#. Soil Mechanics and foundation Engineering:

*: Soil: (2.1.2)

↳ soil is defined as unconsolidated materials, composed of solid particle produced by disintegration of rocks.

classibication of soil:

↳ Soil classibication is the arrangements of soil into dibberent groups such that the soil in a particular group have similar behavior.

field identibication of soil:

↳ Identibication of soil without detailed lab test is known as field identibication of soil.

↳ identibication can be done by:

1. Coarse grained vs fine grained:

Technique: visual inspection.

criteria: fine particles can't be seen by naked eyes.

↳ If more than 50% of soil particles are visible individually, the soil is coarse grained, otherwise fine grained.

2. Gravel vs sand:

Technique: visual inspection

↳ If greater than 50% of soil particles are coarser than 4.75mm sieve size, soil is gravel, otherwise sand.

3. fine sand vs silt:

Technique: sedimentation analysis (Dispersion test).

criteria: fine sand settles immediately while silt takes some time to settle.

↳ If greater than 50% particles settles immediately, soil is tine grained, otherwise silt.

4. silt vs clay: → (5 marks questions)

Technique: (i) Dilatancy test:

- for silt:
  - ↳ fast water appears on surface at pat while shaking on hand & disappears after compression at mid of pat by finger.
- For clay:
  - ↳ takes relatively longer time for water to appear on surface of pat while shaking and doesnot disappear by compression at mid of pat.

(ii) Dry-strength test:

criteria:

- for silt: dry soil piece can be crumbled easily by finger.
- for clay: hard to crumble the piece of dry soil by finger.

(iii) soapy-feeling in hand:

criteria:

↳ soapy feel in hand after touching soil: silt.
otherwise: clay

(iv) Dispersion test:
or,
~~criteria~~: sedimentation analysis.

criteria:

- If particles settles in about 15 minutes to one hour: silt
- If particles donot settle for hour: clay.

5. Plastic clay vs non-plastic clay:

Technique: Rolling soil in between two palm:

criteria:

ⓐ for plastic clay
↳ can be rolled to finer thread.

ⓑ for Non-plastic clay
↳ crumbled during rolling.

Note: for brick preparation → plastic clay is used.

Classification of soil:

↳ Arrangements of soil into different groups such that the soil in a particular group have similar behaviour:

• Based on lab test results:

① MIT classification system. (outdated)

② Textural classification system → Agricultural engineers

③ Unified soil classification system (USCS) → Mostly used by engineers in Nepal.

④ Indian standard soil classification system (ISSCS)

⑤ British soil classification system (BSCS)

⑥ ASTM soil classification system

⑦ AASTHO soil classification system → Road soil classification (i.e. subgrade soil classification)

Q. Necessity of soil classification system ?

a. Uniformity in profession globally.

b. to understand the soil sameway globally.

c. to classify the soil using the standard symbols.

d. to use same criteria to classify the soil.

Unified soil classification system:

- based on airfield classification system that was developed by A. Casagrande in 1948.
- This classification is based on both grain size and plasticity property of soil and is therefore applicable to any use.

Major Division	Division	Minor Division	Classification criteria	For fines 5-12%. Dual name.	
Coarse grained soil. >50% of soil particles are retained on 0.05 mm sieve (Sieve No: 200)	Gravel: >LF > 50% of soil are retained on 4.75 mm sieve (No. 10 sieve)	Pure gravel: fines < 5%	GW $C_u >$ [illegible], $1 < C_c < 3$	GP-GC	C = clayey
			GP	GP-GM	M = silty
		Gravel with fines: fines > 12%	GM Below A-line	GW-GC	P = poorly graded
			GC Above A-line	GW-GM	W = well graded
	Sand: >LF > 50% of soil particles are passed on 4.75 mm sieve. (No. 10 sieve)	Pure sand: fines < 5%	SW $C_u > 6$, $1 < C_c < 3$	SP-SC	G = gravel
			SP otherwise	SP-SM	S = sand
		Sand with fines: fines > 12%	SM Below A-line	SW-SC	
			SC Above A-line.	SW-SM	
Fine-grained soil. < 50% soil particles are retained on 0.075 mm sieve.	LL > 50%	CH	See plasticity chart.		
		MH			
		OH			
	LL < 50%	CL			
		ML			
		OL			
Highly organic clay: > Peat: Pt	criteria: > By visual inspection, smell etc. > Highly compressible in nature.				

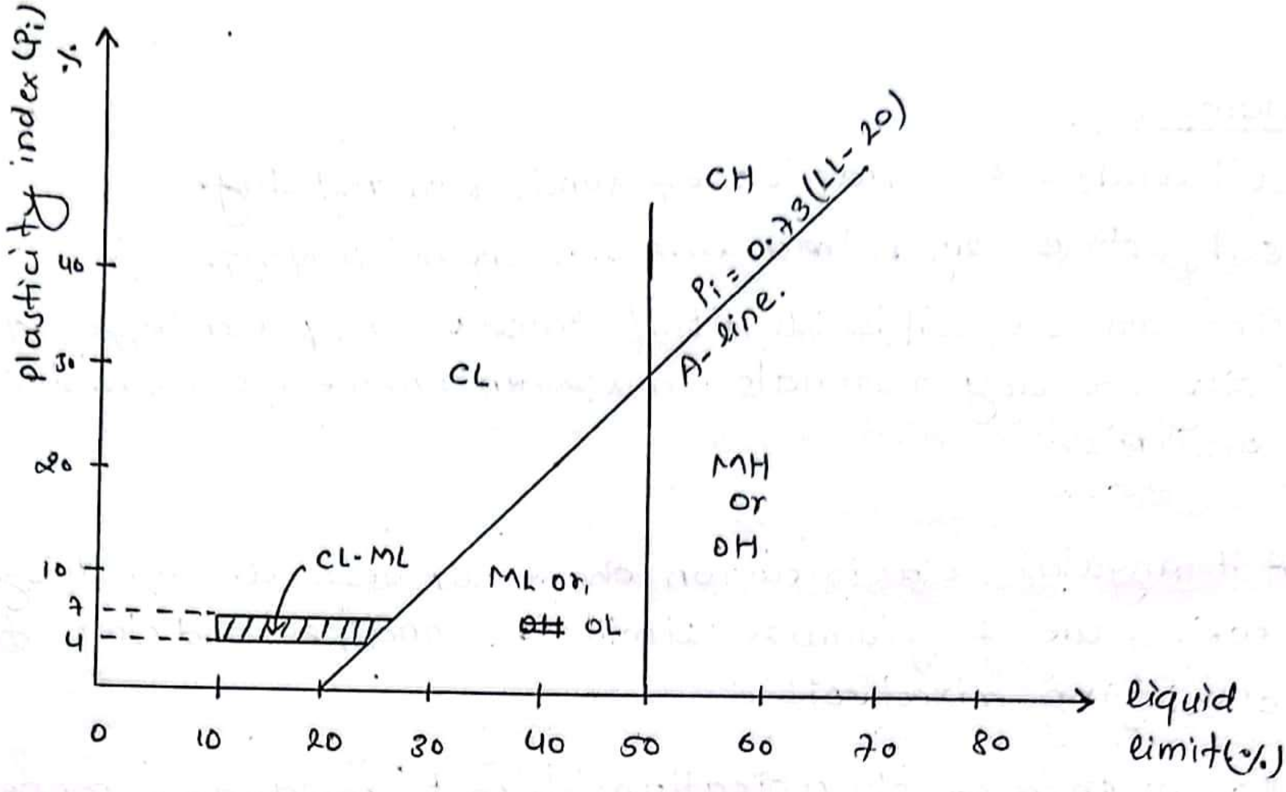


fig: plasticity chart (USCS).

Textural classification:

<u>Assumption:</u>

(i) soil consists of fraction of sand, silts and clay.

(ii) soil particles larger than 2mm size are not present.

(iii) The name of soil is identified based on the percentage of sand, silt and clay materials in a given sample by mechanical analysis.

↳ A triangular classification chart has been developed by making use of grainsize limit for sand, silt and clay for classifying mixed soil.

↳ The method of classification doesnot reveal any properties of the soil other than grain size distribution.

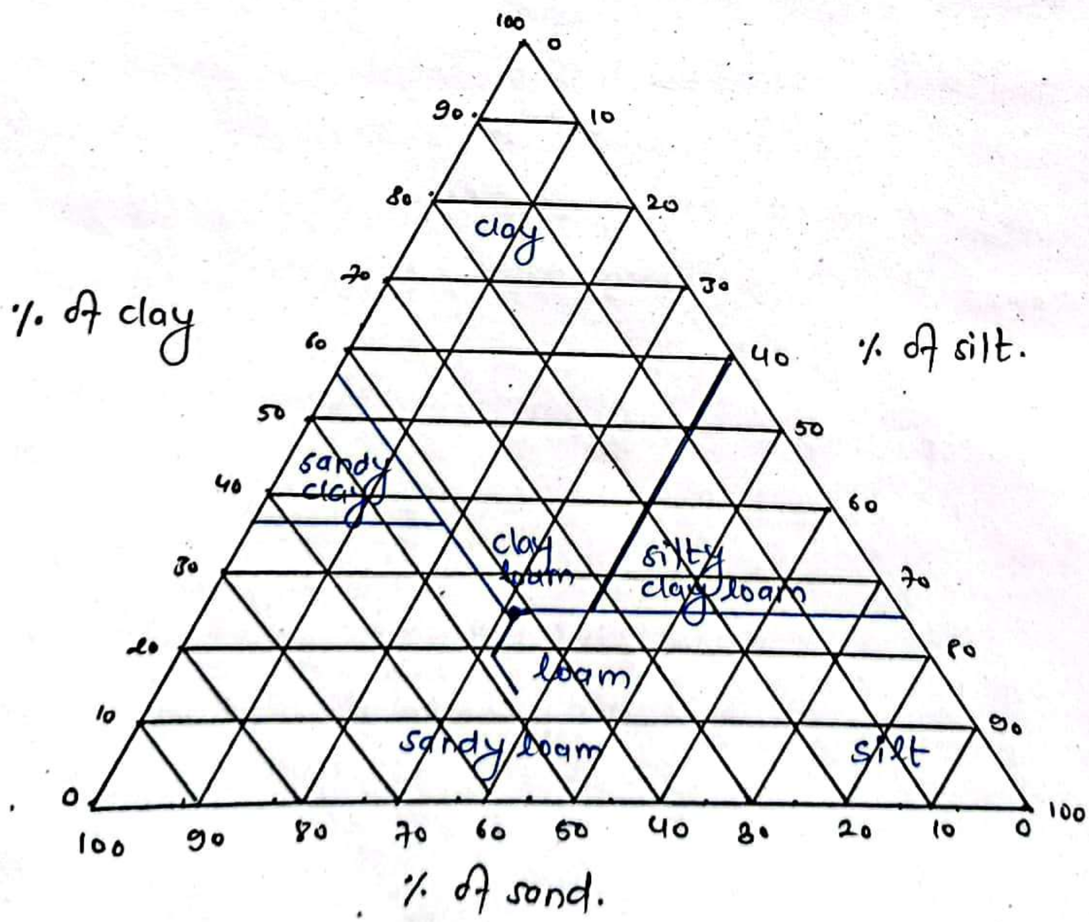
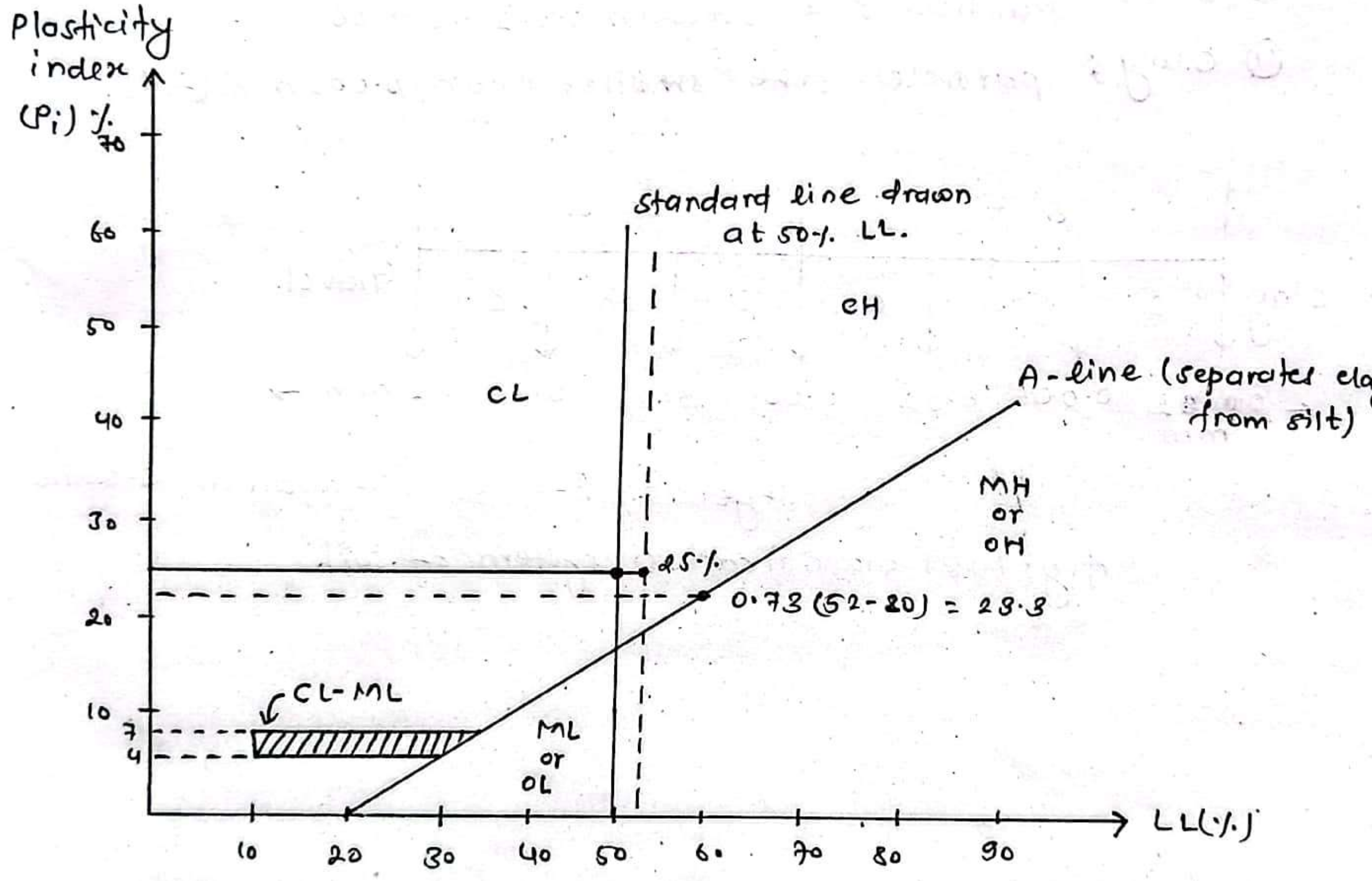


fig: textural classification.

Q. lab test result reveals 35% of soil particles retained on 0.075 mm sieve, LL = 52%, PL = 27%, classify the soil according to USCS.

solution

Since 65% of soil passed on 0.075 mm sieve, the soil is fine grained soil according to USCS.



$P_i = LL - PL$

$= 52\% - 27\%$

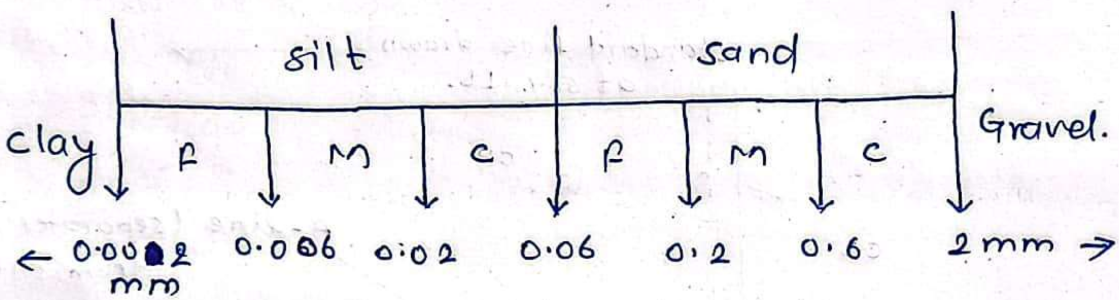
$= 25\%$

Since LL > 50% and lies above A-line, our soil is CH.

i.e. highly plastic inorganic clay.

MIT classification:

- This system of classification was developed by prof. G Gilboy at massachussets institute of technology, USA.
- divided the soil into four groups.
  1. Gravel; particle size greater than 2mm.
  2. Sand; particle size between 0.06 mm to 2mm.
  3. Silt; particle size between 0.002 to 0.06 mm.
  4. clay; particles size smaller than 0.002mm (2μ).



legends: F- fine M- medium C- coarse

fig: MIT classification system of soil.

Permeability of soil:- (2.1.2)

- Permeability of soil is the property by virtue of which soil mass allows water to flow through it.
- It is an engineering property, which is required to be determined for study of soil engineering problems involving flow of water through soils, such as seepage, through dam body & settlement of foundation.
- It is measured in terms of coefficient known as coefficient of permeability or hydraulic conductivity.
- Expressed in terms of cm/sec or m/day or ft./day.

*! Darcy's law: (valid only when reynold's number $Re < 1$)

- According to Darcy's law, for laminar flow condition in a saturated soil, the velocity of flow (v) is directly proportional to hydraulic gradient (i).

i.e $v \propto i$

$\therefore v = ki$

where; k = coeff. of permeability

if $i = 1$ then,

$$\boxed{K = V}$$

Thus, coefficient of permeability or simply permeability is defined as the average velocity of flow that will occur through total c/s area of soil under unit hydraulic gradient.

further, $q = kiA$

where; q = rate of flow or discharge per unit time

A = total c/s area of flow $\perp$er to direction of flow.

Discharge velocity (V) and seepage velocity (V_s):

Discharge velocity = Theoretical velocity

Seepage velocity = Actual velocity

*: Seepage velocity: (V_s)

- Actual velocity of water through soil void (only) is called seepage velocity (V_s).
- It is the rate of discharge percolating per unit c/s area of voids perpendicular to direction of flow.

*: Discharge velocity: (V)

- Velocity obtained on the basis of gross area is known as discharge velocity.
- It is the rate of discharge of soil water through the area of c/s of soil sample.

Since the soil contains solid particles as well as voids and flow takes place only through void, so seepage velocity is always greater than discharge velocity.

From Darcy's law:

- Discharge velocity, $V = \frac{Q}{A}$ or $Q = AV$
- Seepage velocity, $V_s = \frac{Q}{A_n}$ or, $Q = V_s A_n$

; where A_n = Area of void (net area)

From continuity equation:

$$Q = AV = A_n V_s$$

$$\text{or } V_s = V\left(\frac{A}{A_n}\right)$$

$$= V\left(\frac{A \times L}{A_n \times L}\right)$$

$$= V \times \left(\frac{V}{V_v}\right)$$

; V = volume

V_n = volume of void

or, $V_s = V \times \dfrac{1}{\left(\dfrac{V_v}{V}\right)}$

or, $V_s = V \times \dfrac{1}{\eta}$; where η = porosity

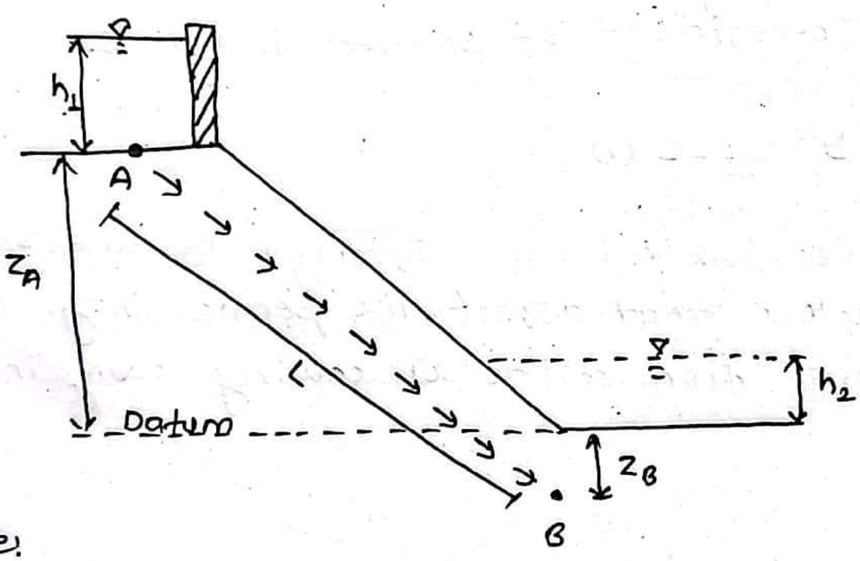
$\therefore\ V_s = \dfrac{V}{\eta}$ ---------- (*)

Darcy's Law:

↳ Darcy law states that "the velocity of water is directly proportional to the hydraulic gradient developed. i.e $v \propto i$

i.e $v = ki$

; i = hydraulic gradient developed. $= \dfrac{h}{l}$.



Note:

Darcy law is applicable for discharge velocity only.

for seepage velocity,

$V_s = K_0 * i$

; where K_0 = coeff. of percolation.

for point A:

elevation head $(h_e) = z_A$

pressure head $(h_p) = h_1$

velocity head $(h_v) = \dfrac{v^2}{2g}$ (very low velocity)

Total head $(h_A) = z_A + h_1$

for point B,

elevation head $(h_e) = -z_B$

pressure head $(h_p) = h_2$

velocity head $(h_v) = 0$

Total head $(h_B) = -z_B + h_2$

Head difference $(h) = h_A - h_B$

$= (h_1 - h_2) + (z_A + z_B)$

↳ Actually in soil mass, seepage velocity occurs.

Factors affecting permeability of soil:-

↳ The general equ^n for coefficient of permeability is:

$$K = C\left(\frac{\gamma_w}{\mu}\right)\left(\frac{e^3}{1+e}\right)D^2 \quad ---- (i)$$

Q. Permeability of soil varies from soil to soil, Justify the statement:

↳ There are numerous factors that affect the permeability of soil. and these factors varies from soil to soil causing change in permeability of soil.

General equ^n:

$$K = C\left(\frac{\gamma_w}{\mu}\right)\left(\frac{e^3}{1+e}\right)D^2 \quad ---- (A)$$

(I) size of soil particles.
(II) shape of soil particles.
(III) void ratio
(IV) viscosity of water
(V) temperature
(VI) entrapped air pockets.
(VII) impurities

(viii) Degree of saturation of soil.
(ix) Adsorbed water.

(i) Size of soil particles:

↳ coarser the soil, higher the permeability.

Hazen's eqn:

$$K = 100\, D_{10}^2$$

; K in cm/sec.

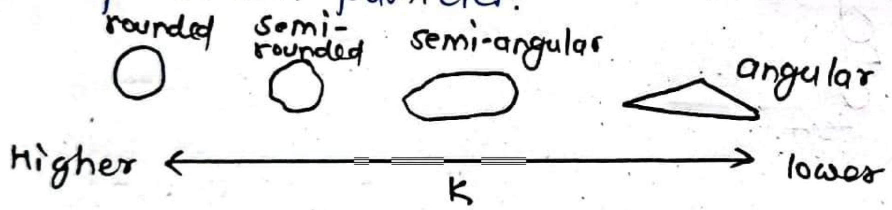
D_{10} = size of soil particles related to 10% finer.

(ii) void ratio:

↳ higher the void ratio, more porous the soil.

$$K \propto \frac{e^3}{1+e}$$

(iii) shape of soil particles:



(iv) viscosity of water: (μ)

$$K \propto \frac{1}{\mu}$$

i.e as μ increases K decreases.

(v) Temperature: (t)

$K \propto t$ i.e. permeability increases as viscosity decreases with temperature.

$$\frac{K_{27}}{K_t} = \frac{\mu_t}{\mu_{27}}$$

(vi) Entrapped air pockets:

↳ obstructs the flow of water.

i.e. K decreases.

(viii) Adsorbed water:

↳ decreases the permeability as it reduces the free area to flow water.

(ix) Degree of saturation:

↳ lower the degree of saturation, lower the permeability and vice-versa.

Determination of coefficient of permeability:

↳ coefficient of permeability can be determined by following methods:

1. Laboratory method (for earthen dam, canal, etc.).
   a. constant head permeability test.
   b. falling head permeability test.
   c. capillary permeability test.
2. Field method
   a. Pumping out test. → unconfined aquifer / confined aquifer
   b. Pumping in test (borehole test)

*: laboratory test:

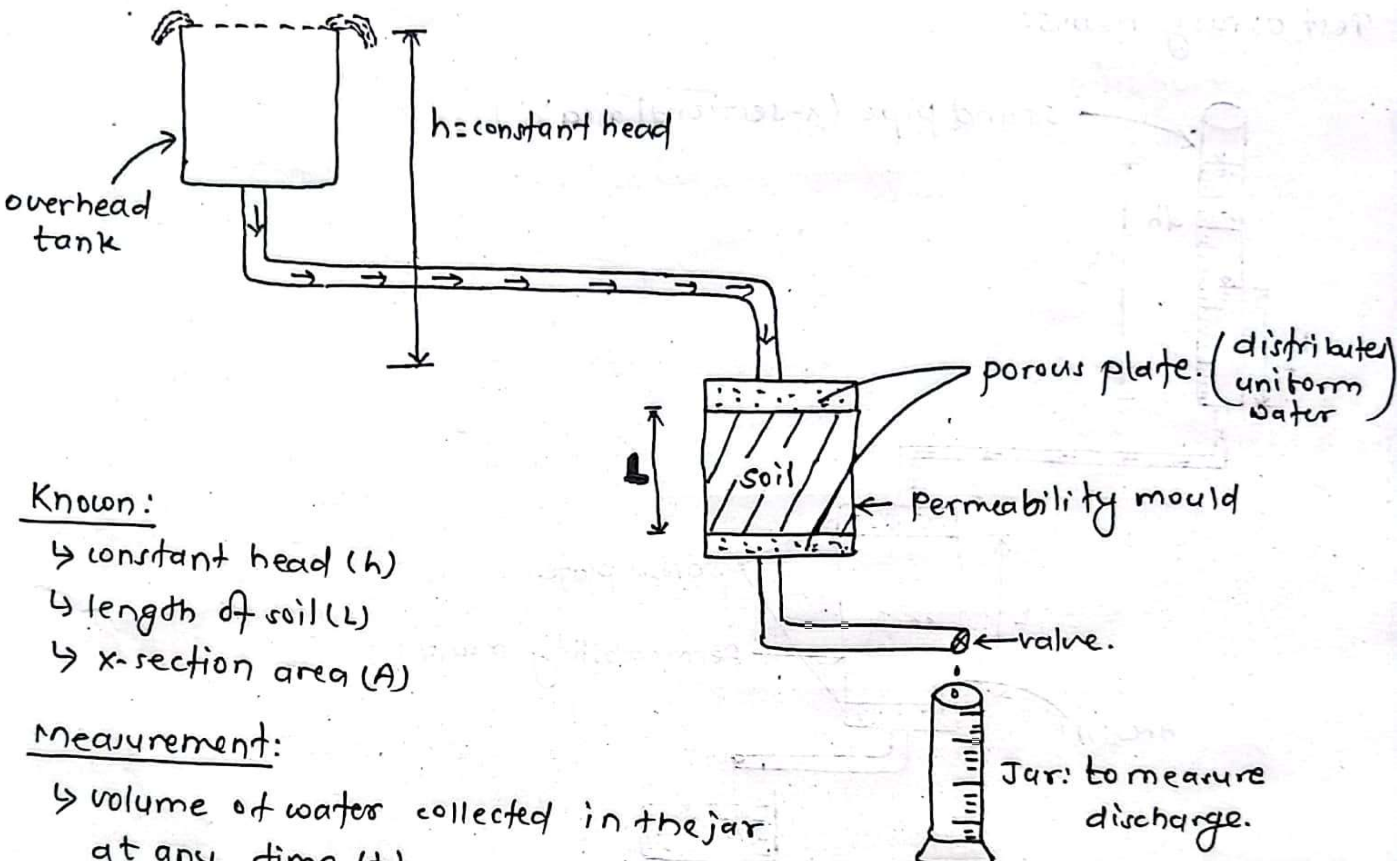
a. constant head method:

Use of the method:

↳ for coarse grained soil.

eg: gravel and sand.

