SHEAR STRENGTH

13.1 INTRODUCTION

Soil is a *particulate* material composed of discrete particles which are relatively free to move with respect to one another. The *mineral skeleton* of soil usually is quite deformable, due to interparticle sliding and rearrangement, even though the individual particles are very rigid.

Thus when a compression load is gradually applied to a soil mass, it will eventually fail due to the movement of the individual soil particles relative to one another which occur along surfaces known as *shear surfaces*. *The maximum resisting stress offered by the soil particles to the deformation due to relative sliding of the particles immediately prior to failure of the soil mass is called as shearing strength of the soil mass.*

Thus the structural strength of soil is primarily a function of its shear strength and as such the ability of a soil mass to support structural load is dependent on its shearing strength. Specially the knowledge of shear strength is essential for:

(i) Evaluation of bearing capacity (Chapter-10) used for the design of foundations.

(ii) Analysis of stability of slopes used for the design of embankments for dams, levees, roads, temporary /permanent excavations etc. (Chapter-12), and

(iii) Estimation of lateral earth pressure required in the design of earth retaining structures such as retaining walls, bulk heads, sheetpile cofferdam, underground structures etc. (Chapter-11).

The shear strength of a soil mass is essentially made up of:

(a) The structural resistance to movement of soil particles due to interlocking of the grains (i.e. density of the soil)

(b) the frictional resistance to sliding between the individual soil grains at their contact points (i.e. angle of internal friction, ϕ of the soil), and

(c) cohesion (adhesion) between surfaces of the soil grains. The cohesion (c) is the resistance due to the forces tending to hold the grains together in a soil mass.

Generally speaking, coarse-grained soils (sands, gravels and their mixtures) derive their shear strength almost entirely from internal friction (ϕ). On the other hand, fine-grained soils (clays, silts and their mixtures) have cohesion (c) as their major component of shear strength. Usually most of the natural soils are mixture of fine-grained and coarse-grained soils and as such their shear strength is dependent on both c and ϕ parameters.

It is therefore convenient to consider the following three conventional soil types for study of shear strength:

(i) Coarse-grained. frictional soil or cohesionless soils ($c = 0$ soils)

(ii) Fine-grained or cohesive soils ($\phi = 0$ soils)

(iii) Cohesive-frictional soils (c-ϕ soils)

Shear strength of a soil mass is greatly influenced by loading and drainage conditions during the loading. A number of different field and laboratory tests are used to evaluate the shear strength of soils. When used under comparable conditions these tests should give similar results

This chapter is deputed for the study of shear strength.

13.2 COULOMB'S LAW OF SHEAR STRENGTH (1773)

As stated in the preceding section, the shear strength of a soil is made up of two major components—friction (ϕ) and cohesion (c).

The intergranular friction (ϕ) is directly proportional to the normal stress acting on shear surface. The cohesion (c) is dependent on the type, size and packing of the soil grains and on the suction properties of the soil.

Coulomb (1773) proposed that the shearing strength of a soil, τ is governed by the straight line equation:

$$\tau = c + \sigma \tan \phi \qquad 13.1$$

Where,

c = apparent cohesion

σ = the normal stress

ϕ = the angle of internal friction or shearing resistance of the soil.

In 1773 when coulomb put forward equation 13.1, the concept of effective stress was not introduced. Following the introduction of this principle by Terzaghi, the equation 13.1 is now expressed in terms of effective stresses, thus:

$$\bar{\tau} = \bar{c} + \bar{\sigma} \tan \bar{\phi}$$

Where,

$\bar{c}$ = apparent cohesion w.r.t. effective stress;

$\bar{\sigma} = \sigma - u$, the effective normal stress (σ being the total stress and u the pore water pressure)

$\bar{\phi}$ = effective angle of internal friction.

The modified Coulomb's equation is diagrammatically shown in Fig. 13.1.

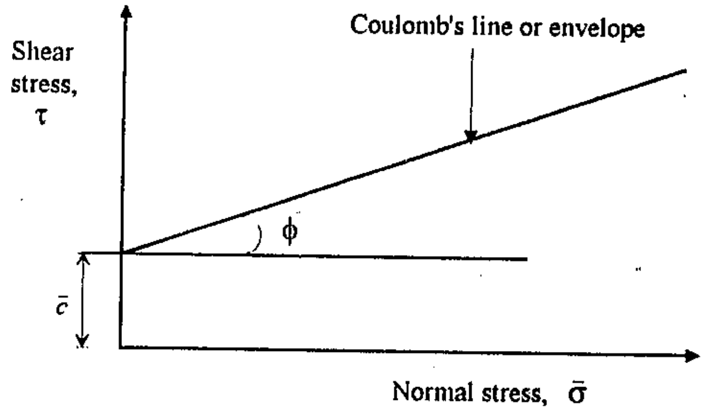


Figure 13.1 *Diagrammatic representation of Coulomb's equation* 13.2.

From equation 13.2, it is evident that the pore water pressure (u) has a major influence on the shearing strength of soils. For coarse-grained soils where the drainage is very good, the total stress (σ) is usually equal to the effective stress ($\bar{\sigma}$). With the fine-grained soils, however, the drainage is very poor and usually considerable time is required before the effective increase is equal to the total stress increase. Hence the rate and length of time of testing is important in the determination of the shear strength of the fine-grained soils. Fig. 13.2 represents the relation between testing time and the effective stress:

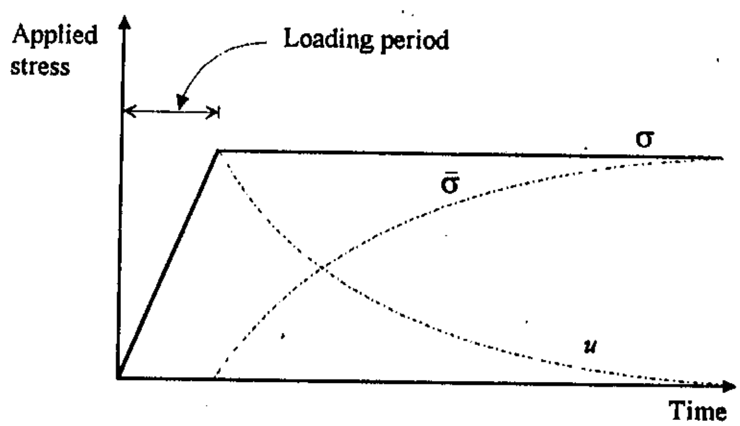


Figure 13.2 *Pore pressure/effective stress/time relationship*

Thus in order to obtain realistic results within a reasonable time for fine-grained soils, pore pressures must be recorded during the progress of the test continuously. For shear strength of fine-grained soils, it is vital that the conditions applicable to the field problem must be considered before deciding on the type of test to be used. This is to ensure that the testing conditions in the laboratory simulate the conditions of the site.

13.3 MOHR CIRCLE OF STRESS

In 1887 O Mohr presented the concept of Mohr circle of stress according to which the stress at any point within a material at equilibrium can be represented by a circle provided the shear stress and the normal stress are plotted using same scale. Mohr circle of stress represents the complete two-dimensional state of stress *at equilibrium* in an element or at a point. The concept of Mohr circle is very useful in geotechnical engineering; and briefly explained under this section.

