STABILITY OF SLOPES IN THE SOFT, GLACIOLACUSTRINE DEPOSITS OF THE THUNDER BAY REGION, N.W. ONTARIO.

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Abstract

The glaciolacustrine deposits along the N. W. shore of Lake Superior, even though soft and often highly sensitive, are generally not prone to excessive slope stability problems. However, under certain geological conditions, human activities, such as deforestation, may lead to more extensive sliding. The associated risk to engineering structures and human life is moderate as long as the extent of human activities in the area remains limited.

Résumé
Les argiles glaciolacustre de la côte nort-ouest du Lac Superieur, quoique molles et trés sensibles, ne sont pas généralement affectés par des problémes de glissement des pentes. Mais dans des conditions géologiques particulièrs, les activités humains tells que le déboisement peuvent causer des glissements de terrain. Les risques aux infrastructures et à la vie humaine sont modestes tant que dévelopement économique de la région reste limité.

1. INTRODUCTION

The glaciolacustrine deposits which are encountered along the N. W. shore of Lake Superior typically are interlayered clay, silt and sand beds of various thicknesses, reflecting the deltaic and lacustrine environment in which they had been laid down at the fringe of glacial lake Algonquin during the last ice age (Zoltai, 1963). The total thickness of these deposits varies generally between 15 and 30 metres, but may exceed 40 metres in some areas. The clays and silts often have a more or less pronounced varved structure and depending on their origin may be medium to high plastic. Even though many of the clays and silts are fairly sensitive, slopes cut into these deposits by rivers and streams are relatively stable and generally are affected only by shallow slumps triggered by undercutting due to toe erosion. However, occasionally, more extensive landsliding has been observed in the area. Two different types of slope failures can be recognized, both of which can be related to previous deforestation. The slopes with low landslide hazards will be discussed, as well as the sites where landsliding had occurred (See Fig.'s 1 and 2).

2. SLOPES IN VARVED FORT WILLIAM CLAYS

The soft, sensitive clay deposits typically encountered in the lowlands of the Thunder Bay area, previously known as Fort William, have been referred to in the literature as Fort William Clays (e.g., Soderman and Quigley, 1965). The Fort William Clays are typically overlain and interlayered with silt and sand beds of various thicknesses, reflecting the delatic environment in which they had been laid down at the fringe of glacial Lake Algonquin during the last ice ages (Quigley 1980). The clays show a more or less pronounced varved structure and depending on the clarity of the varved structure have been subdivided into “massive silty clay” and “varved clay” (e.g., Palmer and Belshaw 1980; Folkes and Crooks 1985; Ng 1984). The varved Fort William Clays consist of dense sandy silt layers and soft silty clay layers, with thicknesses ranging between 3 and 20 mm for the silty layers and between 1 and 5 mm for the clay layers, with liquidity indices of around 1.0 for both materials. Despite the high liquidity index values, *in situ* vane tests resulted typically in undrained shear strength values between 35 and 70 kPa. The sensitivities based on field vane testing were generally in the range of 5-9, denoting fairly sensitive clays. The Atterberg limits (w_L = 21-48% and I_p = 8-29% for the silt layers; w_L = 44-77% and I_p = 26-57% for the clay layers) indicate when plotted in a plasticity chart that the data points fit into the typical pattern suggested by Milligan *et al.* (1962) for Canadian varved clays. Carbonate contents between 5 and 6% for the silt layers and between 8 and 20% for the clay seams were determined at Lakehead University (Eigenbrod and Burak, 1991) as well as in previous studies by Soderman and Quigley (1965), suggesting a high degree of cementation. Montmorillonite contents of 5-15% of the bulk material were reported by Soderman and Quigley (1965). A summary of the average index properties of the varved Fort William Clay specimens tested by Eigenbrod and Burak (1991) can be found in Table 1.

