Chapter - 8
Shear strength of Soil

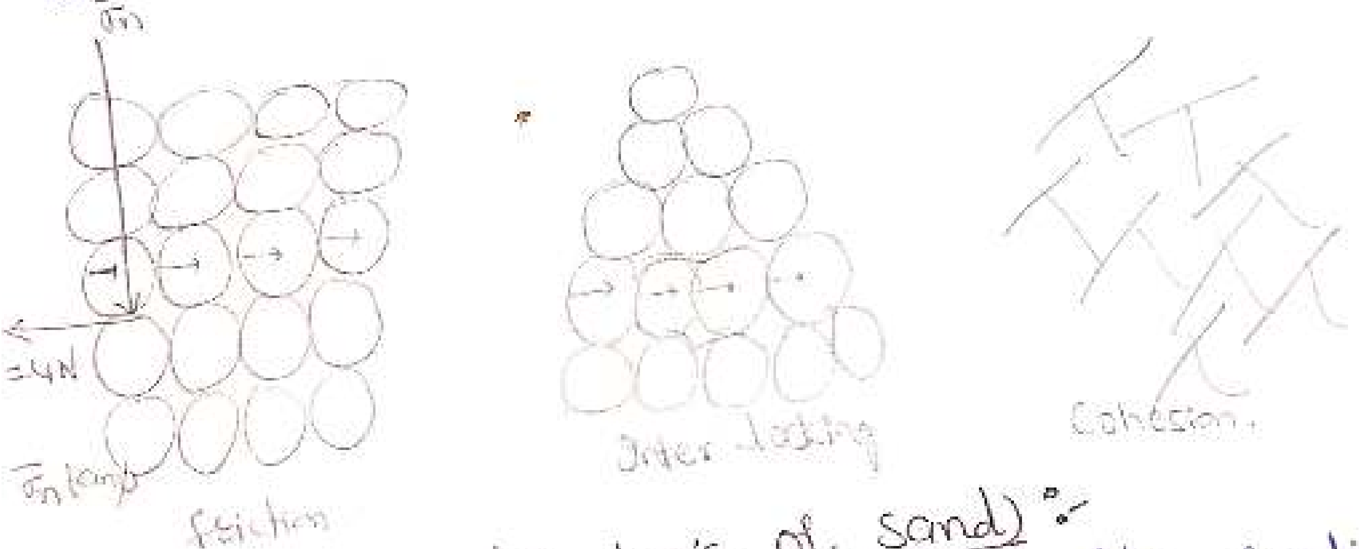
(i) Shear strength is the resistance against the shear deformation offered by the soil grains.

(ii) Soil may derive its shear strength from following parameters:-

(a) Friction in b/w particles due to sliding and rolling (sand, silt and gravel).

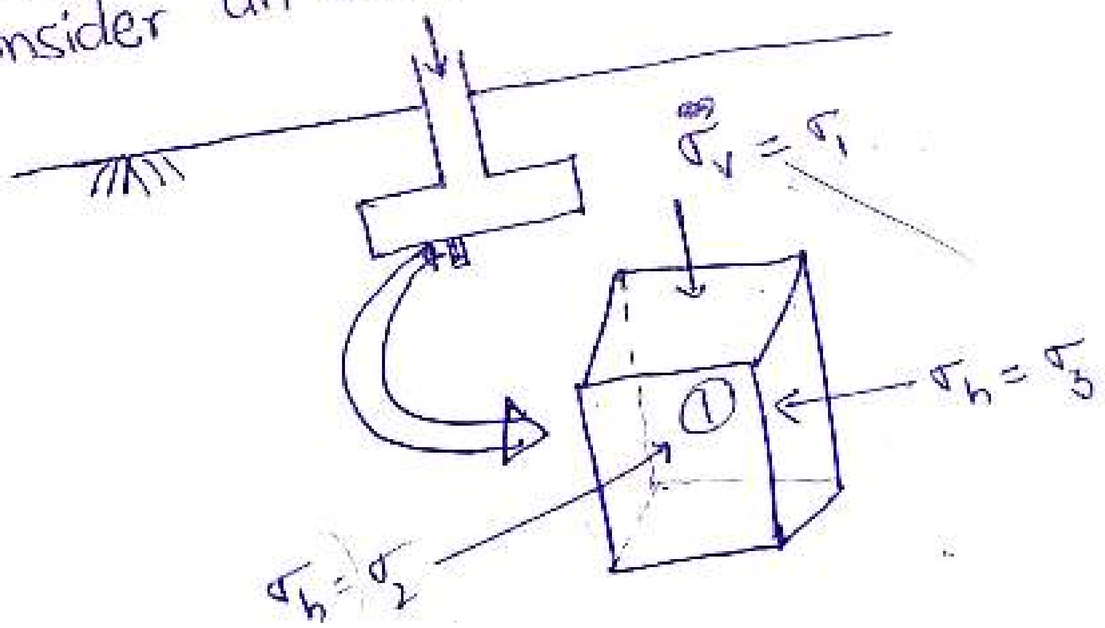
(b) Interlocking b/w particles (dense sand and gravel).

(c) Intermolecular attraction due to cohesion or adhesion (silt and clay).



Mohr Theory (Analysis of Sand):-

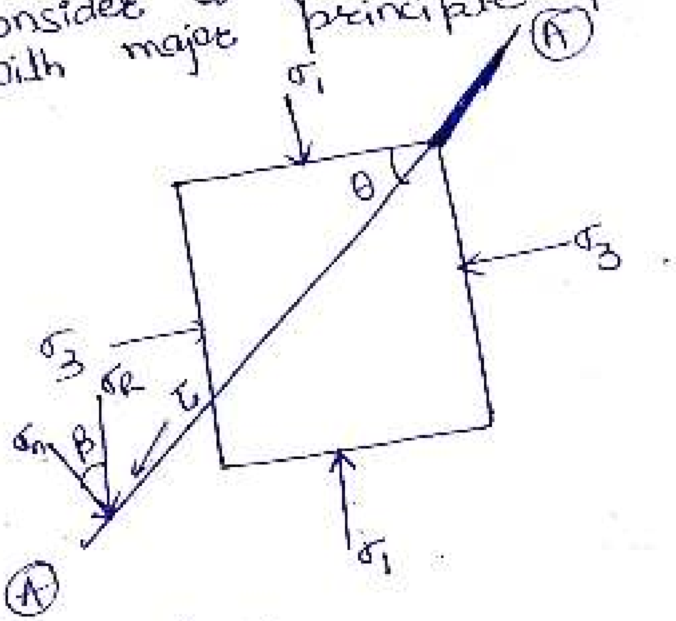
Consider an element just below the footing.



σ_1 = major principle stress
σ_2 = intermediate principle stress $(\sigma_2 = \sigma_3)$
σ_3 = minor principle stress
↓ is the stress where shear failures not occur

2-D Analysis

Consider a plane (A)-(A) at an angle of θ with major principle plane



Normal stress

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

or

$$\sigma_n = \sigma_1 \cos^2\theta + \sigma_3 \sin^2\theta$$

Shear stress

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

Resultant stress

$$\sigma_R = \sqrt{\sigma_n^2 + \tau^2}$$

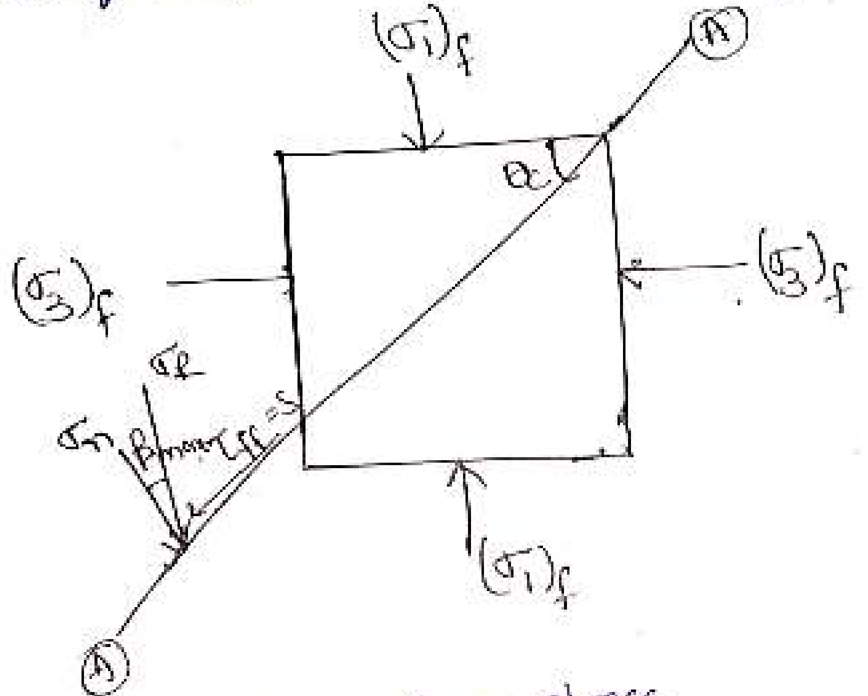
Angle of obliquity

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

Scanned with CamScanner

- When we increase the loading, the shear failure will occur on that plane in which resultant stress σ_R is most inclined with the normal of that plane, such a plane is called critical plane or failure plane and angle of critical plane is called critical angle of failure plane (θ_c).

<u>At failure</u> :-



τ_{ff} → Shear stress

→ At the time of failure

→ On the failure plane

- Shear strength (S) = Shear stress produced on failure plane at limiting condition (just before the failure)

- On this failure plane, σ_R is most inclined with normal of that plane hence β_{max} is called maximum angle of obliquity which is also termed as Internal Angle of reiction or angle of repose.

$\beta_{max.} = \phi$ = Internal angle of friction.

