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ENGINEERING GEOLOGY AND GEOTECHNICAL STUDY OF DRYNOCH LANDSLIDE, BRITISH COLUMBIA

D.F. VANDINE

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ENGINEERING GEOLOGY AND GEOTECHNICAL STUDY OF DRYNOCH LANDSLIDE, BRITISH COLUMBIA

Abstract

Drynoch landslide, a large earthflow along Thompson River in southwestern British Columbia, has been moving slowly for many hundreds of years and for the past one hundred years has disrupted transportation facilities which cross its toe. The slide has been noted and studied during the past century by numerous scientists and engineers. This report assimilates much of the pertinent existing data and presents an interdisciplinary engineering geology and geotechnical study of the slide based on airphoto interpretation, surface and subsurface site investigations, laboratory testing, and stability analyses.

The slide material is a mixture of Pleistocene glacial till, remoulded Tertiary sediments, and Cretaceous rock fragments. The origin of the slide is related to an excess accumulation of montmorillonite-rich Tertiary sediments and clay-rich glacial till (possibly from a remnant valley glacier), and topographic conditions suitable for instability. Drynoch landslide is probably a zonal infinite-slope type of failure, the result of highly plastic, montmorillonite-rich clay layers, the availability of groundwater, and the presence of high pore water pressures.

A suggested method to retard movement of the slide is by means of drainage wells installed to depths below the failure zones. One means of minimizing localized instability and erosion of the toe of the landslide is by toe loading.

Résumé

Le glissement de terrain de Drynoch, représenté par une vaste coulée de terre le long de la rivière Thompson (sud-ouest de la Colombie-Britannique) se poursuit lentement, depuis plusieurs centaines d'années. Depuis cent ans, il perturbe les services de transport qui traversent l'extrémité de la coulée. Au siècle dernier, de nombreux chercheurs et ingénieurs ont vu et étudié ce glissement. Dans le présent rapport, on fait une synthèse d'une grande partie de l'information disponible à ce sujet, et l'on présente une étude à la fois de géologie appliquée et géotechnique de ce glissement de terrain, basée sur la photo-interprétation, des études du site en surface et en profondeur, des essais effectués au laboratoire et des analyses de stabilité.

Le matériau déplacé est un mélange de till pléistocène, de sédiments tertiaires remaniés et de débris rocheux crétacés. Le glissement est dû à l'accumulation excessive de sédiments tertiaires riches en montmorillonite et de till très argileux (peut être issus d'un glacier de vallée résiduel); la topographie accentue l'instabilité du terrain. Le glissement de terrain de Drynoch est probablement un type de rupture zonale à pente infinie, résultant de la présence de couches argileuses très plastiques et riches en montmorillonite, de la présence d'eau souterraine, et des pressions élevées de l'eau interstitielle.

Pour ralentir ce glissement de terrain, on propose une méthode consistant à installer des puits de drainage au-dessous des zones de rupture. Un moyen de réduire au maximum l'instabilité localisée et l'érosion du pied de la coulée est d'appliquer une charge à cette extrémité.

ENGINEERING GEOLOGY AND GEOTECHNICAL STUDY OF DRYNOCH LANDSLIDE, BRITISH COLUMBIA

INTRODUCTION

This report presents the results of an engineering geology and geotechnical study of Drynoch landslide. The slide is located in the southwestern interior of British Columbia, on the east side of Thompson River, 8 km south of Spences Bridge and 180 km northeast of Vancouver, British Columbia. It is 5.3 km long and varies from 670 to 120 m wide. There are indications that it has been moving slowly for many hundreds of years. The slide debris consists of a mixture of glacial till, remoulded Tertiary sediments, and Cretaceous rock fragments. Its appearance resembles an ice glacier, and based upon the U.S. Highway Research Board Classification (Varnes, 1958, p. 20-47), Drynoch landslide is classed as a "slow earthflow".

Macoun (1877, p. 122), referring to Drynoch landslide and the historical Cariboo Road built around 1860, stated "a short distance below the bridge, the mountain is constantly sliding towards the river and even in the short space of time since the road was built it has been rebuilt thrice at a more secure distance from the river". Today the Trans-Canada Highway, the mainline of the Canadian Pacific railway, and associated transmission lines cross the toe of the slide (Fig. 1). Because the slide has been moving fairly regularly, albeit slowly, for at least the past 115 years, maintenance and realignment have been required to keep these facilities serviceable. Due to the importance of the facilities, the costs involved in maintenance and realignment, and the particular nature of the landslide, a review of previous studies and a detailed engineering geology and geotechnical study was felt necessary and resulted in this report.

Previous Work

The most extensive previous study of Drynoch landslide was conducted by British Columbia Department of Highways between 1957 and 1963. This work was primarily related to geotechnical aspects of the Trans-Canada Highway realignment across the toe of the slide (Brawner, 1957; Brawner and Readshaw, 1963). The department file (number M2201) on the landslide for the years 1957 and 1973 also contains useful information.

Figure 1. *Oblique aerial photograph looking southward down lower Thompson Valley and across the toe of Drynoch landslide (to left). Transportation routes from left to right are:*

1 - New Trans-Canada Highway;
2 - Canada Pacific railway;
3 - Old Trans-Canada Highway (note how the central portion has been disrupted);
4 - Thompson River (note how the slide has pushed the river against the west bank);
5 - Canadian National Railways.

(GSC 175698)

The landslide was noted and briefly described by a number of people at the turn of the century (Begbie, 1871; Macoun, 1877; Cambie, 1878; Dawson, 1896; Anon, 1909; and Drysdale, 1914). The anonymous article in Engineering Record (1909) was the first to refer to the slide as "Drynoch landslide".

Studies dealing with the regional Quaternary geology that cover the slide area include work by Mathews (1944), Anderton (1970), Fulton and Smith (1978), and Ryder (in press). Bedrock geology of the general area was mapped by Duffell and McTaggart (1952), Jones (1959), and Mamu (1974) studied particular aspects of the local bedrock geology.

Brief engineering geology studies of the landslide were carried out by Cadman et al. (1967) and Bawden (1972). Other references to the slide are found in Armstrong and Fulton (1965), Mollard (1972), and Nasmith (1972).

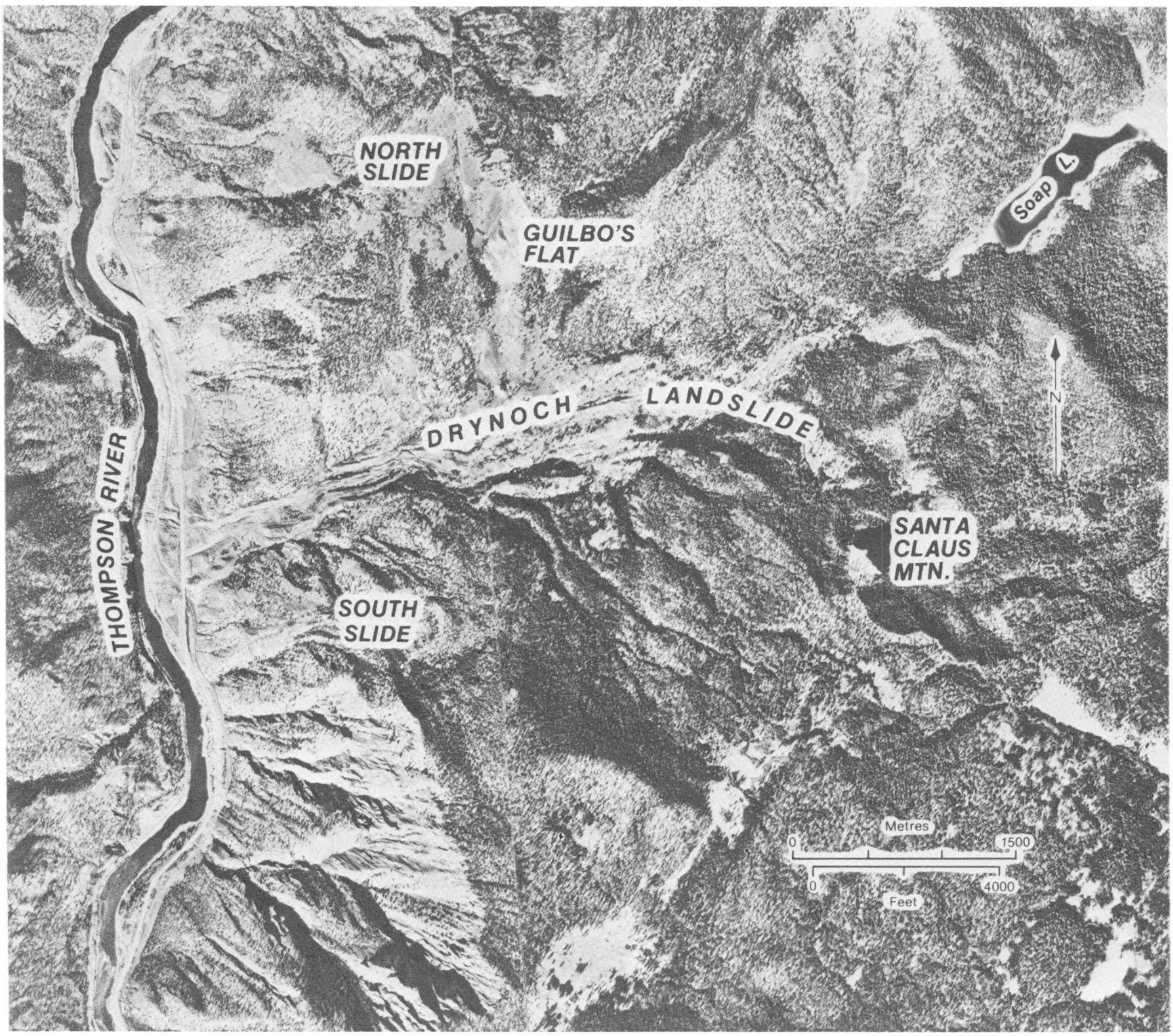


Figure 2. Photomosaic of Drynoch landslide. Some place names are unofficial (British Columbia Lands, Forests and Water Resources airphotos BC 5169-164 and 5170-004). (GSC 175662-5)

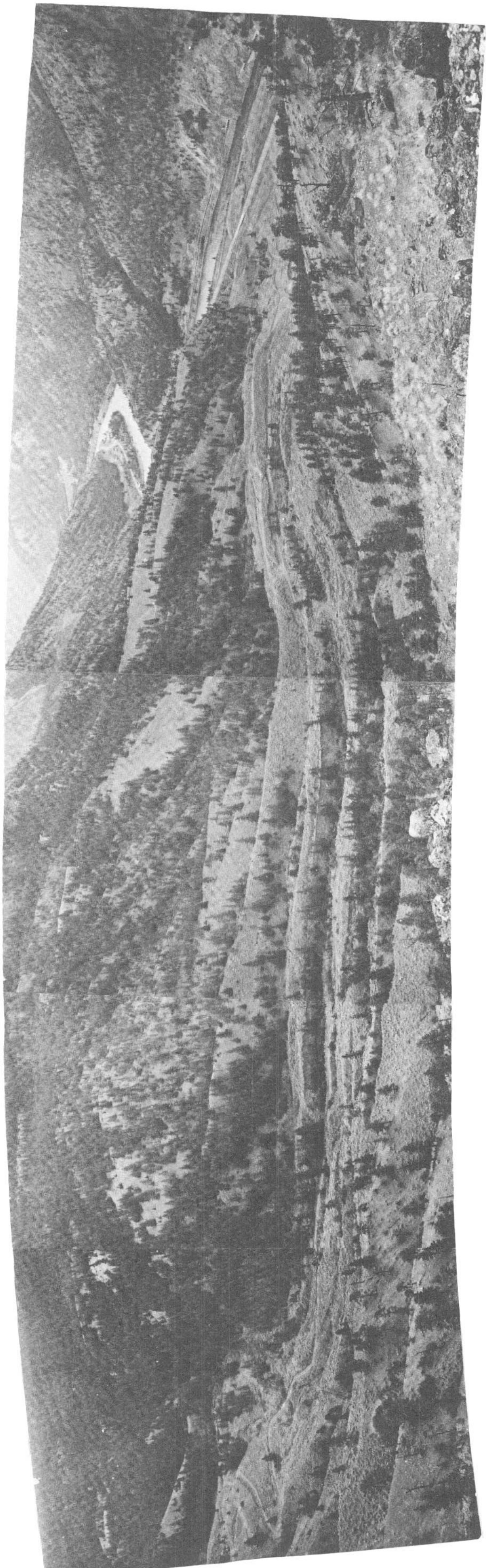


Figure 3. *Panoramic view looking southward over lower portion of Drynoch landslide. Note flow features and lines of vegetation. The slide is moving from left to right. (GSC 175662-5)*

Present Study

In light of the previous studies of Drynoch landslide and its regional geological setting, a broad engineering geology and geotechnical approach incorporating existing data was the basis for this study.

Initially an airphoto interpretation was made of the slide and surrounding area to delineate and summarize the bedrock and surficial geology, surface and subsurface drainage, and surface expressions and rates of slide movement. During summer 1973, these areas were mapped. Previous remedial construction works were located and their present effectiveness was assessed. Three testholes were drilled on the slide to obtain stratigraphic information on the slide material, to recover undisturbed* and disturbed soil and rock samples, to install piezometers, and to determine the depth to bedrock.

Subsequent laboratory testing, including grain size analyses, Atterberg limits, and moisture contents of selected disturbed and undisturbed soil samples was conducted. Some samples were analyzed for the mineralogy of the clay fraction. Estimations of soil shear strength for material from the failure zone were calculated by three methods:

1. both stress-controlled and strain-controlled, drained, triaxial tests were conducted on relatively large (8.5 x 20.3 cm) undisturbed soil samples;
2. drained, direct shear box tests were performed on both undisturbed and remoulded samples (residual shear tests also were done); and
3. a back calculation of slope stability using an infinite slope analysis was carried out.

The results of a monitoring program, conducted by British Columbia Department of Highways between 1957 and 1960, consisting of several lines of movement hubs and water levels in existing testholes, were analyzed along with climatic and hydrologic data, to determine if there was any correlation among these elements.

Existing data, together with observations and results arising from this investigation, lead to the determination of the probable cause and failure mechanism of the slide, and were used to suggest a method to retard its movement.

Acknowledgments

Field work was conducted during 1973 while I was with the Geological Survey of Canada. Laboratory testing was carried out at Geological Survey laboratories in Ottawa and at Queen's University of Kingston and was partially financed by a National Research Council Postgraduate Scholarship. British Columbia Department of Highways generously provided drilling equipment and allowed the use of the department files. Specifically I would like to thank J.S. Scott, Geological Survey of Canada, and R.J. Mitchell, Queen's University, for providing guidance during the project; D.G. McLean who assisted in the field; and staff members of Geological Survey of Canada and Thurber Consultants Ltd. who gave professional support.

* The terms "undisturbed" and "remoulded" when referring to the slide debris are relative terms only, since all the material probably has been disrupted from its original location.

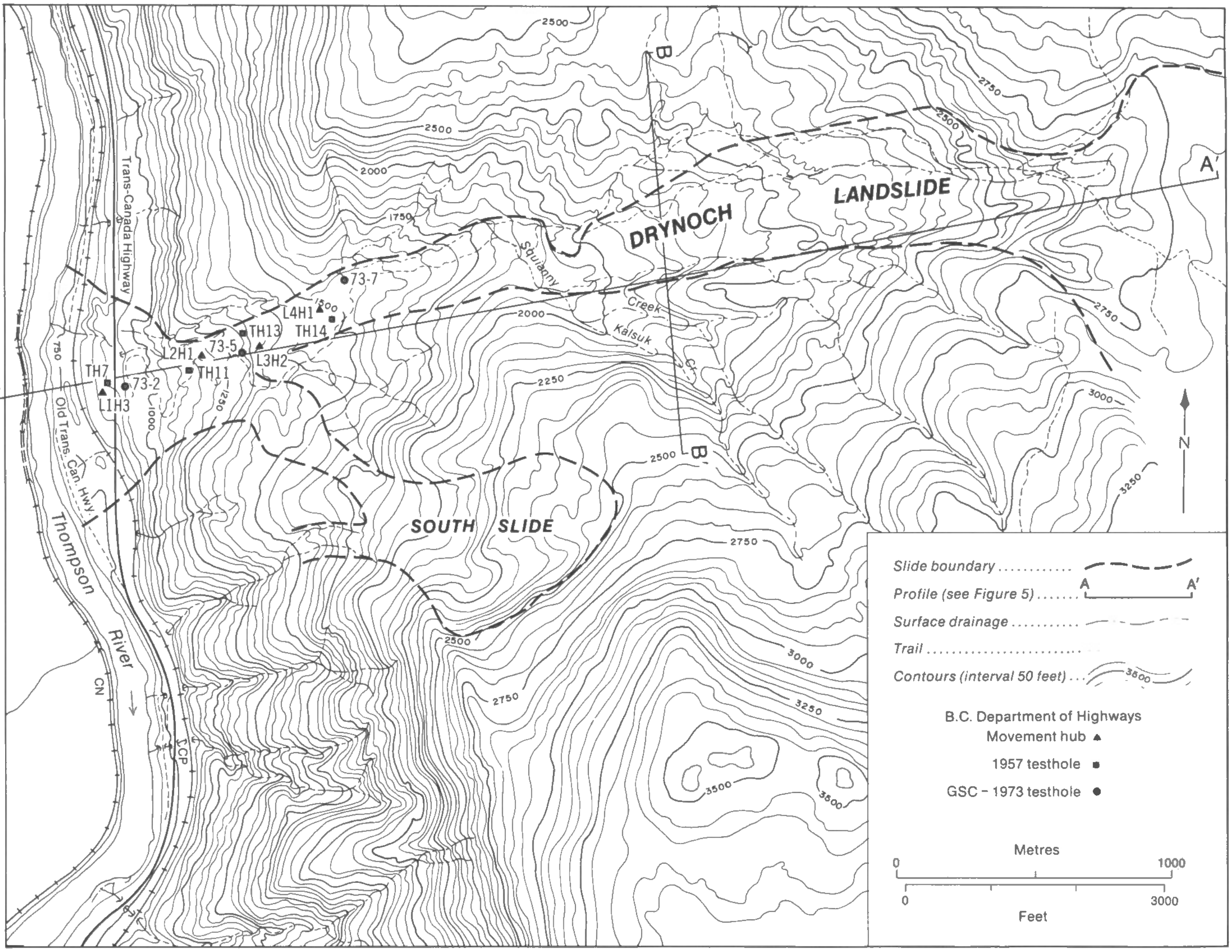


Figure 4. *Contour map of a major part of Drynoch landslide.*

SITE DESCRIPTION AND GEOLOGICAL SETTING

Slide Description

The appearance of Drynoch landslide closely resembles that of a valley glacier. Figures 2 and 3 show obvious flow characteristics which are accentuated by two large ridges bordering either side of the lower portion of the slide and by the lines of vegetation which follow surface and subsurface water courses. Longitudinal faults, transverse cracks and ridges, and a cirque-like headscarp complete the valley glacier analogy.

