CHAPTER 3

Soil Mechanics Principles

For slope stability analyses to be useful, they must represent the *correct problem, correctly formulated*. This requires (1) mastery of the principles of soil mechanics, (2) knowledge of geology and site conditions, and (3) knowledge of the properties of the soils at the site. In this chapter we deal with the principles of soil mechanics that are needed to understand and to formulate analyses of slope stability problems correctly.

DRAINED AND UNDRAINED CONDITIONS

The concepts of drained and undrained conditions are of fundamental importance in the mechanical behavior of soils, and it is worthwhile to review these concepts at the beginning of this examination of soil mechanics principles. The lay definitions of *drained* and *undrained* (drained = dry or emptied, undrained = not dry or not emptied) do not describe the way these words are used in soil mechanics. The definitions used in soil mechanics are related to the ease and speed with which water moves in or out of soil in comparison with the length of time that the soil is subjected to some change in load. The crux of the issue is whether or not changes in changes in load cause changes in pore pressure:

- *Drained* is the condition under which water is able to flow into or out of a mass of soil in the length of time that the soil is subjected to some change in load. Under drained conditions, changes in the loads on the soil do not cause changes in the water pressure in the voids in the soil, because the water can move in or out of the soil freely when the volume of voids increases or decreases in response to the changing loads.
- *Undrained* is the condition under which there is no flow of water into or out of a mass of soil in the length of time that the soil is subjected to some change in load. Changes in the loads on the soil cause changes in the water pressure in the voids, because the water cannot move in or out in response to the tendency for the volume of voids to change.

An example that illustrates these conditions is shown in Figure 3.1, which shows a clay test specimen in a direct shear test apparatus. The permeability of the clay is low, and its compressibility is high. When the normal load P and the shear load T are increased, there is a tendency for the volume of the clay to decrease. This decrease in volume of the clay would take place entirely by reduction of the volume of the voids because the clay particles themselves are virtually incompressible. However, for the volume of the voids in the clay to decrease, water would have to run out of the clay because water is also virtually incompressible.

If the loads P and T were increased quickly, say in 1 second, the clay specimen would be in an undrained state for some period of time. Within the period of 1 second involved in increasing P and T, there would not be enough time for any significant amount of water to flow out of the clay. It is true that even in a period of 1 second there would be some small amount of flow, but this would be insignificant. For all practical purposes the clay would be undrained immediately after the loads were changed.

If the loads P and T were held constant for a longer period, say one day, the state of the clay specimen would change from undrained to drained. This is because within a period of one day, there would be sufficient time for water to flow out of the clay. Within this time the volume of the voids would decrease and come essentially to equilibrium. It is true that equilibrium would be approached asymptotically, and strictly

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

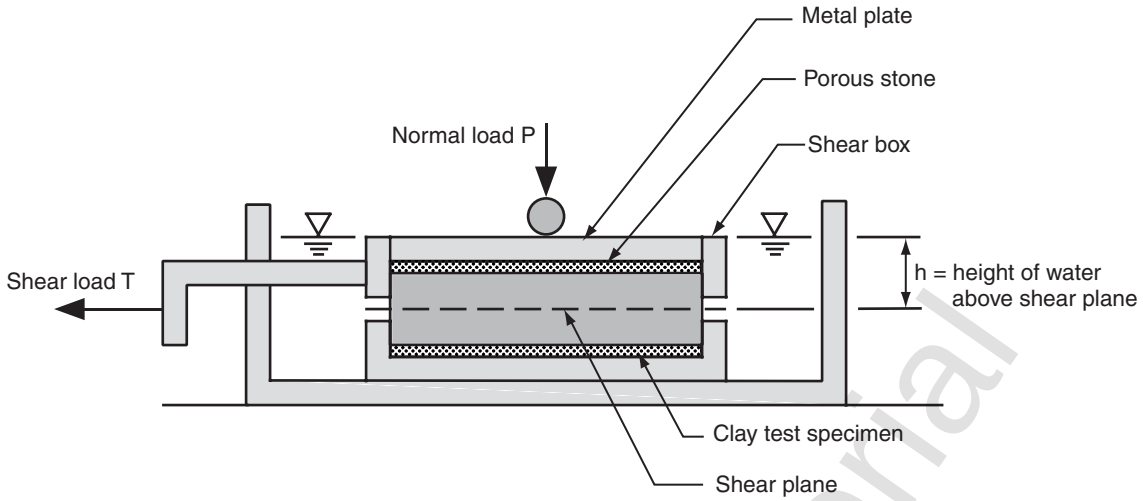


Figure 3.1 Direct shear test apparatus.

speaking, equilibrium would be approached closely but never be reached. However, for all practical purposes the clay would be drained after the loads were held constant for one day.

It is clear from this example that the difference between undrained and drained, as these words are used in soil mechanics, is *time*. Every mass of soil has characteristics that determine how long is required for transition from an undrained to a drained condition. A practical measure of this time is t_{99}, the time required to achieve 99% of the equilibrium volume change, which for practical purposes, we consider to be equilibrium. Using Terzaghi's theory of consolidation, we can estimate the value of t_{99}:

$$t_{99} = 4\,\frac{D^2}{c_v} \tag{3.1}$$

where t_{99} is the time required for 99% of the equilibrium volume change, D the greatest distance that water must travel to flow out of the soil mass (length units), and c_v the coefficient of consolidation (length squared per unit of time). For the test specimen in Figure 3.1, D would be half the specimen thickness, about 1.0 cm, and c_v would be about 2 cm^2/h (19 ft^2/yr). Using these numbers, we would estimate that t_{99} would be 2.0 h. One second after the new loads were applied, the test specimen would be undrained. After 2 hours or longer, the test specimen would be drained.