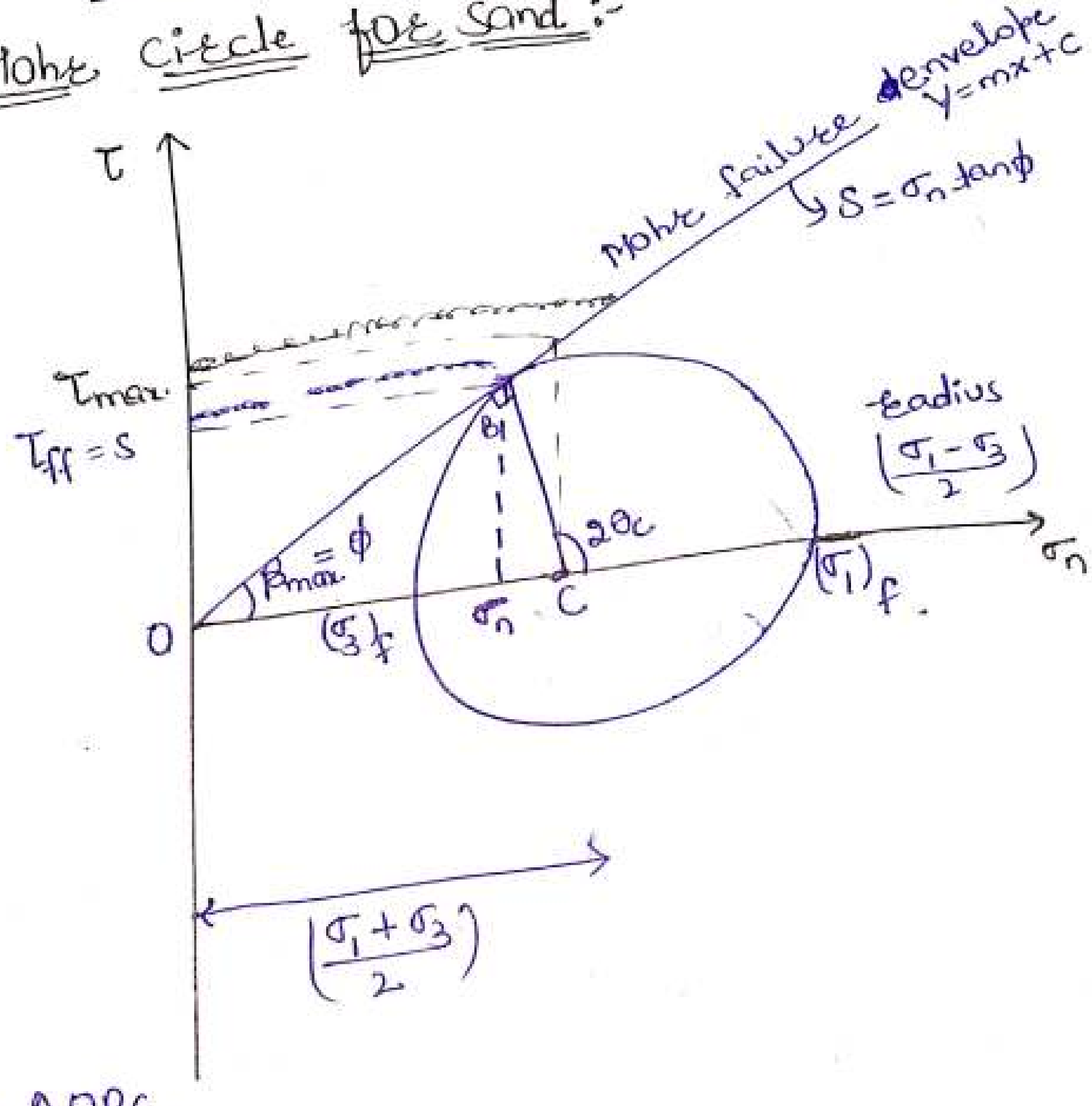
$\tau_{ff} = S$ = shear strength

$\tan\beta_{max.} = \frac{\tau_{ff}}{\sigma_n} = \frac{S}{\sigma_n}$

$\therefore, S = \sigma_n \tan(\beta_{max.})$

$$\boxed{S = \sigma_n \tan\phi}$$

Mohr Circle for sand :-



In ΔOBC,

$\angle O + \angle B + \angle C = 180^\circ$

$\Rightarrow \beta_{max.} + 90^\circ + (180^\circ - 2\theta_c) = 180^\circ$

$\Rightarrow \theta_c = \frac{90 + \beta_{max.}}{2}$

Shear strength - shear stress then failure occur

$\tau > \tau_f$ failure

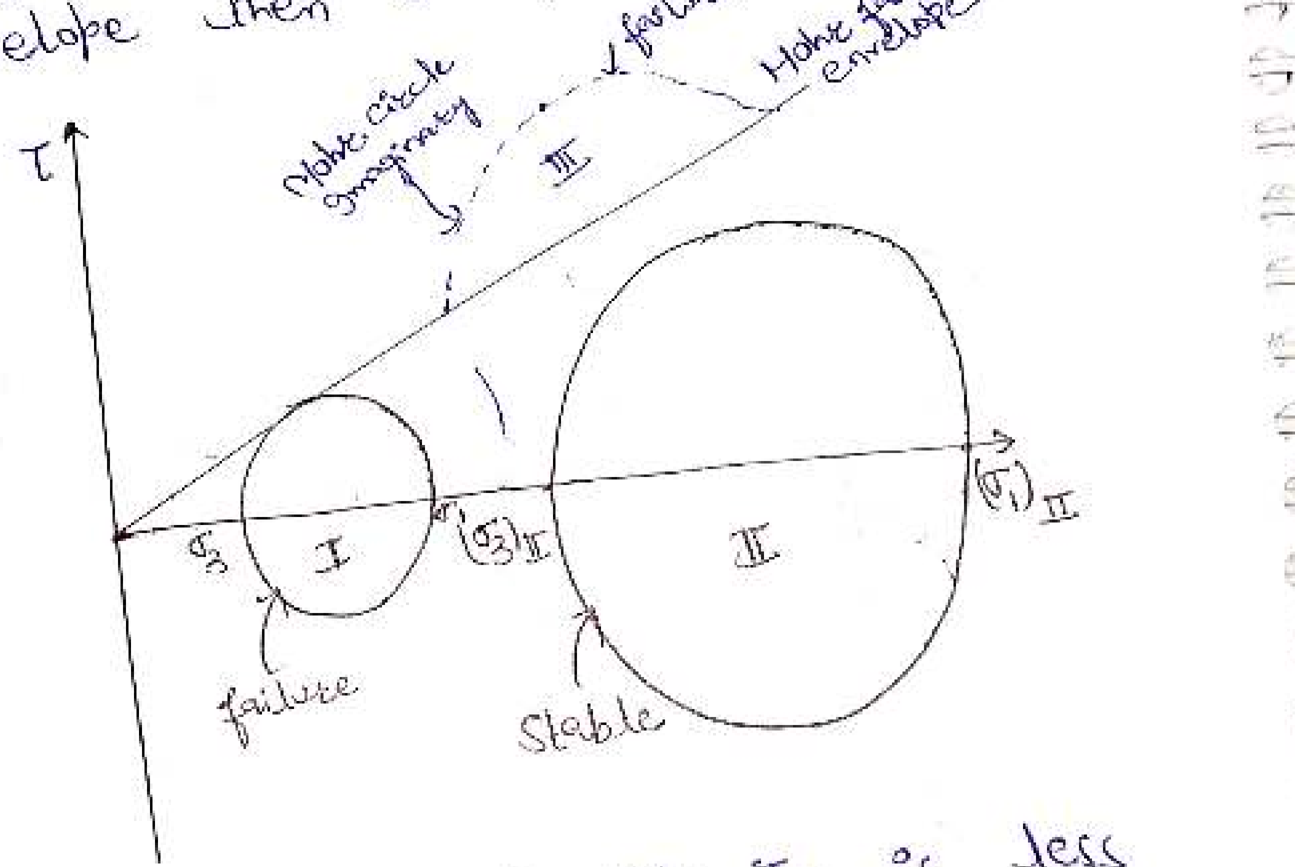
$$\theta_c = 45^\circ + \frac{\beta_{max}}{2}$$

$$\boxed{\theta_c = 45^\circ + \frac{\phi}{2}}$$

↳ with major principle plane.

Note 1:-

Failure will occur on that plane in which shear stress lie on and above the mohr failure envelope whereas if the point is located below the failure envelope then it represents stable condition.

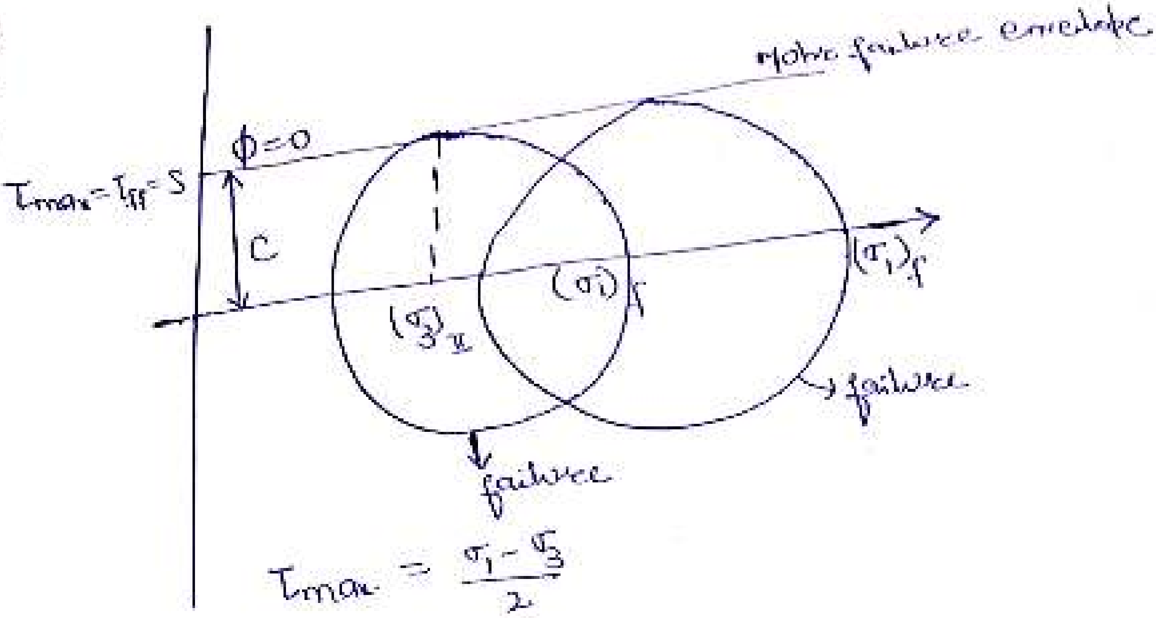


Note 2:-

In case of sand and silt τ_{ff} is less than τ_{max}. but in case of clay $\tau_{ff} = \tau_{max}$.

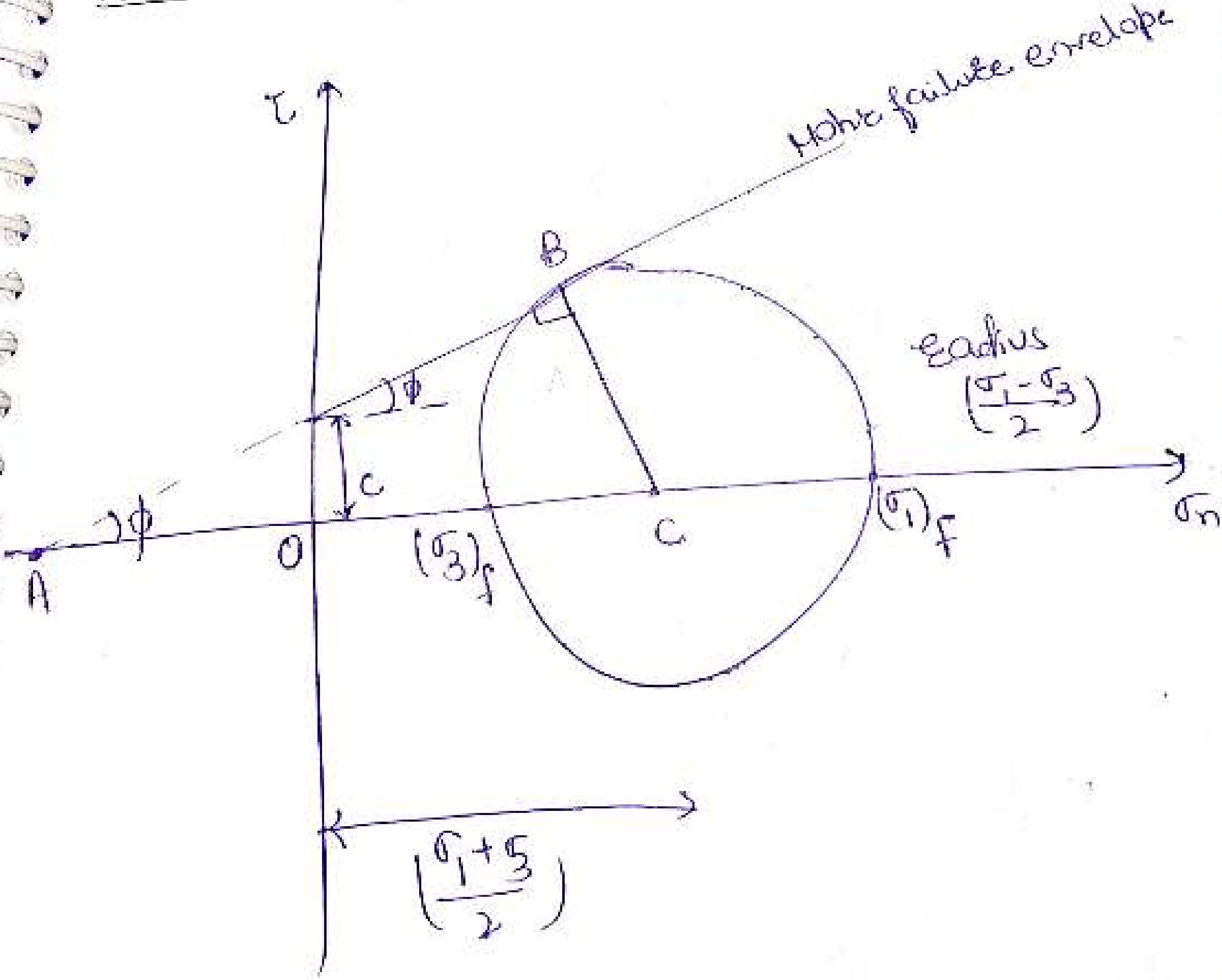
In clays, failure plane will be plane of τ_{max} which forms horizontal Mohr failure envelope.

