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STABILITY OF NATURAL SLOPES IN SENSITIVE CLAY

BY

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LA STABILITE DES TALUS NATURELS D'ARGILE SENSIBLE

SOMMAIRE

L'affaissement des talus naturels d'argile sensible se propage souvent suffisamment pour former un cirque caractéristique plus vaste que la niche de décrochement. Les auteurs décrivent deux affaissements de ce type qui se sont produits dans la région d'Ottawa, Canada, et comparent leur forme avec les talus stables des environs. L'un des affaissements s'est produit il y a plusieurs centaines d'années, mais les modifications subies par le réseau d'écoulement des eaux souterraines depuis l'affaissement de terrain semblent avoir consolidé les talus avoisinants. Le second affaissement s'est produit en 1963 dans une région de glissements fréquents. Les auteurs ont calculé dans chaque cas quelle était la résistance des terrains nécessaire au maintien des talus dans diverses conditions du niveau hydrostatique, et ont comparé les chiffres obtenus avec les valeurs mesurées. Ils en ont conclu qu'une analyse précise des contraintes subies par les terrains de ces talus donne des résultats satisfaisants si les essais sont menés dans l'éventail des contraintes possibles et si les conditions physiques affectant le niveau hydrostatique sont soigneusement mesurées.

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STABILITY OF NATURAL SLOPES IN SENSITIVE CLAY[a]

Cárl B. Crawford[1] M. ASCE, and William J. Eden[2]

INTRODUCTION

The approximations that must be made in the analysis of a natural clay slope are so questionable that actual case records are essential to the development of reliable procedures. Even case records are subject to such a variety of interpretations that, on the basis of an exhaustive study of the most reliable published material, Skempton[3] observed that ". . .from the analysis of actual slips in clay, the values of the shear strength parameters as determined by conventional tests do not necessarily bear any relation to the values which must have been operative in the clay at the time of failure." This is a useful qualification to keep in mind in evaluating past and present case records.

Part of the uncertainty in any stability analysis is caused by the variation of soil properties and groundwater conditions; the more inaccurate analyses, however, are probably the result of incorrect interpretation of failure in the soil tests. When the nature and rate of stress changes during sampling and

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[a]Presented at the August 22-26, 1966, ASCE Soil Mechanics and Foundations Division Conference on Stability and Performance of Slopes and Embankments, held at Berkeley, Calif.

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[3]Skempton, A. W., "Long Term Stability of Clay Slopes," Géotechnique, Institution of Civil Engineers, London, Vol. 14, No. 2, 1964, pp. 75-102.

testing are considered it is not surprising that conflicting interpretations of failure can occur.

RESIDUAL STRENGTH

Skempton dealt primarily with insensitive, overconsolidated clays and specifically excluded the sensitive Leda clays of Eastern Canada from his analysis. The overconsolidated clays described by Skempton tend to expand during shear, especially after the peak strength has been reached. The increasing water content caused by this expansion, together with particle reorientation, local overstressing, and time effects, results in a reduced strength at large strains. This is called the "residual" strength and it is shown that the strength required for long-term stability of slopes in these clays approaches the residual strength. In general it can be concluded that normal testing of insensitive, overconsolidated clays results in peak strengths that are greater than those mobilized in nature and stability computations are consequently on the unsafe side.

Sensitive clays, unlike the soils just described, can exist in nature at rather high void ratios as a result of their bonded structure, even when they are substantially overconsolidated. Under increasing or sustained shearing stresses the bonds are broken and the structure collapses causing a decrease in volume. If the drainage of pore water is too slow, pressures will build up in the water phase until the effective stress on the failure plane approaches zero. This is the state that is thought to exist on the failure plane of a flow slide. It is not at all certain, therefore, that the residual strength concept can be applied to these soils.

LEDA CLAY

Leda clay is the name given to the extensive deposits of sensitive clay in the Ottawa and St. Lawrence River Lowlands of Eastern Canada. This paper will deal with soils in the Ottawa region that are thought to have been deposited in brackish water during a late glacial stage. They have an open, flocculated structure that breaks down under strain to a liquid consistency. The natural soils are often reasonably uniform, composed of 60% to 80% clay-sized particles, with a plasticity index of 30% to 40%. Void ratios up to 2 are common below the drying crust which usually extends to depths of 10 ft to 20 ft. The clays are relatively nonswelling and have a low salt content in the pore water.

In the Ottawa region it has been shown that the undrained shear strength and the preconsolidation pressure of samples of the natural clay are related to their elevation above sea level.[4] Assuming a classical geological history, it is estimated that as much as 200 ft of overburden have been removed from the lower plains since the clay was deposited. The interpretation of the stress-deformation properties of Leda clay is as uncertain as that of the overconsolidated clays described by Skempton, but for different reasons. The influence of rate of loading on resistance to one-dimensional compression[5] and to shearing

[4] Crawford, C. B., and Eden, W. J., "A Comparison of Laboratory Results with In-Situ Properties of Leda Clay," Proceedings, 6th International Conference on Soil Mechanics, Montreal, Vol. 1, 1965, pp. 31-35.

[5] Crawford, C. B., "Interpretation of the Consolidation Test," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 90, No. SM5, Proc. Paper 4056, Sept., 1964.

resistance[6] appears to be especially important for the normal short-term tests. It is questionable whether parameters obtained from such tests are reliable when applied to long-term field problems.

LANDSLIDES IN LEDA CLAY

When a bank fails it is possible to evaluate the actual shearing strength of the soil in place, but the reliability of the evaluation is limited by several factors. The first problem is to obtain a sufficient number of good, representative, undisturbed samples of soil in the field and to preserve them in their natural condition until they are installed in the testing device. Next is the problem of carrying out an appropriate test, which simulates all the important variables encountered in the field, such as stress path, drainage conditions, and rate of strain, and from which simple characteristic parameters can be derived. Finally there is the problem of determining the initial slope and groundwater conditions and the mode of failure. For a retrogressive slide in sensitive clay the initial failure is obscured by subsequent movements.

