MODULE - I

: Soil Exploration & Retaining walls :

Def: Soil Exploration consist's of Determining The profile of The Natural Soil deposit's @ site, taking The Soil Samples and Determinqng The Engg. properties of soil using Laboratory tests as well as In-situ Testing Method's.

Need of Soil Exploration :-

→ it give's The Information to determine The type of foundation Required such as shallow or a deep foundation.

→ Necessary Information with regard's to The Strength & Compressibility charecteristic's of The Sub soil to allow The Design Consultant to Make Recomondation's on The Safe Bearing pressure's or pile Load Capasity.

~~Meth~~

Method's of Soil Exploration

- **Direct Method's**
  - Test-pit's / Trail pit's / Trenches.
- **Semi-direct Method**
  - Boaring
    - Auger Boaring
    - wash Boaring
    - Rotary Boaring
    - Percussion drilling
    - Combination of shell + Auger
- **In-direct-method**
  - Geo-physical method's.

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Test. pit's :-

→ it is The oldest Method of soil Exploration

→ Test pit's give's The detailed visual Examination of The Sub-Surface Meterial In-situ Condition.

→ Thus are The open Type.

→ Disturbed or partially undisturbed samples Can be obtained.

→ Accessible In natural condition of soil.

→ Suitable for Smaller depth's i.e depth's ≤ 3M

→ Cost Increases with depth

→ Greater depth's may also Lead bracing Latteral Support's.

→ G.W.T if encounterd should be Lowerd.

→ Suitable for Small project's were The depth of foundation is Less. or shallow depth's.

Boaring Def:- Drilling bore hole's Into ground to obtain soil / Rock Sample's from Specified or unknown depth's is called.

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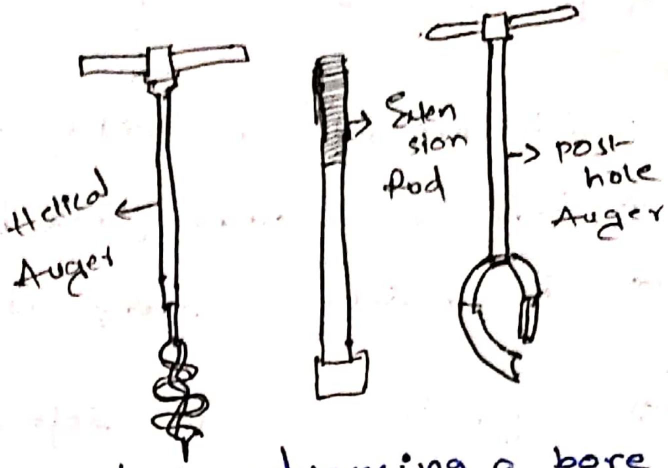
Auger Boring:



→ Auger's are of Two type's.

i) Hand operated Auger's

ii) power Driven Auger's.

→ Auger's are The devices That useful for advancing a bore hole In to The ground.

→ Hand operated Auger's are used for relatively small depth's. i.e 3 to 5 m

→ power Driven Auger's are used for greater depth's i.e upto 60 to 70 m [for continuous flight-auger's].

→ Auger's are Suitable for All types of soil's which are presenting above W.T

→ These are suitable only below W.T in clayey soil's.

Process.

↓

The soil Auger is Advanced by rotating it while pressing in to The soil.

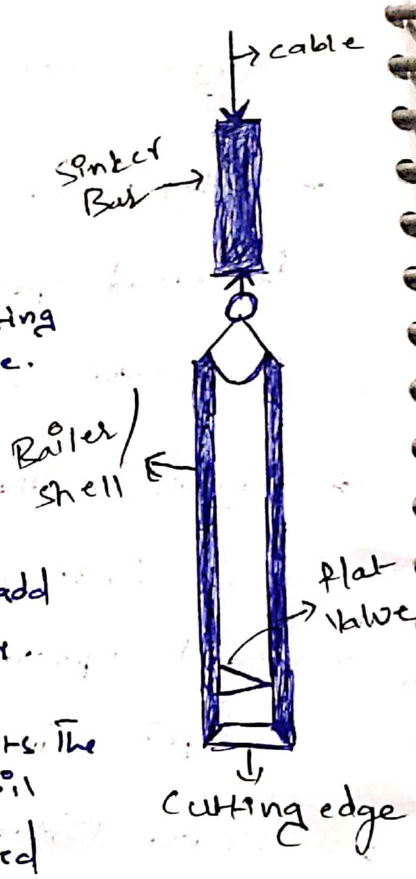
↓

As soon as The Auger get's filled with soil, it is taken out and soil sample is collected.

↓

The Soil Sample's obtained from This type of borings are highly disturbed.

Auger & Shell Boring :-



- → This Method was widely used in India.
- → it is also known as Sand bailer
- → it consists of Heavy duty pipe with cutting Edge.
- → Auger is used for soft clay soils & shell is used for stiff clay.
- → Some time's Sinker bars are added to add weight to The bailer.
- → Raising & Dropping of a shell in a hole cuts The soil
- → Now soil is pushed in to tube/shell & Emptied when full.
- → Shell is used when Auger boaring becomes difficult

Wash Boring :-

- → it was commonly used for Boring in difficult soil's.
- → it was used when the Exploration's are to be done below ~~GWT~~ .W.T where auger is un-suitable.
- → A casing pipe is pushed in & driven with a drop weight
- → Soil particles/ sample is a very disturbed sample which was not suitable for Evaluation of ~~pro~~ soil properties
- → The change in color of water indicates The change of soil strata.

Process

A casting pipe is pushed in and driven with a drop weight

hallow drill bit is screwed to a hollow, drill rod connected to a rope passing over a pulley and supported by a tripod.

water jet under pressure is forced through the rod & the bit into the hole which loosens the soil.

The soil water suspension forced upword is led to a settling tank where the soil particles settle while the water over flow's in to a sumps.

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Rotary Drilling:-

→ it was Normally used in Rocky strata.

→ it was very fast Method & Rock cores also obtained by suitable drill bits.

Process

↓

A Drill bit, is fixed to The lower end of a drill Rod & it is rotated by power

↓

Drilling fluid / bentonite slurry is forced under pressure Through The Drill rod & it comes up bringing The cuttings to surface.

↓

when The soil samples are read, The Drill rod raised & Drill bit is replaced by a sampler.

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Percussion Drilling:-

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Penetration Tests:-

(or) : Field Method's to Determine Bearing capacity:-

→ Here we have Mainly 3 type's of Test's.

i) plate Load Test

ii) Standard penetration Test

iii) Static cone penetration Test.

i) plate- Load Test:-

→ These tests are used to determine The Bearing capacities of soil's in In situ condition.

→ In plate -Load test we are taking a Steel plate of 30 cm or 45 cm or 60 cm or 90 cm.

→ Shape of Steel plate will depend's on shape of footing we design. Ex:- if we are going to design Circular footing Take circular plate. Like wise Rectangular or Square ---

→ This plate (Baseplate) should be kept in a Excavated Pit/trench.

→ Size of trench Should be minimum of 150 cm. width

→ let B_p is width of Base plate.

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→ Now width of Trench ≮ 5 Bp.

→ Depth of Trench will be kept Equal to The Depth of footing wich we have to be designed.

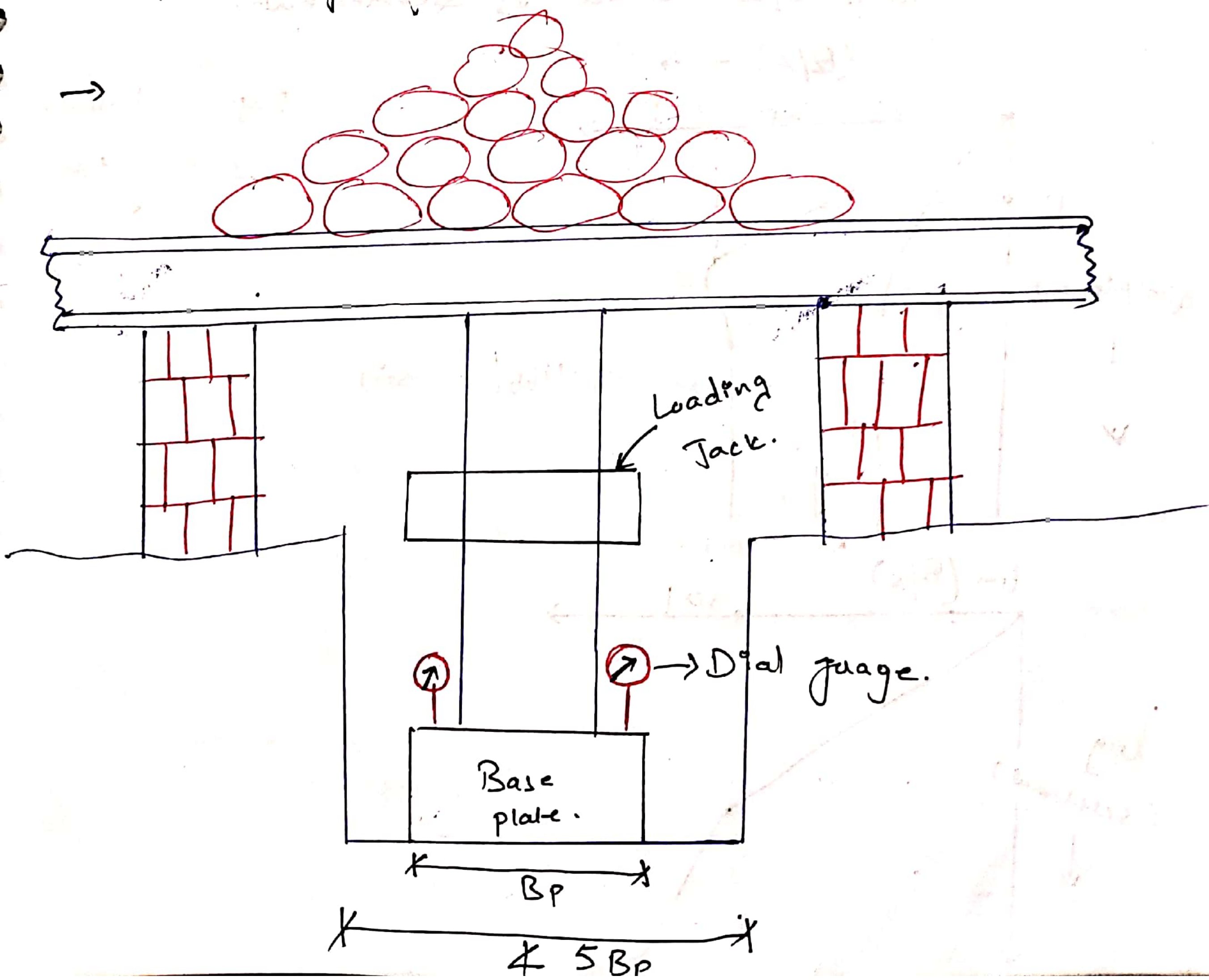
→ on The base plate a loading Jack was arranged & 3 dial gauges are placed to note down The Settlement

→ 3 dial guages are placed @ 120° from Each other.

→ we can use The Single dial guage to take Settlement readings but in case of Differential Settlement single Dial guge reading will not gives accurate results.

→ so we can arrange 3 dial guages ~~reading~~ & Take The average of 3 readings as Settlement.

→



→ to start The test we need to apply The load on girder by Manually or by using Loading jack.

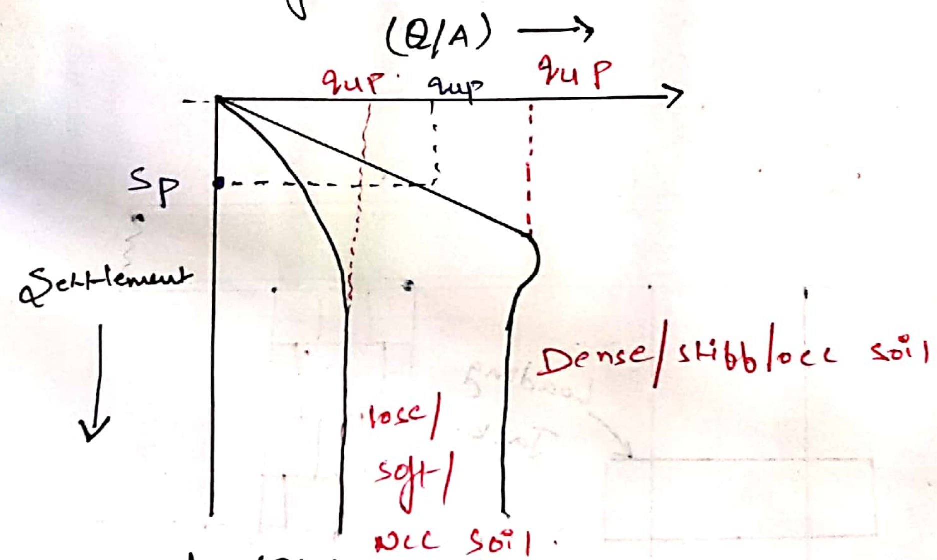
→ when we started loading we can see The deflection's in dial guage.

→ as load ↑ Dial guage reading ↑.

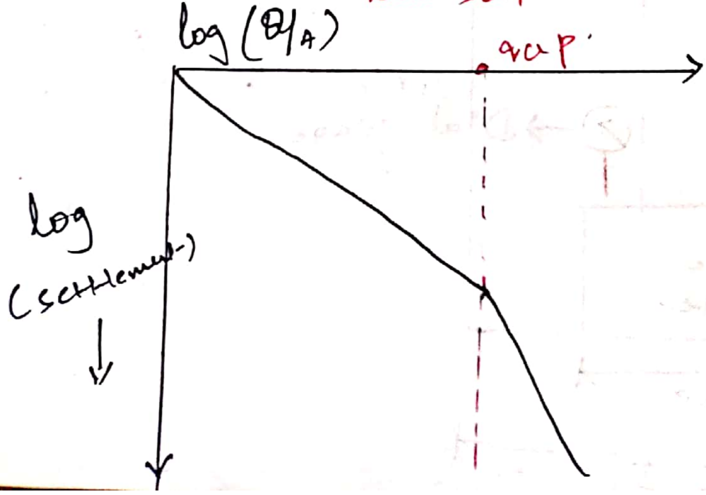
→ at certain point on ↑ of load we can see The large deflection in dialguge

→ it will indicates The soil below plate wase Suddenly Settled i.e Soil was failed @ That loading.

→ we will Stop loading at That point and we draw a graph b/w Load vs Settlement.



qup = ultimate point of failure



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→ for OCC/ stiff / Dense soil q_{up} will be clear. but in case of NCC / lose / soft soil getting q_{up} is difficult because of the progressive settlement.

→ so to get q_{up} in NCC / lose / soft soil we use log-log scale graph

Case (I) for clay soil:- In case of clay soil U.B.C does not depend upon the size of footing.

$$\boxed{q_{uf} = q_{up}} \quad kN/m^2 \rightarrow (1)$$

q_{uf} = U.B.C of footing.
q_{up} = U.B.C of plate.

Case II : in case of cohesionless / or sandy soil is UBC depend's on size of footing. i.e ($q_{uf} \neq q_{up}$)

Assume in linear relationship

$$\frac{q_{uf}}{q_{up}} = \frac{B_f}{B_p}$$

$$\boxed{q_{uf} = \frac{B_f \times q_{up}}{B_p}} \quad kN/m^2 \rightarrow (2)$$

B_f = width of footing
B_p = width of plate.

Housel's Analysis:-

According to Housel the load carrying capacity is influenced by area and perimeter of the footing & properties of the soil.

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→ if Q, A, P are ultimate load, Area of plate & peremeter of the plate respectively. Then as per Housel's.

$$\boxed{Q = Am + Pn}$$

Q = ultimate load
A = Area of plate
P = perimeter
m, n = properties of soil.

for I^st Base plate

$$Q_1 = A_1 m + P_1 n \rightarrow (A)$$

for 2nd Base plate:-

$$Q_2 = A_2 m + P_2 n. \rightarrow (B)$$

Here we have two Eqn's & two unknow's if we solve both we get the values of m & n.

Now Assume A_f = Area of footing

P_f = perimeter of footing.

$$\boxed{Q_f = A_f m + P_f n.} \quad kN.$$

U.B.C Based on Settlement Criteria:-

Is code have suggested permissible values of settlement

for clay => 40mm

for sand => 25mm.

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$$\frac{S_f}{S_p} = \left[\frac{B_f (B_p + 0.3)}{B_p (B_f + 0.3)}\right]^2 \quad \text{(for sandy soils.)}$$

$$\frac{S_f}{S_p} = \left[\frac{B_f}{B_p}\right]^{n+1} \quad \text{[for } c-\phi \text{ soils} \quad \text{Her } n = 0.5]$$

$$\frac{S_f}{S_p} = \left[\frac{B_f}{B_p}\right] \quad \text{[for clay soils].}$$

Here

S_f = Settlement of footing

S_p = Settlement of plate.

in 'm' strictly.

B_f = width of footing (in m)

B_p = width of plate (in m)

from This Method

→ for the given permissible values of Settlement of footing (S_f), Settlement of plate (S_p) can be calculated.

→ Using load Settlement curve, for the values of S_p, q_{up} can be calculated.

→ Now using q_{up}; q_{uf} can be calculated by using Eqn ① & ② for given soils.

Note:- No. further F.O.S is reqd because calculated value of q_{up} has been Estimated considering permissible settlement of footings. q_{up} can be called safe/allowable pressure (in kN/m²)

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Limitation's:-

i) The values of Bearing capacities we get from This test will Show's The ~~Same~~ charecteristics of small depth soil but in actual condition of footing's load distribution will be large.

it will Show's The maximum twice of ~~of~~ width of plate width of soil charecteristics will be shown.

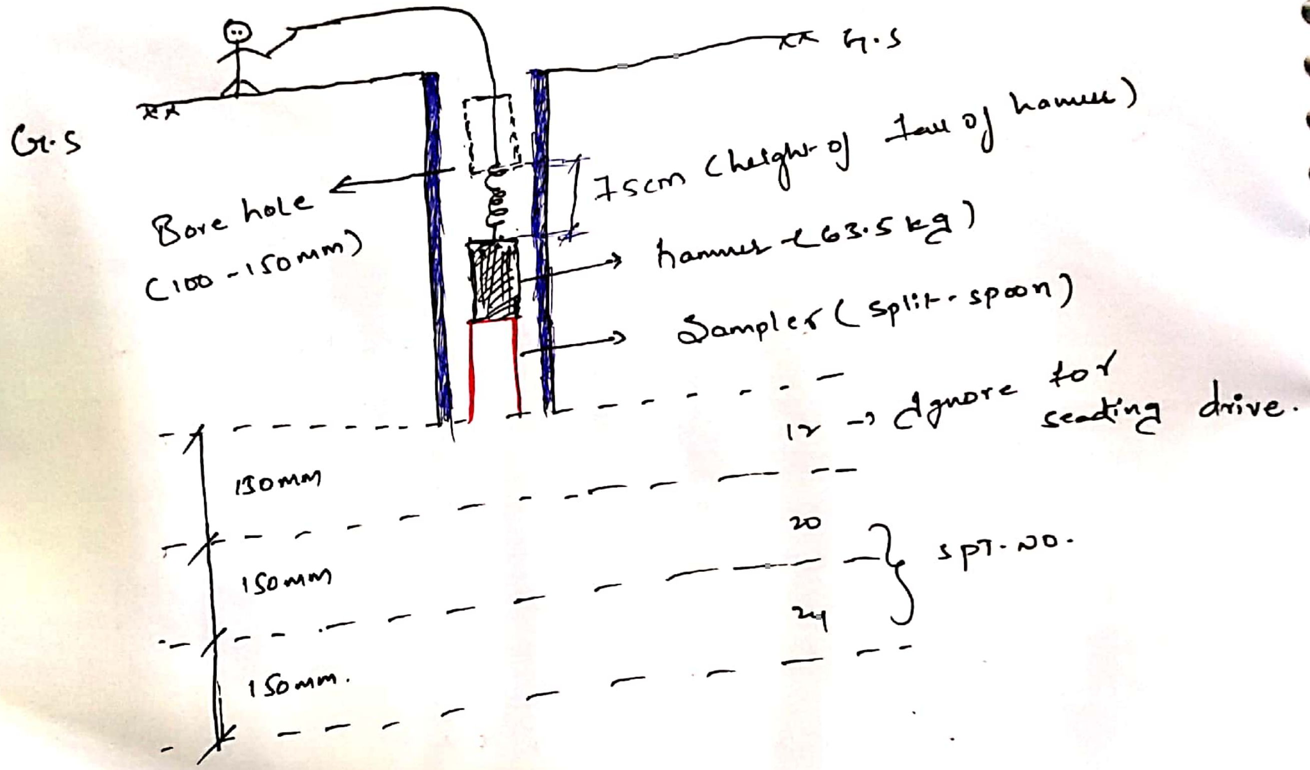
i.e if we take 45 cm plate Then It will show's Maximum 90 cm depth charecteristics.

ii) for clay soil's q_{uf} is $= q_{up}$.

but in clay's Settlement takes ∞ time (unknown)

But The test will be done in finate time.

