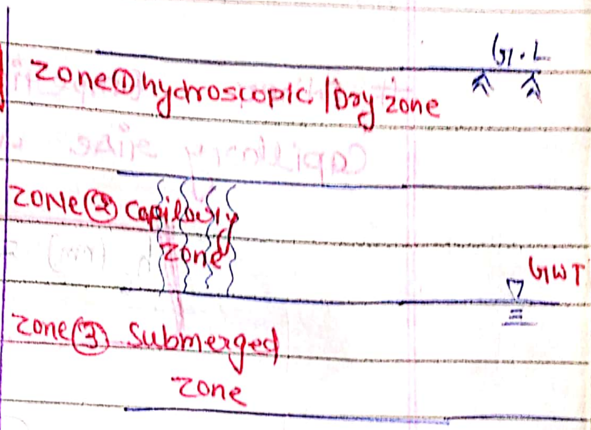
Effective Stress

- Effective stress concept was developed by **Terzaghi**
- Effective stress concept applies to a fully saturated soil.

And Releteds three types of stress
(1) Total Stress (2) Neutral/pore stress (3) Effective Stress

(1) **ZONE-1 :→ Hygroscopic/Dry zone**

- The soil in the zone in the hygroscopic or dry condn hence **Bulk or dry unit weight** should be used

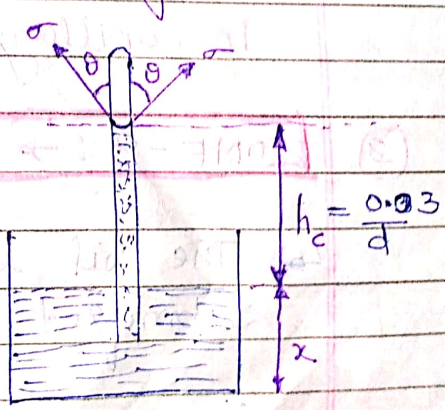


(2) **ZONE2 :→ Capilary zone**

- Soil in the zone is under capilary effect generally soil is saturated but may be partial saturated is given in the question due to capilary effect soil is pulled up above the water table

Height of the capilary rise is given as

$$\boxed{h = \frac{4\sigma\cos\theta}{\rho g d}}$$



σ = Surface tension
d = dia of pipe

$$\boxed{h_c = \frac{0.03}{d}}$$

cm   mm

d = mm
h_c = m

*Capillary water is held above the water table by $\boxed{\text{surface tension}}$

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→ It has been shown that if we take 'd' as $0.2 D_{10}$, we get good result for capillary rise in sand and silts.

for sand and silt

$$\boxed{h_c = \frac{0.03}{0.2 D_{10}}}$$

D_{10} = effective size of the particle in mm.

Other Emperical formula used for capillary rise is as given below.

$$\boxed{h_c\,(cm) = \frac{C}{e D_{10}}}$$

e = void ratio

D_{10} = effective size in $\boxed{cm}$

C = emperical constant = $0.1 - 0.5\ cm^2$

→ Capillary rise is function of $\boxed{\text{pore size}}$.

→ Due to increase in effective stress, shear stress of soil is also increase.

$$\boxed{\sigma \uparrow = \tau \uparrow}$$

→ Due to capillary rise, effective stress in capillary zone increase.

(3) $\boxed{\text{ZONE - 3 :→ Submerged zone}}$

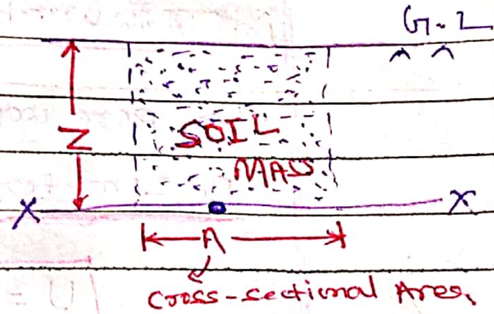
→ The soil ~~is~~ in the zone is under submerged zone.

Total stress :→

↳ It is denoted by σ

↳ Total stress on a plane with in a soil mass is force per unit area of soil mass transmitted in normal direction across a plane.

$$\sigma_{xx} = \frac{\text{wt. of soil}}{\text{cross-sectional Area}}$$



$$\sigma_{xx} = \frac{\text{Vol. of soil} \times \gamma_{soil}}{A}$$

$$\gamma_{soil} = \frac{\text{wt. of soil}}{\text{vol. of soil}}$$

$$\boxed{\text{wt. of soil} = \text{Vol. of soil} \times \gamma_{soil}}$$

vol. of soil

$$\sigma_{xx} = \frac{\cancel{A} \times Z \times \gamma_{soil}}{\cancel{A}}$$

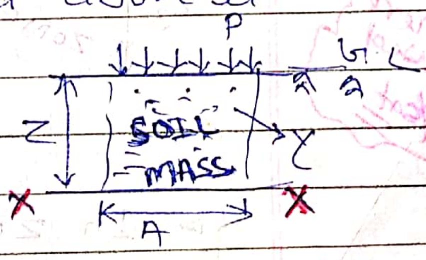
$$\text{Vol.} = A \times \text{length}$$

$$\boxed{\sigma_{xx} = \gamma_{soil} \times Z}$$

- → γ_{bulk} = partial saturated
- → γ_{dry} = dry condn
- → γ_{sat} = saturated condn

↳ It is a stress at any section due to weight of soil and external load above it

$$\boxed{\sigma_{xx} = \frac{P}{A} + \gamma \cdot Z}$$



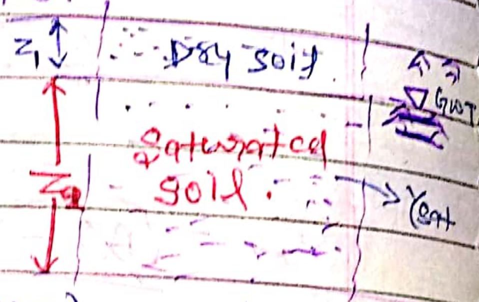
↳ **Total stress** is a physical parameter which can be measured by suitable arrangement such as by $\boxed{\text{Pressure cell}}$

PORE OR NEUTRAL STRESS (U) — Static → Here is used / Flow

- It is denoted by U
- It is the pressure of water filling the void space between solid particles.
- It has no shear component and also measurable quantity
- Pore water pressure is measured using a piezometer or a stand pipe.

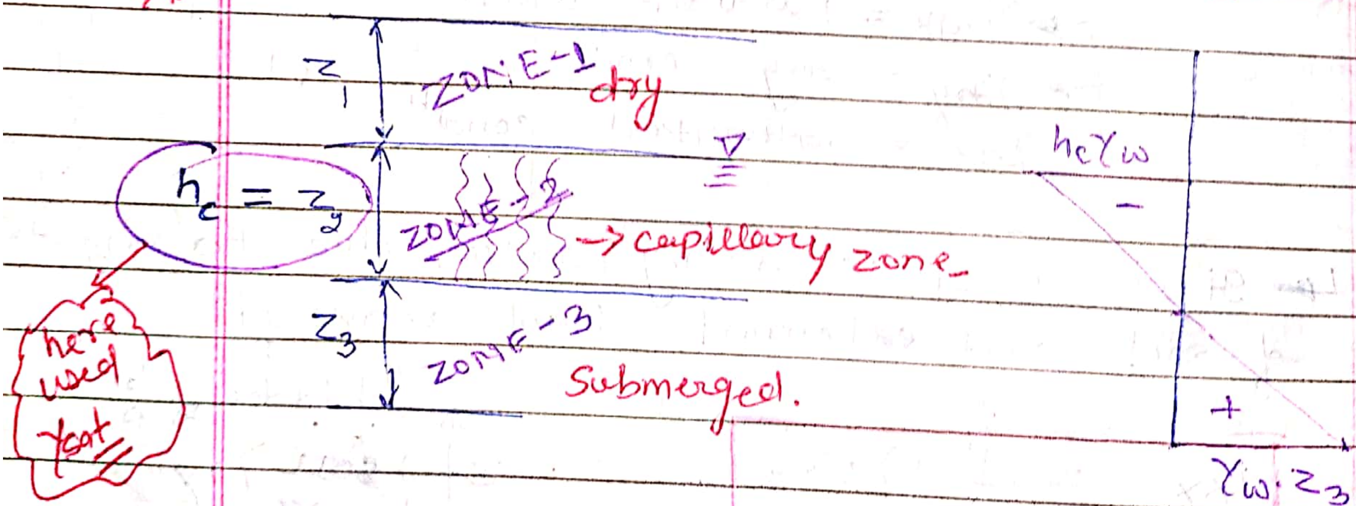
$$\boxed{U = \gamma_w \times z_2}$$

γ_w → unit wt of water

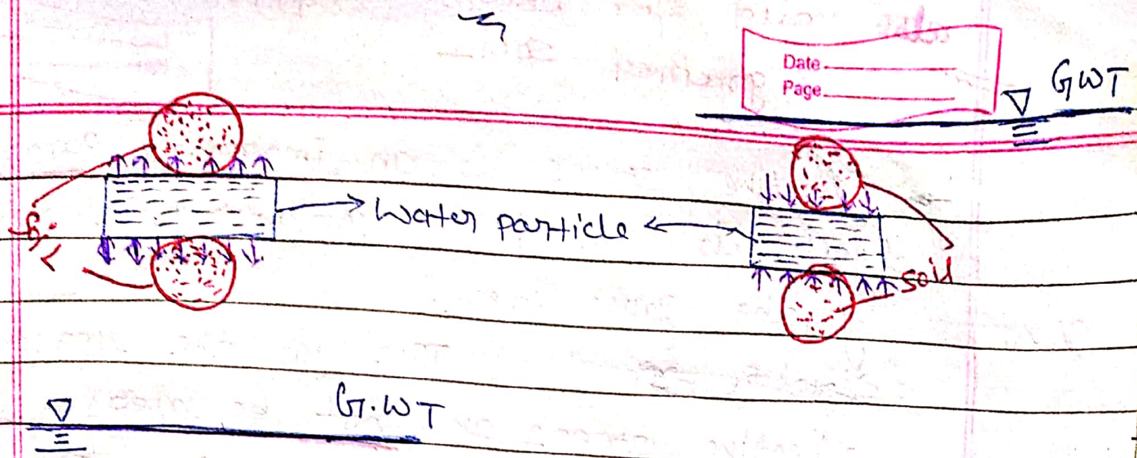


- At Ground water table (GWT) surface Pore pressure is $\boxed{\text{zero}}$
- It is also a $\boxed{\text{Physical parameter}}$

*



- In Zone-1 generally pore water is zero under dry or bulk condn.



Capillary ZONE — Submerged zone

- In capillary zone water is pulled up against gravity hence pore ~~water pressure is~~ tension and Negative
- In submerged zone due to hydrostatic pressure of bottom pore water is in Compression & positive

$U = -ve$ (tension) — $U = +ve$ (compression)

Effective Stress :-

- It is denoted by $\bar{\sigma}$
- Effective stress is the net grain to grain contact pressure which is transmitted from one layer to another.
- ~~Effective~~
- Effective stress is defined as equal to the total stress (σ) minus the neutral stress

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

Capillary ZONE

$\sigma = -7Kg$, $\boxed{u = -2kg}$

$\bar{\sigma} = -7-(-2)$

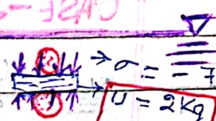
$= -7+2$

$\boxed{= -5kg}$

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

$u \rightarrow$ Negative & tension

Submerged zone.



$\sigma = -7kg$, $\boxed{u = 2kg}$

$\bar{\sigma} = -7-2 = -9kg$

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

$u \rightarrow$ positive & compression

also valid for Coarse & fine grained soil

↳ Effective stress is an imaginary parameter which is sum of the contact force devided by the gross Area.

~~effective st~~ This is the reason why effective stress can not be measured hence it is not physical parameter

NOTE :→ Consolidation, settlement, shear strength, bearing capacity are the function of effective stress and not total stress

CASE – 1 :→ $\boxed{\text{GWT at a Depth } z_1 \text{ from G.L}}$

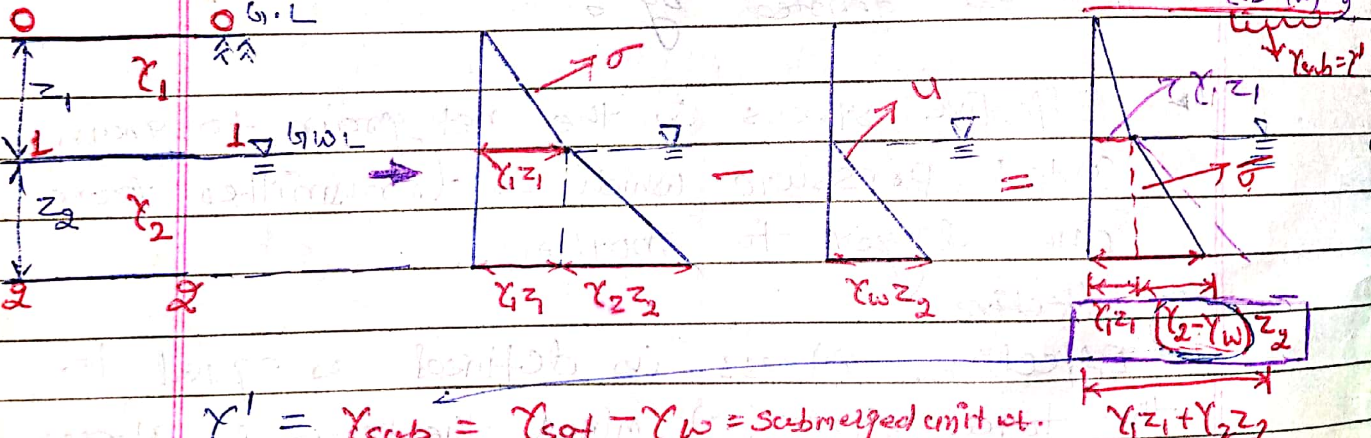
$\sigma_{00} = 0$ $\quad u_{00} = 0$ $\quad \bar{\sigma}_{00} = 0$

$\sigma_{1-1} = \gamma_1 z_1$ $\quad u_{11} = 0$ $\quad \bar{\sigma}_{11} = \gamma_1 z_1$

$\sigma_{2-2} = \gamma_1 z_1 + \gamma_2 z_2$ $\quad u_{22} = \gamma_w z_2$ $\quad \bar{\sigma}_{22} = \gamma_1 z_1 + \gamma_2 z_2 - \gamma_w z_2$

$= \gamma_1 z_1 + (\gamma_2 - \gamma_w) z_2$

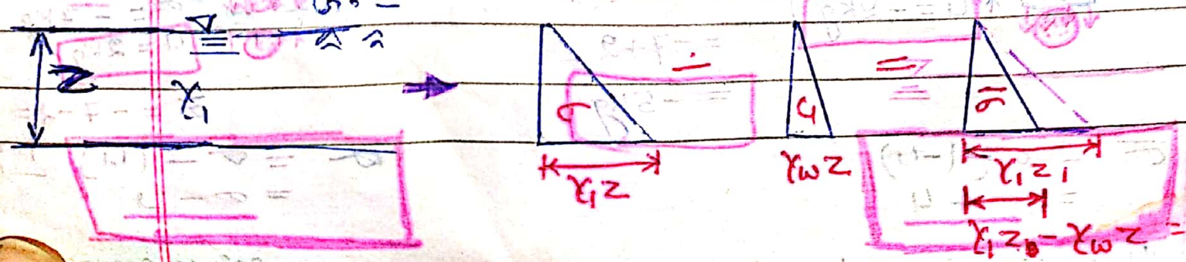
$\gamma_{sub} = \gamma'$



$\gamma' = \gamma_{sub} = \gamma_{sat} - \gamma_w$ = submerged unit wt.

$\gamma_2 - \gamma_w$

CASE – 2 :→ $\boxed{\text{when G.W.T is at G.L}}$



$\gamma_1 z - \gamma_w z = (\gamma_1 - \gamma_w) z$

Soil and water is incompressible

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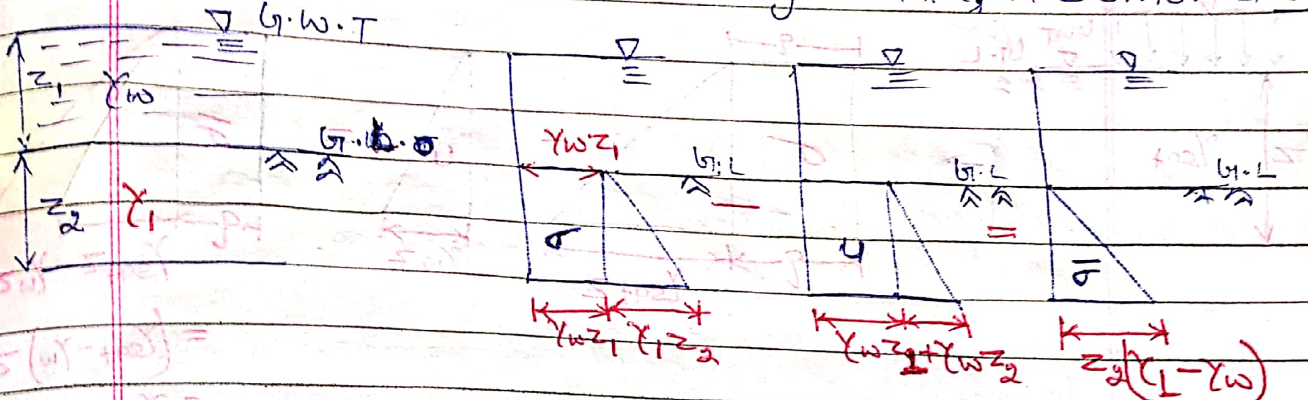
Case → 3 :→ When G.W.T is above G.L

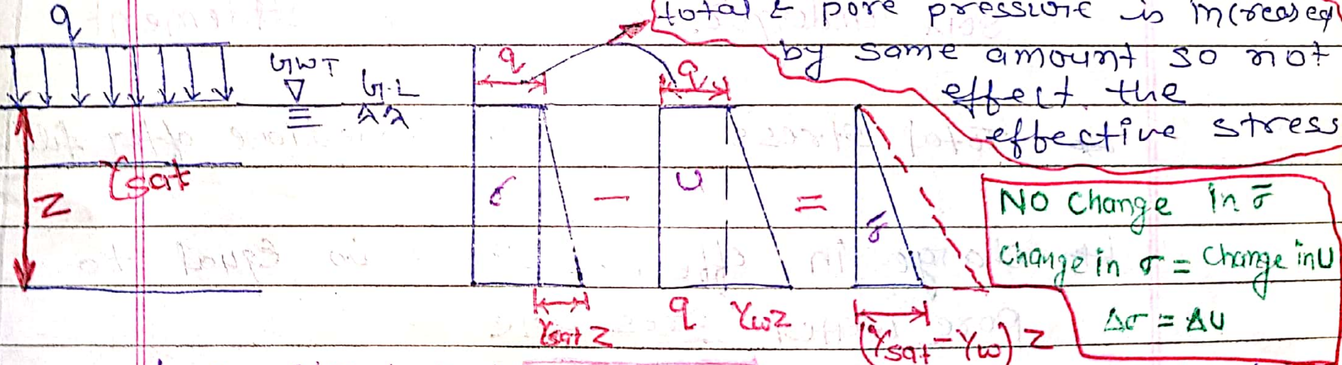
eg:– lake, reservior et.c



NOTE → So we can say when W.T rises above G.L then effective stress below the G.L remain uneffected.

CASE – 4 :→ Effect of Surcharge Appliging

(a) SHORT TERM EFFECT :→ G.W.T at G.L



total & pore pressure is increased by same amount so not effect the effective stress

No change in $\bar{\sigma}$

Change in σ = Change in U

$\Delta\sigma = \Delta U$

↳ In short period due to applied surcharge the total press. and pore press. increased by same amount of the of surcharge hence there is not change in effective stress.