Parenthetically, it should be noted that the use of the direct shear test as an example of drained and undrained conditions is not meant to indicate that the direct shear apparatus is suitable for both drained and undrained shear tests on soils. Direct shear tests are suitable for drained shear tests on soils, but not for undrained tests. Drained direct tests are performed using thin specimens so that D is small, and using a slow rate of shearing so that the specimen is drained throughout the test. Direct shear tests are not good for undrained tests, because the only way to prevent drainage is to apply the loads very quickly, which can result in higher measured strength due to strain rate effects. Triaxial tests are better suited to undrained testing in the laboratory, because drainage can be prevented completely by sealing the test specimens in impermeable membranes. Undrained triaxial tests can therefore be performed slowly enough to eliminate undesirable rate effects, and still be undrained.

Recapitulation

- The difference between undrained and drained conditions is time.
- *Undrained* signifies a condition where changes in loads occur more rapidly than water can flow in or out of the soil. The pore pressures increase or decrease in response to the changes in loads.
- *Drained* signifies a condition where changes in load are slow enough, or remain in place long enough, so that water is able to flow in or out of the soil, permitting the soil to reach a state of equilibrium with regard to water flow. The pore pressures in the drained condition are controlled by the hydraulic boundary conditions, and are unaffected by the changes in loads.

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

TOTAL AND EFFECTIVE STRESSES

Stress is defined as force per unit area. *Total stress* is the sum of all forces, including those transmitted through interparticle contacts and those transmitted through water pressures, divided by the total area. *Total area* includes both the area of voids and the area of solid.

Effective stress includes only the forces that are transmitted through particle contacts. It is equal to the total stress minus the water pressure. The *total normal stress* on the potential shear plane in the test specimen in Figure 3.1 is equal to

$$\sigma = \frac{W + P}{A} \tag{3.2}$$

where σ is the total stress (force per unit of area); W the weight of the upper half of the specimen, porous stone, metal plate, and the steel ball through which the load is applied; P the applied normal load (F); and A the total area (L^2). For a typical direct shear apparatus, with a 102-mm^2 (4-in^2) shear box, W would be about 12.4 N (2.8 lb).

Before any load is applied to the specimen (when $P = 0$), the normal stress on the horizontal plane is

$$\sigma_0 = \frac{12.4 \text{ N}}{0.0103 \text{ m}^2} = 1.2 \text{ kPa} \tag{3.3}$$

The effective stress is equal to the total stress minus the water pressure. Consider the condition before any load is applied to the specimen (when $P = 0$): If the specimen has had enough time to come to a drained condition, the water pressure would be hydrostatic and its value would be governed by the depth of water in the reservoir around the shear box. For a typical direct shear apparatus the depth of water (h in Figure 3.1) would be about 2 in. (about 0.051 m). The corresponding hydrostatic water pressure at the level of the horizontal plane would be

$$u_0 = \gamma_w h = (9.81 \text{ kN/m}^3)(0.051 \text{ m}) = 0.5 \text{ kPa} \tag{3.4}$$

where u_0 is the initial water pressure in the specimen, γ_w the unit weight of water = 9.81 kN/m^3, and h the height of water above the horizontal plane = 0.051 m.

With $\sigma = 1.2$ kPa and $u_0 = 0.5$ kPa, the effective stress is equal to 0.7 kPa:

$$\sigma_0' = \sigma_0 - u_0 = 1.2 \text{ kPa} - 0.5 \text{ kPa} = 0.7 \text{ kPa} \tag{3.5}$$

where σ_0' is the initial effective stress. If a load P = 200 N is applied to the specimen, the change in normal stress would be

$$\Delta\sigma = \frac{200 \text{ N}}{0.0103 \text{ m}^2} = 19.4 \text{ kPa} \tag{3.6}$$

and the total stress after the load is applied would be

$$\sigma = \sigma_0 + \Delta\sigma = 1.2 \text{ kPa} + 19.4 \text{ kPa} = 20.6 \text{ kPa} \tag{3.7}$$

The values of total stress are defined without reference to how much of the force might be carried by contacts between particles or to how much is transmitted through water pressure. Total stress is the same for the undrained and drained conditions. The value of total stress depends only on equilibrium; it is equal to the total of all normal forces divided by the total area.

When the load P is applied rapidly and the specimen is undrained, the pore pressure changes. The specimen is confined within the shear box and cannot deform. The clay is saturated (the voids are filled with water), so the volume of the specimen cannot change until water flows out. In this condition, the added load is carried entirely by increased water pressure. The soil skeleton (the assemblage of particles in contact with one another) does not change shape, does not change volume, and carries none of the new applied load.

Under these conditions the increase in water pressure is equal to the change in total stress:

$$\Delta u = \Delta\sigma = 19.4 \text{ kPa} \tag{3.8}$$

where Δu is the increase in water pressure due to the change in load in the undrained condition. The water pressure after the load is applied is equal to the initial water pressure plus this change in pressure:

$$u = u_0 + \Delta u = 0.5 \text{ kPa} + 19.4 \text{ kPa} = 19.9 \text{ kPa} \tag{3.9}$$

The effective stress is equal to the total stress [Eq. (3.7)] minus the water pressure [Eq. (3.9)]:

$$\sigma' = 20.6 \text{ kPa} - 19.9 \text{ kPa} = 0.7 \text{ kPa} \tag{3.10}$$

Because the increase in water pressure caused by the 200-N load is equal to the increase in total stress, the effective stress does not change.

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

The effective stress after the load is applied [Eq. (3.10)] is the same as the effective stress before the load is applied [Eq. (3.5)]. This is because the specimen is undrained. Water does not have time to drain as the load is applied, so there is no volume change in the saturated specimen. As a result, the soil skeleton does not strain. The load carried by the soil skeleton, which is measured by the value of effective stress, does not change.

