EM 1110-2-1902
31 Oct 2003

US Army Corps
of Engineers®

ENGINEERING AND DESIGN

Slope Stability

ENGINEER MANUAL

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Publication Number: **EM 1110-2-1902**
Title: **Engineering and Design - Slope Stability**
Proponent: **CECW-EW**
Publication Date: **31 October 2003**
Distribution Restriction Statement: **Approved for public release; distribution is unlimited.**
File Format: **Adobe Acrobat.pdf.** Adobe Acrobat Reader software is required to read portable document files (pdf). Click Viewers to access free Adobe Acrobat Reader software and follow installation instructions or go directly to the Adobe homepage at http://www.adobe.com.
File Size: Files 1 MB or larger have had their file size noted.

CECW-EW

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-2-1902

Manual
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Engineering and Design
SLOPE STABILITY

Chapter 1
Introduction

1-1. Purpose and Scope

This engineer manual (EM) provides guidance for analyzing the static stability of slopes of earth and rock-fill dams, slopes of other types of embankments, excavated slopes, and natural slopes in soil and soft rock. Methods for analysis of slope stability are described and are illustrated by examples in the appendixes. Criteria are presented for strength tests, analysis conditions, and factors of safety. The criteria in this EM are to be used with methods of stability analysis that satisfy all conditions of equilibrium. Methods that do not satisfy all conditions of equilibrium may involve significant inaccuracies and should be used only under the restricted conditions described herein. This manual is intended to guide design and construction engineers, rather than to specify rigid procedures to be followed in connection with a particular project.

1-2. Applicability

This EM is applicable to all USACE elements and field operating activities having responsibility for analyzing stability of slopes.

1-3. References

Appendix A contains a list of Government and non-Government references pertaining to this manual. Each reference is identified in the text by either the designated publication number or by author and date.

1-4. Notation and Glossary

Symbols used in this manual are listed and defined in Appendix B. The notation in this manual corresponds whenever possible to that recommended by the American Society of Civil Engineers.

1-5. Basic Design Considerations

a. General overview. Successful design requires consistency in the design process. What are considered to be appropriate values of factor of safety are inseparable from the procedures used to measure shear strengths and analyze stability. Where procedures for sampling, testing, or analysis are different from the procedures described in this manual, it is imperative to evaluate the effects of those differences.

b. Site characterization. The stability of dams and slopes must be evaluated utilizing pertinent geologic information and information regarding in situ engineering properties of soil and rock materials. The geologic information and site characteristics that should be considered include:

(1) Groundwater and seepage conditions.

(2) Lithology, stratigraphy, and geologic details disclosed by borings and geologic interpretations.

(3) Maximum past overburden at the site as deduced from geological evidence.

(4) Structure, including bedding, folding, and faulting.

(5) Alteration of materials by faulting.

(6) Joints and joint systems.

(7) Weathering.

(8) Cementation.

(9) Slickensides.

(10) Field evidence relating to slides, earthquake activity, movement along existing faults, and tension jointing.

c. Material characterization. In evaluating engineering properties of soil and rock materials for use in design, consideration must be given to: (1) possible variation in natural deposits or borrow materials, (2) natural water contents of the materials, (3) climatic conditions, (4) possible variations in rate and methods of fill placement, and (5) variations in placement water contents and compacted densities that must be expected with normal control of fill construction. Other factors that must be considered in selecting values of design parameters, which can be evaluated only through exercise of engineering judgment, include: (1) the effect of differential settlements where embankments are located on compressible foundations or in narrow, deep valleys, and (2) stress-strain compatibility of zones of different materials within an embankment, or of the embankment and its foundation. The stability analyses presented in this manual assume that design strengths can be mobilized simultaneously in all materials along assumed sliding surfaces.

d. Conventional analysis procedures (limit equilibrium). The conventional limit equilibrium methods of slope stability analysis used in geotechnical practice investigate the equilibrium of a soil mass tending to move downslope under the influence of gravity. A comparison is made between forces, moments, or stresses tending to cause instability of the mass, and those that resist instability. Two-dimensional (2-D) sections are analyzed and plane strain conditions are assumed. These methods assume that the shear strengths of the materials along the potential failure surface are governed by linear (Mohr-Coulomb) or nonlinear relationships between shear strength and the normal stress on the failure surface.

(1) A free body of the soil mass bounded below by an assumed or known surface of sliding (potential slip surface), and above by the surface of the slope, is considered in these analyses. The requirements for static equilibrium of the soil mass are used to compute a factor of safety with respect to shear strength. The factor of safety is defined as the ratio of the available shear resistance (the capacity) to that required for equilibrium (the demand). Limit equilibrium analyses assume the factor of safety is the same along the entire slip surface. A value of factor of safety greater than 1.0 indicates that capacity exceeds demand and that the slope will be stable with respect to sliding along the assumed particular slip surface analyzed. A value of factor of safety less than 1.0 indicates that the slope will be unstable.

(2) The most common methods for limit equilibrium analyses are methods of slices. In these methods, the soil mass above the assumed slip surface is divided into vertical slices for purposes of convenience in analysis. Several different methods of slices have been developed. These methods may result in different values of factor of safety because: (a) the various methods employ different assumptions to make the problem statically determinate, and (b) some of the methods do not satisfy all conditions of equilibrium. These issues are discussed in Appendix C.

e. Special analysis procedures (finite element, three-dimensional (3-D), and probabilistic methods).

(1) The finite element method can be used to compute stresses and displacements in earth structures. The method is particularly useful for soil-structure interaction problems, in which structural members interact with a soil mass. The stability of a slope cannot be determined directly from finite element analyses, but the

computed stresses in a slope can be used to compute a factor of safety. Use of the finite element method for stability problems is a complex and time-consuming process. Finite element analyses are discussed briefly in Appendix C.

(2) Three-dimensional limit equilibrium analysis methods consider the 3-D shapes of slip surfaces. These methods, like 2-D methods, require assumptions to achieve a statically determinate definition of the problem. Most do not satisfy all conditions of static equilibrium in three dimensions and lack general methodologies for locating the most critical 3-D slip surface. The errors associated with these limitations may be of the same magnitude as the 3-D effects that are being modeled. These methods may be useful for estimating potential 3-D effects for a particular slip surface. However, 3-D methods are not recommended for general use in design because of their limitations. The factors of safety presented in this manual are based on 2-D analyses. Three-dimensional analysis methods are not included within the scope of this manual.

(3) Probabilistic approaches to analysis and design of slopes consider the magnitudes of uncertainties regarding shear strengths and the other parameters involved in computing factors of safety. In the traditional (deterministic) approach to slope stability analysis and design, the shear strength, slope geometry, external loads, and pore water pressures are assigned specific unvarying values. Appendix D discusses shear strength value selection. The value of the calculated factor of safety depends on the judgments made in selecting the values of the various design parameters. In probabilistic methods, the possibility that values of shear strength and other parameters may vary is considered, providing a means of evaluating the degree of uncertainty associated with the computed factor of safety. Although probabilistic techniques are not required for slope analysis or design, these methods allow the designer to address issues beyond those that can be addressed by deterministic methods, and their use is encouraged. Probabilistic methods can be utilized to supplement conventional deterministic analyses with little additional effort. Engineering Technical Letter (ETL) 1110-2-556 (1999) describes techniques for probabilistic analyses and their application to slope stability studies.

f. Computer programs and design charts. Computer programs provide a means for detailed analysis of slope stability. Design charts provide a rapid method of analysis but usually require simplifying approximations for application to actual slope conditions. The choice to use computer programs or slope stability charts should be made based on the complexity of the conditions to be analyzed and the objective of the analysis. Even when computer programs are used for final analyses, charts are often useful for providing preliminary results quickly, and for providing an independent check on the results of the computer analyses. These issues are discussed in Appendix E.

g. Use and value of results. Slope stability analyses provide a means of comparing relative merits of trial cross sections during design and for evaluating the effects of changes in assumed embankment and foundation properties. The value of stability analyses depends on the validity of assumed conditions, and the value of the results is increased where they can be compared with analyses for similar structures where construction and operating experiences are known.

h. Strain softening and progressive failure. "Progressive failure" occurs under conditions where shearing resistance first increases and then decreases with increasing strain, and, as a result, the peak shear strengths of the materials at all points along a slip surface cannot be mobilized simultaneously. When progressive failure occurs, a critical assumption of limit equilibrium methods – that peak strength can be mobilized at all points along the shear surface -- is not valid. "Strain softening" is the term used to describe stress-strain response in which shear resistance falls from its peak value to a lower value with increasing shear strain. There are several fundamental causes and forms of strain softening behavior, including:

(1) Undrained strength loss caused by contraction-induced increase in pore water pressure. Liquefaction of cohesionless soils is an extreme example of undrained strength loss as the result of contraction-induced pore pressure, but cohesive soils are also subject to undrained strength loss from the same cause.

(2) Drained strength loss occurring as a result of dilatancy. As dense soil is sheared, it may expand, becoming less dense and therefore weaker.

(3) Under either drained or undrained conditions, platy clay particles may be reoriented by shear deformation into a parallel arrangement termed "slickensides," with greatly reduced shear resistance. If materials are subject to strain softening, it cannot be assumed that a factor of safety greater than one based on peak shear strength implies stability, because deformations can cause local loss of strength, requiring mobilization of additional strength at other points along the slip surface. This, in turn, can cause additional movement, leading to further strain softening. Thus, a slope in strain softening materials is at risk of progressive failure if the peak strength is mobilized anywhere along the failure surface. Possible remedies are to design so that the factor of safety is higher, or to use shear strengths that are less than peak strengths. In certain soils, it may even be necessary to use residual shear strengths.

i. Strain incompatibility. When an embankment and its foundation consist of dissimilar materials, it may not be possible to mobilize peak strengths simultaneously along the entire length of the slip surface. Where stiff embankments overly soft clay foundations, or where the foundation of an embankment consists of brittle clays, clay shales, or marine clays that have stress-strain characteristics different from those of the embankment, progressive failure may occur as a result of strain incompatibility.

j. Loss of strength resulting from tension cracks. Progressive failure may start when tension cracks develop as a result of differential settlements or shrinkage. The maximum depth of cracking can be estimated from Appendix C, Equation C-36. Shear resistance along tension cracks should be ignored, and in most cases it should be assumed that the crack will fill with water during rainfall.

k. Problem shales. Shales can be divided into two broad groups. Clay shales (compaction shales) lack significant strength from cementation. Cemented shales have substantial strength because of calcareous, siliceous, other types of chemical bonds, or heat, and pressure. Clay shales usually slake rapidly into unbonded clay when subjected to a few cycles of wetting and drying, whereas cemented shales are either unaffected by wetting and drying, or are reduced to sand-size aggregates of clay particles by wetting and drying. All types of shales may present foundation problems where they contain joints, shear bands, slickensides, faults, seams filled with soft material, or weak layers. Where such defects exist, they control the strength of the mass. Prediction of the field behavior of clay shales should not be based solely on results of conventional laboratory tests, since they may be misleading, but on detailed geologic investigations and/or large-scale field tests. Potential problem shales can be recognized by: (1) observation of landslides or faults through aerial or ground reconnaissance, (2) observation of soft zones, shear bands, or slickensides in recovered core or exploration trenches, and (3) clay mineralogical studies to detect the presence of bentonite layers.

1-6. Stability Analysis and Design Procedure

The process of evaluating slope stability involves the following chain of events:

a. Explore and sample foundation and borrow sources. EM 1110-1-1804 provides methods and procedures that address these issues.

b. Characterize the soil strength (see Appendix D). This usually involves testing representative samples as described in EM 1110-2-1906. The selection of representative samples for testing requires much care.

c. Establish the 2-D idealization of the cross section, including the surface geometry and the subsurface boundaries between the various materials.

d. Establish the seepage and groundwater conditions in the cross section as measured or as predicted for the design load conditions. EM 1110-2-1901 describes methods to establishing seepage conditions through analysis and field measurements.

e. Select loading conditions for analysis (see Chapter 2).

f. Select trial slip surfaces and compute factors of safety using Spencer's method. In some cases it may be adequate to compute factors of safety using the Simplified Bishop Method or the force equilibrium method (including the Modified Swedish Method) with a constant side force (Appendix C). Appendix F provides example problems and calculations for the simplified Bishop and Modified Swedish Procedures.

g. Repeat step f above until the "critical" slip surface has been located. The critical slip surface is the one that has the lowest factor of safety and which, therefore, represents the most likely failure mechanism.

Steps *f* and *g* are automated in most slope stability computer programs, but several different starting points and search criteria should be used to ensure that the critical slip surface has been located accurately.

h. Compare the computed factor of safety with experienced-based criteria (see Chapter 3).

Return to any of the items above, and repeat the process through step *h*, until a satisfactory design has been achieved. When the analysis has been completed, the following steps (not part of this manual) complete the design process:

i. The specifications should be written consistent with the design assumptions.

j. The design assumptions should be verified during construction. This may require repeating steps *b, c, d, f, g,* and *h* and modifying the design if conditions are found that do not match the design assumptions.

k. Following construction, the performance of the completed structure should be monitored. Actual piezometric surfaces based on pore water pressure measurements should be compared with those assumed during design (part *d* above) to determine if the embankment meets safe stability standards.

1-7. Unsatisfactory Slope Performance

a. Shear failure. A shear failure involves sliding of a portion of an embankment, or an embankment and its foundation, relative to the adjacent mass. A shear failure is conventionally considered to occur along a discrete surface and is so assumed in stability analyses, although the shear movements may in fact occur across a zone of appreciable thickness. Failure surfaces are frequently approximately circular in shape. Where zoned embankments or thin foundation layers overlying bedrock are involved, or where weak strata exist within a deposit, the failure surface may consist of interconnected arcs and planes.

b. Surface sloughing. A shear failure in which a surficial portion of the embankment moves downslope is termed a surface slough. Surface sloughing is considered a maintenance problem, because it usually does not affect the structural capability of the embankment. However, repair of surficial failures can entail considerable cost. If such failures are not repaired, they can become progressively larger, and may then represent a threat to embankment safety.

c. Excessive deformation. Some cohesive soils require large strains to develop peak shear resistance. As a consequence, these soils may deform excessively when loaded. To avoid excessive deformations, particular attention should be given to the stress-strain response of cohesive embankment and foundation soils during design. When strains larger than 15 percent are required to mobilize peak strengths, deformations in the embankment or foundation may be excessive. It may be necessary in such cases to use the shearing resistance mobilized at 10 or 15 percent strain, rather than peak strengths, or to limit placement water contents to the dry side of optimum to reduce the magnitudes of failure strains. However, if cohesive soils are compacted too dry, and they later become wetter while under load, excessive settlement may occur. Also, compaction of cohesive soils dry of optimum water content may result in brittle stress-strain behavior and cracking of the embankment. Cracks can have adverse effects on stability and seepage. When large strains are required to develop shear strengths, surface movement measurement points and piezometers should be installed to monitor movements and pore water pressures during construction, in case it becomes necessary to modify the cross section or the rate of fill placement.

d. Liquefaction. The phenomenon of soil liquefaction, or significant reduction in soil strength and stiffness as a result of shear-induced increase in pore water pressure, is a major cause of earthquake damage to embankments and slopes. Most instances of liquefaction have been associated with saturated loose sandy or silty soils. Loose gravelly soil deposits are also vulnerable to liquefaction (e.g., Coulter and Migliaccio 1966; Chang 1978; Youd et al. 1984; and Harder 1988). Cohesive soils with more than 20 percent of particles finer than 0.005 mm, or with liquid limit (LL) of 34 or greater, or with the plasticity index (PI) of 14 or greater are generally considered not susceptible to liquefaction. The methodology to evaluate liquefaction susceptibility will be presented in an Engineer Circular, "Dynamic Analysis of Embankment Dams," which is still in draft form.

e. Piping. Erosion and piping can occur when hydraulic gradients at the downstream end of a hydraulic structure are large enough to move soil particles. Analyses to compute hydraulic gradients and procedures to control piping are contained in EM 1110-2-1901.

f. Other types of slope movements. Several types of slope movements, including rockfalls, topples, lateral spreading, flows, and combinations of these, are not controlled by shear strength (Huang 1983). These types of mass movements are not discussed in this manual, but the possibility of their occurrence should not be ignored.

Chapter 2
Design Considerations

2-1. Introduction

Evaluation of slope stability requires:

a. Establishing the conditions, called "design conditions" or "loading conditions," to which the slope may be subjected during its life, and

b. Performing analyses of stability for each of these conditions. There are four design conditions that must be considered for dams: (1) during and at the end of construction, (2) steady state seepage, (3) sudden drawdown, and (4) earthquake loading. The first three conditions are static; the fourth involves dynamic loading.

Details concerning the analysis of slope stability for the three static loading conditions are discussed in this chapter. Criteria regarding which static design conditions should be applied and values of factor of safety are discussed in Chapter 3. Procedures for analysis of earthquake loading conditions can be found in an Engineer Circular, "Dynamic Analysis of Embankment Dams," which is still in draft form..

2-2. Aspects Applicable to All Load Conditions

a. General. Some aspects of slope stability computations are generally applicable, independent of the design condition analyzed. These are discussed in the following paragraphs.

b. Shear strength. Correct evaluation of shear strength is essential for meaningful analysis of slope stability. Shear strengths used in slope stability analyses should be selected with due consideration of factors such as sample disturbance, variability in borrow materials, possible variations in compaction water content and density of fill materials, anisotropy, loading rate, creep effects, and possibly partial drainage. The responsibility for selecting design strengths lies with the designer, not with the laboratory.

(1) Drained and undrained conditions. A prime consideration in characterizing shear strengths is determining whether the soil will be drained or undrained for each design condition. For drained conditions, analyses are performed using drained strengths related to effective stresses. For undrained conditions, analyses are performed using undrained strengths related to total stresses. Table 2-1 summarizes appropriate shear strengths for use in analyses of static loading conditions.

(2) Laboratory strength tests. Laboratory strength tests can be used to evaluate the shear strengths of some types of soils. Laboratory strength tests and their interpretation are discussed in Appendix D.

(3) Linear and nonlinear strength envelopes. Strength envelopes used to characterize the variation of shear strength with normal stress can be linear or nonlinear, as shown in Figure 2-1.

(a) Linear strength envelopes correspond to the Mohr-Coulomb failure criterion. For total stresses, this is expressed as:

$$s = c + \sigma \tan \varphi \qquad (2\text{-}1)$$

Table 2-1
Shear Strengths and Pore Pressures for Static Design Conditions

Design Condition	Shear Strength	Pore Water Pressure
During Construction and End-of-Construction	Free draining soils – use drained shear strengths related to effective stresses[1]	Free draining soils – Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations if there is no flow, or using steady seepage analysis techniques (flow nets or finite element analyses).
	Low-permeability soils – use undrained strengths related to total stresses[2]	Low-permeability soils – Total stresses are used; pore water pressures are set to zero in the slope stability computations.
Steady-State Seepage Conditions	Use drained shear strengths related to effective stresses.	Pore water pressures from field measurements, hydrostatic pressure computations for no-flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses, or finite difference analyses).
Sudden Drawdown Conditions	Free draining soils – use drained shear strengths related to effective stresses.	Free draining soils – First-stage computations (before drawdown) – steady seepage pore pressures as for steady seepage condition. Second- and third-stage computations (after drawdown) – pore water pressures estimated using same techniques as for steady seepage, except with lowered water level.
	Low-permeability soils – Three-stage computations: First stage--use drained shear strength related to effective stresses; second stage--use undrained shear strengths related to consolidation pressures from the first stage; third stage--use drained strengths related to effective stresses, or undrained strengths related to consolidation pressures from the first stage, depending on which strength is lower – this will vary along the assumed shear surface.	Low-permeability soils – First-stage computations--steady-state seepage pore pressures as described for steady seepage condition. Second-stage computations – total stresses are used; pore water pressures are set to zero. Third-stage computations -- same pore pressures as free draining soils if drained strengths are used; pore water pressures are set to zero where undrained strengths are used.

[1] Effective stress shear strength parameters can be obtained from consolidated-drained (CD, S) tests (direct shear or triaxial) or consolidated-undrained (CU, $\bar{R}$) triaxial tests on saturated specimens with pore water pressure measurements. Repeated direct shear or Bromhead ring shear tests should be used to measure residual strengths. Undrained strengths can be obtained from unconsolidated-undrained (UU, Q) tests. Undrained shear strengths can also be estimated using consolidated-undrained (CU, R) tests on specimens consolidated to appropriate stress conditions representative of field conditions; however, the "R" or "total stress" envelope and associated c and ϕ, from CU, R tests should not be used.
[2] For saturated soils use $\phi = 0$. Total stress envelopes with $\phi > 0$ are only applicable to partially saturated soils.

where

s = maximum possible value of shear stress = shear strength

c = cohesion intercept

σ = normal stress

ϕ = total stress friction angle.

