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US Army Corps
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ENGINEERING AND DESIGN

Slope Stability

ENGINEER MANUAL

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

EM 1110-2-1902

Manual
No. 1110-2-1902

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Engineering and Design
SLOPE STABILITY

1. Purpose. 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.

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

3. Distribution Statement. This publication is approved for public release; distribution is unlimited.

4. Scope of the Manual. This manual is intended to guide design and construction engineer, rather than to specify rigid procedures to be followed in connection with a particular project.

FOR THE COMMANDER:

7 Appendixes
(See Table of Contents)

MICHAEL J. WALSH
Colonel, Corps of Engineers
Chief of Staff

This manual supersedes EM 1110-2-1902, 1 April 1970.

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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:

$$s = c + \sigma \tan \varphi \tag{2-1}$$

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, $\overline{R}$) 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

$$s = c' + \sigma' \tan \varphi' \tag{2-2}$$

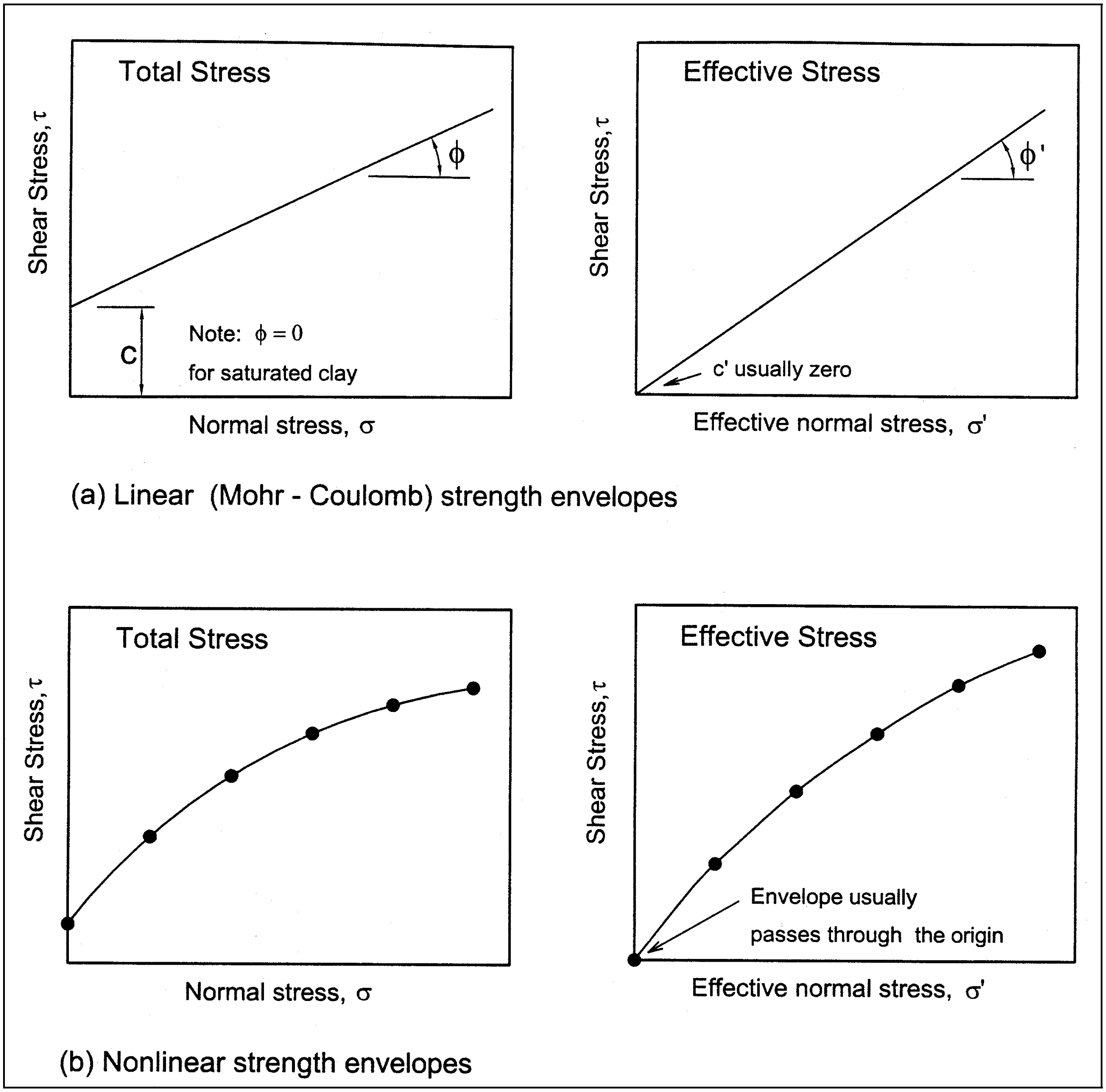


Figure 2-1. Strength envelopes for soils

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.

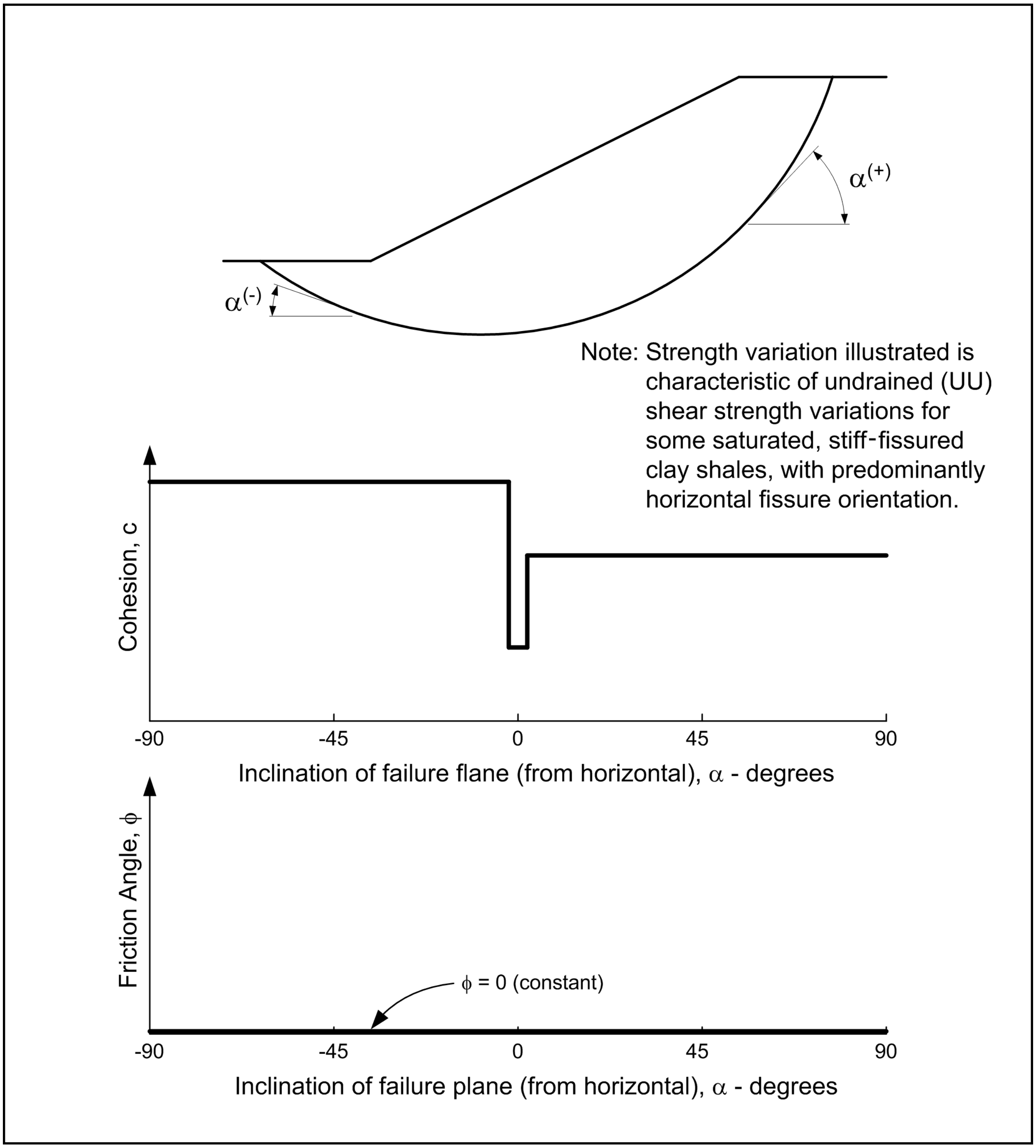


Figure 2-2. Representation of shear strength parameters for anisotropic soil

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:

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

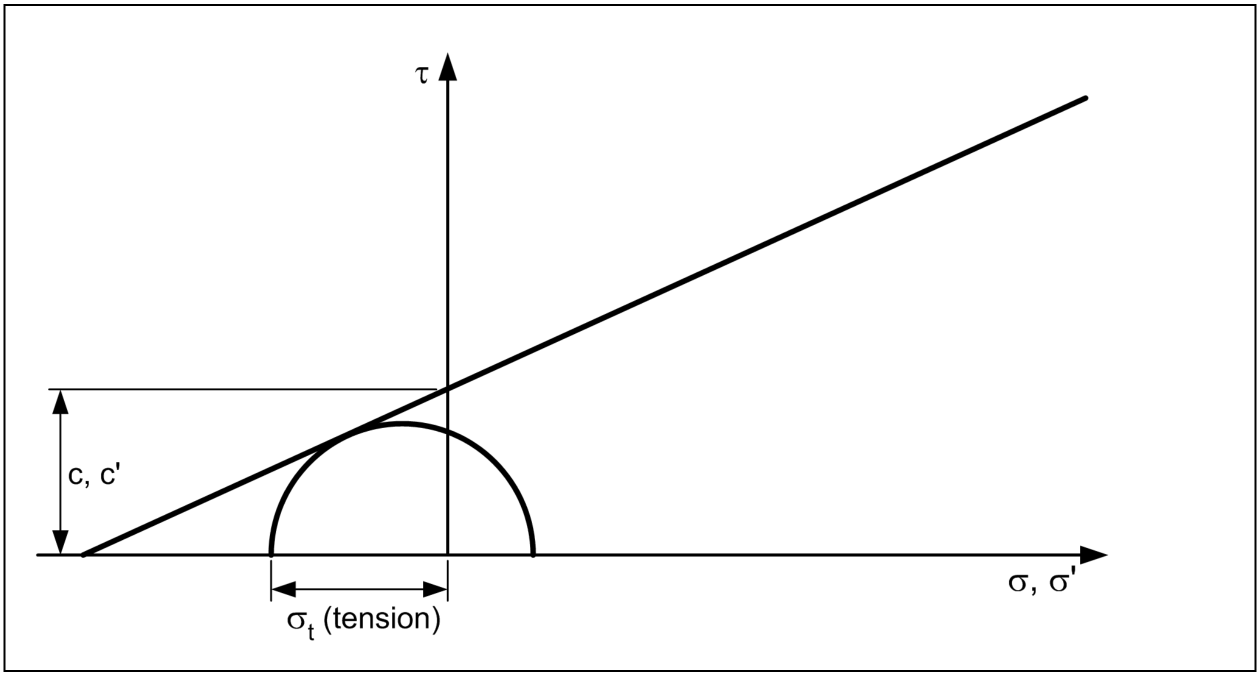


Figure 2-3. Tensile stresses resulting from a Mohr-Coulomb failure envelope with a cohesion intercept

