Failure of Slopes in Weathered Overconsolidated Clay

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Small slides in backslopes in overconsolidated clay are only locally reported and documented, but they constitute a large maintenance expense item in transportation department budgeting. While these failures are usually attributed to drastic changes in water content caused by saturation, there are additional factors that are conducive to the loss of stability but are not considered adequately. These factors are related to reduction in strength and weatherability. The design of slopes uses shear-strength parameters, the values of which are conveniently obtained from triaxial compressive strength tests. However, these values reflect ultimate strength conditions whereas slope failures represent a state of reduced strength. The adjustment from ultimate to reduced strength can be effected by using the Webb technique. When this is done, a drop in the cohesion and interparticle friction values results and, in turn, a substantial reduction in the factors of safety governing slope stability. The weathering of material can be approximated in the laboratory by ultrasonic degradation tests. The data from these tests indicate that the clay content and the plasticity index of the soils in the slopes are higher than those indicated by the conventional standard tests. The slopes, acted on by water, develop higher pore water pressures and less resistance to the forces initiating sliding than predicted. Consequently, slope stability is further reduced and reaches failure or near-failure conditions.

Massive slides of slopes have been well documented in the geotechnical literature. On the other hand, small slides that occur quite frequently are only locally reported, yet they constitute a large maintenance expense item. Seldom are these slope failures subjected to a thorough and intensive investigation. Thus, this study concentrates mainly on small side-slope failures.

Within the topic of slope stability, overconsolidated clay slopes present interestingly unique features that have been described by Bjerrum (1) and Skempton (2) and recently in a comprehensive manner by Fragaszy and Cheney (3). The profound and critical problem of slope stability centers around the prediction of the "shear strength mobilized during undrained failure", since the design of slopes is based on static equilibrium and uses primarily the shear-strength parameters of cohesion and interparticle friction determined in the laboratory. Consequently, the state at which these parameters are evaluated is of paramount importance.

The studies by Bjerrum (1), Skempton (2), Fragaszy and Cheney (3), Gould (4), and Noble (5) emphasized the physical significance of the residual strength concept for overconsolidated material, such as shales. This study, however, is related to the failures that took place within the weathered material derived from Oklahoma shales. The unweathered material was not involved in the failure and therefore is not treated in this study. Whether the weathered material can be considered as definitely overconsolidated may be debatable. However, it is likely that it carries some structural features of its overconsolidation history.

To a great extent, soil structure owes its form and its permanence to the cementation bonds among the soil particles. Studies on the weatherability of shales by Laguros (6) revealed that, when the shales were excavated and put into use, their performance was substandard compared with that predicted through the conventional standard tests for soils. Further investigations (6,7) led to the application of the ultrasonic degradation test. Treatment of shales ultrasonically results in increases in the clay-size soil material and the plasticity index. These increases were attributed to the breakdown of the cementation bonds in the shale and are indicative of the propensity of the material to further weathering. This occurs rather slowly in nature. In the laboratory, however, it can be brought about in approximately 2 hours. Thus, the ultrasonic degradation test constitutes an accelerated simulation of the environmental and other influences in nature. It further provides a tool, at least qualitatively, for predicting the breakdown of the soil structure and the attendant lowering of the factor of safety for slope stability. In addition, the weathered material has a higher initial permeability than its parent shale material; thus, it becomes water saturated with greater ease.

In actual design, the stability of slopes is based on cohesion (c) and interparticle friction angle (ϕ), values obtained from undrained triaxial tests, and a predetermined minimum value of factor of safety. The c and ϕ values are chosen on the basis of an ultimate strength. Failures took place in the slopes that were designed in this manner.

Failures suggest that a reduction in shear strength has taken place. The Webb technique (8) is a method in which the peak strength values obtained from consolidated-drained triaxial tests are modified to give the "reduced" shear-strength values. Under the conditions of this study, c and ϕ values from consolidated-undrained triaxial tests were available, and it was felt that there might be an analogy between these failures and those in overconsolidated clays. Therefore, in an effort to find an explanation for the failures--aside from the fact that there was an augmentation in moisture content--the Webb technique was used.

This paper presents data on cohesion and interparticle friction for ultimate and reduced strength conditions as well as on the breakdown of the interparticle bonds. The use of the data and the analysis of slope stability led to the calculation of new safety factors that are significantly lower than those for which the slopes were designed.

