Chapter 24

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SOFT SENSITIVE CLAYS

1. INTRODUCTION

Sensitivity is defined as the ratio of the undisturbed to the remolded shear strength of a soil and thus expresses the loss of strength when a soil is remolded. A variety of clay deposits may exhibit sensitivity, but some marine clay deposits exhibit very high sensitivity and very low remolded shear strength. Such deposits are termed *quick clays*, which is a direct translation of the term *kvikkleira* used by Norwegian investigators to describe these deposits, common in their country. The Norwegians define quick clays as those soils exhibiting a sensitivity greater than 30 and having a remolded shear strength less than 0.5 kPa (Norsk Geoteknisk Forening 1974). This definition requires the remolded soil to behave like a fluid. It should be noted that this definition of quick clay considers soil behavior only, ignoring geographic location, depositional environment, or any other factors related to origin or material.

In nonsensitive soils, landslide masses generally come to rest at the toe of the slope and act like a stabilizing mass in the new slope geometry. For sensitive clays, however, the remolding involved in the mass movement results in a drastic reduction in shear resistance, causing the remolded clay to behave like a thick liquid so that the slide mass moves away and leaves the new slope unsupported. A new instability may result and a series of retrogressive failures will be triggered that can extend far beyond the crest of the initial slope. In highly sensitive clays, therefore, it is not just the risk of a slope failure that is of concern, but also the area that can be affected by retrogressive sliding.

Their high sensitivity combined with the fluidity of the remolded materials makes quick clays very susceptible to retrogressive landsliding. Rapid and dramatic geotechnical failures are the result (Figure 24-1). This type of retrogressive failure has been referred to as a flow slide or earth flow. The latter term is preferred, since it is in line with current international landslide terminology (see Cruden and Varnes, Chap. 3 in this report). Several cases have been documented in which such retrogressive failures affected areas larger than 20 ha and extended more than 500 m beyond the crest of the initial slopes (Eden and Jarrett 1971; Eden et al. 1971; Tavenas et al. 1971; Brooks et al. 1994; Evans and Brooks 1994).

At present, the risk and extent of retrogression are difficult to evaluate. However, on the basis of studies conducted in eastern Canada, retrogression may be considered to be a consequence of an initial slope failure. In terms of analysis, the first step is to quantitatively examine the initial slope stability. The second step is to evaluate, in a semiquantitative manner, the risk of retrogression that could develop following initial slope failure. In addition to numerical analysis, a certain comprehension of the formation of slopes in sensitive clay deposits is helpful in order to put the processes into a rational framework.

FIGURE 24-1 Aerial view of Lemieux landslide, Ontario, Canada, late morning, Wednesday, June 23, 1993, involving failure of 2.5 to 3.5 million m^3, much of which flowed into South Nation valley causing impoundment of river *(foreground)*. Failure took place within 1 hour on June 20, temporarily damming river with debris. Approximate maximum height of river impoundment is shown during overtopping of dam. Ground surface within failure is about 12 m below original ground surface and slopes gently toward river. Debris flowed about 1.6 km upstream and 1.7 km downstream of landslide (Brooks et al. 1994; Evans and Brooks 1994).

PHOTOGRAPH 1993-254F COURTESY OF GEOLOGICAL SURVEY OF CANADA

2. DESCRIPTION OF SENSITIVE CLAY DEPOSITS

The existence of sensitive clay deposits is related to

1. Mineral sources and depositional conditions responsible for an open soil fabric and a relatively high water content and
2. Geochemical factors responsible for a reduction of the liquid limit and the remolded shear strength.

These factors naturally restrict the geographic distribution and geologic characteristics (including composition, stratigraphy, groundwater geochemistry, and groundwater volumes) of soft sensitive clay deposits.

2.1 Geographic Distribution

Although soft clay deposits are found in many parts of the world, the most extensive areas of highly sensitive clays occur in Scandinavia and eastern Canada. They have also been reported in Alaska, Japan, the former Soviet Union, and New Zealand (Torrance 1987). Small areas of similar deposits may still be discovered, but it appears unlikely that any large areas of such deposits remain undiscovered.

2.2 Geologic Origin

Sensitive clay deposits are generally young marine clays deposited in preglacial and postglacial bodies of water that existed during the retreat of the last Wisconsin ice sheet between 18,000 and 6,000 years before the present (BP). Many of the topographic depressions left by the glaciers formed freshwater lakes, but some were connected to the oceans at some stage.

The weight of the continental ice sheets caused depression of the land surface up to several hundred meters. Isostatic rebounding of the land surfaces occurred after the ice retreated, but this is a relatively slow process that is still continuing in some regions. The enormous amounts of water incorporated into the continental ice sheets caused the ocean volumes to be somewhat less than at present, and the ocean levels dropped worldwide by at least 120 m (Kenney 1964). However, the rate of ice-sheet melting and the consequent rise in sea level were much more rapid than the rebounding of the land surface following ice-sheet removal.

Isostatic rebound and eustatic sea-level changes after the last deglaciation have been well documented (Kenney 1964; Andrews 1972; Hillaire-Marcel and Fairbridge 1978; Quigley 1980). The isostatic rebound was much larger than the increase in sea level. The net result of the interaction between changing land elevations and sea levels was a period of inundation and subsequent reemergence of certain portions of the continental land masses in the time between the melting of the continental ice sheets and the present day.

Sensitive clays are considered to have been deposited in marine, or at least brackish, bodies of water. In North America the last glaciation covered most of Canada, parts of the conterminous northern United States, and part of Alaska. The largest deposits of postglacial marine clays were formed in the Champlain Sea, which occupied the St. Lawrence lowlands from the Gulf of St. Lawrence to the region around the city of Ottawa during the period approximately 12,500 to 10,000 years BP (Elson 1969; Gadd 1975; Hillaire-Marcel 1979).

2.3 Groundwater Geochemistry

The emergence of these postglacial marine clay formations above sea level altered their groundwater regimes, and because of hydraulic or concentration gradients, their pore-water salt concentration, which was initially as high as 32 g/L, was reduced. Present-day pore-water salinity in many sensitive clay deposits has been reduced to values below 1 g/L. Reduction of pore-water cations (salinity) is known to reduce the liquid limits (Rosenqvist 1966) and the remolded shear strength of clays.

Because of an open fabric and high water content retained since deposition, some lacustrine clays and unleached marine clays can also be fairly sensitive. The soft Barlow-Ojibway lacustrine clay in northern Québec Province, for example, has a water content of as much as 100 percent, a plasticity index of 40, a liquidity index of 1.5, and a sensitivity of about 20. However, geochemical factors, such as salinity reduction, are considered necessary for the development of extrasensitive or quick clays, which have a liquidity index greater than 2 and flow when remolded. However, it should be noted that geochemical factors other than salinity reduction, such as the introduction of dispersants, can also lead to an increased liquidity index and sensitivity.

