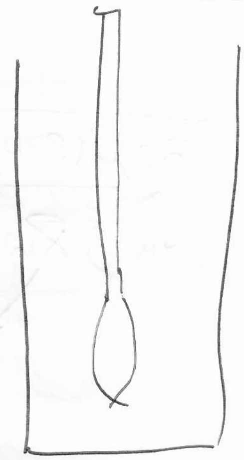
Siddique Sir

Meniscus Correction



$$D = \sqrt{\frac{18\mu}{\gamma_s - \gamma_w} \times \frac{Z_r}{t}}$$

1 poise = 1 dyne-s/cm²

1 gm-sec/cm² = 980.7 poise

$$\gamma_s - \gamma_w = G_s \gamma_w - \gamma_w$$

$$= (G_s - 1)\gamma_w$$

$$D = \sqrt{\frac{18\mu\,(\text{poise})}{(G_s-1)\gamma_w \times 980.7} \times \frac{Z_r\,(\text{cm})}{t\,\text{min} \times 60}} \times 100 \quad \text{mm}$$

$$= \sqrt{\frac{\text{gm-sec} \times \text{cm}^3}{\text{cm}^2 \times \text{gm}} \times \frac{\text{cm}}{\text{sec}}}$$

$$= \text{cm}$$

$$D_{mm} = \sqrt{\frac{30\mu(\text{poise})}{(G_s-1)\,\underbrace{\gamma_w}_{g/cc}\times 980.7} \times \frac{Z_r(\text{cm})}{t(\text{min})}}$$

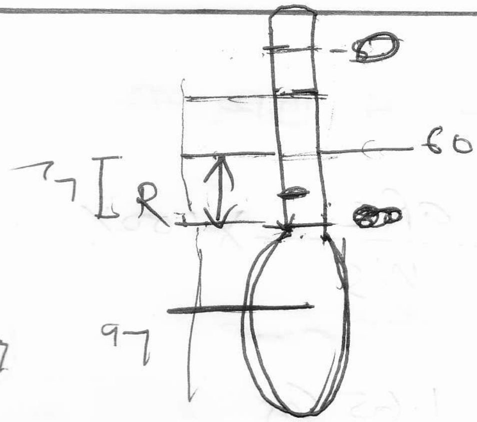
$$\cancel{\frac{18}{1000}}$$

$$D = \sqrt{\frac{3\mu(Ns/m^2)}{(G_s-1)\,\underbrace{\gamma_w}_{kN/m^3}} \times \frac{Z_r(\text{cm})}{t(\text{min})}}$$

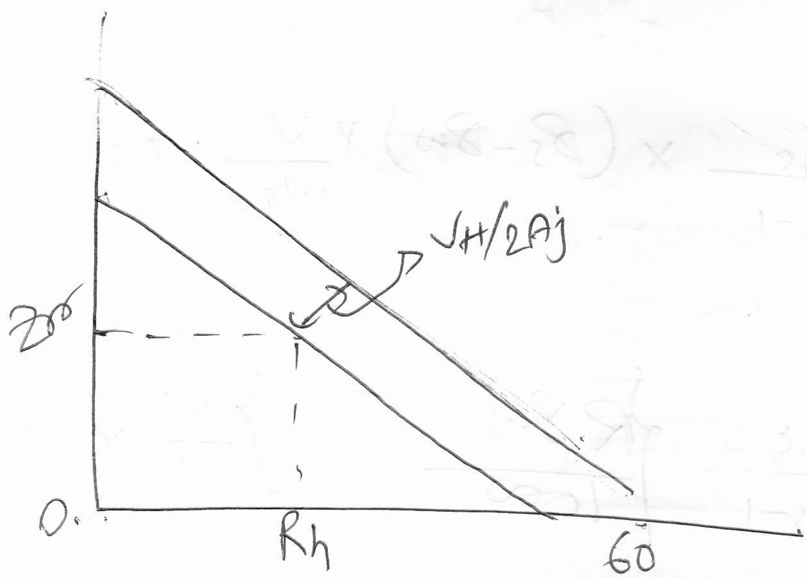
$\frac{V}{A}$

152H

% finer =



$\boxed{Z_r = 16.29 - 0.1641R} \rightarrow 152H$



152H

$$N = \frac{aRc}{Ws} \times 100\%$$

$$a = \frac{1.65\,G_s}{2.65\,(G_s - 1)}$$

$G_s > 2.65,\ a < 1$

$G_s < 2.65,\ a > 1$

$R_c = R_0 - R_{zero} \pm R_T$

151H →

$$N = \frac{G_s}{G_s - 1} \times (\gamma_s - \gamma_w) \times \frac{V}{W_s} \times 100\%$$

151H

$$N = \frac{G_s}{G_s - 1} \left\{ \frac{R\,\gamma_w}{1000} \right\} \frac{V}{W_s} \times 100\%$$

$$\boxed{N = \frac{G_s\,\gamma_w}{10\,(G_s - 1)} \frac{V}{W_s} R_c} \rightarrow 151H$$

Flocculation prevention এর জন্য Dispersing agent use করা।

$$R = 1000(G_s - 1)$$

$$G_s = R/1000 + 1$$

Zero correction always substract করতে হবে।

Temp কম হলে (20°C), বিয়োগ করতে Temperature correction

Temp বেশি হলে (20°C), যোগ করতে Temperature correction

$$\boxed{R_c = R - \text{Zero correction} + C_T (T > 20^\circ C)}$$

No Meniscus correction applied in case of R_c

(*) $G_s = 2.69$

$A_{jar} = 27.8\ cm^2$

$V_H = 58\ cm^3$

$\cancel{R}$ Z_r/L = 12.94 (Without immersion Correction)

$V = 1000\ cc$

$W_s = 50g$

Temp = 29°C, μ at 29°C = $\boxed{0.00818}$ poise

$C_D = 3.0$

$C_T = 3.05$

151H :

D in mm , N(%)

$$D = \sqrt{\frac{30\,\mu(\text{poise})}{980.7\,(G_s - 1)\,\gamma_w} \times \frac{Z_r(\text{cm})}{t(\text{min})}}$$

$$\gamma_w = 0.99598\ g/cm^3$$

$$= \sqrt{\frac{30 \times 0.00818}{980.7\,(2.69 - 1) \times 0.99598} \times \frac{11.9}{15}}$$

$$\boxed{\begin{aligned} Z_r &= Z_r' - \frac{V_H}{2A_j} \\ &= 11.9\ cm \end{aligned}}$$

$$\boxed{D = 0.01086\ mm}$$

$$R_s = R_{obs} - C_D + C_T$$

$$= 28 - 3 + 3.05$$

$$= 28.05$$

$$N = \frac{G_s \gamma_w}{10(G_s - 1)} \frac{V}{W_s} R_s$$

$$= \frac{2.69 \times 0.99598}{10(2.69-1)} \times \frac{1000}{50} \times 28.05$$

$$= \boxed{88.94\%}$$

States of a soil →

Liquid, plastic, solid, Semisolid → 4 States

Atterberg Limit test provides →

Liquid limit, Plastic limit, Shrinkage limit

Solid | Semi solid | Plastic | Liquid → Soil can flow at its own weight

Semi solid → Deform, Remold, with cracks

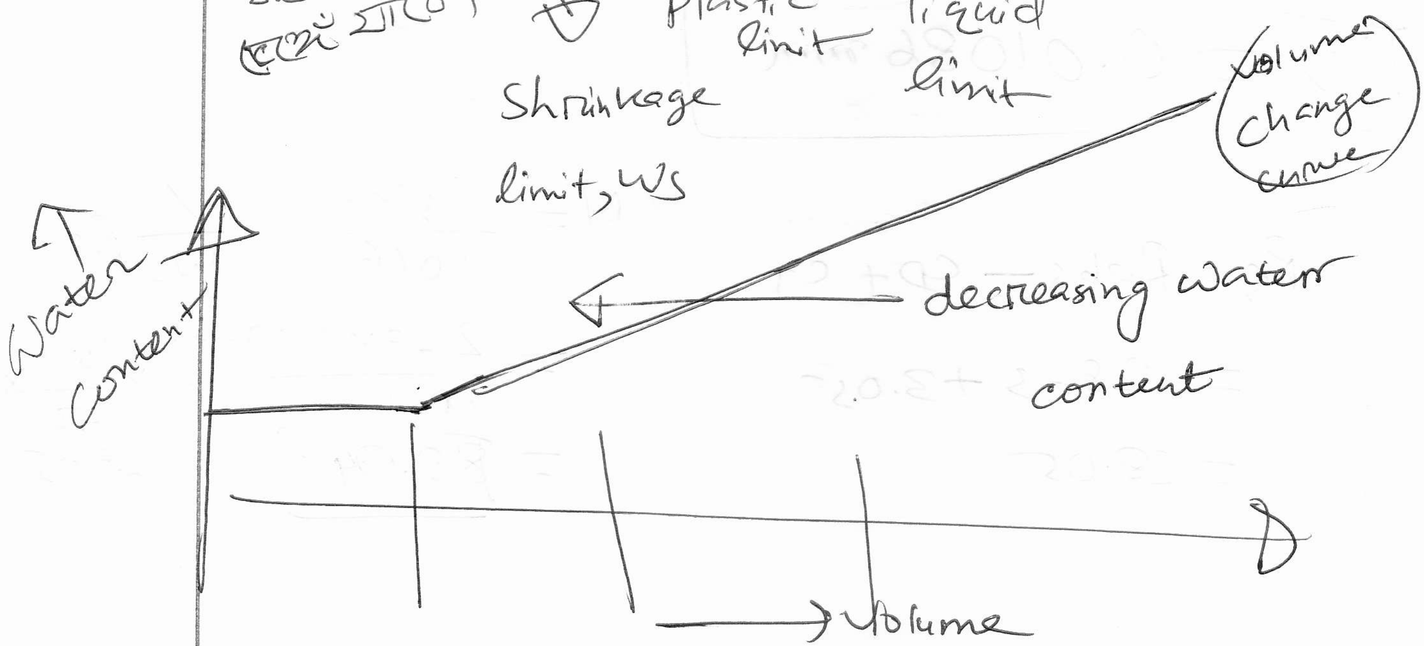
Plastic → Deform the soil, Mold the soil without any cracks

Solid → Remold করা যায়, deform করা যায়, ভেঙে যাবে।

SL → Shrinkage limit, w_s

PL → Plastic limit

LL → liquid limit



29/6/19

Siddique Sir:

Liquid limit is the maximum water content when the soil is at plastic state and the minimum water content when the soil is at the liquid state. It is the boundary between plastic and liquid state.

Similarly, PL, SL can be defined.

Shrinkage limit এর পরে Volume কমবে না Dry করা হলেও।

All these limits are water contents

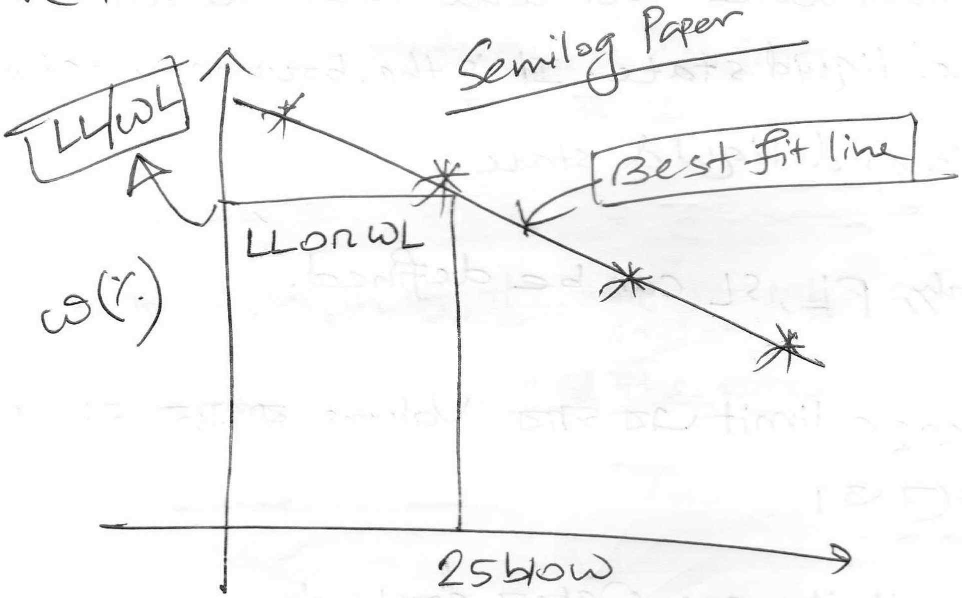
$$\frac{LL/PL/SL}{w_L/w_P/I_P}$$ ASTM 4718

Oven dry soil নিবো #40 sieve দিয়ে যে soil গুলো পাস pass করবে সেগুলোর Liquid limit, plastic limit, Shrinkage limit test করবো।

$$\text{Water content} = \frac{W_w}{W_s} \times 100$$

Casagrande's Method

25টা blow দিলে 1/2" Length এর close groove তৈরি হলে যেই water content তাকে Liquid limit বলে।



log N = Number of blow

Flow curve: Water content (w%) vs log N

slope of the flow curve is called flow index

Different soils have same liquid limit but different flow index.

Flow index কম, তাহ shear strength বেশি quickly কমবে

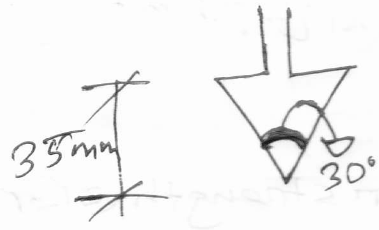
Flow index বেশি, তাহ shear strength কম quickly কমবে।

N	ω(%)
15 →	60.1
20 →	57.9
24 →	56.4
30 →	55.2
35 →	54.1

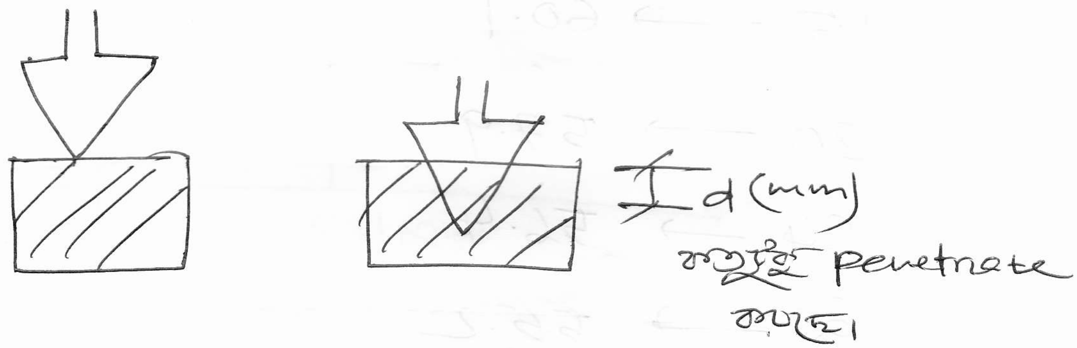
Fall cone method → British method

Weight of cone = 80gm = 0.78N

30° → Angle cone

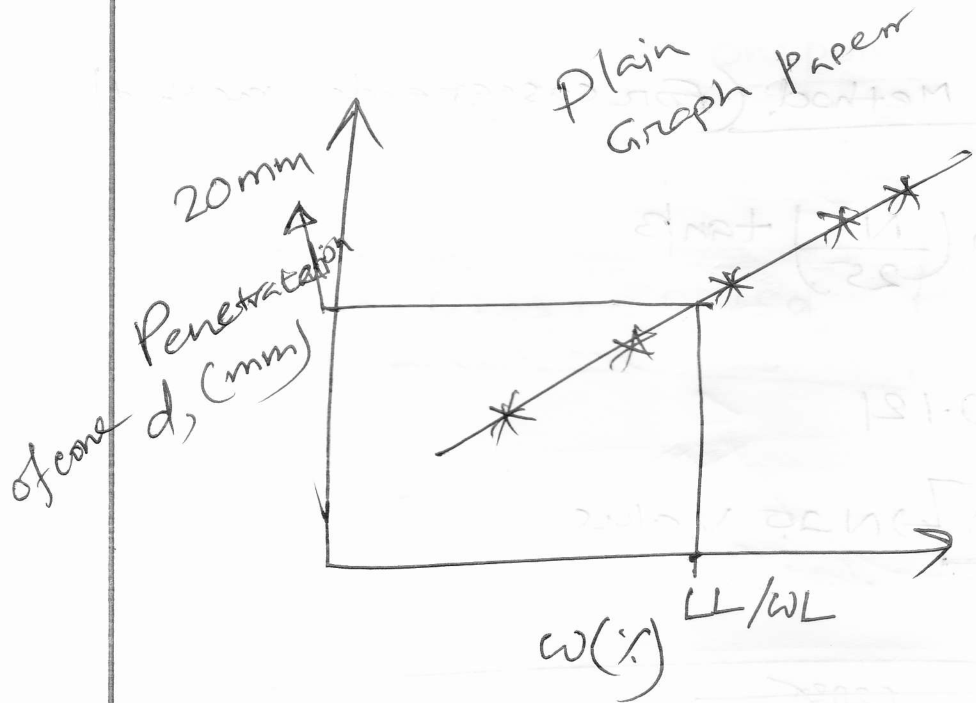


Cone কে free fall করালে এ সয়েলে কতটুকু penetrate করে measure করতে হবে।



Dial এর initial and final reading এর difference থেকে d পাব।

20mm in 5sec এ penetrate করতে যে water content লাগে তাকে Liquid limit বলে।



Fall cone method

d(mm)	w(%)
16 →	33
18 →	40
22 →	54
25 →	63
30 →	83

Casagrande's method can be checked by ~~f~~ one point methods —

One point Method: (For casagrande's method)

