Nipigon River Landslide

R. B. Dodds
Trow Consulting Engineers Ltd., Thunder Bay, Ontario, Canada

J. P. Burak
Trow Consulting Engineers Ltd., Thunder Bay, Ontario, Canada

K. D. Eigenbrod
Lakehead University, Thunder Bay, Ontario, Canada

Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, Paper No. 2.49

Nipigon River Landslide

R. B. Dodds
Trow Consulting Engineers Ltd., Thunder Bay, Ontario, Canada

K. D. Eigenbrod
Professor in Civil Engineering, Lakehead University, Thunder Bay, Ontario, Canada

J. P. Burak
Trow Consulting Engineers Ltd., Thunder Bay, Ontario, Canada

SYNOPSIS A massive landslide occurred on the Nipigon River, north of the Town of Nipigon, Ontario, Canada in the early morning hours of April 23, 1990 and involved an estimated 300,000 cu m of soil. Although there was no loss of life, there were significant environmental and economic impacts.

Discussed in this paper are the investigations carried out after the slide, the events prior to and the factors contributing to, the slide.

INTRODUCTION

A landslide occurred on the Nipigon River, north of the Town of Nipigon, Ontario, Canada in the early morning of April 23, 1990. The slide extended almost 350 m inshore with a maximum width of approximately 290 m.

The force of the landslide caused soil to be pushed into the Nipigon River 300 m upstream and about the same distance downstream. The momentum of the movement was so great that piles of detritus were found in the trees on the opposite bank of the river, likely tossed up when the soil hit the opposite bank. The Nipigon River was approximately 100 m wide at the location of the failure. The islands, formed by the soils pushed into the river, redirected the current and caused subsequent erosion on the west bank of the river opposite the slide. The redirected current was likely the cause of several landslides further south, which occurred within one month after this slide.

A witness residing on the opposite bank heard trees breaking around 1:00 a.m. on April 23, 1990, which is assumed to be the time of the start of the slide. Failure took place retrogressively from the river bank inland and severed the fibre optics cable 300 m back from the river at precisely 4:01 a.m., 3 hours after the presumed start of the slide.

Discussions with two local residents, who know the river quite well, have revealed that prior to the slide the river bank at the location of the initial slide was approximately 6 m high and was undercut. The river at this location was reportedly very deep, with an estimated depth of almost 8.0 m. They also stated that it was common to have river fluctuations of up to 1.2 m over a 1 to 2 day period.

The flow rate in the river is controlled by a hydro-electric dam located 8 km upstream, varying from a high in excess of 500 cms to a low of 113 cms. Records show that the flow in the river was at 350 cms on April 18, 1990,

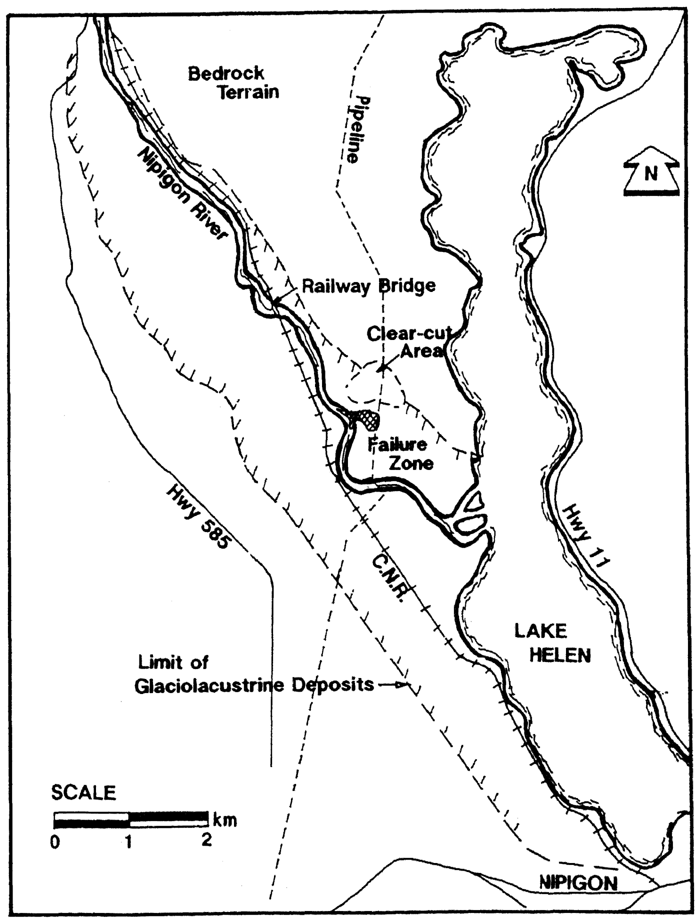

Figure 1 Location of landslide.

reduced to 150 cms by the end of the same day, increased to 320 cms by the middle of the next day and reduced to 120 cms by the beginning of the following day (April 20).