(b) For effective stresses, the Mohr-Coulomb failure criterion is expressed as

$$s = c' + \sigma' \tan \varphi' \qquad (2\text{-}2)$$

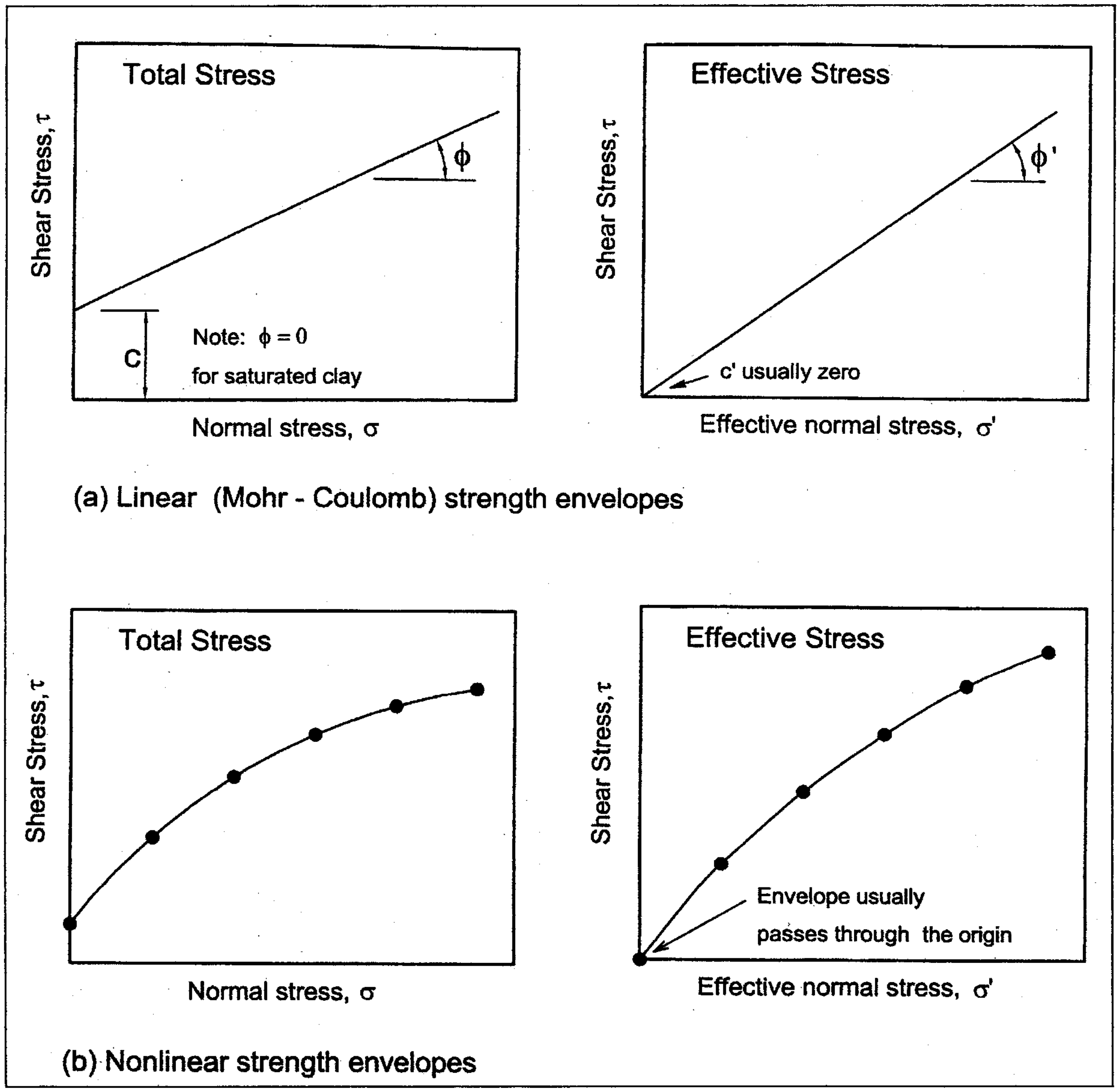

Figure 2-1. Strength envelopes for soils

where

s = maximum possible value of shear stress = shear strength

c' = effective stress cohesion intercept

σ' = effective normal stress

φ' = effective stress friction angle.

(c) Nonlinear strength envelopes are represented by pairs of values of s and σ, or s and σ'.

(4) Ductile and brittle stress-strain behavior. For soils with ductile stress-strain behavior (shear resistance does not decrease significantly as strain increases beyond the peak), the peak shear strength can be used in evaluating slope stability. Ductile stress-strain behavior is characteristic of most soft clays, loose sands, and clays compacted at water contents higher than optimum. For soils with brittle stress-strain behavior (shear resistance decreases significantly as strain increases beyond the peak), the peak shear resistance should not be used in evaluating slope stability, because of the possibility of progressive failure. A shear resistance lower than the peak, possibly as low as the residual shear strength, should be used, based on the judgment of the designer. Brittle stress-strain behavior is characteristic of stiff clays and shales, dense sands, and clays compacted at optimum water content or below.

(5) Peak, fully softened, and residual shear strengths. Stiff-fissured clays and shales pose particularly difficult problems with regard to strength evaluation. Experience has shown that the peak strengths of these materials measured in laboratory tests should not be used in evaluating long-term slope stability. For slopes without previous slides, the "fully softened" strength should be used. This is the same as the drained strength of remolded, normally consolidated test specimens. For slopes with previous slides, the "residual" strength should be used. This is the strength reached at very large shear displacements, when clay particles along the shear plane have become aligned in a "slickensided" parallel orientation. Back analysis of slope failures is an effective means of determining residual strengths of stiff clays and shales. Residual shear strengths can be measured in repeated direct shear tests on undisturbed specimens with field slickensided shear surfaces appropriately aligned in the shear box, repeated direct shear tests on undisturbed or remolded specimens with precut shear planes, or Bromhead ring shear tests on remolded material.

(6) Strength anisotropy. The shear strengths of soils may vary with orientation of the failure plane. An example is shown in Figure 2-2. In this case the undrained shear strength on horizontal planes ($\alpha = 0$) was low because the clay shale deposit had closely spaced horizontal fissures. Shear planes that crossed the fissures, even at a small angle, are characterized by higher strength.

(7) Strain compatibility. As noted in Appendix D, Section D-9, different soils reach their full strength at different values of strain. In a slope consisting of several soil types, it may be necessary to consider strain compatibility among the various soils. Where there is a disparity among strains at failure, the shear resistances should be selected using the same strain failure criterion for all of the soils.

c. Pore water pressures. For effective stress analyses, pore water pressures must be known and their values must be specified. For total stress analyses using computer software, hand computations, or slope stability charts, pore water pressures are specified as zero although, in fact, the pore pressures are not zero. This is necessary because all computer software programs for slope stability analyses subtract pore pressure from the total normal stress at the base of the slice:

$$\text{normal stress on base of slice} = \sigma - u \tag{2-3}$$

The quantity σ in this equation is the total normal stress, and u is pore water pressure.

(1) For total stress analyses, the normal stress should be the total normal stress. To achieve this, the pore water pressure should be set to zero. Setting the pore water pressure to zero ensures that the total normal stress is used in the calculations, as is appropriate.

(2) For effective stress analyses, appropriate values of pore water pressure should be used. In this case, using the actual pore pressure ensures that the effective normal stress ($\sigma' = \sigma - u$) on the base of the slice is calculated correctly.

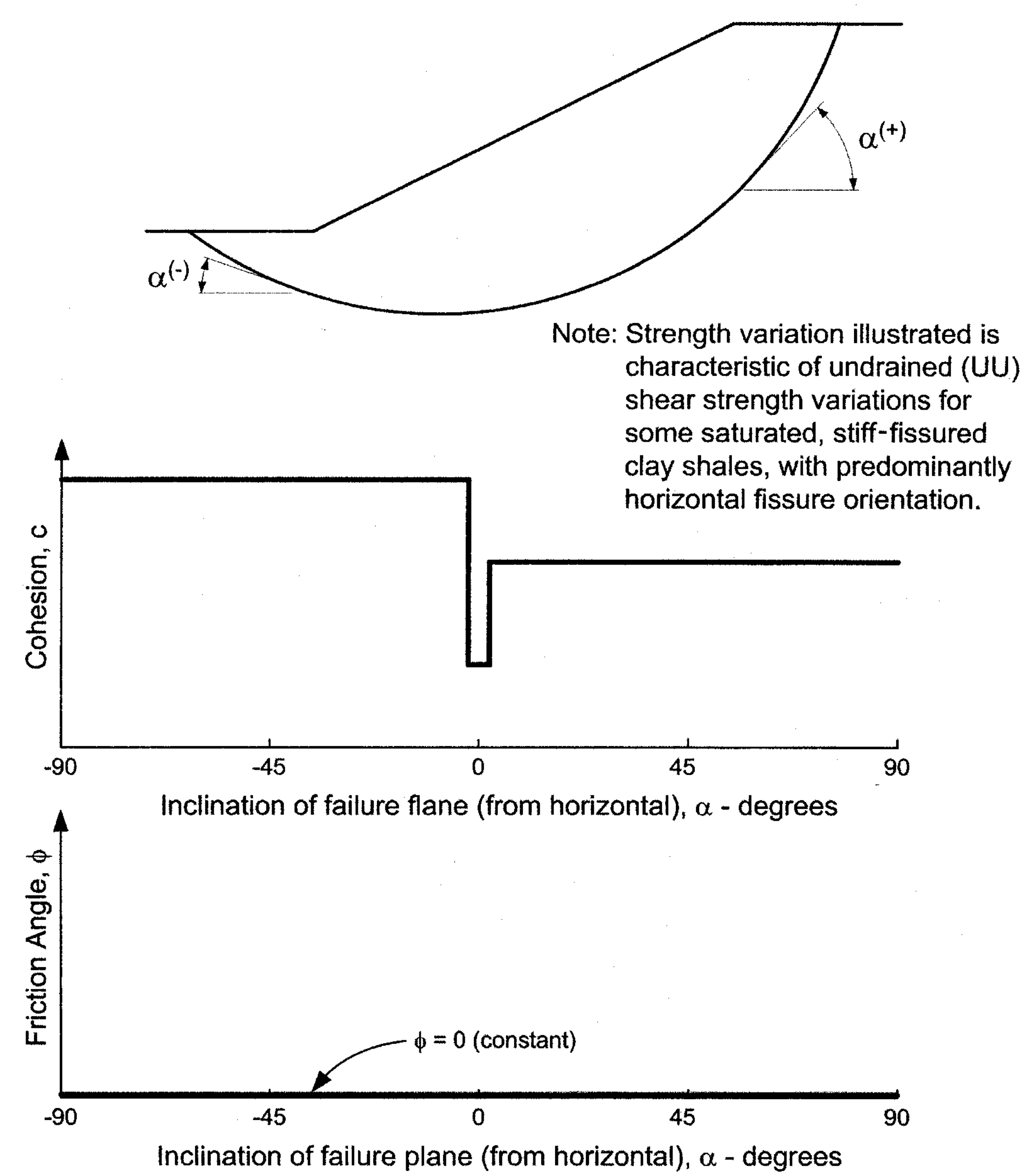

Figure 2-2. Representation of shear strength parameters for anisotropic soil

d. Unit weights. The methods of analysis described in this manual use total unit weights for both total stress analyses and effective stress analyses. This applies for soils regardless of whether they are above or below water. Use of buoyant unit weights is not recommended, because experience has shown that confusion often arises as to when buoyant unit weights can be used and when they cannot. When computations are performed with computer software, there is no computational advantage in the use of buoyant unit weights. Therefore, to avoid possible confusion and computational errors, total unit weights should be used for all soils in all conditions. Total unit weights are used for all formulations and examples presented in this manual.

e. External loads. All external loads imposed on the slope or ground surface should be represented in slope stability analyses, including loads imposed by water pressures, structures, surcharge loads, anchor forces, hawser forces, or other causes. Slope stability analyses must satisfy equilibrium in terms of total stresses and forces, regardless of whether total or effective stresses are used to specify the shear strength.

f. Tensile stresses and vertical cracks. Use of Mohr-Coulomb failure envelopes with an intercept, c or c', implies that the soil has some tensile strength (Figure 2-3). Although a cohesion intercept is convenient for representing the best-fit linear failure envelope over a range of positive normal stresses, the implied tensile strength is usually not reasonable. Unless tension tests are actually performed, which is rarely done, the implied tensile strength should be neglected. In most cases actual tensile strengths are very small and contribute little to slope stability.

(1) One exception, where the tensile strengths should be considered, is in back-analyses of slope failures to estimate the shear strength of natural deposits. In many cases, the existence of steep natural slopes can only be explained by tensile strength of the natural deposits. The near vertical slopes found in loess deposits are an example. It may be necessary to include significant tensile strength in back-analyses of such slopes to obtain realistic strength parameters. If strengths are back-calculated assuming no tensile strength, the shear strength parameters may be significantly overestimated.

(2) Significant tensile strengths in uncemented soils can often be attributed to partially saturated conditions. Later saturation of the soil mass can lead to loss of strength and slope failure. Thus, it may be most appropriate to assume significant tensile strength in back-analyses and then ignore the tensile strength (cohesion) in subsequent forward analysis of the slope. Guidelines to estimate shear strength in partially saturated soils are given in Appendix D, Section D-11.

(3) When a strength envelope with a significant cohesion intercept is used in slope stability computations, tensile stresses appear in the form of negative forces on the sides of slices and sometimes on the bases of slices. Such tensile stresses are almost always located along the upper portion of the shear surface, near the crest of the slope, and should be eliminated unless the soil possesses significant tensile strength because of cementing which will not diminish over time. The tensile stresses are easily eliminated by introducing a vertical crack of an appropriate depth (Figure 2-4). The soil upslope from the crack (to the right of the crack in Figure 2-4) is then ignored in the stability computations. This is accomplished in the analyses by terminating the slices near the crest of the slope with a slice having a vertical boundary, rather than the usual triangular shape, at the upper end of the shear surface. If the vertical crack is likely to become filled with water, an appropriate force resulting from water in the crack should be computed and applied to the boundary of the slice adjacent to the crack.

(4) The depth of the crack should be selected to eliminate tensile stresses, but not compressive stresses. As the crack depth is gradually increased, the factor of safety will decrease at first (as tensile stresses are eliminated), and then increase (as compressive stresses are eliminated) (Figure 2-5). The appropriate depth for a crack is the one producing the minimum factor of safety, which corresponds to the depth where tensile, but not compressive, stresses are eliminated.

(5) The depth of a vertical crack often can be estimated with suitable accuracy from the Rankine earth pressure theory for active earth pressures beneath a horizontal ground surface. The stresses in the tensile stress zone of the slope can be approximated by active Rankine earth pressures as shown in Figure 2-6. In the case where shear strengths are expressed using total stresses, the depth of tensile stress zone, z_t, is given by:

$$z_t = \frac{2c_D}{\gamma}\tan\left(45° + \frac{\phi_D}{2}\right) \qquad (2\text{-}4)$$

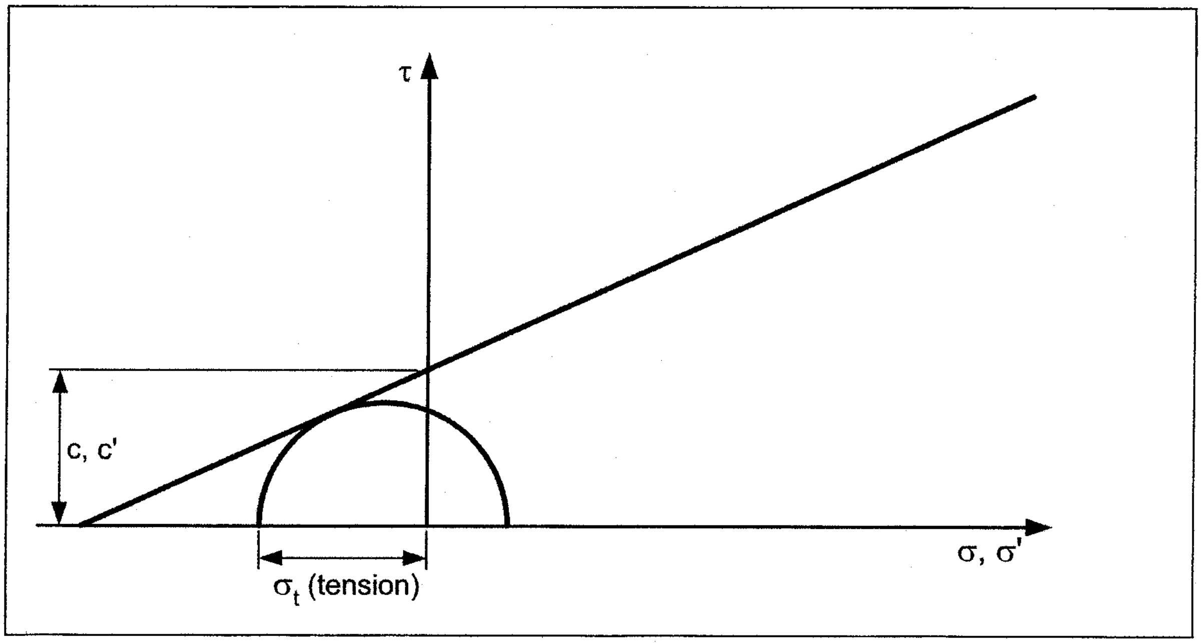

Figure 2-3. Tensile stresses resulting from a Mohr-Coulomb failure envelope with a cohesion intercept

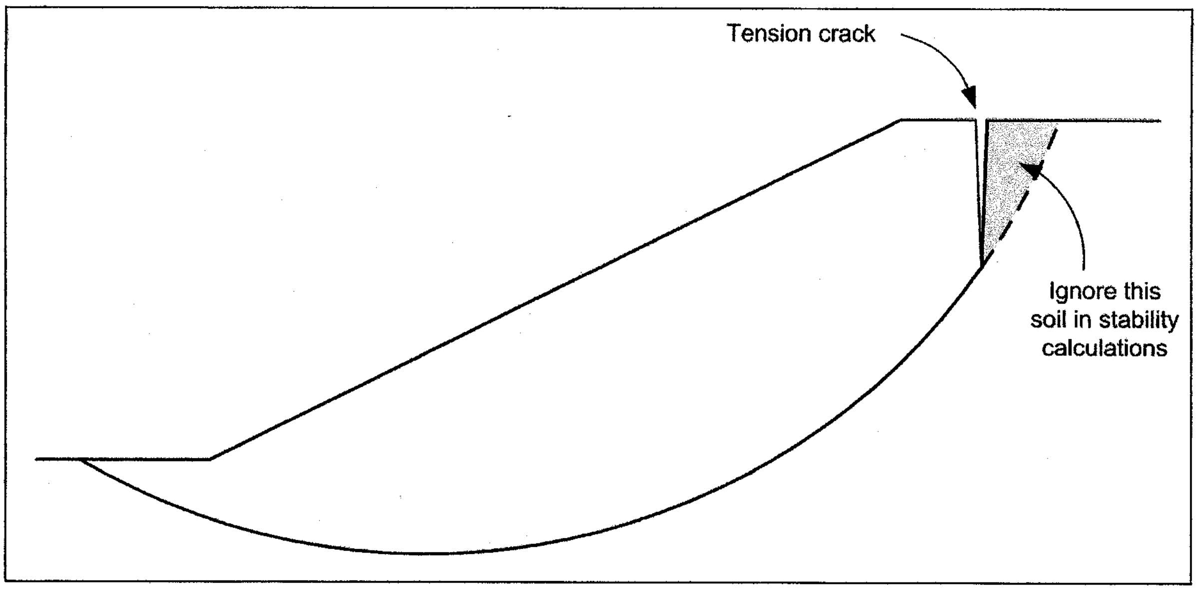

Figure 2-4. Vertical tension crack introduced to avoid tensile stresses in cohesive soils

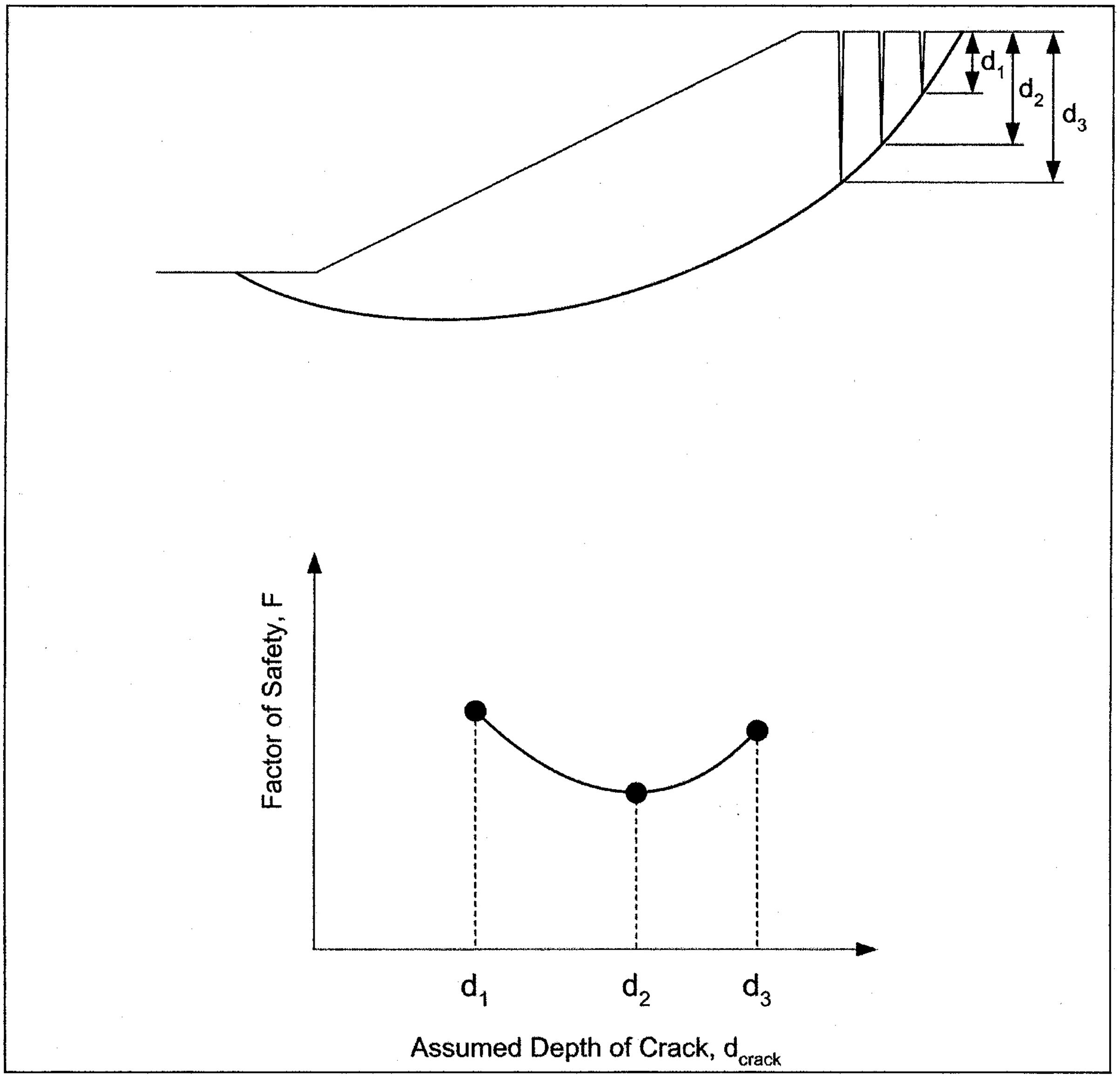

Figure 2-5. Variation in the factor of safety with the assumed depth of vertical crack

where c_D and ϕ_D represent the "developed" cohesion value and friction angle, respectively.

