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THE MECHANICS OF LANDSLIDES IN LEDA CLAY

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The mechanics of landslides in Leda clay[1]

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An appraisal of the drained shear strength of Leda clay under low effective normal stresses has resulted in a new appreciation of its behavior in this stress range which can be applied to consideration of the stability of slopes. Closely spaced planes of weakness existing in the apparently intact clay give rise to dilatant behavior and predominantly frictional shearing resistance. This mode of failure is consistent with field observations that have been compiled from numerous landslides; three of these landslides are analyzed in this paper.

Des mesures de la résistance au cisaillement drainé de l'argile Léda sous de faibles valeurs de la contrainte normale effective a conduit à une nouvelle évaluation du comportement de cette argile dans le domaine des contraintes applicables aux problèmes de stabilité des pentes. Des plans de faiblesse très rapprochés les uns des autres existent dans cette argile apparemment intacte et résultent en un comportement de l'argile caractérisé par la dilatation durant le cisaillement et par une résistance au cisaillement due principalement au frottement. Ce mode de rupture est consistant avec les observations faites sur le terrain et compilées pour de nombreux glissements; trois glissements sont analysés dans cet article.

Landslides are a common feature along the slopes of stream valleys and terraces in areas of Leda clay. The slides appear to be rotational slips which often retrogress a considerable distance into the slope and in some instances become large flowslides. During the past 10 years a number of slope failures in the Ottawa area have been investigated and certain common features have emerged which provide the background for this paper. Some information on landslides at Breckenridge, Rockcliffe, and Orleans are summarized in Table I and these three slope failures are discussed in the paper.

All the slope failures investigated have occurred at exceptionally wet times indicating that an ample water supply is a necessary condition for the initiation of a slope failure. Failures have occurred only in slopes with inclinations greater than 2.5:1 but the height of these slopes varied from 10 to 30 m. Many small shallow slides have been observed to occur and it is believed that the larger slides are initiated by a shallow slide.

The weathered crust on Leda clays is of variable thickness and may extend to a depth of 10 m. It is identified by decreasing field vane strength with increasing depth. The nature of the crust is similar in many respects to the crust on Norwegian clays (Bjerrum *et al.* 1969). The crust consists of two layers: an upper oxidized layer which extends approximately to the depth of seasonal variation of the groundwater table, and a lower layer of gray clay. The upper portion of the crust is strongly fissured and may allow water to move freely through the clay.

Leda clay is considered to be strongly bonded (Crawford 1963; Townsend *et al.* 1969) and the sensitivity generally increases with depth. Physical examination of clay samples indicates that both the weathered clay and the lower clay strata have a closely spaced system of defects that allow strain discontinuities to develop when the sample is distorted. This feature is illustrated in Fig. 1 which shows a piece of clay from a depth of 7 m initially broken into two pieces then fractured into small blocks or 'nodules' by a shearing action. The small nodules are generally prismatic in shape and have well defined planar sides. Similar fractures have been observed in samples taken from depths exceeding 20 m. Individual nodules may be remoulded to a quasi-liquid state. Thus the clay can be considered as being made up of a system of small brittle nodules separated by micro fissures or planes of weakness.

Confidence in the conventional upper bound solution to slope stability problems can only be achieved if all the laboratory and field observations are consistent with the predictions. The depth of the predicted critical circle de-

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TABLE I
Summary of landslides discussed

Location of landslide	Breckenridge Creek, Quebec	Rockcliffe on Ottawa River	Orleans Ontario
Upper terrace elevation	330 ft	180 ft	240 ft
Description of slope	Prominent natural slope	Uniform natural slope	Side hill cut
Height of slope	90 ft	40 ft	30–35 ft
Average inclination	25°	24°	35°
Description of slide	Retrogressive flow slide	Retrogressive slide	Shallow rotational slip
Date of slide	April 1963	April 1967	October 1965
Volume of clay involved	30000 yd^3	30000 yd^3	2000 yd^3
Soil conditions	Uniform Leda clay underlying 4 ft of silt	Stiff gray Leda clay, fairly uniform	Leda clay with silty layers
Conditions contributing to instability	Melting snow heavy rainfall possible toe erosion	Melting snow heavy rainfall toe erosion	Heavy rainfall no toe erosion

pends on the strength parameters (c', Φ') assigned to the soil, and the value of the average normal stress calculated for the failure arc depends, primarily, on the depth of this critical circle. If the strength parameters of the soil vary with the normal stress at failure, the most critical situation may be overlooked by not covering the entire range of possible normal stresses in laboratory tests. Field evidence indicates rather shallow failure surfaces which would suggest that Leda clay should exhibit a high frictional strength component (Φ') when sheared to failure under low normal pressures.

Triaxial Compression Tests

The possibility of shear failure developing under low values of mean effective stress has recently been investigated by carrying out drained triaxial compression tests during which the cell pressure was reduced as the axial load was increased. Tests were carried out with a constant rate of stress change (average test duration approximately one day), with small incremental stress changes (average test duration about one week), and with strain controlled loading (average test duration about 8 h). Test specimens from the sites of the three landslides discussed in this paper (Table I) exhibited similar shear behavior and similar failure conditions. Since the tests on material from the Rockcliffe landslide site are the most extensive, these results will be discussed in some detail throughout the paper with reference to the mechanics of failure. The failure points for test specimens from the two other sites will be presented with reference to the landslide analysis.

