Chapter 6

Strength Properties and Their Measurement

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The methods of limiting equilibrium are frequently used to analyze the stability of a soil or a rock mass (Chapter 7). In such analyses, the shear strength of the material is assumed to be fully developed along the slip surface at failure. This chapter outlines the basic principles that govern the shear strength and the methods that may be used for its measurement.

GENERAL PRINCIPLES

Mohr-Coulomb Failure Criterion

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

$$s = c + \sigma \tan \phi \quad [6.1]$$

where

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

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

$$\sigma_1 = \sigma_3 \tan^2 [(\pi/4) + (\phi/2)] + 2c \tan [(\pi/4) + (\phi/2)] \quad [6.2]$$

where

σ_1 = major principal stress, and
σ_3 = minor principal stress.

Other failure criteria, particularly the modified Tresca and Von Mises, are sometimes used for soils, but their application to landslides has been limited (see later section on common states of stress and stress change).

Effective Stress Versus Total Stress Analysis

Since the shear strength of soils and rocks is strongly influenced by the drainage conditions during loading, those conditions must be properly accounted for in the use of shear strength. A fundamental principle in soil engineering is the use of effective stress (σ'), which is defined as

$$\sigma' = \sigma - u \quad [6.3]$$

where

σ = total stress, and
u = pore pressure.

The shear strength can be expressed consistently in terms of effective stress, or

$$s = c' + \sigma' \tan \phi' = c' + (\sigma - u) \tan \phi' \quad [6.4]$$

where c' and ϕ' are the strength parameters for effective stress. The use of the effective stress parameters requires that the pore pressure be known so that σ' may be evaluated.

In general, 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 pore-pressure change. The excess pore pressure may be either positive or negative, depending on the type of soil and the stresses involved. Under the fully drained, long-term condition, the excess pore pressure is zero, and pore pressure due to groundwater flow can usually be evaluated without serious difficulty. Hence, analysis with the effective stress description of shear strength (Equation 6.4) 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. In this case, the undrained shear strength (s_u) can be used, where $s = s_u$. This is the shear strength description in the common $\phi = 0$ method of analysis. The shear strength usually changes as drainage occurs. 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 undrained shear strength can be used only for short-term or temporary situations.

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. Many stability problems can be approximated by the plane-strain condition in which σ_2' lies near the midpoint between σ_3' and σ_1'. Experimental studies show that the relative value of σ_2' compared with σ_1' and σ_3' exerts some influence on the stress-strain characteristics and the shear strength.

Several common states of stress are shown in Figure 6.1. In the initial state, σ_z' is the effective overburden pressure, $\sigma_r' = K_0 \sigma_z'$ is the lateral pressure, and K_0 is the coefficient of earth pressure at rest. In the stress state beneath the center of a circular loaded area, the vertical stress ($\sigma_z' = \sigma_{z0}' + \Delta\sigma_z'$) is the major principal stress and the radial stress (σ_r') is the minor principal stress. In the stress state below the center of a circular excavation, the vertical stress is the minor principal stress and the radial stress is the major principal stress. For the circular load, the intermediate principal stress (σ_2') is equal to the minor principal stress (σ_3'); for the excavation, it 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' and, in Figure 6.1d, lies between σ_1' and σ_3'.

Another important feature in many stability problems is the rotation of the principal axes during loading or excavation. It has been reported that this reduces the shear strength of some soft clays (*6.50*). The rotation of principal axes is shown in Figure 6.2. Before the excavation of the cut, the state of stress is represented by that shown in Figure 6.1a. After excavation, the major principal stress is in the horizontal direction at the toe (point a). Thus, the principal axes are rotated through an angle of 90°; at point b, a rotation of approximately 45° occurs. At point c, the original principal stress directions remain the same although the values of the stresses change.

Stress-Strain Characteristics

Two stress-deformation curves are shown in Figure 6.3; stress-strain curves are of similar form. In common practice,

Figure 6.1. Common states of stress in soil.

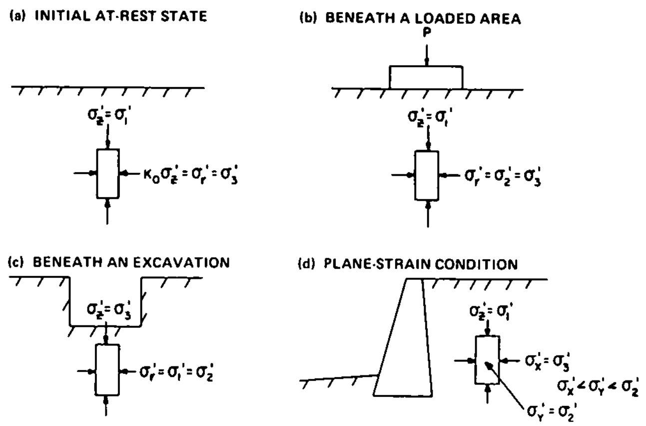
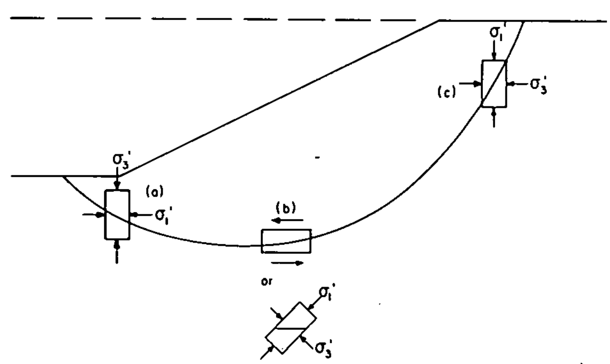


Figure 6.2. Rotation of principal stress axes in a slope.



the strength of the soil is defined as the peak strength (points a and b) 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 slip surface.

Many soils demonstrate strain-softening behavior, as illustrated by curve A. Any of several phenomena may explain the strength decrease, but it is important in design to account for this decrease. The lower limit to strength (point c) may be called the fully softened strength, remolded strength, or residual strength, depending on the type of soil involved (these terms are not synonymous). For such soils, it is unreasonable to assume that all soil along a failure surface reaches its peak strength simultaneously. In fact, the soil at some points will suffer displacements greater than Δ_a before the soil at other points reaches this deformation. In the limit of a large deformation, all strength at all points will be reduced to the strain-softening limit (point c).

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 real structures, the load remains permanently, although in some dynamic situations the peak load may be applied only for short durations. The effect of rate of loading on soil strength, excluding direct drainage effects, may be significant. In general, the undrained strength of soils

Figure 6.3. Typical stress-strain curves for soils.

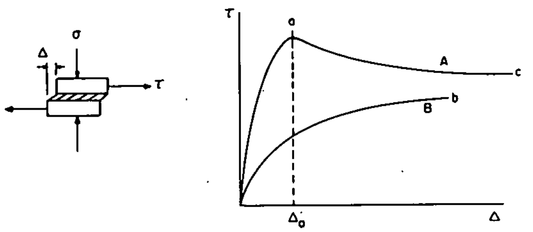
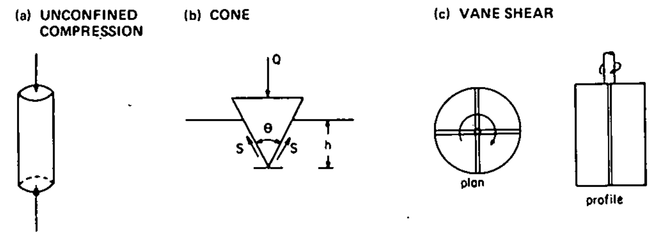


Figure 6.4. Simple test methods for determining soil strength.



increases as the rate of loading increases; however, this effect depends on the specific material and varies over a wide range (*6.22, 6.73*).