Total length of the slide from Thompson River to the farthest scarp is 5.3 km; the width varies from 670 m at the headscarp to 120 m at a point 800 m east of Thompson River. Volume of material involved has been estimated to be approximately 17 000 000 m^3 (Brawner and Readshaw, 1963, p. 5). The average elevation of Thompson River at the toe of the slide is 207 m a.s.l. and of the headscarp is 1070 m. The gradient of the slide surface varies from 25:1 to 3:1 and averages 6.5:1 (8.7°), with gradients steeper in the lower part (Fig. 4, 5).

The rate of movement varies with time and with location in the slide. The depressed central portion of the slide (Fig. 5B), flanked by scarps and ridges on either side, is moving the fastest and has been measured by British Columbia Department of Highways to move between 1 and 3 m/year. The department postulates that movement is seated on a failure plane or failure planes which are parallel to the surface at depths ranging between 7.5 and 18 m (Brawner and Readshaw, 1963, p. 8). At the toe, small isolated circular arc failures occur within the slide material.

During site investigations, many small surface expressions of movement were noted. At the break in slope of the slide surface (Fig. 5A), about 900 m from Thompson River, large transverse tensions cracks up to 2 m wide have developed (Fig. 6) as well as smaller ones near the toe (Fig. 8) where material is constantly being eroded by the river. Longitudinal shear cracks have developed where the more rapidly moving central portion of the slide adjoins the slower moving areas or relatively stable sides (Fig. 7). Vegetated portions of Drynoch landslide, especially those areas at higher elevations, indicate movement by a bent configuration of trees. In two areas within the slide, one approximately

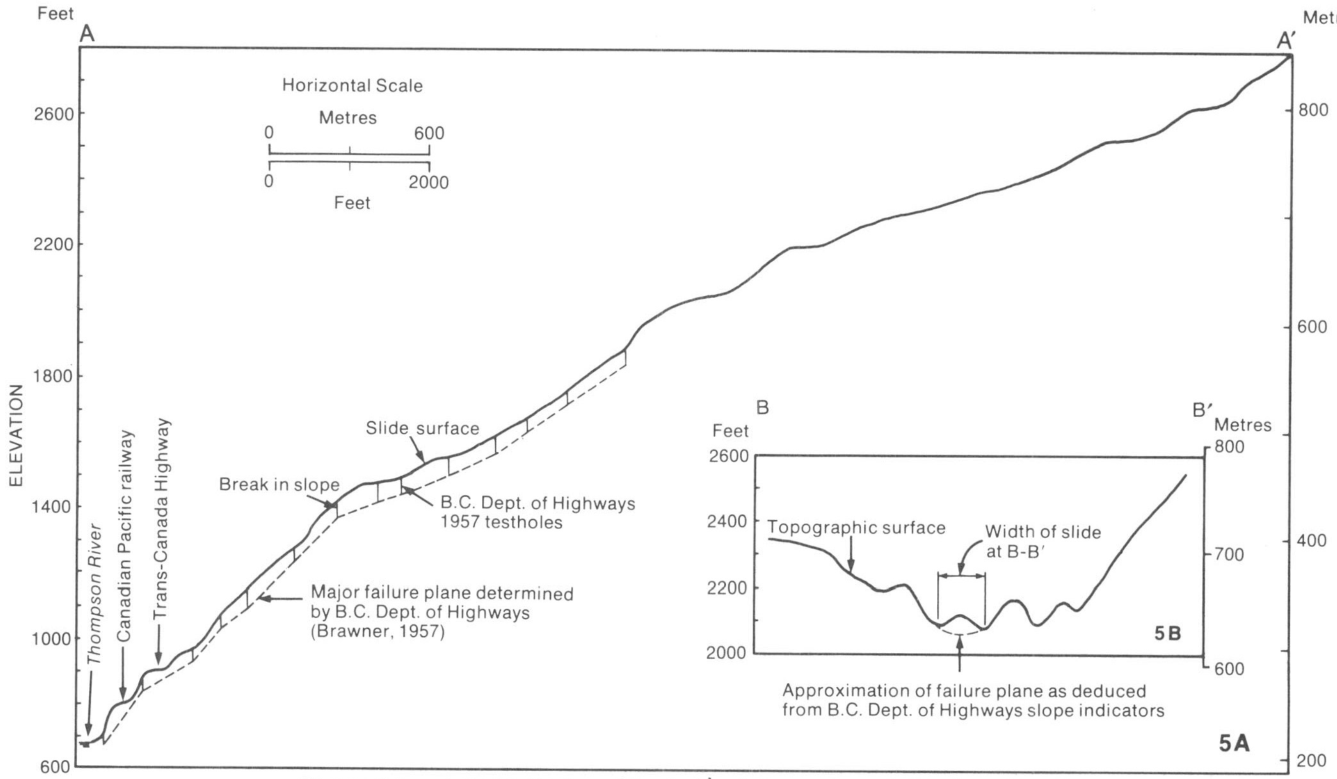


Figure 5A. *Longitudinal profile A-A^1 of Drynoch landslide (see Fig. 4).*

Figure 5B. *Cross-section B-B^1 across Drynoch landslide (see Fig. 4).*

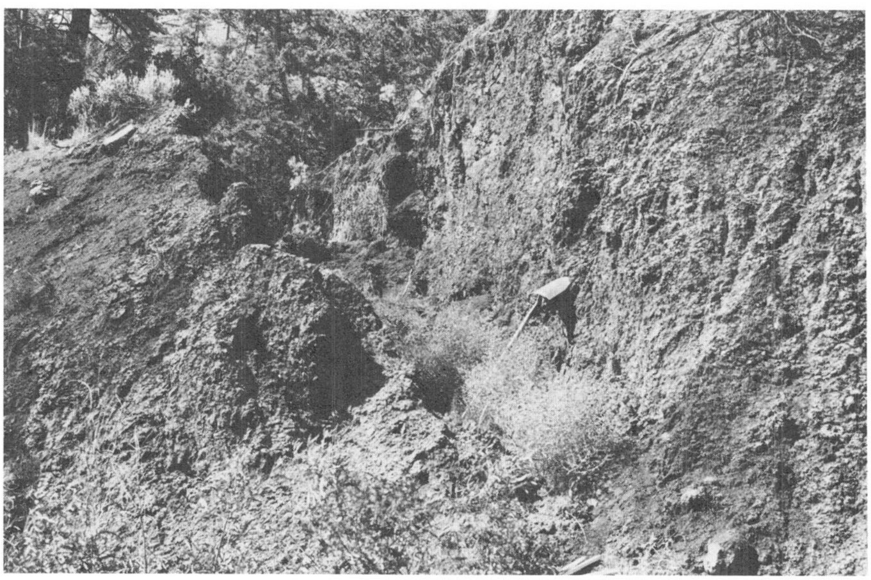


Figure 6. *Large tension crack at break in slope of Drynoch landslide; the crack is 1.8 m wide and more than 3 m deep. (GSC 175637)*

1.2 km east of Thompson River and the other in the gravel pit exposure, slickensides have been noted (Fig. 9) by various observers.

Physiography

Drynoch landslide is situated within the Thompson Plateau, a subunit of the Interior Plateau (Fig. 11). The topography varies from semi-mountainous to gently rolling with mountains rising to 1500 m and the valleys as low as 200 m a.s.l. The area is drained by Thompson and Nicola rivers, both of which are located in U-shaped, glacially eroded valleys. Situated in the rain shadow of the Coastal Mountains, climatically the landslide area is part of the "interior drybelt".

Other pertinent features include Guilbo's Flat, a small plateau to the north; Soap Lake, an evaporite lake to the northeast; and two smaller landslides, one to the north, the other bordering on the south side of the slide (Fig. 10). The small slides show many of the features of and have material similar to the large slide.

Bedrock Geology

Figure 12 and Table 1 summarize the bedrock geology of the Drynoch landslide area. Relatively massive and competent volcanic flow rocks of the Lower Cretaceous Spences Bridge and Kingsvale groups underlie the present limits of the slide and outcrop in the headscarp and along the flanks. One small outcrop of the Kingsvale sedimentary unit is found west of "Santa Claus Mountain" and consists of competent buff to green sandstone. Fragments of these volcanic and sedimentary rocks, especially the Kingsvale Group, are found throughout the landslide debris.

Tertiary sediments of the Kamloops Group are poorly exposed on Guilbo's Flat north of the slide. They consist of poorly indurated and friable impure sandstone, mudstone, shale, and coal (Fig. 13). This material occurs in the slide debris as nonindurated material having been broken down with movement to very small pieces.

Figure 7. Shear cracks developed within Drynoch landslide. Direction of movement is indicated by the head end of the hammer. (GSC 175690-1)

Figure 8. View looking south across the toe of Drynoch landslide showing tension crack of incipient toe failure. Shovel is 1.3 m long. Sage is typical of vegetation on lower portion of landslide. (GSC 175625)

Figure 9. Large slickensides and fluting found on failure plane of Drynoch landslide. Failure plane was exposed in 1961 during construction activity. Photo by British Columbia Department of Highways.

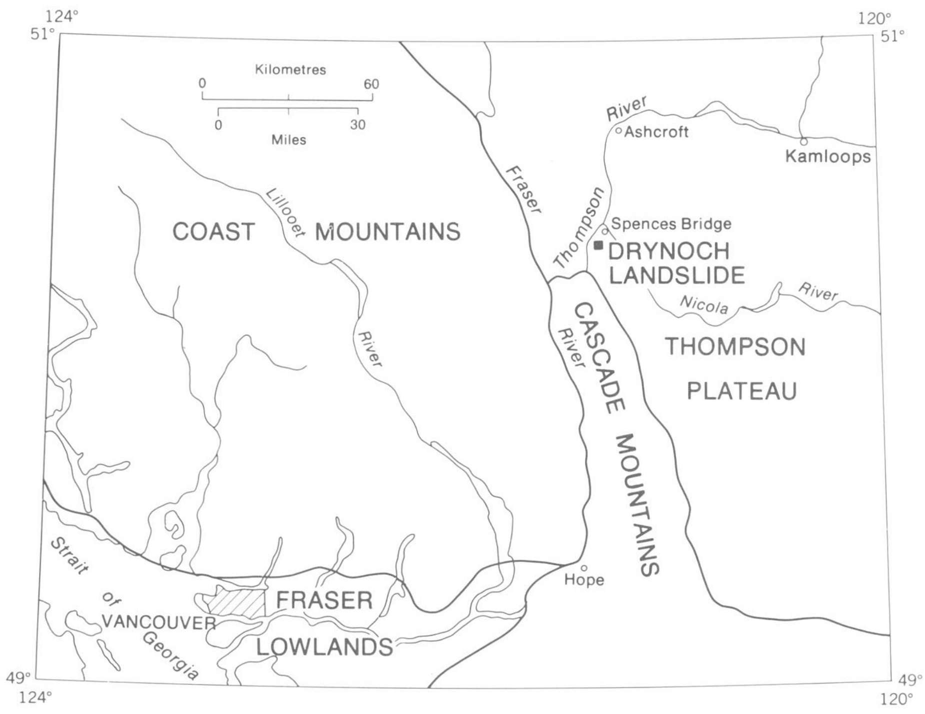
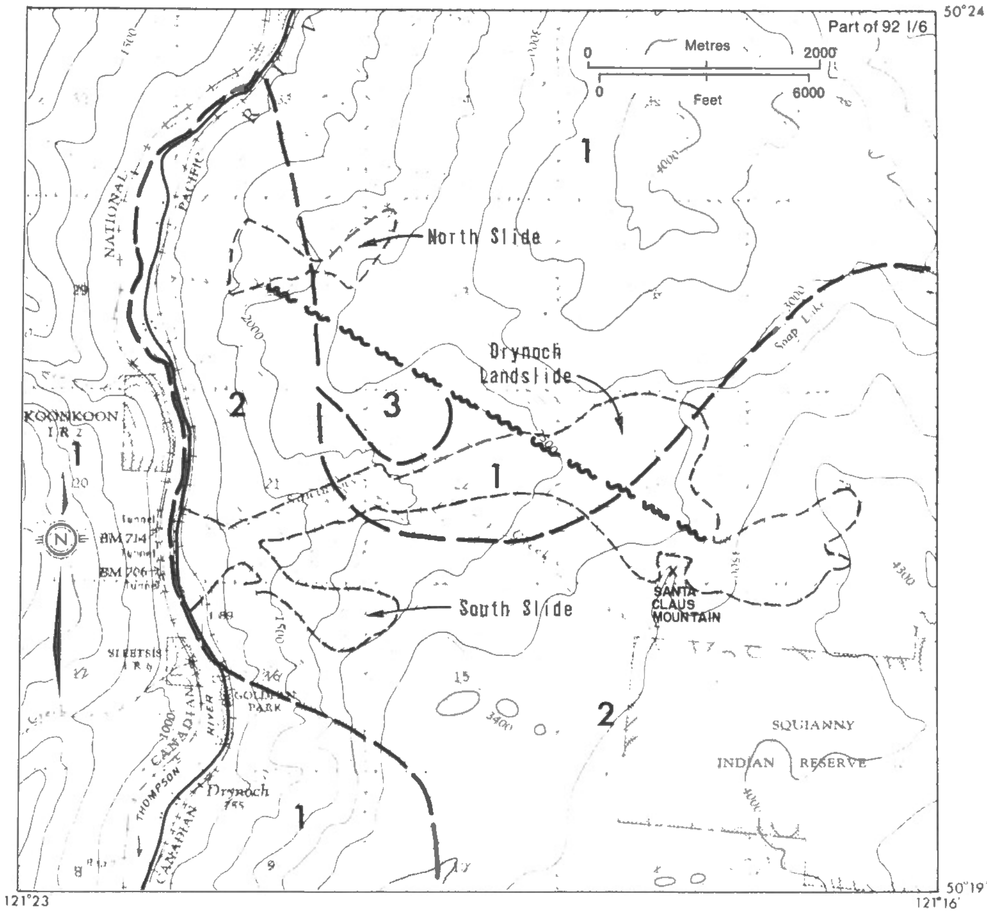


Figure 10

Oblique aerial photograph of Drynoch landslide, looking northeast. Summit in upper left of photo is 1340 m; Soap Lake is 920 m; Thompson River is 208 m a.s.l. Horizontal distance from Thompson River in the foreground to Soap Lake is 5.3 km. Photo by D.A. MacLean.

Figure 11

Location of Drynoch landslide and physiography of southwestern British Columbia (adapted from Holland, 1964).



TERTIARY

KAMLOOPS GROUP

3	*COLDWATER BEDS: sandstone, mudstone, shale, coal*

CRETACEOUS

KINGSVALE GROUP

2	*Andesite, basalt, agglomerate, tuff, breccia, minor sandstone*

SPENCES BRIDGE GROUP

1	*Andesite, basalt, rhyolite, agglomerate, tuff, breccia*

Geological contact (approximate) ....

Major fault ....

Figure 12. *Bedrock geology of Drynoch landslide area (after Duffell and McTaggart, 1952; Manu, 1974).*

A major fault in the area, separating Kamloops and Spences Bridge groups, strikes southeast and dips steeply to the southwest. A series of steeply dipping shear zones up to 0.6 m wide occurs south of and approximately parallel to the slide (Jones, 1959, p. 2). These zones cut the volcanic rock and are filled with "bright red, hematite-rich, clayey fault gouge" (Jones, 1959, p. 2). This same fault gouge was found at approximately 90 m depth by British Columbia Department of Highways during drilling. Other shear zones and faults probably exist but are difficult to find due to poor bedrock exposure and lack of marker horizons.

