Department of Civil Engineering
Study Material

Unit-V

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

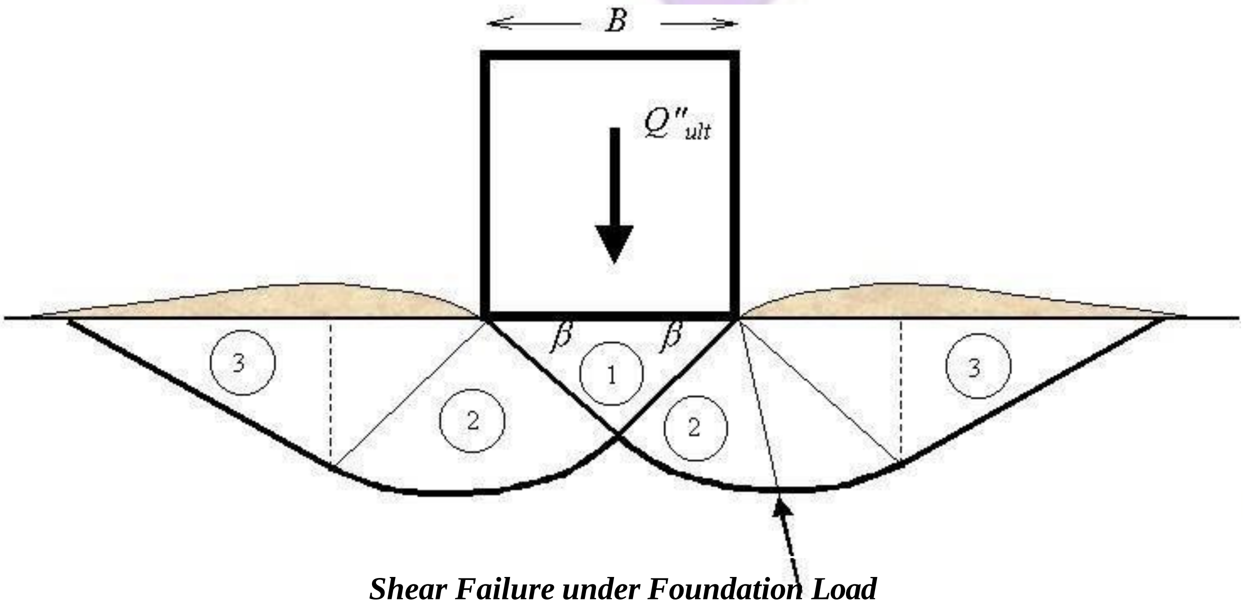
• If the soil fails, the structure founded on it can collapse

Thus shear strength is

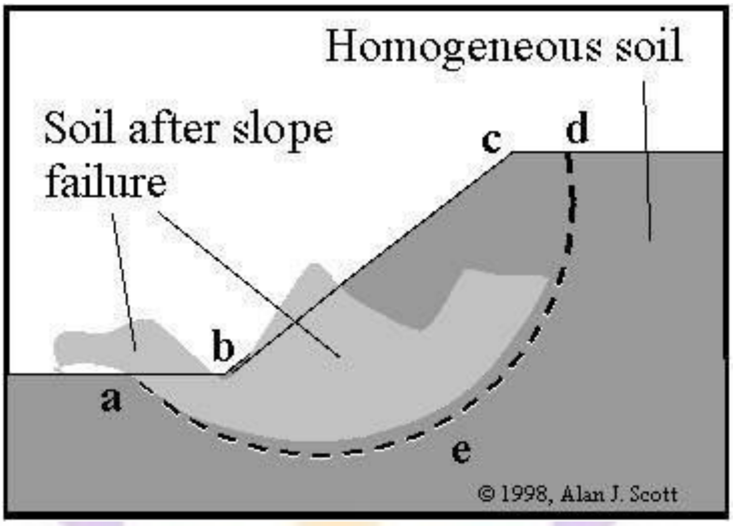
"The capacity of a material to resist the internal and external forces which slide past each other"

Significance of Shear Strength :

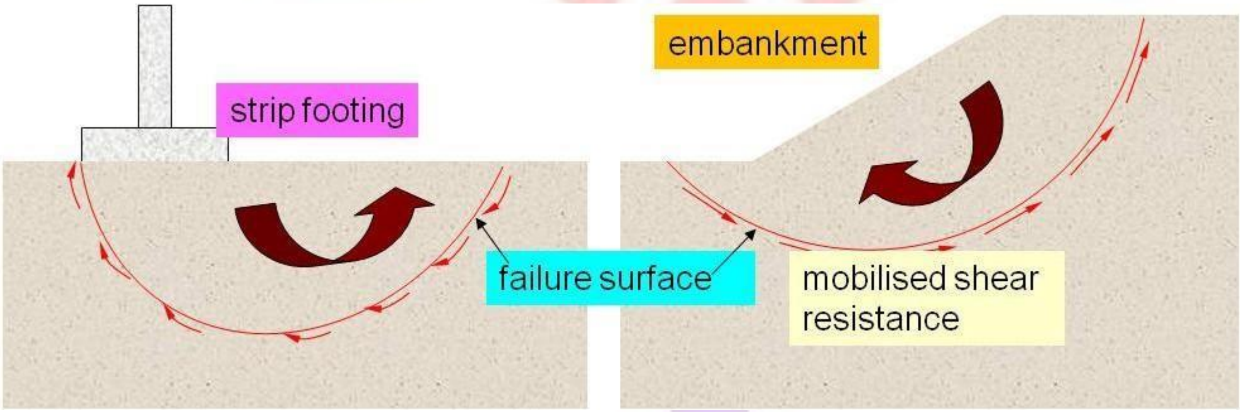
- Engineers must understand the nature of shearing resistance in order to analyze soil stability problems such as;
- Bearing capacity
- Slope stability
- Lateral earth pressure on earth-retaining structure



Shear Failure under Foundation Load



Slope Stability Failure as an Example of Shearing Along Internal Surface



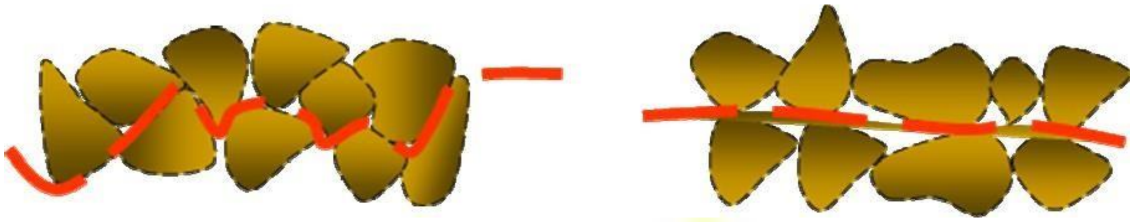
At failure, shear stress along the failure surface reaches the shear

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)

