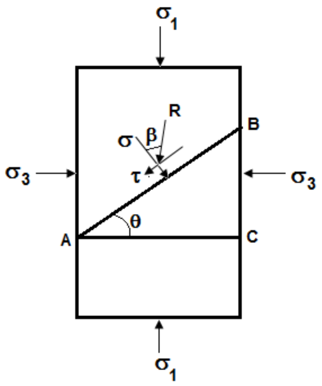
SHEAR STRENGTH OF SOILS

The shear strength of a soil is its maximum resistance to shear stresses just before failure. Soils are seldom subject to direct shear. However shear stresses develop when the soil is subjected to direct compression. The shear failure of a soil mass occurs when the shear stresses induced due to applied compressive loads exceed the shear strength of the soil. Failure in soils occurs by the relative movements of soil particles.

STRESS SYSTEMS WITH PRINCIPAL PLANES PARALLEL TO COORDINATE AXES

At every point in a stressed body, there exist three mutually perpendicular planes on which the shear stresses are zero. These planes are known as principal planes. The plane with maximum compressive stress (σ_1) is known as the major principal plane, and that with minimum compressive stress (σ_3) is known as minor principal plane. The third plane is subjected to a stress value in between σ_1 and σ_3, and is known as the intermediate principal plane.



The critical stress values generally occur on planes normal to the intermediate plane. Consider a plane AB of unit width perpendicular to the plane of the paper, inclined at θ to the major principal plane.

Resolving the forces on the wedge ABC in the horizontal direction

$$\sigma_3(BC\ x\ 1) = \sigma\,(AB\ x\ 1)\sin\theta - \tau\,(AB\ x\ 1)\cos\theta$$

where σ = normal stress on AB
τ = shear stress on AB

Dividing throughout by AB

$$\sigma_3\left(\frac{BC}{AB}\right) = \sigma\sin\theta - \tau\cos\theta$$

$$\sigma_3\sin\theta = \sigma\sin\theta - \tau\cos\theta \qquad \ldots\ldots\ldots\ldots\text{(a)}$$

Resolving forces in the vertical direction

$$\sigma_1(AC\ x\ 1) = \sigma\,(AB\ x\ 1)\cos\theta + \tau\,(AB\ x\ 1)\sin\theta$$

Dividing throughout by AB

$$\sigma_1\left(\frac{AC}{AB}\right) = \sigma\cos\theta + \tau\sin\theta$$

$$\sigma_1\cos\theta = \sigma\cos\theta + \tau\sin\theta \qquad \ldots\ldots\ldots\ldots\text{(b)}$$

Solving equations (a) and (b) for σ and τ

$$\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2}\cos 2\theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

$$\tau = \frac{1}{2}(\sigma_1 - \sigma_3)\sin 2\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

Resultant stress on the plane AB is $R = \sqrt{\sigma^2 + \tau^2}$

Equations (1) and (2) give the normal stress and shear stress on the inclined plane AB making an angle, θ, measured counter-clockwise (CCW) with the major principal plane AC.

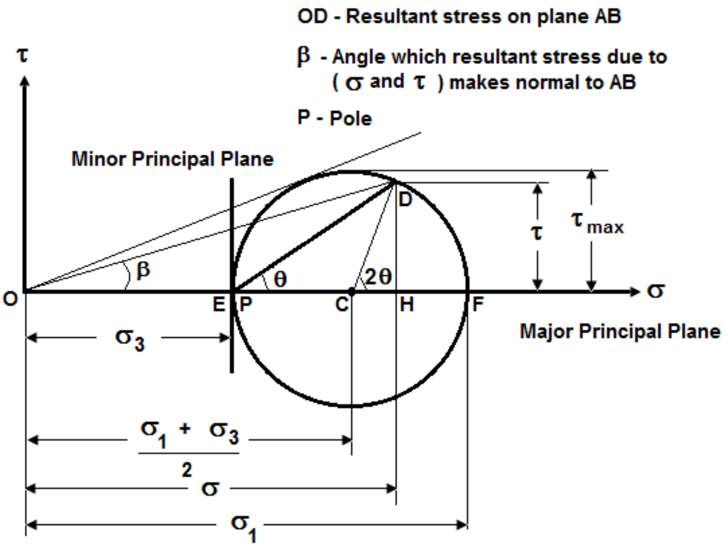
MOHR CIRCLE

Mohr devised a graphical method for the determination of stresses on a plane inclined to the principal planes. ***Mohr circle represents the stress conditions at a given point. The locus of stress coordinates (σ, τ) for all planes through a point is a circle, called the Mohr Circle of stresses.***

PROCEDURE TO DRAW MOHR CIRCLE

1. An origin is selected and the normal stresses are plotted along the horizontal axis and shear stresses on the vertical axis.

2. Compressive stresses are taken as positive and are plotted towards the right of the origin (along +ve x-axis).



3. Shear stress is generally taken as positive if it causes CCW couple at a point inside the wedge ABC.

4. Positive shear stresses are plotted upward from the origin (along +ve y-axis).

5. The point E represents the minor principal stress σ_3 and the point F, the major principal stress σ_1.

6. The point C is the center of the circle with normal stress coordinate equal to $\left(\frac{\sigma_1 + \sigma_3}{2}\right)$.

