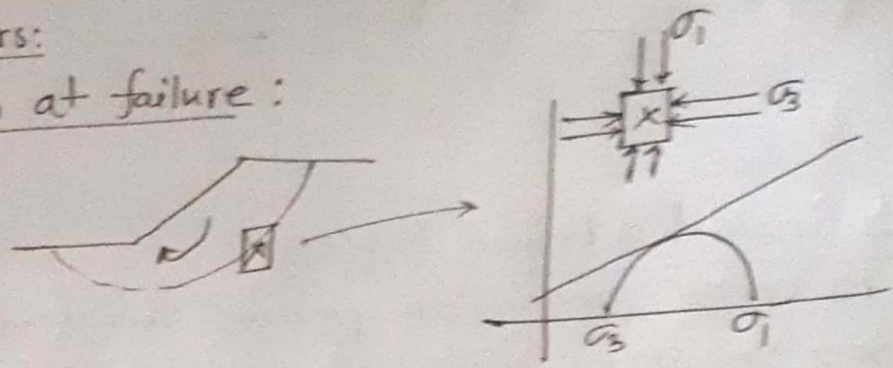
Types of Drainage controlled Triaxial Shear Tests:

1. Consolidated-drained test (CD test)
2. Consolidated-Undrained test (CU)
3. Unconsolidated-Undrained test (UU

Some Parameters:

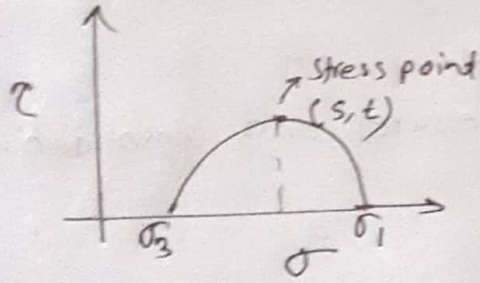
i) $\sigma_1 - \sigma_3$ Relation at failure:



$$\sigma_1 = \sigma_3 \tan^2(45+\phi/2) + 2c\tan(45+\phi/2)$$

$$\sigma_3 = \sigma_1 \tan^2(45-\phi/2) - 2c\tan(45-\phi/2)$$

ii) Stress point:

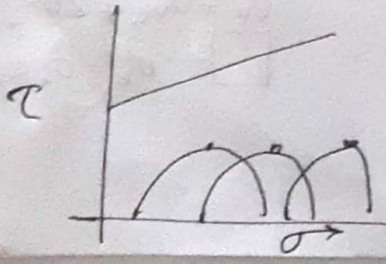


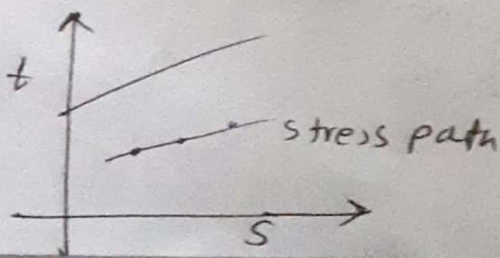
$$s = \frac{\sigma_1 + \sigma_3}{2}$$

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

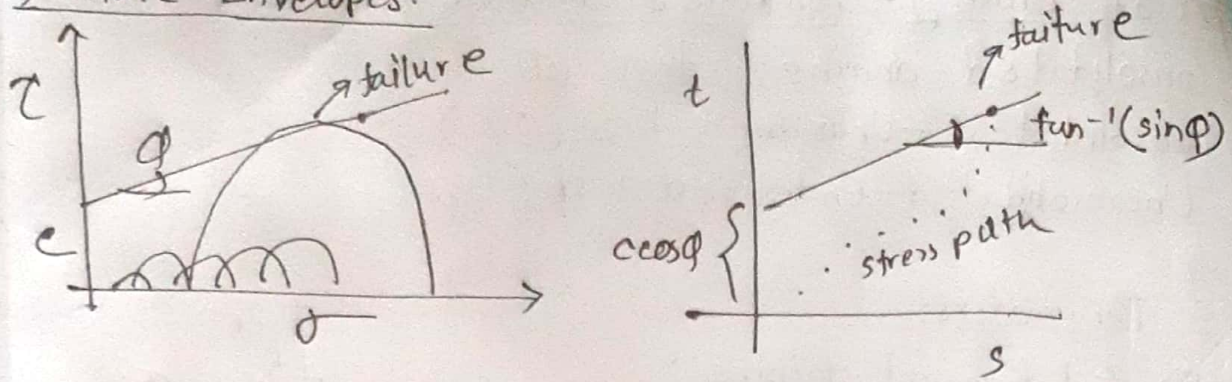
iii) Stress Path:

Results of triaxials tests can be represented with diagrams called stress paths. Stress path is the locus of stress points. It's a convinient way to keep track of the progress in loading with respect to failure envelope.





