Chapter 12

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SOIL STRENGTH PROPERTIES AND THEIR MEASUREMENT

1. INTRODUCTION

Methods of limiting equilibrium are frequently used to analyze the stability of a soil mass (see Chapter 13). In such analyses, the shear strength of the material is assumed to be fully developed along the rupture surface at failure. In this chapter the basic principles that govern shear strength and the methods that may be used for its measurement are outlined. Brief descriptions of the properties of some common soils are provided.

2. GENERAL PRINCIPLES

The basic principles in the description of strength properties are the failure criterion and the effective stress principle. When a failure criterion derived from test data is used to estimate in situ strength, appropriate attention should be given to possible differences between the stress state of the test and that of the in situ soil when subjected to the expected load.

2.1 Failure Criterion

The Mohr-Coulomb criterion is widely used to define failure; it states that the shear strength (s) is

$$s = c + \sigma \tan\phi \qquad (12.1)$$

where

σ = normal stress on rupture surface,
c = cohesion, and
ϕ = angle of internal friction.

In terms of principal stresses, the Mohr-Coulomb criterion becomes

$$\sigma_1 = \sigma_3 \tan^2\left[\left(\frac{\pi}{4}\right)+\left(\frac{\phi}{2}\right)\right] + 2c\tan\left[\left(\frac{\pi}{4}\right)+\left(\frac{\phi}{2}\right)\right] \qquad (12.2)$$

where σ_1 is the major principal stress and σ_3 is the minor principal stress.

A more general formulation, which combines failure with stress-strain behavior, is the yield surface (Drucker et al. 1955) and the critical state (Schofield and Wroth 1968). The yield surface is especially useful when evaluation of deformation is required. For limiting equilibrium analysis, the Mohr-Coulomb criterion is still the most convenient failure criterion.

2.2 Effective Stress Versus Total Stress Analysis

Because the shear strength of soils is strongly influenced by drainage conditions during loading, those conditions must be properly accounted for in the use of shear strength in design. A fundamental principle in soil engineering is the use of effective stress σ', which was first defined by Terzaghi (1936a) as

$$\sigma' = \sigma - u \qquad (12.3)$$

where σ is the total stress and u is the pore pressure. The shear strength can be expressed consistently in terms of effective stress:

$$s = c' + \sigma' \tan\phi' = c' + (\sigma - u) \tan\phi' \qquad (12.4)$$

where c' and ϕ' are the strength parameters for effective stress. For a partially saturated soil, the shear strength can be expressed as (Fredlund et al. 1978)

$$s = c' + (\sigma - u_w) \tan\phi' + (u_w - u_a) \tan\phi'' \qquad (12.5)$$

where

u_a = pore-air pressure,
u_w = pore-water pressure, and
ϕ'' = soil property that reflects influence of suction $(u_w - u_a)$ on strength.

When the soil is saturated, $u_a = 0$ and $u = u_w$. For saturated soils, pore pressure consists of the hydrostatic pore pressure related to groundwater level and the excess pore pressure due to applied loads.

When soils are loaded under undrained or partially drained conditions, the tendency to change volume results in an excess pore pressure, which may be positive or negative depending on the type of soil and the stresses involved. General relations between pore pressure and applied stresses have been suggested. For example, Henkel (1960) proposed that

$$\Delta u = B(\Delta\sigma_{oct} + \alpha \Delta\tau_{oct}) \qquad (12.6)$$

where

α = empirical coefficient,
$\tau_{oct} = \frac{1}{3}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{1/2}$,
$\sigma_{oct} = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$, and
$\sigma_1, \sigma_2, \sigma_3$ = major, intermediate, and minor principal stresses.

For soils tested in the triaxial apparatus or loaded so that $\Delta\sigma_2 = \Delta\sigma_3$, Skempton (1954) proposed that the excess pore pressure be given by

$$\Delta u = B[\Delta\sigma_3 + A\Delta(\sigma_1 - \sigma_3)] \qquad (12.7)$$

where A is an empirical coefficient related to the excess pore pressure developed during shear and B is an empirical coefficient related to the soil's compressibility and degree of saturation. For saturated soils, $B = 1$. For an elastic material, $A = 1/3$. For soils that compress under shear, $A > 1/3$, and for soils that dilate under shear, $A < 1/3$.

Under the fully drained condition, the excess pore pressure is zero, and pore pressure in saturated soils caused by groundwater flow can usually be evaluated without serious difficulty. Hence, analysis with the effective stress description of shear strength (Equation 12.4) is most useful. For partially drained and undrained conditions, the evaluation of excess pore pressure is often difficult. In some cases, a total-stress description of shear strength may be used. One important case is the undrained loading of saturated soils, for which the undrained shear strength ($s = s_u$) can be used. This is the common $\phi = \phi_u = 0$ analysis (Skempton and Golder 1948). The shear strength usually changes as the void ratio changes with drainage. If the change results in a higher strength, the short-term, undrained stability is critical and the stability can be expected to improve with time. On the other hand, if drainage produces a decrease in strength, the long-term, drained stability is critical; the undrained shear strength can be used only for short-term or temporary stability. For partially saturated soils, the prediction of pore-air and pore-water pressures is more difficult. Currently, the only reliable method is in situ measurement.

2.3 Common States of Stress and Stress Change

The Mohr-Coulomb criterion does not indicate any effect of the intermediate principal stress (σ_2') on the shear strength. In practical problems, σ_2' may range from σ_3' to σ_1', depending on the geometry of the problem. The direction of the major principal stress also changes during loading. Experimental studies show that the value of σ_2' relative to σ_3' and σ_1' has an influence on the shear strength.

Several common states of stress are shown in Figure 12-1. In the initial state (*a*), σ_z' is the effective overburden pressure, $\sigma_r' = K_o\sigma_z'$ is the radial or lateral pressure, and K_o is the coefficient of earth pressure at rest. In the stress state beneath the center of a circular loaded area [Figure 12-1(*b*)], the vertical stress is the major principal stress and the radial stress σ_r' is the minor principal stress. The

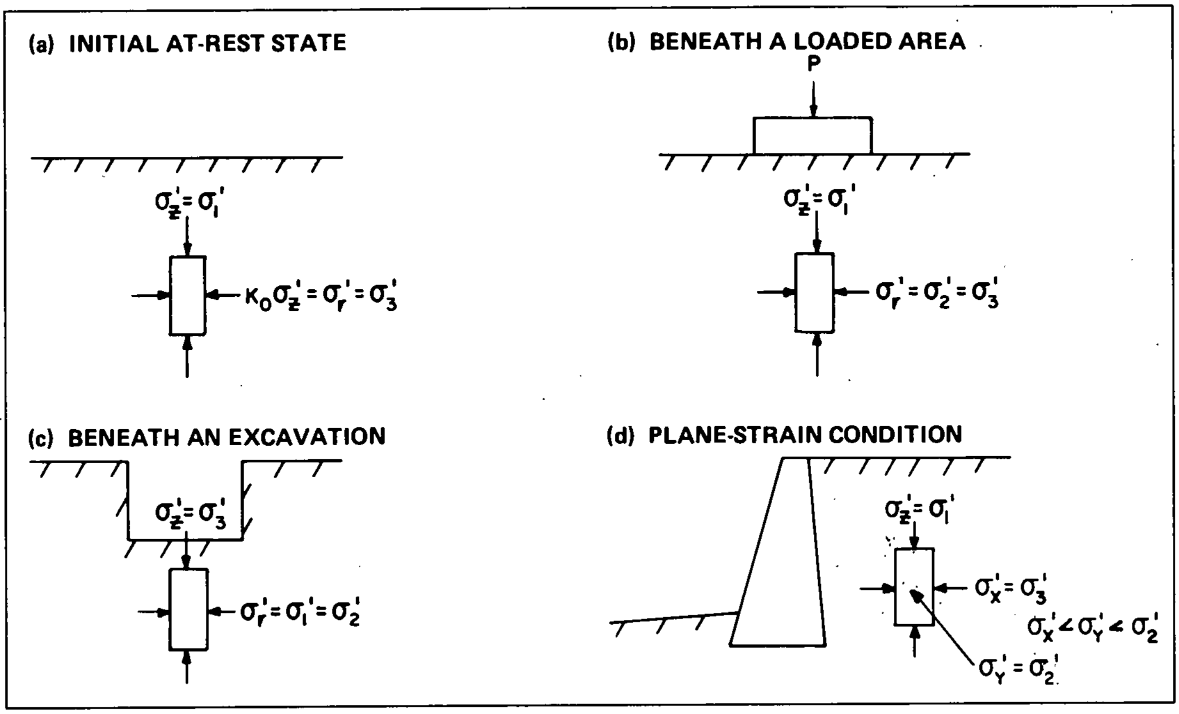


FIGURE 12-1 Common states of stress.