(i) $w_L = w\left(\frac{N}{25}\right)\tan\beta$

$\tan\beta = 0.121$

$\boxed{(20-30)}$ → N 25 value

~~(25~~

$\boxed{N \to 15 \quad 35}$

(ii) $$w_L = \frac{w}{1.3215 - 0.23\log N}$$

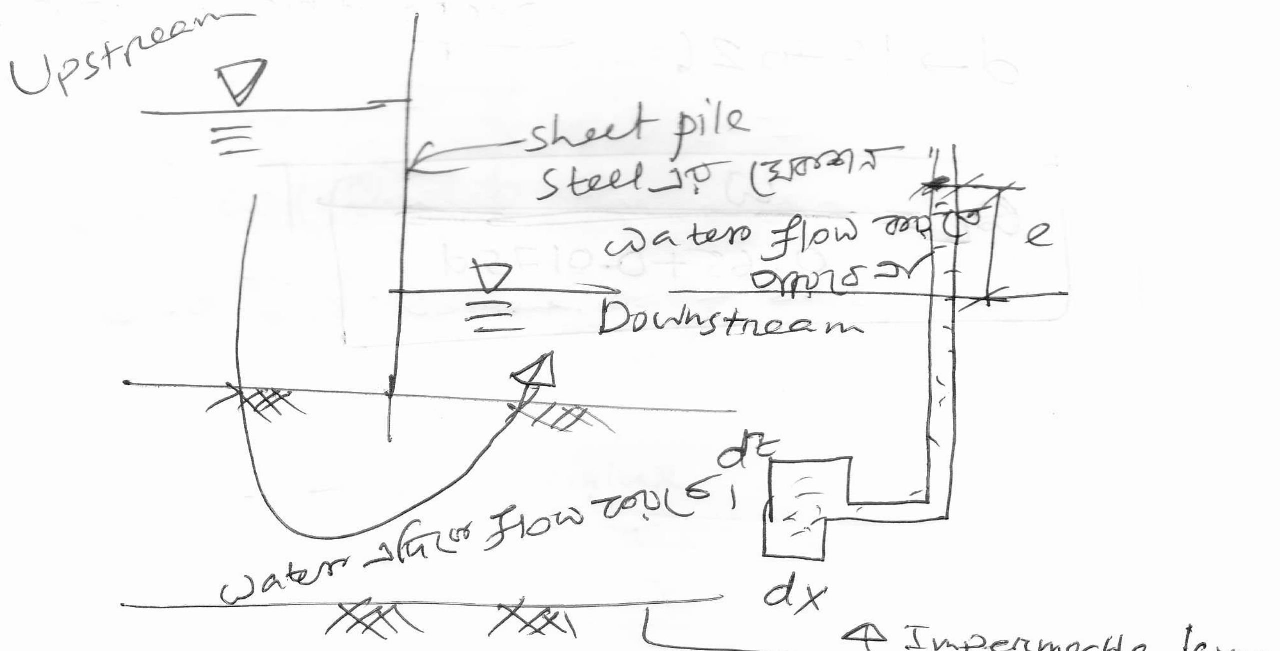
~~$w_L = \frac{w}{0.65}$~~

Cone penetrator / fall ~~#~~ cone method can be checked by one point method —

$d \to 16$ to 26

one point method for fall cone method

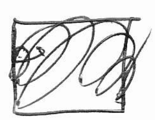
$$\boxed{\omega_L = \frac{\omega}{0.65 + 0.0175d}}$$



$\boxed{\text{Sheet pile}}$ → very expensive

e → Driving head

2-dimensional flow problem



$$\left(\frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z}\right) dx . dy dz = 0$$

$dx . dy . dz \neq 0 \Rightarrow$ volume can't be zero.

$$\therefore \frac{\partial V_x}{\partial x} + \frac{\partial V_z}{\partial z} = 0$$

$$V_x = K_x i_x$$

$$V_z = K_z i_z$$

$$K_x = K_z = K \text{ (isotropic)}$$

$$V_x = K \frac{\partial h}{\partial x}$$

$$V_z = K \frac{\partial h}{\partial z}$$

$$k\left\{\frac{\partial}{\partial x}\left(\frac{\partial h}{\partial x}\right) \times \frac{\partial}{\partial z}\left(\frac{\partial h}{\partial z}\right)\right\}=0$$

$$\nabla^2 h=0 \qquad \frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial z^2}=0$$

Flow through porous media → Laplace eqn.

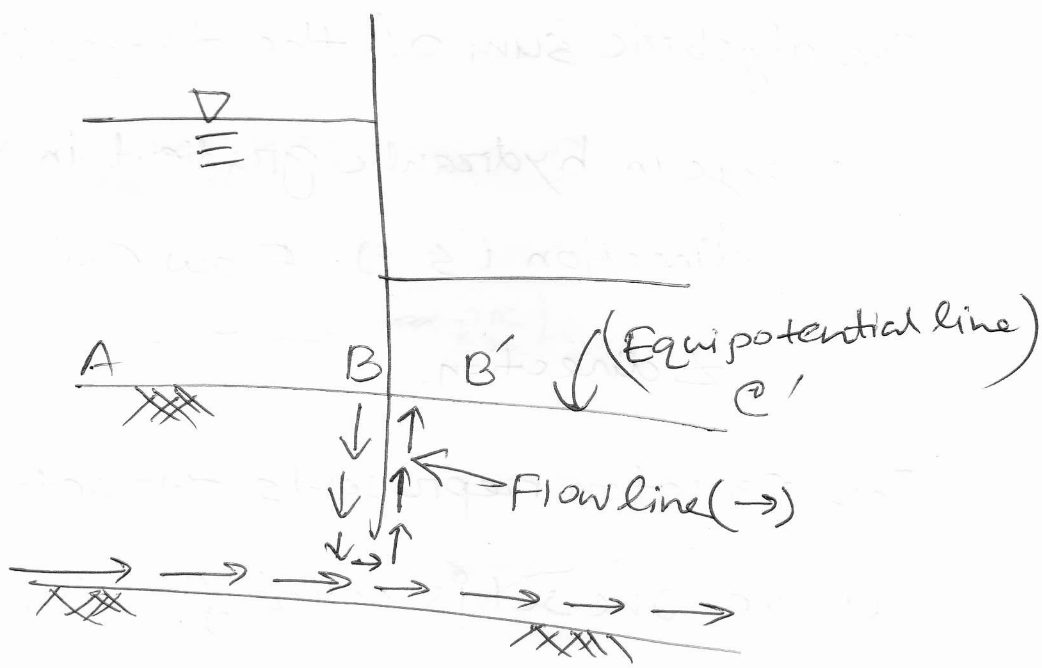
The algebric sum of the ~~tangent of~~ change in hydraulic gradient in X and Z direction is 0. Flow occurs in X and Z direction.

This equation represents two sets of orthogonal curve. One set is called flow line (Streamline) and another set is called equipotential line

Flow line is the line along which water particles move from upstream to downstream.

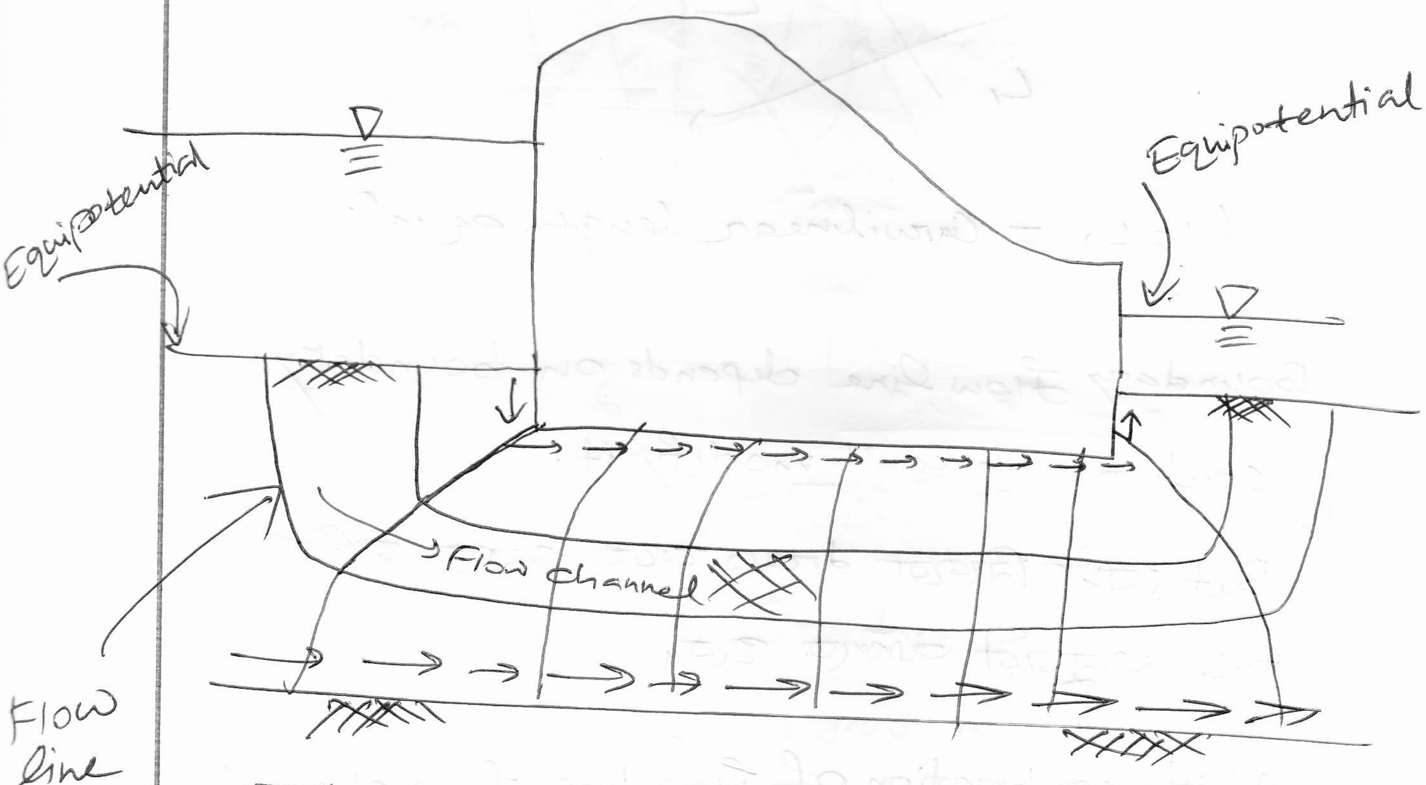
Equipotential line is the line, along which all piezometric readings will be the same.

Identification of boundary conditions:



Boundary কে Equipotential line ভাবো।

The region between two successive flow lines is called flow channel.

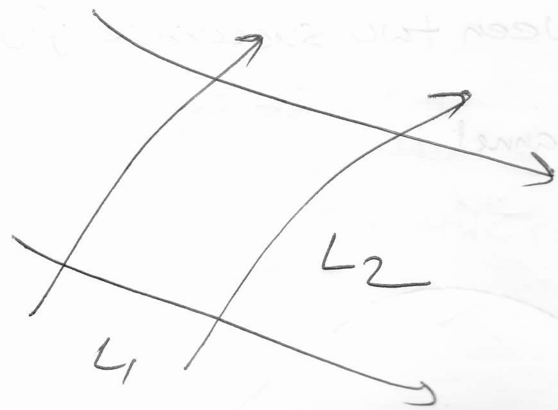


Flow line at the boundary

→ Flow element

The region bounded between two successive flow line and flow line is called flow element.

Flow element generally curvilinear square 2010



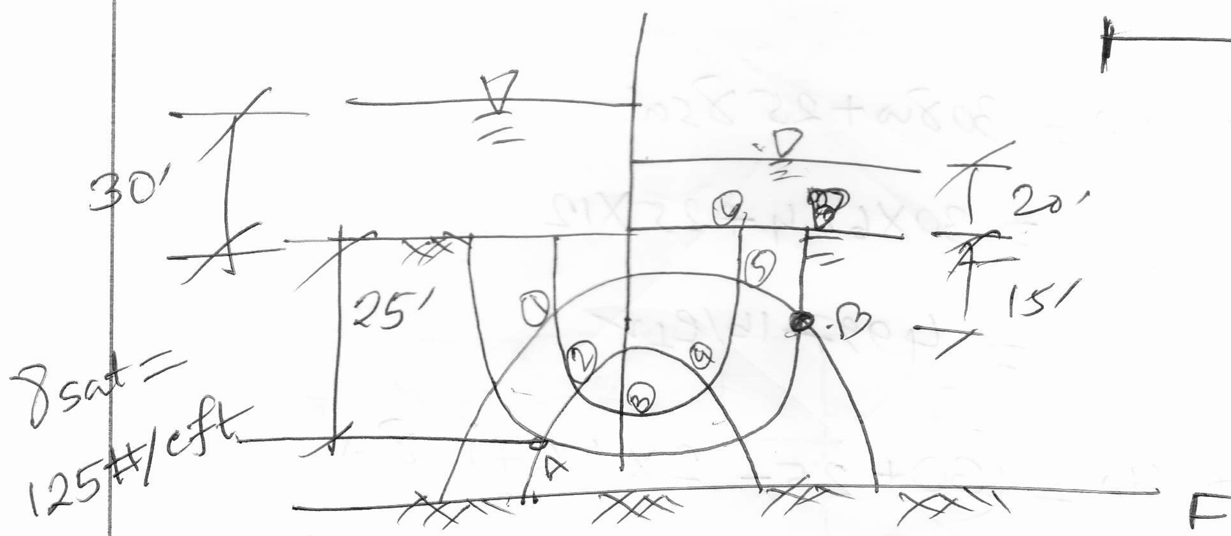
$L_1 = L_2$ = Curvilinear length equal

Boundary flow line depends on boundary conditions. We can't modify it.

But যেটা নিজেরা draw করব ওখানে অবশ্যই rules অনুযায়ী আঁকতে হবে।

At the combination of flow line, flow channel, equipotential line, flow element is called flow net diagram.

Scale dimension



$$\frac{30-20}{6} = 1.67'$$

1) Scale ঠিক করো (Draw the problem)

ii) Identify the boundary condition.

iii) Flow line draw করো (characteristics maintain করে হবে)
(2-3 টা)

iv) Equipotential line flow line এর perpendicular করে Draw করো।

$$P_A = 30\gamma_w + 25\gamma_{sat}$$

$$= 30 \times 62.4 + 25 \times 12$$

$$= 4997\ lb/ft^2$$

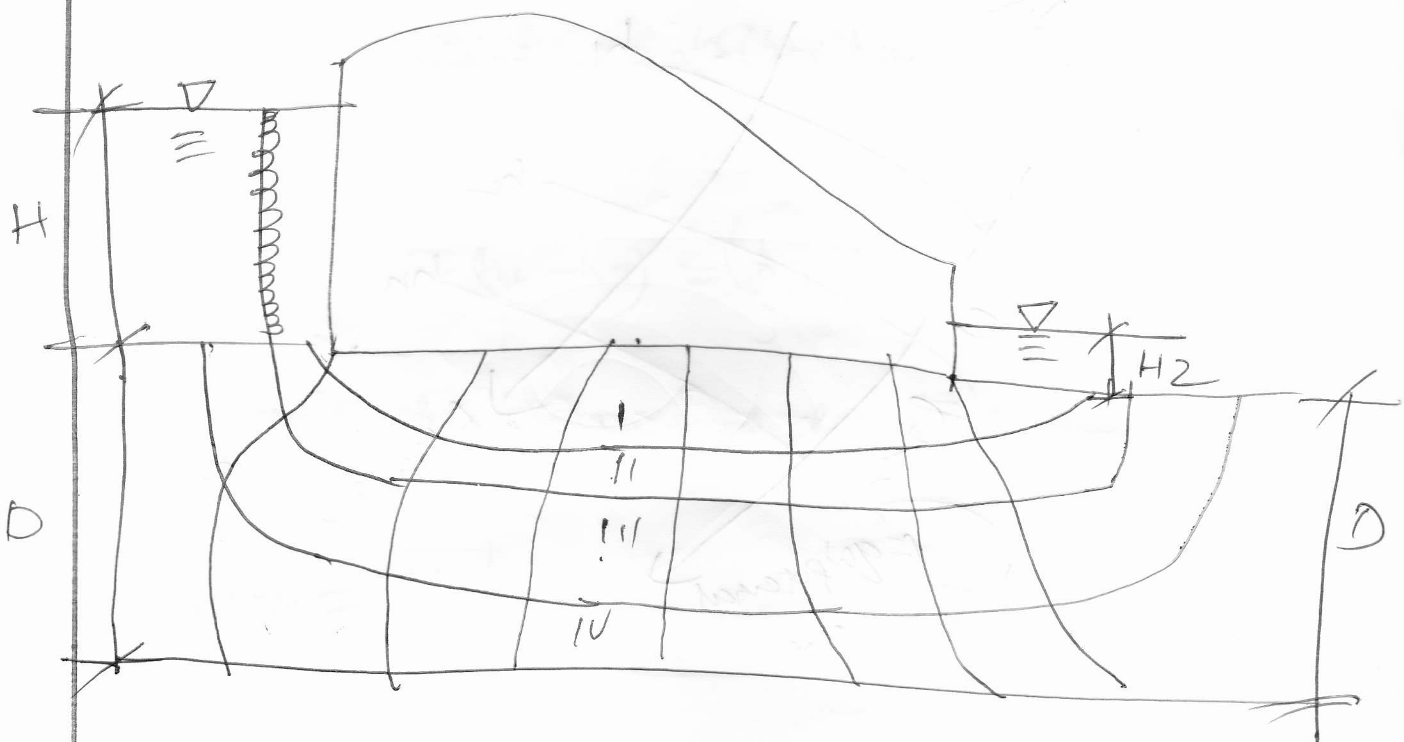
Neutral Stress $\rightarrow$ $$u_A = (30 + 25 - 2 \times 1.67)\ \gamma_w$$