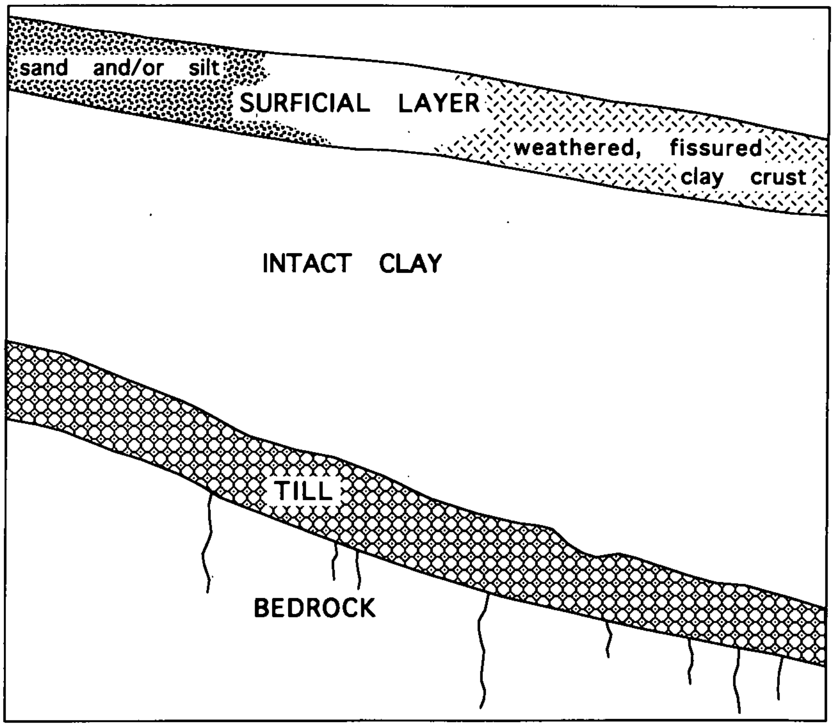
2.4 Composition and Mineralogy

Because of their geologic origin, sensitive clays frequently have a characteristic composition and mineralogy. The silt fraction is most often high, typically in the range of 30 to 70 percent. The fraction with grain sizes smaller than 2 µm generally contains more rock flour than clay minerals, and the clay minerals are predominantly illite (Quigley 1980; Locat et al. 1984).

2.5 Stratigraphy

In large depositional basins, marine clay deposits are generally massive and notably uniform. In small basins, or in the proximal areas of large basins, marine clay deposits can be interlayered with sand and gravel and even with glacial till (Gadd 1975). For most slope stability analyses, the stratigraphy of typical postglacial soft clay deposits can generally be simplified (Figure 24-2). The massive clay formation is confined between two relatively pervious boundary layers. The upper boundary consists of either a weathered fissured crust or sand and silt derived from the final depositional stages. The lower boundary is formed either by till deposited on the bedrock by glaciers or by fissured bedrock. The overall coefficient of permeability in the boundary layers is generally at least two orders of magnitude higher than that in the massive clay deposit.

FIGURE 24-2 Simplified down-valley cross section of typical soft marine clay deposits of eastern Canada (modified from Lefebvre 1986).



3. VALLEY FORMATION AND GROUNDWATER REGIME

The process of clay deposition in shallow marine or brackish environments usually produced an initial deposit with little or no topographic relief and gentle slopes. Subsequent emergence of these deposits subjected them to subaerial erosion. Today the slopes of geotechnical concern in these sensitive clay deposits are the result of erosion by rivers or streams that cut valleys through these deposits. In the Champlain Sea region of eastern Canada, the preconsolidation pressure is, in general, in agreement with the maximum filling level of the valley (Eden and Crawford 1957; Crawford and Eden 1965). In other areas the apparent preconsolidation reflects either cementation (Locat and Lefebvre 1986; Lefebvre et al. 1988) or secondary consolidation (Bjerrum 1967). Stages of valley formation are illustrated in a simplified manner in Figure 24-3, where the valley bottom is seen to gradually approach the lower pervious boundary layer.

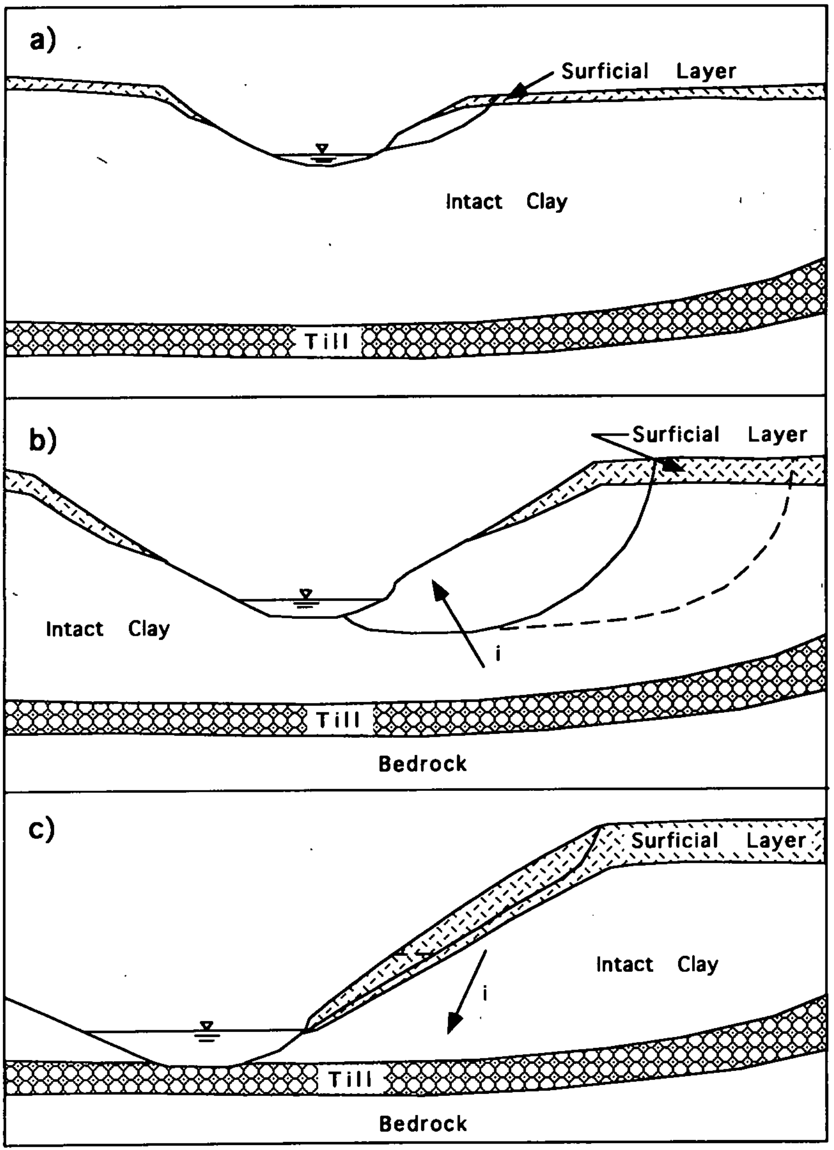


FIGURE 24-3 Stages of valley formation in marine clay deposits (modified from Lefebvre 1986): (*a*) early phase, (*b*) intermediate phase, (*c*) advanced phase.

