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SOIL MECHANICS

→ GATE : 10 – 12 marks / 100.

→ IES : → Objective : 30 – 35 que. / 120 que.
→ Conventional : 60 marks / 250.

BOOKS

→ Gopal Ranjan Rao.
→ K.R. Arora.
→ BC Punmia.

SYLLABUS

SOIL MECHANICS:

* 1. Properties of Soil. (obj + conv. + interview)
2. Classification of Soil. (obj)
3. Effective Stress (obj. + conv)
* 4. Permeability & Seepage (obj + conv + Int)
5. Stress distribution in Soil (obj)
6. Compaction (obj)
* 7. Consolidation (obj + conv. + int.)
* 8. Shear Strength of Soil "
9. Earth Pressure (obj + conv)
10. Stability of Slopes (obj)

FOUNDATION Engg.:

* 11. Shallow foundation (obj + int + conv.)
12. Pile foundation (obj + conv)
13. Soil stabalization & Exploration (obj)

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CHAPTER-1 PROPERTIES OF SOIL.

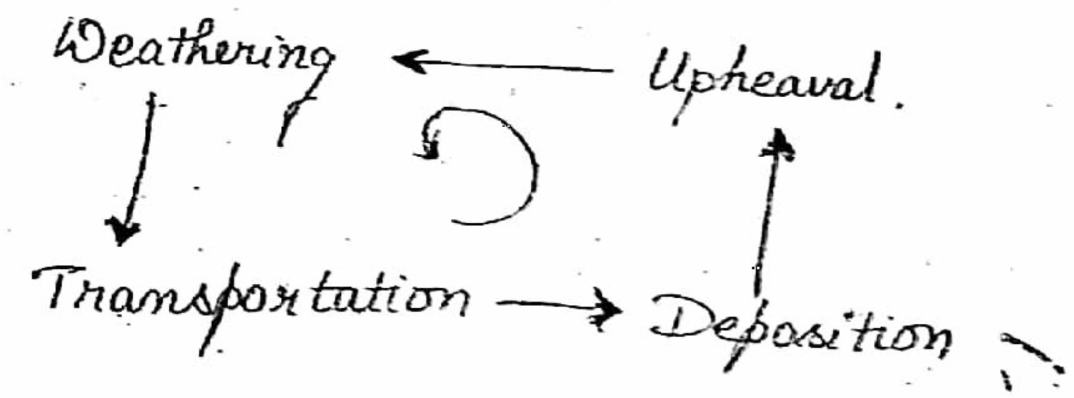
* Karl Terzaghi is the father of soil mechanics, his first book on soil mechanics is "Eardbaumechanic" publised in 1925.

* The process of formation of soil is termed as "Pedelogenesis".

* The soil is formed due to weathering of rocks, which may be carried out either ~~pshy~~ physically or chemically. Physical agency involved in weathering of rocks includes (running water, ice, wind, gravity etc) and chemical agencies involved in weathering includes (oxidation, reduction, carbonation etc)

* If the weathered rock material is retained over parent rock then it is termed as residual soil & if it is transported then it is termed as transported soil.

* Geological steps involved in the formation of soil are :—



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TYPE OF SOIL

1.) Alluvial Soil :- Deposited from suspension in running water (weathering agent is running water, transportation soil)

This type of soil is found along the length of river.

2) Lacustrine Soil :- Formed due to deposition from suspension in fresh still water from lake.

3) Marine Soil :- Deposition from suspension in sea water.

4) Aeoline Soil (Sand-Dunes) :- It is the soil which is transported by wind.

5) Loess Soil :- It is uniformly graded wind blown silt, slightly cemented due to Calcium Compound or montmorillonite (a clay mineral).

When it is wet it becomes soft and compressible because cementing action is lost and is collapsed.

Colluvial (Talus) Soil :- It is formed due to transportation by gravitational force, It is found in mountain Valleys.

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7) Glacial Soil :- It is soil which is transported by ice.

8) Marl Soil :- It is fine graded Calcium Carbonate Soil (due to animal bones) of marine origin, which is formed due to decomposition of animal bones & acquatic plants.

9) Bentonite Soil :- It is chemically weathered volcanic ash.

- Generally used as lubricant in drilling operation
- It is also clay containing a high amount of montmorillonite.
- Highly plastic and have high swelling & shrinkage properties.

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10) Black Cotton Soil :- It is residual formed from Basalt, containing a high amount of clay mineral montmorillonite.

- It is dark in colour & suitable for growing cotton.
- It has high plasticity, high swelling & shrinkage & low shear strength.

11) Laterite Soil :- It is a type of soil formed due to leaching (washing out silicon compound) & accumulation of iron oxide and aluminium oxide)

- Generally found in hilly areas, having humid climate (Western ghats & Eastern Ghats)

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12) Muck Soil :- It is mixture of in organic soil & black decomposed organic matter

13) Peat Soil :- It is highlty organic soil which almost entirely consist of vegetable matters in different stage of decomposition.

- Its colour is very strong black to dark brown and it poses organic odour.
- ~~Its colo~~ Its highly compressible soil.

Note :- Peat & Muck soil are also termed as Cumulose Soil.

14) Loam Soil :- It is mixture of clay, silt and sand.

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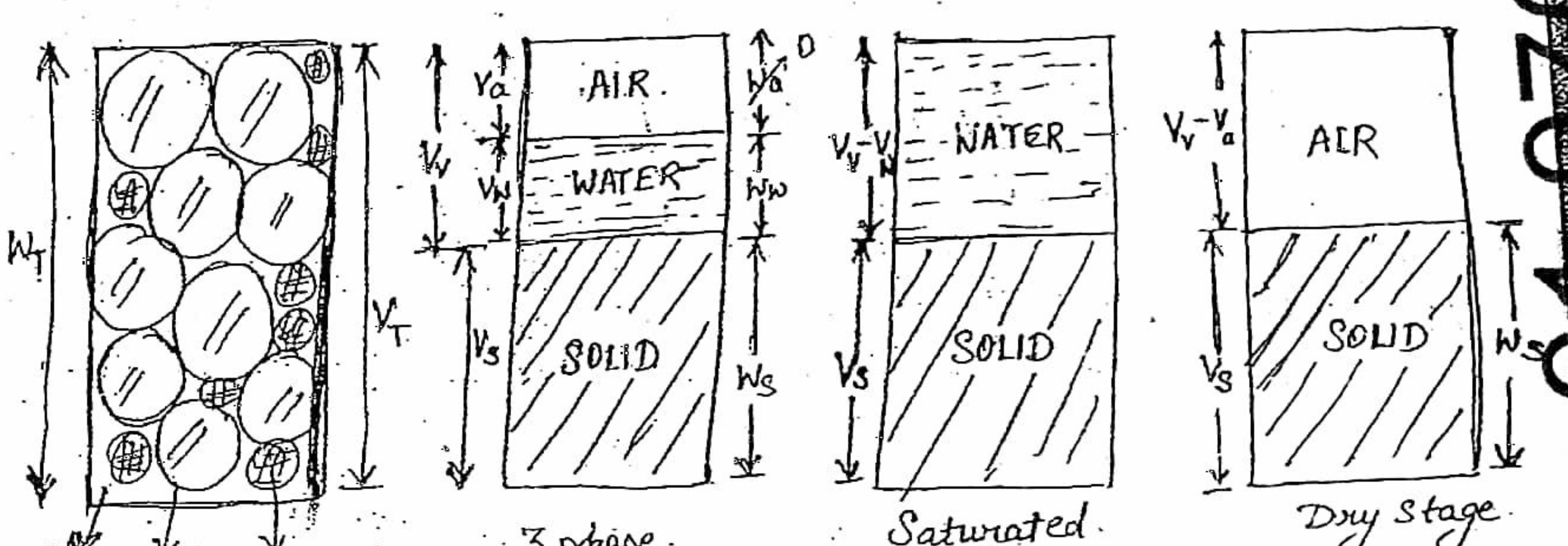
PROPERTIES OF SOIL

V_T = Total volume of soil.

V_v = vol. of voids which is never zero.

$W_a = 0$ (because of wt of air is very less)

V_s = vol. of ~~soilds~~ solids.



3 phase Diagram

Saturated stage 2 phase diagram

Dry stage 2 phase Diagram

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- A soil mass is 3-phase system that may consist of solid, water & air which do not occupy separate spaces but are mixed with each other in definite proportions which in turn determines the properties of soil.

1) Water Content / Moisture Content (w) :-

→ It is defined as ratio of weight of water to the wt of solids present in the soil mass.

$$\boxed{w = \frac{\text{wt. of water}}{\text{wt of solid}} = \frac{W_w}{W_s} \times 100 = \frac{M_w}{M_s} \times 100}$$

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Note 1 : $w \geq 0$

Note 2 : $\boxed{w = \frac{W_w}{W_s}}$

(Adding 1 to both sides)

$$1 + w = \frac{W_w}{W_s} + 1$$

$$1 + w = \frac{W_w + W_s}{W_s}$$

$$1 + w = \frac{W_T}{W_s} \qquad \because W_T = W_w + W_c$$

$$\boxed{W_s = \frac{W_T}{1+w}} \;**$$

Note 3 : Water Content can also be expressed as part of total wt of soil...

$$w' = \frac{\text{wt. of water}}{\text{Total of wt. of soil}} \times 100$$

$$w' = \frac{W_w}{W_T} \times 100 = \frac{M_w}{M_T} \times 100$$

Range of $w' = \underline{0 \leq w' < 100}$ [if $w' = 100$, then $W_w = W_T$ which is not possible]

→ Relation b/w w & w'

$$w' = \frac{W_w}{W_T} = \frac{W_w}{W_w + W_s} \qquad (\because W_T = W_w + W_s)$$

$$w' = \frac{W_w}{W_w\left[1 + \frac{W_s}{W_w}\right]} = \frac{\cancel{W_w}}{\cancel{W_w}\left(1 + \frac{1}{w}\right)} \qquad \because \frac{W_w}{W_s} = w$$

same in all cases.

$$w' = \frac{w}{1+w}$$

$$\boxed{w = \frac{w'}{1 - w'}}$$

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* wt of solids is stable quantity in comparision to total wt. of soil because it does not changes with change in wt of water, hence, engrr. significance (w) is more than (w')

2.) Void Ratio (e) :—

It is defined as the ratio of the ~~voids~~ vol. of voids to the vol. of solids present in the soil mass.

$$\boxed{e = \frac{V_v}{V_s}} \;**$$

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Note 1 :- $\boxed{e > 0}$ $\quad \because e \neq 0 \quad V_v \neq 0$ (never)

Vol. of voids or any other medium could be zero but not for the soil. Thus, void ratio of soil will never be equals to zero. Generally, void ratio is expressed as decimal fraction.

Eg :- 0.4, 0.8, 1.4 etc.

Note 2 :- $e = \frac{V_v}{V_s}$

Adding 1 on both side

$$1 + e = \frac{V_v}{V_s} + 1$$

$$1 + e = \frac{V_v + V_s}{V_s} = \frac{V_T}{V_s} \qquad (\because V_T = V_v + V_s)$$

$$\boxed{V_s = \frac{V_T}{1+e}} \;**$$

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Note 3 :- Though the size of individual void is more in coarse grain soil but the total vol^n of voids in fine grain soil is more than that of coarse grain soil due to more no. of voids.

3) Porosity (η) :

It is defined as ratio of vol^n of voids to the total vol^n of soil of given soil mass.

$$\boxed{\eta = \frac{V_v}{V_T} \times 100}$$

Note 1 $\boxed{0 < \eta < 100}$

$\eta \neq 0$ $\quad$ $\eta \neq 100$ {because V_v will not be equal to V_T}

Note 2 : Relation b/w e & η

$$\eta = \frac{V_v}{V_T}$$

$$\Rightarrow \eta = \frac{V_v}{V_v + V_s} = \frac{\cancel{V_v}}{\cancel{V_v}\left(1 + \frac{V_s}{V_v}\right)}$$

$$\Rightarrow \eta = \frac{1}{\left(1 + \frac{1}{e}\right)}$$

$$\Rightarrow \boxed{\eta = \frac{e}{1+e}} \;**$$

$$\Rightarrow \boxed{e = \frac{\eta}{1-\eta}} \;**$$

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Note 3 :- Both void ratio and porosity represents vol. of voids in given soil mass but void ratio is more significant because it represent vol^n of voids to vol^n of solids ratio & vol^n of solids is more stable than that of the total vol^n of soil.

4) Degree of Saturation (S) :-

It is defined as ratio of volume of water to the vol^n of voids present in the soil mass.

$$\boxed{S = \frac{V_w}{V_v} \times 100} \;**$$

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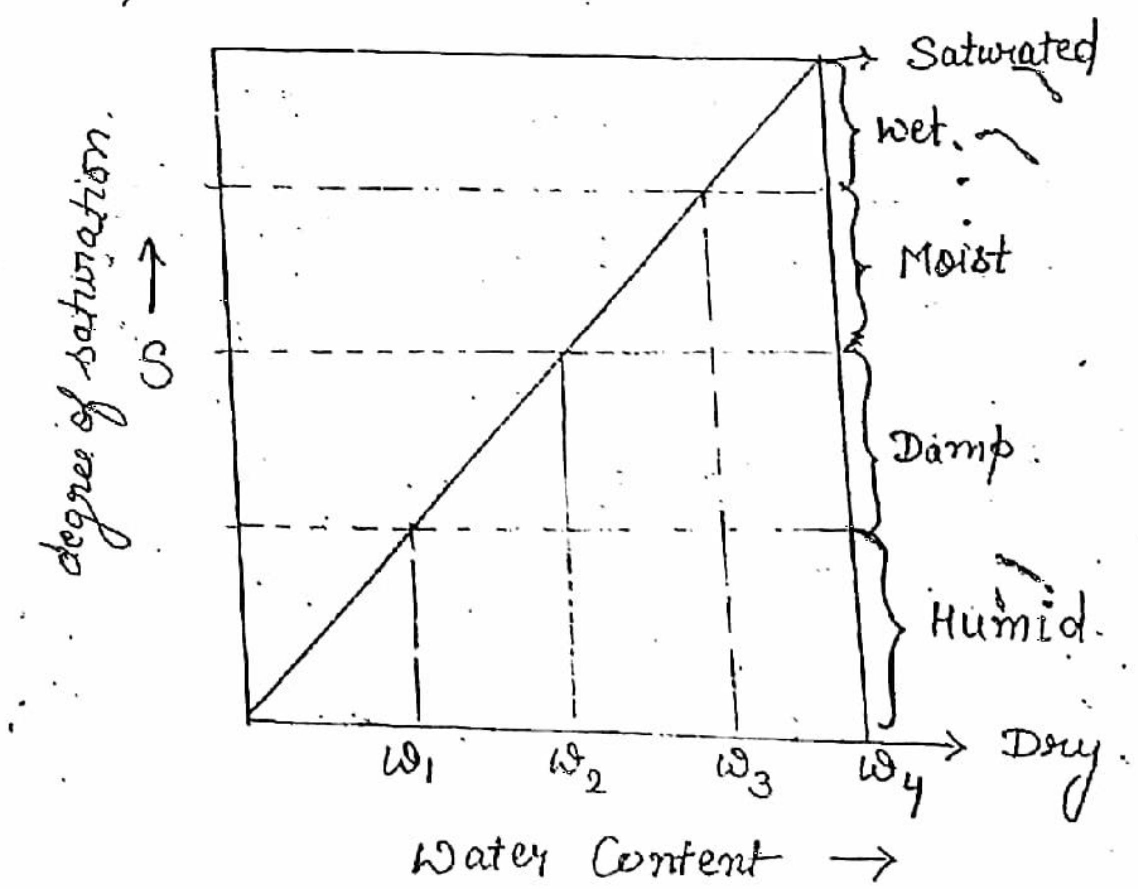
Note 1 :- $\boxed{0 \leq S \leq 100\%}$

Note 2 :- Depending upon degree of saturation of soil mass it can be classified as dry, humid, damped, moist, wet, saturated



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5) Air Content (a_c)

It is defined as the ratio of vol^n of air(V_a) to the vol^n of voids(V_v) present in soil mass.

$$\boxed{a_c = \frac{V_a}{V_v} \times 100}$$

Note 1 :- $\boxed{0 \leq a_c \leq 100}$

Note 2 :- $a_c = \frac{V_a}{V_v}$

$$\Rightarrow a_c = \frac{V_v - V_w}{V_v} = 1 - \frac{V_w}{V_v} = 1 - S.$$

$$\Rightarrow \boxed{S + a_c = 1} \;**$$

6) Percentage Air Voids (η_a) :-

It is defined as the ratio of volume of air (V_a) to the total volume of soil mass (V_T) of given soil.

$$\boxed{\eta_a = \frac{V_a}{V_T} \times 100}$$

Note 1 : $\boxed{0 \leq \eta_a < 100}$ $(\because \eta_a \neq 100)$

Note 2 : $\eta_a = \frac{V_a}{V_T}$

$$\eta_a = \frac{V_a}{V_T} \times \frac{V_v}{V_v} = \frac{V_a}{V_v} \times \frac{V_v}{V_T}$$

$$\boxed{\eta_a = n \cdot a_c} \;**$$

$\left[\because \frac{V_a}{V_v} = a_c \quad \because \frac{V_v}{V_T} = n\right]$

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7) Unit Weight / Density :- (γ, ρ)

Mass $\rightarrow$ kg or gm

wt = mg = N or KN

$$\Rightarrow \rho_w = \frac{1 gm}{cc}, \; \frac{1000 kg}{m^3}$$

$$\Rightarrow (N) \quad W = mg = kg \times 9.81 \, m/s^2 = 9.81 \, N$$

$$\gamma_m = 9810 \frac{N}{m^3} = 9.81 \, KN/m^3$$

a) Bulk Unit weight / Bulk Density

(Existing Condition)

It is defined as Ratio of total wt. of soil in existing condition to the total vol^n of soil mass.

$\rightarrow$ Bulk unit weight $\gamma/\gamma_b = \frac{W_T}{V_T}$

$\rightarrow$ Bulk density $\rho/\rho_b = \frac{M_T}{V_T}$

$\Rightarrow$ Total weight $= W_T = W_s + W_w + \cancel{W_a}^{\,0}$

$\Rightarrow$ Total volume $= V_s + V_v$

b) Saturation unit Weight / Saturated density

It is defined as ratio of total weight of soil in its saturated condition to the total volume of soil mass.

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→ Saturated unit wt. = $\boxed{\gamma_{sat} = \dfrac{W_{sat}}{V_T}}$ → (Total wt. in saturated condition)

→ Saturated density = $\boxed{\rho_{sat} = \dfrac{M_{sat}}{V_T}}$ $\dfrac{kg}{m^3}$

$$W_{sat} = W_s + W_w$$

$$V_T = V_v + V_s = V_w + V_s$$

c) Dry unit weight / Dry Density :—

→ It is defined as the ratio of total weight of soil in its dry stage to the total wt. of soil.

→ Dry unit wt. = $\boxed{\gamma_d = \dfrac{W_d}{V_T} = \dfrac{W_s}{V_T}}$

→ Dry density $\boxed{\rho_d = \dfrac{M_d}{V_T} = \dfrac{M_s}{V_T}}$

Dry weight = $W_s + W_a^{\,0} = W_s$

Total volume = $V_T = V_v + V_s = V_a + V_s$.

NOTE :— → If soil is dry, its bulk unit wt. will be same as of its dry unit wt.

It means :—

→ if $S = 0$ then $\gamma_b = \gamma_d$.

→ If soil is saturated then its bulk unit wt. will be equals to the saturated unit wt.

if $S = 1$ then $\gamma_b = \gamma_{sat}$

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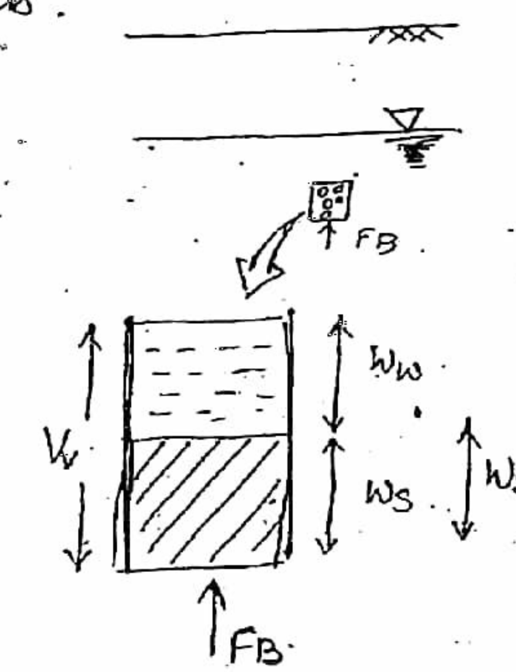
d) Submerged Unit wt / Submerged Density

When the soil mass is submerged below the ground water table, it is being acted upon force of buoyancy in vertically upward direction, magnitude of which is equals the wt. of water displaced by the soil solids, hence, it results in reduced weight of soil solids.

$$\boxed{\gamma' / \gamma_{sub} = \frac{(W_s)_{sub}}{V_T}}$$



$$W_{sat} = W_s + W_w$$

$$(W_s)_{sub} = W_{sat} - F_B$$

$$(W_s)_{sub} = W_{sat} - V_T \gamma_w$$

$$\gamma' / \gamma_{sub} = \frac{(W_s)_{sub}}{V_T} = \frac{W_{sat}}{V_T} - \frac{V_T \gamma_w}{V_T}$$

$$\boxed{\begin{aligned} \gamma' / \gamma_{sub} &= \gamma_{sat} - \gamma_w \\ \rho' / \rho_{sub} &= \rho_{sat} - \rho_w \end{aligned}}$$

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Note :- * Soil in submerged condition will be in saturated stage but soil in saturated condition need not to be in submerged Condition.

* Soil mass below the water table is submerged as well as saturated condition whereas soil mass in capillary zone is in saturated condition only.

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e) Unit weight of solid / Density of Solid

It is defined as ratio of weight of solids to the vol^n of solids present in the given soil mass.

$$\boxed{\gamma_s = \frac{W_s}{V_s}}$$ **

$$\boxed{\rho = \frac{M_s}{V_s}}$$

$V_s \neq V_T$

NOTE** :-

$$\boxed{\gamma_s > \gamma_{sat} > \gamma_{bulk} > \gamma_d > \gamma'}$$

$$\boxed{\gamma' \simeq \frac{1}{2}\gamma_{sat}}$$

8) Specific Gravity :-

(a) True / Absolute Specific Gravity :- (G/G_s)

It is defined as ratio of wt. of solids of given vol^n to the weight of standard fluid (water) of some vol^n (V_s)

OR

It is defined as ratio of unit weight of solids to the unit wt of water.

$$G/G_s = \frac{\text{wt. of solids of given volume}}{\text{wt. of water having same vol}^n}$$

$$\boxed{G = \frac{\gamma_s}{\gamma_w}}$$ **

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Note :- For inorganic soil 'G' in b/w $\boxed{2.6 \text{ to } 2.9}$ **

For organic soil 'G' is in b/w $\boxed{1 \text{ to } 2}$ **

b) Mass / Bulk / Apparent Specific Gravity (G_m)

It is defined as the ratio of wt. of soil of given vol. of the weight of water having same volume (V_T).

OR

It is defined as the ratio of bulk unit weight of soil to the unit weight of water.

$$G_m = \frac{\text{wt. of soil of given vol}^n}{\text{wt. of water having same vol}^n}$$

$$\boxed{G_m = \frac{\gamma_b}{\gamma_w}}$$ *

Note 1 :- $\boxed{G > G_m}$ $(\because \gamma_s > \gamma_b)$

Note 2 :- $\gamma_s = G\gamma_w$ = constant at any temp

$$G_{(27^\circ)}\,\gamma_{w(27^\circ)} = G_{(7^\circ C)}\,\gamma_{w(7^\circ C)}$$

$$(G)_{27^\circ} = \frac{G_{7^\circ C}(\gamma_w)_{7^\circ C}}{(\gamma_w)_{27^\circ C}}$$

→ In India specific gravity is reported at 27°C & if it is required to be computed at any other temp then corresponding change in unit wt. of water should also be considered.

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⇒ SOUL OF SOIL. (Imp)

1) $Se = wG$.

2) $\gamma_b = \dfrac{(G+Se)\gamma_w}{1+e}$

3) $\gamma_{sat} = \dfrac{(G+e)\gamma_w}{1+e}$ (S=1)

4) $\gamma' = \dfrac{(G-1)\gamma_w}{1+e}$

5) $\gamma_d = \dfrac{G\gamma_w}{1+e}$

6) $\gamma_d = \dfrac{\gamma_b}{1+w}$

7) $\gamma_d = \dfrac{(1-n_a)G\gamma_w}{1+wG}$

8) $e = \dfrac{n}{1-n}$

9) $n = \dfrac{e}{1+e}$

10) $W_s = \dfrac{W_T}{1+w}$

11) $V_s = \dfrac{V_T}{1+e}$

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1) Relation b/w S, e, w, G.

$e = \dfrac{V_v}{V_s}$

$\Rightarrow e = \dfrac{V_v}{V_s} \times \dfrac{V_w}{V_w} \Rightarrow \dfrac{V_v}{V_w} \times \dfrac{V_w}{V_s}$

$\Rightarrow e = \dfrac{1}{S} \times \dfrac{V_w}{V_s}$ $\left(\because \dfrac{V_w}{V_v} = S\right)$

$\boxed{Se = \dfrac{V_w}{V_s}}$ — (A)

Also $w = \dfrac{W_w}{W_s}$ $\because \gamma_w = \dfrac{W_w}{V_w} \Rightarrow W_w = V_w \cdot \gamma_w$; $\because \gamma_s = \dfrac{W_s}{V_s} = G \cdot \gamma_w$

$\Rightarrow W_s = V_s \cdot G \cdot \gamma_w$

$\Rightarrow w = \dfrac{W_w}{W_s} = \dfrac{V_w \cdot \gamma_w}{V_s \cdot G \cdot \gamma_w}$

$wG = \dfrac{V_w}{V_s}$ — (B)

From (A) & (B)

$\boxed{Se = wG}$ — (1)

2) Relation γ/γ_b, G, S, e, γ_w.

$\gamma_b = \dfrac{W_T}{V_T} = \dfrac{W_w + W_s}{V_s(1+e)}$ $\because W_T = W_w + W_s$, $V_s = \dfrac{V_T}{1+e}$

$\gamma_b = \dfrac{V_w \gamma_w + V_s \cdot G \cdot \gamma_w}{V_s(1+e)}$

$\gamma_b = \left[\dfrac{V_w}{V_s} + \dfrac{V_s \cdot G}{V_s}\right] \dfrac{\gamma_w}{1+e}$

Now from eqⁿ A $\dfrac{V_w}{V_s} = Se$ $\Rightarrow \boxed{\gamma_b = \dfrac{(G+Se)\gamma_w}{1+e}}$

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3) γ_{sat}

if $S = 1$ then $\gamma_b = \gamma_{sat}$

$$\gamma_b = \frac{(G+Se)\gamma_w}{1+e} = \gamma_{sat}$$

$$\boxed{\gamma_{sat} = \frac{(G+e)\gamma_w}{1+e}} \quad (3)$$

4) γ'

$$\gamma' = \gamma_{sat} - \gamma_w$$

$$\gamma' = \frac{(G+e)\gamma_w}{1+e} - \gamma_w$$

$$\boxed{\gamma' = \frac{(G-1)\gamma_w}{1+e}} \quad (4)$$

5) γ_d

if $S = 0$ then $\gamma_b = \gamma_d$

$$\gamma_b = \frac{(G+Se)\gamma_w}{1+e} = \gamma_d$$

$$\boxed{\gamma_d = \frac{G\gamma_w}{1+e}} \quad (5)$$

6) Relation b/w γ_d, w, γ_b.

$$\frac{W_s}{V_T} = \gamma_d$$

$$\gamma_d = \frac{W_T}{V_T}$$

$$\gamma_b = \frac{W_s(1+w)}{V_T} \qquad \left\{ \because W_s = \frac{W_T}{1+w}, \quad W_T = W_s(1+w) \right.$$

$$\gamma_b = \gamma_d(1+w)$$

$$\boxed{\gamma_d = \frac{\gamma_b}{1+w}} \quad (6)$$

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7) Relation b/w γ_d, η_a, w, G, γ_w.

$$V_T = V_v + V_s = V_a + V_w + V_s.$$

Dividing by V_T at both the sides.

$$1 = \frac{V_a}{V_T} + \frac{V_w + V_s}{V_T}$$

$$\Rightarrow 1 = \eta_a + \frac{V_w + V_s}{V_T} \qquad \left\{ \because \eta_a = \frac{V_a}{V_T} \right\}$$

$$\Rightarrow 1 - \eta_a = \frac{V_w + V_s}{V_T}$$

$$\Rightarrow 1 - \eta_a = \left(\frac{W_w}{\gamma_w} + \frac{W_s}{G\gamma_w} \right)$$

$\left[\because \gamma_w = \frac{W_w}{V_w} \Rightarrow V_w = \frac{W_w}{\gamma_w} \right.$

$\gamma_s = \frac{W_s}{V_s} = G\gamma_w$

$\Rightarrow V_s = \frac{W_s}{G\gamma_w}$

$$= \frac{w \cdot W_s}{\gamma_w} + \frac{W_s}{G\gamma_w} \qquad \because w = \frac{W_w}{W_s} \Rightarrow W_w = w \cdot W_s$$

$$\Rightarrow 1 - \eta_a = \frac{w}{\gamma_w} \cdot \frac{W_s}{V_T} + \frac{W_s}{V_T} \cdot \frac{1}{G\gamma_w}$$

$$= \gamma_d \cdot \frac{w}{\gamma_w} + \frac{\gamma_d}{G\gamma_w}$$

$$1 - \eta_a = \frac{\gamma_d}{\gamma_w}\left(w + \frac{1}{G}\right)$$

$$\Rightarrow \boxed{\gamma_d = \frac{(1-\eta_a)G\gamma_w}{1+wG}} \quad (7)$$

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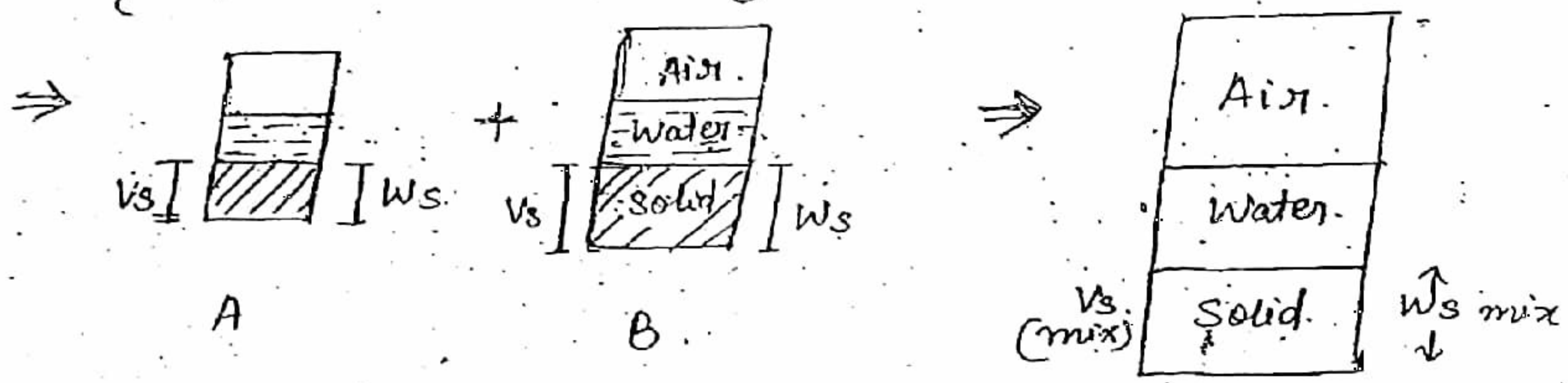
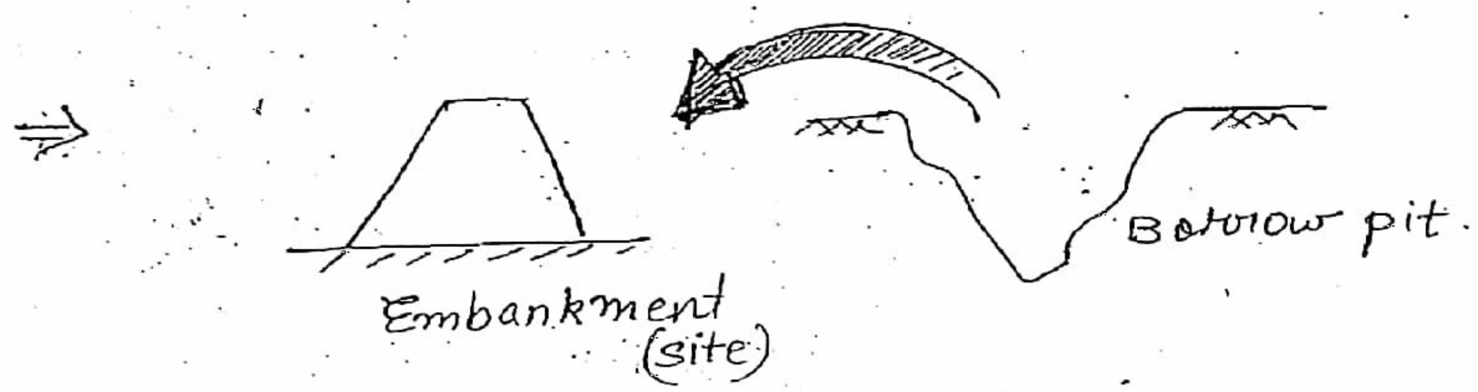
{Refer Point 10 & 11} Formula



w_1, e_1 (A) w_2, e_2 (B) w_{mix}, e_{mix} (C)

$$(W_s)_A + (W_s)_B = (W_s)_{mix}$$

$$(V_s)_A + (V_s)_B = (V_s)_{mix}$$



$$(W_s)_{site} = (W_s)_{Borrow\,pit}$$

$$(V_s)_{site} = (V_s)_{Borrow\,pit}$$

Concept Used in Numerical

Work book

2) $W_s = 28\,g$, $V_v = 10\,cm^3$, $W_w = 9\,g$, $G = 2.7$

find
1) w
2) e
3) S
4) η

(1) $w = \frac{W_w}{W_s} = \frac{9}{28} \times 100\% = 32.14\%$

(2) $e = \frac{V_v}{V_s} = \cancel{\frac{10 \times 2.7}{2.8} = 0.9}$

$$\gamma_s = \frac{W_s}{V_s} = G\gamma_w$$

$$V_s = \frac{W_s}{G\gamma_w} = \frac{28\ gm}{2.7\ gm/cc} = \frac{28}{2.7}\,cc$$

$$e = \frac{10\,cc}{\frac{28}{2.7}\,cc} = \frac{27}{28} = 0.96$$

3) $S = \frac{V_w}{V_v} \times 100 = \frac{9}{10} \times 100 = 90\%$

$\because \gamma_w = \frac{W_w}{V_w}$, $\gamma_w = 1\,gm/cc$, $V_w = \frac{9}{1} = 9\,cc$

4) $\eta = \frac{V_v}{V_T} \times 100$

$$\eta = \frac{e}{1+e} = \frac{0.96}{1.96} = 0.489$$

Q 3) $W_s = 1\,kg$ (A) w = 100%
$W_s = 1\,kg$ (B) 50%
$w_{mix} = ?$

$\Rightarrow (W_s)_A + (W_s)_B = (W_s)_{mix}$

$\Rightarrow \left(\frac{W_T}{1+w}\right)_A + \left(\frac{W_T}{1+w}\right)_B = \left(\frac{W_T}{1+w}\right)_{mix}$

$\Rightarrow \frac{1}{1+1} + \frac{1}{1+0.5} = \frac{2}{1+w}$

$\Rightarrow \frac{1}{2} + \frac{1}{1.5} = \frac{2}{1+w} \Rightarrow \frac{7}{6} = \frac{2}{1+w}$

$w = 0.71 = 71\%$

7) Sample A : $e = 1$, $V_T = 1$
Sample B : $e = 2$, $V_T = 2$
$\eta = ?$

$V_{mix} = 1\,m^3$

$(V_s)_A + (V_s)_B = (V_s)_{mix}$

$\left(\frac{V_T}{1+e}\right)_A + \left(\frac{V_T}{1+e}\right)_B = \left(\frac{V_T}{1+e}\right)_{mix}$

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$$\Rightarrow \frac{1}{1+1} + \frac{1}{1+2} = \frac{1}{1+e}$$

$$\Rightarrow \frac{1}{2} + \frac{1}{3} = \frac{1}{1+e}$$

$$\Rightarrow (1+e) = \frac{1}{0.833}$$

$$e = 0.2$$

$$\Rightarrow \eta = \frac{e}{1+e} = \frac{0.2}{1.2}$$

$$\eta = 0.167 \simeq 17\%$$

22) γ, w, G, γ_w given

$$\gamma_b = \frac{(G+eS)}{1+e}\gamma_w = \frac{(G+wG)}{1+e}\gamma_w$$

$$(1+e) = (1+w)\, G \frac{\gamma_w}{\gamma_b}$$

$$1 + \frac{wG}{S} = (1+w)\, G \cdot \frac{\gamma_w}{\gamma_b}$$

$$\frac{wG}{S} = \frac{(1+w) G\gamma_w}{\gamma_b} - 1$$

$$S = \frac{wG}{\frac{(1+w)G\gamma_w}{\gamma_b} - 1} = \frac{w}{(1+w)\frac{\gamma_w}{\gamma} - \frac{1}{G}}$$

43) $V_s = 3.3$ lakh m^3 | $e = 0.7$

$e = 1.2$ | $V = ?$

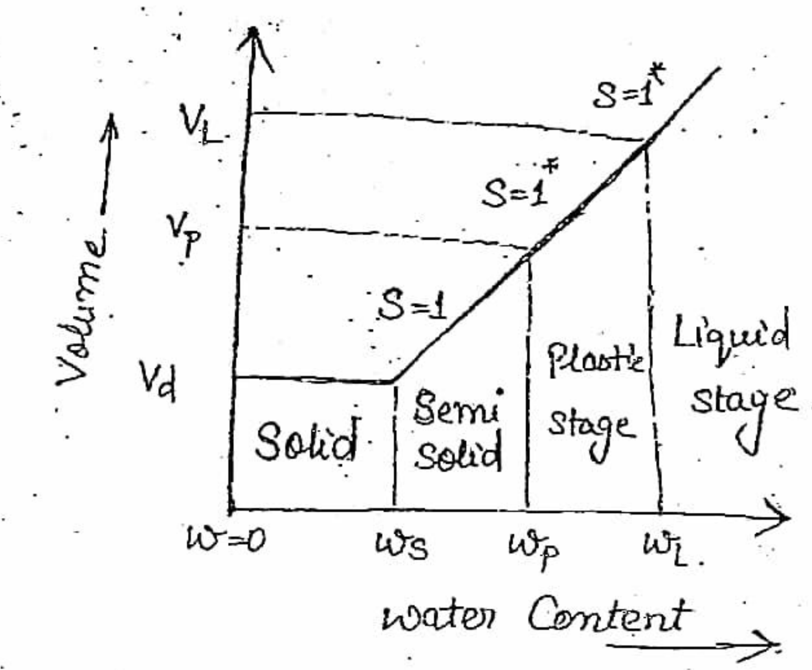
$$\Rightarrow \frac{V_T}{1+e} + \frac{V_T}{1+e} = \frac{V_T}{1+e}$$

$$\Rightarrow \frac{3.3}{1+1.2} = \frac{V_T}{1+0.7}$$

$$\Rightarrow V_T = \frac{3.3\times10^5(1+0.7)}{1+1.2} = 2.55\times10^5$$

9) Consistency of Soil

* Consistency means relative ease with which the soil can be deformed, it also denotes the degree of firmness of soil (which may be termed as soft, stiff or hard).
* It also represents the strength of soil indirectly.
* This term is used for only fine grained soil and is related to water content.



