Selection of Analytical Methods and Strength Parameters for Slope Stability Investigations In Cohesive Soils

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•IN RECENT years methods of stability analysis for slopes in cohesive soils have reached a relatively high degree of refinement. At the same time, soil testing procedures have been perfected to the point where we are now able to determine accurately the shear strength parameters of cohesive soils under a wide variety of testing conditions. However, there still exists the problem of selecting the method of analysis and the shear strength parameters most applicable to a slope under specific existing field conditions. For example, should the stability analysis of a clay slope be based on the $\varphi = 0$ (total stress) concept or should it be on a c', φ' (effective stress) basis? In addition, should the shear strength parameters used be based on peak shear strengths or residual shear strengths? The answers to these and similar questions are of considerable importance to highway engineers concerned with the stability of clay slopes.

Recent studies of landslides in Europe and North America combined with advances in testing techniques provide new insight into selection of the best methods of analysis and strength parameters for various field conditions. This paper will briefly review case histories which bear on this problem, and will attempt to draw conclusions relative to such selection.

BASIC APPROACHES TO STABILITY ANALYSIS

Slope stability problems in clays are of two general types: (a) short-term stability (end-of-construction case), and (b) long-term stability (steady seepage case). The short-term case applies for a short time after a cut is made in a slope. In excavating for a cut, shear stresses are induced which may cause failure in the undrained state. Theoretically it is possible to analyze the stability of a newly cut slope on the basis of either total or effective stresses; however, since it is difficult to ascertain the distribution of pore pressures under these conditions, the $\varphi = 0$ method of analysis (total stress method) has proved more successful. In this method, a circular failure surface is commonly assumed, and the strength along this surface is taken as the undrained shear strength, c, of the soil. If there is a variation in undrained shear strength with depth, it is necessary to divide the failure block into slices and to utilize the shear strength along the failure surface at the base of each slice.

The long-term (steady seepage) case is encountered in natural slopes and should also be considered in stability of embankments. In this case pore pressures are in equilibrium with steady seepage as stresses are applied; thus, no excess pore pressures are induced. This case is analogous to that of the drained shear test, and effective stress parameters should be used.

Two approaches are used most commonly in analysis of the long-term case. As shown in Figure 1, these are the conventional method of slices (17) and the Bishop method (2). Both of these are used on an effective stress basis for the long-term stability case; i.e., they are used with effective stress parameters. The Bishop method

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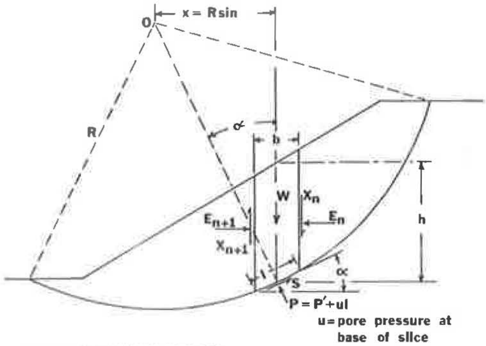

Conventional Method of Slices:

$$F = \frac{\Sigma[c'b \sec\alpha + (W\cos\alpha - ub/\cos\alpha)\tan\phi']}{\Sigma W\sin\alpha}$$

Bishop Method:

$$F = \Sigma\left[\{c'b + (W - bu)\tan\phi'\}\frac{\sec\alpha}{1 + \frac{\tan\alpha\tan\phi'}{F}}\right]\cdot\frac{1}{\Sigma W\sin\alpha}$$

Figure 1. Effective stress analysis by conventional method of slices and Bishop method.

of analysis differs from the conventional method of slices in that it takes into account the forces on the sides of each slice; these are completely ignored in the conventional method of slices. For this reason the Bishop method is theoretically the more accurate of the two. However, the Bishop method is more complicated than the conventional method of slices, and thus is somewhat more costly to use.

ACCURACY OF METHODS OF ANALYSIS AS SHOWN BY CASE HISTORIES

Short-Term Failures of Cuts in Clay Slopes

Factors of safety[1] obtained from analyses of four well-documented short-term failures by the $\varphi = 0$, total stress method are given in Table 1. Many other instances of short-term failures can be found in the literature, but these four are among the best-documented short-term case histories; they also give results which are reasonably representative of the majority of such failures.