Valleys in soft sensitive clays can be found today in different stages of formation, and the groundwater regime evolves according to the valley development. This evolution of the groundwater regime plays an important role in the general state of slope stability and in the morphology of the landsliding that does occur. The groundwater regime in the vicinity of a slope not only determines the pore-pressure distribution in the clay deposits, but also is a prime factor in the general slope stability along the valley sides. In sensitive clay areas the critical location for the water table is often close to ground surface. The groundwater regime can vary widely from one site to another, ranging from a general underdrainage to high artesian conditions. It is difficult to interpret and to extrapolate piezometric readings without knowing the pattern of the groundwater regime. Simple considerations of typical present-day topographic profiles and of the valley erosion process, as shown in Figure 24-3, are helpful in predicting and understanding the groundwater regime.

Once the boundary conditions have been assumed, it is indeed a simple matter to produce a flow net because for practical purposes the permeability of marine clay deposits can be considered isotropic (Tavenas et al. 1983). Figure 24-4 shows groundwater flow patterns for the geometric conditions corresponding to the three stages of valley development shown in Figure 24-3. These flow nets have been determined numerically assuming a permeability in the lower boundary layer of 10 times the permeability in the clay.

In the early stage of valley formation, the groundwater regime is not significantly influenced by the lower pervious boundary [Figure 24-4(*a*)] because the bottom of the valley is well above it. However, as the bottom of the valley is eroded closer to the lower boundary [Figure 24-4(*b*)], high artesian pressures develop. At this stage, because pore-pressure discharge or dissipation is restricted by the relatively impervious clay, the pore pressure in the boundary layer under the bottom of the valley and the lower part of the slope tends to equilibrate with the head existing behind the slope crest. Such a condition is obviously detrimental to stability and favors a deep failure surface. In fact, the artesian pressures evaluated from the flow net of Figure 24-4(*b*) are sufficient to blow up the bottom of the valley.

Groundwater within the lower boundary layer discharges into the river once the valley formation reaches a sufficiently advanced stage [Figure 24-4(c)]. In the vicinity of the slope, the piezometric head is slightly higher than the river level and creates a downward gradient in the clay formation. The surface infiltration into the upper pervious boundary materials is much larger than the flow that can percolate through the intact clay formation. Thus, the water table is essentially unaffected by the downward drainage and generally remains close to the surface inside the upper boundary layer, fluctuating with the season. In the upper boundary layer, the flow can be assumed to be parallel to the ground surface and under almost hydrostatic conditions.

Groundwater regimes along valleys in their final stages of formation have been documented at several sites, especially in the Tyrrell Sea basin of northern Québec Province where valleys appear to have evolved more rapidly because of a clay that is more erodible than that in the St. Lawrence valley (Lefebvre 1986). The groundwater regime deduced from a large number of piezometers at one site in the Broadback River valley is presented in Figure 24-5(*a*) along with the stratigraphy obtained from several borings. Figure 24-5(*b*) presents the results of a numerical analysis by the finite-element method. The permeability coefficients used in these analyses were obtained by permeability tests in open-tube piezometers.

Both the numerical analysis and the piezometric data confirm that strong underdrainage developed in the clay formation when the underlying till formation discharged into the river. The groundwater table remained close to the ground surface in the wet season so that essentially hydrostatic conditions prevailed in the surficial crust. Under the crest of the slope, the pore pressure increased with depth in the upper part of the intact clay formation and then decreased toward the

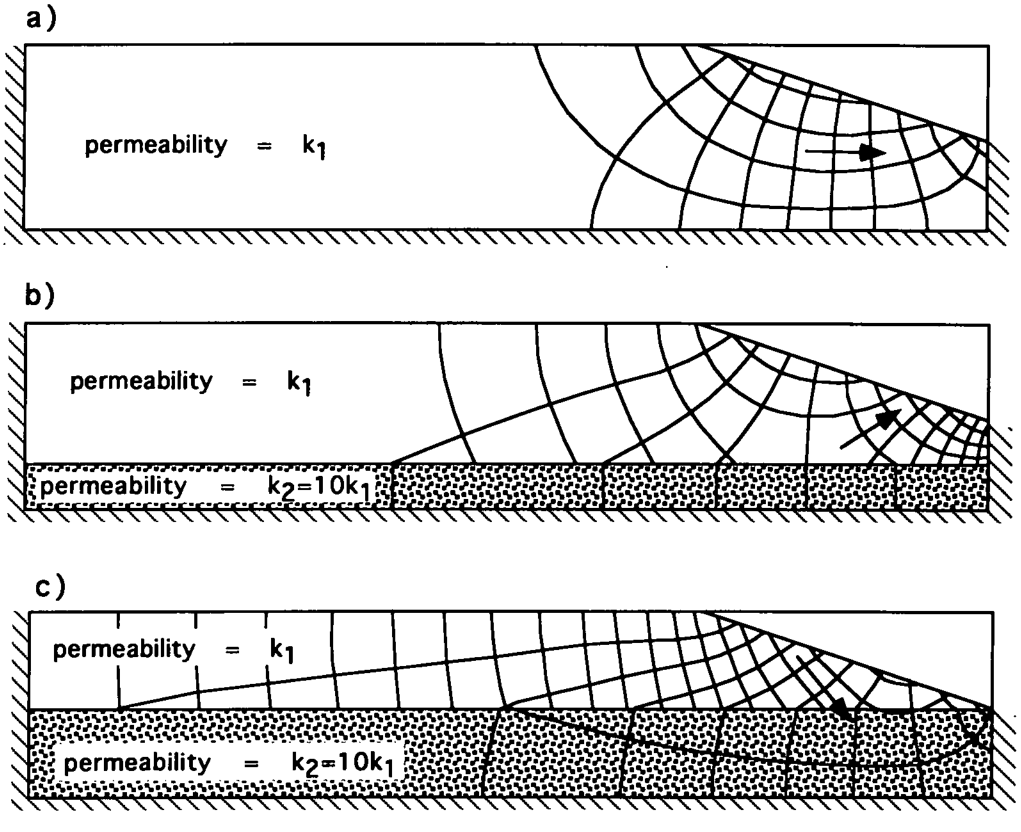


FIGURE 24-4
Flow nets showing groundwater regimes in vicinity of clay slopes corresponding to stages of valley formation shown in Figure 24-3 (modified from Lafleur and Lefebvre 1980):
(*a*) early phase,
(*b*) intermediate phase,
(*c*) advanced phase.