LABORATORY MEASUREMENT OF SHEAR STRENGTH

A variety of methods is available for laboratory measurement of shear strength. The simple methods are designed to determine the shear strength of a sample in a particular condition, such as the water content or void ratio 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 are able to establish the shear strength relation defined by Equation 6.4. These methods allow combinations of normal and shear stresses to be employed and pore pressures to be measured or controlled. The more elaborate tests allow more accurate simulation of the field stress or deformation conditions. For example, triaxial compression tests simulate Rankine's active state, and triaxial extension simulates Rankine's passive state. The plane-strain and simple shear tests may be used to provide a better simulation of the actual deformation conditions in a slope.

Simple Tests

Three types of simple tests are discussed below.

1. The unconfined compression test is usually performed on a cylindrical sample with a diameter-to-length ratio of 1:2 or slightly more. The sample is compressed axially (Figure 6.4a) until failure occurs; the shear strength is taken as one-half the compressive strength.

2. In the cone test, a cone with an angle of θ is forced into the soil (Figure 6.4b) under a force (Q), which may be its own weight. The shear strength is obtained from the relation

$$s_u = KQ/h^2 \qquad [6.5]$$

where

h = penetration, and
K = constant that depends on the angle θ and the weight Q.

Calibration curves for K have been published by Hansbo (*6.34*) and others.

3. In the vane test, 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 6.4c). 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 (*6.21*), the shear strength for vanes with a diameter-to-height ratio of 1:2 is

$$s = (6/7)\,(M/\pi D^3) \qquad [6.6]$$

where

M = torque, and
D = diameter of the vane.

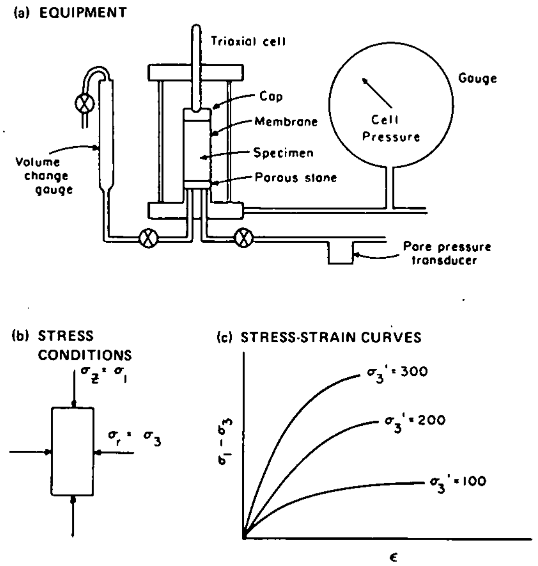
In the application of the results of these simple tests to the analysis of slopes, consideration should be given to the type of soil and loading conditions in situ. The application of these test results is commonly limited to saturated cohesive soils under undrained conditions. The results are all expressed 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 $s = s_u$. If the tests are run slowly or if the soil drains during shear, the results are generally not applicable.

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. Several studies (*6.15, 6.51*) show that even "good" samples may suffer strength losses as great as 50 percent. The effect of sample disturbance is most severe in soft sensitive soils and appears to become more significant as depth of the sample increases. Other factors to be considered include the state of stress and deformation. The directions of the principal stresses and the orientations of failure surfaces in each of these tests are not the same. They may also be quite different from the directions along the actual slip surface in a slope (*6.50*). Hence, caution should be exercised when the results of these simple strength tests are applied to slope stability problems (*6.15*).

Triaxial Test

The triaxial test is a highly versatile test, and a variety of stress and drainage conditions can be employed (*6.10*). The cylindrical soil specimen is enclosed within a thin rubber membrane and is placed inside a triaxial cell (Figure 6.5a). 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 compressive stress (σ_3). Drainage from the specimen is provided through the porous stone at the bottom, which is connected to a volume-change gauge. The volume-change gauge is often an enclosed burette so that back pressure can be applied. Volume compressibility can be determined by

Figure 6.5. Triaxial test.



the use of these measurements. Pore pressures in the sample are also measured by a device connected to the porous stone. Several devices may be used. Electric pressure transducers are becoming more common and are replacing the traditional null indicator and hydraulic cylinder system.

The axial stress (σ_z) may be increased by application of a load through the loading ram. From the known stresses at failure ($\sigma_1 = \sigma_z$ and $\sigma_3 = \sigma_2 = \sigma_r$), Mohr circles or other stress plots can be constructed. Several triaxial tests, each using a different value of cell pressure (σ_3), are usually performed on the same material for the definition of the failure envelope. The principal stresses at failure are then used to construct Mohr circles or other stress plots from which a failure envelope can be obtained. Typical plots of the principal stress difference versus axial strain are shown in Figure 6.5c. The stress-strain behavior is influenced by the confining pressure, the stress history, and other factors. Analytical representations of normalized stress-strain relations have been suggested for some soil types. The sample can also be loaded to failure in extension by increasing the radial stress while maintaining the axial stress constant; then, $\sigma_1 = \sigma_2 = \sigma_r$ and $\sigma_3 = \sigma_z$. These two methods of loading simulate the stress states shown in Figures 6.1b and c respectively.

The following tests are commonly performed with the triaxial apparatus.

1. In the consolidated-drained test (sometimes called a drained test or slow test), the soil is allowed to consolidate completely under an effective cell pressure (σ_3') so that at the end of consolidation the excess pore pressure in the soil is zero. The water content of the specimen after consolidation is w_c. In the triaxial compression test, the axial stress is increased at a slow rate, and drainage is permitted. The rate should be slow enough so that water can drain through the soil, and no excess pore pressure should be allowed to build up. A drained test of soils of low permeability often requires several days.

The volume change during shear in a drained test can result in either an increase or a decrease in the water content. This will depend on the type of soil and the level of stress involved. The water content at failure (w_f) will usually be different from w_c.

Since the excess pore pressure is zero in the drained test, effective stresses are known throughout the test and particularly at failure. In a compression test, the effective radial stress (σ_3') is equal to the cell pressure, and the measured load on the ram can be used to evaluate the effective axial stress (σ_1'). The results of a series of drained tests can be used to evaluate the effective stress strength parameters in Equation 6.4.

2. In the consolidated-undrained test with pore pressure measurement (sometimes called consolidated-quick test), the drainage valves are closed after the initial consolidation of the sample to w_c. Stress changes are applied through the ram, 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. Since the test is run in the undrained condition after consolidation, the water content throughout the test and at failure is w_c. Excess pore pressures developed during the test can be either positive or negative, depending on the type of soil and stress level.

Several equations have been proposed to describe the magnitude of the excess pore pressure developed as a result of stress changes in an undrained soil. For soils tested in the triaxial apparatus, or loaded so that $\Delta\sigma_2 = \Delta\sigma_3$, Skempton (*6.67*) proposed that the excess pore pressure is given by

$$\Delta u = B[\Delta\sigma_3 + A\Delta(\sigma_1 - \sigma_3)] \qquad [6.7]$$

where

B = empirical coefficient related to the soil's compressibility and degree of saturation, and
A = empirical coefficient related to the excess pore pressure developed because of shear of soil.

General relations between pore pressure and applied stresses have been suggested. For example, Henkel (*6.37*) proposed

$$\Delta u = B(\Delta\sigma_{oct} + \alpha\Delta\tau_{oct}) \qquad [6.8]$$

where

α = empirical coefficient similar to A;
τ_{oct} = octahedral shear stress, equal to $\frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$; and
σ_{oct} = octahedral normal stress, equal to $(\sigma_1 + \sigma_2 + \sigma_3)/3$.

The consolidated-undrained test is sometimes performed without pore-pressure measurement. Obviously, effective stresses are not known during this test or at failure. The application of shear strength measured in this test to any field problem involves assumptions of excess pore pressure that are of questionable validity in most cases; thus, this test is not recommended. To relate strength parameters obtained from consolidated undrained tests without pore-pressure measurement to field conditions is difficult.

3. In the unconsolidated-undrained test (sometimes

called undrained test or quick test), no drainage is allowed during any part of the test. When the cell pressure is applied, a pore-water-pressure change (Δu_c) occurs in the soil. When the axial stress is applied, additional pore-pressure changes (Δu_a) occur. These pore-pressure changes are not usually measured, so test results must be interpreted by the use of total stresses. 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, therefore, similar to the simple tests defined in the earlier section on simple tests.