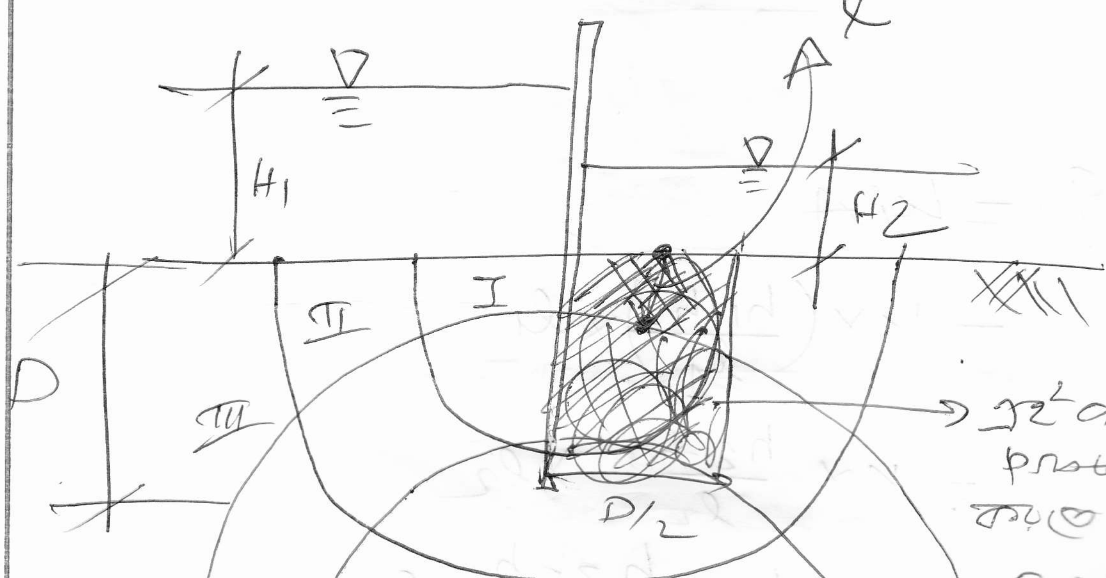
$$= 3223.58\ lb/ft^2$$

$$P_A' = P_A - u_A$$

$$P_B = 20\gamma_w + 15 \times \gamma_{sat}$$

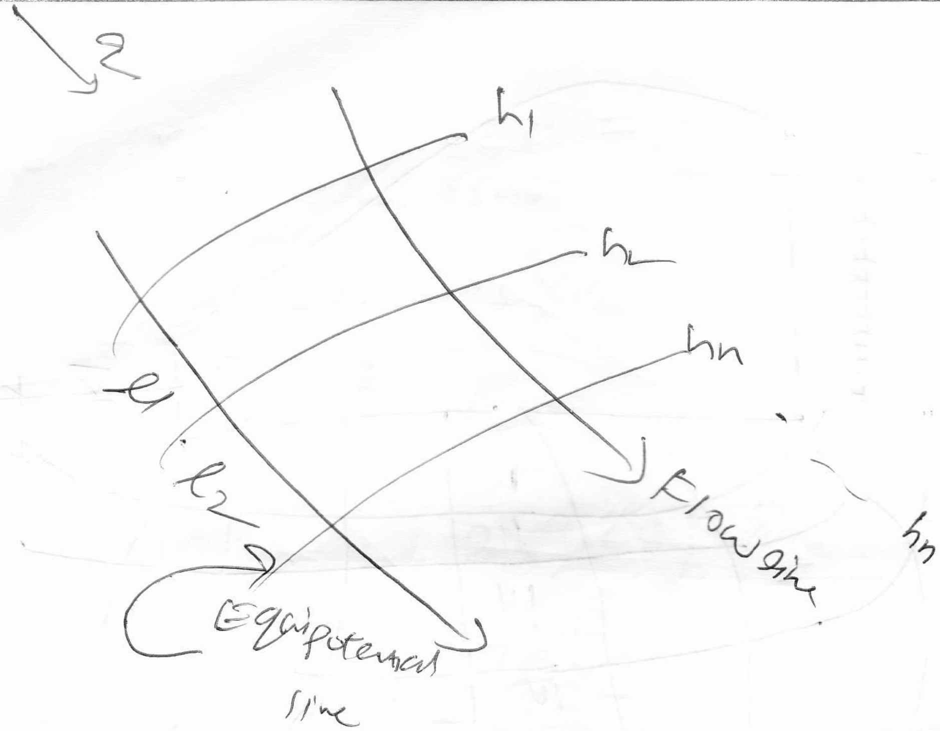
$$u_B = 35\gamma_w + 1.67\gamma_w$$





এই area protect করতে হবে Quick sand condition exist না করে।

2



No flow could occur across the channel.

$$q = kiA$$

$$= k \times \left(\frac{h_1 - h_2}{l_1}\right) \times l_1$$

$$= k \times \frac{h_2 - h_3}{l_2} \times l_2$$

$$= k \times \frac{h_3 - h_4}{l_3} \times l_3$$

$$q = K(h_1 - h_2) = K(h_2 - h_3) = K(h_3 - h_2)$$

$$(h_1 - h_2) = (h_2 - h_3) = (h_3 - h_2)$$

প্রতিটি element এ equal drop হবে।

$$\boxed{N_D = 11}$$

$$H_1 - H_2 = H$$

$$q = \frac{K \times H}{N_d}$$

$N_F \rightarrow$ Number of flow channel.

$$q = \frac{K \times H \times N_f}{N_d}$$

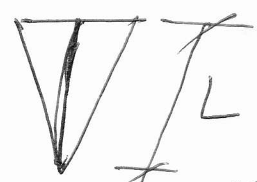
Quantity of water flowing from upstream to downstream / seepage loss / loss

Exit gradient, $i_{ext} = \frac{h}{2}$

$\boxed{i_{exit}} < 1 \, (I_c)$

└→ Factor of safety (3–4) 0.25–0.33

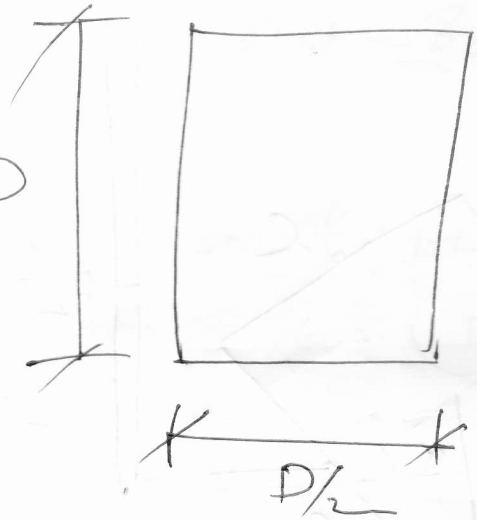
<u>Harza's criteria</u>



Triangular area rite

Sheet pile → only → Tarzaki's criteria

D → Depth of embankment

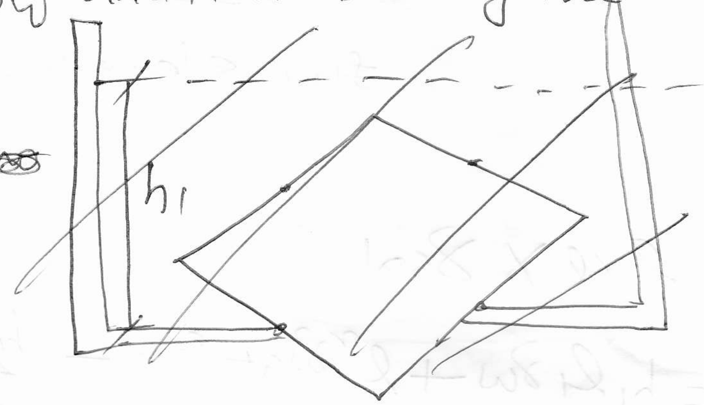


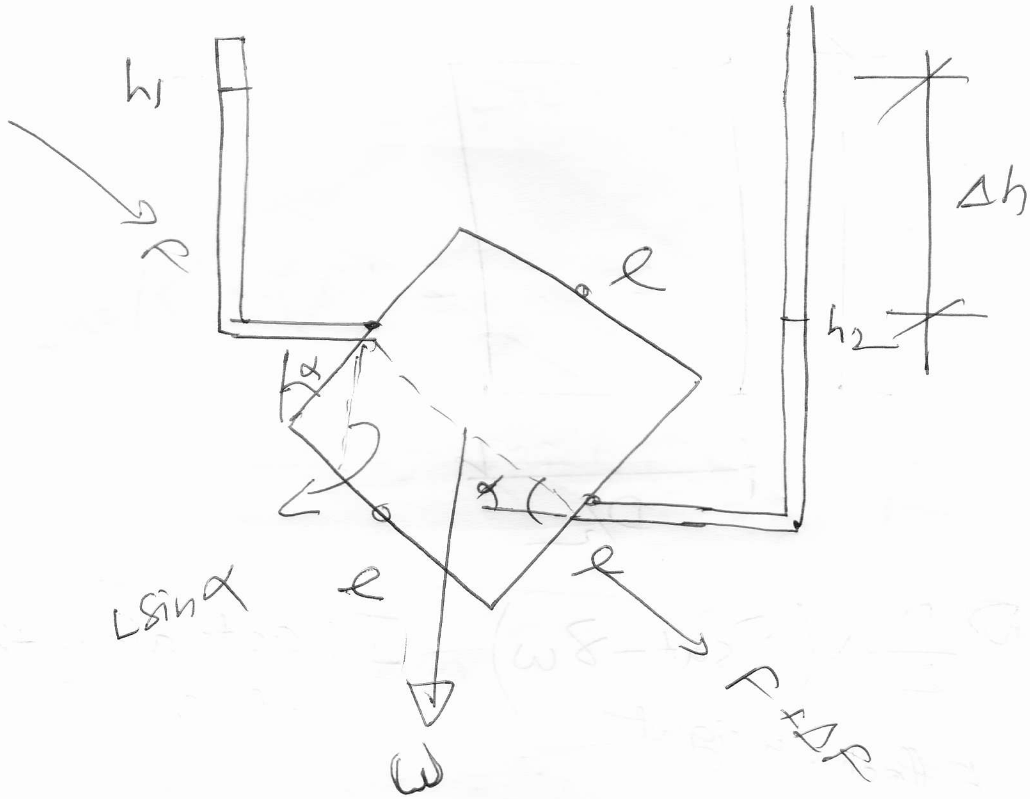
∇ $\cancel{D}\, \frac{D}{2} \times (\gamma_{sat} - \gamma_w)$ It acts along the direction of flow

Effective weight

=

Seepage force per unit volume of soil sample is $i\gamma_w$ and it acts along the direction of flow.





F → Hydrostatic force

W → Effective weight of flow element

$$W = \ell \times \ell \times \gamma_{sat}$$

$$\Delta F = h_1 \ell_1 \gamma_w + \ell^2 \gamma_{sat} \sin\alpha - h_2 \ell \gamma_w$$

$$\boxed{h_1 + \ell \sin\alpha = h_2 + \Delta h}$$

$$\boxed{h_2 = h_1 + \ell \sin\alpha - \Delta h}$$

$$\Delta F = l^2(\gamma_{sat} - \gamma_w)\sin\alpha + \Delta h\, \gamma_w\, l$$

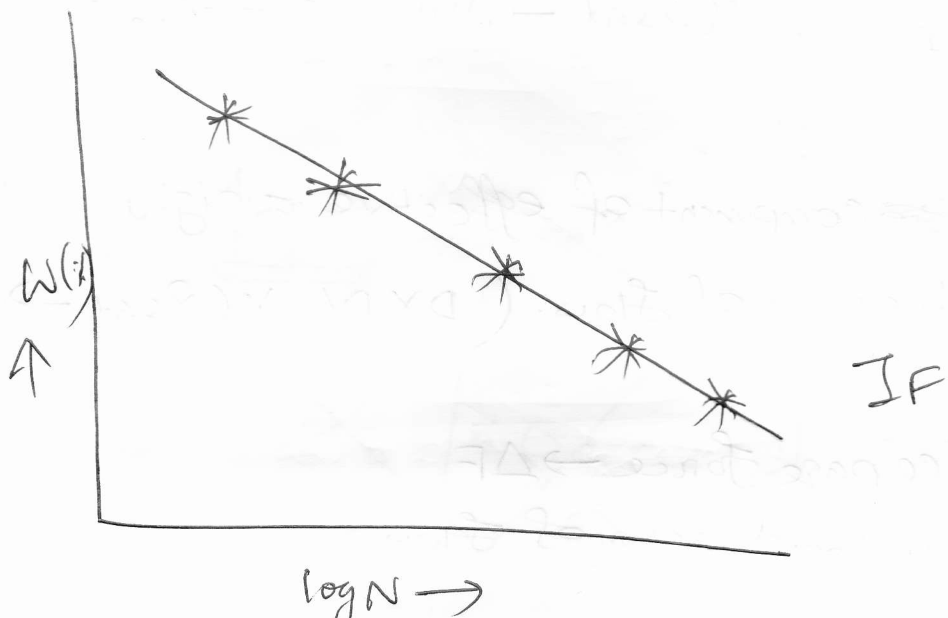
~~ΔF~~ component of effective weight in the direction of flow. $\left(D \times D/2 \times (\gamma_{sat} - \gamma_w)\right)$

Seepage force $\rightarrow \Delta F$
in the direction of flow

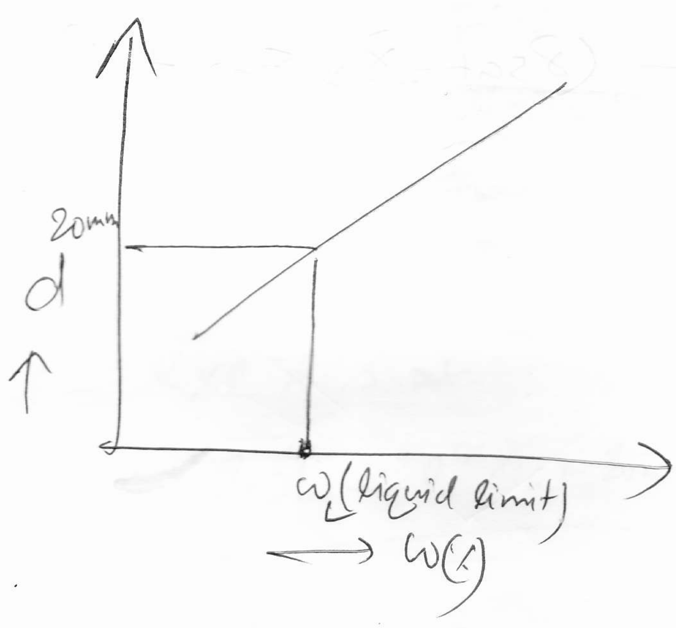
Seepage force per unit volume =

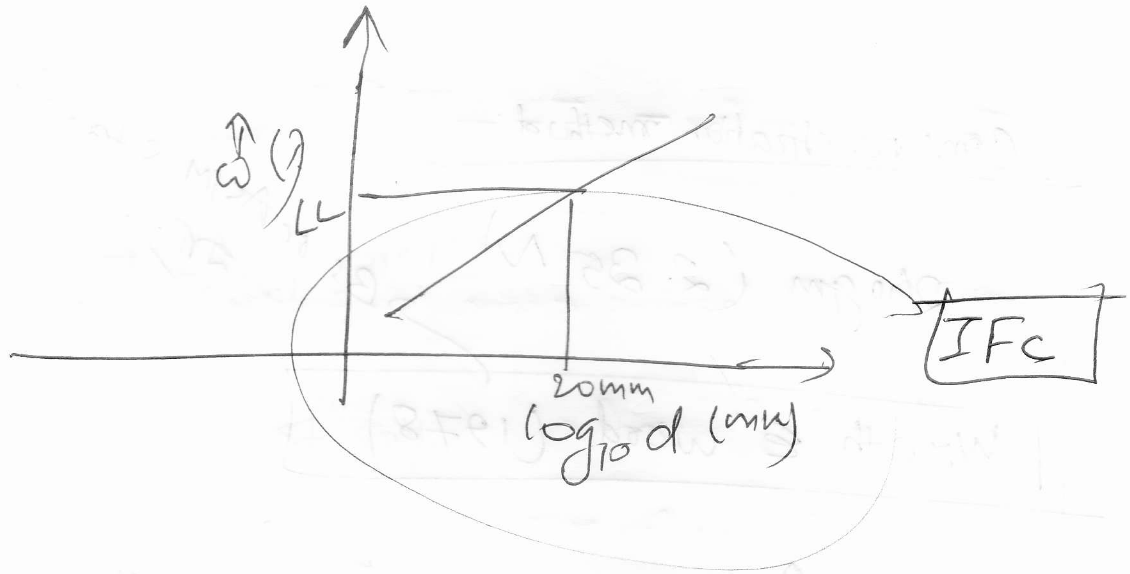
$$(\gamma_{sat} - \gamma_w)\sin\alpha + \frac{\Delta h}{l} \times \gamma_w$$

$$= (\gamma_{sat} - \gamma_w)\sin\alpha + i\gamma_w$$



$\boxed{\text{Cone 80g (0.78 N)}}$





Plasticity index:

Which is measure of plasticity of soil.

