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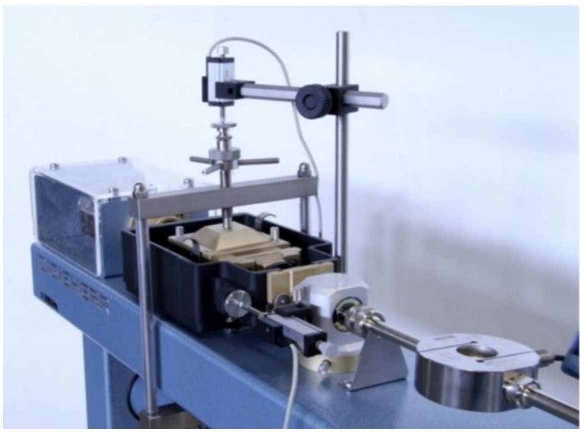
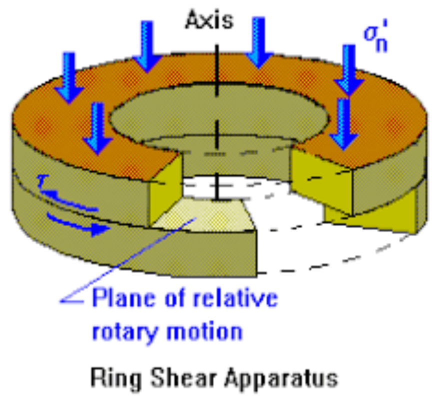
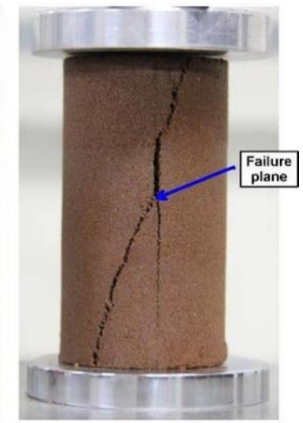
College of Engineering

Department of Civil and Architectural Engineering







Foundation Engineering I (CVEN 330) – Fall 2020

Shear Strength of Soil – Lab Measurement

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Shear Strength of Soil – Lab Measurement

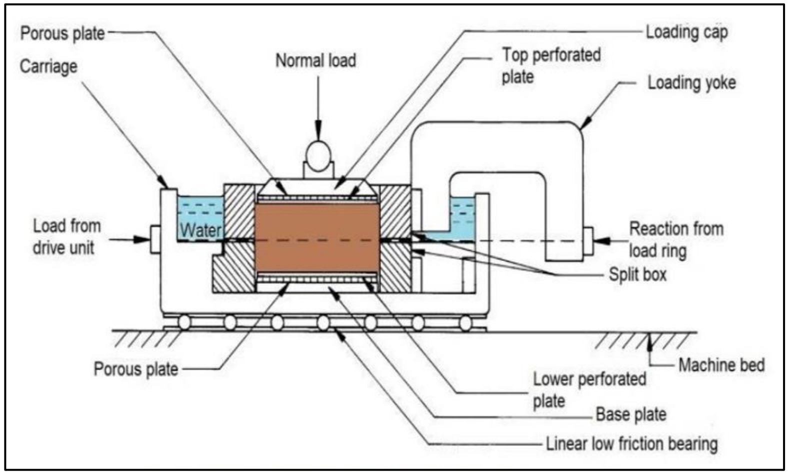
All laboratory tests for determining stress, strain, and strength of soils are conducted in **two steps**. In the first step, an initial condition, preferably similar to the preconstruction condition existing in the field, is established in the soil specimen. The second step in the testing procedure is intended to simulate or represent a construction-related process that subjects the soil to changes in shear stresses.

Various modes of shear that may be encountered under field conditions are approximated by using different types of apparatus and methods of application of shear stress. **Either a deformation is imposed** (often at a constant rate) on the specimen and the resulting changes in stress are measured, **or increments of stress are imposed** on the specimen and deformations are measured. A test of the former type, which is carried out under *strain control*, is more convenient, but the latter, which is carried out under *stress control*, is more realistic.

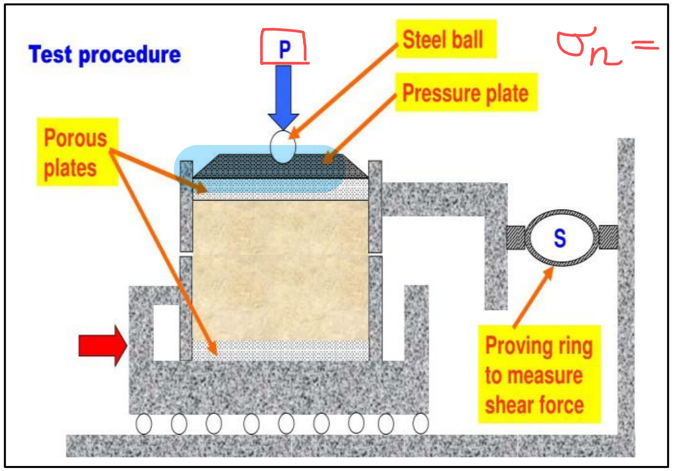
The following is a brief description of the equipment and procedure for lab tests that are widely used for investigating the shear strength of soils.

1- Direct Shear Test

The oldest method for investigating the shearing resistance of soil is the *direct-shear test*. The apparatus consists of an upper box that is stationary and a lower one that can be moved in a horizontal direction. The specimen is located between two porous stones (perforated plates) that serve as drains during the first and second steps of the test. Because the specimen is confined by the rigid upper and lower shear boxes, volume changes of the specimen during both the first and second steps are measured by a vertical deformation dial gauge in contact with the upper porous stone.



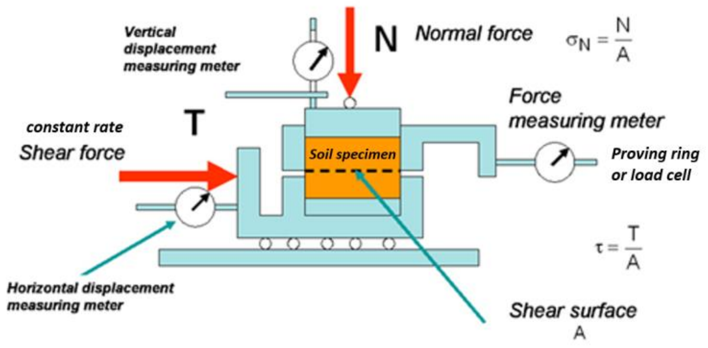
Typical setup for a direct shear test



$\sigma_n = P/A$

Simplified schematic for direct shear box

In the first step, a vertical load N (i.e., stress $\sigma_n = \dfrac{N}{A}$) is applied to the upper stone (loading cap) and is maintained long enough that the vertical load is carried entirely by effective vertical stresses within the specimen. This accomplished by reading the vertical deformation (vertical displacement) dial with time and interpreting the progress of consolidation as for an oedometer (consolidation) test.



Normal and shear stress application in a direct shear test

In the second step, the lower box is subjected to a constant rate of horizontal displacement, and the imposed horizontal shear force T (i.e., shear stress $\tau = \dfrac{T}{A}$) is measured by a proving ring or load cell that keeps the upper box stationary. The lower box is displaced at such a rate that no appreciable shear-induced pore water pressures develop during the second step. The shear-induced volume changes are measured with the vertical deformation dial gauge.

As explained before, drained tests (e.g., direct shear test) are usually carried out on soils that in the field respond in *drained* fashion to typical natural and constructed-related shearing process. For example, permeable granular soil, with the exception of saturated sands subjected to dynamic loading, display a drained response to most natural and man-made events. Critical instability conditions in stiff clays and shales also develop most often in a drained condition. Therefore, drained direct shear test is appropriate for granular soils and stiff clays and shales.

