Analysis of Slope Failure in Overconsolidated Fissured Residual Soils: A Case Study

STEVEN F. PUTRICH, ROY H. BORDEN, AND PHILIP C. LAMBE

A case study is presented of a well-documented slope failure within the Durham Triassic Basin of North Carolina that indicates the appropriateness of using noncircular failure geometrics and suggests the applicability of utilizing postpeak strengths obtained from drained direct shear tests. A companion paper in this Record presents an experimental investigation of the drained shear strength of soils from three sites within this basin.

This paper is the second of two papers in this Record dealing with slope stability analyses in overconsolidated fissured residual soils. In the first paper an experimental investigation of the drained shear strength of soils from three sites within the Durham Triassic Basin was described. Multistage direct shear tests were utilized and the results were found to be comparable with those that have been reported in the literature for other overconsolidated soils.

In this paper an analysis is presented of a slope failure that occurred in 1978 at the intersection of the Norfolk-Southern Railroad Underpass and US-64 located in the Durham Triassic Basin, the northern portion of the Deep River Triassic Basin deposit of North Carolina. Figure 1 shows the Deep River Basin deposit in relation to the series of Triassic deposits located along the Atlantic Seaboard and the general location of the site investigated in this study.

Geologically the Deep River Basin contains Triassic sedimentary rocks that are clastic deposits consisting of claystone, shale, siltstone, and sandstone. These deposits are characterized by abrupt changes in composition. The sediments of the Deep River Basin are composed largely of debris eroded from nearby pre-Triassic metamorphic and igneous rocks. In places they contain large amounts of debris derived from nearby granite intrusives. These sediments were deposited as alluvial fans, as stream-channel and floodplain deposits, and as lake and swamp deposits. Because of the nature of the depositional environment, a large percentage of the clays and clayey silts are fissured. Because the soils were deposited under wet conditions, subsequent drying and shrinkage caused cracks and openings classified as fissures.

Triassic sedimentary deposits are inclined to the southeast at angles ranging from 5 to 45 degrees. This dipping of the nonhomogeneous deposits is believed to have a major impact on the stability of cut slopes in Triassic deposits. The geometry of the failed slope mass appears to be dependent on the orientation of the soil layering. As was reported by Leith and Fischer (*1*), on the basis of a statewide survey of highway cuts, the slope failure frequency in the Triassic Basin was anomalously high. It can be inferred that the characteristic southeasterly dip of the Triassic sedimentary soil profile is a major factor in inducing a large number of slope failures.

The significance of this analysis and its application to evaluating the use of laboratory direct shear tests similar to those conducted as a portion of this study is that the Railroad Underpass/US-64 slope failure was extensively investigated and documented by the Geotechnical Unit of the North Carolina State Department of Transportation (NCDOT) in 1978. Therefore, there are sufficient field data to evaluate, with reasonable confidence, the maximum shear resistance mobilized at failure.

GEOTECHNICAL EVENTS AT THE RAILROAD UNDERPASS/US-64 SITE

The original project at this site, including the Norfolk-Southern Railroad/US-64 bridge overpass structure, was completed in late 1972 under the direction of the U. S. Army Corps of Engineers. In 1973 several slides of varying size occurred along the length of the railroad on the west side of the excavation adjacent to the bridge structure, including one relatively large, first-time slide. This slide forced the rerouting of rail traffic and posed a threat to the integrity of the bridge overpass. Several remedial measures were taken to correct the slide:

1. Clay material was trucked to the site and used to reshape the slope to its original configuration, and
2. Drain holes in the US-64 bridge deck over the Norfolk-Southern Railroad track were plugged. A 4-ft-wide concrete gutter was poured on the fill slopes beneath the drains of the bridge structure to facilitate surface drainage.

In May 1978 approximately 5 years after the original slide, a second slope failure occurred in the same general location. Figure 2 shows two views of the failed slope. In a subsequent inspection conducted in 1978 by the Geotechnical Unit of NCDOT, the following observations were made about the geometry and subsurface conditions after the failure:

1. The failed soil mass was approximately 110 ft long,

S. F. Putrich, Department of Civil Engineering, North Carolina State University, Raleigh, N. C. 27695-7908. Current affiliation: Woodward-Clyde Consultants, Wayne, N. J. R. H. Borden and P. C. Lambe, Department of Civil Engineering, North Carolina State University, Raleigh, N. C. 27695-7908.

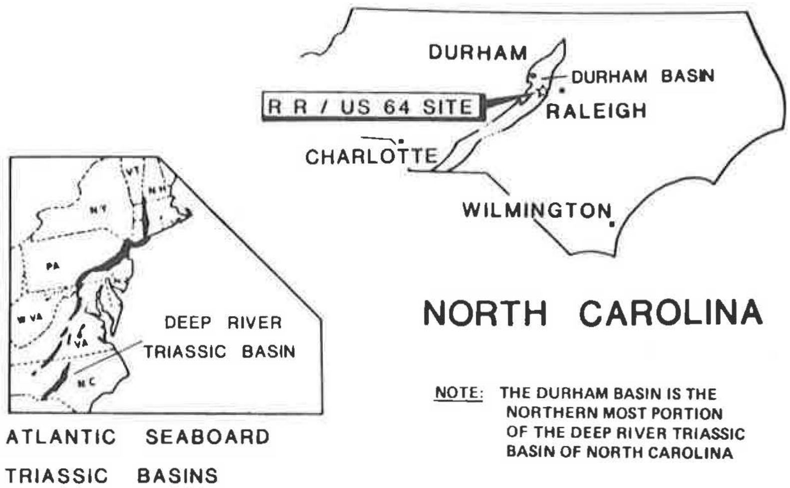

FIGURE 1 Location of Deep River Basin and Railroad Underpass/ US-64 site.

extending southward from the bent at the western end of the bridge structure. Figure 3 shows the extent of the failure and the direction of movement of the failed soil mass into the excavation. The opposite side of the excavation, in which the bedding planes dipped away from the excavation, remained stable.

2. The slope before failure was approximately 25 ft high with a 1.5 horizontal to 1 vertical slope. The location of the 1978 slide appeared to be the same as that observed in the 1973 slide at the site. Therefore this was considered a second-time slide (i.e., soil mass sliding along a previous failure surface).