Total pore water press. = static pore water press + Excess pore water pressure

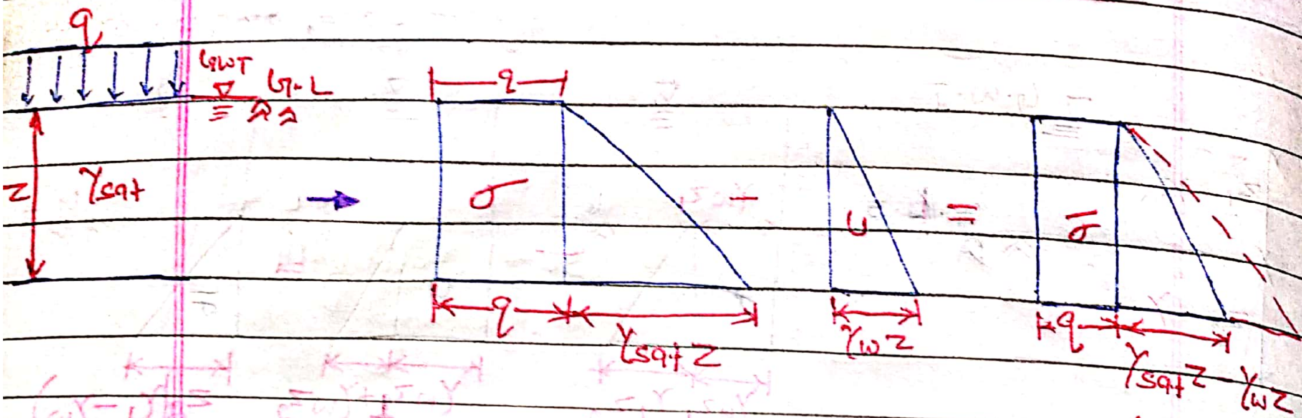
Excess pore water pressure = load

develop due to sudden application of load.

load = E.P.W.P

Only one case of capillary action where effective stress is greater ~~than the total stress~~

(b) LONG TERM EFFECT: → GWT at G.L



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

$= \gamma_{sub} z$

↳ If surcharge continue to ~~act~~ ~~for~~ long time then ~~this~~ excess pore water pressure (q) reduces to zero by ~~following~~ flowing the pore water pressure into surrounding low pore water pressure Area.

↳ Due to this ~~void~~ the volume of void below surcharge reduce, and ~~reduction~~ hence soil undergoes consolidation settlement.

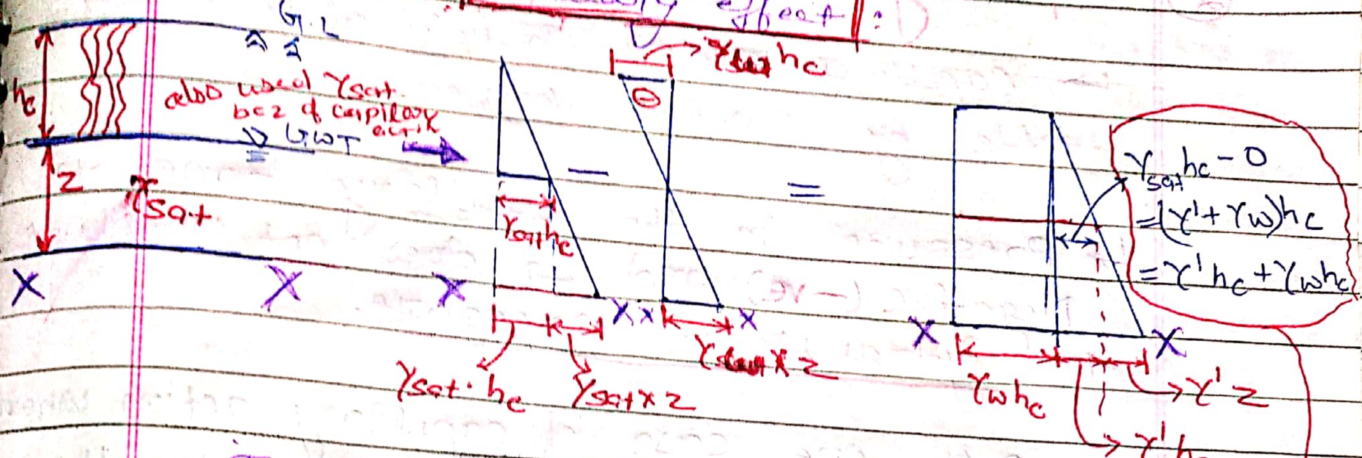
↳ Total stress same as immediate after filling

↳ change in effective stress is Equal to pore water pressure

$$\boxed{\Delta \bar{\sigma} = \Delta u}$$

↳ Reduction of excess pore water pressure as drainage takes place is called **Dissipation**. Dissipation is complete, soil is said to be **Drained condition**

CASE – 5 :→ Capillary effect :



$$\sigma_{xx} = \gamma_{sat} \cdot h_c + \gamma_{sat} \cdot z$$

$$u_{xx} = \gamma_w z$$

$$\bar{\sigma}_{xx} = (\gamma_{sat} \cdot h_c + \gamma_{sat} \cdot z) - (\gamma_w z)$$

$$= \gamma_{sat} \cdot h_c + \gamma_{sat} \cdot z - \gamma_w z$$

$$= \gamma_{sat} \cdot h_c + (\gamma_{sat} - \gamma_w) z$$

$$= \gamma_{sat} \cdot h_c + \gamma' z$$

$$= \gamma' h_c + \gamma_w h_c + \gamma' z$$

$$= \gamma' h_c + \gamma' z + \gamma_w h_c$$

$\gamma_{sat} \cdot h_c - 0$
$= \gamma_{sat} \cdot h_c$
$= (\gamma_w + \gamma') h_c$
$= \gamma_w h_c + \gamma' h_c$

Capillary in soil.

(Ground water can exist in two forms)

① Phreatic or Gravitional water

② Capillary water

↳ Subjected to gravitation force & saturated the void completely

↳ generally called GWT

↳ At GWT surface pressure is atmospheric pressure and below which pressure is hydrostatic

(2) Capillary water :—

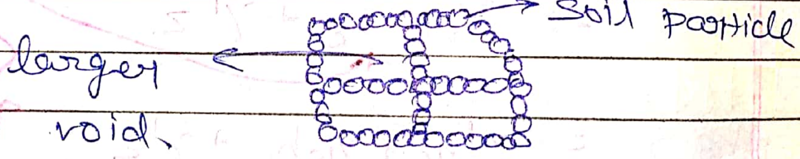
- Capillary water is held above water table by surface tension.

- Pressure in the capillary zone is Negative (−ve) and it does not contribute to hydraustatic pressure.

- Only one case of capillary action where effective stress ($\bar{\sigma}$) is greater than total stress (σ)

- Bulking of sand also occure bcz of capillarity.

capillarity produces apparent cohesion which holds the particles in clusters, enclosing honey combs



Honey combed structure.

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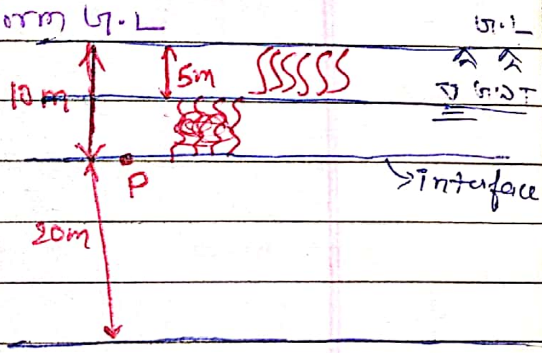
① Q → A 10m thick clay layer is underlain by sand layer of 20m depth the GWT is 5m below surface of clay layer. the Soil above G.W.T is capillary saturated if now water table rises to surface. Then the effective stress at point P on the interface of two soils will

(a) increase by $5\gamma_w$
(b) remain unchange
(c) decrease by $5\gamma_w$
(d) decrease by $10\gamma_w$

Initial condition → GWT at 5m form G.L



$\sigma_{P_i} = \gamma_{sat} \times 10$

$u_{P_i} = -(5\gamma_w)$

$\bar{\sigma}_{P_i} = \gamma_{sat} \times 10 - (-5\gamma_w)$
$= 10\gamma_{sat} + 5\gamma_w$

Final condition: → GWT is rises to surface of G.L

$\sigma_{P_f} = \gamma_{sat} \times 10$

$u_{P_f} = -\gamma_w \times 10$

$\bar{\sigma}_{P_f} = 10\gamma_{sat} - (10\gamma_w)$

Change $= \bar{\sigma}_{P_f} - \bar{\sigma}_{P_i}$

$= (10\gamma_{sat} + 5\gamma_w) - (10\gamma_{sat} + 10\gamma_w)$

$= \cancel{10\gamma_{sat}} + 5\gamma_w - \cancel{10\gamma_{sat}} - 10\gamma_w$

$= -5\gamma_w$

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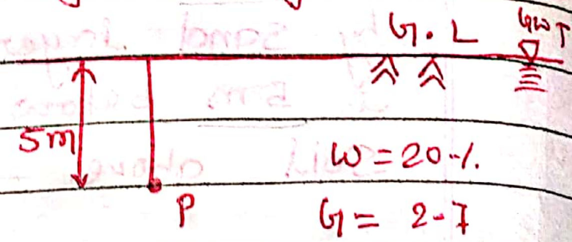
(2) Q→ The ground condⁿ at a sight are given below (!)

$w = 20\% = 0.20$

$G = 2.7$

$\gamma_w = 10 KN/m^3$



$w = 20\%$

$G = 2.7$

$\gamma_w = 10 KN/m^3$

Q - (1) $\gamma_{sat} = ?$

(2) $\sigma, U, \& \bar{\sigma}$ at P

(1) $\gamma_{sat} = \dfrac{(G+e)\gamma_w}{1+e}$ $\qquad \boxed{S \cdot e = wG}$

$= \dfrac{(G + wG)\gamma_w}{1 + wG}$

$= \dfrac{(1+w)G\gamma_w}{1 + wG}$

$= \dfrac{(1+0.20)\, 10 \times 2.7}{1 + 2.7 \times 0.20}$

$= 2.7 \times 7.79 \, KN/m^3$

$= 21.033 \, KN/m^3$

(2) $\sigma_p = \gamma_{sat} \times 5m$

$= 21.033 \times 5$

$= 105.16 \, KN/m^2$

$U_p = \gamma_w \times 5m$

$= 10 \times 5$

$= 50 \, KN/m^3$

$\bar{\sigma}_p = \sigma_p - u_p$

$= 103.16 - 50 = 55.16 \, KN/m^3$

3 Q → A sand layer found at sea floor under 20m water table has relative density 40%, $e_{max} = 1$, $e_{min} = 0.5$, $G_s = 2.67$. Assuming Sp. Gravity of sea water = 1.03 & $\gamma_w = 9.81\,kN/m^3$ for fresh water.

(1) What would be $\bar{\sigma}$ in kPa at 30m depth into the sand layer.

(2) What would be change in $\bar{\sigma}$ at 30m depth into the sand layer if sea water level rizes permanently by 2m.

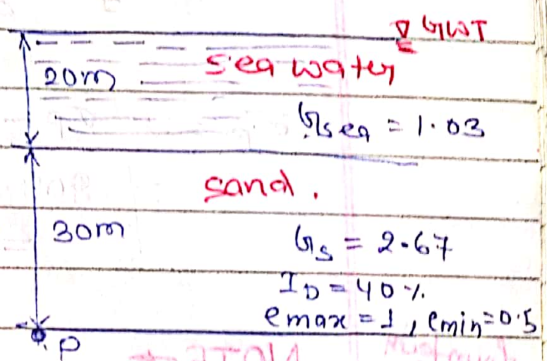
$I_D = 40\%$

$e_{max} = 1$, $e_{min} = 0.5$

$G_s = 2.67$

$G_{sea} = 1.03$

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



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

$$0.40 = \frac{1 - e}{1 - 0.5}$$

$$0.40 \times 0.5 = 1 - e$$

$$e = 1 - 0.2 = \boxed{0.8}$$

$$\boxed{\gamma_{sat} = \frac{(G + e)\gamma_{sea}}{1 + e}}$$

bcz here sea water is fill in the pores of soil so we required γ_{sea}.

$$= \frac{(2.67 + 0.8) \times 10.104}{1 + 0.8}$$

$$\boxed{\gamma_{sat} = 19.47\,kN/m^3}$$

$$G_{sea} = \frac{\gamma_{sea}}{\gamma_w}$$

$$\gamma_{sea} = G_{sea} \cdot \gamma_w$$

$$= 1.03 \times 9.81$$

$$\boxed{\gamma_{sea} = 10.104}$$

(1) $\sigma_p = 20 \times \gamma_{sea} + \gamma_{sat} \times 30$

$= 20 \times 10.104 + 19.47 \times 30$

$=$ [illegible] kN/m^2

$u_p = 50 \times \gamma_{sea}$

$= 50 \times 10.104$

$= 505.2$

$\bar{\sigma}_p = \sigma_p - u_p$

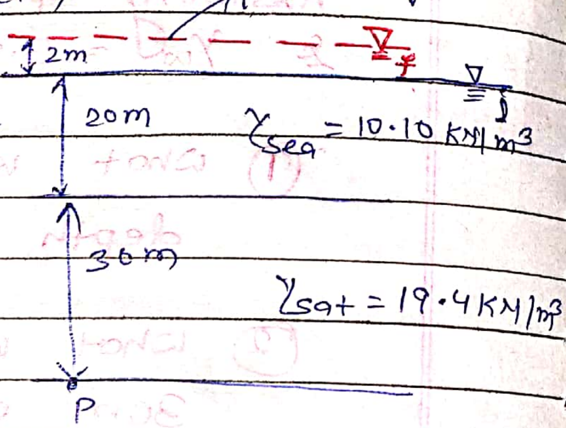
$= 786.14 - 505.2$ [illegible]

$\bar{\sigma}_p = \sigma_p - U_p$

$= 786.18 - 505.2$

$= 280.98 \; KN/m^2$

② When water table permonetly rise 2m.



$\sigma_p = 2 \times \gamma_{sea} + 20\gamma_{sea} + 30\gamma_{sat}$

$= 22\gamma_{sea} + 30\gamma_{sat}$

$= 22 \times 10.104 + 30 \times 19.4$

$= 804.288 \; KN/m^2$

$U_p = 2 \times \gamma_{sea} + 20\gamma_{sea} + 30\gamma_{sea}$

$= 52\gamma_{sea} = 52 \times 10.104$

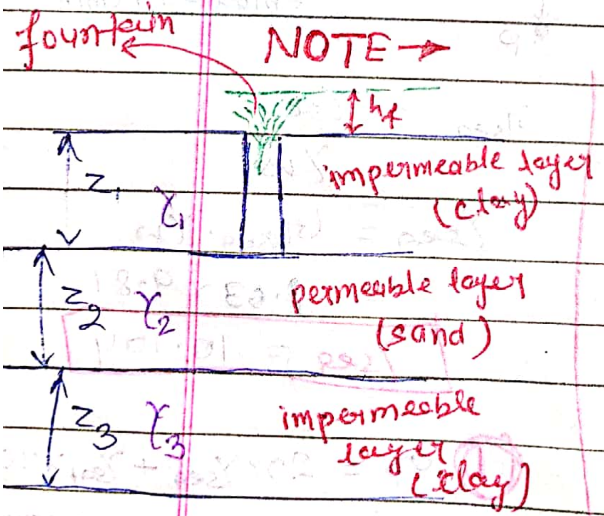
$= 525.408$

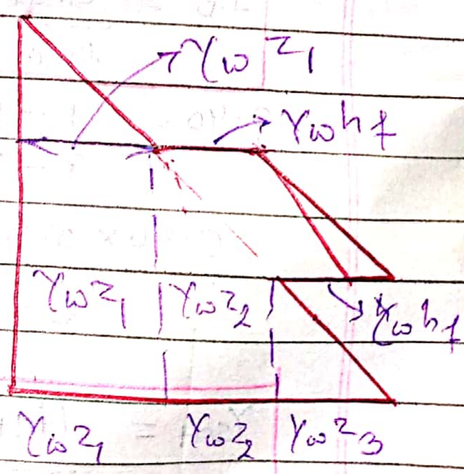
$\bar{\sigma}_p = \sigma_p - U_p$

$= 804.288 - 525.408$

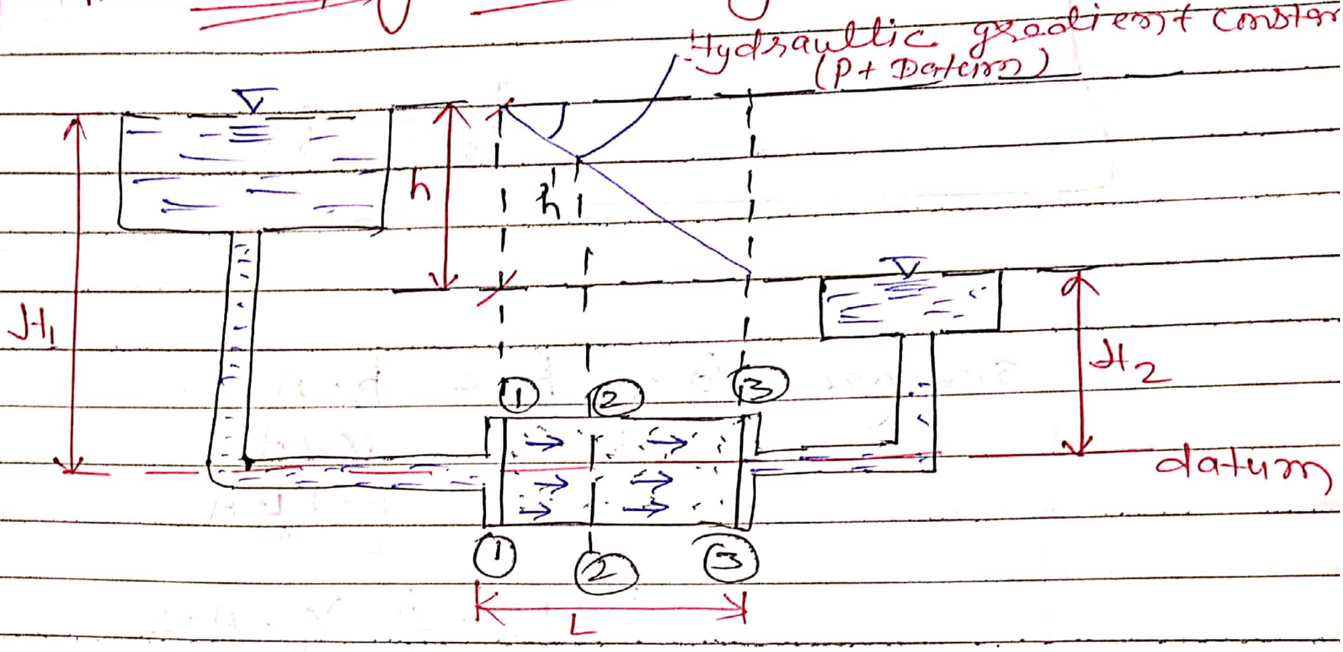
$= 278.88 \; KN/m^2$

NOTE →





* Seepage Analysis *



→ Seepage force is a drag force exerted by on the soil molicules which always acts in the direction of flow. It is due to viscous friction of water on solids and is directly proportional to the hydraulic head of water at that point.

Seepage pressure at ① - ① $= h\gamma_w$

" " at ② - ② $= h'\gamma_w$

" " at ③ - ③ $= 0$

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→ Seepage pressure $(p_s) = h\gamma_w = \frac{h}{L} \times \gamma_w \cdot L$

$$\boxed{p_s = i\gamma_w L}$$

Seepage force $F_s = p_s \cdot A$

$= i\gamma_w L A$

$= \frac{h}{L} \cdot \gamma_w L \cdot A$

$F_s = \frac{h}{L} \cdot \gamma_w \cdot A \cdot L$

$$\boxed{F_s = i\gamma_w \cdot V}$$

$$\boxed{\text{Seepage force/unit volume} = i\gamma_w}$$

* Special Case :-

If ~~The~~ seepage flow in verticle direction then it will increase or decrease the vertical effect stress (grain to grain contact pressure)

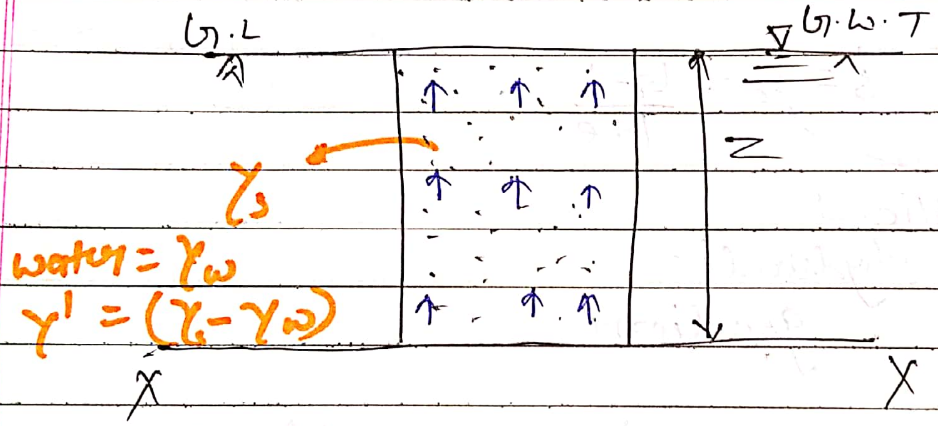
$$\boxed{\begin{array}{c}\text{Net effective verticle pressure} \\ = \bar{\sigma} \pm p_s\end{array}}$$

If flow in down ward

$$\boxed{\bar{\sigma}_{net} = \bar{\sigma} + p_s}$$

If flow in upward

$$\boxed{\bar{\sigma}_{net} = \bar{\sigma} - p_s}$$



If hydraulic head of water available at section XX = h

$$p_{s(xx)} = iz\gamma_w = h\gamma_w$$

$$\bar{\sigma}_{net} = \bar{\sigma}_{xx} - p_s$$

$$= \gamma' z - iz\gamma_w$$

If net effective stress becomes zero

$\bar{\sigma}_{net} = 0$

$$i \cancel{z} \gamma_w = \gamma' \cancel{z}$$

$$i = \frac{\gamma'}{\gamma_w}$$

$$i = \frac{\left(\frac{G-1}{1+e}\right)\cancel{\gamma_w}}{\cancel{\gamma_w}}$$

$$i = i_c = \frac{G-1}{1+e}$$

Critical hydraulic gradient

→ In cohesionless soil such as sand and fine gravels if net effective stress ($\bar{\sigma}_{net}$) b/w particle reduced becomes zero due to upward seepage force, then in such condition the particle will start floating and may flow with water such condition is called Quick sand condition.