The writers have, from time to time, investigated a number of small river bank failures in the Ottawa area with uncertain results. There is evidence of many flow slides in the area but only one large one has occurred in recent years and has been subject to extensive study. This recent slide and an older classical flow slide are described herein. Both slides are located in similar soils but about 15 miles apart. The recent slide occurred in 1963 in a deep ravine near Breckenridge Station, Quebec. This ravine, which cuts through a large plain below the Gatineau Hills northwest of the City of Ottawa, contains many landslide scars and has had some bank failures in recent years. The older slide is just south of the Ottawa River and about a mile east of the Ottawa City limits. The slide is actually located in an old terrace of the Ottawa River a short distance west of Green Creek in an area of steep banks that appear to be quite stable.

In each of the two creek valleys (Breckenridge and Green Creek) the banks were scanned on aerial photographs and the steepest and highest sections of bank were noted. These slopes were then scaled with a stereo-plotter and the more critical ones were checked in the field by stadia survey. More than 30 slopes were selected in this way for stability analyses. For a first approximation each slope was reduced to a straight line and analysed with the stability coefficients developed by Bishop and Morgenstern.[7] In this way the cohesion, in terms of effective stresses, required at limiting equilibrium for various assumed values of soil density, friction angle, and groundwater conditions was computed for each slope. Some of these results are given in Table 1. One slope, Number 9, was analyzed using Bishop's slices method on the actual ground contours and the computed c' increased from 549 psf to 610 psf with the more accurate method. All of these slopes were then analyzed again using an

[6] Crawford, C. B., "Cohesion in an Undisturbed Sensitive Clay," Géotechnique, Institution of Civil Engineers, London, Vol. 13, No. 2, 1964, pp. 132-146.

[7] Bishop, A. W., and Morgenstern, N., "Stability Coefficients for Earth Slopes," Géotechnique, Insititution of Civil Engineers, London, Vol. 10, No. 4,1960, pp. 129-150.

TABLE 1.—COMPUTED COHESION (c') REQUIRED FOR STABILITY[a]

Slope No.	Slope Angle, in degrees	Slope Height, in feet	Required Cohesion, in pounds per square foot	
			By stability coefficient	By computer
(a) Green Creek Area				
1	21	61	515	580
2	20½	64	540	580
3	25½	47	446	400
4	17½	68	502	470
5	18½	62	478	510
6	24	56	512	645
7	17	33	239	250
8	17	65	471	420
9	24	60	549	585
10	12½	45	232	225
11	23	60	537	580
31	17½	66	490	495
32	16	65	455	480
33	15	50	---	302
34	19½	63	507	---
(b) Breckenridge Area				
12	27	89	860	930
13	25	85	796	815
14	14½	46	285	265
15	20½	55	457	515
16	17½	51	378	405
17	34	36	392	485
18	16	44	301	330
19	29	40	407	395
21	20½	79	657	540
24	19	73	576	580
25	14½	69	427	385
26	12	54	261	265
27	18½	93	716	712

[a]assumed $\phi' = 15°$, $r_u = 0.62$, $\gamma = 100$ pcf.

IBM 1620, Mark II computer[8] in which a more complex representation of slope was possible. The computer results are also shown in Table 1.

GREEN CREEK SLIDE

Description and Soil Tests.—The Green Creek slide, as shown in Fig. 1, is a classical example of an earthflow in which approximately 30 acres of upland

FIG. 1.—AIR VIEW OF GREEN CREEK SLIDE

flowed over about 70 acres of lowland to a depth of 15 ft to 20 ft. The upland, which is about 100 ft higher than the lowland, appears to be a terrace left by the Ottawa River. The crater is clearly separated from the flow material by a ridge that marks the former toe of slope. Many similar craters have been ob-

[8]Irwin, W., "The Use of a Digital Computer for Solving Slope Stability Problems," Computer Program No. 19, Div. of Bldg. Research, National Research Council, Ottawa, Canada, 1964, 8 pp.

served a few miles down-stream in the Ottawa River valley but usually all of the flowed material has been washed away. A recently obtained age determination by the Geological Survey of Canada on a piece of wood from beneath the apron of the slide suggests that it is more than 1,000 yr old (Geological Survey of Canada, No 550, 1140 ± 150 yr Before Present.)

