GATE – CIVIL ENGINEERING

GEOTECHNICAL ENGINEERING

Prof.B.Jayarami Reddy
Professor and Head
Department of Civil Engineering
Y.S.R. Engineering College of
Yogi Vemana University,
Proddatur, Y.S.R.(Dt.), A.P-516360.
E.mail : bjrcivilgate@gmail.com

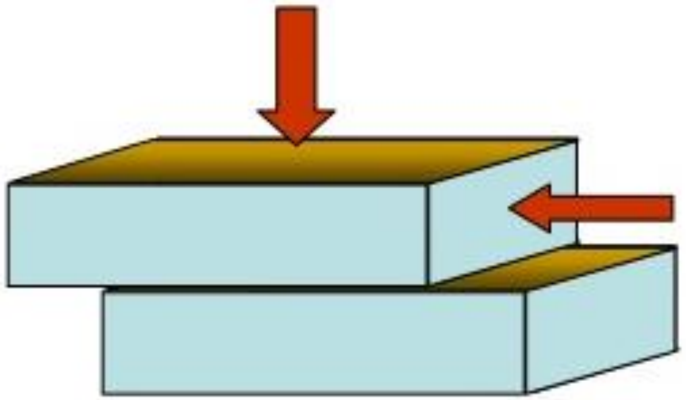
8.10 Shear Strength of Soils

Prof. B. Jayarami Reddy

8.10 Shear Strength of Soils

Shear strength:

- Shear strength is defined as the resistance to shearing stresses and a consequent tendency for shear deformation.
- Shear strength is the principal engineering property which controls the stability of a soil mass under loads. It governs
  - Bearing capacity of soils
  - Satiability of slopes in soils
  - Earth pressure against retaining structures.
- A soil derives its shear strength from
  - i. Resistance due to the interlocking of particles
  - ii. Frictional resistance between the individual soil grains, which may be sliding friction, rolling friction or both.
  - iii. Adhesion or Cohesion between soil particles.



- Granular soils or sands may derive their shear strength from sources i. and ii.
- Cohesive soils or clays may derive their shear strength from sources ii. and iii.
- Highly plastic clays exhibit their shear strength from source iii. alone

Cohesion:

- Cohesion is the property of the soil holding its soil properties together.
- Cohesion is an important property of fine grained soil. eg. Clays
- Cohesive soil is the soil which possess actual cohesion. eg. clays
- Cohesionless soil is the soil which does not posses any cohesion. eg. dry sand.

Structural Resistance:

- The structural arrangement of soil particles also affects the shear stress.
- The same soil may exhibit remarkably different shear strength at different void ratio and at different rate of loading.

Friction:

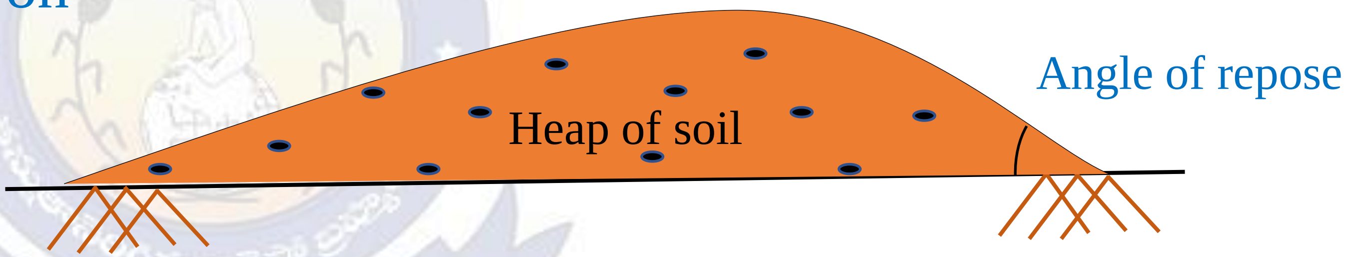
- The internal friction between soil particles resists the shearing friction of soil mass.
- The friction may be either sliding friction or a rolling friction or combination.

Angle of Internal friction: (ϕ)

Angle of internal friction or friction angle is the measure of the resistance of soil to sliding along a plane.

Angle of internal friction ϕ depends on

- Shape of the particles
- Surface roughness
- Type of inter locking
- Lateral pressure



Angle of Repose:

- Angle of repose is the angle at which a heap of soil forms naturally.
- Angle of repose is different from angle of internal friction.
- For a dry loose sand, both angle of repose and angle of internal friction are approximately equal.

Prof. B. Jayarami Reddy

Mohrs theory

- Mohrs failure envelope is a curve since the angle of internal friction is known to decrease slightly with increase in stress.

$$s = \sigma_f . \tan\phi$$

Coulomb's Law:

According to Coulomb's law, the shearing strength of soil consists

i. Cohesion between particles.

ii. Friction between particles.

Coulomb's suggested the straight line equation which is also called as Mohr coulomb equation. $s = c + \sigma . \tan\phi$

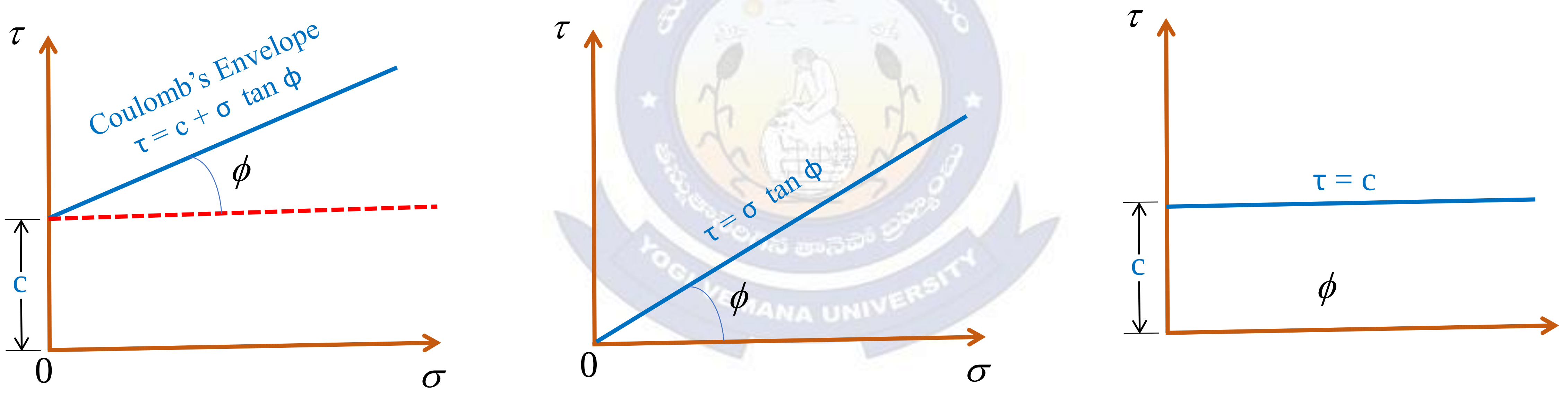
s = Shear stress at the failure of soil.

σ = Normal stress acting on the plane

c = Cohesion

ϕ = Angle of internal friction or Angle of shearing resistance.

- c , ϕ are called shear strength parameters
- c and ϕ are not constants for a particular soil, they depend on the drainage conditions and test conducted.



Coulomb's envelope for a c – φ soil

Pure sand "c = 0" or "ϕ"

Pure clay "φ = 0" or "c"

The Effective stress principle (Terzaghi's concept)

- In Mohr Coulomb equation the $c-\phi$ parameters are not fundamental properties of the soil, they depend upon a number of factors such as water content, drainage conditions, conditions of testing.
- Terzaghi established that the normal stresses which control the shear strength of soil are the effective stresses and not the total stresses.

$$s = c' + \sigma' \tan\phi$$

c' : effective cohesion

ϕ' : effective angle of shearing resistance

σ' : effective stress $\qquad \sigma' = \sigma - u$

u : pore water pressure

Different types of tests based on drainage conditions

- A cohesionless or a coarse grained soil may be tested for shearing strength either in the dry condition or in the saturated condition.
- A cohesive or fine grained soil is usually tested in the saturated condition
- Shear parameters c and ϕ vary with the type of test or drainage conditions

Unconsolidated Undrained test (or) Quick test (UU test or Q test):

- No drainage is permitted either during consolidated stage and shear stage.
- Use the Perspex discs both at the top and bottom of the specimen so that no pore water can escape. Start the axial loading immediately after constant lateral pressure is applied.
- There is no dissipation of pore pressure during the test
- Test can be conducted in few minutes (5-10 minutes)
- Undrained tests are performed only on soils of low permeability clays
- Used for short term stability analysis for foundations, excavations, earth dams.

Consolidated Undrained test (CU test or R-test):

- Drainage is permitted until the consolidation is completed (During the application of normal stress)
- Use the porous discs and saturate the specimen.
- No drainage is permitted during shear stage (during the application of shear stress)

- It is also called as R-test as the alphabet R falls between the alphabet Q used for quick test and the alphabet S used for slow test.
- Volume change do not takes place during shear and excess pore pressure develops.
- Used for sudden drawdown cases.

Consolidated Drained Test (or) Slow test (CD-test or S-test):

- Drainage is permitted throughout the test during the application of both normal stress and shear stress so that full consolidation occurs
- Use the porous discs and saturate the specimen.
- Excess pore pressure is set up at any stage of test.
- Generally takes 2 to 5 days.
- Volume change takes place
- Cohesionless soils and for soils having high permeability.
- Used for ascertaining long term stability.

For undrained test, c_u , ϕ_u parameters are used

For drained test, effective shear strength parameters c', ϕ' are used.

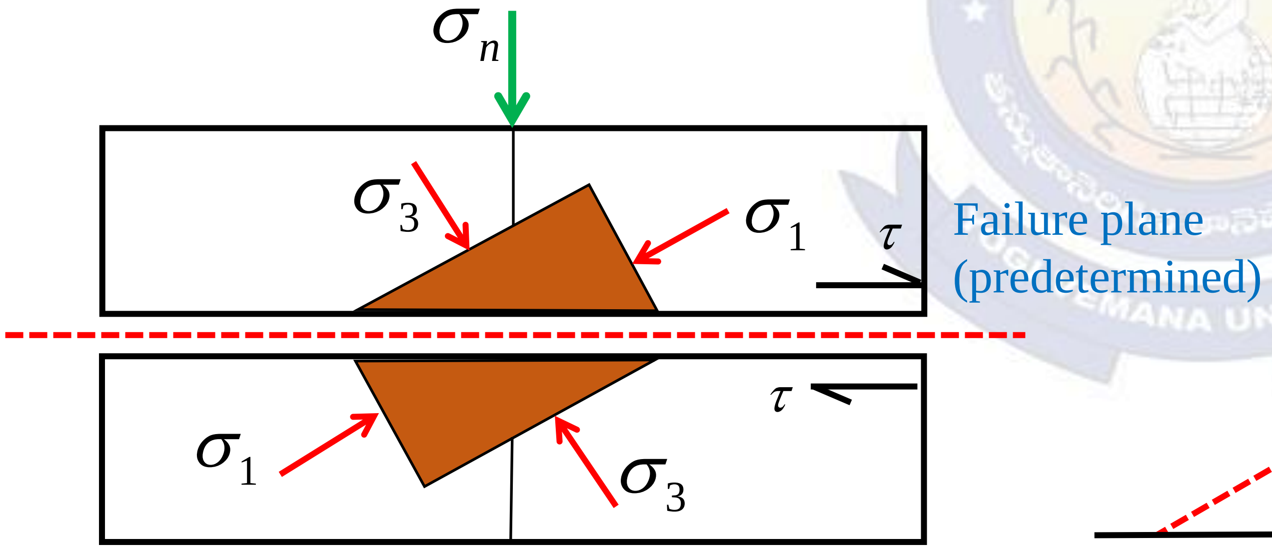
Prof. B. Jayarami Reddy

Shearing strength tests:

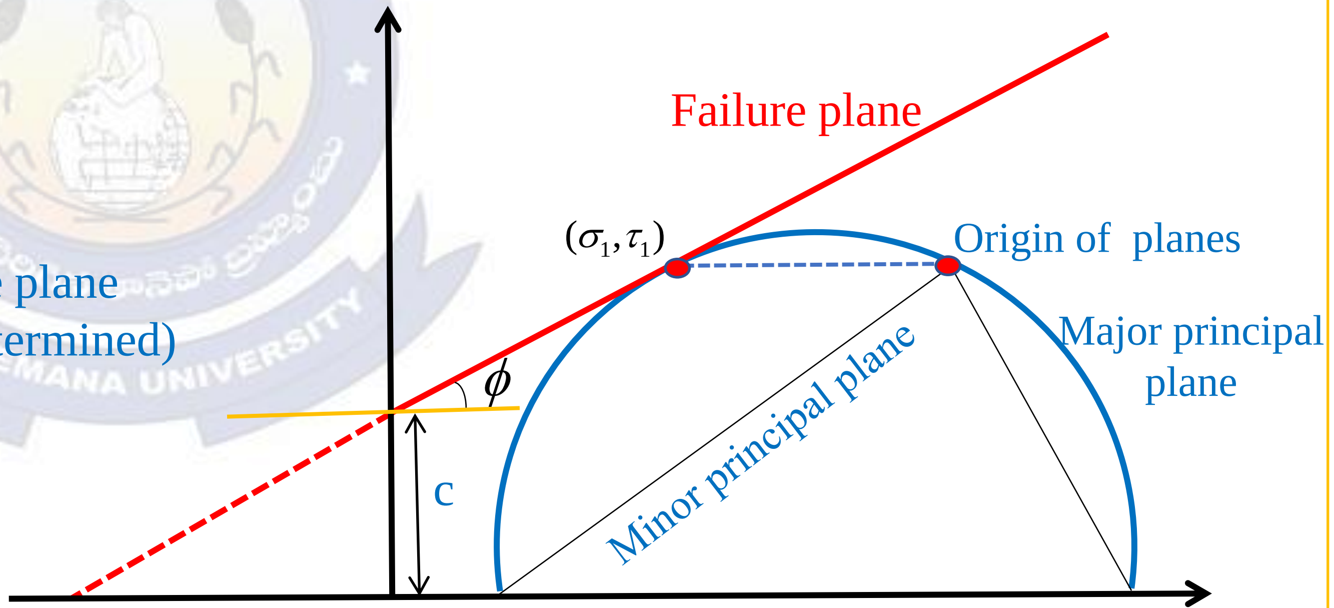
1. Laboratory tests
    a. Direct shear test
    b. Triaxial compression test
    c. Unconfined compression test
    d. Laboratory vane shear test
    e. Torsion test
    f. Ring shear test
2. Field tests
    a. Vane shear test
    b. Penetration test

Direct Shear test

- Soil specimen 60×60×25 mm is used.
- Generally conducted on cohesionless soils as CD test. It can be used for any of the three conditions [UU, UD, CD]
- Used for testing the granular soil
- In direct shear test, the failure plane is horizontal and principal plane is inclined at 45^0 to the horizontal.



