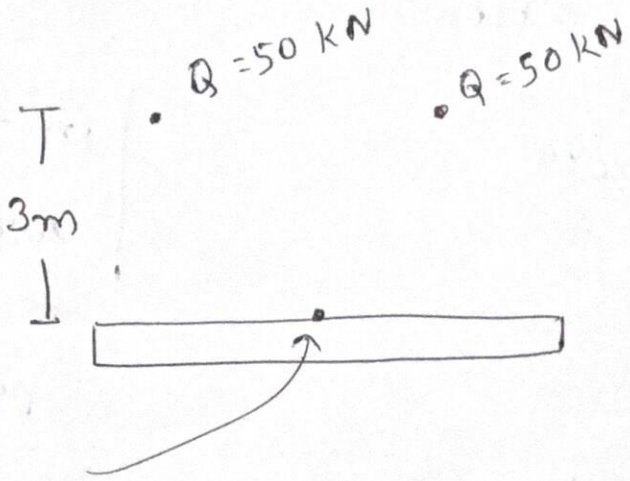
$\sigma_{h\theta} = \sigma_h \cos^2(1.1\theta)$

0.64

two approaches

find on perpendicular distance then use

Depth $n = \frac{h}{H}$ $\sigma_H =$ $\sigma_H \times 0.64$



if at center, then simply find one, the multiply by 2

Line ld

CT — Point ld

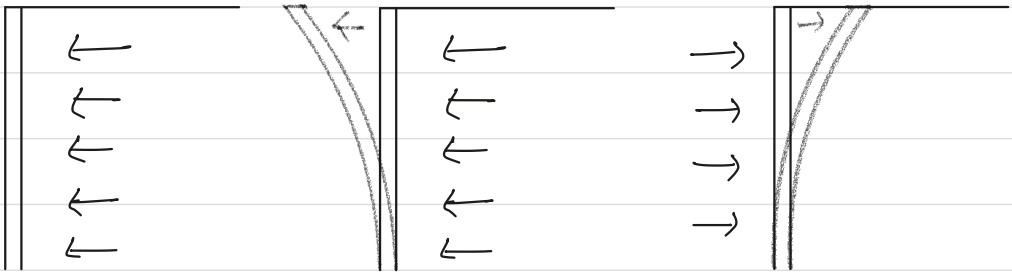
Rough / CT - 1 22/12

Lateral Earth pressure

Pressure exerted by soil against an engineering structure.

Depends on,

1. Physical chars — cohesionless soil
   cohesive soil
   C-ϕ soil ← both cohesion & angle of internal friction
2. Deformation chars — no deform
   lateral expansion
   lateral contraction



no deformation → Stiff material more resistance Earth pressure is at rest

lateral expansion → Active earth pressure → Soil is giving Id → K_a

lateral contraction → Passive earth pressure → Soil is resisting Id → K_p

K_a, K_p depends on failure direction

maximum lateral stress at which soil fails

K_a, K_p → coefficient of active/passive earth pressure

Ratio of lateral stress to vertical stress at active/passive earth pressure

$\frac{\sigma_h}{\sigma_v}$

$K_p > K_o > K_a$

why ↑ on passive soil is also resisting

For Earth pressure at rest → K_0

derived empirically (by convention)

Normally consolidated soil, $K_0 = 1 - \sin\phi'$

Over consolidated soil, $K_0 = (1 - \sin\phi')\, OCR^{\sin\phi'}$

Procedure: determine vertical pressure

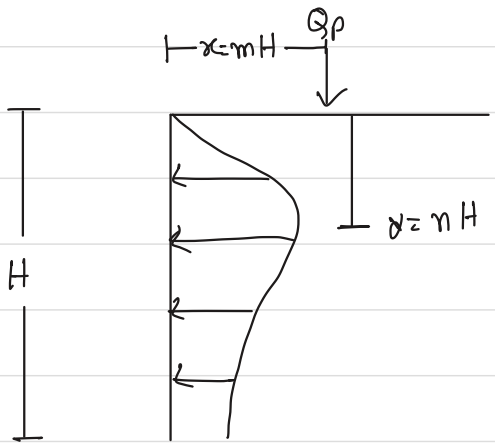
find $K_a / K_p / K_0$

determine lateral pressure

Total thrust = area

Point of action = centroid

Effect of surcharge — Point ld



$$m = \frac{x}{H}$$

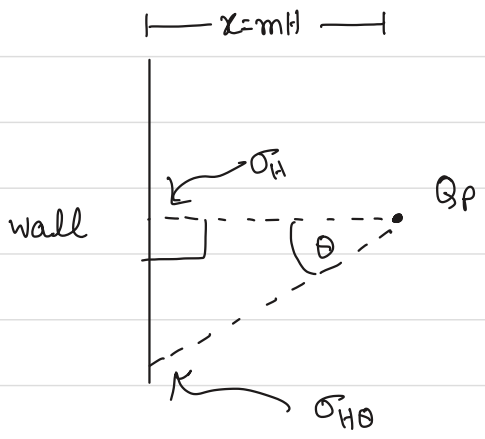
$$n = \frac{y}{H}$$

For $m > 0.4$, $\sigma_H = \frac{1.77\,Q_p}{H^2} \cdot \frac{n^2}{(m^2+n^2)^3}$

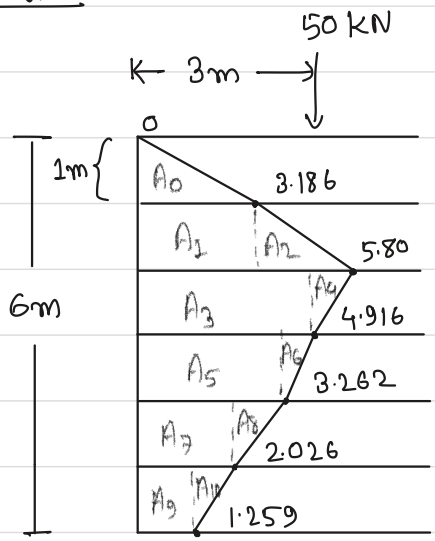
$m \leq 0.4$, $\sigma_H = \frac{0.28\,Q_p}{H^2} \cdot \frac{n^2}{(0.16+n^2)^3}$

} ld at a point perpendicular of loading point & wall

$\sigma_{H\theta} = \sigma_H \cos^2(1.1\theta)$ ⟶ when not perpendicular



Practice



$$m = \frac{x}{H} = \frac{3}{6} = 0.5 \quad (>0.4)$$

$$n = \frac{y}{H} = \frac{y}{6}$$

$$\sigma_{1t} = \frac{1.77 Q_l}{H^2} \cdot \frac{n^2}{(n^2+m^2)^3}$$

$$= 2.458 \frac{n^2}{(n^2+0.25)^3}$$

y (m)	n	σ_H
0	0	0
1	0.167	3.186
2	0.333	5.80
3	0.5	4.916
4	0.667	3.262
5	0.833	2.026
6	1	1.259

Area → force

$A_0 = 0.5 \times 3.186 \times 1 = 1.593$ kN

$A_1 = 3.186 \times 1 = 3.186$ kN

$A_2 = 0.5 (5.8 - 3.186) \times 1 = 1.307$ kN

$A_3 = 4.916 \times 1 = 4.916$ kN

$A_4 = 0.5 (5.8 - 4.916) \times 1 = 0.442$ kN

$A_5 = 3.262 \times 1 = 3.262$ kN

$A_6 = 0.5 \times (4.916 - 3.262) \times 1 = 0.827$ kN

$A_7 = 2.026 \times 1 = 2.026$ kN

$A_8 = 0.5 (3.262 - 2.026) \times 1 = 0.618$ kN

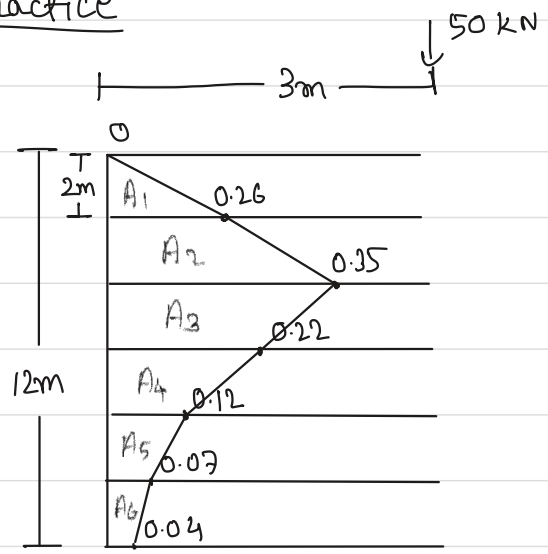
$A_9 = 1.259 \times 1 = 1.259$ kN

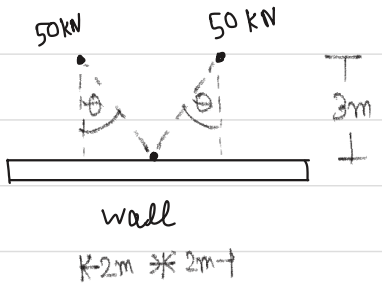
$A_{10} = 0.5 (2.026 - 1.259) \times 1 = 0.384$ kN

use trapezium instead

$\therefore$ Total thrust = $\Sigma A = 19.82$ kN

Practice





~~K-2m * 2m-1~~

$$\theta = \tan^{-1}\frac{2}{3} = 33.69^\circ$$

$$m = \frac{x}{H} = \frac{3}{12} = 0.25 \quad [\leq 0.4]$$

$$n = \frac{y}{H} = \frac{y}{12}$$

$$\sigma_H = \frac{0.28Q}{H^2}\,\frac{n^2}{(0.16+n^2)^3}$$

$$= 0.0972 \times \frac{n^2}{(0.16+n^2)^3}$$

$$\sigma_{H\theta} = \sigma_H \cos^2(1.1 \times 33.69)$$

$$= 0.637\,\sigma_H$$

Depth, y (m)	n	σ_H	$\sigma_{H\theta}$
0	0	0	0
2	0.17	0.41	0.26
4	0.33	0.54	0.35
6	05	0.35	0.22
8	0.67	0.20	0.12
10	0.83	0.11	0.07
12	1	0.06	0.04

$A_1 = 0.5 \times 0.26 \times 2 \quad = 0.26 \text{ KN}$

$A_2 = \frac{0.26+0.35}{2} \times 2 = 0.61 \text{ KN}$

$A_3 = \frac{0.35+0.22}{2} \times 2 = 0.57 \text{ KN}$

$A_4 = 0.12 + 0.22 = 0.34 \text{ KN}$

$A_5 = 0.12 + 0.07 = 0.19 \text{ KN}$

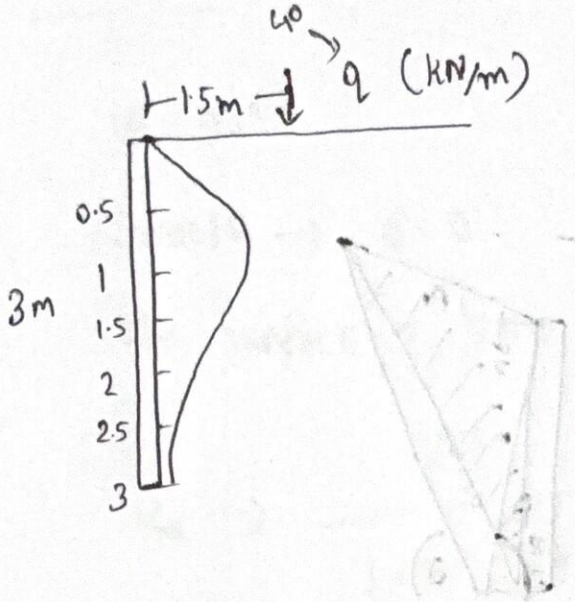
$A_6 = 0.07 + 0.04 = 0.11 \text{ KN}$

$\therefore$ Total thrust $= \Sigma A = 2.08$ KN [for single point ld]

$\therefore$ Total thrust for both ld

$= 2.08 \times 2 = 4.16$ KN

Line ld (Eff of surcharge)



max at H/3

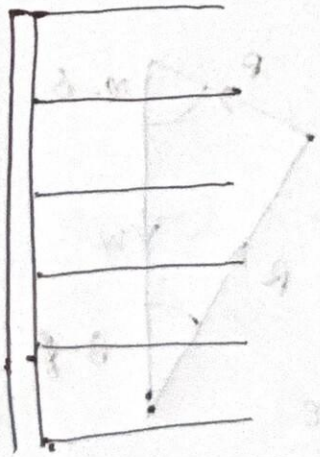
therefore, min 6 sections

$m > 0.4,\ \sigma_H = \frac{4q}{\pi H} \frac{m^2 n}{(m^2+n^2)^2}$

$\leq 0.4,\ \sigma_H = \frac{q}{H} \frac{0.203 n}{(0.16+n^2)^2}$

1. $m = \frac{1.5}{3} = 0.5 \quad (m > 0.4)$

$\sigma_h = \frac{4 \times 40}{\pi \times 3} \times \frac{0.5^2 \times n}{(0.25+n^2)^2} = 4.24 \frac{n}{(0.25+n^2)^2}$



Depth	n	σ_n
0		
0.5		
1		
1.5		
2		
2.5		
3		

$A_i \rightarrow \qquad \Sigma A_i = --$

Columbs Theory Lateral Earth Pressure

Assumptions

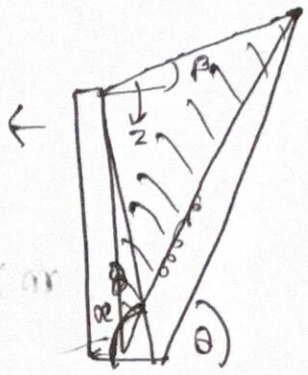
(1)

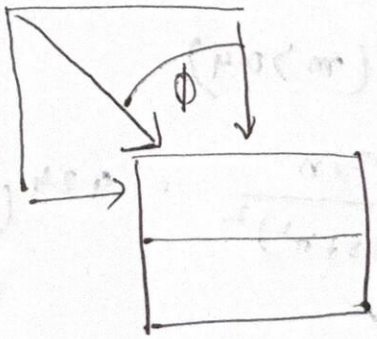
(2)

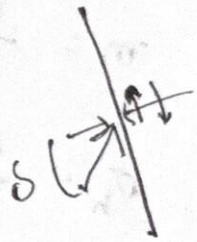
(3)

(4)

(5)



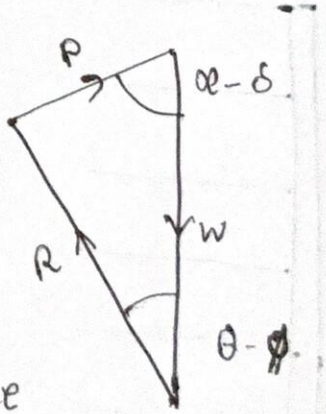




active pressure

$\delta = 2/3$ or $3/4$ of ϕ





only unknown → θ

↳ cause failure plane assumed

Compared to Rankine's

$\alpha = 90°$

Smooth → $\delta = 0$

Hor. surface → $\beta = 0$

[Justify the Rankine's formula, - starting w/ columb's theory]

$$K_a \rightarrow \frac{\cos^2\phi}{1 \times 1 \times \left[1 + \sqrt{\frac{\sin^2\phi}{1}}\right]^2}$$

[connection $1 + \sqrt{\ldots}$]

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

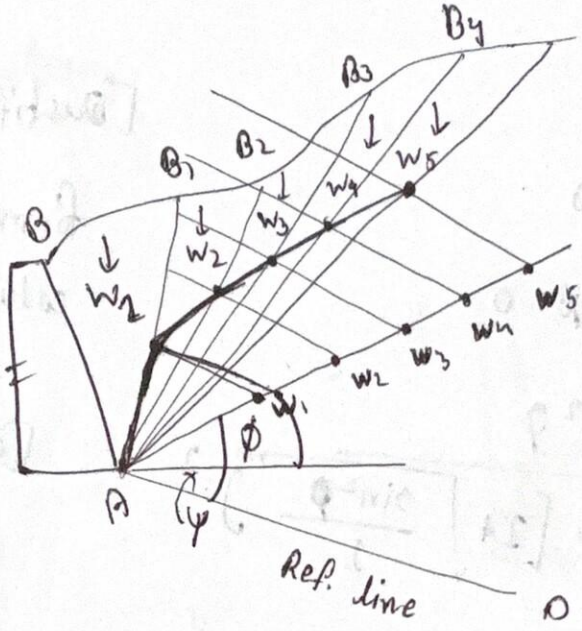
$$\Rightarrow \frac{1 - \sin^2\phi}{(1 + \sin\phi)^2}$$

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

$$K_p \rightarrow \frac{\ldots}{\ldots \left[1 - \sqrt{\ldots}\right]}$$

Culmann's method

Hor. ver. scale same



W → 1m × 1m → $1m^2$ → $1m^3$

W_1 → 87 box → $87\ m^3$ → multiply w/ unit wt

Assume 10 KN → 1 inch

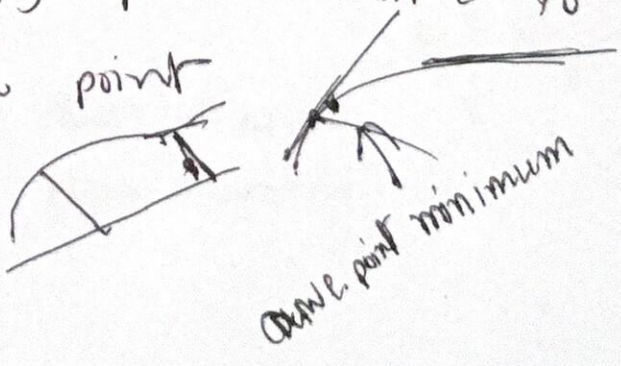
90 KN → 9 inch ; 47 KN → 4.7 inch

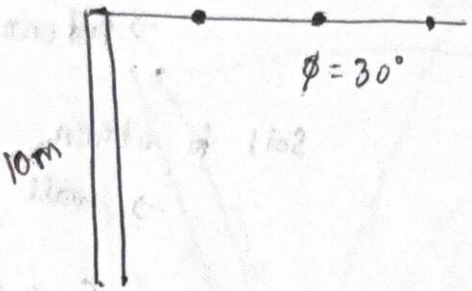
place W at that dist

Then Ref line's parallel line to W_1

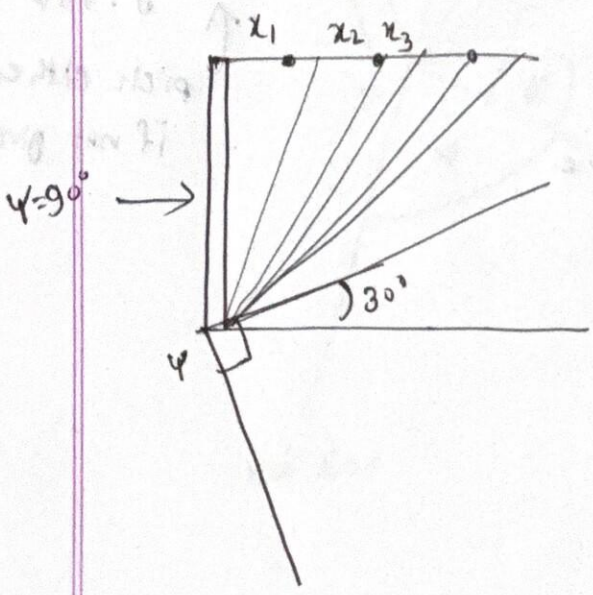
Identify curvature point

Then connect to





min 5



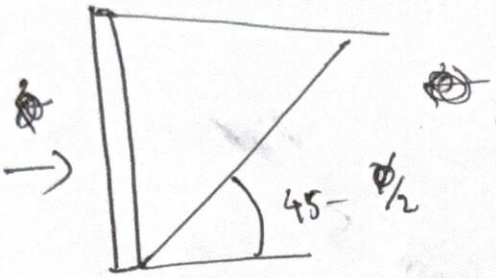
$A_1 = 10 \times x_1$

$A_2 = 10 \times x_2$

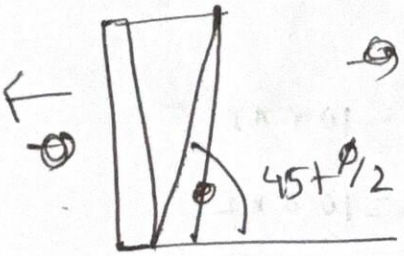
...