Although there was no loss of life or private property, there were significant environmental and economic impacts as a result of the slide. The movement of the soil caused a natural gas pipeline, which operates under a pressure of 7 MPa, to be displaced laterally up to 8.3 m westward toward the river, leaving it suspended without soil support for a length of 75 m. The pipeline was not ruptured, although it did have a sharp bend in it. A major fibre optics cable located in the pipeline right-of-way was ruptured. The increased sediment load in the Nipigon River caused difficulties with the residential water supply for the Town of Nipigon, located about 13 kms downstream, and was also the cause of difficulties with the boiler feedwater of a paper mill further downstream in the Town of Red Rock. The river is a sensitive fish habitat and it is suspected that the sediment adversely affected fish feeding routines for several years.

Soon after the landslide took place, Trow Consulting Engineers Ltd., (Trow) was commissioned by the owners of the natural gas pipeline and by the Ontario Ministry of Natural Resources (MNR) to carry out a soils investigation to:

a) establish the causes of the slide;

b) assess the risk of further slides taking place in the vicinity;

c) assess the feasibility of relocating the gas pipeline or rehabilitating the slide area; and

d) advise on the operational procedures of the hydro-electric dam located upstream of the landslide site.

Further investigation and study were carried out the following year by Ontario Hydro, operators of the dam upstream of the slide area, as well as by the Department of Civil Engineering of Lakehead University in Thunder Bay. Monitoring of the instruments and analyses are on-going. The investigations included air-photo interpretations, the advancement of boreholes, in-situ shear vane testing, triaxial testing, electric piezocone readings, installation of standard and modified piezometers and thermistors and installation of slope indicator casings. Various slope stability analyses were also carried out.

GENERAL GEOLOGY

The area studied is shown on Figure 1 and covers the stretch of the Nipigon River between the mouth into Lake Helen and a railway bridge upstream of the landslide. The predominant landform in this area is a glaciolacustrine plain and delta consisting of sands and silts. The local relief is relatively low with generally poor drainage conditions. The sides of river valleys which have been deeply incised into these fine grained deposits are frequently highly dissected and are susceptible to rill-and-gully erosion on freshly cut ditch slopes and highway backslopes. Higher and steeper natural and man-made slopes are subject to the development of small failures (Mollard and Mollard, 1981).

Figure 2 is a reproduction of an aerial photograph taken shortly after the occurrence of the landslide and shows the Nipigon River and the areas where failures have occurred. Some of the slumps are most likely due to the deflected current from this landslide under study here. It can be seen, as a common factor, that the slides are located on the outside sections of the river meanders - where bank erosion is at its greatest.

Air photo analyses showed that this area of the river has experienced bank failures in the past - even prior to the construction of the upstream hydro dam in 1931. This section of land is very wet with poor surface drainage, possibly adding to the relative instability of the banks in this area.

