LECTURE 30

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

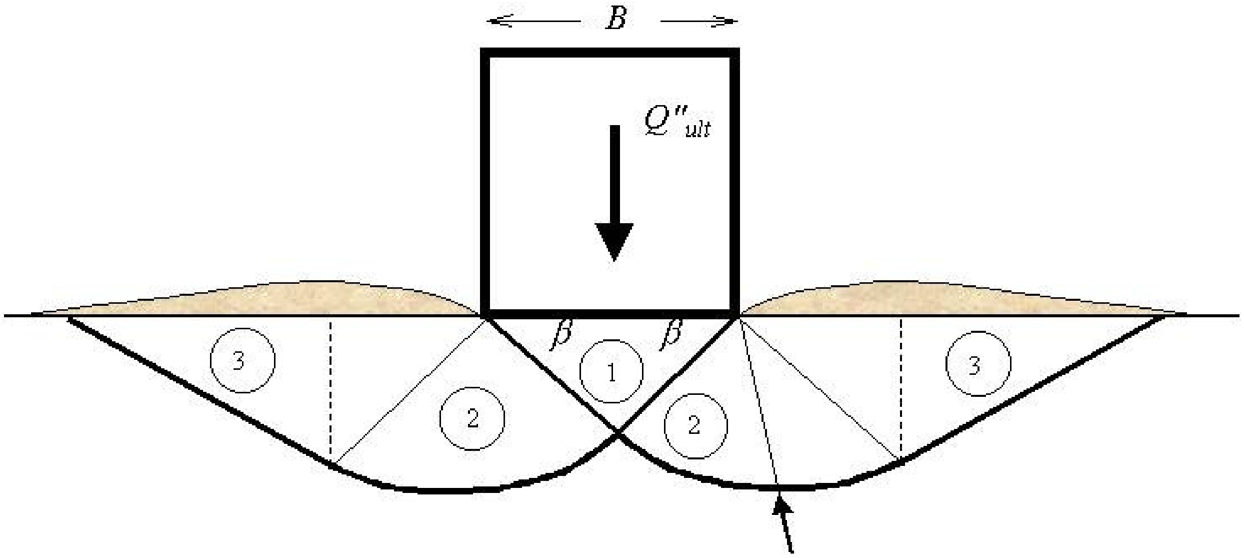
• If the soil fails, the structure founded on it can collapse

Thus shear strength is

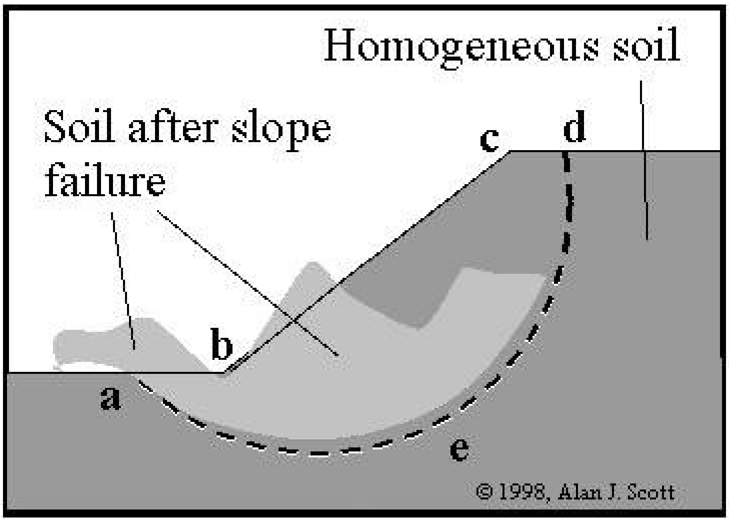
“The capacity of a material to resist the internal and external forces which slide past each other”

Significance of Shear Strength :

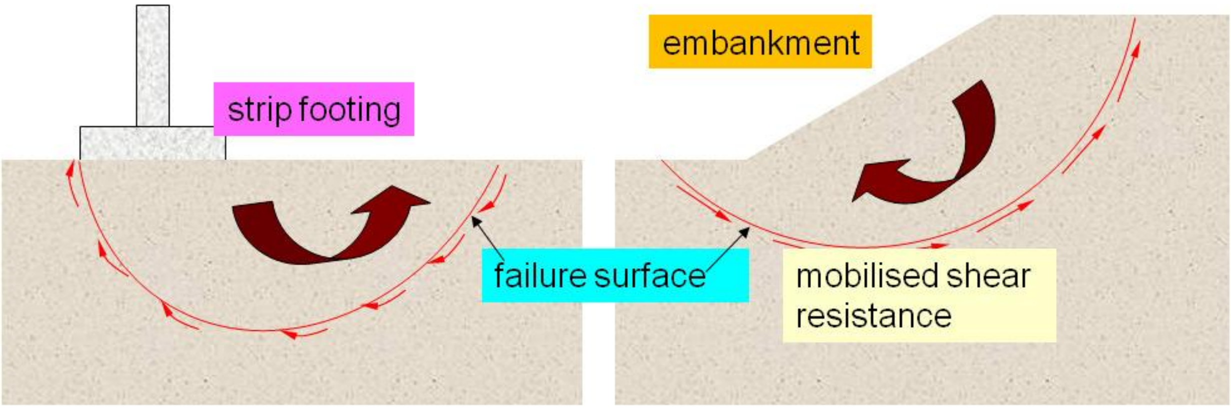
- Engineers must understand the nature of shearing resistance in order to analyze soil stability problems such as;
- Bearing capacity
- Slope stability
- Lateral earth pressure on earth-retaining structure



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)