7. The Mohr circle is drawn with C as the center and EF as the diameter. Each point on the circle gives σ and τ on a particular plane.

8. Point D on the circle gives the stresses on the plane AB inclined at angle θ to the major principal plane. The line DE makes an angle θ with the σ axis. The angle DCF subtended at the center is twice angle DEC.

9. From the figure

$$OH = OC + CH$$

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

$$DH = \tau = \frac{1}{2}(\sigma_1 - \sigma_3)\sin 2\theta$$

10. The point E is a unique point which is known as pole P, which is known as the pole P or the origin of planes.

If a line is drawn from any point, say D, on the Mohr circle, parallel to the plane, say AB, whose stresses are represented by that point, it will intersect the circle at pole P.

Once the pole has been located, the stresses on any other plane making an angle α with the major principal plane can be determined graphically by drawing a line through the pole and making angle α with the σ-axis. The coordinates of that point obtained by the intersection of this line with the circle give the stresses on that plane.

Line OD represents the magnitude of the resultant stress on the inclined plane AB. The angle of obliquity of the resultant with the normal of the plane AB is equal to β.

IMPORTANT CHARACTERISTICS OF MOHR CIRCLE

1. The maximum shear stress is mathematically equal to $\frac{\sigma_1 - \sigma_3}{2}$ and its occurs on a plane inclined at 45^o to the principal planes.

2. The resultant stress on plane AB is equal to $\sqrt{\sigma^2 + \tau^2}$ and its angle of obliquity with the normal to the plane is equal to angle β given by

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

3. The maximum angle of obliquity β_{max} is obtained by drawing a tangent to the circle from the origin O.

$$\beta_{max} = sin^{-1}\left[\frac{(\sigma_1 - \sigma_3)/2}{(\sigma_1 + \sigma_3)/2}\right] = sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right)$$

4. ***The plane of maximum obliquity is the criterion governing failure.*** The shear stress τ_f on the plane of maximum obliquity is less than the maximum shear stress τ_{max}.

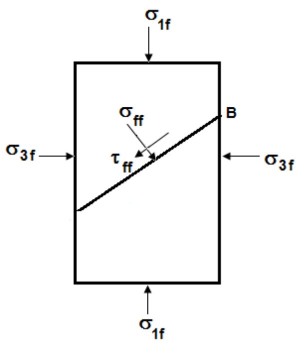
5. Mohr circle represents all possible combinations of shear and normal stresses at the stressed point.

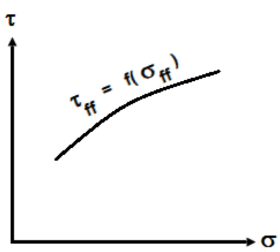
MOHR FAILURE CRITERION AND MOHR FAILURE HYPOTHESIS

A material fails when the shear stress on the failure plane at failure reaches a value which is a unique function of the normal stress on that plane.

$$\tau_{ff} = f(\sigma_{ff})$$

The first subscript *f* refers to “failure plane” and the second subscript *f* refers to “at failure”.





For the soil element at failure, principal stresses at failure are σ_{1f} and σ_{3f} and the normal and shearing stresses on the failure plane are σ_{ff} and τ_{ff}.

Taking data from several tests carried out on different samples with different σ_1 and σ_3 range for each test on the same material, a series of Mohr's circles, each circle representing data from one test, can be plotted.

A curve drawn tangential to the Mohr circles is called Mohr failure envelope. The Mohr failure envelope expresses the functional form of the relationship between τ_{ff} and σ_{ff} as $\tau_{ff} = f(\sigma_{ff})$.

Inferences from Mohr failure Envelope and Mohr Stress Circle

1. If the stress condition at a given point is represented by a Mohr circle which lies below the Mohr failure envelope, then every plane passing through this point has a shearing stress which is smaller than the shearing strength. This represents a stable condition.

2. Circles lying above the Mohr envelope cannot exist because it is not possible for shear stress to exceed shear strength. Failure would have already occurred as soon as Mohr circle touches the envelope.

3. Any Mohr circle, whose tangent is the failure envelope, represents a condition wherein the point of tangency gives the stress conditions on the failure plane at failure. Shear stress on failure plane now is the shear strength.

4. Mohr failure hypothesis identifies the point of tangency of the Mohr envelope to the Mohr circle at failure.

5. Mohr envelope is unique for a given material and is independent of the stresses induced in the material while Mohr stress circle is directly related to the stresses imposed by the loading and is unrelated to the material strength.

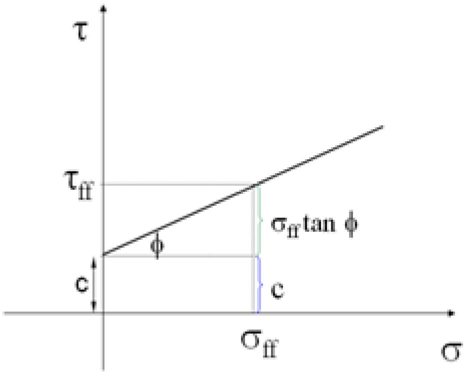
6. In the complete Mohr circle, the failure planes, mutually perpendicular, form angles of $\pm\theta_f$ with the major principal planes.