Table 1. Summary of average index properties of varved Fort William Clay for the specimens tested

	Clay seam	Silt seam	Bulk sample
w_N (%)	45	27	39
w_L (%)	48	21	38
w_P (%)	21	18	19
I_P (%)	28	3	23
I_L	0.9	1.0	0.75
% < 2μ	43	16	23
c_u (kPa)	-	-	35-70
Layer thickness (mm)	1.0-2.0	2.0-5.0	-

Slopes cut into these deposits are typically fairly steep and only when they are undercut by toe erosion, they experience shallow break-offs or surficial sloughing.
A typical section of one of the slopes affected by undercutting along the Kaministiquia River in Thunder Bay (Site 1 on Fig. 2) is shown on Fig. 3. The entire slope is approximately 25 metres high and contains the typical irregular sequence of sands, silts and clays. The clay beds stand in almost vertical benches up to a height of 5.0 metres. The clay benches tend to break-off in slabs of less than 1.0 metre thickness. The varved clay material often liquefies after break-off, carrying parts of vegetation to the bottom of the slope.

Strength tests, direct shear tests, as well as consolidated isotropical undrained (CIU) triaxial tests with pore-pressure measurements, indicated significant effects of the varved structure on the stress-strain behaviour of the clay (Eigenbrod and Burak, 1991). Different strength parameters were observed depending on whether failure occurred across or parallel to the varves. The failure modes for shear parallel to the varves depended also on the stress levels applied. (See Fig. 4) This behaviour is similar to observations reported for New Liskeard varved clays by Lacasse and Ladd (1977) who termed it "double envelope concept." At low consolidation pressures, below the preconsolidation load of 200 kPa, the specimens generally sheared along the silt seams of the varved clay, indicating that the silt was weaker than the clay at low confining stresses. Dilation resulted in *B*-values less than one. Slightly anisotropic elastic deformation behaviour with lower stiffness in the horizontal than in the vertical was indicated, for some specimens almost up to the points of failure. The overconsolidated shear strength was characterized by c' = 12 kPa and ϕ' = 31°, the critical-state shear strength for the overconsolidated specimens by c' = 0 and ϕ' = 37°.

At high stress levels, above the preconsolidation load, specimens sheared largely in the clay seams or sometimes in both, clay and silt seams. Anisotropic elastic behaviour was indicated during initial shear. From the pattern of stress paths and low initial pore-pressure responses it can be assumed that consolidation of the clay seams occurred initially during undrained shear due to internal dissipation of pore pressures into dilating silt seams. A cohesion intercept that was obtained for the normally consolidated clay specimens can be related to carbonate cementation of the varved clay. The critical strength parameters for the normally consolidated varved clay are c' = 0 and ϕ' = 22-29°.

The low initial pore-pressure responses during shearing might explain why slopes in varved Fort William Clay are relatively stable and so far have not exhibited any liquefaction failures. The low horizontal stiffness for varved Fort William Clay might be responsible for large lateral movements that had been observed in the clay for many harbourfront structures in Thunder Bay.

3. SHALLOW SLOPE FAILURES IN THE KAMINISTIQUIA RED CLAY

Shallow ground movements can be commonly observed on grassy slopes of the Upper Kaministiquia River valley (Site 2 on Fig. 2), whereas forested slopes appear stable unless they are undercut by natural or man-made causes (Eigenbrod and Kaluza 1999). The slope failures occur in the red high-plastic clays of the area which typically are underlain by massive clean sand and gravel deposits. The slides are characterized by generally well defined boundaries, (See Fig. 5) indicating that the slope failures were initiated in the grass-covered slopes, after they had been cleared of the original forest cover. To date, only two slides impacted on human development: one slide has been infringing on a residential building and its water supply, and another slide is the cause for continuous movements along an embankment of the Trans-Canada Highway.

The slide which has exhibited slow, recurrent movements towards the residential building (see Fig. 6) will be presented in this paper. The slide is located along a northeast-facing slope with an average inclination of 3.5 (horizontal) to 1 (vertical). It was reported that material had to be removed periodically from the toe of the slide to protect the building at the base of the slope from the moving soil masses. Fortunately, the movements towards the building slowed down considerably after a pit was excavated by the owner into the slope portion just south of the building through the red clay into the underlying sand deposits. Subsequently, the overlying red clay has been slumping into the cut, redirecting the thrust of the movements away from the building towards the cut. In the area of the slide the clay is about 7-10 m thick. Some seepage was noticed occasionally in the fractured red clay during early summer, whereas the underlying sand never exhibited any free water. A 5 m deep water well located about halfway up the slope and entirely within the red clay has been drawing water almost continuously for more than 20 years. This well had to be reset twice since its initial installation in 1978 due to the ongoing slope movements. The surface of the slide area is dissected by numerous open cracks, further indicating continuous ground movements. A clearly defined toe could be recognized near the bottom of the slope, where the moving soil mass had overridden the underlying stable ground.