All of the tests described above are usually begun by increasing the cell pressure to the desired stress level. This applies an isotropic or hydrostatic stress to the sample. This initial condition differs from the initial condition in situ (Figure 6.1a) if 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. This can be accomplished by consolidating the specimen under a cell pressure and an axial load. Experimental results with a wide variety of soils show that effective stress strength parameters determined from isotropically or anisotropically consolidated tests are essentially the same. The stress-strain curves, however, are significantly different. If the stress-strain relation must be determined, anisotropic consolidation should be used.

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 (*6.30, 6.39*). In plane-strain tests, the sample is consolidated anisotropically with zero lateral strain ($\epsilon_x = \epsilon_y = 0$). After this, the sample is loaded to failure by increasing either σ_z or σ_x and maintaining $\epsilon_y = 0$. The two methods of loading can be used to simulate the stress conditions at points c or a of Figure 6.2. Plane-strain tests can be conducted under undrained, consolidated-undrained, or drained conditions in manners similar to those described for triaxial tests.

Direct Shear Test

The direct shear test is shown in Figure 6.6. The soil specimen is enclosed in a box consisting of upper and lower halves; porous stones on top and bottom permit drainage of water from the specimen. The potential plane of failure is a-a. A normal stress (σ_z') is applied on plane a-a 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 vertical stress (σ_z') and the shear stress (τ_{xz}) at point b (Figure 6.6b) are known, but σ_x' is not. The directions of the principal stresses are approximately as shown in Figure 6.6c. Assuming that point a (Figure 6.6d) represents the conditions at failure, a Mohr circle can be constructed. The foregoing represents the common interpretation of the direct shear test. More elaborate analyses have been presented by Hill (*6.41*) and Morgenstern and Tchalenko (*6.57*).

Figure 6.6. Direct shear test.

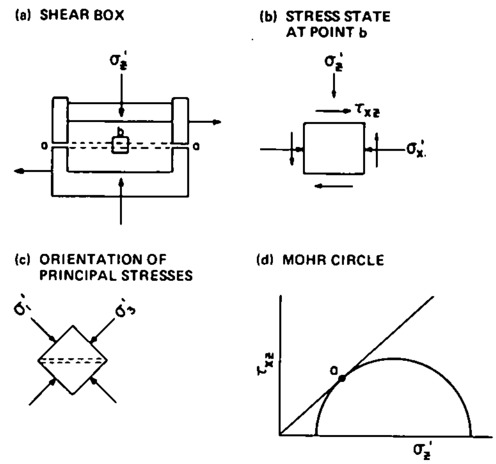
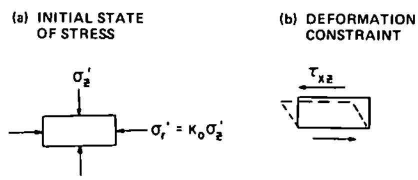


Figure 6.7. Simple shear test.



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 loading is carried out slowly, so that no excess pore pressure develops; hence, the drained condition is obtained.

In saturated clays, the direct shear test can be performed at a rapid rate so that the time duration is too short for any appreciable amount of water to flow into or out of the sample. This is an undrained condition, and excess pore pressures of unknown magnitude are usually developed in the soil. Consequently, this is essentially a simple test, and the shear stress at failure represents the undrained shear strength (s_u).

Simple Shear Test

Several simple shear tests have been developed, but the one described by Bjerrum and Landva (*6.17*) is most commonly used for testing undisturbed samples. The cylindrical specimen is enclosed in a rubber membrane reinforced by wire. This allows the shear deformation to be distributed fairly uniformly through the sample, as shown in Figure 6.7b. In the test, the sample is consolidated anisotropically under a vertical stress (Figure 6.7a) and sheared by application of stress τ_{xz} (Figure 6.7b). 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 adjusting the vertical stress (σ_z) continuously during the test. In the simple shear test, the principal axes are in the vertical and horizontal directions initially. At failure, the horizontal plane becomes the plane of maximum shear strain. This condition approximates that at point b of Figure 6.2.

SHEAR STRENGTH PROPERTIES OF SOME COMMON SOILS

Unlike steel or concrete, whose material properties are known or closely controlled, soil has material properties that are unique at every site. For that reason, the strength properties of the soil should be investigated at every site. Within broad groupings, however, the strength characteristics of many soils are similar. Appreciation of these characteristics can be helpful in planning detailed investigations.

Figure 6.8. Typical failure envelope for cohesionless soils.

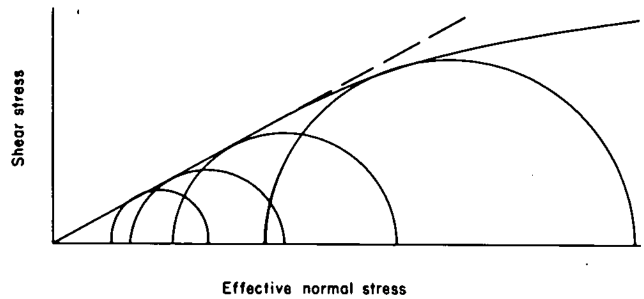
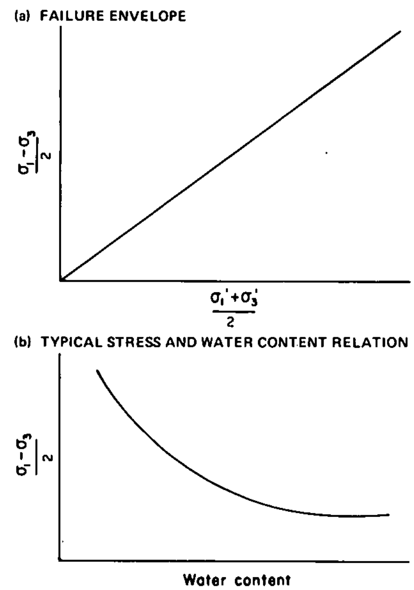


Figure 6.9. Strength behavior of soft saturated clay soils.



Cohesionless Soils

Granular soils, such as gravel, sand, and nonplastic silts, are called cohesionless soils. The effective stress failure envelope of a cohesionless soil is approximately a straight line passing through the origin. This means that, for those soils, $c' = 0$ in Equation 6.4. The value of ϕ' ranges normally from about 27° to 42° or more and depends on several factors. For a given soil, the value of ϕ' increases as relative density increases. If one considers several 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. 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° (*6.42*).

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 (1460 lbf/in^2) indicate that the failure envelope is curved, as shown in Figure 6.8 (*6.7, 6.77*). The high normal stresses apparently cause crushing of grain contacts and result in a lower friction angle. Another important factor is the difference in the values of ϕ' as measured by different types of tests. The ϕ' measured in triaxial tests, which permit change in the radial strain, is as much as 4° to 5° smaller than the ϕ' measured in plane-strain tests (*6.33*). This difference has also been observed in field problems.

In ordinary construction situations, 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 sustained. Without excess pore pressures, effective stresses can be estimated from the groundwater levels. Stability analyses can be performed by using the effective stress strength parameters.

For silty soils, the permeability may be sufficiently low that excess pore pressures will develop during construction. When this is the case, the pore pressures must be measured or estimated if an effective stress analysis is to be performed.

The undrained response of sands and gravels is required for only a few situations. Saturated loose sand may fail so rapidly that excess pore pressures are sustained. Similarly, under rapid loading the undrained shear strength may be applicable (see the later section on soil behavior under repeated loads).

Soft Saturated Clays and Clayey Silts

Soils containing significant amounts of clay and silt are called cohesive soils. Because of the low permeability of fine-grained soils, undrained or partially drained situations are common. This is a most important difference between cohesionless soils and cohesive soils. Another important distinction between normally consolidated or lightly overconsolidated clays and heavily overconsolidated clays is based on the kind of excess pore pressures developed in these soils during shear. The general characteristics of normally consolidated clays will be discussed first. Extremely sensitive normally consolidated clays are not discussed here but in the later section on sensitive soils.

