December 15, 1975

MEMORANDUM TO: J. R. Harbison
State Highway Engineer
Chairman, Research Committee

SUBJECT: Research Report No. 435; "Effects of Water on Slope Stability;" KYHPR-68-48, HPR-1(9), Part II; and KYP-72-38, HPR-PL-1(11), Part III B.

This is the nineteenth report dedicated to stability of embankments since 1962; fourteen addressed case histories, analyses, and remedies; additionally, four memorandum reports presented case analyses and proposed remedies; numerous fill failures have been inspected and impromptu recommendations offered. Three of the reports involved basic properties of soils, and one provided an advanced type of computer program for stability analyses. A forthcoming report will provide another, more encompassing program.

At the inception of KYHPR-68-48, eight landslides were selected for intensive investigation; data were compiled and narratives were written. The case histories were found to be somewhat unwieldy and not summarily definitive for the purpose intended. Consequently, the final report on Study KYHPR-68-48 was deferred. In the interim, through KYP-72-38, other information was merged; and we are now submitting a far more comprehensive report. The readership of such a report will likely be limited to soils engineers. R. C. Deen presented the report at the Ohio River Valley Soils Seminar in October and received many complimentary comments with respect to the depth of the research and originality. The discoveries regarding factors of safety are very significant.

The report will be forwarded to FHWA as final submission under KYHPR-68-48; the case histories will be retained in the files.

Respectfully submitted,

Jas. H. Havens
Director of Research

JHH:gd
Attachment
cc's: Research Committee

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16. Abstract

A brief state-of-the-art review of the effects of water on slope stability and the techniques for analysis is presented. The effective stress principle and basic considerations of slope stability, including design factors of safety, are discussed briefly. The derivations and effects of seepage forces and rapid drawdown on effective stress are also presented. Various conditions of external loading produce changes in effective stress. These changes are discussed and limiting conditions which should be analyzed are mentioned.

Limitations of total stress analyses are discussed in detail. It appears that, for soils having a liquidity index of 0.36 or greater (normally consolidated), the undrained shear strength gives factors of safety close to the actual factor of safety. For soils with a liquidity index less than 0.36 (overconsolidated), the undrained shear strength gives factors of safety that are too high; but the strength parameters can be corrected by the empirical relationship presented herein. Data also show that the difference between vane and calculated shear strength increased as the plasticity index and(or) the liquid limit increased. An empirical relationship for correcting vane shear strength is presented.

A discussion of effective stress analysis, including differences between peak and residual ϕ angles for normally consolidated and overconsolidated soils, is presented. The residual ϕ angle decreases logarithmically with increasing clay fraction. The "critical" state of a clay is also defined. Shear strength parameters of a clay tested in that state correspond to the theoretical strength of an overconsolidated clay which has undergone a process of softening. To test a clay in the critical state, it is suggested herein the soil should be remolded to a moisture content equal to 0.36 times the plastic index plus the plastic limit.

Water may cause unstable conditions in earth slopes due to changes in geometry. Erosion of the toe or the slope can induce damaging stress. Piping through heaving or erosion of subsurface layers can cause damage. Construction of side-hill embankments can cause damming, resulting in a rise in the water table.

Methods of water detection are also summarized. These include tracers, electrical resistivity, and water table observations. The latter method apparently is the most successful. A discussion of ways to monitor water pressures, including the types and operations of piezometers, is given. Finally, suggested guidelines for the design of earth slopes are included.

17. Key Words			18. Distribution Statement
Effective Stress Total Stress Slope Stability Liquidity Index Critical State	Shear Strength Design Erosion Damming Piping Water Detection	Water Monitoring Clay	

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Research Report
435

EFFECTS OF WATER
ON SLOPE STABILITY

KYHPR-68-48; HPR-1(9), Part II
Final Report

by

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Federal Highway Administration
U. S. DEPARTMENT OF TRANSPORTATION

October 1975

INTRODUCTION

In the planning, design, construction, and maintenance of engineered structures, the engineer must be congnizant of potential problems that might be associated with the stability of man-made and natural slopes. To minimize the risk associated with the design and construction of slopes, a knowledge of the general setting is essential in the recognition of potential or actual landslides. The development of or potential for landslides is dependent to a large extent upon the character, stratigraphy, and structure of the underlying rocks and soils; on the topography, climate, and vegetation; and on surface and underground waters. These factors vary widely from place to place and their variations are reflected in the kinds and rates of landslide movements that result from their interactions.

This paper has been prepared as a review of the effects on slope stability of only one of these categories of the overall general setting of slope design. This report is a brief summary of the current state of the art of the effect of water on the stability of slopes and on the selection of techniques to analyze such stability considerations. Methods used for the detection and monitoring of water are also summarized.

BASIC CONSIDERATIONS

SHEAR STRENGTH AND EFFECTIVE STRESS PRINCIPLE

The oldest and most widely used expression for the shear strengths, s, of soils is the Coulomb failure criterion,

$$s = c + \sigma \tan \phi, \qquad 1$$

where c is the cohesion, σ is the total normal stress on the failure surface, and ϕ is the angle of shearing resistance. As expressed by this equation, the strength and deformation characteristics of soils are governed by the total stresses. The strength parameters c and ϕ may vary widely, depending upon test procedures. Thus, application of Coulomb's failure criterion is limited to conditions which duplicate those existing during the test in which ϕ and c are determined.

A more general failure criterion having greater application was introduced by Terzaghi [1]. The shear strength is considered to be a function of the effective normal stress (Figure 1),

$$\sigma' = \sigma - u, \qquad 2$$

where u is the pore water pressure, and is given by

$$s = c' + \sigma' \tan \phi'. \qquad 3$$

The strength components ϕ' and c' are now referred to as effective stress parameters. Equation 3 is commonly referred to as the Coulomb-Terzaghi failure criterion. Equation 2, it should be noted, is applicable only to saturated soils. For partially saturated soils, Equation 2 takes on a more complex form [2]. Pore water pressures in Equation 2 are due to the free pore water existing between soil particles. Pressures existing in the tightly bound pore water around soil particles are as yet unknown. Consequently, the effective stress principle as proposed by Terzaghi is perhaps semi-empirical [3]. Nevertheless, the practical validity of Equation 2 has been widely established.

SLOPE STABILITY

A complete review of methods for evaluating the stability of a slope is beyond the scope of this report. A comprehensive review of various slope stability methods has been presented elsewhere [4, 5]. However, a few remarks considered essential to this review are included herein to provide a complete summary of the state of the art.

Most techniques to analyze slope stability are based on the concept of limiting plastic equilibrium. Such an assumption is reasonable for low safety factors. Bishop [6] showed that local overstressing occurs when the safety factor (by a slip circle method) is less than 1.8. His analysis was for a typical earth dam using relaxation techniques and assuming idealized elastic behavior of the soil. Since most earth embankments have safety factors less than 1.8, Bishop reasoned that a state of plastic equilibrium must exist in at least part of the slope, and therefore the safety factor may be defined as the ratio of available shear strength of the soil to the strength required to maintain equilibrium (see Figure 1), or

$$\Sigma\tau = \Sigma(s/F) = \Sigma (c'/F) + \Sigma(\sigma - u) \tan \phi'/F, \qquad 4$$

where F is the safety factor. The parameters ϕ' and c' are obtained either from consolidated-drained triaxial tests or consolidated-undrained triaxial tests with pore pressure measurements. In terms of total stresses, Equation 4 becomes

$$\Sigma\tau = \Sigma(s/F) = \Sigma(s_u/F), \qquad 5$$

where s_u is the undrained strength of the soil. The undrained strength may be obtained from an unconsolidated-undrained triaxial test (or unconfined compression test). Regardless of whether the effective stress method (Equation 4) or total stress approach (Equation 5) is used in the stability analysis of a slope,

$$h_u = u/\gamma_w$$

EQUIPOTENTIAL LINE

σ τ
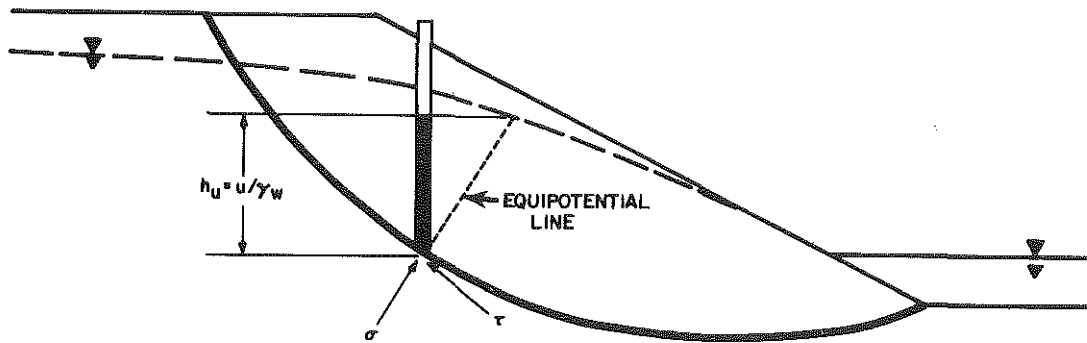
IN A STABLE SLOPE:

$$\Sigma \tau = \Sigma \frac{s}{F} = \Sigma \frac{c'}{F} + \Sigma \frac{(\sigma - u) \tan \phi'}{F}$$

AT FAILURE:

$$F = 1.0; \quad \Sigma \tau = \Sigma s$$

Figure 1.   Effective Stress Concepts and the Stability of Slopes.

the definition of the safety factor is the same -- the ratio of the available shear strength to the shear stress required for equilibrium.

When the safety factor is factored (or moved to the left of the summation sign) in either Equation 4 or 5, it is assumed that the safety factor is constant along the entire shear surface. This assumption is made in practically all slope stability methods. Hence, the value of the safety factor is an overall average. Therefore, as Johnson noted *(7)*, the safety factor obtained from the various methods of analyzing the stability of slopes does not necessarily constitute a reserve of unused strength; rather, the safety factor is a working element of the design process. As Bishop *(6)* noted, it is a quantitative estimate of the stability of a slope.

A particular problem which arises in the analysis of earth slopes is the matter of selecting a design safety factor. Obviously, this selection will significantly affect the economics of the design and may often times depend on the level of risk the designer is willing to assume, or the level of risk dictated by the situation. The type of structure, whether a dam, highway embankment, or building structure, also influences the selection. A review of several soil mechanics textbooks and papers reveals two different viewpoints concerning the selection of a numerical value of the design safety factor.

Some authors, for instance, recommend a design safety factor of at least 1.5 regardless of the type of analysis used to determine the stability of a slope. Bjerrum *(8)*, discussing the short-term stability of an embankment on a soft foundation, stated that many

embankments are designed for a safety factor of 1.4 to 1.5. He further suggested that a safety factor of 1.3 may be adequate if field vane shear strengths are corrected as outlined in his paper or as suggested below. Where special risks are involved or where low permeabilities may delay an increase in the safety factor, Bjerrum implied that a safety factor larger than 1.3 should be used. Johnson *(7)* recommended a design safety factor of 1.5 in all cases. Where a base failure and a sinking of the embankment is likely to occur at the end of construction, Terzaghi and Peck *(9)* recommended a value of 1.5. Lambe and Whitman *(10)* indicated that a safety factor of at least 1.5 is commonly used, provided the shear parameters have been selected on the basis on good laboratory tests, the soils involved were homogeneous, and a careful estimate of the pore pressures have been made. In cases involving nonhomogeneous and stiff fissured clays, those authors noted that more caution should be exercised. Other authors apparently feel that a safety factor lower than 1.5 is permissible. For instance, Tschebotarioff *(11)*, "NAVDOCKS" *(12)*, and Seelye *(13)* suggested a safety factor as low as 1.25 for temporary loading or controlled loading of highway embankments. Some authors do not make reference to a design safety factor although slope stability was oftentimes discussed or was a concern of the reference. The use of safety factors lower than 1.5 apparently evolved from Bishop and Bjerrum's important paper *(14)*. A design safety factor as low as 1.25 should be used cautiously. Whenever possible, local experience should be reviewed before a minimum design safety factor is selected.

CALCULATION OF SHEAR STRESSES ALONG THE FAILURE SURFACE

Whenever a slip occurs, the safety factor may be assumed equal to one and

$$\Sigma \tau = \Sigma s = \Sigma [c' + \sigma' \tan \phi'], \qquad 6$$

or the shear stress is equal to the shear strength of the soil. Equation 6 forms the basis of comparing the shear strength obtained from tests with the in situ shear strength. Unfortunately perhaps, a failure condition is the only opportunity for making such a comparison. If it is assumed that the parameter c' tends to zero when a slip occurs, then the in situ parameter ϕ'_c may be calculated. Rearranging terms and making the substition $\sigma' = \sigma - u_f$, Equation 6 becomes

$$\Sigma \phi'_c = \Sigma \tan^{-1} [\tau/(\sigma - u_f)], \qquad 7$$

where u_f is the pore pressure in the slope at the time of failure. Normally, direct calculations of ϕ'_c are not made, but rather several ϕ'_c values are assumed and corresponding safety factors are calculated and plotted as a function of safety factor. A ϕ'_c value corresponding to a safety factor of one can thus be obtained from such a plot. For "first-time failures" the c'-equal-zero assumption may be questionable. In this event, an approach described by Crawford and Eden (15) may be used. Various combinations of c' and ϕ' may be assumed which yield a safety factor of one. The corresponding c' - ϕ' combinations may then be plotted (ϕ' as function of c'). The ϕ' and c' parameters obtained from a strength test may be plotted on the above diagram and compared. In view of the present state of knowledge, however, the c'-equal-zero assumption appears reasonable.

EFFECT OF WATER ON THE COMPUTED ϕ'_c-VALUE

As shown by Equation 7, the computed ϕ'_c is a function of the pore water pressure existing at the time of failure. Hence, an accurate value of the pore pressure existing at the time of failure must be known to make a valid comparison between field and laboratory shear strengths. An example of the effect of pore pressures is summarized in Figure 2. In this example, slope stability analyses were performed on two cross sections passing through the slide. These analyses were made using computerized solutions of Janbu's generalized procedure of slices (16, 17). Finally, a weighted ϕ'_c value was obtained by a method suggested by Bishop and Bjerrum (14). If the water level in this particular highway slide is assumed to be along the shear surface (that is, u equal zero), then the computed ϕ'_c value is approximately 11 degrees. If the water level is assumed to coincide with the groundline, then the computed ϕ'_c is equal to about 19 degrees.

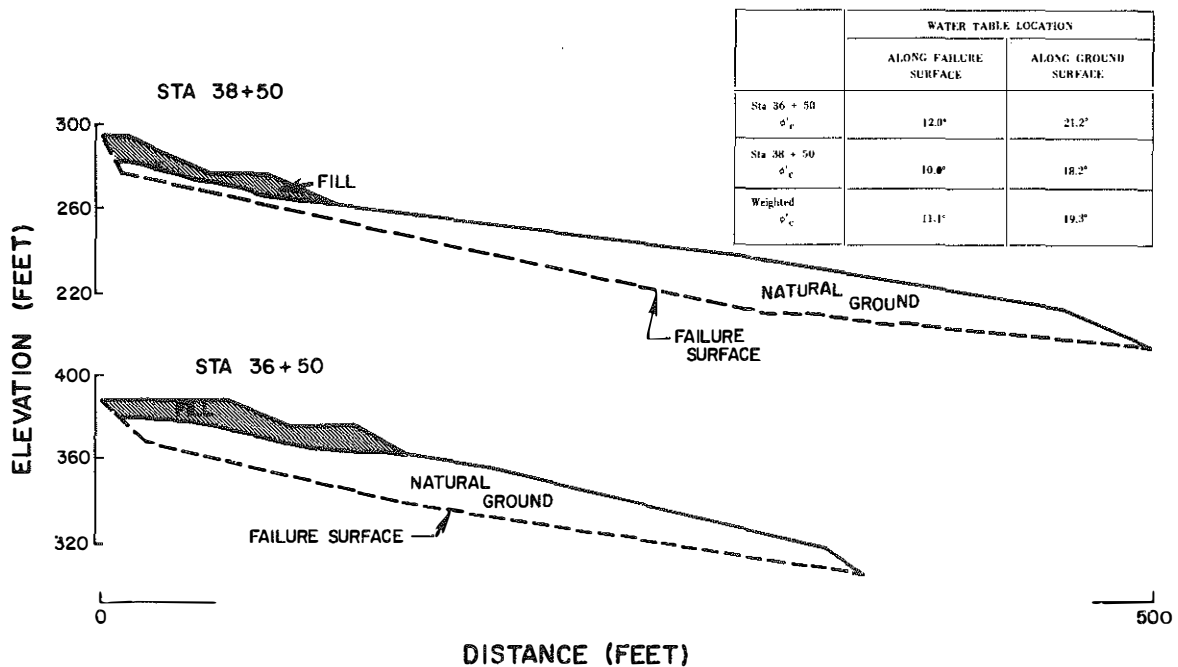| | WATER TABLE LOCATION | |
	ALONG FAILURE SURFACE	ALONG GROUND SURFACE
Sta 36 + 50 ϕ'_c	12.0°	21.2°
Sta 38 + 50 ϕ'_c	10.0°	18.2°
Weighted ϕ'_c	11.1°	19.3°

Figure 2.   Illustration of the Effect of Pore Water Pressures on the Computed Value of the Angle of Shearing Resistance, ϕ'_c (effective stress parameter c' assumed equal to zero).

An accurate determination of the pore water pressure in a landslide at failure poses certain difficulties. Even when piezometers are installed, measurements obtained may not correspond to the pore pressures existing at the time of failure, particularly in cases where a slope failure is preceded by a heavy rainfall and where field personnel may not be present at the time of failure. Frequently in "very active" embankment slides where remedial plans must be developed quickly, time may not permit the installation of piezometers. Hence, water level observations from boreholes may have to be used as a measure of pore pressures. In cohesive soils of low permeability, sufficient time must elapse after drilling for the water level to reach equilibrium (steady state seepage). Otherwise, erroneous pore pressures may be obtained. In such cases, the depth of water in the borehole should be plotted as a function of time so the equilibrium state may be identified. Another problem which may arise is the flow of surface water into the borehole or seepage of surface water down along the piezometer tubing. Hence, all borehole observation wells or piezometer installations should be sealed around the casing or tubing and the casing or tubing capped at the surface to prevent such leakages.

EFFECTIVE STRESS REDUCTION

Slope stability problems involving a reduction in effective stress and, therefore, a loss of shear strength may be divided as proposed by Bishop and Bjerrum *(14)* into two main classes as follows:

1. Pore pressure is an independent variable and does not depend on the magnitude of the total stresses acting in the soil. The pore pressure at a point in the slope is controlled by the water level or flow pattern of underground water or seepage. Engineering problems which come under this class include the long-term stability of slopes and earth fills and the stability of slopes of sand or gravel subject to a rapid drawdown of the water level adjacent to those slopes.

2. Pore pressure is a dependent variable and is a function of stress change. Some engineering problems in this category include the initial, or short-term, stability of a saturated clay foundation subjected to the rapid loading of an embankment or structure construction, the initial stability of an open cut or sheet-piled excavation in clay, and the stability of clay slopes subjected to rapid drawdown.

Effective stress analyses of problems in Group 1 involve the use of ϕ' and c' parameters obtained from drained triaxial tests or consolidated-undrained triaxial tests with pore pressure measurements. Pore pressures used in the effective stress analysis are obtained from piezometers or from a flow net.

The type of analysis used in solving problems in Group 2 depends on whether load is applied or removed and on whether the least favorable pore pressure distribution occurs initially or at some time after construction. Depending on whether loads are applied or removed, the pore pressures in a slope will either increase or decrease, respectively, with time until they finally reach an equilibrium condition with the prevailing ground water level in the soil mass.