3. Scarps and cracks were observed both on the surface of and within the failed soil mass.

4. Two major naturally occurring soil units were present in the cut opposite and adjacent to the failed zone (i.e., the west side of the excavation); the upper unit was a tan and gray fissured clayey sand with varying sand-clay ratios (tan unit), and the basal unit was a dark brown to maroon silty clay with irregular seams of light tan and gray fissured clay in the upper portion of the layer (maroon unit). The contact between the upper and lower units undulates and was identifiable by a thin layer of slightly clayey coarse sand.

5. As part of the postfailure investigation by NCDOT in an effort to locate the failure surface and identify the subsurface conditions, three test borings and a series of rod soundings

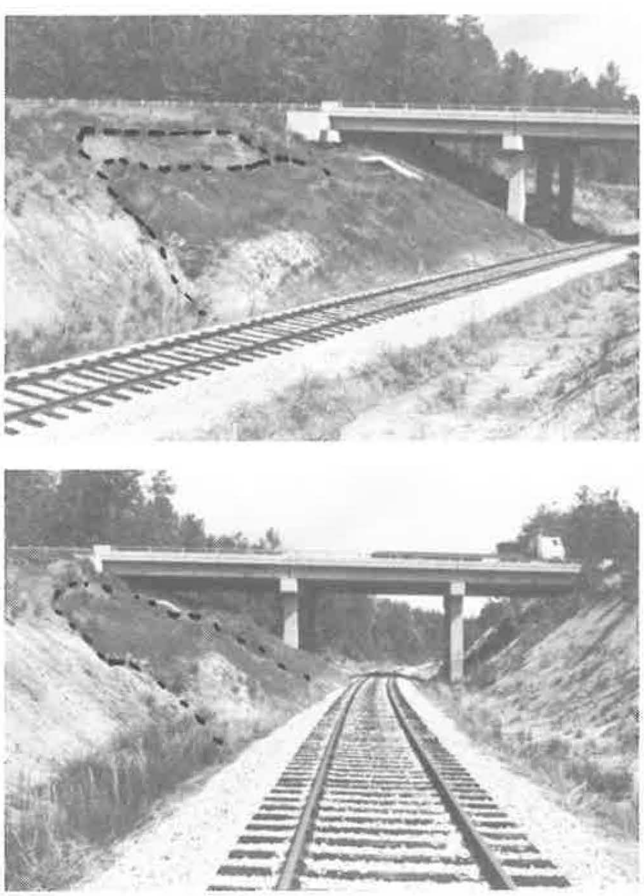

FIGURE 2 Railroad Underpass/US-64 slope failure.

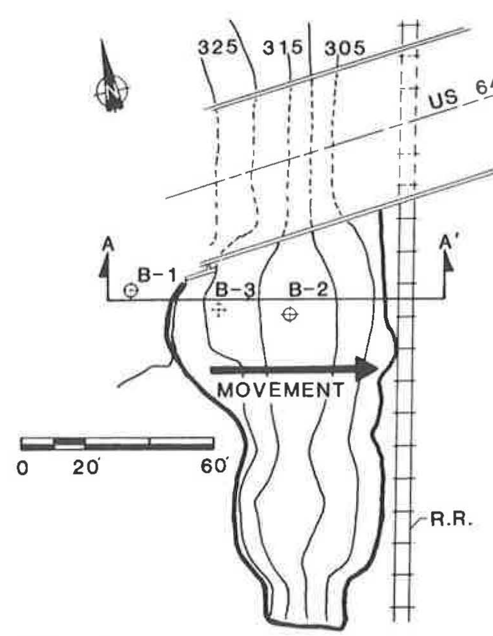

FIGURE 3 Extent of slope failure.

were made at the site shortly after failure occurred, within and adjacent to the failed soil mass. In situ vane shear tests were conducted in the vicinity of the failure surface, and soil samples were recovered in order to conduct laboratory soil classification and shear strength tests. Slope indicator pipe with attached well point was installed in two of the borings located in the failed soil mass in order to monitor further slope movement and postfailure groundwater levels.

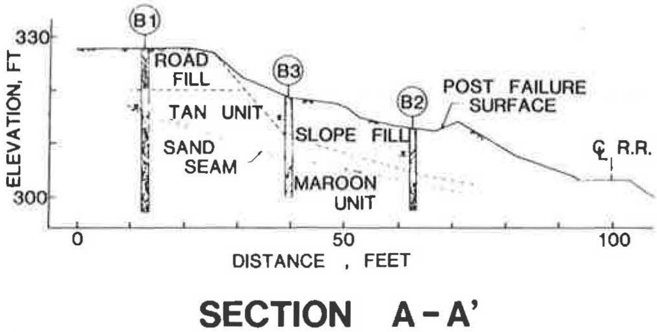

FIGURE 4 Generalized subsurface profile: Section *A–A′*.

Figure 4 shows the generalized subsurface profile (Section *A–A′*) at the Railroad Underpass/US-64 site as defined by Borings B-1, B-2, and B-3 completed by NCDOT during their postfailure investigation. The soil profile can be divided into the following soil units:

1. The top unit is fill material used to construct the existing roadway embankment in 1972. This occurs only in Boring B-1.
2. The second unit is a red clay soil brought to the site to reshape the slope after the initial failure in 1973. This red clay is the surface unit in Borings B-2 and B-3.
3. The third unit is the tan unit, previously described.
4. In between the third and basal unit is a thin layer of slightly clayey coarse sand found in Borings B-1 and B-3.
5. The basal unit is the maroon unit previously described.

The following postfailure surface water and drainage conditions were noted:

1. The perforated pipes used to drain water from beneath the bridge approach slab (part of the original bridge and roadway construction) were clogged.
2. The ditches along the toe of the failed slope were filled with soil debris from other smaller slope failures, which impeded drainage in the ditch.
3. Ditches constructed above the crest of the cut slopes designed to drain off surface water had reduced capacity due to siltation and vegetation. The surface drainage conditions at the site were considered poor.
4. There was excessive surface erosion at the site because of heavy surface runoff from precipitation. This was shown by the undermining of a pile cap supporting the bridge structure immediately adjacent to the failed slope.
5. It was noted that the original design report described the railroad excavation as a "wet" cut, with portions of the excavation below the natural groundwater table. At the time of the NCDOT investigation, groundwater was observed breaking out near the toe of the failed slope, and the face of slope was wet.