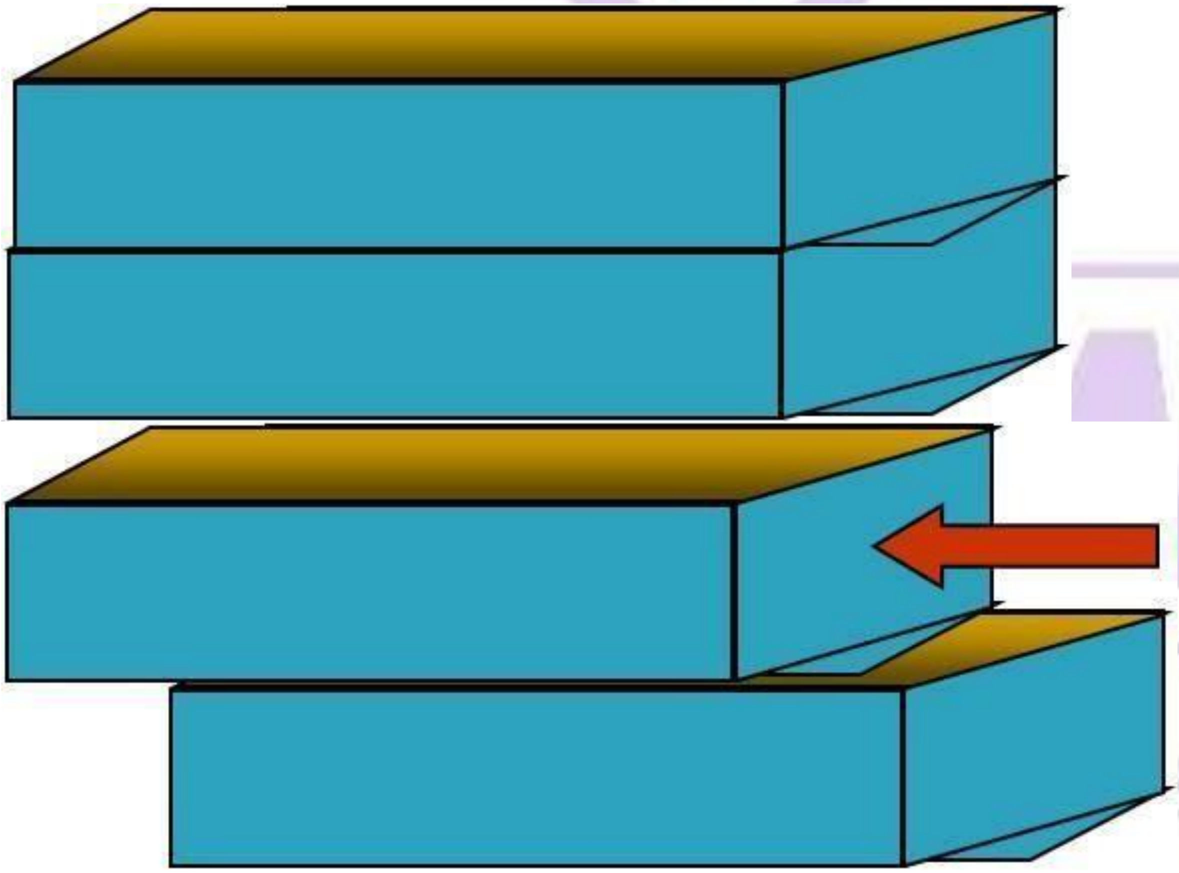
• Cementation between sand grains

• Electrostatic attraction between clay particles

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

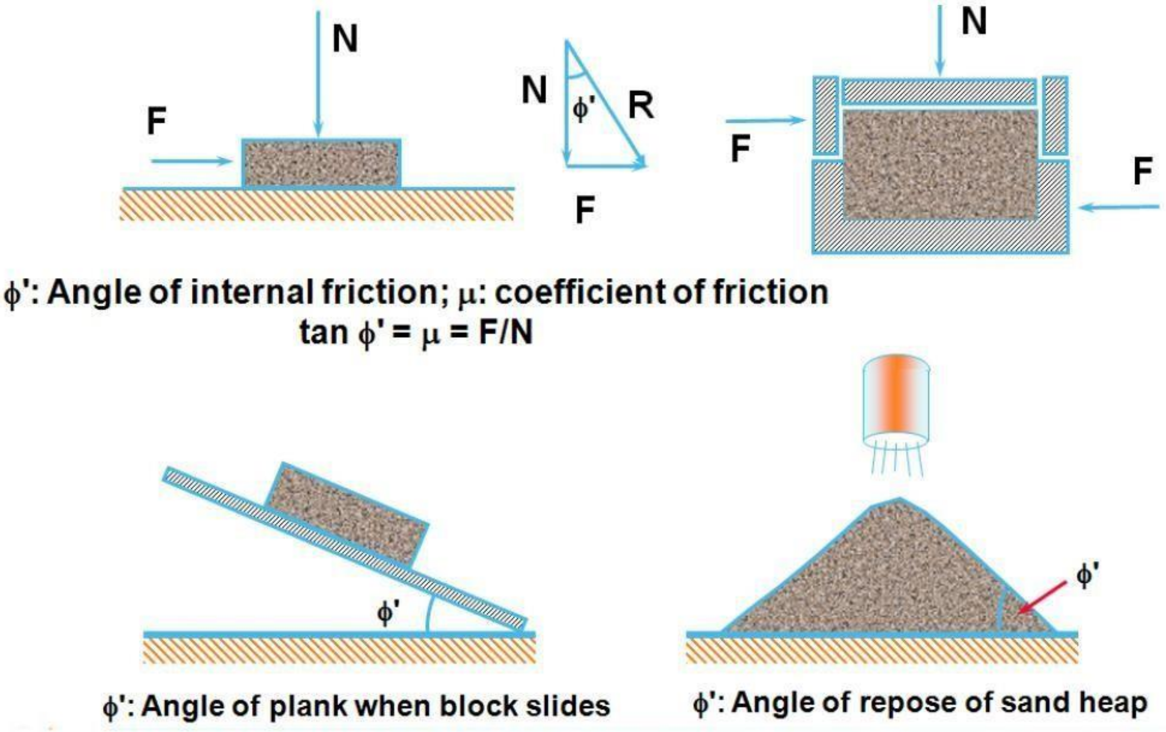
Cohesion :

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

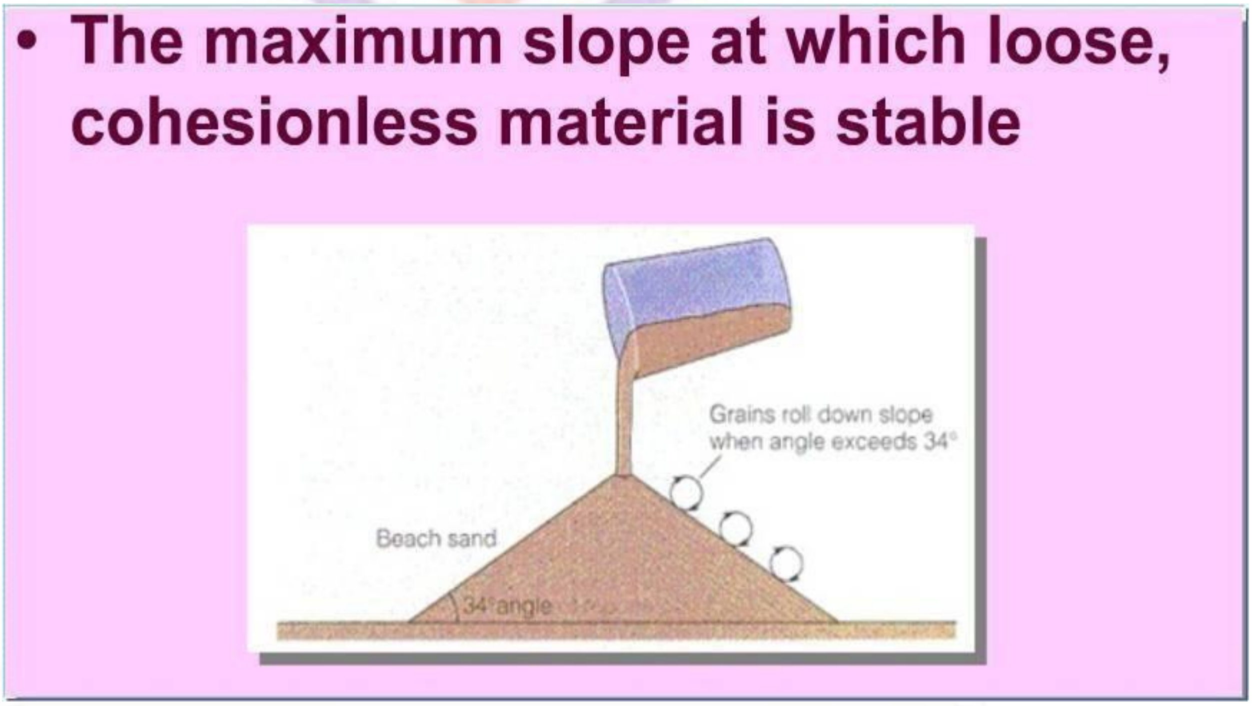


Internal Friction :

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).

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.

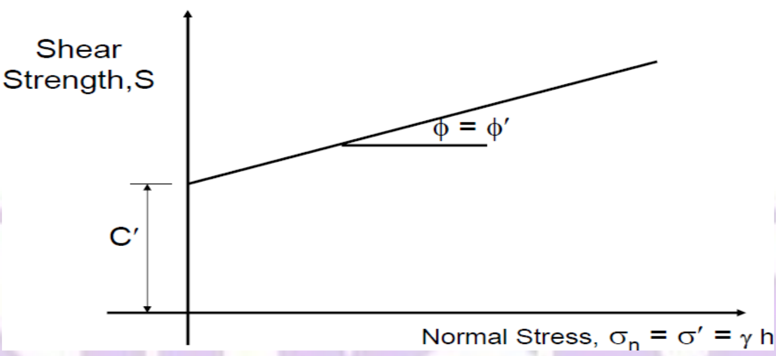
– *Initial state*: State can be describe by terms such as: loose, dense, overconsolidated, normally consolidated, stiff, soft, etc.

– *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.

Mohr-Coulomb Failure Criterion



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

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

where

τ_f = shear strength

c = cohesion; c′ =effective cohesion

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

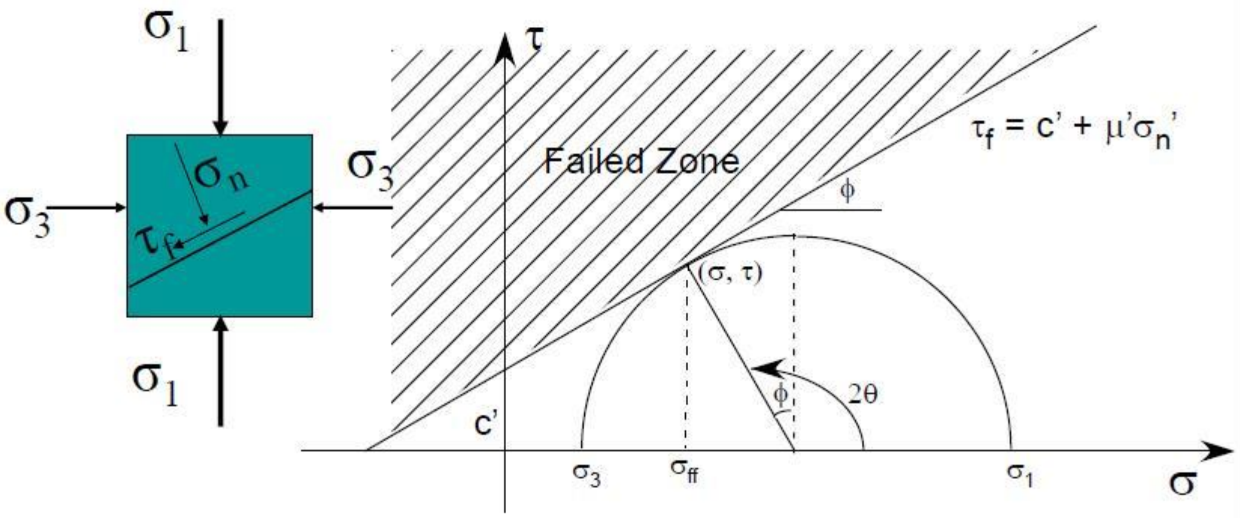
μ = coefficient of friction; μ' = effective coefficient of friction.