Consistency Curve

Note 1:– If soil is partially saturated $(0 < S < 1)$ upto the just saturation, any change in water content does not result in change in volⁿ of voids, volⁿ of soil, void ratio and porosity.

But if soil is supersaturated i.e $(S = 1^*)$, any change in water content leads to corresponding change in volⁿ of voids & volⁿ of soil.

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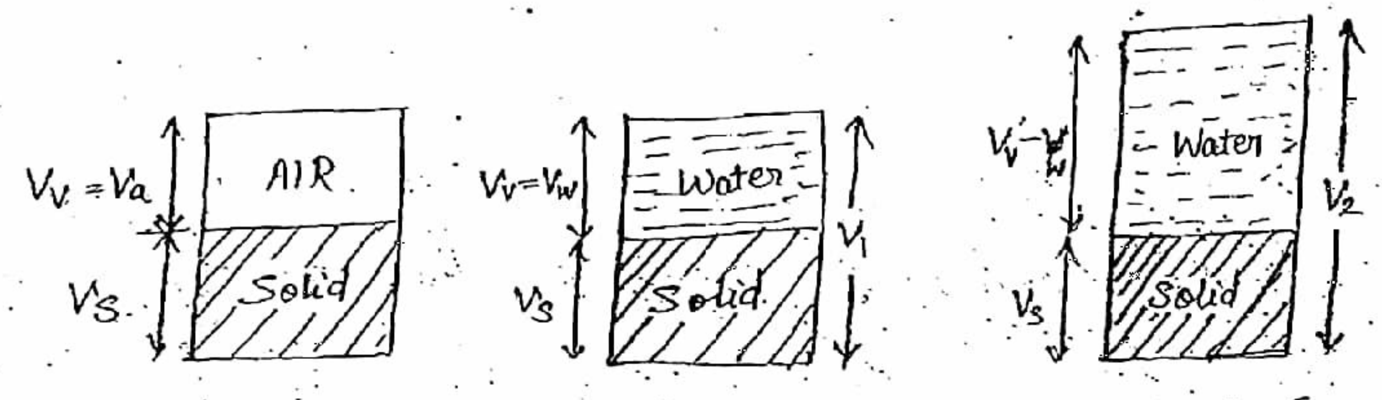

$S=0$ $S=1$ $S=1^*$ (Super saturated)

Note 2 :-

$$\frac{dy}{dx} = \frac{V_L - V_P}{W_L - W_P} = \frac{V_L - V_d}{W_L - W_d} = \frac{V_P - V_d}{W_P - W_S} = \text{constant}$$

* Atterberg analysed consistency of soil in 4 stages as liquid, plastic, semi-solid and solid stage.

* The water content at which soil passes from one stage of consistency to another stage of consistency is known as consistency limit or Atterberg limits.

LIQUID LIMIT (W_L)

* It is the minimum water content at which soil has tendency to flow.
* At liquid limit soil passes from liquid stage of consistency to plastic stage of consistency and vice-versa.
* All soils at liquid limit passes same negligible shear strength of $2.7\ KN/m^2$.

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Type of soil	Liquid limit (W_L)
Gravel	→ Non plastic
Sand	→ Non plastic
Silt	→ 30 – 40
Clay (alluvial soil)	→ 30 – 150
Clay (Black Soil)	→ 400 – 1500
Clay (Bentonite Soil)	→ 500 – 800

* Soils having higher value of liquid limit posses high compressibility (vol^n change in the soils are more)

⇒ METHODS FOR DETERMINATION OF LIQUID LIMIT

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(1) Cassagrande's Apparatus.

Liquid limit may also be defined as the minimum water content at which a part of soil cut by a tool of standard dimension flows together by a distance of $\frac{1}{2}$ inch (approx 12mm) under the impact of 25 blows, where height of fall in each blow is adjusted to 1cm

⇒ Cassagrande's tool or ASTM tool that can be used for cutting the grooves.

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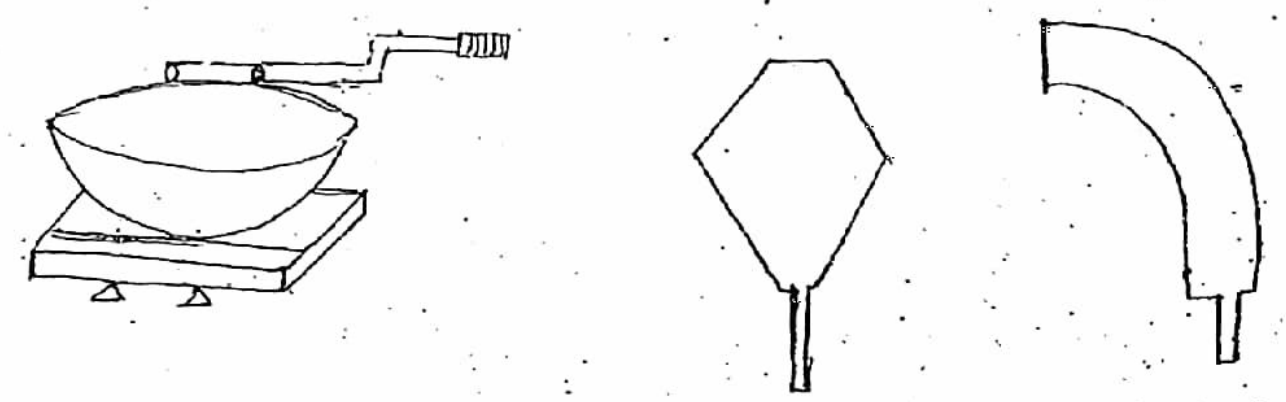

Cassagrande Tool   ASTM tool

Before the test   After the test

* In order to find the liquid limit, no. of blow required by the soil, to follow a distance of 1/2 inch corresponding to particular water content is noted. This test is repeated for same soil at different water content and corresponding number of blows are noted. The result of test are represented in the ~~flow~~ form of a curve known as flow curve from which water content corresponding to 25 no. of blow is interpolated and is called liquid limit.

In flow curve, water content is expressed on Y-axis & no. of blows in log scale on the X-axis.

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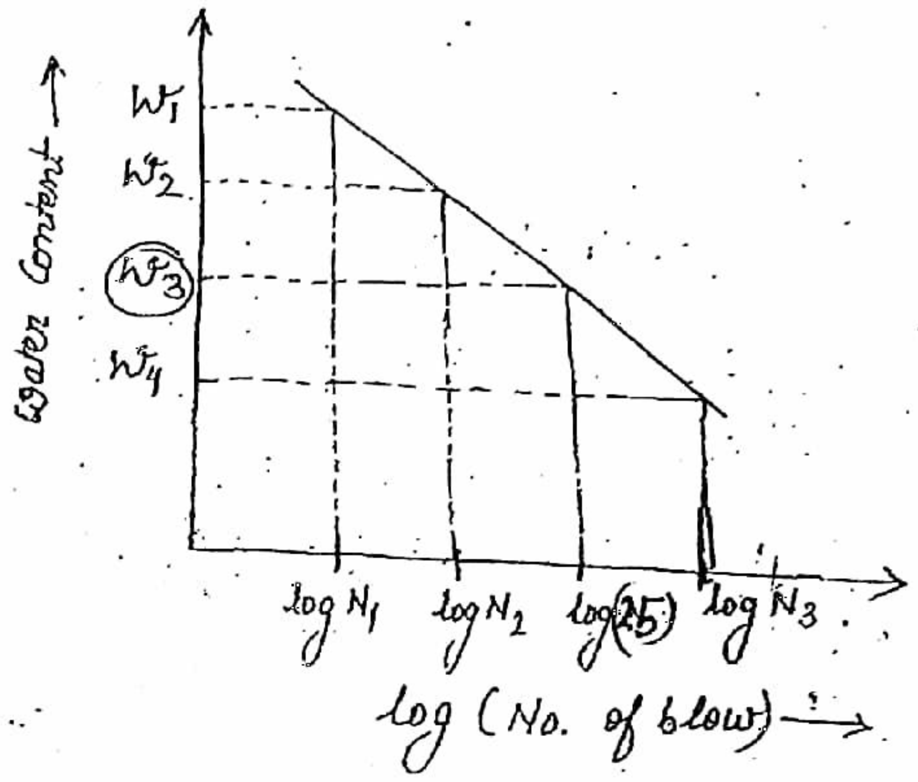

Flow Curve.

* The slope of flow curve is termed as flow Index (I.F) which represents the rate of loss of shear strength of soil with res in Water Content.

$$\text{Slope of flow Curve} = \frac{W_1 - W_2}{\log N_1 - \log N_2}$$

$$\boxed{\text{Flow index} = \left| \frac{W_1 - W_2}{\log (N_1/N_2)} \right|}$$

$$\boxed{I_f = \frac{W_1 - W_2}{\log\left(\frac{N_2}{N_1}\right)}} ***$$

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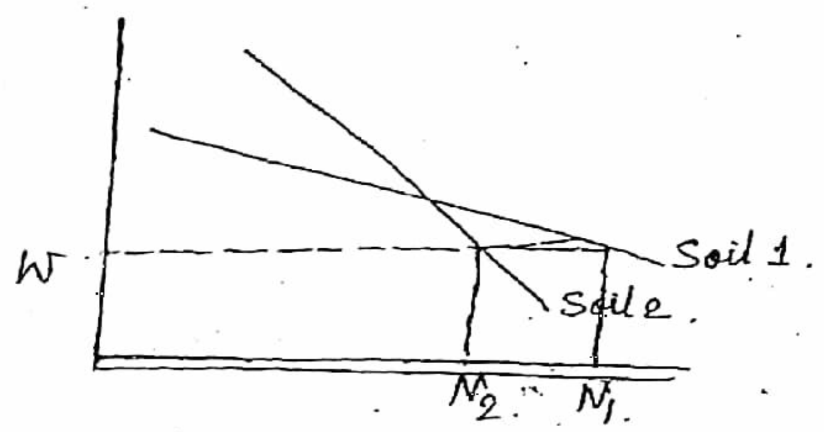
Note 2 :– Higher the value of flow index for particular soil, lower will be its shear strength.

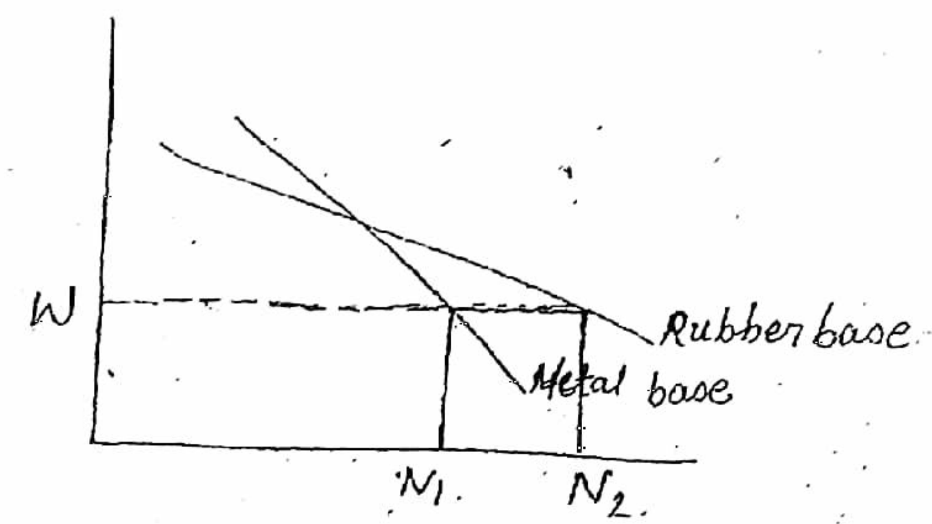
If $I_{f1} > I_{f2}$ (SS = shear strength)

then $(SS)_2 < (SS)_1$



Note 3 :– If the rubber base of Cassagrande apparatus is replaced by metal base then I_f will res.

$(I_f)_{metal\ base} > (I_f)_{Rubber\ base}$



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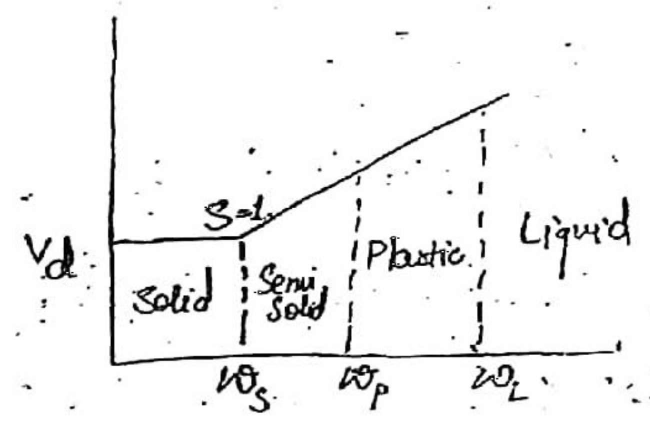
Consistency Curve



One Point Method

1. liquid limit can also be computed using the single observation of Casagrande test emphirically.

$$w_L = w_y \left(\frac{N}{25}\right)^x$$

where w_y = water content corresponding to N no. of blows

$x = 0.068$ to 1.21

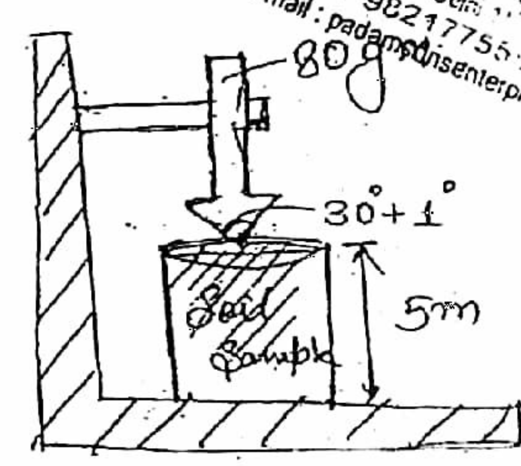
if N = 20 to 30, then,

$x = 0.1$

Cone penetrometer Test

Recommeded by IS code (IS-2720)

* In this test cone penetrometer having vertex angle of $30° \pm 1°$ and total sliding mass of 80gm is used.



* The sample of soil to be tested is placed in cylindrical vessel having dia & height of 5 cm each below the penetrometer.
* The cone is released & allow to penetrate the soil for 30sec.
* Liquid limit is taken as that water content which the penetrometer gives the penetration of 20mm in the soil.

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Plastic limit (w_P)

1. It is defined as minimum water content at which soil is at plastic stage of consistency or behaves as plastic material.

2. At plastic limit soil passes from plastic ~~of~~ stage of consistency to semisolid stage of consistency & vice versa.

Type of Soil	w_P(%)
Gravel	Non - plastic
Sand	Non plastic
Silt	20 - 25
Clay (Alluvial soil)	25 - 50
Clay (Black soil)	100 - 50.

From determination point of view. Plastic limit is defined as minimum water content at which soil begins to crumble or crack when rolled into 3 mm dia thread.

Shrinkage limit (w_s):

It is defined as max water content beyond which further reduction in the water content of soil does not leads to the reduction in the volume of soil.

It is min water content at which soil is just completely saturated.

At shrinkage limit soil passes from semisolid stage of consistency to the solid stage of consistency & vice versa.

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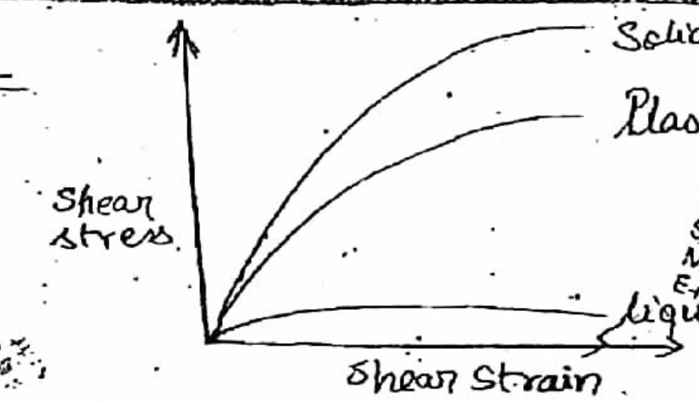
NOTE 1



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

I_f jyade Shear Strength

2.7

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NOTE 2: High compressible soil posses lower shrinkage limit while liquid limit is very high for such soils.

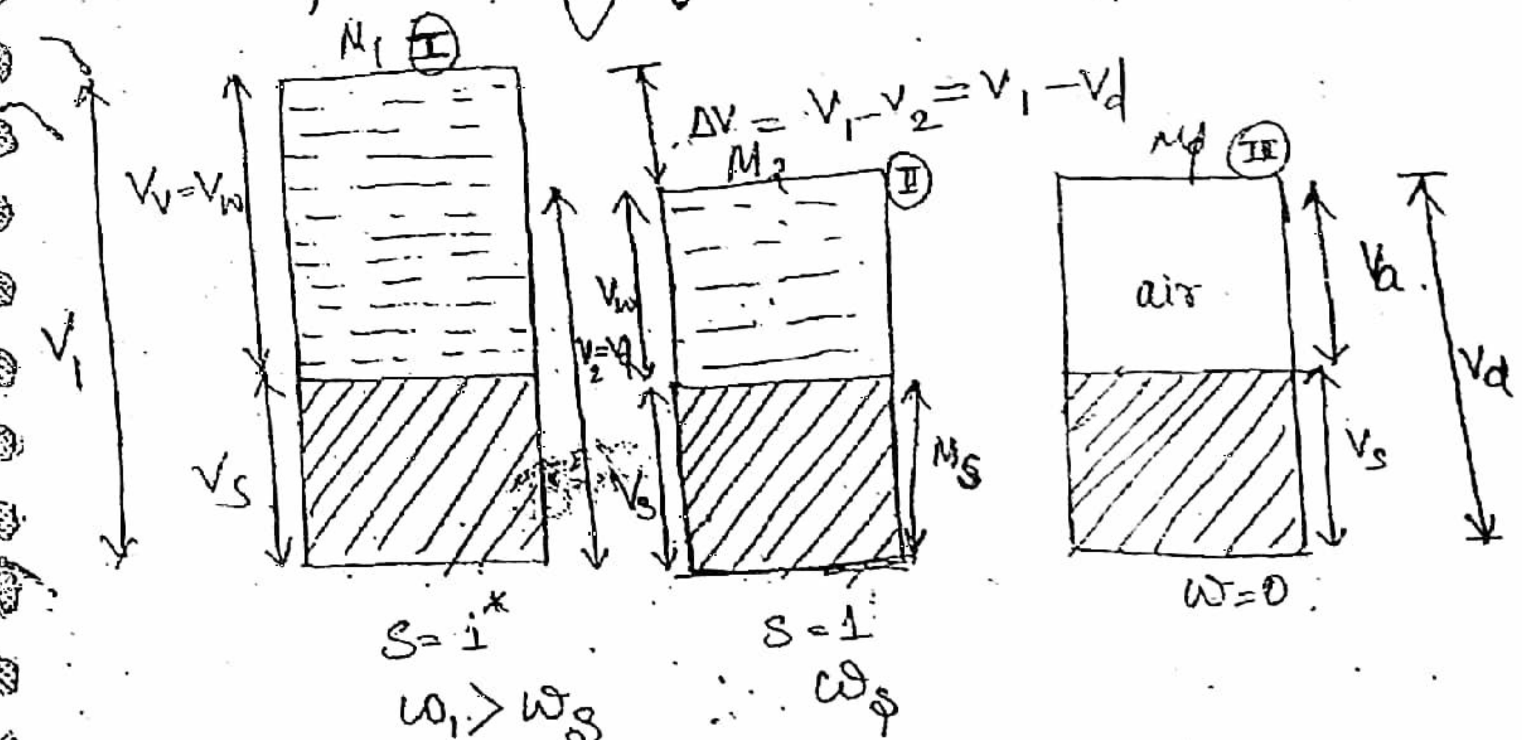
$w_L \uparrow$ compressibility $\uparrow$ shrinkage $\downarrow$.

Determination of Shrinkage limit

1. Consider a sample of ~~soil~~ soil having water content greater than shrinkage limit. Let M_1 & V_1 be the mass & volume of the soil at this stage.

The soil is subjected to drying ~~and the~~ as result of which it volume decrease ~~and~~ at a particular stage of drying, when its water content equal the shrinkage limit let M_2 & V_2 be the mass & vol of the soil.

On complete drying let mass M_d and volume is V_d.



Note.
$M_d = M_s$
$V_d \neq V_s$
$V_2 = V_d$

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I approach $w_s = w_1 - \Delta$ water content

$$w_s = w_1 - \frac{\text{wt of water}}{\text{wt of solid}}$$

$$\boxed{w_s = w_1 - \frac{(V_1 - V_d)\,\rho_w}{M_d}}$$ ***

⇒ Mass in water in I $= M_1 - M_s = M_1 - M_d$.

⇒ Loss of water in II from I $= \Delta V \cdot \rho_w = (V_1 - V_2)\rho_w$
$= (V_1 - V_d)\,\rho_w$

⇒ Mass of water in II = Mass of water in I − loss
$= (M_1 - M_d) - (V_1 - V_d)\,\rho_w$

⇒ water content in I $= \dfrac{\text{wt of water in I}}{\text{wt of solid in}} = \dfrac{M_1 - M_d}{M_s} = w_1$

⇒ water content in II $= \dfrac{\text{wt of water in II}}{\text{wt of solid}} = \dfrac{(M_1 - M_d) - (V_1 - V_d)\rho_w}{M_s}$

$$w_s = \frac{M_1 - M_d}{M_s} - \frac{(V_1 - V_d)\rho_w}{M_s}$$

$$\boxed{w_s = w_1 - \frac{(V_1 - V_d)\,\rho_w}{M_d}}$$ ** $\because M_s = M_d$

II Approach

⇒ Volume of water in II $= V_2 - V_s = V_d - V_s$

⇒ Mass of water in II $=$ Volume $\times \rho_w$
$= (V_d - V_s)\,\rho_w$

⇒ Water content in II $= \dfrac{\text{Wt of water}}{\text{Mass of solid}}$

$$w_s = \frac{(V_d - V_s)\,\rho_w}{M_s} = \frac{V_d}{M_s}\rho_w - \frac{V_s}{M_s}\rho_w$$

R = shrinkage ratio
w_s = shrinkage limit

$$= \frac{\rho_w}{\rho_d} - \frac{\rho_w}{\rho_s}$$

$$\boxed{w_s = \frac{1}{R} - \frac{1}{G}}$$ **

$\because R = \dfrac{\rho_d}{\rho_w}$

$G = \dfrac{\rho_s}{\rho_w}$

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Shrinkage index (I_s)

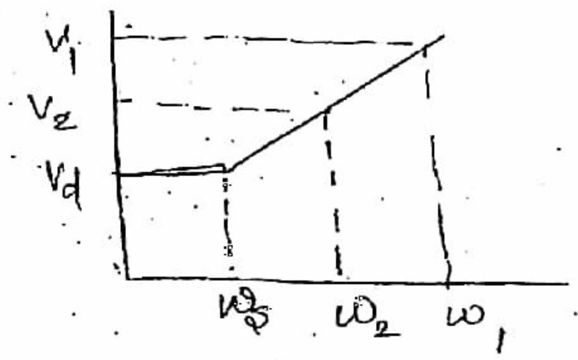
Shrinkage index represents range of semi solid stage of consistency.

$$\boxed{I_s = w_p - w_s}$$

Shrinkage Ratio (R)

1. It is defined as the ratio of (decrease in volume of soil expressed as the % of its dry volume) to the corresponding (change in water content) above the shrinkage limit.

$$\boxed{R = \left(\frac{\dfrac{V_1 - V_2}{V_d}}{w_1 - w_2}\right) \times 100}$$



⇒ if $w_1 = w_s$ then $V_2 = V_d$.

$$\boxed{R = \frac{\dfrac{V_1 - V_d}{V_d}}{w_1 - w_s} \times 100}$$

Relation b/w R, γ_d & γ_w

⇒ $w_1 - w_2 = \dfrac{(V_1 - V_2)\rho_w}{M_s} = \dfrac{(V_1 - V_2)\rho_w}{M_d}$ —— (I)

⇒ we know $R = \dfrac{(V_1 - V_d)/V_d}{w_1 - w_s} \times 100$

$R(w_1 - w_s) = \dfrac{V_1 - V_d}{V_d}$ —— (II)

(II) ÷ (I)

$$\frac{R(\cancel{w_1 - w_s})}{(\cancel{w_1 - w_s})} = \frac{(\cancel{V_1 - V_d})/V_d}{\dfrac{(\cancel{V_1 - V_2})\rho_w}{M_d}}$$

$$R = \frac{M_d}{V_d} \times \frac{1}{\rho_w}$$

$$\boxed{R = \rho_d/\rho_w \Rightarrow R = \frac{\gamma_d}{\gamma_w}}$$

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Volumetric Shrinkage (V.S)

Volumetric Shrinkage is defined as %es in vol. of soil expressed as % of its dry volume when water content is reduced from its given value upto the shrinkage limit.

$$\boxed{V.S = \frac{V_1 - V_d}{V_d} \times 100}$$

we know that $R = \frac{(V_1 - V_d)/V_d}{(w_1 - w_s)} \times 100$

$$\therefore \boxed{R(w_1 - w_s) = V.S.}$$

Plasticity index (I_p)

* It the range of consistency in which soil shows plastic properties or behaves as plastic material.
* It is defined as difference b/w liquid limit & plastic limit of soil

$$\boxed{I_p = W_L - W_P}$$

Type soil	I_p
Gravel	0
Sand	0
Silt	10–15
clay (Alluvial soil)	15–100
Clay (Black soil)	100–250

I_p	Description
0	Non plastic
<7	low plastic
7–17	Medium plastic
>17	High plastic

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Note 1 → When non-plastic soil like sand is mixed with plastic soil like clay then plasticity index of soil (mixture) will reduced which is given as :-

$$\boxed{I_{p(mix)} = \frac{I_{p_1} \cdot X_1 + X_{p_2} \cdot X_2}{X_1 + X_2}} \text{ (weighted mean)}$$

Note 2 :- Plasticity is defined as property of soil by virtue of which it undergoes deformation without cracking, fracturing or rupturing.

Plasticity of soil is due to minerals (K, I, M) And adsorbed water on it.

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Consistency Index & Liquidity Index

These indexes represent insitu behaviour of the soil on consistency stage on the field or degree of firmness of the soil.

$$\boxed{I_c = \frac{W_L - W_n}{W_L - W_P} = \frac{W_L - W_n}{I_p}}$$

W_n = natural water content.

$I_c < 0$ — $W_n > W_p$ — Liquid stage

$I_c > 1$ — $W_n < W_p$ — ~~plastic stage~~ semisolid & solid

$0 < I_c < 1$ — $W_p < W_n < W_L$ — plastic stage

Liquidity Index

$$\boxed{I_L = \frac{W_n - W_p}{W_L - W_p} = \frac{W_n - W_p}{I_p}}$$

$I_L > 1$ — $W_n > W_L$ — Liquid stage

$0 < I_L < 1$ — $W_p < W_n < W_L$ — plastic stage

$I_L < 0$ — $W_n < W_p$ — solid & semisolid

$$\boxed{I_L + I_c = 1}$$

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I_c	I_L	Consistency Stage	Description
<0	>1	Liquid	Liquid
0 – 0.25	1 – 0.75	Plastic	Very soft
0.25 – 0.5	0.75 – 0.5		soft
0.5 – 0.75	0.5 – 0.25		medium stiff
0.75 – 1	0.25 – 0		Stiff
>1	<0	semisolid	very stiff to hard
>1	<0	solid	Hard to very hard

Compressibility More
Shrinkage less

Toughness Index (I_t)

It represents strength of soil at its plastic limit. It is defined as ratio of plasticity index to the flow index.

$$\boxed{I_t = \frac{I_p}{I_f}}$$

* Toughness index generally varies b/w 0 to 3. If for any soil toughness index is less than 1 it is considered to be easily breakable at plastic limit.

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Density index / Relative Density / Degree of Denseness (I_D)

* Density index is used to represent the relative compactness of natural coarse grain soils (cohensionless soil)
* It is defined as

$$\boxed{I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100}$$

where e_{max} = max void ratio in its losest state.
e_{min} = min void ratio in its densest state.
e = natural void ratio.

we know $\gamma_d = \frac{G\gamma_w}{1+e}$ $\qquad \gamma_d \propto \frac{1}{e}$

$$e = \frac{G\gamma_w}{\gamma_d} - 1$$

$$I_D = \frac{\left(\frac{G\gamma_w}{\gamma_{d(min)}} - 1\right) - \left(\frac{G\gamma_w}{\gamma_d} - 1\right)}{\left(\frac{G\gamma_w}{\gamma_{d(min)}} - 1\right) - \frac{G\gamma_w}{\gamma_{d(max)}} - 1}$$

$$\boxed{I_D = \frac{\frac{1}{\gamma_{d(min)}} - \frac{1}{\gamma_d}}{\frac{1}{\gamma_{d(min)}} - \frac{1}{\gamma_{d\,max}}}}$$

I_D (%)	Description
0 – 15	Very loose soil
15 – 35	loose soil
35 – 65	medium soil
65 – 85	Dense soil
85 – 100	Very Dense soil.

Note 1 :- Density Index is use to represent compactness of cohensionless soil only because of large uncertainity involved in computation of e_{max} in its loosest state in laboratory for cohesive soil.

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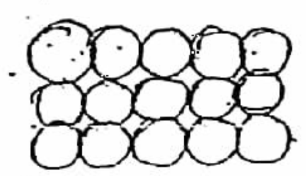
Note 2. Density index is better indicator of denseness of soil than void ratio & dry density because it represents the denseness in absolute term.

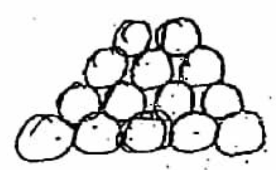
* For the soil having higher value of I_D, higher degree of denseness, stability, shear strength would be observed & lower permeability & compressibility would be observed.

For spherical & uniformally ~~well~~ graded soil.



$e_{max} = 0.91$

Loosest soil



$e_{min} = 0.35$

Densest soil.

Senstivity & Thixotropy.

Consistency of undisturbed sample of clay is changed upon remoulding even at same water content

This change in consistency or decrease in degree of firmness or decrease in strength take place due to following factors :-

1) Permanent of destruction of soil solid upon remoulded ~~remoulded (permanent loss)~~ (permanent loss)

2) Reorientation of water molecule in the adsorbed layer soil solids (recoverable).

3) This loss of strength is represented in terms of its sensitivity which denote the degree of disturbance of sample upon remoulding.

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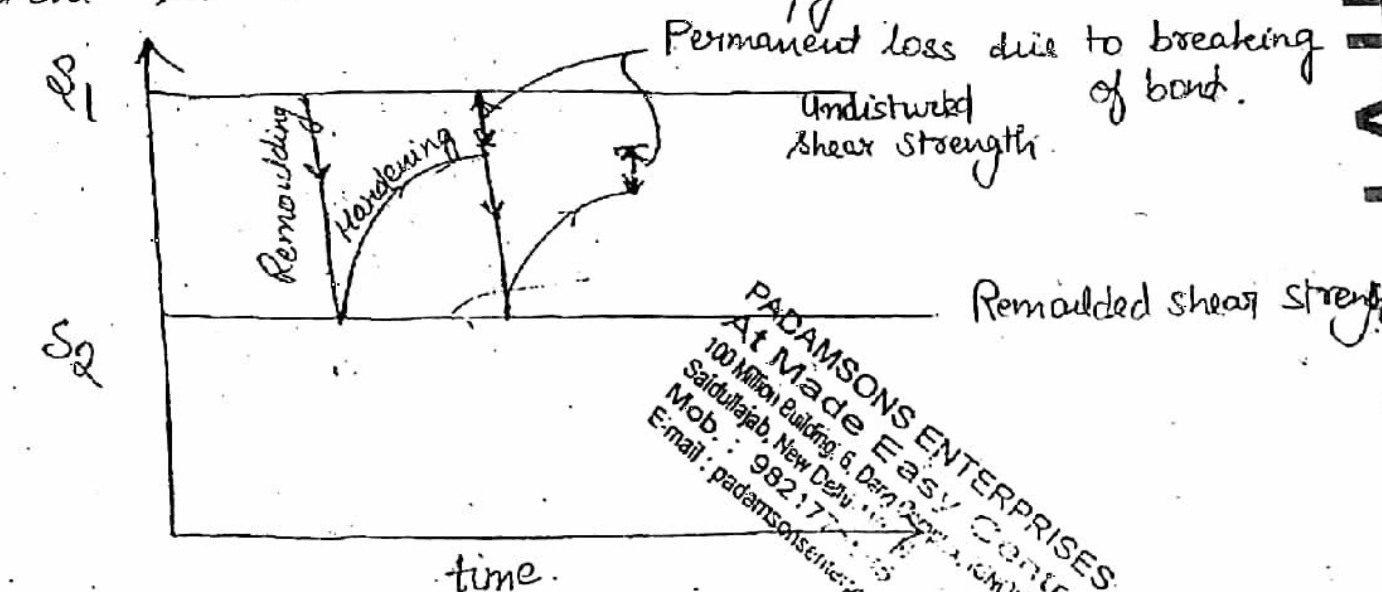
Sensitivity is defined as ratio of unconfined compressive strength (UCS) of soil in its undisturbed state to unconfined compressive strength of soil in its remoulded state.

$$\boxed{S_t = \frac{(UCS)_{undisturbed}}{(UCS)_{remoulded}}}$$

Sensitivity	Description
< 1	In sensitive.
2 – 4	Normal / Less sensitive (Honey Comb)
4 – 8	Medium sensitive (Honey comb / flocculant)
8 – 16	Extra sensitive (flocculant)
> 16	unstable / Quick

Thixotropy

Over a period of time soil regains its a part of its lost strength. This property of soil by virtue of which it regains a part of its lost shear strength at constant water content is termed as thixotropy.



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Activity

The behaviour or plasticity of soil depends upon type of mineral (K, I, MM) and amount of adsorbed water present in the soil.

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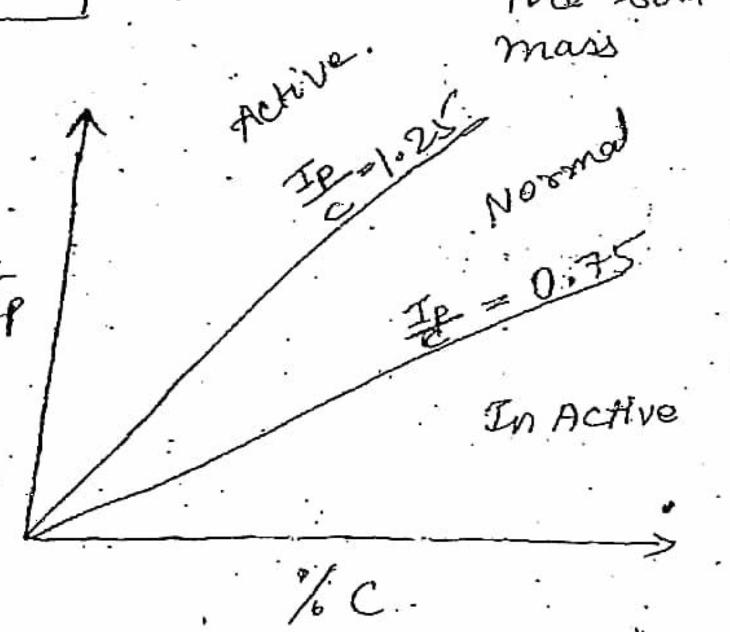
* Skempton defined a parameter termed as activity which represents the volume change in soil with change in water content or compressibility.

* Activity is defined as ratio of plasticity index of the soil to the % of particles present in it having size less than 2μ (clay size)

$$\text{Activity} = \frac{I_p}{C}$$

where C = % clay size particle present in the soil mass

Activity	Description
<0.75	In Active
0.75 – 1.25	Normal
>1.25	Active



Clay Mineral	Activity
(K) Kaolinite	0.4 – 0.5
(I) Illite	0.5 – 1
(MM) Montmorillonite	1 – 7
Na-montmorillonite	4 – 7
Ca-montmorillonite	1.5

Note → Highly active soil are not considered suitable for construction.