As a result of this second failure in 1978, a tie-back pile and timber retaining wall was constructed to hold the sliding mass in place and ensure the integrity of the bridge structure on the site. Figure 5 shows two views of the slope rehabilitation project with the retaining wall in its final stages of construction. Approximately 1 year after the installation of the retaining wall, water was observed seeping through the timber retaining wall at ground elevation. The face of the slope at that time was reported to be saturated and the toe ditch filled with water. These were signs that there was still a problem with on-site groundwater seepage and surface water drainage. To correct the drainage problem, a small wall was constructed back of the toe ditch so that the ditch would be free draining. In addition, 6-in. perforated pipes were installed in the slope to drain the wet slope through the new wall.

More than 10 years after the completion of the railroad and bridge structure, the west side of the railroad excavation at the site has remained stable at a slope of approximately 1 horizontal to 1 vertical. Conversely, the east side of the excavation with a slope of approximately 1.5 horizontal to 1 vertical and soil bedding planes dipping into the cut has experienced two slope failures in this same period of time. With the assumption that the soil-strength and pore-pressure regimes on both sides of the excavation are approximately equal, it can be inferred that the

FIGURE 5 Slope rehabilitation.

orientation of the subsurface profile with respect to the cut has a significant influence on the probability of slope failure in this type of soil deposit. In general, an excavation in the Deep River Triassic Basin made parallel to the direction of the dipping soil layers (i.e., northwest–southeast direction) would have the lowest probability of sliding. This statement is based on the assumption that the variables of slope geometry, soil properties, and pore pressure are constant.

The slope failures at the Railroad Underpass/US-64 site along with the anomalously high number of slope failures in highway cuts within the Triassic Basin deposit are strong evidence to support the need for designing natural slopes with consideration for local geologic characteristics. The geologic environment in the Deep River Triassic Basin deposits is characterized by inclined soil bedding planes, overconsolidated fissured soils, and abrupt changes in soil composition. It is critical that strength parameters utilized in the design of slopes be applicable to sliding along these inclined bedding planes and appropriate in the evaluation of long-term stability.

SLIDE MECHANISMS AND ANALYTICAL TECHNIQUE

The following three factors are believed to have had a major impact on the behavior of this slope:

1. Geometry (including orientation of soil layers) and soil type within the failed soil mass,
2. Soil strength in the failure zone, and
3. Pore-water pressure.

Geometry of Sliding Mass

In order to perform an analysis after the slide, one must determine the geometry, including the surface and subsurface boundaries, of the soil mass that slid. In an effort to determine the location and angle of inclination of the sliding surface, NCDOT personnel completed a series of probe soundings within and adjacent to the failed soil mass in addition to the test borings previously cited. Visual observations and surveys were used to identify the exposed boundaries of the failed slope.

Boring logs for the test borings drilled are given in Figure 6. Included in the logs are data on plasticity and percent passing the No. 200 sieve and the standard penetration values as a function of depth. As shown in the test boring logs, the standard penetration data indicate a significant increase in resistance to penetration as a function of depth into the maroon unit. These data, in conjunction with results from numerous rod soundings, suggest that the sliding surface under the failed soil mass wedge was restricted by the maroon unit. The slope geometry before and following the failure and the estimated failure surface are shown in Figure 7 and correspond to Section *A–A′* within the failed soil mass (see Figure 3 for location of Section *A–A′*).

As shown in Figure 7, the estimated failure surface for this 1978 slide is located in the vicinity of the interface of the tan and maroon soil units. On the basis of field observations, this is thought to be the sliding surface for the previous slope failure

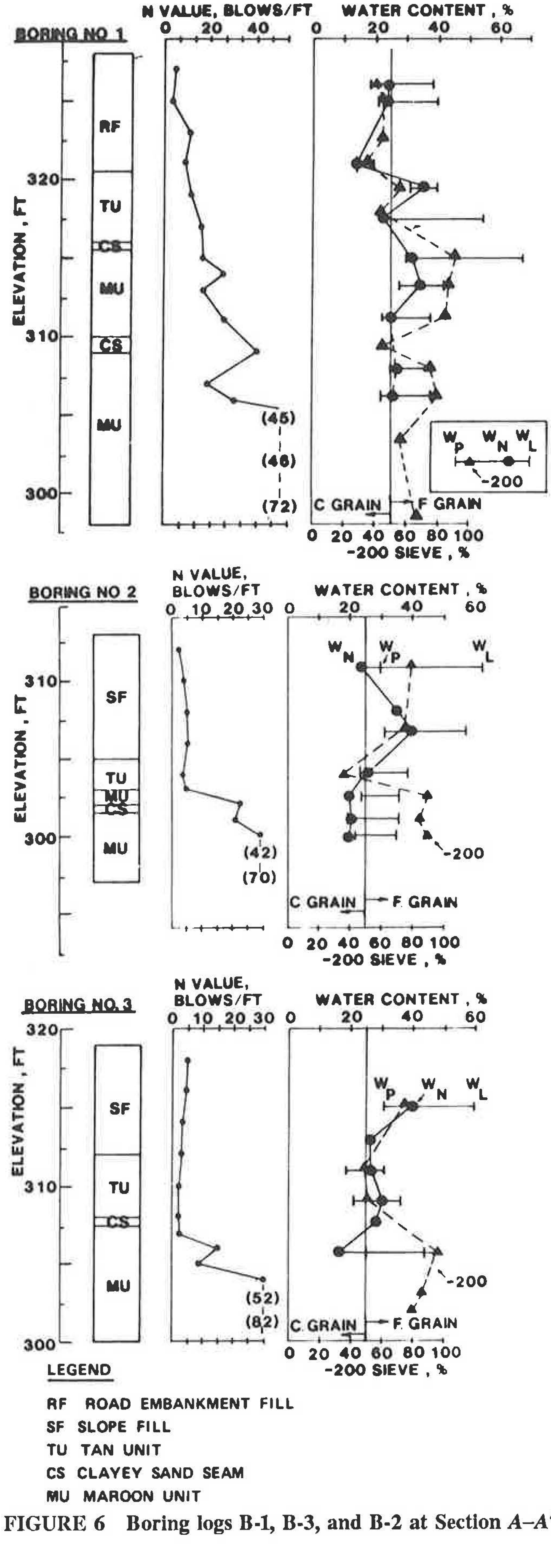

FIGURE 6 Boring logs B-1, B-3, and B-2 at Section *A–A′*.