Most of the specimens from the Rockcliffe site were tested on a stress path defined by $p = (\sigma_1' + \sigma_2' + \sigma_3')/3$ = constant. The 'constant p' tests eliminate volume changes due to changes in the mean normal stress so that the volume changes due to shear (increases in deviatoric stress) can be measured. In addition to the conventional vertical orientation, several specimens were oriented horizontally and at 45° for shear testing. Two sizes of test specimens (nominal cross-sectional areas of 10 cm^2 and 40 cm^2) were used in order to investigate possible size effects.

All tests were carried out under a back pressure and filter paper strips were used to facilitate draining. The triaxial cells were equipped with rotating bushings to reduce piston friction.

Stress–strain curves from several of the constant p tests are plotted in Figs. 2 and 3. Figure 2 shows a comparison between the behavior of vertical and horizontal specimens. Figure 3 shows the behavior of large diameter

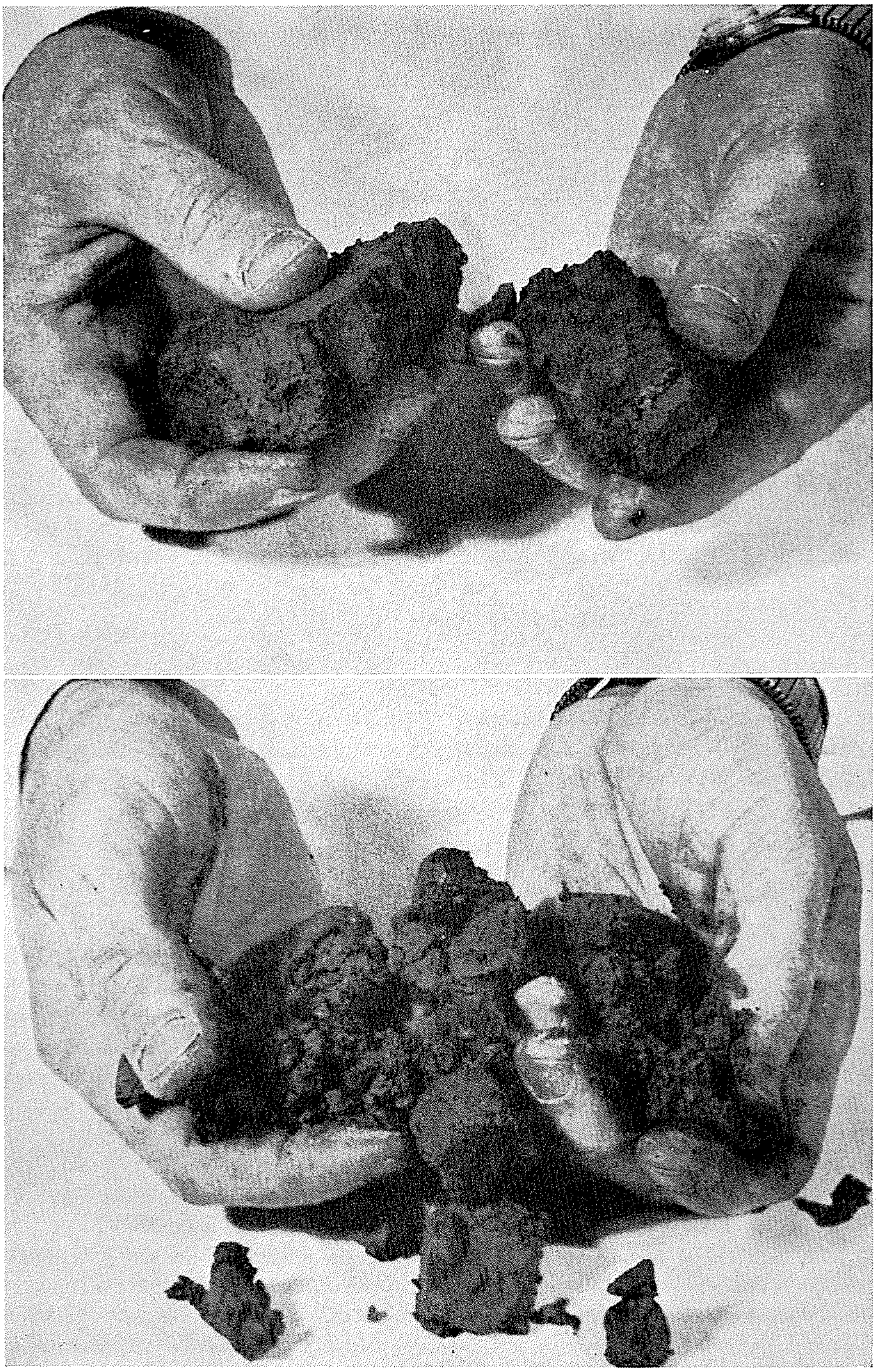

FIG. 1. Nodular composition of Leda clay.

specimens and the behavior of specimens in strain controlled tests. All the specimens tested developed a shear plane at (or near) the maximum deviatoric stress with these planes intersecting the physical horizontal direction at angles between 45 and 63°. The angle of shear