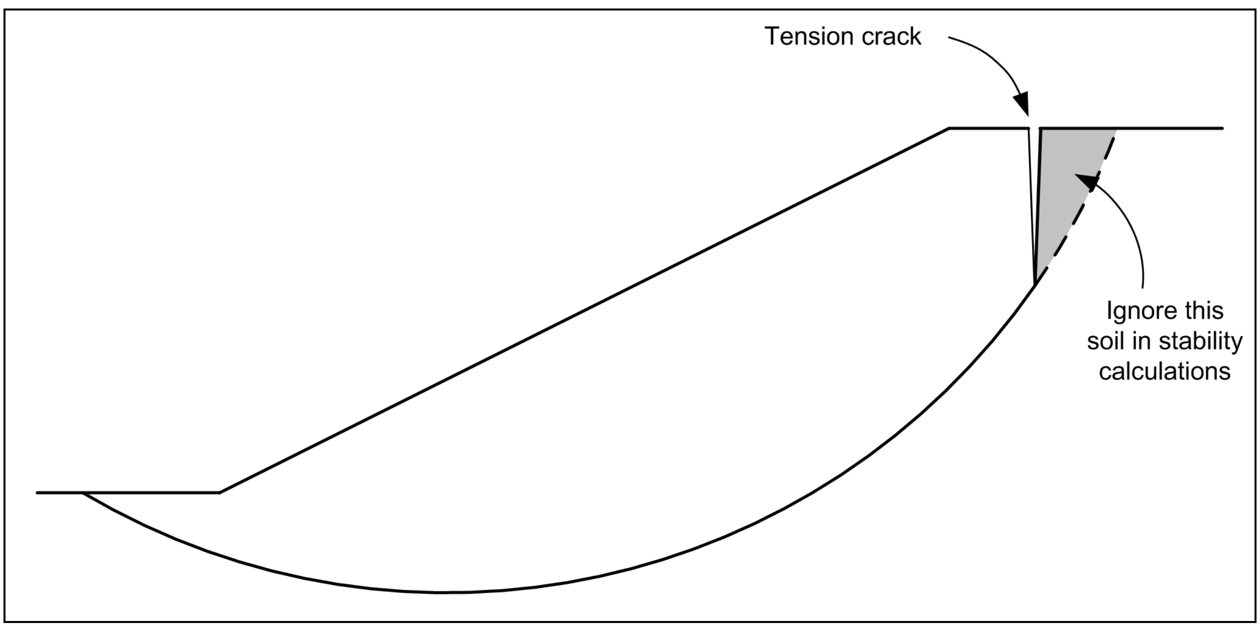


Figure 2-4. Vertical tension crack introduced to avoid tensile stresses in cohesive soils

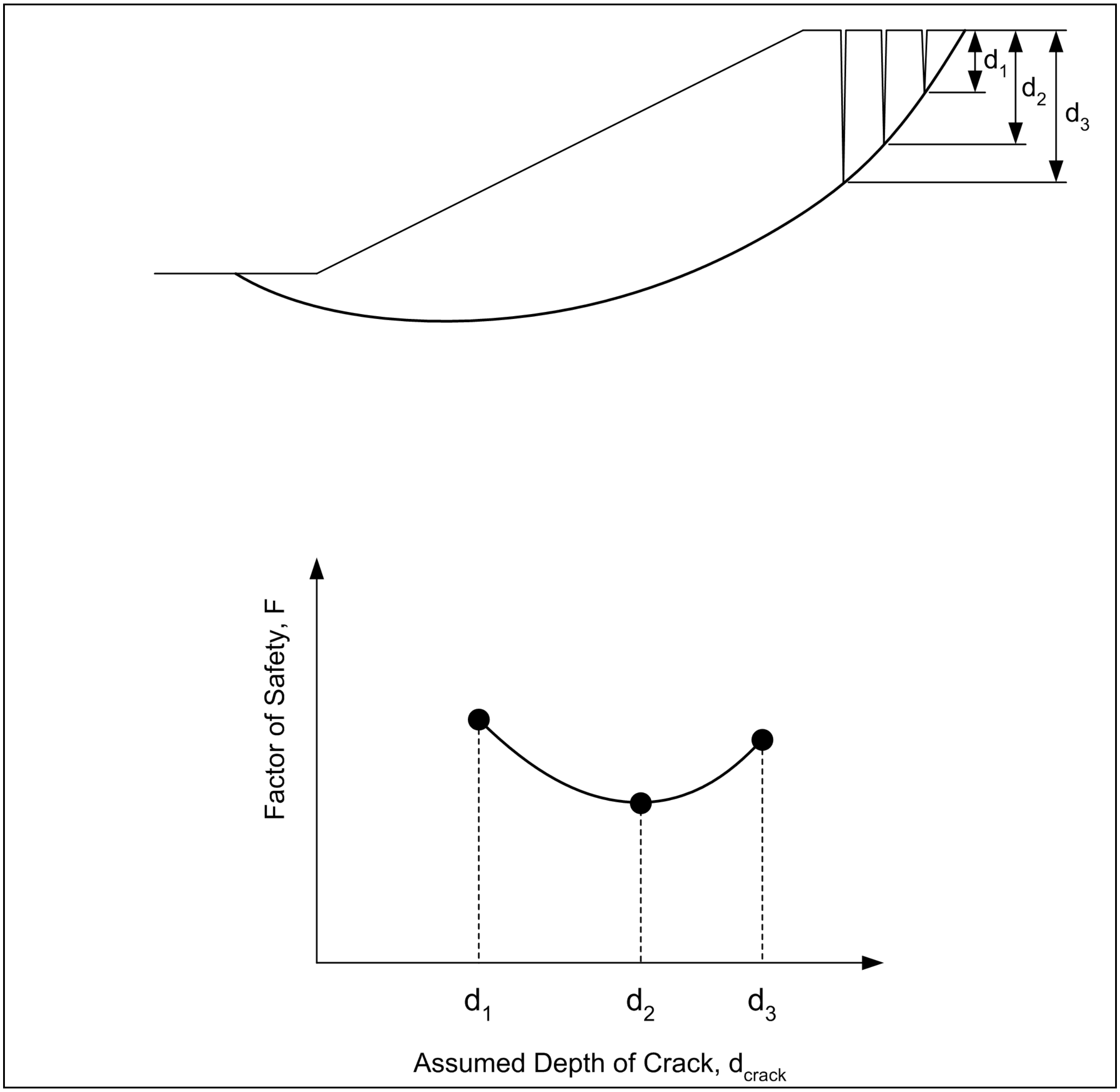


Figure 2-5. Variation in the factor of safety with the assumed depth of vertical crack

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

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

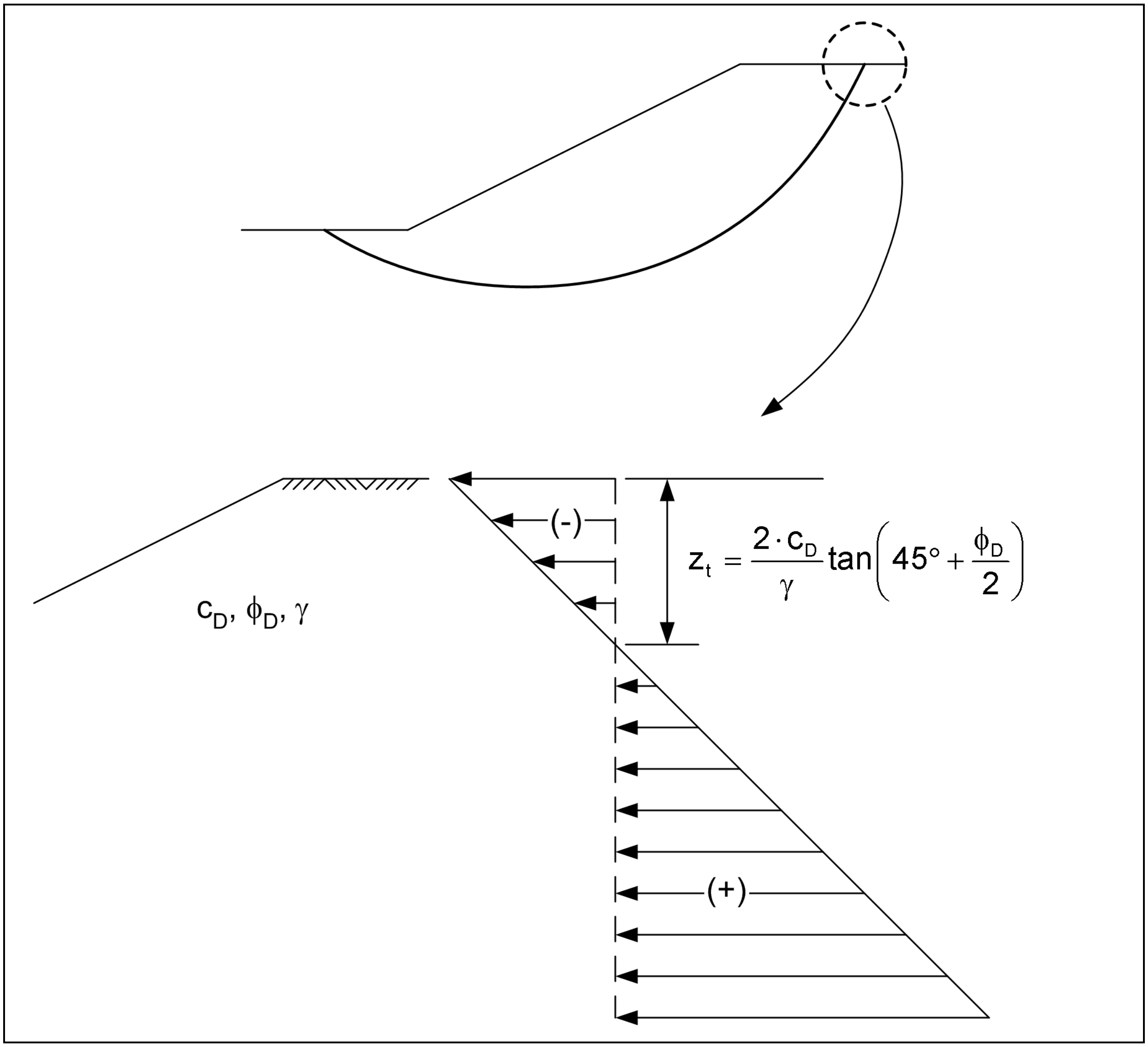


Figure 2-6. Horizontal stresses near the crest of the slope according to Rankine active earth pressure theory