SEEPAGE FORCES

Forces resulting from the seepage of water through a slope have a significant effect on the stability of a slope. This particular problem is frequently encountered in highway cut slopes and sidehill fills. Several case studies *(18-28)* show that the flow of water through slopes is a major cause of highway embankment failures in Kentucky. The general effect of seepage on the stability of a slope can be illustrated by analyzing the infinite slope shown in Figure 3. Seepage through the slope is assumed parallel to the slope and the water level is assumed to coincide with the groundline. Forces acting on a free body include the seepage force, the buoyant weight of the element, the resultant of the boundary normal effective stresses, and the resultant of the boundary shear stresses. From the flow net, the gradient i is

$$i = L \sin i/L = \sin i, \qquad 8$$

where L is the distance along the base of the stratum between the equipotential lines and i is the angle between the horizontal and the top flowline. Summing forces perpendicular to the stratum base yields

$$N = \gamma_b A \cos i, \qquad 9$$

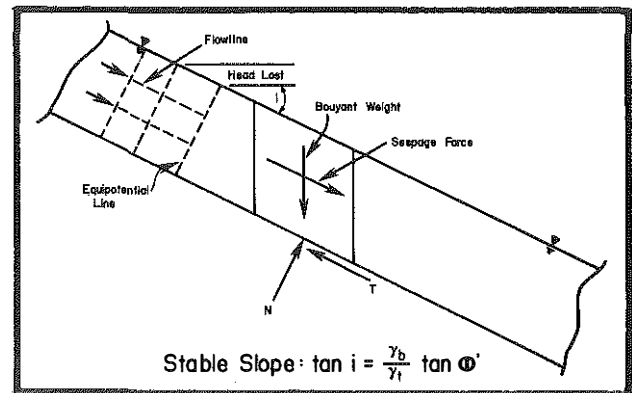where γ_b is the buoyant unit weight and A is the area

Figure 3.   Infinite Slope With Seepage.

of the element. Summing forces parallel to the slope gives

$$T = \gamma_b \, A \sin i + \gamma_w \, A \sin i \qquad 10$$
$$= \gamma_t \, A \sin i,$$

where γ_t is the total unit weight of the soil. Since

$$\tan \phi' = \gamma_t \, A \sin i \, / \, \gamma_b \, A \cos i \qquad 11$$
$$= (\gamma_t / \gamma_b) \tan i$$

then

$$\tan i = (\gamma_b / \gamma_t) \tan \phi'. \qquad 12$$

If typical values of γ_b and γ_t (62 and 124 pounds per cubic foot (993 and 1986 kilograms per cubic meter)) are assumed, the maximum stable slope of noncohesive soils ($c' = 0$), such as sands or normally consolidated clays, is

$$\tan i \approx 1/2 \tan \phi'. \qquad 13$$

If, for instance, the effective angle of shearing resistance, ϕ', of the soils in the slope in Figure 3 is 30 degrees, the maximum stable slope is four horizontal to one vertical. Without seepage, the maximum stable slope is two horizontal to one vertical.
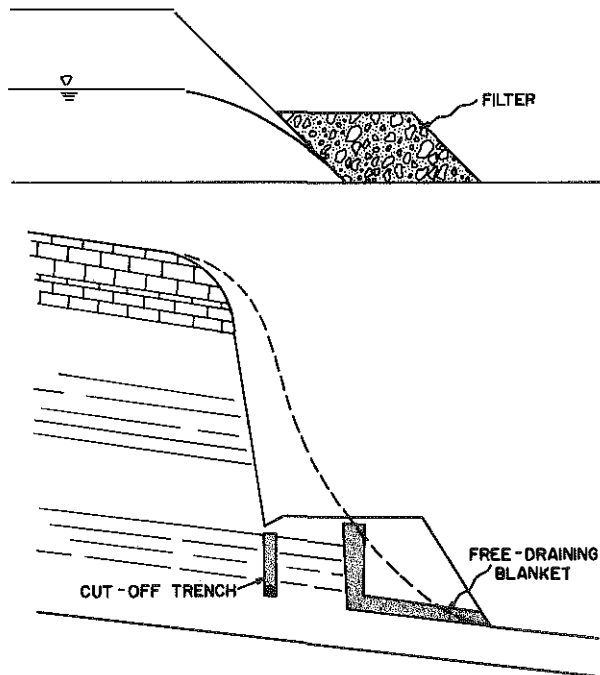
If the slope in Figure 3 contains overconsolidated clays, Equation 12 must be modified. However, regardless of whether or not clay has a cohesion value, the shear stresses and effective normal stresses must satisfy the relationship

$$\tau / \sigma' = (\gamma_t / \gamma_b) \tan i. \qquad 14$$

The effective normal stress acting at the base of the element in Figure 3 is

$$\sigma' = \gamma_b \, H \cos^2 i, \qquad 15$$

where H is the vertical distance from the groundline to the base of the slice. Substituting into Equation 3, the shear strength is

$$\tau = c' + (\gamma_b \, H \cos^2 i) \tan \phi'. \qquad 16$$

Substituting the expressions for σ' and τ into Equation 14 yields

$$\tan i = (\gamma_b / \gamma_t) \, [(c' / \gamma_b \, H \cos^2 i) + \tan \phi']. \qquad 17$$

If c' equals zero, then Equation 17 is the same as Equation 12.

Measures to minimize the effect of seepage on the stability of a slope mainly involve designing some type of drainage system to maintain the phreatic surface at a lowered elevation within the project area. Examples are illustrated in Figure 4.

Figure 4.   Design Measures to Minimize the Effects of Seepage and Rapid Drawdown on the Stability of a Slope.
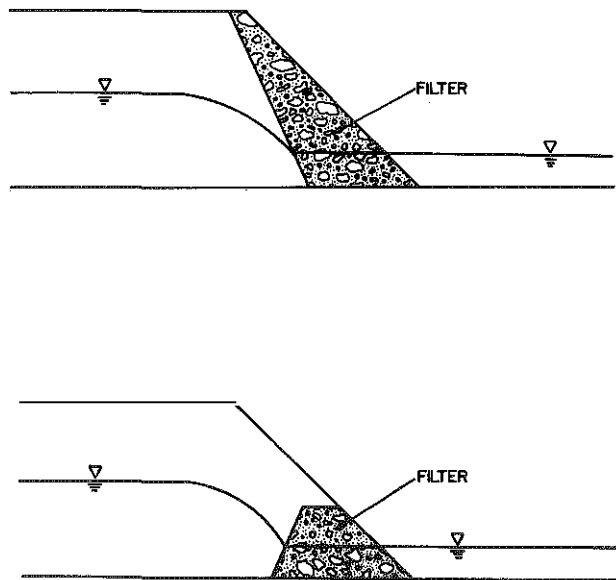
RAPID DRAWDOWN

Rapid drawdown is a sudden lowering in the level of water standing against a slope. This particular situation is commonly encountered, for example, in the design of bridge approach embankments at river crossings. In such a case, rapid drawdown occurs when the river falls following a flood. Other examples of rapid drawdown include the lowering of a reservoir adjacent to the upstream slope of an earth dam, lowering of the water level next to a natural slope, and a drop in the sea level next to a slope. Highway slope failures due to rapid drawdown have occassionally been observed in Kentucky. Most of these failures have occurred at highway stream crossings.

Figure 5 illustrates the three stages necessary for the development of a rapid drawdown condition. Prior to a rise in the water level (Figure 5a), the pore pressures, u_1, in the slope are zero, or negative if the soils are cohesive and in a semi-dessicated state. An analysis of an element of the slope shows that the resultant effective normal force, is (summing forces perpendicular to the base of the element)

$$N'_1 = N_1 - u_1 = X H_F \gamma_t \cos i, \qquad 18$$

where N_1 is the resultant (total) normal force, X is the width of the element, γ_t is the total unit weight, i is the angle formed by a horizontal line and the base of the element, and H_F is the height of the element. Since pore pressures are zero, the resultant effective normal force is equal to the resultant total normal force. The resultant shear force, T_1, found by summing forces parallel to the base of the element, is

$$T_1 = X H_F \gamma_t \sin i. \qquad 19$$

Figure 5b illustrates conditions at high water. Water has begun to seep into the slope (Line A-A'); after some time, the water level will tend to seek an equilibrium level (Line A-B) or hydrostatic state. Assuming the water level has reached the equilibrium level (Line A-B), pore pressures, u_2, in the slope may be expressed as

$$u_2 = \gamma_w [H_F + H_w], \qquad 20$$

where H_F is the height of the element and H_w is the depth of water over the element. The above assumption is conservative and reasonable since most slope failures will occur near the top break point of the slope and, in many cases, the high water level will exist for a sufficient time to enable the water level to reach the equilibrium level (Line A-B). Analysis of Stage 2 shows that

$$N'_2 = N_2 - u_2 \qquad 21$$
$$= X H_F (\gamma_t - \gamma_w) \cos i$$
$$= X H_F \gamma_b \cos i.$$

The resultant shear force is

$$T_2 = X H_F \gamma_b \sin i. \qquad 22$$

It is noted that there is a reduction in the resultant effective normal force ($N'_1 > N'_2$) and in the resultant shear force ($T_1 > T_2$) as a result of a rise of the water level to some hydrostatic condition. If negative pore pressures exist in the slope, the rise in the water level also causes the clay to swell and thus reduces the shear strength of the soil.

After rapid drawdown (Figure 5c), the level of water standing against the slope is lowered. The magnitudes of pore pressures at this stage are a function of the type of soil in the slope and the degree of saturation. Therefore, both cohesive and noncohesive soils must be considered. For cohesive soils of low permeability, such as clays, the drawdown time is much less than the consolidation time of the soils. Immediately following rapid drawdown, the total pore pressures, u_3, in the slope will equal the pore pressures, u_2, before drawdown plus the change in the pore pressures, Δu, due to a change in the water load against the slope, or

$$u_3 = u_2 + \Delta u \qquad 23$$
$$= \gamma_w [H_F + H_w] + \Delta u.$$

If the soils in the slope are near saturation or if the shear stresses are sufficiently large to endanger the stability of the slope, then a conservative estimate of the change of the pore pressures in the slope *(29)* is

$$\Delta u = - \gamma_w H_w. \qquad 24$$

Hence, a conservative estimate of the pore pressures in slopes composed of cohesive soils is

$$u_3 = [\gamma_w H_F + \gamma_w H_w] - \gamma_w H_w, \qquad 25$$

or simply

$$u_3 = \gamma_w H_F. \qquad 26$$

Consequently, the resultant effective normal force is (assuming the base is permeable)

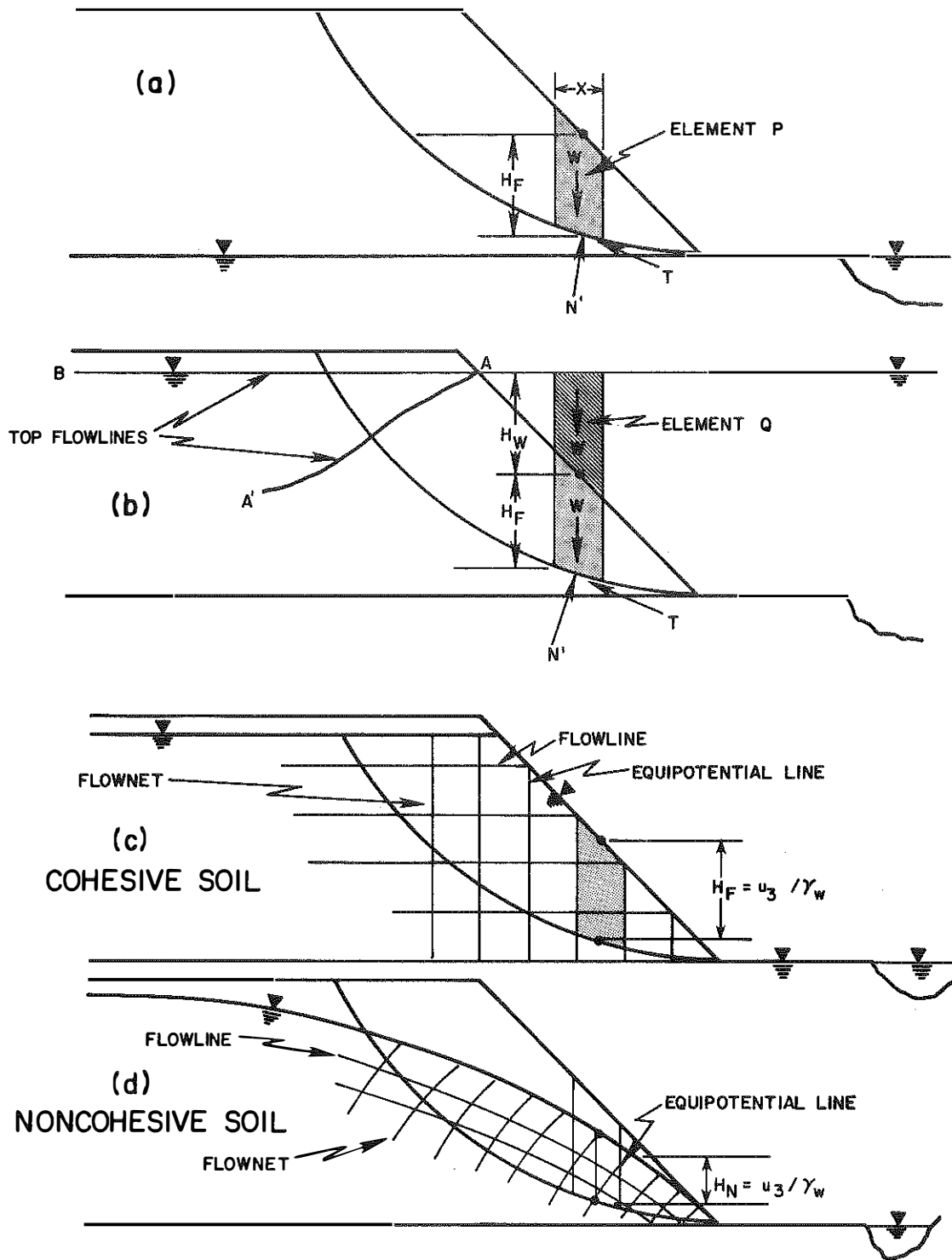$$N'_3 = N_3 - u_3 = X H_F \gamma_b \cos i. \qquad 27$$

6

Figure 5.   Development of a Rapid Drawdown Condition: (a) Before a Rise in the Water Level, (b) High Water Level, (c) Rapid Drawdown in Cohesive Soils, and (d) Rapid Drawdown in Noncohesive Soils.

For this case (cohesive soils), the resultant effective normal force remains the same as in Stage 2. The resultant shear force is

$$T_3 = X H_F \gamma_t \sin i. \qquad 28$$

Such increase $(T_3 > T_2)$ occurs because there is a tendency of water to flow out of the slope immediately after drawdown. Water in the slope will continue to seep out of the slope until an equilibrium condition is reached, as shown in Figure 5a.

In the case of noncohesive soils (sands and gravels), pore pressures in the slope are obtained from a flow net. The drawdown time is usually much more than the consolidation time of the soils. Although a complete analysis would involve drawing several flow nets, generally only one, corresponding to the condition immediately after drawdown, is analyzed. This represents the most critical condition (Figure 5d). Pore pressures are much less than those in the slope composed of cohesive soils:

$$u_3 = \gamma_w [H_F + H_w] + \Delta u. \qquad 29$$

Since water will flow from the slope rapidly as the water level is lowered, pore pressures in the slope will be (assuming a permeable base)

$$u_3 = \gamma_w H_N, \qquad 30$$

where H_N is as shown in Figure 5d.

Stability of a slope subjected to rapid drawdown can be determined from an effective stress analysis. Pore pressures for the effective stress analysis can be obtained for flow nets for the various conditions described and illustrated in Figure 5.

EXTERNAL LOADING

A cursory examination of various slides indicates that a slope may be subjected to several types of loading during service. In addition, the strength of the soil is not constant but changes with effective stress. It is important, therefore, to consider the strength and loading changes in the stability analysis. The analysis of relative changes in the applied stresses and the strength is an important part of the total slope stability investigation. Through this type of study, the engineer obtains a clear picture of the changes in the stability of the slope throughout various stages of the project. In this way, he is able to determine those stages which are most critical and select those for more detailed investigations. Other less critical times may be disregarded.

Two limiting conditions for soil behavior are well established. The conclusions that can be drawn from these conditions are so informative that the engineer is able, by extrapolation, to arrive at general conclusions for the less simple situations. Illustrated in Figure 6 is a situation in which an embankment is constructed over a deposit of clay. Stresses applied at a point, A, within the foundation material increases as the height of the embankment is increased and reaches a maximum at the end of the construction period. Initially, the pore water pressure is equal to the hydrostatic pressure. As the embankment is placed, pore pressures are increased since it is assumed there is no drainage, and thus no dissipation of pore water pressures takes place during the relatively short construction period. As one limiting condition, therefore, it can be assumed that the foundation material is loaded in an undrained condition. After construction, the stresses applied by the embankment loading remain constant. The excess hydrostatic pressures, however, tend to dissipate with
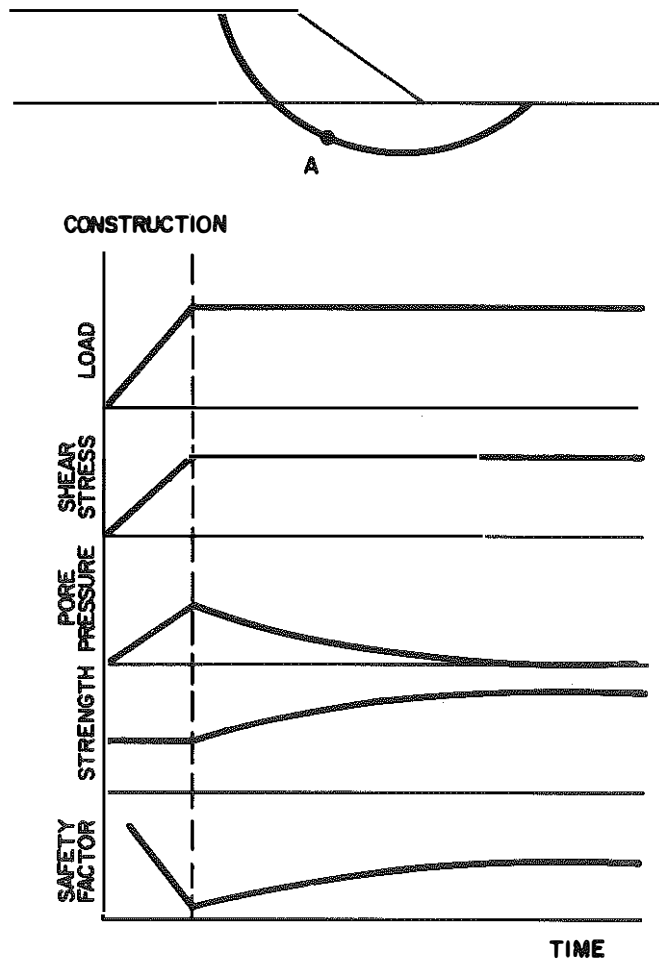

Figure 6.   Stability   Conditions   for   an Embankment-Loaded Foundation.

time. At some time in the future, the excess pore pressures are reduced to zero. As pore pressures are dissipated, there is a reduction in the void ratio of the material and a corresponding increase in the effective stress and the shear strength. The second limiting condition is attained at some long time after construction when the excess hydrostatic pressures approach zero (that is, the drained state). Since, at this late time in the life of the embankment, the excess hydrostatic pressures are zero, the effective stresses can be calculated from the known loads, the weights of the materials involved, and the hydrostatic pressures. The shear strength then can be determined from the effective stress parameters, c' and ϕ'.

The two limiting conditions in the example illustrated above involve forces that can be readily calculated. The end-of-construction stage can be studied using a total stress analysis (ϕ-equal-zero analysis) and the undrained shear strength. The long-term stability of the embankment can be investigated using an effective stress analysis (the excess hydrostatic pressures equal zero) and the effective stress strength parameters. If the distribution of the pore water pressures are known, it is possible, by means of the effective stress analysis, to evaluate the stability of the slope at any other time.

A second example of the application of limiting conditions is illustrated in Figure 7. Here a cut is made in a clay. As the excavation of the soil progresses, the average overburden pressure at some given point, A, is reduced and results in a decrease in pore water pressures. Applied shearing stresses at Point A increase to a maximum at the end of construction. As in the previous example, it can be considered that the limiting condition of no drainage during construction still applies. Therefore, at the end of construction, the shear strength remains equal to the undrained strength. With time, pore pressures increase, accompanied by a swelling of the clay and a reduction in the shear strength of the material. As before, the second limiting condition is reached after a long time -- when the excess hydrostatic pore pressures are equal to zero. Again the strength of the materials can be represented by the effective stress parameters.