The developed shear strength parameters are expressed by:

$$c_D = \frac{c}{F} \tag{2-5}$$

and

$$\phi_D = \arctan\left(\frac{\tan\phi}{F}\right) \tag{2-6}$$

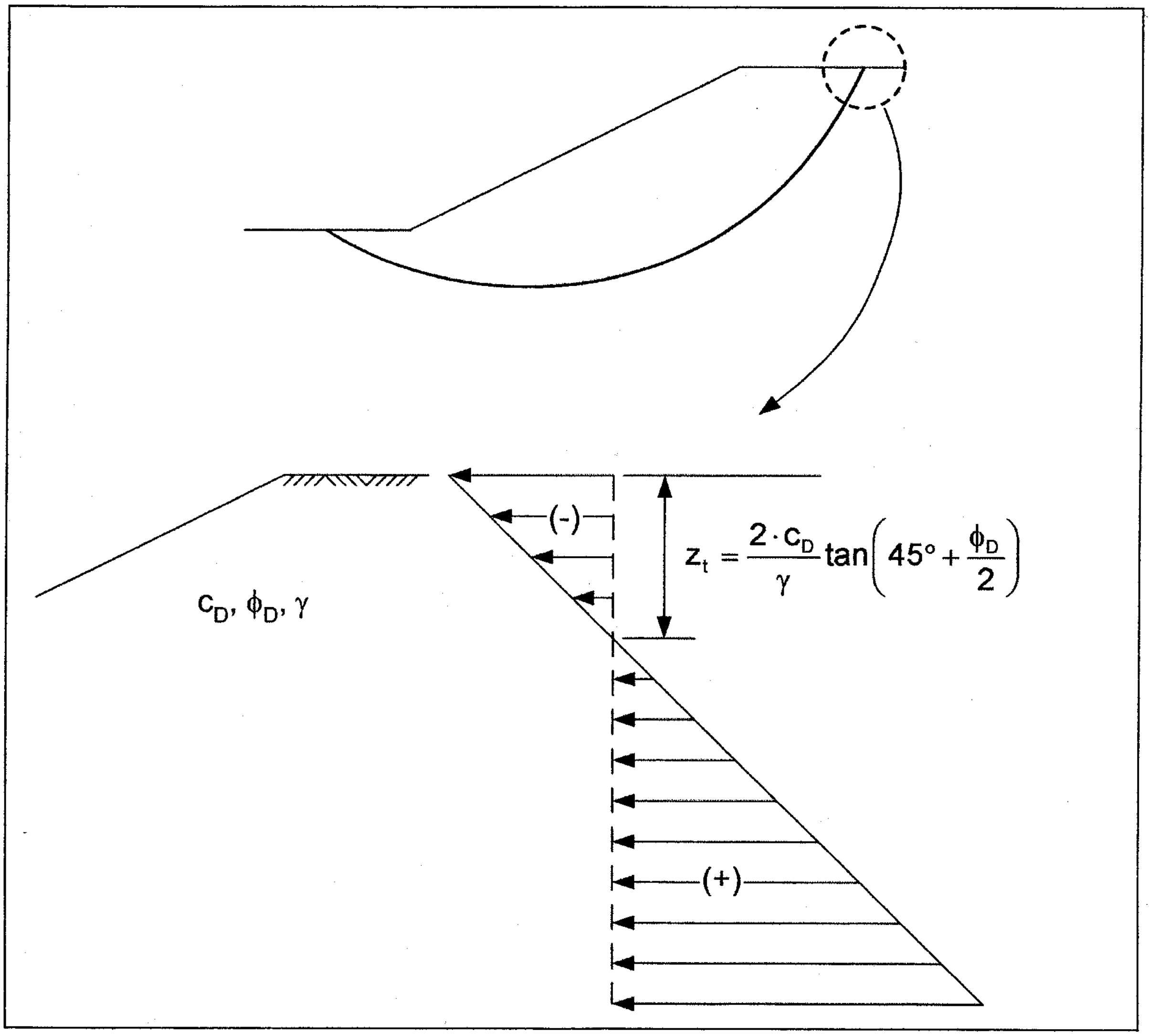

Figure 2-6. Horizontal stresses near the crest of the slope according to Rankine active earth pressure theory

where c, ϕ, and F are cohesion, angle of internal friction, and factor of safety.

In most practical problems, the factor of safety can be estimated with sufficient accuracy to estimate the developed shear strength parameters (c_D and ϕ_D) and the appropriate depth of the tension crack.

(6) For effective stress analyses the depth of the tension crack can also be estimated from Rankine active earth pressure theory. In this case effective stress shear strength parameters, c' and ϕ' are used, with appropriate pore water pressure conditions.

2-3. Analyses of Stability during Construction and at the End of Construction

a. General. Computations of stability during construction and at the end of construction are performed using drained strengths in free-draining materials and undrained strengths in materials that drain slowly. Consolidation analyses can be used to determine what degree of drainage may develop during the construction period. As a rough guideline, materials with values of permeability greater than 10^{-4} cm/sec usually will be fully drained throughout construction. Materials with values of permeability less than 10^{-7} cm/sec usually will be essentially undrained at the end of construction. In cases where appreciable but incomplete drainage is expected during construction, stability should be analyzed assuming fully drained and completely undrained conditions, and the less stable of these conditions should be used as the basis for design. For undrained conditions, pore pressures are governed by several factors, most importantly the degree of saturation of the soil, the density of the soil, and the loads imposed on it. It is conceivable that pore pressures for undrained conditions could be estimated using results of laboratory tests or various empirical rules, but in most cases pore pressures for undrained conditions cannot be estimated accurately. For this reason, undrained conditions are usually analyzed using total stress procedures rather than effective stress procedures.

b. Shear strength properties. During construction and at end of construction, stability is analyzed using drained strengths expressed in terms of effective stresses for free-draining materials and undrained strengths expressed in terms of total stresses for materials that drain slowly.

(1) Staged construction may be necessary for embankments built on soft clay foundations. Consolidated-undrained triaxial tests can be used to determine strengths for partial consolidation during staged construction (Appendix D, Section D-10.)

(2) Strength test specimens should be representative of the soil in the field: for naturally occurring soils, undisturbed samples should be obtained and tested at their natural water contents; for compacted soils, strength test specimens should be compacted to the lowest density, at the highest water content permitted by the specifications, to measure the lowest undrained strength of the material that is consistent with the specifications.

(3) The potential for errors in strengths caused by sampling disturbance should always be considered, particularly when using Q tests in low plasticity soils. Methods to account for disturbances are discussed in Appendix D, Section D-3.

c. Pool levels. In most cases the critical pool level for end of construction stability of the upstream slope is the minimum pool level possible. In some cases, it may be appropriate to consider a higher pool for end-of-construction stability of the downstream slope. (Section 2-4).

d. Pore water pressures. For free-draining materials with strengths expressed in terms of effective stresses, pore water pressures must be determined for analysis of stability during and at the end of construction. These pore water pressures are determined by the water levels within and adjacent to the slope. Pore pressures can be estimated using the following analytical techniques:

(1) Hydrostatic pressure computations for conditions of no flow.

(2) Steady-state seepage analysis techniques such as flow nets or finite element analyses for nonhydrostatic conditions.

For low-permeability soils with strengths expressed in of total stresses, pore water pressures are set to zero for purposes of analysis, as explained in Section 2-2.

2-4. Analyses of Steady-State Seepage Conditions

a. General. Long-term stability computations are performed for conditions that will exist a sufficient length of time after construction for steady-state seepage or hydrostatic conditions to develop. (Hydrostatic conditions are a special case of steady-state seepage, in which there is no flow.) Stability computations are performed using shear strengths expressed in terms of effective stresses, with pore pressures appropriate for the long-term condition.

b. Shear strength properties. By definition, all soils are fully drained in the long-term condition, regardless of their permeability. Long-term conditions are analyzed using drained strengths expressed in terms of effective stress parameters (c' and ϕ').

c. Pool levels. The maximum storage pool (usually the spillway crest elevation) is the maximum water level that can be maintained long enough to produce a steady-state seepage condition. Intermediate pool levels considered in stability analyses should range from none to the maximum storage pool level. Intermediate pool levels are assumed to exist over a period long enough to develop steady-state seepage.

d. Surcharge pool. The surcharge pool is considered a temporary pool, higher than the storage pool, that adds a load to the driving force but often does not persist long enough to establish a steady seepage condition. The stability of the downstream slope should be analyzed at maximum surcharge pool. Analyses of this surcharge pool condition should be performed using drained strengths in the embankment, assuming the extreme possibility of steady-state seepage at the surcharge pool level.

(1) In some cases it may also be appropriate to consider the surcharge pool condition for end of construction (as discussed in Section 2-3), in which case low-permeability materials in the embankment would be assigned undrained strengths.

(2) For all analyses, the tailwater levels should be appropriate for the various pool levels.

e. Pore water pressures. The pore pressures used in the analyses should represent the field conditions of water pressure and steady-state seepage in the long-term condition. Pore pressures for use in the analyses can be estimated from:

(1) Field measurements of pore pressures in existing slopes.

(2) Past experience and judgement.

(3) Hydrostatic pressure computations for conditions of no flow.

(4) Steady-state seepage analyses using such techniques as flow nets or finite element analyses.

2-5. Analyses of Sudden Drawdown Stability

a. General. Sudden drawdown stability computations are performed for conditions occurring when the water level adjacent to the slope is lowered rapidly. For analysis purposes, it is assumed that drawdown is very fast, and no drainage occurs in materials with low permeability; thus the term "sudden" drawdown. Materials with values of permeability greater than 10^{-4} cm/sec can be assumed to drain during drawdown, and drained strengths are used for these materials. Two procedures are presented in Appendix G for computing slope stability for sudden drawdown.

(1) The first is the procedure recommended by Wright and Duncan (1987) and later modified by Duncan, Wright, and Wong (1990). This is the preferred procedure.

(2) The second is the procedure originally presented in the 1970 version of the USACE slope stability manual (EM 1110-2-1902). This procedure is referred to as the USACE 1970 procedure and is described in further detail in Appendix G. Both procedures are believed to be somewhat conservative in that they utilize

Chapter 3
Design Criteria

3-1. General

a. Applicability. This chapter provides guidance for analysis conditions and factors of safety for the design of slopes. Required factors of safety for embankment dams are based on design practice developed and successfully employed by the USACE over several decades. It is imperative that all phases of design be carried out in accord with established USACE methods and procedures to ensure results consistent with successful past practice.

(1) Because of the large number of existing USACE dams and the fact that somewhat different considerations must be applied to existing dams as opposed to new construction, appropriate stability conditions and factors of safety for the analysis of existing dam slopes are discussed as well.

(2) The analysis procedures recommended in this manual are also appropriate for analysis and design of slopes other than earth and rock-fill dams. Guidance is provided for appropriate factors of safety for slopes of other types of embankments, excavated slopes, and natural slopes.

b. Factor of safety guidance. Appropriate factors of safety are required to ensure adequate performance of slopes throughout their design lives. Two of the most important considerations that determine appropriate magnitudes for factor of safety are uncertainties in the conditions being analyzed, including shear strengths and consequences of failure or unacceptable performance.

(1) What is considered an acceptable factor of safety should reflect the differences between new slopes, where stability must be forecast, and existing slopes, where information regarding past slope performance is available. A history free of signs of slope movements provides firm evidence that a slope has been stable under the conditions it has experienced. Conversely, signs of significant movement indicate marginally stable or unstable conditions. In either case, the degree of uncertainty regarding shear strength and piezometric levels can be reduced through back analysis. Therefore, values of factors of safety that are lower than those required for new slopes can often be justified for existing slopes.

(2) Historically, geotechnical engineers have relied upon judgment, precedent, experience, and regulations to select suitable factors of safety for slopes. Reliability analyses can provide important insight into the effects of uncertainties on the results of stability analyses and appropriate factors of safety. However, for design and construction of earth and rock-fill dams, required factors of safety continue to be based on experience. Factors of safety for various types of slopes and analysis conditions are summarized in Table 3-1. These are minimum required factors of safety for new embankment dams. They are advisory for existing dams and other types of slopes.

c. Shear strengths. Shear strengths of fill materials for new construction should be based on tests performed on laboratory compacted specimens. The specimens should be compacted at the highest water content and the lowest density consistent with specifications. Shear strengths of existing fills should be based on the laboratory tests performed for the original design studies if they appear to be reliable, on laboratory tests performed on undisturbed specimens retrieved from the fill, and/or on the results of in situ tests performed in the fill. Shear strengths of natural materials should be based on the results of tests performed on undisturbed specimens, or on the results of in situ tests. Principles of shear strength characterization are summarized in Appendix D.

Table 3-1
Minimum Required Factors of Safety: New Earth and Rock-Fill Dams

Analysis Condition[1]	Required Minimum Factor of Safety	Slope
End-of-Construction (including staged construction)[2]	1.3	Upstream and Downstream
Long-term (Steady seepage, maximum storage pool, spillway crest or top of gates)	1.5	Downstream
Maximum surcharge pool[3]	1.4	Downstream
Rapid drawdown	1.1-1.3[4,5]	Upstream

[1] For earthquake loading, see ER 1110-2-1806 for guidance. An Engineer Circular, "Dynamic Analysis of Embankment Dams," is still in preparation.

[2] For embankments over 50 feet high on soft foundations and for embankments that will be subjected to pool loading during construction, a higher minimum end-of-construction factor of safety may be appropriate.

[3] Pool thrust from maximum surcharge level. Pore pressures are usually taken as those developed under steady-state seepage at maximum storage pool. However, for pervious foundations with no positive cutoff steady-state seepage may develop under maximum surcharge pool.

[4] Factor of safety (FS) to be used with improved method of analysis described in Appendix G.

[5] FS = 1.1 applies to drawdown from maximum surcharge pool; FS = 1.3 applies to drawdown from maximum storage pool.

For dams used in pump storage schemes or similar applications where rapid drawdown is a routine operating condition, higher factors of safety, e.g., 1.4-1.5, are appropriate. If consequences of an upstream failure are great, such as blockage of the outlet works resulting in a potential catastrophic failure, higher factors of safety should be considered.

(1) During construction of embankments, materials should be examined to ensure that they are consistent with the materials on which the design was based. Records of compaction, moisture, and density for fill materials should be compared with the compaction conditions on which the undrained shear strengths used in stability analyses were based.

(2) Particular attention should be given to determining if field compaction moisture contents of cohesive materials are significantly higher or dry unit weights are significantly lower than values on which design strengths were based. If so, undrained (UU, Q) shear strengths may be lower than the values used for design, and end-of-construction stability should be reevaluated. Undisturbed samples of cohesive materials should be taken during construction and unconsolidated-undrained (UU, Q) tests should be performed to verify end-of-construction stability.

d. Pore water pressure. Seepage analyses (flow nets or numerical analyses) should be performed to estimate pore water pressures for use in long-term stability computations. During operation of the reservoir, especially during initial filling and as each new record pool is experienced, an appropriate monitoring and evaluation program must be carried out. This is imperative to identify unexpected seepage conditions, abnormally high piezometric levels, and unexpected deformations or rates of deformations. As the reservoir is brought up and as higher pools are experienced, trends of piezometric levels versus reservoir stage can be used to project piezometric levels for maximum storage and maximum surcharge pool levels. This allows comparison of anticipated actual performance to the piezometric levels assumed during original design studies and analysis. These projections provide a firm basis to assess the stability of the downstream slope of the dam for future maximum loading conditions. If this process indicates that pore water pressures will be higher than those used in design stability analyses, additional analyses should be performed to verify long-term stability.

e. Loads on slopes. Loads imposed on slopes, such as those resulting from structures, vehicles, stored materials, etc. should be accounted for in stability analyses.

Chapter 4
Calculations and Presentations

4-1. Analysis Methods

a. Selection of suitable methods of analysis. The methods of analysis (computer program, charts, hand calculations) should be selected according to the complexity of the site or job and the data available to define the site conditions.

(1) Use of a reliable and verified slope stability analysis computer program is recommended for performing slope stability analyses where conditions are complex, where significant amounts of data are available, and where possible consequences of failure are significant. Computer programs provide a means for efficient and rapid detailed analysis of a wide variety of slope geometry and load conditions.

(2) Slope stability charts are relatively simple to use and are available for analysis of a variety of short-term and long-term conditions. Appendix E contains several different types of slope stability charts and guidance for their use.

(3) Spreadsheet analyses can be used to verify results of detailed computer analyses.

(4) Graphical (force polygon) analyses can also be used to verify results of computer analyses.

b. Verification of analysis method. Verification of the results of stability analyses by independent means is essential. Analyses should be performed using more than one method, or more than one computer program, in a manner that involves independent processing of the required information and data insofar as practical, to verify as many aspects of the analysis as possible. Many slope stability analyses are performed using computer programs. Selection and verification of suitable software for slope stability analysis is of prime importance. It is essential that the software used for analysis be tested and verified, and the verification process should be described in the applicable design and analysis memoranda (geotechnical report). Thorough verification of computer programs can be achieved by analyzing benchmark slope stability problems. Benchmark problems are discussed by Edris, Munger, and Brown (1992) and Edris and Wright (1992).

4-2. Verification of Computer Analyses and Results

a. General. All reports, except reconnaissance phase reports, that deal with critical embankments or slopes should include verification of the results of computer analyses. The verification should be commensurate with the level of risk associated with the structure and should include one or more of the following methods of analysis using:

(1) Graphical (force polygon) method.

(2) Spreadsheet calculations.

(3) Another slope stability computer program.

(4) Slope stability charts.

The historical U.S. Army Corps of Engineers' approach to verification of any computer analysis was to perform hand calculations (force polygon solution) of at least a simplified version of the problem. It was acceptable to simplify the problem by using fewer slices, by averaging unit weights of soil layers, and by simplifying the piezometric conditions. While verification of stability analysis results is still required, it is no longer required that results be verified using graphical hand calculations. Stability analysis results can be verified using any of the methods listed above. Examples of verifications of analyses performed using Spencer's Method, the Simplified Bishop Method, and the Modified Swedish Method are shown in Figures 4-1, 4-2, 4-3, and 4-4.

b. Verification using a second computer program. For difficult and complex problems, a practical method of verifying or confirming computer results may be by the use of a second computer program. It is desirable that the verification analyses be performed by different personnel, to minimize the likelihood of repeating data entry errors.

c. Software versions. Under most Microsoft Windows™ operating systems, the file properties, including version, size, date of creation, and date of modification can be reviewed to ensure that the correct version of the computer program is being used. Also, the size of the computer program file on disk can be compared with the size of the original file to ensure that the software has not been modified since it was verified. In addition, printed output may show version information and modification dates. These types of information can be useful to establish that the version of the software being used is the correct and most recent version available.

d. Essential requirements for appropriate use of computer programs. A thorough knowledge of the capabilities of the software and knowledge of the theory of limit equilibrium slope stability analysis methods will allow the user to determine if the software available is appropriate for the problem being analyzed.