Depth of bedrock is variable throughout the slide area but commonly is within 120 m. Generally bedrock occurs at or very near the surface about the entire slide periphery. In the lower part of the slide, bedrock appears as large bluffs on either side; in the upper part it appears in the headscarps. Within the slide bedrock outcrops approximately 900 m and 1.6 km east of Thompson River. A short distance from the outcrop 900 m east of the river, holes drilled by British Columbia Department of Highways 120 m into the slide material did not reach bedrock. This outcrop divides the slide into two, the upper part having a lower surface angle than the lower part (Figure 5).

Based on drilling results, Horcoff (1960) suggested that a buried bedrock channel may exist under the toe of Drynoch landslide. He postulated that this was the channel used by Thompson River until debris from the slide pushed the river course west to its present location. The river presently flows in a bedrock channel.

A geophysical profile of the slide would be the only economic way to determine the depth to bedrock accurately.

Surficial Geology

Distribution of surficial material, based on airphoto interpretation and field work, is shown in Figure 14. The uplands surrounding the slide generally consist of a variable thickness of glacial till overlying bedrock. The till is composed of "unsorted, angular to rounded fragments of rocks, of very variable size and composition, in a clayey to sandy matrix" (Anderton, 1970, p. 25). On steep slopes a veneer of colluvium (a mixture of residual soil, local bedrock fragments, and some glacial till) covers bedrock. Along Thompson River and underlying the slide debris, at least two benches of well washed and stratified fluvial sand and gravel occur, separated by a till-like deposit. One exposure of the upper fluvial bench in turn is overlain by eolian sand, volcanic ash, alluvial fan debris, and slide debris (Fig. 15).

The slide debris is a heterogeneous mixture of pebbles (which appear to have been transported in glacial drift), rock fragments up to 1 m in diameter which were derived from local Cretaceous volcanic and sandstone bedrock, and a matrix of sandy clay (Fig. 16). There appears to be no continuity in the stratigraphy of the slide debris either in areal distribution or with depth, although some horizontal flow features exist.

In the central portion of the slide a series of ridges exist at the surface along both sides. These ridges (Fig. 17), which in appearance resemble lateral moraines, are regular and symmetrical in cross-section and up to 800 m long. The material within these ridges varies from uniform medium grained sand to a sandy clayey diamicton.

Table 1. Summary of geological formations

Period	Epoch	Group	Lithology
Quaternary	Pleistocene and Recent		glacial drift, glaciofluvial, eolian, volcanic ash, alluvial fan, slide debris
		unconformity	
Tertiary	Eocene or earlier	Kamloops Group (Coldwater Beds)	sandstone, mudstone, shale, coal
		unconformity	
			andesite, basalt, agglomerate, tuff, breccia
		Kingsvale Group	
			conformity
Cretaceous	Lower Cretaceous		sandstone
		unconformity	
		Spences Bridge Group	andesite, basalt, rhyolite, agglomerate, tuff, breccia

Figure 13. Outcrop of folded Tertiary sedimentary beds north of Drynoch landslide. Dark bands are mudstone and shale; light bands are sandstone beds. (GSC 175668)

Hydrogeology

Drynoch landslide is located in the Squianny Creek catchment basin, which is 16.1 km^2 in area. It is drained by Squianny Creek, Katsuk Creek, and small unnamed tributaries (Fig. 18), which are fed by surface runoff, groundwater, and one small apparently perennial spring southeast of the slide. Although many creeks are intermittent, all the water that they do carry ultimately reaches Drynoch landslide. Surface water follows gullies and crevasses formed by the slide; some becomes groundwater by flowing into tension and shear cracks within the slide and eventually enters Thompson River by following permeable substrata within or beneath the slide material. Both surface and some subsurface drainage paths can easily be traced by the resulting vegetation distribution and type.

Soap Lake and a number of smaller lakes located northeast of the slide are fed by groundwater and runoff. Testholes drilled by British Columbia Department of Highways encountered a bedrock ridge, 10 m high, separating these lakes and the slide; no natural surface or subsurface outlets for these lakes has been found. They are evaporite or alkali lakes, common in the interior of British Columbia, and characterized by salt deposits along their shorelines (Fig. 19).

GEOTECHNICAL INVESTIGATIONS

Subsurface Investigations

In 1906 the Canadian Pacific Railway advanced a number of borings near the toe of Drynoch landslide from which they inferred shallow bedrock at depths up to 6 m (Brawner, 1957, p. 1). British Columbia Department of Highways drilled extensively in the lower section of the slide in the late 1950's and early 1960's. A summary of their findings is outlined below (Brawner and Readshaw, 1963, p. 6-8):

1. "materials encountered throughout the slide generally comprise well represented amounts of clay, silt, sand, and gravel with occasional boulders...The soil profile varies considerably from hole to hole...The upper horizon...is of loose to medium density. Below this depth the material becomes increasingly dense. Material in this zone...is practically impervious and was generally found to be relatively dry";
2. "high underground pore water pressures were encounterd in practically all test holes. Considerable variation was noted for adjacent holes"; and
3. "the depth of the failure zone or zones varied from 7.6 m to 18.3 m and averaged 14.6 m".

Depth to the failure zone was determined initially by measuring the depth at which standpipes were bent or broken and later by slope indicators. The failure zone was found to be 4.5 to 6.0 m thick, and its cross-sectional profile was determined to be dish-shaped, intersecting the surface of the slide on the north and south sides (Fig. 5B).

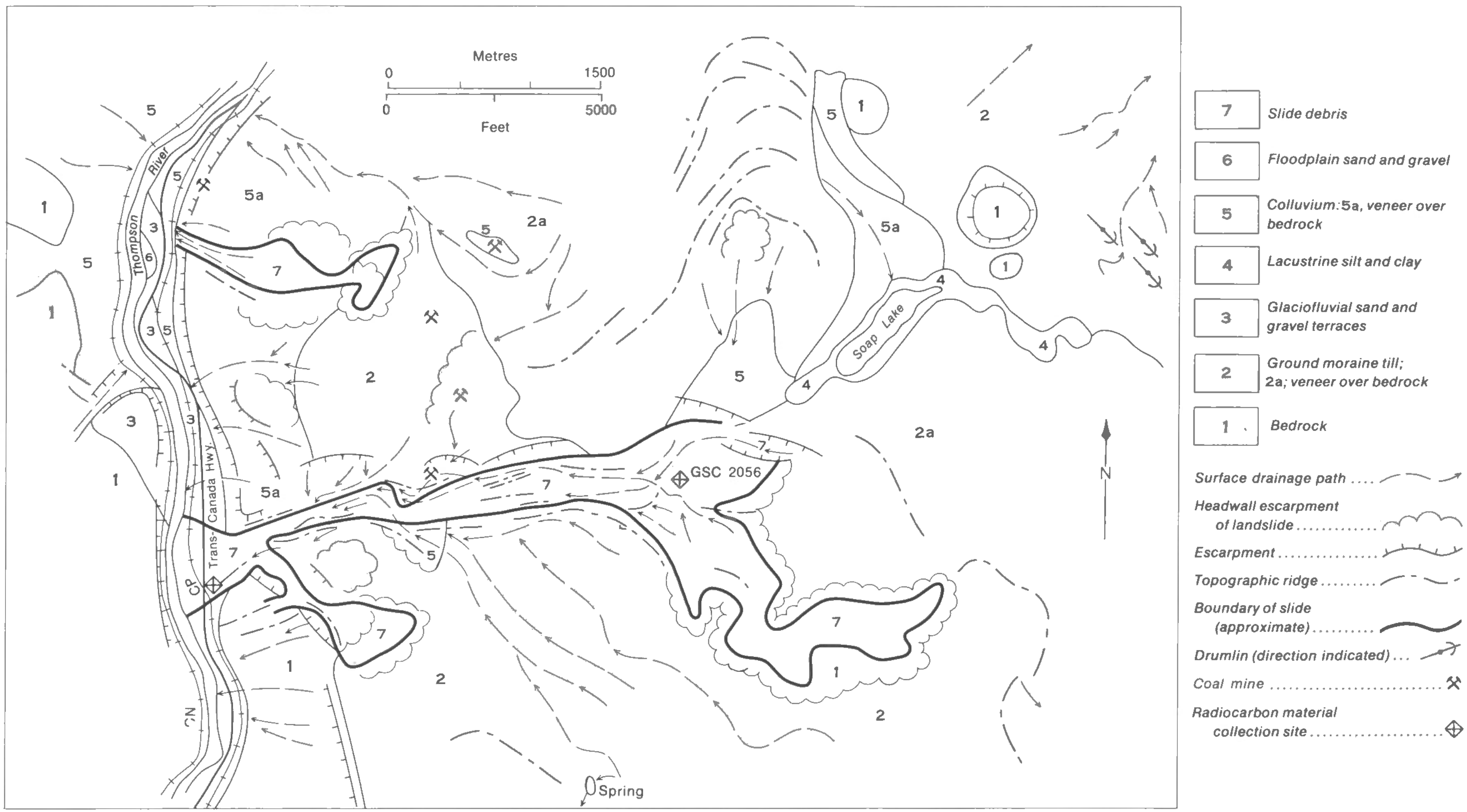


Figure 14. *Surficial materials of Drynoch landslide area based on airphoto interpretation.*

Figure 15

An exposure of unconsolidated material found in a gravel pit near the toe of Drynoch landslide. Shovel is approximately 1.3 m long. (GSC 175588)

Three testholes were drilled in 1973 (Fig. 4) with a BBS-2 rotary drill (Appendix 1). The soil profile indicates an erratic mode of deposition and possibly several successive slides, with fluvial sand and gravel deposited in the interim between sliding. The failure zones appear to correlate well with highly plastic, clay-rich horizons:

Testhole	Depth of Failure Zone
TH 73-2	1.5 - 7.5 m (5 - 25 ft)*
TH 73-5	8.7 - 13.1 m (28.5 - 43 ft)
TH 73-7	7.3 - 10.0 m (24 - 33 ft)

Coarse grained permeable zones above and below these horizons keep the fine grained material in the failure zones saturated. Bedrock was not encountered at the end of testholes 73-2 and 73-5 (16 and 20 m, 54 and 66 ft, respectively) but was encountered in testhole 73-7 (37.5 m, 125 ft).

Nests of piezometers were installed in each testhole to determine pore water pressure and hydraulic gradients within the more permeable material. Unfortunately, after installation, monitoring of the piezometers was not continued, therefore no new piezometric data were obtained.

Laboratory Program

During fieldwork 147 disturbed samples of rock and soil from the slide and surrounding area were collected; 40 disturbed soil samples, 28 undisturbed samples, and 3 m of cored bedrock were collected during the drilling program. A representative sample number of each type of soil was chosen for laboratory testing.

Engineering Properties of the Soil

Figure 20 shows the grain size distribution of 73 samples from the landslide area. The slide material appears to have an even distribution of gravel, sand, and fine grained material and grades finer than material from the surrounding area. In the fine fraction (Fig. 21), the slide material is almost equally divided between silt and clay, whereas the surrounding area contains more silt than clay.

Figure 22 indicates that there is little difference between the "activity" (plasticity index vs. % clay >0.002 mm) of material surrounding the slide and that of the slide itself and that neither are abnormally active.

* At one time the zone of failure in TH 73-2 was probably deeper but British Columbia Department of Highways removed large amounts of material from this area in 1961 in an attempt to stabilize the slide.

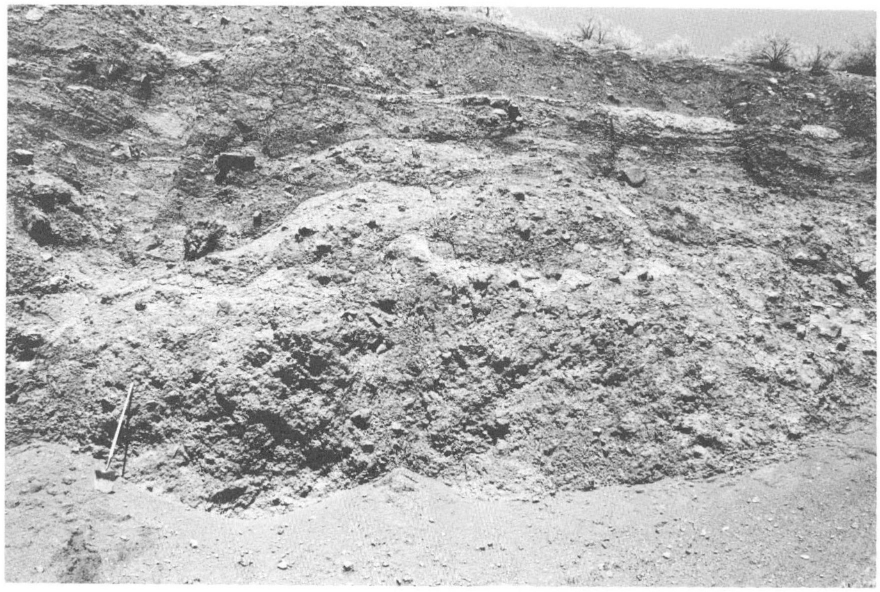


Figure 16

Slide debris exposed on the north side of Drynoch landslide. Note the variety of grain size of granular material and the flow patterns within the exposure. Shovel is approximately 1.3 m long. (GSC 175589)

Figure 17

A ridge, resembling a lateral moraine, on south side of Drynoch landslide. The slide is to the right of the photograph. Shovel is approximately 1.3 m long. (GSC 175646)

The natural moisture content of most soil samples tested is less than the plastic limit. This coincides with earlier results determined by British Columbia Department of Highways (Brawner, 1957, p. xiv).

Mineralogy of the Clay Fraction

During investigation of Drynoch landslide by British Columbia Department of Highways in the early 1960's three clay mineral analyses were carried out by Dr. R.E. Grim, University of Illinois, on samples of the slide material. The relative abundance of each mineral was similar for the three samples:

montmorillonite (sodium dominant)	80%
kaolinite	15%
quartz	5%
feldspar	trace

From these results he suggested that the clay fraction would probably have:

1. high colloidal properties;
2. high Atterberg limits;
3. high expansion on wetting and high shrinkage on drying;
4. high activity; and
5. little or no shear strength with modest amounts of water.

In 1972 the Geological Survey of Canada analyzed three samples from the area surrounding the slide for mineralogy of the clay fraction (Appendix; W.F. Bawden, unpublished report, 1972). Eighteen additional samples from the failure zone of the slide and several samples of nonindurated Tertiary sediments were analyzed in 1973 (Appendix 2). From these analyses it is noted that:

1. montmorillonite is the dominant clay mineral of the slide material, Tertiary sediments, and the material surrounding the slide;

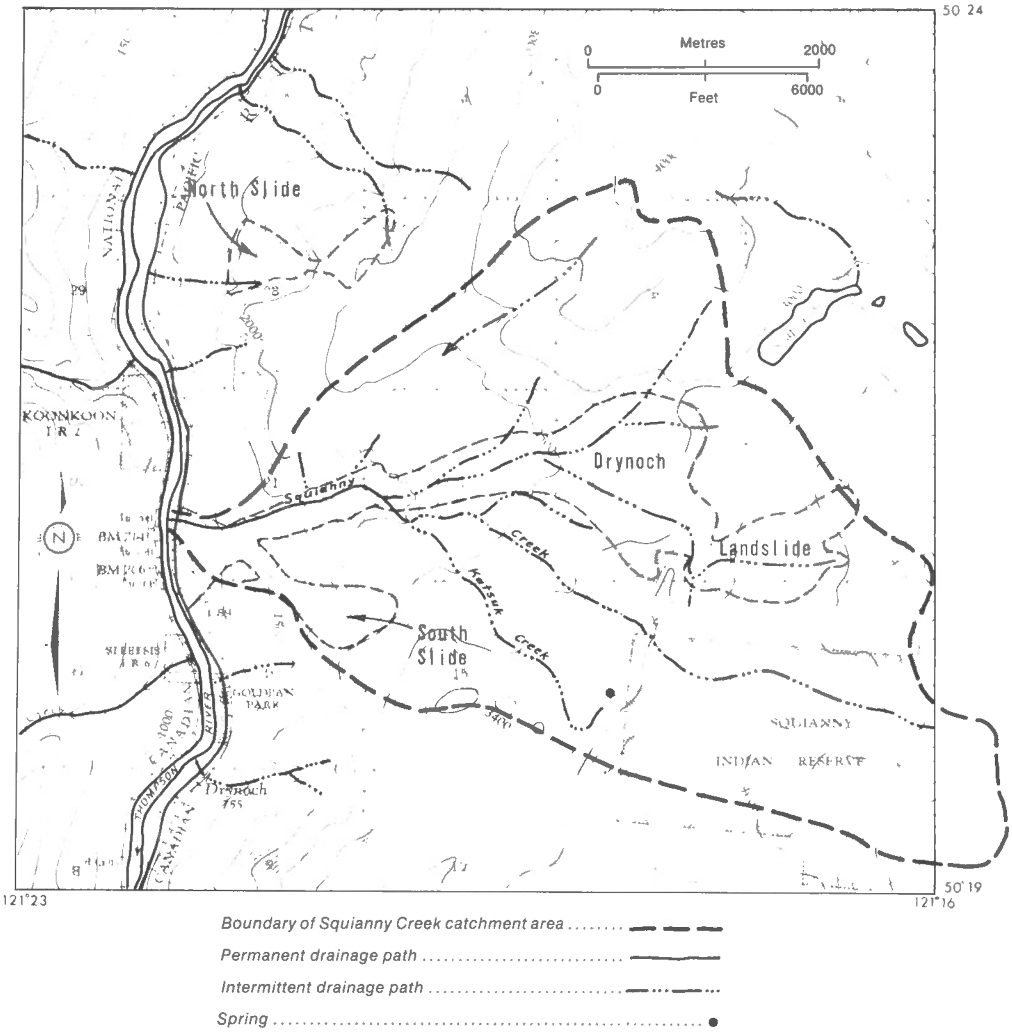


Figure 18. *Surface drainage and catchment basin of Drynoch landslide area.*

2. the average montmorillonite content increases from the toe of the slide upwards to a maximum in the Tertiary sediments of Guilbo's Flat; and
3. the clay content of the Tertiary sediments is almost entirely montmorillonite.

