TIMOTHY D. STARK

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Soil Strengths from Back Analysis of Slope Failures

J. Michael Duncan[1], F.ASCE and Timothy D. Stark[2], A.M. ASCE

Abstract

The advantages and limitations of using back analysis to evaluate soil strengths are discussed. It is shown that even under the simplest conditions it is not possible to determine unique values of the shear strength parameters c' and ϕ' by back analysis. Analysis of 24 landslides in the Orinda formation illustrates the use of back analysis to determine regional strength parameters, and provides a basis for evaluating their accuracy.

Introduction

When slope failures of the types shown in Fig. 1 occur, the shear strength of the soil is mobilized along the full length of the slip surface. An estimate of this strength can be made by performing what has come to be called a "back analysis." Starting from the result -- that the factor of safety is equal to one -- analyses are performed to determine what the strength of the soil must have been for the failure to have occurred.

Determining soil strengths by back analysis avoids many of the problems associated with laboratory testing, and is widely used, especially in connection with landslide repair studies. This technique is an effective method of accounting for important factors that may not be well represented in laboratory tests, such as the structural fabric of the soil, the influence of fissures on the strength of the soil, and the effects of pre-existing shear planes within the soil mass.

However, it is not possible to determine shear strength values uniquely through back analysis. At best, one slope failure provides a single piece of information, which is insufficient to determine the values of two strength parameters (c' and ϕ'). In principle it is possible to determine values of both c' and ϕ' if the depth of the slip surface is known. In practice, however, as a result of progressive failure and the fact that the position of the slip surface may be controlled by strong or weak layers within the slope, the values of c' and ϕ' cannot be inferred from the position of the slip surface. This is illustrated by back analysis of the Northolt slide in London clay.

[1] University Distinguished Professor, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA, 24060
[2] Assistant Professor, Department of Civil Engineering, University of Illinois, Urbana, IL, 61801

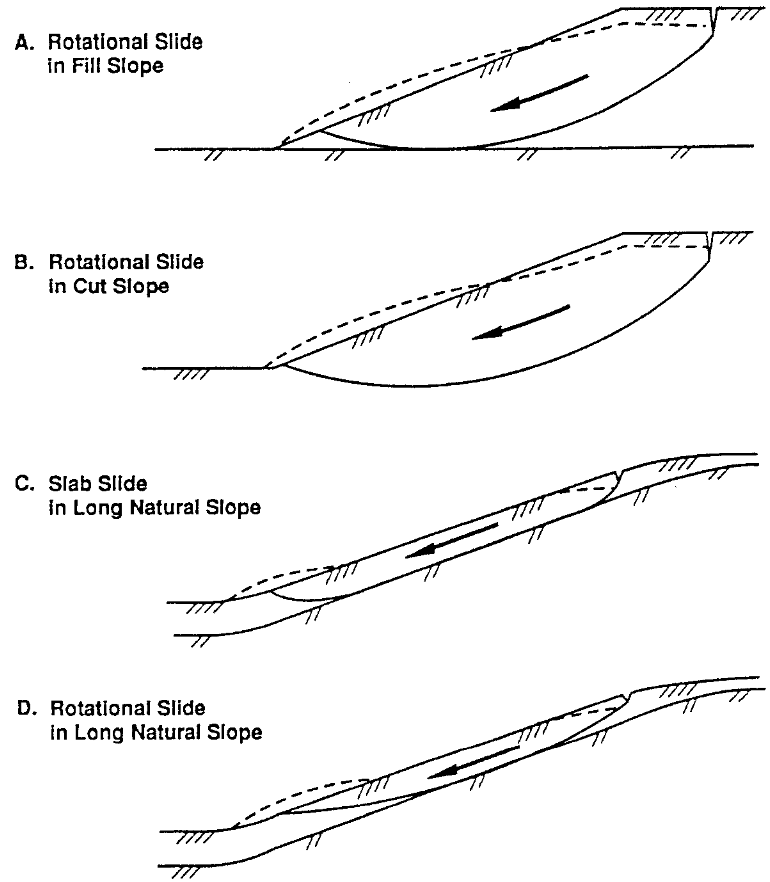


Fig. 1 - Mechanisms of Slope Failures in Soil

In most cases the information regarding the conditions under which a slide occurred is incomplete to some degree, and this lack of complete information reduces the reliability of the back calculated strengths. If the lack of information concerning the conditions at the time of failure is not extensive, and if the necessary assumptions are supported by local experience and good judgment, it is possible to derive values of c' and ϕ' that are of value in analyzing the stability of slopes in the same geologic formation. Where many slides in the same formation can be back analyzed, a measure of the reliability of the back calculated strength parameter values can be developed. This is illustrated by back analysis of slides in the Orinda formation, in Contra Costa County, California.