Mohr circle in clay



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

Mohr circle for silt :-



In ΔABC,

$$\sin\phi = \frac{BC}{AC} = \frac{BC}{AO+OC} = \frac{\frac{\sigma_1-\sigma_3}{2}}{c\cdot\cot\phi + \left(\frac{\sigma_1+\sigma_3}{2}\right)} \quad \left\{\because \tan\phi = \frac{c}{AO}\right\}$$

$$\frac{\sigma_1-\sigma_3}{2} = c\cdot\cot\phi\cdot\sin\phi + \left(\frac{\sigma_1+\sigma_3}{2}\right)\sin\phi$$

Multiplied by 2, w/g

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

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

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

$$\left\{\begin{array}{l}\because \cos\phi = \sqrt{1-\sin^2\phi} = \sqrt{(1-\sin\phi)(1+\sin\phi)} \\ \therefore \dfrac{\cos\phi}{1-\sin\phi} = \dfrac{\sqrt{(1-\sin\phi)(1+\sin\phi)}}{(1-\sin\phi)} = \sqrt{\dfrac{1+\sin\phi}{1-\sin\phi}}\end{array}\right\}$$

$$\therefore \sigma_1 = \sigma_3\left(\frac{1+\sin\phi}{1-\sin\phi}\right) + 2c\sqrt{\frac{1+\sin\phi}{1-\sin\phi}}$$

$$\left\{\because \tan^2\left(45+\frac{\phi}{2}\right) = \frac{1+\sin\phi}{1-\sin\phi}\right\}$$

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

↳ At plastic equilibrium

Plastic equilibrium means on the verge of failure

Mohr - Coulomb's theory of shear strength :-

According to this theory shear strength of soil depends upon

(i) Cohesion in between particles on failure plane.

(ii) Internal angle of friction which accounts for interlocking and frictional resistance both (Dense sand have greater frictional angle).

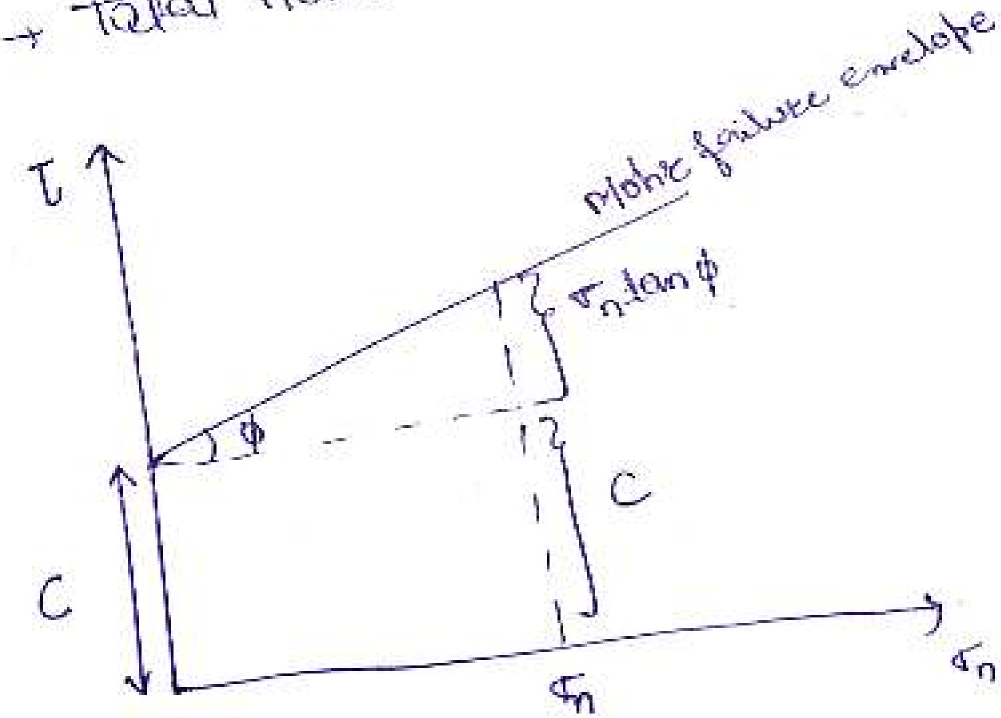
(iii) Normal stress on critical plane or failure plane which increases with the depth in soil.

$$S = C + \tan\phi \, \sigma_n$$

$C \rightarrow$ Total cohesion

$\phi \rightarrow$ Total friction angle

$\sigma_n \rightarrow$ Total normal stress on critical plane.



The above theory is found to be incorrect for the condition when water table is present hence the above theory is modified and improved by Terzaghi which is called as modified Mohr Columb's Theory. It is valid in all type of conditions.

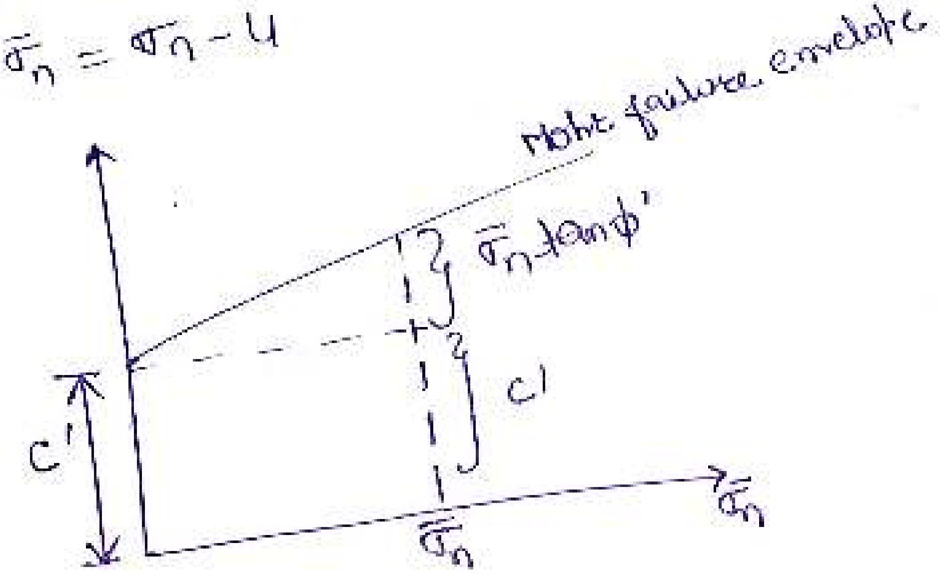
$$\boxed{S = c' + \bar{\sigma}_n \tan\phi'}$$

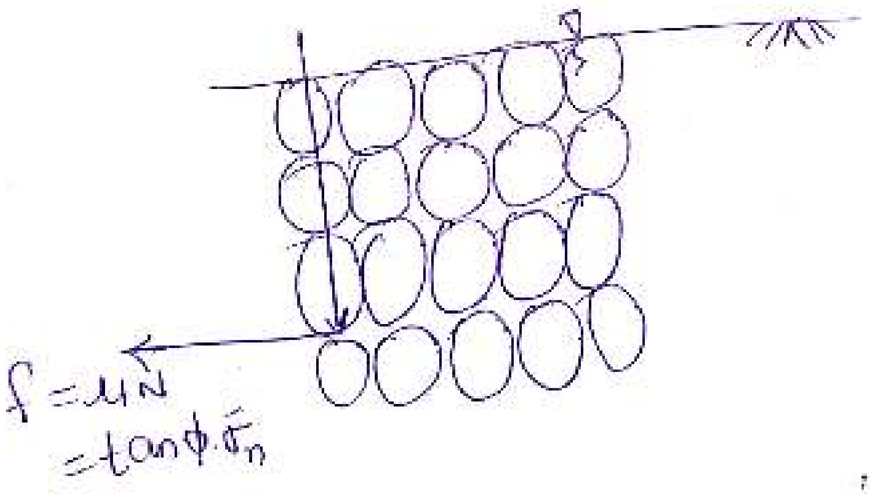
$c' \rightarrow$ Effective cohesion

$\phi' \rightarrow$ Effective friction Angle

$\bar{\sigma}_n \rightarrow$ Effective normal stress on critical plane

$\bar{\sigma}_n = \sigma_n - u$





Limitation of Mohr's Coulomb's Theory:-

i) Mohr failure envelope is approximated to a straight which is found invalid for over-consolidated soil. Practical result shows that Mohr failure envelope in Over consolidated soil is slightly curved.

ii) The Analysis is 2-D in which σ_1 and σ_3 is considered whereas actual stress condition in soil is 3-D in which $\sigma_2 = \sigma_3$.

x ——— x ——— x

In Lab

Types of Shear parameters:-

1.) Undrained / Total Shear parameter $[C_u, \phi_u$ or $C, \phi]$:-

– If saturated soil mass is subjected to shear loading and soil is undrained it means pore pressure will develop. In such case total stress should be used.

Note:-
If fast loading is done on saturated clay sample then in short term condition undrained / total shear parameters should be determined.

2.) Drained | Effective shear parameter [c_d, ϕ_d or c', ϕ']:-

- If loading rate is slow and expulsion of pore water completes during loading then pore pressure will become zero. In such case effective stress should be used.

Note:-
Loading on sand at slow rate for short term and long term condition both whereas in clay for long term condition under slow loading rate, drained | effective shear parameters should be determined.

Note:-
c and ϕ are not fundamental properties of soil, these are depend on the type of test, water content and drainage condition under which the testing is done. Test must represents the field condition. Field condition could be drained or undrained condition, sudden or gradual loading, granular or clayey soil.