For the failure condition, a stability analysis should logically result in a factor of safety of 1.0. Thus, as shown in Table 1, the $\varphi = 0$ method of analysis provides realistic results when applied to short-term failures in normally consolidated to slightly overconsolidated clays which do not contain fissures or similar discontinuities. However, in the case of fissured overconsolidated clays such as those found in the Massena and Bradwell cuts, factors of safety were obtained which were unrealistically high. For the Massena cut this discrepancy was probably due to the presence of fissures in the clay. These fissures lowered the field shear strength but had little effect on laboratory shear strength because the laboratory specimens were too small to contain fissure systems (i.e., the specimens were from solid blocks between the fissures).

The high factor of safety calculated for the Bradwell cut has been ascribed to two factors by Skempton and LaRochelle (16). Of major importance was the loss in strength due to fissuring of the clay. Also of significance was the fact that the rate of loading in the undrained shear test had an effect on the measured shear strength. A time to

TABLE 1

SHORT-TERM FAILURES OF CUTS IN CLAY SLOPES INVESTIGATED BY $\varphi = 0$ ANALYSIS

Locality	Clay Type	Factor of Safety by $\varphi = 0$ Analysis	References
Huntspill Cut, England	Non-fissured, normally consolidated or slightly overconsolidated	0.9	Skempton and Golder (15)
Congress Street Expressway, Chicago	Non-fissured, normally consolidated or slightly overconsolidated	1.11	Ireland (6)
Skattmanso, Sweden	Non-fissured, normally consolidated or slightly overconsolidated	1.1	Cadling and Odenstad (4)
Massena, New York	Fissured, slightly overconsolidated	1.4	Bazett et al (1)
Bradwell, England	Fissured, overconsolidated	1.8	Skempton and LaRochelle (16)

[1]As used here factor of safety is considered to be the sum of the resisting moments divided by the sum of the moments tending to cause failure. For a discussion of the concept of factor of safety as applied to soil and foundation engineering, see Jumikis (7).

TABLE 2

LONG-TERM STABILITY OF NATURAL CLAY SLOPES DETERMINED BY $\varphi = 0$ AND c',φ' ANALYSES
Case I—Stable Slopes in Normally Consolidated Intact Clays

Locality	Factor of Safety by		References
	$\varphi = 0$ Analysis	c',φ' Analysis (Bishop Method)	
Drammen, Norway (Profile C)	0.58	1.26	Kjaernsli and Simons (8)
Bakklandet, Norway (Profile A)	0.75	1.85	Bjerrum and Kjaernsli (3)
Borregaard, Norway	0.85	1.25	Bjerrum and Kjaernsli (3)

failure of approximately 15 minutes was used to obtain the undrained shear strengths used in Skempton and LaRochelle's stability analyses. A test of 15-minute duration is convenient when many such tests are to be conducted, but it obviously corresponds to a rate of strain far greater than that to which the clay is subjected during excavation. For this reason Skempton and LaRochelle conducted a series of undrained triaxial tests with times to failure ranging from 15 minutes to 8 days, and noted an unmistakable tendency for strength to decrease with increasing time to failure. They found the undrained shear strength of London clay to be about 20 percent lower in a 7-day test than in the conventional 15-minute test. This loss of strength with time undoubtedly was part of the reason for the seeming anomaly in the results of the Skempton and LaRochelle analyses. The change in strength with time was found to be a result of migration of pore water toward the shear zone in a specimen due to pore pressure gradients set up upon application of load. This phenomenon was previously reported by Olson (9) who noted that the tendency for increase in moisture content along the failure zone is particularly great in heavily overconsolidated clays.

Long-Term Stability of Stable Natural Clay Slopes in Normally Consolidated Intact Clays

Table 2 compares factors of safety for stable natural slopes (i.e., slopes which had not failed) in normally consolidated clays in Norway as determined by total stress and effective stress methods. The clays were all of the intact variety; i.e., they were free of fissures and similar discontinuities.

Each of these slopes was analyzed to assess the validity of the two approaches to slope stability analysis as applied to stable natural clay slopes. The total stress analyses were based on the $\varphi = 0$ method; the Bishop method was used for the effective stress analyses. The three cases analyzed all involve normally consolidated intact clays. No similar data could be found for stable slopes in overconsolidated clays of either the intact or fissured varieties.