Advantages and Limitations of Back Analysis

Back analysis is an attractive procedure for determining soil strengths because it avoids many of the problems associated with laboratory and in situ tests. The principle advantages of back analysis are:

(1) It avoids problems with disturbance, because back calculated strengths are representative of the soil in its natural state.

(2) It provides strength values that are representative of a failure plane having an area many orders of magnitude larger than the failure plane in any laboratory or in situ test.

(3) It gives a measure of the shear strength of the soil mass that reflects the influences of soil fabric, fissures, and pre-existing shear planes.

(4) In most cases it involves a much longer time to failure than laboratory or in situ tests.

(5) Studies have shown that, where conditions are simple and accurately known, shear strengths determined by back analysis are in good agreement with values determined through very extensive laboratory test programs (Chandler, 1970; Chandler, 1977; Early and Skempton, 1972).

The principle limitation of back analysis is the fact that, unlike laboratory tests, the stress conditions at the time of failure are not precisely known, and cannot be varied to determine how shear strength varies with changes in effective stress. A slope failure provides a single piece of data, and assumptions are inevitably required to determine values of c' and ϕ' by back analysis.

Although back analysis has been found to be effective where conditions are simple, studies by Hutchinson (1969), Hutchinson, et al. (1980), Leroueil and Tavenas (1981), and Hencher, et al. (1984) have shown that the results of back analysis are tenuous when it is applied to complex or poorly defined conditions.

Saito (1980) pointed out that reliable values of c' and ϕ' can only be evaluated through back analysis if the pore pressures used in the analyses reflect accurately the conditions at the time of failure. Leroueil and Tavenas (1981) showed for one case that assuming the phreatic surface in a 25 foot high slope was three feet higher than it actually had been at failure would result in a 50 percent increase in the value of c' calculated by back analysis.

Cooper (1984) performed sensitivity studies to examine the consequences of possible errors in the values of various parameters involved in back analyses. Analyses were performed using ranges of assumed values of c', ϕ', and pore pressure. Calculations were done to determine the effects of errors in the assumed values of these quantities on the calculated factor of safety after stabilization of the slope. In the case where a slope is stabilized by flattening, errors due to incorrectly assumed values of c', ϕ', or pore pressure tend to be self-compensating, and have relatively small effects on the calculated values of factor of safety after stabilization. In the case where a slope is stabilized by drainage, errors due to incorrectly assumed values of c' or ϕ' also tend to be self-compensating, but errors due to incorrectly assumed pore pressures are not self-compensating, and can be quite significant. Assuming that the pore pressures were

higher than they actually were at failure is unconservative, because it results in back calculated strengths that are too high.

Nonuniqueness of Back Analyzed Strength Parameters

As noted previously, one slope failure provides only one piece of information -- that the factor of safety was equal to unity for the conditions prevailing at failure. In effect, this is the same as having only one equation to work with -- only one unknown can be evaluated. To evaluate c' and ϕ' by back analysis, one of these parameters must be assumed before the other can be calculated.

Theoretical relationship between c' and ϕ' and the position of the slip surface. Saito (1980) and Li and Zhao (1984) have suggested that the magnitudes of both c' and ϕ' can be determined by considering the position of the actual slip surface together with the fact that the factor of safety should be equal to unity.

Saito (1980) reasoned that, while the factor of safety for the actual slip surface should be equal to one at failure, the factors of safety for slip surfaces slightly inside and slightly outside the actual slip surface should be greater than one. He suggested that unique values of c and ϕ (total stress strength parameters) could be back calculated using trial and error to find values that satisfy this condition. He also suggested that the same procedure could be used to determine unique values of c' and ϕ' by back analysis, provided that the pore pressures at the time of failure were known.

Li and Zhao (1984) described a method of estimating the values of c' and ϕ' that involved using an estimated value of the average effective normal stress on the actual slip surface, together with the condition of moment equilibrium. The actual slip surface was approximated as a circular arc. They imposed the same requirement as Saito -- that the factor of safety calculated for the circle representing the actual slip surface should be equal to unity, and the factors of safety for circles with slightly shorter and slightly longer radii should be larger.