intermediate principal stress (σ_2') is equal to the minor principal stress (σ_3'). In the stress state below the center of a circular excavation [Figure 12-1(c)], the vertical stress is the minor principal stress and the radial stress σ_r' is the major principal stress. The intermediate principal stress (σ_2') is equal to the major principal stress (σ_1'). Slopes and retaining structures can be approximated by the plane-strain condition in which the intermediate principal strain (ε_2) is zero. Then the intermediate principal stress (σ_2') is σ_y', oriented as shown in Figure 12-1(*d*), and has a value between σ_1' and σ_3'.

Another important feature in many stability problems is the rotation of the principal axes during loading or excavation and its effect on the shear strength of soft clays (Ladd and Foott 1974). The rotation of principal axes is shown in Figure 12-2. Before the excavation of the cut, the state of stress is represented by that shown in Figure 12-1(*a*). After excavation, the major principal stress is in the horizontal direction at the toe (Point A, Figure 12-2). Thus, the principal axes are rotated through an angle of 90 degrees; at Point B, a rotation of approximately 45 degrees occurs. At Point C, the original principal stress directions remain unchanged although the values of the stresses change.

2.4 Stress-Strain Characteristics

Two stress-deformation curves are shown in Figure 12-3. A soil sample is sheared under a normal stress σ and a shear stress τ. The shear displacement is Δ. In common practice, the strength of the soil is defined as the peak strength (Points *a* and *b* in Figure 12-3) measured in the test. When this is used in a stability analysis, the tacit assumption is that the peak strength is attained simultaneously along the entire rupture surface.

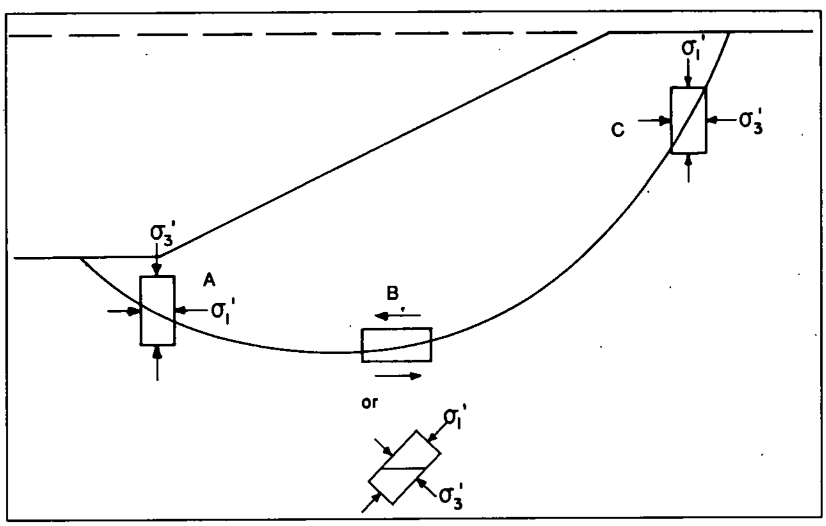


FIGURE 12-2 Directions of principal stresses in a slope.

Many soils demonstrate strain-softening behavior, as illustrated by Curve A in Figure 12-3. Any of several mechanisms may be used to explain the strength decrease, but it is important to account for this decrease in design. For strain-softening soils, it is unreasonable to assume that the soil reaches its peak strength simultaneously at all points along a failure surface. In fact, the soil at some points on the rupture surface will suffer displacements greater than Δ_a before the soil at other points reaches this deformation. This phenomenon was called "progressive failure" by Terzaghi and Peck (1948). In the limit of large deformation, the strength at all points will be reduced to that represented by Point c in Figure 12-3. This strength is called the residual strength.

2.5 Effect of Rate of Loading

The difference between the rate of loading applied in a laboratory shear test and that experienced in the slope is usually substantial. Most laboratory and in situ tests bring the soil to failure within several hours or at most a few days. For most slopes the load is permanent, although some dynamic loads may be applied for short durations. The effect of rate of loading on soil strength, excluding effects of consolidation, may be significant. In general, the undrained strength of soils increases as the rate of loading increases; however, this effect depends on the specific material and varies over a wide range (Casagrande and Wilson 1951; Skempton and Hutchinson 1969).

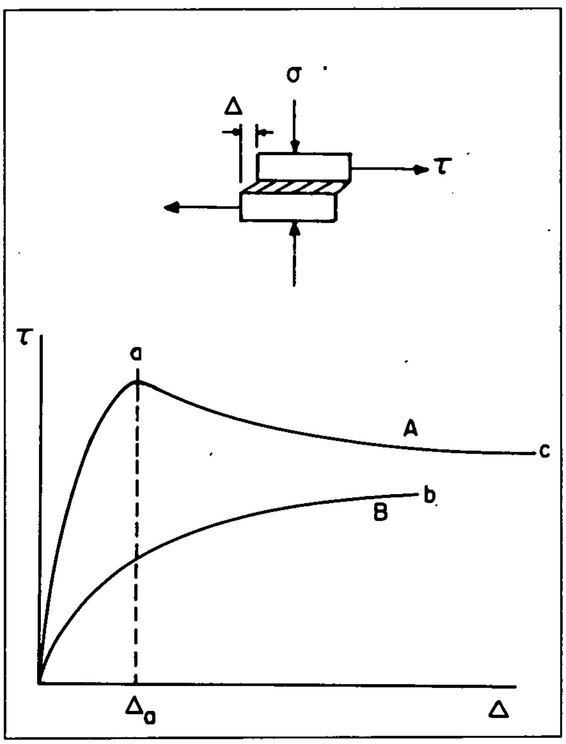


FIGURE 12-3 Typical stress-deformation curves.

3. LABORATORY MEASUREMENT OF SHEAR STRENGTH

Various methods are available for laboratory measurement of shear strength. The simple methods are designed to determine the shear strength of a sample in a particular state, such as the water content of the soil in situ. These methods are most often used to determine the undrained shear strength (s_u) of saturated cohesive soils. More elaborate laboratory tests allow combinations of normal and shear stresses to be employed and pore pressures to be measured or controlled. Then it is possible to establish the shear strength relation defined by Equation 12.4. The most elaborate tests allow simulation of the field stress or deformation conditions. For example, triaxial compression tests simulate the stress state in Figure 12-1(*b*), and triaxial extension tests simulate the stress state in Figure 12-1(*c*) (see Section 3.2). Plane-strain compression and extension tests and simple shear tests can provide better simulation of the stress states in Figure 12-2 (see Section 3.3). A comprehensive summary of stress states in various laboratory tests and constitutive relations that can be deduced from test results was given by Desai and Siriwardane (1984).