Since a stable natural slope is in a state of steady seepage, it would be expected that strength parameters determined on an effective stress basis would provide more reasonable results than those obtained from undrained shear tests. This expectation is supported by the factors of safety in this table. Since the slopes in question were stable, their true factors of safety should be greater than 1.0. Thus, for all three cases, total stress analyses based on undrained shear strengths resulted in calculated factors of safety which were much too low. In contrast, the factors of safety obtained in the effective stress analyses are reasonable values for stable slopes at least in that they are greater than 1.0.

TABLE 3

LONG-TERM STABILITY OF NATURAL CLAY SLOPES DETERMINED BY $\varphi = 0$ AND c',φ' ANALYSES
Case II—Slides in Normally Consolidated and Overconsolidated Intact Clays

Locality	Type of Clay	Factor of Safety by			References
		$\varphi = 0$ Analysis	c',φ' Analysis		
			Method of Slices	Bishop Method	
Drammen, Norway	Normally consolidated	0.47	0.80	1.01	Kjaernsli and Simons (8)
Lodalen, Norway	Slightly overconsolidated	1.01	0.85	1.05	Sevaldson (11)
Selset, England	Overconsolidated	2.7	—	1.05	Skempton and Brown (14) and Skempton (12)

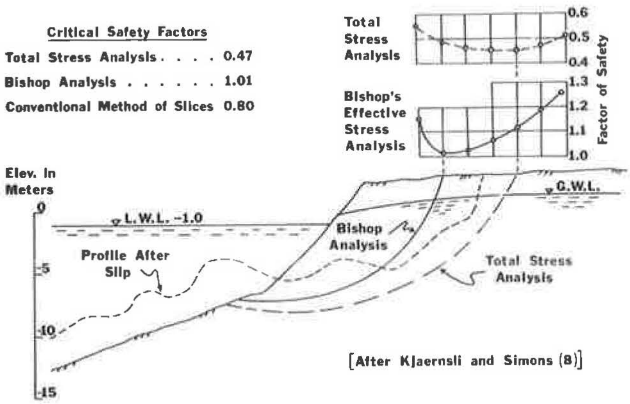

Figure 2. Slope failure on the Drammen River with results of stability analyses using total stress method, Bishop method, and conventional method of slices.

Long-Term Stability of Slides in Natural Clay Slopes in Normally Consolidated and Overconsolidated Intact Clays

Table 3 gives factors of safety calculated for slides which have occurred under long-term conditions in intact clays. The analyses were by means of (a) the $\varphi = 0$ method, total stress approach; (b) the conventional method of slices, effective stress basis; and (c) the Bishop method, effective stress basis.

One of the best-documented studies of slides under long-term stress conditions is that by Kjaernsli and Simons (8) of the Drammen River slide in Norway. Figure 2 shows a cross section of this slide. This profile was located a short distance from the stable profile noted in Table 2. The failure circles shown are critical circles as determined by the $\varphi = 0$ and Bishop methods of analysis. Factors of safety are also plotted for other possible slip circles. As shown in Table 3 and Figure 2, a factor of safety of only 0.47 was obtained for this failure by the $\varphi = 0$ total stress analysis. This is obviously too low to be valid. The conventional method of slices on an effective stress basis resulted in a factor of safety of 0.80, which, while more reasonable than that found in the $\varphi = 0$ analysis, is still too low. The Bishop method, using strength parameters determined on an effective stress basis, provided a reasonable factor of safety of 1.01.

The same general trend is shown in the other analyses in Table 3. In all three cases factors of safety determined by the Bishop method are very close to the true factor of safety of 1.0. The values determined by the conventional method of slices are too low by 15 to 20 percent, and the results of the $\varphi = 0$ analyses are erratic. There is little doubt that of the three approaches the Bishop method provides the most satisfactory results.

Long-Term Stability for Slides in Natural Slopes in Fissured, Overconsolidated Clays

Few well-documented stability analyses of slides in overconsolidated clays containing fissures or similar discontinuities are to be found in the literature. Of these, four of the best are given in Table 4.

As shown in this table, factors of safety obtained by Skempton in use of the $\varphi = 0$ analysis are completely unrealistic for failure conditions in fissured, overconsolidated soils. This is probably due both to the fact that total stress analysis is not well-suited to long-term conditions and to the existence of fissures with resultant lowering of the shear strength.