If the position of the slip surface is controlled by the positions of strong or weak layers within the slope, the principle used by Saito and by Li and Zhao does not apply. For example, consider a case where a layer of weak soil overlies a layer of firm soil or rock, and the actual slip surface is tangent to the top of the firm layer. If ϕ' was assumed to be zero (not a reasonable assumption), a value of c' could be found by back analysis that would make the factor of safety for the actual slip surface equal to unity. The factor of safety for a slip surface slightly inside the actual one would be higher, because, for completely cohesive strength, the shallower the slip surface, the higher the factor of safety. The factor of safety for a slip surface slightly deeper than the actual one would also be higher than unity, because it would cut into the firm layer.

If ϕ' was assumed to be some small value greater than zero, the results of the back analysis would be the same as described in the preceding paragraph. Thus for any set of conditions involving a slide in a weak layer overlying a strong layer, there would be a range of values of c' and ϕ', some reasonable and some unreasonable, which would satisfy the condition that the factor of safety of the actual slip surface should be lower than the factors of safety for neighboring slip surfaces. It is thus clear that the procedures suggested by Saito (1980) and by Li and Zhao (1984) cannot be used to determine unique values of c' and ϕ' if the position of the slip surface is controlled by strong and weak layers within a slope.

Reanalysis of the Northolt slide in London clay. While it is clear that the procedures suggested by Saito (1980) and by Li and Zhao (1984) cannot be applied where there are layers of differing strengths within a slope, it remains to be seen if these methods can be used to determine values of c' and ϕ' uniquely in cases where the slope contains only one type of soil. To explore this possibility, the writers have reanalyzed the slide at Northolt, in London clay, which was described by Skempton (1964).

The Northolt slide was considered a good case for application of the procedures suggested by Saito (1980) and by Li and Zhao (1984) because the values of c' and ϕ' for the slide had been determined by extensive laboratory tests and previous back analysis, and because the position of the phreatic surface and portions of the actual slip surface were known.

A cross- section through the Northolt slide is shown in Fig. 2. The slide was excavated in 1903, was steepened in 1936, and failed in 1955. Based on his 1964 back analyses of the slide, Skempton concluded that the strength of the clay within the slope was represented by c'= 140 psf and ϕ'= 18 degrees. These values fit within the range of c' and ϕ' values for London clay, which vary between the following limits:

c'= 320 psf, ϕ'= 20 degrees -- peak strengths

c'= 0, ϕ'= 16 degrees -- residual strengths.

Back analyses to determine values of c' and ϕ' were performed following the principles outlined by Saito (1980), and by Li and Zhao (1984), using both circular and noncircular slip surfaces.

The results of analyses using circular slip surfaces are shown in Fig. 2A. Slip circle B was used by Skempton as a reasonable representation of the actual slip surface, portions of which are shown in the figure. Slip circles A and C were chosen just inside and just outside circle B. The analyses were performed using Bishop's Modified method of analysis (Bishop, 1954). By repeated trials, it was found that circle B had a calculated factor of safety, F = 1.00 for c'= 20 psf, ϕ'= 24 degrees, while circles A and C both had F = 1.02 for the same values of c' and ϕ'.

Thus the values c'= 20 psf and ϕ'= 24 degrees satisfy the criteria suggested by Saito (1980) and by Li and Zhao (1984) for determining unique values of c' and ϕ' that are consistent with the position of the actual slip surface. However, these values are not reasonable as compared with the extensive studies of the shear strength of London clay done by Skempton (1964, 1977). As compared with the results of laboratory tests, ϕ' = 24 degrees is an unreasonably high value for London clay. As indicated above, tests have shown that the reasonable range for ϕ' for London clay is 16 degrees to 20 degrees.

The results of analyses using noncircular slip surfaces are shown in Fig. 2B. Slip surface E was drawn through the observed slip surface locations, and slip surfaces D and F were drawn just inside and just outside surface E. The analyses were performed using Spencer's method (Spencer, 1967). Using repeated trials, it was found that for c' = 15 psf and ϕ' = 25 degrees, F = 1.00 for slip surface E, while F = 1.03 and 1.02 for slip surfaces D and F.