Test arrangement:



Known:

- constant head (h)
- length of soil (L)
- x-section area (A)

Measurement:

- volume of water collected in the jar at any time (t).

$$q = \frac{V}{t}$$

from Darcy's law:

$$q = kiA$$

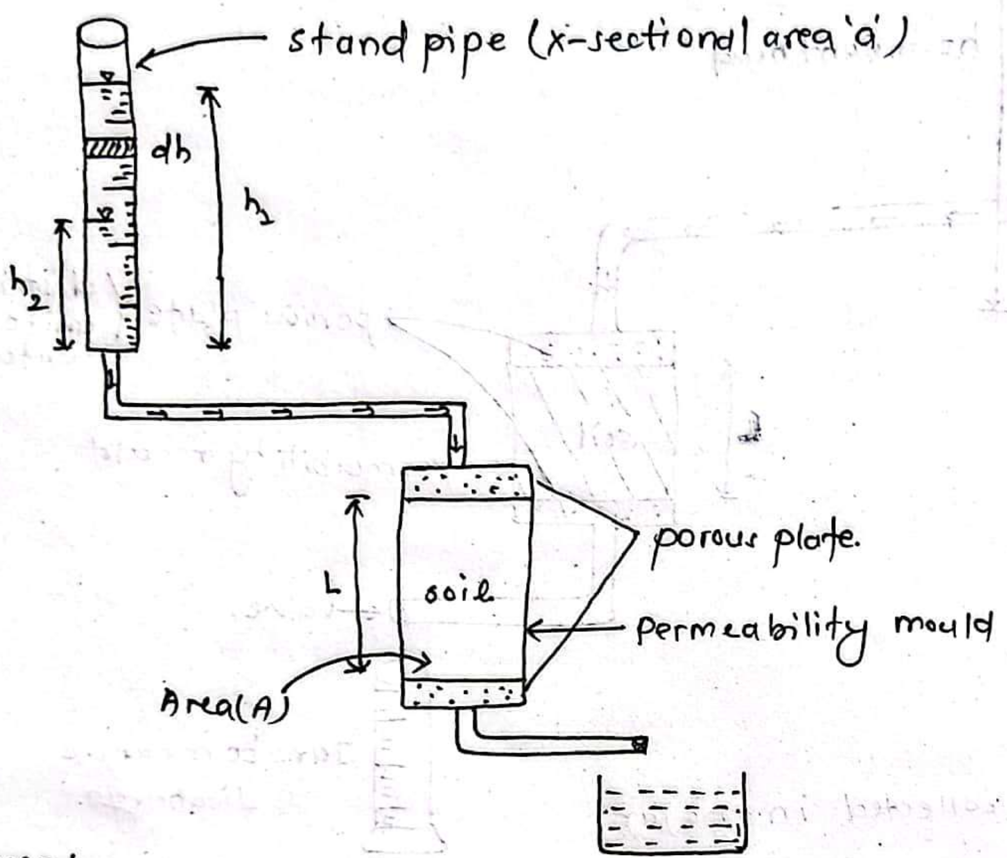
$$= k \times \frac{h}{L} \times A.$$

$$k = \frac{q \times L}{h \times A} \quad \text{------} \quad (*)$$

Use of method: for coarse grained soil.
eg: gravel and sand.

b. Falling head method:

Test arrangements:



Known:

- length of soil (L)
- c/s area of soil (A)
- c/s area of stand pipe (a).

Measurement:

- time 't' required to fall water from h_1 height to h_2 height in stand pipe.

from Darcy's law:

$$q = kiA$$

$$\frac{dv}{dt} = k \times \frac{h}{L} \times A$$

$$-a \times \frac{dh}{dt} = k \times \frac{h}{L} \times A$$

$$-L \times a \times \frac{dh}{dt} = k \times h \times A$$

or, $$-La \int_{h_1}^{h_2} \frac{dh}{h} = kA \int_1^2 dt$$

or, $$-La \times \ln\left(\frac{h_2}{h_1}\right) = kA(t_2 - t_1)$$

or, $$K = \frac{-L \times a \times \ln\left(\frac{h_2}{h_1}\right)}{A \times t}$$

$$\text{or, } K = \frac{L \times a \times \ln\left(\frac{h_1}{h_2}\right)}{A \times t}$$

$$\therefore K = 2.303 \times \frac{a \times L}{A \times t} \log\left(\frac{h_1}{h_2}\right) \text{ ------ } (*)$$

Use of the method:-

↳ for finegrained soil:

eg: silt and clay.

Q. During variable head permeability test, 45 sec time required to fall 500mm height to 200 mm height at stand pipe. Calculate the time to fall another height of 100mm.

Solution:

$$K = 2.303 \frac{a \times L}{A \times t} \log\left(\frac{h_1}{h_2}\right)$$

so,

$$K = 2.303 \times \frac{aL}{At} \log\left(\frac{500}{200}\right) \text{ ---- } (I)$$

$$K = 2.303 \frac{a \times L}{A \times t} \log\left(\frac{200}{100}\right) \text{ ---- } (II)$$

solving equ^n (I) and (II).

$$2.303 \frac{a \times L}{A \times t_1} \log\left(\frac{500}{200}\right) = 2.303 \times \frac{a \times L}{A \times t} \log\left(\frac{200}{100}\right)$$

$$\frac{1}{t_1} \times \log\left(\frac{500}{200}\right) = \frac{1}{t} \times \log\left(\frac{200}{100}\right)$$

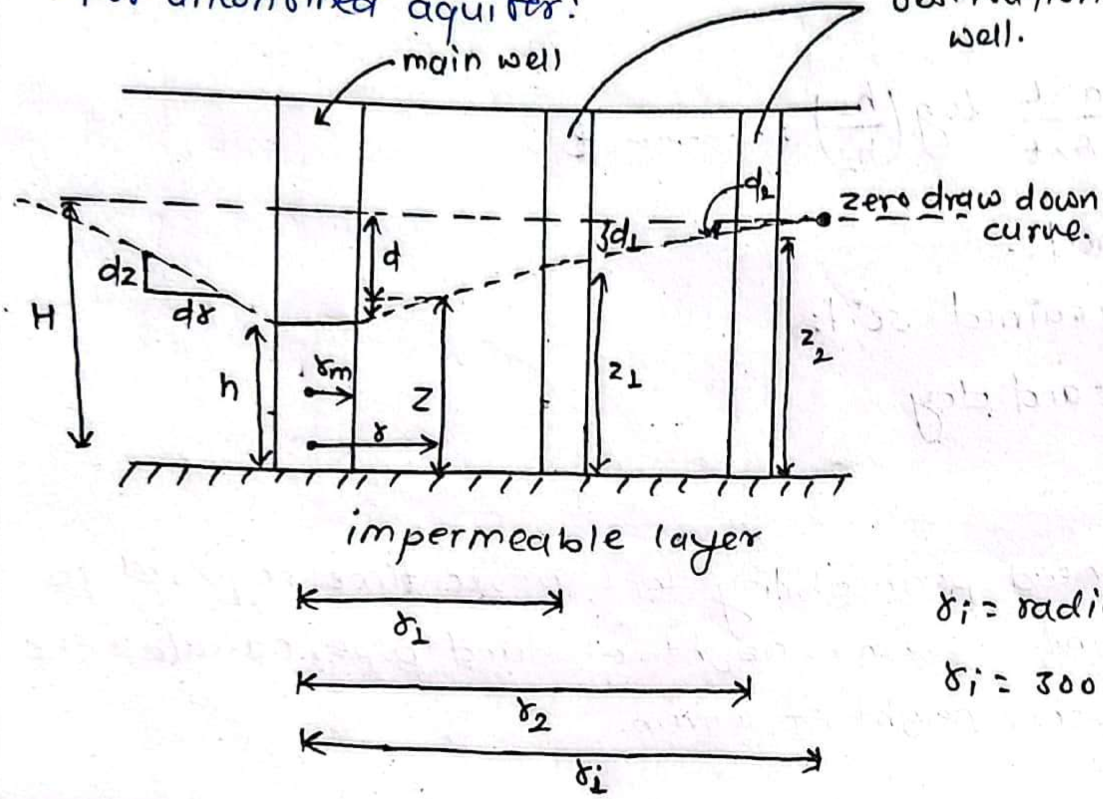
$$\frac{1}{45} \times \log\left(\frac{500}{200}\right) = \frac{1}{t} \times \log\left(\frac{200}{100}\right)$$

$$\therefore t = 34.041 \text{ sec.}$$

*: Field method:

a. Pumping out test:

- For unconfined aquifer:



r_i = radius of influence

$r_i = 3000\, d\sqrt{K}$

; d is in meter.

Known:

- height of water level (H)
- radial distance of observation well r_1 and r_2.
- radius of main well (r_m).

Measurement:

- constant discharge (q) from main well.
- drawdown (d, d_1, d_2)

From Darcy's law:

$$q = kiA$$

$$q = K \times \frac{dz}{dr} \times (2\pi r \times z)$$

$$q \times \frac{dr}{r} = K \times 2\pi \times z\,dz$$

$$q \times \int_{r_1}^{r_2} \frac{dr}{r} = K \times 2\pi \times \int_{z_1}^{z_2} z\,dz$$

$$q \times \ln\left(\frac{r_2}{r_1}\right) = K \times \cancel{2}\pi \times \frac{(z_2^2 - z_1^2)}{\cancel{2}}$$

$$K = \frac{q \times \ln\left(\frac{r_2}{r_1}\right)}{\pi (z_2^2 - z_1^2)}$$

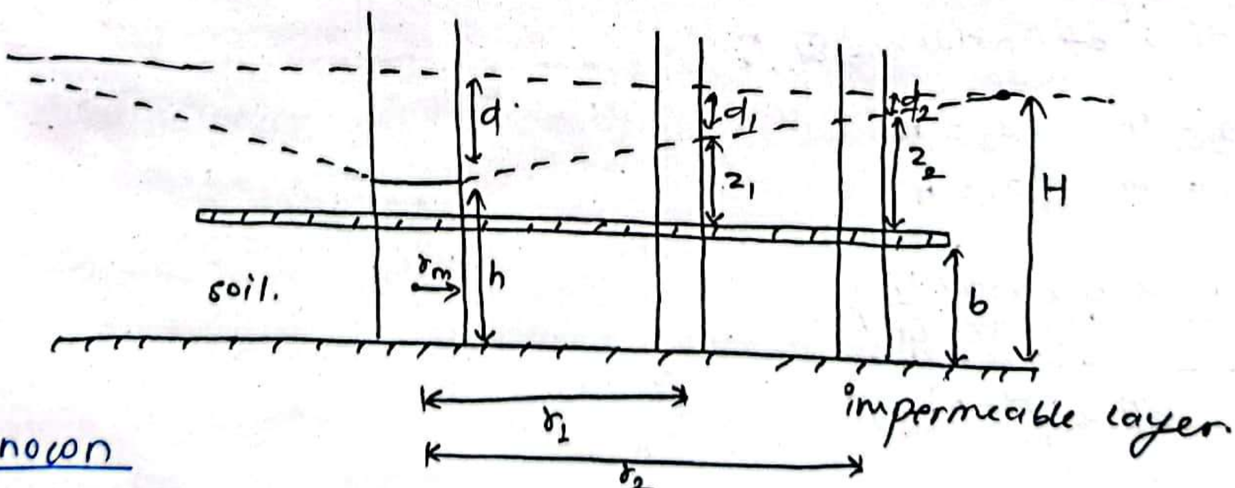
$$\therefore K = \frac{2.303\, q \log\left(\frac{r_2}{r_1}\right)}{\pi (z_2^2 - z_1^2)} \quad ----- (*)$$

For radius of influence,

$$K = \frac{2.303\, q \times \log\left(\frac{r_i}{r_m}\right)}{\pi \times (H^2 - h^2)}$$; r_i = radius of influence (zero-draw down).

~~8. Factors affecting permeability of soils~~

6. confined aquifer:



Known

- Height of water (H)
- radial distance of observation well (r_1 & r_2)
- radius of main well (r_m)
- spacing between two rock layer, b.
-

Measurements:

- constant discharge 'q' from well.
- draw down: d, d_1, d_2

from darcy's low

$$q = kiA$$

$$q = k \times \frac{dz}{dr} \times (2\pi r b)$$

$$q \times \frac{dr}{r} = k \times 2\pi b \times dz$$

$$q \times \int_{r_1}^{r_2} \frac{dr}{r} = k \times 2\pi b \times \int_{z_1}^{z_2} dz$$

$$q \times \ln\left(\frac{r_2}{r_1}\right) = k \times 2\pi b \times (z_2 - z_1)$$

$$k = \frac{2.303\, q \log\left(\frac{r_2}{r_1}\right)}{2\pi b (z_2 - z_1)} \quad \text{------} \; (*)$$

for radius of influence, r_i

$r_2 = r_i \qquad z_2 = H$

$r_1 = r_m \qquad z_1 = h$

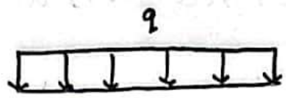
$$k = \frac{2.303 \times q \times \log\left(\frac{r_i}{r_m}\right)}{2\pi b (H - h)}$$

2.1.3 Effective stress:

* Types of stress on soil:

a. Surcharge:

↳ stress due to dam, reservior, canal, embankment, swimming pool and other structures.



b. Overburden pressure:

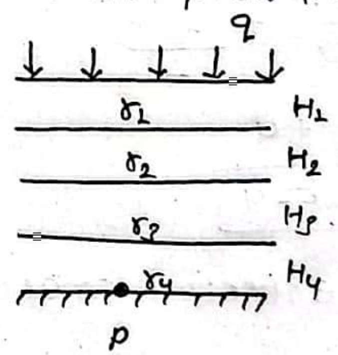
↳ stress due to soil layers above the point.

$$= \Sigma \gamma H$$

c. Total stress (σ):

↳ sum of stress due to surcharge and over burden pressure..

$$\sigma = q + \Sigma \gamma H$$



At point, P

Total stress (σ) $= q + \gamma_1 H_1 + \gamma_2 H_2 + \gamma_3 H_3 + \gamma_4 H_4$

d. Porewater pressure (u):

↳ stress transferred through water to water contact in void is known as porewater pressure.

Mathematically, $u = \gamma_w h_w$

↳ It is also known as neutral stress as it doesnot have shear stress.

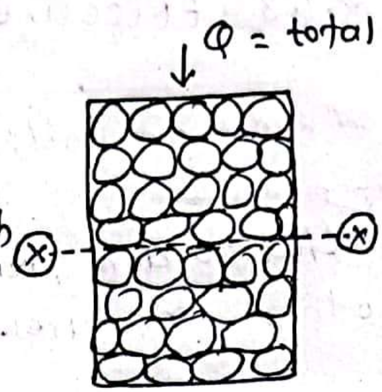
e. Effective stress ($\bar{\sigma}$ or σ'):

↳ The stress transferred through soil particles contact i.e. soil skeleton is known as effective stress.

↳ shear strength of a soil is a function of effective stress.

- It is known as effective stress since this stress is responsible for decrease in void ratio and increase in frictional resistance of soil mass.

Principle of effective stress:



N = Normal load transferred through particle to particle contact.

At x-x: surface:

Q = load due to particle to particle contact + load due to water to water contact.

$Q = \Sigma N + u \times A$

; u = pore water pressure.
A = area of c/s of soil.

$\frac{Q}{A} = \frac{\Sigma N}{A} + \frac{u \times A}{A}$

$\sigma = \bar{\sigma} + u$

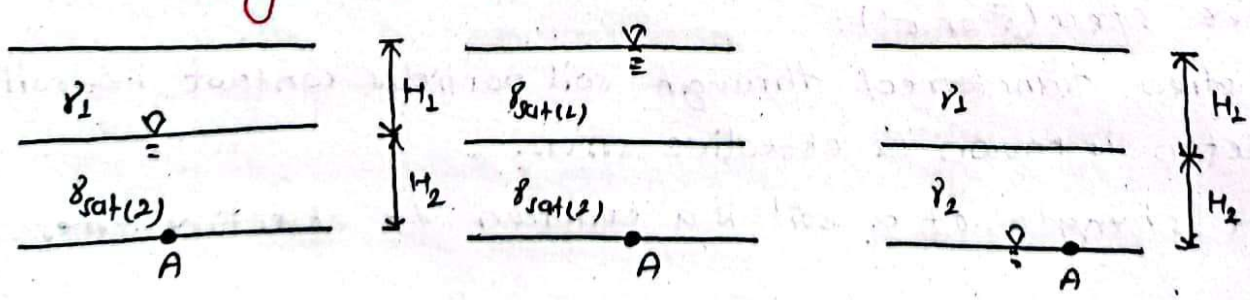
$\boxed{\therefore \bar{\sigma} = \sigma - u}$

i.e effective stress = Total stress - porewater pressure.

Factors affecting effective stress:-

a. position of groundwater table.
b. vertical flow of water
c. capillary rise
d. surcharge applied (q).

a. Position of groundwater table:



case (i) case (ii) case (iii).

$\sigma = \gamma_1 H_1 + \gamma_{sat(2)} H_2$ $\sigma = \gamma_{sat(1)} H_1 + \gamma_{sat(2)} H_2$ $\sigma = \gamma_1 H_1 + \gamma_2 H_2$

$u = \gamma_w H_2$ $u = \gamma_w (H_1 + H_2)$ $u = 0$

$\bar{\sigma} = \gamma_1 H_1 + \gamma_2' H_2$ $\bar{\sigma} = \gamma_1' H_1 + \gamma_2' H_2$ $\bar{\sigma} = \gamma_1 H_1 + \gamma_2 H_2$

; where $\gamma_2' = \gamma_{sat(2)} - \gamma_w$ & $\gamma_1' = \gamma_{sat(1)} - \gamma_w$

↑ ↑

sub-merged unit weight:

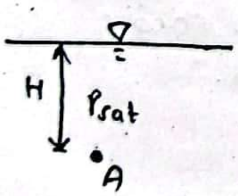
Note:
for same soil layer:
$\gamma_{sat} > \gamma > \gamma_{dry} > \gamma_{sub}$

- Comparing first and second case: (Rise of ground water table)
  ↳ Rise of ground water table makes reduction of effective stress

- comparing first and third case : (Fall of GWT)
  ↳ Fall of GWT. makes increase in effective stress.

6. vertical flow of water:

case : i



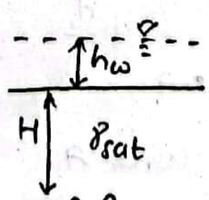
At point A:

$\sigma = \gamma_{sat} \times H$

$u = \gamma_w \times H$

$\bar{\sigma} = \gamma_{sub} \times H$

case : ii



At point A

$\sigma = \gamma_w h_w + \gamma_{sat} \times H$

$u = \gamma_w (h_w + H)$

$\bar{\sigma} = \gamma_{sub} \times H.$

↳ Although effective stress seems equal,
During flow of water through soil mass, water acts certain force in soil to overcome resistance of soil particle, the force is known as seepage force.

- for downward flow: seepage force acts downward so effective stress increases by value of seepage force.

- for upward flow: seepage force acts upward so effective stress decreases by value of seepage force.

c. capillary rise:

↳ negative porewater pressure is developed within capillary rise zone due to suction pressure of surface tension force.

stress $\bar{\sigma} = \sigma - u$

$\bar{\sigma} = \sigma - (-\gamma_w h_c)$

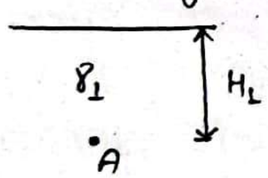
$\therefore \bar{\sigma} = \sigma + \gamma_w h_c$

i.e. effective stress increases with capillary rise zone.

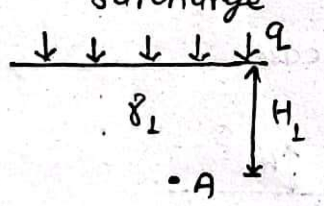
d. surcharge applied:

↳ increase in surcharge increases effective stress and vice-versa.

No surcharge



surcharge



(Note:) Water has infinite compressive strength and zero shear strength.

At point A:

$\sigma = \gamma_1 H_1$

$u = 0$

$\therefore \bar{\sigma} = \gamma_1 H_1$

At point A:

$\sigma = q + \gamma_1 H_1$

$u = 0$

$\therefore \bar{\sigma} = q + \gamma_1 H_1$

Note: q should not exceed capacity of soil. If exceed soil failure takes place.

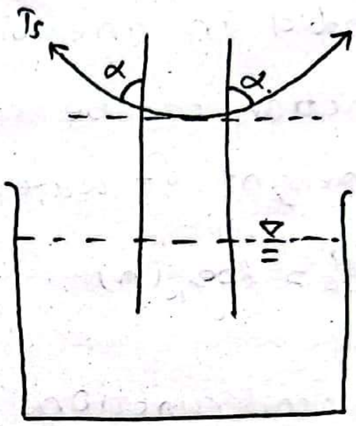
Capillary phenomena in soils:

↳ The void or pores of a soil mass, acts as a series of capillary tube extending vertically above the water table. The rise of water in soil pores is caused due to existence of surface tension, which pulls up the water against the downward gravitational force.

T_s = surface tension force.

↳ adhesive force between tube surface and water molecule.

↳ height of water rises upto equilibrium condition.

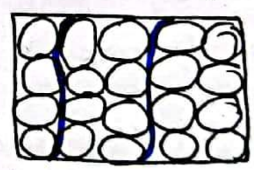


Weight of water = vertical component of tension force.

$$\frac{\pi d^2}{4} \times h_c \times \gamma_w = T_s \times \pi d \times \cos\alpha$$

$$h_c = \frac{4T_s \cos\alpha}{\gamma_w d}$$

i.e. $h_c \propto \frac{1}{d}$.



Tubes formed by series of connected voids.

Q. Why capillary rise phenomena is seen most effective in terai region of Nepal?

↳ In soil mass:

- size of tube formed depends on size of voids.
- size of voids depends on size of soil particles.
- smaller the soil particles, smaller the voids formed.
- smaller the void, smaller the tube formed.

so high capillary rise in smaller soil particles i.e. finer soil. (i.e. cases in terai region).

Soil sunction:

↳ Soil sunction is defined as the state of soil when it is under reduced pressure.

↳ It is the state of negative porewater created by capillary attraction in fine soils and unsaturated soils.

Measurement:

↳ It is measured in terms of height of the water column (h) suspended in the soil.

↳ Soil sunction can be represented as a common logarithm of the height of water column (h) substituted in cm.

$$P_F = \log_{10}(h)$$

*: Features of soil sunction:

(i) Formed by complex energy state that is created by the interaction between the soil, water and air present in the soil, which create negative pressure called soil sunction.

(ii) Soil sunction is studied under unsaturated soil as compared to saturated soil.

*: Factors affecting soil sunction are:

(i) Particle size of soil grain.
(ii) History of drying and wetting cycle.
(iii) Angle of contact.
(iv) Soil structure.
(v) Temperature of soil.
(vi) Water content of soil.
(vii) Dissolved salt in soil.
(viii) Denseness of soil.

Quick sand condition:

↳ In upward flow value of effective decreases by seepage force

↳ When a upward flow is increased, a stage is reached at which effective stress is reduced to zero. The condition so developed is known as quick sand condition.

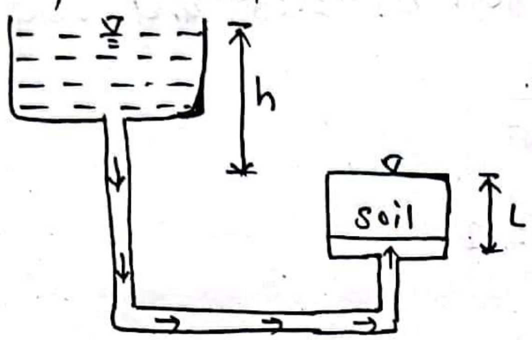


fig: upward flow of water causing quick sand condition.

Hydraulic condition:

↳ Continuous upward flow such that effective stress completely balanced by seepage force. Then the resultant effective stress becomes zero(0).

*! Note:

↳ Quick sand condition is achieved when head causing the flow equals to the thickness of specimen.

Proof:

from above figure

$\sigma = \gamma_{sat} \times L$

$u = \gamma_w (h + L)$

$\bar{\sigma} = \gamma_{sat} \times L - \gamma_w (h + L)$

$\bar{\sigma} = \gamma_{sub} \times L - \gamma_w h$

for quick sand condition,

$\bar{\sigma} = 0$

$0 = \gamma_{sub} \times L - \gamma_w \times h$

$$\frac{h}{L} = \frac{\gamma_{sub}}{\gamma_w}$$

Also,

$$\frac{h}{L} = i \;\&\; \frac{G-1}{1+e} = \frac{\gamma_{sub}}{\gamma_w}$$

Taking sp. gravity of soil as 2.65 and void ratio 0.65.

value of $i = \frac{2.65-1}{1+0.65}$

$$i = \frac{1.65}{1.65} = 1$$

So, $\frac{h}{L} = 1$

$\Rightarrow h = L$ ------- (*)

Summary:

(i) Quick sand is not a special type of soil. It is hydraulic condition.

(ii) A cohesionless soil becomes quick when the effective stress is equal to zero.

(iii) The cohesive soil doesnot become quick even if effective stress is equal to zero, as it still possesses some strength equal to cohesion intercept.

(iv) A quick condition is most likely to occur in silt and fine sand.

Note: quick sand condition is due to ~~liquification~~ flow in which liquification is caused due to shaking.

Q. Quick sand condition is due to

(a) hydraulic condition (b) properties of soil.

(c) both a and b. (d) None of above.

Analysis of quick sand condition:

$$i_{critical} = \frac{G-1}{1+e}$$

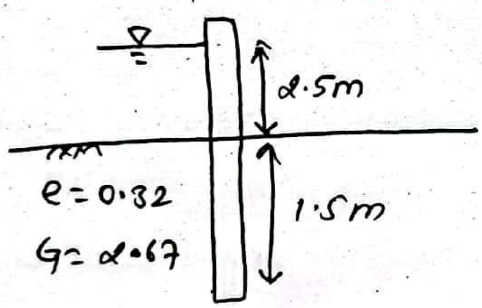
$$i_{developed} = \frac{\gamma_{sub}}{\gamma_w} = \frac{h}{L}$$

For any project:-

$$FOS = \frac{i_{critical}}{i_{developed}}$$

- If FOS > 1, No quick sand condition.
- If FOS < 1, Possibility of quick sand condition.

Q. Is there a possibility of quick sand condition:



solution:

$$i_{critical} = \frac{G-1}{1+e} = \frac{2.67-1}{1+0.32} = \frac{1.67}{1.32} = 1.265$$

$$i_{developed} = \frac{h}{L} = \frac{2.5}{(1.5+1.5)} = 0.833$$

Here, L = path of water travelled = 2×1.5 = 3

$$FOS = \frac{i_{critical}}{i_{developed}} = \frac{1.265}{0.833} = 1.518 > 1$$

Hence no quick sand condition.

Note:

If the quick sand condition occurs, then the embeded depth of sheet pile should be increased so that $i_{developed}$ decreases and FOS increases.