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

the lower of the drained or undrained strength to compute the stability for sudden drawdown. However, the 1970 procedure employs assumptions that may make it excessively conservative, especially for soils that dilate or tend to dilate when sheared. Further details and examples of the procedures for sudden drawdown are presented in Appendix G.

b. Analysis stages. The recommended procedure involves three stages of analysis. The purpose of the first set of computations is to compute the effective stresses along the shear surface (on the base of each slice) to which the soil is consolidated prior to drawdown. These consolidation stresses are used to estimate undrained shear strengths for the second-stage computations, with the reservoir lowered. The third set of computations also analyzes stability after drawdown, using the lower of the drained or undrained strength, to ensure that a conservative value of factor of safety is computed.

c. Partial drainage. Partial drainage during drawdown may result in reduced pore water pressures and improved stability. Theoretically such improvement in stability could be computed and taken into account by effective stress stability analyses. The computations would be performed as for long-term stability, except that pore water pressures representing partial drainage would be used. Although such an approach seems logical, it is beyond the current state of the art. The principal difficulty lies in predicting the pore water pressures induced by drawdown. Approaches based on construction of flow nets and numerical solutions do not account for the pore pressures induced by shear deformations. Ignoring these shear-induced pore pressures results in errors that may be on the safe side if the shear-induced pore pressures are negative, or on the unsafe side if the shear-induced pore pressures are positive. For a more complete discussion of procedures for estimating pore water pressures resulting from sudden drawdown, consult Duncan, Wright, and Wong (1990) and Wright and Duncan (1987).

2-6. Analyses of Stability during Earthquakes

An Engineer Circular, “Dynamic Analysis of Embankment Dams,” still in draft form, will provide guidance concerning types of analyses and design criteria for earthquake loading.

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.

3-2. New Embankment Dams

a. Earth and rock-fill dams. Minimum required factors of safety for design of new earth and rock-fill dams are given in Table 3-1. Criteria and procedures for conducting each analysis condition are found in Chapter 2 and the appendices. The factors of safety in Table 3-1 are based on USACE practice, which includes established methodology with regard to subsurface investigations, drilling and sampling, laboratory testing, field testing, and data interpretation.

b. Embankment cofferdams. Cofferdams are usually temporary structures, but may also be incorporated into a final earth dam cross section. For temporary structures, stability computations only must be performed when the consequences of failure are serious. For cofferdams that become part of the final cross section of a new embankment dam, stability computations should be performed in the same manner as for new embankment dams.

3-3. Existing Embankment Dams

a. Need for reevaluation of stability. While the purpose of this manual is to provide guidance for correct use of analysis procedures, the use of slope stability analysis must be held in proper perspective. There is danger in relying too heavily on slope stability analyses for existing dams. Appropriate emphasis must be placed on the often difficult task of establishing the true nature of the behavior of the dam through field investigations and research into the historical design, construction records, and observed performance of the embankment. In many instances monitoring and evaluation of instrumentation are the keys to meaningful assessment of stability. Nevertheless, stability analyses do provide a useful tool for assessing the stability of existing dams. Stability analyses are essential for evaluating remedial measures that involve changes in dam cross sections.

(1) New stability analysis may be necessary for existing dams, particularly for older structures that did not have full advantage of modern state-of-the-art design methods. Where stability is in question, stability should be reevaluated using analysis procedures such as Spencer's method, which satisfy all conditions of equilibrium.

(2) With the force equilibrium procedures used for design analyses of many older dams, the calculated factor of safety is affected by the assumed side force inclination. The calculated factor of safety from these procedures may be in error, too high or too low, depending upon the assumptions made.

b. Analysis conditions. It is not necessary to analyze end-of-construction stability for existing dams unless the cross section is modified. Long-term stability under steady-state seepage conditions (maximum storage pool and maximum surcharge pool), and rapid drawdown should be evaluated if the analyses performed for design appear questionable. The potential for slides in the embankment or abutment slope that could block the outlet works should also be evaluated. Guidance for earthquake loading is provided in ER 1110-2-1806, and an Engineer Circular, "Dynamic Analysis of Embankment Dams," is in draft form.

c. Factors of safety. Acceptable values of factors of safety for existing dams may be less than those for design of new dams, considering the benefits of being able to observe the actual performance of the embankment over a period of time. In selecting appropriate factors of safety for existing dam slopes, the considerations discussed in Section 3-1 should be taken into account. The factor of safety required will have an effect on determining whether or not remediation of the dam slope is necessary. Reliability analysis techniques can be used to provide additional insight into appropriate factors of safety and the necessity for remediation.

3-4. Other Slopes

a. Factors of safety. Factors of safety for slopes other than the slopes of dams should be selected consistent with the uncertainty involved in the parameters such as shear strength and pore water pressures that affect the calculated value of factor of safety and the consequences of failure. When the uncertainty and the consequences of failure are both small, it is acceptable to use small factors of safety, on the order of 1.3 or even smaller in some circumstances. When the uncertainties or the consequences of failure increase, larger factors of safety are necessary. Large uncertainties coupled with large consequences of failure represent an unacceptable condition, no matter what the calculated value of the factor of safety. The values of factor of safety listed in Table 3-1 provide guidance but are not prescribed for slopes other than the slopes of new embankment dams. Typical minimum acceptable values of factor of safety are about 1.3 for end of construction and multistage loading, 1.5 for normal long-term loading conditions, and 1.1 to 1.3 for rapid drawdown in cases where rapid drawdown represents an infrequent loading condition. In cases where rapid drawdown represents a frequent loading condition, as in pumped storage projects, the factor of safety should be higher.

b. Levees. Design of levees is governed by EM 1110-2-1913. Stability analyses of levees and their foundations should be performed following the principles set forth in this manual. The factors of safety listed in Table 3-1 provide guidance for levee slope stability, but the values listed are not required.

c. Other embankment slopes. The analysis procedures described in this manual are applicable to other types of embankments, including highway embankments, railway embankments, retention dikes, stockpiles, fill slopes of navigation channels, river banks in fill, breakwaters, jetties, and sea walls.

(1) The factor of safety of an embankment slope generally decreases as the embankment is raised, the slopes become higher, and the load on the foundation increases. As a result, the end of construction usually represents the critical short-term (undrained) loading condition for embankments, unless the embankment is built in stages. For embankments built in stages, the end of any stage may represent the most critical short-term condition. With time following completion of the embankment, the factor of safety against undrained failure will increase because of the consolidation of foundation soils and dissipation of construction pore pressures in the embankment fill.

(2) Water ponded against a submerged or partially submerged slope provides a stabilizing load on the slope. The possibility of low water events and rapid drawdown should be considered.

d. Excavated slopes. The analysis procedures described in this manual are applicable to excavated slopes, including foundation excavations, excavated navigation and river channel slopes, and sea walls.

(1) In principle, the stability of excavation slopes should be evaluated for both the end-of-construction and the long-term conditions. The long-term condition is usually critical. The stability of an excavated slope decreases with time after construction as pore water pressures increase and the soils within the slope swell and become weaker. As a result, the critical condition for stability of excavated slopes is normally the long-term condition, when increase in pore water pressure and swelling and weakening of soils is complete. If the materials in which the excavation is made are so highly permeable that these changes occur completely as construction proceeds, the end-of-construction and the long-term conditions are the same. These considerations lead to the conclusion that an excavation that would be stable in the long-term condition would also be stable at the end of construction.

(2) In the case of soils with very low permeability and an excavation that will only be open temporarily, the long-term (fully drained) condition may never be established. In such cases, it may be possible to excavate a slope that would be stable temporarily but would not be stable in the long term. Design for such a

condition may be possible if sufficiently detailed studies are made for design, if construction delays are unlikely, and if the observational method is used to confirm the design in the field. Such a condition, where the long-term condition is unstable, is inherently dangerous and should only be allowed where careful studies are done, where the benefits justify the risk of instability, and where failures are not life-threatening.

(3) Instability of excavated slopes is often related to high internal water pressures associated with wet weather periods. It is appropriate to analyze such conditions as long-term steady-state seepage conditions, using drained strengths and the highest probable position of the piezometric surface within the slope. For submerged and partially submerged slopes, the possibility of low water events and rapid drawdown should be considered.

e. Natural slopes. The analysis procedures in this manual are applicable to natural slopes, including valley slopes and natural river banks. They are also applicable to back-analysis of landslides in soil and soft rock for the purpose of evaluating shear strengths and/or piezometric levels, and analysis of landslide stabilization measures.

(1) Instability of natural slopes is often related to high internal water pressures associated with wet weather periods. It is appropriate to analyze such conditions as long-term, steady-state seepage conditions, using drained strengths and the highest probable position of the piezometric surface within the slope. For submerged and partially submerged slopes, the possibility of low water events and rapid drawdown should be considered.

(2) Riverbanks are subject to fluctuations in water level, and consideration of rapid drawdown is therefore of prime importance. In many cases, river bank slopes are marginally stable as a result of bank seepage, drawdown, or river current erosion removing or undercutting the toe of the slope.

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.