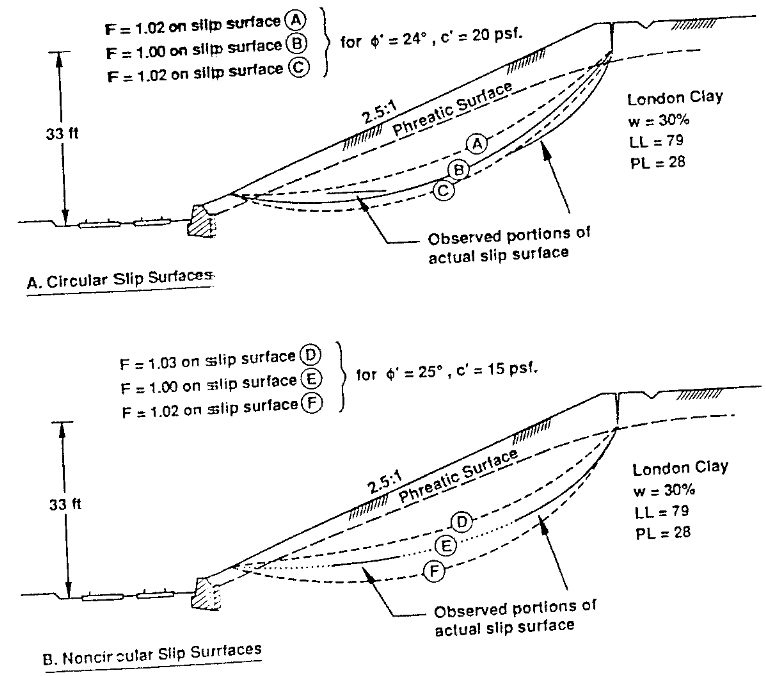


Fig. 2 - Back Analysis to Match Observed Slip Surface for Northolt Slip (Cross-section from Skempton, 1964)

Thus the results achieved using noncircular slip surfaces are essentially the same as those from analyses with circular slip surfaces: the value of ϕ' found using the criterion that the actual slip surface should have a lower factor of safety than neighboring slip surfaces is unreasonably high as compared with laboratory tests.

The writers believe that the reason for this discrepancy is that the Northolt slope failed progressively (Skempton, 1964). Bjerrum (1967) and Skempton (1964, 1977, 1985) have shown that it is common for failure surfaces in overconsolidated clays to develop progressively. Bjerrum's studies suggested that failure surfaces around slopes in overconsolidated clays propagate inward from the toe of the slope gradually over a period of time before failure occurs. As a result, the conditions that prevail when the slip surface is developing may differ from the conditions when the slide occurs. Skempton (1977) indicated that tens of years may be required for pore pressures to equalize around an excavated slope in clay. Thus the slip surface at Northolt probably began to develop while the pore pressures within the slope were lower than they were when the slip occurred. A change in pore pressures after part or all of the slip surface developed appears to explain why the actual slip surface is not consistent with all of the conditions at the time of failure.

From these considerations it seems clear that there will be few, if any, circumstances where the position of the slip surface can be used to determine unique values of the strength parameters c' and ϕ'. If the position of the slip surface is controlled by strong and weak layers within the slope, or if progressive failure has occurred, values of c' and ϕ' calculated assuming that the actual slip surface should be more critical than any other may be unreasonable. Therefore it is preferable to assume the value of ϕ', using good judgment and whatever experience can be brought to bear, and to calculate the value of c' that corresponds to F = 1.00.

Table 1 provides typical values of ϕ' for the fully softened condition and for the residual strength condition. Following the recommendations of Skempton (1964, 1977, 1985), fully softened shear strengths are appropriate where no sliding has occurred previously, and residual strengths should be used where there has been sufficient shearing deformation to result in reorientation of clay particles parallel to the direction of shearing. Field shearing displacements of three feet or more are sufficient to reduce the strengths of clays to their residual values.

Table 1. Typical Values of ϕ' for the Fully Softened and the Residual Strength Condition, after Ladd et al. (1977) and Mitchell (1976)

Plasticity Index	Value of ϕ' (degrees)	
	Fully Softened	Residual
0 - 10	30 - 40	18 - 30
10 - 20	25 - 35	12 - 25
20 - 40	20 - 30	10 - 20
40 - 80	15 - 25	7 - 15

Regional Study of Landslides in the Orinda Formation

Another procedure for determining values of c' and ϕ' is through back analysis of several slides in the same material (Lane, 1961). The Orinda formation, in Contra Costa County, California, is highly prone to landsliding, and affords a good example of an area where this technique can be applied effectively. In the early 1960s. Radbruch and Weiler (1963) studied the landslides in an 8.5 square mile area within the Orinda formation. The area they studied is indicated on the map in Fig. 3. They found that 195 landslides occurred in this area in a period of two years.