(a) Conditions of stress in the shear box



(b) Mohr's circle for direct shear test

Prof. B. Jayarami Reddy

Merits:

- It is relatively simple test
- It is identically suited for conducting drained tests on cohesionless soils.
- As the thickness of the sample is relatively small, the drainage is quick and pore pressure dissipates very rapidly. The consolidated drained and the consolidated undrained tests take relatively small period.

Demerits:

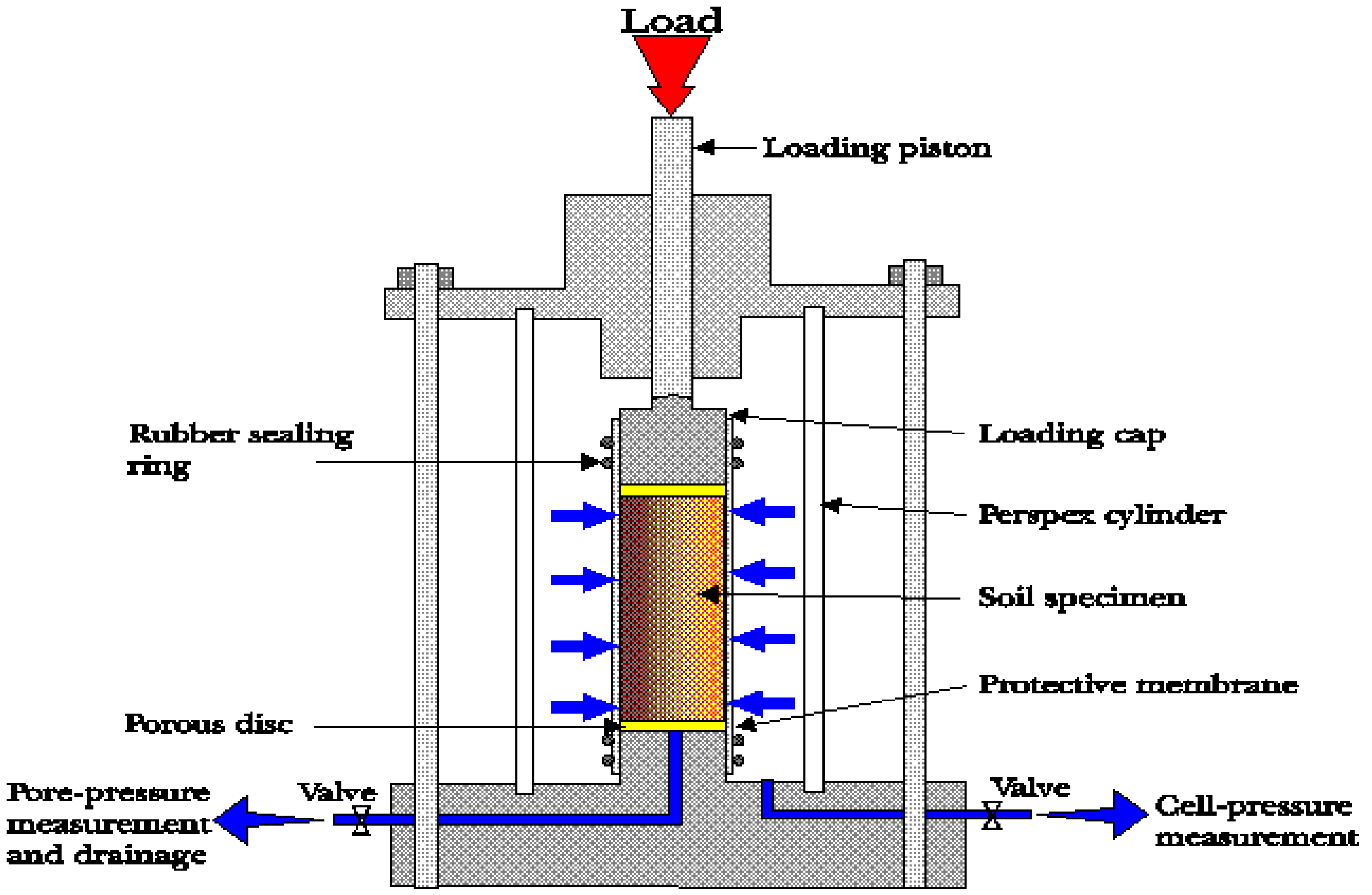
- Little control on drainage of soil. Consequently only drained tests can be conducted on highly permeable soil
- Measurement of pore pressure is not possible.
- The orientation of failure plane is fixed (horizontal) this plane may not be the weakest plane.
- The stress conditions are known at failure. The conditions prior to failure are intermediate and therefore the Mohr circle cannot be drawn.

Prof. B. Jayarami Reddy

- The stress distribution across the soil sample is very complex (not uniform).
- The area under shear gradually decreases as the test progresses. But the corrected area cannot be determined and therefore, original area is taken for the computation of stresses.

Triaxial compression test:

- Triaxial compression test is used for the determination of shear characteristics of all types of soils under different drainage conditions.
- The specimen is subjected to three compressive stresses in mutually perpendicular directions
- Height of specimen is twice its diameter
- 3D stress system is achieved by initial application of alround fluid pressure or confining pressure through water. It is constant throughout the test and an axial loading is increased gradually at a uniform rate until the specimen fails in shear
- The minor and intermediate principal stress is equal to the cell pressure.



Triaxial apparatus

Prof. B. Jayarami Reddy

- As the cell pressure is applied, pore water pressure develops in the specimen which can be measured using pore pressure measurement apparatus connected to the pore pressure line after closing the value of the drainage.
- If the pore pressure is dissipated, the pore water line is closed, the drainage line opened and connected to a burette. The volume decrease of the specimen due to consolidation is indicated by the water drained into the burette.
- The axial strain associated with the application of additional axial stress can be measured by means of a dial gauge recording the downward movement of the loading position.
- The additional axial stress applied is measured with the aid of proving ring and dial gauge.
- For an undrained test $\Delta V = 0$.

Procedure:

Specimen preparation:

- Air dried soil is to be considered.
- Mix up with required amount of water to give a required density for a given volume.
- Compact the soil in a mould to a desired density.
- Press hollow cylindrical cutters into the compacted soil and obtain the specimen of required size.

Mounting of specimen:

- Keep either the perpex disc or the pours stone on the pedestal according to the test requirements and mount the specimen.
- Cover the specimen with the stretcher and place it on the pedestal.
- Place the top loading pad with either perspex disc or porous disc on top of the specimen.

- Remove the sheath stretcher away.
- Assemble the cell taking care to maintain some minimum clearance between the plunger and loading pad.
- Open the air-vent on the cell top.
- Open the valves of cell pressure and pore pressure measuring assembly.
- Apply cell pressure by foot pump - pore water pressure develops in the specimen.
- Close the valve of the drainage line.
- Measure the pore water pressure.
- Close the pore water line
- Open the drainage line and connect to a burette
- The volume decrease of the specimen due to consolidation is indicated by the water drained into the burette.
- Measure the axial strain corresponding to additional axial stress.
- Close the drainage line
- Measure the pore water pressure.

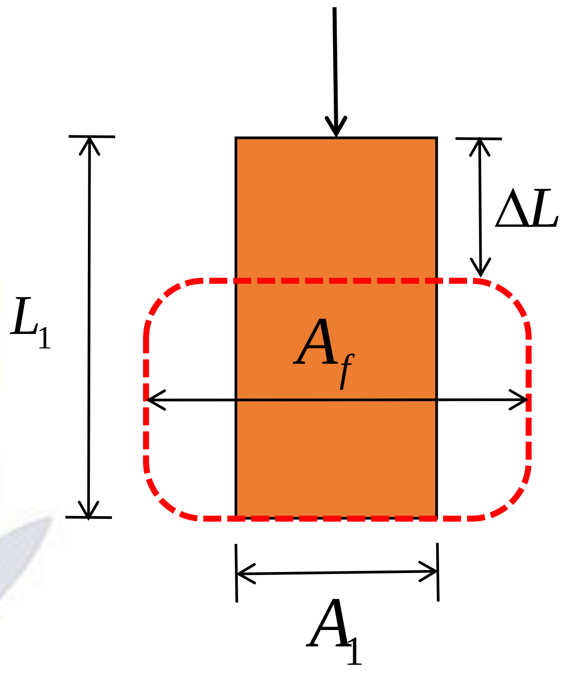
Stress Analysis:

$\sigma_3 = \sigma_c =$ cell pressure = confining pressure

$\sigma_d = (\sigma_1 - \sigma_3)$ =deviator stress = $\dfrac{\text{additional load}}{\text{area at failure}} = \dfrac{P}{A_f}$

$\sigma_1 =$ Major principal stress

Area at failure, $A_f = \dfrac{V_1 + \Delta V}{L_1 - \Delta L}$



$V_1 =$ Initial volume of specimen

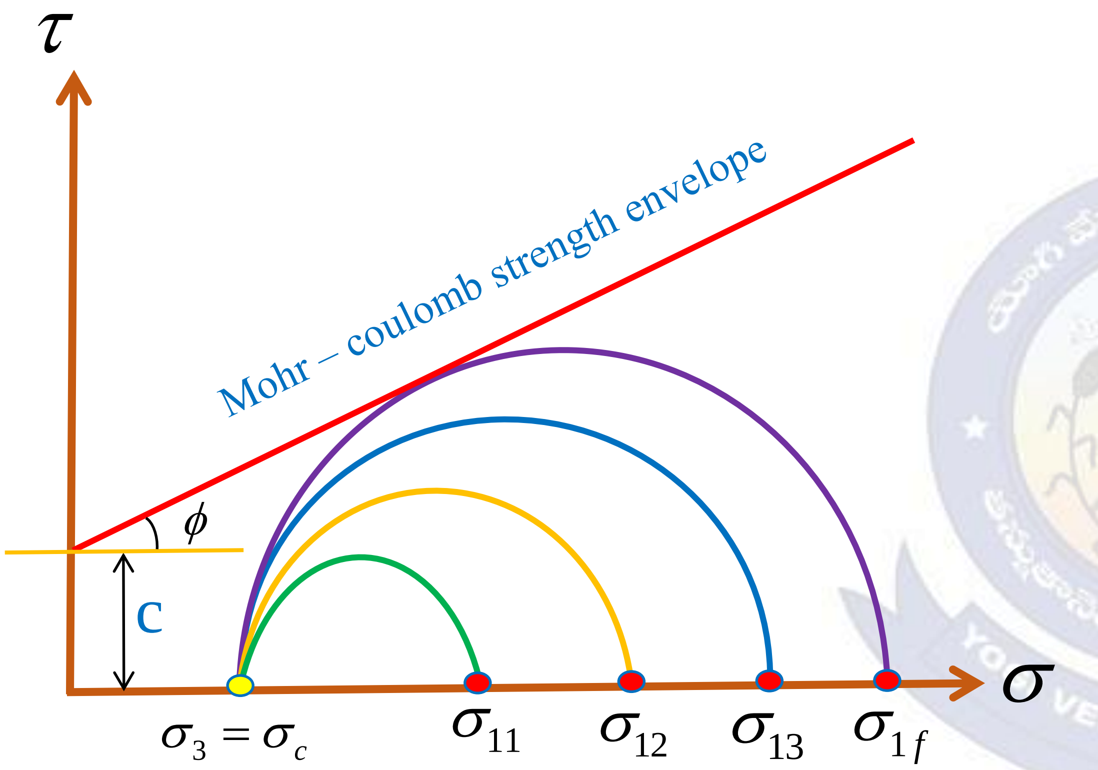
$L_1 =$ Initial length of specimen

$\Delta V =$ Change in volume of specimen

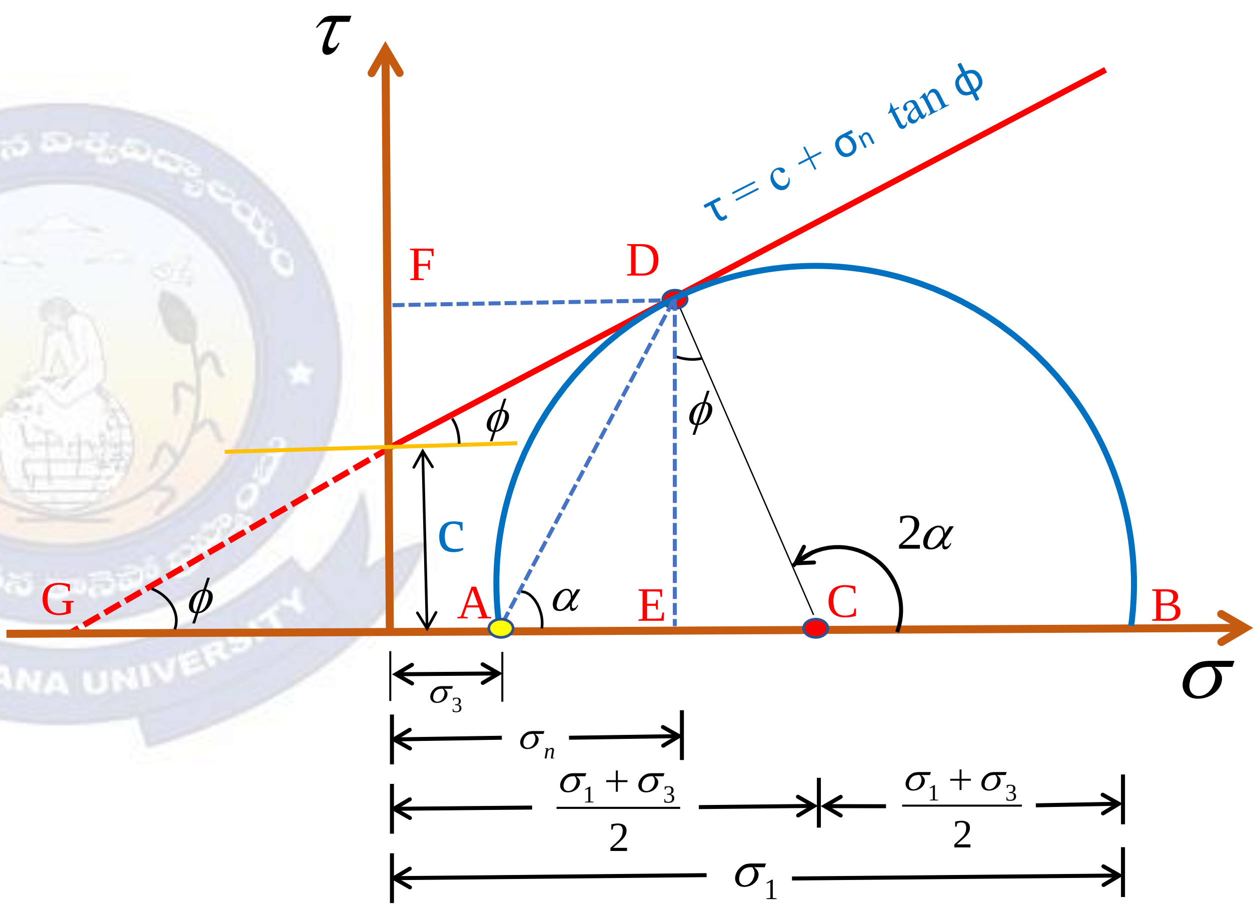
$\Delta L =$ Change in length of the specimen