Graph paper next class

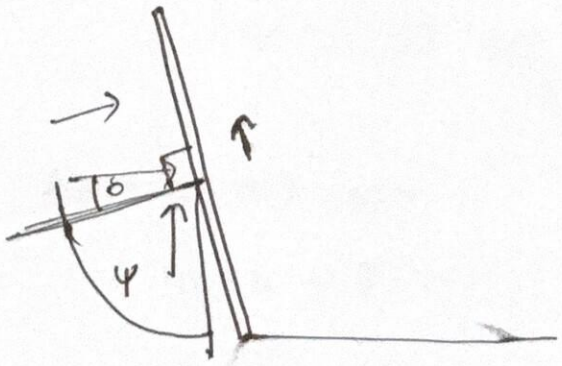
Passive



active



Passive

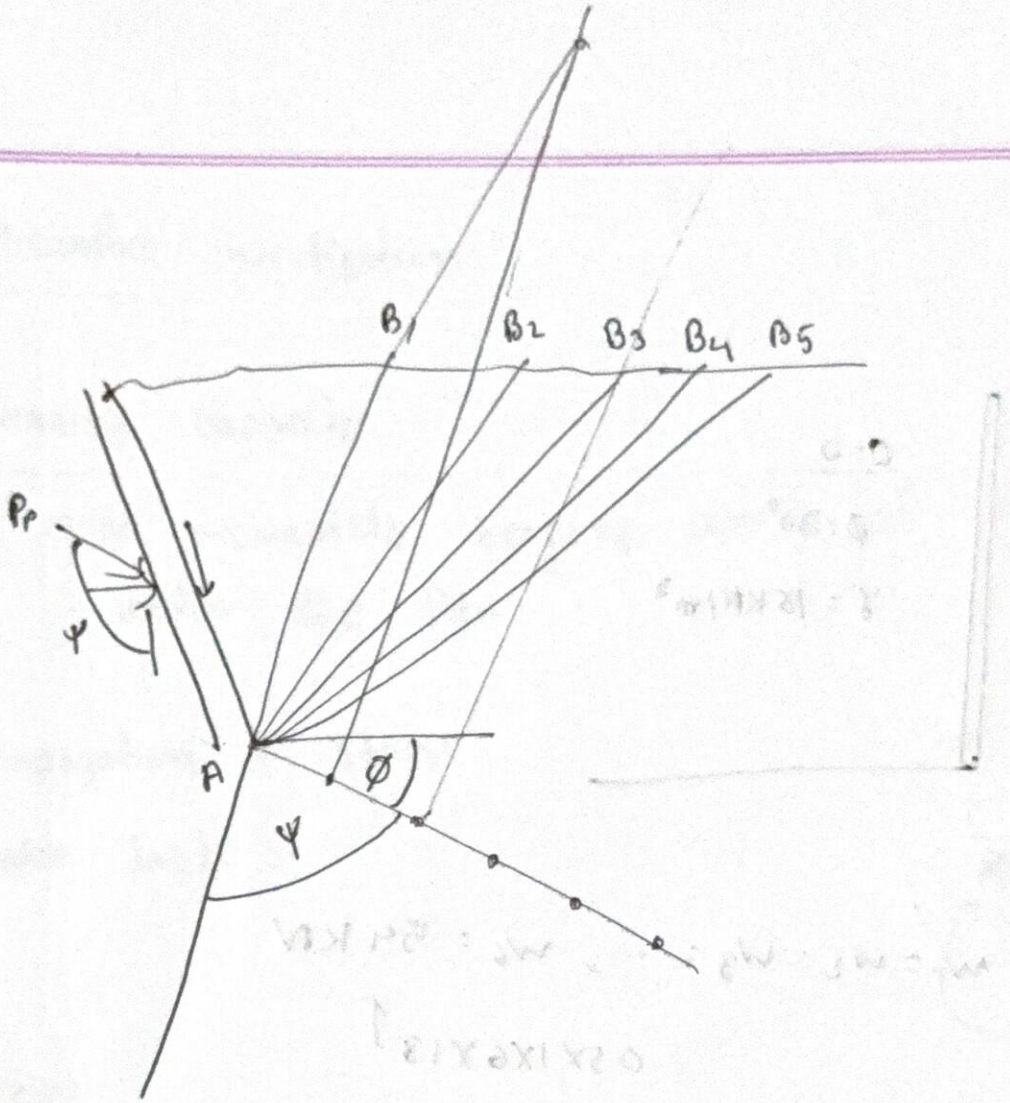


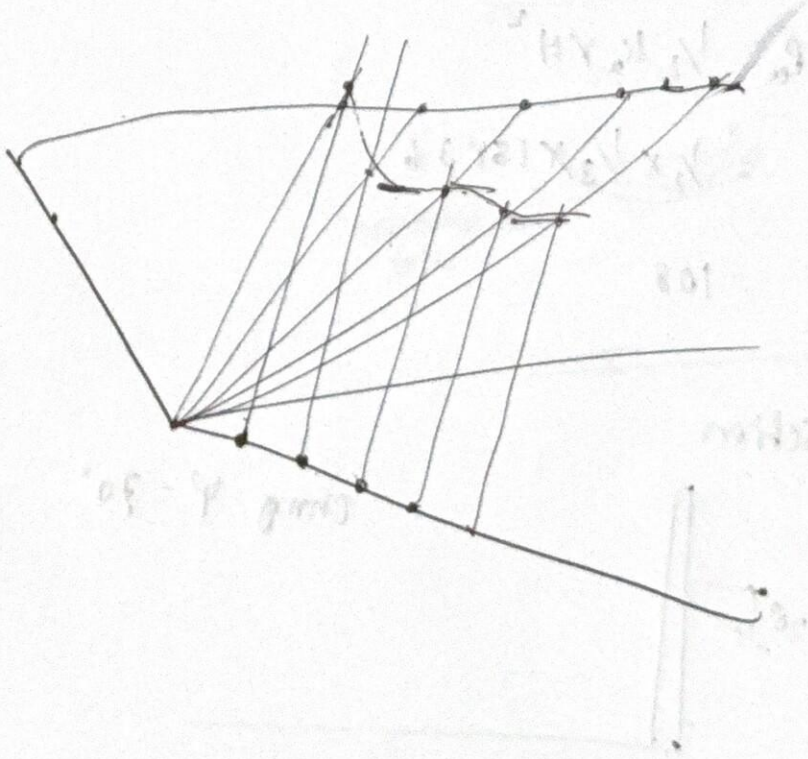
Soil to soil failure
→ internal fric

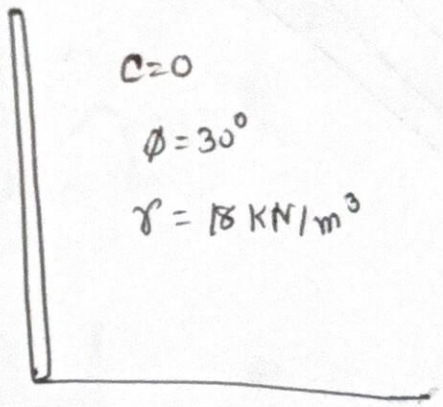
Soil to steel
→ wall friction

$\delta = 0.67\phi$
0.75ϕ

pick either
if not given


B1
B2
B3
B4
B5
Pr
ψ
A
ϕ
ψ
Ref line





$W_1 = W_2 = W_3 = \cdots, W_6 = 54\ KN$

$0.5 \times 1 \times 6 \times 18$ ↑

Rankin's

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

$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 36$

$= 108$

if friction



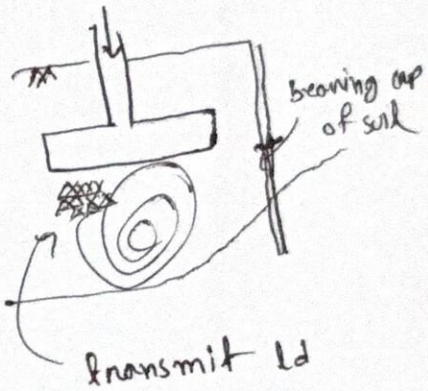
Subsurface investigation

Bearing capacity

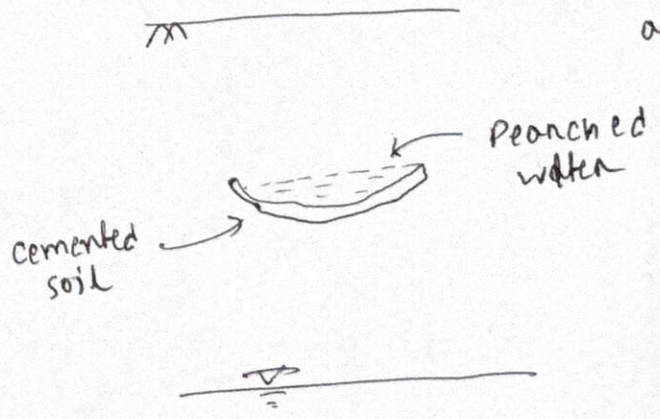
more accurately, bearing capacity of subsurface soil under the fdn

Propagation of stress

water table



Plan



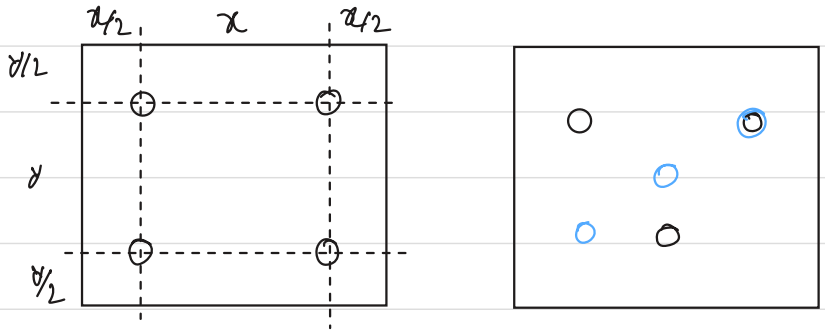


L12 21/01

Subsoil investigation

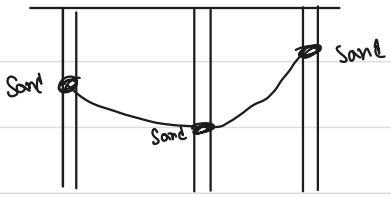
Cost of exploration

Number & location of exploration boring



Spacing of boring

1/2 boring under bridge piers

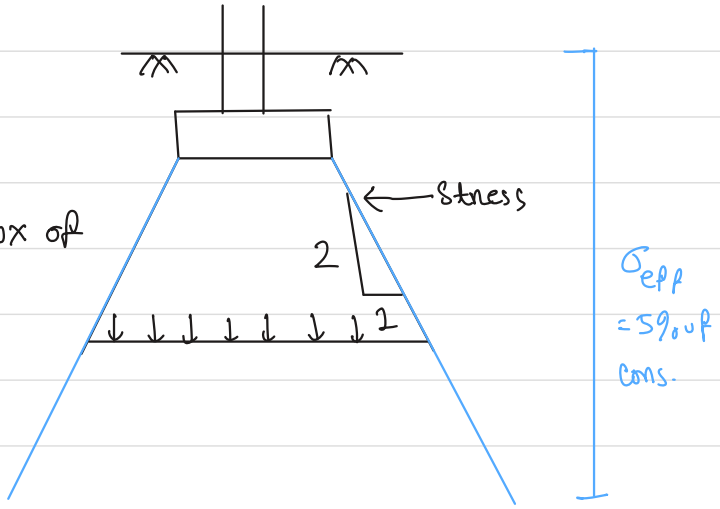


Depth of boring

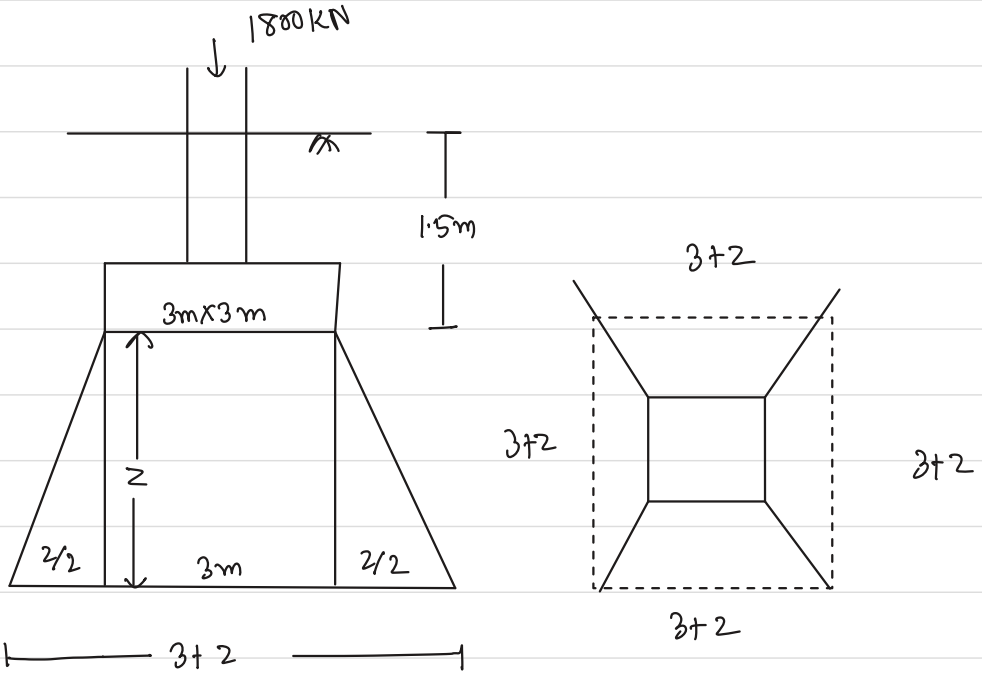
Until 10% of org stress by proposed construction

Until stiffer layer → SPT

10% of sigma where area is 10x of footing



Example



Gross stress increase

$$= \frac{1800}{3 \times 3} = 200 \text{ kPa}$$

Net stress increase

$$= 200 - \gamma D_f$$

$$= 200 - 16 \times 1.5 = 176 \text{ kPa}$$

10% of net stress increase

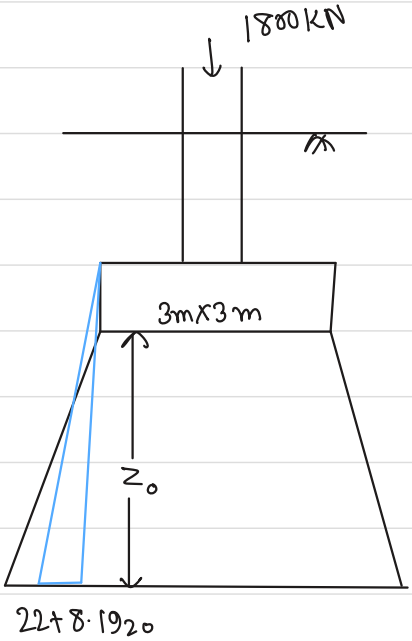
$$= 0.1 \times 176 = 17.6 \text{ kPa}$$

$$\frac{176 \times 3 \times 3}{(3+z)(3+z)} = 17.6$$

$$\Rightarrow z = 6.5 \text{ m}$$

∴ Total depth of boring

$$1.5 + 6.5 = 8 \text{m}$$



$1.5 \times 16 + 1.0(16 - 9.81) + (z_0 - 1)(18 - 9.81)$

$= 30.1 + 8.19 z_0 - 8.19$

$= 22 + 8.2 z_0$

$$\frac{176 \times 3 \times 3}{(3 + z_0)(3 + z_0)} = 0.05(22 + 8.19 z_0)$$

$z_0 = 12.8\ m$

$\rightarrow 12.8 + 1.5 = 14.3$

Use min of two methods

888 Narrow building $h_B <$ ...

For narrow conc $z_b = 3s^{0.7}$

wide $z_b = 6s^{0.7}$

s -> num of equiv stories

50 KN/m²

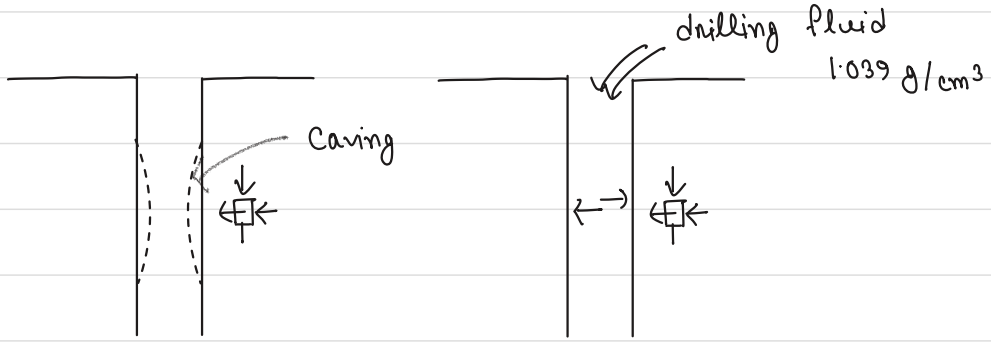
$s = \frac{50}{10} = 5$

narrow, $z_b = 3 \times 5^{0.7} =$ ...

Next → method of boring

L13 27/01

Method of Boring



Suitability ---

Augar Boring looks like screw
in-built casing

Wash Boring

** describe briefly the method of wash boring

Percussion or Cable Tool Drilling
only for stiffen soil

Rotary Drilling

ODEX Drilling

Soil Sample

i) Representative → no chng (grain size analysis, atterberg, specific grav)

ii) Non-representative → add/remove of constituents (field identif, ...)

i) Undisturbed

ii) Disturbed → not possible to the point major – absence of triaxial pressure upon hole

Justify the statement all undisturbed → representative

not all rep... → undisturbed

water content, arrangement may chng

Soil sampling



Block sampling

...

Soil Samplers

Exploration sampler

Drive sampler → (i) Open drive (ii) Piston drive sampler (similar to syring)

cone barrel

3 types ...

Depending on chars/ thickness (3 types)

- Thick wall (split spoon sampler, SS)
- Thin wall (Shelby tube, ST)
- Composite

Sample disturbance

- cutting edge
- Inside wall ...
- ...