Characteristics of the Orinda formation. The Orinda formation, of Pliocene age, includes conglomerate, sandstone, siltstone, and claystone beds. The predominant structure within the area studied by Radbruch and Weiler is a group of northwest-southeast trending parallel folds, which are offset by several faults. Valleys have formed parallel to the axes of the folds, the largest being the one that contains San Pablo Reservoir. The relief in the area is as much as 600 feet. Anti-dip and dip slopes are about equally numerous, and landslides occur on slopes of both attitudes.

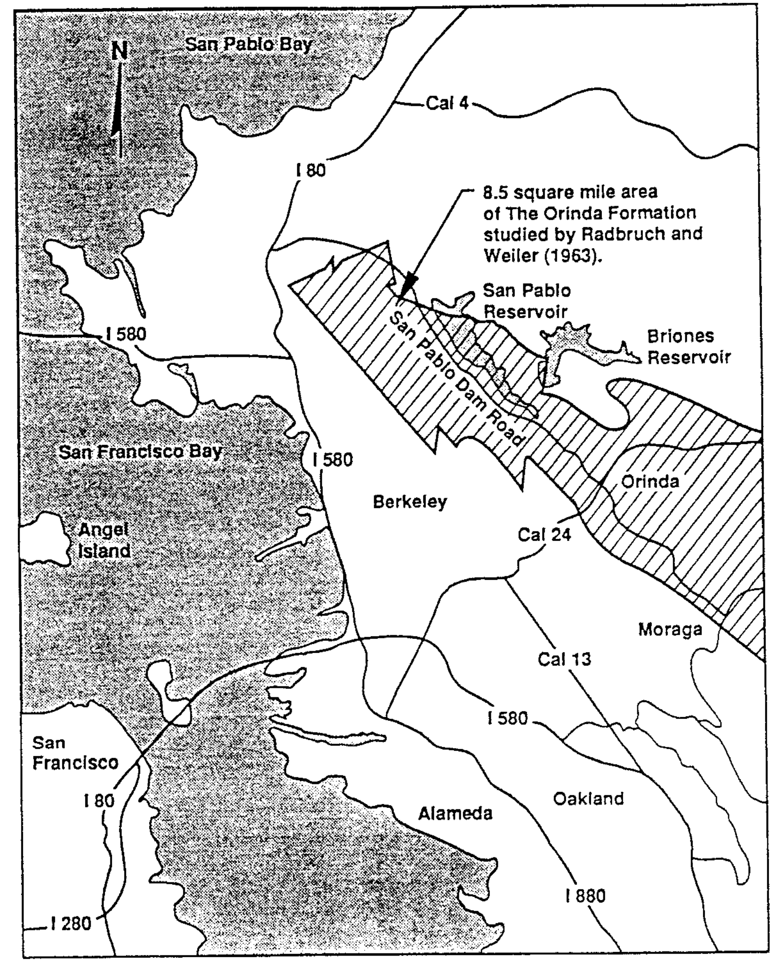


Fig. 3 - Map of the San Francisco Bay Area, Showing the Part of the Orinda Formation Studied by Radbruch and Weiler (1963)

Soil cover is sparse within the Orinda formation. In most locations the weathered rocks of the Orinda are exposed on the hillsides. Weathering extends to a depth of three feet to 20 feet. The weathered materials of the Orinda have very low shear strengths. Although they are rocks from a geologic point of view, they behave like soils, and the landslides that occur within them are of the types that take place on slopes in soil. The weathered rock materials that are exposed on the hillsides are almost always clayey, and they usually classify as clays of low to medium plasticity (CL) by the Unified Soil Classification System. Typical values of the Atterberg limits are LL = 30 to 50, and PL = 15 to 25.

Characteristics of the landslides in the Orinda formation. The materials of the formation are highly prone to landsliding, as indicated by the fact that 195 landslides occurred in an 8.5 square mile area of the Orinda in a two-year period. Many landslides occur on natural slopes inclined at 20 degrees or more, and rarely on slopes flatter than 20 degrees. Slopes of excavations that are deep enough so that they remove the more weathered, weaker material often remain stable at inclinations steeper than 20 degrees.

Most of the slides that occur in either cut slopes or fill slopes in the Orinda formation are rotational slides, of the type shown in Fig. 1A and 1B. Most of the landslides that occur on natural slopes are slab slides or rotational slides on long slopes, as shown in Fig. 1C and 1D. Both the heads and the toes of these slides lie within the slope, and the length of the slope is greater than the length of the slide. In many cases it is not possible to determine by inspection in the field whether a particular slide is a slab slide or a rotational slide, because their surface expressions are virtually the same.

