Module-I : Soil Mechanics :- / GITE.

Diffrent Scientist's given Diff Defination's for soil's.

As per Agriculturist :- upper crust layer supporting plant life.

As per Geologist :- Loose / weakly cemented rock fragments which are jointed by weak Molecular forces.

i.e. Rock is subjected to weathering by various Natural Agencies like Air, wind, temp, Humidity will result in formation of "Soil"

** → The father of Soil Mechanics "Karl Terzaghi" is given an another Defenation as follows.

Soil is an "un-consolidated" Naturally occuring Material.

Loose

→ the classified soil's broadly in to two type's.

organic soil	In-organic soil

→ it consist's of Decomposed Material.

→ Asper an Agriculturist These type of soil's are benfit.

→ But - As a Civil Engg concern we dont need Thaye Type of soil's.

→ A Civil Engineer point of view Soil should be

IN compressible → it does not allow the settlement of structure

High in shear strength → it mean's that soil must be having sufficient strength to bear load's

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Problems faced by Geo-Technical Engineers:-

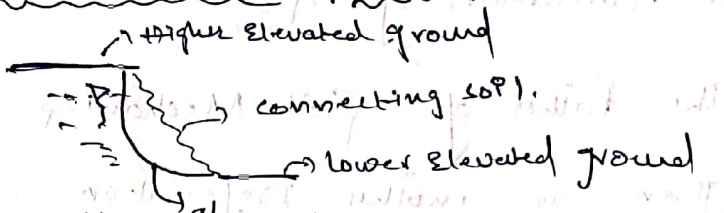
i) Settlement:- It means that the vertical downward movement of structure in to soil.

→ As per civil Engg concern we don't want much more settlement given by Indian standard code.

→ It depends on type of soil & type of footing.

ii) poor drainage characteristics of soil:-

Ex:- slope-failure:-



i) two diff elevated ground's are connected by soil & ii) water table is present @ near the soil then the pore water pressure develops and soil fail's.

→ Apart from these two case problem's in many cases soil is also used as an construction Material.

Ex:

(1) Dam:-



→ Commonly in village's to store the water Earthen Dam's are used. Due to potential head diff upstream water try to move Down-stream

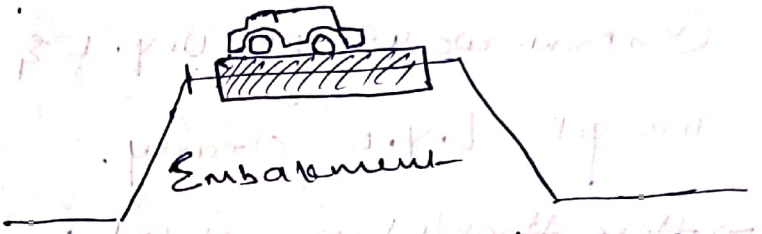
→ so the is a path followed by water is known as flow-line.

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→ So hue pore water pressure Exerted into Dam Meterial.

→ To avoid such things core is made @ center with Claye's soil's & remaining with grave's. Due to permeability Issoe's.

(2) Embakment:-



→ hue soil is osed an embakment for rod's or railways.

→ hue Soil is subjected to Dynamic load's.

→ Soil behaviar changes in Dynamic condition Than The Static condition.

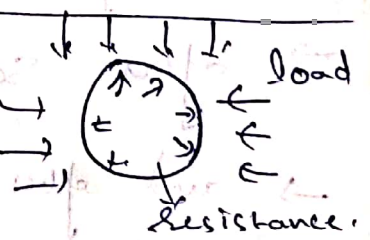
(3) Tunnel's:-

→ As The population ↑ There is no Space To Construct on-ground road way's so many of rail/road projects are underground Tunnel's.

→ Tunnel is a making a hole in To ground.

→ her Soil acting it's sey as load

& Soil " " " " resisting Material



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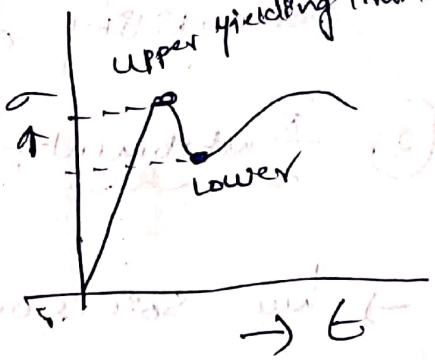
∴ Complexity of soil:-

① In case of Construction Materials like Steel & concrete

They have a certain behaviour under application of load

→ i.e as stress ↑ then strain also ↑

@ a point we will get U.Y.L & then we get L.Y.L clearly.



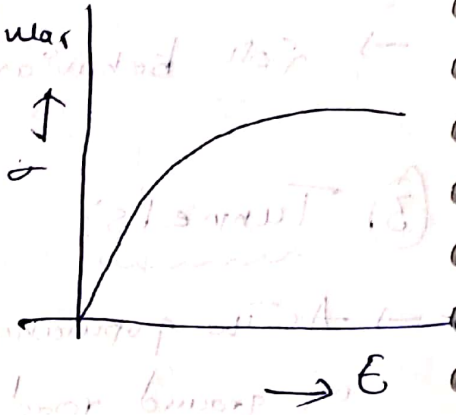
→ Here Hooke's law is valid ie $\sigma \propto \epsilon$

$$\sigma = E\epsilon$$

→ But in a case of soil there is no linear relationship b/w stress & strain.

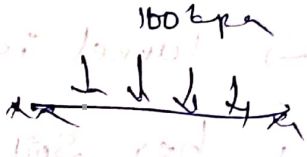
→ as stress ↑ & strain goes on ↑

i.e $\sigma \propto \epsilon$

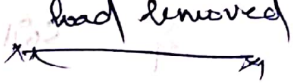


② Role of stress History:-

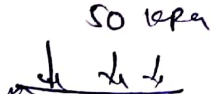
Ex:- Take a case where the soil is initially subjected to 100 kpa load.



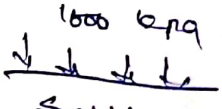
→ after some year's load is removed



→ and after some more year's if we apply load < 100 kpa then there is no settlement



→ if we apply load > 100 kpa then will be occurence of settlement.



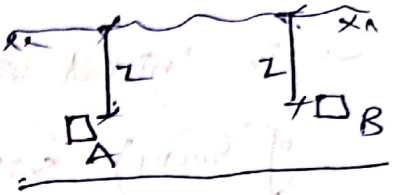
→ all these things we need to study in clayey soil's

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(3) Soil Sample Varies with Space:

→ Below G.L soil properties change's from one point to another point.

→ Ex:- Take two soil sample's below G.L. (A & B) @ same depths 'z'. Thus the shear strength parameters (c & ϕ (or)



(Strength of soil)$_A$ ≠ (Strength of soil)$_B$

→ This is known as spatial variation of soil

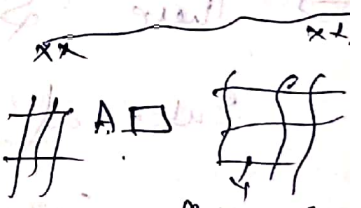
→ This may lead's to Differential settlement's

→ Ex:- Leaning tower of pisa.

(4) Testing:-

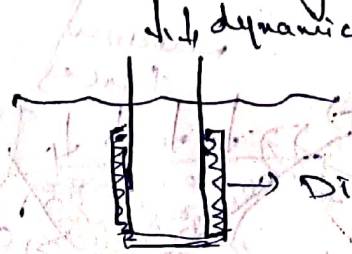
Ex:- if we taken a soil sample @ A.

But near by the sample A there may have joint's or fracture's if we tested @ A we got high strength. if we designed footings/foundations for this strength soil may fail



(5) Getting of Truly undisturbed soil Sample in field:-

→ while collecting soil samples thwth sampler's soil is disturbing due to dynamic activity



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:- formation of soil :-

→ before formation of soil we need know geological cycle.

→ it includ's following stages..

→ in initial stages due to lateral forces (Re-arrangement of rocks) soil is subjected to up-heavel forces.

→ Because of these forces a Land mass is created.

→ This Land mass in now subjected to weathering action Due to Sun, wind, rain, Temp--

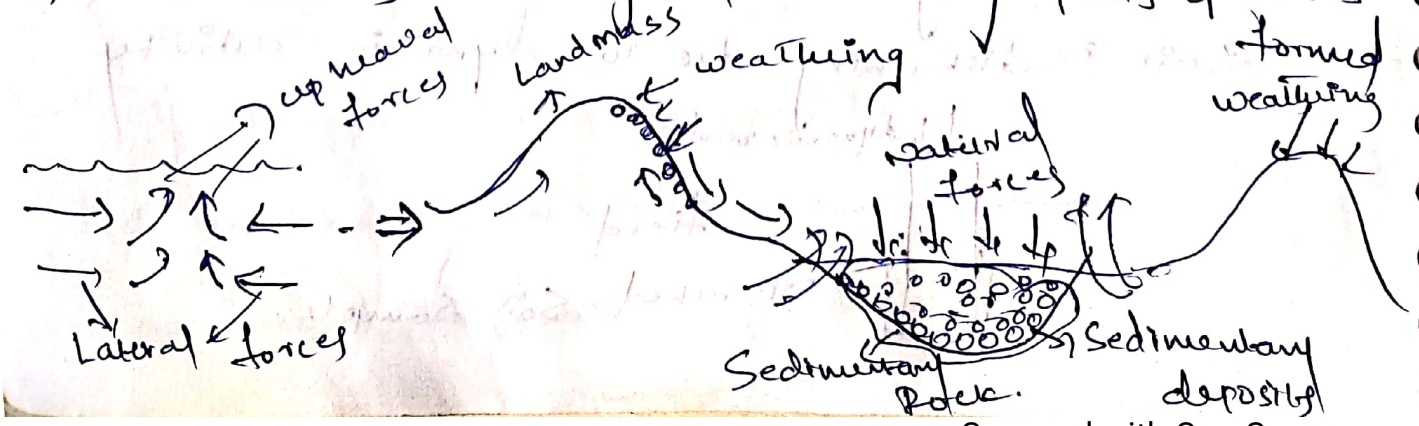
→ Due to this weathering soil in loose pockets is Eroded & fly's back into a low laying area & Deposited which results in formation of Sedimentary deposits.

→ These Sedimentary dyposits when subjected to External forces they'll form Sedimentary rock.

→ Again this Rock subjects to up-heaval forces & forms a hill/Land mass.

→ again weathering takes place. This process goes on continued.

→ So rock is disintigrated in to no. of parts & soil is



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→ Qs. here soil is formed mainly due to 2 Actions

① Due to physical forces. ② chemical forces.

→ Due to these forces sandy/ gravely soils are formed.

→ formed due to oxidation & Reduction ...

→ Here soils retains their Mineralogical properties of parent Rock.

→ due to this forces clays / silty soils form's.

→ They Dont have mineralogical Composition of parent Rock

→ So the soil's are formed Due to Weathering action.

→ During weathering the Diff soil's are formed Due to Diff weathering forces. They listed below.

(Transportation activity)

(Transportation)

① Alluvial soil → Transported by Running water (River)

② Aeolian soil → Transported by wind.

③ Glacial soils → Transported by ice.

④ Lucustrine soil's → Deposited in lake's.

⑤ Colluvial soil's → Deposited Thro Action of Land slides / slope wash.

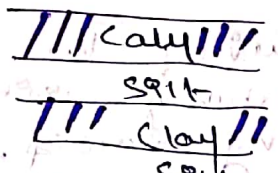
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Example's for Tansported Soil's.

① Loesses → Weakly cemented soil
It is cemented by → CaCO3 &
→ Mont morillonite (present in clay)
↓
It is major mineral which responsible for Swelling & Shrinkage of B.C soil

② Bentonite / Drilling mud: → formed by Volcanic Ash

③ Varved clay → It is Thin layer's of clay & Silt.



→ Deposited during rainy season / Monsoon
→ Deposited during summer.

④ Organic Soil's / Cumulose Soil's

(i) peat
→ highly organic
→ Dark Brown colour

(ii) Humus
→ Fully Decomposed organic Materials

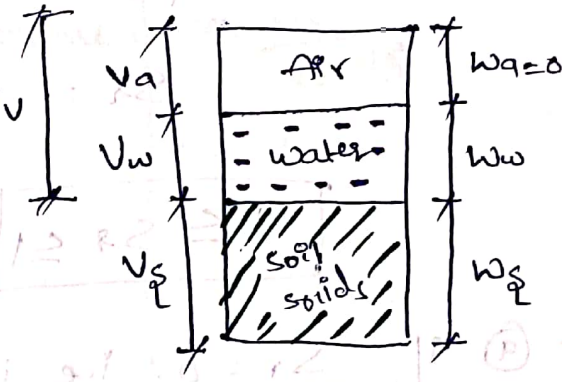
(iii) Muck.
→ we can find it in wetlands & poor drainage like swamps. (ಜೌಗು & ಕೆಸರು)

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Soil / solid - Water - Air Relationship :-

→ Wkt Matter is present in 3 states → solid, Liquid, Gas.

→ All These 3 state's of matter is for soil is defined in one diagram. known as phase Diagram. since there is 3 phases in soil it is known as 3-phase Diagram.



① Void Ratio (e) $V_a + V_w = V_v$

Wkt $V_v = V_a + V_w$.

$$e = \frac{V_v}{V_s} : \frac{\text{Volume of Voids}}{\text{Volume of Solid's}}.$$

Wkt Sandy soil's → more particle size
Clayey soil's → finer particles.

Even Though $e_{clay} >> e_{sandy\ soil}$.

Bcz finer particles are having less void's space bw- when we calculate Total void ratio all small void space's come together & They have more void ratio.

commonly $e > 0$ ie $\boxed{0 < e < \infty}$

$e = 0 \Rightarrow$ Absence of void's [Solid Material)

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(2) Water content:- (we) (w)

$$w = \frac{w_w}{w_s} = \frac{\text{weight of water}}{\text{weight of solid's.}}$$

$w \geq 0$

(3) Degree of saturation (S_r)

→ it is an indication of haomuch volume of water presenting in void's i.e

$$S_r = \frac{V_w}{V_v} = \frac{\text{Vol. of water}}{\text{Vol. of void's}}$$

$$\therefore \boxed{0 \leq S_r \leq 1}$$

(a) if $S_r = 0$. i.e if there is no water Then soil is having 2 phase Diagram,

i.e soil is Dry.



(b) if $S_r = 100\%$ or 1 Then also it shows Two phase Dia.

i.e here void's are completely filled with water

→ here soil said To be fully saturated soil.



(4) Air Content:- (a_c)

$$a_c = \frac{V_a}{V_v} = \frac{\text{Vol. of air}}{\text{Vol. of void's}}$$

$a_c = 0$ for saturated soil

$a_c = 1$ for dry soil

Where $S_r = \frac{V_w}{V_v}$.

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$$Sr + ac = \frac{V_w}{V_v} + \frac{V_a}{V_v} \Rightarrow \frac{V_w + V_a}{V_v}$$

We know from (3) phase Dia $V_a + V_w = V_v$. So

$$Sr + ac = \frac{V_v}{V_v} = 1$$

$$\boxed{Sr + ac = 1} \rightarrow (1)$$

or $\boxed{ac = 1 - Sr}$

(5) Porosity (n)

$$n = \frac{V_v}{V} : \frac{\text{Volume of voids}}{\text{Total volume}}$$

(or)

$$n = \frac{V_v}{V}$$

$$n = \frac{V_v}{V_v + V_s}$$

$$n = v$$

Total volume $\boxed{V = V_a + V_w + V_s}$

$\boxed{V = V_v + V_s}$

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

$$1 + e = \frac{V_v + V_s}{V_s} \Rightarrow \boxed{1 + e = \frac{V}{V_s}} \rightarrow (2)$$

We know $n = \frac{V_v}{V} \Rightarrow n - 1 = \frac{V_v}{V} - 1 \Rightarrow n - 1 = \frac{V_v - V}{V}$

We know $V = V_v + V_s$

$V_s = V - V_v$.

$\boxed{-V_s = V_v - V}$ so $n - 1 = \frac{-V_s}{V}$

$$\boxed{1 - n = \frac{V_s}{V}} \rightarrow (3)$$

On comparing eqns (2) & (3) we get

$$\boxed{1 + e = \frac{1}{1 - n}}$$

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(6) percent Air void (n_a)

$$n_a = \frac{V_a}{V} = \frac{\text{Vol air}}{\text{Total Voil.}}$$

$n_a = n \times a_c$ Bcz $n = \frac{V_v}{V}$ & $a_c = \frac{V_a}{V_v}$

$n_a = n[1 - S_r]$ → (4) $\frac{V_v}{V} \times \frac{V_a}{V_v} = \frac{V_a}{V} = n_a$

→ In soil Mechanics

weight → N.

Mass → kg.

$kg \times \frac{m}{sec^2} = N$. i.e $W = mg$

Density [Mass Density] = $\frac{M}{V} = \rho_w$ (Bulk Mass Density).

it is the Ratio b/w mass of soil solid to volume of soil solid

Unit weight of soil = $\frac{W}{V}$ = $\frac{\text{Total weight of soil}}{\text{Total volume of soil}}$

w.k.t $W = W_w + W_s$ &

$V = V_v + V_s$ = $\frac{W_w + W_s}{V_w + V_s}$

& $W_a = 0$.

it is also called as Bulk Unit weight (γ_b)

for fully saturated soil $\gamma_b = \gamma_{sat}$. i.e $V_a = 0$.

$\gamma_{sat} \Rightarrow (W_w + W_s / V_w + V_s)$

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Specific gravity (G_s) :- The ratio b/w weight of soil solids

To the weight of water is G_s.

$$G_s = \frac{W_s}{W_w} = \frac{\text{weight of soil solids}}{\text{wt of water}}$$

W.K.T $W_w = V_w \times \gamma_w$. so $G_s = \frac{W_s}{V_w \times \gamma_w}$

@ $4^\circ C$ $\gamma_w = 9.81\ kN/m^3$.

Mass density $\rho_w = 1\ \frac{gm}{cc}$ (or) $1000\ \frac{kg}{m^3}$.

(or)

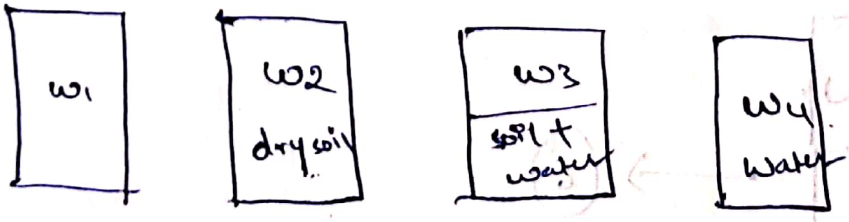
It is also defined as U.W of soil solids to unit wt of water

i.e $G_s = \frac{\gamma_s}{\gamma_w}$ ⟶ (5) But $\gamma_s = \frac{W_s}{V_s}$ so

γ_s → unit wt of soil solids

$$G_s = \frac{W_s}{V_s \times \gamma_w}$$

Det. of sp. gravity :-



where W_1 = wt of Empty sp. gravity bottle.

$W_2 = W_1 +$ dry soil

$W_3 = W_2 +$ water [without any Entrapped Air]

$W_4 = W_1 +$ water

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G_s = wt of soil solids / wt of water Volume Equalent To That of water

$$G_s = \frac{w_2 - w_1}{(w_4 - w_1) - (w_3 - w_2)}$$

for most of soil's G_s ranges from 2.5 to 2.9

G_s for sands = 2.65

Other G_s is - 'True' sp. gravity.

But G_m is apparent / mass sp. gravity

$$G_m = \frac{\gamma_b}{\gamma_w} \qquad \gamma_b = \frac{W}{V}$$

if soil is dry $G_m(dry) = \frac{\gamma_d}{\gamma_w}$

if soil is saturated $G_m[saturated] = \frac{\gamma_{sat}}{\gamma_w}$

Submerged / Buoyant unit weight (γ')

$$\gamma' = \gamma_{sat} - \gamma_w \quad \longrightarrow (6)$$

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

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Bulk unit wt- (γ_b) (γ_{bulk}) (γ)

$$\gamma_b = \frac{W_0}{V_0} \Rightarrow \frac{W_w + W_s}{V_w + V_s} = \gamma_b$$

Dry unit weight- (γ_d)

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

Saturated unit weight- (γ_{sat})

$\gamma_{sat} = \frac{W}{V}$ But- Here void spaces are completely filled with water So

W.K.T $V = V_a + V_w + V_s$ (with $V_a \to 0$)

$V = V_w + V_s$

$$\gamma_{sat} = \frac{W}{V_w + V_s}$$

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① Relationship b/w e, Sr, w, Gs

Wkt $e = \frac{V_v}{V_s}$ if we multiply & Devid with V_w

$$e = \frac{V_v}{V_s} \times \frac{V_w}{V_w} \Rightarrow \left(\frac{V_v}{V_w}\right)^{\frac{1}{Sr}} \times \frac{V_w}{V_s}$$

$$e = \frac{1}{Sr} \times \frac{V_w}{V_s}$$

$$e = \frac{1}{Sr} \times \frac{w_w}{\gamma_w} \Big/ w_s/\gamma_s.$$

$$e = \frac{1}{Sr} \times \frac{w_w \times \gamma_s}{w_s \times \gamma_w}$$

$$e = \frac{1}{Sr} \times w \times \frac{\gamma_s}{\gamma_w}.$$

$$\boxed{e = \frac{1}{Sr} \times w \times G_s.}$$

$$\boxed{e \times Sr = w \cdot G_s.} \rightarrow (1)$$

Sum Becomes

Bw wkt $Sr = \frac{V_w}{V_v}$

$\frac{1}{Sr} = \frac{V_v}{V_w}$

$\gamma_w = \frac{w_w}{V_w}$

$V_w = w_w/\gamma_w.$

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

$V_s = \frac{w_s}{\gamma_s}.$

Bw wkt $w = \frac{w_w}{w_s}$

Eqwkt $G_s = \frac{\gamma_s}{\gamma_w}$

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(3) Relationship b/w f_{bulk}, G_f, e_1 S_r & w :-

W.K.T $f_b = \frac{w}{v} \Rightarrow \frac{w_w + w_s}{v_a + v_s + v_w}$ — Bcz $w_a = 0$

But $v_a + v_w = v_v$. so $f_b = \frac{w_w + w_s}{v_v + v_s}$

$$f_b = \frac{w_s\left[\frac{w_w}{w_s} + 1\right]}{v_s\left[\frac{v_v}{v_s} + 1\right]}$$

$\therefore \frac{w_w}{w_s} = w$

$\therefore \frac{v_v}{v_s} = e$

so $f_b = \frac{w_s[w+1]}{v_s[e+1]}$

$\therefore f_s = \frac{w_s}{v_s}$

$\therefore G_f = \frac{f_s}{f_w}$

$f_s = G_f \cdot f_w$

so $f_b = f_s\left[\frac{w+1}{e+1}\right]$

$$\boxed{f_b = G_f f_w \left[\frac{w+1}{e+1}\right]} \rightarrow (8)$$

But W.K.T $e_1 S_r = w \cdot G_f \Rightarrow w = \frac{e S_r}{G_f}$

Sub in Eqn (8) we get

$$f_b = G_f f_w \left[\frac{\frac{e_1 S_r}{G_f} + 1}{e+1}\right]$$

$$f_b = \cancel{G_f} \cdot f_w \left[\frac{e \cdot S_r + G_f}{\cancel{G_f}(e+1)}\right]$$

$f_b = G_f$

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$$\rho_b = \left(\frac{G_s + e \cdot S_r}{1+e}\right)\rho_w \longrightarrow (9)$$

from Eqn no (9) we can derive two eqn's i.e

$\rho_{sat} \Rightarrow$ in ρ_{sat}, $S_r = 1$ so Eqn becom's

$$\rho_{sat} = \left(\frac{G_s + e}{1+e}\right)\rho_w \longrightarrow (10)$$

if soil is in dry condition then $S_r = 0$. so

$$\rho_d = \frac{G_s \rho_w}{1+e} \longrightarrow (11)$$

Relation B/w ρ_d, ρ_{bulk}, w.

$$\rho_b = \frac{W}{V} = \frac{w_w + w_s}{V_v + V_s}$$

$$\rho_b = \frac{w_s\left[\frac{w_w}{w_s} + 1\right]}{V}$$

B/w w.r.t

$$\begin{cases} \frac{w_w}{w_s} = w \\ \frac{w_s}{V} = \rho_d \end{cases}$$

so $$\rho_b = \rho_d \cdot [w + 1] \longrightarrow (12)$$

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Relationship b/w G_s, n_a, r_d & w

W.K.T Total volume

$$V = V_a + V_w + V_s$$

it is divided by V on B.S

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

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

$$1 = n_a + \frac{V_w}{V} + \frac{V_s}{V}$$

$$1 - n_a = \frac{V_w}{V} + \frac{V_s}{V}$$

$$1 - n_a = \frac{w_w/r_w}{V} + \frac{w_s/r_s}{V}$$

$$1 - n_a = \frac{w \cdot w_s/r_w}{V} + \frac{w_s/r_s}{V}$$

$$1 - n_a = \frac{w_s}{V}\left[\frac{w}{r_w} + \frac{1}{r_s}\right]$$

$$1 - n_a = r_d\left[\frac{w}{r_w} + \frac{1}{r_s}\right]$$

$$1 - n_a = \frac{r_d w}{r_w} + \frac{r_w \times r_d}{r_w \times r_s}$$

if multiply & devide with r_w

$$1 - n_a = \frac{r_d}{r_w}\left[w + \frac{r_w}{r_s}\right]$$

$$1 - n_a = \frac{r_d}{r_w}\left[w + \frac{1}{G_s}\right]$$

But $n_a = \frac{V_a}{V}$

But $r_w = \frac{w_w}{V_w}$; $\therefore V_w = \frac{w_w}{r_w}$

But $r_s = \frac{w_s}{V_s}$; $V_s = \frac{w_s}{r_s}$

But $w = \frac{w_w}{w_s}$; $w_w = w \cdot w_s$

But $r_d = \frac{w_s}{V}$

But $G_s = \frac{r_s}{r_w}$; $\frac{1}{G_s} = \frac{r_w}{r_s}$

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$$1-na = \frac{\gamma_d}{\gamma_w}\left[\frac{w \cdot G_s + 1}{G_s}\right]$$

$$(1-na)\,\gamma_w \times G_s = \gamma_d\,[w G_s + 1]$$

$$\boxed{\gamma_d = \left[\frac{(1-na)(\gamma_w \times G_s)}{(1+wG_s)}\right]} \longrightarrow (13)$$

This expression is useful in Compaction of soils.

Review of Relations

(1) $e \cdot s_r = w G_s$

(2) $a_c = 1 - s_r$

(3) $n_a = n \cdot a_c$

(4) $\gamma_{bulk} = \left(\frac{G_s + e \cdot s_r}{1+e}\right)\gamma_w$

(5) $\gamma_d = \frac{G_s \gamma_w}{1+e}$

(6) $\gamma_{sat} = \left(\frac{G_s + e}{1+e}\right)\gamma_w$

(7) $\gamma_d = (1-na)\frac{G_s \gamma_w}{(1+wG_s)}$

(8) $\gamma_{bulk} = \gamma_d (1+w)$

(9) $\gamma_{bulk} = \gamma_w\left(\frac{1+w}{1+e}\right)G_s$

(10) $\gamma_d =$

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∴ Short cut To Remember formulas below

W.K.T $e \cdot Sr = w \cdot G_s$.

Step ① Add G_s on B.S

$$G_s + e \cdot Sr = wG_s + G_s$$

$$G_s + e \cdot Sr = G_s[1+w]$$

Step ② Devided by $1+e$ on B.S.

$$\left(\frac{G_s + e \cdot Sr}{1+e}\right) = \frac{G_s \cdot [1+w]}{[1+e]}$$

if u compare with sm's ⑧ & ⑨ u get

$$\frac{\gamma_d}{\gamma_w} = \left(\frac{G_s + e \cdot Sr}{1+e}\right) \quad \& \quad \frac{\gamma_d}{\gamma_w} = \frac{G_s \cdot [1+w]}{[1+e]}$$

W.K.T $\frac{\gamma_d}{\gamma_w} = G_m$ (Caparent Sp. gravity).

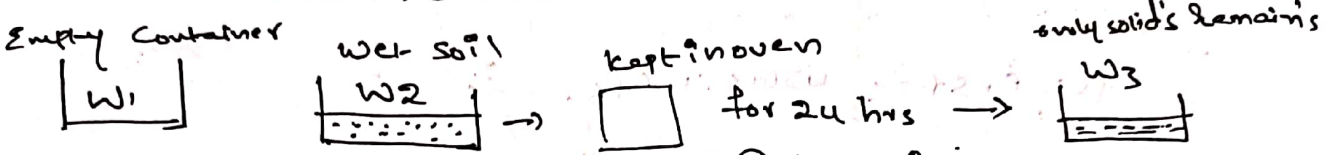
So $$\boxed{G_m = \left[\frac{G_s + e \cdot Sr}{1+e}\right] = \frac{G_s\,[1+w]}{[1+e]}}$$

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Det. of water content (w) :-

To Det w.c of soil in laboratory we have two Methods.

