SHEAR STRENGTH OF SOIL

Necessity of studying Shear Strength of soils :

• Soil failure usually occurs in the form of "shearing" along internal surface within the soil.

Shear Strength:

• Thus, structural strength is primarily a function of shear strength.

• The strength of a material is the greatest stress it can sustain

• The safety of any geotechnical structure is dependent on the strength of the soil

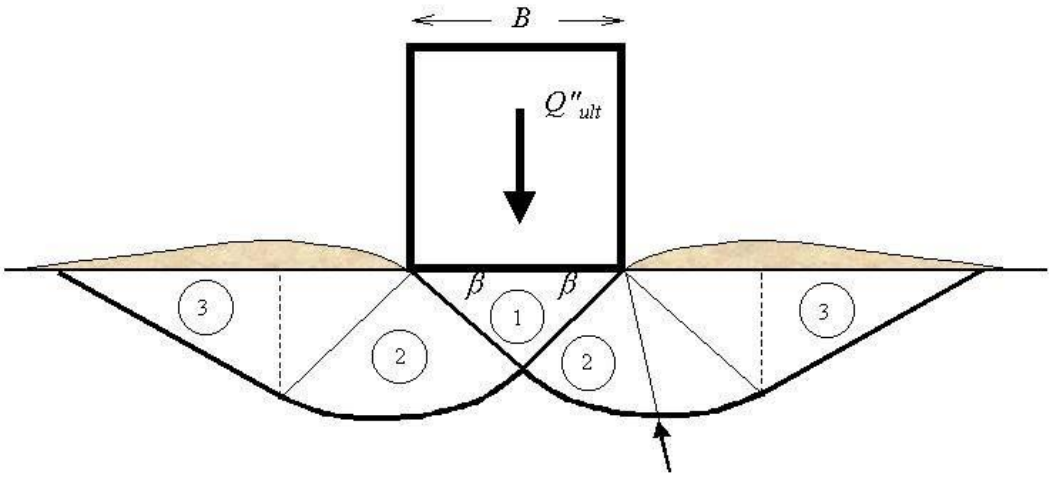
• If the soil fails, the structure founded on it can collapse

Thus shear strength is

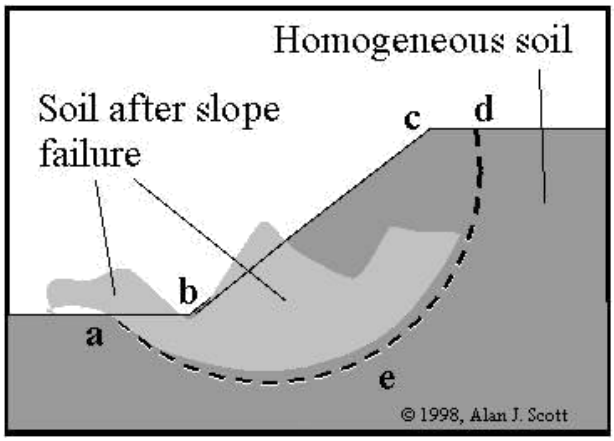
"The capacity of a material to resist the internal and external forces which slide past each other"

Significance of Shear Strength :

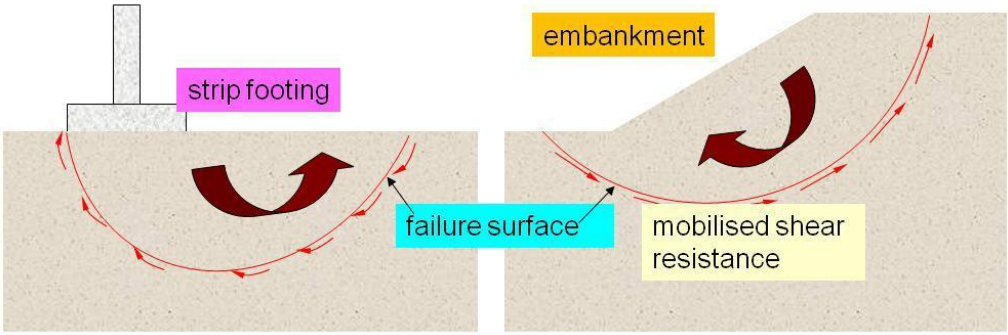
- Engineers must understand the nature of shearing resistance in order to analyze soil stability problems such as;
- Bearing capacity
- Slope stability
- Lateral earth pressure on earth-retaining structure



Shear Failure under Foundation Load



Slope Stability Failure as an Example of Shearing Along Internal Surface



At failure, shear stress along the failure surface reaches the shear

Thus shear strength of soil is

"The capacity of a soil to resist the internal and external forces which slide past each other"

Shear Strength in Soils :

- The shear strength of a soil is its resistance to shearing stresses.
- It is a measure of the soil resistance to deformation by continuous displacement of its individual soil particles.
- Shear strength in soils depends primarily on interactions between particles.