If the load is maintained over a period of time, drainage will occur, and eventually the specimen will be drained. The drained condition is achieved when there is no difference between the water pressures inside the specimen (the pore pressure) and the water pressure outside, governed by the water level in the reservoir around the direct shear apparatus. This condition will be achieved (for practical purposes) in about 2 hours and will persist until the load is changed again. After 2 hours the specimen will have achieved 99% equilibrium, the volume change will be essentially complete, and the pore pressure on the horizontal plane will be equal to the hydrostatic head at that level, u = 0.5 kPa.

In this drained condition the effective stress is

$$\sigma' = 20.6 \text{ kPa} - 0.5 \text{ kPa} = 20.1 \text{ kPa} \quad (3.11)$$

and all of the 200-N load is carried by the soil skeleton.

Recapitulation

- *Total stress* is the sum of all forces, including those transmitted through particle contacts and those transmitted through water pressures, divided by total area.
- *Effective stress* is equal to the total stress minus the water pressure. It is the force transmitted through particle contacts, divided by total area.

DRAINED AND UNDRAINED SHEAR STRENGTHS

Shear strength is defined as the maximum value of shear stress that the soil can withstand. The *shear stress* on the horizontal plane in the direct shear test specimen in Figure 3.1 is equal to the shear force divided by the area:

$$\tau = \frac{T}{A} \quad (3.12)$$

The shear strength of soils is controlled by effective stress, whether failure occurs under drained or undrained conditions. The relationship between shear strength and effective stress can be represented by a Mohr–Coulomb strength envelope, as shown in Figure 3.2. The relationship between τ and σ' shown in Figure 3.2 can be expressed as

$$s = c' + \sigma'_{ff} \tan \phi' \quad (3.13)$$

where c' is the effective stress cohesion, σ'_{ff} the effective stress on the failure plane at failure, and ϕ' the effective stress angle of internal friction.

Sources of Shear Strength

If a shear load T is applied to the test specimen shown in Figure 3.1, the top of the shear box will move to the left relative to the bottom of the box. If the shear load is large enough, the clay will fail by shearing on the horizontal plane, and the displacement would be very large. Failure would be accompanied by development of a rupture zone, or break through the soil, along the horizontal plane.

As the upper half of the specimen moved to the left with respect to the lower half and the strength of the soil was mobilized, the particles within the rupture zone would be displaced from their original positions relative to adjacent particles. Interparticle bonds would be broken, some individual particles would be broken, particles would rotate and be reoriented into new positions, and particles would slide across their contacts with neighboring particles. These movements of the particles would be resisted by the strength of interparticle bonds, by frictional resistance to sliding, and by forces from adjacent particles resisting displacement and reorientation. These types of resistance are the sources of shear strength in soils.

The two most important factors governing the strengths of soils are the magnitude of the interparticle contact forces and the density of the soil. Larger interparticle contact forces (larger values of effective stress) and higher densities result in higher strengths. As τ increases, the shear displacement (Δx) between the top and the bottom of the shear box would increase, as shown in Figure 3.3. This shear displacement results from shear strains in the rupture zone. The shear displacements in direct shear tests can be measured easily, but shear strains cannot be determined, because the thickness of the shear zone is not known. While the direct shear test can be used to measure the shear strengths of soils, it provides only qualitative information about stress–strain behavior. It is possible to determine whether soils are ductile (shear resistance

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

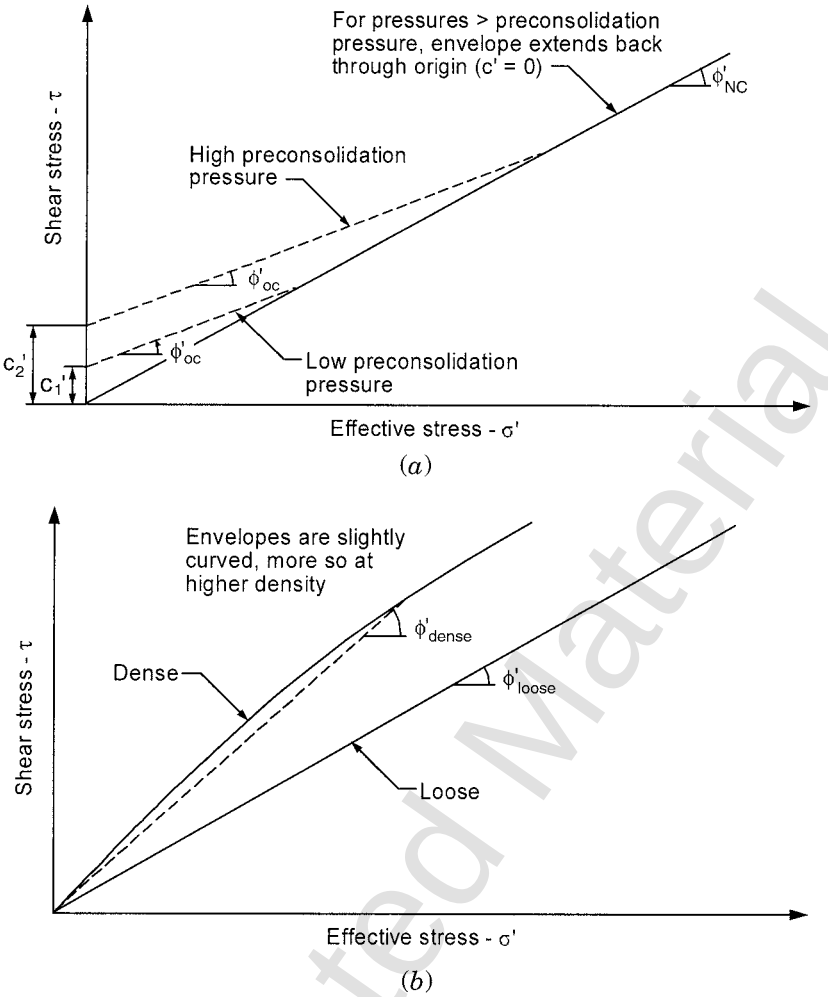


Figure 3.2 Effective stress shear strength envelopes: (*a*) for clay; (*b*) for sands, gravels, and rockfill.

remains high after failure) or brittle (shear resistance decreases after failure).