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 tc as the *drained angle of friction*. Typical values of ϕ' for some granular soils are giver in Table 11.1

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



$2\theta = 90 + \phi'$, or

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

Again, from Figure 11.3,

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

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

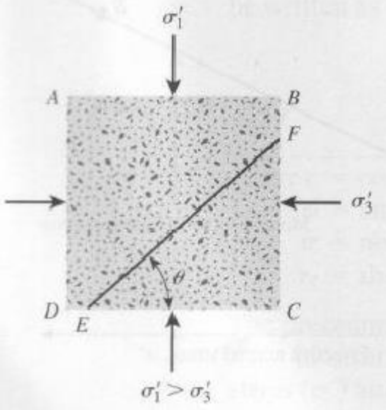


Figure Inclination of failure plane in soil with major principal plane

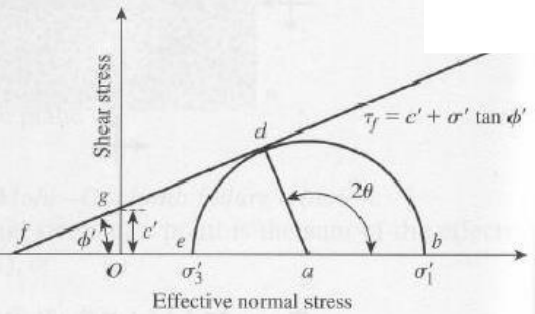


Figure Mohr's circle and failure envelope

Also,

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

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

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

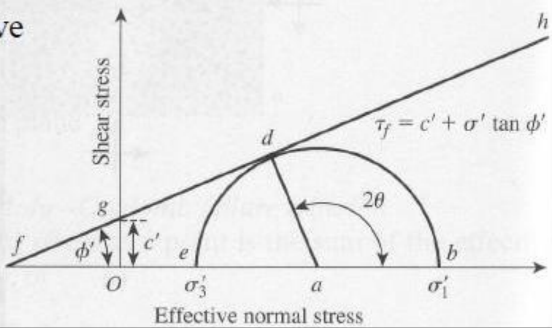
or

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

However, From trigonometric equalities we have

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

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



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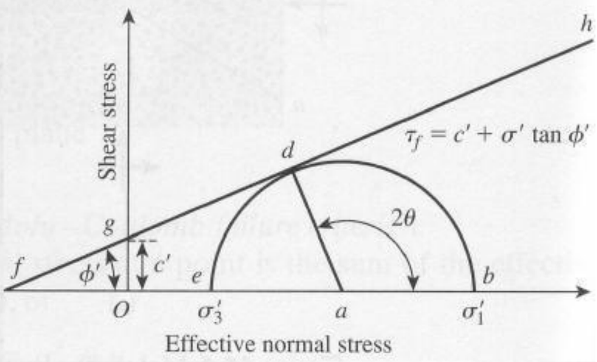


Thus,

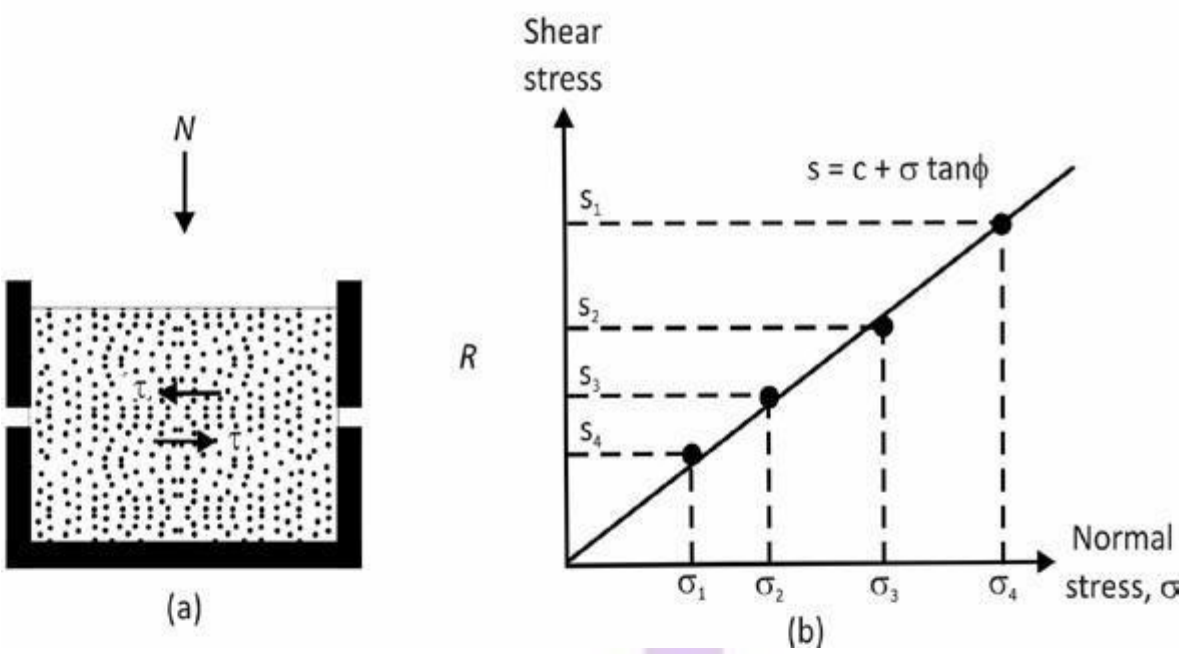
$$\sigma'_1 = \sigma'_3 \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