More specifically in geotechnical engineering, we shall be interested in the state of stress in the plane that contains the major and minor principal stresses, σ_1 and σ_3. Given the magnitude and direction of σ_1 and σ_3, it is possible to compute normal and shear stress in any other direction. Concept of Mohr circle of stress is explained below.

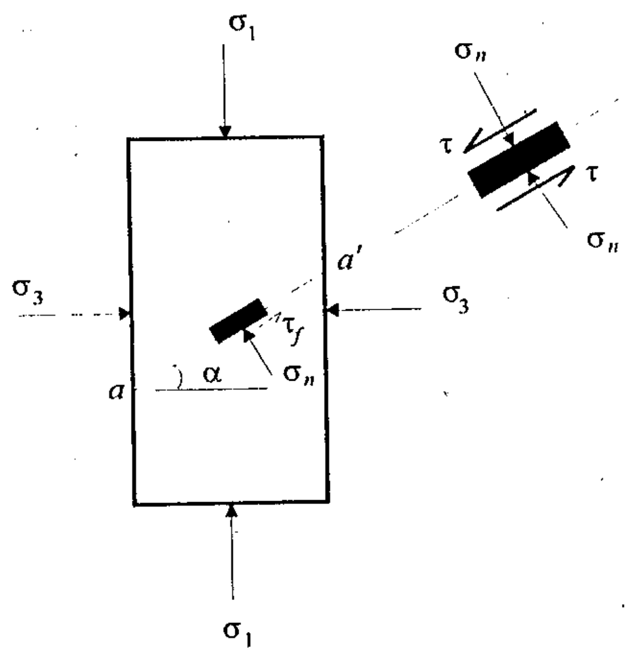


Figure 13.3 *Vertical cross-section through a cylindrical soil sample subjected to confined compression (triaxial) showing shear plane aa' , normal stress* σ_n, *and shear stress,* τ.

The stress conditions of Fig. 13.3 can be analyzed by Mohr's circle of stress as shown in Fig. 13.4.

In Fig. 13.4. the line *FE* often called the *Coulomb's line* or *rupture line*, represents the conditions of shear failure in accordance with Coulomb's law. Mohr circle shown in Fig. 13.4. touching this line at point *E*, represents a condition of incipient failure. Any circle falling below line *FE* would denote stable soil conditions.

The normal stress, σ_n and the shear stress, τ, on an inclined shear plane (*aa'*, Fig. 13.3) can be geometrically demonstrated on Mohr's graph as follows:

$$\sigma_n = OB = OC - CB$$

But $\quad OC = \frac{\sigma_1 + \sigma_3}{2}$

$CB = (CE)\cos 2\alpha,$

And $\quad CE = CA = CD = \frac{\sigma_1 - \sigma_3}{2}$

Therefore,

$$\sigma_n = OB = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2}\cos 2\alpha \qquad 13.3$$

Similarly,

$$\tau = BE = (CE)\sin 2\alpha = \frac{\sigma_1 - \sigma_3}{2}\sin 2\alpha \qquad 13.4$$

Thus each point on the circle gives the pair of stresses acting on a rupture plane of specific inclination, α.

A tangent *t-t*, drawn to the circle at point *E*, has the equation.

$\tau = c + \sigma \tan\phi$ Coulomb's equation for shear strength of soil

The slope of this line, $\tan\phi$, physically means the co-efficient of internal friction of the soil: ϕ is the angle of internal friction, and c is the cohesion.

c and ϕ are actually test co-efficients obtained by special apparatus and by special methods of testing.

Fig. 13.5 represents the Mohr's circle of stress for non-cohesive and cohesive soils.

In this method, the normal stresses (σ_1, σ_3, σ_n) acting on the cylindrical soil sample (triaxial test) are plotted as abscissa and shear stress (τ) as ordinates using same scale along the axes. The difference in the principal stresses, ($\sigma_1 - \sigma_3$) is called as deviator stress. A circle of diameter $(\frac{\sigma_1 - \sigma_3}{2})$ is drawn. Since the shear stresses are zero on planes where principal stresses are acting, the ends of the stress diameter or Mohr's circle of stress have the co-ordinates (σ_1, 0) and (σ_3, 0).

In coarse-grained soils (Dry sand at gravels) cohesion is insignificant, negligible and the Mohr's circle of stress takes the shape of Fig. 13.5 (i). In this case the ratio of the principal stresses, σ_1/σ_3 is the same for all the circles tangent to the line *OE*.

$$\sin\phi = \frac{CE}{OC} = \frac{1/2(\text{difference of principal stresses})}{1/2(\text{sum of principal stresses})} = \frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}$$

From this it is deduced that:

$$\frac{\sigma_1}{\sigma_3} = \frac{1+\sin\phi}{1-\sin\phi} \qquad 13.5$$

Equation 13.5. is used in calculation of earth pressures.