FOR QUICK SAND CONDITION :→

Where net grain to grain contact presure becomes zero

→ And such condition occure at hydraulic gradient or above it called Critical hydrallic graident

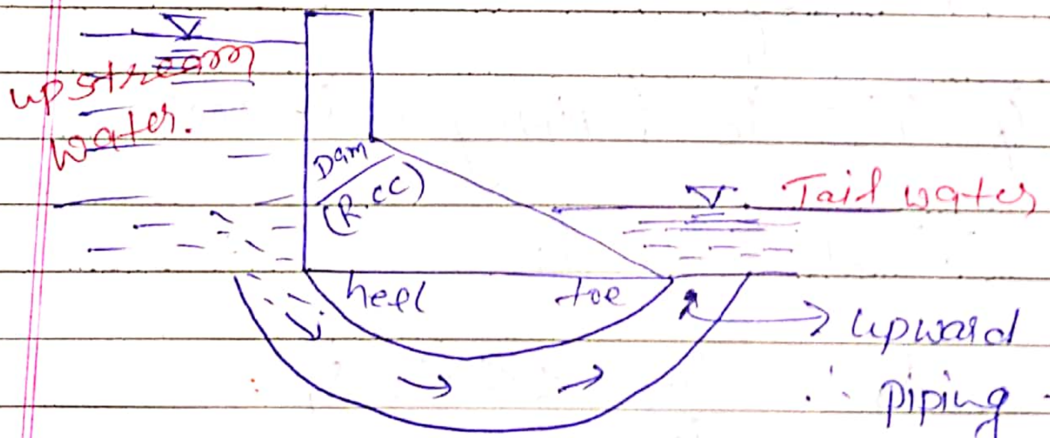
$G_s = 2.6$ to 2.7

$$\boxed{i_c = \frac{G-1}{1+e}}$$

$e = 0.6$ to 0.7

$$= \frac{2.6-1}{1+0.6} = \frac{1.6}{1.6} = 1$$

Quick Sand Sand condition may resut in piping failure below hydraulic structure near the toe.



→ upward seepage
∴ piping failure may occure
if $\boxed{i \geq i_c}$

F.O.S agains piping

$$\boxed{F_c = \frac{i_c}{i}}$$

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* Flow Net *

→ It is a graphical representation of energy flow through the flow filled It is graphical solution of Laplace equation.

For 2D

(a) For Isotropic soil :-

$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0$$ $\nabla^2 \phi = 0$, H = initial head

Square flow net.

(b) For non-isotropic soil

$$K_x \frac{\partial^2 H}{\partial x^2} + K_y \frac{\partial^2 H}{\partial y^2} = 0$$ distorted flow Net

$$\frac{K_x}{K_y} \cdot \frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0$$

$$\boxed{x = x_T \sqrt{\frac{K_x}{K_y}}}$$ → Scale of new flownet.

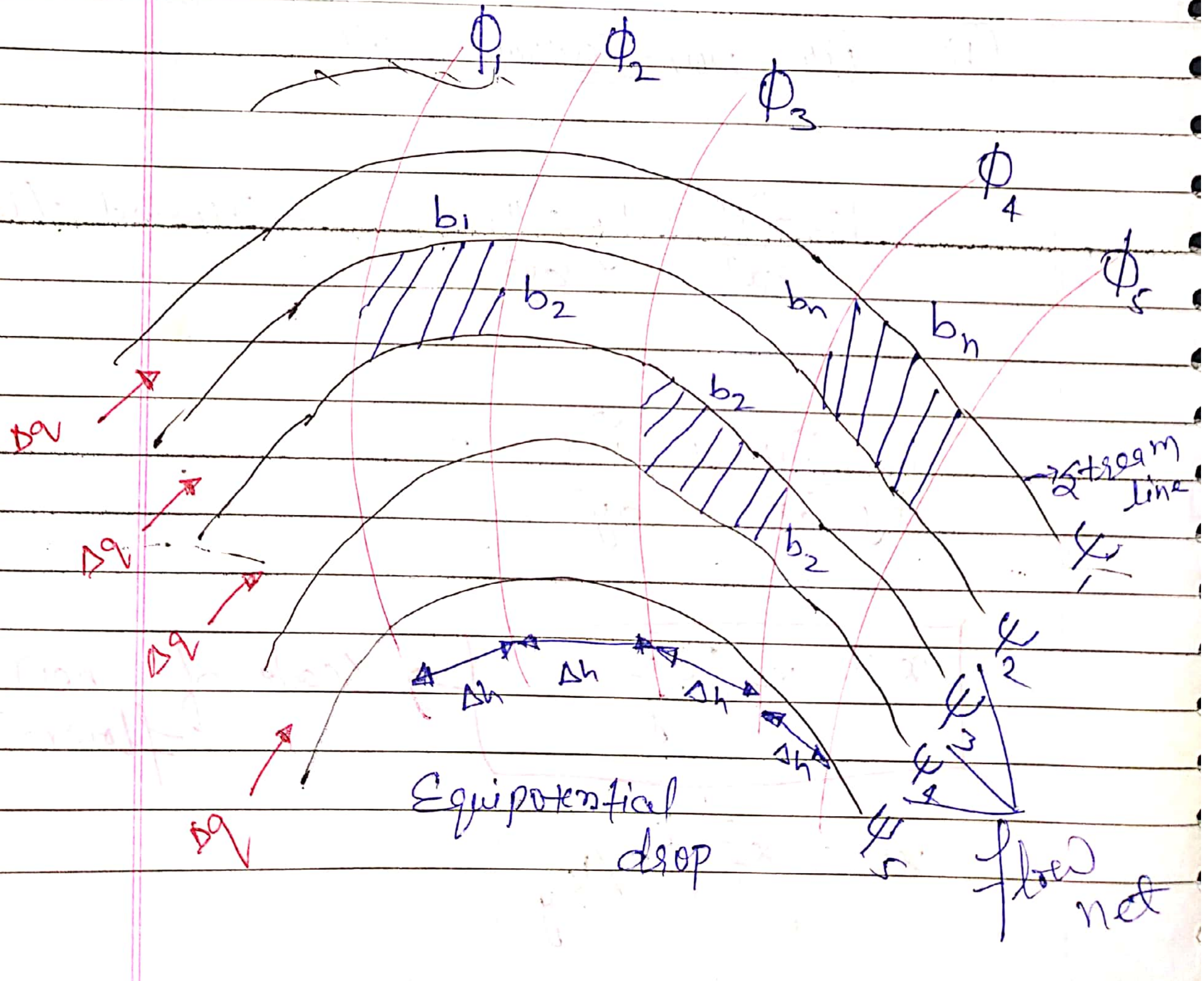
$$x^2 = x_T^2 \left(K_x / K_y \right)$$

$$\frac{K_x}{K_y} \cdot \frac{\partial^2 H}{\partial x_T^2} \times \frac{1}{\frac{K_x}{K_y}} + \frac{\partial^2 H}{\partial y^2} = 0$$

$$\boxed{\frac{\partial^2 H}{\partial x_T^2} + \frac{\partial^2 H}{\partial y^2} = 0}$$

We can say that flow net plotted on Scale x_T & y then it will be a square flownet

* Properties of Flownet :-



ϕ = equipotential function

ψ = stream line function

$\phi_1 > \phi_2 > \phi_3 > \phi_4 > \phi_5$

(1) ϕ line or ψ line cut each other orthogonally (90°)

(11) Two ϕ-line or two ψ line never cross each other

(111) The flow field are square, for homogenious and Isotropic whereas non-Isotropic these are rectangular may be curvilinear or linear.

(4) Head loss through each flow field is equal and is called equipotential drop

Let total head loss b/w U/s & d/s section

$= H_L = H_1 - H_2$

if N_d = No. of equipotential drop.

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

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(5) The discharge through each flow Channel is Δq and remains Constant. that means no component of flow is perpendicular to ψ = line.

discharge per unit width of soil.

$$\boxed{\Delta q = \psi_1 - \psi_2 = \psi_2 - \psi_3 = \psi_3 - \psi_4 = \psi_4 - \psi_5 = \cdots}$$

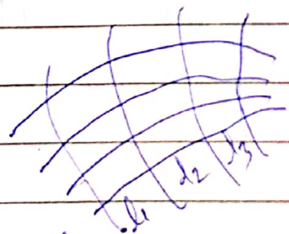
If N_f = No of flow channels in a flow net & Q = total discharge

$$\boxed{\Delta q = \frac{Q}{N_f}}$$

$$\boxed{\Delta q = V_1(b_1 \times L) = V_2(b_2 \times L) = V_n(b_n \times L)}$$

(6) In Case of conversion flow net the size of the last flow field in minimum. therefore its component to check that hydraulic gradient at exit (i.e) should be less than (i_c)

$$\boxed{i_e = \frac{\Delta h}{l_e}}$$



l_e = length of exit flow field

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(7) A given flow net remain same even if the direction of flow is reversed. because flownet depends only on boundary condition not on direction of flow.

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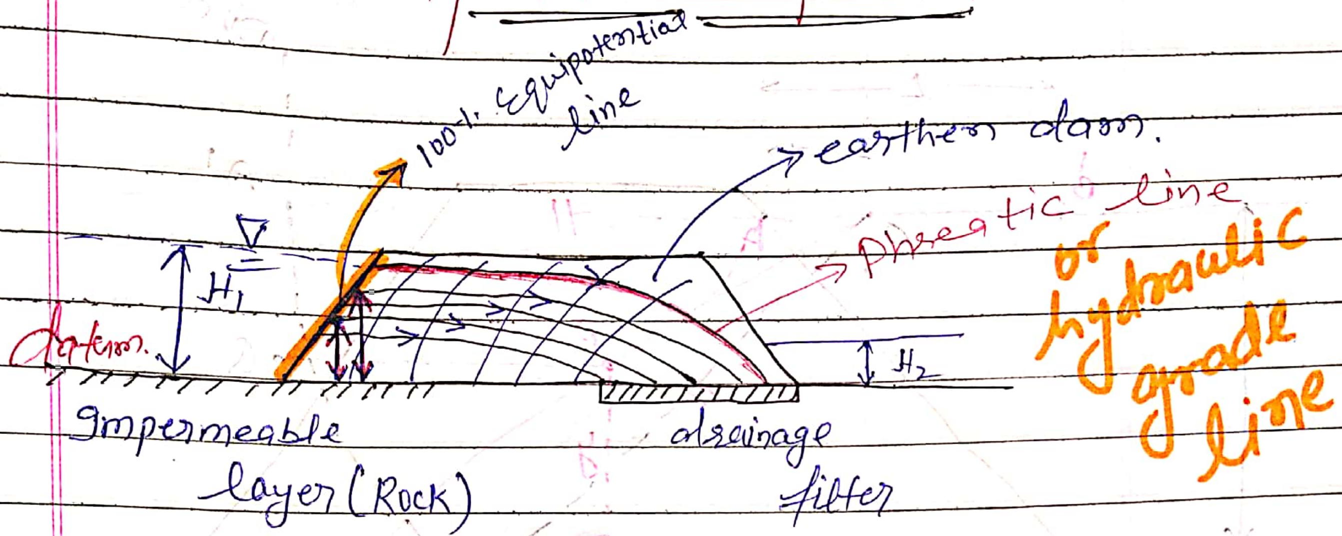
Discharge through earthen dam (draining filter is provided):—



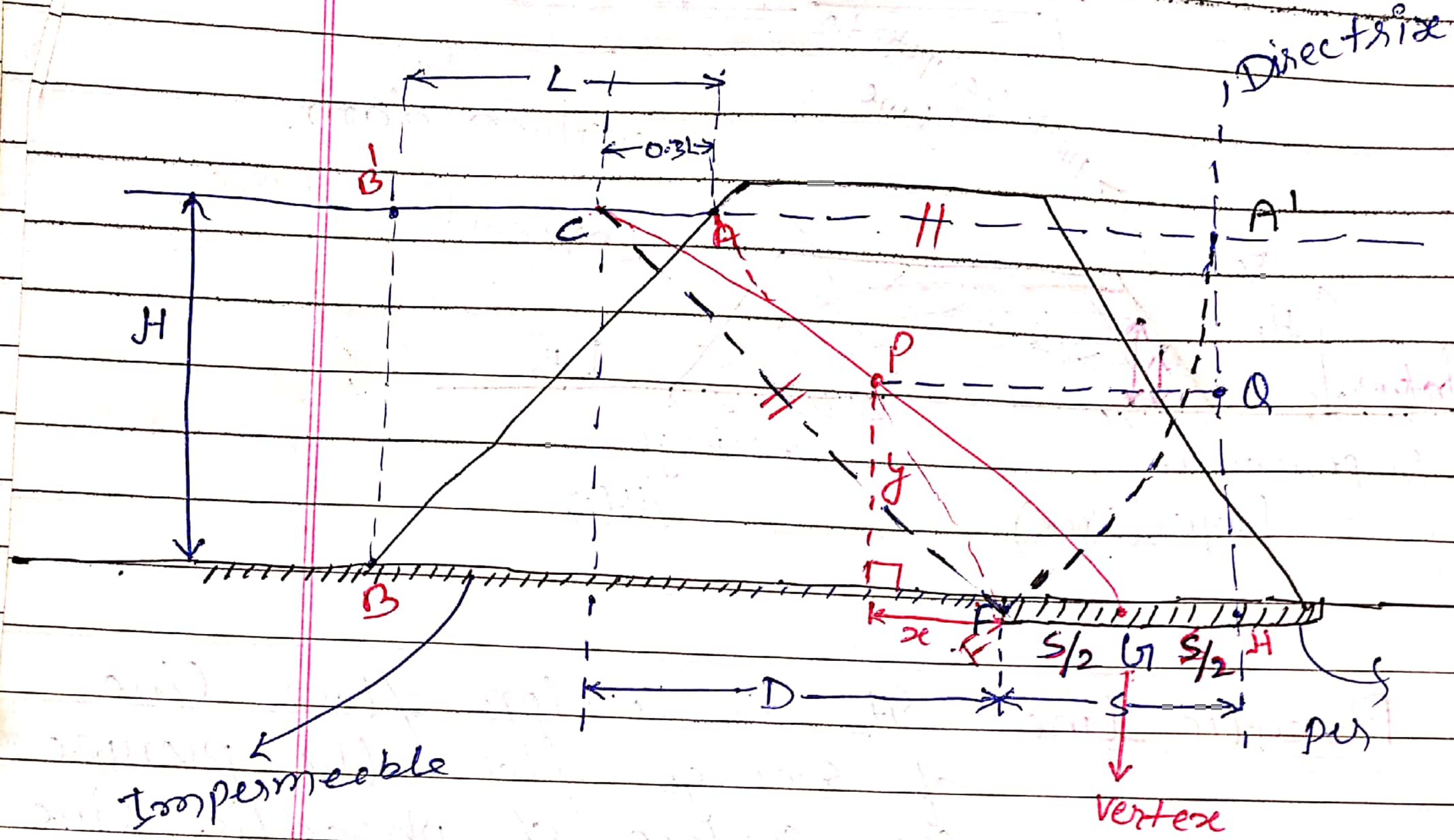
Phreatic line:— It is a top flow line of seepage flow. the pressure on surface of phreatic line is atmospheric. It is also called hydraulic grade line. below phreatic line. seepage flow is under hydrostatic pressure

100% equipotential line:— The upstream wetted surface of earthen dam is 100% equipotential line.

$H_1 - y_1 + y_1 = H_1$

$H_1 - y_2 + y_2 = H_1$

Procedure to draw phreatic line



$\boxed{FG = GH}$

FH = Focal length = S

C = Entery point of base parabola

F = Focus of parabola.

(1) Phreatic line follows a path of a parabola whose focus is the junction of permeable and impermeam surface.

(2) The entery point of parabola is located at point C

on water surface where AC is equal to $\underline{0.3L}$

(iii) To find directrix and arc is draw with radius CF which cuts CA produce at A'

i.e

$$\boxed{CA' = CF}$$

(iv) Draw verticle line through A' to meet filter at H.

A'H is the directrix

FH is focal length

Mark G such that $FG = GH = \frac{S}{2}$

For any point 'p'

$$PF = \sqrt{x^2+y^2} = PQ$$

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

$$\cancel{x^2}+y^2 = S^2+\cancel{x^2}+2Sx$$

$$\boxed{y = \sqrt{S^2+2Sx}}$$ → eqn of parabola (phreatic line)

If D is horizontal distance b/w energy point (C) and focus

$$CF = CA'$$

$$\sqrt{D^2+H^2} = D+S$$

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$$\boxed{S = \sqrt{D^2 + H^2} - D}$$

Seepage discharge through body of Dam per unit width of dam is

$$q = KiA$$

$$q = K \cdot \frac{dy}{dx} (y \times 1)$$

from eqn — ① $y = \sqrt{S^2 + 2Sx}$

$$\frac{dy}{dx} = \frac{1}{2} \frac{2S}{\sqrt{S^2 + 2Sx}}$$

$$q = K \cdot \frac{dy}{dx} \cdot y$$

$$q = K \left(\frac{S}{\sqrt{S^2 + 2xS}} \right) \sqrt{S^2 + 2xS}$$

$$\boxed{q = KS}$$

$$K = \sqrt{K_x \cdot K_y} \quad \text{— 2D}$$

$$K = \sqrt[3]{K_x \cdot K_y \cdot K_z} \quad \text{— 3D}$$

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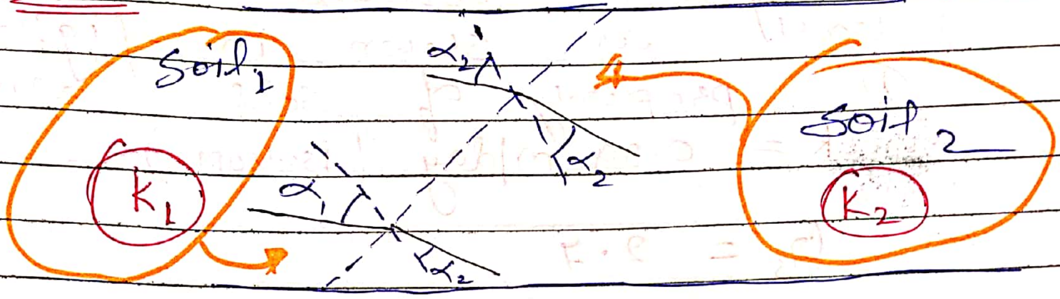
Note :-



$$\boxed{\frac{\tan\alpha_1}{\tan\alpha_2} = \frac{K_1}{K_2}}$$

Q → A ~~Hog~~ Homogenious and Isotropic dam is 20m high and is constructed on a foundation. the K_x 4.8×10^{-8} m/s and $K_y = 1.6 \times 10^{-8}$ m/s water level on up stream is 18m and down stream is dry. The shape factor of flownet is 2/9 estimate the seepage discharge through unit length of dam.

$K_x = 4.8 \times 10^{-8}$ m/s
$K_y = 1.6 \times 10^{-8}$ m/s

$\frac{N_f}{N_d} = \frac{2}{9}$

$H_1 = 18$ m
$H_2 = 0$ m.