-Shear failure occurs when the stresses between the particles are such that they slide or roll past each other

Components of shear strength of soils

Soil derives its shear strength from two sources:

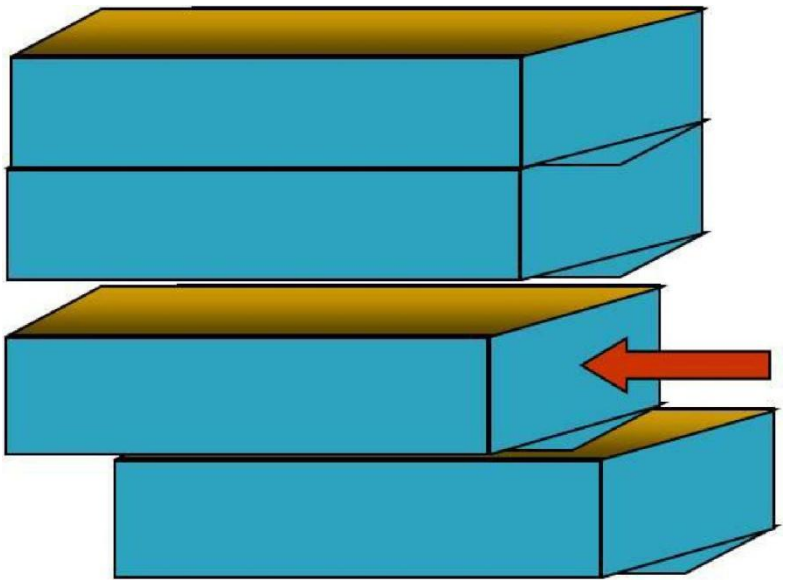
– Cohesion between particles (stress independent component)

- Cementation between sand grains
- Electrostatic attraction between clay particles

– Frictional resistance and interlocking between particles (stress dependent component)

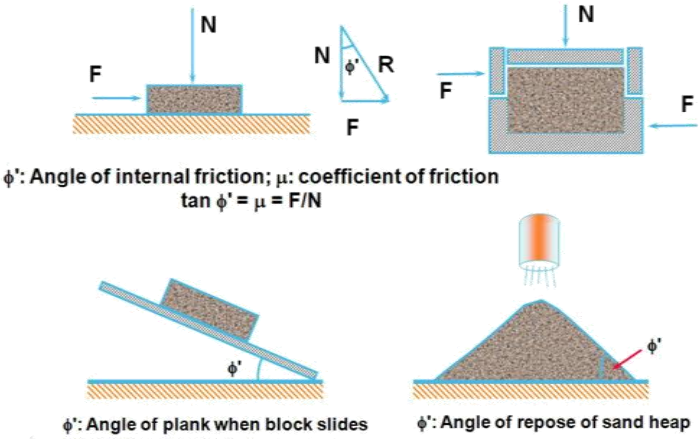
Cohesion :

Cohesion (C), is a measure of the forces that cement particles of soils

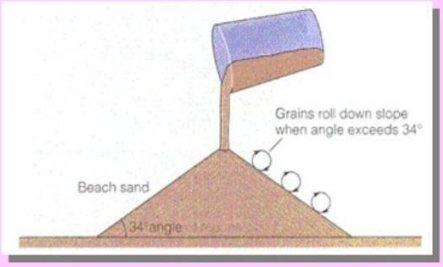


Internal Friction :

Internal Friction angle (*f*), is the measure of the shear strength of soils due to friction



- **The maximum slope at which loose, cohesionless material is stable**



Angle of Repose

Angle of Repose determined by:

Particle size (higher for large particles)

Particle shape (higher for angular shapes)

Shear strength (higher for higher shear strength)

Stresses:

Gravity generates stresses (force per unit area) in the ground at different points.

Stress on a plane at a given point is viewed in terms of two components:

Normal stress (σ) : acts normal to the plane and tends to compress soil grains towards each other (volume change)

Shear stress (τ): acts tangential to the plane and tends to slide grains relative to each other (distortion and ultimately sliding failure).

LECTURE 31

Factors Influencing Shear Strength:

The shearing strength, is affected by:

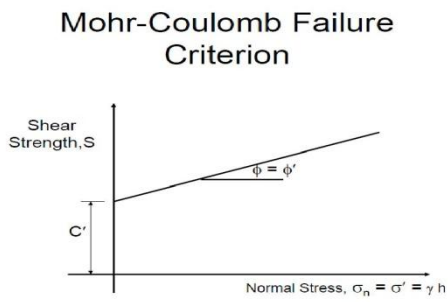
– *soil composition*: mineralogy, grain size and grain size distribution, shape of particles, pore fluid type and content, ions on grain and in pore fluid.

– *Initial state*: State can be describe by terms such as: loose, dense, overconsolidated, normally consolidated, stiff, soft, etc.

– *Structure*: Refers to the arrangement of particles within the soil mass; the manner in which the particles are packed or distributed. Features such as layers, voids, pockets, cementation, etc, are part of the structure.