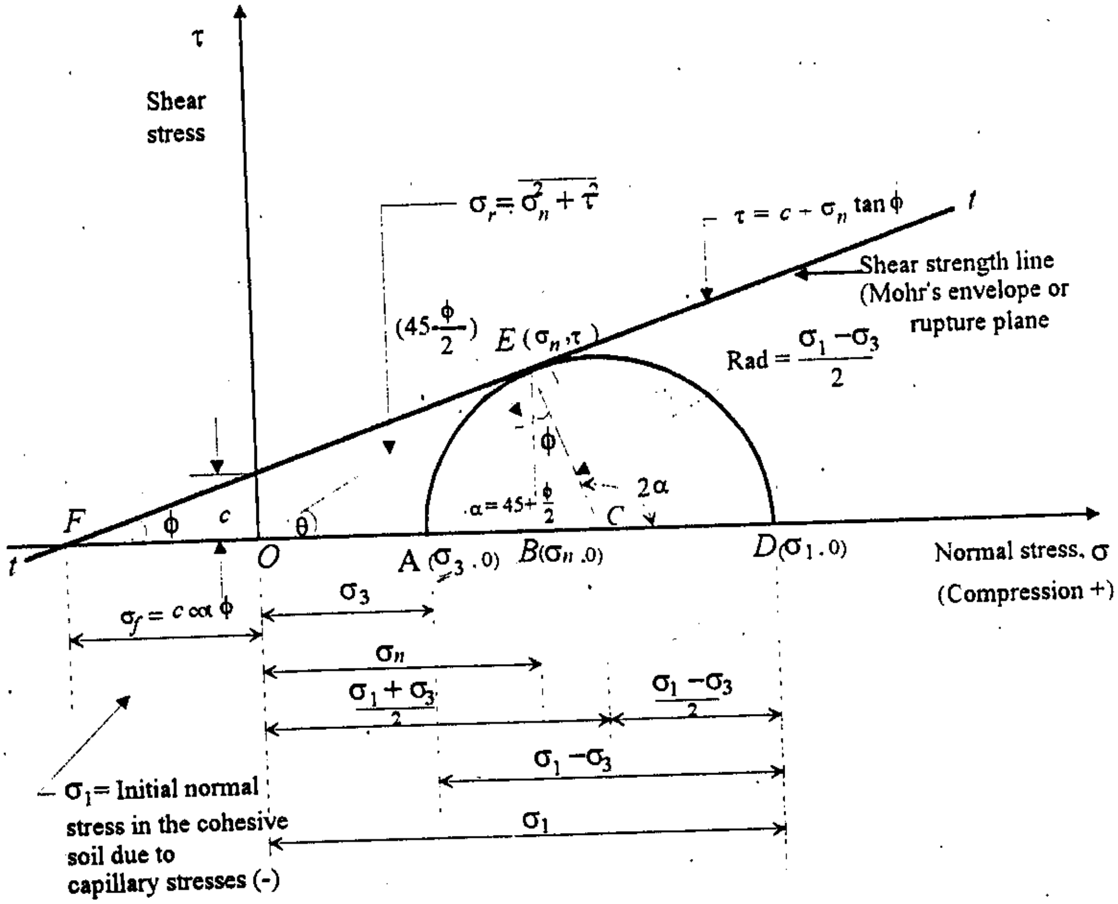


Figure 13.4 *Mohr's circle of stresses*

13.4 METHODS OF DETERMINING SHEAR STRENGTH

Shear strength of soils can be determined using:

(i) Laboratory tests; and

(ii) Field tests.

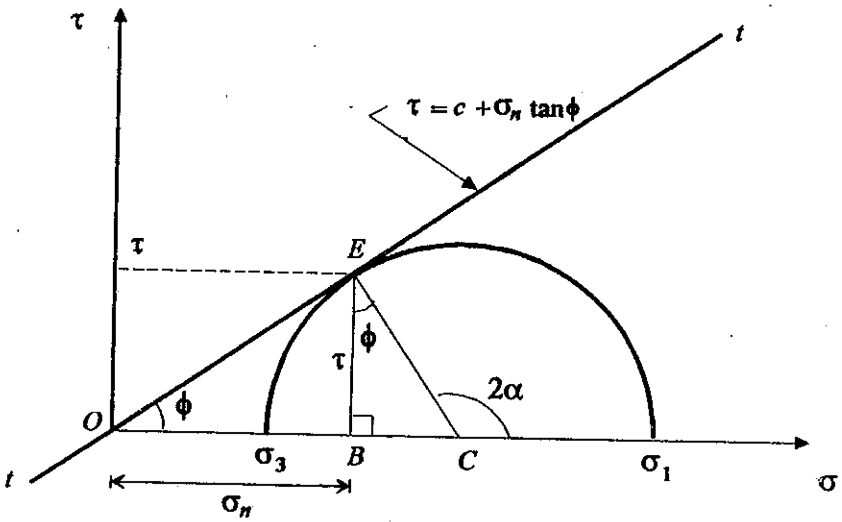
- **Laboratory tests**

(a) Direct shear test (ASTM D3080)

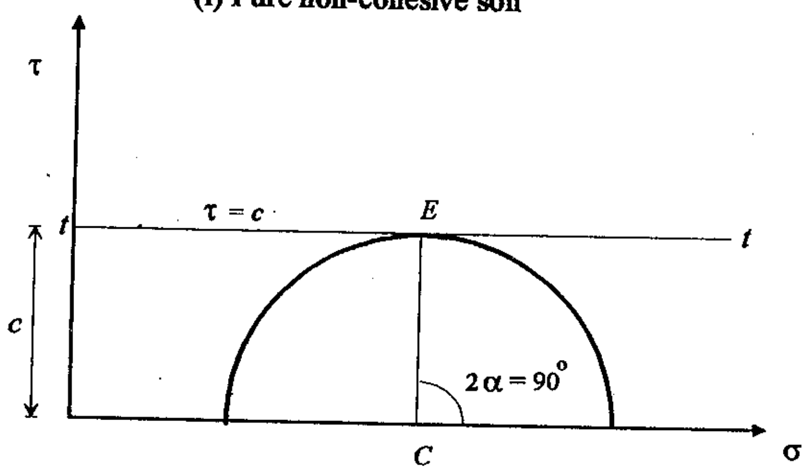
(b) Unconfined compression test (ASTM D2166)

(c) Triaxial compression test (ASTM D2850)

(d) Vane shear test (ASTM D2573)



(i) Pure non-cohesive soil



(ii) Pure cohesive soil

Figure 13.5 *Mohr's circle of stresses for different soils*

Unconfined compression test can be used for determining shear strength only of cohesive soils and the vane shear is suitable only for soft clays particularly sensitive clays. Direct shear and triaxial test, however, can be used to investigate cohesive and cohesionless soils both.

For details of shear tests, the readers may refer to any soil mechanics testing manual or respective test standards shown against each test. Brief description, however, for each test is presented in the succeeding sections.

13.5 DIRECT SHEAR TEST

This is relatively a simple shear test in which the shearing force is applied at a constant rate of strain until shearing failure occurs. In this test, soil samples are placed in a metal shear box of square or round shape spliced at mid height, as shown in Fig. 13.6. Thus in this test, the sample is made to fail on a *pre-determined horizontal shear plane*. The shearing force is measured by means of a horizontal proving ring from which the peak shearing stress is determined. Horizontal and vertical deformations can be recorded using displacement dial gauges installed for this purpose. The normal force to the plane of shear failure can be varied and drainage of the sample can be controlled using solid or porous plates placed at the bottom and top of the sample.

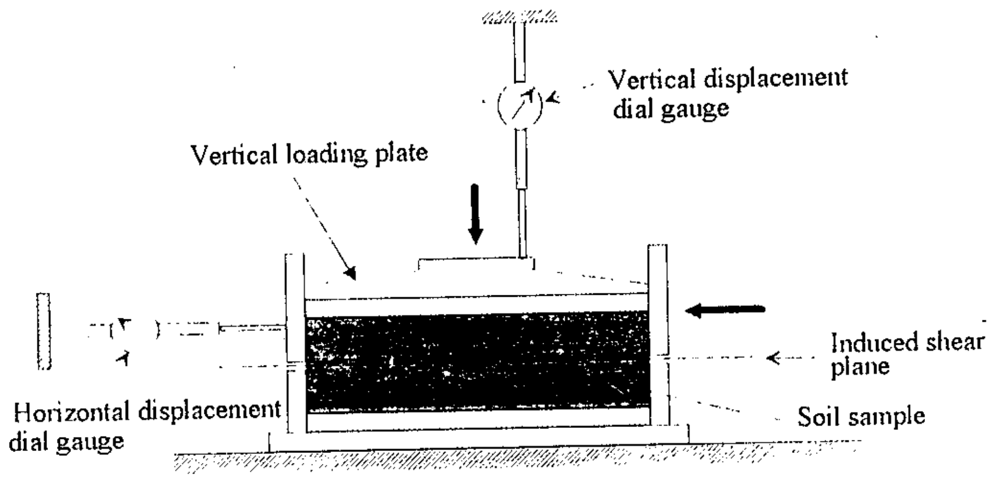


Figure 13.6 *Diagrammatic sketch of direct shear apparatus*

Typical curve of shearing stress, τ, against horizontal deformation δ_h for a given normal stress, σ, on a compacted dense sand is shown in Fig 13.7 (i). From this it is evident that the shearing stress reaches a peak value of τ_{max}, and then decreases while shearing still continues. In Fig 13.7 (ii), τ_{max} is plotted against σ for a series of tests performed under different normal stresses, σ_n and c & ϕ parameters are determined as shown in Fig. 13.7 (ii) below:

COMMENTS ON DIRECT SHEAR TEST

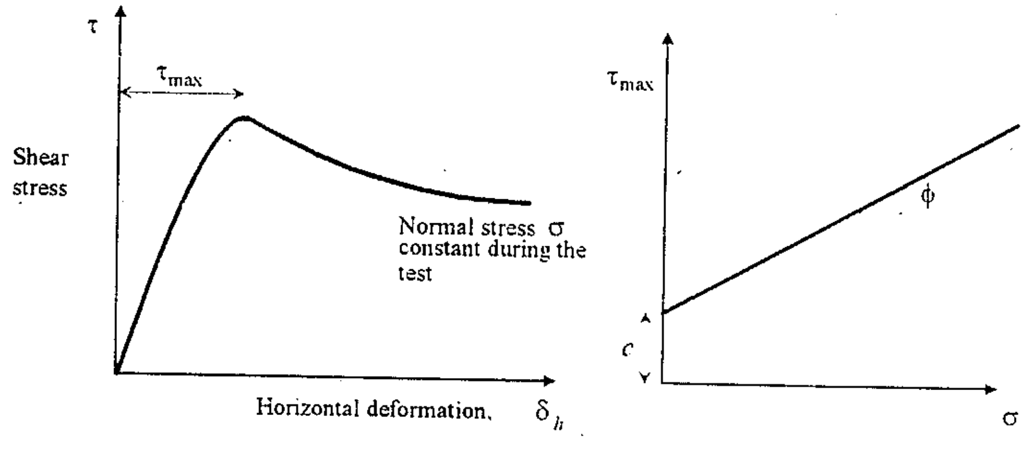
The direct shear although simple and relatively rapid has following major disadvantages:

(i) There is a little control over the drainage conditions. The measurement of a vertical displacement and hence of volume change is not accurate.

(ii) The failure plane is pre-determined, which may not be the weakest plane.

(iii) The actual distribution of shearing stress over the failure plane is not known.

(iv) The area of failure plane decreases during the test. The correct area should be used to compute normal and shear stresses.



(i) Shear stress vs. horizontal deformation (ii) τ_{max} against normal stress for a series of tests

Figure 13.7 *Direct shear test plots*

13.6 TRIAXIAL COMPRESSION TEST.

This is the most complex but accurate shear test. In this test a cylindrical soil sample of height to diameter ratio 2 to 3 is loaded in all three dimensions. although the analysis is reduced to two dimensions as a result of the lateral stresses (cell pressure. σ_3) being equal in all directions. Fig. 13.8 represents a typical test cell layout.

The soil sample enclosed in a rubber membrane and generally has porous platens on each end, is placed in the water tight perspex cell. Water is pumped into the cell and its pressure raised to σ_3 (cell pressure) which acts in all directions. A vertical load is then applied and recorded using proving ring. until shear failure occurs. Since the cell pressure σ_3 was acting all around the sample. and additional vertical stress of ($\sigma_1-\sigma_3$) will cause the failure of the sample and as such this additional stress is known as *deviator stress*. The major and minor principal stresses would be σ_1 and σ_3 respectively. Vertical displacement of the sample can be recorded using strain gauges or dial gauge. If desired, the pore water pressure and volume changes can also be monitored during the test.

Tests are carried out under different cell pressures and results plotted as Mohr's circles. The tangent drawn to the circles gives the values of c & ϕ as shown in Fig 13.9.

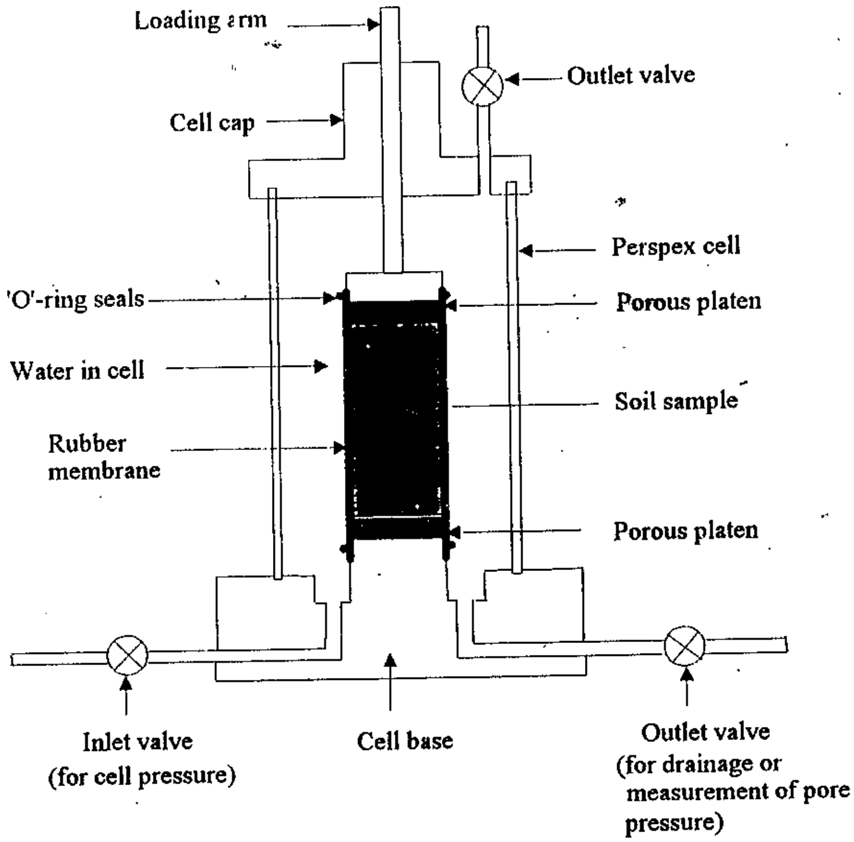
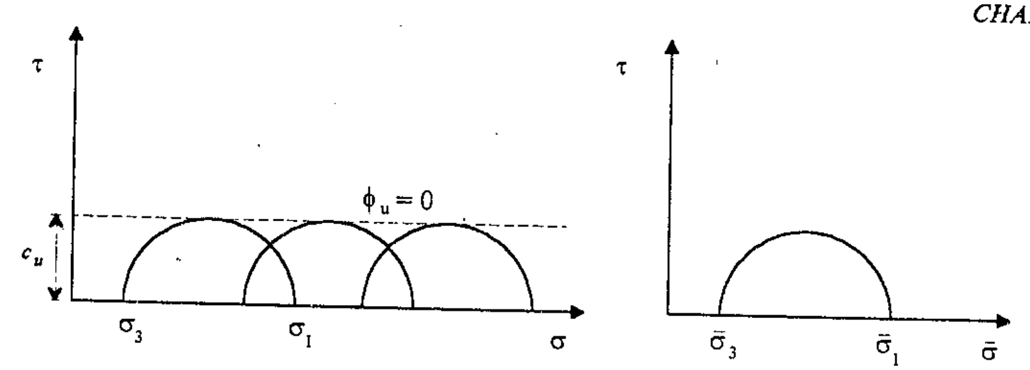