(1) To verify that data are input correctly, a cross section of the problem being analyzed should be drawn to scale and include all the required data. The input data should be checked against the drawing to ensure the data in the input file are correct. Examining graphical displays generated from input data is an effective method of checking data input.

(2) The computed output should be checked to ensure that results are reasonable and consistent. Important items to check include the weights of slices, shear strength properties, and pore water pressures at the bottoms of slices. The user should be able to determine if the critical slip surface is going through the material it should. For automatic searches, the output should designate the most critical slip surface, as well as what other slip surfaces were analyzed during the search. Checking this information thoroughly will allow the user to determine that the problem being analyzed was properly entered into the computer and the software is correctly analyzing the problem.

e. Automatic search verification. Automatic searches can be performed for circular or noncircular slip surfaces. The automatic search procedures used in computer programs are designed to aid the user in locating the most critical slip surface corresponding to a minimum factor of safety. However, considerable judgment must be exercised to ensure that the most critical slip surface has actually been located. More than one local minimum may exist, and the user should use multiple searches to ensure that the global minimum factor of safety has been found.

(1) Searches with circular slip surfaces. Various methods can used to locate the most critical circular slip surfaces in slopes. Regardless of the method used, the user should be aware of the assumptions and limitations in the search method.

Appendix D
Shear Strength Characterization

D-1. Introduction

Selection of shear strength for use in slope stability computations is covered in this chapter. Shear strengths are usually determined from laboratory tests performed on specimens prepared by compaction in the laboratory or undisturbed samples obtained from exploratory soil borings. The laboratory test data may be supplemented with in situ field tests and correlations between shear strength parameters and other soil properties such as grain size, plasticity, and Standard Penetration Resistance (N) values. This chapter focuses primarily on shear strength selection from laboratory test data.

D-2. Definition of Shear Strength

a. Shear strength for all of the slope stability analyses described in this manual is represented by a Mohr-Coulomb failure envelope that relates shear strength to either total or effective normal stress on the failure plane (Figure D-1). In the case of total stresses, the shear strength is expressed as:

$$s = c + \sigma \tan \phi \qquad \text{(D-1)}$$

where

c and ϕ = cohesion intercept and friction angle for the failure envelope

σ = total normal stress on the failure plane

For effective stresses the shear strength is expresses as:

$$s = c' + (\sigma - u) \tan \phi' \qquad \text{(D-2)}$$

where

c' and ϕ' = intercept and slope angle for the failure envelope plotted in terms of effective stresses

σ and u = total normal stress and pore water pressure, respectively, on the failure plane

The shear strength parameters, c and ϕ or c' and ϕ', are determined from laboratory shear test data. The stresses from each test representing failure are plotted and a suitable failure envelope is drawn. The specific way in which the data are plotted, selection of the point representing failure (failure criteria), and whether effective or total stresses are used to plot the data depend on the type of test, loading conditions, and several other factors which are covered in the following sections of this appendix.

b. Theoretically the failure envelope is tangent to all of the Mohr's circles representing the stresses at failure (Figure D-2a). However, in actual practice there will be variations among samples tested, such that the failure envelope represents a "best-fit" to the data from several tests (Figure D-2b). Also, when the failure envelope is derived from direct shear tests, the complete state of stress is not known -- only the stresses on the horizontal plane are known. The horizontal plane is assumed to be the failure plane, and the failure envelope is drawn through the series of points representing the values of τ and σ on the horizontal plane from each test.

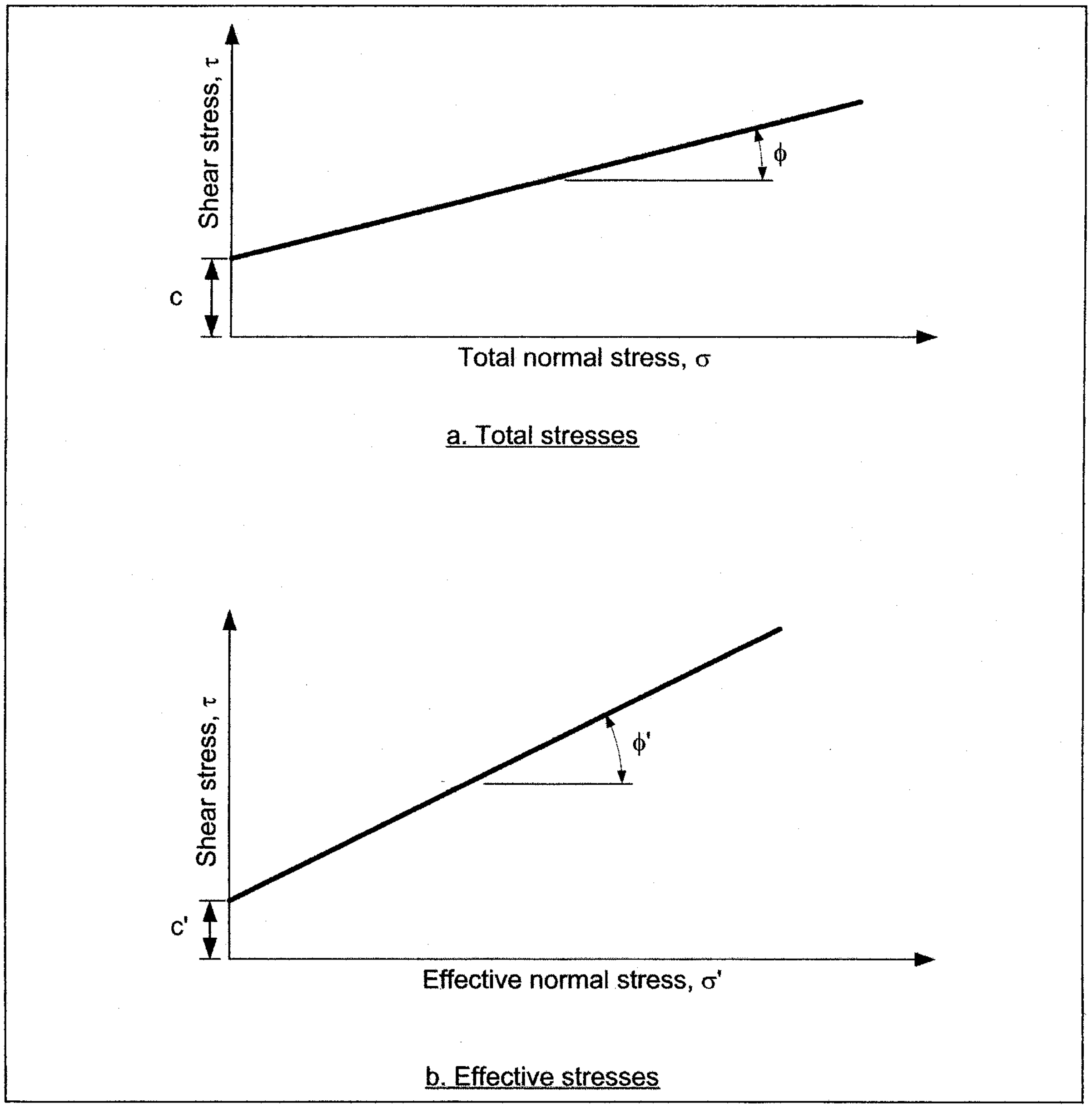

Figure D-1. Failure envelopes for total and effective stresses

c. Sometimes failure envelopes are curved, as shown in Figure D-3. Examples include the failure envelope obtained from Unconsolidated-Undrained tests on compacted soils and the residual shear strength envelope determined from consolidated-drained shear tests. In these cases the appropriate curved envelope, as illustrated in Figure D-3, is determined and used in the stability analyses, rather than values of cohesion and friction angle.

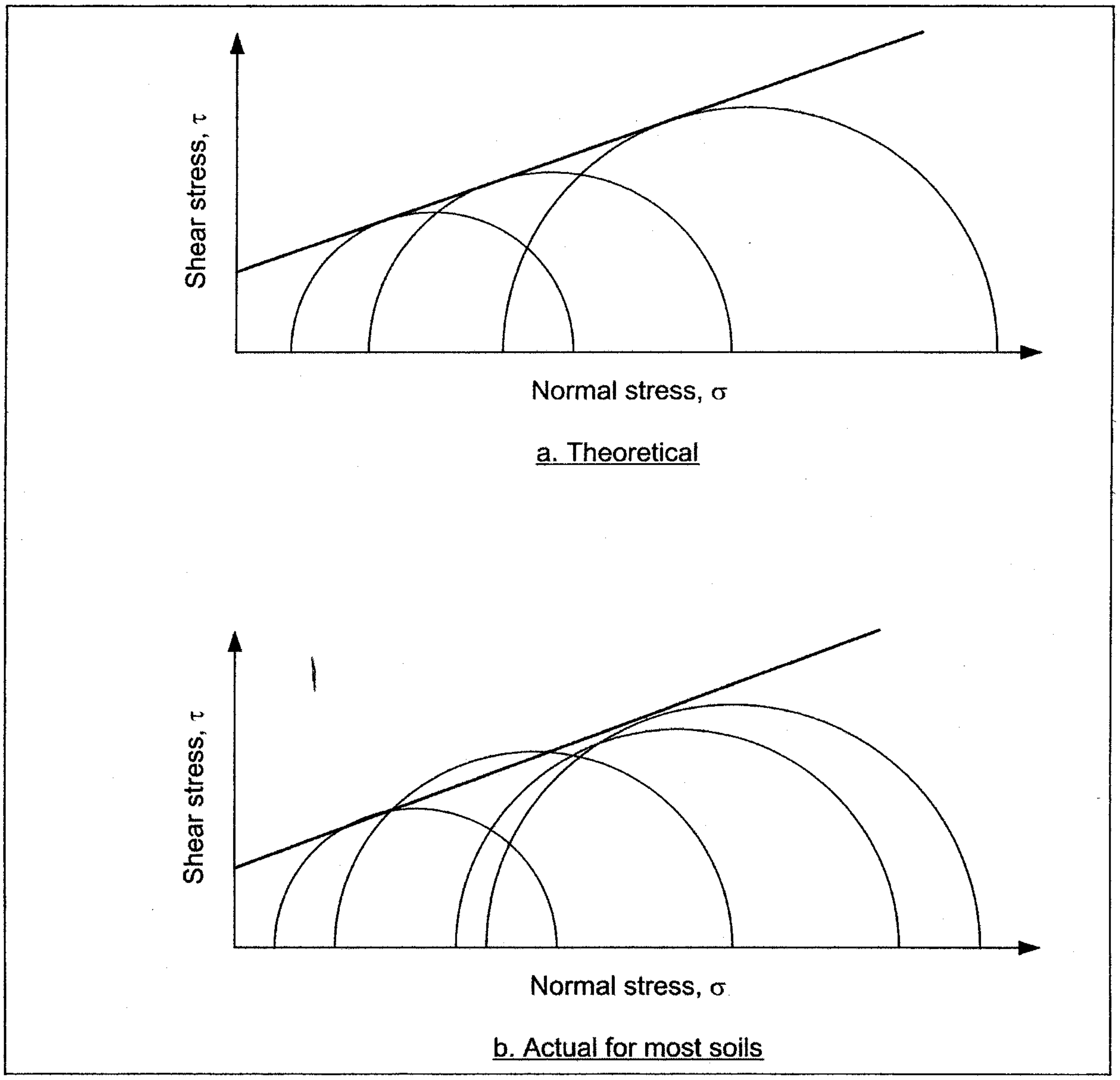

Figure D-2. Mohr's circles and failure envelopes

D-3. Types of Laboratory Strength Test Procedures

a. Most laboratory tests are performed using either triaxial compression or direct shear test equipment. A two-stage loading procedure is used in each of these tests. In the first stage, a confining stress is applied. In the triaxial test, the confining stress is applied by increasing the cell, or all-round pressure on the sample. In the direct shear test, the confining pressure is applied by applying a vertical load to the horizontal plane, which becomes the eventual failure plane. The normal stress on the vertical plane in the direct shear device increases when the stress is applied to the horizontal plane, but the stress on the vertical plan is not known.

(1) The second stage of a strength test involves shearing the specimen. In the triaxial test, the axial load is gradually increased (load control), or the specimen is deformed slowly in the axial direction and the axial load is measured (deformation control), to shear the specimen. In the direct shear test, the horizontal shear

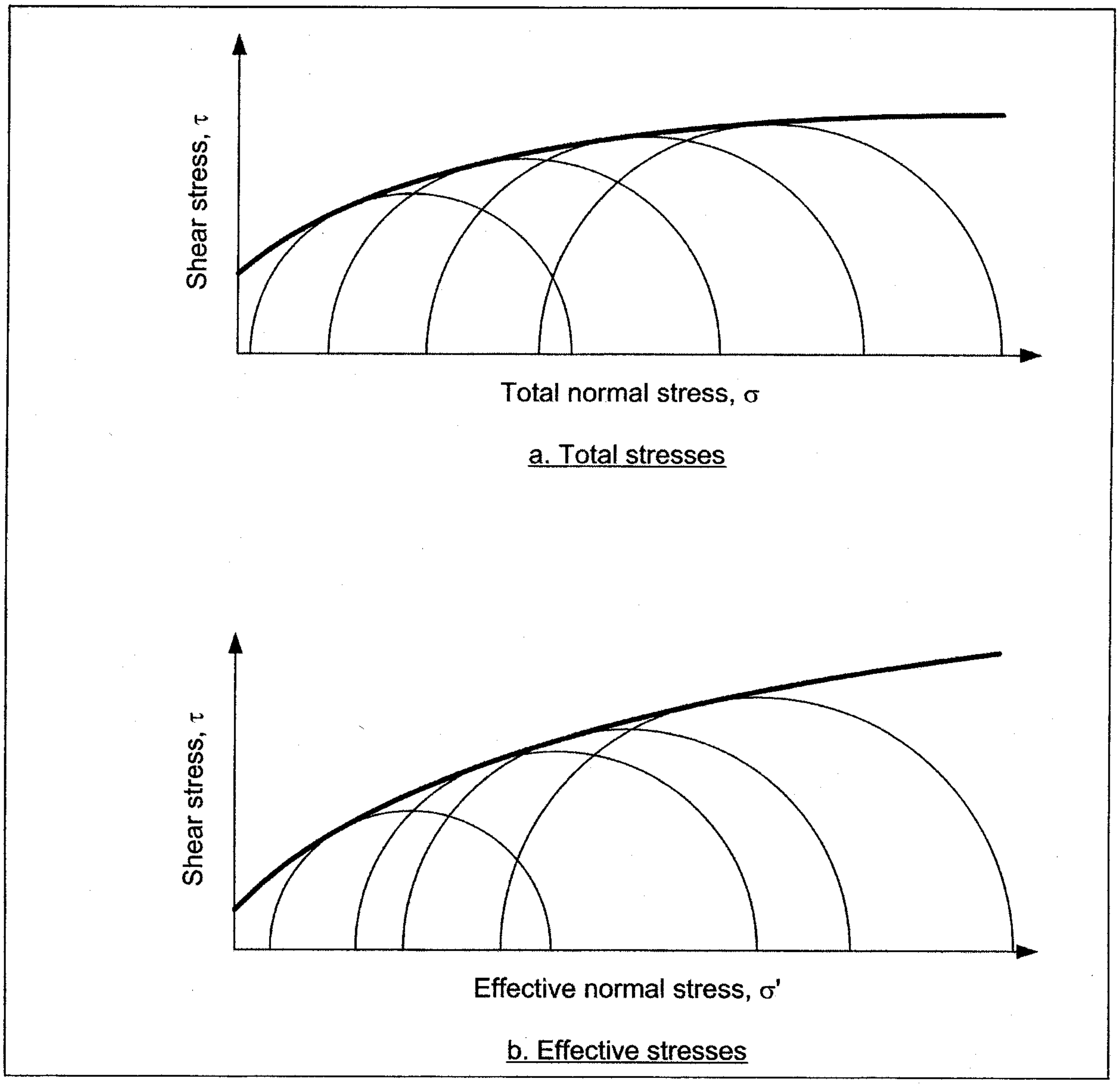

Figure D-3. Curved strength envelopes

load is gradually increased (load control) or the specimen is deformed horizontally by displacing the upper half of the shear box horizontally relative to the lower half and measuring the resulting load (deformation control).

(2) In the triaxial test, drainage of water into or out of the specimen can be controlled during application of both the confining stress and the shear stress. Depending on the drainage allowed in these phases of the test, three different types of test are possible -- Unconsolidated-Undrained (UU or Q), Consolidated-Undrained (CU or R), and Consolidated-Drained (CD or S). The three loading procedures are illustrated in Figure D-4. The loading procedures are intended in part to simulate conditions of loading and drainage in the field. The loading condition used in the laboratory test depends on the stability condition that strengths are being measured for, e.g., end-of-construction, steady-state seepage, or rapid drawdown.

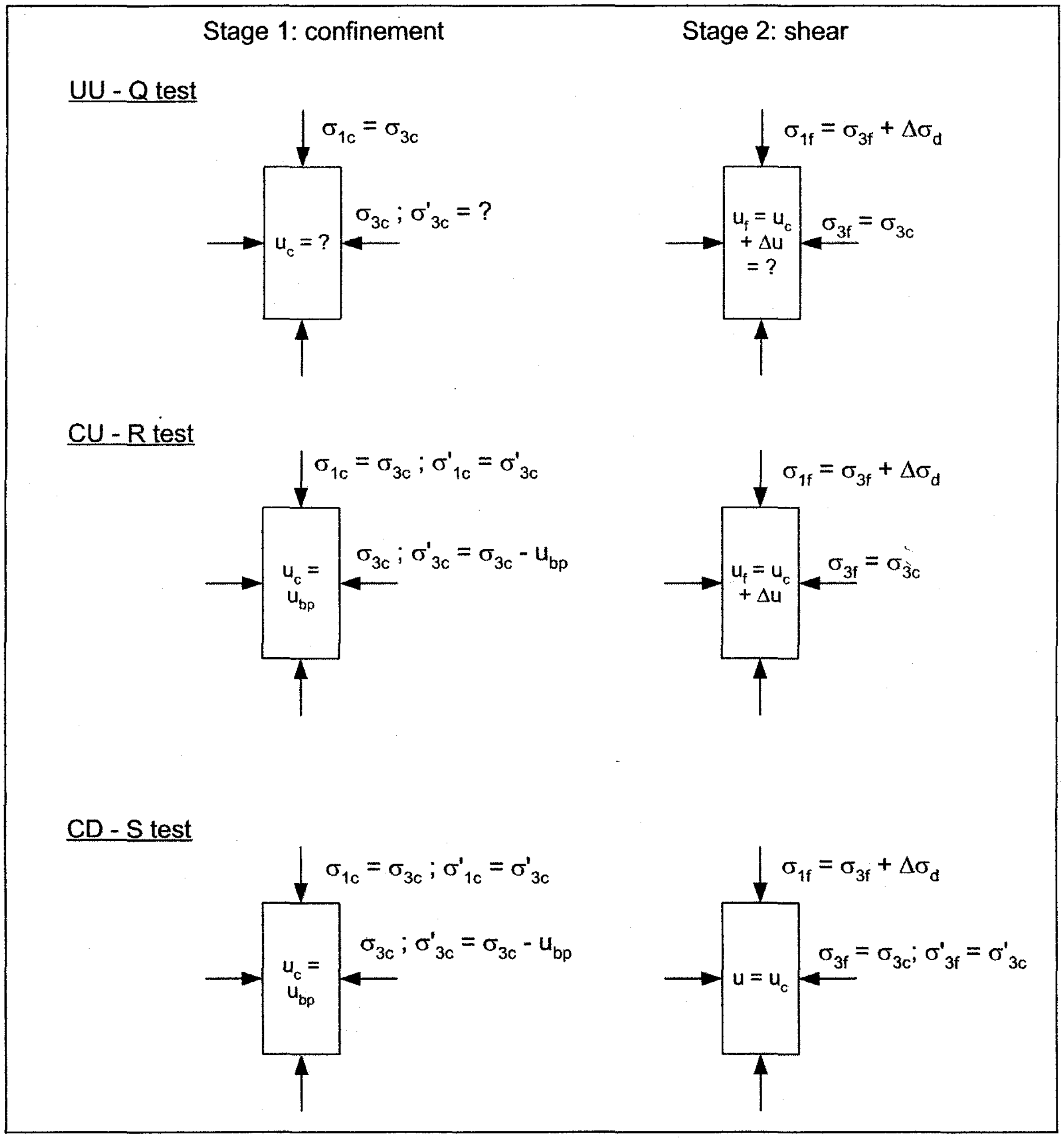

Figure D-4. Three types of triaxial shear tests

(3) In the direct shear test, drainage cannot be prevented. Thus, only the consolidated-drained (CD or S) loading procedure can be used. All three loading procedures (UU or Q, CU or R, and CD or S) are discussed in the following text.

b. Unconsolidated-Undrained (UU or Q) test procedure. No drainage is allowed in a UU test during the application of either the confining pressure or shear stress. The confining pressure is applied and the specimen is sheared shortly afterward. Rates of loading are relatively fast, and need only be slow enough to avoid inertial effects and allow adequate time for recording data. Typical rates of loading for shear produce failure of the specimen in 10 to 20 minutes.