The surficial red clay is a homogeneous, highly plastic silty clay with a liquid limit ranging between 50 and 78% and a plasticity index between 26 and 46%. In the upper 2-3 m the clay is slightly overconsolidated and intensely fractured due to desiccation and frost action. It is medium stiff at a water content between 30 and 35%. Effective strength parameters obtained for the surficial clay portions from consolidated, drained direct shear tests varied between $\phi' = 28^{o}$ and 32^{o} at a cohesion $c' \cong 0$, indicating fissured or fully softened conditions (Skempton 1970). In the excavation at the base of the slope occasionally a low-plastic, layered, cemented silt was found between the red clay and the underlying grey sand. The sand is a very clean, fine-to-medium-grained material with a silt content of less than 2%. The sand was found to be quite dense, with a moisture content of less than 5%. Well-defined cross-beddings which were apparent in the excavation can be referred to its fluvial origin. Slope indicators installed in the slope identified shallow intermittent ground movements in the fractured upper zone of the clay deposit (See Fig. 7). This fractured upper clay zone represents a shallow aquifer which is separated by the unfissured lower clay zone from the underlying sand deposits. In-situ permeabilities of the fractured clay were measured to 10^{-6}-10^{-7} m/s. The shallow fractured clay zone provided a reasonably reliable supply of water over a period of more than 20 years, even though it was recharged largely by surface water infiltration.

During the observation period from 1989 to 1994 piezometric pressures in the upper fractured clay zone typically reached maximum values in early summer, at the same time when ground movements were identified. The back analysis of slope movements (using a pore-pressure coefficient $r_u = 0.5$) indicated reactivated movements at fully softened strength conditions ($c' = 0$, $\phi' = 30)^{o}$. Since fully softened strength rather than residual strength is governing the present movements, it can be assumed that shear movements have not been sufficiently large for the development of residual shear strength and therefore must have been initiated relatively recently.

The well-defined boundaries of the sliding mass within the grass-covered slope support this interpretation. Thus, it was concluded that the shallow slides in the area which typically were observed in the grass-covered clay slopes are a result of the removal of the original forest cover and occurred subsequently in the cleared slopes, at a time when pore-water pressures were sufficiently high. Deforestation can result in considerable increases in groundwater tables due to a decrease of evaporation as had been demonstrated by various field studies (e.g. Kittredge 1948; Peck and Williamson 1987). An attempt was made to quantify the change in evapotranspiration subsequent to clearing of the original forest cover in the Kaministiquia area, and to determine the resultant increase in groundwater levels and its effect on slope stability (Eigenbrod and Kaluza, 1999). Because of the difficulties of measuring evapotranspiration in the field, a numerical model (Morton, 1976) was used for its assessment on the basis of meteorological data in combination with data suggested by Viessman et al. (1989) for different vegetation covers. The resultant groundwater recharge for grassland shows a pattern similar to that observed in the field for the groundwater levels, with maximum values each year in early summer. On the other hand, for forestland the predicted recharge pattern was clearly different from the observed groundwater pattern within the grassy slopes. Thus, even though no data exist to compare the forest-land recharge pattern predictions with actual measurements of the groundwater fluctuations within the forested slopes, it was suggested that change in vegetation cover during forest clearing is a dominant factor in the development of shallow landslides in the slopes of the Kaministiquia area.

4. NIPIGON RIVER LANDSLIDE

The Nipigon River landslide is a retrogressive slide which occurred in soft sensitive glaciolacustrine deposits along the Nipigon River north of the town of Nipigon (See Fig. 1) on April 23, 1990. The Nipigon River flows south from Lake Nipigon into Lake Superior. Along the course of the river a number of hydro power plants are operated by Ontario Hydro, resulting in operational river level fluctuations.