Of the 195 slides that were recorded by Radbruch and Weiler, a total of 39 were suitable for back analysis. These include 24 slides on natural slopes, 10 slides on cut slopes, and 5 slides on slopes in fills that were constructed using fill materials derived from the Orinda formation. The characteristics of the slides are summarized in Table 2.

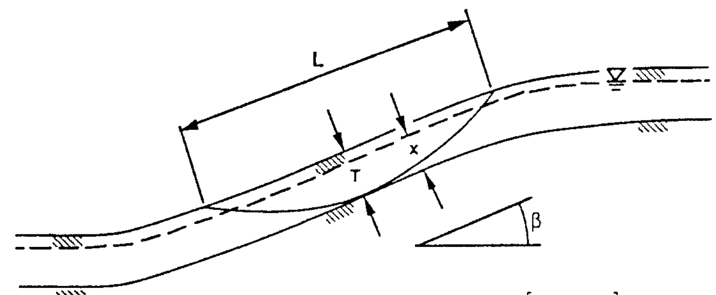
Table 2. Characteristics of Back Analyzed Slopes in the Orinda Formation

Type of Slope	Slide Length	Slope Angle
Natural (24)	Max = 600 ft	Max = 35°
	Min = 20 ft	Min = 16°
	Ave = 75 ft	Ave = 23.5°
Cut (10)	Max = 225 ft	Max = 41°
	Min = 20 ft	Min = 22°
	Ave = 70 ft	Ave = 29°
Fill (5)	Max = 200 ft	Max = 35°
	Min = 20 ft	Min = 23°
	Ave = 60 ft	Ave = 31°

The remaining slides were considered unsuitable because they involved more than one type of material (natural, cut, or fill), because they had complex geometries (variations in slope angle within the slide area), or because they were described by Radbruch and Weiler as earthflows or rockfalls rather than slumps (which are termed slab slides or rotational slides in this paper).

Analysis procedures for natural slopes. For purposes of analysis the slides on natural slopes were assumed to be rotational slides of the type shown in Fig. 1D. This mechanism is considered to be more realistic than the infinite slope mechanism, which neglects end effects at the head and the toe of the slide. Consideration of the end effects results in values of c' that are somewhat smaller, for the same value of ϕ', than would have been calculated if the infinite slope mechanism had been used in the back analyses. The analyses were done using the procedure shown in Fig. 4. The equations shown in Fig. 4 were developed by analyzing a number of rotational slides on long slopes using Bishop's Modified Method (Bishop, 1954), and relating the results to the expressions for factor of safety from infinite slope analyses. It was found that the factors of safety for rotational slides and for the infinite slope condition could be related through the

adjustment factors C_1 and C_2 shown in Fig. 4. Both C_1 and C_2 are equal to 1.00 for infinite slope conditions, and they have the values shown in the figure for rotational slides on long slopes. The expressions in Fig. 4 made it easy to calculate the factor of safety for a given slope, or to calculate the value of c' for F = 1.00.



$$A = \left[1 - \frac{X}{T}\frac{\gamma_w}{\gamma_m}\right]$$

$$F = C_1 A \frac{\tan \phi'}{\tan \beta} + C_2 B \frac{c'}{\gamma_m H} \qquad B = \frac{1}{\cos^2\beta \, \tan \beta}$$

$$C_1 = \left[1 + 36 \sin \beta \left(\frac{T}{L}\right)^2\right]$$

$$c' = \frac{\gamma_m H}{C_2 B}\left[F - C_1 A \frac{\tan \phi'}{\tan \beta}\right] \qquad C_2 = \left[1.5 + 10\left(\frac{T}{L}\right)^2\right]$$

$$H = \frac{T}{\cos \beta}$$

F = factor of safety
ϕ' = effective stress angle of internal friction
c' = effective stress cohesion intercept
γ_m = moist (total) unit weight of soil
γ_w = unit weight of water
T = thickness of layer containing slide (measured normal to ground surface)
X = distance from phreatic surface to base of layer (normal to ground surface)
H = vertical distance from top of layer to base
β = slope angle

Fig. 4 - Analyses of Rotational Slides on Long Slopes

The value of c' for F = 1.00 was calculated for each of the slopes for assumed values of ϕ' ranging from 20° to 40°. This resulted in 24 values of c' for each assumed value of ϕ'. In these calculations, the position of the phreatic surface was assumed to correspond to X/T = 0.8. Landslides in the Orinda formation occur during periods of heavy rainfall, and the phreatic surfaces within the slopes are usually at or near the ground surface at the time of failure. Assuming X/T = 0.8 results in lower values of back calculated strength than would be calculated if it was assumed that X/T =

1.00. This can be seen in Fig. 4 where c' is expressed as a function of A, and A is expressed as a function of X/T. Reducing the value of X/T results in a higher value of A and thus a lower value of c'. The value of γ_m was assumed to be 135 pounds per cubic foot, based on data from several sites.