$\Delta V = 0$ for undrained test

Prof. B. Jayarami Reddy



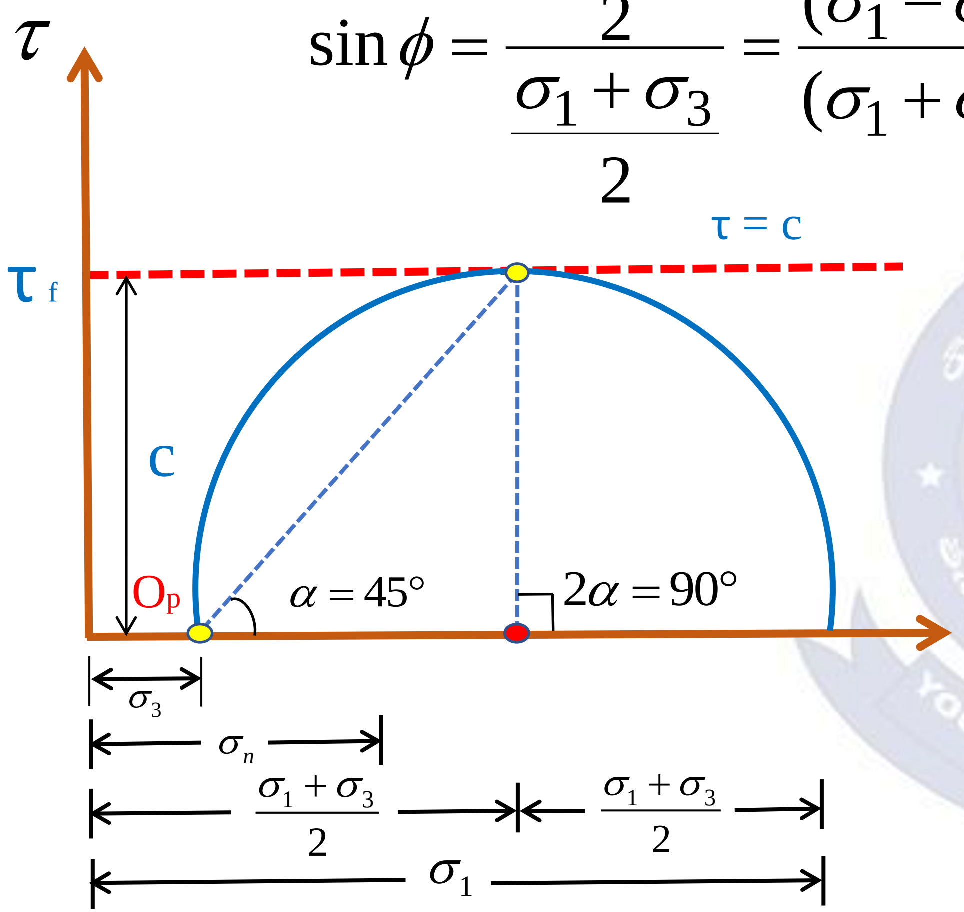
Mohr's circles during triaxial test



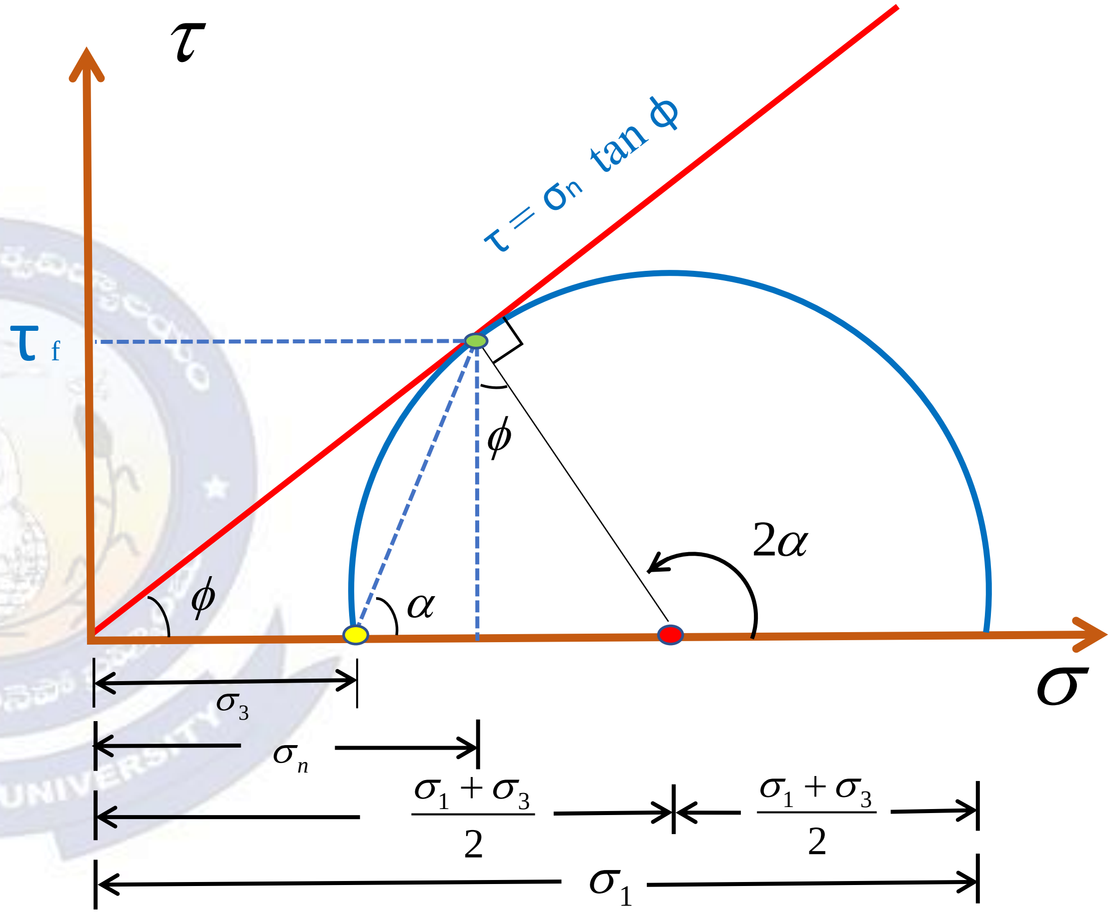
Mohr's circle at failure for general c-φ soil

Prof. B. Jayarami Reddy

$$\sin\phi = \frac{\dfrac{\sigma_1 - \sigma_3}{2}}{\dfrac{\sigma_1 + \sigma_3}{2}} = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 + \sigma_3)}$$



Mohr’s circle for a pure cohesive soil or c – soil at failure



Mohr’s circle at failure for a pure frictional or φ soil

Prof. B. Jayarami Reddy

Merits :

- The stress distribution on the failure plane is uniform
- The specimen is free to fail on the weakest plane.
- There is complete control over drainage conditions
- Pore pressure changes and the volumetric changes can be measured directly.
- The state of stress at all intermediate stages up to failure is known. The Mohr circle can be drawn at any stage of shear.

Demerits:

- The drained test takes a longer period in comparison with a direct shear test.
- The consolidation of the specimen in the test is isotropic, where as in the field the consolidation is generally anisotropic.

Prof. B. Jayarami Reddy

Plastic Equilibrium:

A material is said to be in plastic equilibrium when every point of it is at the verge of failure.

At plastic equilibrium, $\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$

$$\sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi}$$

$$\sigma_h = \sigma_1 \cos^2\alpha + \sigma_3 \sin^2\alpha$$

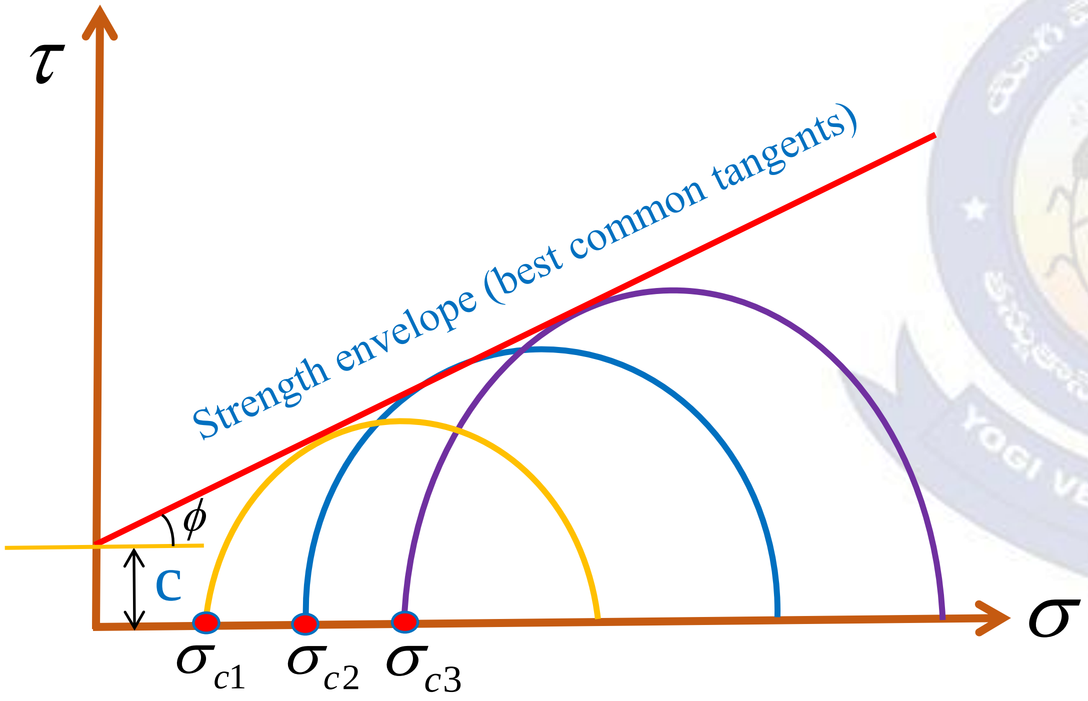
Where $N_\phi = \tan^2\phi$

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

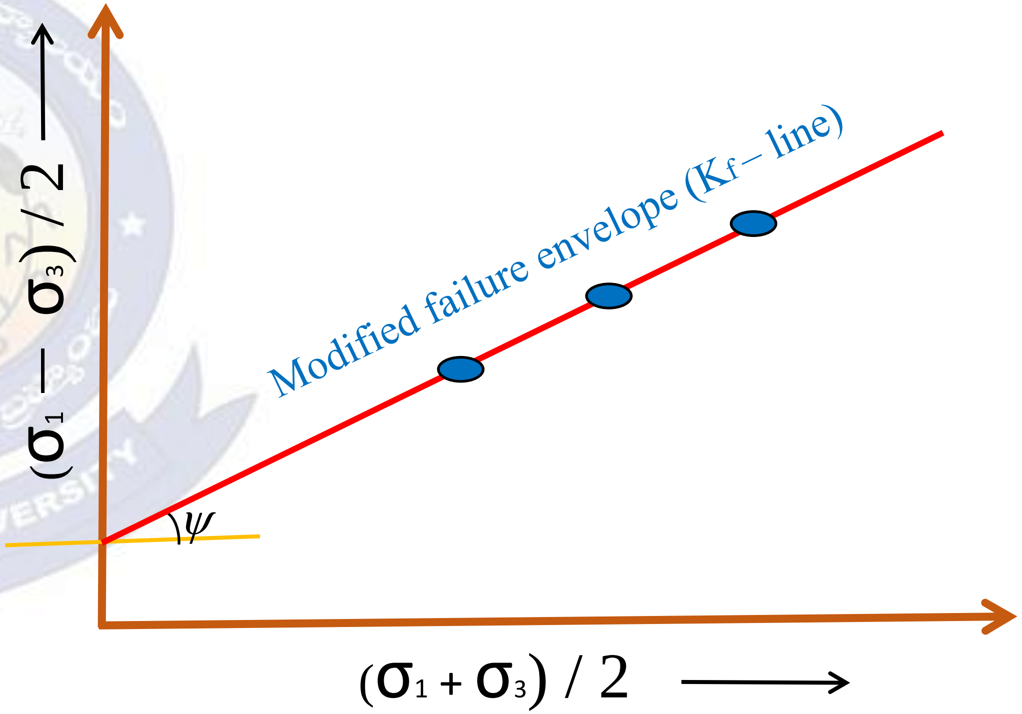
The critical shear plane occurs at an angle of $\alpha = 45^0 + \dfrac{\phi}{2}$ with reference to major principal plane.