① Oven Drying Method :-



$$\text{Water w.c} = \frac{W_w}{W_s} = \frac{W_2 - W_3}{W_3 - W_1} \times 100 = \%$$

Why we need to kept only @ 105 - 110°C. ?

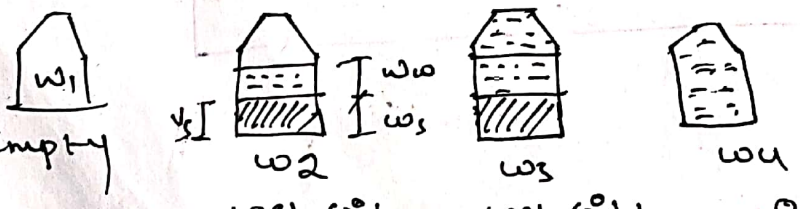
Answer :- Commonly we have 3 type's of water preseting in soil

1) Structural water
2) Held water (capilary force) → (held due to capilary force)
3) free flowing water

→ These two type's of water removed in oven @ 105° to 110°C

→ Structural water Not used in Soil mechanics ; it was taken care by Aggricultural Engg. To remove structural water we need to maintain 666° - 700°C

②. Pycnometer Method :- (Vol = 900 - 1000 cc or 1 lit).



$$\text{Water } w = \frac{W_w}{W_s} = \frac{(W_2 - W_1) - W_s}{W_s}$$

$$= \left[\frac{W_2 - W_1 (G_s - 1)}{(W_3 - W_4)(G_s)} - 1 \right] \times 100$$

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:- Index properties of soil :-

There are two properties of soil.

1) Engineering property.

→ The Engineering properties of soil's are used To Evaluate the shear strength of soil.

→ These properties can be determined in field Through field Tests.

Ex:- SPT, CPT, PLT, PLT

→ On Conducting above Test's we will get The relevent soil parameter's which are use full for Design of a foundation.

→ But conducting these Tests are not Easy to conduct & Include's huge money.

→ In-order to reduce Ebbett's we Determine

2) Index properties of soil :-

→ These can be determined in Lab. so indirectly we can det shear strength of soil. by correlating Index-properties & shear strength parameter's.

To Save Time & money.

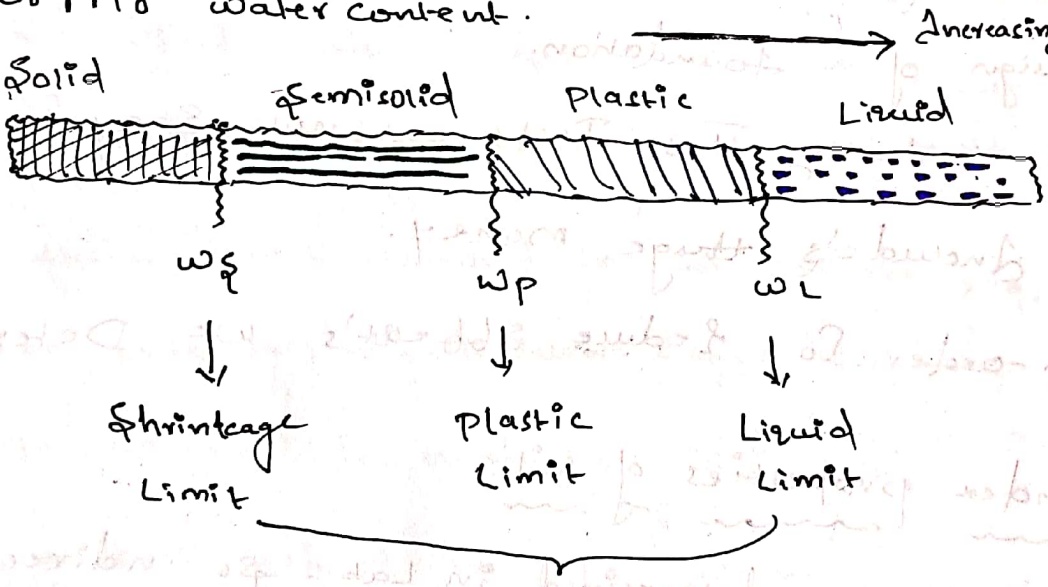
→ Index properties are given Below

1) Water content
2) Specific Gravity
3) Density (INSITU) (field)
4) Grain-size
5) Atterberg Limit's (clay's).
   - → Shrinkage limit (w_s)
   - → plastic limit (w_p)
   - → Liquid limit (w_L)

"Atterberg" limit's :- (clayey soils).

→ These limites are Discovered by scientist "Atterberg".

→ Here we see how total volume of soil will vary w.r.to water content.



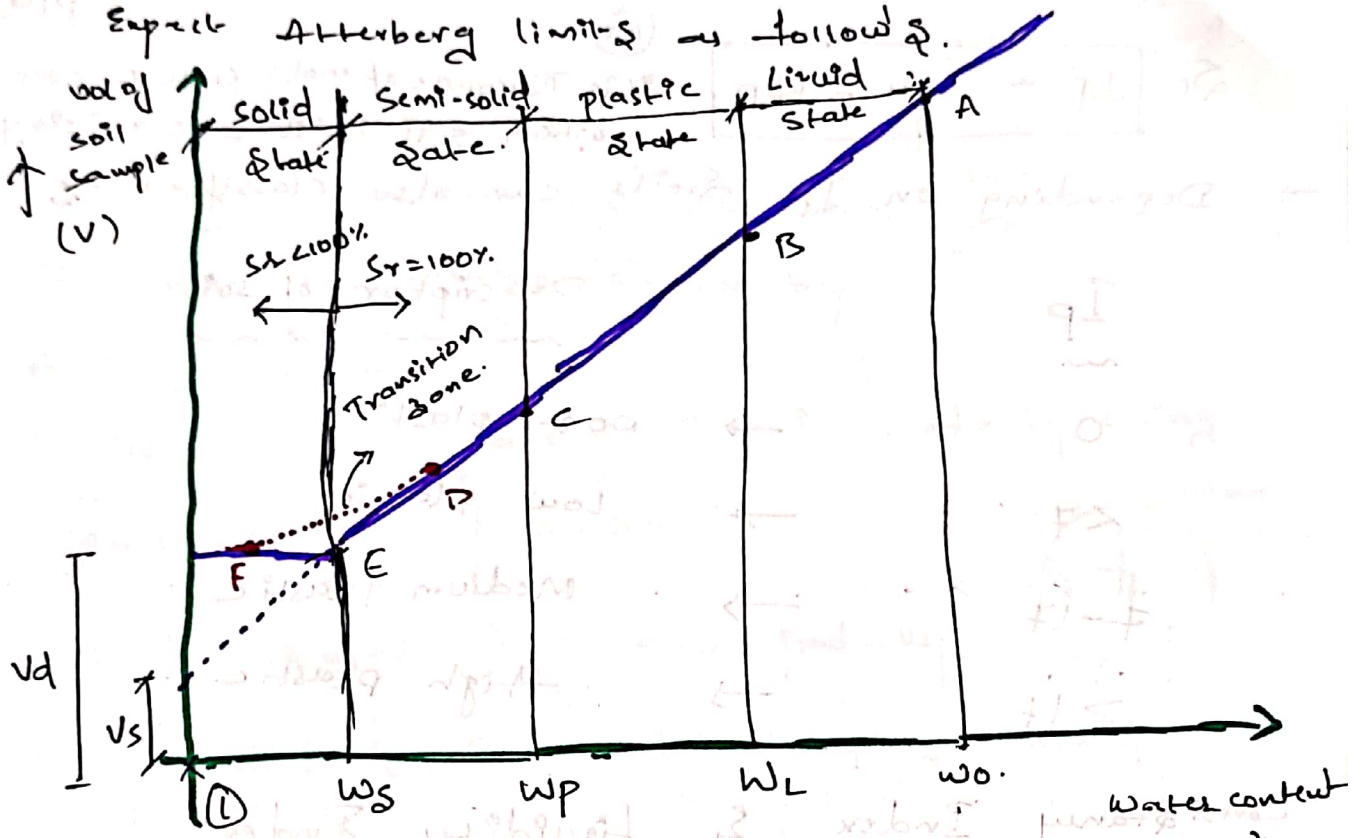
Def:- Atterberg Limit's are water content's where the soil behaviour changes.

-) for all soil's shear strength @ LL is

$$w_L = 2.7 \text{ kpa or } kN/m^2$$

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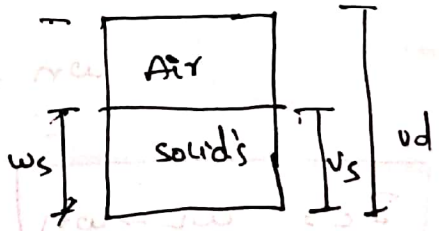
→ To Explain The Atter-berg Limit's: Take a soil sample which is initially fully saturation condition i.e $S_r = 100\%$ and Then allow it to drain. Then we can Expect Atterberg limits as follows.



Vd = volume of dry sample; Vs = volume of soil solid's.

→ in whole proces on drainage Two Things are not changing

ie Vs (vol of soil solids) & Ws (wt of soil soild).



Ws & Vs = not changing.

→ Bcz solid's are incompressible.

→ if we draw phause Dia for ①

→ up to Ws The soil is compleatly saturated. after this Limit Drying is occurd. i.e water is replaced by Air.

→ poin 'D' shows's Entry of Air stary & 'f' point shows's Compleat air Entry i.e solid state.

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Definations:-

Plasticity Index:- (I_p).

→ it gives the idea about the range @ which soil is plastic

So $\boxed{I_p = w_L - w_p}$ (or) This the range of moist. content over which soil exhibits plasticity.

→ Depending on I_p soils can also classified as

I_p		Description of soil.
0	→	non-plastic
< 7	→	low plastic
7 – 17	→	Medium plastic
> 17	→	High plastic.

Consistancy Index & Liquidity Index:-

→ These both limits depends on w.c @ natural state.

Here natural w.c is w_n i.e

w_n = natural w.c / insitue w.c. Shortcut-

$$I_c = \frac{w_L - w_n}{I_p}$$

$I_p = w_L - w_p$

$$I_L = \frac{w_n - w_p}{I_p}$$

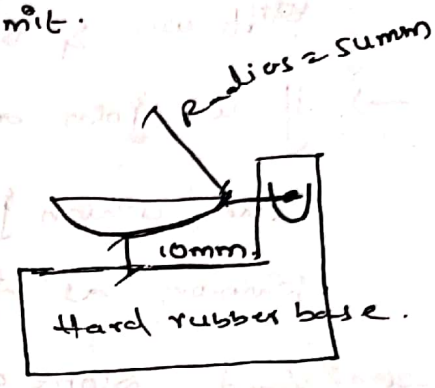
$$I_c + I_L = 1$$

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Liquid limit:- The limiting w.c between Liquid state & plastic states indicates the liquid limit.

Laboratory Det. of LL:-

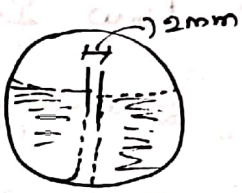
Casagrande's Method:-



→ Soil passing through 425 μ Sieve has Taken.

→ it is placed in casagrande's cup by adding un-known w.c.

→ in These cup A groove of 2mm is made by using casagrande's Tool / ASTM Tool.

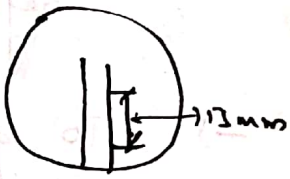


→ Now it is Rotated @ 2 Rev/s Gradually

→ Now this cup is Subjected to Tamping with a height of 10 mm.

to Induce moment in Seperated soil.

→ The No. of Blows which Req To close 13 mm of Seperated soil is Mesured. along Total length of groove. (13mm or ½ inch)



→ If we know corresponding w.c by oven drying

→ By repating this Tests on ↑ w.c we can see the variation in no. of Blows.

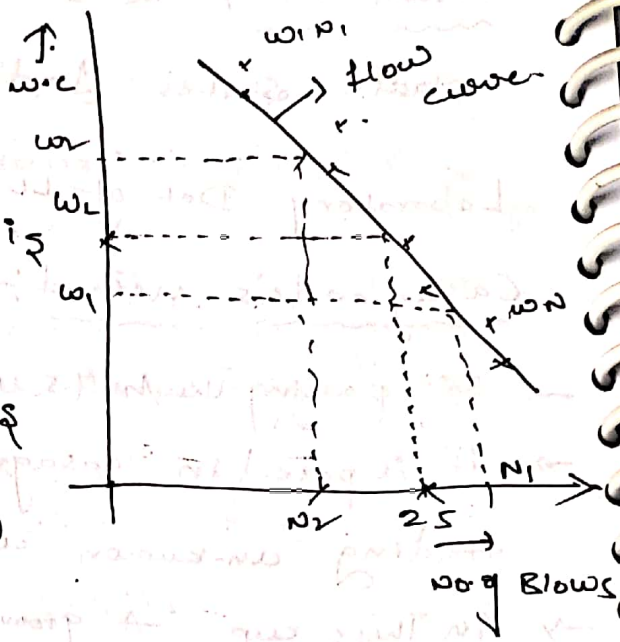
→ Now a graph is Drawn b/w W.C Vs No. of Blows.

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→ So we get no. of points for diff w.c & Blow's.

→ if we join all of them (or) line which join's all points is known as flow curve.

→ And slope of flow curve is called flow Index. (I_f)



→ flow Index indicate's rate @ which soil looses shearing resistance with an ↑ in w.c.

→ Graph is plotted on logarithmic scale.

→ Slope of flow curve is $w_2 - w_1 = -I_f [\log (N_1/N_2)]$

→ w.c corresponding to 25 no. of Blows is defined as Liquid limit.

$$I_f = \frac{w_2 - w_1}{\log_{10}\left(\frac{N_1}{N_2}\right)}$$

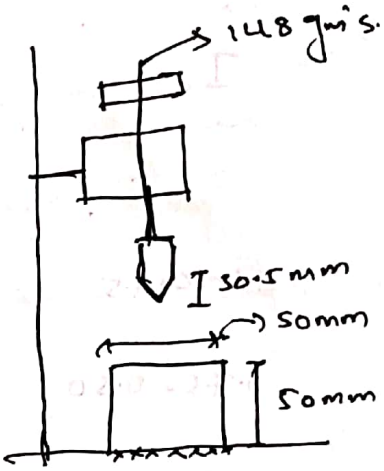
→ To get an flow curve we need to have min of 3 value above 25 blow's & 25 blow's below.

Demerit:

→ we are not doing it in original state we are doing it in remoulded/ disturbing state

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Cone-penetrometer Test:-



→ here the penetration of a standard cone into a saturated soil sample is measured for 30sec.

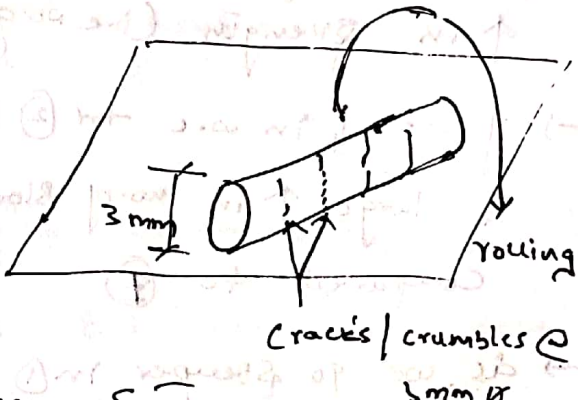
→ If the penetration is < 20mm. the wet soil is taken out & mixed thoroughly with water and the test is repeated till the penetration is in b/w 20-30 mm.

→ The w.c corresponding to 25mm penetration is taken as Liquid limit w_L.

Determination of plastic limit:

Def:- it is the limiting w.c b/w plastic & semi solid state of soil (or) As per Technical

→ water content @ which the soil crumbles when rolled into thread's of 3mm Dia.



→ it is done in repeated trail's and avg value of no. of trail's are taken.

→ Crumbld portion is kept in oven so that we can Det. w.c.

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Soil classification Based on Soil Consistancy

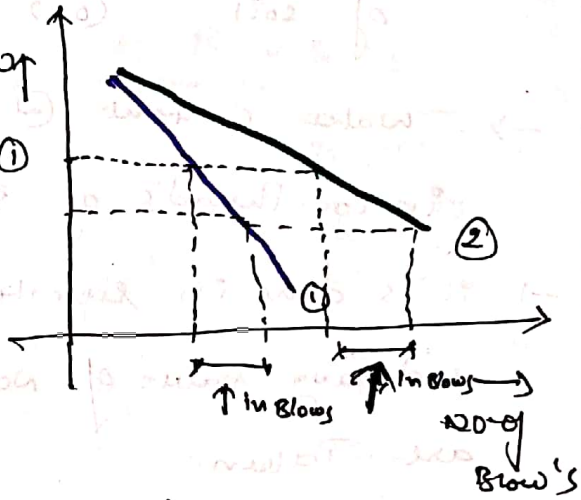
I_c	I_L	Consistancy
>1	<0	very stiff
1-0.75	0-0.25	Stiff
0.75-0.50	0.25-0.50	Medium Stiff
0.50-0.25	0.50-0.75	Soft
0.25-0	0.75-1.00	very soft
<0	>1.0	Liquid state.

Toughness Index:- (I_T)

To explain this we need to go for flow curve., if we take Two curve's one is steeper & another is flatter

→ if we change the (I_f) in w.c will up ↑ in strength (i.e no. of Blow's) ↑ for ①

→ if we ↓ in w.c for ② we can see a large ↑ in no. of Blows (strength) compared to ①



→ as we go steeper in ① strength goes on ↓.

→ So we may conclude that- The Toughness (shear strength) is Indirectly proportional To Slope of flow Curve.

i.e $\text{Toughness} \propto \frac{1}{I_f}$

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So Toughness Index (I_T) is given by

$$I_T = \frac{I_P}{I_f}$$ ★★

if two soil's having same plasticity index. Then w.k.t

$I_T \propto \frac{1}{I_f}$ ∴ So the soil which got flatter flow curve has got higher shear strength.

Shrinkage limit & Determination:-

→ from Liquid to plastic — Semisolid state Soil remain's fully Saturated. & in reducing the w.c in these three stages changes the volume (Δ, in vol) of soil sample.

→ Then there is a stage occure's where further reduction in w.c does not cause any reduction in volume @ that point Soil changes to Semi Solid state to Solid state.

→ after this point in we dry the soil to reduce w.c Soil volume remain's same. But pore water gets reduced.

→ i.e soil does not shrink anymore.

→ when w.c of soil is reduced and @ a particular w.c soil stop's shrinking that w.c is called Shrinkage limit of that Soil.

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(or) SL is lowest w.c @ which soil is fully saturated.

→ after this limit soil begins to dry, air enters into void's & soil colour also changes.

Determination:

→ To Det. S.L soil is air Dried and Seive through 425 μ Is Sieve. To get fine grained soil's only.

→ Take 30 gm's of Sieved soil in Evaporating dish

→ Now add unknown w.c To soil Such that To bring Consistancy To flow.

→ a Shrinkage dish made up of Steel is Taken

→ Empty weight noted as W_{dish}.

→ To Det it's vol filled with Hg.

→ Excess of Hg is removed by Pressed glass on Top of dish.

→ wt of Hg in Dish is Taken & noted Down as $W_{mercury}$

from this Volume det. as $V_{mcury} = \frac{W_{mcury}}{G_{mcury}}$.

so $V_{mcury} = V_{dish}$.

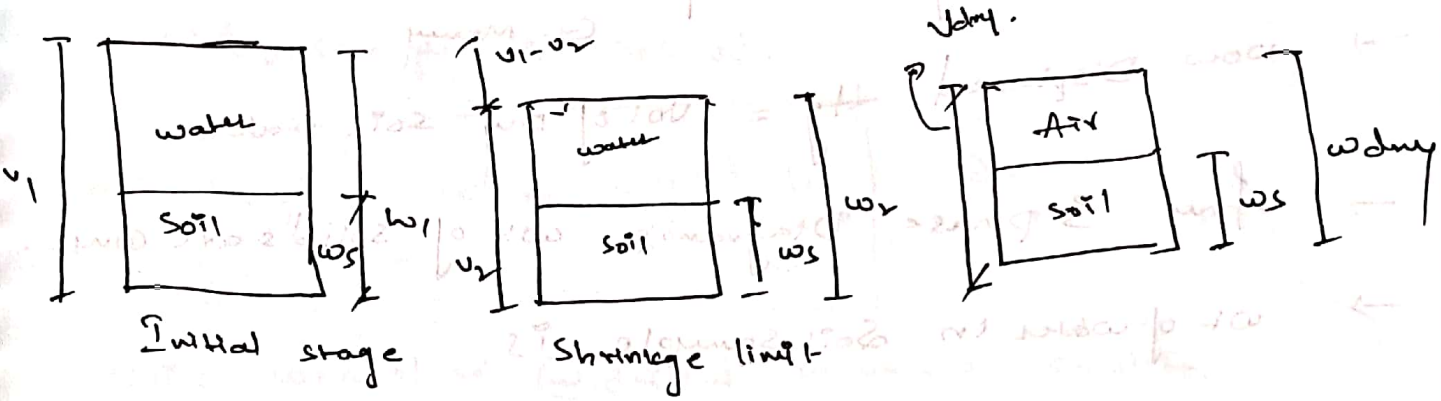
→ Now Soil Sample is Taken in Dish with 3 layers and Excess soil is trimed off.

→ By soil filling wall's of dish applied with Lubricant

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→ Now Dish was Removed & Take the weight of soil pat as W_1. & volume of soil as V_1

$V_1 = V_{Dish}$.



Initial stage Shrinkage limit

→ Now soil pat in dish is allow to Dry in air till colour changes from Dark to light. Then Dish is Dried in oven @ 110°.

→ Now pore water is dried up in place of water air is filled. Here Vol doesnot changes.

→ Wt of Dry soil is taken by Removing Dish as W_{Dry}.

Now $W_{Dry} = W_s$.

→ we allready know's weight of Dry soil i.e W_{Dry}.

But V_{dry} (vol) doesn't known.

it is Mesured by Mercury Displacement method:-

→ Here we Take a glass container & fill it with Hg. & Place it in a pan. Such that Hg should not Expelled.

→ Now Dry soil pat is Removed & placed in Top of Mercury Surface & Submerged it into Hg by

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pressing with glass plate.

→ Mercury is displaced is collected and weighed.

→ Vol is det. by $V_{mercury} = \frac{W_{mercury}}{G_{mercury}}$

→ Now Displaced Hg = Vol of Dry soil pat.

→ frm 3 phase Diagram's. wt of solid's are const.

→ wt of water in soil sample is

wt of initial soil sample − wt of Dry soil sample

$W_1 - W_{Dry}$.

→ in 2nd stage water is dried so volume reduced.

it was given by $(V_1 - V_2)$

→ To get wt of dried water we do $(V_1 - V_2) \times \gamma_w$. i.e

wt of water need to lose @ initial state to reach

at Shrinkage limit $= (V_1 - V_2)\gamma_w$.

→ Now wt of water @ S.L $\boxed{(W_3) = (W_1 - W_{dry}) - (V_1 - V_2)\gamma_w}$.

Now w.k.t S.L is nothing But w.c.

$$\therefore w.c = \frac{W_w}{W_s} = \frac{(W_1 - W_{dry}) - (V_1 - V_2)\gamma_w}{W_{Dry}}$$

$$S.L = \underbrace{\frac{(W_1 - W_{dry})}{W_{dry}}}_{①} - \underbrace{\frac{(V_1 - V_2)\gamma_w}{W_{dry}}}_{②}$$

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→ ① represents w.c of soil @ initial state & it was represented by w_i.

→ & in ② V_2 @ S.L is equal to vol of sample @ Dry state. So V_2 can written as V_d.

Now
$$S.L = w_i - \frac{(V_1 - V_d)\gamma_w}{W_{dry}}$$ **

w_i = w.c of soil @ initial saturation condition. (Det by oven drying).

V_1 = vol of soil @ initial saturation condition.

V_d = Dry vol of soil sample.

W_{dry} = Dry wt of soil sample

γ_w = Unit wt of water.

Shrinkage Ratio: (R).

R = vol change expressed as percent of dry volume to the corresponding change in moist. content from initial value to S.L.

$$R = \frac{w_d}{V_d \gamma_w} * 100$$ But $\frac{w_d}{V_d} = \gamma_d$ @ S.L.

So
$$\boxed{R = \frac{\gamma_d}{\gamma_w}} \quad [@ S.L)$$

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Index properties.

- Grain. Size Distribution
  - Mechanical Analysis (coarse grained soil)
    - Dry method
    - wet Method
  - Hydrometer Analysis (fine grained soil)
- consistancy limits (fine grained soil)
  - → Liquid limit
  - → plastic limit
  - → plasticity Index
  - → Shrinkage limit

Grain Size Distribution (GSD):-

→ for grain's > 0.075mm we can use Sieving method

→ for grain's in Range of 0.075 mm > d > 0.5 µm Hydrometer Test is used.

Sieve Analysis:- (coarse grained soil's d > 0.075 mm)

→ A sieve Analysis is laboratory Test That Mesure The G.S.D of soil by passing it Thrh a Series of sieve's.

→ it is a Screening process in which Corse fraction of soils are Seperated by Mean's of Series of sieves.



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Procedure:-

→ Take an oven-dried soil sample of any weight in to top sieve and shake it either manually or by mechanical means ~~for~~ until all soil grains are retained on finest sized sieve which they can possibly pass.

→ Now weight retained on each sieve is calculated.

→ for each sieve compute the % finer (N).

→ Then plot the graph b/w % finer (y-axis) & particle (or) sieve size (x-axis (log graph) to get a G.S.D curve.

Ex:- wt of dry soil is taken for sieve Analysis = 500 gms.

Sieve no.	Grain dia (mm)	weight retained (g)	cumulative weight (g)	cumulative wt passing (g) (500 - cumulative)	(490/500) x 100 % finer by weight (%)
4.75	4.75	10	10	490	98%
2.36	2.0	165	175	325	65%
1.18	1.0	100	275	225	45%
425 μ	0.425	85	360	140	28%
300 μ	0.300	60	420	80	16%
150 μ	0.150	20	440	60	12%
75 μ	0.075	40	480	20	4%
pan	pan	20			

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Here 20 gms passed Through 75 μ sieve.

% gravel (> 4.75mm) = 100 - 98 = 2%.

% Course Sand (4.75 - 2.00) = 98 - 65 = 33%.

% medium Sand (2.00 - 0.425 μ) = 65 - 28 = 37 %.

% fine Sand (0.425 - 0.075 mm) = 28 - 4 = 24%

% of silt & clay fraction (< 0.075 mm) = 4 %. (< 12%)

Note: if % of silt & clay fraction is passing Through 75 μ is < 12 %. Then Hydrometer Analysis (or) Wet method Analysis → Not required.

if > 12 Then 2nd Hydrometer.

→ Now plot The Graph.

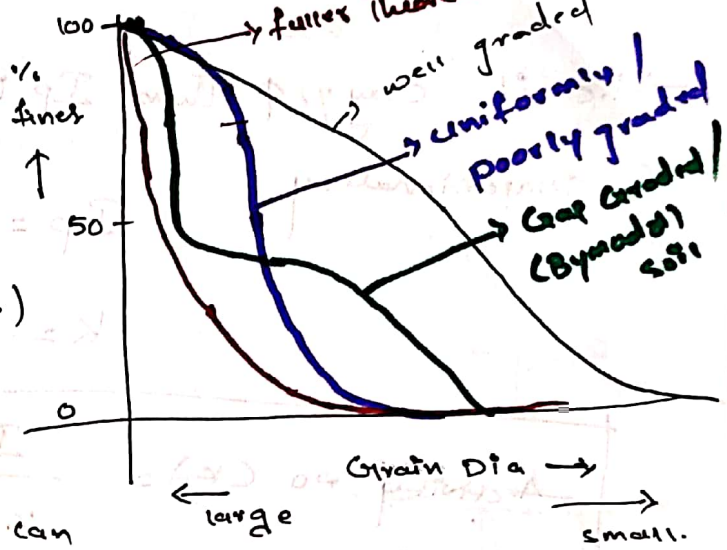
→ Now find out D10 & D10.



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Typical Grain Size Distribution Curve's:

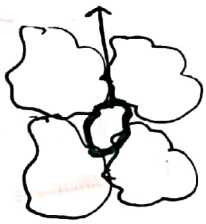
→ if we plot the graph b/w % finer to Grain diameter Then we get different curve's for diff soil's. with diff Compositions of grain's.



→ in uniformly graded soil's soil is having uniform size particles. Ex:- (Desert soils)

→ in Gap graded soil initially up to some extent graph is uniformly graded & Then we can see a change due to absence of certain dia of particles.