OTHER CONSIDERATIONS

Any process which increases the supply of water to an earth mass may potentially lead to the development of excess pore water pressures. Extended periods of rainfall, for example, could result in significant changes in pore pressures. When an embankment is placed, it probably exists in a condition wherein pore water pressures are in a negative state (Figure 8). This, of course, increases the effective stress and causes an increase in the shearing resistance of the
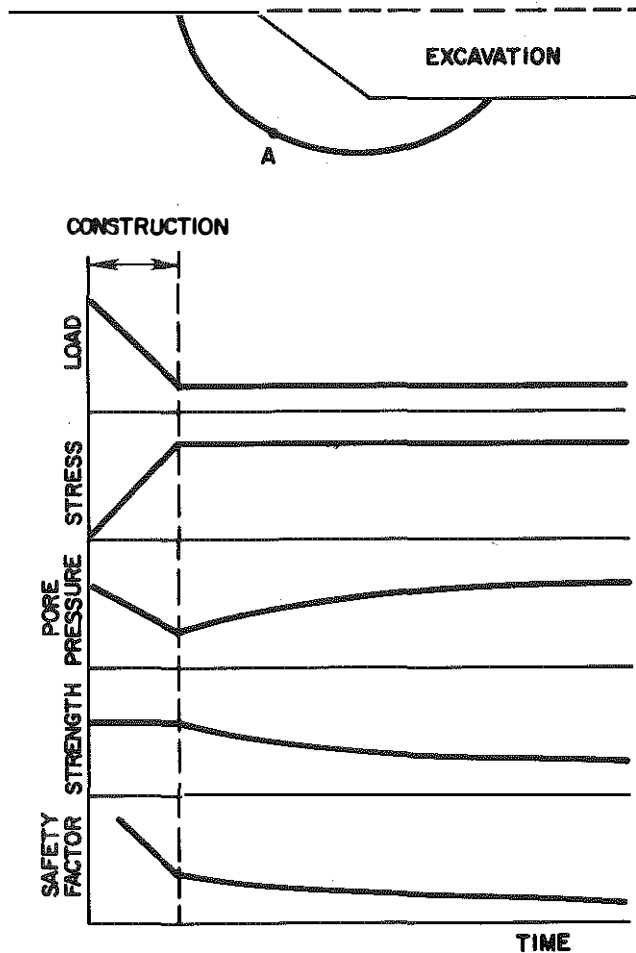
Figure 7. Stability Conditions for an Excavated Slope.

embankment material. After a prolonged period of excessive rainfalls, the negative pore pressures may tend toward zero and may even become positive with a rise in the water table, or a flow of water in the slope may be established which increases pore pressures. Such a change in pore water pressures causes a significant decrease in the effective stress and an attendant decrease in the strength of the embankment. If the strength decrease is sufficient, the embankment slope may become sufficiently unstable that failure occurs.

Any impoundment of water in a position that seepage will enter the zone of potential slope failure may eventually lead to the failure of that slope. The ponding may be a result of natural impoundments of waters high above the slope or might be the result of man's activities, as indicated in Figure 9. Here again, the ingress of water into a potentially unstable zone may result in increased pore pressures, causing reduced strengths and thereby leading to possible failures.

9
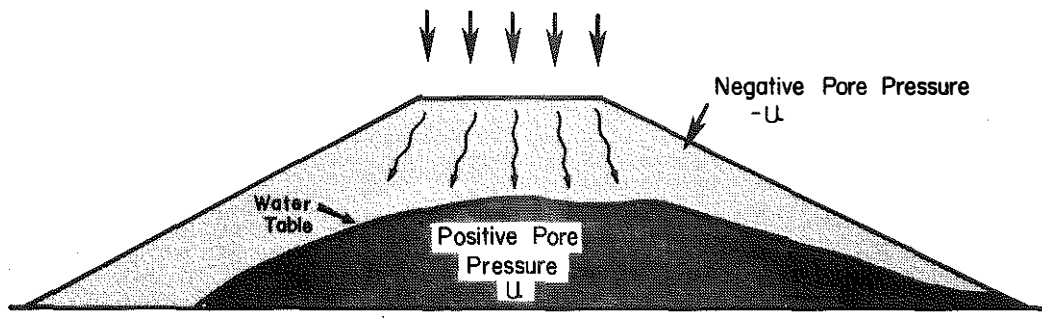
$$\text{AFTER PLACEMENT:}$$
$$S = C' + (\sigma - (-u)) \tan \phi'$$

$$\text{AFTER RAINFALL:}$$
$$-u \rightarrow u$$
$$S = C' + (\sigma - u) \tan \phi'$$

Figure 8.   Effect of Excessive Rainfalls on the Shear Strength of a Clayey Embankment.
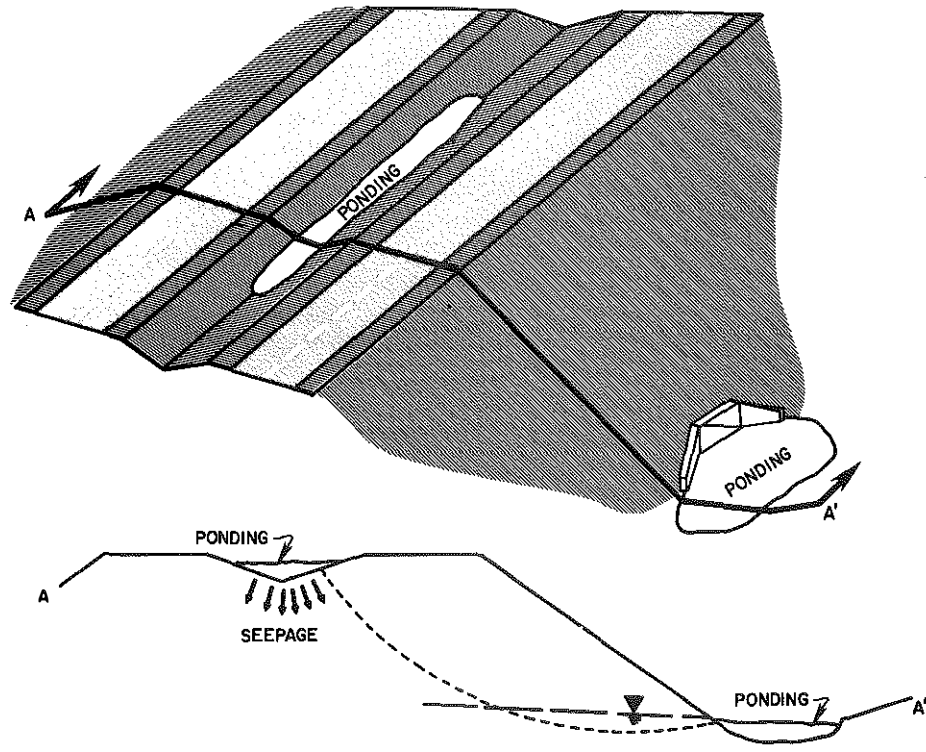
Figure 9.   Seepage through an Embankment due to Impoundment of Water.

REDUCTION IN
SHEAR STRENGTH PARAMETERS

LIMITATIONS OF TOTAL STRESS ANALYSES

The conventional approach to determining the stability of an embankment located on a clay foundation consists of using the undrained strength, s_u, obtained from unconsolidated-undrained tests (or unconfined compression tests), field vane shear tests, or Dutch cone penetration tests (30) in a total stress analysis. If the total stress analysis yields a safety factor of approximately 1.25 to 1.50, the design is considered adequate to prevent failure during construction. The short-term or end-of-construction stability is usually considered the most critical condition since excess pore pressures in the foundation usually reach maximum values at the end of the loading period, as shown in Figure 6. As the excess pore pressures dissipate, the safety factor increases and finally reaches a maximum value when Δu equals zero. Hence, the long-term safety factor is usually considered to be greater than the short-term safety factor obtained from the total stress analysis. In the case of cut slopes and excavations, the ϕ-equal-zero analysis has often been used to assess the end-of-construction safety factor of such slopes. If the ϕ-equal-zero analysis yields a safety factor of about 1.5, it is usually assumed the slope will remain stable during construction. As shown in Figure 7, the value of the safety factor is a maximum during excavation; as pore pressures in the slope increase, the safety factor decreases. However, application of conventional procedures to the design of embankments founded on clay foundations or to cut slopes without regard to the stress history and moisture state of the clays in the foundation or cut slope may lead to erroneous conclusions concerning the safety factor. There are four situations where application of the ϕ-equal-zero analysis may yield too high safety factors based on the a priori arguments shown in Figures 6 and 7. In such situations, the safety factor obtained from a ϕ-equal-zero analysis based on laboratory or field undrained shear strengths may lead to a false impression concerning the stability of the slope; that is, the undrained shear strengths obtained from laboratory or field tests may be larger than the actual (back-computed) shear strengths existing at failure. These situations are

    (1) long-term stability of cut and natural slopes,
    (2) short-term stability of loads on very soft foundations,
    (3) short-term stability of loads on overconsolidated clays and clay shales, and
    (4) short-term stability of a cut or excavated slope in overconsolidated clays and clay shales.

Supportive evidence showing that the use of the ϕ-equal-zero analysis in the above situations may lead to unconservative safety factors is described below. Finally, a method is proposed for predicting the probable success of a ϕ-equal-zero analysis.

Long-Term Stability of Cut and Natural Slopes -- Bishop and Bjerrum (14), summarizing results of a number of failures in natural slopes and cuts, showed that application of the ϕ-equal-zero analysis to slopes where pore pressure and water content equilibrium have been attained is unreliable. In these cases, the ϕ-equal-zero analysis gave safety factors ranging from 0.6 for sensitive soils (sensitivity of a soil is defined as the ratio of undisturbed to remolded strength, and sensitive soils are those which have large values of sensitivity) to 20 for heavily overconsolidated soils (soils which are at equilibrium under a stress less than the stress to which they were once consolidated); however, all of those slopes failed (F = 1.0). The primary reason for the differences between the in situ shear strength and the shear strength obtained from the undrained test is that the pore pressure in the undrained test is a function of applied stress, and these pore pressures are not necessarily equal to the in situ pore pressures. A second reason for the differences is that there is, apparently, a migration of water to the failure zone in a slide (this is true only for overconsolidated clays); an example (from an investigation by Henkel and Skemption (31)) is a slide where the moisture content in the failure zone was some ten percent greater than the moisture content of material above and below the very thin (2-inch (50-mm) thick) failure zone. A ϕ-equal-zero analysis based on the undrained strength of samples from the failure zone of this slide gave a safety factor of about one; based on the undrained strength of samples from above and below the shear zone, a safety factor of above four was obtained.

Examination of case records of long-term failures in cuts and natural slopes revealed that higher safety factors are associated with low to negative values of liquidity index while low safety factors are associated with high values of liquidity index, defined (1) as

$$LI = (w - PL)/PI, \qquad 31$$

where w is the natural moisture content of the soil, PL is the plastic limit, and PI is the plasticity index. In the data cited by Bishop and Bjerrum, there were four cases where the safety factor was near one; the liquidity indices ranged from 0.20 to 1.09. In the other cases, the liquidity indices ranged from about 0.19 to -0.36 while the safety factors ranged from 1.9 to 20. For the case discussed above, the liquidity index of samples from outside the failure zone was 0.0 and the factor of safety was calculated to be above four. The liquidity index

of samples from the failure zone was 0.4, and the safety factor based on the undrained strength of those samples was near one.

Short-Term Stability of Loads on Soft Foundations -- Bjerrum *(8)* assembled a number of case records which showed that procedures normally used to determine the initial or short-term stability of embankments on soft clay foundations are unsatisfactory. In those cases, the undrained shear strength was obtained from in situ vane tests. Safety factors obtained from a ϕ-equal-zero analysis ranged from a low of 0.86 to a high of 1.65. However, the embankments failed in all cases (F = 1.0). Of the 14 cases cited by Bjerrum, eight had safety factors larger than 1.30; in the other cases, the safety factors ranged from 0.86 to 1.17. Where large safety factors were obtained, liquid limits of the clay foundations generally exceeded 90 percent. Where the lower safety factors prevailed, liquid limits were generally below 90 percent. The liquidity indices ranged from 0.49 to 1.75 while plasticity indices ranged from 16 to approximately 108. Hence, there was poor agreement between the calculated shear strength, assuming a safety factor of one, and the strength obtained from in situ vane shear tests for cases with high liquid limits. Arranging the data in order of decreasing values of plasticity indices, Bjerrum observed that the difference between vane and calculated shear strengths increased as the plasticity index of the clay increased (a similar statement could be made concerning the liquid limit). Correction factors, μ, can be derived by plotting safety factors (from observed cases) as a function of plasticity index (Figure 10) and noting that

$$F = (s_u)_{vane}/(s_u)_{calculated} \qquad 32$$
$$= f(PI) = 1/\mu.$$

Using a least squares polynomial method of approximation and a second-degree fit, the safety factor may be expressed as

$$F = 0.747 + 0.0153\ PI \qquad 33$$
$$- 0.00007\ PI^2,$$

and the corrected shear strength may be expressed as

$$(s_u)_{calculated} = (s_u)_{vane}/(0.747 \qquad 34$$
$$+ 0.0153\ PI - 0.00007\ PI^2).$$

Bjerrum's data (based on vane test strength parameters) indicated that the use of uncorrected, in situ vane shear strength parameters should be used cautiously in designing embankments on soft clay foundations.

Data assembled by Bishop and Bjerrum *(14)* representing end-of-construction failures of footings, fills, and excavations on saturated clay foundations are plotted (as circle points) and compared to Bjerrum's data (triangular points) in Figure 10. Liquidity indices of the former data ranged from about 0.25 to 1.44. The undrained strength of the soils in these analyses were obtained primarily from unconsolidated-undrained tests. While Bjerrum's data showed that the difference between vane and back-computed shear strengths increased as the plasticity index of the clay increased, Bishop and Bjerrum's data, in marked contrast, showed that the computed shear strength and laboratory shear strength were almost equal (i.e., safety factor approximately one). Liquid limits of the clay foundations of Bishop and Bjerrum's cases generally did not exceed 90 percent.

Short-Term Stability of Embankments Founded on Overconsolidated Clays and Clay Shales -- A number of short-term failures of embankments located on overconsolidated soils have occurred, even though the ϕ-equal-zero analysis indicated the embankment slopes should have been stable. For example, Beene *(32)* described the failure of a large earth dam (near Waco, Texas) founded on an overconsolidated (pepper) clay shale during construction. According to Wright *(4)*, an analysis of this dam using the ϕ-equal-zero analysis and shear strengths from unconsolidated-undrained tests indicated the embankment should have been stable (F $\approx$ 1.3). Peterson, et al. *(33)* listed several cases of short-term failures (failed during construction) of embankments located on lightly overconsolidated clays, although the ϕ-equal-zero analysis indicated those embankments should have been stable. In particular, Peterson, et al. described two slides ("Seven Sisters Dikes") that occurred during construction. Based on total stress analyses, Slide "S-1" had a safety factor of about 1.6 and Slide "S-6" had a safety factor of 1.31 to 1.65. Peterson also cited another case of a dam ("Northridge") that failed during construction where the total stress analysis was unsuccessful in predicting the performance (safety factor was 1.23). Another example was an embankment failure *(19)* which occurred on I 64 in Kentucky. This embankment was located on overconsolidated foundation clays. A ϕ-equal-zero analysis based on unconfined compression tests yielded a safety factor of about four. Samples for this slide were obtained from the toe of the embankment prior to failure, which occurred approximately 4 years after construction. Three sections passing through the slide were analyzed using Bishop's *(6, 34)* and Janbu's methods; a weighted value of the safety factor was obtained. If this analysis had been performed prior to construction, it certainly would have been concluded that the embankment as designed would have been safe.

In each of the situations described above, the back-computed shear strength was generally less than the undrained shear strength obtained from laboratory or field tests.

Short-Term Stability of a Cut or Excavated Slope in Overconsolidated Clays and Clay Shales -- The ϕ-equal-zero analysis is oftentimes used to determine the short-term stability of a cut or excavated slope. As shown in Figure 7, the short-term safety factor is usually a maximum during or near the end of construction. However, stability of cuts in overconsolidated clays and clay shales may not always conform to the concept shown in Figure 7. For instance, Skempton and Hutchinson *(35)* described two slides which occurred in a deep excavation for a nuclear reactor at Bradwell, England. These slides are also cited in Bishop and Bjerrum's data. The excavation was in a stiff, overconsolidated London clay. Based on a ϕ-equal-zero analysis and undrained shear strengths, the short-term safety factors were about 1.8 to 1.9; but the slopes failed during construction.

Proposed Method of Predicting Success in a ϕ-Equal-Zero Analysis -- Peck and Lowe *(36)* presented a plot (Figure 11) of a portion of Bishop and Bjerrum's data (long-term failures in cuts and natural slopes) which showed that the computed safety factor of failed slopes,

obtained from a ϕ-equal-zero analysis using undrained strength, was apparently a function of the liquidity index. Peck suggested the possibility of using this curve as an empirical basis to determine correction factors to apply to strength parameters for undrained analyses and to assess the possible success of a slope design.

Plotting additional portions Bishop and Bjerrum's data (safety factor as a function of liquidity index), a distinctive division can be observed. All data in Figure 12 represent slope failures where the ϕ-equal-zero analysis was performed using undrained shear strength parameters. In failures where the clay soils had a liquidity index equal to or greater than approximately 0.36, the ϕ-equal-zero analysis based on undrained strengths gave safety factors of approximately one. Hence, the ϕ-equal-zero analysis correctly predicted the in situ shear strength. When the liquidity index is greater than 0.36, safety factors estimated from a ϕ-equal-zero analysis should have an accuracy within $\pm$ 15 percent. Consequently, end-of-construction design safety factors as low as 1.3 may be justified in many routine designs. However, in cases where soils have exceptionally low permeabilities or where special risks are involved, a safety factor of 1.5 should be considered. Where the undrained strength is obtained from in situ vane shear tests, the vane strength should be corrected.
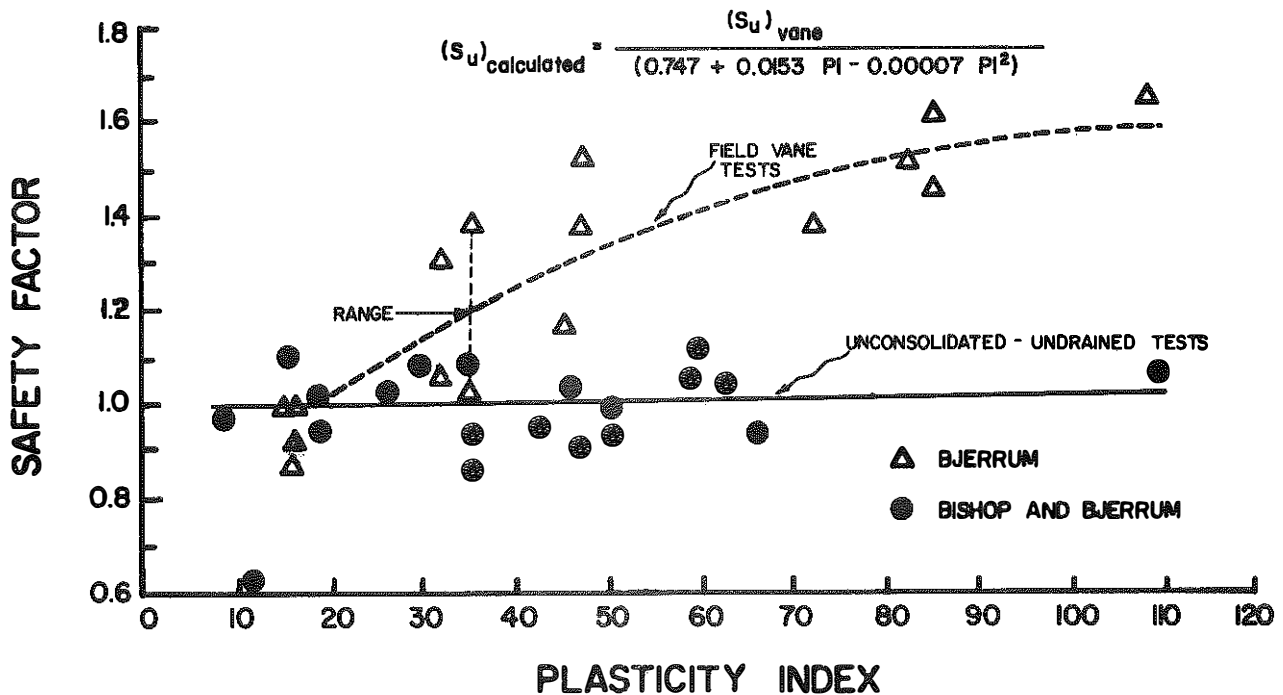
Figure 10.   Factor of Safety as a Function of Plasticity Index (data from Table I of Bjerrum's *(8)* paper and Table II of Bishop and Bjerrum's *(14)* paper). The curved-line fit is by the authors and is based on a re-examination of Bjerrum's data. All slopes represented in the diagram failed, i.e., F = 1.0.
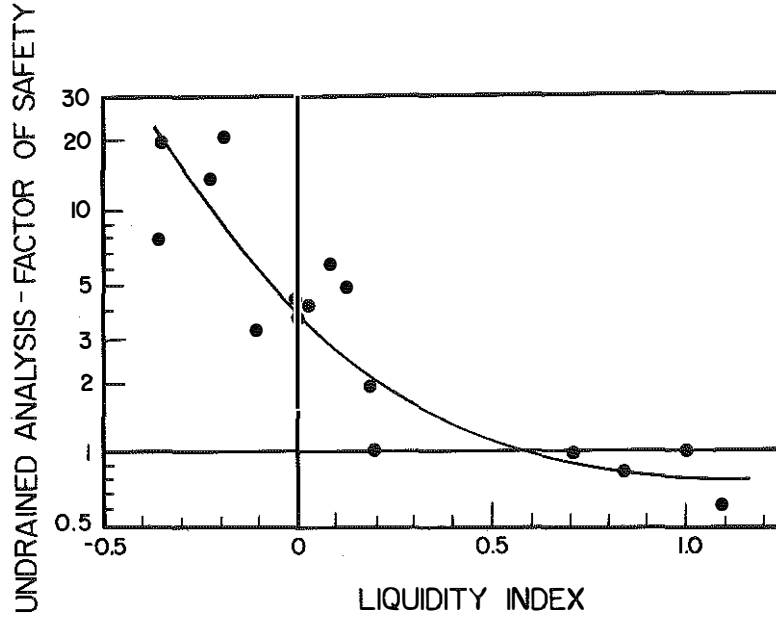
Figure 11.   Factor of Safety as a Function of Liquidity Index (after Peck and Lowe *(36)* and Table V of Bishop and Bjerrum's data *(14)* for long-term failures in cuts and natural slopes). All slopes represented in the diagram failed, i.e., F = 1.0.
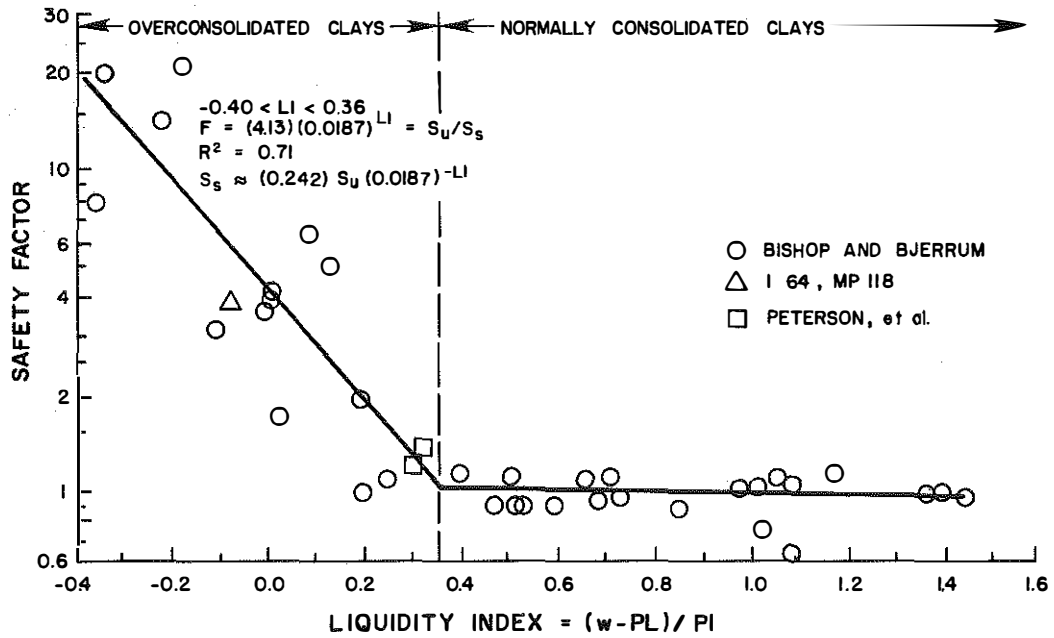