Drained Strength

Drained strength is the strength of the soil when it is loaded slowly enough so that no excess pore pressures are induced by applied loads. In the field, drained conditions result when loads are applied slowly to a mass of soil, or where they persist for a long enough time so that the soil can drain. In the laboratory, drained conditions are achieved by loading test specimens slowly so that excess pore pressures do not develop as the soil is loaded.

Imagine that the direct shear test specimen shown in Figure 3.1 reached a drained condition under the load of 200 N and was then loaded to failure by increasing T slowly so that excess pore pressures did not develop. As shown by Eq. (3.11), the effective stress on the horizontal plane at equilibrium under the 200-N load would be 20.1 kPa, and it would remain constant as the clay was sheared slowly.

The strength of the specimen can be calculated using Eq. (3.13). If the clay is normally consolidated, c' would be zero. The value of ϕ' would probably be between 25 and 35° for normally consolidated sandy or silty clay. As an example, suppose that ϕ' is equal to 30°. The drained strength of the clay would be

$$s = c' + \sigma'_{ff} \tan \phi' = 0 + (20.1)(0.58) = 11.6 \text{ kPa} \tag{3.14}$$

where $c' = 0$ and $\tan \phi' = \tan 30° = 0.58$.

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

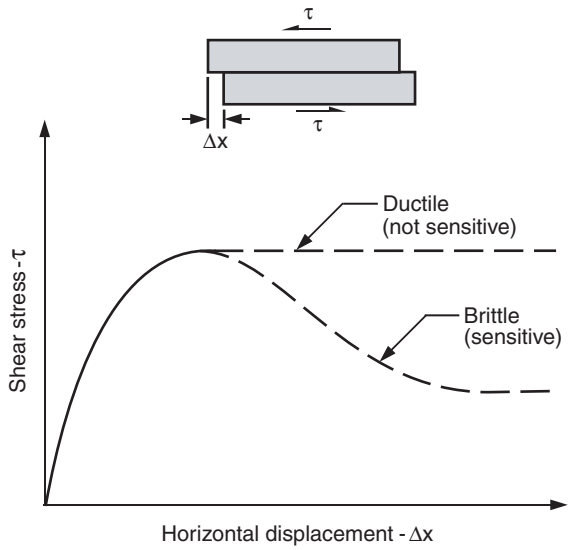


Figure 3.3 Shear stress–shear displacement curves for direct shear test.

Volume Changes During Drained Shear

Whether a soil tends to compress or dilate when sheared depends on its density and the effective stress that confines it. In dense soils the particles are packed tightly together, and tight packing results in a great deal of interference between particles when they move relative to one another. In very dense soils, particles cannot move relative to each other unless they ride up over each other, which causes dilation.

Higher effective stresses tend to prevent dilation, because work is required to cause the soil to expand against the effective confining pressure. If the effective confining pressure is high enough, the soil may not dilate. Instead, as shearing takes place, individual particles will be broken.

In soils with low densities the soil particles are farther apart on average, in a loose assemblage. As a loose soil is sheared, particles tend to fall into the gaps between adjacent particles, and the volume of the soil decreases.

The lower the density and the higher the effective stress, the more likely the soil is to compress when sheared. Conversely, the higher the density and the lower the confining pressure, the more likely the soil is to dilate. In clays, density is governed primarily by the highest effective stress to which the clay has been subjected.

A *normally consolidated soil* is one that has not been subjected to an effective stress higher than the present effective stress, and its density is the lowest possible for any given effective stress. As a result, normally consolidated clays tend to compress when sheared.

An *overconsolidated clay* is one that was subjected previously to higher effective stress and thus has a higher density than that of a normally consolidated soil at the same effective stress. As a result, overconsolidated soils compress less when sheared than do normally consolidated soils, or if the previous maximum effective stress was much higher than the effective stress during shearing, the clay will dilate.

Pore Pressure Changes During Undrained Shear

The tendency of normally consolidated and lightly overconsolidated clays to compress when sheared results in increased pore pressures when shear stresses increase under undrained conditions. The tendency of heavily overconsolidated soils to dilate when sheared results in negative changes in pore pressures when shear stresses increase under undrained conditions. Thus, when clays are sheared under undrained conditions, the effective stress on the potential failure plane changes, becoming lower in normally consolidated soils and higher in heavily overconsolidated soils.

Undrained Strength

Undrained strength is the strength of the soil when loaded to failure under undrained conditions. In the field, conditions closely approximating undrained condtions result when loads are applied to a mass of soil faster than the soil can drain. In the laboratory, undrained conditions are achieved by loading test specimens so rapidly that they cannot drain, or by sealing them in impermeable membranes. (As noted previously, it is preferable to control drainage through the use of impermeable membranes rather than very high rates of loading, to avoid high strain rates that are not representative of field conditions.)

Imagine that the direct shear test specimen shown in Figure 3.1 reached a drained condition under the load of 200 N and was then loaded to failure by increasing T rapidly. As shown by Eq. (3.11), the effective stress on the horizontal plane at equilibrium under the 200-N load, before the shear load was increased, would be 20.1 kPa. The pore pressure before the shear load was increased would be 0.5 kPa, as shown by Eq. (3.4).