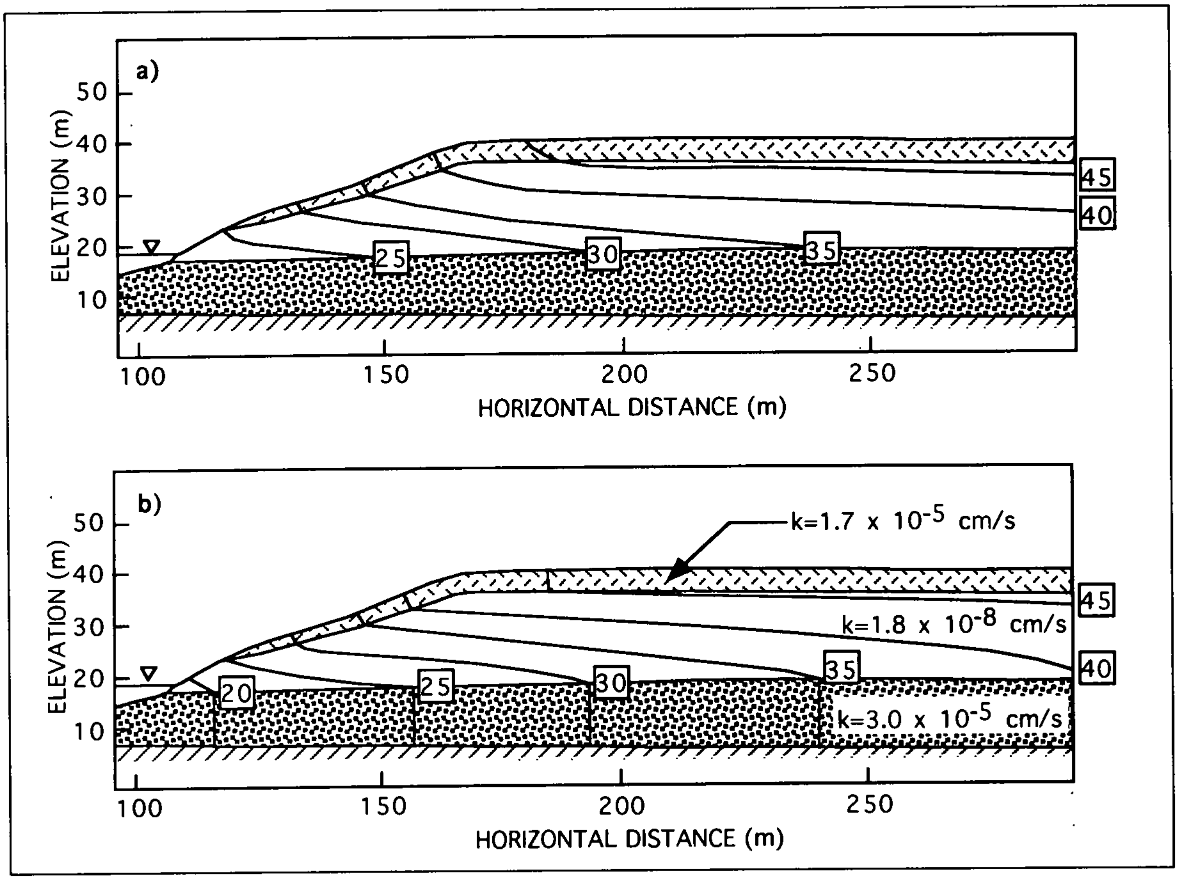


FIGURE 24-5 Equipotential lines obtained from (*a*) piezometer readings November 1980 and (*b*) finite-element analyses at Broadback River site, northern Québec Province, Canada (modified from Lefebvre 1986).

lower boundary value. Such general underdrainage is obviously beneficial for stability.

In contrast, the artesian pore pressure that is characteristic of the intermediate stage of valley formation [Figure 24-4(*b*)] creates conditions that may lead to deep landslides involving large masses of sensitive clays and to subsequent retrogressive failures.

When the lower-boundary material discharges into the river, at the later stages of valley formation [Figure 24-4(c)], the valley walls have reached a phase of equilibrium and stability. Because of important underdrainage at this stage, landsliding in response to bank erosion is generally shallow and does not penetrate deeply into the sensitive intact clay. Risk of retrogression is then fairly low. At this stage, slopes as high as 30 m and as steep as 30 degrees have been found to stand in normally consolidated deposits (Lefebvre et al. 1991). Cases of retrogressive failure have been identified in regions reflecting this final stage of valley formation, but only under conditions of exceptionally severe toe erosion and undercutting (Lefebvre et al. 1991).

4. STABILITY ANALYSIS

As noted in Chapter 13, slope stability methods evaluate the state of stress on a potential failure surface and compare the shear stresses with the available shear strength in the form of a safety factor. The stress evaluation for effective stress analysis is more complex because the effective normal stress on the potential failure surface must be calculated in order to evaluate the shear strength. In such analyses, realistic pore-pressure distribution and shear-strength parameters must be used to obtain a correct evaluation of the available shear strength.

4.1 Shear Strength

The shear-strength parameters must correctly model the shear strength that can be mobilized on the failure surface of a slope that has been formed by the erosion processes described in the previous sections. For that purpose, one must distinguish between those conditions causing an initial failure of a slope that has existed for some time and those conditions causing the almost instantaneous retrogressive failures taking place after a first slide.

As shown in Figure 24-6, starting from the maximum level of sediment filling, the slope that exists today is the result of a slow process of erosion that has decreased the confining stress and increased the shear stress. The stress state on a soil element lying on a potential failure surface can thus be considered the end point of a drained stress path (A to B, Figure 24-6). When stress state B approaches the strength envelope closely enough, a slight reduction in effective stress or increase in shear stress caused by any number of reasons is sufficient to make the slope susceptible to incipient failure.

Actual failure may never occur, or it may occur following a slight additional change to the stress state resulting from such causes as a higher water table during a wet season or erosion at the toe of the slope. This failure will obviously be undrained. Because of the overconsolidated state of the soil, if the failure is triggered by an undrained increase of shear stress due to rapid toe erosion, the undrained stress path (*BO*) will not deviate much from the drained stress path (*BN*). Thus, an effective stress analysis that implies a drained stress path remains valid until the incipience of failure and even then does not introduce a significant error into the analysis of an initial landslide.

However, the situation is completely different for retrogressive failures. The shear stresses are applied almost instantaneously and create an undrained loading. Stability consideration for the role of retrogression must therefore be based on the undrained shear strength.

4.2 Effective Stress Parameters

The effective stress parameters for analysis of an initial failure must describe the shear strength for the range of stresses existing on a potential failure surface; these parameters are generally determined by drained triaxial compression tests. The reconsolidation, or cell, pressure must be fairly low in order to have a state of stress at failure in the range of those stresses existing in the field on the potential failure surface. For this purpose, triaxial specimens are commonly reconsolidated at cell pressures ranging from 5 to 30 kPa in order to have, at failure, a state of stress that is in the overconsolidated range.

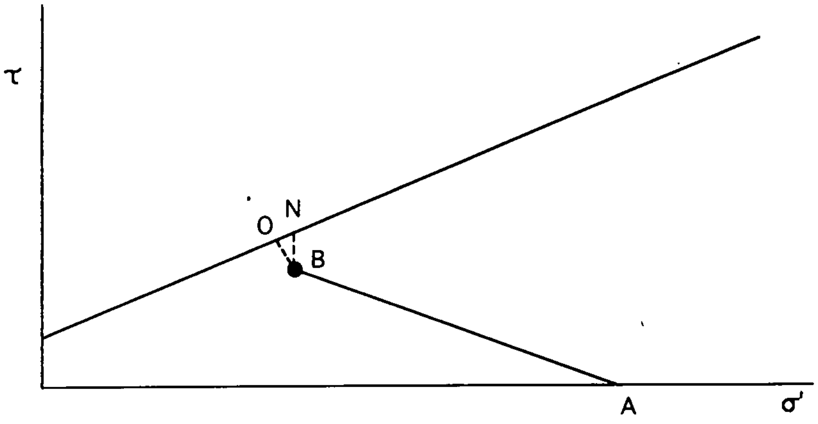


FIGURE 24-6 Approximate stress path of clay element lying on potential failure surface in slope.