Figure 13.8 *Triaxial cell apparatus*

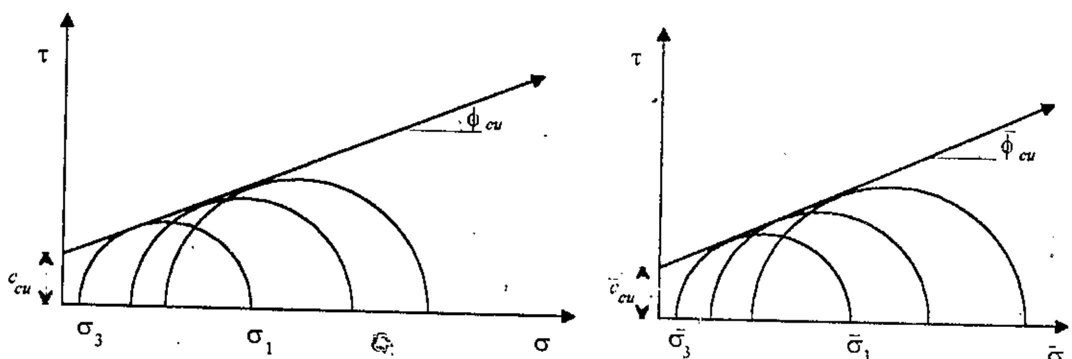
Comparison of direct shear and triaxial tests.

Direct shear	Triaxial
(i) Soil sample is made to fail along a pre-determined plane which may not be the weakest plane.	Sample is free to fail along the weakest plane. The failure plane is not pre-determined.
(ii) There is little control over drainage conditions. Arrangement for pore pressure measurement are not provided.	There is proper control over drainage conditions. Arrangement for measurement of pore pressure are provided.
(iii) Undrained test on sand cannot be performed properly and the results are not reliable.	Any type of test can be performed on any soil type.
(iv) There is unequal distribution of shear stress over the sliding plane	The stress distribution is relatively uniform.
Effective stress cannot be computed.	Effective stress at various stages can be computed.



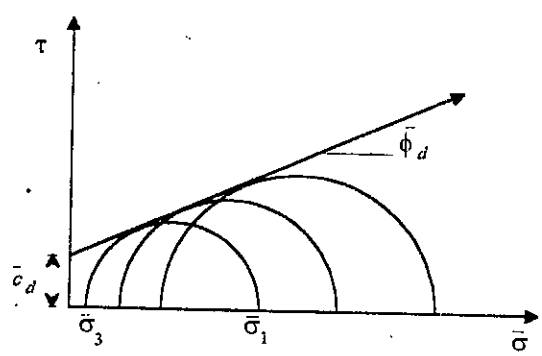
(i) Total stresses (ii) Effective stresses

(a) Unsonsolidated-undrained (UU) test on saturated soil



(i) Total stresses (ii) Effective stresses

(b) Consolidated-undrained (CU) test



(c) Consolidated-drained (CD) test

Figure 13.9 *Triaxial tests for various drainage conditions (see section* 13.10*)*

13.7 UNCONFINED COMPRESSION TEST (UCT)

This test actually is a special form of a triaxial test where the confining cell pressure, is kept zero during the test. Thus the cylindrical soil sample is crushed to failure without applying any lateral pressure like a concrete cylinder crushing test. Although the test can be done in the laboratory using triaxial apparatus, it is move usual to use a much simple portable piece of equipment known as unconfined compression test apparatus as shown in Fig 13.10. Due to its portable nature the equipment can be shifted to the site and UCT is generally done right in the field.

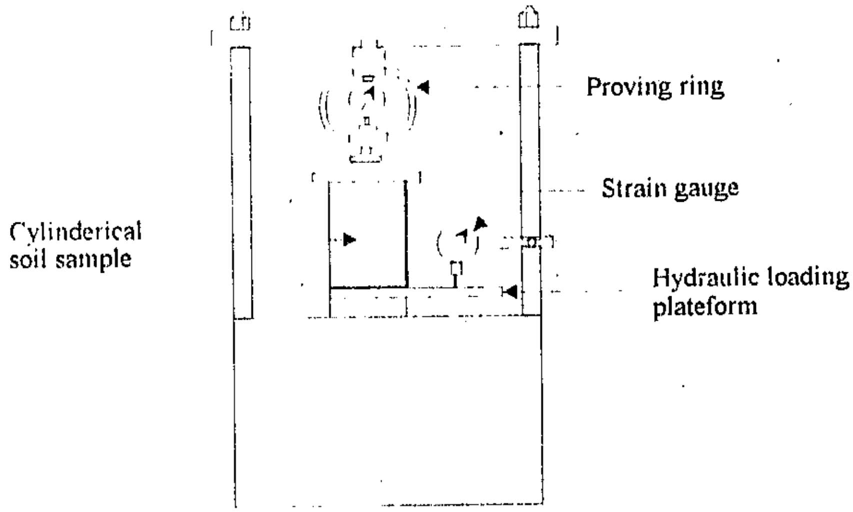


Figure 13.10 *Unconfined compression test apparatus*

In this test a cylindrical soil sample of height to diameter ratio 2 to 3 is compressed at a constant rate of strain (about 1.25 mm/min.) in a loading frame shown in Fig. 13.10 until cracks have definitely developed or stress strain curve is well past its peak or 20% deformation is achieved.

The test is usually rapid and without drainage taking place and since there are no lateral stresses (cell pressure, σ_3), the Mohr's circle of stress passes through the origin as shown in Fig.13.11(ii).