3.1 Simple Shear Tests

Three types of simple shear tests are the unconfined compression, cone, and vane shear tests.

The unconfined compression test is usually performed on a cylindrical sample with a diameter-to-length ratio of 1:2. The sample is compressed axially [Figure 12-4(*a*)] until failure occurs; the shear strength is taken as one-half the compressive strength.

In the cone test, a cone with an angle θ is forced into the soil [Figure 12-4(*b*)] under a force (Q), which may be its own weight. The shear strength is obtained from the relation

$$s_u = \frac{KQ}{h^2} \tag{12.8}$$

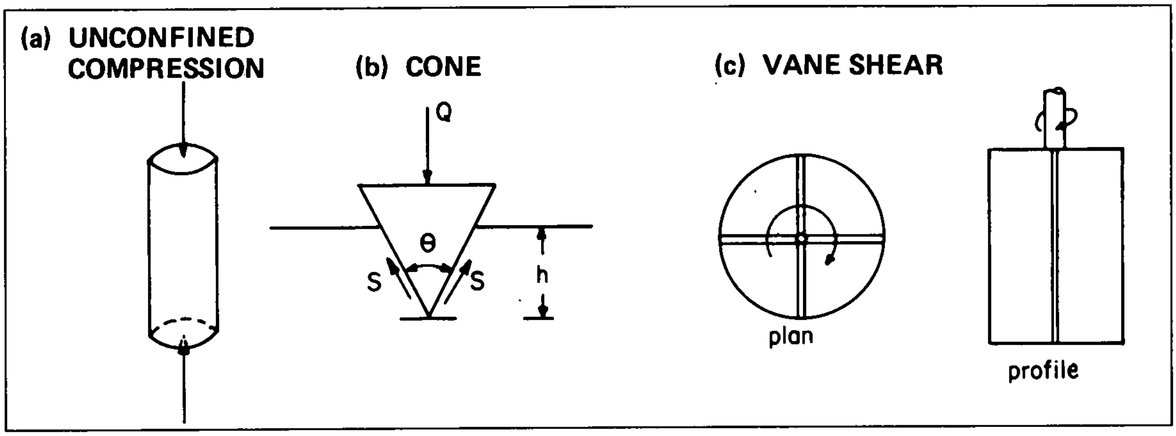


FIGURE 12-4 Simple tests for measurement of soil strength.

where h is the penetration and K is the constant that depends on the angle θ and the weight Q. Calibration curves for K were published by Hansbo (1957).

In the laboratory vane test (Cadling and Odenstad 1950), a vane is pushed into the soil specimen, and a torque is applied to the stem to produce shear failure over a cylindrical surface [Figure 12-4(c)]. The shear strength is obtained by equating the torque measured at failure to the moment produced by the shear stresses along the cylindrical surface. According to Cadling and Odenstad, the shear strength for vanes with a diameter-to-height ratio of 1:2 is

$$s = \frac{6}{7}\left(\frac{M}{\pi D^3}\right) \tag{12.9}$$

where M is the torque and D is the diameter of the vane.

Application of the results of these simple tests to the analysis of slope stability should include consideration of the type of soil and loading conditions in situ. The test results are expressed in terms of total stress because pore pressures are not measured. When the soil is brought to failure rapidly under undrained conditions, the shear strength is defined by $s = s_u$. Hence, the application of these test results is commonly limited to saturated cohesive soils under undrained conditions.

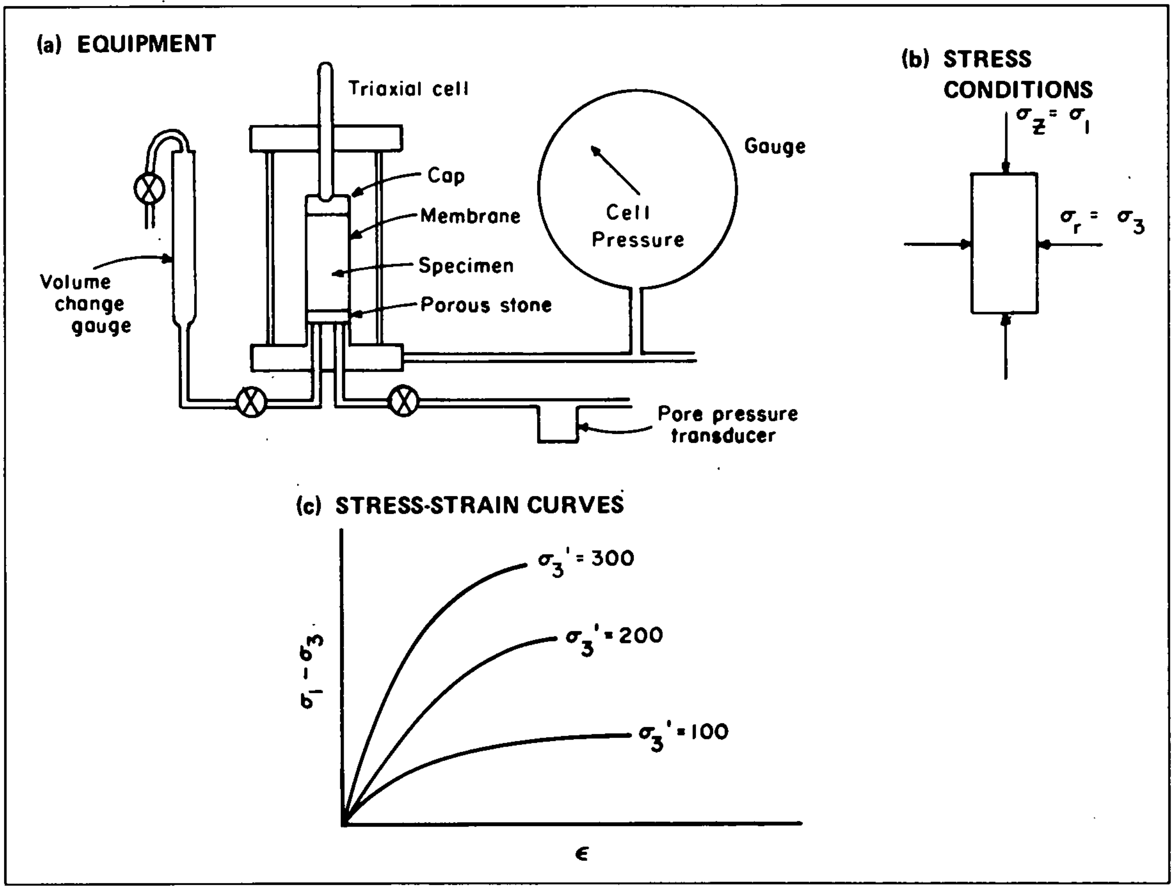
It is usually assumed that the measured strength is equal to the in situ strength. However, a major uncertainty is the effect of sampling disturbance on strength. Even "good" samples may suffer strength losses as great as 50 percent (Ladd and Lambe 1964; Clayton et al. 1992; Hight et al. 1992). The effect of sample disturbance is most severe in soft sensitive soils and appears to increase with the depth from which the sample is taken. The state of stress is another factor that should be considered. The directions of the principal stresses and the orientations of rupture surfaces in these simple tests are not the same. They may also be quite different from directions of the principal stresses and the orientation of the failure surface in a slope. Hence, caution should be exercised when the results of these simple strength tests are applied to slope-stability problems (Bjerrum 1973).