2.1.4 Seepage Analysis :-

- Flow of water through a soil mass under hydraulic gradient is called seepage.
- It is the result of permeability of soil.

Effects of seepage:

(i) water loss in dam, reservoir, canal etc.

(ii) sub-surface erosion.

Two dimensional flow equation: Laplace Equation:

Assumptions:

(i) soil mass is homogenous

(ii) soil mass is isotropic

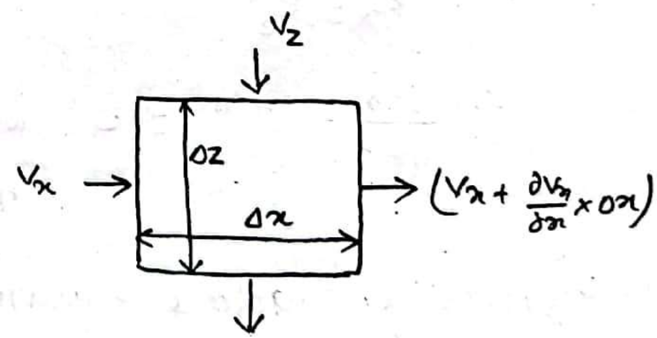
(iii) continuity of flow.

(iv) Two dimensional flow i.e in Hz. and Vt. direction

(v) Darcy's law of flow is valid.

(vi) No volume change in soil mass.

Derivation:



consider an element of size δx and δz and of unit thickness perpendicular to plane of figure.

let V_x and V_z be the velocity components in x and z-directions.

The corresponding velocity components at exit will be,

$$\left(V_x + \frac{\partial V_x}{\partial x} \times \delta x\right) \text{ and } \left(V_z + \frac{\partial V_z}{\partial z} \cdot \delta z\right)$$

According to assumption,

water entering into an element is equal to water leaving the element,

$$V_x(\delta z \cdot 1) + V_z(\delta x \cdot 1) = \left(V_x + \frac{\partial V_x}{\partial x} \times \delta x\right)(\delta z \cdot 1) + \left(V_z + \frac{\partial V_z}{\partial z} \cdot \delta z\right)(\delta x \cdot 1)$$

or, $V_x \cdot \Delta z + V_z \cdot \Delta x = V_x \cdot \Delta z + \frac{\partial V_x}{\partial x} \cdot \Delta x \, \Delta z + V_z \cdot \Delta x + \frac{\partial V_z}{\partial z} \cdot \Delta z \cdot \Delta x$

or, $\frac{\partial V_x}{\partial x} \cdot \Delta z \cdot \Delta x + \frac{\partial V_z}{\partial z} \cdot \Delta z \cdot \Delta x = 0$

$$\therefore \frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z} = 0 \quad \text{------} \quad (*)$$

This is continuity equation for two dimensional flow.

Also, from Darcy's law:

$v = ki$ where $i_x = \frac{\partial h}{\partial x}$

$v_x = K_x i_x$

and $v_z = K_z i_z$ $i_z = \frac{\partial h}{\partial z}$

↳ substituting these values in above equation:

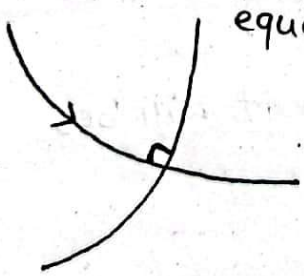
$$K_x \times \frac{\partial^2 h}{\partial x^2} + K_z \times \frac{\partial^2 h}{\partial z^2} = 0$$

for isotropic soil:

$K_x = K_z$

$\therefore \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0$ which is the laplace equation for two dimensional flow.

↳ Physically laplace equation for two dimensional flow represents two curves:



equipotential line : equal head

flow line : line along which water flows.

a. Flownet:

↳ The graphical representation of seepage with possible flow line and headloss pattern is flow net.

↳ It is based on laplace equation.

b. Flow line:

↳ Path traced by seepage water within soil mass.

c. Equipotential line:

↳ Imaginery line joining points having equal heads.

d. Equipotential drop:

↳ Equal head loss in flow field.

↳ $\Delta h = \frac{H}{N_d}$; H = head causing the flow.
N_d = no. of drop.

e. Flow channel:

↳ Space between two consecutive flow line.

d. Flow field:

↳ Space between two adjacent flow line and two consecutive equipotential line.

*: Construction of flow net:

↳ By two methods:

(i) Hit and trial : manual method (Graphical method).

(ii) Finite element method : Analytical method.

*: Characteristics requirements of flownet:-

(i) Flow lines or equipotential line should meet or cross normally.

(ii) Flow lines and equipotential line should be smooth and continuous.

(iii) Flownet should valid boundary condition.

(iv) Box formed in between flow line and equipotential line be - square ...

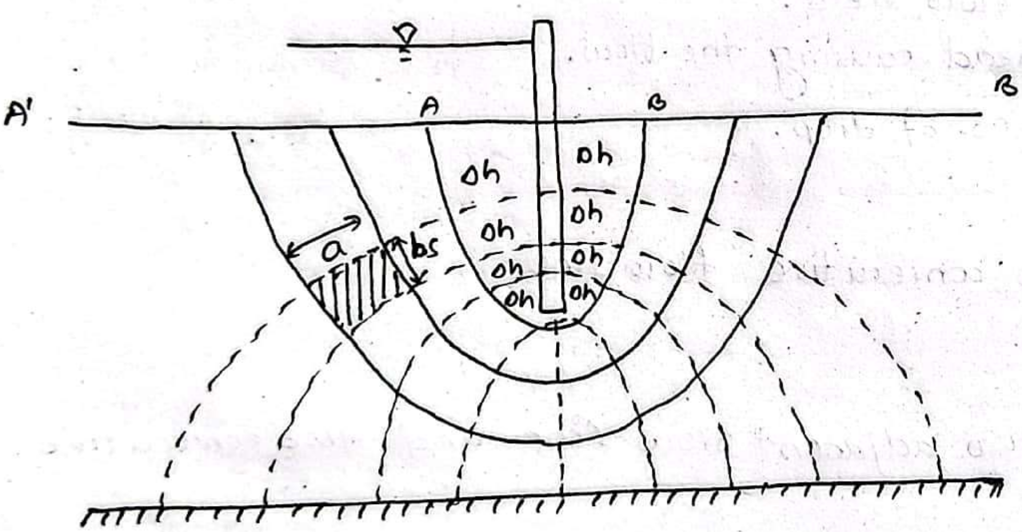
*: Graphical method of construction of flownet:-

- Most extensively used method of construction of flow net.
- detail explanation not required......

Uses of flownet:

1. determination of seepage discharge.
2. determination of seepage force
3. determination of uplift pressure on dam.
4. calculation of exit hydraulic gradient.

(i) determination of seepage discharge:



let q = discharge through flownet

from darcy law,

seepage through one flow channel:

$$\Delta q = kiA$$
$$= k \times \frac{\Delta h}{b} \times (a \times 1)$$

Total discharge

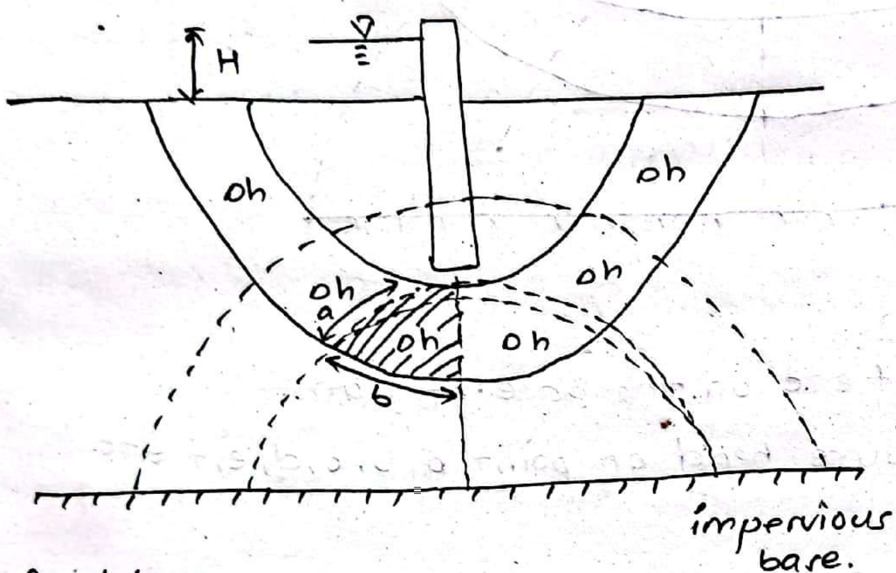
$$(q) = \Delta q \times N_f$$
$$= k \times \frac{\Delta h}{b} \times a \times N_f$$
$$= k \times \frac{H}{N_d} \times \frac{a}{b} \times N_f \quad (\because \Delta h = \frac{H}{N_d})$$

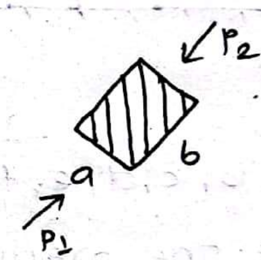
for square box.

$a = b$

$\therefore q = K \times H \times \frac{N_f}{N_d}$. Equ^n for discharge per unit length.

(ii) calculation of seepage force.





At front face:

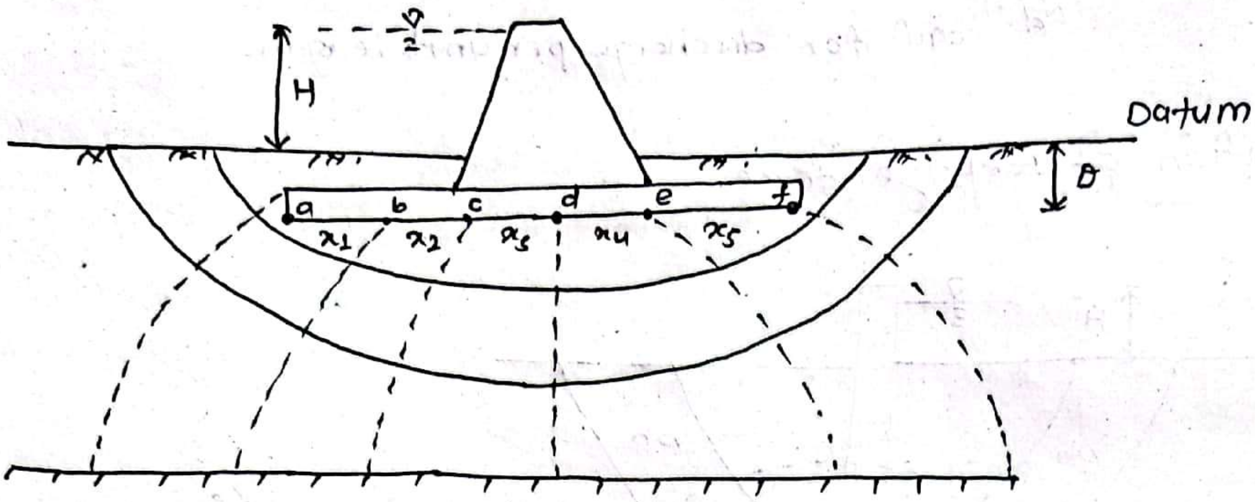
$$P_1 = (H - 2\Delta h)\gamma_w$$

At back face:

$$P_2 = (H - 3\Delta h)\gamma_w$$

$$\begin{aligned} SF \text{ on soil} &= (P_1 - P_2) \times A \\ &= \{(H - 2\Delta h) - (H - 3\Delta h)\} \times A \\ &= (\cancel{H} - 2\Delta h - \cancel{H} + 3\Delta h) \times A \times \gamma_w \\ &= \Delta h \times A \times \gamma_w \\ &= \Delta h \times \gamma_w \times A \end{aligned}$$

(iii) Calculation of uplift pressure on dam:



Procedure:

a. Mark a, b, c, d, e, f etc on the base of dam.

b. Calculate the pressure head on point a, b, c, d, e, f etc.

for point a,

$h_t = H - \Delta h - 0.30\Delta h = H - 1.30\Delta h$

; h_t = total head.

Also,

$h_e = -D$

$h_v = 0$

Now, total head, $h_t = h_e + h_v + h_p$

$H - 1.30\Delta h = -D + 0 + h_{p(a)}$

or, $h_{p(a)} = (H - 1.30\Delta h) + D$

$\therefore$ General equation,

$(h_p)_n = (H - n\Delta h) + D$

$(h_p)_b = (H - 2\Delta h) + D$

$(h_p)_c = (H - 3\Delta h) + D$

$(h_p)_d = (H - 4\Delta h) + D$

$(h_p)_e = (H - 5\Delta h) + D$

$(h_p)_f = (H - 5.5\Delta h) + D$

c. Draw pressure diagram.

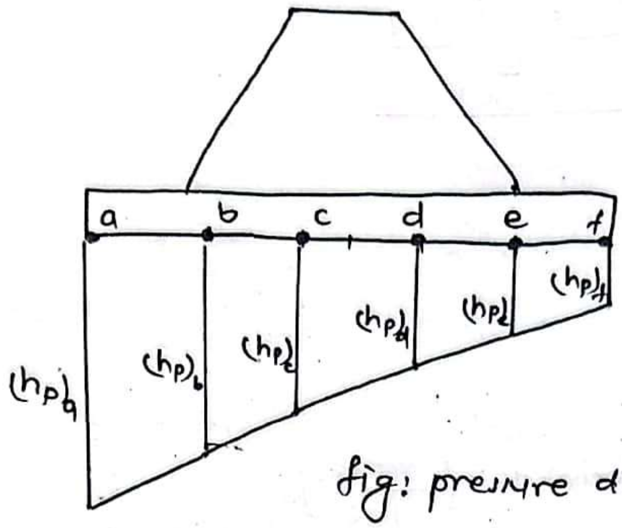


fig: pressure diagram

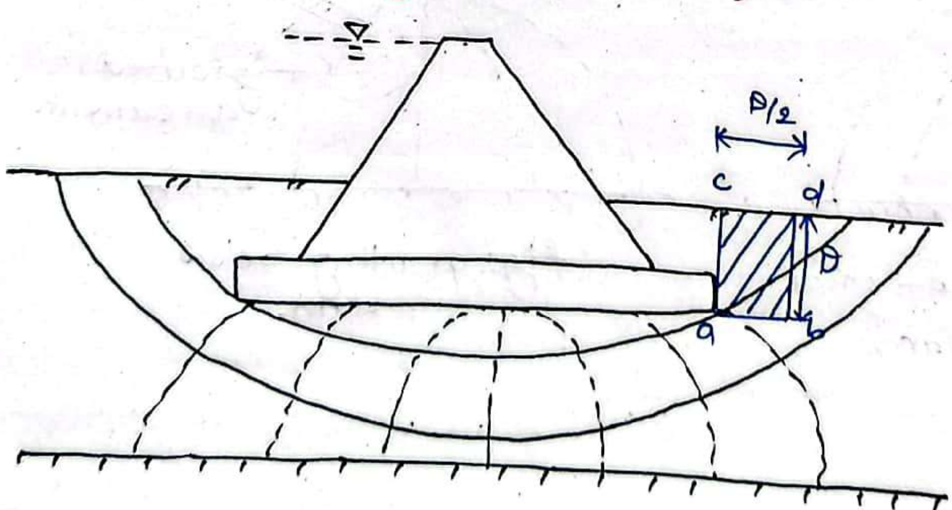
d. Uplift pressure on dam (per unit length)

U = Area of pressure diagram

$$= \frac{1}{2}\left[(hp)_a + 2(hp)_b + 2(hp)_c + 2(hp)_d + 2(hp)_e + (hp)_f \right]$$

Note: This uplift force is resisted by weight of dam.

(iv) Calculation of exit gradient (i_{exit}):



Procedure

a. Mark rectangle abcd of D x D/2 size at toe side.

b. Calculate pressure heads at points a, b, c, d respectively.

c. calculate exit gradient:

$$i_{exit} = \frac{\dfrac{(h_p)_a + (h_p)_b}{2} - \dfrac{(h_p)_c + (h_p)_d}{2}}{D}$$

Analysis;

$$FOS = \frac{i_{critical}}{i_{developed}} \geq 1.5$$

where

$$i_{critical} = \left(\frac{G-1}{1+e}\right)$$

Special technique to reduce seepage discharge and exit gradient:

(i) Increase depth of foundation.

(ii) Introduce sheet pile below dam.

(iii) Grouting: before or after construction.

Note:

1) Rigid pavement मा हुने pumping को solution पनि grouting हो।

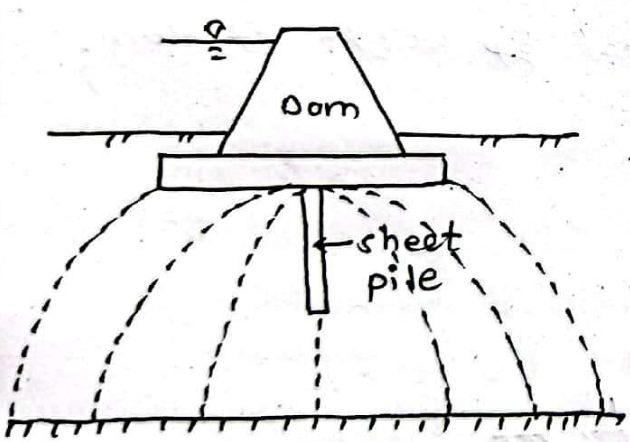


fig: introduction of sheet pile below dam.

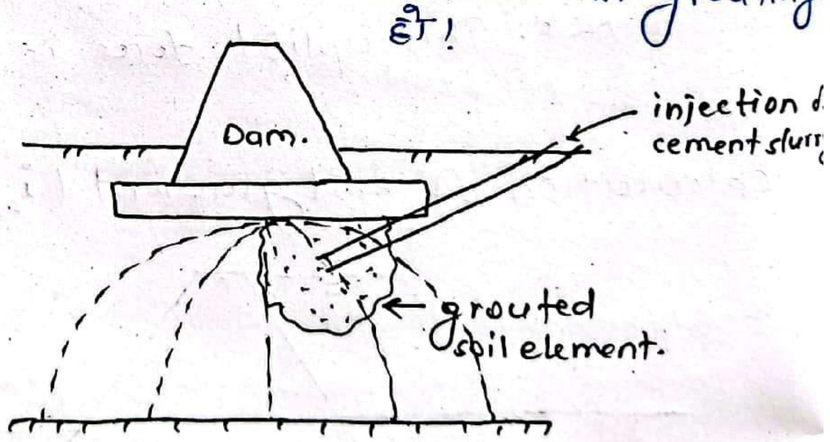


fig: grouting below dam.

Seepage Analysis through Earthen Dam: Unconfined seepage:

↳ The seepage analysis is carried out in earthen dam to analyze the pheratic line, the pore pressure within the dam or in its foundation, the exit gradient at downstream face, the amount of seepage flow that may pass through the dam's cross-section

Causes of seepage in earthen dam:

↳ seepage in earthen dam primarily depends on the properties of soil such as

- plasticity of soil
- the gradation
- the degree of compaction
- cohesion between particles etc.

Also, poor design practices will permit seepage in earthen dam.

Effects of seepage:

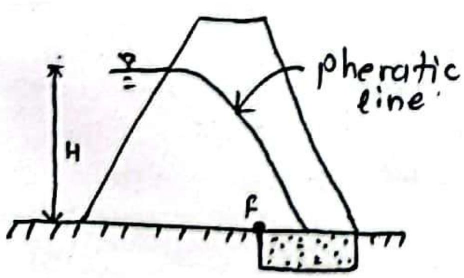
(i) Water loss

(ii) sub-surface erosion if not prevented, continuous erosion forms pipe which may collapse the dam.

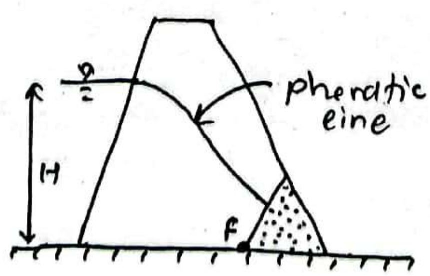
(iii) Heaving and slope instability

(iv) Piping.

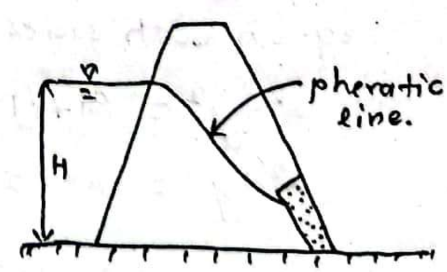
To prevent piping, filter layer is constructed in earthen dam:



Hz- filter



Toe filter



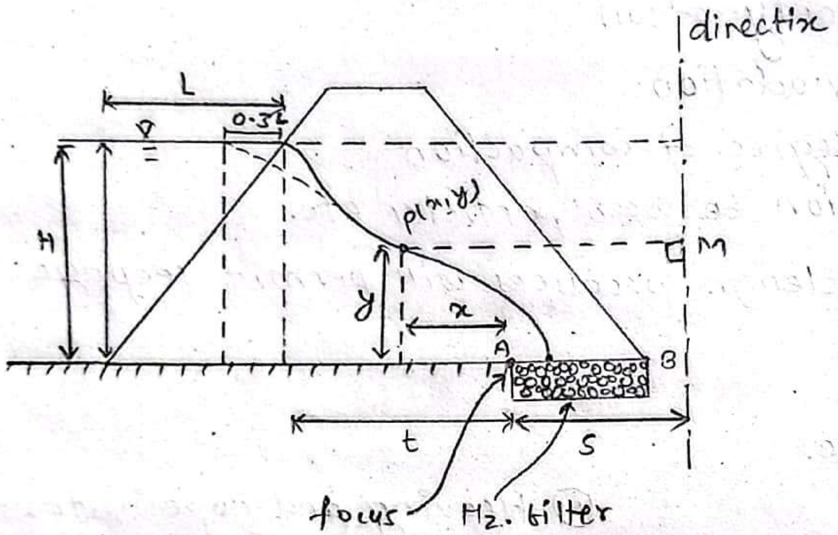
Inclined filter.

Phreatic line:

↳ The topmost flow line of seepage water through earthen dam is known as phreatic line.

↳ position of pheratic line depends on height of retained water, position & size of retained water.

↳ Height of retained water varies, so the position of pheratic lines varies with height of water retained. so, seepage through dam is unconfined seepage.



↳ shape of phreatic line: parabola.

*! Equation of phreatic line:

from properties of parabola,

$PF = PM$

$\sqrt{x^2+y^2} = x+s$

sq. on both sides

$x^2+y^2 = (x+s)^2$

$\cancel{x^2}+y^2 = \cancel{x^2}+2xs+s^2$

$y^2 = 2xs+s^2$

$y = \sqrt{2xs+s^2}$ ---- *

which is equn of phreatic line.

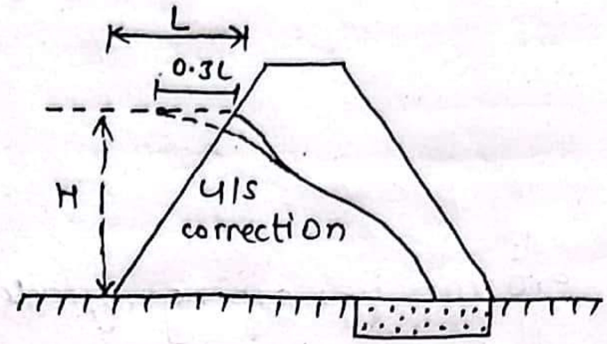
for $x=t$, $y=H$

so, $H = \sqrt{2ts+s^2}$ $\therefore s = \sqrt{t^2+H^2} - t$ ------ **

* Construction of phreatic line:

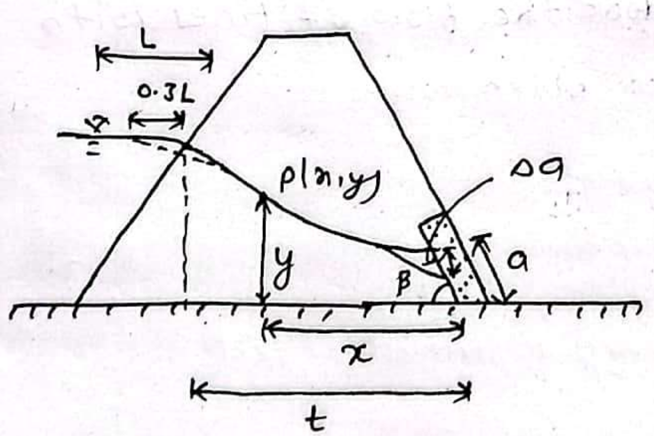
step 1: Draw phreatic line using equ^n $s = \sqrt{t^2+H^2} - t$

step 2: carryout upstream correction as shown in figure,



step: 3 make the downstream correction using standard table.

β	30°	60°	90°	120°	150°	180°
$\Delta a/a$	0.36	0.32	0.28	0.20	0.10	0



↳ Discharge : from flow net:

$$q = KH\frac{N_f}{N_d}$$

↳ Discharge using equations : (phreatic line equ^n)

$$y = \sqrt{2xs+s^2}$$

↳ from Darcy's law:

$$q = KiA$$

$$; A = y \times 1$$

$$i = \frac{dy}{dx} = \frac{1}{2} \times \frac{2s}{\sqrt{2xs+s^2}} = \frac{s}{y}$$

$$q = K \times \frac{s}{y} \times (y \times 1)$$

$$q = K \times S$$

$$; S = iA$$

Design of graded filter:

- To protect the sub-surface erosion i.e. piping, graded filter can be constructed in earthen dam.
- But the filter layer should satisfy the two requirement or criteria:

① Piping requirement:

- the filter layer should block the flow of fines with seepage water from dam.

criteria: Terzaghi's

$$\frac{D_{15}\,(\text{filter soil})}{D_{85}\,(\text{dam soil})} < 5$$

⑪ Permeability requirement:

- the filter layer should provide easier path to flow seepage water from dam body.
- If the water is blocked within dam body, it creates excess pore water pressure which may threat the stability of dam.

criteria: Terzaghi's

$$\frac{D_{15}\,(\text{filter soil})}{D_{15}\,(\text{dam soil})} > 5$$

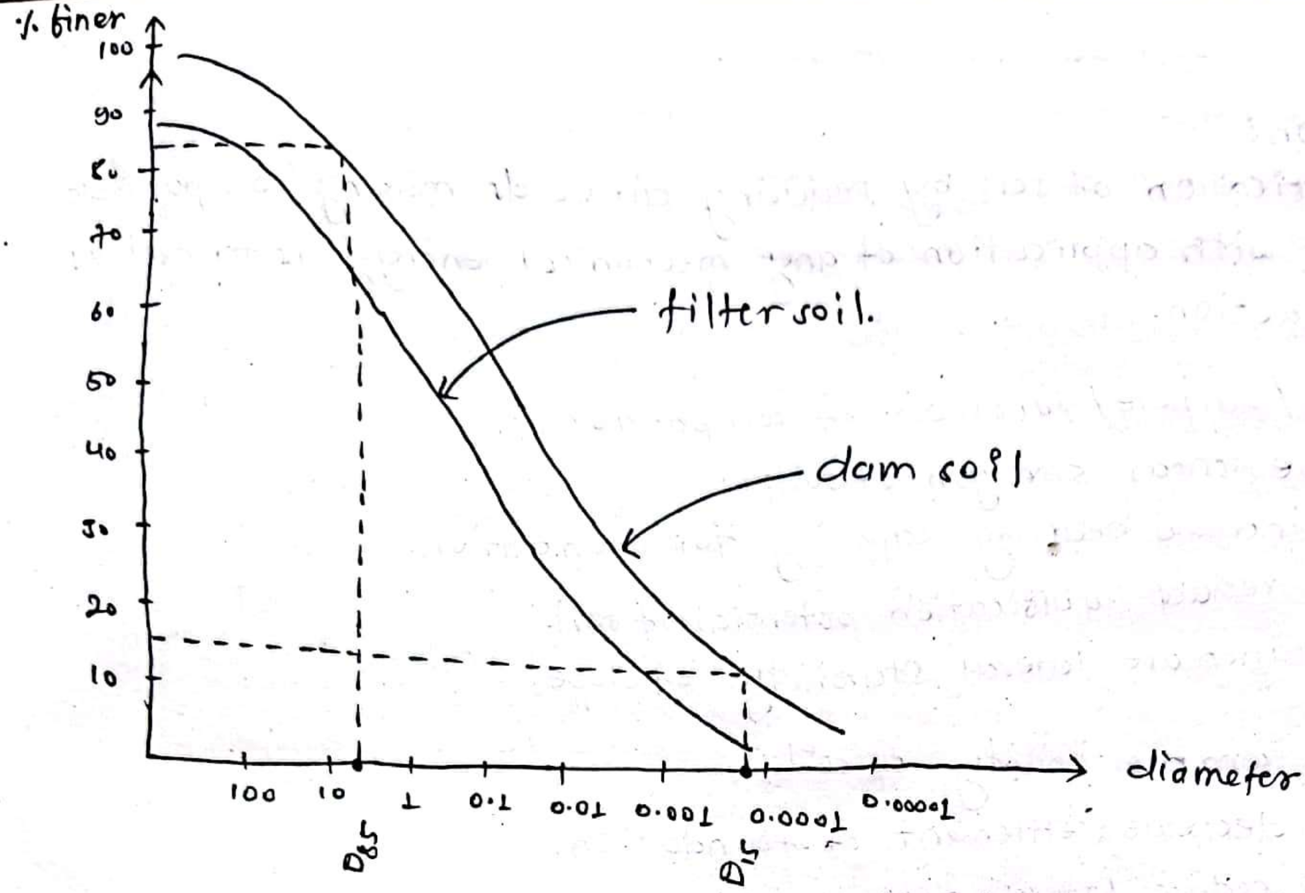


fig: grain-size distribution curve.