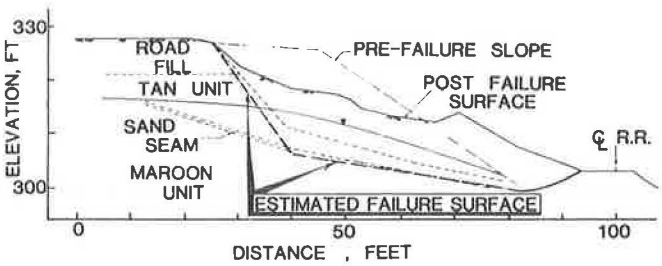

FIGURE 7 Slope geometry and estimated failure surface.

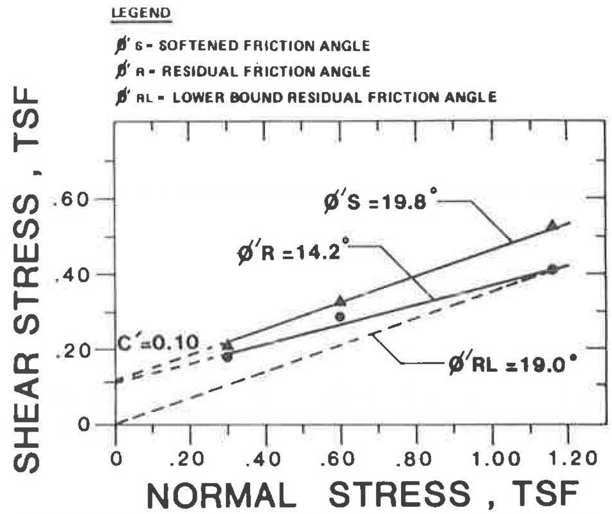

FIGURE 9 Drained direct shear test results: shear stress versus normal stress (*2*).

that occurred at the site 5 years earlier. Therefore, the shear strength of soil in the vicinity of this interface is of particular interest for the slide analysis.

Soil Strength

In the summer of 1983, a program of drained direct shear tests was undertaken, and the gray and tan fissured clay seam from the upper portion of the maroon soil unit was tested. The results of these direct shear tests for the Railroad Underpass/US-64 site are presented in Figures 8 and 9. The stress-versus-displacement results demonstrate that this material loses strength with continued horizontal displacement beyond the peak strength and are in general agreement with the idealized response of stiff, fissured clay during a drained direct shear test as shown in Figure 10 [after Skempton (*2*)].

For the fissured clays found at the referenced site, a peak strength value is an unconservative estimate of the available in situ shear strength with regard to stability of cut slopes in these soils. This post-peak-strength phenomenon can be explained by the progressive failure theory, which states that the peak strength is passed at any one point along a failure surface within a cut slope because of fissures or discontinuities that act as stress concentrators forcing that point to pass the peak strength with a given amount of displacement. Once the peak strength has been passed at one point along the failure surface, the stress is shifted to another point, causing it to pass the peak, and so on. In this way a progressive failure can be initiated, and the strength along the entire or the majority of the length of the slip surface will decrease as a function of displacement to a lower strength range bounded by the softened and residual strengths. Because this is a previously failed slope, it would be expected that a shear strength significantly less than a peak strength would be mobilized at failure. This lowered shear strength is attributed to the accumulated displacement from movements of the soil mass during the initial slide.

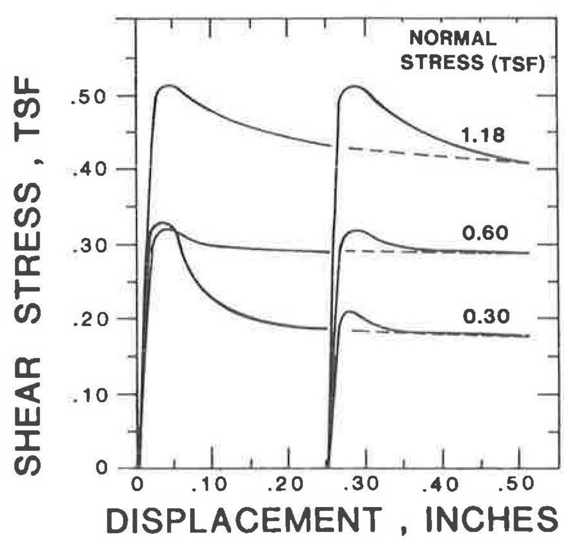

FIGURE 8 Drained direct shear test results: shear stress versus displacement.

The softened strength and zero cohesion residual strength indicated in Figure 10 are generally recognized as the upper and lower boundaries for the range of in situ strength mobilized for slope failures in overconsolidated fissured clays. The softened strength is defined as the peak stress response for a "remolded" normally consolidated clay (i.e., critical state condition) and the residual strength is defined as that value of stress at which further accumulated displacement will not result in a lowering of strength (i.e., steady-state condition) and is generally recognized as an indication of the in situ strength mobilized in a second-time slope failure.

Shear strength envelopes corresponding to the softened, residual, and lower-bound residual conditions for the fissured clay tested are shown in Figure 9. A dashed line has been used to indicate that the shear strength below the lowest stress increment in the strength envelopes is not known. The dashed portion of these lines is a continuation of the slope of the strength envelopes for the stress range tested to the vertical intercept. An apparent effective cohesion (c') is given for the purpose of defining the linear equation of the envelope.

In light of the small amount of displacement used in this test series compared with that in studies by LaGatta (*3*) and Lupini et al. (*4*) that specifically investigated the amount of displacement required to mobilize the minimum strength of clay soils, the lower residual envelope has been introduced. This is a best estimate of the "practical" lower limit of shear strength of this fissured soil. A dashed line is used to indicate that this is a theoretical envelope. As it will be shown in the analysis of the referenced slide, the lower-bound residual significantly under-

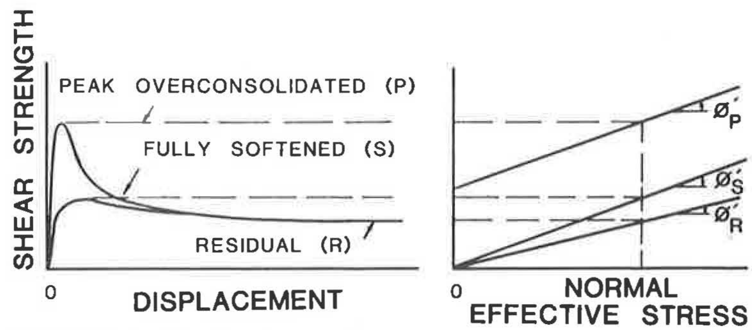

FIGURE 10 Idealized stress-displacement response of stiff fissured clay (2).

estimates the lower-bound in situ shear strength. In the companion paper to this analytical study by Borden and Putrich, a detailed explanation of the range of shear strength results and testing technique used for the series of direct shear tests performed in this study is given. Subsequently it will be shown how the laboratory-determined shear strength may be applied to the slope failure under consideration.