Silt Sand Gravel

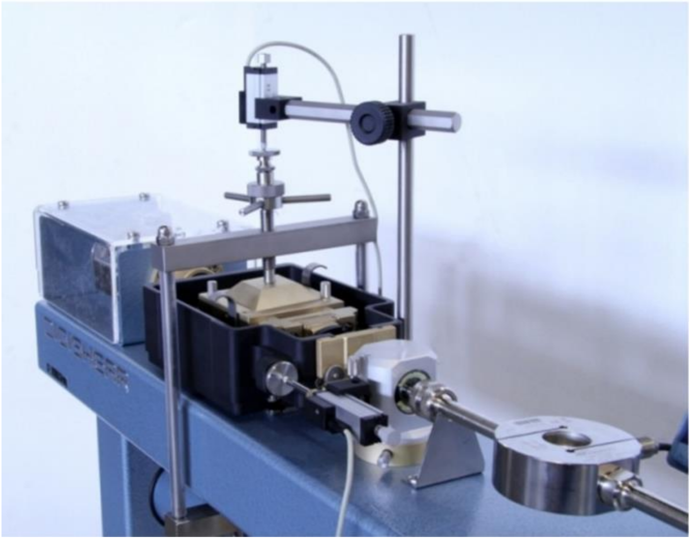
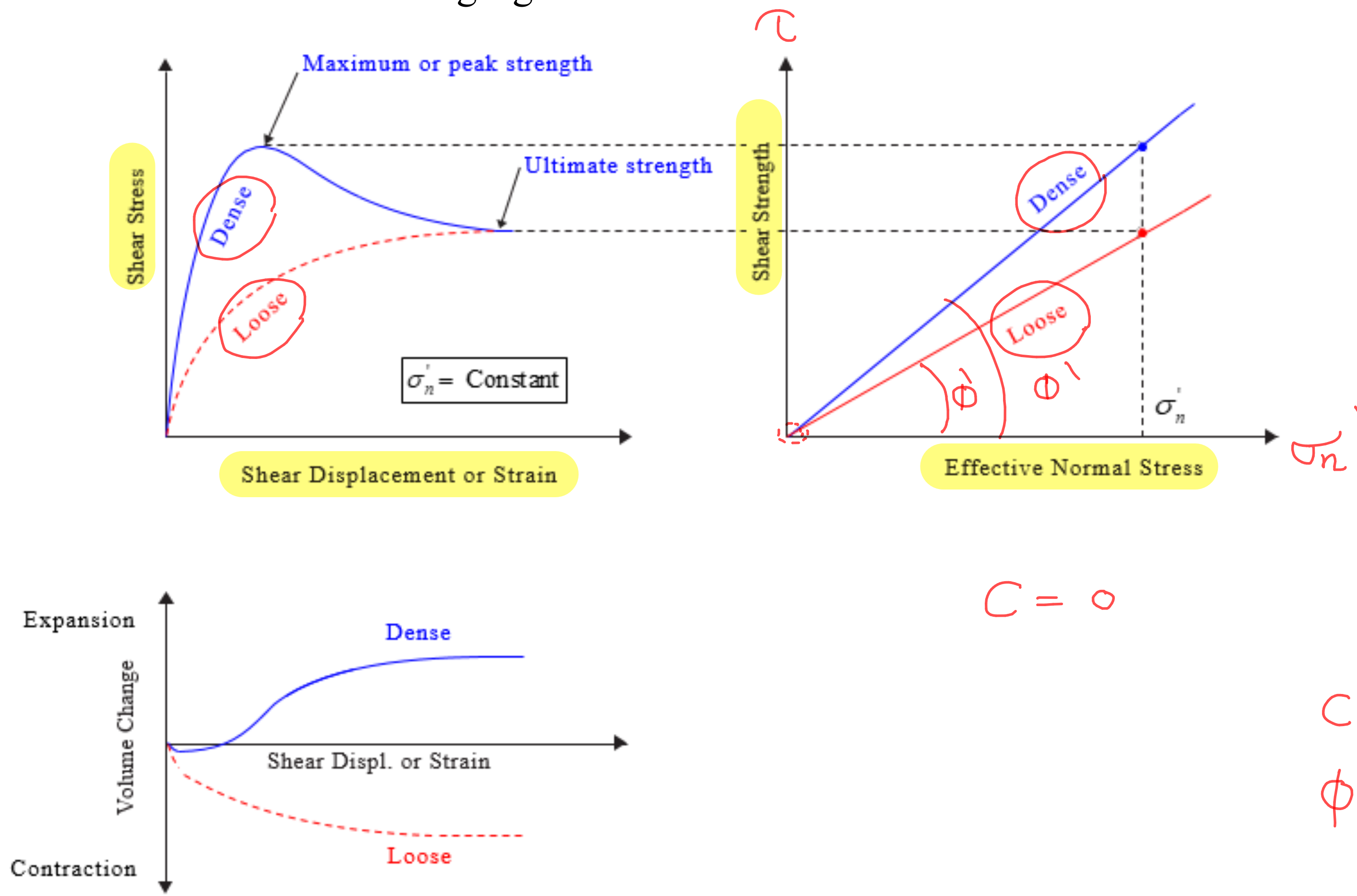


Photo of specimen container, carriage, loading yoke, and force and displacement meters in a direct shear test

1.1 Drained shear strength behavior of saturated granular soils

Sand

Shear strength behavior that is typically yielded from drained direct shear tests on saturated granular soils is shown in the following figure.



C = 0

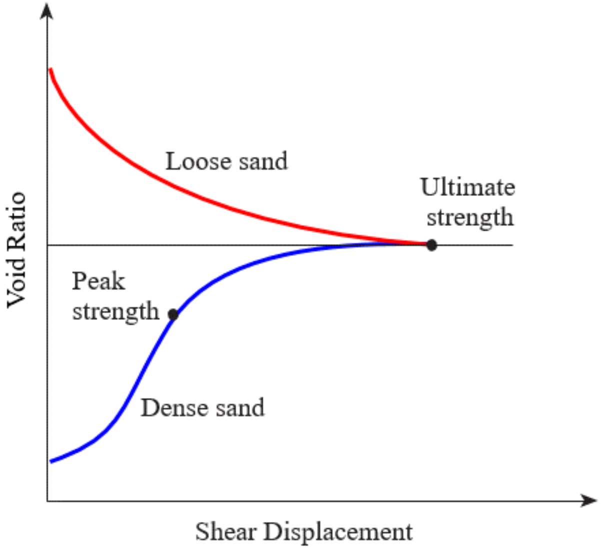
C

φ

Typical drained shear strength behavior of saturated granular soils

It should be noted that the shear behaviour for the same granular soil depends on the initial void ratio or relative density. The loose sample experienced large compression whereas the dense sample, after a small decrease in volume, dilated strongly (due to particle interlocking) with the net volume increase at both maximum (peak) and ultimate stresses. In both samples, the rate of volume increase with respect to horizontal strain was maximum at the maximum stress and was almost zero at the ultimate condition.

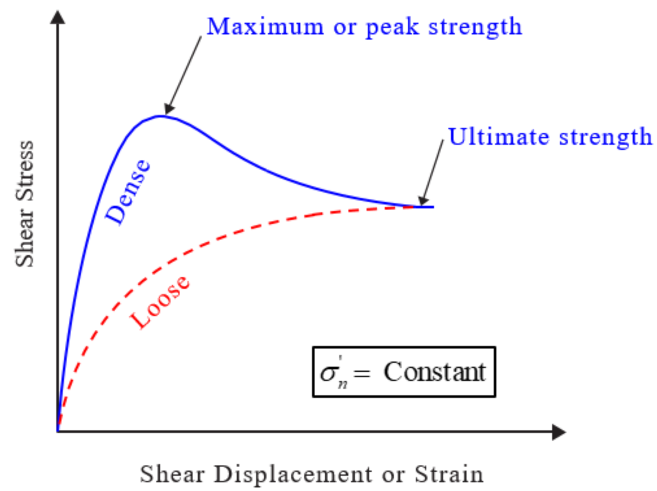
As shown, dense sand tends to dilate when sheared, whereas loose sand tends to contract until both of them reach a certain void ratio (critical void ratio) at which no volume change occurs.



Change in void ratio of sand with shear displacement

Notice that the maximum shear strength of a loose sand is equal to the ultimate shear strength of that sand when sheared at a dense condition. Therefore, the ultimate strength is independent of the initial relative density.

It should be also noted that the mechanisms of shear resistance generation in soils composed of **dry** clay minerals are almost identical to those generating shearing resistance in granular soils. The only difference is that we do not consider dry clays. We are usually concerned with the shearing resistance of clay-water systems.



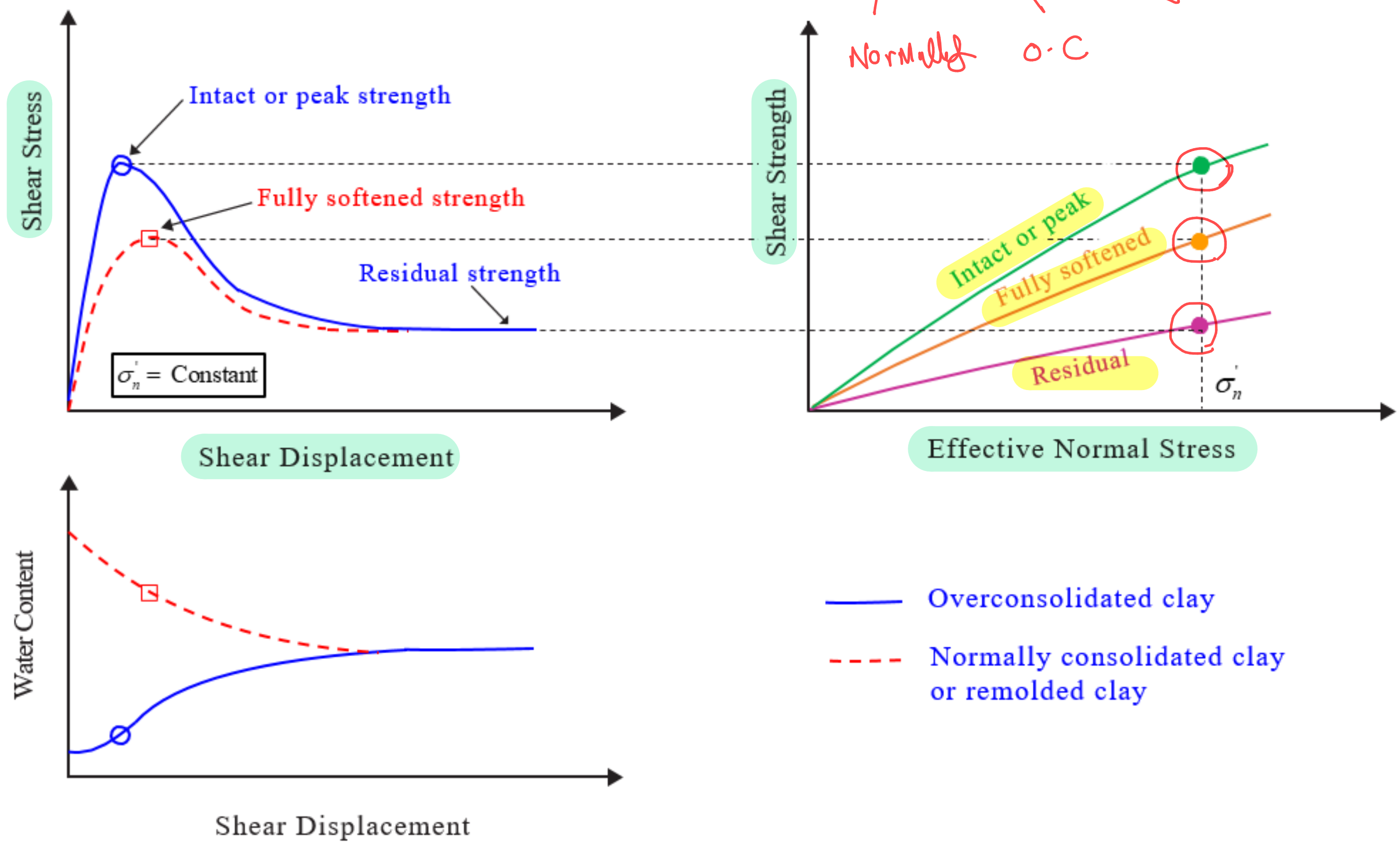
1.2 Drained shear strength behavior of stiff clay