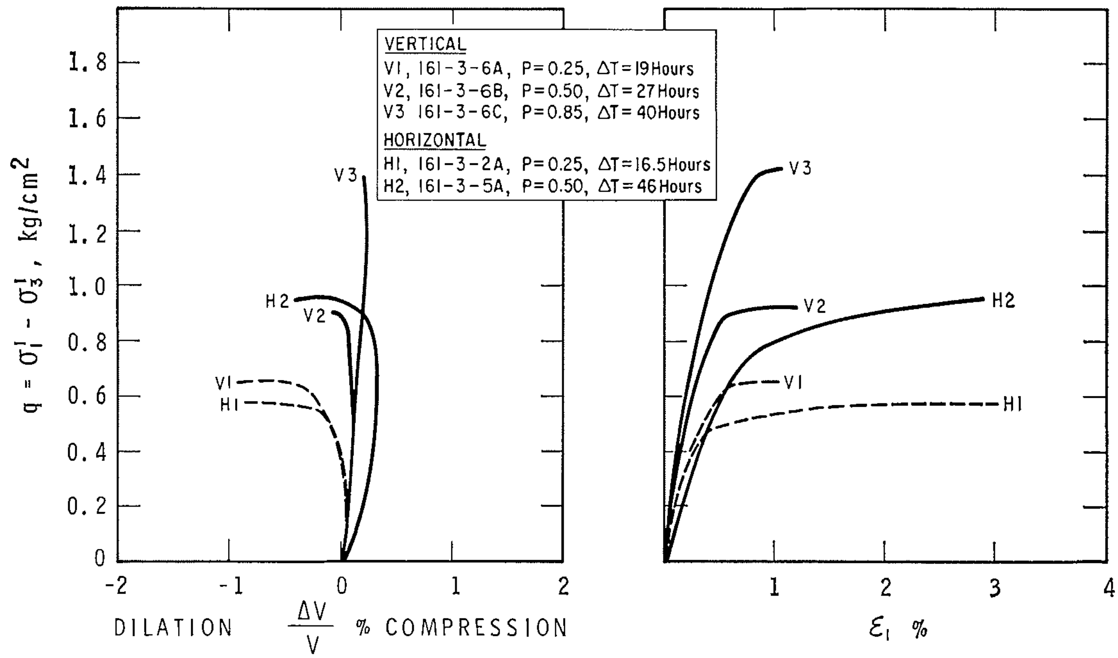

FIG. 2. Shear behavior of Leda clay under low confining pressures.

decreased as the mean normal stress at failure increased. Failure points (maximum deviatoric stress) for all tests are plotted in Fig. 4 with respect to the commonly used plane stress parameters

[1] average stress, $s = (\sigma_1' + \sigma_3')/2$

[2] maximum shearing stress, $t = (\sigma_1' - \sigma_3')/2$

From these test data the following observations were made regarding the behavior of the clay under low values of average stress:

(1) Maximum shearing resistance is reached at strains of the order of 1%.

(2) Significant dilation (increase in volume) occurs prior to and during failure. Rates of dilation near failure were great enough that changes in the level of the water in the burette could be observed visually. The general dilatancy apparently increased the bulk permeability of the specimen by several orders of magnitude.

(3) The failure envelope is approximated, in terms of conventional planar shear parameters, by a low value of c' and a high value of Φ'.

(4) In the low stress range, the failure envelope is reasonably independent of specimen orientation, stress path, and rates of testing. Test specimens of 10 cm^2 nominal area appear to be large enough to eliminate any significant size effect. At higher stress levels, such as shown on the horizontal portion of the envelope in Fig. 4 orientation and rates of testing might have an influence.

(5) The post-peak decrease in shearing resistance is about 10% of the peak shearing resistance at a distortional strain of 5%.

(6) The failure envelope does not appear to change with the depth of samples (below the oxidized crust). For slope stability analysis the samples should be obtained, however, from a depth corresponding to the middle portion of the most probable failure surface.

The strength of Leda clay in the low stress region appears to be predominantly frictional, i.e. dependent on normal stress. Dilatant behavior is attributed to the existence of structural defects (closely spaced hairline fractures or 'closed fissures') which allow strain discontinuities to occur within the material. Features illustrated in Fig. 1 were observed in the test specimens after failure.

Failure points for specimens from the Breckenridge and Orleans landslide sites are plotted in Fig. 5. These points are approximated by linear failure envelopes for use in the slope

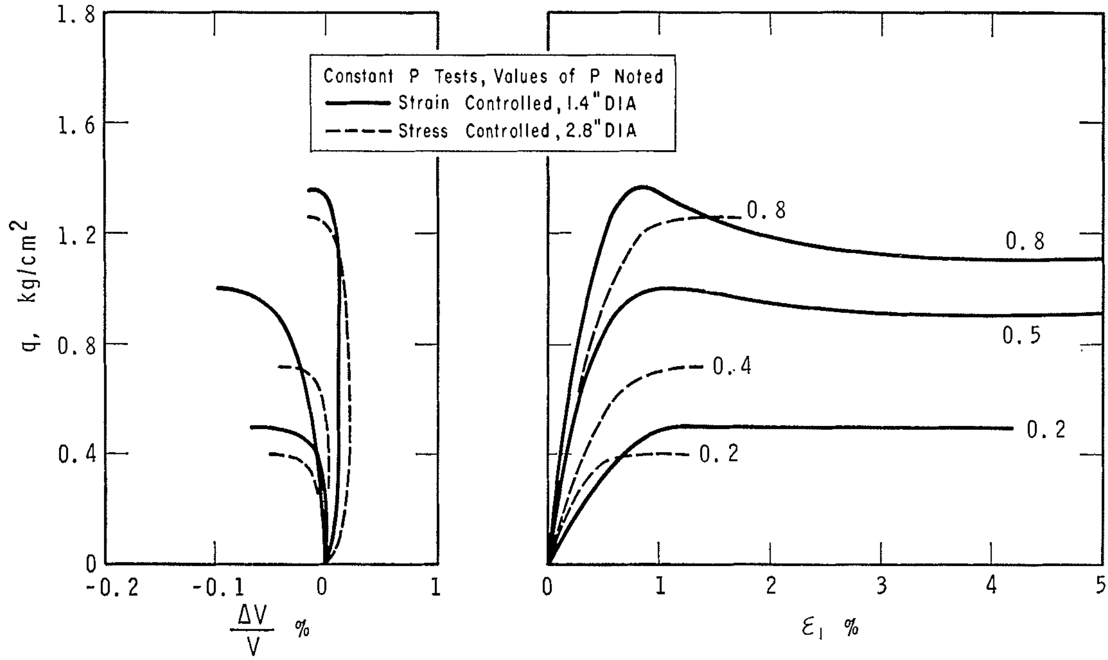

FIG. 3. Additional triaxial test data.