(1) The Unconsolidated-Undrained test procedure is used to measure the shear strength where there will be no drainage (no change in water content) when the soil is loaded. The objective of this test procedure is to measure the shear strength of the soil at the same water content that the soil will have in the field. It is important that the specimens being tested have the proper water content, which represents the field condition. Specimens of natural soils must be at the field water content and not allowed to dry out or absorb water between the time they are sampled and the time they are tested. Compacted specimens should be prepared at moisture content representing expected field conditions. The shear strength of compacted clays decrease with decreasing dry density and increasing water content. Test specimens should be compacted to the lowest dry density and highest water content that will be permitted under the specification, to ensure that the strength in the field will not be lower than that measured in the laboratory tests.

(2) Results from tests performed using Unconsolidated-Undrained loading procedures are always plotted using total stresses. Thus, the shear strength is expressed in terms of total stress, using c and ϕ. Pore water pressures are not measured and are unknown.

c. Consolidated-Undrained (CU or R) test procedure. The Consolidated-Undrained test is used for several purposes and, depending on the purpose, pore water pressure may or may not be measured during shear. Each stage of the loading procedure is described further below, followed by discussion of how the test data are used.

(1) Consolidation stage. The first stage of Consolidated-Undrained loading is the consolidation stage, where the confining pressure is applied and the specimen is given time to consolidate fully. During this stage, the specimen is also saturated using back-pressure saturation techniques. Back-pressure saturation is done by increasing both the total confining stress and pore water pressure in equal increments until the specimen is saturated. Each increment of confining pressure is normally allowed to remain for some time to permit water to flow into the specimen and air to dissolve into the pore water. Increments should be small enough, and equilibration times long enough to avoid the specimen's being subjected to undesirably high effective stresses during back-pressure saturation. Until the specimen is saturated, increasing the confining pressure causes the effective stress to increase during the time required for the internal pore water pressures to equilibrate with the back pressure. It is desirable to back-pressure saturate specimens before consolidation to the final test confining pressures. In that way, reliable volume changes can be measured during consolidation by measuring the amount of water that flows into or out of the specimen as they consolidate. The volume changes should be recorded at suitable time intervals and plotted versus time to determine when consolidation is completed. The volume change-time data are also used to estimate the required times for shearing the specimen, as described in Section D-3.c(2) below.

(2) Shear stage. Once consolidation is complete, the drainage valve is closed to prevent further drainage as the axial load is increased to shear the specimen. In most Consolidated-Undrained shear tests, the pore water pressures developed during this stage of the test are measured. Pore water pressures are usually measured by measuring the water pressure in porous stones or disks at one or both ends of the specimen. Because of the end restraint at the two ends of the specimen, the strains in the specimen will not be uniform and, thus, the induced pore water pressures will not be entirely uniform over the height of the specimen. In order to measure representative values of the pore water pressures, the specimen should be sheared slowly enough for pore water pressures to equalize over the height of the specimen. To measure representative values of the pore water pressures, the specimen should be sheared slowly enough for pore water pressures to equalize over the height of the specimen. Suitable loading rates can be calculated from the time-volume change data recorded during the consolidation phase of the test. Axial load, axial deformation, and pore water pressure readings should be taken during shear. Specimens may be sheared using load control or deformation

control to increase the axial load on the specimens. Either method is adequate for measuring the peak load that the specimen can withstand. However, to measure the soil resistance beyond the point where the peak load is reached, it is necessary to control the deformation rate and measure the load. This is especially important for normally and slightly overconsolidated clays and some loose sands, where the peak effective stress shear strength parameters (c' and ϕ') may be developed at strains larger than the strains at which the peak load in undrained shear is reached.

(3) Use of data. Shear strength data from Consolidated-Undrained tests are used in four different ways for slope stability computations:

- To determine the effective stress shear strength parameters for long-term, steady-state seepage analyses.
- To determine the relationship between undrained shear strength and effective consolidation pressure (τ_{ff} vs. σ'_{fc}) for analyses of rapid drawdown.
- To estimate undrained-shear strengths and reduce effects of sample disturbance for end-of-construction stability analyses.
- To estimate undrained shear strength for analyses of staged construction of embankments.

These uses are each discussed separately below.

(a) By plotting the effective stresses at failure from Consolidated-Undrained tests, the Mohr-Coulomb failure envelope for effective stresses (c' and ϕ') can be determined. The failure envelope for effective stresses from Consolidated-Undrained tests is, for practical purposes, the same as the failure envelope from consolidated-drained (CD or S) tests. The failure envelope from either test can be used in slope stability computations for the long-term, steady-state seepage condition. Consolidated-Undrained (CU or R) tests are usually preferred over consolidated-drained (CD or S) tests for determining the effective stress failure envelope for clays, because Consolidated-Undrained tests can be performed more quickly. The time required for nonuniform pore water pressures to equalize in the specimen in a Consolidated-Undrained test is less than the time required for a specimen to fully drain during shear in a consolidated-drained test.

(b) Results of Consolidated-Undrained shear tests are also used to relate undrained shear strength to effective consolidation pressure for use in stability analyses for rapid drawdown. Further discussion of the plotting and use of the data for rapid drawdown analyses is presented in Appendix G.

(c) Data from Consolidated-Undrained shear tests can be used to estimate the undrained shear strength of saturated soils for use in analyses for end-of-construction stability. By reconsolidating specimens in the laboratory, it is possible to reduce some of the effects of sample disturbance. However, care must be used to avoid increasing the strength, and overestimating the undrained shear strength. When Consolidated-Undrained shear test procedures are used to estimate undrained shear strength the undrained shear strength is expressed as $S_u = (\sigma_1 - \sigma_3)/2$ and is related to the effective consolidation pressure. Two approaches may be used to do this. One approach is the SHANSEP approach suggested by Ladd and Foott (1974);[1] the other is the "recompression" technique suggested by Bjerrum (1973). These are explained more fully below. The SHANSEP procedure suggested by Ladd and Foott (1974) involves the following steps:

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[1] Reference information is presented in Appendix A.

- Step 1: The variations of the present effective vertical stress and the maximum past pressure with depth are established. The present vertical effective stress is the current effective overburden pressure, calculated using unit weight and the groundwater level. The maximum past pressure is determined from consolidation tests on high-quality, undisturbed test specimens.

- Step 2: Consolidated-Undrained tests are performed using consolidation pressures that are higher than the maximum past pressure. If the clay is overconsolidated, the test specimens are allowed to swell after consolidation to achieve a suitable range of overconsolidation ratios, encompassing the range of values in the field. The test results are used to establish a relationship between the normalized shear strength [$S_u/\sigma'_{vc} = ½ (\sigma_1-\sigma_3)/\sigma'_{vc}$] and overconsolidation ratio.

- Step 3: Undrained shear strengths applicable to the field are estimated by multiplying the normalized strength, S_u/σ'_{vc}, determined in Step 2, by the effective vertical stress, σ'_{vc}, determined in Step 1.

Once undrained shear strengths are determined in the manner described, they are represented in the slope stability computations as cohesion values, c, with $\phi = 0$.

- The SHANSEP procedure removes some of the effects of sample disturbance but also alters the structure of the soil. Alteration of the structure can lead to values of shear strength that are not representative of those in the field. This approach is not recommended for heavily overconsolidated soils, or for soils that have distinct structure or cementation bonds.

- The "recompression" technique proposed by Bjerrum (1973) involves reconsolidating specimens to the same effective stress that the specimens currently experience in the field. Although this approach results in specimens that have somewhat lower water contents than in the field (and therefore higher shear strengths), it produces less change in the soil structure than the SHANSEP approach. The recompression technique is recommended over the SHANSEP approach for heavily overconsolidated soils, but the approach may overestimate the shear strength even for these soils and should be used cautiously.

- Further details of the SHANSEP and "recompression" procedures can be found in the cited references. The shear strength obtained from Consolidated-Undrained tests using either the SHANSEP or recompression technique is assigned as a cohesion value with ϕ = zero (these techniques apply only to saturated soils). The advantage of using Consolidated-Undrained tests, rather than Unconsolidated-Undrained tests to estimate the undrained shear strength is that some of the effects of sample disturbance can be reduced. However, care must be exercised to ensure that strengths are not overestimated. In addition, the offsetting effects of such factors as anisotropy and creep may need to be accounted for if the effects of sample disturbance are eliminated.

(d) The fourth use of Consolidated-Undrained shear tests is to measure shear strengths for use in analyses of staged construction of embankments. This is discussed in Section D-10.

d. Consolidated-Drained (CD or S). Complete drainage is allowed during the application of both the confining pressure and shear for consolidated-drained loading. Either the triaxial or direct shear apparatus may be used for testing. The two stages of loading (consolidation and shear) are described separately below, followed by a discussion of how the test data are used.

(1) Consolidation stage. The consolidation stage for consolidated-drained (CD or S) loading procedures is the same as the consolidation stage for Consolidation-Undrained (CU or R) test procedure. Back-pressure saturation is used to ensure complete saturation and to allow accurate measurements of volume change during

both consolidation and shear, by measuring the amount of water that flows into or out of the specimen. Volume change-time data from the consolidation phase of the test are used to estimate rates of loading and times to failure for the shear phase. Saturation is also important to eliminate effects of capillary stresses that would influence the strength measurements and their interpretation if the soil is partly saturated. The results of Consolidated-Drained shear tests are plotted using effective stresses that are equal to the total stresses minus the measured pore water pressures. One of the advantages of triaxial tests over direct shear test is that the specimen can be back-pressure saturated. Direct shear devices should only be used for testing soils which are either already saturated or will become saturated once placed in the direct shear apparatus and submerged.

(2) Shear stage. Specimens are sheared by slowly increasing either the axial load, in the case of triaxial tests, or the horizontal shear load, in the case of direct shear tests. It is very important to shear the specimen slowly enough that the soil can completely drain and no excess pore water pressures are developed. For some heavily overconsolidated clays and clay shales, the loading rates may need to be so slow that failure is reached only after several days or even weeks of shear. Suitable rates of loading to achieve complete drainage can be estimated from the volume change-time data recorded during the consolidation phase of the test. During triaxial shear the axial load, axial deformation, and volume changes of the specimen are recorded at various time intervals. The axial load-deformation data are used to determine the point where the specimen has failed, while the volume change information is used to make corrections for the change in cross-sectional area of the specimen that occurs during shear.

(3) Usage. Consolidated-Drained loading procedures are used to determine the effective stress shear strength parameters of freely draining soils. These soils will drain with relatively short testing times and the consolidated-drained loading procedure comes closest to representing the loading for long-term, drained conditions in the field. Consolidated-Drained tests procedures are also used to measure the residual shear strength of clays using direct shear or torsional shear equipment. The direct shear and torsional shear equipment allow for large strains, like those needed to measure residual shear strengths. When residual shear strengths are not needed and the soils are fine-grained, triaxial tests using Consolidated-Undrained (CU or R) procedures with pore water pressure measurements are preferred over Consolidated-Drained procedures because of the shorter testing times required for CU tests.

D-4. "Modified" Mohr-Coulomb Diagrams

Because the complete state of stress is known in the triaxial test, Mohr's circles of stress can be plotted on a Mohr diagram. However, it can be difficult to judge what is a "best-fit" line tangent to a number of circles. A more convenient techniques is to plot the data on a "modified" Mohr-Coulomb diagram where, rather than plotting circles, a point is plotted to represent the stresses at failure in each test. The diagrams used for this purpose are called "modified" Mohr-Coulomb diagrams. Several different forms of modified Mohr-Coulomb diagrams can be used. All modified Mohr-Coulomb diagrams are based on the fundamental relationship between the principal stresses and the Mohr-Coulomb shear strength parameters, c' and ϕ' or c and ϕ. Referring to Figure D-5 and the triangle formed by points, *def*, the following expression can be written:

$$\sin\,\phi = \frac{\dfrac{(\sigma_1 - \sigma_3)}{2}}{\dfrac{c}{\tan\,\phi} + \dfrac{(\sigma_1 + \sigma_3)}{2}} \qquad \text{(D-3)}$$

Equation D-3 can be rearranged to obtain a number of different relationships between the principal stresses and the shear strength parameters, c and ϕ. Two of the most useful forms of Equation D-3 and the resulting modified Mohr-Coulomb diagrams are described in the following text.

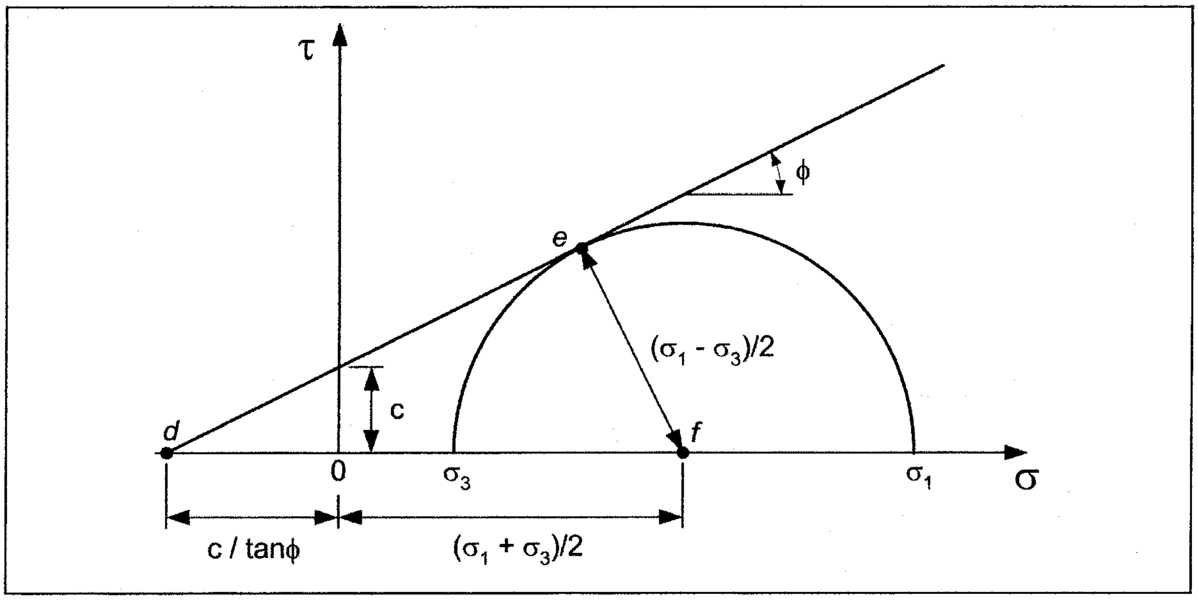


Figure D-5. Mohr's circle of stress used to derive equations for Modified Mohr-Coulomb diagram

a. "p-q" diagrams. One of the most commonly used modified Mohr-Coulomb diagrams is a "p-q" diagram. For this diagram, $q = (\sigma_1 - \sigma_3)/2$ is plotted vs. $p = (\sigma_1 + \sigma_3)/2$, as shown in Figure D-6a. The basis for such a plot can be seen by rewriting Equation D-3 in the form:

$$q = c \cos \phi + p \sin \phi \tag{D-4}$$

which can also be written as:

$$q = d + p \tan \psi \tag{D-5}$$

Equation D-5 expresses a linear relationship between the quantities q and p. The parameter d is the intercept and tan Ψ is the slope of the line on the modified Mohr-Coulomb diagram shown in Figure D-6a. The slope, tan Ψ, is related to the friction angle, φ, by the expression:

$$\tan \psi = \sin \phi \tag{D-6}$$

or

$$\phi = \arcsin (\tan \psi) \tag{D-7}$$

Similarly, the cohesion on a Mohr-Coulomb diagram is related to the friction angle (φ) and the intercept (d) on a modified Mohr-Coulomb diagram by:

$$c = \frac{d}{\cos \phi} \tag{D-8}$$

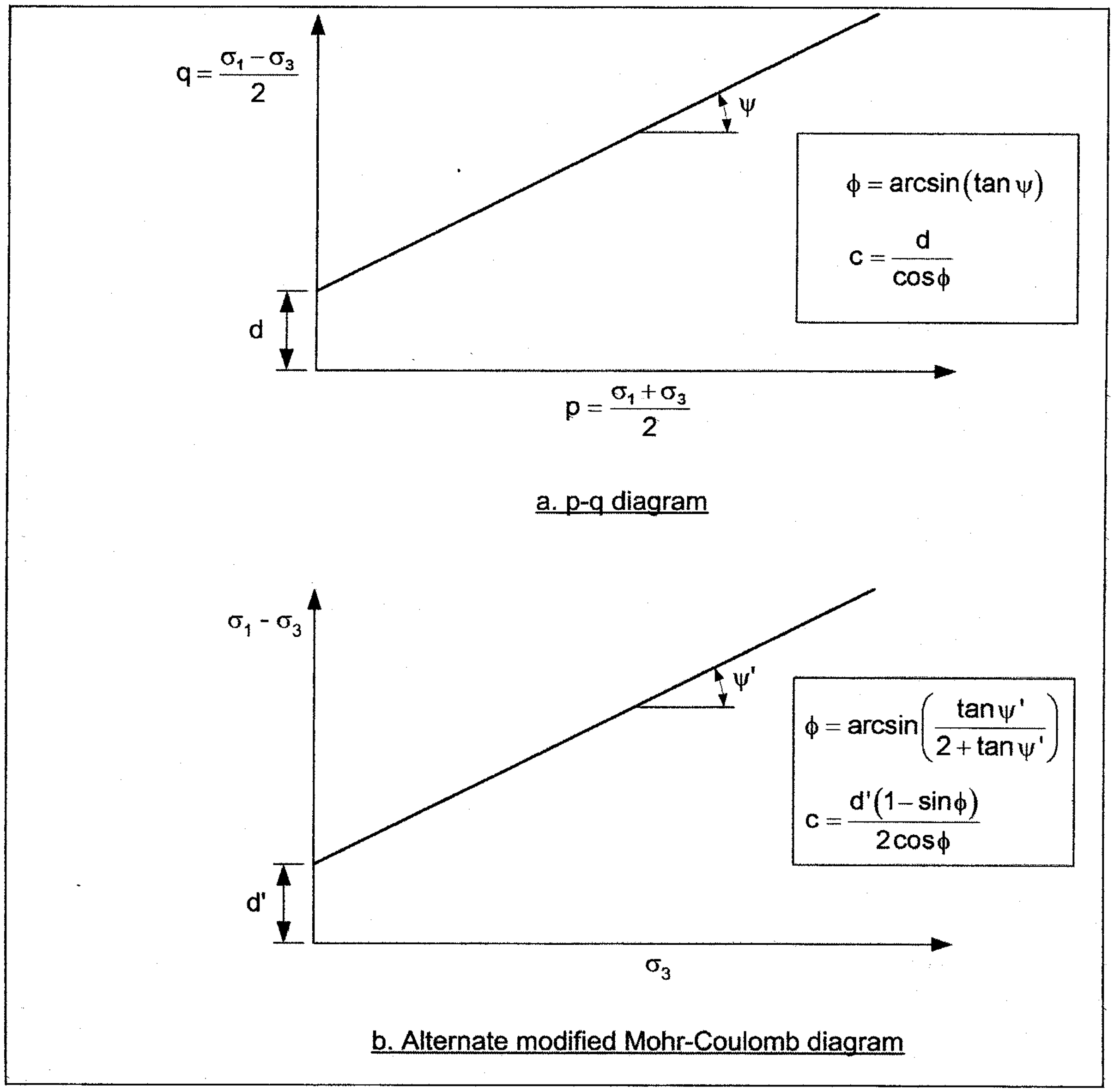


Figure D-6. Modified Mohr-Coulomb diagrams

A p-q diagram can be used to plot the results of triaxial shear tests and determine the Mohr-Coulomb shear strength parameters. To do so, the values of $q = (\sigma_1 - \sigma_3)$ and $p = (\sigma_1 + \sigma_3)$ at failure are determined for each test and plotted on the diagram. A straight line is then drawn to fit the data and the slope ($\tan \Psi$) and intercept (d) are determined. Once the intercept and slope are found, Equations D-7 and D-8 are used to calculate the friction angle, ϕ, and cohesion, c. Care must be exercised in presenting "p-q" diagrams and reviewing such diagrams prepared by others, because an entirely different set of axes and quantities from the ones described above are sometimes used and referred to as "p-q" diagrams. The alternative nomenclature defines p and q for triaxial compression tests as follows:

$$p = \frac{1}{3}(\sigma_1 + 2\sigma_3) \qquad \text{(D-9)}$$

and

$$q = (\sigma_1 - \sigma_3) \tag{D-10}$$

This notation was suggested by Roscoe, Schofield, and Wroth (1958) and has appeared in a number of texts and reference books, e.g., Head (1986), Budhu (2000). To avoid confusion, any "p-q" diagram should always have the axes labeled so that they show the relationship to the principal stresses, $q = (\sigma_1 - \sigma_3)/2$ and $p = (\sigma_1 + \sigma_3)/2$, instead of simply using the notation "p" and "q", which is subject to ambiguity.

b. Alternate modified Mohr-Coulomb diagram. Another form of modified Mohr-Coulomb diagram that is useful is one in which the principal stress difference $(\sigma_1 - \sigma_3)$ is plotted vs. the confining pressure, σ_3, as shown in Figure D-6b. The basis for such a plot can be seen by rewriting Equation D-3 as:

$$(\sigma_1 - \sigma_3) = \frac{2c\ \cos\ \phi}{1 - \sin\ \phi} + \frac{2\ \sin\ \phi}{1 - \sin\ \phi}\sigma_3 \tag{D-11}$$

This requires somewhat more algebraic manipulation than is required to write Equation D-4, but Equation D-11 can be shown to be a valid form of Equation D-3. Equation D-11 can be written as:

$$(\sigma_1 - \sigma_3) = d' + \sigma_3\ \tan\ \psi' \tag{D-12}$$

where

$$d' = \frac{2c\ \cos\ \phi}{1 - \sin\ \phi} \tag{D-13}$$

and

$$\tan\ \psi' = \frac{2\ \sin\ \phi}{1 - \sin\ \phi} \tag{D-14}$$

From Equation D-14, the following equation can be written:

$$\phi = \arcsin\left(\frac{\tan\ \psi'}{2 + \tan\ \psi'}\right) \tag{D-15}$$

and from Equation D-13, the following equation can be written.

$$c = \frac{d'\ (1 - \sin\ \phi)}{2\ \cos\ \phi} \tag{D-16}$$

By plotting the results of triaxial tests in the form of $(\sigma_1 - \sigma_3)$ vs. σ_3 and fitting a straight line through the data points, the cohesion and friction angle can be determined from the slope and intercept of the line using Equations D-15 and D-16. A modified Mohr-Coulomb diagram like the one shown in Figure D-6b is particularly useful and instructive for plotting stress paths from triaxial tests (Section D-5). The horizontal axis represents the confining pressure in the test, σ_3, while the vertical axis is directly related to the applied axial load used to shear the specimen, $(\sigma_1 - \sigma_3)$. Thus, the two axes correspond to the two independently controlled and measured stresses in the triaxial test.