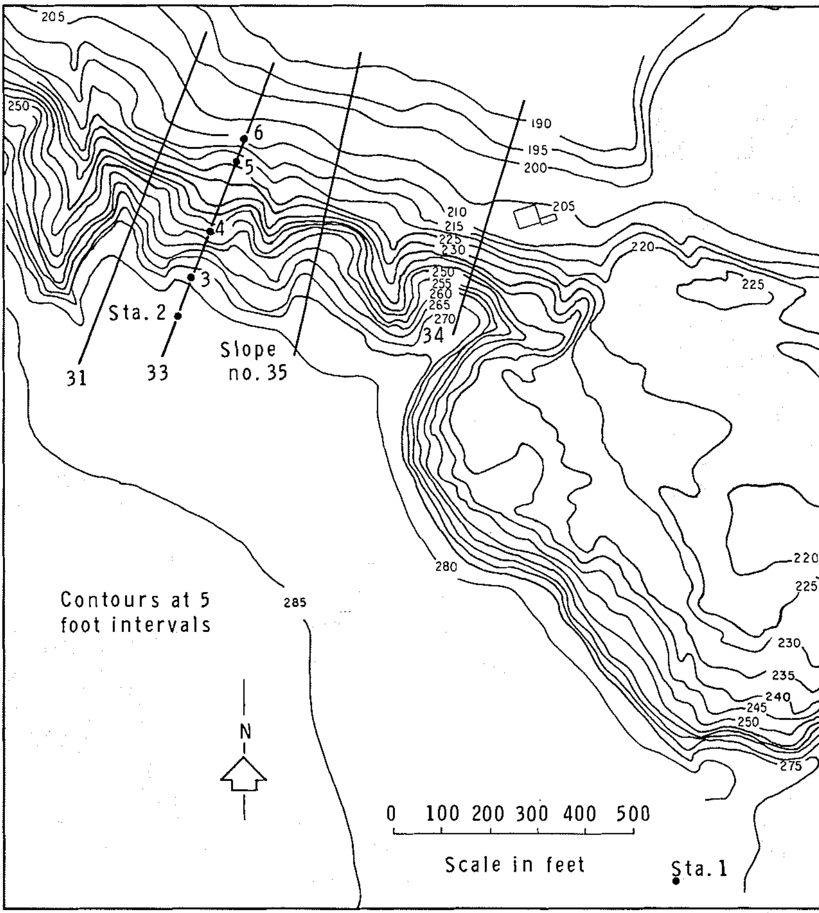

FIG. 2.—PLAN OF GREEN CREEK SLOPES (LOCATION INDICATED ON FIG. 1)

The original bank shown in Fig. 1 is oriented approximately east-west with the flow toward the north. The natural ground has been disturbed at the south-east end of the crater by the removal of borrow for a nearby roadfill. The contours of the present ground surface west of the crater are shown in Fig. 2. The locations of borings and field tests and four of the slopes that have been studied are also shown. Continuous samples were obtained to a depth of 80 ft at

Station 1 using a 54-mm, thin-walled piston sampler.[9] Classification tests from Station 1 are shown in Fig. 3. Recent observations in excavations through the apron of the slide have revealed the original soil profile, including a surface peat layer.[10] Limited tests show the remoulded soil in the apron to be some-

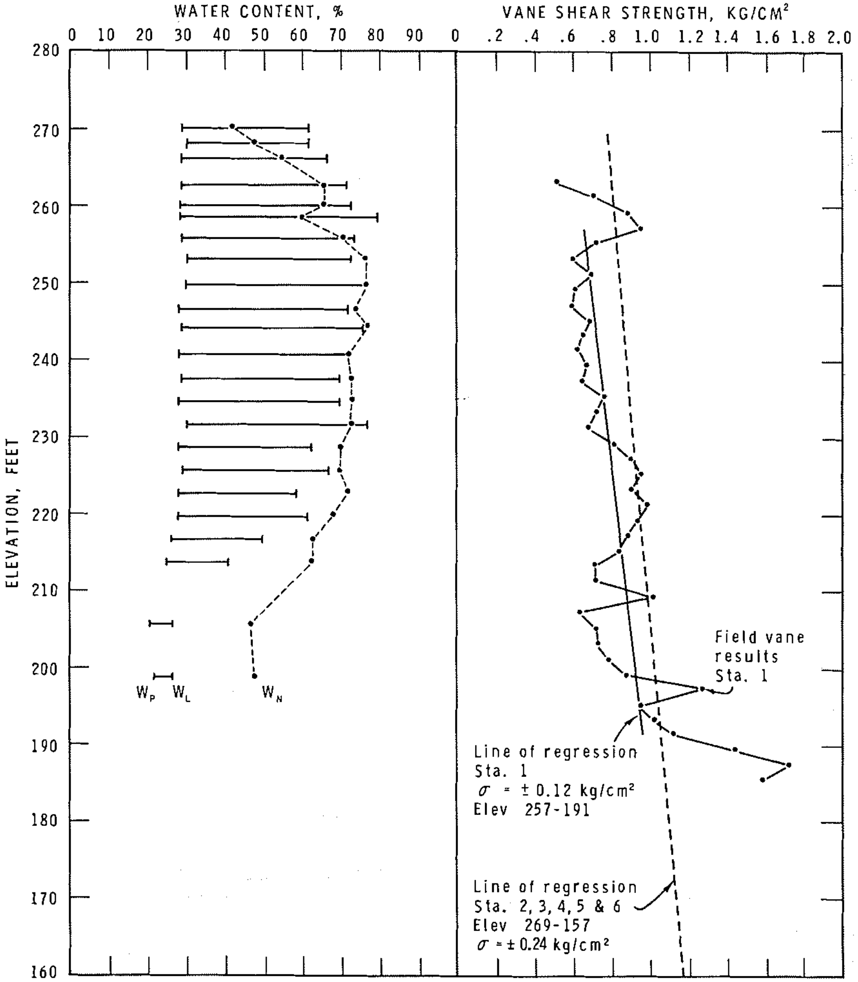

FIG. 3.—PROFILE OF TEST RESULTS, GREEN CREEK

what drier (w = 65%) than the natural soil at Station 1 and to have a field vane strength equal to about one half the natural undisturbed strength. The flowed

[9] Bjerrum, L., "Geotechnical Properties of Norwegian Marine Clays," Géotechnique, Institution of Civil Engineers, London, Vol. 4, No. 2, 1954, pp. 49-69.

[10] Eden, W. J., "Buried Soil Profile under Apron of an Earth flow," Geological Society of America Bulletin, 1967.

soil appears therefore to have been reconsolidated but not enough to regain its original strength.

In 1961, three piezometers, at depths of 20 ft, 40 ft, and 60 ft, were located on slope 33 about 24 ft downslope from Station 3. These revealed a substantial downward flow of groundwater which had not been expected. Six additional piezometers were installed in November, 1964, and all are shown on the cross-section in Fig. 4. Regular measurements have indicated that maximum pore pressures occur in May and minimum pressures in August or September. Based on the highest annual pressures the contours of pore-pressure ratio are sketched on Fig. 4. (The pore-pressure ratio is the ratio of pore water pressure to total weight of soil above a point in the ground, $r_u = u/\gamma h$.) It is easily observed that the downward flow of water will have a substantial effect on the effective stresses in the ground and on the stability of the slope.