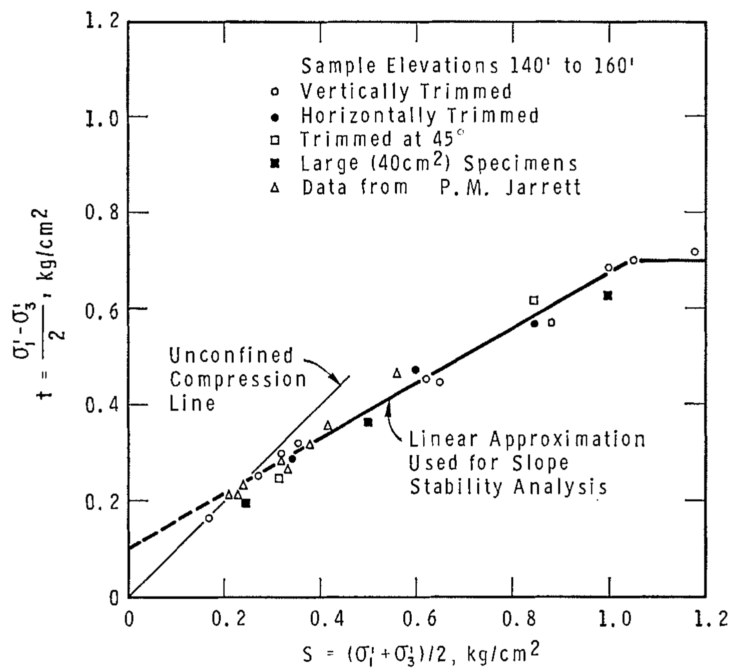

FIG. 4. Strength of Leda clay from the Rockcliffe site.

stability analysis. Failure envelopes obtained for Leda clay from other sites near Ottawa are also drawn to show that the low stress frictional behavior occurs extensively and is not dependent on the elevation of the clay terrace.

Not all Leda clays exhibit the closely spaced

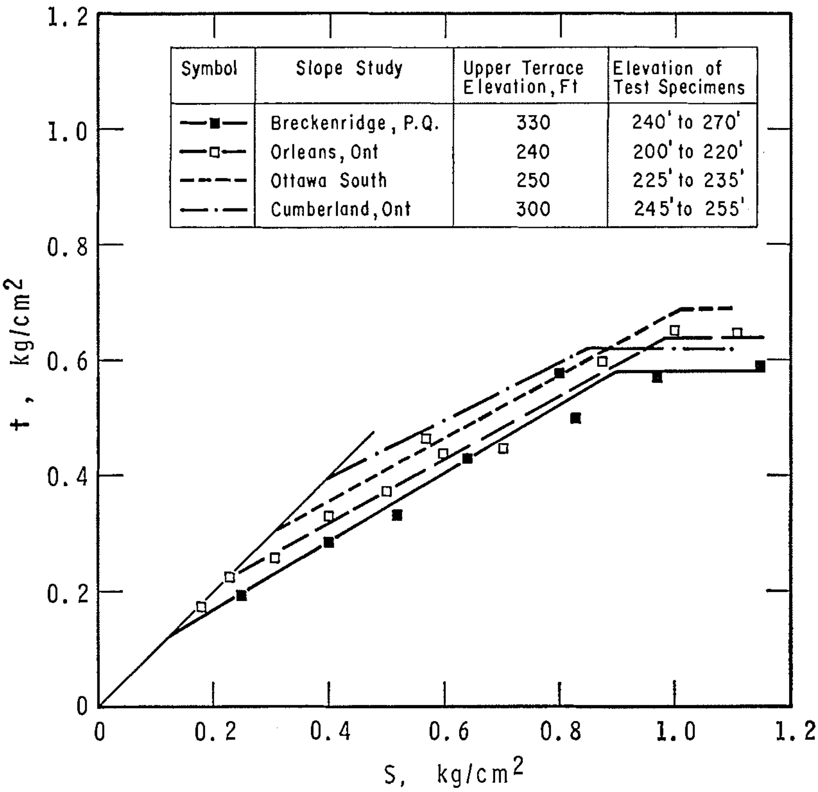

FIG. 5. Failure envelopes for Leda clay from various sites.

fissures and the very stress dependent strength behavior described above. Samples from the Green Creek valley east of the City of Ottawa and from the Outardes II site in Quebec behaved as an intact stiff clay in the low stress range. With the triaxial test, the minimum strength obtained was that indicated by the intersection of the nearly horizontal portion of the failure envelope with the unconfined compression line shown in Fig. 4. These clays exhibited high cohesion with the strength at low stress levels being due primarily to cementation bonds (Townsend *et al.* 1969). Failures took place on well defined planes such as shown in Crawford 1963, indicating brittle, well-bonded material.