D-5. Stress Paths

a. Stress paths are plots representing the successive states of stress in a laboratory test. Although stress paths may be drawn to represent the stresses during both consolidation and shear, stress paths are most useful for the shearing stage of the test. Although a number of different diagrams can be used to plot stress paths; the two types of Modified Mohr-Coulomb diagrams described in Section D-4 are probably the most widely used and useful. While stress paths can be plotted for all three types of loading (UU, CU, and CD), only stress paths from Consolidated-Undrained (CU or R) shear tests are useful, and only they are covered in this section.

b. Stress paths for Consolidated-Undrained shear tests can be plotted for either total or effective stresses. The stress paths for total stresses are shown in Figure D-7 on both p-q diagrams and the alternate $(\sigma_1 - \sigma_3)$ vs. σ'_3 diagram described earlier. On the p-q diagram, the total stress path is along a 45-degree line extending from the horizontal axis to the failure envelope. If the stresses decrease once failure is reached, the stress path will move back along the initial loading path. On the alternate modified Mohr-Coulomb diagram shown in Figure D-7, the total stress path rises vertically to the failure envelope because the total confining pressure does not change. If the strength drops off once failure is reached, the stress path will drop back vertically along the initial loading path.

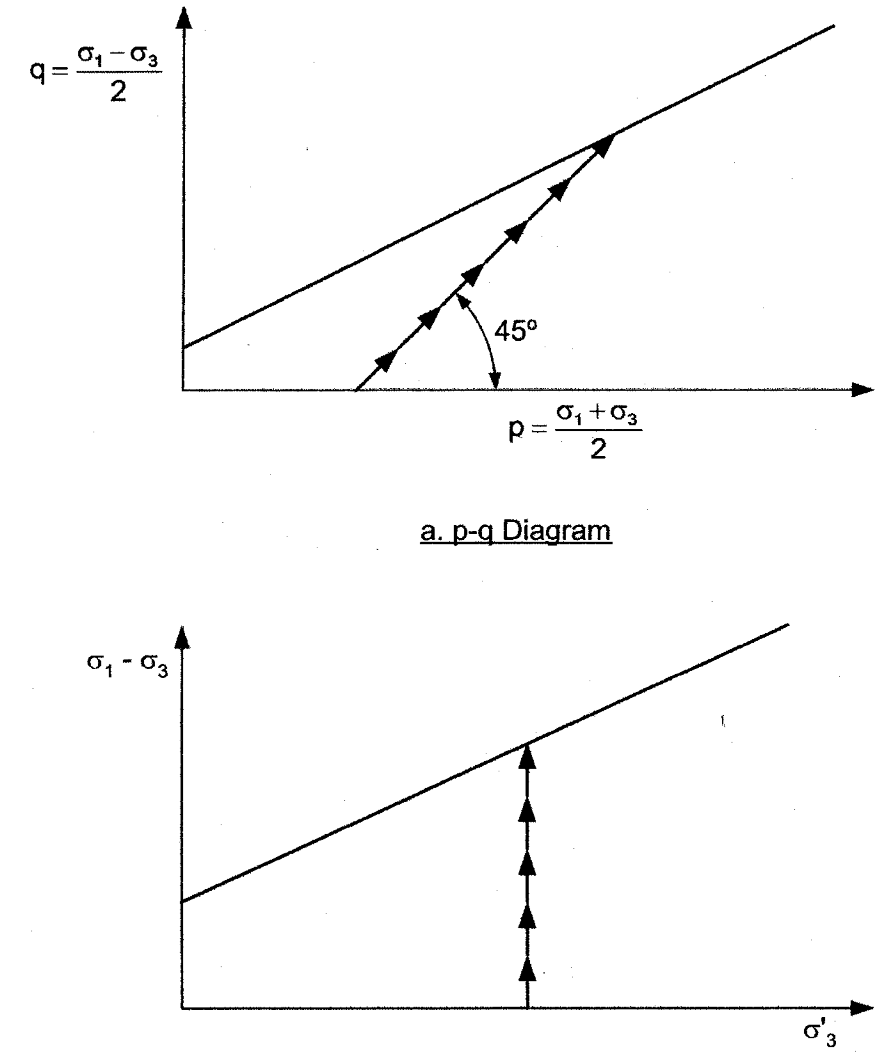
c. Effective stress paths during shear for Consolidated-Undrained loading are shown on a p-q diagram in Figure D-8 for a soil which tends to compress when sheared to failure (Figure D-8a) and for a soil which tends to dilate when sheared to failure (Figure D-8b). A broken 45-degree line extending from the initial stress point toward and across the failure envelope is also shown in each figure. The effective stress paths lie to the left of the 45-degree line when the pore water pressures increase during shear and to the right of the line when pore water pressures decrease during shear, i.e., when the soil tends to dilate. The horizontal distance between the 45-degree line and the stress path represents the change in pore water pressure Δu during shear.

d. Effective stress paths plotted on an alternate, $(\sigma_1 - \sigma_3)$ vs. σ'_3, diagram are shown in Figure D-9. Stress paths are shown for a soil that compresses during shear and for a soil that dilates during shear. Broken lines are drawn on each diagram extending vertically from the initial stress point upward and across the failure envelope. Stress paths which lie to the left of these vertical lines represent stresses where the pore water pressure has increased during shear, while those to the right of the line represent decreases in pore water pressure. The horizontal distance between the vertical line and points on the stress paths represents the change in pore water pressure Δu during shear.

D-6. Failure Criteria

Mohr-Coulomb failure envelopes are determined by plotting stresses at failure and drawing a suitable line or curve either tangent to a series of circles or through a series of points. To define the stresses at failure, a suitable criterion that defines what is meant by "failure" must be established. The criterion chosen depends on the type of test, the type of soil, and the use that will be made of the failure envelope. Failure criteria are discussed for each type of test and loading condition in the sections below.

a. *Unconsolidated-Undrained (UU or Q) test.* For Unconsolidated-Undrained shear tests, failure is usually taken as the point of maximum axial stress, $(\sigma_1 - \sigma_3)_{max}$. However, if large strains are required to reach a peak axial stress, or if the test data show no peak, it is appropriate to use some value of strain as the failure criterion. The ASTM Standard for Unconsolidated-Undrained shear tests suggests that the stress at



a. p-q Diagram

b. Alternate modified Mohr-Coulomb diagram

Figure D-7. Total stress paths for shear plotted on modified Mohr-Coulomb diagrams

15 percent axial strain should be taken as the stress at failure if no peak is reached prior to that point (ASTM 1999). This recommendation is reasonable and should be followed unless the use of stresses at larger strains can be justified. Stresses less than the peak stress may also be used as the failure stresses when strain compatibility is of concern (Section D-9). Ordinarily it will not be possible to draw the failure envelope so that it is precisely tangent to the Mohr's circles on a conventional Mohr diagram or precisely through the points that are plotted on a modified diagram. Consequently, the envelope should be drawn to fit the data in a manner that seems reasonable. Prior Corps of Engineers' practice has been to draw the strength envelope in a position such that data from two-thirds of the tests lie above the failure envelope. This recommendation is reasonable.

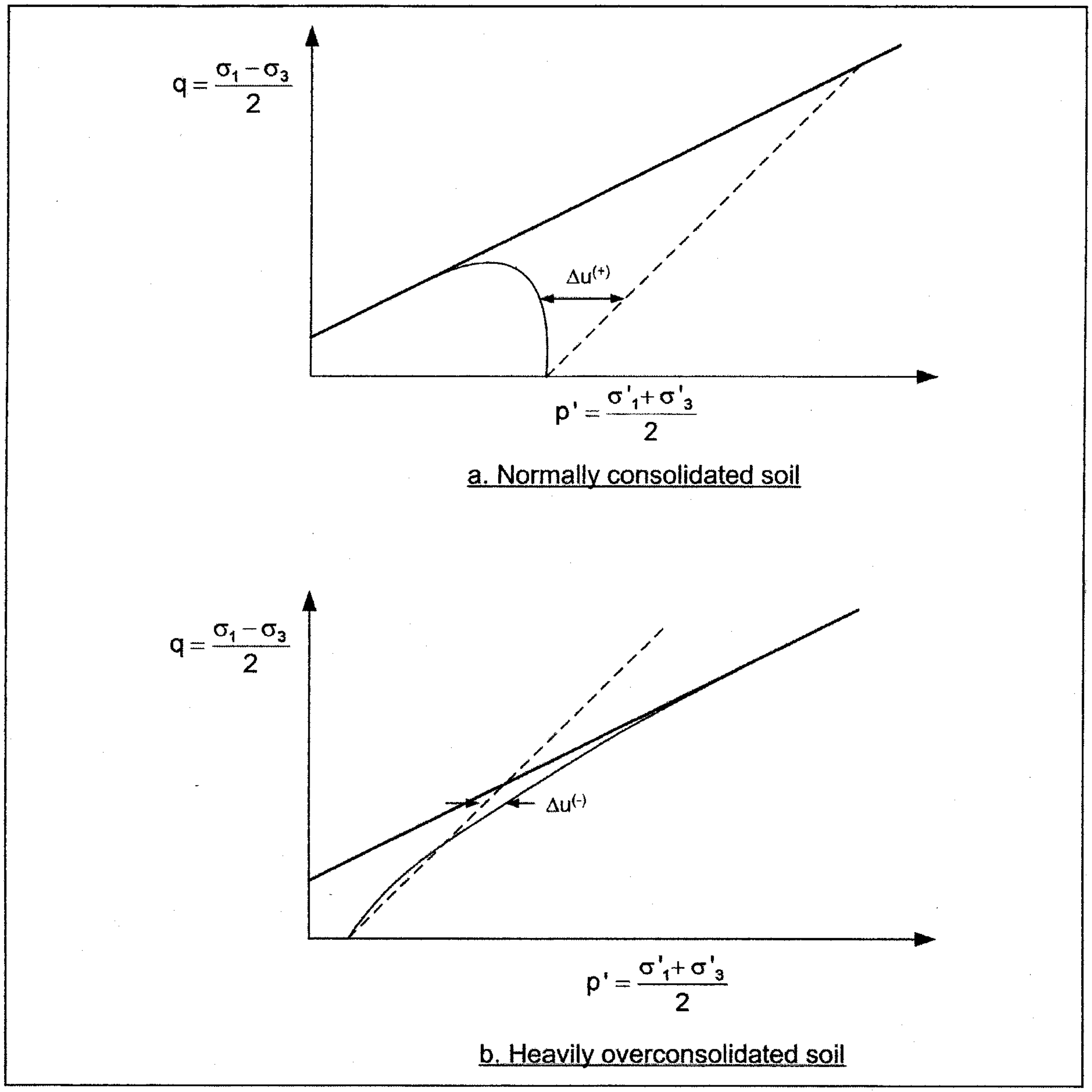


Figure D-8. Effective stress paths for shear plotted on p-q diagrams

b. Consolidated-Undrained (CU or R) test. How failure is defined for CU tests depends on the use that will be made of the results. Different criteria are appropriate depending on whether effective stress shear strength parameters or undrained shear strengths are being determined.

(1) Effective stress shear strength parameters. The appropriate failure stresses for determining effective stress shear strength parameters, c' and ϕ', are best determined by plotting the effective stress paths for the shear phase of the tests. A typical series of effective stress paths is shown on a modified Mohr-Coulomb diagram in Figure D-10. The failure envelope should be drawn such that it is approximately tangent to the stress paths, as shown in this figure. This criterion is referred to as "stress path tangency." Although variations in soil and among the samples tested will probably make it impossible to draw a failure envelop

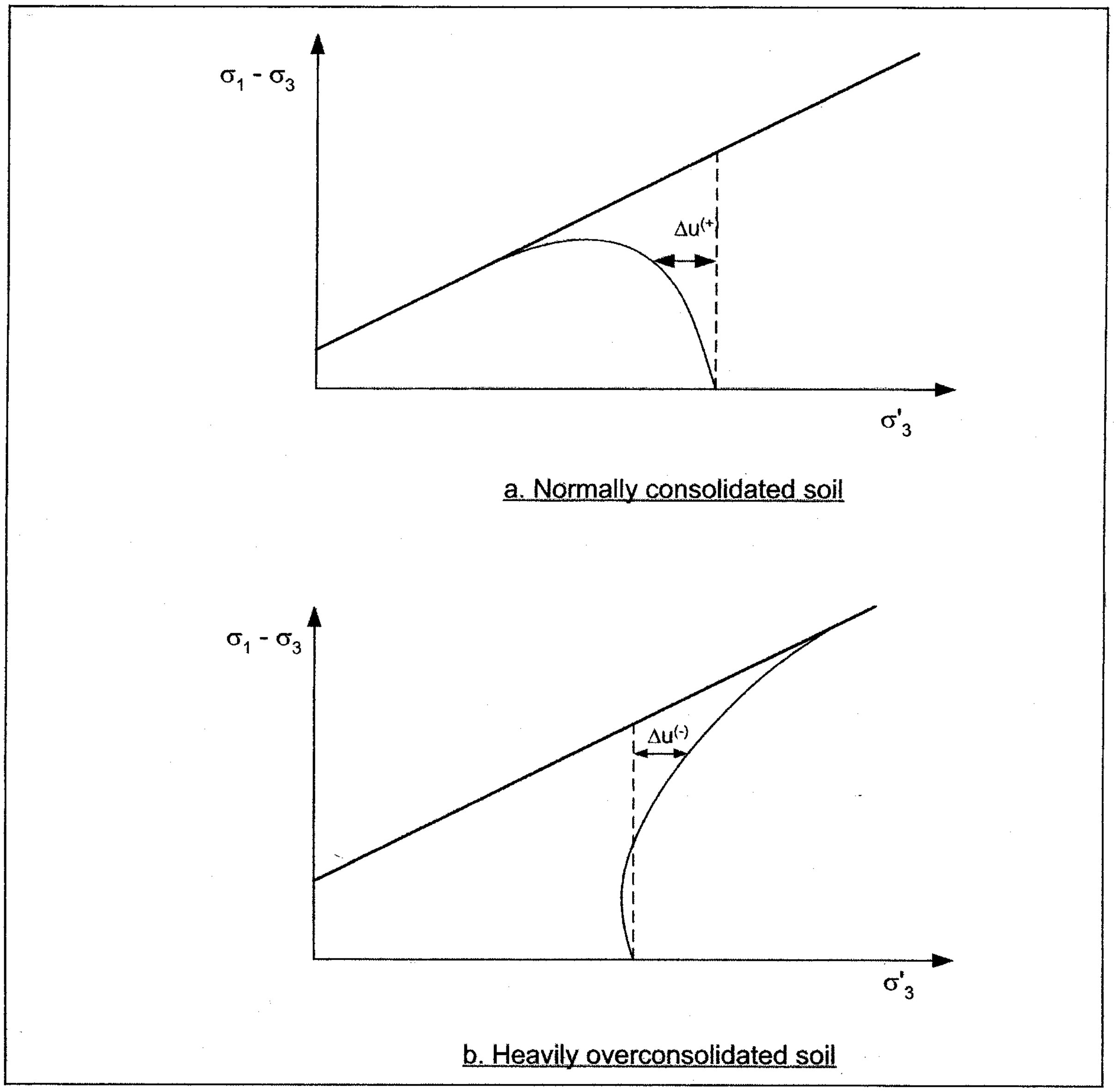


Figure D-9. Effective stress paths for shear plotted on alternate modified Mohr-Coulomb diagrams

that is precisely tangent to all stress paths, the envelope should be drawn as close to tangent as possible, with about two-thirds of the points of tangency above the line, and one-third below.

- The failure envelope may also be drawn by plotting the Mohr's circles of stress on a conventional Mohr-Coulomb diagram. In this case, it is much more difficult to determine when stress path tangency occurs; the particular set of stresses and Mohr's circle where stress path tangency occurs cannot be readily identified from the numerical test data. If several or all Mohr's circles representing the stresses at various stages of loading during each test are plotted, the number of circles becomes large and diagrams become complex and unclear.

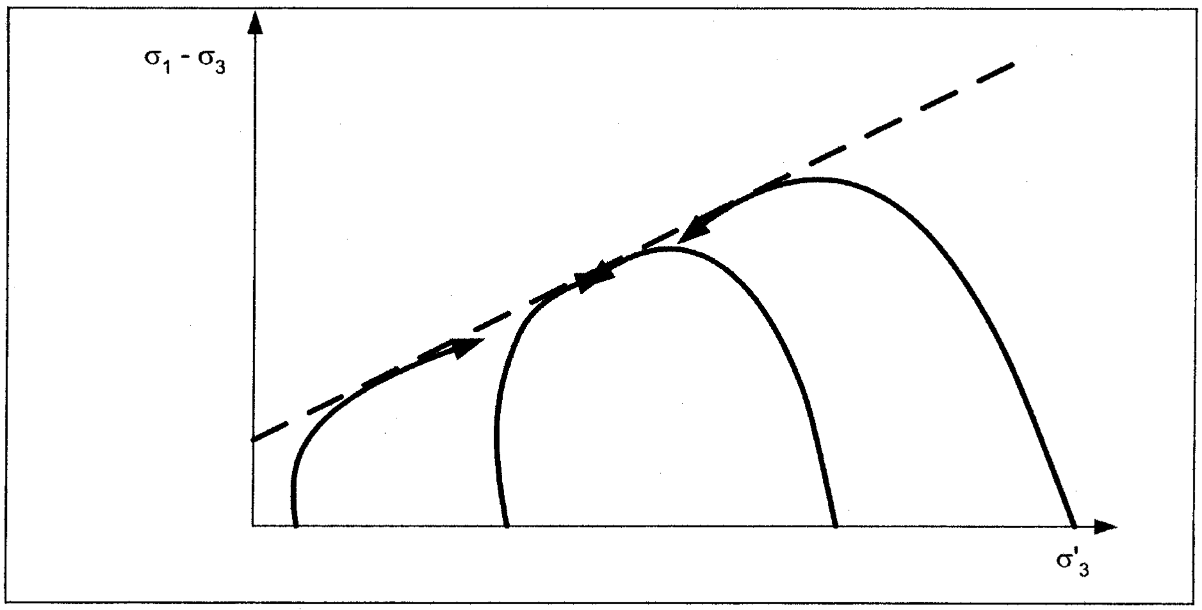


Figure D-10. Effective stress paths for Consolidated-Undrained shear tests plotted on a modified Mohr-Coulomb diagram

- One way of determining where the peak effective stress shear strength parameters are developed without plotting stress paths or Mohr circles for all the stresses during loading is to compute and examine the effective principal stress ratios, σ'_1/σ'_3, during shear for each data point recorded during the test. If the failure envelope passes through the origin of the Mohr-Coulomb diagram (c' = 0), the maximum value of the effective principal stress ratio, $(\sigma'_1/\sigma'_3)_{max}$, coincides with the point of stress path tangency. By calculating σ'_1/σ'_3 and determining where the maximum value occurs, the point of stress path tangency can be determined. The stresses at the point of tangency can then be used to plot the Mohr's circles on a Mohr-Coulomb diagram. However, this approach for determining the point of stress path tangency is only valid when the cohesion intercept, c', is zero.