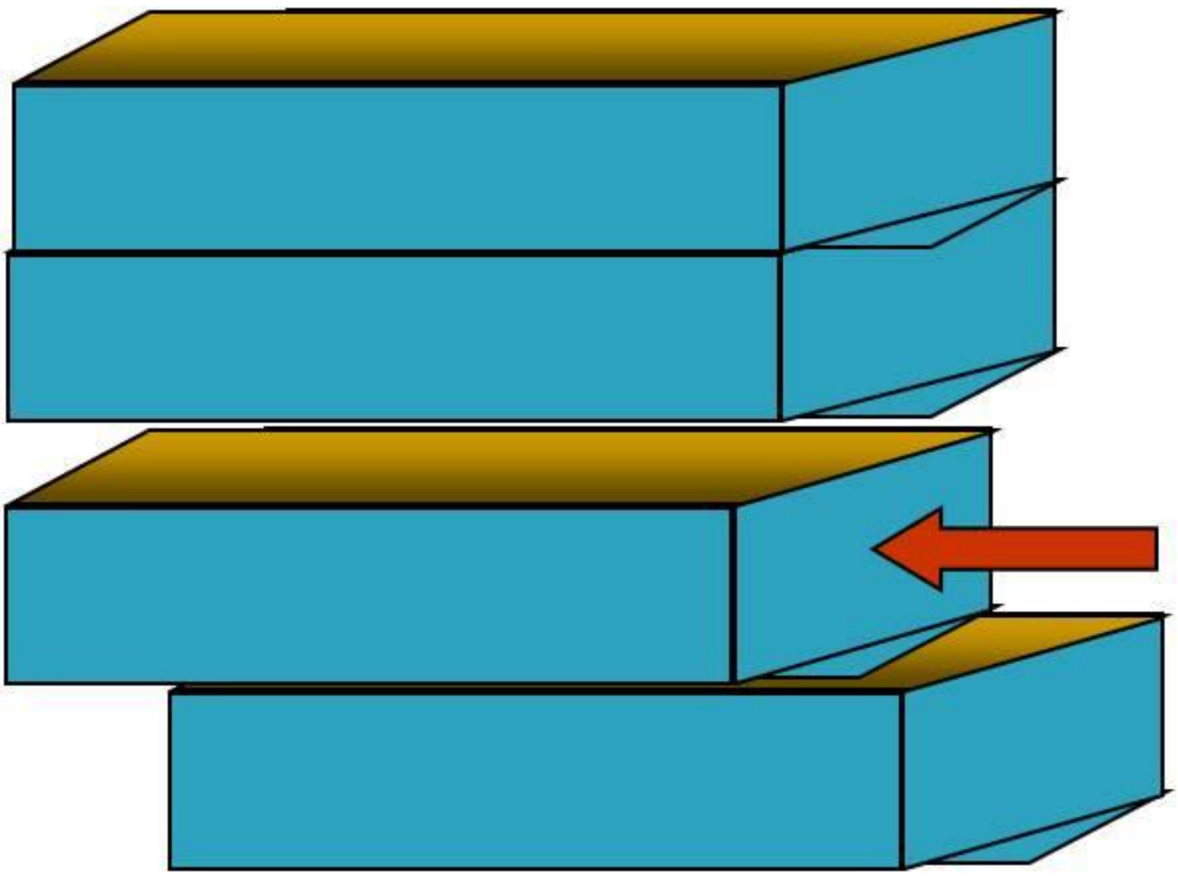
• Cementation between sand grains

• Electrostatic attraction between clay particles

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

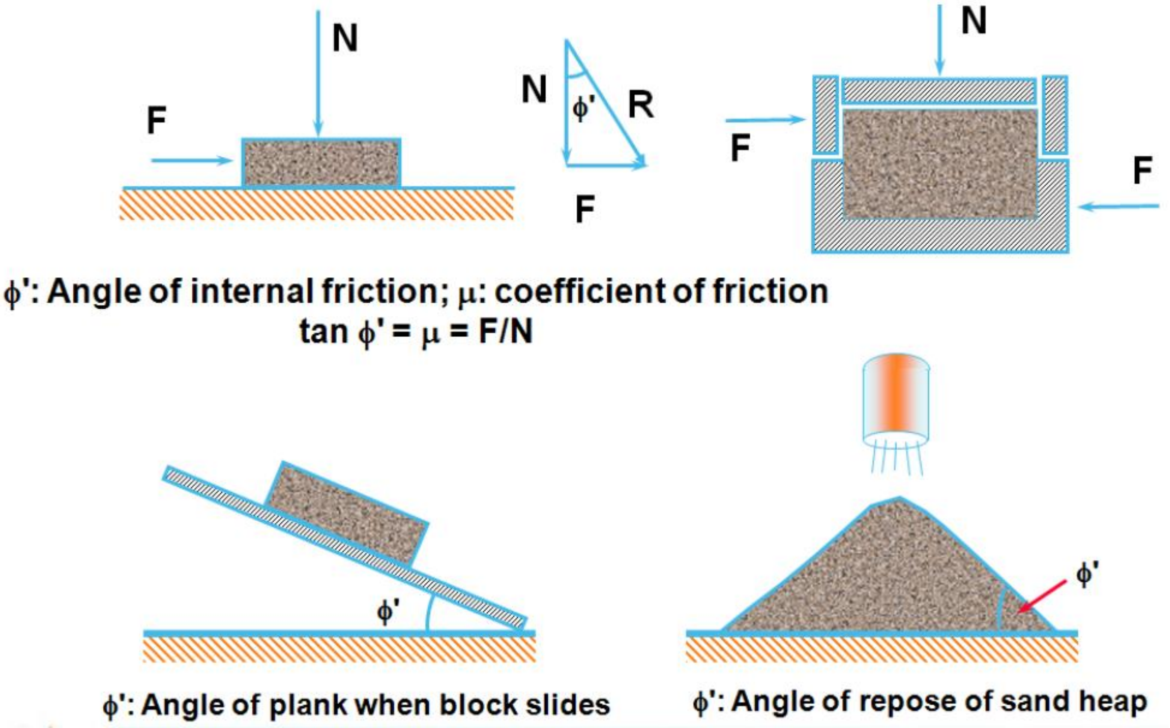
Cohesion :

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

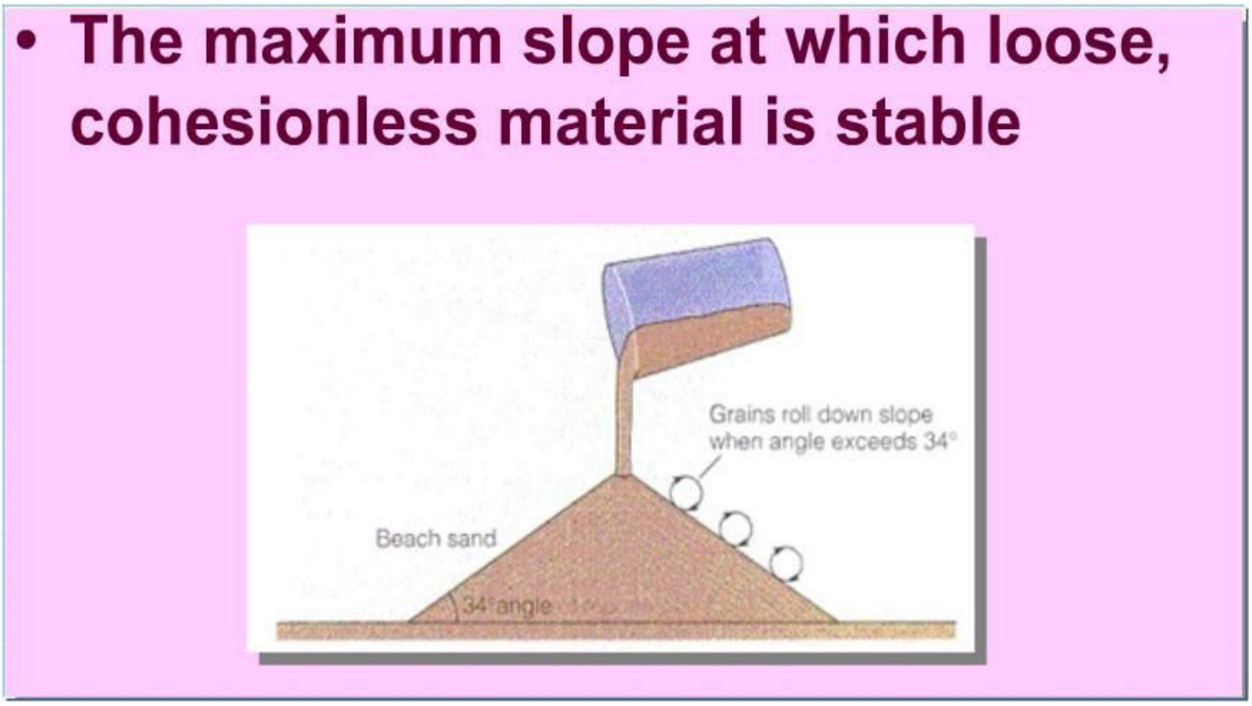


Internal Friction :

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



friction



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:

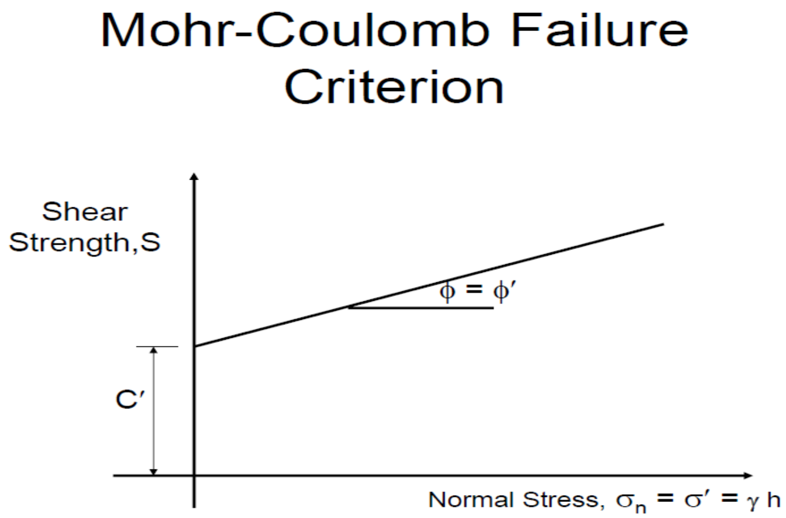
– *soil composition*: mineralogy, grain size and grain size distribution, shape of particles, pore fluid type and content, ions on grain and in pore fluid.

– *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.

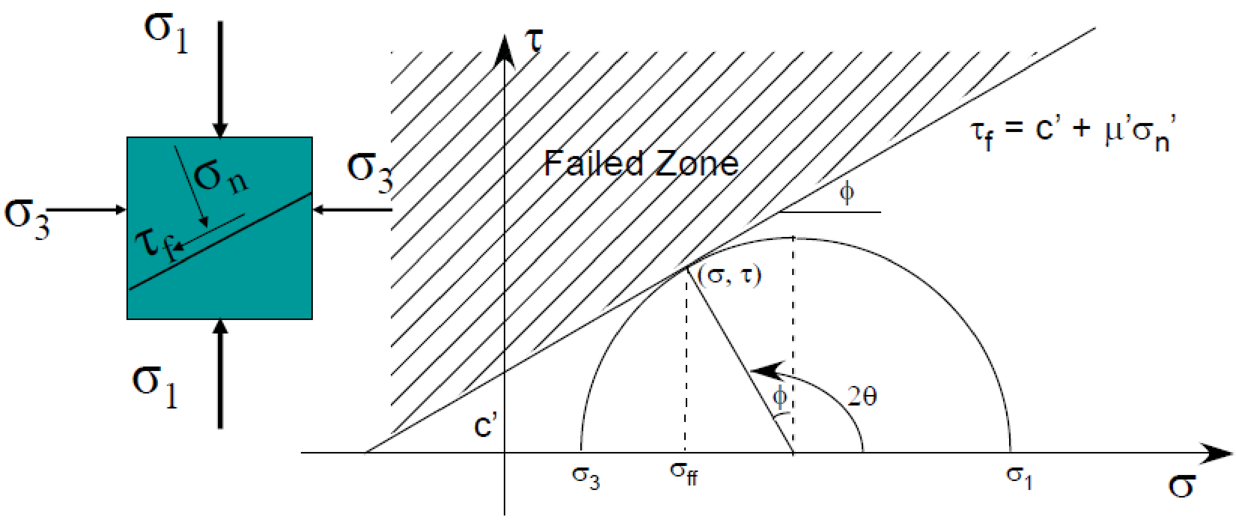
Thus, Eqs. (11.2) and (11.3) are expressions of shear strength based on tota stress and effective stress. The value of c' for sand and inorganic silt is 0. For nor mally consolidated clays, c' can be approximated at 0. Overconsolidated clays have values of c' that are greater than 0. The angle of friction, ϕ', is sometimes referred to as the *drained angle of friction*. Typical values of ϕ' for some granular soils are given in Table 11.1

Under Revision

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



$$2\theta = 90 + \phi', \text{ or}$$

$$\theta = 45 + \frac{\phi'}{2}$$

Again, from Figure 11.3,

$$\frac{\overline{ad}}{\overline{fa}} = \sin \phi'$$

$$\overline{fa} = fO + Oa = c' \cot \phi' + \frac{\sigma_1' + \sigma_3'}{2}$$

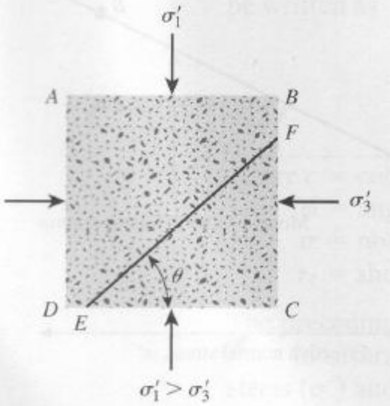


Figure Inclination of failure plane in soil with major principal plane

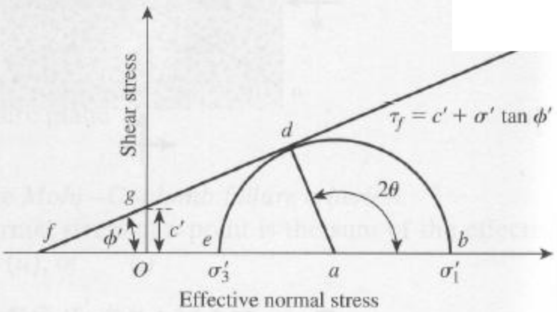


Figure Mohr's circle and failure envelope

Also,

$$\overline{ad} = \frac{\sigma_1' - \sigma_3'}{2}$$

Substituting Eqs. (11.6a) and (11.6b) into Eq. (11.5), we obtain

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

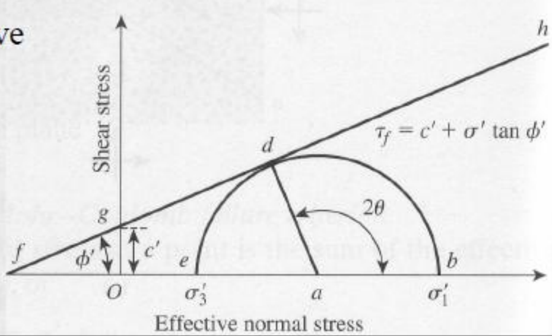
or

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

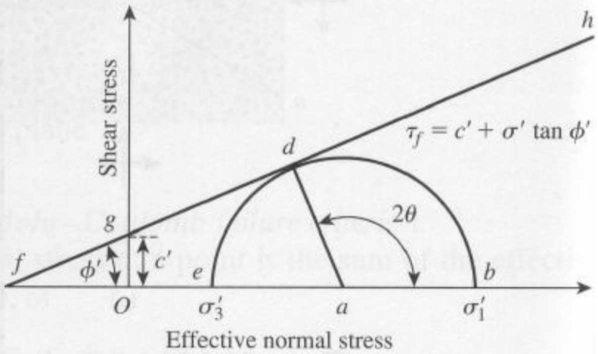
However, From trigonometric equalities we have