2.1.5 Compaction of soil:

*: Compaction:

↳ Densification of soil by reducing air voids making soil particles closer with application of any mechanical energy is termed as compaction.

Advantages / purpose / function of compaction:

a. Increase shear strength of soil:

↳ increase bearing capacity for foundation.

↳ reduce liquification potential of soil.

↳ increase lateral stability of slope.

b. Reduce compressibility of soil:

↳ decrease settlement of foundation

↳ reduce lateral deformation on slope

↳ reduce undulation on pavement.

c. Reduce permeability of soil:

↳ reduce water loss in dam, canal, reservoir etc.

d. Reduce shrinkage and swelling potential of soil:

↳ reduce adverse effect on soil.

↳ reduce pavement failure.

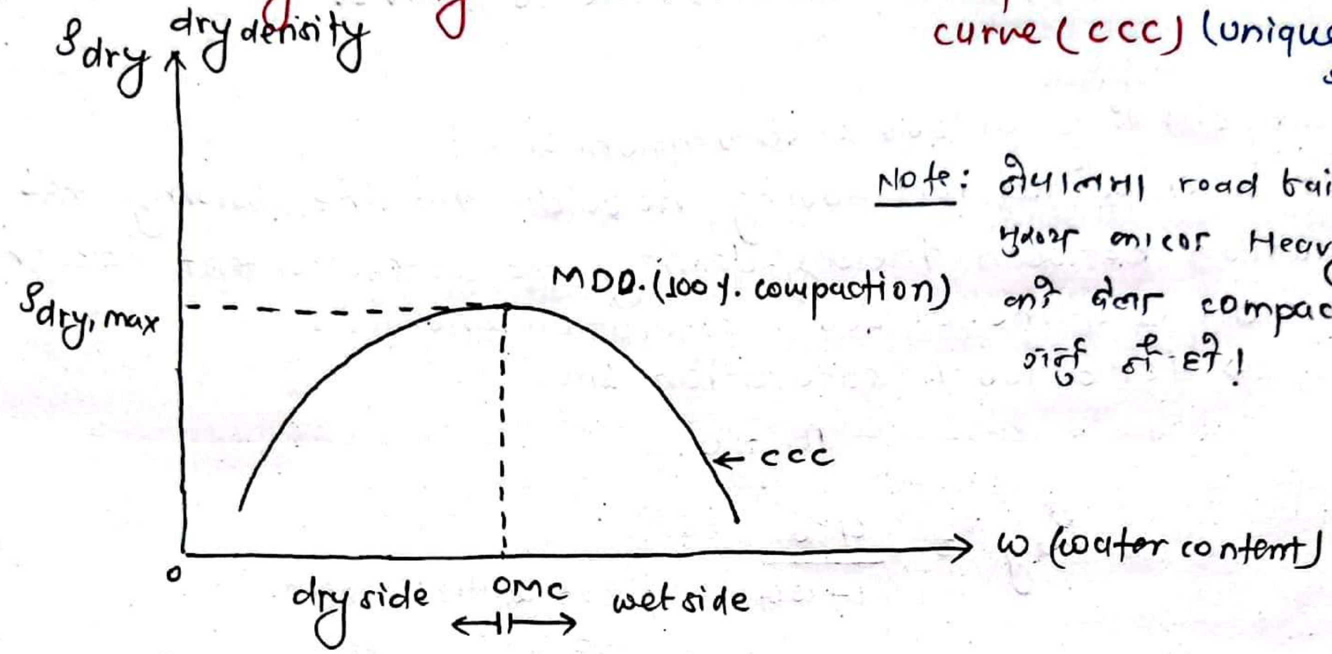
Concept / Principle of compaction:

↳ Water is sprinkled on soil so that water wet the soil particles, act as lubricant between soil particles and reduce friction so that most of the applied energy can be used to make the soil partic[les] closer.

Degree of compaction:

↳ compaction should be measured interms of dry density, so dry density achieved after compaction is also known as degree of compaction.

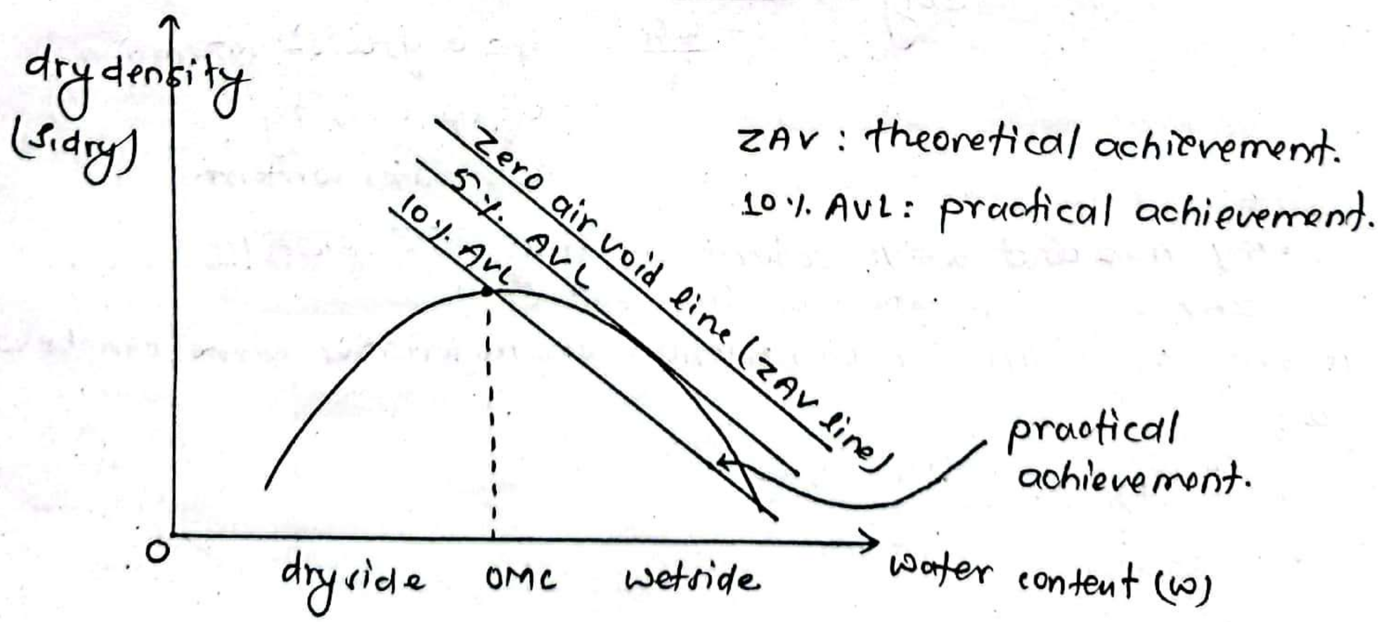
Moisture - dry density relation for soil : Compaction characteristics curve (CCC) (unique for each soil.)



<u>Note</u>: नेपालमा road failure हुनुको, मुख्य कारण Heavy rainfall को ठीक compaction गर्नु नै हो!

↳ compaction characteristics curve is used for simulation of field compaction.

↳ lab-test of compaction is necessary to determine the compaction characteristics..



Q. Why ccc always lie left side of ZAV-line?

Ans: 5 to 10 %. air void can't be removed by applying any mechanical energy, so compaction characteristics curve always lies left side of ZAV. line.

↳ but, ccc lies between 5 %. to 10 %. AVL.

zero-air void line or 100 %. saturation lines:

↳ For a compacted soil having no voids, the line showing the relation between the dry-density and water content is called zero air void line. or 100 %. saturation line.

↳ for a 100 %. saturation line,

$$n_a = 0.$$

$$\therefore \rho_{dry} = \frac{G\rho_w}{1+wG} \quad ; w = \text{water content.}$$
$$G = \text{sp. gravity}$$

saturation line:

↳ for a compacted soil, the line showing relation between the water content and dry density of a soil for a constant degree of saturation (S_r) is known as saturation line.

$$\rho_{dry} = \frac{G\rho_w}{1+\frac{wG}{S_r}} \quad ; S_r = \text{degree of saturation}$$
$$G = \text{sp. gravity}$$
$$w = \text{water content}$$

Note:

- 5 %. AVL and 95 %. saturation line एक हैं!!!
- ZAV and 100 %. saturation line एक हैं!!!

↳ General equation for compaction characteristics curve can be written as;

$$\rho_{dry} = \frac{(1-n_a)\,G\rho_w}{1+wG}$$

; n_a = air content

w = water content.

G = sp. gravity

Lab test of compaction:

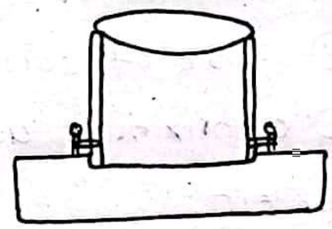
- There are two types of laboratory test for compaction:
  They are:
  1. standard proctor test (SPT) (AASTHO test).
  2. Modified standard proctor test (Modified AASTHO test).

Objective of lab test:

- to simulate field compaction. (જાતે સાઇટ માં roller ચલાવીએ ઈ જાતે પણ ચલાવે
- to determine ρ_{dry} & OMC for compaction
  i.e to draw compaction characteristics curve.

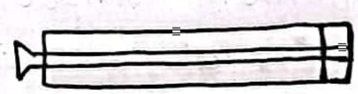
Apparatus,

- Rammer.
- compaction mould.



compaction mould

Comparision of standard proctor test (SPT) and modified standard proctor test:



Rammer.

S.No.	Parameter	Standard Proctor test (SPT)	Modified standard proctor test (Modified AASTHO)
1.	Wt. of rammer	2.6 kg	4.9 kg
2.	Free fall height of rammer	310 mm	450 mm
3.	No. of layer of soil to fill mould	3	5
4.	No. of drop of rammer for each layer	25	25
5.	Energy applied	592 kJ/m³	2700 kJ/m³

6.	simulate	light compaction	Heavy compaction
7.	Use	Urban road, district road	Highway, feeder road (if soil contains >20% gravel)

Factors affecting compaction:

↳ Factors which affects compaction are:

① moisture content

② energy applied

③ mode of energy application

④ type of soil.

⑤ admixture used.

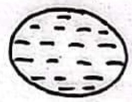
① Moisture content:

↳ Best on omc compaction.

- Wet of optimum compaction

  For fine soil: disperse structure formed in compacted soil.

  ↳ shear strength low



- Dry of optimum compaction

  for fine soil: flocculated structure formed

- edge to edge contact
- face to face contact

} between soil particles.

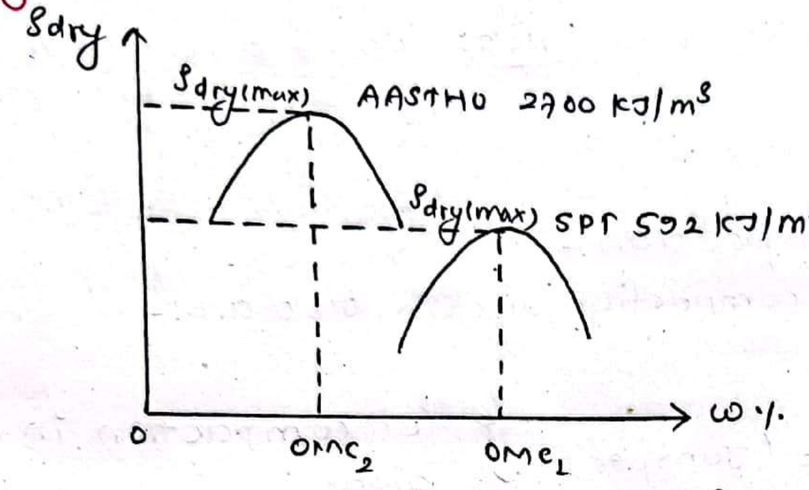
↳ shear strength high & low compressibility.

Properties	Flocculated structure	Disperse structure.
Shear strength	High	low
compressibility	low	High
swelling	High	low
shrinkage	low	High.

Key note:-

- for road: better in dry of optimum compaction.
- for dam, canal: better wet of optimum compaction.

(ii) Energy applied:



; $OMC_2 < OMC_1$

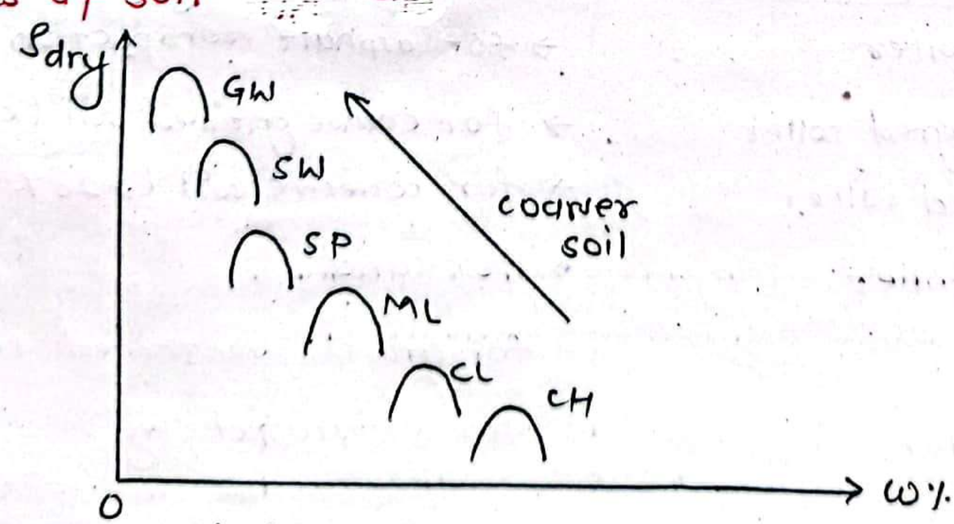
↳ Higher the energy applied, larger the $\rho_{dry(max)}$ can be achieved even in lower OMC.

(iii) Mode of energy application.

static loading
Kneading
vibrating
Dynamic loading

↓ Better compaction.

(iv) Types of soil



- Coarser the soil: Better the compaction.
- Best: for well graded soil.

(v) Admixture used:

↳ Properties of soil can be enhanced by considerably mixing admixture like cement, lime, bitumen etc in soil before compaction.

↳ calcium chloride has been widely used chemical additive for compaction.

Note: use of cement 5% in soil can prevent pavement failure in Nepal.

↳ Increases useful life by 5 to 10%.

Field Method of compaction:-

↳ Different methods of compaction used in field are:-

① Using roller

② using vibrating plate (Jumper) } for compaction in narrow area:

eg:
→ inside the trench of water supply.
→ inside building
→ behind retaining wall. (Backfill soil compaction).

③ Using roller
↓
for compaction in widely spread area:

eg:
→ base, sub-base, sub-grade
→ compaction of road, runway etc.
→ dam, embankment compaction.

Types of roller:	suitability / uses:
(i) steel drum roller	→ for asphalt compaction
(ii) Pneumatic tyred roller	→ for coarse grained soil (< 20% fine
(iii) sheep footed roller	→ for cohesive soil (> 20% fines)
(iv) vibrating roller	→ for sand
(v) Grid roller	→ for gravel, weathered rock
(vi) Impact roller	→ for saturated clay समुद्रको, किनारमा., खोलाको किनार

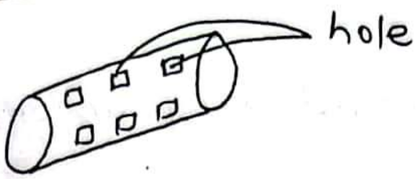


fig: grid roller

↳ coarse grained soil & rock मा use गरिन्छ!

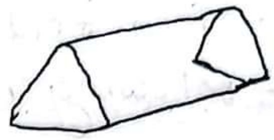


fig: impact roller

↳ river side, ocean side मा soil compaction गर्न use गरिन्छ!

Quality control and field control of compaction:

↳ Quality control is essential process for ensuring the reliability and consistency of compaction methods:

↳ compaction quality control can be done by using compaction specification.

*: Compaction specification:

↳ Tender document having clear description about the properties of materials (soil to be compacted, process of compaction to be used and quality of compacted soil) to be achieved is known as compaction specification.

↳ It consists of two parts:

ⓐ Process specification:

↳ Process specification consists of clear description about

(i) type of roller

(ii) thickness of each soil layer

(iii) size of roller

(iv) No. of passes each soil layer.

ⓑ Product specification:

↳ Product specification describes about the properties of soil to be achieved.

(i) $\delta_{dry(max)}$

(ii) OMC

Field control of compaction:

↳ Quality of compaction in field can be controlled using anyone or combination of following methods:

ⓐ control in process.
ⓑ control in product.

ⓐ control in process:

↳ In this method the supervisor should be present in the field during compaction and check following parameter:

(i) type of roller being used
(ii) size of roller being used
(iii) thickness of each soil layer
(iv) No. of passes on each soil layer.

↳ Make sure according to specification.

ⓑ control in product:

↳ In this method the inspector should be present in field after immediate compaction and check,

- measure field density (ρ)
- moisture content (w)

$$\rho_{dry} = \frac{\rho}{1+w} \geq 95\% \text{ of } \rho_{dry(max)}$$

$$w = omc \pm 2\%$$

↳ 500 m interval मा field test गर्ने
↳ moisture content check गर्ने

Nepal मा mostly
- sand replacement method
- core cutter method

use गरिन्छ !

2.1.7 consolidation and settlements:

*: compressibility:

↳ Soilmass shows reduction in volume under the effect of applied compressive stress, this property is termed as compressibility of soil.

↳ compressibility varies from soil to soil.

- compressibility (property) ⟶ consolidation (phenomenon) ⟶ settlements. (result)

*: Types of compressibility /consolidation:

① Elastic compression
↳ upto 1 week.

(II) Primary consolidation
↳ compression of water and water void to expulsion of water.

(III) secondary consolidation.
↳ plastic rearrangement of soil particles.

Causes of compressibility of soil:

① Elastic compression:
↳ immediate compression (within 7 days of loading)
↳ compression of soil particles, air and airvoids.
↳ small compression.

(II) Compression of water and water void to expulsion of water.
↳ takes long time
↳ gives large compression specially in clay.

(III) compression due to plastic rearrangement of soil particles
↳ small compression.

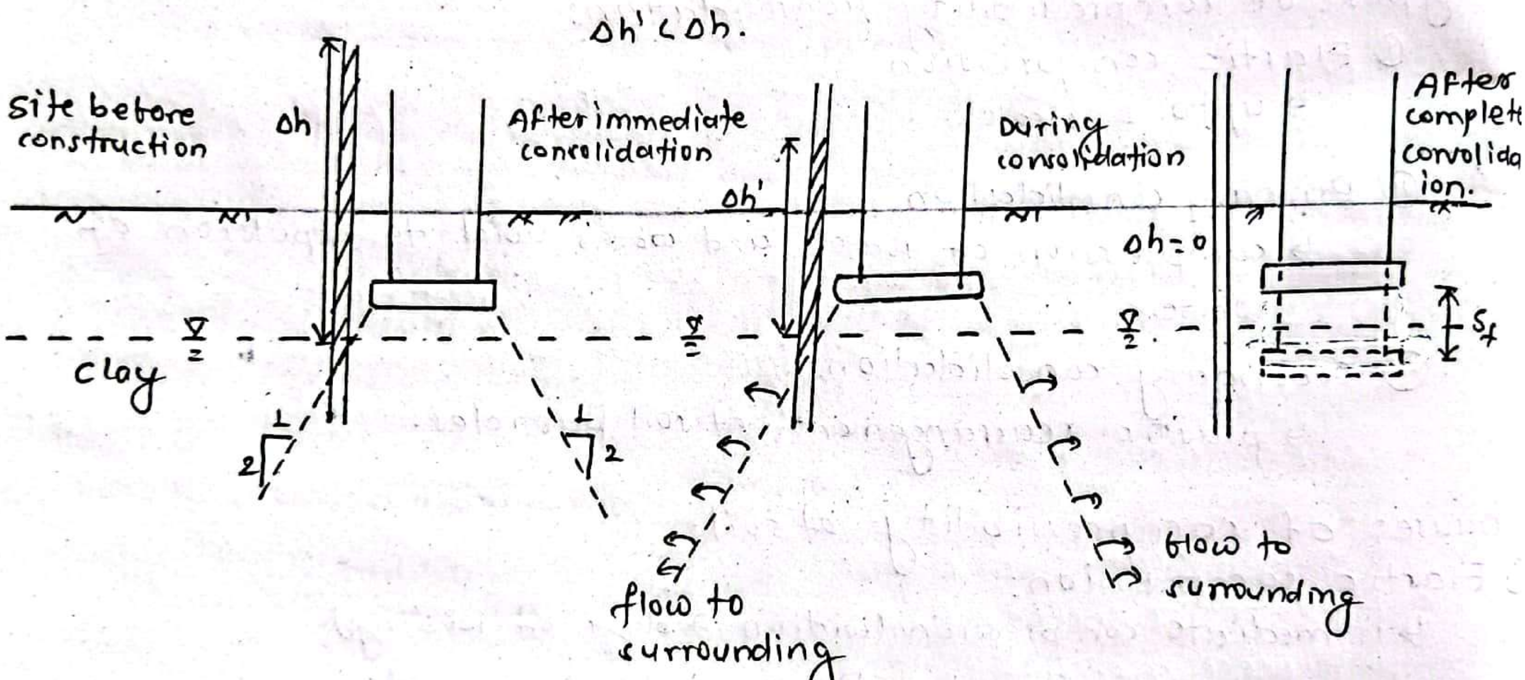
*: Consolidation:

↳ Compression of soil mass due to expulsion of water from void under the effect of applied compressive stress is termed as consolidation.

*: Principle of consolidation:

↳ When a soil mass is subjected to a compressive force, there is decrease in volume of soil mass, this reduction in volume of saturated soil mass due to expulsion of water from voids under the effect of static pressure is called consolidation.

↳ Principle of consolidation can be illustrated by:



↳ After construction, the excess pore water pressure (u_o) is developed within load affecting area.

↳ Water from void of load affecting area starts to flow to the surrounding soil having low porewater pressure. Water void start to be empty and the empty void start to be compressed by applied compressive stress resulting the settlement.

↳ The process is continued until the excess pore water pressure is completely dissipitated i.e $u_o = 0$. so the consolidation is the time dependent phenomena.

↳ consolidation gives large settlement for fine soil, specially clay.
• consolidation of sand is negligible, so generally not considered.

Lab test for consolidation:

α: 1-D Oedometer test:

Objective:

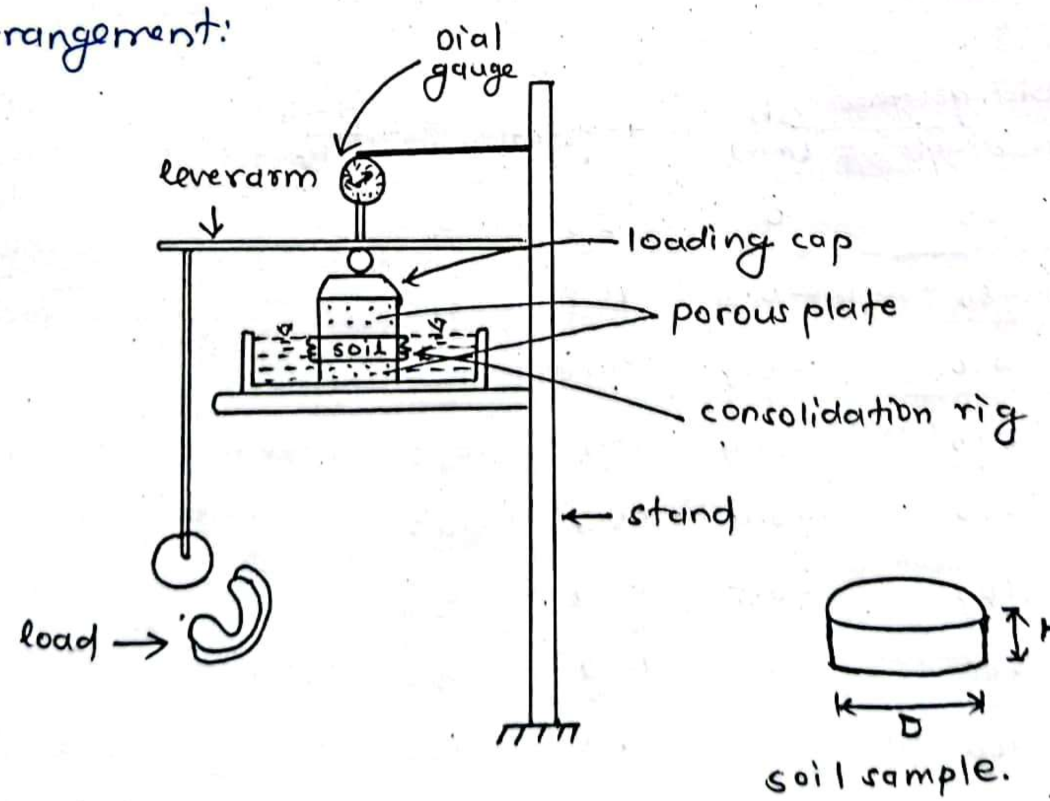
↳ To determine the rate and magnitude of settlement in soils.

↳ settlement values obtained by this test are due to primary consolidation, only which is 90% of total consolidation.

Apparatus required:

i. Oedometer
ii. Dial gauge
iii. stop watch
iv. spatula
v. weighing balance
vi. vernier calipers
vii. Oven
viii. water reservoir

Test arrangement:



D = 60 mm
H = 20 mm.

Test procedure:

(i) set up as shown in figure.

(ii) Loading:

↳ increase stage by stage, each load for 24 hrs with dial gauge observation,

1st stage load: 10 kN/m^2 → 20 kN/m^2 → 40 kN/m^2 → 80 kN/m^2 → 160 kN/m^2 → 320 kN/m^2 → 640 kN/m^2.
(last stage loading)

(iii) Unloading:

↳ decrease stage by stage load for 24 hrs with dial gauge observation.