Black Cotton soil consist of high amount of montmorillonite. Thus, shows large vol. change in water change.

Collapsibility

Soil which shows large decrease in volume with increase in its water content without any Inc in pressure are termed as collapsible soil.

Loess

Collapsibility of soil is measured in form of a parameter termed as collapsible potential which is defined as ratio of decrease in volume of soil to its original volume with Inc in water content.

$$\text{Collapsible potential} = \frac{\Delta V}{V_0} = \frac{\Delta H}{H_0}$$

H_0 & V_0 = initial thickness & volume.

ΔH & ΔV = change in thickness & volume.

Difference b/w Organic & Inorganic soil

1) Physical Observation

2) For organic clays liquid limit of ovendry sample is 0.7 times than liquid limit of air dried / moist sample.

3.) Generally organic soils have liquid limit values greater than 50% but their plastic limits are also higher ∴ their plasticity index values are not higher in comparison to liquid limit.

$\frac{0.35}{1.5}$

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Saidullajab, New Delhi-110030
Mob.: 9821775585
E-mail: padamsonsenterprises@gmail.com

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Q 12) $w = 35\%$ $G_m = 1.90$ $G_{m_2} = 1.75$ $G = ?$

$G_m = \frac{\gamma_{sat}}{\gamma_w} \quad \gamma_d = \frac{G\gamma_w}{1+w} = \frac{1.90 \times 9.81}{1.35}$

$G_m = 1.09 \quad G = 2.77$

Saturated $G_m = \frac{\gamma_{sat}}{\gamma_w} = 1.90$ Oven dried $\frac{\gamma_d}{\gamma_w} = 1.75$

$\Rightarrow \frac{(G+e)\gamma_w}{(1+e)\gamma_w} = 1.90$ $Se = wG$, $e = 0.35 \times G$

$\Rightarrow \frac{G + 0.35G}{1 + 0.35G} = 1.90$

$\boxed{G = 2.77}$

$\Rightarrow w_s = \frac{1}{R} - \frac{1}{G}$

$= \frac{\gamma_w}{\gamma_d} - \frac{1}{G} = \left(\frac{\gamma_w}{1.75\gamma_w} - \frac{1}{2.77}\right) \times 100 = 21.04\%$

13) $e = 0.67$ $G = 2.65$ $w = 12\%$ $V_w = ?$ $V = 100\, m^3$

$S = 1$

$V = V_s + V_v = 100$

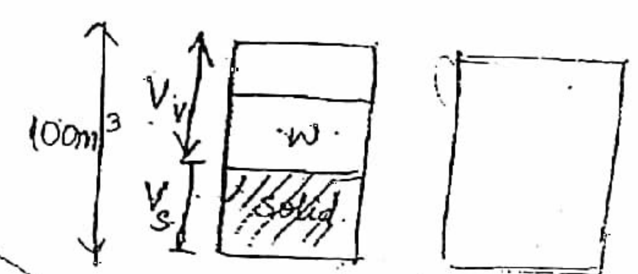
$e = \frac{V_v}{V_s} = 0.67$

$V_v = 0.67 \times V_s$

$1.67\, V_s = 100$

$V_s = 59.88$

$V_v = 40.119$



$e = 0.67$ $e = 0.67$

$G = 2.65$ $w_2 = 0.24$

$w_1 = 12\%$

$Se = wG$

$w = \frac{Se}{G} = 0.24$

$W_s = \frac{G}{1+e}$

$\therefore w = 0.24$

$\frac{\text{wt of water}}{\text{wt of solid}} = 0.24$

13) $e = 0.67$ $S = 1$

$G = 2.65$ $\Rightarrow e = 0.67$

$w_1 = 12\%$ $w_2 = \frac{0.67 \times S}{2.65} = 0.25$

$\gamma_d = \frac{W_s}{V_T} = \frac{W_s}{100} = \frac{G\gamma_w}{1+e}$

$\frac{W_s}{100} = \frac{2.65 \times 9.81}{1 + 0.67}$

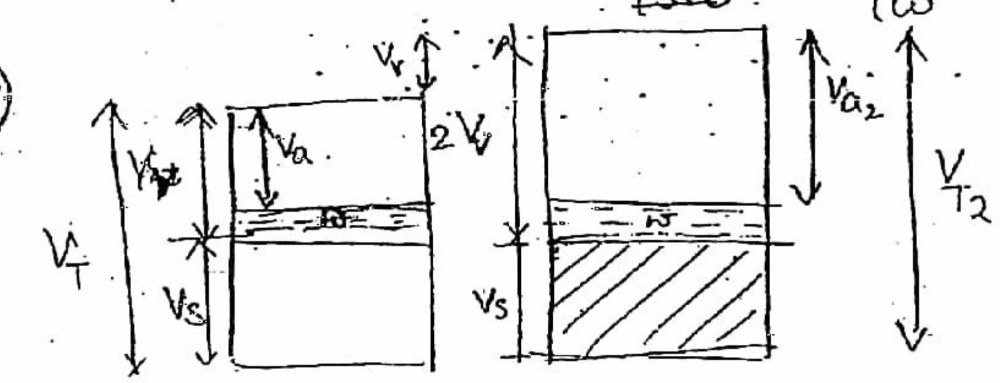
$W_s = 1556$ kN

$\therefore$ water added $= (0.25 - 0.12)\, W_s = (0.25 - 0.12)\, 1556$

$= 202.28$

$\Rightarrow$ Volume added $= \frac{\text{wt}}{\gamma_w} = \frac{202.28}{9.81} = 20.6\, m^3$

15)



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$a_c = \frac{V_a}{V_v}$ $\eta_{a_2} = ?$

$\eta_a = \frac{V_a}{V_T}$ $\eta_{a_2} = \frac{V_{a_2}}{V_{T_2}}$

$\eta_{a_2} = \frac{V_{a_2}}{V_{T_2}} = \frac{V_a + V_v}{V_T + V_v} = \frac{\left(\frac{V_a}{V_v} + 1\right)}{\left(\frac{V_T}{V_v} + 1\right)} = \frac{a_c + 1}{\left(\frac{1}{n} + 1\right)}$

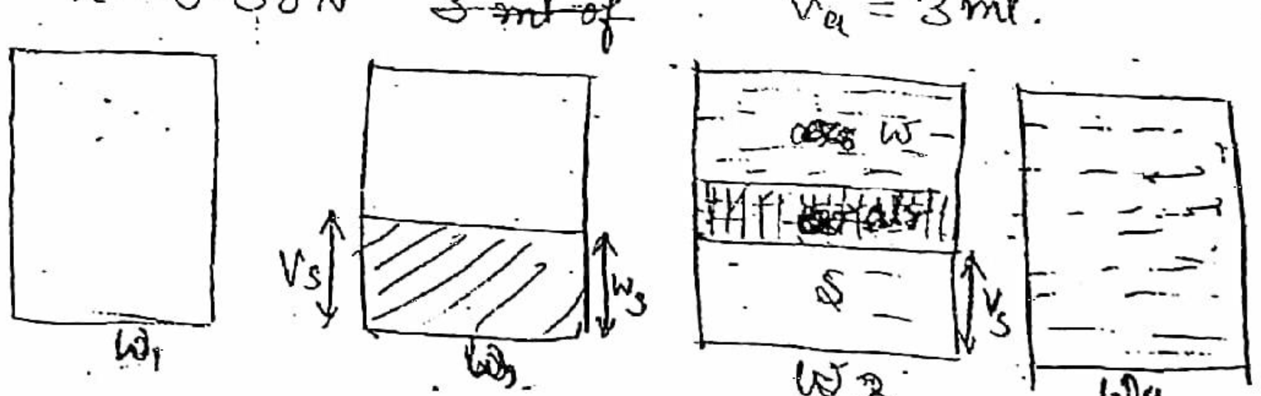
$\eta_{a_2} = \frac{(a_c + 1)}{\left(\frac{a_c}{\eta_a} + 1\right)}$ $\because \eta_a = n \cdot a$, $n = \frac{\eta_a}{a}$

33) wt of dry soil $= 1.04$ N $= (W_2 - W_1)$

wt of bottle + soil + water $= 5.38$ N (W_3)

wt of bottle + water $= 4.78$ N. (W_4)

$w = 5.38$ N ~~3 ml of~~ $V_a = 3$ ml.



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$$G = \frac{W_2 - W_1}{(W_4 - W_3) + (W_2 - W_1)} = \frac{1.04}{4.756 - 5.38 + 1.04} = \underline{2.5}$$

Error in W_3

Corrected W_3 = W_3 + wt of water having 3ml = ~~5.38N~~

$5.38N + 3ml \times \frac{1gm}{cc} \times 10^{-3} kg \times \frac{9.81m}{s^2}$

$= 0.029N$

$\Rightarrow$ Corrected $\omega_3 = 5.409$

$\Rightarrow G = 2.68$ corrected

$\Rightarrow$ Error $= \frac{2.689 - 2.5}{2.5} = \underline{7.4\%}$

V_d. $\frac{V_s}{}$

$\gamma_s = \frac{W_s}{V_s}$

$\gamma_d = \frac{W_s}{V_s}$

41) $w_s = 18\%$ $w_L = 45\%$ $V_d = ?$

$V_T = 23\ cm^3$. $G = 2.73$

$$\omega_s = \frac{1}{R} - \frac{1}{G}$$

$$18 = \frac{1}{R} - \frac{1}{2.73}$$

$\frac{1}{R} = 17.63 \Rightarrow R = 0.056$

$$R = \frac{(V_L - V_d)/V_d}{W_L - W_s} = \frac{(23 - V_d)/V_d}{0.45 - 0.18}$$

0.546

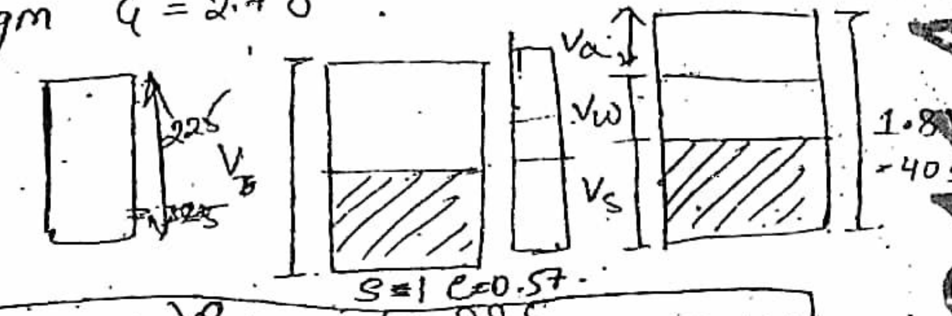
$\frac{23}{V_d} \cdots =$

$V_d = \underline{15.96\ m^3}$

$V_{sd} = 225\ cc$ $W_s = 390\ gm$ $G = 2.78$

$e = ?$ $\omega_s = ?$

$\omega_s =$



$$\cancel{w = \frac{W_\omega}{W_s}} = \frac{(V_1 - V_2)\rho_w}{W_s} = \frac{405 - 225}{390} = 0.46$$

$$\omega_s = \left[\omega_1 - \frac{(V_1 - V_d)}{M_d}\right] \times 100$$

47) $V_d = 225\ cm^3$ $M_s = 390\ gm$ $G = 2.72$. $e = ?$ $\omega_s = ?$

1) $\rho_d = \frac{G\rho_w}{1+e} = \frac{M_d}{V_d} = \frac{390}{225} = 1.73\ gm$

$\frac{2.72 \times 1}{1+e} = 1.73 \Rightarrow \underline{e = 0.57}$

2) $\omega_s = \frac{1}{R} - \frac{1}{G} = \frac{\rho_w}{\rho_d} - \frac{1}{2.72} = \frac{1}{1.73} - \frac{1}{2.72} = 0.209 = \underline{20.9\%}$

(3) $\omega_s = \omega_1 - \left(\frac{(V_1 - V_d) \times \rho_w}{M_d}\right)$

$0.209 = \omega_1 - \left(\frac{1.08 V_d - V_d}{M_d}\right) \times 1 \frac{gm}{cc}$

$\omega_1 = 0.255 = \underline{25.5\%}$

48) $V = \pi r^2 h = 196250\ mm^3 = 196.250\ cc$. $G = 2.69$

$w = 15\%$ $\eta_a = 20$ $\omega_s = ?$ $W_w =$

$w = \frac{W_w}{W_s}$ $\eta_a = \frac{V_v}{V_T}$ $\gamma_d = \frac{(1-\eta_a) G \gamma_w}{1 + wG}$

$\Rightarrow Se = wG$ $V_v = 39.25$ $e = 0.2$

$\gamma_d =$

Sol $\rightarrow$ $\Rightarrow \rho_d = \frac{(1-\eta_a) G \rho_w}{1 + wG} = \frac{W_d}{V_T}$

$\rightarrow \frac{(1 - 0.20) \times 2.69 \times 1}{} = \frac{W_{dry}}{196.25}$

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$W_{dry} = 301.06$

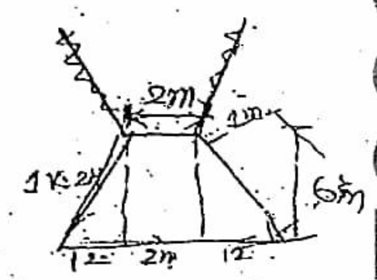
Mass of water requ $= w \times W_s = 20 \times 301.06\,gm$
$= 0.15 \times 301.06\,gm \times \frac{1\,gm}{cc}$
$= 45.16\,cc$

Total mass of soil $= 301.06 + 45.16 = \underline{346.22}$

49 Q) $\gamma_b = 15\,KN/m^3$ $w = 8\%$ (embankment)

$\gamma_d = 20\,KN/m^3$ $w = 10\%$ (borrow pit)

$G = 2.7$ volumes of embankment $= \frac{1}{2} \times (26+2) \times 6 = 84\,m^2$



for 1m $= \underline{84\,m^3}$

$W_t = 80\,KN$.

$\cancel{\gamma_b = \frac{W_s}{V_T} = \frac{G\gamma_w}{1+w}}$

$\cancel{= \frac{W_s}{V_T} = \frac{2.65 \times 9.81}{1+0.08} = 24.07}$

$\gamma_d = \frac{\gamma_b}{1+w}$

OR

$\gamma_d = \frac{W_s}{V_T} \Rightarrow 20 = \frac{W_s}{84} \Rightarrow (W_s)_{emb} = 1680\,KN$

$\Rightarrow (W_s)_{emb} = (W_s)_{BP}$

$\Rightarrow (W_s)_{emb} = \frac{(W_s)_{BP}}{1+w}$

$\Rightarrow 1680 = \frac{(W_s)_{BP}}{1+0.10}$

$\Rightarrow (W_s)_{BP} = 1848$

$\left(\frac{W_T}{V_T}\right)_{BP} = \gamma_b = 15$

$V_T = \frac{W_T}{\gamma_b} = \frac{1814.14}{15} = 120.96\,m^3$

of trip $= \frac{1848}{80} = 23$ trip

Embankment $\gamma_d = \frac{G\gamma_w}{1+e} \Rightarrow 20 = \frac{2.78 \times 9.81}{1+e}$

$\Rightarrow e = 0.368$

$Se = wG$

$S = \frac{wG}{e} = \frac{0.1 \times 2.7}{0.36} = 0.84 = 84\%$

Borrow pit $\gamma_d = \frac{G\gamma_w}{1+e} = \frac{\gamma_b}{1+w}$

$\frac{2.7 \times 9.81}{1+e} = \frac{15}{1+0.08}$

$e = 0.907$

$S = \frac{wG}{e} = \frac{0.08 \times 2.7}{0.907} = 0.238$

$= \underline{23.8\%}$

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CHAPTER-2 Classification of Soil

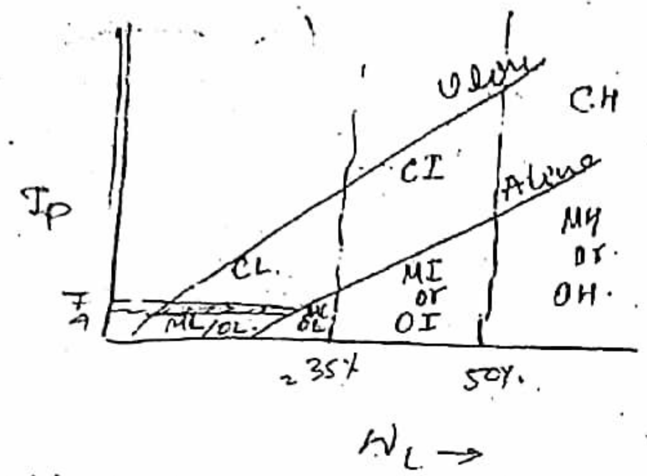
GI index

GI more quality lower as more c liquidity more
more d plasticity more
more a particle finess
b

$(4.75 < d < 80) >$ soil $(75\mu < d < 4.75)$

G : 4.75m. G > S Gravel
S : 0.075 m.

fines < 5% C_u & C_c is chkd
fines > 12% I_p chked



Coarse grained soil

→ If $W_L < 29.58\%$
if $I_p > 7$ then Clay
& $I_p < 7$ then check A.line

→ If $W_L > 29.58\%$
I_p of soil > I_p of A then clay
I_p of soil < I_p of A then silt

Flocculant → head to head
dispersed → face to face
head to face

Octahedron $-\frac{1}{3} \times 6 + 3 = +1$

Type of bond

Valency $H^+ - Cl^-$
electron exchange
Hydrogen bond
Ionic bond
Water bond is the weakest bond

water bond
Note Kerosene
Non polar

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Kaolinite → China ware China clay → Kaolinite.
It is in whole electrically neutral structure but in presence in water
they become charged

Montmorillonite 20 time thickness increases on addition of water.
Isomorphous substitution takes place ~~in the~~

Illite ionic bond more than water less than hydrogen.

1) 40% gravel 4.75 mm
50% sand 0.0075 mm
10% silt 0.002 mm

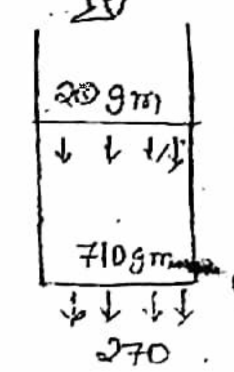
$C_u = \frac{D_{60}}{D_{10}} = \frac{4.75}{0.075}$

Cumulative	% (100 − c)
40	60
90	10
100	0

2) 980 g 0.075 mm
270 g 0.002 mm
0.425 mm

cum % 20

1000 g



$W_L = 40\%$ $W_p = 18\%$
$I_p = W_L - W_p$ $I_p > 7$

1) 75 μ retained 73%
Coarse
2) S > G ⇒ sand
3) % finess → 27%
> 12%

$I_p = 40 - 18 = 22$.
for Coarse grain chk for Ip line

$I_p = (I_p)_A = 0.73(W_L - 20)$
$= 0.73(40-20)$
$= 14.6 < 29.8$
$(I_p)_{A\,line} < I_p$ soil
Clay,

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(i) 38, 60.

(ii) $w_{n_A} = 40\%$ $w_{n_B} = 50\%$

$w_{P_A} = 25\%$ $w_{P_B} = 30\%$

$(w_L)_A = 38\%$ $(w_L)_B = 60\%$

(ii) $I_P = (w_L - w_P)$

$I_{P_A} = 13\%$ $I_{P_B} = 30\%$

(iii) $I_c = \frac{w_L - w_n}{I_P}$

$I_{C_A} = \frac{38-40}{13} = -0.15$ (liquid limit)

$I_{C_B} = \frac{60-50}{30} = 0.33$ (plastic)

$(I_c)_B > (I_c)_A$

B is better for foundation

(iv) $(w_L)_B > (w_L)_A$

B is more compressible

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

$(I_f)_A = \frac{0.73 - 0.37}{\log\left(\frac{30}{38}\right)}$

$= 10.45$

$(I_f)_B = \frac{0.65 - 0.58}{\log\frac{5}{40}}$

$= 8.38$

$(I_f)_A > (I_f)_B$

∴ Rate of loss in SS will be more for soil A

loss of Shear strength in plastic limit then toughness index is found out.

(vi) Soil B: $(I_P)_{A\ line} = 0.73(60-20) = 29.2$

$(I_P)_{soil} > (I_P)_{A\ line}$

⇒ inorganic clay

Soil A: $(I_P)_{A\ line} = 0.73(38-20) = 13.14$

$(I_P)_{soil\ A} = 13$

$(I_P)_{soil_A} \cong (I_P)_{A\text{-}line}$

Probability of silt more hence soil A & B may not contain organic matters.

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	% passing 0.075 mm	4.75 mm
Soil A	14	92
Soil B	75	75

Soil (A)

4.75 mm — 92 gm — 86% Sand

0.075 — 14 gm — %

Coarse (86% retained)

Fine (78) > 9 (B%) ⇒ Sand

⇒ $I_P = w_L - w_P = 16 - 8 = 8\%$

⇒ $I_P > 7$ & Fines 14% > 12% case (b)

$\boxed{SC}$ $C_u > 6$

Soil (B)

25% 4.75 Sand

75% 0.075

OH

(1) $w_L = 58$

$w_L > 50$ High compressi...

(2) $I_P = (58 - 14) = 44$

(3) $(I_P)_{A\ line} = 0.73(w_L - 20) = 27.72$ 27.74

(4) Check

$(I_P)_{U\ line} = 0.9(w_L - 8)$

$= 0.9(58-8) = 45$

$I_{P(A\ line)} < I_P(soil) < I_{P(U\text{-}line)}$ ⇒ $\boxed{CH}$ clay

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CHAPTER – 3 EFFECTIVE STRESS

Total stress

* At any given plane section in the soil mass, total stress (load/area) is due to the self wt. of the soil (solid + water) or due to the applied overburden pressure (uniform large area surcharge).

* This total stress further consist of two different components {
1. Effective Stress (intergranular stress)
2. Pore water pressure (Neutral pressure)

$$\boxed{\sigma = \bar{\sigma} + \bar{U}}$$

Effective Stress

* It is the stress which is being transfered in soil mass by ~~stay~~ grain to grain contact which tends to force the particles to come into closer state of contact resulting in its decreased void ratio, increased degree of denseness & mobilisation of shear strength.

∵ These stresses are being transfered by grain to grain these stresses are also termed as intergranular stress.

Pore Water Pressure

It is the pressure which is being transmitted by pore fluid & is equal to the weight of fluid (wt. of water column) above the conserned section in soil mass. This water pressure acts all around the soil solids hence does not tends to force the soil solids into closer

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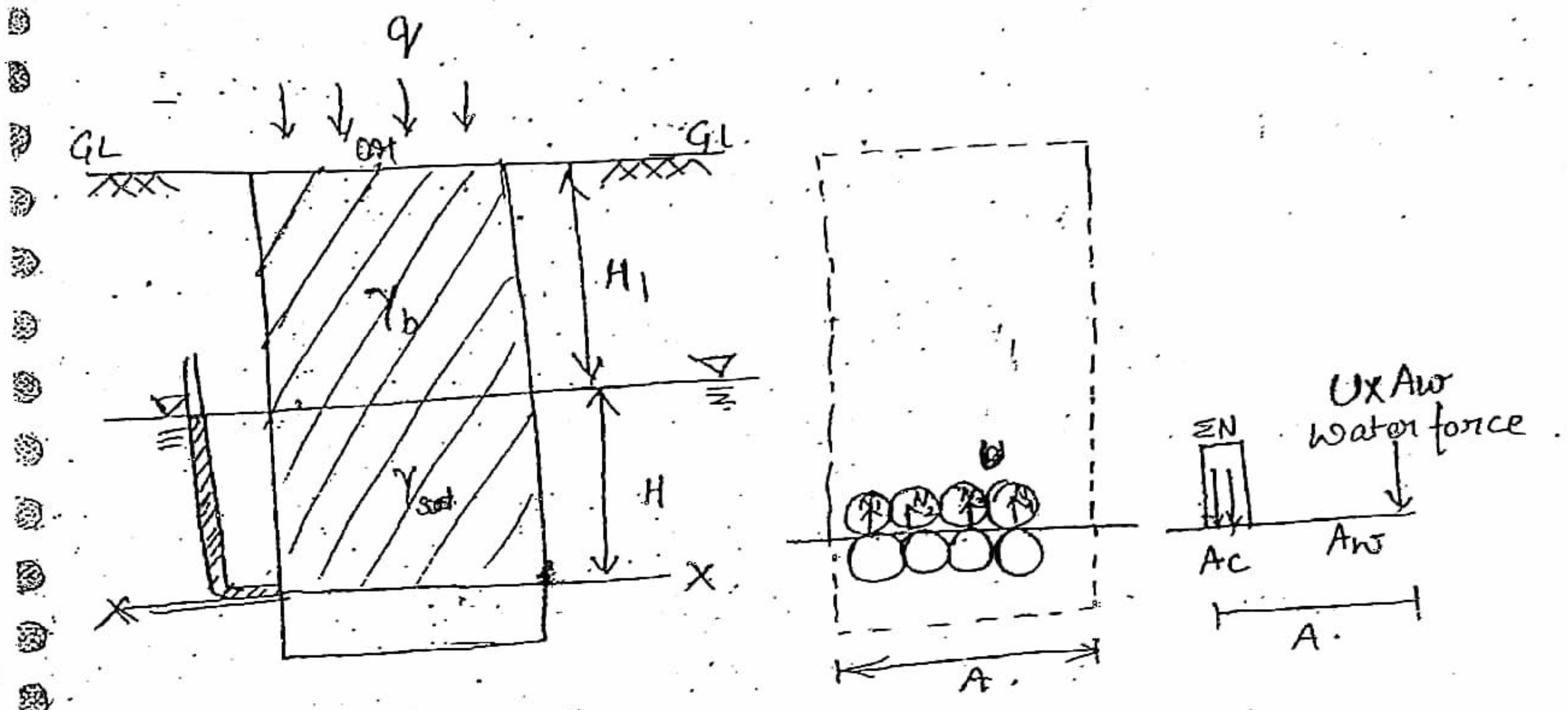
state of contact. It does not have any shear component. It is also termed as neutral pressure.



$$\text{Total Stress} = \frac{\text{Total Wt}}{\text{Area}} = \frac{\text{wt. of (solid + water)}}{\text{Area}} = \frac{\text{Vol.} \times \gamma}{\text{Area}}$$
$$= \frac{A \times h\gamma}{A} = h\gamma$$

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at section X – X

Total stress $(\sigma) = \gamma_b H_1 + \gamma_{sat} H$

if surcharge applied

$(\sigma) = \gamma_b H_1 + \gamma_{sat} H + q$

Total stress = Effective Stress + Pore water Pressure

$$\text{Pore water pr}^n = \frac{\text{wt. of water}}{\text{Area of water}}$$
$$= \frac{A_w \times \gamma_w \times H}{A_w} = H\gamma_w$$

U = pore water pressure = pressure head × γ_w
= $H \times \gamma_w$

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Effective stress = Total stress − Pore water pressure

$= \gamma_{sat} H + \gamma_b H_1 - \gamma_\omega H$

$= \gamma_b H_1 + (\gamma_{sat} - \gamma_\omega) H$

$$\boxed{\bar{\sigma} = \gamma_b H_1 + \gamma' H}$$

Or.

$$\boxed{\bar{\sigma} = \gamma_b H_1 + \gamma' H + q}$$ when q surcharge applied.

Alternate method

Plane x − x

Total force $= \Sigma N + U \cdot A_w$ ∵ water pressure × Area = force;

Total stress $= \dfrac{\text{Total force}}{\text{Area}}$

$\sigma = \dfrac{\Sigma N + U \cdot A_w}{A} = \dfrac{\Sigma N}{A} + \dfrac{U \cdot A_w}{A}$

$A = A_c + A_w$

∵ $A_c \ll A_w$

∴ $A = A_w$

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

$$\boxed{\sigma = \bar{\sigma} + U}$$

∵ $\boxed{\bar{\sigma} = \dfrac{\Sigma N}{A}}$

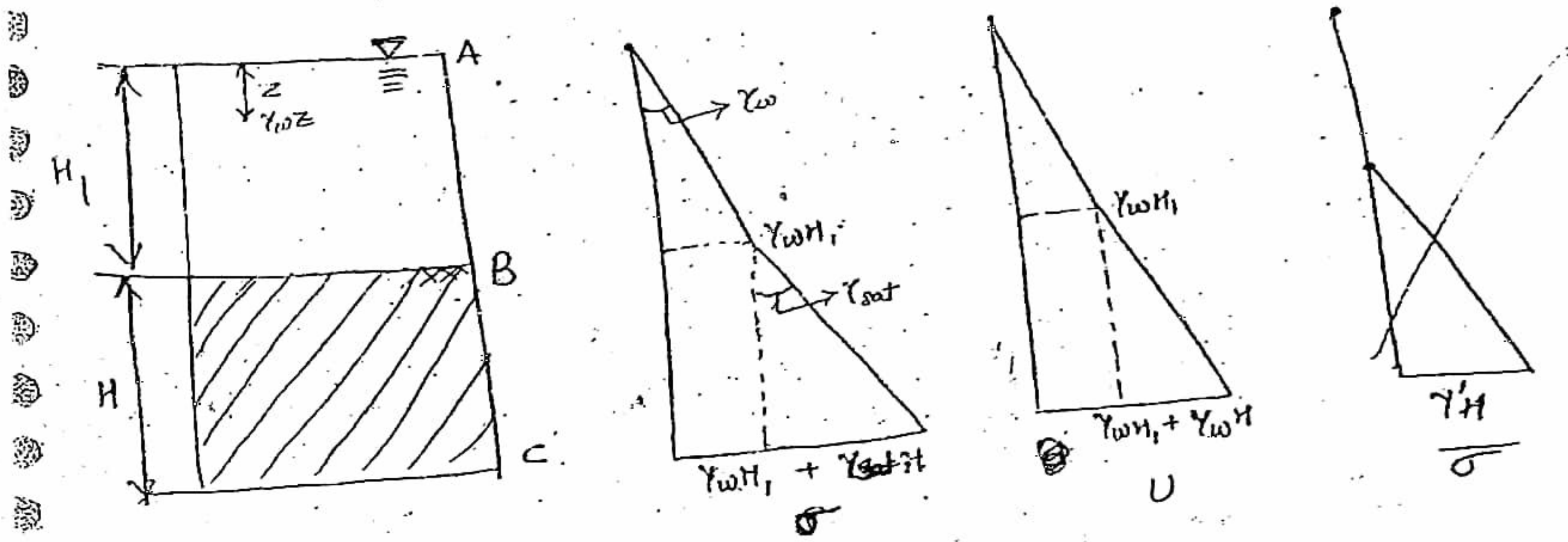
NOTE :- Area of contact (A_c) of soil ~~grain~~ is very small hence, A_c cannot be determined ~~practically~~ hence, (σ'') cannot be determined practically.

Hence actual contact stress $\sigma'' = \dfrac{\Sigma N}{A_c}$

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

$\sigma'' \gg \bar{\sigma}$ ∵ $A_c \ll A$

Case I Submerged Soil mass



Point A	Point B	Point C
$\sigma = 0$	$\sigma = H_1 \gamma_\omega$	$\sigma = \gamma_\omega H_1 + \gamma_{sat} H$
$U = 0$	$U = H_1 \gamma_\omega$	$U = \gamma_\omega (H_1 + H)$
$\bar{\sigma} = 0$	$\bar{\sigma} = 0$	$\bar{\sigma} = \gamma_{sub} H$

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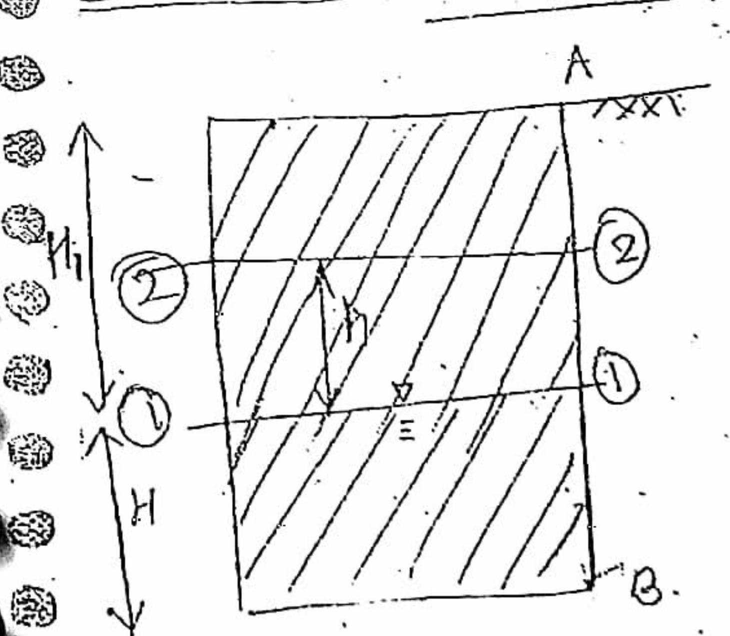
NOTE :- If the water table is above the ground level, then variation in GWT level will not affect effective stress (for eg:- rivers, lakes, ponds)

But if the water table is below the ground level its variation will affect the effective stress

CASE II Water table rising. (GWT below the ground level)



Effective stress at 'B' when WT at (1−1)

$(\bar{\sigma}_B)_{1-1} = \sigma - U$

$= (\gamma_b H_1 + \gamma_{sat} H) - \gamma_\omega H$

$(\bar{\sigma}_B)_{1-1} = \gamma_b H_1 + \gamma' H$

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Effective stresses ~~when~~ at B when GWT is at 2-2

$(\bar{\sigma}_B)_{2-2} = \sigma - u$

$= [\gamma_b(H_1 - h) + \gamma_{sat}(H+h)] - \gamma_w(H+h)$

$= \gamma_b(H_1 - h) + \gamma'(H+h)$

Change in stress

$\Delta(\bar{\sigma}) = (\bar{\sigma}_B)_{2-2} - (\bar{\sigma}_B)_{1-1}$

$= \gamma_b(H_1 - h) + \gamma'(H+h) - \gamma_b(H_1 - h) - \gamma'(H_1 + h)$

$= -\gamma_b h + \gamma' h$

$\boxed{\Delta\sigma = (\gamma' - \gamma_b)h}$ $\qquad \Delta\bar{\sigma} = -ve$

$\because \gamma_b > \gamma'$

implies effective stress ↓es with ↑es in level of water.

NOTE Effective stress will reduce with increase in ground water table if the ground water table is below the ground level & vice versa.