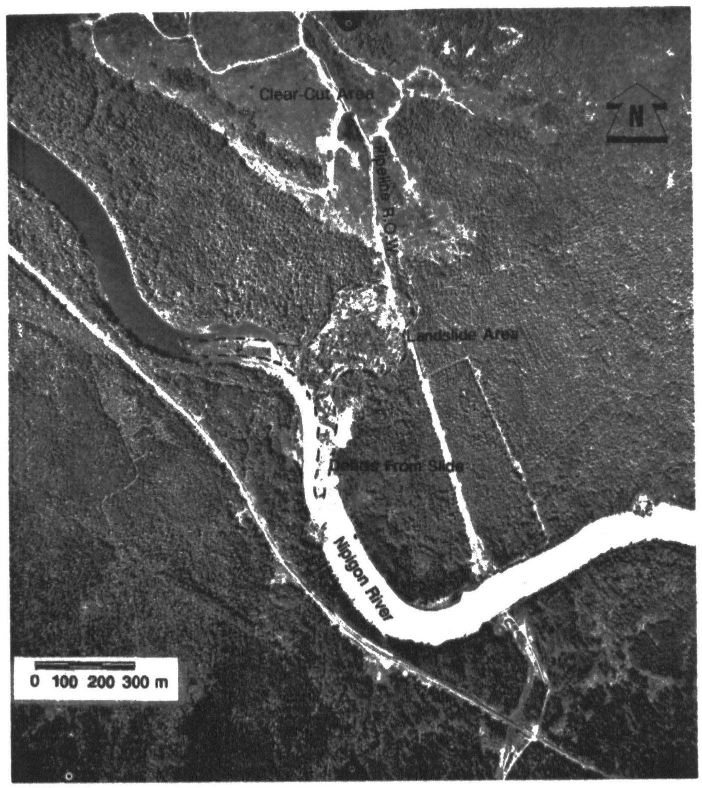

Figure 2 Aerial photograph of landslide.

FIELD WORK

The locations of boreholes and ground instrumentation installed as part of the investigation of the landslide are shown on a site plan on Figure 3 and in a stratigraphic section on Figure 4. Field work has been carried out independently by three agencies.

<u>Trow Investigation</u>

The field work for the initial and immediate investigation by Trow consisted of advancing two boreholes at the river bank immediately north of the failure zone. Borehole No. 1 was drilled to a depth of 21.9 m and Borehole No. 2 to a depth of 12.8 m. A slope indicator casing to measure on-going soil movements was installed in Borehole No. 1. One piezometer and one stand-

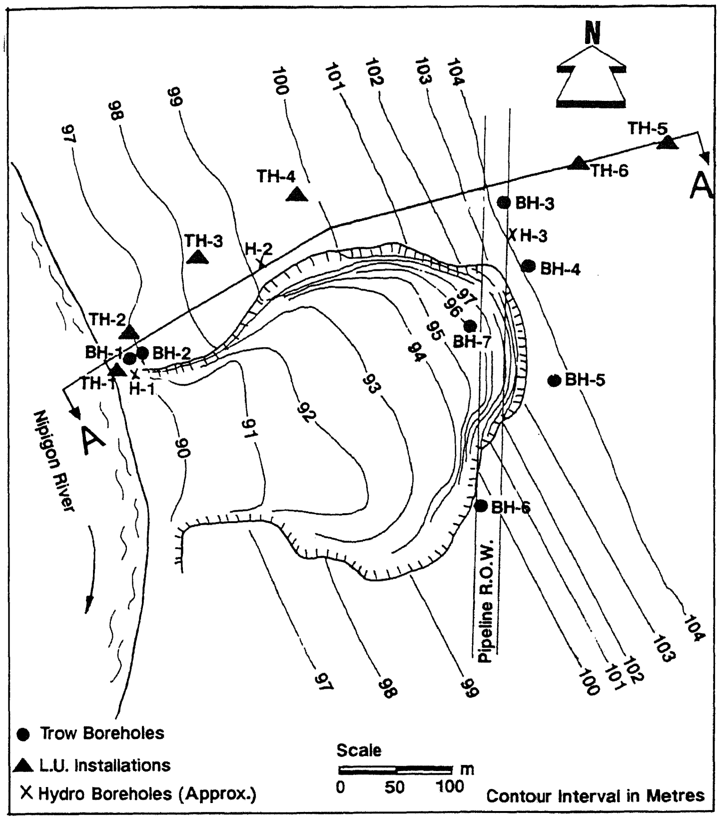

Figure 3 Site Plan

pipe were installed in Borehole No. 2 to at depths of 10.8 m and 6.4 m respectively. In addition, five boreholes were advanced near the natural gas pipeline.

Soil samples were taken at 1.5 m intervals with a standard split spoon sampler. Attempts were also made to obtain Shelby tube samples in the cohesive strata; however, due to the very soft-loose nature of the soils relatively undisturbed samples could not be obtained.

In the more cohesive soils, field vane tests were performed to measure undisturbed shear strength and remoulded shear strength. From these results the sensitivity of the soil was determined.

Ontario Hydro Investigation

In July 1991, 3 boreholes were advanced by Ontario Hydro (Boreholes H-1, H-2 and H-3). Undisturbed samples were recovered by means of a piston sampler. Soil stratigraphy and undrained strength profiling was achieved by means of an electric piezocone and Geonor shear-vane tests. Pore pressure dissipation tests were performed in clayey layers with the piezocone to estimate the coefficient of consolidation of the in-situ deposits.