However, the results obtained by both Skempton and Ringheim using the effective stress approach also are high. Since considerable care was taken in testing these soils, it is probable that the shear strengths obtained in the laboratory were correct for

TABLE 4

LONG-TERM STABILITY OF NATURAL CLAY SLOPES DETERMINED BY $\varphi = 0$ AND c', φ' ANALYSES

Case III—Slides in Fissured, Overconsolidated Clays

Locality	Factor of Safety by		References
	$\varphi = 0$ Analysis	c', φ' Analysis (Bishop Method)	
Jackfield, England	4.0	2.06	Skempton (12)
Sudbury Hill, England	—	2.1	Skempton (12)
Kensal Green, England	3.8	1.6	Skempton (12, 13)
South Saskatchewan River Damsite, Canada	—	Approximately 3-4	Ringheim (10)

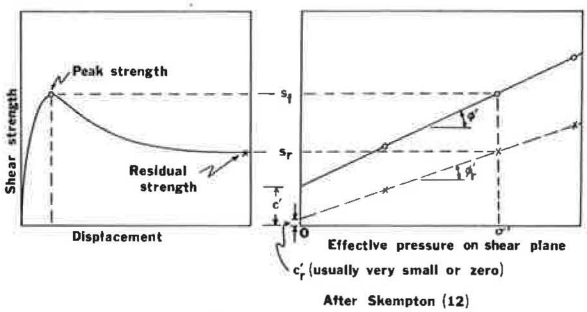

Figure 3. Shear strength characteristics of an overconsolidated clay on effective stress basis.

the conditions of testing, but that these testing conditions did not represent existing field conditions in these fissured clays.

In 1936, Terzaghi (18) first speculated on the reduction of strength of overconsolidated clays due to fissuring. Skempton (12) has since presented a strong case for this concept based on his studies of the failures shown in Table 4 and of other failures in fissured, overconsolidated clays in the United Kingdom. In these studies, Skempton has postulated that the effective shear strength along an individual fissure or other existing failure surface in a clay approximates the residual shear strength obtained from slow, drained direct shear tests in a shear box apparatus in which the clay is subjected to displacements amounting to several inches.

Figure 3 shows the shear characteristics obtained in such a test of an overconsolidated clay. As the clay is strained, it builds up increasing resistance to deformation. Under a given effective pressure, however, there is a definite limit to the resistance the clay can offer; this is the "peak strength", s_f. In an ordinary direct shear test, the test is terminated shortly after the peak strength has been defined, and s_f is referred to simply as the "shear strength" of the clay (under a given effective pressure) on an effective stress basis. If the test is continued to larger deformation, however, it is found that the strength of the clay decreases as the displacement increases. This process, which Skempton refers to as "strain-softening", is not without limit, because ultimately a certain "residual strength", s_r, is reached which the clay maintains even when subjected to large displacements.

If three such drained shear tests are conducted under three different effective pressures, the peak and residual strengths, when plotted against effective pressure as shown in Figure 3, will show a relationship approximately in accordance with the Coulomb-Terzaghi law. Peak strength can therefore be expressed by

$$s_f = c' + \sigma' \tan \varphi'$$

Since test results have almost invariably shown that c'_r is very small, it can be assumed that residual shear strength can be determined from

$$s_r = \sigma' \tan \varphi'_r$$

It should be noted that the difference between peak and residual strengths is small for normally consolidated clays. Thus, the concept of residual shear strength is of real interest only in the case of overconsolidated clays.

The analogy between residual shear strength and the shear strength available on existing failure planes is reasonable. In cases where failure tends to occur entirely along an existing failure surface, as might be the case where material is to be removed from the toe of an old existing slide block, it would thus seem practicable to use residual shear strength parameters in the stability analysis.