Fuller packing

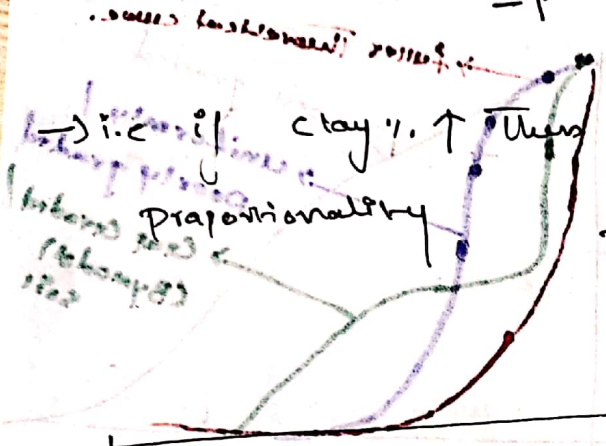


→ if we have larger particles & the space b/w (void) particles are filled with smaller particles of diff size's Then we get fuller Theoretical curve. [known as fuller packing]

Activity No:- it was given by Skempton (1953)

→ He has observed that-

$I_p \propto$ % finer than 2μ (or) clay %.



→ i.e if clay % ↑ then I_p is also ↑. So. if we remove proportionality

$I_p = k \times$ (% finer than 2μ).

k = Activity Number.

$$\text{Activity No } (k) = \frac{I_p}{\text{\% finer than } 2\mu}$$

→ Depends on activity no soil classified as below.

Activity No.	Description of soil
> 0.75	Inactive.
0.75 - 1.25	Normal
> 1.25	Active.

Consistancy Indices:- → (plasticity Index) (I_p)

→ Consistancy Index (I_c)

→ Liquidity Index. (I_L)

→ Toughness Index. (I_T)

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Determination of field/insitue Density:-

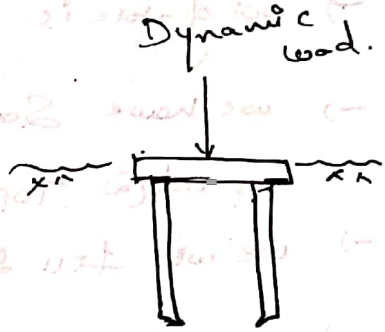
→ we have 3 Method's to Det field Density.

a) Core cutter Method
b) Sand. Replacement Method
c) Water Displacement Method

} in 3 Method's insitue unit weight-

$\gamma_{bulk} = \frac{W_{soil}\ (Total\ wt)}{V_{soil}\ (Total\ vol)}$

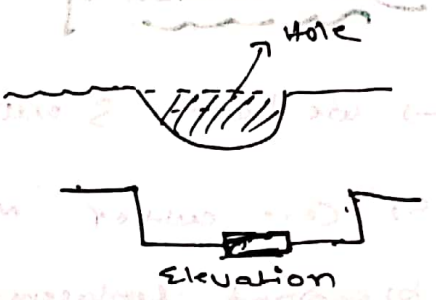
a) Core cutter Method :-



→ volume of Core = (900-1000 cc)
→ @ Top we place Dolly
→ Drive This Setup into soil by Hammer by giving Dynamic load.
→ now remove Sorounding soil to remove Core. & Take weight of soil inside core.
→ Now $\frac{weight}{volume}$ we get γ_b.
→ Not applicable for gravely type of soil.
→ Not applicable for Sandy soil's also.

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Sand Replacement Method



→ it is having a pan with Hole.

→ Kept the pan on G.L & Dig Hole.

→ Excavated Soil weight is calculated & volume of Hole is also calculated

Now $\gamma_b = \frac{\text{Wt of Excavated soil}}{\text{Volume of Hole.}}$



→ Vol of Hole is calculated by Sand Replacement Method.

→ we have Sand pouring cylender opend @ Top.

→ we will fill Soil (Dry soil) & Take wt as W_1



→ Now Take Cylender To Ground & Place it @ Top of Hole & open Shutter.

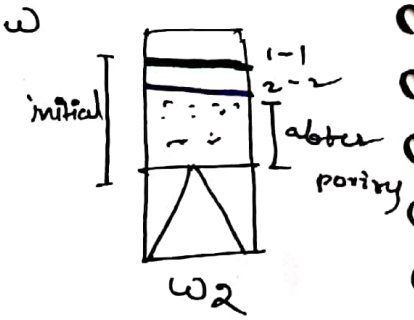


→ Now Sand will filled inside Hole. as well as an additional cone is formed.

→ Angle made by cone with G.L is called as

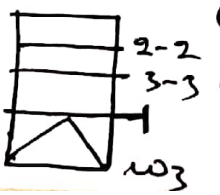
Angle of Repose

→ Now close valve & Sand is reduced Now Take reading as W_2



→ Diff b/w $W_2 - W_1$ gives wt of sand inside Hole + Additional Cone weight

→ Now kept cylender on flat surface with level 2-2 & open Shutter it will form's a Cone & Now Take weight as W_3



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→ Now weight gives the cone weight.

→ Now addition cone weight is Deducted.

→ Now weight of sand inside hole.



→ So volume of Hole = $\frac{W \text{ of sand}}{\rho_{sand}}$

→ ρ_{sand} is calculated by keeping this cylinder on calibrating pan which volume is known.

→ So to get ρ_{sand} = $\frac{\text{wt of sand in calibrating pan}}{\text{volume of calibrating pan.}}$

ρ_{bulk} = $\frac{\text{wt of soil Excavated from Hole}}{\text{wt of whole.}}$

(2) Water Displacement method

→ for clayey soils only.

: Soil - Classification system :

→ We have Different types of soil's based on their structure.

→ Bw- universally Accepted 3 classification system's are introdused by US. Engineer's.

→ The first man in the world Casagrand (1948) has developed a classification system namely Unified Soil classification system [USCS] During world war-II

→ These classification system is particularly made for Highway | Airfield pavement construction.

→ we have Another Soil classification system AASHTO (Americal Association of State Highway & Transport officials)

→ we have also Indian Soil classification system [ISCS]

→ we adopted USCS, with minor modification's in plasticity chart.

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Unified Soil classification System (USCS):-

→ USCS has given Ranges Depending on parameter when we call soil as wellgraded, uniformly graded.

→ The Range b/w uniformly graded & well graded soil's are given and also they given soil classification system based on particle sise's.

→ 75 μ sieve is also known as [Us. sieve no 200]

→ This classification system say's that-

Step 1:-

① if More Than 50% of soil Sample is retained on 75 μ Sieve Then soil said to be Coarse grained soil

Step-2:-

→ Now Take collect all the soil which retained on 75 μ Sieve & Sieve all the soil Sample Then 4.75 mm sieve

→ if the soil retain on 4.75 mm sieve is greater than 50% then soil is said to be Gravelly or else Sandy Soil.

→ after getting CGr soil now we can use G.S.D curve to get well graded & poorly graded soil's.

→ It can be done by two parameter's.

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① Co-efficient of uniformity (C_u)

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

D_{10} = Effective size of particle (we can find out from GSD graph)

② Co-efficient of curvature (C_c)

$$C_c = \frac{D_{30}^2}{D_{10} \times D_{60}}$$

all Diameters of particles are got from GSD graph.

→ if $C_u > 4$ & C_c in b/w 1-3 Then soil is said to be well graded gravel (GW) or if soil fail's This condition Said to be poorly graded gravel (GP)

→ This system is having notation's mentioned below.

prefix		Suffix.	
G →	gravel	W →	well graded
S →	Sand	P -	poorly graded
C →	clay	C -	clayey
M →	silt	m -	silty
O →	organic	L →	$W_L < 50\%$
Pt →	peat.	H →	$W_L > 50\%$

→ if more Than 50% retained on 75 μ Sieve Then we go for Sieve analysis.

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→ if More than 50% of soil Sample is passed Though 75 μ Sieve Then Soil Said to be fine grained soil.

→ Then we can classify the Soil Based on plasticity Index chart. (Sieve analysis not req).

→ if soil is having more than 12% of finer fraction Than 75 μ Then we may call it is Gm (silty gravel) or Gc [clayey gravel].

→ Gc & Gm should be Distinguished by plasticity index chart.

→ Here we have A-line Eqn [Arthur casagrande's line]

$$A\text{ line } I_p = 0.73(w_L - 20)$$

→ if the point is above A-line we can call it as clay.

→ if the point is below A-line we can call it as silt. (or) organic soil.

→ if I_p (plasticity index) is particulary in b/w 4 to 7 we need To give Dual symbol. Ex:- Gc-Gm.

→ Border line cases also req to give dual symbol's.

Her Low plasticity $w_L < 35\%$

Intermediate plasticity $35\% \leq w_L < 50\%$

High plasticity $w_L > 50\%$

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→ If we get more than 50% of soil Retain's on 75 'u' is called Coarse grained soil.

→ Now Sieve it Then 4.75 mm sieve. more than 50% of coarse grained soil retain's on 4.75 mm sieve Then it is gravel if more than 50% of soil passing Then 4.75 mm Then it is Sand.

Sand ← >50% Passing | 50% Retained → gravel. (4.75)

→ In gravel if we consider % of (clay + silt) or % fined grained soil (X)

if (X) is < 5% Then soil is → GW / GP.

if (X) is > 12% Then soil is → GC (Clayey gravel) Clies Above A-Line) / GM (Silty gravel) Clies Below A-Line)

if 5 < X < 12 Then soil may (Dual symbols) → GW-GC / GW-GM / GP-GC / GP-GM.

GP & GC is Taken from Cc & Cu

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Fine grained soil ← 0.75 mm Sands. → >50% passing

% of fine grained soil (x)

if x < 5 %. Then soil → S.W (well graded sand)
→ S.P. (poorly graded sand)

for well graded sand's $C_u > 6$ &

$C_c \approx 1-3$.

Else it is poorly graded Sand.

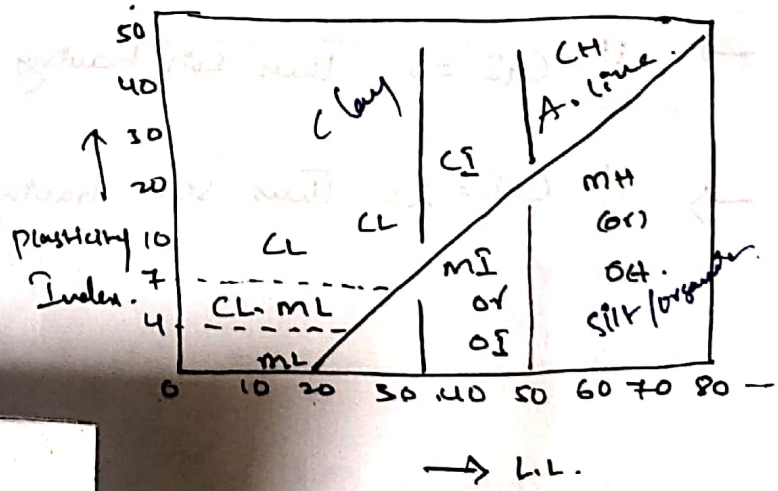
if x > 12 % → SC (clayey sand) (above A-line)
→ SM (silty sand) (Below A-line)

if x is in b/w 5-12 → SW-SC
→ SW-SM
→ SP-SC
→ SP-SM.

Clayey Sand means Clay % is Less Than Sand %.

→ if > 50% of sand passing thru 0.75 mm sieve Then we can use plasticity Index chart.

$I_f \uparrow \propto I_p \uparrow$.



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AASHTO - soil classification system:-

AASHTO = American Association of state highway & Transport officials.

AASHTO classification system grouped the soil into 8 catagiries.

A_1, A_2, A_3, A_4, A_5, A_6, A_7, A_8.

→ above all group's A_8 => is organic soil's. [peat]

→ A_1 – A_7 = group's are depending on Group Index (GI)

→ GI is mesure of shear strength of soil.

$$GI = 0.2a + 0.005ac + 0.01bd$$

limits.

a = % of finer than #75 μ. (x) – 35 [0 to 40]

b = "x" – 15 [0 – 40]

c = w_L – 40%. [0 – 90]

d = I_p – 10% [0 – 20].

→ if we get any "-ve" value then Replace it by number '0'.

→ Group index lies b/w $0 < GI < 20$.

→ if GI = 0 – Then soil having good strength (i.e suitable for construction)

→ if GI = 20 Then soil having poor strength (i.e Not suitable for construction).

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Indian standard Soil classification system (ISSES)

→ ISSCS adopted USCS i.e unified Soil classification system's with minor modification.

→ classification of C.G Soil's are similar to that (USCS) unified Soil classification.

→ To Ensure the Soil is coarse grained we need to Sieve the Soil Thrh 75 μ Sieve.

→ if more than 50% of soil is retaind on 75 μ Sieve then Soil is said to be coarse-grained

→ other wise soil is fine grained soil.

→ coarse-grained soil:- after confirming Soil is C.G.

→ Now for further classification Soil should Sieve Thrh 4.75mm Sieve

→ if now more than 50% of soil is Retaind on 4.75mm Sieve Then Soil is said to be Gravely soil

→ if more than 50% of soil is passing Thrh 4.75mm Sive Then Soil said to be Sandy Soil

→ Now for further classification we need to do the Mechanical Sieve analysis. & plot GSD curve.

→ Based on ~~on~~ GSD curve the soil if further classified.

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D_{10} → Effective size.

D_{30} = 30% of soil is finer than this size.

D_{60} = 60% of soil is finer than this size.

after getting D_{10}, D_{30}, D_{60} we will get C_c, C_u. i.e

i) co-efficient of uniformity: (C_u):

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

ii) co-efficient of curvature, (C_c)

$$C_c = \frac{D_{30}^2}{D_{60} \cdot D_{10}}$$

for Sand:

$C_u > 6$

$C_c \simeq 1$ to 3

} if these two conditions satisfy soil is said to be well graded Sand (SW) Else Soil is said to be poorly graded Sand (SP).

for Gravel:

$C_u > 4$

$C_c = 1$ to 3

} if these conditions satisfy soil is said to be well graded gravel (GW) or else soil is poorly graded gravel (GP).

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If The soil is fine-grained soil:- further classification is done by plasticity chart. (with minor modification).

Modifications are:- CI, MI & OI.

i.e clay of Intermediate Compressibility.

CL → clay of Low Compressibility.

ML → Silt of " "

OL → organic soil of " "

CH → clay of High compressibility

MH → Silt of " "

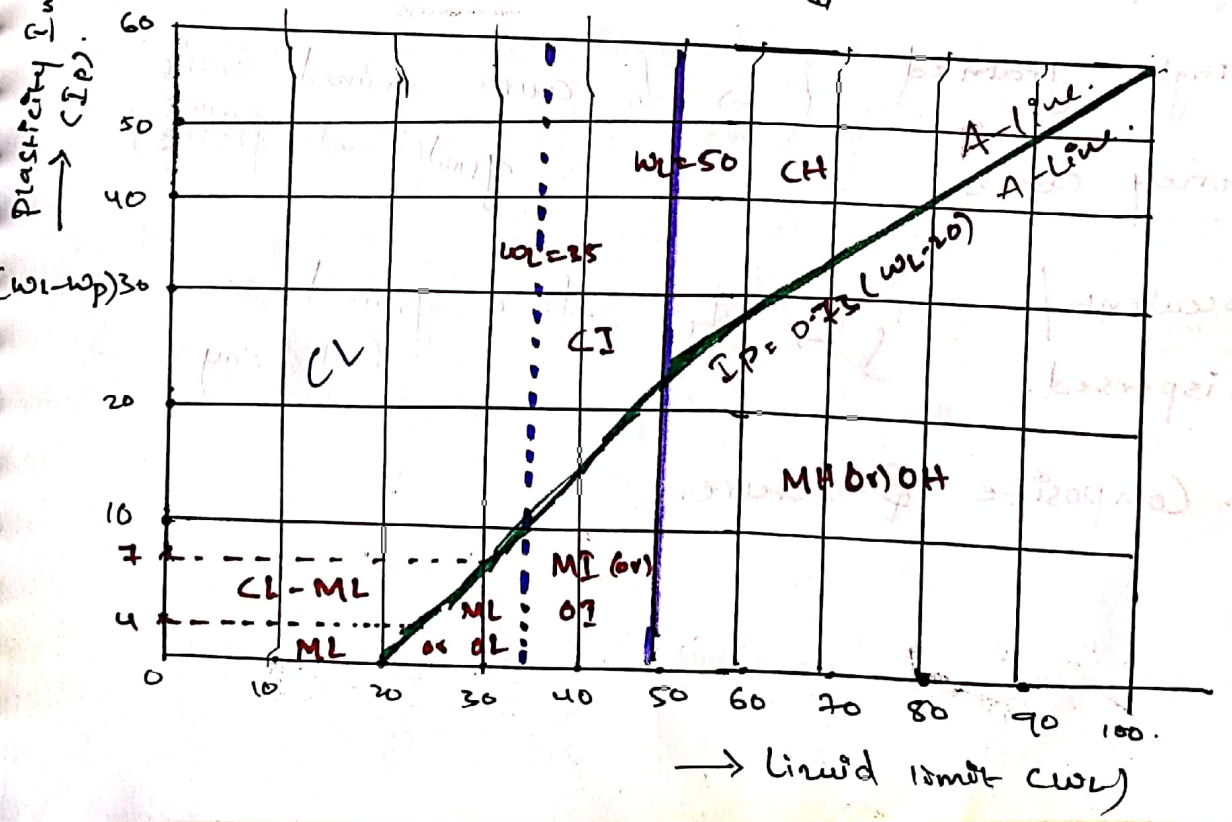
OH → organic soil of " "

CI → Clay of intermediate Compressibility.

MI → Silt of " "

OI → organic soil of " "

(plasticity chart As per Indian standard classification).



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Specific Surface Area :- (SSA).

Def :- it is defined as Total Surface of the individual grain's per drymass of the grain's.

$$SSA = \text{Surface Area} / (\text{volume} \times \rho).$$

SSA is inversely proportional to the particle size.

i.e as the particle size ↑ then SSA ↓.

% of water adsorbed on to mineral can be given as

$$w.c = (SSA) \times \text{thickness of layer of water} \times (\rho w).$$

Soil - structure (or) Soil fabric :-

Def :- Arrangement of the grain's in relation to each other

As per Terzaghi & Casagrande :-

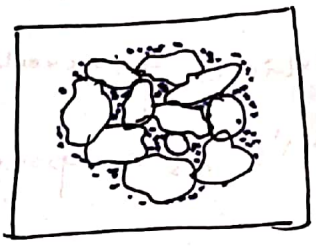
The type of Soil Structure's are

1. Single grained } → for course grained soil's (course gravel (sands & silts))
2. Honey comb.
3. flocculent / Dispersed. } → for fine grained soil's. (silt & clay)
4. Composite structure.

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Single grained structure:-

→ Each grain Each grain touches several of it's neighbour's in such a way that the Aggrigate is stable. Even though there is no force of attraction b/w grain's.



particle size > 0.02 mm (for sands & silts).

→ The arrangement may be dense or loose, and the properties are greatly influenced by denseness or looseness.

★★

Loose state:- High void ratio & Low unit wt

Dense state:- Low void ratio & high unit wt.

→ For granular soil's (sand's & gravel). particles are assumed to be spheres shape of equal size.

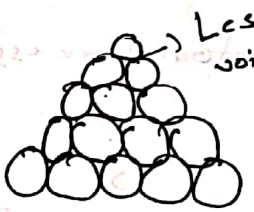
→ Loosest & Densest arrangements we can obtain from these equal spheres of simple cubic & pyramidal type of packing.



Simple cubic stru-

→ Loosest state ⇒ Simple cubic

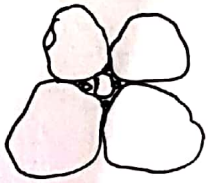
Pyramidal packing ⇒ Densest state.



pyramidal packing.

→ Ex:- ~~Pyram~~ Billard Balls are arranged in Both way's as shown in fig.

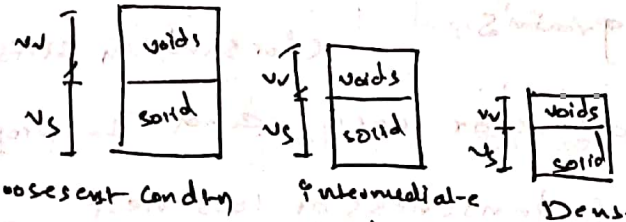
→ But in the field soil particles may not be perfect sphear's. There may chance of small particle's occupy void spaces b/w large one's & ↓ void ratio.



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-> on other hand The irregularity in the shape of Particles generally tends to ↑ void ratio of Soil's as compared to ideal spheres.





Loosest condition
void ratio = e_{max}

Intermediate condition
Void ratio = e

Densest condition
Void ratio = e_{min}.

e_{max} = Max. void ratio in loosest State.

e_{min} = minimum " " Densest- "

e = In situ / Intermediate void ratio.

** Max void ratio simple cubic structure is 0.91

** Min void ratio for pyramidal structure is 0.35

Relative Density (D_r) or Density Index (I_D)

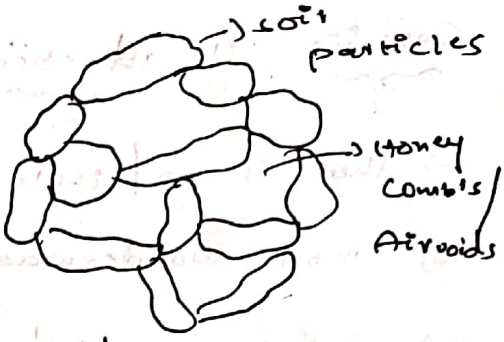
-> It is used to describe the degree of compaction (packing or togetherness) of coarse grained soils.

$$D_r = \frac{(e_{max} - e)}{(e_{max} - e_{min})}$$

D_r (%)	classification:
< 15	very loose
15-35	Loose
35-65	medium Dense
65-85	Dense
> 85	Very dense.

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Honeycomb Structure:-



grain's of silt's < 0.02mm & > than 0.002mm.

→ Honeycomb Structure will collapse on wetting so that- volume of soil ↓.

→ it is sensitive to vibration's / shocks which lead's to collapse Structure. & volume decreases.

→ grain's coming in contact- are held by forming miniature arche's. bridging over relatively large void spaces & create a structure called Honeycomb Structure with large void's.

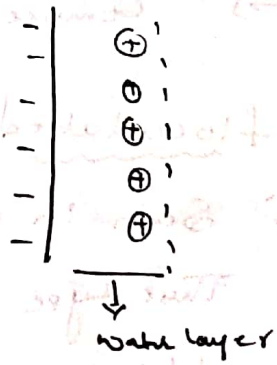
→ if Structure collap's ~~Soil~~ it deduced to again a single grained particle arrangement's.

Structure of fine grained Soil's (clayey's):-

→ consider a soil particles which are suspend & seperated by a water layer. the forces acting on them are.

i) Vander wall's forces of attraction

ii) net repulsion b/w two tvely charged ionised adsorbed water layers.



Case i: if the asorbed layer is Thick

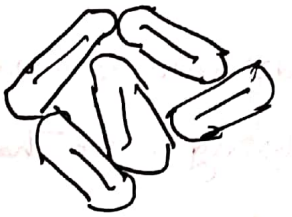
→ net repulsion forces ↑

→ vanderwal forces ↓.

→ So particle's will Remain Dispersed)
(i.e Settle's Endipendently)

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Case (ii) if absorbed layer is Thin:



→ There is no / little repulsive forces

→ More wanderwaal force of attraction

→ So momment of particles will be enough to bring them into contact. This process is called flocculation.

→ group of particles settle together is called flocculation.

Dispersed Structure of clay's:



→ It occurs in remoulded clay's.

→ It clay's orginal @ one place & Transported To some where by any natural transport

→ due to transportation Edge to face orientation changed to face to face orientation.

→ So net force become repulsive

→ Low Shear Strength having } ☆☆

→ Low permeability

→ Compressibility ↑ (fine clay & colloid's)

flocculated Structure:- [Ex: marine clay]



<0.002 mm

→ Seen in clay particles.

→ This type of structure formed due to net attractive forces b/w particle

→ Orientation - Edge to face.

→ High - Shear Strength

→ Low - Compressibility, High - permeability. } ☆☆

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Composite Structure:-

→ if our soil is having combination of Diff. soil's then there is presence of different structure's in single sample. such type structure is known as composite structure.



Remoulding:-

→ Loss of strength of cohesive soils. Loss of original structure due to transportation / loading condition's.

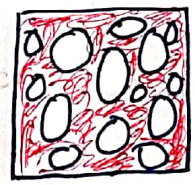
Thixotropy ☆☆☆☆

The gain of strength in the soil with passage of time after remoulding is called Thixotropy

→ This is due to gradual re-orientation of molecules of water in adsorbed water layer & re-establishment of chemical equilibrium.

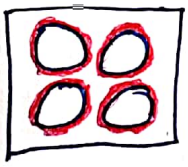
→ The recovery of strength is not 100%.

Absorbed water



→ water held in voids of soil
→ which is not transfered by gravity
→ occures throughout the body of soil
→ Rate is same throughout body

Adsorbed water:-



→ water held on surface of body
→ it is due to adhesion
→ it is a surface characteristic.
→ Initial rate is high & then decreases
→ Also called hygroscopic water

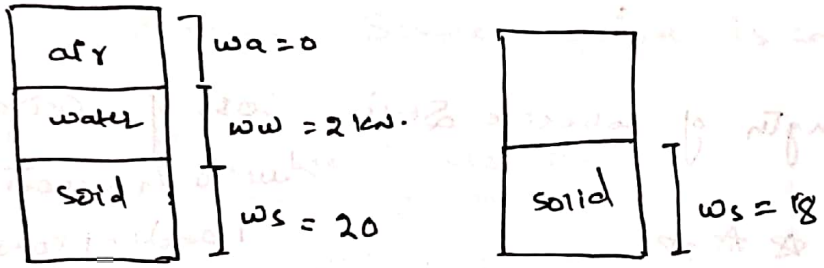
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: Problems :- (GATE- Level)

① $1 m^3$ of wet soil weighs 20 kN, it's dry wt is 18 kN, Sp. gravity is 2.67 kN, Determine, w.c, porosity, void ratio degree of saturation & Also draw phase diagram.

Initially soil is wet condition

after Drying W_s is given 18 so Dried water weight is = 2kN



Initial

w.k.t $w.c \Rightarrow w = \frac{w_w}{w_s} \Rightarrow \frac{2}{18} = \frac{1}{19} = \boxed{11.11\% = w}$

w.k.t $\gamma_b = \frac{W}{V} \Rightarrow \frac{\text{Total wt}}{\text{Total vol}} = \frac{20}{1} = 20.$

& we have $\gamma_b = \frac{G_s (1+w) \gamma_w}{1+e} \Rightarrow 1 = \frac{2.67 (1+11.11) 9.81}{1+e}.$

$\Rightarrow \boxed{e = 0.455}$

we have $1+e = \frac{1}{1-n} \Rightarrow 1+0.455 = \frac{1}{1-n} \Rightarrow \boxed{n = 0.313}$

we have $eS_r = wG_s \Rightarrow S_r = w.G_s/e \Rightarrow \frac{11.11 \text{ (0.1111)} \times 2.67}{0.455} \Rightarrow \boxed{S_r = 0.645}$

$\boxed{S_r = 64.55\%}$

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(2). w.c of saturated soil & sp. gravity of soil solid's were found to be 30% & 2.7 resp. Taking $\rho_w = 10\ kN/m^3$, find (ρ_{sat}) saturated unit wt & void ratio of soil.

Δw

=> given $w.c = 30\% = 0.300$ & $G_s = 2.7$; $\rho_w = 10\ kN/m^3$

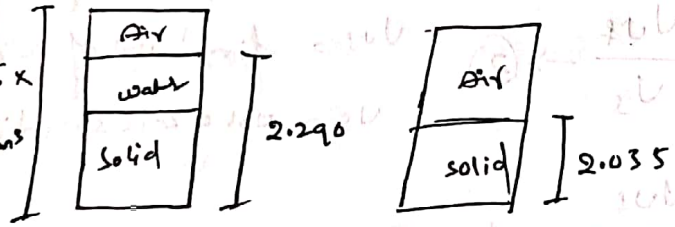
$\rho_{sat} = ?$ & $e = ?$

=> we have $eS_r = G_s w$, for saturated soil $S_r = 1$ so

$e = G_s . w \Rightarrow e = 2.7 \times 0.3 \Rightarrow \boxed{e = 0.81}$

& $\rho_{sat} = \left(\frac{G_s + e}{1+e}\right)\rho_w \Rightarrow \rho_{sat} = \left(\frac{2.7+0.81}{1+0.81}\right)10 \Rightarrow \boxed{\rho_{sat} = 19.392\ kN/m^3}$

(3) a soil sample in it's natural state has a mass of 2.290 kg & vol of $1.15 \times 10^{-3}\ m^3$. after oven dried the mass of sample is 2.035 kg. $G_s = 2.68$, find porosity, n=?



so wt of water => 2.290 - 2.035

=> 0.125

= 12.5%.

w.k.t $\rho_b = \frac{G(1+w)}{(1+e)}\rho_w$, $\rho_b = \frac{w}{V} = \frac{2.290}{1.15\times10^{-3}} = 1.9913 \times 10^{-3} = \rho_b$.