NP → Non plastic soil (Sand)

Plasticity index = 0

PI → classification of soil based on plasticity index

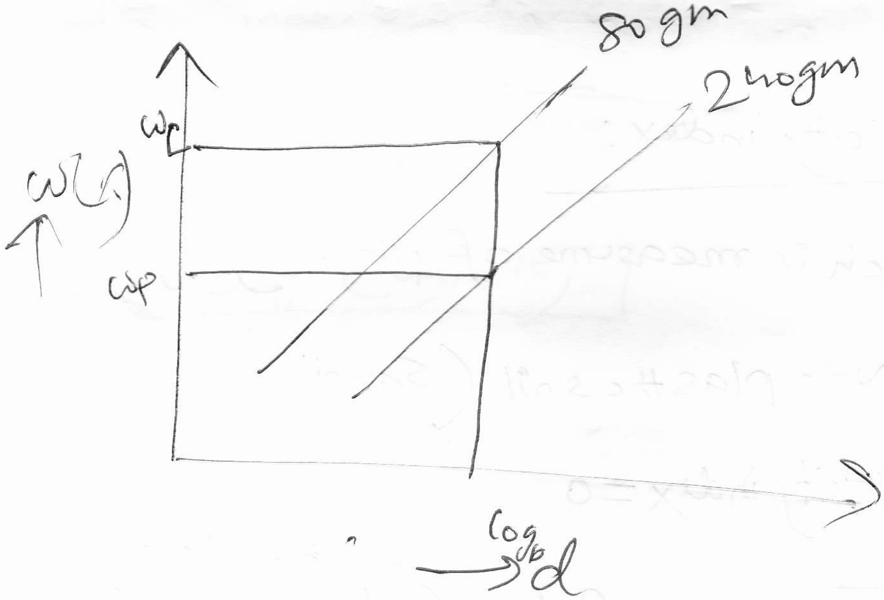
ওই এ দেখো।

Atkins 1997

Cone penetration method;

240gm (2.35 N)

$\boxed{\text{Wroth \& Wood (1978)}}$ → BS or ASTM standard ar;



$\longrightarrow$ 1) I_F

$\longrightarrow$ 2) I_p or PI

Plasticity of soil.

Liquidity index :

$$\text{Liquidity index} = \frac{NMC - PL}{LL - PL}$$

$NMC = PL$, $LI = 0$

$NMC = LL$, $LI = 1$

0.07

Toughness index, $I_T = \dfrac{PI\ (I_P)}{I_f}$

Two soils having same plasticity index, but different flow index.

যার flow index কম হয় বেশি tough হবে;

Shear strength বেশি হবে।

clay < 0.002 mm

Sand, silt, clay থাকলে 4 cycle

Activity/ Sensitivity

Unconfined compression test

Unconfinded compressive strength for undisturbed streng

Unconfined " " " disturbed/ remolded strength

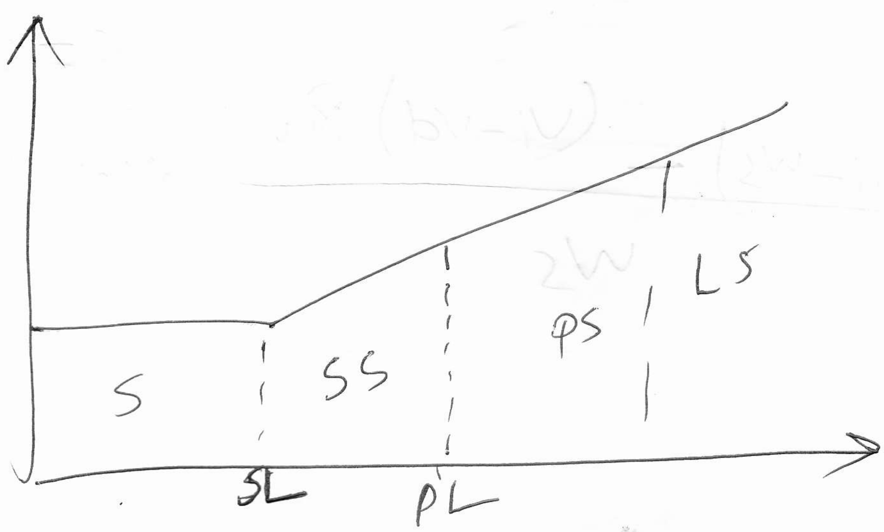
$$I_c = \frac{LL - W_n}{LL - PL}$$

consistency index

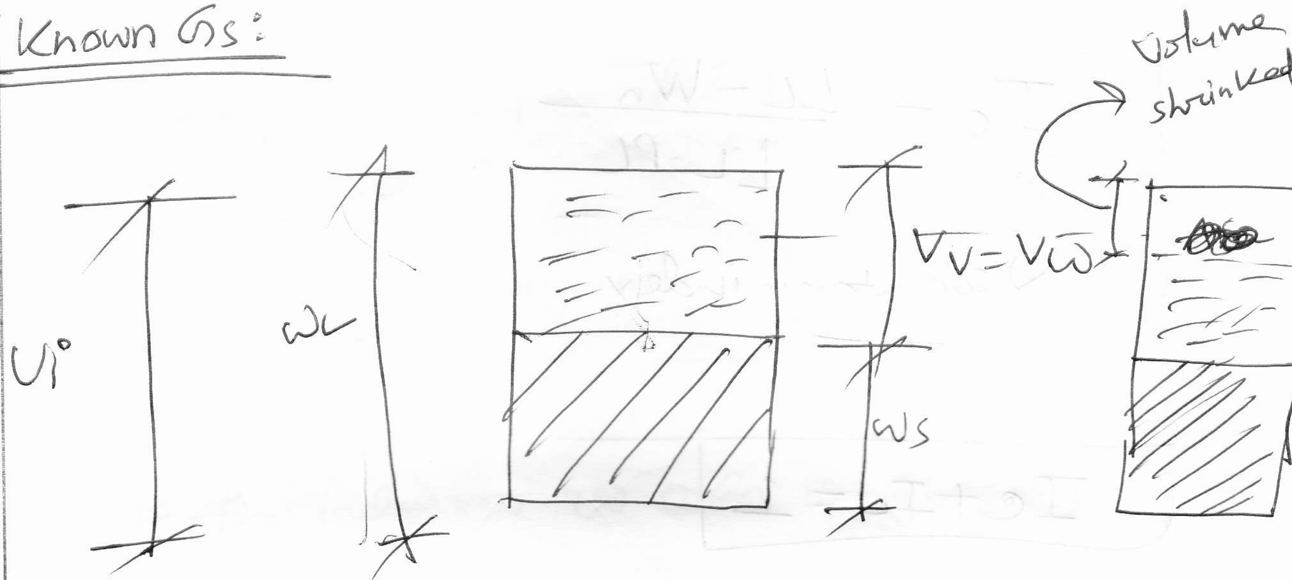
$$\boxed{I_c + I_L = 1}$$

$\boxed{\text{B-Value}}$ or,

Water plasticity ratio, I_L = Liquidity index

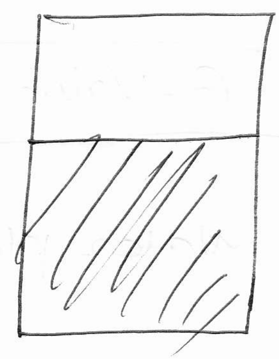


Known G_s:



$\boxed{w > w_s}$

Unknown G_s:



$w = 0$

$$= \frac{(W_i - W_s) - (V_i - V_d)\gamma_w}{W_s} \times 100\%$$

Known G_s:

$$SL = \frac{(V_d - V_s)\,\gamma_w}{W_s}$$

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

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

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

$$\boxed{SL = \frac{\left(V_d - \frac{W_s}{G_s \gamma_w}\right) \times \gamma_w}{W_s}}$$

$$= \frac{V_d \gamma_w - \frac{W_s}{G_s}}{W_s}$$

W_s

$$= \boxed{\frac{V_d \gamma_w}{W_s} - \frac{1}{G_s}}$$

$$= \frac{1}{R} - \frac{1}{G_s}$$

$$R = \frac{W_s}{\gamma_w V_d}$$ (shrinkage ratio)

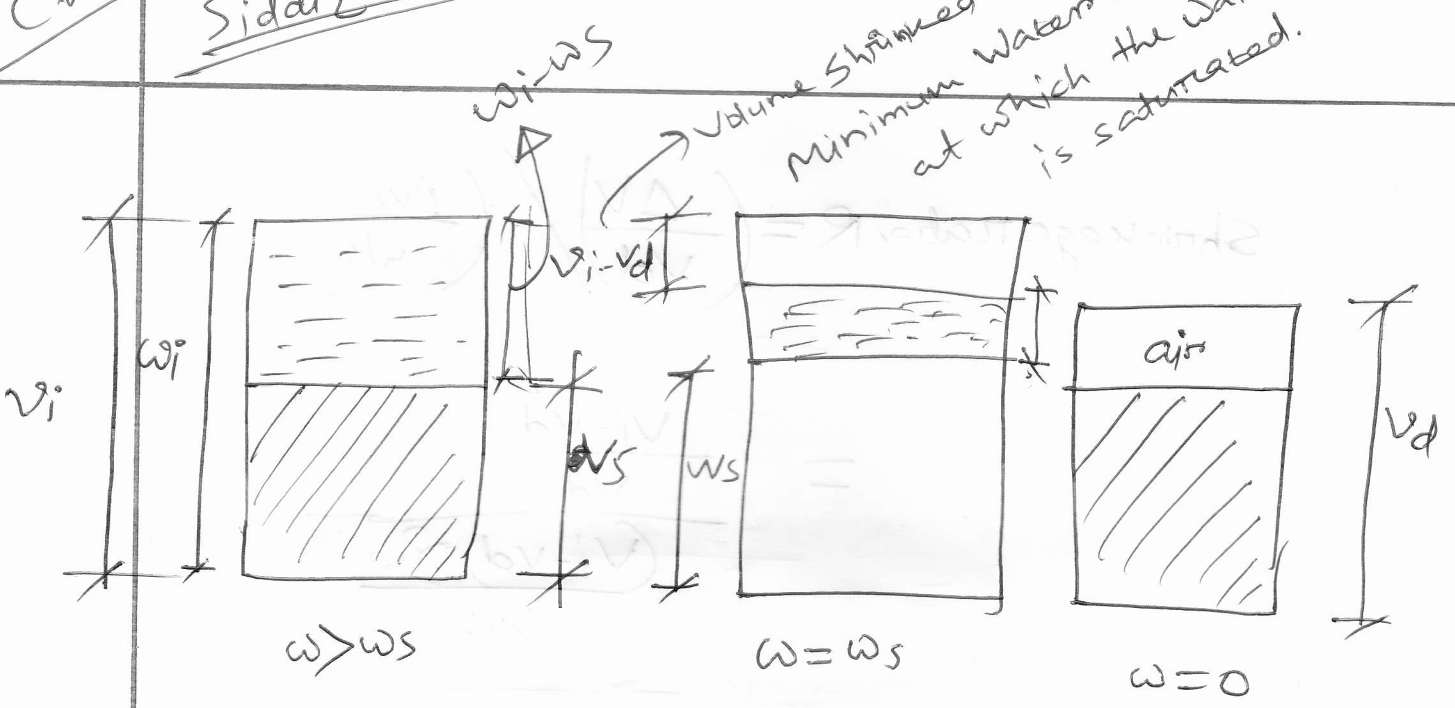
$$SL = \left[\frac{1}{R} - \frac{1}{G_s}\right] \times 100$$

$$\cancel{\frac{SL}{100}} = \cancel{\frac{1}{\rho}}$$

$$\frac{1}{G_s} = \frac{1}{R} - \frac{SL}{100}$$

$$\Rightarrow \boxed{G_s = \frac{1}{\frac{1}{R} - \frac{SL}{100}}}$$

C.W. Siddique Sir



(*) Derive the equation of shrinkage for known G_s and Unknown G_s.

$$\omega_s = \frac{\left[(W_i - W_s) - (V_i - V_d)\gamma_w\right]}{W_s} \times 100$$

$$\omega_s = \omega_i(\%) - \frac{(V_i - V_d)\gamma_w}{W_s} \times 100$$

$$\omega_s(\%) = \frac{(V_d - V_s)\gamma_w}{W_s} \times 100 = \left[\frac{V_d\gamma_w}{W_s} - \frac{1}{G_s}\right] \times 100$$

$$\gamma_s = G_s\gamma_w$$

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

$$\text{Shrinkage ratio, } R = \left(\frac{\Delta V}{V_d}\right) \Big/ \left(\frac{\Delta w}{w_s}\right)$$

$$= \frac{\dfrac{V_i - V_d}{V_d}}{\dfrac{(V_i - V_d)\,\gamma_w}{W_s}}$$

$$= \boxed{\frac{W_s}{V_d\,\gamma_w}}$$

$$\cancel{\frac{1}{G_s} = \left[\frac{1}{R} - \frac{1}{G_s}\right] \times 100}$$

$$\Rightarrow \cancel{G_s = \frac{1}{\left(1/R - 1/G_s\right) \times 100}}$$

$$\boxed{G_s = \frac{1}{1/R - \dfrac{w_s(\%)}{100}}}$$

LL or ω_L

PL or ω_p

* $PI/I_p = LL - PL$

* Flow index

* Toughness Index $= \dfrac{I_p}{I_F}$

* Activity $= \dfrac{PI}{\text{Clay content}}$ → less than 0.002 mm as per MIT, BS Standard, AASHTO

* Liquidity Index or B value or water plasticity ratio $= \dfrac{NMC - PL}{LL - PL}$

Water plasticity ratio কম হলে soil is very stiff

বেশি হলে soil is soft

Consistency index, $I_c = \dfrac{LL - \omega_n}{LL - PL}$

Relative ~~cont~~ consistency

$I_L + I_c = 1$ or 100%

(1)

$$\gamma_{db} = \frac{\gamma_{bulk\,b}}{1+w} = \frac{19.1}{1+95/100} = 17.44\ kN/m^3$$

$$\frac{V_b}{V_f} = \frac{1+e_b}{1+e_f} = \frac{\dfrac{G_s\gamma_w}{\gamma_{db}}}{\dfrac{G_s\gamma_w}{\gamma_{df}}}$$

$$= \frac{\gamma_{df}}{\gamma_{db}} = \frac{71.5}{17.44}$$

$$V_b = V_f \times \frac{71.5}{17.44}$$

$$= 42000 \times \frac{71.5}{17.44}$$

$$= 17\ 2190.367\ m^3$$

$$\text{Truckload} = \frac{67.5}{4.5} \times 172190.367$$

$$= 2582855.5\ kN$$

(e)

$$\gamma_{bulk}(@\, 9.5\%\, MC) = \gamma_d(1+w)$$
$$= 71.5 \times \left(1 + \frac{9.5}{100}\right)$$
$$= 78.29\ kN/m^3$$

$$\gamma_{bulk}(@\, 15\%\, MC) = \gamma_d(1+w)$$
$$= 71.5\left(1 + \frac{15}{100}\right)$$
$$= 82.225\ kN/m^3$$

γ_d

$W_w = w\,(W_s)$

Weight of water added per m^3 of soil $= (82.225 - 78.29)\ kN$

$= 3.935\ kN$.

