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CLASS: 078 BCE SEC: CH ROLL NO: 078 BCE 178

SUBJECT: Soil Mechanics

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[illegible]

Never Stop Learning.....

Soil is a naturally occurring loose or soft deposit which is also a part of the earth's crust and produced as a result of weathering, disintegration, decomposition of rocks or decay of vegetation.
(3D)

Soil Mechanics

↳ It can be defined as a application of laws of mechanics and hydraulics in order to solve the engineering problems of with certain modification (if required) related to soil.

Soil → Deposit → voids → air → 2 phase

↳ air + water → 3 phase
↳ water → 2 phase

Formation of soil:

Soil

Decay: vegetation

Decomposition:

Disintegration: } Rock (eng. use)

[Agriculture → Humus
↳ fertile
↳ compressible
↳ weak
↳ not for engineering]

Formation of soil:

1. Biological factors (Decay) → Urination by animals, Burrowing animals

2. Decomposition (chemical composition change)

↳ Chemical factors → oxidation; hydration;
carbonation; hydrolysis;
Leaching

3. Physical factors (Disintegration)

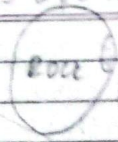
(physical parameters change, shape, size, smoothness, etc.)

↳ Abrasion

↳ Thermal change

↳ Ice Formation

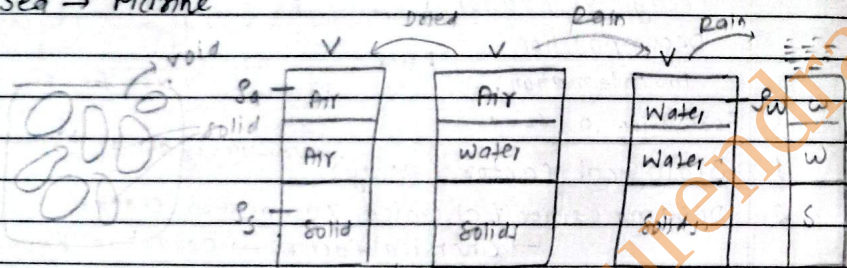
Soil breaks from contact plane
 Voids are interconnected & also connected to atmosphere



soil deposited away from place of origin
 Transported

Transporting factors:

1. Water → Alluvial soil
2. Gravity → colluvial soil
 ↳ landslides
3. Air → Aeolian
4. Glaciers → Glacial
5. Lake → Lacustrine
6. Sea → Marine



2-Phase (dry soil) (dry density) ρ_d
 3-Phase diagram (wet/moist soil) (wet/moist bulk density) ρ
 2-Phase (saturated soil) (saturated density) ρ_{sat}

* water-table → fluctuates

↳ soil water content level

ρ → soil (wet/moist)

$\rho_{sub} = \rho_{sat} - \rho_w$

Unless & until force is applied the volume of soil remains constant for soil which has formed by disintegration of rock (engineering material)
 (Not for agricultural soil)

$$\Rightarrow \rho_{sat} > \rho > \rho_d > \rho_{submerged}$$

ρ_d → dry density

ρ_{sat} → sat. density

ρ_{sub} → weight (ρ smallest)

In soil, density can be increased by adding water (mass ↑)
 ↳ or removing air from soil (volume ↓) → more stronger

Terminologies

	Mass	Volume
Air	M_a	V_a
Water	M_w	$V_w = e V$
Solids	M_s	V_s
Total	M	V

$$(i) \text{ Void ratio } (e) = \frac{V_v}{V_s} \quad (\text{fraction or \%})$$

$$\Rightarrow V = V_v + V_s$$

$$V = V_a + V_w + V_s$$

→ for formula derivation, use (assume) $V_s = 1$

$$\text{So, } e = \frac{V_v}{1}$$

$$\rho = \frac{M}{V} = \frac{M_w + M_s}{V} \quad [M_s = V_s \cdot \rho_s = \rho_s]$$

$$(ii) \text{ Porosity } (\eta) = \frac{V_v}{V}$$

$$(iii) \text{ Degree of saturation } (S_x \text{ 'or' } S) = \frac{V_w}{V_v}$$

$$(iv) \text{ Sp. Gravity of solids; } G = \frac{\rho_s}{\rho_w} \quad (\rho_s = G \rho_w)$$

$$(v) \text{ Sp. Gravity of soil, } G_t = \frac{\rho}{\rho_w} \rightarrow \rho_d / \rho / \rho_{sat} \text{ (acc. to condition)}$$

$$(vi) \text{ Water content, } w = \frac{M_w}{M_s}$$

$$[M_w = w M_s]$$

Thus,

$$\rho = \frac{M}{V} = \frac{M_w + M_s}{V} = \frac{w G \rho_w + G \rho_w}{V}$$

$$\left[\rho = \frac{G(1+w) \rho_w}{V} \right] \quad \leftarrow$$

$$\rho = \frac{M}{V} = \frac{M_w + M_s}{V} = \frac{w M_s + M_s}{V}$$

$$\left[\rho = \frac{M_s(1+w)}{V} \right] \quad \leftarrow$$

$$\left[\rho_d = \frac{M}{V} = \frac{M_s}{V} \right] \quad \leftarrow$$

$$\left[\rho = \rho_d(1+w) \right] \quad \leftarrow \left[\gamma = \gamma_d(1+w) \right]$$

Mass of soil equal to Mass of solid $\Rightarrow$ only for dry soil

$$\# \quad e S_x = w G$$

$$\frac{V_v}{V_s} \cdot \frac{V_w}{V_v} = \frac{M_w}{M_s} \cdot \frac{\rho_s}{\rho_w} = \frac{V_w}{V_s}$$

$$\left[\rho = \frac{G(1+w) \rho_w}{1+e} = \frac{(G + e S_x) \rho_w}{1+e} \right] \quad \leftarrow$$

S_x = Degree of saturation

• When, $S_x = 0$, soil becomes dry.

Then

$$\left[\rho_d = \frac{G \rho_w}{1+e} \right] \quad \leftarrow$$

• When, $S_x = 1$, soil becomes saturated

$$\left[\rho_{sat} = \frac{(G + e) \rho_w}{1+e} \right] \quad \leftarrow$$

$$\rho_{sub} = \rho_{sat} - \rho_w = \frac{(G + e - 1) \rho_w}{1+e}$$

$$= \frac{(G + e) \rho_w}{1+e} - \rho_w$$

$$= \frac{(G + e - 1 - e) \rho_w}{1+e}$$

$$\left[\rho_{sub} = \frac{(G - 1) \rho_w}{1+e} \right] \quad \leftarrow \left[\gamma_{sub} = \frac{(G - 1) \rho_w}{1+e} \right] \quad \leftarrow$$

$$\# \quad \left[\rho = \frac{G(1+w) \rho_w}{V} \right] \quad \left[\rho = \frac{(G + e S_x) \rho_w}{1+e} \right] \quad \left[\rho = \rho_d(1+w) \right]$$

$$\left[\rho_d = \frac{G \rho_w}{1+e} \right] \quad \left[\rho_{sat} = \frac{G + e}{1+e} \rho_w \right] \quad \left[\rho_{sub} = \frac{G - 1}{1+e} \rho_w \right]$$

$$\# \text{ void ratio } (e) = \frac{V_v}{V_s}$$

$$\text{porosity } (n) = \frac{V_v}{V}$$

$$\text{Deg. of saturation } (S_x) = \frac{V_w}{V_v}$$

$$\# \text{ Water void ratio } (e_w) = \frac{V_w}{V_s}$$

$$\% \text{ air void} = \frac{V_a}{V} = n_a$$

$$\text{Air content } (q_c) = \frac{V_a}{V_v}$$

$$\# e \cdot S_x = wG$$

$$\# \left\{ \begin{aligned} S_d &= \frac{G(1-n_a)S_w}{1+wG} \end{aligned} \right.$$

we have

$$\frac{V}{V} = \frac{V_a}{V} + \frac{V_w}{V} + \frac{V_s}{V}$$

water content for saturation of given soil is always constant called saturated water content (w_{sat})

$$\# e \cdot S_x = wG$$

For saturation, $S_d = 1$

$$\Rightarrow [e = w_{sat} G]$$

$$\# q_c = \frac{V_a}{V_v} = \frac{V_v - V_w}{V_v} = 1 - S_x$$

$$\# n_a = \frac{V_a}{V} = \frac{V_v - V_w}{V} = n - \frac{V_w}{V} \times \frac{V}{V_v}$$

$$n_a = n - S_x n$$

$$[n_a = n(1 - S_x)]$$

$$[n_a = n \cdot q_c]$$

$$\# \rho = \frac{G + e S_x}{1 + e} \rho_w$$

$$\rho = \frac{G \rho_w}{1 + e} + \frac{e}{1 + e} S_x \rho_w$$

$$[\rho = \rho_d + n S_x \rho_w]$$

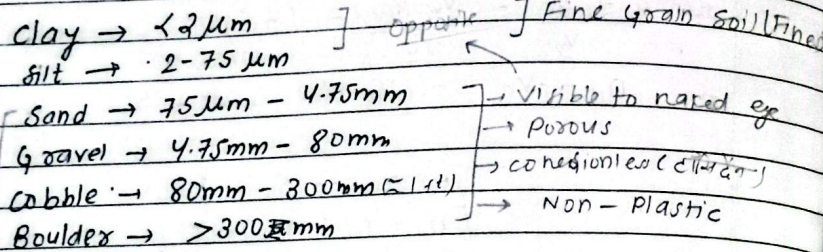
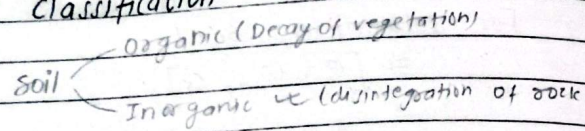
$$\# \rho_{sat} = \frac{G + e}{1 + e} \rho_w = \frac{G}{1 + e} \rho_w + \frac{e}{1 + e} \rho_w$$

$$[\rho_{sat} = \rho_d + n \rho_w]$$

↓

$$[\rho = \rho_d + (\rho_{sat} - \rho_d) S_x]$$

Broad Classification



- Loam → Mixture of clay, silt & sand (fertile soil)
↳ Agricultural engineer
- Caliche → Mixture of silt, sand & gravel
- Peat → Organic soil
- Calcareous soil → having high % of Calcium Carbonate
- Marl → Calcareous soil which is green in colour
- Hardpan → offers high resistance to drilling (like rock)
↳ well graded soil
↳ Compacted
- Varved clay → less found in nature
↳ Sedimentary deposit of alternate layers of silt & clay.
- Black cotton soil → high % of clay

Clay → formed by decomposition (chem decomposition)

Illite

Kaolinite

Montmorillonite

Black cotton soil

↳ highly swelling & shrinkage

↳ weak

↳ compressible

↳ low bearing capacity

↳ not suitable for construction

finest clay

Vol can increase up to 20% of its wt on exposure to water.

Bentonite → G.I. (Slurry) that comes out during boring
↳ expensive than cement (sold in bags)

↳ Properties same as Black cotton soil

formed by decomposition of rock

formed by decomposition of volcanic rock (or volcanic ash)

$$w = \frac{M_w}{M_s} = \frac{M_{\text{moist}}}{M_{\text{dry}}}$$

Determination of water content (w)

$$w = \frac{M_w}{M_s} \left(\text{can be } \frac{M_w}{M_s} \text{ or } \frac{M_w}{M_s} \right)$$

1. Lab Method

↳ Oven-drying Method

↳ Pycnometer Method

Accurate water content

Time taking

Costly

dry mass

2. Field Method

↳ a. Alcohol Method

↳ b. sand Bath Method.

rough

Temp: 60°C (for dry soil)
Temp: 80°C (for soil having high gypsum)

1. (a) Lab Oven-Drying Method (24 hr)

↳ most accurate

carry 2% of water to avoid loss of water of soil

dry container

M_1

Temp: 105°C to 110°C (for fine soil)

Time → 4 hrs (sandy soil)

12-16 hrs (clay)

Wet soil

sample (i.e. the result given by sample must represent results in field)

constant

(110°C ± 5°C)

M_2

kept in oven (Temp = 110°C)

↳ may burn hands

↳ kept for 24 hours

dry soil

M_3

Thus,

$$M_w = M_2 - M_3$$

$$M_s = M_3 - M_1$$

$$w = \frac{M_w}{M_s} = \frac{M_2 - M_3}{M_3 - M_1}$$

→ If organic soil is mixed in given soil then
 ↳ Temp should be maintained at $60^\circ - 80^\circ\text{C}$ in oven else the organic matter may burnt out

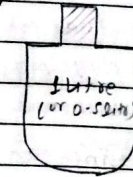
→ Sample taken must be representative
 (represent whole mass of soil in field)

More than 90% passing through	Quantity
→ 425 μm sieve	25 gm
→ 2 mm sieve	50 gm
→ 4.75 mm sieve	200 gm
→ 10 mm sieve	300 gm
→ 20 mm sieve	500 gm
→ 40 mm sieve	1000 gm

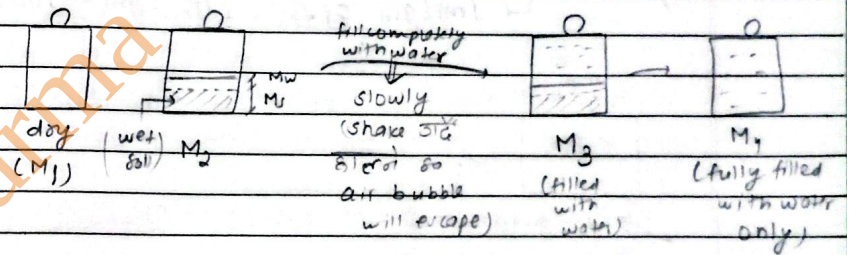
9. (b) Pycnometer Method → Quick results (10-20 min)

(G)

↳ w can be found out only when G of soil is already known.
 (For finding G next experiment may be done)



For 1 litre container → 200-400 gm of sample



$$M_3 - M_s + \left(\frac{M_s}{G}\right) p_w = M_4 \quad \left[\frac{p_s}{p_w} = G\right]$$

$$\text{or, } M_s = \frac{M_3 - M_4}{1 - \frac{p_w}{G}} = \frac{M_3 - M_4}{\left(\frac{G-1}{G}\right)}$$

$$\left[M_s = \frac{M_3 - M_4}{\left(\frac{G-1}{G}\right)} \right] \sim$$

$$w = \frac{M_w}{M_s} = \frac{M_2 - M_1 + M_s}{M_s} = \frac{M_2 - M_1}{M_s} + 1$$

$$\left[w = \frac{M_2 - M_1}{M_3 - M_4} \left(\frac{G-1}{G} \right) + 1 \right]$$

→ If air bubble is present → result may vary
 → If water is present in container or around it then result may vary
 → Soil must not come out while shaking

Field Method

2(a) Alcohol Method - 10 minutes

↳ Quick method

↳ not accurate as oven-drying method

↳ not used for soil that contains a large proportion of clay, org. matter, gypsum, or any calcareous material.

Wet soil M_2

✓ Methylated spirit → increase rate of evaporation during drying

✓ Alcohol → Methylated spirit

↳ 1ml/gm of soil (For 1gm → 1ml)

Ignite

Dry soil M_3

→ (may not be burnt evenly) → not accurate

$$w = \frac{M_w}{M_s} = \frac{M_2 - M_3}{M_3 - M_1}$$

✓ NO control over temp

2(b) Sand Bath Method (20-60 min)

↳ used when facility of oven is not available

↳ not for org soil & soil which contain high % of gypsum

↳ NO control over temperature

M_1

Wet soil M_2

piece of white paper (becomes wet in wet soil)

Sand 3mm

flame

when white paper becomes brown the soil is (abt. said to be dried to burn) (M_3)

$$w = \frac{M_w}{M_s} = \frac{M_2 - M_3}{M_3 - M_1}$$

Determination of Specific Gravity: G_s

Specific gravity of solids → G_s (no void)

Should be dried first → soil broken finely

Sp. gravity of soil → G_s (void present)

↳ not broken

↳ no drying

1. Pycnometer method

2. Conical flask Method

3. Gas jar Method

4. Density bottle Method

proves some (container diff.)

50ml

container soil

Pycnometer → 1ltr. → 200-300gm

conical flask → 1ltr. → 80-100gm

Gas jar → 1ltr. → 200-400gm

Density bottle → 50ml → 5-10gm.

Four readings

M_1

M_2
(dry container)

M_3
(filled with water)

M_4
(filled with water only)

$$M_3 - M_2 + \frac{M_s}{\rho_w} = M_4$$

$$\text{on } M_3 - M_2 + \frac{M_s}{\rho} = M_4$$

$$\text{on } G = \frac{M_3 - M_2}{(M_4 + M_2) - (M_1 + M_3)}$$

$$\left[G = \frac{M_2 - M_1}{(M_4 + M_2) - (M_1 + M_3)} \right]$$

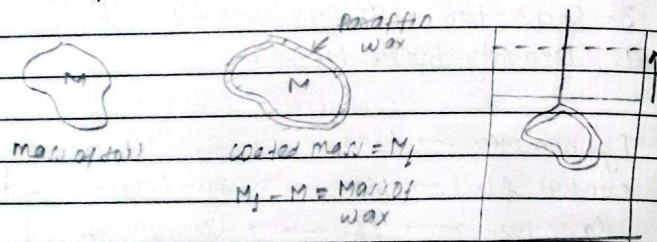
$$\left[G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)} \right]$$

Determination of field density $\therefore \rho = \frac{M}{V}$

↳ By maintaining the volume (most difficulty)

1. Water displacement Method
2. Sand Replacement Method
3. Core Cutter Method

1. Water displacement Method



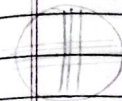
$$V_{\text{wax}} = \frac{M_1 - M}{\rho_{\text{wax}}}$$

$$V_1 = \text{Vol}^{\text{m}} \text{ of water displaced} \\ = \text{Vol. of wax} + \text{Vol. of soil} \\ \text{Vol}^{\text{m}} \text{ of soil} = V_1 - \frac{(M_1 - M)}{\rho_{\text{wax}}}$$

$$\text{Density} = \frac{M}{\text{Vol}^{\text{m}} \text{ of soil}}$$

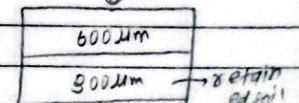
2. Sand Replacement Method. → with help of standard sand.

If not available (Std. sand) → sieve Dried sand (oven)

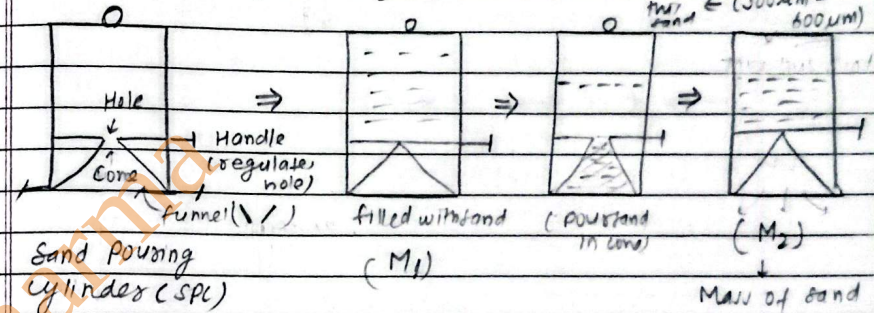


D_s → 600 μm

d_s → 300 μm

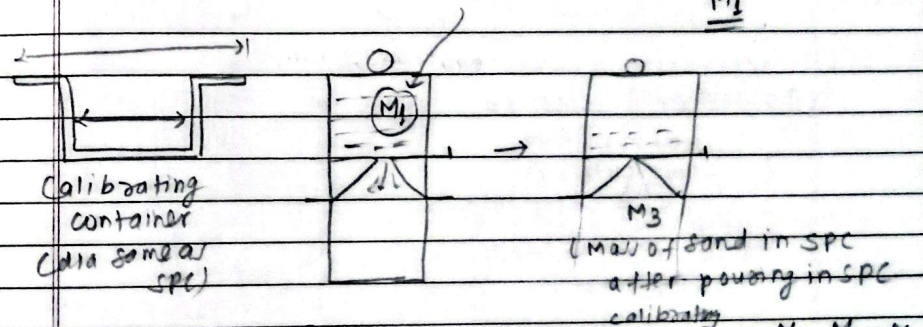


A. Determination of density of sand.



$$\left[\Delta \text{ Mass of sand in cone } (M_2) = M_1 - M_2 \right]$$

fill back the released mass back to SPC s.t mass become $\frac{M_1}{2}$



$$\text{Mass of sand in container} = M_1 - M_3 - M_c$$

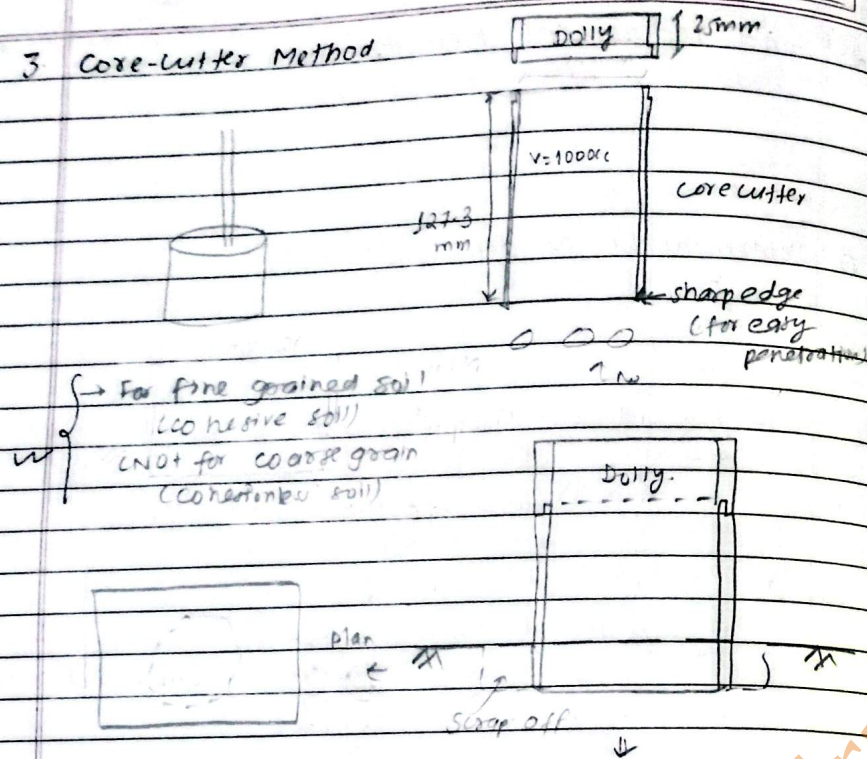
$$\text{Vol}^{\text{m}} \text{ of calibrating container} = V$$

[Dia → vernier calliper & Ht.]

$$\text{Density of sand, } \rho_{\text{sand}} = \frac{M_1 - M_3 - M_c}{V}$$

[Elaborate] Mass container 100g, after divide 10

3 Core-cutter Method



* Dolly → only used to ensure that core cutter is filled completely for uniform load distribution.

Mass of core cutter = M_1

Mass of core cutter with soil = M_2

Mass of soil $M = M_2 - M_1$

2. Sand Replacement Method

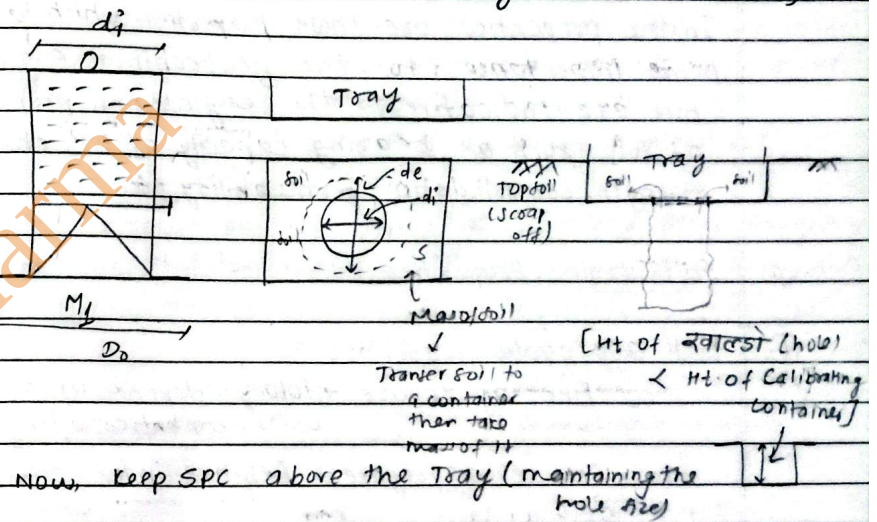
B. Determination of density of soil (in field).

↳ Spectro

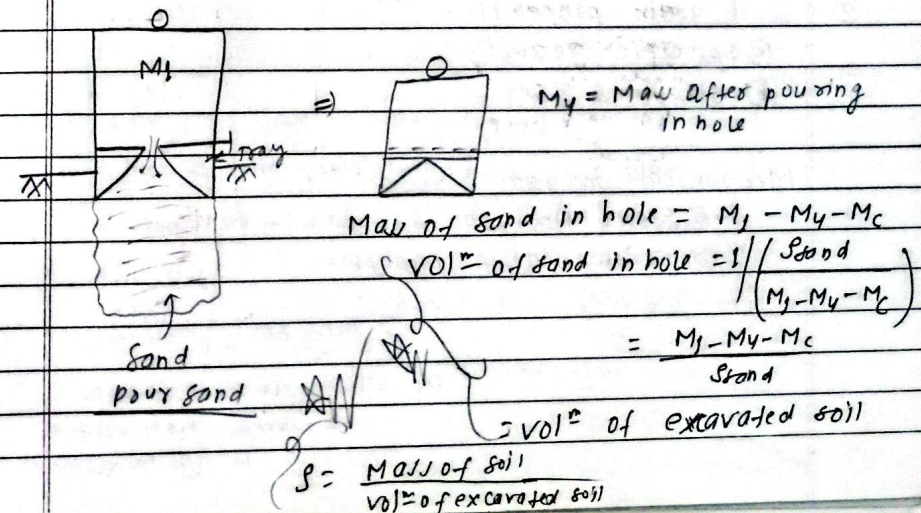
↳ specially done for coarse grain soil

where core-cutter Method is not feasible

[But, can do for fine grain soil as well but takes more time & money than core-cutter]



Now, keep SPC above the Tray (maintaining the hole size)



$M_4 = M_{av}$ after pouring in hole

Mass of sand in hole = $M_1 - M_4 - M_c$

Volⁿ of sand in hole = $\frac{M_1 - M_4 - M_c}{\rho_{sand}}$

= $\frac{M_1 - M_4 - M_c}{\rho_{sand}}$

= Volⁿ of excavated soil

$\rho = \frac{\text{Mass of soil}}{\text{Volⁿ of excavated soil}}$

Properties of Engineering properties (used directly in structure design) for major projects

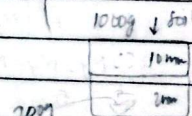
- Index properties of soil
 - ↳ not used for design of structure
 - ↳ takes less time, expense for test
 - ↳ gives range of eng prop but not accurate
 - ↳ for minor projects (taking min^m value of eng prop)

Index properties are those properties which are not prime importance to the geotechnical engineer but are indicative to the engineering properties of soil such as bearing capacity of soil, shear strength, consolidation, permeability, etc.