Pore Pressure

In order to conduct a slide analysis in terms of effective stresses, the pore pressure within the cut slope must be known. The pore pressure (u) is defined as the sum of the initial pore pressure before construction (u_0) and the excess pore pressure (Δu) due to changes in total stress as a result of excavating for the cut slope. In this study the initial stage is an undrained unloading followed later by drainage. Therefore, the pore pressure initially is influenced to a large degree by the changes in total stress to which the soil mass was subjected during excavation. When an excavation for a cut slope is made, the total stresses decrease, so that with time the pore pressure increases from the end-of-construction value u to $u_0 + (-\Delta u)$. These rising pore pressures result in a reduction in the factor of safety. Therefore, a cut that was designed to be safe using total stresses (undrained failure analysis) may slide after a period of time when the negative excess pore pressure decreases. This behavior is demonstrated in Figure 11 [after Bishop and Bjerrum (5)] and indicates that the drained condition is more critical than the undrained condition. As shown in the plot of factor of safety versus time, the factor of safety continues to decrease until the pore pressure reaches a steady-state condition ($\Delta u = 0$). Skempton (6) refers to this condition as the long-term stability, where the pore pressure is determined directly from the flow net. The effective stress analysis has been employed in numerous slope stability studies, including ones by Lambe et al. (7), Wu (8), Vaughan and Walbanke (9), and Peck (10).

The available data from the Railroad Underpass/US-64 postfailure investigation were used to estimate the pore pressures in the cut slope at the time of failure. The pore-pressure data from this site were obtained from well measurements (slope indicator pipe with well points attached to the bottom of the pipe) within and immediately outside the failed soil mass. These wells are not piezometers and therefore did not furnish pore-pressure data directly but gave an estimate of the groundwater level within the slope (i.e., phreatic surface). The well measurements taken immediately after the slope failure were used to evaluate the location of this phreatic surface. The pore pressures were determined directly from the location of the phreatic surface with respect to the sliding surface by assuming horizontal flow. The location of the outbreak of groundwater on the face of the failed slope observed during the NCDOT investigation was also used to estimate the position of the groundwater level. The estimated location of the phreatic surface at the time of failure is shown in Figure 7.

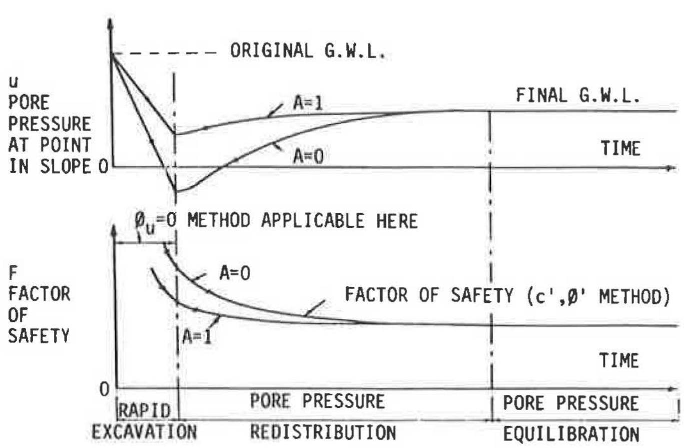

FIGURE 11 Pore pressure and factor of safety as function of time for cut slope in clay (5).

Excessive erosion and poor surface drainage were observed in the field investigations in 1973 and 1978 after each of the slope failures. It can be inferred that this site was subject to heavy surface runoff from precipitation. Much of this runoff was retained on the surface of the slope and allowed to infiltrate because of poor surface drainage. Sangrey et al. (11) state that this infiltration contributes to two factors that adversely affect the stability of slopes: increase of the unit weight of the soil mass as saturation increases and increase of the pore pressure (uplift force) on the potential sliding surface by raising the phreatic surface. Sangrey et al. further commented that a vast majority of landslides are related directly or indirectly to precipitation. This infiltration effect was indirectly accounted for in this analysis by using the well measurements taken shortly after failure. Change in the level of the phreatic surface because of surface infiltration was assumed to be reflected in the postfailure well readings.

It may be concluded that the buildup of pore pressures due to both dissipation of negative excess pore pressure induced by excavation and the infiltration of precipitation because of poor surface drainage acts as a triggering mechanism in the case of a marginally stable slope. There is strong evidence indicating that this was the case in this slope failure.

Analytical Technique

In describing the problem of an unstable slope, one may say that sliding occurs when the sum of the forces actuating movement becomes equal to the sum of the maximum forces resisting the movement. As used in this study, the term "slide analysis" refers to the situation where a slide has occurred. The slide analysis was performed by the wedge method, which uses the equilibrium factor-of-safety approach outlined in the *Navy Design Manual* (12).

Based on the postfailure field investigation by the Geotechnical Unit of NCDOT, the landslide would be described as a partly rotational block failure according to the movement terminology used by Varnes (13). Movements of the soil mass involved the sliding and partial rotation of soil blocks (wedges) as shown in Figure 12. Because the layering of the soil within the slope was inclined into the cut, the soil mass had an inherent tendency to slide into the cut. The sliding surface was planar (noncircular), oriented parallel to the inclination of the soil layering, and occurred in the vicinity of the interface between the tan and maroon units.

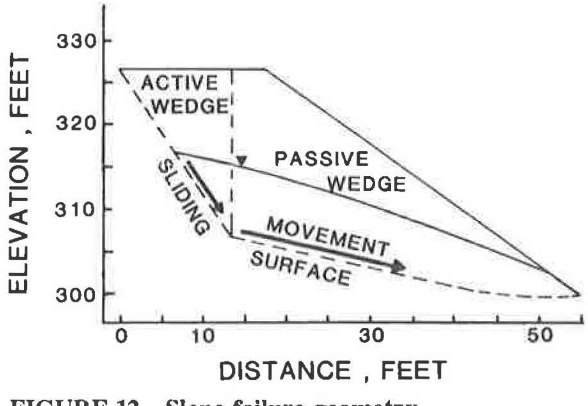

FIGURE 12 Slope failure geometry.