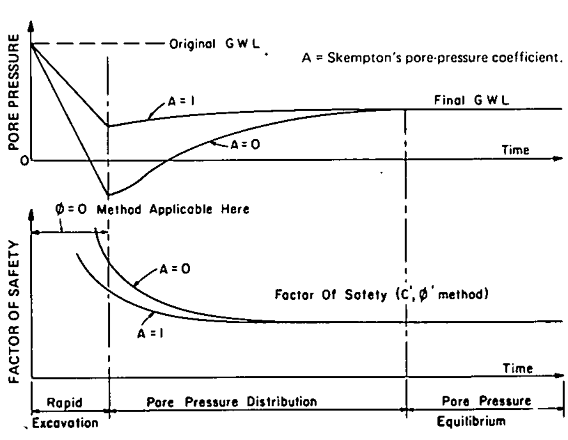
A clay soil is considered to be normally consolidated if the consolidation pressures before shear are equal to or greater than the preconsolidation stress (p_c'). When a series of drained triaxial tests is conducted on a normally consolidated soil, the failure envelope is a straight line that passes through the origin (Figure 6.9a); thus, $c' = 0$. A relation between strength and water content is shown in Figure 6.9b. If consolidated-undrained tests are performed on a normally consolidated soil, positive excess pore pressures develop. The effective stresses at failure will define the same failure envelope in consolidated-undrained tests as in drained tests ($c' = 0$ and $\phi' = \phi_d'$). As a result of the positive excess pore pressures, however, the undrained strength ($\sigma_1 - \sigma_3$) will be less than the drained strength of a sample initially consolidated under the same stresses. This characteristic can be applied as a design principle.

If the load or stress change in the field induces positive excess pore pressures (as in the case of a fill), the undrained strength will be lower than the drained strength. An initially stable design can usually be expected to increase in stability with time as the excess pore pressure dissipates, the water content decreases, and the strength increases. On the other hand, if negative excess pore pressures are induced (as in the case of an excavation), the undrained strength will be larger than the drained strength. Failure may occur sometime after construction, even though the slope is stable in the undrained state. Bishop and Bjerrum (*6.9*) have described several examples.

A good source of information on the reliability of theoretical models is the failure of real slopes. In a number of careful investigations, the factor of safety of the slope that failed was compared with the measured shear strengths. If the theory and soil properties used are correct, the safety factor of a slope at failure should be unity. The results of these studies show that, for normally consolidated or lightly overconsolidated homogeneous clays of low sensitivity, analysis using the undrained shear strength is reasonably accurate for immediate stability. For long-term stability, the effective stress analysis is also consistently accurate. Several studies on bearing capacity failure likewise show reasonable agreement. Summarized results of several case studies are given in Table 6.1. Eight cases are given of short-term failure immediately after or during construction. These are undrained conditions, and the analyses were made by using the undrained shear strength (s_u). The computed factors of safety are all close to unity and thus show that failure should have occurred according to theoretical predictions.

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 general unloading of the excavation (*6.9*). 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 soil will decrease in strength with time and drainage. In this case, the long-term or drained stability will be critical for a normally consolidated clay. An example of this situation is shown in Figure 6.10. Table 6.1 gives four cases of long-term failures in soft clay soil and the calculated safety factors.

Figure 6.10. Changes in pore pressure and factor of safety during and after excavation of a cut in clay (*6.9*).



The examples of field investigations given in Table 6.1 for short-term conditions of soft clay soils all involve undrained behavior of normally consolidated or lightly overconsolidated clays. To date, more than 50 slope failures and foundation failures in such soils have been investigated and reported. For more than 90 percent of those, the discrepancy between calculated and observed safety factors is less than 15 percent. Since most of the clays investigated are fairly uniform deposits, the accuracy would be less for nonuniform clay deposits. Many factors contribute to this uncertainty, but strength anisotropy and rate of loading are probably two of the most important. Bjerrum (*6.14, 6.15*) reviewed notable cases in which large discrepancies between prediction and performance were observed in highly plastic and organic clays. For those cases, the use of undrained shear strength, as measured by the unconfined compression or vane shear tests, tends to overestimate the safety factor under undrained conditions. Available case studies of drained failure in normally consolidated clays are too few to support a statement about the accuracy of predictions. However, the reliability of these predictions appears to be about the same as that for undrained failure of normally consolidated clays.

Heavily Overconsolidated Clays

Geological and stress histories are important considerations in the behavior of heavily overconsolidated clays. The presence of fissures, which may be due to passive failure under high values of K_0 (*6.11, 6.20, 6.68*) or other causes such as shrinkage, has an important influence on the strength of soils. The characteristics of some fissures and the strengths along the fissures are described by Skempton and Petley (*6.75*). The shear strength of laboratory specimens of fissured clays is strongly dependent on the number, shape, and inclination of fissures in the specimen (*6.54*). The presence of fissures is less likely in small specimens, which are often trimmed from intact soil between the fissures. Hence, the measured strength tends to increase as the size of test specimen decreases (*6.52, 6.78*). Thus, to extrapolate from the laboratory shear strength to the in situ shear strength is often difficult, and frequently in situ load tests must be conducted.

Most heavily overconsolidated clays show stress-strain relations that suggest general strain softening (Figure 6.3, curve A). Several concepts may be used to explain this strain-softening behavior. Consider the test results from a series of heavily overconsolidated clay specimens. If the peak strength is used to describe failure, an effective stress failure envelope as shown by curve A in Figure 6.11a is obtained. The failure envelope is approximately a straight line and, if extrapolated to the axis of $\sigma' = 0$, there is a cohesion intercept (c'). If the effective stresses at failure are used, results from both drained and undrained tests describe the same envelope. Laboratory tests using normal stresses that are close to the normal stresses in the field should be performed because research (*6.11, 6.66*) 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.

As time and drainage increase and the negative pore pressure dissipates, a heavily overconsolidated clay will absorb

Table 6.1. Examples of stability prediction in soft and overconsolidated clays.

Clay	Condition	Site	LL (%)	PL (%)	w_c (%)	Shear Strength Parameters s_u (kPa)	c' (kPa)	ϕ' (deg)	ϕ'_r (deg)	Method of Analysis	Factor of Safety	Remarks	Reference
Soft	Short term	Congress Street, Chicago	32	18	25	40 to 70				$\phi = 0$	1.11 to 0.9	Circular failure	Ireland (***6.43***)
		Welland, Ontario											
		Test cut	53	27	35	60				$\phi = 0$	1.01	Analysis by Bjerrum (***6.15***)	Kwan (***6.48***)
		Channel	40	20	30	29 to 43				Janbu	0.69 to 0.94	Analysis by Bjerrum (***6.15***)	Conlon, Tanner, and Coldwell (***6.26***)
		Portsmouth, New Hampshire, test fill	38	22	50	10 to 18				Simplified Bishop	0.86 to 0.92	Test fill	Ladd (***6.49***)
		Rangsit, Thailand	77	37	65	20 to 30				–	1.08 to 1.26	Loading test	Brand, Muktabhant, and Taechathummarak (***6.19***)
		England											
		Chingford Reservoir	145	36	90	14				$\phi = 0$	1.05	Fill	Skempton and Golder (***6.72***)
		Newport	60	26	50	18				$\phi = 0$	1.08	Fill	Skempton and Golder (***6.72***)
		Huntspill	75	28	56	15				$\phi = 0$	0.90	Cut slope	Skempton and Golder (***6.72***)
	Long term	Norway											
		Drammen	35	18	35	–	11	32.5		Simplified Bishop	1.01		Kjaernsli and Simons (***6.47***)
		Lodalen	36	18	31	40 to 60	10	27		Bishop	1.00 to 1.07		Sevaldson (***6.64***)
		Kimola, Finland											
		Upper canal	54	23	44	50	12	27.6		Simplified Bishop	1.16		Kankare (***6.45***)
		Great slide	53	26	53	30	5	27.7		Simplified Bishop	0.97		Kankare (***6.45***)
Overconsolidated	Short term	England											
		Selset	26	13	12		9	30		Bishop	1.05	Peak strength	Skempton and Brown (***6.71***)
		Bradwell 1	95	30	33	77				$\phi = 0$	~1.0	Peak strength corrected for fissures and time to failure	Skempton and LaRochelle (***6.74***)
		Bradwell 2	95	30	33	72				$\phi = 0$	~1.0	Peak strength corrected for fissures and time to failure	Skempton and LaRochelle (***6.74***)
		Valdarno, Italy	75	45	48		100 to 200	27 to 30		Block analysis	~1.0	Failure controlled by orientation of discontinuities	Esu (***6.31***)
	Long term	Caneleira, Brazil	45	30	–		30	40		–	1.0	Peak strength	Vargas and Pichler (***6.76***)
		England											
		Brown London clay (12 cases)	90[a]	30[a]	–		0	20[a]		–	0.8 to 1.0	Fully softened	Skempton (***6.70***)
		Lias clay (12 cases)	60[a]	28[a]	–		0	23 (±1½)		Morgenstern and Price	0.75 to 1.0	Fully softened	Chandler (***6.24***)
		Sudbury Hill	82	28	31		0	20		Morgenstern and Price	1.05	Fully softened	Skempton and Hutchinson (***6.73***)