$\therefore$ for $4.5\,m^3$ (1 truck Load), weight of water added $= 3.935 \times 4.5$

$= 17.7075\ kN$

Volume of water added $= \dfrac{17.7075}{9.81} = 1.805\ m^3$

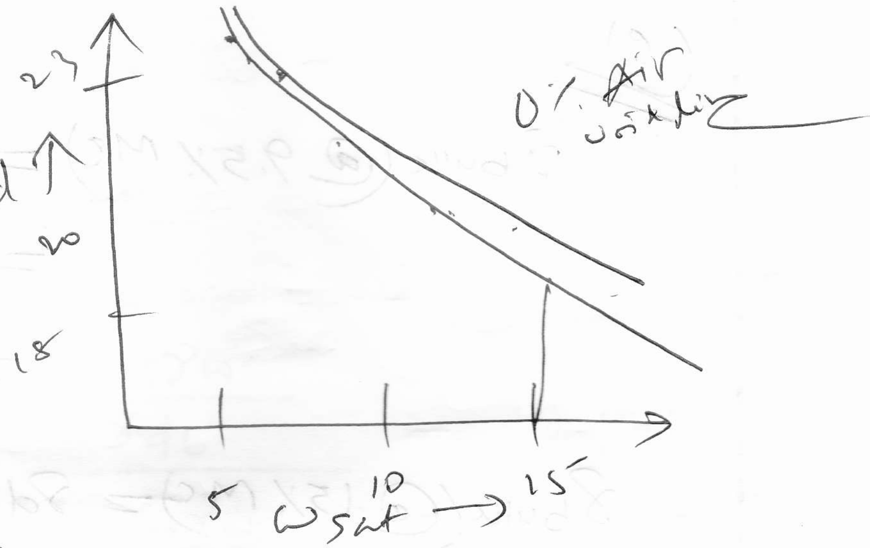
$= 1805.04$

$\boxed{4.5 m3} \rightarrow$

67.5 kN

(9)

63000 $\gamma d \uparrow$



$S_r = 1 = \frac{\omega G_s}{e}$

$\boxed{\omega_{sat} G_s = e}$

$\gamma d = \frac{G_s \gamma_w}{1+e}$

$= \frac{G_s \gamma_w}{1 + \omega_{sat} \times G_s}$

$= 23$

20.6

18.7

Volumetric shrinkage, $V_s (\%) = \frac{V_i - V_d}{V_d} \times 100$

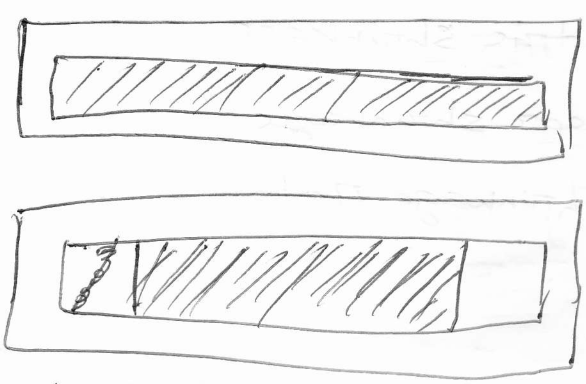
$$V_s = R\,[\,w_i(\%) - w_s(\%)\,]$$

= Shrinkage ratio [water content initially − shrinkage limit]

Shrinkage limit (বেশি হওয়া ভালো, কম খারাপ,

Linear shrinkage (BS Method) বেশি হলে খারাপ, কম হলে ভালো।

Linear shrinkage = $\frac{\text{Change in length}}{\text{Original length}}$



$$L_s = 100\left[1 - \sqrt[3]{\frac{100}{100 + V_s(\%)}}\right]$$

Degree of shrinkage, $S_d = \left(\frac{V_i - V_d}{V_i}\right) \times 100$

Sd (limits)

~~Sta~~ (Schedig's Classification)

Good < 5%

(5-10)% → Medium soil

(10-15)% → ~~Medium~~ poor soil

More than 15 → Very poor soil

$\boxed{\text{Shrinkage limit}}$

→ Degree of shrinkage

→ Volumetric shrinkage

→ Linear shrinkage

→ Shrinkage ratio

Problem.

$62.5\ cm^3 = V_i$

$117.3\ gm = W_i$

$47.8\ cm^3 = V_d$

$85.6\ gm = W_s$

1) Shrinkage limit, (i) G_s, iii) V_s, iv) S_d,

v) L_s

(i) $SL = w_i(\%) - \frac{(V_i - V_d)\gamma_w}{W_s} \times 100$

$= \frac{117.3 - 85.6}{85.6} \times 100 - \frac{(62.5 - 47.8) \times 1 \times 100}{85.6}$

$= 19.86$

(ii) $R = \frac{W_s}{V_d \gamma_w} = \frac{85.6}{47.8 \times 1} = 1.8$

$V_s = \frac{V_i - V_d}{V_d} \times 100 = 30.75\%$

~~γ_{bulk}~~

Dish + Saturated soil = 38.78 g = 28.13 gm

Dish + oven dried soil = 30.46 g = 19.81 gm
(cake)

wt of dish = 10.65 g

Vol. of dish = 16.29 cm³ = V_i

Vol. of ovendry soil cake = 10 cm³ = V_d

<u>find:</u> $\boxed{w_s, R, G_s, V_s, L_s}$

#200 sieve দিয়ে More than 50% soil pass করে ~~[illegible]~~ গেলে fine grained, More than 50% retained হলে coarse grained.

50% এর ~~[illegible]~~ উপরে #4 sieve দিয়ে pass করলে coarse fraction হবে sand

50% এর উপরে #4 sieve এ retained হলে coarse fraction হবে gravel.

SIDDIQUESIR:

GW → Gravel, Well graded

GP → Gravel, poorly graded

SW → ~~Silt~~ sand, well graded

SP → ~~Silt~~ sand, poorly graded

$\Big\}$ < 5% fine

$Cu \geq 4,\ 1 \leq Cz \leq 3 \longrightarrow GW$

$Cu \geq 6,\ 1 \leq Cz \leq 3 \longrightarrow GP$

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

$$Cz = \frac{(D_{30})^2}{D_{60} \times D_{10}}$$

Well graded Gravel → less than 15% Sand

Well graded gravel with sand → 15% or more sand

Poorly graded sand → less than 15% gravel

Poorly graded sand with gravel → ~~[illegible]~~ 15% or more gravel

ASTM ~~D1557~~

AASHTO → T180

Std. Proctor test →

ASTM D698, AASHTO T99

soil is compacted in mold having volume $\frac{1}{30}$ ft^3.

The diameter of the mold is 4". During the lab test, the mold is attached to a baseplate at the bottom and to an extension at the top.

· compacted in 3 layers by a hammer that delivers 25 blows to each layer.

$\boxed{5.5 \text{ lb}}$ and a drop of 12"

$$\gamma_{bulk} = \frac{W}{V_m} = \frac{W}{1/30} = \boxed{W \times 30}$$

weight of compacted soil

$$\text{Compactive effort} = 5.5 \times \frac{12}{12} \times 25 \times 3 \times 30$$
$$= 12375 \text{ lb-ft/cft}$$

$$\text{Compactive effort} = 10 \times \frac{18}{12} \times 25 \times 5 \times 30$$
$$= 56250 \text{ lb-ft/cft}$$

CW.

Siddique Sir—

GC-GM → Silty clayey gravel

SC-SM → Silty clayey sand

Granular Material:

Silt clay material:

$\boxed{\text{AASHTO M145}}$

Granular material ≤ 35% passing #200

More than 35% passing #200 → Silt clay Material:

% passing (% finer)
#10 (2.0 mm)
#40 (0.425 mm)
#200 (0.075 mm)

$\boxed{LL, PL}$

$\boxed{PI = LL - PL}$

$\boxed{\text{A-1, A-3, A-2}}$ $\boxed{\text{A-4, A-5, A-6, A-7}}$

Granular Material   Silt clay Material

A-1: Stone fragments, gravels, coarse sand, few fines, low plasticity.

~~A-2~~ A-1-a: #10 passing → Max (50%)

#40 passing → Max (30%)

#200 passing → Max (15%)

A-1~~a~~: LL-not applicable for A-1-a

PI → Max 6

A-1-b: #10 (50max) → Not applicable

#40 passing → 50% max

#200 passing → 25% max

A-3 : Clean Fine Sand

#10 → N/A

#4 → 51 min^m

#200 → 10 max

LL → N/A

PI → Not plastic

Subgrade material এর Quality তে উত্তম —

A-1 → Excellent

A-3 → Very Good

A-2 → Good

Serially, A-1, A-3, A-2 বলতে হবে।

A-2: Silty or clayey gravel and sand.

Fair to Poor Subgrade Material

A-2-4 → LL = 40 max, PI = 10 max

A-2-5 → LL = 41 min, PI = 10 max

A-2-6 → LL = 40 max, PI = 11 min

A-2-7 → LL = 41 min, PI = 11 min.

A-4
A-5
A-6
A-7

Fair to Poor

LL, PI → same to A2-4, A-2-5, A-2-6, A-2-6, A-2-7

$\boxed{\#200 \text{ passing} \rightarrow 36 \text{ min}}$ → A-4, A-5, A-6, A-7

$\boxed{\#200 \text{ passing} \rightarrow 35 \text{ max}}$ → A-2-4, A-2-5, A-2-6, A-2-7

$\boxed{\text{A-7.}}$

same criteria

→ A-7-5 → $PI \leq (LL-30)$ → $PL \geq 30$

→ A-7-6 → $PI > (LL-30)$ → $PL < 30$

Group Index : $\boxed{\text{\% finer No. \#200 sieve, LL, PI}}$

function of

$$GI = \underbrace{(F-35)\left[0.2+0.005(LL-40)\right]}_{\text{Partial GI for LL}} + \underbrace{0.01(F-15)(PI-10)}_{\text{Partial GI for PI}}$$

F → % finer # 200 sieve

LL → Liquid limit

PI → Plasticity Index

GI expressed as a whole number

GI = 22.7 হলে

GI = 23 নিবে

① GI → Negative হলে Zero হবে।

② Whole No. এ express করতে।

③ GI for A-1-a, A-1-b, A-2-4, A-2-5, A-3 is Always zero. (As GI becomes Negative).

④ A-2-6, A-2-7 এর বেলা GI calculate করতে partial group index for PI

⑤ ~~A-2~~ A-4, A-5, A-6, A-7 এর জন্য দুই term calculate করবে।

$PL \geq 30 \rightarrow$ A-7-5
$PL < 30 \rightarrow$ A-7-6

Soil A	Soil B
LL = 51	LL = 48
PL = 24	PL = 31
% 200 passing = 75,	% passing 200 sieve = 80
Ans: A-7-6	A-7-5 ~~Better~~
GI = 20.4 (20)	GI = 22.5 (23)

% passing #200 → 95%

LL = 60,
PL = 20,
GI = 42

GI যত বেশি তত খারাপ,
GI যত কম তত ভালো

% passing #200 → 95%

Flow net diagram → ct

Effectivestress

Pore water pressure

A soil profile is shown in the figure.

If a uniformly distributed load is applied at the ground surface. What is the settlement of the clay layer caused by the primary consolidation?

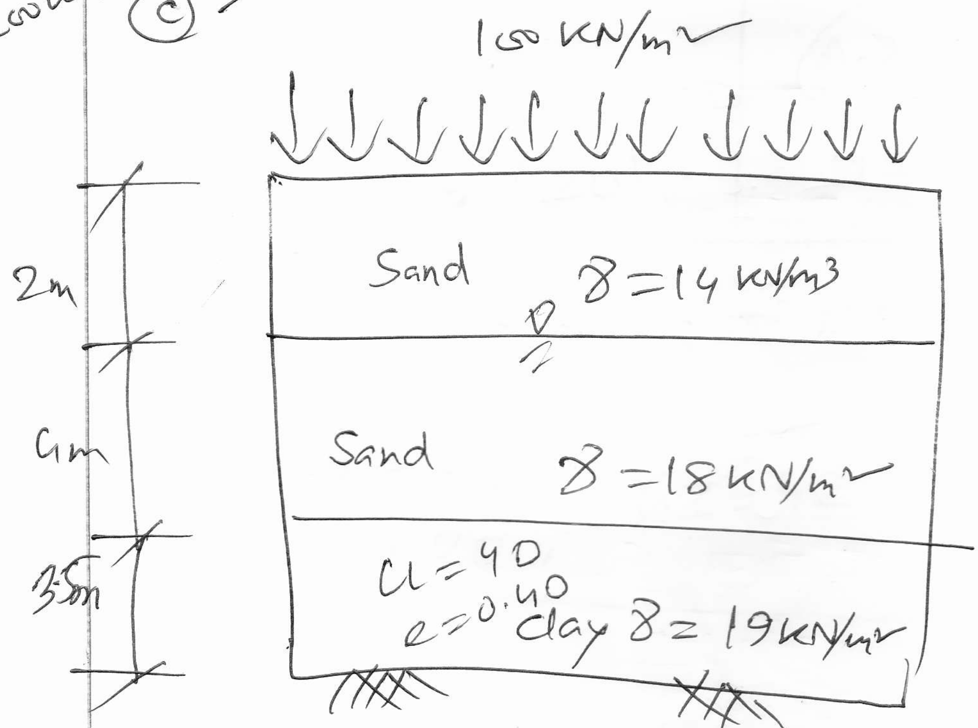
(a) The clay is Normally consolidated. Given pre-consolidation pressure = 200 KN/m²

(b) OC clay

Pre Consolidation Pressure = 200 KN/m²

Pre consolidation pressure = 150 KN/m²

(c)



$$C_c = 0.009 \times (LL - 10)$$
$$= 0.009 \times (40 - 10)$$
$$= 0.27$$

$$P_o = 14 \times 2 + (18 - 9.81) \times 4 + (19 - 9.81) \times \cancel{3.5}\ 1.75$$
$$= \cancel{92.925}\ \text{KN/m}^2$$
$$76.8425$$

$$S = H \times \frac{C_c}{1 + e_0} \log \frac{P_o + \Delta p}{P_o}$$

$$= 3.5 \times \frac{0.27}{1 + 0.80} \log \left(\frac{\overset{76.8425}{\cancel{92.925}} + 100}{\underset{76.844}{\cancel{92.925}}} \right)$$

$$= \cancel{0.216\,m}\ 0.191\,m$$

(b)

$\cancel{S = HC_s}$ $C_s = 15\%$ of C_c
$$= 0.0405$$

$$S = 3.5 \times \frac{0.0405}{1 + 0.80} \log \left(\frac{\cancel{200}\ 176.8425}{76.8425} \right) = 0.038\ \text{m}$$

(c)

$$\boxed{S = \frac{H C_s}{1+e_0} \log \frac{P_c}{P_0}} +$$

$$\frac{H e_c}{1+e_0} \log \left(\frac{P_0 + \Delta P}{P_c} \right)$$

$$= \underline{0.068\,m}$$

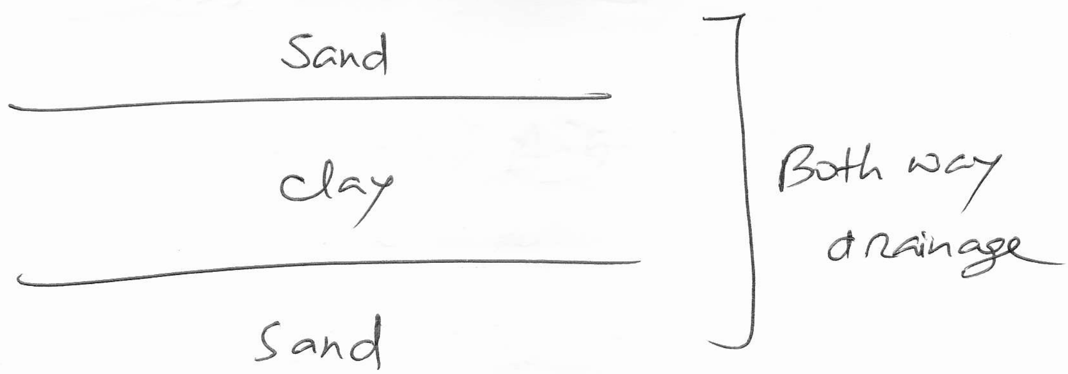
$$= \frac{3.5 \times 20/100 \times 0.27}{1+0.80} \log \left(\frac{150}{76.8425} \right) +$$

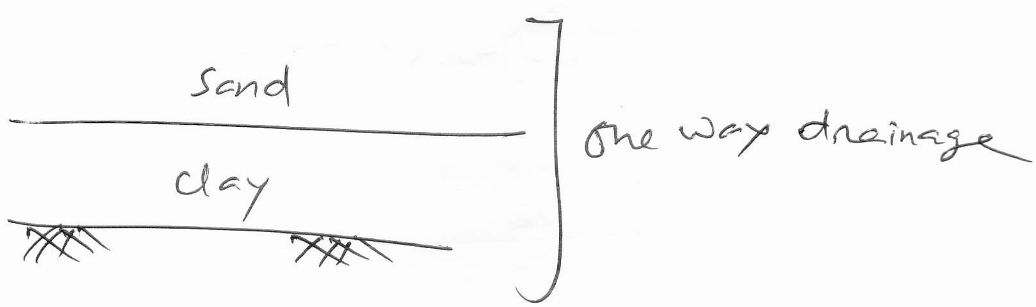
$$\frac{3.5 \times 0.27}{1+0.80} \log \left(\frac{76.8425 + 150}{150} \right)$$

continuous footing, strap footing, wall footing.

Ultimate settlement

This magnitude will remain same irrespective of drainage condition.