Mohr-Coulomb Failure Criteria:

This theory states that a material fails because of a critical combination of normal stress and shear stress, and not from their either maximum normal or shear stress alone.



$$\tau_f = c + \sigma_n \tan\phi = c + \mu\sigma_n$$

$$\tau_f = c' + \sigma_n' \tan\phi' = c' + \mu'\sigma_n'$$

where

τ_f = shear strength

c = cohesion; c′ =effective cohesion

φ = angle of internal friction; φ′ = effective angle of internal friction

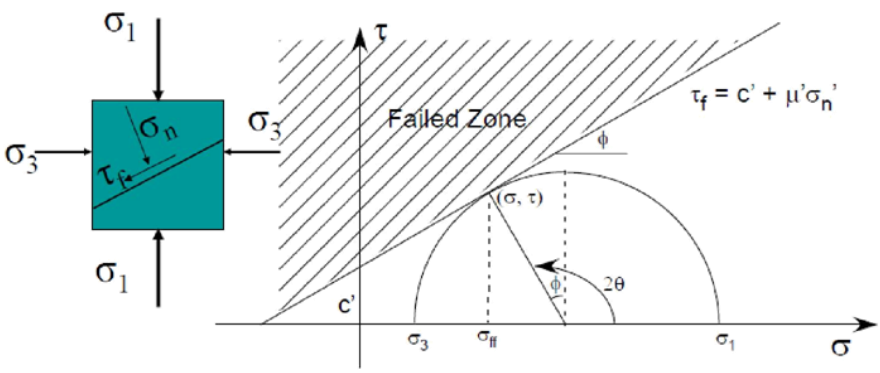
μ = coefficient of friction; μ' = effective coefficient of friction.

Thus, Eqs. (11.2) and (11.3) are expressions of shear strength based on tota stress and effective stress. The value of c' for sand and inorganic silt is 0. For nor mally consolidated clays, c' can be approximated at 0. Overconsolidated clays hav values of c' that are greater than 0. The angle of friction, ϕ', is sometimes referred t as the *drained angle of friction.* Typical values of ϕ' for some granular soils are give

Table Typical Values of Drained Angle of Friction for Sands and Silts

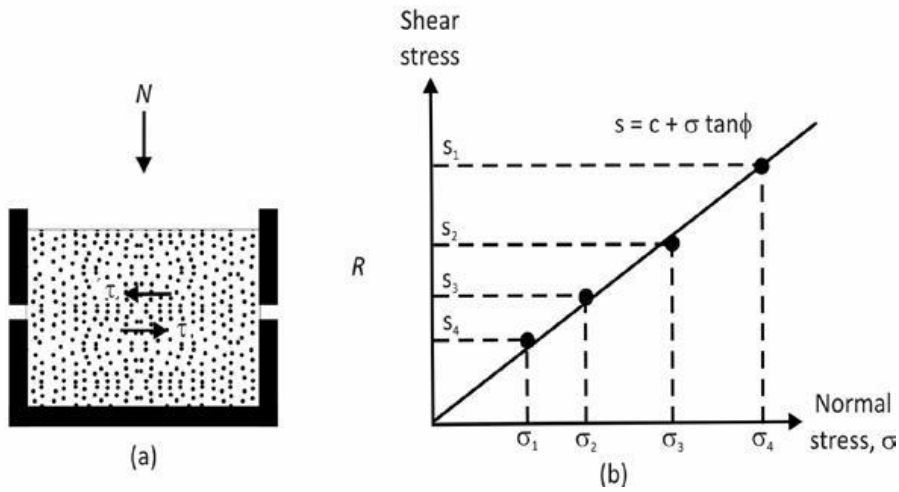
Soil type	ϕ' (deg)	$\mu = \tan\phi'$
Sand: Rounded grains		
Loose	27–30	0.51-0.58
Medium	30–35	0.58-0.70
Dense	35–38	0.70-0.78
Sand: Angular grains		
Loose	30–35	0.58-0.70
Medium	35–40	0.70-0.84
Dense	40–45	0.84-1.00
Gravel with some sand	34–48	0.67-1.11
Silts	26–35	0.49-0.70

Mohr-Coulomb shear failure criterion



Direct Shear Test:

Dry sand can be conveniently tested by direct shear tests. The sand is placed in a shear box that is split into two halves . A normal load is first applied to the specimen. Then a shear force is applied to the top half of the shear box to cause failure in the sand. The normal and shear stresses at failure are