- (a) Soil grain Properties: Specific gravity, Shape etc.
- (b) Soil aggregate properties
 - Fine aggregate → totally depend upon water content, consistency limit
 - Coarse aggregate → Relative index

- (c) Soil grain properties
 - (a) specific gravity
 - (b) shape & size

- Mechanical analysis → naming/classification of soil
 - ↳ (a) Sieve Analysis → coarse soil → sand/gravel
 - ↳ (b) sedimentation Analysis → < 75µm



Fine soil → silt/clay → < 75µm
 For silt/clay in sieve analysis
 ↳ mass loss happens and it attaches to sieve

Mechanical analysis → also called particle size analysis, is method of separation of soil into diff fraction based on particle size

Sieve analysis → Mass loss < 2%
 ↳ done for coarse soil
 ↳ can also be done if fine soil are mixed provided Fines < 5% and mass loss < 2%, if analysis is done properly

Sedimentation analysis → (Fines > 5% in coarse soil), then we cannot maintain mass loss < 2% in sieve analysis
 ↳ for silt/clay - fine grained soil.

Mixed soil
 For coarse soil if Fines < 5% → Dry sieve Analysis
 For coarse soil if Fines > 5% → wet sieve Analysis
 (difficult to use, avoidable, fines which comes from here are done analysis by sedimentation analysis)

#	Dry sieve Analysis	Gravel fraction	cover	top
conco				
75mm	80mm		80	
37.5mm	40mm		40	
	20		20	
	10		10	
	4.75mm	Gravel fraction	4.75	
	2	retain: 4.75mm		
	1mm	↓ passing		
	750µm	sand fraction		
	600µm			
	425µm			
	300µm			
	212µm			
	150µm			
	75µm			

Retained mass are weighed in a pan.
 (mass loss < 2%)

750 μ	
600	
425	
300	
212	
150	
75	
Vibroater (10 min)	

Done in 3 phases

→ Gravel fraction

→ sand fraction

→ separate (S₂ & S₃)

→ At lab done at once

→ sometimes done size determination by eye judgement

Table for taking sample	
Max size	Quantity
80mm	→ 65kg
20mm	→ 6.5kg
4.75mm	→ 0.5kg

Mass of sample = M

S.N.	Sieve size	Mass retained	% Mass retained	Cumulative % retained	% Fineness
1	40	M ₁	$P_1 = (M_1/M) \times 100$	$C_1 = P_1$	$N_1 = 100 - C_1$
2	20	M ₂	$P_2 = (M_2/M) \times 100$	$C_2 = P_1 + P_2$	$N_2 = 100 - C_2$
3	4.75	M ₃	P_3	$C_3 = P_1 + P_2 + P_3$	$N_3 = 100 - C_3$
4	600 μ	M ₄	P_4	C_4	$N_4 = 100 - C_4$
5	300 μ	M ₅	P_5	C_5	$N_5 = 100 - C_5$
6	75 μ	M ₆	P_6	$C_6 = \sum_{i=1}^6 P_i$	$N_6 = 100 - C_6$

Wet sieve Analysis.

Wet soil

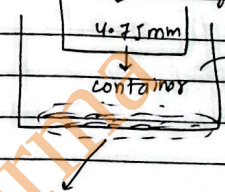
↓
dry soil

↓
Take sample

↓
Water mix

↓
slurry

↓
washing

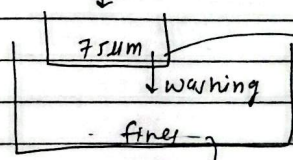


Take this soil passed thr. 4.75mm Sieve

Take retained soil as Gravel fraction

↓
Dry (in oven for 24 hrs)

↓
Dry sieve Analysis (retained)



↓
sedimentation.

S.N.	Sieve size	Mass retained	% mass retained	cumulative % retained	% finer
1	40	150	15%	15%	85%
2	20	200	20%	35%	65%
3	4.75	300	30%	65%	35%
4	600	50	5%	70%	30%
5	300	75	7.5%	77.5%	22.5%
6	75	225	22.5%	100%	0%
		1000g	100%	100%	

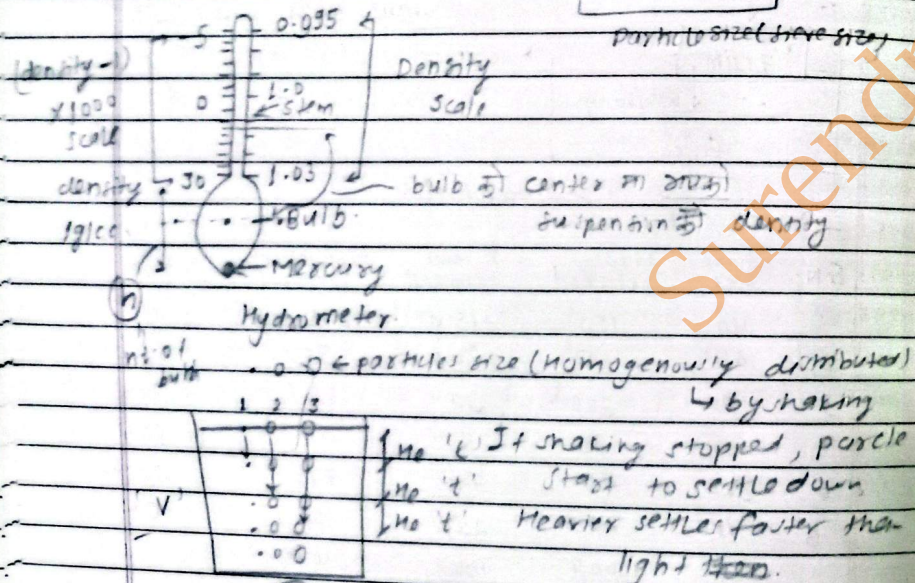
coefficient of uniformity (C_u) = D_{60} / D_{10}
 coefficient of curvature (C_c) = $\frac{D_{30}^2}{D_{10} \times D_{60}}$

Use of particle distribution curve:

- Is useful in estimating permeability of soil
- helpful in estimating the suitability of drainage for highways
- In estimating shear strength of soil.
- In estimating consolidation property of soil.
- In finding the mode of deposition of soil.

Sedimentation Analysis

1. Hydrometer Analysis:-



M_s = mass of solid
 M_s - mass shaken in V vol =

Hydrometer measures sp. gravity of suspension at a point indicated by center of immersed volume.

water $\rightarrow$ mass of solid
 $V_{ml} \rightarrow M_s$
 $1ml \rightarrow \left(\frac{M_s}{V}\right) = m_s$

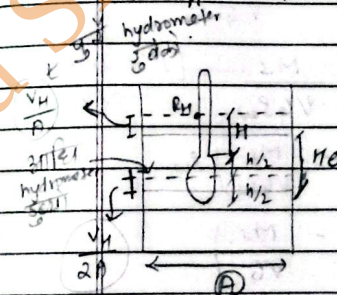
After some time (settling down) 't',

$V_{ml} \rightarrow M_p$ still in water.
 $1 \rightarrow \frac{M_p}{V} = m_p$ due to small size couldn't settle down completely
 fine particle

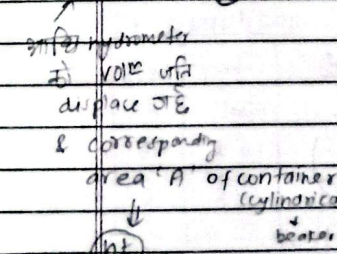
% fines = $N = \frac{m_d}{m_s} \times 100$ But we don't know size of particle limit below which particle couldn't settle!

$V_h \rightarrow$ Vol^m of hydrometer.

R_h = hydrometer reading
 H = water level above neck



$H_e = H + h/2 + \frac{V_h}{2A} - \frac{V_h}{2A}$
 $H_e = H + h/2 - \frac{V_h}{2A}$

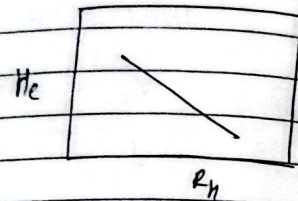


$H_e = H + h/2 - \frac{V_h}{2A}$
 H_e is the center of mass of the hydrometer depth.

Solid line $\rightarrow$ initial imaginary line before immersing hydrometer
 dotted line $\rightarrow$ final " " after " "

R_H	H	H_c
-5	35	
-1	34	
0	30	
30		

Calibration chart/curve
(given for given hydrometer)



Initial density (ρ_i) (before homogeneity);

$$\rho_i = \frac{M}{V} = \frac{\text{Mass of water} + M_s}{V}$$

$$\rightarrow \text{Mass of solids} = \frac{M_s}{V} \quad \left(\begin{array}{l} \text{Mass of solid per} \\ \text{unit vol. of} \\ \text{mixture} \end{array} \right)$$

$$\rightarrow \text{Vol}^e \text{ of solids} = \frac{M_s}{V \cdot \rho_s} = \frac{M_s}{V \cdot G \cdot \rho_w}$$

$$\rightarrow \text{Vol}^e \text{ of water} = 1 - \frac{M_s}{V G \rho_w} \quad [1 \rightarrow \text{unit vol}^e \text{ of mixture}]$$

$$\rightarrow \text{Mass of water} = \left(1 - \frac{M_s}{V G \rho_w} \right) \rho_w$$

$$= \left(\rho_w - \frac{M_s}{V G} \right)$$

Thus,

$$\rho_i = \rho_w - \frac{M_s}{G V} + \frac{M_s}{V}$$

$$= \rho_w + \frac{M_s}{V} \left(1 - \frac{1}{G} \right)$$

$$\rho_i = \rho_w + \frac{M_s}{V} \left(\frac{G-1}{G} \right)$$

After time 't' (some solid settle down)

$$\rho = \rho_w + \frac{M_d}{V} \left(\frac{G-1}{G} \right)$$

$$1000 \cdot (\rho - \rho_w) = \frac{m_d}{V} \left(\frac{G-1}{G} \right) \times 1000$$

$$R_H = \frac{M_s}{100} \left(\frac{G-1}{G} \right) \times 1000$$

$$\left[R_H = \left(\frac{N}{100} \right) \left(\frac{M_s}{V} \right) \left(\frac{G-1}{G} \right) \times 1000 \right] \checkmark$$

If, $V = 1000$

$$R_H = \left(\frac{N}{100} \right) \frac{M_s}{V} \left(\frac{G-1}{G} \right)$$

$$N = \frac{100 R_H}{M_s} \left(\frac{G}{G-1} \right)$$

$$\left[N = \frac{R_H}{M_s} \frac{G}{G-1} \times 100 \right] \checkmark$$

formula.

In practical:

SN	t	R_H	H_c	Particle size (D)	N
1	1 min				
2	2 "				
3	3 "				
4					
5					

Stokes law, $V = \frac{1}{18} \frac{g (G-1) \rho_w}{\eta} \times D^2$

$\left(\frac{R_H}{t} \right)$

$[D = \mu]$

$$\frac{\text{settling velocity}}{60 \times t} = \frac{V_s}{18} = \frac{H_c}{18} = \frac{1}{18} \frac{g(g-1) \rho_w D^2}{\eta}$$

$$D = \sqrt{\frac{0.3 \eta}{g(g-1) \rho_w} \times \frac{H_c}{t}}$$

$t \rightarrow$ in minute

[each particle settles differently, else soil may cohere to form larger size & settle faster]

$\downarrow$
So, Dispersing agent is added

$\downarrow$
So particles do not stick (चिपकने) with each other

$\rightarrow$ For pure water (27°C), hydrometer reading (R_H) should be zero
[$\rho = 1$] $(1-1) \times 1000 = 0$

Corrections

1. Meniscus correction (C_m)
2. Temperature correction (C_t) $\begin{cases} > 27^\circ\text{C} \\ < 27^\circ\text{C} \end{cases}$
3. Dispersing agent correction (C_d) $\rightarrow (-ve)$

• Temperature correction (C_t)

$\begin{cases} C_t \rightarrow > 27^\circ\text{C} \rightarrow +ve \\ C_t < 27^\circ\text{C} \rightarrow -ve \end{cases}$

• Meniscus correction (C_m) $\rightarrow +ve$

$\begin{cases} +0.005 \\ -0.1 \end{cases}$

* Consistency Index

Plasticity of soil

$\begin{cases} \text{non-plastic} \rightarrow \text{relative density index} \\ \text{coarse grain} \rightarrow \text{Gravel, Sand} \rightarrow \text{cohesionless} \\ \text{fine-grained soil} \rightarrow \text{cohesive} \\ \text{fine-grained soil} \rightarrow \text{molded/plastic/can deform} \end{cases}$

fine-grained soil changes its property according to water content

Acc. to water content $\Rightarrow$ shrinking & swelling.

$\begin{matrix} \text{clay} \\ \text{negatively charged} \\ \text{water} \\ \text{Bipolar} \end{matrix} \Rightarrow \begin{matrix} \text{clay} \\ \text{water} \\ \text{Bipolar} \end{matrix} \rightarrow \begin{matrix} \text{clay} \\ \text{water} \\ \text{Bipolar} \end{matrix}$

$\rightarrow$ water act as lubricating agent, & clay can be molded into various shape

So, clay can hold water & molded into shape (plasticity)
 $\rightarrow$ clay & water.

Plasticity: It is an ability of soil to undergo deformation without cracking or deformation

For plasticity essential components,

$\rightarrow$ clay & $\rightarrow$ water.

Soil $\Rightarrow$ clay

कितनी पानी?

vol. of soil $\uparrow$ or $\downarrow$ $\Rightarrow$...
 $\rightarrow$ Soil can't take tensile force
 $\rightarrow$ cracks within negligible force

But can take compressive force
 (no voids left)

consistency limits

water content $\Rightarrow$ from '0' upto saturation limit

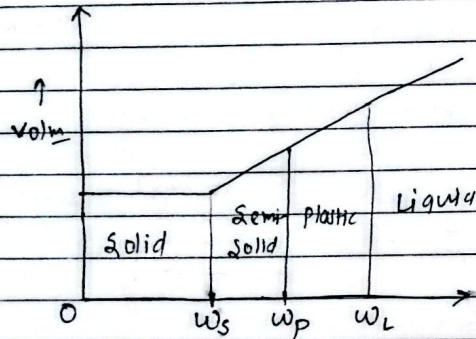
$\rightarrow$ Above satⁿ, soil can hold water if energy is applied (broken)
 w/o changing vol^m

$\Rightarrow$ Consistency limits are those water contents at which soil changes from one state to another

Also called Atterberg's limit

Three

1. Liquid limit (w_L)] Lab.
2. Plastic limit (w_P)
3. Shrinkage limit (w_S)



water content (w) $\rightarrow$

1. Liquid limit (w_L) $\rightarrow$ At soil which soil changes from plastic liquid state to liquid state

$\rightarrow$ It is the minimum water content at which soil begins to flow.

2. Plastic limit (w_P) $\Rightarrow$ At which soil changes from semi-solid state to plastic state.

(water content at which soil thread of 3mm dia can be moulded with crack)

$\rightarrow$ Plastic limit.

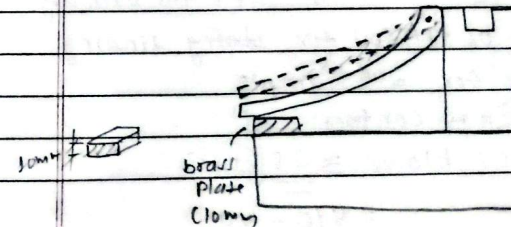
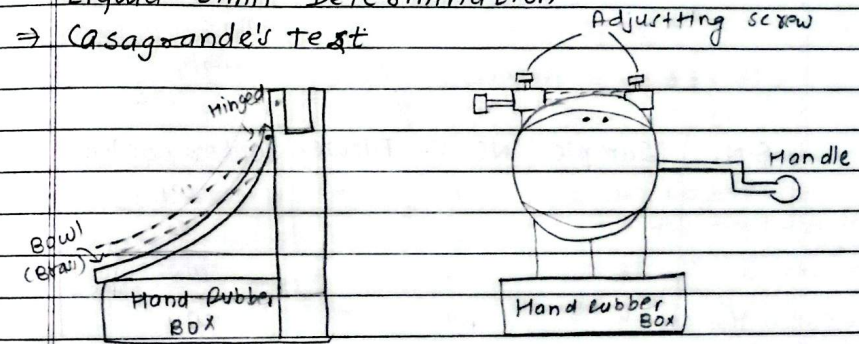
$\Rightarrow$ It is a minimum water content at which soil rolled into thread of 3mm diameter just crumbles (cracks).

3. Shrinkage limit (w_S): At which soil changes from solid state to semi-solid state. (saturation pt.)

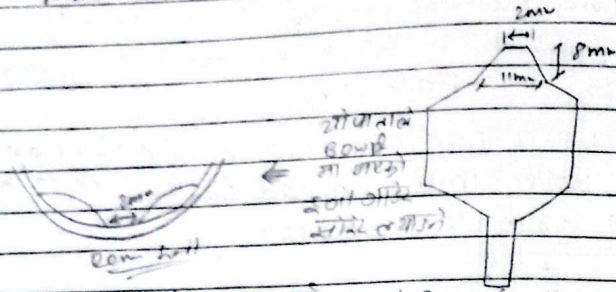
$\Rightarrow$ It is a maximum water content at which further reduction in water content will not cause decrease in volume of soil.

Liquid Limit Determination

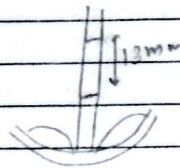
$\Rightarrow$ Casagrande's test



- Sample → Soil through 425μ sieve
Add distilled water to sample



→ handle the bowl with steel for each revolution
Bowl comes down 13mm to have rubber box
& gives solid sound for each revolution
⇒ speed of revolution: 2 rev/sec.



bowl vibrator.
13mm soil comes down reducing gap of 2mm until both halves comes in contact

⇒ till comes in contact in length wise 13mm.

{ NO. of Blows ⇒ for contact of 13mm length }

S.N.	Sample	NO. Of Blows.	water content
1	1	N_1	w_1
2	2	N_2	w_2
3	3	N_3	w_3
4	4	N_4	w_4

mixed soil → (some amount) → left for oven dry
Then, take some of the soil for doing finding
NO. of blows for 2nd sample
Conditions → 13mm contact

↳ NO. of blows $\approx 25 (\pm 15)$
 $= 10 - 40$

flow index → Slope of flow curve obtained from graph of blows and the water content in casagrande's method of determination of liquid limit

Acc. to Casagrande's, Liquid Limit is water content corresponding to 25 no. of blows.

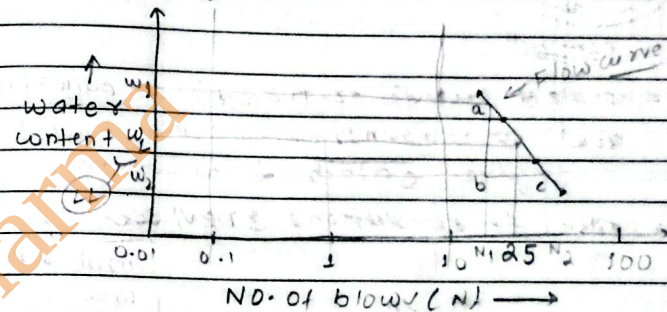
But getting exact 25 No. of blows is not feasible so, ± 15 No. of blows with 25.

i.e. 10-40 N

& Super position is done to find

N: 10-40 & w_L for N=25

→ (Semi-log graph)



From flow curve, we find flow index (I_f)

$$I_f = \frac{ab}{bc} = \frac{w_1 - w_2}{\log N_2 - \log N_1}$$

one pt. method

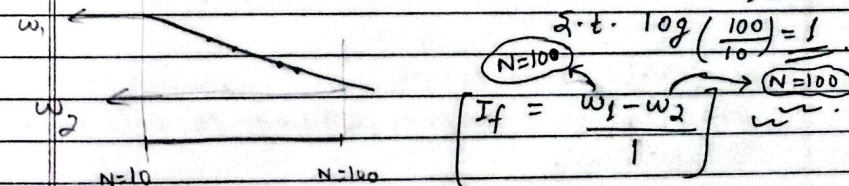
$$w_L (\%) = w_N \left(\frac{N}{25} \right)^n$$

$$\left[I_f = \frac{w_1 - w_2}{\log \left(\frac{N_2}{N_1} \right)} \right]$$

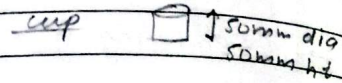
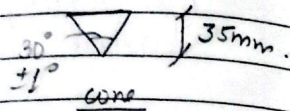
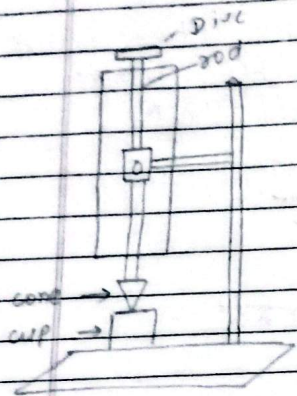
$$\log \left(\frac{N_2}{N_1} \right) = ?$$

$\frac{N_2}{N_1} \rightarrow$ may become fraction & diff. to calculate w/c calculator

So, take, w_1 & w_2 corresponding to the values of NO. of blows = 10 & 100



Cone Penetrometer Test



Mass of Disc + rod + cone = $(80 \pm 0.05)g$

→ Soil: passed thr. 4.25mm sieve

↓
Mix with water

→ Put in cup

→ cone just touch with cup

→ Release cone for 30 sec and note the penetration

→ Find water content of sample

Sample	Penetration	w/c	
1	y_1	w_1	↓ add water
2	y_2	w_2	
3	y_3	w_3	
4	y_4	w_4	

→ Mix water and proceed similar process for other samples along with water content data.

→ Plot: Normal graph.

→ LL is w/c corresponding to penetration of 25mm.

$$M_1 - (V_1 - V_2) \rho_w = M_s$$

Penetration

25mm

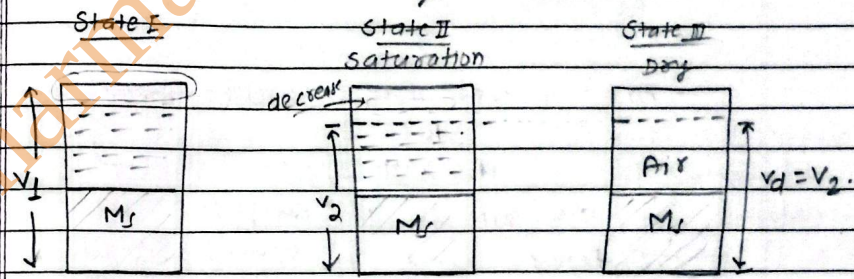
One pt. method,

$$w_s = w_y + 0.01(25 - y)(w_y + 1.5)$$

y (in mm) → penetration when water content is w_y
→ b/w 20 to 30 mm

w/c
↓
LL

Shrinkage limit (w_s)



M_s (total wt)
↓
 M_s

w/c → w_s (shrinkage limit)

$$\text{State II: } w_s = \frac{M_w}{M_s} = \frac{M_1 - M_s - (V_1 - V_2) \rho_w}{M_s}$$

$$= \frac{M_1 - M_s}{M_{ss}} - \frac{(V_1 - V_2) \rho_w}{M_s}$$

$$w_s = w_1 - \frac{(V_1 - V_2) \rho_w}{M_s}$$

w/c of state I

$$\text{Only observing state II: } w_s = \frac{M_w}{M_s} = \frac{(V_2 - V_s) \rho_w}{M_s} = \frac{V_2 \rho_w - \frac{M_s}{M_s} \rho_w}{M_s}$$

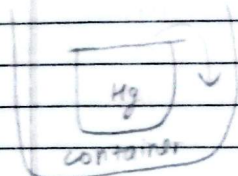
$$w_s = \left(\frac{V_d}{M_s} \right) \rho_w - \frac{\rho_w}{\rho_s} \quad \left[\rho_s = \frac{M_s}{V_s} \right]$$

$$\left[w_s = \frac{\rho_w}{\rho_d} - \frac{1}{G_s} \right] \quad \text{---}$$

- Practical
- Take shrinkage disk:

$\phi = 40 \text{ mm}$
 $h = 15 \text{ mm}$

MASS $\rightarrow M$



press the
excess mercury
with the plate

s.t. excess mercury is
carried into container
(skim off)

measure mass again

Mass of mercury =

Vol of mercury = ?