Borehole H-1 was positioned 8 m east of the bank edge and close to Trow's Borehole No. 1. Borehole H-2 was located about 165 m east of Borehole 1 and Borehole H-3 was positioned just east of the landslide limit and TransCanada Pipelines right-of-way. Open type Geonor piezometers were installed at depths of 6 m, 9 m and 12 m at each

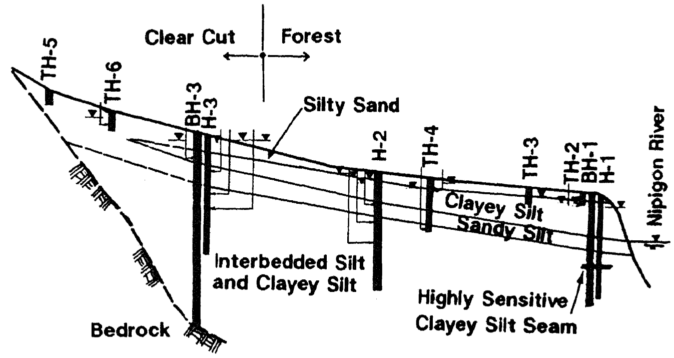

Figure 4 Stratigraphic Section

of the 3 boreholes. A standpipe piezometer was installed in Borehole H-3 at 3 m depth to monitor the perched water table in the surficial sand layer.

Lakehead University Investigation

A series of electrical piezometers together with thermistors were installed by Lakehead University. Four modified semiconductor type piezometers were located in the upper sand at depths between 1.5 m to 2.0 m, and two vibrating wire type instruments were installed at 3.0 metre depth in the upper clayey silt and at 12 metre depth in the lower clayey silt. The ground temperatures were recorded at these points as well as at two additional locations in the clear cut area.

LABORATORY TESTING

Moisture contents, Atterberg limits and grain size analyses were obtained from selected samples as part of the Trow investigation. Table I presents a summary of these results.

TABLE I SUMMARY OF TROW LABORATORY TESTING

SAMPLE	W_L	W_P	I_P	I_L	w	% SAND	% SILT	% CLAY
BH-1 1.5 - 2.1 m						25	71	4
BH-1 3.0 - 3.6 m	45.5	24.7	20.8	0.33	31.5			
BH-1 4.9 - 5.5 m	39.0	18.9	20.1	0.97	38.4	2	74	24
BH-1 12.2 - 12.8m	22.5	15.7	6.8	2.87	35.2	1	69	30
Silt Block						30	64	6
Clay Block						4	68	28

Notes:
1. W_L - Liquid Limit
2. W_P - Plastic Limit
3. I_P - Plasticity Index
4. I_L - Liquidity Index
5. w - Moisture Content

SUBSURFACE CONDITIONS

Generally, a silty SAND stratum is overlying a clayey SILT, beneath which there is another silty SAND/sandy SILT layer. Beneath the lower SILT/SAND, interbedded, very soft, SILT and more plastic CLAY/SILT layers are found. The subsurface conditions can be seen on the geological section shown on Figure 4 and the river bank section shown on Figure 5.

The characteristics of each of the main soil types encountered are summarized in the following sections.

SILTY SAND

This stratum consists of generally loose silty SAND, and ranges in thickness between 1 m to 3 m.

CLAYEY SILT

The clayey SILT stratum generally has clay fractions ranging from 20% to 30%. It characteristically has very high moisture contents, usually near, or even higher than its liquid limit. Measured liquid limits range from 22% to 39% with plastic limits in the order of 16% to 20%.

In-situ field vane tests show that the top section of this stratum has peak undrained shear strengths ranging from 48 to 86 kPa and sensitivities in the order of 2 to 4. Below this, in the more soft and saturated zone, the in-situ strengths range from 19 to 34 kPa with moderate sensitivities ranging from 1.9 to 2.9. The field vane shear values were considerably higher than the piezocone and laboratory triaxial strength values obtained by Ontario Hydro (Radhakrishna et al, 1992). The effective strength parameters of this unit as determined by consolidated undrained triaxial compression tests are $c' = 12.8$ kPa and $\phi' = 30°$. (Radhakrishna et al, 1992)

The boreholes near the gas pipeline showed this stratum to be very soft and wet throughout its depth with the same relatively low shear strengths yet with relatively high sensitivities. This renders the soil subject to easy lique-faction and failure upon disturbance.

SANDY SILT

The sandy SILT stratum generally has sand fractions ranging from 14% to 30%. It is generally compact with SPT values of 18 to 35 blows/ft and moisture contents ranging from 18% to 22%. The stratum appears to be stratified with evidence of interbedded fine sand and silt. This stratum is 3 to 5 m thick.