NOTE: If ground water table is lowered due to pumping out of water then

(a) In short term

soil will be treated as saturated in lowered water table zone $\boxed{\Delta\bar{\sigma} = (\gamma_{sat} - \gamma')h}$

(b) in long term (after a long time)

soil will achieve bulk condition in lowered water zone $\boxed{\Delta\bar{\sigma} = (\gamma_{bulk} - \gamma')h}$

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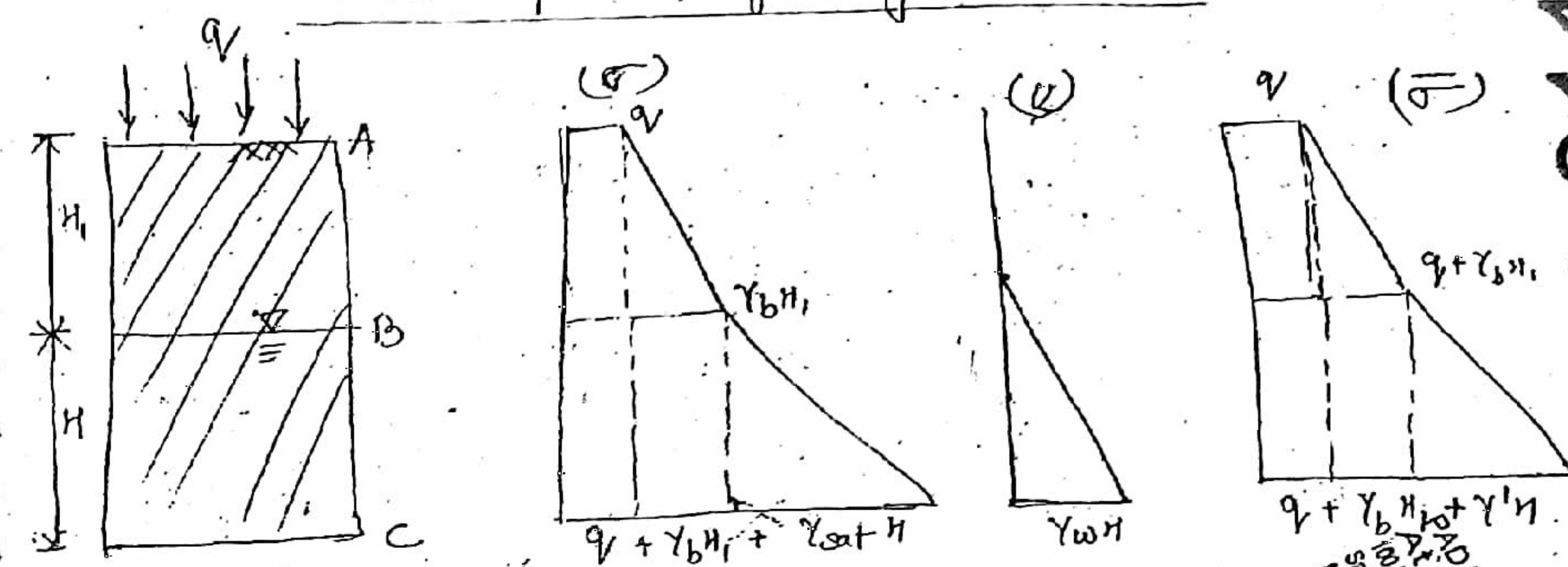
CASE - III Soil Mass with Surcharge & water table at depth H_1 from ground level



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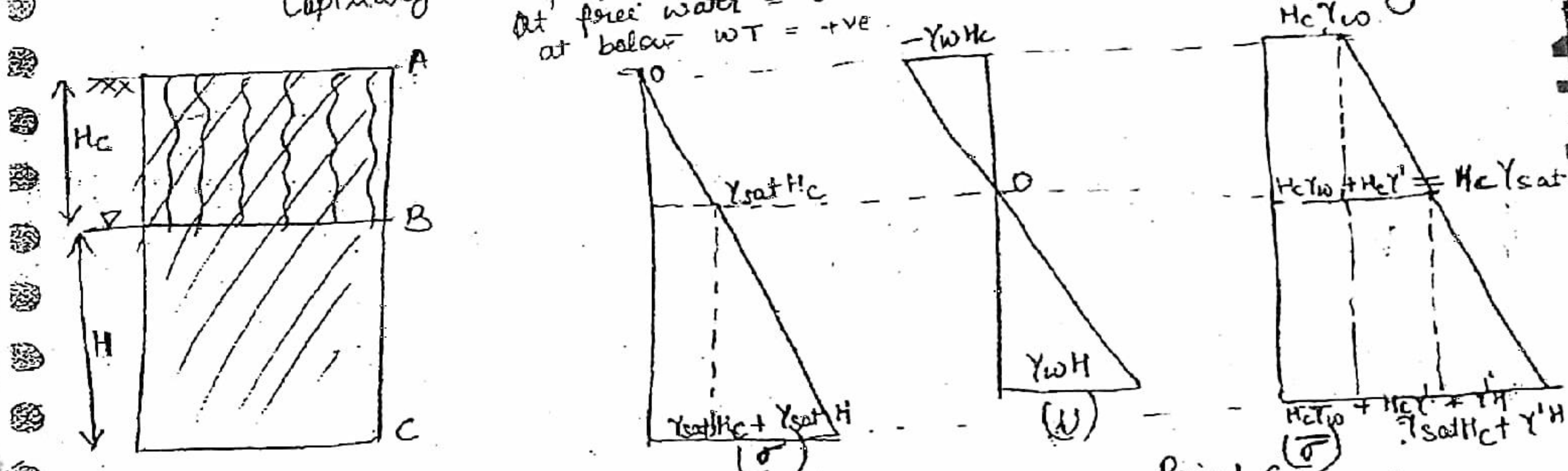
Point A
$\sigma = q$
$u = 0$
$\bar{\sigma} = q$

Point B
$\sigma = q + \gamma_b H_1$
$u = 0$
$\bar{\sigma} = q + \gamma_b H_1$

Point C
$\sigma = q + H_1\gamma_b + \gamma_{sat} H$
$u = \gamma_w H$
$\bar{\sigma} = q + H_1\gamma_b + \gamma' H$

CASE - IV Soil mass with capillary fringes (Soil is saturated above GWT due to capillary rise)

Capillary at top of surface = -ve
At free water = 0
at below WT = +ve



Point A
$\sigma = 0$
$u = -\gamma_w H_c$
$\bar{\sigma} = \sigma - u = 0 - (-\gamma_w H_c)$
$= \gamma_w H_c$

Point B
$\sigma = \gamma_{sat} H_c$
$u = 0$
$\bar{\sigma} = \gamma_{sat} H_c$

Point C
$\sigma = \gamma_{sat} H_c + \gamma_{sat} H$
$u = H\gamma_w$
$\bar{\sigma} = \gamma_{sat} H_c + \gamma' H$

Change in Effective pressure $= H_c \gamma_w$

$\because \gamma_{sat} = \gamma_{sub} + \gamma_w$

$\gamma_{sat} H_c = \gamma_{sub} H_c + \ldots$

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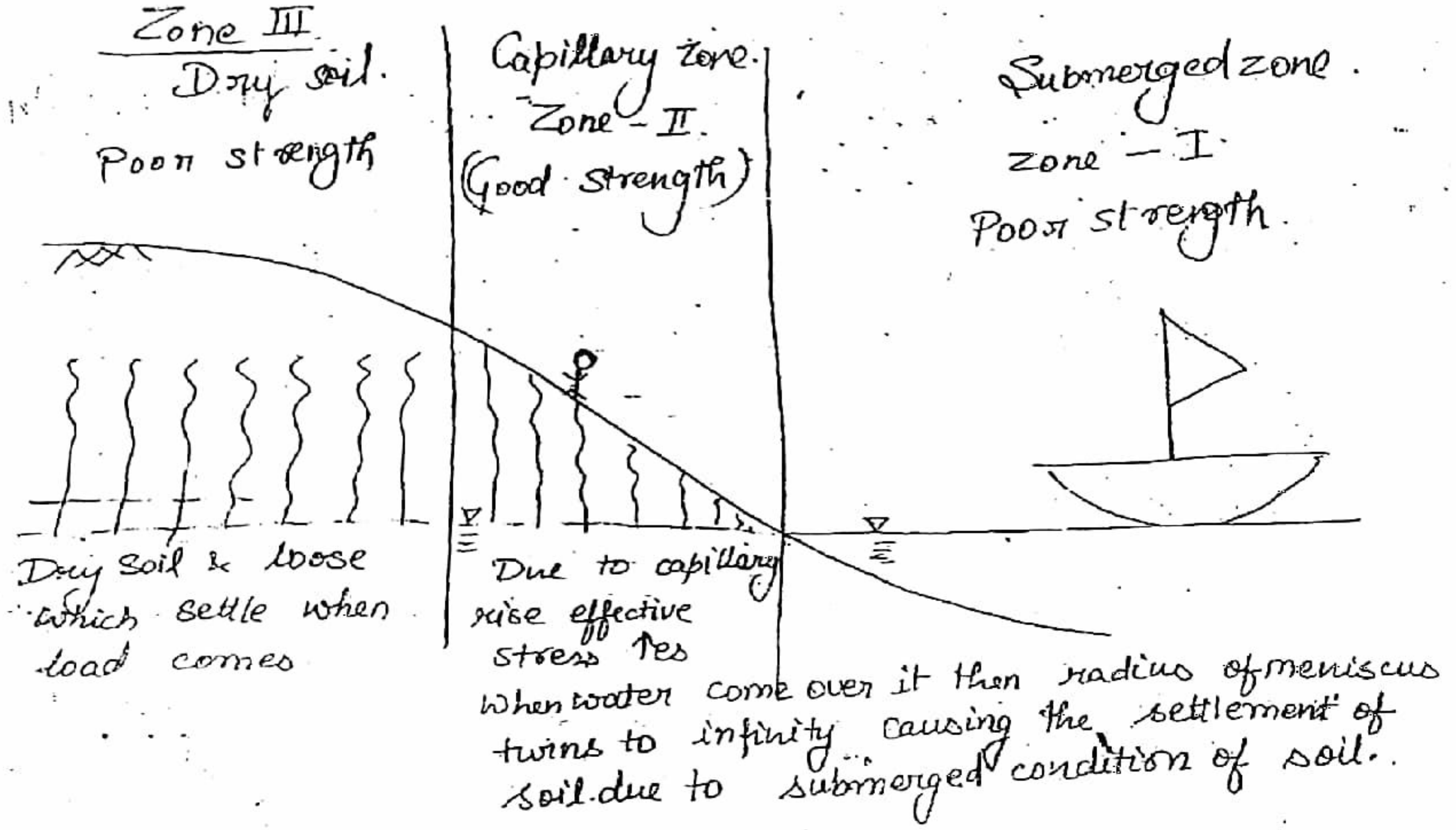
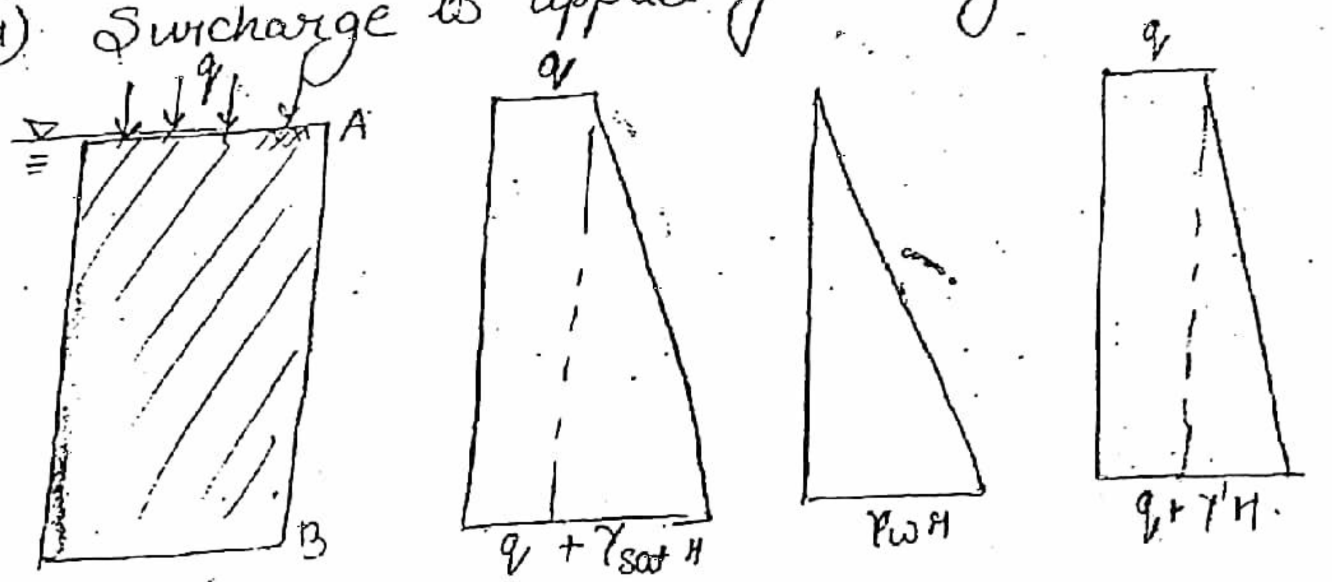
NOTE :- The effect of capillarity is same as in case of surcharge ($q = H_c \gamma_w$), it helps in increasing effective stress.
If the soil would have been saturated above point B due to ground water instead of capillary water, Effective stress in soil would have decreased due to increased pore water pressure.



CASE V : Surcharge is applied on soil mass and water table is on ground level :-

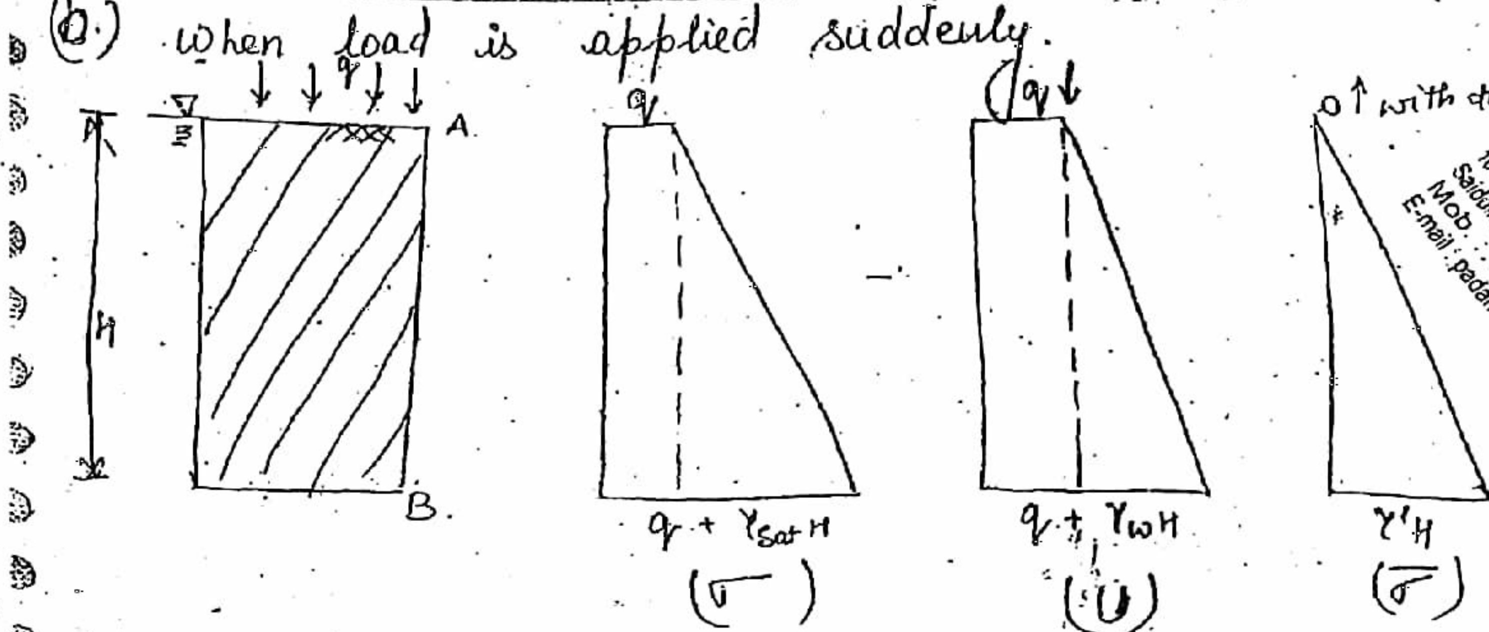
a) Surcharge is applied gradually.



Point A
$\sigma = q$ $U = 0$ $\bar{\sigma} = q$

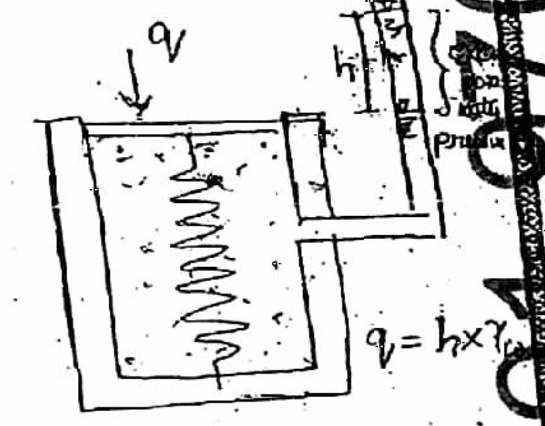
Point B
$\sigma = q + \gamma_{sat} H$
$U = \gamma_w H$
$\bar{\sigma} = q + \gamma' H$

(b) when load is applied suddenly.



At point A.
$\sigma = q$
$U = q$
$\bar{\sigma} = 0$

At point B.
$\sigma = q + \gamma_{sat} H$
$U = q + \gamma_w H$
$\bar{\sigma} = \gamma' H$



NOTE: When water table is present at ground level and surcharge is applied gradually, it is being carried by soil solid.

* But if surcharge is applied suddenly it is carried by water in it which cause ↑es in pore water prⁿ (excess pore water prⁿ) that causes water to seep out from the voids of soil during which water transfer a part of surcharge on the soil solids & when excess pore water pressure is being dissipated then total surcharge is being carried by soil solids.

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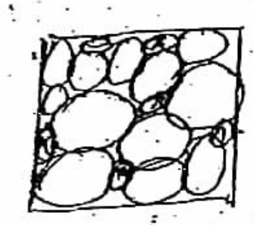
CASE – VI Partially saturated soil.

If the soil is partially saturated soil, air is also present in soil with water hence, in the analysis of effective stress pore air pressure should also be considered along with pore water pressure.



Given by Lambe.

$\bar{\sigma} = \sigma - U_a + X(U_a - U_w)$

U_a = pore air pressure.

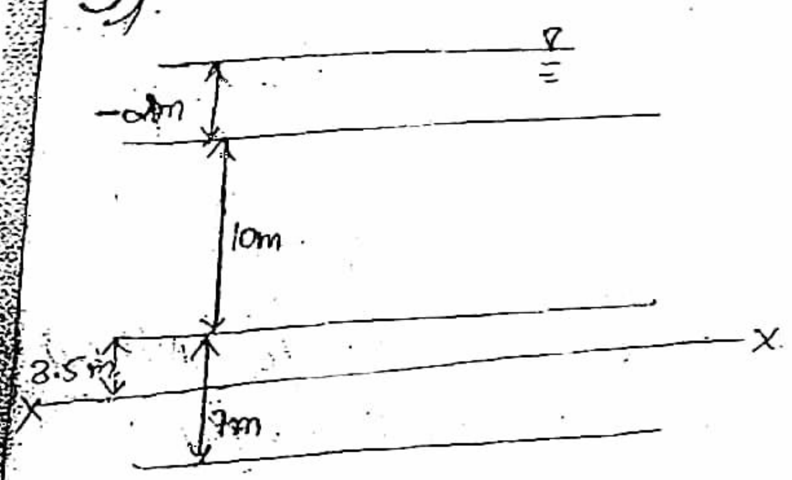
U_w = pore water pressure.

X = fraction of the area of soil occupied by water.

$= \frac{A_w}{A}$

NOTE:- In partially saturated condition pore pressure is neglected in the calculations.

5)



$\bar{\sigma}_1 = 178.445$ $\bar{\sigma}_2 = 198.06$

~~$2\times\gamma_w + 22\times10 + 3.5\times9$~~

~~119.065~~

at XX axis

$\bar{\sigma} = 20$

23) $\bar{\sigma}_1 = 5\times18 + 10\times20 + 5\times20 + 2.5\times20$
$- (10+5+2.5)\,9.81 = 268.325$

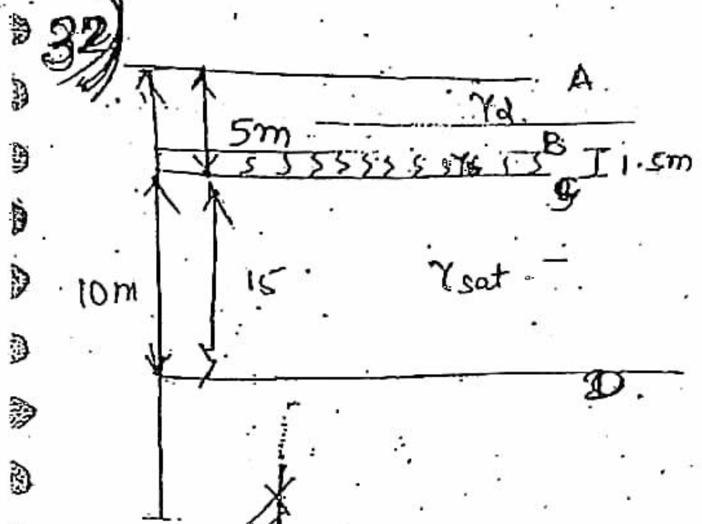
$\bar{\sigma}_2 = 15\times18 + 5\times20 + 2.5\times20 - (5+2.5)\,9.81$

$\Delta\sigma = 78.5\ KN/m^2$

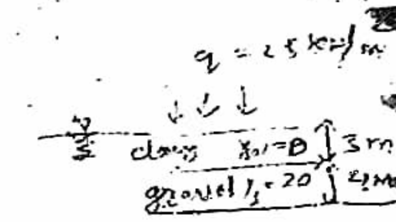
rect $\Delta\sigma = (\gamma_b - \gamma')h$
$= (18 - 10.19)\times10$
$= 78.1$

$\gamma' = 20 - 9.81$
$= 10.19$

32)



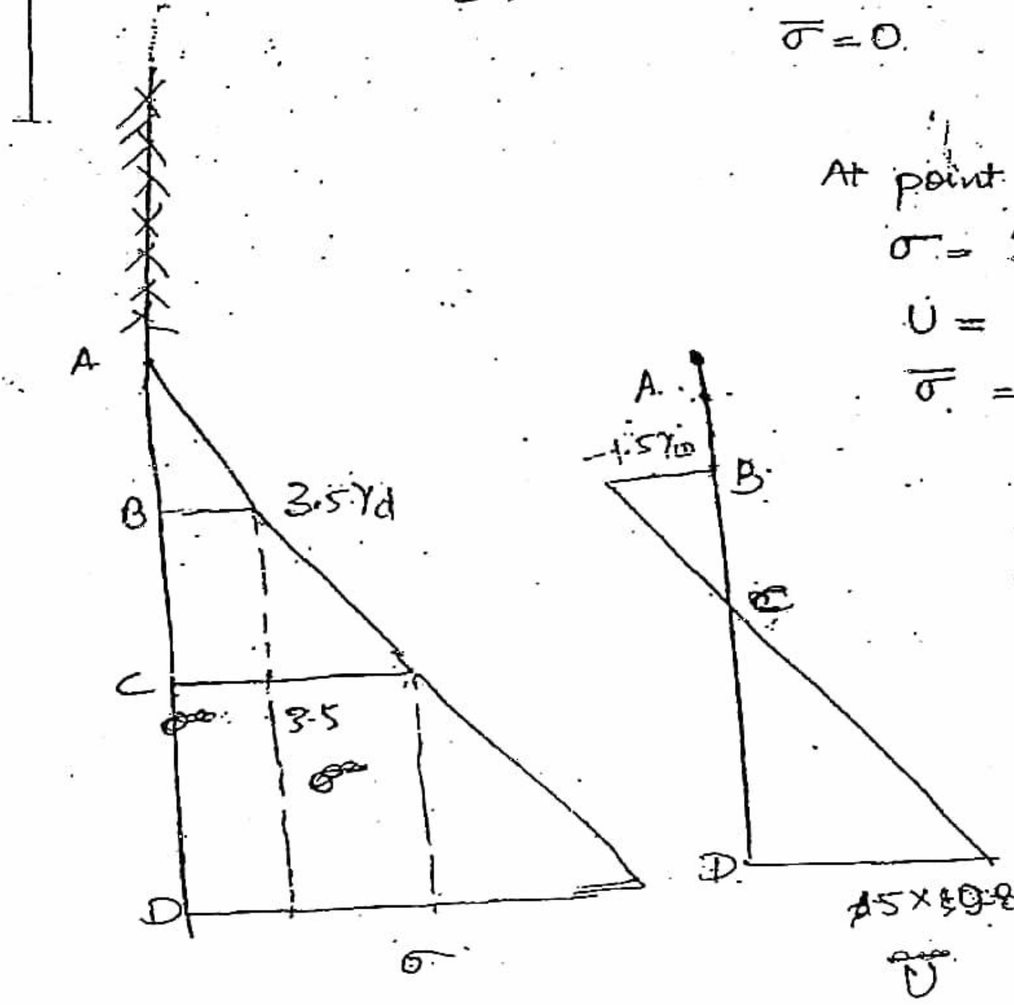
S=0.8
e=0.6
G=2.65
Capillary pr = $-h_c\gamma_w$.

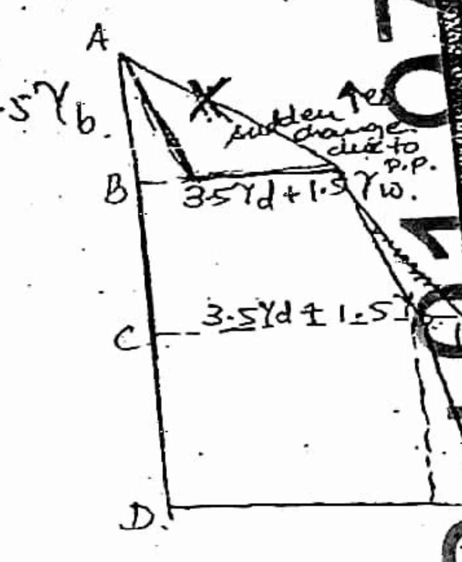


At point A
$\sigma = 0$.
$U = 0$.
$\bar{\sigma} = 0$.

At point B. γ_s
$\sigma = 3.5\gamma_d$.
$U = -1.5\gamma_w$.
$\bar{\sigma} = 3.5\gamma_d + 1.5\gamma_w$.

At point C
$\sigma = 3.5\gamma_d + 1.5\gamma_b$.
$U = 0$.
$\bar{\sigma} = 3.5\gamma_d + 1.5\gamma_b$





$\gamma_b = (G+Se)$

At point D.

$\sigma = 3.5\gamma_d + 1.5\gamma_b + 5\times\gamma_{sat}$

$U = 5\gamma_w$

$\bar{\sigma} = 3.5\gamma_d + 1.5\gamma_b + 5\gamma_{sat} - 5\gamma_w$

$\gamma_d = \frac{G\gamma_w}{1+e} = 16.24\ KN/m^3$

$\gamma_b = \frac{(G+Se)\gamma_w}{1+e} = 19.19\ KN/m^3$

$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w = 19.92\ KN/m^3$

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p – 35   > 60   = 15
p – 15   S 10   35
w_L – 40   X 20   0
I_p – 10   * 20   30

$0.1a + 0.02bd + 0.005ac$

$0.1 \times 15 + 0.02 \times 35 \times 10$

$e_1 = \frac{V_v}{V_s}$

$V_v = V_s$

$0.4 V_s = V_v$

$\therefore V_s = \frac{V_v}{0.4}$

$V_v = \frac{V_i}{0.4}$

$\boxed{V_s = \frac{V_1}{1+e_1}} = \frac{V_2}{1+e_2}$

$\frac{V_1}{1+2} = \frac{V_2}{1.4}$

$\frac{V_1}{V_2} = 1.42$

$V_1 = 1.42 V_2$

$\frac{V_1 - V_2}{V_1}$

$\boxed{V_1 = V_s(1+1)}$

$V_2 = V_{s_2}(1 + 0.60)$

$V_1 = 2V_{s_1}, \quad V_2 = 1.6 V_{s_2}$

$\frac{V_1 - V_2}{V_1} \times 100$

$\frac{2V_{s_1} - 1.6V_{s_2}}{2V_{s_1}} \times 100$

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CHAPTER – 4 PERMEABILITY

* Permeability is the property of soil by virtue of which it allows the flow of fluid ~~of~~ through it.
* Permeability is also termed as hydraulic ~~activity~~ conductivity.
* Permeability of coarse grain soil is compariatively more than that of fine grain soil.

Type of soil	Permeability (cm/sec)
Gravel	> 1
Sand	$1 - 10^{-3}$
Silt	$10^{-3} - 10^{-7}$
Clay	$< 10^{-7}$

Sand is 1000 time more permeable compare to clay.

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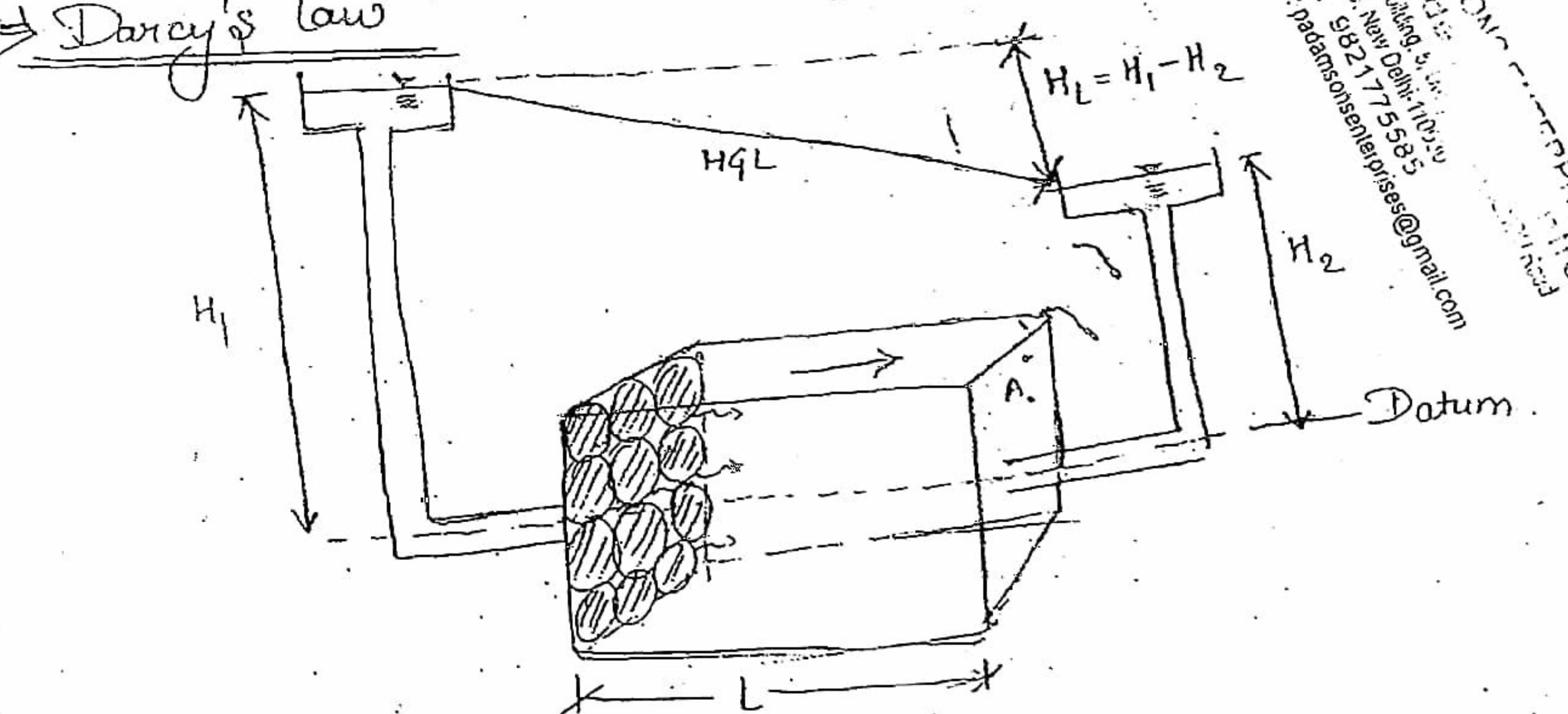
→ Darcy's law



For laminar flow in saturated soil mass, velocity of flow is directly proportional to the hydraulic gradient (i)

$V \propto i$

$\boxed{V = Ki}$

where K = coefficient of permeability
i = hydraulic gradient
$i = \frac{\text{head diff / head loss}}{\text{length}} = \frac{h_L}{L}$

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Discharge : $q = A \cdot v$

$\boxed{q = KiA}$ $\quad \frac{m^3}{sec}, \frac{cm^3}{sec}$

NOTE:- The velocity of flow considered above is average or discharge velocity because total area of c/s is considered.
In actual medium, flow takes place through the interconnecting voids of area of which is much less than total area of c/s hence, actual / true / seepage velocity of flow is much more than average or discharge velocity considered above.

$$q = \frac{Volume}{time} = A \cdot v = A_v \times V_s$$

$$\frac{v}{V_s} = \frac{A_v}{A} = \frac{A_v}{A} \times \frac{L}{L}$$

$$\frac{v}{V_s} = \frac{V_v}{V_T} = \eta$$

V_v = volume of void
V_T = Total volume.

Seepage velocity $\boxed{V_s = \frac{v}{\eta}}$ $\quad \eta$ = porosity

$V_s \propto i$

$V_s = K_p i$

K_p = coeff. of percolation

$$\frac{v}{V_s} = \eta = \frac{Ki}{K_p i}$$

$$\boxed{K_p = \frac{K}{\eta}}$$

Note 1:- $\because \eta < 1$ (always)

$\therefore V_s > v$

Note :- Reynold's No. for laminar flow

Re for flow through pipe $Re \leq 2000$
flow b/w parallel plate $Re \leq 1000$
flow through open channel $Re < 500$
$\boxed{\text{flow through soil} \quad Re < 1}$

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Note :- Darcy's law is not valid in case of gravel because flow through them is turbulent flow.

→ Determination of Permeability

1. Laboratory Method

(a) Constant head permeability test (Suitable for coarse grain soil)
(b) Variable head permeability test (suitable for fine grained soil)
(c) Capillarity permeability test (suitable for partially saturated soil)

2. Indirect Method

(a) Kozney – Carman eqⁿ
(b) Allen Hazen eqⁿ
(c) Consolidation eqⁿ
(d) Terzaghi's eqⁿ
(e) Louden's eqⁿ

3. Field Method

(a) Pumping Out test
(b) Pumping in test

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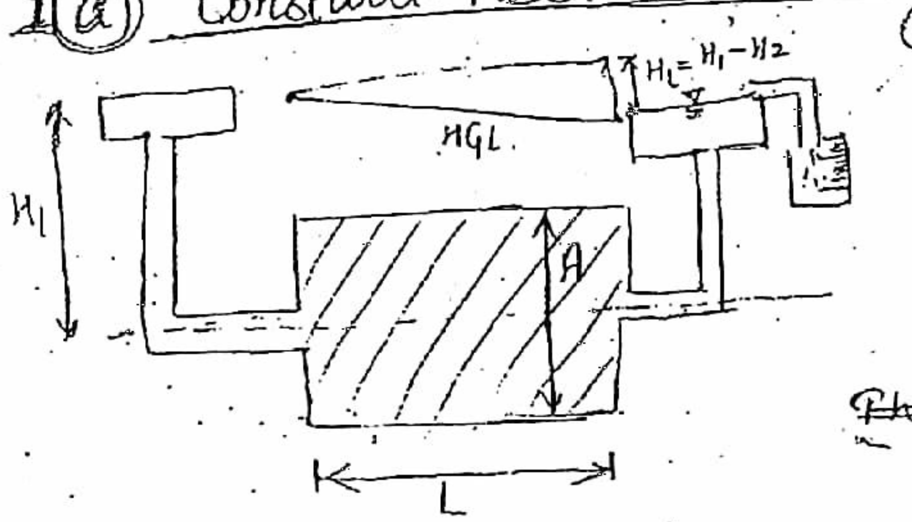
1(a) Constant Head Permeability Test



V volume is collected in time t.

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

$$K = \frac{V}{iAt}$$

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This method is generally used for coarse grained soil for which substantial discharge can be obtained during the test ~~during~~ in time.

In this method water is allowed to flow under constant head.

Let volume 'V' is collected in time 't'.

$$\text{Discharge} = \frac{\text{Volume}}{t} = \frac{V}{t}$$

As per Darcy $q = KiA$.

$$\therefore \boxed{KiA = \frac{V}{t}} \qquad K = \frac{V \cdot L}{t \cdot H_L \cdot A}$$

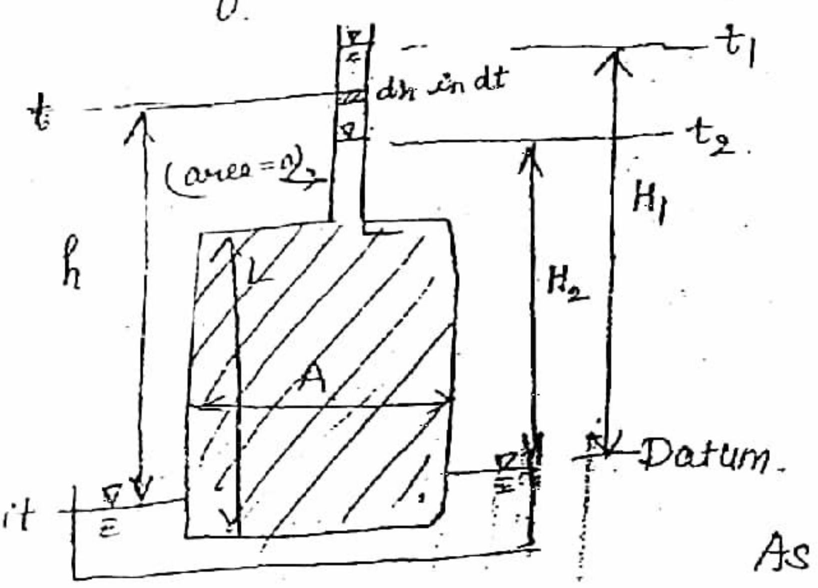
AAA

1(b) Variable Head / Falling Head permeability test

This method is generally used for fine grain soil for which constant head method is not suitable.

In this method, a stand pipe of known area (Area = a) is inserted into the medium & water is allowed to flow through it.

In order to compute the permeability of medium, height of water in standpipe is noted at different times.



Let at time 't' water is moving under head 'h', & it moves 'dh' in time 'dt'

Volume of water flow in dt $(dV) = a \cdot dh$

$$\text{discharge } dq = \frac{dV}{dt} = -a\frac{dh}{dt}$$

As per darcy $q = KiA$.

$$dq = -a\frac{dh}{dt} = KiA$$

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

$$\Rightarrow -a\frac{dh}{dt} = K \cdot \frac{h}{L} \cdot A$$

$$\Rightarrow -a\int_{h_1}^{h_2}\frac{dh}{h} = \frac{K \cdot A}{L}\int_{t_1}^{t_2} dt$$

$$\Rightarrow -a \left| \ln h \right|_{h_1}^{h_2} = \frac{KA}{L} \left| t \right|_{t_1}^{t_2}$$

$$\Rightarrow -a \ln\left(\frac{h_2}{h_1}\right) = \frac{KA}{L}(t_2 - t_1)$$

$$\Rightarrow 2.303\, a \log_{10}\left(\frac{h_1}{h_2}\right) = \frac{KA}{L}(t_2 - t_1)$$

$$\boxed{K = 2.303\,\frac{a}{A} \cdot \frac{L}{t}\log_{10}\left(\frac{h_1}{h_2}\right)} \qquad \boxed{t_2 - t_1 = t \text{ (time interval)}}$$

NOTE :- If in given time interval 't' height of water in stand pipe falls from 'h_1' to 'h_2' & in the same time 't' it falls from 'h_2' to 'h_3' then —

$$K = 2.303\,\frac{a}{A}\,\frac{L}{t}\log_{10}\left(\frac{h_1}{h_2}\right) = 2.303\,\frac{a}{A}\,\frac{L}{t}\log_{10}\left(\frac{h_2}{h_3}\right)$$

$$\frac{h_1}{h_2} = \frac{h_2}{h_3}$$

$$h_2^2 = h_1 h_3$$

$$\boxed{h_2 = \sqrt{h_1 h_3}}$$

It gives the consistency of the readings.