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

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

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

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

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

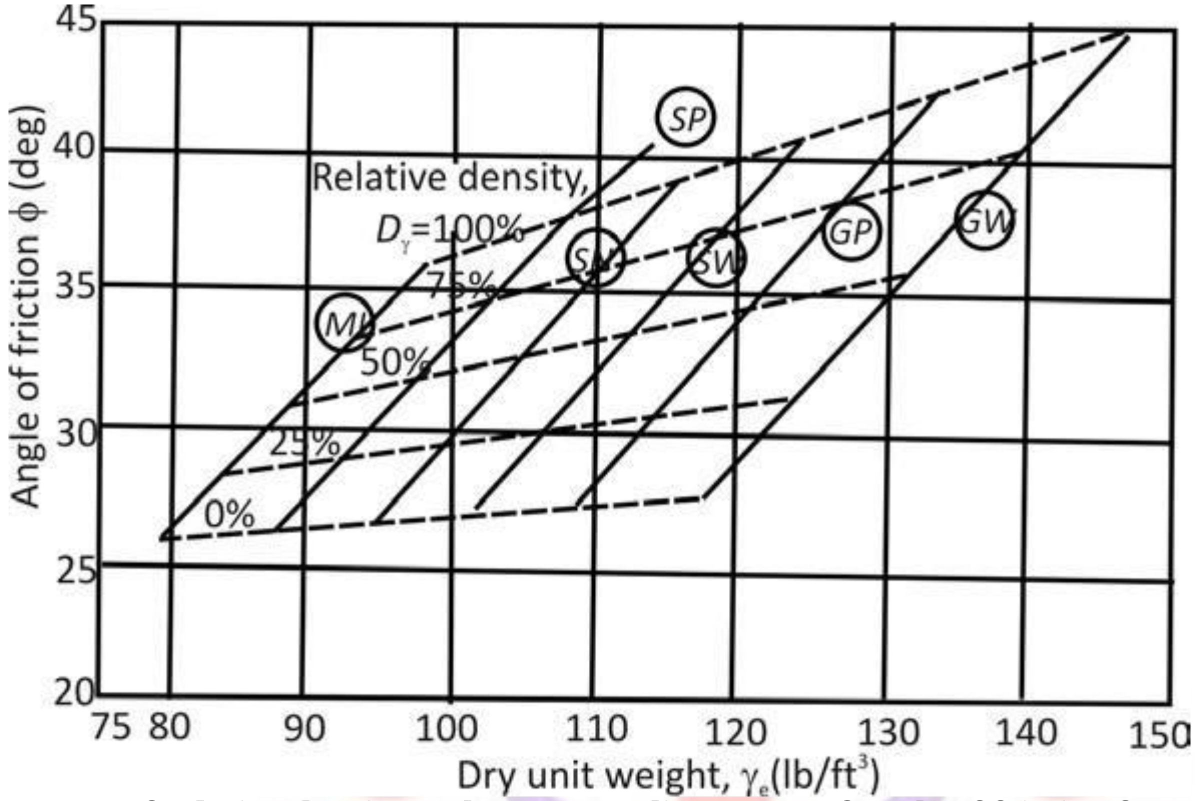
Where

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

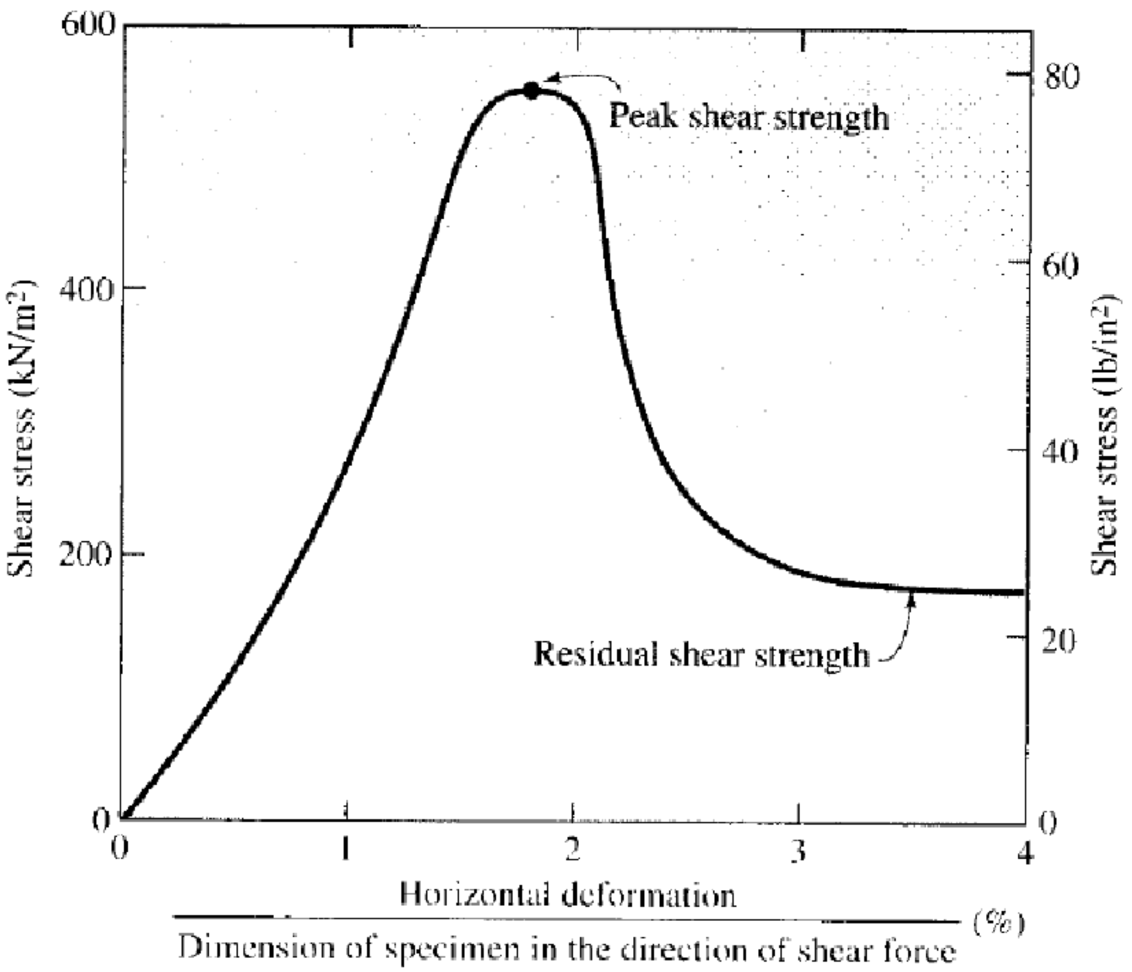
Several tests of this type can be conducted by varying the normal load. The angle of friction of the sand can be determined by plotting a graph of s against σ' (= σ)

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

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



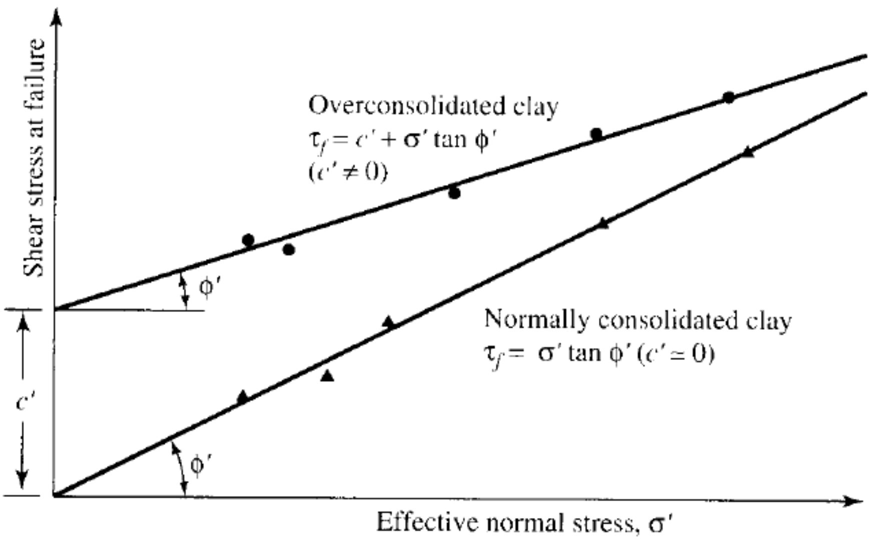


Figure 11.9 Failure envelope for clay obtained from drained direct shear tests

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 (σ3) is applied to the specimen by means of the chamber fluid (generally water or glycerin). An added stress (Δσ) can also be applied to the specimen in the axial direction to cause failure (Δσ=Δσf 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 =σ3=Δσf=σ1=σ′1

Minor Principal effective stress =σ3=Δσ′3

Changing σ3 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≈0). At failure

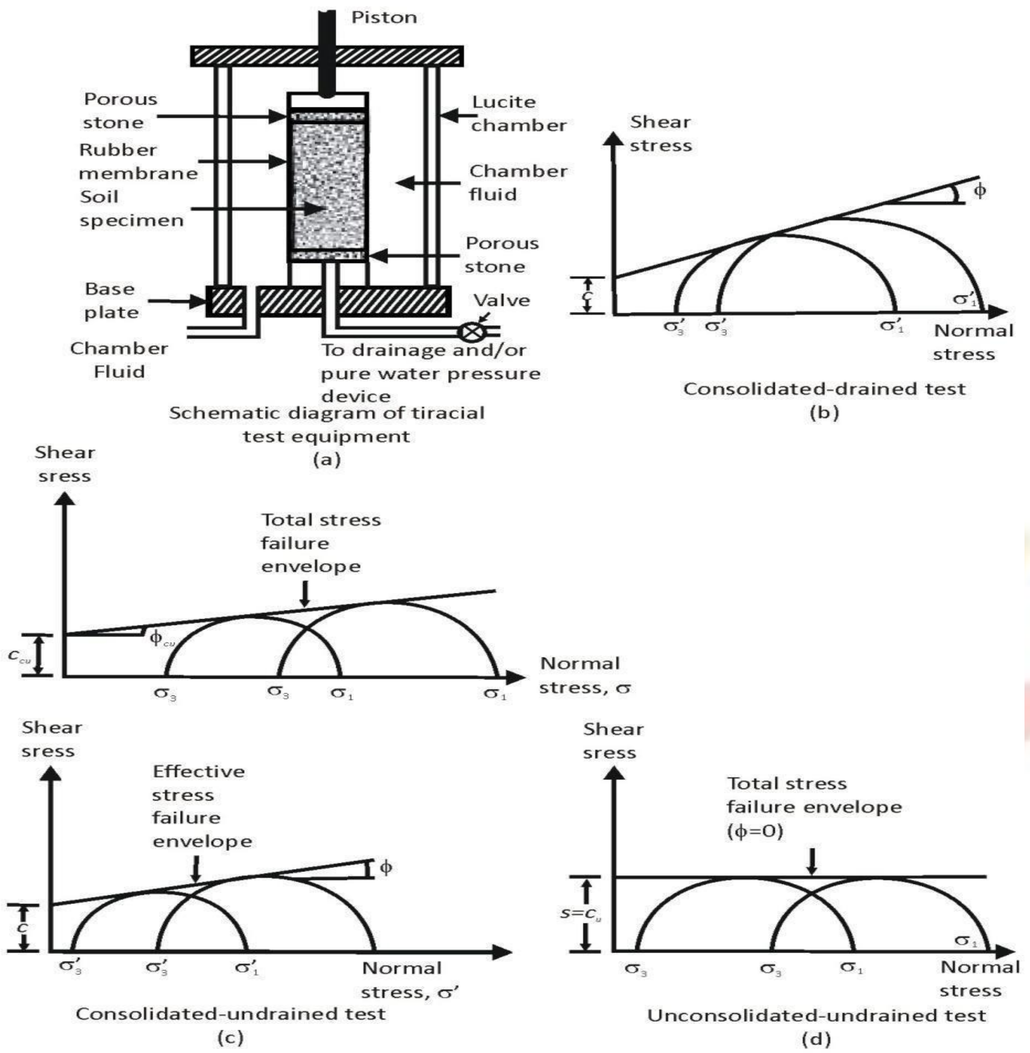
$$\sigma'_1 = \sigma'_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)$$

For consolidated-undrained tests, at failure,

Major Principal total stress =σ3=Δσf=σ1

Minor principal total stress =σ3

Major principal effective stress =(σ3+Δσf)−uf=σ′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 σ1−σ3=Δσf is a constant, irrespective of the chamber confining pressure, σ3.The tangent to these Mohr's circles will be a horizontal line, called the ϕ=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

$$u = u_a + u_d \qquad [1.87]$$

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, Δσ, so

ud=A Δσ

Where

A=Skempton′s pore pressure parameter

However,

Δσ=σ1−σ3

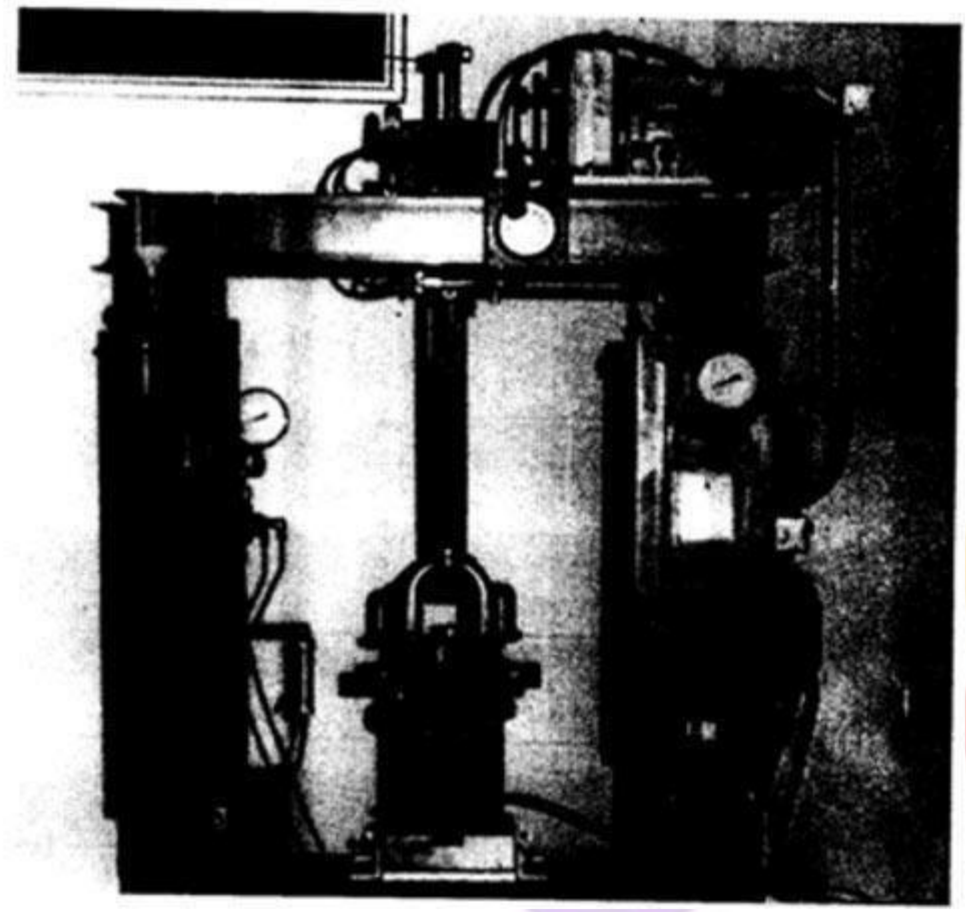
Combining equations gives

u=ua+ud=Bσ3+Aσ1−σ3

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

u=σ3+A(σ1−σ3)