Soil disturbance

Undisturbed soil sample → exactly field condition
except stress

Disturbance depends on

1. Cutting edge
2. Inside wall friction
3. No-return valve

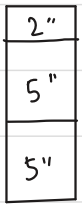
Cutting edge → C_i btwn 1% 3%
C_o !> C_i
A_n ↓↓

Recovery ratio $R_n = \frac{L}{H}$



Rock Quality

$RQD = \frac{10}{12} \times 100\% = 83\%$



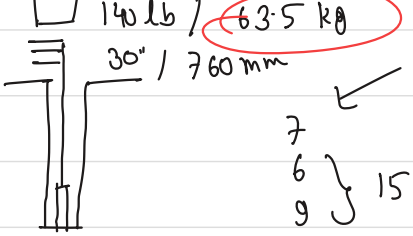
> 4"

Ground water level

Standard Penetration Test (SPT) ← Common Dynimic Penetration Test

Connected

~ 30 blows/min



for 6" penetration

7
6
9 } 15 → SPT value
→ N-value

should be continued until → 3 criteria

Should be continued until : 3 criteria

Connections of SPT value

Standard Penetration Test

140 lb / 63.5 kg

30" / 760 mm

18" { 6" / 150 mm → 7 blws → ignore
6" / 150 mm → 5 blws
6" / 150 mm → 8 blws } + → 13 → SPT value
N - value
SPT - N
N_f

Connection for field procedure

1. For Hammer energy → 60% ideal
2. For Borehole diameter → 100 mm → 1
3. For Drill rod connection
4. For sampler connection

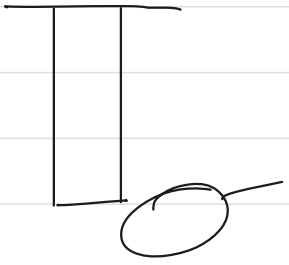
always while ←
→ 45% eff

$$N = 15$$

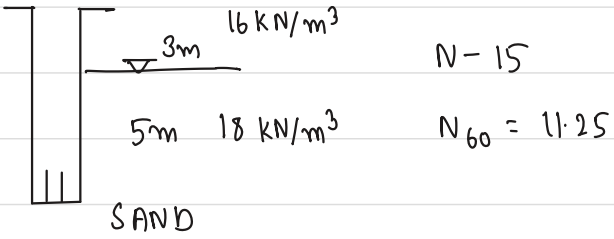
$$N_{60} = \frac{0.45 \times 1 \times 1 \times 1}{0.6} \times 15$$

$$= 11.25$$

← could be fraction



if sandy soil (below G.W.T.)
further connection req



Connection for overburden pressure

$$\sigma_v' = 3 \times 16 + 5\,(18 - 9.81)$$
$$= 88.95 \text{ kPa}$$

$$C_N = 0.77 \log\left(\frac{2000}{\sigma_v'}\right) ; \quad \sigma_o' \geq 0.25 \text{ kN/m}^2$$
$$= 0.77 \log\left(\frac{2000}{88.95}\right) = 1.04$$

$C_N = \sqrt{\frac{1}{\sigma_v'}}$; σ_v' in kg/cm²

$$\left.\begin{matrix} N_{1(60)} \\ (N1)_{60} \end{matrix}\right\} = C_N * N_{60}$$
$$= 1.04 \times 11.25$$
$$= 11.7$$

Efficiency from table

Other connections (C_B, C_S, C_R) from table

$C_R \rightarrow$ depth of SPT sample

Connection for water table / - - -

No longer used

we use overburden connection

SPT & soil prups correlation

Cohesionless soil

> 50 → very dense → tanget

interpolate angle of internal friction

Cohesive soil

Unconfined compressive stren, $q_u (kN/m^2) = 12.5\ N_{60}$

$$C_u = \frac{q_u}{2}$$

$$\tau = C_u + \sigma \tan\phi \rightarrow 0$$

For Bangladesh soil,

$q_u = 14.3\ N$	$LL \leq 35$
$16.9\ N$	$36 \leq LL \leq 50$
$17.8\ N$	$LL > 50$

$$\phi^\circ = 27 + 0.3 N_{60} + 0.00054 N_{60}^2$$

$$\phi^\circ = \sqrt{N_{60}} + 15$$

Rough / Midterm 01/03

SPT — Standard Penetration Test

- Driving a split spoon sampler into soil
- Borehole dia 55 mm - 100 mm
- Hammer weigh 63.5 kg dropped onto a drill rod from 30 in or 750 mm above.

N-value → Number of blows to penetrate 12" (avoid first 6")

Boring to be continued until refusal occurs.
Refusal :

1. Reach 50 blows during any of the three 6" penetration
2. Reach total of 100 blows
3. No significant advancement during 10 successive blows.

Connections :

(A) Standardization

1. For hammer energy → E_H
2. Borehole dia → C_B
3. Drilling rod → C_R
4. Sampler → C_S

$$N_{60} = \frac{E_H C_B C_R C_S N}{0.60}$$

E_H → 0.5 for BD, Donut hammer

C_B → 0.65 – 100 mm → 1
150 mm → 1.05
200 mm → 1.15

C_S → w/ liner → 1.0
w/o liner → 1.2

C_R → len 3 – 4 m → 0.75
4 – 6 m → 0.85
6 – 10 m → 0.95
> 10 m → 1.0

(B) No longer req

(C) Overburden connection factor (C_N)

$$C_N = 0.77 \log\left(\frac{2000}{\sigma_0'}\right) \leq 2.0; \quad \sigma_0' > 0.25 \text{ KN/m}^2$$

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

SPT & soil property :

Cohesionless soil →	N:	0–4	4–10	10–30	30–50	>50
		V. Loose	Loose	Medium	Dense	V. Dense

Cohesive soil →	N:	0–2	2–4	4–8	8–15	15–30	>30
		V. soft	Soft	Medium	Stiff	V. Stiff	Hard

SPT value & consistency of cohesive soil

$$q_u = 12.5 N_{60} \quad (kN/m^2)$$
$$c_u = \frac{q_u}{2}$$
$$\tau = c_u$$

For Bangladesh

$$q_u = 14.3 N \quad LL \leq 35$$
$$= 16.9 N \quad LL \leq 50$$
$$= 17.8 N \quad LL \geq 50$$

Defⁿ angle of internal friction.

$$\phi^o = 27 + 0.3 N_{60} + 0.00054 N_{60}^2$$
$$= \sqrt{N_{60}} + 15$$

Dynamic cone penetration test (DCPT)

100 mm → 100mm → 100 mm → 100 mm

300 mm → # of blows

65 kg → high → high dyna...

low → low "

→ Super "

Dutch cone penetration test (CPT)

Cone resistance

Skin / Side friction

q_c

f_c

$R_f = \frac{f_c}{q_c}$ → ↑ → fine

→ ↓ → sand

Soil profile based on depth vs R_f

Geophysical Method (not impo for exam)

[Skip to page — Soil Report (scope of ...)
[Then p219]

Soil Exploration Report

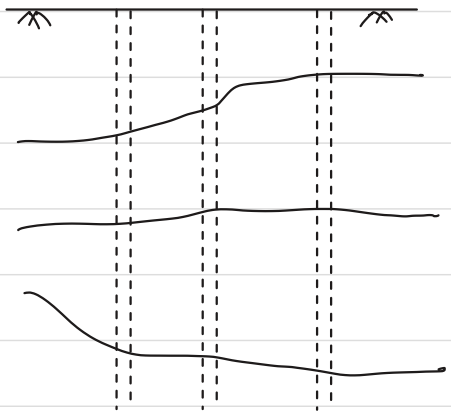
Drainage → via gravity

Dewatering → !(n); pump

Borehole log

Scopes, trace — -- %

--- — --- %

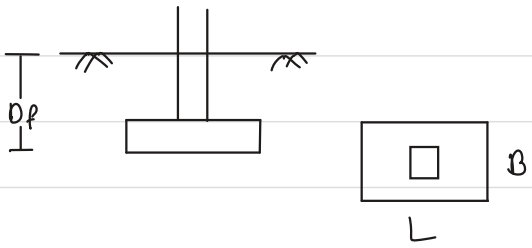


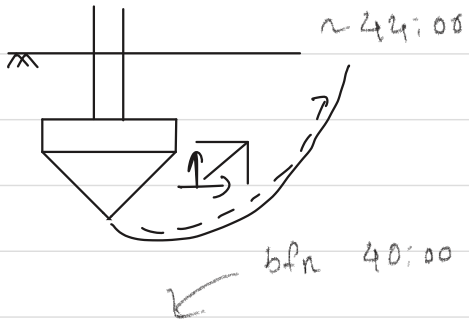
Types of foundation

Shallow fdn (common - footing)

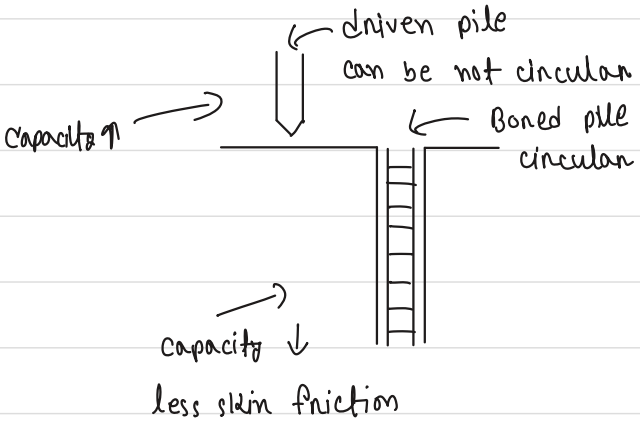
Df < 3m

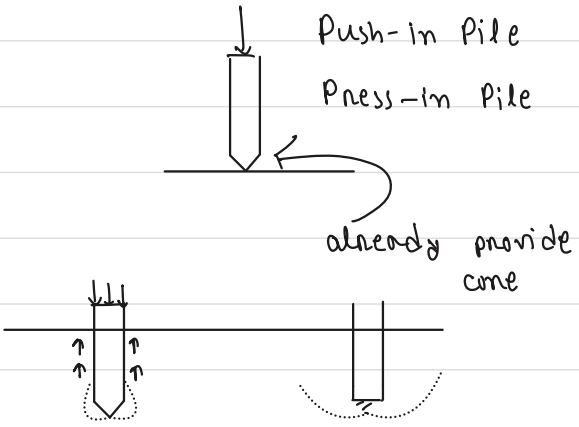
depth less than breadth (B)





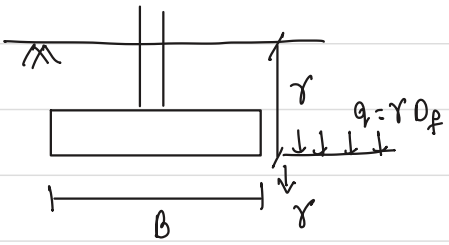
Deep fdn

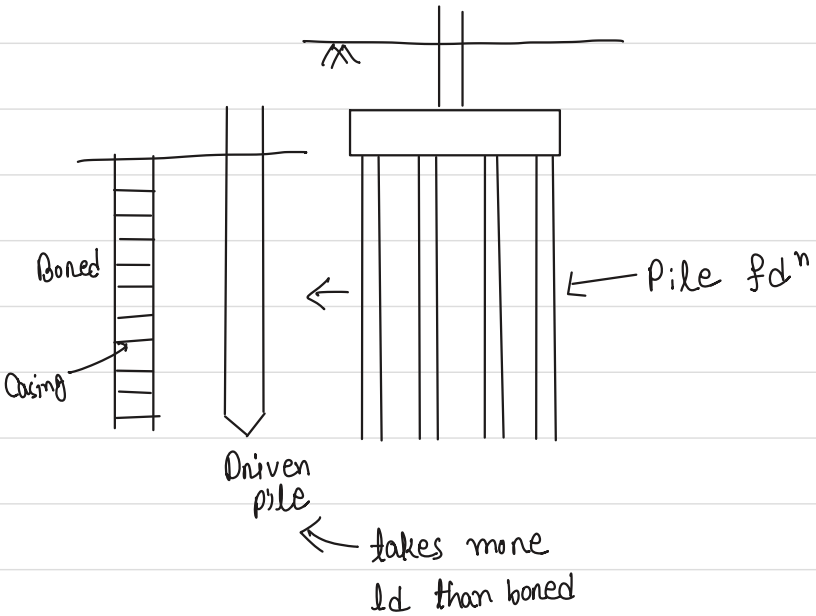




Bearing Capacity of Pile Fdn

$$q_{ult} = cN_c + \frac{1}{2} B\gamma_1 N_\gamma + \gamma_2 D_f N_q$$





→ Static analysis method ← ΣF_x ΣF_y ΣM

→ Field methods

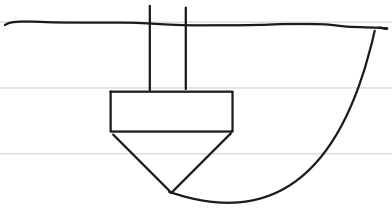
→ Static ld test

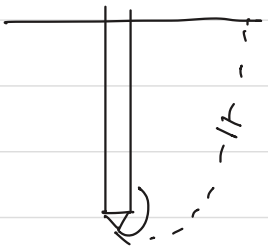
→ Dynamic ld test ← will not be studied

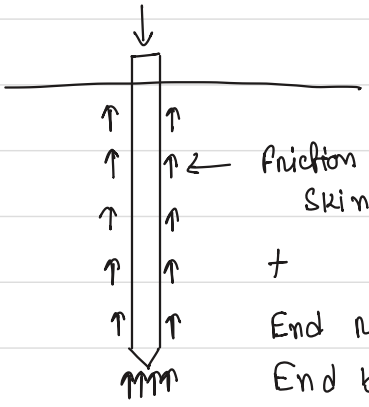
→ Driving formulae ← taks 10 blw on one soil
20 " " another "

Static analysis method

N_c, N_q, N_γ → General bearing capacity factor







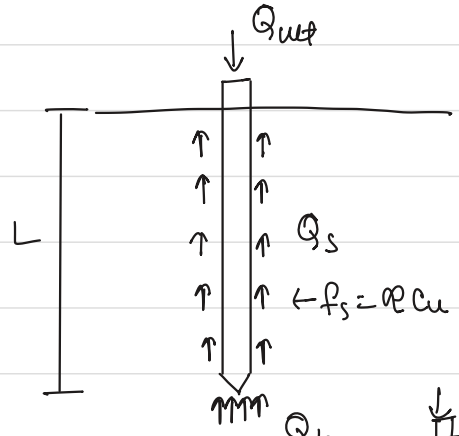
80%

if most dd is taken by skin friction → friction pile

if by end → bearing pile

→ pile in cohesive soil, pile in cohesionless soil

Driven pile in cohesive soil



$Q_{ult} = Q_s + Q_b$

$Q_s = A_s f_s$ $\quad A_s = \pi D L$

$q_{ult} = cN_c + \frac{1}{2}\gamma B N_\gamma + qN_q$ ($\gamma = 0$, $q = 0$)

failure btwn soil particles → C

failure btwn soil & concrete → varies → c

$Q_b = f_b A_b$ $\quad f_b = q_{ult} = cN_c$

↓ 9 (for deep foⁿ)

$f_s = c_u$ ← Shear for cohesive → c, undrained → c_u

C_u = 20 kPa ← disturbance ↓

100 kPa ← " ↑

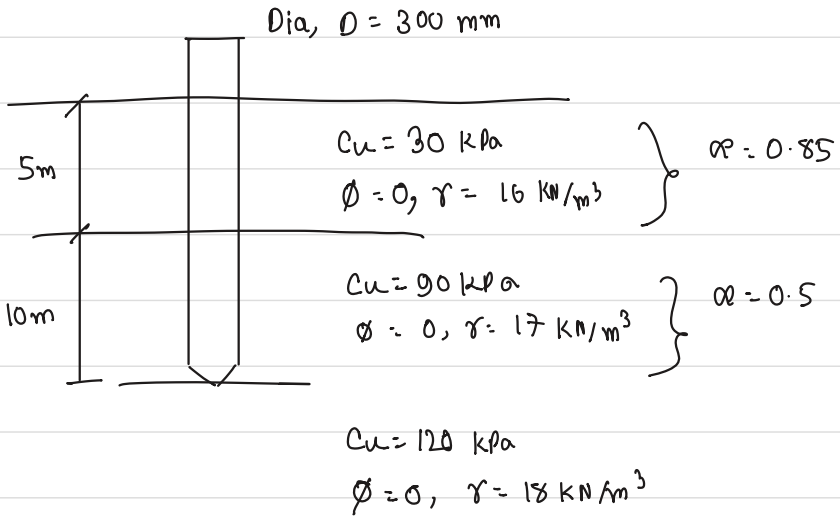
α & C_u

C_u ↑ → α ↓ ← disturbance higher, α ↓

C_u ↓ → α ↑

C_u = 20 kPa → α = 0.90

C_u = 100 kPa → α = 0.45 } interpolate in btwn



$$Q_s = A_s f_s = A_{s1} f_{s1} + A_{s2} f_{s2}$$

$$= (\pi \times 0.3 \times 5)(0.85 \times 30) + (\pi \times 0.3 \times 10)(0.5 \times 90)$$

$$= 120.1 + 424.1 = 544.2$$

$$Q_b = A_b f_b = \left(\frac{\pi\, 0.3^2}{4}\right)(9 \times 120) = 76.3 \text{ kN}$$

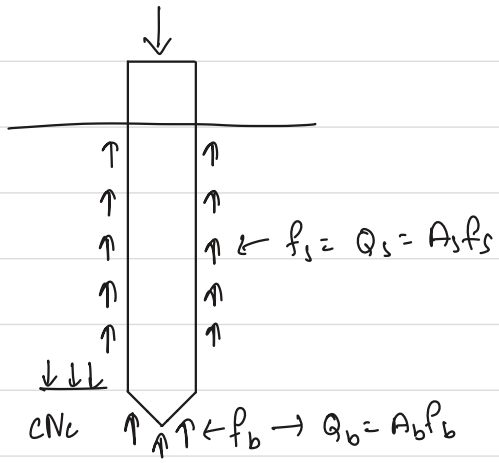
$$Q_{ult} = 544.2 + 76.3 = 620.4 \text{ kN}$$

In general

Cohesive : $c=c$, $\phi=0$: Clay ; NC clay, plastic silt

Cohesionless : $c=0$, $\phi=\phi$: Sand-gravel; NC clay, non-plastic silt

$c-\phi$: $c-\phi$: Both



Alpha method

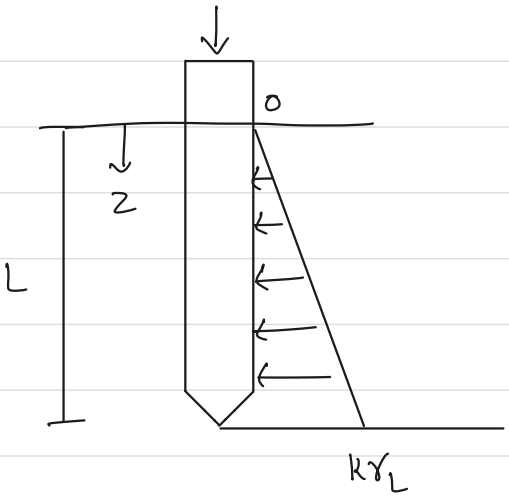
$f_s = \alpha c_u$

$f_b = q N_q$ ← q

$= 9q$

Cohesionless soil:

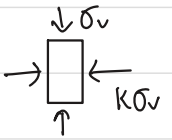
ϕ → when failure btwn soil



$f_s = \sigma \tan\phi = K\gamma_z \tan\phi$

$= K\sigma_v \tan\phi$

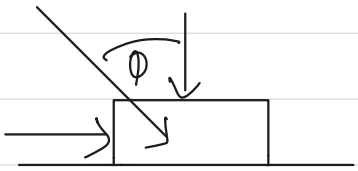
$= \beta \sigma_v$ usually given

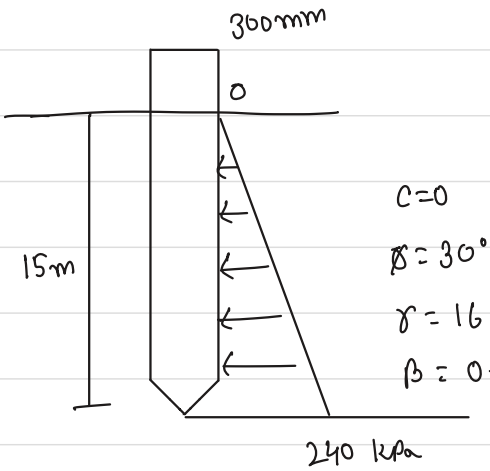


Angle of wall friction = δ

can be measured

if not, $\delta = \frac{2}{3}$ to $\frac{3}{4}\phi$





$C = 0$

$\phi = 30°$

$\gamma = 16\ kN/m^3$

$\beta = 0.9$

Beta method

$f_s = \beta \sigma_{v,avg}$

$= 0.9 \times \frac{240}{2} = 108\ kPa$

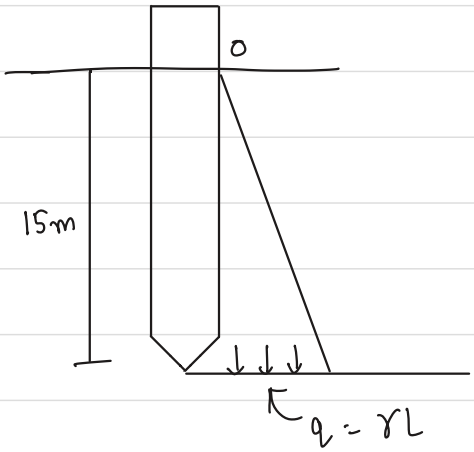
$Q_s = A_s f_s = (\pi \times 0.3 \times 15) \times 108$

$= 1526.8\ kN$

$q_a = cN_c + \frac{1}{2}\gamma B N_\gamma + q N_q$ (0 over c, 0 over γ)