Standard Penetration Test [SPT - test] [IS 2131 - 1981]



→ This test is performed in clean Bore hole of size 100 to 150mm

→ IS 2131 - 1981 is used for this test.

→ In bore hole's, casing and drilling mud can be used when drilling is to be done in soft clay or loose sand

→ This test is ~~perm~~ performed @

i) at Every change in Strata
ii) @ interval's not more than 1.5m

} whichever is less.

→ The Sampler is penetrated ~~top~~ at the bottom of the bore hole by dynamic Mechanism of hammer

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→ The weight of hammer & height of free fall are 63.5 kg and 75 cm Respectively

→ The split-spoon sampler is first allowed to sink by it's own self weight Then no. of blow's are applied.

→ This Test is performed in 3 stages of 150 mm each.

→ The No. of blows Reqd in first 150 mm penetration of Sampler are Ignored for Seating drive.

→ The total No. of blow's Required in last 300 mm penetration of sampler is given as SPT value or SPT Number.

→ Sometime's Entire Sampler Sink's Into The soil due to it's Self weight only. In that case no hammer blow's are reqd SPT values can be given as Zero.

→ The observed value of SPT from field Requires some Correction's.

(i) Over burden Correction

(ii) dilatancy Correction / water table Correction / fine Correction.

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i) Overburden-Correction:-

→ This Correction is mainly used in cohesionless soils or granular soils.

→ it is observed That on ↑ depth / Effective over burden pressure Confining pressure ↑.

→ Due to higher Confining pressure at The larger depth, SPT Number is over-Estimated.

→ But when Confining pressure is less in loose soil or @ The smaller depth's Then SPT-value is under-Estimated.

→ So to magnify The SPT Number over burden Correction is used.

$$N_1 = N_0 \left[\frac{350}{70 + \sigma'}\right]$$

N_1 = Corrected value after overburden Correction.

N_0 = observed value of SPT

σ' = over burden pressure at The test level.

Here ($\sigma' \ngtr 280\ kN/m^2$).

ij $\sigma' \geq 280\ kN/m^2$, Then this Correction is Not used.

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ii) dilatancy / water Table / fine correction:-

→ this correction is applied in fine sand and silt when water table is present.

→ This Correction Shall be applied when water table is "at" or "above" The test level. if w.T is below The test level Thus This correction not Reqd.

→ due to The dynamic action of hammer pore pressure resistance is developed which causes The SPT number to be over Estimated. There for to reduce The SPT value This Correction is Reqd.

$$\boxed{N_2 = 15 + \frac{1}{2}(N_1 - 15)}$$

$(N_2 > 15)$ @ always

N_2 = final Corrected value after w.T correction

N_1 = Corrected value after over burden Correction.

→ if $N_1 < 15$ Then This Correction is not Reqd.

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Cone- penetration -Test:- [CPT] →

CPT is devided in to two type's

CPT → Dynamic Cone penetration Test
CPT → Standard Cone penetration Test

Static cone penetration Test:-

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Rankine's Earth pressure Theory:-

Assumption's:-

① ⇒ Soil is Homogeneous & Isotropic.

Homogeneous → Soil properties at every point is same i.e They should not vary with location or with space.

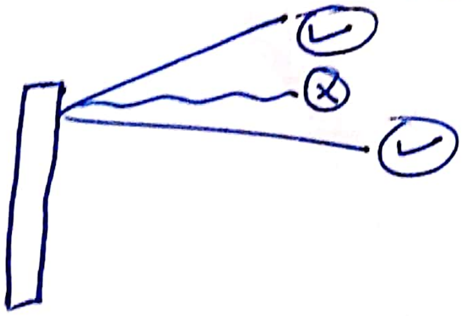
Isotropic → Prop. of soil is same in all-direction's i.e soil prop. does not change with direction

② ⇒ Soil is cohesion less.

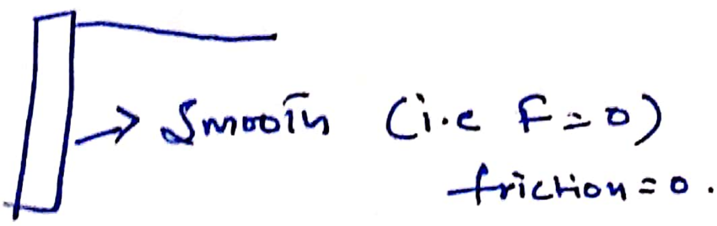
i.e $c = 0$ (sandy).

so we can say That R.E.P.T is applicable only for cohesion less soil's.

③ ⇒ Back fill surface near Retaining structure should be plane i.e Either horizontal or Inclined.



④ ⇒ face of The R.W in contact with The Back fill should be considered as smooth



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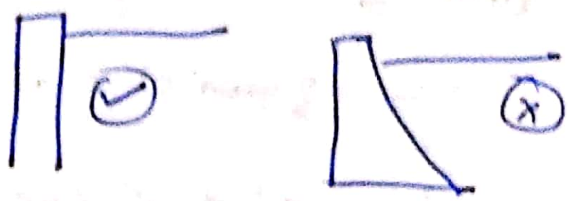
(5) The face of wall in contact with Back fill
⇒ is vertical only.



(6)
⇒ The wall yield's about its base such that the max values of c & ϕ are Mobilised.

[i.e. if we study the failure of wall then we have to calculate failure @ base. it may slide, overturn...]

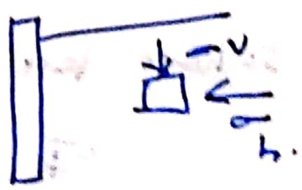
@ failure $c_m, \phi_m = \phi_u, c_u$.

(7)
⇒ Elimental failure is consider.

[i.e. whenever we will calculate soil failure then we will calculate failure of each soil Element]

Application of Rankine's Earth pressure theory :-

Active state :-



→ Let us take a small Element of soil & σ_v', σ_h' are Vertical & Horizontal forces acting on it.

→ as $\sigma_h' < \sigma_v'$ then

σ_h' will be considerd as Minor principle stress

σ_v' " " " " Major " "

so $\sigma_v' = \sigma_1$

$\sigma_h' = \sigma_3$

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→ from Shear strength Theory

$$\sigma_1 = \sigma_3 \tan^2\theta + 2\overset{\to 0}{\cancel{c}}\tan\phi$$

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

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

$$\boxed{\sigma_v' = \sigma_h' \times (ka)}$$ → known eqn

$$ka = \frac{\sigma_h'}{\sigma_v'} = \frac{1}{\tan^2(45+\phi/2)}$$

$$Ka = \boxed{\frac{\sigma_h'}{\sigma_v'} = \frac{1-\sin\phi}{1+\sin\phi}}$$

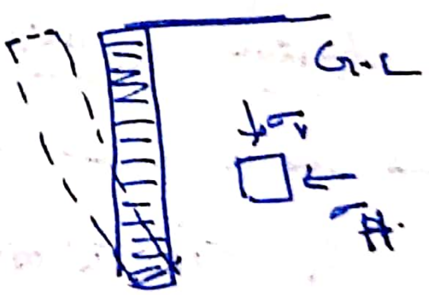
$\sigma_h = ka \times \sigma_v$ here $ka = \frac{1-\sin\phi}{1+\sin\phi}$

so here $ka < 1$ so

$\sigma_H < \sigma_v$

so σ_H = Minor principle stress

σ_v = Major principle stress.



above eqn came from origional eqn

$$\sigma_3 = \sigma_1\left[\frac{1-\sin\phi}{1+\sin\phi}\right] - 2c\sqrt{\frac{1-\sin\phi}{1+\sin\phi}}$$

$$\boxed{\sigma_H = ka\,\sigma_v - 2c\sqrt{ka}}$$ if $c=0$ then $$\boxed{\sigma_H = ka\times\sigma_v}$$

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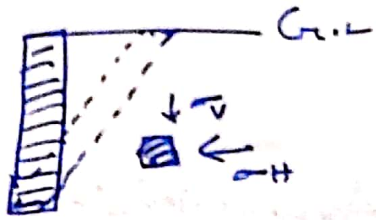
: Passive case:-



$\sigma_H = K_p \times \sigma_V$

$K_p = \frac{\sigma_H}{\sigma_V}$

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

$K_p > 1$

So $\sigma_H > \sigma_V$

$\sigma_H = \sigma_1, \ \sigma_V = \sigma_3.$

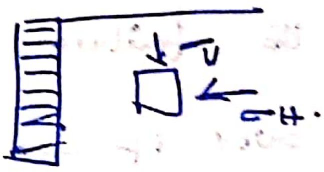
Original Eqn

$\sigma_1 = \sigma_3 \left[\frac{1+\sin\phi}{1-\sin\phi}\right] + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$

$\sigma_H = K_p \times \sigma_V + 2c\sqrt{K_p}.$

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

@ Rest Condition:-



$\boxed{\sigma_H = K_0 \times \sigma_V}$

$\boxed{K_0 = 1 - \sin\phi.}$

Columb's Earth pressure Theory (or) " Wedge Theory

Assumptions:-

1. Soil is Homogeneous, Isotropic, Infinite, plastic, Dry, Cohesionless.

2. The face of wall in contact with Back fill is vertical or Inclined & it is Rough.

3. The failure wedge act as a Rigid Body & stress over it are uniformly Distributed.

4. The failure is Essentially 2-D & Rupture Surface is planer & passes through The Heel of The wall.

5. The location & Direction of resultant Thrust B/w wall & soil is known The point of Application is Taken @ lower third point of The wall By Assuming Triangular Distribution of Earth pressure.

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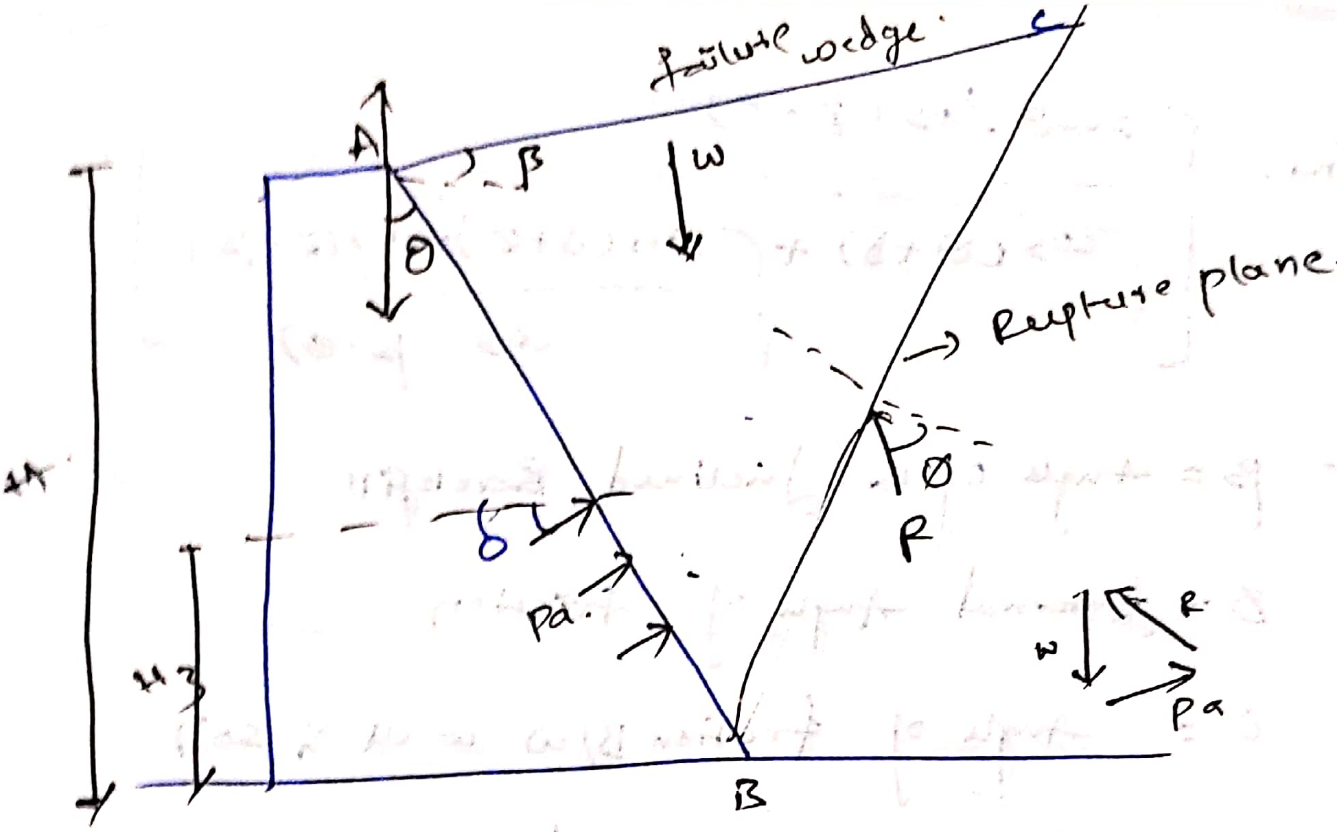

(6) wedge failure is Considered which is under Equilibrium of 3 force.

i) Self wt. of wedge ABC (W) Acting vertically downword direction.

ii) Resultant Reaction R, acts @ downword angle of ø with Normal to Slip plane.

iii) Resultant Reaction 'Pa' Between The wall & soil act @ a downword Angle of δ with Normal of The wall.

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

Here β = Angle of The Inclined Backfill

ϕ = Internal Angle of friction

δ = Angle of friction B/W wall & soil

(generally Assumed as $\frac{2}{3}\phi$)

θ = Angle made by Vertical with Inclined face.

Note: if wall is verticle ($\theta = 0^\circ$), Horizontal Backfill ($\beta = 0$), if wall is Smooth ($\delta = 0$)

Then

$$\boxed{(K_a)_{columbs} = (K_a)_{Rankine}}$$

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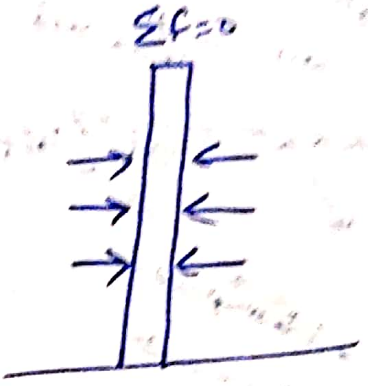
Earth pressure @ Condition:-

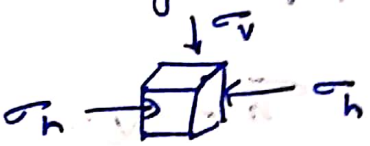
Σf = Net-force.

if $\Sigma f = 0$ wall will not move.



$$K_0 = \frac{\sigma_h}{\sigma_v}$$

So net change in vol of soil = 0.



$\left(\frac{\Delta v}{V_0}\right) = 0$ i.e volmetric strain = 0.

So from som w.r.T

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

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

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

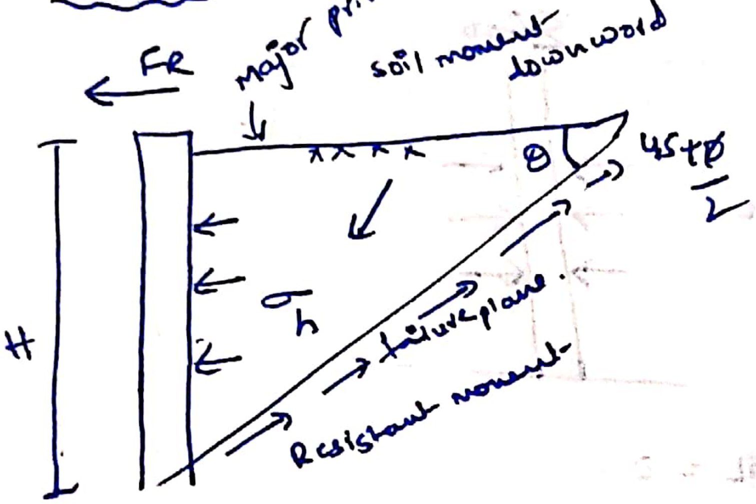
μ = poison's Ratio.

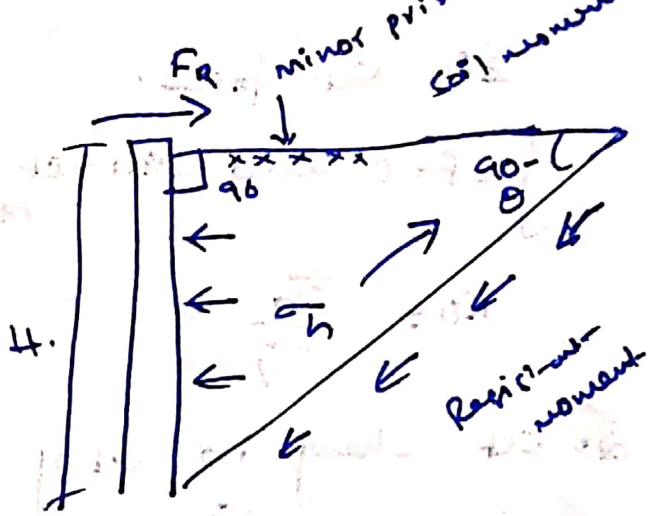
for sandy soil $\boxed{K_0 = 1 - \sin\phi}$

for o.c. clay $\boxed{(K_0)_{oc.clay} = (K_0)_{Nc.clay} \times \sqrt{OCR}}$.

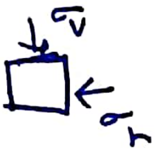
active

passive





$\sigma_h < \sigma_v$



$\sigma_h' = \text{minor} - \sigma_3$

$\sigma_v = \text{Major} = \sigma_1$

from shear strength

Angle B/w failure plane & major principle plane.

$$\theta = \frac{\pi}{4} + \phi/2$$

$\text{Tan}\theta = \frac{H}{La}$

$La = H/\text{Tan}\theta$

$La = H \cot\theta$.

$\sigma_h > \sigma_v$.