COULOMB'S EQUATION AND MOHR COULOMB CRITERION

1. Coulomb observed that one component of the shearing strength, called the intrinsic cohesion (or apparent cohesion) is constant for a given soil and is independent of the applied stress.
2. The other component, namely, frictional resistance, varies directly as the magnitude of the normal stress on the plane of the rupture.
3. Coulomb's equation is written as

$$\tau_f = c + \sigma_{ff} \tan \varphi$$

where τ_f = shear strength of the soil
c = apparent cohesion
σ_{ff} = normal stress on the failure plane at failure.
φ = angle of internal friction



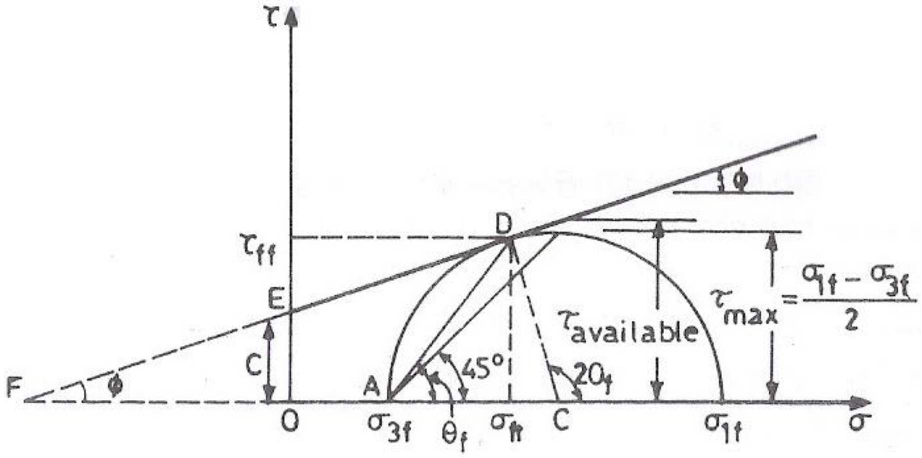
4. c and φ are referred to as **shear parameters**. They are not inherent or fundamental properties of the soil. They are related to the type of the test and the conditions under which these are measured.
5. The graphical representation of Coulomb's equation is a straight line. The intercept made by the straight line on the τ-axis represents cohesion c and the slope of the straight line gives the angle of shearing resistance φ.
6. c and φ are values obtained by plotting the results of a shear test.
7. The angle of failure plane relative to the major principal plane, θ_f, can be expressed in terms of angle of shearing resistance φ as

$$\theta_f = 45 + \frac{\varphi}{2}$$

8. The shear stress on the failure plane is not the maximum shear stress in the element. The maximum shear stress acts on planes inclined at 45° to the major principal plane and is equal to $\frac{1}{2}(\sigma_{1f} - \sigma_{3f})$. $\tau_{ff} < \tau_{max.}$

Mohr coulomb failure criterion in terms of principal stresses at failure

Mohr-Coulomb failure criterion can be expressed in terms of the relationship between the principal stresses σ_{1f} and σ_{3f}.



$$\sin\varphi = \frac{DC}{FC} = \frac{DC}{CO + OF} = \frac{(\sigma_{1f} - \sigma_{3f})/2}{(\sigma_{1f} + \sigma_{3f})/2 + 2c\cot\varphi}$$

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

Rearranging terms,

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

The above equation is also expressed in the form

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

$$\sigma_{1f} = \sigma_{3f}N_\varphi + 2c\sqrt{N_\varphi}$$

where

$$N_\varphi = \frac{1 + \sin\varphi}{1 - \sin\varphi} = \tan^2\left(45^o + \frac{\varphi}{2}\right)$$

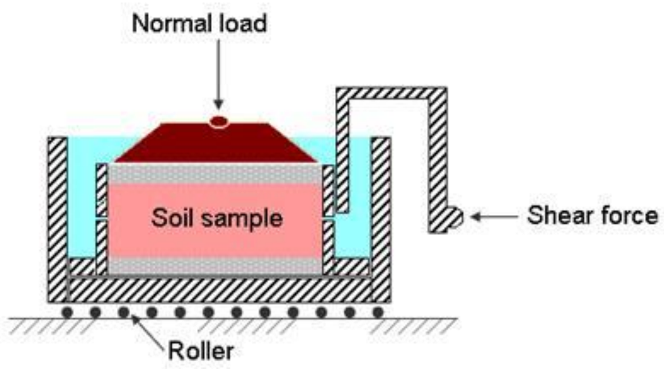
If c = 0 (a purely frictional soil)

$$\sigma_{1f} = \sigma_{3f}\left(\frac{1 + \sin\varphi}{1 - \sin\varphi}\right)$$

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

The above equation is known as obliquity relationship which is valid only for soils having c = 0. The stress coordinates of point D in the figure are the stresses on the plane of maximum obliquity in the soil element. The ratio τ_{ff}/σ_{ff} is a maximum on the plane of maximum obliquity and not on the plane of maximum shear stress. The plane of maximum obliquity is the criterion governing failure.