- When the Mohr-Coulomb failure envelope is curved, test data must be plotted on a conventional Mohr-Coulomb diagram of τ vs. σ' in order to draw the failure envelope. This is necessary because no convenient means exists for transferring a curved envelope from a modified Mohr-Coulomb diagram back to the conventional diagram. However, even in instances where the failure envelope is curved, a modified Mohr-Coulomb diagram and stress paths may be drawn first to establish the point of stress-path tangency and failure. Once the point of stress path tangency is determined from stress-paths plotted on the modified Mohr-Coulomb diagram, the stresses can then be plotted on a conventional τ vs. σ' diagram and the curved Mohr-Coulomb failure envelope can be drawn.

- For many normally consolidated clays and loose sands, the point of stress-path tangency occurs after the maximum axial load is reached. In order to capture the point of stress-path tangency, it is necessary to continue to shear specimens past the point where the maximum load is developed. In order to do so, deformation-controlled loading, rather than load-controlled loading, must be used.

(2) Undrained shear strengths. When Consolidated-Undrained loading procedures are used to determine undrained shear strengths, the failure criterion for plotting the data are the same as those used for UU tests. The peak stress or the stress at a limiting value of strain, e.g., 15 percent axial strain, is used as the failure criterion.

c. Consolidated-Drained (CD or S) tests. Failure in consolidated-drained tests is determined as the point of maximum principal stress difference, $(\sigma_1 - \sigma_3)_{max}$, or at some limiting, maximum value of axial strain. Fifteen percent axial strain is a reasonable value to use as a failure criterion. Heavily overconsolidated, stiff-fissured clays and dense sands sometimes exhibit significant reduction in shearing resistance with strain beyond the peak. In these materials it is not possible to develop the peak strength simultaneously at all points along the shear surface. Also, in slopes where prior sliding has resulted in development of slickensided slip surfaces, the shear resistance has already declined to its residual value. In these instances adequate stability can only be ensured by using residual shear strengths in stability analyses.

D-7 Generalized Stress-Strain-Strength Behavior

An understanding of the stress-strain response of soils is useful in interpreting the results of laboratory shear tests. The stress-strain response of soils in both drained and undrained shear tests is discussed below.

a. Drained loading. Typical stress-strain curves from triaxial shear tests on dense and loose sands are shown in Figure D-11. The upper portion of this figure shows the axial stress-strain curves, while the lower portion shows volumetric strain vs. axial strain curves. Loose sands tend to compress (volume decreases) during shear. The axial stress may increase with increasing strain up to 20 or 25 percent axial strain or even more. Dense sands also tend to compress initially when sheared, but they then expand as they are sheared to larger strains. In dense sands, peak load is reached at much smaller strains than for loose sands, and the stress may then decrease significantly as strains are further increased. If loose and dense specimens of the same sand are sheared to large strains at the same confining pressure, the strengths will become similar at large strains, regardless of the initial density. At large strains, the soil is said to reach a "critical state" or "critical void ratio," and the shearing resistance at these large strains is largely independent of initial density. Normally and heavily overconsolidated clays tend to exhibit stress-strain response similar to those for loose and dense sands. Normally consolidated clays tend to compress throughout shear, developing a peak resistance at 10 to 20 percent axial strain. Heavily overconsolidated clays tend first to compress and then to dilate as they are sheared to large strains. Under drained loading, the peak resistance of heavily overconsolidated clays is usually developed at smaller strains than for normally consolidated clays.

(1) The response to shear of both clays and sands with different stress histories or densities can be illustrated and explained with the concept of a "critical void ratio" or "critical state" first suggested by Casagrande (1936) and later promoted for clays by Roscoe, Schofield, and Wroth (1958). This is illustrated by the diagram of void ratio vs. confining pressure, σ_3, shown in Figure D-12. The curve labeled "critical state" in Figure D-12 represents the void ratios which soils eventually reach when they are sheared to large strains at various confining pressures. If a soil is loose, such that it starts shear at a point above the "critical state" line, the soil will compress (void ratio will decrease), as suggested by the path a-c in Figure D-12. In contrast, if a soil is dense, such that it starts shear at a point below the critical state line, the soil will tend to dilate as large strains are reached and the soil will dilate (void ratio will increase), as suggested by path b-c in Figure D-12). Regardless of the initial density, two specimens of the same soil tested at the same confining stress will tend to reach a similar void ratio and have very similar shear strengths at large strains. The dense soil will, however, have a higher peak strength.

(2) For clays, a similar set of behavioral characteristics is observed. Normally consolidated clays tend to compress and reach a critical state when sheared, while heavily overconsolidated soils tend to expand as they reach the critical state at large strains.

(3) Compacted soils can exhibit stress-strain responses varying from that for normally consolidated soil to that for heavily overconsolidated soils. At low confining pressures compacted clays tend to behave like overconsolidated clays, and at high confining pressures, where the effects of compaction no longer dominate their behavior, they behave more like normally consolidated soils.

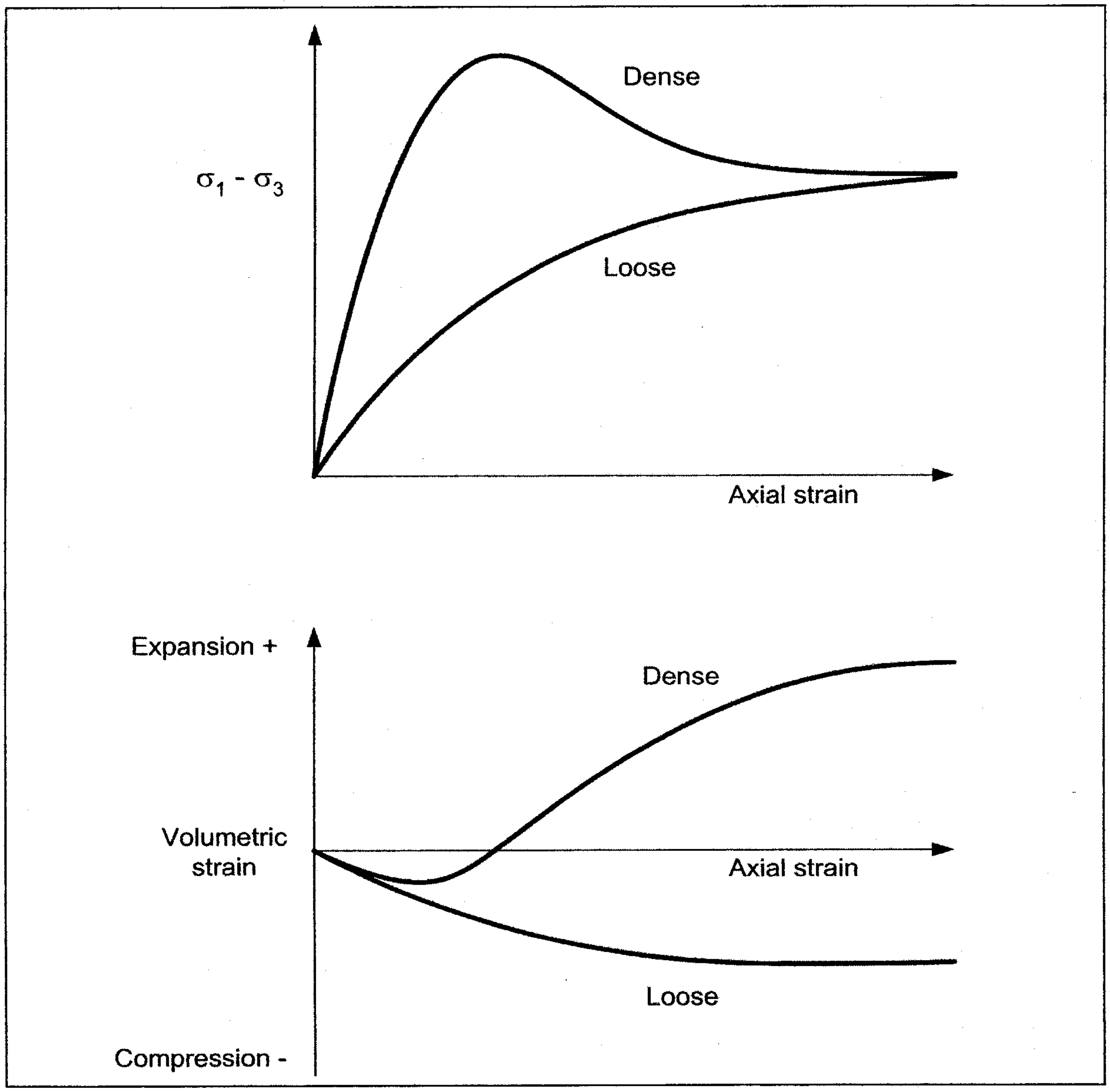


Figure D-11. Typical stress-strain curves from CD-S triaxial shear tests on dense and loose sands

b. Undrained loading. Typical effective stress paths for Consolidated-Undrained triaxial shear tests on normally consolidated and overconsolidated clays are illustrated in Figure D-13. Figure D-13a shows the stress paths on a p-q diagram, while Figure D-13b shows the stress paths on a modified Mohr-Coulomb diagram where (σ_1 - σ_3) is plotted versus σ_3. Note how the stress paths for the normally consolidated clay reach a peak value of (σ_1 - σ_3) prior to becoming tangent to the effective stress failure envelope. If the specimen were being sheared using controlled load, rather than controlled deformation, the postpeak behavior might either be lost or the sample might deform so rapidly as to make pore water pressure and effective stress measurements meaningless (result of unequalized pore water pressures in the specimen). This is the reason that deformation controlled, rather than load controlled loading is preferred for undrained tests on normally and lightly overconsolidated clays.

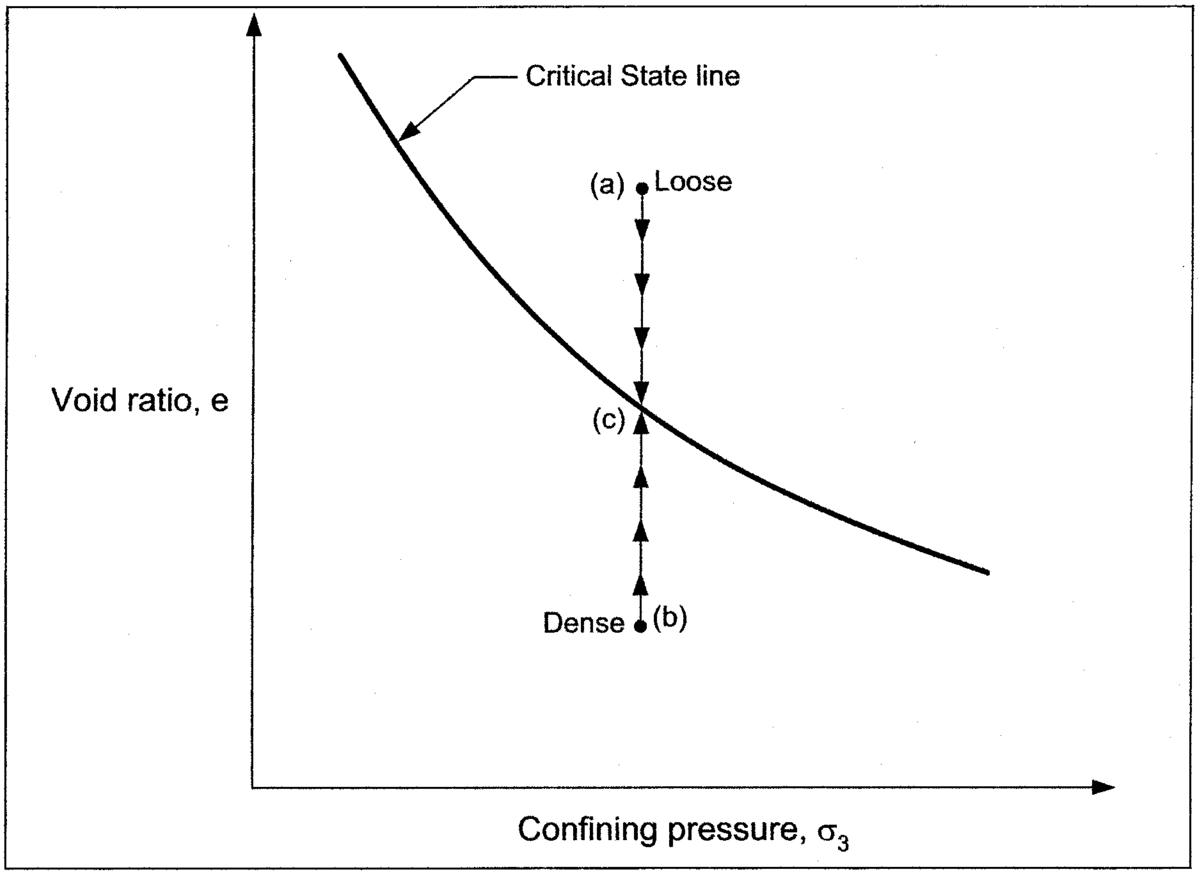


Figure D-12. Critical State line representing combinations of void ratio and confining pressure for soil that has been sheared to large strains – behavior in drained tests

(1) The concept of a "critical state" line presented earlier is applicable to undrained loading as well as drained loading. Figure D-14 shows a critical state line along with a line representing the line for virgin, isotropic consolidation. Specimens that are normally consolidated prior to undrained shear in the triaxial test have loading paths for shear that start along the virgin isotropic consolidation line. The line labeled a-b represents the loading path for a normally consolidated soil specimen that is sheared to large strains, and eventually reaches the critical state line. The pore water pressure in the specimen continually increases during shear, and the effective confining pressure, σ'_3, continually decreases. Similarly, the line labeled c-d represents a loading path for a heavily overconsolidated soil that is sheared to large strains at the same initial effective confining pressure. The pore water pressures at failure are less than they were at the start of shear and the effective stresses have increased. The lengths of the paths a-b and c-d represent the changes in pore pressure during the tests. Movement to the left indicates increase in pore pressure, and movement to the right indicates decrease in pore pressure.

(2) The line labeled e-b in Figure D-14 represents another overconsolidated specimen. However, in this case the specimen has the same void ratio as the normally consolidated specimen. Both specimens have the same void ratio, but prior to shear, the normally consolidated specimen is under a higher confining pressure than the overconsolidated specimen. During shear the pore water pressures in the normally consolidated specimen increase (line a-b) and the pore water pressures in the overconsolidated specimen decrease (line e-b). At large strains (at the critical state line), both specimens have the same void ratio and have reached a

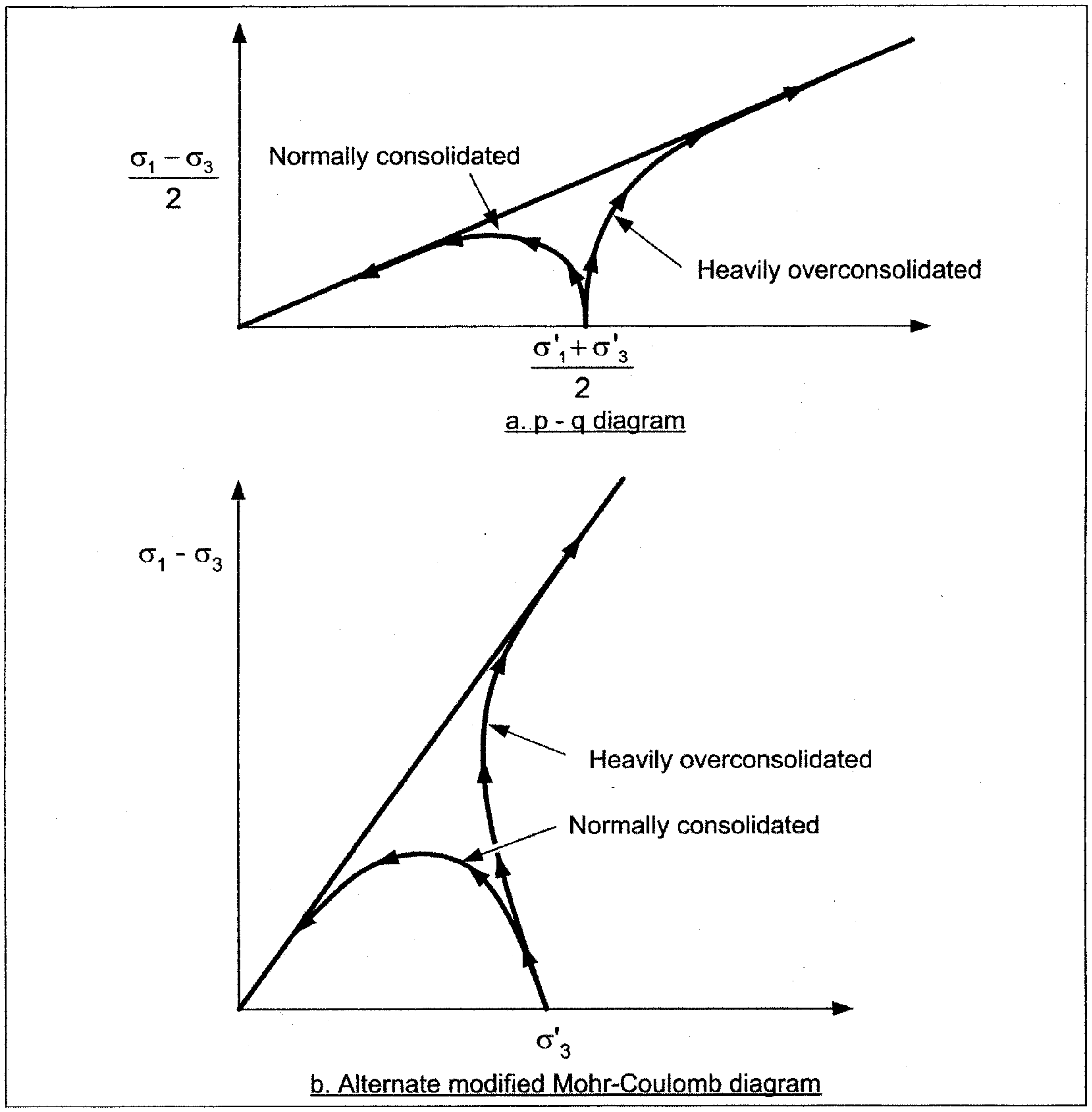


Figure D-13. Effective stress paths on two types of modified Mohr-Coulomb diagrams for normally consolidated and heavily overconsolidated clays

"critical confining pressure," $\sigma'_{3\text{-critical}}$. Both specimens also have the same shear strength at the large strains corresponding to the critical state line, although the peak shear strengths of the two specimens would be quite different.

c. Sample disturbance. Disturbance of soil specimens caused by sampling and handling affects the stress-strain response of soils. Disturbance can sometimes be detected by simply examining the soil response during shear, especially when the soil is either loose sand or normally consolidated clay. Disturbance densifies loose sands and causes them to behave more like dense sands. Similarly, disturbance of normally