As the shear load T was applied without allowing time for drainage, the pore pressure would increase, because the clay is normally consolidated under the 20.1 kPa effective stress. As the shear load T is increased, the pore pressure within a specimen of a typical normally consolidated clay under these conditions would increase by about 12 kPa, and the effective normal stress on the failure plane at failure (σ'_{ff}) would

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

decrease by the same amount. The effective stress on the failure plane at failure would thus be equal to 20.1 kPa − 12 kPa, or about 8 kPa. The undrained shear strength of the clay would thus be about 4.6 kPa:

$$s = c' + \sigma' \tan\phi' = 0 + (8.0)(0.58) = 4.6 \text{ kPa} \tag{3.15}$$

Figure 3.4 shows the stress paths and shear strengths for drained and undrained failure of the direct shear test specimen. The drained stress path is vertical, corresponding to an increase in shear stress and constant effective normal stress on the horizontal plane. The undrained stress path curves to the left, as the increase in shear stress is accompanied by a decrease in effective normal stress due to the increase in pore pressure.

As is typical for normally consolidated clays, the undrained strength is lower than the drained strength. This is due to the fact that the pore pressure increases and the effective stress decreases during undrained shear. For very heavily overconsolidated clays, the reverse is true: The undrained strength is greater than the drained strength, because pore pressure decreases and effective stress increases during undrained shear.

Strength Envelopes

Strength envelopes for soils are developed by performing strength tests on soils using a range of pressures and plotting the results on a Mohr stress diagram, as shown in Figure 3.5. Both effective stress and total stress strength envelopes can be developed. The strength envelopes shown in Figure 3.5 are representative of the results of tests on undisturbed specimens of clay, all trimmed from the same undisturbed sample and therefore all having the same preconsolidation pressure. The effective stress envelope represents the fundamental behavior of the clay, because the strength of the clay is controlled by effective stress and density. The total stress envelope reflects the pore pressures that develop during undrained shear as well as the fundamental behavior in terms of effective stresses.

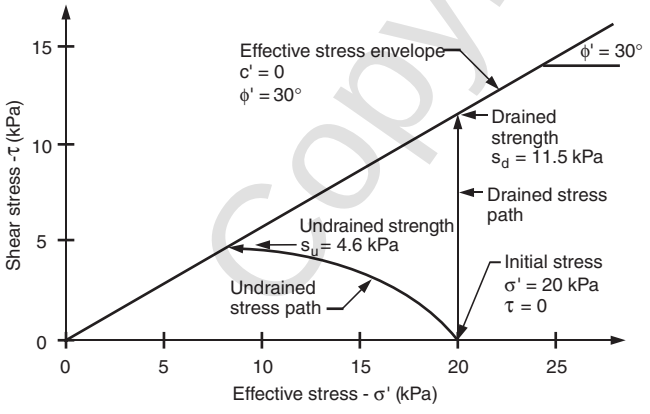


Figure 3.4 Drained and undrained stress paths and shear strengths.

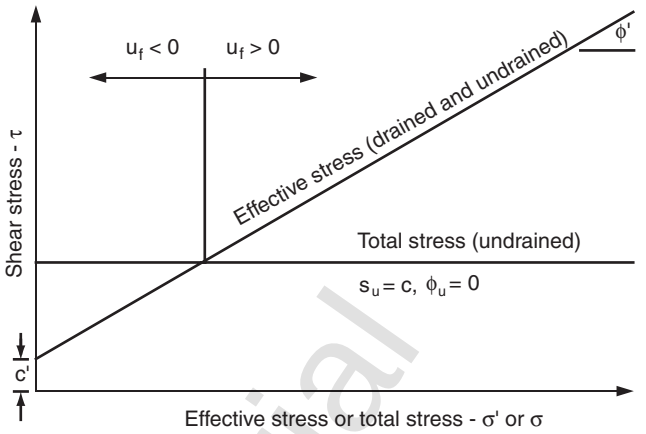


Figure 3.5 Drained and undrained strength envelopes for saturated clay.

Effective stress strength envelopes for clays consist of two parts. At high stresses the clay is normally consolidated, and the high-pressure part of the envelope extends back through the origin. At low stresses the clay is overconsolidated. The strength envelope in this range of pressures does not extend through the origin. The values of the effective stress shear strength parameters c' and ϕ' depend on whether the clay is normally consolidated or overconsolidated. If the clay is tested in a range of pressures where it is normally consolidated, c' is zero and ϕ' is constant. If the clay is tested in a range of pressures where it is overconsolidated, c' is greater than zero and ϕ' is smaller than the normally consolidated value. Because the values of c' and ϕ' that characterize the strength of the clay depend on the magnitude of the stresses in relation to the preconsolidation pressure, it is important that the range of stresses used in laboratory strength tests should correspond to the range of stresses involved in the problem being analyzed.

The total stress envelope is horizontal, representing shear strength that is constant and independent of the magnitude of the total stress used in the test. This behavior is characterized by these relationships:

$$c = s_u \tag{3.16a}$$

$$\phi_u = 0 \tag{3.16b}$$

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

where c is the total stress cohesion intercept, s_u the undrained shear strength, and ϕ_u the total stress friction angle.

The shear strength is the same for all values of total normal stress because the clay is saturated and undrained. Increasing or decreasing the total normal stress results only in a change in pore pressure that is equal in magnitude and opposite in sign to the change in normal stress. Thus, the effective stress is constant, and because the effective stress is constant the strength is constant because strength is controlled by effective stress. Although strength is controlled by effective stress, it is more convenient for some purposes to use the total stress envelope and the corresponding total stress parameters. Use of effective and total stress parameters in stability analyses is discussed later in the chapter.