Figure 12.   Factor of Safety as a Function of Liquidity Index (by the authors; the data is from Tables II, III, and V of Bishop and Bjerrum's paper *(14)*, Peterson, et al. *(33)*, and Kentucky DOT Division of Research).

14

In failures where the clay soils had a liquidity index less than about 0.36, the ϕ-equal-zero analysis based on undrained strengths gave safety factors which were much too high. Consequently, the ϕ-equal-zero analysis using laboratory undrained strength parameters overestimated the in situ shear strength and, therefore, give a false hnpression concerning the stability of a slope. The ϕ-equal-zero analysis in such cases is not applicable. For clay soils with a liquidity index less than 0.36, the safety factor appears to be a function of the liquidity index. Assuming a straight-line fit and using the method of least squares, the following relationship was obtained:

$$F = (4.23) (0.0187)^{LI}. \qquad 35$$

Since the safety factor can be expressed as

$$F = s_u/s_s, \qquad 36$$

where s_u is the laboratory undrained shear strength obtained from unconsolidated-undrained tests and s_s is the corrected laboratory shear strength, or the "softened" shear strength, the corrected laboratory shear strength may be expressed as

$$s_s \approx (0.242)s_u (0.0187)^{-LI}. \qquad 37$$

This is an approximate expression for correcting the laboratory strength and should be used cautiously. Obviously, more studies are needed to improve the correlation between the safety factor and liquidity index. The corrected laboratory shear strength given by the above empirical equation is believed to represent the "softened" state of overconsolidated clays and clay shales at failure. It is interesting to recall the results obtained by Henkel and Skempton *(31)*. The liquidity index of samples from the failure zone was 0.40; based on the laboratory undrained shear strength of the failure zone samples and ϕ-equal-zero analyses, the safety factor was close to one. Based on the undrained shear strength of samples from above and below the failure zone, the safety factor was 4.0 and the liquidity index was zero, suggesting that, when an overconsolidated clay slope fails, the liquidity index in the failure zone increases to 0.36 or more.

From Figure 12, it appears that overconsolidated clays and normally consolidated clays can be approximately distinguished on the basis of liquidity index. If a clay has a liquidity index greater than 0.36, it might be considered normally consolidated; and if the liquidity index is less than 0.36, the soil is overconsolidated.

The mechanism leading to the development of a "softened" failure zone in overconsolidated clays is not clearly understood. The mechanism appears to be time dependent, since many of the slopes represented in the left portion of Figure 12 were long-term failures, and is significantly affected by the presence of water. However, a few cases in the left portion of the figure represent short-term failures. Consequently, time to failure (or time for a progressive failure to develop *(37)*) may depend more on the rate of migration of water to an overstressed zone and on the level of applied stress. In searching the literature, only a few cases of failures were found which gave the time to failure and liquidity indices of the soils. These cases are shown in Figure 13. Although this plot is highly idealized, it does suggest, perhaps, that the time of failure in overconsolidated soils, or the time for the "softening" effect to develop, is very long for soils having large negative values (less than about 0.0 or -0.1) of liquidity index. Where the liquidity index is greater than 0.0 or 0.1, time to failure is quite variable. However, much more research is needed to show the general validity of the above concept.

The above concepts apply only to clays. They do not necessarily apply to other soil types, particularly soils of low plasticity. To illustrate this point, the Atterberg limits corresponding to the above case records are plotted on the plasticity chart. Figure 14 represents those cases where the soils were "normally consolidated" (LI > 0.36). The majority of those cases are concentrated in the CH region, although some of the cases fall in the CL region. None of those cases fall in the ML and ML-CL region. For the "normally consolidated" cases, the safety factor was equal to 1.0 ± 15 percent, except for seven cases (Bjerrum's data *(8)*) involving clays with liquid limits greater than about 70 percent. The safety factors for these cases ranged from about 1.38 to 1.65. It appears the ϕ-equal-zero analysis using undrained shear strengths may be reliable for normally consolidated clays having liquid limits up to approximately 70 to 90 percent. For clays having liquid limits in excess of 70 to 90 percent, the ϕ-equal-zero analysis based on undrained shear strengths is unreliable. For overconsolidated soils (LI < 0.36), the cases, as shown in Figure 15, are generally concentrated in the CH region.

LIMITATIONS OF EFFECTIVE STRESS ANALYSES

Problems in Triaxial Testing -- When "undisturbed samples" are obtained, the in situ stresses on those samples are altered -- generally relieved. The sample will normally respond with a small increase in volume. This induces negative pore pressures within the sample, increasing the shear strength of the sample. In soils that are overconsolidated, this process took place by erosion of part of the overburden under which the sample was originally consolidated.
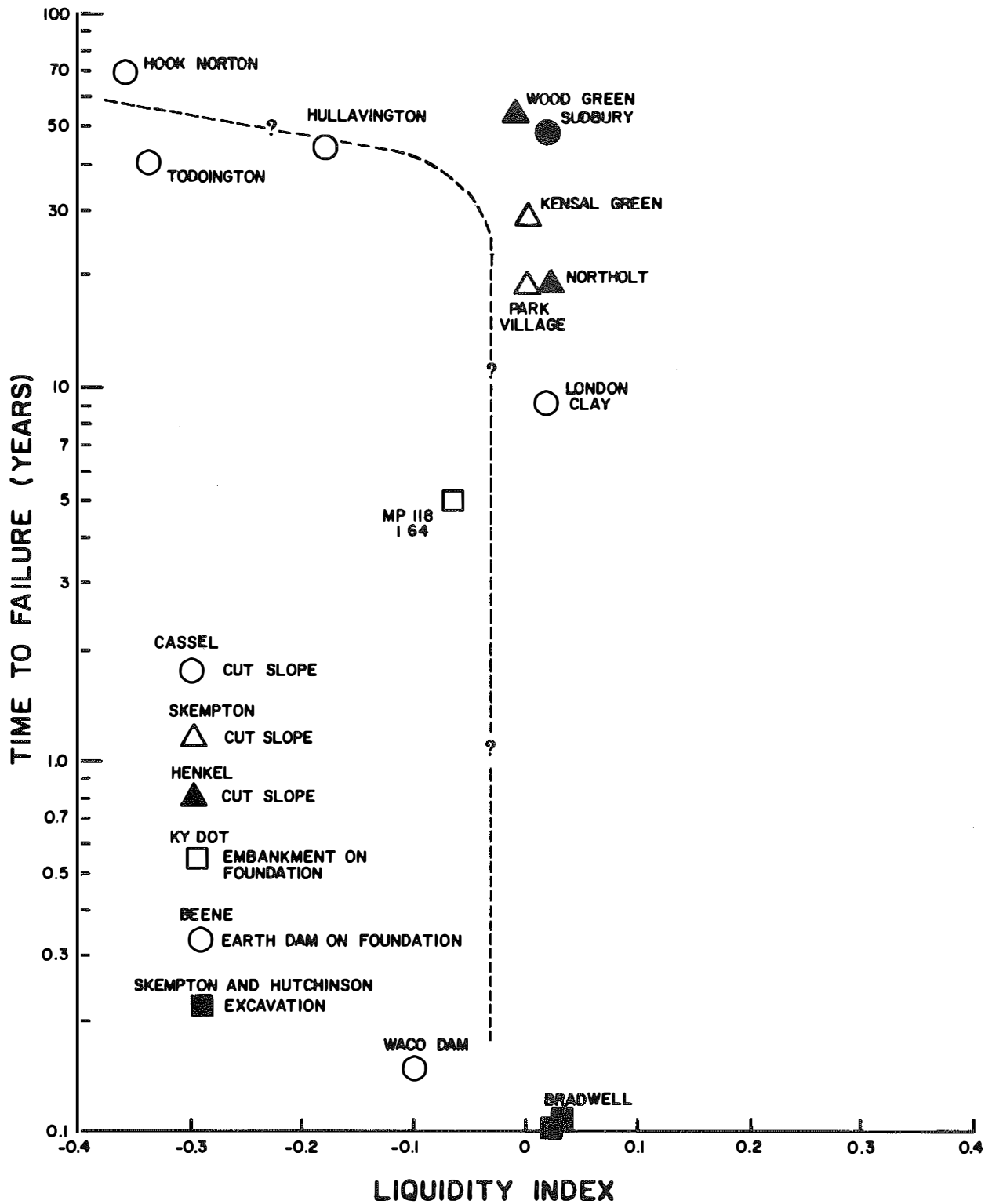
Figure 13. Time to Failure of Overconsolidated Clays and Shales as a Function of Liquidity Index (data from Cassel *(38)*, Skempton *(39)*, Henkel *(40)*, Beene *(32)*, Skempton and Hutchinson *(35)*, and Kentucky DOT Division of Research).

16
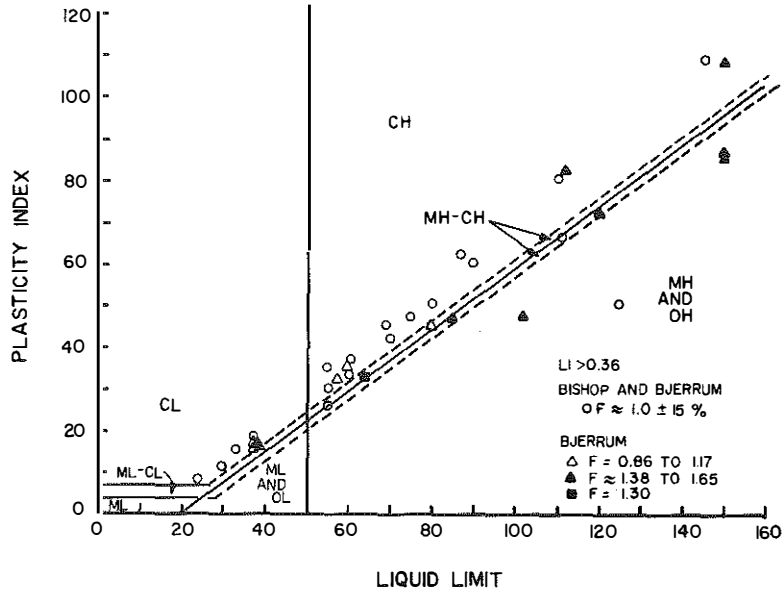
Figure 14.   Plasticity Chart Showing the Atterberg Limits of Slope Failures in Normally Consolidated Clays (data from Bishop and Bjerrum *(14)*, Bjerrum *(8)*, and Kentucky DOT Division of Research). The liquidity index for these cases was greater than 0.36. Factor of safety for these cases was within 1.0 ± 15 percent, except for seven cases of Bjerrum's data where the safety factor ranged from 1.38 to 1.65.
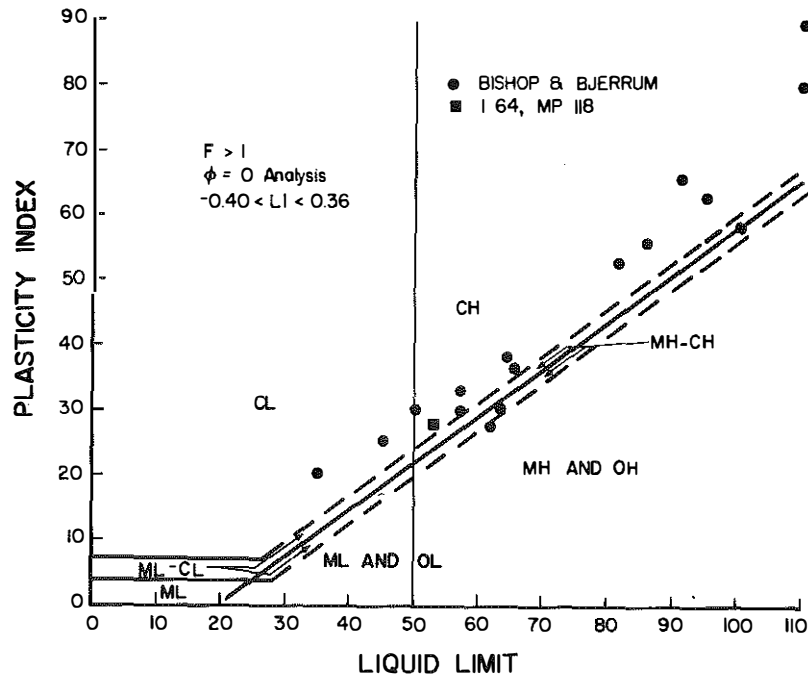
Figure 15.   Plasticity Chart Showing the Atterberg Limits of Slope Failures for Overconsolidated Clay (data from Bishop and Bjerrum *(14)* and Kentucky DOT Division of Research). The liquidity index for these cases was less than 0.36. Factor of safety for these cases ranges from about 1.0 to 20.0.

17

When an overconsolidated sample is placed in a triaxial chamber under some predetermined consolidation pressure, this, hopefully, will cause the sample to consolidate and eventually become saturated. If the consolidation pressure is less than the preconsolidation pressure or previous in situ stresses, the sample probably will not become saturated and negative pore pressures will still exist. To saturate highly overconsolidated samples or samples that have a high degree of cementation, pressures as high as 700 psi (4.8 MPa) may be necessary.

Although negative pore pressures in partially saturated samples generally increase the overall shear strength, it can increase or decrease the individual components, ϕ and c, depending upon such things as rate of loading, amount of strain, consolidation and preconsolidation pressures, degree of saturation, sample orientation, pore pressure gradients within the sample, and possibly pore pressure lag to sensing devices.

Negative pore pressures will increase the effective stress, as can be seen in Equation 2. If there is a large pore pressure gradient in the sample so that small or negative pore pressures exist at the shear zone but a large pore pressure exists near the sensing device, a larger than normal Mohr's circle will result. However, the circle will be shifted toward the origin because of larger pore pressures at the base. This will give a larger apparent c'. Hence, large values of c' should be used cautiously. However, if negative pore pressures exist uniformly throughout the sample, the circle will be shifted from the origin, decreasing ϕ' but increasing the apparent c'. Pore pressure gradients can generally be reduced if the sample is strained at a slow enough rate to allow equalization. This will result in more accurate values of the effective strength parameters.

The above examples illustrate problems encountered when trying to interpret effective strength parameters. In many cases, therefore, effective stress parameters, like total stress parameters, may overestimate actual available strength.

**Other Limitations of Effective Stress Analyses –** In the conventional design of earth slopes, if the total stress analysis yields a safety factor of about 1.0 to 1.25, then an effective stress analysis is performed to assess the probable long-term safety factor. Such an approach requires the effective stress parameters be determined; normally, these parameters are determined from laboratory isotropically consolidated, drained triaxial tests (CID) or isotropically consolidated, undrained triaxial tests (CIU) with pore pressure measurements. Generally, these tests yield approximately the same results. Although an effective stress analysis could be used to analysis the short-term or end-of-construction case, such an approach is normally not used since it requires that excess pore pressures be known. Methods of predicting excess pore pressures are particularly difficult to use and results obtained from such methods are highly questionable. Consequently, the assumption is made that Δu is equal to zero, and an effective stress analysis is performed based on that assumption. If the safety factor is sufficiently high (about 1.5), and if consolidation tests indicate the soils will drain fairly rapidly, it may be assumed the slope can be constructed safely. However, if the safety factor is relatively low, piezometers may be installed to monitor pore pressures during construction. If pore pressures are known, then an effective stress analysis may be performed during construction to assess the probable stability of the slopes.

Uncertainties in the application of the effective stress approach to the design of earth slopes arise in the selection of shear strength parameters, ϕ' and c', and evaluation of pore pressures. Although the effective stress method has been successfully applied to normally consolidated and very lightly overconsolidated clays and silty clays having an intact structure (clays free of fissures or joints), the method is not successful when applied to the design of slopes composed of overconsolidated clays and clay shales. Even considering that the latter soil types are very prevalent and that much research has been directed toward studying the characteristics of those soils, it is those materials which have the greatest tendency to invalidate current design concepts. Overconsolidated soils pose the greatest design dilemma to engineers.