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

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



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



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

Thus,

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

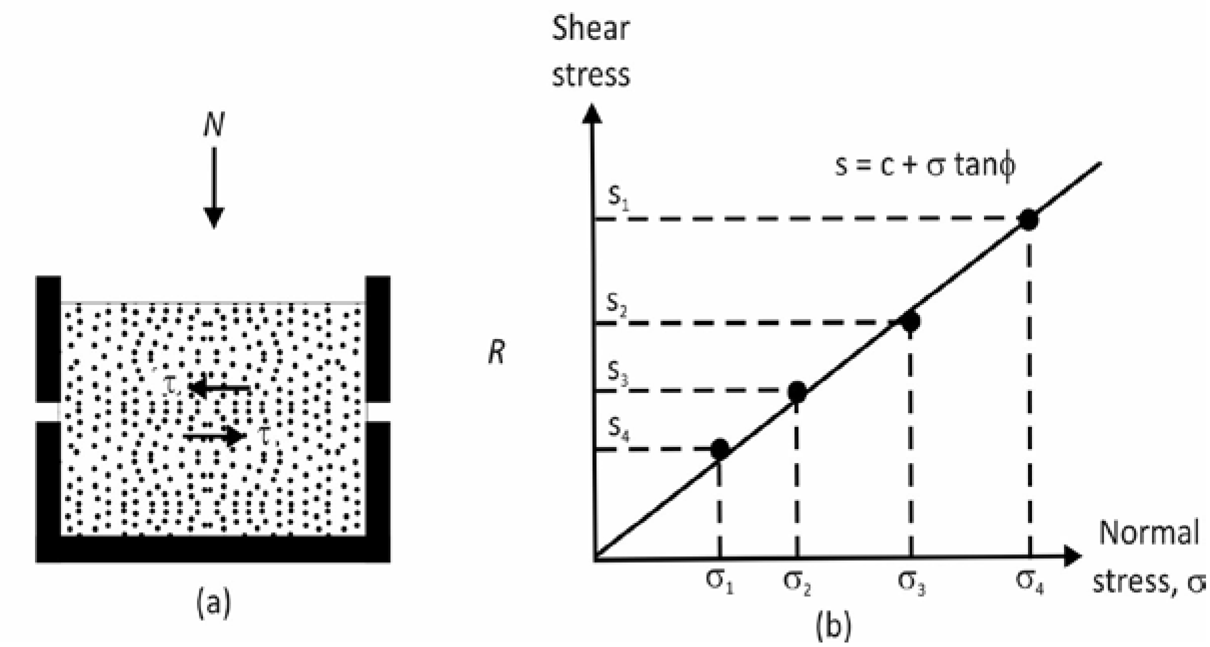
An expression similar to Eq. (11.8) could also be derived using Eq. (that is, total stress parameters c and ϕ), or

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

LECTURE 32

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



Direct shear test in sand: (a) schematic diagram of test equipment; (b) plot of test results to obtain the friction angle, ϕ