$\sigma_h = \text{major} = \sigma_1$

$\sigma_v = \text{minor} = \sigma_3$.

$90 - \theta$

$90 - (45 + \frac{\phi}{2})$

$= 45 - \frac{\phi}{2}$

$Lb = H \cot(90-\theta)$

$= H \cot(45 - \phi/2)$

$\boxed{Lp > La}$

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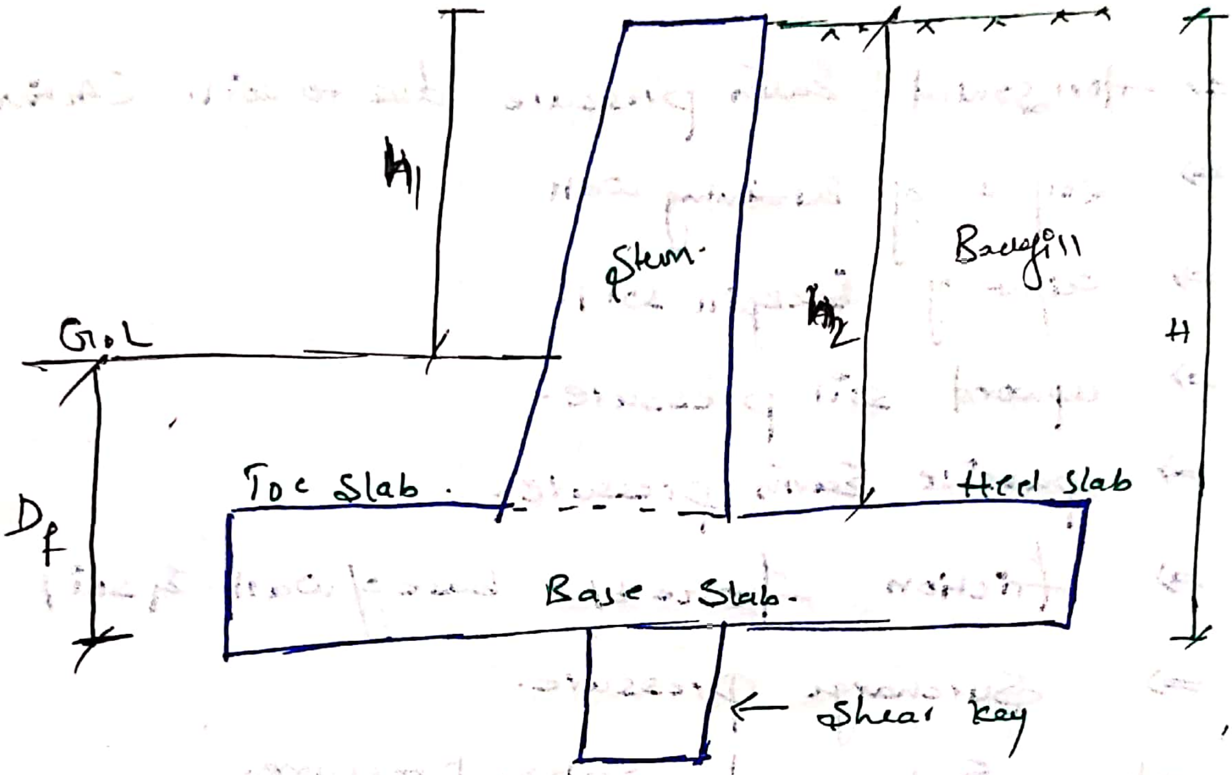
Stability checks for R.W :-



⇒ Stem can be Rectangular or Trepizoidal.

H = Total height of R.W.

h_2 = Height of Stem above & below G.L

h_1 = " " R.W above G.L

D_f = D_f = Depth of foundation from below G.L to End of Base Slab.

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:- forces acting on Retaining wall :-

=> Horizontal Earth pressure due to soil. (Active E.P)

=> Self wt of Retaining wall

=> weight of backfill soil.

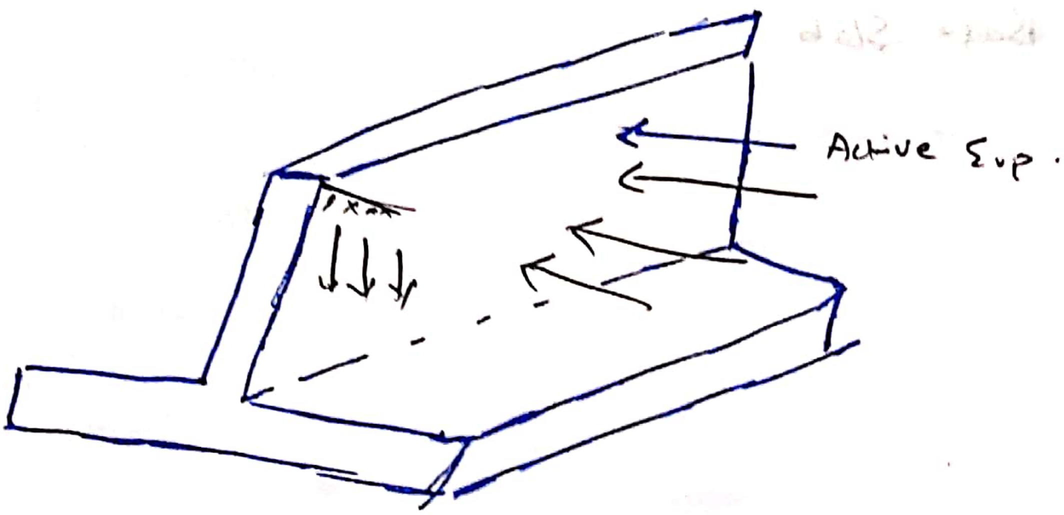
=> upword soil pressure.

=> passive Earth pressure.

=> friction force b/w base of wall & soil

=> Surcharge Pressure.

=> Submerged water pressure.



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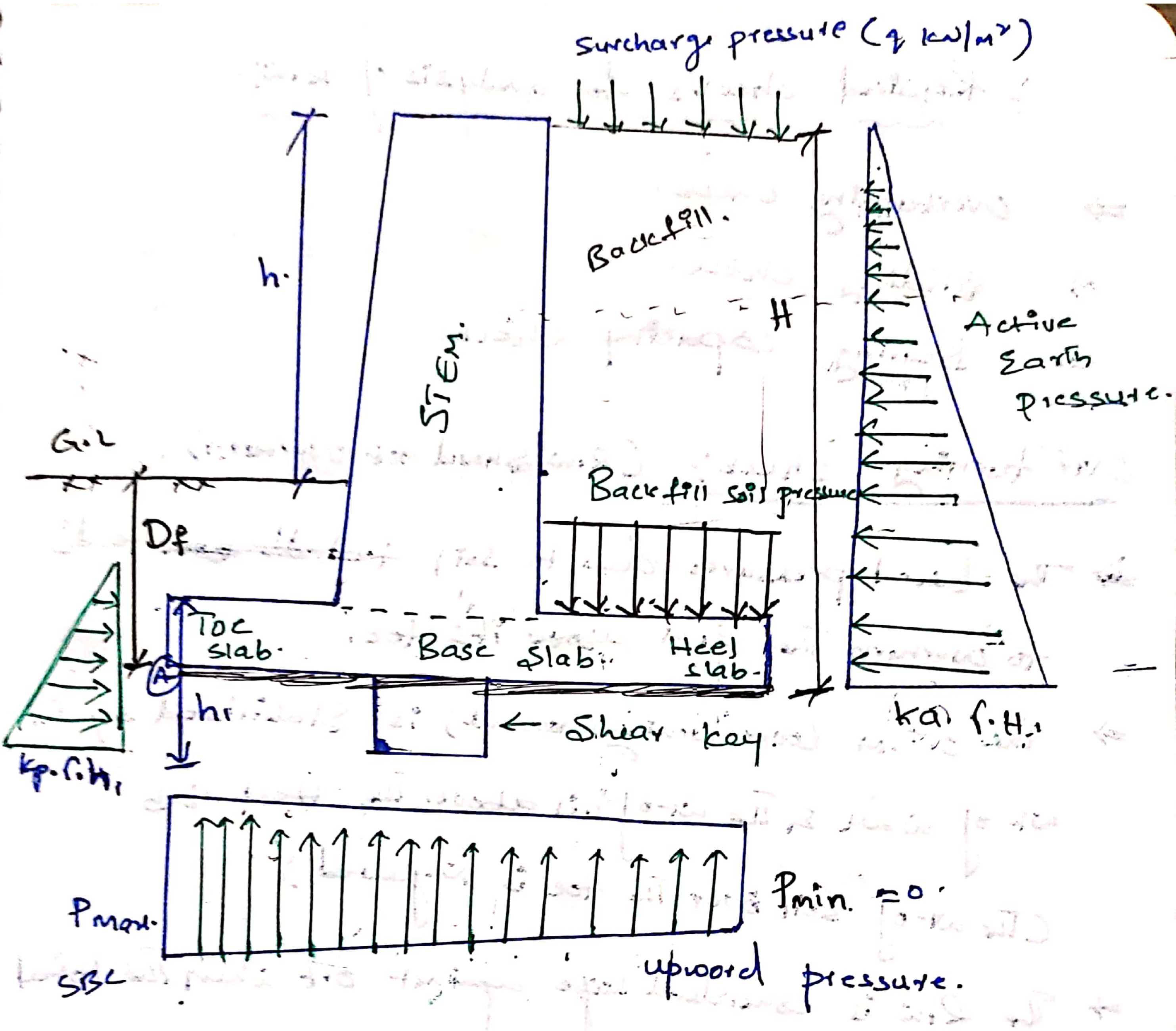


$$K_a = \frac{1-\sin\phi}{1+\sin\phi} \qquad K_p = \frac{1+\sin\phi}{1-\sin\phi}$$

ϕ = Angle of Internal friction of soil.

As per formula $\boxed{K_a = \frac{1}{K_p}}$

Required check's for analysis of R.W:-

⇒ Overturning check.

⇒ Sliding check.

⇒ Bearing capacity check.

Over turning check:- (R.W should not overturn).

⇒ The lateral pressures due to soil ~~tends to~~ tend's to overturn the R.W above it's toe.

⇒ The O.T.M (over turning moment) is stabilised by the wt of wall & the wt of soil above the Heel slab (The wt of soil over the toe is neglected).

⇒ The R.W is considerd safe against o.t when the total stabilizing or Resisting moment is atleast 100% greater than the o.t.M.

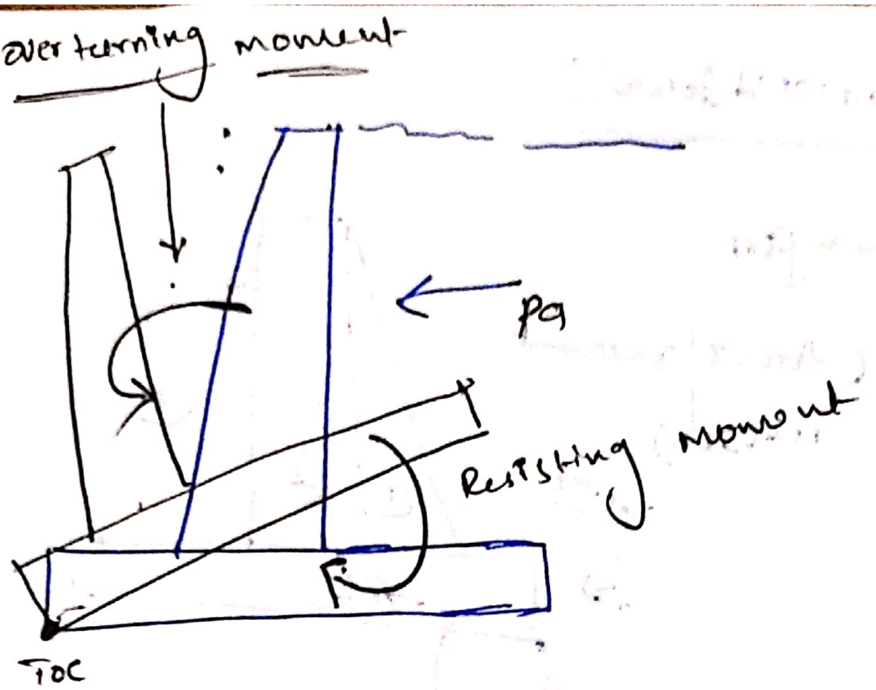
Overturning check:-

$$\text{Fos of overturning} = \frac{\text{Resisting moment}}{\text{Overturning moment}}$$

Resisting moment = Moment due to vertical forces (down wards)

Overturning moment = Moment due to Hz forces.

Fos overturning $>= 1.5$



Horizontal forces: (Lateral forces):

Active Earth pressure (pa) = area of active pressure triangle

$= \frac{1}{2} \times (ka \cdot \rho \cdot H) \times H$

$= \frac{1}{2} \times ka \cdot \rho \cdot H^2$

Moment of point A = over turning moment Mo (Anticlock wise)

Moment = force x Ler dist

$= \text{force} \times \text{Ler dist}$

$\Rightarrow pa \times H/3$

$\Rightarrow \frac{1}{2} \times ka \times \rho \cdot H^2 \times \frac{H}{3}$

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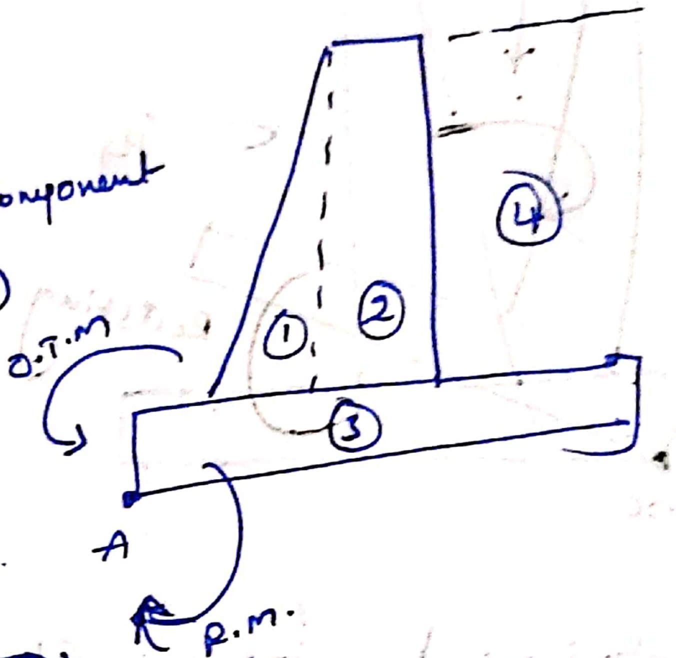
Verticle forces : (down word forces):-

i) pressure due to Back fill

ii) Self wt of R.W. (Area of component 1+2+3+4)



Self wt of R.W = Total wt of diff components of R.W

Downword force due to Back fill soil = total wt of back fill soil

Moment @ point A = Resisting moment M_R (clockwise)

= Force × ⊥er dist

ΣV = total vertical forces

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Sliding Check:-

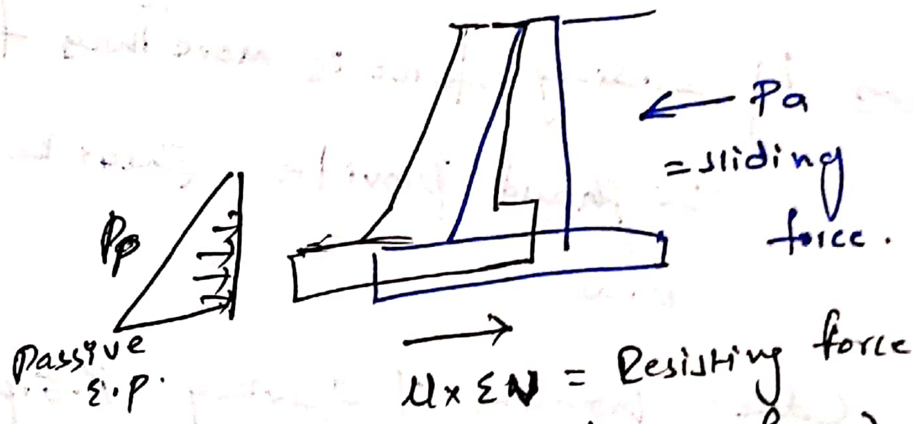
$$\text{FOS of Sliding} = \frac{\text{Resisting force}}{\text{sliding force}}$$

$$\frac{(\mu \Sigma V + P_p)}{P_{ah}} >= 1.55$$

Sliding force = P_a = horizontal Component

Resisting force = $\mu \times \Sigma V + P_p$.

P_p = passive force.



$\mu \times \Sigma N$ = Resisting force (friction force).

μ = Co-eff of friction

ΣV = Sum of Total vertical force.

Sliding Check:-

=) The Lateral E.p on stem tries to slide the R.W away from Back fill soil such force Exert on R.W considerd as sliding force.

=) Resisting lateral force is acting from opposite directiy such resisting force is considerd as "frictional force"

=) Eventually sliding force can be Resicted by friction force which is generated b/w Base of wall & soil

=) if sliding force is more than frictional force, we should provide shear key @ the bottom of R.W

(due to providing of shearkey P.E.p ↑)

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* **Bearing Capacity Check:-**

=> Soil pressure acts on the Bottom of The wall from downword to upword

=> it varies linearly with more pressure on toe side & less pressure on heel side.

=> These upword soil presure Should not be more than SBC of soil & It should not less than zero.

=> To satisfy This Criteria, Middle Third rule should be Satisfy....

=> As per middle Third rule, The Resultant forces of the forces from RW, it should be fall within Middle Third portion

=> if we cant satisfy middle Third rule, should change size of R.W.

=> Eventually due to upword pressure of soil, the Base of wall Should be in Compression

$$P_{max} = \frac{\Sigma V}{A} + \frac{M}{Z} < SBC$$

$$P_{min} = \frac{\Sigma V}{A} - \frac{M}{Z} >= 0.$$

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Here $M = \Sigma V \cdot e$.

ΣV = total Verticle forces

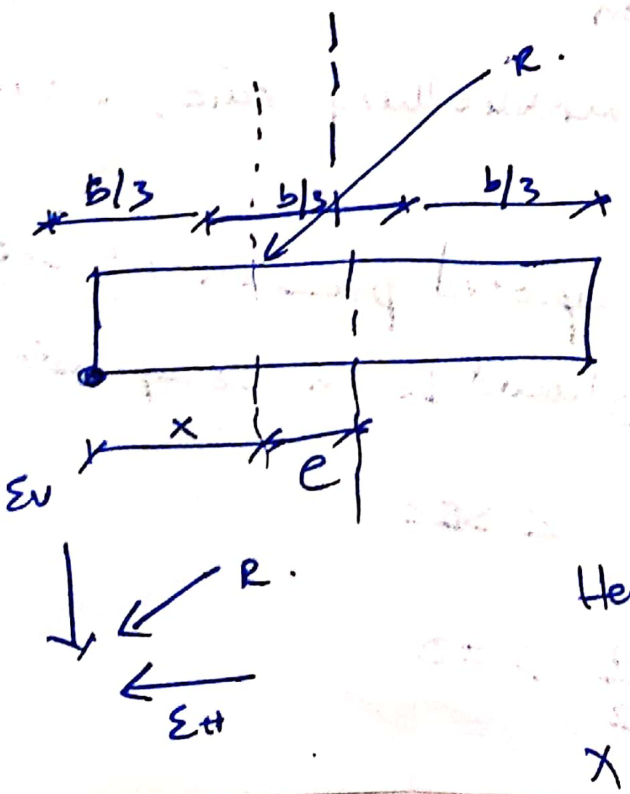
e = Eccentricity of Resultant forces from Toe.