Strength envelop:

To evaluate the shear parameters c and ϕ at least two sets of σ_1 and σ_3 are required. Plot the Mohr circles for each set and draw the best tangent to the circles as the strength envelope



Mohr's circles for triaxial tests with different cell pressures and strength envelope



Alternative procedure of evaluating shear strength parameters

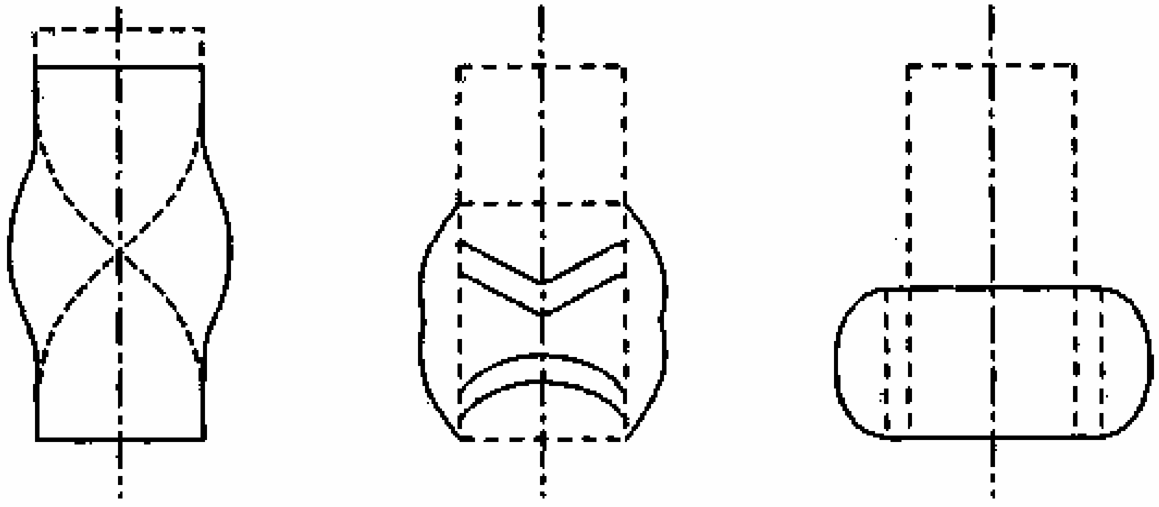
Prof. B. Jayarami Reddy

Types of the failure of a triaxial compression test specimen

Brittle failure: Well defined shear plane

Semi plastic failure: Shear cones and some lateral bulging

Plastic failure: Well expressed lateral bulging



Failure patterns in triaxial compression tests

Prof. B. Jayarami Reddy

Unconfined compression test:

- It is a special form of triaxial test in which the confining pressure is zero.
- The test can be conducted only on undisturbed or remoulded cohesive soils which can withstand without confinement. It is a quick or undrained test.
- Minor principal stress, $\sigma_3 = 0$
- Major principal stress, $\sigma_1 = \dfrac{P}{A_f}, \quad A_f = \dfrac{A_1}{1-\varepsilon} = \dfrac{A_1}{1-\dfrac{\Delta L}{L}}$

P = Axial load

A_1 = Initial area

A_f = Area at failure

$$\sigma_1 = \sigma_3 \tan^2 \alpha + 2c \tan \alpha_f \qquad (\sigma_3 = \sigma_2 = 0)$$

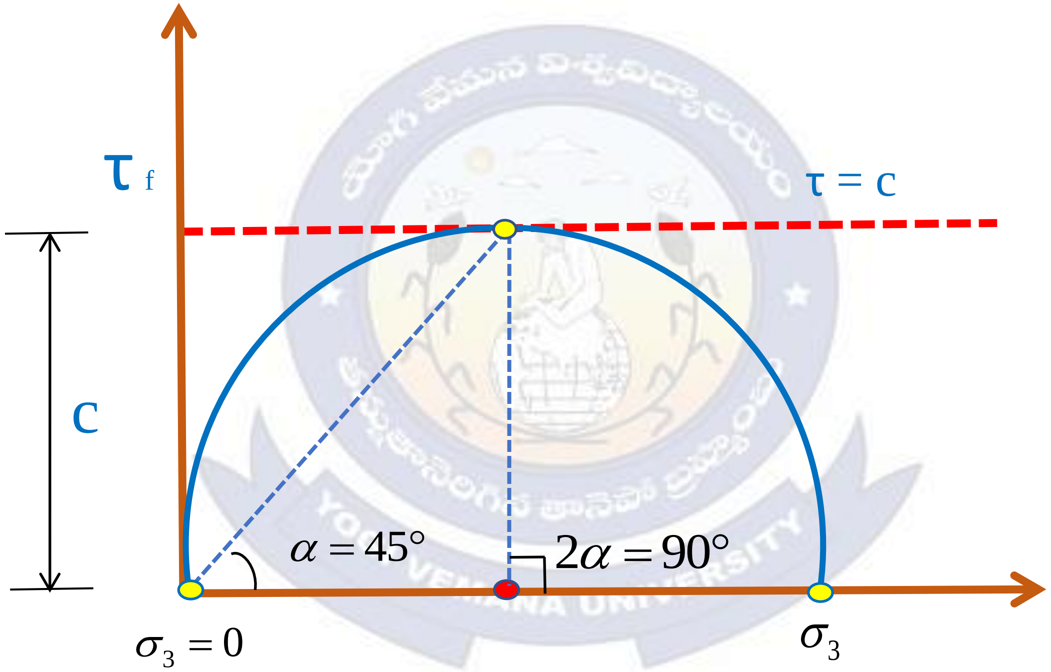
$$\sigma_1 = 2c_u . \tan \alpha = 2c_u \tan\left(45 + \frac{\phi}{2}\right) = 2c_u \tan(45+0)$$

$$\sigma_1 = 2c_u \quad ; c_u = \frac{\sigma_1}{2} \quad ; c_u = \frac{q_u}{2}$$

q_u = Unconfined compressive strength.

Prof. B. Jayarami Reddy

- It is ideally suited for measuring the unconsolidated undrained shear strength of saturated clay.
- Test cannot be conducted on fissured clays.



Mohr's circle for unconfined compression test

Prof. B. Jayarami Reddy

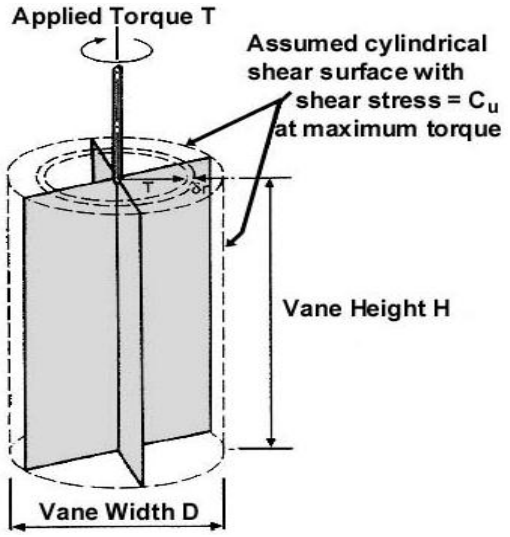
The Vane shear test:

Used to determine undrained shear strength of soft clays.

The test can be conducted in the field/ laboratory.

Torque, $T = \pi d^2 c\left[\frac{H}{2}+\frac{d}{6}\right]$ when both ends part take in shearing.

$= \pi d^2 c\left[\frac{H}{2}+\frac{d}{12}\right]$ when only bottom end part take in shearing.



c = undrained cohesion

D = overall diameter of vane

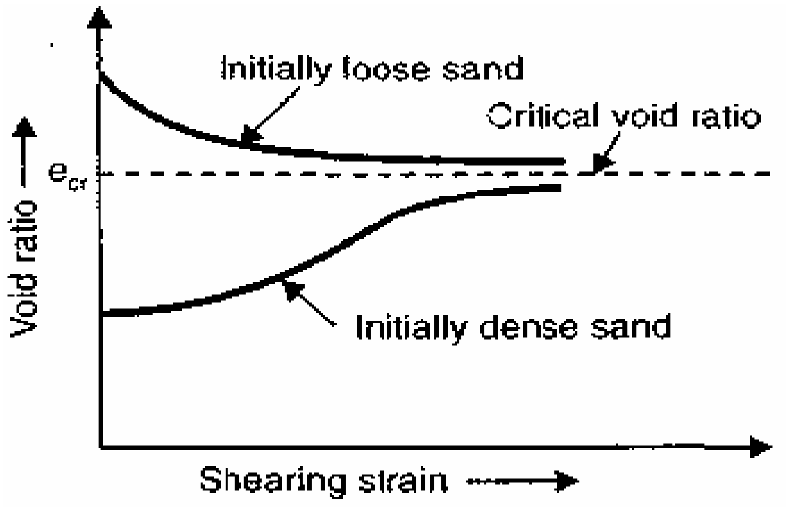
H = Height of vane, as per IS recommendation, $H=2d$.

The sensitivity of soil can be conveniently determined by this test.

This test is not suitable when clay contains sand or silt laminations or fissured clay.

Prof. B. Jayarami Reddy

Critical void ratio:



Effect of initial density on changes in void ratio

- At large strains both initially loose and initially dense sands attain nearly the same void ratio at which further strain will not produce any volume changes. Such a void ratio is usually referred to as the Critical Void Ratio.
- Sands with initial void ratio greater than critical value will tend to decrease in volume during shearing, while sands with initial void ratio less than the critical will tend to increase in volume.

Prof. B. Jayarami Reddy

Shear characteristics of cohesionless soils:

- In case of loose sand the specimen bulges and ultimately falls by sliding simultaneously on numerous planes. The failure is known as Plastic Failure.
- In the case of dense sand, the specimen shows a clear failure plane and the failure is known as Brittle failure.

The factors affecting shear strength of cohesionless soils are.

1. Shape of Particles: The shearing strength of sands with angular particles having sharp edges is greater than that with rounded particles.

2. Gradation: A well graded sand exhibits greater shear strength than a uniform sand.

3. Denseness: The greater the denseness, the greater is the strength.

4. Confining Pressure: The shear strength increases with an increase in Confining pressure.

5. Loading: The angle of shearing resistance of sand is independent of the rate of loading.

6. Type of Minerals: If the sand contains Mica, it will have a large void ratio and lower value of .

7. Capillary Moisture: The sand may have apparent cohesion due to capillary moisture.

Prof. B. Jayarami Reddy

Factors affecting Shear strength of Cohesive Soils:

Clay content: As the clay content increase, the angle of internal friction decreases.

Drainage conditions: The cohesive soils have very low strength just after the application of load when undrained conditions exist.

Confining Pressure: The shear strength of clays increase with an increase in the confining pressure.

Plasticity Index: The value of decreases with an increase in plasticity index of the clay.

Stress History: The values of shear strength parameters depend upon stress history. Over consolidated clays have greater shear strength than normally consolidated clays.

Disturbance: The shear strength of disturbed sample is less than that of the undisturbed samples.

The friction angle varies with particle size.

Prof. B. Jayarami Reddy

Angle of internal friction, $\phi' = 36^0 + \phi_1 + \phi_2 + \phi_3 + \phi_4$

$\phi' = 36^o$ represents the value for average conditions.

ϕ_1: Influence of grain shape on

$= 1^o$ for angular grains

$= 0^o$ for sub-angular grains

$= -3^o$ for rounded grains

$= -6^o$ for well rounded grains

ϕ_2: Influence of grain size on ϕ'

$= 0$ for sand

$= 1^o$ for fine gravel

$= 2^o$ for Medium and coarse gravel

$$\boxed{\phi(\text{Poorly graded soil}) < \phi(\text{Well graded soil})}$$

ϕ_3: Correction factor for gradation

$= -3^o$ for poorly graded soil

$= 0$ for medium uniformity

$= 3^o$ for well graded soil

ϕ_4: Correction factor for relative density

$= -6^o$ for loosest packing

$= 0$ for medium density

$= 6^o$ for densest packing

Prof. B. Jayarami Reddy

Pore pressure Parameters

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

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

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

$$\Delta u = \Delta u_1 + \Delta u_2$$

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

$$\Delta u_1 = B.\Delta\sigma_3 \qquad \Delta u_2 = B.A\left(\Delta\sigma_1 - \Delta\sigma_3\right)$$

Δu_1 = Change in pore pressure due to an increase in cell pressure

Δu_2 = Change in pore pressure due to an increase in deviator stress

$$B = \frac{1}{1 + n\frac{c_w}{c_v}}$$

n = porosiy of soils

c_w = compressibility of pore fluid

c_v = compressibility of soils

When the soil is dry, $c_w << c_v, B = 1$

When the soil is fully saturated $c_w >> c_v \Rightarrow \frac{c_w}{c_v} \Rightarrow \infty \Rightarrow$ $B = 0$

$B \rightarrow 0 \ to \ 1$

$$A = \frac{\Delta u}{\sigma_1 - \sigma_3}$$

- A can be any value i.e. A<0, A>0, A>1
- B→0 to1

Prof. B. Jayarami Reddy

- A is not constant and depends on the type of soil and stress condition
- A and B are not constants
- B → UU test
- A → CU test

Prof. B. Jayarami Reddy

Shear Strength of Soil

(Numerical Questions)

Prof. B. Jayarami Reddy

1. The following results were obtained from a direct shear test on cohesive soil sample.

Normal stress, kN/m^2	160	240
Shear stress at failure, kN/m^2	110	130

i. The angle of internal friction and cohesion of the soil sample respectively are

ii. If a triaxial test is carried out on the same soil sample with a cell pressure of 150 kN/m^2, the deviator stress at failure is ....

Ans: ϕ =14.03^0, C =70 kN/m^2 and σ_d =275.3 kN/m^2

$$c = ? \qquad \phi = ?$$

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

$$110 = c + 160 \tan\phi$$

$$130 = c + 240 \tan\phi$$

Solving the above equations $c = 70\ kN/m^2 \quad : \quad \phi = 14.03^\circ$

Prof. B. Jayarami Reddy

$$\sigma_3 = 150\text{kN/m}^2$$

$$\sigma_d = \text{Deviated stress}$$

$$= \sigma_1 - \sigma_3$$

$$\alpha = 45^0 + \frac{\phi}{2} = 45^0 + \frac{14.03}{2} = 52.01^0$$

$$\sigma_1 = \sigma_3 \tan^2 \alpha + 2c \tan \alpha$$

$$= 150 \tan^2 52.01 + 2 \times 70 \tan 52.01^0$$

$$\sigma_1 = 425.3\text{kN/m}^2$$

$$\sigma_3 = 425.3 - 150 = 275.3\text{kN/m}^2$$

2. The results of the triaxial compression test conducted under undrained conditions on the soil sample are as follows:

Diameter of the sample = 40 mm

Height of the sample = 80 mm

Cell pressure =100 kN/m^2

Increase in volume at failure = 1.2 ml.

Axial compression = 5 mm.

The deviator stress and the total stress of the sample respectively are

Ans: 442 kN/m^2 and 542 kN/m^2

$$\sigma_d = \frac{\mathrm{P}}{\mathrm{A_f}}$$

$$d = 40\text{mm} \qquad L = 80\text{mm} \qquad \sigma_3 = 100\text{kN/m}^2 \qquad P = 600N$$

$$\Delta V = 1.2ml = 1.2\text{cm}^3 = 1.2\times10^3 mm^3$$

$$\Delta L = 5mm$$

Prof. B. Jayarami Reddy

$$V=\frac{\pi\times(40)^2}{4}\times 8=100.53\times 10^3\,mm^3$$

$$A_f=\frac{V+\Delta V}{L-\Delta L}=\frac{100.53\times 10^3+1.2\times 10^3}{80-5}\Rightarrow A_f=1356.4mm^3$$

$$\sigma_d=\frac{P}{A_f}=\frac{600}{1356.4}=0.422\text{N/mm}^2\Rightarrow\sigma_d=442\text{kN/m}^2$$

$$\text{Total Stress},\sigma_1=\sigma_d+\sigma_3$$

$$\sigma_1=442+100=542\text{kN/m}^2$$

Prof. B. Jayarami Reddy

3. In an unconfined compression test, cylindrical specimen of a saturated soil fails at an axial stress of 160 kN/m^2. The failure plane makes an angle of 50^o with the horizontal. The shear parameters are …..