(iv) Failure Envelopes:



☐ Consolidated Drained Test (CD)

* no excess pore water pressure throughout the test.
* very slow shear buildup to avoid buildup of pore water pressure. This can take days, which is not desirable

* Gives c' and ϕ'

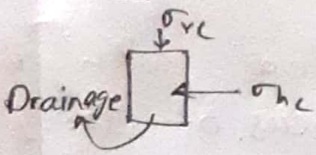
* Use c' and ϕ' to analyse fully drained situations.
* Very uncommon

Procedure:

- Drainage connection is opened.
- Dissipation of excess pore pressure and consolidation takes place. Thus $u=0$

Total, σ	=	u	+	Effective, σ'

Step: 1 At the end of consolidation:



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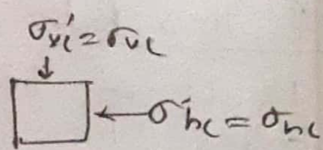
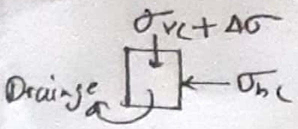


Step: 2 During axial stress increase



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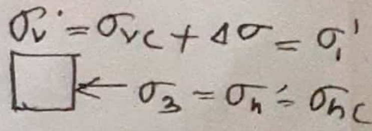
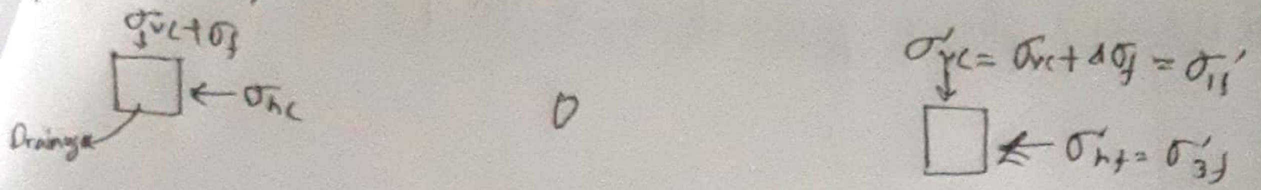
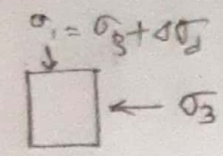


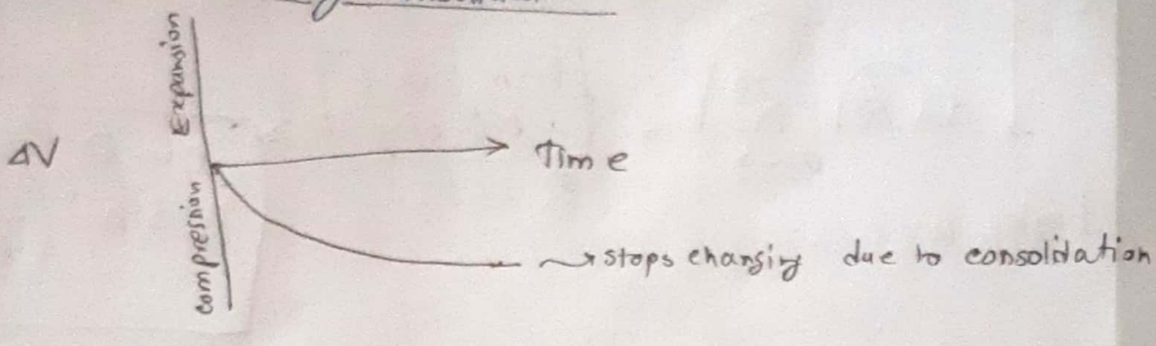
Step: 3 At failure:



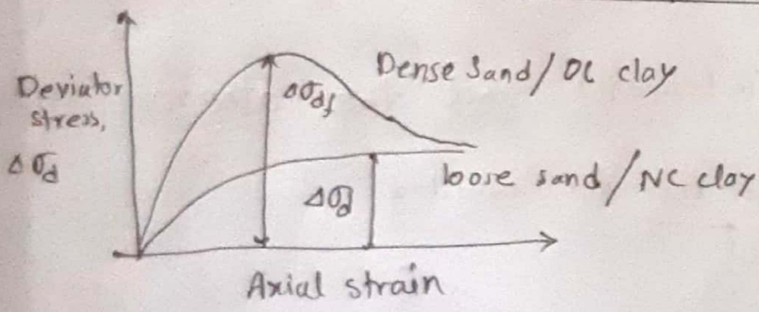
Deviator stress, $\sigma_d = \sigma_1 - \sigma_3$