intack

Fully softened

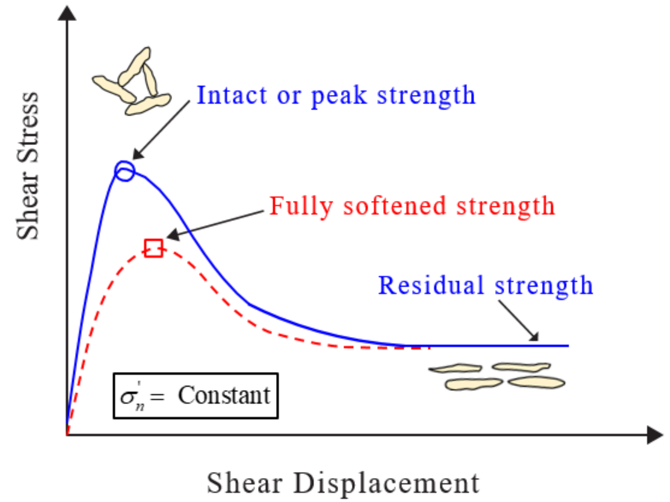
Shear strength behavior that is typically yielded from drained direct shear tests on stiff clays and shales is shown in the following figures.

Normally O.C



Typical drained shear strength behavior of plastic clays

By the time of the peak shear strength is mobilized, there is little change in structure of the overconsolidated clay. After **peak** (or intact) strength, fissures, joints open up and diagenetic bonds breaks and clay particles disaggregate and take up water (adsorbed double layer and free water). As a result, the overconsolidated clay softens. By the time most of the softening is completed, a **fully softened** strength is reached.



If the shearing process is continued, the shearing resistance continues to decrease until finally a **residual** strength is reached. The loss in strength between fully softened and residual is brought about primarily by the orientation of the platy clay particles along the shear plane and therefore, by a reduction in edge-to-face interaction among the clay particles.

It should be noted that the fully softened shear strength of overconsolidated clay is approximately equal to the maximum or peak shear strength of the same clay when sheared after complete remolding. The relative drop from the peak strength to residual strength increases with increasing plasticity.

Advantages and disadvantages of Direct shear test

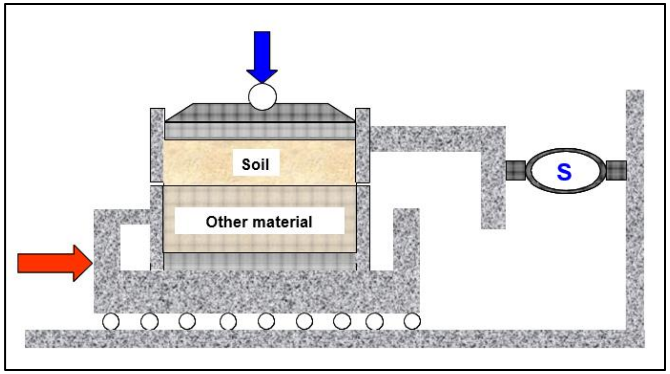
Advantages:

i) Relative simplicity and low cost.

ii) Clay or shale specimen can be oriented to measure the shear strength a long a plane of weakness.

iii) Shearing resistance after large shear displacement can be measured by using the reversing direct shear method in which the direction of the shearing is reversed several times to accumulate large shear displacement needed to mobilize residual strength. [Note: typical direct shear box has a maximum travel (forward and reverse) of 10 mm]

iv) Suitable for interface testing.

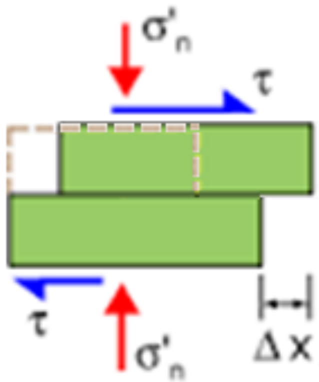


Disadvantages:

i) The change in area of the surface of sliding as the test progresses. This problem is minimized by restricting the shearing displacement (Δx) in each direction to about 5mm.

ii) Shear failure doesn't take place simultaneously at every point of the potential surface of sliding. Progressive failure starts at the two edges and proceeds toward the center. Therefore, the peak value of the shearing resistance indicated by the test results is lower than the real peak value.

iii) Failure occurs along predetermined failure plane (i.e., does not simulate the field condition).



Numerical Example #1:

Direct shear tests were performed on a dry sand. The specimen cross section was 60 mm × 60 mm. Test results are given in the following table

Test	Normal force (N)	Shear force at failure (N)
A	180	109
B	360	227
C	540	328

Determine graphically the friction angle of the tested sand (ignore the effect of changing sheared areas during test)

Solution

For Test A:

$$\sigma'_N = \frac{180}{60 \times 60} = 0.05\ \overbrace{N/mm^2}^{MPa} \xrightarrow{10^3} = 50\ kPa$$

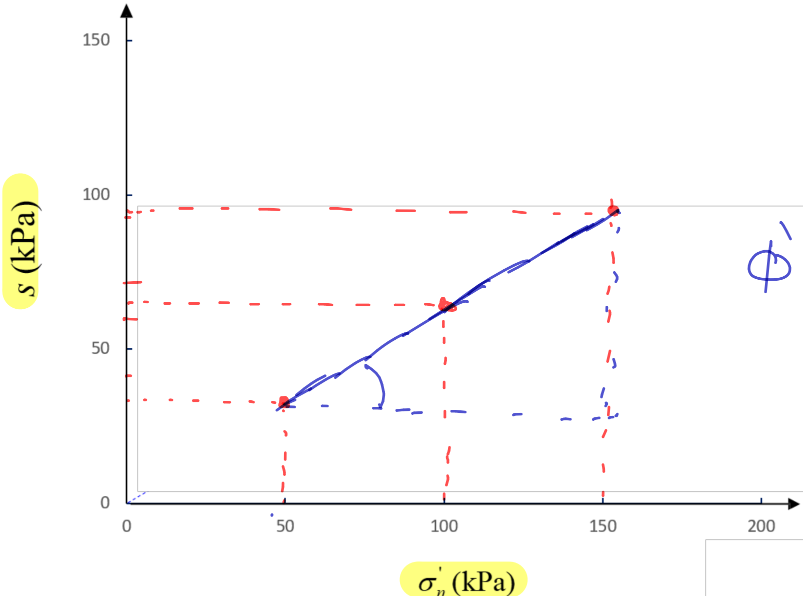
$$S = \frac{109}{60 \times 60} = 0.03\ N/mm^2 = 30\ kPa$$

Solution (Continued)

Test B $\quad \sigma_n' = \frac{360}{60 \times 60} = 100 \text{ kPa}$

$$s = \frac{227}{60 \times 60} = 63.1 \text{ kPa}$$

Test C $\quad \sigma_n' = 150 \text{ kPa} \qquad s' = 91 \text{ kPa}$

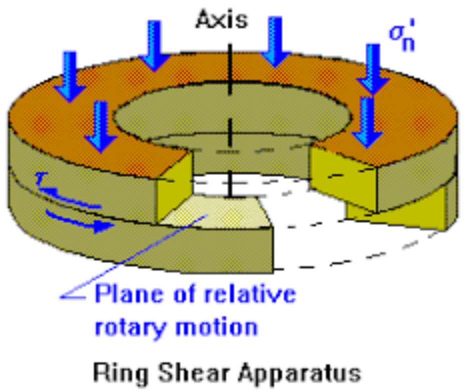


$$\phi' = \tan^{-1} \frac{91 - 30}{150 - 50}$$

$$= 31.4^\circ$$

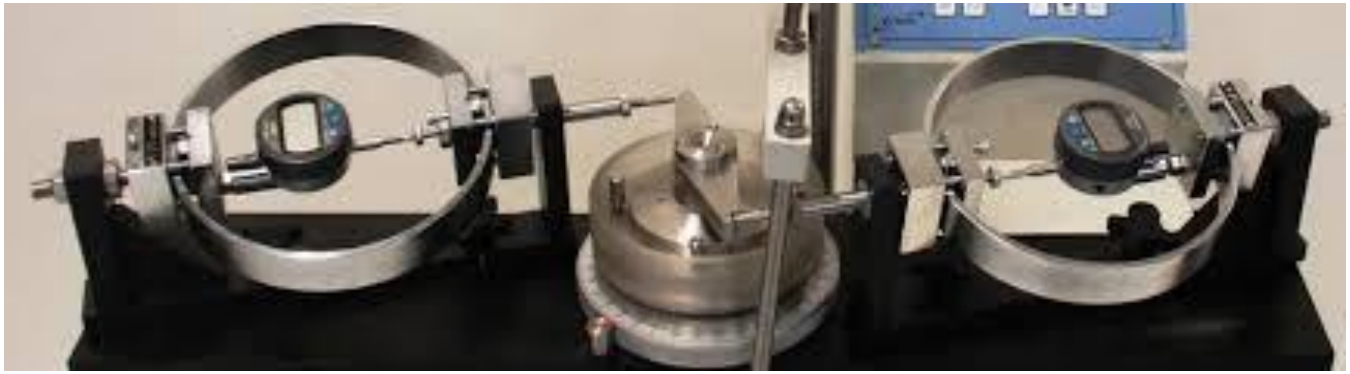
2- Torsional Ring-Shear Test

The reversing direct shear test leaves open the question whether the effect of many back-and-forth displacements is indeed equivalent to a unidirectional displacement of the same total magnitude. The torsional ring-shear apparatus has been devised to permit shear-stress measurements over unlimited displacements. In this test, the soil specimen has the shape of a ring with a rectangular cross-section. The inside and outside diameter of the specimen are confined by metal rings, and the top and bottom are in contact with annular porous plates with sharped fins to minimize slip of soil-porous plate interface.