If the clay were only partly saturated, the undrained strength envelope would not be horizontal. Instead, it would be inclined and shaped as shown in Figure 3.6. As the total normal stress increases, the strength also increases, because changes in total stress do not cause equal increases in pore pressure. As the total stress applied to a partly saturated specimen is increased, both the pore pressure and the effective stress increase. This occurs because, with both water and air in the voids of the clay, the pore fluid (the mixture of water and air) is not incompressible, and only part of the added total stress is carried by the pore fluid. The balance is carried by the soil skeleton, which results in an increase in effective stress.

How much of a change in total stress is borne by the change in pore pressure and how much by change in effective stress depends on the degree of saturation of the clay. At degrees of saturation in the range of 70% and lower, the change in pore pressure is negligible and virtually all of the change in total stress is reflected in change in effective stress. At degrees of saturation approaching 100%, the opposite is true: Virtually all of the change in total stress is reflected in change in pore pressure, and the change in effective stress is negligible. This behavior is responsible for the curvature of the total stress envelope in Figure 3.6: The degree of saturation increases as the total confining pressure increases. Therefore, at low values of total stress, where the degree of saturation is lower, the envelope is steeper because changes in effective stress are a larger portion of changes in total stress. At high values of total stress, where the degree of saturation is higher, the envelope is flatter because changes in effective stress are a smaller portion of changes in total stress.

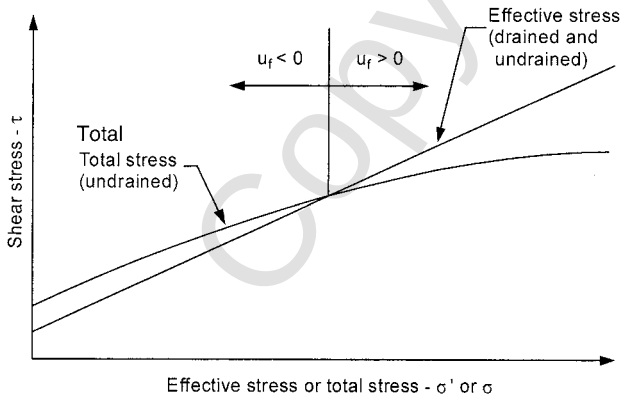


Figure 3.6 Drained and undrained strength envelopes for partly saturated clay.

Recapitulation

- *Shear strength* is defined as the maximum shear stress that the soil can withstand.
- The strength of soil is controlled by effective stresses, whether failure occurs under drained or undrained conditions.
- *Drained strength* is the strength corresponding to failure with no change in effective stress on the failure plane.
- *Undrained strength* is the strength corresponding to failure with no change in water content.
- Effective stress strength envelopes represent fundamental behavior, because strength is controlled by effective stress and density.
- Total stress strength envelopes reflect the pore pressures that develop during undrained shear, as well as fundamental behavior in terms of effective stress.
- Total stress strength envelopes for saturated clays are horizontal, corresponding to $c = s_u$, $\phi_u = 0$. Total stress envelopes for partly saturated clays are not horizontal, and ϕ_u is greater than zero.

BASIC REQUIREMENTS FOR SLOPE STABILITY ANALYSES

Whether slope stability analyses are performed for drained conditions or undrained conditions, the most basic requirement is that equilibrium must be satisfied in terms of total stresses. All body forces (weights), and all external loads, including those due to water pressures acting on external boundaries, must be included in the analysis. These analyses provide two useful results: (1) the total normal stress on the shear

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

surface and (2) the shear stress required for equilibrium.

The *factor of safety* for the shear surface is the ratio of the shear strength of the soil divided by the shear stress required for equilibrium. The normal stresses along the slip surface are needed to evaluate the shear strength: Except for soils with $\phi = 0$, the shear strength depends on the normal stress on the potential plane of failure.

In effective stress analyses, the pore pressures along the shear surface are subtracted from the total stresses to determine effective normal stresses, which are used to evaluate shear strengths. Therefore, to perform effective stress analyses, it is necessary to know (or to estimate) the pore pressures at every point along the shear surface. These pore pressures can be evaluated with relatively good accuracy for drained conditions, where their values are determined by hydrostatic or steady seepage boundary conditions. Pore pressures can seldom be evaluated accurately for undrained condtions, where their values are determined by the response of the soil to external loads.

In total stress analyses, pore pressures are not subtracted from the total stresses, because shear strengths are related to total stresses. Therefore, it is not necessary to evaluate and subtract pore pressures to perform total stress analyses. Total stress analyses are applicable only to undrained conditions. The basic premise of *total stress analysis* is this: The pore pressures due to undrained loading are determined by the behavior of the soil. For a given value of total stress on the potential failure plane, there is a unique value of pore pressure and therefore a unique value of effective stress. Thus, although it is true that shear strength is really controlled by effective stress, it is possible for the undrained condition to relate shear strength to total normal stress, because effective stress and total stress are uniquely related for the undrained condition. Clearly, this line of reasoning does not apply to drained conditions, where pore pressures are controlled by hydraulic boundary conditions rather than the response of the soil to external loads.