SLOPE CHARACTERIZATION

Slope Failure Conditions

From a relatively large number of cases, the three basic slope failures selected for discussion represent typical occurrences. The common feature of these backslope sections is that they were cut through soil material that developed in place from the parent shale material.

The geometry of failure of the three slopes has the same pattern. Slope 3 is selected as representing this pattern, and its cross section is depicted in Figure 1. The events that preceded the slope failures and the field investigations following the failures indicate that a substantial augmentation of water content took place in the slope soil materials. In slope 1, ponding of water was observed on the top of the bench constructed to drain the water away from the cut, and for slopes 2 and 3 heavy rainfall is reported to have preceded the failure.

A very important feature common to all such slope failures, as shown in Figure 1, is that the weathered shale material was involved in the failure but not the more stable unweathered shale. Field investigations indicated a rotational block-type failure. The slip surface of failure coincided fairly well with the weathered-unweathered shale interface. Evidence of tension cracks was found in the material not involved in failure. Whether these cracks were there before failure is not known.

Since the failure took place soon after rainfall, strength mobilization seems to have occurred under undrained conditions.

Slope Material Properties

The shales from which the slope materials are derived are primarily marine deposits that at one time or another were subjected to high overburden pressures (6). Thus, the parent shales are classified as overconsolidated. Later on, the shales weathered, yielding the material found in the backslopes.

In the areas where the slides occurred, the in-place unit weights of the weathered shales vary from 1.5 to 1.8 Mg/m^3 and their natural moisture contents vary from 11 to 23 percent.

Table 1 summarizes the important properties of the slope soil materials obtained from Shelby tube samples. The geologic information was obtained from Sheerar (9). The engineering properties were determined by using the standard tests. The plasticity and clay content of the shales are reported in terms of two values. The first value is obtained by using the standard American Society of Testing and Materials tests; the second value is obtained by first treating the shale sample ultrasonically for 2 hours (6,7) and then running the standard tests for plasticity index and gradation. It is noteworthy that in all cases both the clay content and the plasticity index of the shales show increases after ultrasonic treatment. In fact, the augmentation of the clay-size material is quite marked, especially in the soil from slope 3.

Within a short time after failure, soil borings were obtained from the failed material. Consolidated-undrained (CU) triaxial strength tests, simulating the most probable field conditions, were run on the samples obtained from the field. Three different lateral pressures--68.9, 137.8, and 206.7 kPa (10, 20, and 30 psi)--were used. Stress-strain data to failure loads were recorded. Strength envelopes from these data are shown in Figure 2. By using the Webb technique (8), the shear-strength parameters at postfailure condition of reduced strength were calculated and are shown in Figure 3. The straightline envelopes of Figures 2 and 3 lead to the graphical determination of the c and ϕ values, both for the ultimate and the reduced strength conditions. These values are presented in Table 1.

For the ultimate strength, cohesion values varied from 83 to 159 kPa. For the reduced strength, they were between 28 and 55 kPa. The angle of friction values varied from 13° to 16° for the ultimate strength and from 11° to 12° for the reduced strength.

ANALYSIS OF FAILURE

The safety factors for slope stability were computed by using the circular arc method. The factors were found to range from 1.25 to 1.28 for the ultimate strength and from 1.04 to 1.08 for the reduced strength. Thus, it is unlikely that the ultimate strength values existed at the time of failure. The reduced strength values, calculated by using the Webb technique, indicate great potential for failure.

Another factor involved in failure stems from the geomorphology of these weathered shales, which have undergone further intensive weathering. This seems to have contributed to further reduction in the factor of safety.

To measure the change due to weathering and to substantiate it, the data obtained from the ultrasonic degradation test are used. These data indi-

Figure 1. Cut slope 3.

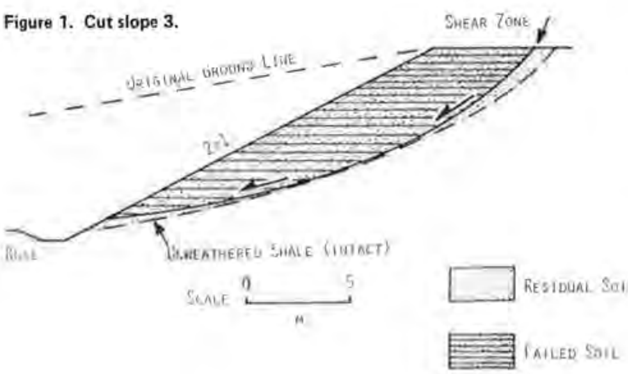


Table 1. Geotechnical and strength properties of slope materials.