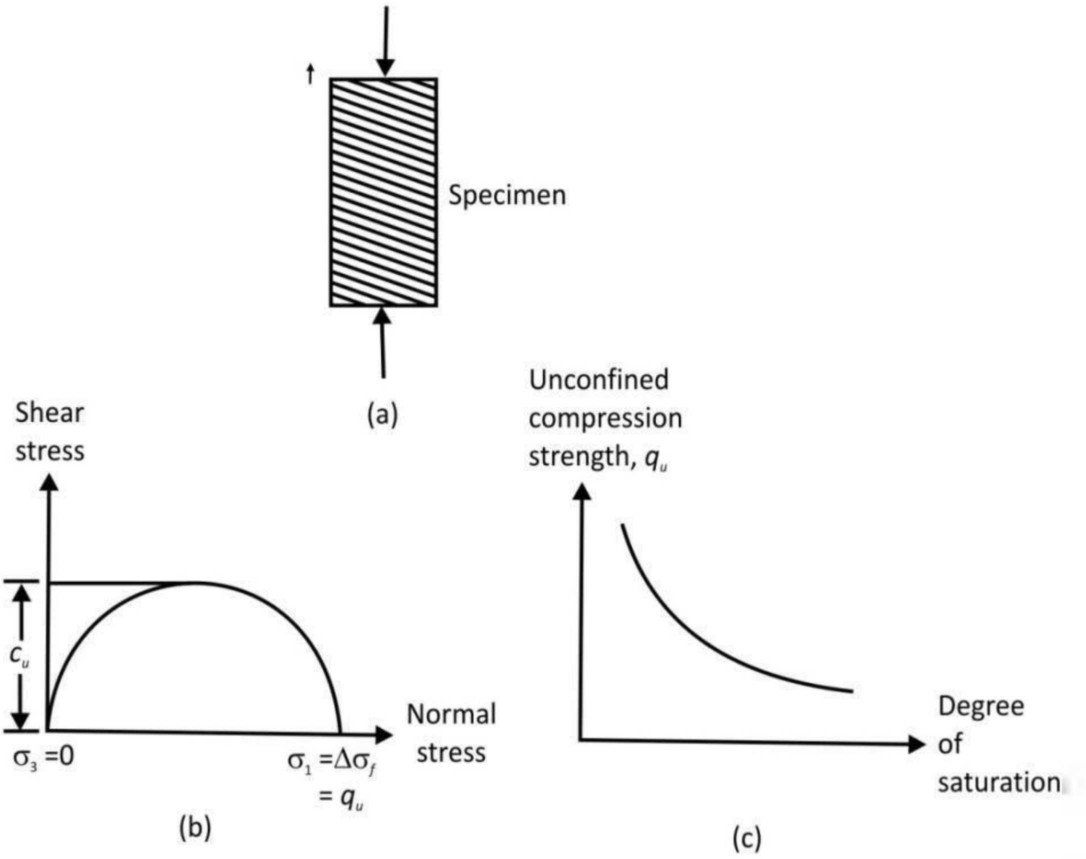
The value of the pore water pressure parameter A at failure will vary with the type of soil. Following is a general range of the values of A at failure for various types of clayey soil encountered in nature.

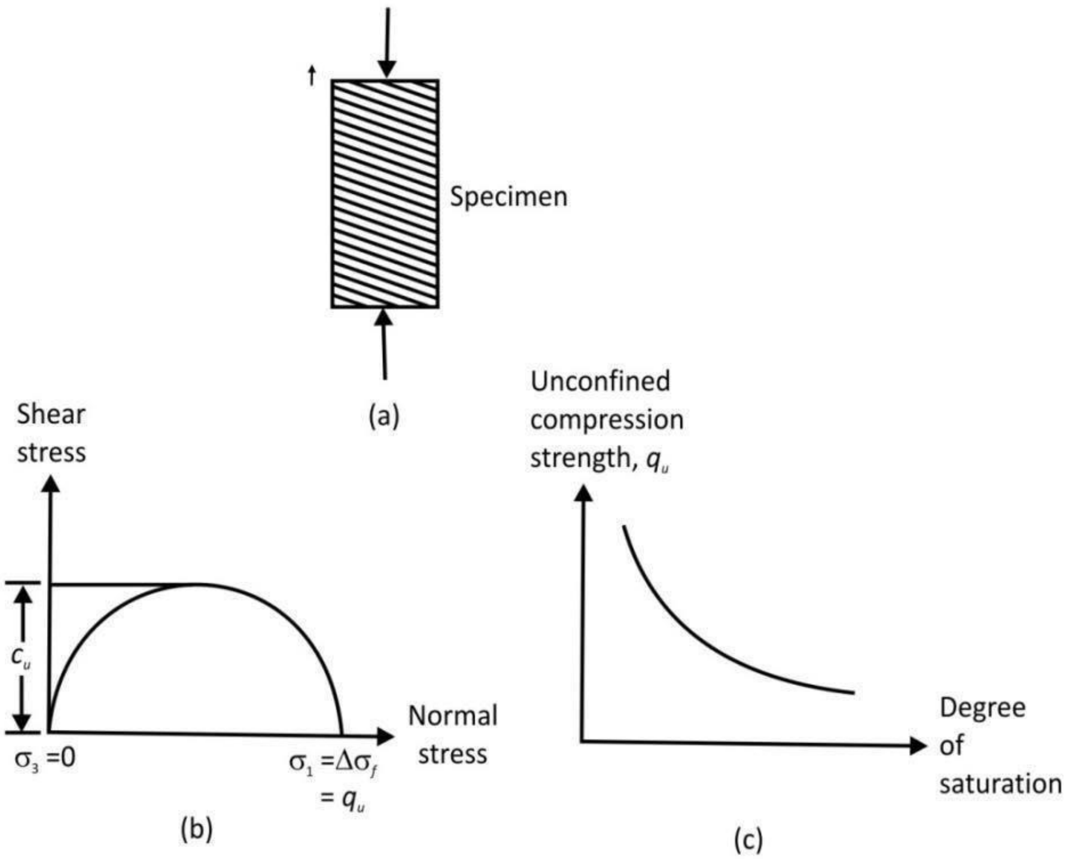


Triaxial test equipment

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)

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

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

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

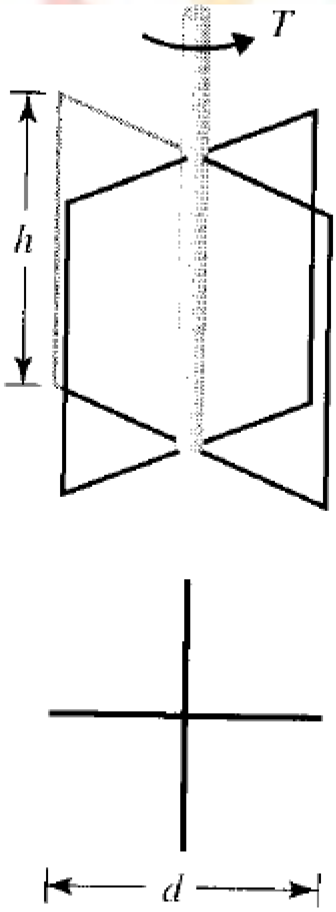
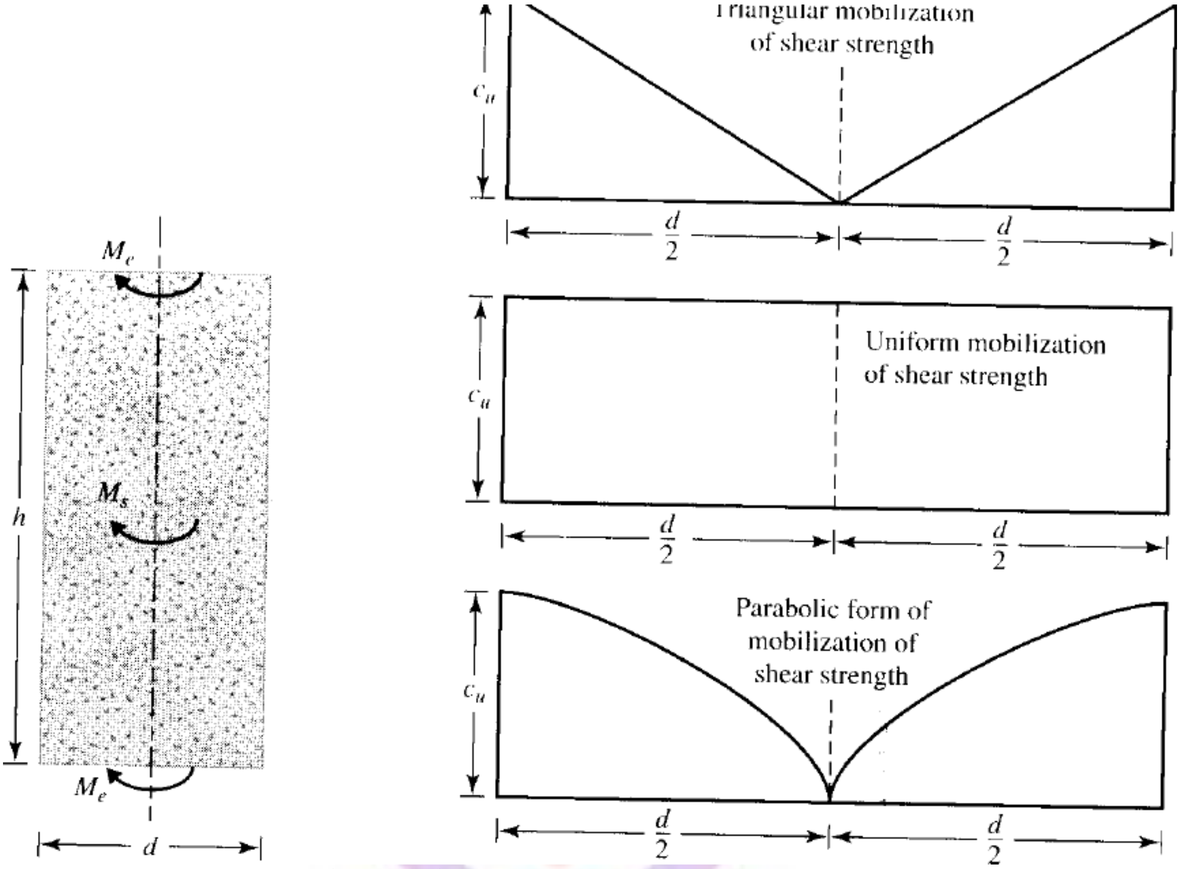