The slide started at the river bank and extended uphill for about 350 metres with a maximum width of approximately 280 metres. The soil movements were completed within a period of 3 hours, after they severed a fibre optic telephone cable and displaced a gas pipeline laterally by about 8 metres over a length of 75 metres without rupture. The force of the landslide moved debris by about 300 metres upstream and downstream (See Fig. 8), suggesting maximum velocities in the order of 25 km/hr. The temporarily increased silt load in the Nipigon River disrupted the residential water supply further downstream and affected the local lake-trout fishery. Apart from the large 1990 slide discussed here, many smaller slumps and erosion features have been observed along the river banks. No other retrogressive slides of similar extent have been reported in recent history, however, air-photo studies indicated that similar sized landslides had occurred before.

The soil-stratigraphy and the piezometric conditions as delineated from testholes and piezocone testing (Eigenbrod, 1998) are summarized on the geological section Fig. 9. Four distinct soil strata can be identified, dipping gently towards the river. Of particular importance is the sandy silt layer which extends continuously from the uphill rock outcrop to the river bank and probably acts as an aquifer or subsurface drainage for the area. In the upper slope portions the sandy soil layers contain less fines (less than 10%) and are more pervious then in the lower slope where the fine content goes up to 70%. Therefore, surface water from the upper slope portions which infiltrates into the sandy surface layer, may flow into the interconnected lower sandy silt stratum.

The groundwater conditions at failure are not known, but can be estimated from the piezometric pressures recorded subsequently between May 1990 and September 1993. A general decrease in pore water pressure was observed during the winter months and maximum values in spring and early summer. A sudden rise in piezometric head by 1.8 m within less than a week could be recognized in the upper slope at the end of March 1993 (See Fig. 10). Temperature measurements identified that the ground in the deforested upper slopes was thawed out about two months earlier than in the lower-lying forested slope portions, thus permitting early infiltration of surface water into the ground.

It could be demonstrated (Eigenbrod, 1998), that the retrogressive landslide which occurred in the soft sensitive glaciolacustrine deposits along the Nipigon River in April 1990, was associated with elevated pore water pressures in the slope, which were caused by increased recharge in the upstream slopes and restricted discharge in the lower slopes (See Fig. 11). From a FEM analysis of the flow-conditions in the slope it was concluded that a sudden pore pressure build-up uphill led to pore pressure increases further downhill with maximum pore pressures occurring one month later. In a total-stress FEM analysis a zone of elevated shear stresses could be identified at 6 to 8 metre depth as a result of unloading from the initial slope failure at the river bank. An effective stress evaluation indicated that for the elevated pore water pressures identified in early spring, together with the elevated shear stresses, a potential for liquefaction existed in the lower sandy silt layer, which lead to the retrogressive land slide observed in April of 1990. The potential for failure was not apparent at pore pressures which most likely were valid prior to deforestation of the upper slopes.

5. DISCUSSION

The three sites will be discussed by comparing them in terms of the risks related to landsliding, using a safety cube analysis (Amann et. al 1999).

In a safety cube the hazard scenarios of engineering problems (e.g. the factors mitigating landsliding), the associated risks and consequences and the appropriate safety measures can be conveniently correlated to each other (See Fig. 12).

5.1 Kaministiquia River slopes in Fort William deposits

Along the slopes cut into the Fort William deposits by the Kaministiquia River and its tributaries only shallow slides must be expected as a result of toe erosion by the river. Larger slides are possible, if very large loads were applied to the top of oversteepened slopes. The risks associated with sliding along these slopes are relatively large for locations within the limits of the City of Thunder Bay, where residential development often is close to the valley edge. The risks, however, are predictable, as the destruction of property will proceed at the same rate as the rate of undercutting by the river. Similarly the risk of loss of human life is not very high, as it is more associated with the destruction of engineering structures than with the sliding event. Landsliding can be easily avoided by preventing toe-erosion, e.g. by placement of riprap along the toe of the slope for erosion control and by restricting development close to the edge of the river bank. Where existing development is already close to oversteepened slope portions, structures may be abandoned and removed, or alternatively secured by appropriate retaining structures (DST, 1982). Such measures have been successfully implemented along the river slopes in Thunder Bay.