$$\sigma' = \frac{N}{A}$$

$$S = \frac{R}{A}$$

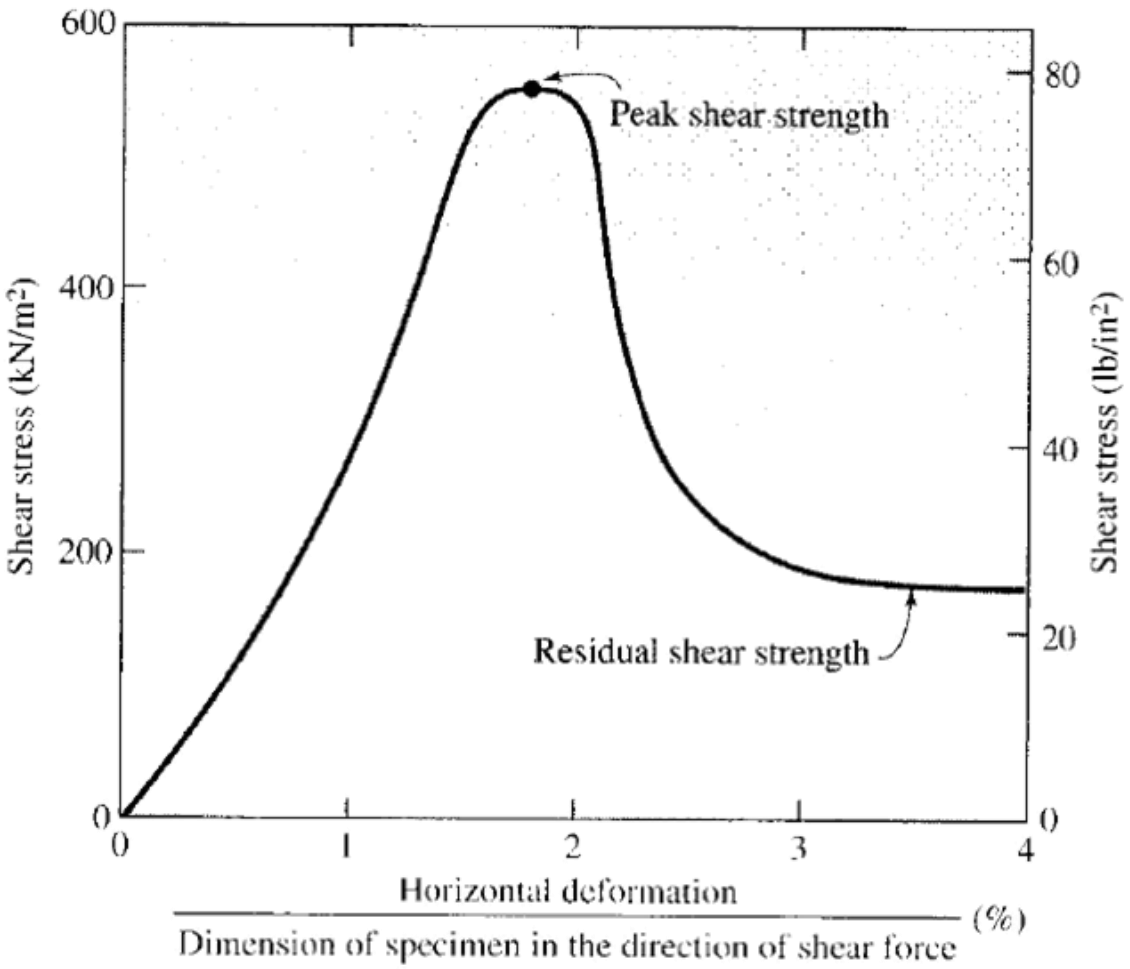
Where

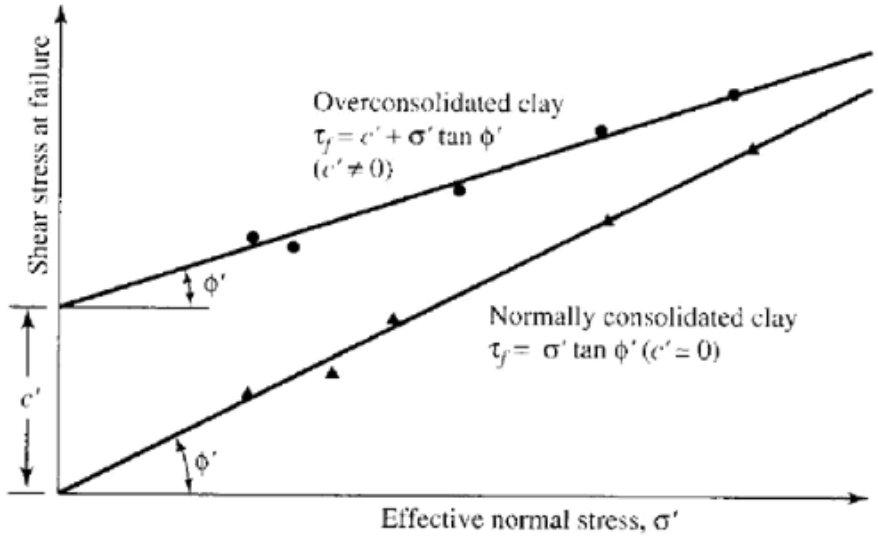
A = Area of the failure plane in soil-that is, the area of cross section of the shear box