British Columbia Department of Highways had water samples analyzed from Soap Lake and several testholes in Drynoch landslide (Appendix 3). Sodium is the dominant cation in the groundwater which substantiates Grim's suggestion that sodium is the dominant ion in the montmorillonite.

Shear Strength of Soil from the Failure Zone

One disturbed and nine undisturbed samples were selected from the failure zone for shear strength determinations by triaxial and direct shear box tests. A full description of each sample is given in Appendix 4. Generally, material in the failure zone has a high clay content and behaves as a plastic clay even though it contains a large percentage of coarse grained material.

Triaxial Tests

Ripley and Associates (1960) carried out 8 stress controlled, undrained, triaxial shear strength tests on undisturbed soil samples, 3.6 cm in diameter and 7.6 cm high, obtained from Drynoch landslide. The tests showed a shatter of results because "for specimens of the small dimensions tested, the distribution of gravel sizes in any given specimen could have an appreciable effect on its shearing strength", but "the slope of the rupture envelope probably ranges between 14° and 17°". In an attempt to eliminate particle size effects, the present triaxial tests were performed on larger samples, 8.5 cm in diameter and 20.3 cm high.

When the tests were conducted in 1960, the fact that the long term strength of a soil is approximated better in the drained condition and the concept of residual shear strength of soil (Skempton, 1964) were generally unknown. Therefore to duplicate more closely field conditions at failure and to expose any residual shear strength of the soil, a series of triaxial tests was performed in the drained condition, and both stress and strain controlled tests were performed.

Five standard drained, strain controlled triaxial tests were carried out. The applied confining pressures attempted to duplicate overburden pressures, and the rate of strain (2% per day) was determined using the method of Bishop and Henkel (1962).

Three drained, stress controlled triaxial tests also were carried out under standard procedures. An estimate of the failure load was determined beforehand, and as that load was reached, successively smaller load increments were applied to the sample after pore water pressure from the previous increment had reached equilibrium. Results of the triaxial tests are plotted in Figure 23 and summarized in Table 2. From the failure envelope the following estimates of shear strength parameters were made: α'* = 20°, a' = 0.25 kg/cm^2, corresponding to ϕ' = 21.4°, c' = 0.26 kg/cm^2.

Figure 24 shows slickenside features developed on the failure plane of a sample from 23.5 feet depth in testhole 2. Slickensides generally indicate alignment of clay particles and subsequent residual shear strength characteristics. Even though the strain controlled tests were taken to an axial strain of 10.8%, no significant strain softening occurrred (Fig. 25).

Direct Shear Tests

To confirm shear strength test results obtained from the triaxial tests and to investigate further any residual shear strength characteristics of soil, two series of direct shear box tests were carried out. The rate of strain used in all tests, to assure drained conditions, was 0.0117 cm/min; the maximum

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*Shear strength parameters

α' slope of the failure envelope in q' vs. p' space
a' q intercept of the failure envelope in q' vs. p' space
ϕ' friction angle based on effective stresses
c' cohesion intercept based on effective stresses

Figure 19

View to the east over Soap Lake, an evaporite lake found northeast of Drynoch landslide. The white rim around the lake is an evaporite, sodium carbonate. (GSC 175609)

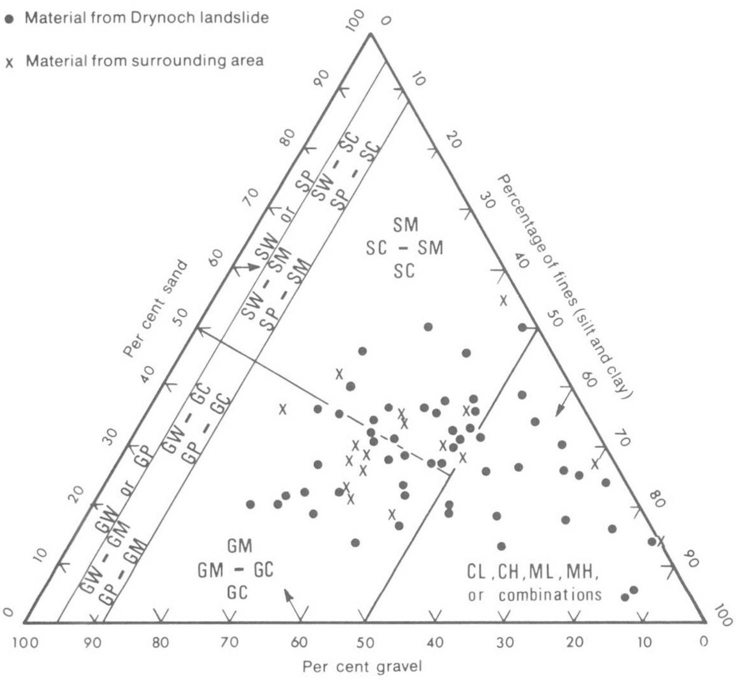


Figure 20. *Distribution of grain size for 73 samples from Drynoch landslide area, superimposed on Unified Soil Classification Identification triangle.*

displacement the apparatus would allow was 0.71 cm (approximately 12% strain). It was assumed failure occurred at this displacement unless a peak shearing resistance occurred under a smaller displacement.

The first series of tests was performed on three undisturbed samples from one shelby tube, taken from a probable zone of failure in testhole 73-7. Confining normal loads used for the tests were determined by estimating the average overburden load on the sample in the field. Residual shear strength tests were carried out following a procedure outlined by Noble (1973, p. 706).

The second series of tests was performed on the same material, but the soil was remoulded*, sieved to remove any material greater than #4 sieve size, and reconsolidated under the same testing loads as used previously. Again peak and residual shear tests were performed on the samples.

The results of the direct shear tests are shown in Figure 26. Soil strength parameters for the undisturbed samples are $c' = 0$, $\phi' = 21.3°$, for the remoulded samples, $c' = 0$, $\phi' = 15°$. In both series of tests, the corresponding residual tests show a scatter of shear strengths, and in several cases samples actually appear stronger than the corresponding undisturbed or remoulded peak strength. No estimate of residual shear strength has been made for these data.

STABILITY STUDIES

Rate of Movement

Vertical airphotos taken in 1951, 1960, 1965, 1969, and 1972 show that certain features within Drynoch landslide are moving constantly towards Thompson River. Three features that show this movement clearly are: the old TransCanada Highway, the Canadian Pacific railway (Fig. 1), and a section of logging road about 1.2 km from the river (Fig. 27). The rate of movement of the slide, estimated from the movement of the logging road, is 67 m in 21 years, averaging 3.2 m/year. This estimate corresponds well to measurements made by British Columbia Department of Highways in which the horizontal component of movement varied from 0.65 cm/month in the winter to 20-30 cm/month in the summer (Brawner, 1960, p. 59). The rate of movement varies throughout the slide, but generally maximum movement occurs in the centre of the slide and diminishes towards the edges.

Movement vs. Climatic Conditions

In 1957 British Columbia Department of Highways established several lines of movement hubs within Drynoch landslide and surveyed them regularly for a number of years to obtain an accurate record of the movement. During this period, water levels in neighbouring testholes were also monitored. Data for four typical movement hubs and testholes for a three year period (June 1957 to June 1960) were compared with climatic and hydrologic records of neighbouring stations for the same period.

Spences Bridge has no climatic records after 1909, therefore long term climatic records of the neighbouring communities were studied; it was found that the precipitation at Ashcroft (1600 ft, 488 m a.s.l.) approximates precipitation within the catchment basin of Drynoch landslide

* The term "remoulded" when referring to slide debris is a relative term only, since all the material has been disrupted from its original location at one time or another.

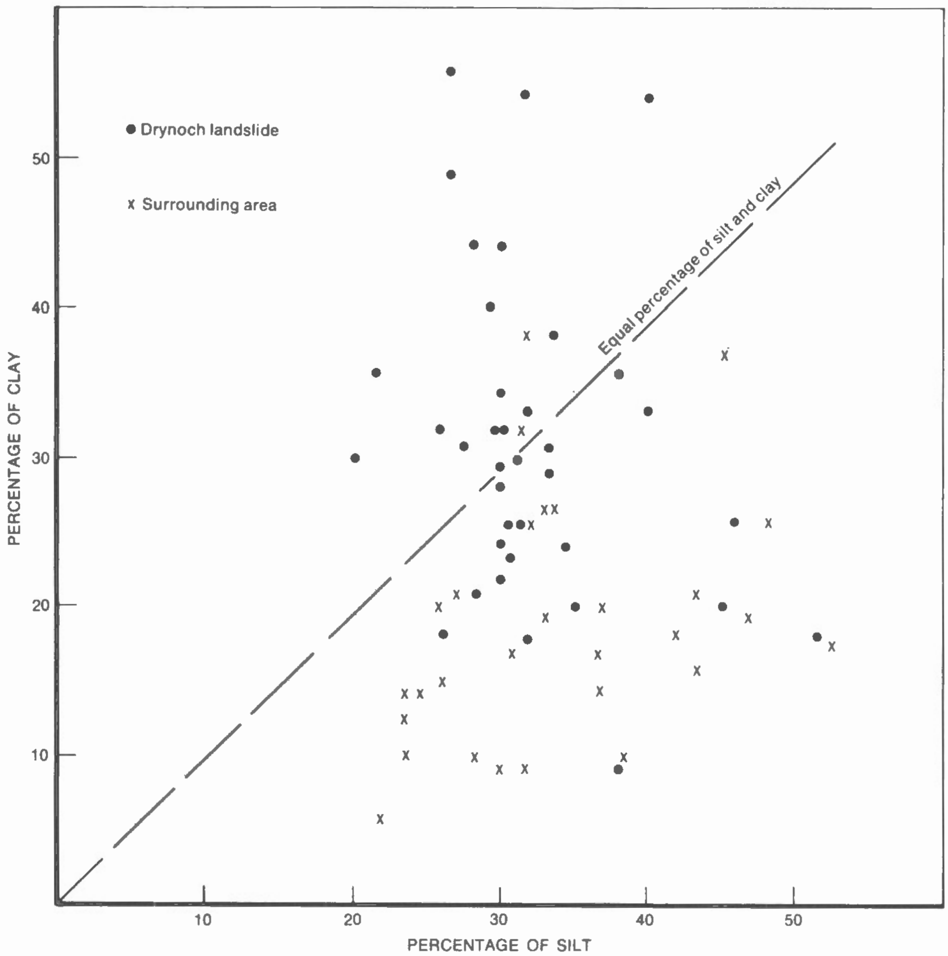


Figure 21. Distribution of fine fraction between silt and clay particle size for 66 samples from Drynoch landslide area.

(Appendix 5). Hydrologic data were obtained from a Canada Department of Environment gauging station along Thompson River immediately downstream of the slide.

Appendix 6 indicates a reasonable correlation between peak spring runoff, high groundwater levels, and maximum rate of movement, and very little correlation between the amount of precipitation, groundwater levels, and rate of movement. The latter observation is to be expected because of the small amount of precipitation and the climatic probability for a great deal of evapotranspiration in the region.

Stability Analysis – Back Calculations

Most landslide studies include an analysis of slope stability based upon soil parameters determined in the laboratory and an assumed geometry of the failure plane. Because the soil strength parameters obtained from the laboratory testing program (Fig. 22, 23) for Drynoch landslide are not consistent or conclusive, a different approach was taken.

Figure 5A indicates that the failure plane is relatively long and generally parallels the slide surface. Therefore, considering an infinite-slope (rather than a circular arc or slide block) analysis to be appropriate, and assuming that because the slide is moving the factor of safety is equal to 1.0, a back calculation was carried out to determine what strength parameters should approximate the soil condition. The parameters obtained from this back calculation then were compared to those obtained from the laboratory testing program.

For this back calculation, two infinite-slope analyses were used: an extension of Bishop's (1955) slip circle analysis and Skempton and Delory's (1957) infinite-slope analysis (see Appendix 7 for detailed calculations). The soil parameters obtained from these analyses are $c' = 0$ (assumed for both cases) and $\phi' = 17.1°$ and $\phi' = 17.4°$, respectively. A summary and comparison of the various shear strength parameters determined for the soil from Drynoch landslide is presented in Table 3.

The triaxial test results compare reasonably with the "undisturbed" direct shear test results, and the "remoulded" (<#4) direct shear test results compare reasonably with the back calculations and the test results of Ripley and Associates (1960).

DISCUSSION

Inferred Quaternary History

The inferred Quaternary history of the Drynoch landslide area is constructed from field mapping, subsurface information, and a general knowledge of the geological and glacial history of lower Thompson Valley. A generalized stratigraphic column of the unconsolidated deposits is shown in Table 4.

Prior to Fraser Glaciation (the last major period of glaciation), an interglacial period existed from before 36 000 to about 25 500 years ago (Armstrong et al., 1965, p. 326). A deposit of well sorted sand and gravel overlain by a glacial till along Thompson River and just south of Drynoch landslide may have been deposited by an aggrading river during this interval (Anderton, 1970, p. 25). From some time after 20 000 until about 10 000 years ago (Fulton and Smith, 1978, p. 979) ice from the Fraser Glaciation covered this area to a maximum thickness of 2500 m (Duffell and McTaggart 1952, p. 69). The resulting drift accounts for the glacial till in the uplands and the till overlying sand and gravel near the toe of Drynoch landslide.

Anderton (1970, p. 66) has suggested that after downwasting of most of the Fraser Glaciation ice, and prior to 9000 years ago, an ice plug remained somewhere in lower Thompson Valley. Acting as a dam, such a plug would account for the thick deposit of lacustrine "white silts" upstream along Thompson and Nicola rivers. Some time before 9000 years ago (Anderton, 1970, p. i), this ice plug disintegrated allowing southward drainage along lower Thompson River. The enlarged river probably produced the large accumulation of steeply dipping sand and gravel downstream along the river and the upper bench off fluvial sand and gravel found at the toe of the slide.

Instability along Thompson River was prevalent upon deglaciation due to oversteepening of valley walls and release of lateral support by the glaciers (Anderton, 1970, p. 61). Possibly, small landslides in the vicinity of Drynoch landslide, such as those found north and south of the slide (Fig. 14),

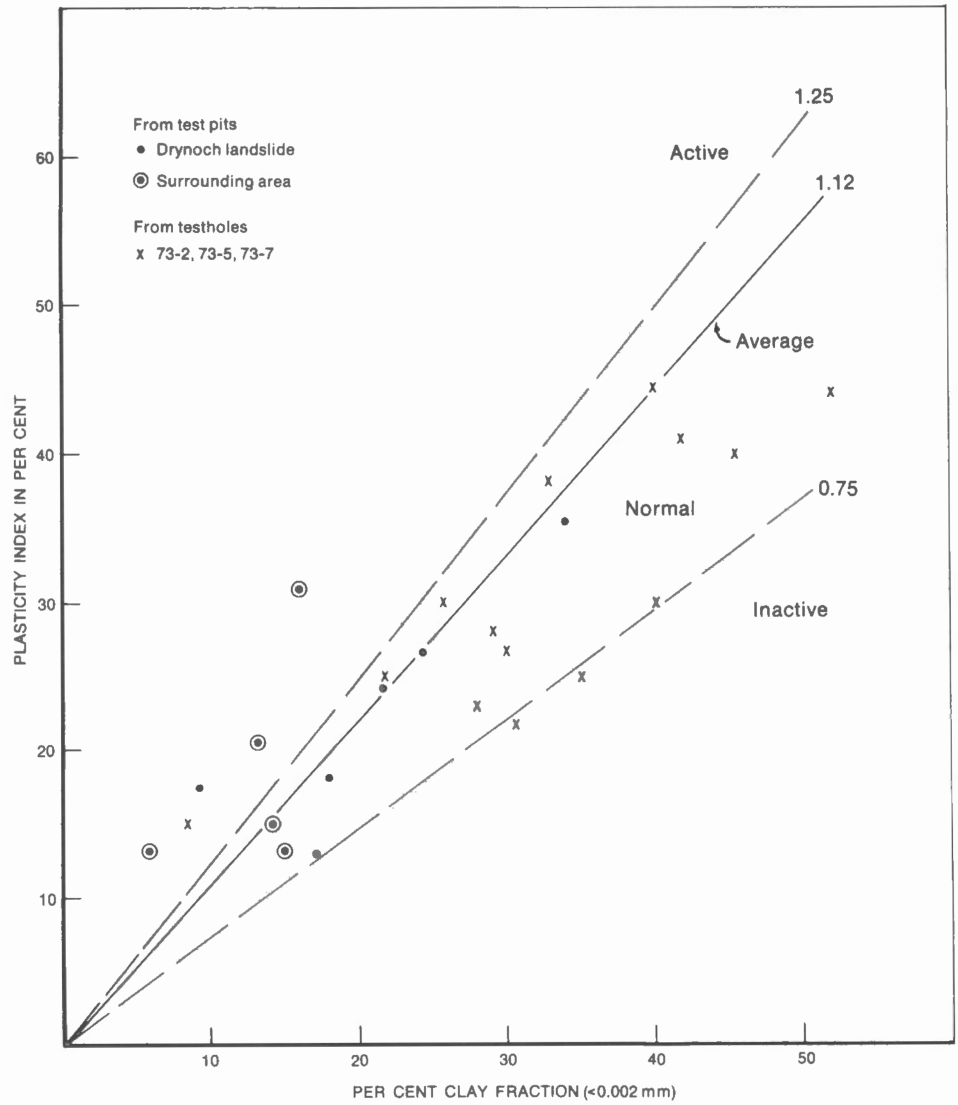


Figure 22. Activity (plasticity index vs. % clay) of 25 samples from Drynoch landslide area.

occurred after the ice plug melted. Most of the slide spoil from these older slides probably was removed by an enlarged Thompson River.