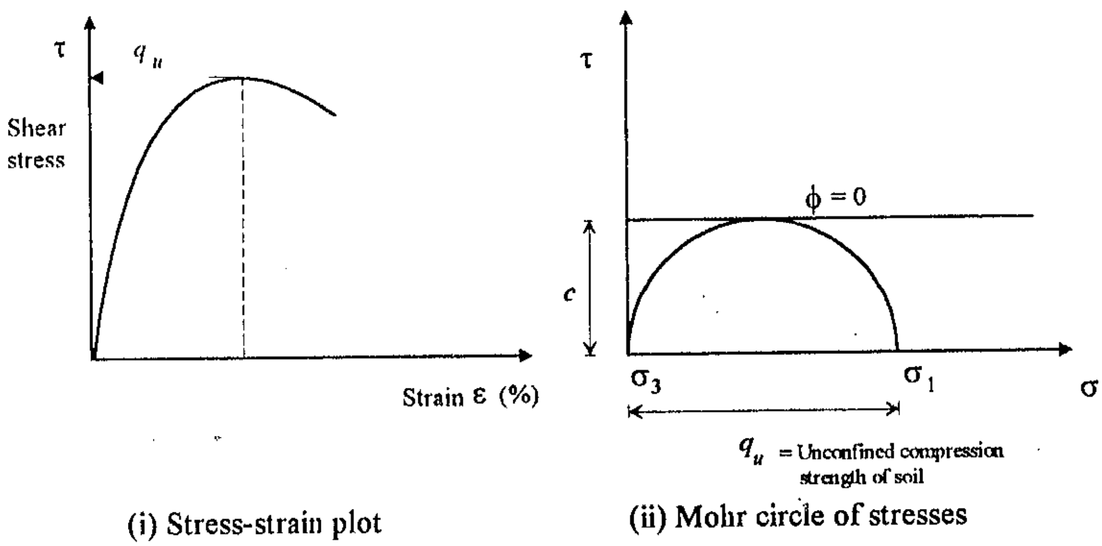


Figure 13.11 *Unconfined compression test*

Thus $c = q_u/2$ 13.6

Since in this test it is assumed that the volume of sample remains constant, the cross-sectional area of the sample at any stage of test, A is given by:

$$A = \frac{A_o}{1-\varepsilon} \qquad 13.7$$

Where,

A = initial x-sectional area of the test sample = $\frac{\pi}{4}D^2$ (D being the sample dia.)

ε = axial strain at any stage of test.$\Delta L/L_o$

Where,

ΔL = change in sample length i.e. vertical deformation during the test at any stage of test.

L_o = initial sample length.

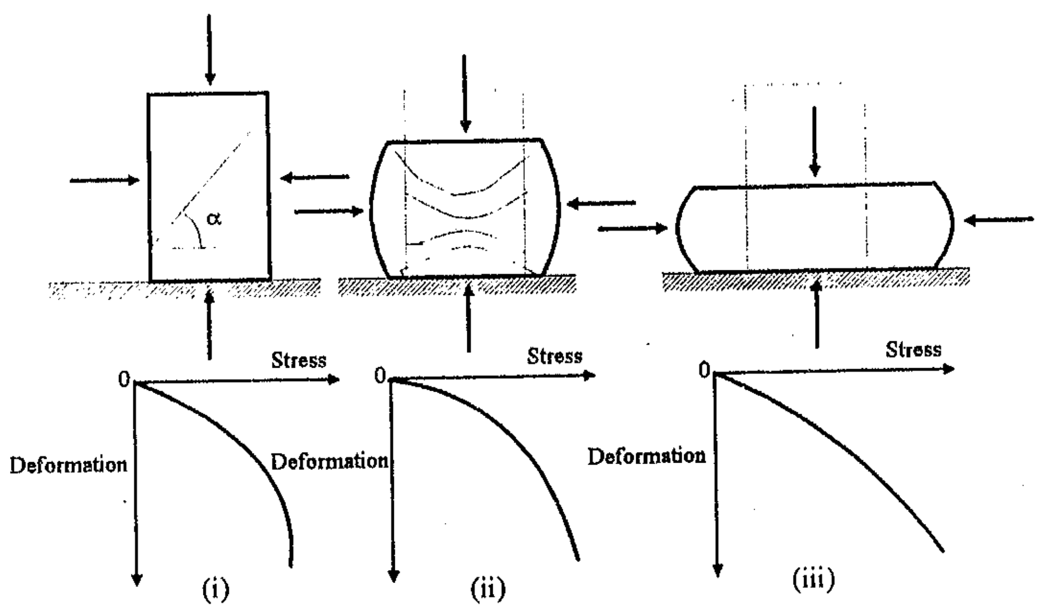
Let,

P = compression load at failure,

$q_u = P/A$ = unconfined compression strength 13.8

13.8 MODES OF FAILURE IN TRIAXIAL TEST AND UNCONFINED COMPRESSION TEST

Typical failure modes of triaxial/unconfined compression test sample are depicted in Fig. 13.12.



(a) Brittle failure with clearly defined failure plane. Common in OCC and dense sand.

(b) Intermediate failure with shear cones and bulging. Common for medium dense sand and lightly OCC.

(c) Plastic failure with considerable bulging. Common for loose sand and NCC.

Figure 13.12 Typical failure modes of compression tests

13.9 LABORATORY VANE SHEAR TEST (LVST)

This is also a rapid test, used either in the field or in the laboratory to determine the undrained shear strength of soft cohesive sensitive clays. Sensitive clays are those soft clays which looses part of their shear strength when disturbed.

In this test a cruciform vane of the form shown in Fig. 13.13 is used. A torque is applied to the shaft of the vane until failure occurs due to the shearing on the cylinder of diameter d, and height, h. Vane blades are pushed into the soil and rotated at a constant rate of 1° per minute by a worm gear and wheel arrangement.

The shearing strength of clay (c) is computed using:

$$\text{Torque} = T = c(\pi dh)\frac{d}{2} + c(\pi\frac{d^2}{4})\frac{1}{3}d \times 2$$

(sides of cylinder) (ends of cylinder)

$$c = \frac{T}{\pi d^2(\frac{h}{2}+\frac{d}{6})} \qquad 13.9$$

Where only the bottom end is sheared.

$$c = \frac{T}{\pi d^2(\frac{h}{2}+\frac{d}{12})} \qquad 13.10$$

Where,

T = maximum torque at failure in Kg-cm or ft-lb

h = height of vane in cm or ft.

d = diameter of vane in cm or ft.

13.10 LABORATORY SHEAR TEST CONDITIONS

As stated already that shearing strength of soils is greatly influenced by loading and drainage conditions maintained during the test. With respect to loading and drainage conditions, the laboratory tests can be divided into the following three main categories:

(a) Unconsolidated-Undrained (UU Test) or *Quick Shear Test.*

(b) Consolidated-Undrained (CU Test) or *Consolidated Quick Test.*

(c) Consolidated-Drained (CD Test) or *slow Test.*

Mohr circle diagrams for these three test conditions are given in Fig. 13.9 and brief descriptions are discussed in the succeeding sections.