Ans: $\phi = 10^0$ and $c_u = 67.12$ kN/m^2

$$\sigma_1 = 2c \tan\alpha$$

$$q_u = 2c.1$$

$$q_u = 2c_u.\tan\alpha$$

$$160 = 2c_u.\tan 54^0$$

$$c_u = 58.12$$

$$\alpha = 45 + \phi/2$$

$$c_u = \frac{q_u}{2}$$

Pure shear

$\phi = 0$

$\alpha = 45^0$

Prof. B. Jayarami Reddy

$$54 = 45 + \phi/2$$

$$\phi = 18$$

For c-ϕ soils

$$\alpha > 45^0$$

$$\sigma_1 = 2c\tan\alpha \Rightarrow q_u = 2c_u.\tan\alpha \Rightarrow c_u = ?$$

Prof. B. Jayarami Reddy

4. A vane shear test was conducted on a soft clay deposit. The failure occurred at a torque of 64 Nm. Afterwards the vane is rotating rapidly and the test was repeated in remolded sample. The torque at failure in the remolded soil was 22 Nm. The size of the vane and diameter across the blades are 120 mm and 80 mm respectively. The sensitivity of the soil is ……….

Ans: S = 2.91

$$d = 38mm \qquad L = 76mm \qquad P = 30N \qquad \delta L = 11mm \qquad \alpha = 50^0$$

$$A_0 = \frac{\pi}{4}(38)^2 \Rightarrow A_0 = 1134.1\text{mm}^2$$

$$A_f = \frac{A_0}{1-\varepsilon} = \frac{A_0}{1-\frac{\Delta L}{L}} = \frac{1134.1}{1-\frac{11}{76}} = 1 \Rightarrow A_f = 1326.4\text{mm}^2$$

$$q = \frac{P}{A_f} = \frac{50}{1326} = 0.0226\text{N/mm}^2 q = 226\text{kN/m}^2$$

Prof. B. Jayarami Reddy

$$\alpha = 45^0 + \phi/2 \Rightarrow 50 = 45 + \frac{\phi_u}{2} \Rightarrow \phi_u = 10^0$$

$$q_u = 2c_u \tan\alpha$$

$$22.6 = 2c_u \tan 50^0$$

$$c_u = 948 \text{kN/m}^2$$

Prof. B. Jayarami Reddy

5. The properties of soil in a 3 m high embankment are c' =50 kN/m^2, ϕ' = 20^0 and γ = 16 kN/m^3. Skempton pore pressure parameters are found from the triaxial test, A = 0.5 and B = 0.9. The height of embankment was raised from 3 m to 6 m. Assuming that the dissipation of pore pressure during the stage of construction is negligible and that lateral pressure is half of the vertical pressure, then the shear strength of soil at base of embankment just after increasing the height of embankment is ……..

Ans: 73.14 kN/m^2

$T_{undisturbed} = 64\text{N-m}$ $\quad$ $T_{disturbed} = 22\text{N-m}$

$H = 120\text{mm}$ $\quad$ $D = 80\text{mm}$ $\quad$ $S = ?$

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

$$T \alpha C_u$$

Prof. B. Jayarami Reddy

$$S = \frac{C_{u(undisturbed)}}{C_{u(remoulded)}} = \frac{T_{undisturbed}}{T_{disturbed}} = \frac{64}{22}$$

$$S = 2.909$$

Prof. B. Jayarami Reddy

6. An unconfined compression test was conducted on an undisturbed soil sample of clay. The sample has diameter 38 mm and length 76 mm. The load at failure was 30 N. The axial deformation of the sample is 11 mm. If the failure plane made an angle of 50^0 with the horizontal, the undrained shear strength parameters are:

Ans: c_u= 9.48 kN/m^2,ϕ_u =10^0

$$c' = 50\text{kN/m}^2 \qquad \phi' = 20^0 \qquad \gamma = 16\text{kN/m}^2$$

Skemptons A=0.5, B=0.9

Δu: Increase in pore pressure

$$\Delta u: \mathrm{B}\left[\Delta\sigma_3 + \mathrm{A}\left(\Delta\sigma_1 - \Delta\sigma_3\right)\right]$$

$$\Delta\sigma_1 = \text{Increase of vertical stress}$$

$$= \gamma h = 16 \times 3 = 48\text{kN/m}^3$$

$$\Delta\sigma_3 = \frac{1}{2}.\Delta\sigma_1 = \frac{1}{2}\times 48 = 24\text{kN/m}^2$$

$$\Delta u = 0.9\left[24 + 0.5\left(48 - 24\right)\right] = 22.4\text{kN/m}^2$$

$$\tau_f = c' + \sigma' \tan\phi'$$

σ' : Effective vertical stress at the base of the emabnkment

$$\sigma' = \left(\sigma_1 + \Delta\sigma_1\right) - \Delta u = \left(16\times 3 + 48\right) - 32.4 = 63.6\text{kN/m}^2$$

τ_f = Shear strength of soil at base

$$\tau_f = c' + \sigma' \tan\phi' = 50 + 63.6\tan 20^0 = 73.14\text{kN/m}^2$$

Prof. B. Jayarami Reddy

7. An unconfined compressive strength of soil was found to be 80 kN/m^2. A soil sample of same soil failed at a deviator stress of 180 kN/m^2 and at a cell pressure of 90 kN/m^2. The shear strength parameters are ………

$$q_u = 80\text{kN/m}^2 \qquad \sigma_d = 180\text{kN/m}^2 \qquad \sigma_3 = 90\text{kN/m}^2$$

$$C_u = \frac{q_u}{2} = \frac{80}{2} = 40\text{kN/m}^2$$

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2C\tan\alpha$$

$$\sigma_1 = \sigma_3 + \sigma_d = 90 + 180 = 270\text{kN/m}^2$$

$$270 = 90\tan^2\alpha + 2\times 40\times\tan\alpha$$

$$\tan\alpha = 1.34 \Rightarrow \alpha = 53.26$$

$$\alpha = 45 + \frac{\phi}{2} \Rightarrow 53.26 = 45 + \frac{\phi}{2} \Rightarrow \phi = 16.65$$

Prof. B. Jayarami Reddy

8. In a triaxial test, a soil specimen was consolidated under a cell pressure of 600 kN/m^2 and the increased pore pressure reading was 420 kN/m^2. Axial load was then increased to give a deviator stress of 550 kN/m^2 and pore pressure reading of 640 kN/m^2. The pore pressure parameters A and B respectively are

Ans: A =1.66 and B = 0.7

$$\Delta\sigma_3 = 600\text{kN/m}^2 \qquad \Delta u_1 = 420\text{kN/m}^2$$

$$\Delta\sigma_d = 550\text{kN/m}^2 \qquad \Delta u_2 = 640\text{kN/m}^2$$

$$\Delta\text{u: B}\left[\Delta\sigma_3 + \text{A}\left(\Delta\sigma_1 - \Delta\sigma_3\right)\right]$$

$$= \Delta u_1 + \Delta u_2$$

$$= \text{B}.\Delta\sigma_3 + \text{B.A}\left(\Delta\sigma_1 - \Delta\sigma_3\right)$$

Prof. B. Jayarami Reddy

Δu_1 : change in pore pressure due to change of cell pressure

Δu_2 : change in pore pressure due to change in cell pressure

$$\Delta u = B.\Delta\sigma_3$$

$$420 = B \times 600 \Rightarrow B = \frac{420}{600} \Rightarrow B = 0.7$$

$$\Delta u_2 = B.A\left(\Delta\sigma_1 - \Delta\sigma_3\right)$$

$$640 = 0.7.A.(550) \Rightarrow A = 1.66$$

Prof. B. Jayarami Reddy

Shear Strength of soils

Previous GATE Questions

Prof. B. Jayarami Reddy

1. In a drained triaxial compression test, a sample of sand fails at deviator stress of 150 kPa under confining pressure of 50 kPa. The angle of internal friction (in degree, round off to the nearest integer) of the sample, is …. GATE CE 2020

Ans. 37

For sample of sand, c=0

Deviator stress, $\sigma_d = 150\,kPa$

Confining pressure, $\sigma_3 = 50\,kPa$

Angle of internal friction, $\phi = ?$

Major Principal Stress, $\sigma_1 = \sigma_3 + \sigma_d$ = 50 + 150 = 200 KPa

$$\sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi}$$

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

$$200 = 50\tan^2\left(45^o + \frac{\phi}{2}\right) \Rightarrow \phi = 36.87^o$$

Prof. B. Jayarami Reddy

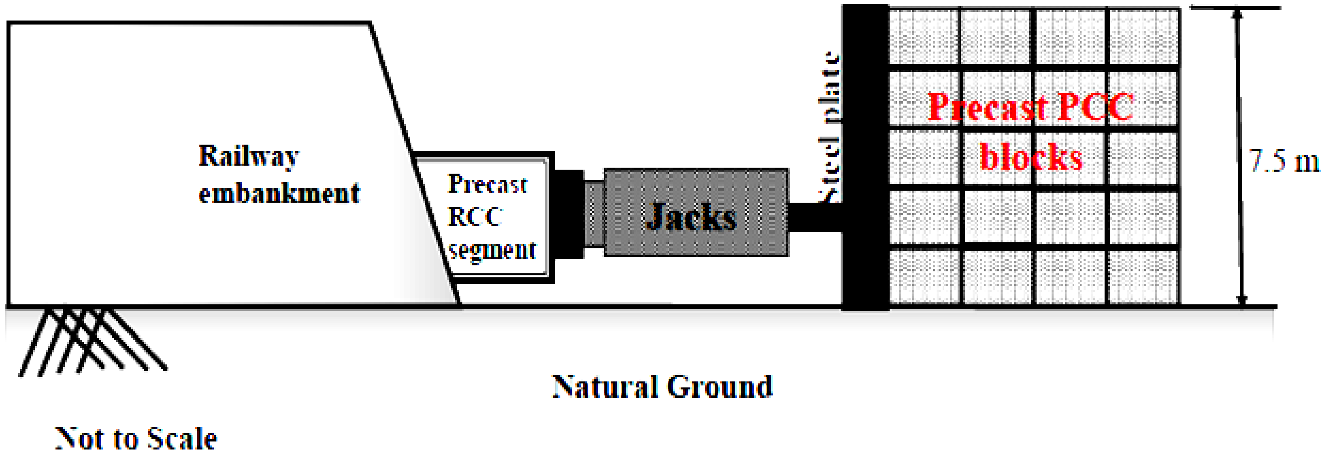
(OR)

$$\sin\phi = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} = \frac{200 - 50}{200 + 50} = \frac{150}{250} = 0.6$$

$$\phi = 36.87^{o}$$

Prof. B. Jayarami Reddy

02. A 3 m×3 m square precast reinforced concrete segments to be installed by pushing them through an existing railway embankment for making an underpass as shown in the figure. A reaction arrangement using precast PCC blocks placed on the ground is to be made for the jacks. CE1 2019



At each stage, the jacks are required to apply a force of 1875 kN to push the segment. The jacks will react against the rigid steel plate placed against the reaction arrangement. The foot print area of reaction arrangement on natural ground is 37.5 m^2. The unit weight of PCC block is 24 kN/m^3. The properties of the natural ground are: c =17 kPa; $\phi = 25^0$ and γ =18 kN/m^3. Assuming that the reaction arrangement has rough interface and has the same properties that of soil, the factor of safety (round off to 1 decimal place) against shear failure is...

Prof. B. Jayarami Reddy

Ans. 2.02

Factor of safety, $F = \frac{S}{\tau} = \frac{c + \sigma \tan\phi}{\tau}$

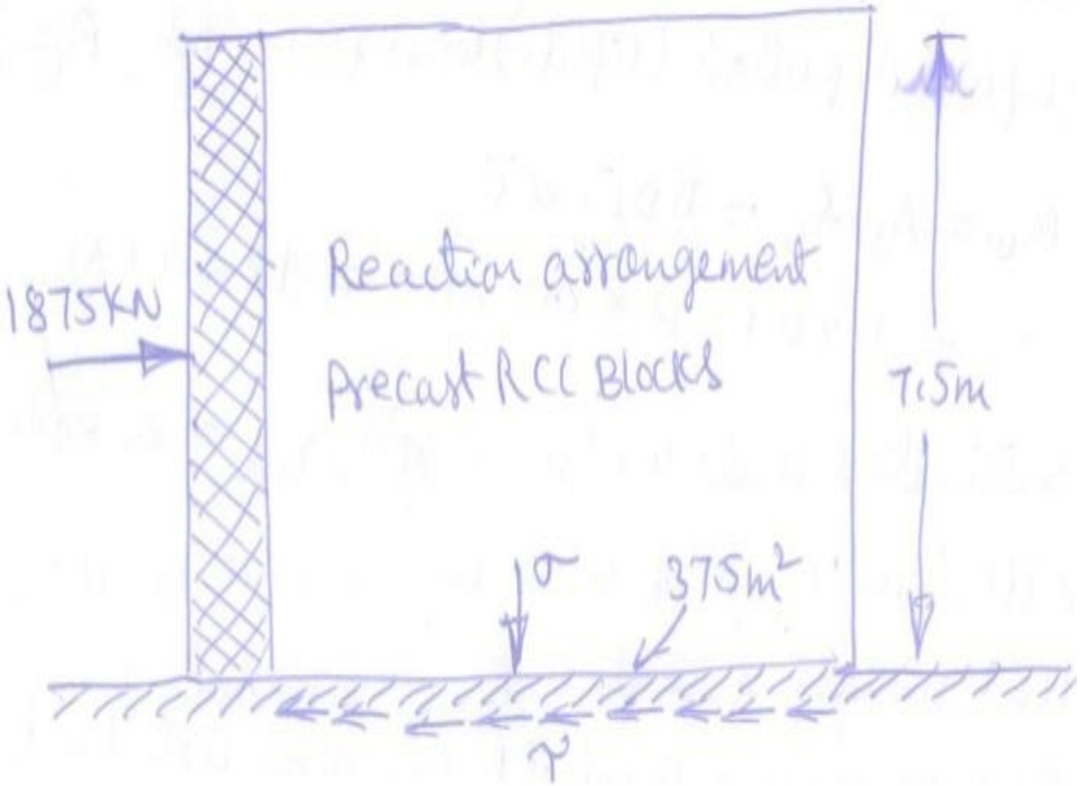
Weight of precast PCC blocks, $W = V.\gamma$

$$\sigma = \frac{W}{A} = \frac{V.\gamma}{A} = \frac{A.H.\gamma}{A} = 7.5 \times 24 = 180\,\mathrm{kN/m^2}$$

$$S = c + \sigma \tan\phi = 17 + 180 \tan 25^0 = 100.94\,\mathrm{kN/m^2}$$

$$\tau = \frac{jack\ force}{area} = \frac{1875}{37.5} = 50\,\mathrm{kN/m^2}$$

$$F = \frac{100.94}{50} = 2.02$$



Prof. B. Jayarami Reddy

03. The total horizontal and vertical stresses at a point in a saturated sandy medium are 170 kPa and 300 kPa, respectively. The static pore-water pressure is 30 kPa. At failure, the excess pore-water pressure is measured to be 94.50 kPa, and the shear stresses on the vertical and horizontal planes passing through the point and zero. Effective cohesion is $C' = 0$ kPa and effective angle of internal friction is $\phi' = 36^0$. The shear strength (in kPa, up to two decimal places) at point is…. CE2 2018