The Quaternary history of the immediate Drynoch landslide locality has been interpreted by R.J. Fulton (pers. comm., 1968) from an exposure in a gravel pit near the toe of the landslide (Fig. 15). The lower 6 m of the exposure consists of the upper fluvial sand and gravel terrace mentioned previously. Up to 0.6 m of eolian sand lies in a depression on top of the river terrace. Charcoal and cultural material found in the sand have been dated at 7530 years (GSC-530, Lowden et al., 1969, p. 31), indicating that Thompson River had receded and the shore could support human activity by that time. Less than 3.7 m of poorly sorted and washed gravel in a sandy matrix overlies the sand; the deposit dips to the west and probably originated from an alluvial fan on Squianny Creek. Alluvial fans are characteristic at the mouths of most major creeks in the area. A volcanic ash lens up to 5 cm thick is found within the fan material and has been correlated with the Mazama ash fall of 6600 years ago. The overlying slide debris is a mixture of Cretaceous bedrock fragments, poorly indurated Tertiary sediments, and glacial till, interbedded with Squianny Creek fluvial sand and gravel. The components have intermixed to appear as a sandy clay "till" with pebbles of angular volcanic bedrock and rounded to subrounded till-like stones.

During highway construction in 1961, roots were uncovered along a failure plane of the slide and were dated at 3175 ± 150 years (I-462, Armstrong and Fulton, 1965, p. 103). Charcoal from an Indian fire pit (found by the writer) in the upper portion of the slide has been dated at 900 ± 50 years (GSC-2056). These dates provide little information on the historical movement of the slide except that it was probably moving about 3000 years ago and that the upper portion was relatively stable about 1000 years ago. No precise date for beginning of movement has been found, but it is evident that the slide debris overrode the exposed Mazama ash after 6600 years ago.

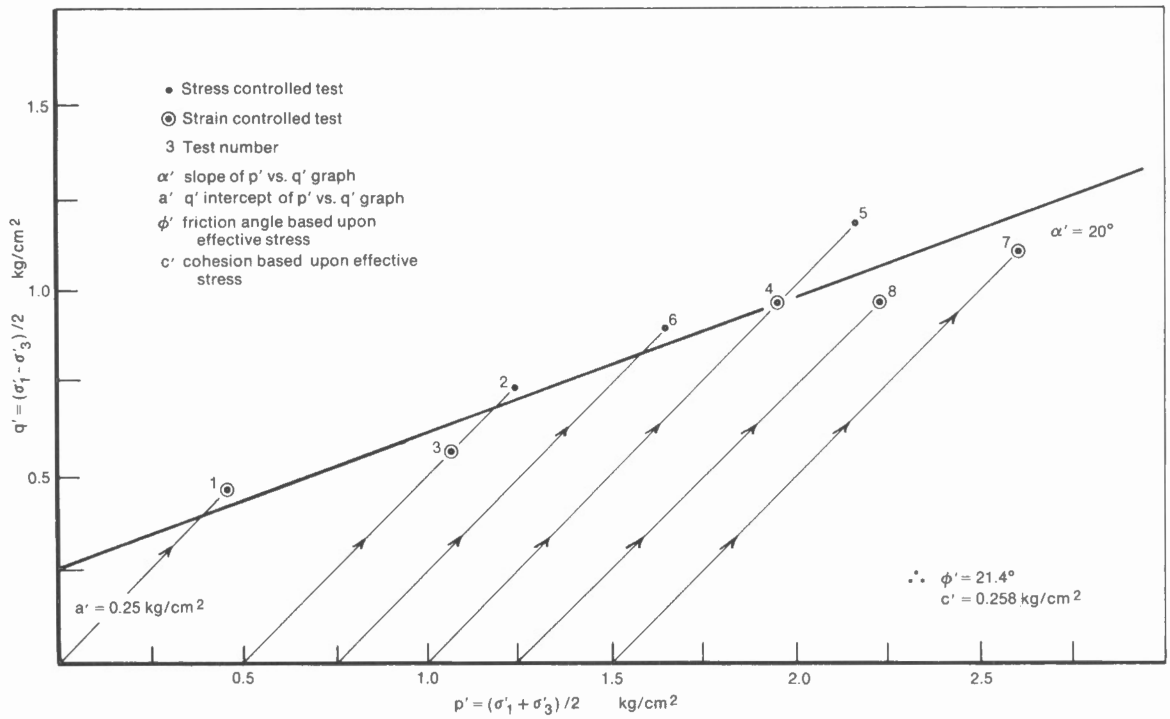


Figure 23. *Triaxial test results of samples from Drynoch landslide, plotted on a p' (hydrostatic stress) vs. q' (deviator stress) diagram.*

Table 2. Summary of triaxial test results

			Water Content (%)		at failure		σ_3'†	σ_1'-σ_3'†† at failure
Testhole	Depth (ft)	USC*	Before	After	€%**	θ*** (degrees)	kg/cm²	
73-2	5-7	CH	19.9	22.0	12	60,60	0	0.92
73-2	10-12	CH	22.0	23.6	7	55,50	0.5	1.50
73-2	17-19	CH	24.3	24.5	6	60,60	0.5	1.14
73-2	23.5-25.5	CH	18.6	20.0	11	60	1.0	1.92
73-5	30-31.5	CL	20.3	22.1	6	56,70	1.0	2.36
73-5	35-37	CH	16.3	16.9	14	48,60	0.75	1.80
73-5	40-42	CH	14.9	17.9	8	60	1.5	2.20
73-7	35-37	CH	18.1	17.5	10	60	1.25	1.92

* Unified Soil Classification
** Strain ($\Delta L/L$)
*** Angle of the failure plane to the horizontal
† Applied confining pressure
†† Deviator stress

Origin of Drynoch Landslide

The first hypothesis on the origin of Drynoch landslide was advanced by Brawner and Readshaw (1963, p. 4):

> "it is not hard to conceive of the coal measures [part of the Tertiary sediments] continuing across the cirque areas, and these having first slid out as a slump or a block glide carrying considerable amounts of montmorillonite clays to the toe of this initial slide together with a considerable flow of water, and the slide having then continued as an earthflow."

Two later hypothesis are reported in Bawden (1972, p. 140):

> "The slide area is an old preglacial valley with a very thick till deposit, and the slide is strictly a till slide with no bedrock slippage involved (J.S. Scott, personal communication)"; and

> "The slide was initiated as a bedrock slump, carrying the till cover with it. As the slide progressed the bedrock could be broken up and incorporated with the till (H.W. Nasmith, personal communication)".

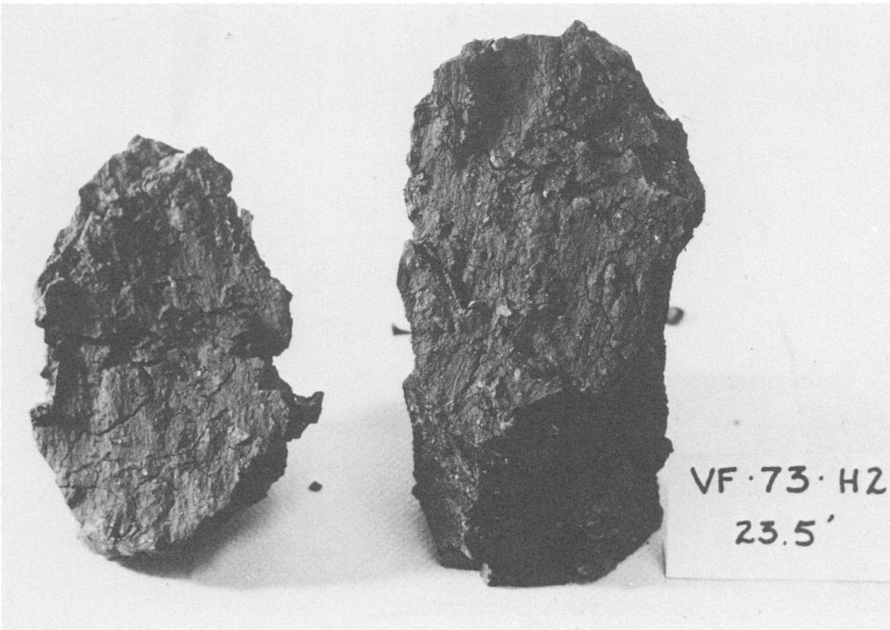


Figure 24

Photograph showing slickensides formed during a laboratory triaxial shear test on a 8.56 cm diameter sample from debris of Drynoch landslide. Photo by Queen's University.

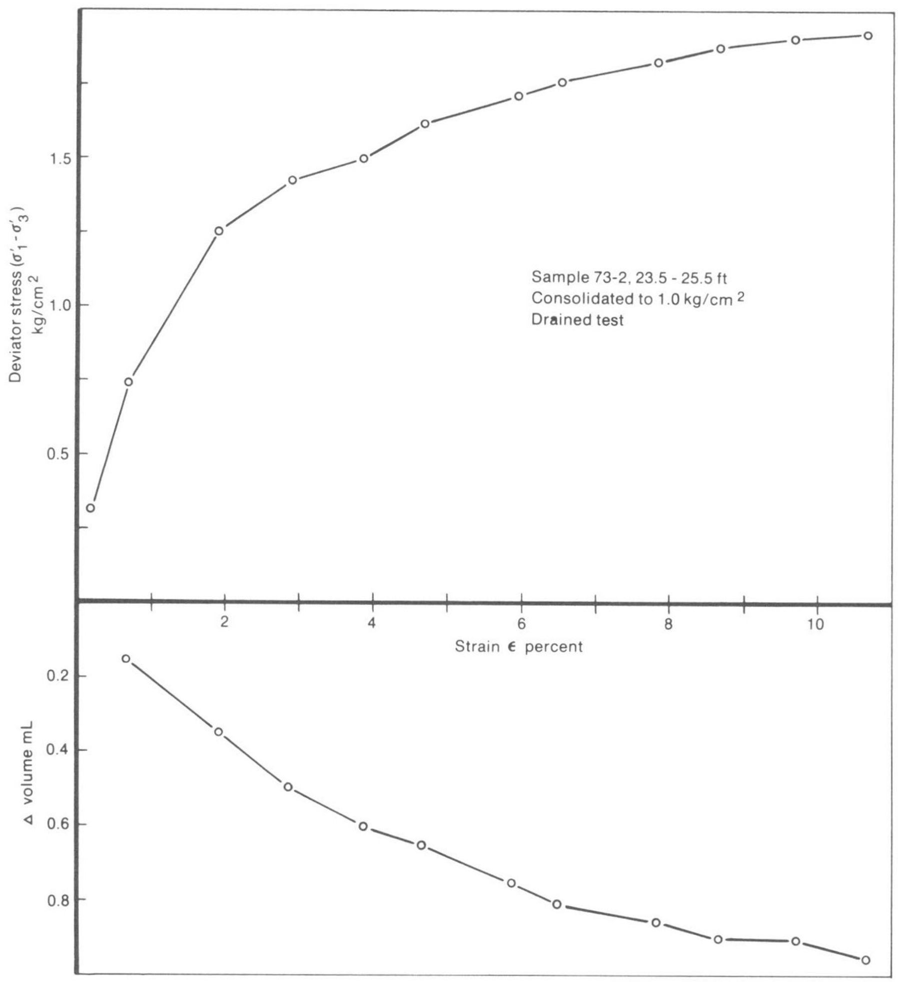


Figure 25. *Typical deviator stress vs. strain curve and change in volume vs. strain curve in strain controlled triaxial test for sample from Drynoch landslide.*

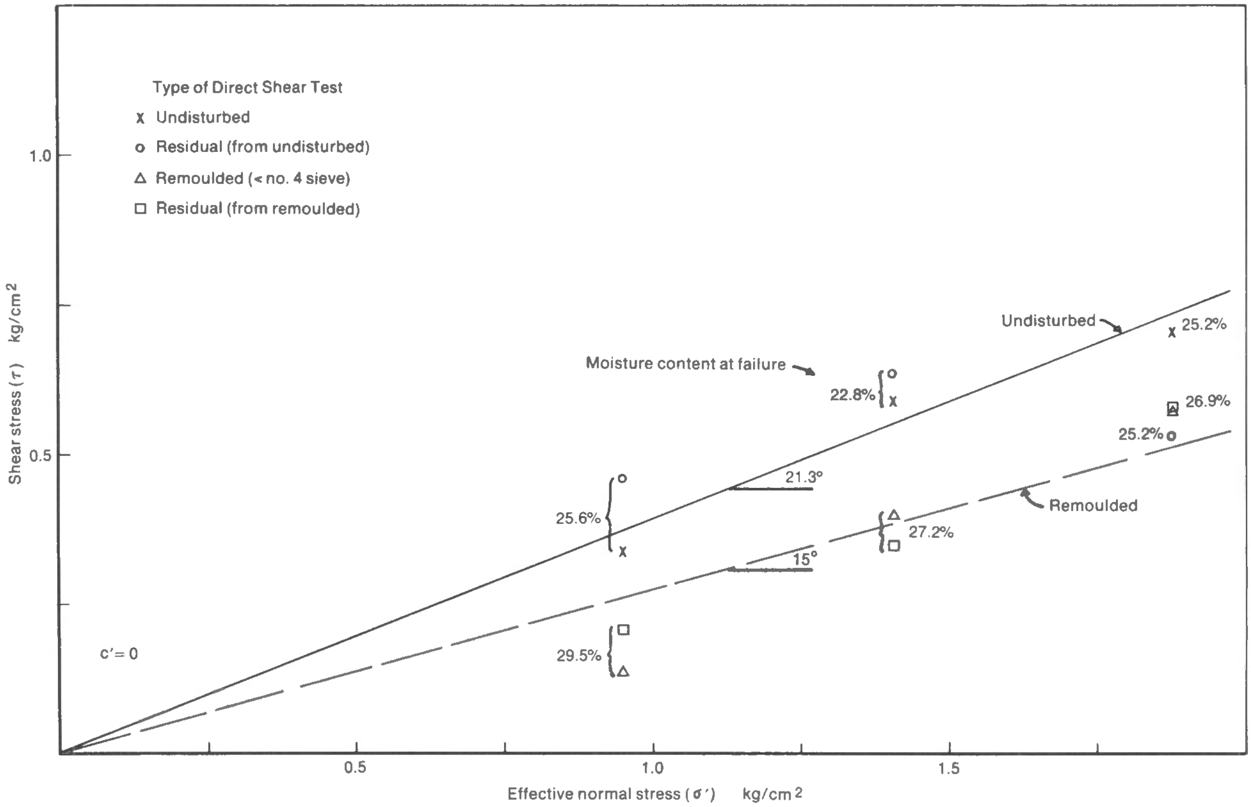


Figure 26. *Direct shear test results, sample 73-3 30 ft (10 m) from Drynoch landslide.*

In post-Tertiary time, poorly consolidated Tertiary lake sediments, which cover the Cretaceous bedrock, possibly extended over most of Squianny Creek valley (the area presently occupied by Drynoch landslide). In time, a predecessor of Squianny Creek possibly eroded away much of the Tertiary sediments and formed a deep channel in the volcanic rock (as disclosed by drilling). This downcutting possibly caused localized landslides and slumping of the Tertiary sediments such as presently found on Guilbo's Flat. These localized failures had to occur either prior to glaciation or during an interglacial period, because the scarps and Tertiary sediments are blanketed with a thin deposit of glacial till. No matter when the instability occurred, it would have added a large amount of montmorillonite-rich material to Squianny Creek valley.