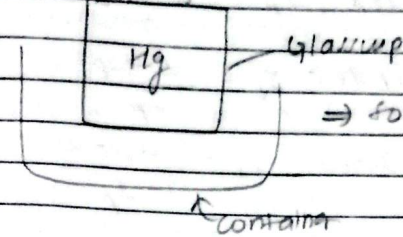
then, volume of shrinkage disk is

Known

→ Oiling of shrinkage disk done (soil doesn't stick to disk)
Then, soil is filled in it and left for oven dry

M_1 (initial mass (not oven dried))
→ oven dry (we get ~~from~~ soil from it)

→ Then take glass cup ($h = 25 \text{ mm}$, $\phi = 50 \text{ mm}$) and pour mercury in it



⇒ soil displaces some amt of mercury & V_d can be known (amt of displaced vol)

$$w_s = \frac{M_w}{M_s} = \left(\frac{M_1 - M_2}{M_s} \right) - \frac{(V_1 - V_2) \rho_w}{M_s} = w_1 - \frac{(V_1 - V_2) \rho_w}{M_s}$$

Shrinkage Ratio (SR)

Condⁿ of volⁿ diff expressed in term of dry volume to change in corres. w/c diff overall.

$$SR = \left(\frac{V_1 - V_2}{V_d} \right) \times 100$$

$$SR = \left(\frac{V_1 - V_d}{V_d} \right) \times \frac{M_s}{(V_1 - V_d) \rho_w} \quad \left\{ \left(w_1 - w_s \right) \right\}$$

$$SR = \left(\frac{M_s}{V_d} \right) \times \frac{1}{\rho_w}$$

$$\left[SR = \frac{\rho_d}{\rho_w} \right] \quad \text{---}$$

Thus,

$$\left[w_s = \frac{1}{S.R} - \frac{1}{G} \right] \quad \text{---}$$

Plasticity Index, $I_p = w_L - w_p$

Liquidity Index, $I_L = \frac{w - w_p}{I_p}$
 ↳ how close to the liquid limit

Consistency Index, $I_c = \frac{w_L - w}{I_p}$ (relative consistency)
 ↳ shows soil is how close to plastic limit

→ $I_p \uparrow \rightarrow$ clay content $\uparrow$

Table 1:

Consistency	$I_c (\%)$
very soft	0-25
soft	25-50
medium	50-75
stiff	75-100
Hard	>100

stiff/hard $\rightarrow I_c \uparrow$

Table 2:

Consistency	I_L
Semi solid to solid	-ve
stiff	0
soft	1
liquid	>1

flowy $\rightarrow I_L \uparrow$

Table 3:

I_p	Plasticity
0	Non-plastic
<7	low-plastic
7-17	Med. plastic
>17	highly plastic

plasticity $\rightarrow I_p \uparrow$

Plasticity Index $\rightarrow$ range in which soil remains plastic
 ↳ still is how close to plastic limit / liquid limit.

⇒ toughness Index, $I_t = \frac{I_p}{I_f}$ } It is a measure of shearing strength of soil at plastic limit

rate at which a soil mass loses its shear strength with increase in water content

Flow index $\rightarrow$ in water content
 (Flow curve) $\rightarrow$ steeper the slope faster it loses its shear strength. compared to flatter slope

Activity of soil (A)

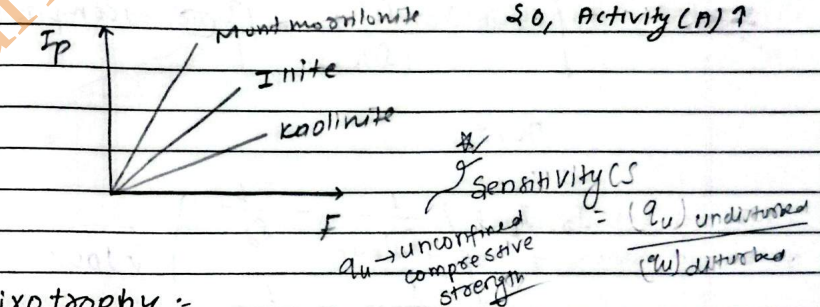
$$A = \frac{I_p}{F}$$

F = % of clay fraction

→ measures water holding capacity of soil (clay $\Rightarrow$ -ve charge)

→ Montmorillonite $\Rightarrow$ finest clay $\rightarrow$ holds most water (steepest slope)
 → clay $\uparrow \Rightarrow I_p \uparrow$ } But, as a whole ratio $(\frac{I_p}{F}) \uparrow$
 ↳ F $\uparrow$

So, Activity (A) $\uparrow$



Thixotropy :-

When the disturbed soil sample allowed to stand without further disturbances and change in water (moisture) content, it will regain its strength, this property is called thixotropy.

Coarse grained soil

relative Index (density index) (I_p/I_d)

It is a ratio of difference between maximum void ratio and natural void ratio and to the difference between max. void ratio and minimum void ratio. expressed in percentage.

$$i.e. I_p/I_d = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

$e_{max} \Rightarrow$ maximum void ratio.
 $e_{min} \Rightarrow$ minimum void ratio.
 $e \Rightarrow$ natural void ratio.

$$s_d = \frac{G_s w}{1+e}$$

$$\Rightarrow [e = \frac{G_s w}{s_d} - 1] \text{---(I)}$$

$$[e_{max} = \frac{G_s w}{(s_d)_{min}} - 1] \text{---(II)} \Rightarrow \text{loosest form of soil}$$

$$[e_{min} = \frac{G_s w}{(s_d)_{max}} - 1] \text{---(III)} \Rightarrow \text{compacted}$$

Thus,

$$I_D, I_P = \left(\frac{\frac{1}{(s_d)_{min}} - \frac{1}{s_d}}{\frac{1}{(s_d)_{min}} - \frac{1}{(s_d)_{max}}} \right) \times 100\%$$

$$[I_D, I_P = \left(\frac{s_d - (s_d)_{min}}{(s_d)_{max} - (s_d)_{min}} \right) \frac{(s_d)_{max}}{s_d} \times 100\%]$$

Compactness of Soil

$(I_p) (\%)$

very loose

< 15

loose

$15 - 35$

medium

$35 - 65$

dense

$65 - 85$

very-dense

> 85

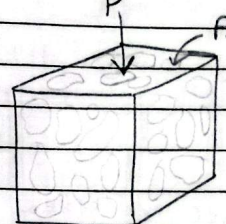
compactness $\uparrow$

* Principle of Effective Stress, Capillarity & Permeability

Effective Stress Principle

$$\sigma = \frac{P}{A} \Rightarrow P = \sigma A$$

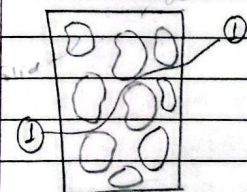
Saturated Soil Sample (Solid Swated)
 NO air void.



$$\text{Total stress } (\sigma) = \frac{\text{Total load } (P)}{\text{Area } (A)}$$

$$P = P_s + P_w$$

$P_s =$ load resisted by solids
 $P_w =$ load resisted by water



Section 1-1 $\rightarrow$ passing through contact points of solids

$\rightarrow$ Area $\neq A$

But, for simplicity Assume it as 'A'

$$A = A_s + A_w$$

$A_s =$ Area occupied by solids $\rightarrow$ Area occupied by water

According to Research,

$$A_w \approx 97\% A$$

$$\text{Thus, } A_w \approx A$$

$$\text{Now, } P = P_s + P_w$$

$$\sigma = \frac{P_s}{A} + \frac{P_w}{A_w}$$

$$P = \sigma = \frac{P_s}{A} + \frac{P_w}{A_w} \quad (\text{or Neutral stress})$$

Thus,

$$\sigma = \frac{P_s}{A} + U$$

Sum of horizontal force on solids, $\Sigma H = 0$

Sum of vertical force, $\Sigma V = P_s (U)$

Thus,

$$\sigma = \frac{\Sigma V}{A} + U$$

$$\sigma = \frac{\Sigma N}{A} + U$$

$$[\sigma = \bar{\sigma} + U]$$

$$\bar{\sigma} = \frac{\Sigma N}{A} = \text{effective stress}$$

$$[\bar{\sigma} = \sigma - U]$$

effective stress

water may flow away but solid always exist and need to carry greater amount of load. So, effective stress comes into play.

analyse & design by taking only the stress taken by solids

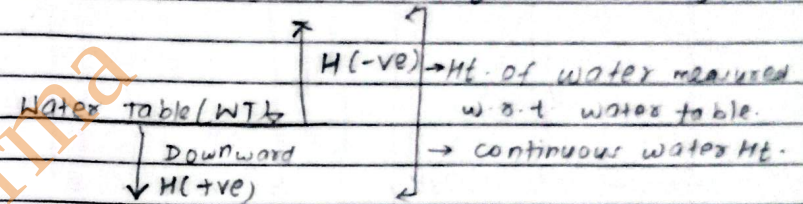
→ 3 Diagram σ ; $\bar{\sigma}$ & U & design by taking effective stress ($\bar{\sigma}$)

$$U = \text{wt. of water} = \frac{A \times \gamma_w \times H}{A} = \gamma_w \times H$$

stress due to material, $\sigma = \text{Ht. of material} \times \text{unit weight}$

$$\sigma = \frac{\gamma \times H \times A}{A} = \gamma H$$

For water pressure, take ht. from ground water table [continuous ht.]



Hydrostatic condition

→ water flows → head lost
→ Velocity very less (water doesn't flow - static)
So, velocity head neglected.

Case I: Dry Soil

At AA,

$\sigma, \bar{\sigma}, U \rightarrow ?$

No applied load, $P = 0$

$$\sigma = \frac{0}{A} = 0$$

$$U = 0 \quad [\text{Dry soil: NO WT \& } H = 0]$$

$$\bar{\sigma} = \sigma - U = 0$$

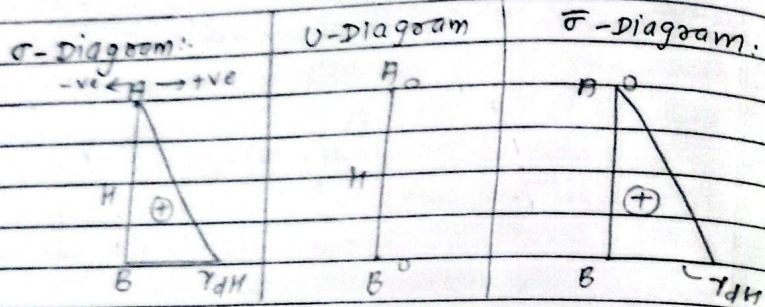
At BB,

load $\rightarrow$ wt. of soil b/w AA & BB [No applied load]

$$\sigma = \gamma_d H$$

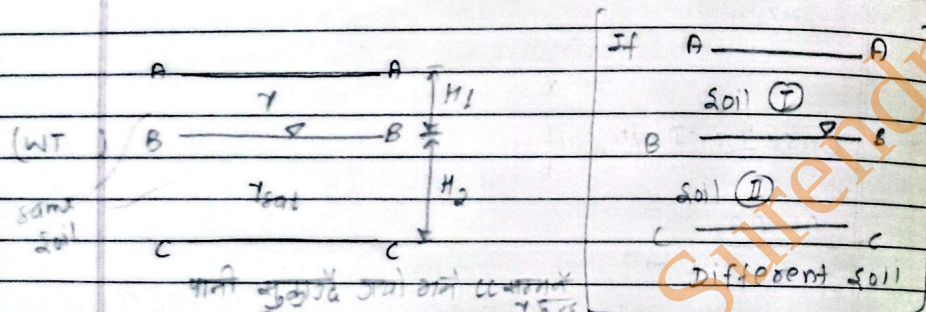
$$U = 0 \text{ [No WT]}$$

$$\bar{\sigma} = \sigma - U = \gamma_d H$$



Case II: Wet $\neq$ Partially Saturated Soil

$\hookrightarrow$ Same Soil as before.



At AA,

$$\sigma = 0$$

$$U = \gamma_w H_1 \text{ (wt. of soil)}$$

$$\text{So, (not of water)}$$

$$U = -\gamma_w \times 0 = 0$$

$$\bar{\sigma} = \sigma - U = 0$$

At BB

$$\sigma = \gamma H_1 \text{ (Soil wt.)}$$

$$U = -\gamma_w \times 0 \text{ at water table } = 0$$

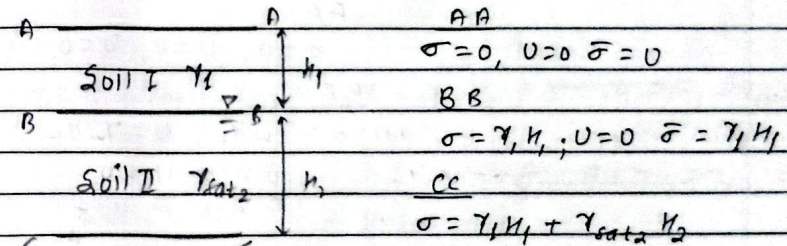
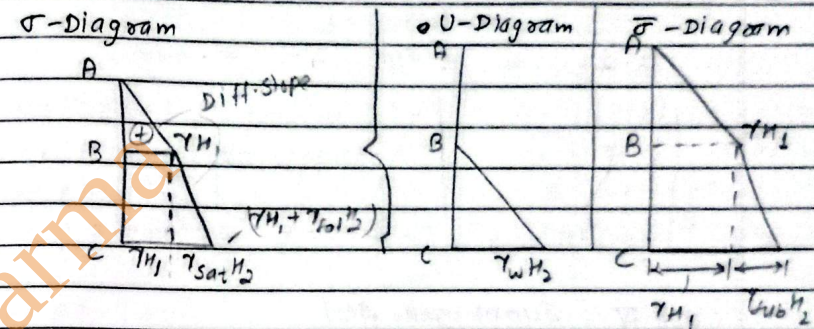
$$\bar{\sigma} = \sigma - U = \gamma H_1$$

At CC,

$$\sigma = \gamma H_1 + \gamma_{sat} H_2$$

$$U = \gamma_w H_2$$

$$\bar{\sigma} = \sigma - U = \gamma H_1 + \gamma_{sat} H_2 - \gamma_w H_2 = \gamma H_1 + H_2 (\gamma_{sat} - \gamma_w) = \gamma H_1 + H_2 \gamma_{submerged}$$



AA
 $\sigma = 0, U = 0, \bar{\sigma} = 0$

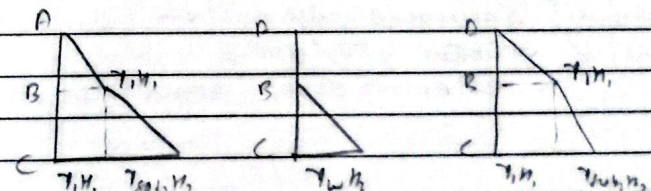
BB
 $\sigma = \gamma H_1, U = 0, \bar{\sigma} = \gamma H_1$

CC
 $\sigma = \gamma H_1 + \gamma_{sat} H_2$

$$U = \gamma_w H_2$$

$$\bar{\sigma} = \gamma H_1 + \gamma_{sat} H_2 - \gamma_w H_2$$

$$\bar{\sigma} = \gamma H_1 + \gamma_{sub} H_2$$

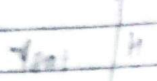


Case III: Saturated Soil

At AA,

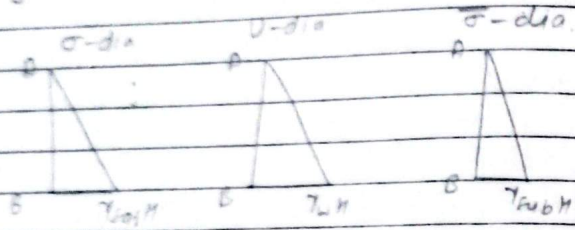
$$\sigma = 0, U = 0, \bar{\sigma} = 0$$

A - - - - - A



At BB,

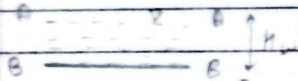
$$\sigma = \gamma_{sat} H, U = \gamma_w H, \bar{\sigma} = \gamma_{sub} H.$$



Case IV: Submerged Soil

AA

$$\sigma = 0, U = 0, \bar{\sigma} = 0$$



BB

$$\sigma = \gamma_w H_w, U = \gamma_w H_w$$

$$\bar{\sigma} = \sigma - U = 0$$

(At B the stress is solid stress, $\bar{\sigma} = 0$)

CC

$$\sigma = \gamma_w H_w + \gamma_{sat} H; U = \gamma_w (H_w + H)$$

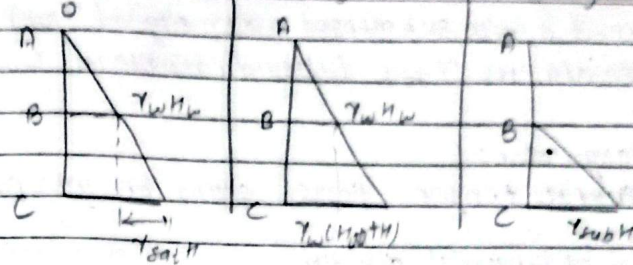
$$\bar{\sigma} = \sigma - U = \gamma_{sub} H$$

once soil becomes saturated soil in ← (Same as saturated case)
effective stress remains same/constant.

σ -Diagram

U-Diagram

$\bar{\sigma}$ -Diagram



5. Surcharged soil

↳ external force applied

At AA,

$$\sigma = q, U = 0, \bar{\sigma} = \sigma - U = q$$

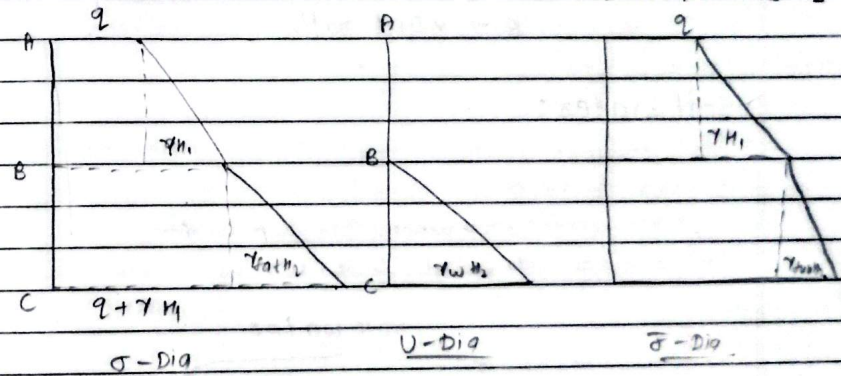
At BB,

$$\sigma = q + \gamma_1 H_1, U = \gamma_w H_1, \bar{\sigma} = q + \gamma_1 H_1 - \gamma_w H_1$$

At CC,

$$\sigma = q + \gamma_1 H_1 + \gamma_{sat} H_2; U = \gamma_w H_2; \bar{\sigma} = q + \gamma_1 H_1 + \gamma_{sat} H_2 - \gamma_w H_2$$

$$\bar{\sigma} = q + \gamma_1 H_1 + \gamma_{sub} H_2$$



- In σ -dia, no submerged density (γ_{sub})
 In $\bar{\sigma}$ -dia, no γ_{sat} (shouldn't write in γ_{sat} form)

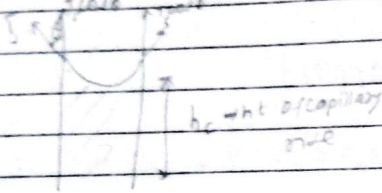
6. Capillary rise:

↳ Surface tension: force acting on interface

↳ tangential direction

STF $\rightarrow$ force per unit length.

diff layers contact plane.



$$T \cos \theta = \frac{\pi}{4} D^2 h_c \gamma_w$$

$$\Rightarrow \left[h_c = \frac{4 T \cos \theta}{\gamma_w D} \right]$$

In soil mechanics, we

Terzaghi & PECK

Father of
soil mechanics.

$$h_c = \frac{C}{e \cdot D_{10}}$$

$D_{10} \rightarrow$ effective diameter

$$C = [0.1 - 0.5] \text{ cm}^2$$

$e \rightarrow$ void ratio

Soil water:

↳ water passes thr. soil unless it finds impermeable layer

Water table $\rightarrow$ sub-surface or depth at which water is top surface.

water flows $\rightarrow$ high head $\Rightarrow$ low head

soil water

Free water/gravitational water

↳ (Flow from higher head to lower head freely)
 ↳ higher elev. to low elev.

held water

• (1) Free-water $\rightarrow$ The water which flows ^{freely} from higher head (elev.) to lower head under gravity force.

• (2) Held-water $\rightarrow$ The water which is held in the pores of soil and cannot flow under gravity action

↳ water is mineral composition of mineral of soil

(a) Structural water $\rightarrow$ the water which is chemically combined with crystal structure of mineral.

(b) Adsorbed water $\rightarrow$

↳ The water which is absorbed by the oven-dried sample from the atmosphere is called adsorbed water

↳ A water present (remained) in air-dried sample of soil.

(c) Capillary water

↳ The water which is above the groundwater table due to the capillary action is known as capillary water

↳ Finer soil: more capillary rise

↳ infinite no. of capillary tube in soil voids

↳ capillary rise say 1cm (void filled with water)

↳ then water table rises 1cm after 1cm soil saturated

silty clay $\rightarrow$ capillary rise

sand, gravel $\rightarrow$ cap. rise xx.

contd

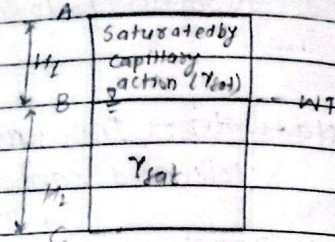
6. Capillary rise

At AA

$$\sigma = 0,$$

$$U = -\gamma_w H_1 \quad (\text{Ht. of water above W})$$

$$\bar{\sigma} = \sigma - U = \gamma_w H_1$$



At BB,

$$\sigma = \gamma_{sat} H_1 ; U = 0 ; \bar{\sigma} = \gamma_{sat} H_1$$

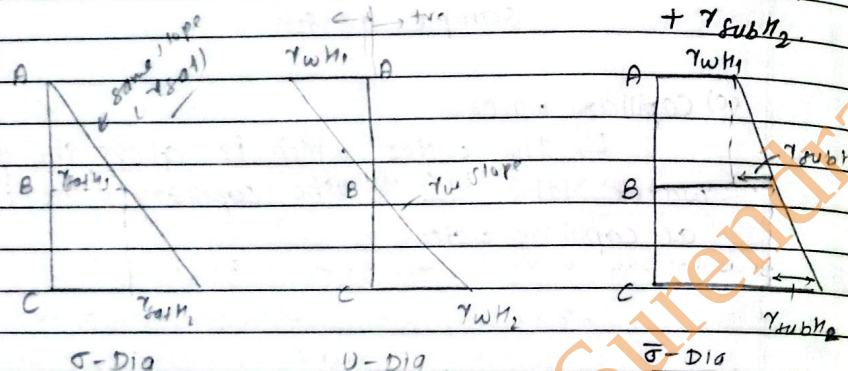
$$= \gamma_{sub} H_1 + \gamma_w H_1$$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$

At CC,

$$\sigma = \gamma_{sat} H_1 + \gamma_{sat} H_2 ; U = \gamma_w H_2 ; \bar{\sigma} = \gamma_{sat} H_1 + \gamma_{sub} H_2$$

$$= \gamma_{sub} H_1 + \gamma_w H_1$$



• same soil is saturated $\rightarrow$ same slope.

• same soil is submerged $\rightarrow$ same slope

Diff. soil $\rightarrow$ Diff. slope.

contd.

The effective stress diagram of surcharged case is similar to capillary rise case

7. Partial Capillary rise:

At AA,

$$\sigma = 0, U = 0, \bar{\sigma} = 0$$

Not continuous water from W

At BB (Just below) to AA

$$\sigma = \gamma_1 H_1, U = -\gamma_w H_2,$$

$$\bar{\sigma} = \gamma_1 H_1 + \gamma_w H_2$$

At BB (Just above)

$$\sigma = \gamma_1 H_1, U = 0, \bar{\sigma} = \gamma_1 H_1.$$

At CC

$$\sigma = \gamma_1 H_1 + \gamma_{sat, H_2}, U = 0, \bar{\sigma} = \gamma_1 H_1 + \gamma_{sat, H_2} = \gamma_1 H_1 + \gamma_{sub, H_2} + \gamma_w H_2$$

At DD,

$$\sigma = \gamma_1 H_1 + \gamma_{sat, H_2} + \gamma_{sat, H_3}, U = \gamma_w H_3,$$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$

$$\bar{\sigma} = \gamma_{sub, H_3} + \gamma_1 H_1 + \gamma_{sub, H_2} + \gamma_w H_2$$

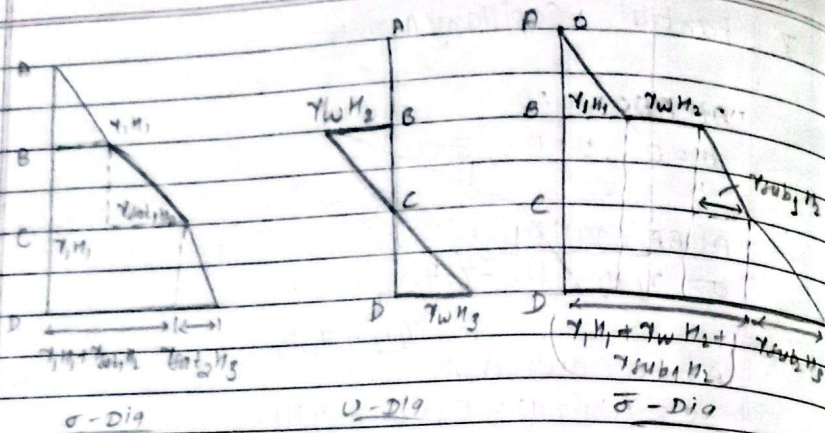
BB $\rightarrow$ Just above no water $\rightarrow$ no capillary rise so, $U = 0$

BB $\rightarrow$ Just below: there is water of capillary rise $\rightarrow U = -\gamma_w H_2$

of two soil $\Rightarrow$ Interface of two soils (or soil) $\rightarrow$ two conditions should be considered.