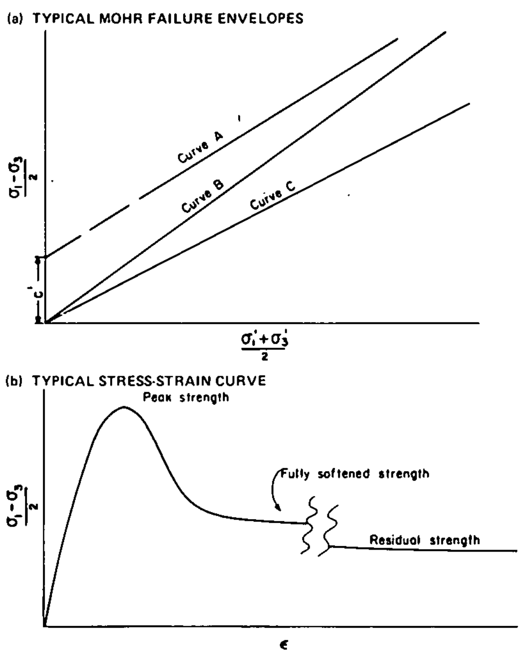
Table 6.1. Continued.

Clay	Condition	Site	LL (%)	PL (%)	w_c (%)	Shear Strength Parameters: s_u (kPa)	c' (kPa)	ϕ' (deg)	ϕ'_r (deg)	Method of Analysis	Factor of Safety	Remarks	Reference
		Northolt	79	28	30		15[b] 0[c]	20[b]	 13[c]	Morgen-stern and Price	1.0	R = 0.60	Skempton and Hutchinson *(6.73)*
		Jackfield	44	22	21		0		<19	Morgen-stern and Price	1.1	Residual	Henkel and Skempton *(6.38)*
		Sevenoaks	65	26	25		0		16	Fellenius	1.0	Residual	Skempton and Petley *(6.75)*
		River Beas Valley, India	41	25	—		0		15 to 20	Block analysis	1.0	Residual	Henkel and Yudhbir *(6.40)*
		Saskatchewan, Canada	115	23	32				6	Block analysis	0.9	Residual	Bjerrum *(6.12)*
		Balgheim, Germany	61	25	37				17	Block analysis	1.0	Residual	Bjerrum *(6.12)*
		Sadnes, Norway	60	30	36				12 to 18	—	1.0	Residual	Bjerrum *(6.12)*

Note: LL = liquid limit; PL = plastic limit; w_c = water content; s_u = undrained shear strength; c' = cohesion intercept in terms of effective stress; ϕ' = angle of internal friction in terms of effective stress; ϕ'_r = residual angle of internal friction; R = residual factor = (peak strength - average strength at failure)/(peak strength - residual strength); and 1 kPa = 0.145 lbf/in^2.

[a]Average. [b]Peak. [c]Residual.

Figure 6.11. Shear strength levels developed by heavily overconsolidated clays.

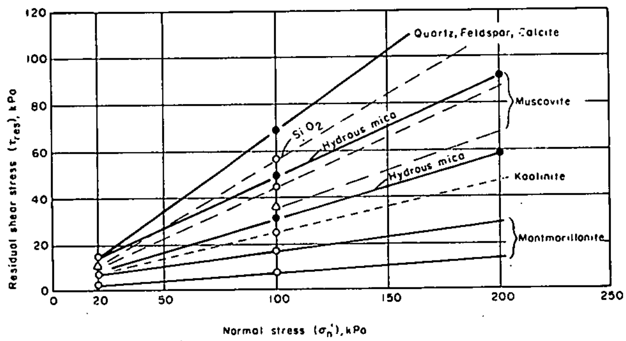


water. In landslides, drainage is facilitated by opening of fissures after stress release; this leads to a softening of the clay (*6.13*). The water content, at least in the failure zone, will increase significantly as the strength is reduced to the fully softened strength (Figure 6.11b). For example, several studies (e.g., *6.36, 6.75*) included data on the differences between soil properties along discontinuities and the same properties for the adjacent "intact" clay. In one case, the water content was 44 percent along the failure surface of London clay and 30 percent in the adjacent intact clay. The failure envelope of a fully softened strength can be defined by curve B in Figure 6.11a. This fully softened strength is typically the same as 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 on the order of 10 to 100 percent. The significance of using the fully softened strength in the long-term design of slopes has been discussed by several authors (e.g., *6.24, 6.70*).

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 is formed (*6.46, 6.57*). In natural slopes, slickensides may be developed along surfaces of old landslides, bedding planes, or zones of deformation caused by tectonic forces. Along these surfaces, the shear strength may approach the residual strength (*6.69*); this concept has been the subject of extensive study in the field and in the laboratory. The failure envelope, curve C in Figure 6.11a, is typical for the residual strength. The straight line passing through the origin defines the residual angle of internal friction (ϕ'_r). Kenney (*6.46*), as a result of an extensive series of laboratory direct shear tests, concluded that ϕ'_r is dependent on soil mineralogy. As shown in Figure 6.12, massive minerals, like quartz, feldspar, and calcite, have high ϕ'_r values, which are little different from the values of the peak strength parameter (ϕ') for these soils. On the other hand, the various clay mineral groups all show significant differences between ϕ' and ϕ'_r. The largest difference was found in montmorillonitic clays, which have ϕ'_r below 10°.

Both long-term and short-term stability must be considered in the design of slopes in heavily overconsolidated clays. Because of the low permeability, the time required to develop the fully drained condition may be many years (*6.12*). For most slopes in heavily overconsolidated clays, the excess pore pressures immediately after construction are negative. Thus, the undrained strength will be greater than the drained strength. As time and drainage increase, the negative pore pressures dissipate and water is drawn into the

Figure 6.12. Residual shear strengths of soil minerals (*6.46*).



Note: 1 kPa = 0.145 lbf/in²

sample. As water content increases, strength decreases. Under field conditions, 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.

The behavior of heavily overconsolidated clays is obviously much more complicated than that of normally consolidated soils. In the application of shear strength concepts to a slope stability analysis, the most difficult problem is to select the appropriate operating strength. A further complication is that the strength changes are strain-softening (Figures 6.3 and 6.11). As noted in the earlier section on stress-strain characteristics, it is unreasonable to expect all of the soil along a failure surface to reach peak strength at the same time. If the soil at some parts of a failure surface has not yet reached the peak or has passed the peak, the operating strength must be chosen to represent the average of the strengths along the entire failure surface (see Chapter 7). The use of the fully softened strength may be too conservative (*6.25*).

Another factor that may affect the stability of slopes in heavily overconsolidated clays is the effect of residual stress. Residual stresses released by excavation may be temporarily resisted by bonding within the soil. If these bonds deteriorate with time and weathering, the stress release will occur over a period of time and progressive failure may occur (*6.12, 6.25, 6.73*).