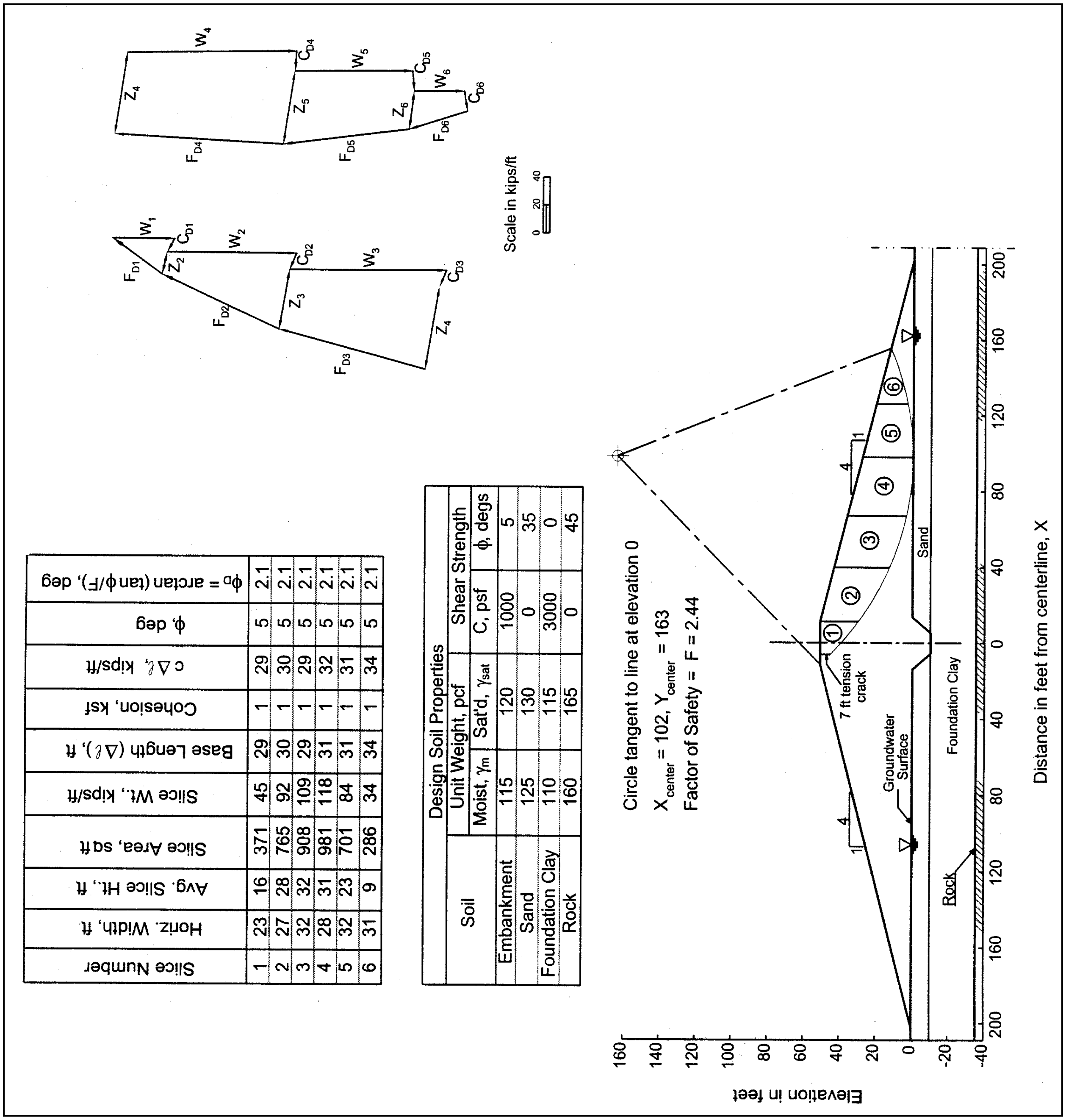
d. Essential requirements for appropriate use of computer programs. A thorough knowledge of the capabilities of the software and knowledge of the theory of limit equilibrium slope stability analysis methods will allow the user to determine if the software available is appropriate for the problem being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



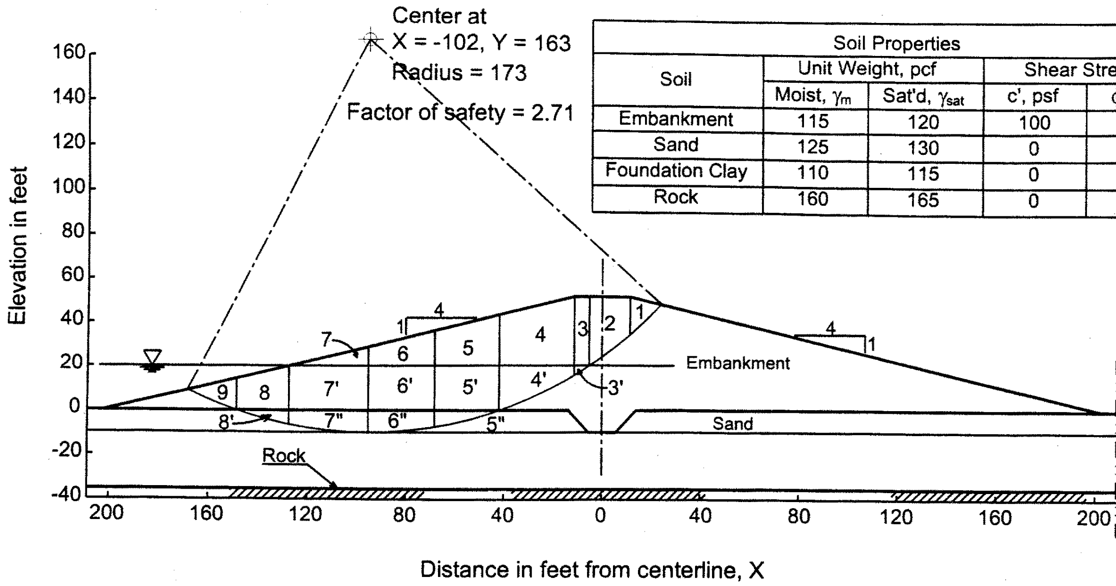
Slice Number	Horiz. Width, ft	Avg. Slice Ht., ft	Slice Area, sq ft	Slice Wt., kips/ft	Base Length ($\Delta\ell$), ft	Cohesion, ksf	$c\Delta\ell$, kips/ft	ϕ, deg	ϕ_D = arctan (tan ϕ/F), deg
1	23	16	371	45	29	1	29	5	2.1
2	27	28	765	92	30	1	30	5	2.1
3	32	32	908	109	29	1	29	5	2.1
4	28	31	981	118	31	1	32	5	2.1
5	32	23	701	84	31	1	31	5	2.1
6	31	9	286	34	34	1	34	5	2.1

Design Soil Properties				
Soil	Unit Weight, pcf		Shear Strength	
	Moist, γ_m	Sat'd, γ_{sat}	C, psf	ϕ, degs
Embankment	115	120	1000	5
Sand	125	130	0	35
Foundation Clay	110	115	3000	0
Rock	160	165	0	45

Figure 4-1. Hand verification using force equilibrium procedure to check stability computations performed via Spencer's Method – end-of-construction conditions

(a) During an automatic search, the program should not permit the search to jump from one face of the slope to another. If the initial trial slip surface is for the left face of the slope, slip surfaces on the right face of the slope should be rejected.

(b) In some cases, a slope may have several locally critical circles. The center of each such locally critical circle is surrounded by centers of circles that have higher values for the factor of safety. In such cases, when a search is performed, only one of the locally critical circles will be searched out, and the circle found may not be the one with the overall lowest factor of safety. To locate the overall critical

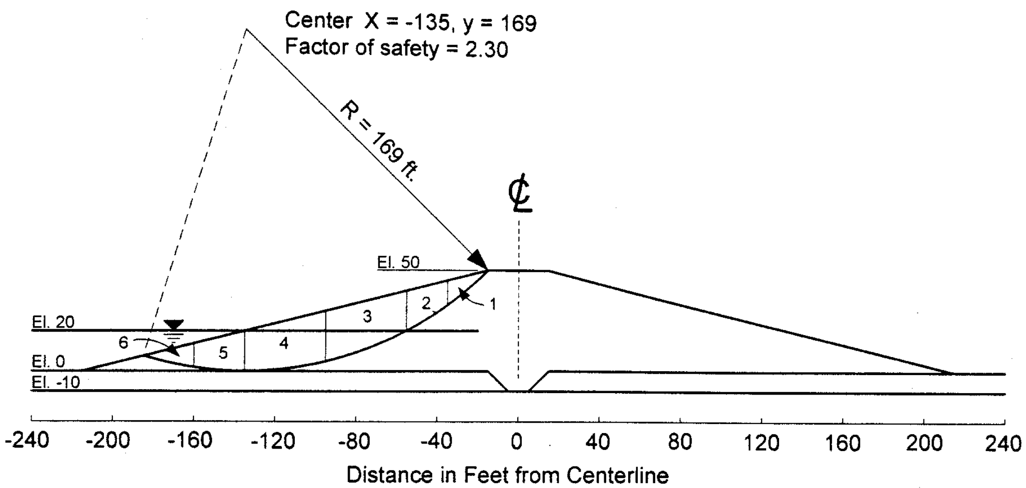


Soil Properties				
Soil	Unit Weight, pcf		Shear Strength	
	Moist, γ_m	Sat'd, γ_{sat}	c', psf	ϕ', degs
Embankment	115	120	100	25
Sand	125	130	0	35
Foundation Clay	110	115	0	28
Rock	160	165	0	45

Slice Number	Horizontal Width (b), ft	Average Slice Height, ft	Slice Area, sq. ft	Total Unit Weight (γ), pcf	Partial Weights, kips	Total Weight (W), kips	Base Inclination (α), deg	W sin(α)	Ht. of Surface Water, ft	Avg. Surface Press., ksf	Base Inclination (α), deg	Surface Load (P), kips/ft	Horiz Moment Arm (d_h), ft	Vert Moment Arm (d_v), ft	Moment (MP), kip-ft	Piezometric Height, ft	Pore Water Pressure (u), ksf	Cohesion (c), ksf	Friction Angle (ϕ), deg	c'b + (W + Pcosβ - ub)tanϕ	m_α(F=2.71)	[c'b+(W + Pcos β - ub)tan ϕ)] m_α
1	11	7	83	115	10	10	45	7	-	-	-	-	-	-	-	0	0.00	0.1	25	6	0.83	7
2	20	22	436	115	50	50	38	31	-	-	-	-	-	-	-	0	0.00	0.1	25	25	0.89	28
3 3`	10	30 3	311 34	115 120	36 4	40	32	21	-	-	-	-	-	-	-	3	0.20	0.1	25	19	0.94	20
4 4`	29	26 13	766 385	115 120	88 46	134	25	57	-	-	-	-	-	-	-	13	0.83	0.1	25	54	0.98	55
5 5` 5``	29	19 20 4	554 579 109	115 120 130	64 70 14	147	15	37	-	-	-	-	-	-	-	24	1.48	0	35	73	1.03	71
6 6` 6``	29	12 20 9	344 580 254	115 120 130	40 70 33	142	5	12	-	-	-	-	-	-	-	29	1.80	0	35	63	1.02	62
7 7` 7``	33	4 20 8	136 660 278	115 120 130	16 79 36	131	-6	-13	-	-	-	-	-	-	-	28	1.77	0	35	51	0.97	52
8 8`	25	17 3	421 85	120 130	51 11	62	-15	-16	3	0.19	14	5	45	146	399	23	1.46	0	35	21	0.90	23
9	20	7	139	120	17	17	-23	-7	9	0.55	14	11	68	152	1176	16	0.98	0.1	25	6	0.85	7
								129							1574							326

$$F = \frac{326}{129 - \frac{1574}{173}} = 2.71$$

Figure 4-2. Verification of computations using a spreadsheet for the Simplified Bishop Method – upstream slope, low pool