$\rho_{bulk} = 1991.3$, $\rho_b = \frac{\rho_{bulk}}{\rho_w} \Rightarrow \frac{1991.3}{1000} = 1.991$

$1.991 = \frac{2.68(1+0.125)}{(1+e)} \Rightarrow \boxed{e = 0.5141}$

w.k.t $1+e = \frac{1}{1-n}$ $\quad 1+0.5141 = \frac{1}{1-n}$

$\boxed{n = 0.34}$

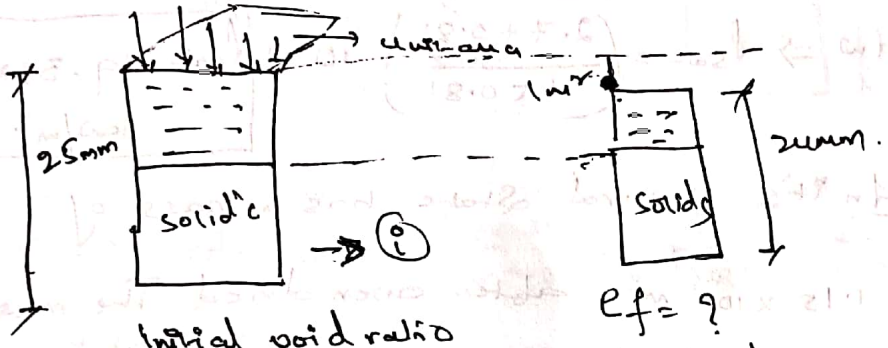
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(4) Clay sample originally 25 mm thick @ $e = 1.12$ was subjected to compressive load. After the clay sample was completely consolidated, it's thickness was measured to be 24 mm. Compute final void ratio.

Ans:- Here clay sample is completely consolidated.

i.e soil is fully saturated, for fully saturated soil

w.k.t $S_r = 100\% = 1$.



Initial void ratio
$e_i = 1.12$

$e_f = ?$
final void ratio.

Here soil volume is incompressible i.e V_s does not change

V_{vi} = initial vol of void's
V_{vf} = final vol of void's.
V_s = vol of soil solid's.

$e_i = \frac{V_{vi}}{V_s} \rightarrow (1)$ Eq

$e_f = \frac{V_{vf}}{V_s} \rightarrow (2)$

$1.12 = \frac{V_{vi}}{V_s}$

$V_s = \frac{V_{vi}}{1.12}$ from (1)

$V_s = \frac{V_{vf}}{e_f}$ from (1)

V_f = final volume.
V_i = Total vol @ initial stage

Now Total volume from dia (i) $V_i = V_{vi} + V_s$

$V_i = V_s \times 1.12 + V_s \Rightarrow V_i = 2.12 V_s \rightarrow (3)$

$V_f = e_f V_s + V_s = V_s[1 + e_f] \rightarrow (4)$

from (3) $V_i = 2.12 V_s \Rightarrow 25 = 2.12 V_s \Rightarrow V_s = 11.793$ mm.

fm (4) $V_f = [1 + e_f) V_s \Rightarrow 24 = (1 + e_f) 11.793 \Rightarrow$ **$e_f = 1.035$**

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Method -2

$1+e_i = \frac{V_{v_i}}{V_s} + 1$

$1+e = \frac{V}{V_s}$ → const.

$\boxed{1+e \propto V}$ ★★

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

$\frac{25}{24} = \frac{2.12}{e_f + 1}$

$\boxed{e_f = 1.03}$

Relation b/w V_1 (1+e) with γ_d.

$\gamma_d = \left[\frac{G}{1+e}\right]\gamma_w$.

⇒ provided no soil is added which is of dist sp. Gravity i.e G = Const

$\gamma_d \propto \frac{1}{1+e}$ (or) $S_d \propto \frac{1}{1+e}$.

So $\boxed{\frac{V_1}{V_2} = \frac{1+e_1}{1+e_2} = \frac{\gamma_{d_2}}{\gamma_{d_1}}}$ ★★

(1), (2).

(5). an embakment is made by using soil from barrowpit. Both properties are given below.

Barrowpit

$w = 15\%$.

$e = 0.6$

$V_1 = ?$ ($\gamma_{d_2} = ?$)

$G = 2.7$

Common for both

Embakment

$V_2 = 4000\ m^3$

$w = 18\%$.

$\gamma_{d_2} = 1.76\ ton/m^3$.

findout-

(a) Vol. Excavated from barrowpit

(b) quantity of water Rqd for Embakment construction in tons.

We have relation $\frac{V_1}{V_2} = \frac{1+e_1}{1+e_2} = \frac{\gamma_{d_2}}{\gamma_{d_1}}$

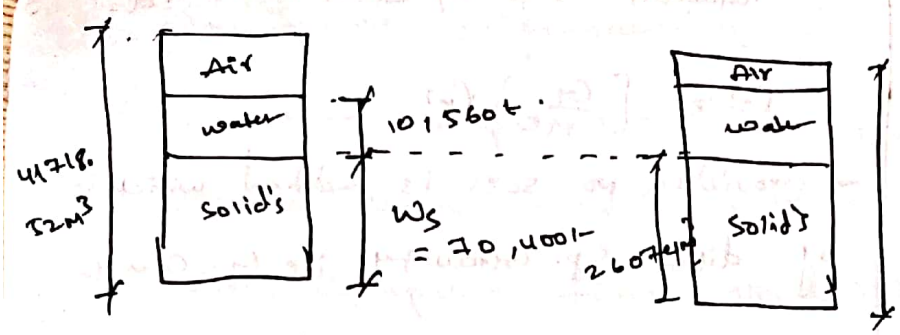
$\frac{V_1}{V_2} = \frac{\gamma_{d_2}}{\gamma_{d_1}}$ ⇒ $\gamma_{d_1} = \frac{G_s(\gamma_w)}{1+e_1}$ → (1) bcz $\gamma_w = 1000\ kg/m^3$

$\gamma_{d_1} = \frac{2.7 \times 1}{1+0.6}$

$\gamma_{d_1} = 1.6875\ t/m^3$

$\frac{V_1}{4000} = \frac{1.76}{1.68}$ ⇒ $\boxed{V_1 = 4187.52}$

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Barrow pit

Embankment.

w.k.t $\rho_d = 1.6875$ t/m³

$\rho_d = \frac{W_s}{V} \Rightarrow W_s = \rho_d \times V$

$= 1.6875 \times 41718.52$

$= 70,400$ t.

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

$W_w = w \times W_s \Rightarrow 0.15 \times W_s$

$\Rightarrow 10,560$ t.

$\rho_w = 1$ t/m³.

Now w.k.t $G_s = \frac{W_s}{V_s}$.

$V_s = \frac{70,400}{2.7} = 26074$ m³

$V_{air} = 41718.52 - 26074 - 10056 \Rightarrow 5084.45$ m³

$w = 0.18$

$W_w = 0.18 \times 70400$

$= 12,672$ ton.

Quantity of water to be added

$W_w = (12672 - 10560)$ t

$W_w = 2112$ ton.

$\frac{W_w}{\rho_w}$ = vol. of water to be added

$V_w = 2112$ m³

(6) A soil is compacted to prepare Subgrade per an Airport pavement (specifications are given), such that the soil has brought from (2) Barrow pits. (specifications given) find out which location gives Less Transportation cost

Airport pavement

$e_1 = 0.75$

$V_1 = 600000$ m³

Barrow pit e Transportation cost

A ⟶ 0.8 ⟶ Rs 10/ m³

B ⟶ 1.7 ⟶ Rs 5/ m³.

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Soln:-

we have $\frac{V_1}{V_2} = \frac{1+e_1}{1+e_2} = \frac{\gamma_{d_1}}{\gamma_{d_2}}$

for Borrow pit (A)

$\frac{V_1}{V_2} = \frac{1+e_1}{1+e_2} \Rightarrow \frac{600000}{V_A} = \frac{0.75+1}{1+0.8} \Rightarrow V_A = 6,17,142.86\ m^3$

So Total cost @ A = $V_A \times 10 \Rightarrow 6,17,142.8$

for Borrow pit B

$\frac{V_1}{V_2} = \frac{1+e_1}{1+e_2} = \frac{600000}{V_B} = \frac{0.75+1}{1+1.7} \Rightarrow V_B = 9,25,714.3\ m^3$

Total cost @ B = $9,25,714 \times 5 \Rightarrow 46,28,571.43$

$\Rightarrow$ So Borrowpit B is Economical.

problem A soil sample weighing 1605 kN/m³, has a w.c of 28%, $G_s = 2.7$; Det, γ_d, e, η, S_r = ?

w.k.t $\gamma_b = \gamma_d [w+1]$

$\gamma_d = \frac{\gamma_b}{w+1}$ we get γ_d = 12.89

$\gamma_d = \frac{w \times G_s}{1+e}$ we get e; e = 1.055

$e.S_r = w.G_s$ we get S_r = 71.7%

$1+e = \frac{1}{1-\eta}$ we get η = 51%

(2) A sample of soil at a vol of 1000 cc & wt of 17.5 N, $G_s = 2.52$, if the dry u.w of soil is 15.8 kN/m³. Det w, e, γ_{sub}, & S_r.

$\gamma_b = \frac{W}{V} = \frac{17.5 \times 10^{-3}\ kN}{1000 \times 10^{-6}\ m^3} =$

$\gamma_d = \frac{\gamma_b}{w+1}$ & γ_d = 15.8

we get w = 10.8%

$\gamma_d = \frac{G_s \gamma_w}{1+e}$ we get e = 0.596

$\gamma_{sat} = \left(\frac{G+e}{1+e}\right)\gamma_w$ γ_{sat} = [illegible] kN/m³

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

9.532 kN/m³

(3) Soil sample has a vol of $1 \times 10^{-6}\ m^3$ & weighs 19N. The w.c of soil is 15%, $G_s = 2.7$, find γ_d, e, S_r, & w.c @ saturation

Take $\gamma_w = 9.81\ kN/m^3$

$e.S_r = w.G_s$

$S_r = 48.16$

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A sample of clayey soil of vol $1 \times 10^{-3} m^3$ of weighing 0.01762 kN after being dried in oven has a weight of 0.01368 kN. if $G_s = 2.69$, find the following from it's original condition

i) w.c, ii) void ratio iii) Air void's, iv) Saturated u.w

v) dry unit wt

① dry $V = 1 \times 10^{-3}$

$W_{wet} = 0.01762$ kN.

$W_{dry} = 0.01368$ kN. $\Rightarrow W_s$.

$G_s = 2.69$

$w = ?$ $e = ?$, ~~$n = ?$~~ $n_a = ?$, $\gamma_{sat}, \gamma_d = ?$

$W_w = W_{wet} - W_{dry}$

$= 0.01762 - 0.01368$

$= 0.00394$ kN.

Water content $(w) = \frac{W_w}{W_s} \times 100 = \frac{0.00394}{0.01368}$.

$= 28.8\%$.

we have $G_s = \frac{\gamma_s}{\gamma_w}$ $\gamma_s = \frac{W_s}{V_s}$

$G_s = \frac{W_s}{V_s \gamma_w}$

$V_s = \frac{W_s}{G_s \times \gamma_w}$

$V_s = \frac{0.01368}{2.69 \times 9.81} \Rightarrow 5.183 \times 10^{-4}$

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Also $\gamma_w = \frac{w_w}{V_w}$.

$V_w = \frac{w_w}{\gamma_w} \Rightarrow \frac{0.00394}{9.81} = 4.016 \times 10^{-4} m^3$

Also $V_v = V_a - V_s$

$\Rightarrow 1 \times 10^{-3} - 5.183 \times 10^{-4}$

$= 4.817 \times 10^{-4} m^3$.

$V = V_v + V_s$

$V_s = V - V_v$

and $V_a = V_{v_a} - V_w$.

$\Rightarrow 4.817 \times 10^{-4} - 4.0 \times 10^{-4}$

$= 8.01 \times 10^{-5}$

② Void ratio $(e) = \frac{V_v}{V_s} \Rightarrow \frac{4.817 \times 10^{-4}}{5.183 \times 10^{-4}}$

$= 0.929$

% air void's

porosity $(\eta) = \frac{e}{1+e} \times 100$.

$\frac{0.929}{1+0.929} \times 100$.

$= 48.16$.

% air void's $= (n_a) = \frac{V_a}{V} \times 100$

$V_{sat} = \left(\frac{G+e}{1+e}\right) \gamma_w$.

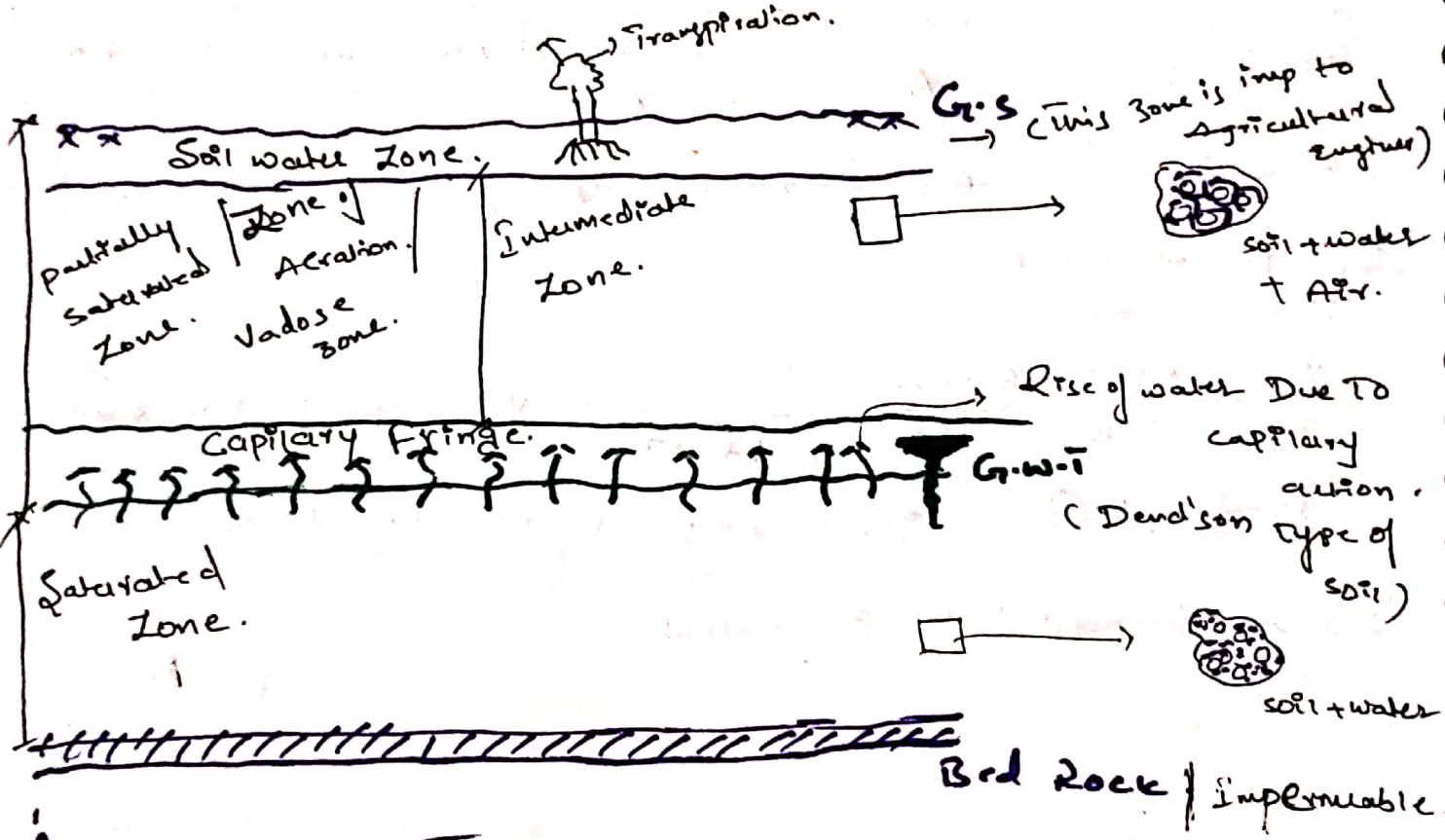
$= ?$

$\gamma_d = ?$

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Permeability, Effective Stress & Seepage Through Soils:-

Distribution of water below G.S / Soil water:-



Flow of water Through Soils:-

Def:- The path Taken by water along the pore space's within the soil mass.

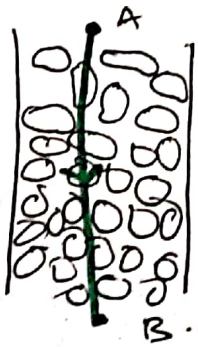


Note:- To occure the flow of water there must be Head diff. & water will flow from Higher (Head) Energy level to lower Energy level

→ if There is no-Head Dift Then There is no flow Takes place. This condition is known as "Hydrostatic condition"

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→ Here water is flowing from A-B. Because of Head difference. and water does not take Straight line path. But it may passes Through along Contact points or pore spaces of soil Medium This path is known as winding path or tortuos path





→ But in large scale we can Take it is line & This path is known as flow-path.

→ According to Bernoulli Eqn, Total head @ a point in water under motion can be given as Sum of pressure, velocity & Elevation head's. i.e

$$h = \frac{P_w}{\gamma_w} + \frac{v^2}{2g} + Z.$$

$\frac{P_w}{\gamma_w}$ = pressure head

v = velocity

Z = Elevation.

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÷ Permeability :-

→ To occur permeability there should be presence of void's.

Ex: if we take soil & a Rock Sample then permeability will occurs in soil's Bcz of presence of void's.

Def:- it is the property of soil which allows the flow of water through it's interconnected void's.

(Vol of single void) gravel > (vol of single void) clay

(Total no. of void's) clay > (Total no. of void's) gravel.

(Total vol of void's) clay > (Total vol of void's) gravel.

→ Even though Total vol. of void's in clay greater than gravel then is less permeability in clay's (fine grained soil's)

→ Bcz of interconnectivity between void spaces.

→ due to this interconnectivity a tube like structure is formed in soil's so permeability occurs

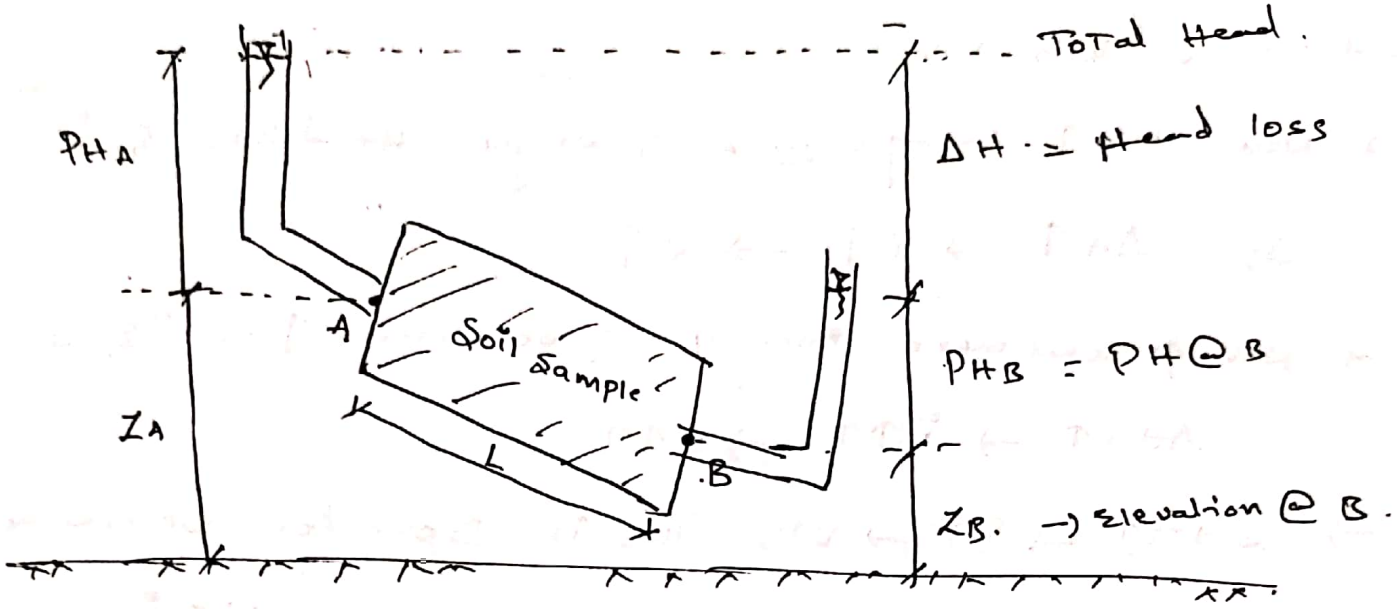
→ where as in clay we have dispersed structure.

So. In coarse grained soil's → permeability ↑ due to (void's are interconnected)

in fine grained soil's → permeability ↓ [no-interconnectivity].

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Darcy's Law of Permeability:-



according to Bernoulli Total Head (TH) = pressure head + velocity Head + Elevation Head

$$TH = PH + \cancel{VH}^{0} + EH$$

→ But Bernoulli is given for pipe flow. in pipe the velocity is much higher when comparing with soil [or negligible].

→ So we may Assume velocity Head $VH = 0$. So.

$$TH = PH + EH.$$

So $TH_{@A} = PH_A + EH_A$ & $TH_B = PH_B + EH_B$.

→ Due to flow of water we may have some Head loss ΔH. i.e

$\Delta H = PH_A - PH_B$. & if L is the length covered by water in soil sample so we have

Hydraulic graidient $i = \frac{\Delta H}{L}$

Here @ A Energy is high so.

$TH_A > TH_B$ & water will flow from A to B

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→ let us assume a case where There is no Diff in head b/w A & B. i.e There is no change in Head so $\Delta H = 0, i = 0$, Then $V = 0$.

→ Now we ↑ Energy @ A & we get Head loss & i.

So $\Delta H \uparrow \rightarrow i \uparrow \rightarrow V \uparrow$

→ Now ↑ some more Energy & we observe ↑ in i & V.

$\Delta H \uparrow\uparrow \rightarrow i \uparrow\uparrow \rightarrow V \uparrow\uparrow$

→ so $\Delta H \uparrow \rightarrow i \uparrow \rightarrow V \uparrow$. This is Done for Several no. of soil Sample's & plot → graph b/w V & i.

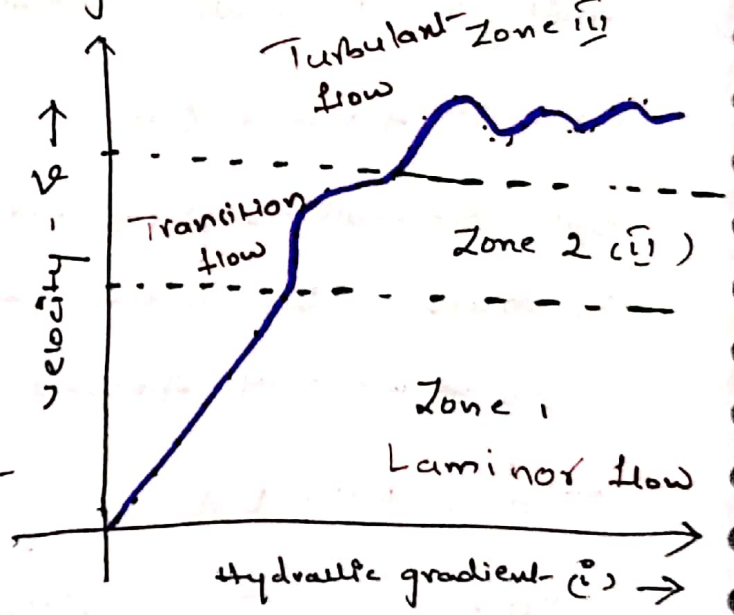
→ Here velocity is function of hydraulic gradient- i.e

$y = f(x)$

$\hookrightarrow i$.

→ so we can't change Axis

→ In Zone (1) we have a linear relationship so we get St. line



→ Above all the Study was done by Darcy.

→ As per his observation's for Zone-1 he has given a law called "Darcy's law of permeability".

→ According to Darcy's in Zone I

$V \propto i$

$V = k \times i$

q = no-unit

So k units = V units

k = co-efficient of permeability [cm/s) (m/s)

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→ This Theory is valid only for Laminar flow.

→ i.e it valid only for "coarse Silt & fine Sand's" only

~~Be~~ not valid for gravel's & clay's.

Bcoz in gravel Permeability is more so we get Non-linear relationship & in clay's also permeability is ↓ (less) so here also we get Non-linear relation

→ here V = discharge velocity. (m/s)

$$V = \frac{Q}{A}$$

, Q = Quantity of Discharge

A = Area of crossection.

→ Here we are using Total crossectional Area. But the water is flowing only Through void spaces.

→ So it is valid only for discharge velocity.

→ if we consider the velocity which passing Through void's is very high Bcz void's are small. so the velocity through the void's is given by Seepage velocity.

→ ~~Be fore~~ To Det- Seepage velocity we have Continuity Eqn

$A_1V_1 = A_2V_2$

$AV = A_vV_s$

$V_s = \frac{A}{A_v} V \frac{L}{L}$

$V_s = \frac{v}{v_v} \times V$

$$\boxed{V_s = \frac{V}{\eta}}$$

V_s = Seepage velocity.

A V = Total Crossectional Area & Discharge velocity

A_v = Area of void's } for single void.

V_s = Seepage velocity }

A x L = Total vol V

$A_v \times L$ = vol of void's. (V_v)

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

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∴ W.K.T $V_s > V$. But Darcy's law is valid only for Discharge velocity.

$V_s \propto i$

$\boxed{V_s = k_0 i}$ k_0 = co-efficient of percolation.

factors Affecting The permeability :-

(a) $K = C d_{10}^2 \left(\frac{e^3}{1+e}\right)\left(\frac{\gamma_w}{\mu}\right)\left(\frac{1}{k_s \cdot SSA^2}\right)\left(\frac{1}{T^2}\right) \times A \times \frac{1}{B} \times \frac{1}{C} \times \frac{1}{D}$.

i) Size of particle :- from

Allen Hazen's Eqn :- $k = C d_{10}^2$ i.e $k \propto d$.

Here C = const & d = diameter of particle.

i.e $d_{10} \uparrow = k \uparrow$. , ($C \rightarrow 1$ to 1.5, D_{10} = in mm)

ii) Specific Surface Area :-

from Loudon's Eqn $\log_{10}(k \cdot SSA^2) = a + bn$.

SSA = Sp. Surface Area, & SSA $\uparrow \Rightarrow k \downarrow$.

In fine grained soils SSA $\uparrow \rightarrow k \downarrow$

for course grained soil SSA $\downarrow \rightarrow k \uparrow$.



(b) we have Kozeny Carman's Eqn

$k = \frac{C^3}{1+e}\left(\frac{\gamma_w}{\mu}\right)\left(\frac{1}{k_s \cdot SSA^2}\right) \cdot \frac{1}{T^2}$

(c)

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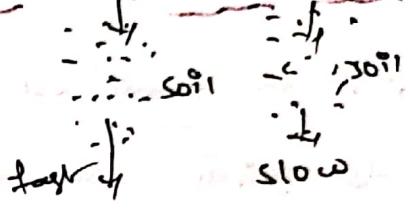
iii) void ratio :- (e) :- from above eqn

$k \propto e$ So $k \uparrow \rightarrow e \uparrow$

iv) Dynamic viscocity :- (μ) again from above Eqn

$k \propto \frac{1}{\mu}$ So $\mu \uparrow \Rightarrow k \downarrow$

SS $\uparrow$ — $k \downarrow$.



v) Tortuosity :- (T) (Zig Zag way of water)

$k \propto \frac{1}{T}$ So T $\uparrow$ Then $k \downarrow$.

vi) Degree of saturation (A)

$k \propto A$, A $\uparrow$ So $k \uparrow$

fully saturated → more D.O.S → more slow → $k \uparrow$

partially saturated → D.O.S less → $k \downarrow$.

vii) Entrapped gasses (B)

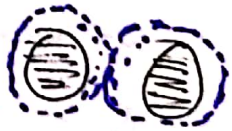
$k \propto \frac{1}{B}$ So B $\uparrow$ $k \downarrow$

viii) Impurities (C) :-

$k \propto \frac{1}{C} \Rightarrow$ C $\uparrow$ $k \downarrow$.

ix) adsorbed water (D)

Absorbed water is holding Bw surface water of soil. Here water held @ surface of soil & close the space of voids So $k \downarrow$



i.e $k \propto \frac{1}{D}$ So D $\uparrow$ $k \downarrow$.