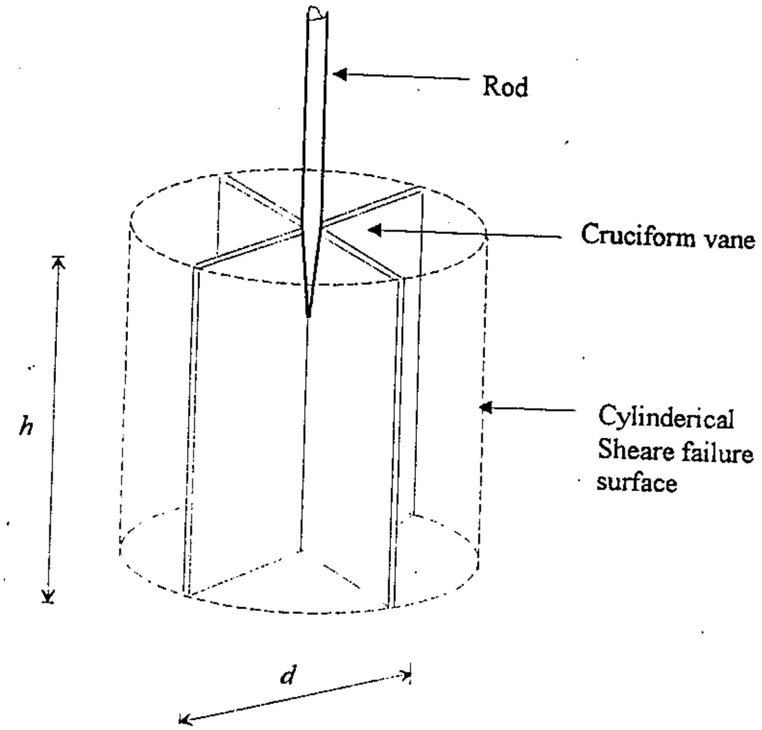


Figure 13.13 *Vane shear test apparatus*

• **UU-Test (Quick Test)**

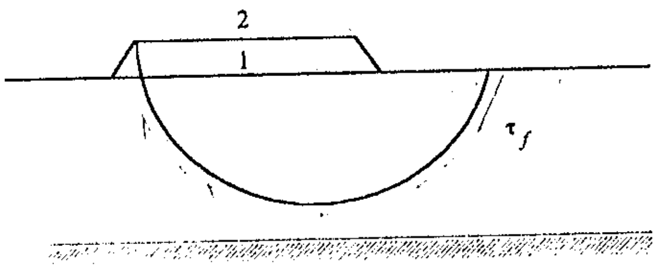
In this test no drainage is allowed as the testing proceeds to failure. Soil sample is sheared immediately after application of the normal load and no time for sample consolidation is allowed either before or during the shear test. To assure that during testing the void ratio of the soil sample would change as little as possible, the shearing force is applied rapidly and the entire test is completed within a period of about 5 to 10 minutes. Usually pore pressure are not measured in this test. The test is a *total stress test* and it yields total stress shear parameters (c_u, ϕ_u); but in principle, it is possible to measure pore pressure in UU tests.

In engineering practice, we mostly have to deal with a relatively quick shear loading where the excess pore pressure has no time to dissipate, or there is no time to adjust or equalize the pore pressure. Under such conditions the resistance of an earth mass to sliding under certain conditions result in smaller values than those obtained from tests. Therefore, the most unfavourable conditions are to be considered, the sudden loading of a soil mass till failure. Accordingly the shear test in the laboratory is to be performed quickly. Typical examples of the use of this test are the determination of shear strength in temporary excavations; calculation of bearing capacity of cohesive soils used in the design of foundations; and slope stability analysis of earth dams during construction.

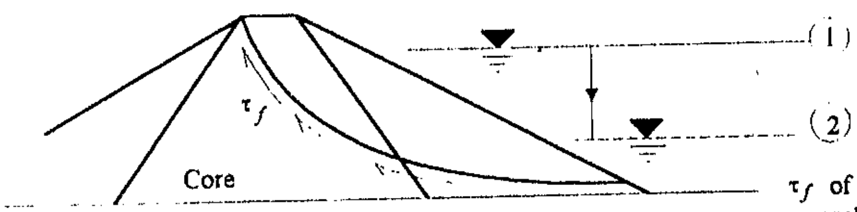
• **CU-TEST (THE CONSOLIDATED-QUICK TEST)**

In this test the normal load is applied and the sample is allowed to consolidate allowing drainage during consolidation process to reduce pore pressure to zero. Once this condition is reached, one normal stress is increased rapidly without allowing any. drainage, until the sample fails. Provided pore pressure is measured during the shearing phase, the results can be expressed in terms of total or effective shear parameters (i.e. $\bar{c}_{cu}$ & $\bar{\phi}_{cu}$).

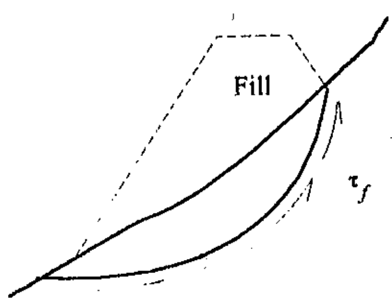
In practice CU strengths are used for stability problems where the soils have first become fully consolidated and are at equilibrium with the existing stress system. Then for some reasons *additional* stresses are applied quickly, with no drainage occurring. Practical examples include stability slopes of earthen dams during rapid drawn down. Fig. 13.14 represents some examples where effective stresses of CU test results are applied.



(a) Embankment raised (2) subsequent to consolidation under its original height (1).



(b) Rapid drawdown behind an earth dam. No drainge of the core. Reservoir level falls from 1 to 2.



(c) Rapid construction of an embankment on a natural slope

Figure 13.14 *Some examples of CU analysis for clays (after Ladd, 1971)*

CD-Test (Slow Test)
In CD test, soil consolidates under normal load and the drainage is allowed during consolidation. On completion of consolidation, drainage conditions to be allowed while one normal stress is increased at a rate such that no pore pressure can develop. The resulting shear strength parameters are in terms of effective stresses only (i.e. $\bar{c}_d$ & $\bar{\phi}_d$).

In practice, CD parameters are used in long term stability problems of clayey soil slopes and the long term lateral pressures on walls retaining cohesive soils. Examples of CD test use are given in Fig. 13.15.

Slow test on clays may take 4 to 6 weeks to complete and this test is usually used in research. This test is not very popular for clays:

Since non-cohesive soils are free draining soils and consolidate quickly, these soils are tested usually by CD test.

13.11 FIELD SHEAR TESTS.

Undisturbed sampling of non-cohesive soils and soft sensitive clays, if not impossible, is difficult and expensive. Usually shear strength of these is determined using field shear tests. Now-a-days numerous varieties of field shear tests are conducted for this purpose. Several very common routine tests are:-

(i) Standard Penetration Test, SPT (ASTM D1586).

This test provides fairly good estimate of shear strength of non-cohesive soils.

(ii) Cone Penetration Test, CPT (ASTM D3441)

Used for soft clays and loose to medium dense sands.

(iii) Field Vane Shear Test, FVST (ASTM D2573)

(iv) Pressuremeter Test PMT (Menard 1956)

Used for variety of soils and rocks

Brief descriptions of these tests have been given in Chapter-14 of this book.

13.12 SHEAR STRENGTH OF SANDS AND CLAYS

- **Sands**

Due to relatively large particle size (less surface area) non-cohesive soils have little or no cohesion and their shear strength is mainly due to frictional resistance between the particles including sliding and rolling friction as wall as interlocking of the grains. Thus the major contributing parameter towards the shear strength of granular soils such as sand is the internal friction angle, (ϕ) and the cohesion, $c = 0$

The most critical condition with regard to shear strength of non-cohesive soils occurs at construction stage or upon application of load. In the case of non-cohesive soils, any water contained in the voids at construction time or upon application of load will be drained out almost immediately due to high permeability of the soils. As a result the shear strength parameters are w.r.t. effective stress. Thus, the shear strength of non-cohesive soils will remains more or less the same throughout the life of the structure.