Material Properties

Soil	Cohesion	ϕ	Unit Weight
Embankment	200 psf	25 deg	115 pcf
Foundation Sand	0	35 deg	130 pcf
Foundation Clay	0	27 deg	115 pcf

Slice	Horizontal Width (b), ft	Avg. Slice Ht., ft	Slice Area, sq ft	Slice Weight (W), kips	Piezometric Ht.-Center, ft	Ht. of Surface Water, ft	Press. at top of Slice, ksf	Slope at Top of Slice, deg	Length of Top of Slice, deg	Surface Load (P), kips	Slice Base Length ($\Delta\ell$), ft	Pore Water Pressure (u), ksf	$U_b = u\Delta\ell$, kips	$c'\Delta\ell$, kips	$C_D = c'\Delta\ell/F$, kips	$\phi_D = \arctan(\tan(\phi')/F)$, deg
1	20	6	122	14	0	0	0	14	21	0	26	0.00	0	5.3	2.3	11.5
2	20	16	326	37	0	0	0	14	21	0	24	0.00	0	4.8	2.1	11.5
3	40	23	897	103	8	0	0	14	41	0	40	0.47	20	8.5	3.7	11.5
4	40	23	904	104	18	0	0	14	41	0	43	1.10	44	8.1	3.5	11.5
5	25	16	399	46	19	3	0.20	14	26	5	24	1.19	30	5.0	2.2	11.5
6	25	6	148	17	15	9	0.58	14	26	15	26	0.96	24	5.1	2.2	11.5

Figure 4-3. Hand verification of computations using the Modified Swedish Method – upstream slope, low pool (Part 1 of 2, computed forces)

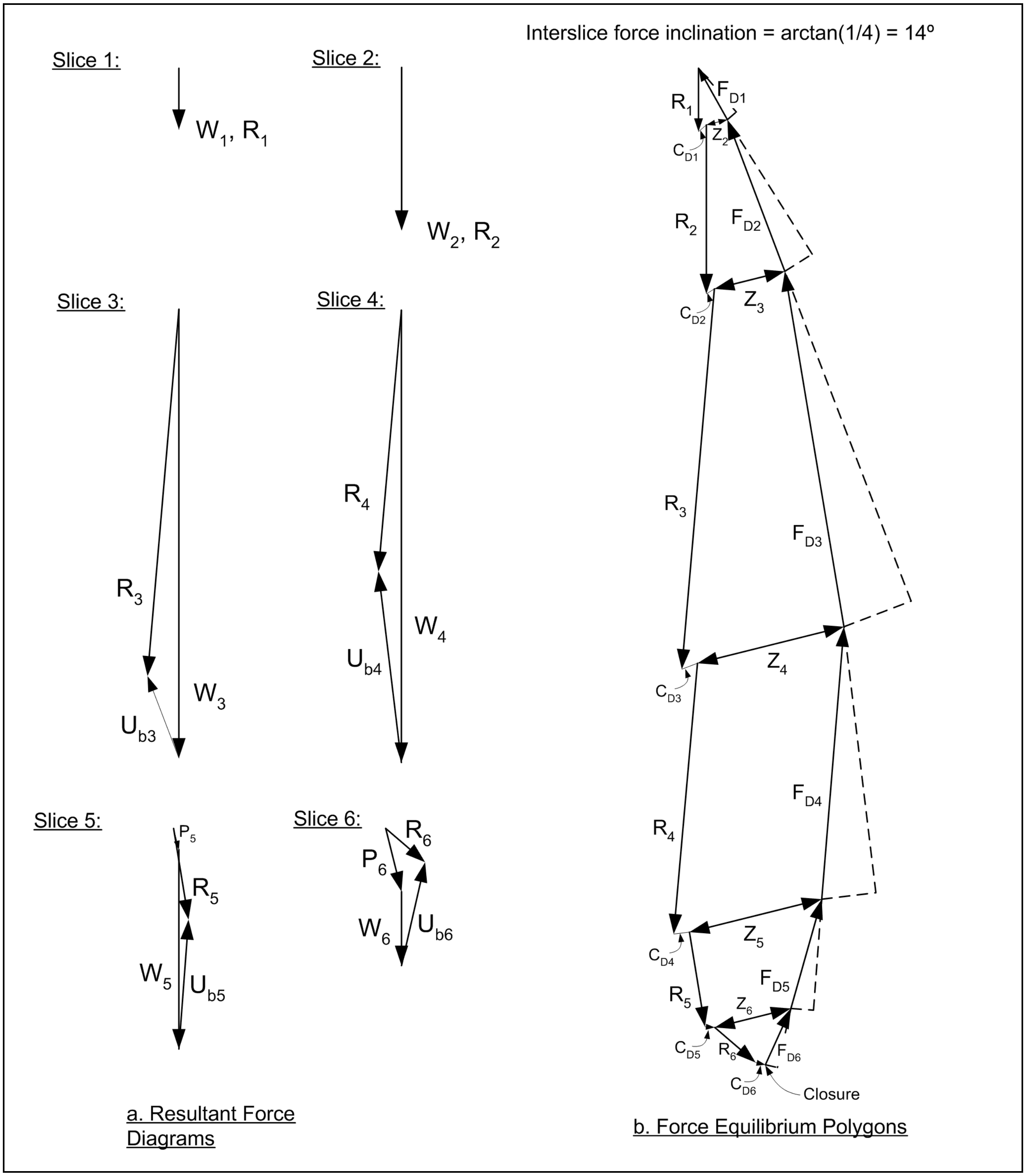


Figure 4-4. Hand verification of computations using the Modified Swedish Method (Part 2 of 2 – force polygons)

circle, several automatic searches should be performed using different starting points for the centers of the circles. The values of the factor of safety for each of the critical circles located by these independently started searches should then be compared by the user to determine the overall minimum factor of safety, and the location of the corresponding critical circle. This requires the user to perform several independently started searches for a given problem.

(c) An alternative approach is to perform analyses for a suite of circles with selected center points, and to vary the radii or depths of the circles for each center point. The computed factors of safety can be examined to determine the location of the most critical circle and the corresponding minimum factor of safety.

(2) Searches with noncircular slip surfaces. As with circular slip surfaces, various methods are used to search for critical noncircular slip surfaces. In all of these methods, the initial position of the slip surface is specified by the user and should correspond to the estimated position of the critical slip surface.

(a) In most methods of limit equilibrium slope stability analysis, the equilibrium equations used to compute the factor of safety may yield unrealistic values for the stresses near the toes of slip surfaces that are inclined upward at angles much steeper than those that would be logical based on considerations of passive earth pressure. Trial slip surfaces may become excessively steep in an automatic search unless some restriction is placed on their orientation.

(b) Because procedures for searching for critical noncircular slip surfaces have been developed more recently than those for circles, there is less experience with them. Thus, extra care and several trials may be required to select optimum values for the parameters that control the automatic search. The search parameters should be selected such that the search will result in an acceptably refined location for the most critical slip surface. The search parameters should be selected so that the final increments of distance used to shift the noncircular slip surface are no more than 10 to 25 percent of the thickness of the thinnest stratum through which the shear surface may pass.

4-3. Presentation of the Analysis and Results

a. Basic requirements. The description of the slope stability analysis should be concise, accurate, and self-supporting. The results and conclusions should be described clearly and should be supported by data.

b. Contents. It is recommended that the documentation of the stability analysis should include the items listed below. Some of the background information may be included by reference to other design documents. Essential content includes:

(1) Introduction.

(a) Scope. A brief description of the objectives of the analysis.

(b) Description of the project and any major issues or concerns that influence the analysis.

(c) References to engineering manuals, analysis procedures, and design guidance used in the analysis.

(2) Regional geology. Refer to the appropriate design memorandum, if published. If there is no previously published document on the regional geology, include a description of the regional geology to the extent that the regional geology is pertinent to the stability analysis.

(3) Site geology and subsurface explorations. Present detailed site geology including past and current exploration, drilling, and sampling activities. Present geologic maps and cross sections, in sufficient number and detail, to show clearly those features of the site that influence slope stability.

(4) Instrumentation and summary of data. Present and discuss any available instrumentation data for the site. Items of interest are piezometric data, subsurface movements observed with inclinometers, and surface movements.

(5) Field and laboratory test results.

(a) Show the location of samples on logs, plans, and cross sections.

(b) Present a summary of each laboratory test for each material, using approved forms as presented in EM 1110-2-1906, for laboratory soils testing.

(c) Show laboratory test reports for all materials. Examples are shown in Figures 4-5, 4-6, 4-7, 4-8, 4-9, and 4-10.

(d) Discuss any problems with sampling or testing of materials.

(e) Discuss the use of unique or special sampling or testing procedures.

(6) Design shear strengths. Present the design shear strength envelopes, accompanied by the shear strength envelopes developed from the individual test data for each material in the embankment, foundation, or slope, for each load condition analyzed. An example is shown in Figure 4-11.

(7) Material properties. Present the material properties for all the materials in the stability cross section, as shown in Figure 4-12. Explain how the assigned soil property values were obtained. In the case of an embankment, specify the location of the borrow area from which the embankment material is to be obtained. Discuss any factors regarding the borrow sites that would impact the material properties, especially the natural moisture content, and expected variations in the materials in the borrow area.