Landslides at Breckenridge, Rockcliffe, and Orleans

The main features of the landslides to be discussed in this section have been tabulated in Table I. The landslide at Breckenridge has been described in some detail by Crawford and Eden (1967). Detailed descriptions including site plans, piezometric levels, sample and test records, of the Orleans and Rockcliffe landslides are contained in Eden and Jarrett (1970) and Mitchell (1970) respectively.

All three landslides occurred at periods when the ground water table was high. The actual failure took place during heavy rainfalls so that the analyses were conducted initially by assuming the slopes to be fully saturated. The stability analyses were carried out using an I.B.M. System 360 computer following Arseneault's (1967) program for Bishop's stability equation. This program requires specification of one point on the failure surface and an approximation of the ground water table by an average value of r_u where r_u for each slice is defined by Bishop and Morgenstern (1960) as $r_u = (\mu/\gamma h)$ where μ = pore water pressure at base of slice

h = height of the slice

γ = wet unit weight of the soil.

For the clays at the Breckenridge and Rockcliffe sites, the wet unit weight is close to

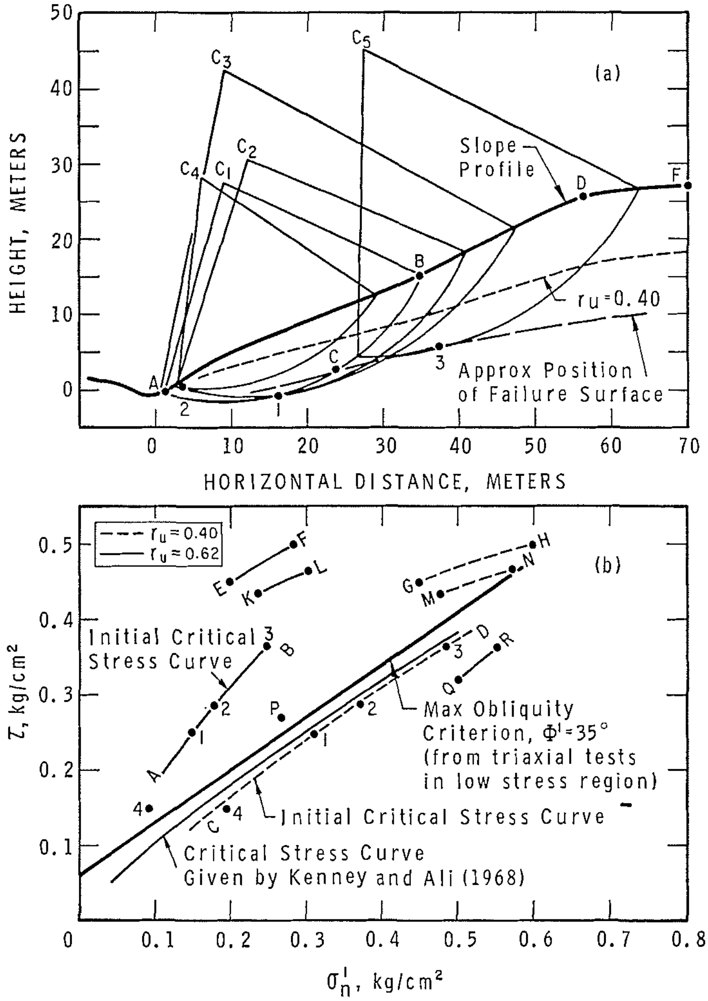

FIG. 6. Analysis of the Breckenridge landslide. (*a*) trial circles; (*b*) critical stress curves.

1.6 g/cm³; an r_u of 0.62 corresponds to full saturation of the slope under hydrostatic ground water conditions.

A convenient method of presenting a slope stability analysis is to calculate 'critical stress curves' (Kenney 1967) for given slope conditions. Various combinations of c' and Φ' that yield a factor of safety of unity for a particular slip circle will define an average stress point, $(\bar{\tau}, \bar{\sigma}_n)$, in (τ, σ'_n) space. A critical stress curve forms the upper limit of all average stress points and passes through the average stress points of critical circles. A critical stress curve represents a possible field failure envelope for the specified slope conditions and may only be compared directly with laboratory tests carried out in the appropriate range of normal stresses. Critical stress curves and average stress points are calculated for various values of r_u in the following analysis.

Breckenridge Slide

Because of the geometry of the Breckenridge slope it is believed that the initial slide did not involve the total slope height (Kenney and Ali 1968). In Fig. 6*a* various possible circular slip surfaces for the Breckenridge slope are drawn with centers c_1 to c_4 and passing through points 1 and 2 in the vicinity of the failure surface. Stability analyses based on these failure surfaces and two values of r_u yield the critical stress curves AB and CD in Fig. 6*b*. The critical stress curves can be compared directly with the maximum obliquity criterion (c', Φ') obtained from the triaxial test data using the relationships: $c' = d_0 \tan \Phi'/\tan \Psi$ and $\Phi' = \arcsin(\tan \Psi)$ where d_0 is the ordinate intercept of the failure line in (t, s) space and Ψ is the slope of this line. This comparison suggests that a small initial slide would occur if the groundwater table were high enough to give an average r_u factor somewhat greater than 0.4.