DIRECT SHEAR TEST



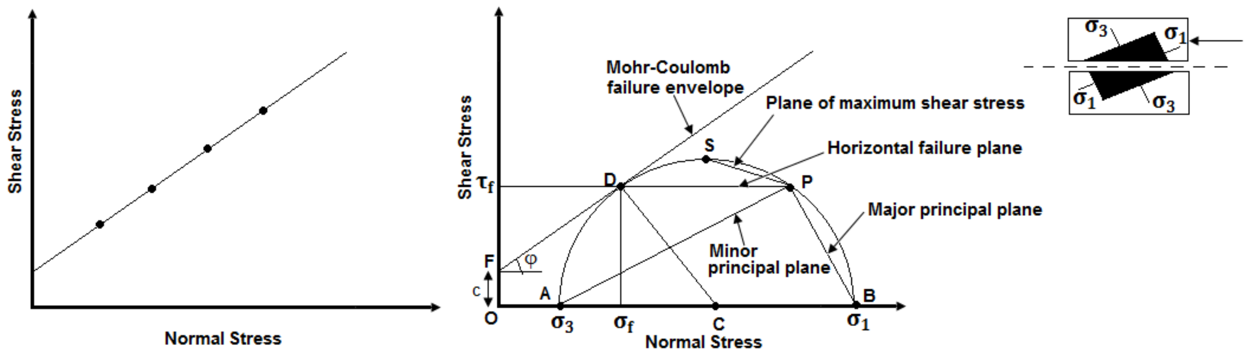
The test is carried out on a soil sample confined in a metal box of square cross-section which is split horizontally at mid-height. A small clearance is maintained between the two halves of the box. The soil is sheared along a predetermined plane by moving the top half of the box relative to the bottom half. The box is usually square in plan of size 60 mm x 60 mm. A typical shear box is shown.

If the soil sample is fully or partially saturated, perforated metal plates and porous stones are placed below and above the sample to allow free drainage. If the sample is dry, solid metal plates are used. A load normal to the plane of shearing can be applied to the soil sample through the lid of the box.

Tests on sands and gravels can be performed quickly, and are usually performed dry as it is found that water does not significantly affect the drained strength. For clays, the rate of shearing must be chosen to prevent excess pore pressures building up.

As a vertical normal load is applied to the sample, shear stress is gradually applied horizontally, by causing the two halves of the box to move relative to each other. The shear load is measured together with the corresponding shear displacement. The change of thickness of the sample is also measured.

A number of samples of the soil are tested each under different vertical loads and the value of shear stress at failure is plotted against the normal stress for each test. Provided there is no excess pore water pressure in the soil, the total and effective stresses will be identical. From the stresses at failure, the failure envelope can be obtained.



Orientation of principal planes at failure in direct shear test

1. Plot point D(σ_f, τ_f) i.e., stresses on the failure plane at failure.
2. If c = 0, draw the failure envelope through the origin O and point D. Measure angle of shearing resistance, φ.
3. At point D, draw a normal DC to the failure envelope. The point C, where the normal intersects the horizontal axis is the centre of the Mohr circle.
4. With C as centre and CD as radius, draw the Mohr circle. Read the principal stresses at failure σ_1 and σ_3 at points B and A.
5. From D, draw a horizontal line to cut the Mohr circle at P. P is the pole or origin of planes.
6. BP is the orientation of the major principal plane and AP is the orientation of minor principal plane.
7. PS is the plane of maximum shear stress.

The test has several **advantages:**

- The sample preparation is easy. The test is simple and convenient.

- As the thickness of the sample is relatively small, the drainage is quick and pore pressure dissipates very rapidly. Consequently consolidated-drained and consolidated-un drained tests take relatively small period.

- It is ideally suited to conduct drained tests on cohesionless soils.

- Large samples can be tested in large shear boxes, as small samples can give misleading results due to imperfections such as fractures and fissures, or may not be truly representative.

- Samples can be sheared along predetermined planes, when the shear strength along fissures or other selected planes are needed.

The **disadvantages** of the test include:

- The failure plane is always horizontal in the test, and this may not be the weakest plane in the sample. The stress distribution on the failure plane is not uniform. Failure of the soil occurs progressively from the edges towards the centre of the sample.

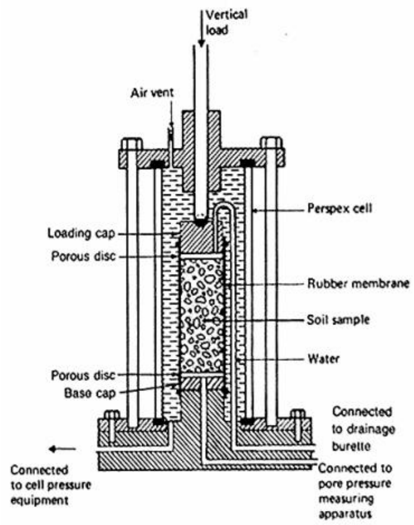
- The stress conditions are known only at failure. The conditions prior to failure are indeterminate and therefore, the Mohr circle cannot be drawn.

- There is no provision for measuring pore water pressure in the shear box and so it is not possible to determine effective stresses from undrained tests.

- The shear box apparatus cannot give reliable undrained strengths because it is impossible to prevent localised drainage away from the shear plane.

TRIAXIAL TEST

The triaxial test is carried out in a cell on a cylindrical soil sample having a length to diameter ratio of 2. The usual sizes are 76 mm x 38 mm and 100 mm x 50 mm. Three principal stresses are applied to the soil sample, out of which two are applied water pressure inside the confining cell and are equal. The third principal stress is applied by a loading ram through the top of the cell and is different to the other two principal stresses. A typical triaxial cell is shown in figure.