5.2 Shallow Slides in Kaministiquia

The shallow slides in the upper Kaministiquia River valley are directly related to forest clearing and the associated increase of the ground water table in the shallow aquifers found in the fractured upper crust of the Kaministiquia red clay slopes. The ground movements are slow and limited in size and as a result the risk to engineering structures, such as building and roadways is low and of concern only if they are located in the area or at the lower fringe of the slide. There is practically no risk to human life. Ground movements can be controlled by keeping the groundwater level low in the fractured surface zone, e.g. by punching vertical drains through the intact clay into the underlying sand and gravel horizon, or by a system of trench drains cut to the base of the sliding soil mass. The other option is doing nothing and avoiding or by-passing the slide. This is acceptable due to the small size of the slides, as long as low-density development is maintained in the area.

5.3 Nipigon River Slide

Even though small slumps and erosion features commonly occur along the Nipigon River, large retrogressive slides are not common events. The specific geological and topographic conditions which govern the development of liquefaction slides, are not prevalent over the entire area. If these conditions do not exist, the triggering human factors, such as deforestation and rapid-draw-down conditions in early spring, are of little consequences. When considering the risks associated with the Nipigon Landslide, it can be stated that the impact of the slide would have been very limited and of little concern, if the Trans-Canada fiber-optic cable would not have been ruptured and the Trans-Canada-Gas pipeline would not have been damaged. Thus, the degree of future development, will determine the required remedial measures such as detection of the critical areas by a geotechnical investigation, implementation of safety features (by-passing of critical areas; stabilisation and preventive measures and/or installation of early warning systems in the critical areas), or doing nothing and accepting fully the landslide risk.

6. SUMMARY AND CONCLUSION

The glaciolacustrine deposits along the N.W. shore of Lake Superior, even though soft and often highly sensitive, are generally not prone to slope stability problems. However, under certain geological conditions, human activities, such as deforestation, may lead to more extensive sliding. The associated risk to buildings, engineering structures and human life is moderate as long as the extent of human activities in the area remains limited.

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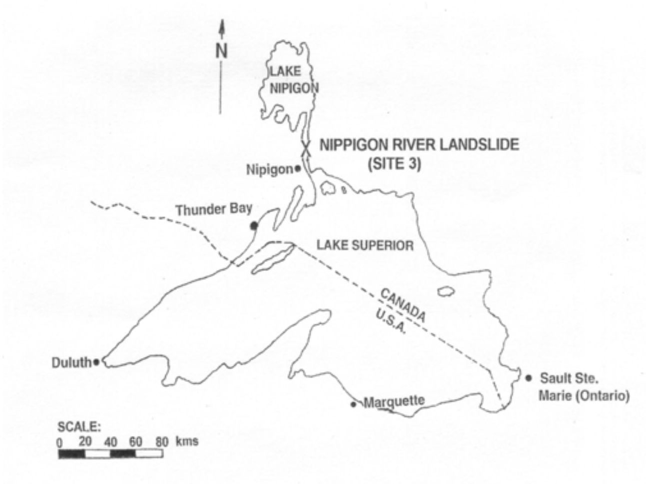

Fig. 1 Site plan of general area.

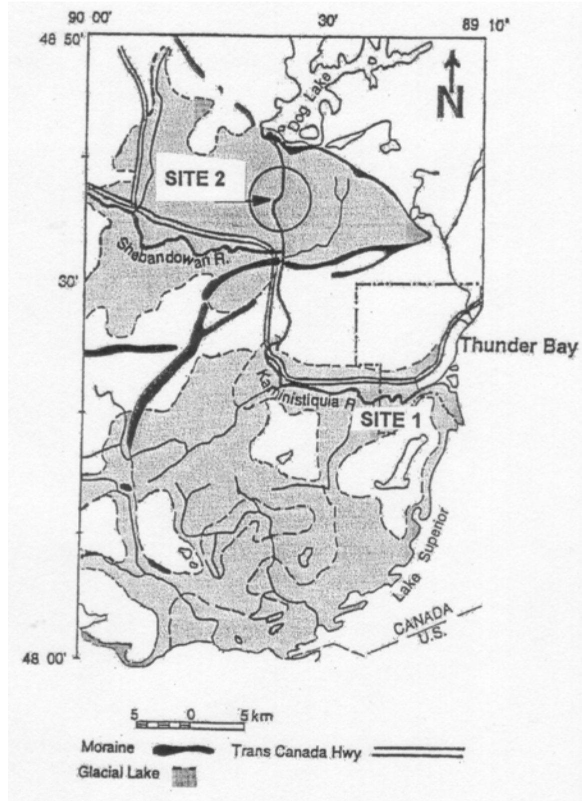

Fig. 2 Site plan of Kaministiquia River area (Sites 1 and 2).