One way drainage এ settlement দেরিতে হয়
Two " " এ " দ্রুত হয়।

Clay layer thickness অনেক বেশি হলে

Clay thickness

3m এর বেশি হলে দুই layer এ জোড়া করে নিতে হবে।

Siddique Sir:

AASHTO -> Soil classification

Rules for GI -> Ref. Das

#10 -> 98%

#40 -> 74%

#200 -> 60%

LL -> 42
PL -> 33
PI = 9 => A-5

LL = 50
P.L = 21
A-7-6

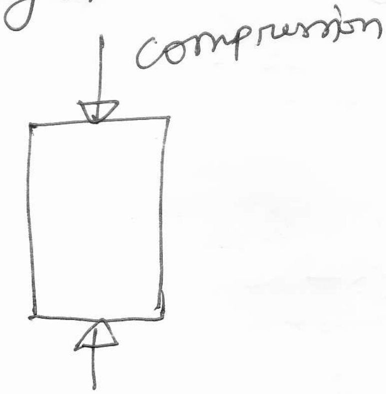
$PI = 0.73 \times (LL - 20)$

CL/CH
ML/MH
OL/OH

Soil is very weak in tension, can only take compressive loads

Shear strength

→ Resistance to shear stress



Load দিলে Mobilized/Developed shear stress আসবে,

Mobilized shear stress < shear strength of soil

→ soil won't fail

Sliding capability:

Strength parameters:

Undrained shear → very fast,

Drained shear → slow, পানি বের হওয়ার সময় থাকবে

Cases when shear strength parameter is required →

I) Slope stability Analysis :

II) Bearing capacity : shear strength parameters

III) Active earth pressure, passive earth pressure
for retaining walls, shore pile

Shear strength depends on.
3 major components —

I) Interlocking between soil particles →
cohensionless soil, coarse grained soil

II) ~~Elct~~ Frictional resistance between individual
particles → Cohesionless particles, sandy soil

$\boxed{\text{sliding, Rolling friction}}$

III) Adhesion between individual particles →
cohesive soil, clayey soil,
high plastic clay

shear strength parameters:

ϕ-soil → frictional soil

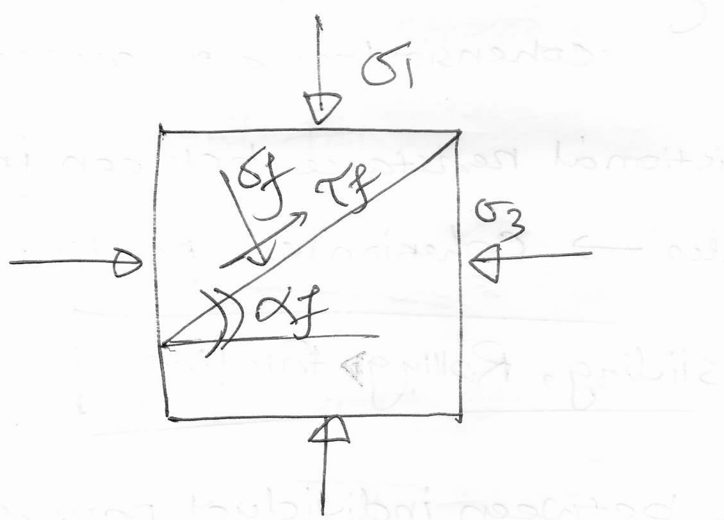
c-soil → ~~frictional~~ Cohesive soil

c-ϕ soil → frictional-cohesive soil

এগুলো drained Undrained হতে পারে।

Unconfined compression test → clay (c) বের করব → $c = q_u/2$

Mohr-coulomb Failure Criteria:



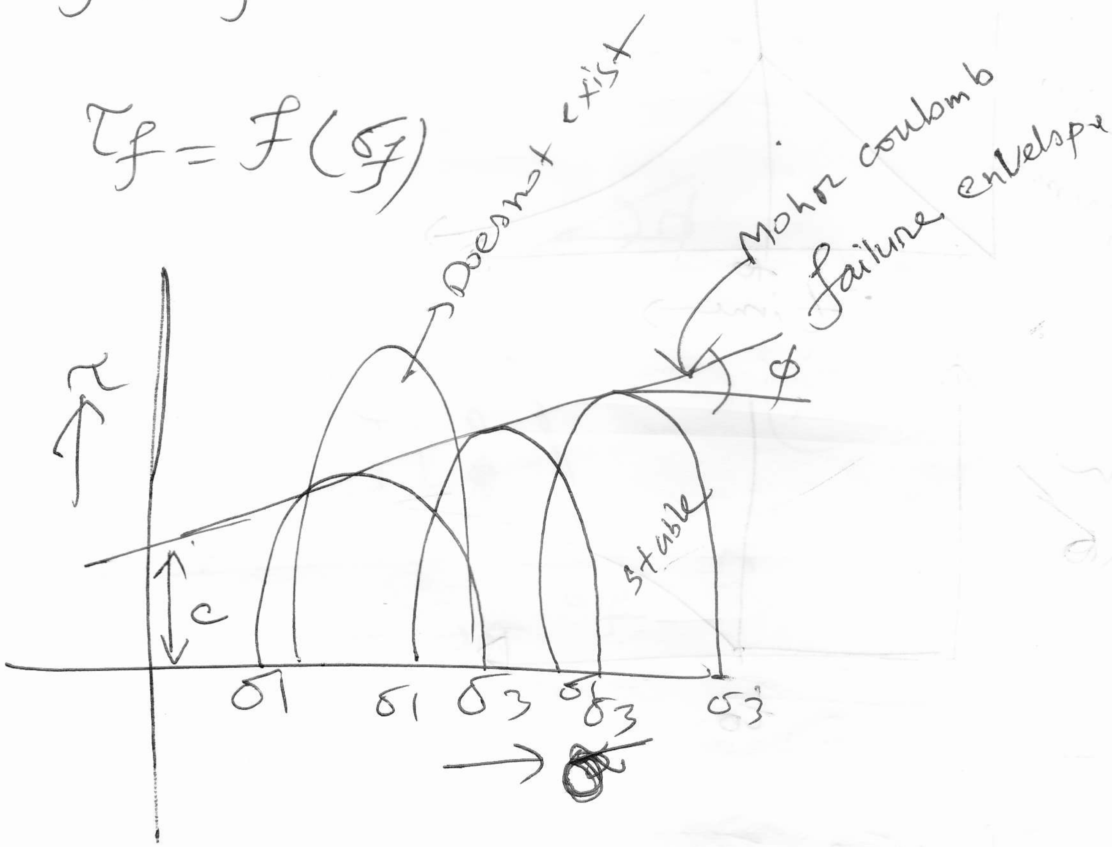
σ_1 → Major principal stress

σ_2 → Intermediate principal stress

σ_3 → Minor Principal Stress

τ_f, σ_f

$\tau_f = f(\sigma_f)$

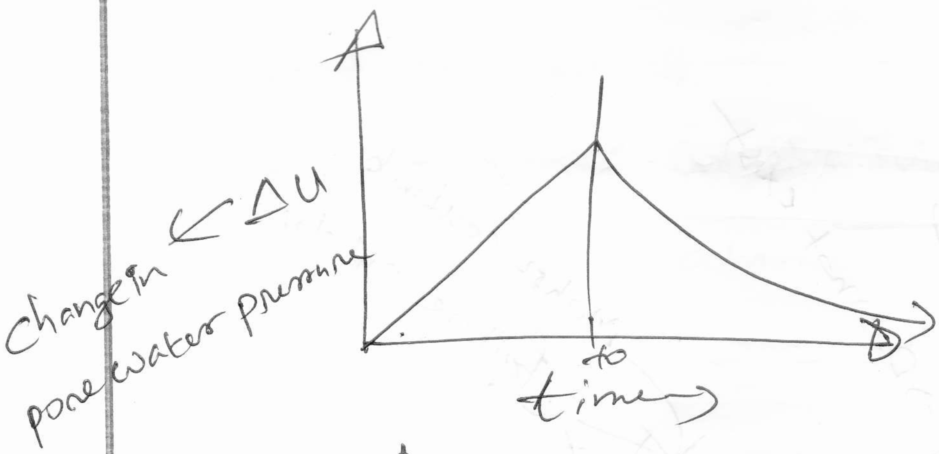


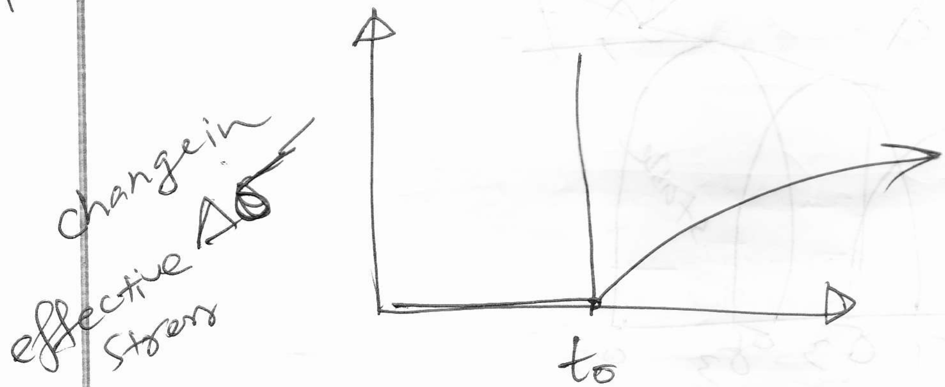
$\tau_f = c + \sigma_f \tan\phi$

$\boxed{\sigma_1 = \sigma_3 + \Delta\sigma}$

Deviatoric stress

Major stress



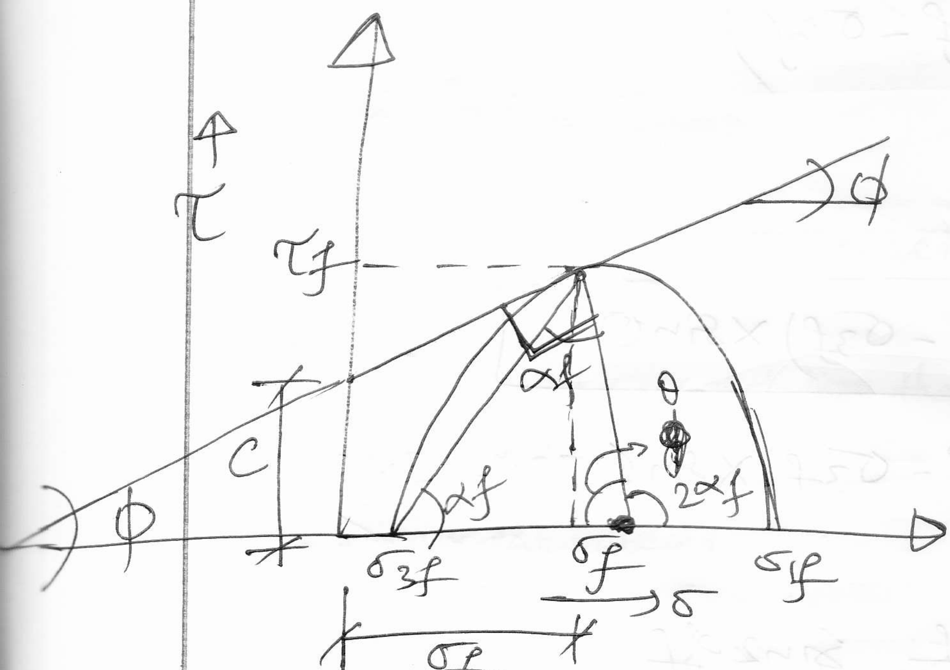


t_0 = Upto construction process

Initially load apply করলে applied stress, change pore water pressure হিসেবে থাকে। কোনো Volume change হয় না।

কিন্তু সময়ের সাথে সাথে pore water pressure dissipate করে। Effective stress বাড়তে থাকে। Volume change হয়।

Effective stress এর বেলায়, soil sample এর ক্ষেত্রে।



(Same scale এ আঁকতে হবে)

$\theta = 180° - 2\alpha_f$

$180° - 90° - \phi = 180° - 2\alpha_f$

$\Rightarrow 2\alpha_f = 90° + \phi$

$\Rightarrow \alpha_f = 45° + \phi/2$

Angle of failure

Angle of friction

একই soil এর c, φ test condition এর উপর নির্ভর করে।

$\phi = 30°$

$\alpha_f = 45° + 30°/2$

$\Rightarrow \alpha_f = 60°$

$$R = \frac{1}{2}(\sigma_{1f} - \sigma_{3f})$$

$$\tau_f = R \sin\theta$$

$$\tau = \boxed{\frac{1}{2}(\sigma_{1f} - \sigma_{3f}) \times \sin\theta}$$

$$= \frac{1}{2}(\sigma_{1f} - \sigma_{3f}) \times \sin(180^\circ - 2\alpha_f)$$

$$\tau_f = \frac{\sigma_{1f}\,\sigma_{3f}}{2} \sin 2\alpha_f$$

V.V.I.

$$\boxed{\tau_f = \frac{\sigma_{1f} - \sigma_{3f}}{2} \cos\phi}$$

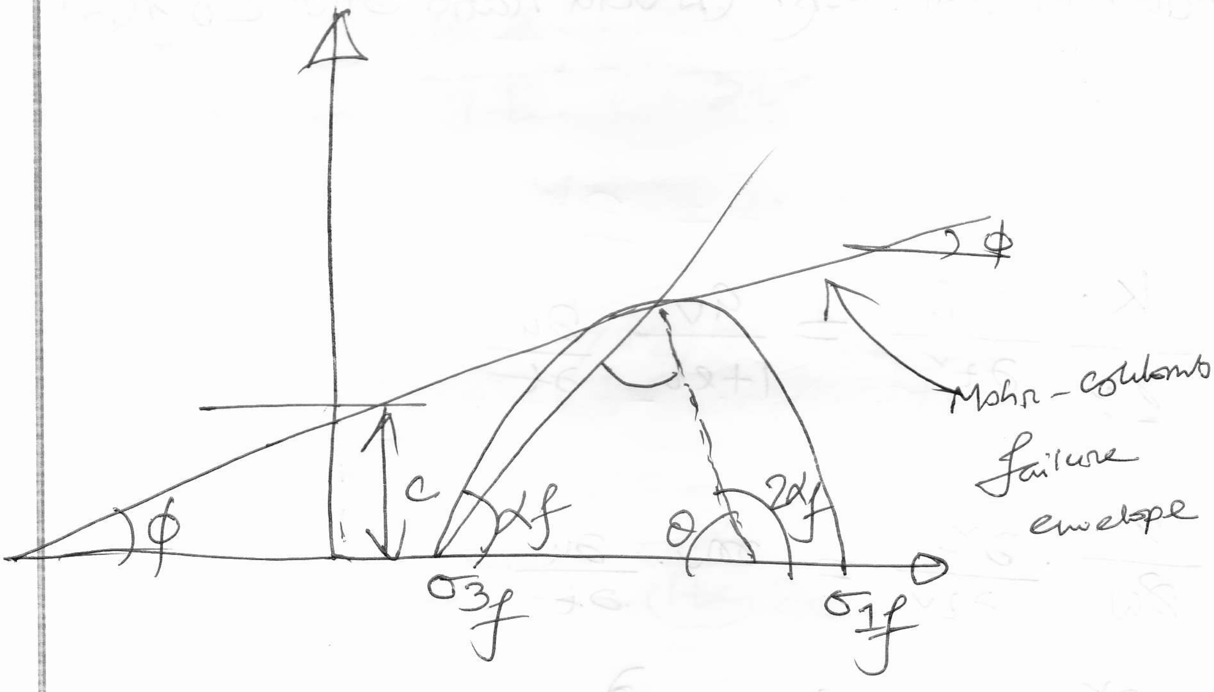
$$\sigma_f = \sigma_{3f} + \frac{1}{2}(\sigma_{1f} - \sigma_{3f}) - R\cos\theta$$

$$\boxed{\sigma_f = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\alpha_f}$$

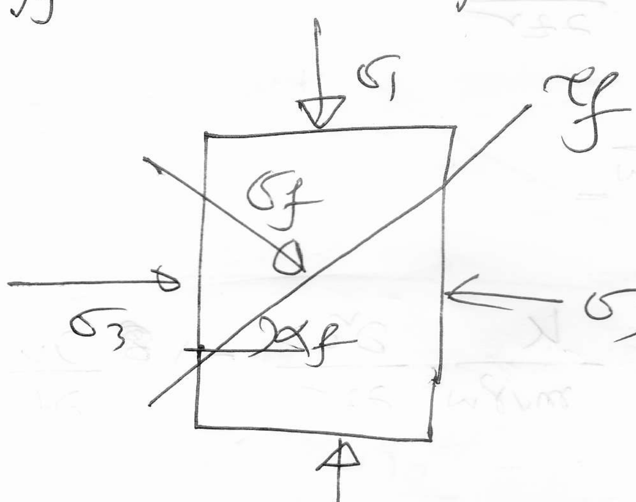
V.V.I.

$$\boxed{\sigma_f = \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2} \sin\phi}$$

Siddique Sir:



$$\tau_f = c + \sigma_f \tan\phi \quad (c, \phi)$$



The difference between major principal stress at failure and minor principle stress at failure is called deviator or deviatoric stress.