A variable r_u factor (varying from 0.50 at the top of the slope to 0.62 at the toe to approximate the conditions considered applicable at the time of the slide) was used in a modified computer program to obtain a critical toe circle for $\Phi' = 35°$. This circle lies between circles c_1 and c_2 in Fig. 6*a* and gives a factor of safety, F.S. = 0.95 (point *P* in Fig. 6*b*, where

$$\text{F.S.} = \frac{\text{allowable shear stress, } \tau_{\text{all}} = c' + \sigma_n \tan \Phi'}{\text{average shear stress, } \bar{\tau}\text{, computed for the critical circle}}$$

the trial circle numbers). A failure envelope ($c' = 0.12$ kg/cm^2, $\Phi' = 33°$) was chosen to approximate the triaxial test data in Fig. 4. The circles with centers at c_3, c_4, and c_7 in Fig. 7*a* yield stress points 3, 4, and 7 which lie on this failure line. As the circle with center c_3 was calculated to be critical for $\Phi' = 33°$ the analysis indicates that the slope would be at a condition of limiting equilibrium when fully saturated under a hydrostatic groundwater condition.

Assuming that the initial slide occurred along the arc given by circle c_1 in Fig. 6*a* the first retrogressive slide was analyzed using the geometry ACBDF. The computer was restricted to searching for critical circles through point 3 (Fig. 6*a*) on the failure surface and yielded the critical stress curves EF and GH in Fig. 6*b*. A second retrogressive slide was analyzed in a similar manner by approximating the geometry of the slope after the most critical circle (center c_5 in Fig. 6*a*) was assumed to slide during the first retrogressive failure. This analysis yielded the critical stress curves KL and MN in Fig. 6*b*. These critical stress curves all lie above the failure criterion in Fig. 6*b* indicating that retrogressive sliding would occur at average r_u values of 0.4. These calculations make no allowance, however, for the stabilizing effect of that spoil material which comes to rest in the landslide crater.

Finally, the existing stable back scarp was analyzed with respect to circles tangential to the projected failure surface. These computations yielded the critical stress curve QR in Fig. 6*b* and predicted the stability of the scarp under the extreme conditions of full hydrostatic saturation.

Rockcliffe Slide

The landslide at Rockcliffe (Mitchell 1970) was the largest of several that occurred along the south bank of the Ottawa River near Ottawa during the unusually wet spring of 1967. The level clay terrace supporting the Rockcliffe Air Base drains toward the Ottawa River; field observations after the slide indicated that the groundwater table was quite near the ground surface at the time of the slide.

Average stress points are plotted in Fig. 7*b* for several trial circles shown in Fig. 7*a* (the stress points are numbered to correspond to the trial circle numbers).

Personnel working in the vicinity of the slide claimed that a hut located on the upper terrace (Fig. 7*a*) was missing immediately after they heard the noise caused by the initial slide. Stability analysis suggests that the initial failure surface would intersect the upper terrace quite close to the hut. Some approximate calculations regarding the retrogressive slips indicate the gross instability of the slope remaining after the initial slide. The stability of the post failure ground surface (Fig. 7*a*) was considered with respect to trial circles tangential to the projection of the failure plane and with respect to trial circles emerging at the toe of the back scarp. In both cases a safety factor in excess of unity was obtained.

Orleans Slide

The landslide at Orleans (Eden and Jarrett 1970) occurred in a road cut 5 years after construction. Groundwater studies conducted after the slide indicate that the average r_u factor would have exceeded 0.4 at the time of the slide. Full saturation of the Orleans slope corresponded to an r_u factor of 0.59. A stability analysis has been carried out for these limits of r_u.

Three critical circles, as shown in Fig. 8*a*, were analyzed and gave the average stress

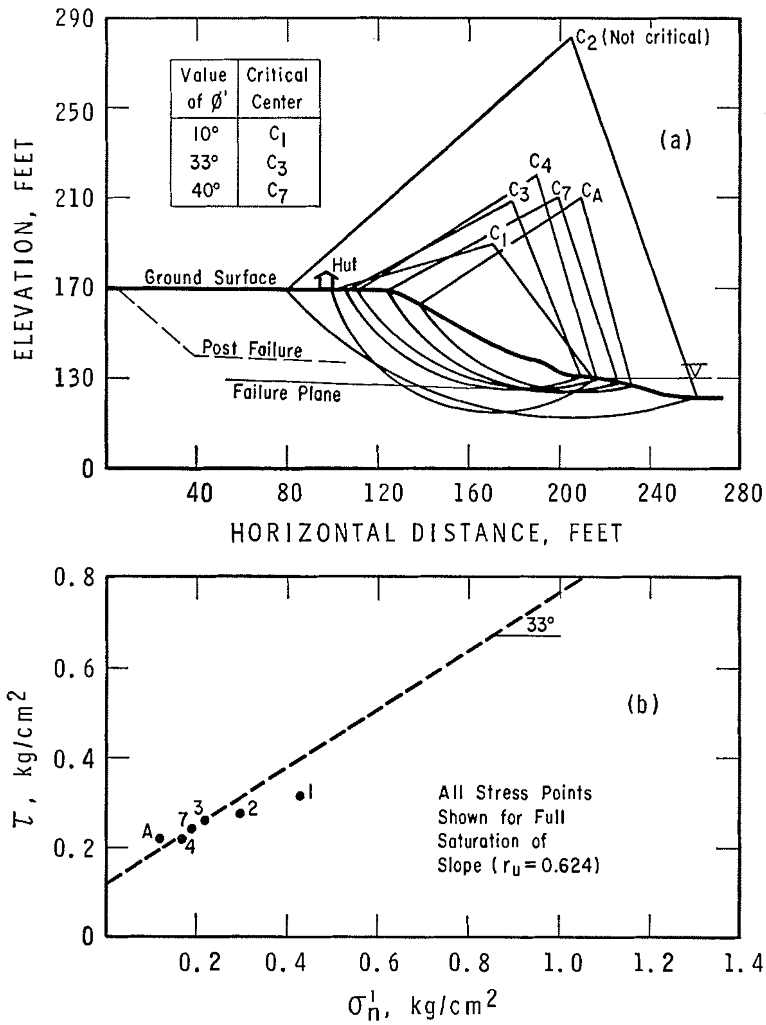

FIG. 7. Analysis of the Rockcliffe landslide. (*a*) critical circles; (*b*) critical stress points.