Diagram of vane shear test equipment



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

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

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

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)

Chemical Modification or Stabilization

The transformation of soil index properties by adding chemicals such as cement, fly ash, lime, or a combination of these, often alters the physical and chemical properties of the soil including the cementation of the soil particles. There are the two primary mechanisms by which chemicals alter the soil into a stable subgrade:

1. Increase in particle size by cementation, internal friction among the agglomerates, greater shear strength, reduction in the plasticity index, and reduced shrink/swell potential.
2. Absorption and chemical binding of moisture that will facilitate compaction.

Design Procedures

Criteria for Chemical Selection

When the chemical stabilization or modification of subgrade soils is considered as the most economical or feasible alternate, the following criteria should be considered for chemical selection based on index properties of the soils.

1. Chemical Selection for Stabilization.

   a. Lime: If PI > 10 and clay content (2μ) > 10%.

   b. Cement: If PI ≤ 10 and < 20% passing No. 200.

   Note: Lime shall be quicklime only.

2. Chemical Selection for Modification

   a. Lime: PI ≥ 5 and > 35 % Passing No. 200

   b. Fly ash and lime fly ash blends: 5 < PI < 20 and > 35 % passing No. 200

   c. Cement and/ or Fly ash: PI < 5 and ≤ 35 % Passing No. 200

   Fly ash shall be class C only.
   Lime Kiln Dust (LKD) shall not be used in blends.
   Appropriate tests showing the improvements are essential for the exceptions listed above.

Suggested Chemical Quantities For Modification Or Stabilization-

a. Lime or Lime By-Products:	4% to7 %
b. Cement:	4% to 6%
c. Fly ash Class C:	10% to 16%

% for each combination of lime-fly ash or cement-fly ash shall be established based on laboratory results.

Strength requirements for stabilization and modification

The reaction of a soil with quick lime, or cement is important for stabilization or modification and design methodology. The methodology shall be based on an increase in the unconfined compression strength test data. To determine the reactivity of the soils for lime stabilization, a pair of specimens measuring 2 in. (50 mm) diameter by

4 in. (100 mm) height (prepared by mixing at least 5% quick lime by dry weight of the natural soil) are prepared at the optimum

moisture content and maximum dry density (AASHTO T 99). Cure the specimens for 48 hours at 120o F (50o C) in the laboratory and test as per AASHTO T 208. The strength gain of lime soil mixture must be at least 50 psi (350 kPa) greater than the natural soils. A strength gain of 100 psi (700 kPa) for a soil-cement mixture over the natural soil shall be considered adequate for cement stabilization with 4% cement by dry weight of the soils and tested as described above

In the case of soil Stabilization, enhanced subgrade support is not accounted for in pavement design. However, an approved chemical (LKD, cement, and fly ash class C) or a combination of the chemicals shall attain an increase in strength of 30 psi over the natural soils when specimens are prepared and tested in the same manner as stabilization.

Lime Stabilization.

Lime reacts with medium, moderately fine and fine-grained soils to produce decreased plasticity,increased workability, reduced swelling, and increased strength. The major soil properties and characteristics that influence the soils ability to react with lime to produce cementitious materials are pH, organic content, natural drainage, and clay mineralogy. As a general guide, treated soils should increase in particle size with cementation, reduction in plasticity, increased in internal friction among the agglomerates, increased shear strength, and increased workability due to the textural change from plastic clay to friable, sand like material.

The following procedures shall be utilized to determine the amount of lime required to stabilize the subgrade. Hydrated or quick lime and lime by-products should be used in the range of 4 ± 0.5% and 5 ± 1% by weight of soil for modification respectively. The following procedures shall be used to determine the optimum lime content.

Perform mechanical and physical tests on the soils.

Determine the separate pH of soil and lime samples.

Determine optimum lime content using Eades and Grim pH test.

• A sufficient amount of lime shall be added to soils to produce a pH of 12.4 or equal to the pH of lime itself. An attached graph is plotted showing the pH as lime content increases. The Optimum lime content shall be determined corresponding to the maximum pH of lime-soil mixture. (See Figure 4.0 A).

• Representative samples of air-dried, minus No. 40 soil is equal to 20 g of oven-dried soil are weighed to the nearest 0.1 g and poured into 150-ml (or larger) plastic bottles with screw on tops.

• It is advisable to set up five bottles with lime percentages of 3, 4, 5, 6, and 7. This will insure, in most cases, that the percentage of lime required can be determined in one hour. Weigh the lime to the nearest 0.01 g and add it to the soil. Shake the bottle to mix the soil and dry lime.

• Add 100 ml of CO_2-free distilled water to the bottles.

• Shake the soil-lime mixture and water until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.

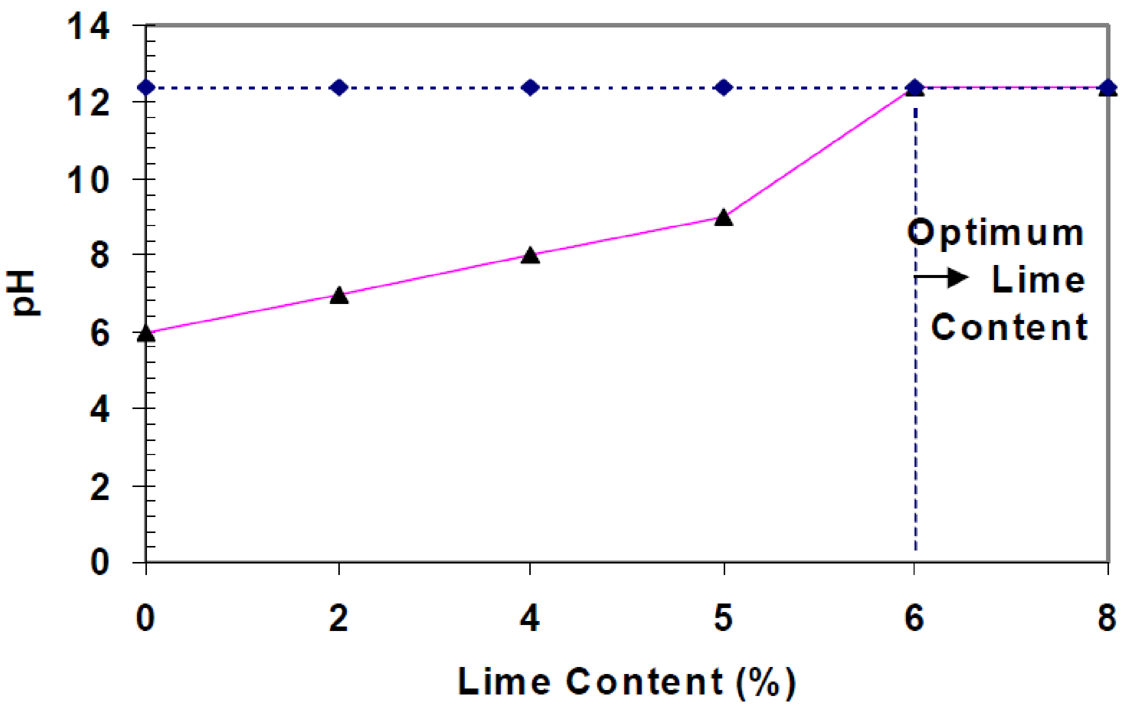
• Shake the bottles for 30 seconds every 10 minutes.

• After one hour, transfer part of the slurry to a plastic beaker and measure the pH. The pH meter must be equipped with a Hyalk electrode and standardized with a buffer solution having a pH of 12.00.