As the end of glaciation neared, the ice in southwestern British Columbia downwasted. A small lateral valley glacier could have remained longer in the Squianny Valley. This could have formed an ice dam as hypothesized by Anderton (1970, p. 66) and would have contributed additional glacial drift and large amounts of local bedrock fragments to the Squianny Valley. Immediately after glaciation, removal of lateral support afforded by the glacier in Thompson Valley, and erosion by the enlarged rivers, would have caused instability in the area due to mechanisms as indicated by Anderton (1970, p. 58) and Kujansuu (1972, p. 19). By 7350 years ago an increase in the stability along Thompson Valley allowed Indians to settle at the mouth of Squianny Creek. During this period of relative stability, eolian sand was deposited on Thompson River terraces, an alluvial fan was formed at the mouth of Squianny Creek, and volcanic ash blanketed the area 6600 years ago.

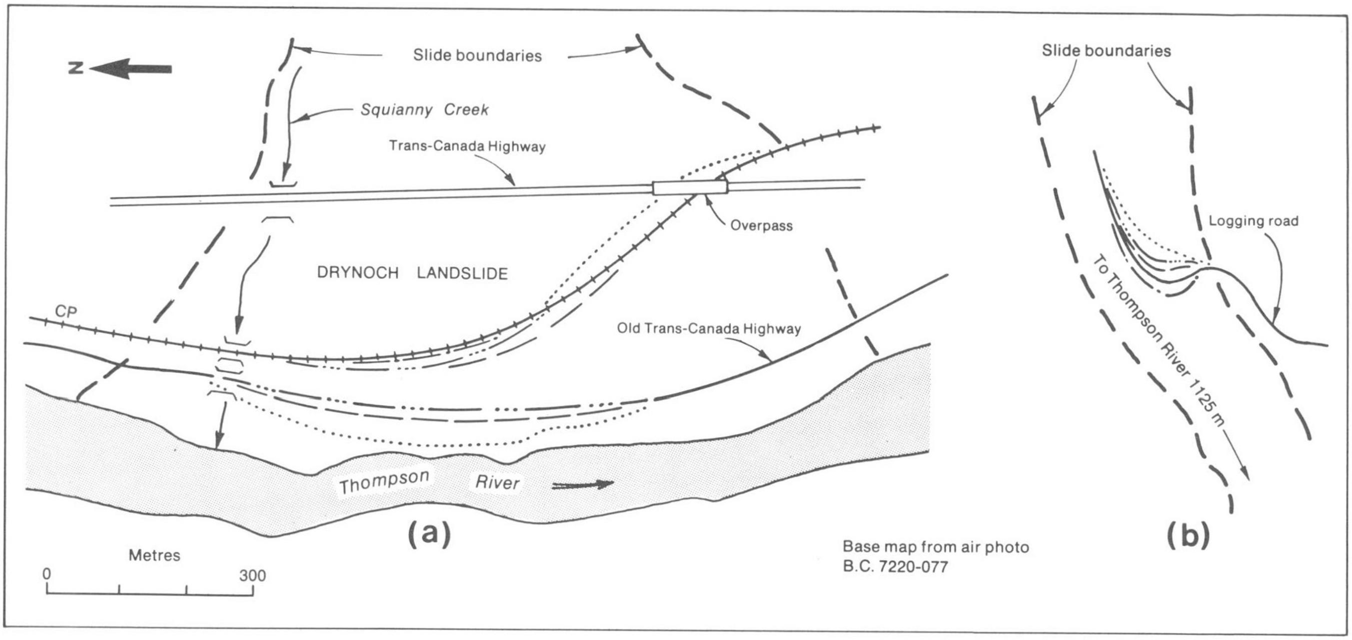
After glaciation, groundwater and surface water within the present Drynoch landslide catchment basin accumulated. This water would have reduced the shear strength of the deep remoulded valley sediments, rich in sodium montmorillonite. When the shear strength of a portion of the material was reduced sufficiently, instability produced a plastic earthflow. Evidence of the plastic nature of the initial slide is demonstrated by data from testhole 73-2 (Appendix 1), in which a sequence of overburden material, similar to that in the previously mentioned gravel pit (Fig. 15), was covered by slide debris and not removed. Several sporadic earthflows, each containing a small volume of material, appear evident from the nature of the slide debris which exhibits abundant flow features and interbedded coarse and fine grained material.

At one time a much larger volume of material filled the valley as evidenced by: (1) bedrock, presently exposed, but which is covered on top with slide debris; (2) ridges of slide debris along Squianny Valley which stand 6 to 9 m higher than the present debris (not to be confused with the features resembling lateral moraines); and (3) the manner in which slide debris has pushed Thompson River westwards into a new channel (Fig. 2).

From the initial instability, Drynoch landslide probably has moved steadily but at irregular rates throughout various parts of the slide.

In summary, the following factors may account for the earthflow:

1. an accumulation of Tertiary sediments which are very poorly indurated and contain a high percentage of sodium montmorillonite in the clay fraction;
2. a glaciation followed by deglaciation which would have added unconsolidated material to Squianny Creek valley in the form of glacial drift and locally derived rock fragments from highly fractured rock cirque-like features;



Year	Symbol	Remarks
Old Trans-Canada Highway		
1951	....	Movement of slide prior to 1951 caused considerable disruption of designed curve, bringing it close to edge of Thompson River.
1960	—···——···—	Between 1951 and 1957 highway was realigned uphill. Further disruption is evident on left side of sketch.
1965	-— — — —	Movement continued downhill as maintenance of old highway was discontinued on completion of new highway.
1969 & 1972	————	Old highway has been washed out by toe erosion and subsequent toe failure.
New Trans-Canada Highway		
1969	════	Built in 1961, this road has undergone little movement because most of the failure plane was removed during construction.
Canadian Pacific railway		
1951	....	Movement prior to 1951 created a sharp curve.
1960	—···——···—	CP Rail realigned tracks and removed some of the curve prior to construction of overpass by B.C. Department of Highways.
1965	— — — —	Movement increased sharpness of curve.
1969	+++++++++	Realignment in 1967 decreased curve once more.
1972	-·—·—·—	Downhill movement continued between 1969 and 1972.
Logging Road		
1951	....	Continual downhill movement increased sharpness of curve of logging road.
1960	—···——···—	
1965	— — — —	
1969	————	
1972	··—·—·—	

Figure 27. *Sketch showing movement of Drynoch landslide as interpreted from vertical airphotos.*

Table 3. Summary of shear strength parameters

Method of Estimation	c'* (kg/cm^2)	Ø'** (degrees)
Triaxial Test	0.26	21.4
Failure angle (45 + Ø'/2)	-	30.0
Direct Shear Test:		
(a) Undisturbed***	0	21.3
(b) Remoulded <#4	0	15.0
Back Calculation:		
(a) Bishop's (1955) method	0 (assumed)	17.1
(b) Skempton and Delory's (1957) method	0 (assumed)	17.4
Ripley and Associates (1960)	0.34	14-17

* Effective cohension

** Effective angle of friction

*** The terms "undisturbed" and "remoulded", when referring to this slide debris are relative terms only, since all this material has been disrupted from its original location.

Table 4. Generalized Quaternary stratigraphic column

Epoch	Approximate date years B.P.[1]	Unit and Origin	Lithology
	<6 600	Landslide	Heterogeneous mixture of pebbles and rock fragments in a sandy clay matrix
		unconformity	
Recent		Alluvial fan	Subrounded pebbles in sandy matrix
	6 000	Volcanic ash	Volcanic ash
		Alluvial fan	Subrounded pebbles in sandy matrix
		unconformity	
	7 350	Eolian sand	Silty sand, artifacts
		unconformity	
	> 9 000	Glaciofluvial gravel terrace	Stratified sand and gravels
		unconformity	
Pleistocene	11 000-24 500	Glacial drift	Unsorted rock pebbles in a clayey to sandy matrix
		unconformity	
	24 500-36 000	Interglacial river and lake deposits	Well sorted sands and gravels

[1] years B.P. – years before present (1950)

Sources: Anderton (1970)
Armstrong et al. (1965)

3. a bedrock topography that resulted in a large catchment basin and an inclined surface suitable for a slide to occur (the inclined bedrock surface could be either the result of an ancient bedrock slump or a fault in the bedrock); and
4. a climate resulting in high snowmelt and spring runoff.

Mechanism of Failure

Initially, the mechanism of failure postulated for Drynoch landslide involves the sliding of saturated landslide debris on a bedrock surface (Cambie, 1878; Dawson, 1896; and Anonymous, 1909). Brawner and Readshaw (1963, p. 22) postulated that the presence of montmorillonite clay and high pore water pressures are the prime contributing factors.

Material from the failure zones of Drynoch landslide contains a large percentage of highly plastic, clay-sized material composed primarily of sodium montmorillonite, a clay material that absorbs large amounts of water and subsequently becomes very weak. The presence of sodium montmorillonite probably attributed to the slickensides noted in various parts of the slide and the low angle of failure plane with the horizontal. Both features are characteristic of landslides in swelling clays.

Relatively low Atterberg limits and normal activities of the material from the failure zones were obtained even though the soil contained a high percentage of montmorillonite. These abnormalities may be due to desiccation during sample storage or due to the ASTM (American Society for Testing and Materials) procedure for determining Atterberg limits of swelling clays and shales (Lagatta, 1970). From the shear strength tests and the back calculation stability analyses (Table 3), it appears that the slide material is failing in a "remoulded" state rather than an undisturbed state. Its strength parameters are $c' = 0$ (assumed) and $\phi' = 14°$ to $17.5°$.

A possible explanation for the fact that no residual shear strength value was obtained from the laboratory tests (although slickenside were produced and the material contains considerable amounts of montmorillonite) is the large percentage of granular material in the slide. Even though large samples were tested, the granular proportion had enough effect to produce a failure angle averaging approximately 60° corresponding to a ϕ' value of 30°.

As mentioned earlier, evidence of high pore water pressure within the slide was noted during drilling in the later 1950's (Brawner and Readshaw, 1963, p. 7). Unfortunately, no additional pore water pressure data are available from the study; but strictly from a visual site investigation, the possibility of high pore water pressure is apparent. Relief within the Drynoch landslide catchment basin is approximately 1070 m. Patton (1973, p. 3), in his argument for the neo-classical groundwater flow system, stated that pore water pressures greater than hydrostatic should be expected in areas of high relief, especially if materials with relatively low permeabilities are present. But even in classical groundwater flow theory, the possibility of artesian flow in an area with such great relief is high.

The abundance of surface water from snowmelt, spring runoff, and a spring within the catchment basin also contribute to the instability of Drynoch landslide. Surface water enters the slide by way of tension and shear cracks and is distributed through layers of coarser grained, relatively more permeable material and failure zones. Water is a necessary condition to cause swelling and softening of montmorillonite. High pore water pressures could exist in the shear zones which may act as minute confined aquifers. These conditions could result in a zonal infinite-slope type failure.

In summary, the mechanisms of failure are:

1. the swelling and softening of the sodium montmorillonite-rich clay fraction in the failure zone assisted by large quantities of groundwater; and
2. high pore water pressures which, when they occur in the shear zones, may produce a zonal infinite-slope type failure.

REMEDIAL WORKS

Previous Attempts

The initial solution to the Drynoch landslide problem was not to attempt to retard or stop the movement but rather to rebuild the Cariboo Road every few years at a safer distance from Thompson River (Macoun, 1877). In 1906 the Canadian Pacific Railway diverted some surface water before it entered the slide by means of a ditch filled in with a tile drain, loose rock, and cedar poles (Anonymous, 1909, p. 728). The movement was reduced somewhat, but the ditch did not penetrate deep enough into the slide material and therefore all water was not diverted.

In 1954 Professor R.B. Peck, University of Illinois, suggested that a large cut-off ditch, extending to bedrock and backfilled with gravel, should be placed across the narrowest section of the slide approximately 0.8 km east of Thompson River. This would cut off seepage through the lower portion of the slide. Dr. Peck postulated that "sliding material from above would accumulate on the upper part of this block and reduce the gradient towards the head of the slide until ultimately most of the ground above the trench would also become stable" (in Appendix of Brawner, 1957, p. iii). The Canadian Pacific railway did not feel this large amount of construction was justified at the time, and nothing was done.

In the late 1950's British Columbia Department of Highways undertook to upgrade the Fraser/Thompson River Highway to Trans-Canada Highway standards. The road across the toe of Drynoch landslide had been such a maintenance problem for the past century, and to bypass or bridge the slide proved to be uneconomical, therefore between 1957 and 1963 British Columbia Department of Highways carried out a study and subsequent remedial work to halt or at least reduce movement of the slide.

The proposed method of stabilization was a combination of surface and subsurface drainage works. Surface drainage works consisted of:

1. draining all swamps and water holes within the slide and providing permanent drainage channels from such areas; and
2. diverting Squianny Creek across and out of the slide area to a diversion channel cut in the north bank.

Several different approaches to subsurface drainage were proposed and tested. The initial design consisted of three 1.5 m diameter, 10 gauge corrugated metal tunnels, burrowed into the slide at three different areas. These, accompanied by vertical well points, were to relieve excess pore water pressure. The first well only produced 20% of the water that was expected and lowered the groundwater table only 1.2 m instead of the anticipated 5 m. It was realized that 8 or 10 tunnels would be required rather than the initial estimate of 3, and due to economic reasons this method of subsurface drainage was abandoned. Shortly afterwards the test tunnel collapsed due to shifting ground and differential movement.

Next, the effectiveness of electro-osmosis to lower the groundwater table was tested. It was lowered only 1.8 to 2.4 m immediately around the installation. Lack of success was attributed to the granular nature of the soil and mechanical difficulties. It was concluded that, for a permanent method of reducing pore water pressures, electro-osmosis would be uneconomical.

The next approach to lower pore water pressure involved the use of several methods simultaneously.

1. A 15 to 19 m deep cutoff trench was constructed at the narrowest section of the slide, as suggested in 1954 by Dr. Peck.
2. Horizontal drains, penetrating the slide from berms (formed during construction of the trench and highway cut), were connected to a collector system to remove water from the slide.
3. The angle of repose of the slide between the cutoff trench and the highway cut was reduced. It was hoped that effective stresses would be reduced, therefore increasing stability.
4. Lime was added to the portion between the cutoff trench and the highway cut to aid stabilization by ion exchange.
5. Tension and shear cracks continually were filled to prevent surface water from entering the body of the slide.

Construction began in 1961 and was completed the following year. Some difficulties were encountered during construction due to shifting ground, interception of a failure plane, and general instability. Unfortunately, the deep cutoff trench could not be completed to its design depth, and the design number of berms had to be reduced.

It was hoped that these remedial measures would last for 20 years. Initial results indicated a reduction in movement of the upper portion of Drynoch landslide and a stabilizing effect on the lower portion. In subsequent months and years, however, the following observations were made (British Columbia Department of Highways field reports): June 1962: large movements noted in the backslope of the cutoff trench; January 1963: slide pushing highway right-of-way fence downhill; April 1965: considerable fresh failure is showing between highway and cut-off trench; slide material is encroaching upon highway ditch; fence has been pushed down; April 1966: movement between 15 December 1965 and 24 March 1966 is approximately 51 cm; July 1970: there are signs that Drynoch slide may be starting to move; there appear to be two tension cracks on the road.

Continual movement pushed material from the upper portion of the slide over the cutoff trench and plugged and broken the horizontal drains, collector systems, and diversion pipes. The lime stabilization program was carried out successfully, but it was concluded that it was not on a large enough scale to add permanent stability to the slide (J.D. Austin, pers. comm., 1973).

Since the major remedial construction work was completed, minor remedial measures have been carried out. These include placing polyethelene over portions of the slide to prevent infiltration of surface water, filling in the old Squianny Creek bed, and diverting water from the spring southeast of the slide area into another catchment basin (during the present study it was noted that the diversion dam was broken). Canadian Pacific railway realigned their railbed in 1967 and cut back a large portion of slide debris which was encroaching upon the tracks.

Suggestions for Retarding Movement

The present series of strength tests indicates material from the failure zone of Drynoch landslide is best approximated by a frictional material with little or no cohesion. As such, stability is less affected by the amount of material above the failure plane and more affected by the height of the groundwater table. Therefore, the most practical method of retarding movement of the slide, and the prime method already attempted, is to decrease the groundwater table. A common factor, which resulted in only partial success of previous attempts to retard movement, was the underestimation of the amount of movement and force of the moving slide material. Accordingly, drainage structures were broken or disrupted soon after installation (some during construction), so as to substantially decrease their effectiveness.

From this study, it is suggested that the overall stability of Drynoch landslide could be increased by the use of large diameter drainage wells (to depths below the failure zones), with an accompanying pumping system. This method was considered in 1957 by British Columbia Department of Highways but was rejected because of (1) the need for continual maintenance; (2) high cost of operation; (3) difficulty in drilling large diameter holes; and (4) the possibility of movement shearing off the wells (Brawner, 1957, p. 21).