What is the +

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

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

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

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

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

$$\Rightarrow \sigma_1(1 - \sin\phi) = \sigma_3(1 + \sin\phi) + 2c\cos\phi$$

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

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

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

$\phi \rightarrow \phi', \phi_u$

$\boxed{\sigma_1}$ or $\boxed{\sigma_1'}$ → Effective Majorr principal stress

↓

Total major principle stress

$\sigma_1' = (\sigma_1 - u)$

$\sigma_3' = (\sigma_3 - u)$

$\boxed{\sigma_3}$ or $\boxed{\sigma_3'}$

↓ Total Minor Principal Stress

↓ Effective minor principal stress

ϕ_u → Undrained co-efficient of angular friction

ϕ' → Effective " " " "

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

Cohension less soil ~~Undrained soil~~ এর

জন্য $c=0$ (Sandy soil)

$$\Rightarrow \boxed{\frac{\sigma_1}{\sigma_3} = \tan^2(45^\circ + \phi/2)}$$

Sandy soil

$$\boxed{\frac{\sigma_3}{\sigma_1} = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45^\circ - \phi/2)}$$

Sand এর জন্য ϕ_u not applicable,

Sand, gravel খুব quickly drain হয়ে যায়।

~~OC clay~~ →

Undrained অবস্থায় Load দিলে Excess pore water pressure develop হয়।

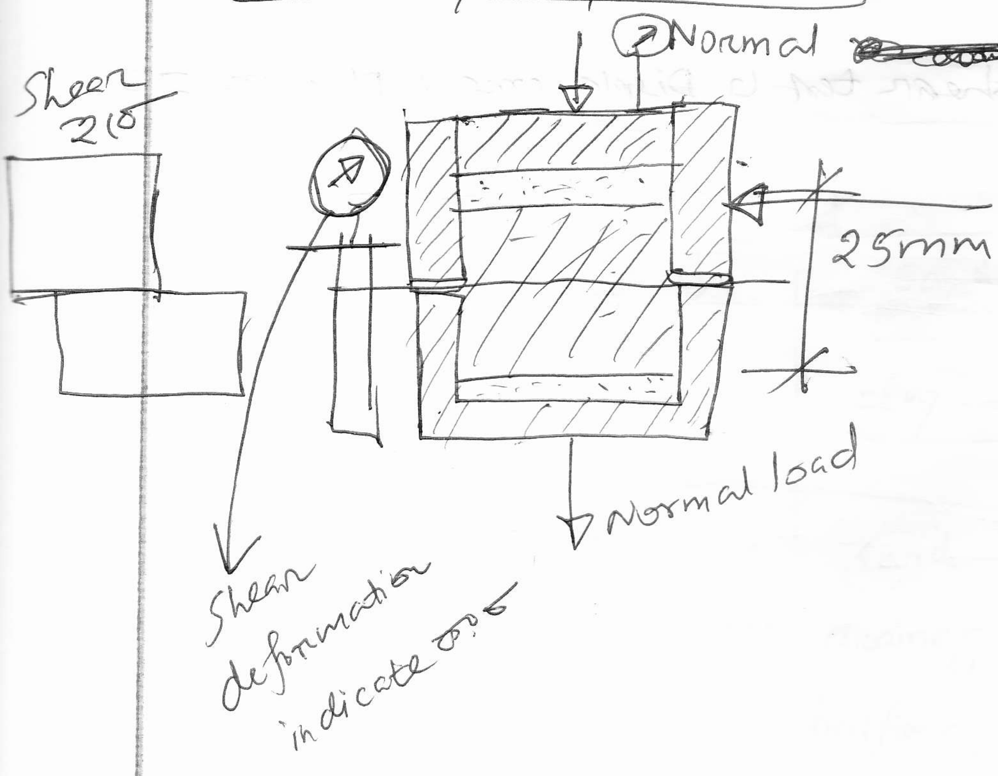
→ Direct shear on dry sand

→ Triaxial compression test

→ Unconfined compression test. (Pure compacted clay)

→ Field Vane shear test

$\boxed{\text{In situ/Inplace/Field}}$ Same word

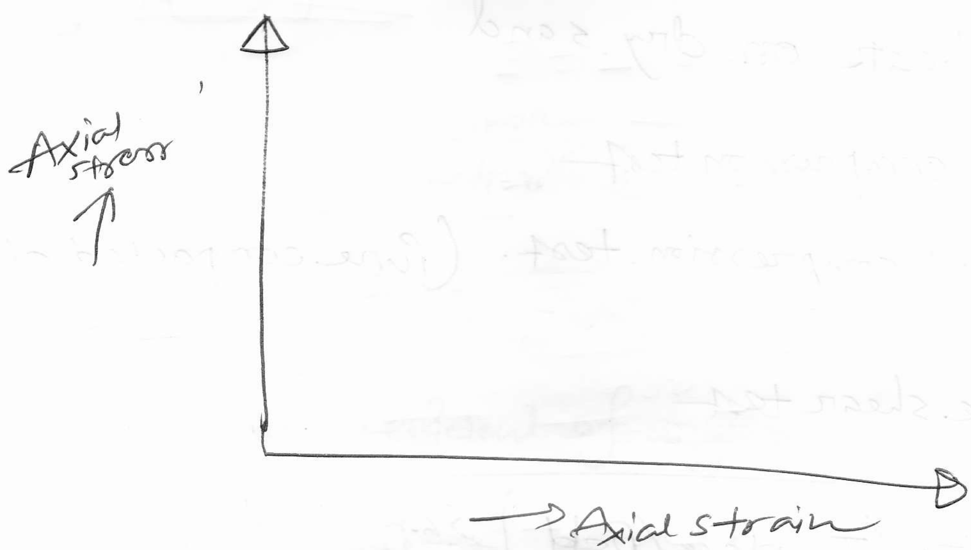


For different Shearing loads, Strain মাপে (Normal Strain or shear strain)

Porous stone must have higher permeability than sand for proper drainage

Porous stone is provided for drainage.

Un-confined compression test ↳



But direct shear test ↳ Displacement plot করা হ

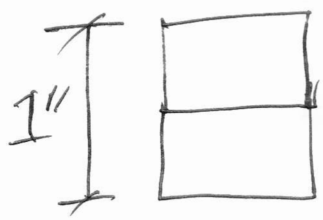
Siddique Sir

Direct shear Test →

Dry sand, Saturated sand

Sand → Loose dense

clay → NC clay , OC clay



Volume change

Initial volume = $A_o h_o$

~~$\Delta V = \Delta V$~~

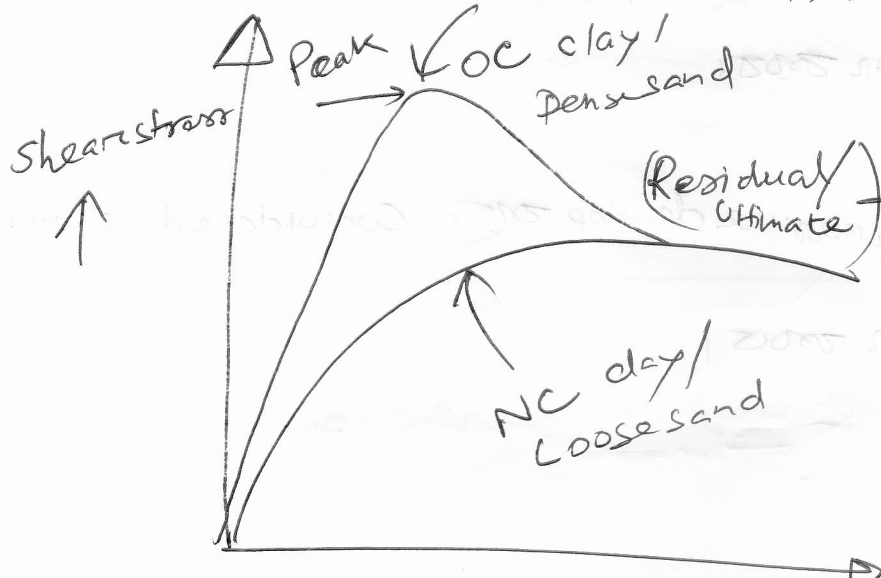
$$\text{Vol. change in } \% = \frac{\Delta V}{V_o}$$

$$= \frac{A \times \Delta h}{A_o h_o}$$

$$= \frac{\Delta h}{h_o} \times 100$$

Loose sand এবং NC clay এর behavior same.
Dense sand এবং OC clay " " " } Shear

└→ Preloaded, Pre-consolidated



1 Sample
3 specimens

→ shear Deformation / Displacement (mm)

Sample will fail in a pre-determined failure plane (Horizontal)