Peak and Residual Strengths – Figure 16 shows typical stress-strain curves for normally consolidated and overconsolidated clays tested similarly under drained conditions. As the samples are loaded, both reach a peak strength. As the overconsolidated soil is strained beyond the peak strength, the shear resistance of the overconsolidated soil decreases until at large strains the strength falls to a (nearly) constant value. This lower limit of resistance is referred to as the "residual" or "ultimate" strength of the soil *(1, 3, 37, 39, 41)*. With increasing displacements, after the peak strength has been attained, the shear resistance of the normally consolidated clay may fall only slightly. After large strains, the shear resistance of the overconsolidated and normally consolidated clay coincide. In heavily overconsolidated plastic clays, there is a large difference in the peak and residual strengths. In silty clays and soils of low plasticity, this difference is very small. With an increase in clay content, this difference increases even in normally consolidated clays, although not as much as in overconsolidated clays. The softened state of an overconsolidated clay may be defined as shown in Figure 16 -- the intersection of a horizontal line projected from

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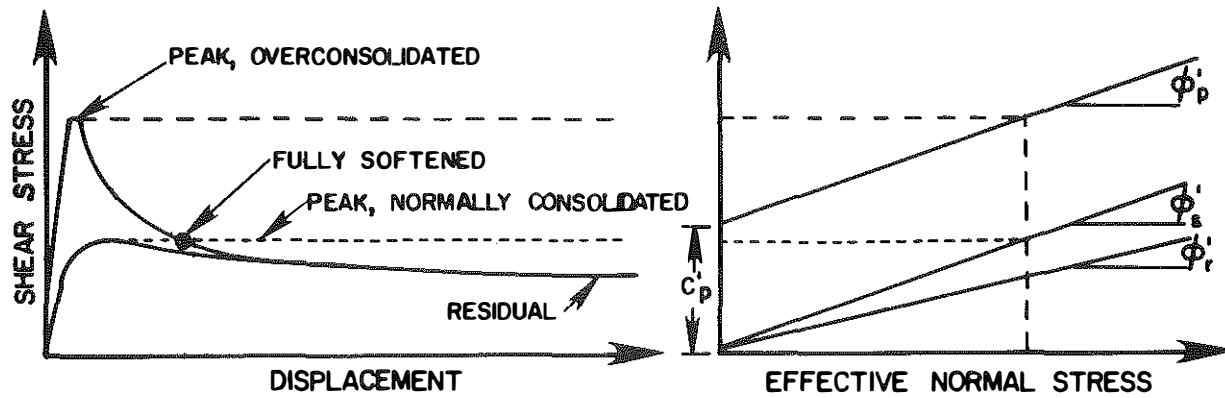
(a) SHEAR CHARACTERISTICS OF REAL CLAYS
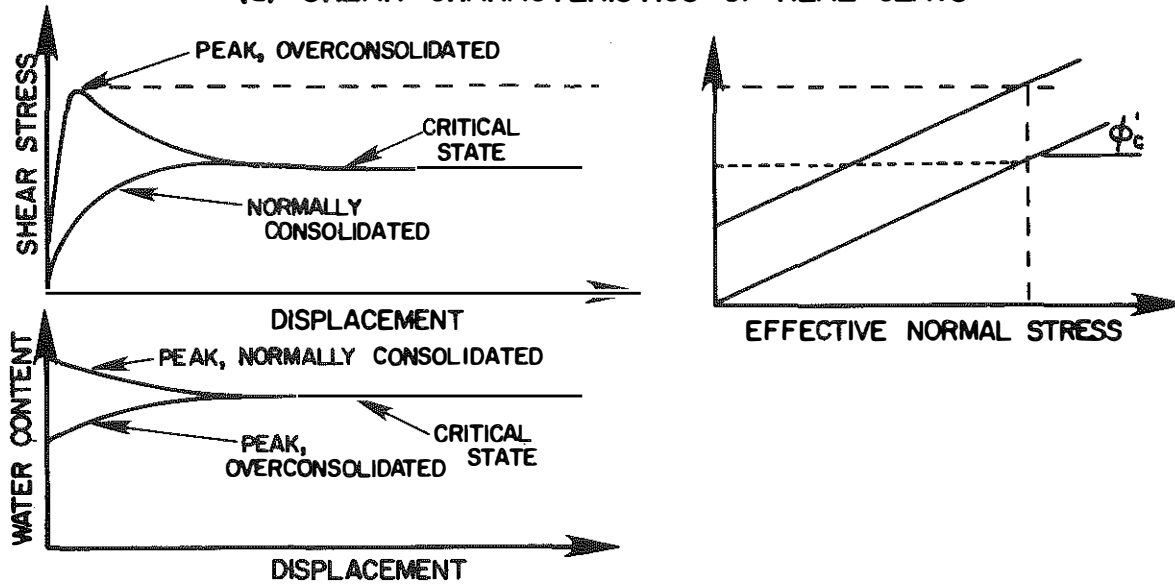
(b) SHEAR CHARACTERISTICS OF IDEAL CLAYS

Figure 16. Typical Stress-Strain Curves for Normally Consolidated and Overconsolidated Clays (same clay having different stress histories).

the peak strength of the normally consolidated clay and the stress-strain curve of the overconsolidated clay.

The critical state of a normally consolidated clay can be defined *(41)* as the state (in a drained condition) in which any further increment in shear distortion will not result in any change in water content. The water content at the critical state is equal to that ultimately attained by an overconsolidated clay due to expansion during shear. Since the critical-state shear strength of real clays cannot readily be determined, a practical approximation to the critical state might be obtained from strength tests on remolded clay. Shear strength parameters obtained in this manner may then be a practical approximation to the fully softened strength of an overconsolidated clay. The shear strength obtained in this manner corresponds to the theoretical limiting

strength of an overconsolidated clay which has undergone a process of softening as described by Terzaghi *(1)*. It is suggested herein that the critical state shear strength might be obtained from triaxial tests performed on samples (from normally consolidated clay) remolded at a water content given by

$$w_c = (0.36) \; PI + PL. \qquad 38$$

The water content given by Equation 38 may well be the water content of a clay at the critical state. Whenever an overconsolidated day is sheared, the water content in the failure zone increases to w_c; whenever a normally consolidated clay is sheared, the water content decreases to w_c.

19

To develop a rapid means of determining the shear strength parameter, ϕ'_p, attempts have been made to correlate the peak parameter (obtained from triaxial tests performed on normally consolidated clays) with plasticity index. In Figure 17, data given by Kenney *(42)* and Bjerrum and Simons *(43)* have been combined. There is considerable scatter in the data and a poor correlation between ϕ'_p and plasticity index, although there is a trend. At best, the curve gives an approximate indication of the magnitude of ϕ'_p.

The ultimate strength may be obtained from a consolidated-drained, direct shear test. The sample may be sheared in one direction or reversed several times. The sample is usually sheared at a very slow rate (2 to 4×10^{-4} inch per minute (0.005 to 0.010 mm per minute)). Generally, these tests show that the residual cohesion parameter, c'_r, is usually zero or very small. The cohesion for the normally consolidated sample is also usually zero. For the overconsolidated sample, the peak effective stress parameter, ϕ'_p, is usually large. The residual effective stress parameter, ϕ'_r, appears to be mainly dependent on the clay fraction. This dependence is illustrated in Figure 18. Using the method of least squares and assuming a straight-line fit, the residual shear strength parameter may be evaluated by

$$\phi'_r = 68.2 - 30.2 \, (\log \text{CF}) \qquad 39$$

where CF is the clay fraction (percent < 0.002 mm).

Peak Strength from Triaxial Test -- Bjerrum *(37)* assembled data on a number of failures of natural and cut slopes in overconsolidated clays and clay shales which showed that the average shear stress along the failure surface was much smaller than the shear strength measured from laboratory triaxial tests. The liquidity indices of these clays ranged from -0.51 to 0.25. In Figure 19, the back-computed effective stress angle of shearing resistance is plotted as a function of the peak effective stress parameter obtained from triaxial tests. Even neglecting the cohesion, the data plots below the line of equality. If residual shear strengths are used, there is better agreement between the computed shear strengths and those determined by direct shear tests (Figure 20).

Table 1 summarizes results of six case studies of highway embankment failures in Kentucky and illustrates some difficulties associated with using peak strengths from triaxial tests. Slope inclinometers were used to locate the shear zones and cased boreholes were monitored over a period of several months to locate the phreatic surface. Shear strengths of the soils in the embankment and foundation were obtained from consolidated-undrained triaxial tests with pore pressure measurements. Pore pressures and loads were monitored using electrical pressure transducers and strip-chart recorders. A back pressure was used to saturate the samples. Results of the CIU tests were plotted using the effective stress path technique *(45)*. In some cases, the
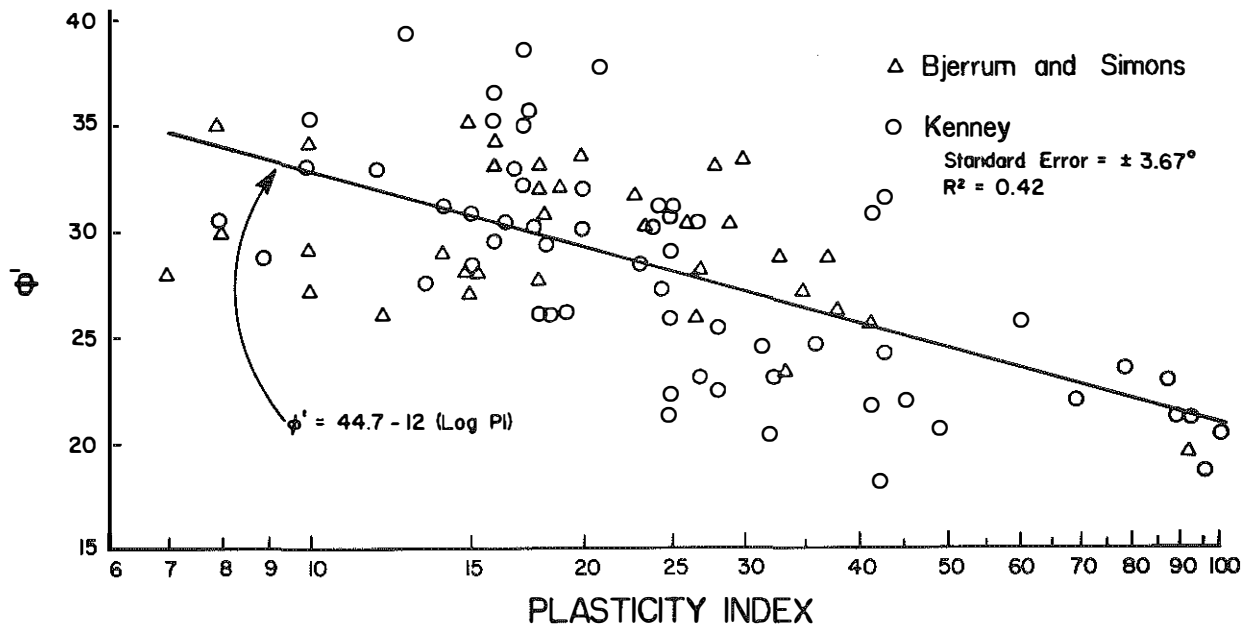
Figure 17. Peak Shear Strength Parameter, ϕ'_p, as a Function of Plasticity Index (data from Kenney *(42)* and Bjerrum and Simmons *(43)*) The parameter, ϕ'_p, was obtained from triaxial tests performed on normally consolidated clays. Curve was fitted by the authors.
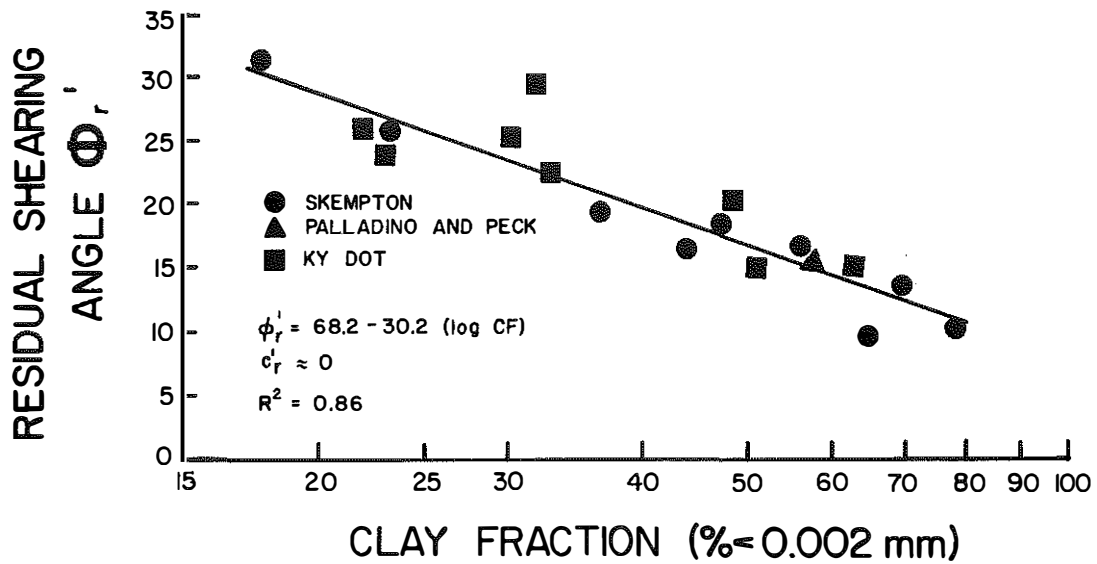
Figure 18. Residual Shear Strength Parameter, ϕ'_r, as a Function of Clay Fraction (data from Skempton *(39)*, Kentucky DOT Division of Research, and Palladino and Peck *(44)*). Curve was fitted by the authors.
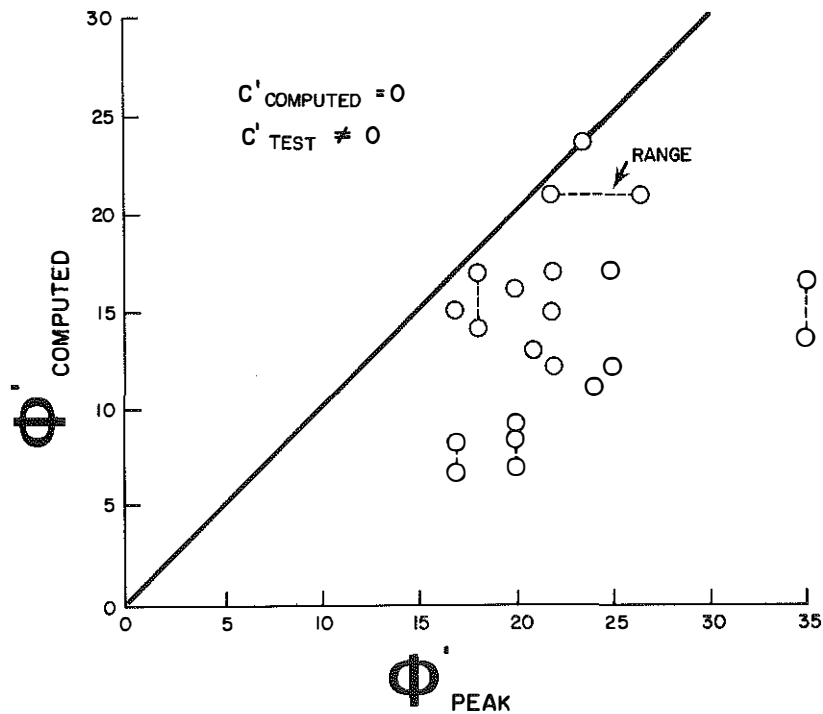
Figure 19. Back-Computed Shear Strength Parameter as a Function of Peak Shear Strength Parameter from Triaxial Tests (data from Bjerrum *(37)* and Skempton *(39)*).
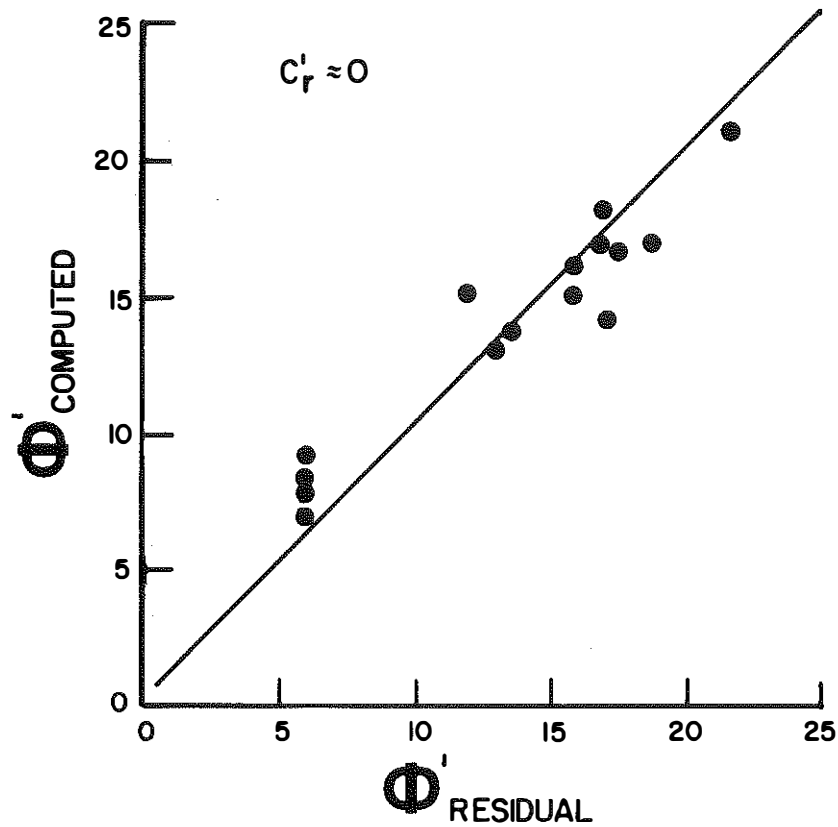
Figure 20.   Back-Computed Shear Strength Parameter as a Function of Residual Shear Strength Parameter (data from Bjerrum *(37)* and Skempton *(39)*).

TABLE I.   SUMMARY OF STABILITY ANALYSES

| LOCATION | STRENGTH PARAMETERS | | | | UNIFIED CLASSIFICATION | | SAFETY FACTOR | |
| | EMBANKMENT | | FOUNDATION | | | | | |
	ϕ' (degrees)	c' (lb/ft^2)	ϕ' (degrees)	c' (lb/ft^2)	EMBANKMENT	FOUNDATION	PEAK-STRENGTH ANALYSIS	$c' = 0$ ANALYSIS
I 64, MP 44, Shelby County	30.0	42	24.2	474	CL	CL	1.01	0.72
I 64, MP 118, Bath County	30.0 0.0	0 5227	20.8 0.0	530 4992	CL	CH and CL	1.28* 3.90**	0.97*
I 64, MP 188, Boyd County	29.4	0 to 476	24.0	0	CL	CL	>0.99	0.99
Bluegrass Parkway, MP 21	27.1	243	31.3	0	CL	OL - ML	1.45	1.19
Bluegrass Parkway, MP 44	21.8	230	.	.	CL	.	1.03	0.73
Western Kentucky Parkway, MP 96	26.7	0	25.1	475	CL	CL	2.08	1.12

*Weighted value of three cross sections
**ϕ' - equal - zero analysis

peak strength correctly predicted the average shear stress along the slide (F ≈ 1.0); in other cases, the triaxial shear strength was too large (F > 1.0).

Slope Design Dilemma -- Observations *(39)* suggest the rate of development of a continuous sliding surface in a clay slope prior to failure varies from one type of clay to another; in the stiffer clays, the rate may be very small; delay of the failure may be on the order of years. Data in Figure 13 suggest that, for clay soils having liquidity indices less than approximately -0.1 to -0.2 (very stiff clays), the failure delay may be several years. In slopes where the liquidity indices are greater, the delay in failure may be very short. Hence, engineers charged with the responsibility of designing slopes face the dilemma of having to decide which shear strength -- peak, residual, or some intermediate strength -- to use in a stability analysis. Use of residual shear strength may be very conservative and expensive, especially in cases where temporary cuts are made in overconsolidated clays.

GEOMETRY CHANGES

EROSION

Erosion of concern here is that action of flowing water either through a soil medium or over a soil surface which "breaks loose" individual soil particles and transports them to new locations. This action, over a period of time, can cause geometric changes of a slope with subsequent serious distress or complete failure of an earth structure.

Some soils are more susceptible to erosion than others. Sands and silts are generally the most susceptible, particularly those that are poorly graded with very little fines to act as a bond. This could include soils with group symbols such as SP, SM, ML, MH, and OL (Unified Classification) or A-3 soils (AASHTO Classification). Loess is particularly susceptible to erosion. It is composed of very fine sands and silts and is deposited above the water table by winds and is more permeable in the vertical direction, accounting for its unusual characteristic of eroding on vertical slopes. Clays are somewhat less susceptible than sands or silts, particularly when compacted. The best erosion-resistant soils are the coarse-grained gravels and gravel-sand and gravel-clay mixtures with group symbols GW, GP, GM, and GC.

Toe Erosion – One major cause of distress to earth structures is due to the erosion of material at the toe of a slope produced by flow of surface water in drainage channels. This can include intermittent flow such as rain water in unpaved ditches or continuously flowing streams.

The removal of material at the toe induces stresses in the structure because of loss of support. This will initiate large shear strains producing a failure surface as shown in Figure 21. An alternative method of failure would be a small slump at the toe that would cause a slump to occur higher on the slope, with this mode of failure progressively working its way up the slope.

To prevent erosion of the toe from intermittent drainage, ditches should be paved with asphalt or concrete. Construction of check dams in the flowline to reduce the velocity of flow, thereby reducing erosion, can also be used. Methods used to prevent toe erosion by rivers or streams include construction of small rock berms, dumping of cyclopian stone, and placing concrete paving or asphalt membranes or filter mats at the face of the toe.