A = Base Area of retaining wall.

Middle Third rule:-

⇒ If The Resultant force acts within The middle third, The Bearing pressure diagram is trapezoidal & 100% of The Base will be in Compression

⇒ $P_{max} < SBC$, if it will more Than SBC Then RW Base will be fail.

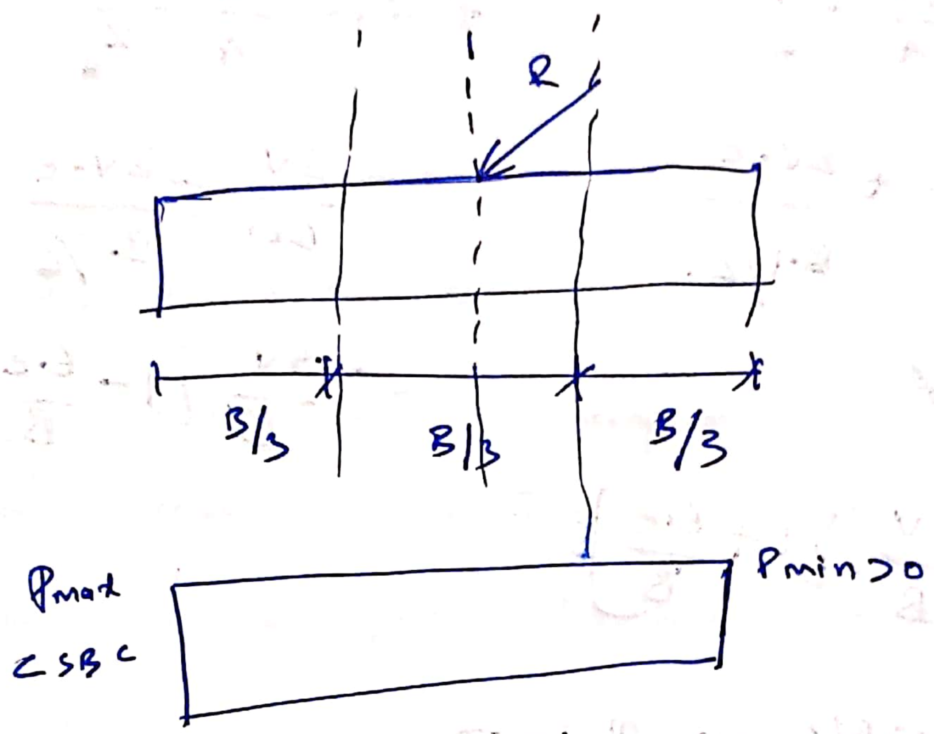


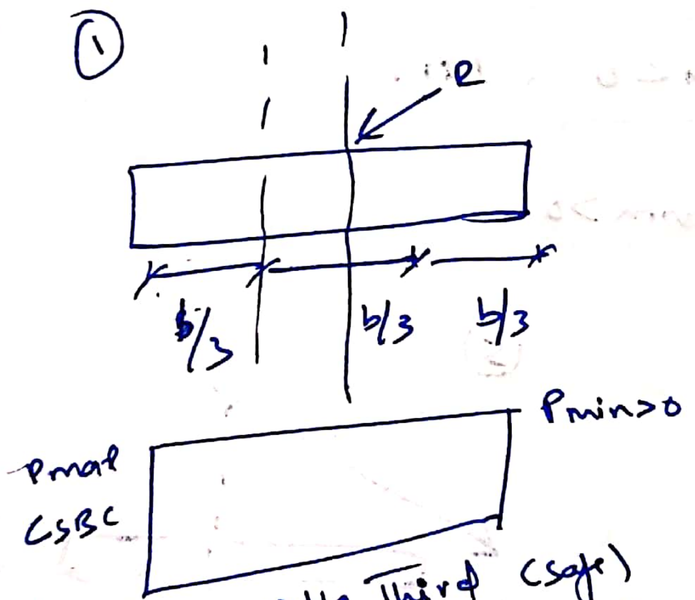
e = Eccentricity from center of Base to the dist. where resultance force is acting.

Here $x = \dfrac{\Sigma \text{moment about Toe}}{\Sigma V}$

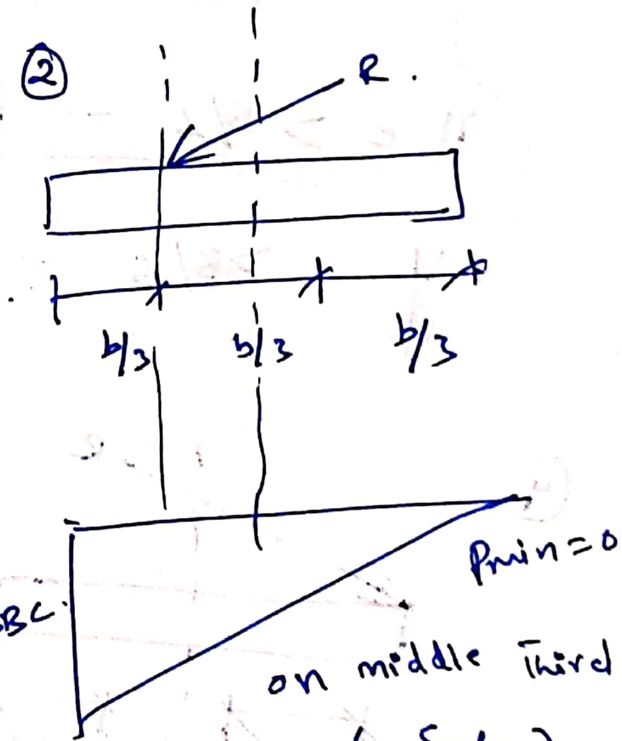
$x = \dfrac{M_r - M_o}{\Sigma V}$, $e = \dfrac{b}{2} - x$

Middle Third portion.

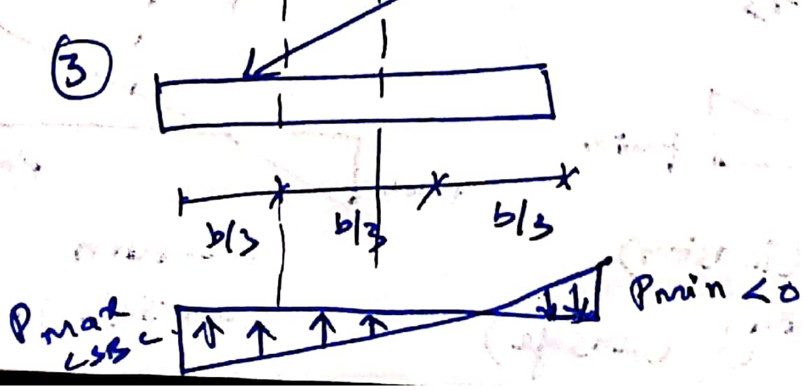




within middle third (safe)



on middle third (Safe)



(un-safe.)

(outside of middle third)

$$P_{max} = \frac{\Sigma v}{A} + \frac{M}{Z}$$

$$= \frac{\Sigma V}{B \times 1} + \frac{\Sigma V \cdot e}{B \cdot 1/6}$$

1m length of wall
B = width

$$\Rightarrow \frac{\Sigma V}{B}\left[1 + \frac{6 \cdot e}{B}\right]$$

$$P_{min} = \frac{\Sigma v}{A} - \frac{M}{Z}$$

$$= \frac{\Sigma V}{B \times 1} - \frac{\Sigma V \cdot e}{B \cdot 1/6}$$

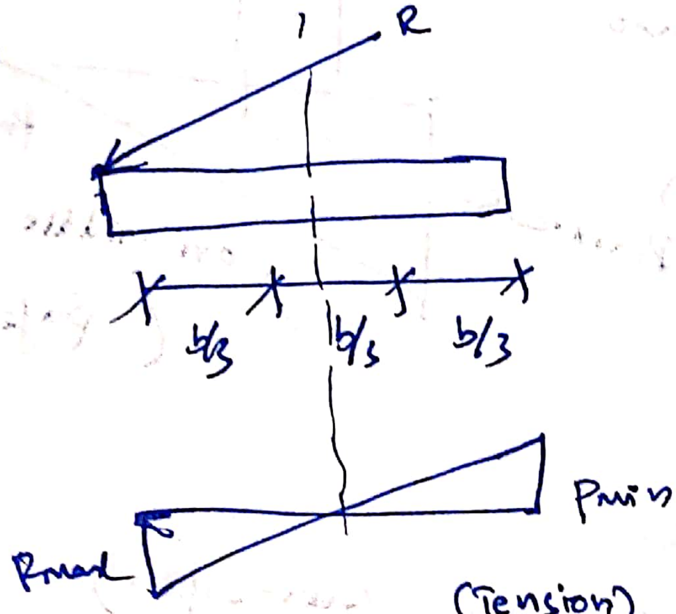
$$= \frac{\Sigma V}{B}\left[1 - \frac{6 \cdot e}{B}\right]$$

if $e = B/6$ -- $P_{min} = 0$

if $e > B/6$ -- $P_{min} < 0$ ~~...~~.

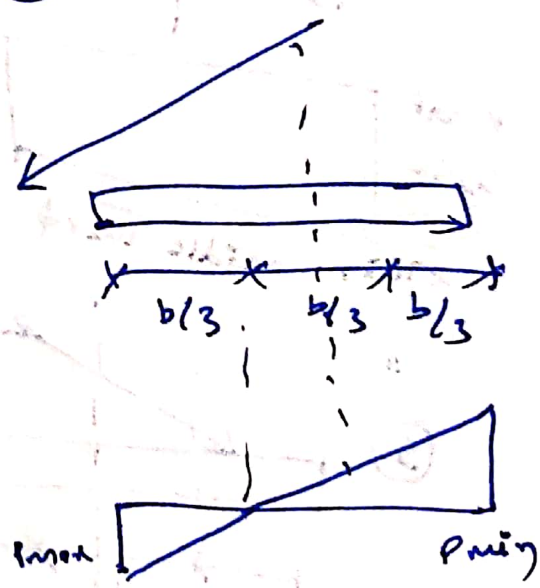
if $e > B/6$ -- $P_{min} > 0$

(4)



On-verge of over turning. (unsafe)

(5)



over turn
(un-safe).

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Example problem's for cohesion-less soils:-

Here we have 4 type's of problem's

1. Earth pressure In general case => with vertical wall & Horizontal Backfill [Dry or moist soil without-Surcharge]
2. Earth pressure with surcharge with Horizontal Backfill
3. Earth pressure with Inclined Backfill & surcharge.
4. Effect of submerged soil Earth pressure with Horizontal Backfill.

Case - I

Earth pressure In general case - with vertical wall and Horizontal Backfill (dry or moist soil without-surcharge)

Check the stability of Cantilever R.W which is read to support a bank of Earth 4m height above G.L. Dimension's of Row are given in Diagram.

=> Assume S.B.C of soil is 160 kN/m^2

=> U.W of soil is 16 kN/m^3

=> Angle of Internal friction of soil is 30°

=> co-Efficient of friction b/w concrete & soil is 0.5 use M25 & Fe415

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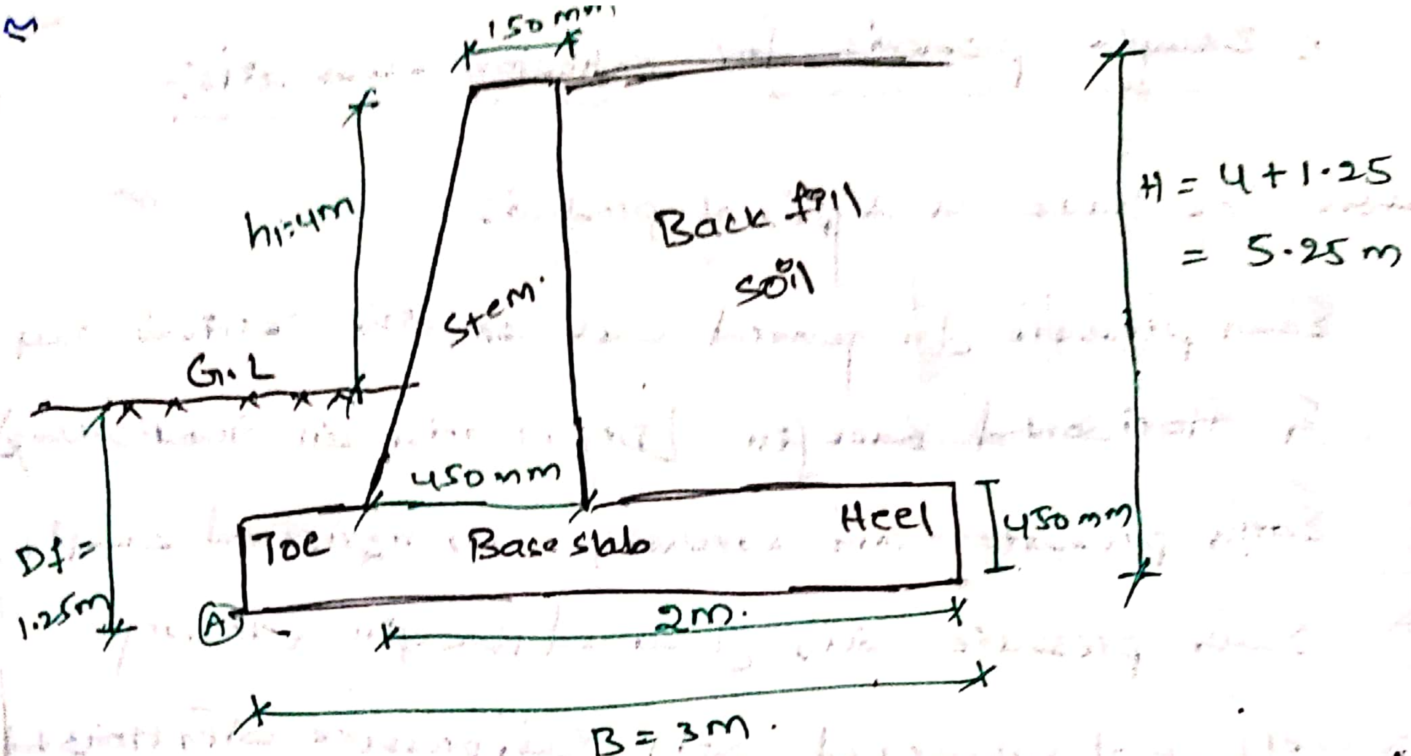


Soln:- We know $k_a = \dfrac{1-\sin\phi}{1+\sin\phi}$ & $k_p = \dfrac{1+\sin\phi}{1-\sin\phi}$

given $\phi = 30^\circ$ (Angle of internal friction)

So $k_a = \dfrac{1-\sin 30^\circ}{1+\sin 30^\circ}$, $k_p = \dfrac{1+\sin 30^\circ}{1-\sin 30^\circ}$

$k_a = \dfrac{1}{3}$ $\qquad$ $k_p = 3.$

Overturning check:-

$$\text{Fos of overturning} = \frac{\text{Resisting moment } (M_R)}{\text{Overturning moment } (M_O)}$$

M_R = Moment due to Verticle forces (down word)

M_O = Moment due to H_z forces

Fos. of overturning ≥ 1.5

Soln:- $k_a = \dfrac{1-\sin\phi}{1+\sin\phi}$ $k_p = \dfrac{1+\sin\phi}{1-\sin\phi}$

Given $\phi = 30^\circ$

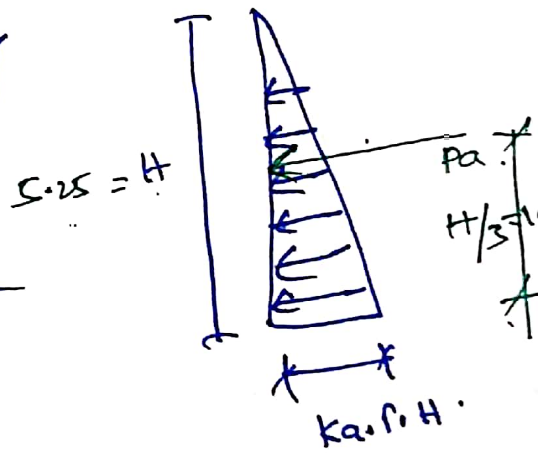
$k_a = \dfrac{1-\sin 30^\circ}{1+\sin 30^\circ} = \dfrac{1}{3}$ $k_p = \dfrac{1+\sin 30^\circ}{1-\sin 30^\circ} = 3.$

① Check for overturning :-

$$FOS = \frac{\text{Resisting Moment } (M_R) \Rightarrow \text{verticle forces } (\downarrow)}{\text{Overturning moment } (M_O) \Rightarrow \text{Hz forces } (\leftarrow) \; P_a.}$$

Overturning moment (M_O) => (due to Hz forces).

Hz forces. => P_a = lateral Earth pressure.

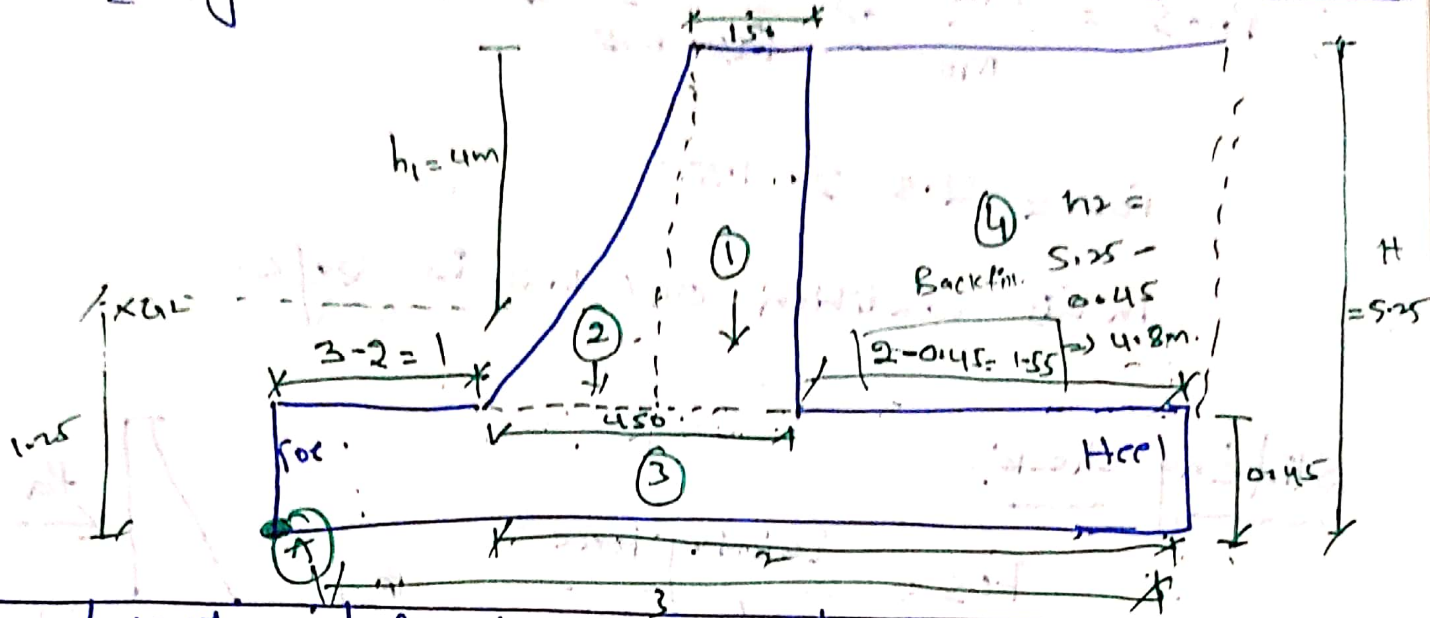


force (Pa) => Area of Triangle	Distance from Bottom	Moment
=> $\frac{1}{2}$ x Base x Height => $0.5 \times (K_a \cdot \gamma \cdot H) \times H$. => $0.5 \times \frac{1}{3} \times 16 \times (5.25)^2$	H/3 => 5.25/3 => 1.75	force x ⊥ar dist => 73 x 1.75 => $\boxed{128.625}$

=> 72.765

≅ 128.