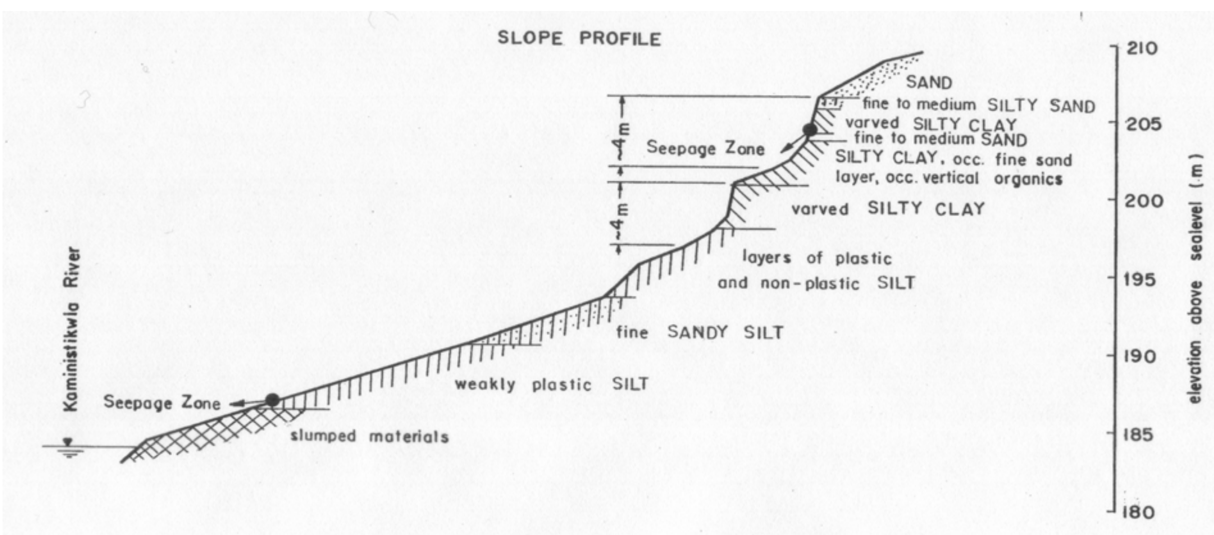

Fig. 3 Lower Kaministiquia River (Site 1). Geological section of typical slope (after DST, 1982).

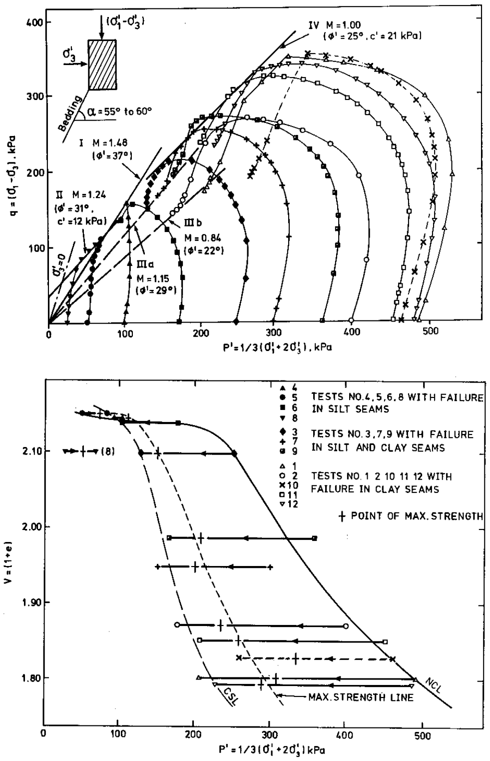

Fig. 4 Fort William Clay (Site 1). Undrained, effective stress paths in q-p' and V-p' space for shear along bedding planes.