Triaxial cell

The soil sample is placed inside a rubber sheath which is sealed to a top cap and bottom pedestal by rubber O-rings. For tests with pore pressure measurement, porous discs are placed at the bottom, and sometimes at the top of the specimen. Filter paper drains may be provided around the outside of the specimen in order to speed up the consolidation process. Pore pressure generated inside the specimen during testing can be measured by means of pressure transducers. A valve controls drainage within the specimen. The valve can be either closed thereby preventing drainage or opened which allows for drainage.

Before commencing the test, the sample must be 100% saturated.

The triaxial compression test consists of two stages:

First Stage or Consolidation Phase: The first phase is the consolidation phase during which cell pressure or confining pressure is gradually increased to the required amount. This produces a uniform confining stress all round the specimen equal to the minor principal stress, σ_3. During this phase the soil may or may not be allowed to consolidate depending on the type of test being performed.

Initially there is no stress on the specimen and the Mohr circle appears as a point at the origin. As the cell pressure is increased during the consolidation phase, the Mohr circle still remains a point as $\sigma_1 = \sigma_3$. However, the Mohr circle moves to the right along the σ- axis.

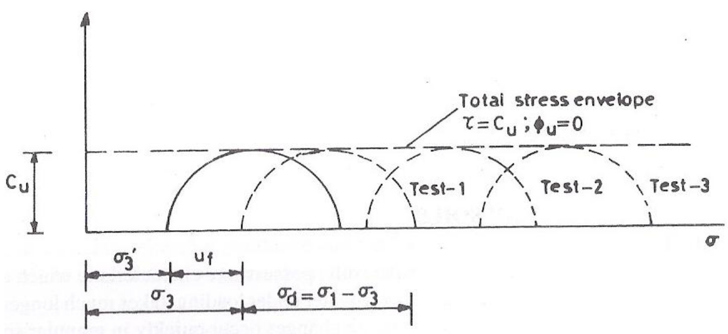
Second Stage or Shear Phase: In this, additional axial stress (also called deviator stress, $\sigma_1 - \sigma_3$) is applied to the top of the specimen which induces shear stresses in the sample. Since there are no shear stresses on either the top or sides of the specimen, these are principal planes. The major principal stress is applied to the top of the specimen. The axial stress is continuously increased until the sample fails.

During the second stage, when additional axial stresses are applied, shear stresses are induced in the soil. The resulting pore pressure is not equal to the applied axial stress and may even be negative, under certain conditions.

During both the stages, the applied stresses, axial strain, and pore water pressure or change in sample volume can be measured.

TEST TYPES
There are several test variations, and those used mostly in practice are:

UU (UNCONSOLIDATED UNDRAINED) TEST



In this test, cell pressure during the consolidation phase is applied without allowing drainage. Then keeping cell pressure constant, deviator stress is increased to failure without drainage. Thus, the drainage valve is closed throughout the test. This is a total stress test and does not usually intend measuring the pore pressures. It is also called Q- or Quick test since there is no waiting for the consolidation in the first phase or for drainage during the second phase.

Since the drainage valve is closed during the consolidation phase, no consolidation occurs and the soil does not gain any strength. If three tests are conducted on identical samples, each with an increasing confining pressure, **the Mohr circles at failure for all the tests will have the same diameter**. The total stress failure envelope (line tangential to all the Mohr circles) will have a zero slope (horizontal line) and intersects the τ-axis at c_u. This is known as the φ = 0 condition. ***Saturated clays loaded under undrained conditions fail under φ = 0 condition.*** The **failure envelope cannot be drawn in terms of effective stresses for a saturated soil under undrained conditions** due to the fact that an increase in confining pressure results in an equal increase in pore water pressure and hence **only one Mohr circle can be drawn in terms of effective stresses and it has the same diameter as the total stress Mohr circle.**

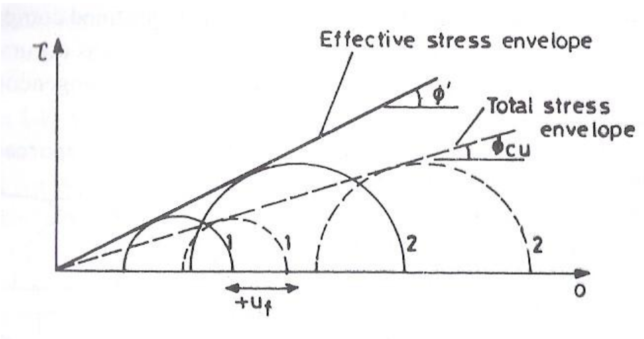
In the UU test, if pore water pressure is measured, the test is designated by $\overline{UU}$.