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

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

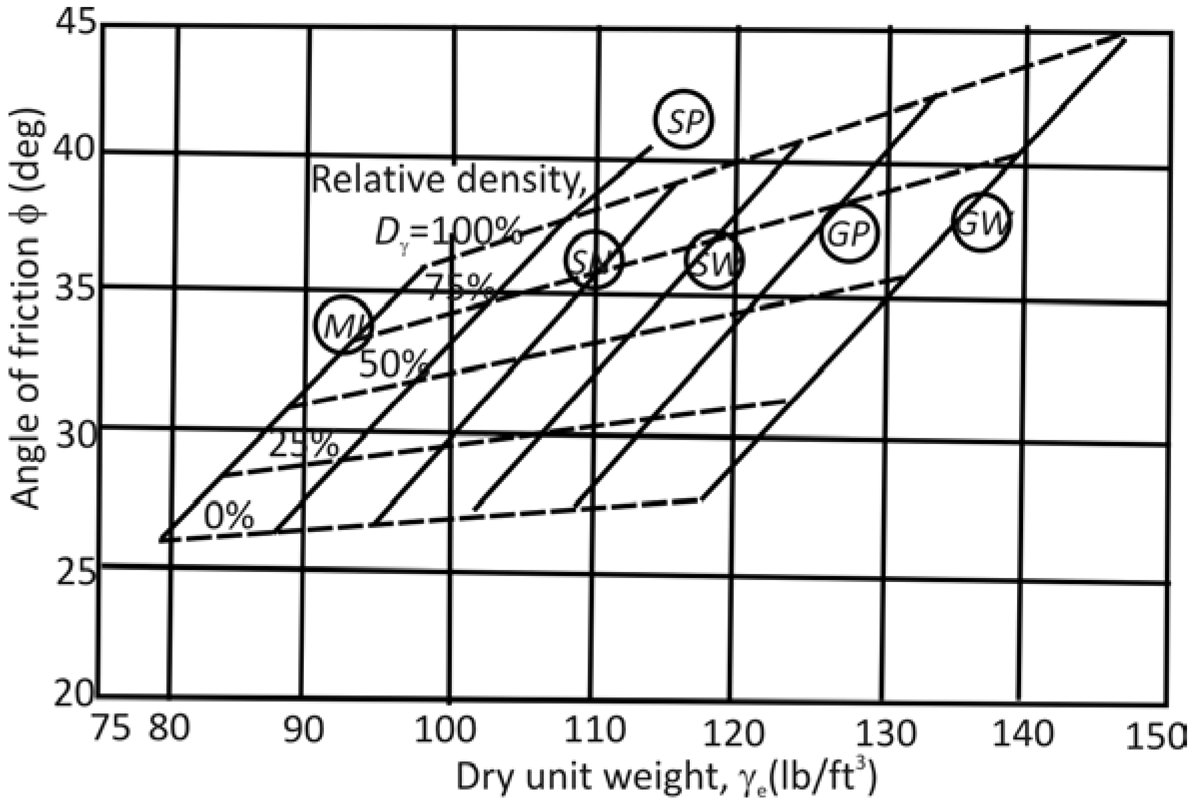
Where

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

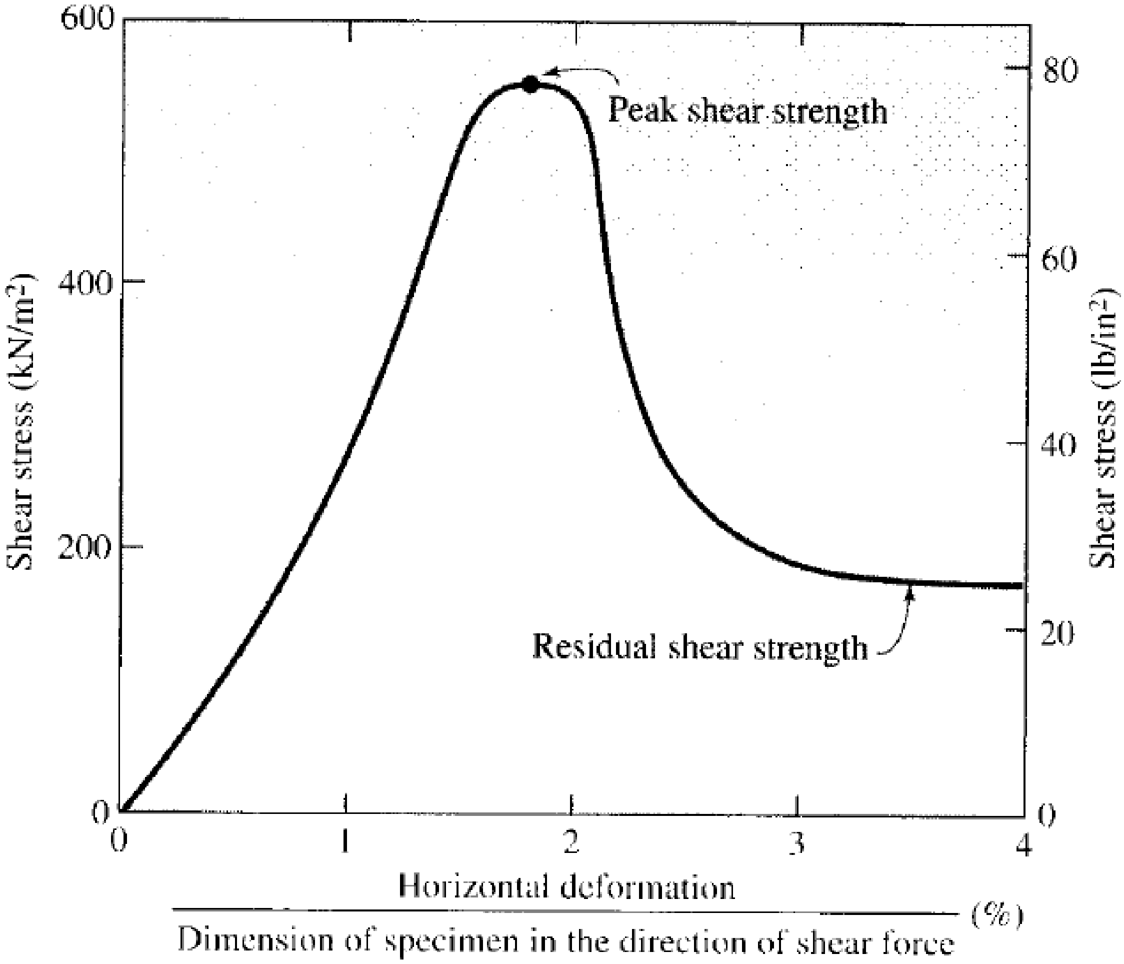
Several tests of this type can be conducted by varying the normal load. The angle of friction of the sand can be determined by plotting a graph of s against σ' $(= \sigma)$

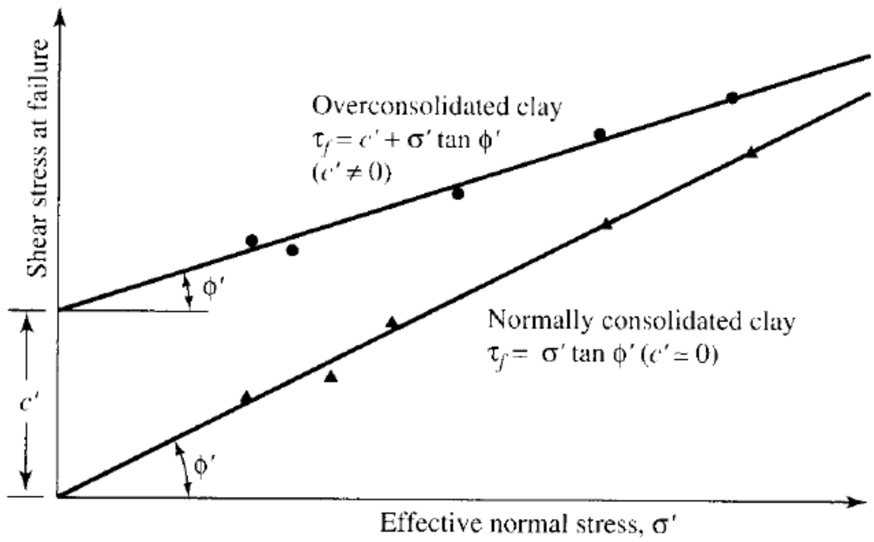
$$\phi = \tan^{-1}\left(\frac{s}{\sigma'}\right)$$

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**



Range of relative density and corresponding range of angle of friction for coarse-grained soil





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

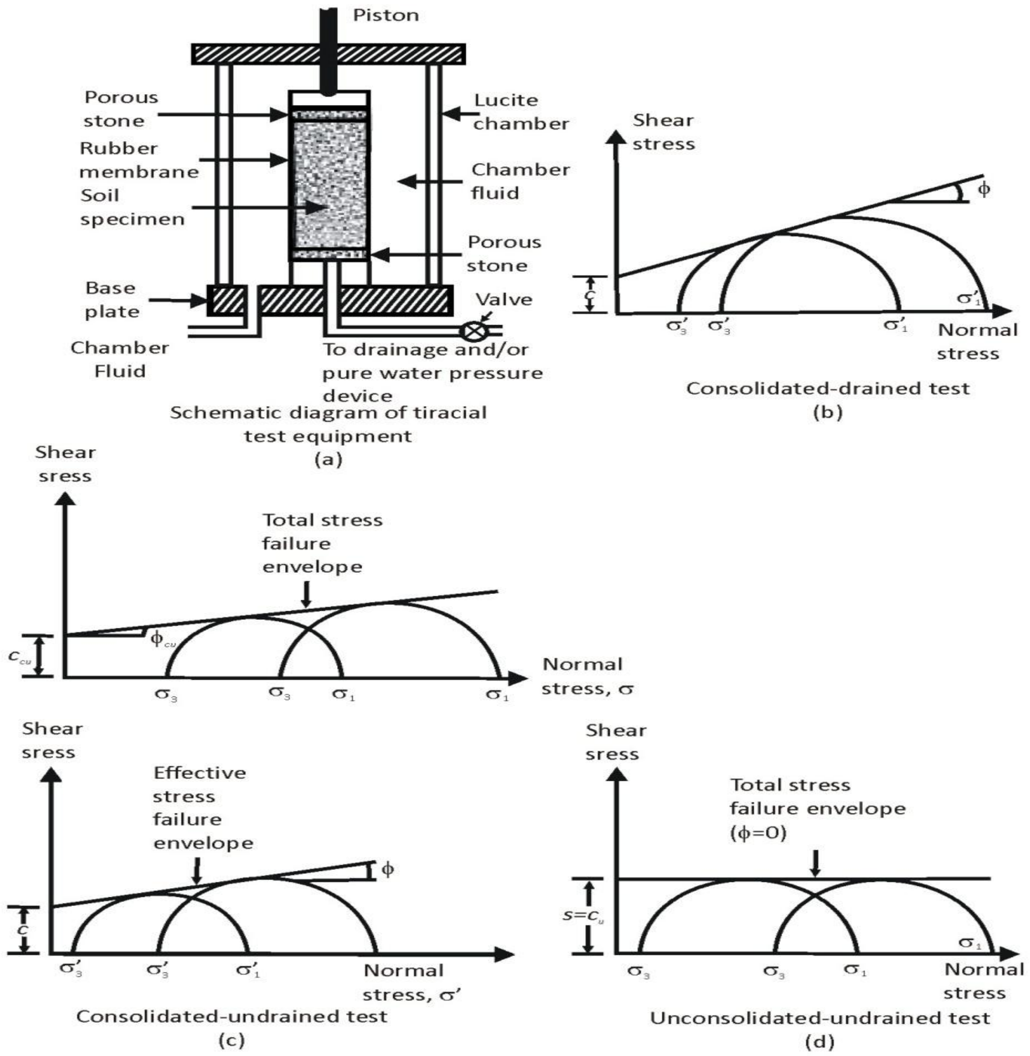
$$\sigma'_1 = \sigma'_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)$$