Before (6) $\rightarrow$ Interface of air and soil so not considered above part

Interface $\rightarrow$ 2 part. $\rightarrow$ above (air) $\rightarrow$ below (soil)



effective stress dia $\rightarrow$ saturated soil to submerged
form on 3/13/20

water pressure = $\gamma_w H_w$ $\rightarrow$ Total head = Elevation + pressure
Piezometric head (for static condition)

$\rightarrow$ Hydraulic head $\rightarrow$ head which causes the flow of water from higher head to lower head
Hydraulic gradient $\rightarrow$ head lost per unit length of flow
hydraulic head equal to diff in head b/w entry & exit pt

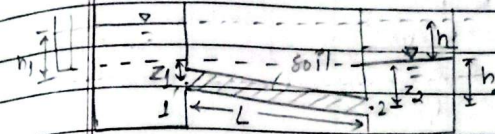
hydraulic gradient $(i) = \frac{h}{L}$

* Flow condition

- Downward flow

- Upward Flow

downward flow

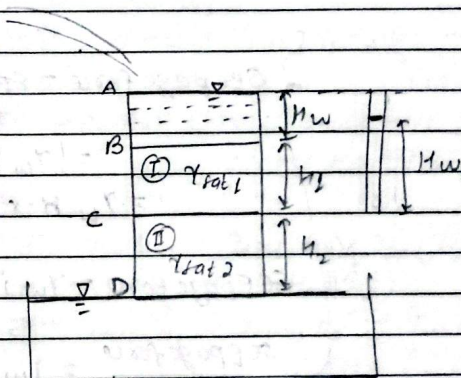


pipe head loss & velocity head not considered

1 $\rightarrow$ entry pt. 2 $\rightarrow$ exit pt
head loss $\rightarrow$ only in soil

(1) Total head = $h_1 - z_1$

(2) exit point $\rightarrow$ total head
 $h_2 - z_2 = 0$



At AA, $\sigma = 0$, $U = 0$, $\bar{\sigma} = 0$

At BB, $\sigma = \gamma_w H_w$, $U = \gamma_w H_w$, $\bar{\sigma} = 0$

At CC, $\sigma = \gamma_w H_w + \gamma_{sat} H_1$, $U = \gamma_w (H_w + H_1)$, $\bar{\sigma} = \gamma_{sub} H_1 + \gamma_w (H_w + H_1)$

At CC, $\sigma = \gamma_w H_w + \gamma_{sat} H_1 - \gamma_w H_w$
 $U = \gamma_w H_w$, $\bar{\sigma} = \sigma - U = \gamma_{sub} H_1 + \gamma_w (H_w + H_1 - H_w)$

$$\bar{\sigma} = \gamma_{sub} H_1 + \gamma_w (H_w + H_2 - H_{loss})$$

→ due to downward flow of water

→ effective stress

increased by

factor of $\gamma_w \times \text{head loss}$

→ $\gamma_w \times \text{Head loss} = \text{seepage pressure}$

→ $H_w + H_2 - H_{loss} = h$

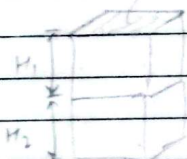
At DD, (atmospheric)

$$\sigma = \gamma_w H_w + \gamma_{sat} H_1 + \gamma_{sat} H_2$$

$$U = 0 \text{ (due to atm. pressure)}$$

$$\bar{\sigma} = \gamma_{sub} H_1 + \gamma_{sub} H_2 + \gamma_w (H_w + H_2)$$

$h(\text{head loss})$



Seepage force = seepage pressure

$\times \text{Flow area}$

$$= (\gamma_w h) \times A$$

$$= \gamma_w h \times A \times (H_1 + H_2)$$

$$\text{Seepage force} = \gamma_w i \cdot V$$

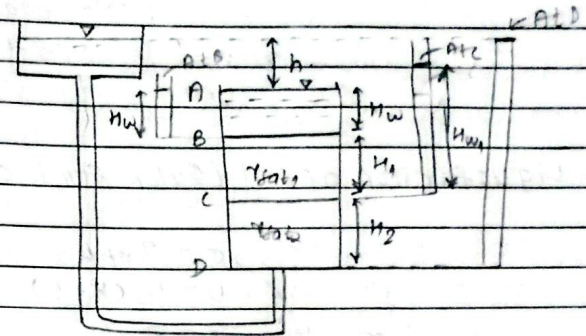
$$\frac{\text{seepage force}}{V} = \gamma_w i$$

$i \rightarrow \text{Hydraulic gradient} = \frac{\text{head loss}}{\text{Total length}}$

$$i = \frac{h}{H_1 + H_2}$$

Constant for given case

Upward flow



$$\text{At AA, } \sigma = 0, U = 0, \bar{\sigma} = \sigma - U = 0$$

$$\text{At BB, } \sigma = \gamma_w H_w, U = \gamma_w H_w, \bar{\sigma} = 0$$

$$\text{At CC, } \sigma = \gamma_w H_w + \gamma_{sat} H_1$$

$$U = \gamma_w H_w$$

$$\bar{\sigma} = \sigma - U = \gamma_{sub} H_1 + \gamma_w (H_w + H_2 - H_{loss})$$

seepage pressure

In upward flow, effective pressure decreases by seepage pressure

$$\text{At DD, } \sigma = \gamma_w H_w + \gamma_{sat} H_1 + \gamma_{sat} H_2$$

$$U = \gamma_w (h + H_w + H_1 + H_2)$$

$$\bar{\sigma} = \gamma_{sub} H_1 + \gamma_{sub} H_2 - \gamma_w h$$

seepage pressure

(Against decrease)

by seepage pressure

$h \rightarrow \text{variable}$

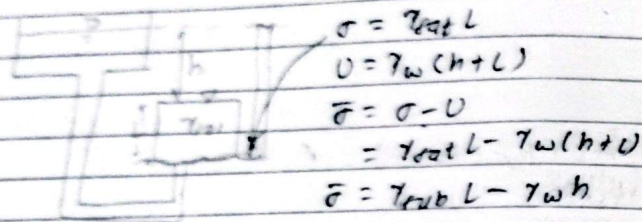
(higher the water tank higher than)

At some condition, $\bar{\sigma} = 0$

Soil is stronger than water

At $\bar{\sigma} = 0$, soil turned into liquid state
 So, soil can't bear any load
 and called liquefaction of soil
 (or) Quick sand condition)

Liquefaction of soil (Quick sand condition)



For, $\bar{\sigma} = 0$ (by $\uparrow h$)

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

$$\text{or, } \frac{h}{L} = \frac{\gamma_{sub}}{\gamma_w} \quad h \rightarrow \text{max}$$

① Critical hydraulic gradient $= i_c = \frac{h}{L} = \frac{\gamma_{sub}}{\gamma_w}$

For, $i_c = 1$, then quick-sand condition arises.

Condition for Quick sand or Liquefaction of soil

The condition at which soil loses its total shear strength & behaves like liquid.

shear strength, $[s = c + \bar{\sigma} \tan \phi]$

Modified/ Revised eqⁿ:

$s \rightarrow$ shear strength

$c \rightarrow$ cohesion

$\bar{\sigma} \rightarrow$ effective strength.

So, even, for $\bar{\sigma} = 0$, $s \neq 0$ if $c \neq 0$, (not Quick-sand)
 * Conditions:

- (i) $\bar{\sigma} = 0$
 - (ii) $c = 0$
 - (iii) $s = 0$
- $\rightarrow s = 0 \rightarrow$ Quick sand condition
 for sand, frictionless (cohesion-less soil)

(iv) upward flow condition

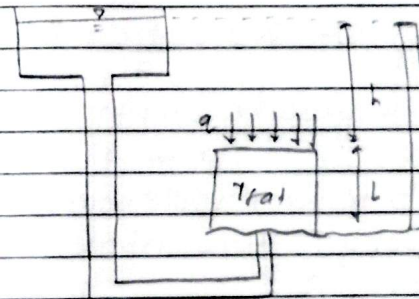
(v) Solid particles moves in upward direction along with water molecule which look like boiling of water.

Therefore, it is known as boiling of sand/ Boiling condition

(vi) Water- Soil behaves like liquid losing all its shear strength, having density twice that of water. ($\frac{\gamma_{sub}}{\gamma_w} = 2$)

(vii) Thus, $i_c \approx 1$.

Effect of surcharge in Liquefaction of soil



For, $\bar{\sigma} = 0$

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

$$\text{or, } h = \frac{\gamma_{sub} L + q}{\gamma_w}$$

$$\left[h = \frac{\gamma_{sub} L}{\gamma_w} + \left(\frac{q}{\gamma_w} \right) \right]$$

- ✓ → Head req. to cause quick sand condition is increased by factor of $\frac{2}{\gamma_w}$
- Surcharge increases the stability against quick-sand condition.

6. Permeability

- It is an ability of soil which allows the flow of water through its cross-section.
- It can be defined as an ease through which water can pass through soil mass.

Darcy's Law:

$$\text{Rate of flow} \longrightarrow q = \frac{\text{discharge}}{t} = \frac{Q}{t}$$

Rate of flow or discharge through cross-sectional area A , $[q \propto iA]$ is directly proportional to product of hydraulic gradient i and cross-sectional area (A)

i.e.

$$q \propto iA$$

$$\text{or } q = KiA$$

where, K = coefficient of permeability; }
hydraulic conductivity; }
constant of proportionality }

Flow velocity

$$V = \frac{q}{A} = \frac{KiA}{A} = Ki \quad \text{--- (1) } \quad A \rightarrow \text{flow area.}$$

v = flow velocity

From eqⁿ

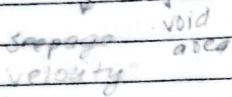
- Permeability can be defined as flow velocity under unit hydraulic gradient.

For, $i = 1$

$$[V = K]$$

- Permeability is the discharge through unit cross-sectional area under unit hydraulic gradient.

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~~$$\text{Or, } V = V_s \frac{A_v}{A} \times \frac{H}{H}$$~~

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

$$[V = V_f \times T] \rightarrow \text{priority}$$

$$\frac{q}{A} = k i = V = V_s n$$

$$V_s = \left(\frac{k}{n} \right) i$$

$$[v_j = k_p i] \quad (k_p > k)$$

$$[v_s > v]$$

- Lab method:

पानी की कति दियो १०००; १००० पर ५०००

↳ कुन माटोमा कुनैको त्यसै 106 ग ठगपुन सकिन्छ

↳ economic

↳ undisturbed condition मा लव मा ल्याउन सकिने sample.

Undisturbed sample $\rightarrow$ of clay

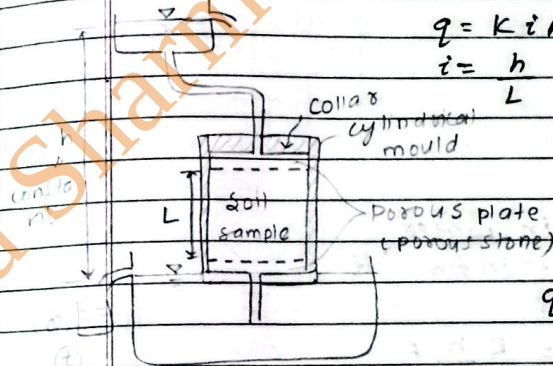
Else, field method (If undisturbed sample cannot be brought to lab).

→ metallic mould of int. dia 100mm
& ht. 127.3mm & 1000ml capacity

(i) Constant head method:

$$q = K i A$$

$i = h$; $A \rightarrow \text{cls area.}$



✓ porous plate

✓ Normally, Vol. of
soil sample $\rightarrow 1000 \text{ cc.}$

- Porous stone

↳ पानी मात्र दिने

पथो माहो जानु

अएन

↳ Why?
↳ kept.

तब: पानी में
भाटे जाऊ

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

$$\frac{\Phi}{t} = K \frac{h}{L} \cdot A$$

$$[k = \frac{Q \cdot L}{h A t}]$$

porous stone

इसे permeability
माने गन्दा
धरे हुए पद

A. $K_{\text{power}} \times 10 \times K_{\text{ball}}$

So, all water flows
w/o disturbance

Soil must be saturated
(before practical
soil permeability
must be saturated)

upper porous plate

so that soil
doesn't come out
during solota-
tion of soil
by upward flow

Have time, porous stone kept in water (submerged) over

Noting, porous stone kept in boiling

water for 8-10 min.

saturating in $\Rightarrow$ by upward flow
of water (pipe coming from
deeper water pipe)

→ A vertical glass tube (burette) is fitted to permeameter. (permeameter is same as of constant head method)
 ↳ The time sep to flow water from initial head h_1 to head h_2 is taken.

(ii) Falling head method (variable head)

→ Keep porous plate

→ saturated soil

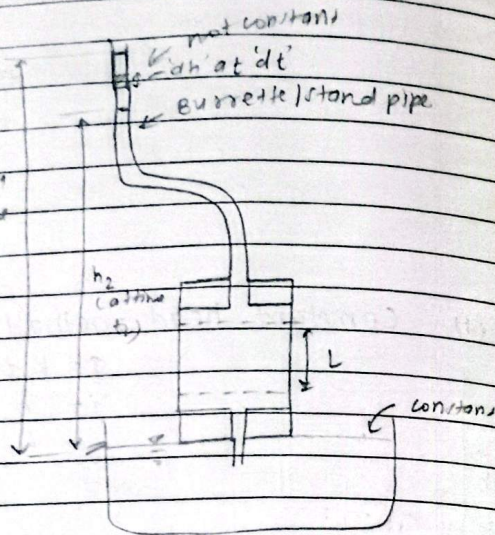
porous plate same as before.

h_1 = Initial head in Burette at $t=0$

h_2 = head in Burette after time 't'

A = c/s area of sample.

a = c/s area of Burette.



Let, 'dh' be the fall in water head in Burette in time 'dt'

Then, $\frac{dV}{dt} = \text{var.}$
 $-a \frac{dh}{dt} = K \frac{h}{L} A$

on $\int_{h_1}^{h_2} \frac{dh}{h} = -\frac{KA}{aL} \int_0^t dt$

or, $\ln\left(\frac{h_2}{h_1}\right) = -\frac{KA}{aL} \times t$

or $\ln\left(\frac{h_1}{h_2}\right) = \frac{KA}{aL} \times t$

on $\left[K = \frac{aL}{At} \ln\left(\frac{h_1}{h_2}\right) \right] \checkmark$

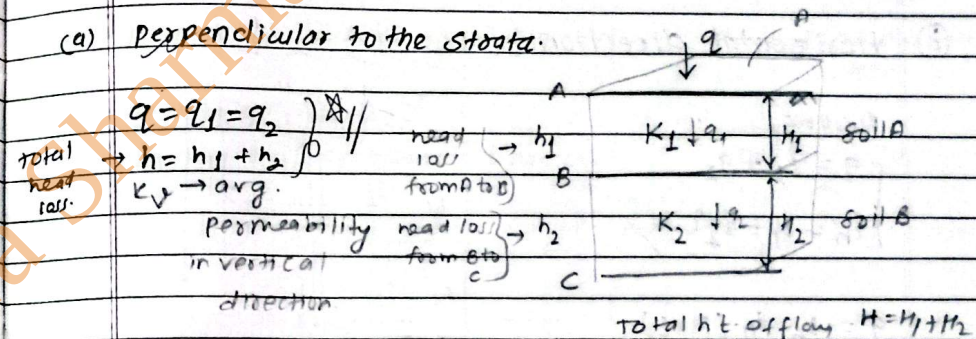
$$K = \frac{2.303 aL}{At} \log_{10} \left(\frac{h_1}{h_2} \right)$$

Burette kept at constant temp. & time.

Permeability $\uparrow$ → छोटी सी Burette फलन पठा
 eg. sand लोई सी Burette फलन पठा
 Permeability $\downarrow$ → बड़ी Pipe फलन पठा
 So, that measurement of time is easier.

* permeability of stratified soil

(a) perpendicular to the strata.



We know,

$q = K i A$ $A \rightarrow$ same for each layer.

$q = K_v i_v A = K_v \left(\frac{h_1 + h_2}{h_1 + h_2} \right) A = K_v \left(\frac{h}{H} \right) A$

$q_1 = K_1 i_1 A = K_1 \left(\frac{h_1}{H} \right) A$

$q_2 = K_2 i_2 A = K_2 \left(\frac{h_2}{H} \right) A$

Thus,

$h = h_1 + h_2$

or $\frac{qH}{K_v A} = \frac{qH_1}{K_1 A} + \frac{qH_2}{K_2 A}$
 or $\left[\frac{H}{K_v} = \frac{H_1}{K_1} + \frac{H_2}{K_2} \right] \checkmark$
 $\left[K_v = \frac{H}{\frac{H_1}{K_1} + \frac{H_2}{K_2}} \right] \checkmark$

$$K_v = \frac{H_1 + H_2}{\frac{H_1}{K_1} + \frac{H_2}{K_2}}$$

$$= \frac{\sum_{i=1}^n H_i}{\sum_{i=1}^n \left(\frac{H_i}{K_i} \right)}$$

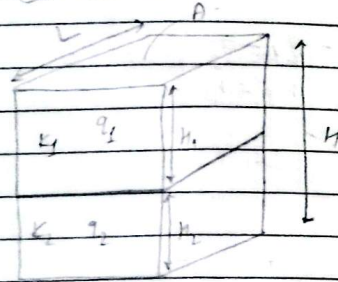
$$K_v = \frac{H}{\sum_{i=1}^n \left(\frac{H_i}{K_i} \right)}$$

⑤ Horizontal direction: (Along the strata)

Assume,

$$q = q_1 + q_2$$

$$[i_h = i_1 = i_2]$$



$$q = K_h i_h (H \times L)$$

$$q_1 = K_1 i_1 (H_1 \times L)$$

$$q_2 = K_2 i_2 (H_2 \times L)$$

Thus,

$$K_h i_h (H \times L) = K_1 i_1 (H_1 \times L) + K_2 i_2 (H_2 \times L)$$

$$\text{or } [K_h = \frac{H_1 K_1 + H_2 K_2}{H}]$$

$$[K_h = \frac{H_1 K_1 + H_2 K_2}{H_1 + H_2} = \frac{\sum_{i=1}^n H_i K_i}{H}]$$

Hydraulic Gradient (i) $\rightarrow$ slope of flow line
($i = \frac{\Delta h}{L}$)

* Factors affecting permeability 'K'

Kozeny & Karman eqⁿ $[K = \frac{C D^2 (e^3)}{(1+e)} \frac{\gamma_w}{\eta}]$

e = void ratio.

D = particle size

C = constant (shape factor)

η = viscosity.

$$K = C (D_{10})^2 \quad K \rightarrow \text{cm/sec}$$

$$\{C \rightarrow 100 \text{ to } 150\} (D_{10})$$

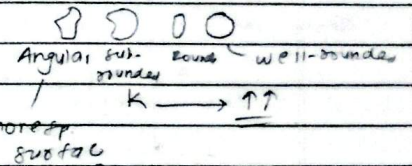
directly proportional to square of particle size.

(i) $K \propto D^2$

(ii) $K \propto C$
dependent upon C

$$K \propto \frac{1}{\text{specific surface}}$$

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



(iv) $K \propto \frac{1}{\eta}$

$$[\eta \propto \frac{1}{\text{Temp.}}] \Rightarrow [K \propto \text{Temp. of soil}]$$

(v) Strata.

(vi) Degree of saturation (S_r)

$$S_r \uparrow K \uparrow$$

(vii) Adsorbed water

(void space में पानी)

(doesn't flow due to gravity)

resist flow of water

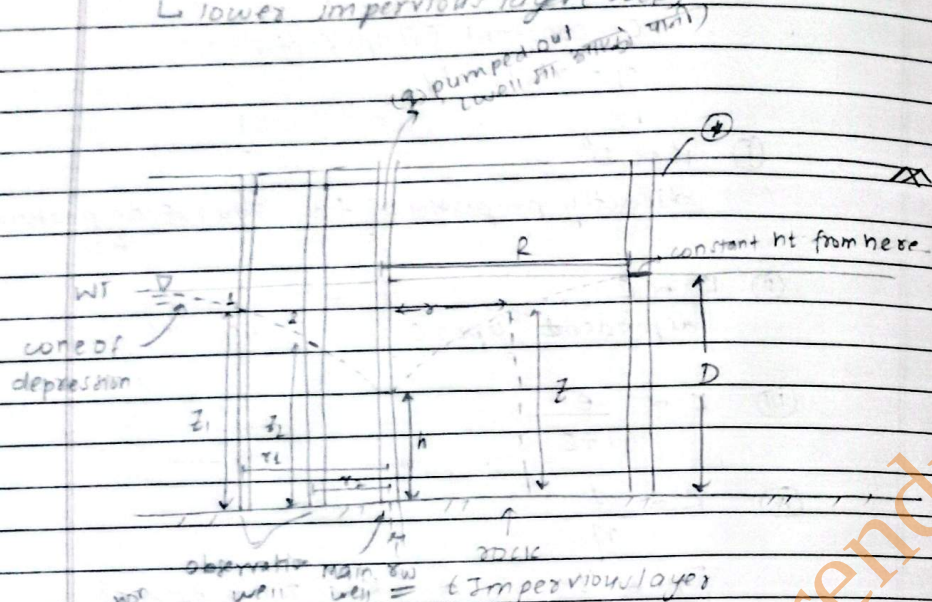
$K \downarrow$ due to Adsorbed water.

(viii) Impurities

* Field Methods

* Unconfined aquifer

- sandy, gravel, coarse grains
- connected to atmosphere above
- lower impervious layer (rock)

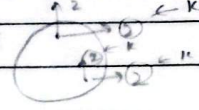
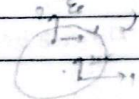


* Dupit's assumption:-

- (i) Laminar flow
- (ii) Flow is steady
- (iii) 'K' is constant
- (iv) Soil is homogeneous and isotropic

Soil mass of diff
pt. in same prop

each dist. on
K is same



(v) Well is perfect

(penetrated)
→ well which is bored upto the impervious rock

(vi) Well का पानी निकालने की मात्रा, (but we only know no. distance or slope cannot be determined)

Flow is towards the well which is horizontal & radial.

(vii) Hydraulic gradient is very small

(viii) Darcy's law is valid.

r_w → radius of well

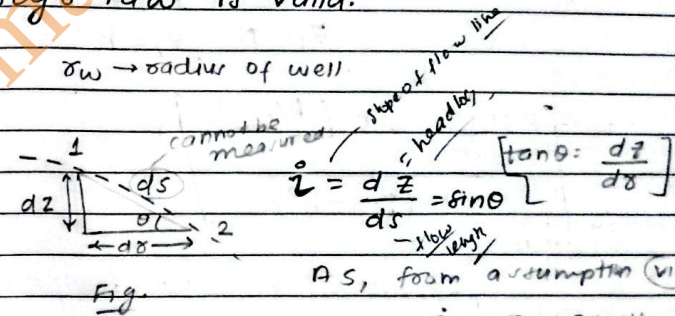


Fig.

As, from assumption (vii)

i → very small

So, $\theta \rightarrow 0$

$$i = \sin \theta \approx \tan \theta \approx \frac{dz}{dr}$$

$$\left[i = \frac{dz}{dr} \right]$$

From Darcy's law,

$$q = K i A$$

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

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

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

$$k = \frac{q \ln(r_1/r_2)}{\pi(z_1^2 - z_2^2)}$$

$$\left[k = 2.303 \cdot \frac{q}{\pi} \frac{\log_{10}(r_1/r_2)}{z_1^2 - z_2^2} \right]$$

If we can know 'R' → radius after which const. level in well.

$$\text{So, applying, } \begin{cases} r_1 = R \\ r_2 = r_w \end{cases}$$

$$z_1 = D, \quad z_2 = h$$

$$\left[k = \frac{q \ln(R/r_w)}{\pi(D^2 - h^2)} \right]$$

D → ht. of WT from impervious layer.

* Confined Aquifer

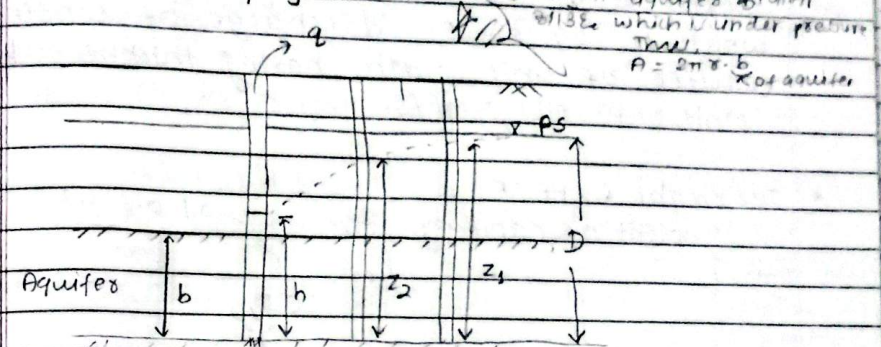


Fig. show a confined aquifer of thickness, 'b' lying b/w two aquicludes. In confined aquifer, water pressure is indicated by piezometric surface (PS). PS is water table equivalent to confined aquifer. Initially, PS is horizontal & after pumping, it is depressed and cone of depression is formed.

$$q = K i A$$

$$\text{or, } q = K d z \cdot 2\pi r \cdot b$$

$$\text{or, } \frac{q}{2\pi b} \frac{dr}{r} = K dz$$

Integrating,

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

$$\text{or, } \frac{q}{2\pi b} \ln\left(\frac{r_1}{r_2}\right) = K(z_1 - z_2)$$

$$\text{or, } K = \frac{q \ln(r_1/r_2)}{2\pi b(z_1 - z_2)}$$

* coefficient of transmissibility

$$= b \times K$$

thickness of aquifer

discharge of aquifer of unit width having thickness, b.

coefficient of transmissibility is the coefficient which is equal to discharge of confined aquifer of unit width having thickness b .
Unit = $m \cdot m/s = m^2/s$.

* Terzaghi & PECK

height of capillary rise, $h_c = \frac{0.1-0.5 \text{ cm}^2}{C}$
Soil }
effective diameter.
We don't use, $h_c = \frac{4 \gamma_w \cos \theta}{\gamma_w D}$

Seepage

→ Flow of water through any permeable medium is seepage

1. Flowline:

The path followed by flowing water

2. Equipotential line:

→ Different flow lines at equal head

→ The line joining diff. pts. of varies flow lines having equal potential

3. Flow channel:

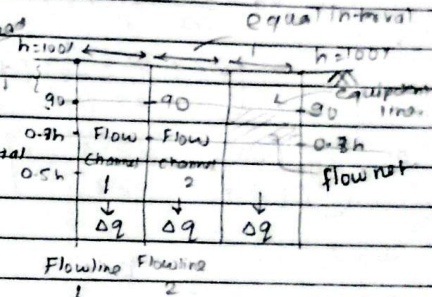
→ The space between adjacent flowlines

→ Flow is constant

→ in each flow channel.

4. Flow net

→ The network of flow lines and equipotential line meeting right angle to each other.



→ Flow channel → space b/w two adjacent flowlines

→ For homogeneous soil, discharge through each flow channel same.