• Record the pH for each of the lime-soil mixtures. If the pH readings go to 12.40, then the lowest percent lime that gives a pH of 12.40 is the percentage required to stabilize the soil. If the pH does not go beyond 12.30 and 2 percentages of lime give the same readings, the lowest percent which gives a pH of 12.30 is the amount required to stabilize the soil. If the highest pH is 12.30 and only 1 percent lime gives a pH of 12.30, additional test bottles should be started with larger percentages of lime.

d. Atterberg limits should be performed on the soil-lime mixtures corresponding to optimum lime content as determined above.

e. Compaction shall be performed in accordance with AASHTO T 99 on the optimum lime and soil mixture to evaluate the drop in maximum dry density in relation to time (depending on the delay between the lime-soil mixing)



pH vs. Lime Content

Figure 4.0 A

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In the case of stabilization, the Unconfined Compression Test (AASHTO T 208) and California Bearing Ratio (AASHTO T 193, soaked) or resilient modulus (AASSHTO T 307) tests at 95%compaction shall be performed in addition to the above tests corresponding to optimum lime-soil mixture of various predominant soils types.

Cement Stabilization

The criteria for cement percentage required for stabilization shall be as follows. The following methodology shall be used for quality control and soil-cement stabilization.

1. Perform the mechanical and physical property tests of the soils.
2. Select the Cement Content based on the following:

AASHTO Classification	Usual Cement Ranges for Stabilization (% by dry weight of soil)
A-1-a	3 – 5
A-1-b	5 – 8
A-2	5 – 9
A-3	7 – 10

Suggested Cement Contents

3. Perform the Standard Proctor on soil-cement mixtures for the change in maximum dry unit weight in accordance with AASTO T 134.
4. Perform the unconfined compression and CBR tests on the pair of specimens molded at 95% of the standard Proctor in case of stabilization. A gain of 100 psi of cement stabilization is adequate enough for stabilization and % cement shall be adjusted. Although, there is no test requirement for the optimum cement content when using cement to modify the subgrade. An amount of cement 4% ± 0.50% by dry weight of the soil should be used for the modification of the subgrade.

THERMAL STABILASTION

iv) *Soil Stabilization by Thermal Process :* It is described above that the method of burning moulded articles of soil was perfected from Vedic period by Indians. Sankalia[1] had stated that from protohistoric period the method of manufacture statues by *cire perdue* technique was known to Indians. The technique requires a mould with inner and outer layers of soil with cavity in between. The outer layer of the mould was prepared of stabilized soil. It is described that for preparing statues of copper, silver and gold from such a mould, the soil to be used should be mixed with rice husk, cotton and salt and must be ground thoroughly. After three days the outer part of the mould is prepared of this stabilized soil[24].

For melting metals like gold, silver, iron etc., it is necessary to place them in a capsule (*mūṣa*). This is placed in a furnace. These capsules were prepared of stabilized soil such that it can bear the intense heat of a furance. More information on this aspect of soil stabilization can be had from Satya Prakasha[25].

GROUTING

The soil improvement techniques are effective for each of the allowed or required disturbance of existing structures. The following methods, which imply a low level of vibration, are useful to improve liquefiable ground by solidification:

(a) Compacting grouting ;

(b) Permeation grouting ;

(c) Jet grouting;

Compaction Grouting-

Compaction grouting is a soil injection with low workability cement paste that remains homogeneous without entering in the soil pores. The cement mass extends, soil is moved and finally compacted. The liquefaction improvement using compaction grouting divides into the following categories:

(a) Treatment under existing structures;

(b) Treatment in urban areas with low levels of vibration and noise;

(c) Treatment in narrow areas.

The execution of compaction by injection technology using bottom-up method takes place as follows. In the first stage, injection pipes set up on the foundation soil of the existing or future foundations using drilling machines. The injection process begins. Mixture injected through the pipes pushes the surrounding soil; then the injection pipes raises about 0.3-1.5 m and the process renews. The "in steps" injection process continues until the whole thickness of the soil layer is treated. Injection stabilizes soil layer by density and pressure increasing. The injection process is used when a controlled lifting of the soil surface or existing structures affected by local settlements are necessary (Plescan & Rotaru, 2010; Morales & Morales, 2003; Welsh et al, 2002).

Orense *et al.* (2010) reported that Compaction grouting involves the injection of a very stiff grout (soil-cement-water mixture with sufficient silt sizes to provide plasticity, together with sand and gravel sizes to develop internal friction) that does not permeate the native soil, but

results in controlled growth of the grout bulb mass that displaces the surrounding soil. The primary purpose of compaction grouting is to increase the density of soft, loose or disturbed soil,

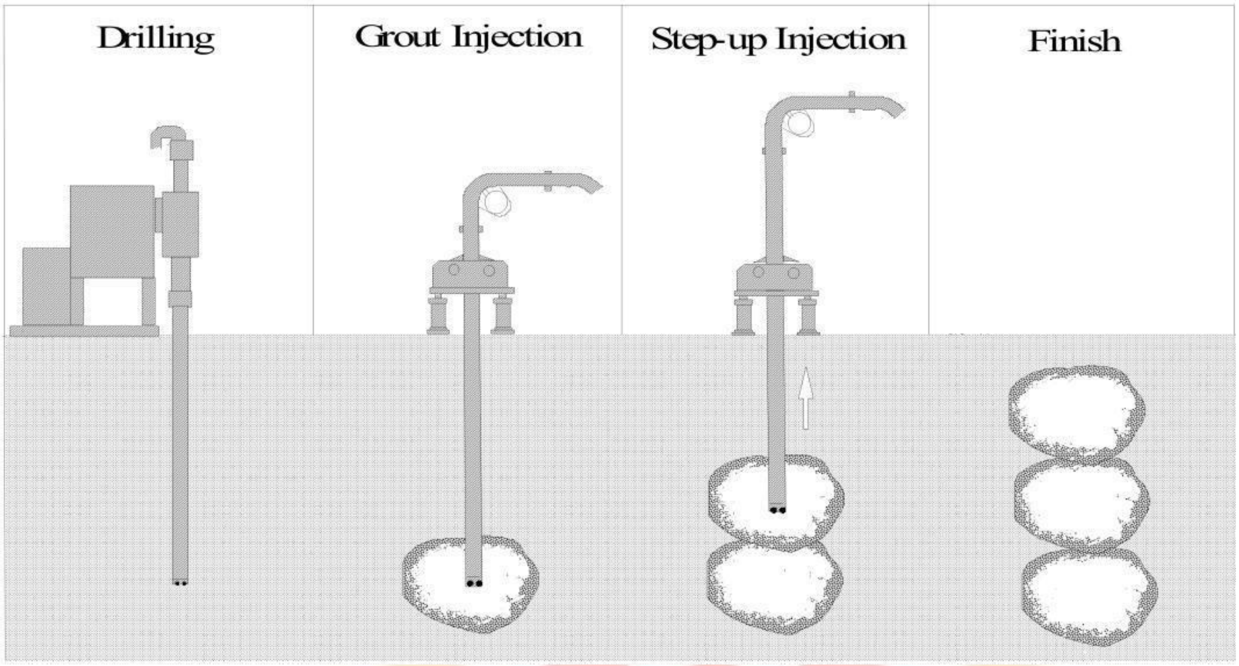
typically for settlement control, structural re-leveling, increasing the soil's bearing capacity, and mitigation of liquefaction potential.

Also Orense (2008) had a review of two case histories on the application of compaction grouting as liquefaction remediation was presented. One case involved the implementation in an open unrestricted space such as airport runways, while the second one was under an existing

manufacturing plant. Based on the discussion, several important observations regarding the effectiveness of the technique were addressed. The post-treatment data suggested that compaction grouting was capable of producing the improvement in SPT resistance required to mitigate

liquefaction risk. The method of construction, whether "bottom-up", "top-down" or combination of the two, affected the level of effectiveness and the resulting ground heave. The method was most effective on sandy soil with fewer fines content. In addition, compaction grouting also

increased the strength and the lateral earth pressure of the ground. This method is shown



Compaction grouting implementation (Orense, 2008).

Permeation Grouting-

Permeation grouting consists of the injection of a low-viscosity fluid in the soil pores without changes in the soil physical structure. The main goal of permeation grouting is both to strengthen soils through particle cementation (to stabilize the links between particles) and to waterproof ground by filling its pores with injected fluid. This method improves the soil physical and mechanical characteristics, successfully stabilizes the excavation walls in soft soils, controls the groundwater migration in order to implement the underpinnings at the existing foundations and prevents the effects of earthquakes – compaction and soil liquefaction. Permeation grouting is a technology used to mitigate liquefaction that is suitable for un-compacted soils solidification in order to reduce the risks of compaction and liquefaction that may occur as result of possible earthquakes (Plescan and Rotaru, 2010).

The process is quiet flexible and it can be designed with a minimal disruption at the surface and therefore, it is advantageous for use in urban areas or areas with limited access. During grouting process, injection pressures Based on the field trials and the soil conditions, the injection pressures and the grout volumes will be justified to meet the intended performance.

Particulate grouts (e.g. cement or bentonite) are generally used for medium to coarse grained sands, such that the particles in the grout easily percolate through the formation. Micro fine cement is also used for fine grained sands where Ordinary Portland Cement cannot percolate through the formation. Chemical grouts (e.g. silicates) are used in formations with smaller pore spaces, but are limited to soils coarser than fine grained sands. The process of permeation grouting is schematically shown in Figure 2. Quality Control & Quality Assurance Like any other grouting improvement process, the quality control during permeation grouting is very important to ascertain the effectiveness of the technique. As such, the process parameters such as grout are usually limited to prevent fracturing or volume change in the natural soil/rock formation. injection pressures and the grout volumes will be justified to meet the intended performance.