640 kN/m^2 → 160 kN/m^2 → 40 kN/m^2 → 0 kN/m^2.

*: 24 hrs dial gauge observation:

0 sec, 15 sec, 30 sec, 1 min, 2 min, 4 min, 8 min, 16 min, 30 min, 1 hrs, 2 hrs, 4 hrs, 8 hrs, 16 hrs, & 24 hrs.

Calculation:

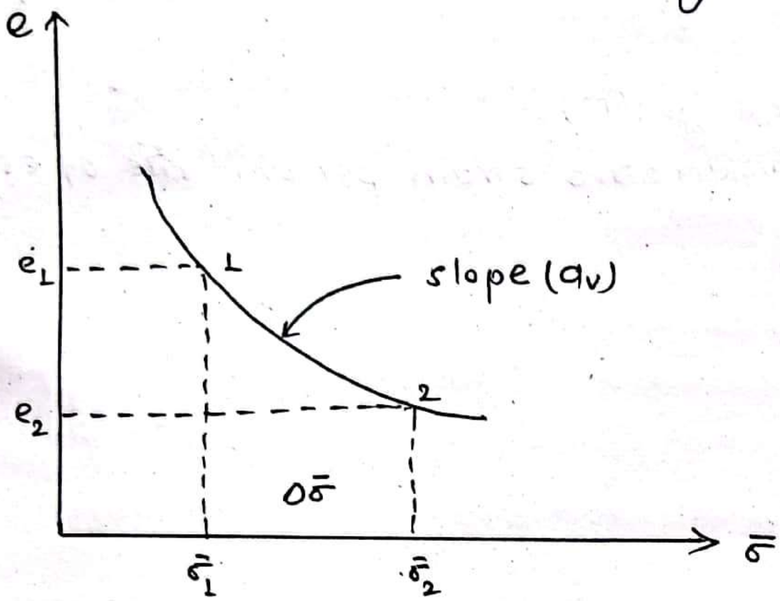
Stress (σ) kN/m^2	Dial gauge reading (div.)	δh (mm)	$H = H_0 - \delta h$	$e = \frac{H - H_s}{H_s}$	
0	800	0	20	e_0 = ---	
10	780	-0.2	19.8	e_1 =	
20	750	-0.3	19.5	e_2 =	
40	710	-0.4	19.1	e_3 =	reduces
80	650	-0.4	18.7	e_4 =	
160	600	-0.5	18.2	e_5 =	
320	500	-1	17.2	e_6 =	
640	400	-2	16.2	e_7 =	
160	440	0.4	16.6	e_8 =	
40	490	0.5	17.1	e_9 =	increases.
0	550	0.6	17.7	e_{10} =	

Height of solid, $H_s = \frac{m_s}{\rho_s \times A} = \frac{m_s}{G\rho_w \times A}$; $A = \frac{\pi D^2}{4}$

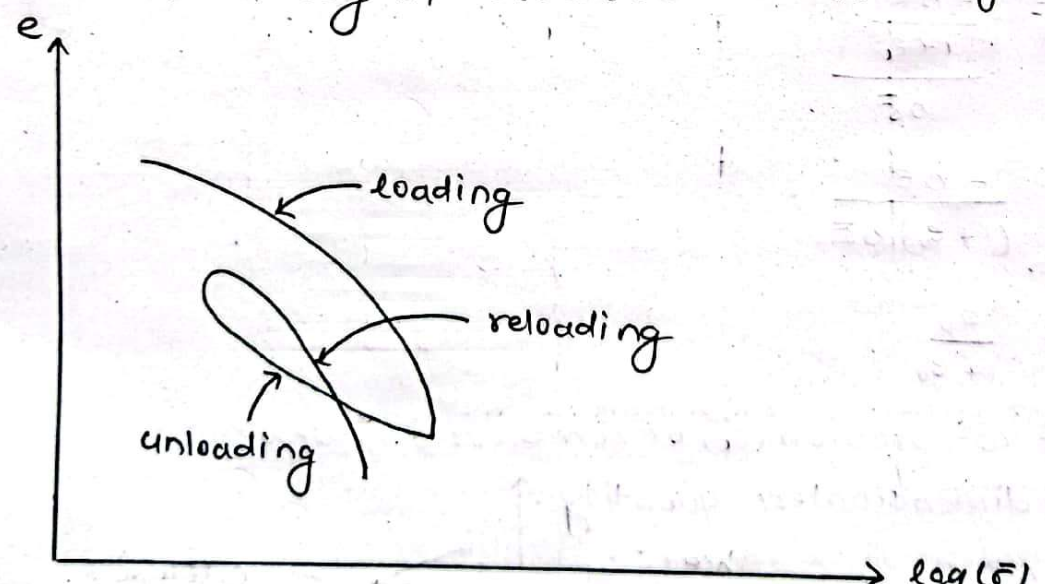
; D = dia. of sample.

Result:

(i) void ratio (e) vs effective stress diagram ($\bar{\sigma}$)



(ii) void ratio (e) vs log of effective stress ($\bar{\sigma}$) diagram.



Some coefficients:

(i) Coefficient of compressibility (a_v):

↳ It is defined as the decrease in void ratio per unit rise of effective stress.

$$a_v = \frac{-\Delta e}{\Delta \bar{\sigma}} \qquad \text{unit: } m^2/kN$$

↳ arithematic scale मा e vs $\bar{\sigma}$ को plot को slope हो।

(ii) Coefficient of volume change: (m_v)

↳ It is defined as the volumetric strain per unit rise of effective stress.

$$m_v = \frac{\frac{-\Delta V}{V}}{\Delta \bar{\sigma}}$$

from, 3-φ diagram,

$$\frac{-\Delta V}{V} = \frac{-\Delta e}{1+e_0}$$

so, above relation can be written as

$$m_v = \frac{\frac{-\Delta e}{1+e_0}}{\Delta \bar{\sigma}}$$

$$\text{or, } m_v = \frac{-\Delta e}{(1+e_0)\,\Delta \bar{\sigma}}$$

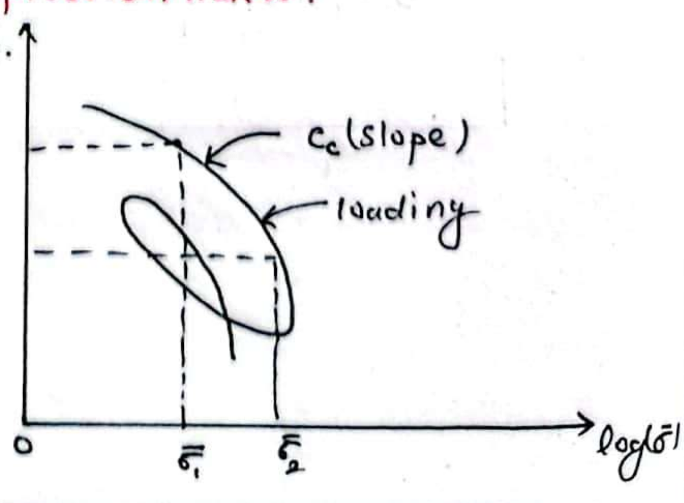
$$\therefore m_v = \frac{a_v}{1+e_0}$$

(iii) Coefficient of compression (C_c) or compression index:

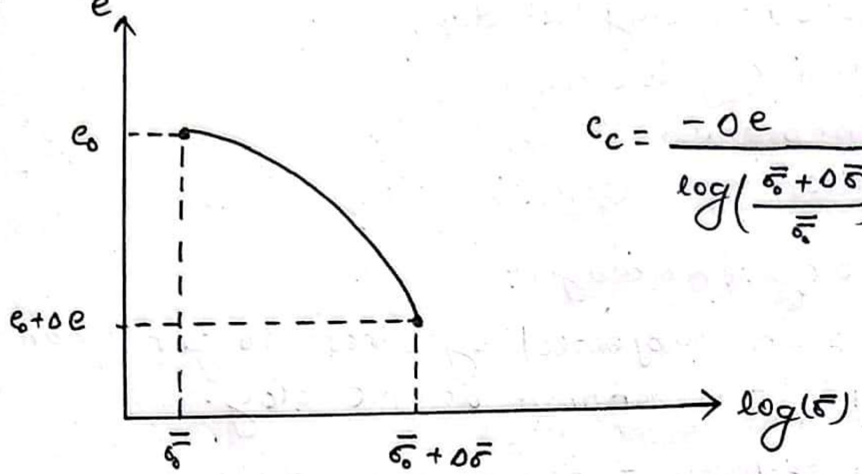
↳ unitless or dimensionless quantity.

$$C_c = \frac{-\Delta e}{\log\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)} = \frac{-\Delta e}{\log(\Delta \bar{\sigma})}$$

↳ logarithmic scale मा $\bar{\sigma}$ vs e को plot हो।



for insitu condition:



$$C_c = \frac{-\Delta e}{\log\left(\frac{\bar{\sigma}_0+\Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

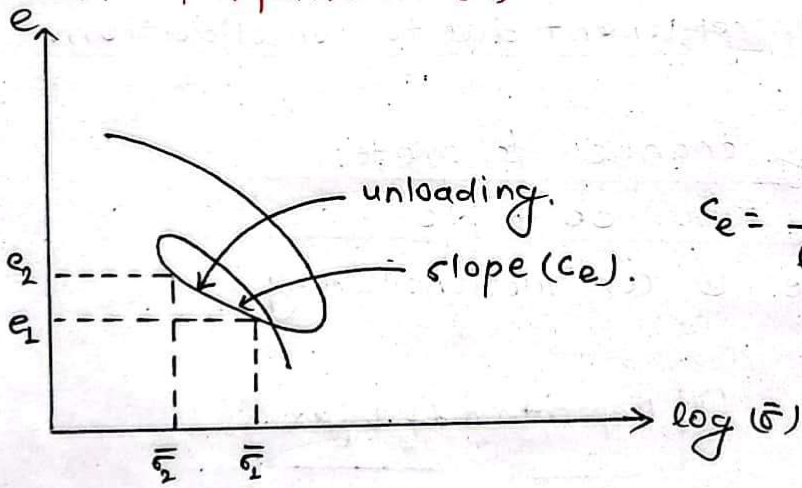
; $\bar{\sigma}_0$ = initial effective stress on clay before loading.

$\Delta\bar{\sigma}$ = increase in effective stress after loading.

empirical formula,

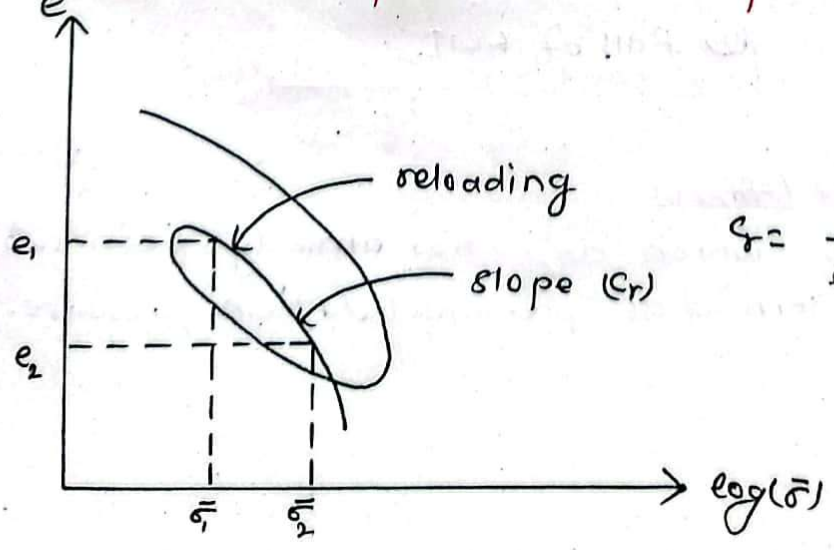
$$C_c = 0.09(LL-10)$$

(iv) coefficient of expansion (C_e) or swell index:



$$C_e = \frac{\Delta e}{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_2}\right)}$$

(v) coefficient of recompression: or recompression index (C_r):



$$C_r = \frac{-\Delta e}{\log\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)}$$

Also,

$$\boxed{C_r < \frac{1}{10} C_c}$$

Some important terms:-

a. Normally consolidated clay (NC clay)
b. Over consolidated clay (OC clay)
c. Preconsolidation pressure (σ_{max})
d. Over consolidation ratio (OCR).

a. Normally consolidated clay: (NC clay):

↳ If the clay has not been subjected by stress larger than present stress on clay, it is known as NC clay.

↳ NC clay gives large settlement due to consolidation.

b. Over consolidated clay (OC clay):

↳ If the clay already has been subjected in its past life by the stress larger than present stress, the clay is OC clay.

↳ OC clay gives small settlement due to consolidation.

*: Causes: By which clay changes its state:

S.N	NC to OC	S.N.	OC to NC
(i)	Dismental of structure	(i)	construction: loading
(ii)	Erosion of layer clay ⇒ clay	(ii)	Deposition of layer clay ⇒ clay
(iii)	Rise of GWT	(iii)	Fall of GWT.

c. Preconsolidation pressure (σ_{max}):

↳ The maximum stress by which clay has already been subjected in its past life is termed as pre-consolidation pressure.

Determination of preconsolidation pressure:

→ By using data obtained from consolidation test:

*: casagrande method for determination of pre-consolidation pressure:

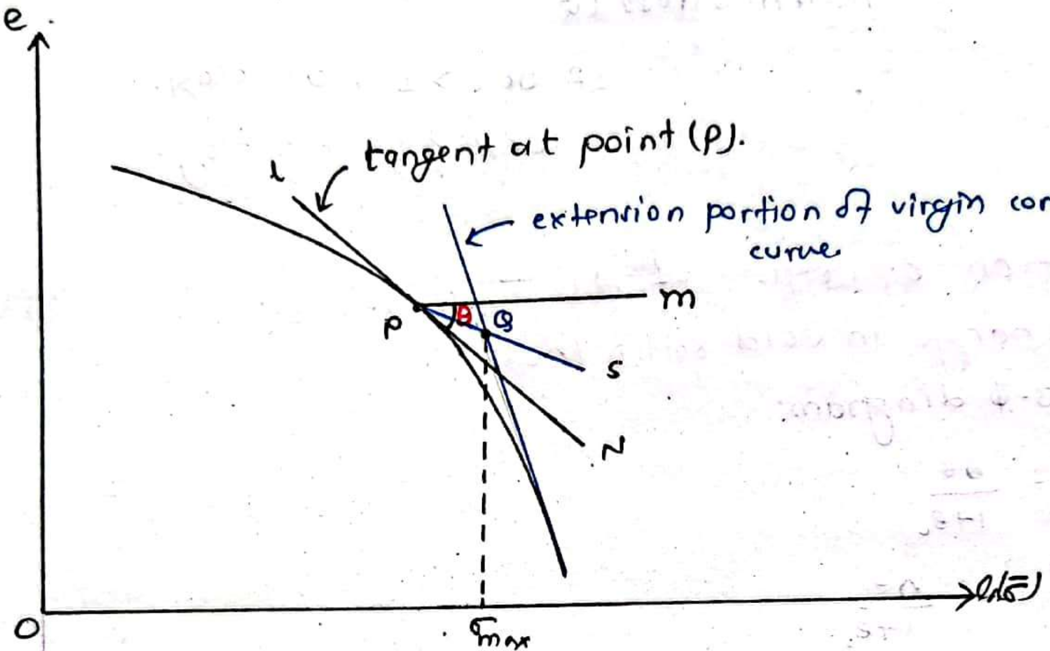


fig: consolidation curve for saturated clay showing the procedure for finding preconsolidation pressure.

Here, P = max^m curvature point

pm = Hz. line at point P.

lN = tangent at point P.

Ps = bisector of ∠mPN = θ

σ_{max} = max^m pressure or pre consolidation pressure.

• NC line is also known as virgin line of consolidation.

Procedure:

(i) select point, P having maximum curvature on consolidation curve.

(ii) Draw pm, horizontal line at point P.

(iii) Draw a tangent line lN at point P.

(iv) Bisect the angle made by line lN and pm (Ps = bisector of ∠mPN)

(v) Extend straight portion of virgin curve to intersect at bisector Ps.

(vi) Draw a vertical line from point Q to hz. axis indicating σ_{max} as preconsolidation pressure.

d. Overconsolidation ratio (OCR):

↳ It is the ratio of maximum stress to the present stress.

$$OCR = \frac{\sigma_{max}}{\text{present stress } (\bar{\sigma})}$$

If $OCR > 1$, OC clay.

$OCR < 1$, NC clay.

*: Calculation of settlement due to consolidation:

(i) Using change in void ratio (Δe):

from 3-ϕ diagram:

$$\frac{\Delta V}{V} = \frac{\Delta e}{1+e_0}$$

$$\frac{\Delta H \times A}{H \times A} = \frac{\Delta e}{1+e_0}$$

$$\Delta H = \frac{\Delta e}{1+e_0} \times H$$

$$S = \frac{\Delta e}{1+e_0} \times H \text{ ---- (i)}$$

; H = thickness of clay.

Q. A clay layer of 5m thickness and 0.32 void ratio was found in site. After 7 years of building construction, the void ratio of clay is found to be 0.22, then calculate the settlement due to consolidation occured in the 7 years.

Solution

clay thickness (H) = 5m

$e_1 = 0.32$

$e_2 = 0.22$

$\Delta e = 0.32 - 0.22 = 0.10$

Now, settlement due to consolidation,

$$S = \frac{\Delta e}{1+e_0} \times H$$

$$= \left(\frac{0.10}{1+0.32}\right) \times 5 = 0.38\,m \text{ Ans.}$$

(ii) Using change in a_v or m_v:

from σ-ϕ diagram,

$$\frac{\Delta v}{v} = \frac{\Delta e}{1+e_0}$$

$$\Rightarrow S = \frac{\Delta e}{1+e_0} \times H \text{ --- (i)}$$

also, $a_v = \dfrac{\Delta e}{\Delta \bar{\sigma}}$ $\Rightarrow \Delta e = a_v \times \Delta \bar{\sigma}$

substituting this in equn (i)

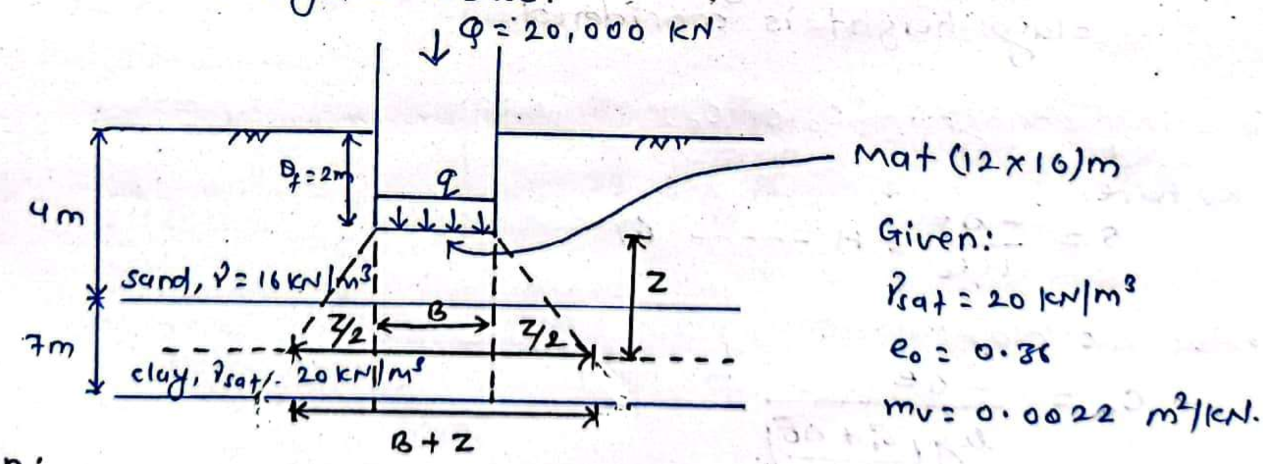
$$S_f = \frac{a_v \times \Delta \bar{\sigma}}{1+e_0} \times H$$

also, $\dfrac{a_v}{1+e_0} = m_v$

so, $S_f = m_v \times \Delta \bar{\sigma} \times H$ ----- (ii)

; where $\Delta \bar{\sigma}$ = increase in effective stress on clay after construction.

Q. Calculate the settlement due to foundation construction for the figure shown in figure below:



Solution;

As we know foundation distributes load in 1:2 (H:V) ratio. construction is required for calculating settlement.

Here, $Z = (4-2) + \frac{7}{2} = 2 + 3.5 = 5.5$ m

$$q = \frac{Q}{A} = \frac{20{,}000}{12 \times 16} = 104.167 \text{ kN/m}^2$$

$$\Delta\bar{\sigma} = \frac{\left(\frac{Q}{A} - \gamma D_f\right) \times B \times L}{(B+z) \times (L+z)}$$

$$= \frac{\left(\frac{20{,}000}{12\times16} - 16\times2\right) \times (12\times16)}{(12+5.5)(16+5.5)}$$

$$= \frac{(104.167 - 32)(12\times16)}{376.25}$$

$$= 36.82 \ kN/m^2$$

Also, $m_v = 0.0022 \ m^2/kN$

and $e_0 = 0.36$

we have,

$S_f = m_v \times \Delta\bar{\sigma} \times H$.

$= 0.0022 \times 36.82 \times 7$ ($\because$ consolidation only takes in clay)

$= 0.51548 \ m$

$= 515.48 \ mm$

Note: consolidation only takes in clay, Thus while taking H, only clay height is considered.

(iii) Using change in C_c or C_r:

we have,

$$S = \frac{-\Delta e}{1+e_0} \times H \text{ ----- (i)}$$

- For NC clay:

$$C_c = \frac{-\Delta e}{\log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

$$\Rightarrow \Delta e = \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right) \times C_c$$

substituting this value in equⁿ (i)

$$S = \frac{C_c \times \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}{1+e_0} \times H$$

$\therefore S_f = \frac{C_c}{1+e_0} \times H \times \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$

Q. For the same above questions if

$C_c = 0.27$

$e_0 = 0.36$

calculate the settlement of foundation.

solution;

Here, $\Delta\bar{\sigma} = \frac{\left(\frac{Q}{A} - \gamma D_f\right) \times (B \times L)}{(B+z)(L+z)}$

$= \frac{\left(\frac{20000}{12\times16} - 16\times2\right)\times(12\times16)}{(12+5.5)\cdot(16+5.5)}$

$= 36.86\ kN/m^2$

$\bar{\sigma}_0 = 16\times4 + 20\times8.5 - 9.81\times8.5$

$= 99.65\ kN/m^2$

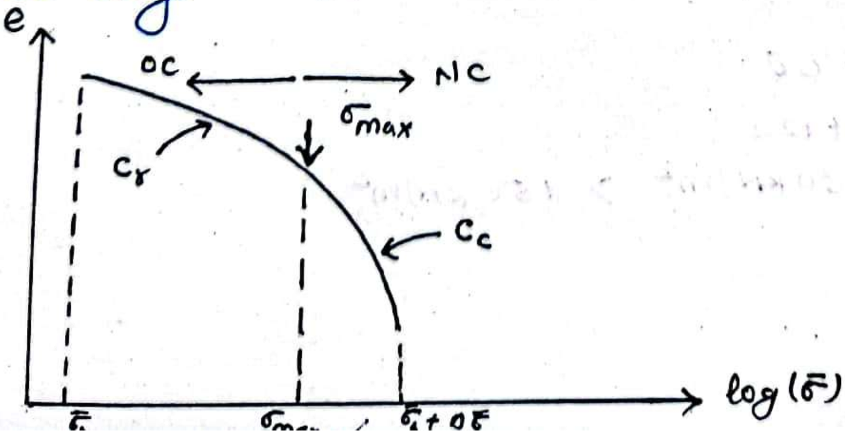
Now, $S_f = \frac{C_c}{1+e_0} \times H \times \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$

$= \frac{0.27}{1+0.36} \times 7 \times \log\left(\frac{99.65 + 36.86}{99.65}\right)$

$= 0.1899 m$

$= 189.95 mm$

• For OC clay



↳ If present stress on clay:

$$\bar{\sigma}_0 + \Delta\bar{\sigma} < \sigma_{max}$$

S_f = settlement on OC clay

and

$$S_f = \frac{C_r}{1+e_0} \times H \times \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

↳ If present stress on clay:

$$\bar{\sigma}_0 + \Delta\bar{\sigma} \geq \sigma_{max}$$

S_f = settlement on OC clay stage + settlement on NC clay stage.

$$S_f = \frac{C_r}{1+e_0} \times H \times \log\left(\frac{\sigma_{max}}{\bar{\sigma}_0}\right) + \frac{C_c}{1+e_0} \times H \times \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\sigma_{max}}\right)$$

Q. A clay 7m thickness with 110 kN/m² effective stress in mid is found in site. After construction of building, increase in effective stress is found to be 120 kN/m²,
Lab test of clay gives
$C_r = 0.07$; $C_c = 0.32$, $e = 0.27$
$\sigma_{max} = 150$ kN/m²,
calculate the settlement due to consolidation.

Solution:

$H = 7$ m $\qquad C_r = 0.07$

$\bar{\sigma}_0 = 110$ kN/m² $\qquad C_c = 0.32$

$\Delta\bar{\sigma} = 120$ kN/m² $\qquad e = 0.27$

$\sigma_{max} = 150$ kN/m²

Present stress $= \bar{\sigma}_0 + \Delta\bar{\sigma}$

$= 110 + 120$

$= 230$ kN/m² > 150 kN/m²

so, $S_f = \frac{C_r}{1+e_0} \times H \times \log\left(\frac{\bar{\sigma}_{max}}{\bar{\sigma}_0}\right) + \frac{C_c}{1+e_0} \times H \times \log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_{max}}\right)$

$= \frac{0.07}{1+0.27} \times 7 \times \log\left(\frac{150}{110}\right) + \frac{0.32}{1+0.27} \times 7 \times \log\left(\frac{110+120}{150}\right)$

$= 0.3793 \text{ m}$

$= 379.39 \text{ mm}$

Q: Differentiate between compaction and consolidation:

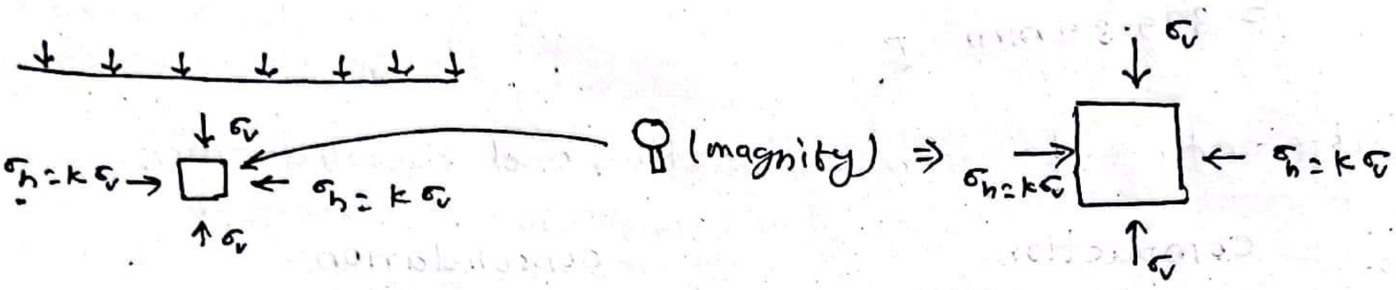
S. No.	Compaction	Consolidation
I.	Densification of soil mass by removing of air voids by application of mechanical energy.	Compression of soil mass due to expulsion of water from void under the effect of applied compressive stress.
II.	It is rapid and dynamic process.	It is a long term process occuring lifelong period.
III.	Applies to cohesive as well as cohesionless soil.	Applies to cohesive soil only.
IV.	Brought about by artificial or human agency.	Brought about by load or natural agency.
V.	soil involved is partially saturated.	soil involved is fully saturated.