A dense sand tends to expand (dilates) during shear while a loose sand will decrease in volume. Dense sands have higher shear strength than that of loose sands.

Fig. 13.16 represents typical stress-strain curves for loose and dense sands of CD tests performed using triaxial apparatus. Testing a saturated cohesionless sample in UU or CU condition is meaningless.

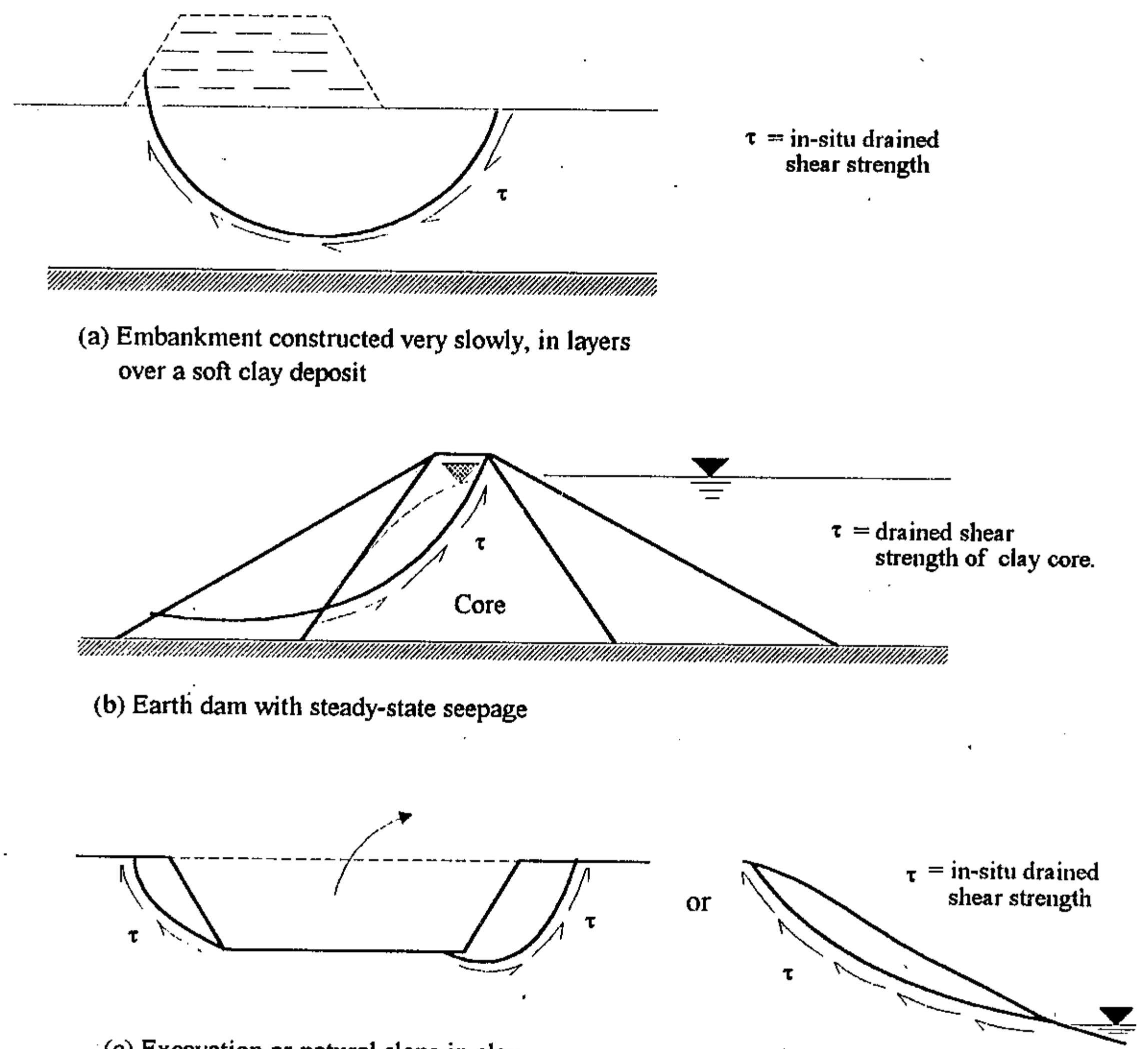


Figure 13.15 *Some examples of CD analyses for clays (after Ladd, 1971)*

Also Table 13.1. presents the listing of some typical field problems with appropriate shear strength parameter to be used.

Table 13.1 Typical field problems and appropriate shear strength parameters

Type of construction	Load	Critical time	Analysis and comment
Foundation on saturated clay and passive earth pressures on retaining walls	Positive	End of construction	Total stress c_u, $\phi_u = 0$, analysis gives acceptable solutions.
Earth dam construction. Embankment fill. May involve construction with several loading periods	Positive	End of construction	Effective stress $\overline{c}$, $\overline{\phi}$. Stage construction will give some dissipation of excess pore-water pressure. Hence measure pore-water pressure over field range to check the factor of safety for each stage.
Earth pressures on walls backfilled with partially saturated materials ($\phi_u \neq 0$)	Positive	End of construction or long-term	Total stress, c_u, $\phi_u = 0$. Effective stress $\overline{c}$, $\overline{\phi}$. Check any seepage pore-water pressures.
Active pressure on driven and then dredged sheet pile walls	Negative	Usually long-term but possibly during construction also.	Effective stress $\overline{c}$, $\overline{\phi}$ Pore-water pressure may develop from water table behind the wall.
Permanent cuts. Stability of natural slopes	Negative	Long-term	Effective stress $\overline{c}$, $\overline{\phi}$ or preferably c_d, ϕ_d. Pore-water pressures from steady seepage or static condition may develop. $\overline{c}$ is often not reliable and taken as zero. With some fissured over-consolidated clays the remoulded parameters. $c_r = 0$, ϕ_r should be used.
Temporary excavations. Slope stability. Base heave of intact clays.	Negative	During construction	Total stress c_u, $\phi_u = 0.$, c_u preferably measured using unloading triaxial test.
Temporary excavations. Slope stability. Base heave of non-intact clays.	Negative	During construction	Effective stress $\overline{c}$, $\overline{\phi}$. Quick drainage of non-intact materials makes an undrained analysis unreliable. Often requires pore-water pressures to be estimated which may prove to be difficult.

Note: (i) Positive load is the structural load added due to construction.
(ii) Negative load is load removed during excavation.

Failure may be defined as:

(1) maximum principal stress difference, $(\sigma_1 - \sigma_3)_{max}$

(2) maximum principal effective stress ratio, $\left(\frac{\overline{\sigma}_1}{\overline{\sigma}_3}\right)_{max}$

(3) $\tau = \frac{(\sigma_1 - \sigma_3)}{2}$ at a prescribed strain.