Property	Slope 1	Slope 2	Slope 3
County	Pawnee	Love	McIntosh
Geologic system	Pennsylvanian	Cretaceous	Pennsylvanian
Physiographic region	Sandstone Hills	Red River	Prairie Plain
Geologic formation	Vamoosa (Kanwaka)	Caddo (Washita)	Senora (Senora)
Group character	Soillike	Soillike	Rocklike
Design slope	2:1	3:1	2:1
Depth of cut (m)	21.30	9.10	9.10
Year of construction	1963	1961	1963
Year of failure	1964	1964	1966
Annual rainfall (cm)	86	127	104
Clay minerals	Kaolinite, illite	Kaolinite, illite, montmorillonite	Kaolinite, illite, montmorillonite
$<2\mu$clay (%)			
Standard ASTM D424-63 (1972)	39	70	14
After ultrasonic test	48	84	65
Plasticity index			
Standard ASTM D424-59 (1971)	23	38	6
After ultrasonic test	25	40	22
Triaxial test			
Ultimate strength			
c_u (kPa)	118	159	83
ϕ_u (°)	13	14	16
Safety factor	1.25	1.28	1.25
Residual strength			
c_R (kPa)	35	55	28
ϕ_R (°)	11	11	12
Safety factor	1.04	1.08	1.04

Figure 2. Ultimate strength envelopes from triaxial test data.

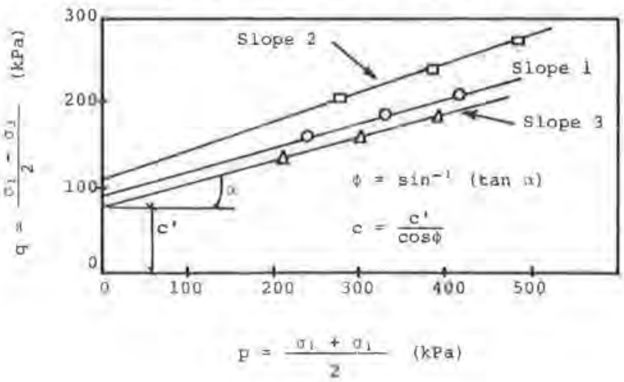
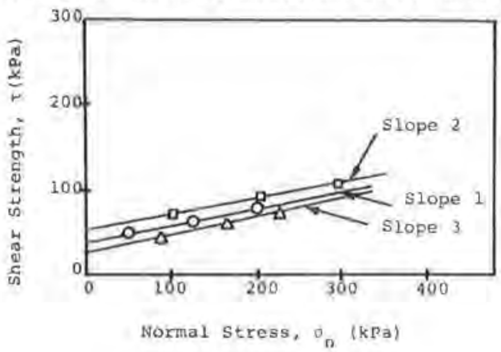


Figure 3. Residual strength envelopes computed by using the Webb technique.



cate that the 2-μm clay content increased 11, 14, and 51 percent for slopes 1, 2, and 3, respectively. The increase in the plasticity index was less pronounced for slopes 1 and 2 but substantial for slope 3. This augmentation in clay-size particles suggests that the soils, after slope construction and because of intensive weathering effects, may become more plastic and contain more clay-size particles than originally designed for. Higher plasticity and higher clay content result in lower permeability and the development of higher pore pressures under saturation; thus, the slope soil material is rendered more vulnerable to failure than anticipated during the design phase.

It should be pointed out that the ultrasonic test presents the soil material in its ultimately degraded state. That the slopes studied had reached that stage cannot be verified. However, it is certain that they had undergone some degree of degradation.

CONCLUSIONS

The stability analysis of slopes based on reduced strength, with the c_R and ϕ_R parameters computed from CU triaxial test data, predicts more realistically the critical field conditions. The potential of the slope soil material to develop high pore pressures and become less permeable in the field is strongly suggested by the increase in the 2-μm clay-size material and the plasticity index determined from the predictive ultrasonic degradation test data.

ACKNOWLEDGMENT

This investigation was a phase of a study supported by the Oklahoma Department of Transportation. We wish to express our appreciation to Willard McCasland of the Oklahoma Department of Transportation for his cooperation. The findings and conclusions do not necessarily reflect the official views or policies of the Federal Highway Administration or the U.S. Department of Transportation.

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Publication of this paper sponsored by Committee on Soil and Rock Properties.