Schematic 3D view of soil specimen of the torsional ring-shear apparatus

In the first step, the specimen is subjected to an effective vertical stress. In the second step, the specimen is sheared by rotating the lower half while the upper half reacts against a torque arm, held in place by a proving ring (or load cell) at each end, that measures the tangential load. Failure occurs on the horizontal plane that passes through the boundary between the upper and lower confining rings. The average shear stress on the failure surface can be calculated with the knowledge of the torque applied to the upper annular porous plate.



Torque arm held in place by two proving rings in torsional ring-shear apparatus

Torsional ring-shear apparatus is mainly suited for measurement of the drained residual shear strength of clays and shales. Its only advantage over the direct shear test is that uninterrupted shear displacement of any magnitude can be readily achieved on the horizontal slip surface and without change in the area of such surface. Its disadvantages include most of those of the direct shear apparatus.

The apparatus is frequently used to measure the peak strength of remolded clay (i.e., fully softened strength). Results of such test type are very comparable to those of the direct shear test and lead to measured fully softened friction angles that are 2° to 3° less than those measured using the Triaxial compression test. This slight disagreement is basically due to having different modes of shear.

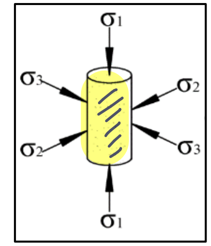
Conslidation σ_3

3- Triaxial Test

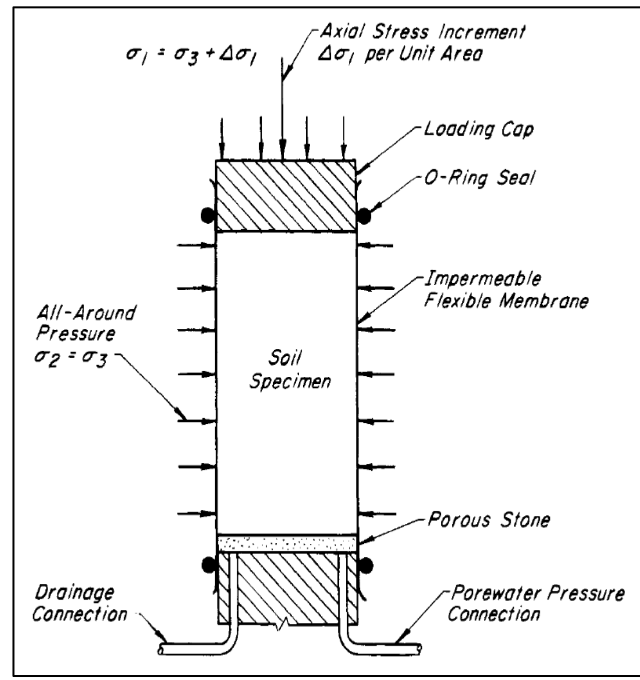
$\Delta\sigma_1 \Rightarrow$ vertical

$\sigma_1 = \sigma_3 + \Delta\sigma_1$

In a triaxial test a cylindrical specimen of soil typically 38mm or more in diameter and with a height to diameter ration of 2, is subjected to an equal all-round pressure (σ_3) known as the *cell pressure*, in addition to an axial stress $(\sigma_1 - \sigma_3)$ or $\Delta\sigma_1$, known as the *deviator stress*, that may be varied independently of the cell pressure. The axial stress may be positive or negative. If it is positive, the test is a *triaxial compression test*, if negative, a *triaxial extension test*. The latter test type is outside the scope of this course.

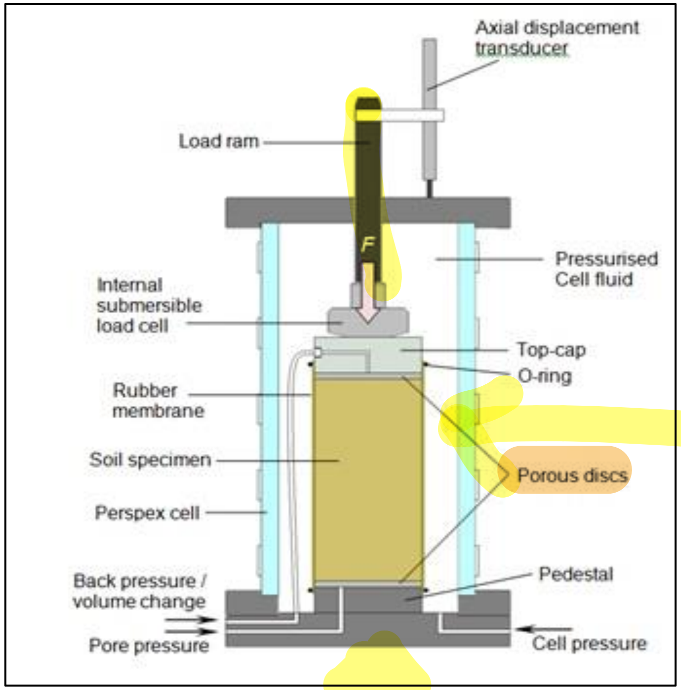


Triaxial stresses on soil specimen



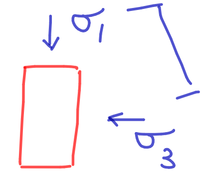
Soil specimen in a triaxial test

The cylindrical surface of the specimen is covered by a rubber membrane sealed by rubber O-rings to a pedestal at the bottom and a cap at the top. The assemblage is contained in a chamber into which cell fluid may be admitted under any desired pressure; this pressure acts laterally on the cylindrical surface of the specimen through the rubber membrane **and** vertically through the top cap. The additional axial stress is applied by means of a piston passing through a frictionless bushing at the top of the chamber.



Basic features of typical triaxial apparatus

Photo of the triaxial test apparatus



3.1 Types of Triaxial Compression Tests

In triaxial apparatus, a porous disk is placed against the bottom of the specimen and is connected to the outside of the chamber by tubing. By means of this connection, the pressure in water contained in the pores of the specimen can be measured if drainage is not allowed. In the first step of triaxial test (consolidation stage), the soil specimen is subjected to a uniform all-around cell pressure (σ_3). In the second step (shearing stage), the specimen is subjected to increasing axial stress ($\Delta\sigma_1$) until failure. The types of triaxial tests can be defined based on the allowance of water drainage [i.e., turning on or off the drainage connection (the pore pressure connection)] valve as follows:

a- Consolidated-Drained Test (CD Test): Valve is ~~off~~ on during the 1st and 2nd steps. (No − u_F)

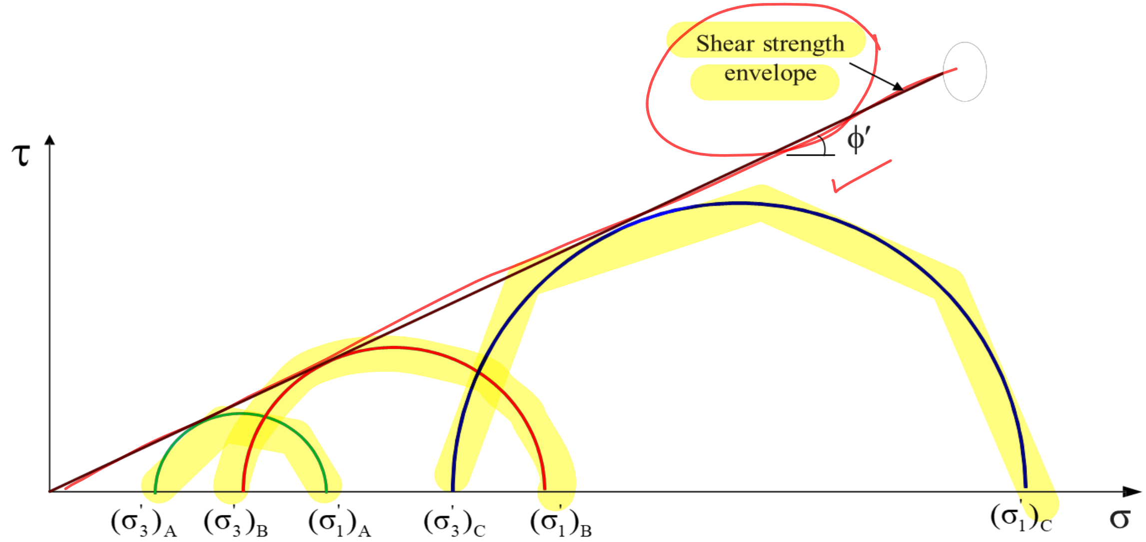
b- Consolidated-Undrained Test (CU Test): Valve is on during the 1st step and off during the 2nd step. u_F

c- Unconsolidated-Undrained Test (UU Test): Valve is off during the 1st and the 2nd step.