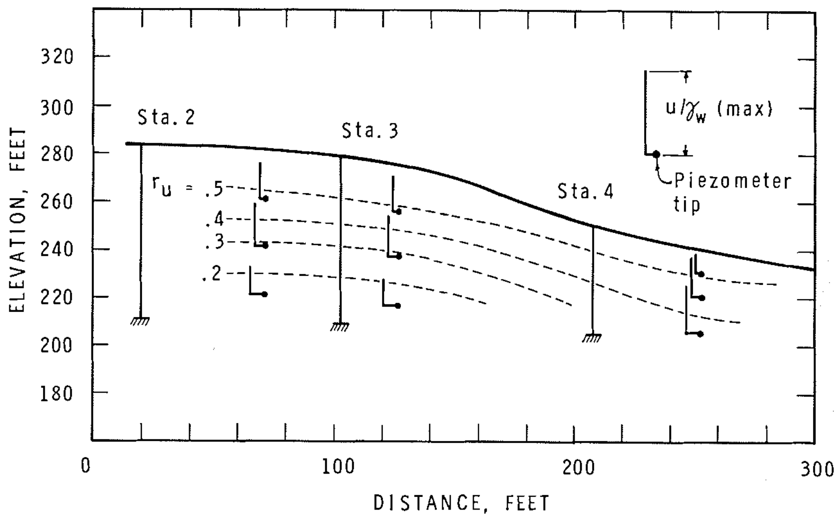

FIG. 4.—SLOPE 33, GREEN CREEK

Stability Analysis.—Of all the slopes in the Green Creek area, slopes 9 and 34 are closest to limiting equilibrium. Slope 34, adjacent to the landslide crater, is assumed to represent the slope at the location of the initial failure. The cross-section shown in Fig. 5 was therefore analyzed in detail using the pore pressures measured on slope 33 to estimate the maximum value of r_u.

Previously Irwin[8] had prepared Bishop's method of analysis for an IBM 1620 computer but the program was rewritten for an IBM 360 model 50 computer which has since become available. This machine has a greater capacity and permitted more extensive computations. It was assumed that the materials in the slope are homogeneous with c', ϕ', and γ being constant, and that the pore water pressures can be represented by an average constant for a number of circles. Accordingly, for $F = 1$ and γ = 100 pcf, the required c' was determined for various assigned values of ϕ' and r_u. The maximum value of c' was determined for r_u = 0.62 (groundwater table at surface and horizontal

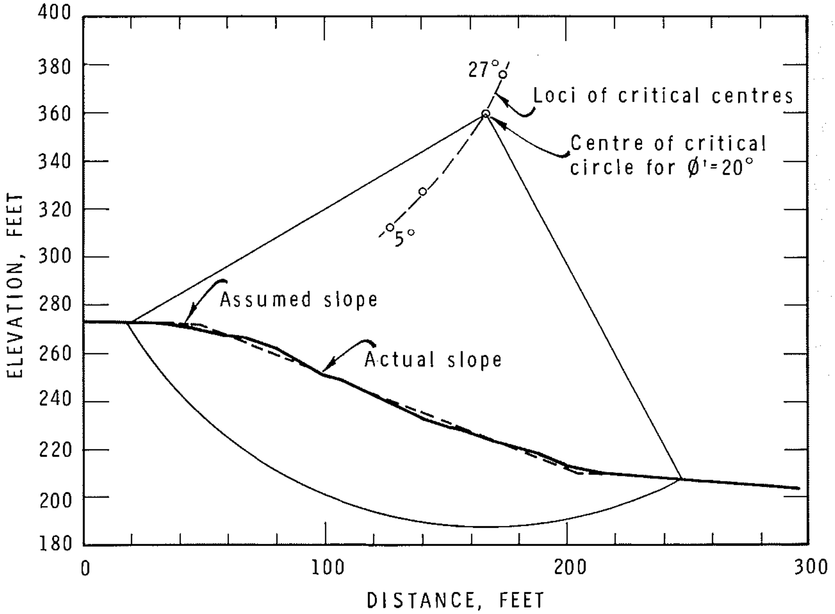

FIG. 5.—SLOPE 34, GREEN CREEK

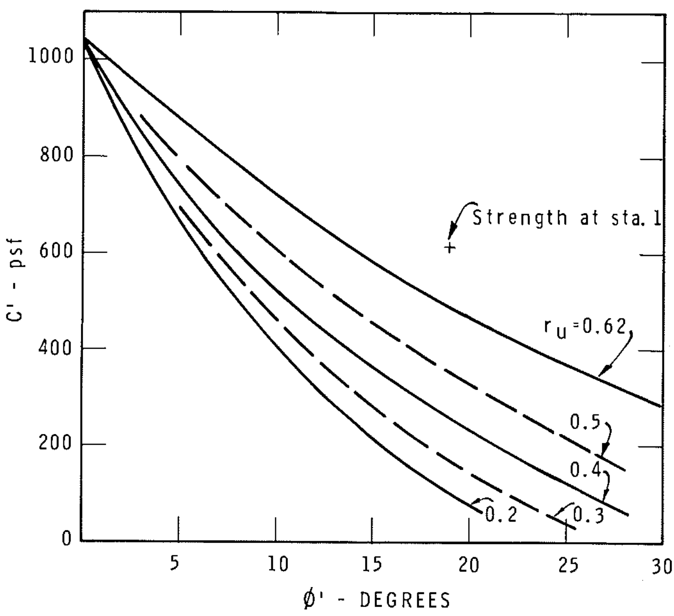

FIG. 6.—STRENGTH REQUIRED FOR STABILITY OF SLOPE 34

flow) and for various values of ϕ', the critical circle changing in each case. With r_u = 0.2 and 0.4, the maximum values of c' were again determined for values of ϕ'. The results of these solutions are presented in Fig. 6 and one of the most critical circles, with r_u = 0.62 and ϕ' = 20° is shown in Fig. 5. Although this critical circle passes much below the toe of slope it has been deduced that the actual failure plane emerged near the toe because there still exists a ridge at the lip of the crater that appears to be the lower part of the original bank (see Figs. 1 and 2).