(c) Capillarity Permeability Test

* This test is also known as horizontal capillarity test
* This test is used to find permeability of medium & capillary rise in it.
* It is suitable for partially saturated soil.
* In this test partially saturated sample of soil is placed in a cylindrical glass tube having dia of 4 cm & length of 35

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Horizontal distance travelled by the water in these sample (h) at different times is noted in order to analyse its permeability and capillary rise in it which is determine using following relation:-

$$\boxed{\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2K_u}{S.n}\left(h_0 + h_c\right)}$$

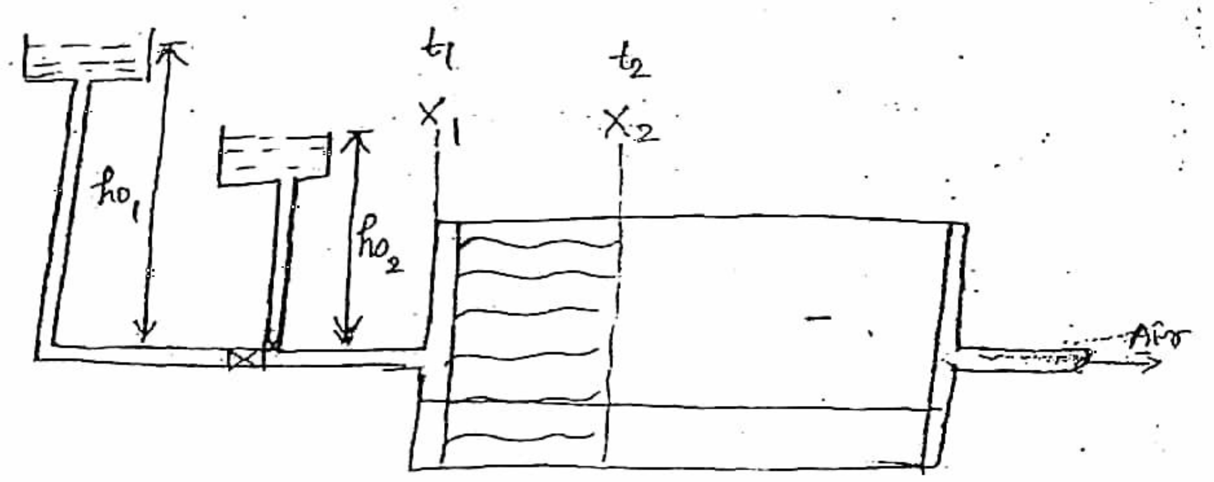
K_u = permeability of partially saturated soil

h_c = capillary rise in soil.

S = degree of saturation

η = porosity

h_0 = head under which flow is taking place



$$\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2K_u}{S.n}\left(h_{o_1} + h_c\right) \quad \text{—— (1)}$$

$$\frac{x_2'^2 - x_1'^2}{t_2' - t_1'} = \frac{2K_u}{S.n}\left(h_{o_2} + h_c\right) \quad \text{—— (2)}$$

∵ Both permeability & capillary rise of the medium has to be determine, two sets of observation are being noted in which water is allowed to flow under constant head h_{o_1} & h_{o_2}.

The capillary is rise is observed in ~~partly~~ partially saturated soil because in completely saturated soil meniscus is destroyed thus no capillary rise.

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2) In direct Method

(a) ** Kozney – Carman Eq^n.

$$K = \frac{1}{K_K}\cdot\left(\frac{\gamma}{\mu}\right)_{fluid}\cdot\frac{e^3}{1+e}\times d^2$$

for water
$$\boxed{K = \frac{1}{K_K}\times\frac{\gamma_\omega}{\mu}\times\frac{e^3}{1+e}\times d^2}$$

Or

$$\boxed{K = \frac{1}{K_K}\times\frac{\gamma_\omega}{\mu}\times\frac{e^3}{1+e}\times\frac{1}{S_s^2}}$$

where K_K = Kozney – Carman constant

γ = unit weight of fluid

μ = dynamic viscosity of fluid

e = void ratio.

d = particle size

S_s = specific surface area.

* for spherical particle.

$S_s = \frac{\text{surface area}}{\text{volume}}$

$= \frac{4\pi r^2}{\frac{4}{3}\pi r^3} = \frac{3}{r} = \frac{6}{d}$

$S_s \propto \frac{1}{d}$.

* If particles are not spherical and it is passing through sieve of size ~~capit~~ A & ~~b~~ retained on sieve of size B then

$$\boxed{S_s = \frac{6}{\sqrt{A.B}}}$$

A
B

(b) *** Allen – Hazen Eq^n

$$K = C D_{10}^2$$

$$\boxed{K = 100\, D_{10}^2} \quad \text{cm/sec}$$

D_{10} → effective size (cm)

C = constant - 100

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(c) Consolidation eq^n

$$\boxed{K = C_v m_v \gamma_w}$$

C_v = coeff. of consolidation
m_v = Coeff. of volume compressibility

(d) Terzaghi's Eq^n

$K = 200\, e^2 D_e^2$ cm/sec

where e = void ratio.
D_e = effective size of sphere for which ratio of volume to surface area is same for given soil particle

soil particle

$(S_s)_{sphere} = (S_s)_{particle}$

$\frac{6}{D_e} = \left(\frac{S.A}{Volume}\right)_{particle}$

(e) Louden's eq^n

$\log_{10}(K_s . S_s^2) = a + b\eta$

$a = 1.3$
$b = 5.1$
η = porosity
S_s = sp. surface area.

3 Field Method

a) Pumping Out test

* This method is suitable for large area of influence
* In this method water is pumped out from the medium to find its permeability.

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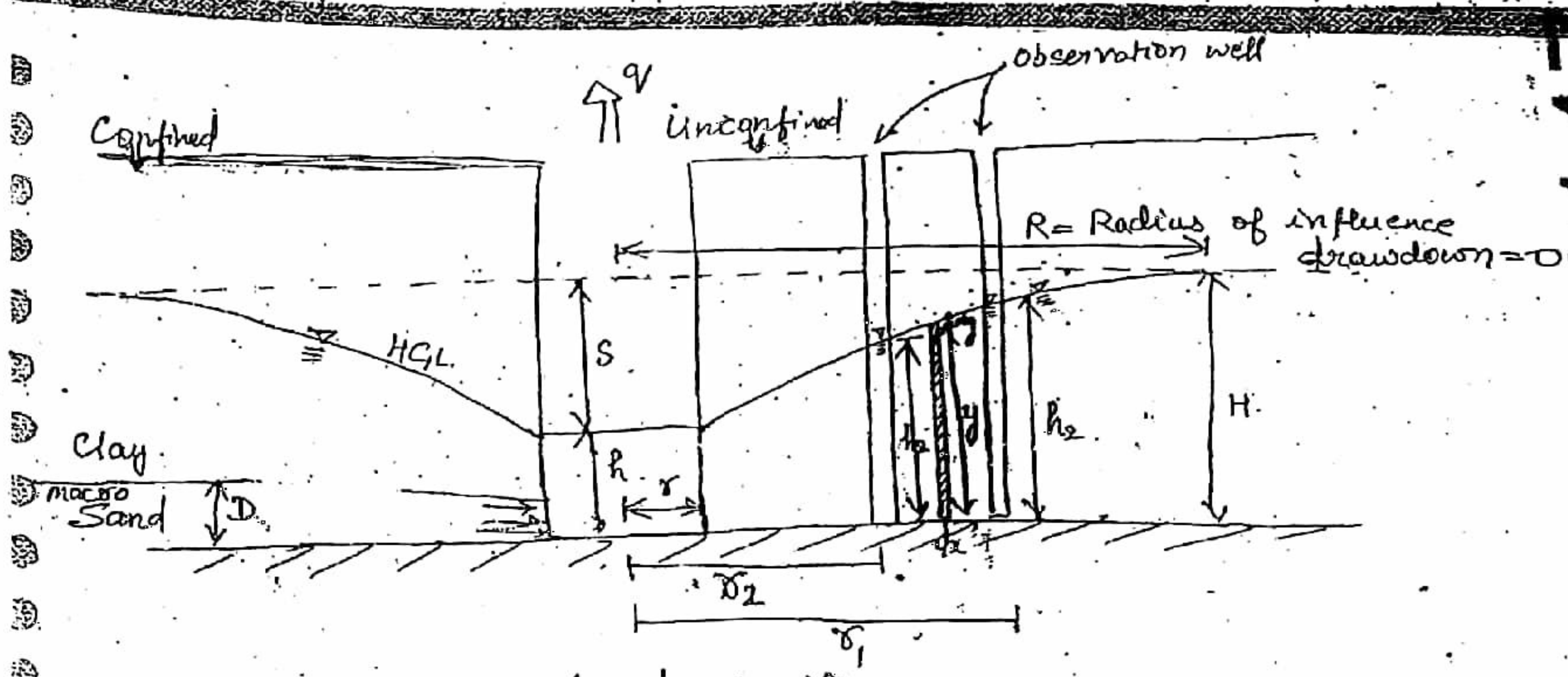

a) Well in unconfined aquifer

1) Thiem's theory

as per darcy $q = KiA$

$q = K.\frac{dy}{dx}.A \quad \left[\because i = \frac{dy}{dx}\right]$

Discharge contributing area

$A = 2\pi x.y$

$q = K.\frac{dy}{dx}.2\pi x.y$

$q\int_{r_1}^{r_2}\frac{dx}{x} = K.2\pi\int_{h_1}^{h_2} y\,dy$

$q\,|\ln x|_{r_1}^{r_2} = \cancel{K.2\pi}\ 2.303q\log_{10}\left(\frac{r_2}{r_1}\right) = K.2\pi\left(\frac{h_2^2}{2}-\frac{h_1^2}{2}\right)$

$$\boxed{K = \frac{2.303q\log\left(\frac{r_1}{r_2}\right)}{\pi(h_1^2-h_1^2)}}$$

2) Dupit's theory

$$\boxed{K = \frac{2.303q\log\left(\frac{R}{r}\right)}{\pi(H^2-h^2)}}$$

R = radius of influence (150 m − 300 m)
$R \approx 3000\, S\sqrt{K}$
S = drawdown.

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b) well in confined aquifer

Theim Theory

$$\boxed{K = \frac{2.303\, q \log\left(\frac{r_1}{r_2}\right)}{2\pi D\,(h_1 - h_2)}}$$

Dupit theory

$$\boxed{K = \frac{2.303\, q \log\left(\frac{R}{r}\right)}{2\pi D\,(H-h)}}$$

$\boxed{h_1 - h_2 = S}$ drawdown

NOTE :-

1. For accurate result theim's theory is used but the work involved for collecting the observation is more.

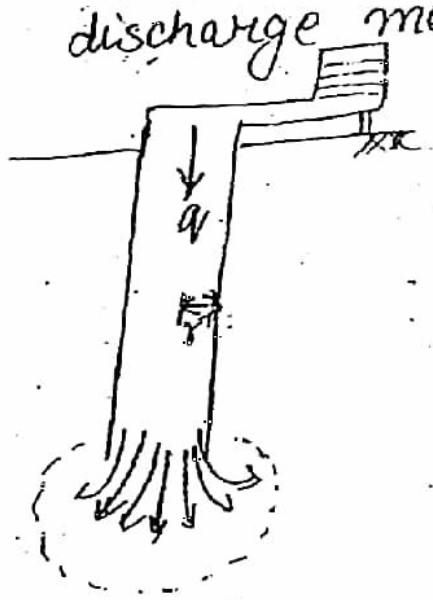
3 b) Pumping In Test

This test is suitable for small area of influence in which water is pumped into the medium in order to find its permeability.

(i) Open End test

In this test an open end pipe is sunk into the medium and soil from the pipe is taken out.

Water is pumped into the medium through the pipe by discharge measuring device under constant head.



USBR 1961 $\boxed{K = \frac{q}{5.5\, r h}}$

r = radius of well.

h = head under which flow takes place

= gravity head + pressure head (if any)

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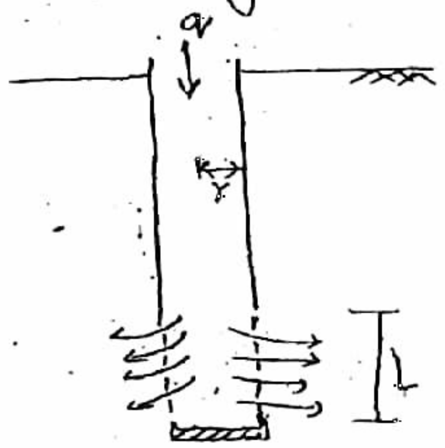
* Spherical flow will be there.

(ii) Packer's test

In this test pipe is plugged at the bottom & perforated over length 'L' at its surface from bottom, is sunk into the medium & water is allowed to flow through it under constant head.



if $L > 10r$

$$K = \frac{q}{2\pi L h} \log_{10}\left(\frac{L}{2r}\right)$$

if $r \le L \le 10r$

$$K = \frac{q}{2\pi L h} \sinh^{-1}\left(\frac{L}{2r}\right)$$

Factors Affecting Permeability.

1.) Particle size

$\boxed{K \propto d^2}$ or $\boxed{K \propto D_{10}^2}$

Coarse grain have permeability more than fine grain

2.) Shape of particle

$\boxed{K \propto \frac{1}{S_s^2}}$

3) Void ratio

$\boxed{K \propto \frac{e^3}{1+e}}$ or $\boxed{K \propto e^2}$

Keeping all other factors constant (particle size kept constant)

d → constant. (for Kozney)

D_e → constant (for Terzaghi's)

for example :- Loose sand have more permeability than that of dense sand because loose sand have high void ratio in comparison to dense sand (particle size is same)

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4.) Fluid property

$$\boxed{K \propto \left(\frac{\gamma}{\mu}\right)_{fluid}}$$

$K \propto \frac{\gamma_w}{\mu}$ for Water

$\because \mu \propto \frac{1}{T^\circ C}$

$$\boxed{K \propto T^\circ C}$$

Note :-

It is difficult to compare & analyse the 2 soil sample because permeability is dependent on both medium properties & fluid properties thus coefficient of intrinsic permeability or absolute permeability is use which dependent only upon medium properties.

$$K = f(\text{fluid properties})(\text{Medium properties})$$

Intrinsic Permeability

$$K_0 = f(\text{medium property})$$

$$\boxed{K_0 = \frac{K}{\left(\frac{\gamma}{\mu}\right)_{fluid}}}$$

K cm/sec; $\frac{\text{cm/sec}}{\text{N/m}^3 \,/\, \text{Nm/sec}^2} = \frac{m^2}{m^2}$

$$\frac{\text{cm/sec}}{\dfrac{\text{N/m}^3}{\text{Nm/sec}^2}}$$

5) Degree of Saturation

$$\boxed{K \propto \text{Degree of Saturation}}$$

because higher is the S lower is the presence of air which results reduced resistance in the form of air block.

6) Entrapped gases

$$\boxed{K \propto \frac{1}{\text{entrapped gases}}}$$

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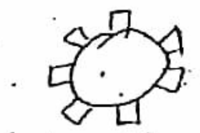
7) Impurities

$$\boxed{K \propto \frac{1}{\text{impurities}}}$$

8) Adsorbed Water

$$\boxed{K \propto \frac{1}{\text{Adsorbed Water}}}$$



Higher is the presence of adsorbed water in the medium lower is its permeability because lower is the area available to the flow of the fluid.

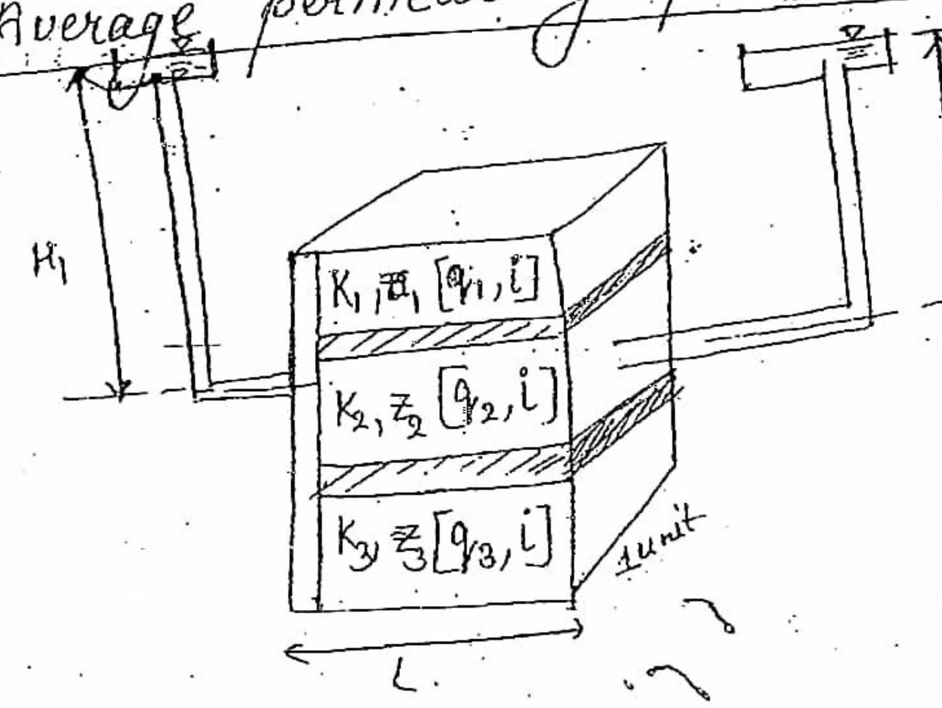
It is assumed that in the presence of adsorbed water void ratio is approximately reduced by 0.1

8) Structure of Soil

Permeability parallel to bedding plane is more than the permeability perpendicular to the bedding plan.

Permeability parallel to Bedding plane

IES (a) Average permeability parallel to Bedding plane



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Total discharge $q = q_1 + q_2 + q_3 + \text{----} \quad (A)$

As per darcy $q = K_{avg} \cdot i \cdot \{(z_1 + z_2 + z_3 + \text{---})\} * 1$

$$q = K_{avg} \cdot i \cdot \Sigma z_i$$

Individual layer

$q_1 = K_1 \, i \, A = K_1 \cdot i \cdot z_1 * 1$

$q_2 = K_2 \, i \, z_2$

$q_3 = K_3 \, i \, z_3$.

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from eqⁿ A.

$q = q_1 + q_2 + q_3 + \cdots$

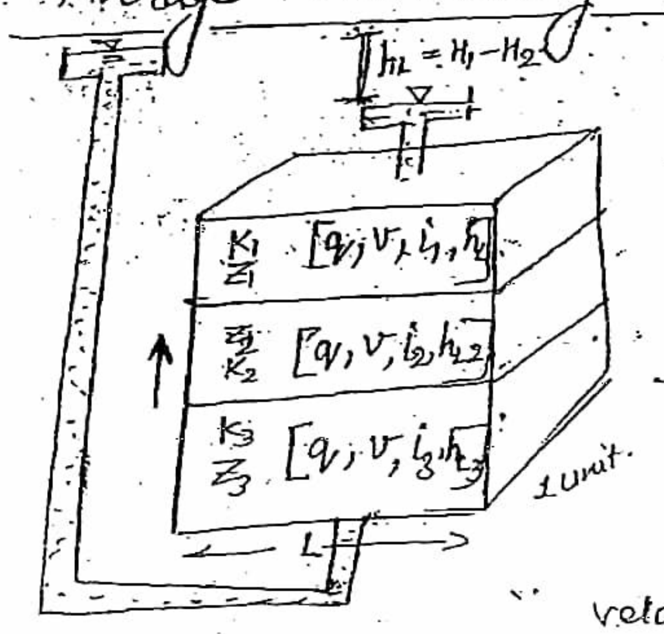
$K_{avg_{II}} \cdot \cancel{i} \cdot (\Sigma z_i) = K_1 \cancel{i} z_1 + K_2 \cancel{i} z_2 + K_3 \cancel{i} z_3 + \cdots$

$$K_{avg_{II}} = \frac{K_1 z_1 + K_2 z_2 + K_3 z_3 + \cdots}{z_1 + z_2 + z_3 + \cdots}$$

$$\boxed{K_{avg_{II}} = \frac{\Sigma K_i z_i}{\Sigma z_i}}$$

(b) Average Permeability perpendicular to bedding plane

Discharge will be same.



Total head loss.

$H_L = h_1 + h_2 + h_3 + \cdots$ (A)

Hydraulic Gradient $= i = \frac{H_L}{\text{Length}}$

$$= \frac{H_L}{z_1 + z_2 + z_3 + \cdots}$$

$$i = \frac{H_L}{\Sigma z_i}$$

velocity $= Ki = K_{avg} \cdot i = K_{avg} \cdot \frac{H_L}{\Sigma z_i}$ — (B)

For individual layer.

$i_1 = \frac{h_1}{z_1} \cdot i_2 = \frac{h_2}{z_2} \quad i_3 = \frac{h_3}{z_3}$ & ---

$V = K_1 i_1 = K_2 i_2 = K_3 i_3 = \cdots$

$V = K_1 \cdot \frac{h_1}{z_1} = K_2 \cdot \frac{h_2}{z_2} = K_3 \frac{h_3}{z_3} = \cdots$ — (C)

from (A) $H_L = h_1 + h_2 + h_3 + \cdots$

from (B) & (C)

$$H_L = \frac{V \cdot \Sigma z_i}{K_{avg}} = \frac{\cancel{V} z_1}{K_1} + \frac{\cancel{V} z_2}{K_2} + \frac{\cancel{V} z_3}{K_3} + \cdots$$

$$K_{avg} = \frac{z_1 + z_2 + z_3 \cdots}{\frac{z_1}{K_1} + \frac{z_2}{K_2} + \frac{z_3}{K_3} + \cdots} \rightarrow \boxed{K_{avg} = \frac{\Sigma z_i}{\Sigma \frac{z_i}{K_i}}}$$

WORKBOOK

13) $h_{LB} = 19 h_{LA}$. $K_A = 3 \times 10^{-5}$ $K_B = ?$

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

Discharge Same

$$\frac{K_A}{K_B} = \frac{\left(\frac{h_L}{L}\right)_B}{\left(\frac{h_L}{L}\right)_A} \Rightarrow \frac{3 \times 10^{-5}}{K_B} = \frac{19}{1}$$

$K_B = 0.15 \times 10^{-5}$

$= 1.5 \times 10^{-6}$

15) $\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2K_u}{Sn}(h_0 + h_c)$

$\frac{7^2 - 1.5^2}{7} = \frac{2K_u}{Sn}(60 + h_c)$

$\frac{18.5^2 - 7^2}{24} = \frac{2K_u}{Sn}(180 + h_c)$

$0.54 \times \quad 98.38 + 0.54 h_c = 60 + h_c$

$38.38 \quad h_c = 84.65$ mm

$K = 6.86 \times 10^{-3}$ cm/min.

$12.2 = \quad K_u(180$

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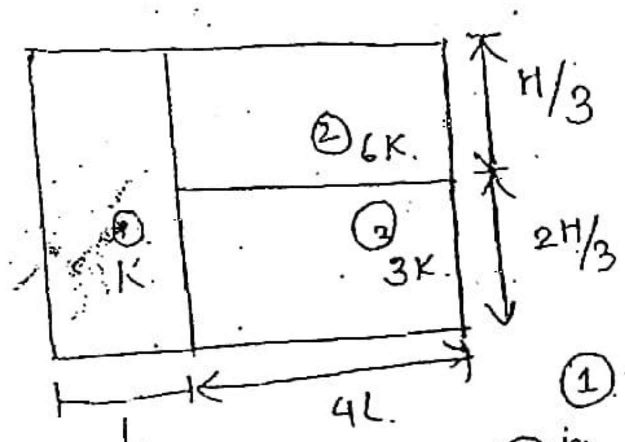
16) $e_A = 2e_B \qquad D_{eA} = \frac{1}{3} D_{eB}$

$$\frac{h_{cA}}{h_{cB}} = \frac{\frac{C}{e_A D_{eA}}}{\frac{C}{e_B D_{eB}}} = \frac{3 e_B D_B}{2 e_B \times D_B}$$

28)



$K_{avg} = \frac{\Sigma H_i}{\Sigma \frac{H_i}{K_i}} = \frac{L + 4L}{\frac{L}{K} + \frac{4L}{9K}}$

$= \frac{5\cancel{L} \times 9K}{13 \cancel{L}} = \frac{45K}{13}$

① — ②

① K_{avg} ② — ③ $= \frac{\frac{6KH}{3} + \frac{2H}{3} \times 3K}{\frac{H}{3} + \frac{2H}{3}}$

② $\frac{5L}{\frac{L}{K} + \frac{4L}{4K}} = \frac{5}{2}K \qquad = 4K$

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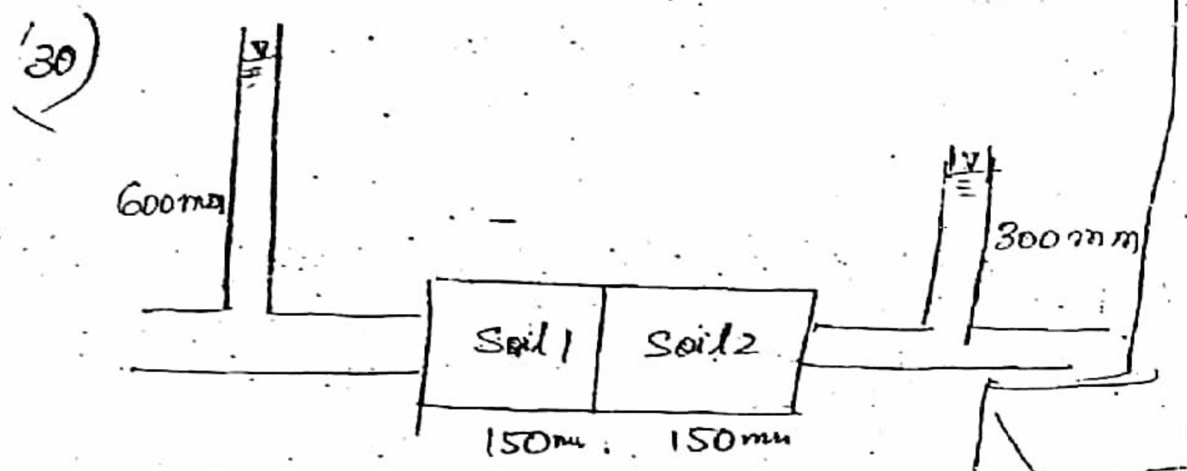
30)



$H_L = 300\ mm$

$A = 80cm^2 = 80\times10^4\ mm^2$

$t = 15\ min$

$q = \boxed{200\ ml} = 2\times10^5\ mm^3$

$K_1 = 0.02\ mm/sec$

$\frac{200\ ml}{15\ min} = q_{avg} = q_1 = q_2$

$\Rightarrow K_{avg}\cdot i\cdot A = K_{avg}\left(\frac{300}{300}\right)\times 80cm^2$

$\frac{200000\ mm^3}{15\times60\ sec} = \left[\frac{150+150}{\frac{150}{0.02}+\frac{150}{K_2}}\right]\times 80cm^2$

$\frac{150}{0.02} + \frac{150}{K_2} = 108000$

$K_2 = 0.045\ mm/sec$

$q = K\boxed{iA} = K\boxed{iA}$

$i = \frac{2\times10^4}{0.02\times80\times10^4}$

$i = 1.25$

$K = \frac{q}{iA} = \frac{2\times10^5}{1.25\times80\times10^4}$

$q = \frac{200}{15} = \frac{2\times10^5}{15\times60}$

$q = 22.22\ mm^3/sec = K\,i\,A$

$\Rightarrow$ ~~$K = 0.028$~~

34)

500m

$h_L = (92-42)m = 50\ m$

$e = 0.9$ $\quad K = 1\times10^{-3}\ m/s$

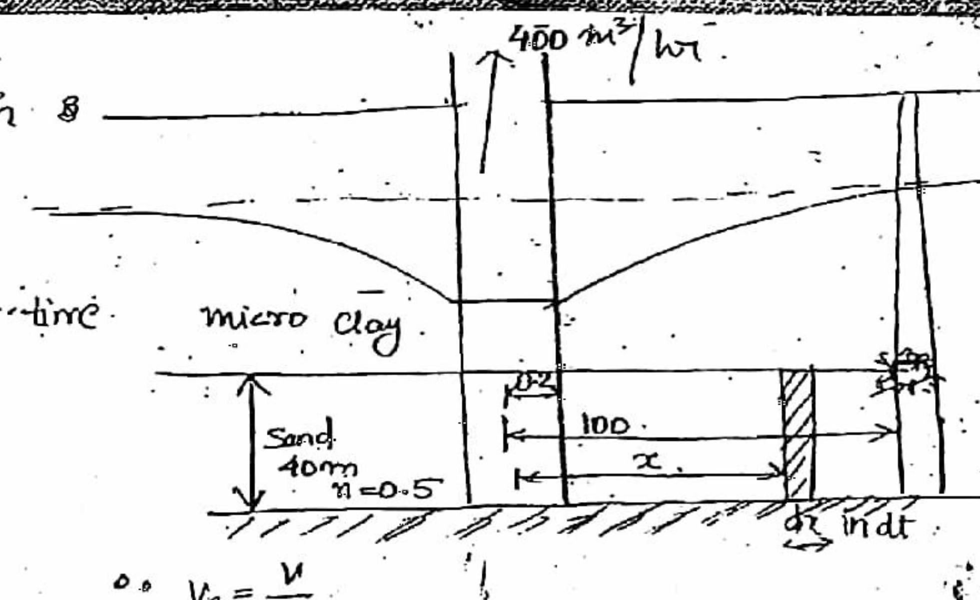
$t = ?$ $\quad V_s = \frac{v}{n} = \frac{Ki}{n} = \frac{10^{-3}\times0.1}{0.47}$

$V_s = 2.1\times10^{-4}$ sec

$t =$ 28 days $\approx$ 27.4 days

35)

Tracer will move with seepage velocity.



Let tracer move 'dx' distance in dt time

Velocity $= \frac{-dx}{dt}$

As per darcy

$q = A\cdot V$
$\quad = A\cdot(V_s\cdot n)$ $\quad \therefore V_s = \frac{v}{n}$

Discharge contributing area. $\quad A = 2\pi x\cdot d$

$q = (2\pi x\cdot d)\times(V_s)\times n = 2\pi x d\times\frac{-dx}{dt}\cdot n$

$= 2\pi n d\,\frac{-x\,dx}{dt}$

$\int_0^t q\cdot dt = -2\pi n d\int_{100}^{0.2} x\,dx = -2\pi n d\left.\frac{x^2}{2}\right|_{100}^{0.2}$

$= 2\pi n d\,\frac{(100^2-0.2^2)}{2}$

$qt = \pi n d\,(100^2-0.2^2)$

$t = \frac{\pi\times0.5\times40\times(100^2-0.2^2)}{400\ \frac{m^3}{hr}} \Rightarrow t = 1570.79\ hrs$

$t = 65.45\ day$

$R.C = \frac{\gamma_d\ field}{\gamma_d\ lab\ max}$

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Seepage Analysis | Important

When water flows through saturated soil mass total head at any point consist of pressure head, datum head (elevation head), and velocity head.

∵ flow through the soil mass is considerably less, velocity head is neglected in the soil hence, total head at any point consist of datum head & pressure head which is also termed as hydraulic head.

Total head (hydraulic head) = Pressure head + Datum head.

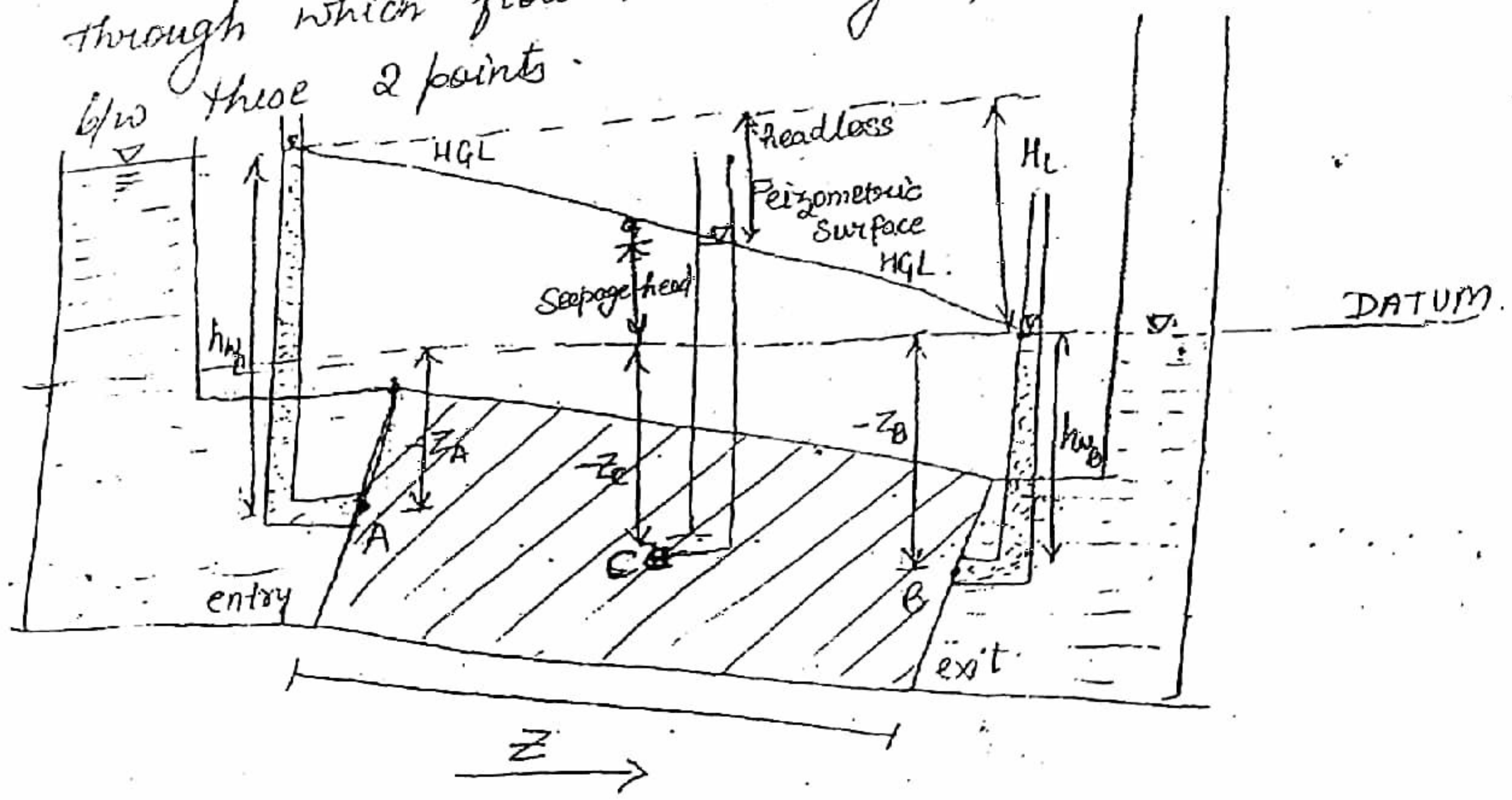
$$\boxed{H = h_w \pm Z}$$

* Elevation head at a point is the vertical distance of that point measured from an assumed ~~from~~ datum plane which is normally taken at the tail waterlevel elevation for ~~convinience~~ convenience.

* If a peizometer or an open stand pipe is inserted at a point of flow, water would stand upto a particular height inside the peizometer termed as pressure head.

Total head = Pressure head + Datum head.

The difference b/w total head at any 2 points in a soil through which flow is occuring represents the head loss b/w these 2 points.



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	A (Entry)	(exit) B	C
Datum head	$-Z_A$	$-Z_B$	$-Z_C$
Pressure head	h_{wA}	h_{wB}	h_{wC}
Total head (hydraulic head)	$h_{wA} - Z_A$	$h_{wB} - Z_B = 0$ ($\because h_{wB} = Z_B$)	$h_{wC} - Z_C$

Hydraulic head difference under which flow is taking place (Seepage head)

$$h = H_{entry} - H_{exit} = H_A - H_B$$

$$h = (h_{wA} - Z_A) - (\cancel{h_{wB} - Z_B})^{0}$$

$$h = (h_{wA} - Z_A)$$

Hydraulic gradient $(i) = \frac{\text{Head diff}}{\text{Length}} = \frac{\text{Head Loss}}{\text{Length}}$

$$i = \frac{H_{entry} - H_{exit}}{\text{Length}} = \frac{H_L}{\text{length}}$$

$$\boxed{i = \frac{h}{Z}}$$

→ Seepage Pressure

When water flows through saturated soil mass, it exert pressure over soil skeleton by virtue of frictional drag this pressure exerted by water is termed as seepage pressure.

If 'h' is the hydraulic head under which seepage takes place (or seepage head) then seepage pressure is given as:

seepage pressure $(p_s) = h\gamma_w$
$= \text{seepage head} \times \gamma_w$.

we know that $i = \frac{h}{Z}$

$$\boxed{p_s = iZ\gamma_w}$$

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Seepage force $P_s = p_s \cdot A = iZ\gamma_w \cdot A$

Sp. Seepage force $P_{ss} = \dfrac{iZ\gamma_w A}{\text{Volume}} = \dfrac{iZ\gamma_w A}{Z \cdot A}$

$$\boxed{P_{ss} = i\gamma_w}$$

NOTE :- * Seepage pressure always acts in the direction of flow. vertical effective stress may be increased or decreased due to seepage, depending upon the direction of flow.

1) If flow takes place in vertically downward direction, effective stress is ~~stress~~ increased because seepage pressure also acts in downward direction.

2) If flow take place in vertically upward direction effective stress is to reduce b/c seepage prⁿ act in upward direction.