Analyses of Drained Conditions

Drained conditions are those where changes in load are slow enough, or where they have been in place long enough, so that all of the soils reach a state of equilibrium and no excess pore pressures are caused by the loads. In drained conditions pore pressures are controlled by hydraulic boundary conditions. The water within the soil may be static, or it may be seeping steadily, with no change in the seepage over time and no increase or decrease in the amount of water within the soil. If these conditions prevail in all the soils at a site, or if the conditions at a site can reasonably be approximated by these conditions, a drained analysis is appropriate. A *drained analysis* is performed using:

- Total unit weights
- Effective stress shear strength parameters
- Pore pressures determined from hydrostatic water levels or steady seepage analyses

Analyses of Undrained Conditions

Undrained conditions are those where changes in loads occur more rapidly than water can flow in or out of the soil. The pore pressures are controlled by the behavior of the soil in response to changes in external loads. If these conditions prevail in the soils at a site, or if the conditions at a site can reasonably be approximated by these conditions, an undrained analysis is appropriate. An *undrained analysis* is performed using:

- Total unit weights
- Total stress shear strength parameters

How Long Does Drainage Take?

As discussed earlier, the difference between undrained and drained conditions is time. The drainage characteristics of the soil mass, and its size, determine how long will be required for transition from an undrained to a drained condition. As shown by Eq. (3.1):

$$t_{99} = 4\,\frac{D^2}{c_v} \tag{3.17}$$

where t_{99} is the time required to reach 99% of drainage equilibrium, D the length of the drainage path, and c_v the coefficient of consolidation.

Values of c_v for clays vary from about 1.0 cm^2/h (10 ft^2/yr) to about 100 times this value. Values of c_v for silts are on the order of 100 times the values for clays, and values of c_v for sands are on the order of 100 times the values for silts, and higher. These typical values can be used to develop some rough ideas of the lengths of time required to achieve drained conditions in soils in the field.

Drainage path lengths are related to layer thicknesses. They are half the layer thickness for layers that are bounded on both sides by more permeable soils, and they are equal to the layer thickness for layers that are drained only on one side. Lenses or layers of silt or sand within clay layers provide internal drainage, reducing the drainage path length to half of the thickness between internal drainage layers.

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

Values of t_{99} calculated using Eq. (3.17) are shown in Figure 3.7. For most practical conditions, many years or tens of years are required to reach drainage equilibrium in clay layers, and it is usually necessary to consider undrained conditions in clays. On the other hand, sand and gravels layers almost always reach drainage equilibrium quickly, and only drained conditions need be considered for these materials. Silts fall in between sands and clays, and it is often difficult to anticipate whether silt layers are better approximated as drained or undrained. When there is doubt whether a layer will be drained or undrained, the answer is to analyze both conditions, to cover the range of possibilities.

Short-Term Analyses

Short term refers to conditions during or following construction—the time immediately following the change in load. For example, if constructing a sand embankment on a clay foundation takes two months, the short-term condition for the embankment would be the end of construction, or two months. Within this period of time, it would be a reasonable approximation that no drainage would occur in the clay foundation, whereas the sand embankment would be fully drained.

For this condition it would be logical to perform a drained analysis of the embankment and an undrained analysis of the clay foundation. There is no problem with performing a single analysis in which the embankment is considered to be drained and is treated in terms of effective stresses, and in which the foundation is considered to be undrained and is treated in terms of total stresses.

As discussed earlier, equilibrium in terms of total stresses must be satisfied for both total and effective stress analyses. The only differences between total and effective stress analyses relate to the strength parameters that are used and whether pore pressures are specified. In the case of short-term analysis of a sand embankment on a clay foundation, the strength of the sand would be characterized in terms of effective stresses (by a value of ϕ' for the sand), and the strength of the clay would be characterized in terms of total stresses (by values of $s_u = c$ varying with depth, with $\phi_u = 0$ for a saturated clay).

Pore pressures would be specified for the sand if the water table was above the top of the clay or if there was seepage through the embankment, but pore pressures would not be specified for the clay. There would, of course, be pore pressures in the clay. However, because the strength of the clay is related to total stress, it would be unneccesary to specify these nonzero values. Because most computer programs subtract pore pressures when they are specified, specifying pore

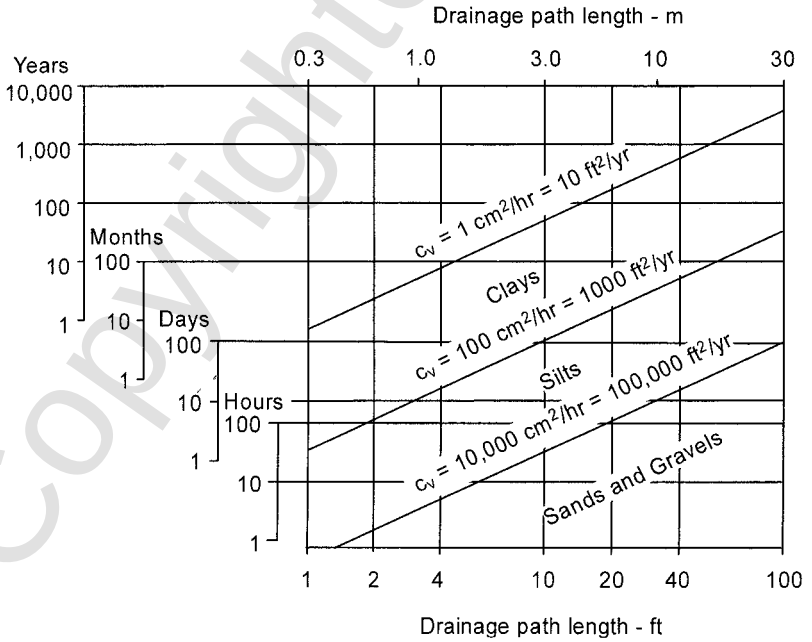


Figure 3.7 Time required for drainage of soil deposits (t_{99} based on Terzagh's theory of consolidation).

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

pressures for soils that are being treated as undrained can result in errors. Therefore, for soils that are treated in terms of total stresses, pore pressures should be set to zero, even though, in fact, they are not zero. (In the particular case of $\phi = 0$, no error will result if pore pressures are not specified as zero, because strengths are independent of normal stress, and misevaluating normal stress does not result in strengths that are wrongly evaluated.)