→ For head = 0, flow rate, head loss b/w two adjacent equipotential line is constant

* Laplace equation:-

→ Graphical solution of Laplace equation is flow net

Assumptions

(i) → Flow is two dimensional

(ii) → Flow is laminar

(iii) → Flow soil is laminar

(iv) → Water and solid are incompressible

(v) → Solid is homogeneous and isotropic

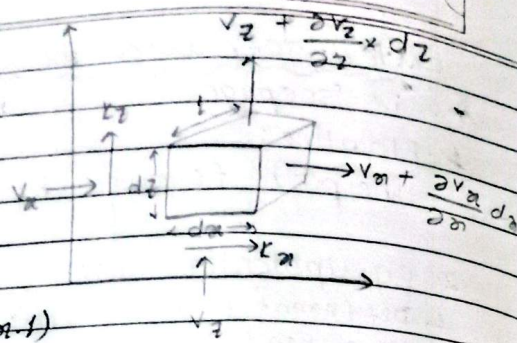
(vi) → Darcy's law is valid.

2D (2D) → Steady flow
2D (2D) → Soil is homogeneous, isotropic & saturated.
→ stresses are neglected.
→ Darcy law valid.

Discharge is constant

$$q_{in} = q_{out}$$

$$q = V_x \times (dz \cdot 1) + V_z \times (dx \cdot 1) = (V_x + \frac{\partial V_x}{\partial x} dx)(dz \cdot 1) + (V_z + \frac{\partial V_z}{\partial z} dz)(dx \cdot 1)$$



or $\nabla \cdot \mathbf{v} = 0 \rightarrow$ Laplace continuity equation.

$$q = k i A$$

$$x: k \rightarrow k_x$$

$$z: k \rightarrow k_z$$

$$v = k i$$

$$x: V_x = k_x \frac{\partial h}{\partial x} \Rightarrow \frac{\partial V_x}{\partial x} = k_x \frac{\partial^2 h}{\partial x^2}$$

$$V_z = k_z \frac{\partial h}{\partial z} \Rightarrow \frac{\partial V_z}{\partial z} = k_z \frac{\partial^2 h}{\partial z^2}$$

Thur,

$$k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0$$

As, soil is isotropic, $k_x = k_z = k$

$$\left[\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2} = 0 \right] \rightarrow \text{Laplace 2D equation in terms of head loss.}$$

equipotential line

velocity potential (ϕ)
It is a scalar quantity of time and space whose partial derivative in any direction gives the velocity in that direction.

$$\frac{\partial \phi}{\partial x} = V_x \quad \& \quad \frac{\partial \phi}{\partial z} = V_z$$

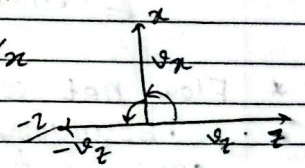
$$\Rightarrow \frac{\partial^2 \phi}{\partial x^2} = \frac{\partial V_x}{\partial x} \quad \& \quad \frac{\partial^2 \phi}{\partial z^2} = \frac{\partial V_z}{\partial z}$$

Thur, From ①,

$$\left[\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \right] \rightarrow \text{Laplace 2D equation in terms of velocity potential } (\phi)$$

flow line
Stream function (ψ) $\rightarrow$ scalar qty of time & space

$$\frac{\partial \psi}{\partial x} = -V_z \quad ; \quad \frac{\partial \psi}{\partial z} = V_x$$



$$\frac{\partial \phi}{\partial x} = \frac{\partial \psi}{\partial z} \quad \& \quad \frac{\partial \phi}{\partial z} = -\frac{\partial \psi}{\partial x}$$

$$\frac{\partial^2 \phi}{\partial x^2} = \frac{\partial^2 \psi}{\partial x \partial z} \quad \& \quad \frac{\partial^2 \phi}{\partial z^2} = -\frac{\partial^2 \psi}{\partial x \partial z}$$

Thur, from above eq =

$$\frac{\partial^2 \psi}{\partial x^2} - \frac{\partial^2 \psi}{\partial z^2} = 0$$

Thur, stream function satisfies Laplace equation.

$$d\phi = \frac{\partial \phi}{\partial x} dx + \frac{\partial \phi}{\partial z} dz = 0 \quad \left\{ \begin{array}{l} \text{Since } \phi = \text{constant} \\ \text{is a scalar quantity} \\ \text{not dependent on direction} \end{array} \right.$$

$$\text{or } v_x dx + v_z dz = 0$$

$$\Rightarrow \left[\frac{dz}{dx} \right]_{\phi} = - \frac{v_x}{v_z}$$

$$d\psi = \frac{\partial \psi}{\partial x} dx + \frac{\partial \psi}{\partial z} dz = 0 \quad \left\{ \begin{array}{l} \psi \rightarrow \text{scalar quantity} \\ \text{is constant} \end{array} \right.$$

$$\text{or } -v_z dx + v_x dz = 0$$

$$\Rightarrow \left[\frac{dz}{dx} \right]_{\psi} = \frac{v_z}{v_x}$$

Hence, ψ & ϕ are orthogonal. ($m_{\psi} \times m_{\phi} = -1$)

* Flow net constant construction:

• Boundary condition (B.C.):

confined flow $\rightarrow$ All B.C. known

unconfined flow $\rightarrow$ if all boundary condition not known

Boundary conditions	FL	EPL
(i) 1 st flow line		100% epl
(ii) Last flow line $\rightarrow$ impervious layer (rock)		
(iii) 100% equipotential line		
(iv) 0% equipotential line		0% epl

* Impervious layer is last flowline.

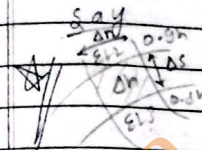
* Every equipotential line meet the impervious layer (last flow line) at right angle.

Sheet pile is a long, thin section of material (often steel, timber or reinforced concrete) with interlocking edge.
 $\rightarrow$ provide earth retention and support during construction & excavation.
 $\rightarrow$ retain soil or water creating stable barrier.

Sheet pile:

$\rightarrow$ Temp. str. (reversible)

$\rightarrow$ Interface of soil & sheet pile: shortest path to travel



Boundary condition:

1. 1st flow line = BCD

2. Last flow line = XY

3. 100% equipotential line = AB

4. 0% equipotential line = DE

Other flow lines $\rightarrow$ 1st to 1st flow line

interval b/w flow lines $\rightarrow$ self judgement

* Max^m flow line: 3-4

flow net $\rightarrow$ square (approx) (rotation) (2/3/4) may not be equal

equipotential line $\rightarrow$ s.t. flow net square

(same interval as FL)

b/w two EPL $\rightarrow$ equal head loss



* Use of flow net

(a) Calculation of seepage discharge:

$N_f \rightarrow$ No. of flow lines/channel # 3

$N_d \rightarrow$ No. of equipotential drop = 7

(Equipotential drop & equipotential line)

$N_f \downarrow$ then $N_d \downarrow$ But ratio of N_f and N_d

$N_f \uparrow$ then $N_d \uparrow$ $\frac{N_f}{N_d} = \text{constant}$

(Whatever may be interval)

$\Delta q \rightarrow$ discharge through single channel

$q \rightarrow$ total discharge

$$q = N_f \times \Delta q \quad \text{flow area}$$

$$= N_f \times K \times \left(\frac{\Delta h}{\Delta s} \right) (\Delta n \times l)$$

as $\Delta n \approx \Delta s$

$$= [N_f \times K \times \Delta h]$$

$$= N_f \times K \times \Delta h$$

$$\left(\Delta h = \frac{h}{N_d} \right)$$

$$\left[q = \frac{N_f}{N_d} (K \cdot h) \right] \text{ we}$$

(b) Hydraulic gradient $i = \frac{\Delta h}{\Delta s}$ can be calculated.

$$i = \frac{\Delta h}{\Delta s}$$

$$\Delta h = h \leftarrow \text{known}$$

$$N_d \leftarrow \text{count}$$

It in any flow net, (upward flow), (right pass)

$i \rightarrow$ opposite

then quick sand condition. (sheet pile failure)

(c) Total head

At any pt. (say P),

$$h_p = h - (\Delta h)_{\text{up to } P}$$

$$= \text{Initial head} - n \times \Delta h$$

[n can be 1, 2, 3, ...] no. of head loss, head lost b/w two eqv. lines

(d) Pressure head,

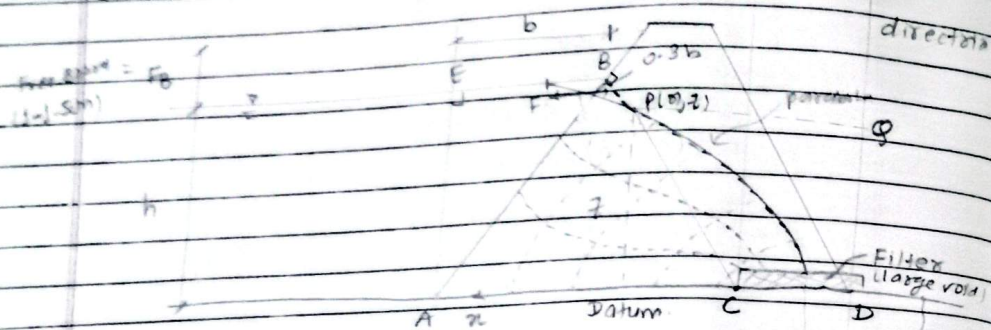
$$\text{At } P, (h_p)_p = \text{Total head} - \text{Elevation head}$$

$$= h_p - [-(h_e)_p]$$

$$= h_p + (h_e)_p \quad \text{below datum}$$

- ① head loss static zone
② slope

2. Earth Dam



Filter criteria

① $K_{filter} > K_{soil}$

② But not large enough st. to allow percolate flow

AB → each pt. 100% head $z = h$
(only water should pass not soil)

100% equipotential line → AB

0% equipotential line → CD

Last flow line → AC (total unit area cut area)

1st FL → Phreatic line

AE → vertical line

BE → b

E → from 30% of b from B

Draw parabola from E to filter S-1
top edge of filter (just above C) is
focus of parabola.

1st FL (but not from F)

Acc to Caragrande's → parabola 1st FL

$$CP = p\phi$$

$$\text{or } \sqrt{x^2 + z^2} = a + s$$

$$\text{or } x^2 + z^2 = a^2 + s^2 + 2as$$

$$[z^2 = s^2 + 2as]$$

$$[z = \sqrt{s^2 + 2as}]$$

For F,

$$z = \frac{b}{a}$$

$$[h^2 = s^2 + 2sd]$$

$$\frac{dz}{ds} = \frac{1}{2\sqrt{s^2 + 2as}} \times 2s$$

$$\left[\frac{dz}{ds} = \frac{s}{\sqrt{s^2 + 2as}} \right]$$

Now make parabola from eq ①

x	1m	2	3	...
z	...	...	...	...

But actually, water comes from B,
join the parabola to B (3rd) to parabola
forming a smooth curve which is
first flow line

$$q = K i A$$

$$= K \left(\frac{dz}{ds} \right) (z - 1)$$

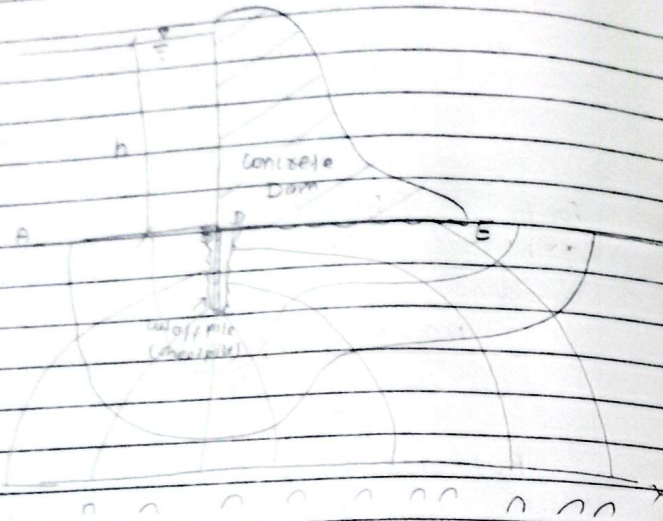
$$= K \frac{s}{\sqrt{s^2 + 2as}} \times \sqrt{s^2 + 2as} \times 1$$

$$[q = Ks]$$

Filter at → perforated pipes are kept

designed acc. to discharge

3 Concrete Dam

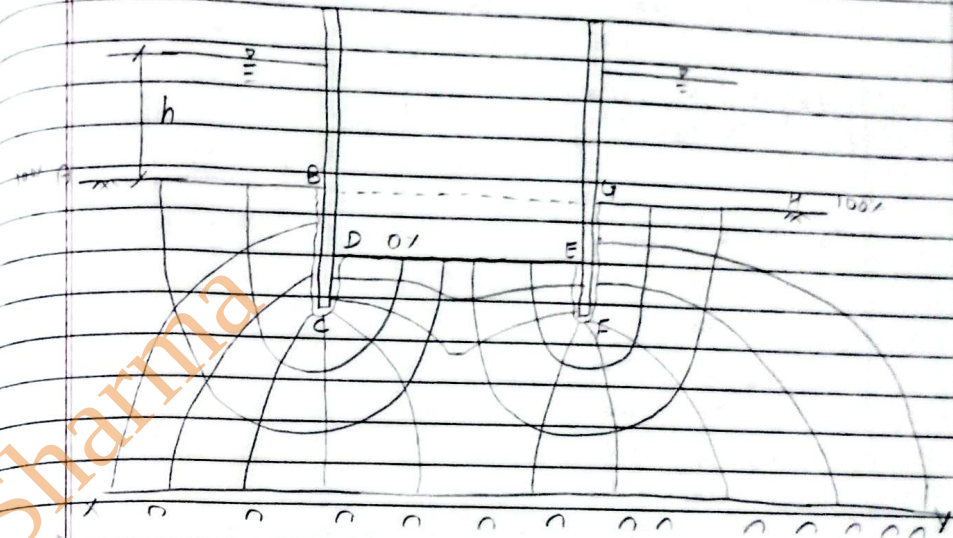


First FL = BCDE

100% Equipotential line = AB

0% Equipotential line = EF

4. Trench (cutting area) of 22 ft x 22 ft



First FL = BCD/GFE

100% Equipotential line = AB/GH

0% Equipotential line = DE

Last FL = XY

Anisotropic soil

(not square flow net)

$$\nabla \left[k_x \frac{\partial^2 h}{\partial x^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0 \right] \text{ (not Laplace)} \quad \left(\frac{\partial v_x}{\partial x} + \frac{\partial v_z}{\partial z} = 0 \right) \text{ (1)}$$

$$dx_t = \sqrt{\frac{k_z}{k_x}} dz$$

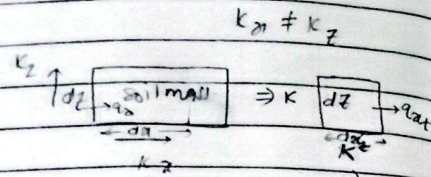
$$v_x = t_x i_x$$

$$= k_x \frac{\partial h}{\partial x}$$

$$= k_x \frac{\partial h}{\partial x_t} \times \frac{\partial x_t}{\partial x}$$

$$= k_x \frac{\partial h}{\partial x_t} \sqrt{\frac{k_z}{k_x}}$$

$$\left[v_x = \sqrt{k_x k_z} \frac{\partial h}{\partial x_t} \right]$$



Soil mass compressed st. permeability becomes equal (compressed in x-dir only) (isotropic soil mass)

$$\frac{\partial v_x}{\partial x_t} = \sqrt{k_x k_z} \frac{\partial^2 h}{\partial x_t^2} \times \frac{\partial x_t}{\partial x}$$

$$= \sqrt{k_x k_z} \frac{\partial^2 h}{\partial x_t^2} \times \sqrt{\frac{k_z}{k_x}}$$

$$\left[\frac{\partial v_x}{\partial x} = k_z \frac{\partial^2 h}{\partial x_t^2} \right]$$

→ Anisotropic soil is compressed st. it becomes isotropic w/o affecting dz
st. (k_x = k_z = k)
→ compressed from dx to dx_t

Thus, from (1),

$$k_z \frac{\partial^2 h}{\partial x_t^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0$$

$$\text{on } \left[\frac{\partial^2 h}{\partial x_t^2} + \frac{\partial^2 h}{\partial z^2} = 0 \right] \rightarrow \text{Laplace form}$$

for compressed isotropic soil, & initial soil

$$v_x = q_x = k_x \frac{\partial h}{\partial x} \quad (\Delta z = 1)$$

$$q_{x_t} = k \frac{\partial h}{\partial x_t} \quad (\Delta z = 1)$$

equating,

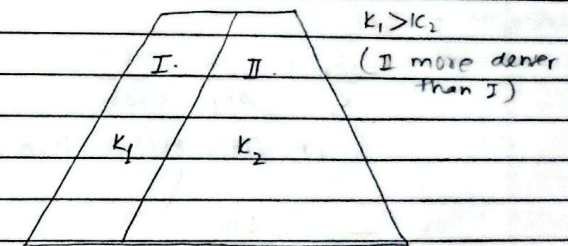
$$k_x \frac{\partial h}{\partial x} (\Delta z = 1) = k \frac{\partial h}{\partial x_t} (\Delta z = 1)$$

$$\text{or, } k = k_x \left(\frac{\partial x_t}{\partial x} \right)$$

$$k = k_x \times \sqrt{\frac{k_z}{k_x}}$$

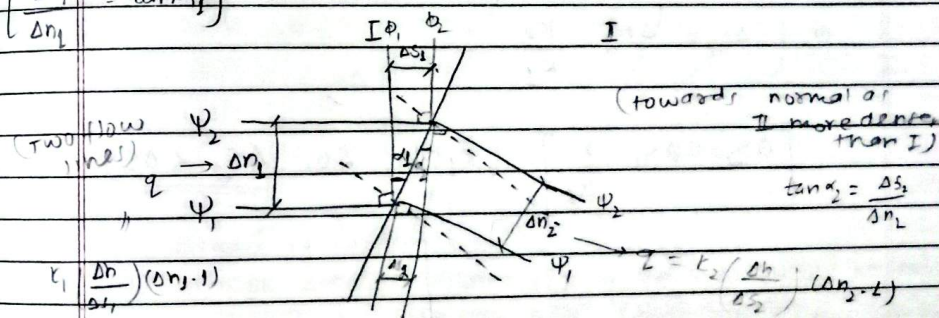
$$[k = \sqrt{k_x k_z}] \text{ ✓}$$

* For non-homogenous soil.



(compare with Refraction of light)

$$\left[\frac{\Delta s_1}{\Delta n_1} = \tan \alpha_1 \right]$$



(towards normal as II more dense than I)

$$\tan \alpha_2 = \frac{\Delta s_2}{\Delta n_2}$$

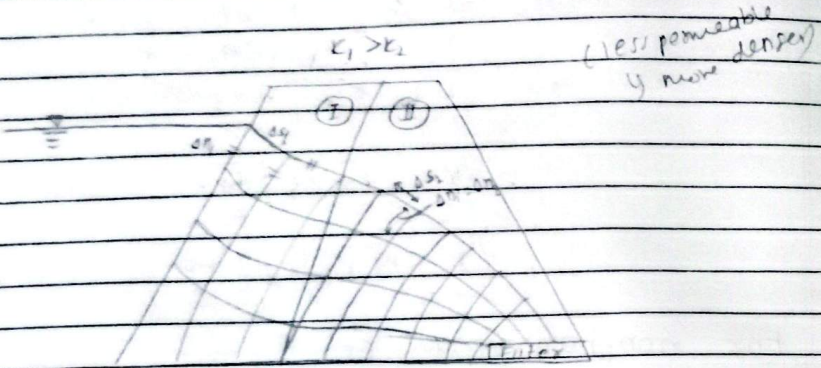
$$k_1 \left(\frac{\Delta h}{\Delta s_1} \right) (\Delta n_1 = 1)$$

$$= k_2 \left(\frac{\Delta h}{\Delta s_2} \right) (\Delta n_2 = 1)$$

Since q remains same

$$k_1 \cdot \left(\frac{\Delta h}{\Delta S_1} \right) (\Delta n_1 + 1) = k_2 \left(\frac{\Delta h}{\Delta S_2} \right) (\Delta n_2 + 1)$$

$$\frac{k_1}{k_2} = \frac{\tan \alpha_1}{\tan \alpha_2} > 1 \quad \left(\frac{k_1}{k_2} = \frac{\Delta S_1}{\Delta n_1} \cdot \frac{\Delta n_2}{\Delta S_2} \right)$$



$$\frac{k_1}{k_2} = \frac{\Delta S_1}{\Delta n_1} \cdot \frac{\Delta n_2}{\Delta S_2}$$

(As first soil is homogeneous, $\Delta n_1 = \Delta S_1$)

$$\text{or } \frac{k_1}{k_2} = \frac{\Delta n_2}{\Delta S_2}$$

$$\text{or } \Delta S_2 = \Delta n_2 \cdot \frac{k_2}{k_1}$$

($\Delta n \rightarrow$ space b/w two flow lines taken same)

$$\left[\Delta S_2 = \Delta S_1 \cdot \frac{k_2}{k_1} \right] \quad (k_1 > k_2 \text{ so, } \Delta S_2 < \Delta S_1)$$

denser / less permeable medium at
short distance travel of flow

For formula, need to travel less distance in less permeable

Failure types of hydraulic structures

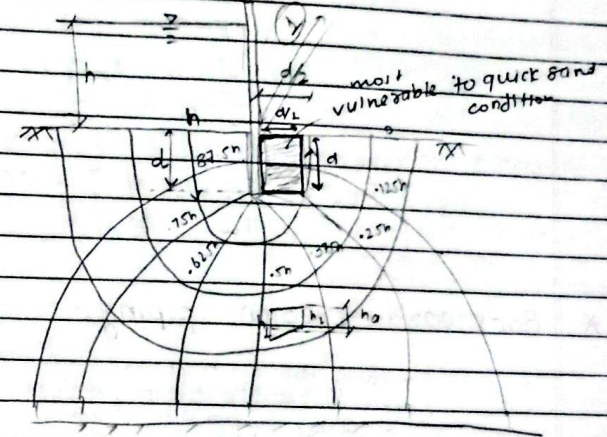
1. Heave piping

$$N_f = 4$$

$$N_d = 8$$

$$\Delta h = \frac{h}{N_d}$$

$d \rightarrow$ depth of pile below ground surface



According to Terzaghi,

If $d/2$ Downstream is safe then all structure is safe

$$\downarrow F_{\text{soil}} = \uparrow F_{\text{soil}}$$

$$\left(\frac{d}{2} \cdot d \cdot 1 \right) \times \gamma_{\text{sub}} = \gamma_w \cdot h_a \cdot \left(\frac{d}{2} \cdot 1 \right)$$

$h_a \rightarrow$ water height at upstream side of structure

$$h_a = \frac{h_1 + h_2}{2} \quad (\text{from flow net})$$

$$\text{force downward} = \frac{d}{2} \times d \times 1 \times \gamma_{\text{sub}}$$

$$F_{\text{soil}} \uparrow = \gamma_w h_a \left(\frac{d}{2} \cdot 1 \right)$$

pressure given by water on soil
check if $d/2$ part of soil is safe

$$h_a = \frac{0.75h + 0.25b}{2}$$

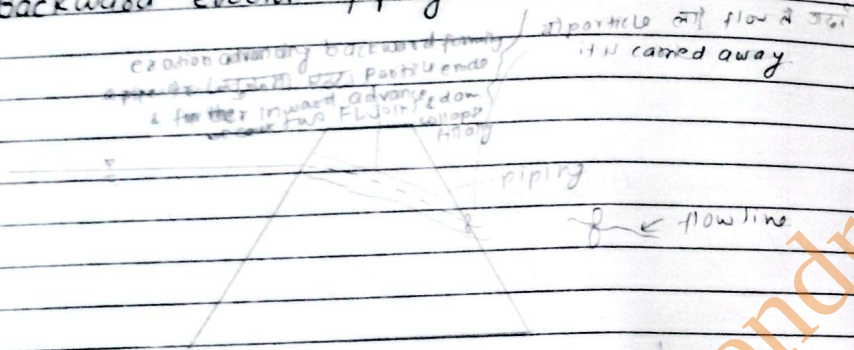
For safety, downward force > upward force

$$FDS = \frac{\text{Downward force}}{\text{Upward force}}$$

$$= \frac{\left(\frac{d}{2} \cdot d \cdot \gamma_{\text{sub}}\right)}{(\gamma_w \cdot h_a \cdot d_s)}$$

$$FDS = \frac{\gamma_{\text{sub}} \cdot d}{\gamma_w \cdot h_a} \quad \text{--- acc to norms}$$

* Backward erosion piping:



Soil particles flowing with water, then two flow lines meet increasing the flow. Thus further, larger particles flow along with water thus forming pipe like structure & erosion of soil mass occurs.

→ Concrete is always pervious
↳ crack

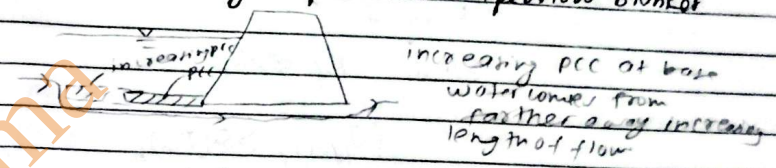
* TO avoid erosion piping.

1. Filter design
2. Increasing the length of flow
 - (a) width

(b) cutoff pipe

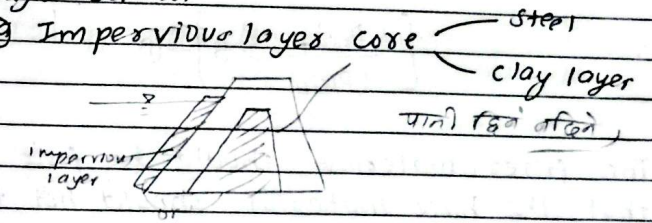
↳ for air piping prevention

(c) Increasing upstream impervious Blanket



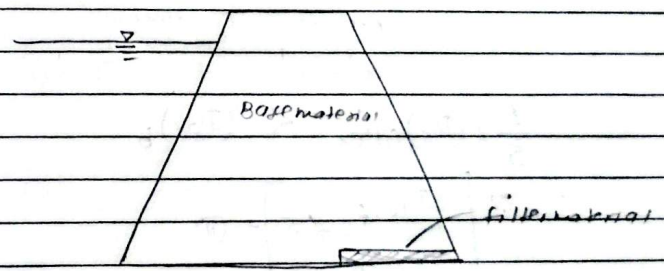
3. Seepage control

(a) Impervious layer core



* Filter design

- ↳ cheaper option
- ↳ dam in front particle / soil → Base material
- ↳ Filter → filter material



Criteria for filter.

(previous)

→ The filter material should be coarse enough such that the percolating water can easily pass without building the pore water pressure.