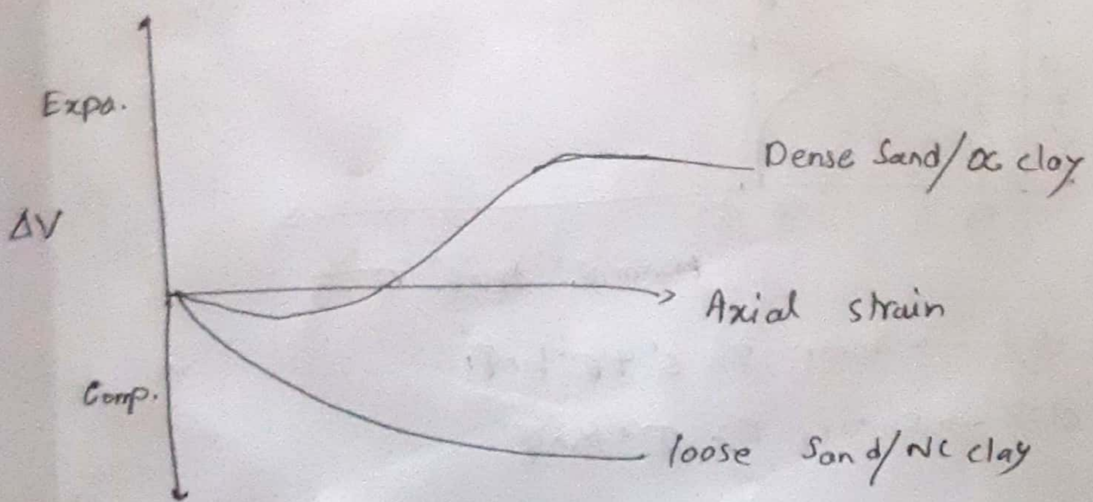


* Volume change during consolidation:

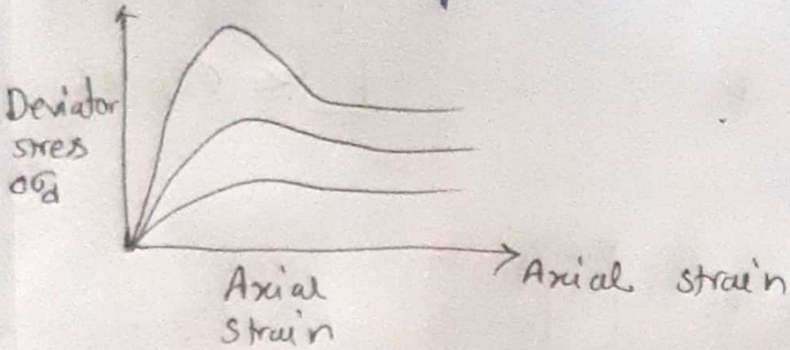


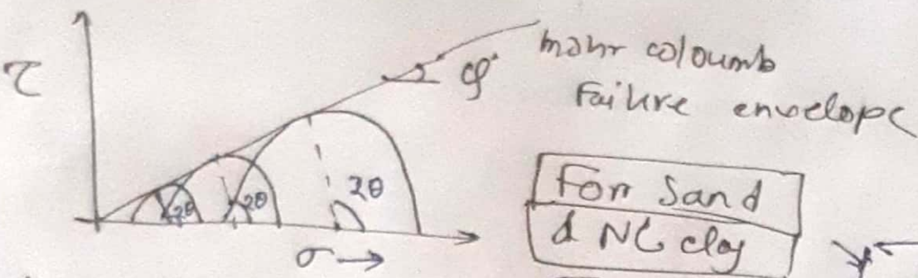
* Stress-strain-volume relationship during shearing:

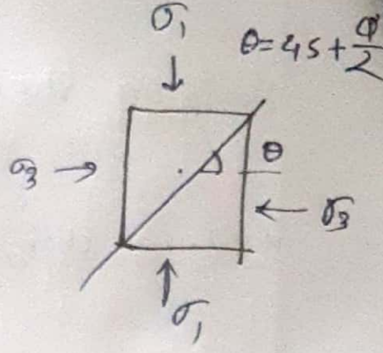




* Determine strength parameters c and φ:







For Sand & NC clay

Here, u=0 ∴

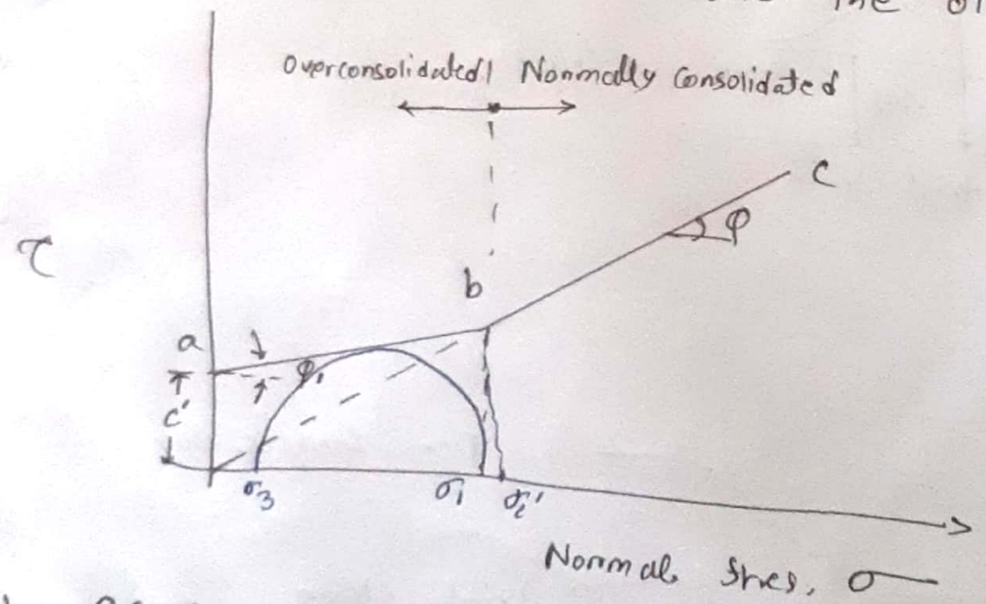
$c = 0$

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

For OC clay $c \neq 0$

when clay is overconsolidated with previous chamber pressure σ_3, we get two distinct branches in failure envelope.