2.1.6 Shear strength of soil:-

Introduction:

↳ The maximum shear stress that the soil mass can resist upto failure condition is termed as shear strength of soil.

$$S = \tau_{failure}.$$



∴ σ_v = vertical stress
σ_h = Hz. stress.

*: shear strength of soil is due to the:

1. interlocking between particles.
2. frictional resistance
3. intermolecular attraction force.

*: for coarse-grained soil:
eg: sand and gravel:
↳ due to 1, & 2

*: for c-φ soil:
eg: silty soil
↳ due to 2 & 3

*: for purely cohesive soil
eg: clay
↳ due to 3 only.

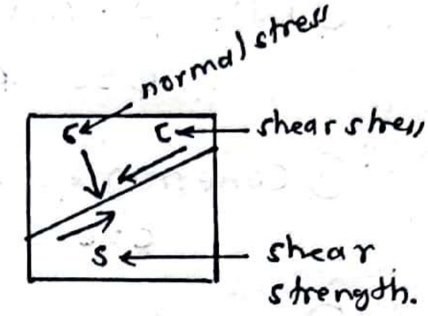
↳ Due to applied stress two types of stress is introduced in soil mass:

(i) shear stress

↳ stress acting along parallel to plane.

(ii) Normal stress

↳ stress acting perpendicular to plane.



Note:

↳ shear stress ले fail गर्छ

↳ Normal stress ले fail हुनबाट resist गर्छ!

↳ $\sigma_N < \sigma_S$ बढ्ने अनुपात.

*: Shear strength:

↳ The maximum shear stress that the soil mass can resist upto failure condition is termed as shear strength of soil.

$$S = \tau_{failure}$$

↳ It is the inherent property of soil and varies from soil to soil.

*: Causes of shear strength of soil:

a. Cohesive force (c):

↳ surface attraction force between soil particles which is significant in clay.

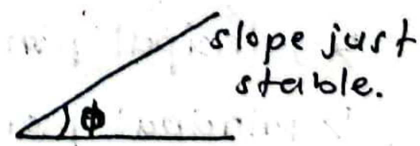
↳ Measured in terms of parameter 'c' i.e. cohesion (Unit kN/m^2).

b. Frictional Force: (ϕ):

↳ Friction between soil particles which is significant in coarse particles, eg: sand & gravel.

↳ Measured in terms of parameter (ϕ).

field density मा { • angle of internal friction or,
• angle of shearing resistance or,
• angle of repose (it measured in very loose condition)

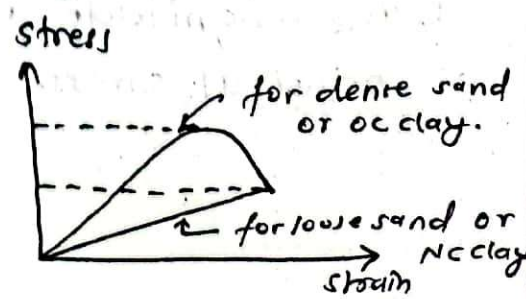


c. Interlocking between the soil particles:

↳ sand: dense to very dense sand

↳ clay: OC clay

Here, c and ϕ are strength parameter.



↳ Depending upon the value of c & ϕ, the soil are classified as:

① Pure cohesive soil : clay

$\phi = 0$ & c = very large.

② Cohesionless soil: sand, gravel.

$c = 0$ & ϕ = very large

③ Mixed soil: sandy clay

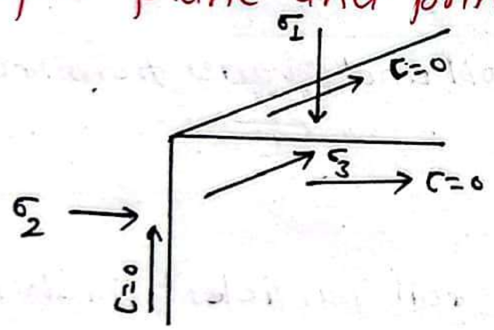
$c-\phi$ soil : clayey sand

: gravely clay

: sandy gravely clay.

decreasing proportion.

*: Principle plane and principle stress:



Principal plane:

↳ The plane within the soil mass where no shear stress developed i.e. $\tau = 0$, is known as principal plane.

↳ Three mutually perpendicular planes can be found within soil mass where no shear stress developed.

↳ principal plane ($\tau = 0$) can be horizontal, vertical or inclined.

↳ principal plane is the plane along which soil doesnot fails.

Principal stress:

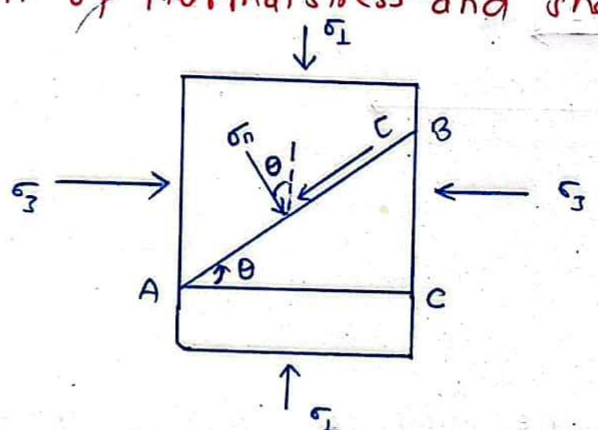
↳ The magnitude of direct stress on a principal plane is called principal stress.

↳ Principal plane can be classified into different types depending upon the value of principal stress:

Principal plane	Principal stress:
① Major principal plane	→ Major principal stress is acting (σ_1)
(ii) Intermediate principal plane	→ Intermediate principal stress is acting (σ_2)
(iii) Minor principal plane	→ Minor principal stress is acting (σ_3)

Note: $\sigma_1 > \sigma_2 > \sigma_3$

✱! Calculation of Normal stress (σ) and shear stress (τ) on any plane:



σ_n = normal stress

τ = tangential stress / shear stress

θ = angle of inclination of plane with Hz. in ACW direction.

Methods:

a. Analytical methods:

Applying equilibrium on triangle ABC.

$$\Sigma H_f = 0 \quad (\leftarrow +)$$

$$\sigma_3 \times BC + \tau \times AB \times \cos\theta - \sigma_n \times AB \times \sin\theta = 0 \quad \text{---- (i)}$$

$$\Sigma V_f = 0 \quad (\uparrow +)$$

$$\sigma_1 \times AC - \tau \times AB \sin\theta - \sigma_n \times AB \cos\theta = 0 \quad \text{---- (ii)}$$

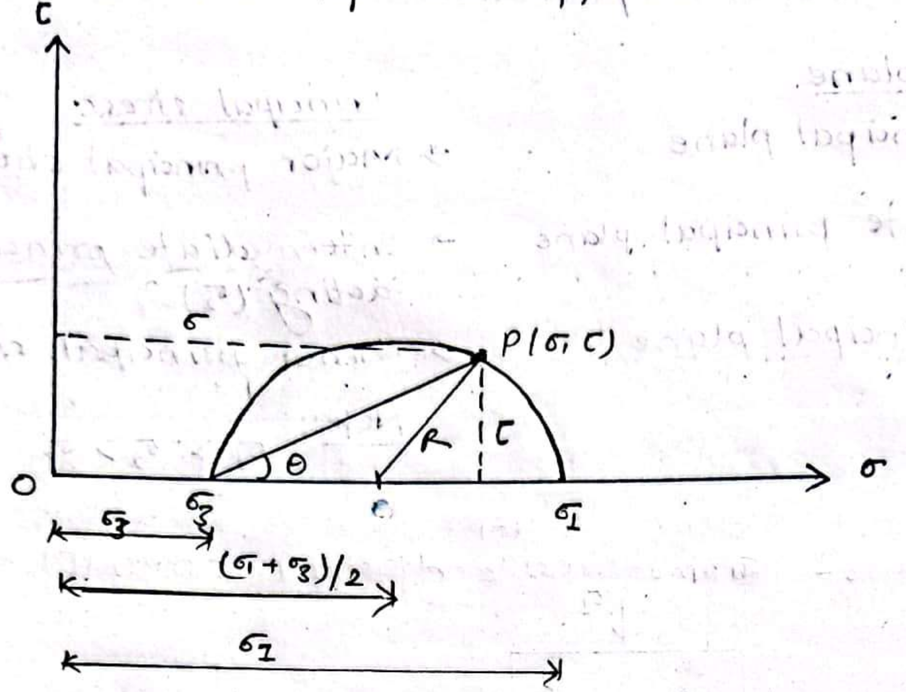
Solving these two equations;

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta \quad \text{and,}$$

$$\sigma_n = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta .$$

6. Graphical method: Mohr's circle method

↳ Mohr's circle represents applied stress on soil.

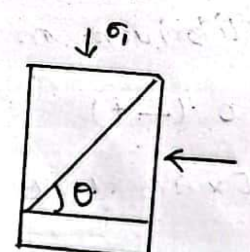


Here, $R = \dfrac{\sigma_1 - \sigma_3}{2}$

Q. Calculate τ and σ on a plane inclined by 10° with horizontal in ACW direction.

Given $\sigma_1 = 90\ kN/m^2$

$\sigma_3 = 30\ kN/m^2$



Solution:

we have

$$\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta$$

$$= \frac{90 + 30}{2} + \frac{90 - 30}{2} \cos(2 \times 10^\circ)$$

$$= 88.190\ kN/m^2$$

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta$$

$$= \frac{90 - 30}{2} \sin(2 \times 10^\circ)$$

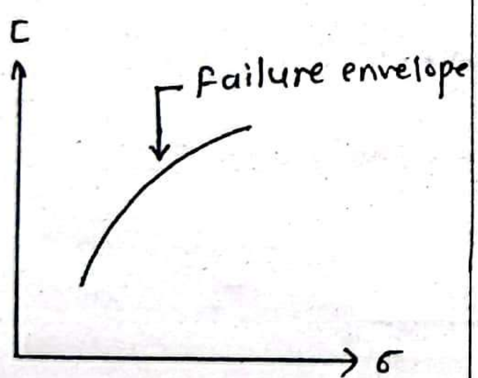
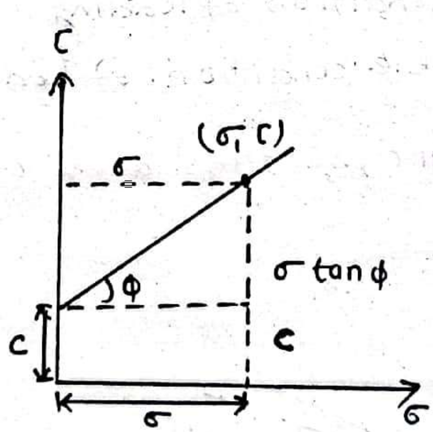
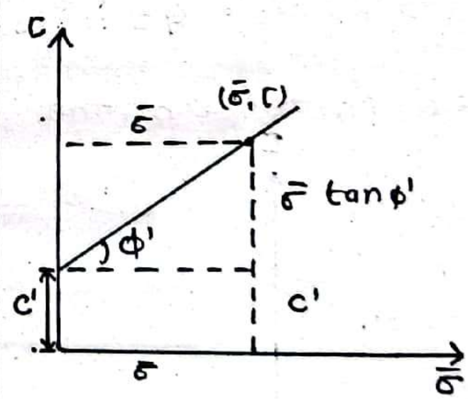
$$= 10.26\ kN/m^2$$

Theory of shear strength:

Failure envelope:

- Failure envelope is the line tangent to the mohr's circle at which failure occurs when it just touches the mohr's circle.

Comparision of Mohr, coulomb and Terzaghi's theory for shear strength

Mohr criteria	Coulomb criteria	Terzaghi's criteria.
- Initially, mohr states that shear strength of soil is a function of normal stress on failure plane $\tau = f(\sigma)$ ---- (i)  - Failure envelope: locus of shear strength.	- later on coulomb added on mohr's theory stating shear strength also depends on intra-particles attraction i.e. surface force (c). $\tau = c + \sigma \tan\phi$ ---(i)  - c & φ are shear strength parameters in terms of total stress	- Finally, terzaghi's modified mohr-coulomb theory stating that the shear strength is function of effective stress not of total stress. $\tau = c' + \bar{\sigma} \tan\phi'$ ---(i)  - c' and φ' are shear strength parameter in term of effective stress.

Failure criteria:

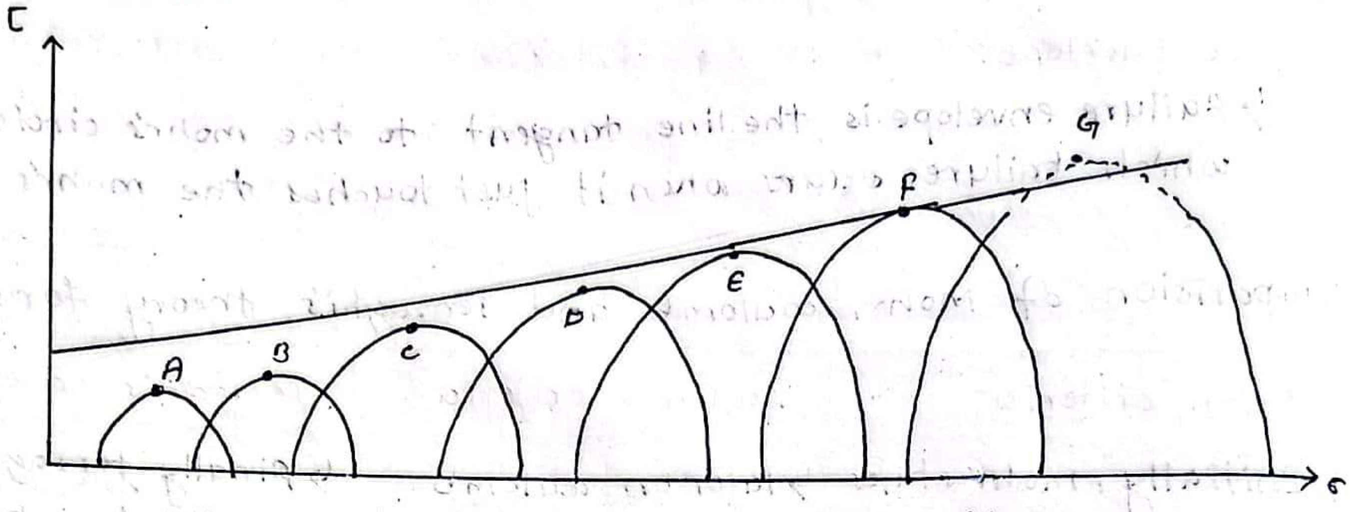


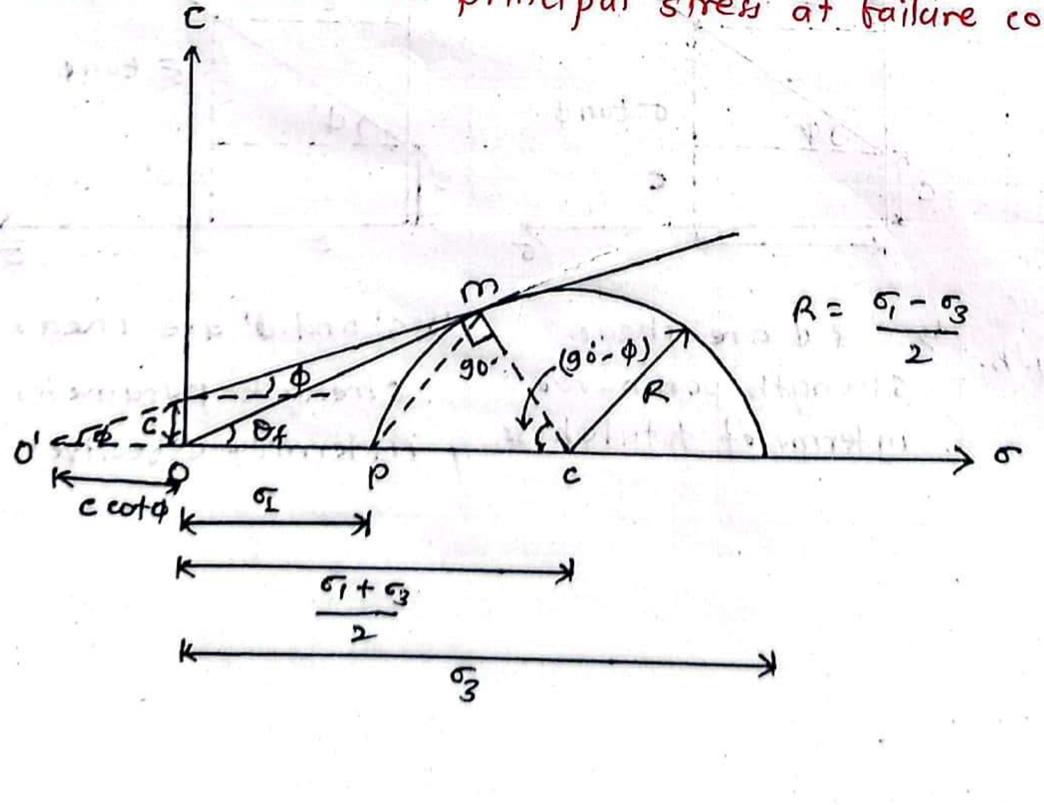
fig: diagram showing failure criteria.

↳ size of mohr's circle increases with increase of applied stress.

↳ The soil mass at loading condition on which mohr circle touches failure envelope is failure criteria.

- A, B, C, D, E : safe condition of loading
- F : Failure condition of loading
- G = impossible condition of loading.

Relation between the principal stress at failure condition:



from $\Delta O'MC$:

$$\sin\phi = \frac{CM}{O'C}$$

$$\sin\phi = \frac{CM}{O'O + OC}$$

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

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

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

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

$$\sigma_1 = \sigma_3 \tan^2(45^\circ + \phi/2) + 2c\tan(45^\circ + \phi/2)$$

$$\sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi}$$

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

↳ failure angle with horizontal:

$$\theta_f = (45^\circ + \phi/2).$$

for cohesionless soil, $c = 0$

$$\therefore \sigma_1 = \sigma_3 N_\phi.$$

*: Types of shear strength test: lab test:

↳ The determination of shear strength of soil involve the plotting of failure envelope and calculating shear strength parameter for the necessary condition.

(i) Direct shear test. (sandy soil)

(ii) Unconfined compression test. (clay)

(iii) Tri-axial test. (all type, specially clay)

(iv) Vane shear test (clay).

↳ Based on drainage condition, triaxial test are further classified as:

a. Undrained test (UU test)

b. Consolidated undrained test. (CU test)

c. Consolidated drained test. (CD test).

(i) Direct shear test:

Objective: to determine c and ϕ of soil.

Applicable: on sand or sandy soil.

Test arrangement:

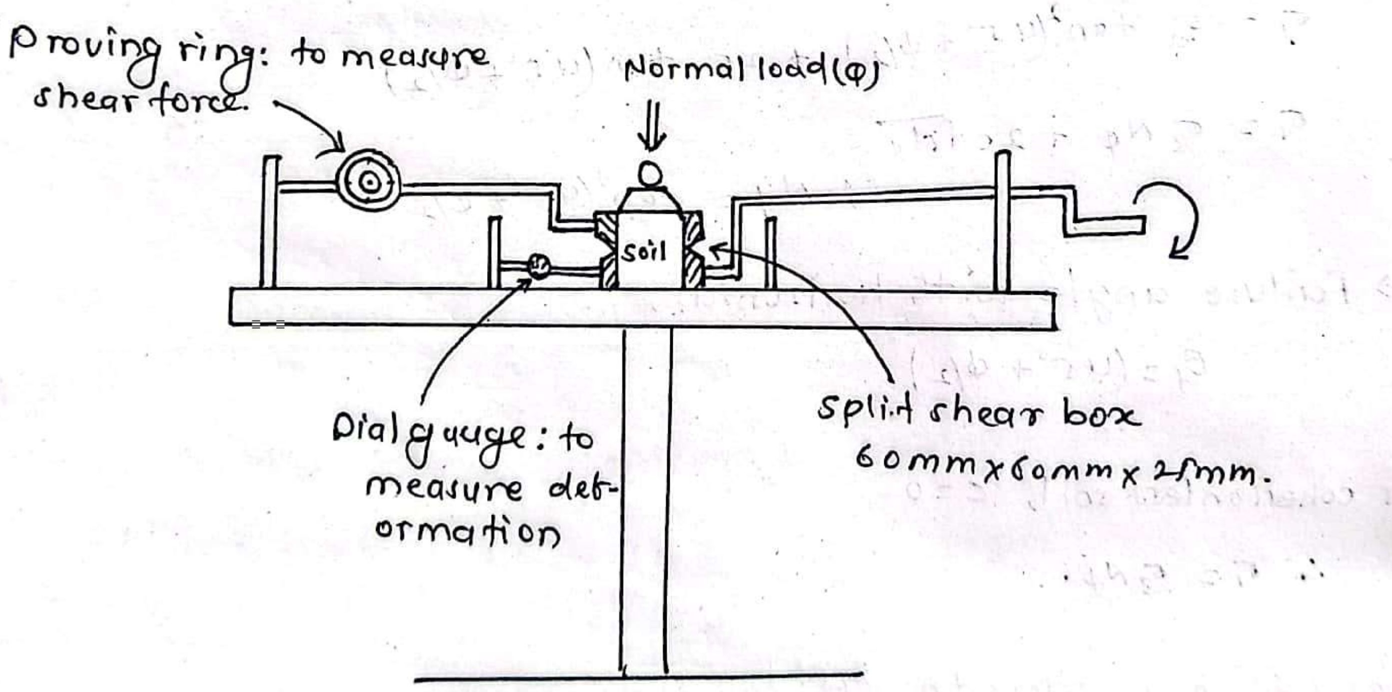


fig: experimental setup for direct shear test.

Measurement:

Test	Normal load	Shear force at failure (F)
I	Q_1	F_1
II	Q_2	F_2
III	Q_3	F_3

calculation,

Test	Normal stress $\sigma = Q/A$	shear stress at failure (τ) = F/A
I	σ_1	τ_1
II	σ_2	τ_2
III	σ_3	τ_3

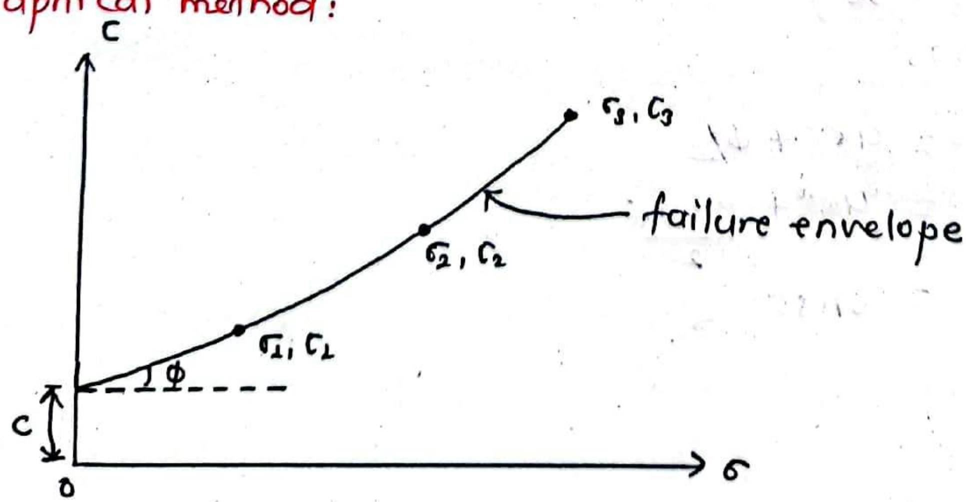
Analysis:

(i) Analytical method:

$$c, \phi \left[\begin{array}{l} \tau_1 = c + \sigma_1 \tan\phi \\ \tau_2 = c + \sigma_2 \tan\phi \\ \tau_3 = c + \sigma_3 \tan\phi \end{array} \right] \begin{array}{l} c, \phi \\ c, \phi \end{array}$$

↳ Take average value of c and ϕ.

(ii) Graphical method:



Q. A sand sample is tested on direct shear machine applying 90 KN load as normal load and fails at 60 KN shear load. If the size of the sample was 60mm x 60mm, calculate shear strength parameter and failure angle.

Solution:

$Q = 90\,KN$

$F = 60\,KN$

$A = 60mm \times 60mm$

$\sigma = Q/A = \frac{90}{60 \times 60} = \underline{0.025\ KN/mm^2}$

$\tau = F/A = \frac{60}{60 \times 60} = \underline{0.0167\ KN/mm^2}.$

We know,

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

for sand $c = 0$

$\therefore \tau = \sigma \tan\phi$

or, $\frac{F}{A} = \frac{Q}{A} \times \tan\phi$

or, $F = Q \times \tan\phi$

or, $\tan\phi = \frac{F}{Q}$

or, $\tan\phi = \frac{60}{90}$

or, $\phi = \tan^{-1}\left(\frac{60}{90}\right)$

$\therefore \phi = 33.70°$

Failure angle $= 45° + \phi/2$

$= 45° + \frac{33.70°}{2}$

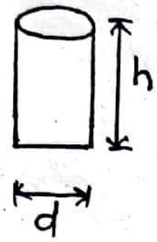
$= 61.85°$ Ans.

(ii) Unconfined compression test: (Undrained test)

Objective: to find out undrained cohesion (c_u) or undrained shear strength (S_u) of soil.

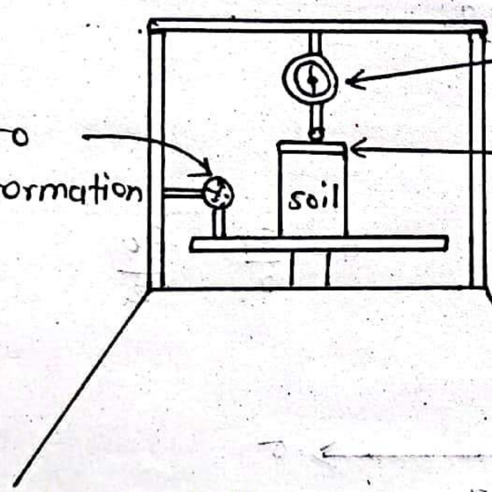
Applicable: on clay (mostly saturated clay)

soil sample:



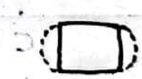
- prepared from UD soil.
- $d = 38$ mm
- $h = 76$ mm.
- loading rate: 1.25 mm/min.