Ans. 52.52

At a point in a saturated sandy soil,

Horizontal stress, σ_3 =170 kPa

Vertical stress, σ_1 = 300 kPa

Static pore water pressure, u = 30 kPa

At failure, excess pore water pressure, u_1 = 94.50 kPa

$\bar{\sigma}_1 = 300 - 30 - 94.50$ =175.50 kPa

$\bar{\sigma}_3 = 170 - 30 - 94.50$ =45.5 kPa

Effective cohesion, $C' = 0$

Prof. B. Jayarami Reddy

Effective angle of internal friction, $\phi' = 36^0$

Shear strength at point $X,\ \tau = ?$

For failure plane,

$$\alpha = 45 + \frac{\phi}{2} \Rightarrow \alpha = 45^0 + \frac{36}{2} = 63^0$$

$$\bar{\sigma}_n = \frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2} + \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}.\cos 2\alpha = \frac{175.5 + 45.5}{2} + \frac{175.5 - 45.5}{2}\cos 2 \times 63$$

$$= 110.5 - 38.21 = 72.29 \text{ kPa}$$

Shear strength, $\tau = C + \sigma \tan\phi = 0 + 72.29 \tan 36^0$ $= 72.29$ kPa

Prof. B. Jayarami Reddy

04. A conventional drained triaxial compression test was conducted on a normally consolidated clay sample under an effective confining pressure of 200 kPa. The deviator stress at failure was found to be 400 kPa. An identical specimen of the same clay sample is isotropically consolidated to a confining pressure of 200 kPa and subjected to standard undrained triaxial compression test. If the deviator stress at failure is 150 kPa, the pore pressure developed (in kPa, up to one decimal place) is …. CE1 2018

Ans. 125

For drained triaxial compression test:

Confining pressure, $\bar{\sigma}_3$ =200 kPa

Deviator stress at failure, σ_d = 400 kPa

$\sigma_d = \bar{\sigma}_1 - \bar{\sigma}_3 \Rightarrow 400 = \bar{\sigma}_1 - 200 \Rightarrow \bar{\sigma}_1$ = 600 kPa

For clay sample,

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2C\, tan\, \alpha$$

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

For undrained triaxial compression test:

Confining pressure, σ_3 =200 kPa

Deviator stress at failure, σ_d =150 kPa

$\sigma_d = \sigma_1 - \sigma_3 \Rightarrow 150 = \sigma_1 - 200 \Rightarrow \sigma_1$ =350 kPa

Let u be the pore pressure developed

$$\bar{\sigma}_1 = 350 - u$$

$$\bar{\sigma}_3 = 200 - u$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$350 - u = (200 - u)\tan^2\left(45 + \frac{30}{2}\right)$$

$$350 - u = (200 - u)3 \Rightarrow 2u = 250 \Rightarrow u = 150 \text{ kPa}$$

Prof. B. Jayarami Reddy

05. Following are the statements related to the stress path in a triaxial testing of soils:

P: If $\sigma_1 = \sigma_3$, the stress point lies at the origin of the p-q plot.

Q:If $\sigma_1 = \sigma_3$, the tress point lies on the p-axis of the p-q plot.

R: If $\sigma_1 > \sigma_3$, both the stress points p and q are positive.

For the above statements, the correct combination is CE2 2017

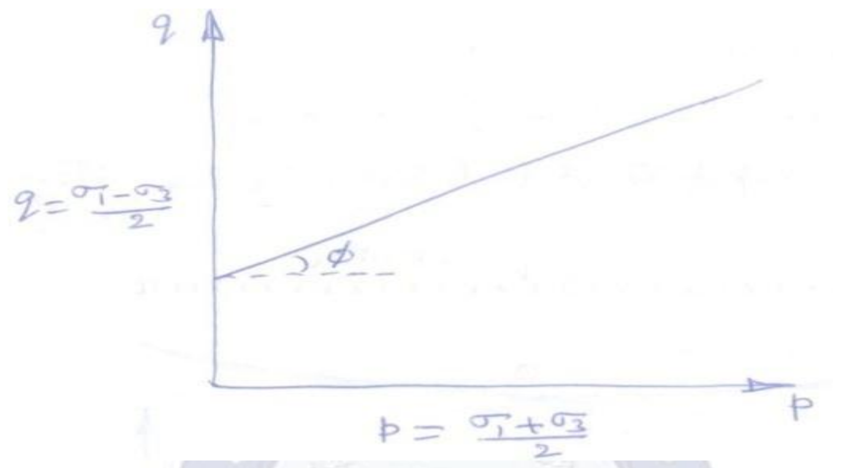
a. P-False; Q-True; R-True

b. P-True; Q-False; R-True

c. P-False; Q-True; R-False

d. P-True; Q-False; R-False

Ans. a



If $q=\frac{\sigma_1-\sigma_1}{2}$ =0 $p=\frac{\sigma_1+\sigma_1}{2}=\sigma_1$

Hence, the stress point lies on the p axis of p-q plot. The statement p is false and Q is true.

If $\sigma_1>\sigma_3$ $q=\frac{\sigma_1-\sigma_3}{2}>0$ $p=\frac{\sigma_1+\sigma_3}{2}>0$

Hence, both the stress points are positive. The statement R is true.

Prof. B. Jayarami Reddy

06. A consolidated undrained triaxial compression test is conducted on a normally consolidated clay at a confining pressure of 100 kPa. The deviator stress at failure is 80 kPa, and the pore-water pressure measured at failure is 50 kPa. The effective angle of internal friction (in degrees, up to one decimal place) of the soil is CE1 2017

Ans. 26.4

Confining pressure, σ_3 =100 kPa

Deviator stress at failure, σ_d =80 kPa

Pore water pressure measured at failure, U= 50 kPa

Effective angle of internal friction, $\phi' = ?$

$\sigma_1 = \sigma_3 + \sigma_d$ = 100 + 80 = 180 kPa

$\bar{\sigma}_1$ = 180-50 = 130 kPa

$\bar{\sigma}_3$ = 100-50 = 50 kPa

For normally consolidated clay, c=0

$$\bar{\sigma}_1 = \bar{\sigma}_3 . N_\phi + 2C\sqrt{N_\phi}$$

Prof. B. Jayarami Reddy

$$\bar{\sigma}_1 = \bar{\sigma}_3 . \tan^2\left(45^0 + \frac{\phi}{2}\right)$$

$$130 = 50.\tan^2\left(45^0 + \frac{\phi}{2}\right)$$

$$45^0 + \frac{\phi}{2} = 58.2^0$$

$$\phi = 26.4^0$$

Prof. B. Jayarami Reddy

07. A drained triaxial compression test on a saturated clay yielded the effective shear strength parameters as c' = 15 kPa and $\phi' = 22^0$ .Consolidated Undrained triaxial test on an identical sample of this clay at a cell pressure of 200 kPa developed a pore water pressure of 150 kPa at failure. The deviator stress (expressed in kPa) at failure is… CE1 2016

Ans. 104.38

Effective shear strength parameters:

$c' = u$ =15 kPa and $\phi' = 22^0$

Consolidated undrained test

Cell pressure, $\sigma_3 =$ 200 kPa

Pore water pressure, u = 150 kPa

Deviator stress at failure, $\sigma_d = ?$

$\sigma_3' = \sigma_3 - u$ =200-150=50kPa

$$\sigma_1' = \sigma_3 \tan^2\left(45 + \frac{\phi^1}{2}\right) + 2c' \tan\left(45 + \frac{\phi}{2}\right)$$

Prof. B. Jayarami Reddy

$$= 50\tan^2\left(45+\frac{22}{2}\right)+2\times 15\tan\left(45+\frac{22}{2}\right)$$

$$\sigma_1' = 154.377\text{kPa}$$

$$\sigma_1' = \sigma_1 - u$$

$$154.377 = \sigma_1 - 150$$

$$\sigma_1 = 304.377 \text{ kPa}$$

$$\sigma_d = \sigma_1 - \sigma_3 = 304.377\text{-}200 = 104.38 \text{ kPa}$$

Prof. B. Jayarami Reddy

08. In the consolidated undrained triaxial test on a saturated soil sample, the pore water pressure is zero CE1 2016

a. during shearing stage only

b. at the end of consolidation stage only

c. both at the end of consolidation and during shearing stages

d. under none of the above conditions.

Prof. B. Jayarami Reddy

09. Stress path equation for tri-axial test upon application of deviation stress is, $q = 10\sqrt{3} + 0.5p$. The respective values of cohesion, (in kPa) and angle of internal friction, are: CE2 2015

a. 20 and 20^0 **b**. 20 and 30^0 c. 30 and 30^0 d. 30 and 20^0

Ans.b

Stress path equation for triaxial test upon application of deviator stress is given by

$$q = 10\sqrt{3} + 0.5p$$

Cohesion, C =?

Angle of internal friction, $\phi = ?$

Stress path equation for triaxial test is given by

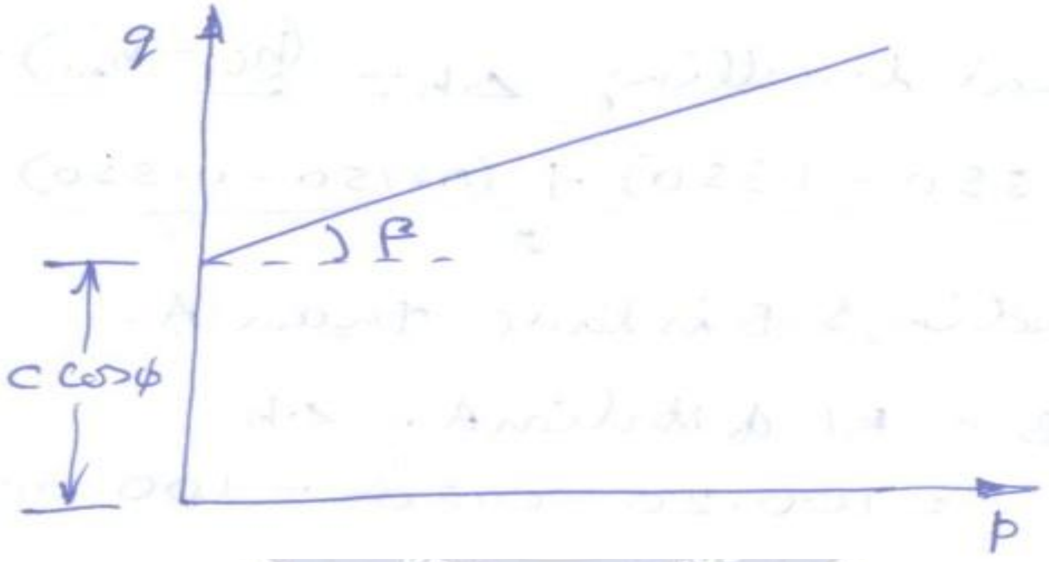
$$\frac{\sigma_1 - \sigma_3}{2} = C\cos\phi + \frac{\sigma_1 + \sigma_3}{2}.\sin\phi$$

On comparison of the equations, $C\cos\phi = 10\sqrt{3}$

$Tan\beta = \sin\phi$ =0.5 $\Rightarrow$ $\phi = 30^0$

Prof. B. Jayarami Reddy

$C\cos 30^0 = 10\sqrt{3} \Rightarrow C = 20 kN/m^2$



Prof. B. Jayarami Reddy

10. In an unconsolidated undrained Triaxial test, it is observed that an increase in cell pressure from 150 kPa to 250 kPa leads to a pore pressure increase of 80 kPa. It is further observed that, an increase of 50 kPa in deviator stress results in an increase of 25 kPa in the pore pressure. The value of Skemption's pore pressure parameter B is: CE1 2015

a. 0.5 b. 0.625 **c.** 0.8 d. 1.0

Ans. c

Increase in cell pressure, $\Delta\sigma_3 = 250 - 150$ =100kPa

Increase in pore pressure, $\Delta U = 80\,\text{kPa}$

Increase in deviator stress, $\Delta\sigma_d = 50\,\text{kPa}$

Increase in pore pressure, $\Delta U = 25\,\text{kPa}$

Skempton pore pressure parameter, $B = \dfrac{\Delta U}{\Delta\sigma_3} = \dfrac{80}{100}$ =0.8

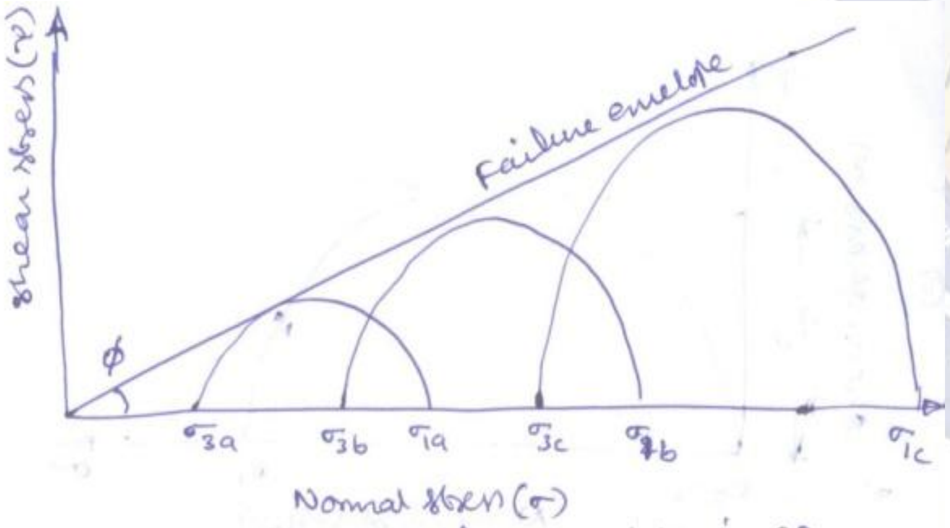
Prof. B. Jayarami Reddy

11. For a saturated cohesive soil, a tri-axial test yields the angle of internal friction as zero. The conducted test is CE1 2014