When tested under low confining stress in the overconsolidated range, sensitive clays exhibit brittle behavior (Figure 24-7). A sharp peak strength is mobilized at relatively small axial deformation on a triaxial stress-strain curve; the strength rapidly decreases after the peak and stabilizes on a plateau. The shear strength remains approximately constant beyond an axial deformation of 4 to 5 percent. The strength of these eastern Canadian sensitive clays at large deformations is routinely determined at an axial deformation of 8 percent. Study of many case records for locations in eastern Canada has shown that it is the large-deformation, or postpeak, strength that is mobilized during the initial failure of a natural slope or long-term excavation slope (Lefebvre and

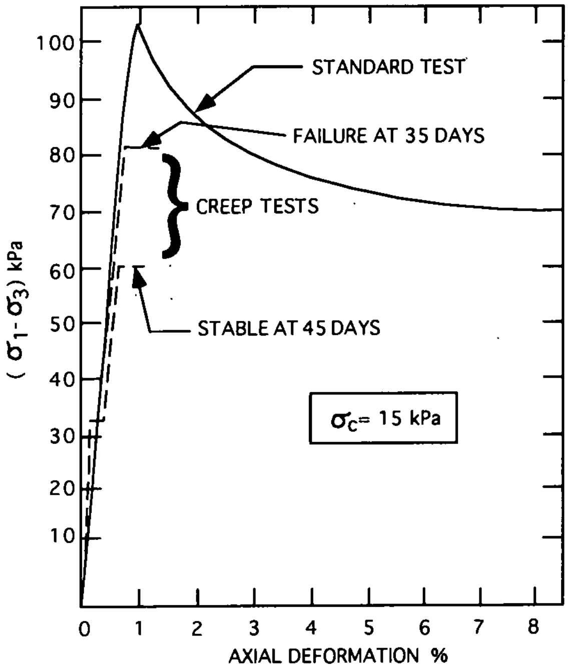


FIGURE 24-7 Creep and standard drained triaxial tests on sensitive Nicolet clay, eastern Canada (modified from Philibert 1976).

LaRochelle 1974; Lefebvre 1981). Skempton (1964) reached a similar conclusion, namely, that for stiff overconsolidated clays, it is the strength at large deformations that is mobilized during a slope failure.

The use of the postpeak strength in performing stability analyses of sensitive clays can be justified by progressive failure along the failure surface or by the fact that the peak strength in sensitive clay cannot be mobilized under sustained load. Triaxial drained creep tests performed on sensitive St. Lawrence clays (Figure 24-7) have shown that creep failure develops for any loading above the strength level corresponding to large deformations (Lefebvre 1981). In any event, the strength at large deformations may be considered to form a stability threshold in terms of either deformation or time. The postpeak strength is generally determined by means of an isotropically consolidated triaxial drained test with a rate of loading of 1 to 2 percent per day to allow drainage; the test is continued to an axial deformation of 10 to 12 percent.

Sampling disturbance is generally a significant factor with sensitive clay in the overconsolidated state. The peak strength is, for example, much affected by sampling disturbance. However, good commercial tube sampling is sufficient to determine the strength at large deformations. When tube sampling methods are used, the scatter in the results may be larger than when block samples are used. In principle, the large-deformation strength envelope can be determined for overconsolidated samples by using undrained tests with pore-pressure measurement. However, difficulties arise in obtaining this determination accurately by such methods because the stress paths all tend to reach the failure envelope at the same stress state.

4.3 Normalized Shear Strength

A large data bank has accumulated over the years on the postpeak shear strengths of the sensitive Champlain Sea clays of eastern Canada. For specimens reconsolidated in the laboratory to the same confining pressure, the strength at large deformations appears to be influenced by the stress history; that is, it increases with preconsolidation pressure. Figure 24-8 presents test results on block sample specimens of Champlain Sea clays from nearly 20 sites (Lefebvre 1981). The data are normalized by the preconsolidation pressure, σ_p'. The data from all these sites, located over a region about 700 km long, plot in a surprisingly narrow band. They represent a range of natural water content from 40 to 90 percent, plasticity index from 5 to 40, and preconsolidation pressure from 100 to 600 kPa.

The normalized postpeak envelope (Figure 24-8) exhibits some curvature, with the shear resistance initially increasing rapidly with effective stress and then flattening toward the normally consolidated envelope. Because this normalized envelope is common to all Champlain Sea clay sites, it can be used as a first approximation to derive the strength parameters from σ_p'. As indicated by the curvature of the envelope, for the same confinement stress range, ϕ' increases with σ_p'. If the parameters are determined for an identical confinement pressure (σ_3') range of 5 to 20 kPa, the normalized envelope of Figure 24-8 provides an estimate of ϕ' that increases from 29 to 43 degrees, whereas σ_p' increases from 100 to 400 kPa and c' remains approximately constant at 7 kPa. These values are summarized in Table 24-1. Test results from tube samples generally correlate with the envelope defined in Figure 24-8 but exhibit a larger scatter.

The postpeak strength defined for the soft sensitive Champlain clays is surprisingly high and appears somewhat paradoxical when compared with the residual strength determined for stiff overconsolidated clays, which is defined by a zero cohesion intercept and a value for ϕ' that sometimes is as low as 10 to 15 degrees. The high postpeak c' and ϕ' values for the Champlain clay are primarily related to its mineralogy. The high silt and rock flour content and the relatively low fraction of illitic clay minerals result in a fairly large friction angle. In addition, as shown by the correlations established with σ_p', the shear strength at large deformations is still influenced by the stress history and may be different from a true residual strength.

The correlations established for Champlain clay should be checked before they are adopted to describe the characteristics of sensitive clays from other regions of the world. From limited data obtained from analyses of block samples of sensitive clays from four sites in the Tyrrell Sea basin in northern Québec Province, the postpeak parameters for the Tyrrell Sea clays appear to be slightly lower than those for Champlain clays. Figure 24-9 shows a comparison of the normalized postpeak strength envelopes for the Tyrrell Sea and Champlain Sea clays.

Marine clay deposits are generally massive and fairly homogeneous; thus the use of a circular failure surface is generally appropriate. Bishop's modified method is most often used for stability analyses of soft clay slopes at sites in eastern Canada. The relatively high ϕ' that is used for the stability analysis of an initial failure in sensitive clay slopes points to the importance of a good understanding of the pore-pressure distribution.

5. ESTABLISHING RISK OF RETROGRESSION

Although the analysis of an initial landslide failure on a slope composed of sensitive clay appears fairly straightforward, there is much more uncertainty in the evaluation of the risk of retrogression following this initial failure. The potential retrogression distance is indeed an important parameter in assessing the risk of damage.