Resisting moments = Moment due to vertical forces (↓ direction)



Element No	Name of element	force / load (kN) Self wt of element ⇒ vol of Element × density of material	Distance from point (A) (m)	Moment about point (A)
1	Stem = Rectangle	$(0.15 \times 4.8 \times 1) \times 25 = 18$	$\left(\frac{0.15}{2}\right) + (0.45 - 0.15) + 1$ ⇒ 1.375	force × ⊥ dist. 18×1.375 ⇒ 24.75
2	Stem = triangle	$\left[\frac{1}{2}(0.45 - 0.15) \times 4.8\right] \times 25 = 18$	$\frac{2}{3}(0.3) + 1$ ⇒ 1.2	18×1.2 ⇒ 21.6
3	Base of slab	$(1 \times 0.45 \times 3) \times 25 = 33.75$	⇒ $B/2 = \frac{3}{2}$ ⇒ 1.5	⇒ 33.75×1.5 ⇒ 50.62
4	Backfill meterial	$[1 \times (2 - 0.45) \times 4.8] \times 16 = 119.04$	⇒ $1 + 0.45 + \left(\frac{1.55}{2}\right)$ ⇒ 2.225	119.04×2.225 ⇒ 264.84
		$\Sigma V = 188.79$ kN.		$\Sigma M_1 = 361.839$ kN/m²
5	Shearkey	$(1 \times 0.45 \times 0.5) \times 25$ ⇒ 5.62	$1 + \left(\frac{0.45}{2}\right)$ ⇒ 1.225	⇒ 5.62×1.225 ⇒ 6.89

ΣV_2 ⇒ $188.79 + 5.62$ ⇒ 194.45

$\Sigma M_r = 361.839 + 6.89$ ⇒ 368.7

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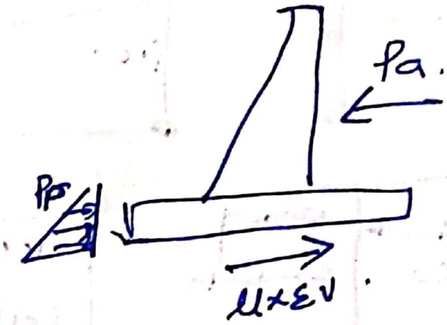
$$F.O.S = \frac{M_R}{M_O} \Rightarrow \frac{361.89}{128.625} \Rightarrow 2.8$$

$$FOS = 2.8 > 1.5$$

In this condition our R.W is safe.

Sliding check:-



$$FOS = \frac{\text{Resisting force}}{\text{sliding force}}$$

$$FOS = \frac{(\mu \times \Sigma V + P_P)}{P_a}$$

P_a = Horizontal component / A.E.P.

Resisting force $\Rightarrow \mu \times \Sigma V + P_P$. ($P_P$ = P.E.P)

$$P_a = \frac{1}{2} \times \text{Base} \times \text{hight} \Rightarrow \frac{1}{2} \times (k_a \times \gamma \times H) \times H$$

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

$$\Rightarrow 0.5 \times \frac{1}{3} \times 16 \times (5.25)^2$$

$$\Rightarrow 73.5 \text{ kN}.$$

$$P_p = \left(\frac{1}{2} \times \text{base} \times \text{height}\right) \Rightarrow \frac{1}{2} \times (k_p \times \gamma \times 0.45) \times 0.45$$

$$\Rightarrow \frac{1}{2} \times 3 \times 16 \times (0.45)^2$$

$$\Rightarrow 4.86 \text{ kN}.$$

$$\Sigma V = 188.79.$$

$$\text{fos} = \left(\frac{\mu \times \Sigma V + P_p}{P_a}\right)$$

$$\Rightarrow \frac{(0.5 \times 188.79) + 4.86}{73.5}$$

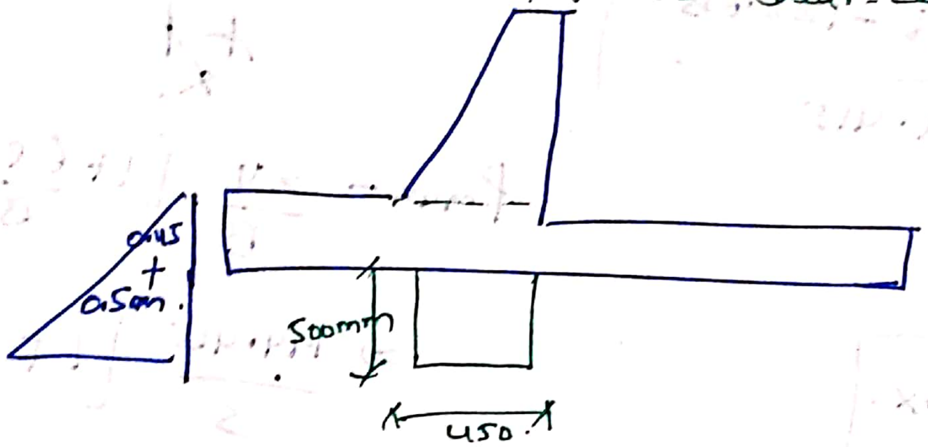
$$\Rightarrow \left(\frac{99.26}{73.5}\right) \Rightarrow 1.35$$

fos for sliding is 1.55

But we got fos < 1.55 ie 1.35

So our R.W is un-safe.

So we should provide - Shear - key



after providing shear key ΣV change.

& passive E.P is also changes

$P_p \Rightarrow \left(\frac{1}{2} \times \text{Base} \times \text{height}\right) \times \text{hight}$

$\Rightarrow \frac{1}{2} \times (K_p \times \gamma \times 0.9) \times 0.9$

$P_p \Rightarrow 0.5 \times (3 \times 16 \times 0.95) \times 0.95 \Rightarrow 21.66$ kN.

$\text{fos} = \frac{118.86}{73.5} \Rightarrow 1.61$

$1.61 > 1.55$

So. R.W. is Safe.

Resisiting force. $[(0.5) \times (194.415)] + 21.66$

$\Rightarrow 94.39 + 21.66 \Rightarrow 118.86$ kN.

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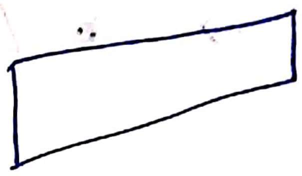
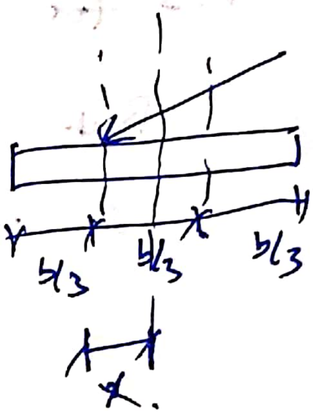
Check for B.C

$$P_{max} = \frac{\Sigma V}{B}\left[1+\frac{6e}{B}\right] < SBC$$

$$P_{min} = \frac{\Sigma V}{B}\left[1-\frac{6e}{B}\right] \geq 0$$



$$x = \frac{\Sigma \text{moment about toe}}{\Sigma V}$$



$$x = \frac{M_r - M_o}{\Sigma V}$$

$$x = \frac{368.98 - 128.625}{194.415}$$

$x = 1.23$

$$\boxed{e = \frac{B}{2} - x}$$

$$e = \frac{B}{2} - 1.23$$

$\Rightarrow 1.5 - 1.23$

$$\boxed{e \Rightarrow 0.27}$$

$e < B/6$

$0.27 < \frac{3}{6}$

$\Rightarrow \boxed{0.27 < 0.5}$

$$P_{max} = \frac{\Sigma V}{B}\left[1+\frac{6e}{B}\right]$$

$$\Rightarrow \frac{194.415}{3}\left[1+\frac{6 \times 0.27}{3}\right]$$

$P_{max} \Rightarrow 99.79 < SBC$

$\Rightarrow 160$

$\boxed{\Rightarrow 99.79 < 160}$ Safe

$$P_{min} = \frac{\Sigma V}{B}\left[1-\frac{6e}{B}\right]$$

$$\Rightarrow \frac{194.415}{3}\left[1-\frac{6 \times 0.27}{3}\right]$$

$P_{min} \Rightarrow 29.81$ kN ≥ 0

Safe Condition.

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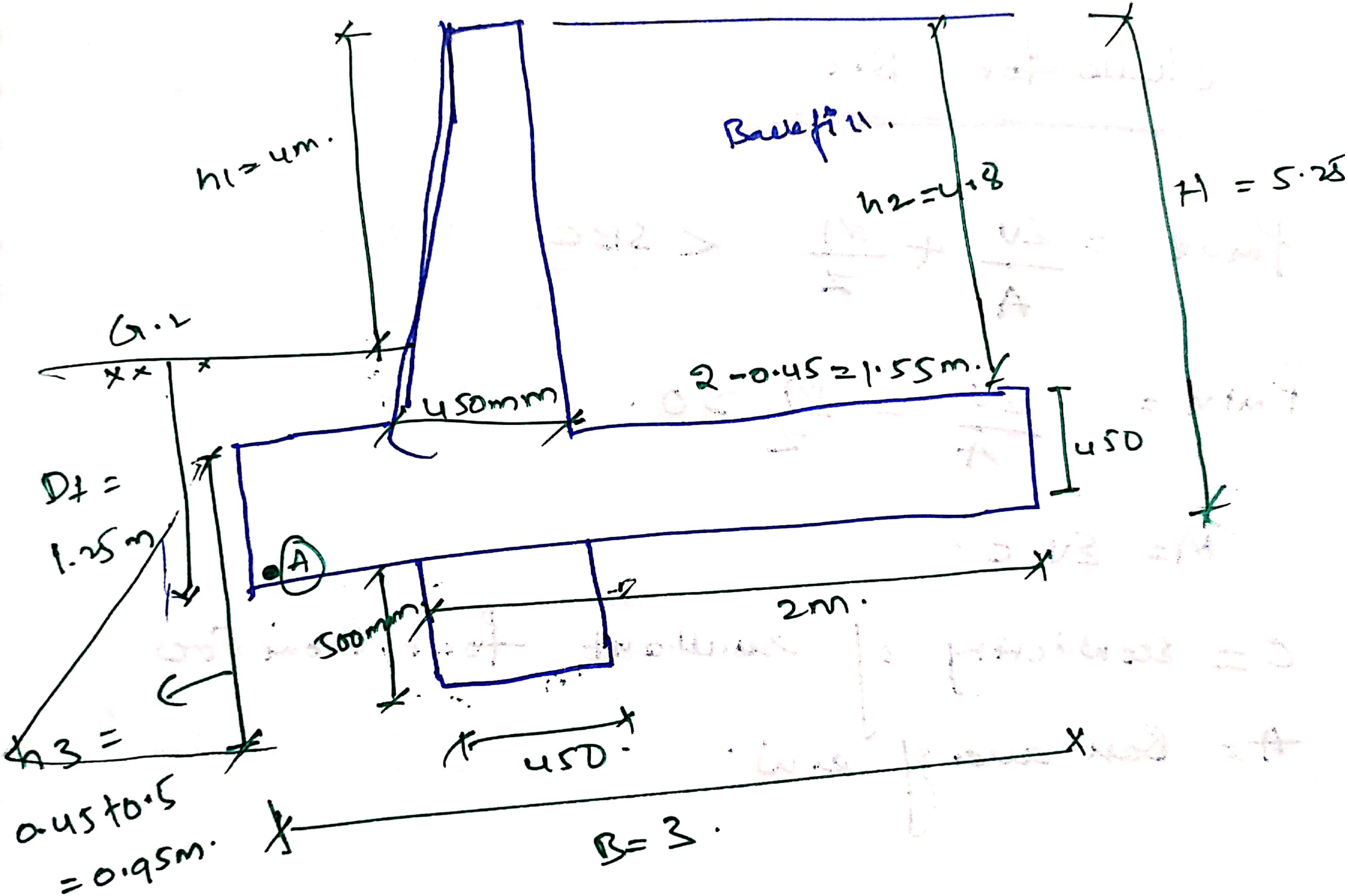

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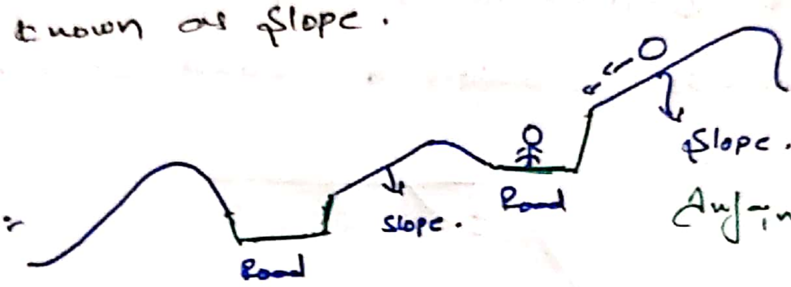
: Module- 2 :-

: Slope Stability & Earth pressure Theories :-

Introduction:-

Slope:- when the Soil is formed in Inclined position Then it is known as slope.

Mountain



Infinite slope

Canal:-

finite slope.

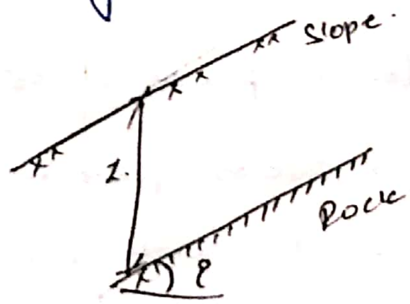
Slope's are Two type's:- → Infinite Slope
→ finite Slope.

Infinite Slope:-

⇒ Infinite Slope's are not bounded. [Ex: Mountain]

⇒ failure Surface is parallel to the Surface of slope.

i.e



i = Angle of slope.

⇒ on Increasing the z value slope failure will occure i.e Soil Shear's

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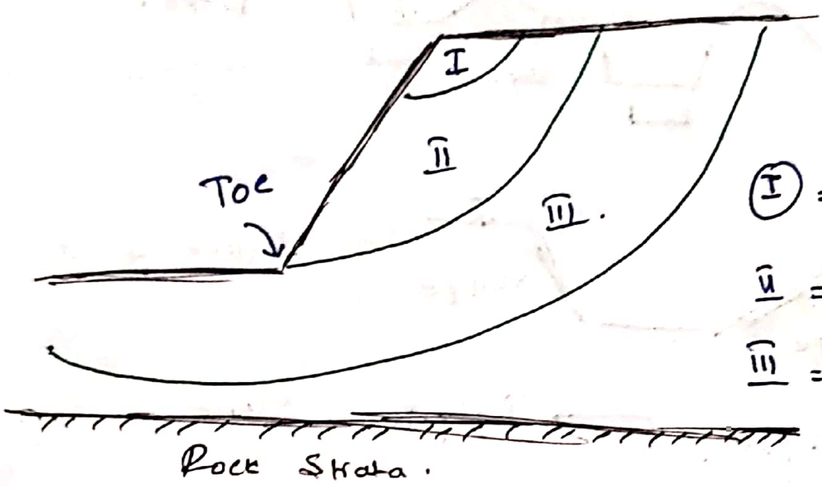
② finite Slope's:-

⇒ They are Bounded from Top & bottom. (Ex:- canal)

⇒ failure Surface is circular or Spiral.

⇒ finite slope's are having different failure mode's unlike infinite slope's.

to Explain This Consider a canal Section.



Ⓘ = face failure

II = Toe failure

III = Base failure.

⇒ Face failure is occured when The Soil in The upper part is weaker than the soil of lower part Then weak Soil will Shear's. Such failure is called face failure.

⇒ Toe- failure:-

whenever upper & lower parts of soil's are homogeneous Then in That condition due to increasing in stresses The failure is occured. such failure is called as Toe-failure.

⇒ Base- failure:- (opposite to face failure).

when The lower part of soil is weak comparitively to The upper part of soil Then Base-failure is occured.

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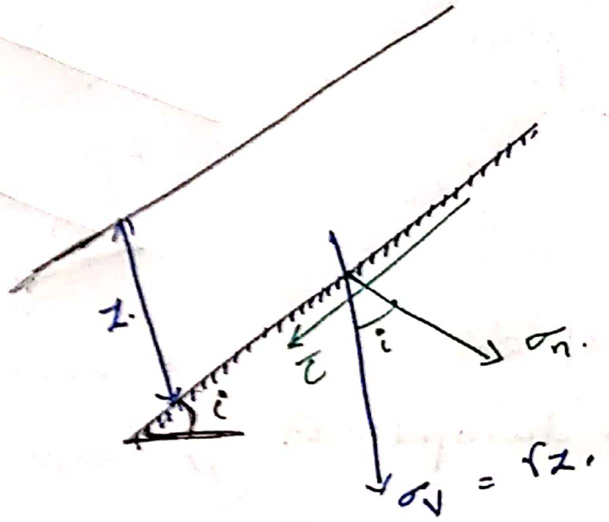
Analysis of Infinite Slope :-



$\sigma_v = \gamma z \cdot \cos i$

σ_n = Normal Stress / Normal component

τ = Shear Stress or Tangential Stress. ☆☆

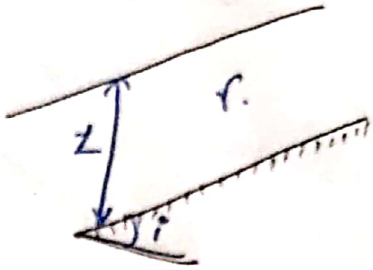
$\sigma_n = \sigma_v \times \cos i \Rightarrow (\gamma z \times \cos i) \times \cos i \Rightarrow \boxed{\sigma_n = \gamma z \cos^2 i}$

$\tau = \sigma_v \times \sin i \Rightarrow \gamma z \cos i \times \sin i \Rightarrow \boxed{\tau = \gamma z \cos i \times \sin i}$

Case - I : cohesionless soil in Infinite slope :- (sandy soil)

(No - water Table condition)

⇒ Here we will calculate FOS against sliding / shear failure.



$$FOS = \frac{\tau_f}{\tau}$$

i.e $FOS = \dfrac{\text{Shear Strength of soil}}{\text{Shear stresses developed in soil}}$

i = Angle of slope

ϕ = Angle of internal friction

w.r.t $\tau_f = c' + \sigma' \tan\phi'$ from Mohr's columb Theory

So $FOS = \dfrac{\tau_f}{\tau} = \dfrac{\cancel{c'}^{\,0} + \sigma' \tan\phi'}{\tau}$

⇒ here c = 0 Bcz soil is cohesionless (sandy)

⇒ σ' & ϕ' becomes σ & ϕ Bcz no pore water pressure

$$FOS = \frac{0 + \sigma_n \tan\phi}{\tau}$$

$$FOS = \frac{\sigma_n \tan\phi}{\tau}$$

from Eqn ① & ② w.r.t $\sigma_n = \gamma z \cos^2 i$, $\tau = \gamma z \cos i \times \sin i$

So $FOS = \dfrac{(\cancel{\gamma z} \cos^2 i) \tan\phi}{\cancel{\gamma z \cos i} \times \sin i} = \dfrac{\cos i \times \tan\phi}{\sin i} \Rightarrow \dfrac{\tan\phi}{\frac{\sin i}{\cos i}}$

⇒ $\boxed{FOS = \dfrac{\tan\phi}{\tan i}}$

Note :- i) FOS > 1 Then soil is in safe condition

ii) FOS < 1 Then soil is in un-safe condition

⇒ To satisfy FOS > 1 Then $i < \phi$ {for cohesion less soil's only}

Such That soil is in safe condition

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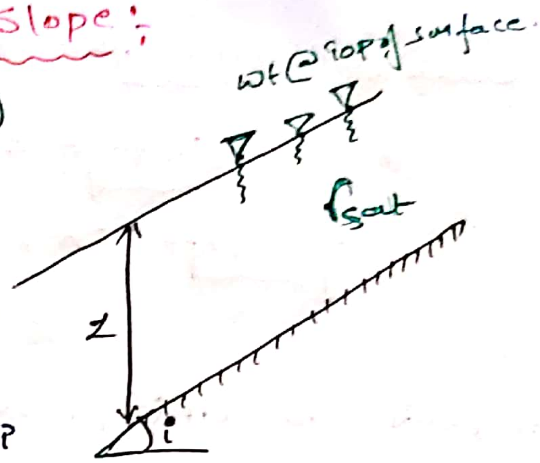
Fully Submerged - Infinite Slope :-

case (ii) (Cohesion less soil i.e $c=0$)



w.r.t $\sigma_n' = \sigma_v' \cos i$

in Submerged condition
$\gamma = \gamma_{sub}$ &
$\sigma_n = \sigma_n'$
Bcz of p.w.p

So $\boxed{\sigma_n' = \gamma_{sub} \times Z \cos^2 i}$

Like wise w.r.t

$$\tau = \sigma_v' \sin i$$

$$\boxed{\tau = \gamma_{sub} Z \cdot \cos i \times \sin i}$$

ϕ' = Effective Angle of Internal friction

ϕ = Total Angle of Internal friction.