INTERBEDDED SILT AND CLAYEY SILT

At greater depth there is an interbedded SILT and clayey SILT stratum. The upper portion of this stratum is stiffer but with increasing depth the silt becomes very soft with a moisture content in the order of 33%. The clayey SILT is darker grey in colour and stiffer than the SILT, with higher plasticity and a moisture content in the order of 56%. The SILT layers are up to 150 mm in thickness, while the clayey SILT layers are generally 25 to 50 mm in thickness. In-situ vane tests showed that this stratum has an undrained shear strength ranging from 19 to 34 kPa. The remoulded strength of this layer ranges from 1.3 to 2.6 kPa indicating a high sensitivity of 10 to 15. A thin, very soft, light grey clayey silt zone of very high sensitivity was encountered at 12.0 m depth.

The liquidity index of the entire unit is greater than 1.0 with a strong susceptibility for liquefaction upon disturbance. The effective strength parameters of this unit near its top portion are $c' = 5$ kPa and $\phi' = 25°$ (Radhakrishna et al, 1992).

The boreholes advanced in the area near the pipeline did not indicate the stiffer clayey SILT in the upper portion of the stratum as was found in the Trow Boreholes, Nos. 1 and 2, but showed low undrained shear strengths throughout its depth (9.6 to 38.3 kPa as compared to 48.0 to 86.0 kPa indicated in the boreholes at the river's edge).

SLOPE INDICATOR READINGS AND INTERPRETATION

A slope indicator casing was installed in Trow Borehole No. 1 to a depth of 21.9 m. Readings were subsequently taken between May 25 and June 25, 1990. The readings indicated that there were movements still taking place within 16 m of the surface in the general direction of the river several months after the slide occurred.

SLOPE STABILITY

Trow Analysis

A stability analysis was performed by Trow on the river bank configuration as drawn in Figure 5. The analysis was performed using the commercial program SLIDE, developed at the University of Toronto. This program is based on Bishop's Simplified Method of Slices with circular slip surfaces.

The piezometric level used in the analysis by Trow was based on the water levels measured in the standpipes and piezometers after installation as well as on field observations and information from the borehole logs. Two water levels were considered:

1) a high groundwater level based on seepage zones observed near the toe of the slope immediately after the slide took place and on zones of free water found during borehole sampling; and

2) a low groundwater level as measured from the standpipes and related to the lowest river elevations measured during the field program.

Material Parameters

Because undisturbed samples could not be obtained in the Trow investigation, the strength parameters were estimated from the Atterberg values, the in-situ vane shear tests, and correlations with the 'N' values from SPT tests.

The soil parameters used in the stability analysis are summarized in Table II.

TABLE II Soil Parameters Used In Stability Analyses

Soil Type	Unit Weight kN/cu.m	Friction Angle ϕ'	Cohesion c' kPa
Sandy SILT	17.6	30°	0
Clayey SILT	19.0	28° (30°)	30 (12.8)
Very Soft Clayey SILT	18.2	28° (25°)	0 (5)