3.2 Triaxial Test

The triaxial test is highly versatile; a variety of stress and drainage conditions can be employed (Bishop and Henkel 1962). The cylindrical soil specimen is enclosed within a thin rubber membrane and placed inside a triaxial cell, as shown in Figure 12-5(a). The cell is then filled with a fluid. As pressure is applied to the fluid in the cell, the specimen is subjected to a hydrostatic stress (σ_3). Drainage from the specimen is provided through the porous stone at the bottom, and the volume change can be measured. Alternatively, if no drainage is allowed, the pore-water pressure can be measured. For partially saturated soils, the pore-water pressure is measured through a fine ceramic stone that prevents air entry. The pore-air pressure is measured through a screen that permits free air entry. Details on testing of partially saturated soils were given by Fredlund and Rohardjo (1993, 260–282).

In triaxial compression, the axial stress (σ_z) is increased by application of a load through the loading ram. From the known stresses at failure ($\sigma_1 = \sigma_z$ and $\sigma_2 = \sigma_3 = \sigma_r$), Mohr circles or other stress plots can be constructed. Results from several triaxial tests, each using a different cell pressure (σ_3), are

FIGURE 12-5
Triaxial test.



used to obtain the failure envelope. Typical plots of the principal stress difference versus axial strain are shown in Figure 12-5(*c*). The stress-strain behavior is influenced by the confining pressure, the stress history, and other factors. The sample can also be loaded to failure in triaxial extension by increasing the radial stress while maintaining the axial stress constant; then $\sigma_1 = \sigma_2 = \sigma_r$ and $\sigma_3 = \sigma_z$. Triaxial compression and extension simulate, respectively, the stress states beneath a loaded area, as shown in Figure 12-1(*b*), and beneath an excavation, as shown in Figure 12-1(*c*).

Principal stresses and pore pressure at failure are used to construct Mohr circles and to obtain the failure envelope. Figure 12-6 shows a series of failure envelopes. The horizontal axis is $\sigma - u_w$, which is the familar pore pressure for saturated soils. For saturated soils $(u_a - u_w) = 0$. When the soil is partially saturated, $(u_a - u_w)$ is not zero. A series of failure envelopes is obtained for different values of $(u_a - u_w)$, such as those shown in Figure 12-6 for values of $(u_a - u_w)$ equal to *a* and *b*.

Tests with the following drainage conditions can be performed with the triaxial apparatus. Tests on saturated soils are considered first.

In the *consolidated-drained test* (also called a drained test or slow test), the soil is allowed to consolidate completely under an effective cell pressure (σ_3'). In the triaxial compression test, the axial stress is increased at a slow rate, and drainage is permitted. The rate should be slow enough that no excess pore pressure is allowed to build up. Since

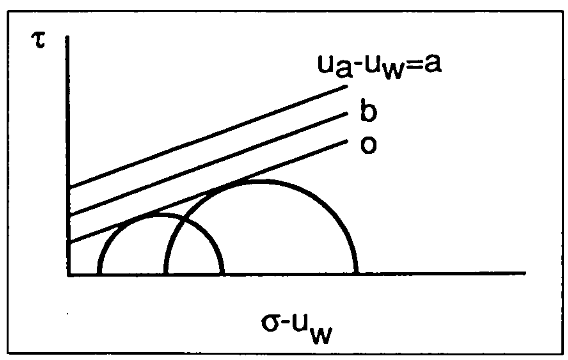


FIGURE 12-6
Failure envelopes.

the excess pore pressure is zero in the drained test, effective stresses are known throughout the test and at failure. The results of a series of drained tests can be used to evaluate the effective stress strength parameters in Equation 12.4.

In the *consolidated-undrained test* with pore-pressure measurement (also called the consolidated-quick test), the drainage valves are closed after the initial consolidation of the sample. The axial stress is increased without drainage, and the excess pore pressure is measured. The pore pressure is subtracted from the total axial and radial stresses to give the effective stresses. The effective stresses at failure from a series of tests are used to define the failure criterion, as in the drained test. The consolidated-undrained test is sometimes performed without pore-pressure measurement. Then the effective stresses at failure cannot be determined. The results could be used to estimate the undrained shear strength after consolidation under σ_3'.

In the *unconsolidated-undrained test* (also called the undrained test or quick test), no drainage is allowed during any part of the test. When the cell pressure is applied, a pore-pressure change occurs in the soil. When the axial stress is applied, additional pore-pressure changes occur. If these pore-pressure changes are not measured, the test results can only be interpreted in terms of total stress. At failure, the undrained shear strength (s_u) is taken to represent the strength at the in situ water content. The unconsolidated-undrained test is then similar to the simple shear tests defined in Section 3.1.

The same three types of triaxial tests can be performed on partially saturated soils. However, because the air voids can be compressed even when there is no drainage, the simplifications derived above for undrained loading do not apply to partially saturated soils. For such soils, the pore-air and pore-water pressures must be measured in all tests in order to construct the failure envelope in terms of effective stress.

Triaxial tests are usually begun by increasing the cell pressure (σ_3) to the desired stress level, thus applying an isotropic or hydrostatic stress to the sample. This initial condition may differ from the initial in situ condition [see Figure 12-1(*a*)], in which the vertical and horizontal principal stresses are different. In situ stresses can be simulated in a triaxial test by using an anisotropic stress state during consolidation. Experimental results that show the effect of stress state on strength properties were reviewed by Ladd et al. (1977).

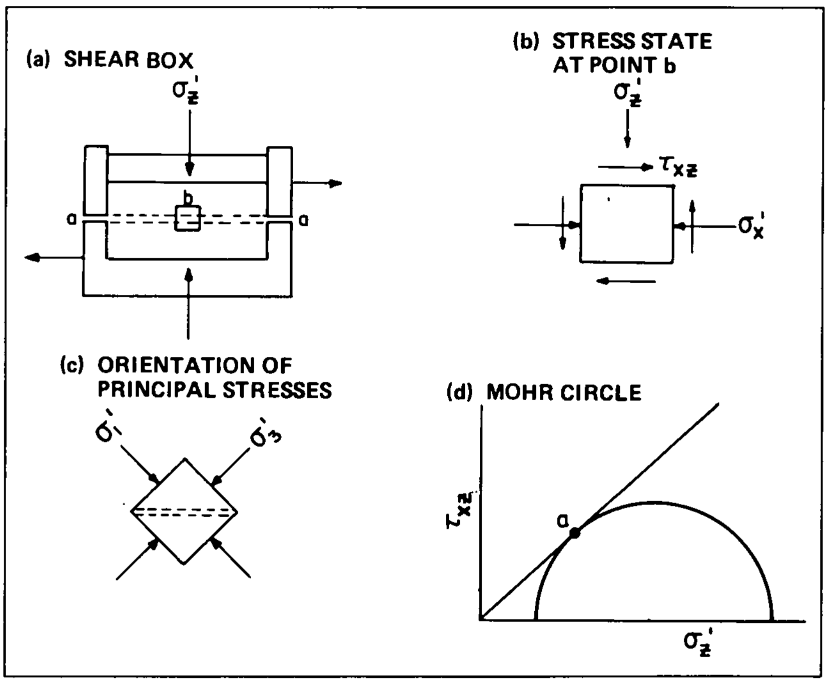
3.3 Plane-Strain Test

The geometry of many geotechnical problems can be approximated by the condition of plane strain, in which the intermediate principal strain (ε_2) is zero. To simulate this condition, plane-strain tests have been developed (Henkel and Wade 1966) in which the sample is consolidated anisotropically with zero lateral strain ($\varepsilon_x = \varepsilon_y = 0$). Then the sample is loaded to failure by increasing either σ_x or σ_z and maintaining $\varepsilon_y = 0$ to simulate, respectively, the stress conditions at Points A or C of Figure 12-2. Plane-strain tests can be conducted under undrained, consolidated-undrained, or drained conditions as described for triaxial tests.