Thus, $K_{filter} > K_{base}$.

In General,

$$K_{filter} = 25 * K_{base}$$

Permeability $K \propto D^2$

Since $K_f \gg K_b$

$$[D_{15}]_f > 5 * [D_{15}]_b$$

$$\left[\frac{[D_{15}]_f}{[D_{15}]_b} > 5 \right] = \text{---} \text{---} \text{---}$$

① The filter materials should be fine enough such that the base material should not be washed away through the filter.

$$\text{Void size of filter} = \frac{1}{5} (D_{85})_{filter}$$

which 15% should be retained s.t.

base material will not be washed away

coarse particles strike filter and

$(D_{85})_b \rightarrow 85\%$ finer fraction 15% coarse of filter

$$\frac{1}{5} * (D_{15})_{filter} < (D_{85})_b$$

$$\Rightarrow \left[\frac{[D_{15}]_f}{[D_{85}]_b} < 5 \right] = \text{---} \text{---} \text{---}$$

Thus, from ① & ②

$$\frac{[D_{15}]_f}{[D_{85}]_b} > 5$$

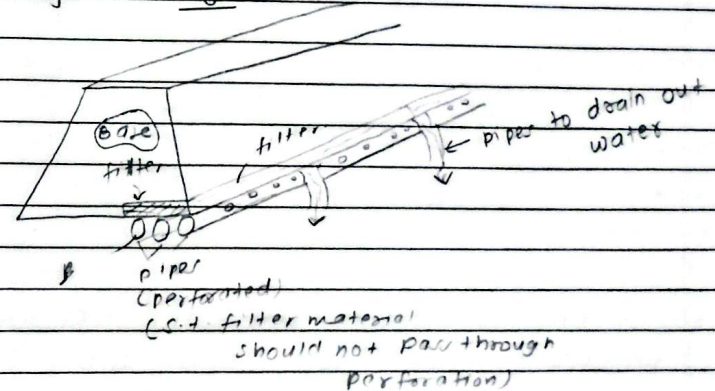
$$\frac{[D_{15}]_f}{[D_{85}]_b} > 5 \Rightarrow \frac{[D_{15}]_f}{[D_{85}]_b}$$

③ The particle size distribution curve of base and filter material should be approximately parallel.

④ The filter material should not contain more than 5% fines.

⑤ The filter material should not contain more than 75mm size particles.

⑥ Graded filter design:



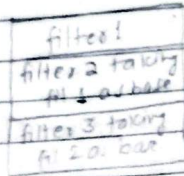
filter (soil) > base material so we do Graded filter design.

f.b.
f.w.
b.

Graded filter

If filter has perforations then layer are added. Designing new perforation layer above it is taken as base material and calculation of filter.

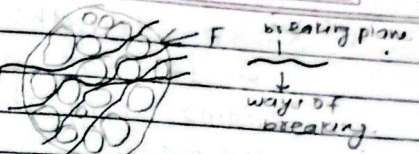
- ⑥ The last layer of Graded filter should not be washed away through the perforations of drainage pipe.



adding filter layer till last layer don't pass through perforation of drainage pipe

Shear Strength

each plane will have component of applied force



develop normal stress (horizontal)



develops shear stress (tangential)

(i) what force distribution is at failure?

(ii) at failure, what stress?

$\sigma_f = \sigma$ at failure (not $\sigma_f = \sigma$ before)

Shear stress $\rightarrow$ shear stress

But, resisting capacity of body tries to resist shear stress

But if resisting capacity $\leq$ Applied stress

failure

each plane has its own resisting capacity

plane with least resisting capacity breaks/fails first

resisting capacity $\Rightarrow$ shear strength

Applied force $\Rightarrow$ shear stress

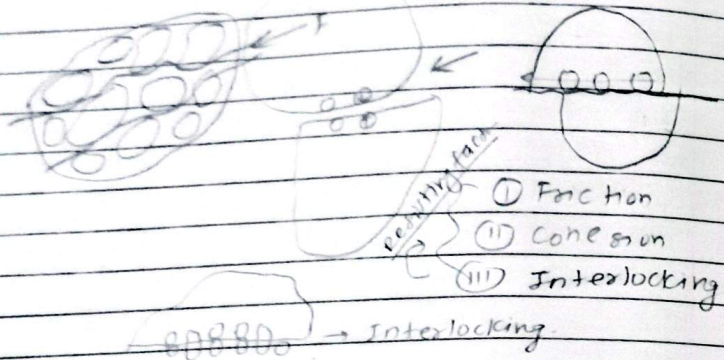
fail in plane $\Rightarrow$ weakest plane

$\rightarrow$ least resisting capacity

Amongst the infinite planes failure occurs in the plane which has least resisting capacity / shear strength. But, during failure that plane develops maximum shear stress.

• Shear strength:-

↳ It is an ability of soil to resist the sliding along the internal surface within the soil mass.



↳ It is also defined as:

Shear strength is also defined as the shear stress at the condition of failure on the failure plane.

→ Failure occurs through plane when with least resisting capacity and at the time of failure shear strength is equal to shear stress for that plane.

Thus, $\tau_{max} = \text{Shear strength}$

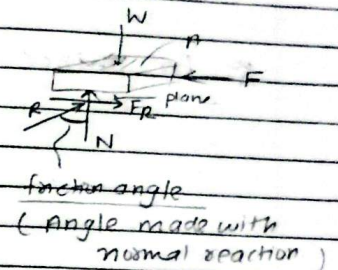
• Contributing factors for shear strength

- (i) Friction } computed in lab
- (ii) Cohesion }
- (iii) Interlocking } in terms of friction
not calculated differently

* Types of soil on the basis of shear strength

- (a) Purely cohesive soil (c -soil; $\phi = 0$ soil)
 $c \rightarrow$ cohesion, $\phi \rightarrow$ friction
- (b) cohesionless soil ($c = 0$ soil 'or' ϕ -soil)
(Frictional soil)
- (c) Cohesive soil (c - ϕ soil)

* If friction angle developed due to friction b/w similar materials then, it is Angle of internal friction (ϕ)



* For dissimilar materials, If friction angle developed due to friction b/w similar particles is called angle of wall friction (δ).
Angle of internal friction (ϕ)

* Principle plane :-

Three plane which are mutually perpendicular to each other, s.t. $\tau = 0$ & $\sigma \neq 0$

- (i) 3 mutually perpendicular to each other
- (ii) $\tau = 0$
- (iii) $\sigma \neq 0$ (only normal stress)

Three - principle planes

- (i) Major plane (Max $\approx$ normal stress)
↳ The principle plane on which max $\approx$ normal stress is acted is called Major principle plane
- (ii) Minor principle plane
↳ Plane experiencing min $\approx$ normal stress
- (iii) Intermediate principle plane
↳ has in b/w value b/w max $\approx$ & min $\approx$ normal stress

Pole : $\rightarrow$ pt. among infinite pts in Mohr's circle.
 $\rightarrow$ origin pt. of infinite number of planes.
 Also, Major PP, minor PP & failure planes originate from it.

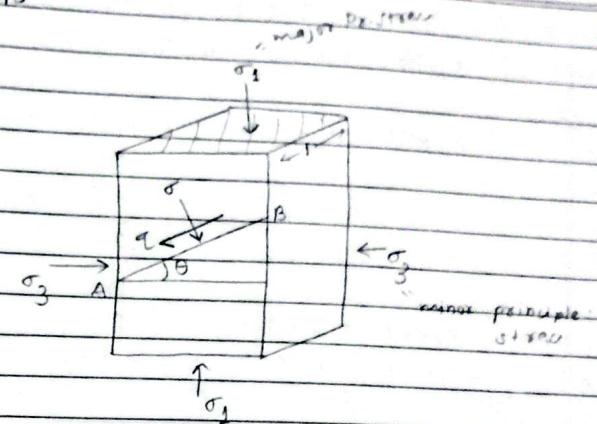
$\rightarrow$ Max^m normal stress acting on Major principle plane is called major principle stress (σ_1)

$\rightarrow$ Similarly,

For Intermediate PP $\rightarrow$ Intermediate principle stress (σ_2)

$\rightarrow$ For minor PP $\rightarrow$ Minor principle stress (σ_3)

2-D Analysis



Major principle plane = Horizontal plane

Minor principle plane = Vertical plane

Principle plane $\rightarrow$ NO. shear stress (τ)

Let us take plane AB, inclined at angle θ

with horizontal plane (Major PP)

AB $\rightarrow$ Not principal plane

So, both normal stress and shear stress.

$\Sigma F_V = 0$ ($\downarrow +ve$)

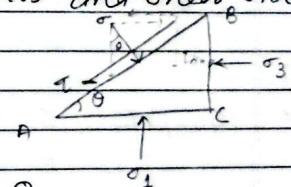
or, $-\sigma_1(AC \cdot 1) + \sigma \cos \theta (AB \cdot 1) + \tau \sin \theta (AB \cdot 1) = 0$

or, $-\sigma_1 \cos \theta + \sigma \cos \theta + \tau \sin \theta = 0 \quad \text{--- (I)}$ [Dividing by $AB, \frac{AC}{AB} = \cos \theta$]

$\Sigma F_H = 0$ ($\leftarrow +ve$)

or, $\sigma_3(BC \cdot 1) - \sigma \sin \theta (AB \cdot 1) + \tau \cos \theta (AB \cdot 1) = 0$

or, $\sigma_3 \sin \theta + \tau \cos \theta - \sigma \sin \theta = 0 \quad \text{--- (II)}$



Q11

$$\cos \theta [\sigma_3 \sin \theta + \tau \cos \theta - \sigma \sin \theta = 0]$$

$$\sin \theta [-\sigma_1 \cos \theta + \sigma \cos \theta + \tau \sin \theta = 0]$$

$$\sigma_3 \sin \theta \cos \theta - \sigma_1 \cos \theta \sin \theta + \tau \cos^2 \theta + \tau \sin^2 \theta = 0$$

$$\text{or } \left[\tau = \frac{(\sigma_1 - \sigma_3) \sin 2\theta}{2} \right] \checkmark$$

$$\sin \theta [\sigma_3 \sin \theta + \tau \cos \theta - \sigma \sin \theta = 0]$$

$$\cos \theta [-\sigma_1 \cos \theta + \sigma \cos \theta + \tau \sin \theta = 0]$$

⊕ ⊖ ⊖

$$\sigma_3 \sin^2 \theta + \sigma_1 \cos^2 \theta - \sigma \sin^2 \theta - \sigma \cos^2 \theta = 0$$

$$\text{or } \left[\sigma = \sigma_1 \cos^2 \theta + \sigma_3 \sin^2 \theta \right] \checkmark$$

$$\text{or } \sigma = \sigma_3 \left(\frac{1 - \cos 2\theta}{2} \right) + \sigma_1 \left(\frac{1 + \cos 2\theta}{2} \right)$$

$$\left[\sigma = \left(\frac{\sigma_3 + \sigma_1}{2} \right) + \left(\frac{\sigma_1 - \sigma_3}{2} \right) \cos 2\theta \right] \checkmark$$

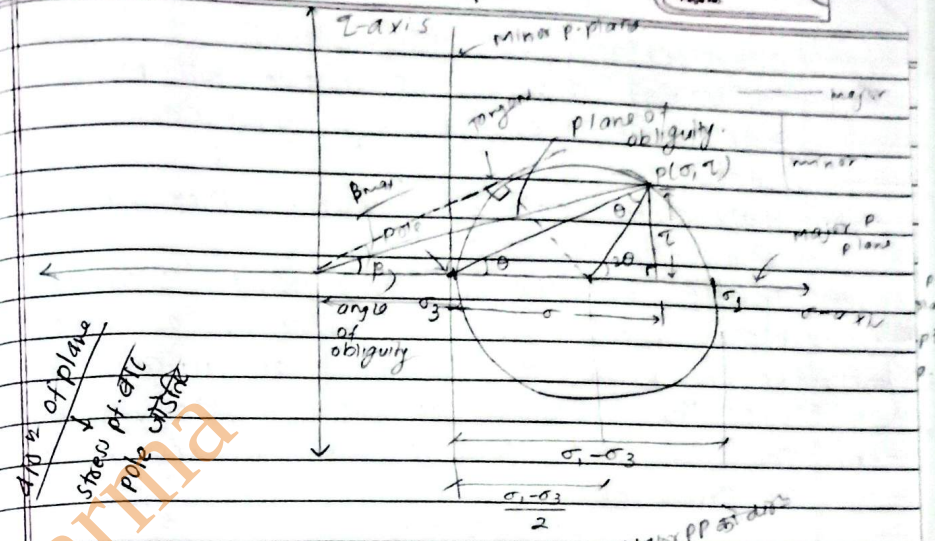
eg: ⊗ → Analytical method of finding σ and τ for given inclination θ

* Graphical Solution (Mohr's Circle)

~ principle plane ⇒ mutually ⊥
(not always ⊥ to Vt)
(may be inclined)
~ pole is among infinite points among
the circumferential points from which
 θ is measured

Identification → locate σ_1 & σ_3 (Major & minor stress)
~ pole
~ major principle stress pt etc
~ minor principle stress pt etc
~ principle plane ⇒ cut through

Mohr's circle doesn't talk about failure plane rather it talks about any plane that any plane may be failure plane.



45° of plane
stress pt etc
pole etc

$$\text{From fig } \sin 2\theta = \frac{\tau}{\left(\frac{\sigma_1 - \sigma_3}{2} \right)}$$

$$\Rightarrow \tau = \left(\frac{\sigma_1 - \sigma_3}{2} \right) \sin 2\theta$$

From fig,

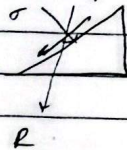
$$\sigma = \sigma_3 + \frac{\sigma_1 - \sigma_3}{2} + \left(\frac{\sigma_1 - \sigma_3}{2} \right) \cos 2\theta$$

$$\left[\sigma = \left(\frac{\sigma_1 + \sigma_3}{2} \right) + \left(\frac{\sigma_1 - \sigma_3}{2} \right) \cos 2\theta \right]$$

Characteristics of Mohr-circle:

- (i) It is not necessary to draw full Mohr's circle, half circle is enough (any upper part or lower part)
- (ii) Half-circle can be drawn either above or below the σ -axis
- (iii) The coordinate of any point 'P' on the Mohr's circle give the value of ' σ ' and ' τ ' developed on the plane ~~the~~ inclined at an angle ' θ ' with the horizontal or major principle plane.
- (iv) The resultant stress on the inclined plane can be given by

$$R = \sqrt{\tau^2 + \sigma^2}$$

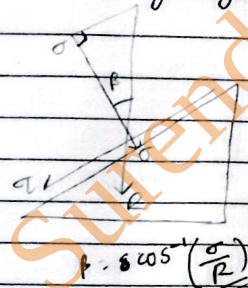


- (v) The angle ' β ' made by the line joining P with the origin is known as angle of obliquity.

where,

$$\beta = \tan^{-1} \left(\frac{\tau}{\sigma} \right)$$

Also, Angle ' β ' is made by resultant with normal stress (σ).



- (vi) ' β ' becomes maximum when line of obliquity is tangent to Mohr's circle.

From Mohr's circle,

$$\sin \beta_{\max} = \frac{\text{Radius}}{\sigma_3 + \text{Radius}} = \frac{\frac{\sigma_1 - \sigma_3}{2}}{\frac{\sigma_1 + \sigma_3}{2}}$$

$$\beta_{\max} = \sin^{-1} \left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} \right)$$

- (vi) When, $\theta = 45^\circ$, τ become max. whole value is equal to radius of Mohr's circle.

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

for, $\theta = 45^\circ$

$$\tau = \frac{\sigma_1 - \sigma_3}{2} = \tau_{\max} = \text{Radius}$$

- (vii) The value of β at the point of max^m shear stress is always less than $\beta_{\max}$.

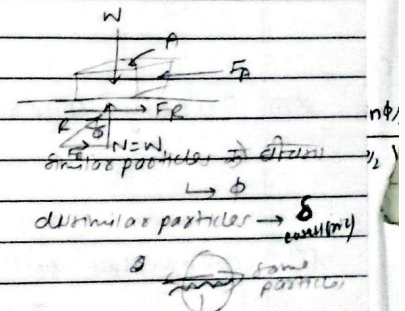
$\neq 45^\circ$

* Mohr's Failure Theory (Adduces only failure condition)

The material will fail when the shearing stresses on the plane along which it is assumed to fail is a unique function of the normal stress, acting on that plane.

$$\text{i.e. } \tau = f(\sigma)$$

Cohesion-less soil $\Rightarrow C=0$
 $\phi \neq 0$



C-phi soil

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

Mohr's Coulomb's Theory.

C=0 soil

$$\tan \phi = \frac{F}{W}$$

$$\frac{F}{A} = \frac{W}{A} \tan \phi$$

$$[\tau = \sigma \tan \phi]$$

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

cohesion ϕ shearing angle

c & $\phi \Rightarrow$ combinedly called shear parameters.

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

revised shear parameter in term of total stress

* Revised Mohr-Coulomb's Theorem

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

c' & ϕ' are called shear parameters in terms of effective stress.

* Purely cohesive soil ($\phi = 0$ & $c \neq 0$)

$$\tau = c \quad \left\{ \begin{array}{l} \tau = c' \end{array} \right.$$

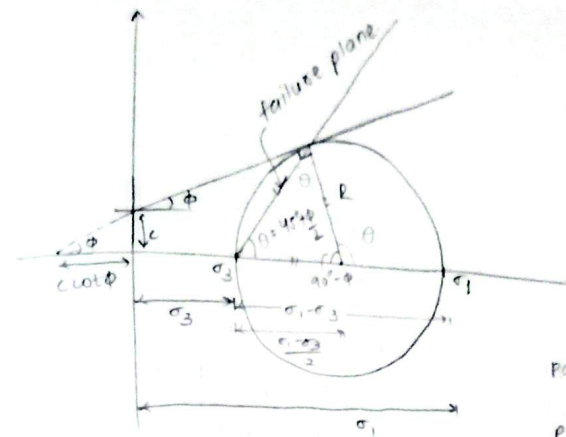
(i) cohesionless soil ($c = 0$, $\phi \neq 0$)

$$\tau = \sigma \tan \phi \quad \left\{ \begin{array}{l} \tau = \bar{\sigma} \tan \phi' \end{array} \right.$$

(ii) cohesive soil, ($c-\phi$ soil)

$$\tau = c + \sigma \tan \phi \quad \left\{ \begin{array}{l} \tau = c' + \bar{\sigma} \tan \phi' \end{array} \right.$$

$\propto$ All eqs of straight line



Pole of σ_1 major principal stress
 of τ $\rightarrow$ major principal stress
 pole to failure stress pt
 $\rightarrow$ failure plane.

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

$$\text{or, } \sin \phi = \frac{\sigma_1 - \sigma_3}{2 c \cot \phi + \sigma_1 + \sigma_3} \quad \left\{ \begin{array}{l} \text{or, } \end{array} \right.$$

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

$$\text{or, } \sigma_1 = \frac{2 c \cos \phi + \sigma_3 + \sigma_3 \sin \phi}{\sin^2 \phi + \cos^2 \phi - \sin \phi}$$

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

$$\text{or, } \sigma_1 = \left[\frac{\cos \phi/2 + \sin \phi/2}{\cos \phi/2 - \sin \phi/2} \right]^2 \sigma_3 + 2 c \frac{(\cos \phi/2 + \sin \phi/2)}{(\cos \phi/2 - \sin \phi/2)}$$

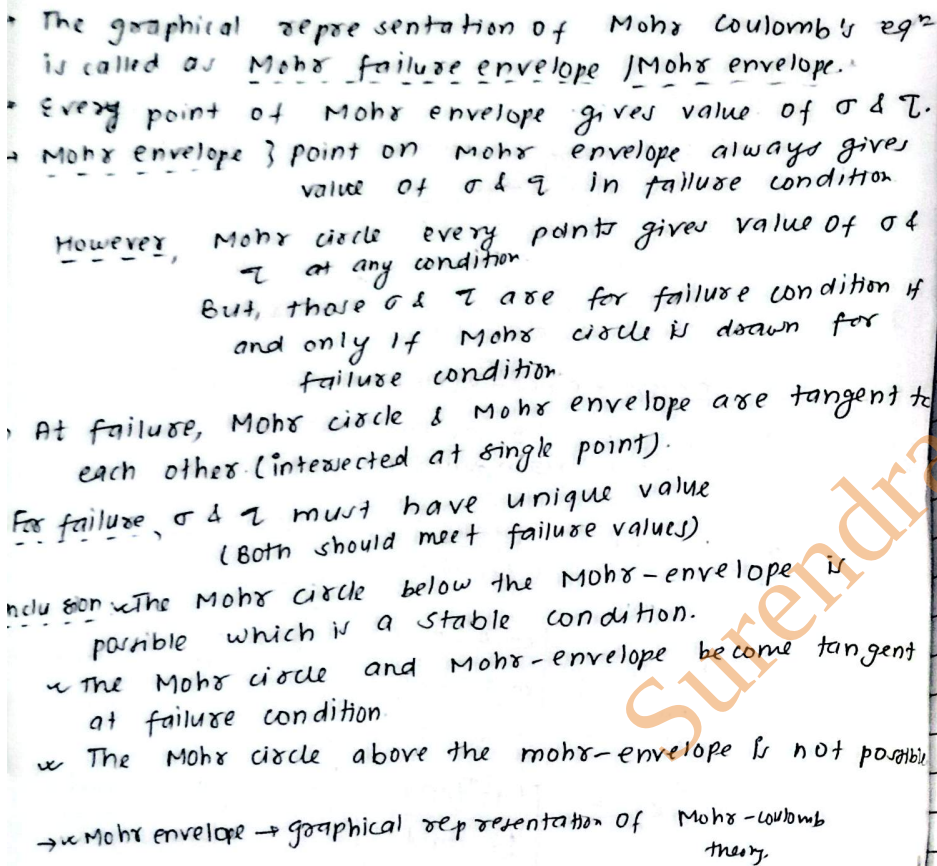
$$\text{or, } \sigma_1 = \left[\frac{1 + \tan \phi/2}{1 - \tan \phi/2} \right]^2 \sigma_3 + 2 c \frac{(\cos \phi/2 + \sin \phi/2)}{\cos \phi/2 - \sin \phi/2}$$

$$\text{or, } \left[\sigma_1 = \left[\tan(45^\circ + \phi/2) \right]^2 \sigma_3 + 2 c (\tan 45^\circ + \phi/2) \right]$$

obviously
 eqⁿ which
 gives relation
 b/w σ_1 & σ_3
 is called as
 flow ratio

$$\left[\sigma_1 = \sigma_3 N_\phi + 2 c \sqrt{N_\phi} \right]$$

$$N_\phi = \text{Flow ratio} = \tan^2(45^\circ + \phi/2)$$



1. Direct Shear Test ~
2. Triaxial Shear Test
3. Vane - Shear Test
4. Unconfined compression shear test ~

1. Consolidation stage

↳ stage at which normal stress / normal load is applied and consolidation ~~is~~ applied happens.

↳ stage at which shear stress (or shear force) is applied till the sample fails.

(No force applied)
Variable stress applied till failure
(Can be applied in two way) (i) stress controlled test
(ii) strain controlled test

1. Unconsolidated undrained test:

→ normal stress applied in first stage but consolidation not allowed.

→ normal load असाउने बिलिङ shear load असाउने
s.t. पानी सिक्कल कपाअंश

↳ quick

↳ Water is not allowed to be drained out in both consolidation and shear stage.

⇒ Drainage of water is not possible in both stage,

⇒ quick / fast stage.