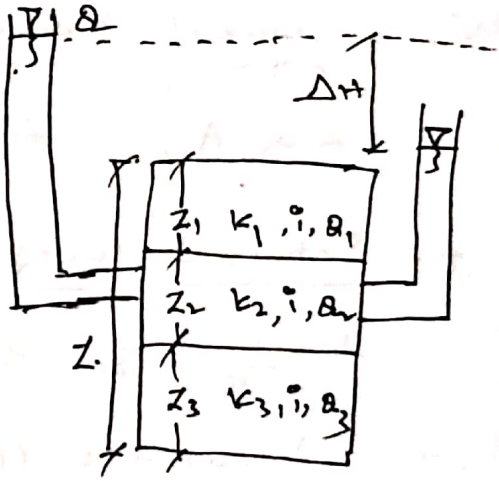
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Hydraulic conductivity / co-efficient of permeability of stratified Layer's:-

case (I) :- when flow occure's parallel to no. of layer's:-

→ As we know that soil properties change's from point to point. so as depth goes on increases we may get diff type of soil layer's.

→ let - depth's of strata's z_1, z_2 & z_3 & co-eff of permeability k_1, k_2 & k_3. & water flowing parallel in strata. i.e Left-To Right. [i due to head loss]



→ for all layer's headloss is ΔH

→ & Length of soil sample 'L' is same.

→ so we may say that hydraulic gradient 'i' is same for all soils.

→ B.C.Z ΔH & L is same.

→ we are giveng a Discharge Q so that it will divide in to diff soil types as Q_1, Q_2 & Q_3

So we may say that-

$$Q = Q_1 + Q_2 + Q_3.$$

$$AV = A_1v_1 + A_2v_2 + A_3v_3$$

$$(Z \cdot 1)(k_{eq} \cdot i) = (Z_1 \cdot 1)(k_1 i) + (Z_2 \cdot 1)(k_2 i) + (Z_3 \cdot 1)(k_3 i)$$

$$(k_{eq})i = \frac{i(Z_1 k_1 + Z_2 k_2 + Z_3 k_3)}{Z}$$

w.k.t $A = B \times D$.

$D = Z$ &

$B = 1$ i.e unit width

from Darcy's $V = ki$

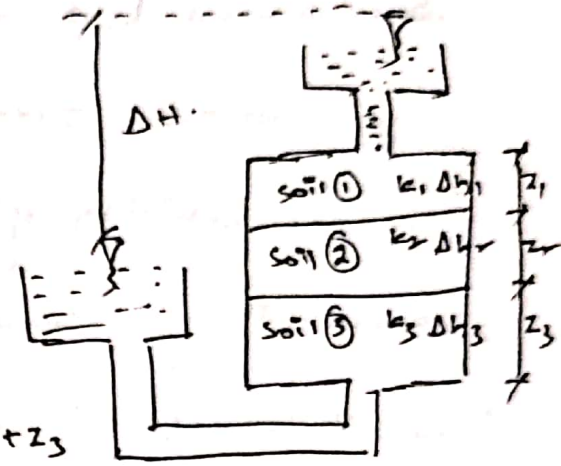
k_{eq} = equivalent

w.k.t $Z = Z_1 + Z_2 +$

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So $$k_{eq} = \frac{k_1 z_1 + k_2 z_2 + k_3 z_3}{z_1 + z_2 + z_3}$$

Case(ii) When water flows normal To soil layers:-



w.k.t $\Delta h = \Delta h_1 + \Delta h_2 + \Delta h_3$.

there i is not same for all soils bcz ΔH & L is not same for all soils.

w.k.t $i = \frac{\Delta h}{L} \Rightarrow \Delta h = iL$.

So here $L = z$.

$iz = i_1 z_1 + i_2 z_2 + i_3 z_3$ $\quad z = z_1 + z_2 + z_3$

$\frac{v}{k_{eq}} \cdot z = \frac{v}{k_1} z_1 + \frac{v}{k_2} z_2 + \frac{v}{k_3} z_3$ $\quad$ w.k.t $v = k_1 i$

$q = \frac{v}{k}$.

→ here v does not changes bcz crosssectional Area of All Sample is same So discharge is same Thus 'v' is also same.

$$\frac{z}{k_{eq}} = \frac{z_1}{k_1} + \frac{z_2}{k_2} + \frac{z_3}{k_3}.$$

$$k_{eq} = \frac{z}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \frac{z_3}{k_3}}$$ & w.k.t $z = z_1 + z_2 + z_3$

$$\Rightarrow k_{eq} = \frac{z_1 + z_2 + z_3}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \frac{z_3}{k_3}}$$

By observing on several Experiments it was confirmed that

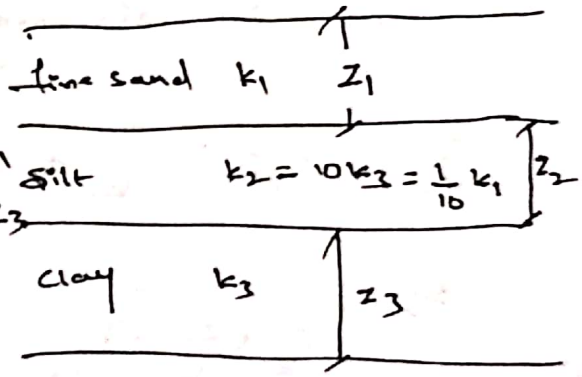
The value of $(k_{eq})_{horizontal} > (k_{eq})$ in vertical.

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Problem on stratified layer's: (Gate- 2015 Qstn). (2m).

Q:- A homogeneous silt deposit consists of a silt layer ~~of~~ equal thickness of lower and upper sandy layer. @ top and clay layer below. permeability of the silt layer is 10 time's the permeability of the clay layer & one tenth of the permeability of sand layer. Thickness of silt layer is 2 times the thickness of sandy layer & two third of the thickness of the clay layer. The ratio of Equivalent- Horizontal & Equivalent- Vertical permeability of Deposit is ?

$$\frac{k_{eq\,(Hz)}}{k_{eq\,(vertical)}} = ?$$



$$k_{eq\,(H)} = \frac{k_1 z_1 + k_2 z_2 + k_3 z_3}{z_1 + z_2 + z_3}.$$

$$\Rightarrow \frac{k_1 z_1 + \frac{1}{10} k_1 2(z_1) + (0.01 k_1)(3z_1)}{z_1 + 2z_1 + 3z_1}$$

$10k_3 = \frac{1}{10} k_1$

$k_3 = \frac{1}{100} k_1$

$= 0.01 k_1$

$2z_1 = \frac{2}{3} z_3$

$z_3 = 3z_1$

$$k_H = \frac{k_1 z_1 [1 + 0.2 + 0.03]}{6 z_1}$$

$$k_H = 0.205\, k_1$$

$$k_V = \frac{z_1 + z_2 + z_3}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \frac{z_3}{k_3}} \Rightarrow \frac{z_1 + 2z_1 + 3z_1}{\frac{z_1}{k_1} + \frac{2z_1}{0.1 k_1} + \frac{3z_1}{0.01 k_1}}$$

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$$K_V = \frac{6K_1}{\frac{K_1}{K_1}[1 + 20 + 300]} \Rightarrow \frac{6K_1}{321} \Rightarrow 0.0187K_1$$

$$\text{Now } \frac{K_H}{K_V} = \frac{0.205K_1}{0.0187K_1} \Rightarrow 10.97$$

Determination of co-efficient of permeability :-

- Indirect Method
  - → Allen Hazen Method
  - → Louden's Eqn
  - → Kozeny Carman's Eqn
  - → Consolidation Eqn.

    $C_k = C_v m_v \cdot f(w)$.
- field Method
  - → pumping-in-method
  - → pumping-out-method
    - wc-will study in WRE
- Laboratory Method.
  - → constant head permeability Test
  - → falling/variable head permeability Test
  - → capilary permeability test.

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Constant Head permeability Test:- (coarse grained soil's only).

Ex:- Gravel & sand

From Darcy's law W.K.T

$V \propto i$

$V = k \times i$

$k = \frac{V}{i}$

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

$k = \frac{V}{\frac{\Delta h}{L}}$

$k = \frac{V \times L}{\Delta h}$.

W.K.T

$Q = A \times V$

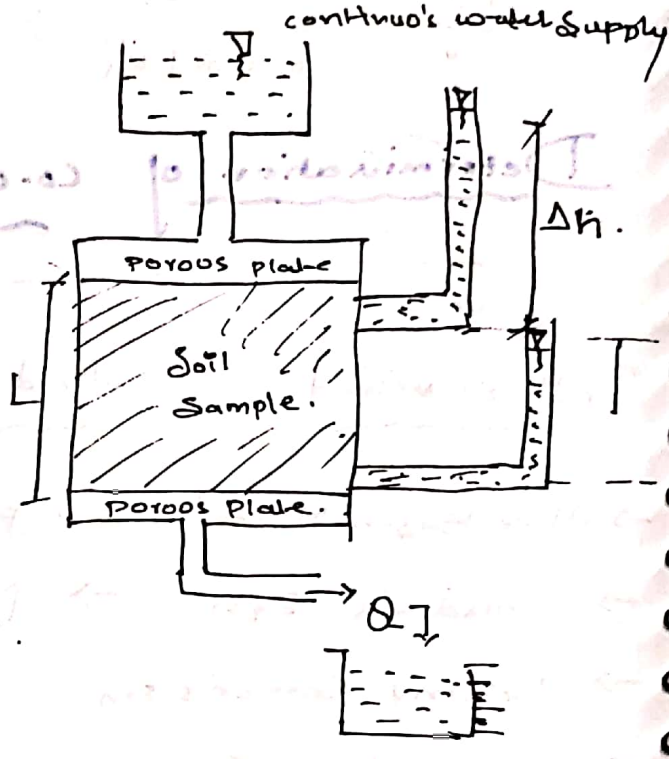
$V = \frac{Q}{A}$.

$k = \frac{Q \times L}{A \times \Delta h}$

$Q = \frac{V}{t}$.

$$\boxed{k = \frac{V \times L}{t \times A \times \Delta h}}$$

Units: cm/s or m/day.



Here. V = Volume of soil sample

L = Length of Soil Sample

Length of flow path

t = time in Sec.

A = Area of cross section

Δh = Head Loss

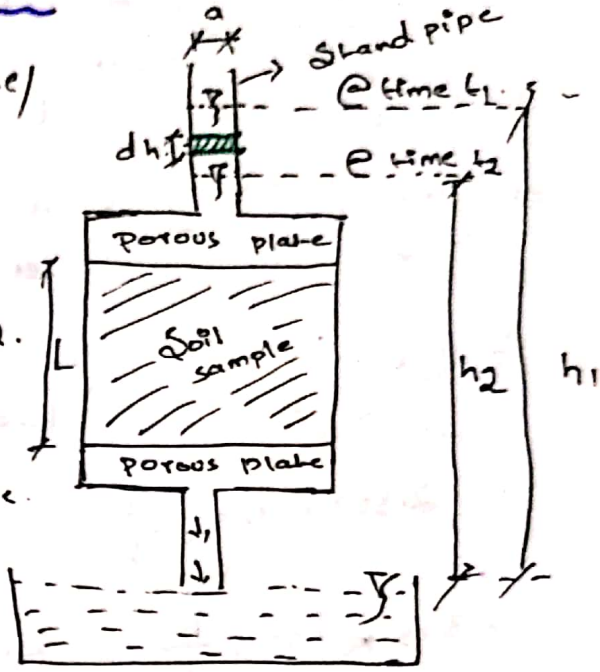
→ Here Δh Should be maintain Constant.

→ To maintain Const head water Should Supply Continuously.

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Falling Head Permeability Test: / Variable Head Permeability Test:

for [Silt's & fine sand's]



→ here we will take known Area of Tube/ Stand pipe (a).

→ initially @ time t_1 we have head h_1 & @ time t_2 we have h_2.

→ ∴ discharge in tube is Equal to discharge in Soil Sample.

So we have frm Darcy's law.

$Q = k.i.A$ (Soil sample).

$$Q \Rightarrow k \times \frac{h}{L} \times A \quad \text{(for soil sample)} \rightarrow (1)$$

here $i = \frac{\Delta h}{L}$ Let $\Delta h = h$.

w.k.t same discharge is occured for tube also.

So Let us take @ any intermediate time 'dt' head diff is 'dh'

So for Tube discharge we have

$$Q = \frac{V}{t}$$ i.e velocity / time.

$$Q = \frac{a \times dh}{dt}$$

$$Q = a \cdot \frac{dh}{dt} \rightarrow (2)$$ This is for Tube.

(w.k.t Area of crossection of tube = a.
& Length discharged = dh.
So $v = (a \times dh)$
t = Small time 'dt'.

(1) = (2) So we have

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

{ -ve sign is due to as t ↑ head (dh) ↓ }

By Integration on B.S.

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

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

$$\Rightarrow -\left[\log h\right]_{h_1}^{h_2} = \frac{k \cdot A}{L \cdot a} \left[t\right]_{t_1}^{t_2}$$

$$\therefore \left[\log \frac{h_2}{h_1}\right] = \frac{k \cdot A}{L \cdot a} \left[t_2 - t_1\right]$$

$$-2.303 \times \log_{10} \frac{h_2}{h_1} = \frac{k \cdot A}{L \cdot a} (t_2 - t_1)$$

$$2.303 \times \log_{10} \frac{h_1}{h_2} = \frac{k \cdot A}{L \cdot a} (t_2 - t_1) \qquad t_2 - t_1 = T$$

$$\boxed{k = 2.303 \cdot \frac{a \cdot L}{AT} \log_{10} \left[\frac{h_1}{h_2}\right]}$$

calculation of missing reading

Suppose @ time t_1 = 3:00 pm we get head h_1

t_2 = 3:15 pm → we forgetten (h_2) missing.

t_3 = 3:30 pm → we got head h_3

$k = 2.303 \frac{aL}{AT} \log_{10} \left[\frac{h_1}{h_2}\right]$ → un-known → ① for t_1 & t_2

$k = 2.303 \frac{aL}{AT} \log_{10} \left[\frac{h_2}{h_3}\right]$ un-known for t_2 & t_3 by ②

∴ ① = ②

$$\log_{10} \left(\frac{h_1}{h_2}\right) = \log_{10} \left(\frac{h_2}{h_3}\right)$$

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

here all properties i.e a, L, A, T is same so k is also same if we evaluate

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Problem:- (1) Gate-2012 -(2m)

two Soil Specimens with identical Dimensions were Subjected to falling head permeability tests in laboratory under identical conditions. The fall of water head was measured after an identical Time Interval. The ratio of Initial to final water heads for the test involving the first specimen was 1.25. If the co-eff. of permeability of the Second specimen is 5 times that of the first, the Ratio of Initial to final water heads in the test involving the Second Specimen is

(A) 3.05 (B) 3.80 (C) 4.00 (D) 6.25

Sol:- Let us for soil sample - (1)

Let Co-of permeability = k_1

initial head - h_1

final head = h_2

given $\left(\frac{h_1}{h_2}\right) = 1.25$

Soil Sample (2)

$\Rightarrow k_2$.

initial = h_3

final = h_4.

given $k_2 = 5k_1$

$\left(\frac{h_3}{h_4}\right) = ?$

~~for~~ ~~k_T~~ = ~~$2.303 \log_{10}$~~

$k_1 = 2.303 \cdot \frac{aL}{AT} \log\left(\frac{h_1}{h_2}\right)$ $\quad k_2 = 2.303 \frac{aL}{AT} \log\left(\frac{h_3}{h_4}\right)$

$\frac{k_2}{k_1} = \frac{\log\left(\frac{h_3}{h_4}\right)}{\log\left(\frac{h_1}{h_2}\right)}$

$\frac{5k_1}{k_1} = \frac{\log\left(\frac{h_3}{h_4}\right)}{\log\left(\frac{h_1}{h_2}\right)}$

$5 = \frac{\log\left(\frac{h_3}{h_4}\right)}{\log(1.25)}$

$\Rightarrow 5 = \frac{\log\left(\frac{h_3}{h_4}\right)}{0.0969}$.

$\boxed{\frac{h_3}{h_4} = 3.05}$ → (A)

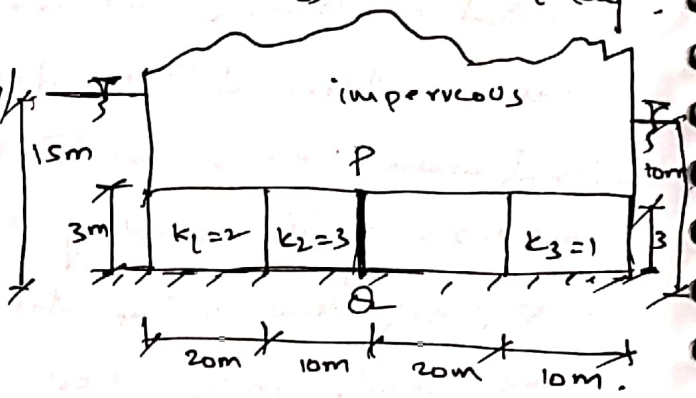
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(2) - gate- 2016:-

Seepage is occuring through a porous media shown in fig.

The hydraulic conductivity values (k_1, k_2, k_3) are in m/day.

The seepage discharge (m^3/day/m)

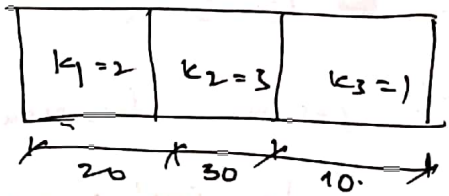


Through porous media @ section PQ is

(A) $\frac{7}{12}$ (B) $\frac{1}{2}$ (C) $\frac{9}{16}$ (D) $\frac{3}{4}$

→ we have initial head

$h_1 = 15m$, final head $= h_2 = 10m$ Q →



$\Delta h = 15-10 = 5m$.

Area = $L \times B \Rightarrow 3 \times 1 = 3\ m^2$ (1 is for unit width).

⇒ flow is ⊥ er to soil layers so Avg Equivalent k is

$$k_{eq} = \frac{z_1+z_2+z_3}{\frac{z_1}{k_1}+\frac{z_2}{k_2}+\frac{z_3}{k_3}} \Rightarrow \frac{20+30+10}{\frac{20}{2}+\frac{30}{3}+\frac{10}{1}}$$

$$k_{eq} = \frac{60}{10+10+10} = \frac{60}{30} = 2$$

$k_{eq} = 2$ m/day.

Now discharge $Q = k.i.A.$

$$Q = 2 \times \frac{5}{60} \times 3 \times 1$$

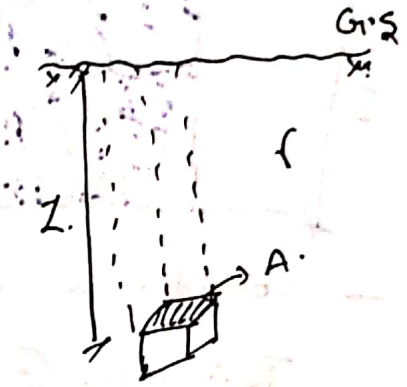
$$Q = \frac{1}{2}\ m^3/day/meter \quad (B).$$

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: Effective Stress & Seepage Through Soils :-

→ Before getting in to Effective Stress we kneed to know Total Stress & pore water pressure.

Total Stress :- (σ).



→ Consider a Soil Element below the G.L.

→ Due to wt of soil above that point stresses are acting on Element.

→ These stress are due to self wt of soil.

→ Let u.w of soil = γ, depth of element = Z.

→ Surface Area of Element = A.

→ Now Total weight = unit wt x volume.

$\Rightarrow \gamma \times (Z \times A)$.

So Total Stress σ = $\frac{\text{weight}}{\text{Area}}$ $\Rightarrow$ $\frac{\gamma \times Z \times A}{A}$

$$\boxed{\sigma = \gamma \times Z}$$ kN/m², N/mm² ...

→ it is also known as overburden pressure.

Pore water pressure :- (u).

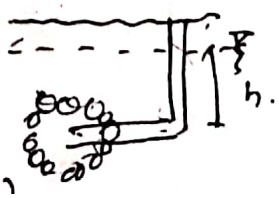
The pressure Exerted by the water which is presenting in void / pore spaces in on and around the Mineral / soil grain's is known as p.w.p.



→ p.w.p under No flow condition i.e hydrostatic condition is given by

$$\boxed{u = \gamma_w \times h}$$

h = pressure head (rised in piezometer)

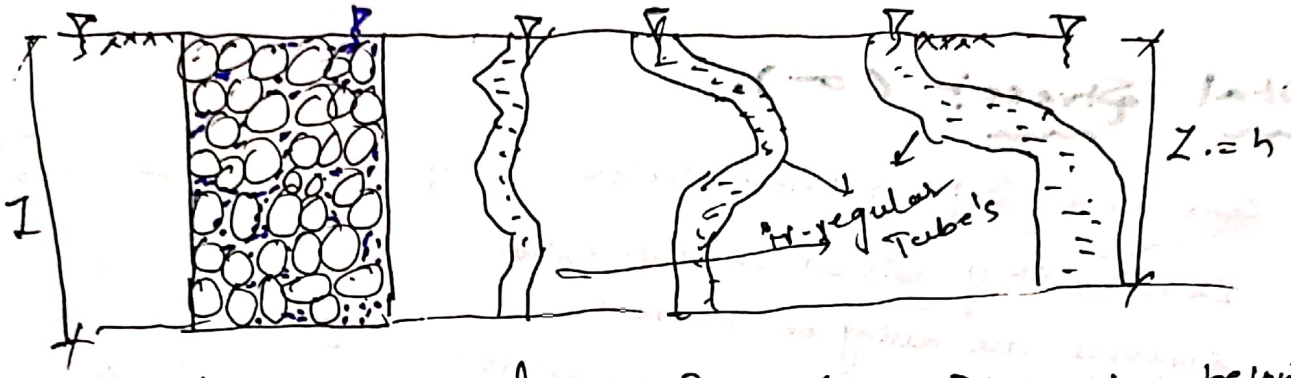


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→ Piezometer's are used upto a small depth's only.

→ But p.w.p is present's @ greater depth's below G.L's also.

→ So at those situation's p.w.p can be calculated as below.



→ Consider a soil layer below G.L's presenting below W.T also. @ a depth Z.

→ Here void's are compleatly filled with water and here void's are inter-connected.

→ as the void's are inter connected they form tube like structure. Some of them are may be irregular also.

→ But all the water rises to height 'Z'. So the p.w.p is given by $\boxed{u = \gamma_w \times Z}$

→ ~~height~~ here higher 'h' is replaced by depth Z.

Effective Stress (σ') (1936).

⇒ According to Terzhagi Effective Stress is given by

$$\boxed{\sigma' = \sigma - u}$$

σ = Total Stress.

u = p.w.p.

Defi: it is the pressure transmitted Through the contact area of soil solid's. i.e

$$\sigma' = \frac{\text{Contact force}}{\text{Area.}}$$

contact

⁂

→ Valid only for Saturated soil's.

Later The Defi. of Terzaghi's Eqn has proved wrong. by

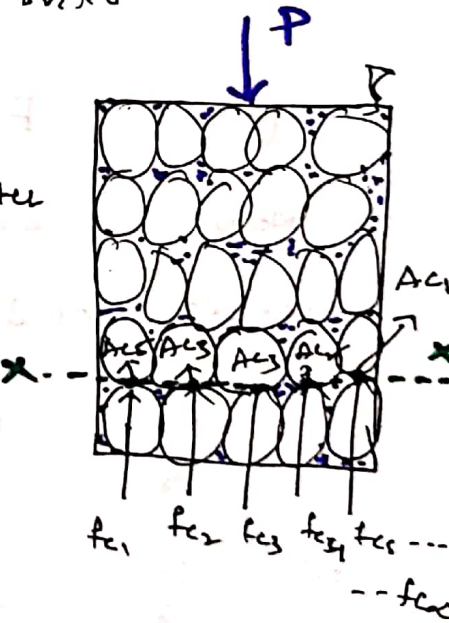
Lambe & Whitman (1969):

→ They Took a fully Saturated Soil Sample. & considered a Verticle load 'p' coming onto soil.

→ Consider a Section x–x in soil vol.

→ there load 'p' is Taken by soil + water

i.e p = load Taken by soil Solid's Thrgh Thir Contact area. + load Taken by water.



→ in b/w Two particle we have a contact

@ Each & Every contact point we have an up ward contact force $f_{c_1} \cdots f_{c_n}$ To resist load 'p'.

i.e $f_{c_1} + f_{c_2} + \cdots f_{c_n} = \Sigma f_c = F_c$ = Total contact force.

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So F_c = load Taken by soil solid's

→ Now we need To find out load Taken by water.

w.k.t load = pressure x Area.

→ let us consider Total Crosssectional Area of soil Sample is 'A'. & @ Each & Every Contact point we have contact area's i.e $A_{c1}, A_{c2}, A_{c3} \ldots A_{cn}$.

→ now water area Can be calculated by Total crosssectional Area − contact area. i.e

$A_{c1}, A_{c2} \ldots A_{cn} = \sum A_c = A_c$.

$\Rightarrow (A - A_c) \Rightarrow$ Area of water.

→ and water pressure is 'u' i.e p.w.p. So

$$P = F_c + u(A - A_c).$$

→ Now we need To calculate Total Stress.

w.k.t Total Stress = $\frac{\text{load}}{\text{Area}}$

$$\Rightarrow \frac{P}{A} = \frac{F_c}{A} + u\left(\frac{A - A_c}{A}\right)$$

$$\Rightarrow \frac{P}{A} = \frac{F_c}{A} + u\left[1 - \frac{A_c}{A}\right].$$

→ let consider Section x − x.

→ @ xx − we have crosssectional Area is A & Contact area is $A_{c1} \ldots A_{cn} = A_c$.

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$\Rightarrow$ here A_c is very very small on comparing with 'A'

$\Rightarrow$ if we devide A_c with A i.e $\frac{A_c}{A}$ it almost becomes 0

i.e $\frac{A_c}{A} \approx 0$. So Eqn Becomes.

$$\frac{P}{A} = \frac{F_c}{A} + u.$$

$$\boxed{\sigma = \frac{f_c}{A} + u.} \longrightarrow (1).$$

from Terzaghi's Eqn we have $\sigma' = \sigma - u$

$$\boxed{\sigma = \sigma' + u.} \longrightarrow (2)$$

on Comparing (1) & (2) we get

$$\boxed{\sigma' = \frac{F_c}{A}.} = \boxed{\frac{\text{Total contact force}}{\text{Total Cross sectional Area.}}}$$

But as per Terzhagi $\sigma' = \frac{\text{Contact force}}{\text{Contact Area.}}$ so this is wrong.

But $\sigma' = \frac{F_c}{A}$ is also physically not meaning full.

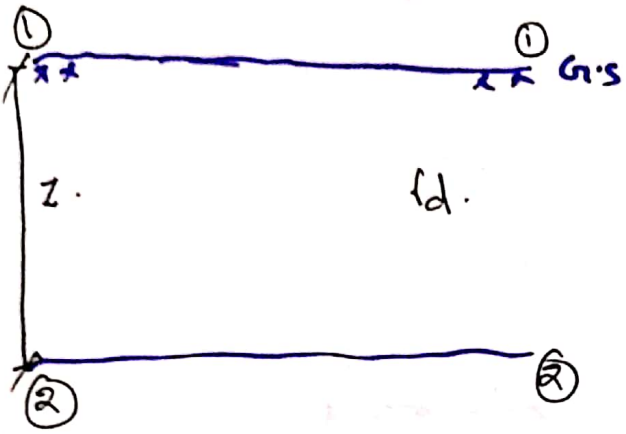
i.e $\frac{F_c}{A_c}$ is ok But $\frac{f_c}{A}$ is not Measurable.

$\Rightarrow$ So we never Measure σ' directly. Until & unless we mesure σ & u.

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Variation of Effective Stress with Water Table :-

Case (i): No-water Table (Dry soil) :-



→ consider a soil sample depth 'z' with Dry condition.

→ @ Section ①-① $\sigma_{11} = 0$ Bcz There is no additional soil is present above ①-①. & Total Stress is due to soil wt only

& $u_{11} = 0$ (i.e p.w.p = 0) Bcz Soil is dry.

So w.k.t $\sigma' = \sigma - u$

$\Rightarrow \sigma_{11}' = 0 - 0 = 0.$

@ ②-② $\Rightarrow \sigma_{22} = fd \times z$, $u_{22} = 0.$

$$\boxed{\sigma_{22}' = fd\, z.}$$

Case (ii): wt is at any depth

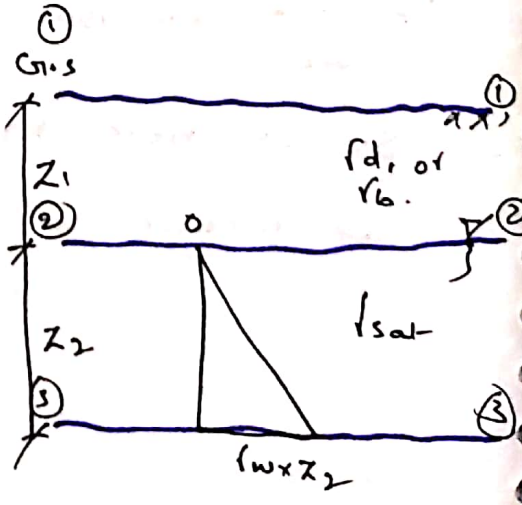
@ ①-①: $\sigma_{11} = 0$, $u_{11} = 0$. So

$\sigma_{11}' = 0.$

@ ②-② : $\sigma_{22} = fd \times z_1$, $u_{22} = 0$

$\sigma_{22}' = fd\, z_1$

Bcz w.t starts @ ②-② only.