3.4 Direct Shear Test

The direct shear test is shown in Figure 12-7. The soil specimen is enclosed in a box consisting of upper and lower halves; porous stones on the top and the bottom permit drainage of water from the specimen. If the loading is carried out slowly, no excess pore pressure develops and the drained condition is obtained. The potential plane of failure is *a-a*. A normal stress (σ_z') is applied on plane *a-a*

FIGURE 12-7
Direct shear test.



through a loading head, and the shear stress is increased until the specimen fails along plane *a-a*. A stress-deformation curve is obtained by plotting the shear stress versus the displacement. Because the thickness of shear zone *a-a* is not precisely known, the shear strain cannot be determined. The test gives the value of τ_{xz} at failure. The average vertical stress (σ_z') and shear stress (τ_{xz}) are known, but σ_x' is not known. The directions of the principal stresses for Point *b* [Figure 12-7(*b*)] are approximately as shown in Figure 12-7(*c*). Assuming that Point *a* [Figure 12-7(*d*)] represents the conditions at failure, a Mohr circle can be constructed. The failure envelope is obtained from several tests, each using a different effective normal stress, performed on specimens of the same soil. The values of τ_{xz} at failure are plotted against the values of σ_z'. The foregoing represents the common interpretation of the direct shear test. More elaborate analyses were presented by Hill (1950), Morgenstern and Tchalenko (1967), and Jewell and Wroth (1987).

In saturated clays the direct shear test can be performed at a rapid rate so that the test duration is too short for any appreciable amount of water to flow into or out of the sample. This is an undrained condition, and an excess pore pressure of unknown magnitude is developed in the soil. Consequently, this is essentially a simple test, and the shear stress at failure represents the undrained shear strength (s_u). Test procedures for partially saturated soils were given by Fredlund and Rohardjo (1993, 282–254).

3.5 Simple Shear Test

Several simple shear tests have been developed, but the one described by Bjerrum and Landva (1966) is the most commonly used for undisturbed samples. The cylindrical specimen is enclosed in a rubber membrane reinforced by wire. This enclosure allows the shear deformation to be distributed fairly uniformly through the sample, as shown in Figure 12-8(*a*). The sample is consolidated anisotropically under a vertical stress [Figure 12-8(*a*)] and sheared by application of stress τ_{xz} [Figure 12-8(*b*)]. The simple shear test can be performed under undrained, consolidated-undrained, and drained conditions. Zero volume change during shear in an undrained test can be maintained by continuously adjusting the vertical stress σ_z' during the test. In the simple shear test, the initial principal axes are in the vertical and horizontal directions. At failure the horizontal plane becomes the plane of maximum shear strain. This condition approximates that at Point *B* of Figure 12-2.

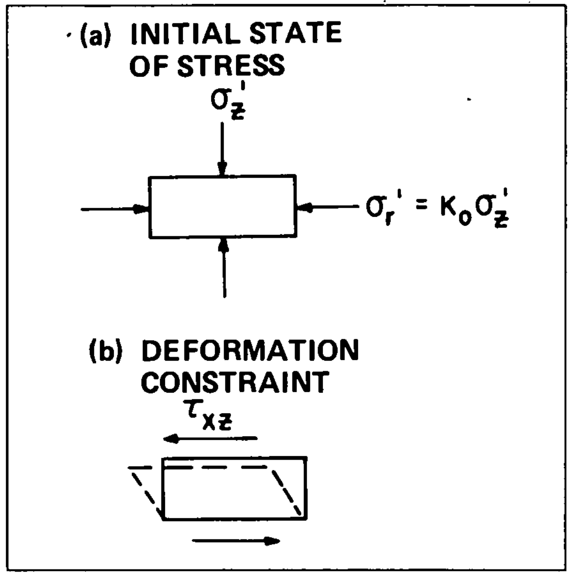


FIGURE 12-8
Simple shear test.

4. IN SITU MEASUREMENT OF SHEAR STRENGTH

A variety of in situ tests are available for measurement of shear strength. Several of these are described in Chapter 10. Although theories have been proposed to interpret the test results, the stress state in most in situ shear tests is not precisely known. Hence empirical correlation is often relied upon for interpretation of the results. Various correlations have been published for different soils. For a comprehensive review of in situ tests, see the discussions by Orchant et al. (1988) and Lunne et al. (1989).

4.1 Standard Penetration Test

The standard penetration test uses a split-spoon sampler, which is driven with a specified energy into the bottom of a borehole. The number of blows (*N*) required to advance the sampling spoon through a distance of 30 cm may be used to estimate the angle of internal friction ϕ' of sands (Peck et al. 1974). More data are available on the correlation of *N* with relative density D_r (Gibbs and Holtz 1957; Marcuson and Bieganousky 1977). Correlation between D_r and ϕ' was given by Schmertmann (1975). Correlation between *N* and shear strength of cohesive soils is not reliable.

4.2 Cone Penetration Test

In the cone penetration test a cone of specified diameter and shape is attached to the end of a drill rod and advanced at a controlled rate. The force required is measured. This test can provide a continuous record of the resistance. For cohesionless soils, bearing capacity equations can be used to compute ϕ' from the penetration resistance. Empirical correlations between cone resistance and ϕ' are also available. Different correlation equations can give very different values of ϕ' (Mitchell and Lunne 1978; Wu et al. 1987a). The undrained shear strength of cohesive soils can be estimated from the cone resistance by the following relation:

$$s_u = \frac{q_c - \sigma_v}{N_k} \tag{12.10}$$

where

s_u = undrained shear strength,
q_c = cone resistance,
σ_v = total overburden stress, and
N_k = an empirical constant.

The value of N_k ranges between 10 and 20 and depends on the overconsolidation ratio and plasticity index (Aas et al. 1986).

4.3 Field Vane Test

The field vane test is similar to the laboratory vane test described in Section 3.1. It measures the shear strength along a vertical cylindrical surface and the top and bottom surfaces of the cylinder. An equation similar to Equation 12.9 is used to calculate the shear strength. The coefficient depends on the assumption for stress distribution on the cylindri-cal surface and the top and bottom surfaces (Donald et al. 1977). Since the rupture surface in a stability problem may be quite different from that in a vertical cylinder, the shear strength may also be different from that measured in the vane test because of stress and material anisotropy. Correction factors based on an analysis of the stress state were suggested by Wroth (1984), and empirical correction factors were suggested by Bjerrum (1973).

4.4 Pressuremeter Test

In the pressuremeter test (Menard 1965; Baguelin et al. 1972), the cylindrical instrument is expanded against the wall of the borehole until failure in the surrounding soil occurs. Elastic-plastic solutions for expansion of a cylindrical cavity by Gibson and Anderson (1961) and subsequent modifications are used to obtain the shear strength and stress-strain properties of the soil. Consistent results have been obtained for the undrained shear strength of saturated clays and drained shear strength of cohesionless soils (Wroth 1984).

4.5 Borehole Shear Test

The borehole shear test is analogous to the direct shear test. The two halves of a cylinder, which is split lengthwise, are pressed under a controlled pressure against the opposite walls of a borehole. The cylinder is then pulled vertically, shearing the soil (Handy and Fox 1967). The failure surface is not determined; hence the normal and shear stresses are not accurately known. For cohesionless soils with high permeability, the test may be considered a consolidated-drained test. For cohesive soils, the drainage condition is uncertain. The borehole shear test was analyzed by Lutenegger and Hallberg (1981) and Handy et al. (1985).

4.6 Dilatometer Test

The dilatometer is a flat blade that contains a flexible membrane on one side (Marchetti 1975). The blade is pushed into the soil to reach the desired depth, and the membrane is inflated by pressure and pushes back the adjacent soil. The pressures required to "lift off" the membrane and to displace the soil 1 mm are recorded. These pressures are used to estimate the horizontal in situ stress. Empirical correlation between the horizontal in situ stress and s_u/σ_v' is used to estimate the s_u of clays (Marchetti 1980). The force required to advance the dilatometer is measured and used to estimate the bearing capacity and then ϕ' of cohesionless soils (Schmertmann 1982).