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 $=\sigma_3 - u_f = \sigma'_3$

Changing σ_3 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

$s = c_{cu} + \sigma \tan\phi_{cu}$

Where c_{cu} and ϕ_{cu} are the consolidated-undrained cohesion and angle of friction respectively (Note: $c_{cu} \approx 0$ 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$=\sigma_3 = \Delta\sigma_f = \sigma_1$

Minor principal total stress $=\sigma_3$

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, $\sigma 3$.The tangent to these Mohr's circles will be a horizontal line, called the $\phi=0$ 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

$ud=A\ \Delta\sigma$

Where

A=Skempton′s pore pressure parameter

However,

$\Delta\sigma=\sigma 1-\sigma 3$

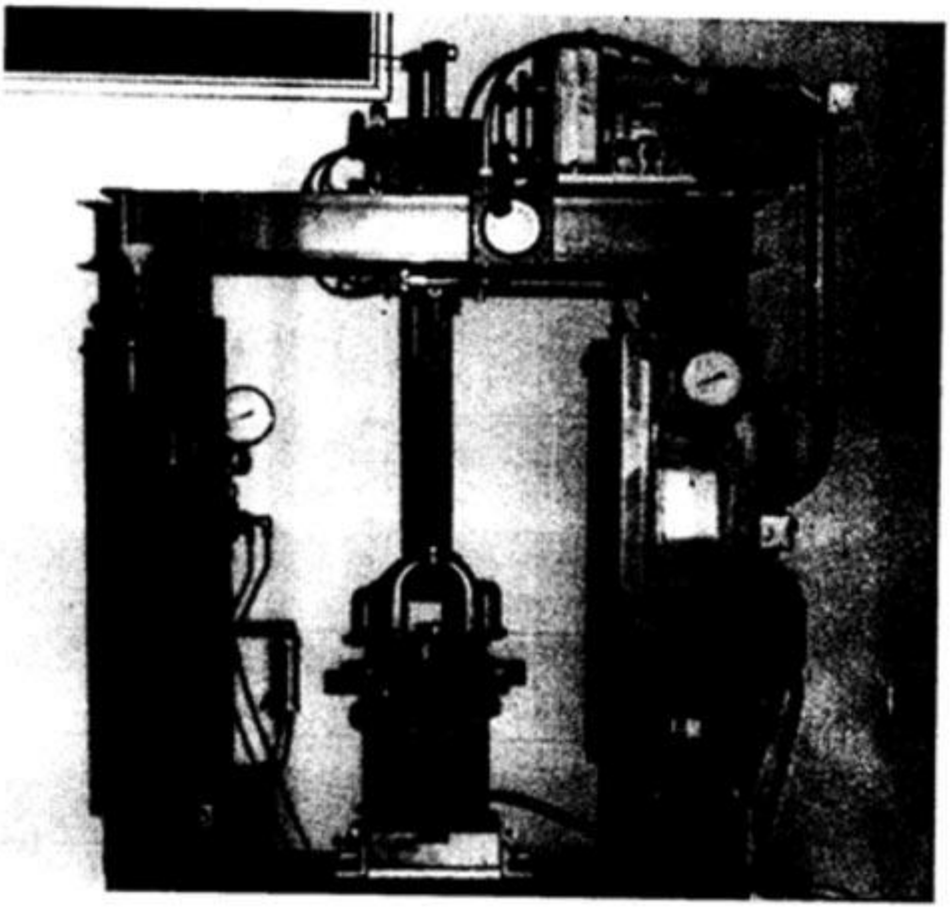
Combining equations gives

$u=ua+ud=B\sigma 3+A\sigma 1-\sigma 3$

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.