Test arrangement:



calibration: 1 div = 0.01 mm

$$A_0 = \frac{\pi d^2}{4}$$



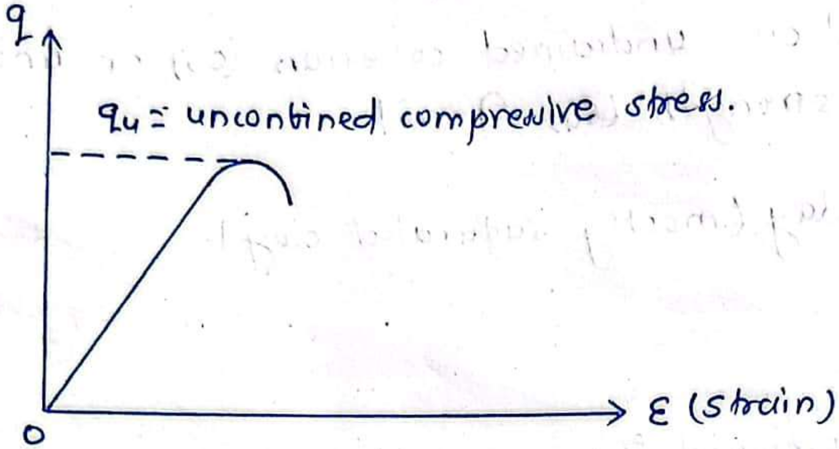
$$A = \frac{A_0}{1-\varepsilon}$$

soil sample after loading

Measurement and calculation:

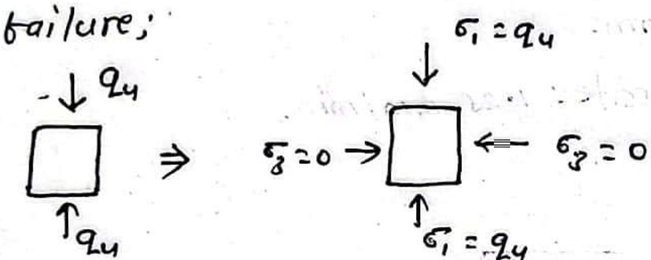
Dial gauge reading (div.)	Proving ring reading (div.)	compressive load Q = col. 2 × const.	Deformation Δh = col. 1 × 0.01 (mm)	strain $\varepsilon = \frac{\Delta h}{h}$	Area $A = \frac{A_0}{1-\varepsilon}$	Compressive stress $q = Q/A$
0	0	0	0	0	A_0	0
10	-	-	-	-	-	-
20	-	-	-	-	-	-
30	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
upto failure	-	-	-	-	-	-

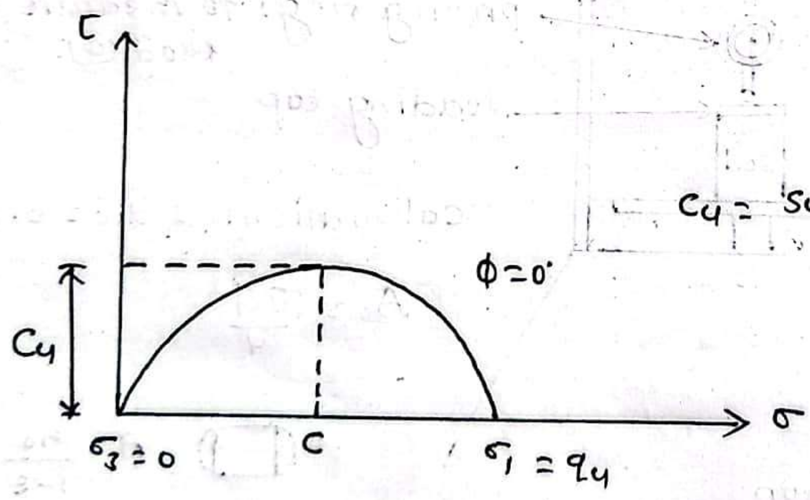
Result:



Analysis:

At failure;





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

Q. A clay sample of 38 mm diameter and 76 mm height fails at 27N load after 7mm deformation during unconfined compression test. calculate the undrained cohesion.

solution;

At failure,

$q = 27$ N

$\Delta h = 7$mm

$h = 76$mm

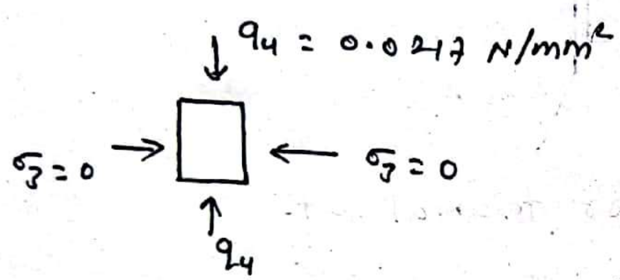
$d = 38$mm.

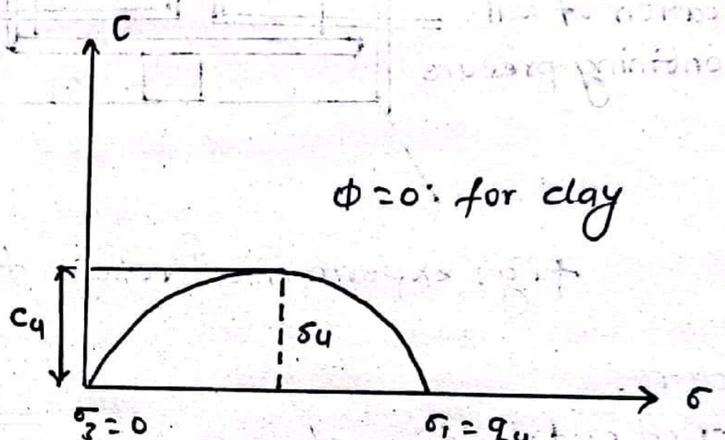
$$A_0 = \frac{\pi d^2}{4} = \frac{\pi \times 38^2}{4} = 1134.114 \ mm^2$$

$$\varepsilon = \frac{\Delta h}{h} = \frac{7}{76} = 0.0921$$

$$A = \frac{A_0}{1-\varepsilon} = \frac{1134.114}{1-0.0921} = 1241.16 \ mm^2$$

$$q_u = \frac{P}{A} = \frac{27}{1241.16} = 0.0217 \ N/mm^2$$





$$\therefore c_u = \frac{q_u}{2} = \frac{0.0217}{2} = 0.01085 \ N/mm^2.$$

Note:

In Nepal:

- clay → unconfined compression test.
- sand → direct shear test.

(iii) Tri-axial test:

Objective: to find c and ϕ of soil.

Applicable: for all types of soil but special test for clay.

Test arrangement:

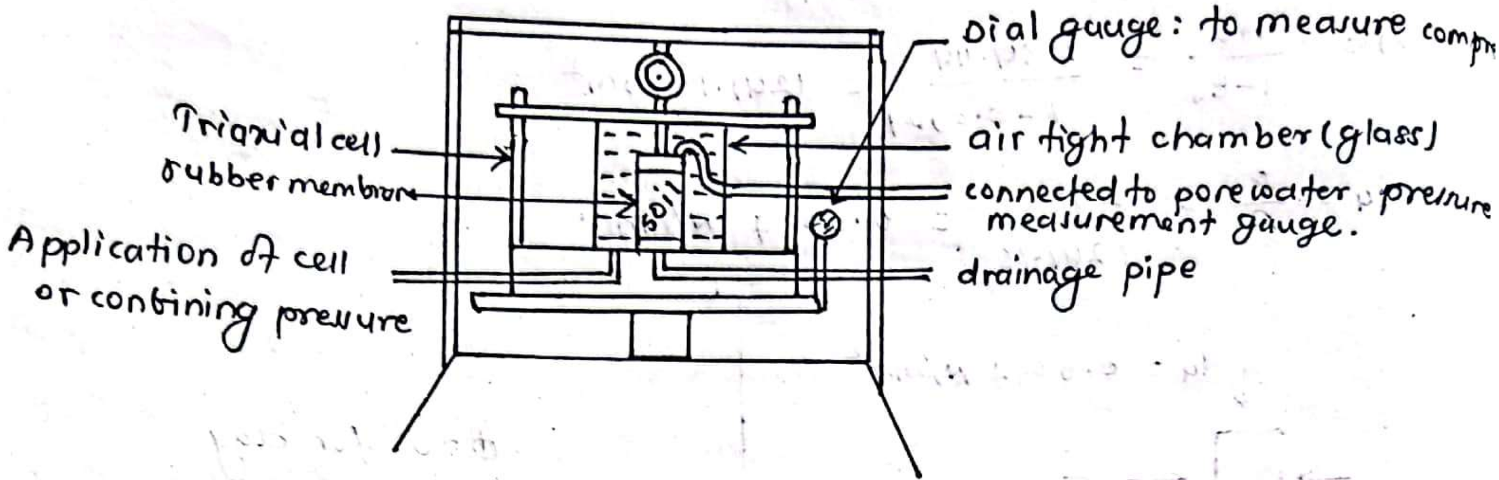


fig: experimental setup for triaxial test.

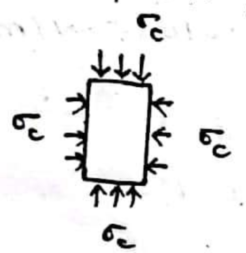
Procedure:

↳ carried out in two stage.

stage (i):

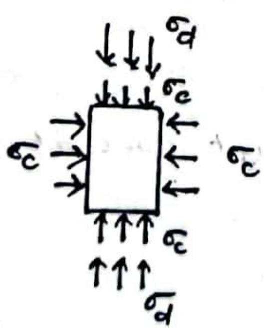
↳ Application of cell pressure.

↳ generally 20, 30, 40 cell pressure use হয়।



stage (ii): shearing of soil

↳ Application of mechanical or deviator stress. (σ_d)



$\sigma_1 = \sigma_c + \sigma_d$

$\sigma_3 = \sigma_c$; σ_d = deviator stress.

Types of triaxial test:

↳ Depending on drainage condition at two stage of triaxial test there are of three types:

a. Unconsolidated undrained test. (UU test)

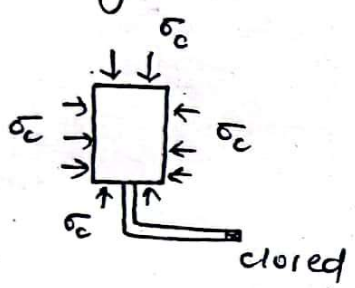
b. consolidated undrained test (CU test)

c. consolidated drained test (CD test)

a. Unconsolidated undrained test (UU-test)

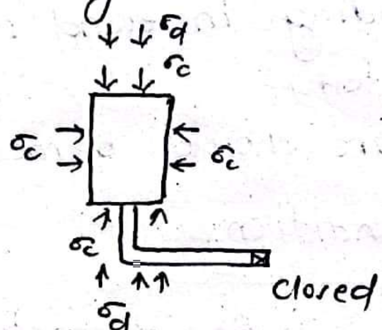
↳ At the two stage of the test.

- stage - I:



unconsolidated

- stage - II:



undrained

↳ second stage start immediately

↳ fast rate of loading in second stage.

↳ quick test (q- test)

↳ pore water pressure should be measured.

Relevant field condition:

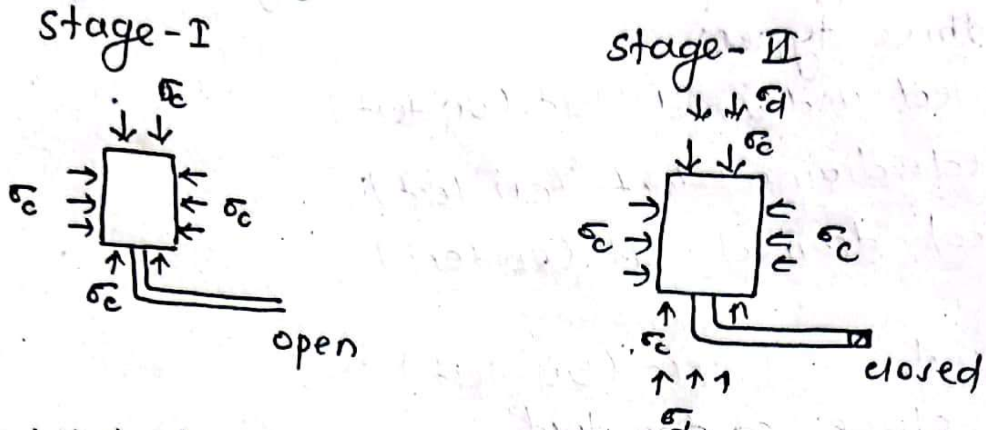
clay

↳ Rapid construction

↳ shearing of clay by dam is equivalent to UU test.

b. Consolidated Undrained test: (CU test)

↳ At two stage of test:



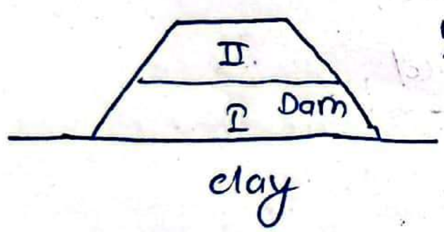
↳ sufficient time is provided for consolidation of stage-I.

↳ fast rate of loading in second stage

↳ Rapid test (R-test)

↳ pore water pressure should be measured.

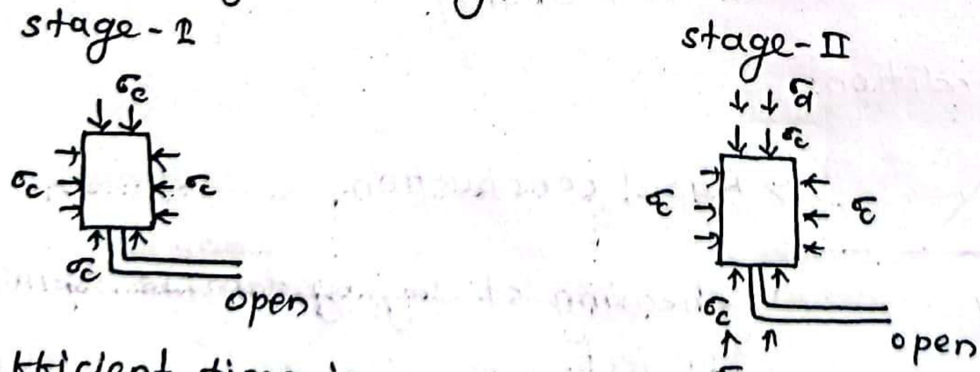
Relevant field condition:



↳ Rapid construction but in two stage with time gap.

c. Consolidated drained test: (CD test)

↳ At two stage of loading.



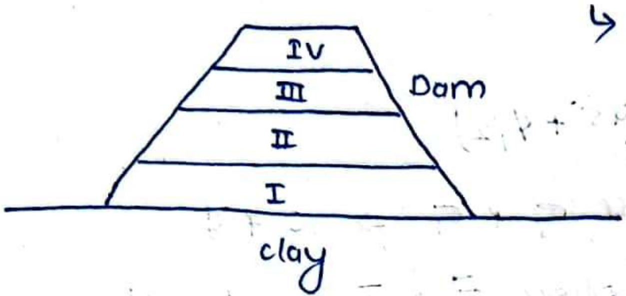
↳ sufficient time is provided for consolidation at stage-I.

↳ slow rate of loading in second stage

↳ slow test (S-test)

↳ Pore water pressure should not be measured.

Relevant field condition:



↳ very slow construction but in many stages with time gap.

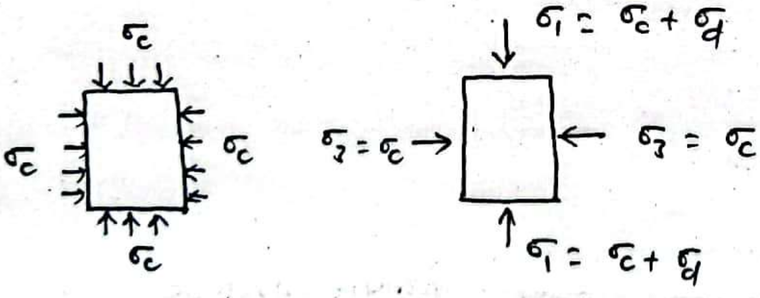
Note:

↳ Generally unconsolidated undrained test (UU test) is performed.

Measurement:

Test	Cell Pressure	Deviator stress at failure (σ_d)	Porewater pressure at failure
I	$\sigma_c(I)$	$\sigma_d(I)$	$u_{(I)}$
II	$\sigma_c(II)$	$\sigma_d(II)$	$u_{(II)}$
III	$\sigma_c(III)$	$\sigma_d(III)$	$u_{(III)}$

Calculation;



Test	$\sigma_3 = \sigma_c$	$\sigma_1 = \sigma_c + \sigma_d$	$\bar{\sigma}_1 = \sigma_1 - u$	$\bar{\sigma}_3 = \sigma_3 - u$
I	$\sigma_3(I)$	$\sigma_1(I)$	$\bar{\sigma}_1(I)$	$\bar{\sigma}_3(I)$
II	$\sigma_3(II)$	$\sigma_1(II)$	$\bar{\sigma}_1(II)$	$\bar{\sigma}_3(II)$
III	$\sigma_3(III)$	$\sigma_1(III)$	$\bar{\sigma}_1(III)$	$\bar{\sigma}_3(III)$

Analysis:

(I) Analytical method:

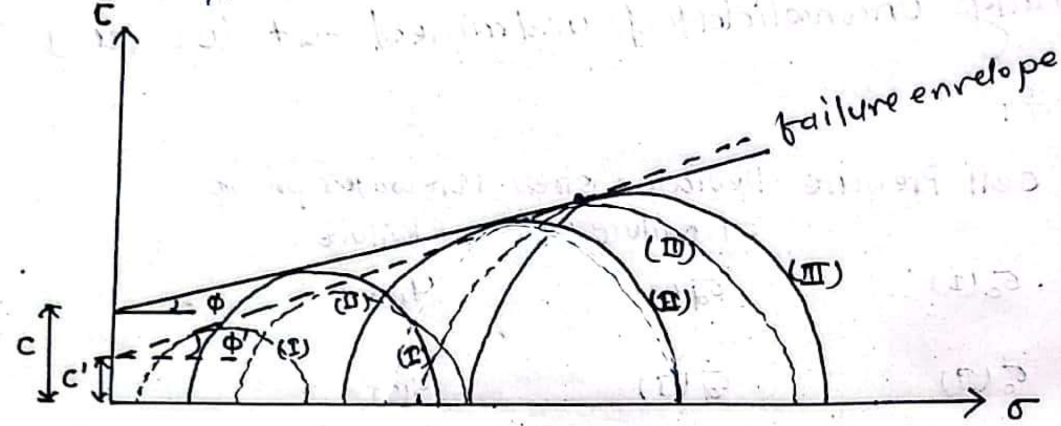
$$\sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi} \quad ; N_\phi = \tan^2(45^\circ + \phi/2).$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 N_{\phi'} + 2c'\sqrt{N_{\phi'}}$$

Total stress σ_3 & σ_1 $\Rightarrow$ c & ϕ

Effective stress $\bar{\sigma}_3$ & $\bar{\sigma}_1$ $\Rightarrow$ c' & ϕ'

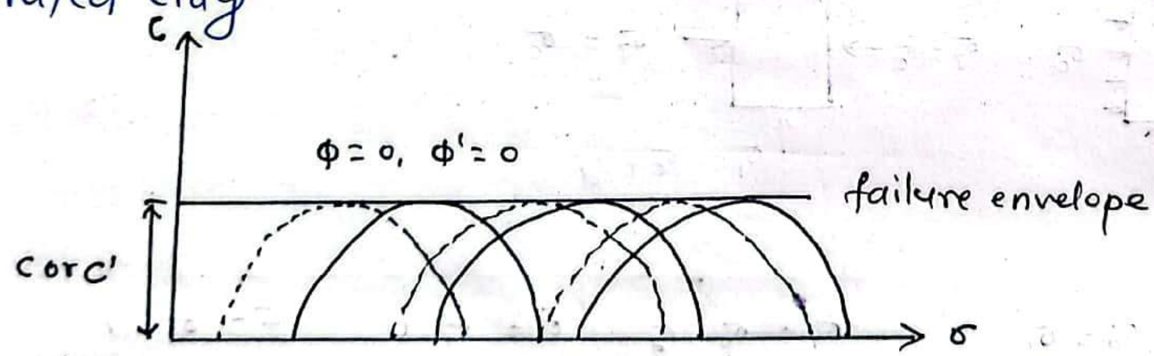
(II) Graphical method:



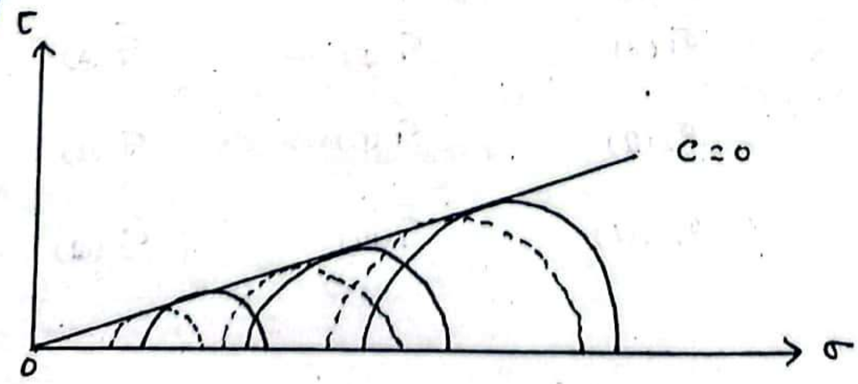
$c' < c$
$\phi' > \phi$ } Always hold's true.

*: Some-special results:

- Saturated clay



- NC clay

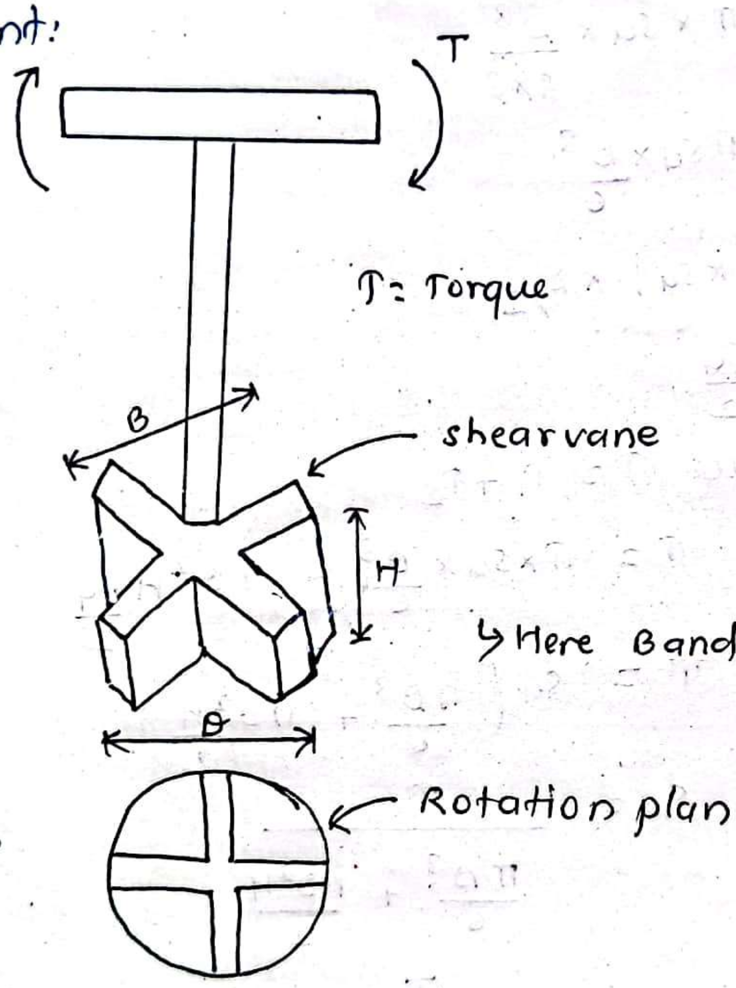


(iv) Vane shear test: (for clay)

Objective: to find undrained cohesion (c_u) or undrained shear strength (S_u).

Applicable: on clay

Test arrangement:



↳ Here B and H are known value.

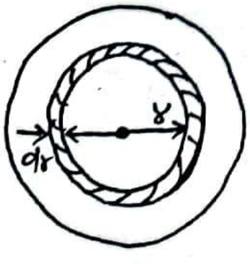
Here D = B

Measurement:

↳ Maximum torque (T) required to rotate vane during full depth drive of vane into soil by push and rotation.

Derivation:

Torque (T) = Resistance by soil at x-section (T_1) + Resistance by soil at surface area (T_2).



Here $T_1 = 2\int_0^{D/2} (2\pi r\,dr) \times s_u \times r$

; $2\pi r \times dr$ = area of strip.

2 is for both top & bottom resistance.

$$T_1 = 2\int_0^{D/2} 2\pi r\,dr \times S_u \times r$$

$$= 4\pi \times S_u \times \int_0^{D/2} r^2 dr$$

$$= 4\pi \times S_u \times \left[\frac{r^3}{3}\right]_0^{D/2}$$

$$= 4\pi \times S_u \times \frac{D^3}{8\times 3}$$

$$= \pi S_u \times \frac{D^3}{6}$$

$$T_2 = (\pi D \times H \times S_u) \times D/2$$

$$= \frac{\pi D^2 H S_u}{2}$$

So, total torque $T = T_1 + T_2$

$$T = \pi \times S_u \times \frac{D^3}{6} + \frac{\pi D^2 H S_u}{2}$$

$$T = S_u\left(\frac{\pi D^3}{6} + \frac{\pi D^2 H}{2}\right)$$

$$S_u = \frac{T}{\frac{\pi D^3}{6} + \frac{\pi D^2 H}{2}}$$

$$S_u = \frac{T}{\pi D\left(\frac{D^2}{6} + \frac{DH}{2}\right)} \quad \text{----- (i)}$$

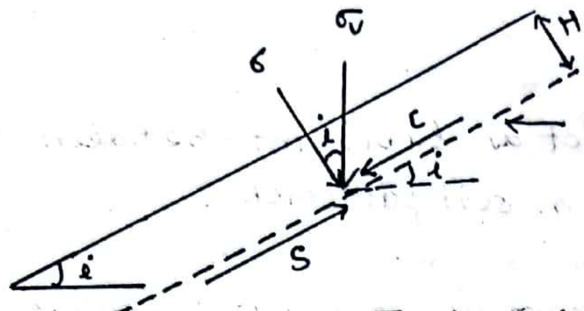
Equⁿ ① is used for both top & bottom resistance case.

↳ for bottom resistance only,

$$S_u = \frac{T}{\pi D\left(\frac{D^2}{12} + \frac{DH}{2}\right)} \quad \&$$

2.1.8 Stability of slopes:

concept:



$\sigma_v = \rho H \cos i$

If $S > C$: stable slope

$S \approx C$: critical slope

$S < C$: failure / unstable slope.

↳ The surface along which the soil mass slides when the failure of an earth slope occurs is known as critical s. of failure.

*: Causes of slope failure:

↳ The causes of slope failure or slope instability can be categorized into two main group.

(I) Increase in shear stress on soil.

(II) Decrease in shear strength of soil.

(I) Increase on shear stress on soil: (C)

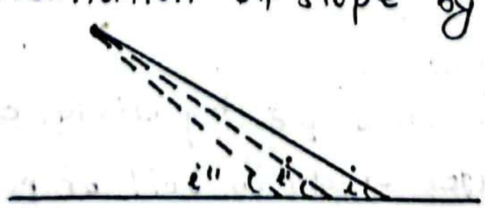
a. Loading: By construction.

b. Raining:

↳ increase the weight of soil on slope due to wetting.

↳ when water flows along the slope, then shear stress increases by seepage force.

↳ increase in inclination of slope by erosion due to continuous rainfall.



$i'' > i' > i$

c. dynamic loading: earthquake.
↳ certain components of dynamic loading acts downward on slope increasing shear stress.

(II) Decrease in shear strength of soil: (S)

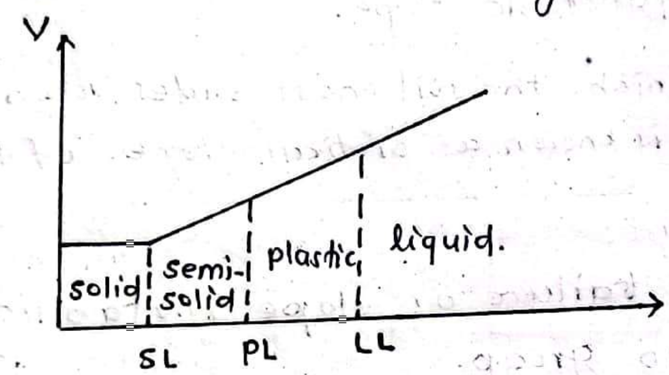
a. Raining:

↳ wet the soil particles; act as lubricant between soil particles and reduce friction between soil particles.