Determination of shear parameters (c and ϕ):-

1.) Laboratory test →

(i.) Triaxial test
(ii.) Unconfined compression test
(iii.) Direct shear test
(iv.) Vane shear test
(v.) Ring shear test
(vi.) Torsion balance test

2) Field Method :
(i.) Vane shear test
(ii.) SPT
(iii.) CPT

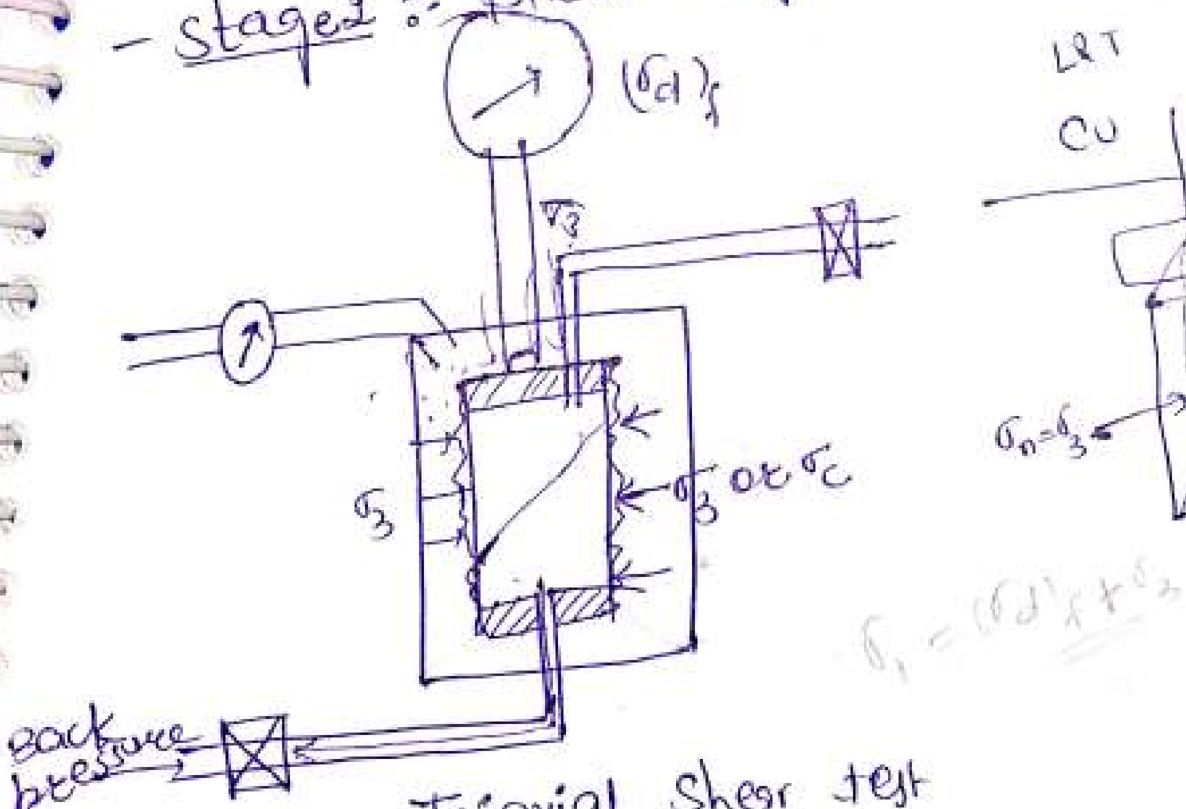
3) Empirical Equation :-

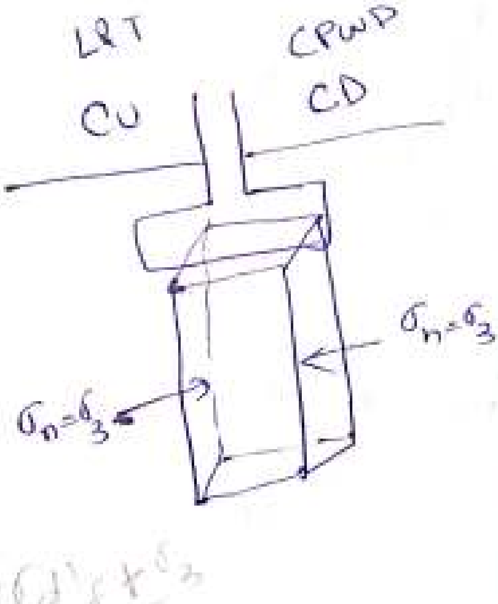
$$\frac{C_u}{\bar{\sigma}_n} = 0.11 + 0.0037 I_p$$

$C_u \rightarrow$ undrained cohesion
$\bar{\sigma}_n \rightarrow$ Eff. stress on critical plane
$I_p \rightarrow$ Plasticity Index in %.

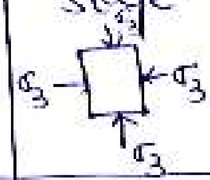
(i.) Triaxial Shear Test :-

- A cylindrical soil specimen is prepared from a saturated soil mass which is enclosed inside impermeable rubber membrane. The length of the sample is usually 2 times of its dia.
- This test is conducted in two stages.
- Stage 1 :- Confining pressure stage / cell pressure stage.
- Stage 2 :- Shear stage / Deviator stage.





Triaxial Shear test

Type of Triaxial test	Stage I confining stage	Stage II Shear stage	Suitable field condition
			
Unsolidated Undrained (Q) (UU Test / Quick test)	X	X	clay in short term
Consolidated Drained (S) CD test / Slow test	✓	✓	clay in long term & sand in long term and short term
Consolidated Undrained (CU Test) R	✓	X	Sudden drawdown in hydraulic structure.

Note:-

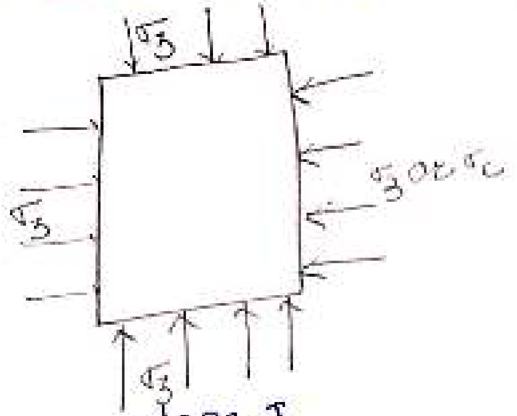
Unsolidated drained test is not performed practically because confining pressure acts from very long time and if soil is unconsolidated in long time then it cannot be drained in small period of shear loading.

Procedure of Triaxial shear test:-

(1.) Triaxial test is performed in two stages.

Stage 1:-

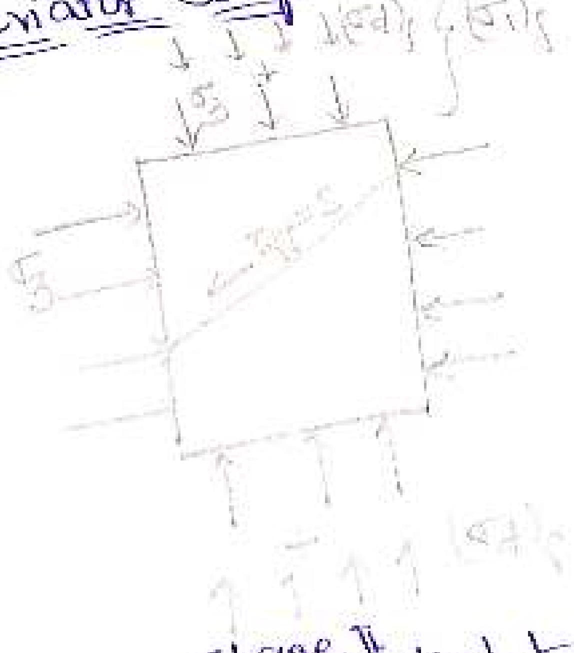
Confining pressure stage / cell pressure stage / Consolidation stage.



Stage I

— All around confining pressure (σ_3 or σ_c) is applied using external water pressure.
— If test is unconsolidated then drainage valves will be closed and if test is consolidated then drainage valves will be opened.

Stag II :-
Shear stage | Deviator stage :



— Confining pressure stage II is kept constant and additional axial stress is applied called as deviator stress (σ_d). Deviator stress is gradually increases until the soil fails in shear.

- If test is undrained then valves will be closed and if test is drained then valves will be opened.

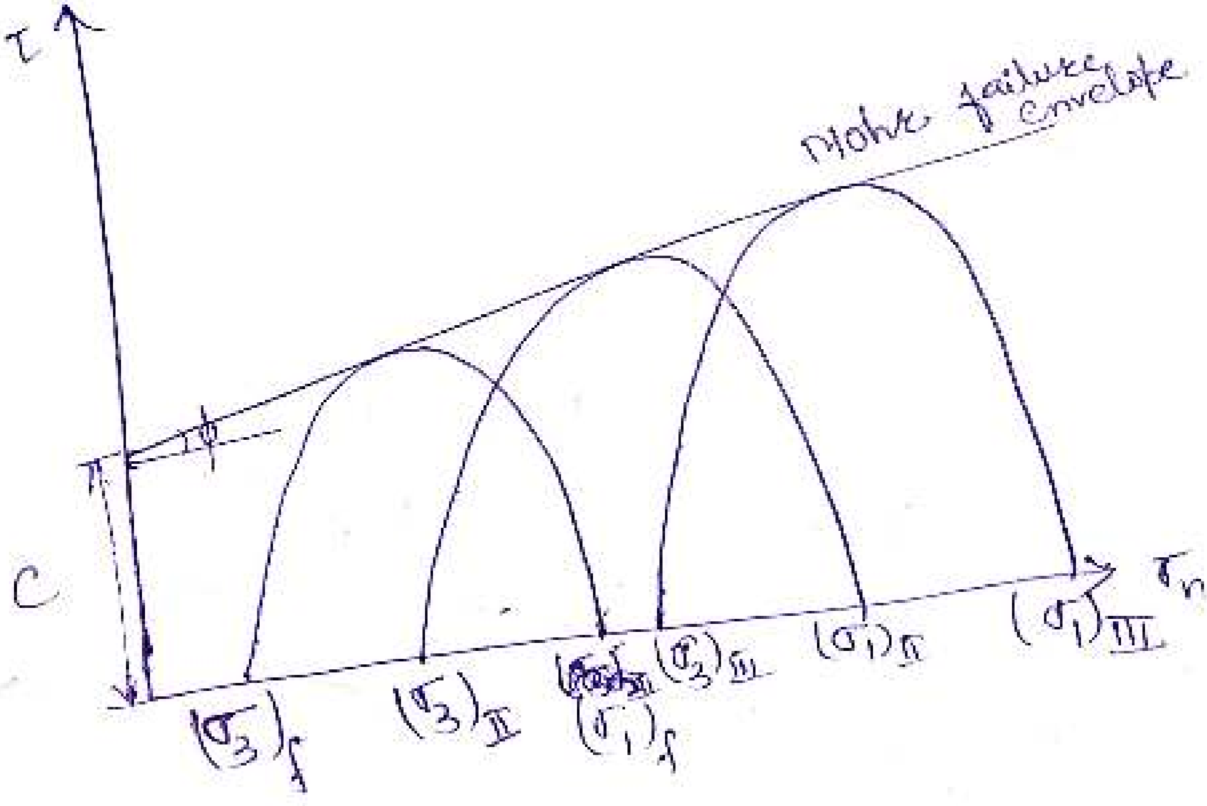
Note 1:-

The deviator stress at failure is termed as Confined compressive strength (CCS). **

$$\boxed{CCS = (\sigma_d)_f = (\sigma_1 - \sigma_3)_f = \frac{P}{A_f}}$$

$A_f \rightarrow$ area of cross-section at failure.

- For σ_1 and σ_3 at failure, a mohr circle is drawn and test is repeated on another sample of same soil with changed value of minor principal stress (σ_3) and corresponding σ_d and σ_1 is noted.
- Mohr circle for each case is drawn and common tangent to these mohr circles represents Mohr failure envelope from which C and ϕ can be determined.



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

Note 2:-

- If Mohr circle are plotted for total stress then total / undrained shear parameters can be obtained.
- If we used effective stress to draw the Mohr circle then effective / drained shear parameters can be obtained.