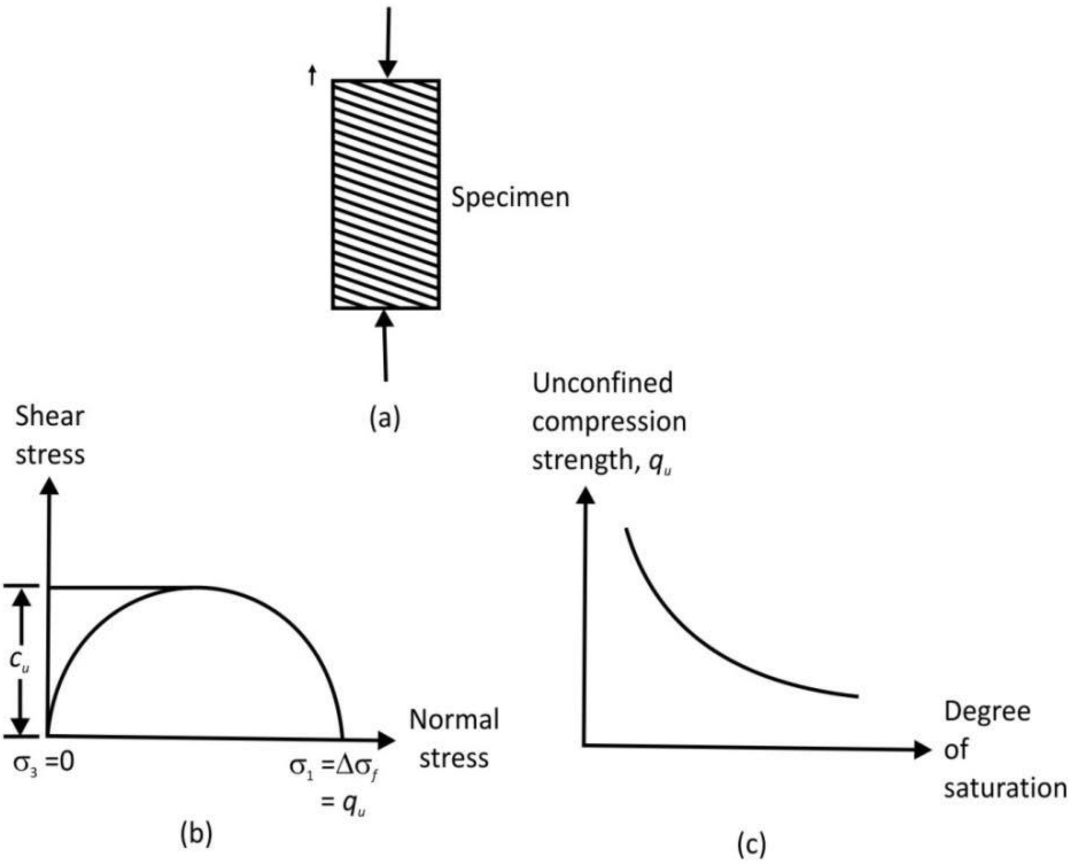


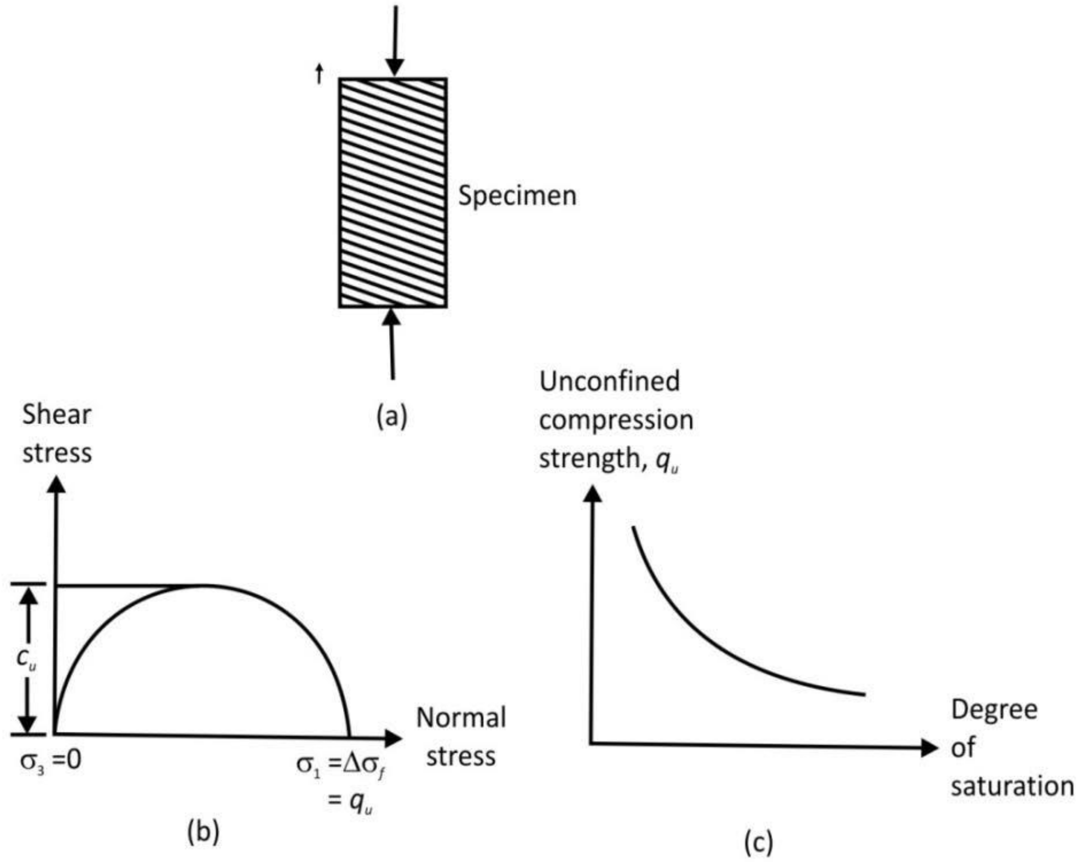
Triaxial test equipment

LECTURE 34

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





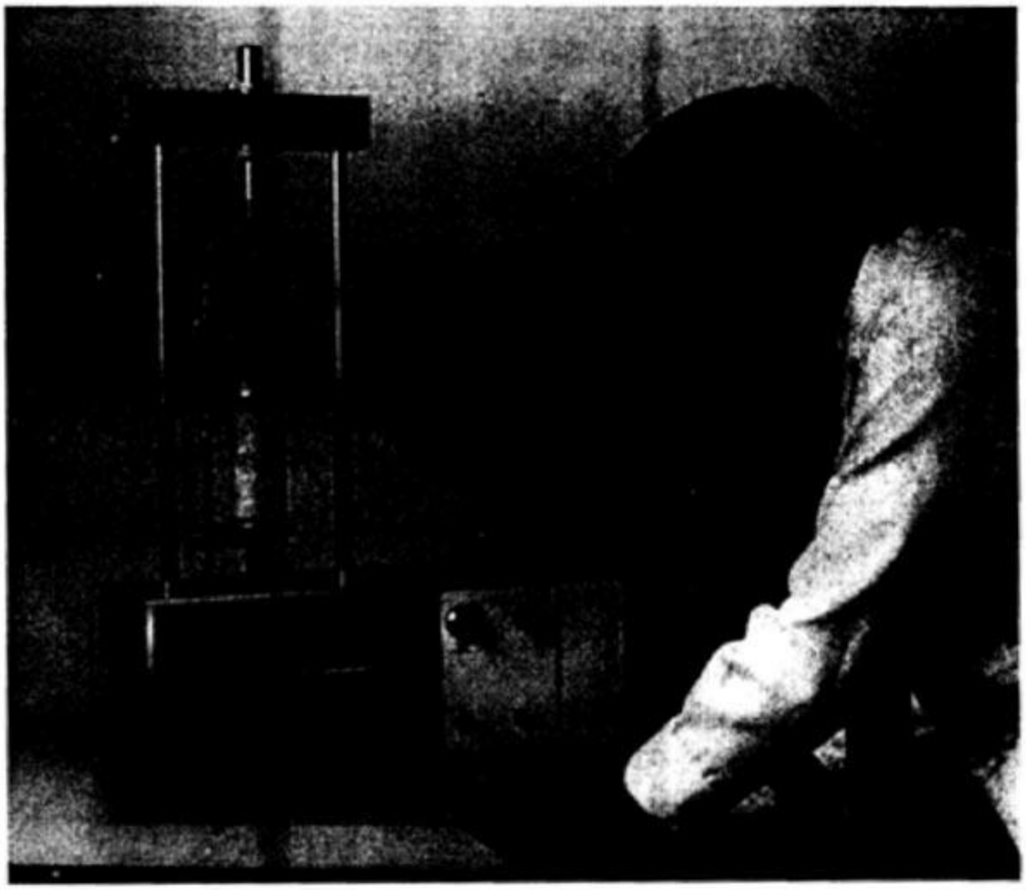
Unconfined compression test: (a) soil specimen; (b) Mohr's circle for the test; (c) variation of qu with the degree of saturation

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



compression test.

Unconfined compression test in progress (courtesy of Soiltest, Inc., Lake Bluff, Illinois)

LECTURE 35

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

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

where d : diameter of the shear van c/z : height of thc shear vane

For the calculation of M., investigator sh avea everal t ypeso f distributicln of shear strength mobilization at thc ends of the soil cylinder:

l. Triangular. Shear strength mobilization is c,, at the periphcry 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 rom 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