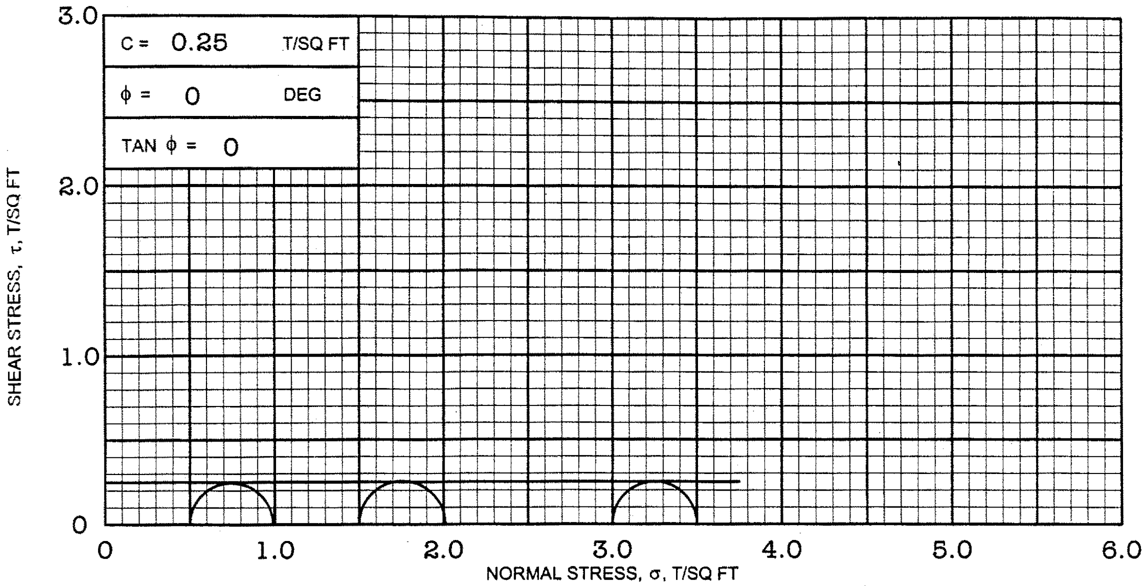
(8) Groundwater and seepage conditions. Present the pore water pressure information used in the stability analysis. Show the piezometric line(s) or discrete pore pressure points in the cross section used in the analysis, as shown in Figure 4-12. If the piezometric data are derived from a seepage analysis, include a summary of the seepage analysis in the report. Include all information used to determine the piezometric data, such as water surface levels in piezometers, artesian conditions at the site, excess pore water pressures measured, reservoir and river levels, and drawdown levels for rapid drawdown analysis.

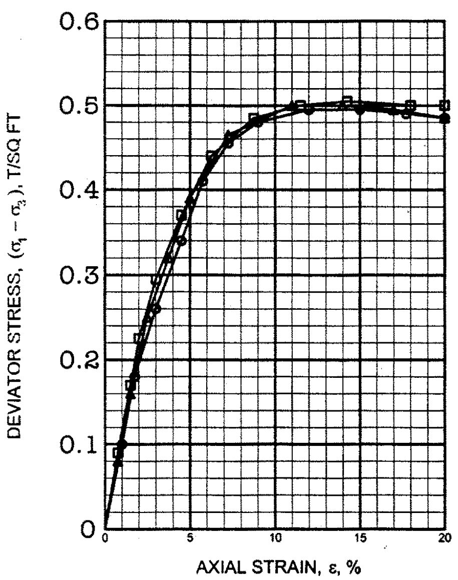
(9) Stability analyses.

(a) State the method used to perform the slope stability analysis, e.g., Spencer's Method in a given computer program, Modified Swedish Method using hand calculations with the graphical (force polygon) method, or slope stability charts. Provide the required computer software verification information described in Section 4-1.

(b) For each load condition, present a tabulation of material property values, show the cross section analyzed on one or more figures, and show the locations and the factors of safety for the critical and other significant slip surfaces, as shown in Figure 4-12. For circular slip surfaces, show the center point, including the coordinates, and the value of radius.

(c) For the critical slip surface for each load condition, describe how the factor of safety results were verified and include details of the verification procedure, as discussed previously.





SPECIMEN NO.			○ 1	□ 2	△ 3	
INITIAL	WATER CONTENT, %	w_o	28.9	29.3	31.6	
	DRY DENSITY LBS CU FT	γ_{d_o}	93.0	89.8	88.3	
	SATURATION, %	S_o	96.1	90.2	93.8	
	VOID RATIO	e_o	0.812	0.877	0.909	
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LBS CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE T/SQ FT	u_b				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.5	1.5	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1-\sigma_3)_{max}$	0.49	0.51	0.50	
TIME TO $(\sigma_1-\sigma_3)_{max}$, MIN		t_f	30	30	30	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1-\sigma_3)_{ult}$	0.485	0.485	0.500	
INITIAL DIAMETER, IN.		D_o	1.40	1.40	1.40	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

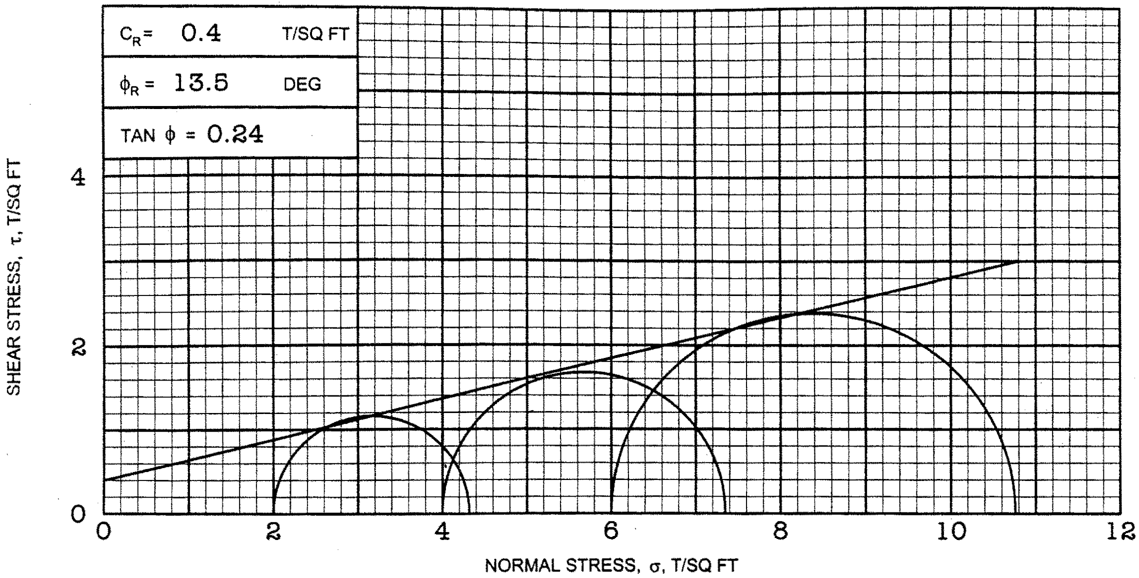
CONTROLLED - STRAIN TEST

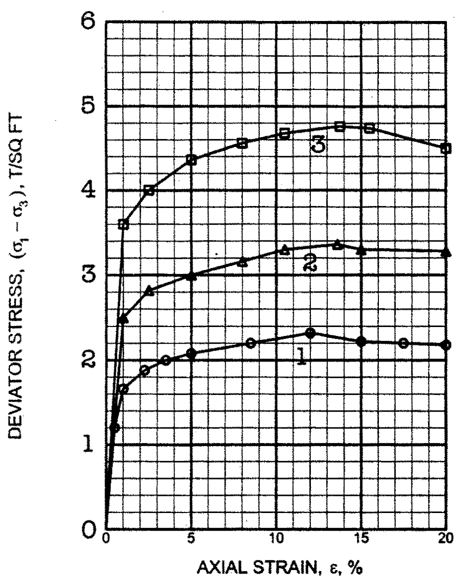
DESCRIPTION OF SPECIMENS SILTY CLAY (CL), GRAY

LL 28	PL 20	PI 8	Gs 2.70	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST Q

REMARKS	PROJECT YATESVILLE DAM	
	BORING NO. UD-84-5	SAMPLE NO. 3
	DEPTH/ELEV 16.0/17.5	
	LABORATORY ORD	DATE 21 AUG 84
	TRIAXIAL COMPRESSION TEST REPORT	

Figure 4-5. Triaxial compression test report for Q (unconsolidated-undrained) tests





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	27.3	28.3	28.0	
INITIAL	DRY DENSITY LBS CU FT	γ_{d_o}	98.2	94.6	95.9	
INITIAL	SATURATION, %	S_o	100	97.8	99.7	
INITIAL	VOID RATIO	e_o	0.717	0.782	0.758	
BEFORE SHEAR	WATER CONTENT, %	w_c	25.4	24.6	24.2	
BEFORE SHEAR	DRY DENSITY LBS CU FT	γ_{d_c}	103.9	103.4	105.0	
BEFORE SHEAR	SATURATION, %	S_c	100	100	100	
BEFORE SHEAR	VOID RATIO	e_c	0.622	0.630	0.605	
BEFORE SHEAR	FINAL BACK PRESSURE T/SQ FT	u_b	5.76	5.76	5.76	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	2.0	4.0	6.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	2.32	3.35	4.76	
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	793	913	920	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	2.18	3.28	4.50	
INITIAL DIAMETER, IN.		D_o	1.35	1.36	1.35	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

CONTROLLED - STRAIN TEST

DESCRIPTION OF SPECIMENS SILTY CLAY (CL), GRAY

LL 27	PL 17	PI 10	Gs 2.70	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R-BAR

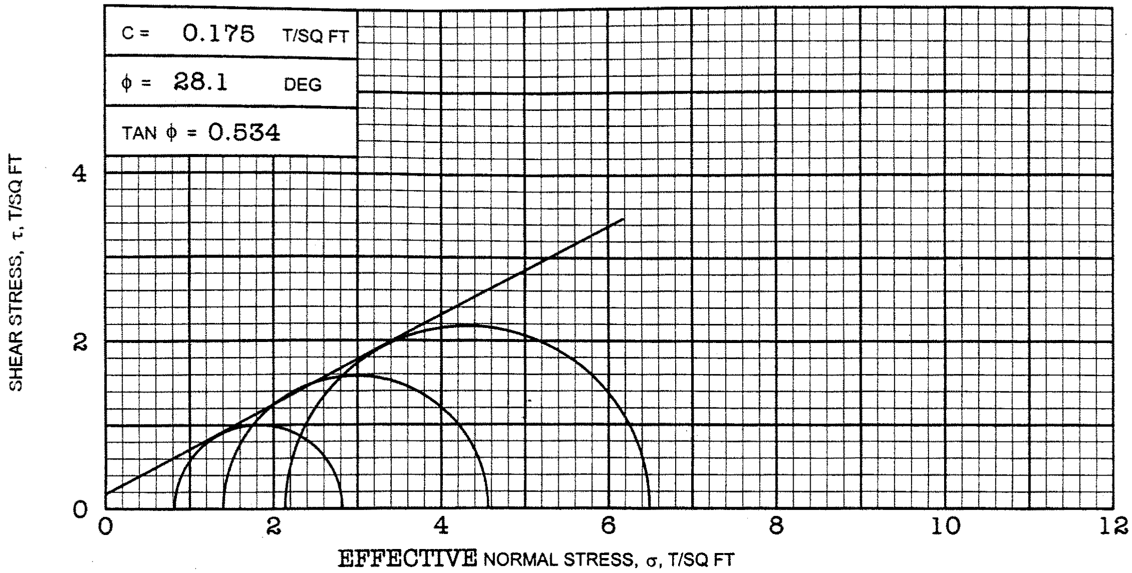
REMARKS

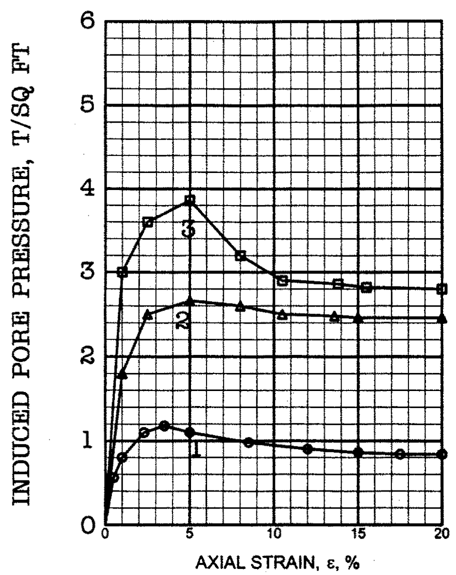
PROJECT YATESVILLE DAM	
BORING NO. UD-84-3	SAMPLE NO. 1
DEPTH/ELEV 18.0/19.8	
LABORATORY USAEWES	DATE 31 JULY 1984