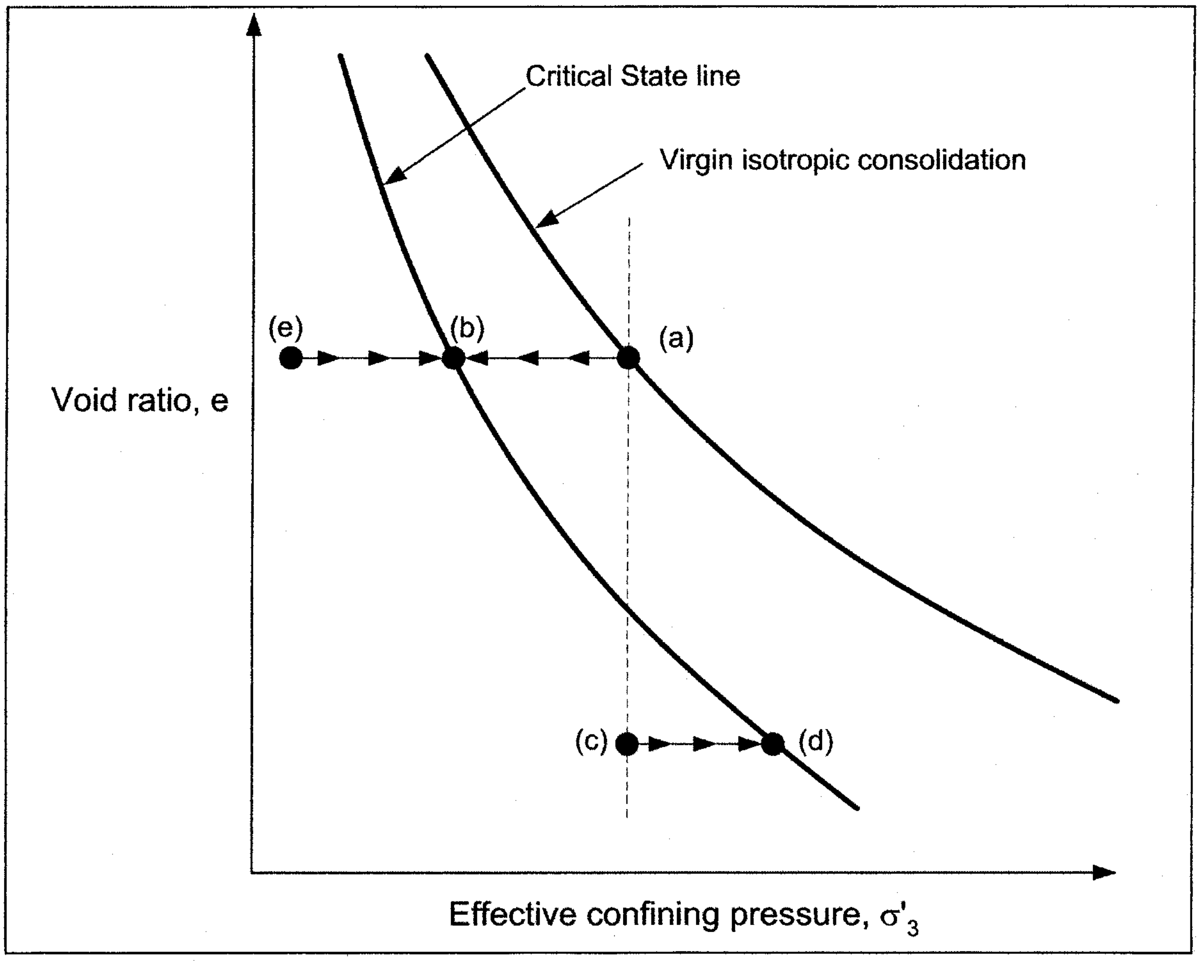


Figure D-14. Critical State line representing combinations of void ratio and confining pressure for soil that has been sheared to large strains – behavior in undrained tests

consolidated soil specimens makes them respond more like overconsolidated clays because disturbance causes the pore water pressure to increase and the effective stress to decrease. The axial strain at which the peak strength is developed in Unconsolidated-Undrained (UU or Q) tests may increase when specimens are disturbed, and instead of being in the typical range of 1 to 6 percent expected for normally consolidated clays, may increase to 10 percent or more if the specimens are disturbed. Pore water pressures generated during shear will also be lower if the specimens are disturbed. However, in the case of Unconsolidated-Undrained shear tests, the pore water pressures that exist prior to shear will be higher because of disturbance effects. The higher pore pressures at the start of shear will offset the lower pore water pressures developed during shear, such that sample disturbance will reduce the undrained shear strength measured in Unconsolidated-Undrained tests.

D-8. Curved Strength Envelopes

a. The strength envelopes for many soils are curved, rather than linear, as shown in Figure D-15. If the curvature is small and the range of stresses of interest is small, a curved failure envelope can be approximated by a straight line for purposes of analysis, as shown in Figure D-15b. However, if the envelope is distinctly curved over the range of stresses of interest, use of a straight line failure envelope may significantly

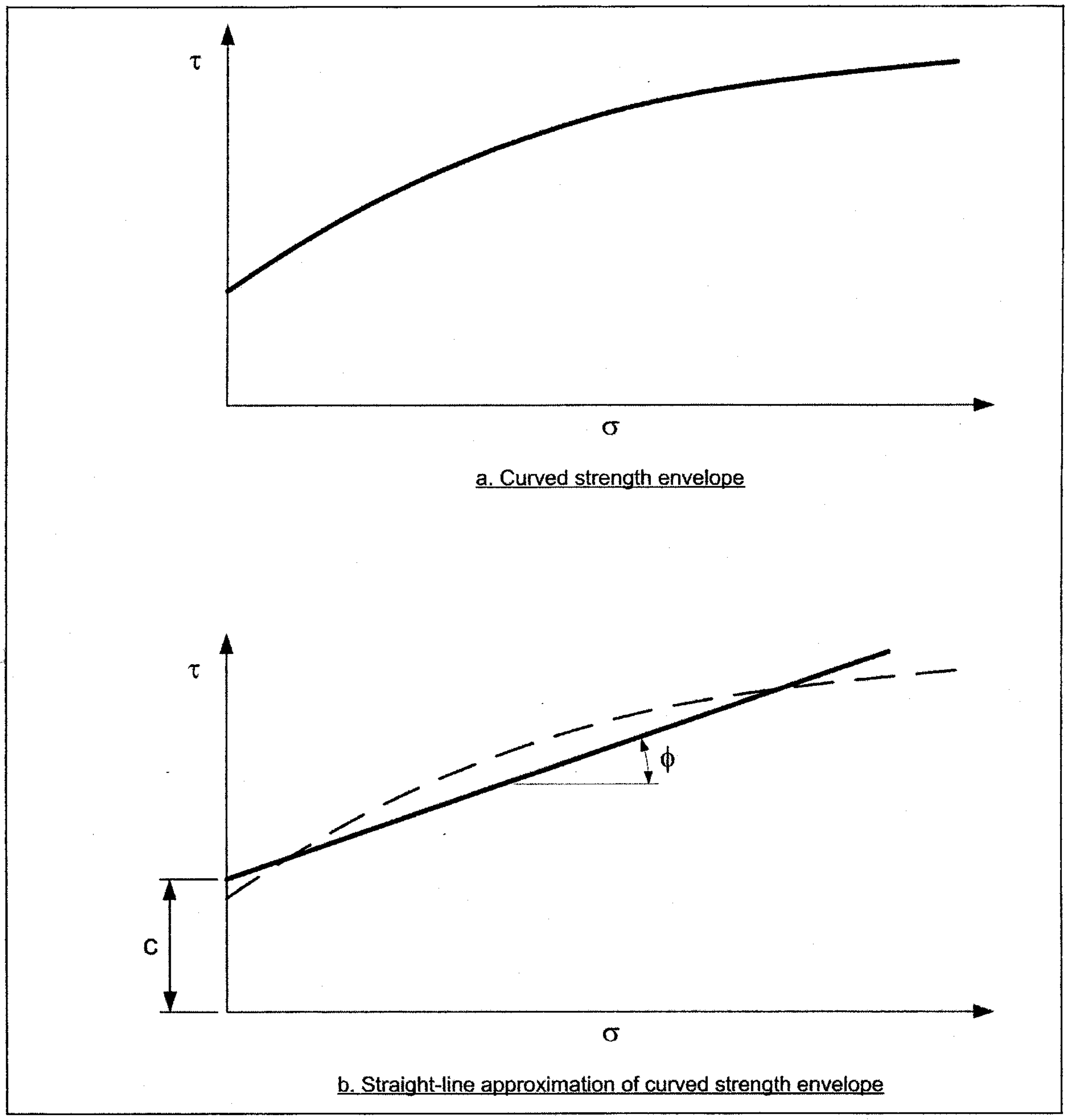


Figure D-15. Curved strength envelope and straight-line approximation

overestimate the shear strength at low and at high stresses. Extrapolation to higher or lower stresses can result in an overestimate of shear strength, and a factor of safety with regard to slope stability that is too high.

b. It is especially important that laboratory tests be conducted using a range in confining pressures that represents the range of stresses expected along potential sliding surfaces. Once tests have been conducted using a suitable range in stresses, an appropriate decision can be made regarding how the strength envelope will be represented. It is relatively easy to use curved strength envelopes in slope stability computations with software that is currently available, and there is little advantage to using a liner strength envelope when the data suggest otherwise.

c. Frequently, when the strength envelope is curved, the results of strength tests are reported in terms of a secant friction angle, ϕ_{secant} (Figure D-16). For example, Duncan, Horz, and Yang (1989) present useful correlations for the friction angle of a number of soils in terms of the secant friction angle at one atmosphere confining pressure, and the reduction in friction angle $\Delta\phi$, with each ten-fold increase in confining pressure. Stark and Eid (1994, 1997) present correlations for residual and fully softened shear strengths in terms of the secant friction angle at selected values of effective normal stress. Correlations such as those by Duncan, Horz, and Yang (1989) and Stark and Eid (1994, 1997), are useful for estimating shear strength values for preliminary stability analyses and to supplement data from laboratory tests. However, for slope stability computations it is usually preferable to express the strength envelope in terms of a continuous function of shear strength versus normal stress, rather that as a series of discrete values of secant friction angle. This can be done by selecting suitable values of normal stress, σ_j, and determining the corresponding friction angle, ϕ_{secant} for each value of σ_j. Values of the respective shear strength are then computed as:

$$s_j = \sigma_j \tan \phi_{secant\text{-}j} \tag{D-17}$$

where

σ_j = one of a number of values of normal stress

s_j and $\phi_{secant\text{-}j}$ = corresponding values of secant friction angle and shear strength

The values of shear strength s_j are plotted against the corresponding values of σ_j to develop a nonlinear strength envelope like the one shown in Figure D-17, which is then used in the slope stability computations.

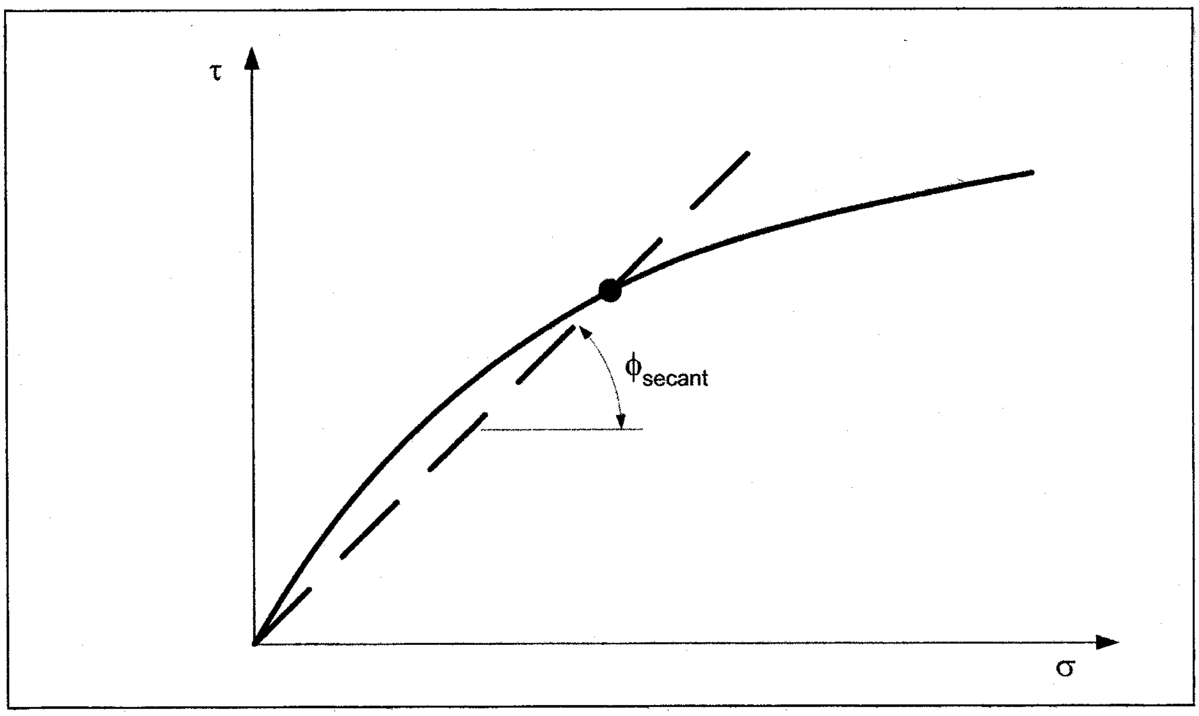


Figure D-16. Curved strength envelope and equivalent secant friction angle

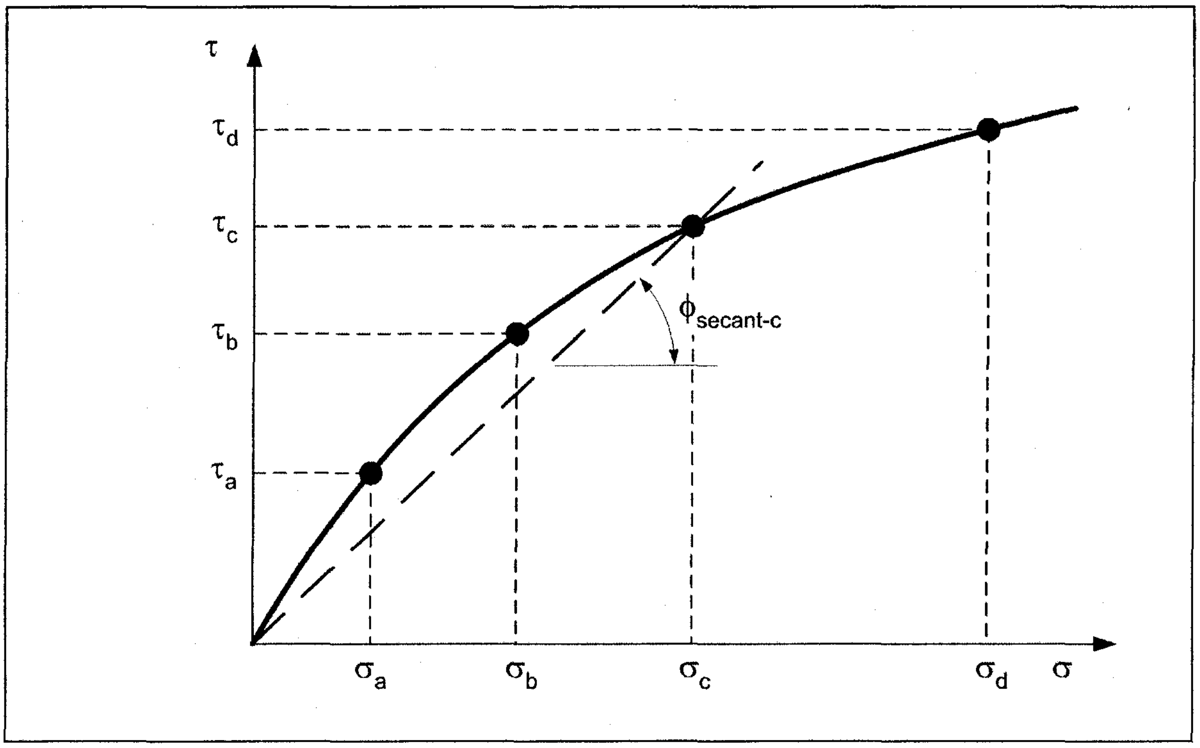


Figure D-17. Construction of curved strength envelope using secant friction angles

D-9. Strain Compatibility

a. "Strain Compatibility" is a term used to refer to the variation in the stress-strain properties of soils along a slip surface. In an actual slope, both the strains that are developed and the stress-strain properties will vary such that it is unlikely that the peak shear strength will be developed simultaneously along the full length of a slip surface. This is important if the soils within the slope exhibit significant strain softening. In such cases, it is appropriate to adopt lower shear strengths or to require higher factors of safety.

b. Several approaches have been suggested for handling issues of strain incompatibility. Koutsoftas and Ladd (1985) have suggested a procedure for determining an "equivalent" strain that is used as the failure criterion for determining shear strengths from laboratory test data. Also, Chirapuntu and Duncan (1975) have developed procedures for reducing shear strengths when highly dissimilar soils exist along a potential slip surface. For many soils and slope conditions, no special provisions are necessary. For example, Wright, Kulhawy, and Duncan (1973) showed that for overall factors of safety of at least 1.5, the peak strength of the soil is not fully reached at any point along the potential slip surface, even though the strains and factor of safety may vary significantly. When the possibility of reduced strengths exists, it is helpful to compute the factor of safety using the ultimate (residual) shear strength. This represents a lower bound for shear strength and should indicate if there is any possibility of failure due to strain incompatibility problems. When such a conservative estimate for shear strengths is used, lower than conventional values of factor of safety are acceptable.

D-10. Staged Construction

a. In staged construction, an embankment is built in increments and the foundation soil is allowed to consolidate fully or partially under each stage so that the increased strength will increase stability during subsequent states. Staged construction is generally used when the foundation soils are so weak that the entire embankment cannot be built in a single increment. Analyses for stage construction require special consideration in developing shear strength parameters. One approach is to use Consolidated-Undrained tests to measure the strengths, taking into account increases in strength resulting from increases in effective consolidation pressure. This involves the following steps:

- Step 1: Initial effective stresses and maximum past pressures are determined. Initial stresses are computed from unit weights and the groundwater levels prior to embankment construction. Maximum past effective stresses are determined from oedometer tests on high quality, undisturbed samples.

- Step 2: Normalized shear strengths, S_u/σ'_{vc}, are determined for various overconsolidation ratios by performing Unconsolidated-Undrained shear tests using the SHANSEP procedure.

- Step 3: Pore water pressures and effective stresses in the field are estimated using an appropriate consolidation analysis. The consolidation analysis should take into account the initial stresses, the increase in stress because of added embankment loads, and the subsequent consolidation because of dissipation of excess pore water pressures.

- Step 4: Undrained shear strengths are estimated using the information from Steps 1, 2, and 3. Undrained shear strengths are calculated by multiplying the appropriate values of normalized shear strength by the effective vertical stress, thus accounting for consolidation.

- Step 5: Stability analyses are performed using undrained shear strengths. The undrained shear strengths are assigned as values of cohesion, c, and ϕ is equal to zero.

b. The above procedure requires assumptions about how the initial excess pore water pressures are generated by the embankment loads, especially regarding the pore water pressures beneath and beyond the toe of the slope. Also, a relatively complex analysis of consolidation is required to account for the variation in stresses and excess pore water pressures in the vertical and horizontal directions. Finally, the shear strength is usually related to the vertical effective stress in the field, which, unlike the stresses in the laboratory, is seldom the major principal stress during consolidation. More uncertainty exists in analyses of staged construction than for other cases, and this should be taken into consideration when selecting appropriate shear strength values and factors of safety for design.

c. An alternative approach to the undrained strength approach described above is to perform an effective stress analysis using effective stress shear strength parameters (c' and ϕ') and estimated pore water pressures. This approach requires the same relatively complex consolidation analysis used in the first approach and, thus, the second approach is also subject to the same errors. The effective stress approach will also give different values for the factor of safety as the result of fundamental differences between total and effective stress factors of safety: The effective stress approach is based on pore water pressures at working stress levels, rather than values at failure, while the undrained strength approach is based on pore water pressures generated at failure. Because there is no experience to guide selection of safety factors for the effective stress approach, it should not be used.

D-11. Partially Saturated Soils

a. Partially saturated soils present special problems when treated using effective stresses. Significant progress has been made in understanding how partially saturated soils behave and the role of effective stresses (Fredlund and Rahardjo 1993; Fredlund 2000). This work indicates that the simple expression for effective stress where the pore water pressure is subtracted from the total stress to evaluate the effective stress is not valid and that the Mohr-Coulomb shear strength equation for effective stresses in saturated soils is not valid for unsaturated soils. Rigorous treatment of effective stresses in partially saturated soils is beyond the scope of this manual.

b. Fortunately, consideration of effective stresses in unsaturated soils can be avoided for many practical slope stability problems. To evaluate strength and stability at the end of construction, Unconsolidated-Undrained shear tests are performed to measure the shear strength. In this case the shear strengths are expressed as a function of total stresses, and the approach is valid for both saturated and unsaturated soils. For long-term stability and stability during rapid drawdown, the soil may be fully or only partially saturated. However, if the soil is below the groundwater table or beneath the phreatic surface, the pore water pressures are positive and the soil is assumed to be saturated for design purposes. If the soil is above the water table or in a zone of capillarity and where pore pressures are negative, the beneficial effects of negative pore water pressures are conservatively neglected by assuming that the pore water pressures are zero. Conventional effective stress shear strength parameters are used for both the saturated (positive pressure) and partially saturated (zero pressure) zones. The effective stress shear strength parameters are measured on specimens that are fully saturated prior to laboratory testing, regardless of the saturation that may exist in the field.

c. For cases where substantial portions of the slope are partially saturated and long-term stability is being evaluated, neglecting negative pore water pressures can be very conservative. In such cases, some account of negative pore water pressures may be appropriate (Fredlund 1989, 1995). Beneficial effects of negative pore water pressure can easily be destroyed by rainfall and infiltration of surface water, as well as a rise in ground water table. Thus, negative pore water pressures should be included in stability computations with great caution. Use of negative pore water pressures is not recommended for design of dams and similar critical structures where the consequences of failure are great.