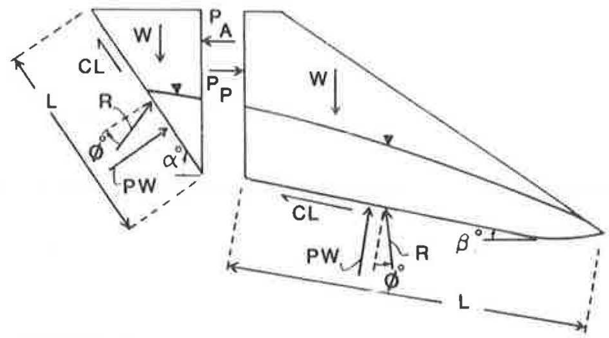

FIGURE 13 Wedge analysis model (12).

The failed mass may be thought of as two wedges moving downward and outward. If one imagines a wall between the two wedges, the wedge to the left of the wall corresponds to the active wedge and the wedge to the right of the wall is the passive wedge. Figure 13 shows a free-body diagram of both the active and passive wedges and a description of the forces involved. The equations used to determine the active and passive wedge forces, P_a and P_p, shown in Figure 13, are as follows:

$$P_a = [W_a - c_{am}L \sin(\alpha) - P_{aw} \cos(\alpha)] \tan(\alpha - \phi'_{am}) - [c_{am}L \cos(\alpha) - P_{aw} \sin(\alpha)]$$

$$P_p = [W_p - c_{pm}L \sin(\beta) - P_{pw} \cos(\beta)] \tan(\beta - \phi'_{pm}) + [c_{pm}L \cos(\beta) - P_{pw} \sin(\beta)]$$

where

- P_a, P_p = resultant horizontal force for active and passive wedges, respectively;
- W_a, W_p = total weight of soil and water in active and passive wedges above sliding surface;
- c_{am}, c_{pm} = mobilized cohesion acting along sliding surface of active and passive wedges where $c_m = c/F_s$ and F_s = factor of safety;
- α, β = inclination of active and passive sliding surface with respect to horizontal;
- L = length of sliding surface of active and passive wedges;
- ϕ'_{am}, ϕ'_{pm} = mobilized effective friction angle acting along sliding surface of active and passive wedges where $\phi'_m = \tan^{-1}(\tan\phi'/F_s)$;
- P_{aw}, P_{pw} = resultant force due to water pressure on potential sliding surface of active and passive wedges; and
- R_{am}, R_{pm} = result of normal and tangential forces on sliding surface considering mobilized friction angle of material.

The resultant horizontal force due to water pressure on the interface of the active and passive wedges is not shown in these equations. These forces have been assumed equal and opposite and therefore cancel out.

The following five steps outline the procedure used to determine the resultant P_a and P_p:

1. Determine the necessary constants: weight (W_a and W_p), water pressure force (P_{aw} and P_{pw}), and the length of the sliding plane (L_a and L_p);
2. Select a set of strength parameters (ϕ' and c', or c);
3. Determine the mobilized strength parameters in terms of a trial factor of safety (ϕ'_m and c'_m or c_m);
4. Solve for P_a and P_p in terms of the mobilized parameters and the constants from Steps 3 and 1, respectively; and
5. Repeat Steps 3 and 4 for a range of trial safety factors.

Steps 2 through 5 were repeated for a range of strength values, including the unconfined compressive strength and in situ vane shear strength.

The final step in the procedure of this slide analysis was to plot curves for the factor of safety versus values of P_a and P_p as a function of shear strength acting along both the active and passive failure surfaces. A sample of this simple procedure is shown in Figure 14. The intersection of any two curves (P_a and P_p, respectively) indicates that equilibrium between the active and passive wedges has been satisfied and corresponds to a factor of safety for that set of strength parameters.

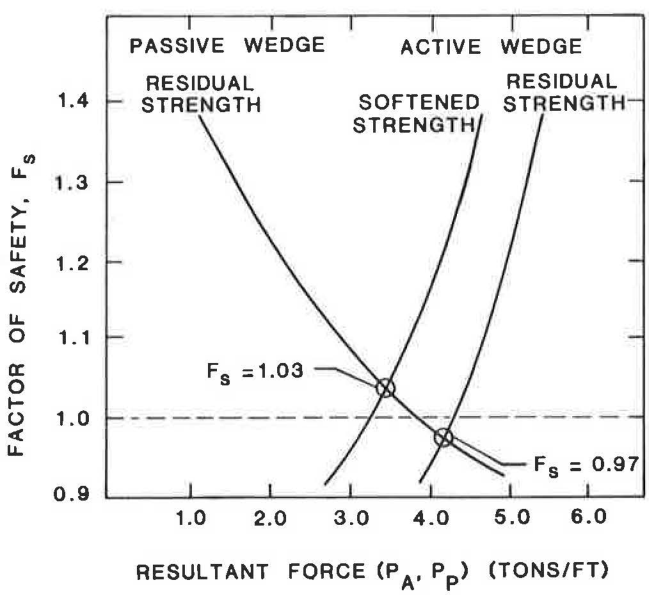

FIGURE 14 Equilibrium factor of safety.

SLIDE ANALYSIS RESULTS

Slide analyses were made for the Railroad Underpass/US-64 site by using the previously described slope failure geometry and pore-pressure conditions. Various shear strengths from drained direct shear tests conducted for this study were investigated. In addition, undrained shear strengths generated from unconfined compression tests and field vane tests completed by the NCDOT Geotechnical Unit were also evaluated.

The slide analyses were conducted by using the softened, residual, and lower-bound (zero-cohesion) residual strengths from the direct shear tests for the passive portion of the sliding surface. Only the softened and residual strengths were considered for the active portion of the sliding surface. There is evidence to suggest that the failed soil mass in the first and second slope failures of the Railroad Underpass/US-64 site slid along the bedding planes in the vicinity of the interface between the tan and maroon units represented by the passive sliding surface. Therefore a lower strength range was evaluated for the soils along the passive surface that would have accumulated more displacement than the soils on the active surface. There was no evidence to suggest that the active sliding surface was the same for both the first and second slope failures.