points 1, 2, and 3 plotted in Fig. 8*b* for the two limits of r_u. A comparison of these stress points with the failure criterion indicates that failure would not occur on the deeper circles (calculated critical for $\Phi' < 20°$). For the condition of full saturation instability is predicted for circles with centers c_1 and c_2. The factor of safety calculated for circle c_1 is 0.84. For an average r_u value of 0.40 the slope would be considered stable with respect to these circles.

A closed solution was developed for the stability analysis using the following additional assumptions: the circle passes through the toe of the slope, intersects the upper terrace at an angle of $45 + \Phi'/2$, and has its center located vertically over the toe. This circle is shown, for $\Phi' = 33°$, in Fig. 8*a* and appears to approximate the observed failure surface reasonably well. The closed solution gives the average stress point A in Fig. 8*b* for the condition of full saturation. The factor of safety predicted by the latter calculation is 1.05. A similar analysis of the profile ABCDE in Fig. 8*a* gave a factor of safety of 1.10 for the slope remaining after the slide.

Effect of Ground Water Level

The analysis of these landslides with reference to critical stress curves (or average stress points for critical circles) emphasizes the critical role of the groundwater conditions in slope instabilities in Leda clay. A special triaxial test

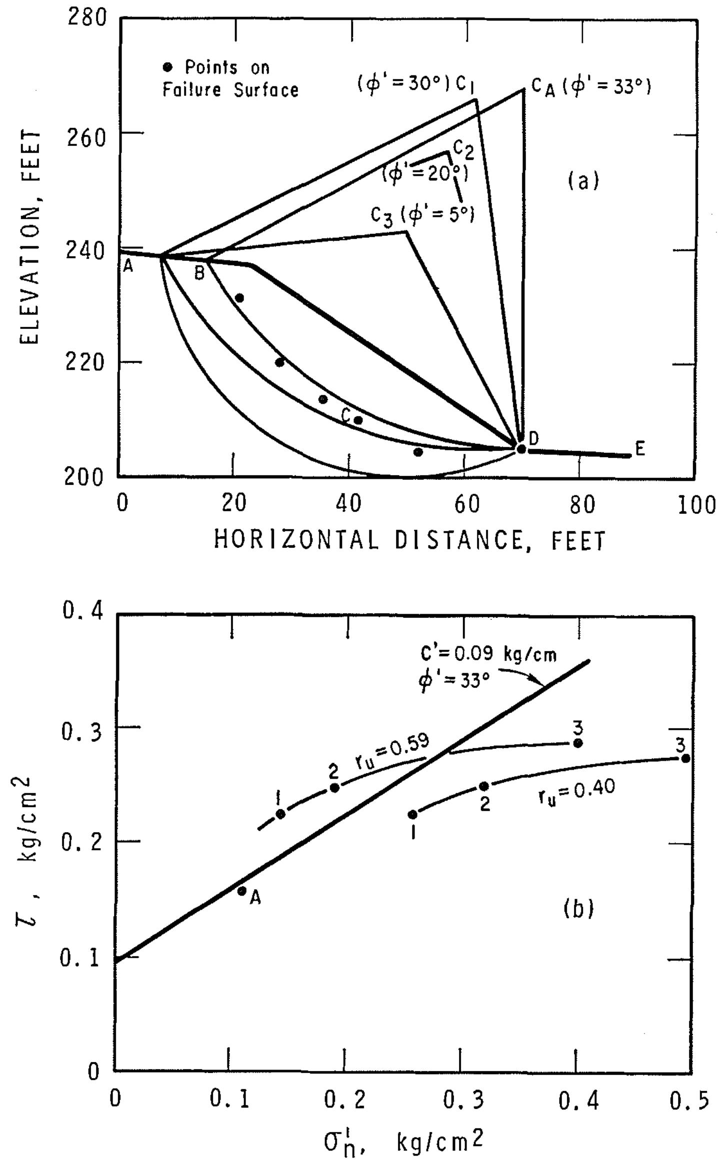

FIG. 8. Analysis of the Orleans landslide. (*a*) critical toe circles; (*b*) critical stress curves.

was carried out where the specimen was loaded in increments (drained) to about 85% of its failure load (stresses of $\sigma'_1 = 1.0$ kg/cm^2 and $\sigma'_3 \simeq 0.25$ kg/cm^2). It was found that the deformation of the specimen could be controlled at this stress level by the pore water pressure. A slight increase in the back pressure (about 0.5 m head of water or 0.05 kg/cm^2) would result in axial deformation readily observable on the dial gauge. The deformation could be arrested by lowering the back pressure to its prior value. This behavior suggests that a temporary rise in groundwater pressures may cause slope deformation without complete collapse. Extended periods of high groundwater pressures may result in sufficient strain to develop tension cracks near the top of the slope. Complete failure may depend on the availability of surface water entering via these tension cracks to satisfy the dilatant tendency and prevent suction pressures in the failure zones. In view of these considerations it appears that the assump-