So $fos = \frac{\tau_f}{\tau} = \frac{\cancel{c'}^{0} + \sigma_n' \tan\phi'}{\tau}$

$fos = \frac{\cancel{\gamma_{sub}} \times \cancel{Z} \cos^{\cancel{2}} i \tan\phi'}{\cancel{\gamma_{sub}} \cdot \cancel{Z} \cdot \cancel{\cos i} \times \sin i} \Rightarrow \frac{\cos i \tan\phi'}{\sin i}$ on re-arranging

we get $Fos = \frac{\tan\phi'}{\left(\frac{\sin i}{\cos i}\right)} \Rightarrow \boxed{Fos = \frac{\tan\phi'}{\tan i}}$

Note:- To Increase The Fos we should keep less i value Than The ϕ' value.

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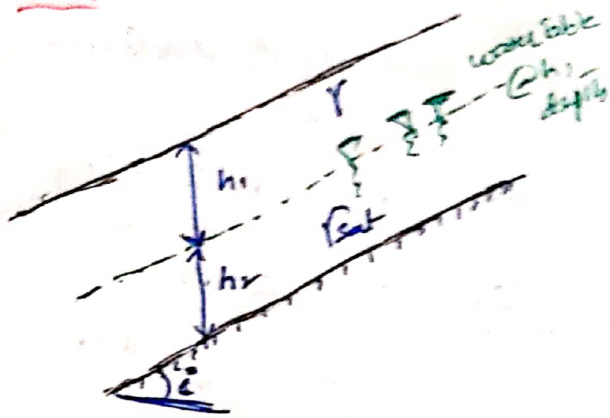
Case-(iii) water Table at any depth In cohesionless soil and Seepage occurs parallel to The Surface of slope in Infinite Slope:-



=> In this case WT is presenting @ a depth of h_1 from surface.

=> So The soil which presenting above WT is may be dry soil or may be partially saturated.

=> In case of dry soil $\gamma = \gamma_d$ & for partially saturated soil $\gamma = \gamma_{bulk}$.

Here we assuming dry soil i.e $\underline{\gamma}$.

So w.r.T $\sigma'_n = \sigma'_v \times \cos i$ Her σ'_v is Effective Stress

w.r.T {Effective stress = Total stress − p.w.p}

$\sigma'_n = (\gamma \times h_1 + \gamma_{sat} \times h_2 - \gamma_w \times h_2)\cos i \times \cos i$

&

w.r.T $(\sigma_v = \gamma z \times \cos i)$

$\sigma'_n = [\gamma h_1 + h_2(\gamma_{sat} - \gamma_w)]\cos^2 i$

γ_w = unit wt of water

$$\boxed{\sigma'_n = [\gamma h_1 + h_2 \times \gamma_{sub}]\cos^2 i}$$

{w.r.T $\gamma_{sat} - \gamma_w = \gamma_{sub}$}

$\tau = \sigma_v \times \sin i$

$$\boxed{\tau = [\gamma h_1 + \gamma_{sat} h_2]\cos i \cdot \sin i}$$

(=> In calculation of τ we are not considering p.w.p Bcz The pressure due to Seepage & p.w.p cancells Each other. Bcz they Both are in same direction)

$Fos = \frac{\tau_f}{\tau} = \frac{\cancel{c'}^{\,0} + \sigma'_n \tan\phi'}{\tau}$

$Fos = \frac{(\gamma h_1 + \gamma_{sub} h_2)\cos^{\cancel{2}} i \tan\phi'}{(\gamma h_1 + \gamma_{sat} h_2)\cancel{\cos i} \times \sin i}$

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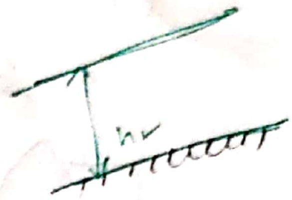
$$FOS = \left(\frac{\gamma h_1 + \gamma_{sub} h_2}{\gamma h_1 + \gamma_{sat} h_2}\right) \frac{\tan\phi'}{\tan i}$$

Note: water reaches to the top surface & if seepage occurs.

i.e $h_1 = 0$ & Total height $= h_2$

Substitute these values in above eqn we get

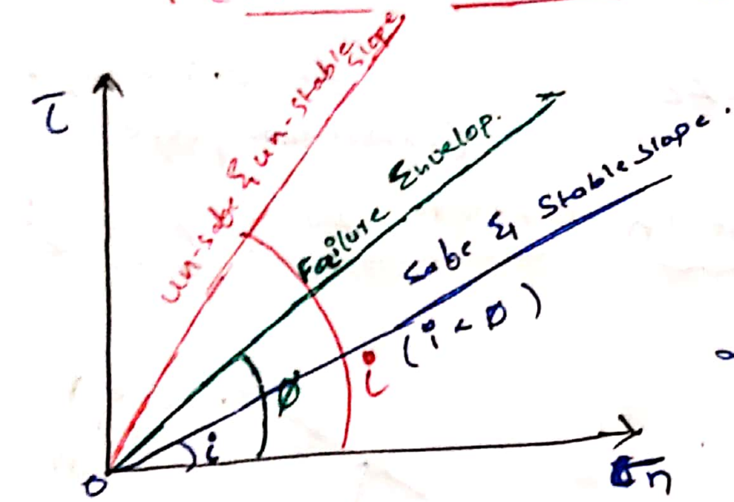


$$\boxed{FOS = \frac{\gamma_{sub}}{\gamma_{sat}} \times \frac{\tan\phi'}{\tan i}}$$

$$FOS \simeq \frac{1}{2} \times \frac{\tan\phi'}{\tan i}$$

In this case minimum f.o.s should be > 2.

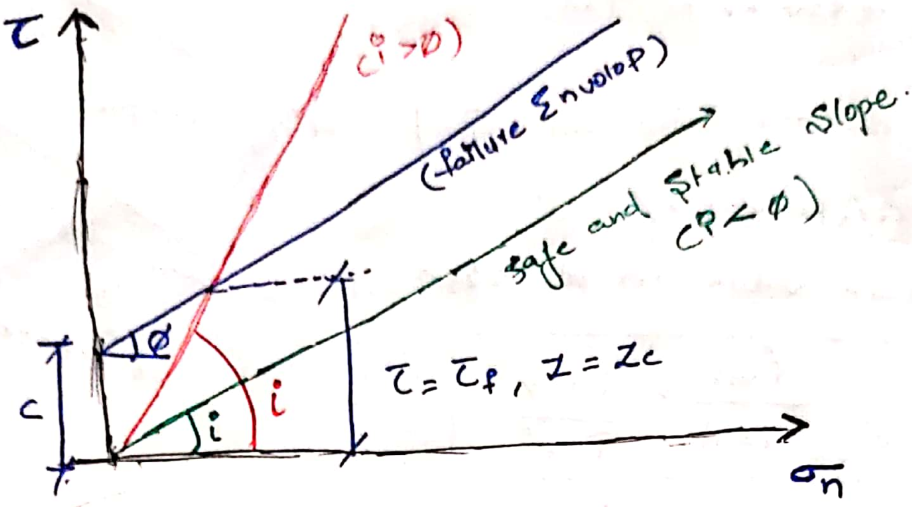
Infinite slope in cohesion less soil :-



- ⇒ w.k.t if $c = 0$, then failure Envelop starts from origin (O)
- ⇒ failure Envelop shows the state of strength.
- ⇒ Mohr's circle below failure Envelop shows the state of stress.
- ⇒ if mohr's circle is below failure Envelop then soil is safe
- ⇒ if mohr's circle touches or exceeds (i.e stress > strength) then soil fail's.

Note: In case of cohesion less soil's in infinite slope angle of slope (i) should be less than angle of internal friction (or) angle of shearing resistance (ϕ).

Infinite Slope in Cohesive Soil [c-ϕ soil]:-



Note:- In case of cohesive soil in Infinite Slope.

⇒ if $i < \phi$ Then it is Safe and Stable but

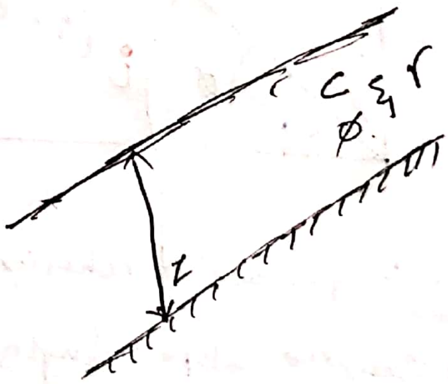
⇒ if $i > \phi$ Then also it can be Safe and Stable

if $\tau < \tau_f$, $z < z_c$

z_c: critical depth (i.e Max depth of slope).

Case (IV). Infinite Slope in cohesive Soil [c-ϕ soil]:-

$$FOS = \frac{\tau_f}{\tau} = \frac{c + \sigma_n \tan\phi}{\tau}$$



$$FOS = \frac{c + \gamma z \cos^2 i \tan\phi}{\gamma z \cos i \tan i} \longrightarrow (1)$$

$$FOS = \frac{c}{\gamma z \cos i \tan i} + \frac{\cancel{\gamma z} \cos^{\cancel{2}} i \tan\phi}{\cancel{\gamma z} \cancel{\cos i} \tan i}$$

$$\boxed{FOS = \frac{c}{\gamma z \cos i \cdot \tan i} + \frac{\tan\phi}{\tan i}} \longrightarrow (2)$$

Compare This eqn with case-(i) In This case FOS is ↑ due to cohesion. So we can keep $(i > \phi)$

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For Critical Condition, Fos = 1.

If Fos = 1, Then $z = z_c$, $\tau_f = \tau$

Substitute above values in Eqn ①. we get

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

$$\gamma z_c \cos i \sin i = c + \gamma z_c \cos^2 i \tan\phi$$

$$c = \gamma z_c \cos i \sin i - \gamma z_c \cos^2 i \tan\phi$$

$$c = \gamma z_c \cos^2 i \left[\frac{\sin i}{\cos i} - \tan\phi\right]$$

$$\frac{c}{\gamma z_c} = \left[\tan i - \tan\phi\right] \cos^2 i = S_n.$$

S_n = Taylor's Stability Number.

So
$$\boxed{S_n = \left[\tan i - \tan\phi\right] \cos^2 i \qquad S_n = \frac{c}{\gamma z_c}.}$$ E_1

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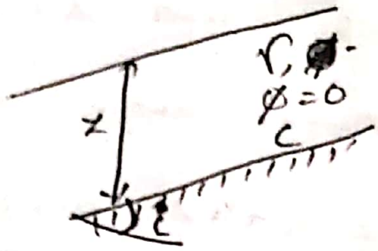
Infinite Slope in clay soil ($\phi = 0$)



$$FOS = \frac{\tau_f}{\tau} = \frac{c + \sigma_n \tan\phi}{\tau}$$

but $\phi = 0$. So

$$FOS = \frac{c}{\tau} \quad [\because \tan 0 = 0.]$$

$$\boxed{FOS = \frac{c}{\gamma z \cos i \sin i}}$$

for critical condition w.k.T $FOS = 1$, $z = z_c$.

So $$1 = \frac{c}{\gamma z_c \cos i \sin i}$$

$$\boxed{\cos i . \sin i = \frac{c}{\gamma z_c}}$$

w.k.T $\frac{c}{\gamma z_c} = S_n$ i.e Taylor's Stability number

So $$\boxed{S_n = \cos i \times \sin i}$$

Short cut to Remember above formule:-

w.k.T for $c-\phi$ soil $S_n = (\tan i - \tan\phi)\cos^2 i$

put $\phi = 0$ in above S_n

$$S_n = (\tan i - \tan 0)\cos^2 i$$

$$S_n = \tan i \times \cos^2 i$$

$$S_n = \frac{\sin i}{\cancel{\cos i}} \times \cos^{\cancel{2}} i$$

$$\boxed{S_n = \sin i \times \cos i} \rightarrow \text{for clay soil}$$

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Stability Analysis by Swedish Arc Method :-

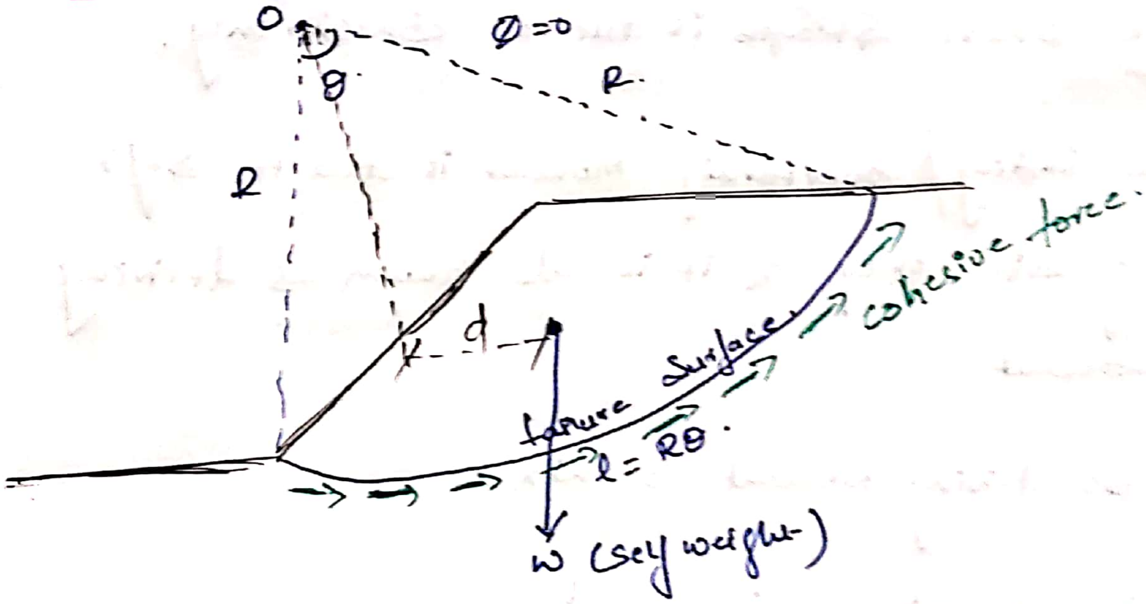
=> this Method is used to Study The Analysis of finite Slope's.

Finite Slope:- If The Slope is of finite dimension & Bounded by top & bottom Surface is termed as finite slope.

=> In finite Slopes failures occures due to rotation. & failure Surfaces are may be Circular or Spiral.

=> This Method is applicable for c & c-ϕ Soil's.

(i) Check for Stability of c - Soil:-



=> let us take a finite Slope which is having a slip failure Surface.

=> Consider The moments about 'O' which is having Radiu 'R' & making An angle θ with failure Surface.

=> here 'w' is Self wt of soil which acts Vertically downwords

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⇒ here length of Arc i.e failure surface is 'l'.

$l = R \times \theta$.

⇒ here soil's is slides due to self wt 'w'.

⇒ cohesive forces which are developed in soil will Resist The sliding moment of soil.

⇒ so cohesive forces are formed due to shear strength of soil's.

w.r.t shear strength $\tau_s = c + \sigma \tan\phi$

here $\phi = 0$ so

$$\boxed{\tau_s = c.}$$

i.e shear strength is due to cohesion only.

⇒ here sliding / overturning moment is due to self wt of soil only & it is also known as deriving moment.

⇒ So driving moment $= w \times d$.

Resisting moment = Force × Leaver Arm.

Arc length $(l) = R \cdot \theta$.

Resisting force = $c \times$ are of plane

$= c \times R\theta \times 1$

$= c \times R\theta$.

Resisting moment $= cR\theta \times R$

$= cR^2\theta$.

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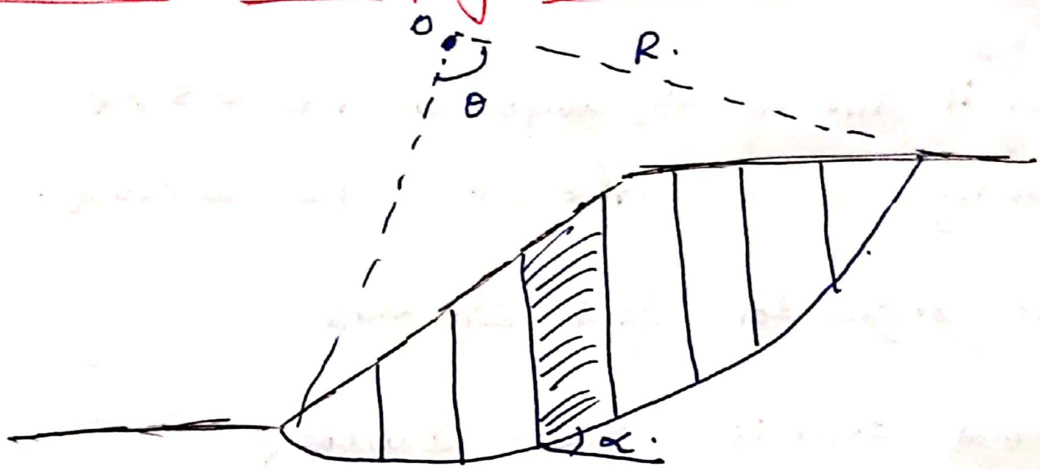
$$Fos = \frac{\text{Resisting Moment}}{\text{deriving moment}} = \frac{cR^2\theta}{w \times d}.$$

$$\boxed{Fos = \frac{cR^2\theta}{w \times d}}$$

if $fos > 1$ Slope is safe.
$fos < 1$ slope fails/un-safe.
$fos = 1$ critical condition.