Results of back analyses. The variation of the back calculated values of c' with the assumed values of ϕ' for natural slopes is shown in Fig. 5. As the assumed value of ϕ' increases, the back calculated value of c' decreases. The range between the maximum and the minimum values of c' is about 150 psf, a surprisingly narrow range considering the variations in slope heights and slope angles among the 24 slopes.

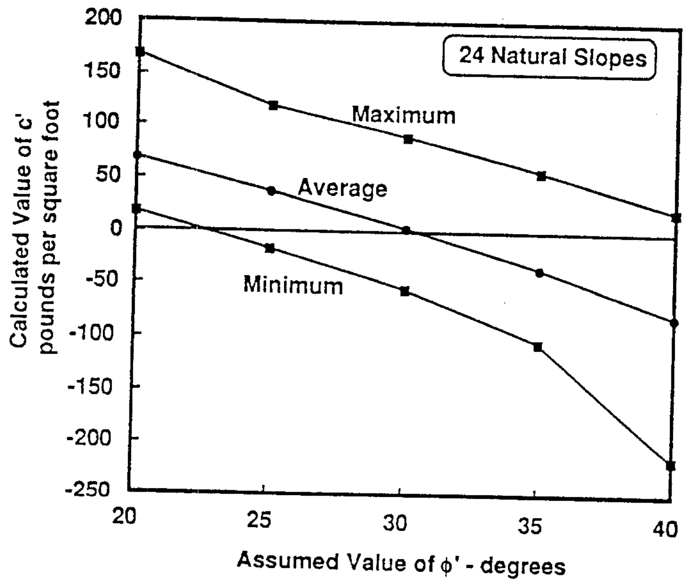


Fig. 5 - Varitation of Calculated Value of c' with Assumed Value of ϕ' for Failures on 24 Natural Slopes in the Orinda Formation

Given the information shown in Fig. 5, two important questions remain: (1) What are the best values of c' and ϕ' for analysis of slopes in the Orinda formation, and (2) What minimum factor of safety should be used in conjunction with these values of c' and ϕ'?

To answer these questions, the factor of safety of each of the slopes was calculated using each assumed value of ϕ' and the corresponding average value of c'. The results are shown in Fig. 6. The values of maximum safety factor shown in Fig. 6 provide a measure of the potential inaccuracy involved in using the average value of c' in stability analyses. Because all of the slopes failed, the correct result in each case is F = 1.00. Where values of F > 1.00 are calculated, the results are unconservative, and the margin between the calculated value of F and 1.00 is a measure of the unconservative error.

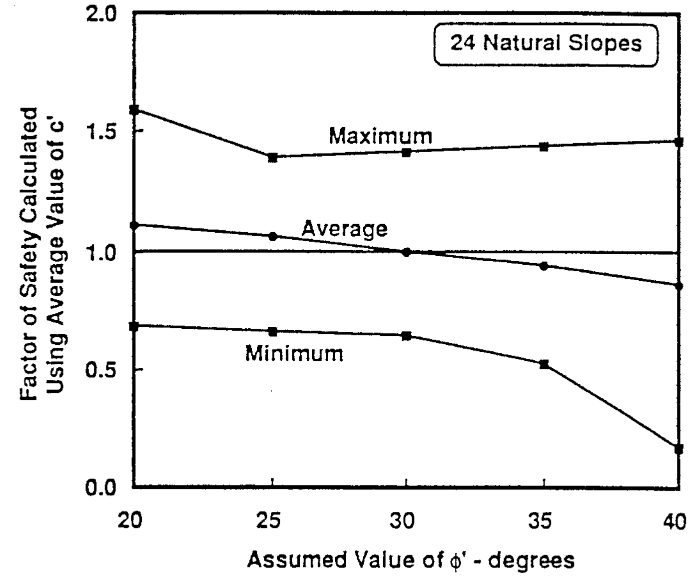


Fig. 6 - Varitation of Factor of Safety Calculated Using the Average Back Calculated Value of c' with the Corresponding Assumed Value of ϕ'

A reasonable design procedure would be to use the average value of c' with the corresponding value of ϕ', and to ensure that the calculated factor of safety is larger than the maximum value shown in Fig. 6. This should result in a safe design, provided the actual conditions are not worse than those assumed in the analyses. Thus the maximum factor of safety shown in the figure provides guidance with regard to the value of F that should be used to achieve a safe and reliable result.