$\Rightarrow$ also called U test or Φ -test

unconsolidated underlined given

2. Consolidated undrained (C-U test)

- Normal stress is applied until sample reaches its failure stage (till its capacity)
- Consolidation is allowed completely (to 1st stage) slow
- Then shear stress is allowed till failure (quick) there no draining out of water

↳ R-test

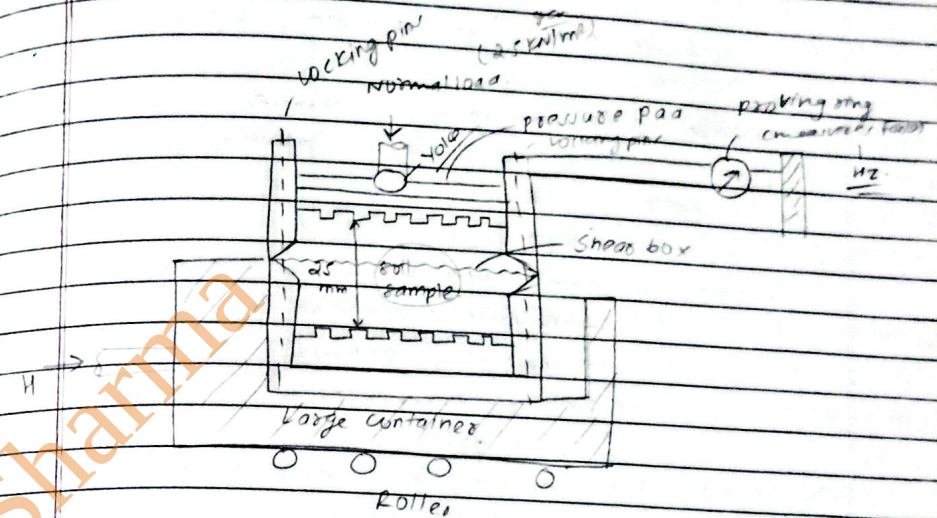
3. Consolidated drained (C-D-Test) | S-Test

- Normal stress applied and let the sample stay for consolidation (slow)
- The shear stress is allowed till failure along with drainage (slow)
- ↳ Slow test
- Normal stress applied, consolidation allowed.
- Slow shear load (at constant rate) ^{increased} until failure, along with drainage of water

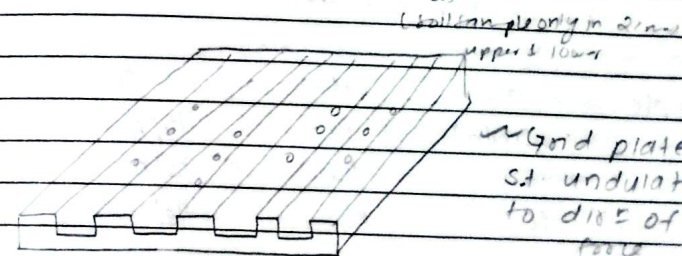
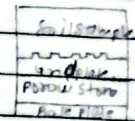
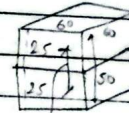
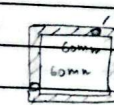
Two machines — Stress controlled
— Strain controlled

- ~ Stress measure — easy than strain
- ~ Strain difficult — use that controller
- Strain controlled

1. Direct Shear Test :-



Shear box



Grid plate

- ① Plain (if want undrained sample)
- ② Perforated (lower portion) (lower undulation)
- ↳ If want drainage

✓ Soil sample can be

undisturbed (field etc. sample)
remoulded (lab etc. sample)

After setting up sample, pin fixation

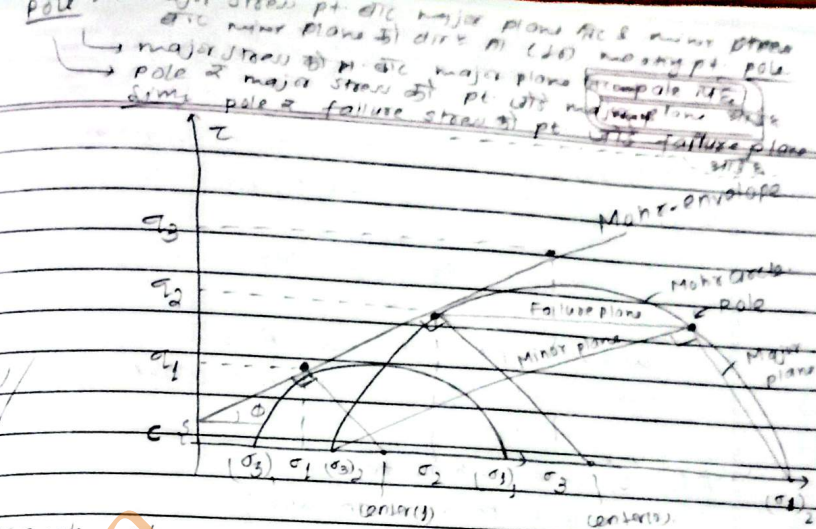
- ① Normal stress $\rightarrow 25 \text{ kN/m}^2$ applied
- ② After setting the sample, pin fixation
- ③ τ force applied
 [on the face lower portion is separated from upper portion (if shearing force developed in plane of failure is equal to applied force (H))
 and lower portion rolls along the lower roller]
 At that condition max. shear stress develops (which is equal to shear strength)

Plane of failure $\rightarrow$ Intersection plane of upper & lower portion of shear box

- ④ Further, Normal stress $\uparrow$
& next shear strength is calculated

sample	σ	H	τ
1	25 kN/m^2	H_1	$H_1/A = \tau_1$
2	50	H_2	$H_2/A = \tau_2$
3	100	H_3	$H_3/A = \tau_3$

⑤ Then plot in normal graph (τ vs σ)

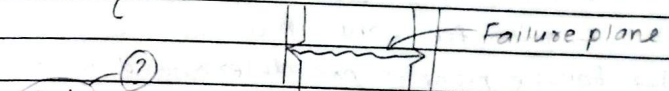


Each pt. on Mohr-envelope gives stress combination during failure.

At that condition Mohr envelope tangent to Mohr-circle

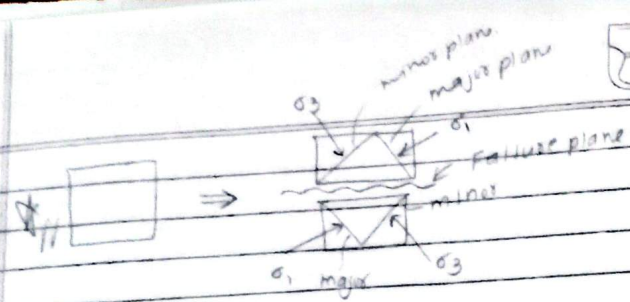
$\rightarrow$ Mohr circle two points give $(\sigma_1 \& \sigma_3)$

Failure plane is pre-determined which may not be the weakest plane $\Rightarrow$ horizontal plane



Pole $\rightarrow$ Don't know major & minor pr. plane

- $\rightarrow$ But know failure plane
- $\rightarrow$ every plane originate from pole
- $\rightarrow$ can be anywhere in circumference of Mohr-circle
- $\rightarrow$ Here, failure pt. etc failure plane $\rightarrow$ dir = σ_1 stress meet σ_3 pt. in Mohr's circle is pole



Failure pt. can't be pole.
 Principle plane $\perp$ to each other

Adv. of Direct shear test

- ↳ cheap method (economically)
- ↳ easy (sample preparation is easy)
- ↳ quick method (30 min to 1 hr)

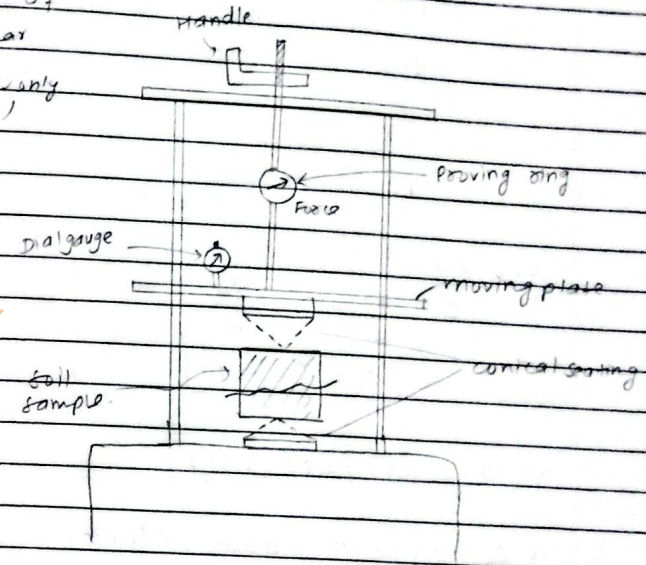
Disadv. of Direct shear test

- ↳ Pore water pressure cannot be measured for undrained test
 ~ (thus, only drained test is possible)
- ↳ test for cohesionless soil only (sand, gravel)
- ↳ Failure plane is pre-determined which may not be weakest plane.

2. Unconfined Compression Test:- (Clay)

↳ cylindrical sample

↳ Special case of triaxial shear test (where $\sigma_3 = 0$)



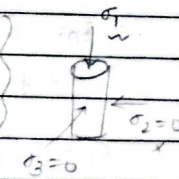
* Test done for

- ↳ cohesive soil only
- ↳ soil which can stand on its own
- ↳ Saturated clay
- ↳ even small amt of mixture of sand (cohesionless soil) gives inaccurate result.
- ↳ For $\phi = 0$ soil.

(A) $P \rightarrow$ Force at failure
 $A \rightarrow$ c/s area of soil

Soil sample

↳ only vertical stress

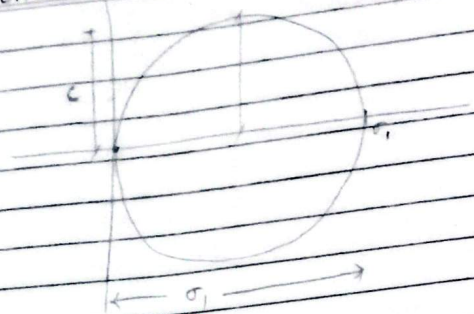


$\sigma_1 =$ Axial stress at failure

$$= \frac{P}{A}$$

= unconfined compressive strength (q_u)

Mohr's circle



From fig.

$$c = \frac{\sigma_1}{2} = \frac{q_u}{2}$$

$$[\sigma_1 = 2c = q_u]$$

✓ only for pure clay.

✓ compression causes strain area increase,

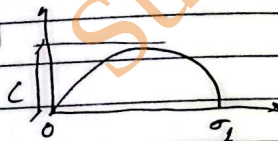
$$\text{correction in area, } A = \frac{A_0}{1 - \epsilon}$$

(volⁿ remain constant)

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

For $\sigma_3 = 0$ (No confining stress)

& $\phi = 0$ (cohesive soil)



$$\sigma_1 = 2c \tan 45^\circ$$

$$[\sigma_1 = 2c] \sim$$

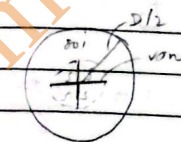
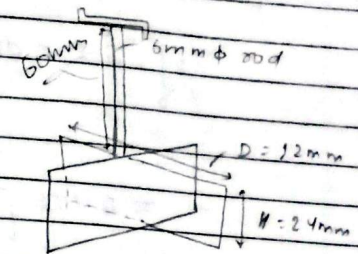
Vane - Shear test:

→ for lab as well as field

→ economic

→ cohesive soil

(which can stand on its own)

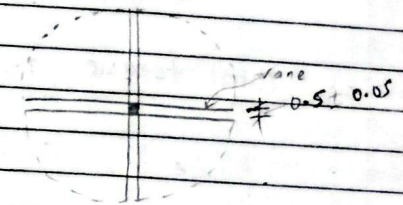


S → soil

after one rev. vane is

rotated 90°

(top; bottom & its circumference)



T = applied Torque

→ shear strength developed at failure plane (Resisting)

Along circumference,

T_f = resisting torque developed along circumference

$$= (S) \times \pi D H \cdot \frac{D}{2} = \frac{\pi D^2 H S}{2}$$

shear strength during failure

Area of contact along circumference

center to circum

(Area of moment)

T_2 = Resisting Torque developed on top & bottom planes

$$= \int_0^{D/2} S \cdot 2\pi r \cdot dr \cdot x$$

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

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

Each elemental ring develops moment differently for different moment arm

Total torque, $T = T_1 + T_2$

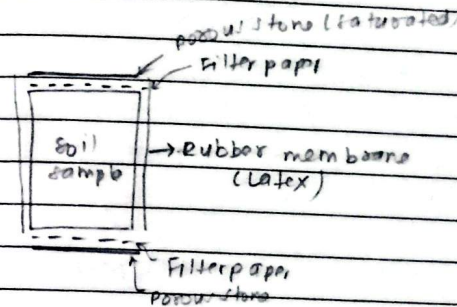
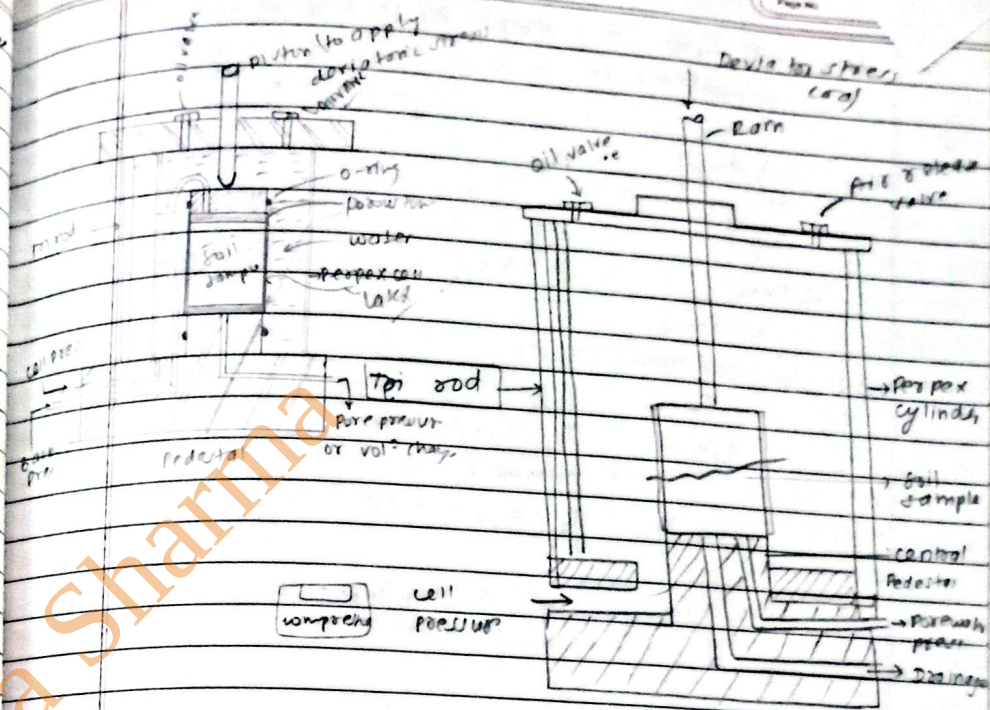
$$T = \frac{\pi D^2 H}{2} \cdot S + \frac{\pi D^3}{6} S$$

$$\Rightarrow \left[S = \frac{T}{\frac{\pi D^2 H}{2} + \frac{\pi D^3}{6}} \right]$$

Say value completely diff. then Bottom surface diff. stress

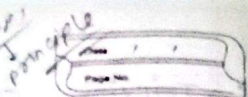
$$S = \frac{T}{\frac{\pi D^2 H}{2} + \frac{\pi D^3}{12}}$$

* Triaxial Shear Test:

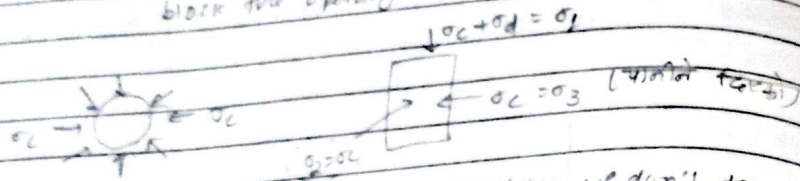


- Soil sample → take saturated (weakest state of soil)
- Inject water with the help of compressor like say 5 KN/m² cell pressure.
- water filled throughout perspex cylinder
- rubber membrane doesn't let water in perspex cylinder to be mixed to water in soil sample

No more stress
in side of
to apply
pressure

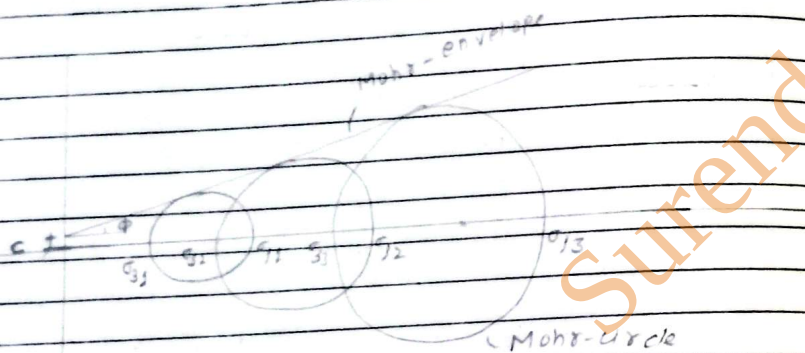


all valve? → 2 or 3 drops of oil is kept which will block the opening to prevent the leakage.



In 1-D analysis we don't take

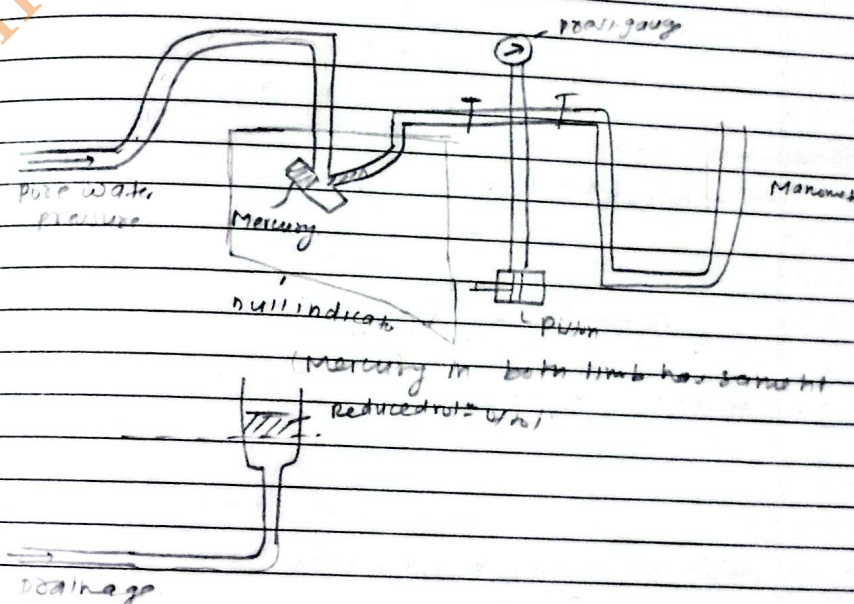
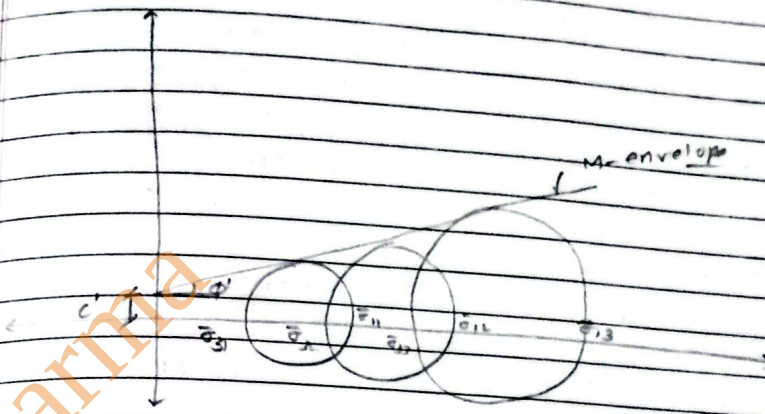
Sample No.	Cell pressure σ_c	σ_3	σ_d	σ_1 ($\sigma_c + \sigma_d$)	σ_3 $\frac{\sigma_1 - \sigma_3}{2}$
1	$\sigma_{c1} = 5$	$\sigma_{31} = \sigma_{c1} = 5$	σ_{d1}	$\sigma_{11} = \sigma_{c1} + \sigma_{d1}$	σ_{11}
2	$\sigma_{c2} = 10$	$\sigma_{32} = \sigma_{c2} = 10$	σ_{d2}	$\sigma_{12} = \sigma_{c2} + \sigma_{d2}$	$(\sigma_{12}, \sigma_{d2})$
3	$\sigma_{c3} = 20$	$\sigma_{33} = \sigma_{c3} = 20$	σ_{d3}	$\sigma_{13} = \sigma_{c3} + \sigma_{d3}$	



~ w.r.t. eff. stress c', ϕ'
~ w.r.t. total stress c, ϕ
 $\sigma = \bar{\sigma} + u$
undrained test → u

$\sigma = \sigma' + u$
If undrained test is done

→ Mohr's circle in terms of σ_3 & σ_1 is drawn
→ take load of σ above loading frame.



(Mercury in both limb has same height)
Reduced to u

Drainage conditionally
non-saturated

1) tangent

toucher certain length rather than

2) Mohr's circle

↳ difficult averaging

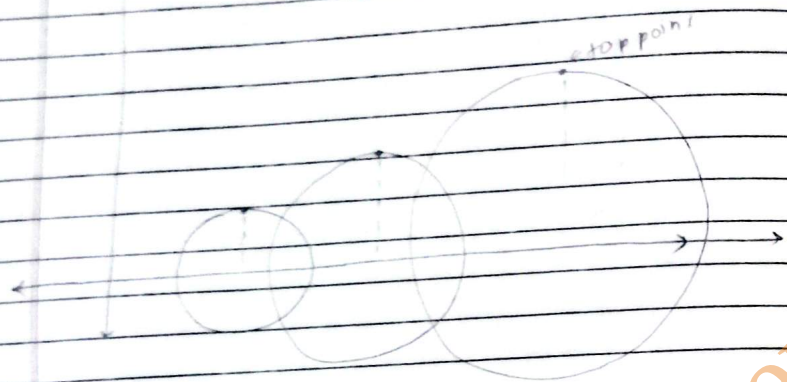
↳ we don't draw Mohr's envelope to

find c & ϕ

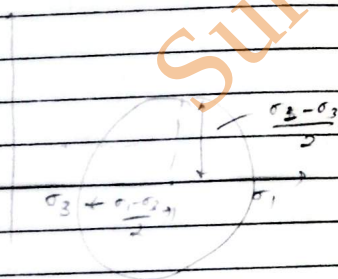
rather draw modified failure

rather draw modified failure

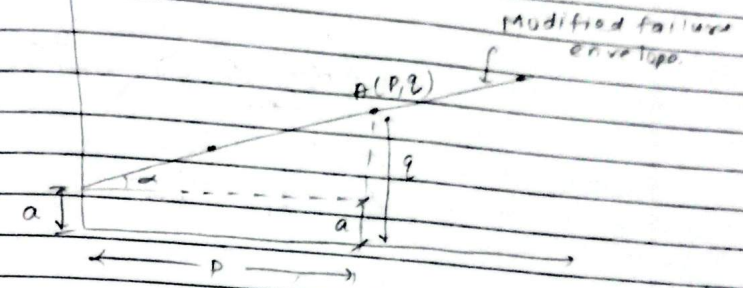
envelope



For each circle



Plotting points for every Mohr's circle



$$p = \frac{\sigma_1 + \sigma_3}{2}$$

$$q = \frac{\sigma_1 - \sigma_3}{2}$$

$$\Rightarrow q = a + p \tan \alpha$$

$$\frac{\sigma_1 - \sigma_3}{2} = a + \frac{\sigma_1 + \sigma_3}{2} \tan \alpha$$

$$\Rightarrow (\sigma_1 - \sigma_3) = 2a + (\sigma_1 + \sigma_3) \tan \alpha \quad \text{--- (i)}$$

$$\text{Also, } \sin \phi = \frac{\sigma_1 - \sigma_3}{\frac{\sigma_1 + \sigma_3}{2} + c \cot \phi}$$

$$\Rightarrow \sigma_1 - \sigma_3 = 2c \cos \phi + (\sigma_1 + \sigma_3) \sin \phi \quad \text{--- (ii)}$$

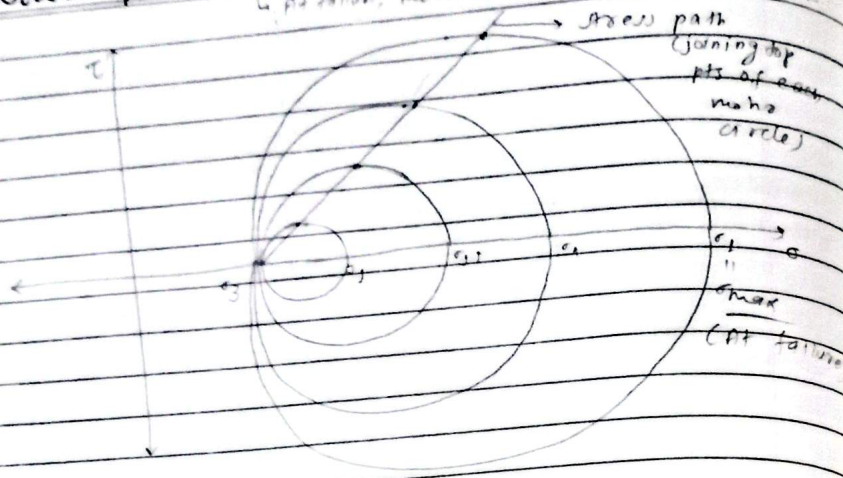
Equating (i) & (ii)

$$a = c \cos \phi$$

$$[\tan \alpha = \sin \phi \Rightarrow \frac{\phi}{\alpha} = \sin^{-1}(\tan \alpha) \omega]$$

$$[c = a \sec \phi] \omega$$

* Stress path:- Mohr's test, $\sigma_1 \uparrow \sigma_3 \uparrow$
 Mohr's circle tangents to failure envelope
 At failure, Mohr's circle tangents to failure envelope



gradual inc in $\sigma_1 \rightarrow$ gradual inc in size of Mohr's circle

At failure condition, Mohr's circle is larger

stress path is the line obtained by joining top pts of Mohr's circles

(top pts of Mohr's circle)
 $[\sigma_1 = \sigma_3 + \sigma_d]$ NOT top pts of Mohr's circle at failure

* Types of stress paths

1 Total stress path (TSP) $\rightarrow \frac{\sigma_1 + \sigma_3}{2}, \frac{\sigma_1 - \sigma_3}{2}$

2 Effective stress path (ESP) $\rightarrow \frac{\sigma_1 + \sigma_3}{2}, \frac{\sigma_1 - \sigma_3}{2}$

3 Total stress minus static pore water pressure path (TSSP) $\left(\frac{\sigma_1 + \sigma_3}{2} - U, \frac{\sigma_1 - \sigma_3}{2} \right)$

* Triaxial Test: Interpretation

Two soils

(1) Normally consolidated soil

(2) Over-consolidated soil

Typical are to consolidated
 1.5 $\rightarrow$ under consolidated

first time σ_1 soil is under stress $\rightarrow$ NC
 But σ_1 soil is under stress $\rightarrow$ NC

overconsolidated if $\sigma_1 = \text{prev. load} \Rightarrow$ Not attain
 if $< \text{prev. load} \Rightarrow$ Not attain
 Normally consolidated if $> \text{prev. load} \Rightarrow$ attain

Normally consolidated soil, $u = +ve$ (positive pore water pressure)

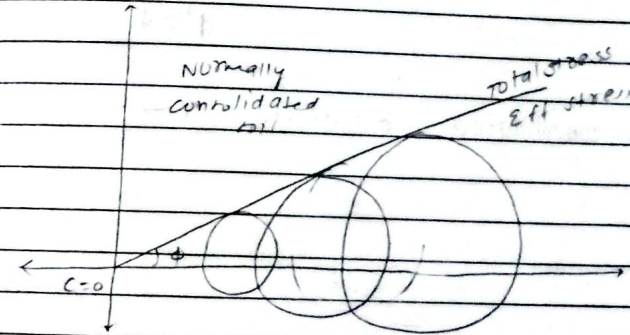
over-consolidated soil, $u = -ve$ (negative pore water pressure)