CD Test —— $\left\|\begin{matrix}\sigma_1 = \bar{\sigma}_1 + \cancel{u} \\ \sigma_3 = \bar{\sigma}_3 + \cancel{u}\end{matrix}\right\|$ — $S^*_{n} \rightarrow \left\{\begin{matrix} c_d, \phi_d \\ \text{or} \\ c', \phi' \end{matrix}\right\}$

UU Test and CU Test —

- $\left\|\begin{matrix}\sigma_1 = \bar{\sigma}_1 + u \\ \sigma_3 = \bar{\sigma}_3 + u\end{matrix}\right\|$ — S^*_{n} — $\left\{\begin{matrix} c_u, \phi_u \\ \text{or} \\ c, \phi \end{matrix}\right\}$
- $\left\|\begin{matrix}\bar{\sigma}_1 = \sigma_1 - u \\ \bar{\sigma}_3 = \sigma_3 - u\end{matrix}\right\|$ — S^*_{n} — $\left\{\begin{matrix} c_d, \phi_d \\ \text{or} \\ c', \phi' \end{matrix}\right\}$

Note 3:- Area of cross section at the time of failure

Area (A_f):-

Let A_0 = Initial c/s Area $= \frac{\pi}{4} D^2 = \frac{V_0}{L_0}$

$$\{\text{Area}\}_{\text{failure}} = \left(\frac{\text{volume}}{\text{Length}}\right)_f$$

$$A_f = \frac{V_0 \pm \Delta V}{L_0 - \Delta L} = \frac{V_0 \left[1 \pm \frac{\Delta V}{V_0}\right]}{L_0 \left[1 - \frac{\Delta L}{L_0}\right]}$$

Absinal

$$\boxed{A_f = A_0 \left[\frac{1 \pm \varepsilon_v}{1 - \varepsilon_i}\right]} \quad **$$

where $\varepsilon_v \rightarrow$ volumetric strain $\frac{\Delta V}{V_0}$

$\varepsilon_i \rightarrow$ Axial strain $= \frac{\Delta L}{L_0}$

If UU Test is conducted on soil mass

$\Delta V = 0 \quad \varepsilon_v = 0$

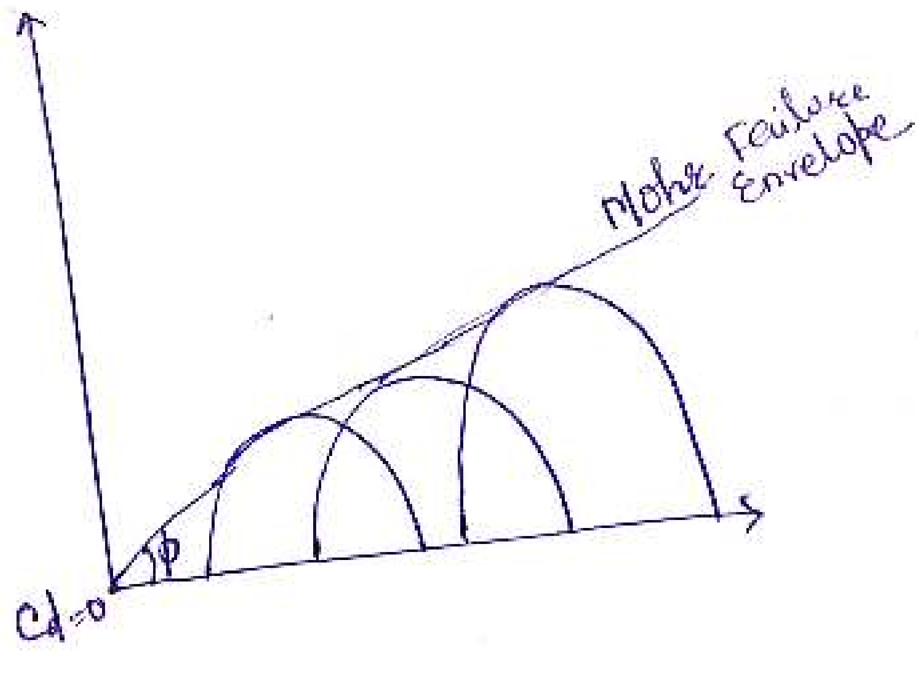
$$\boxed{A_f = \frac{A_0}{1 - \varepsilon_i}}$$

Notes:-

Type of Mohr Failure Envelope:-

- Denpending upon type of soil and loading condition and drainage condition, following types of Mohr failure Envelope may be obtained:-

i.) For sand:-

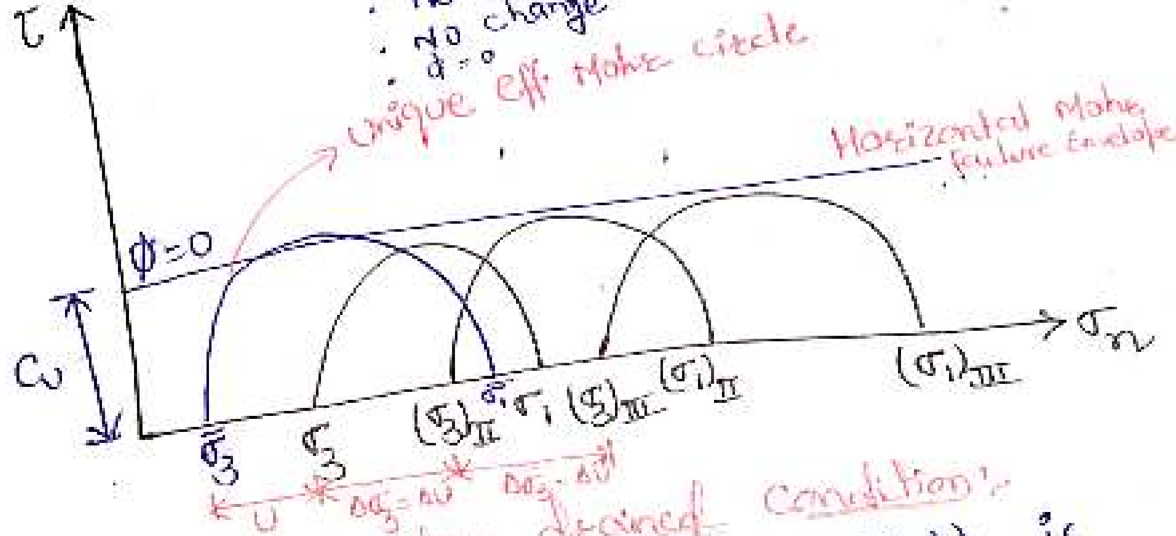


ii) Saturated clay under undrained condition:-

Immediate - initial time then UU test.

- If unconsolidated undrained test (UU Test) is conducted on saturated clay then the behaviour of clay is cohesive and no friction is observed.
- When test results are plotted then radius of all the Mohr circles is found to be equal and if Mohr circles are plotted in effective stress then unique effective Mohr circle is obtained

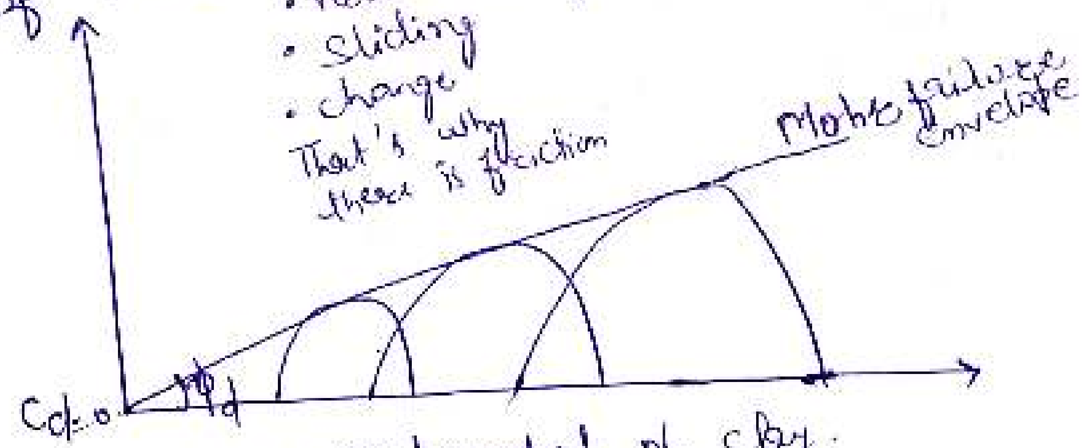
· No volume change
· No sliding
· No change
· $\phi = 0$



iii) Saturated clay under drained condition:-

- If consolidated drained Test (CD Test) is conducted on saturated clays then behaviour of N_c clay is similar to that of sand.

· volume change
· sliding
· change
That's why there is friction

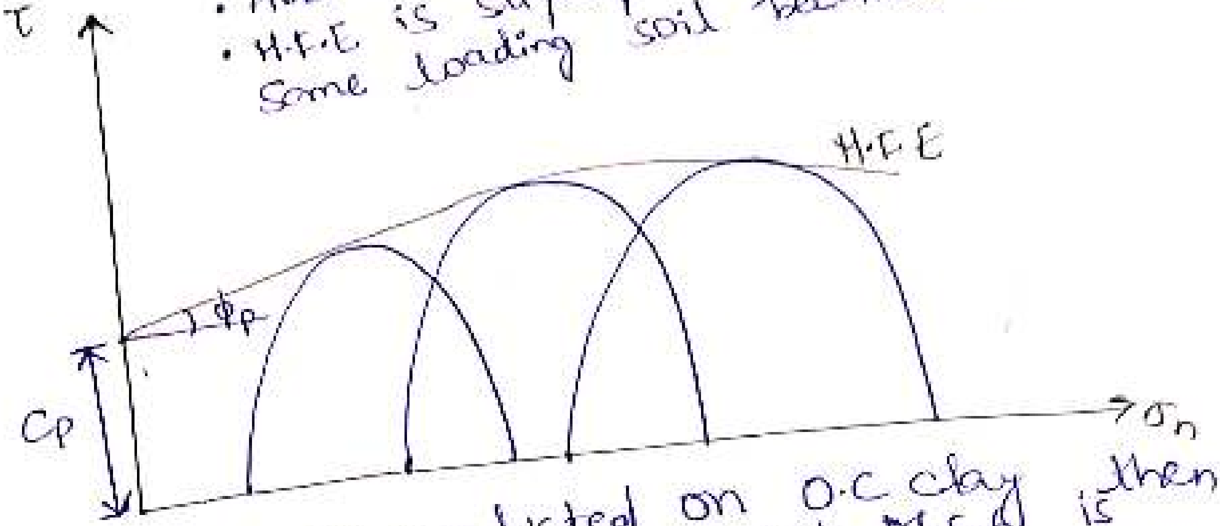


For Saturated N_c clay.

iv) Partially saturated clay :-

- Partially saturated clay under drained and Undrained condition behaves similar to that of silt (C-ϕ soil) and Mohr failure Envelope obtained slightly curved.

• Air will also take pressure
• M.F.E is slightly curved ∵ after some loading soil becomes saturated.



v) If CD Test is conducted on O.C clay then it behaves like silt (C-ϕ) soil. M.F.E is slightly curve.

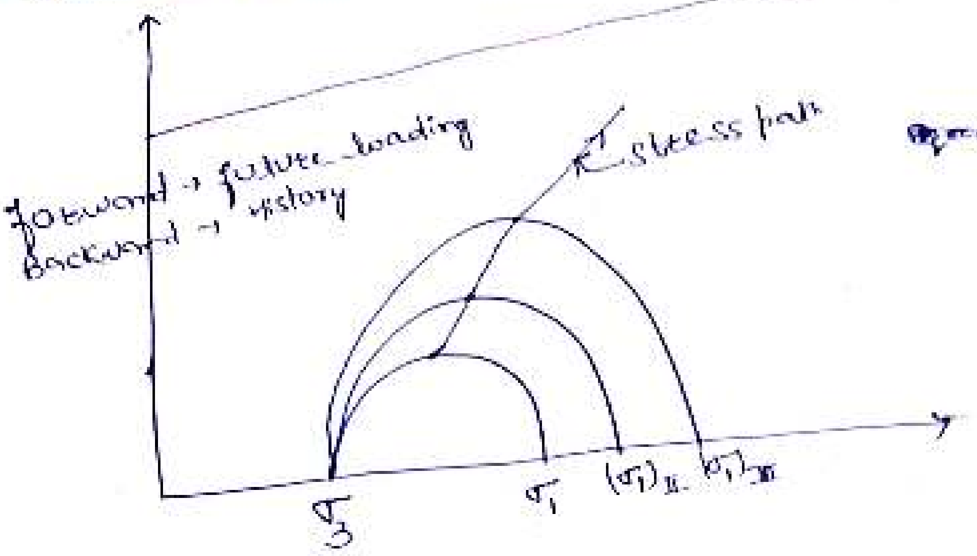
⊕ If CU test is conducted on saturated clay then it behaves like silt (C-ϕ soil).