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@ ③-③ :- $\sigma_{33} = \gamma_d \times z_1 + \gamma_{sat} \times z_2$; $u_{33} = \gamma_w \times z_2$

BC3 $\boxed{u = \gamma_w \times h}$ So

$$\sigma'_{33} = \gamma_d z_1 + \gamma_{sat} z_2 - \gamma_w z_2$$

$$= \gamma_d z_1 + z_2 [\gamma_{sat} - \gamma_w]$$

$$\boxed{\sigma'_{33} = \gamma_d z_1 + z_2 \gamma_{sub}} \quad (\because \gamma_{sat} - \gamma_w = \gamma_{sub})$$

on comparing cas(i) & case(ii) we may say That- Effective Stress ↓ on increasing w.t (or) p.w.p 'u'.

BC3 in case(i) we have $\gamma_d z$ & in case (ii) we get γ_{sub}. which is very very less & $(z_1 + z_2 = z)$ so σ' ↓.

Case (iii) wt. is @ G.L :-

@ ①-① :- $\sigma_{11} = 0$, $u_{11} = 0$ So

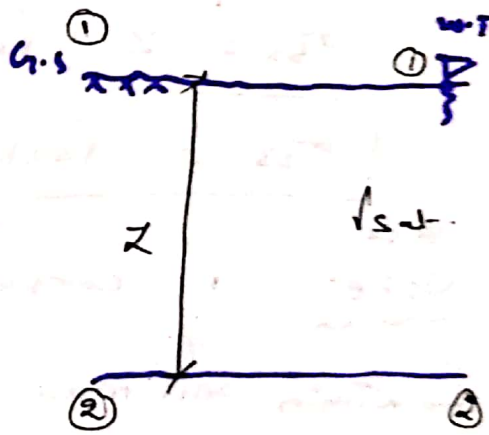
$$\sigma'_{11} = 0.$$

@ ②-② :- $\sigma_{22} = \gamma_{sat} \times z$, $u_{22} = \gamma_w \times z$.

$$\sigma'_{22} = \gamma_{sat} z - \gamma_w z.$$

$$= (\gamma_{sat} - \gamma_w) z.$$

$$\boxed{\sigma'_{22} = \gamma_{sub} \times z.}$$

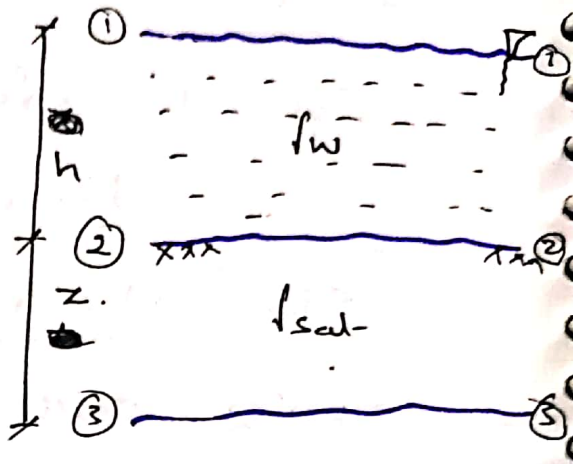


if we compare case (iii) with remaining cases Here σ' is further ↓ due to ↑ in w.t.

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Case-IV) if w.t is above G.L :-

→ These Type of cases possible in pond's, Lake's, & ea's b.cz Here w.t is above G.L.



@ ①-① $\sigma_{11} = 0, \; u_{11} = 0, \; \sigma'_{11} = 0.$

@ ②-② $\sigma_{22} = h \times \gamma_w, \; u_{22} = \gamma_w \times h.$

$\sigma'_{22} = h\gamma_w - h\gamma_w = 0.$

$\sigma_{22}' = 0.$

@ ③-③ :- $\sigma_{33} = \gamma_w \times h + \gamma_{sat} \cdot z, \quad u_{33} = \gamma_w [h+z]$

$\sigma'_{33} = \gamma_w h + \gamma_{sat} z - \gamma_w h - \gamma_w z$

$\sigma'_{33} = z [\gamma_{sat} - \gamma_w]$

$$\boxed{\sigma'_{33} = \gamma_{sub} \times z.}$$

★★

Note:- on comparing all cases we may conclude That-

→ Within soil mass if we ↑ w.t (p.w.p) Then σ' is ↓. up to Ground surface.

→ above G.L if we increase ↑ w.t (or p.w.p) There is no change in Effective stress.

(01)

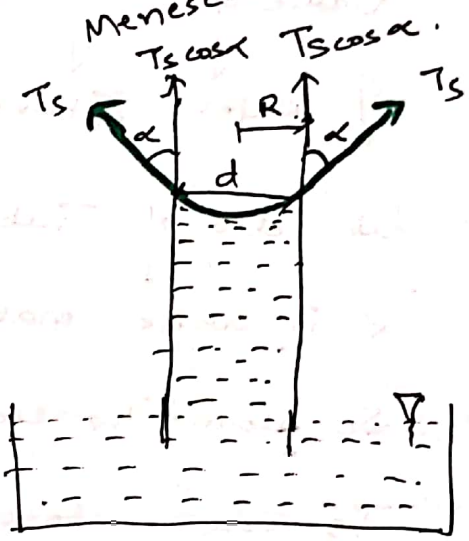
Above The G.L Specially In case of River's, ocean's, & ea's, ponds if w.t fluctuate's Then There is no change in Effective stress.

i.e "Effective stress remain's same"

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Capilarity or Capilary Effect:-

Basic Def: Rise of liquid (or) water level inside the capilary tube due to surface tension is known as capilarity. and this effect is known as capilary effect.



→ Here we will take a water tub insert a glass tube.

→ Here water rises up to a certain level by forming Menescos. due to surface tension T_s.

→ Commonly we have two types of force's
① Atraction → cohesion (cohesion force.
→ Adhesion.
② Repulsion.

Cohesion force:- When there is a atraction b/w same type of particle's then that force is known as cohesion.

Adhesion:- when there is a attraction b/w two diff meterial's then it is known as Adhesion.

→ capilary action is occured due to Adhesion forces bcz here glass & water are two diff meterials.

→ due to adhesion forces water is risen. up to some height which is known as height of capilary rise (hc)

→ Risen water is known as capilary water

→ This effect is known as capilarity.

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⇒ Here Height of rise is limited To a certain value.
it will rise To Some finite distance.

→ Here water is risen up to a point where The value of Surface Tension is Equal To The weight of water stored in That Tube.

→ let dia of Tube is 'd'.

→ 'α' is angle making with Tube.

→ So around The perimeter of Tube in upward direction "Ts cosα" force is acting all around The tube.

⇒ So resultant force is given by.

$(\pi d)\ T_s \cos\alpha \uparrow = \left(\frac{\pi}{4} d^2 \times h_c\right) \rho_w \cdot g \downarrow$ (due To gravity)

perimeter × Surface tension = wt of water in cylindrical Tube.

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

hc is dependent on α value.
i.e α = 0 Then cosα = 1. → max value

So $$h_{c\,max} = \frac{4 T_s}{\rho_w \cdot d}$$

d = diameter of menisus.

If we wrote in radias. Then $R = \frac{d}{2} \Rightarrow d = 2R$

$$h_{c\,max} = \frac{2 T_s}{\rho_w R}$$

R = Radius of Menisus.

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Capilarity in soil:-

⇒ ** There is no capilarity in fully saturated soil's

** capilarity occures in partially saturated soil's only

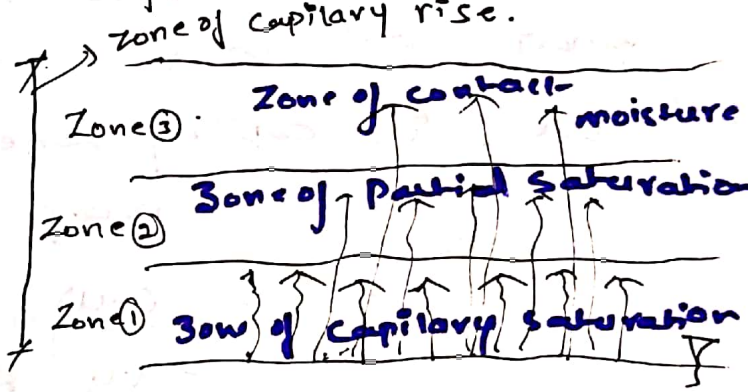
⇒ Here the role of Tube is taken by soil solid's.

⇒ Consider a soil strata having w.t @ a certain depth.

⇒ above w.t we have partially saturated soil & below w.t we have fully saturated soil.

⇒ Due to capilary action the water level is raised up to certain height.

⇒ Here water will raise against gravity.



→ Raise of water height is depends on Type of soil & diff conditi's.

→ in Zone ① - soil is fully saturated.

Zone ② - Soil is partially saturated.

Zone ③ - Soil is having only some moisture.

⇒ when the water is raising against gravity we may say that water is in Tension. This is known as capilary Tension.

⇒ So w.k.t water exerts p.w.p. So in Tension p.w.p = -ve.

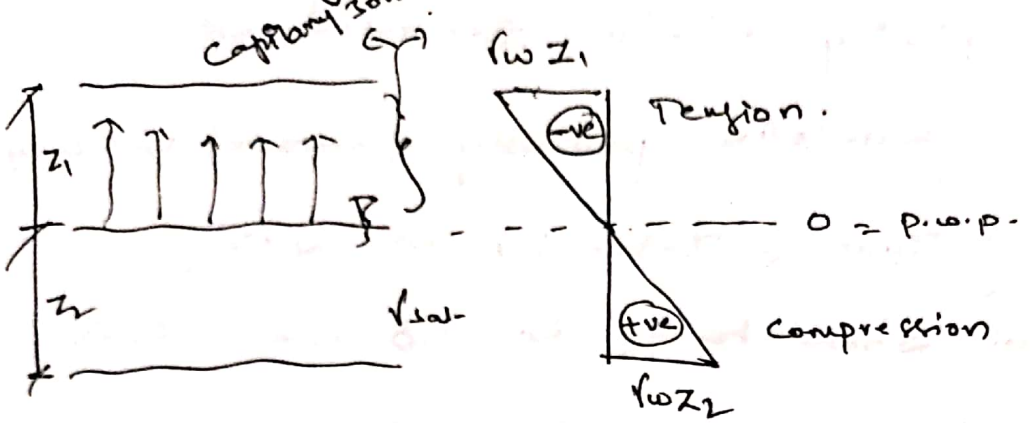
⇒ Below w.t p.w.p = Compression = +ve.

Ex:-



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Destribution of p.w.p (u):-



Effective stress in capilary Zone:- ★★★

w.k.t $\sigma' = \sigma - u$.

But in capilary Zone water is in Tension so p.w.p = $(-u)$ so

$\sigma' = \sigma - (-u)$

$\boxed{\sigma' = \sigma + u.}$ ★★

So Effective stress ↑.

Ex:- It is very difficult to walk away from sea coast But very Easy to walk near the sea-coast.

Bcz near sea cost we have capilary Zone so σ'↑.

How σ' increases due to capilarity?

→ Initially water is in Tension known as Capilary Tension.(T)

→ water Transmit this pressure to soil Through compression form known as capilary pressure.

→ as compression occurs in soil solid's e ↓ u.w ↑ So σ'↑

How the water is Hold b/w solid particles?



→ in B.w solid particles There is some adsorbed water

→ water will form like menescos & Surface Tenfion occurs

→ Due to This Surface Tenfion contact force b/w particles ↑.

→ so water hold's, & strength ↑.

→ due to Surface Tenfion contact force ↑ This is known as "contact pressure".

Why Capilarity does not occure in fully saturated soil?

=> In fully saturated soil's ~~[illegible]~~ a distance b/w soil grain's is very high. i.e Radius of Menescos 'R' ↑. & we have

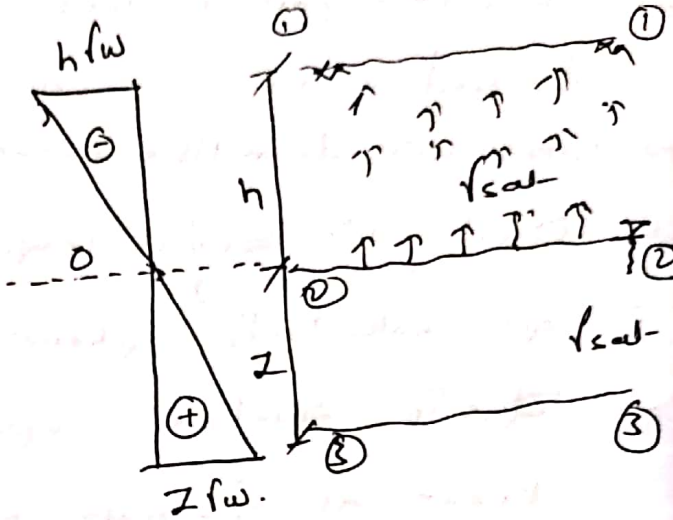
$$h_{cmax} = \frac{2T_s}{\gamma_w \cdot R}$$

here $h_c \propto \frac{1}{R}$.

→ so if R ↑ h_c ↓. for fully saturated soil's $R = \infty$ Then $h_c = 0$.

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Effect of capilarity on Effective stress:-



@ section ① - ①

$\sigma_{11} = 0, \quad u_{11} = -h\gamma_w.$

$\sigma'_{11} = h\gamma_w.$

@ section ② - ②

$\sigma_{22} = h \times \gamma_{sat}, \quad u_{22} = 0.$

@ section ③ - ③

$\sigma_{33} = h\gamma_{sat} + z\gamma_{sat}, \quad u_{33} = z\gamma_w.$

$\sigma'_{33} = h\gamma_{sat} + z\gamma_{sat} - z\gamma_w.$

$= h \times \gamma_{sat} + z(\gamma_{sat} - \gamma_w)$

$$\boxed{\sigma'_{33} = h\gamma_{sat} + z \cdot \gamma_{sub}.}$$

~~From this~~ we

→ on comparing with previous cases of Effective stress here we have ↑ in Effective stress.

→ in previous cases we didn't got any Effective stress @ Top layer. but here we got $\sigma' = h\gamma_w$.

i.e ~~σ~~ σ' ↑.

→ ↑ in Effective stress is $\boxed{\sigma' = h\gamma_w}$

i.e height of capilary rise × U.W of water.

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Seepage Through Soil's:-

:- permeability:-

→ property of soil which allow's the water through it's interconnected void spaces.

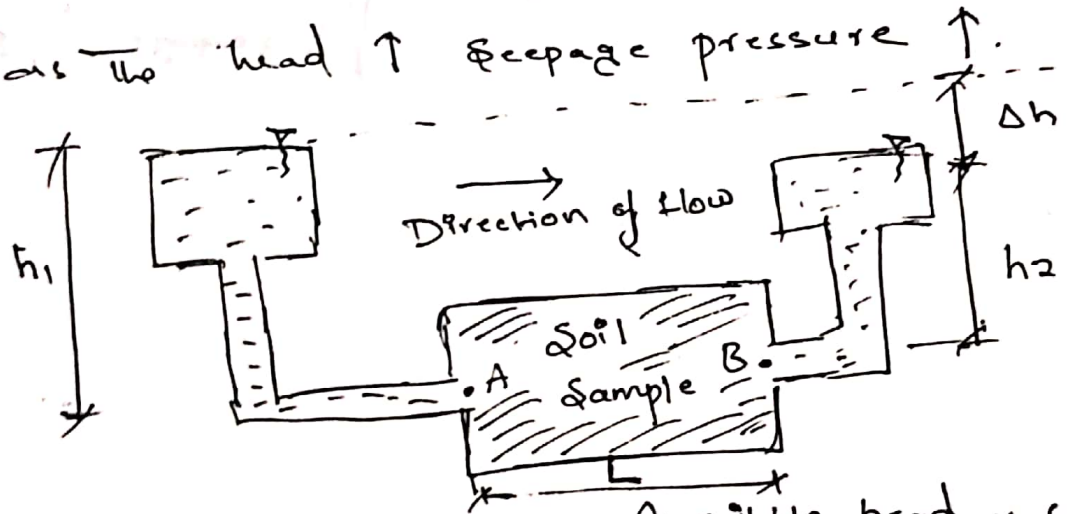
:- Seepage:-

→ The process of flow of water is known as seepage.

→ Seepage & permeability occur's when there is a head difference in two end's.

→ due to this head difference water exert's some pressure on the soil which is known as "seepage pressure."

→ as the head ↑ seepage pressure ↑.



here seepage pressure = Available head x u.w of water
i.e head diff

⇒ $h \times \gamma_w$ or $\Delta h \gamma_w$.

So now $h = h_1 - h_2$

Seepage force = Seepage pressure x Area.

$= h\gamma_w \times A$.

$\Rightarrow h\gamma_w \times A \times \frac{L}{L}$

$\Rightarrow \frac{h}{L} \times \gamma_w (A \times L)$

Let A is crossectional Area of soil sample

L = length of flow

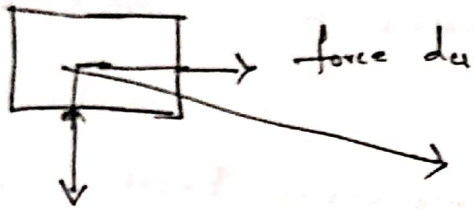
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Seepage force. $\Rightarrow i \gamma_w \times V$ ($\because A \times L =$ volume).

Now

Seepage force per unit volume (i.e $V = 1\,m^3$)

$= i \times \gamma_w$. (KN/m³).

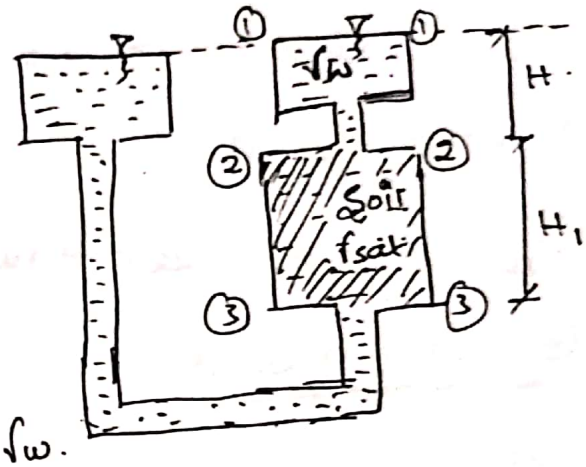


$f_R = \sqrt{(\text{force due to seepage})^2 + (\text{force due to Effective stress})^2}$

Effect of Seepage on Effective Stress:-

Case (i): No-flow-condition:-

=> there no flow occuring due to there is no-head diff.



@ (1)-(1) $\sigma_{11} = 0, \; u_{11} = 0$

$$\boxed{\sigma'_{11} = 0}$$

@ (2)-(2) $\sigma_{22} = \gamma_w \times H, \; u_{22} = H \times \gamma_w$.

$$\text{So } \boxed{\sigma'_{22} = 0}$$

@ (3)-(3)

$\sigma_{33} = \gamma_w \times H + \gamma_{sat} H_1, \; u_{33} = (H_1 + H)\gamma_w$.

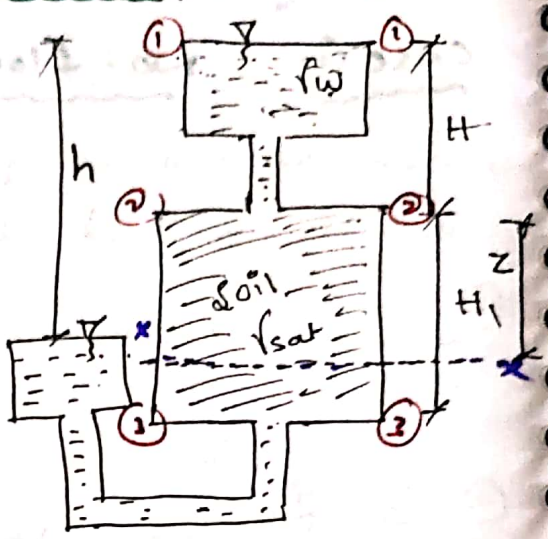
$\sigma'_{33} = \gamma_w H + \gamma_{sat} H_1 - H_1 \gamma_w - H \gamma_w$.

$\sigma'_{33} = H_1 (\gamma_{sat} - \gamma_w) \Rightarrow$

$$\boxed{\sigma'_{33} = H_1 \gamma_{sub}}$$

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Case ii; Downward flow condition:



@ Section ①-①

$\sigma_{11} = 0, \quad u_{11} = 0, \quad \sigma'_{11} = 0.$

@ Section ②-②

$\sigma_{22} = \gamma_w \times H, \quad u_{22} = +\gamma_w, \quad \sigma'_{22} = 0.$

@ Section ③-③

$\sigma_{33} = \gamma_w \times H + \gamma_{sat} \times H_1, \quad u_{33} = (H + H_1)\gamma_w - h\gamma_w.$

(2) =

⇒ Here seepage pressure acting ↓ down word direction & p.w.p acting ↑ upword

So resultant = p.w.p - & seepage pressure.

(Available head) x Unit wt.

$\Rightarrow (H + H_1)\gamma_w - h\gamma_w.$

So $\sigma'_{33} = \gamma_w H + \gamma_{sat} H_1 - H_1\gamma_w - H\gamma_w + h\gamma_w$

$\Rightarrow H_1\gamma_{sat} - H_1\gamma_w + h\gamma_w.$

$\Rightarrow H_1(\gamma_{sat} - \gamma_w) + h\gamma_w$

$\Rightarrow H_1\gamma_{sub} + h\gamma_w.$

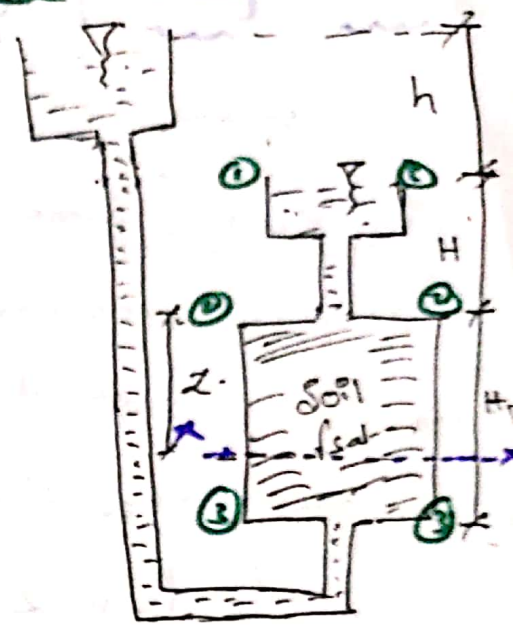
$$\boxed{\sigma'_{33} = H_1\gamma_{sub} + h\gamma_w}$$

on comparing case ① & ② we have ↑ in Effective stress in downward condition.

This is due to densification of soil due to seepage pressure.

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Case (ii) up-word flow condition:-



@ section ①-①

$\sigma_{11} = 0, \quad u_{11} = 0, \quad \sigma'_{11} = 0.$

@ section ②-②

$\sigma_{22} = H\gamma_w, \quad u_{22} = H\gamma_w, \quad \sigma'_{22} = 0.$

@ section ③-③

$\sigma_{33} = H\gamma_w + H_1\gamma_{sat},$

$u_{33} = (H + H_1)\gamma_w + h\gamma_w$ } B.C.s p.w.p & seepage pressure are in same direction }

$\sigma'_{33} = H\gamma_w + H_1\gamma_{sat} - H\gamma_w - H_1\gamma_w - h\gamma_w$

$$\boxed{\sigma'_{33} = H_1\gamma_{sub} - h\gamma_w}$$

$$\boxed{\sigma'_{xx} = \gamma_{sub} z - i\gamma_w z.}$$ → Like in case (ii).

So here we conclude that in up-word seepage condition effective stress decreases by $h.\gamma_w$ times.

Consider a case (i)

Now we need to Det effective stress @ any intermediate section x-x. Then

we have Eqn for case(ii) is $\sigma'_{33} = \gamma_{sub} H_1 + h\gamma_w.$

$\sigma'_{33} = \gamma_{sub} \times H_1 + h\gamma_w \times \frac{H_1}{H_1} \Rightarrow \gamma_{sub} H_1 + \left(\frac{h}{H_1}\right)\gamma_w H_1.$ → i

$$\boxed{\sigma'_{33} = \gamma_{sub} H_1 + i\gamma_w H_1}$$

So @ any section x-x is given by

$$\boxed{\sigma'_{xx} = \gamma_{sub} \times z + i\gamma_w z.}$$

$$\boxed{i = \frac{h}{H_1}}$$

$i = \frac{\text{Total Head Loss}}{\text{Total length of flow path}}$

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Quick Sand / Sand Boiling condition or Piping failure:-

→ We know that when water is flowing through soil it exerts some pressure on soil which is known as "seepage pressure".

→ in upword flow condition of seepage we have effective stress as below.

$$\sigma' = \gamma_{sub} \times z - i z \gamma_w$$

→ As p.w.p goes on increases i.e ($i z \gamma_w$) Total stress goes on ↓ i.e ($\gamma_{sub} \times z$) @ a point- p.w.p = Total stress occures.

→ @ that point effective stress becomes '0'.

→ let effective stress is become '0'.

Then $0 = \gamma_{sub} \times z - i z \gamma_w$.

$$0 = \gamma_{sub} \times z - i_c z \gamma_w$$

{ B.C whenever effective stress becomes '0' then i is called as i_c }

where i_c = Critical hydraulic gradient.

So $\gamma_{sub} \times z = i_c z \gamma_w$

$$i_c = \frac{\gamma_{sub}}{\gamma_w}$$

$$i_c = \frac{\frac{G-1}{1+e}\gamma_w}{\gamma_w}$$

$$i_c = \frac{G-1}{1+e}$$

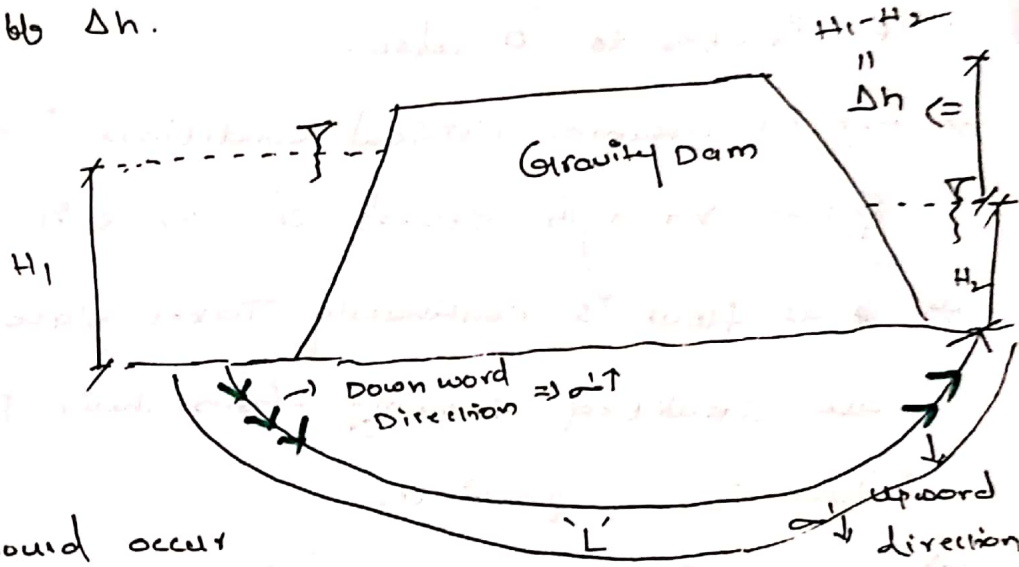
{ i.e $\gamma_{sub} = \left(\frac{G-1}{1+e}\gamma_w\right)$

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⇒ what is use of i_c ?

The use is whenever we constructing any hydraulic structure or a simple structure if we use any gradient value greater than i_c then soil get's quicksand condition i.e effective stress becom's '0'.

Ex:- Considera gravity dam (made up of concrete). having head diff Δh.



→ Here flow should occur due to head diff. but gravity dam doesn't allow flow through it's body so water flow below Dam.

⇒ Initially water will flow Downword & then upword direction.

⇒ Let us assume we know length of flow path (L)

So we have $\Delta h = H_1 - H_2$,

L = length of flow path.

$$\boxed{i_{exit} = \frac{\Delta h}{L}.}$$

if this $i_{exit} < i_c$. Then No Quick sand condition occur's Hence structure is safe.

if $i_{exit} > i_c$ Then Quicksand condition occur's Hence structure is un-safe

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How we can say a structure is safe or not?