Several tests of this type can be conducted by varying the normal load. The angle of friction of the sand can be determined by plotting a graph of s against σ' $(= \sigma)$

$$\phi = \tan^{-1}\left(\frac{s}{\sigma'}\right) \qquad [1.83]$$

For sands, the angle of friction usually ranges from 26° to 45°, increasing with the relative density of compaction. The approximate range of the relative density of compaction and the corresponding range of the angle of friction for various coarse-grained soils is shown in **figure**





Failure envelope for clay obtained from drained direct shear tests

LECTURE 33

Triaxial Tests

Triaxial compression tests can be conducted on sands and clays shows a schematic diagram of the Triaxial test arrangement. Essentially, it consists of placing a soil specimen confined by a rubber membrane in a Lucite chamber. An all-round confining pressure ($\sigma 3$) is applied to the specimen by means of the chamber fluid (generally water or glycerin). An added stress ($\Delta\sigma$) can also be applied to the specimen in the axial direction to cause failure ($\Delta\sigma=\Delta\sigma f$ at failure). Drainage from the specimen can be allowed or stopped, depending on the test condition. For clays, three main types of tests can be conducted with Triaxial equipment:

Triaxial test:

1. Consolidated-drained test (CD test)
2. Consolidated-undrianed test (CU test)
3. Unconsolidated-undrained test (UU test) Major

Principal effective stress $=\sigma 3=\Delta\sigma f=\sigma 1=\sigma'1$ Minor Principal

effective stress $=\sigma 3=\Delta\sigma'3$

Changing $\sigma 3$ allows several tests of this type to be conducted on various clay specimens. The shear strength parameters (c and φ) can now be determined by plotting Mohr's circle at failure, as shown in figure and drawing a common tangent to the Mohr's circles. This is the Mohr-Coulomb failure envelope. (Note: For normally consolidated clay, $c\approx 0$). At failure

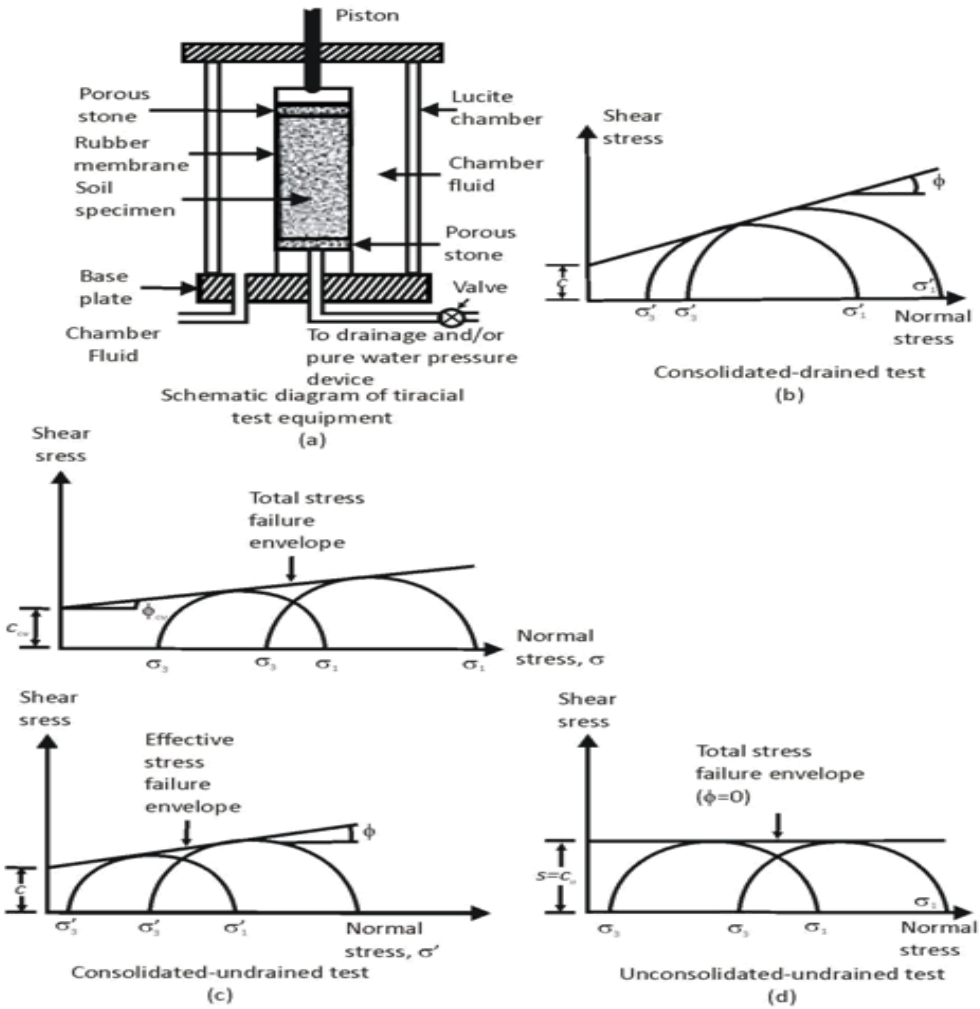
$$\sigma'_1 = \sigma'_3 \tan^2\left(45+\frac{\Phi}{2}\right) + 2c\tan\left(45+\frac{\Phi}{2}\right)$$