The various operating strengths of heavily overconsolidated clays are also illustrated by the case histories given in Table 6.1. The undrained failures are all short-term failures of cut slopes. Since slopes in heavily overconsolidated clays will become less stable with time, any design using the undrained strength must be considered temporary and the undrained strength should only be used if the clay is intact. Fissured clays soften and drain so rapidly that undrained conditions should not be assumed in design. The long-term examples are grouped more or less according to the type of strength at the time of failure. The number of examples given is not indicative of the actual distribution of failures in natural slopes.

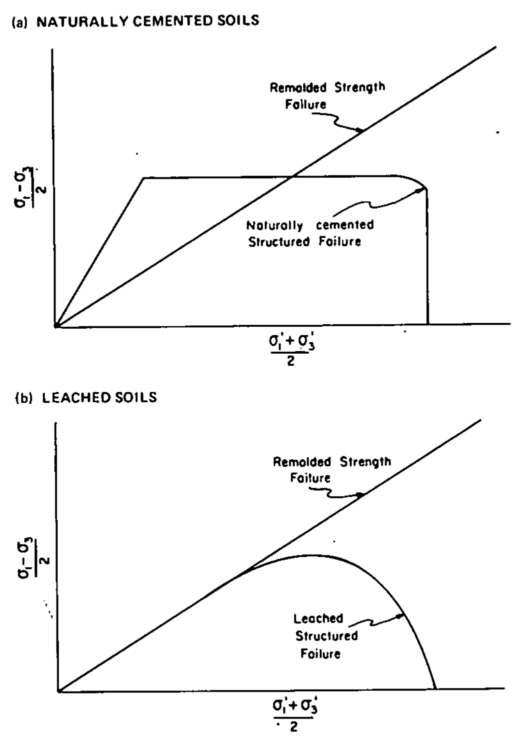
Sensitive Soils

Sensitivity is defined as the ratio of the peak undrained shear strength to the undrained shear strength of a sample remolded at a constant water content. Causes of clay sensitivity are discussed by Mitchell and Houston (*6.55*). The most dramatic landslides in sensitive soils are the flow slides occurring in Pleistocene marine clays in the Scandinavian countries of northern Europe and in the St. Lawrence River valley of eastern North America. The sensitivity of these soils is due to either leaching or natural cementation.

The effective stress failure envelope for both leached and naturally cemented sensitive clays is distinctly different from that of other soft clays, and the strengths of both types of sensitive soil are dominated by structure. Special testing techniques must be used to define these failure envelopes. As shown in Figure 6.13a, the strength envelope for a naturally cemented sensitive clay is a unique curve in stress space (*6.59*). The failure envelope encloses a low stress region within which the cemented structure remains intact. When stress changes in a slope exceed the limits of the failure curve, the structure is destroyed. Then high positive excess pore pressures are produced, and the soil behaves as a remolded soil. Thus, the failure envelope for the remolded soil (Figure 6.13a) governs the strength only after rupture of the cemented structure and has no influence on initial failure.

The cemented structures of these sensitive soils can be broken down by consolidation, particularly during isotropic consolidation of a triaxial test specimen. The natural state of stress for a sensitive cemented soil lies within its domain of natural cementation. In the design of slopes in these soils, it is usually desirable to keep the stresses within these same limits. Therefore, stresses used in laboratory tests should also lie within these limits. A common mistake in testing these materials is to use a high cell pressure, which destroys the structure during consolidation (*6.15*). In this case the failure envelope for remolded soil is obtained, and it may not be applicable to the actual stress conditions in the field.

Figure 6.13. Strength envelopes for sensitive soils.



For a leached sensitive soil (Figure 6.13b), the strength envelope falls entirely below the failure envelope for the remolded soil. For both the leached and naturally cemented sensitive soils, the failure envelope is a unique characteristic for each soil. A separate set of tests must be used to determine the strength of each soil. Laboratory studies also show that the strength of these soils is highly anisotropic and that compression, extension, and simple shear tests commonly give different strength envelopes (*6.16, 6.56, 6.59*).

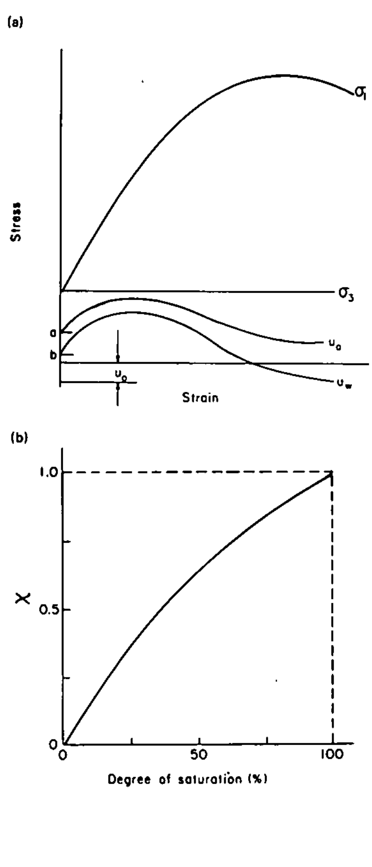
The difficulties in sampling and testing sensitive soils have resulted in a great deal of emphasis on field measurement of their strengths. The vane test is used extensively; corrections to account for anisotropy and strain rate have been suggested by Bjerrum (*6.15*).

Partially Saturated Soils

For partially saturated soils, effective stress analysis usually is not possible because excess pore pressures are not known. In this case, the pore pressure consists of two components: pore-air pressure and pore-water pressure. To perform an effective stress analysis of a slope requires that both components and their interaction be known. Coarse granular soils, even when partially saturated, pose little problem because excess pore pressures dissipate rapidly through drainage. On the other hand, partially saturated clays, particularly compacted clays, are a common problem. The process of compaction usually utilizes high pressures, so that the compacted clay resembles in some respects a heavily overconsolidated clay. The strength characteristics of compacted clays are therefore similar to those represented by curve A in Figure 6.11.

Figure 6.14 shows the behavior of air and water pressures during an undrained triaxial test. In an unsaturated soil, menisci exist at the air-water interfaces and the water is under capillary tension. Hence, the initial pore-water pressures have a negative value equal to u_0. After the all-around pressure (σ_3) is applied, the compression of these air spaces brings about a volume change and a rise in the air and water pressures; these are shown as points a and b in Figure 6.14a. The difference between the pore-air pressure (u_a) and the pore-water pressure (u_w) is equal to the capillary tension. As σ_1 is increased, both u_a and u_w change with strain.

Figure 6.14. Strength of partially saturated soils (*6.8*).



In a system with both air and water under pressure, the effective stress may be written as (*6.8*)

$$\sigma' = \sigma - u_a + \chi(u_a - u_w) \quad [6.9]$$

in which χ is a coefficient that depends on the degree of saturation. This relation is evaluated experimentally. To determine the quantity χ for an unsaturated soil requires that both the air and the water pressures be known as well as c′ and ϕ', the shear strength parameters of the saturated soil. For an unsaturated soil with given air and water pressures, the value of χ must be such that it satisfies

$$s = c' + [\sigma - u_a + \chi(u_a - u_w)] \tan \phi' \quad [6.10]$$

Equation 6.10 may be rewritten as

$$\chi = [s - c' - (\sigma - u_a) \tan \phi'] / [(u_a - u_w) \tan \phi'] \quad [6.11]$$

In Equation 6.11, c′ and ϕ' are known for the soil from tests on saturated specimens, and s, σ, u_w, and u_a are measured for the particular specimen in a triaxial test. The appropriate value of χ can then be calculated directly. The objective is to establish the relation between χ and the degree of saturation (S_r). Therefore, at different degrees of saturation, a curve like the one shown in Figure 6.14b can be obtained. This relation between χ and S_r is a soil property and must be known if the values of c′ and ϕ' are to be applied to the unsaturated soil.