$= q N_q$

[28:30]



$N_q = 15$ [graph, L/d]

$f_b = q N_q$

$= 240 \times 15 = 3600\ kPa$

$Q_b = A_b f_b = \frac{\pi d^2}{4} \times 3600$

$= \frac{\pi \times 0.3^2}{4} \times 3600$

$= 254.5\ kN$

$Q_{ult} = Q_s + Q_b$

$= 1526.8 + 254.8 = 1781.3\ kN$

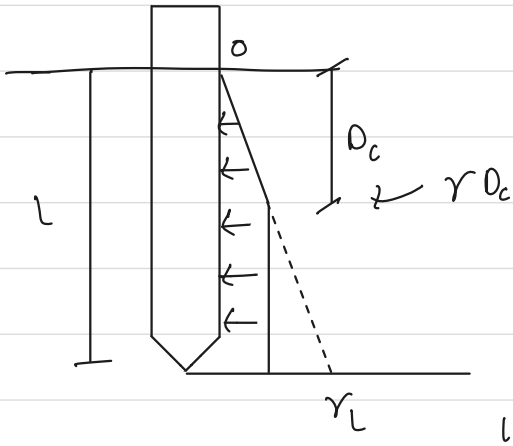
$F_s = 2$ to 3 ← if pile ld test is performed on site

← No pile ld test

$Q_a = \frac{1781.3}{3} = 593.7\ kN$

Regarding β method ← critical depth

After certain depth, vert. pressure no longer increase



$f_s = \beta \sigma_v$

avg from two

density of soil

Loose sand, $D_c = 10 \cdot D$ [40:00]

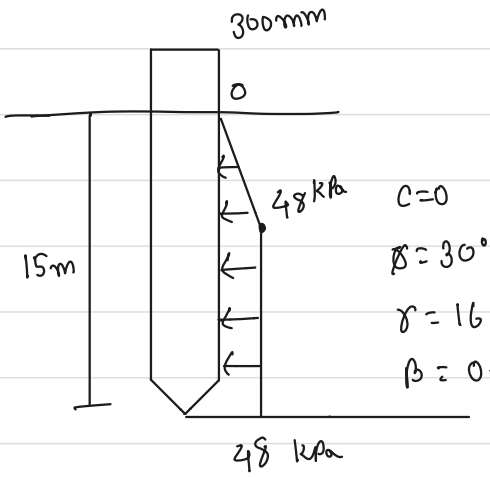
Med " , $D_c = 15 D$ ← 16–30

Dense, $D_c = 20 D$

> 30

$Q_s = Q_{s1} + Q_{s2}$

$= A_{s1} f_{s1} + A_{s2} f_{s2}$



$C = 0$

$\phi = 30°$

$\gamma = 16\ kN/m^3$

$\beta = 0.9$

$D_c = 10D = 10 \times 0.3$

$= 3\ m$

$Q_{s1} = A_{s1} f_{s1} = (\pi \times 0.3 \times 3)(0.9 \times \frac{48}{2})$

$= 61.07\ KN$

$Q_{s2} = A_{s2} f_{s2} = (\pi \times 0.3 \times 12)(0.9 \times 48)$

$= 488.5\ KN$

$Q_s = Q_{s1} + Q_{s2} = 0$

$= \cdots = 549.57\ KN$

$Q_b = A_b f_b = (\frac{\pi \times 0.3^2}{4})(48 \times 15)$

$= 50.89\ KN$

$Q_u = Q_s + Q_b = 600.49\ KN$

$Q_a = \frac{600.49}{3} = 200.15\ KN$

On ques: considering critical depth or omit
Pile ld test to be done → S.F = 2

Usually limits are imposed on f_s, f_b (w/o critical depth)
E.g., BNBC, $f_{b,max} = \cdots$

$f_{s,min} = \cdots$

if on ques limiting values are prov.

Bearing capacity of pile fdn

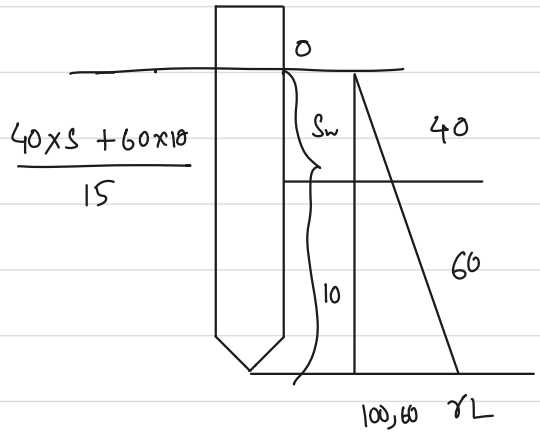
Small ltr → stress
Cap ltr → force

Skin friction : End bearing :

Cohesive → αC_u (alpha method) —— $9 C_u$

Cohesionless → $\beta \sigma_{v,avg}$ (beta method) —— qN_q ← γL

$\lambda(\sigma_{v,avg} + 2C_{u,avg})$ (λ method) — $9C_u$, qN_q, $(9C_u + qN_q)$



$f_s = 9 C_u$

$\beta = K \tan \delta$

For bored pile → $\frac{1}{3}(f_s)$; $\frac{1}{2}(q_s)$

Per column more than one pile normally
If pile dia > 2', may be one pile; preferrably not

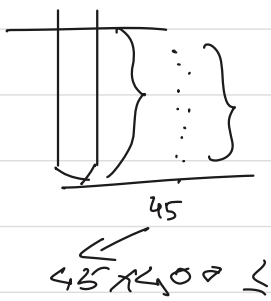
Capacity based on SPT value :

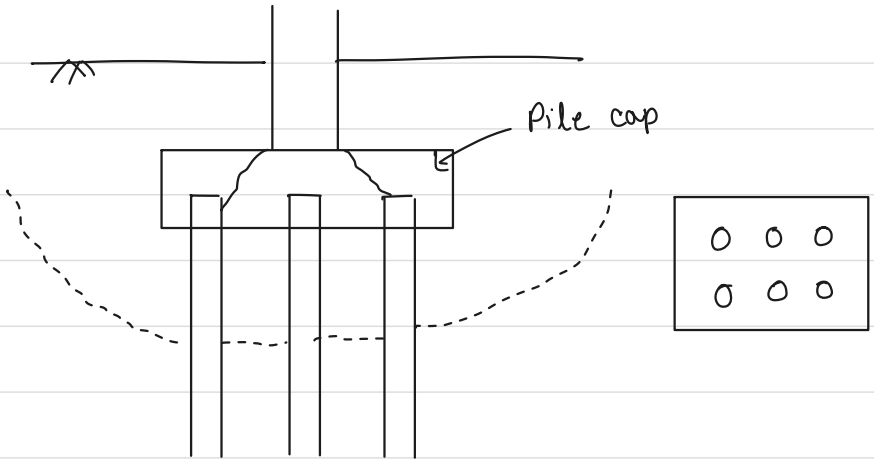
$N \rightarrow N_{60} \rightarrow (N_1)_{60}$ ← normally not considered; but for EQ, liquefaction

$\bar{N}$ → avg

End bearing — Skin friction

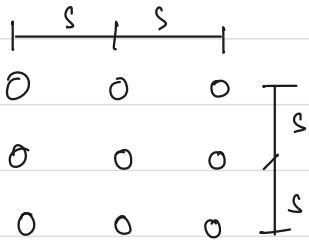
Sand = $400N$ (≤ 10000) — $= 2\bar{N} \leq 60$ kN/m² ;





Pile fdn

Piled fdn ⟶ Pile cap in contact w soil

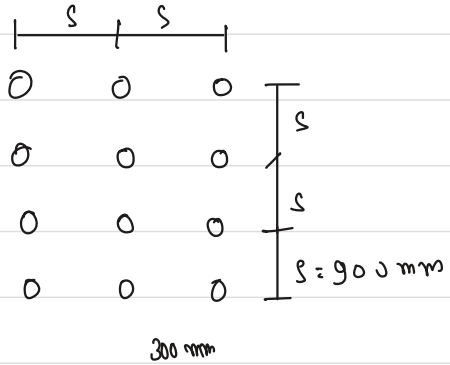


$s = 2d$ to $6d$

$= 3d$

Efficiency of pile:

$$\eta_g = 1 - \left[\frac{(n_1-1)n_2 + (n_2-1)n_1}{90\, n_1 n_2}\right]\theta$$



$n_1 = 4$

$n_2 = 3$

$\theta = \tan^{-1}\left(\frac{d}{s}\right) = \tan^{-1}\left(\frac{0.3}{0.9}\right)$

$= 18.4°$

$$\eta_g = 1 - \left[\frac{(4-1)\times 3 + (3-1)\times 4}{90 \times 4 \times 3}\right] \times 18.4$$

$= 1 - 0.29 = 0.71$

$Q_s = 250$ KN

$Q_b = 390$ KN

$Q_u = Q_s + Q_b = 640$ KN

$Q_{u,gnp} = 640 \times 0.71 = 454$ KN

Block method

$Q_s = A_s f_s$

$= (2.1 + 3.0) 2 \times 15 \times 0.7 \times 50$

$= 5355$ KN

$Q_b = 9 \times 100 \times 2.1 \times 3$

$= 5580$ KN

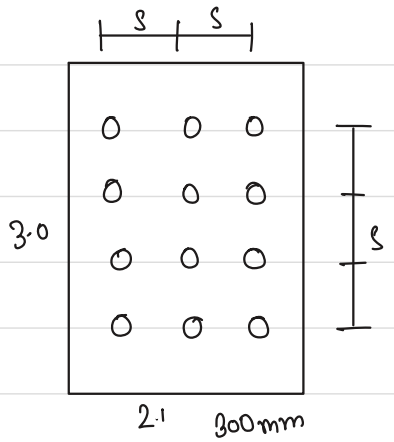
$Q_u = 5355 + 5580$

$= 10935$ KN

$C_u = 50$ kPa

$\alpha = 0.7$

$L = 15$ m



Efficiency = ...

$Q_s = A_s f_s$

$= (\pi \times 0.3 \times 15) \times 0.7 \times 50$

$= 494.5$ KN

$Q_b = 9 \times 100 \times \pi \times 0.3^2$

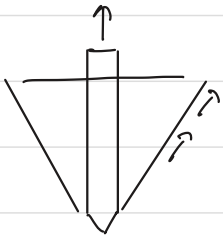
$= 63.6$ KN

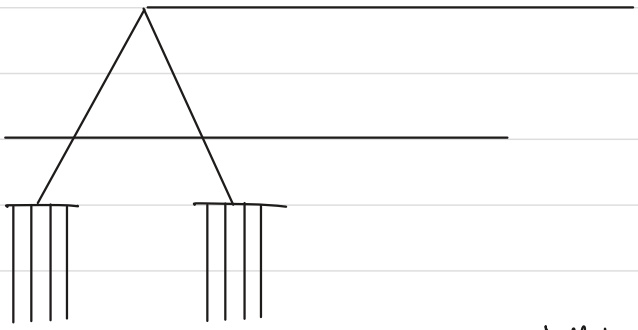
$Q_u = 494.5 + 63.6 = 558$ KN

$Q_{ug} = 558 \times 12 = 6696$ KN

Not for exam:

Uplift





~ 55:00

building	40mm	20mm
allowable	Settlement	dif. settlement

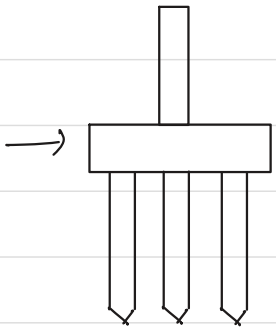
For lateral ld

wind ld, high rise

failure

Check → bearing cap

→ settlement

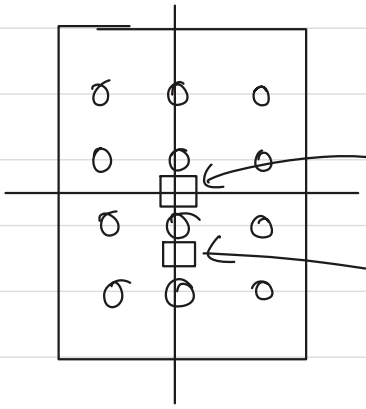


lateral movement/ settlement

normally, 1/2 of vertical sett. allowed

compressive frc, uplift, lateral ld

exact → site pile test



if at center → vert

in such case, least ld on one side

highest ld on another

if ld test performed → SF = 2 ← saves money

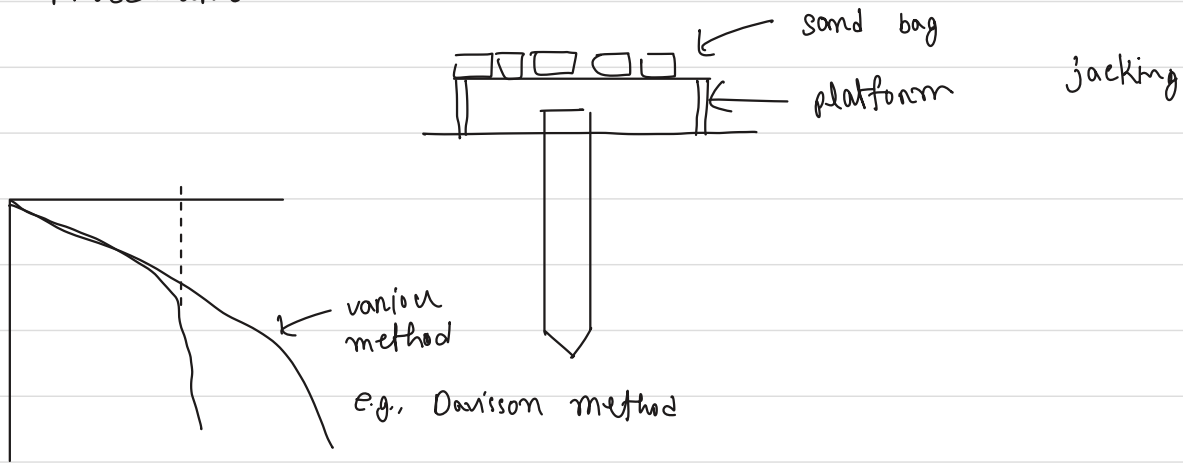
else → SF = 3

hence in big proj, pile ld test performed

Pile ld test

Double of design ld

procedure



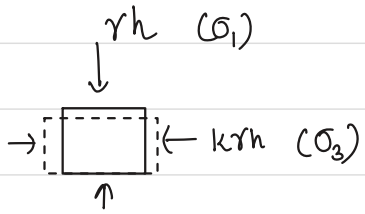
Rough / Final

27/04

Lateral Earth Pressure

(Ref. #187; Principles of Fdn Engg. B.M. Das

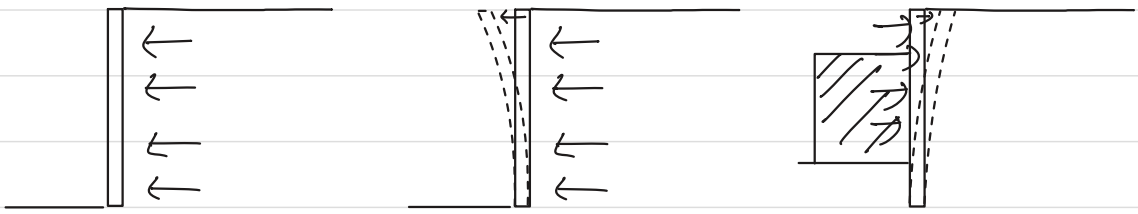
Earth pressure — Pressure exerted by soil against an engineering struct



Depends on →

1. Physical chars: cohesionless soil ($\tau = \sigma \tan\phi$)
   cohesive soil ($\tau = c$)
   c-φ soil ($\tau = c + \sigma \tan\phi$)

2. Deformation chars:



No deformation
Earth pressure
at rest
$K_o = \frac{\sigma_3}{\sigma_1}$

Lateral expansion
Active Earth pressure
Deforms away from
soil
Soil is causing to deform
$K_a = \frac{\sigma_h}{\sigma_v}$

Lateral contraction
Passive Earth pressure
Deforms toward soil
Soil is resisting
$K_p = \frac{\sigma_h}{\sigma_v}$

$K \rightarrow$ coeff of lateral earth pressure; $K_p > K_0 > K_a$

Requires wall movement to achieve passive/acting stage

On passive cond. soil is also resisting

Earth pressure at rest: when retaining wall may be restrained from moving, the lateral earth pressure on the wall at any depth is called at-rest earth pressure.

$$\sigma_n = K_0 \sigma_v = K_0 \gamma h$$

For normally consolidated soil: $K_0 \approx 1 - \sin\phi'$

Over-consolidated soil: $K_0 = (1 - \sin\phi')\, OCR^{\sin\phi'}$

} empirical

$\phi' \rightarrow$ Drained friction angle of soil (effective)

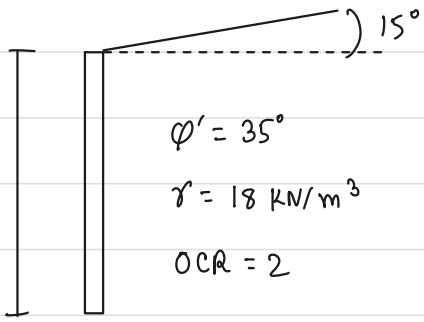
OCR $\rightarrow$ Over consolidation ratio

$\rightarrow$ Past eff. vert. stress / current eff. vert stress

OCR = 1 $\rightarrow$ normally consolidated

> 1 $\rightarrow$ over consolidated

P21/ Example



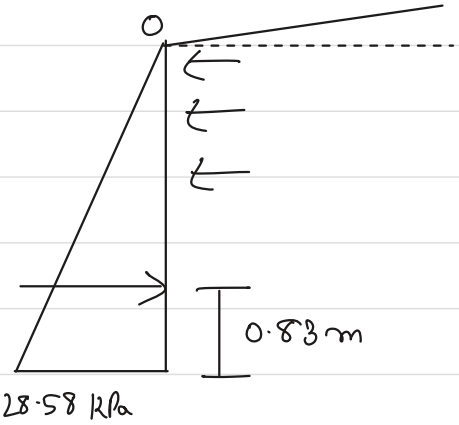
Step 1: $K_o = (1 - \sin\phi')\, OCR^{\sin\phi'}$

$= (1 - \sin 35^\circ) \times 2^{\sin 35^\circ}$

$= 0.635$

Step 2: $P_o(h) = K_o \gamma h$

$= 0.635 \times 18h \quad = 11.43h$



Step 3: $P_o(0) = 0$

$P_o(2.5) = 28.58$ kPa

Step 4: Total force $= \frac{1}{2} 28.58 \times 2.5$

on, point ld : 35.73 kN/m-len

Loc of line of action, $\bar{y} = \frac{1}{3} \times 2.5 = 0.83$m

Step 5: Connection factor $= (1 + 0.5 \tan\beta)^2$

$= (1 + 0.5 \tan 15^\circ)^2 \quad = 1.29$

Connected point ld $= 35.73 \times 1.29 = 46.09$ kN/m

Lateral Earth Pressure Theories:

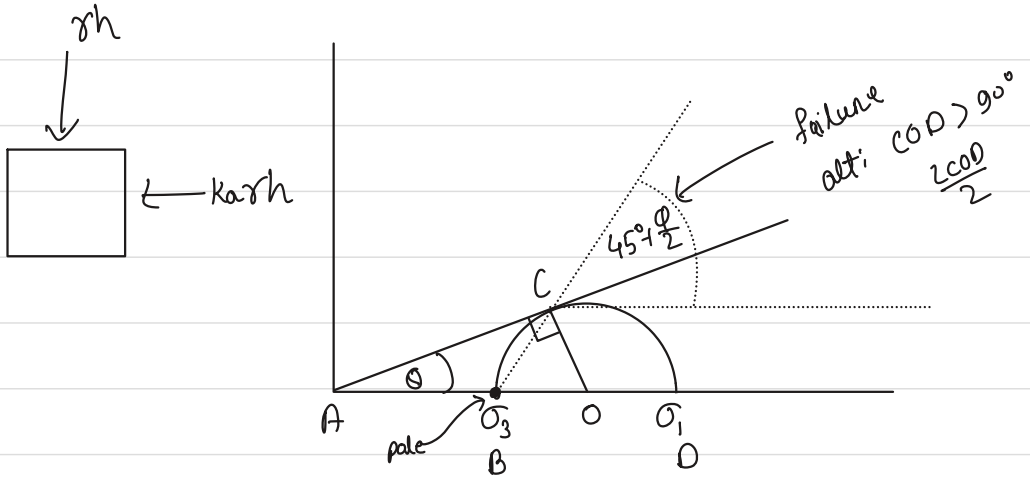
Rankine & Coulomb

Rankine's Earth Pressure Theory:

Assumptions:

(i) Soil mass is semi-infinite, homogeneous, cohesionless
(ii) Ground surface is horizontal or inclined
(iii) Retaining wall is vertical, smooth
(iv) Conjugate relation exists between vertical & hor. pressures

Active Earth Pressure (Cohesionless soil)

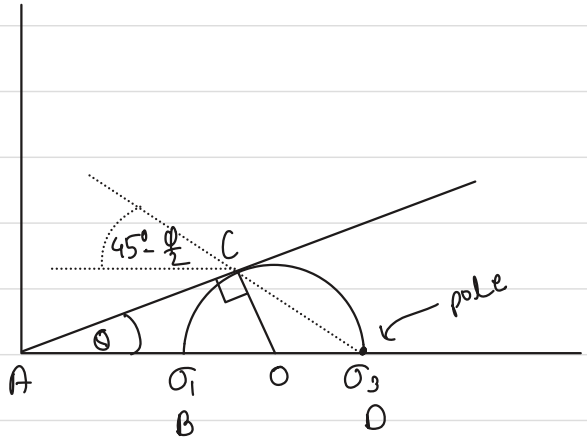


$$K_a = \frac{\sigma_3}{\sigma_1} = \frac{AB}{AD} = \frac{AO-OB}{AO+OD} = \frac{AO-OC}{AO+OC}$$

$$= \frac{1-\frac{OC}{AO}}{1+\frac{OC}{AO}} = \frac{1-\sin\phi}{1+\sin\phi}$$

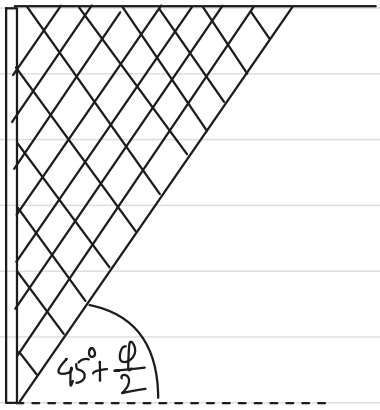
$$\Rightarrow K_a = \tan^2\left(45°-\frac{\phi}{2}\right)$$

Passive earth pressure (cohesionless soil)

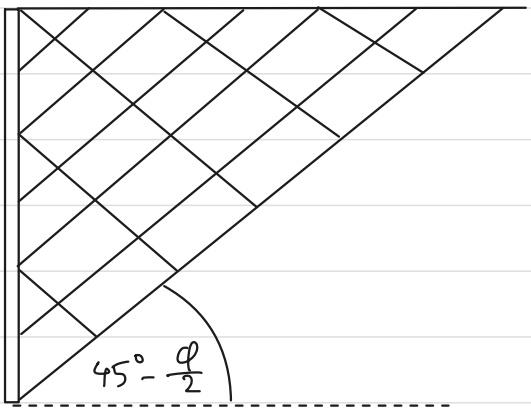


$$K_p = \tan^2\left(45° + \frac{\phi}{2}\right)$$

Failure planes:



Active condition



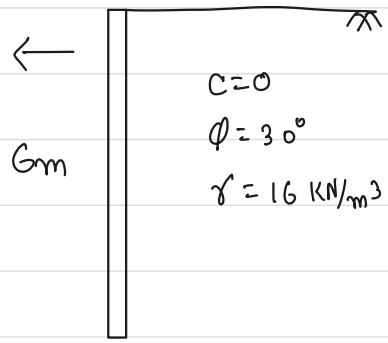
Passive condition

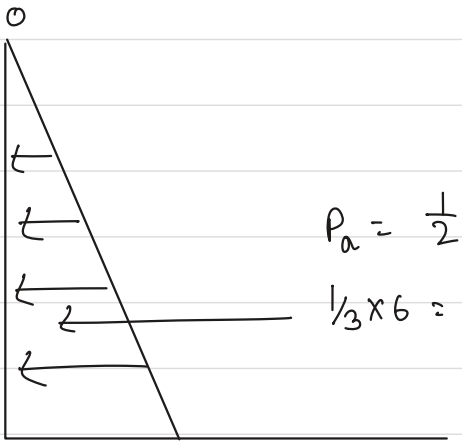
#

$$K_a = \tan^2\left(45^\circ - \frac{\phi}{2}\right)$$

$$= \tan^2\left(45^\circ - \frac{30^\circ}{2}\right)$$

$$= 0.333$$





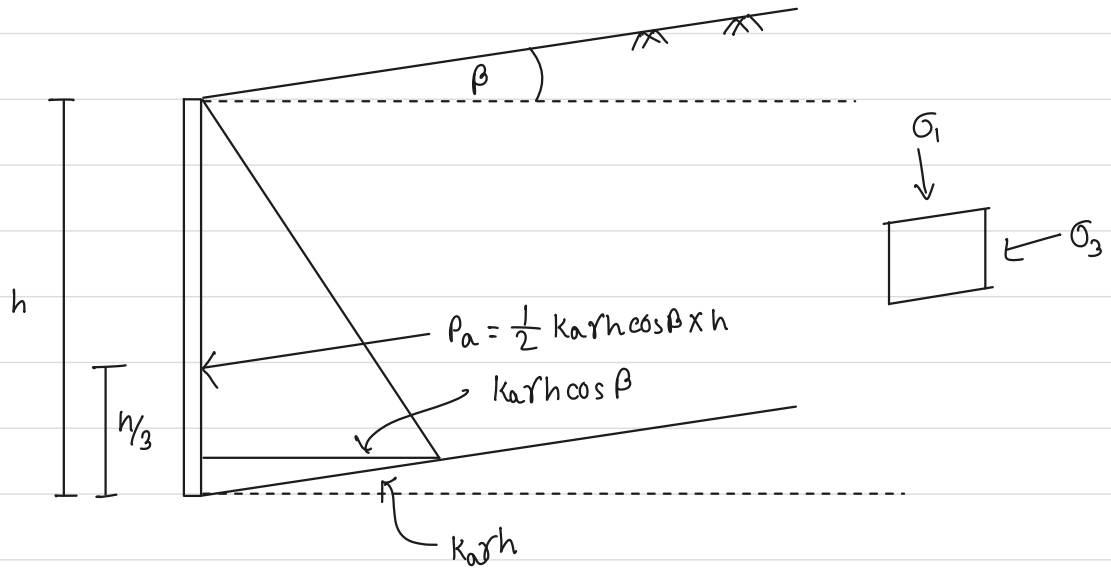
$P_a = \frac{1}{2} 32 \times 6 = 96\ KN/m$

$1/3 \times 6 = 2\ m$

$K_a \gamma h$

$= 32\ KPa$

Active earth pressure: cohesionless, inclined ground surface



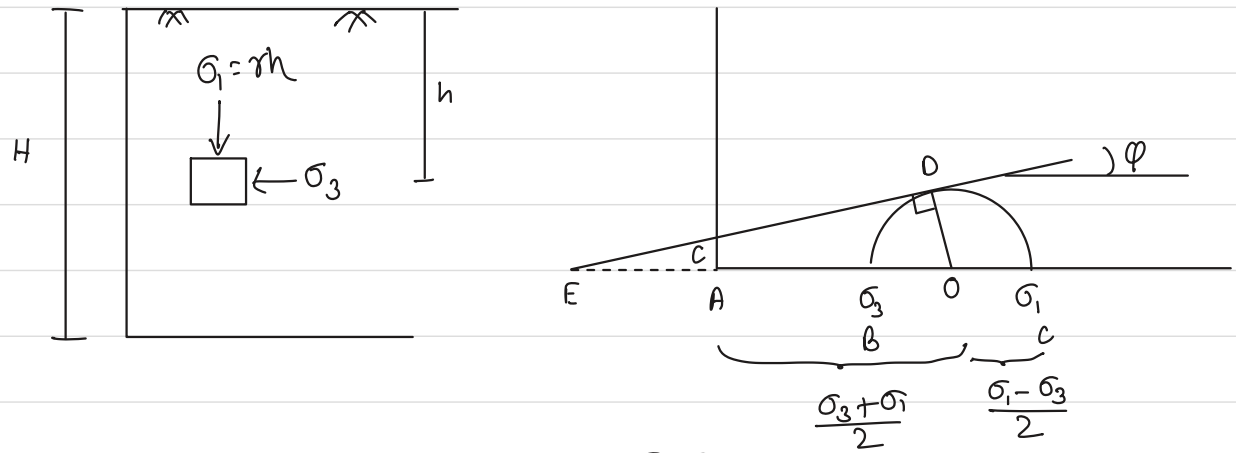
$$K_a = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

Passive earth pressure: inclined, $c = 0$ soil

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

C-φ soil:

Active earth pressure:



$$\sin\varphi = \frac{DO}{EO} = \frac{DO}{EA+AO} = \frac{\frac{\sigma_1-\sigma_3}{2}}{c\cot\varphi + \frac{\sigma_3+\sigma_1}{2}}$$

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

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

$$\Rightarrow 2c\cos\varphi = \sigma_1 - \sigma_1\sin\varphi - \sigma_3 - \sigma_3\sin\varphi$$

$$\Rightarrow 2c\cos\varphi = \sigma_1(1-\sin\varphi) - \sigma_3(1+\sin\varphi)$$

$$\Rightarrow \sigma_3 = \sigma_1\frac{1-\sin\varphi}{1+\sin\varphi} - 2c\frac{\cos\varphi}{1+\sin\varphi}$$

Now, $$\frac{\cos\varphi}{1+\sin\varphi} = \frac{\sqrt{1-\sin^2\varphi}}{(\sqrt{1+\sin\varphi})^2}$$

$$= \frac{\sqrt{1-\sin\varphi}}{\sqrt{1+\sin\varphi}} = \sqrt{K_a}$$

$$\therefore \sigma_3 = \sigma_1 K_a - 2c\sqrt{K_a}$$

Active earth pressure (ne):



$$\sin\phi = \frac{DO}{EO} = \frac{DO}{EA+AO} = \frac{\frac{\sigma_1-\sigma_3}{2}}{c\cot\phi + \frac{\sigma_1+\sigma_3}{2}}$$

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

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

$$\Rightarrow 2c\cos\phi = \sigma_1 - \sigma_1\sin\phi - \sigma_3 - \sigma_3\sin\phi$$

$$= \sigma_1(1-\sin\phi) - \sigma_3(1+\sin\phi)$$

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

Now, $\frac{\cos\phi}{1+\sin\phi} = \frac{\sqrt{1-\sin^2\phi}}{\left(\sqrt{1+\sin\phi}\right)^2}$

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

$\therefore \sigma_3 = \sigma_1 K_a - 2c\sqrt{K_a}$

Depth of tension crack (h_c):

$$\sigma_3 = K_a \gamma h - 2c\sqrt{K_a}$$

At $h = 0$, $\sigma_3 = -2c\sqrt{K_a}$ (i.e, pressure is neg → tension)

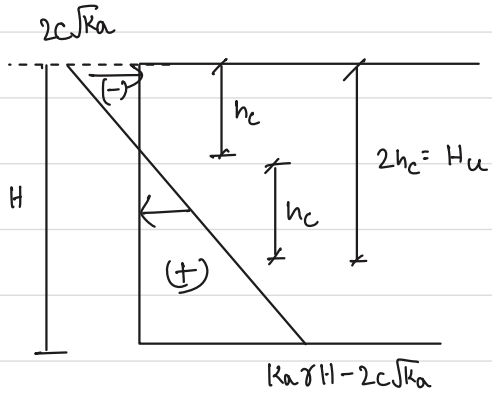
$$\sigma_3 = 0 \Rightarrow K_a \gamma h_c - 2c\sqrt{K_a} = 0$$

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

$$= 2c \frac{1}{\sqrt{K_a}\gamma}$$

$$= 2c \frac{1}{\gamma\sqrt{\tan^2(45^\circ - \frac{\phi}{2})}}$$

$$= \frac{2c}{\gamma} \tan(45^\circ + \frac{\phi}{2})$$



Theoretical unsupported ht, $H_u = 2h_c = \frac{4c}{\gamma}(\tan 45^\circ + \frac{\phi}{2})$

At $2h_c$, resultant force = 0

For purely cohesive soil ($\phi = 0$): $h_c = \frac{2c}{\gamma}$

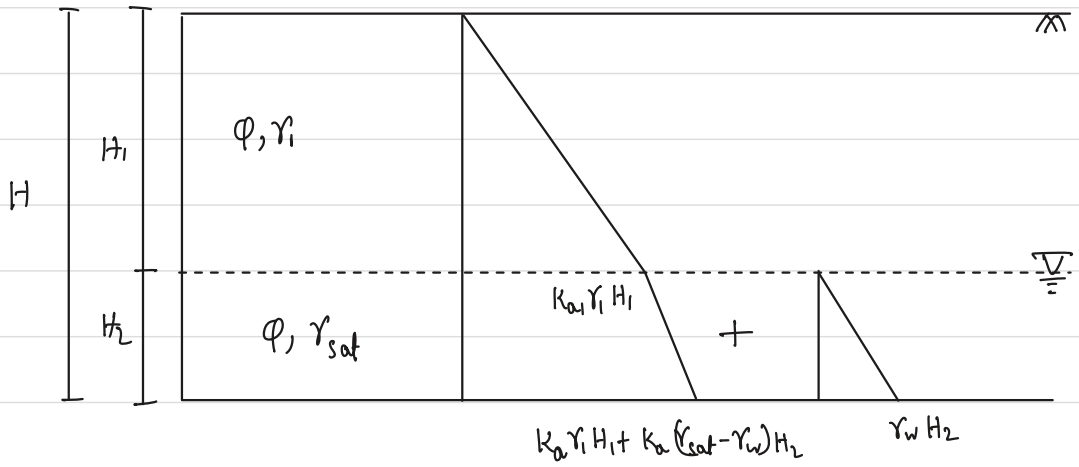
Total thrust area: only pos portion as soil doesn't resist any tension (i.e., won't pull wall)

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

Passive earth pressure:

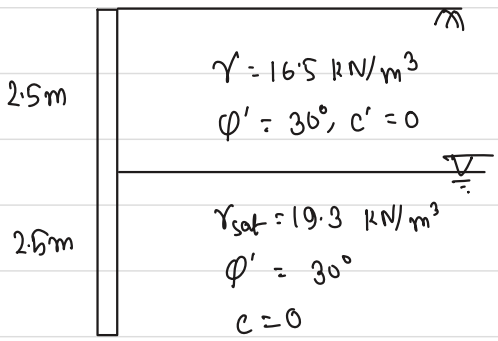
$$\sigma_3 = K_p \sigma_1 + 2c\sqrt{K_p}$$

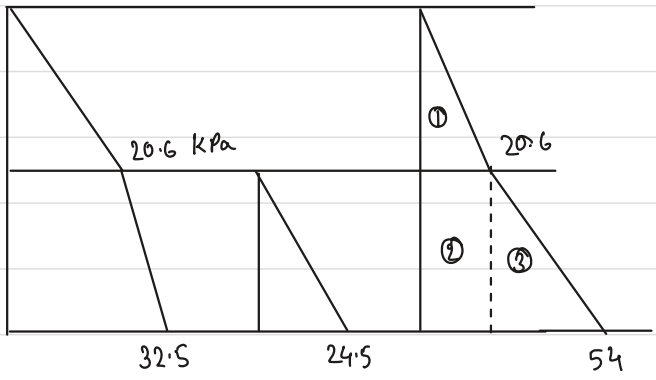
Effect of water table:



[Pore water pressure reduces vertical pressure → ↓ lateral pressure
water induces hydrostatic pressure]

Ex. 12.1





OCR = 1

$K_{o1} = (1 - \sin\varphi')\, OCR^{\sin\varphi'}$

$= (1 - \sin 30°) = 0.5$

$K_{o2} = 0.5$

$P_o = A_1 + A_2 + A_3$

$= 0.5 \times 20.6 \times 2.5 + 20.6 \times 2.5 + 0.5 \times (54 - 20.6) \times 2.5$

$= 25.75 + 51.5 + 41.75$

$= 119\ kN/m$

$\bar{x} = \dfrac{\Sigma \gamma_i A_i}{\Sigma A_i}$

$= \dfrac{25.75 \times (2.5 + \frac{2.5}{3}) + 51.5 \times 1.25 + 41.75 \times \frac{2.5}{3}}{119}$

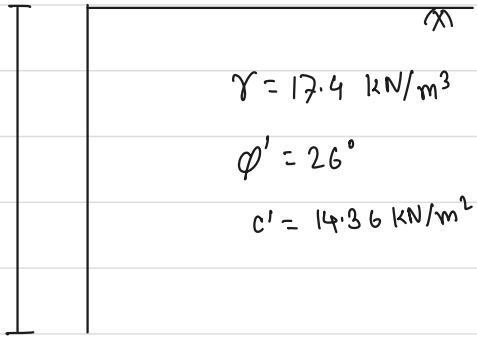
$= 1.55\ m$

Ex. 12.2

$$K_a = \tan^2(45° - \frac{\phi}{2})$$

$$= \tan^2(45° - 26°/2)$$

$$= 0.3905$$



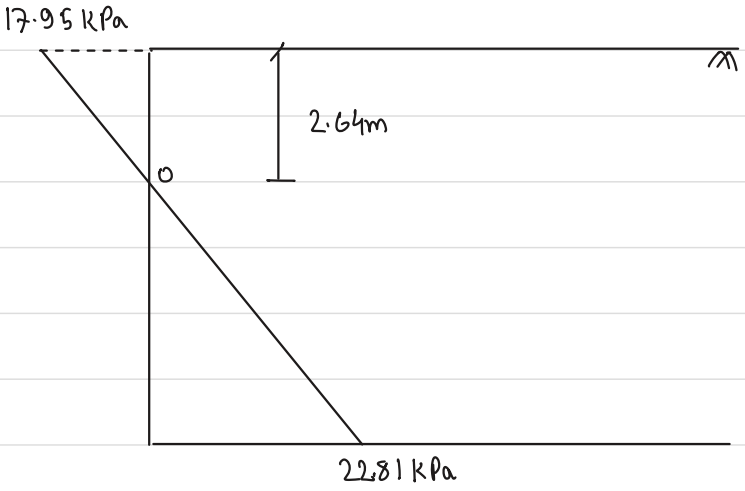
Def^n h_c:

$$\sigma_3' = K_a \gamma h_c - 2c\sqrt{K_a} = 0$$

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

$$\Rightarrow 0.3905 \times 17.4 \times h_c = 2 \times 14.36 \times \sqrt{0.3905} = 17.95 \text{ kPa}$$

$$\Rightarrow h_c = 2.64 \text{ m}$$



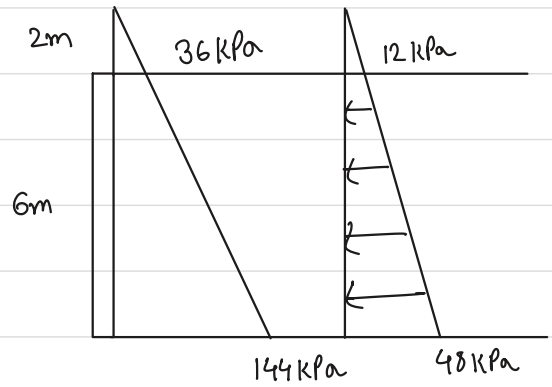
After tensile cracks appear:

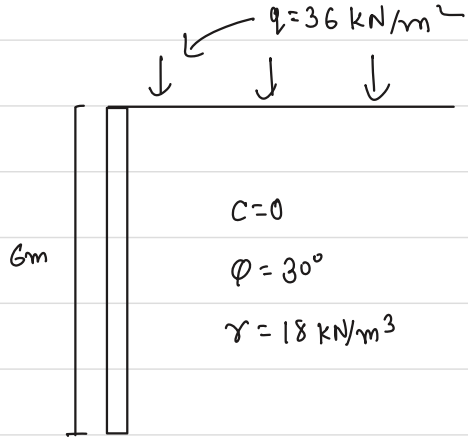
$$P_a = 0.5 \times 22.81 \times (6 - 2.64)$$

$$= 38.32 \text{ kN/m}$$

$$\bar{y} = \frac{1}{3}(6 - 2.64) = 1.12 \text{ m}$$

Effect of uniform load:





$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.3333$$

Equiv ht, H_u: $36 = 18 \times H_u \Rightarrow H_u = 2m$

Alt: $H_u = 2m$

$\therefore P_a(h) = K_a \gamma (h + H_u)$

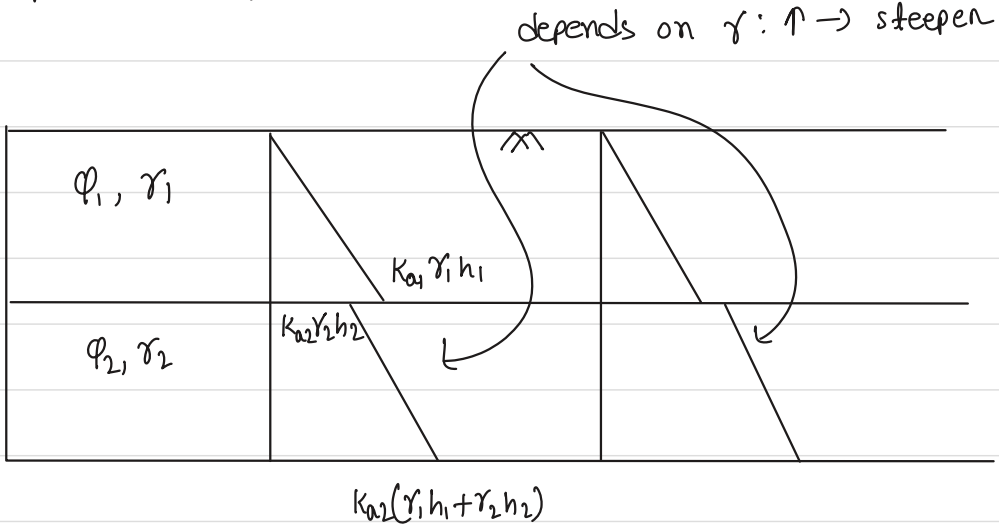
$= 0.3333 \times 18(2+h)$

$= 6(2+h)$

$P_a(0) = 12 KPa$

$P_a(6) = 48 KPa$

Effect of stratification:



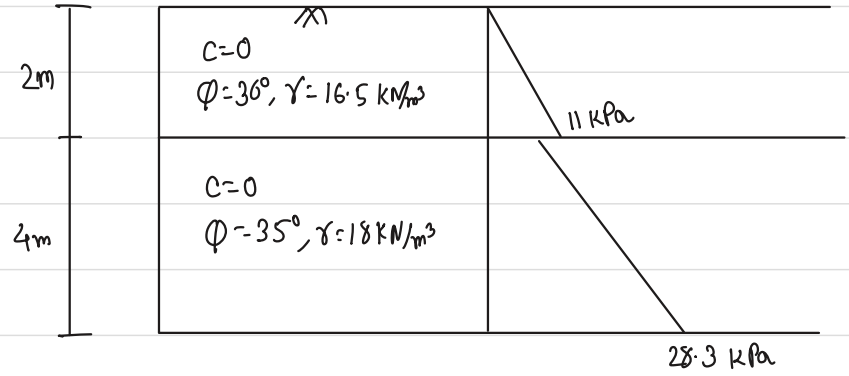
if $\varphi_2 > \varphi_1$ if $\varphi_2 < \varphi_1$

$K_{a1} = \tan^2(45° - \frac{30°}{2})$

$= 0.3333$

$K_{a2} = \tan^2(45° - \frac{35°}{2})$

$= 0.2710$



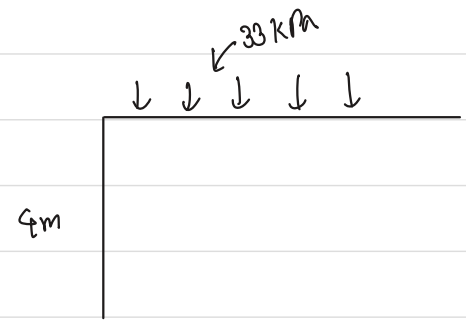
$H_u = \frac{33}{18} = 1.83$ m

$\therefore P_{a2} = K_{a2} \gamma_2 (h + H_u)$

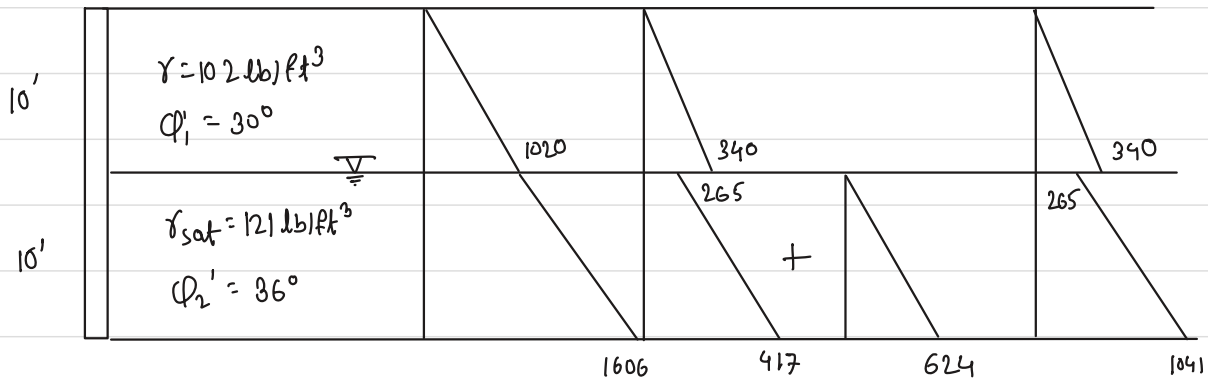
$= 0.270 \times 18 (1.83 + h)$

$= 4.86 (1.83 + h)$

$\therefore P_{a2}(4) = 28.3$ kPa



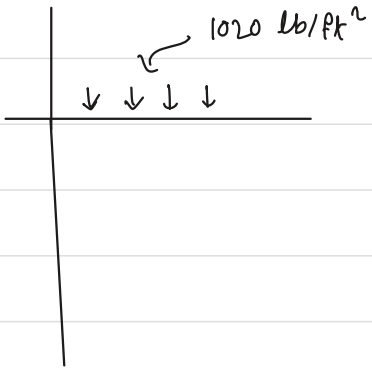
Ex. 12.3 Active



Layer 1: $K_{a1} = \tan^2(45° - \frac{\phi}{2}) = 0.3333$

$P_v(10') = \gamma h = 102 \times 10 = 1020 \text{ lb/ft}^2$

$P_a(10') = K_{a1} P_v = 340 \text{ lb/ft}^2$



Layer 2: $K_{a2} = \tan^2(45° - \frac{\phi}{2}) = 0.2596$

$H_e = \frac{1020}{121 - 62.4} = 17.41 \text{ ft}$

$P_a(h) = (\gamma - \gamma_w)(H_e + h) K_{a2}$

$= 58.6 \times (17.41 + h) \times 0.2596$

$P_w(10') = 62.4 \times 10 = 624 \text{ lb/ft}^2$

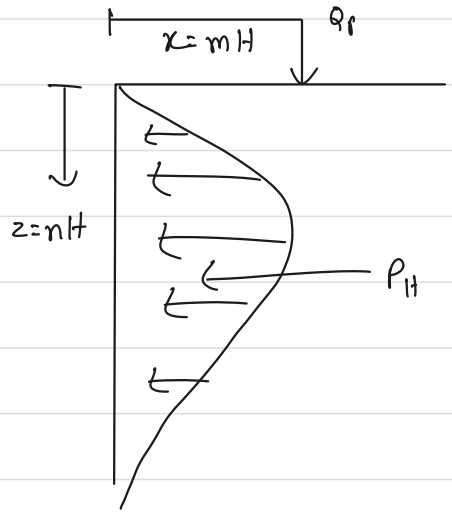
∴ Point $Ld = 0.5 \times 340 \times 10 + 265 \times 10 + 0.5 (1041 - 265) \times 10$

$= 1700 + 2650 + 3880$

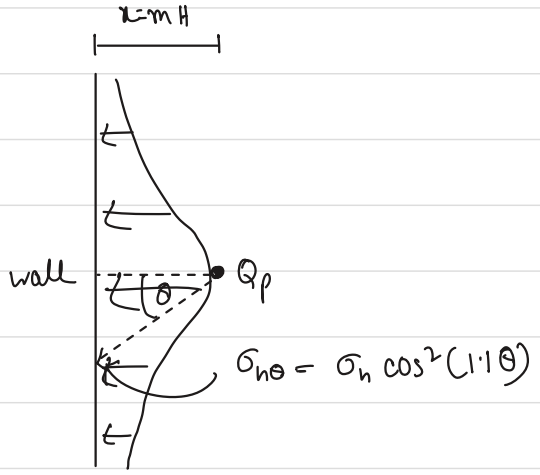
$= 8230 \text{ lb/ft}$

$\bar{y} = \ldots$

Effect of point ld:



vert. section



top view

For $m \leq 0.4$: $\sigma_H = \frac{0.28Q}{H^2} \frac{n^2}{(m^2+n^2)^3}$; $m = \max(0.4, m)$

$m > 0.4$: $\sigma_H = \frac{1.77Q}{H^2} \frac{n^2}{(m^2+n^2)^3}$

At least 6 sections

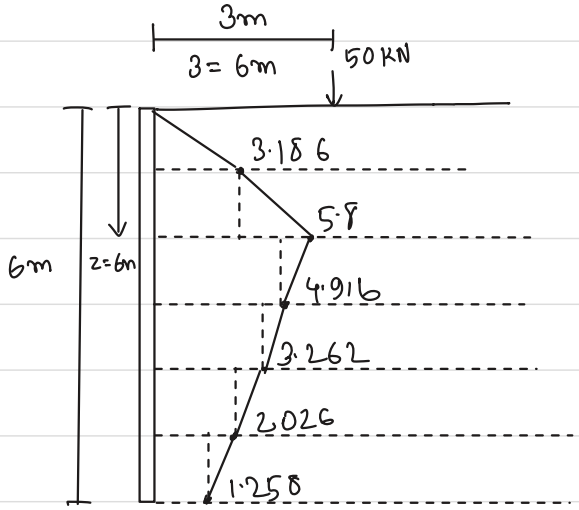
#

$$m = \frac{3}{6} = 0.5 > 0.4$$

$$\therefore \sigma_H = \frac{1.77Q}{H^2} \frac{n^2}{(m^2+n^2)^3}$$

$$= \frac{1.77 \times 50}{6^2} \frac{n^2}{(0.5^2+n^2)^3}$$

$$= 2.458 \frac{n^2}{(0.25+n^2)^3}$$



z	n	q_H
0	0	0
1	0.167	3.186
2	0.333	5.800
3	0.5	4.916
4	0.667	3.262
5	0.833	2.026
6	1	1.258

Area A → force

$A_1 = 0.5 \times 3.186 \times 1 = 1.593$

$A_2 = \frac{3.186+5.8}{2} \times 1 = 4.493$

$A_3 = \frac{5.8+4.916}{2} \times 1 = 5.358$

$A_4 = \frac{4.916+3.262}{2} \times 1 = 4.089$

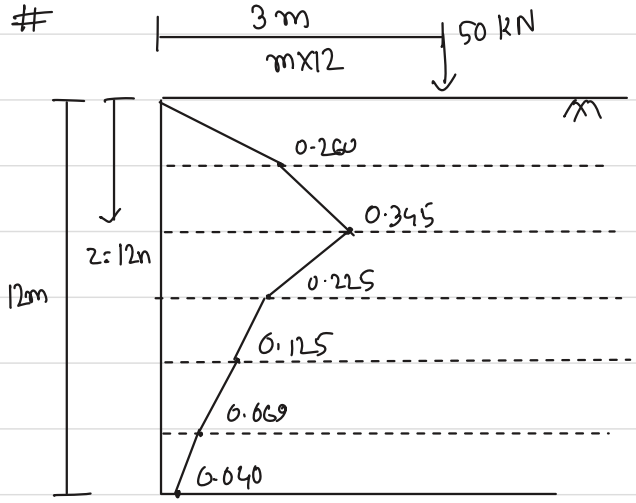
$A_5 = \frac{3.262+2.026}{2} \times 1 = 2.644$

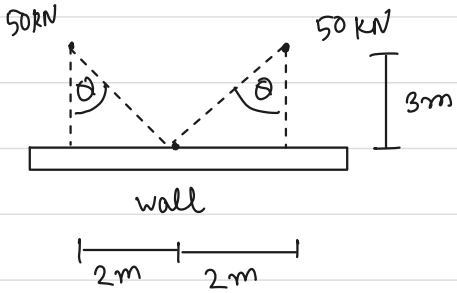
$A_6 = \frac{2.026+1.258}{2} \times 1 = 1.642$

$\therefore \Sigma A = 19.819$

$\bar{x} = \dots$

#





$\theta = \tan^{-1}\frac{2}{3} = 33.69°$

$m = \frac{3}{12} = 0.25 \leq 0.4$

$n = z/12$

$\sigma_{H\theta}$	z	n	σ_H	$\sigma_{H\theta}$
	0	0	0	0
	2	0.167	0.408	0.260
	4	0.333	0.542	0.345
	6	0.5	0.353	0.225
	8	0.667	0.196	0.125
	10	0.833	0.108	0.069
	12	1	0.062	0.040

$$\sigma_H = \frac{0.28Q}{H^2}\frac{n^2}{(n^2+0.16)^3}$$

$$= \frac{0.28 \times 50}{12^2} \times \frac{n^2}{(0.16+n^2)^3}$$

$$= 0.097\frac{n^2}{(0.16+n^2)^3}$$

$\sigma_{H\theta} = \sigma_H \cos^2(1.1\theta)$

$= \sigma_H \times 0.637$

Area, A → Force:

$A_1 = 0.26$

$A_2 = 0.61$

$A_3 = 0.57$

$A_4 = 0.35$

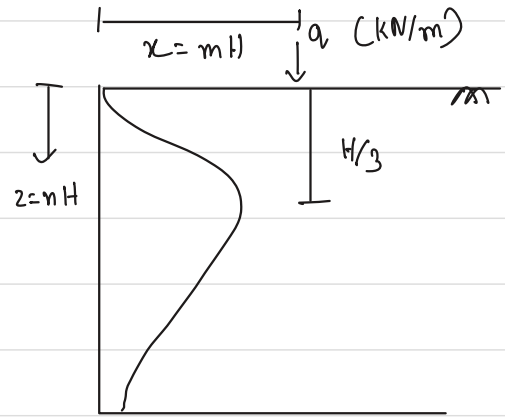
$A_5 = 0.19$

$A_6 = 0.11$

Total thrust for single load = $\Sigma A = 2.08$ KN

for both load = $2 \times \Sigma A = 4.16$ KN

Effect of line load:

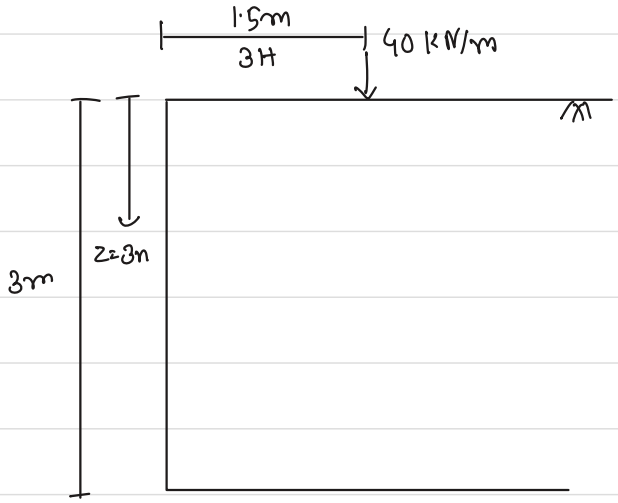


$m = 0.4 \rightarrow \frac{4}{\pi} \times m^2 = 0.2037\ldots \rightarrow 0.203$

$m > 0.4: \quad \rho_H = \frac{4q}{\pi H} \frac{m^2 n}{(n^2+m^2)^2}$

$m \leq 0.4: \quad \rho_H = \frac{q}{H} \frac{0.203\, n}{(n^2+m^2)^2}; \quad m = 0.4$

#



$m = \frac{1.5}{3} = 0.5 > 0.4$

$n = \frac{z}{3}$

$\therefore \rho_h = \frac{4q}{\pi H} \frac{m^2 n}{(n^2+m^2)^2}$

$= 4.244 \frac{n}{(n^2+0.25)^2}$

ρ_h:

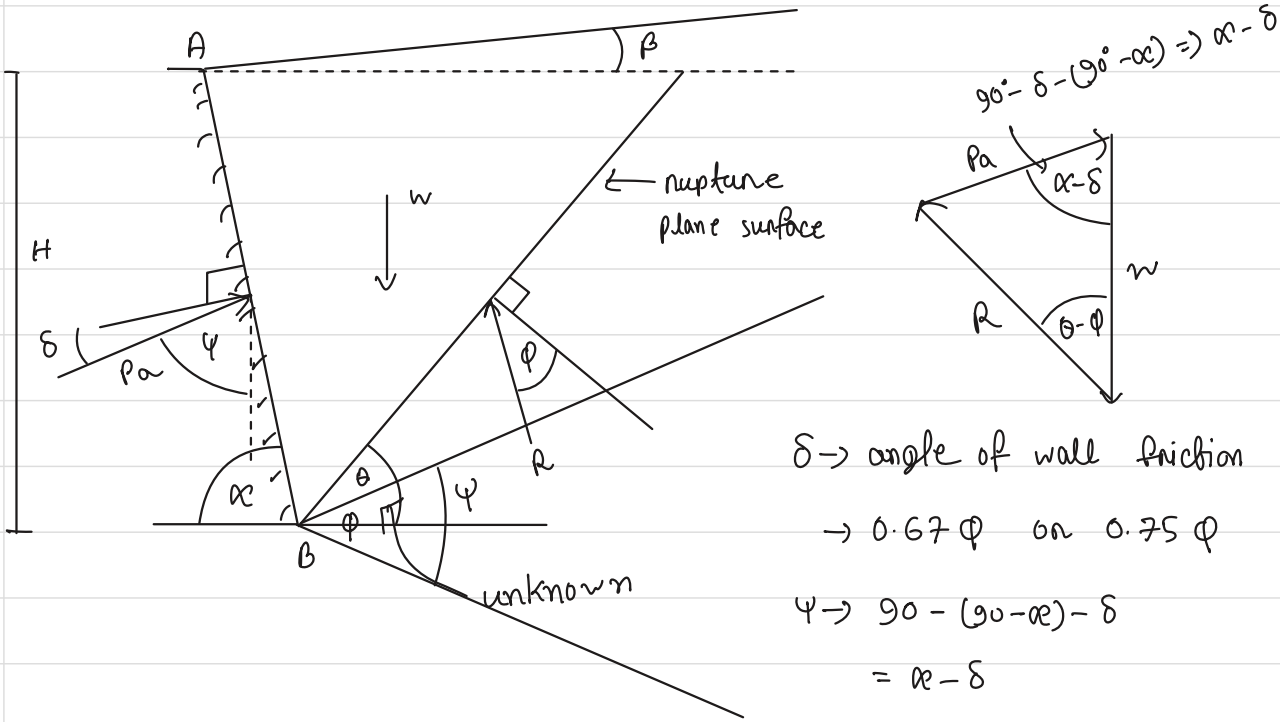
z	n	ρ_h
0	—	—
0.5	—	—
1	—	—
1.5	—	—
2	—	—
2.5	—	—
3	—	—

$\Sigma \rho_h = \ldots$

Coulomb's theory of lateral earth pressure:

Assumptions:

(i) Soil is homogenous, isotropic
(ii) Failure wedge is a rigid body
(iii) Both the rupture & pressure surfaces are plane
(iv) There may be friction between wall & soil
(v) Failure is two dimensional
(vi) Soil is cohesionless

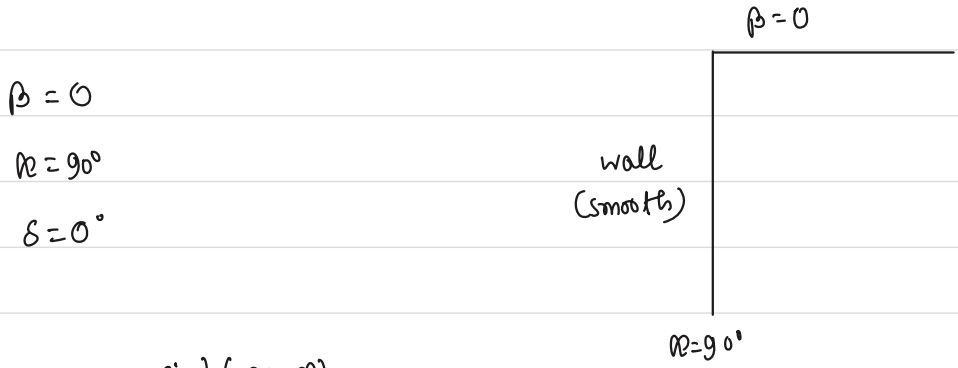


δ → angle of wall friction
→ 0.67 φ or 0.75 φ

ψ → 90 - (90 - α) - δ
= α - δ

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

Compared to Rankine's:



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

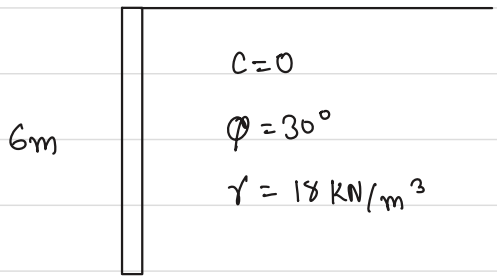
$$= \frac{\cos^2\phi}{1\times 1\times\left[1+\sqrt{\frac{\sin\phi\sin\phi}{1\times 1}}\right]^2}$$

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

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

$$= K_a$$

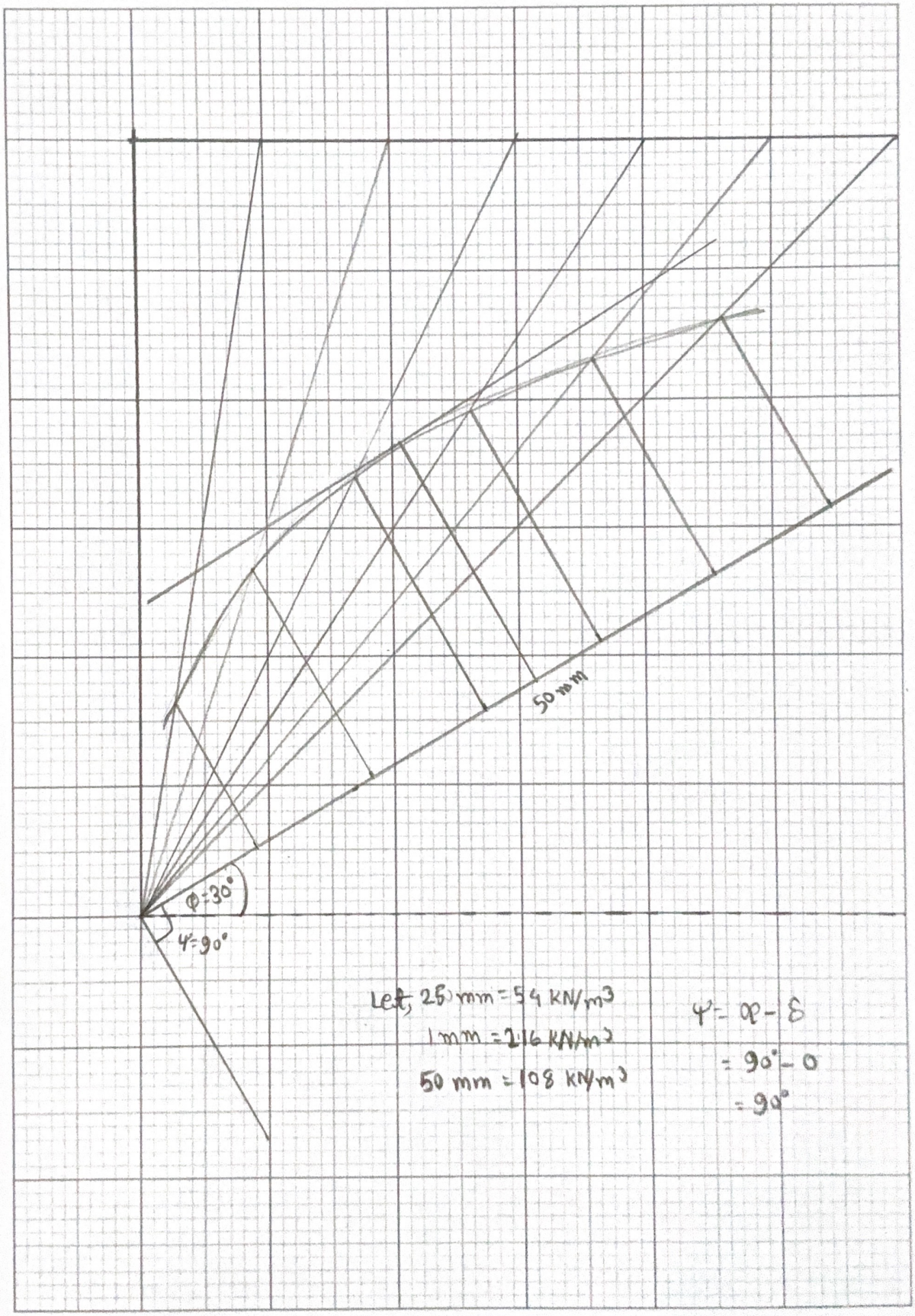
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failure plane: 45°+

Taking 6 trial wedges $w_1 = w_2 = \cdots = w_6 = \frac{1}{2} \times 2 \times 6 \times 18$

$= 54$ KN



Let, 25 mm = 54 KN/m³

1 mm = 2.16 KN/m³

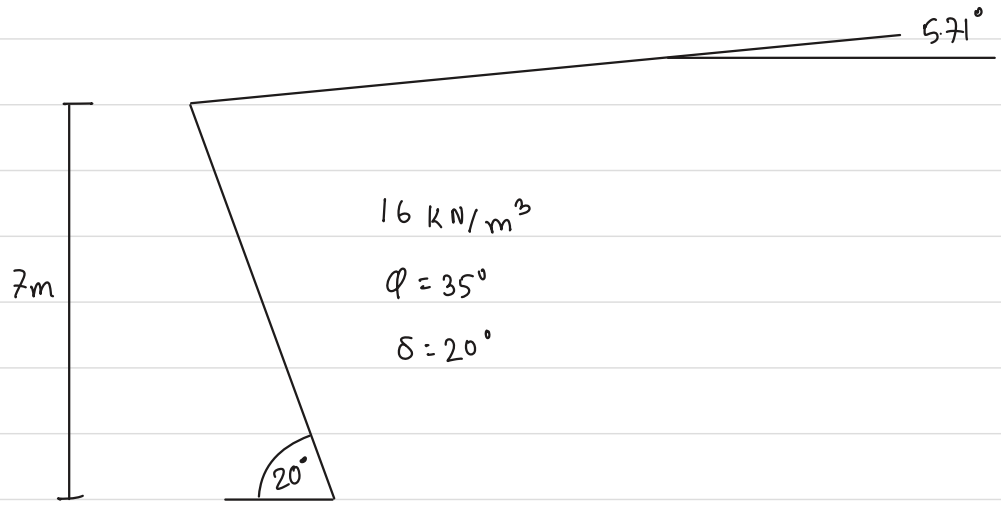
50 mm = 108 KN/m³

$\psi = \alpha - \delta$

$= 90° - 0$

$= 90°$

Ex. 11.13



Subsurface investigation

Planning: 4 phases:

1. Obtain available info
2. Field recon
3. Preli. investigation
4. Detailed "

Info req:

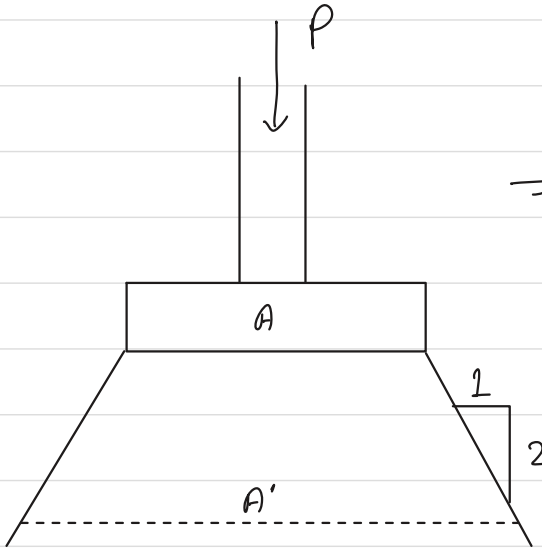
1. Nature of deposit
2. Depth, thickness of each strata
3. GWT loc
4. Engg. props

Considerations

1. Cost of exploration
2. Number & location of boring
3. Spacing of boring
4. Depth of boring

Depth of boring

Technique 1: Until net increase in stress caused by the proposed struct is less than 10% of max value.



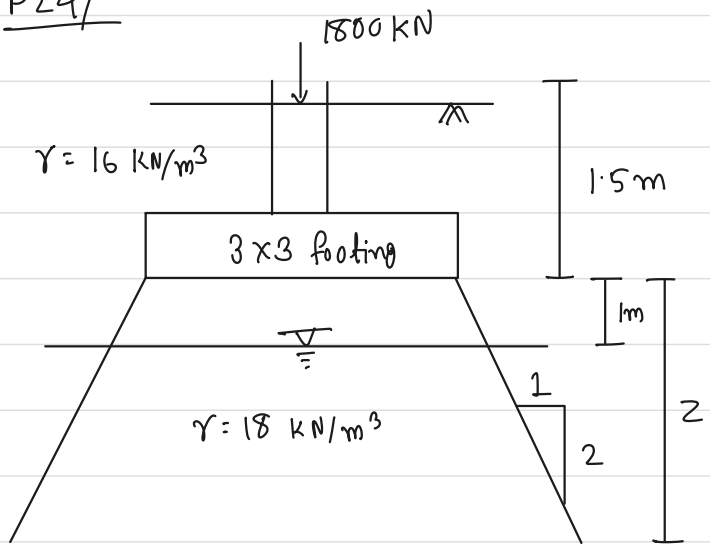
$\rightarrow q_{max} = \frac{P}{A}$

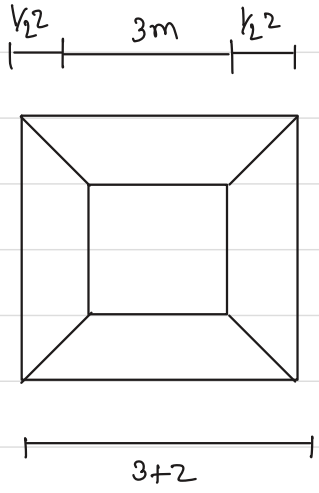
10% of $q_{max} = \frac{P}{A'}$

$A' = 10 \times A$

Technique 2: Until stress becomes less than 5% of eff. vert. stress caused by soil wt

P24/





Gross increase = 1800/9 = 200 kPa

Net increase : $200 - \gamma D_f$

$= 200 - 16 \times 1.5 = 176$ kPa

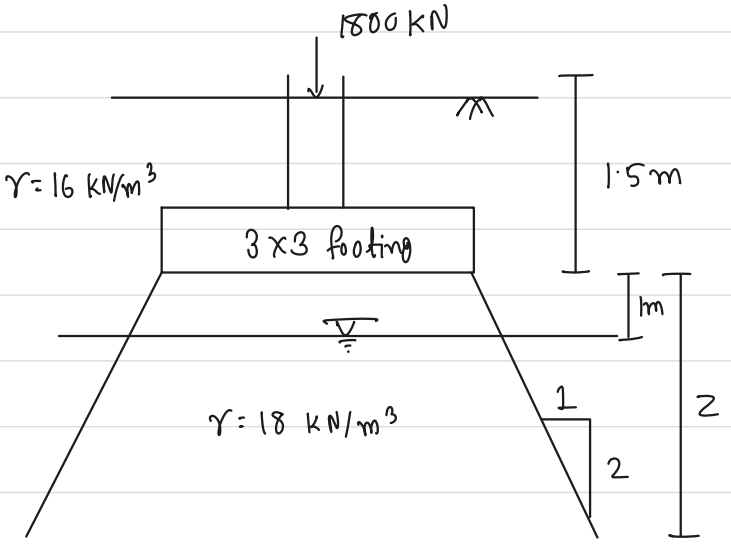
Prev ld gets replaced by new load

Depth of boring: 10% of 176 = 17.6 kPa

$17.6 = \frac{176 \times 3 \times 3}{(3+z)^2} \Rightarrow z = 6.5$ m $\quad \left[p = \frac{F}{A}\right]$

Total depth = 1.5 + 6.5 = 8 m

Technique 2:



Net increase in stress = 17.6 kN

$$\frac{17.6 \times 3 \times 3}{(3+z)^2} = 0.05 \left[16 \times 2.5 + (18 - 9.81) \times (z-1) \right]$$

$\therefore z = 4\ m$

Use min of two

Empirical eqn

Narrow: $L/B > 10$ $\quad z_b = 3S^{0.7}$

Wide: $z_b = 6S^{0.7}$

Total depth $= z_b + d_f$

S → # of equiv. stories

10 KN/m² → 1 storey

20 KN/m² → 2 storey

method of boring

1. Auger boring
2. Hollow stem auger boring
3. Wash boring
4. Percussion boring
5. Rotary drilling
6. ODEX drilling

Auger boring

soils which can stay open w/o casing

Upto 10 m for hand op

30 m " power driven

Hollow stem auger boring

Upto 50 m

hollow stem facilates prevention of caving

wash boring

very popular

works on any soil except for very stiff, hard, cemented, or rock layer

A hollow drill bit is connected to a hollow drill rod which is connected to tripod with rope as a pulley system.

The hole is advanced by a sequence of jetting and chopping of the soil using the drill bit. The separated soils is then transported upward through the annular space btwn drill bit & wall. The the soil settles in a tank while overflown water is repurposed for drilling

Soil is very disturbed & only provide identification of strata. Primarily used for advancing.

Soil sample

Depending on soil contituents compared to institu:

i) Representative: There is no change in constituents but soil's structure, void ratio, water content may be altered.
ii) Non-representative: Soil may be mixed w/ other strata, its constituents may be missing.

Depending on structure & arrangement:

i) Undisturbed: Sample is an exact replica of institu; i.e., all constituents are present in similar structure & arrangement. Though not feasible (stress release)
ii) Disturbed: Any sort of disturbance

Soil sampling:

Block sampling: Best quality
soil surrounding the sample is excavated and collected in a box

Borehole sample: Hole is formed upto sampler depth by any method, then sampler is used to collect

Soil sampler: Tool by which soil is collected

Exploration sampler
Drive sampler
Cone barrel

Drive sampler — Open drive
Piston drive sampler → Stationary
Retracted
Depending on thickness Free moving

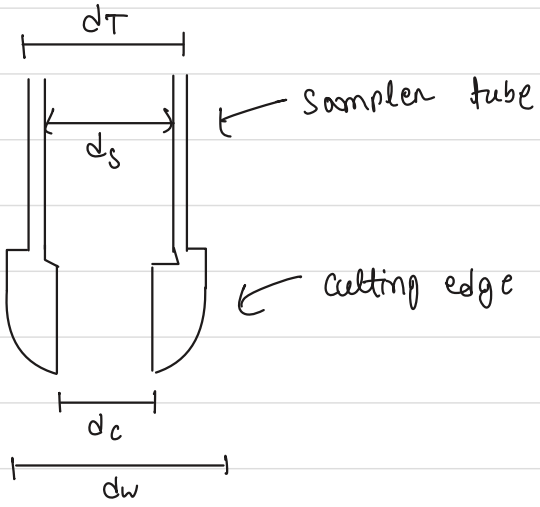
i) Thin wall sampler (Shelby spoon sampler, ST)
ii) Thick " " (Split spoon sampler, SS)
iii) Composite sampler

Sample disturbances

Three stage: Prior sampling (removal of overburden pressure)
During sampling (displacement of soil)
After sampling (during transportation)

Depends on: Cutting edge
Inside wall friction
Non-return valve

Cutting edge:



Inside clearance ratio,

$$C_i = 100 \times \frac{d_s - d_c}{d_c} \rightarrow 1\% - 3\%$$

Outside clea - —

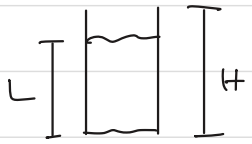
$$C_o = 100 \times \frac{d_w - d_T}{d_T} \rightarrow \geq C_i$$

Area ratio,

$$A_R = 100 \frac{d_w^2 - d_c^2}{d_c^2} \quad \geq 20\% \quad \geq 10\%$$

stiff soft

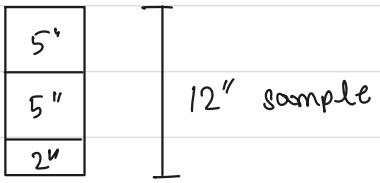
Recovery ratio: $R_R = \frac{L}{H}$ (98% - 96%)



Rock quality designation (RQD)

$$RQD = \frac{\Sigma (\text{portions} \geq 4'')}{\text{Total len}}$$

$$= \frac{5+5}{12} = 83\%$$



Collected rock sample 12"

Found cracks

Excellent	Good	Fair	poor	very poor
90—100	75-90	50-75	25-50	0-24

SPT – Standard Penetration Test

- Drives a split spoon sampler into soil
- Borehole dia — 55 mm to 100 mm
- A 63.5 kg hammer dropped onto a drill rod from 30 in or 750 mm above

N-value — Number of blows req to penetrate 12" (6" + 6") after initial 6"

Boring to be continued until refusal occurs:

i) 50 consequtive blows in any 6"
ii) Total of 100 blows
iii) No significant advancement after 10 successive blows

Correction:

(a) Standardization:

$$N_{60} = \frac{E_H C_B C_S C_R}{60} \times N$$

E_H = Hammer eff
C_B = Borehole dia corr.
C_S = Sampler corr
C_R = Drill rod corr

(B) Not rea anymore

(C) Overburden corr. factor — for sandy soil

$$C_N = 0.77 \log\left(\frac{2000}{\sigma_o'}\right) \leq 2.0; \qquad \sigma_o' \geq 0.25 \text{ kN/m}^2$$

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

SPT & Soil Props:

← reliable

Cohesionless:	0–4	4–10	10–30	30–50	>50	
	v. loose	loose	medium	dense	v. dense	
Cohesive:	0–2	2–4	4–8	8–15	15–30	>30
	v. soft	soft	medium	stiff	v. stiff	hard

↗ not reliable

SPT & consistency of clay:

$$q_u = 12.2 N_{60} \quad (kN/m^3)$$

$$c_u = \frac{q_u}{2}$$

$$\tau = c_u$$

$$q_u = 10 N_{60} \; ; \; PI > 30$$

For Bangladesh

$q_u = 14.3N$ PI 0–35

16.9 N 36–50

17.8 N >50

Defⁿ of angle of internal friction:

$$\phi_0' = 27 + 0.3 N_{60} + 0.00054 N_{60}^2$$ [note N_{60}, not $(N_1)_{60}$]

Dynamic Cone Penetration Test (DCPT)

100 mm → 100 mm → 100 mm → 100 mm

300 mm # of blows

65 kg hammer

Dutch cone Penetration Test (CPT)