The risk of retrogression is broadly related to two conditions. First, the clay has to be sensitive enough that the debris of the initial landslide will move away, or flow out, instead of creating a stabilizing mass at the toe of the newly created slope. Second, the newly created slope must have an unstable geometry, or a geometry such that the induced shear stresses exceed the undrained shear strength of the clay.

5.1 Role of Sensitivity and Remolded Shear Strength

Because factors other than clay sensitivity, such as slope geometry and stratigraphy, play a role, it is difficult to establish a sensitivity threshold below which the debris from the initial landslide will not move away and prevent retrogression. However, large retrogressive failures, for example, the typical "bottleneck" earth flow, develop generally in very sensitive or quick clays. In fact, the remolded shear strength controls the susceptibility of the earth mass to flow. Figure 24-10 presents values of sensitivity as a function of the remolded shear strength for a dozen sites of slope failure in Champlain clays (Grondin 1978). Those sites where retrogression has or has not developed are also identified. The remolded shear strengths and sensitivities shown in Figure 24-10 were determined in the laboratory. At sites where retrogressive failures developed, the remolded shear strength was lower than 1 kPa and the sensitivity was generally higher than 40. Figure

Table 24-1
Postpeak Strength Parameters for Sensitive Clays from Champlain Sea Deposits of Eastern Canada (modified from Lefebvre 1981)

RANGE OF CONSOLIDATION STRESS (kPa)	σ_p' (kPa)	c_m' (kPa)	ϕ_m' (degrees)
5–20	100	7.4	28.7
5–20	200	7.7	34.7
5–20	300	7.7	39.8
5–20	400	7.5	43.6

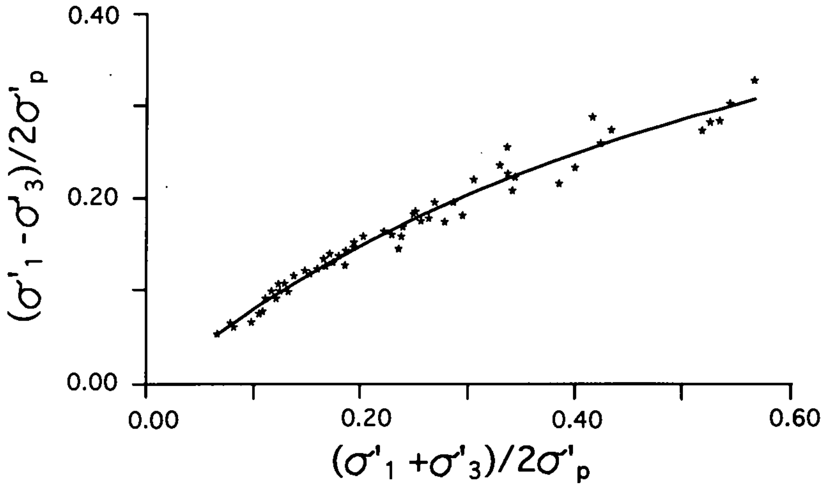


FIGURE 24-8
Normalized postpeak strength envelope from block samples of Champlain Sea sensitive clay (Lefebvre 1981).

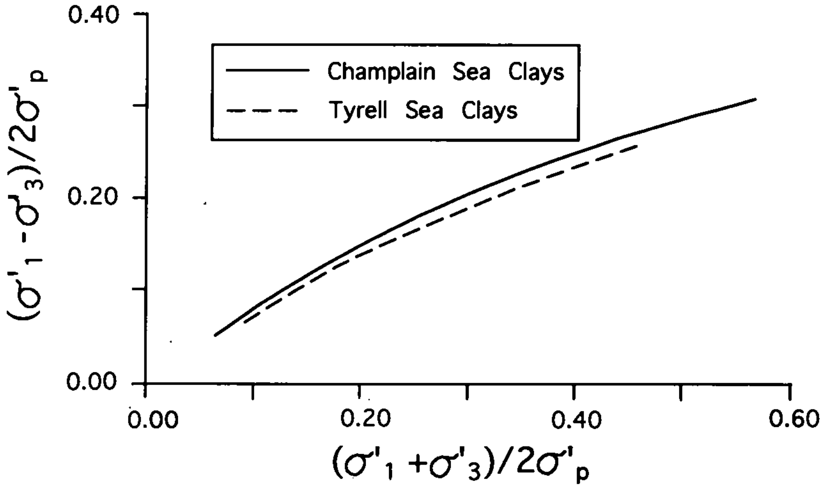


FIGURE 24-9
Comparison of normalized postpeak strength envelopes for Champlain Sea and Tyrell Sea sensitive clays.

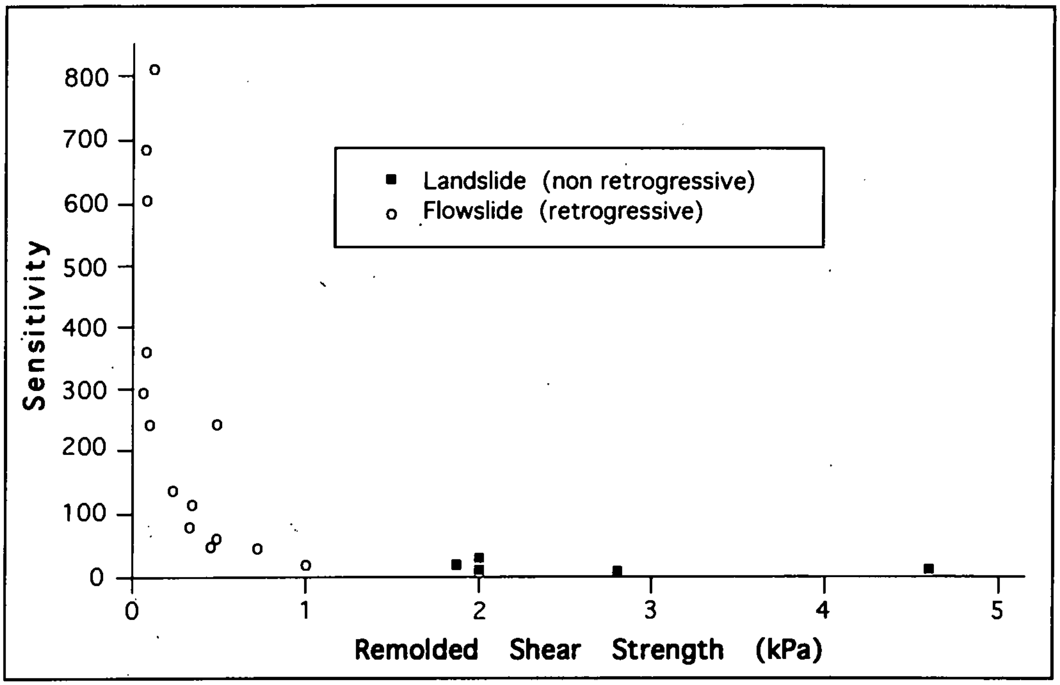


FIGURE 24-10
Relation among type of landslide failure, sensitivity, and remolded shear strength of sensitive clays (modified from Grondin 1978).