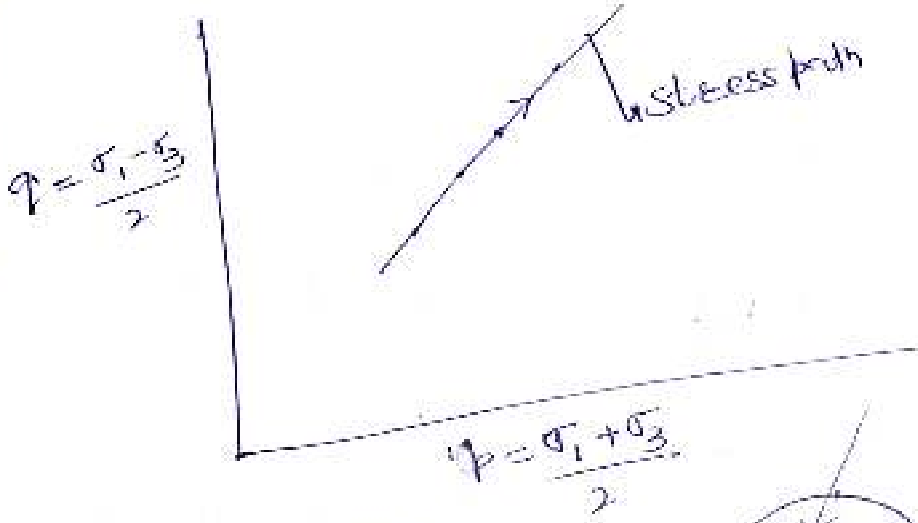
Notes :-

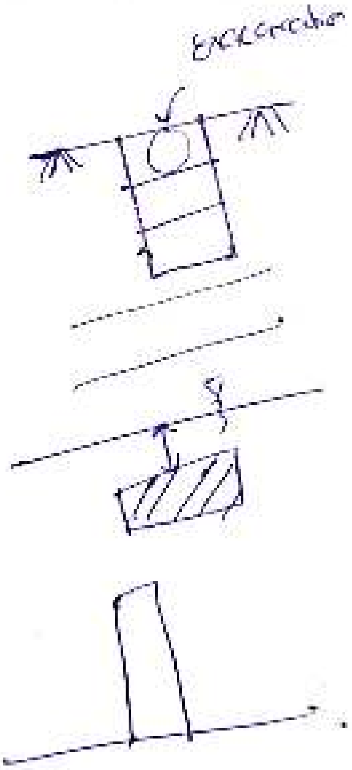
Stress path :-

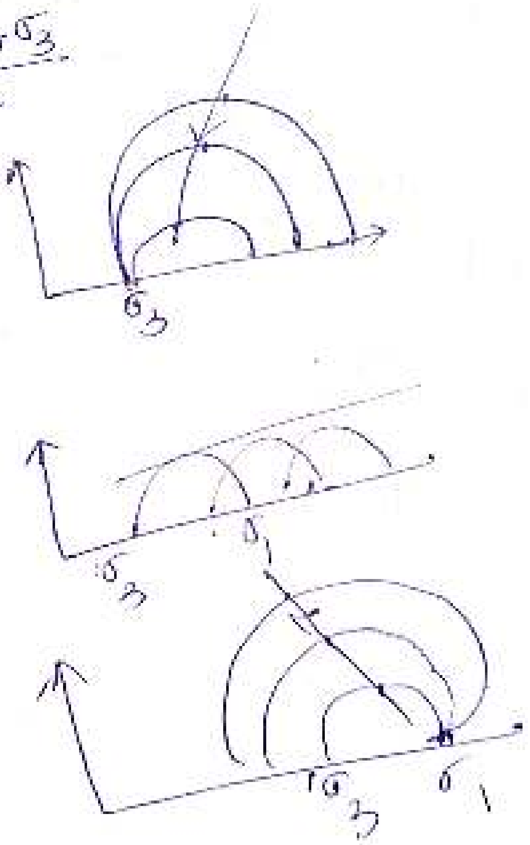
- A stress path is a curve which shows change in stresses as the load acting on the soil specimen changes. In other words, it is the locus of points of maximum shear stress experienced by a specimen as the load change take place. It can also represents in p-q plot.

M.F.E


Forward → future loading
Backward → history
stress path
σ_3
σ_1
$(\sigma_1)_{II}$
$(\sigma_1)_{III}$


stress path
$q = \frac{\sigma_1 - \sigma_3}{2}$
$p = \frac{\sigma_1 + \sigma_3}{2}$


Excavation


σ_3
σ_3
σ_1
σ_3
σ_1

Merits of Triaxial Shear Test:-

i) Failure plane is not predetermined
ii) It is the weakest plane.
iii) Drainage condition is controlled.
iv) There is mechanism to measure pore pressure.
v) Suitable for all types of soil.
vi) The stress distribution is much more uniform.
vii) Results are accurate.

2. UNCONFINED COMPRESSION TEST:-

- It is special case of triaxial test in which confining pressure is zero it means there is no first stage therefore no rubber membrane is required.
- Without rubber membrane dry soil and sand cannot be held in position hence this test can be conducted in saturated silt and clay but is more stable for saturated clay.
- The saturated sample is subjected to axial loading and net deviator stress at failure is σ_d.

Note:-

- Deviator stress at failure is termed as unconfined compressive strength (q_u). **

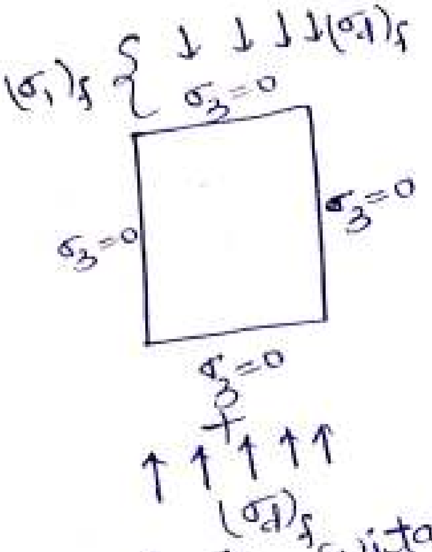
$$\boxed{UCS = (\sigma_d)_f = (\sigma_1 - \cancel{\sigma_3}^{0})_f = \frac{P}{A_f}}$$

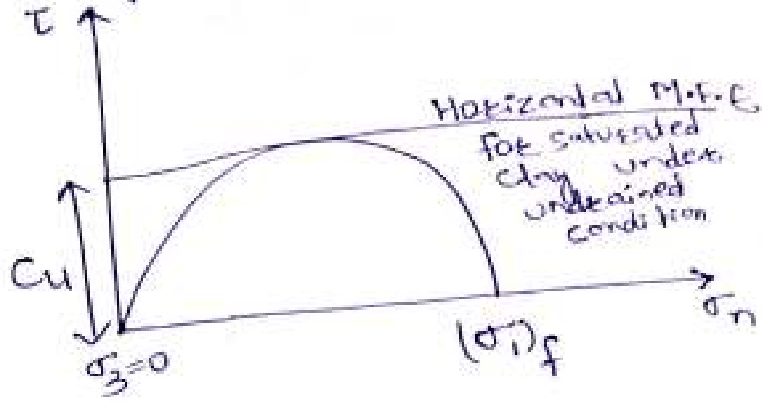
$A_f \rightarrow$ Area of c/s at failure.

Unique eff. Mohr circle $\rightarrow$ UU Test

Unique Mohr circle $\rightarrow$ UCS test

- Because $\sigma_3 = 0$ hence σ_1 at failure will be same for each sample of same soil hence unique Mohr circle is obtained which passes from origin.





- This test is suitable for clay under undrained condition for which Mohr failure envelope is a horizontal line.

Unconfined compressive strength

$$UCS = (\sigma_d)_f = (\sigma_1 - \cancel{\sigma_3}^{\,0})_f$$

$$\sigma_1 = \cancel{\sigma_3}^{\,0} \tan^2\left(45 + \frac{\phi}{2}\right) + 2C\tan\left(45 + \frac{\phi}{2}\right)$$

$$\sigma_1 = 2C\tan\left(45 + \frac{\phi}{2}\right)$$

$$\boxed{UCS = 2C\tan\left(45 + \frac{\phi}{2}\right)}$$ → Silt

If soil is clay under undrained condition $(\phi = 0)$

$$\boxed{UCS = 2C}$$ **

or

$$\boxed{C = \frac{UCS}{2}}$$ **

3. VANE SHEAR TEST :-

- In this test, there is no mechanism to measure pore pressure and drainage is not controlled.
- It is preferred for soil saturated clay like marine clay to fine undrained shear strength, liquid limit and sensitivity.
- The vane is punched into the soil and torque is applied.
- The vane is calibrated to a spring having torsional stiffness K. The torque at shear failure can be determined as

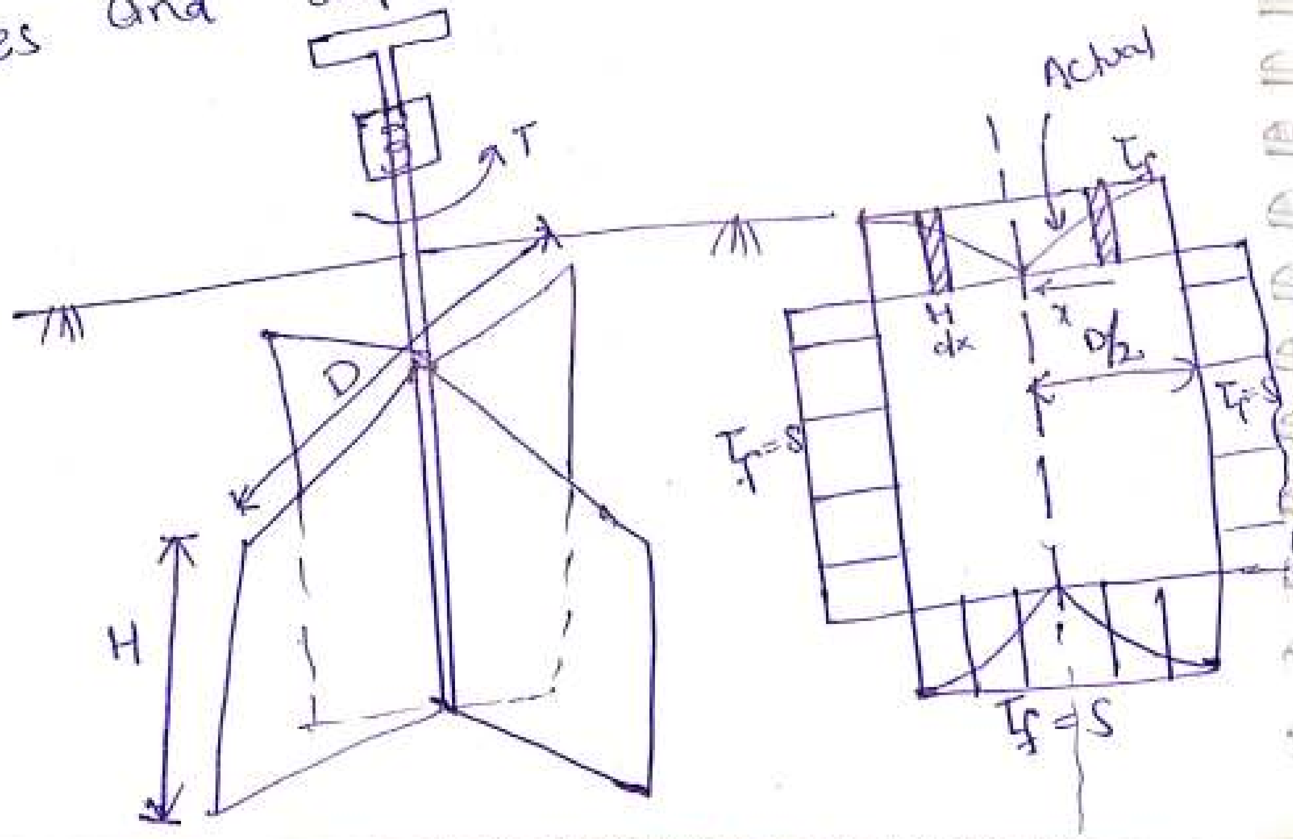
$$\boxed{T = K\theta}$$

where $\theta \rightarrow$ angle of rotation of vane at the shear failure.