5. SHEAR STRENGTH PROPERTIES OF SOME COMMON SOILS

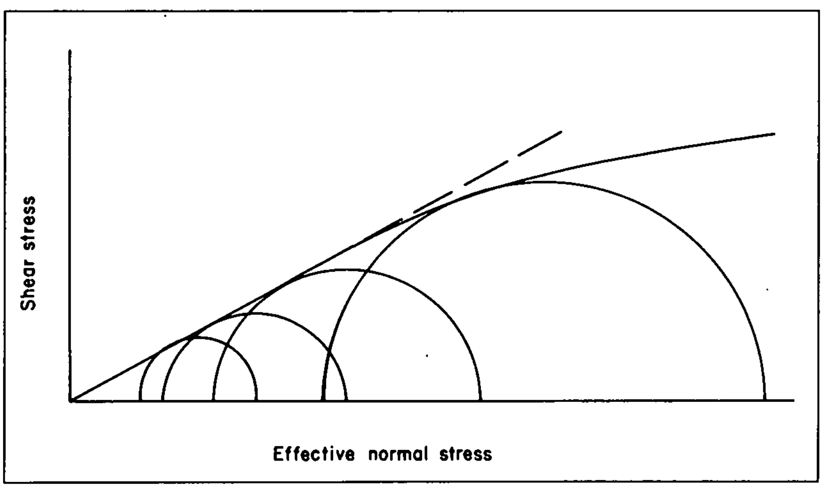
The strength characteristics of natural soils are strongly influenced by the geologic processes of soil formation. Hence, it is possible to identify strength characteristics that are common to soils within broadly defined groups. Groups with well-defined characteristics include saturated cohesionless soils, soft saturated cohesive soils, heavily overconsolidated clays, sensitive soils, and residual soils and colluvium. More detailed discussions of slopes involving specific soil conditions are given in Chapters 19 through 25.

5.1 Saturated Cohesionless Soils

Cohesionless soils, such as gravel, sand, and nonplastic silts, have effective stress failure envelopes that pass through the origin. This means $c' = 0$ in Equation 12.4. The value of ϕ' ranges from about 27 to 45 degrees or more and depends on several factors. For a given soil the value of ϕ' increases as the relative density increases. For different soils at the same relative density, the value of ϕ' is affected by particle-size distribution and particle shape. The value of ϕ' for a well-graded soil may be several degrees greater than that for a uniform soil of the same average particle size and shape. The same is true when a soil composed of angular grains is compared with one made up of rounded grains. The effect of moisture on ϕ' is small and amounts to no more than 1 or 2 degrees (Lambe and Whitman 1969, 147–149; Holtz and Kovacs 1981, 514–519).

The failure envelope, which is a straight line at low pressures, cannot be extended to high confining pressures. Tests with effective normal stresses above 700 kPa indicate that the failure envelope is curved, as shown in Figure 12-9 (Bishop 1966; Vesic and Clough 1968). The high normal stresses apparently cause crushing of grain contacts and result in a lower friction angle. This factor is particularly important in uncemented carbonate sands (Datta et al. 1982). Another important factor is the difference in the values of ϕ' as measured by different types of tests. The value of ϕ' measured in triaxial tests, which permit change in the radial strain, is 4 to 5 degrees smaller than that measured in plane-strain tests (Ladd et al. 1977).

FIGURE 12-9
Failure envelope for cohesionless soils.



In ordinary construction sandy and gravelly soils of high permeability can be considered to be loaded in the drained condition. Volume changes occur rapidly, and no excess pore pressures are developed. Without excess pore pressure, effective stresses can be estimated from the groundwater levels. Stability analyses can be performed by using effective stress strength parameters. For silty soils the permeability may be sufficiently low that excess pore pressure will develop during construction. When this is the case, the pore pressure must be measured or estimated if an effective stress analysis is to be performed.

The undrained response of fine sands and silts is important in some situations. Loose saturated sands and silts may develop large excess pore pressures after reaching peak strength. Because of the large pore pressure, the strength drops and the ultimate strength is well below the peak strength. Large displacements occur as a result of this loss of strength. This phenomenon was called *liquefaction* by Casagrande (1936). Large pore pressures may also develop in saturated sands of medium to high density under cyclic loading. In a triaxial test, if an axial stress is repeated for a number of cycles, the pore pressure may become equal to the confining pressure, thus reducing the effective minor principal stress (σ_3') to 0 (Seed and Lee 1967). Large strain increments will occur with each cycle of stress as the pore pressure approaches σ_3. If the soil dilates with strain, the pore pressure soon drops and the soil becomes stable. This phenomenon is called *cyclic mobility* (Castro 1975). The behavior of a sand under cyclic loading may range from liquefaction to cyclic mobility, depending on whether the sand is contractive or dilative.

Liquefaction and cyclic mobility are important considerations in the evaluation of slope stability under earthquake loadings. Lade (1992) and Sladen et al. (1985) formulated conditions for liquefaction in terms of the yield surface and the critical state, respectively. Methods for assessing the potential for liquefaction and cyclic mobility were reviewed by Casagrande (1976), Seed (1979), Poulos et al. (1985), and Seed et al. (1985).

5.2 Normally Consolidated and Lightly Overconsolidated Clays and Clayey Silts

Because of the low permeability of fine-grained soils, undrained or partially drained conditions are common in normal construction. The general characteristics of normally consolidated to lightly overconsolidated clays are described in this section. Very sensitive normally consolidated clays are described in Section 5.4.

A clay soil is considered to be normally consolidated if the consolidation pressure before shear is equal to or greater than the preconsolidation pressure (σ_p'). When a series of drained triaxial tests is conducted on a normally consolidated soil, the failure envelope is a line that passes through the origin [Figure 12-10(*a*)]; thus, $c' = 0$. If consolidated-undrained tests are performed on a normally consolidated soil, positive excess pore pressures develop. As a result, the undrained shear strength $s_u = 1/2(\sigma_1 - \sigma_3)$ will be less than the drained shear strength of a sample initially consolidated under the same stresses. The respective stress paths of the drained and consolidated-undrained tests are shown as *ca* and *cb* in Figure 12-10(*a*). A typical relation between strength and water content is shown in Figure 12-10(*b*). The results of consolidated-undrained tests can also be used to obtain the ratio s_u/σ_3', which is a constant for normally consolidated soils (Skempton 1948). For lightly overconsolidated soils, s_u/σ_3' is a function of the overconsolidation ratio σ_3'/σ_p' (Ladd and Foott 1974).

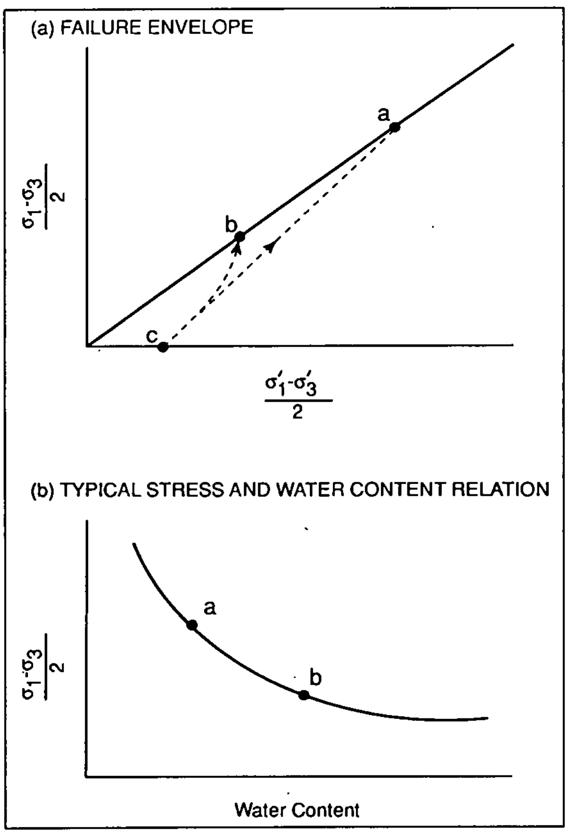


FIGURE 12-10 Strength properties of soft saturated clays.