Effective Stress $= Z\gamma' \pm p_s$

$= Z\gamma' \pm iZ\gamma_w$

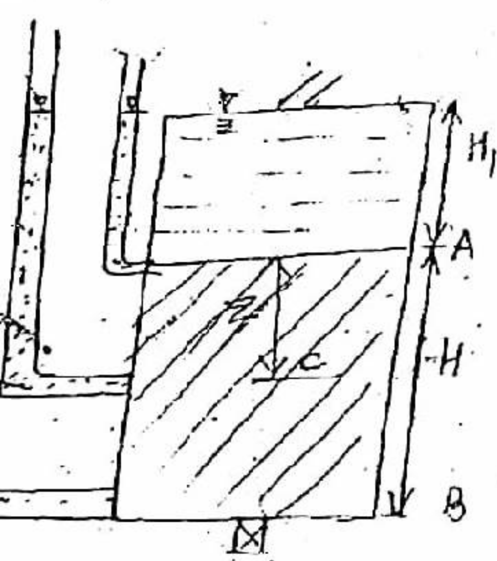
if flow is upward

$$\boxed{\bar{\sigma} = Z\gamma' - iZ\gamma_w}$$

if flow is downward

$$\boxed{\bar{\sigma} = Z\gamma' + iZ\gamma_w}$$

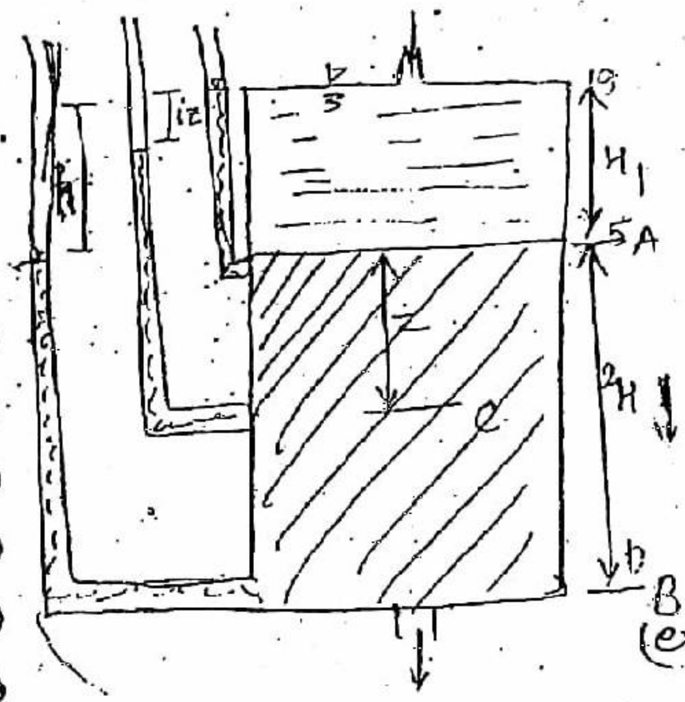
No flow Condition



Datum

	A	B	C
σ	$\gamma_w H_1$	$\gamma_w H_1 + \gamma_{sat} H$	$\gamma_w H_1 + \gamma_{sat} Z$
U (Pressᴿ head $\times \gamma_w$)	$\gamma_w H_1$	$(H_1 + H)\gamma_w$	$(H_1 + Z)\gamma_w$
$\bar{\sigma}$	0	$\gamma' H$	$\boxed{\gamma' Z}$

Case 2 ~~Downard~~ Downward flow Condition



	A	B	C
Datum head	H	0	$H - Z$
Pressure head	H_1	$H + H_1 - h$	$H_1 + Z - iZ$
Total head	$H + H_1$	$H + H_1 - h$	$H + H_1 - iZ$

Hydraulic head under which flow is taking place

$= H_{entry} - H_{exit} = (H + H_1) - (H + H_1 - h)$

Hydraulic gradient $i = \dfrac{\text{Head diff}}{\text{length}} = \dfrac{h}{H}$

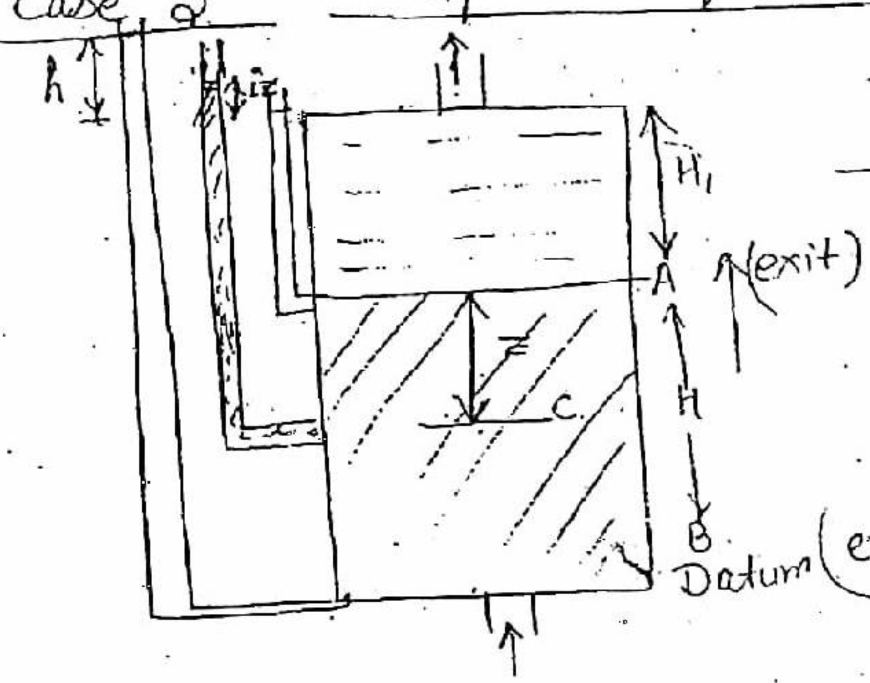
Hydraulic gradient $i = \dfrac{\text{head loss}}{\text{length}} = \dfrac{(h_L)_{entry \to C}}{Z}$

$(h_L)_{entry \to C} = iZ$

$(H)_{entry} - (H)_C = iZ$

	A	B	C
σ	$H_1\gamma_w$	$\gamma_{sat} H + \gamma_w H_1$	$\gamma_w H_1 + \gamma_{sat} Z$
$U = P.H \times \gamma_w$	$H_1\gamma_w$	$(H + H_1 - h)\gamma_w$	$(H_1 + Z - iZ)\gamma_w$
$\bar{\sigma}$	0	$H\gamma' + h\gamma_w$	$\boxed{Z\gamma' + iZ\gamma_w}$

Case 3 Upward flow Condition



	A	B	C
Datum head	H	0	$H - Z$
Pressure head	H_1	$H + H_1 + h$	$Z + H_1 + iZ$
Total head	$H + H_1$	$H + H_1 + h$	$H + H_1 + iZ$

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Hydraulic head under which flow takes place

$= H_{entry} - H_{exit} = (H+H_1+h) - (H+H_1)$

$= h$

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

Hydraulic gradient $i = \frac{\text{Head loss}}{\text{length}} = \frac{(h_L)_{c \to exit}}{z}$

$(h_a)_c - (H_a)_{exit} = iz$

	A	B	C
σ	$\gamma_w H_1$	$\gamma_{sat} H + \gamma_w H_1$	$\gamma_w H_1 + \gamma_{sat} z$
U	$H_1 \gamma_w$	$(H+H_1+h)\gamma_w$	$(H_1+z+iz)\gamma_w$
$\bar{\sigma}$	0	$H\gamma' - h\gamma_w$	$\boxed{\gamma' z - iz\gamma_w}$

Special Case

When flow takes place in upward direction seepage pressure also act in upward direction & effective stress is reduced.

If seepage pressure equals the submerged wt of soil mass then, effective stress reduced to zero.

In such case cohesionless soil mass losses all its shear strength & hence have the tendency to flow along with the water. This phenomena in which soil particles leaves the soil mass & flow along with the water is termed as quick sand, piping, boiling or floating condition.

for quick sand condn. — Effective Stress approach

$\bar{\sigma} = z\gamma' - p_s = 0$

$z\gamma' = p_s = iz\gamma_w$

$\gamma' = i\gamma_w$

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$i_c = \frac{\gamma'}{\gamma_w} = \left(\frac{G-1}{1+e}\right)\frac{\gamma_w}{\gamma_w}$ downward submerged = seepage pressure

$$\boxed{i_c = \frac{G-1}{1+e}}$$

This hydraulic gradient at which quick sand condn. occurs is critical gradient, piping gradient, floating or bursting gradient.

* To avoid quick sand condition hydraulic gradient should always be less than critical hydraulic gradient

$i < i_c$

$$\boxed{FOS = \frac{i_c}{i}}$$

* For fine sand for which specific gravity is 2.65 & void ratio is approximately 0.65 then approximately critical hydraulic gradient is 1.

$i_c = \frac{G-1}{1+e} = \frac{2.65-1}{1+0.65} \boxed{\cong 1}$

NOTE:- Quick sand is not a type of sand but is a flow condition in cohesionless soil mass when effective stress is reduced to zero due to upward flow condition.

Quick sand generally occurs in sand and coarse silt and is not found in gravel, clay & fine silt.

* In cohesive soils like clay shear strength is not reduced to zero even if effective stress is reduced to zero as soil particles are held together due to their inherent cohesion.

Shear strength $S = C + \bar{\sigma}_n \tan\phi$

In Sand

$C = 0$

when quick sand condn.

$\bar{\sigma}_n = 0$

$S = \bar{\sigma}_n \tan\phi$

$S = 0$

In Clay

$C \neq 0$

at quick sand condn.

$\bar{\sigma}_n = 0$

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

$S = C \neq 0$

8.3

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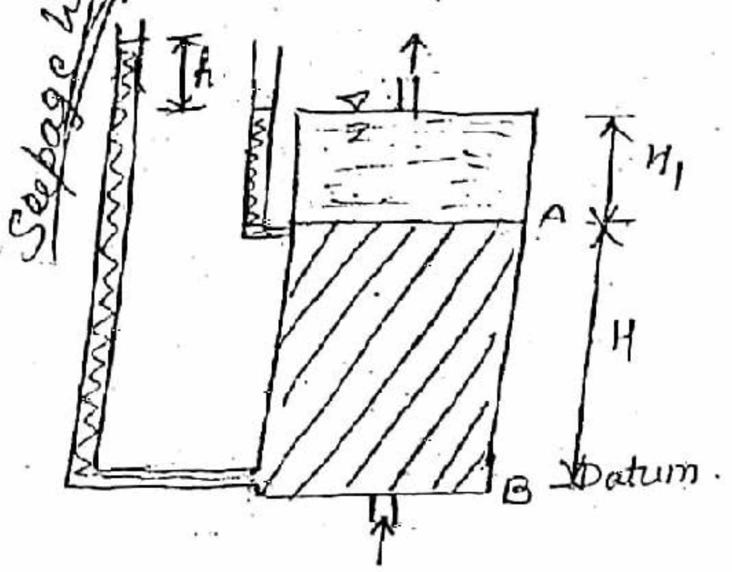
In gravel and coarse sand which are highly permeable soil high discharge is required to obtain critical hydraulic gradient which in natural condition do not practically possible.

* Quick sand condition can also be analysed using equillibrium of critical section. (Total stress approach)



$\Sigma F_{Y_B} = 0.$

$(H_1\gamma_w + \gamma_{sat} H) \cdot A - (H + H_1 + h)\gamma_w = 0$

Downward total stress — pore water pressure

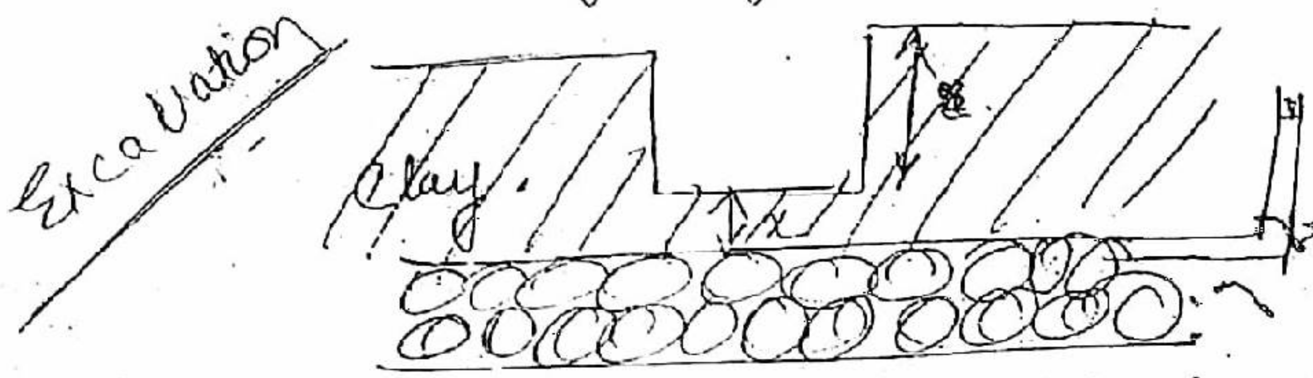
$\gamma_{sat} - H\gamma_w - h\gamma_w = 0.$

$H\gamma' - h\gamma_w = 0.$

$\frac{\gamma'}{\gamma_w} = \frac{h}{H} = i_c.$ $\left[\because i = \frac{h}{H}\right]$

$i_c = \frac{\gamma'}{\gamma_w} = \left(\frac{G-1}{1+e}\right)\frac{\gamma_w}{\gamma_w}$

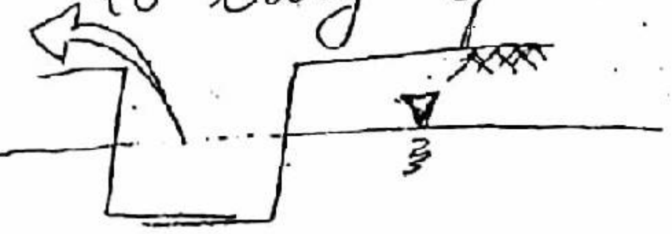
* Quick Sand also occur when sand under artesian pressure is over lained by clay



Total stress approach

$\gamma_{sat} \cdot x = h\gamma_w$

quick sand condition is found generally found when excavation is done below the G.W.T level and water is pumped out from excavated area to cary of the engineering activities



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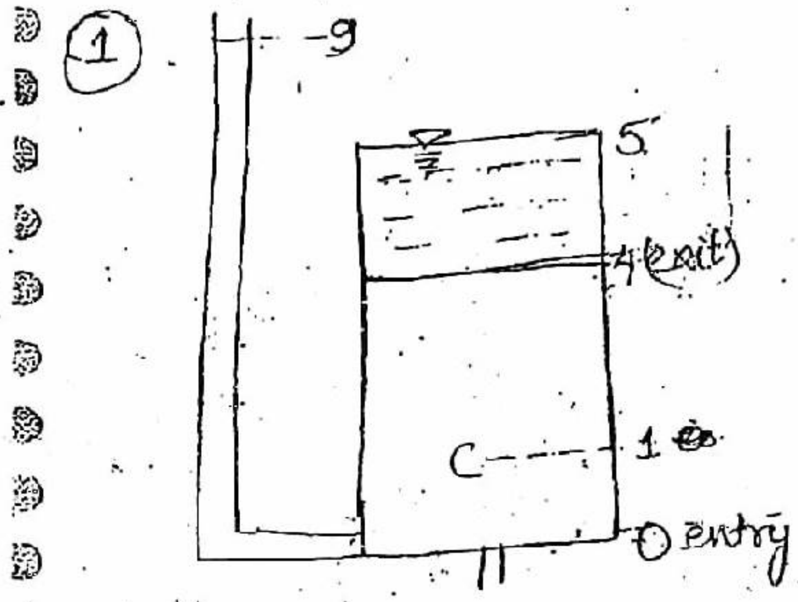
• For each case 1 & 2 determine pressure head, elevation head & total head at Entry point, exit point & point C.

(1)



	Datum	Pressure	Total
9	9	0.	9.
0 entry	0	9.	9.
4 exit	4	1.	5
5	5	[illegible]	
1	1	7.	8

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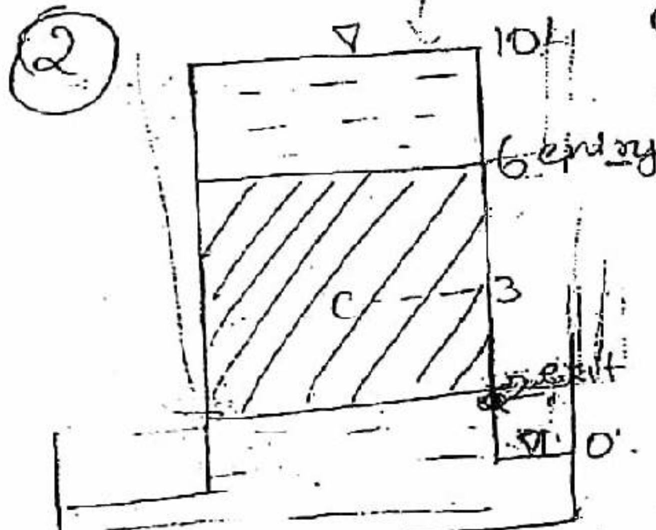
$i = \frac{9-5}{4} = 1.$

$\frac{9-C}{1} = 1$

$C = 8$

(2)



	Datum	Pressure	Total
entry (6)	6	4	10
exit (2)	2	-2	0
C 3	3.	-0.5	2.5

$i = \frac{10}{4} = 2.5$

$\frac{10-C}{3} = 2.5$

$C = 10 - 7.5$
$= 2.5$

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(3) FOS = 2 $\quad h = 1.8m$

$e = 0.678$

$i_c = \frac{G-1}{1+e} = 0.995$

$FOS = 2 = \frac{i_c}{i}$

$i = 0.49$

~~$i = iz\gamma_w$~~

~~$P_s = h\gamma_w = iz\gamma_w$~~

$(1.2+H)\gamma_w = iz$

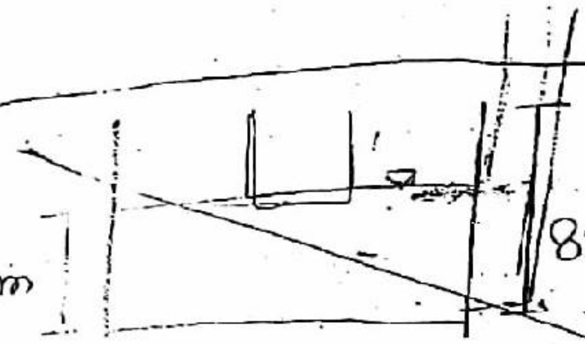
~~H = 2.4~~ $\quad H = 2.4m$

H = ? sand

1.2m

Hydraulic gradient $i = \frac{\text{head diff}}{\text{length}}$

$0.49 = \frac{\text{Seepage head}}{H+1.2}$



3m

8m

$P_s = 3m \quad H = 3m$

$w = 0.30$

$G = 2.7$

$e = 0.81$

$es = wG$

$i_c = 0.93$

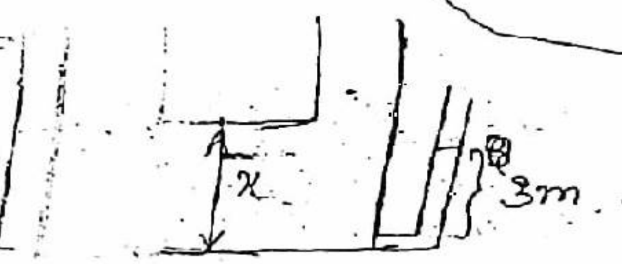
Let D be excation.

$i = \frac{3}{8-D} = 0.93$

$8 - D = 3.19$

$D = 4.80m$

(37)



8m

x

3m

$\gamma_{sat} X = h \times \gamma_w$

$\gamma_{sat} = \left(\frac{G+wG}{1+wG}\right)\gamma_w$

$19.02 X = 3 \times 10$

$\gamma_{sat} = 19.02 \ KN/m^3$

$X = 1.54$

depth required = 6.45 m.

1

$K_{avg} = \frac{3+2+4}{\frac{3\times10^{-3}}{3\times10^{-3}} + \frac{2}{6.5\times10^{-2}} + \frac{4}{7\times10^{-4}}} = 1.33 \times 10^{-3}$

ΔH

$K_{avg} \times i = K_3 \, i$

$\Rightarrow 1.33\times10^{-3} \times \frac{\Delta H}{9} = 7\times10^{-4} \times \frac{0.50}{2}$

$\Delta H = 1.18 \ m$

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~~$4\gamma_w + 5\gamma_{sat} = 3\gamma_w$~~ $\quad i = \frac{3}{6} = \frac{1}{2}$

~~$\frac{3}{6} \times 6 \times 9.81 =$~~ $\quad$ ~~$6\gamma_w - 5\gamma_{sat} = 3\gamma_w$~~

σ

35)

$z\gamma' + iz\gamma_w$

$= 5\times(18-9.81) + \frac{3}{6}\times5\times9.81$

$= \underline{65.4}$

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E-mail: padamsonsenterprises@gmail.com

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100 Million Building, 6, Jury Complex, IGNOU Road
Saidullajab, New Delhi-110030
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Flow Net & Laplace eqⁿ

If seepage takes place in 2 dimension it can be analysed using laplace eqⁿ which represents the loss of energy head in resistive medium.

$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0 \quad \text{for isotropic medium.}$$

$$K_x \frac{\partial^2 H}{\partial x^2} + K_y \frac{\partial^2 H}{\partial y^2} = 0 \quad \text{for non-isotropic medium}$$

The graphical solution of laplace eqⁿ is flownet which represents the description of equipotential line and streamline.

Assumptions in flow net

1) Darcy's law is valid.
2) Soil is homogeneous & isotropic.
3) Pore fluid & soil solids are incompressible.

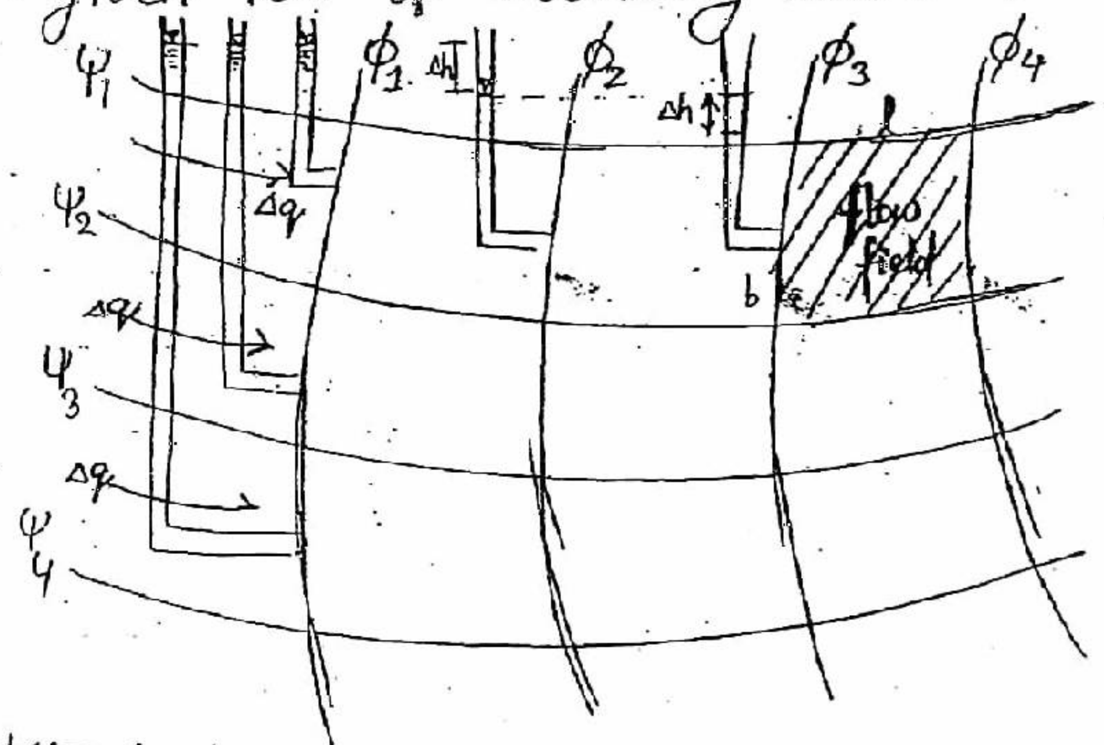
Properties of Flow Net

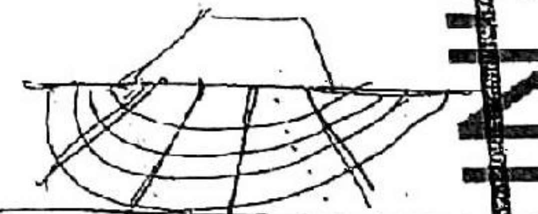
1) Equipotential line (ϕ line) and streamline (ψ line) intersect each other perpendicularly.
2) There can be no flow across the flow line & velocity of flow is always ⊥ to equipotential line.
3) Loss of head b/w 2 equipotential lines is always same and is termed as equipotential drop.

Area bounded b/w 2 equipotential lines & two flow lines is known as flow field which are approximately square in isotropic medium, that may be linear or curvilinear, while in non-isotropic medium they are approximately rectangular which may be linear or curvilinear.

5) The area b/w two flow lines is known as flow channel and discharge through each flow channel is same.

6) If water levels are reversed on upstreams & downstream side without changing boundary condition, there will be no change in flow net, it means flow-net is unique for given set of boundary condition.





Total Discharge

$$\boxed{q = \Delta q \times N_f}$$

N_f = no. of flow channel

N_f = No. of flow line − 1.

Equipotential drop

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

N_d = no. of equipotential drop.

N_d = no. of equipotential line − 1.

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Application of flow Net

Flow Net can be used for determination of :-

1. Seepage discharge
2. Hydrostatic pressure (pore water pressure)
3. Seepage pressure
4. Exit gradient

⇒ **Determination of Seepage discharge**

Let Δq be the discharge through the flow field.

Let length of flow net is unit

⇒ As per Darcy

$$\Delta q = K \cdot i \cdot A$$
$$= K \times \frac{\Delta h}{L} \times (b \times 1)$$
$$\Delta q = K \Delta h \frac{b}{L}$$

Case (a) :- Medium is isotropic flow field is square $b \simeq L$

$$\Delta q = K \cdot \Delta h \qquad \left(\because \Delta h = \frac{H}{N_d}\right)$$
$$\Delta q = K \frac{H}{N_d}$$

Total discharge $q = \Delta q \times N_f$

$$\boxed{q = KH \frac{N_f}{N_d}} \quad m^3/s/m \text{ length of dam.}$$

$\frac{N_f}{N_d}$ = shape factor

N_f = no. of flow channel

N_d = no. of equipotential drops.

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Case b for Non-isotropic ~~so~~ medium

$$K_x \frac{\partial^2 H}{\partial x^2} + K_y \frac{\partial^2 H}{\partial y^2} = 0 \quad \text{—(1)}$$

$$\boxed{X = X_T \sqrt{\frac{K_x}{K_y}}}$$

$$\Rightarrow x^2 = X_T^2 \frac{K_x}{K_y}$$

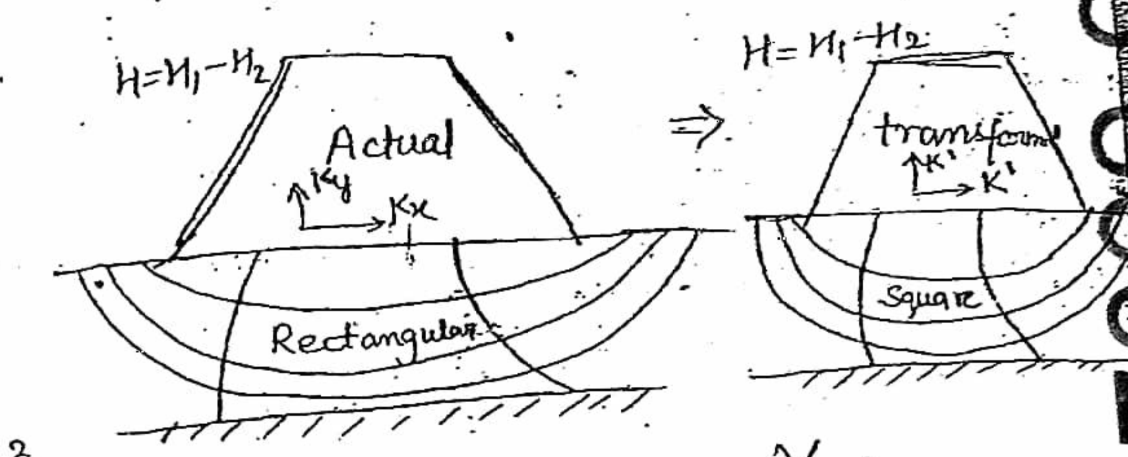
$$\Rightarrow \partial x^2 = \partial X_T^2 \frac{K_x}{K_y}$$

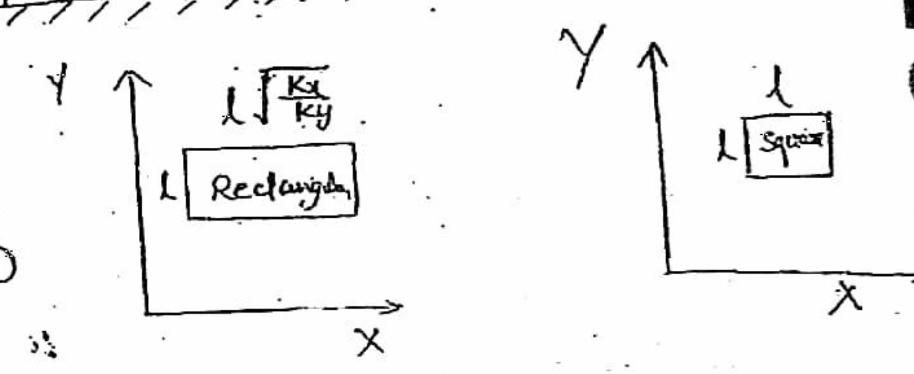
Putting in (1)

$$\Rightarrow \cancel{K_x} \frac{\partial^2 H}{\partial X_T^2 \cdot \frac{\cancel{K_x}}{K_y}} + K_y \frac{\partial^2 H}{\partial y^2} = 0$$

$$\Rightarrow K_y \frac{\partial^2 H}{\partial X_T^2} + K_y \left(\frac{\partial^2 H}{\partial y^2}\right) = 0$$

$$\Rightarrow \frac{\partial^2 H}{\partial X_T^2} + \frac{\partial^2 H}{\partial y^2} = 0$$





Discharge through actual flow field $= \Delta q = K_x \frac{\Delta h}{l\sqrt{\frac{K_x}{K_y}}} \times (1 \times 1)$

Discharge through transformed flow field $= \Delta q = K' \frac{\Delta h}{l} \times l \times 1$

Discharge from both the section would be same

$$\Delta q = K_x \frac{\Delta h}{\sqrt{\frac{K_x}{K_y}}} = K' \Delta h$$

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$$K' = K_x \sqrt{\frac{K_y}{K_x}}$$

$$\boxed{K' = \sqrt{K_x \cdot K_y}} \quad \text{equivalent permeability}$$

$$K' = \sqrt[3]{K_x K_y K_z} \quad 3\text{-D.}$$

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Total discharge

$$q = \Delta q \times N_f = \left\{K' \frac{\Delta h}{K} \times 1\right\} \times N_f$$

$$\boxed{q = K' H \frac{N_f}{N_d}} \quad K' = \text{equivalent permeability}$$

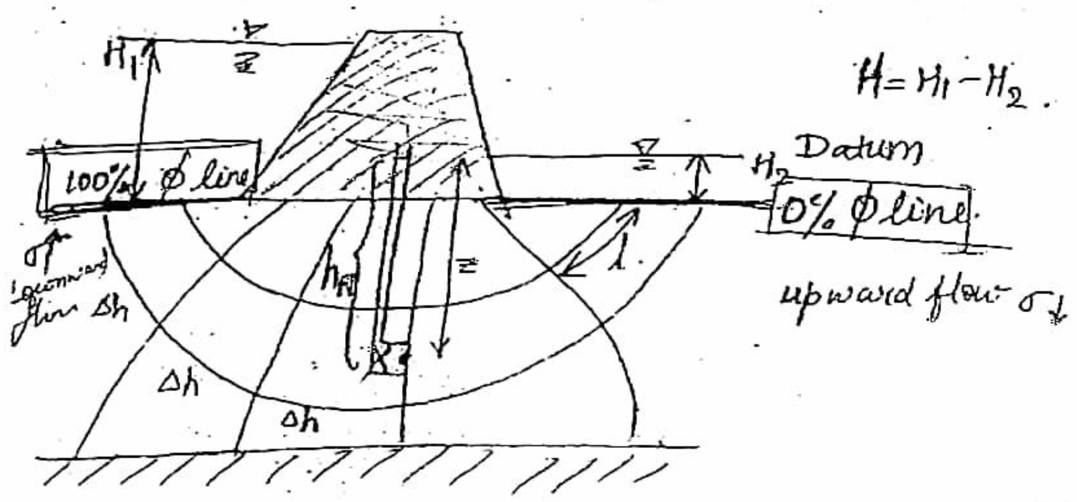
2) Determination of Seepage pressure

Let 'h' ~~be~~ is seepage head in medium at any given point after n equipotential drop then'

$$\boxed{h = H - n\Delta h}$$

$\therefore$ $\boxed{\text{seepage pressure} = \text{seepage head} \times \gamma_w}$

$$\boxed{p_s = h\gamma_w = (H - n\Delta h)\gamma_w}$$



$H = H_1 - H_2$

3) Determination of hydrostatic pressure (pore water pressure)

If 'h_w' is pressure head at any given point in the medium after 'n' equipotential drops then hydrostatic pressure at that point is given as.

$$\boxed{U = h_w * \gamma_w}$$

$$T.H = P.H \pm \text{Datum head}$$

$$P.H. = (H - n\Delta h) \mp z$$

$$\boxed{h_w = (H - n\Delta h) \mp z}$$

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Determination of Exit gradient: (governing criteria for quick sand condition)

* It is the hydraulic gradient at the downstream side of flow at which percolating water leaves the soil mass and emerges out at free water.

* If 'Δh' is head across the last flow field under which flow takes place and 'l' is length of last flow field over which this head is lost then

$$\boxed{i_{exit} = \frac{\Delta h}{l}} \qquad \Delta h = \frac{H}{N_d}$$

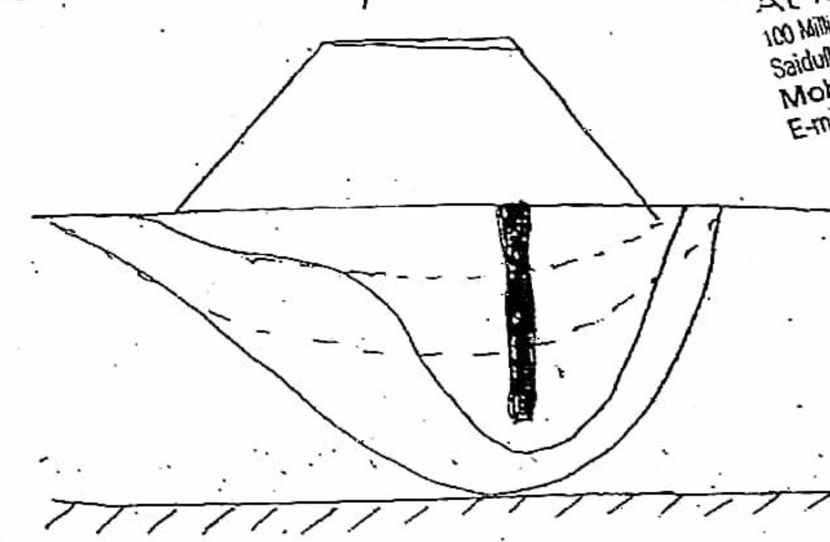
NOTE:- Exit gradient is governing criteria for piping failure

* To avoid piping FOS of 6 to 7 is provided in case of fine sand ~~FOS~~ FOS of 4 to 5 is provided in coarse sand.

$$\boxed{FOS = \frac{i_c}{i_{exit}}}$$

Sheet pile

* Piping can be avoided by providing sufficient length of vertical cut-off, viz sheet pile walls which helps in increasing length of flow hence helps in reducing i_{exit} and increasing FOS.



length of flow $\uparrow$
i_{exit} $\downarrow$
FOS $\uparrow$

inverted filter

* Piping can be avoided by providing protective filter, graded filters or inverted filter which helps in both avoiding the erosion of soil particles & reducing the uplift pressure at the base of dam

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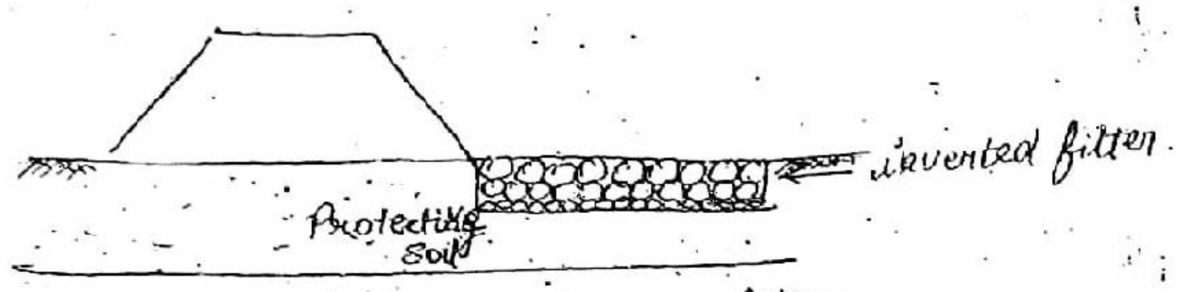


Terzaghi's
→ Design considerations for ~~design~~ protective filter.

IES

$$\frac{(D_{15})_{filter}}{(D_{85})_{protective\ layer}} < 5 \quad \text{~~gives the size of the largest~~}$$

$$4 < \frac{(D_{15})_{filter}}{(D_{15})_{protective\ layer}} < 20$$

$$\frac{(D_{50})_{filter}}{(D_{50})_{protective\ layer}} < 25$$

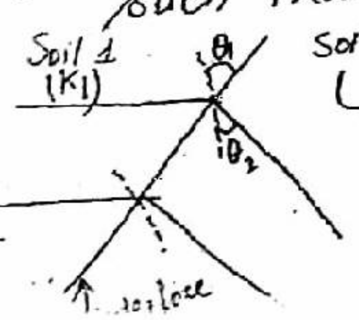
[1] The 1^{st} condition ensures that no significant erosion of particles of protected soil through the filter medium takes place

[2] The 1^{st} part of 2^{nd} condition ensures that sufficient loss of head inflow through filter takes place without built up of excess uplift pressure.

[3] The ~~seco~~ 2^{nd} part of 2^{nd} condition & 3^{rd} condition are additional design consideration.