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

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

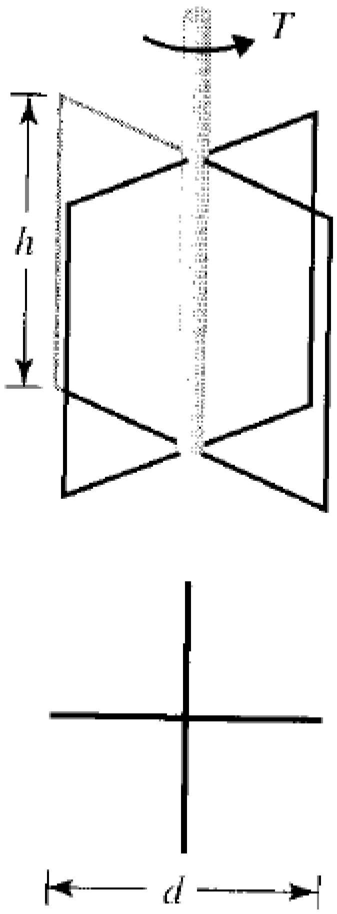
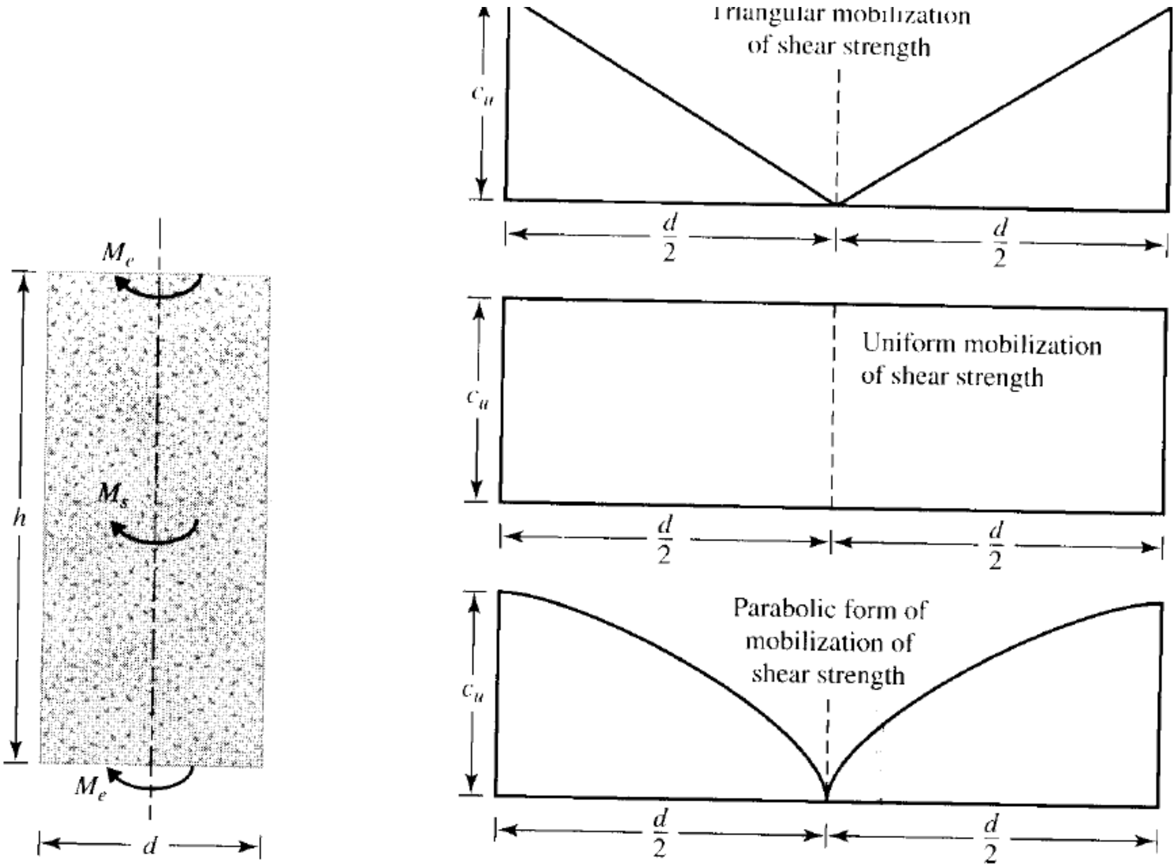


Diagram of vane shear
test equipment



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

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

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

LECTURE 36

STABILIZATION OF SOIL:

It is the policy of the Indiana Department of Transportation to minimize the disruption of traffic patterns and the delay caused today's motorists whenever possible during the construction or reconstruction of the State's roads and bridges. INDOT Engineers are often faced with the problem of constructing roadbeds on or with soils, which do not possess sufficient strength to support wheel loads imposed upon them either in construction or during the service life of thepavement. It is, at times, necessary to treat these soils to provide a stable subgrade or a working platform for the construction of the pavement. The result of these treatments are that less time and energy is required in the production, handling, and placement of road and bridge fills and subgrades and therefore, less time to complete the construction process thus reducing the disruption and delays to traffic.

These treatments are generally classified into two processes, soil modification or soil stabilization. The purpose of subgrade modification is to create a working platform for construction equipment. No credit is accounted for in this modification in the pavement design process. The purpose of subgrade stabilization is to enhance the strength of the subgrade. This increased strength is then taken into account in the pavement design process. Stabilization requires more thorough design methodology during construction than modification. The methods of subgrade modification or stabilization include physical processes such as soil densification, blends with granular material, use of reinforcements (Geogrids), undercutting and replacement, and chemical processes such as mixing with cement, fly ash, lime, lime byproducts, and blends of any one of these materials. Soil properties such as strength,compressibility, hydraulic conductivity, workability, swelling potential, and volume change tendencies may be altered by various soil modification or stabilization methods. Subgrade modification shall be considered for all the reconstruction and new alignment projects.

When used, modification or stabilization shall be required for the full roadbed width including shoulders or curbs. Subgrade stabilization shall be considered for all subgrade soils with CBR of less than 2.INDOT standard specifications provide the contractor options on construction practices to

achieve subgrade modification that includes chemical modification,replacement with aggregates, geosynthetic reinforcement in conjunction with the aggregates, and density and moisture controls. Geotechnical designers have to evaluate the needs of the subgrade and

include where necessary, specific treatment above and beyond the standard specifications.Various soil modification or stabilization guidelines are discussed below. It is necessary for designers to take into consideration the local economic factors as well as environmentalconditions and project location in order to make prudent decisions for design.

It is important to note that modification and stabilization terms are not interchangeable.

Mechanical Stabilization:

This is the process of altering soil properties by changing the gradation through mixing with other soils, densifying the soils using compaction efforts, or undercutting the existing soils and replacing them with granular material.

A common remedial procedure for wet and soft subgrade is to cover it with granular material or to partially remove and replace the wet subgrade with a granular material to a pre-determined depth below the grade lines. The compacted granular layer distributes the wheel loads over a wider area and serves as a working platform.To provide a firm-working platform with granular material, the following conditions shall be met.

1. The thickness of the granular material must be sufficient to develop acceptable pressure distribution over the wet soils.
2. The backfill material must be able to withstand the wheel load without rutting.
3. The compaction of the backfill material should be in accordance with the Standard Specifications.

Based on the experience, usually 12 to 24 in. (300 to 600mm) of granular material should be adequate for subgrade modification or stabilization. However, deeper undercut and replacement may be required in certain areas

The undercut and backfill option is widely used for construction traffic mobility and a working platform. This option could be used either on the entire project or as a spot treatment. The equipment needed for construction is normally available on highway construction projects.

Geosynthetic Stabilization

Geogrid has been used to reinforce road sections. The inclusion of geogrid in subgrades changes the performance of the roadway in many ways (6). Tensile reinforcement, confinement, lateral spreading reduction, separation, construction uniformity and reduction in strain have been identified as primary reinforcement mechanisms. Empirical design and post-construction evaluation have lumped the above described benefits into better pavement performance during the design life. Geogrid with reduced aggregate thickness option is designed for urban area and recommendations are follows;

Excavate subgrade 9 in. (230 mm) and construct the subgrade with compacted aggregate No. 53 over a layer of geogrid, Type I. This geogrid reinforced coarse aggregate should provide stable working platform corresponding to 97 percent of CBR. Deeper subgrade problem due to high moisture or organic soils requires additional recommendations. Geogrid shall be in accordance with 918.05(a) and be placed directly over exposed soils to be modified or stabilized and overlapped according with the following table.

SPT blow Counts per foot (N)	Overlap
> 5	12 in. (300 mm)
3 to 5	18 in. (450 mm)
less than 3	24 in. (600 mm)