$q = \frac{N_f}{N_d} \times \sqrt{K_x \cdot K_y} \times 18$

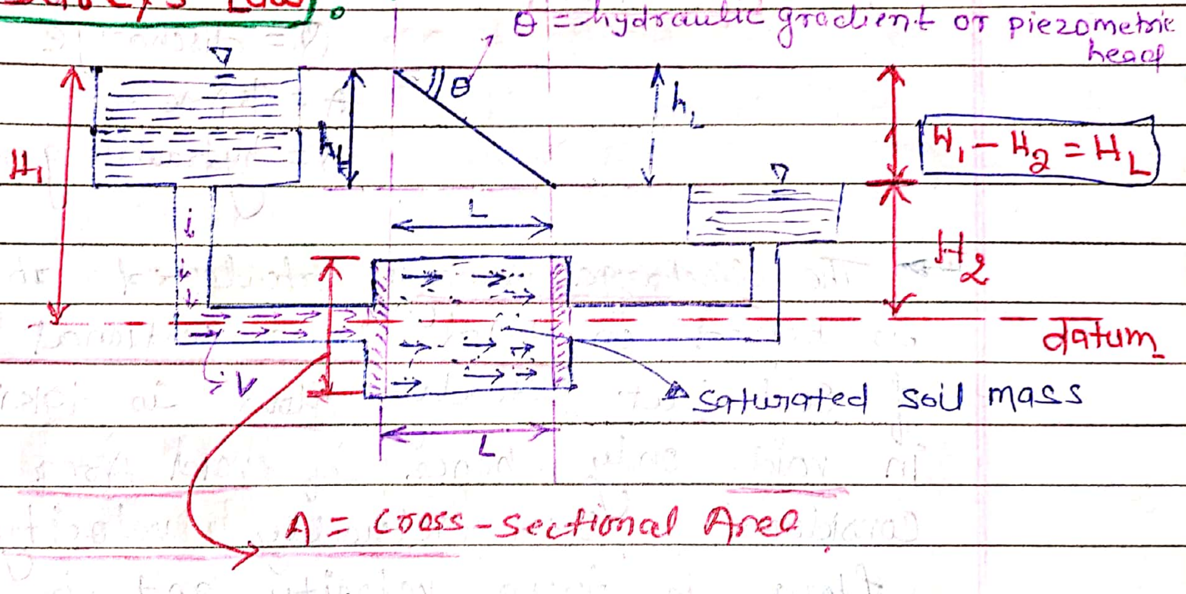
$= \frac{2}{9} \times \sqrt{4.8 \times 10^{-8} \times 1.6 \times 10^{-8}} \times 18$

$= 11.08 \times 10^{-8}$ $m^3/sec/m$.

Permeability

- Knowledge of Permeability is essential in settlement of buildings, yields of wells, Seepage through and below the earth surface.
- It controlls the hydraulic stability of soil masses.
- The Permeability of soils is also required in the design of filters used to prevent piping in hydraulic structures and subgrade drainage.
- rate of Consolidation of compressible soils.

* Darcy's Law :→



Head Loss $(H_L) = H_1 - H_2$

$\tan\theta = \boxed{\frac{h_L}{L} = i} =$ hyraullic grdient line or Piezometric head

A/c to Darcy's Law:

↳ A/c to darcy's law for laminar condition in saturated soil mass the discharge velocity is directly proportional hydraulic gradient

$V_d \propto i$

$$\boxed{V_d = Ki}$$

K = co-efficient of permeability
m/sec or cm/sec.

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

$$\boxed{Q = KiA}$$

$$\boxed{Q = A \cdot V \qquad V = \frac{Q}{A}}$$

Q = discharge
A = c/s Area.
i = hydraulic gradient $= \frac{h_L}{L}$

↳ The discharge velocity calculated above is based on total cross-sectional Area of soil but Actually flow is taking place in void only. hence if void Area is considered then actually velocity of flow is true velocity and is called **Seepage velocity (V_s)**

$$V_s = \frac{Q}{A_v}$$

$$Q = V_s A_v$$

$$A V_d = V_s A_v$$

$$V_s = \frac{A \cdot V_d}{A_v}$$

$$V_s = \frac{A \cdot L\, V_d}{A_v \times L} = \frac{V_d}{V_v / V} = \boxed{\frac{V_d}{n} = V_s}$$

Q = AV

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$\boxed{V_s = \dfrac{V_d}{n}}$ $n \rightarrow$ porosity

↳ $\boxed{n < 1}$, V_s is always greater than V_d

$\boxed{V_s > V_d}$

but always used V_d

Typical values of permeability are as listed in the table.

Soil	Permeability (K)
Gravel	> 1 cm/se
Sand	$1 - 10^{-3}$ cm/sec.
Silt	$10^{-3} - 10^{-7}$ cm/sec.
Clay	$< 10^{-7}$ cm/se.

Co-efficient of percolation (Kp)

$$V_s = K_p\, i$$

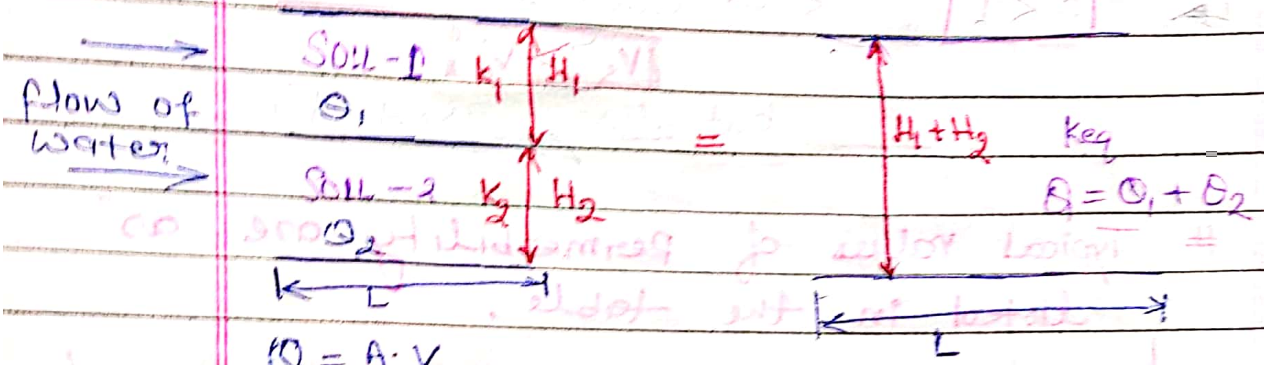
$$\boxed{\frac{V_d}{n} = K_p\, i}$$

$$\frac{K \cancel{i}}{n} = K_p \cancel{i}$$

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

Permeability (K) of stratify soil.

(A) When flow is parrelled to the bed.



$Q = A \cdot V$

$= A K i$

$= A K \frac{h_L}{L}$

$Q_1 = K_1 \frac{h_{L_1}}{L}(H_1)$

$+$

$Q_2 = K_2 \frac{h_{L_2}}{L}(H_2)$

$\boxed{Q = Q_1 + Q_2}$

$Q = K_{eq} \frac{h_L}{L}(H_1 + H_2)$

Now

$Q = Q_1 + Q_2$

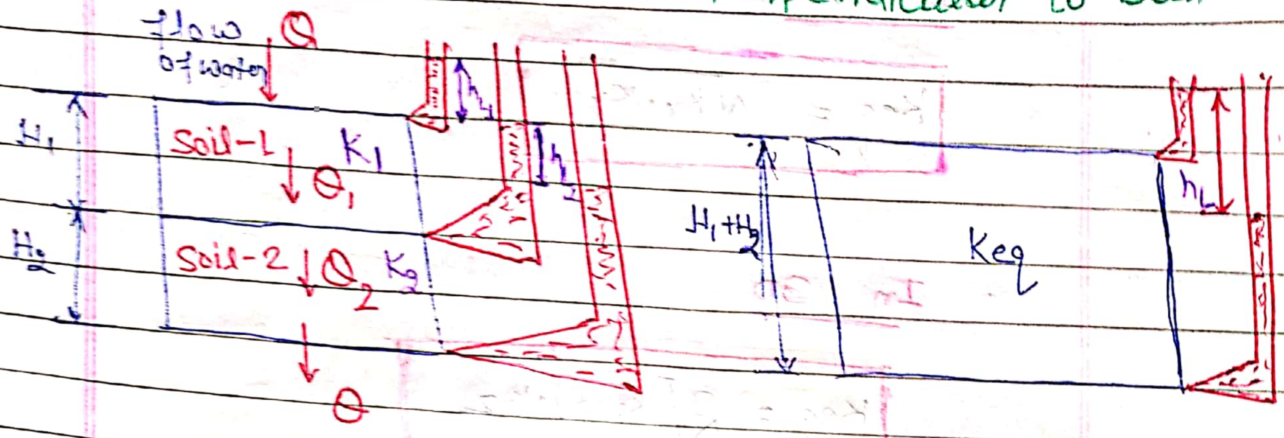
$K_{eq} \frac{\cancel{h_L}}{\cancel{L}}(H_1 + H_2) = K_1 \frac{\cancel{h_{L_1}}}{\cancel{L}}(H_1) + K_2 \frac{\cancel{h_{L_2}}}{\cancel{L}}(H_2)$

$H_L = H_{L_1} = H_{L_2}$

Set the piezometer height at same of soil

$$\boxed{K_{eq} = \frac{K_1 H_1 + K_2 H_2}{H_1 + H_2}}$$

(B) When water flow is perpendicular to Bed.



$$* \boxed{Q_1 = Q_2 = Q} \qquad \boxed{h_L = h_{L_1} + h_{L_2}}$$

$$Q_1 = K_1 i_1 A = K_1 \frac{h_{L_1}}{H_1} \cdot A \Rightarrow h_{L_1} = \frac{Q_1 H_1}{K_1 A}$$

$$Q_2 = K_2 i_2 A = K_2 \frac{h_{L_2}}{H_2} \cdot A \Rightarrow h_{L_2} = \frac{Q_2 H_2}{K_2 A}$$

$$Q = K_{eq} i \cdot A = K_{eq} \frac{h_L}{(H_1+H_2)} \cdot A \Rightarrow h_L = \frac{Q(H_1+H_2)}{K_{eq} A}$$

$$\boxed{h_L = h_{L_1} + h_{L_2}}$$

$$\frac{\cancel{Q_1} H_1}{K_1 \cancel{A}} + \frac{\cancel{Q_2} H_2}{K_2 \cancel{A}} = \frac{\cancel{Q}(H_1+H_2)}{K_{eq} \cancel{A}} \qquad \left\{ Q_1 = Q_2 = Q \right.$$

$$\boxed{K_{eq} = \frac{H_1 + H_2}{\frac{H_1}{K_1} + \frac{H_2}{K_2}}}$$

In 2D

$$\boxed{K_{eq} = \sqrt{K_x \cdot K_y}}$$

In 3D

$$\boxed{K_{eq} = \sqrt[3]{K_x \cdot K_y \cdot K_z}}$$

Factor Affecting Permeability :→

(1) Particle Size :→ If void ratio is same the course soil are more Permeable.

A/c to Allen Hazen :-

Co-efficient of Permeability of soil is proportional to the square of a representative particle size

$K \propto D_{10}^2$

$\boxed{K = C D_{10}^2}$ K = Permeability of soil, cm/sec

D_{10} = effective size of particle, mm

↳ C = Constant ↳ (0.40 – 1.02) if D_{10} in mm

↳ C = 100, if D_{10} in cm

↳ ~~This~~

(2) EFFECT of VOID Ratio :→ If Particle Size is same then loose soil are more permeable than dense soil.

A/c to Tylor :→

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

C = Shape factor.

$\boxed{K = C\frac{e^3}{1+e}}$ $\boxed{K = Ce^2}$

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$$\boxed{\frac{K_1}{K_2} = \frac{C_1 \frac{e_1^3}{1+e_1}}{C_2 \frac{e_2^2}{1+e_2}}}$$

C_1 & C_2 are depends upon placing of soil grain and shape characteristics of pores — shape factor

For sand :→

C changes only slightly with void ratio i.e. ⇒ $C_1 = C_2$

$$\boxed{\frac{K_1}{K_2} = \frac{\frac{e_1^2}{1+e_1}}{\frac{e_2^2}{1+e_2}}}$$

$$\boxed{\frac{K_1}{K_2} = \frac{e_1^2}{e_2^2}}$$

FOR Silt and clay :→

$$\boxed{\frac{\log_{10} K_1}{\log_{10} K_2} = \frac{e_1}{e_2}}$$

(3) Shape Particle :→ Effect of shape of particle is expressed in terms of Specific surface area (S_A)

→ Permeability of soil is inversely proportional to squre of S_A

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

$$\boxed{S_A = \frac{\text{Surface Area}}{\text{Volume}}}$$

eg:–

Sphare :–

$$S_A = \frac{4\pi r^2}{\frac{4}{3}\pi r^3} = \boxed{\frac{3}{r}}$$

=

FOR same VOID RATIO

Rounded → $K\uparrow$, $S_A\downarrow$

Angular → $K\downarrow$, $S_A\uparrow$

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④ **Degree of Saturation :→** Partially Saturated soil the air void present create resistence in the flow of water. Hence less permeability.

$\boxed{K \propto S}$

$\boxed{\begin{array}{l} S_{Full} \rightarrow K \uparrow \\ S_p \rightarrow K \downarrow \end{array}}$

⑤ **Structure of Soil :→**

↳ In stratified soil, permeability is $\boxed{\text{more}}$ in Parrellel to the bed and Permeability is $\boxed{\text{Less}}$ in perpendicular to the bed.

Two clays of same void ratio

→ Flocculated structure (K_1)
→ dispersed structure (K_2)

in both case permeability is differed

$K_1 \neq K_2 \quad (K_2 < K_1)$

K is decreases basically bez of reduction in size of void available for flow.

⑥ **Impurities :→** if Impurities is $\boxed{\text{more}}$ then permeability is $\boxed{\text{decreases}}$

↳ ~~More is impurities~~ ~~less~~

(7) Effect of Permeant (its viscosity and temperature)

→ Permeability of soil is also depends upon the $\boxed{\text{density (or unit weight)}}$ or its $\boxed{\text{viscosity}}$

$\boxed{\rho \text{ or } \gamma}$ $\boxed{\mu}$

$$\boxed{K \propto \frac{\gamma}{\mu}}$$

γ = Unit wt. of fluid (generally water)

μ = viscosity of fluid

$\boxed{\mu \propto \frac{1}{\text{Temp}}}$

so

$\boxed{K \propto \text{Temp}}$

$$\boxed{\frac{K_1}{K_2} = \frac{\gamma_{w_1}}{\mu_1} \Big/ \frac{\gamma_{w_2}}{\mu_2} = \frac{\gamma_{w_1}}{\gamma_{w_2}} \cdot \frac{\mu_2}{\mu_1}}$$

→ K is $\boxed{\text{more}}$ affected by change in viscosity than change in unit weight.

bcz unit weight of water does not change much over large range of temp.

$$\boxed{\frac{K_1}{K_2} = \frac{\mu_2}{\mu_1}}$$

→ $\boxed{\text{Greater}}$ ~~the~~ is the viscosity $\boxed{\text{lower}}$ is the permeability

- A/c to IS: 2720 Part 17 :→ Permeability of soil shall be expressed at $\boxed{27°C}$

$$\boxed{K_{27} = \frac{\mu_T}{\mu_{27}} K_T}$$

K_{27} = value of K at 27°C

K_T = " " " " T°C

μ_{27} = viscosity of Permeant at 27°C

μ_T = " " " " T°C

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⑧ Effect of effective Stress :→

↳ As effective stress increases, void ratio decrease and consequently permeability decrease.

$$\boxed{\bar{\sigma} \uparrow \rightarrow e \downarrow \rightarrow K \downarrow}$$

⑨ Adsorbed water :→ more is absorption capacity less is the permeability.

NOTE →

Sand ← $\boxed{K > 10^{-3}}$ → determined by constant head method.

$\boxed{10^{-7} < K < 10^{-3}}$ → determined by falling head.

$\boxed{K < 10^{-7}}$ → determined by consolidation formula

Coefficient of absolute permeability / specific permeability or intrinsic permeability : — K_o

↳ This parameter is independent of property of fluid and depends only on soil property.

$$\boxed{K_o = K \cdot \frac{\mu}{\gamma_w}}$$

K_o In $\boxed{m^2}$ or $\boxed{cm^2}$ or $\boxed{darcy}$

$1 \text{ Darcy} = 0.987 \times 10^{-8} \text{ cm}^2$

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Determination of co-efficient of Permeability

→ LABORATORY Method

→ Constant-head Permeability test :-

→ variable/falling head Permeability test

→ capillary - permeability test.

→ FIELD METHOD

→ Pumping - out test (large area required)

→ pumping - out test (Small area required)

→ INDIRECT ~~TEST~~ METHOD

→ Computation from the particle size and its specific surface (used when particle size known)

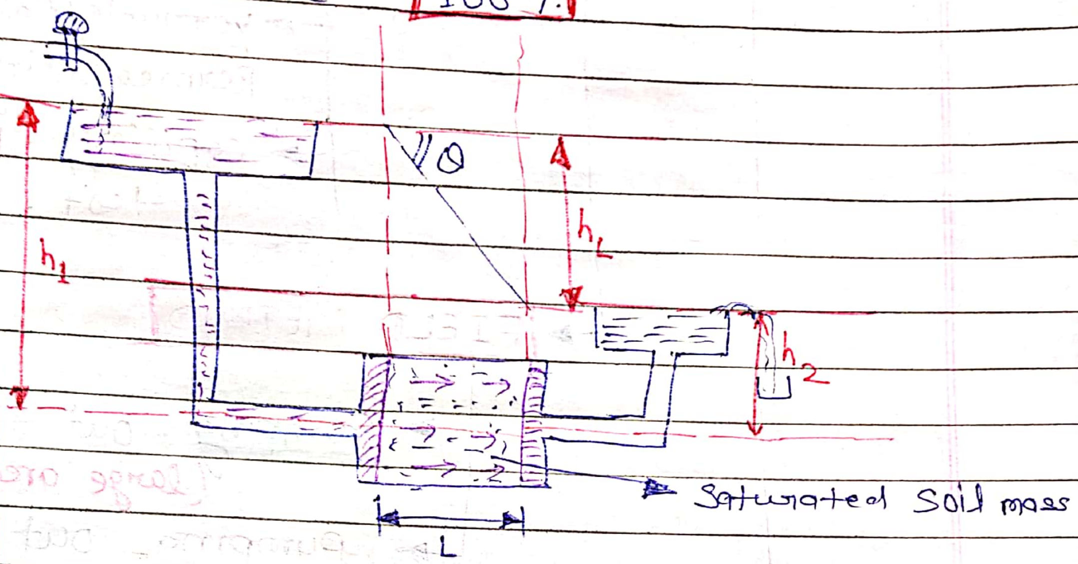
→ Computation from the consolidation test data. (used when co-efficient of volume change (m_v) has been determined by consolidation test on the soil.

LABORATORY METHODS

* CONSTANT HEAD PERMEABILITY TEST :-

↳ This test is generally used for Course soil i.e Sand.

↳ Degree of saturation of soil should be 100 %.