The shear strength equation may be written as

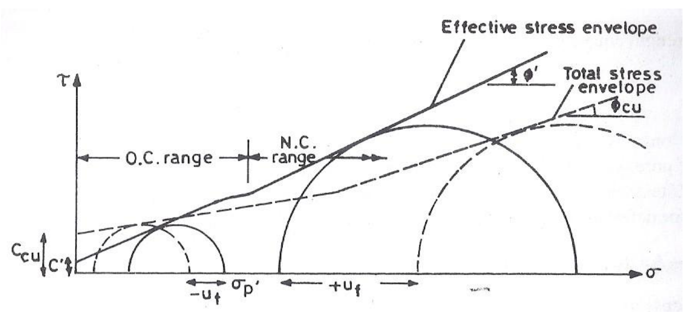
$$\tau_f = c_{uu} = \frac{\sigma_1 - \sigma_3}{2}$$

CU (CONSOLIDATED UNDRAINED) TEST

In this, drainage is allowed during cell pressure application or consolidation phase. At the end of the first stage there is no excess pore water pressure in the specimen. During the second phase or shearing phase, without allowing further drainage, deviator stress is increased keeping cell pressure constant. It is also called as an R-test. The pore pressure is measured during the deviator stress application. When the specimen is sheared under undrained condition, the excess pore water that develops can be positive or negative.



In normally consolidated clays there is a tendency towards volume decrease under undrained conditions during shear (that is, for pore water to escape). If this volume change is not allowed to occur, positive pore water pressures develop. Consequently, effective stress Mohr circles lie to the left of total stress circles (i.e. effective stress < total stress). **Both effective stress and total stress failure envelopes for the normally consolidated clays are straight lines that pass through the origin** i.e. c_{cu} =c' = 0. Also, $\varphi_{cu} \cong 1/2\ \varphi'$.



In overconsolidated clays there is tendency towards volume increase under undrained conditions during shear. If this volume increase is no allowed to occur, negative pore water pressures develop. Consequently, effective stress Mohr circles lie to the right of total stress circles (i.e. effective stress > total stress). **The failure envelope for an overconsolidated clay is not a straight line but a curve.** An overconsolidated clay shows a cohesion intercept c_{cu} in terms of total stresses and c' in terms of effective stresses. **c_{cu} is always greater than c'.** φ_{cu} can be slightly greater than or smaller than φ'.

For tests involving drainage in the first stage, when Mohr circles are plotted in terms of total stresses, the diameter increases with the confining pressure.

In the CU test, if pore water pressure is measured in the second stage, the test is symbolized as $\overline{CU}$.

The shear strength equation may be written as

(a) For normally consolidated clay

In terms of total stresses, $\tau_f = \sigma \tan \varphi_{cu}$

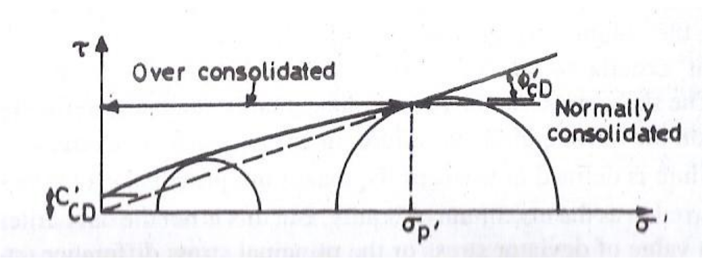
In terms of effective stresses, $\tau_f = \sigma' \tan \varphi'_{cu}$

(b) For over consolidated clay

In terms of total stresses, $\tau_f = c_{cu} + \sigma \tan \varphi_{cu}$

In terms of effective stresses, $\tau_f = c'_{cu} + \sigma' \tan \varphi'_{cu}$

CD (CONSOLIDATED DRAINED) TEST



This is similar to **CU test** except that as deviator stress is increased, drainage is permitted. The drainage valve is kept open in both the phases of the test. During the shearing phase, the rate of loading must be slow enough to ensure no excess pore water pressure develops. Therefore, **at any stage of the test, the total stresses are the effective stresses**. Thus, this is an effective stress test. It is also called an S- or Slow test.

For the normally consolidated clay, the failure envelope is a straight line that passes through the origin. For the overconsolidated clays, the failure envelope is curved and gives an a value of effective cohesion.

(a) For normally consolidated clay $\tau_f = \sigma' \tan \varphi'_{CD}, \quad c'_{CD} = 0$

(b) For over consolidated clay $\tau_f = c'_{CD} + \sigma' \tan \varphi'_{CD}$

Significance of Triaxial Testing

The first stage simulates in the laboratory the in-situ condition that soil at different depths is subjected to different effective stresses. Consolidation will occur if the pore water pressure which develops upon application of confining pressure is allowed to dissipate. Otherwise the effective stress on the soil is the confining pressure (or total stress) minus the pore water pressure which exists in the soil.

During the shearing process, the soil sample experiences axial strain, and either volume change or development of pore water pressure occurs. The magnitude of shear stress acting on different planes in the soil sample is different. When at some strain the sample fails, this limiting shear stress on the failure plane is called the shear strength.

ADVANTAGES AND DISADVANTAGES OF TRIAXIAL TEST
Advantages

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

Disadvantages

- The apparatus is elaborate, costly and bulky.
- The drained test takes a longer period as compared with that in a direct shear test.
- The strain condition in the specimen is not uniform due to frictional restraint produced by the loading cap and the pedestal disc.
- It is not possible to determine the cross-sectional area of the specimen accurately at large strains, as the assumption that the specimen remains cylindrical does not hold good.
- The test simulates only axis-symmetrical problems. In the field, the problem is generally 3-demnsional.
- The consolidation of the specimen is isotropic; whereas in the field its ia anisotropic.