Note: Numbers in brackets are Ontario Hydro's values for comparison

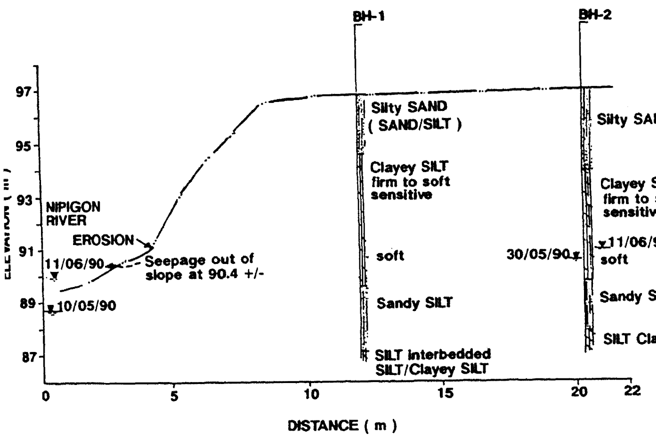

Figure 5 River Bank Configuration

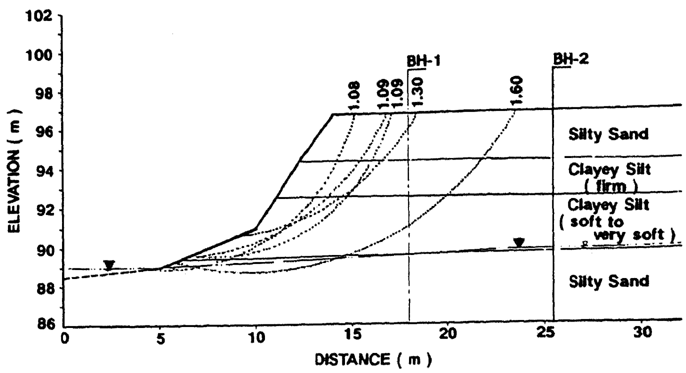

Figure 6 Slope stability analysis - Case 1

Results of the Slope Stability Analysis

This stability analysis was used for discussion purposes only and as an indication of how the river bank's stability would be affected by different groundwater conditions.

In particular it was attempted to determine the groundwater conditions at the time of failure.

Four main cases were investigated and the results are presented on Figures 6 to 9, showing the slope configuration and the respective groundwater conditions used. Each of the cases is described below:

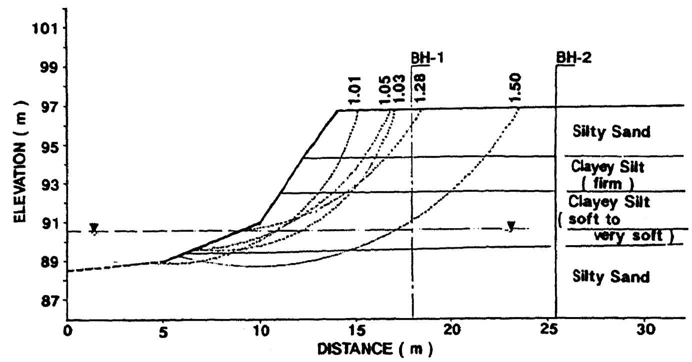

Figure 7 Slope stability analysis - Case 2

Case No.1

In Case No. 1 the conditions of the slope at the time of the soils investigation were investigated with a low river water level (prior to increase of flow) and the lowest measured groundwater elevations in the standpipe and piezometer. The results are shown on Figure 6.

Case No. 2

In Case No. 2 the condition of a high river level with the groundwater level at the same level as the river elevation was analyzed. The results are shown on Figure 7.

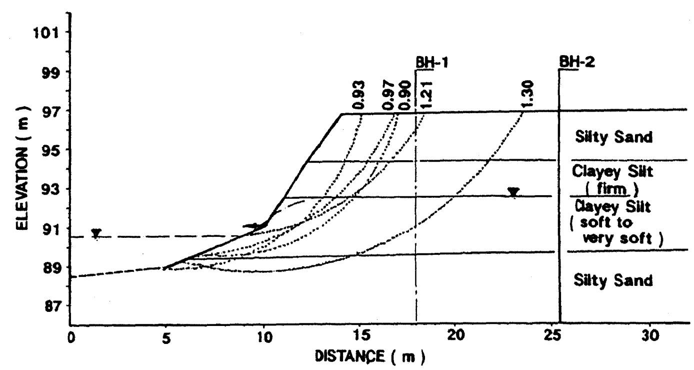

Figure 8 Slope stability analysis - Case 3

Case No. 3

In Case No. 3 spring conditions were simulated with high river levels and a high groundwater level with seepage at the toe, as was observed during the field program. These results are shown on Figure 8.

Case No. 4

In Case No. 4 spring conditions were simulated with a high groundwater level, including seepage at the toe of the slope, and low river levels for minimum flows. This situation is similar to a drawdown condition. The results for this case are shown on Figure 9.

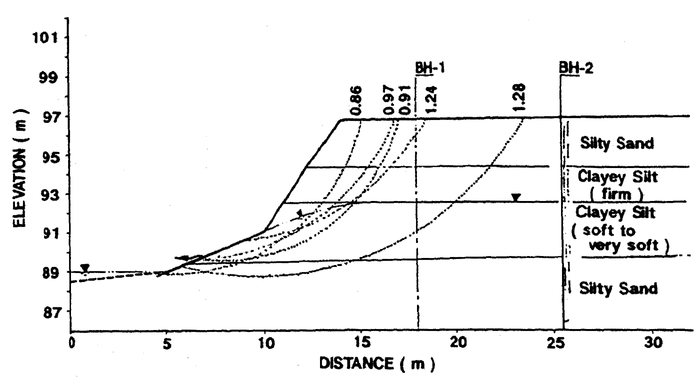

Figure 9 Slope stability analysis - Case 4

The results of the analyses are summarized in Table III.