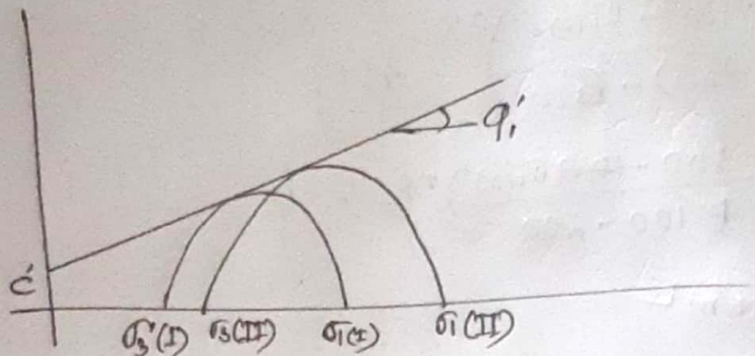
One has a cohesion intercept and the otherone dosen't.



ab = OC clay → Shear, $\tau = c' + \sigma' \tan\varphi_1$

bc = NC clay → Shear, $\tau = \sigma' \tan\varphi'$

If two specimens of Over consolidated soil are given,



Then the shear strength parameters are

$$\varphi' = 2\left\{\tan^{-1}\left[\frac{\sigma_1'{(1)} - \sigma_1'{(2)}}{\sigma_3'{(1)} - \sigma_3{(2)}}\right]^{0.5} - 45^\circ\right\}$$

And,

$$C = \frac{\sigma_1'{(1)} - \sigma_3'{(1)} \tan^2\left(45 + \frac{\varphi_1'}{2}\right)}{2\tan\left(45 + \varphi_1'/2\right)}$$

CI-15: A particular soil failed under a major principle stress of 300 kN/m^2 with a corresponding minor principle stress of 100 kN/m^2. If, for the same soil, the minor principle stress had been 200 kN/m^2, determin what the major stress would have been if (a) $\varphi = 30^\circ$ (b) $\varphi = 0^\circ$

overconsolidated যদি বলা না থাকে। → general case

(a) solution:

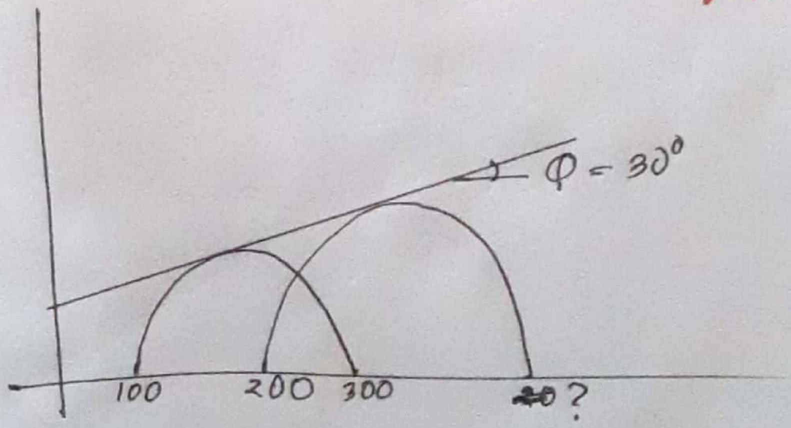
$\varphi = 30^\circ$

$\sigma_{1(1)} = 300$

$\sigma_{3(1)} = 100$

$\sigma_{3(2)} = 200$

$\sigma_{1(2)} = ?$



we know,

$$\varphi = 2\left\{\tan^{-1}\left(\frac{\sigma'_{1(1)} - \sigma'_{1(2)}}{\sigma'_{3(1)} - \sigma'_{3(2)}}\right)^{0.5} - 45^\circ\right\}$$

$$\Rightarrow 30^\circ = 2\left\{\tan^{-1}\left(\frac{300 - \sigma'_{1(2)}}{100 - 200}\right)^{0.5} - 45^\circ\right\}$$

$\therefore \sigma'_{1(2)} = 600 \text{ kN/m}^2$

(b) $\varphi = 0^\circ$ same as $\rightarrow \sigma'_{1(2)} = 2\sigma'_{1(1)}$ 2(0)