For consolidated-undrained tests, at failure,

Major Principal total stress $=\sigma 3=\Delta\sigma f=\sigma 1$

Minor principal total stress $=\sigma 3$

Major principal effective stress $=(\sigma 3+\Delta\sigma f)-uf=\sigma'1$



Minor principal effective stress =σ3− uf=σ'3

Changing σ3 permits multiple tests of this type to be conducted on several soil specimens. The total stress Mohr's circles at failure can now be plotted, as shown in figure , and then a common tangent can be drawn to define the failure envelope. This total stress failure envelope is defined by the equation

s=ccu+σtanφcu

Where ccu and φcu are the consolidated-undrained cohesion and angle of friction respectively (Note: ccu≈0 for normally consolidated clays)

Similarly, effective stress Mohr's circles at failure can be drawn to determine the effective stress failure envelopes.

They follow the relation expressed in equation .

For unconsolidated-undrained triaxial tests

Major principal total stress=σ3=Δσf=σ1

Minor principal total stress =σ3

The total stress Mohr's circle at failure can now be drawn, as shown in figure. For saturated clays, the value of $\sigma_1-\sigma_3=\Delta\sigma_f$ is a constant, irrespective of the chamber confining pressure, σ_3.The tangent to these Mohr's circles will be a horizontal line, called the $\varphi=0$ condition. The shear stress for this condition is

$$s = c_u = \frac{\Delta\sigma_f}{2}$$

Where

c_u = undrained cohesion (or undrained shear strength)

The pore pressure developed in the soil specimen during the unconsolidated-undrained triaxial test is

$$u = u_a + u_d$$

The pore pressure u_a is the contribution of the hydrostatic chamber pressure, σ_3. Hence

$$u_a = B\sigma_3$$

Where

B=Skempton's pore pressure parameter

Similarly,the pore pressure ud is the result of added axial stress, $\Delta\sigma$, so

$u_d=A\,\Delta\sigma$

Where

A=Skempton's pore pressure parameter

However,

$\Delta\sigma=\sigma_1-\sigma_3$

Combining equations gives

$u=u_a+u_d=B\sigma_3+A\sigma_1-\sigma_3$

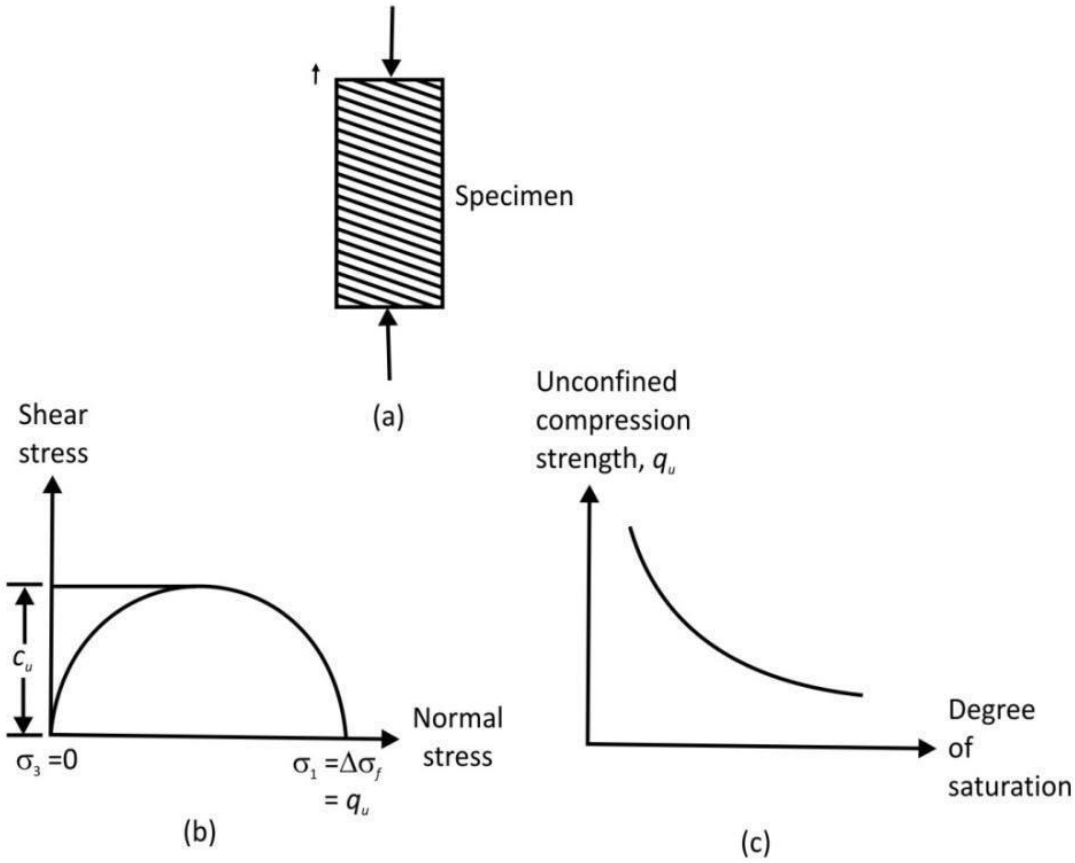
The pore water pressure parameter B in soft saturated soils is 1, so