Since the method was first proposed in 1957, drilling and pumping methods have improved considerably, a fact that would reduce the initial drilling, operation, and maintenance costs. Five successful deep drainage wells, used in Montana on a landslide with similar geometry, were installed in 1967 for $65 000 plus "nominal annual expense for inspection and maintenance to ensure continued proper functioning" (Noble, 1973, p. 708). Pumping would only be necessary during certain seasons of the year and could be activated automatically, thereby decreasing operating costs further.

The possible shearing of drainage wells is not as grave a problem as it first appears. If a drainage well was designed to be installed on the downslope side of a large diameter drillhole, and the upslope side of the hole was backfilled with loose sand (capped with an impervious cover), the sand would deform upon movement and would reduce stress on the well and ancillary equipment. Even if the drainage well did shear, to redrill and establish a new drainage well would be far less expensive in the long term than to install a costly, non-repairable permanent drainage system.

The design, number, locations, and cost estimate for these drainage wells are not within the scope of this present study.

Accompanying the installation of drainage wells, proper surface drainage should be reimplemented and nominal annual maintenance carried out to ensure continued effectiveness of the wells. Other supplementary methods to retard movement include forestation of the slide surface with deep rooted vegetation and filling small diameter testholes with dry slaked lime which would be dissolved and distributed by groundwater and thereby cause partial cementing of the soil.

For the more localized problem of toe stability of Drynoch landslide, a possible solution would be to (1) remove existing toe spoil to produce a lower slope angle; (2) compact the exposed surface to prevent surface water infiltration; and (3) load the toe with rock fill which could be placed upon the bedrock that is exposed at low water levels. This would both increase the stability of the toe and provide bank protection from Thompson River. If properly designed, the structure would not interfere with the flow of the river or force the water to undercut the opposite bank and the Canadian National railway embankment.

SUMMARY AND CONCLUSIONS

Drynoch landslide, a large earthflow in southwestern British Columbia, has been moving slowly for hundreds of years. It has been studied previously by geologists and by civil engineers. This study looks at the slide from an integrated engineering geological and geotechnical viewpoint.

The various bedrock and surficial deposits of Drynoch landslide and the surrounding area were mapped and studied, and the geological history of the slide area was reconstructed. This, supplemented by an analysis of particle size distribution, clay mineralogy, and the relationship of movement, climate, and hydrogeological data, leads to the following observations and conclusions:

1. Drynoch landslide debris consists of a heterogeneous mixture of Pleistocene glacial till, remoulded Tertiary sediments, and fragments of local Cretaceous bedrock. The slide debris contains a slightly greater fraction of clay than silt sized particles, of which a large percentage is montmorillonite.
2. Drynoch landslide has an origin intimately related to the geological and glacial history of the area. Tertiary sedimentary rocks provided a large amount of sodium montmorillonite-rich material to Squianny Valley. Glaciation and subsequent deglaciation added additional clay-rich material to the valley, produced some east-west trending glacial features, and contributed to the breakdown and inclusion of Cretaceous rock fragments in the slide debris. The topography provides a large catchment basin and an inclined surface (produced by bedrock faulting and/or slumping) where large amounts of unconsolidated material can accumulate and move downhill.

A laboratory testing program revealed the character of soil from both the failure plane and surrounding area. This information supplemented by site investigation and literature review enabled the following conclusions to be made:

1. Drynoch landslide is probably a zonal infinite-slope type of failure. Within the zones of failure, the highly plastic clay fraction fails at a shear strength approximated by remoulded strength parameters.
2. Factors that influence the failure mechanism include the presence of montmorillonite in the clay fraction, the availability of groundwater, and the presence of high pore water pressures.

Based upon the results of the present study several recommendations for future study are suggested. A detailed study of glacial features within the Drynoch landslide area would verify the hypothesis of a remnant valley glacier in Squianny Creek valley. The discrepancy between the high montmorillonite content of the clay fraction and the relatively low values of Atterberg limits and activity warrants further investigation.

For design of the suggested drainage well system to retard downhill movement of the slide, additional geotechnical studies are recommended. These include reactivating a movement monitoring system, obtaining additional information on the depth and thickness of the failure zone or zones, obtaining additional piezometric and permeability data, and obtaining an accurate determination of depth to bedrock. Similar studies should be carried out for the design of stabilizing measures for the toe of Drynoch landslide.

A landslide, such as Drynoch, is a natural geologic process associated with valley development. The mechanisms of a landslide are, and can be analyzed by, a series of simple mechanical processes. Therefore to understand more fully all the processes involved, the study of a landslide should combine geological and geotechnical engineering.

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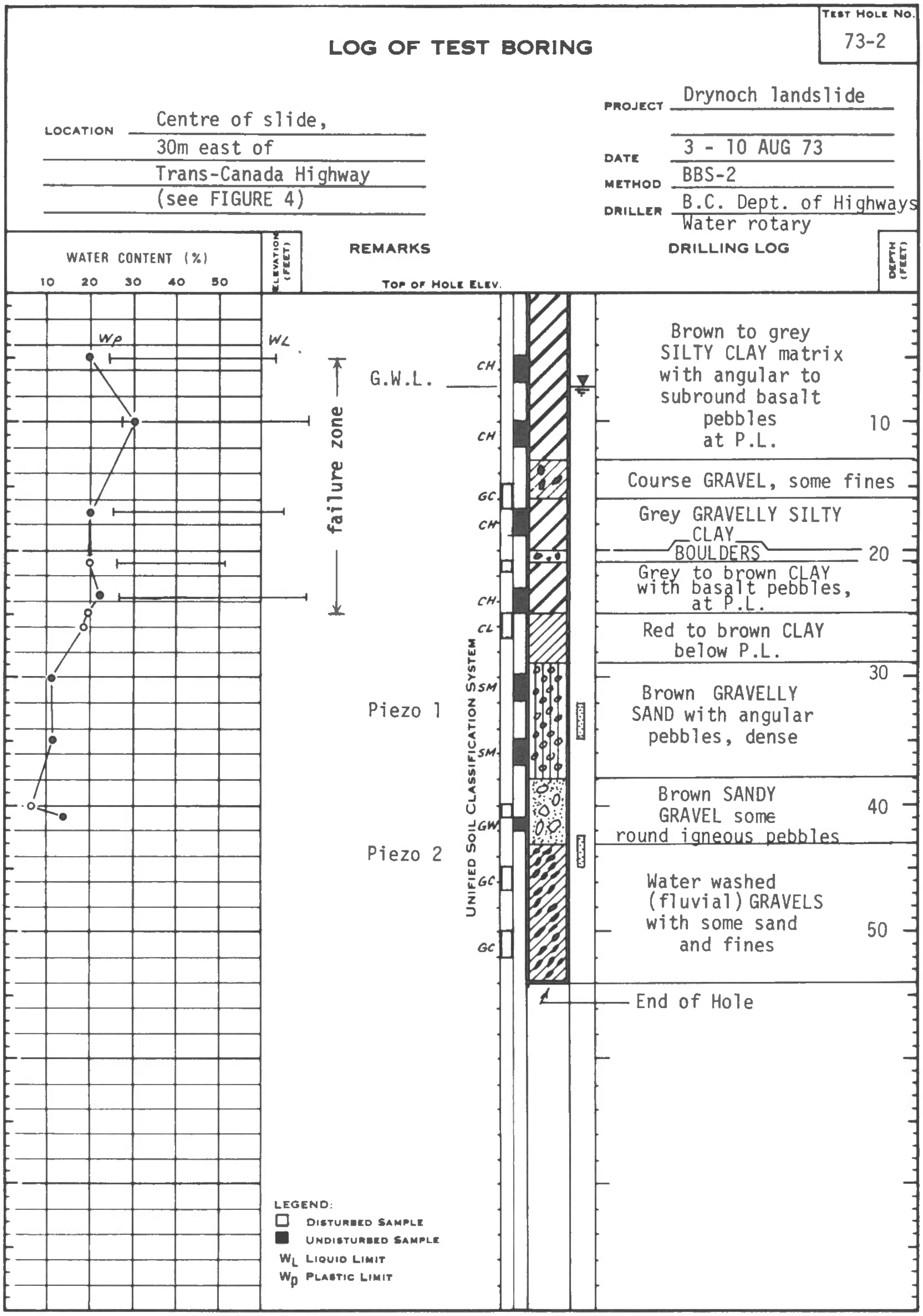

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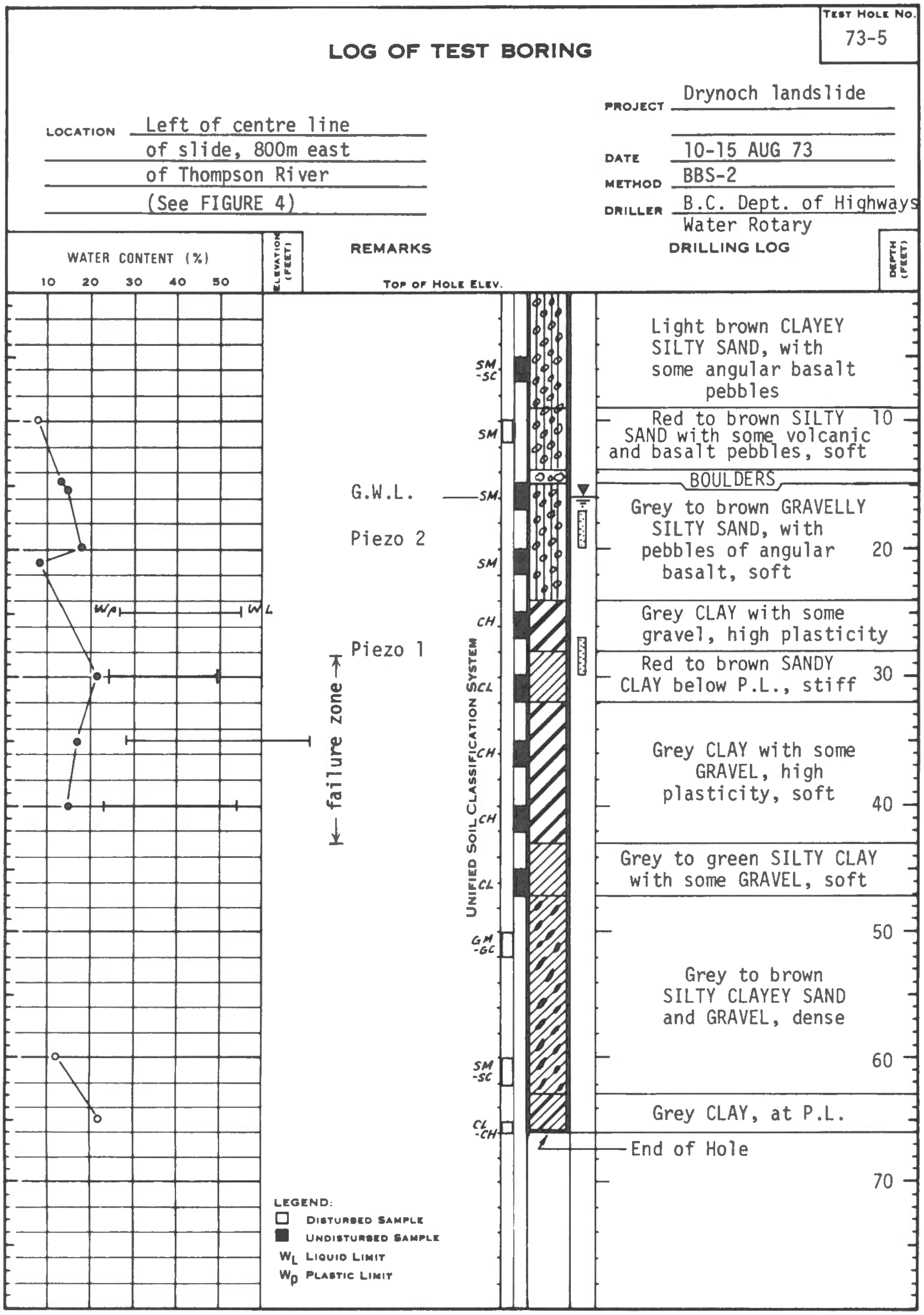

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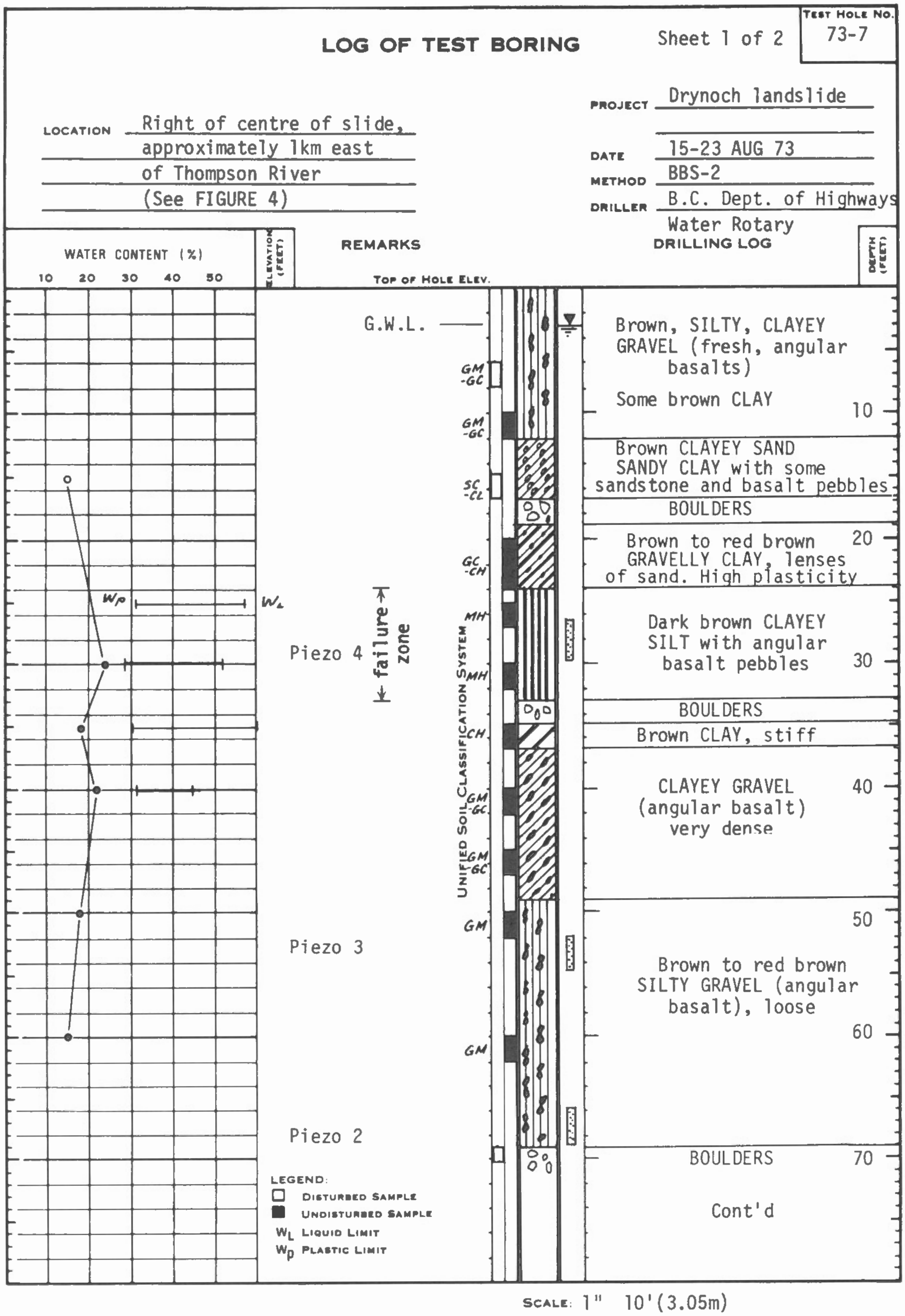