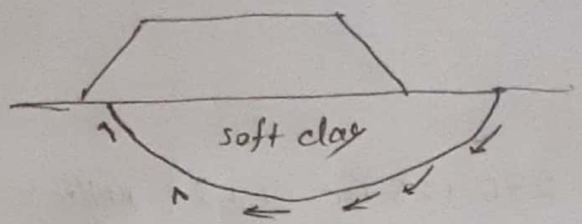
Ex: 12.6

As for sand and nonmally consolidated clay, $c=0$

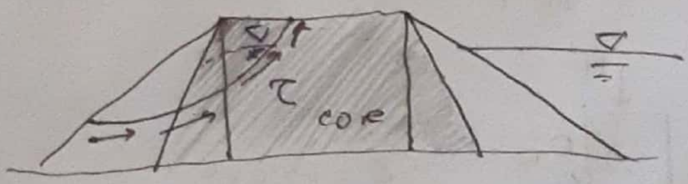
∴ Angle of fiction, $\phi = \sin^{-1}\left(\frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'}\right)$

* Some practical uses:

1. Embanument constructed veny slowly, in layers over a soft clay deposit.



2. Earth dam with steady state seepage



3. Excavation or Natural slope

#Example 12.3: A consolidated-drained triaxial test was conducted on a normally consolidated clay. The results are as follows:

$\sigma_3 = 276$ kN/m²

$(\Delta\sigma_d)_f = 276$ kN/m²

Determine

a) Angle of friction, ϕ'

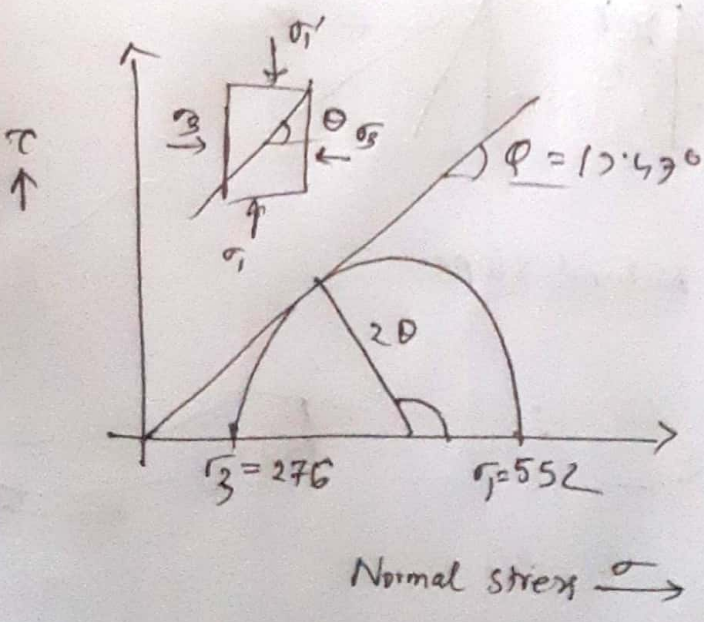
b) Angle θ that can b the failure plane makes with the major principle plane.

Answer:

a) $\sigma_1 = \sigma_3 + \Delta\sigma_d = 276 + 276 = 552$ kN/m²

$\therefore \phi' = \sin^{-1}\left(\frac{552-276}{552+276}\right) = 19.47°$

b) $\theta = 45 + \frac{\phi'}{2} = 45 + 19.47/2 = 54.73°$



From Ex 12.3

a) Find the normal stresses ~~σ~~ σ' and the shear stress τ_f on the failure plain

b) Determine the effective normal stress on the plane of maximum shear stress.

Solution:

a) We know angle of the failure plane is 2θ

$$\therefore \sigma_f' = \frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2} \cos 2\theta$$

$$= \frac{552+276}{2} + \frac{552-276}{2} \cos(2 \times 54.73)$$

$$= 368.03 \text{ kN/m}^2$$

$$\therefore \tau_f = \frac{\sigma_1' - \sigma_3'}{2} \sin 2\theta$$

$$= \frac{552-276}{2} \sin(2 \times 54.73) = 130.12 \text{ kN/m}^2$$

b) we know maximum shear plane is at 90° of the major principle plane,

$\therefore 2\theta = 90°$

$\therefore$ normal stress at plane of max shear,

$$\sigma = \frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2} \cos 2\theta$$

$$= 414 \text{ kN/m}^2$$

Example: 12.5 The equation of the ~~equa~~ effective stress failure envelope for normally consolidated clay soil is $\tau_f = \sigma' \tan 30°$. A drained triaxial test was conducted with the same soil at a chamber-confining pressure of 69 kN/m². Calculate the deviator stress, at failure.

Solution:

~~$\theta = 45° + \phi/2 = 45 + 30/2 =$~~

$\phi' = 30°$

$\sigma_3 = 69$

$\sigma_1 = ?$

We know, $\phi' = \sin\left(\frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'}\right)$

$\therefore \sigma_1 = 207$

$\therefore \sigma_1 = \sigma_3 + \Delta\sigma_d$

$\Delta\sigma_d = 207 - 69 = 138$ kN/m²

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or, $\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2)$

$= \sigma_3 \tan^2(45 + \phi/2)$ $\because c = 0$ for N/C soil

$= 69 \tan^2(45 + 30/2)$

$\sigma_1 = 207$

$\therefore \Delta\sigma_d = \sigma_1 - \sigma_3 = 207 - 69 = 138$ kN/m²

CU Test:

~~No~~ Drainage allowed during cell pressure, thus $u_c = 0$

After deviator stress applied drainage stopped, $\therefore u_d = \ldots$

$\therefore$ Also, $u_d = A\,\Delta\sigma_d$

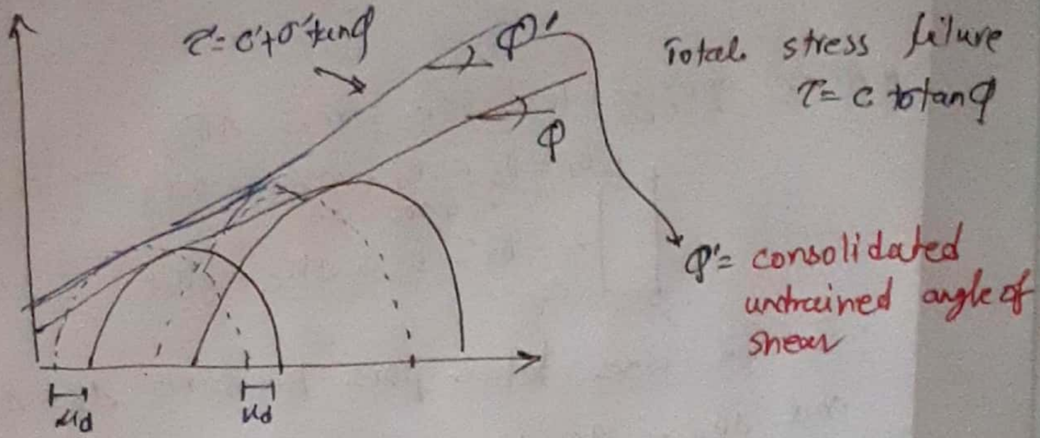
$\therefore$ Total pore pressure, $u = u_d + u_c$

$= A\sigma_d + 0$

$\therefore \therefore u = A\,\Delta\sigma_d = u_d$

; A = Skempton's parameter

$\therefore A = \frac{u_d}{\sigma_d}$



~~φ = sin⁻¹~~

major total pressure at failure: $\sigma_3 + \Delta\sigma_d = \sigma_1$

" effective " " : $\sigma_1 - \Delta u_d = \sigma_1'$

Minor total " " : σ_3

" effective " : $\sigma_3 - u_d = \sigma_3'$

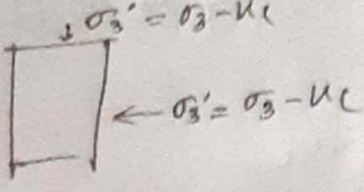
$$\therefore \phi = \sin^{-1}\left(\frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'}\right)$$

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

$$\therefore \phi' = \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3 - 2\Delta u_d}\right)$$

UU Test:

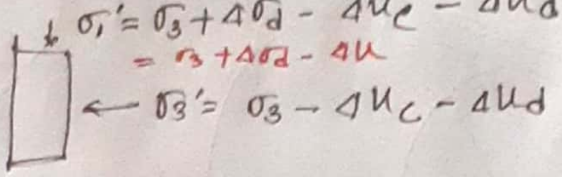
* Drainage not allowed in any cases even in cell pressure.
* During cell pressure,



In this case, $\Delta u_c = B \Delta \sigma_3$ → increase of cell pressure

(B → 1 if fully saturated soil)

* After applying deviator stress $\Delta \sigma_d$,



(Δu_d → due to deviator pressure)

∴ In this case total pore pressure, $\Delta u = \Delta u_c + \Delta u_d$

and $\Delta u_d' = AB\,\Delta\sigma_d$; $A = \frac{A_0}{1-\epsilon_2} = \frac{A_0}{1 - \Delta H/H_0}$

∴ total pwp $\Delta u = u_c + u_d$

$$= B\sigma_3 + AB\sigma_d$$

Problem

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

→ Skempton's pore water pressure equation

If it's fully saturated then we know, $B = 1$

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

$\Delta u = \sigma_3 + u_d$

∴ σ_1' becomes, $\sigma_1' = \sigma_3 + \Delta\sigma_d - \Delta u = \cancel{\sigma_3} + \Delta\sigma_d - \cancel{\sigma_3} - u_d$

$\therefore \sigma_1' = \Delta\sigma_d - u_d$

Ex: 12.7

A specimen of saturated sand was consolidated under an all-around pressure of 105 kN/m². The axial stress was then increased and drainage was prevented. The specimen failed when ~~increased~~ a the axial deviator stress reached 70 kN/m². The pore water pressure at failure was 50 kN/m². Determine

a) Consolidated-Undrained angle of shearing stress, ϕ

b) Drained friction angle, ϕ'