NOTE:- Flow through Non homogeneous medium

Flow lines get deflected at interface of two dissimilar soils when it passes from one soil to another soil such that,



$$\boxed{\frac{K_1}{K_2} = \frac{\tan\theta_1}{\tan\theta_2}}$$

where θ_1 & θ_2 are angle made by flow line with the normal to interface before & after getting deflected

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Phreatic line

* Phreatic line is the topmost flow line below which ~~flo~~ seepage takes place.
* It is used to separate saturated soil mass from unsaturated soil mass.
* Pressure below the phreatic line is hydrostatic & pressure on & above phreatic line is atmospheric.
* Phreatic line follows the path of base parabola for which any point (focus) & point lying on it is equidistance from a point a line (directrix)

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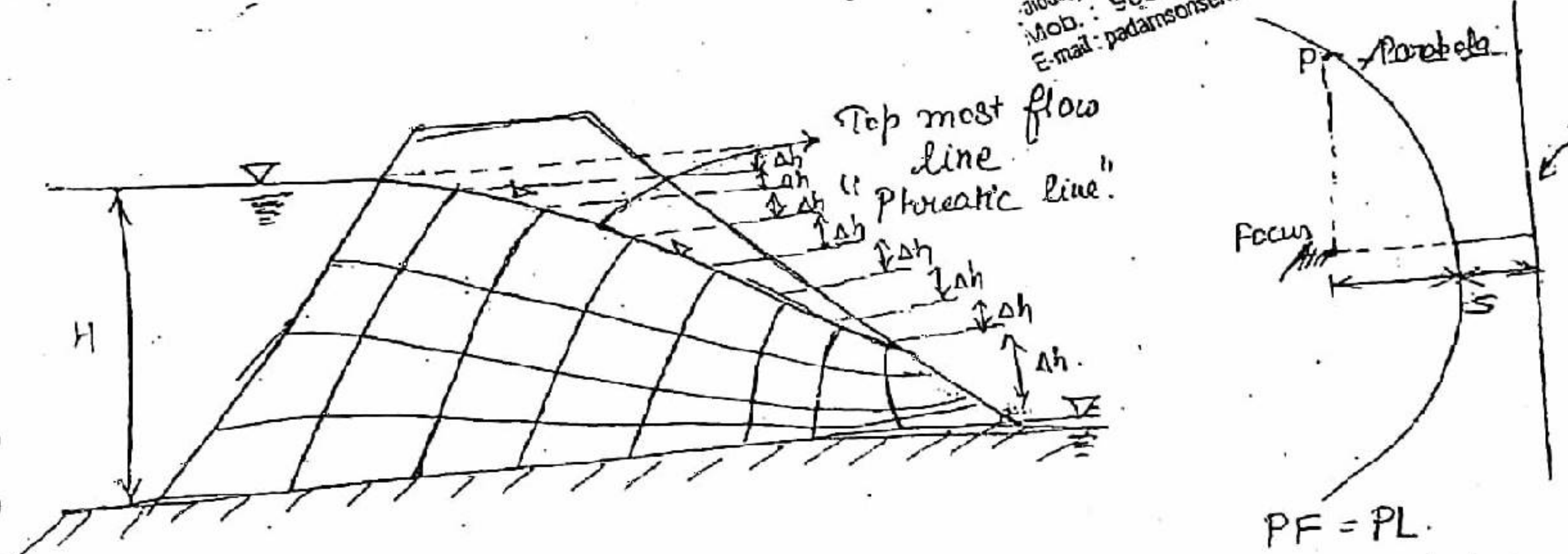

PF = PL
S = focal distance

Procedure to draw phreatic line when drainage filter is provided (Given by Casagrande).

1. Let BD (length = L) is the horizontal projection of upstream face AB of dam on water surface level. Phreatic line starts from point C, distance 0.3L from point B.
2. Focus F is the intersection of permeable & impermeable medium.
3. Draw a circle with point C as centre and CF as radius, to meet the horizontal projection of water surface at I. Vertical tangent to this circle at point I

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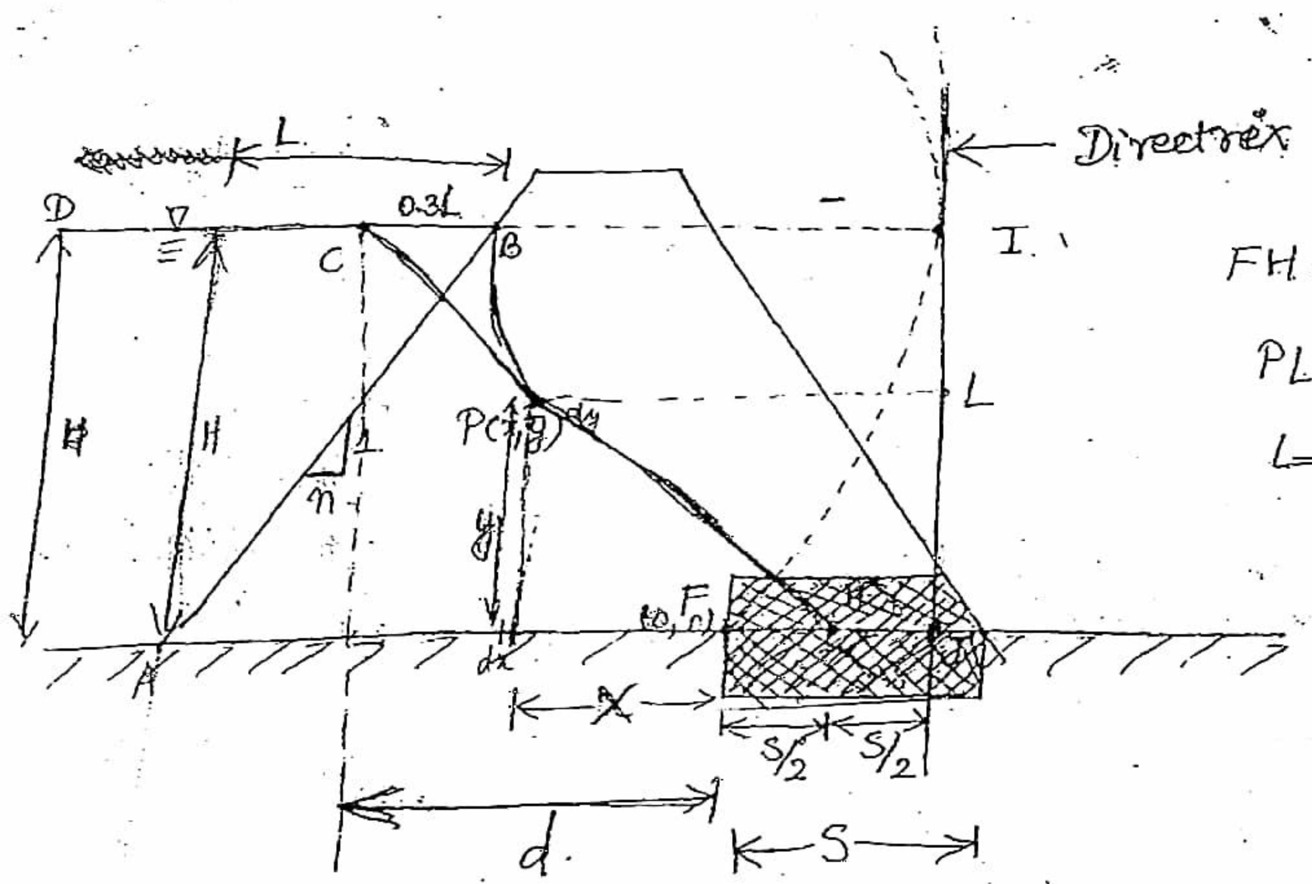
represents the directrix of parabola.

4) The vertex 'H' of parabola lies in b/w focus and directrix such that $FH = HJ = S/2$ where S is focal distance.

5) To locate the intermediate points $P(x, y)$ on phreatic line draw a circle with focus F as the centre and radius of $X+S$ which meet the vertical line drawn at a distance of X from point F, which represents required point $P(x, y)$.

$$PF = PL = X+S.$$

6) Join all the intermediate points. As phreatic line is topmost flow line it must start from point B and should be perpendicular to equipotential line AB. Hence, an entry correction is made to obtain phreatic line.



Discharge through Dam, considering unit length of Dam

As per darcy $\quad q = KiA$

$i = \dfrac{dy}{dx} \quad A = y \times 1$

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

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Point $P(x, y)$ $\quad PF = PL$

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

$$\cancel{x^2} + y^2 = \cancel{x^2} + S^2 + 2xS$$

on diff : $2y \dfrac{dy}{dx} = 2S \Rightarrow \dfrac{dy}{dx} = \dfrac{S}{y}$

$$① \Rightarrow q = K \frac{dy}{dx} \cdot y = K \cdot \frac{S}{y} \cdot y$$

$$\boxed{q = K \cdot S} \quad m^3/s / m \text{ length of Dam.}$$

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Point 'C' (d, H)

$CF = CI.$

$$\sqrt{d^2+H^2} = d+S$$

$$\boxed{S = \sqrt{d^2+H^2} - d}$$

(d = Base width $- 0.7L$ − length of filter.)

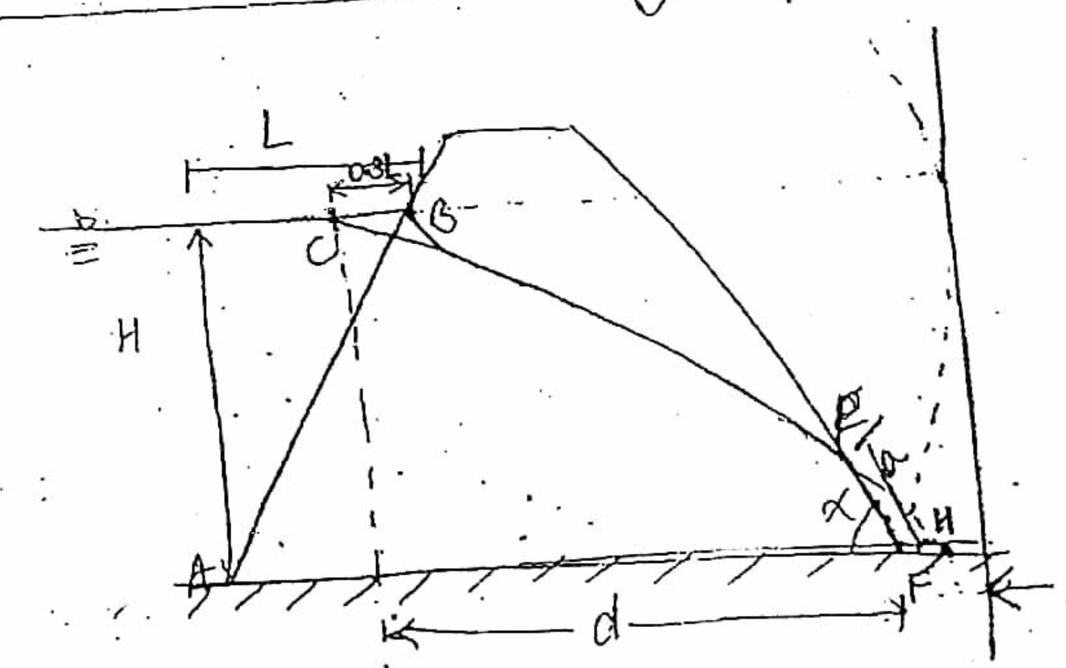
Non isotropic Medium

$\boxed{q = K'S} \quad S = \sqrt{d_T^2 + H^2} - d_T$

$d = d_T \left(\dfrac{K_x}{K_y}\right)$

K' = equivalent permeability $= \sqrt{K_x K_y}$

Phreatic line when drainage filter is not provided



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if $\alpha < 30^\circ$

$q = K a \sin\alpha \, \cancel{\tan\alpha}$. $\quad a = \frac{d}{\cos\alpha} - \sqrt{\frac{d^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}}$

If drainage filter is not provided, base parabola meets the downstream side of ~~of~~ dam at point ~~P~~ E and phreatic line extends upto focus of the parabola F (toe of dam) where EF represent drainage face 'a' which always remains wet hence, in order to avoid its damage stone pitching is provided downstream side.

if $30^\circ < \alpha < 60^\circ$

$q = K a \sin^2\alpha$.

$a = \sqrt{d^2 + H^2} - \sqrt{d^2 + H^2\cot^2\alpha}$

Pg 25

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

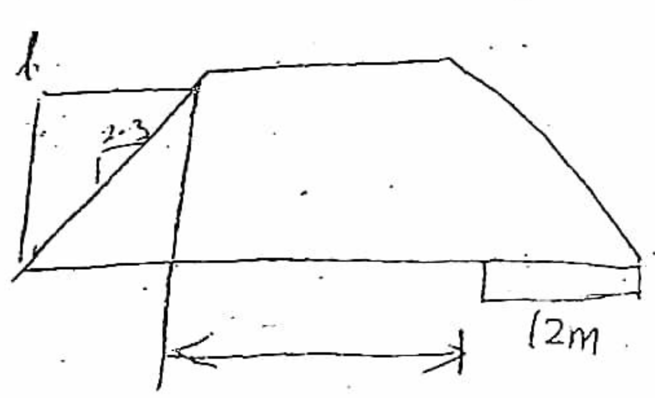
$= 3.8\times10^{-6}\times10^2\times6.3\times\frac{3\times100}{10}\times1.100$

$= 7.186\ cm^3/s$

28) $i_{exit} = \frac{\Delta h}{l} = \cancel{\quad}$ $\quad \Delta h = \frac{H}{N_d} = \frac{5}{9}$

$i_{exit} = 0.55$

~~27)~~ 41)



$q = K'S$.

$K' = \sqrt{K_x K_y}$

$l = 32.2m$.

$0.7\,l = 22.54$.

$d = 80 - 22.54 - 12$.

$= 45.46$

$d = d_T\sqrt{\frac{K_x}{K_y}}$

$d_T = 30.495$

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$S = \sqrt{d_T^2 + H^2} - d_T = 3.06$.

$q = 16.42\times10^{-7}\ m^3/sec/m$ length

$= 0.0162\ cm^3/sec/m$ length of dam // Ans

~~Δ n 3~~ ~~H-14~~

$N_d = 18$ $\quad$ x after 3 equipotential drop.

~~at x~~ Hydraulic head $= H - n\Delta h = H - 3\Delta h$.

$\Delta h = \frac{H}{N_d} = \frac{14}{18}$.

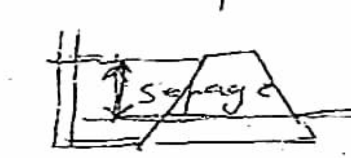
Hydraulic head $= 14 - 3\times\frac{14}{18} = 11.67m$.

Total head = Datum head + Pressure head

Pressure head $= 11.67 - 6 = 5.67$.

Pore water pressure $= 5.67\,\gamma_w = 55.62\,kN/m^2$

Seepage pressure $= 11.67\times\gamma_w$.



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CHAPTER – 6. Stress Distribution in Soil

⇒ Boussinesq's theory.

$\sigma_z = K_B \times \frac{Q}{Z^2}$

$K_B = \frac{3}{2\pi}\left[\frac{1}{1+\frac{r^2}{z^2}}\right]^{5/2}$

$\sigma_z = \frac{3Q z^3}{2\pi (r^2+z^2)^{5/2}}$ $\qquad \frac{\partial \sigma_z}{\partial z} = 0$

$\Rightarrow \frac{\partial}{\partial z}\left[\frac{3Qz^3}{2\pi} \times (r^2+z^2)^{-5/2}\right] = 0$

$\Rightarrow \frac{3Q}{2\pi}\left[3z^2 \times (r^2+z^2)^{-5/2} + z^3 \times -\frac{5}{2}(r^2+z^2)^{-7/2} \times 2z\right] = 0$

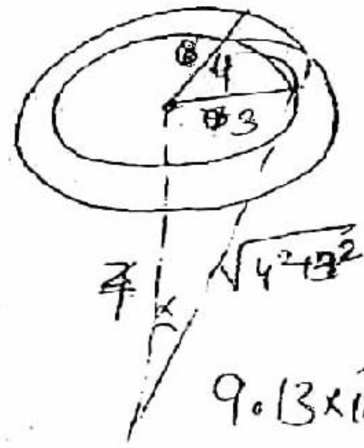
$\frac{3z^2}{(r^2+z^2)^{5/2}} - \frac{5z^4}{(r^2+z^2)^{7/2}} = 0$

$3 - \frac{5z^2}{(r^2+z^2)} = 0 \Rightarrow 3 = \frac{5z^2}{r^2+z^2}$

$3r^2 + 3z^2$

$3r^2 = 2z^2$

$\frac{r}{z} = \sqrt{\frac{2}{3}}$



7, $\sqrt{4^2+3^2}$ $\quad (1 - \cos^3\alpha)$

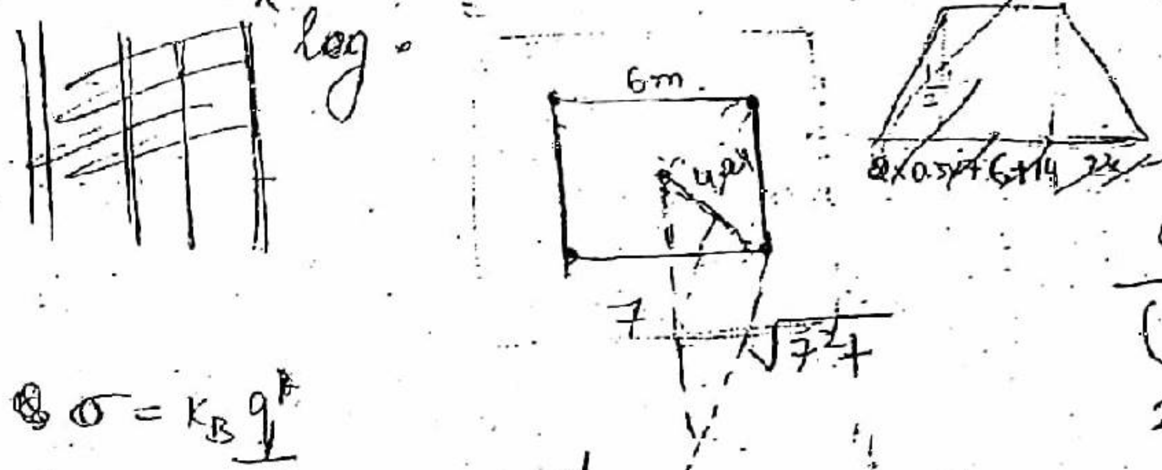
$9.13\times10^{-5} - 1.49\times10^{-4}$

$0.913 \qquad 1.49\times10^{-4}$

$\sigma = 31.68$

WORKBOOK

(14) Since the load is transfered on each point of below the log.



$B+2nz = 20$

$\frac{qB^2}{(B+2nz)^2}$

2800

$\Rightarrow \sigma = K_B \frac{Q}{z^2}$

$= \frac{3}{2\pi}\left(\frac{1}{1+\frac{r^2}{z^2}}\right)^{5/2} \frac{Q}{z^2}$

$\Rightarrow \sigma_z = \frac{3}{2\pi}\left(\frac{1}{1+\left(\frac{4.24}{7}\right)^2}\right)^{5/2} \frac{Q}{z^2} = 44.56\ kN/m^2$

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(6) $W_T = 1.8\ kg$ $\quad w_1 = 23g$ $\quad w = 3g$

$V = 944$ $\quad w_4 = 20g$ $\quad w = \frac{W_w}{W_s}$

Practical $\gamma_b = \frac{20}{944} = 0.021$ $\qquad = \frac{3}{20} = 0.15$

$\gamma_d = \frac{\gamma_b}{1+w}$ = ~~18.83 kN/m³~~

Theoretical $\gamma_d = \frac{G\gamma_w}{1+wG}$

$= 18.85\ kN/m^3$

(17) $e = 0.5$ $\quad e_{max} = 0.75$ $\quad e_{min} = 0.35$ $\quad G = 2.67$

$I_D = 0.625 = 62.5\%$

$R.C = \frac{\frac{1}{1+e}}{\frac{1}{1+e_{min}}} = \frac{1+e_{min}}{1+e} \times 100 = 90\%$

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Compressibility :- Compressibility of soil is defined as gradual +es in the volume of soil on application of load on soil mass.

* The volume change may be due to :-
1. expulsion & compression of pore air.
2. Expulsion of pore water.
3. Plastic readjustment of soil solids.

Compressibility
- Initial Compression or Compaction — Due to compression & expulsion of pore air.
- Consolidation
  - Primary Consolidation — Due to Expulsion of pore water
  - Secondary Consolidation — Due to plastic rearrangement of soil solids. (Occurs only high plastic soils)

*** Initial Compression Or Compaction**

It is an instantaneous process which occurs immediately after loading & is due to compression & expulsion of pore air.

Primary Consolidation

begins when soil reaches to full saturation (after the expulsion of pore air) and remains saturated during entire process of primary consolidation. It is due to expulsion of pore water. Primary consolidation completes when

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expulsion of pore water stops.

It is time taking phenomenan which depends upon permeability of soil, loading condition & length of drainage path.

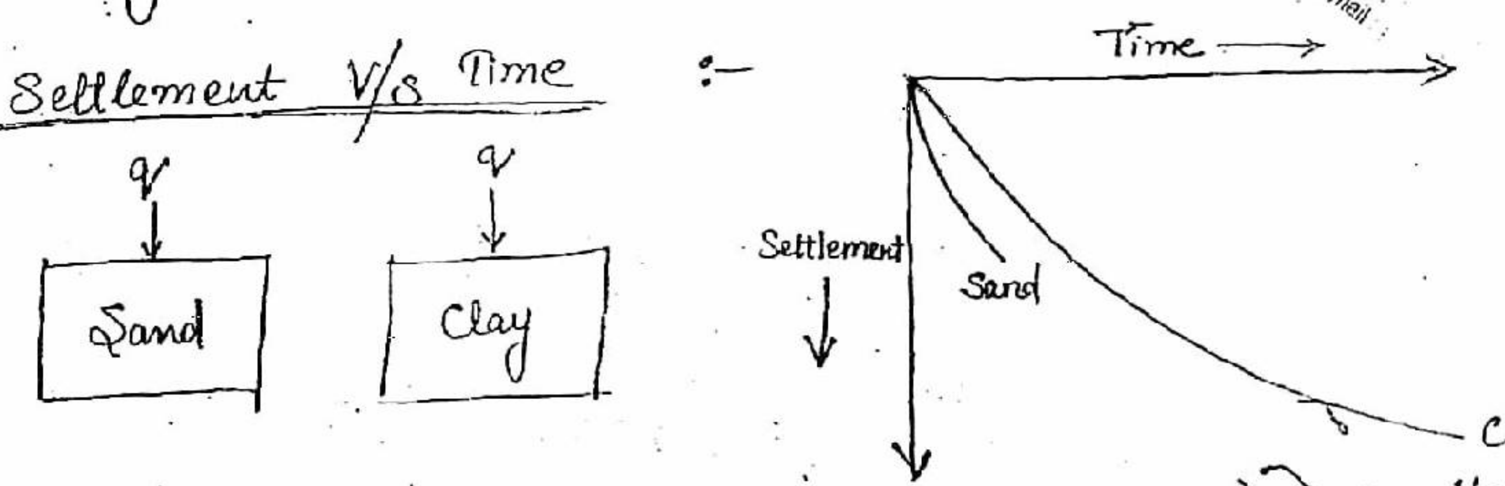
*** Secondary Consolidation**

Its start after completion of primary consolidation. If load is constant & volume change is recorded with passage of time without expulsion of pore water then volume change is due to secondary consolidation.

It occurs due plastic readjustment of soil molecules. In Coarse grain soil (gravel & soil) secondary consolidation is insignificant but in highly plastic clays, secondary consolidation can be 10% to 20% of total settlement.

*** Primary Consolidation Characteristics**

1. Settlement V/s Time :-



The rate of consolidation in sand is more due to its high permeability, but total settlement of clay is much greater than that of sand at same loading rate having same thickness of sample of both the soils.

The settlement in clay is high because of its high void ratio (porosity).

Thus for all practical purposes compressibility of granular soils may be neglected on account of consolidation.

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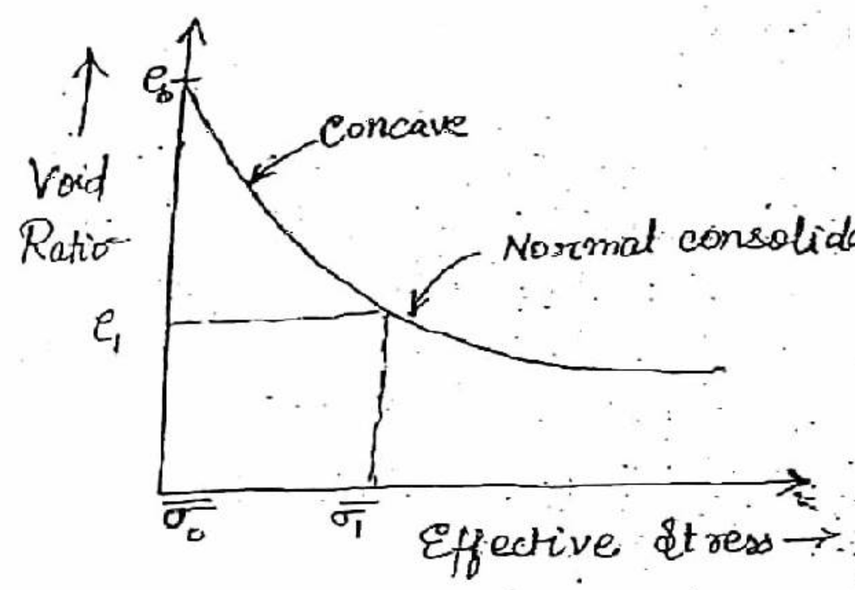
2 Void Ratio V/s Effective Stress (Normal Consolidation Curve or Normal Compration Curve)

Normal consolidated soils are those which are subjected to first time in the history to the present applied effective stress.

(Present stress > Past Stress) ⇒ Normal Consolidated



The Magnitude of slope of above curve is termed a coefficient of compressibility.

$$\text{Slope of } e \text{ vs } \bar{\sigma} = \frac{e_0 - e_1}{\bar{\sigma}_0 - \bar{\sigma}_1}$$

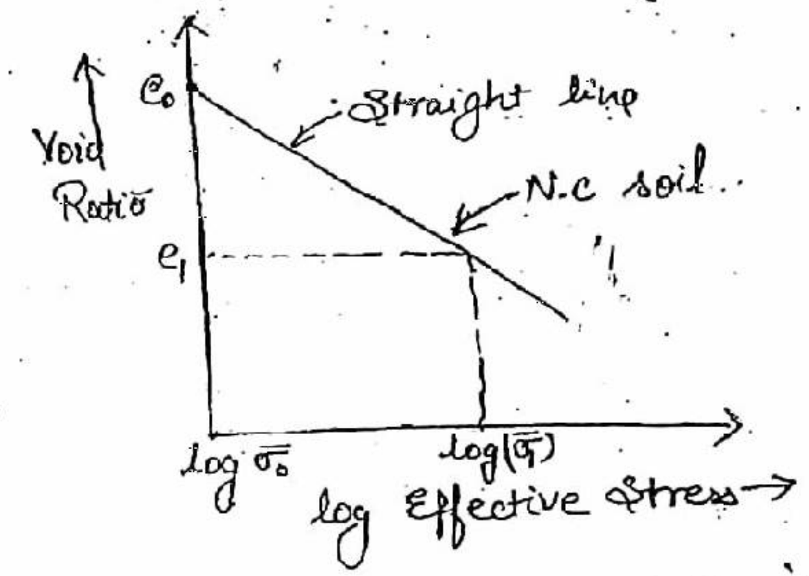
$$\text{Coefficient of compressibility} = \left|\frac{e_0 - e_1}{\bar{\sigma}_0 - \bar{\sigma}_1}\right|$$

$$\boxed{a_V = \frac{\Delta e}{\Delta \bar{\sigma}}}$$

∴ a_V is not constant for whole curve hence not used widely.

3 Void Ratio V/s log Effective stress

If graph of effective stress in log scale and void ratio in arithematic scale then straight line is obtained.



The magnitude of slope of above curve is termed as coefficient of compression or compression index.

$$\text{Slope of } e \text{ v/s } \log(\bar{\sigma}) = \frac{e_0 - e_1}{\log(\bar{\sigma}_0) - \log(\bar{\sigma}_1)}$$

$$\text{Coefficient of Compression} = \left|\frac{e_0 - e_1}{\log(\bar{\sigma}_0) - \log(\bar{\sigma}_1)}\right|$$

$$\boxed{C_c = \frac{\Delta e}{\log \frac{\bar{\sigma}_1}{\bar{\sigma}_0}}}$$

C_c more of any soil at same effective stress settlement of that soil will be more V_V more ie ΔV - more ie settlement more

* For ~~many~~ ~~soil~~ most of the clay C_c lies b/w 0·1–0·8. Greater C_c means soil is more compressible. It means, within same change in stress, it will be compressed more.

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Emphirical Relations of Calculation of C_c

1) For undisturbed clay with medium sensitivity.

$$\boxed{C_c = 0.009\,(W_L - 10)}$$ **

$W_L \rightarrow$ liquit limit in %

2) For remoulded clays with medium to low sensitivity

$$C_c = 0.007\,(W_L - 7)$$

3) For Organic clays

$$C_c = 0.0115\, W_n$$

$W_n \rightarrow$ Natural water content in soil.

4) If initial void ratio is known

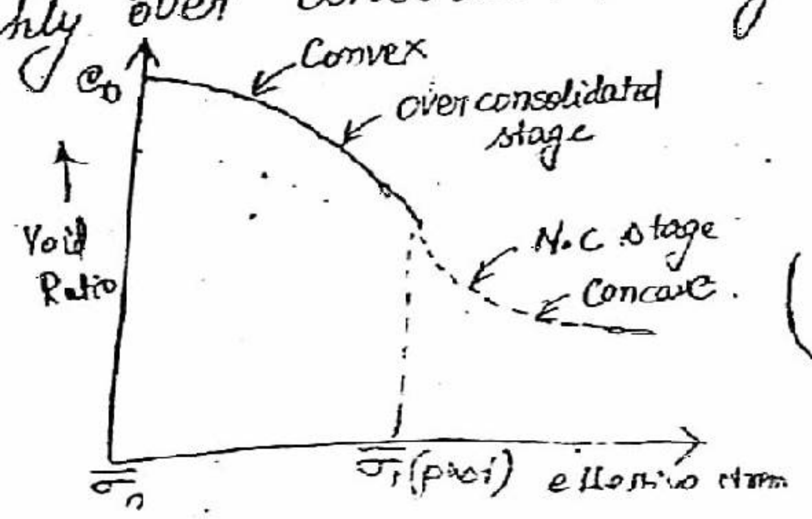
$$\boxed{C_c = 1.15\,(e_0 - 0.35)}$$

$e_0 \rightarrow$ initial void Ration

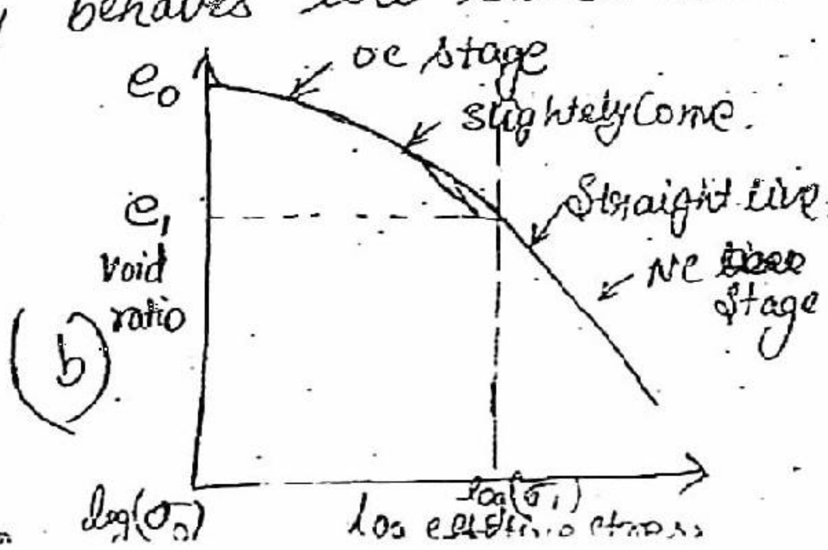
4 Overconsolidated soils / Pre Consolidated soil.

If presently applied effective stress is less than applied effective stress in the past then soil is said to be in over consolidated stage (present is less than past).

Over consolidated soils shows less volume change and highly over consolidated clay behaves like dense sand



(a)



(b)

Slope of curve (b) e v/s $\log \bar{\sigma}$ in O.C. stage $= \dfrac{e_0 - e_1}{\log \bar{\sigma}_0 - \log \bar{\sigma}_1}$

Coefficient of Recompression $= \dfrac{\Delta e}{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}$

$$\boxed{C_r = \left|\frac{\Delta e}{\log\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_0}\right)}\right|}$$

Generally $C_r = \frac{1}{5}$ to $\frac{1}{10}$ of C_c.

Over Consolidation Ration (O.C.R)

It is defined as the ratio of max effective stress applied in past to the max effective stress applied in present.

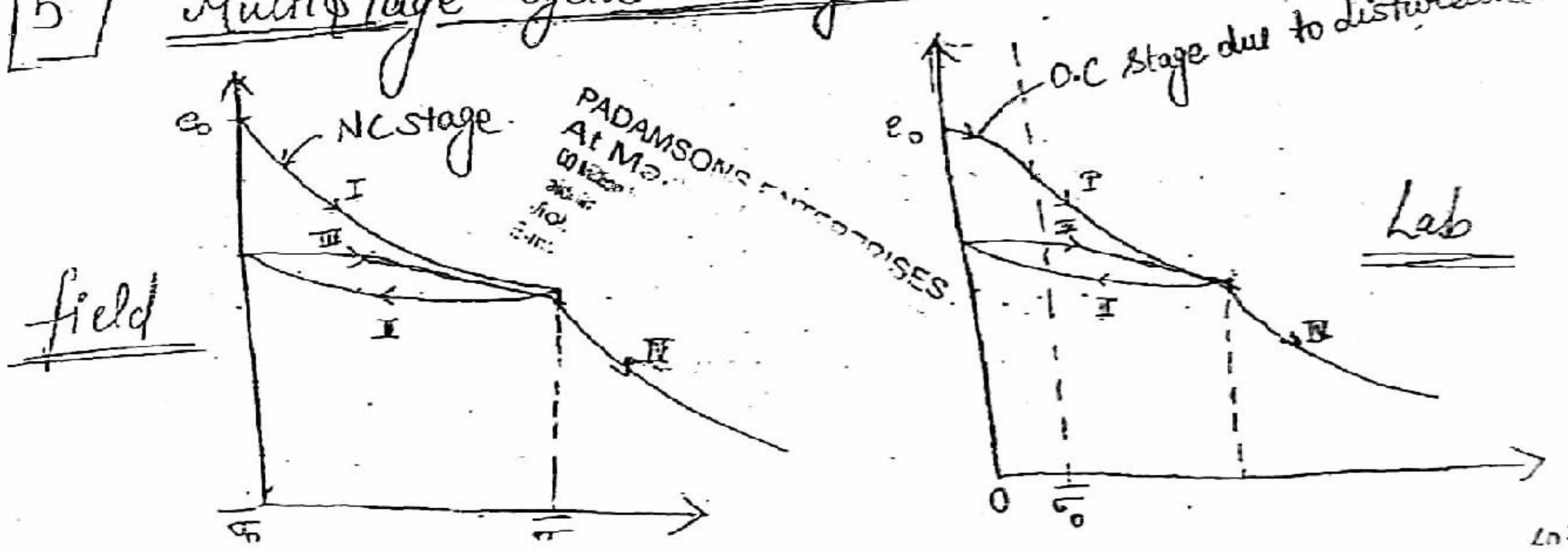
$$\boxed{O.C.R = \frac{\text{Max}^n \text{ effective stress applied in Past}}{\text{Max}^n \text{ effective stress applied in Present}}}$$

if OCR $> 1 \longrightarrow$ Over consolidated stage (Past > Present)

if OCR $\leq 1 \longrightarrow$ normal consolidated stage (Present $\geq$ Past)

NOTE :- Dessication of clay, Removal of construction load, removal of glaciers, Rise of water table are some example of over consolidation.

5 Multistage Cyclic loading



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I → Normal consolidated curve / virgin compression curve

II → Rebound / Swelling curve

III → Over consolidation curve

IV → Normal Consolidation curve.

Terzaghi's One Dimensional Consolidation Theory

Assumption

1) Soil mass is homogeneous & isotropic.

