Chapter 2
Design Considerations

2-1. Introduction

Evaluation of slope stability requires:

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

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

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

2-2. Aspects Applicable to All Load Conditions

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

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

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

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

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

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

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

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

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

[1] Effective stress shear strength parameters can be obtained from consolidated-drained (CD, S) tests (direct shear or triaxial) or consolidated-undrained (CU, $\overline{R}$) 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

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

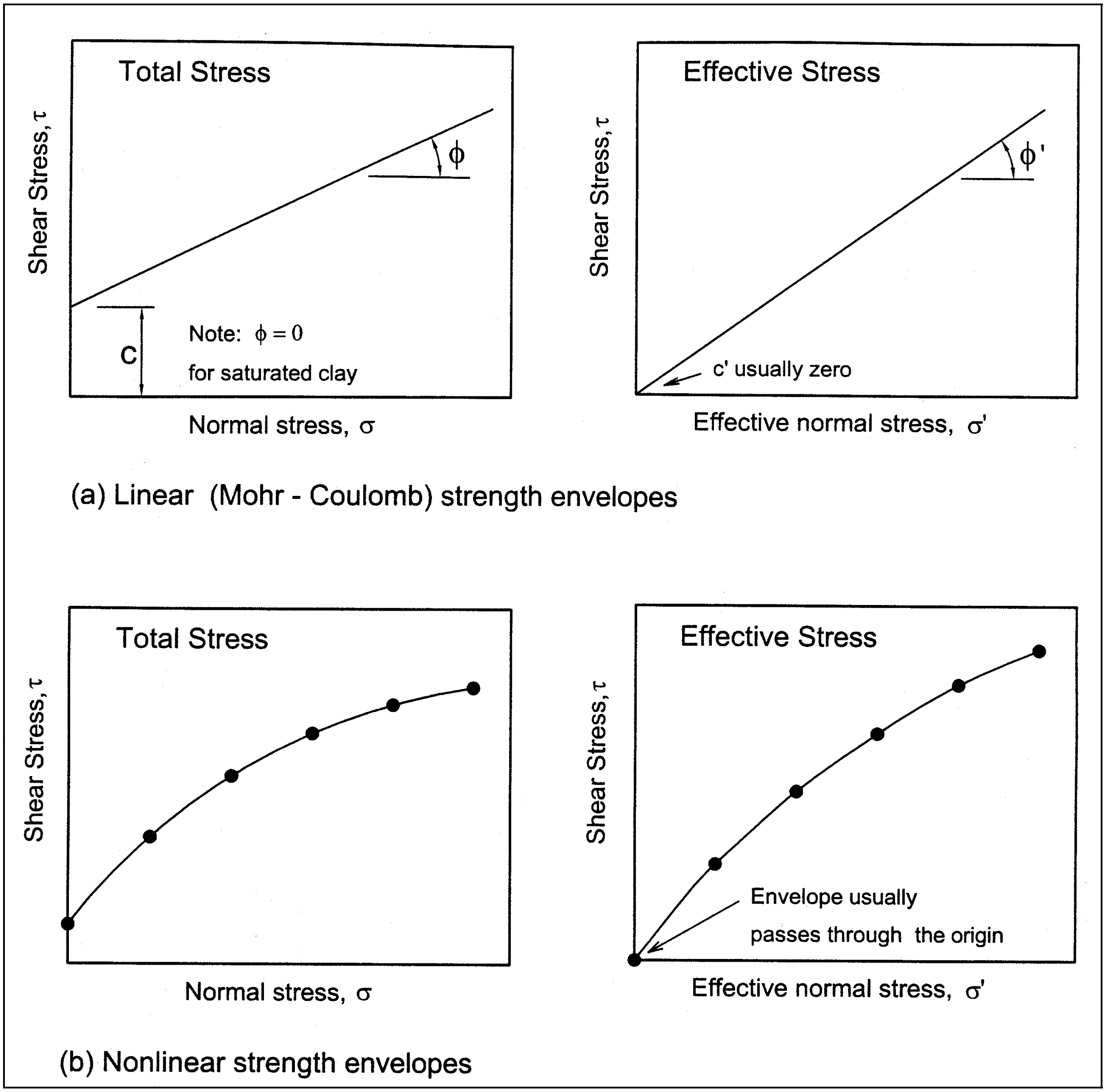

Figure 2-1. Strength envelopes for soils

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.