The triaxial test has many **advantages** over the direct shear test:

- The soil samples are subjected to uniform stresses and strains.
- Different combinations of confining and axial stresses can be applied.
- Drained and undrained tests can be carried out.
- Pore water pressures can be measured in undrained tests.
- The complete stress-strain behaviour can be determined.

UNCONFINED COMPRESSION TEST

This test is applicable only to fully saturated, non-fissured clays. This is a special case of triaxial test in which the confining pressure (σ_3) is zero. It is not performed in a triaxial cell. A cylindrical soil specimen, usually of the same standard size as that for triaxial compression, is loaded axially until failure takes place. **Since the specimen is laterally unconfined, the test is known as unconfined compression test.** No latex membrane is necessary to encase the specimen. The axial or vertical compressive stress is the major principal stress and the other two principal stresses are zero.

Since the test is quick, water is not allowed to drain out of the sample. Hence, this is a total stress test. **The test produces only one Mohr circle which is tangential to the τ-axis.**

The relationship between principal stresses at failure is given by

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

As $\sigma_{3f} = 0$,

$$\sigma_{1f} = 2c_u\sqrt{\frac{1+\sin\varphi}{1-\sin\varphi}}$$

c_u indicates that the test is undrained.
As saturated clays loaded under undrained conditions fail under $\varphi = 0$ condition,

q_u

$$\sigma_{1f} = 2c_u$$

σ_{1f} is called the unconfined compressive strength and is denoted by q_u.

$$\therefore\ q_u = 2c_u$$

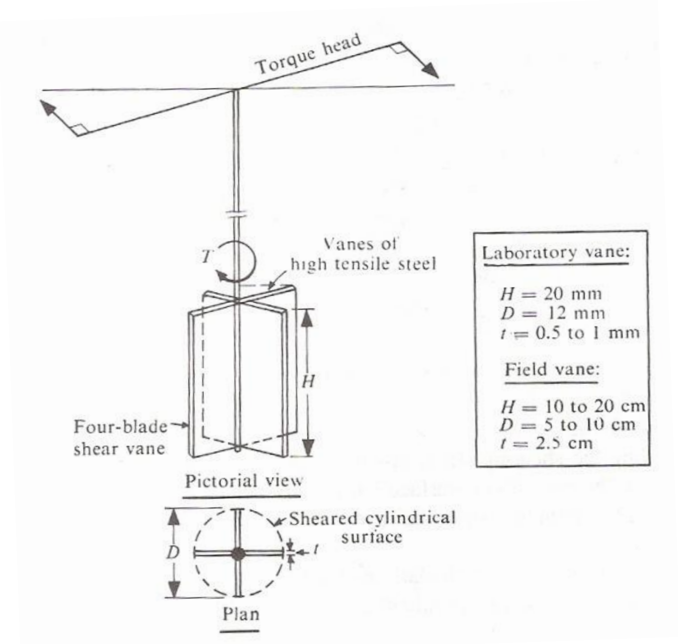
The undrained shear strength of saturated clay is

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

VANE SHEAR TEST

It is difficult to obtain undisturbed samples of soft saturated clay deposits. The shear strength of such sensitive clays may be significantly altered during the process of sampling and handling. The in-situ shear strength of such soils may be determined using vane shear or shear vane.

The shear vane consists of four steel plates welded orthogonally to a steel rod. The shear vane is gently pushed into the soil up to the required depth or at the bottom of the bore hole. Torque is then applied gradually to the upper end of the torque rod until the soil fails in shear, due to rotation of the vane. The torque is measured by noting the angle of twist.



Shear failure occurs over the surface and the ends of a cylinder having a diameter *D*, equal to the diameter of the vane.

The total shearing resistance of the soil at failure is

$$= \pi D H c_u + 2\int_0^{D/2} (2\pi r\, dr) c_u$$

where c_u is the unit undrained shearing resistance and *r* is the radius of the sheared surface.

The moment of the total shearing resistance about the centre is the torque *T* at failure

$$T = \pi D H c_u . \frac{D}{2} + 2\int_0^{D/2} (2\pi r\, dr) c_u . r$$

$$T = c_u \pi \left[\frac{D^2 H}{2} + \frac{D^3}{6}\right]$$

$$c_u = \frac{T}{\pi D^2 \left[\frac{H}{2} + \frac{D}{6}\right]}$$

If the test is carried out such that the top end of the vane does not shear the soil (as in the case of a test in a borehole)

$$T = c_u \pi \left[\frac{D^2 H}{2} + \frac{D^3}{12}\right]$$

If after the initial test, the vane is rotated rapidly several times, the soil becomes remoulded and the shear strength of the remoulded clay can be calculated, and thus the sensitivity of the clay can be calculated.

$$Sensitivity\ (S_t) = \frac{Shear\ strength\ of\ undisturbed\ soil}{Shear\ strength\ of\ remoulded\ soil}$$

SKEMPTON'S PORE PRESSURE PARAMETERS

Pore pressure parameters, first introduced by Skempton, are empirical coefficients which are used to express the response of pore pressure to changes in total stress under *undrained conditions*. They are dimensionless and indicate the part of total stress that manifests as excess pore water pressure for no drainage condition.