It can be seen that, for $\phi' = 25°$ and c' = 40 psf, the maximum calculated factor of safety is 1.4. For values of ϕ' either smaller or larger than 25°, the maximum calculated factor of safety is larger. These results thus indicate: (1) The values c' = 40 psf and $\phi' = 25°$ appear to be optimum choices. (Note that the maximum factor of safety calculated using c' = 0 and $\phi' = 30°$ is only very slightly higher than 1.4; therefore c' = 0 with $\phi' = 30°$ might be considered to be an equally good alternative.) (2) If these values of c' and ϕ' were to be used for analysis of the stability of slopes in the Orinda formation, the minimum factor of safety should be somewhat larger than 1.4. A minimum value of F = 1.5 seems appropriate.

Analysis Procedures for Cut and Fill Slopes. These same types of analyses were also done for the 10 cut slope failures and the 5 fill slope failures, with the results shown in Table 3.

Table 3. Optimum Values of c' and ϕ' Based on Back Analysis of Slope Failures in the Orinda Formation.

Type of Slope	c'	ϕ'	Ave F	Max F
Natural Slopes (24)	40 psf	25°	1.06	1.39
Cut Slopes (10)	10 psf	35°	1.01	1.30
Fill Slopes (5)	40 psf	30°	1.02	1.14

For convenience the same failure mechanism was used for analysis of the cut and fill slopes as was used for the natural slopes. Although this mechanism is not as representative of the actual failure mechanisms in cut and fill slopes, the results are useful because they provide some interesting information about the reliability of back calculated strength parameters.

The most significant aspect of the results shown in Table 3 is that the values of maximum factor of safety for cut and fill slopes are smaller than those for natural slopes. It might be inferred, therefore, that it would be appropriate to use smaller factors of safety for cut and fill slopes, on the basis that the back calculated values of c' and ϕ' are more reliable. However, this is clearly not the case. The principle factor influencing the value of the maximum factor of safety is the number of slopes analyzed. If only two slopes were analyzed, using the same procedures, the results would seem perfect -- the calculated factor of safety would be exactly 1.00 for both slopes. Thus, to arrive at some measure of the uncertainty in this type of regional back analysis study, it is necessary to analyze a significant number of slope failures. Clearly five is too small a number. Ten also appears to be a little too small. The writers believe that the minimum number of back analyzed failures that will provide a reasonable indication of the reliability of the result is about 15 to 20.

Conclusions

Back analysis is a useful procedure for determining soil strengths from slope failures. Its advantages include the fact that it avoids problems with soil disturbance, and that it gives a measure of the shear strength of the soil mass that is representative of a large area, and which reflects the influences of soil fabric, fissures, pre-existing shear planes, and long-term loading. Where soil conditions are not complex, and where the conditions at the time of failure are known with some degree of accuracy, shear strengths determined by back analysis have been found to be in good agreement with values determined through extensive programs of laboratory testing

It has been suggested that the the magnitudes of both c' and ϕ' can be determined by considering the position of the actual slip surface within a slope. However, if the position of the slip surface is controlled by strong and weak layers within the slope, or if progressive failure has occurred, this is not true. The best procedure appears to be to assume the value of ϕ', using good judgment and whatever experience can be brought to bear, and to calculate the value of c' that corresponds to F = 1.00. Table 1 provides some guidance for estimating values of ϕ'. Attempting to find values of c' and ϕ' that make the actual slip surface more critical than any other does not improve the results, and may in fact result in values of c' and ϕ' that are less representative of the actual strength characteristics of the soil.

Although back analysis is a useful procedure for estimating soil strengths, it cannot provide unique values of c' and ϕ' for the soil involved in a slide, and it cannot be used without assumptions. A thorough understanding of the slope failures analyzed, and the application of good judgment in formulating the necessary assumptions, are prerequisites for achieving reasonable and reliable results.

Acknowledgments

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