- There are two mechanisms to conduct this

(a) ~~two~~ way shearing :-

In this case, vane is punched into the ground such that top of vane is at some depth below the ground level.
In this case, shearing will occur at sides and top and bottom both.



$\Rightarrow T = (T_1)_{side} + (T_2)_{top \& bottom}$

$\Rightarrow T_1 = \text{force} \times \text{distance} = [\tau_f \times \pi D H] \times \frac{D}{2} = \tau_f \frac{\pi D^2 H}{2}$

$\Rightarrow T_2 = 2 \times [\tau_f \times 2\pi x \cdot dx] \cdot x$

$T_2 = 4\pi \tau_f \cdot \int_0^{D/2} x^2 \cdot dx$

$T_2 = 4\pi \tau_f \left| \frac{x^3}{3} \right|_0^{D/2}$

$T_2 = \frac{\cancel{4}}{3} \pi \tau_f \cdot \frac{D^3}{\cancel{8}\, 2}$

$T_2 = \frac{\pi \tau_f D^3}{6}$

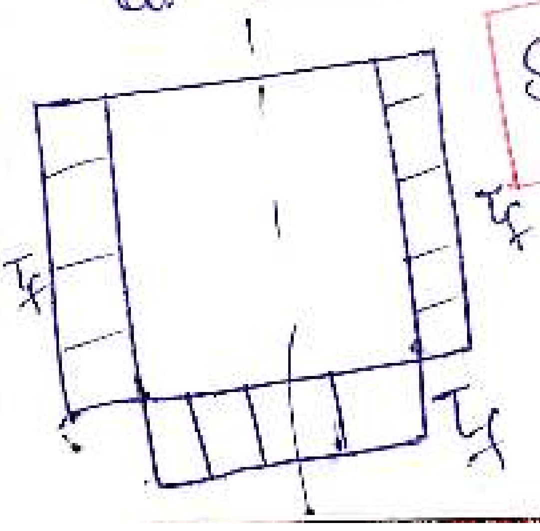
$\therefore T = T_1 + T_2 = \tau_f \pi D^2 \frac{H}{2} + \tau_f \cdot \frac{\pi D^3}{6}$ **

$$\boxed{S = \tau_f = \frac{T}{\pi D^2 \left(\frac{H}{2} + \frac{D}{6}\right)}}$$

B.) one way shearing :-

- If vane is punched such that top ~~and~~ of vane is at G.L or performed in a bore hole then shearing will occur at sides and bottom only.

$$\boxed{S = \tau_f = \frac{T}{\pi D^2 \left(\frac{H}{2} + \frac{D}{12}\right)}}$$ **



Determination of sensitivity of clay :-

$$S_t = \frac{(UCS)_{undisturbed}}{(UCS)_{remoulded}} = \frac{(2C)_{undisturbed}}{(2C)_{remoulded}}$$

or

$$S_t = \frac{\text{Undisturbed vane shear strength}}{\text{Remoulded vane shear strength}}$$

$$S_t = \frac{\dfrac{(T)_{undisturbed}}{\pi D^2\left(\frac{H}{2}+\frac{D}{6}\right)}}{\dfrac{(T)_{remoulded}}{\pi D^2\left(\frac{H}{2}+\frac{D}{6}\right)}} = \frac{(T)_{undisturbed}}{(T)_{remoulded}}$$

$$\text{Shear strength } (S) = C + \sigma_n \tan\cancel{\phi}^{\,0}$$
$$= \underline{\underline{C}}.$$

4. **DIRECT SHEAR TEST :-**

There is no mechanism to measure pore pressure and drainage is not controlled hence this test is preferred under drained condition. However, it can be conducted under undrained condition also.

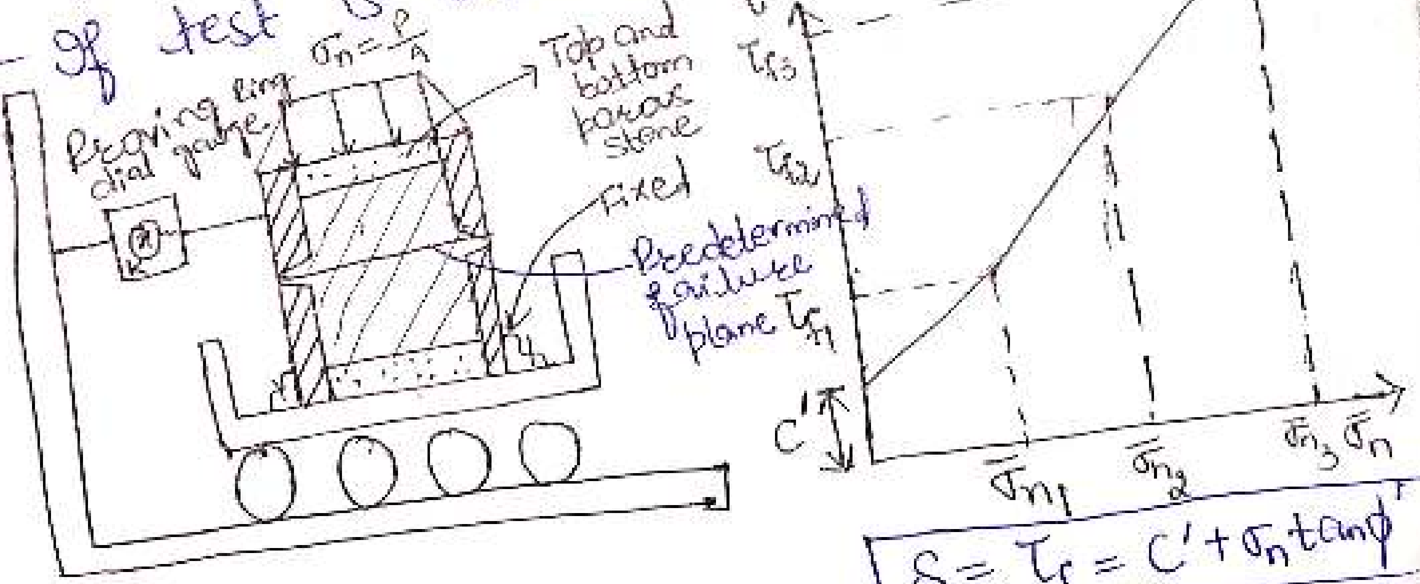
- Shear box is either square or circular having size 60 to 90 mm.

- There are two mechanism to conduct this test :-
- i) Strain controlled (Motor and screw)
- ii) Stress controlled (with the help of pulley & weight).
- Strain controlled is prefered in most of the cases.
- A saturated soil mass is placed in shear box. Shear Box has two parts, upper and lower which are separated to each other.
- A normal force is applied from the top and water is allowed to dissipate out.
- when expulsion of pore water stops then shearing is ~~restored~~ introduced on a horizontal plane.
- At constant normal stress ($\sigma_n = P/A$) shear stress at failure is determined as $\boxed{\tau_f = \frac{kN}{A}}$

where $k \rightarrow$ proving ring constant

$N \rightarrow$ proving ring reading at failure.

- σ_n and τ_f are plotted on the graph and test is repeated at different values of σ_n and (corresponding shear stress at failure (τ_f is noted)) and plotted on the graph.
- If test is drained then $\sigma_n = \bar{\sigma}_n + \cancel{u}^0$



$$\boxed{S = \tau_f = C' + \bar{\sigma}_n \tan\phi'}$$

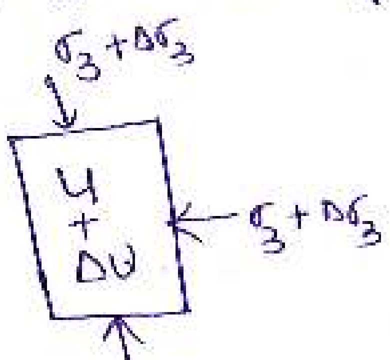
Limitations :-

- Failure plane is predetermined which may not be the weakest plane.
- There is no mechanism to measure pore pressure.
- Drainage is not controlled.
- Stress distribution is not uniform.

Pore pressure parameters :- (skempton)

- If it is not possible to measure pore pressure by practical means then theoretical approach given by skempton can be adopted.
- Parameter B and A represents the response of change in pore pressure due to change in vertical pressure under undrained condition.
- The pore pressure change can be classified into two stages :-

Stage I :- Confining pressure stage :- The parameter B represents the ratio of change in pore pressure to the change in confining pressure.



$$\boxed{B = \frac{\Delta u_c}{\Delta\sigma_c} = \frac{\Delta u_c}{\Delta\sigma_3}}$$

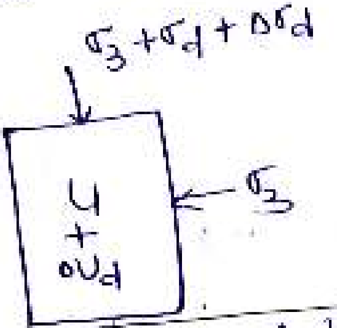
For dry stage :→ $B = 0$

For saturated stage :→ $B = 1$

For partially saturated stage :→ $0 < B < 1$

Stage II :- Shear stage / Deviator stage :-

- The parameter $\bar{A}$ is defined in terms of another parameter A' such that $\bar{A}' = A \cdot B$.
- The parameter $\bar{A}'$ represents the ratio of change in pore pressure to the change in deviator stress in shear stage under undrained condition.



$$\boxed{\bar{A} = A \cdot B = \frac{\Delta u_d}{\Delta \sigma_d} = \frac{\Delta u_d}{\Delta \sigma_1 - \Delta \sigma_3}} \quad **$$

Note 1 :-

- The parameter A depends upon degree of saturation, O.C.R, strain in soil and stratification of soil.
- The value of parameter A may be as low as -0.5 (for dense sand and OCC) and may be as highest +3 for loose sand.
  over consolidated clay
- If confining pressure and deviator stress both changes then total change in pore pressure can be determined as

$$\Delta U = \Delta U_c + \Delta U_d$$

$$\Delta U = B \cdot \Delta \sigma_3 + \bar{A} (\Delta \sigma_1 - \Delta \sigma_3)$$

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

$$\boxed{\Delta U = B (\Delta \sigma_3 + A(\Delta \sigma_1 - \Delta \sigma_3))} \quad **$$