In one-dimensional loading situation, the excess pore water pressure that is induced initially is equal to the applied vertical stress. But **in the case of triaxial test when the deviator stress is applied and shear stresses develop, the pore water pressure that is induced will not be equal to the applied deviator stress.** Its magnitude will depend on the type of soil, and its stress history. It can be positive or negative.

Skempton's equation may be written in the following form.

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

where, $\Delta u =$ increase in pore water pressure when no drainage is permitted
$A, B =$ Skempton's pore pressure parameters

The above equation can be split into two parts as

$$\Delta u_1 = B\Delta\sigma_3$$

$$\Delta u_2 = BA(\Delta\sigma_1 - \Delta\sigma_3)$$

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

Δu_1 is the change in pore pressure due to an increase in cell pressure $\Delta\sigma_3$

Δu_2 is the change in pore pressure due to an increase in deviator stress $(\Delta\sigma_1 - \Delta\sigma_3)$.
B varies from 0 to 1 depending on the degree of saturation (S). For S = 100 %, B = 1. The relation between S and B is not linear.

For B =1,

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

For a partially saturated sample,

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

where $\bar{A} = AB$

For a completely saturated sample, $\bar{A} = A$

Parameter B varies with the stress range. Hence, while evaluating A from $\bar{A}$, the value of B corresponding to appropriate deviator stress range must be used.

Parameter B can be determined from a $\overline{UU}$ test (UU test in which pore pressure is measured). The cell pressure is increased by $\Delta\sigma_3$ and the corresponding increase in pore pressure Δu_1 is measured in the first stage of the triaxial test.

$$B = \frac{\Delta u_1}{\Delta \sigma_3}$$

Parameter $\bar{A}$ is measured during the second stage of the triaxial test. If Δu_2 is the pore pressure increase due to an increase in deviator stress of $(\Delta\sigma_1 - \Delta\sigma_3)$ with cell pressure being constant,

$$\bar{A} = \frac{\Delta u_2}{(\Delta\sigma_1 - \Delta\sigma_3)}$$

For a fully saturated soil, A can be determined from a $\overline{CU}$ test as

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

Like parameter B, parameter A is also not a constant and is different for different soils and stress conditions.

USE OF SKEMPTON'S PARAMETERS

Skempton's pore pressure parameters are very useful in field problems where pore pressures that are induced consequent to change in total stress may have to be computed. During the construction of an earth embankment over a soft clay deposit, if the rate of construction is such that pore water pressure in the foundation soil cannot dissipate, undrained loading condition will prevail. Computation of pore pressure using Skempton's parameters will estimate the chances of endangering the foundation soil due to excessive pore pressures.

STRESS-STRAIN BEHAVIOUR OF SANDS

Sands are usually sheared under drained conditions as they have relatively higher permeability. This behaviour can be investigated in direct shear or triaxial tests. The two most important parameters governing their behaviour are the **relative density (I_D)** and the magnitude of the **effective stress (σ′).** The relative density is usually defined in percentage as

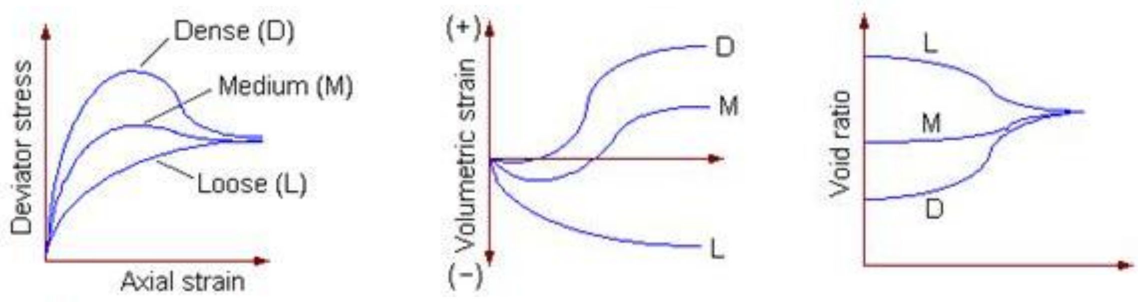
$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} x100$$

where $\boldsymbol{e_{max}}$ and $\boldsymbol{e_{min}}$ are the maximum and minimum void ratios that can be determined from standard tests in the laboratory, and ***e*** is the current void ratio. This expression can be re-written in terms of dry density as

$$I_D = \left(\frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}} \right) x \frac{\gamma_{d\,max}}{\gamma_d} x100$$

where γ_{dmax} and γ_{dmin} are the maximum and minimum dry densities, and γ_d is the current dry density. Sand is generally referred to as dense if I_D > 65% and loose if < 35%.

The influence of relative density on the behaviour of saturated sand can be seen from the plots of CD tests performed at the **same effective confining stress.** There would be no induced pore water pressures existing in the samples.



For the dense sand sample, the deviator stress reaches a peak at a low value of axial strain and then drops down, whereas for the loose sand sample, the deviator stress builds up gradually with axial strain. The behaviour of the medium sample is in between. The following observations can be made:

- All samples approach the same ultimate conditions of shear stress and void ratio, irrespective of the initial density. The denser sample attains higher peak angle of shearing resistance in between.
- Initially dense samples expand or dilate when sheared, and initially loose samples compress.