SHEET 1 OF 3

TRIAXIAL COMPRESSION TEST REPORT

Figure 4-6. Triaxial compression test report for R-bar (consolidated-undrained) tests – total stress envelope





SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	27.3	28.3	28.0	
	DRY DENSITY LBS CU FT	γ_{d_o}	98.2	94.6	95.9	
	SATURATION, %	S_o	100	97.8	99.7	
	VOID RATIO	e_o	0.717	0.782	0.758	
BEFORE SHEAR	WATER CONTENT, %	w_c	25.4	24.6	24.2	
	DRY DENSITY LBS CU FT	γ_{d_c}	103.9	103.4	105.0	
	SATURATION, %	S_c	100	100	100	
	VOID RATIO	e_c	0.622	0.630	0.605	
	FINAL BACK PRESSURE T/SQ FT	u_b	5.76	5.76	5.76	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	0.82	1.40	2.14	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1-\sigma_3)_{max}$	2.00	3.16	4.35	
TIME TO $(\sigma_1-\sigma_3)_{max}$, MIN		t_f	793	913	920	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1-\sigma_3)_{ult}$	2.18	3.28	4.50	
INITIAL DIAMETER, IN.		D_o	1.35	1.36	1.35	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

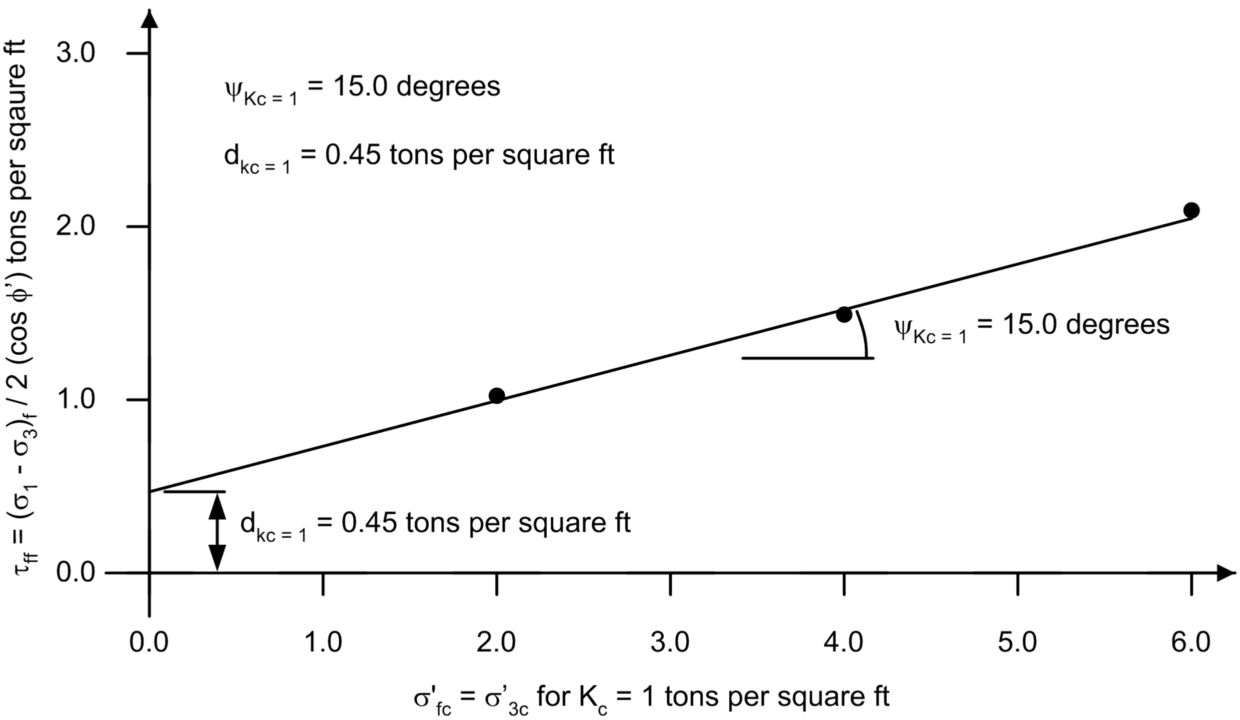
CONTROLLED - STRAIN TEST

DESCRIPTION OF SPECIMENS SILTY CLAY (CL), GRAY

LL 27	PL 17	PI 10	Gs 2.70	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R-BAR

REMARKS	PROJECT YATESVILLE DAM	
	BORING NO. UD-84-3	SAMPLE NO. 1
	DEPTH/ELEV 18.0/19.8	
	LABORATORY USAEWES	DATE 31 JULY 1984
SHEET 2 OF 3	TRIAXIAL COMPRESSION TEST REPORT	

Figure 4-7. Triaxial compression test report for R-bar (consolidated-undrained) tests – effective stress envelope

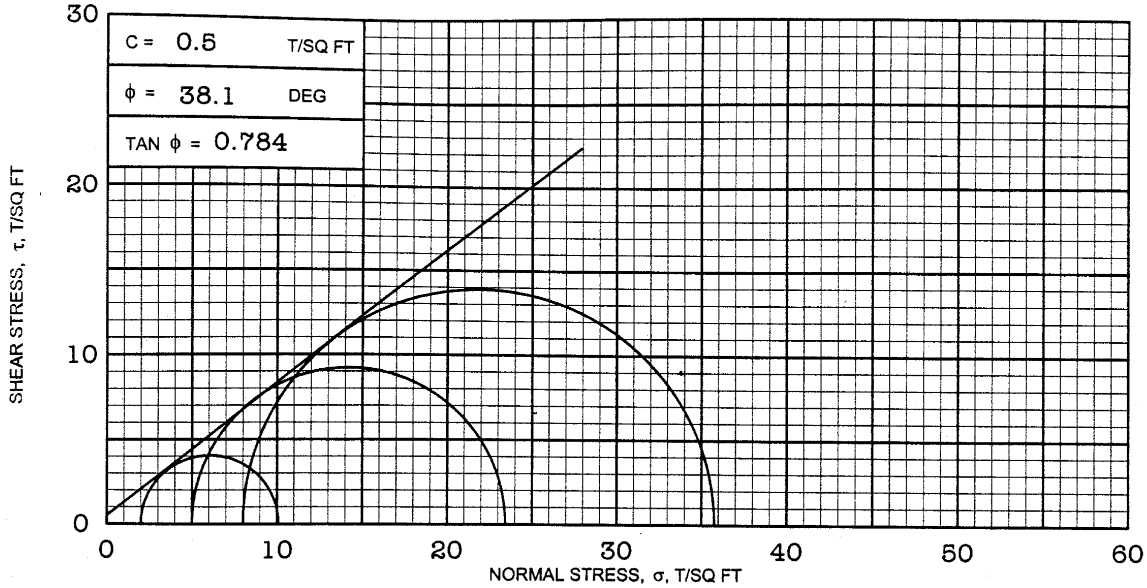


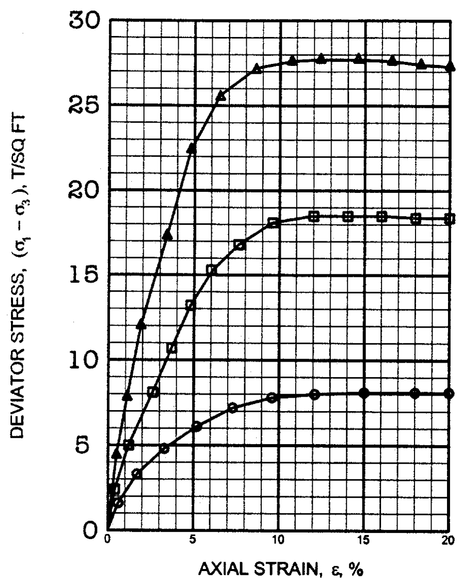
Test	$\sigma'_{fc} = \sigma'_{3c}$ (tsf)	$(\sigma_1 - \sigma_3)$ (tsf)	τ_{ff} (tsf)
1	2.00	2.32	1.02
2	4.00	3.35	1.48
3	6.00	4.76	2.10

Note: ϕ' = 28.1 degrees, cos ϕ' = 0.88

Sheet 3 of 3

Figure 4-8. Variation of τ_{ff} with σ'_{fc} for R-bar test with isotropic consolidation





SPECIMEN NO.			○ 1	□ 2	△ 3	
INITIAL	WATER CONTENT, %	w_o	4.9	4.4	4.5	
	DRY DENSITY LBS CU FT	γ_{d_o}	120.1	120.7	120.6	
	SATURATION, %	S_o	34.9	31.6	32.3	
	VOID RATIO	e_o	0.372	0.365	0.366	
BEFORE SHEAR	WATER CONTENT, %	w_c	12.9	12.4	11.1	
	DRY DENSITY LBS CU FT	γ_{d_c}	122.8	124.1	127.5	
	SATURATION, %	S_c	100	100	100	
	VOID RATIO	e_c	0.341	0.327	0.292	
	FINAL BACK PRESSURE T/SQ FT	u_b	4.00	4.00	4.00	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	2.00	5.00	8.00	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	8.09	18.44	27.76	
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	150	110	130	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	8.09	18.41	27.43	
INITIAL DIAMETER, IN.		D_o	14.88	14.88	14.88	
INITIAL HEIGHT, IN.		H_o	32.00	32.00	32.00	