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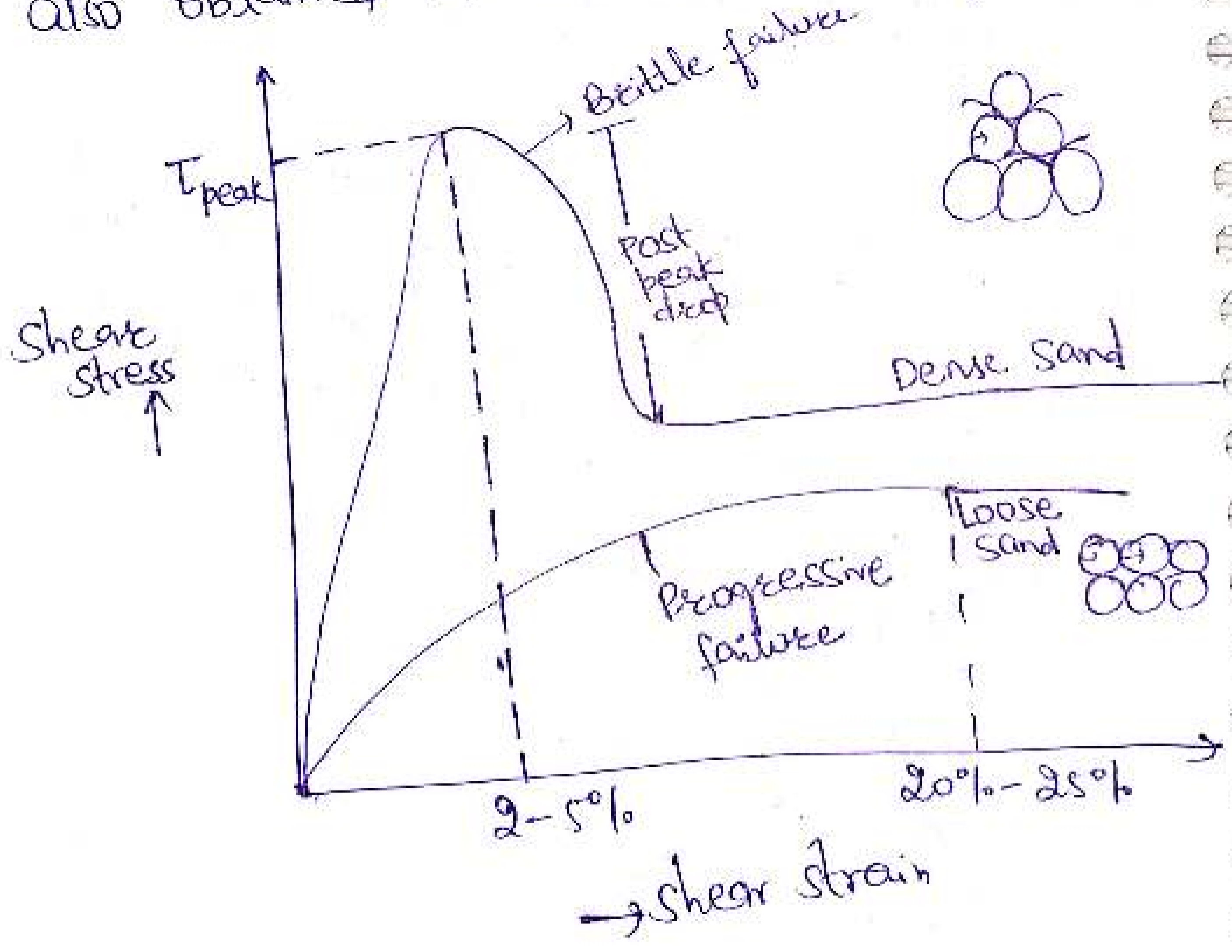
Shear characteristics of Sand :-

- In Dense sand the shear strength is due to Interlocking and friction both. Interlocting resistance may be 20% to 30% of total And at 2% to 5% of strain interlocking fails and sudden decrease in shear strength is recorded. This type of failure is termed as Brittle failure which can also be obtained in Undisturbed sensitive clay and O.C.C.

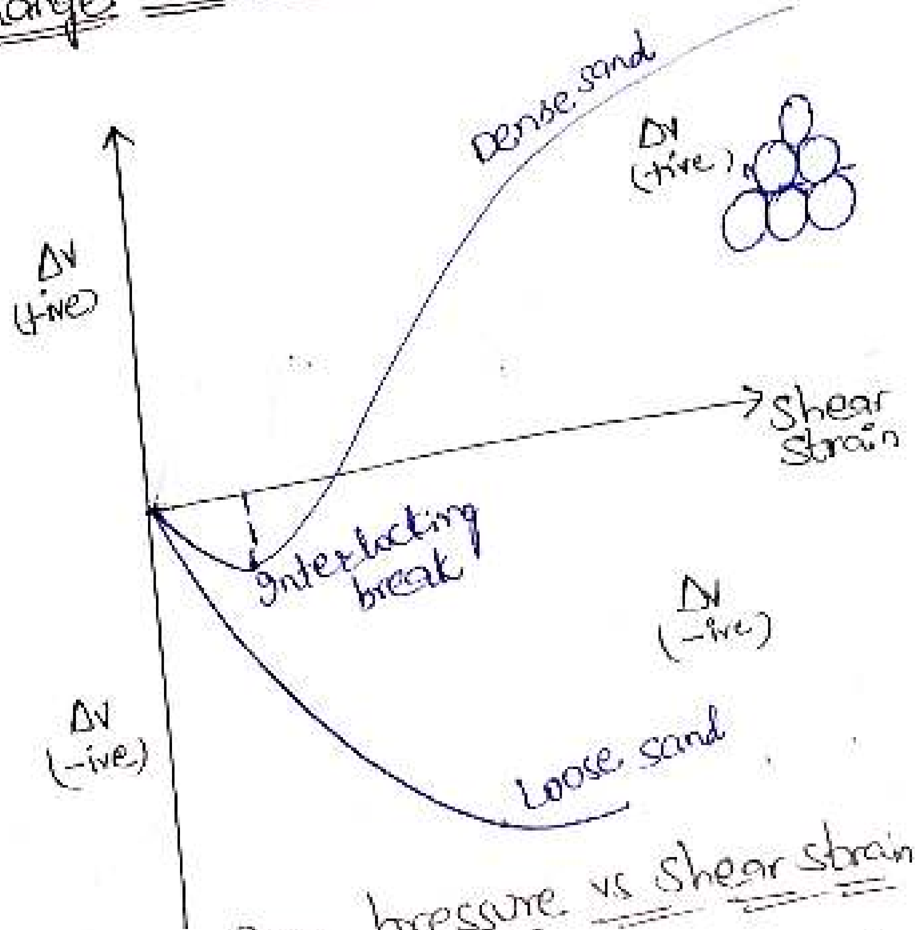
- Loose sand have only frictional resistence. A loose sand volume continue to decrease and clear failure does not occur hence failure is assumed when strain is reached 20% to 25%. This type of failure is termed as progressive failure which is also obtained in remoulded clay and NCC



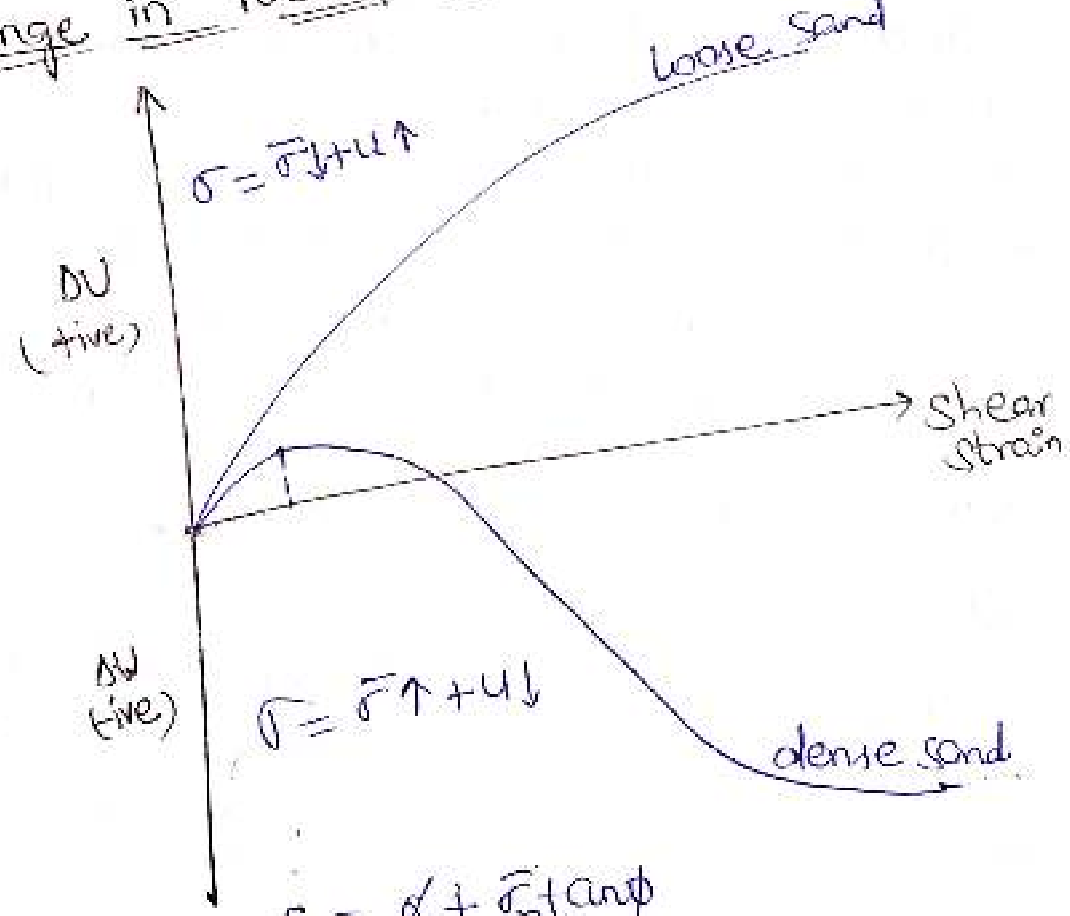
Effect of seismic loading on saturated sand:-

Change in volume vs. shear strain:-

(I.) change in volume

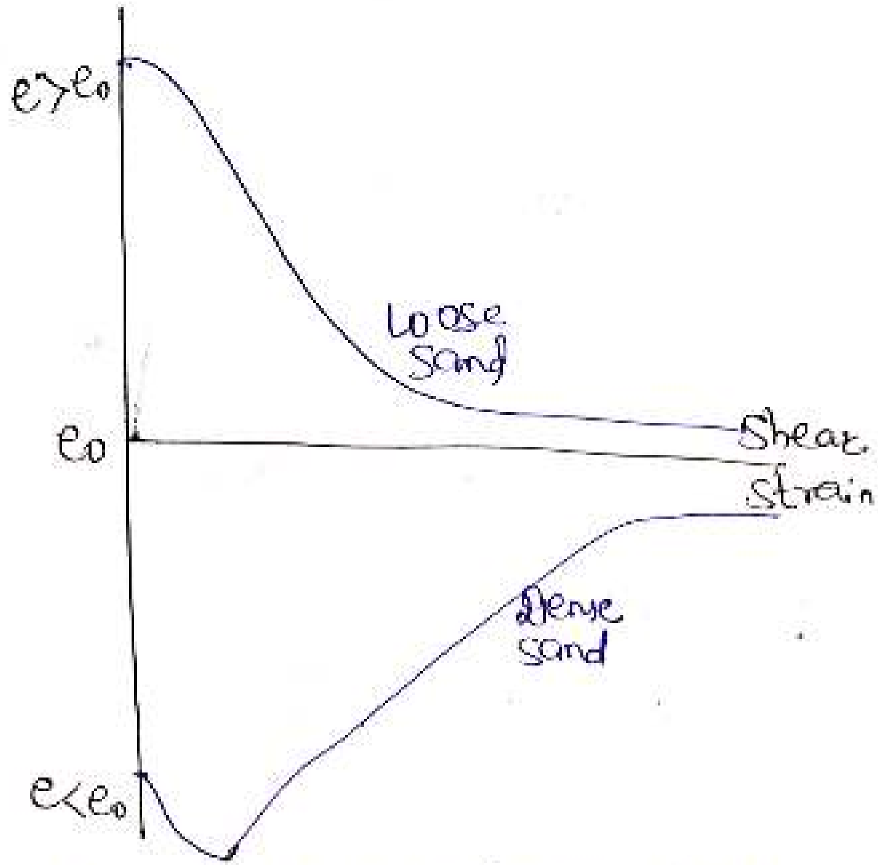


(II) Change in Pore pressure vs Shear strain :-



$S = c + \bar{\sigma}_n \tan\phi$

III.) Void Ratio vs. Shear Strain :-



$e_0 \rightarrow$ critical void Ratio ; [at which volume chang will be neglible in soil, due to shearing.]

- In dense saturated sand, due seismic disturbance, volume increases after interlocking break as a result pore pressure change becomes negative hence effective stress increases consequently shear strength increases and failure does not occur.

Liquefaction :-

- In loose saturated sand, due to seismic disturbance (earthquake) or dynamic loading, volume decreases hence pore pressure change becomes +ve. Due to built up of high pore pressure, effective stress decreases and sometimes it may decreases to zero. Consequently decrease in shear strength is recorded and large settlement of foundation suddenly occurs along with vertical upward flow of muddy water. Such a phenomenon is called liquefaction of sand.

Generally, it is observed near the sea and rivers during Earthquake.

In loose sand → Fine sand liquefaction.