Slope Erosion – Slopes of earth structures which are inadequately protected will also be eroded by surface water, possibly causing undue stress on the structure, resulting in failures similar to those described under toe erosion. Some examples of slope erosion include the formation of gullies on an embankment slope due to inadequate sod (Figure 22), eroding action of waves from large bodies of water on the slopes of embankments, dams, or banks, and erosion of slopes from high velocity streams as would often occur during flood stage.

Slopes are often protected by placing sod or sowing grass, as in the case of highway embankments. Concrete or asphalt paving, cyclopian stone, and in a few cases even steel plates have been used as protection against high velocity streams and on the upstream face of dams. Hand-placed riprap can also be used but is not recommended in places where trash such as driftwood might dislodge some of the stones leaving unprotected places on the slope.

Piping -- Many earth structures have failed by formation of pipe-shaped discharge channels or tunnels in the subsurface or by loss of support at the toe of such structures from erosion caused by discharge of large volumes of water under considerable pressure heads. This is defined as failure by piping.

There are two basic processes that can produce piping. One is erosion of the subsurface starting at springs in the toe and continuing upstream along lines of flow until the structure is undermined. The second process occurs when the pressure head on the water that percolates upward through the soil at the toe is greater than the bouyant weight of the soil. This results in a heaving of the soil mass at the toe of the structure and is defined as piping by heaving.

A typical example of subsurface erosion is shown in Figure 23. A stiff clay or other cohesive, erosion-resistant material will often overlay an easily
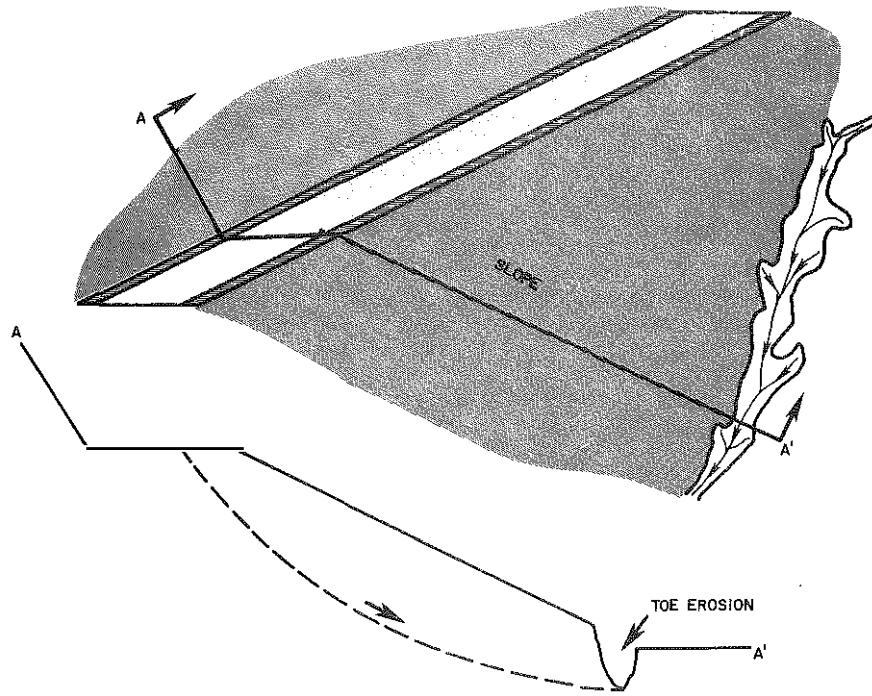
Figure 21.　Erosion at the Toe of an Embankment.
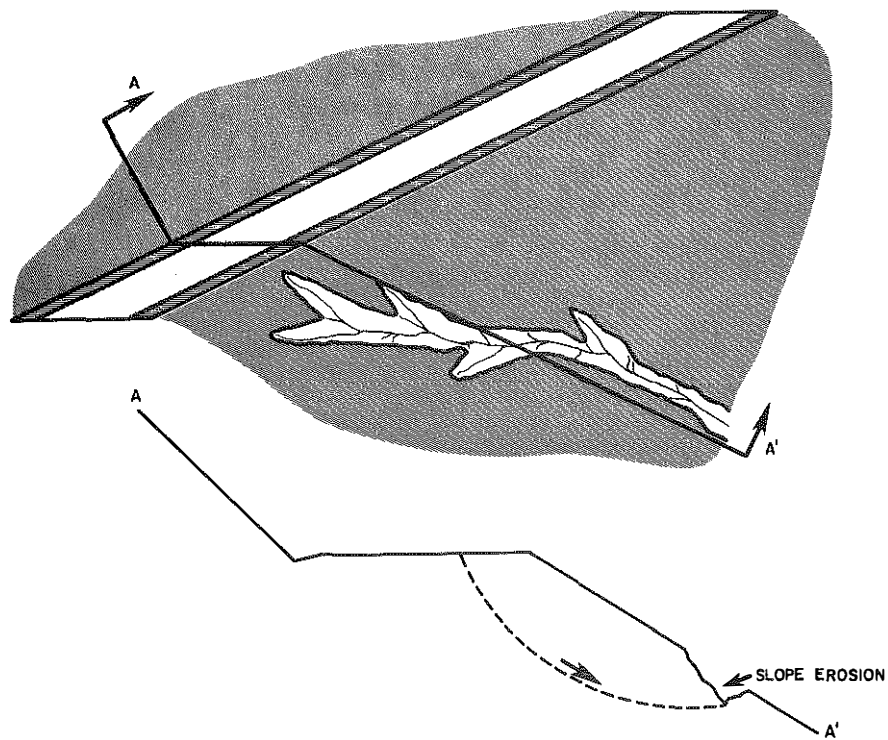
Figure 22.　Formation of Gullies on an Embankment Slope due to Inadequate Protection.
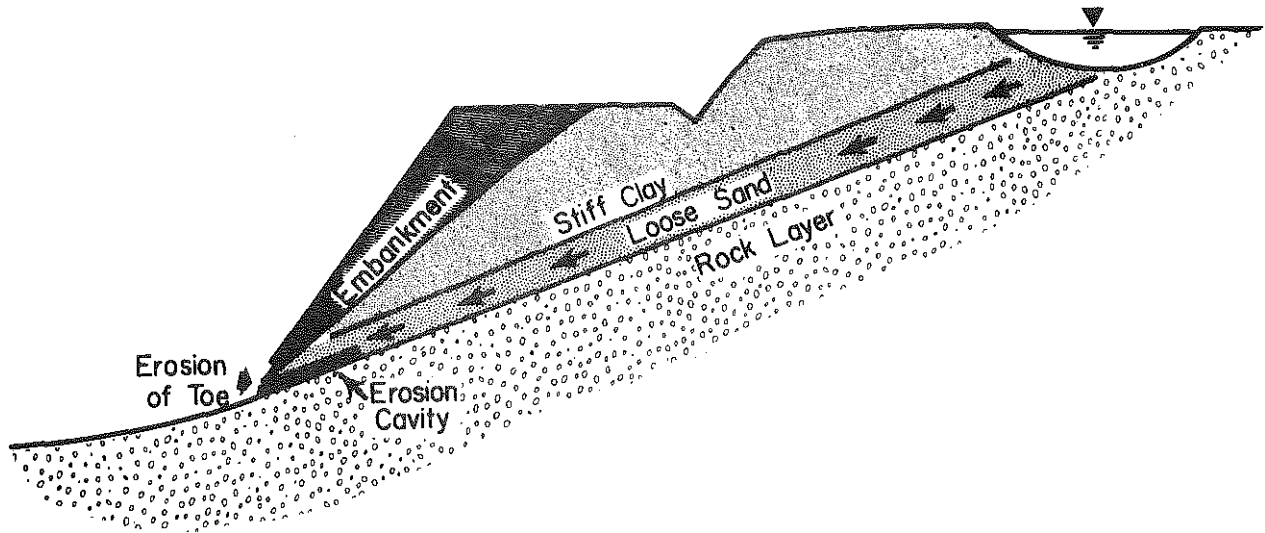
Figure 23. Illustration of Subsurface Erosion due to Piping.

erodible material including loose fine sands or silts. The foundation for the erodable material would be an impermeable layer. Water would flow through the loose material, emerging at the toe as a small spring. As material at the toe breaks loose and is carried away in the discharge, channels or pipes are formed. These continue to enlarge in diameter and erode toward the source of water until enough of the foundation material is removed to cause structural failure.

Inverted filters can be used to prevent this type of piping. When small springs are first noted in the toe area, before any major erosion has occurred, the area should be covered with an inverted, graded filter. This will prevent erosion of the toe material, allow disipation of pore pressures, and provide free drainage. If a graded filter is necessary, it could be designed using specifications recommended by Terzaghi (1).

Piping due to heave first begins with a drastic decrease in the effective stress in the material at the toe of the earth structure. As the pressure head on the water percolating upward approaches the bouyant weight of the soil, the effective stress approaches zero. This greatly increases the permeability of the soil, allowing the flow paths of the water to straighten and widen, thus permitting more flow. The soil surface then rises, springs appear, and erosion begins. This condition is illustrated in Figure 24.

A number of methods can be used to prevent heaving and, consequently, piping depending on site conditions (geometry, soil types, etc.). Relief wells at the toe can be drilled to lower piezometric levels. Toe drains (Figure 4) can be used in new construction -- for free drainage without erosion. Sheet piles driven at the toe can be used to decrease pore pressures by creating longer flow lines and greater head losses before

the flow emerges at the toe. Also, loaded inverted filters can be used to increase the effective stress at the toe. The equation for the factor of safety against heave is

$$F = \gamma_b/u. \qquad 40$$

The factor of safety against heave may be increased by the added weight of the filter material as follows:

$$F = (\gamma_b + \gamma_d)/u \qquad 41$$

where γ_d is the in-place unit weight of the added filter material. Assuming the material allows free drainage, pore pressures in the filter would be zero. Loaded inverted filters do not alter the pore pressure at the toe but simply increase the effective stress through added weight.

DAMMING

Construction of side-hill embankments as shown in Figure 25 often complicate stress and pressure distributions causing an imbalance of forces that initially were in equilibrium. This is particularly evident in the case of changes in the level of the phreatic surface.

In a hillside, the phreatic surface is in equilibrium as the seepage water drains toward the toe. However, when the embankment is constructed, a number of things can happen. Consolidation will most likely be initiated in the foundation, decreasing the permeability. If the foundation is a slow-draining material, pore pressures will increase and the effective stress will be reduced, thus decreasing shearing resistance in the foundation and causing greater instability. If, however, the foundation is a free-draining material and no pore pressures are built up during consolidation, then the

25
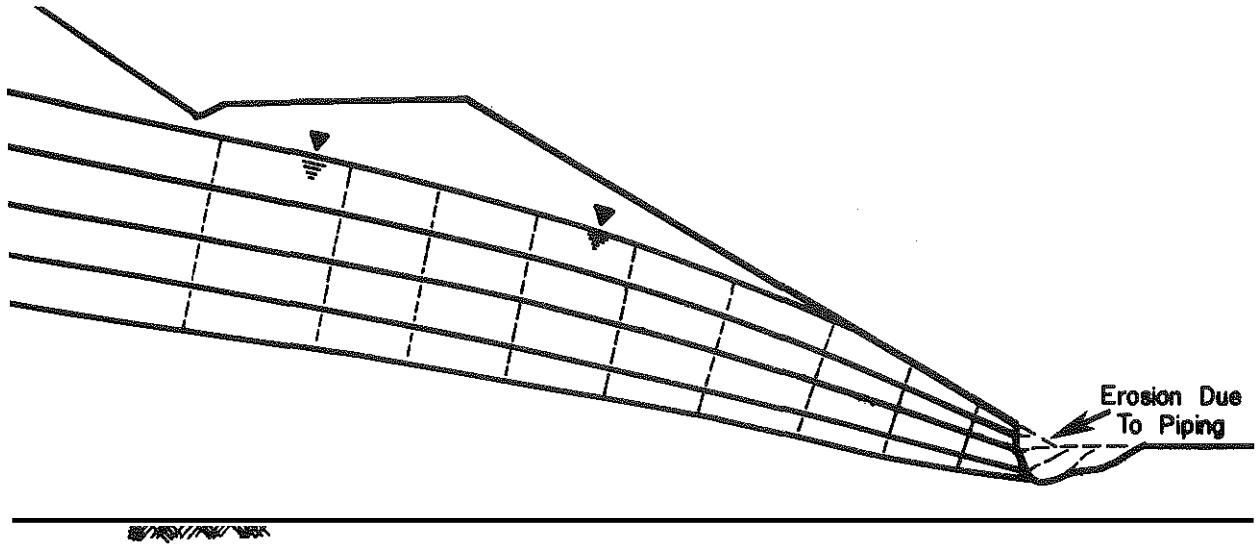
FACTOR OF SAFETY AGAINST HEAVE:
$$F = \gamma_b \text{ (at toe)}/U \text{(at toe)}$$

Figure 24.   Illustration of Heaving of a Soil Mass at the Toe of an Embankment.
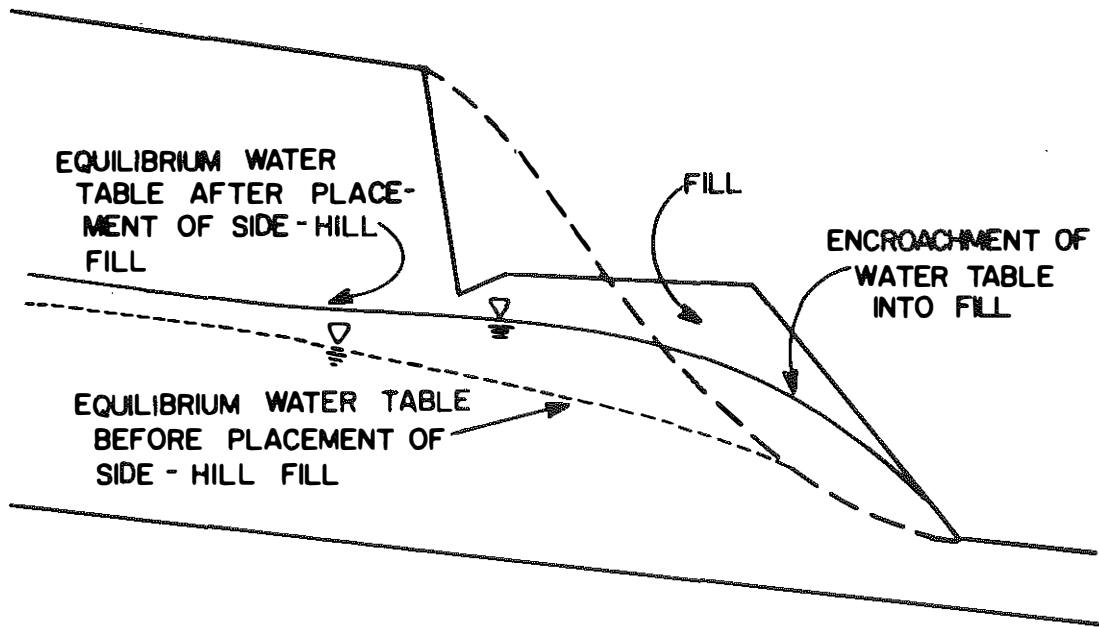
Figure 25.   Illustration of Damming Caused by the Construction of a Side-Hill Embankment.

decreased permeability will prevent future seepage from draining so freely, causing pore pressures to eventually rise.

The hindrance to free drainage of seepage water by the newly-placed embankment causes it to behave similar to a dam and initiates a rise in the phreatic surface. This rise will continue with time until a new equilibrium condition is reached. The foundation and new embankment then reach a state of somewhat greater instability because effective stresses have been reduced, seepage forces are present, and shear strength parameters are possibly reduced because of softening.

A number of methods can be used to prevent or reduce the effects of damming (Figure 4). Interceptor trenches, backfilled with granular materials, have been used in the ditchline at the top of the embankment and at the toe of the slope to intercept and lower the phreatic surface. Relief wells or horizontal drains may also be used to lower piezometric heads. However, none of the above methods, when used as corrective measures, will be very effective in poorly draining materials, such as heavy clays, that are already saturated.

A corrective measure that has been used successfully in saturated, poorly draining materials is construction of either rock or earth berms at the toe of embankments. This increases stability by adding weight to the passive wedge at the toe and increasing resisting forces.

The effects of damming can be prevented on new construction or reconstruction by placing a properly designed drainage blanket between the embankment and original ground. This prevents encroachment of the phreatic surface into the embankment.

DETECTION AND MONITORING

DETECTION

Locating and tracing ground water have been done by two old and effective but rather expensive and time-consuming methods -- borings and excavation. There have been efforts to develop methods which are as effective as borings and excavation but require less time and can be accomplished at lower costs. Several methods, such as the use of tracers, water table observations, and electrical resistivity, have been considered and studied for use in Kentucky (23).

Tracers -- By injecting a tracer in drill holes in suspected sources, movement of the tagged ground water can be traced by bailing water samples from other drill holes and measuring the concentration of the tracer material from each sampling point. Knowing the distance of the sampling holes from the injection hole and the time for the marked water to travel that distance, the velocity of the ground water movement can be calculated. Thus, speed, direction, and location of the ground water can be determined.

There are many types of tracer materials, each yielding good results under certain condidtions. Some classes of materials that have been used include dyes, chemicals, suspended particles, dissolved gases, bacteria, and radioactive isotopes.

Satisfactory results are more likely in relatively homogeneous soils having medium to high hydraulic conductivity. Therefore, poor results will be expected in heterogeneous soils with low hydraulic conductivities. An ideal tracer material must be economical, safe, not present in the original water, capable of following the water movement without altering it, non-absorbent or nearly so, capable of being detected in low concentrations, and non-reactive with the porous medium. Important factors that restrict the use of otherwise suitable tracers are absorption, dispersion, filtration, and public acceptance.

Laboratory and field work with tracers in Kentucky have been rather limited. Nevertheless, one tracer (a fluorescent dye) was tested in the laboratory and at three different field sites, and a chemical tracer (hydraulic lime) was investigated at a fourth site.

In the laboratory, a large permeameter consisting of a 3-inch (76-mm) inside diameter by 9-foot (2.7-m) long plexiglass tube was constructed and used to study the ability of fluorescent dyes to resist the filtering and absorbing action of soil. The permeameter was filled with fine, silty sand and stopped at both ends with porous disks. A solution of fluorescent dye was circulated through the sand column. After several passes through the sand filter, the solution appeared as strong as originally injected into the column, and it was concluded that sand will not remove fluorescent dyes from solution for moderate percolation distances. However, this process of rerunning the solution through the sand filter continued for a week, and the dye at the effluent end became somewhat less detectable. When the sand was emptied from the cylinder, it was found that a portion of the dye had been absorbed by the sand material.

In the field, tracers were used at four sites; all sites were either actual or potential landslides on highway embankments in Kentucky. At three sites, the same fluorescent dye (four colors -- purple, orange, red, and blue) investigated in the laboratory was used. Ultraviolet light was used to detect the presence of fluorescence in wet areas of the sites after the dye was introduced into suspected sources of seepage. In total darkness, the dyes fluoresced in distinctly different colors; but in semi-darkness, it was difficult to differentiate between red and orange and between blue and purple. The dye

was used only to verify whether the suspected source was a real one or not. No effort was made to calculate the velocity of ground water flow, and no attempts were made to measure the actual concentration of the dye in wet areas.

The use of fluorescent dyes to detect and trace water in unstable slopes in Kentucky soils has not been too successful. At one site where there was a moderate flow over short distances, the use of the dye did verify suscepted sources of several springs at the toe of an unstable embankment. It was concluded that the dye did not "seep" through the soil mass but traveled through defined channels. At other sites, the use of dyes was completely unsuccessful. Continuous observations of seepage water at the toes of the slides never indicated a trace of the dye. It was concluded that the dye was absorbed by the soil or diluted by the large volumes of water so its concentration in the seepage water at the toe could not be detected.

Hydrated lime has yet to be used successfully to trace seepage waters. Water samples obtained from drill holes at frequent intervals over a period of several months at one site were tested in the laboratory for pH, conductivity, calcium content, etc. Analysis of the of the water samples did not indicate any change due to lime.

Water Table Observations -- A network of auger holes is drilled in the area under study, and water table measurements are recorded over a considerable period of time. From these measurements, ground water contour maps can be plotted to show the change of the water table level with time. This method provides an accurate picture of the ground water level, and the movement of the water is inferred from the gradients observed on the contour maps.