To evaluate the shear strength by using Equation 6.10 requires also that the air pressure (u_a) be known. If the soil is compacted at a moisture content on the dry side of optimum, most of the air voids are continuous, and the approximation $u_a = 0$ is commonly used. When this condition does not hold, the air pressure must be estimated by the same method used to evaluate the water pressure. Alternate methods using the total stress to analyze the immediate stability are described by Blight (*6.18*) and others (*6.2*).

In dry climates, some partially saturated soils have cemented bonds at the contacts. If these soils are saturated by flooding, the bonds may break leading to collapse of the natural structure. Such soils often are called collapsible soils; when their strengths are evaluated, consideration should be given to the water content under operating conditions (*6.1, 6.4*).

Residual Soil and Colluvium

The weathering of rock produces residual soil. On flat topography, residual soil remains where formed and is called eluvium. It may develop to great depths, and natural or human-made slopes may be cut into the eluvium. Natural rock slopes also weather to produce residual soil, which will be acted on by forces of gravity. The soil and rock debris that moves down the face of a slope is called colluvium.

Residual soils have a wide range of properties, depending on the parent material and the degree of weathering (*6.29, 6.53*). In the initial stages of weathering, coarse rock fragments are produced; the ultimate product of weathering is clay. Between these extremes is usually a mixture of grain sizes, and the behavior of residual soils is similar to that described in previous sections for the soils with similar particle size distributions. Some residual soils are also cemented. More detailed information about residual soils and their properties is given by Deere and Patton (*6.29*).

Colluvium presents some particular slope stability problems (*6.27, 6.53*). The extremely heterogeneous nature of the material makes sampling and testing difficult. In some cases, loose structure and large grain size result in high values of permeability. This characteristic is often accentuated by the presence of less permeable layers immediately below the colluvium. Under this condition, infiltrating surface water flows through the colluvium, generally parallel to the slope, and the consequence is a decrease in stability. Where large landslides have occurred in the past, slickensides may be found along old slip surfaces. The shear strength along those surfaces may be close to the residual shear strength.

Rocks

The shear strength of rock masses is almost always determined by the configuration of the joints and the nature of the joint surfaces (*6.28*), as explained in Chapter 9. Numerous theoretical and laboratory studies (*6.5, 6.6, 6.32, 6.44*) have been conducted to evaluate the apparent strength of rock masses with closed block joints (Figure 6.15a). These studies show that failure of jointed rock depends on several geometrical parameters, including orientation and spacing of joints, joint characteristics, and stress state, and on the strengths of the intact rock and the joint filling (see Chapter 9). In many field problems, the spacing of joints is small relative to the height of slope and the size of the failure surface.

Under these conditions, failure of a jointed rock mass involves the interaction of sliding along the joints, dilatancy, separation and rotation of the blocks, and possible fracture of the intact rock. Most experimental studies indicate that for this type of failure the failure envelope for jointed rocks is nonlinear (Figure 6.15b). Each situation is unique, so such curves must be determined experimentally for the particular situation. With certain joint orientations, failure will occur along a single joint or joint set. This commonly happens in the field when the orientation of joints or other discontinuities is close to that of the slope so that block or wedge sliding becomes possible. To analyze this condition requires that the strength of the rock along the joints be known.

The shear strength of a joint or other discontinuity depends on the characteristics of the joint. The strength of a freshly fractured joint surface will be different from that of a joint that is highly weathered and full of debris. Because fresh rock surfaces may weather rapidly, it is important to recognize the potential for change in the strength of a joint or joint system during the lifetime of an engineering project. Experimental measurements of joint strengths can be made by the use of laboratory or field direct shear tests.

A stress-displacement curve of a rough joint is similar to curve A in Figure 6.3. The peak strength (point a) is obtained when the large projections along the joint are sheared off. Beyond this point, the strength decreases and approaches the residual strength. The residual strength represents the strength of the joint after the projections have been sheared off.

Typical results of drained tests in which effective stresses could be measured are given in Table 6.2. The strength is almost always frictional in character, even when large amounts of debris are present in the joint.

Figure 6.15. Strength of rock masses with closed block joints (*6.32*).

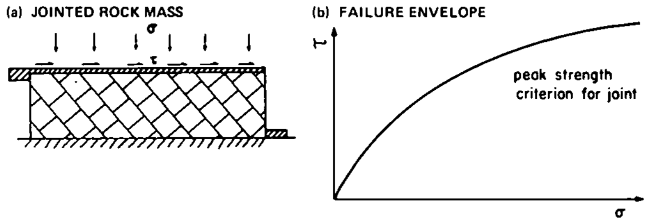


Table 6.2. Residual angle of friction obtained from sandblasted, rough-sawed, and residual surfaces of various rocks (*6.5*).

Rock	Moisture	σ' (kPa)	ϕ'_r (deg)
Amphibolite	Dry	98 to 4 100	32
Basalt	Dry	98 to 8 300	35 to 38
	Wet	98 to 7 700	31 to 36
Conglomerate	Dry	294 to 3 300	35
Chalk	Wet	0 to 390	30
Dolomite	Dry	98 to 7 100	31 to 37
	Wet	98 to 7 100	27 to 35
Gneiss (schistose)	Dry	98 to 7 900	26 to 29
	Wet	98 to 7 700	23 to 26
Granite			
Fine grained	Dry	98 to 7 400	31 to 35
	Wet	98 to 7 300	29 to 31
Coarse grained	Dry	98 to 7 200	31 to 35
	Wet	98 to 7 400	31 to 33
Limestone	Dry	0 to 490	33 to 39
	Wet	0 to 490	33 to 36
	Dry	98 to 7 000	37 to 40
	Wet	98 to 7 000	35 to 38
	Dry	98 to 8 100	37 to 39
	Wet	98 to 8 100	35
Porphyry	Dry	0 to 980	31
	Dry	4 021 to 13 000	31
Sandstone	Dry	0 to 490	26 to 35
	Wet	0 to 490	25 to 33
	Wet	0 to 290	29
	Dry	294 to 2 900	31 to 33
	Dry	98 to 6 900	32 to 34
	Wet	98 to 7 200	31 to 34
Shale	Wet	0 to 290	27
Siltstone	Wet	0 to 290	31
	Dry	98 to 7 400	31 to 33
	Wet	98 to 7 100	27 to 31
Slate	Dry	0 to 1 100	25 to 30

Note: 1 kPa = 0.145 lbf/in^2.

SOIL BEHAVIOR UNDER REPEATED LOADS

Ground motion during earthquakes subjects slopes to repeated loading. Consider again the slope shown in Figure 6.2. The stresses shown are those acting under static conditions. When subjected to earthquake motions, additional stresses of a cyclic nature are induced in the soil. The nature of the stresses at point b is shown in Figure 6.16. For simplicity, it is assumed here that the ground motion during an earthquake consists only of shear waves propagating vertically through the soil. The cyclic stress (τ^e) during an earthquake usually consists of a series of irregular pulses, as shown by the stress-time plot in Figure 6.16c.

Several methods can be used to calculate the response of a slope to repeated loading (see Chapter 7). Some require use of a constitutive relation for the soil, and others use the soil strength under dynamic loading. When most natural soils are subjected to earthquake or other types of repeated loading, the resulting fluctuations in stress produce irreversible changes in pore pressures. These produce long-term and short-term changes in soil strength, and this must be recognized in the design of slopes to resist earthquakes and other kinds of dynamic loads. Strength changes in soils subjected to repeated loading have been the subject of extensive research in recent years (*6.3, 6.23, 6.58, 6.60, 6.62*). In some of those studies, pore pressures and effective stresses were determined (*6.23, 6.60, 6.65*). In most cases, however, only total stress analysis was possible because rapid rates of loading did not permit the measurement of pore pressures. The present state of knowledge is based on contributions from both types of study.

Figure 6.16. Stresses under dynamic loading.