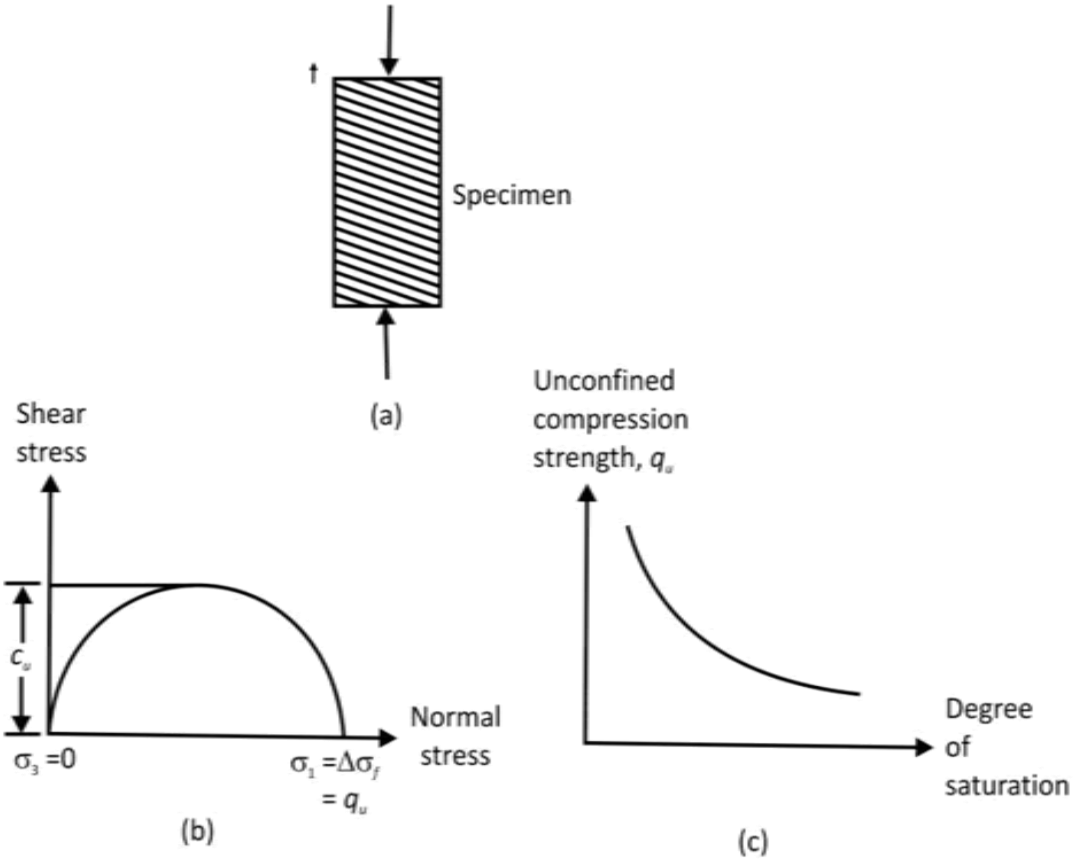
$u=\sigma_3+A(\sigma_1-\sigma_3)$

The value of the pore water pressure parameter A at failure will vary with the type of soil. Following is a general range of the values of A at failure for various types of clayey soil encountered in nature.

UNCONFINED COMPRESSION TEST

The unconfined compression test is a special type of unconsolidated-undrained Triaxial test in which the confining pressure σ3=0, as shown in figure. In this test an axial stress, Δσ, is applied to the specimen to cause failure (that is, Δσ=Δσf). The corresponding Mohr's circle is shown in figure . Note that, for this case, u





Unconfined compression test: (a) soil specimen; (b) Mohr's circle for the test; (c) variation of qu with the degree of saturation

Major principal total stress =Δσf=qu

Minor principal total stress = 0

The axial stress at failure, Δσf=qu is generally referred to as the unconfined compression strength. The shear strength of saturated clays under this condition (φ=0,

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

The unconfined compression strength can be used as an indicator for the consistency of clays. Unconfined compression tests are sometimes conducted on unsaturated soils. With the void ratio of a soil specimen remaining constant, the unconfined compression strength rapidly decreases with the degree of saturation shows an unconfined

Vane Shear Test:

Fairly reliable results for the undrained shear strength, c,, (S : 0 concept), of very soft to medium cohesive soils may be obtained directly from vane shear tests. The shear vane usually consists of four thin, equal-sized steel plates welded to a steel torque rod. First, the vane is pushed into the soil. Then torque is applied at the top of the torque rod to rotate the vane at a uniform speed. A cylinder of soil of height ft and diameter r/ will resist the torque until the soil fails. The undrained shear strength of the soil can be calculated as follows. If I is the maximum torque applied at the head of the torque rod to cause failure, it should be equal to the sum of the resisting moment of the shear force along the side surface of the soil cylinder (M.) and the resisting moment of the shear force at each end (M,,)

$$T = M_s + \underbrace{M_e + M_e}_{\text{Two ends}}$$

The resisting moment can be given as

$$M_s = \underbrace{(\pi d h) c_u}_{\text{Surface area}} \underbrace{(d/2)}_{\text{Moment arm}}$$

l. Triangular. Shear strength mobilization is c,, at the periphery of the soil cylinder and decreases lineaarly to zero at the center.