↳ Let

Quantity of collected in graduated vessel in Time t is Q

Hence

~~Discharge(q) = Quanti~~

$$\boxed{\text{Discharge } (q) = \frac{\text{Quantity } (Q)}{\text{Time } (t)}}$$

q = discharge

A = c/s Area

L = length of flow

h_L = difference in monometer

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

q also know

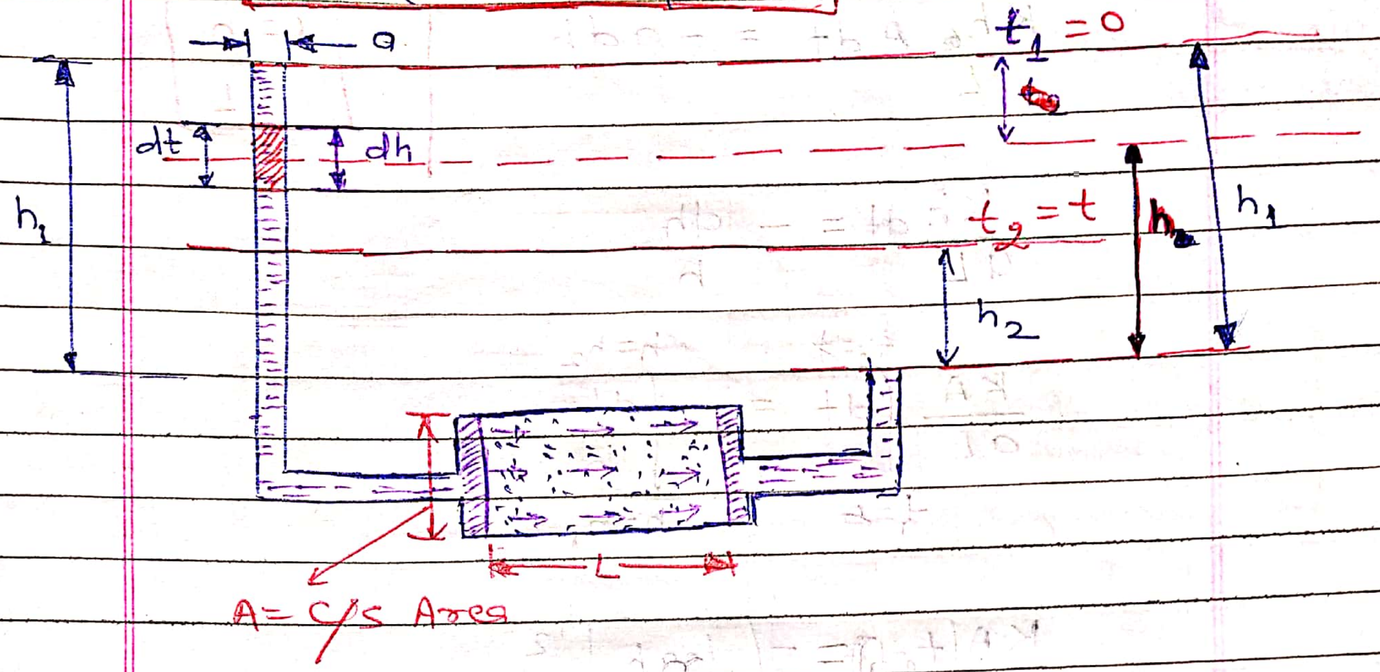
$$q = K \cdot i \cdot A$$

$$\frac{Q}{t} = K \frac{h_L}{L} A \Rightarrow \boxed{K = \frac{QL}{h_L A t}} \Rightarrow \frac{qL}{h_L A}$$

$\frac{Q}{t}$

FALLING HEAD PERMEABILITY TEST →

↳ This ~~test~~ is generally used for $\boxed{\text{fine soil}}$ $\boxed{\text{undisturbed specimen}}$



Let

time $\boxed{t_1 = 0}$ → Head difference b/w up & down stream level be $\boxed{h_1}$

time $\boxed{t_2 = t}$ → Head differene b/w up & down stream level be $\boxed{h_2}$

Let,

At any intermediate stage the head difference $\boxed{h}$ and it fall by $\boxed{dh}$ in $\boxed{dt}$ time.

Now

volume of water fall in time interval dt

Q = discharge

$$\boxed{V = Q \times dt} \qquad \left[Q = \frac{\text{Volume}}{\text{time}} \right.$$

Volume of water pass through soil. at dh.

$$\boxed{V = -a \times dh}$$

By continuity

$$Q \times dt = -a \times dh$$

$$KiA\,dt = -a \times dh$$

$$\left\{ \begin{array}{l} V = Ki \\ \dfrac{Q}{A} = Ki \\ Q = KiA \end{array} \right.$$

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$$K i A\,dt = -a\,dh$$

$$\frac{K h}{L} A\,dt = -a\,dh \qquad \boxed{i = \frac{h}{L}}$$

$$\frac{KA}{aL} dt = -\frac{dh}{h}$$

$$\frac{KA}{aL}\int_{t_1=0}^{t_2=t} dt = -\int_{h=h_1}^{h=h_2} \frac{dh}{h}$$

$$\frac{KA}{aL}(t_2-t_1) = -\left[\log h\right]_{h_1}^{h_2}$$

$$\frac{KA}{aL}(t_2-t_1) = -\left[\log h_2 - \log h_1\right]$$

$$\frac{KA}{aL}(t_2-t_1) = \ln h_1 - \ln h_2$$

$$\frac{KA}{aL}(t_2-t_1) = \ln \frac{h_1}{h_2}$$

$$\boxed{K = \frac{aL}{A(t_2-t_1)} \ln \frac{h_1}{h_2}}$$

$$\boxed{K = \frac{2.303\,aL}{A(t_2-t_1)} \log_{10} \frac{h_1}{h_2}}$$

NOTE → In above test conducted in two stages. i.e in first stage head fall from h_1 to h_2 in t time and its second stage heads falls from h_2 to h_3 in same time t then.

$$\boxed{K = \frac{2.303\,aL}{At}\log_{10}\frac{h_1}{h_2} = \frac{2.303\,aL}{At}\log_{10}\frac{h_2}{h_3}}$$

$$\log_{10}\frac{h_1}{h_2} = \log_{10}\frac{h_2}{h_3}$$

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

$$h_2^2 = h_1 h_3$$

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

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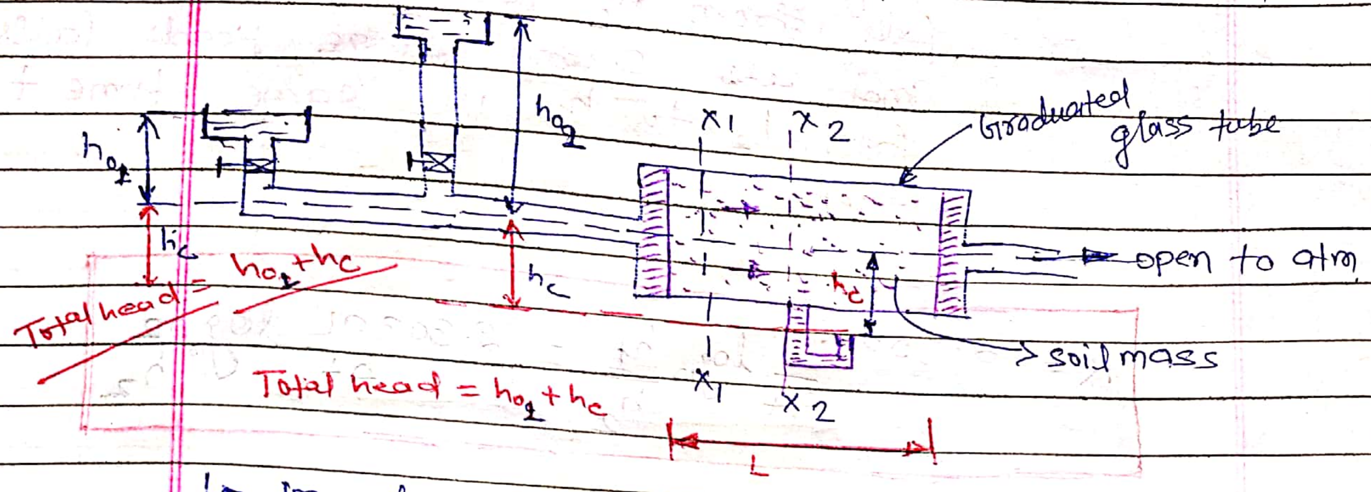
Capillary Permeability test :-

- This test is used for unsaturated soil.



- In first stage smaller head h_{o_1} is connected to soil mass and water moves from x_1 to x_2 in time t_1 to t_2

$$\boxed{\frac{x_2^2 - x_1^2}{t_2 - t_1} = \frac{2K}{S \cdot n}\left(h_{o_1} + h_c\right)}$$

- In 2nd stage larger head h_{o_2} is connected to soil mass let water surface moves from x'_1 to x'_2 in the time t'_1 to t'_2

$$\boxed{\frac{x_2'^2 - x_1'^2}{t'_2 - t'_1} = \frac{2K}{S \cdot n}\left(h_{o_2} + h_c\right)}$$

K = Co-efficient of permeability

S = Degree of saturation

n = porosity

h_c = capillary head.

FIELD TEST METHODS

Pumping out test :→

(a) Unconfined aquifer :→

(1) Well is penetrated upto the bottom of aquifer such that comes only from side of the well not from the bottom.

(2) Study state seepage is stablized towards the wells.

this result in the water table being downw drawdown to fall a cone of depression.

(3) The hydraulic gradient at distance from the centre of well is assumed to be curved.

* **Drawdown :→** Drawdown at some distance r from the centre of well is Equal to fall in water level from original ground water level.

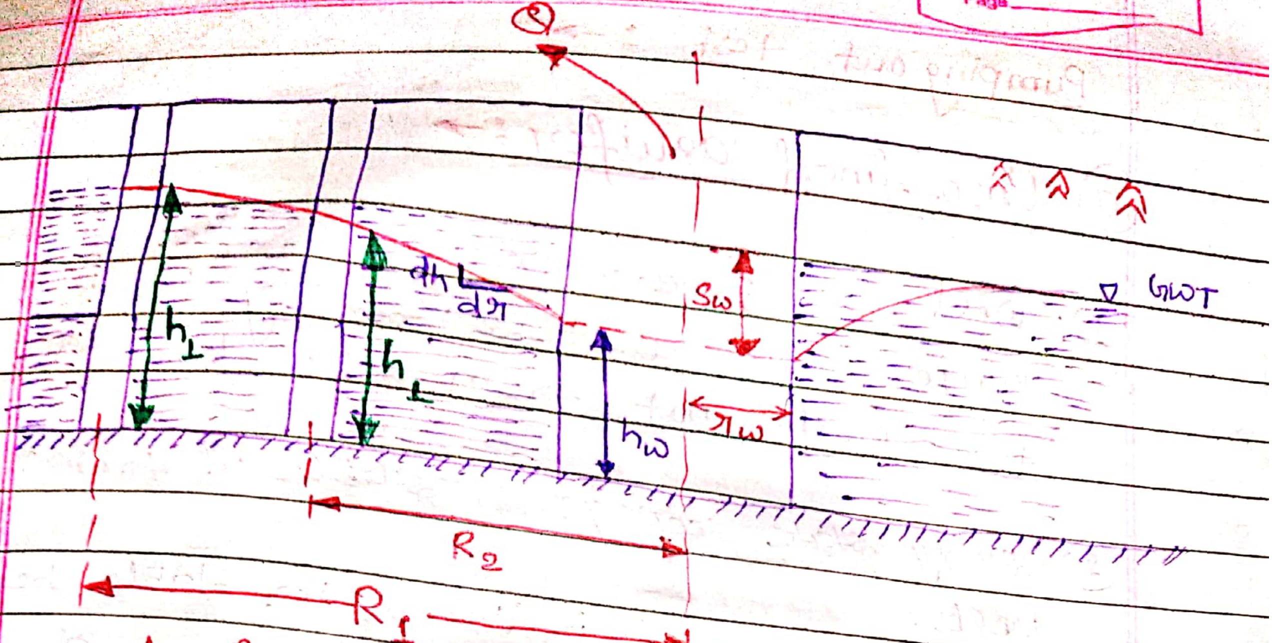
Radius of Influence :→ Distance from the centre of well such that drawdown is Equal to zero

→ Radius of Influence varies b/w 150 to 300m

A/c to Sichardt

$$\boxed{R = 3000d\sqrt{K}}$$

d = drawdown of well (m)
R = Radius of Influence (m)
K = m/sec.



As per darcy

$$Q = K S A$$

$$Q = K \cdot \frac{dh}{dr} \cdot A \rightarrow A = 2\pi r h$$

$$K = \frac{Q \cdot dr}{2\pi h dh}$$

$$K \int_{h_1}^{h_2} h\,dh = \int_{R_1}^{R_2} \frac{Q}{2\pi} \cdot \frac{dr}{r}$$

$$K\left(\frac{h_2^2 - h_1^2}{\cancel{2}}\right) = \frac{Q}{\cancel{2}\pi} \ln\left(\frac{R_2}{R_1}\right)$$

$$\boxed{K = \frac{Q}{\pi\left(h_2^2 - h_1^2\right)} \ln\left(\frac{R_2}{R_1}\right)}$$

$R_2 \rightarrow r_w$

$h_2 \rightarrow h_w$

$$\boxed{K = \frac{Q}{\pi\,(h_w^2 - h_1^2)} \ln\left(\frac{R_w}{R}\right)}$$

$$\boxed{K = \frac{Q \times 2.303}{\pi\,(h_w^2 - h_1^2)} \log_{10} \frac{R_w}{R}}$$

(b) Confind Aquifer :-

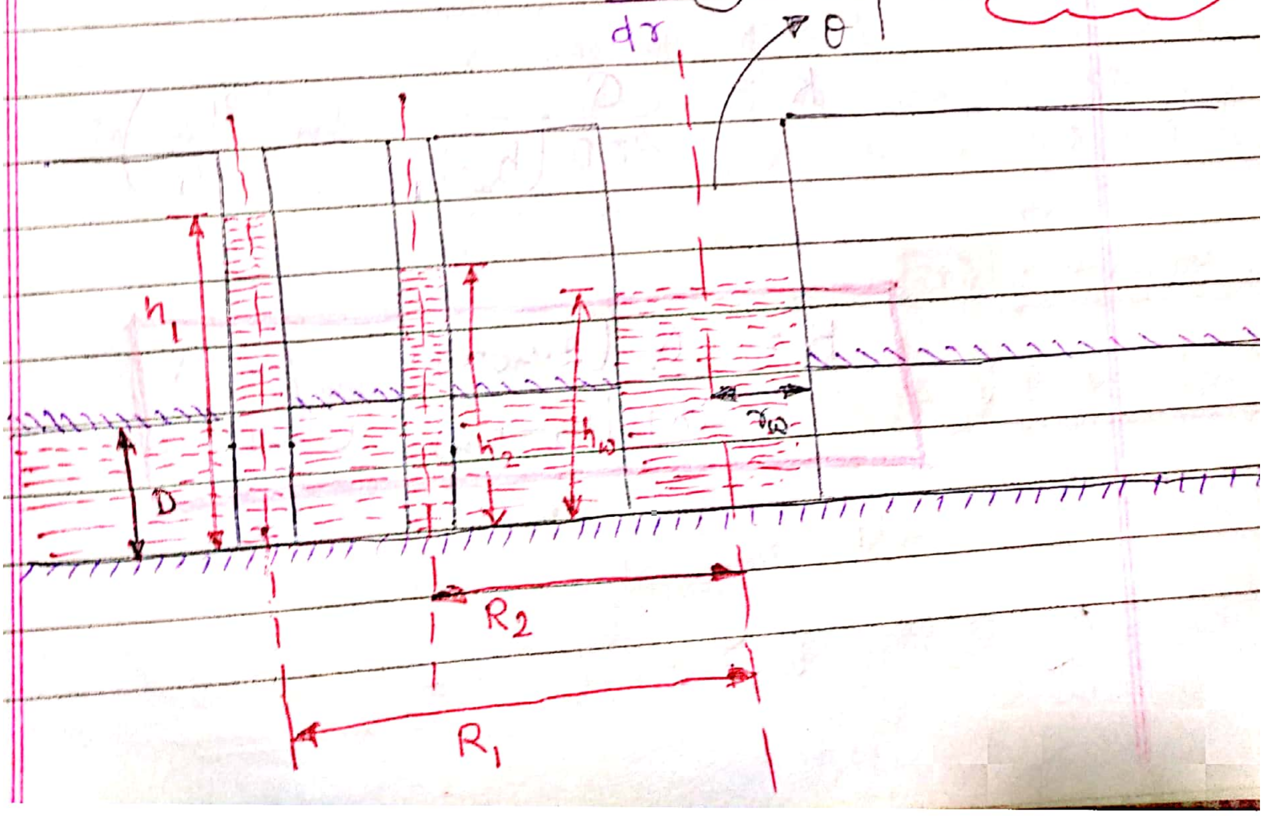
A confined flow condⁿ occurs when the aquifer is confined both above and below by impermeable strata.

A/c darcy's law

$$Q = KiA$$

$$= K \cdot \frac{dh}{dr} \, A$$

$A = 2\pi r B$



$$Q = K \frac{dh}{dr} \cdot A$$

$$Q = K \frac{dh}{dr} (2\pi r D)$$

$$\boxed{K = \frac{Q \cdot dr}{r} \times \frac{1}{2\pi D \times dh}}$$

$$K\,dh = \frac{Q}{2\pi D} \cdot \frac{dr}{r}$$

$$K \int_{h_1}^{h_2} dh = \frac{Q}{2\pi D} \int_{r_1}^{r_2} \frac{dr}{r}$$

$$K(h_2 - h_1) = \frac{Q}{2\pi D} \ln\left(\frac{r_2}{r_1}\right)$$

$$K = \frac{Q}{2\pi D (h_2 - h_1)} \ln\left(\frac{r_2}{r_1}\right)$$

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

INDIRECT METHOD

① Kozeny - Carman Equation :→

γ_w → unit wt of water.

$$\boxed{K = \frac{1}{K_K} \cdot \frac{\gamma_w}{\mu} \cdot \frac{e^3}{1+e} \cdot \frac{1}{S_A^2}}$$

K_K → Shape factor for Kozeny . Carman

μ = Co-efficient of viscosity

S_A = ~~Shape factor~~ Specific surface Area (i.e sp. area per unit vol. of solid)

Specific Area for spherical particle

$$\boxed{S_A = \frac{\pi D^2}{\frac{\pi D^3}{6}} = \frac{D}{6}}$$

↳ If particle size b/w a & b the sp. surface Area = $\boxed{\frac{6}{\sqrt{ab}}}$

② Allen Hazen's Formula :→

→ Co-efficient of permeability of soil is proportional to the square of representative particle size.

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

↳ C = 100 to 150 if D_{10} is in cm (K → cm/sec)

↳ C = 1 to 1.5 if D_{10} is in mm (K → cm/sec)

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③ :→ Loudon's formula :→

$$\boxed{\log_{10}(KS_A^2) = a + bn}$$

n = porosity
a & b = Constant
S_A = sp. Surface Area.

④ Consolidation formula :→

↳ This method is suitable for fine grained soil. whose $K < 10^{-7}$ cm/sec for which Permeability test can not be conducted in Laboratry

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

C_v = Co-efficient of consolidation
m_v = Co-efficient of volume change.

Q→ Estimate the value of co-efficient of Permeability for a uniform gradded sand of size $D_{10} = 0.15$ mm obtained from sieve analysis G = 2.67 by Allen Hazen formula

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

Let,
C = 1 bcz D_{10} in mm

$$K = 1 \times (0.15)^2 = 0.0225 \text{ cm/sec.}$$
$$= 0.225 \text{ mm/sec}$$

SHEAR STRENGTH OF SOIL

→ Soil will always fail in shearing, what be the type of loading.

→ It never occure by crushing of soil particles.

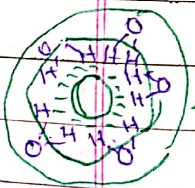
→ Shear Strain of soil is the resistence offered by the soil grains against shear deformation.

→ failures occures when shear stress at any point is exceeded by ~~strength~~ Shear strength of that point.

→ soil may desire its Shear strength from following parameter.

(a) Frictional Strength :→ because of friction interlocking of soil grains

(b) Cohesion :—



(i) True Cohesion :→ Because of electrostatic Force of attraction.

(ii) Apperent cohesion → Due to negative pressure in the pores which Cause attraction b/w the particles.

Methods to findout Shear Strength of SOIL

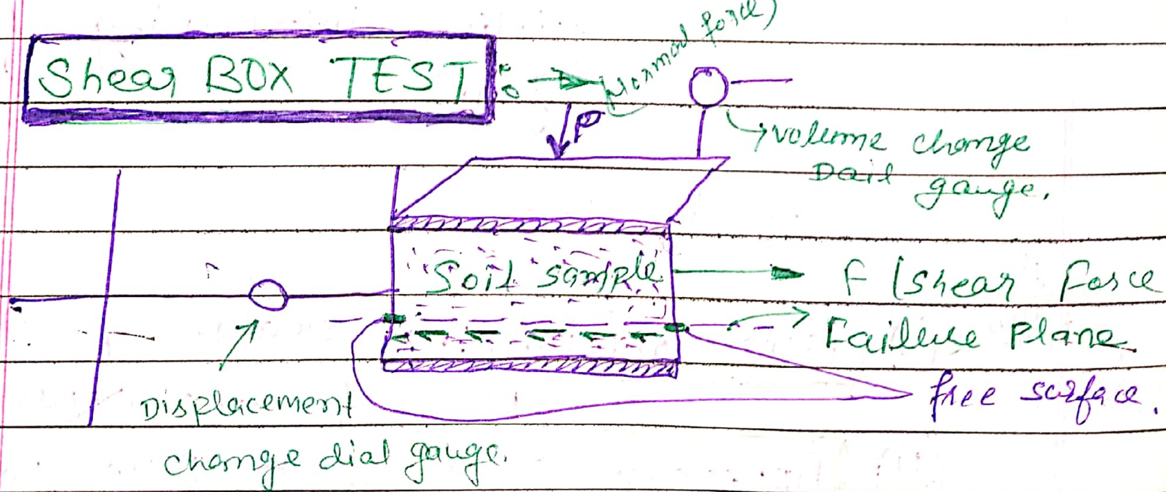
① Direct Shear test (BOX Shear test) :-
This is the oldest test

② Triaxial compression test
(a) Consolidated drain test (CD)
(b) Consolidated undrain test (CU)
(c) Unconsolidated undrain test (UU)
(d) unconfined compression test

③ Torsion Balance shear test

④ Ring shear test
⑤ vane shear test
⑥ Standard penetration test (SPT)
⑦ Cone penetration test (C.P.T)

① **Shear BOX TEST** :-



Advantage :- ① Quick, Inexpensive and easy to do
② Sample preparation is easy

{Box is square used}

Disadvantage: — (i) Drainage cannot controlled and pore water pressure can not be measured.