In the case of clays containing fissures or similar discontinuities, however, it probably is not reasonable to assume that over 30 to 40 percent of the critical slip surface will actually occur along these discontinuities. In this case, use of residual shear strength parameters would be too conservative an approach because the true strength would lie between the peak and residual values. To aid in understanding this concept, Skempton has devised a "residual factor", R, which he attempts to apply to existing

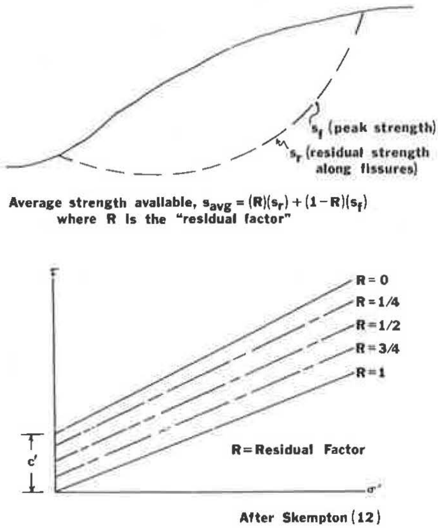

Figure 4. Relationship of residual factor to stability of fissured clay slopes.

fissured, overconsolidated clays to aid in stability analysis (Fig. 4). Completely unfissured clays have R values of zero, and peak strengths are used in stability analyses of such clays. If it is possible to develop a new failure surface entirely along an old surface, R is equal to 1, and residual strengths are used. For cases in between (i. e., fissured clays), R would be based on the proportion of slip surfaces to be developed along existing fissures. The choice of R value is obviously difficult, but ever poor approximations of R will result in stability analyses of fissured, overconsolidated clays which are more reasonable than would be the case if the lowering of available shear strength due to the presence of these discontinuities is not considered.

As noted in the discussion of Table 1, fissuring also has an effect on the short-term available shear strength of overconsolidated clays. Skempton and LaRochelle (16) illustrate how this short-term shear strength of a fissured clay can be approximated from the effective stress parameters. Consider an intact clay with an undrained shear strength, c_u = 2000 psf, and with the following shear strength parameters on an effective stress basis:

$$\text{if R} = 0: \quad c' = 350 \text{ psf} \quad \varphi' = 20°$$

$$\text{if R} = 1: \quad c'_r = 0 \quad \varphi'_r = 15°$$

Let us arbitrarily consider the confining pressure, σ'_n, at which the undrained shear strength as given above for the non-fissured clay is equal to the drained shear strength. This pressure can be calculated from the relationship

$$2000 \text{ psf} = 350 \text{ psf} + \sigma'_n \tan 20°$$

The confining pressure will thus be σ'_n = 4500 psf.

If we further envision this same clay in the ground under an effective pressure of 4500 psf but in a fissured state so that R = 0.2, we can calculate a value of undrained shear strength for the fissured clay as follows:

$$\text{if R} = 0.2: \quad c' = 280 \text{ psf}, \quad \varphi' = 19°,$$

$$c_u = 280 + 4500 \tan 19° = 1800 \text{ psf}$$

Therefore the degree of fissuring indicated here would result in a reduction of undrained shear strength of 200 psf or 10 percent of the strength of the unfissured clay. A reduction of strength of this magnitude due to fissuring could easily result in the discrepancies in factors of safety noted for the short-term stability cases for fissured clays in Table 1.

SUMMARY

The case histories discussed here provide considerable insight into the efficacy of total and effective stress methods of slope stability analysis as applied to clay slopes under varying field conditions. In addition, they give an indication as to which shear strength parameters should be used with these methods.

For short-term stability analysis of non-fissured clays, the $\varphi = 0$ total stress method is satisfactory. For short-term stability of fissured, overconsolidated clays, the $\varphi = 0$ analysis is not safe unless account is taken of reduced strength due to the presence of fissures. Also, it must be remembered that for overconsolidated clays the rate at which the undrained shear test is conducted has considerable effect on the laboratory undrained shear strength, and that undrained shear strengths obtained in the common 15-minute test may be considerably higher than those in existence in the slope at time of failure.

For long-term stability of slopes in non-fissured clays, analysis on an effective stress basis is preferable to use of the $\varphi = 0$ total stress analysis. Assuming use of the effective stress approach, the Bishop method results in what appear to be better analyses than the conventional method of slices, the method of slices producing factors of safety for the failure state which are 15 to 20 percent too conservative.

Effective stress methods of analysis should also be used for long-term stability of slopes in overconsolidated clays containing fissures or other discontinuities. However, because of the reduced strength of the clay due to fissuring, it is necessary to utilize shear strength parameters which represent a compromise between peak shear strength and residual shear strength as determined in laboratory shear tests. More studies of the effects of fissure systems on the stability of slopes in overconsolidated clays are needed before we can accurately determine the shear strengths applicable to meaningful analyses of such slopes.

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