Ans:

a) Here, $\sigma_3 = 105$

$\Delta\sigma_d = 70$

$\Delta u_f = 50$

$\therefore \sigma_1 = \sigma_3 + \Delta\sigma_d = 105 + 70$

$= 175$

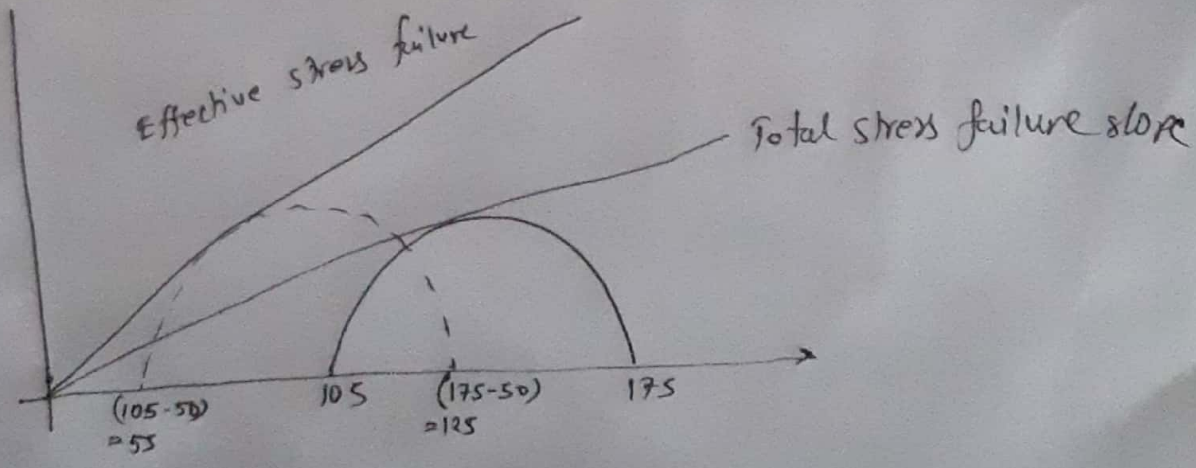
$\therefore$ Consolidated-undrained angle, $\phi = \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right)$

$= 14.47^\circ$

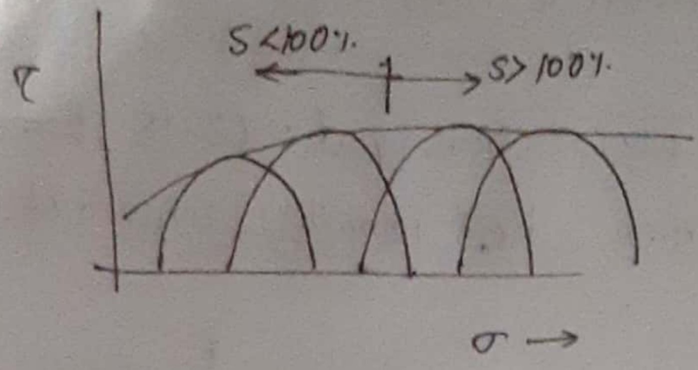
b) Drained friction angle, $\phi' = \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3 - 2\Delta u_f}\right)$

$= \sin^{-1}\left(\frac{175 - 105}{175 + 105 - 2 \times 50}\right)$

$= 22.88^\circ$ Ans.

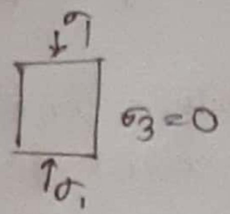


* Effect of degree of saturation on failure envelope.



[4] Unconfined Compression Test (UC).

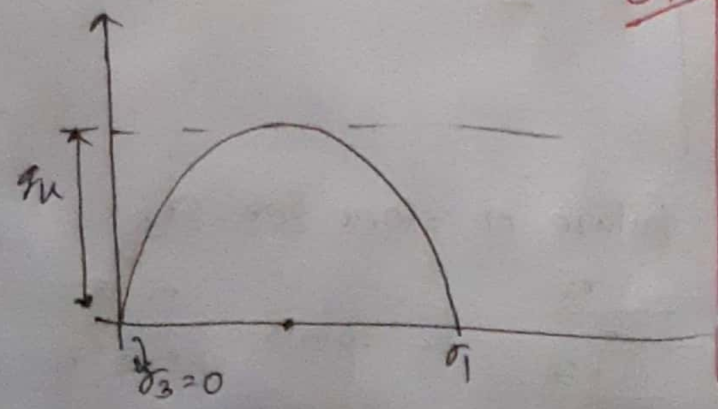
* speical type of UU Test
* Commonly used for clay speciments.
* In this case, confining pressure is $\sigma_3 = 0$



* Only vertical axial pressure is applied
* There's no minor principle stress

∴ Shear stress, $\tau_f = \frac{\sigma_1 - \sigma_3}{2} = \frac{\sigma_1}{2} = \frac{q_u}{2} = c_u$

∴ $\boxed{\tau_f = \frac{q_u}{2}}$ q_u = unconfined compression stress.