TABLE III Summary of Stability Analysis

Case No.	Groundwater	River Level	Minimum Factor of Safety
1	low	low	1.08
2	same as river	high	1.01
3	high	high	0.93
4	high	low	0.86

Ontario Hydro (Radhakrishna et al, 1992) also performed a stability analysis on a similar slope configuration using the commercial computer program PC-SLOPE by GEO-SLOPE Programming Ltd., applying Bishop's simplified method along with the soil suction option. The Factors of Safety for various conditions were determined and compared with a base case. Their results yielded Factors of Safety ranging from 0.85 to 1.1.

COMMENTS

The results of both analyses are very similar, with Factors of Safety close to one in all cases. The most significant drop in stability was found to be related to an increase of the piezometric pressure head by about four metres during spring. The results, which indicate that the river banks have marginal stability, even under the most favourable groundwater and river level conditions, are supported by evidence of numerous old and new slides throughout the glaciolacustrine deposit along the river.

Examination of the results show further that the most critical slip circles are near the toe of the slope. The surficial silty sand stratum is unstable in some areas, suggesting minor surface sloughing on the upper bank. The slip surfaces reaching further back into the slope showed much higher factors of safety.

Thus, the analysis indicates that the slide started as a small slip at the river bank and did not fail as a whole entity, but retrogressed uphill after initial failure occurred.

This assumption is also supported by the following evidence:

1. A witness residing on the west bank of the river opposite the slide area heard the trees cracking and breaking at the river bank around 1:00 a.m. on April 23, 1990 whereas the Bell Telephone fibre optics cable was severed at the pipeline at 4:01 a.m., indicating that the slide retrogressed from the river bank to the pipeline (300 m) in a 3 hour time period; and

2. the detritus in the slide pushed out into the river shows no evidence that materials from behind the riverbank was riding up over material in place, as would be the case if the slide had started at the pipeline.

The fact that the slide debris was moving against the very swift current of the river for about 400 metres before it stopped, as well as the lateral deflections of the gas pipeline, suggest that the forces mobilized by the soil movement were very large. It appears also that the movements of soil were intermittent and may have been interrupted by several pauses before the final movements occurred which caused the rupture of the telephone cable about three hours later.

The main causes for the retrogression of the slide appear to be higher than normal groundwater levels in combination with highly sensitive soil deposits. The slide kept retrogressing after the initial slip occurred at the river bank and the slope oversteepened. Blocked drainage due to the debris of the initial slide led to the continued series of failures.

The depth of the slide seems to be at, or near, the lower sandy SILT stratum and may be determined by a seam of highly sensitive soil which was detected at Ontario Hydro Borehole H-1 just below the lower sandy SILT stratum (shown on Figure 4). The slide kept retrogressing upslope until sufficient resistance was met. It appears that the pipeline helped to stop the landslide retrogression.

The higher than normal groundwater pressure existed at the time of the landslide due to warm weather and heavy rainfall immediately prior to the slide and the recent timber harvesting operations uphill of the slide area. During a site visit on April 24, 1990, the day after the slide occurred, it was apparent that the frost was out of the ground in the cut-over area with water ponding on the surface, whereas in the tree covered areas and in the landslide zone, the ground was still frozen. Thus, in the cut-over areas, surface water could penetrate into the ground, recharging the groundwater in the soils of the failure area, which were still capped and confined by the overlying frozen ground. Similarly, some recharging could also have taken place in the cleared pipeline right-of-way. Figure 10 illustrates these points.

The qualitative observations in April 1990 were confirmed by ground temperature measurements on May 12, 1992 at various locations within the forested area and within the clear-cut area uphill from the landslide. While the ground surface within the forested area was at, or below, 0°C down to a depth of about 0.5 to 1.0 metres, the ground within the clear-cut area was completely thawed, ranging between +10°C near the ground surface to a low of +3°C at 1.0 m depth.

It should be noted that the clear-cut area uphill of the landslide is a shallow south and southwest facing bowl-shaped slope which absorbs sun radiation in the early spring. Surface water from snow melt and precipitation is collected in this area, readily recharging the groundwater in the lower lying slope portions. The surficial soils in the clear-cut slope area are fine sands, which taper out towards the rock outcrops further uphill and most likely connect with the two sand horizons further downhill (see geological section on Figure 4).