External water pressures acting on the surface of the foundation or the embankment would be specified for both materials, because external water pressures are a component of total stress, and they must be included to satisfy equilibrium in terms of total stress.

Long-Term Analyses

After a period of time, the clay foundation would reach a drained condition, and the analysis for this condition would be performed as discussed earlier under "Analyses of Drained Conditions," because *long term* and *drained conditions* carry exactly the same meaning. Both of these terms refer to the condition where drainage equilibrium has been reached and there are no excess pore pressures due to external loads.

For the long-term condition, both the sand embankment and clay foundation would be characterized in terms of effective stresses. Pore pressures, determined from hydrostatic water levels or steady seepage analyses, would be specified for both materials. External water pressures on the surface of the foundation or embankment would be specified for both materials, as always; these must be included to satisfy equilibrium in terms of total stress.

Progressive Failure

One of the fundamental assumptions of limit equilibrium analyses is that the strength of the soil can be mobilized over a wide range of strains, as shown by the curve labeled "ductile" in Figure 3.3. This implicit assumption arises from the fact that limit equilibrium analyses provide no information regarding deformations or strains.

Progressive failure is a strong possibility in the case of excavated slopes in overconsolidated clays and shales, particularly stiff-fissured clays and shales. These materials have brittle stress–strain characteristics, and they contain high horizontal stresses, often higher than the vertical stress. When an excavation is made in stiff fissured clay or shale, the excavated slope rebounds horizontally, as shown in Figure 3.8. Finite element studies by Duncan and Dunlop (1969), (1970) showed that shear stresses are very high at the toe of the slope, and there is a tendency for failure to begin at the toe and progress back beneath the crest, as shown in Figure 3.8.

Immediately after excavation of the slope (at time t_1), the stresses at point A might just have reached the peak of the stress–displacement curve, and the stresses at points B and C would be lower. With time, the slope would continue to rebound into the cut, due to a delayed response to the unloading from the excavation, and possibly also due to swelling of the clay as its water content increases following the reduction in stress. At a later time (t_2), therefore, the displacements at *A*, *B*, and *C* would all be larger, as shown in Figure 3.8. The shear stress at point *A* would decease as it moved beyond the peak, and the shear stresses at

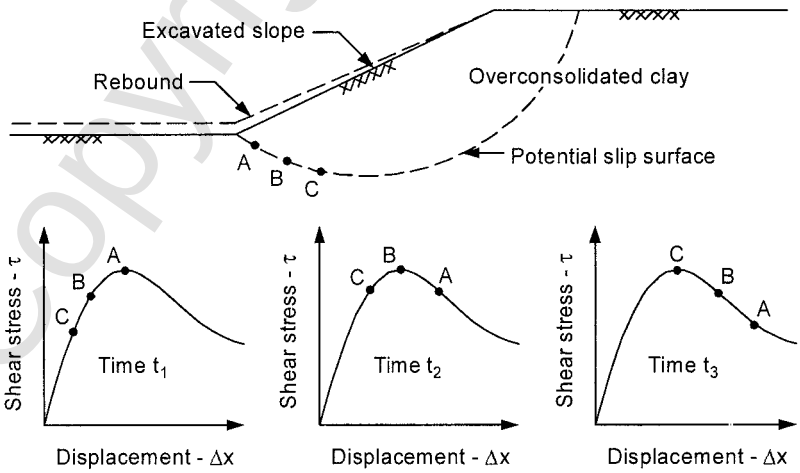


Figure 3.8 Mechanism of progressive failure of an excavated slope in overconsolidated clay.

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com

points B and C would increase. At a later time (t_3), the displacement at point B would be large enough so that the shear stress there would fall below the peak. Through this process, progressively, failure would spread around the slip surface, without ever mobilizing the peak shear strength simultaneously at all points along the slip surface.

Because progressive failure can occur for soils with brittle stress–strain characteristics, peak strengths should not be used for these soils in limit equilibrium analyses; using peak strengths for brittle soils can lead to inaccurate and unconservative assessment of stability. As discussed in Chapter 5, experience with slopes in overconsolidated clays, particularly fissured clays, shows that *fully softened strengths* are appropriate for these materials in cases where slickensides have not developed, and *residual strengths* are appropriate in conditions where slickensides have developed.

Recapitulation

- Equilibrium must be satisfied in terms of total stress for all slope stability analyses.
- In effective stress analyses, pore pressures are subtracted from total stresses to evaluate the effective stresses on the shear surface.
- In total stress analyses, pore pressures are not subtracted. Shear strengths are related to total stresses.
- The basic premise of total stress analyses is that there is a unique relationship between total stress and effective stress. This is true only for undrained conditions.
- Total stress analyses are not applicable to drained conditions.
- The time required for drainage of soil layers varies from minutes for sands and gravels to tens or hundreds of years for clays.
- In short-term conditions, soils that drain slowly may best be characterized as undrained, while soils that drain more quickly are best characterized as drained. Analyses of such conditions can be performed by using effective stress strength parameters for the drained soils and total stress strength parameters for the undrained soils.
- When effective stress strength parameters are used, pore pressures determined from hydraulic boundary conditions are specified. When total stress strength parameters are used, no pore pressures are specified.
- An implicit assumption of limit equilibrium analyses is that the soils exhibit ductile stress–strain behavior. Peak strengths should not be used for materials such as stiff fissured clays and shales which have brittle stress–strain characteristics, because progressive failure can occur in these materials. Using peak strengths can result in inaccurate and unconservative evaluations of stability.

Copyrighted Material

Copyright © 2005 John Wiley & Sons Retrieved from: www.knovel.com