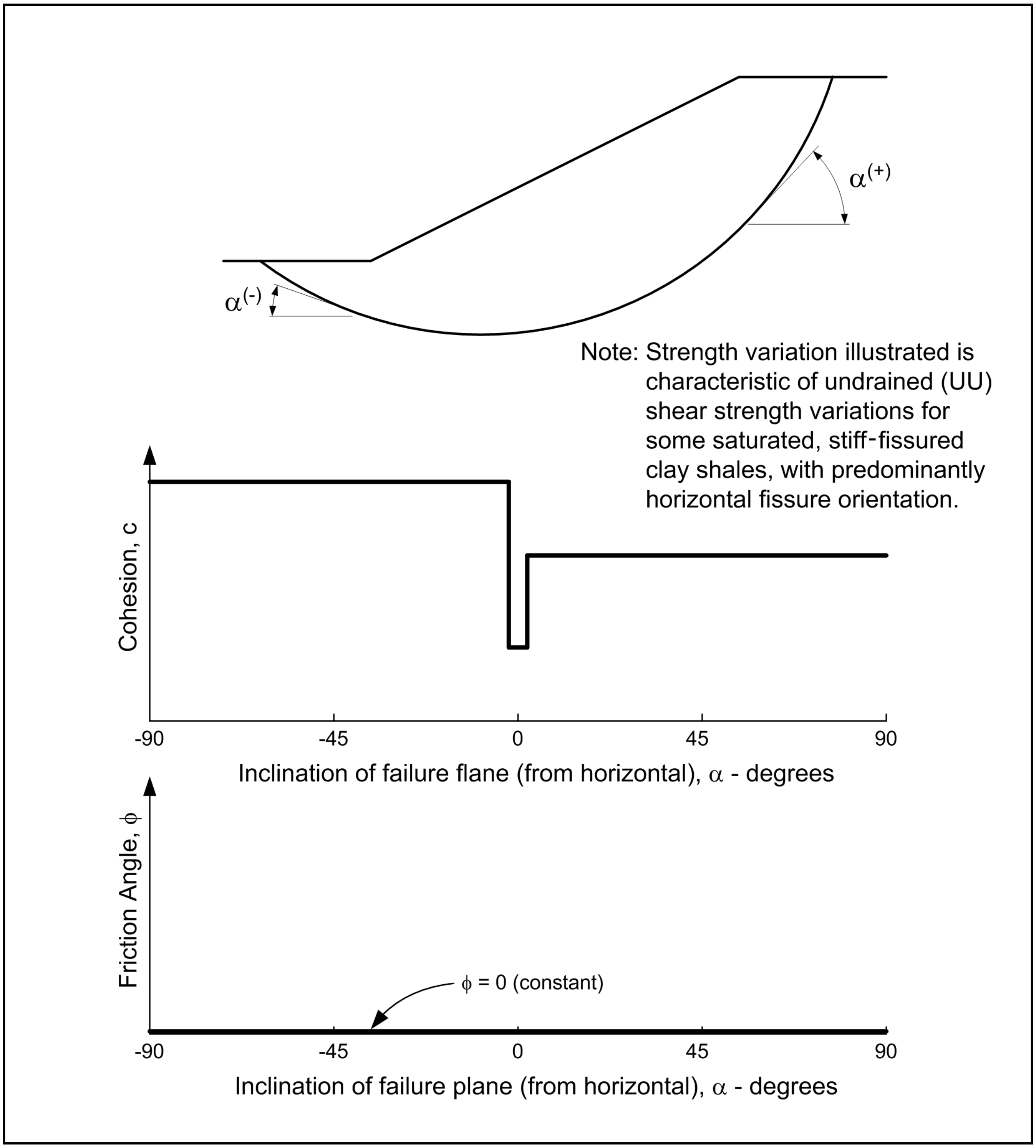

Figure 2-2. Representation of shear strength parameters for anisotropic soil

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:

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

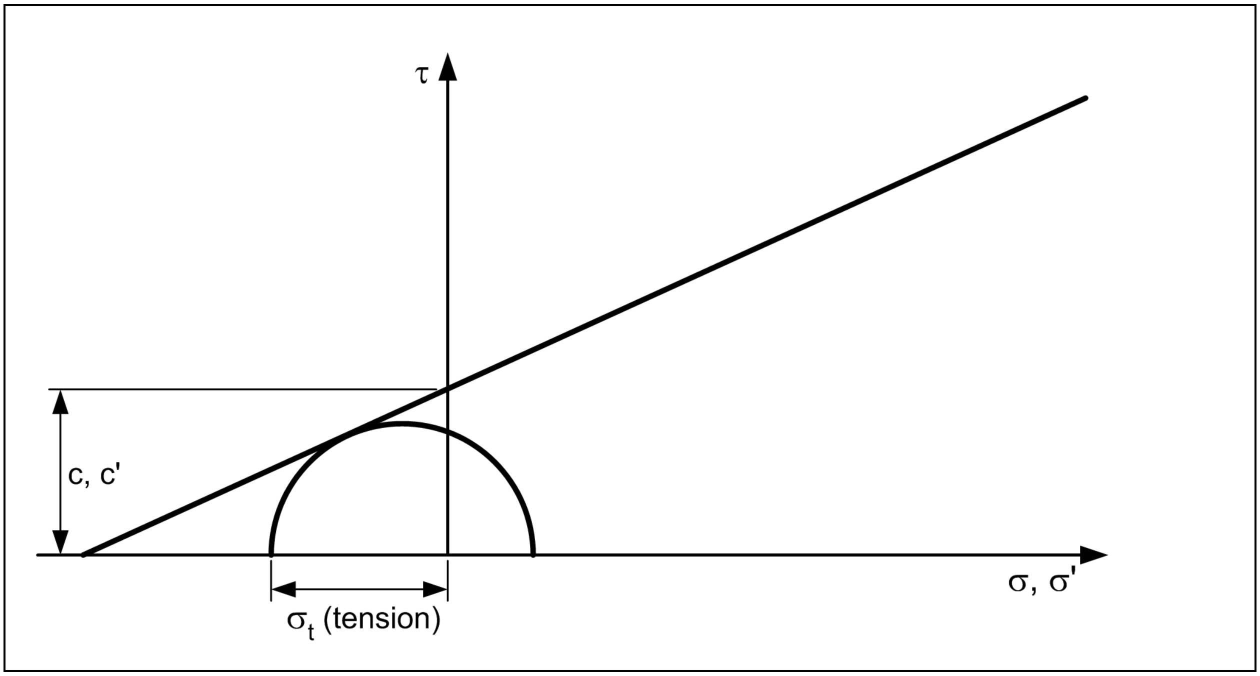

Figure 2-3. Tensile stresses resulting from a Mohr-Coulomb failure envelope with a cohesion intercept

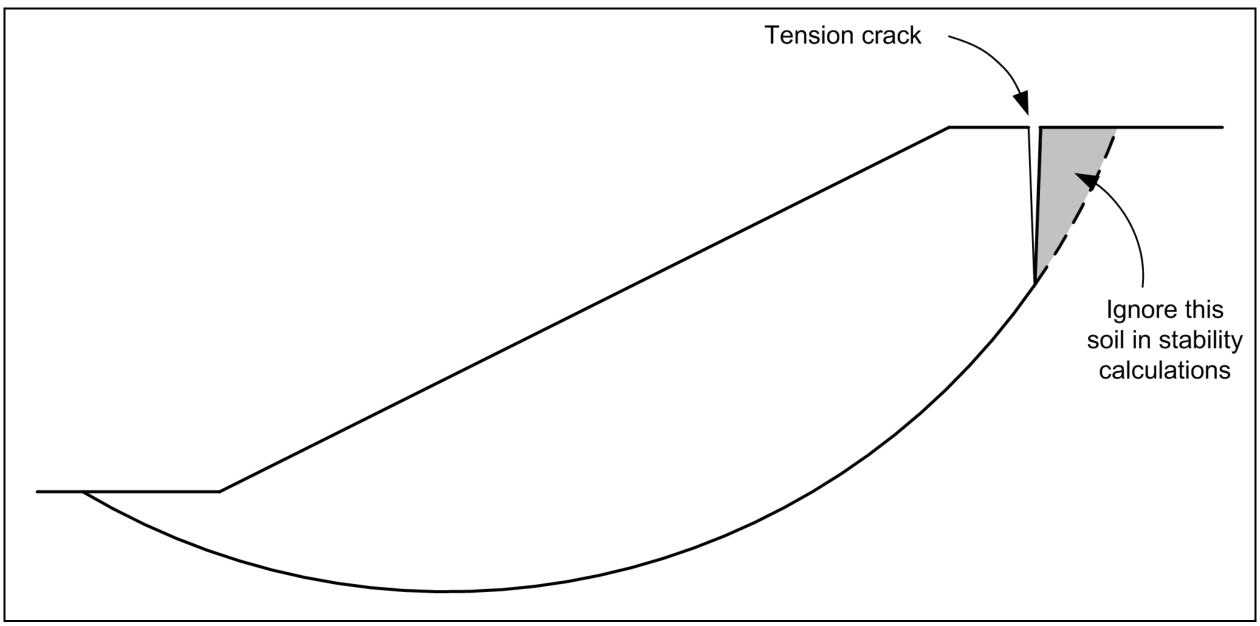

Figure 2-4. Vertical tension crack introduced to avoid tensile stresses in cohesive soils