CT-15 UU Parameters

$\phi = 0$

$\sigma_3 = 0$

$\tau_{max} = \frac{\sigma_1}{2} = \frac{q_u}{2}$

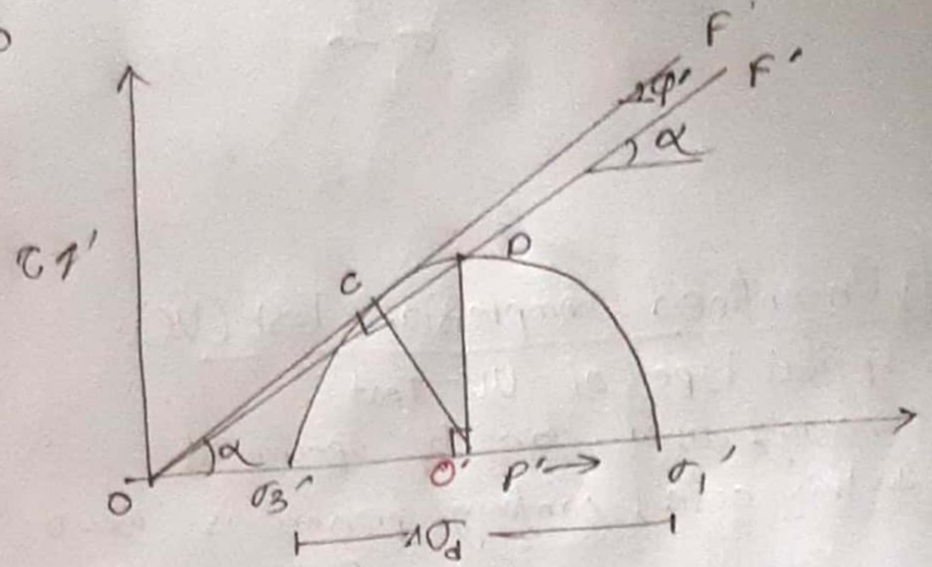
Stress Paths:

Let's consider a specimen is subjected to consequencial loads in a CD test.

Calculating c and ϕ using stress path: CT-15,

Considering the specimen is normally consolidated,

$\therefore$ c=0



Here OF' = modified failure envelope = k_f line

OF = Failure envelope line

$\therefore$ If q' & p' are stress points, then the k_f line can be expressed as $p' = q' \tan\alpha$

Now, at D point, $p = \frac{\sigma_1' + \sigma_3'}{2}$, $q = \frac{\sigma_1' - \sigma_3'}{2}$

$$\therefore \tan\alpha = \frac{DO'}{OO'} = \frac{\frac{\sigma_1' - \sigma_3'}{2}}{\frac{\sigma_1' + \sigma_3'}{2}}$$

$$\therefore \tan\alpha = \frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'}$$

Again, from the failure envelope line Of,

$$\sin\phi = \frac{O'C}{OO'} = \frac{\frac{\sigma_1' - \sigma_3'}{2}}{\frac{\sigma_1' + \sigma_3'}{2}} \quad \therefore \sin\alpha = \frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'}$$

$\therefore \sin\phi = \tan\alpha$

$\therefore \phi' = \sin^{-1}(\tan\alpha)$

$\therefore c = 0$

$\phi' = \sin^{-1}(\tan\alpha)$ Answer

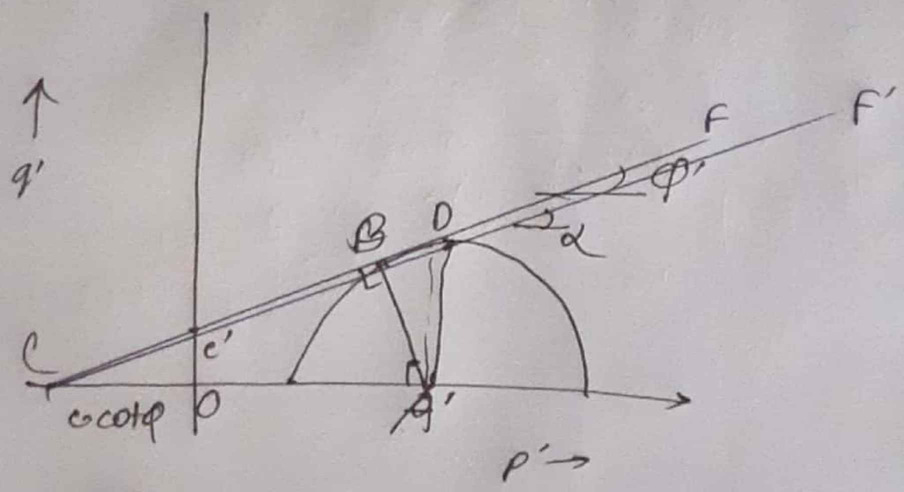
Example: ES 12'10 – (CT–15)

How will calculate c and ϕ for overconsolidated soil?

Solution:

Let, the k_f line for over consolidated line be

$q' = m + p'\tan\alpha$



Now, $\sin\phi' = \dfrac{AB}{AC} =$

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

$\therefore \dfrac{\sigma_1' - \sigma_3'}{2} = c\cos\phi' + \left(\dfrac{\sigma_1' + \sigma_3'}{2}\right)\sin\phi$ —(i)

$\therefore q' = m + p'\tan\alpha$ —(ii)

$\therefore$ From (i) & (ii)

$c\cos\phi = m \Rightarrow c = \dfrac{m}{\cos\phi}$ & $\sin\phi = \tan\alpha$, $\phi = \sin^{-1}(\tan\alpha)$