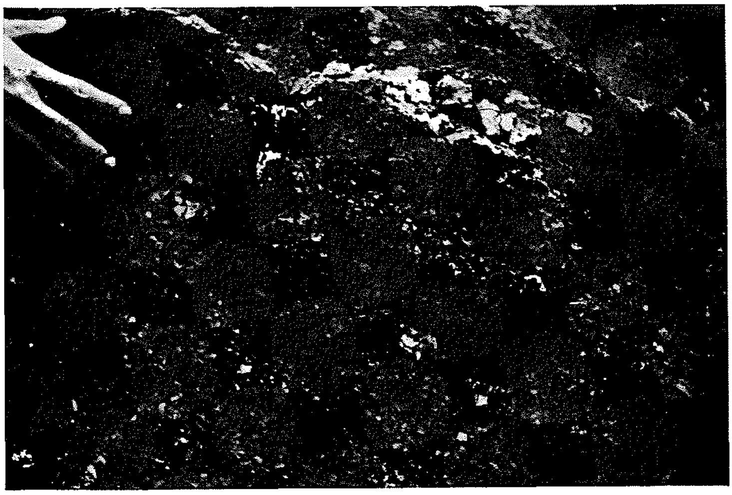

FIG. 9. Nodular composition of landslide debris.

tion of full saturation may not be overly conservative in the analysis of the stability of natural slopes in Leda clays. The need for accurate records of transient groundwater pressures has led to the recent installation of several electronic transducer piezometers in the Ottawa area.

Multiple Failure Surfaces

The failure points plotted in Fig. 4 indicate a curved failure condition. In isolation those points contributed by P. M. Jarrett[2] give a planar strength criterion $c' = 0.08$ kg/cm^2, $\Phi' = 45°$ as compared to the values $c' = 0.12$ kg/cm^2, $\Phi' = 33°$ obtained by the authors. When used in the stability analysis both sets of strength parameters yield a safety factor close to unity. This somewhat surprising result occurs because these strength parameters define two planar failure criteria that intersect close to the value of the average normal stress on the critical circles. The absolute values of the strength parameters appear to be less important than ascertaining that the strength tests and stability analysis involve comparable normal stresses.

The curved shape of the failure condition is associated with the variation in dilatant tendencies, dependent primarily on the value of the mean normal stress. This failure condition is similar in shape to the critical stress curve and indicates that, although one slip circle may be theoretically critical, many slip surfaces may exist simultaneously at a nearly critical equilibrium. Thus an extensive zone of soil may be deforming at the time of failure. An extensive zone of deformation is indicated by the multiple tension cracks and 'stepped' ground surface profile observed at the back scarp of several landslides. Extensive deformation (and dilation) together with multiple slippage could reasonably be associated with a flow slide. Much of the 'flow' material would be transported as a flexible nodular blanket riding on a slippery surface of remoulded clay. This nodular composition was noted in the debris of several landslides which were investigated by the authors. Figure 9 shows this feature in the debris of a flow slide that occurred near Louiseville, Quebec, in April 1969.

Conclusions and Discussion

The following conclusions are drawn from the information presented in this paper:

(1) Leda clay may exhibit a dilatant behavior and a predominantly frictional shearing resistance when tested at low values of average stress.

(2) The frictional strength parameters are supported by field observations regarding the location of failure surfaces and the critical role of groundwater in slope instabilities.

(3) Strengths derived from drained triaxial tests are in good agreement with strengths derived from the analysis of slope failures.

(4) Dilatant behavior is attributed to the

[2]P. M. Jarrett (1969), private communication of results from triaxial tests carried out at the University of Glasgow, Scotland.

development of discontinuities on pre-existing planes of weakness in the clay and results in the clay shearing into small (1 mm to 1 cm) blocks. Triaxial specimens of 10 cm^2 cross-sectional area appear to be representative of the mass behavior. This behavior may give rise to multiple failure surfaces during slope instabilities.

Beyond the association of slope failures with abundant ground and surface water, the detailed geological and climatic conditions contributing to these instabilities are not yet fully understood. The spring seasons of 1967 and 1969 were both accompanied by numerous instabilities along the south bank of the Ottawa River. Both these periods were preceded by winter seasons during which early and continuous snow cover prevented deep frost penetration. This factor may contribute to early infiltration of melting snow cover and spring rains (with less surface runoff) and high groundwater tables.

Perhaps the most important aspect yet unexplained is the development of the closely spaced hairline fractures in the clay. Large fissures, apparent in the oxidized crust, are attributed to stress relief and shrinkage of the clay due to loss of moisture. Similar fissures may appear in clay exposed by erosion or excavation. Microfissuring may occur as a result of stress relief due to overburden removal and slope cutting. The formation of microfissures by this mechanism may depend primarily on the relationship between the swelling pressures and the strength of the 'cementation bonds' in the clay. A second possible cause of microfissuring may be the fatiguing effect of seasonal changes in temperature and/or groundwater pressures on the 'cemented' clay. These factors can only be resolved by detailed studies of the physical and chemical properties of the clays.

Recognition of the microfissured nature of some Leda clays and their behavior under shear strains has been instrumental in understanding the mechanics of landslides. The earlier view that Leda clay slopes failed in a brittle manner, with the clay being completely remoulded into a quasi-liquid state during flow slides, has been largely discredited in favor of the mechanistic picture which is developed in this paper. An understanding of the causes and distribution of microfissures may lead to a future recognition of the conditions essential for a flow type of slide.

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