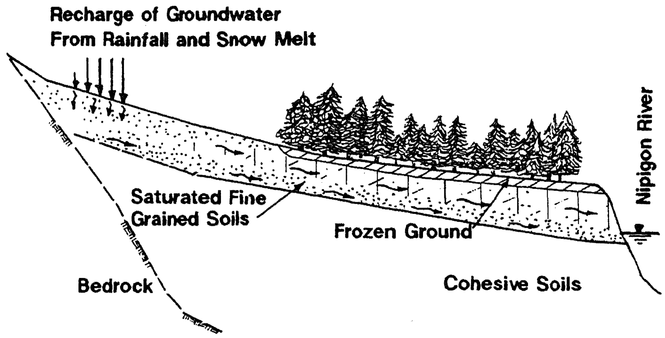

Figure 10 Model for soil saturation.

CONCLUSIONS AND RECOMMENDATIONS

The landslide that retrogressed from the east bank of the Nipigon River eastward to disrupt the gas pipeline likely started as a small slide at the river bank. Small slides are common along the outside bends of the Nipigon River, whereas retrogressive failures of the extent observed on April 23, 1990, are not.

Contributing factors to the river bank failure that started the landslide include:

a) toe erosion at the river bank as a result of the presence of easily erodible material and of high flow velocities;

b) higher than normal groundwater pressure in the river bank area, as evidenced by seepage out of the bank;

c) lowering of water levels in the Nipigon River which probably occurred more quickly than the river bank soils could drain, thus reducing the factor of safety and creating the potential for localized surface failures (rapid drawdown condition).

Probable factors that caused the localized river bank failure to retrogress back to the pipeline include:

a) the soil deposits in the area are weak and rather sensitive, leading to significant reductions in strength when disturbed;

b) the higher than normal groundwater pressure in the soils back from the river decreased their shear strength and stability. The high groundwater pressure appeared to be partially due to the spring weather conditions in 1990 and high groundwater recharge in the recently cut-over areas and along the gas pipeline right-of-way.

The nature of the glaciolacustrine soil deposits which this section of the Nipigon River traverses makes this area prone to bank failures. New slides will likely continue to occur at the river banks, as they have in the past before any development. It is possible that some of the small slips could result in large retrogressive failures such as the April 23, 1990 landslide, if similar conditions would develop.

Man-caused activities that may have contributed to one or more of the factors mentioned above include:

a) frequent rapid changes in the river level due to dam operations would increase the risk of small localized bank slides. In an uncontrolled river, fluctuations would be less frequent and would take place over a longer period of time, allowing the river bank soils to drain;

b) tree harvesting uphill of the area contributed to earlier, and increased, infiltration and thus to high groundwater pressures in the soil deposits downslope;

c) the cleared right-of-way constructed by the gas pipeline company could also have contributed to increased infiltration and thus to high groundwater pressures.

The following recommendations were made to reduce the risk of both small local river bank slips and larger retrogressive type movements:

1) Tree harvesting uphill from the river within the glaciolacustrine deposits should not take place unless an engineering study is undertaken to develop strategies that would ensure that tree cutting would not contribute to increased groundwater recharge into these deposits.

2) Operation of Ontario Hydro's dams on the Nipigon River should be controlled such that rapid drawdown conditions along the river banks would be avoided or minimized at critical time stages. Engineering studies should be undertaken to investigate seasonal ground-water levels and groundwater drainage patterns in order to establish acceptable times within which river lowering would be less likely to trigger bank failures. These studies may establish operational restrictions to reduce bank failures at certain times of the year or under certain weather conditions.

3) Areas in which surface water was ponding along the pipeline right-of-way should be identified and drainage should be provided as needed.

REFERENCES

Mollard, D. G. and Mollard, J. D. (1981), Frazer Lake Area (NTS 52 H/SE), District of Thunder Bay; Ontario Geological Survey, Northern Ontario Engineering Geology Terrain Study 42, 30 p Accompanied by Map 5052, scale 1:100,000.

Radhakrishna et al (1992), "Nipigon River Landslide", First Canadian Symposium on Geotechnique and Natural Hazards, Vancouver, British Columbia.

Trow Ontario Limited (1990), "Geotechnical Investigation and Stability Analyses - Nipigon River landslide - TransCanada Pipelines", Job Reference No. F-90139-A/G

Trow Ontario Limited (1990), "Soils Investigation and Analysis - Nipigon River Landslide", Job Reference No. F-90141-A/G