(ii) This is suitable for sand and gravels. bcz they drain Rapidly

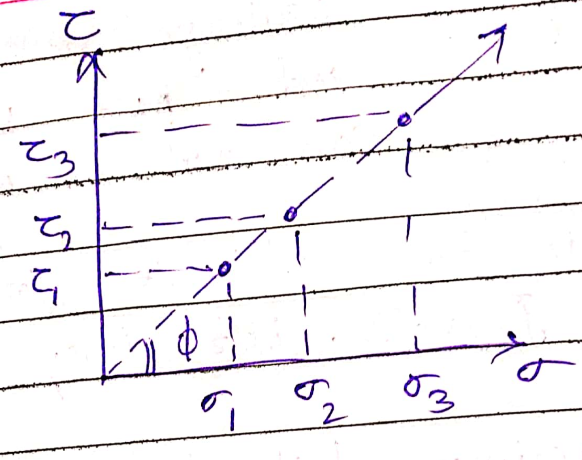
(iii) Specimen fail along (Predetermine Plane not the weakest Plane

(iv) Not possible to perform undrain test which is Import for Clay

(v) Area of specimen under the normal load and shear does not remain constant but we take it as constant for calculation of stress.

Result of test: — various sample are tested up to failure at different normal loading and shear force required for failure is found out corrosponding to normal load at time of failure.

Sample	Normal load	Shear load	Normal Stress	Shear stress
1	P_1	F_1	$\frac{P_1}{A} = \sigma_1$	$\frac{F_1}{A} = \tau_1$
2	P_2	F_2	$\frac{P_2}{A} = \sigma_2$	$\frac{F_2}{A} = \tau_2$
3	P_3	F_3	$\frac{P_3}{A} = \sigma_3$	$\frac{F_3}{A} = \tau$



Straight line Equation

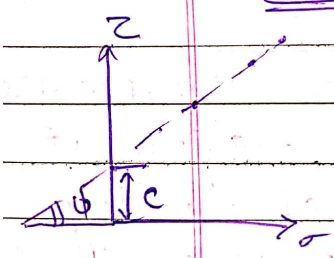
$$y = mx$$

Similarly

$$\tau = \tan\phi \cdot \sigma$$

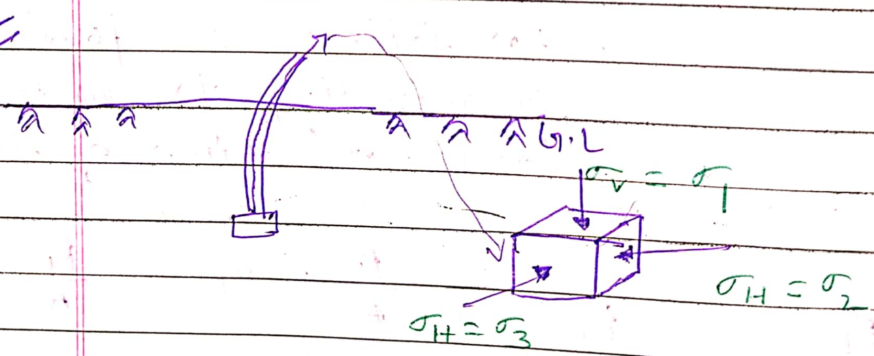
$\boxed{\tau = \sigma \tan\phi}$ for cohessionless soil

For Cohessive



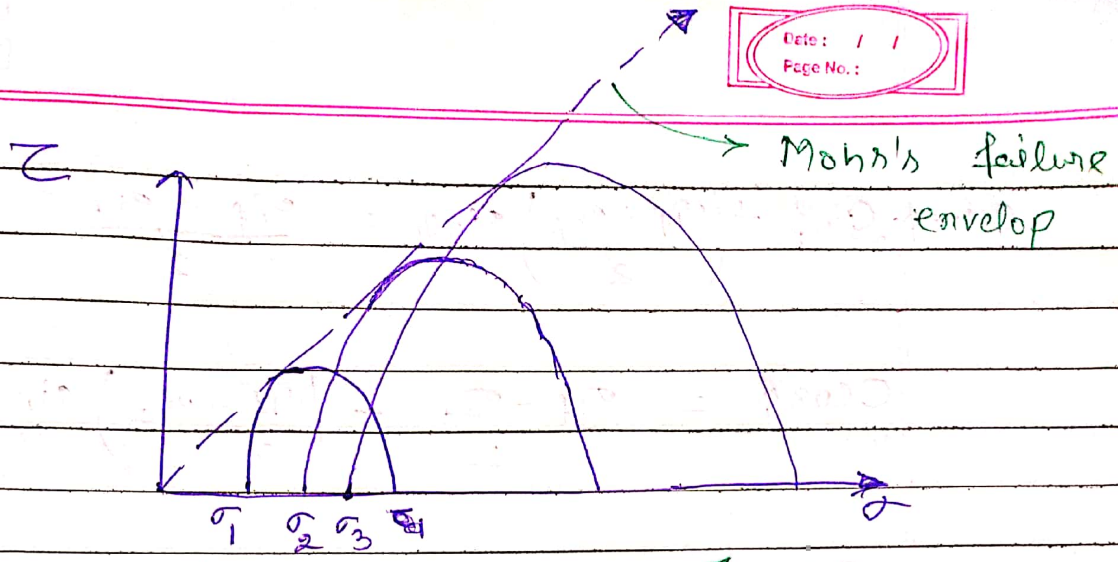
$\boxed{\tau = \sigma \tan\phi + c}$ (i.e. $\cancel{\tau = \sigma \tan\phi} + c$

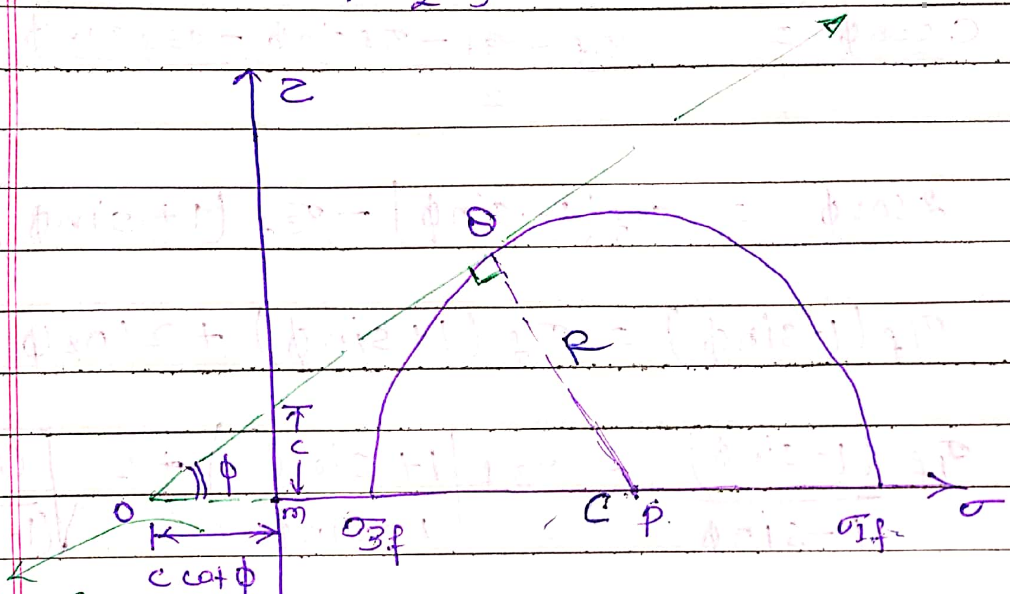
$\boxed{\text{i.e. } y = mx + c}$



$\sigma_1 =$ maxm principle stress

$\sigma_2 = \sigma_3 =$ minor principless





$$\tan\phi = \frac{c}{om}$$

$$om = \frac{c}{\tan\phi} = c\cot\phi$$

$$\boxed{QP = R = \frac{\sigma_{1f} - \sigma_{3f}}{2}} \qquad \boxed{C = \frac{\sigma_{1f} + \sigma_{3f}}{2}}$$

$$OP = om + mP$$

$$= c\cot\phi + \frac{\sigma_{1f} + \sigma_{3f}}{2}$$

$$\sin\phi = \frac{QP}{OP}$$

$$\sin\phi = \frac{\frac{\sigma_{1f} - \sigma_{3f}}{2}}{c\cot\phi + \frac{\sigma_{1f} + \sigma_{3f}}{2}}$$

$$c \cot\phi \cdot \sin\phi + \left(\frac{\sigma_{1f} + \sigma_{3f}}{2}\right)\sin\phi = \frac{\sigma_{1f} - \sigma_{3f}}{2}$$

$$c\cos\phi = \frac{\sigma_{1f} - \sigma_{3f}}{2} - \left(\frac{\sigma_{1f} + \sigma_{3f}}{2}\right)\sin\phi$$

$$c\cos\phi = \frac{\sigma_{1f} - \sigma_{3f} - \sigma_{1f}\sin\phi - \sigma_{3f}\sin\phi}{2}$$

$$2\cos\phi = \sigma_{1f}(1 - \sin\phi) - \sigma_{3f}(1 + \sin\phi)$$

$$\sigma_{1f}(1 - \sin\phi) = \sigma_{3f}(1 + \sin\phi) + 2\cos\phi$$

$$\sigma_{1f}\frac{(1-\sin\phi)}{1-\sin\phi} = \sigma_{3f}\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2\sqrt{\frac{\cos^2\phi}{(1-\sin\phi)^2}}$$

$$\sigma_{1f} = \sigma_{3f}\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c\sqrt{\frac{1-\sin^2\phi}{(1-\sin\phi)^2}}$$

$$\boxed{\sigma_{1f} = \sigma_{3f}\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}}$$

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$$\sin\phi = 2\sin\frac{\phi}{2} \cdot \cos\frac{\phi}{2}$$

$$\sin^2\phi + \cos^2\phi = 1$$

$$1+\sin\phi = \sin^2\phi + \cos^2\phi + 2\sin\frac{\phi}{2}\cdot\cos\frac{\phi}{2} = \left(\sin\frac{\phi}{2} + \cos\frac{\phi}{2}\right)^2$$

$$1-\sin\phi = \left(\sin\frac{\phi}{2} - \cos\frac{\phi}{2}\right)^2$$

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

$$= \frac{\left(1 + \tan\frac{\phi}{2}\right)^2}{\left(1 - \tan\frac{\phi}{2}\right)^2}$$

$$= \frac{\tan 45 + \tan\frac{\phi}{2}}{1 - \tan\frac{\phi}{2}\cdot\tan 45}$$

$$\boxed{\tan^2(A+B) = \frac{\tan A + \tan B}{1 - \tan A\cdot\tan B}}$$

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

$$\boxed{\sigma_{1f} = \sigma_{3f}\tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\left(45+\frac{\phi}{2}\right)}$$

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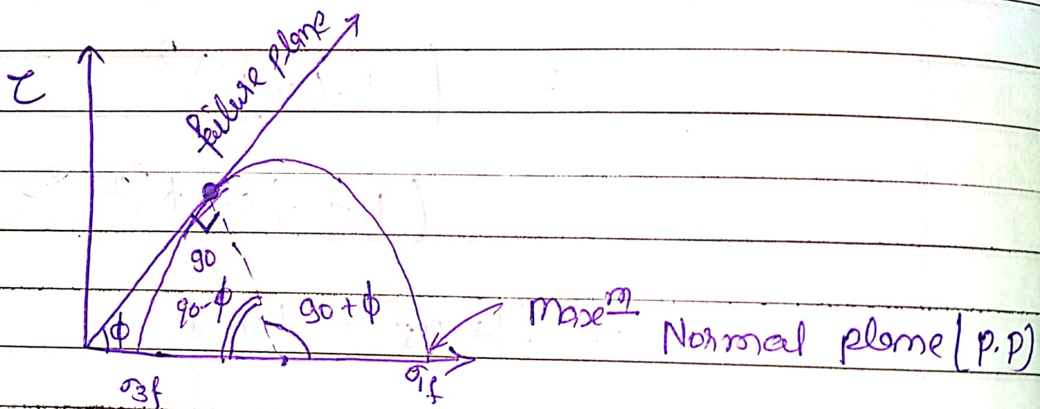
* C and ϕ are collectively called shear strength parameters in which

$$\boxed{\begin{aligned} C &= \text{Cohesive parameter} \\ \phi &= \text{frictional parameter} \end{aligned}}$$

Shear strength of soil

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



→ The failure plane will be inclined at an angle of $\left(45+\frac{\phi}{2}\right)$ with the major principle plane

$$\boxed{\underset{\text{in mohr circle}}{90+\phi} = \underset{\text{in actual condn}}{45+\frac{\phi}{2}}}$$

→ The failure plane will be inclined at an angle of $45-\frac{\phi}{2}$ with the minor principle plane

$$\boxed{\text{in Mohr circle } 90-\phi = 45-\frac{\phi}{2} \rightarrow \text{in actual}}$$

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Q→ A direct shear test on sand gives the following results at time of failure

① Normal load = 288 kN (P_1)
Shear load = 173 N (F)
C/S Area = 36 cm^2 (A)
Calculate – ① the angle of internal friction
(II) Find the magnitude and direction of principle stress from failure plane

$$\sigma = \frac{P_1}{A} = \frac{288}{36} = 8\ N/cm^2$$

$$\tau = \frac{F}{A} = \frac{173}{36} = 4.805\ N/cm^2$$

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

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

$$\frac{4.805}{8} = \tan\phi$$

$$\phi = \tan^{-1}\left(\frac{4.805}{8}\right)$$

$$= 30.99$$

$$\simeq 31$$

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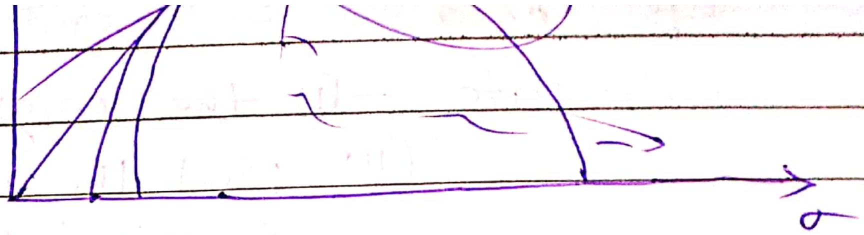
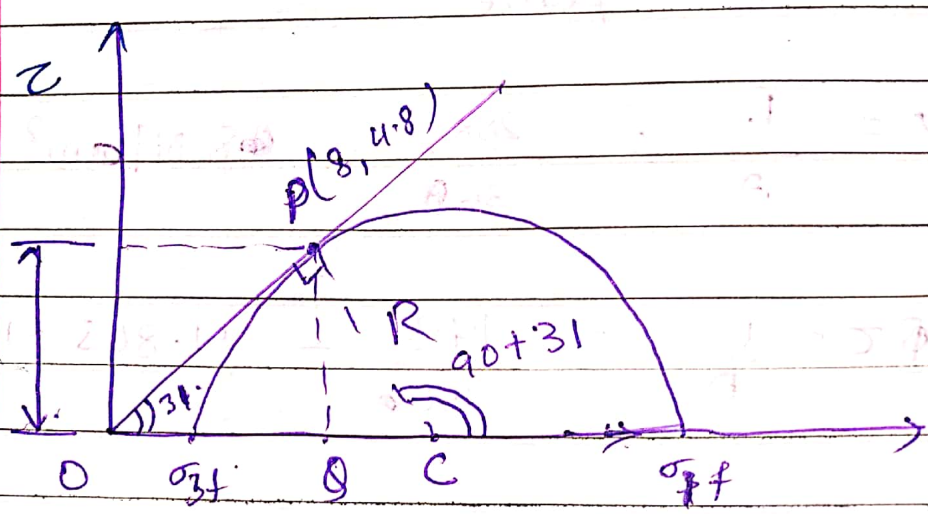




In ΔOPQ

$$OP = \sqrt{OQ^2 + QP^2}$$

$$= \sqrt{8^2 + 4.8^2}$$

$$OP = 9.33$$

In ΔOPC

$$\tan\phi = \frac{PC}{OP}$$

$$PC = OP \tan\phi$$

$$= 9.93 \tan(30.99) = 5.60$$

$$\cos\phi = \frac{OP}{OC}$$

$$OC = \frac{OP}{\cos\phi} = \frac{9.33}{\cos(30.99)}$$

$$= 10.88$$

in mohrs circle $= 121$
then
in actual $= \frac{121}{2} = 60.5$

$\sigma_{1f} = C + R \Rightarrow 10.88 + 9.33 = 20.21$

$\sigma_{3f} = C - R \Rightarrow 10.88 - 9.33 = 1.55$

(2) Triaxial test :→

- This is the most commonly used shear strength test
- It is suitable for all types of soil.
- Drainage can be controlled whatever be type of soil (i.e sand can be tested in undrain condition and clay can be tested in drain condition.)

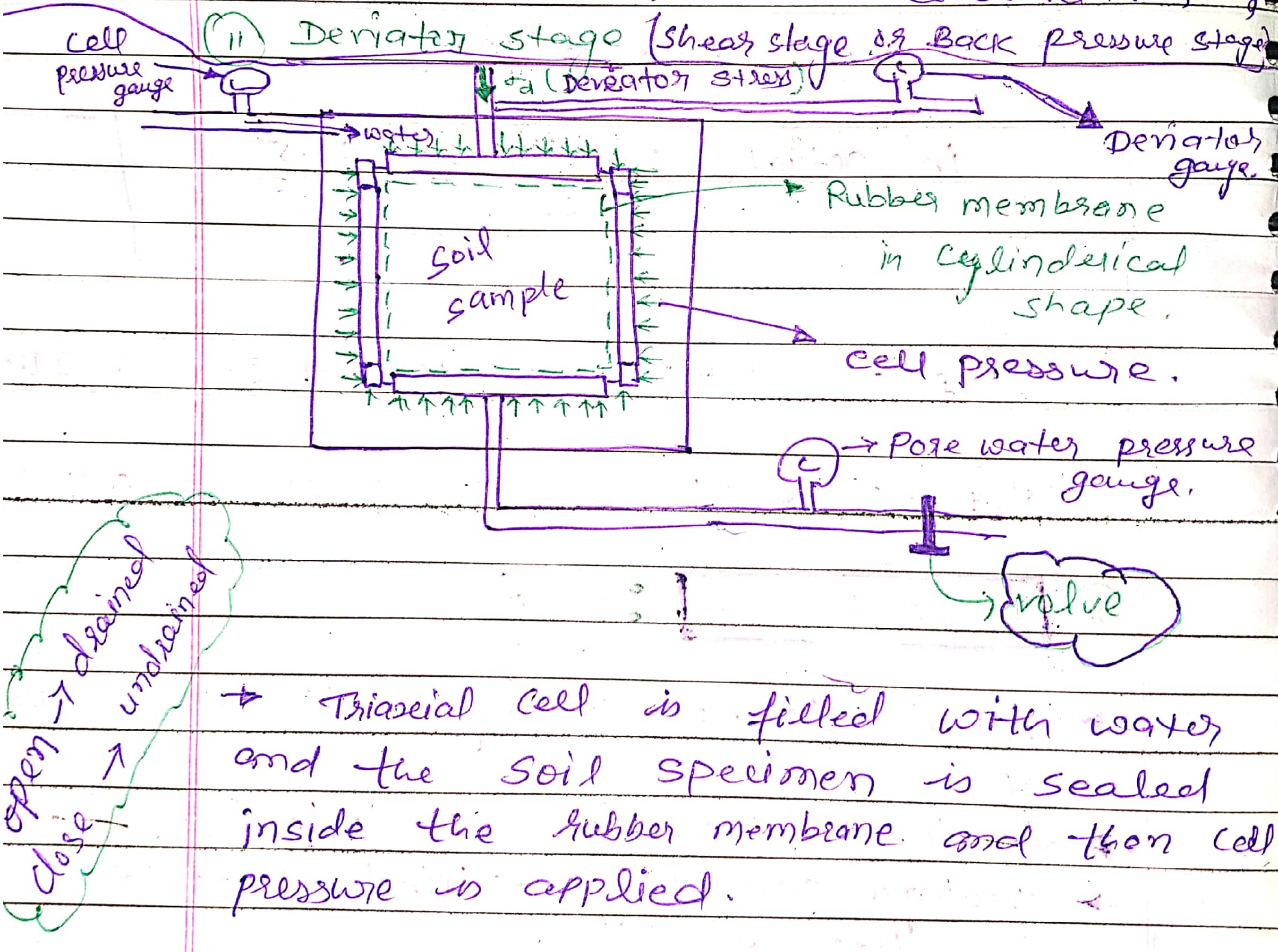
- excess pore water pressure generated can be measured

- volume changes can also be measured
- failure plane is not forced

Triaxial test is completed in two stages

(I) Cell pressure stage (confining stage or consolidation stage)

(II) Deviator stage (shear stage or Back pressure stage)



- Triaxial cell is filled with water and the soil specimen is sealed inside the rubber membrane and then cell pressure is applied.