If the load or stress change in the field induces positive excess pore pressures, as may be the case for a fill, the undrained strength will be lower than the drained strength. A slope based on an initially stable design can be expected to increase in stability with time as the excess pore pressure dissipates and the strength increases. The increase in strength with consolidation makes possible the construction in stages of embankments over soft soils. The strength gained under each stage of loading permits the addition of the next load increment. The ratio s_u/σ_3' is used to estimate the s_u under a given σ_3'. When a slope is made by excavation, there is a simultaneous increase in shear stress due to the slope and a decrease in mean normal stress due to the unloading of the excavation. In a saturated, normally consolidated soil, the increase in shear stress produces a positive excess pore pressure, and the decrease in normal stress produces a negative excess pore pressure. The net excess pore pressure in various parts of the slope depends on the relative values of these two effects. If the excess pore pressure is negative, the strength will decrease with time and drainage. In this case, the long-term or drained stability will be critical for a normally consolidated clay. Bishop and Bjerrum (1960) described several examples.

Failures of real slopes are good sources of information on the reliability of theoretical models. In a number of careful investigations, the factor of safety of the slope that failed was compared with the measured shear strength. If the theory and soil properties used are correct, the computed safety factor of a slope at failure should be unity. Results from many case histories show that for normally consolidated or lightly overconsolidated homogeneous clays of low sensitivity, total stress analysis with undrained shear strength gives a reasonably accurate estimate of immediate stability. For long-term stability under drained conditions, effective stress analysis with c' and ϕ' also gives satisfactory results. Comprehensive reviews and case histories were given by Bishop and Bjerrum (1960) and Bjerrum (1973), and more recent examples were

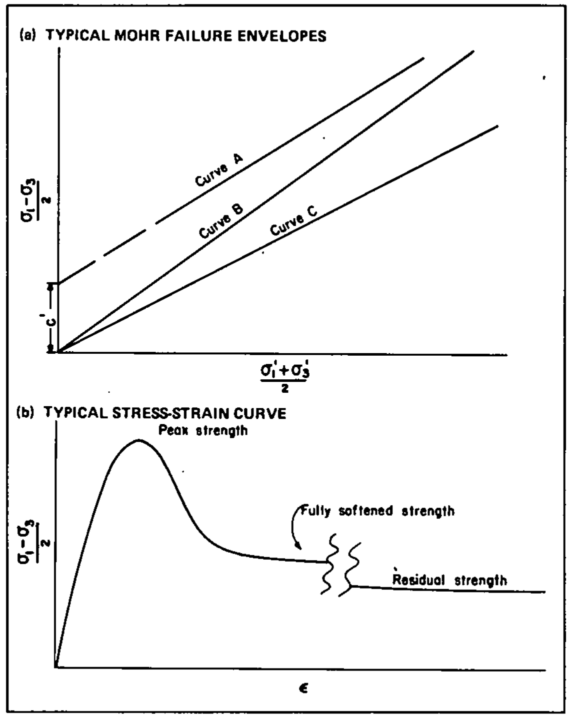


FIGURE 12-11 Strength properties of stiff overconsolidated clays.

presented by Ladd (1991), Pilot et al. (1982), and Tavenas and Leroueil (1980). For most of the reported failures in these case histories, the discrepancy between calculated and observed safety factors is less than 15 percent. Since most of the clays investigated are fairly uniform deposits, an accuracy of ±15 percent may be the best that can be achieved in practice.

It should be noted that earlier studies, such as that by Skempton and Golder (1948), used unconfined compressive strengths obtained from samples of small diameter. These conditions would tend to underestimate the in situ shear strength because of sample disturbance. By comparison, the study by Duncan and Buchignani (1973) used large-diameter samples and unconsolidated-undrained triaxial tests. The computed safety factor was 1.17 for the slope that failed. Various factors that could affect the strength were estimated, and the difference between observed strength and that measured in triaxial tests was attributed to the difference in strain rate. In cases reported by Ladd and Foott (1974), strength anisotropy was accounted for by performing triaxial compression and extension tests and direct simple shear tests. Hence, errors of different nature and magnitude are involved in the estimates of undrained shear strength in the different case histories.

Soft clays may also develop high pore pressures under cyclic loading (Thiers and Seed 1969). Sangrey et al. (1969) showed that a yield surface in effective stress space exists for cyclic stresses. When cyclic stresses reach the yield surface, large strains develop. Measurement and evaluation of strength losses due to complex states of static and cyclic stresses were described by Andersen et al. (1988).

5.3 Heavily Overconsolidated Clays

Most heavily overconsolidated clays show strain softening in their stress-strain curves (Figure 12-3, Curve A). The peak strengths of a series of heavily overconsolidated clay specimens give an effective stress failure envelope shown by Curve A in Figure 12-11(*a*). The failure envelope is approximately a straight line and if extrapolated to the axis at $\sigma' = 0$ gives a cohesion intercept (c'). Laboratory tests should be performed using normal stresses that are close to the normal stresses in the field because research has shown that the failure envelope for the peak strength of heavily overconsolidated clays is curved in the low-stress region and passes through the origin.

When loaded in the drained condition, a heavily overconsolidated clay will absorb water, and the absorbed water leads to a softening of the clay. The water content will increase significantly as the strength is reduced to the fully softened strength [Figure 12-11(*b*)]. The failure envelope for fully softened strength is shown as Curve *B* in Figure 12-11(*a*). The fully softened strength is close to the strength of the same soil in the normally consolidated condition. The strain necessary to develop fully softened strength in a heavily overconsolidated clay varies from one soil to another, but is at least 10 percent.

When much larger shear displacements take place within a narrow zone, the clay particles become oriented along the direction of shear, and a polished surface, or *slickenside*, forms. In natural slopes, slickensides may be developed along rupture surfaces of old landslides, bedding surfaces, or zones of deformation caused by tectonic forces. Along these surfaces the shear strength may approach the residual strength. The failure envelope, Curve C in Figure 12-11(*a*), is a line passing through the origin, and the residual angle of internal friction is ϕ_r'. Results of laboratory direct-shear tests indicate that ϕ_r' is dependent on soil mineralogy. As shown in Figure 12-12, "massive minerals," such as quartz, feldspar, and calcite, have high values of ϕ_r' that are close to the values of ϕ' for peak strength. However, clay minerals show significant differences between values of ϕ' and ϕ_r'. The largest difference was found in montmorillonitic clays, for which ϕ_r' was less than 10 degrees (Kenney 1967). The relationship between mineralogical composition and ϕ_r' makes it possible to correlate ϕ_r' with index properties such as plasticity index (Lupini et al. 1981) or liquid limit (Mesri and Cepeda-Diaz 1986). Because of the importance of strain-softening in heavily overconsolidated clays, choice of the strength for a stability analysis of a slope should consider progressive failure as described in Section 2.4. The design strength must be chosen to represent the average of the strengths along the entire rupture surface. For examples of analysis of progressive failure, see studies by Bjerrum (1967) and Duncan and Dunlop (1969).