Based on detailed observations along slope 33, the average r_u along the critical circle through slope 34 is estimated to be about 0.3. It is probable that with natural forest conditions, long-term wet cycles, and higher Ottawa River levels over a thousand-year period the groundwater pressures would be considerably higher than at present.[10] The writers believe that the r_u value could have been as high as 0.6.

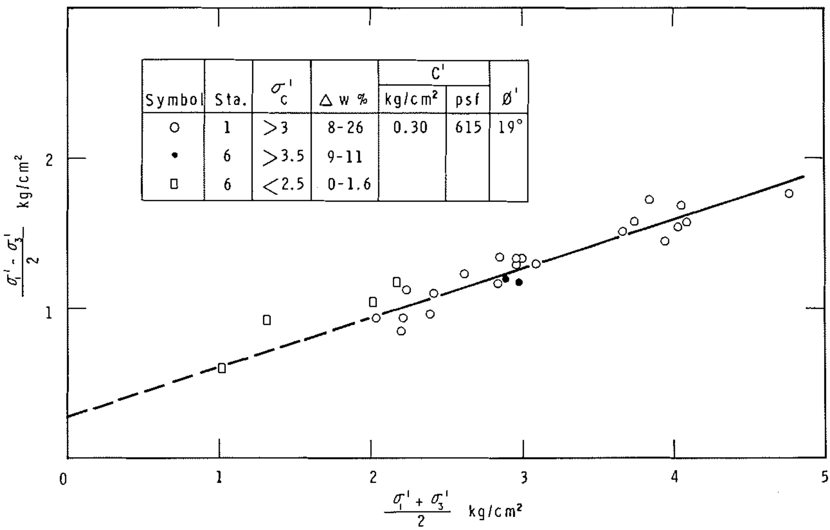

FIG. 7.—STRENGTH TEST RESULTS, GREEN CREEK

Measured Shear Strength.—The undrained shear strength of the natural clay around the landslide has been measured by field vane at Stations 1 to 6. The lines of regression and the standard error shown in Fig. 3 reveal fairly uniform soil conditions. Between El. 200 and El. 240, which cover most of the critical sliding surface, the undrained shear strength at slope 33 (Stations 2 to 6) averages about 0.9 kg per sq cm. At the same elevations at Station 1 the average shear strength by field vane test is about 0.8 kg per sq cm and by laboratory compression test is about 0.6 kg per sq cm.

Twenty-one consolidated-undrained triaxial compresssion tests were carried out on 1-1/2-in.-diam specimens, 3 in. high obtained between El. 200 and El. 240 at Station 1. All specimens were loaded at rates of strain between 1% per hr and 2% per hr. The results, averaged by the method of least squares, gave c' = 0.30 kg per sq cm (615 psf) and ϕ' = 19° as shown in Fig. 7. All of these specimens were consolidated under cell pressures of 3 kg per sq cm or more and since this exceeds the preconsolidation pressure of approximately 2.5 kg per sq cm, the water contents were reduced in proportion to the cell pressure by amounts ranging from 8% to 26%. The initial water contents averaged 72% and the lowest final water content was 45%.

In 1964, a shallow boring was made between Stations 5 and 6 near the toe of slope 33. Six consolidated-undrained tests were made on specimens at

FIG. 8.—AIR VIEW OF BRECKENRIDGE SLIDE

average El. 231, where the preconsolidation pressure was measured to be about 3.5 kg per sq cm. Four of the tests were made at cell pressures of 2.5 kg per sq cm or less and they gave higher than average strengths as shown in Fig. 7. Unfortunately insufficient results are available to establish reliable shear parameters in this range.

Strength Comparisons.—The shear strength available according to tests and that required for stability may now be compared on Fig. 6. It will be seen that according to the triaxial tests the factor of safety would be slightly greater than one even if the piezometric head on the slip plane was equal to the height of soil above it (r_u = 0.62). Under present conditions (r_u = 0.3) the average effective stress normal to the slip surface is about 1 kg per sq cm and the factor of safety is equal to about 1.6.

BRECKENRIDGE SLIDE

Description and Soil Tests.—The Breckenridge slide occurred during a heavy rainstorm in the late evening of April 20, 1963 destroying one of the pole structures of a power line. Retrogressive slips occurred intermittently for the