The auger-hole method has been used extensively by the Kentucky Bureau of Highways to observe water table levels. The method was found to be, as expected, a good and reliable one. Continuous water table measurements in the drill holes permit the plotting of water table contour maps which show water table gradients. The latter, in turn, indicate the direction of movement of the ground water. This method, although reliable, involves extensive drilling and, thus, is time-consuming and expensive. No attempt has been made to calculate the actual hydraulic conducitivity because the interest is normally in determining maximum water table gradients and directions of flow rather than the velocity of flow. The rate of flow would, however, be useful in estimating the time needed for an embankment to become saturated.

Figure 26 is a typical example of a water table contour map. Soon after soil movement was detected at this site, observation wells were drilled to monitor the water table. Elevations of the water table were obtained over a considerable time, and contour maps prepared. The average of water table readings over a period of 1 year have been plotted in Figure 26. The figure shows that water seeps from a nearby hill south of the eastbound shoulder, between Stations 6923+00 and 6925+00, in a northwesterly direction. It also indicates there is a mound of water along the westbound shoulder between Stations 6922+10 and 6923+15.

Electrical Resistivity -- The resistivity technique can be used in landslide investigations because of the effect of higher water contents at the slip surface or shear zone upon the measured resistivity values. Several states have used this method in demonstration tests on landslide problems; only in a few instances, however, has a complete and thorough survey of a landslide been made.

This method is based on the measurement of earth electrical resistivity in the area under investigation. The application of this method is not simple, however, due to the many different types and conditions of subsurface soils. At times, different earth materials under different conditions have approximately the same electrical resistivity. This sometimes causes confusion and makes it difficult to differentiate between a wet layer of one type of soil and a dry layer of a soil of a different type. Therefore, knowledge of the geology of the area under investigation is essential when this method is used, and calibration tests with the resistivity apparatus over exposures of formations believed to be typical of those in the area must be performed.

Water in the pores of soil alters the conductivity of the soil-void system to such an extent that resistivity measurements can be used to assess the hydrological condition of the subsurface. It is impossible to recognize the presence of water by a specific value of earth conductivity. However, where the presence of an acquifer has been established by bore holes or wells, it is possible to correlate resistivities with water-bearing formations.

An investigation was undertaken to establish a correlation between some function of resistivity and a corresponding measure of moisture content. Three methods were used to determine representative moisture content values for a specified depth interval:

  (1) using the moisture content at the centriod of the zone of influence of the resistivity measurements (approximated by a semicircle),

  (2) using the moisture content at the specified depth, and

  (3) using the moisture content at the center of the specified depth interval.

Moisture content values were plotted against values of a resistivity function for the same depth interval. Two functions of resistivity were used:
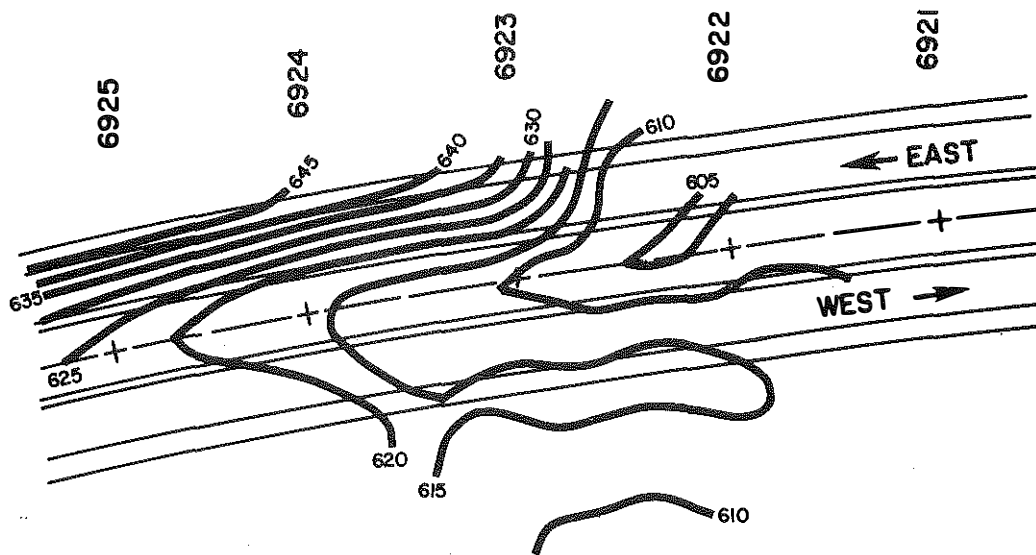
WATER TABLE CONTOURS

Figure 26.   Example of a Water Table Contour Map.

(1) the ratio of the specified depth to the accumulative resistivity at that depth, and
(2) the specific resistivity at the specified depth.

Apparent specific resistivity values are of limited use since it is impossible to recognize water by a specific value of soil resistivity. Water in the pores of soil changes the resistivity to such an extent that the resistivity of the earth minerals is almost negligible. Thus, the moisture content and electrical conductivity of water are the major factors that affect earth resistivity, and the specific value of resistivity will depend greatly on the conductivity of the pore water. However, depth/accumulative resistivity versus moisture content curves showed an increase in the value of depth/accumulative resistivity for an increase in moisture content.

Since depth/accumulative resistivity versus moisture content curves do show a correlation between resistivity and moisture content, though disappointingly weak, it follows that a knowledge of the variation of resistivity over an area will reveal some information of the variation of mositure content over the same area. This information can be obtained by plotting contours of depth/accumulative resistivity (see Figure 27 for an example). Higher values of the resistivity function correspond to higher moisture contents. The contours indicate higher moisture contents in the area right of Stations 46+50 and 47+00, the area of failure, and thus

show excellent agreement with actual conditions.

Summary -- Clean, fine sand did not remove fluorescent dye from solution for moderate percolation distances, but the dye became somewhat less detectable due to its absorption by sand for longer percolation distances. Conventional monitoring equipment (ultraviolet light) was not sufficiently efficient to monitor low concentrations of the dye. The tracer method was not dependable for the purpose of locating seepage waters and was shown to require further improvement. However, it was used to verify a suspected source of seepage when the ground water was believed to have traveled through channels or very porous material.

The water table observation method was the most definitive and useful of the methods studied for tracing and locating seepage water.

The electrical resistivity method did not yield very accurate and dependable results. However, when the results were correlated with actual moisture conditions, they showed fairly good agreement. Knowledge of the geology of the area under study and the electrical properties of the subsurface material prior to resistivity testing is essential to obtain meaningful results. However, in the case of landslides, it was rather difficult to apply such calibration investigations since each slide area has different subsurface conditions which are peculiar to the slide itself.

29

46+50 46+75 47+00 47+25 47+50 47+75

.010

.020

.024

← SOUTH

NORTH →

.010

.020

.030

.040

.050

CONTOURS OF DEPTH/ACCUMULATIVE RESISTIVITY
DEPTH — 0'- 12'
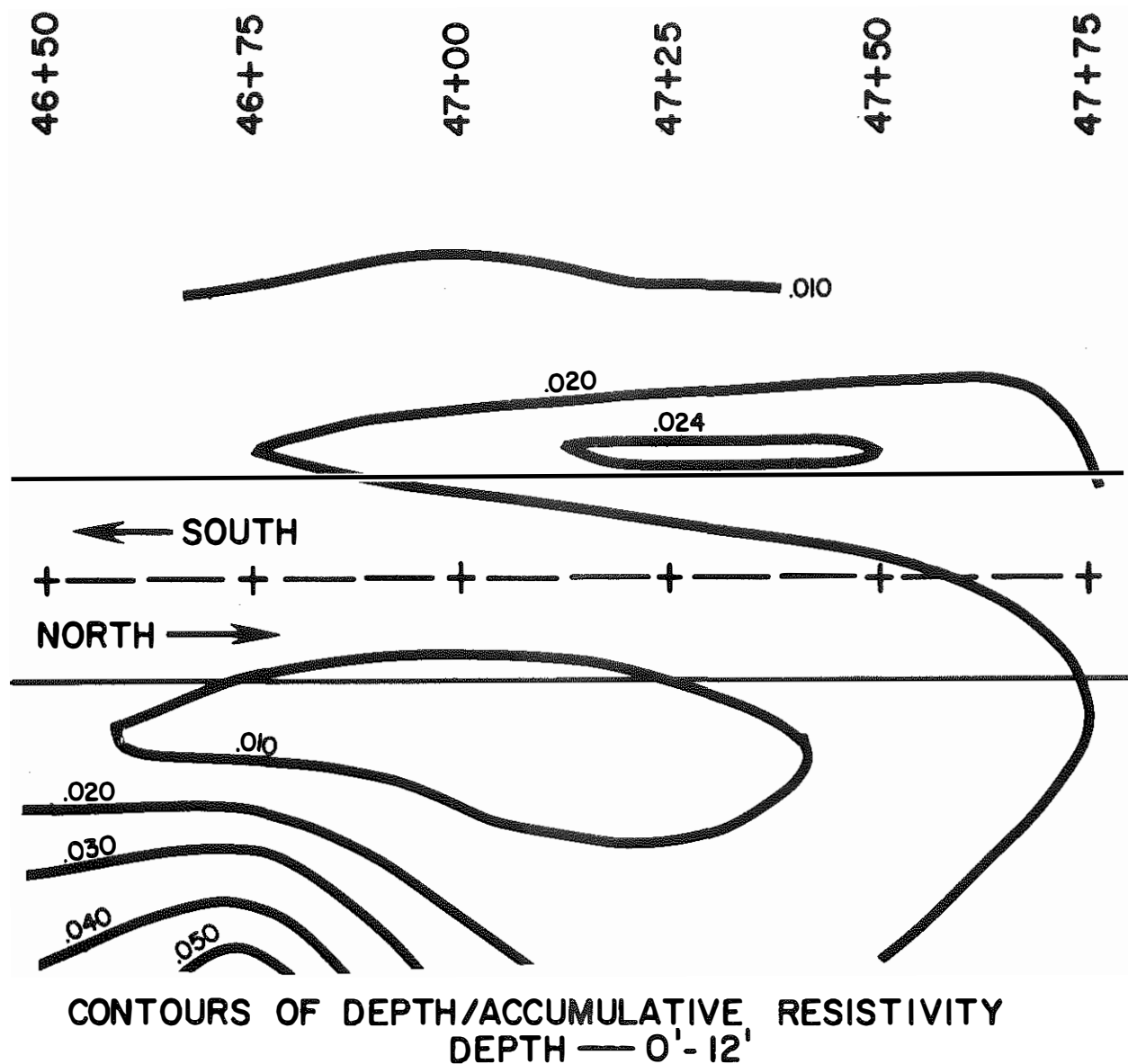
Figure 27.   Example of Contours of the Ratio of Depth to Accumulative Resistivity.

MONITORING

Since it has been shown that the strength characteristics of a soil mass are significantly influenced by effective stresses, it is necessary that geotechnical engineers be able to measure and predict changes in pore water pressures. In many situations, it is impossible to predict adequately pore water pressure changes. In such cases, it is necessary to measure and record pore water pressures at strategically selected points in the soil mass as a means of construction control or to check stability of foundations and slopes.

The basic instrument for measuring pore water pressures is the piezometer. A piezometer consists of

a porous element placed in the ground so the soil water is continuous through the pores of the element. Provision is made to measure either the level of the water in the piezometric system or to measure the pressure of the water in the system, thus providing a measure of pore water pressure in the soil mass at that particular point. To adequately measure pore water pressures, it is necessary that any piezometer should

    (1)   record the water pressure accurately within known limits of error,

    (2)   cause a minimum of disturbance to the soil in which the element is placed,

    (3)   respond quickly to changes in ground water

conditions,

(4) be rugged and reliable and remain stable over long periods of time, and

(5) be capable of recording pore water pressures either continuously or intermittently, as required.

The basic problem with any piezometric system is that a finite flow of water from the adjacent soil into the porous element is required to pressurize the system. Thus, the measuring system is unable to record a change in pore pressures immediately. As pore pressure conditions in a soil change, it is necessary that a flow of water into or out of the piezometer element occur before equilibrium is reached. This requires a finite time and depends primarily on the soil permeability. It can also be seen that the permeability of the piezometric porous element can be important in such instrumentation. It has generally been concluded that pore water pressures will be measured adequately if the piezometric element is at least ten times more permeable than the surrounding soil mass.

Piezometer Types – The simplest ground water recording technique is that described previously -- to observe the water level in an open borehole. The surface area through which the water enters the borehole is normally large. Unless the soil is coarse grained, a large time lag results. Different layers of soil, which may be under different water pressures, are interconnected by the borehole and the level of the water in the borehole may have little if any relationship to pore water pressures at a particular point. Some of the disadvantages of the open borehole may be overcome by using casing extended to the level at which water pressure measurements are desired. Even so, leakage through casing joints and seepage from adjacent strata may still occur and obscure small changes in pore water pressures.

To reduce the time required for equalization of pressures between the piezometer and the soil, a riser pipe of small diameter is used and connected to a porous element. The small diameter pipe requires less volume of water flow to occur before the instrumentation is able to detect a pressure change. The annulus between the riser pipe and the borehole is backfilled and the porous element, usually referred to as a tip or well point, is connected to the riser pipe and placed in a layer of sand or gravel. Porous tips are usually about 1 1/2 to 2 inches (38 to 50 mm) in diameter and up to 1 1/2 feet (0.6 m) long. The most common tip is a nonmetallic ceramic stone developed by Casagrande. Such tips are susceptible to damage during placement; this has lead to the development of other types of porous tip elements comprised of various porous protective sheaths or filters made of metals.

Because of the time lag associated with the flow of water from fine-grained soils into or out of the piezometer, it is common practice to provide the piezometer with two lines or standpipes. One of the lines passes to the bottom of the piezometric element and the other terminates at the top of the element. Such an arrangement allows the piezometer to be flushed after installation to remove air bubbles.

The electrical piezometer makes use of a diaphragm which is deflected by the water pressure in the soil mass. The deflection of the diaphragm is measured by means of various types of electrical transducers. Such a device has a very small time lag and thus is very sensitive. The most usual methods to measure the deflection of the diaphragm are vibrating wire gages, resistance strain gages, or capacitance strain gages. There are several types of electrical piezometers -- the main differences between types being the type of transducer element used. Such piezometers are useful in situations requiring rapid response or where dynamic loadings are expected.

The pneumatic piezometer consists of a porous tip which contains a pressure-sensitive valve. When the water pressure on either side of the valve is equalized, the pressure in the standpipe line is a measure of the pore water pressures in the soil. Such piezometers have very small time lags in that a very small volume change is required to operate the valve. The instrument is simple to operate and has a long-term stability sometimes not provided by other piezometer types.

A number of piezometer types are available for use in special circumstances. Special considerations must be given to the measurement of pore water pressures in compacted embankments inasmuch as such soil structures often exist in a partially saturated state. Consequently, both air pressures and water pressures exist in the pore spaces. Over the years, specially designed shapes of twin-line hydraulic piezometers and electrical piezometers have been developed to measure pore water pressures in such circumstances. Special devices have also been developed to measure pore water pressures at the boundaries between subsurface structures (such as walls, piles, and culverts) and the surrounding soil mass. Special consideration must be given in such installations to the environment in which the piezometer will be used, in particular to the problems of installation and protection during construction.

Installation and Operation – Most piezometers are installed within a permeable layer near the base of the borehole, sealed both above and below to isolate the point of water pressure measurement from adjacent soil layers. If the in situ permeability of the soil is very low, there is difficulty in forming a plug or seal, with a permeability of less than that of the surrounding soil,

above and below the piezometric element.

Air can enter the piezometer system through the walls of the tubing, which may be slightly permeable to air, or through the porous tip if the soil is partially saturated. Air dissolved in the soil water may also eventually accumulate in the piezometer system. To control the entry of air through the piezometer tip and to eliminate air from the system, it is necessary to flush the system periodically. It is also necessary to protect exposed portions of piezometer systems from vandalism and from construction or maintenance activities.

Three techniques normally used to record piezometric readings include mechanical methods, electrical methods, and manometers. The simplest method is to use a single plumbline to detect the water surface. It is also possible to use a coaxial cable which will indicate that the end of the line is at the water level when an electrical circuit is closed. When the water level rises above the level of the measuring instrumentation, a Bourdon pressure gage may be used. However, such gages are not too accurate and very small changes in high pressures values may be difficult to detect. The electrical transducer system used in some piezometers is a relatively expensive method of recording data. It is not always reliable under field conditions, especially under long-term usage; it is impossible to recalibrate once the piezometer has been installed. It does have the advantage that the elevation of the piezometric tip does not control the position of the recording station. Also, the electrical piezometer is able to provide continuous and automatic recordings. The most reliable method of recording is the mercury monometer system. Such systems do not require calibration or zero correction, and their sensitivity does not alter with the pressure range.

CONCLUDING REMARKS

A study of the various problems illustrated in this paper demonstrates the importance of knowing the distribution of excess hydrostatic pore water pressures. To make reasonable estimates of slope performance, it is necessary to have knowledge of expected changes in pore water distributions within the earth mass. The engineer is in somewhat of a dilemma with regard to water as it effects the stability of earth slopes. It is desirable to have water present in appropriate amounts and properly distributed to facilitate the placement of earth materials in engineered constructions. On the other hand, the presence of water in unknown and(or) uncontrolled quantities and manners may invalidate design analyses and result in significant problems during and after construction. A review of the many case histories concerning unstable slopes throughout the world illustrate that the presence of water and its associated excess pore pressures have been at least a partial, if not a significant, factor related to the instability of the slopes.

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A

B

DESIGN DRAINED SHEAR STRENGTHS, φ' and c'
(CONSIDER THE FOLLOWING)
o Upper Limit -- Obtain φ' and c' from CID and(or) CIU Tests (see Note 7)
o Intermediate -- Estimate from Normally Consolidated Samples (CID and(or) CIU Tests) Remolded at

$w \approx 0.36 \, PI + PL$ (see Note 8)

o Lower Limit -- φ' Obtained from Consolidated-Drained, Direct Shear Tests; Preliminary Estimate May Be Obtained from

$\phi' \approx 68.2 - 30.2 \log CF$

o Shear Strength of Fill (see Note 3)

DESIGN DRAINED SHEAR STRENGTH, φ' and c'
o Establish φ' and c' from CID or CIU Tests
o Shear Strength of Fill (see Note 3)

DESIGN SAFETY FACTOR
o Consider F ≈ 1.5 for Intermediate Shear Strength
o Consider F ≈ 1.1 for Lower Limiting Shear Strength (see Note 9)

DESIGN SAFETY FACTOR
o Consider F ≈ 1.5

F Too Low ←

o From Total Stress Analysis, Is Safety Factor:
o Too Low?
o Okay?

F Okay

o See Note 10
o LI > 0.36? -- Effective Stress Analysis Is Optional
o LI > 0.36?
o LI < 0.36?

LI > 0.36

LI > 0.36 (Optional)
LI < 0.36

STABILITY OF SLOPE: EFFECTIVE STRESS ANALYSIS (DRAINED STRENGTH) -- LONG-TERM ANALYSIS
o Select Slope Stability Method (see Note 6)
o Assume Change In Pore Pressure Is Zero, Δu = 0
o Obtain Pore Pressures from Groundwater Level Observations and Flownet

Stage Loading

o Redesign?
o Stage Loading?

Redesign

F Too Low

o F Too Low?
o F Okay?

F Okay →

o Special Loading Conditions?

No →

Yes

REDESIGN CONSIDERATIONS
o Decrease Slope
o Add Berms
o Change Drainage Conditions

C

SPECIAL LOADING CONDITIONS
o Rapid Drawdown -- Consider F (As Low As) ≈ 1.2 (see Note 11)
o Toe Erosion
o Slope Erosion
o Piping
o Damming -- Obtain Pore Pressures from Predicted Flownet (see Note 12)
o Earthquake
o Perform Effective Stress or Total Stress Analysis

STAGE LOADING DURING CONSTRUCTION
o Select Slope Stability Method (see Note 6)
o Effective Stress Analysis (Short-Term) and Consolidation Analysis; Must Estimate Pore Pressure Changes, Δu (see References 3 and 15), and Stress Distribution within Soil Mass
o Change In Pore Pressure (Undrained Conditions):

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

Obtain from Triaxial Tests
o Obtain Initial Pore Pressures from Groundwater level Observations and Flownet
o Where F Is Low, Consider Installing Piezometers to Monitor Pore Pressures -- Check Stability during Construction

F Too Low

o F Too Low?
o F Okay?