Table 1 gives the results of the analyses in terms of equilibrium factors of safety for the various shear strengths investigated. In evaluating the results presented in Table 1, the following observations were made:

1. The drained strength parameters govern the long-term stability of this cut slope. The logic for this behavior was described by Skempton (*14*), who concluded, on the basis of field evidence, that the failure of cut slopes in overconsolidated fissured London clays occurred years after excavation because of the slow dissipation of excess negative pore pressure. This position was stated earlier by Vaughan and Walbanke (*9*). This same pore-pressure equilibration is occurring in cut slopes in the overconsolidated residual soils of the Triassic Basin. A general indication of the overconsolidation ratio was obtained from consolidation test results on a fissured clay from a nearby site, as presented in the companion paper of this analytical

TABLE 1 EQUILIBRIUM FACTOR OF SAFETY AS A FUNCTION OF SHEAR STRENGTH

Drained Direct Shear Test			Undrained Shear Test[a]	Field Vane Test[b]
Active Wedge	Passive Wedge	Factor of Safety		
τ_s	τ_s	1.17	2.3	2.5
τ_r	τ_s	1.09		
τ_s	τ_r	1.03		
τ_r	τ_r	0.97		
τ_s	τ_{rl}	0.86		
τ_r	τ_{rl}	0.8		
τ_s[c]	—	1.25		
τ_r[c]	—	1.18		

Note: τ_s = softened strength; τ_r = residual strength; τ_{rl} = lower-bound residual strength.

Undrained shear strength values given refer to results of tests conducted by the NCDOT as a part of the postfailure geotechnical investigation completed in 1978.

The drained shear strength value used for the slightly clayey coarse sand layer (ϕ' = 32 degrees) is assumed. This value is considered a conservative estimate of shear strength based on the range of typical shear strengths for sands in overconsolidated soil deposits.

Equilibrium factor of safety values less than 1.0 indicate instability for the given slope geometry, pore pressure, and shear strength.

[a] s_u = 0.44 tsf.

[b] s_u = 0.55 tsf (0.9) = 0.49 tsf.

[c] Coarse sand layer, estimated ϕ' = 32 degrees.

study (Borden and Putrich in this Record). The maximum past effective stress was found to be 1.7 tsf, indicating an overconsolidation ratio of 3 to 4. The delay of 1 year and 5 years between the end of construction and the occurrence of the first and second slope failures at the site provides evidence that pore-pressure equilibration played a role in the failures.

2. For the failure geometry and pore-pressure regime used in these slide analyses, the residual shear strength appears to closely match the in situ shear strength mobilized for the slope failure investigated. This finding is in agreement with the general design approach for cut slopes in overconsolidated fissured clays that have experienced previous sliding.

3. The lower-bound (zero-cohesion) residual strength is considered to be the lower limit of in situ strength that would be predicted for failure in cut slopes. This is consistent with results shown in Table 1, which indicate that the lower-bound residual strength underestimated the resistance mobilized at failure, as indicated by factors of safety less than 1.

4. The analyses based on the unconfined compressive strength and vane shear strength confirm that use of the undrained strength is an unconservative approach to the analysis of overconsolidated slopes as shown in Table 1. With factors of safety greater than 2.0, it is apparent that undrained shear strength overestimates the available in situ shear strength when the slide analysis technique described is used.

5. As seen in the results of the NCDOT postfailure investigation, the contact between the upper and lower tan and maroon fissured clay units was identified by a slightly clayey coarse sand layer. The majority of the failure surface (i.e., the passive surface) was located in the vicinity and oriented parallel to the interface plane of the tan and maroon units. There was therefore an interest in evaluating the possibility that the failure surface was located in this sand layer. An analysis was conducted by using an assumed shear strength (ϕ' = 32 degrees) considered a conservative estimate on the basis of the range of typical shear strength values for sands in overconsolidated deposits. The resulting factor of safety using this strength value for the sand layer was higher than that determined for the drained strengths for the fissured clay evaluated. Therefore, on the basis of these results, the failure surface was most likely located in the fissured clay and not in the sand layer.

EVALUATION OF TEST RESULTS

In order to verify the results of the analyses performed, the slope was evaluated for the average shear and normal stresses acting on the failure surface at failure (i.e., F_s = 1.0) by using the wedge analysis with a vector approach. These average stresses were then plotted on the diagram of shear versus normal stress shown in Figure 15. Included in this figure are the strength envelopes generated from the direct shear tests conducted on the fissured clay from the Railroad Underpass/US-64 site. The average stresses from both the active and passive sliding surfaces are indicated by points *A* and *P*, respectively.

Figure 15 indicates that average stresses approximately equal to or slightly greater than the residual shear strength governed in the long-term stability of the slope. This finding is consistent with the results of the wedge analysis using the equilibrium factor-of-safety approach, which indicated that the residual

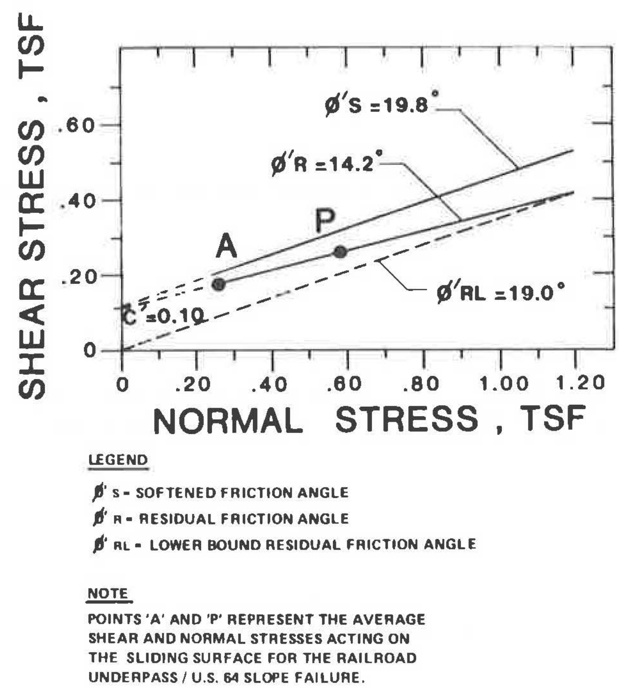

FIGURE 15 Average stress at failure for the Railroad Underpass/US-64 site.

strength closely matched the in situ shear strength acting at failure for the previously failed slope.