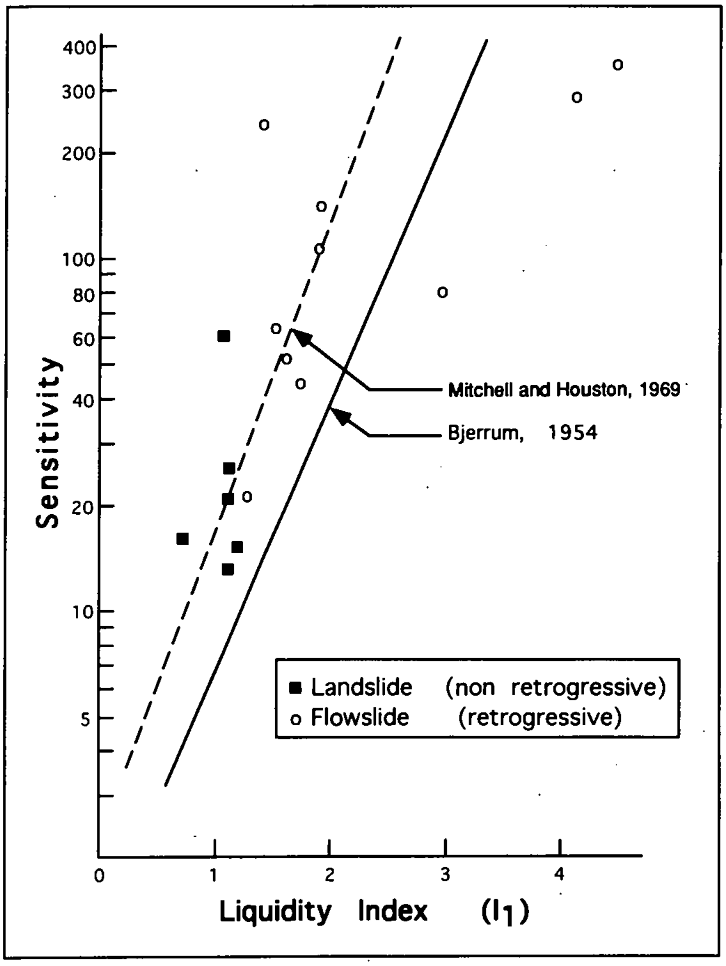


FIGURE 24-11
(below)
Risk of retrogression in terms of sensitivity and liquidity index; data points are those shown in Figure 24-10 (modified from Grondin 1978).

24-10 clearly shows that the quick clays with remolded shear strengths lower than 0.2 kPa are more prone to flow than those with remolded shear strengths between 0.5 and 1.0 kPa.

At its liquid limit, the remolded shear strength of a typical clay is about 1.7 kPa (Wroth and Wood 1978). The data shown in Figure 24-10 indicate that the liquidity index has to be above unity for a site to have retrogression potential. Relationships between sensitivity and liquidity index have indeed been proposed by many authors (Skempton and Northey 1953; Bjerrum 1954; Mitchell and Houston 1969). Some of these relations are reproduced in Figure 24-11 together with the data for the sites shown in Fig-ure 24-10. The criteria of Mitchell and Houston (1969) suggest that a sensitivity of 40 corresponds to a liquidity index of about 1.5 to 2.0. Clays at sites where retrogression developed (from Figure 24-10) generally have a sensitivity higher than 40 and a liquidity index exceeding 1.5. Therefore, the liquidity index, which is routinely available, is a good indicator of the susceptibility of these clays to flow.

5.2 Stability Number

The susceptibility of the clay to flow is not sufficient to induce retrogressive earth-flow failures. The slope geometry after the initial failure must be such that the shear stress in the new slope exceeds the undrained shear strength of the clay. Therefore, the potential distance of retrogression is difficult to evaluate. Nevertheless, the risk of retrogression after the occurrence of an initial failure can be assessed at least qualitatively. From analytical studies and examination of more than 40 case records, Mitchell and Markell (1974) proposed that retrogression can occur only if the stability number, $N_s = \gamma H/Cu$, is larger than 6 within the depth of potential failure.

This stability number is similar to the coefficient used in the evaluation of bearing capacity, as if the stress γH imposed by the new slope were creating an undrained extrusion of the clay below. The larger the value of $\gamma H/Cu$, the larger will be the retrogression distance before equilibrium. Mitchell and Markell (1974) found a general relationship between $\gamma H/Cu$ and the distance of retrogression, although other factors such as sensitivity, stratigraphy, and topography also appear to play a significant role in limiting retrogression.

The groundwater regime influences the effective-stress profile and consequently the undrained shear strength profile, especially in normally consolidated deposits. In the final stage of valley formation, deposits may be affected by strong underdrainage, as described earlier, which causes the undrained shear strength to increase rapidly with depth in the lower half of the clay deposit. Under these circumstances, the most critical position for a retrogression surface will be much higher on the slope than would otherwise be the case because of the unique relationships between the $\gamma H/Cu$ values and elevation (or depth).

The liquidity index and the ratio $\gamma H/Cu$ are probably the most useful parameters for assessing the risk of occurrence of retrogressive failures. However, because retrogressive landslides can only be triggered after the initial failure has occurred, securing the stability of a slope against an initial failure is the logical approach in preventing large retrogressive earth flows.

6. CONCLUSIONS

In sensitive clay deposits, landslides are an important component in the development of young river valleys. The primary cause of instability is changes of geometry related to natural erosion or sometimes to human intervention. In slopes in which stability has already become marginal, some slight seasonal variations in the groundwater regime may trigger a landslide. For the long term, however, the groundwater regime in the vicinity of slopes evolves with the development of valleys in a way that affects the general stability of the valley walls and the morphology of the landslides. Detailed piezometric readings are required to permit a rational interpretation of the groundwater regime. Piezometric data should not be considered isolated data values but a reflection of a groundwater regime controlled by boundary conditions.

Approaches to stability analyses of sensitive clay slopes are based largely on experience and the study of many case records. However, consideration of the genesis of clay slopes justifies the type of analysis as well as the approach for determination of the shear-strength parameters. Landslides in sensitive clays are notorious for large and very rapidly enlarging retrogressive earth flows that can be triggered as a result of the initial failure. Expertise has been developed in the evaluation of stability against an initial failure using effective stress analysis, proper pore-pressure data, and postpeak strength parameters. However, it remains difficult to assess whether an initial failure will generate a sequence of retrogressive earth flows. Slopes composed of sensitive clays with a stability number $\gamma H/Cu$ larger than 6 and a liquidity index higher than 1.5 are generally considered as presenting a risk of retrogressive failure following an initial failure.

REFERENCES

Andrews, J.T. 1972. Post-Glacial Rebound. In *The National Atlas of Canada*, Canada Department of Energy, Mines, and Resources, Ottawa, Ontario, pp. 35–36.

Bjerrum, L. 1954. Geotechnical Properties of Norwegian Marine Clays. *Geotechnique*, Vol. 4, pp. 49–69.

Bjerrum, L. 1967. Engineering Geology of Norwegian Normally Consolidated Marine Clays as Related to Settlements of Buildings. *Geotechnique*, Vol. 17, pp. 81–118.