- This test is conducted in two stages in which drainage valve will be closed or open as our desire.

① **Cell pressure Stage** →

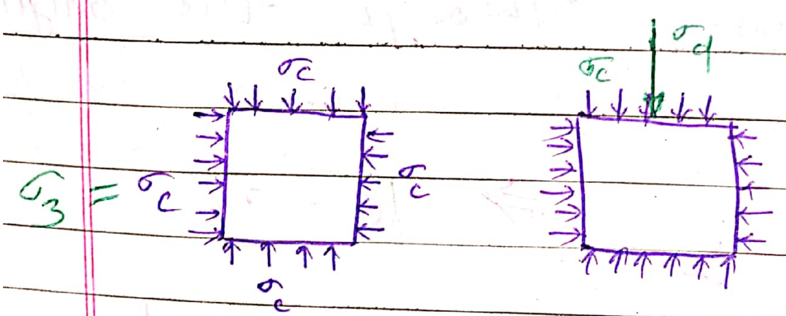
→ All round stress called confining pressure or cell pressure is applied using external water pressure

→ If test is unconsolidated the drainage valve will be closed. But if test is consolidated the drainage valve will be open and expulsion of pore water pressure is permitted

→ When expulsion pore water is stop, 1st Stage will be completed (soil sample is not dry)

② **Deviator Stage** : →

→ Confining pressure is kept constant and additional axial stress is applied known as deviator stage. deviator stress increase gradually until the soil fail in shear



$\sigma_1 = \sigma_c + \sigma_d$

$\sigma_1 = \sigma_3 + \sigma_d$

$$\boxed{\sigma_{df} = \sigma_{1f} - \sigma_{3f}}$$

At the time of failure.

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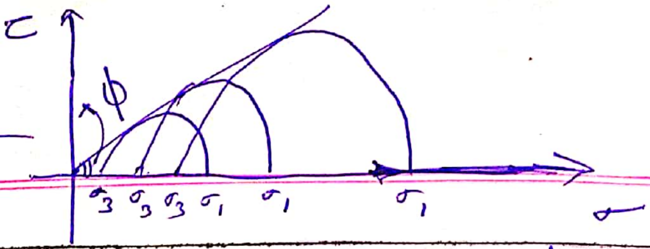


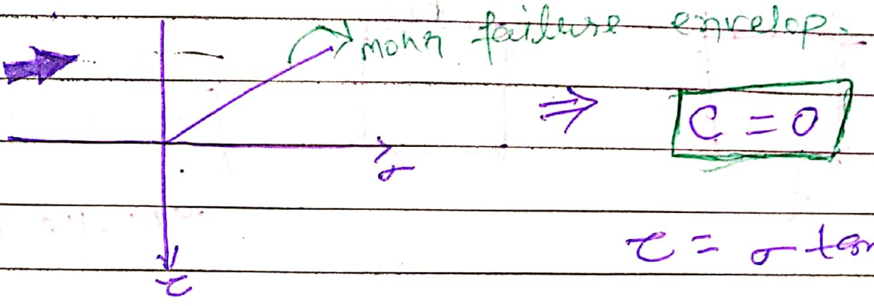
Types of triaxial test

(1) Consolidated drain test :→ Slow test

- Drainage is allowed in both the stages, Hence this test takes longest time
- that why this is also called Slow test.
- The confining pressure generated in 1st stage represents the horizontal confinment in field.
- Loading rate is so slow, so as to allow the water is expelled out.
- Pore water pressure does not buildup any stage
- The shear strength parometers are obtained are corrosponding to effective stress or total stress.

(a) * If the soil normally consolidated mohr envelop passes through the origin

By the experiment



$\Rightarrow$ $\boxed{C = 0}$

$\tau = \sigma \tan \phi$

$= \bar{\sigma} \tan \phi'$

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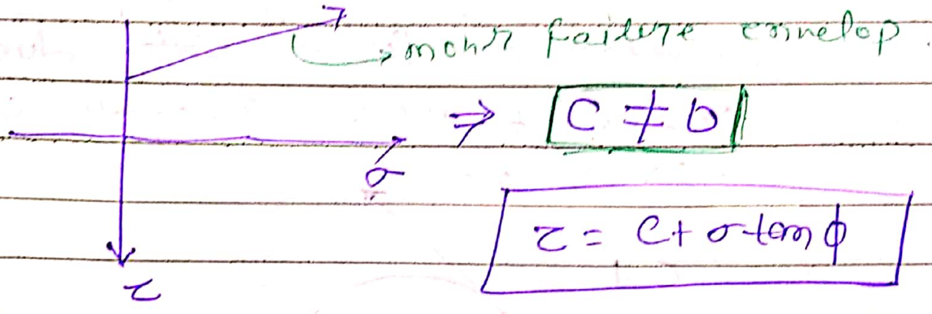
b) * If the soil over consolidated some shear strength due to preconsolidation is exists and hence even at zero normal stress. It will have some shear strength



$\Rightarrow$ $\boxed{C \neq 0}$

$$\boxed{\tau = C + \sigma \tan\phi}$$

APPLICATION :→

① This test is suitable coarse grained soil.

② **consolidated undrain test :→**

→ 1st stage takes around 24 hrs and 2nd stage takes 2hrs approximetly

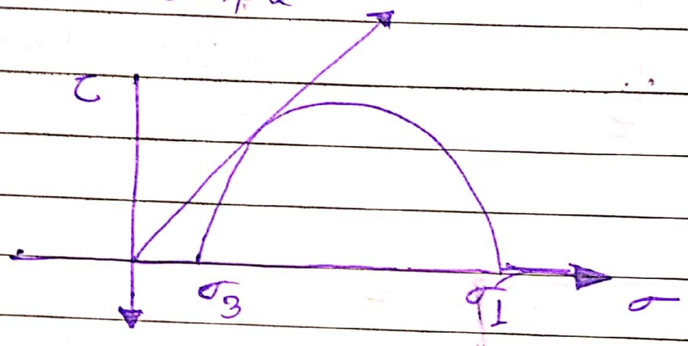
→ In this test we find out total stress parameter however if pore water pressure is measured, we can also calculate effective stress parameter also. (C' & φ')

If we express effective stress parameter the test symbol is denoted by $\overline{CU}$

→ The strength of soil under CU testing is primarly governed by confining stress at the consolidation has been done in 1st stage.

* No strength will be developed in the 2nd stage.

→ In the CU testing as the confinment is increased progeresively larger mohr circle is obtained Because ~~initial~~ due to larger initial confinment more strength comes into the sample



* → for normally consolidated soil $\boxed{C_u = 0}$

$\boxed{C_{cu} = 0}$

Cohessive strength of soil in Consolidated undrain test

$\boxed{C_{cu} \neq 0}$ → for over conssolidated soil.

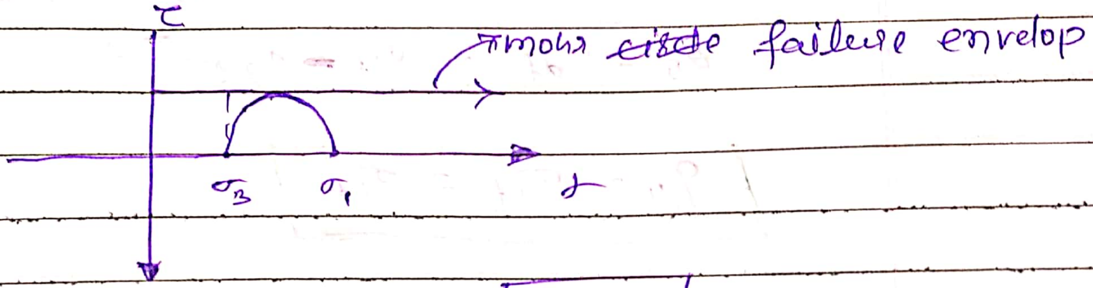
(3) Unconsolidated undrain test :— (Quick test)

Consalidation is not pemitted in both the stages.

Therefore It take 15 minutes only. So this called quick test

→ We generally perform total stress analysis in this case to compare the field situation in which excess pore water pressure can not be measured.

→ Strain does not build up in both the stages that why mohr failure envelop becomes horizontal.



$\boxed{\tau = c}$

→ This test is suitable for fine grained soil. which have low permeability

→ Loading rate is very fast.

(4) Unconfined Compression test :→

→ No Confinement is applied i.e $\sigma_3 = 0$
→ Hence only one mohr circle is obtained.
→ It is special case of triaxial test in which confining pressure is zero. It means there is no 1st stage (cell pressure stage)

→ Therefore no Rubber membrane is required. without Rubber membrane dry soil and sand can not be held in position

- Hence this test can be conducted in saturated silt and clay ~~out at~~

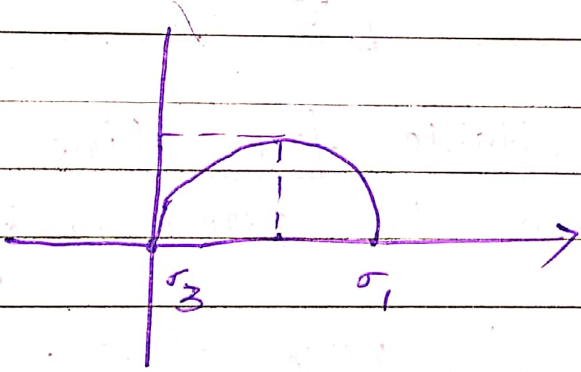
- But it is more suitble for clay

- $\sigma_d = \sigma_1 - \sigma_3$

at failure [σ_d is also known as unconfined compressive strength q_u]

$$q_u = \sigma_{df} = \sigma_{1f} - \cancel{\sigma_{3f}} \rightarrow 0$$

$$\boxed{q_u = \sigma_{df} = \sigma_{1f}}$$



$$C = \frac{\sigma_{1f}}{2} = \frac{\sigma_{df}}{2} = \frac{q_u}{2}$$

$$\boxed{C = \frac{q_u}{2}}$$

Q. The undrain cohesion of remoulded clay soil is $10 KN/m^2$. if the sensitivity of clay is 20 corresponding remoulding compressive strength is

(a) $5 KN/m^2$ (b) $10 KN/m^2$

(c) $20 KN/m^2$ (d) $200 KN/m^2$

$\cancel{S_t = \frac{q_u(\text{undisturbed})}{q_u(\text{remoulded})}}$

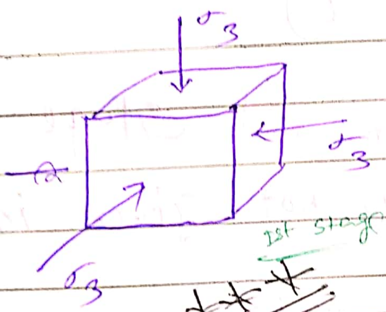
$$\boxed{q_u = 2C}$$

$q_u = 2 \times 10$
$= 20$

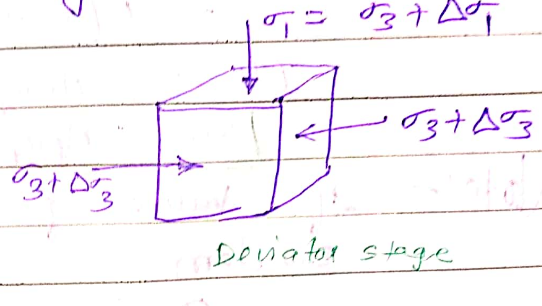
* Skempton Pore pressure Co-efficient :-

→ Pore pressure co-efficient are used to express response of pore water pressure to change in total stress in undrain condition.

→ It is not possible to measure pore pressure by practical, then theoritical approch is given by Skempton.



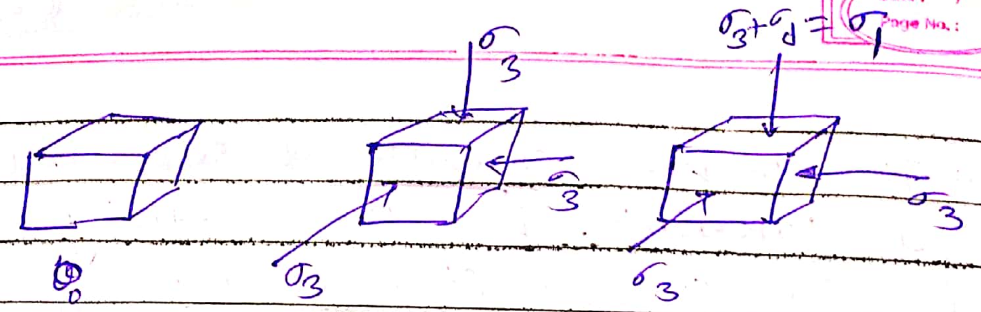
1st stage



Deviator stage

$$\boxed{\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]}$$

A & B are pore pressure co-efficient



1st stage   2nd stage

U_0 U_1 U_2

	$\Delta\sigma_1 = \sigma_3$	$\Delta\sigma_1 = \sigma_d$
	$\Delta\sigma_3 = \sigma_3$	$\Delta\sigma_3 = 0$

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$$

$$= B[\sigma_3 + A(\cancel{\sigma_3 - \sigma_3})]$$

$$\Delta u = B\sigma_3$$

$$\boxed{\frac{\Delta u}{\sigma_3} = B} \rightarrow \text{By 1st stage}$$

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$$

$$= B(0 + A(\sigma_d - 0))$$

$$\Delta u = AB\sigma_d$$

$$\boxed{\Delta u = A_f\sigma_d} \rightarrow \text{from 2nd stage}$$

Notes:- If value of B is not given in problem

① for saturated soil

$$\boxed{B = 1}$$

② For dry soil

$$\boxed{B = 0}$$

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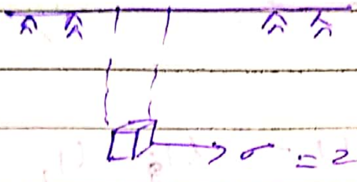
Q-1 A saturated clay sample was obtained from field without allowing the water content and void ratio to change the sample was subjected to following stress in the field.


$\sigma = 240$ kpa
$u = 140$ kpa.

Q.① On sampling what are the stress acting on the sample immediately.

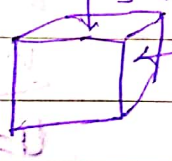
$\bar{\sigma} = \sigma - u$
$= 240 - 140$
$= 100$ kpa.

total stress $\sigma = 0$
effective stress $\bar{\sigma} = 100$ kpa (effective does not change ~~immediatly~~ even load is applied or removed

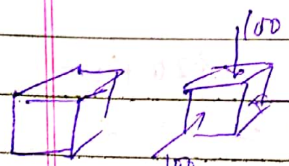
$\sigma = \bar{\sigma} + u$
$u = -\bar{\sigma} + \sigma$
$= -100 + 0$

pore water pressure (u) = -100 kpa

2.Q → sample was placed on triaxial cell and cell pressure was applied at 100 kpa while keeping the drainage valve closed. what are stress on it


$\sigma_3 = 100$ kpa.
$\sigma_3 = 100$ kp

$\bar{\sigma} = (\sigma - u) \Rightarrow (100 - 0)$
$\bar{\sigma} = 100$ kpa
$\sigma = 100$ kpa
$u = 0$


100
100
100
$u = -100$
$\bar{\sigma} = 100$
$\sigma = 0$

$\Delta u = u_1 - u_0$
$100 = u_1 - 100$
$\boxed{u_1 = 200}$

$\boxed{\Delta u = B\left(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right)} \Rightarrow \Delta u = 1\left(100 + 1A(100 - 100)\right)$
$\Delta u = 100$ kPa.

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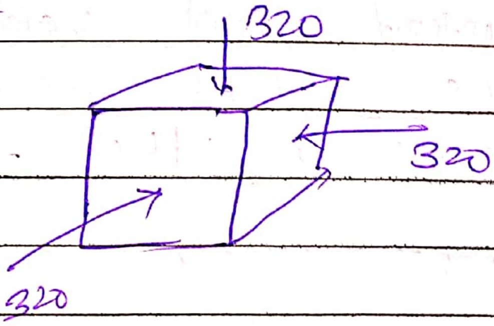
Q.(3) Cell pressure is now raise to 320 KPa with drainage valve closed what stress on it



$\sigma = 320$ Kpa.

$u = 220$ Kpa

$\bar{\sigma} = 100$ Kpa

$u = u_f - u_1$

$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3))$

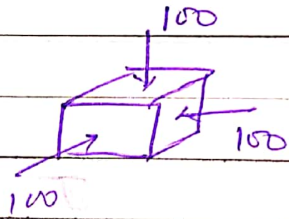
$\Delta u = 1\,[220 + A(220 - 220)]$

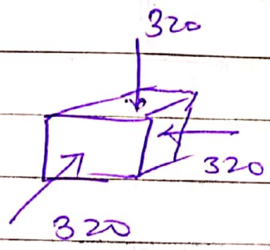
$\Delta u = 220$

$u_2 - u_1 = 220$

0

$\boxed{u = 220}$



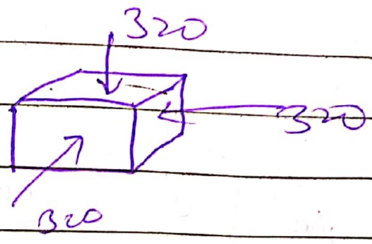


(4) Q→ Drainage is now open and soil is allowed to ~~to~~ consolidate under ~~soil~~ cell pressure. Find the stress.

Soln: →

valve open

$u = 0$ | due to open valve Excess pore water pressure becomes zero



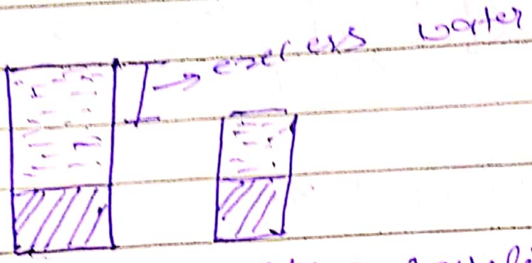
$\sigma = 320$ Kpa

$u_2 = 220$ Kpa

$\bar{\sigma} = 100$ Kpa,

$\sigma = 320$ kpa.
$\bar{\sigma} = 320$ kpa

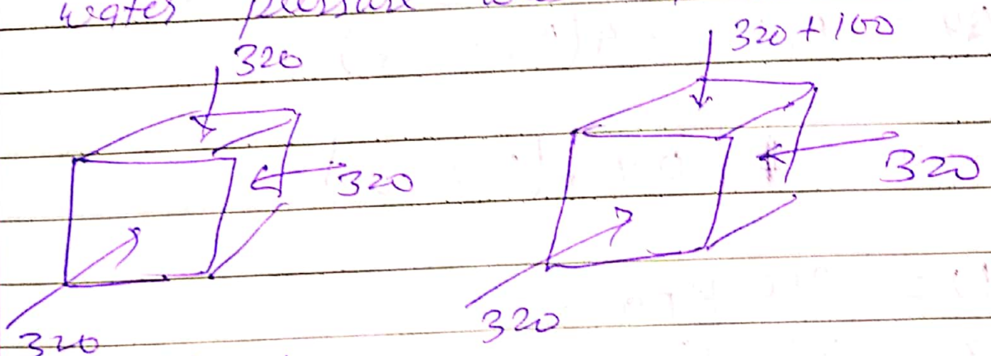
Q. (5) What will the nature of change in water content and degree of saturation after the consolidation.



After consolidation
$S = 100\%$. (Does not change)

Water content and void ratio are decreases. But degree of saturation remain constant.

Q → (6) Drainage valve is now closed, and addition axial pressure is applied with cell pressure remaining constant at 320 kpa when axial pressure was 100 kpa, the pore water pressure was measure as 80 kpa.