SPT-N value

Cone resistance q_c → ↑ → fine
Side friction f_s → ↓ → coarse

$q_c = 0.4 N$

Friction ratio, $R_f = \frac{f_s}{q_c}$ → ↑ → fine, ↓ → sand

R_f → 0.0 – 0.5 → loose gravel
0.5 – 2.0 → sand, gravel
2 – 5 → clay sand mix
>5 → clay, peats

Soil Exploration Report:

(i) Scope of investigation: why, what type of project
(ii) General description of proposed structure: building, stories
(iii) Geologic condition of site: fill soil, natural soil
(iv) Drainage condition: expulsion of water (drainage, dewater)
(v) Details of boring: bore dia, depth
(vi) Description of subsoil condition
(vii) GWT
(viii) Probable fdn alts
(ix) Anticipated construction problems
(x) Limitation of investigation

Boring log:

Trace	few	little	some	mostly
0-5%	5-10%	15-25%	35-45%	>50%

55% sand, 31% silt, 24% clay

sandy soil with some silt & little clay

Soil Exploration Report (Re):

1. Scope of investigation
2. Description of proposed struct
3. Geologic condition of site
4. Drainage facilities
5. Details of boring
6. Subsoil condition
7. GWT
8. Probable fdn alts
9. Potential problems
10. Limitation of investigation

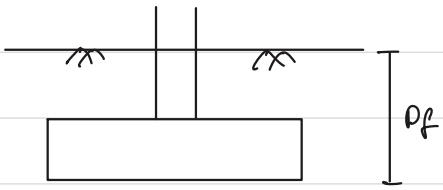
Trace	few	little	some	mostly
0–5	5–10	15–25	30–45	>50

Bearing capacity of Pile Foundation

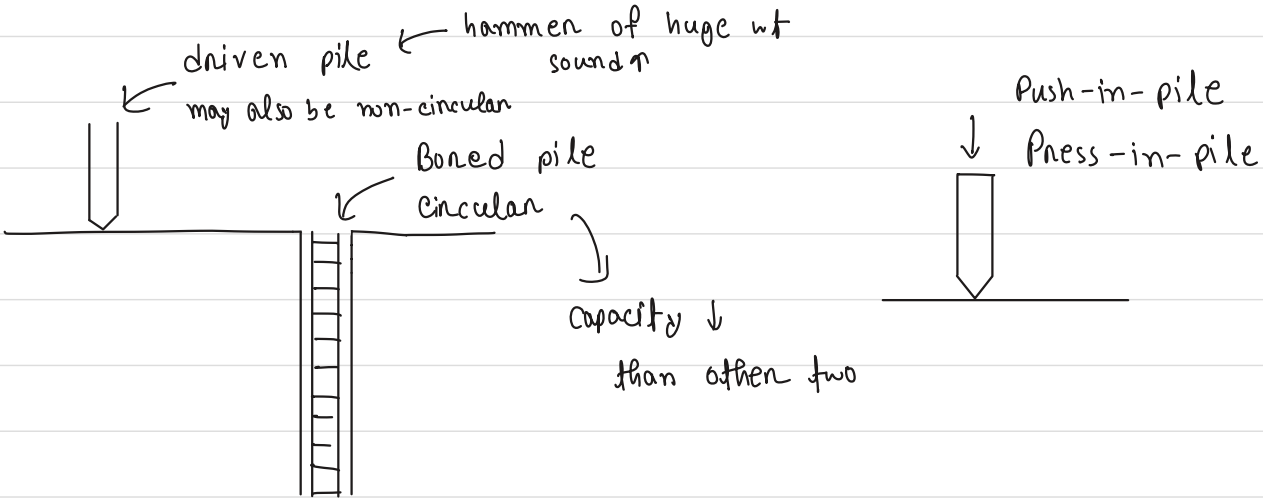
Types of foundation:

Shallow foundation: $D_f < 3m$

$D_f < B$

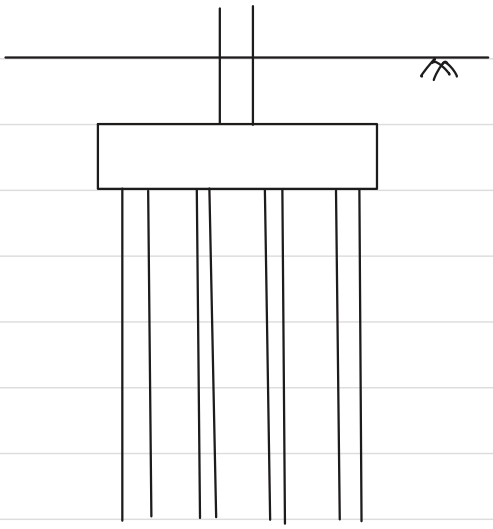


Deep foundation:



Driven pile: impact driven, pre-formed piles

Push-in-pile: hydraulic system using wt of existing struct



Geotechnical resistance of pile:

Static analysis methods
Field method:
Static ld
Dynamic ld
Driving formulae

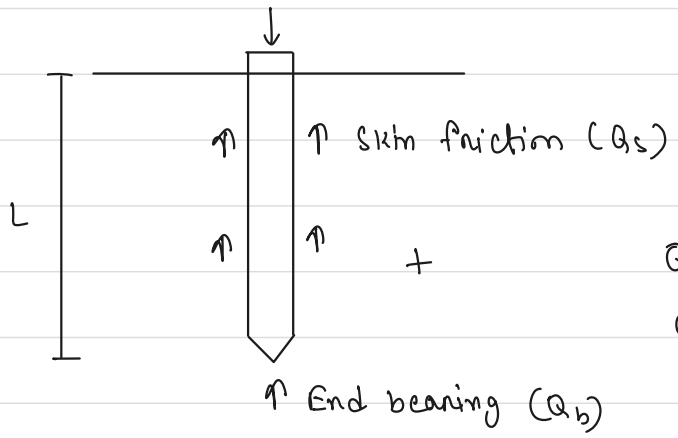
Static analysis method: Single pile:

Load is carried by: Skin friction
End bearing

$$q_{ult} = cN_c + \frac{1}{2} B\gamma N_\gamma + qN_q$$

Driven pile in cohesive soil: α - method

$q_{ult} = cN_c + \frac{1}{2}B\gamma N_\gamma + qN_q$; $\tau = c_u$



$Q_u = Q_s + Q_b$

$Q_s = A_s f_s$; $A_s = 2\pi rL = \pi DL$

$f_s = \alpha c_u$

$Q_b = f_b A_b$; $f_b = q_{ult} = cN_c$

7 to 9

9 for deep

f_s → unit skin friction resistance

→ adhesion between pile & soil

→ $f_s = c_a = \alpha c_u$

→ α → adhesion factor

c_u → soil — soil slip

soil — concrete slip → varies, lowered

steeper soil → more reduce

steeper soil → more disturbance

→ reduced more

Soften

c_u → 20 kPa → less disturbance → α ↑

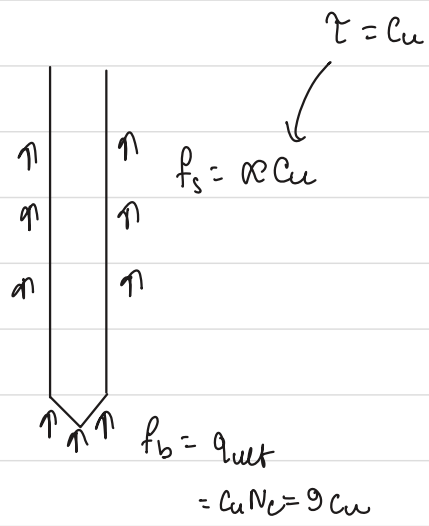
→ 100 kPa → more " → α ↓

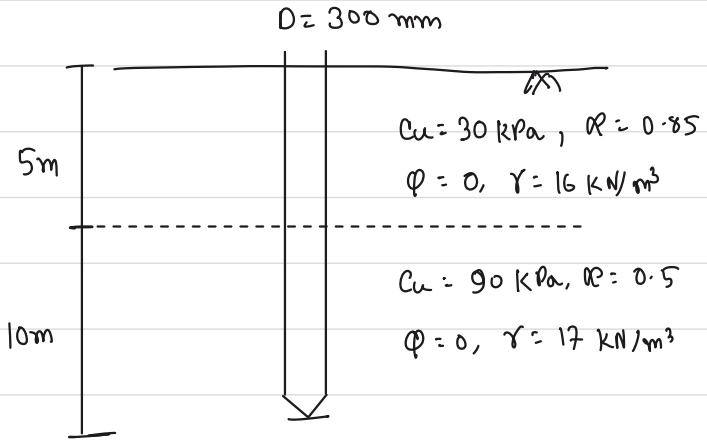
c_u = 20 kPa → α = 0.9

= 100 kPa → α = 0.45

} interpolate

#





$Q_{ult} = Q_s + Q_b$

$Q_s = A_1 f_{s1} + A_2 f_{s2}$

$= (\pi \times 0.3 \times 5)(0.85 \times 30) + (\pi \times 0.3 \times 10)(0.5 \times 90)$ kN

$= 120.2 + 424.1 = 544.3$ kN

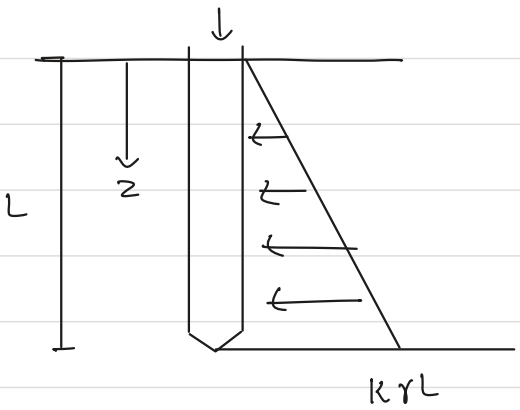
$Q_b = A f_b = A C_u N_c$

$= \pi \frac{0.3^2}{4} \times (90+30) 9 = 76.3$ kN

$\therefore Q_{ult} = 620.6$ kN

Driven pile in cohesionless soil: β method:

$\beta = K \tan\delta$; $\delta = \frac{2}{3}\phi$ or $\frac{3}{4}\phi$



$Q_s = A_s f_s$; $f_s = \sigma \tan\phi$

$= K\gamma z \tan\phi$

$= K\sigma_v \tan\phi$

$= \beta \sigma_{v,avg}$

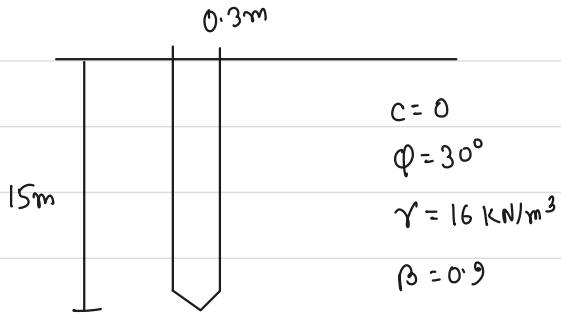
$Q_b = A_b f_b = f_b = q N_q$

$Q_{ult} = Q_s + Q_b$

$Q_s = \beta \sigma_{v,avg} \times A_s$

$= 0.9 \times \frac{0+16}{2} 15 \times (\pi \times 0.3 \times 15)$

$= 1526.8$ kN



$Q_b = (\cancel{cN_c}^{0} + \cancel{\frac{1}{2}B\gamma N_\gamma}^{0} + qN_q) A_b$

$= qN_q \times A_b$

$= 16 \times 15 \times 15 \times \frac{\pi 0.3^2}{4}$ [N_q from graph]

$= 254.5$ kN

$L/D = 50$

$\therefore Q_{ult} = 1781.3$ kN

$Q_a = \frac{Q_{ult}}{3} = 594$ kN

Considering critical depth: After certain depth, pressure no longer inc.

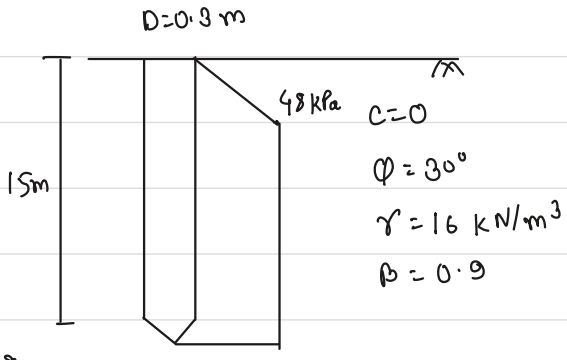
Loose sand : $D_c = 10D$ ← $\phi = 20-30$

med sand : $D_c = 15D$ $\phi = 30-36$

Dense sand : $D_c = 20D$ $\phi = 35+$

$\phi = 30° \rightarrow$ soil is Loose

$\therefore D_c = 10d = 3\,m$



$Q_s = A_{s1} f_{s1} + A_{s2} f_{s2}$

$= \pi D L_1 \times \beta \sigma_{v1,avg} + \pi D L_2 \times \beta \sigma_{v2,avg}$

$= \pi \times 0.3 \times 3 \times 0.9 \times \frac{48}{2} + \pi \times 0.3 \times 12 \times 0.9 \times 48$

$= 61.07 + 488.58\ kN$

$= 549.65\ kN$

$Q_b = A_b f_b = \frac{\pi D^2}{4} \times q N_q$

$= \frac{\pi \times 0.3^2}{4} \times 48 \times 15$

$= 50.89$

$\therefore Q_{ult} = 600\ kN$

c-ϕ soil : λ-method

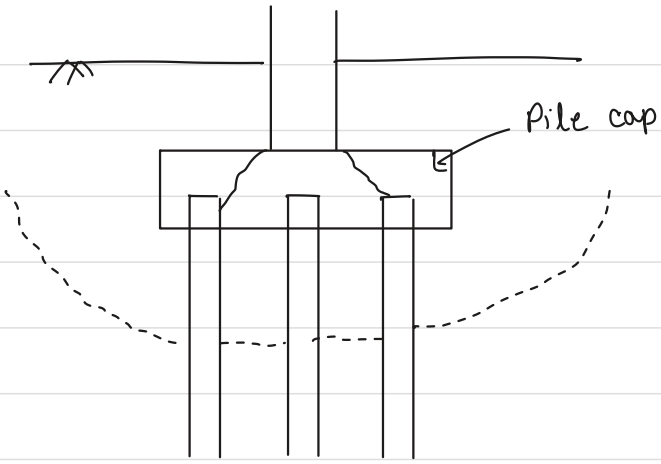
	Skin friction	End bearing
cohesive	αc_u	$c_u N_c = 9 c_u$
cohesionless	$\beta \sigma_{v,avg}$	$q N_q$
c-ϕ soil	$\lambda(\sigma_{v,avg} + 2c_{u,avg})$	$9c_u$ or qN_q or $(9c_u + qN_q)$

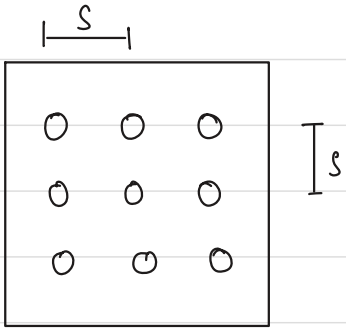
For bored pile: $f_s = 1/3\, f_s$; $f_b = 1/2\, f_b$

Capacity based on SPT value:

	Skin friction (avg N)		End bearing	
Sand	$2\bar{N}$	≤ 60 KPa	400 N	≤ 1000 KN/m^2
Silt	$2.67\bar{N}$		300 N	
Clay	$5\bar{N}$	≤ 75 KPa	45 N	

Group piles:





S → 2d to 6d

→ 3d

Group efficiency:

$$\eta_g = 1 - \left[\frac{(n_1-1)n_2 + (n_2-1)n_1}{90 n_1 n_2} \right] \theta$$

n_1 = # of rows

n_2 = # of cols

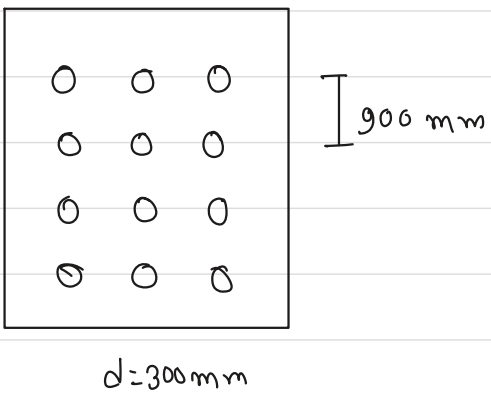
$\theta = \tan^{-1} d/s$

$\theta = \tan^{-1} \frac{d}{s} = 18.43°$; $n_1 = 4$; $n_2 = 3$

$$\eta_g = 1 - \left(\frac{(n_1-1)n_2 + (n_2-1)n_1}{90 n_1 n_2} \right) \theta$$

$$= 1 - \left[\frac{3\times3 + 2\times4}{90\times3\times4} \right] \times 18.43$$

$$= 0.71$$



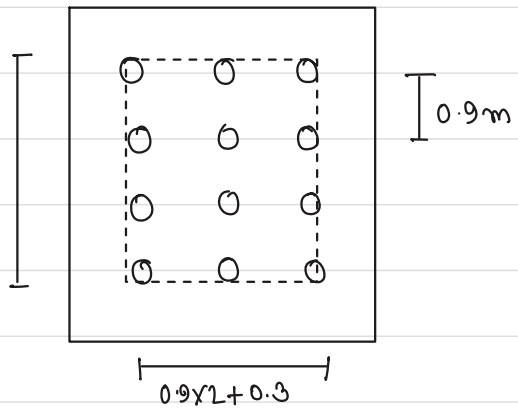
d = 300 mm

For single pile → Q_u

group → $Q_{u,grp} = Q_u \times n_g$

Block method:

Composite pile:



$Q_s = A_s f_s \quad (f_s = \alpha C_u)$

$= [\{(2.1+3)\times 2\} \times 15 \times] [0.7 \times 50]$

$= 5355$ kN

$Q_b = A_b f_b \quad (f_b = cN_c)$

$= (3 \times 2.1)(100 \times 9)$

$= 5670$ kN

$\therefore Q_u = 11025$ kN

L = 15 m $\alpha = 0.7$

$C_u = 50$ kPa $C_{u,end} = 100$ kPa

Single pile:

$Q_s = A_s f_s = \pi DL \times \alpha C_u$

$= \pi \times 0.3 \times 15 \times 0.7 \times 50 = 494.8$ kN

$Q_b = A_b f_b = \pi D^2/4 \times cN_c$

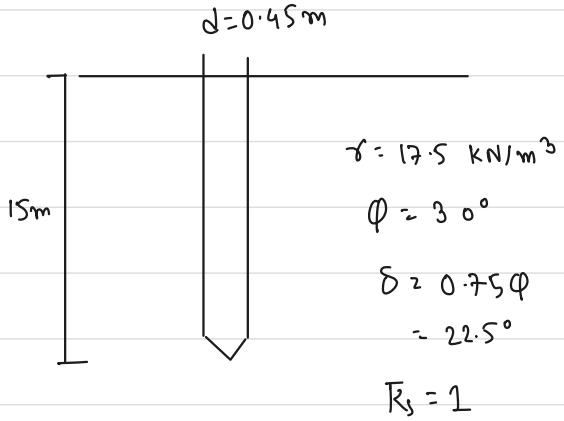
$= \pi \frac{0.3^2}{4} \times 100 \times 9 = 63.6$ kN

$\therefore Q_u = 558.4$ kN

$\therefore Q_{ug} = 558.4 \times (3 \times 4) = 6700$ kN

$\left.\right\}$ eff. $= \frac{6700}{11025} = 0.61$

Ex. 15.1



$\beta = K \tan\delta = 1 \times \tan 22.5°$

$= 0.414$

$Q_s = A_s f_s = \pi D L \times \beta \sigma_{v,avg}$

$= \pi \times 0.45 \times 15 \times 0.414 \times \frac{17.5 \times 15}{2}$

$= 1152$ kN

$L/D = 33.3$

$Q_b = A_b f_b = \pi D^2/4 \times q N_q$

$= \pi \frac{0.45^2}{4} \times 17.5 \times 15 \times 17$

$= 710$ kN

$\therefore Q_u = 1862$ kN

$\therefore Q_{all} = \frac{Q_u}{F_s} = \frac{1862}{2.5} = 745$ kN