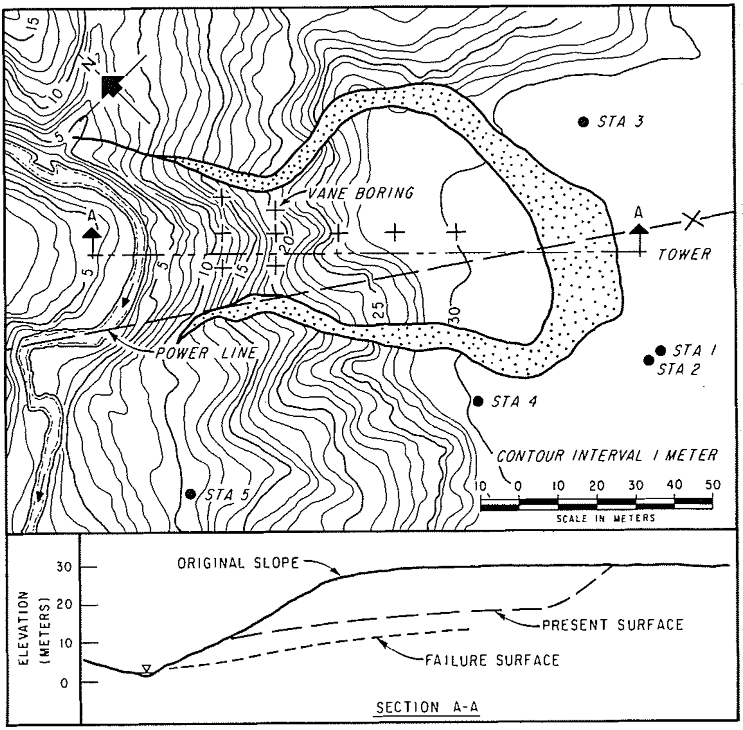

FIG. 9.—PLAN OF BRECKENRIDGE SLIDE AREA

next three days. About 30,000 cu yd of clay were involved in the slide. The upland is at about El. 330 and the stream bed is at El. 240. Unlike the Green Creek region this is an area of active sliding with visible scars of relatively recent landslides. The slide is shown in Fig. 8 and a contour plan of the area before failure is shown in Fig. 9. This plan was prepared by the Photogrammetric Research Section of the Division of Applied Physics, National Research Council from photographs taken in 1961 by the Royal Canadian Air Force. The locations of the landslide and of test borings and field vane tests also appear on the plan.

Field work was carried out at the site immediately following the slide. An examination of the scarp showed a surface cap of about 3 ft or 4 ft of sandy silt overlying clay that appeared to be highly stratified near the surface. Borings revealed more uniform clay extending to a depth of more than 90 ft. Continuous samples were obtained with the thin-walled piston sampler at Station 1 and vane tests were carried out at Stations 2 to 5 (Fig. 9). The results of tests are shown in Fig. 10. The vane tests indicate a drying crust to a depth of about 20 ft and evidence of drying or case hardening along the

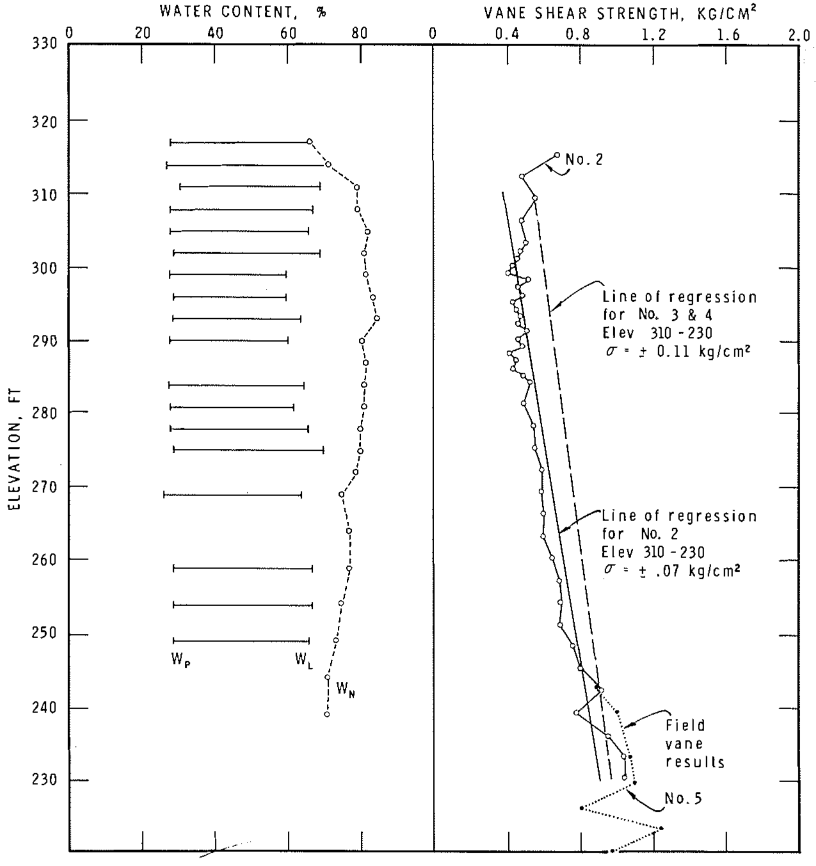

FIG. 10.—PROFILE OF TEST RESULTS, BRECKENRIDGE

slope. The preconsolidation pressure at this site is less than that at Green Creek, averaging less than 2 kg per sq cm.

Vane tests were made in the crater of the landslide at locations shown in Fig. 9. In most of these locations the failure plane of the slide was easily indentified by a change from erratic to consistent strength measurements. This failure plane is sketched on section A-A in Fig. 9 and is seen to pass above the toe of slope. Vane tests prependicular to this section suggests that the failure surface is dished.