* Consolidated drained test

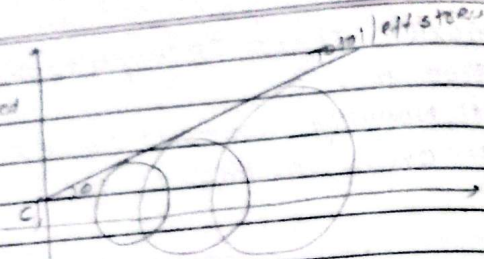
$u = 0$

$\sigma = \bar{\sigma}$

NC soil $\rightarrow$ first time, $c = 0$



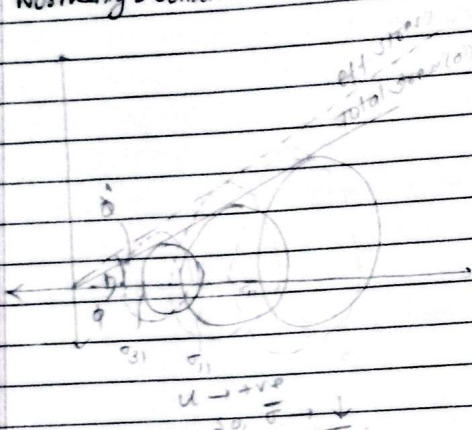
over-consolidated soil



* Consolidated undrained test:
 $u \neq 0$

Normally-consolidated soil

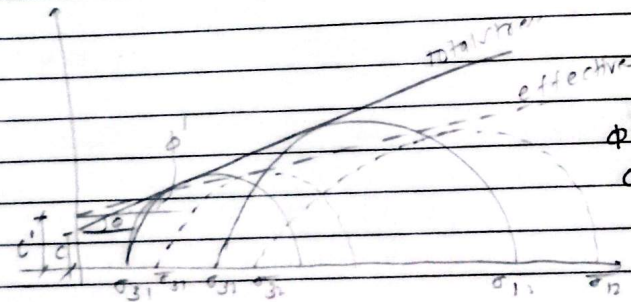
$$(\bar{\sigma} = \sigma - u)$$



- $u \rightarrow +ve$
- $u \neq 0$
- NC $\rightarrow C=0$
 $\hookrightarrow u \rightarrow +ve$ $\sigma'_3 < \sigma_3$
- OC $\rightarrow C \neq 0$
 $\hookrightarrow u \rightarrow -ve$ $\sigma'_3 > \sigma_3$

$$\phi' > \phi$$

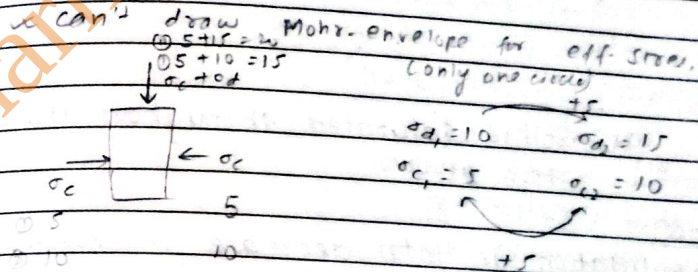
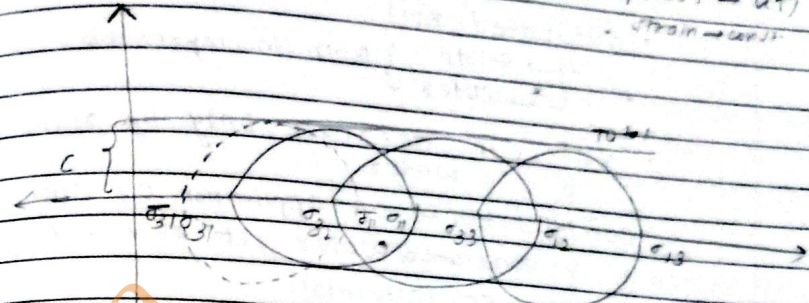
Over-consolidated, $\bar{\sigma} = \sigma - u$, $u \rightarrow -ve$



$$\left. \begin{matrix} \phi' < \phi \\ c' > c \end{matrix} \right\}$$

For UU test on saturated clay, ϕ is not observed. $\hookrightarrow$ unique effective Mohr circle obtained

* Unconsolidated undrained test
($u \neq 0$)



$$\begin{aligned} &\rightarrow \sigma_c \uparrow u \uparrow \} \sigma'_3 \& \sigma'_1 \\ &\rightarrow \sigma_c = \sigma_3 \quad \text{const.} \\ &\rightarrow \sigma'_1 = \sigma_d + \sigma_c \end{aligned}$$

CONSOLIDATION

↳ For saturated (or nearly saturated) soil :
↳ Natural process

- Saturated soil
 - ↳ Solids } Both incompressible
 - ↳ Water }
- Vol^m can decrease only by removal of water

Consolidation → on application of static load
some amt of water gets removed from soil (saturated)

- ↳ still the soil is saturated
- ↳ that doesn't mean air replacing water void rather the water void gets decreased

★ → Soil is saturated throughout the consolidation process

Hence,

Consolidation is total decrease in volume of saturated soil by removal (squeezing) out of water from water void of soil by app of static load.
↳ Volume decrease → 1-dimensionally (only by decrease of ht.) - But area remains same

* Stages of Consolidation

1st stage

applied load $\xrightarrow{\text{distributed}}$ water (P_w)
solid (P_s)

↳ Thus, solids start to move (motion) & cannot carry load & transfer the load to water

↳ Hence, load on water increases and consequently pressure on water increases

↳ Thus, total load is carried by water only

End: $P_w = P$ (Total load)
↳ max.

& max = pore water pressure developed in water

(Excessive pore water pressure)
& $P_s = 0 \rightarrow$ gains final stable position.

→ Their vol^m change in this stage is negligible

→ The decrease in the volume that takes place immediately after application of load is known as initial consolidation (1st stage)
↳ due to movement of solids

□ 2nd stage.

At start: End condition of initial consolidation stage.

$$\rightarrow U = 0$$

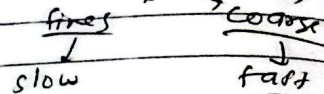
$$\rightarrow P_w = P$$

→ Water has max. pressure

→ Solid has attained final stable condition & thus, it can carry load

→ Water having max. pressure transfer load to soil gradually and water gets drained out

→ Drainage speed → depends on permeability (k)



At End, $P_s = P$ (Complete transfer of load to solid)

$$P_w = 0$$

→ NO more dec. in vol^m of water.

But still has some amt. of water in soil
→ Still has some amt. of water in soil
as no load left to drain it out

→ Vol^m change due to discharge of water is not negligible

→ The decrease in the volume due to the gradual expulsion or the removal of water through the voids of the soil is called Primary consolidation (2nd stage)

□ 3rd stage (Secondary consolidation)

→ Solid & water reaches its final state now (no more dec.) (Still, dial gauge settles)

→ As water is gradually discharged, the size of void decrease & it is assumed that solid rearrange within the newly shaped void of soil.

→ negligible dec. than that of initial stage

⇒ The consolidation that takes place even after the primary consolidation is secondary consolidation.

It is assumed that it occurs, when solid particles rearrange within the new solid. So, small change in vol^m.

Settlement: due to applied load

→ The decrease in height of soil is known as settlement

→ happens in each stage

→ Settlement that takes place during initial consolidation ⇒ immediate elastic settlement (S_i)

→ settlement that takes place during primary consolidation ⇒ Primary consolidation settlement or just settlement (S_c)

→ settlement during sec. consolidation ⇒ Secondary consolidation settlement (S_s)

① Total settlement

$$S_t = S_i + S_c + S_s$$

Settlement → Foundation eng. =

* Settlement due to natural causes.

It occurs due to

(1) Landslide

(2) Vibration/shocks

artificial (blasting)
 or natural (EQK)
 usually for sandstone (rearrange)
 (consolidation)

(3) Mines

(4) Burrowing Animal

(5) Cavities → Porosity

water dissolves limestone and makes voids
 cavities sinkhole/leaching.

(6) Thermal changes

clay hold water so thermal expansion
 swelling & shrinking

(7) Change in surrounding.

increasing in strength & density

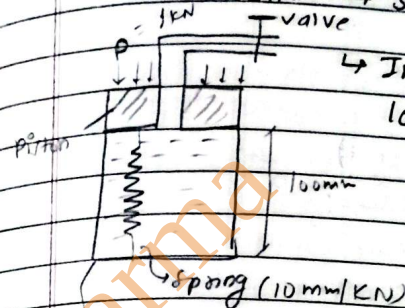
* Terzaghi's Spring Piston Analogy Experiment.

↳ For primary consolidation

Start: $P_w = P$, $P_s = 0$ → (solid in final stable position)

Pore water press → max = excessive

Solid → stable position → ext. pore development in pw



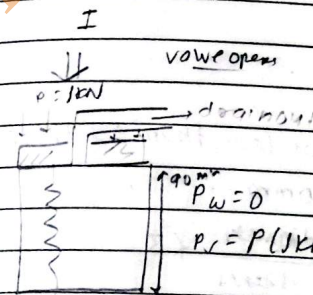
↳ Initially, valve is closed, when load is applied, load is taken by water only.

$$P_s = 0$$

$$P_w = P$$

↳ because water can't compress on spring, $P_w = P$

→ load is applied & load is taken by water only as water can't compress because valve is closed & piston can't compress. So load taken only by water.



When valve is opened,

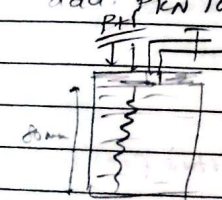
$$P_w = 0$$

$$P_s = P$$

some vol of water is drained out (because of load)

(10 mm) → water discharge

Finally, valve closed & add. 1 kN load added.

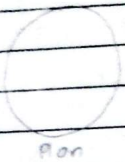
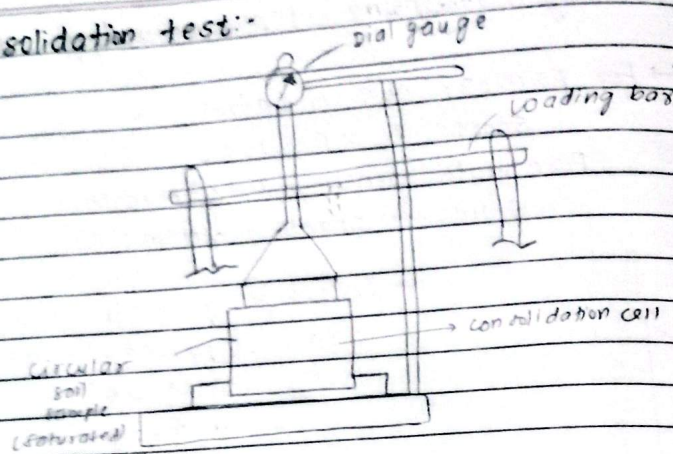


valve → permeability

In this exp

water is analogous to water
 Spring → solid
 valve → permeability

Consolidation test:-



④ thickness = not less than of the following

- ① 20mm
- ② $\frac{\text{diameter}}{\text{thickness}} \times 3$
- ③ Not less than 10 times the max. size of particle,

⑤ 2 Types of consolidation cells

- ↳ Floating / Free end cell
- ↳ Fixed end cell

floating → upper & lower porous stone free to move

fixed → one fixed (lower one) & upper porous stone to move.

→ consolidation cell has direct connection to water supply.

→ load on loading bar, reading on dial gauge.

→ Soil sample is squeezed.

→ water is drained out to container directly connected to consolidation cell

→ Soil sample is taken from the level of foundation which swells up when taken to consolidation cell. Then the setting pressure is provided to soil sample (2.5 - 5 kN/m²) and dial gauge reading is taken after 24 hrs (assuming that soil has gone to its natural state)

→ Now the load is increased by double factor (until 24 hrs)

Dial gauge reading → sq. root of $\sigma - t$ plotting
with t in hours
(1 min, 4, 9, 16, 25, 36)
49, 1, --- 24 hrs

S.N.	Applied stress (kN/m ²)
1	5
2	10

→ To start the exp. first setting load or pressure of 2.5 - 5 kN/m² is applied in order to nullify the swelling effect of soil to bring it to original condition.

→ $\sigma \rightarrow$ constant After 24 hrs same σ becomes $\bar{\sigma}$ (during exp. there is $pwp(u)$) but after that there is no $pwp(u)$.

Then after when there is change in sample/specimen it is recorded either after 24 hrs or after attaining constant value or earlier.

S.N. Applied stress (σ) Final dial gauge reading

1	5	
2	10	
3	20	
4	40	
5	80	
6	160	
7	320	
8	640	
9	1280	
10	320 (75% stress)	
11	80 (again only 25% loading kept)	
12	20	
13	0	

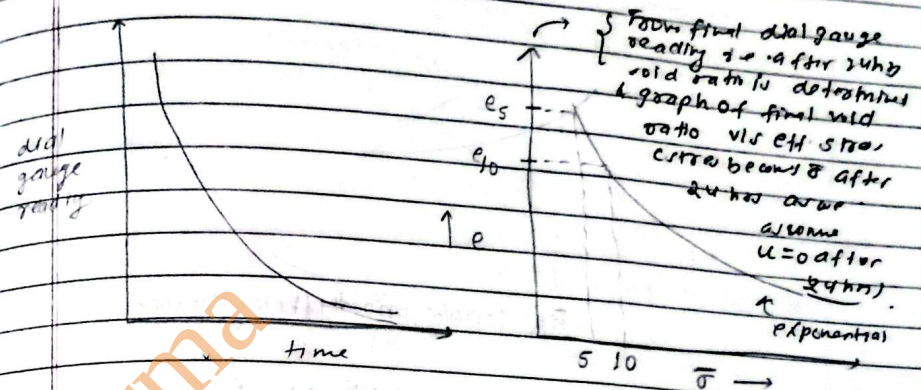
Applied stress (σ) depends upon structure
(may have to inc load for no of day)

→ Double loading until loading requirements is met

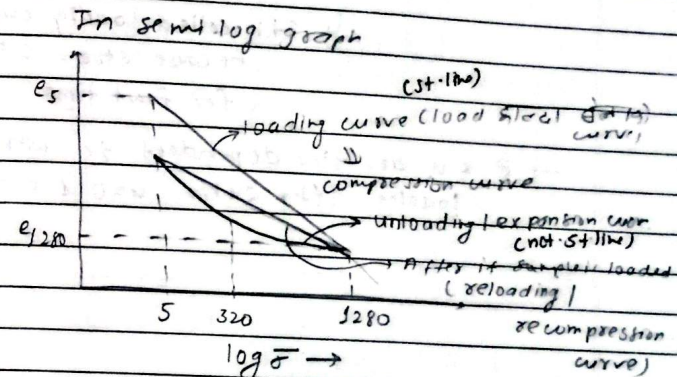
- Load increased till stress developed becomes equal or just greater than stress. It will be induced after const. of structure.
- After stress reaches peak value, load is removed by the 75% of present load till all load ceases out. The reading of unloading stage is taken after 24 hrs of unloading.

- For water content, whole mass (sample) is kept for drying (dry mass of soil sample in consolidation cell is taken by subjecting whole soil sample to final water content by oven dry).
- During unloading, water tries to come towards the soil sample developing -ve pwp. (contains soil).
- During loading, water is squeezed out of the consolidation cell (soil to container).

Plot graph for dial gauge reading vs time for each loading stress.



Normal graph



- धरें धरें → loading curve
- धरें धरें धरें → unloading curve
- धरें धरें धरें धरें → reloading
- If New house to be constructed smaller than previous ⇒ No test needed (reloading)
- But, if New house to be constructed larger than previous ⇒ Test req.

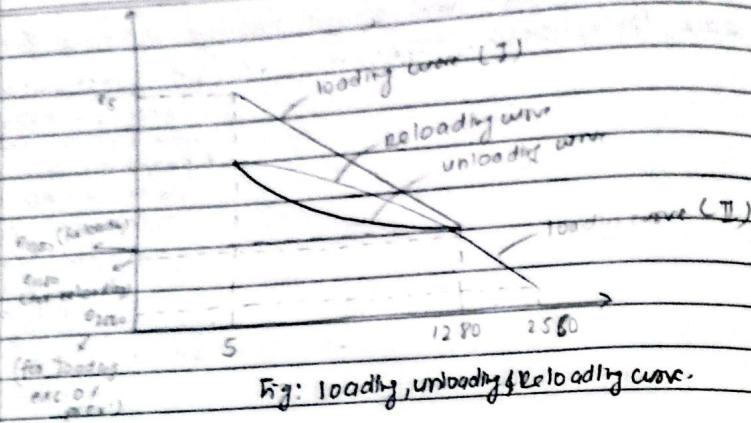


Fig: loading, unloading & reloading curve.

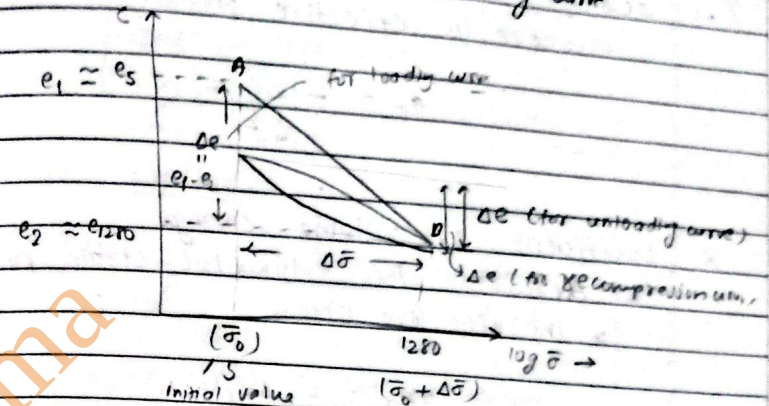
loading curve (II) → This obtained if further more than 1280 kN/m² applied.

↳ It is called loading curve not reloading because stress 2560 kN/m² is applied for first time.

→ $\bar{\sigma}$ & u are time dependent i.e. with time for a particular loading after 24 hrs $u=0$ & $\sigma = \bar{\sigma}$.

* (1) Compression Index (C_c)

↳ Slope of compression curve/loading curve



$$C_c = \frac{e_1 - e_2}{\log(\bar{\sigma}_0 + \Delta\bar{\sigma}) - \log(\bar{\sigma}_0)}$$

Initial value $\bar{\sigma}_0$ $\bar{\sigma}_0 + \Delta\bar{\sigma}$ Increment in load

$$= \frac{-\Delta e}{\log\left(1 + \frac{\Delta\bar{\sigma}}{\bar{\sigma}_0}\right)}$$

e decreases with loading

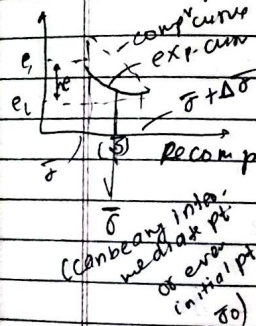
(2) Expansion index

increment in e for exp. curve

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

Recompression index (C_r)

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



* Coefficient of compressibility (a_v)
 → defined as the decrease in void ratio per unit increase in effective stress.

$$a_v = \frac{-\Delta e}{\Delta \bar{\sigma}} \quad \text{change}$$

* Coefficient of volume change
 → defined as the volumetric strain per unit increase in the effective stress

$$m_v = \frac{-\Delta V/V}{\Delta \bar{\sigma}} \quad \text{(vol. dec. as stress ↑)}$$

Relation b/w a_v & m_v

$$V = V_s + V_v \quad (e = \frac{V_v}{V_s})$$

$$V = V_s (1 + e) \quad \text{--- (I)}$$

Differentiating,

$$\Delta V = V_s \Delta e \quad \text{--- (II)}$$

Div. (II) by (I),

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

$$m_v = \frac{-\Delta V/V}{\Delta \bar{\sigma}} = (-) \frac{\Delta e / (1 + e)}{\Delta \bar{\sigma}}$$

$$\left[m_v = \frac{a_v}{1 + e} \right] \quad \left[a_v = \frac{-\Delta e}{\Delta \bar{\sigma}} \right]$$

cc. coeff. of compressibility

* Finding $e = ?$

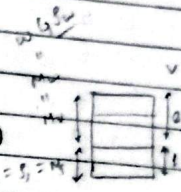
2 Methods

→ Ht. of solid method

→ Change in void ratio method.

Ht. of solid method

$$H_s = \frac{V_s}{A} = \frac{M_s / \rho_s}{A} = \frac{M_s}{\rho_s \times A}$$



H_0 → initial thickness of sample

H → thickness of sample after any time t

$\pm \Delta H$ { Unload → $H \uparrow$ (Swell) → +
 Load → $H \downarrow$ (Compression) → -

$$[H = H_0 \pm \Delta H]$$

$$e = \frac{V_v}{V_s} = \frac{V - V_s}{V_s} = \frac{H - H_s}{H_s} \quad (A \rightarrow \text{const.})$$

$$[e = \frac{(H_0 \pm \Delta H) - H_s}{H_s}]$$

ΔH → from dial gauge reading.

eg: Sample calculation
 $\sigma = \bar{\sigma}$

S-N.	Applied stress (σ)	Final dial gauge reading	ΔH (mm)	$H_0 \pm \Delta H$ (mm)	$H - H_0$	$e = \frac{H - H_0}{H_0}$
1	0	490	0	25	90.75	0.75
2	10	482	-0.08 (dec)	24.92	10.67	0.744
3	20	470	-0.12	24.80	10.55	0.74
4	40	431	-0.39	24.41	10.16	0.713
5	80	390	-0.41	24.00	9.75	
6	160	343	-0.47	23.53	9.28	
7	320	295	-0.48	23.05	8.80	
8	640	249	-0.46	22.59	8.34	
9	0	364	+1.15	23.74	9.48	0.666

L.C. of dial gauge = 0.01 mm

Given $H_0 = 25 \text{ mm}$

$A = 50 \text{ cm}^2$

$M_s = 190.24 \text{ gm}$ (final soil sample)

$W_s = 24.94 \text{ gm}$ (last soil)

→ Solⁿ

$$H_s = \frac{M_s}{G_s \cdot A} = \frac{190.24}{2.67 \times 50} = 1.425 \text{ cm}$$

$$= 14.25 \text{ mm}$$

$$e = \frac{H - H_s}{H_s}$$

② Change in void ratio method:

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

$$\Delta e = \frac{\Delta H}{H} (1+e)$$

For final sample,

$$e_{\text{last}} = \frac{W_s}{S_s} = \frac{0.2494 \times 2.67}{1}$$

$$= 0.666$$

$\Delta H \rightarrow$ diff. each time

$$\Delta e = \frac{\Delta H}{H_{\text{last}}} (1+e_{\text{last}})$$

$$\Delta e = \frac{\Delta H}{H_e} (1+e)$$

For $e = e_0$ (last value)

$$\Delta e = \frac{25 - 23.74}{23.74} (1 + 0.666)$$

$$\Delta e = \frac{\Delta H (1 + 0.666)}{23.74} = 0.0702 \Delta H$$

→ Backward calculation to obtain initial void ratio

S-N	ΔH (mm)	Δe	e
	-0.006		0.752
	-0.008		0.746
	-0.027		0.738
	-0.029		0.711
	-0.031		0.682
	-0.034		0.651
	-0.032		0.617
	+0.081		0.585
			0.666

Calculation of settlement:

(a) using m_v

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

$$m_v = -\frac{\Delta H/H}{\Delta \bar{\sigma}}$$

$$\Delta H = (-) m_v \cdot \Delta \bar{\sigma} \cdot H = S \quad \text{Settlement}$$

(b) Using change in void ratio (Δe)

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

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

$$\Delta H = \frac{\Delta e (1+e)}{1+e} (H)$$

$$\Delta e = \frac{\Delta H (1+e)}{H} \quad \text{--- (a)}$$

Case I:

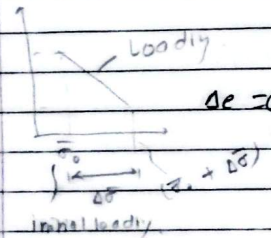
Normally consolidated soil

↳ loading curve

↳ C_c (for loading curve)

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

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



Substitute value of Δe in eqⁿ (a)

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

Case II

Over consolidated soil

↳ recompression curve (C_r)

(b) Preconsolidation pressure ($\bar{\sigma}_c$)

↳ overconsolidated soil in which max^m prev.

↳ The max^m load or prev. experienced in the OC soil is called pre-consolidation pressure.

Cases:

(a) $\bar{\sigma}_0 < (\bar{\sigma}_0 + \Delta \bar{\sigma}) < \bar{\sigma}_c$ (initial part of recompression)

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

$$S = -\Delta H = \frac{C_r H}{1+e} \log \left(\frac{\bar{\sigma}_0 + \Delta \bar{\sigma}}{\bar{\sigma}_0} \right)$$

(b) Loading at or initial

$$\bar{\sigma} < (\bar{\sigma} + \Delta \bar{\sigma}) = \bar{\sigma}_c$$

$$S = -\Delta H = \frac{C_c H}{1+e} \log \left(\frac{\bar{\sigma}_c}{\bar{\sigma}} \right)$$

(c) $(\bar{\sigma} + \Delta \bar{\sigma}) > \bar{\sigma}_c$ (greater than prev. loading)

upto prev. loading $\rightarrow C_r$
Then $\rightarrow C_c$

$$S = \frac{C_r H}{1+e} \log \left(\frac{\bar{\sigma}_c}{\bar{\sigma}} \right) + \frac{C_c H}{1+e} \log \left(\frac{\bar{\sigma} + \Delta \bar{\sigma}}{\bar{\sigma}_c} \right)$$

dia $\rightarrow 14.8 \text{ cm} / 14.9 \text{ cm}$

ht $\rightarrow 12.8 \text{ cm}$

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Trial 2 :- (18% water)

2250

depth = 1.7 cm

wt. of base with
soil = 9.1740 kg

wt. of can with
soil sample = 11.818 g
(2)

Trial 3 :- (23% water)

wt of base with
soil = 9.4340 kg

depth = $\frac{1.2 + 2}{2} = 1.6 \text{ cm}$

wt of can with
soil sample = 14.566 g
(1)

Trial 4 :- (28% water)

wt of base with soil = 9.4630 g

depth = $\frac{2.5 + 1.8}{2} =$

wt of can with
sample = 12.039 g
(1023)

wt. of soil = 3.070 kg

(Base) wt. of cone = 5.995 kg
(only base)

Trial 1:

wt. of water taken = 13% of 3 kg
= 0.39 kg
= 390 gm
= 390 ml

(1) → 40 ml

(2) → ~~500 ml~~ 570 ml

wt. of Base with ^{compacted} soil → 9.2936 kg

wt. of empty can = 4.403 g
(1023)

wt. of empty (112) = ~~6.497 g~~ 6.529 g

empty (2) = 6.318 g

empty (1) = 6.514 g

wt. of can with compacted soil sample

(112) = 18.188 g

(1023) → 10.370 g

(2) → 10.984 g

(112) → 16.677 g

(1) → 13.165 g