2, IJni.form.S hears trengthm obilization is constant (that is, c)f room the periphery to the center of the soil cylinder.

3. Parabolic. Shear strength mobilization is c,, at the periphery of the soil cylinder and dccreases parabolically to zero at the center.

These variations in shear strength mobilization are shown in Figure .In general, the torque, I at failure can be expressed as

$$T = \pi c_u \left[\frac{d^2 h}{2} + \beta \frac{d^3}{4} \right]$$

$$c_u = \frac{T}{\pi \left[\frac{d^2 h}{2} + \beta \frac{d^3}{4} \right]}$$

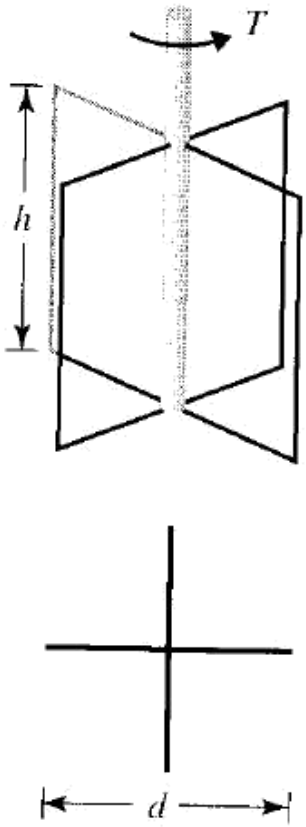
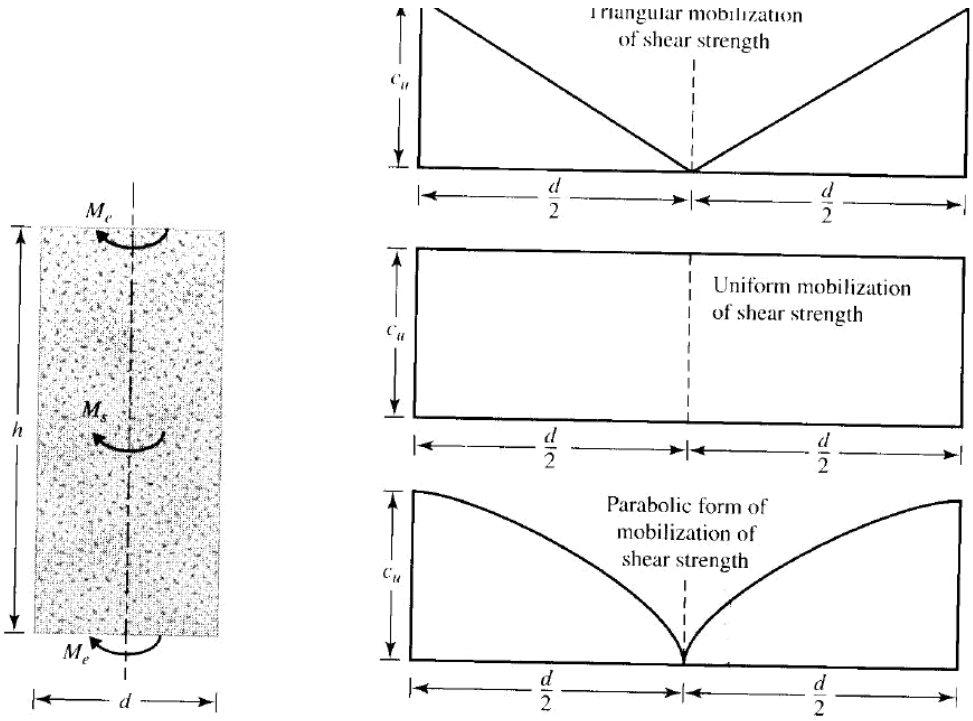


Diagram of vane shear test equipment



(a) resisting moment of shear force; (b) variations in shear strength mobilization

$$c_u\,(\mathrm{kN/m^2}) = \frac{T\,(\mathrm{N \cdot m})}{(366 \times 10^{-8})\underset{\substack{\uparrow \\ (\mathrm{cm})}}{d^3}}$$

$$c_u\,(\mathrm{lb/ft^2}) = \frac{T\,(\mathrm{lb \cdot ft})}{0.0021\underset{\substack{\uparrow \\ (\mathrm{in.})}}{d^3}}$$