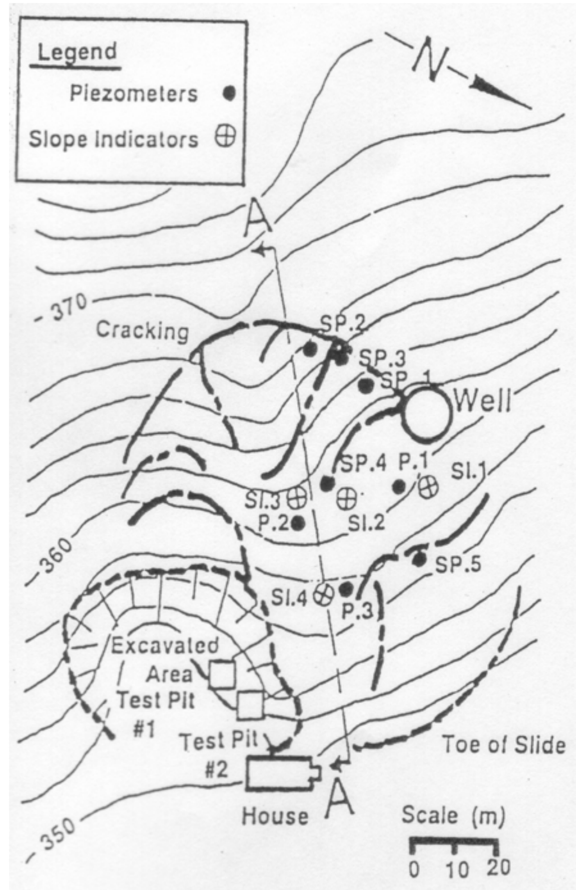

Fig. 6 Shallow slides in upper Kaministiquia River valley (Site 2). Site plan of slope failure investigated.

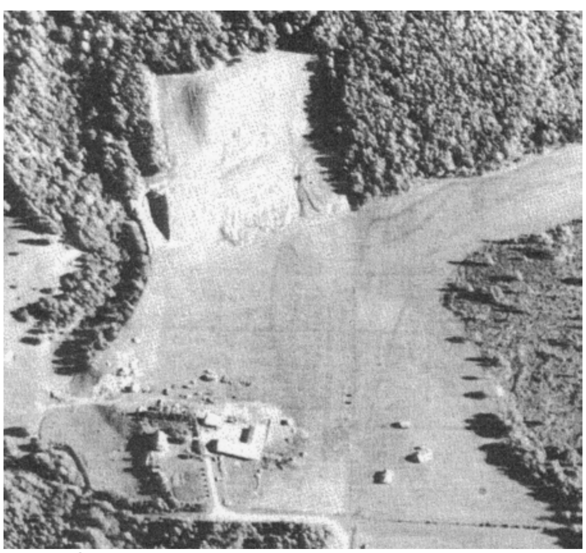

Fig. 5 Shallow slides in upper Kaministiquia River valley (Site 2). Air photo of typical slide.

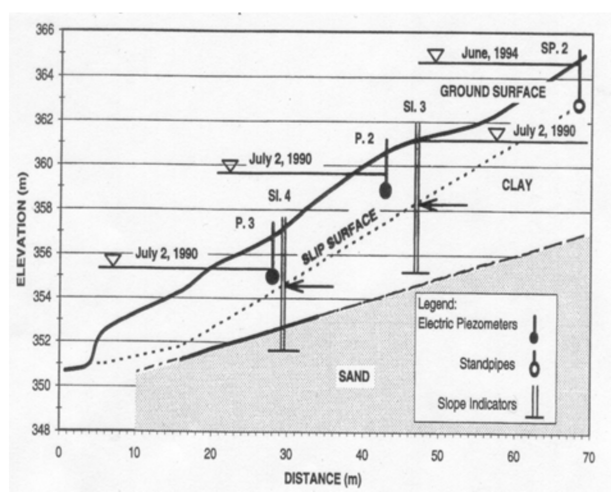

Fig. 7 Shallow slides in upper Kaministiquia River valley (Site 2). Geological section of slope with location of slip plane.