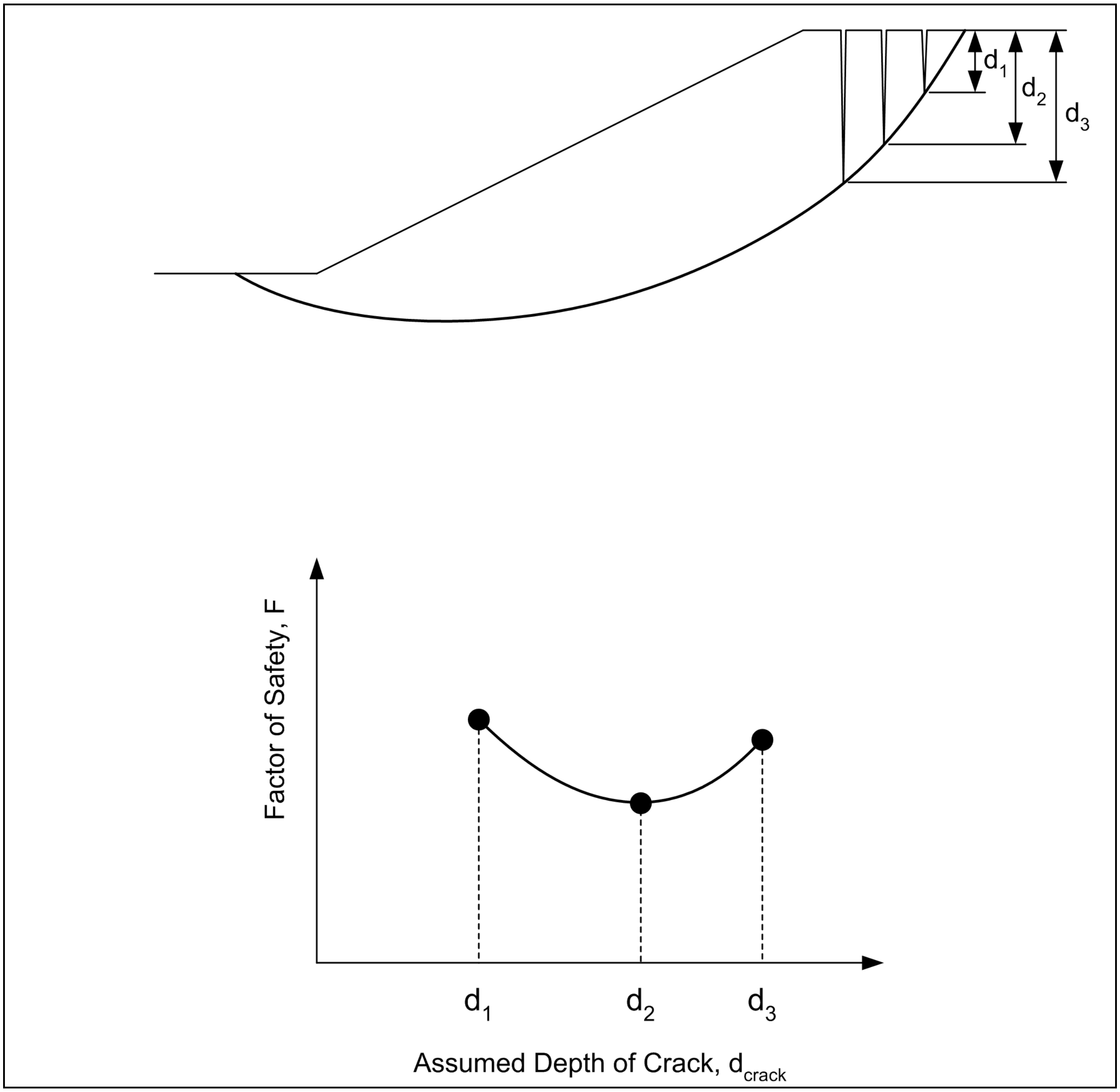

Figure 2-5. Variation in the factor of safety with the assumed depth of vertical crack