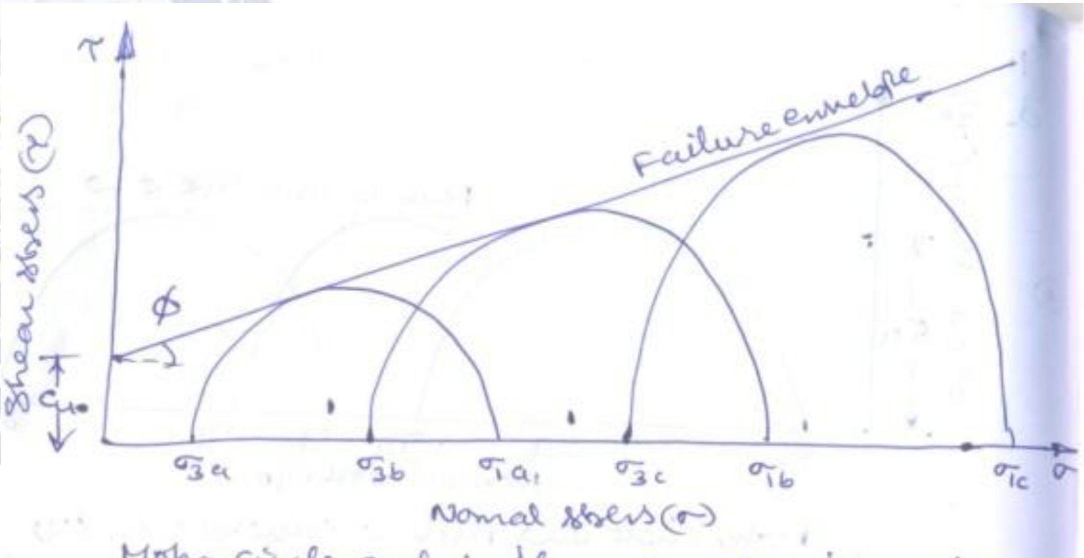
a. Consolidated Drained (CD) test b. Consolidated Undrained (CU) test

c. Unconfined compression (UC) test **d.** Unconsolidated Undrained (UU) test

Ans. d

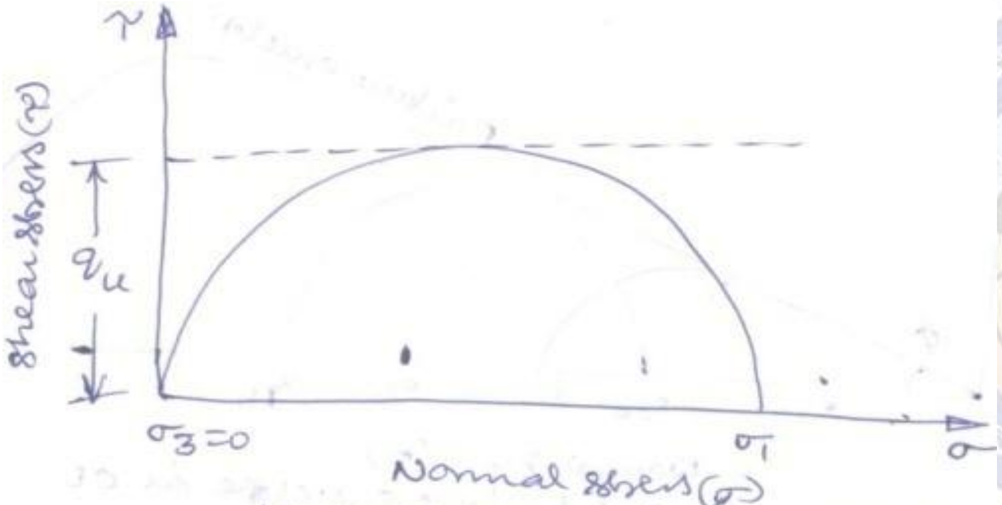


Mohr circle and failure envelope in CD triaxial test.

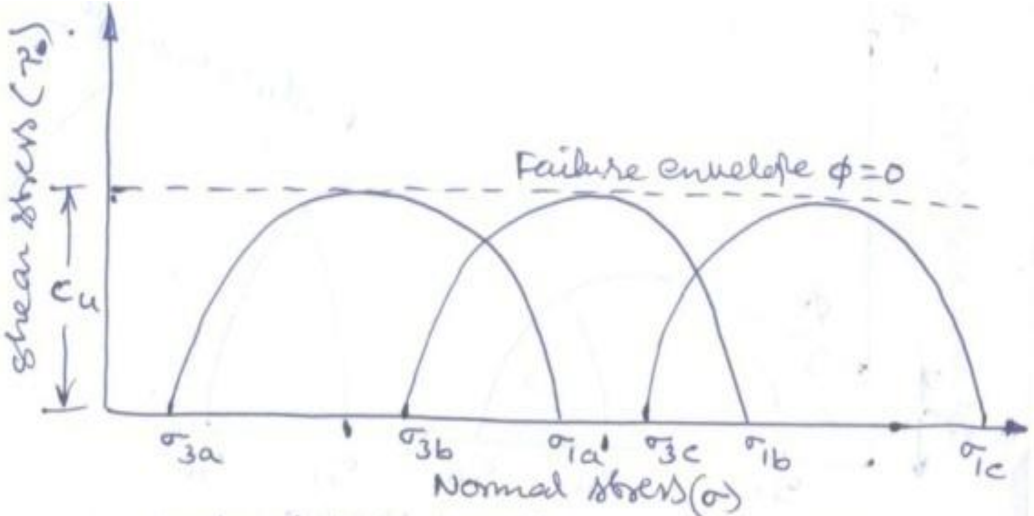


Mohr circle and failure envelope in CU triaxial test

Prof. B. Jayarami Reddy



Unconfined compression (UC) test



Mohr circle and Failure envelope in UU triaxial test

Prof. B. Jayarami Reddy

12. . The effective stress friction angle of a saturated, cohesionless soil is 38°. The ratio of shear stress to normal effective stress on the failure plane is 2012

a. 0.781 b. 0.616 c. 0.488 d. 0.438

Ans. a

Friction angle of soil, $\phi = 38^0$

The coulomb's law on the plane of failure is $\tau = c + \sigma.\tan\phi$

For cohesionless soils, C=0

τ : Shear stress

σ : Normal stress

C : Cohesion

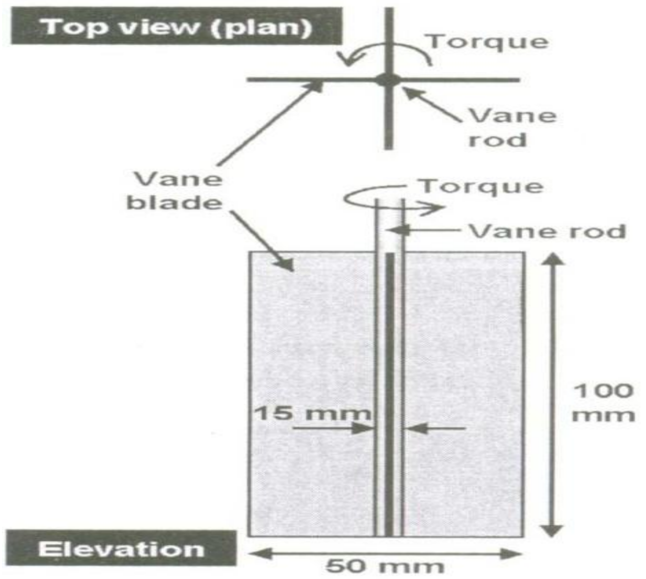
$$\frac{\tau}{\sigma} = \tan\phi \quad \tan 38^0 = 0.781$$

Prof. B. Jayarami Reddy

13. A field vane shear testing instrument (shown alongside) was inserted completely into a deposit of soft, saturated silty clay with the vane rod vertical such that the top of the blades were 500 mm below the ground surface. Upon application of a rapidly increasing torque about the vane rod, the soil was found to fail when the torque reached 4.6 Nm. Assuming mobilization of undrained shear strength on all failure surfaces to be uniform and the resistance mobilized on the surface of the vane rod to be negligible, what would be the peak undrained shear strength (rounded off to the nearest integer value kPa) of the soil? 2011

a. 5 kPa **b.** 10 kPa c. 15 kPa d. 20 kPa

Prof. B. Jayarami Reddy



Ans. b

Torque at failure $T = c_u \cdot \pi D^2 \left(\frac{H}{2} + \frac{D}{6} \right)$

T: Torque = 4.6 Nm

C_u: Undrained shear strength

D: Diameter of the vane = 50 mm

H: Height of the vane = 100mm

$$4.6 = C_u \pi \ (50)^2 \left[\frac{100}{2} + \frac{50}{6} \right] \times 10^{-9}$$

$C_u = 10 \times 10^3 \text{ N/m}^2 = 10 \text{ kPa}$

Prof. B. Jayarami Reddy

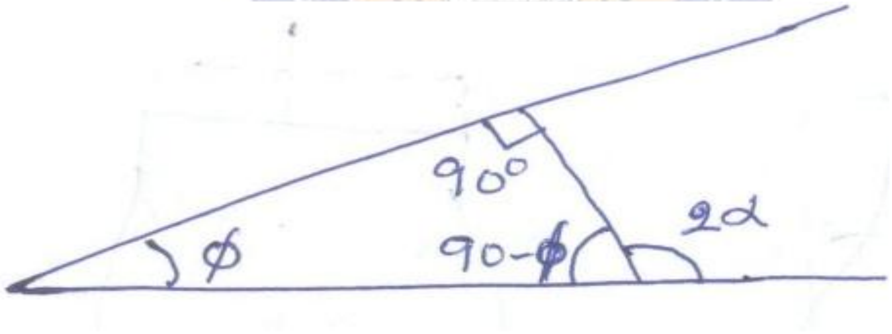
14. For a sample of dry, cohesionless soil with friction angle, , the failure plane will be inclined to the major principal plane by an angle equal to 2011

a. ϕ b. 45 c. $45^0 - \frac{\phi}{2}$ d. $45^0 + \frac{\phi}{2}$

Ans.d

The angle made by the failure plane with the vertical is $45^0 + \frac{\phi}{2}$ i.e., with the plane of which the major principal stress acts.

The angle made by the failure plane with the horizontal is $45^0 - \frac{\phi}{2}$ for the passive case.



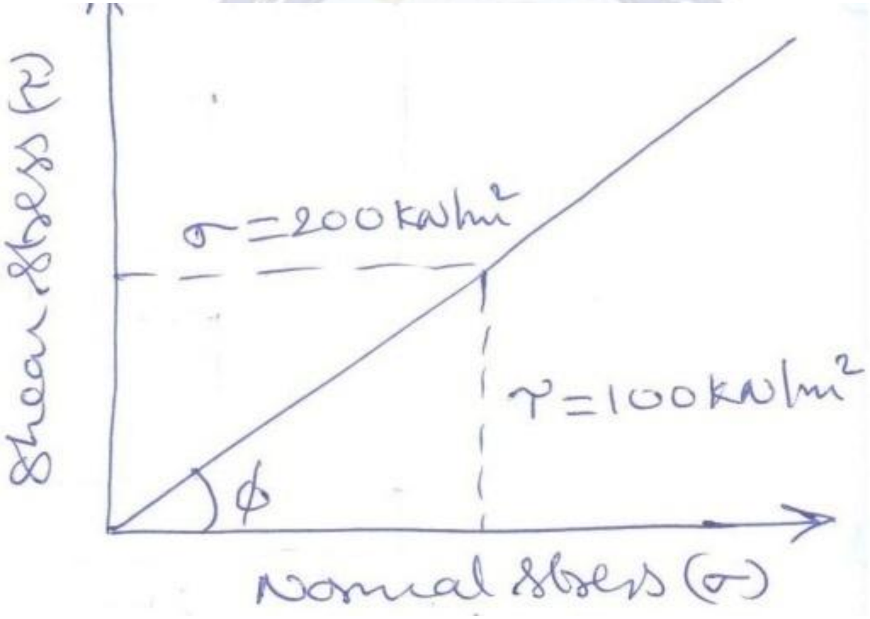
$2\alpha = 90° + \varphi$ $\alpha = 45° + \phi / 2$

Prof. B. Jayarami Reddy

15. A direct shear test was conducted on a cohesionless soil ( ) specimen under a normal stress of 200 kN/m^2. The specimen failed at a shear stress of 100 kN/m^2. The angle of internal friction of the soil (degrees) is 2008

a. 26.6 b. 29.5 c. 30.0 d. 32.6

Ans. a



Cohesion, $C = 0$

Normal stress $\sigma = 200$ kN/m^2

Shear stress $\tau = 100$ kN/m^2

Angle of internal friction $\phi = ?$

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

$$\tan\phi = \frac{\tau}{\sigma} = \frac{100}{200} = 0.5$$

$$\phi = \tan^{-1}(0.5) = 26.6^0$$

Prof. B. Jayarami Reddy

16. A clay soil sample is tested in a triaxial apparatus in consolidated-drained conditions at a cell pressure of 100 kN/m^2. What will be the pore water pressure at a deviator stress of 40 kN/m^2 ? 2007

a. 0 kN/m^2 b. 20 kN/m^2 c. 40 kN/m^2 d. 60 kN/m^2

Ans. a

In the consolidated drained test, the soil sample is first consolidated under an appropriate cell pressure. By keeping the cell pressure constant, the soil sample is then sheared by applying the deviator stress so slowly that excess pore water pressure does not develop during the test. Thus at any stage of the test, the total stresses are the effective stresses.