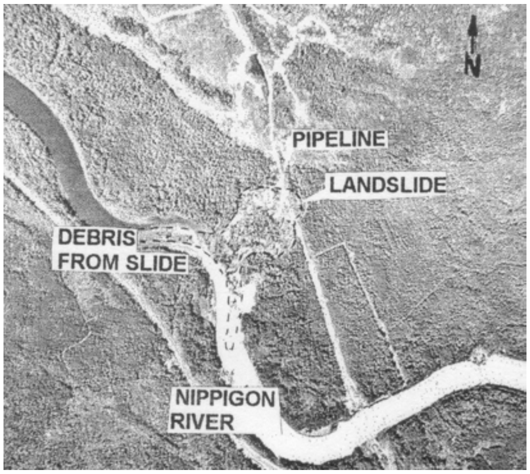

Fig. 8 Nipigon River landslide (Site 3). Air photo of area.

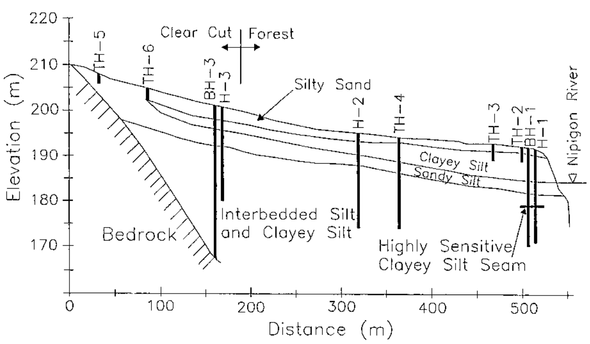

Fig. 9 Nipigon River landslide (Site 3). Geological section of slide.

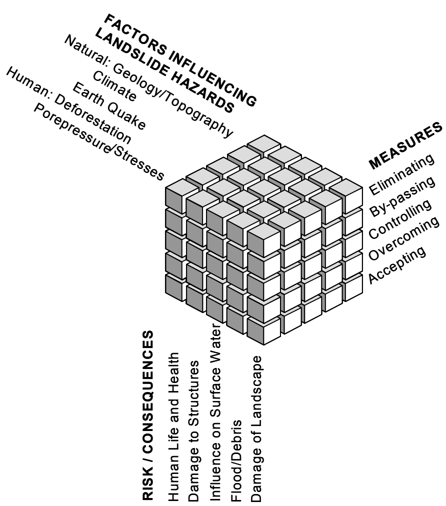

Fig. 12 Safety Cube (after Schneider, see Amann et al. 1999)

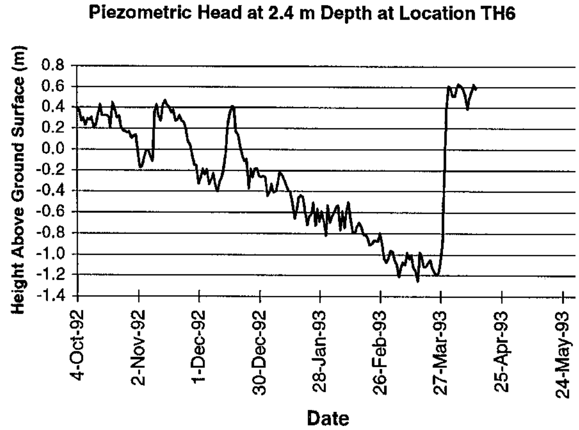

Fig. 10 Nipigon River landslide (Site 3). Pore water pressures at TH6 between Oct. 1992 and April 1993.

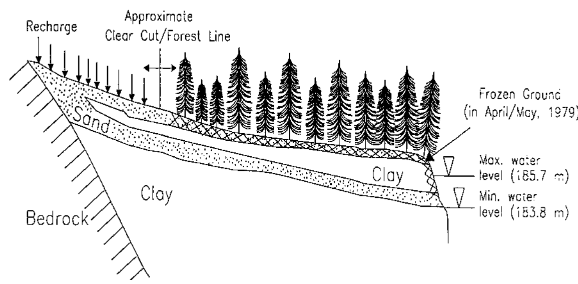

Fig. 11 Nipigon River landslide (Site 3). Schematic presentation of slope section with condition leading to high ground water levels at failure.