Check for Stability of c-ϕ soil:-



⇒ In This Method c-ϕ soil is assumed to be devided in to no. of slices of Equal width having unit Thickness

⇒ No. of slices Should be > 5 (or) (6-12).

⇒ Each slice of soil mass acts like indipendent column & it is assumed That There is no forces b/w adjacent slices.

⇒ slice is making An Angle 'α' with horizontal.

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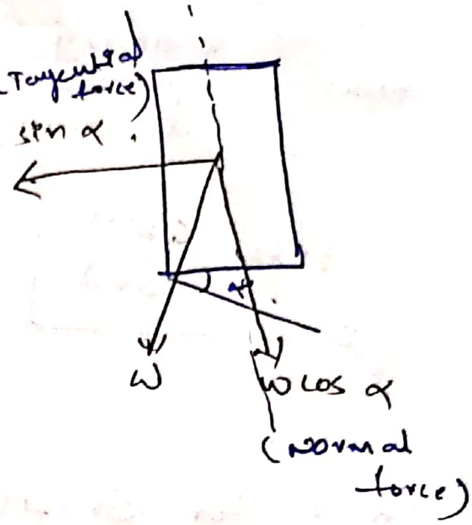
Consider a single slice.

⇒ say wt 'w' acting vertically downwards.



& w is having two components.

Vertical & horizontal.

Hz component = $w \sin\alpha$

Vertical " = $w \cos\alpha$.

⇒ Sliding is due to Hz component i.e $w \sin\alpha$.

⇒ Resisting " " " Vertical " i.e $w \cos\alpha$.

⇒ if we consider for Total soil mass

Normal force is $\Sigma N = \Sigma w \cos\alpha$

Tangential force $\Sigma T = \Sigma w \sin\alpha$.

⇒ here driving moment is due to tangential forces.

⇒ force × leaver Arm.

driving moment ⇒ $\Sigma \sin\alpha \times R$.

⇒ Resisting moment is due to Shear Strength.

⇒ Shear Strength is due to c & ϕ.

we k.T c value from previous sum

$C = cR^2\theta$.

Resisting moment due to friction is

~~Cφ tanφ~~

σ is normal stress so

$\sigma = (\Sigma N \tan\phi)\ R.$

Resisting moment $= CR^2\theta + \Sigma N \tan\phi\ R.$

So $fos = \dfrac{CR^2\theta + \Sigma N \tan\phi R}{\Sigma T \times R.}$

$$\boxed{fos = \frac{CR\theta + \Sigma w \cos\alpha \tan\phi}{\Sigma w \sin\alpha}}$$

$fos < 1$ unsafe.

$fos > 1$ safe.

$fos = 1$ critical conditn.

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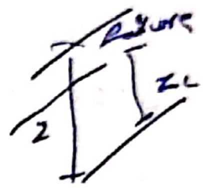
Factor of safety w.r.t different components of soil:-

⇒ FOS of cohesive soil can be represented in terms of height also.

1) F.O.S of cohesive soil in term's of height

$$F.O.S_{(H)} = \frac{H_c}{H} \text{ or } \frac{Z_c}{Z}.$$



H_c: Critical height of slope

H = Total height of slope.

2) FOS in term's of cohesion.

i) $H < H_c$ (or) $Z < Z_c$

$$(FOS)_c = \frac{c}{c_m}.$$

c = Total cohesion

c_m = Mobilised (utilised) cohesion

w.r.t $S_n = \frac{c}{\gamma \times Z_c}$

③ F.O.S w.r.t friction [Angle of internal friction]

⇒ it is the ratio of available frictional strength to the mobilised frictional strength

$$(FOS)_\phi = \frac{\sigma_n \tan\phi}{\sigma_n \tan\phi_m}.$$

$$FOS_\phi = \frac{\tan\phi}{\tan\phi_m}.$$

$$\boxed{\text{Max } S_n = 0.261}$$

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Taylor's Stability Number Method :-

⇒ In this method stability of slope is analysed with the help of Taylor's stability number which is taken from Taylor's chart on the basis of given c & ø values.

w.r.t from previous derivation's,

Taylor's stability no (Sn)

$$\boxed{S_n = \frac{c}{\gamma \times H_c} \text{ or } \frac{c}{\gamma \times Z_c}}$$

{ if soil is dry then γ
if soil is submerged $\gamma = \gamma_{sub}$
if soil is saturated $\gamma = \gamma_{sat}$ }

we can modify the Sn in to various forms.

w.r.t $F = \frac{Z_c}{Z}$.

so $Z_c = F \times Z$.

i.e $S_n = \frac{c}{\gamma \times Z_c}$

$$\boxed{S_n = \frac{c}{\gamma \times F \times Z}}$$ in term's of F.O.S

w.r.t ⇒ $F = \frac{c}{c_m}$.

$c_m = \frac{c}{F}$.

w.r.t $\frac{c}{F} = c_m$ if we keep it in above eqn

$$\boxed{S_n = \frac{c_m}{\gamma \times Z}}$$

Here Sn is in terms of mobilised cohesion.

$$\boxed{F = \frac{c}{\gamma S_n H}}$$

Culmann's Graphical Method for Estimating Active Earth pressure:-

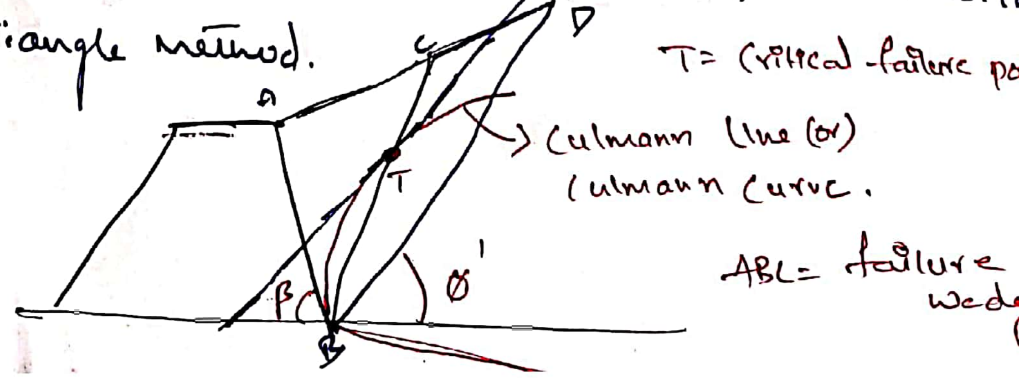
=> In 1866 Culmann's given Earth pressure Theory graphically to Estimate Active Earth pressure.

=> it was depending on coloumb's Earth pressure Theory i.e coloumb's wedge Theory.

=> In This Theory The failure wedge is Experiencing diff type's of Surcharge load's [Ex:- UDL (kN/m²), Line load (k... where as in diff wedge Theories only a single type of load is taken in to account.

=> due to Varies type's of Surcharge load's on failure wedge There will be a critical failure plane is formed, along with The Culmann's line. (or) Culmann Curve.

=> So Culmann's Considered to determine The active Earth Pressure by Culmann line (or) Culmann Curve with force triangle method.



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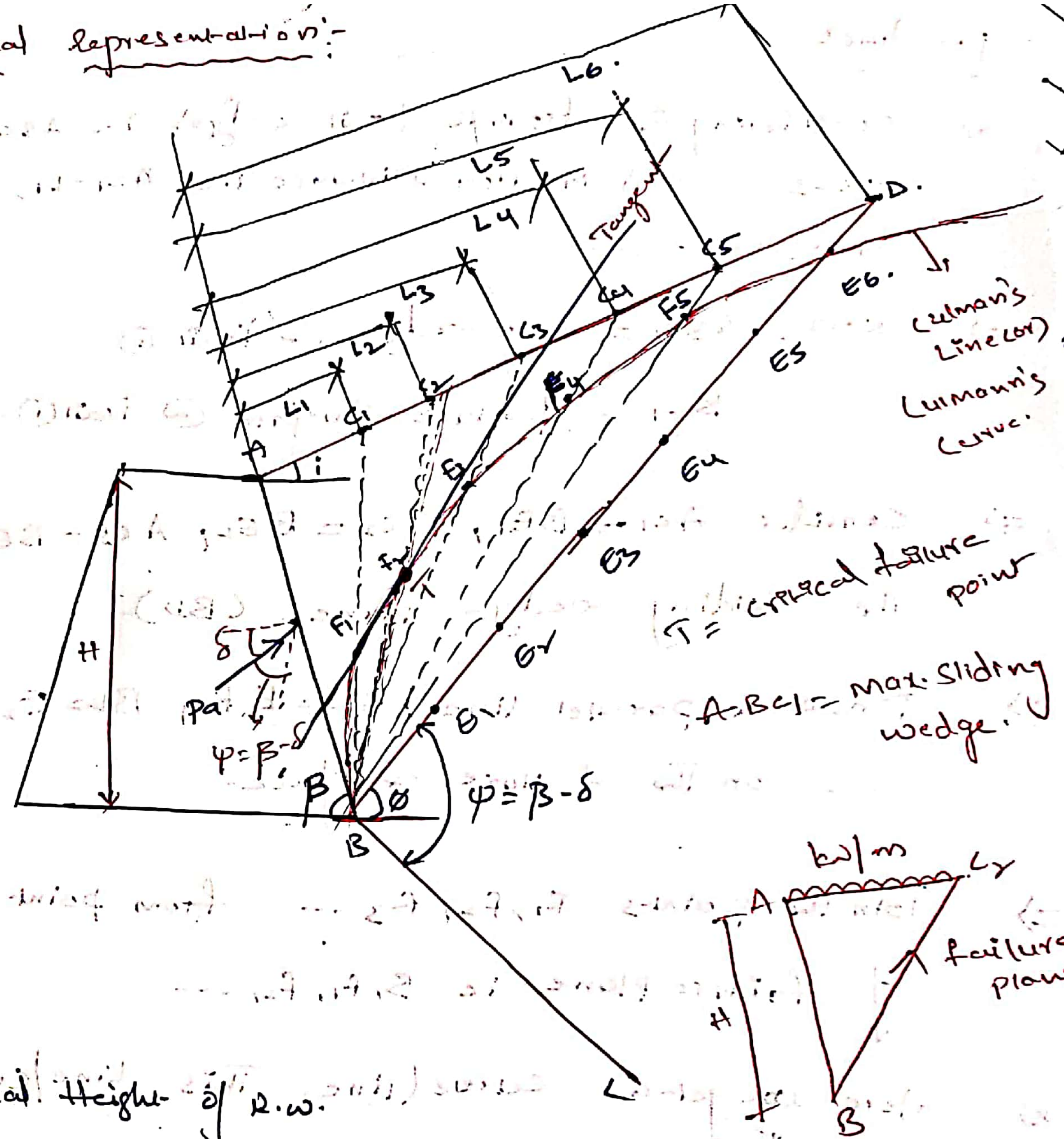
graphical representation:-



H = vertical Height of R.w.

i = Slope angle of Back fill

Ø = Angle of failure plane.

Procedure:-

⇒ Considering the triangle trail wedge's i.e ABC_1, ABC_2, ABC_3 .... & Mention Distance i.e $AC_1 = L_1$, $AC_2 = L_2$, $AC_3 = L_3$, ----

⇒ Now, ABC_1 = wt of wedge @ Trail ①
BC_1 = failure Surface @ Trail ①.

⇒ Consider $AC_1 = BE_1$, $AC_2 = BE_2$, $AC_3 = BE_3$ --- on the sliding wedge plane, (BD).

⇒ Draw a parellel line. $BL = E_1F_1$, $BL = E_2F_2$, $BL = E_3F_3$ --- on the failure Surface.

⇒ join the points F_1, F_2, F_3 --- from point B to End of failure plane i.e B, F_1, F_2, ---

⇒ Here we get a curve line. This Line/curve we called as "Culumnn's line/Culumann's Curve"

⇒ Draw a tangent line to Culmann's Curve, this tangent should be parellel to sliding/failure plane

⇒ Tangent Intersecting point on Curve is taken as failure point

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Module - III

3) Explain briefly various types of shear failures in shallow foundations. Mention the parameters to decide type of shear failure?

Ans: There are three Methods that shallow foundation failure occurs.

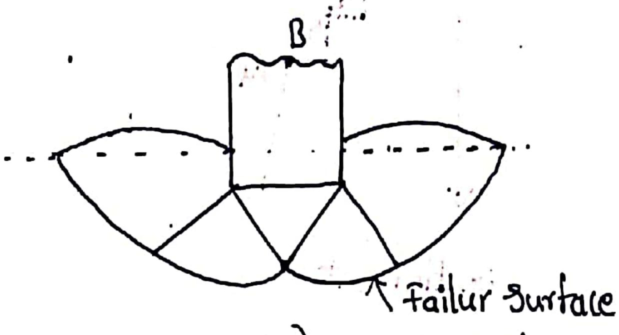
a) General Shear failure
b) Local shear failure
c) Punching shear failure

a) General Shear Failure:

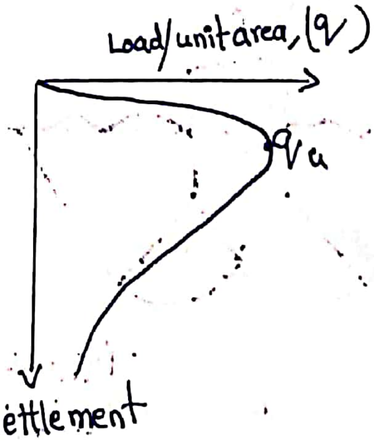
A gradual increase in the load on the foundation will increase the settlement of the footing and rises the pressure under the foundation.

When the pressure under the foundation reaches the ultimate pressure that the soil can bear, the foundation will fail suddenly.

These types of foundation failures occurs in low compressible soils.



(a)



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The ultimate pressure that soil can bear is the ultimate bearing capacity of the foundation. As indicated in the above, when the load reaches the ultimate bearing capacity (P_u), it fails

1) This types of shallow foundation failure occurs in dense sand, (or) stiff cohesive soil.

2) Dense and stiff soils are low compressible leads to shear failure.

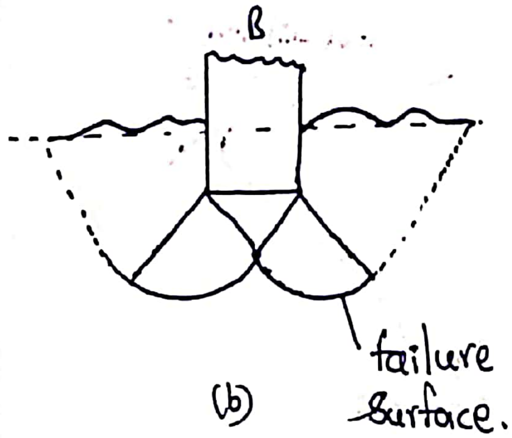
3) Tilting of foundation occurs when fail.

ii) Local shear failure :-

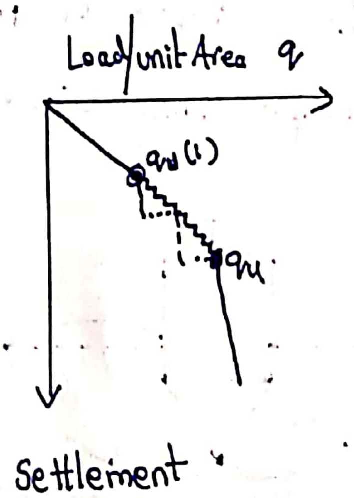
Local shear failure occurs in medium compressible soil.

An increase in the pressure in the foundation increases the settlement of the foundation and the failure surface in the soil gradually extent outwards from the foundation.

Failure surface does not extend to the ground surface as the soil is compressible (or) due to the deeper depth of the foundation.



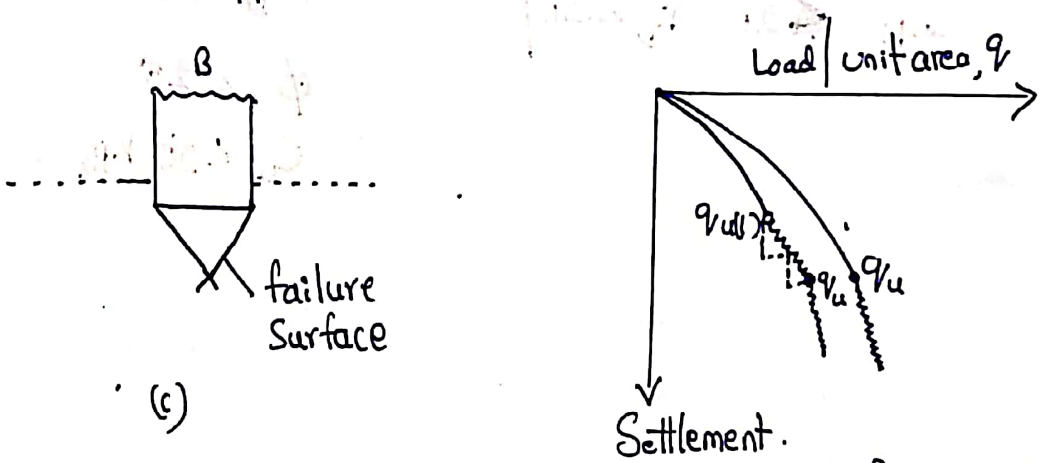
(b)



Punching Shear failure:- This type of failure occurs when the foundation is rested on fairly loose soil where the settlement of the foundation is higher.

Shear surfaces are not developed due to the Compressibility of soil.

The soil under the foundation compressed and it cannot bear the pressure applied from the foundation.



(c)

The above figure indicates the failure surface and the load settlement curve of the foundation.

The parameters to decide type of shear failure.

General Shear failure:- This failure is accompained by low strain (<5%) in a soil considerable ϕ (ϕ > 36°) and large "N" (N > 30) and High relative density I_D (I_D > 70)

i.e. strain < 5%

I_d > 70

N > 30

ϕ > 36°

C_u > 100 kPa

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Local/Punching Shear Failure :-

large strain (> 10 to 20%) in a soil with considerably low ϕ ($\phi < 28°$) and low N (N < 5) having low relative density I_D ($I_D \nless 20\%$)

i.e. ~~large strain > 20%~~

~~N > 5~~

~~$I_D \nless 20$, $I_D < 20$~~

i.e. strain > 20%

N < 5

$I_D < 20$

$\phi < 28°$

$C_u < 50$ kPa

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(4) Explain with neat sketch, various types of Shallow foundation? (5)

Ans:- Shallow Foundation :- Foundation is placed immediately lowest part of the Super structure, is termed as Shallow Foundation.

Types of Shallow Foundations are:

1) Strip Footing
2) Spread or Isolated Footing (or) Individual Footing
3) Combined footing
4) Strap (or) Cantilever footing
5) Mat (or) Raft foundations.

(1) Strip Footing:-

a) A strip footing is provided for a load-bearing wall

b) A strip footing is also provided for a row of columns which are so closely spaced that their spread footings overlap (or) nearly touch eachother.

c) In such a case, it is more economical to provide a strip footing than to provide a number of spread footings in one line.

d) A strip footing is also known as Continous footing.

e) Strip footing also prevent the horizontal distortion of Individual foundation and strengthen them.