↳ rise of water on slope increases porewater pressure on slope and reduce effective stress on slope.

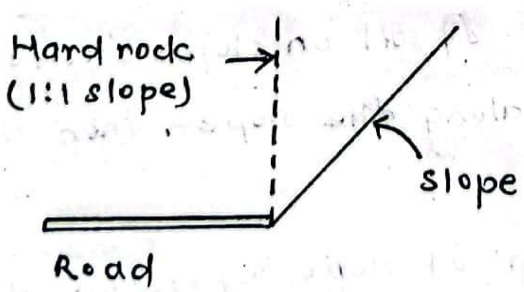
↳ For clayey slope:

- increase in moisture change the behaviour in soil.



b. Erosion
↳ loss of fines of soil on slope, loss in binding material, reduction in cohesion.

c. Weathering of rock on slope.
↳ reduction in strength of soil.



d. Cyclic dynamic loading:
↳ loosen the soil mass on slope by disturbance.
↳ Generates or elongates cracks in soil or rocks on slope.

Note:

↳ Many slope failures are associated with the excessive presence of water during heavy rainfall and flood. The introduction of such water content to the soil contributes to both increase in shear stress as well as reduction in shear strength due to increase in pore water pressure.

Types of slope:

↳ Slopes can be categorized into two types:

1. Finite slope.
2. Infinite slope.

① Finite slope:

↳ slope of same inclination, i extended to short distance.

eg: slope of dam, embankment.

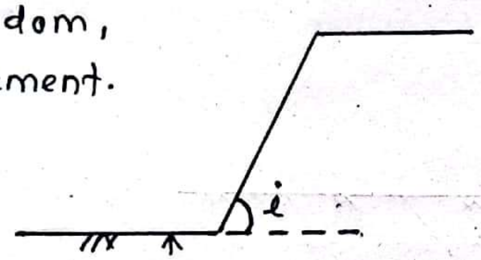


fig: finite slope.

⑪ Infinite slope:

↳ slope of same inclination extended to very large distance.

eg: natural slope.

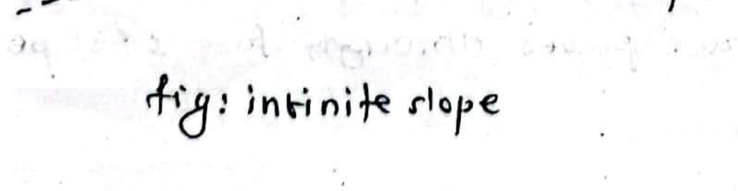
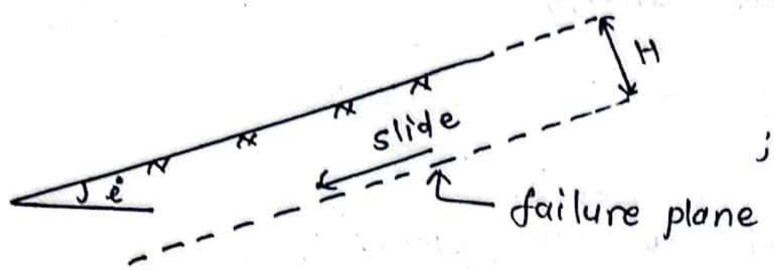


fig: infinite slope

Types of slope failure:

↳ A slope may undergo to any types of failure:

① Translational or plane failure:-

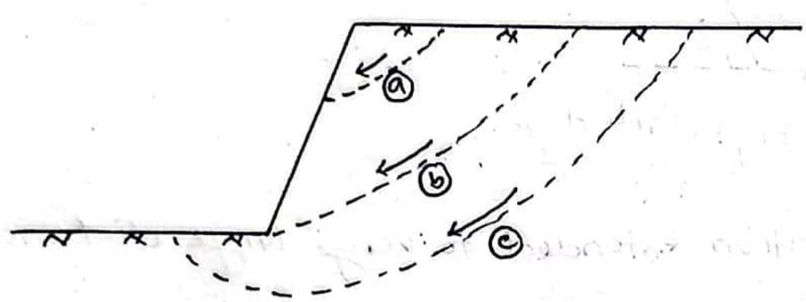


; i = inclination of slope with Hz.

↳ soil layer slide down making the failure plane parallel to the surface.

↳ potential : infinite slope

⑪ Rotational failure:



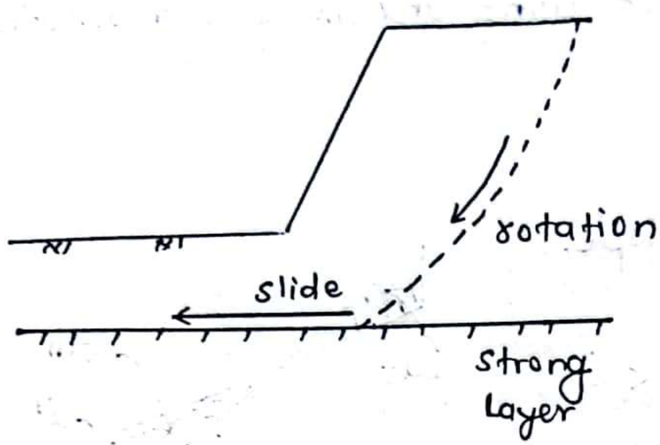
↳ Failure wedge rotate about a point making spoon shape failure surface.

↳ potential: finite slope.

↳ Rotational failure can be further be classified as:

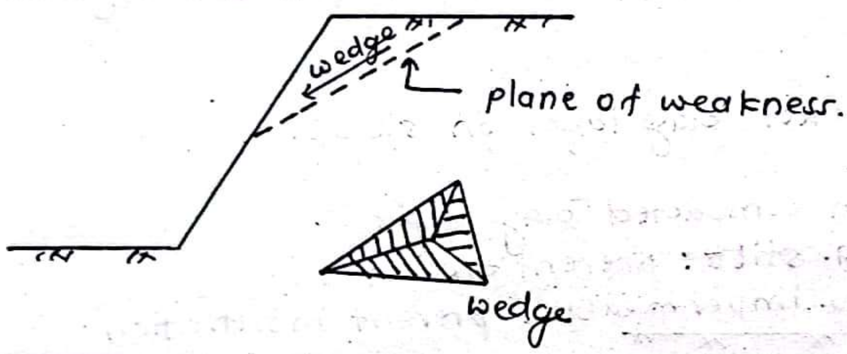
- type (a) : slope rotation failure.
  ↳ failure surface passes through face of slope.
- type (b) : toe failure.
  ↳ failure surface passes through toe of slope.
- type (c): base failure.
  ↳ failure surface passes below toe of slope.

(iii) Compound / composite failure:



↳ combination of both rotational and translational failure.

(iv) Wedge failure:



↳ triangular wedge fails down making angular failure surface.
↳ occurs usually in dry condition.

Critical failure surface & FOS:

↳ In case of infinite slope, it is generally assumed that failure surface is plane, whereas in finite slope comprising of cohesive soil, it is not always possible to determine exact form of critical surface.

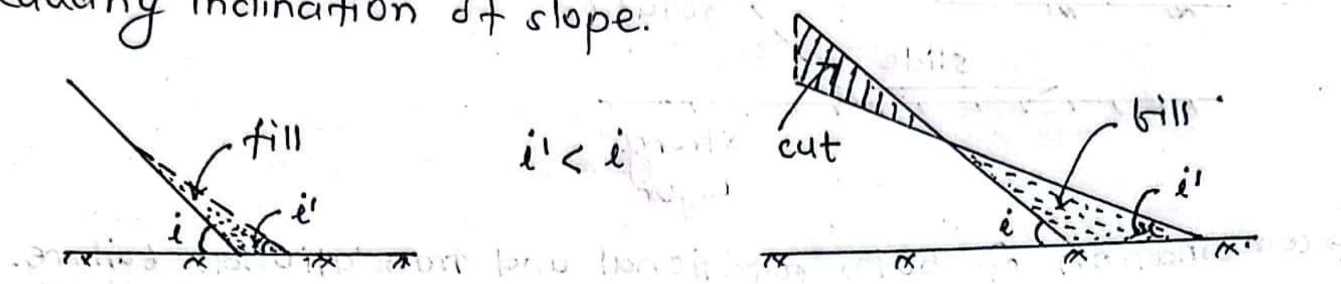
↳ The critical failure surface is that surface at which the shear resistance is minimum i.e. FOS is minimum.

Preventive measures of slope failure:

↳ slope failure can be prevented using any one or combination of following technique:

① Removal of load

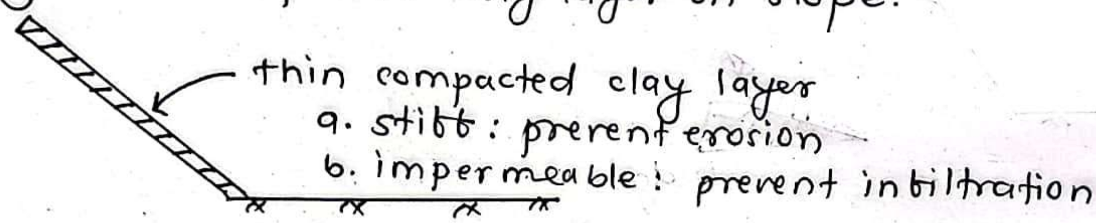
② Reducing inclination of slope.



③ Bio-engineering

↳ application of vegetative system with small civil engineering structures.

④ Creating thin compacted clay layer on slope.



↳ this can be done on silty or sandy slope without vegetation.

⑤ Proper water management on slope:

↳ By constructing surface drainage and sub-surface drainage, benching etc.

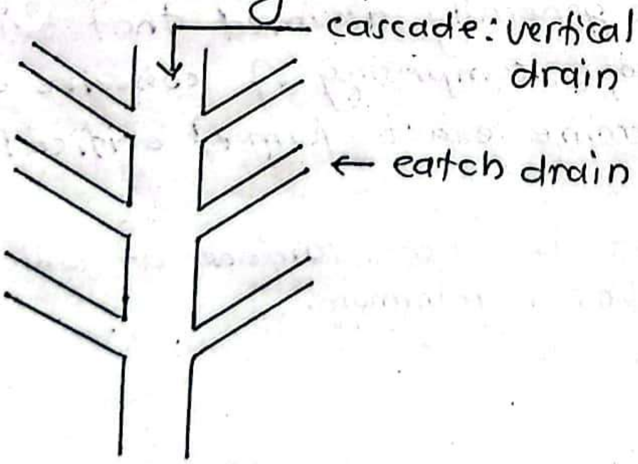
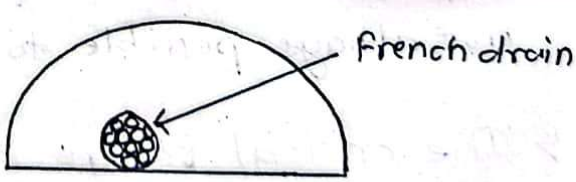


fig: surface drainage



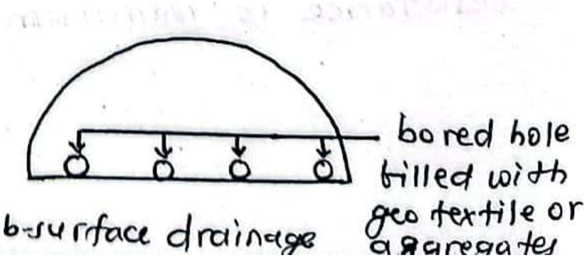
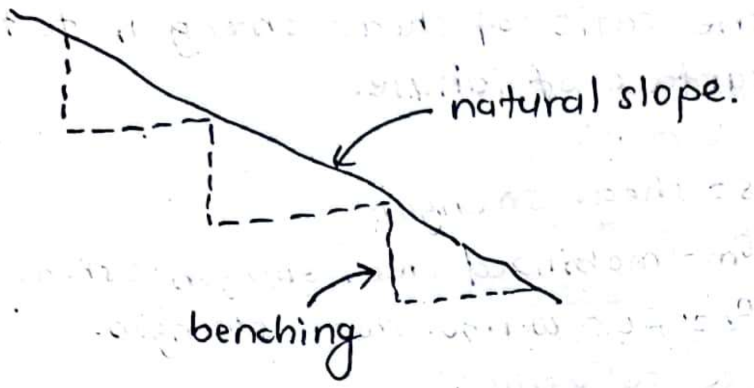
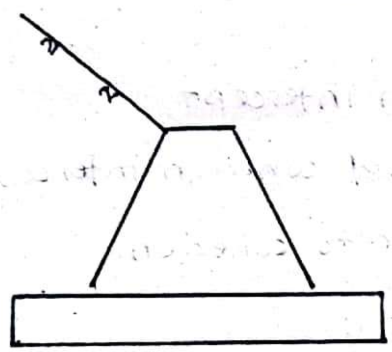


fig: sub-surface drainage

Benching



(vi) construction of retaining wall:



(vii) Rock slope stability: Rock bolting

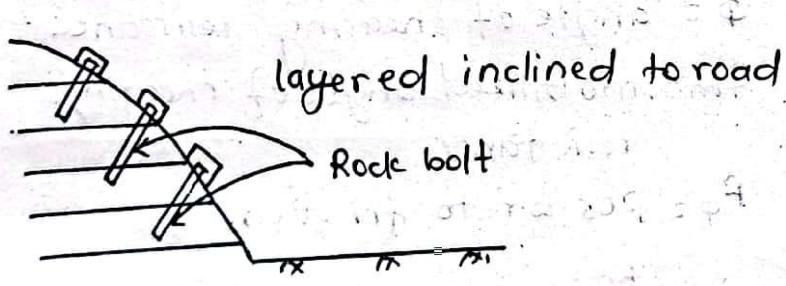
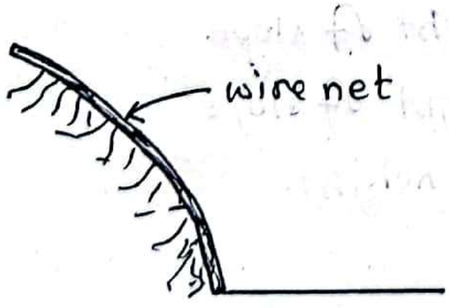
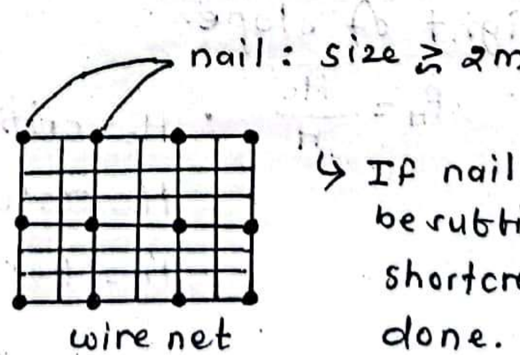


fig: rock bolting

- Incase of weathered rock:





↳ If nail cannot be be sufficient to handle, shortcreting can be done.

Condition for factor of safety:

a. Factor of safety w.r. to shear strength:

↳ It is defined as the ratio of shear strength to the shear stress along the surface of failure.

$$F_s = \frac{s}{\tau_m}$$; s = shear strength

τ_m = mobilized shear strength (shear stress)

F_s = FOS w.r. to shear strength.

b. Factor of safety w.r. to cohesion:

↳ It is defined as the ratio of available cohesion intercept, to mobilized cohesion intercept.

$$F_c = \frac{c}{c_m}$$; c = cohesion intercept

c_m = mobilized cohesion intercept.

F_c = FOS w.r. to cohesion.

c. Factor of safety w.r. to friction:

↳ It is defined as the ratio of available shear strength to mobilized frictional strength.

$$F_\phi = \frac{\tan\phi}{\tan\phi_m}$$; ϕ = angle of shearing resistance

ϕ_m = mobilized angle of shearing resistance.

F_ϕ = FOS w.r. to friction,

d. Factor of safety w.r. to height:

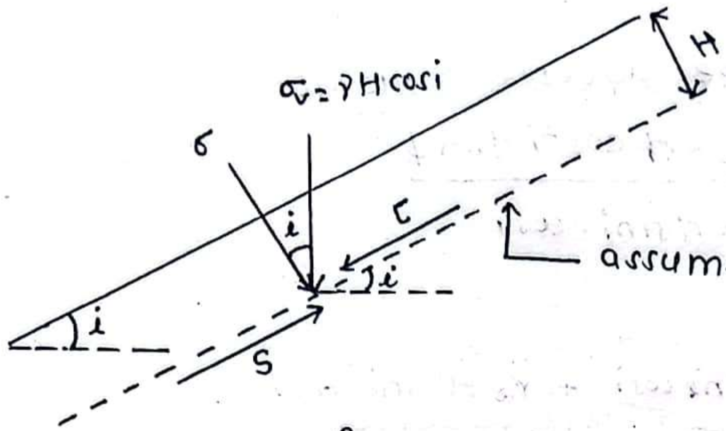
↳ It is the ratio of critical height of slope to actual height of slope.

$$F_H = \frac{H_c}{H}$$; H_c = critical height of slope

H = actual height of slope

F_H = FOS w.r. to height.

*: Stability Analysis of Infinite slope:



from figure;

$$\sigma = \sigma_v \cos i = \gamma H \cos^2 i$$

$$\tau = \sigma_v \sin i = \gamma H \cos i \cdot \sin i$$

Analysis procedure:

(i) Assume failure plane.

(ii) calculate FOS

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

$$= \frac{c + \sigma \tan\phi}{\tau}$$

$$= \frac{c + \gamma H \cos^2 i \tan\phi}{\gamma H \cos i \cdot \sin i}$$ (General equ^n for all types of soil)

- If

FOS > 1, stable

FOS = 1, critical

FOS < 1, failure

(iii) Critical plane calculation

↳ having least FOS.

* Most affected parameter:

(a) water condition

1. dry slope

$\gamma = \gamma_{dry}$

2. Submerged slope.

$\gamma = \gamma_{sub}$

3. Seepage along slope: adverse situation

$$FOS = \frac{c + \gamma_{sub} \times H \cos^2 i \tan\phi}{\gamma_{sat} \times H \sin i \cdot \cos i}$$

Proof:

$$\tau = \underbrace{\gamma_{sub} H \sin i \cos i}_{\text{shear stress}} + \underbrace{\gamma_w H \sin i \cdot \cos i}_{\text{seepage force}}$$

$$\tau = (\gamma_{sub} + \gamma_w) H \cos i \sin i$$

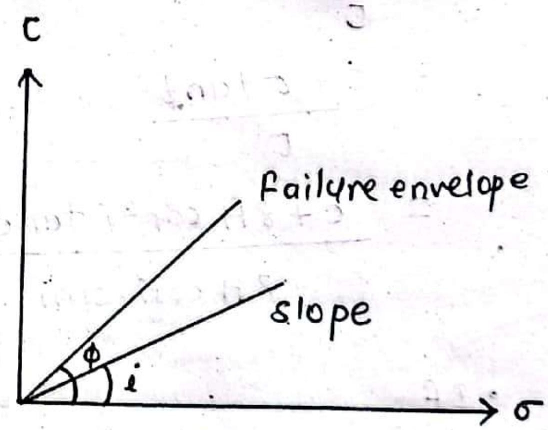
$$\therefore \tau = \gamma_{sat} H \sin i \cos i$$

⑥ Soil type:

① Sand: cohesionless soil.

$c = 0$

$$FOS = \frac{\tan\phi}{\tan i}$$



$i < \phi$: stable

$i = \phi$: critical.

$i > \phi$: failure.

Proof:

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

$c = 0$,

$$FOS = \frac{0 + \cancel{\gamma H} \cos^{\cancel{2}} i \tan\phi}{\cancel{\gamma H} \cancel{\cos i} \sin i}$$

$$FOS = \frac{\tan\phi \cdot \cos i}{\sin i}$$

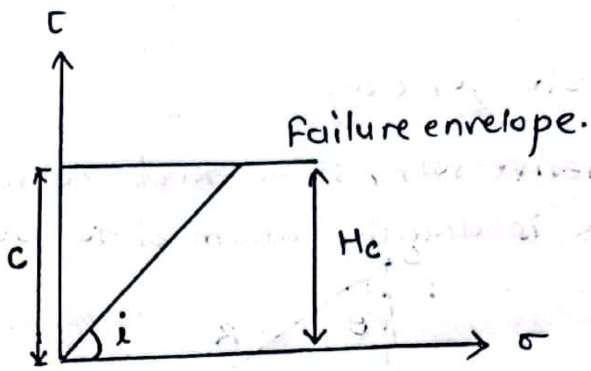
$$FOS = \frac{\tan\phi}{\frac{\sin i}{\cos i}}$$

$$\therefore FOS = \frac{\tan\phi}{\tan i}$$

2. Pure cohesive soil: clay

$\phi = 0°$

$$FoS = \frac{c}{\gamma H \cos i \cdot \sin i}$$

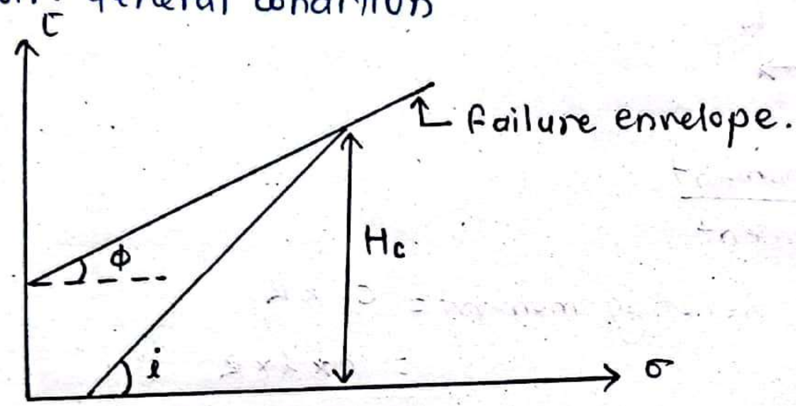


$H < H_c$: stable.

$H = H_c$: critical.

$H > H_c$: unstable.

3. Mixed soil: General condition



$H < H_c$: stable

$H = H_c$: critical

$H > H_c$: unstable.

*: Stability Analysis of finite slope:

↳ Graphical or semi-graphical methods are used for analysis of finite slope.

Analysis procedure:

↳ Assume failure circle.

↳ calculate FoS for each failure circle.

↳ Circle having least FoS is critical circle.

Methods:

I. $\phi = 0$ method

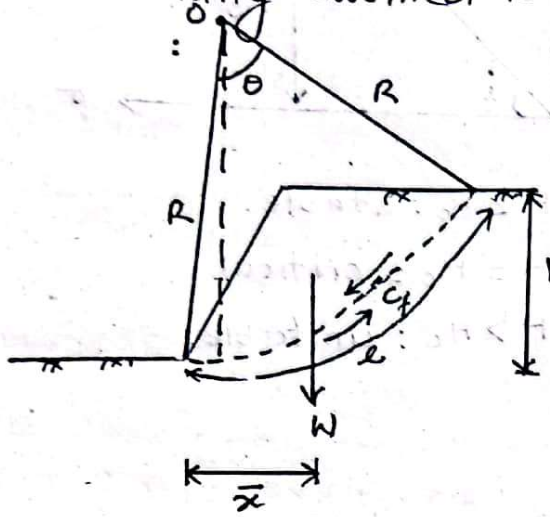
II. c-ϕ method

III. Taylor's stability number method

(i) $\phi = 0$ method:

↳ Applicable for clay

↳ In cohesive soil, slope tends to rotate along a curve. The failure surface is usually assumed to be an arc of a circle.



$$FOS = \frac{\text{Resisting moment}}{\text{Driving moment}}$$

; Resisting moment $= c_f \times R$

$= c \times l \times R$

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

$= cR^2\theta$; θ is in radian

Driving moment $= W \times \bar{x}$

$$\therefore FOS = \frac{cR^2\theta}{W \times \bar{x}}$$

(ii) c-φ method: method of slices

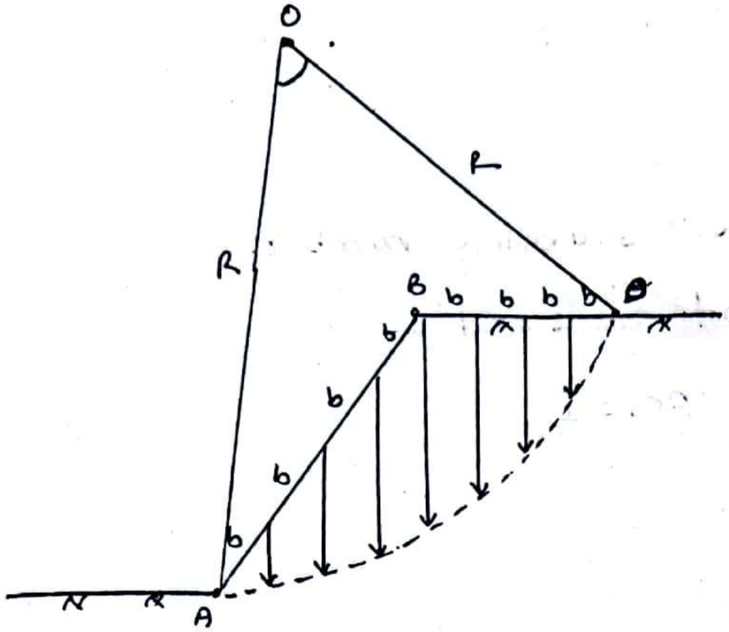
Assumptions:

① The slip circle is cylindrical i.e. arc of circle in section.

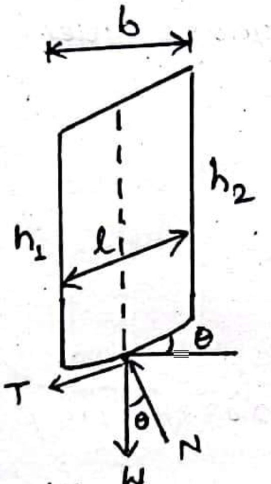
② The sliding soil mass is assumed to consist of no. of vertical slices.

③ The force of interaction between adjacent slices are neglected.

→ Applicable: on mixed soil.



↳ Divide the assumed failure wedge into number of equal slices.
↳ For each slices: measure on graph the following parameter.



$h_1 = ?$

$h_2 = ?$

$\theta = ?$

θ = slice और centre का H_2. संग बनाएको angle.

↳ Calculate the driving moment and resisting moment for each slice.

Resisting moment $= c \times l + N \tan\phi$

Driving moment $= T$; where $N = W\cos\theta$

Now, for a number of slices, $T = W\sin\theta$

$l = b \times \sec\theta$

$W = A \times \rho$

$= \left(\frac{h_1 + h_2}{2}\right) \times b \times \rho$

↳ $FOS = \dfrac{\text{Resisting moment}}{\text{Driving moment}}$

$$= \frac{\Sigma c \times l + \Sigma N \tan\phi}{\Sigma T}$$

(iii) Taylor's stability number: method:

$$S_n = \frac{c}{\gamma H \times FOS}$$

; S_n = taylor's stability number.

S_n depends on i and ϕ.

for critical height (H_c), FOS = 1

$$\therefore H_c = \frac{c}{\gamma \times S_n}$$

Q. Illustrate the types of slope failure with suitable sketches: [5 marks] (PSC-2078/12/19)

Q. A vertical cut is made in a clay deposit, find the critical height. (2ıt language question मा ।शिएन्त).
consider,

$c = 30\ kN/m^2$

$\phi = 0°$

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

FOS = 1 and

$S_n = 0.261$ [5 marks] (PSC-2078/12/19)

solution:

Since FOS is given, Here critical height is to be found out.

$$S_n = \frac{c}{\gamma H \times FOS}$$

$$\text{for } H_c = \frac{c}{\gamma \times S_n \times FOS} = \frac{39}{0.261 \times 16 \times 1} = 7.18\,m$$ Rk