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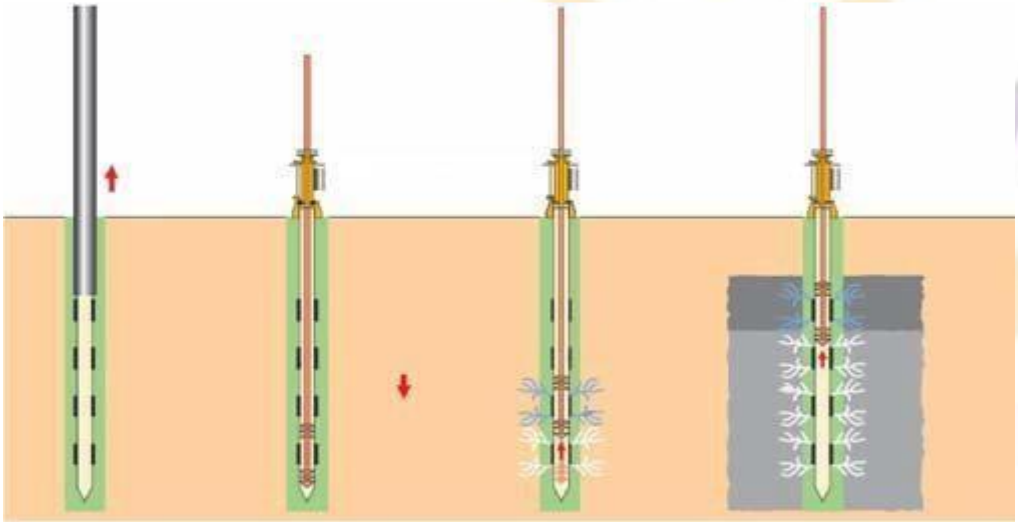
grouting improvement process, the quality control during permeation grouting is very important to ascertain the effectiveness of the technique. As such, the process parameters such as grout

pressure, flow rate, volume of grout for corresponding depth are monitored throughout the construction process. Post construction in-situ permeability tests are conducted after sufficient curing period to validate the effectiveness of permeation grouting (Raju & Valluri, 2010).



Schematic showing process of Permeation Grouting(Raju & Valluri, 2010).

A newly permeation grouting technique using a colloidal silica has been developed to prevent the liquefaction of sandy ground beneath existing structures. In the case of cyclic torsional shear test with the treated sand, a relatively large strain is developed in the early stage of loading;

however, both the development of shear strain and the decrease of mean effective stress resulted without any collapse and liquefaction also the remarkable improvement of cyclic shear strength by the colloidal silica treatment can be exhibited. This phenomenon leads to the expansion of

dilation region across which it faced the phase transformation line and the failure line. (Ohno et al, 2005)

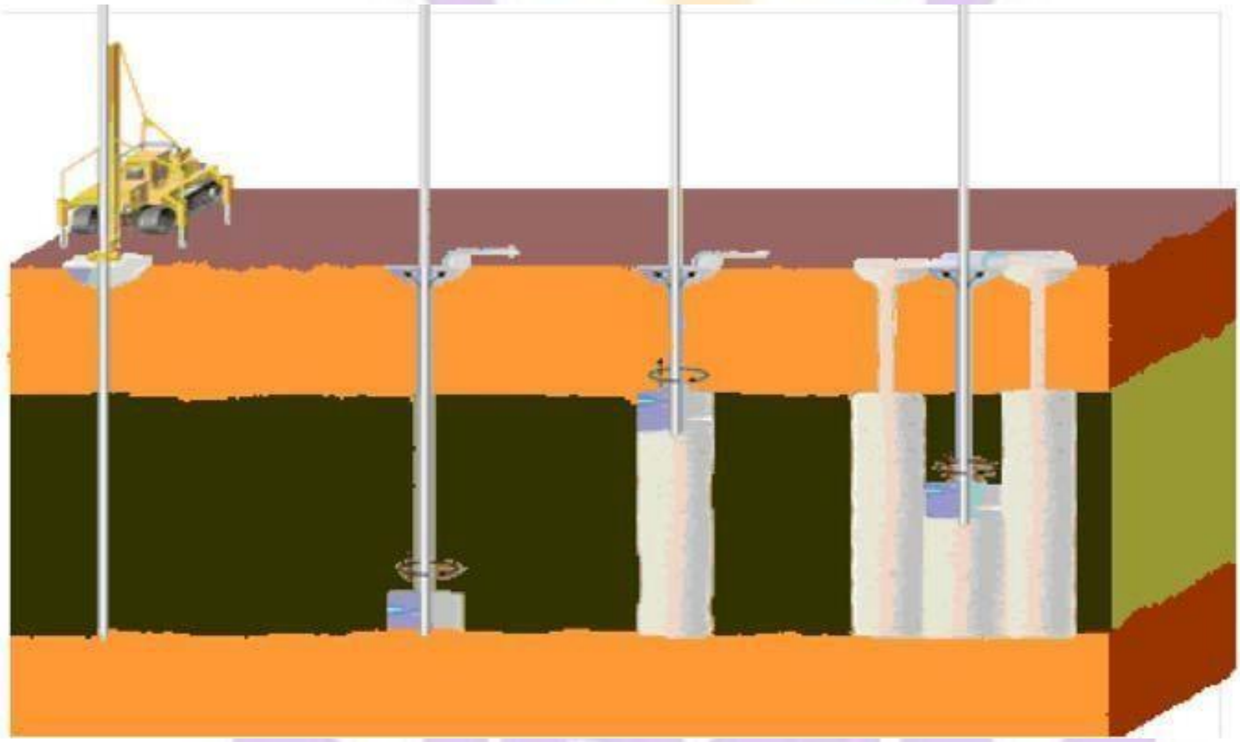
In order to study the deformation and the strength characteristics of the treated sand with the colloidal silica, a series of laboratory tests has been performed on the treated sand such as monotonic and cyclic torsional shear tests. In the case of cyclic torsional shear test with the

treated sand, a relatively large strain is developed in the early stage of loading; however, both the development of shear strain and the decrease of mean effective stress resulted without any collapse and liquefaction. As the results, the remarkable improvement of cyclic shear strength by

the colloidal silica treatment can be exhibited. This phenomenon leads to the expansion of dilation region across which it faced the phase transformation line and the failure line. When the performance based design for the silica-treated ground are carried out, it is necessary for the dynamic deformation analysis to produce the appropriate constitutive model for the treated sand

based on the test results (Oka et al. 2003). Their report indicates that the improved sand exhibited greater liquefaction strength curves. Based on the results of laboratory studies, a cyclic elastoplastic model has been proposed in order to describe the behavior of improved sand. Then, the cyclic elasto-plastic model has been implemented into the fully coupled effective stress based FEM program to perform numerical analysis of liquefaction of improved ground. It has been revealed that the improvement by permeating grouting method is very

effective in increasing the
resistance against liquefaction.
Jet Grouting-
Applications of the jet grouting system fall into three broad categories: underpinning or excavation support, stabilization of soft or liquefiable soils, groundwater or pollution control. The method consists of soil injection of a mixed fluid at high pressure forming jets that erode and
replace the existing soil with the injection mixture. In general this method begins by drilling
small-diameter holes (90-150 mm) up to the final injection depth. Cement mixture is injected into the soil with a metal rod that runs a rotational and withdrawal motion whilst. This technology is useful to underpinnings of existing foundations, to support excavations in cohesive less soils, to
control the groundwater migration and to improve the strength of liquefiable soil (plescan & rotaru, 2010; Geotechnical news, 2008). This method is shown in the Figure .



Jet grouting method (Plescan & Rotaru, 2010). Cooke (2000) studied the use of jet grouting under an embankment slope at existing highway
bridges to mitigate the risk of earthquake-induced liquefaction damage. The jet-grouted zone helped to limit movements of the abutment by containing and limiting the shear deformations that occurred in the liquefiable soils under the embankment that were softened due to the development
of excess pore water pressures during shaking. The limitation of the deformations was dependent on the strength and stiffness of the jet-grouted zone, which in the cases evaluated did not fail, and its ability to resist the increased overturning forces during shaking. The performance of a jetgrouted
zone is highly dependent on its strength. The strength assumed for the jet grouted material was high and resulted in no material failure during shaking (Geotechnical news, 2008).
Also Geotechnical News (2008) reported from Olgun (2003), the soils were improved to
increase bearing support for shallow foundations and to reduce liquefaction potential of the sand layers. Surcharge fills with wick drains were used to improve the soft clays, and jet-grouted columns were used to provide increased bearing support in the clays and prevent liquefaction of

the loose sands. Jet-grouted column spacing and diameters were selected on the basis of footing spacing, footing loads, floor slab loads, and judgment. A primary and secondary grid of columns was installed in a rectangular pattern to provide blanket treatment. In addition Olgun showed that

jet grouted columns do not stiffen the ground by attracting the seismic shear stresses. they do not reduce shear stresses on the soft ground. They may still act as vertical support if there is enough bearing capacity and side resistance from layers that did not liquefy. It is possible rely

on jet grouted columns to provide bearing support and reduce settlements if liquefaction is limited to a specific zone. It is clear that there could be liquefaction mitigation only if the entire liquefiable zone is treated. A total replacement of potentially liquefiable material by jet grouting,

will avoid the liquefaction likelihood.

Yilmaz *et al.* (2008) performed a study on the soil improvement in Beydag dam against liquefaction of alluvium at the dam site. Peak acceleration on rock was estimated to be 0.32 g for an earthquake having magnitude of 7. Liquefiable soils, which consisted of two separate layers of diatomaceous silt and one layer of volcanic ash beneath the downstream toe of Wick up Dam,

were stabilized using 4.3 m diameter jet grouting columns. These liquefiable strata extended to depths up to 26 m. The dam had a square grid of intersecting jet grout piles at the downstream side of upstream wall having an area replacement ratio of about 10%. Depending upon the shear

modulus ratio, G, between jet grouted column and soil, it was found that stress reduction coefficient changes with area replacement ratio. Cyclic stress ratio (CSR) after ground improvement is calculated by multiplying stress reduction coefficient with CSR before treatment. Thus, it was possible to calculate the area replacement ratio required to reach the intended factor of safety. It was found that 10% area replacement ratio may reduce CSR at least about 50%.