F Okay

o Consider Economics of Design
o Select Final Slope Geometry (see Note 13)

FLOW CHART AND SUGGESTED GUIDELINES
FOR THE
DESIGN OF EARTH SLOPES

SUBSURFACE INVESTIGATION
- o Boring Plan
- o In Situ Shear Strength
- o Disturbed and Undisturbed Soil Samples
- o Soil and Rock Profiles
- o Geologic Features and Settings
- o Ground Water Levels and Flow Patterns
- o Design and Construction – Local Experience
- o Geophysical Tests (see Reference 1)

↓

ROUTINE LABORATORY SOILS TESTS
- o Visual Descriptions
- o Atterberg Limits, LL, PL, PI
- o Particle-Size Analyses, CF (< 0.002mm)
- o Natural Water Contents, w
- o Unconsolidated-Undrained Triaxial Tests and(or) Unconfined Compression Tests, Referred to as UU Tests
- o Plot PI and LL on Plasticity Chart
- o Unit Weights
- o Liquidity Indices, LI

↓

SLOPE GEOMETRY
- o Establish Soil Profiles from Field Borings and Plots of LL, PL, w, s_u, CF, and LI as a Function of Depth
- o Select Preliminary Slope Geometry

↓

◇ Decision:
- o LI < 0.36?
- o LI > 0.36 and LL > 70%?
- o LI > 0.36 and LL ≤ 70%?

Branches: **LI < 0.36** (left), **LI > 0.36 and LL > 70%** (right), **LI > 0.36 and LL ≤ 70%** (center)

LI < 0.36 branch:

DESIGN UNDRAINED SHEAR STRENGTH, s_s
(CONSIDER THE FOLLOWING OPTIONS, ESPECIALLY WHEN WATER IS PREVALENT)
- o Estimate "Softened" Strength (see Note 2) from

$$s_s \approx 0.242\ s_u\ (0.0187)^{LI}$$

- o Estimate s_s from UU Tests on Samples Remolded at

$$w_c \approx 0.36\ PI + PL$$

and Consolidated (No Drainage) to Estimated Average In Situ Overburden Pressures
- o Estimate s_s (= q_f) from Plot of w_f (Failure Water Content) as a Function of q_f; Obtain Necessary Data from CID and(or) CIU Tests Performed on Samples (Normally Consolidated) Remolded at

$$w_c \approx 0.36\ PI + PL$$

Enter Graph of q_f vs w_f with w_c To Obtain q_f (= s_s)
- o s_u from UU Tests (Uncorrected) – Peak Strength
- o Might Include Shear Strength of Fill in Analysis (see Note 3)

↓

DESIGN SAFETY FACTOR, F
- o Consider F ≈ 1.3 to 1.5 Using Corrected Strength, s_s (see Note 4)

LI > 0.36 and LL ≤ 70% branch (center):

DESIGN UNDRAINED SHEAR STRENGTH, s_u
- o s_u from UU Tests and(or)
- o s_u from Field Vane Tests – Correct according to

$$(s_u)_{field} = (s_u)_{vane}/(0.747 + 0.0153\ PI + 0.0007\ PI^2)$$

- o Might Include Shear Strength of Fill in Stability Analysis if LL of Foundation is Relatively Low
- o If LL of Foundation is Near 70 Percent, Consider Ignoring Strength of Fill – Assume Vertical Crack in Fill
- o If LL of Foundation is Intermediate, Might Assume Fill Has Same Shear Strength as Foundation (see Note 3)

↓

DESIGN SAFETY FACTOR, F
- o For Homogeneous Soils, Low Risks, Relatively High Permeabilities, and Shear Strengths from UU Tests and(or) Corrected Field Vane Tests: F (At Least) ≈ 1.3 (see Note 5)
- o For Heterogeneous Soils, High Risks, Relatively Low Permeabilities, and Shear Strengths from UU Tests and(or) Corrected Field Vane Tests: F (At Least) ≈ 1.5

LI > 0.36 and LL > 70% branch (right):

DESIGN UNDRAINED SHEAR STRENGTH
- o s_u from UU Tests and(or)
- o s_u from Field Vane Tests – Correct According to

$$(s_u)_{field} = (s_u)_{vane}/(0.747 + 0.0153\ PI + 0.0007\ PI^2)$$

- o Ignore Strength of Fill – Assume Vertical Crack in Fill (see Note 3)

↓

DESIGN SAFETY FACTOR, F
- o For Homogeneous Soils, Low Risks, Relatively High Permeabilities, and Corrected Field Vane Strengths: F (As Low As) ≈ 1.3 (see Note 5)
- o For Heterogeneous Soils, High Risks, Low Permeabilities, or Undrained Strengths from UU Tests: F (At Least) ≈ 1.5

All branches lead to:

STABILITY OF SLOPE: TOTAL STRESS ANALYSIS (UNDRAINED SHEAR STRENGTH, s_u) – END-OF-CONSTRUCTION
- o Select Slope Stability Method (see Note 6)
- o Uncorrected Shear Strength
- o Repeat For Corrected Shear Strength if Correction Made

(C) →

↓

◇ Decision:
- o LI ≤ 0.36?
- o LI > 0.36?

LI ≤ 0.36 → (A)   **LI > 0.36** → (B)

35

TECHNICAL NOTES AND COMMENTS
ON
FLOW CHART AND SUGGESTED GUIDELINES
FOR THE
DESIGN OF EARTH SLOPES

NOTE 1 -- These design guidelines apply mainly to the design of slopes in clayey soils against a "first-time" failure. In using these guidelines, considerable judgement and knowledge concerning principles of soil mechanics and testing techniques are required, especially in the selection of design safety factors. The intent of these guidelines is not necessarily to present an absolute design method but rather to propose a framework for identifying situations where an analysis may yield safety factors which may lead to false impressions concerning the stability of a slope. Additionally, these guildlines may serve the purpose of providing the user with a "checklist" of items which should be considered in the design of earth slopes.

NOTE 2 -- The methods listed for estimating the "softened" shear strength are untested and should be used cautiously. These methods are merely suggested as possible ways of obtaining the undrained shear strength that might prevail in the failure zone of a slope composed of overconsolidated plastic clay.

NOTE 3 -- Whether or not to include the peak strength, some intermediate shear strength, or no shear strength of the embankment (located on a clayey foundation) in a slope design presents a difficult problem. Based on experience in Kentucky, in several cases involving embankments located on overconsolidated clays (LI < 0.36) and where complete failure occurred, the shear plane passed through the fill material at an angle (from visual observations) and was not vertical. These observations were also confirmed by slope inclinometer measurements. Hence, these observations strongly indicated that the shear strength of the fill material was fully, or at least partially, mobilized.

In contrast, where embankments were located on soft clay foundations (LI > 0.36), failures were generally preceded by what appeared to be vertical cracks in the fill material. Data presented by Bjerrum (2) indicated that in many cases where liquid limits of the clayey foundations were greater than 70-90 percent, embankment failures were preceded by the formation of cracks in the fill material. Placement of a stiff fill material on a soft clay foundation initially generates large tensile stresses in the fill material. Since soils cannot sustain large tensile stresses, there is a tendency for a vertical crack to form in the stiffer material. Hence,

the assumption that a vertical crack will occur in the stiffer fill material, and, therefore, the fill has no shear strength appears reasonable. In some cases cited by Bjerrum where the liquid limits of the clayey foundations were below approximately 70-90 percent, the shear strength of the fills was apparently fully mobilized.

NOTE 4 -- The use of a design safety factor of 1.3 to 1.5 and the "softened" shear strength may lead to a conservative design. However, use of this shear strength is suggested as a means of assessing the stability of a slope near failure. Whether or not the stability of the slope can conform to the safety factors shown and remain within the realm of an economical design must be judged by the designer.

NOTE 5 -- A design safety factor as low as 1.3 should be used discriminately. Although a number of cases (where LI > 0.36) showed an accuracy of ± 15 percent may be expected in the estimate of safety factor (3) based on undrained shear strength, to rely on such accuracy, considering the number of factors that influence the final evaluation of an embankment and its foundation, may be expecting too much. The safety factor must compensate for many factors not explicitly considered in the analyses.

Such factors as sampling disturbance, sample orientation in the laboratory test, anisotropic properties of the soils, sample size, and rate of shearing of the laboratory sample influence results obtained from laboratory tests in varying degrees. Additionally, the variability (both horizontally and vertically) of shear strengths at a given site poses a particularly difficult problem in the selection of a foundation design shear strength(s). Moreover, any engineer who has been charged with the responsibility of investigating a failure and whose intent is to compare laboratory shear strength with the in situ shear strength prevailing at the time of failure may have a tendency toward preconceived notions regarding the failures; that is, the engineer realizes or assumes the safety factor at failure is one and, in selecting the laboratory shear strength for the analysis, such an assumption may bias or influence the final arguments published by the investigator. A pragmatical examination of many published case histories which form the basis for using low safety

factors *(3)* was beyond the scope of these design guidelines. Nevertheless, such examination might be useful in the development of better guidelines than those suggested herein and might provide additional evidence that safety factors as low as 1.3 are justifiable for clays having liquidity indices greater than 0.36.

NOTE 6 -- Practically all slope stability analysis techniques are based on the concept of limiting plastic equilibrium. The problem which arises in formulating a method for determining the stability of a slope is that the soil mass bounded by the slope and the assumed shear surface is statically indeterminate. To make the problem statically determinate, assumptions must be made concerning unknown quantities. This situation has led to the evolution of some 20 methods for computing the safety factor of a slope and undoubtedly has contributed, at least from a practitioner's viewpoint, to some confusion concerning the selection of a slope stability analysis method. The basic differences among the many methods involve the assumptions required to obtain statical determinacy and the particular conditions of equilibrium which are satisfied. Furthermore, some methods are restricted to circular shear surfaces. Although a complete statement of these differences is beyond the scope of this discussion, a few comments are offered below to provide some perspective to the problem of selecting a slope stability method.

Slope stability methods may be broadly divided into two categories:

1. *Methods which Consider only the Equilibrium of the Soil Mass Bounded by the Shear and Slope Surfaces* -- Included in this category are such procedures as the friction circle method, Frohlich's method, the ϕ-equal-zero method, the logarithmic spiral method, Culman's plane shear surface method, and Bell's method. The former three methods are limited to a circular arc; Bell's method is applicable to any general shape of the shear surface. Except for Bell's method, these procedures are applicable only to homogeneous soils. These methods have limited use.

2. *Methods in which the Soil Mass Bounded by the Slope and Shear Surface Is Divided into a Number of Slices* -- The basis of the method of slices is that the normal stress acting at a point on the shear surface is mainly influenced by the weight of soil lying above that point. A distinctive advantage of the procedure of slices over the methods (except Bell's method) mentioned above is that the method of slices is applicable to multilayered soils where the shear strength is a function of the normal stresses acting at the base of the shear surface. The method of slices can be used to investigate the effect of forces acting at the sides of the slices. The method also provides opportunity for judging the reasonableness of a solution; usually, the distribution of stresses along the shear surface is reasonable if the distribution of the side forces and their location yield a reasonable distribution of stresses within the soil mass. The various methods of slices may be broadly divided into three categories based on the number of equilibrium equations satisfied by the method:

A. *Methods which Satisfy Overall Moment Equilibrium* -- Included in this classification is Fellenius' ordinary method of slices *(4)*. This method has been used extensively for many years because the method is applicable to multilayered soils and is very amenable to hand calculation. In the ordinary method of slices, only the overall moment equation is satisfied; side forces on each slice are ignored, and it is assumed the forces on the side of each slice have zero resultant in the direction normal to the failure arc for that slice. The method is applicable only to circular shear surfaces. As shown by Bishop *(5)*, the ordinary method of slices is inaccurate when applied to ϕ'-c' soils. For these types of soils, Bishop showed that, in many problems, the ordinary method of slices may yield safety factors 10 to 15 percent below the range of equally correct answers. Where high pore pressures are present, this method may yield safety factors which may be in error as much as 60 percent. For c'-soils (ϕ' equal zero or very small), the ordinary method of slices gives answers which are essentially the same as those obtained from more accurate methods.

Also included in this category is Bishop's modified procedure of slices *(5)*. In this method, overall moment and vertical (implicitly satisfied) force equilibrium equations are satisfied. However, for individual slices, neither moment or horizontal equilibrium are completely satisfied. Bishop's modified procedure is applicable only to circular arcs. Although equilibrium conditions are not completely satisfied, Bishop's method is, nevertheless, a very accurate slope stability procedure and is recommended for most routine work where the shear surface may be approximated by a circle.

B. *Force Equilibrium Methods* -- Several methods have been proposed which satisfy only the overall and individual slice vertical and horizontal forces equilibrium; moment equilibrium is not explicitly considered in these procedures. However, these methods may yield accurate solutions if the side force assumption (inclinations of the side forces) are made in such manner that the assumed inclinations of the side forces generally satisfy implicitly moment equilibrium. Arbitrary assumptions of the inclinations of the side forces have a large influence on the safety factor obtained from force equilibrium methods. Depending on the inclinations of the side forces, a range of safety factors

may be obtained in many problems. Force equilibrium methods should be used cautiously, and the user should be well aware of the particular side-force assumption used.

Force equilibrium procedures include Lowe and Karafieth's method, Corps of Engineers' modified Swedish method, Seed and Sultan's two sliding-blocks method, and the Corps of Engineers' three sliding-wedges method. The assumption that the inclinations of the side forces are horizontal, although conservative, generally produces low safety factors when compared to methods which use more reasonable side force inclinations. In general, safety factors obtained from force equilibrium procedures should be viewed cautiously and should be verified using other methods which consider all three equilibrium conditions.

C. *Moment and Force Equilibrium Methods* -- In these methods, efforts are made to satisfy all equilibrium conditions -- overall and individual slice moment equilibrium and vertical and horizontal force equilibriums. Earlier methods which satisfy all three equilibrium conditions include Peterson's (1916) method, Raedschelder's (1948) method, and Fellenius' (1936) rigorous method. These procedures were solved using a graphical procedure and are similar to several procedures developed recently. Use of these methods has been limited due to the time required and complexity involved in obtaining a solution.

Beginning in 1954, there apparently was some renewed interest in slope stability procedures which satisfy all equilibrium conditions. Methods which attempted to satisfy all conditions of equilibrium were published in 1954 by Bishop *(5)* and by Janbu *(6)*. The former procedure is applicable to circular arcs while the latter method is applicable to any general shape of the shear surface. Bishop worked several problems using a rigorous method -- all equilibrium conditions satisfied. He compared the solutions of these numerical examples which included the interslice forces and found that the vertical interslice shear force could be set equal to zero without introducing significant errors -- typically less than one percent. Hence, Bishop's modified procedure which set the vertical interslice force equal to zero gave approximately the same result as Bishop's rigorous procedure which satisfied all equilibrium conditions. Bishop also showed that the ordinary method of slices was inaccurate when applied to ϕ'-c' soils. In the method proposed by Janbu, force equilibrium conditions are completely satisfied; moment equilibrium is only partially satisfied. However, this condition does not significantly affect the accuracy of Janbu's method.

Beginning in 1965, renewed interest occurred again in slope stability methods that satisfy all conditions of equilibrium. Probably such renewed interest may partly be attributed to improved computer technology and availability. Whatever the method of derivation, the solution of the equilibrium equations necessitates the use of successive approximations. Hence, such methods can readily be solved using the electronic computer. In 1965, Morgenstern and Price *(7)* presented a generalized method which satisfies equilibrium conditions. In 1967, Whitman and Bailey *(8)* solved several problems using Morgenstern and Price's procedure and Bishop's modified method and found the resulting difference was seven percent or less -- usually the difference was two percent or less.

In 1967, Spencer *(9)* investigated the factors affecting the accuracy of Bishop's modified version. Spencer showed that Bishop's method, which does not satisfy one of the force equilibrium equations, yields reasonably accurate answers because of the insensitivity of the moment equilibrium equation to the slope of the resultant interslice forces. In 1973, Spencer *(10)* generalized his procedure and introduced a very accurate slope stability method (all equilibrium conditions satisfied). A significant result of Spencer's work was that the resultant interslice forces could be assumed parallel without introducing significant errors. This assumption and finding simplifies the numerical solution of this method.

The geometry and relative locations of different soil types within the earth mass oftentimes dictate the method selected for determining the stability of a slope. In many instances, especially if the project is very important, several methods should be used. For overall suitability and reasonable accuracy and in cases where the geometry of the situation indicates the shear surface may be circular, especially in cases where an embankment is located on a soft and deep foundation and in cut slopes, Bishop's modified procedure *(5)* is recommended. Spencer's procedure *(9)* as published in 1967 may also be used. However, in applying Spencer's method to fairly deep shear surfaces, some difficulty may be encountered in obtaining a reasonable "thrust line" -- a term used by Bishop *(5)* to describe the line passing through the points of action of the interslice forces. A solution of Spencer's method yields the thrust line and the reasonableness of the solution can be judged. Generally, a solution is assumed to be reasonable if the thrust line is located between a distance of about one-third to one-half of the height of the slice above the shear surface and shear stresses on the sides of the slices do not exceed the shear resistance of the soil. What constitutes a reasonable thrust line is, however, questionable since an assumption must be made concerning the stress distribution between slices to compute the interslice stresses. Some uncertainty is associated with the choice of a "proper" stress

distribution between slices, especially in nonhomogeneous soils. The choice of a triangular stress distribution for homogeneous soils may be reasonable; however, such a choice for nonhomogeneous soils may not be reasonable. Bishop (5), in formulating his method, chose to ignore this criterion; he reasoned that, since the slip surface assumed is only an approximation, overstress may be implied in the adjacent soil.

In cases where the potential failure surface may not be circular, that is, the failure surface may be composed of curved and plane segments or entirely of plane segments, Spencer's generalized method (10), as proposed in 1973 and based on the assumption that interslice resultant forces are parallel, and Janbu's generalized procedure of slices (6) are recommended. These methods are particularly applicable to cases where the failure may be of a sliding wedge form or to cases involving long, shallow failures. These methods should also probably be used, in addition to Bishop's method, in checking the stability of an embankment located on both overconsolidated and normally consolidated soils. In these cases, a sliding wedge failure should be assumed. In using Janbu's method, the thrust line must be assumed to obtain a solution. Consequently, several positions of the thrust line should be assumed to determine the effects of the different locations of the thrust line on the computed safety factors. In using Janbu's method, the user must be aware of the fact that a convergent solution may not always be obtained.

The use of Morgenstern-Price's method and Spencer's generalized procedure (the interslice forces are not parallel) is not generally recommended for routine work, but rather these methods should mainly be used to verify solutions obtained from simpler methods. However, in designs involving large sums of money and if the expertise is available, these methods should be used.

Several methods mentioned above have been programmed for the computer. Wright (11) has programmed several of the methods. A computer program developed by Bailey and Christian (12) is based on Bishop's modified procedure, and the program also yields solutions based on a modified form of the ordinary method of slices. This is an excellent computer program. Yoder and Hopkins (13) have also computerized Bishop's modified procedure.

Additionally, Hopkins and Mayes (14) have computerized Janbu's generalized procedure of slices. In using any computer program, the user should understand thoroughly the limitations and capabilities of the slope stability procedure used in the computer program as well as the limitations and capabilities of the computer program. In general, development of a computer program free of errors is oftentimes very difficult. At a minimum, the user should thoroughly understand the slope stability method used in a computer program in order to recognize erroneous results; and it is highly desirable that the user have at least a working knowledge of computer programming. Results obtained from slope stability computer programs should always be reviewed and interpreted by experienced engineers. Even then, errors and oversights may occur.

NOTE 7 -- CID -- Consolidated Isotropically, Drained Triaxial Tests. CIU -- Consolidated Isotropically, Undrained Triaxial Tests.

NOTE 8 -- This procedure is suggested as a possible means of approximating the "normally consolidated" or "softened" shear strength of an overconsolidated clay. This procedure is untested and should be used cautiously.

NOTE 9 -- These design safety factors may lead to conservative designs. Whether or not the stability of the slope can conform to the safety factors shown and remain within the realm of an economical design must be judged by the designer.

NOTE 10 -- In many routine designs where the level of risk may be small, the liquidity index is greater than 0.36, and the soils are homogenous and drain fairly rapidly, a check of the stability of a slope using an effective stress analysis might be considered to be optional. Whether or not such an analysis should be made must be left to the judgment of the designer.

NOTE 11 -- The likelihood that rapid drawdown may occur during construction should be considered since high excess pore pressures may exist in the embankment foundation during that period.

NOTE 12 -- Prediction of some future equilibrium seepage pattern resulting from damming is a difficult problem. Observations in Kentucky at several sites involving side-hill embankments and where groundwater levels were monitored over a period of several months showed that oftentimes the groundwater levels were located in the lower half of the embankment. The geologic setting of the site should be investigated to determine the likelihood of seepage into the embankment.

NOTE 13 -- Economics of different designs based on the possible range of shear strengths which might prevail at the site at some future date should be reviewed. A plot of the cubic yardage of soils required for each slope design as a function of safety factor may be an useful aid in selecting the final slope design. Where the original groundline may vary considerably within a given site, several sections may have to be analyzed before the final slope is selected.

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