$u_0 = 0$ $\quad U_1 = 80$ kpa

$$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3))$$

$$80 - 0 = 1(\cancel{320} + A(400 - 0)) \Rightarrow \frac{80}{400} = A$$

$$\boxed{A = 0.8}$$

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(7) Q→ Had an axial stress of 200kPa beeing achieved the sample has failed

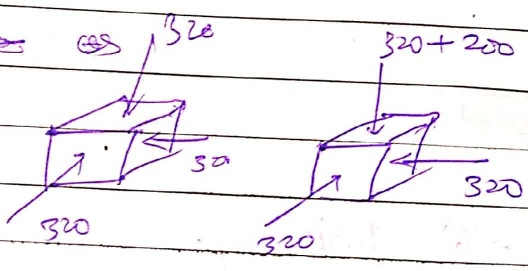
(a) if A = 1 then what is the value σ_{1f}

(b) find the value of ϕ' if $c = 0$

$\sigma_1 = 200\,kP + 320$

$\sigma_d = \sigma_{1f} - \sigma_{3f}$

3



$u = 0$

$\sigma = 320$

PWP = 0

$\Delta\sigma_1 = 200$

$\Delta\sigma_3 = 0$

$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3))$

$u - 0 = B[0 + 1(200 - 0)]$

$u = 200$ kPa

$\sigma = 520$ kPa

$\bar{\sigma}_{1f} = 320$ kPa

$\bar{\sigma}_{3f} = 320 - 200$

$= 120$ kPa

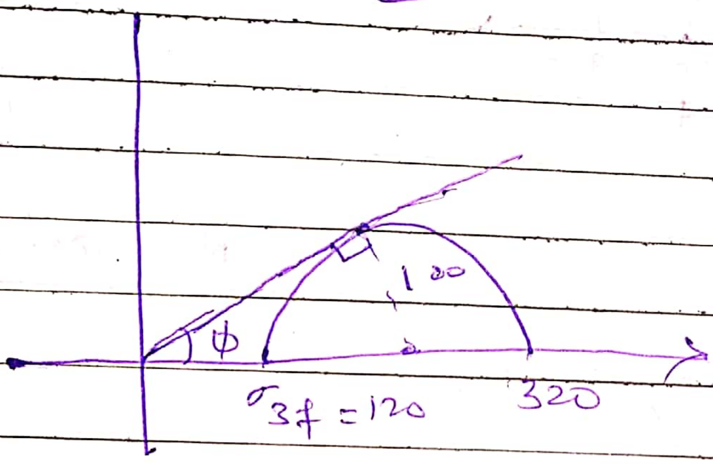
(b) $\tau = \cancel{c} + \sigma \tan\phi$

$$\sigma_{1f} = \sigma_{3f} \tan^2\left(45 + \frac{\phi}{2}\right) + 2\cancel{c}^{\,0} \tan\left(45 + \frac{\phi}{2}\right)$$

$$320 = 120 \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$\frac{32\cancel{0}}{12\cancel{0}} = \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$\boxed{\phi = 27.03}$$

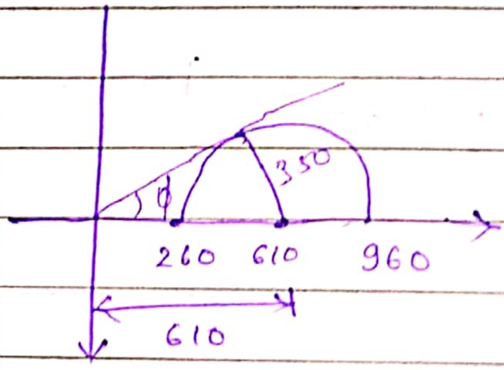


$$C = \frac{120 + 320}{2} = \cancel{80}\ 250$$

$$R = \frac{320 - 120}{2} = \frac{200}{2} = 100$$

$$\sin\phi = \frac{10\cancel{0}}{25\cancel{0}}$$

Q→ A sample of dry sand are tested in triaxial and direct shear test. in the triaxial test the sampled failed when the major and minor Principal stress where 960 kpa and 260 kpa respectively (a) what shear strain could be expected in the direct shear stress if normal stress of 230 kpa.



$$C = \frac{960+260}{2} = \frac{1220}{2} = 610$$

$$R = \frac{960-260}{2} = \frac{700}{2} = 350$$

$$\sin\phi = \frac{35\cancel{0}}{61\cancel{0}}$$

$$\phi = \sin^{-1}\left(\frac{35}{61}\right)$$

$$= 35.01$$

In Shear Box test $(\sigma = 230)$

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

$$= 230 \tan(35.01)$$

$$= 161.10 \text{ Kpa}$$

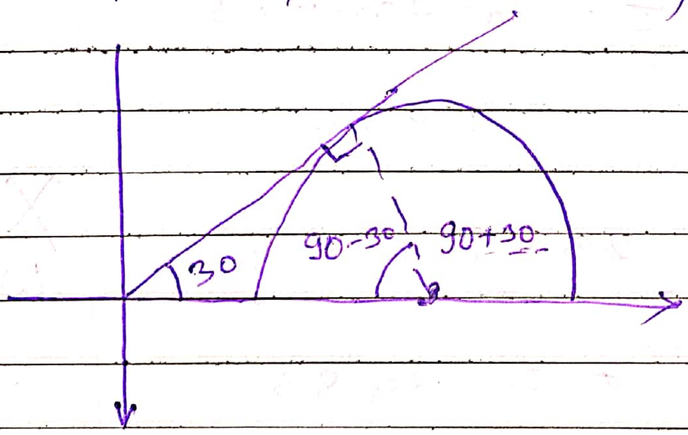
2nd method

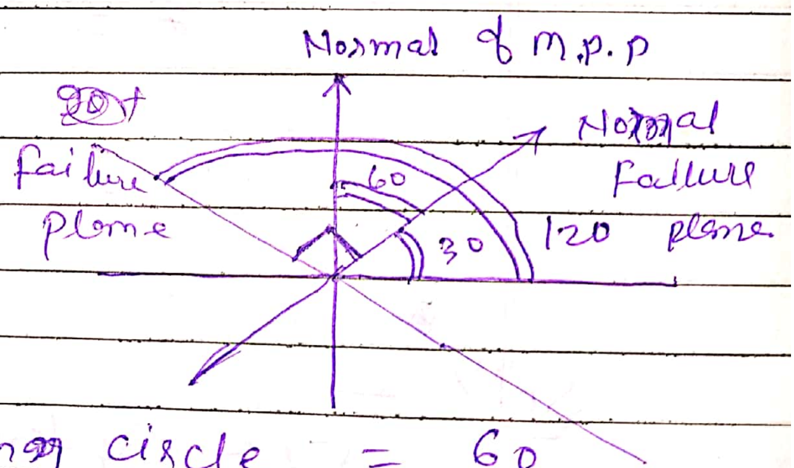
$$\sigma_{1f} = \sigma_{3f} \tan^2\left(45 - \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right)$$

$c = 0$ (∵ sand)

$$\boxed{\sigma_{1f} = \sigma_{3f} \tan^2\left(45 - \frac{\phi}{2}\right)}$$

Q→ If angle of shear resistance is 30° what will be the angle between the normal of major principle plane and failure plane.

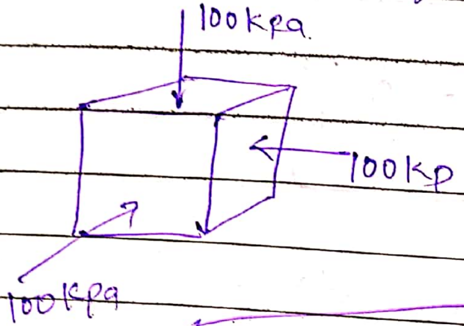




in mohr circle = 60

actual in = 30°

Q. A CU test was conducted on a soil sample with cell pressure σ_c = 100kpa if C = 0 ϕ = 13.3° ~~30~~ ~~30.3°~~ and C' = 0 then ϕ' = 30° What was the pore water pressure at failure



σ = 100kpa

τ = 100 tan 13.3

= 23.63

23 = $\bar{\sigma}$ tan 30

$\bar{\sigma}$ = 39.83

$\boxed{U = 60.17}$

X wrong approach.

Sol^n :—

$$\sigma_{1f} = \sigma_{3f} \tan^2\left(45 + \frac{\phi}{2}\right)$$

σ_{3f} = 100 kpa

c = 0 $\quad$ ϕ = 13.3

σ_{1f} = 159.74 kpa.

Initial suction → Pore water pressure is negative

let pwp $= u$

$$\bar{\sigma}_{1f} = \sigma_{1f} - u$$

$$\bar{\sigma}_{1f} = 159.74 - u \quad \text{— (I)}$$

$$\bar{\sigma}_{3f} = 100 - u \quad \text{— (II)}$$

$$\bar{\sigma}_{1f} = \bar{\sigma}_{3f} \tan^2\left(45 + \frac{\phi'}{2}\right)$$

$$C = 0 \;,\; \phi = 30$$

$$159.74 - u = (100 - u) \tan^2\left(45 + \frac{30}{2}\right)$$

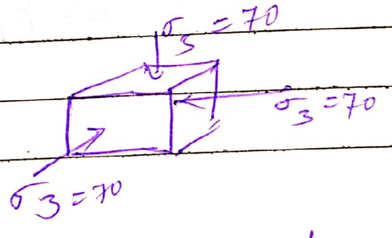
$$159.74 - u = 300 - 3u$$

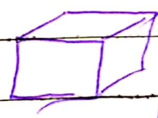
$$2u = 300 - 159.74$$

$$\boxed{u = 70.125 \text{ kPa}}$$

Q → A triaxial test specimen from a Staturated clay sample is subjected to cell pressure is 70 kpa. initial suction in the speciment 35 kpa. If $C' = 0$ and $\phi' = 20°$. and a constant $A_f = -0.2$ then determine the shear strength of soil.

$$\sigma_3 = 70 \text{kpa} \;,\; \phi' = 20° \;,\; C' = 0 \;,\; A_f = -0.2$$

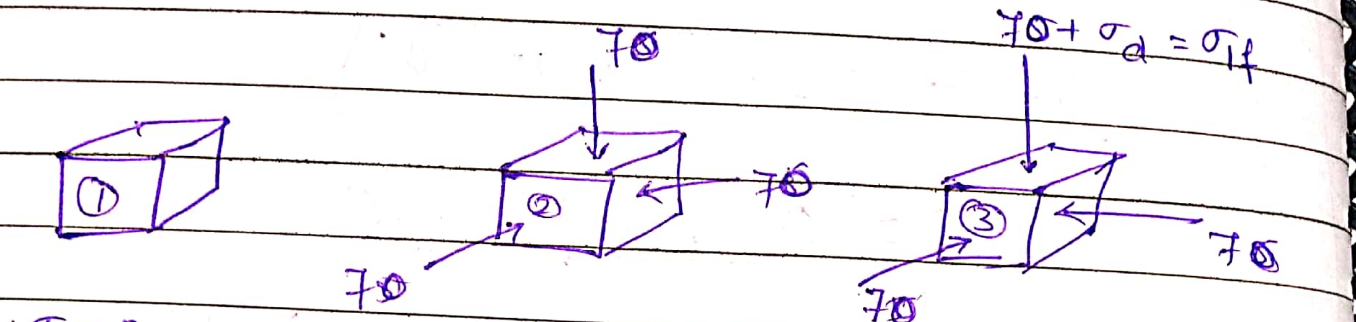




$$\left(\begin{array}{l}\sigma = 0 \\ u = -35 \text{ kpa}\end{array}\right)$$

Initial Suction = −ve

$$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)) \Rightarrow ~~B \& 2 \ldots~~$$



$\sigma = 0$ — Cell pressure stage — Deviator stage.

$u = -35$ kpa — Stage

suction → (−ve) — u_2

from ① and ② system.

$$\Delta\sigma_1 = 70$$

$$\Delta\sigma_3 = 70$$

$$\Delta u = B\left(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right)$$

$$\Delta u = 1\left(70 + \cancel{A(70-70)}^{\,0}\right)$$

$$u_2 - u_1 = 70 \Rightarrow u_2 - (-35) = 70$$

$$u_2 = 70 - 35$$

$$= 35 \text{ kpa.}$$

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again using relation and

from IInd and IIIrd stage

$$\Delta\sigma_1 = \sigma_d$$
$$\Delta\sigma_3 = 0$$

$$\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3))$$

$$\Delta u = AB\sigma_d$$

$$u_3 - u_2 = A_f\sigma_d$$

$$\boxed{u_3 - 35 = -0.2\sigma_d}$$

from IIIrd and (1)

$$\Delta\sigma_1 = 70 + \sigma_d$$
$$\Delta\sigma_3 = 70$$
$$U_3 = ?$$
$$U_2 = -35$$

$$U_3 - 35 = 1[70 + A(\cancel{70} + \sigma_d - \cancel{70})$$

$$u_3 - 35 = 70 + A_f\sigma_d$$

$$U_3 = 35 \mp 0.2\sigma_d$$

* Using relation.

$\overline{\sigma_{1f}} = \sigma_{1f} - U_3$

$= 70 + \sigma_d - 35 + 0.2\sigma_d$

$= 35 + 1.2\sigma_d$

$\overline{\sigma_{3f}} = \sigma_{3f} - U_3$

$= 70 - 35 + 0.2\sigma_d$

$= 35 + 0.2\sigma_d$

again using relation.

$\overline{\sigma_{1f}} = \overline{\sigma_{3f}} \tan^2\left(45 - \frac{\phi'}{2}\right)$

$35 + 1.2\sigma_d = (35 + 0.2\sigma_d)\tan^2\left(45 + \frac{30}{2}\right)$

~~35 + 0.2~~ $\boxed{\tan^2 55 = 2.039}$

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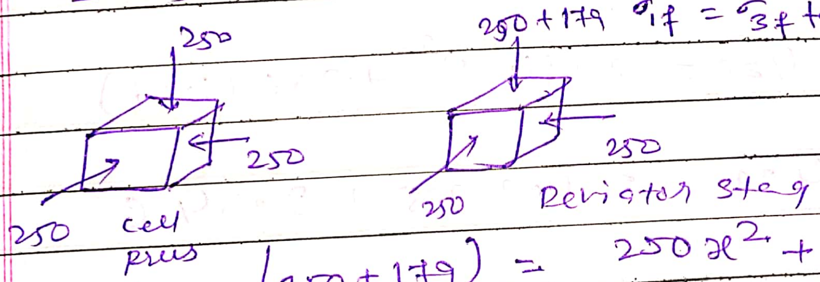
d Consolidated undrain triaxial test where performed on two identical specimen of saturated clay with pore pressure measurement the observation are as follow.

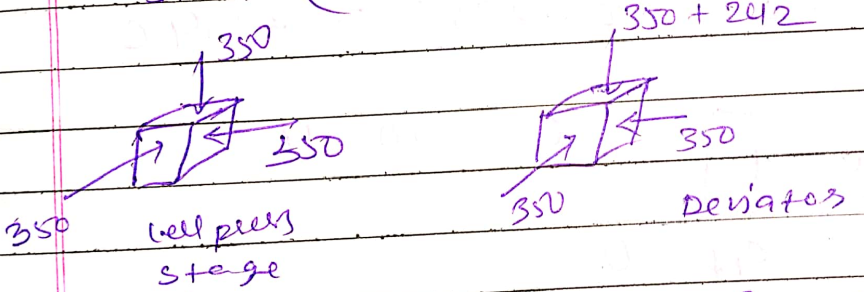
Sample	cell pressure at failure	deviator stress at failure	pore water pressure at failure
(1)	250 Kpa	179	102 Kpa
(2)	350 Kpa	242 Kpa	145 Kpa.

determine the shear strength parameter in terms of total stress as well as effective stress

$$\sigma_{1f} = \sigma_{3f}\,\underbrace{\tan^2\left(45+\frac{\phi}{2}\right)}_{x^2} + 2c\,\underbrace{\tan\left(45+\frac{\phi}{2}\right)}_{x}$$



$$(250+179) = 250x^2 + 2cx \quad \text{— (1)}$$



$$(350+242) = 350x^2 + 2cx \quad \text{— (ii)}$$

from (1) & (2)

$$429 = 250x^2 + 2cx$$
$$592 = 350x^2 + 2cx$$
$$-163 = -100x^2$$

$$x = \frac{163}{100} = 1.27$$

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$$\tan\left(45+\frac{\phi}{2}\right) = 1.27$$

$$45+\frac{\phi}{2} = \tan^{-1}(1.27)$$

$$\phi = \left(\frac{51.78+45}{2}\right)\times 2$$

$$= \cancel{48.28}\ 13.56$$

from Eq - 1

$$429 = 250x^2 + 2cx$$

$$429 = 250(1.27)^2 + 2\times c\,(1.27)$$

$$429 = 403.225 + 2.54c$$

$$429 - 403.225 = 2.54c$$

$$c = 10.14$$

$\boxed{\text{Sample - 1}}$

$$\bar{\sigma}_{1f} = \sigma_{1f} - U$$

$$= 429 - 101$$

$$= 328$$

$$\bar{\sigma}_{3f} = \sigma_{3f} - U$$

$$= 592 - 101$$

$$= 491$$

Vane Shear test :-

- This test is used in lab as well in field.

- The mechanism of this test is same for lab and field.

- In this test there is no mechanis major pore pressure and drainage facility is provided. Hence it can be conducted only undrain condition.

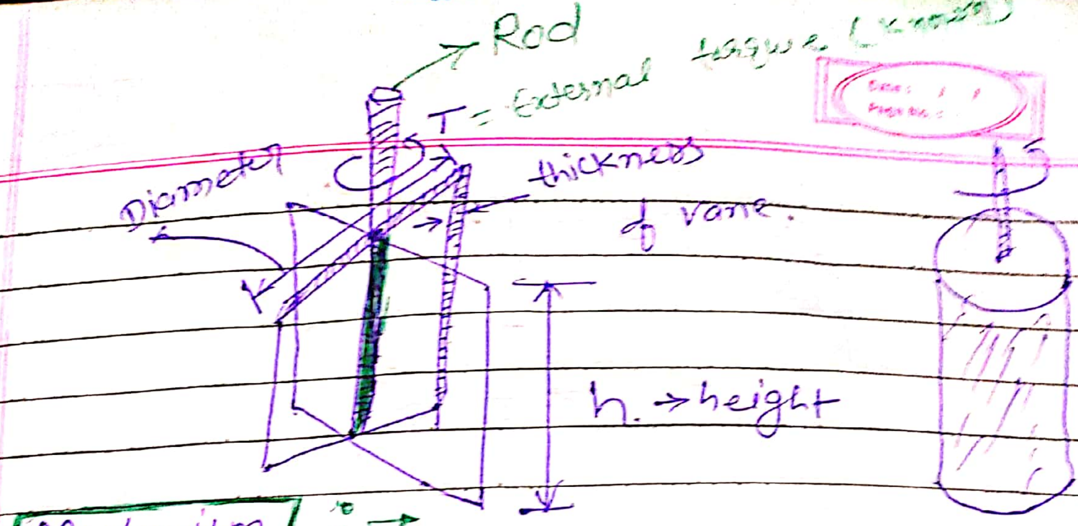
- It is suitable for soft saturated clay

Marine clay

- To find undrain shear strength and also ~~find~~ we used to find out sensitivity of clay

SIZE OF VANE SHEAR Instrument

	Lab	Field.
Height	20mm	10 to 20 cm
Diameter	12mm	5 to 10 cm.
thickness of vane	0.5 to 1mm	2 to 3mm.



Mechanism :→

→ The vane is punch into the soil. and torque is applied by rotating the vane 6°/minutes.

$T_{external} = T_{side} + T_{bottom} + T_{top}$.

Condⁿ

① If vane completely inserted in soil.

$$T_{external} = T_{side} + 2T_{bottom}$$

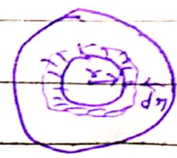
② If vane is inserted ground level.

$$T_{external} = T_{side} + T_{bottom}.$$

* $$T_{side} = (\tau \times \pi dL) \times \frac{d}{2}$$

$$= \frac{\tau \pi d^2 L}{2}$$

* T_{bottom} or top $$= (\tau \cdot dA)\, r$$



$$= \int_0^{d/2} \tau \times (2\pi r . dr)\, r$$

$$= \tau \times 2\pi \int_0^{d/2} r^2 dr$$

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$$= 2\pi \tau \times \left[\frac{r^3}{3}\right]_0^{d/2}$$

$$= 2\pi \tau \cdot \frac{d^3}{24}$$

$$= \tau \times \frac{\pi d^3}{12}$$

$$T_{external} = T_{side} + 2\,T_{bottom}$$

$$= \tau \times \frac{\pi d^2 l}{2} + 2 \times \tau \times \frac{\pi d^3}{12}$$

$$T_{external} = \tau\left[\frac{\pi d^2 l}{2} + \frac{\pi d^3}{6}\right]$$

$$\boxed{\tau = \frac{T_{external}}{\frac{\pi d^2 l}{2} + \frac{\pi d^3}{6}}}$$

(2) When vane is inserted ground level.

$$\boxed{\tau = \frac{T_{external}}{\frac{\pi d^2 l}{2} + \frac{\pi d^3}{12}}}$$

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