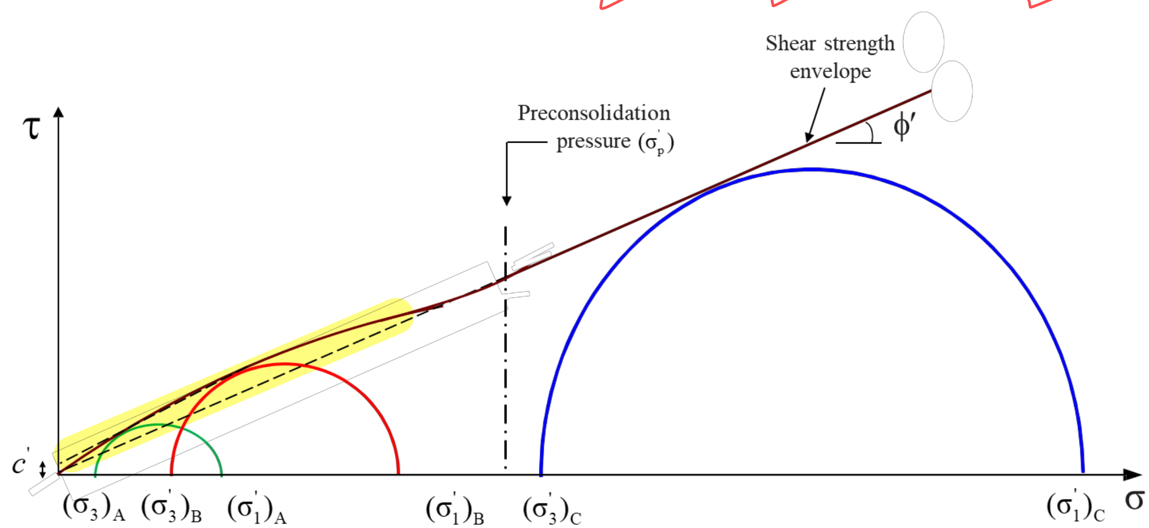
3.1.1 CD Tests

By definition, pore water pressure is not allowed to be developed either during the consolidation stage or during the shearing stage. Loading during both stages is to be applied slowly to assure having zero pore water pressures. As a result, the **total** principal stresses at failure (σ_1 and σ_3) will be the same as the **effective** principal stresses (σ_1' and σ_3').

It is a common practice to conduct the test on three different specimens of the same soil (σ_3 will be different for each test and consequently different σ_1 is needed to cause failure). Mohr's rupture circles and the associated shear strength envelope can then be drawn to subsequently determine the shear strength parameter(s).



Schematic of typical CD triaxial test results for sand, remolded clay, or normally consolidated clay



Schematic of typical CD triaxial test results for stiff clay tested at stresses across the overconsolidated and normally consolidated ranges

3.1.2 CU Tests

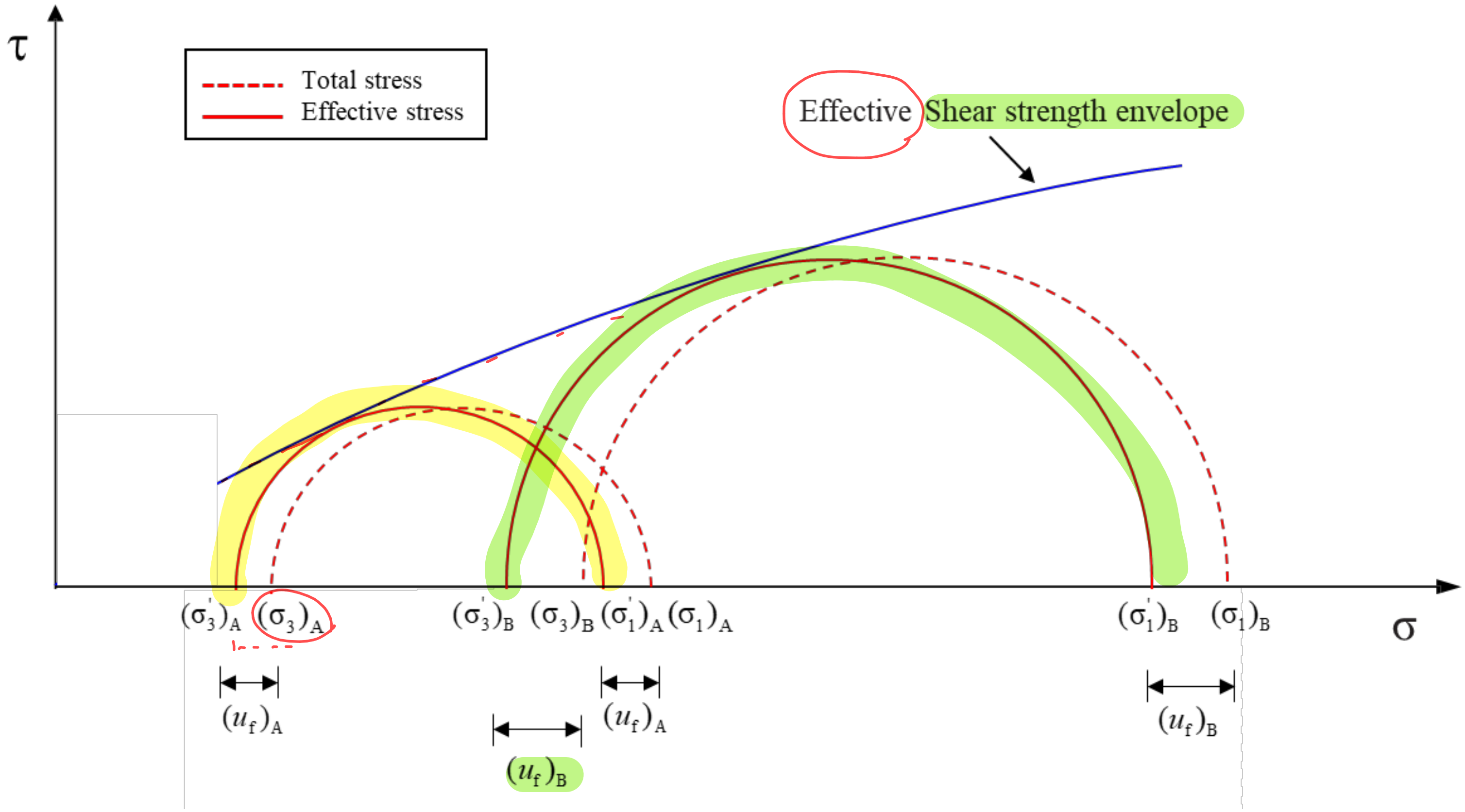
Drainage is allowed during the consolidation stage (i.e., step #1) while it is not in the shearing stage. As a result, step #2 of this test can be faster than that of the CD test.

$\therefore \; \sigma_3' \neq \sigma_3$

and $\sigma_1' \neq \sigma_1$

$\sigma_1' = \sigma_1 - u_F$

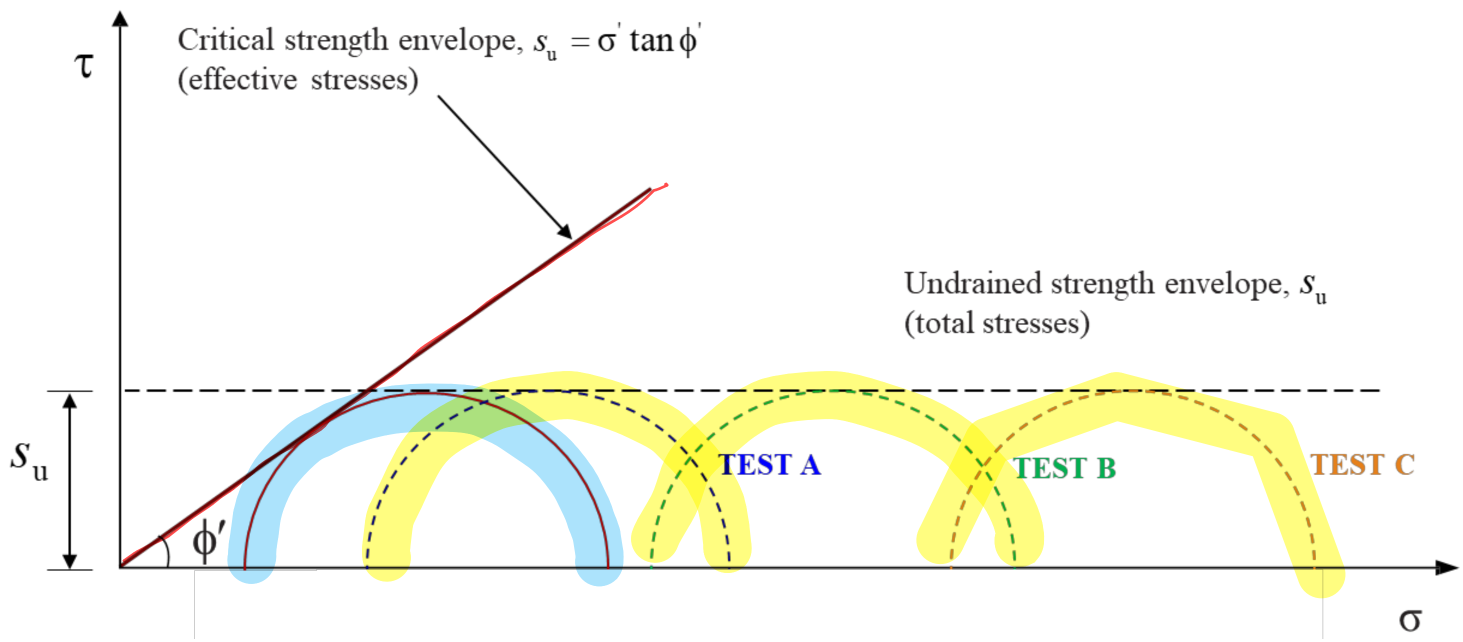
By measuring the shear-induced pore water pressure at failure (u_f), we can calculate $\sigma_1' = \sigma_1 - u_f$, and then draw Mohr's circle using the effective principal stresses. The effective stress circles will have the same diameter as those of the corresponding total stress circles but shifted to the left (on the $\tau - \sigma$ plot) by the magnitude of u_f for each test. The shear strength failure envelope should be drawn (defined) as the tangent of the **effective** stress circles.



Note : Effective shear strength envelopes similar to those yielded from the CD test can be also developed using the effective shear stresses calculated based on the CU test results of the same soil.

3.1.3 UU Tests

No drainage is permitted from the specimen under the influence of either the confining pressure or the axial stress. As a result, pore water pressures at failure (u_f), that may be measured or not, will be due to imposing both of the confining pressure and the axial stresses. This leads to **several total stress** Mohr's circles of failure and **one effective stress** Mohr's circle as shown in the following figure.