Brooks, G.R., J.M. Aylsworth, S.G. Evans, and D.E. Lawrence. 1994. *The Lemieux Landslide of June 20, 1993, South Nation Valley, Southeastern Ontario—A Photographic Record.* Miscellaneous Report 56. Geological Survey of Canada, Ottawa, Ontario, 18 pp.

Crawford, C.G., and W.J. Eden. 1965. A Comparison of Laboratory Results with In Situ Properties of Leda Clay. In *Proc., Sixth International Conference on Soil Mechanics and Foundation Engineering*, Montreal, Canada, University of Toronto Press, Vol. 1, pp. 31–35.

Eden, W.J., and C.B. Crawford. 1957. Geotechnical Properties of Leda Clay in the Ottawa Area. In *Proc., Fourth International Conference on Soil Mechanics and Foundation Engineering*, London, England, Butterworths Scientific Publications, London, Vol. 1, pp. 22–57.

Eden, W.J., and P.M. Jarrett. 1971. *Landslide at Orleans, Ontario*. Technical Paper 321. Division of Building Research, National Research Council of Canada, Ottawa, Ontario.

Eden, W.J., E.D. Fletcher, and R.J. Mitchell. 1971. South Nation River Landslide. *Canadian Geotechnical Journal*, Vol. 8, No. 3, pp. 446–451.

Elson, J.A. 1969. Late Quaternary Marine Submergence of Quebec. *Revue de Géographie de Montréal*, Vol. 23, pp. 247–259.

Evans, S.G., and G.R. Brooks. 1994. An Earthflow in Sensitive Champlain Sea Sediments at Lemieux, Ontario, June 20, 1993, and Its Impact on the South Nation River. *Canadian Geotechnical Journal*, Vol. 31, No. 3, pp. 384–394.

Gadd, N.R. 1975. Geology of Leda Clay. In *Mass Wasting: Proc., Fourth Guelph Symposium on Geomorphology*, Geo Abstracts Ltd., Norwich, England, pp. 137–151.

Grondin, G. 1978. *Etude de la susceptibilité aux coulées d'argile*. M.Sc. thesis. Department of Civil Engineering, Université de Sherbrooke, Québec, Canada.

Hillaire-Marcel, C. 1979. *Les mers post-glaciaires du Québec: quelques aspects*. Thèse de doctorat d'Etats Sciences Naturelles, Université Pierre et Marie Curie, Paris, France (2 vols).

Hillaire-Marcel, C., and R.W. Fairbridge. 1978. Isostacy and Eustacy of Hudson Bay. *Geology* (Geological Society of America), Vol. 6, pp. 117–122.

Kenney, T.C. 1964. Sea Level Movements and the Geologic Histories of the Post-glacial Marine Soils at Boston, Nicolet, Ottawa and Oslo. *Geotechnique*, Vol. 14, pp. 203–230.

Lafleur, J., and G. Lefebvre. 1980. Groundwater Regime Associated with Slope Stability in Champlain Clay Deposits. *Canadian Geotechnical Journal*, Vol. 17, No. 1, pp. 44–53.

Lefebvre, G. 1981. Strength and Slope Stability in Canadian Soft Clay Deposits. *Canadian Geotechnical Journal*, Vol. 18, No. 3, pp. 420–442.

Lefebvre, G. 1986. Slope Instability and Valley Formation in Canadian Soft Clay Deposits. *Canadian Geotechnical Journal*; Vol. 23, No. 3, pp. 261–270.

Lefebvre, G., and P. La Rochelle. 1974. The Analysis of Two Slope Failures in Cemented Champlain Clays. *Canadian Geotechnical Journal*, Vol. 11, No. 1, pp. 89–108.

Lefebvre, G., C.C. Ladd, and J.J. Paré. 1988. Comparison of Field Vane and Laboratory Undrained Shear Strength in Soft Sensitive Clays. In *Laboratory and Field Vane Shear Strength Testing*, Special Technical Publication, ASTM, Philadelphia, Pa., pp. 233–246.

Lefebvre, G., P. Rosenberg, J. Paquette, and J.-G. Lavallée. 1991. The September 5, 1988, Landslide on the La Grande River, James Bay, Québec, Canada. *Canadian Geotechnical Journal*, Vol. 28, No. 2, pp. 263–275.

Locat, J., and G. Lefebvre. 1986. The Origin of Structuration of the Grande-Baleine Marine Sediments Quebec, Canada. *Quarterly Journal of Engineering Geology*, Vol. 19, pp. 365–374.

Locat, J., G. Lefebvre, and G. Ballivy. 1984. Mineralogy, Chemistry and Physical Properties Interrelationships of Some Sensitive Clays from Eastern Canada. *Canadian Geotechnical Journal*, Vol. 21, No. 3, pp. 530–540.

Mitchell, J.K., and W.N. Houston. 1969. The Causes of Clay Sensitivity. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 95, No. SM3, Paper 6568, pp. 845–871.

Mitchell, R.J., and A.R. Markell. 1974. Flowslides in Sensitive Soils. *Canadian Geotechnical Journal*, Vol. 11, No. 1, pp. 11–31.

Norsk Geoteknisk Forening. 1974. *Retningslinjer for Presentasjon av Geotekniske Undersökelser*. Oslo, 16 pp.

Philibert, A. 1976. *Etude de la résistance au cisaillement d'une argile Champlain*. M.Sc. thesis. Faculté des Sciences Appliquées, Université de Sherbrooke, Québec, Canada.

Quigley, R.M. 1980. Geology, Mineralogy and Geochemistry of Canadian Soft Soils: A Geotechnical Perspective. *Canadian Geotechnical Journal*, Vol. 17, No. 2, pp. 261–285.

Rosenqvist, I. Th. 1966. Norwegian Research into the Properties of Quick Clay—A Review. *Engineering Geology* (Elsevier), Vol. 1, pp. 445–450.

Skempton, A.W. 1964. Long-term Stability of Clay Slopes. *Geotechnique*, Vol. 14, pp. 75–102.

Skempton, A.W., and R.D. Northey. 1953. The Sensitivity of Clays. *Geotechnique*, Vol. 3, pp. 30–53.

Tavenas, F., J.-Y. Chagnon, and P. La Rochelle. 1971. The Saint-Jean-Vianney Landslide: Observations and Eye-Witness Accounts. *Canadian Geotechnical Journal*, Vol. 8, No. 3, pp. 463–478.

Tavenas, F., P. Jean, P. Leblond, and S. Leroueuil. 1983. The Permeability of Natural Soft Clays, Part II: Permeability Characteristics. *Canadian Geotechnical Journal*, Vol. 20, pp. 645–660.

Torrance, J.K. 1987. Quick Clays. In *Slope Stability* (M.G. Anderson and K.S. Richards, eds.), John Wiley and Sons, New York, pp. 447–473.

Wroth, C.P., and D.M. Wood. 1978. The Correlation of Index Properties with Some Basic Engineering Properties of Soils. *Canadian Geotechnical Journal*, Vol. 15, pp. 137–145.