2) Soil is fully saturated and remains saturated throughout the process of consolidation. it means initial compression is not considered

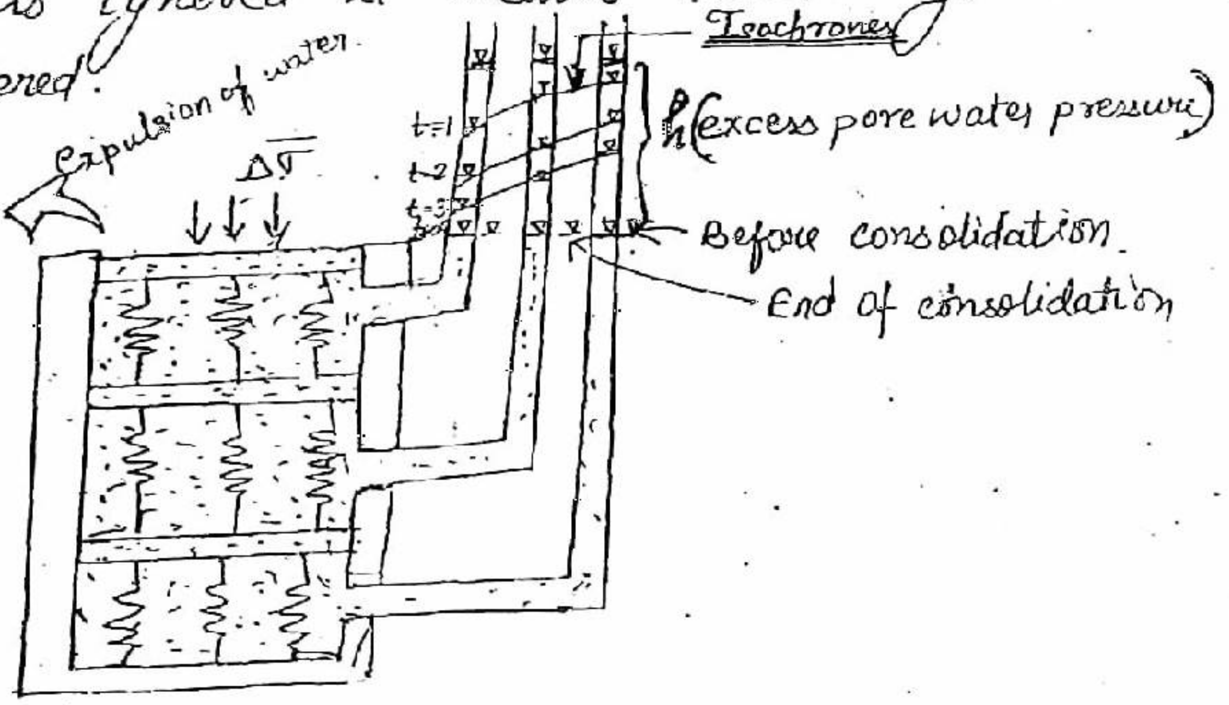
3) The consolidation is one dimensional, it means flow of expelling water is unidirectional (in vertical) and there is no change in area (volume change is due to change in depth)

4) Darcy's law is valid

5) Strains coming on soil sample are small

6) The hydrodynamic lag (time taken for expulsion of water) is considered where as plastic lag (time taken by the soil in readjustment) is ignored however plastic lag is found to be exist it means time required for plastic readjustment is ignored it means secondary consolidation is not considered.

Spring Analogy:



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Theory book Ques

1) Application of total stress.

2) at $t=0$ (Entire load is taken by water) (I)

$\Delta\sigma = \cancel{\Delta\bar{\sigma}}^{0} + U_i$

$\Delta\sigma = U_i = h\gamma_w$ (Excess pore water pressure)

3) at any time 't' (Pore water ~~pressure~~ will expelled out & effective stress will increase) (II)

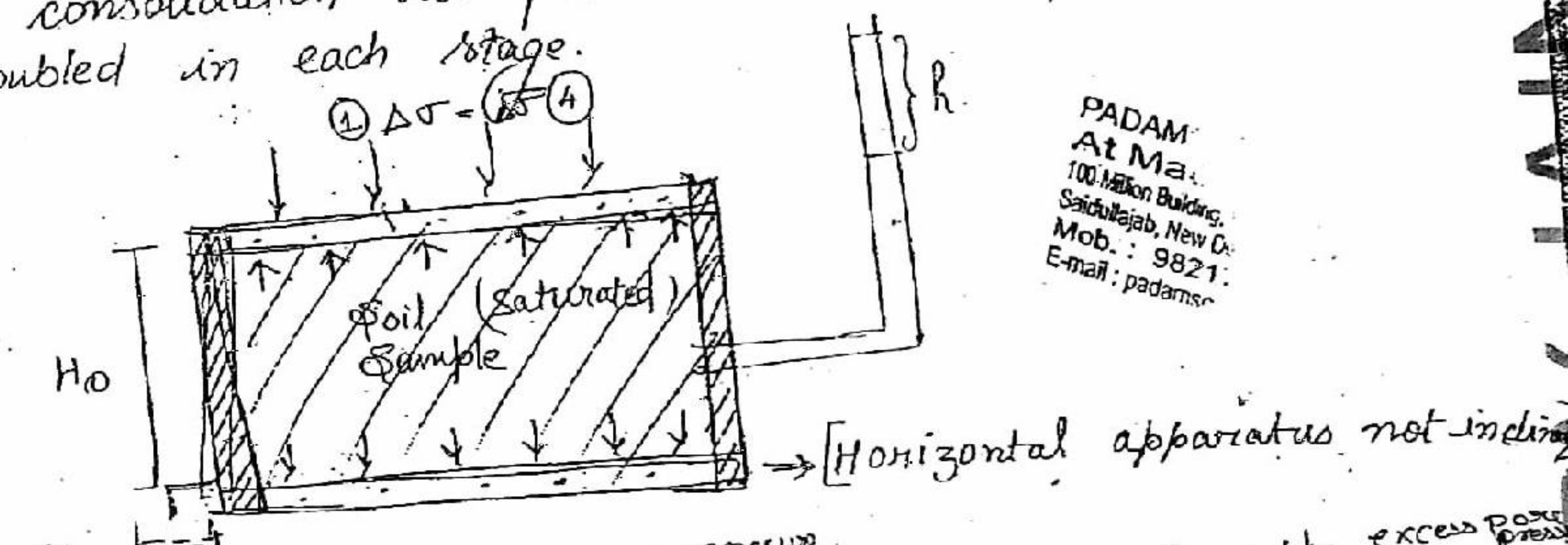
$\Delta\sigma = \Delta\bar{\sigma}\uparrow + U_i\downarrow$

4) at $t=\infty$ (Entire load is taken by effective stress) (III)

$\Delta\sigma = \Delta\bar{\sigma} + \cancel{U_i}^{0}$

* In order the simulate field condition consolidation test is being perform over the soil sample as per Terzaghi's theory of One 1-D consolidation in consolidometer or oedometer, in which settlement of given sample of soil is noted at different time intervals.

* In consolidation test pressure on soil specimen is doubled in each stage.



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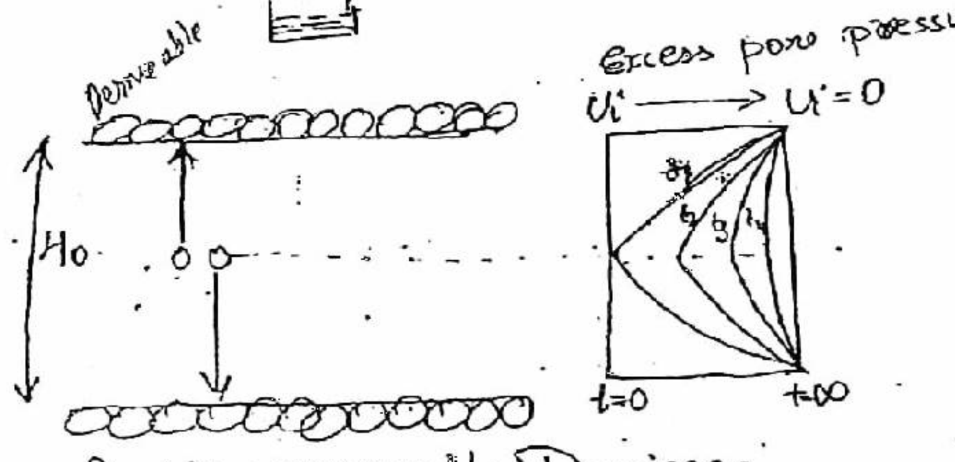
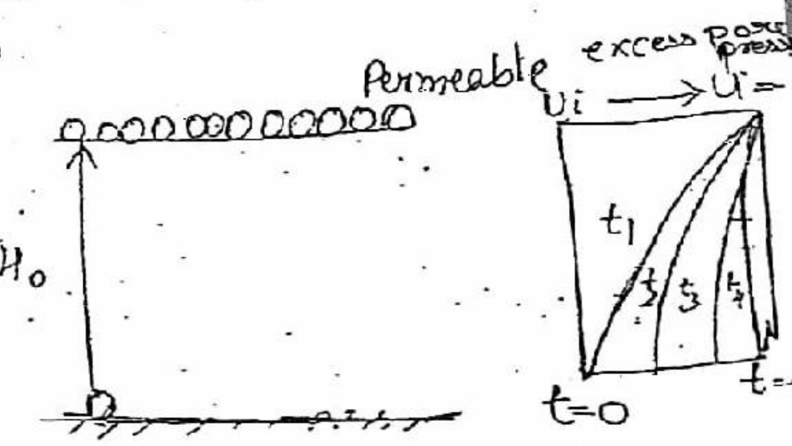


2-Way ~~permeable~~ Drainage

Length of Drainage $= \frac{H_0}{2}$



One way Drainage.

Length of Drainage path = H_0

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Terzaghi One Dimensional Consolidation Equation

$$\boxed{\frac{\partial U}{\partial t} = C_v \cdot \frac{\partial U^2}{\partial z^2}}$$

1.) where, $\frac{\partial U}{\partial t}$ = rate of change of pore water pressure which represent rate of consolidation.

2) C_v = coefficient of consolidation;

Coefficient of Consolidation $\boxed{C_v = \frac{K}{m_v \gamma_w}}$

K = coefficient of permeability.

m_v = coefficient of volume compressibility or modulus of volume change.

$$m_v = \frac{-\frac{\Delta V}{V_0}}{\Delta \sigma} = \frac{\frac{-\Delta e}{1+e_0}}{\Delta \sigma} = \frac{-\Delta e}{\Delta \sigma} \times \frac{1}{1+e_0}$$

Coefficient of Volume compressibility $\boxed{m_v = \frac{a_v}{1+e_0}}$

a_v = coefficient of compressibility.
e_0 = Initial Void Ratio.

Note

	coefficient of	formula	Symbol	Unit
1	Compressibility	$\frac{\Delta e}{\Delta \sigma}$	a_v	m^2/KN
2	compression	$\frac{\Delta e}{\log\left(\frac{\sigma_1}{\sigma_0}\right)}$	C_c	unit less
3	recompression	$\frac{\Delta e}{\log\left(\frac{\sigma_1}{\sigma_0}\right)}$	C_r	unit less
	Consolidation	$\frac{K}{m_v \gamma_w}$	C_v	m^2/sec
	Volume compressibility	$\frac{a_v}{1+e_0}$	m_v	m^2/KN

$\frac{m/sec}{m^2/KN \times KN/m^3}$

$\frac{m/sec}{\frac{m^2}{KN} \cdot \frac{KN}{m^3}}$

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Solutions of Terzaghi Equation

1.) Degree of Consolidation (% U)

* It is fraction of ultimate consolidation which is completed at any stage of time during consolidation.
* It is defined for primary consolidation process only.
* At the beginning (t=0) % U = 0
* At the End of primary consolidation (t=∞) % U = 100%

(a) when settlement at any stage is known

Let 'Δh' is settlement at any time let 'ΔH' is ultimate consolidation settlement at the end of primary consolidation

$$\boxed{\% U = \frac{\Delta h}{\Delta H} \times 100}$$ **

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(b) when pore water pressure is known

Let at any time "t" excess pore water pressure "u"
At the beginning of consolidation, excess pore pressure is 'ui'
At the end of consolidation (t=∞) % excess pore pressure = 0

$$\boxed{\% U = \frac{U_i - U}{U_i - 0} \times 100 = \frac{U_i - U}{U_i} \times 100}$$

$\boxed{\Delta\sigma = U_i}$

(c) when void ratio is known

Let e_0 = void ratio at beginning of consolidation
e = Void ratio after any time 't'
e_{100} = void ratio at the end of primary consolidation.

$$\boxed{\% U = \frac{e_0 - e}{e_0 - e_{100}} \times 100}$$

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2. Time factor (T_v)

It is the parameter which relates to the degree of consolidation (U) & time required for that consolidation and is given as :-

$$\boxed{T_v = C_v \cdot \frac{t}{d^2}} \;**$$

where $\Rightarrow C_v$ = coefficient of consolidation.

$C_v = \frac{K}{m_v \gamma_w}$ (property of soil)

$= \frac{K}{\frac{a_v}{1+e_0} \cdot \gamma_w} \Rightarrow \frac{K}{\frac{\Delta e}{1+e_0} \cdot \frac{\gamma_w}{\Delta \bar{\sigma}}}$

$t \propto \frac{1}{K}$ permeability
$t \not\propto \Delta\bar{\sigma}$ effective stress
~~t~~ void
$t \propto a_v$ volume compressibility

$\Rightarrow$ $d \rightarrow$ length of drainage path

1-way drainage $d = H_0$ (more time)

2-way drainage $d = \frac{H_0}{2}$ (time less)

$\Rightarrow$ t = time after which time factor is to be calculated

$\Rightarrow$ T_v = time factor which depends upon degree of consolidation

*

$T_v = \frac{\pi}{4} U^2$	$\% U \le 0.6$
$T_v = -0.9332 \log_{10}(1-U) - 0.0851$	$\% U > 0.6$

$(T_v)_{50} = 0.196$

$(T_v)_{90} = 0.848$

$(T_v)_{100} = \infty$

Theoretically for 100% primary consolidation infinite time is required but primary consolidation is assumed to be complete if degree of consolidation has reached $\geq 90\%$

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$\Delta h \longleftrightarrow t$

$\% U = \frac{\Delta h}{\Delta h} \rightarrow T_v$

$T_v = C_v t / d^2$

Method of Determination of C_v

$\Rightarrow$ C_v depends on type of soil and change in effective stress.

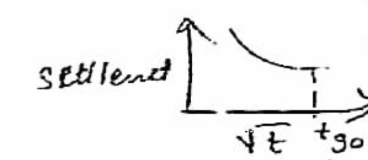
$\Rightarrow$ Generally it had been observed that if liquid limit of increases $C_v \downarrow$es.

$W_L \uparrow \rightarrow$ compressibility $\uparrow \rightarrow a_v \uparrow \rightarrow m_v \updownarrow \rightarrow C_v \downarrow$ $\quad C_v = \frac{K}{m_v \gamma_w}$

Generally for most of the clays value of C_v lies in the range of $5 \times 10^{-4} \frac{mm^2}{sec}$ to $2 \times 10^{-2} \frac{mm^2}{sec}$

$\Rightarrow$ There are 2 methods to find C_v.

1. Taylor's Square Root of Time fitting Method ($\% U = 90\%$)



$$T_v = C_v \frac{t}{d^2}$$

$$(T_v)_{90} = C_v \frac{t_{90}}{d^2}$$

$$0.848 = C_v \frac{t_{90}}{d^2}$$

$$\boxed{C_v = \frac{0.848 d^2}{t_{90}}}$$

where t_{90} = time required for 90% degree of consolidation & is noted as per the observation from the la

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2) Casagrande's logarithmic of Time fitting Method ($U = 50\%$)

$$T_v = C_v \frac{t}{d^2}$$

$$(T_v)_{50} = C_v \frac{t_{50}}{d^2}$$

$$0.196 = C_v \frac{t_{50}}{d^2}$$

$$\boxed{C_v = \frac{0.196\, d^2}{t_{50}}}$$

NOTE :- Square root of time fitting method is better for those soil which have high consolidation settlement.

Settlement Analysis

The total settlement is some of immediate / initial settlement, primary consolidation & secondary consolidation settlement.

$$\boxed{S = S_i + S_c + S_s}$$

⇒ IS Code specification for permissible settlement

1. Total permissible settlement
   (i) for isolated footing on clay = 65mm
   (ii) for isolated footing on sand = 40mm
   (iii) for raft footing on Clay = 65 – 100 mm
   (iv) for raft footing on sand = 40 – 65mm
2. Permissible differential settlement
   (i) for isolated footing on clay = 40 mm
   (ii) for isolated footing on sand = 25mm
   ~~(iii) for isolated footing~~

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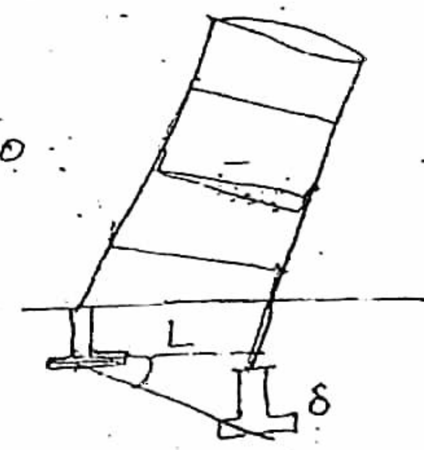
3) Permissible Angular Settlement

for High framed structure $\not> \frac{1}{500}$

To avoid any types of minor damage $\not> \frac{1}{1000}$

Angular settlement $= \frac{\delta}{L}$



⇒ Determination of Immediate / Initial Settlement

(a) For Sand (Cohensionless soil)

Cone penetration test data are used to determine immediate settlement.

$$S_i = \frac{H_0}{C_s} \log \frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}$$

$$C_s = \frac{q_c}{\bar{\sigma}_0} \times 1.5$$

where q_c = static cone penetration resistance value
H_0 = initial thickness of compressible layer
$\bar{\sigma}_0$ = Initial effective overburden pressure at the centre of compressible layer
$\Delta\bar{\sigma}$ = increase in effective stress at the centre of compressible layer.

(b) Elastic Settlement

In saturated Clays there is insignificant immediate settlement because pore water pressure cannot dissipate immediately, however small elastic settlement can occur due to deformation of clay particles and squeezing of pore water.

$$\boxed{S_i = \frac{qB(1-\mu^2)}{E_s} \cdot I_t}$$

Where q = uniform pressure at the base of foundation.
B = width of footing.

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B = width of footing (least lateral dimension)

μ = poisson's ratio. (0.35–0.5)

E_s = young's modulus of soil.

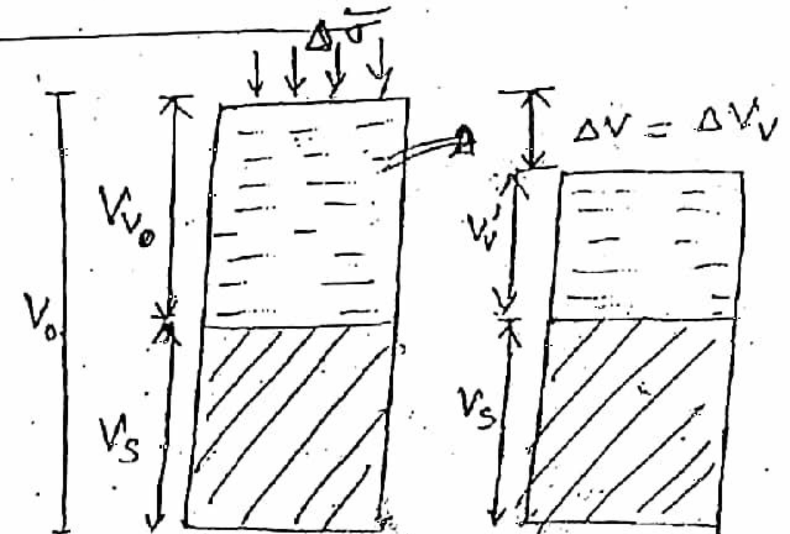
I_t = influence factor or shape factor which depends on l/b ratio.

	Shape	flexible foundation: centre	corner	Avg	rigid
1)	Circle	1	0.64	0.85	0.80
2)	Square	1.12	0.56	0.95	0.82
3)	Rectangle				
	$\frac{L}{B} = 1.5$	1.36	0.68	1.20	1.09
	$\frac{L}{B} = 2.0$	1.52	0.76	1.31	1.22

Determination primary Consolidation settlement

(a) When change in void ratio is known

$$\frac{\Delta V}{V_0} = \frac{A \cdot \Delta H}{A \cdot H_0} = \frac{\Delta H}{H_0} \quad \text{--- (1)}$$



$$\frac{\Delta V}{V_0} = \frac{\Delta V_v}{V_0} = \frac{\Delta V_v}{V_s + V_{v_0}}$$

$$= \frac{\Delta V_v}{V_s\left(1 + \frac{V_{v_0}}{V_s}\right)}$$

$$e_0 = \frac{V_{v_0}}{V_s}$$

$$\frac{\Delta V}{V_0} = \frac{\Delta e}{1+e_0} \quad \text{--- (2)} \qquad \left[\because \frac{\Delta V_v}{V_s} = \Delta e_0, \quad \frac{V_{v_0}}{V_s} = e_0\right]$$

from (1) & (2)

$$\boxed{\Delta H = H_0 \frac{\Delta e}{1+e_0}}$$

where $H_0 \rightarrow$ initial thickness

$\Delta H \rightarrow$ Ultimate Primary Consolidation

$\Delta e \rightarrow$ Change in void ratio

$e_0 \Rightarrow$ initial void ratio

b) When coefficient of volume compressibility is known

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0} = \frac{\Delta e}{1+e_0} \times \frac{\Delta \bar\sigma}{\Delta \bar\sigma}$$

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{\Delta \bar\sigma} \times \frac{\Delta \bar\sigma}{1+e_0} = \frac{a_v}{1+e_0} \times \Delta \bar\sigma$$

$$\because a_v = \frac{\Delta e}{\Delta \bar\sigma}, \quad m_v = \frac{a_v}{1+e_0}$$

$$\frac{\Delta H}{H_0} = m_v \cdot \Delta \bar\sigma$$

$$\boxed{\Delta H = H_0 \, m_v \, \Delta \bar\sigma}$$

c) When coefficient of Compression is known

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0} = \frac{\Delta e}{1+e_0} \times \frac{\log\left(\frac{\bar\sigma_1}{\bar\sigma_0}\right)}{\log\left(\frac{\bar\sigma_1}{\bar\sigma_0}\right)}$$

$$= \frac{\Delta e}{\log\left(\frac{\bar\sigma_1}{\bar\sigma_0}\right)} \times \frac{\log\left(\frac{\bar\sigma_1}{\bar\sigma_0}\right)}{1+e_0}$$

$$\frac{\Delta H}{H_0} = \frac{C_c}{1+e_0} \times \log\left(\frac{\bar\sigma_1}{\bar\sigma_0}\right)$$

$$\boxed{\Delta H = \frac{H_0 \cdot C_c}{1+e_0} \log\left(\frac{\bar\sigma_0 + \Delta\bar\sigma}{\bar\sigma_0}\right)}$$

$\bar\sigma_0$ = initial overburden pressure

$\Delta\bar\sigma$ = change in effective stress.

NOTE:1 — If soil is normally consolidated then use coeff. of compression (C_c) & if soil is over consol. then use coeff. of recompression (C_r).

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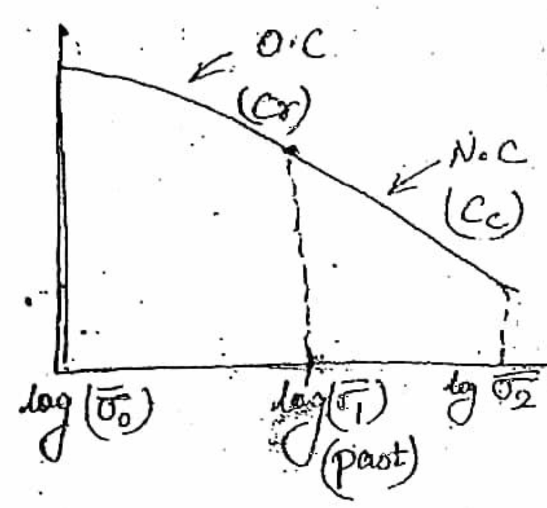
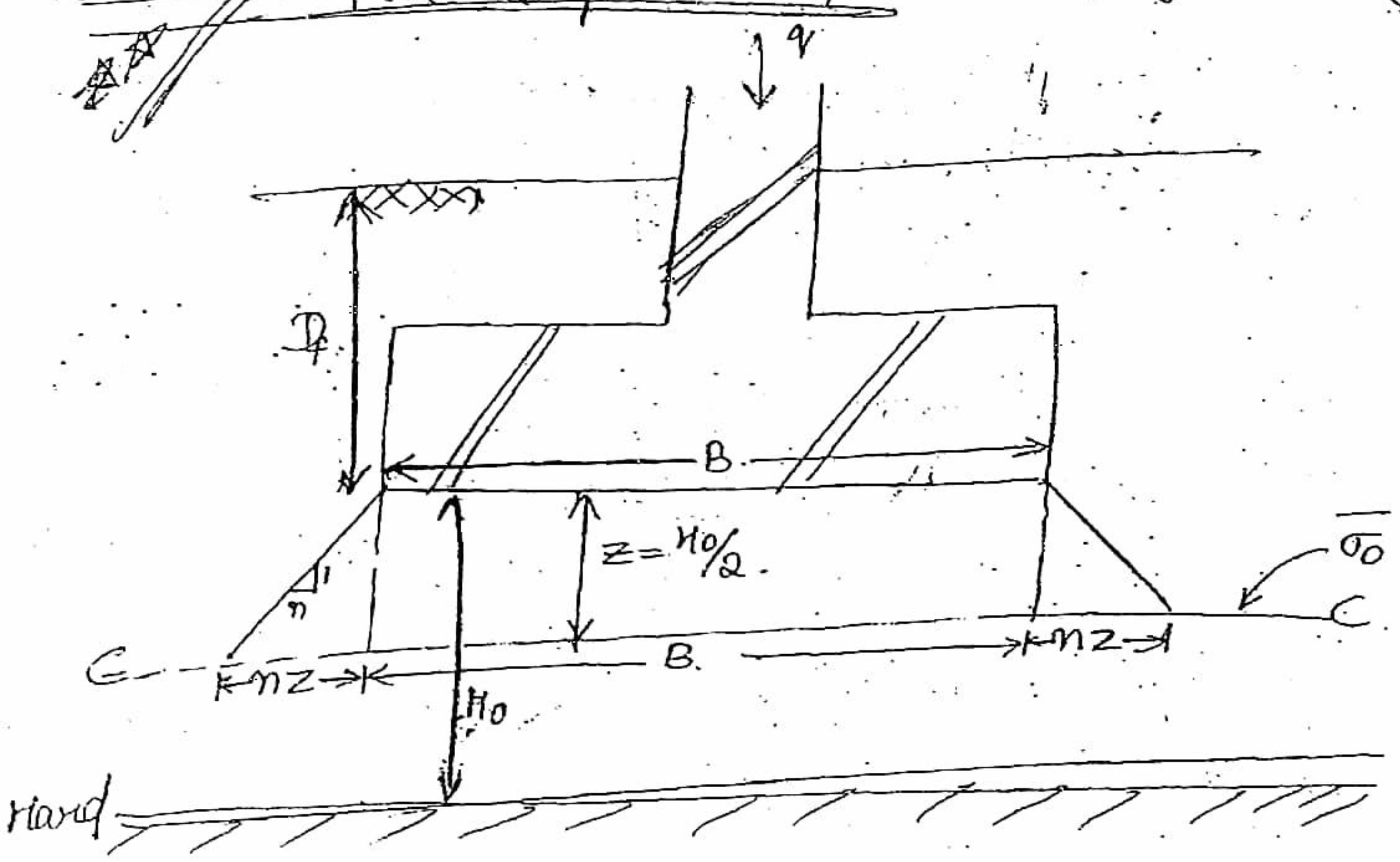
$$\Delta H = \frac{H_0 C_c}{1+e_0}\log\left(\frac{\bar{\sigma}_p}{\bar{\sigma}_0}\right) + \frac{H_0 C_r}{1+e_0}\log\left(\frac{\bar{\sigma}_2}{\bar{\sigma}_1}\right)$$



NOTE2 Five Step Method



(1) Strata.

(2) H_0 taken below foundation.

(3) $\bar{\sigma}_0$ calculation at the midde of the compressible layer.

4) $\Delta\bar{\sigma}$ due to applied load.
load distribution in 1 : n

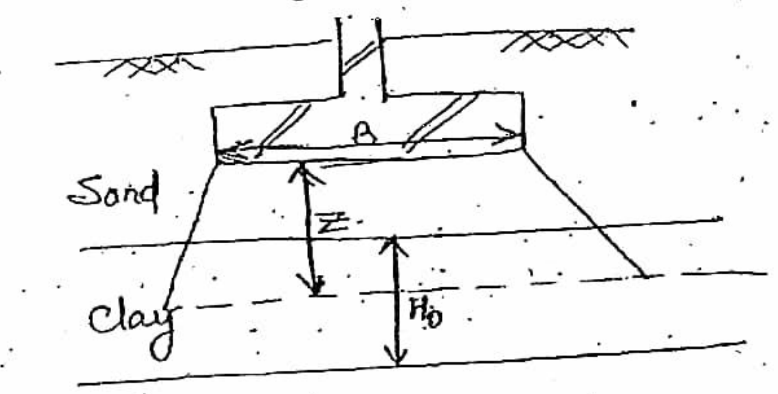
$$\Delta\bar{\sigma} = \frac{\text{force}}{\text{Area}} = \frac{q(B\times L)}{(B+2nz)(L+2nz)}$$

5) Ultimate Consolidation settlement

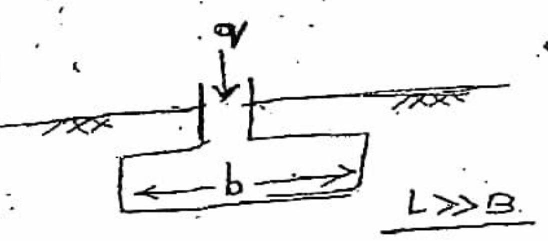
$$\Delta H = \frac{H_0 C_c}{1+e_0}\log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}_0}{\bar{\sigma}_0}\right)$$

NOTE 3 :- Determine Settlement of Clay layer

H_0 → clay layer.
z = taken at the total $\frac{\text{depth}}{2}$



NOTE 4 :- Settlement of Strip footing.



L>>B.

a) if load distribution (1 : n) is given.

$$\Delta\bar{\sigma} = \frac{q(B.L)}{(B+2nz)(L+2nz)}$$

$$\boxed{\Delta\bar{\sigma} = \frac{q.B}{B+2nZ}}$$

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b) if load distribution is not given.

$$\boxed{\Delta\bar{\sigma} = q}$$

NOTE-5 :- Total primary consolidation depends upon

1. compressibility of soil (soil property)
2. Magnitude of stress increase (loading condition)
3. Thickness of compressible layer (H_0)

$C_v = \frac{1}{d^2}$

NOTE-6 :- Ultimate primary consolidation will be same in 2 way and 1 way drainage condition if soil & loading condition is same.

Determination of Secondary Consolidation settlement

It occurs due to plastic readjustment of soil solids.
It is very slow time depending process.

$$S_s = \frac{H_{100}\, C_s}{1+e_{100}}\log\frac{t}{t_{100}}$$

where C_s = secondary Compression index.
H_{100} = thickness of compressible layer after ... consolidation

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e_{100} = void ratio after primary consolidation settlement

t_{100} = time required for 100% primary consolidation approximately $\left(\frac{t}{90}\right)$

t = time at which secondary consolidation is required.

WORKBOOK

4) $H_{o\,sand} = H_{o\,clay} = 3m$.

$a_{v\,sand} = \frac{1}{5} a_{v\,clay}$

$\frac{K_{sand}}{K_{clay}} = 10{,}000$

$\frac{t_{clay}}{t_{sand}} =$

$T_v = C_v \frac{t}{d^2}$

$t \propto \frac{1}{C_v}$

$C_v = \frac{K}{m_v \gamma_w} = \frac{K}{\frac{a_v}{1+e_o} \times \gamma_w}$

$t \propto \frac{a_v}{K}$

$C_v \propto \frac{K}{a_v}$

$\frac{t_{clay}}{t_{sand}} = \frac{a_{v\,clay}}{a_{v\,sand}} \times \frac{K_{sand}}{K_{clay}}$

$= 5 \times 10000$

$= 50000$

(d)

6) $T_v = C_v \frac{t}{d^2}$

$\frac{T_{v_i}}{T_{v_f}} = \frac{9 \times (d/4)^2}{t \times (d/2)^2} = \frac{9}{4t} = 1$

$t = \frac{9}{4} = 2.25$

= 2 yrs 3 mnth

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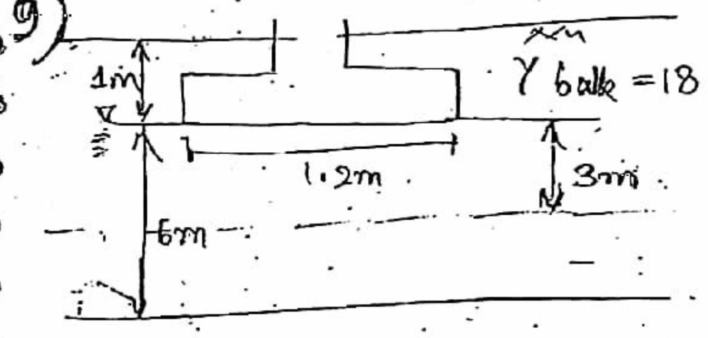
9)



$C_c = 0.08$ $\quad e_o = 0.40$

$\gamma_{sat} = 20\ KN/m^3$

$\bar{\sigma}_o = 3 \times 20 + 1 \times 18 - 3 \times 9.81$

$= 48.57$

$\Delta H = \frac{6 \times 0.08}{1+0.40} \log\left(\frac{48.57+40}{48.57}\right)$ $\quad \Delta\sigma = 40\,KN/m^2$

$\Delta H = 89\ mm$

10) $E_s = 5 \times 10^3\ kPa$. $\quad \mu = 0.4$. $\quad I_f = 1.75$

$S_i = \frac{qB(1-\mu^2)}{E_s} \times I_f$ $\quad kPa = KN/m^2$

$= \frac{40 \times 1.2(1-0.4^2)}{5 \times 10^3} \times 1.75$

37) $H_0 = 2m$ $\quad q = 0.5\ kg/cm^2$ $\quad U(50\%)$

$= 2000mm$ $\quad = 0.5\ kg/mm^2 \times 10^{-2}$ $\quad T_v = 0.196$

$d = \frac{H_0}{2}$ $\quad K = 3\ mm/yr$. $\quad \Delta h = ?$ $\quad t = 1yrs$.

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~~$\Delta H = H_0 m_v \Delta\sigma$~~

$T_v = C_v \frac{t}{d^2}$

$0.196 = \frac{K}{m_v \gamma_w} \times \frac{t}{d^2}$

$m_v = 1.56 \times 10^{-3} \frac{m^2}{KN}$

~~$\Delta H = H_0 m_v \times \Delta\sigma$~~

~~$\Delta H = 1.56 \times 10^{-3}$ mm~~

$= 2 \times 1.56 \times 10^{-3} \times 0.5 \times 9.81$

$100 \times \frac{\Delta h}{\Delta H_0} = U$

~~$\Delta h = 3.12$~~

$\Delta H = H_0\, m_v\, \Delta\sigma$

$= 2 \times 1.56 \times 10^{-3} \frac{m^2}{KN} \times 0.5 \times 9.81 \times \frac{10^{-3} KN}{10^{-4} m^2}$

$\Delta H = 153.03 \times 10^{-3}\ m$

$= 153mm$ (Total settlement But asked after 1)

After 1 yr $= \frac{153.03}{2} = 76.52\ mm$. Ans.

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41) $\sigma_0 = 50\,KN/m^2$ OC $\sigma_1 = 75\,KN/m^2$ NC $\sigma_2 = 40\,KN/m^2$

$H_0 = 2m$ $C_r = 0.05$ $C_c = 0.25$

$e_0 = 1.40$

$$\Delta H = \frac{H_0 C_r}{1+e_0}\log\left(\frac{75}{50}\right) + H_0 \times \frac{0.25}{1+e_0}\log\left(\frac{90}{75}\right)$$

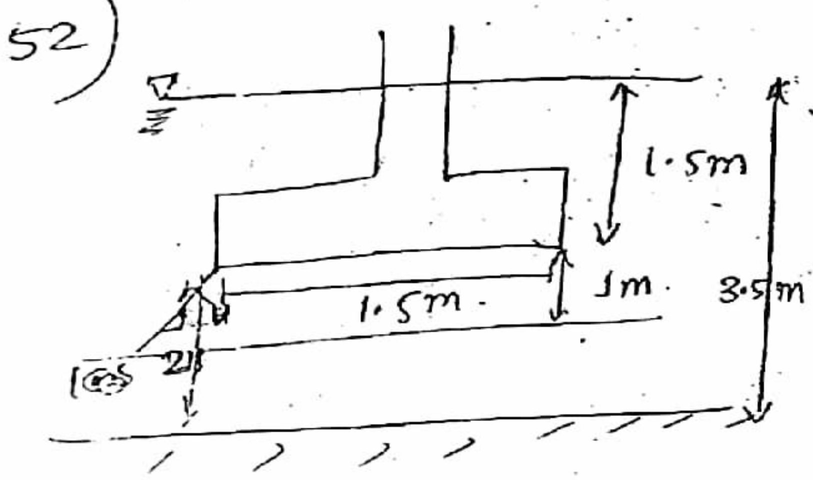
50 75 90

$= 7.34 \times 10^{-3} + 0.0164 = 0.0238m$

$= 23.8mm$ Ans

$= \cancel{0.046m}$

52)



$\gamma_{sat} = 20\,KN/m^3$

$e = 0.8$

$C_c = 0.07$

$q = 225\,KN$

$\bar{\sigma} = 2.5 \times 20 - 2.5 \times 10$ $\gamma_w = 10\,KN/m^3$

$= 25$

$\Delta\bar{\sigma} = \frac{225}{(2.5)^2} = 7.43$

$$\Delta H = \frac{H_0 \times C_c}{1+e_0}\log\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$\Delta H = \cancel{30.1\,mm} = 8.79\,mm$

$\cancel{= 8.79\,mm}$

44) $C_v = 0.003\,cm^2/sec = 0.003 \times 10^{-4}\,m^2/sec$

$\Delta h_2 = 50mm$ $t = ?$

$\Delta h_1 = 30mm$

$t = 2\,yrs$

$H_0 = 20m$

$U = \frac{\Delta h}{\Delta H} \times 100$

$= 60\%$

$T_{v_{60}} = 0.2826 = C_v \frac{t}{(20/2)^2}$

$t = 942 \times 10^4\,sec$

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$T_v = 0.003 \times 10^{-4} \times \frac{2}{10^2}$

$T_v = 0.1892 = \frac{\pi}{4} U^2$

$U = 49\% = \frac{\Delta h}{\Delta H}$

$\Delta H = 61.10mm$

for 50 mm settlement

$\frac{\Delta h}{\Delta H} = \frac{50}{61.10} = 0.81\%$

$U = 81\% \Rightarrow T_v = 0.605 = C_v \frac{t}{d^2}$

$t = 6.39 - 2$ yrs

50 mm settlement next yr = 4.39 yrs

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