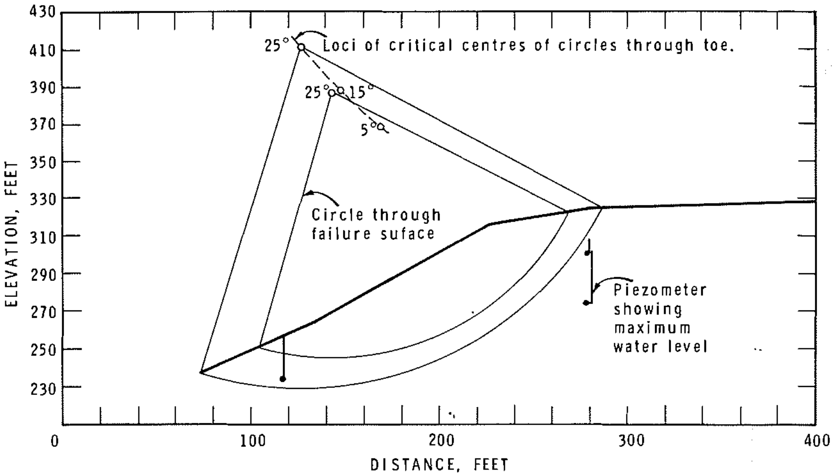

FIG. 11.—SLOPE 12, BRECKENRIDGE

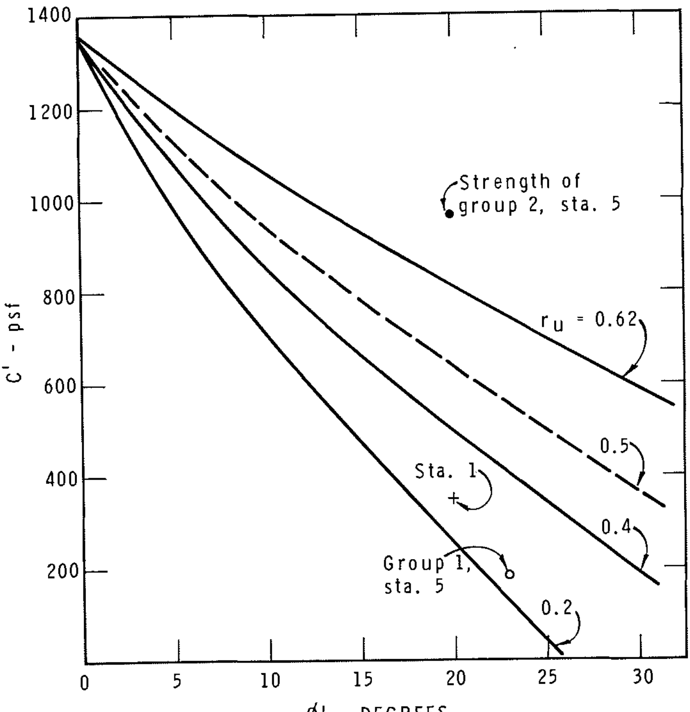

FIG. 12.—STRENGTH REQUIRED FOR STABILITY OF SLOPE 12

Three piezometers were installed as shown in Fig. 11 to check groundwater conditions. These revealed only a slight tendency for downward flow and therefore higher pore pressures than exist at Green Creek. Maximum springtime values suggest an average r_u of approximately 0.5 along the critical failure surface but it is thought that it was probably higher than this at the time of failure.

Stability Analysis.—In the Breckenridge area Slope 12 (Table 1) was considered to be the most critical slope and it is at the location of the landslide. Slope 12, shown as section A-A on Fig. 9 and redrawn in Fig. 11, was therefore subjected to a comprehensive stability analysis similar to that for slope 34 at Green Creek. The results of the stability analysis for various values of r_u and ϕ' are shown in Fig. 12 and the critical circle for r_u = 0.62 and ϕ' =

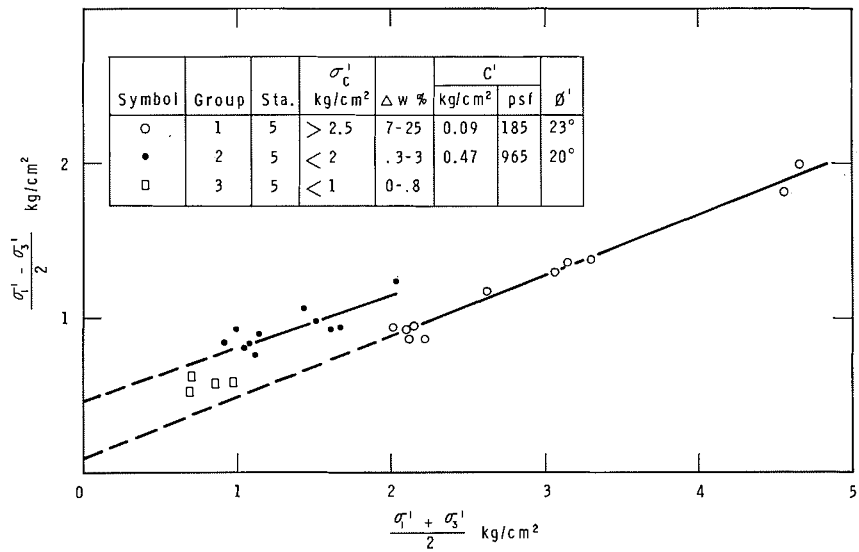

FIG. 13.—STRENGTH TEST RESULTS, BRECKENRIDGE

25° is plotted on Fig. 11. This critical circle passes through the toe of slope, but the actual failure surface, as determined by vane borings, is known to be somewhat higher. Assumed failure circles passing through the actual failure surface were therefore analyzed and found to require slightly less strength for equilibrium than the circle through the toe.

Measured Shear Strength.—Forty-two consolidated-undrained and drained tests (with decreasing lateral stress) were carried out on specimens obtained at Station 1. The average failure envelope for these tests gave c' = 0.17 kg per sq cm (350 psf) and ϕ' = 20°. Almost all of these tests were substantially consolidated in the triaxial cell and consequently the structure of the soil was disturbed.