→ Initially water flow in downward direction & w.r.t if flow occurs in downward direction σ' ↑ Bcz soil Becom's dense.

→ when water reaches to down stream side flow becom's upward direction so w.r.t σ' ↓. and Some times it reaches to '0' also.

→ Let us assume Critical condition i.e $\sigma' = 0$. So shear strength Becom's '0'. So soil is in suspension.

→ as flow is continuosly Takes place Then particles are started flowing from Their position. Then There is no particles.

→ Like wise as flow continuosly taking place Effective stress becom's '0' & soil goes on erod. finally soil below Dam completely got eroded.

→ So below Dam a hallow pipe like structure is formed.

→ There is no soil in This pipe Then sudden settlement of Dam occurs.

→ Whenever settlement occurs concrete is brittle material it shows The cracks.

→ This is also known as piping failure

How we can avoid piping failure?

→ To decrese the ~~ring~~ piping failure we need to bring Safe condition. i.e $i_{exit} > i_c$ to $i_{exit} < i_c$.

⇒ To bring Safe conditiong we need to ↓ the i_{exit} value.

w.k.t $i_{exit} = \frac{\Delta h}{L}$.

Here we have Two way's to ↓ i_{exit} value.

(i) $i_{exit} \downarrow$ if $L \uparrow$.

i.e we need to ↑ the flow path. to ↑ flow path we need to ↑ width of Dam which is not Economical so This case is not possible.

ii) $i_{exit} \downarrow$ if $\Delta h \downarrow$.

To decreas Δh. again we have two options

a) ↑ the Down Stream head (H_2).

This is not possible bcz in downstream side we have villages if $H_2 \uparrow$ Then flood will occurs so villages washedout

b) ↓ up Stream Side i.e (H_2) ↓.

This is also not possible Bcz in upstream side we are storing water for libb purposes.

But we have an option of ↑ the Length of flow by withwithout - ↑ the width of Dam.

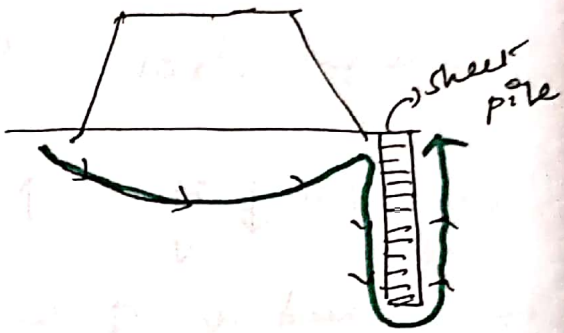
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⇒ To increase length of flow we need to place a sheet-pile in downstream side. (made of concrete / steel / wood depends on Design).

⇒ due to introuding of sheet-pile length of flow path ↑.

⇒ so i_{exit} ↓. so now

$$i_{exit} < i_c = \frac{G-1}{1+e}$$



⇒ so safe structure occurs.

So F.O.S [factor of safety] is given by

$$F_{OS\ (against\ piping\ failure)} = \frac{i_c}{i_{exit}}.$$

why piping not occurs in gravels & clayeys?

Sand

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

$\tau = 0$

so piping occurs

Clay

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

$\tau = c$

So piping not occurs.

Gravel

in gravel with voids are interconnected. it will allow water easily so it is having more permeability i.e

with $Q = KiA$.

$$K = \frac{Q}{iA}.$$

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

if K ↑ i ↓ so structure is safe.

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Flow Net's - characteristics & uses:

Isotropic soil:- @ a particular point properties of soil is same in all directions (i.e x, y, z).

An-Isotropic soil's:- properties of soil will change with direction.

we have Laplace Eqn for An-Isotropic soil as

$$k_x \cdot \frac{d^2H}{dx^2} + k_y \cdot \frac{d^2H}{dy^2} = 0$$ [for two dimension here x & y are direction's H = available head].

if we Consider the Isotropic soil properties will be same in all direction's i.e $k_x = k_y = k_z$

i.e $$\frac{d^2H}{dx^2} + \frac{d^2H}{dy^2} = 0.$$ [for Isotropic soil].

Vol Enter = volume Exit.

=> if we represent Laplace Eqn in graphical form Then it is known as "flow net"

=> it valid for Isotropic & an-Isotropic soil's.

=> it is used to calculate the amount of water flowing Through soil Section. [or to determine seepage discharge]

Stream Line / Flow-Line / Ψ-Line:- which shows the path of flow.. this also known as flow-Line or Ψ-line.

Equipotential Line / Φ-Line:- Line which connects the point's of equal Head or Equal Energy.

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Properties of flow Net:-

(1) Two - stream line's or two Equipotential line's never intersect each other

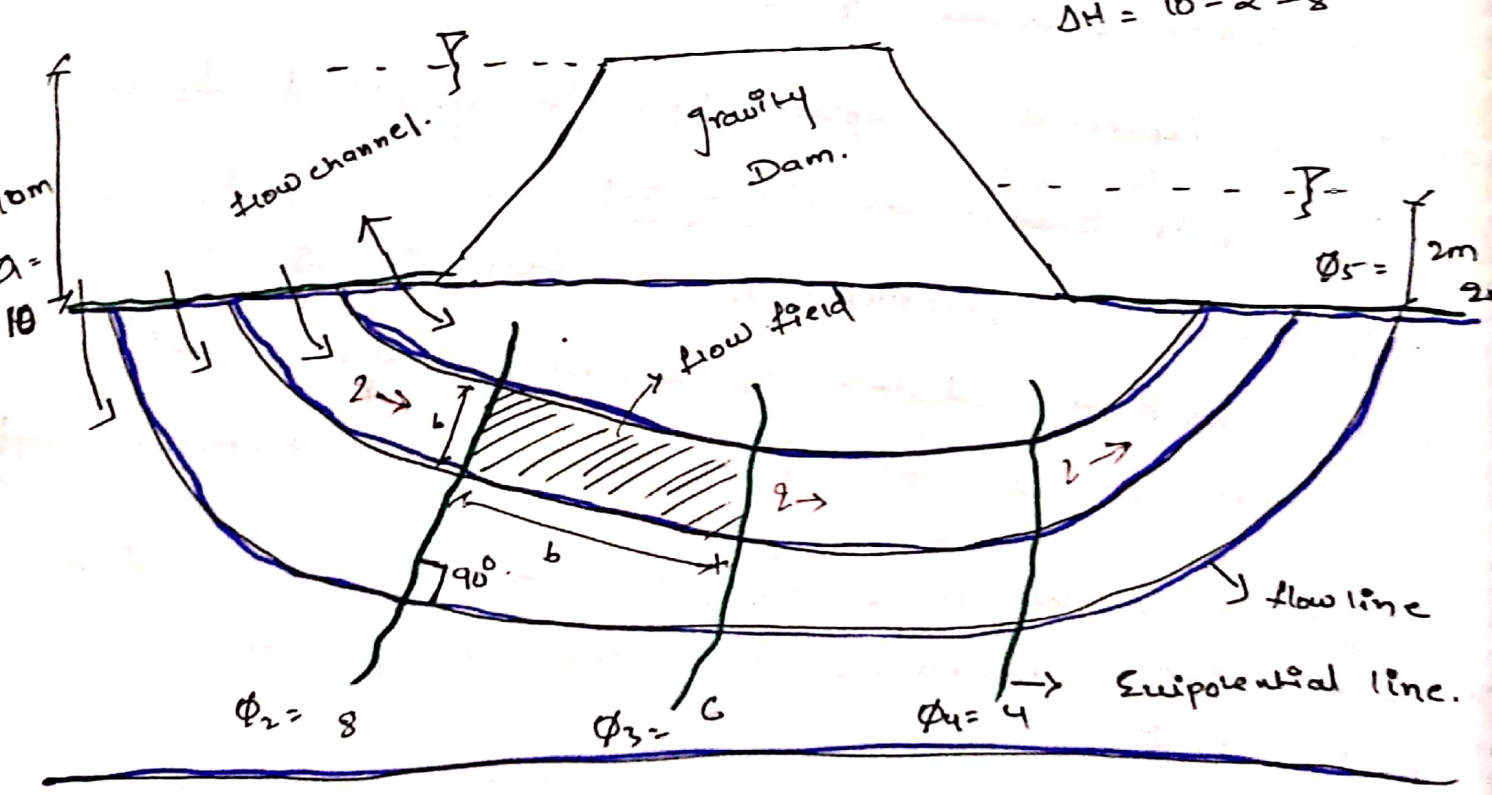
(2) flow lines & Equipotential line's are ⊥ to each other

(3) The Area bounded b/w Two flow-lines is called flow-channel

(4) The Area bounded b/w Two flow lines & two Equi-potential line's is called flow field.

(5) The flow field is square for Isotropic soil and Rectangular for an-Isotropic Soil.

(6) flow Net is unique for The given boundary condition's, it Remain's Same Even The up-stream & downstream Hed's are Reversed.



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=> Consider a gravity dam as shown in figure.

=> Here we have No. of flow lines = 5 (blue-colour)

No. of flow channels = 4

So No of flow Channels (N_f) = No. of flow lines - 1

= 4.

=> Here we have No. of Equipotential Line's = 5.

No. of Equipotential Drop's (N_d) = No. of Equipotential lines - 1

= 5-1 => 4.

=> Head Loss per Drop Δh is given by

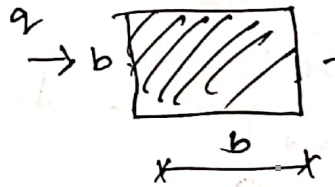
$$\Delta h = \frac{\Delta H}{N_d} \Rightarrow \frac{\text{Total Headloss}}{\text{No. of Drop's}} = \frac{8}{4} = 2$$

=> Now we need to calculate Total seepage discharge occuring from high Hedd to Low Head.

=> Consider Isotropic soil so for Isotropic soil w.k.t

flowe field = square.

=> we have $Q = k.i.A.$



for single flow field

=> assume the small amount of discharge 'q'

Length of flow = b.

Head Loss = Δh.

Consider unit Length i.e L = 1m.

& width = b.

So we have $q = k.\frac{\Delta h}{b} \times [1.b]$

$$q = k.\Delta h.$$

w.k.t $\Delta h = \frac{\Delta H}{N_d}$

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$\therefore$ $\boxed{q = k \cdot \frac{\Delta H}{Nd}}$ → This is discharge per a single channel.

→ we need to calculate Total discharge.

Total Discharge (Q)

Total Discharge = discharge per single channel x No. of flow channels.

$$Q = k \cdot \frac{\Delta H}{Nd} \times Nf$$

$\therefore$ $\boxed{Q = k \cdot \Delta H \times \frac{Nf}{Nd}}$ m^3/s or m^3/day or m^3/min

Note:- we should kept in meters only Bcz we assumed Length as unit length (1 m).

$\boxed{\frac{Nf}{Nd}}$ ⇒ is also called as Shap factor.

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Problems on permeability, Effective stress, Seepage:

① In a falling head permeability test the initial head of 1m dropped to 0.35m in 3hr's. The Dia of stand pipe is 5mm. The soil specimen was 200 mm long & 100 mm dia. The co-eff. of permeability of soil is ____ cm/sec.

a) 4.86×10^{-5} cm/sec b) 4.86×10^{-6} cm/sec.

c) 4.86×10^{-7} cm/sec. d) 4.86×10^{-8} cm/sec.

Sol:- W.K.T in falling head permeability

$$k = 2.303 \times \frac{aL}{AT} \log_{10}\left[\frac{h_1}{h_2}\right].$$

given dia of stand pipe = 5mm => 0.005m. =>

$$a = \frac{\pi}{4}(0.005)^2 \Rightarrow 0.00001925\ m^2$$

$\Rightarrow 1.96 \times 10^{-5}$

Initial head $h_1 = 1$ m.

Final head $h_2 = 0.35$ m. =>

Time T = 3hr's = $3 \times 60 \times 60$ sec. => 64800 sec.

$$A = \frac{\pi}{4} \times (0.1)^2 = 0.00785$$

(100 mm)

L = 200mm => 0.2m.

$$k = 2.303 \times \frac{(0.00001925)(0.2)}{(0.00785)(10800)} \log_{10}\left[\frac{1}{0.35}\right].$$

$k = 4.89 \times 10^{-8}$ m/sec. $\times 100$ cm/sec.

$$\boxed{k = 4.89 \times 10^{-6} \text{ cm/sec.}}$$

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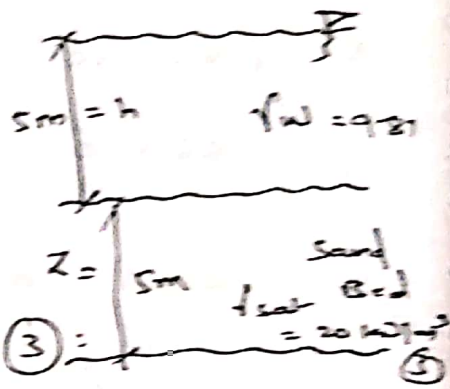
② A river of 5 m deep consists of a sand bed with saturated u.w of 20 kN/m³. $\gamma_w = 9.81$ kN/m³. The effective stress @ 5m from the top of sand bed is → ? Gate-1999.

ⓐ 41 kN/m² ⓑ 51 kN/m² ⓒ 55 kN/m² ⓓ 53 kN/m²

Soln: given $\gamma_{sat} = 20$ kN/m³.

$\gamma_w = 9.81$

$z = 5$ m.



Effective stress @ ⑤ - ③ is

$\sigma_{33} = h\gamma_w + z\gamma_{sat}$

$\Rightarrow 5 \times 9.81 + 5 \times 20 =$

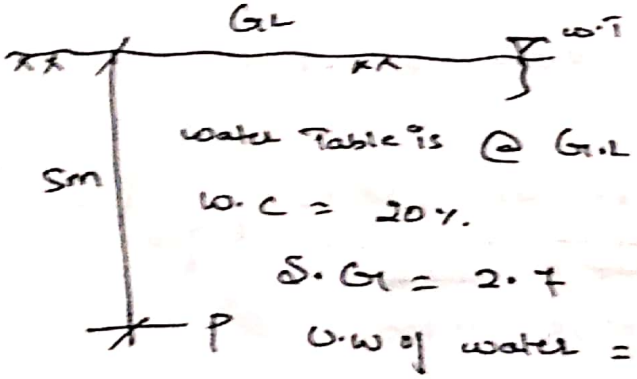
$u_{33} \Rightarrow h\gamma_w + z\gamma_w$

$\Rightarrow (5 \times 9.81 + 5 \times 9.81) =$

$\sigma' = \sigma_{33} - u_{33} \Rightarrow 50.95 \approx 51$ kN/m². $\Rightarrow$ ⓑ.

③ Common data Qstn (Gate-2008-4m).

The ground condition's @ site are shown in below given figure.



a) Saturated u.w of sand is

Ⓐ 15 Ⓑ 18 Ⓒ 21 Ⓓ 24.

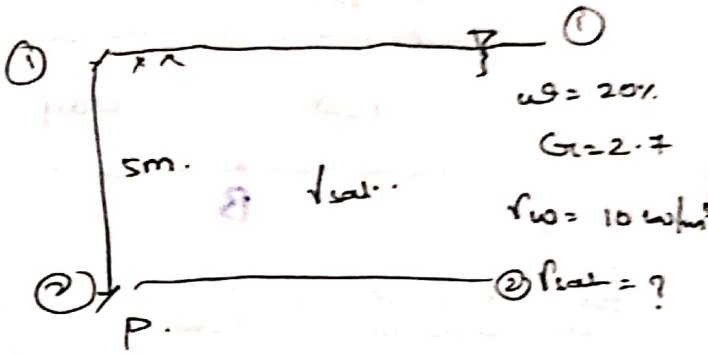
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(b). The Total stress, p.w.p & Effective stress (kN/m²) @ point P. are respectively

(A) 75, 50 & 25 (B) 90, 50 & 40

(C) 105, 50 & 55 (D) 120, 50 & 70.

Soln:-



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

$$e \cdot S_r = G_s \cdot w$$

$$e_{x1} = 2.7 \times 0.2 \Rightarrow \boxed{0.54}$$

$$\gamma_{sat} = \frac{(2.7+0.54)\,10}{(1+0.54)} \Rightarrow 21.03\ kN/m^3 = (C)$$

@ section (2) - (2)

Total stress $\sigma_{z2} = 5 \times 21 \Rightarrow 105\ kN/m^2$.

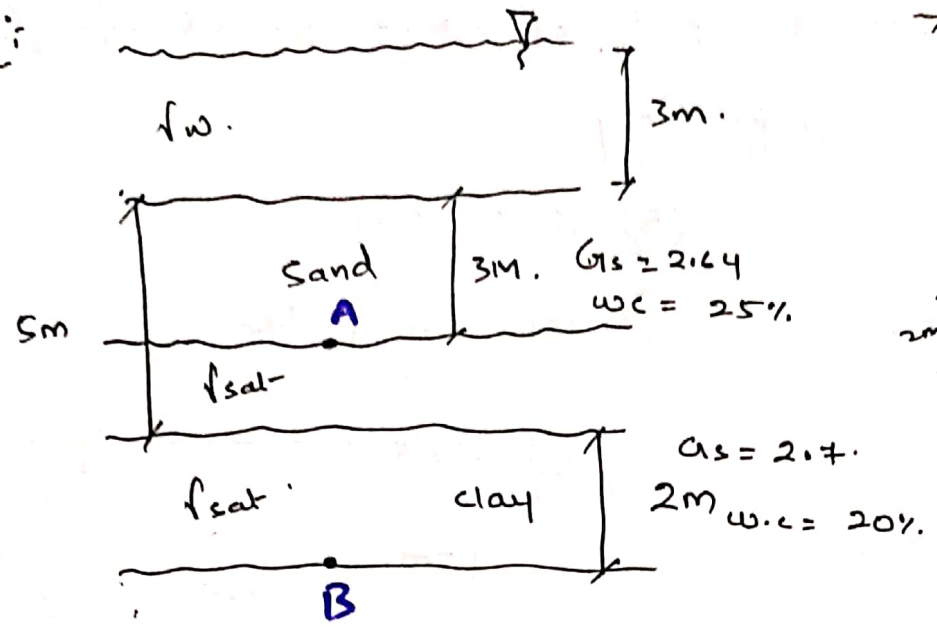
P.W.P $u_{z2} = (5 \times 10) = 50\ kN/m^2$

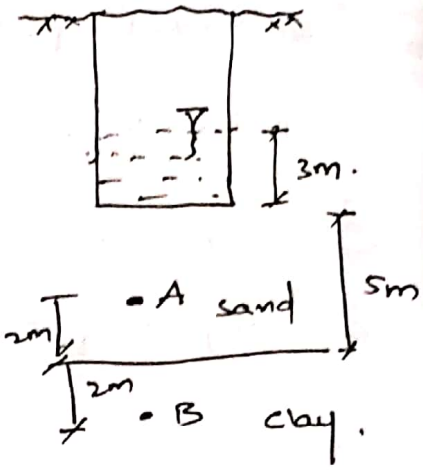
Effective stress $\sigma'_{z2} = \sigma_{z2} - u_{z2} \Rightarrow 105 - 50 \Rightarrow 55\ kN/m^2$

(4) The Depth of water in a well is 3m. Below the Bottom of well lies a layer of sand 5m Thick overlying a clay deposit. The Sp. gravity of sand & clay are resp. 2.64 & 2.70. Their w.c resp. 25 & 20 %. Compute the Total Intergranular & P.W.P @ points A & B as shown in fig.

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Soln:





for point A. $\gamma_{sat} = \left(\frac{G_s - 1}{1+e}\right) \times \gamma_w$.

$e S_r = G_s \cdot w \Rightarrow e = G_s \cdot w \quad [\because S_r = 1]$.

$e = 2.64 \times 0.25$

$e = 0.66$ ∴ 0

$\gamma_{sat} = \left[\frac{2.64 - 1}{1 + 0.66}\right] \times 9.81 \Rightarrow \boxed{9.7}$ kN/m³

Now $\sigma_A \Rightarrow \gamma_w \times 3 + \gamma_{sat} \times 3 \Rightarrow 9.81 \times 3 + 9.7 \times 3$

Total stress $\boxed{\sigma_A \Rightarrow 58.53}$ kN/m²

P.W.P = $u_A = \gamma_w \times 3 + \gamma_w \times 3 \Rightarrow 9.81 \times 3 + 9.81 \times 3$

$\boxed{u_A = 58.86 \text{ kN/m}^2}$

Effective stress = 58

for point B. $\gamma_{sat} = \left(\frac{G_s - 1}{1+e}\right) \gamma_w$, $e = G_s \cdot w$

$\Rightarrow 2.7 \times 0.2$

$\boxed{e = 0.54}$

$\gamma_{sat} = \left(\frac{2.7 - 1}{1 + 0.54}\right) \times 9.81$

$\boxed{\gamma_{sat} = 10.83 \text{ kN/m}^2}$

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Total stress @ B

$$\sigma_B = 3 \times \gamma_w + 5 \times \gamma_{sat(sand)} + 2 \times \gamma_{sat(clay)}$$

$$\sigma_B = 3 \times 9.81 + 5 \times 9.7 + 2 \times 10.83$$

$$\boxed{\sigma_B = 99.59 \text{ kN/m}^2}$$

p.w.p @ B.

$$u_B = 3 \times \gamma_w + 5 \times \gamma_w + 2 \times \gamma_w$$

$$= 3 \times 9.81 + 5 \times 9.81 + 2 \times 9.81$$

$$\boxed{u_B = 98.1 \text{ kN/m}^2}$$

Inter granular pressure = $5 \times 9.7 + 2 \times 10.83$

$$= 70.2 \text{ kN/m}^2$$

(5) A clay layer 3.66m thick rests beneath a deposit of submerged sand 7.92m thick. The top of the sand is located 3.05m below the surface of lake. The saturated u.w of sand is 19.62, clay is 18.36 kN/m³.

Compute (a). Total vertical pressure (b) p.w.p. (c) Effective vertical pressure @ mid height of clay layer

Soln:- @ section (1)-(1)

$$\sigma_{11} = \gamma_w \times 3.05 + \gamma_{sand} \times 7.92 + 1.83 \times \gamma_{clay}$$

$$= 9.81 \times 3.05 + 19.62 \times 7.92 + 1.83 \times 18.36$$

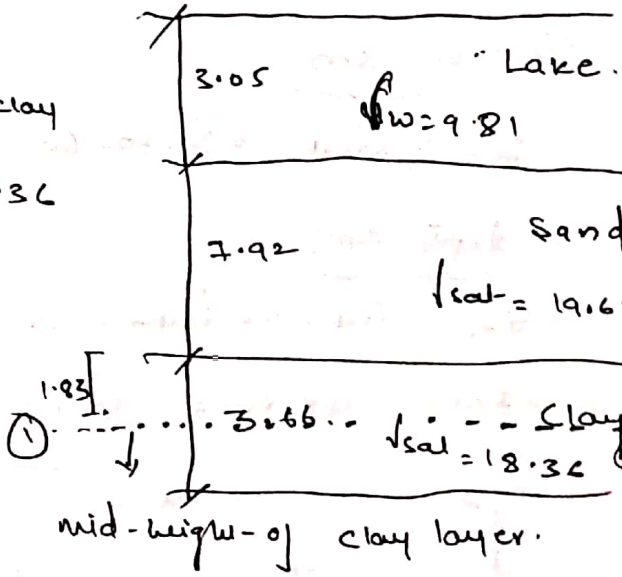
$$\boxed{\sigma_{11} = 218.98 \text{ kN/m}^2}$$

$$u_{11} = 3.05 \times \gamma_w + 7.92 \times \gamma_w + 1.83 \times \gamma_w$$

$$\boxed{u_{11} = 125.568 \text{ kN/m}^2}$$

$$\sigma'_{11} = \sigma_{11} - u_{11}$$

$$\boxed{\sigma'_{11} \Rightarrow 93.35 \text{ kN/m}^2}$$

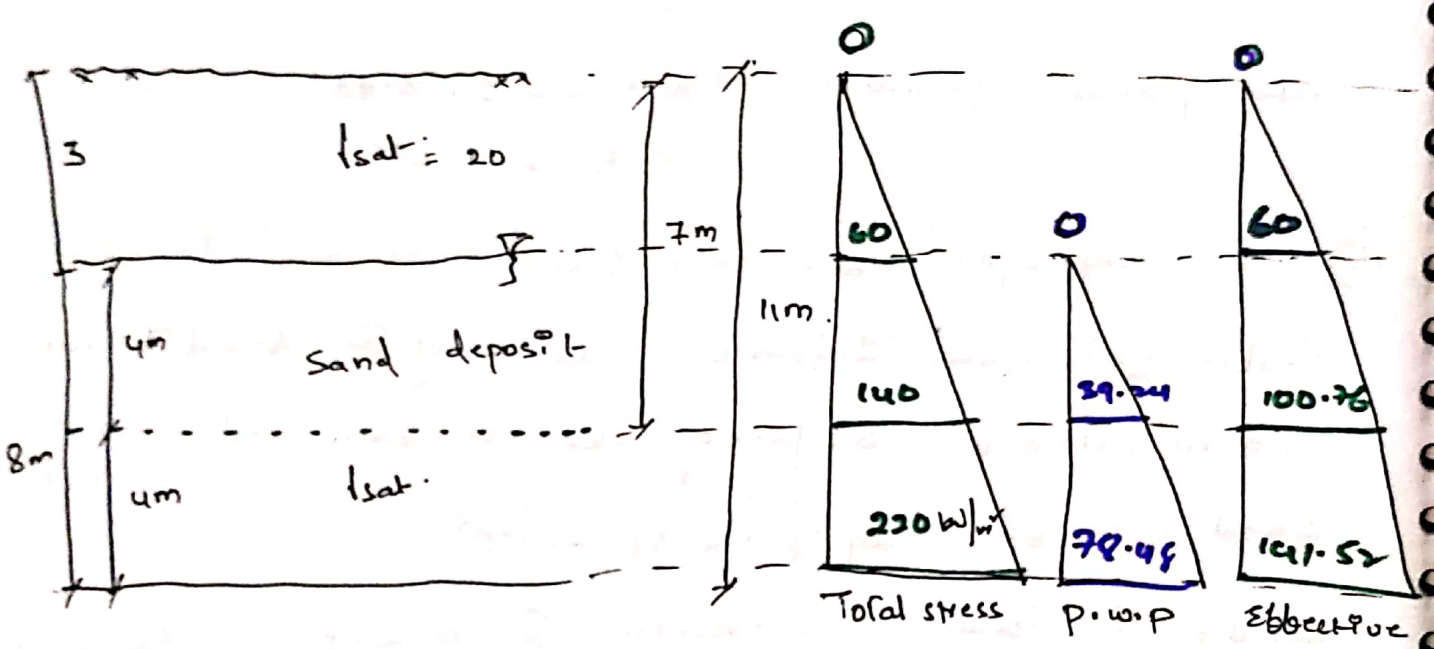


mid-height of clay layer.

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(i) w.t is located @ a depth of 3.0 m below the G.S. in a deposit of sand 11.0m thick. The sand is saturated above w.t. The total u.w of sand is 20 kN/m³. Calculate.

(a) Total pressure (b) p.w.p (c) Effective pressure's @ Depth's 0, 3.0, 7.0 & 11.0 frm G.S & draw pressure distribution diagram. (fig given below).



@ Depth 0m

$\sigma_0 = 0$, $u_0 = 0$ $\boxed{\sigma'_0 = 0}$

@ Depth 3m

$\sigma_{3m} = 3 \times \gamma_{sat}$, $= 3 \times 20 = 60$, $u_{3m} = 0$. $\boxed{\sigma'_{3m} = 60}$

@ depth 7m:-

$\sigma_{7m} = \gamma_{sat} \times 3 + \gamma_{sat} \times 4 \Rightarrow 1(20) = 140$

$u_{7m} = 4 \times \gamma_w = 4 \times 9.81 = 39.24$.