Schematic of UU test envelopes for normally consolidated clay

3.2 Advantages and disadvantages of triaxial compression test

Advantages:

i) Simulates the field stress conditions.

ii) No predetermined failure plane.

iii) Failure takes place simultaneously at every point of the potential surface of sliding.

Disadvantages:

i) Relatively expensive.

ii) Can not be used to measure the residual shear strength of soil.

iii) Can not be used to measure interface shear strength.

Numerical Example #2:

The following results are yielded from CU tests on undisturbed saturated clay. Estimate graphically the shear strength parameters c' and ϕ' of the clay.

Test no.	Chamber pressure, σ_3 (kPa)	Deviator stress at failure, $\Delta\sigma$ (kPa)	Pore water pressure, at failure, u_f (kPa)
A	100	170	15
B	200	260	40
C	300	360	80

σ total

σ' Effective

u

$\sigma' = \sigma - u$

Solution

for test A:

$(\sigma_3')_A = 100 - 15 = 85$ KPa

$(\sigma_1')_A = (100 + 170) - 15 = 255$ KPa

for test B:

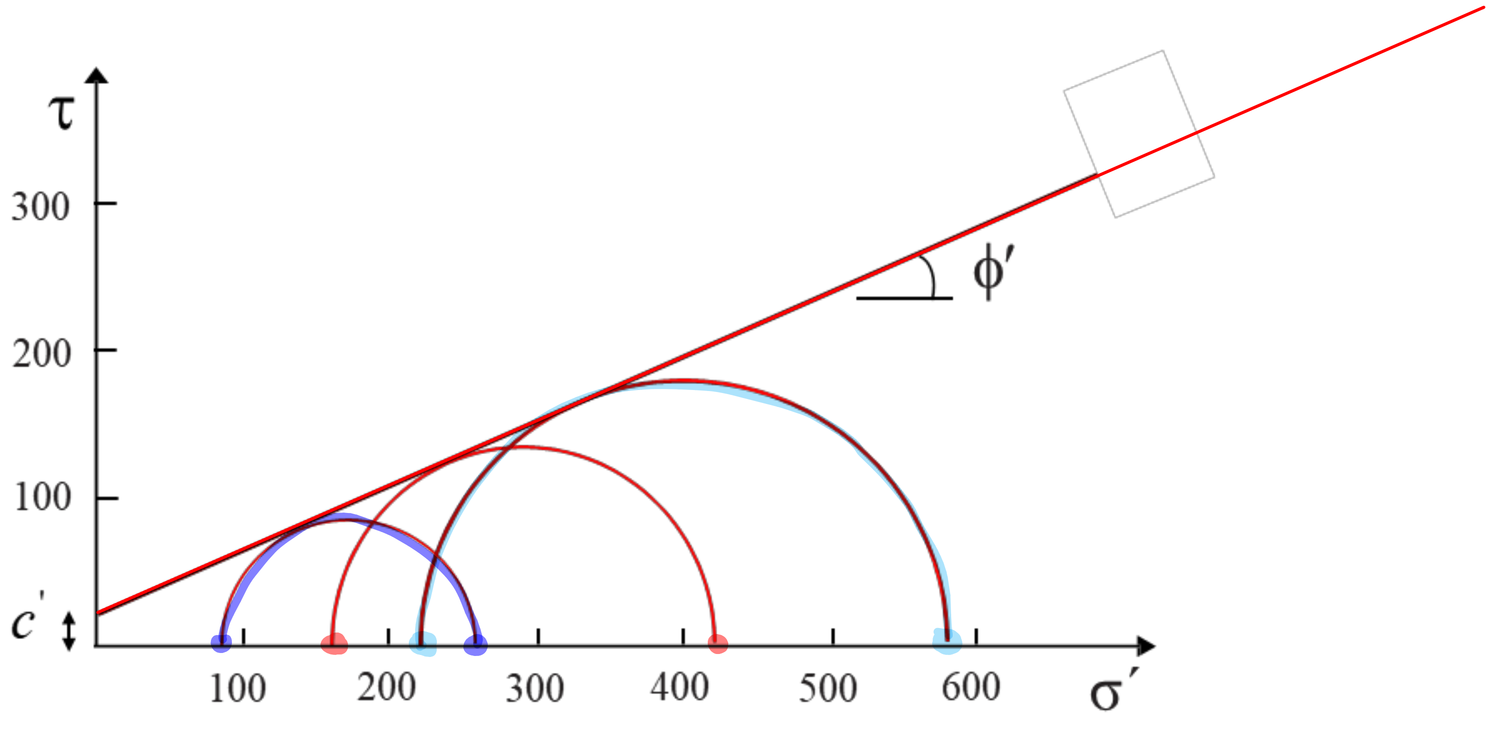
$(\sigma_3')_B = 200 - 40 = 160$ KPa

$(\sigma_1')_B = 200 + 260 - 40 = 420$ KPa

for test C:

$(\sigma_3')_C = 300 - 80 = 220$ KPa

$(\sigma_1')_C = 300 + 360 - 80 = 580$ KPa



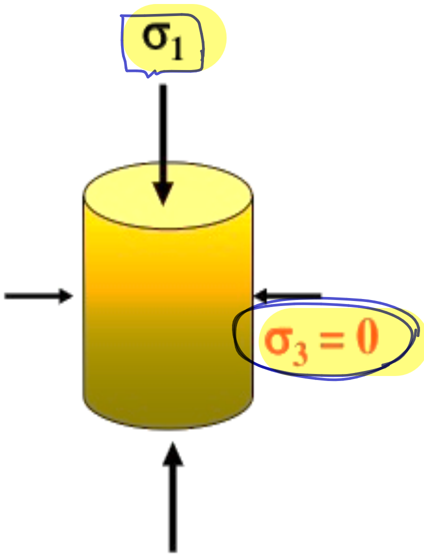
The values of c' and ϕ':

$c' = 17.7$ kPa

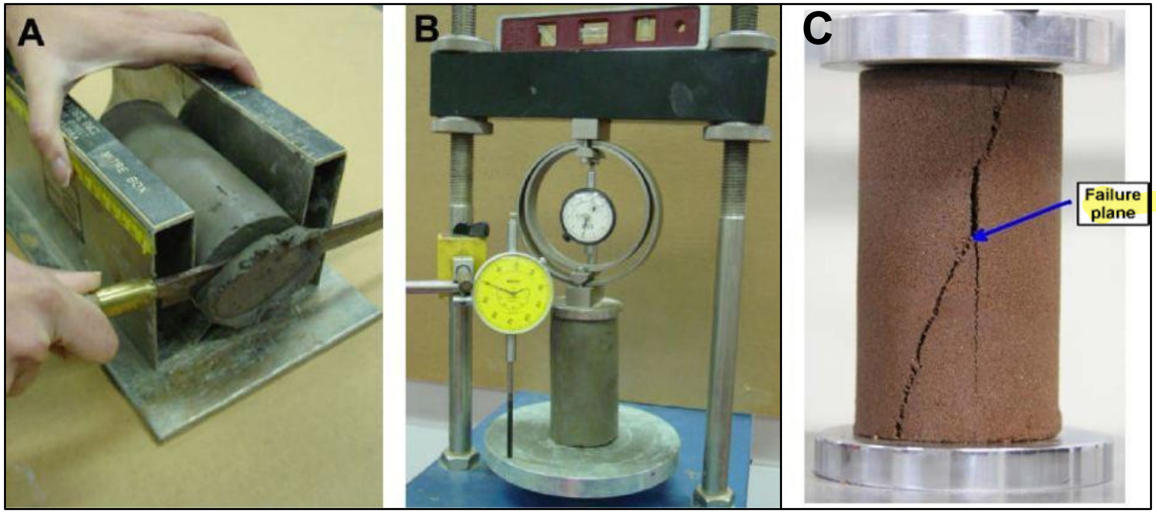
$\phi' = 24^{o}$

4. Unconfined (Uniaxial) Compression Test

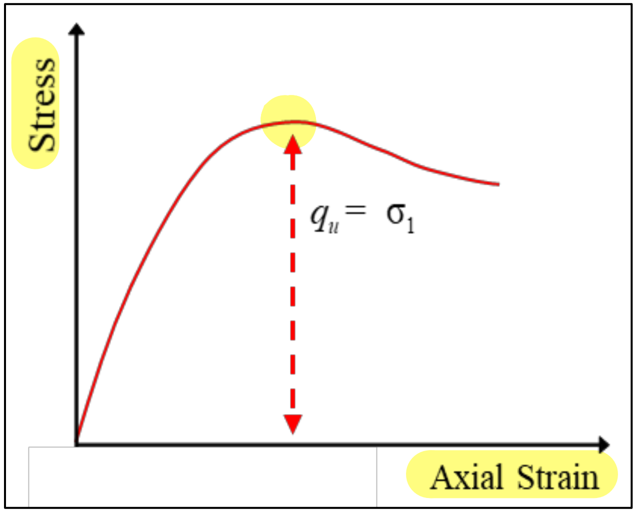
The unconfined compression test is a special case of the unconsolidated- undrained (UU) triaxial compression test. It is used widely to determine the consistency of saturated clays and other plastic soils. A cylindrical vertical specimen with a height-to-diameter ratio of about 2 and typically 38 mm or more in diameter is set up between end plates. A vertical load is applied incrementally at such a rate as to produce strain of about 1 to 2% per minute. This rate is so rapid relative to the drainage of the specimen. The unconfined compressive strength (q_u) is considered to be equal to the load at which failure occurs, or at which the axial strain reaches 20% if there is no sudden failure, divided by the cross-section area of the specimen at the time of failure (i.e.,σ_1).