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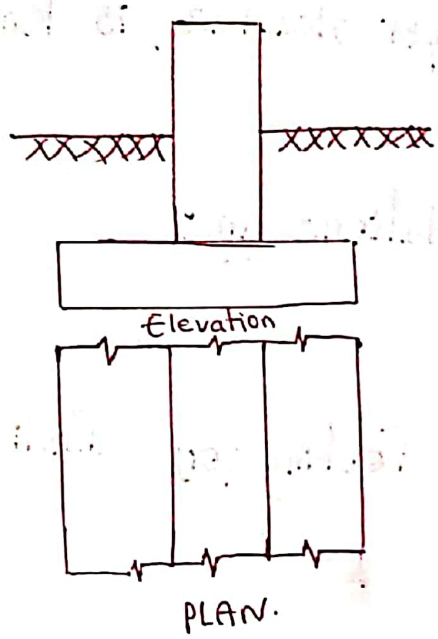
f) In the horizontal direction, the strip footings act as a Continuous carrier under the force influence from the columns/walls.



Strip Footing

2) Spread (or) Isolated Footing (or) Individual Footing

a) A Spread Footing also called as isolated Footing, Pad footing and individual footing is provided to support an individual column.

b) A spread footing is circular, Square (or) Rectangular slab of uniform thickness.

c) Sometimes, it is steeped (or) haunched to spread the load over a large area.

d) Nowadays, Massive foundations are avoided and reinforced spread footing are performed with considerably smaller dimensions due to the use of reinforcement.

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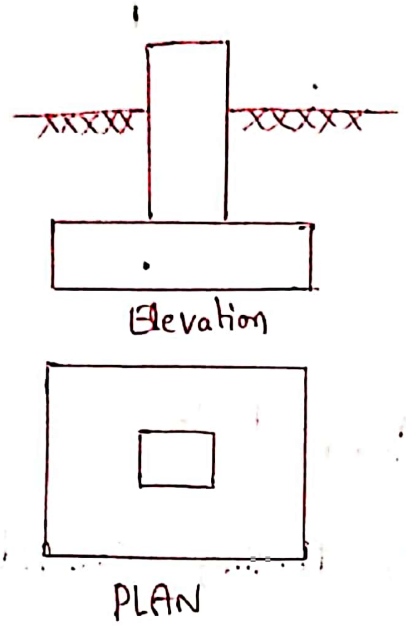
e) The Criterion for the foundation determination of Spread foundation stiffness depends on the soil foundation reactor module and is valid for $K > 0.40$ for Solid foundation i.e $K < 40$ for Flexible foundations.



Spread Footing

③ Combined Footing:

a) A Combined Footing Supports two columns.

b) It is used when the two columns are so close to Each other that their Individual footings would overlap.

c) A combined Footing is also provided when the property line is so close to one column that a spread Footing would be Eccentrically loaded when kept Entirely within the property line.

d) By Combining it with that of an interior column, the load is evenly distributed.

e) A Combined footing may be rectangular (or) trapezoidal in plan

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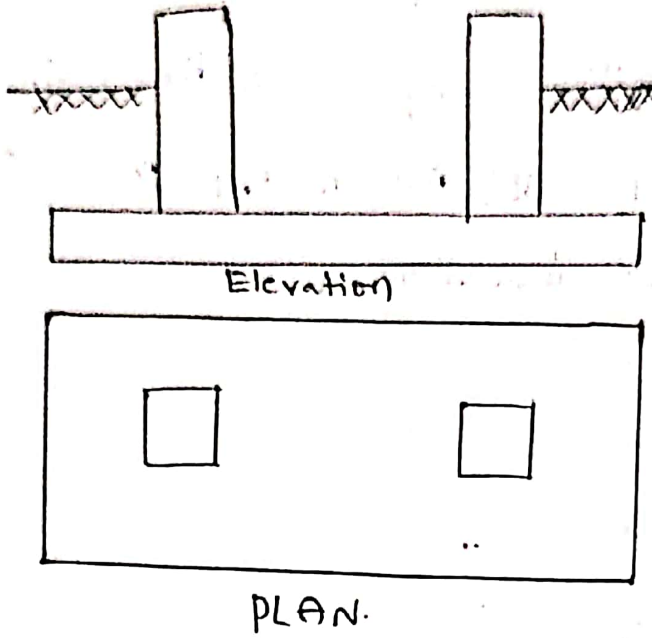

Combined footing.

4) Strap (or) Cantilever Footing

a) A Strap (Cantilever) Footing Consist of two isolated Footing connected with a structural strap (or) lever.

b) The Strap connects the two Footings such that they behave as one unit.

c) The strap is designed as a rigid beam

d) The individual footings are so designed that their Combined line of action passes through the resultant of the total load.

e) a strap footing is more economical than a Combined Footing when the allowable soil pressure is relatively high and the distance betwen the columns is large.

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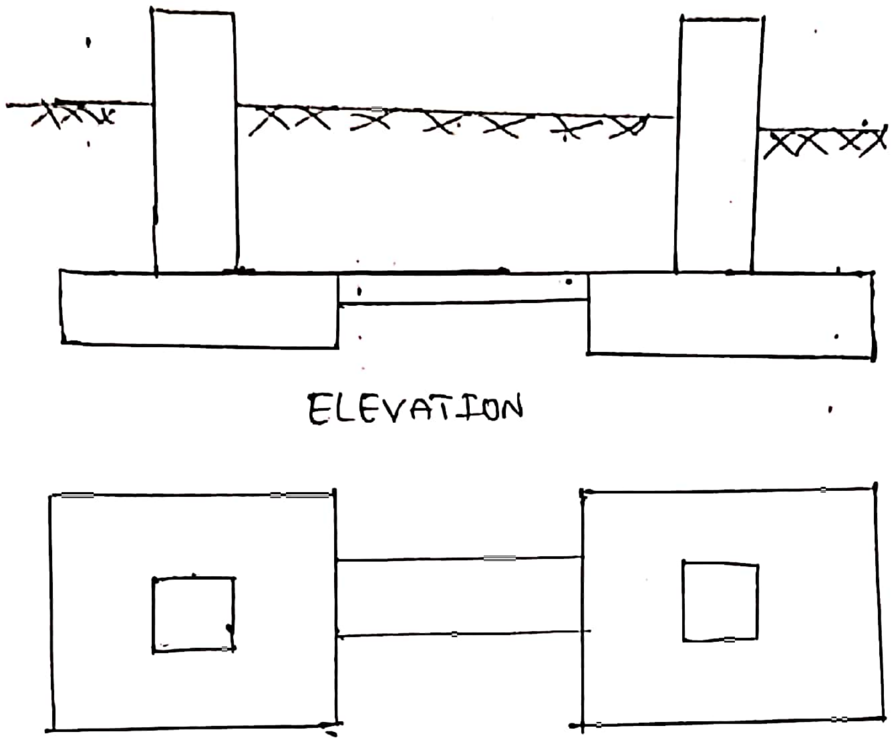

Strap (or) Cantilever footing

⑤ Mat (or) Raft foundation

a) A mat (or) Raft foundation is a large slab supporting a number of columns and walls under the entire structure (or) a large part of the structure.

b) A mat is required when the allowable soil pressure is low (or) where the columns and walls are so close that Individual Footings would overlap (or) nearly touch Eachother

c) Mat foundations are useful in reducing differential settlements on non-homogeneous soils (or) where there is

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Terzaghi Bearing Capacity Theory :-

=> In 1943 Terzaghi's given a Theory to determine the bearing capacity of soil which is extended from prandtl (1920) eqn.

=> Terzaghi made following assumption's for developing bearing capacity eqn to determine ultimate bearing capacity (q_u) for $c-\phi$ soil.

Assumption's :-

i) Soil is semi-infinite, homogeneous & Isotropic.

ii) The Base of footing is Rough.

iii) foundation is Shallow type i.e $\left(\frac{D_f}{B} < 1\right)$.

iv) footing is Continuous / strip footing i.e ($L >> B$) & it makes analysis 2-dimensional along the depth & width.

v) Soil will fail's when it reaches to plastic Equilibrium.

vi) The load is vertical & symmetrical.

vii) The ground surface is horizontal along with foundation

viii) The failure is only by general shear failure.

ix) The stress zone acts only @ Base of footing.

x) The over burden pressure @ foundation level is equivalent to surcharge load i.e ($q_0 = \gamma \times D_f$)

(Here γ = U.W of soil, D_f = Depth of foundation)

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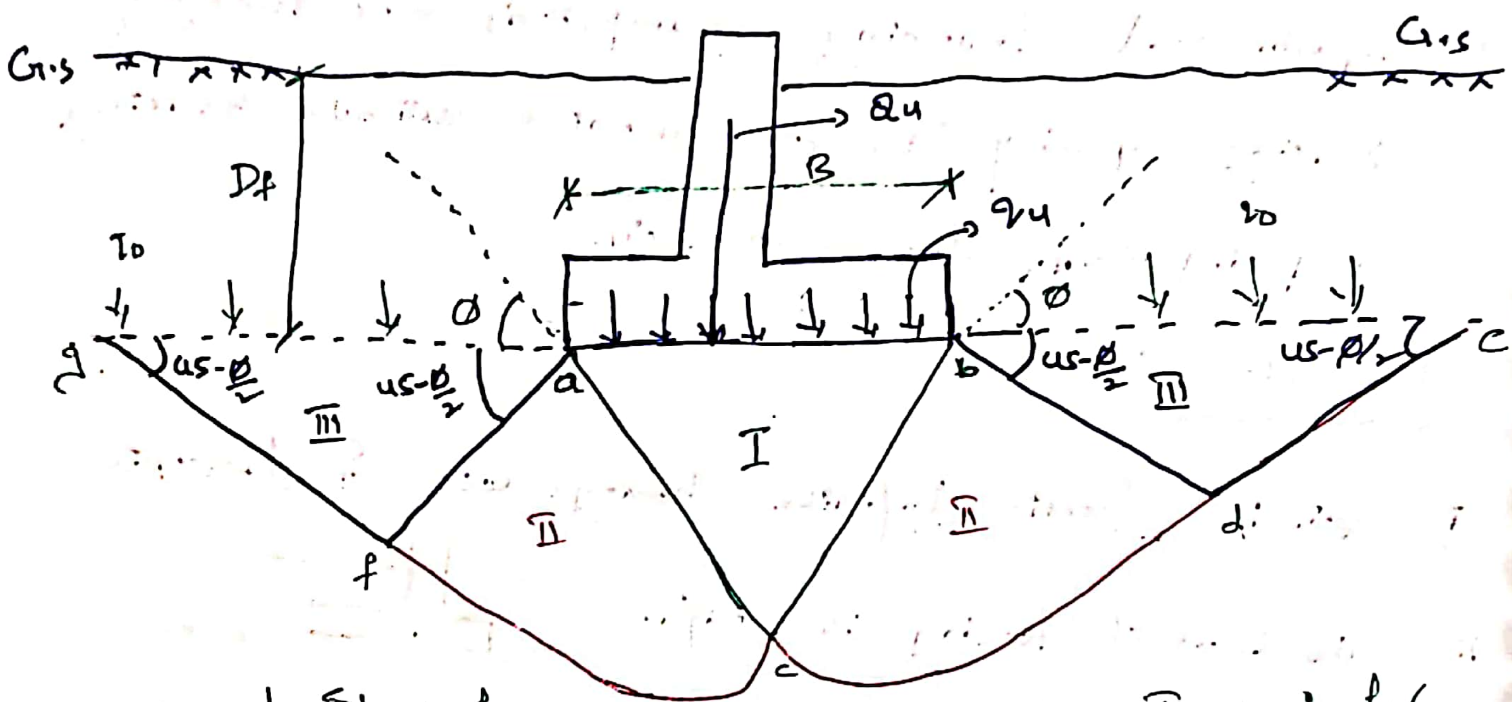
xi) The principle of Superposition is valid

xii) coulomb's law is strictly valid, i.e ($\sigma = c + \sigma \tan\phi$)

failure Mechanism:



general shear failure surface as assumed by Terzaghi for strip footing.

Zone I = Elastic Equilibrium State.

Zone II = radial Shear State.

Zone III = Rankine's passive State.

Here q_u = load per unit area

B = width of footing.

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Ultimate bearing capacity Eqn:-

$$q_u = cN_c + \gamma D_f N_q + \frac{1}{2} \cdot \gamma \cdot B \cdot N_\gamma.$$

$$\boxed{q_u = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma.}$$

Here B = width of footing

D_f = Depth of footing below GL

$\gamma \times D_f$ = Surcharge load @ foundation Level

N_c, N_q, & N_γ = Bearing capacity factor's [depend's on frictional Angle of soil]

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

$N_q = N_\phi e^{\pi \tan\phi}$

$N_\gamma = 1.8 \tan\phi (N_q - 1)$

$N_c = \cot\phi (N_q - 1)$

for purly cohesive soil =
$$\left\{ \begin{array}{l} \phi = 0^\circ \\ N_\phi = 1 \\ N_q = 1 \\ N_\gamma = 0 \end{array} \right.$$

* if the soil is failure in local shear failure then Mobilized parameter's are

$$\boxed{c_m = \frac{2}{3}c}, \quad \boxed{\phi_m = \tan^{-1}\left(\frac{2}{3}\phi\right).}$$

Bearing capacities for Diff soils & footings:-

(i) Cohesive soil - Strip footing:-

$\phi = 0$, $c > 0$, $N_c = 5.7$, $N_\gamma = 0$, $N_q = 1$

$$q_u = cN_c + \gamma D_f N_q + 0.5 \gamma \times B \times N_\gamma.$$

$$= 5.7 N_c + \gamma D_f N_q + 0$$

$$\boxed{q_u = 5.7 N_c + \gamma D_f}$$

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ii) Non- Cohesive Soil - Strip footing :-

$$q_u = cN_c + \gamma D_f N_q + \frac{1}{2} B \gamma N_\gamma$$

here $c = 0$, & $\beta = 0$

$$\boxed{q_u = \gamma D_f N_q + \frac{1}{2} B \times \gamma \times N_\gamma}$$

iii) Cohesive Soil - Square footing :-

$$\boxed{q_u = 1.3\, cN_c + \gamma D_f N_q + 0.4 B \gamma N_\gamma}$$

iv) Non- Cohesive Soil - Square footing :-

$$\boxed{q_u = \gamma D_f N_q + 0.4 B \gamma N_\gamma}$$

v) Cohesive Soil - ~~Circular~~ Rectangular footing :-

$$\boxed{q_u = \left[1 + 0.3 \frac{B}{L}\right] cN_c + \gamma D_f N_q + \left[1 - 0.2 \frac{B}{L}\right] \frac{1}{2} \gamma B N_\gamma}$$

vi) Non- Cohesive Soil - Rectangular footing :-

$$\boxed{q_u = \gamma D_f N_q + \left[1 - 0.2 \frac{B}{L}\right] \times \frac{1}{2} \gamma B N_\gamma}$$

vii) Cohesive Soil - Circular footing :-

$$\boxed{q_u = 1.3\, cN_c + \gamma D_f N_q + 0.3 \gamma B N_\gamma}$$

viii) Non- cohesive Soil - Circular footing :-

$$\boxed{q_u = \gamma D_f N_q + 0.3 \gamma B N_\gamma}$$

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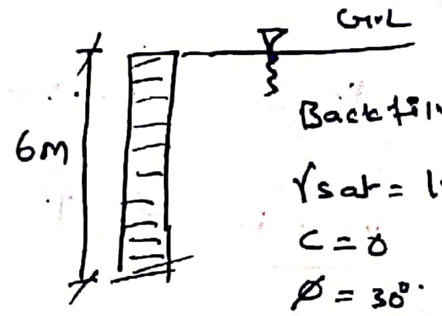
Module-2 Q-7.

(7) A masonry R.W with vertical back has to Retain a backfill of 6m ht. behind it. The G.L is horizontal @ top, & the W.T is upto top of Backfill, assume $\gamma_{sat} = 19\ kN/m^3$, $c=0$, $\phi = 30°$ Solve horizontal Earth pressures (a) wall move's away from Backfill

b) wall move's toword's backfill

c) wall is Rigid.

Soln:- Given Data:-



(a) wall move's away from Back fill:-

So here active Earth pressure is acting. (P_a).

@ active State $k_a = \dfrac{1-\sin\phi}{1+\sin\phi}$

when Back fill is Submerged two types of pressures acts on wal.

(1) active E-P due to submerged wt of soil (P_a)

2) Lateral Earth pressure due to water. (P_w)

So total pressure $\overline{P_a} = P_a + P_w$.

W.K.T $P_a = \frac{1}{2}\gamma k_a H^2$, $P_w = \frac{1}{2}\gamma_w H^2$

So $\overline{P_a} \Rightarrow \frac{1}{2}\gamma_{sat} k_a H^2 + \frac{1}{2}\gamma_w H^2$

W.K.T $\gamma_{sat} = 19$, $k_a = \dfrac{1-\sin\phi}{1+\sin\phi} =$, $H = 6$, $\gamma_w = 9.81$ (Taken)

So $\overline{P_a} = (0.5)(19)(0.33)(6)^2 + (0.5)(9.81)(6)^2$

$\overline{P_a} = 289.44\ kN/m$

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② Wall move's toword's back fill:-

it is in passive condition.

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = 3.$$

So $\bar{P}_p = \frac{1}{2} K_p \cdot \gamma_{sat} \times H^2 + \frac{1}{2} \times \gamma_w \times H^2$

$\Rightarrow 0.5 \times 3 \times 19 \times 6^2 + 0.5 \times 9.81 \times 6^2$

$\bar{P}_p = 1202.58$ kN/m

③ Wall is rigid:-

ⓐ Rest Condition $K_0 = 1 - \sin\phi$

$K_0 = 1 - \sin 30 = 0.5$

So $P_0 = \frac{1}{2} \times K_0 \times \gamma_{sat} \times H^2$

$\Rightarrow 0.5 \times 0.5 \times 19 \times 6^2$

$P_0 \Rightarrow 171$ kN/m

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module - 2

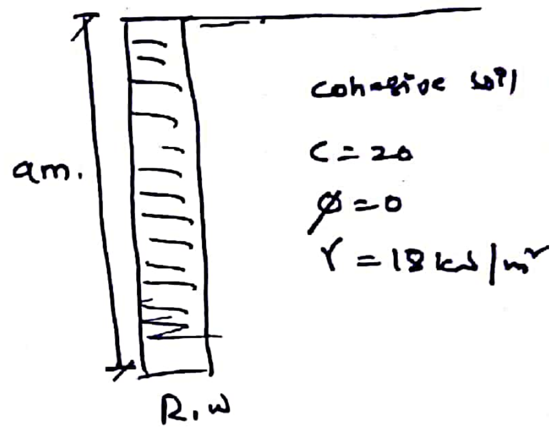
Q 8 : a wall with a smooth vertical back 9 m high supports a purely cohesive soil $c = 20$ kN/m³ & $\gamma = 18$ kN/m² solve

i) Maximum depth of tension crack.

ii) active force before tensile crack

iii) active force after tensile crack occurs.

Soln:-



i) Maximum depth of Tension crack:- (z_c)

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

$$k_a = \frac{1 - \sin 0}{1 + \sin 0} \Rightarrow 1$$

$$Z_c = \frac{2 \times 20}{18 \times 1} \Rightarrow \frac{40}{18} = 2.22$$

ii) active force before tensile crack:-

w.k.T active force before tensile crack is

$$\boxed{P_a = \frac{1}{2} \times \gamma \times H^2 - 2cH}$$

$$\Rightarrow 0.5 \times 18 \times 9^2 - 2 \times 20 \times 9.$$

$$\boxed{P_a = 369 \text{ kN/m.}}$$

iii) active force after Tensile crack:-

active force after Tensile crack is

$$\boxed{P_a = \frac{1}{2}(\gamma H - 2c)(H - z_c)}$$

$$\Rightarrow 0.5(18 \times 9 - 2 \times 20)(9 - 2.22)$$

$$\boxed{P_a = 414.8 \text{ kN/m.}}$$

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