$\sigma'_{7m} = 140 - 39.24 = \boxed{100.76}$

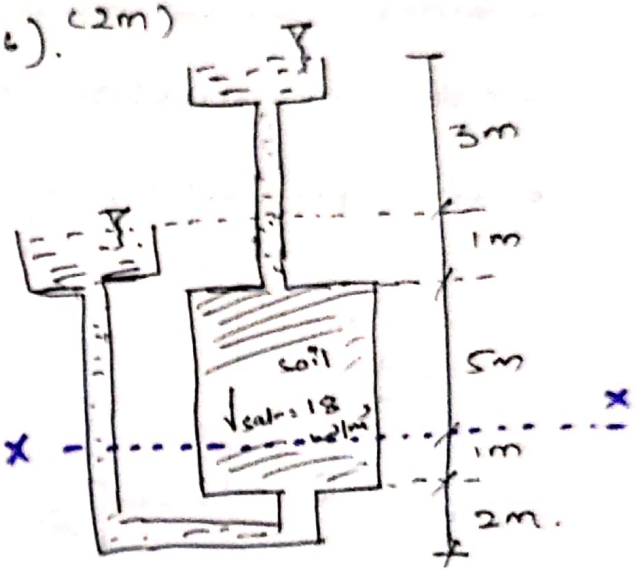
@ depth 11m $\sigma_{11m} \Rightarrow 3 \times 20 + 8 \times 20 = 220$, $u_{11m} = 8 \times 9.81 = 78.48$

$\sigma'_{11m} = \boxed{141.52}$

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Problems on Seepage / flow conditions in soil:-

① A seepage flow condition is shown in fig. The saturated u.w of soil $\gamma_{sat} = 18\ kN/m^3$ using u.w of water $\gamma_w = 9.81\ kN/m^3$. The effective vertical stress (Expressed in kN/m^2) on a plane x-x is _____. (Gate-2016). (2m)



Soln:- from fig it was clear that it is downward seepage condition

given $\gamma_{sat} = 18\ kN/m^3$

$\gamma_w = 9.81\ kN/m^3$.

we have formula for downward condition @ x-x as

$$\sigma'_{xx} = \gamma_{sub} \times z + i z \gamma_w$$

where z = length of soil sample upto section x-x

$z = 5m$

$$i = \frac{h}{L} = \frac{\text{Total head loss}}{\text{Total length of soil sample}} = \frac{3}{6} = 0.5$$

$\gamma_{sub} = (\gamma_{sat} - \gamma_w) \Rightarrow (18 - 9.81) = 8.19$.

So $\sigma'_{xx} = (8.19) \times 5 + (0.5) \times 5 \times 9.81$

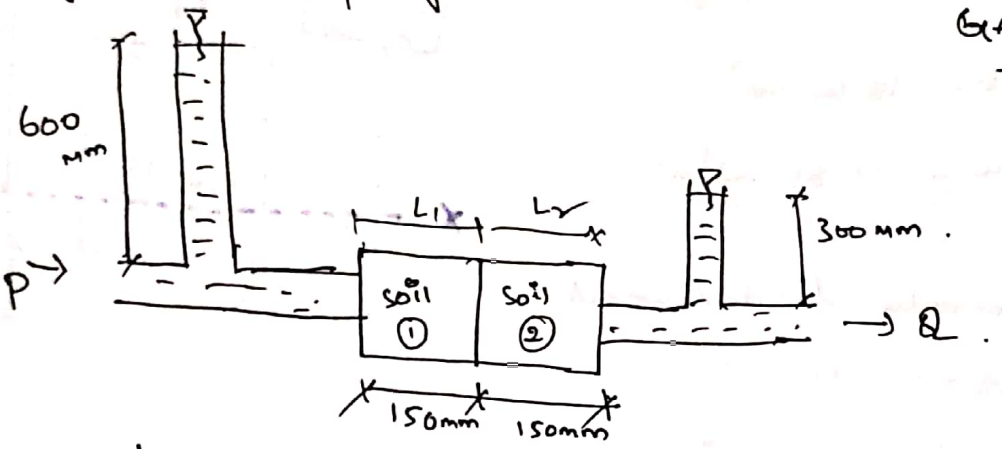
$\sigma'_{xx} = 65.475\ kPa$

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(2) water flows from p to Q through two soil samples, Soil 1 & Soil 2, having cross sectional Area of 80 cm^2 as shown in fig. over a period of 15 min, 200 ml of water was observed to pass through any cross section. The flow conditions can be assumed to be steady state. if the co-eff. of permeability of soil 1 is 0.02 mm/s. The co-eff of permeability of soil 2 in mm/s is _____ ?

GATE - 2016

(2m)



→ ~~here head loss can be 600-300 by see how soil 1 soil 2 samples~~

→ Here ~~vert~~ flow is vertical | normal to soil sample.

⇒ given $Q = 200\ ml \Rightarrow 200 \times 10^{-3}\ L/s \Rightarrow 200 \times 10^{-3} \times 10^{-3}\ m^3$

$$= \frac{200 \times 10^{-3} \times 10^{-3} \times 10^9\ mm^3}{15\ min \times 60\ sec} \quad \frac{}{sec}.$$

$k_1 = 0.02$ mm/s.

$\Delta H = 300$ mm

for soil ① $k_2 = ?$

$\Rightarrow Q = k_1 i_1 A_1$ $\{ Q_1 = Q_2$

$$\frac{200 \times 10^3}{15 \times 60} = 0.02 \times \frac{\Delta h_1}{L_1} \times (80 \times 10^2\ mm^2)$$

$L_1 = 150$

$$\boxed{\Delta h_1 = 208.33\ m}$$

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given total head loss = 300 mm

Head loss for soil ① = 208.33mm

So head loss for soil ② = 300 − 208.33mm.

$$\boxed{\Delta h_2 = 91.67\text{mm}}$$

$$Q = k_2 i_2 A_2$$

$$\frac{200 \times 10^3}{15 \times 60} = k_2 \times \frac{91.67}{150} \times (80 \times 10^2)$$

$$\boxed{k_2 = 0.04545\ \text{mm/s}.}$$

Problem on flow-net & seepage discharge :-

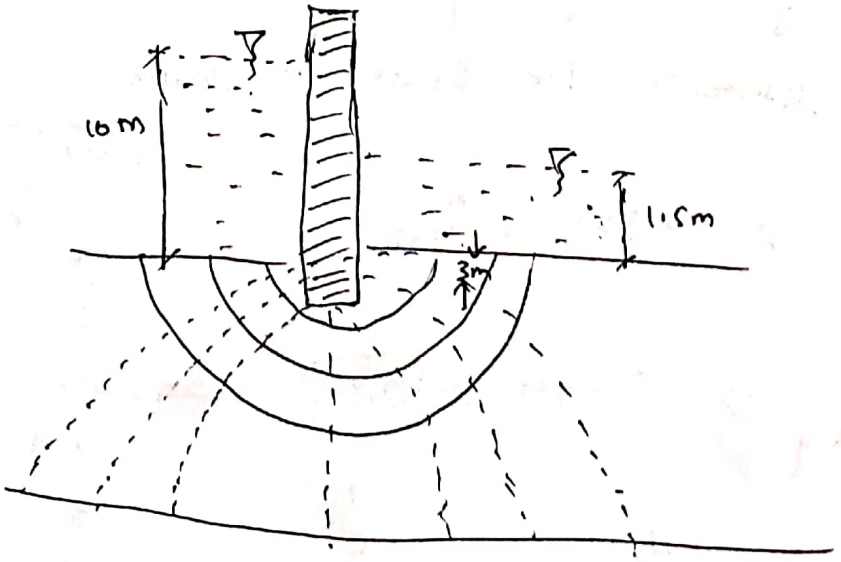
① The flow net around a sheet pile wall is shown in fig. The property of soil are permeability $(k_{0.66}) = 0.09$ m/day (isotropic), $G = 2.70$, void ratio = 0.85. The sheet pile wall and bottom of soil are impermeable. [gate-2012-4m].

a) Seepage loss (in m^3 per day per unit length of wall) of water is

(A) 0.33 (B) 0.38 (C) 0.43 (D) 0.54

b) The F.O.S against the occurence of piping failure is

(A) 3.55 (B) 2.93 (C) 2.60 (D) 0.39.



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To know the Seepage Loss or Discharge we have

$$Q = k \cdot \Delta H \cdot \frac{N_f}{N_d}$$

given $k = 0.09$ m/day.

$\Delta H = 10 - 1.5 \Rightarrow 8.5$m.

N_f = No. of flow channel's = 4 [i.e no. of flow lines − 1]

N_d = No. of Equipotential head drops = [No. of Equipotential lines − 1]

$= 9 - 1 \Rightarrow 8.$

$$Q = 0.09 \times 8.5 \times \frac{4}{8} \Rightarrow \boxed{0.3825}\ m^3/day/\text{meter length of Dam.}$$ (B)

$$(B) \Rightarrow F.O.S = \frac{i_c}{i_{exit}}$$

$$i_c = \frac{G-1}{1+e}$$ given $G = 2.7$, $e = 0.85$

$$i_c = \frac{2.7-1}{1+0.85} = \boxed{0.919}$$

$$i_{exit} \Rightarrow \frac{\Delta H}{L} \Rightarrow \frac{8.5}{?} \Rightarrow \frac{\Delta h}{b} \Rightarrow \frac{\Delta H}{N_d \times b}$$

→ In problem given length of flow field is 3m. & given soil is Isotropic. i.e Square in shape.

$$\Rightarrow \frac{8.5}{8 \times 3} \Rightarrow 0.354 = i_{exit}.$$

→ 3m, 3m

$$i = \frac{\Delta h}{3}$$

$$F.O.S = \frac{0.919}{0.354} \Rightarrow 2.594 \approx \boxed{2.6}$$

$$\Delta h = \frac{\Delta H}{N_d} \Rightarrow i = \frac{\Delta H}{3 \times N_d}$$

Hence Dam is Safe. (C)

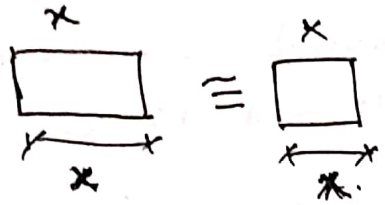
$$= \frac{8.5}{8 \times 3}$$

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Seepage Through an-isotropic soil:-

⇒ Work in isotropic soil is square & in Anisotropic soil it was Rectangle.

⇒ Measurement of seepage Through An-isotropic soil is difficult. So let us assume here also flow field is square. i.e



⇒ So we have discharge formula for isotropic soil as

$$Q = K \cdot \Delta H \cdot \frac{N_f}{N_d}$$

⇒ But in case of isotropic soil's $k_x \neq k_y \neq k_z$.

⇒ So we need to know $k_{eq} = k$.

$$k_{eq} = \sqrt{k_x \cdot k_y}$$

⇒ So the Discharge for an-isotropic soil is given by

$$Q = k_{eq} \cdot \Delta H \cdot \frac{N_f}{N_d}$$

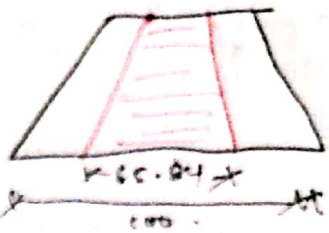
$$\Rightarrow \boxed{Q = \sqrt{k_x \cdot k_y} \cdot \Delta H \cdot \frac{N_f}{N_d}}$$

Problem gate

① Coeff of permeability of a soil in Hz & Vertical directions are 3.46 m/day & 1.5 m/day. Resp. The Base length of concrete Dam resting in this soil is 100m when the flow net is developed for this soil with 1:25 scale factor in vertical direction. The reduced Base length of Dam is — ?

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(A) = 2.65 B) 4.00 C) 6.08 D) 5.43 m.



Given Data; $k_x = 3.46$, $k_y = 1.5$.

Reduced Base length $(X) = \sqrt{\frac{k_y}{k_x}} \cdot x \Rightarrow \sqrt{\frac{1.5}{3.46}} \times 100 \Rightarrow 65.84$ m.

Here scale $\frac{1}{25}$ so Base length $= 65.84 \times \frac{1}{25} = 2.633$ → (4)

(3) a concrete dam (As shown in fig) is constructed across a river over a permeable stratum of soil of limited thickness. The water head's @ up stream & down stream are 16 m, 2m. The flow net constructed under the dam gives $N_f = 4$ & $N_d = 12$. Calculate Seepage loss through Sub-soil, if hydraulic conductivity in Hz. direction is 6×10^{-3} cm/sec & 3×10^{-4} cm/sec in vertical. Calculate Exit gradient if the avg length of the last field is 0.9 m. Assume $e = 0.56$, & $G_s = 2.65$. Det. critical gradient (i_c), comment on the Stability of the floor bed on the Down stream Side.

Soln: Given $h_1 = 16$m, $h_2 = 2$m. $\therefore \Delta H = 16 - 2 = 14$ m.

$N_f = 4$, $N_d = 12$, $k_h = 6 \times 10^{-3}$, $k_v = 3 \times 10^{-4}$

$\therefore Q = \sqrt{k_h \cdot k_v} \times \Delta H \cdot \frac{N_f}{N_d} \Rightarrow \sqrt{(6 \times 10^{-3})(3 \times 10^{-4})} \times 14 \times \frac{4}{12}$

$= 0.626$ cm³/sec.

The head loss per Each potential Drop $= \frac{\Delta H}{N_d} \Rightarrow \frac{14}{12} = 1.17$ m $= \Delta h$

$i_{exit} = \frac{\Delta h}{l} = \frac{1.17}{0.9} = 1.30$.

$i_c = \frac{G-1}{1+e} = \frac{2.65-1}{1+0.56} = 1.06$.

here $i_{exit} > i_c$ Un-Safe [Quick sand condition occurs so provide Sheet pile wall].

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: Compaction :

Def :- The process of increasing the unit wt of soil by forcing the solid's in to a dense state and reducing the air void's.

→ This achieved by applying static or dynamic load to soil

Effect of compaction on soil properties :-

i) Increases shear strength
i.e larger load's may applied to compacted soil's.

ii) Reduces Compressibility :-
i.e under the application of larger load's also there is only smaller settlement.

iii) Reduced permeability :-
i.e reduces the tendency to expand, shrink & liquification

Purpose of Compaction :-

1) Maximum shear strength occures @ minimum void ratio.
2) Larger air voids may lead to settlement of structure during service under working loads.
3) Larger voids if subb may get filled with water which reduces shear strength.
4) ↑ in w.c is also leads to swelling & ↓ in shear strgth of some clay's.

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Proctor's Theory - R.R. Proctor (1930):-

He showed that-

1) There exists a definite relationship b/w soil moisture content & degree of dry density to which soil may compacted.

2) for some amount of compaction energy applied on soil There is one m.c which is termed as O.M.C @ which a particular soil attains Max dry density

→ he proposed tests to determine relationship b/w w.fd (or) e of a compacted soil in a standard manner and to determine the OMC of the soil.

Two Methods

1) Standard proctor Test
2) Modified proctor Test.

Standard proctor Test:-

→ This slope is to determine the relationship b/w w & fd of soils compacted in a mould of given size with 2.5 kg rammer dropped from a height of 305 mm

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Specification's of mould & Rammer:-

→ internal dia = 105 mm.

Height of mould = 116 mm.

mould contain's Detachable collar + base plate.

→ Rammer face ⌀ = 50.8mm, free fall, weight = 2.5 kg

→ height of Drop = 305 mm.

→ No. of layer's = 3

Procedure:-

→ Approximately 5kg soil mixed with known w.c

→ weigh the mould without Baseplate & collar & fix them

→ In the mould, Compact the weighed soil in 3 layers by given 25 blow's for each layer with 2.5 kg Rammer

→ weigh soil + mould.

→ by knowing wt & volume we get bulk unit wt.

→ by repating no. of times

→ Increase the w.c by one or two % points & repeat the above procedure in each increment.

→ continue this Series of determination until there is a decrease in the wet unit wt of compacted soil.

Compactive Energy applied for soil per unit volume.

$$E = \frac{N n W h}{V}$$

N = no. of blws per layer | h = ht of Drop

n = " " layer's | V = vol of mould

W = wt of hammer

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$$E = \frac{25 \times 3 \times 2.5 \times 0.305}{10^3 \times 10^{-6}} = 57187.5 \frac{kg\,m}{m^3} \text{ or } 574 \frac{kJ}{m^3}$$

Dry Density calculations:-

volume of mould = V.

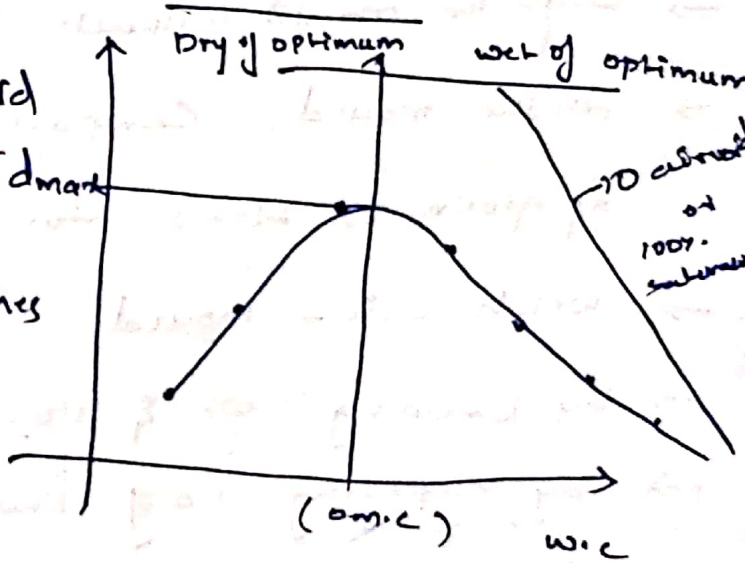
bulk density, $\rho_b = w/V$

dry density $\rho_d = \frac{\rho_b}{1+w}$. $w = w.c$

moisture content - dry unit wt relationship:-

→ w.c w.r.t maximum dry density is referred as optimum moisture content.

→ right-side limb never touches 0 air void's line.



→ Compaction process only decreases the volume of air voids not the volume of solid's.

→ if m.c is less than o.m.c then it is known as Dry of optimum.

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Importance of w.c in compaction:-

→ Initially soil is stiff & difficult to compress so we have low density & high air void's

→ on ↑ in w.c lubricates & soften soil, Reduces surface tension become more workable so we have ↑ w.c & low air void's.

→ on further ↑ in w.c now soil is having combination of air and water so this causes the inhibition the particles coming close so ~~dd~~ density ↓ after crossing o.m.c.

→ This is known as wet optimum

→ To right-side of o.m.c the curve approches Saturation line but never reaches i.e it is never possible to expell air

AASHTO or Modified proctor test:-

→ standard proctor test is not suitable for airway's & highw

→ so US Army crops of Engineer's developed m.p.t which levels of greater compaction's & to produce higher densit

→ Modified proctor test was later adopted by AASHTO & ASTM

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→ specifications:-

→ ⌀ of mould = 105 mm

→ height = 116 mm

→ Rammer face ⌀ = 50.8 mm

wt = 4.5 kg

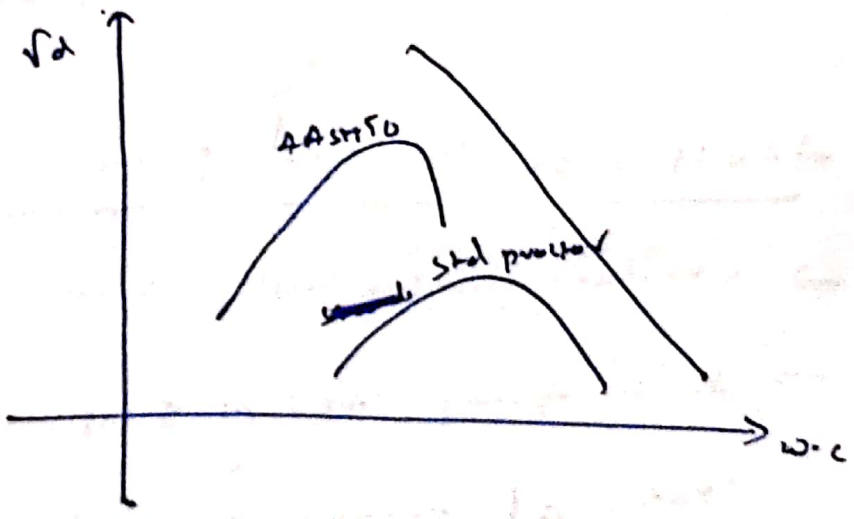
height of Drop = 0.45 m.

No. of blow's = 25 / layer

No. of layer's = 5 layers.

$$E = \frac{N_n W h}{V} = 253125 \text{ kg m/m}^3$$

i.e it is 4.5 Times Extra of std proctor test



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Factors Influencing Compaction:-

→ Moist- content-

→ Soil Type

→ Effect of Compaction
- Effect
  - Nature of Effort
    - Load Duration
    - Area of Contact
  - Amount of Effort

Moisture content:-

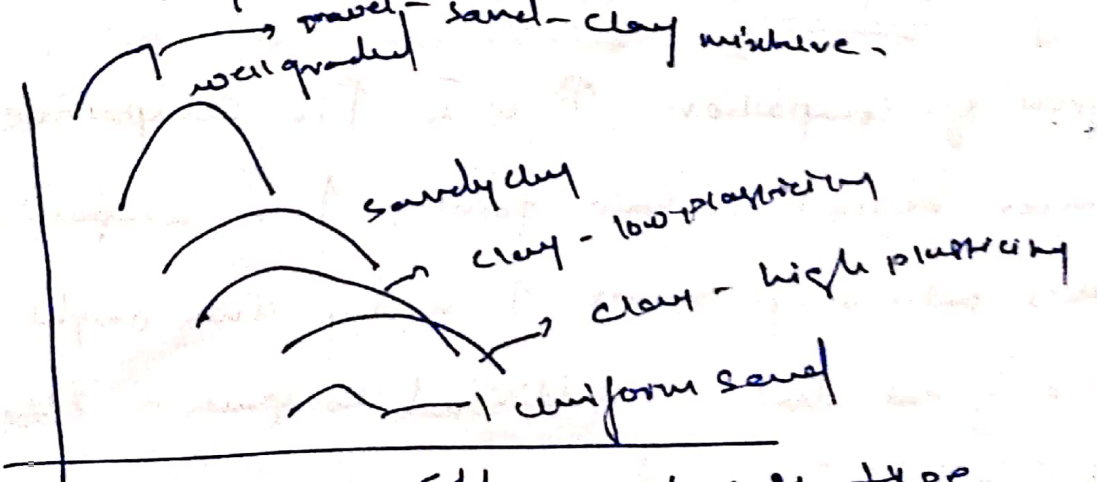
As same as importance of w.c in compaction.

Soil Type:-

→ grain size, shap of soil grain's, amount & type of clay minerals present and the Gs of soil solids, have a great influence on omc & γ_d.

→ In poorly graded sand's γ_d initially ↓ as ↑ in w.c and then ↑ to maximum value with further ↑ in moisture

→ at lower m.c the capillary tension inhibits tendency of the soil particles to move around and can be compacted



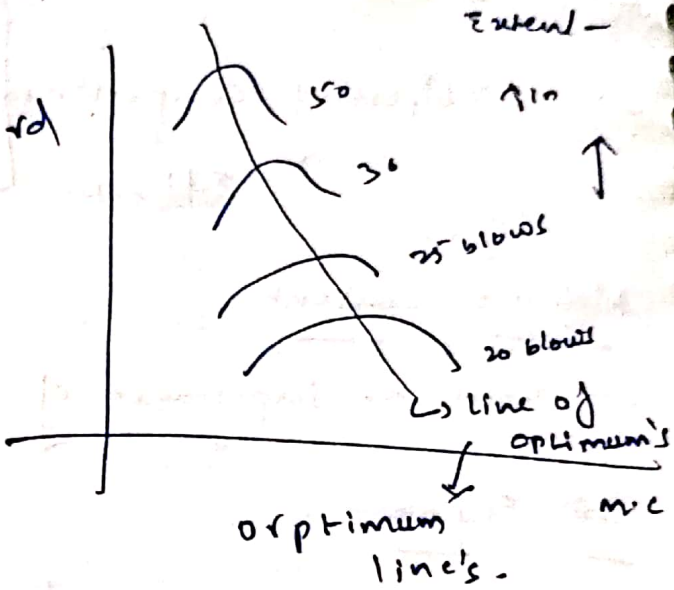
Compaction curve - Effect of soil type.

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Effect of Compactive Effort:-

i) maximum dry u.w ↑ with ↑ in compactive effort

ii) ↑ in compactive effort ↓, the o.m.c to some extent



Nature of Effort – Load duration & Contact area:-

1) Longer time duration load's to Reduced shear strength and greater Compaction.

2) greater Contact area lead's to greater depth of Influence.

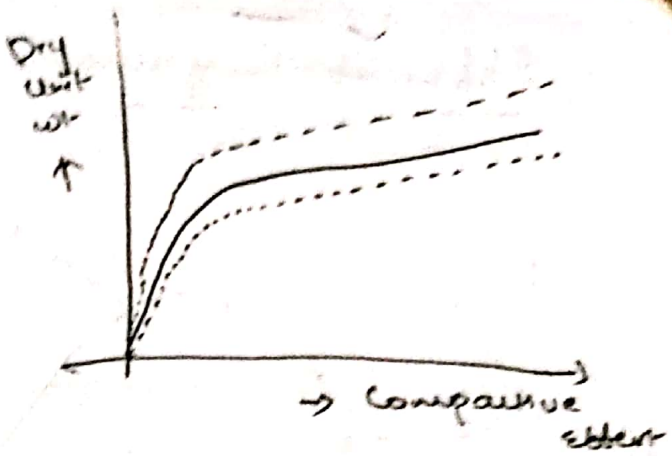
Amount of Compactive Effort:-

→ Degree of Compaction ↑ with ↑ in compactive effort

→ However beyond certain point, ↑ in compactive effort produces only very small ↑ in dry unit weight. i.e it Takes a great deal of additional compactive effort 'E' to see significant ↑ in dry unit weight

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→ This curve shows after a certain point increase in γ_d is very marginal i.e may around (10-20%)



Bulking phenomenon

((Cohessonless soil's) (sandy).

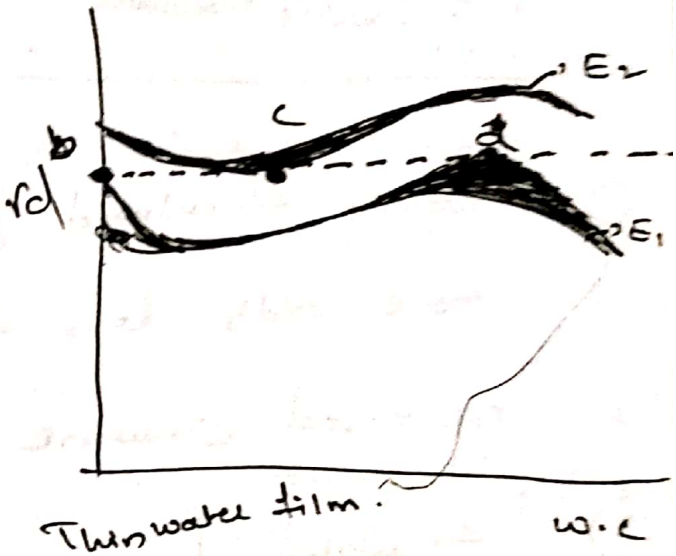
$E_2 > E_1$

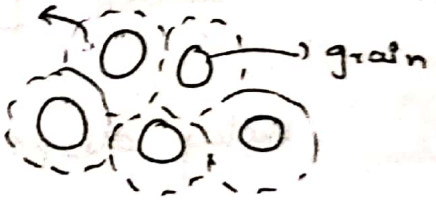
→ in moisture condition for sandy soils all the particles are covered by thin film of water which prevents the particles come closer due to surface tension forces.

b = dry condition

c = moisture condition

a = near (almost) saturation.





→ Surface Tension induces apparent cohesive strength resisting compaction initially so dry density ↓.

→ this is known as Bulking of sand.

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Module-1

Qn. In a specific gravity Test the following Data were

(6) obtained @ the room Temp of 40°C.

wt of pycnometer + water = 720.36 g

wt of pycnometer + solid's + water = 750.46 gms.

wt of soil solid's = 47.50 g.

Determine Sp. gravity of solid's = ?

Empty wt of pycnometer = 680 gms

Soln:- given Data

Let wt of pycnometer + water = W_4 = 720.36 g.

& wt of pycnometer + water + solid's = W_3 = 750.46 gms

Empty wt of pycnometer = W_1 = 680 gm's.

& W_2 = Empy wt of pycnometer + Dry soil

$\Rightarrow$ 680 + 47.50 = 727.5 gms

W.K.T sp. gravity $G_s = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)}$

$G_s = \frac{727.5 - 680}{(720.36 - 680) - (750.46 - 727.5)}$

G_s = 2.78

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Module-3

Q-4 proctor Compaction Test was conducted on a Soil Sample of the following observations were made if V = 950 cc. Det. OMC of MDD.

w.c	7.7	11.5	14.6	17.5	19.5	21.2
wt of soil in (gms)	1700	1890	2030	1990	1960	1920.

Soln:-

⇒ w.k.t OMC is the max w.c @ wich further adding the water will decrease the wt of soil

⇒ So from given Data it was clearly visible that

Max w.c i.e OMC = @ 14.6 % of

wt of soil @ OMC = 2030.

To find out MDD i.e Max. Dry. Density

w.k.t $\gamma_d = \frac{\gamma_b}{1+w}$ ⇒ $\gamma_d = \frac{w/v}{1+w}$

$w = w.c$.

w = wt of soil @ OMC, v = volume.

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So $w = 14.6\%$,

$W = 2030$ gms

$V = 950$ cc.

$$\gamma_d = \frac{2030/950}{1 + 0.146}$$

$$\gamma_d = 1.92 \text{ gm/cc}.$$

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