For most slopes in heavily overconsolidated clays, the excess pore pressure immediately after construction is negative. Thus, the undrained strength will be greater than the drained strength. With drainage, the negative pore pressure dissipates and water is drawn into the sample. As water content increases, strength decreases. The long-term or drained conditions are critical, and there is no assurance that an initially stable slope will remain stable in the long term. Because of the low permeability, the time required to develop the fully drained condition in the field may be several years (Chandler 1984).

In heavily overconsolidated clays and soft shales, fissures and other discontinuities have an important influence on strength (Terzaghi 1936b). The characteristics of discontinuities in some stiff clays were described by Skempton and Petley (1967). The shear strength of laboratory specimens of such clays is strongly dependent on the number, shape, and inclination of discontinuities in the specimen. The presence of discontinuities is less likely in small specimens, which are often trimmed from intact soil between the discontinuities. Hence, the measured strength of laboratory samples tends to be higher than the in situ strength. When the clay contains discontinuities such as fissures and slickensides, the in situ strength will depend on the frequency and orientation of the discontinuities. Skempton and Petley (1967), Skempton and Hutchinson (1969), Chandler (1984), Wu et al. (1987b), and Burland (1990) provided examples of the in situ strength of clays with discontinuities.

5.4 Very Sensitive Soils

Sensitivity is defined as the ratio of the peak undrained shear strength to the undrained shear strength of the same soil after remolding at constant water content. Causes of clay sensitivity were reviewed by Mitchell and Houston (1969). The most dramatic landslides in sensitive soils occur in

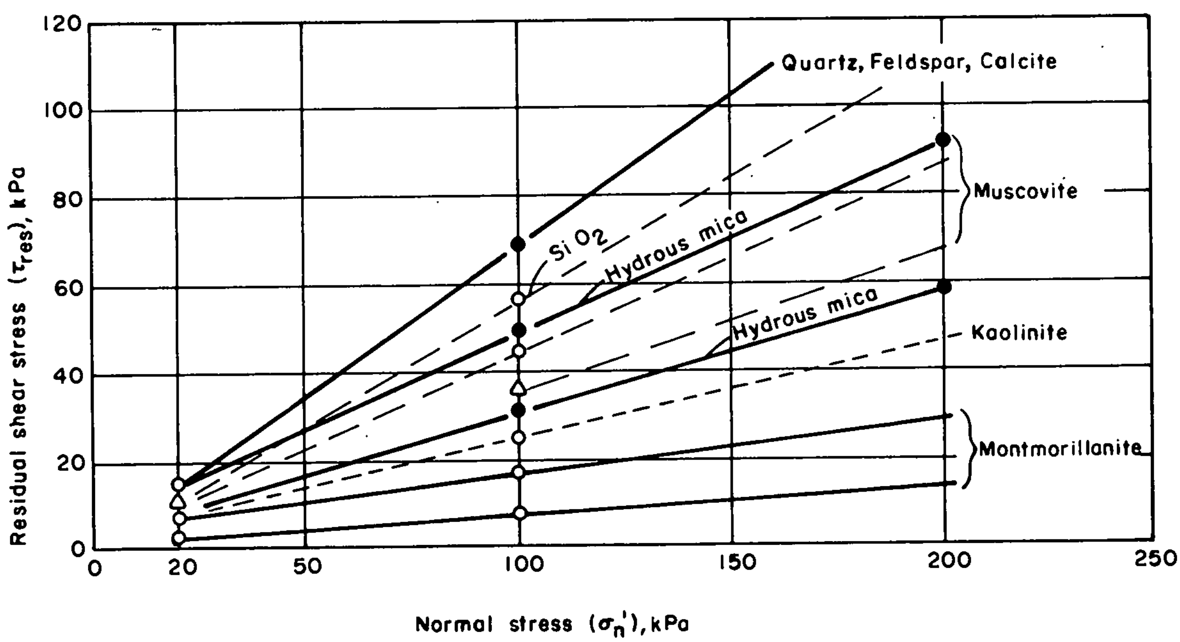


FIGURE 12-12 Residual shear strength of minerals.

Pleistocene marine clays in the Scandinavian countries and in the St. Lawrence River valley of eastern North America. High sensitivity is found in soils that have been subjected to leaching or natural cementation.

Effective stress failure envelopes for sensitive clays differ from those for soft saturated clays, described in Section 5.2. A sensitive clay, consolidated under the K_0 condition, has a yield surface Y_0 as shown in Figure 12-13. When the stress path for a point reaches the yield curve, such as at Point P, the soil structure is destroyed. Then high excess pore pressures are produced, and the shear strength decreases to Point C on the failure envelope for large strains (Tavenas and Leroueil 1977), which corresponds to the critical state (Schofield and Wroth 1968). In the design of slopes in these soils, it is necessary to keep the stresses within the yield curve. Evaluation of slope stability in very sensitive clays is described in Chapter 24.

5.5 Residual Soil and Colluvium

The weathering of rock produces residual soil. In the initial stages of weathering, coarse rock fragments are produced; the ultimate product of weathering is clay. Between these extremes is a soil composed of a mixture of grain sizes. On flat topography residual soil remains where formed. On slopes the weathered material, composed of soil and rock debris, may move down the face of the slope under the force of gravity. The deposit formed by this process is called *colluvium*. Residual soils have a wide range of properties depending on the parent material and the degree of weathering (Deere and Patton 1971; Mitchell and Sitar 1982).

Residual soils frequently exist in the partially saturated state. Some residual soils have high porosity and high permeability. Large changes in moisture content may occur with dry and wet seasons. The shear strength in terms of total stress has been found to be strongly influenced by the moisture content (Foss 1977; O'Rourke and Crespo 1988). Data on shear strength parameters c', ϕ', and ϕ'' are scarce. Tests by Ho and Fredlund (1982) showed that the component $c' + (u_w - u_a) \tan\phi''$ in Equation 12.5 decreases rapidly with decreasing suction, and c' is generally small at saturation. On the other hand, residual soils that are composed of a mineral skeleton held together by cementation may have an undrained shear strength that is unaffected by the water content. Such soils may show appreciable strength in spite of high water content and high porosity (Wallace 1973).

Colluvium presents some particular problems. The extremely heterogeneous nature of the material makes sampling and testing difficult. Results of tests on the finer matrix are likely to underestimate the shear strength.

Failure of slopes on residual soils and colluvium most frequently occurs during wet periods when there is an increase in moisture content and a decrease in suction. Stability of slopes may be determined by means of effective stress analysis and c', ϕ', and ϕ''. It is necessary to estimate the suction $u_w - u_a$ (Anderson and Pope 1984). In many cases in situ measurement of suction is necessary (Brand 1982; Krahn et al. 1989). If the soil is likely to be saturated, an effective stress analysis for the saturated condition provides a conservative estimate of the stability. If a total stress analysis is used, it is necessary to estimate the strength that corresponds to the in situ moisture content.

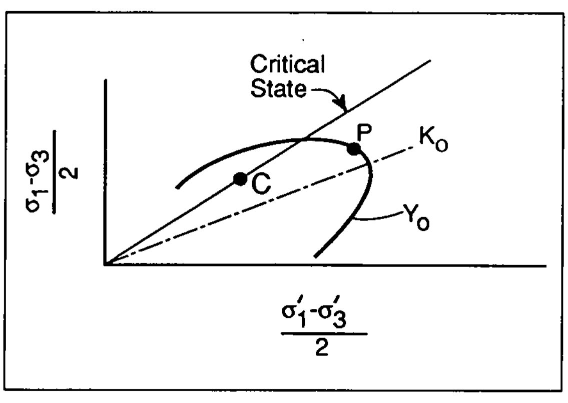


FIGURE 12-13
Strength envelopes of sensitive soils.

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