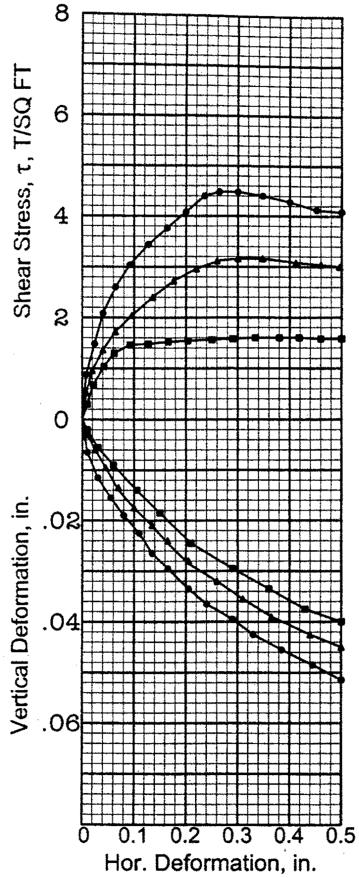
CONTROLLED - STRAIN (0.1%/min) TEST

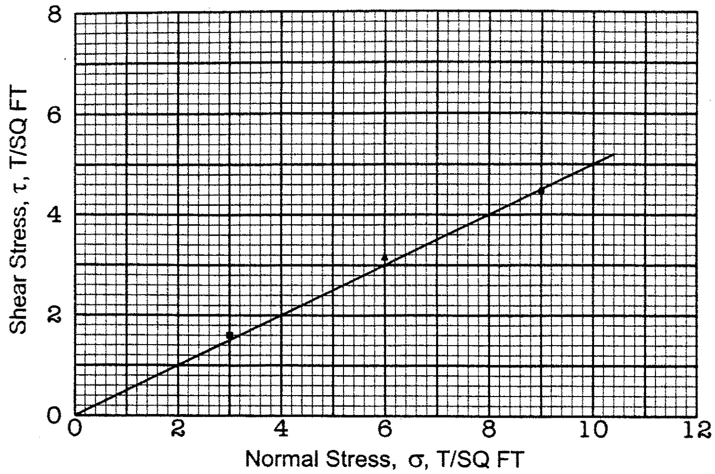
DESCRIPTION OF SPECIMENS Gray & tan poorly graded crusher run sandstone (GP)

LL 20	PL 18	PI 2	Gs 2.64	TYPE OF SPECIMEN REMOLDED	TYPE OF TEST S

REMARKS	PROJECT YATESVILLE DAM	
Specimens requested to be remolded	PROPOSED ROCK EXCAVATION - SANDSTONE	
at field moisture content to a dry	BORING NO. C-82, C-84, C-85	SAMPLE NO.
density of 120 pcf	DEPTH/ELEV 808.5 TO 758.8	
	LABORATORY SADEL	DATE 26 DEC 73
	TRIAXIAL COMPRESSION TEST REPORT	

Figure 4-9. Triaxial compression test report for S (drained) tests – effective stress envelope





Shear Strength Parameters

ϕ' = 27.0 deg

tan ϕ' = 0.510

c' = 0.0 T/sq ft

☐ Controlled stress

☒ Controlled strain

Test No.						
Initial	Water content	w_o	24.4%	24.4%	24.4%	
	Void ratio	e_o	0.769	0.709	0.779	
	Saturation	S_o	87%	95%	86%	
	Dry density lb/cu ft	γ_d	97.2	100.4	96.5	
Void ratio after consolidation		e_c	0.694	0.615	0.610	
Time for 50 percent consolidation, min		t_{50}	16	24	23	
Final	Water content	w_f	23.3%	26.1%	26.4	
	Void ratio	e_f	0.555	0.464	0.424	
	Saturation	S_f	100	100	100	
Normal stress T/sq ft		σ	3.0	6.0	9.0	
Maximum shear stress, T/sq ft		τ_{max}	1.60	3.16	4.47	
Actual time to failure, min		t_f	500	600	600	
Rate of displ., in/min			0.0003	0.0003	0.0003	
Ultimate shear stress, T/sq ft		τ_{ult}	1.58	3.00	4.08	

Type of specimen Undisturbed		3.25 in. square	0.50 in. thick
Classification Brown and Gray Lean CLAY (CL)			
LL 34.1	PL 20.9	PI 13.2	G_s 2.75
Remarks	Project Yatesville Dam		
	Area		
	Boring No. UD-1A	Sample No. 4	
	Depth/Elev 579.5-577.0	Date DEC 1973	
	DIRECT SHEAR TEST REPORT		

Figure 4-10. Direct shear test report – effective stress envelope

Boring	Sample	Depth - ft	Specimen	Moist density	Saturated density	C' tsf	φ' degrees
UD-2	1	42.0 - 44.0	1	126.0	129.9		
UD-2	1	42.0 - 44.0	2	125.4	127.0	0.0	27.7
UD-2	1	42.0 - 44.0	3	124.0	126.5		
UD-2	2	108 - 109.5	1	129.8	134.9		
UD-2	2	108 - 109.5	2	133.5	135.9	0.0	34.6
UD-2	2	108 - 109.5	3	128.6	134.0		
UD-4	3	58.0 - 59.5	1	122.3	124.4		
UD-4	3	58.0 - 59.5	2	121.6	123.9	0.0	25.7
UD-5	1	42.0 - 44.1	1	131.5	132.5		
UD-5	1	42.0 - 44.1	2	129.5	131.4	0.0	38.9
			Average	126.7	129.5	0.0	31.7

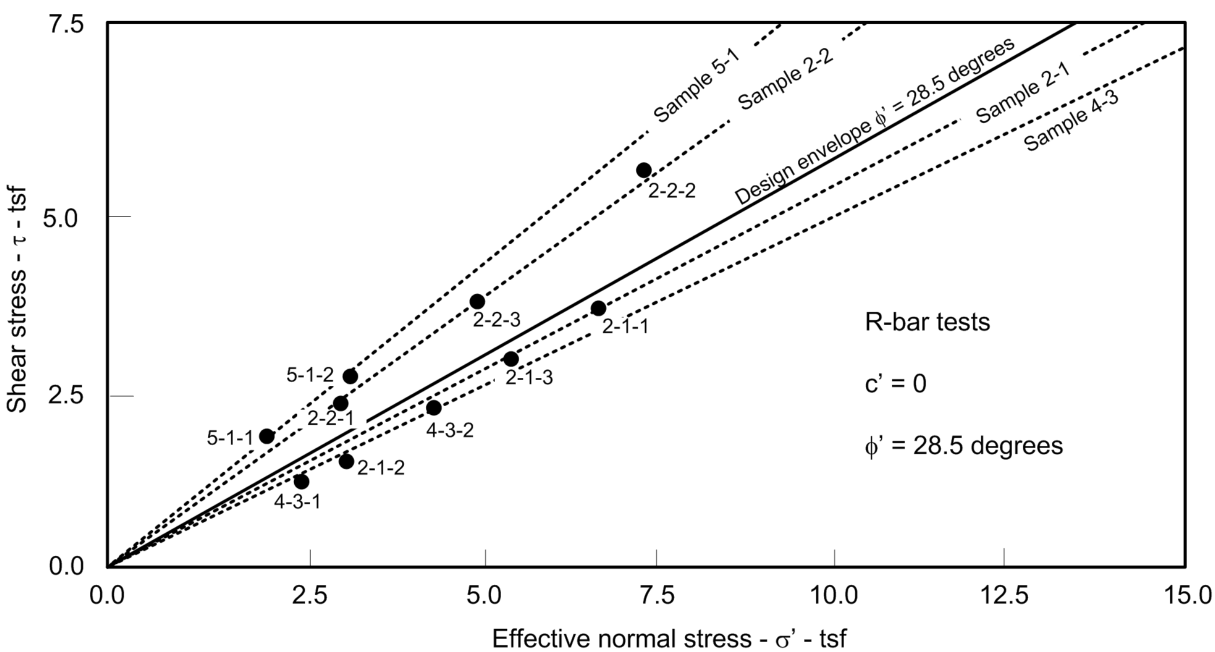


Figure 4-11. Presentation of design strength values

Critical circles			
Tangent elevation	X	Y	Factor of safety
815 ft.	182	1086	2.18 before drawdown 1.87 after drawdown
790 ft.	244	1074	2.15 before drawdown 1.64 after drawdown
775 ft.	280	1110	2.53 before drawdown 2.16 after drawdown

Soil Properties							
Mat'l	γ_{moist} pcf	γ_{sat} pcf	c' psi	ϕ' degs	$d_{K_c=1}$ psf	$\psi_{K_c=1}$ degs	Description
1	127	130	800	21	1700	13	Impervious core
2	132	136	400	18	1600	12	Random impervious
3	140	143	0	36	-	-	Pervious shell
4	114	127	0	28	-	-	Weathered rockfill
5	100	125	0	35	-	-	Proposed rockfill
6	135	140	0	33	-	-	Foundation outwash
7	-	130	0	27	-	-	Silty aluvium

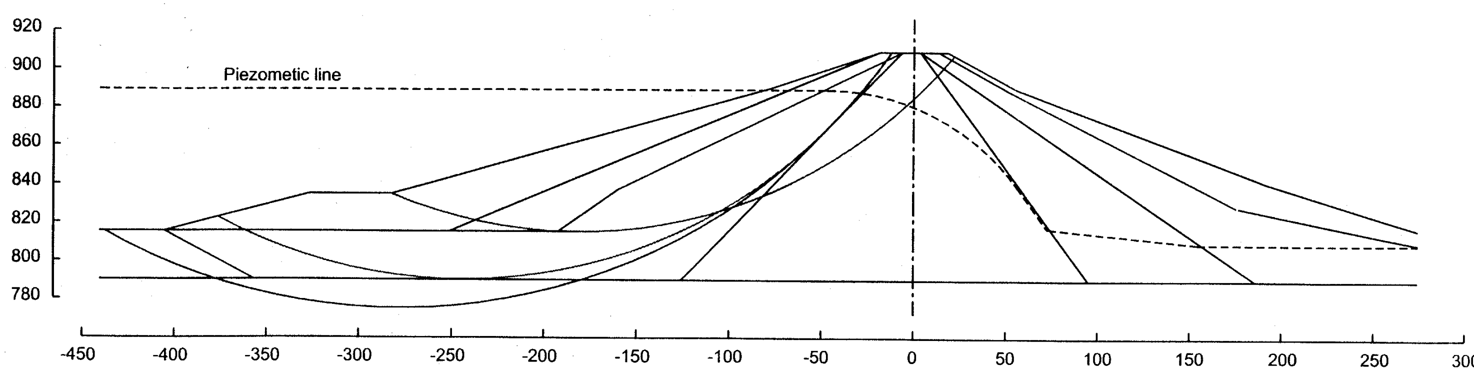


Figure 4-12. Presentation of slope stability analysis results