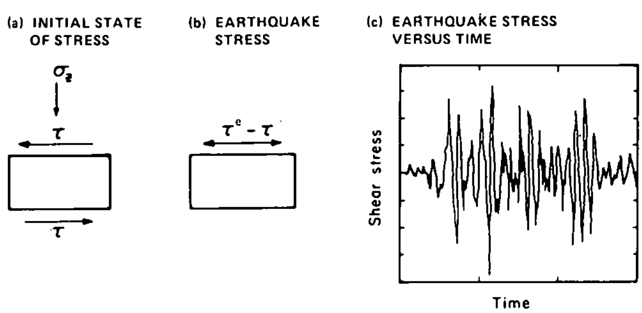


Figure 6.17. Dynamic triaxial test conditions.

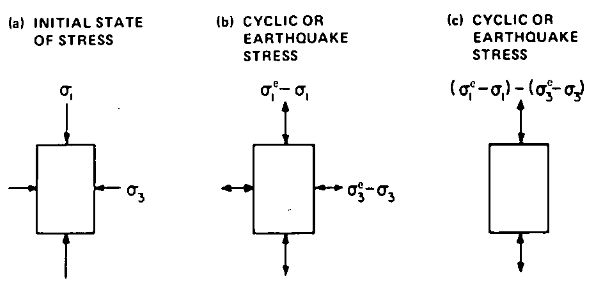
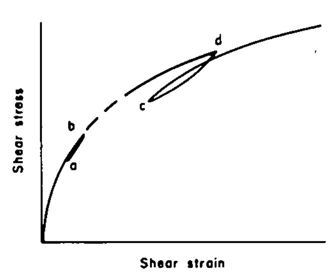


Figure 6.18. Stress-strain relation for cyclic loading.



Repeated Load Tests

Laboratory tests to measure soil strength under repeated loads can be made with the triaxial cell or the simple shear apparatus (*6.63*). Because of difficulties in interpretation and analysis of random loads, such as those shown in Figure 6.16c, current laboratory tests usually employ regular stress pulses. The shape of the stress pulse may be square or triangular or sinusoidal. In the simple shear test, the stress conditions shown in Figure 6.16a and b can be simulated. The sample is consolidated under the static stresses (σ_z and τ), and the peak shear stress during an earthquake is τ^e. Hence, the cyclic stress (τ^e - τ) is applied (Figure 6.16b).

The triaxial test cannot simulate the rotation of principal axes under earthquake loading; therefore, this phenomenon must be ignored. The test procedures are similar to those for static tests discussed in the section on the triaxial test. The principal stresses under static loading are σ_1 and σ_3. The sample is consolidated first under the static stresses (Figure 6.17a), after which the cyclic stresses (σ_1^e - σ_1 and σ_3^e - σ_3) are applied (Figure 6.17b). Because of the relatively short duration of earthquakes, the cyclic stresses are usually applied in the undrained condition. If the soil is saturated, the effective stress does not change under an applied hydrostatic stress. Thus, the loading can be simplified to fluctuation in axial stress only (Figure 6.17c).

Model tests of soil slopes and embankments loaded by means of a shaking table have also been used in design (*6.3*).

Stress-Strain Characteristics

Under repeated loading, the strain produced by a given peak stress is usually different from that produced by a static stress of the same magnitude. Strain continues to increase with successive cycles and depends on several factors, but particularly on the duration of the load, the magnitude of the stresses, and the number of load cycles. A typical stress-strain curve is shown in Figure 6.18. At small strain, the cyclic stress produces the hysteresis loop ab. The shear modulus and damping are equal to the slope of ab and the area enclosed by the loop respectively. At large strains, the hysteresis loop is cd. The shear modulus decreases with strain, and the damping increases with strain. Estimates of the shear modulus and damping factor can be made on the basis of available empirical data (*6.35, 6.65*). For dry soils and soils with low degrees of saturation, the modulus tends to increase with cycles of loading. Figure 6.19 compares soil behavior under repeated loading (curves B and C) with that under a monotonically increasing stress (curve A). The stress-strain relation and pore pressure under repeated loading depend on stress level, stress history, type of loading, number of stress cycles, and degree of saturation.

When a saturated soft clay or loose sand is subjected to a high level of stress (for example, σ_c in Figure 6.19a), positive pore pressures develop (curve C in Figure 6.19b). After a sufficient number of load cycles, the accumulated pore pressure will lead to failure, and the stress-strain curve C (Figure 6.19a) is obtained. Failure occurs as the effective normal stress is reduced (Figure 6.19c). This failure condition is described by several different terms (*6.23, 6.62*). If the sample is loaded to some stress level lower than σ_c, the pore pressure may build up to a certain value and remain at that level (curve B, Figure 6.19b) and the strain will approach a limiting value (curve B, Figure 6.19a). In this case, no failure occurs.

Failure Under Repeated Loading

Failure caused by high pore pressure, as shown in Figure 6.19, is called liquefaction (*6.62*), particularly when applied to cohesionless soils. The relation between cyclic stress level (S) and number of cycles (N) necessary to achieve failure or a particular strain is shown in Figure 6.19d. This is a common and useful relation, particularly when only total stresses are known. At lower levels of cyclic stress, failure does not occur, even under a large number of loading cycles. This is shown as the asymptote in Figure 6.19d; it is called the critical level of repeated loading (*6.60*). Curves such as these make it possible to choose a design stress corresponding to the anticipated number of loading cycles. Alternatively, the critical level of stress may be used for design. In terms of effective stress, the critical level and the corresponding void ratio have been equated (*6.60*) to the "critical state" of the soil, as defined by Schofield and Wroth (*6.61*). Soils other than soft clays and loose sands can also experience strength changes as a result of earthquakes or other repeated loading. The fundamental phenomena that control the strength changes and the states that define the critical level of stress have been considered by Sangrey (*6.59*), and a summary is given in Table 6.3.

Figure 6.19. Behavior of soils under repeated loading.

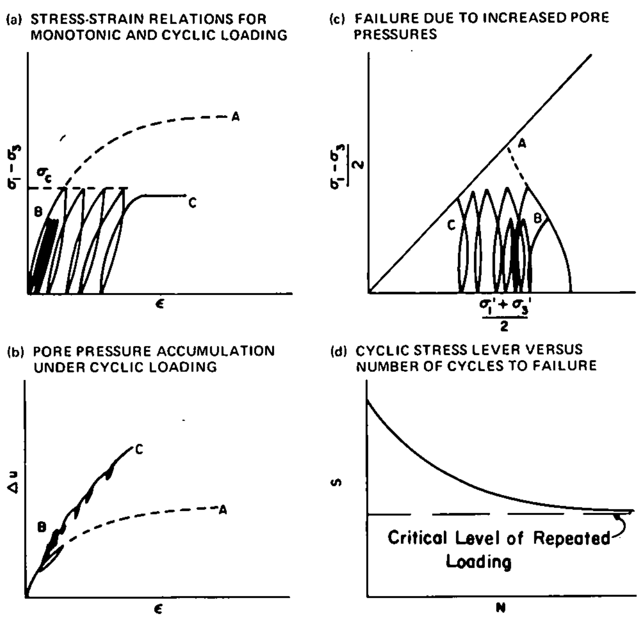


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Table 6.3. Design recommendations for strength of soils subjected to earthquake or other repeated loading (*6.59*).

Soil Type	Critical Level of Stress Under Repeated Loading	Fundamentals of Response
Saturated normally consolidated clay		
Undrained	Strength at critical state; remolded strength	Accumulating positive pore water pressures
Drained	No reduction; strength increases with drainage	Drainage under positive pore-pressure gradient
Saturated overconsolidated clay		
Undrained	Little change from static strength (unless zonal)	Accumulated negative pore water pressure
Drained	Strength decreases with drainage, change in critical state	Negative pore pressures dissipate by increasing water content, decreasing critical state
Extremely sensitive, naturally cemented soils	Reduced strength greater than remolded strength	Fatigue of cementation bonds between particles
Saturated granular materials		
Loose	Strength at critical state or critical void ratio	Accumulation of positive pore pressures
Dense	No reduction from static strength (unless zonal)	Dilation

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