17. A sample of saturated cohesionless soil tested in a drained triaxial compression test showed an angle of internal friction of 30^0. The deviatoric stress at failure for the sample at a confining pressure of 200 kPa is equal to 2006

a. 200 kPa **b.** 400 kPa c. 600 kPa d. 800 kPa

Ans. b

Angle of internal friction, $\phi = 30^0$

Confining or cell pressure, $\sigma_3 = 200\,\text{kPa}$

Cohesion, $C = 0$

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

Angle of the failure plane, $\alpha = 45^0 + \frac{\phi}{2} = 45 + \frac{30}{2} = 60^0$

$$\sigma_1 = \sigma_3.\tan^2\alpha + 2c.\tan\alpha = \sigma_3\tan^2\alpha = 200\tan^2 60^0 = 600\,\text{kPa}$$

Deviatoric stress, $\sigma_d = 600 - 200 = 400\,\text{kPa}$

Prof. B. Jayarami Reddy

18. For a triaxial shear test conducted on a sand specimen at a confining pressure of 100 kN/m^2 under drained conditions, resulted in a deviator stress ( ) at failure of 100 kN/m^2. The angle of shearing resistance of the soil would be

a. 18.43^0 **b.** 19.47^0 c. 26.56^0 d. 30^0 2005

Ans.b

Confining pressure, $\sigma_3 = 100\,\text{kN/m}^2$

Deviator stress, $\sigma_1 - \sigma_3 = 100\,\text{kN/m}^2$

$\sigma_1 = 100 + \sigma_3$ = 100+100 =200kN/m^2

The relationship between σ_1 and σ_3 is, $\sigma_1 = \sigma_3\, N_\phi + 2c.\sqrt{N_\phi}$

The given soil sample is sandy, C=0

$$\sigma_1 = \sigma_3 . N_\phi$$

$$N_\phi = \frac{\sigma_1}{\sigma_3} = \frac{200}{100} = 2$$

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

Prof. B. Jayarami Reddy

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

$$45^0+\frac{\phi}{2}=54.73^0 \qquad \phi=19.47^0$$

(or)

We know that, $\dfrac{\sigma_1}{\sigma_3}=\dfrac{1+\sin\phi}{1-\sin\phi}$

$$\sin\phi=\frac{\sigma_1-\sigma_3}{\sigma_1+\sigma_3}=\frac{100}{200+100}=\frac{1}{3}$$

$$\phi=\mathrm{Sin}^{-1}\left(\frac{1}{3}\right)$$

$$\phi=19.47^0$$

Prof. B. Jayarami Reddy

19. The undrained cohesion of a remoulded clay soil is 10 kN/m^2. If the sensitivity of the clay is 20, the corresponding remoulded compressive strength is 2004

a. 5 kN/m^2 b. 10 kN/m^2 **c.** 20 kN/m^2 d. 200 kN/m^2

Ans. c

Undrained cohesion of remoulded clay soil, C_u= 10 kN/m^2

Sensitivity of the clay = 20

Remoulded compressive strength = q_u

$$c_u = \frac{q_u}{2}$$

$$q_u = 2c_u \quad = 2 \times 10 = 20 \text{ kN/m}^2.$$

$$\text{Sensitivity} = \frac{\text{Undistrubed shear strength}}{\text{Distrubed shear strength}}$$

Prof. B. Jayarami Reddy

20. In an undrained triaxial test on a saturated clay, the Poisson's ratio is 2004

a. $\frac{\sigma_3}{(\sigma_1+\sigma_3)}$ b. $\frac{\sigma_3}{(\sigma_1-\sigma_3)}$ c. $\frac{(\sigma_1-\sigma_3)}{\sigma_3}$ d. $\frac{(\sigma_1+\sigma_3)}{\sigma_3}$

Ans. a

In unconsolidated undrained test on saturated clay, the clay remains unconsolidated and no volume change takes place.

$$\frac{\sigma_3}{E}-\mu.\frac{\sigma_1}{E}-\mu.\frac{\sigma_3}{E}=0$$

$$\sigma_3-\mu(\sigma_1+\sigma_3)=0$$

$$\mu=\frac{\sigma_3}{(\sigma_1+\sigma_3)}$$

Prof. B. Jayarami Reddy

21. In a triaxial test carried out on a cohesionless soil sample with a cell pressure of 20 kPa, the observed value of applied stress at the point of failure was 40 kPa. The angle of internal friction of the soil is 2002

a. 10^0 b. 15^0 c. 25^0 **d.** 30^0

Ans. d

Cell pressure, $\sigma_3 = 20\,\text{kPa}$

Stress at the point of failure, $\sigma_d = 40\,\text{kPa}$

Angle of internal friction, $\varphi = ?$

Major principal stress, $\sigma_1 = \sigma_3 + \sigma_d = 20 + 40 = 60\,\text{kPa}$

For cohesionless soils $\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\varphi}{2}\right)$

$$60 = 20\tan^2\left(45^0 + \frac{\varphi}{2}\right) \Rightarrow \tan^2\left(45^0 + \frac{\varphi}{2}\right) = 3$$

$$45 + \frac{\varphi}{2} = 60^0 \quad \Rightarrow \varphi = 30^0$$

Prof. B. Jayarami Reddy

22. If the effective stress strength parameters of a soil are =10 kPa and , the shear strength on a plane within the saturated soil mass at a point where the total normal stress is 300 kPa and pore water pressure is 150 kPa will be 2002

a. 90.5 kPa **b.** 96.6 kPa c. 101.5 kPa d. 105.5 kPa

Ans. b

Effective cohesion, $c' = 10\,\text{kpa}$

Effective angle of friction, $\varphi = 30^0$

Total normal stress, $\sigma = 300\,\text{kPa}$

Pore water pressure, $u = 150\,\text{kPa}$

Shear strength, $\tau = c' + \sigma' \tan\varphi'$

$$\tau = c' + (\sigma - u)\tan\phi' = 10 + (300 - 150)\tan 30^0 = 96.6\,\text{kN/m}^2$$

Prof. B. Jayarami Reddy

23. The following two statements are made with respect to different sand samples having the same relative density. Identify if they are TRUE or FALSE. 2001

I. Poorly graded sands will have lower friction angle than the well graded sands.

II. The particle size has no influence on the friction angle of sand.

a. II is TRUE but I is FALSE.
b. Both are FALSE statements
c. Both are TRUE statements.
d. I is TRUE but II is FALSE.

Ans. d

Angle of internal friction $\phi' = 36^0 + \phi_1 + \phi_2 + \phi_3 + \phi_4$

$\varphi' = 36^0$ represents the φ' value for average conditions.

ϕ_1 : Influence of grain shape on φ'

= 1^0 for angular grains

= 0^0 for subangular grains

= 3^0 for rounded grains

= -6^0 for well rounded grains

Prof. B. Jayarami Reddy

ϕ_2 : Influence of grain size on φ'

$= 0^0$ for sand

$= 1^0$ for fine gravel

$= 2^0$ for Medium and coarse gravel

φ_3 : Correction factor for gradation

$= -3^0$ for poorly graded soil

$= 0^0$ for medium uniformity

$= 3^0$ for well graded soil

φ_4 : Correction factor for relative density

$= -6^0$ for loosest packing

$= 0^0$ for medium density

$= 6^0$ for densest packing

φ(Poorly graded soil) $< \varphi$ (Well graded soil)   Statement I is true

The friction angle varies with particle size.   Statement II is false

Prof. B. Jayarami Reddy

24. In a drained triaxial compression test, a saturated specimen of cohesionless sand fails under a deviatory stress of 3 kgf/cm^2 when the cell pressure is 1 kgf/cm^2. The effective angle of shearing resistance of sand is about 2000

a. 37^0 b. 45^0 c. 53^0 d. 20^0

Ans. a

Deviatoric stress, $\sigma_d = 3 \text{ kg/cm}^2$

Cell pressure $\sigma_3 = 1 \text{ kg/cm}^2$

Major principal stress, $\sigma_1 = \sigma_3 + \sigma_d$ = 1+3 = 4Kg/cm^2

Angle of shearing resistance, $\varphi = ?$

For a cohesionless sand, C=0

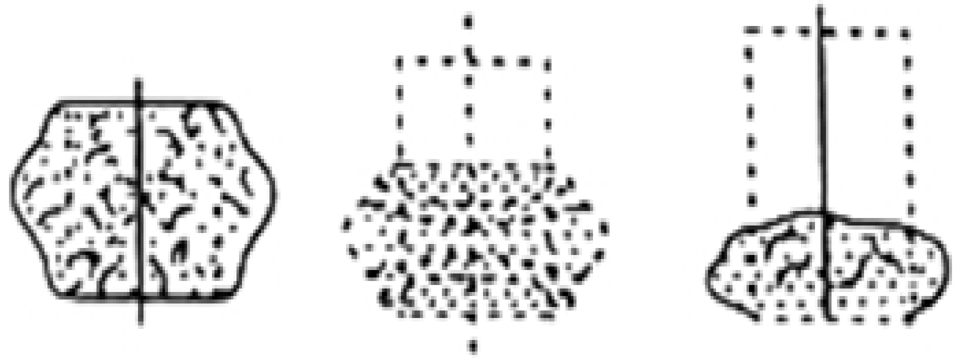
$$\sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi}$$

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

$$4 = 1\tan^2\left(45 + \frac{\varphi}{2}\right) \qquad 45 + \frac{\phi}{2} = 63.4^0 \qquad \phi = 36.87^0$$

Prof. B. Jayarami Reddy

25. Triaxial compression test of three soil specimens exhibited the patterns of failure as shown in the figure. Failure modes of the respectively are 1999



a. brittle, semi-plastic, plastic   b. semi-plastic, brittle, plastic

c. plastic, brittle, semi-plastic   d. brittle, plastic, semi-plastic

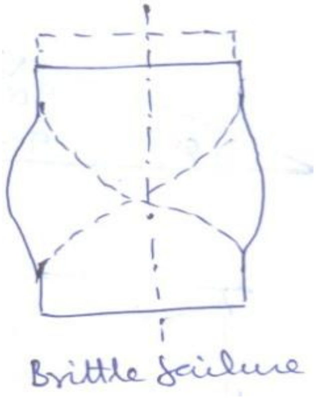
Prof. B. Jayarami Reddy

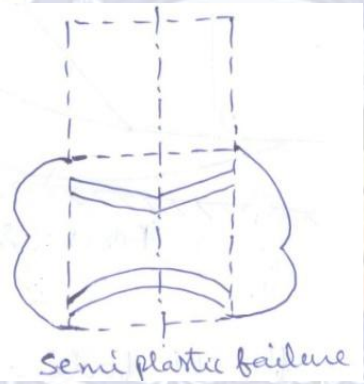
Ans. a

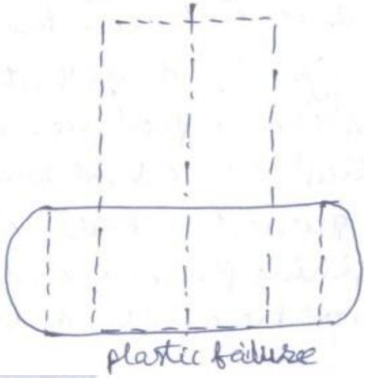
Brittle failure : well defined shear plane

Semi plastic failure : shear cones and some lateral bulging

Plastic failure : well expressed lateral bulging







Failure patterns in triaxial compression test

Prof. B. Jayarami Reddy

26. Vane tester is normally used for determining in-situ shear strength of

a. softly clays b. sand c. stiff clays d. gravel 1997

Ans. a

Vane shear test is normally used for determining in-situ shear strength of soft saturated clay deposits.

Prof. B. Jayarami Reddy

27. The appropriate field test to determine the undrained shear strength of soft clay is

a. plate load test   b. static cone penetration test   1995

c. standard penetration test   **d.** vane shear test

Ans. d

Vane shear test is a field test used to determine the undrained shear strength of soft clay.

28. The unconfined compressive strength of a ‘stiff clay’ falls in the range 1995

a. less than 50 kN/m^2 b. 50 to 100 kN/m^2

c. 100 to 200 kN/m^2 d. Above 200 kN/m^2

Ans. c

Term	Diagnostic features	Undrained compressive strength
Very soft soil	Exudes between fingers when squeezed	<25 kPa
Soft soil	Easily indented by fingers	25 – 50 kPa
Firm soil	Indented only by strong finger pressure	50 – 100 kPa
Stiff soil	Indented by thumb pressure	100 – 200 kPa
Very stiff soil	Indented by thumb nail	200 – 400 kPa
Hard soil	Difficult to indent by thumbnail	400 -1000 kPa

29. In a drained triaxial compression test conducted on dry sand, failure occurred when the deviator stress was 218 kN/m^2 at a confining pressure of 61 kN/m^2. Wht is the efective angle of shearing resistance and the inclination of failure plane to major principal plane? 1993

a. 34^0, 62^0 b. 34^0, 28^0 c. 40^0, 25^0 **d.** 40^0, 65^0

Ans. d

Deviator stress, $\sigma_d = 218 \text{ kN/m}^2$

Confining pressure, $\sigma_3 = 61 \text{ kN/m}^2$

For dry sand, *C*=0

Major principal stress, $\sigma_1 = \sigma_3 + \sigma_d$ =61+218=279kN/m^2

Angle of internal friction $= \varphi$

Inclination of major principal plane $= \alpha$

$$\sigma_1 = \sigma_3 \tan^2 \alpha + 2c \tan \alpha$$

$$279 = 61 \tan^2 \alpha$$

$$\alpha = 65^0$$

$$\alpha = 45^0 + \frac{\phi}{2}$$

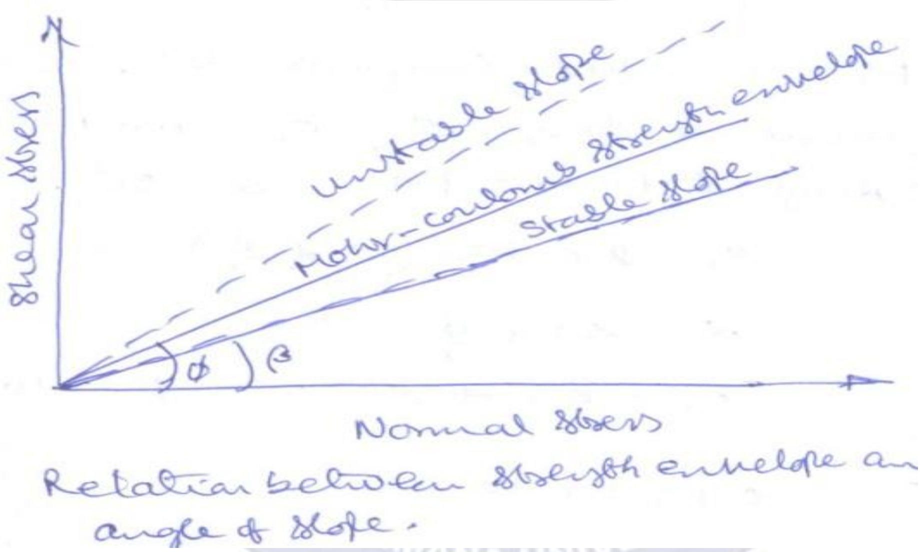
$$65 = 45^0 + \frac{\varphi}{2}$$

$$\varphi = 40^0$$

Prof. B. Jayarami Reddy

30. Write True/False in the answer book. If the Mohr circle for a given state of stress lies entirely below the Mohr envelope for a soil, then the soil will be unstable for that state of stress. 1992

Ans. False



If the Mohr circle for a given state of stress lies entirely below the Mohr envelope for a soil, then the soil will be stable for that state of stress.

If the Mohr circle for a given state of stress lies entirely above the Mohr envelope for a soil, then the soil will be unstable for that state of stress.

31. An unconfined compression test yielded a strength of 0.1 N/mm^2. If the failure plane is inclined at 50° to the horizontal, what are the values of the shear strength parameters? 1992

Ans. $\varphi = 10^0$, $c = 41.9$ kN/m^2

For unconfined compression test, $\sigma_3 = 0$

Compressive strength, $q_u = \sigma_1 = 0.1$ N/mm^2

Angle of failure plane $\alpha = 50^0$

$$\alpha = 45 + \frac{\phi}{2}$$

$$50 = 45 + \frac{\varphi}{2} \Rightarrow \varphi = 10^0$$

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

$$0.1 = 2c \tan 50^0$$

$C = 0.0419$ N/mm^2

Cohesion $c = 0.0419$ N/mm^2 $= 41.9$ kN/m^2

Angle of internal friction, $\varphi = 10^0$

Prof. B. Jayarami Reddy