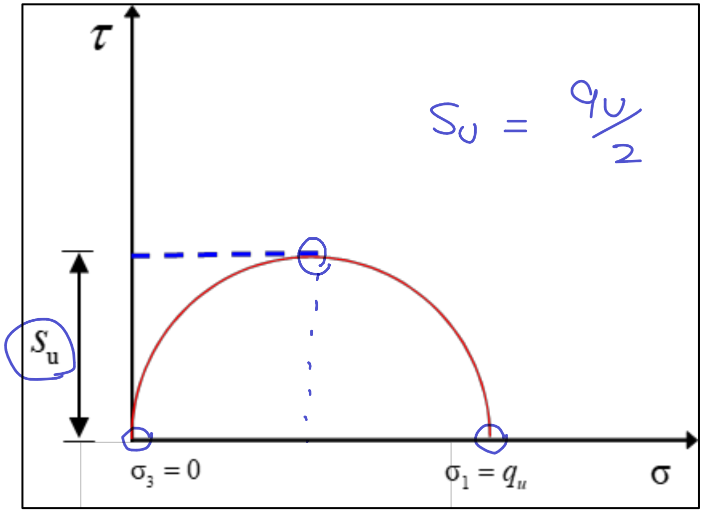
Loading in unconfined compression test



Unconfined compression test: (A) sample preparation; (B) set-up before testing; (C) failed specimen



Typical stress-strain relationship of unconfined compression test



Mohr's rupture circle yielded from unconfined compression test

As shown in the last two figured, the undrained shear strength of soil (s_u) can be estimated as:

$$\boxed{s_u = \frac{q_u}{2}}$$

Where: q_u is the unconfined compressive strength (or the axial vertical stress at failure).

4.1 Unconfined Compressive Strength and clay Consistency

Consistency of saturated clays and other plastic soils can be described based on the value of (q_u) as per the following table.

q_u		Consistency
ton / ft^2	kN / m^2	
0 - 0.25	0 - 25	Very soft
0.25 - 0.5	25 - 50	Soft
0.5 - 1.0	50 - 100	Medium
1 - 2	100 - 200	Stiff
2 - 4	200 - 400	Very stiff
> 4	> 400	Hard

4.2 Sensitivity of Soft Clays

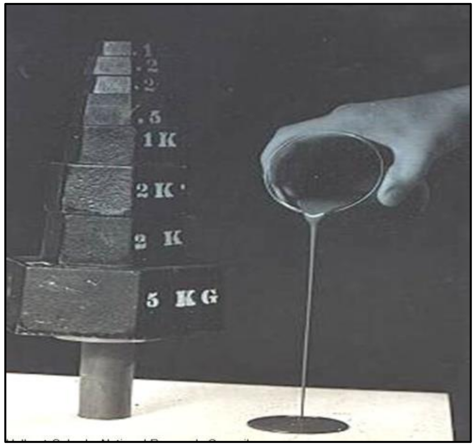
When normally consolidated or slightly overconsolidated natural clays are remolded at their natural water content, the undrained shear strength of the remolded soil is less than the undrained strength of the undisturbed soil. The parameter that is used to express the magnitude of loss in undrained shear strength upon remolding is called **sensitivity**, S_t and is defined as :

$$\boxed{S_t = \frac{s_u\,(\text{undisturbed})}{s_u\,(\text{remolded})}}$$

The difference in the undrained strength of the undisturbed and remolded clay samples can be explained in terms of changes in the structure of soil, i.e., fabric and interparticle forces. A majority of natural soft clays possess a flocculated random fabric which helps to develop short-range contacts and natural clay stabilized at relatively high water contents. The process of remolding rends to reorient clay particles to a face-to-face configuration. Therefore, a significant portion of the short-range are lost and undrained shear strength decreases.

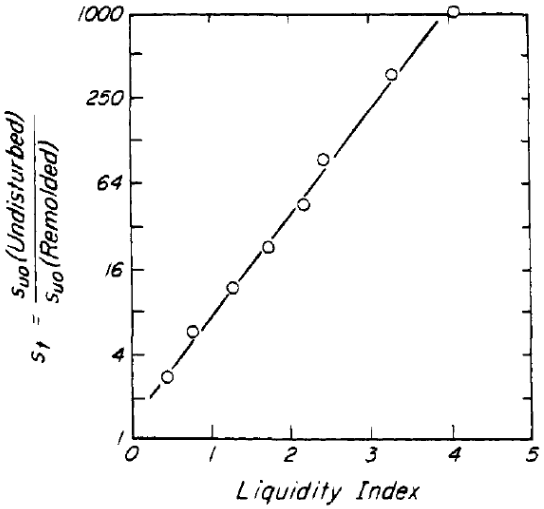
It terms of sensitivity soft clays may be classified as follows:

S_t	Classification
2 - 4	Low sensitivity
4 - 8	Medium sensitivity
8 - 16	High sensitivity
> 16	Quick



Undisturbed and remolded samples of Leda clay from Ontario, Canada

For some soils the loss in strength is so great (S_t can be up to 1000) that becomes almost a liquid upon remolding. These clays are called **quick clays**. An index property which is an excellent indication of sensitivity of clays is the liquidity index $\left(I_l = \dfrac{w - w_p}{I_p} \right)$.



Relation between sensitivity and liquidity index (after Bjerrum 1954)

ϵ

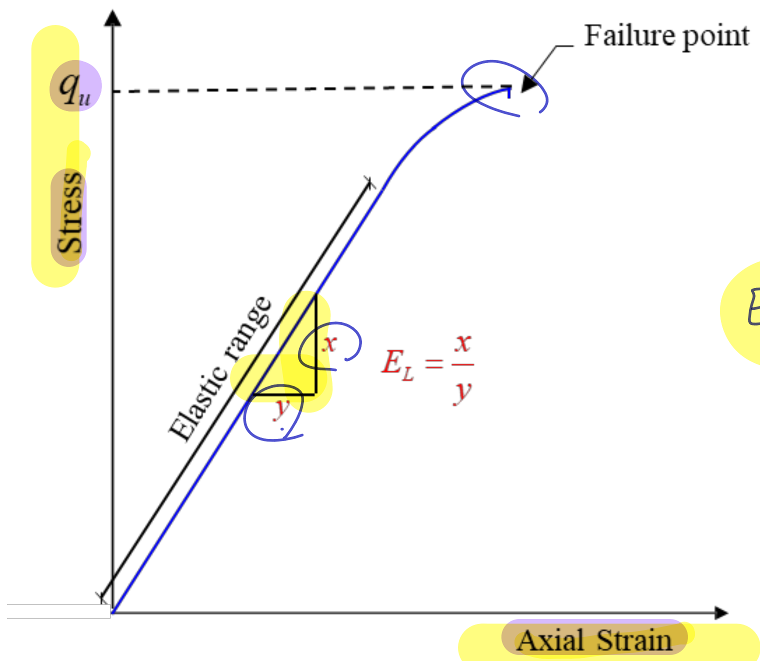
4.3 Unconfined (uniaxial) Compression Test for Rocks

It addition of the degree of fractionation, the unconfined compressive strength of rock (strength of rock substance) is heavily used in classifying rocks and stability analysis of foundation resting on rock masses. Unconfined compression test is usually performed on intact rock specimen with a height-to-dimeter ratio of 2 to 2.5, according to ASTM (American Society for testing and material) and 2.5-3.0 according to ISRM (International Society for Rock Mechanics). Samples are retrieved by drill cores and are selected cautiously in order to be representative of the original rock formation. The minimum diameter of a specimen must be at lest 47 mm.

Rock borehole cores

Rock specimen ready for unconfined compression test

The modulus if elasticity of rock substance (E_L or E_r of rock substance) can be estimated in the lab through plotting the relationship of the stresses and corresponding strains yielded from the unconfined compression test. Such modulus can be used, along with the rock degree of fractionation, to estimate the modulus of elasticity for rock mass (E_m) and consequently calculate settlement of structures resting on fractured rocks.



E_L = slope

Schematic of the stress-strain relationship yielded from the unconfined compression test on rock

4.3.1 Rock Description and ranges of q_u and E_L

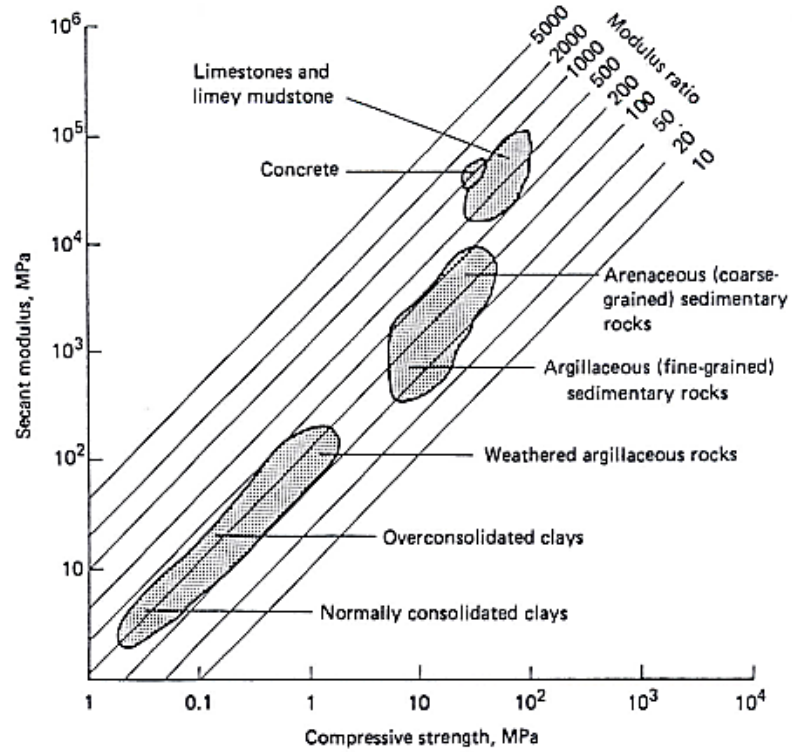
Rock can be described based on the value of q_u as follows:

q_u (MPa)	Description
< 1.25	Very weak
1.25 - 5	Weak
5 - 12.5	Moderately weak
12.5 - 50	Moderately strong
50 - 100	Strong
100 - 200	Very strong
> 200	Extremely strong