Normal Stress,

Shear stress

Normal displacement

Proving ring

Angle of internal friction, ϕ

Consolidated Drained

UU → Pore pressure develop করে। Un-consolidated sample

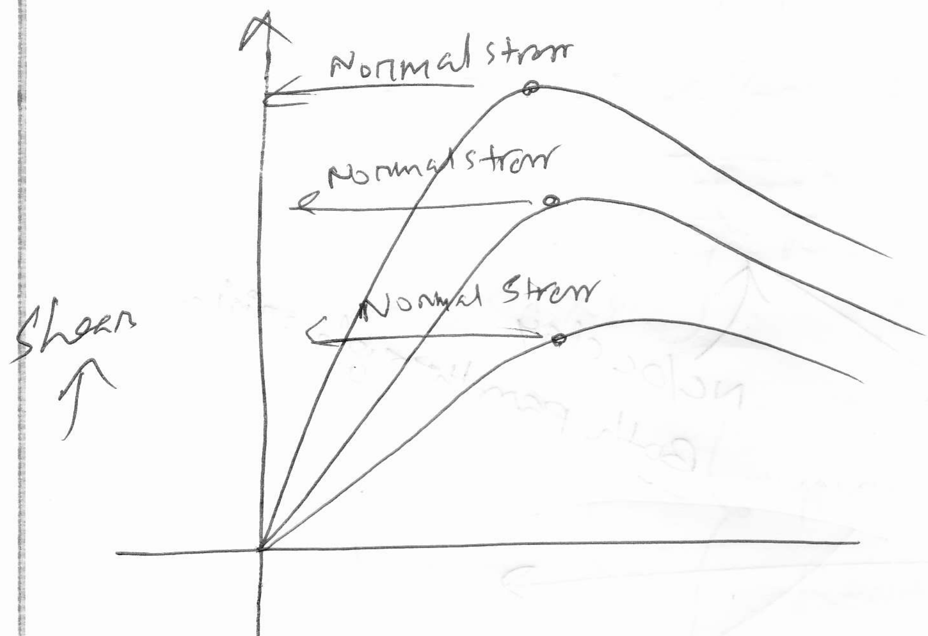
quick shear করে।

CU → Pore pressure develop করে। Consolidated sample

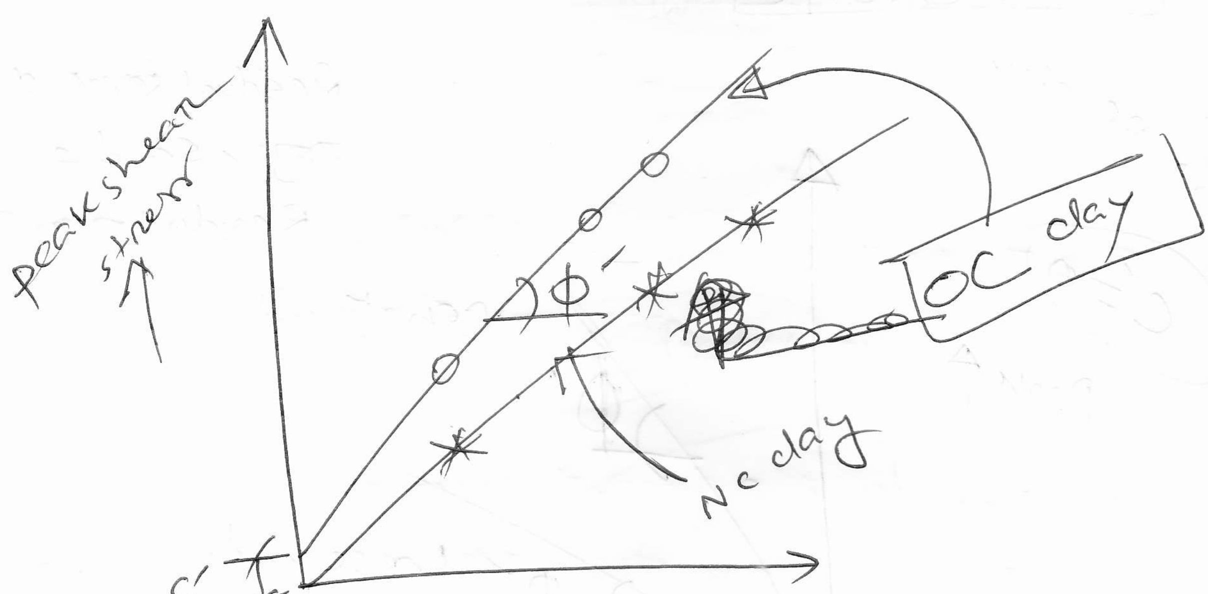
Quick shear করে।

CD → Volume change হবে।

Direct shear → only CD test হবে।



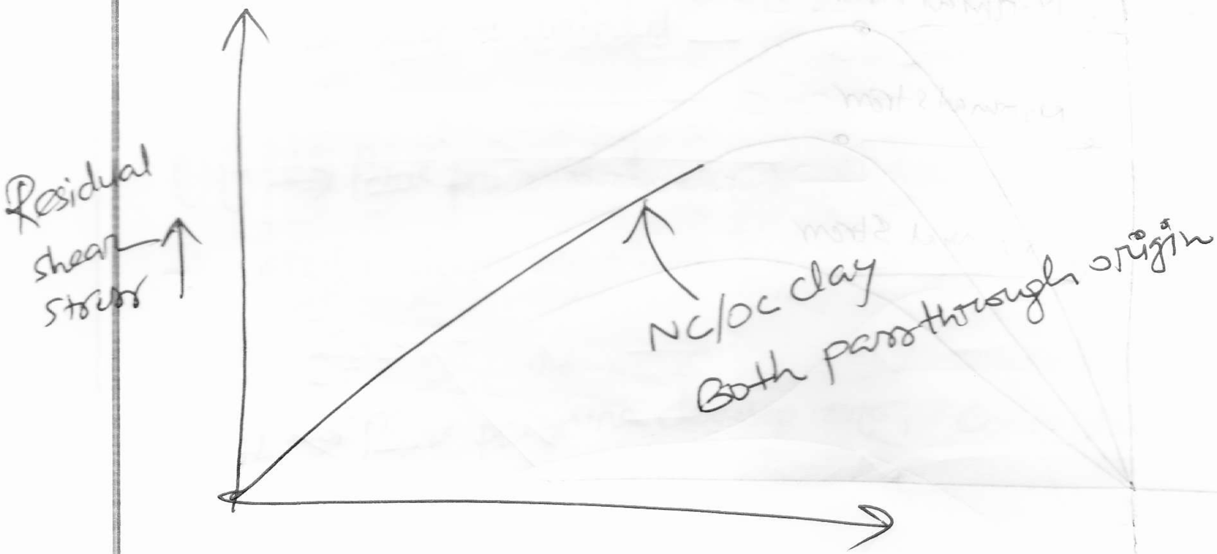
তিনটা Normal Stress এর মান বের করা



Effective normal stress

Effective বলা যায় normal stress কে।

Effective angle of internal friction



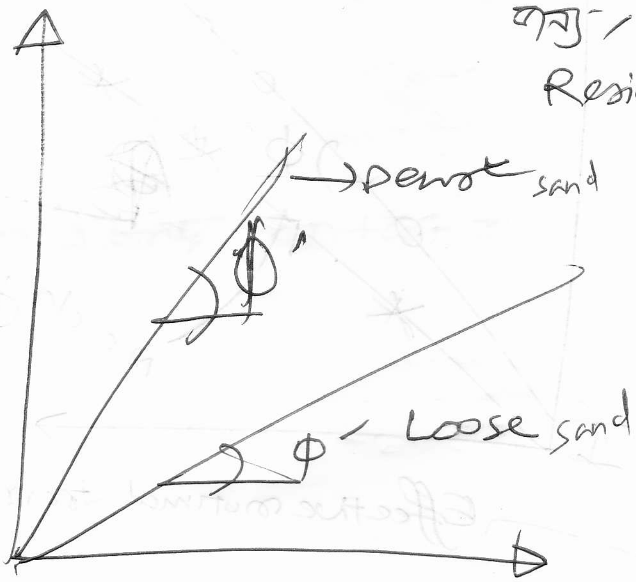
→ σ', Effective normal stress

Failure envelope

Only for OC clay

c' = 0

Peak ↑



Residual থাকবে clay এর জন্য। Sand এর জন্য Residual থাকে না

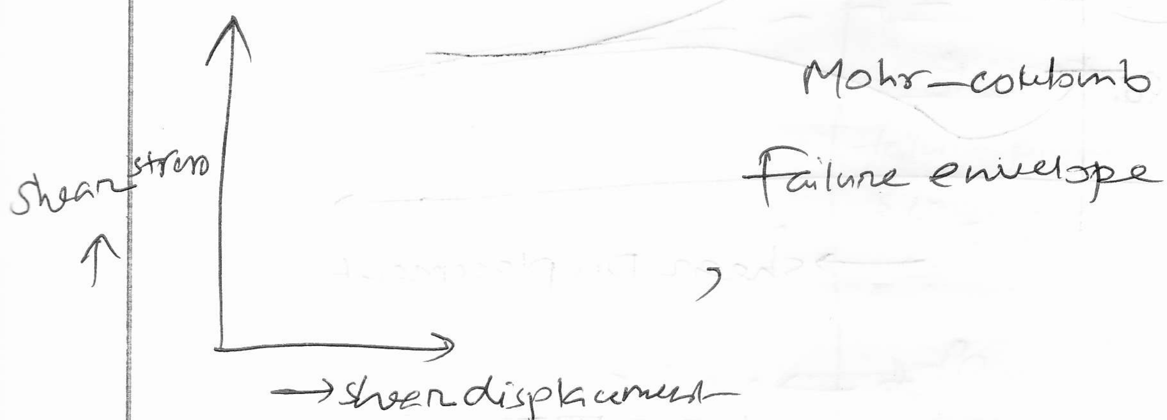
Sand এর জন্য c' = 0, always

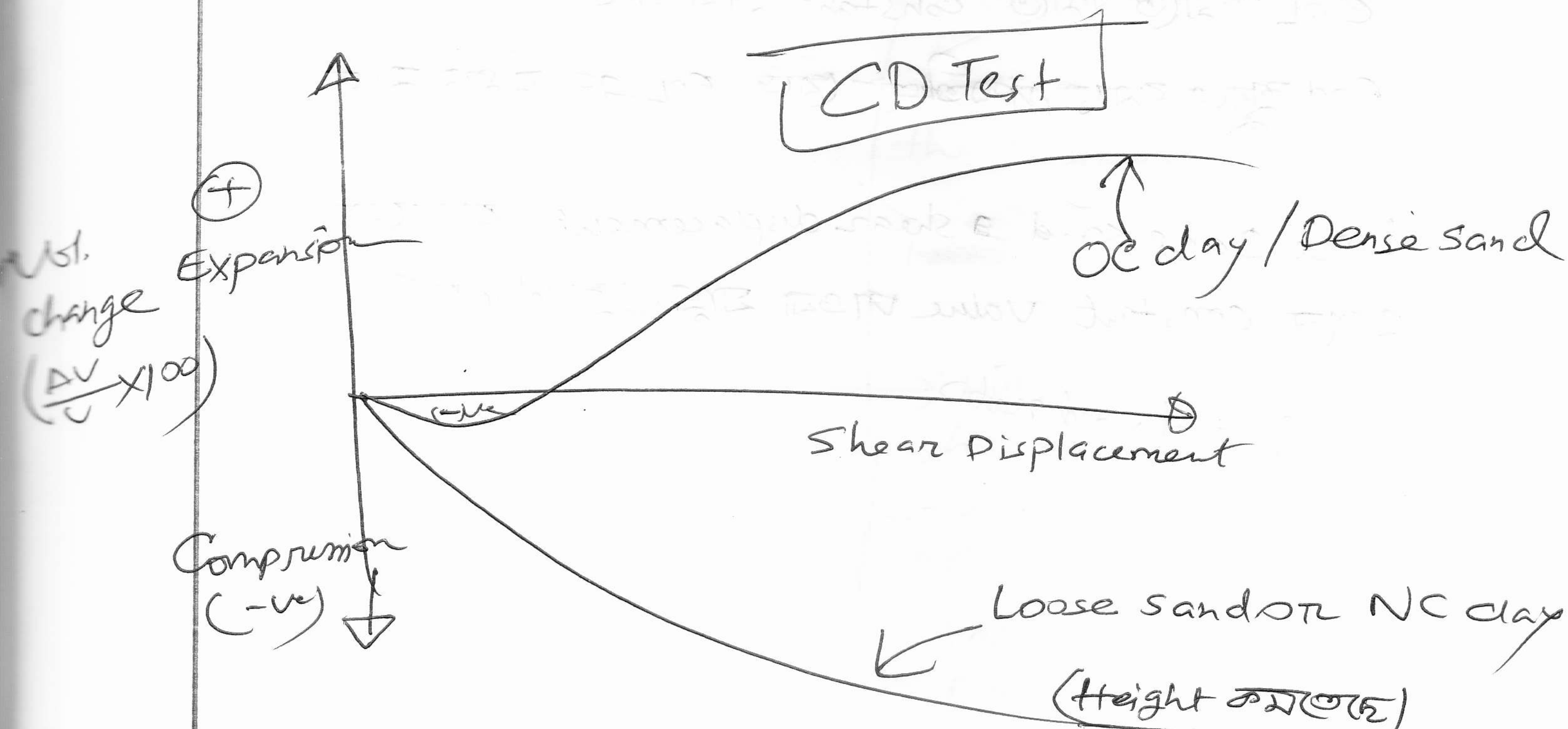
$\phi'(\text{Dense}) > \phi'(\text{Loose})$

OC clay → c′ and ϕ′
NC clay → Only ϕ′, c′=0
} Peak values plot

OC clay → Only ϕ′
NC clay → Only ϕ′
} Residual shear stress plot

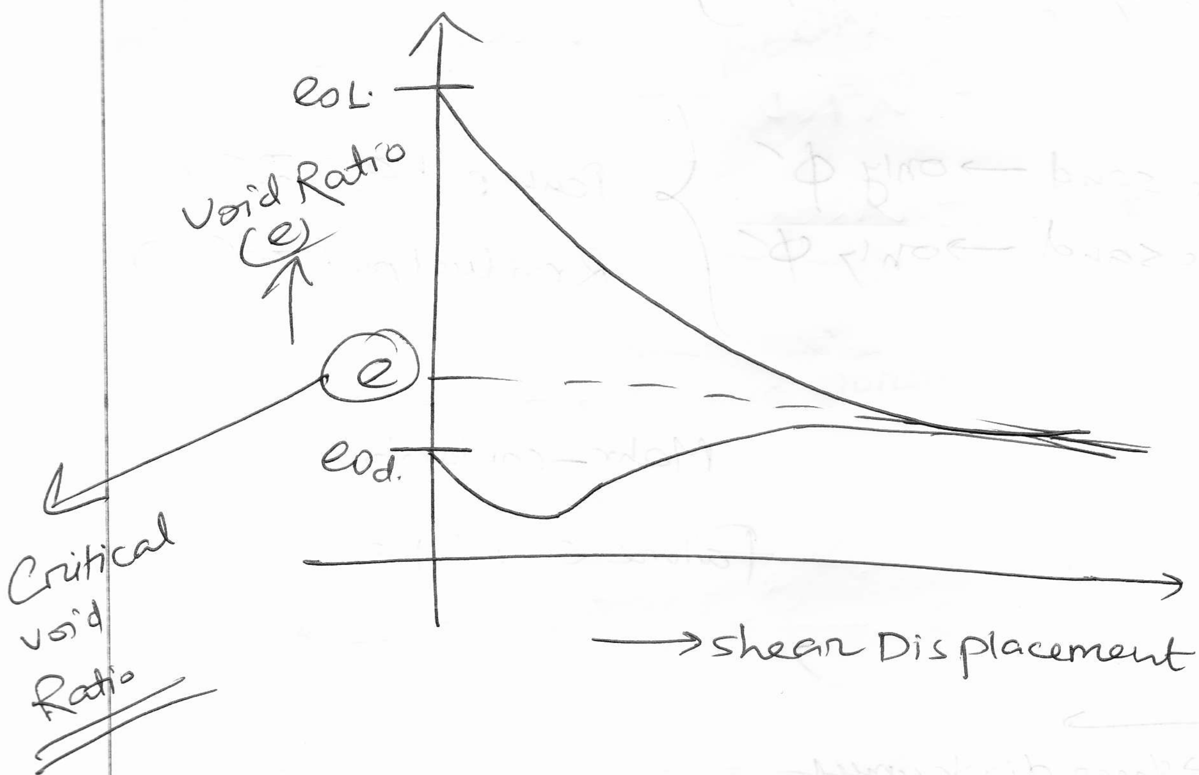
Dense sand → only ϕ′
Loose sand → only ϕ′
} Peak plot থেকে হয়, Residual plot থেকেও





Applicable only for sand — (Not for clay)

$\boxed{\text{Critical void ratio}}$



eoL কমতে কমতে constant হয়ে যাবে

eod শুরুতে কমবে, পরবর্তীতে বেড়ে eoL এর সমান হবে

Loose, Dense sand এ shear displacement বাড়াতে থাকলে constant value পাওয়া যায়। এই value টা Critical void ratio.

Advantages and Disadvantages →

- simple & cheap
- sample preperation for Dry & saturated sandis.
- Rapid volume change
- Quick dissipation of pore water pressure.

- Sample doesn't fail along the weakest plane. It fails along a pre
- shear stress distribution is not uniform on failure plane
- Sample fails progressively. It
- Shear strength on the failure plane is not mobilized simultaneously
- Pore water pressure can't be measured
- Sides of the wall of the shear box gives lateral ~~Shear~~ strain on the sample ~~load~~
- With the progress of test, shear stress decreases
- Area under ~~load~~ normal load decreases with progression of experiment

Dia → 50mm

Ht = 25mm

1 → 150 kN — 158 N → 44 N

2 → 350 N — 257 N → 102 N

3 → 550 N — 363 N → 144.5 N

$$\boxed{\frac{\phi'_{peak},\ c'}{\phi'_{residual}}}$$

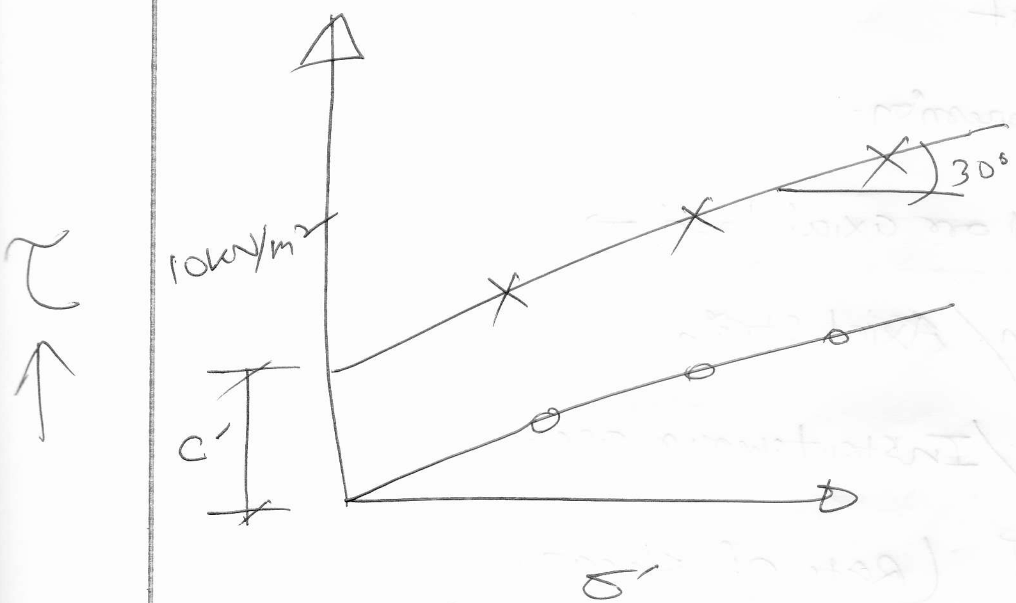
$\tau_f = c' + \sigma'_f \tan\phi_p' \rightarrow OC$

$\tau_f = \phi_f' \tan\phi_{ss}' \rightarrow NC/OC$

$\tau_f = \sigma_f' \tan\phi_p' \rightarrow NC$

CD Test

$\sigma = \sigma'$



$\boxed{\text{BM Das}}$

Problem

$\boxed{1\ psi = 6.89\ kN/m^2}$

Unconfined compression Test:

ϕ', C' → Effective shear stress parameters

CD → strain controlled

slow Test

Unconfined Compression.

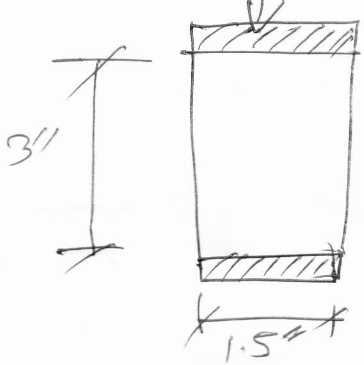
compressive load or axial load →

Axial deformation / Axial strain

Corrected Area / Instantaneous area

$\boxed{1.5 \text{ mm/min}}$ (Rate of shear)

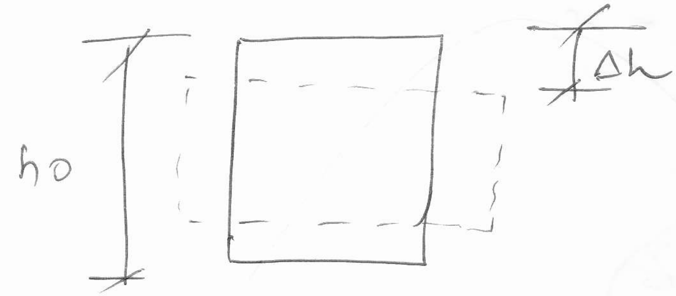
Compressive load / Axial load



$h_o = 3''$

$A_o = 1.767 \text{ in}^2$

Axial deformation



$$h_o \times A_o = A_c \times (h_o - \Delta h)$$

$$\Rightarrow A_c = \frac{A_o \times h_o}{h_o - \Delta h}$$

$$\Rightarrow A_c = \frac{A_o}{1 - \Delta h / h_o}$$

$$\Rightarrow A_c = \frac{A_o}{1 - \varepsilon}$$ $\rightarrow$ Decimal

$$A_c = \frac{A_o \times 100}{100 - \varepsilon(\%)}$$

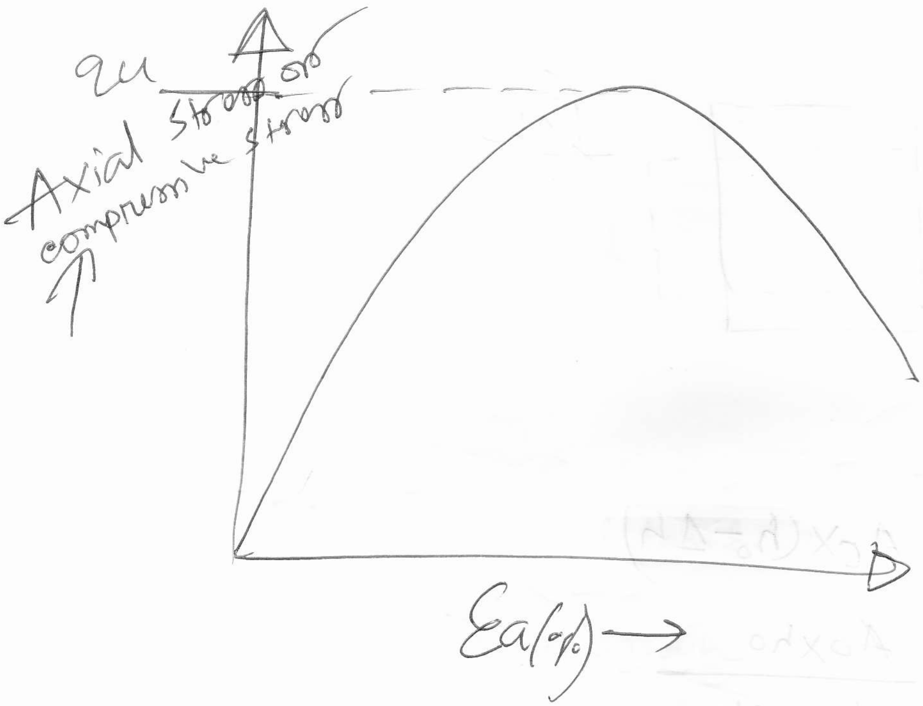
120

Deformation $\delta = 12/100 \times 3 = 0.36$ in

$= 9.144$ mm

12% $\rightarrow \varepsilon =$ 1.52 mm/min

$$\text{Time} = \frac{9.144}{1.52} = 6.016 \text{ min}$$



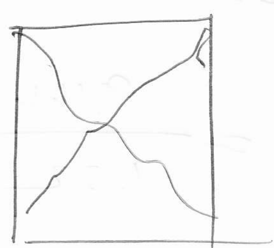
q_u = Unconfined compressive strain

Clays ~~Or~~ Undisturbed soil sample + Remolded.

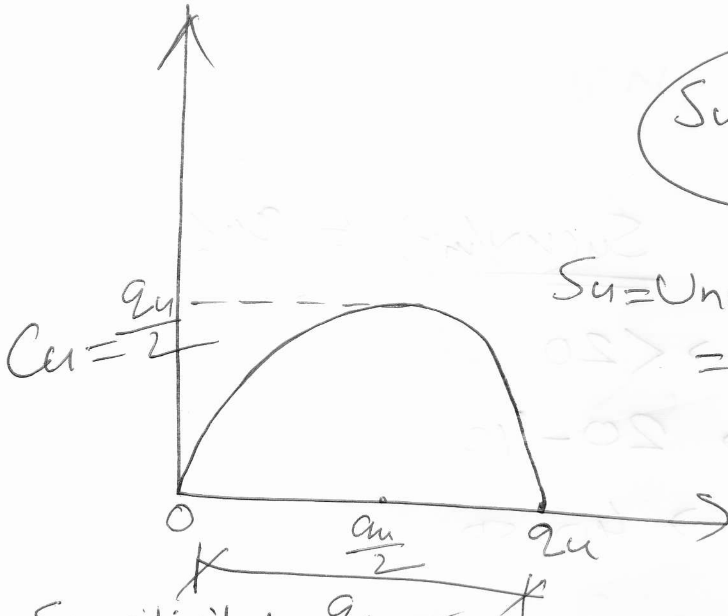
Remolded:

soft sample:

Stiff clay:



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



$S_u = q_u/2$

S_u = Undrained shear strength
= c_u =

Sensitivity: q_u

$$\frac{q_u \text{ (Undisturbed)}}{q_u \text{ (Remolded)}}$$

$\boxed{\text{Sensitivity} < 1}$ Dhaka Clay

$$S_t = \frac{q_u \text{ (Undisturbed)}}{q_u \text{ (Remolded)}}$$

$$= \frac{q_u \text{ (UD)}}{q_u \text{ (R)}}$$

Classification of clays depending on sensitivity.

Deviatory stress at failure = q_u

$\boxed{\text{Consistency :}}$

BS 5930	S_u (kN/m^2) $= q_u/2$
Very soft	< 20
Soft	20–40
Soft to firm	40–50
Firm	50–75
Firm to stiff	75–100
Stiff	100–150
Very stiff/hard	> 150

Decreasing water content (W%)

$$\boxed{A_c = f(\varepsilon_a)}$$

*

$\boxed{\text{76.2 mm sample, diameter} = 38\text{ mm}}$

Compressive Load at failure = 105 N

failure strain ϵ = 10%

Same water content & Density,

$q_u(R) = 20.7\ kN/m^2$

3212.33

$$A_c = \frac{\pi/4 \times (38)^2 \cancel{\times 76.2}}{1 - 10/100}$$

0.0326

$= \cancel{96021.73}$ 1260.12 86919.55

$$q_u(UD) = \frac{105 \times 1000}{\cancel{96021.73}\ 1260.12}$$

96021.7322

$= 83.32$

1.09

$S_u = \frac{q_u}{2}$ 41.66

soft to firm.

$$\frac{83.32}{20.7} = 4.025$$