CONCLUSIONS

On the basis of the results of the direct shear tests performed in this study, an evaluation of a well-documented slope failure at the Railroad Underpass/US-64 site, and the literature cited in this paper, the following conclusions are advanced:

1. The slide analysis performed in this study showed that the residual strength from drained direct shear tests closely approximated the average in situ strength mobilized at failure for the second-time slide at the Railroad Underpass/US-64 site. For overconsolidated fissured clays, a peak shear strength value is an unconservative estimate of the available shear strength with regard to the stability of cut slopes in these soils. In the case of a previously failed slope, a shear strength significantly less than the peak value represents the available in situ strength. This lower strength is due to the accumulated displacement associated with shearing of soil along the sliding surface during a previous slope failure.

2. In overconsolidated soils the use of the undrained shear strength was shown to be unconservative. Factors of safety greater than 2.0 resulted in the analysis of the failed slope using undrained shear strength values from unconfined compression and field vane tests. Drained strength parameters govern the long-term stability of cut slopes because the undrained strength will subsequently decrease with time as the negative pore pressures are dissipated. The delay of 1 year and 5 years between the end of construction and the occurrence of the first and second slope failures provides evidence that pore-pressure

equilibration (strength decreasing from an undrained to drained condition) played a role in the failures for the slope evaluated.

3. The infiltration of precipitation into a cut slope that is already experiencing a gradual buildup of pore pressure due to the dissipation of the negative excess pore pressures induced by excavation can act as a triggering mechanism, causing a marginally stable slope to fail. It is therefore important to evaluate the change in pore-pressure regime that will occur in the slope after the end of construction, with consideration given to surface and subsurface drainage provisions. It is very likely that the increase of pore pressure due to the infiltration of runoff acted as a triggering mechanism for the slope failures at the Railroad Underpass/US-64 site. Poor surface drainage facilitated the retaining and infiltration of runoff into the cut slope, thereby adversely affecting stability.

4. Noncircular failure geometries are most appropriate for describing slides occurring in nonhomogeneous soil deposits, which are characterized by inclined soil layers and planar seams of weak material as was the slope evaluated in this study.

5. The soil stratigraphy within the Durham Triassic Basin is characterized by bedding planes that are inclined, dipping downward in a southeasterly direction. Therefore, an excavation made parallel to the direction of the dipping soil layers (i.e., northwest–southeast direction) would have the lowest probability of sliding, with all other variables constant (i.e., slope geometry, soil properties, and pore pressure). When the orientation of excavations is otherwise, special attention must be given to the dipping of inclined layers into the cut. The importance of this issue is shown by the sliding that has occurred at the Railroad Underpass/US-64 site. The side of the excavation with the soil layers dipping into the cut has experienced two major slides in the same area, whereas on the opposite side of the excavation, the slope has remained stable since construction.

ACKNOWLEDGMENTS

The authors wish to express their sincere appreciation to the North Carolina Department of Transportation, whose assistance made this study possible. The use of field records, photographs, and post-slide investigation reports is gratefully acknowledged. Special thanks are due Dave Bingham, Geotechnical Unit, and Fritz Koch, area geologist.

The authors also wish to thank Woodward-Clyde Consultants, Wayne, New Jersey, for their financial support in the preparation of the final manuscript.

REFERENCES

1. C. J. Leith and C. P. Fisher. *An Investigation of the Stability of Highway Cut Slopes in North Carolina.* Project ERD-110-U. Highway Research Program, North Carolina State University, Raleigh, 1964.
2. A. W. Skempton. First-Time Slides in Over-Consolidated Clays. *Geotechnique*, Vol. 20, No. 3, 1970, pp. 320–324.
3. D. P. LaGatta. *Residual Strength of Clays and Clay-Shales by Rotation Shear Tests.* Harvard Soil Mechanics Series 86. Harvard University, Cambridge, Mass., 1970.
4. J. F. Lupini, A. E. Skinner, and P. R. Vaughan. The Drained Residual Strength of Cohesive Soils. *Geotechnique*, Vol. 31, No. 2, 1981, pp. 181–213.
5. A. W. Bishop and L. Bjerrum. *The Relevance of the Triaxial Test to the Solution of Stability Problems.* NR. 34. Norwegian Geotechnical Institute, Oslo, 1960.
6. A. W. Skempton. Long-Term Stability of Clay Slopes. *Geotechnique*, Vol. 14, 1964, pp. 77–102.
7. T. W. Lambe, F. Silva, and W. A. Marv. Instability of Amuay Cliffside. *Journal of the Geotechnical Engineering Division, ASCE*, Vol. 107, No. GT11, Nov. 1981, pp. 1305–1520.
8. T. H. Wu. *Stability of Slopes in Shale and Colluvium.* Civil Engineering Department, Ohio State University, Columbus, Dec. 1981.
9. P. R. Vaughan and H. J. Walbanke. Pore Pressure Changes and the Delayed Failure of Cutting Slopes in Overconsolidated Clays. *Geotechnique*, Vol. 23, No. 4, 1973, pp. 531–539.
10. R. B. Peck. Stability of Natural Slopes. *Journal of the Soil Mechanics and Foundation Division, ASCE*, Vol. 93, No. SM4, 1967, pp. 403–418.
11. D. A. Sangrey, K. O. Harrop-Williams, and J. A. Klaiber. *Groundwater Response to Precipitation with Application to Slope Stability.* USGS 9-6102-05813. Department of Civil Engineering, Carnegie-Mellon University, Pittsburgh, Pa., 1982.
12. *Navy Design Manual: Soil Mechanics.* NAVFAC DM-7.1. U. S. Navy Department, May 1982.
13. D. J. Varnes. "Slope Movement Types and Processes." In *Special Report 176: Landslides: Analysis and Control* (R. L. Schuster and R. J. Krizek, eds.), TRB, National Research Council, Washington, D. C., 1978, pp. 11–33.
14. A. W. Skempton. "Slope Stability of Cuttings in Brown London Clay." *Proc., 9th International Conference on Soil Mechanics and Foundation Engineering*, Tokyo, Vol. 3, 1977, pp. 261–270.

Publication of this paper sponsored by Committee on Engineering Geology.