Twenty-six triaxial tests were made on specimens obtained between El. 230 and El. 245 at Station 5 and most of them were tested at cell pressures less than the preconsolidation pressure, which is approximately 2.5 kg per sq cm (average of 12 tests) at this elevation. The maximum deviator stresses are plotted in Fig. 13. The results are tabulated in three groups. The first group was tested at consolidation pressures of 2.5 to 6 kg per sq cm and the water content was reduced from 7% to 25% depending on the pressure. These tests are not considered to have any direct practical value because of the disturbance caused by consolidation. The second group was stressed less than the preconsolidation pressure and resulted in higher strengths, $c' = 0.47$ kg per sq cm (965 psf), $\phi' = 20°$. The third group was tested with cell pressures of 0.5 to 1 kg per sq cm and at failure the lateral effective stress was as low as 0.1 kg per sq cm. One of the disadvantages of the triaxial test is that low lateral effective stresses are inevitable in tests in the low stress range and it is thought that this results in premature failure. Therefore these tests were given less emphasis in establishing the failure envelope.

TABLE 2.—SUMMARY OF TEST RESULTS AND COMPUTED SAFETY FACTORS

Slope	Sta.	Test Group	Cell Pressure	c'	ϕ'	r_u	σ'_n	FS
34	1		$>p_n$	615	19	0.62	1050	1.15
						0.3	2130	1.61
12	1		$>p_n$	350	20	0.62	1290	0.63
						0.5	1700	0.76
	5	1	$>p_n$	185	23	0.62	1290	0.56
						0.5	1700	0.76
	5	2	$<p_n$	965	20	0.62	1290	1.12
						0.5	1700	1.26

Strength Comparisons.—The shear strengths available according to tests and that required for stability are compared in Fig. 12. It will be seen that the shear strength parameters obtained by extrapolating the strength of consolidated specimens (Stations 1 and Group 1, Station 5) are quite unrealistic but that tests done at pressures lower than the preconsolidation pressure (Group 2, Station 5) give reasonable results. Assuming a pore pressure ratio of 0.62 at failure, the computed factor of safety based on Group 2 results is 1.12. For an assumed $r_u = 0.5$ the factor of safety is 1.26.

ANALYSIS OF RESULTS

A number of simplifications have been made to assess the reliability of shear strength parameters measured in the laboratory. The parameters themselves have been reduced to single values of c' and ϕ' for comparison with required values in Figs. 6 and 12. Average values for r_u and for σ'_n (effective stress normal to the slip surface) were then estimated to compute the safety factors listed in Table 2.

The safety factor for Slope 34 under present groundwater condition is estimated at 1.61 on the basis of specimens which were substantially consolidated during the test. Tests on unconsolidated specimens from the toe of the slope suggest that the safety factor may be greater. This is compensated by the possibility that a slope steeper and higher than Slope 34 existed at the location of initial failure of the old Green Creek landslide. Indications from other piezometers in the area are that the downward flow of groundwater is a general condition and this is thought to explain the apparent present stability of the Green Creek valley.

At Breckenridge the tests conducted at cell pressures higher than the preconsolidation pressure gave strengths that were too low but tests carried out closer to the working stress range were quite reasonable. It is thought that the r_u existing at the time of failure was close to 0.62 and this gives a factor of safety of 1.12. That the value exceeds unity can be partly explained by the neglect of the possibility of a tension crack full of water at the top of the bank. Other possible reasons are related to intrepretation of the shear tests and to unknown influences of progressive failure. At this time (1967) the quoted factors of safety should be considered only as approximations because of the many simplifications that have been introduced.

Although it was not possible to study the influence of strain rate on the sensitive samples described herein, it is known that strength decreases with decreasing rate of strain in the overconsolidated stress level. Based on very slow loading in the oedometer, there is reason to believe that a satisfactory minimum strength is present in the overconsolidated region. This is probably the value that should be used in a stability analysis.

Another factor that may have a substantial influence on a stability analysis in these sensitive soils is the method of test. The writers believe that failures in these soils are essentially undrained. Effective stresses on the potential failure plane may be decreased rather slowly by naturally increasing pore pressures but when the material begins to strain the breakdown in structure causes a rapid rise in pore pressure and an undrained failure. It follows that the pretreatment of samples should not cause consolidation before shearing and it is apparent from the analysis of the Breckenridge slide that shear strengths measured in the working stress range are required.

CONCLUSIONS

These investigations suggest tentatively that an effective stress analysis of natural slopes in the sensitive clays around Ottawa will yield satisfactory results. Confidence in this approach is limited, however, by the substantial lack of agreement between the critical failure circle and the actual failure surface. Because the actual surface is higher than the computed surface, it may be suggested that the frictional component is higher than the value deduced from the laboratory tests, although other factors have to be considered. For example, tension cracks and fissures, known to exist in the upper layers of soil, would have a greater influence on the strength mobilized along a shallow surface. Also, the pore pressures existing above the toe of slope at the time of failure at Breckenridge were probably high, possibly high enough to initiate a progressive failure. The influence of the present downward gradient in the pore water at Green Creek, however, has a stabilizing effect more than enough to compensate for seasonal pore pressures at the toe. The influence of groundwater conditions is clearly important and failure to measure these conditions adequately may lead to incorrect assessments of stability.

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5324 STABILITY OF NATURAL SLOPES IN SENSITIVE CLAY

KEY WORDS: soil mechanics; slopes stability; shear strength, sensitivity clay (material); slides

ABSTRACT: Failures of natural slopes in sensitive clays often retrogress leaving a characteristic crater which is wider than the opening in the slope. Two actual failures of this type that occurred near Ottawa, Canada are described and compared with stable slopes in the same general area. One of the failures occurred several hundred years ago but changes in ground water conditions since failure appear to have increased the stability of adjacent slopes in that area. The second failure, in an area of active sliding, occurred in 1963. In each case the required strength for stability under various ground water conditions has been computed and compared with measured values. It is concluded that an effective stress analysis of these slopes yields satisfactory results if tests are conducted in the working stress range and ground water conditions are carefully measured.

REFERENCE: Crawford, Carl B., and Eden, W. J., "Stability of Natural Slopes in Sensitive Clay," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM4, Proc. Paper 5324, July, 1967, pp. 419-436.