Range of Values of E and q_u for Some Rocks

Rock Type: Main	Rock Type: Sub-division	Range of Compressive Strength, t/m^2 (MN/m^2)	Range of Modulus of Elasticity, t/m^2 (MN/m^2)
Igneous			
Intrusive (coarse grained)	Granite	7000—18,000 (68.7—17.5)	2.6×10^6—5.0×10^6 (27.5×10^3—49.0×10^3)
	Diorite	7000—18,000 (68.7—176.5)	3.5×10^6—5.6×10^6 (34.3×10^3—54.9×10^3)
	Gabbro	10,000—21,000 (98.0—206.0)	$5.0\times10^6\times8.5\times10^6$ (49.0×10^3—83.4×10^3)
Extrusive (fine grained)	Rhyolite	7000—18,000 (68.7—176.5)	3.5×10^6—5.6×10^6 (34.3×10^3—54.9×10^3)
	Andesite	7000—18,000 (68.7—176.5)	4.2×10^6—6.3×10^6 (41.2×10^3—61.8×10^3)
	Dark coloured basalt	18,000—28,000 (176.5—274.6)	5.0×10^6—9.2×10^6 (49.0×10^3—90.2×10^3)
	Tuff	150—7000 (1.5—68.7)	0.16×10^5—0.7×10^5 (1.5×10^3—6.9×10^3)
Metamorphic			
Foliated (platey)	Mica schist	3500—10,000 (34.3—98.0)	1.4×10^6—3.5×10^6 (13.7×10^3—34.3×10^3)
	Fine grained, dark coloured slate	7000—14,000 (68.7—137.4)	3.5×10^6—5.6×10^6 (34.3×10^3—54.9×10^3)
Banded (imperfectly foliated)	Gneiss	7000—14,000 (68.7—137.4)	2.8×10^6—5.6×10^6 (27.5×10^3—54.9×10^3)
Massive	Quartzite	10,000—25,000 (98.0—245.2)	4.2×10^6—5.6×10^6 (41.2×10^3—54.9×10^3)
	Marble	8500—21,000 (83.4—206.0)	5.0×10^6—7.0×10^6 (49.0×10^3—18.6×10^3)
	Serpentine	700—7000 (6.9—68.7)	0.7×10^6—3.5×10^6 (6.9×10^3—34.3×10^3)
Sedimentary			
Argillaceous	Shale	70—3500 (6.9—34.3)	0.35×10^6—1.4×10^6 (3.4×10^3—13.7×10^3)
	Siltstone	70—3500 (6.9—34.3)	0.35×10^6—1.4×10^6 (3.4×10^3—13.7×10^3)
Silicious	Medium grained sandstone	3000—8500 (29.4—83.4)	0.7×10^6—2.0×10^6 (6.9×10^3—19.6×10^3)
	Coarse grained conglomerate	3500—10,000 (34.3—98.0)	0.7×10^6—3.5×10^6 (6.3×10^3—34.3×10^3)
	Breccia	3500—10,000 (34.3—98.0)	0.7×10^6—3.5×10^6 (6.0×10^3—34.3×10^3)
Calcareous	Limestone	3500—10,000 (34.3—98.0)	1.4×10^6—4.2×10^6 (13.7×10^3—41.2×10^3)
	Dolomite	5000—15,000 (49.0—147.1)	2.8×10^6—5.6×10^6 (27.5×10^3—54.9×10^3)

Note: Values of properties are for rocks tested dry in the laboratory. Elasticity and strength of specimens tested saturated, generally are 8- and 90 per cent of the values shown.



Classification according to uniaxial compressive strength and Young's modulus. (After Deere and Miller, 1966; Hobbs, 1974.)

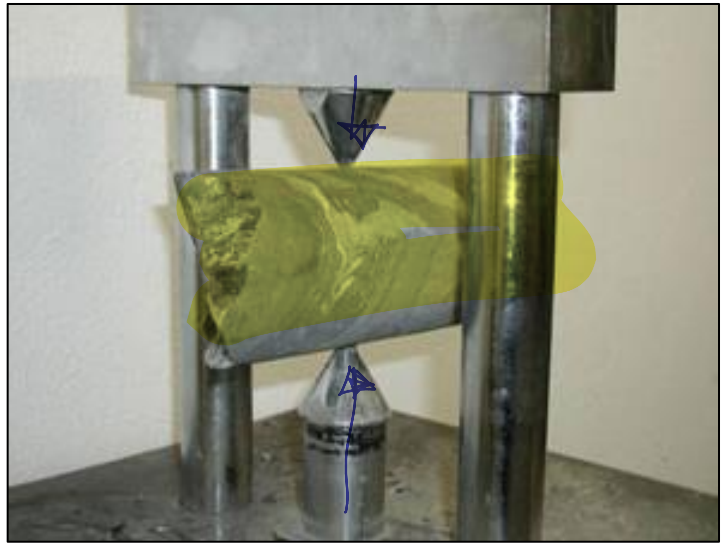
Due to the end effect, the measured unconfined compressive strength (q_u) measured of specimens with height-to-diameter ratio less than 2.0 (i.e., $\frac{L}{D} < 2.0$) should be corrected as follows:

$$\boxed{q_u = \frac{(q_u)_{\text{measured}}}{0.88 + 0.24\frac{D}{L}}}$$

$L/D < 2$

4.3.2 Point Load test

A simplification of uniaxial strength test which can give a rapid and accurate strength index for harder rocks. It is becoming so widely used laboratory test. Performing the unconfined compression test on rocks requires expensive laboratory equipment and carful specimen preparation. The point load test requires a relatively simple loading frame and can be carried out on virtually any shape and size of specimen. The test is usually carried out on an unprepared core, obtained directly from drilling, which is compressed between two conical points loaded from simple hydraulic hand pump. The modulus of elasticity of rock substance $(E_L$ or $E_r)$ **can not** be determined from this test.



Rock sample ready for point load test

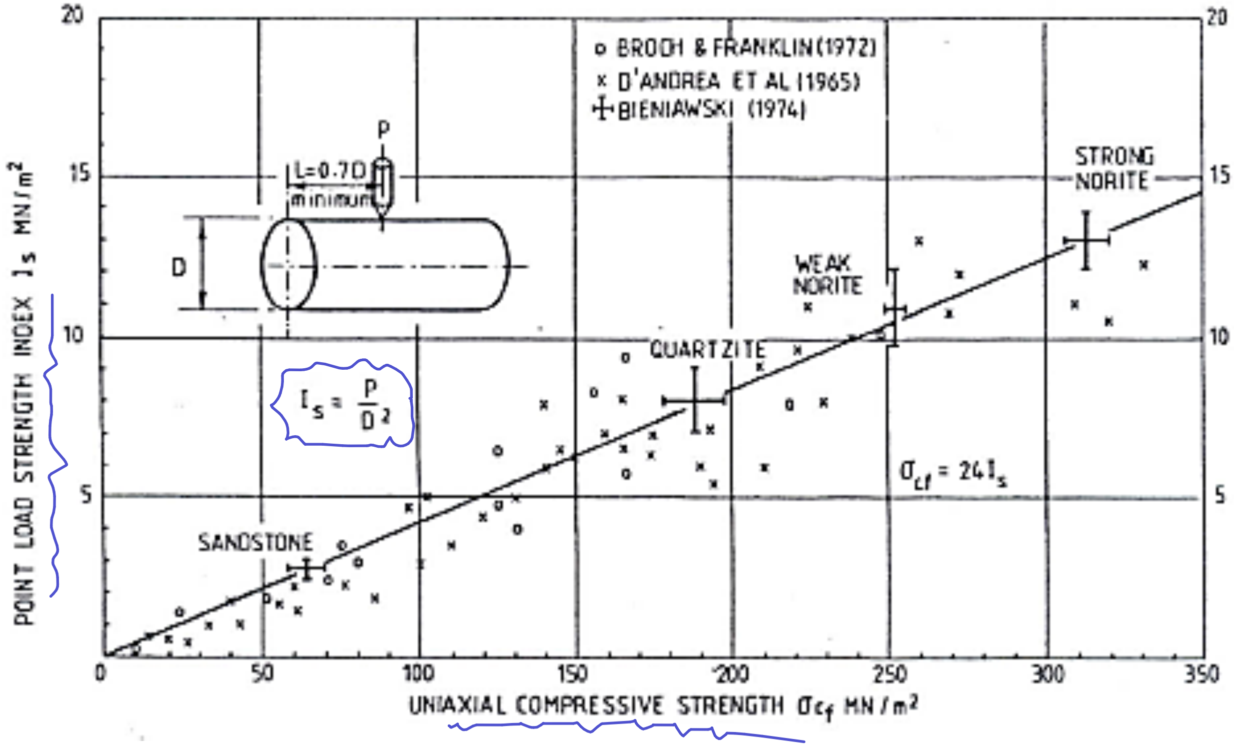
The cores specimen should ideally be NX (54mm) size and have a length at least 1.5 times the diameter (D). The core fails at a relatively low applied force (P) due to tensile failure over the diameter area between the points. The strength at failure is expressed as a point load index (I_s), where

$$\boxed{I_s = \frac{P}{D^2}}$$

$q_u = C\, I_s$

A very close correlation has been shown by various researches between I_s and the uniaxial compressive strength (q_u):

$$\boxed{q_u = CI_s}$$



Relation between point load strength index and uniaxial compressive strength for NX core samples (after Bieniawski 1974)

Where C is a constant equal to 24 for NX (54 mm diameter) cores. Hoek and Bray (1974) suggest modified values of C=17.5 for 20 mm cores, 19 for 30 mm, 21 for 40 mm, 23 for 50 mm and 24.5 for 60 mm cores. They also suggest that the test is only vailed if a clean diameter break occurs between cores. If the fracture runs to another plane or if there are signs of cone penetration and crushing, the results should be rejected.

Diameter break valid test