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

The developed shear strength parameters are expressed by:

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

and

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

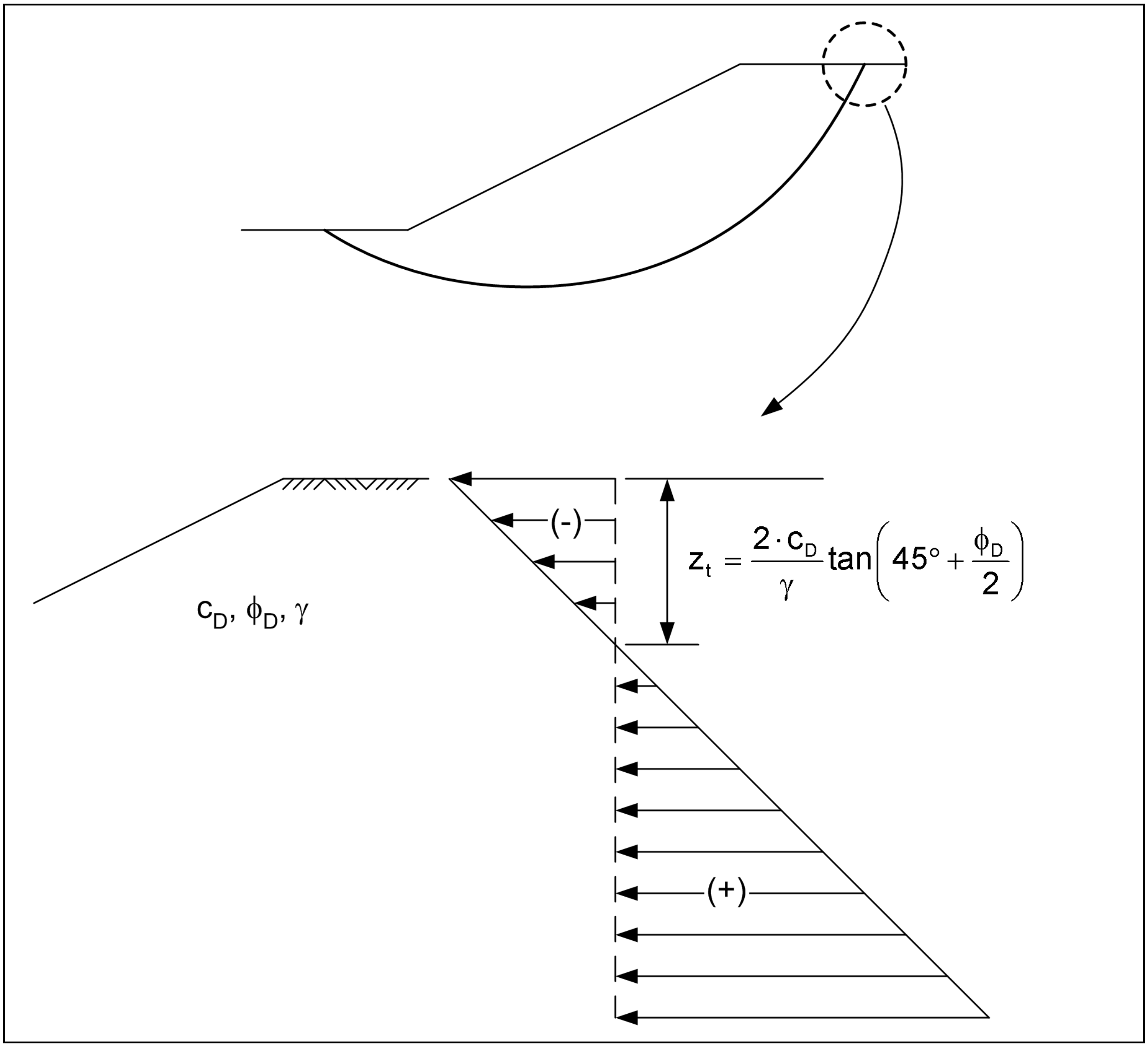

Figure 2-6. Horizontal stresses near the crest of the slope according to Rankine active earth pressure theory

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

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

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

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

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

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

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

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

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

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

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

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

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

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

2-4. Analyses of Steady-State Seepage Conditions

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

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

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

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

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

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

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

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

(2) Past experience and judgement.

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

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

2-5. Analyses of Sudden Drawdown Stability

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

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

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

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.