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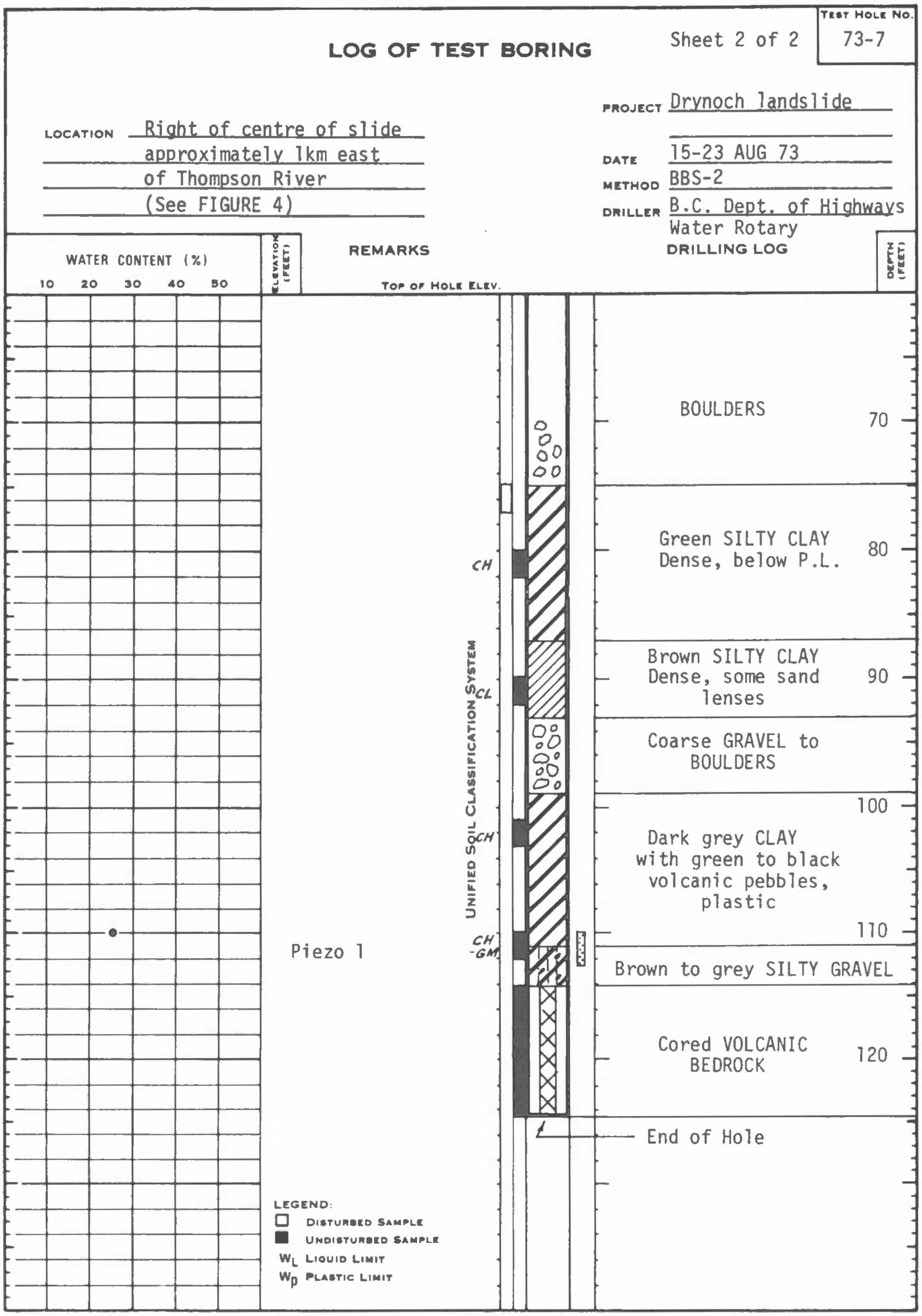


SCALE: 1" = 10' (3.05m)

APPENDIX 2

Relative abundances of clay minerals

Sample No.	Material	Relative Abundances % Illite	Chlorite	Montmor-illonite
Bawden				
SK-DS-1	Material	-	4	96
SK-DS-3	Surrounding	4.0	10	86
SK-DS-4	Drynoch landslide	-	-	100
VanDine				
VF-73-H2001		-	40	60
VF-73-h2003		-	30	70
VF-73-H2006	TH 73-2	-	31	69
VF-73-H2007		-	33	67
VF-73-H2009		-	33	67
VF-73-H5007		-	18	82
VF-73-H5008	TH 73-5	-	34	66
VF-73-H5009		-	28	72
VF-73-H5011		-	22	78
VF-73-H7008		-	19	81
VF-73-H7010		-	29	71
VF-73-H7011	TH 73-7	-	30	70
VF-73-H7012		-	27	73
VF-73-H7014		-	-	100
VF-73-79	Tertiary	-	-	100
VF-73-80	Sediments	-	7	93
VF-73-81	(Guilbo's Flat)	-	-	100
VF-73-82		-	15	85

* See Figure 4 for testhole locations
Sources: unpublished reports, W.F. Bawden, 1972; VanDine, 1974

APPENDIX 3

Water analyses of samples from Soap Lake and testholes from Drynoch landslide

Ion	Soap Lake (mg/L)	Testholes in Slide (ppm)
Na	7606.2	612
K	2860.1	-
CO_3	10862.0	356
HCO_3	2618.0	-
SO_4	842.1	1146
Cl	254.5	71
Ca	2.5	124
Mg	9.6	59
Fe	0.13	-
pH	10.1	8.0
Date	20 March 58	7 January 60

Source: British Columbia Department of Highways, unpublished data

APPENDIX 4

Visual identification of soil samples used for triaxial and direct shear tests

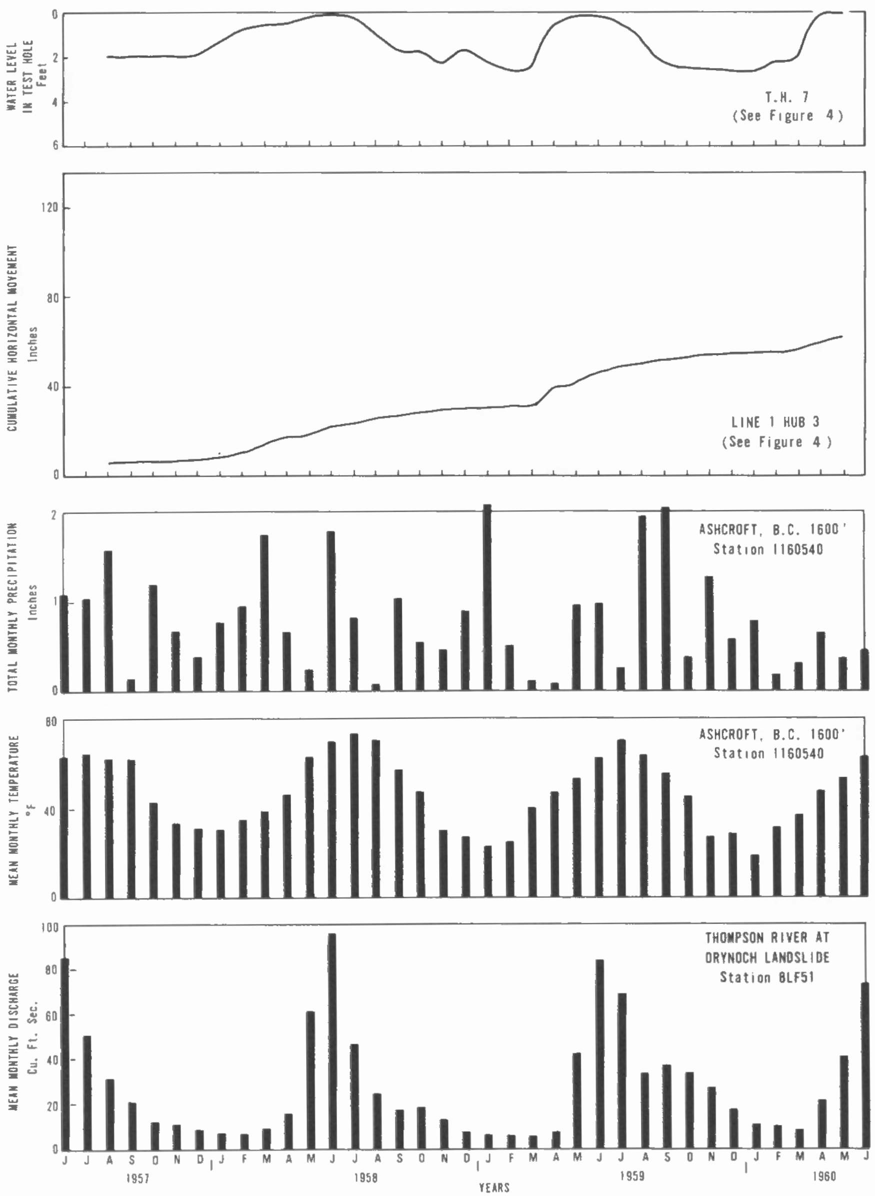
Testhole	Depth (ft)	Test	Visual Description
73-2	5-7	Strain controlled triaxial test	- grey to dark grey - silty clay matrix with many cobbles of fresh, angular basalts; maximum size 3" - stiff, at plastic limit
	10-12	Stress controlled triaxial test	- grey to dark grey - silty clay matrix with some cobbles of basalt and volcanics; maximum size 0.5" - firm to stiff, slightly above plastic limit - small pockets of yellow sand
	17-19	Strain controlled triaxial test	- grey to dark grey - silty clay with some stones of angular basalt; maximum size 0.75" - firm to stiff, at plastic limit
	23.5-25.5	Strain controlled triaxial test	- dark grey to green - clay matrix with some sand lenses (probably disintegrated sandstone) - angular basalt and some green material; cobbles up to 2" - very stiff, at plastic limit but dry
73-5	30-31.5	Stress controlled triaxial test	- dark grey - clay matrix, contains many pebbles of basalt and volcanics - stiff, above plastic limit
	35-37	Stress controlled triaxial test	- dark grey - clay matrix with pebbles of basalt, sandstone and volcanics; maximum size 1" - very stiff, moist to dry, at plastic limit
	40-41.5	Strain controlled triaxial test	- dark grey - clay matrix but contains some layers of wet sand; pebbles are sandstone and volcanics, rounded to subangular - very stiff, at plastic limit
73-7	30-31	Undisturbed, direct shear test	- brown to grey - sandy, silty, clay matrix with some angular basalt; maximum size 0.5" - firm, above plastic limit
	31-32	Remoulded, direct shear test	- brown to grey - sandy, silty, clay matrix with some angular basalt; maximum size 0.5" - firm, above plastic limit
	35-37	Strain controlled triaxial test	- dark brown and grey - clayey, silty, sand matrix, with basalt; maximum size 2" and small volcanics - stiff, moist, at plastic limit

APPENDIX 5

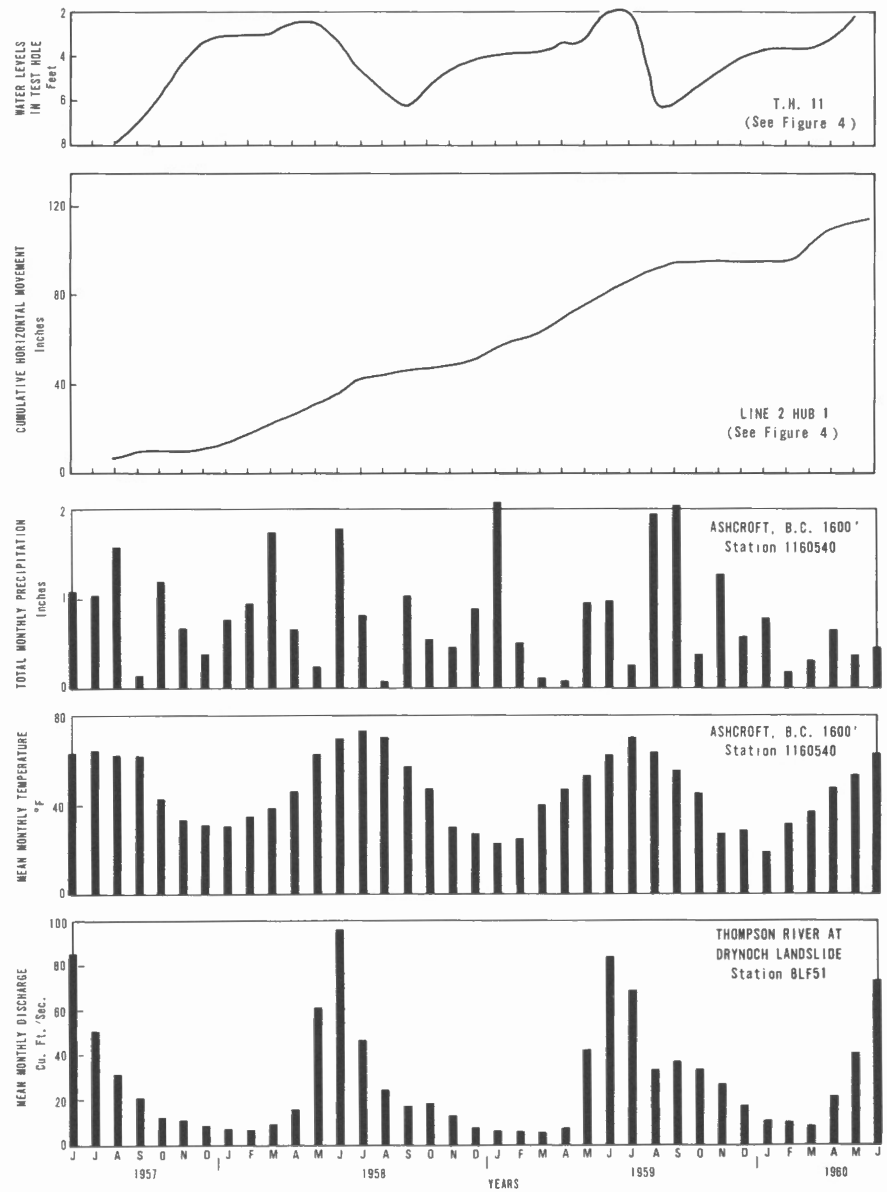
Long term climatic records for Spences Bridge, and Ashcroft British Columbia

Location	January	February	March	April	May	June	July	August	September	October	November	December	Yearly Total
Spences Bridge (750 ft, 230 m a.s.l.)													
(a) Mean daily temperature (°F)	22.1	28.2	40.1	51.2	59.8	65.0	72.2	70.9	60.5	50.1	35.4	30.1	
(b) Mean monthly precipitation (in)	0.87	0.62	0.53	0.32	0.83	0.69	0.58	0.52	0.95	0.75	1.00	0.89	8.55
Ashcroft (1600 ft, 488 m a.s.l.)													
(a) Mean daily temperature (°F)	20.2	25.3	36.4	47.9	57.1	63.2	68.8	66.1	58.1	45.5	32.6	25.2	
(b) Mean monthly precipitation (in)	1.17	0.71	0.48	0.39	0.60	1.32	0.63	1.05	0.64	0.79	0.83	0.86	9.47

Source: Canada Department of Environment, 1974

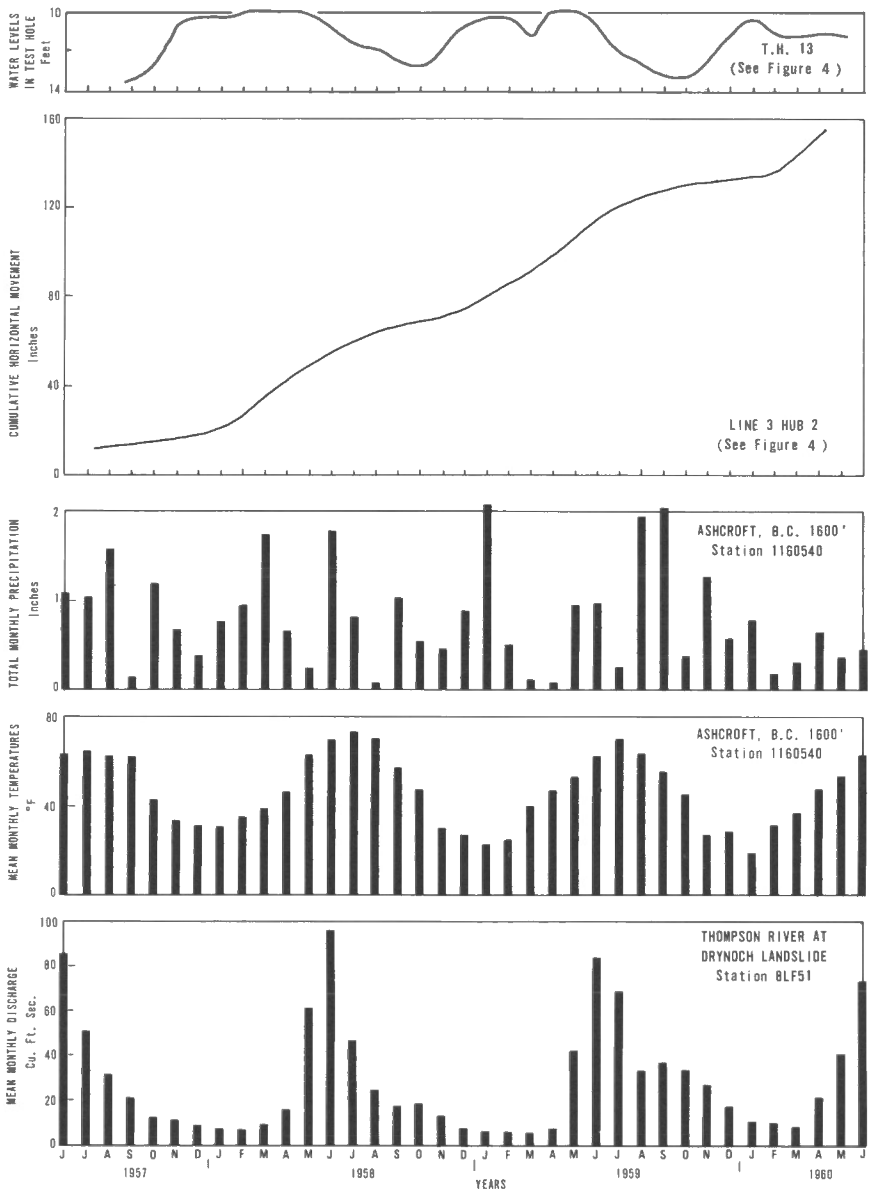


T.H.7; Line 1, Hub 3, June 1957 - June 1960

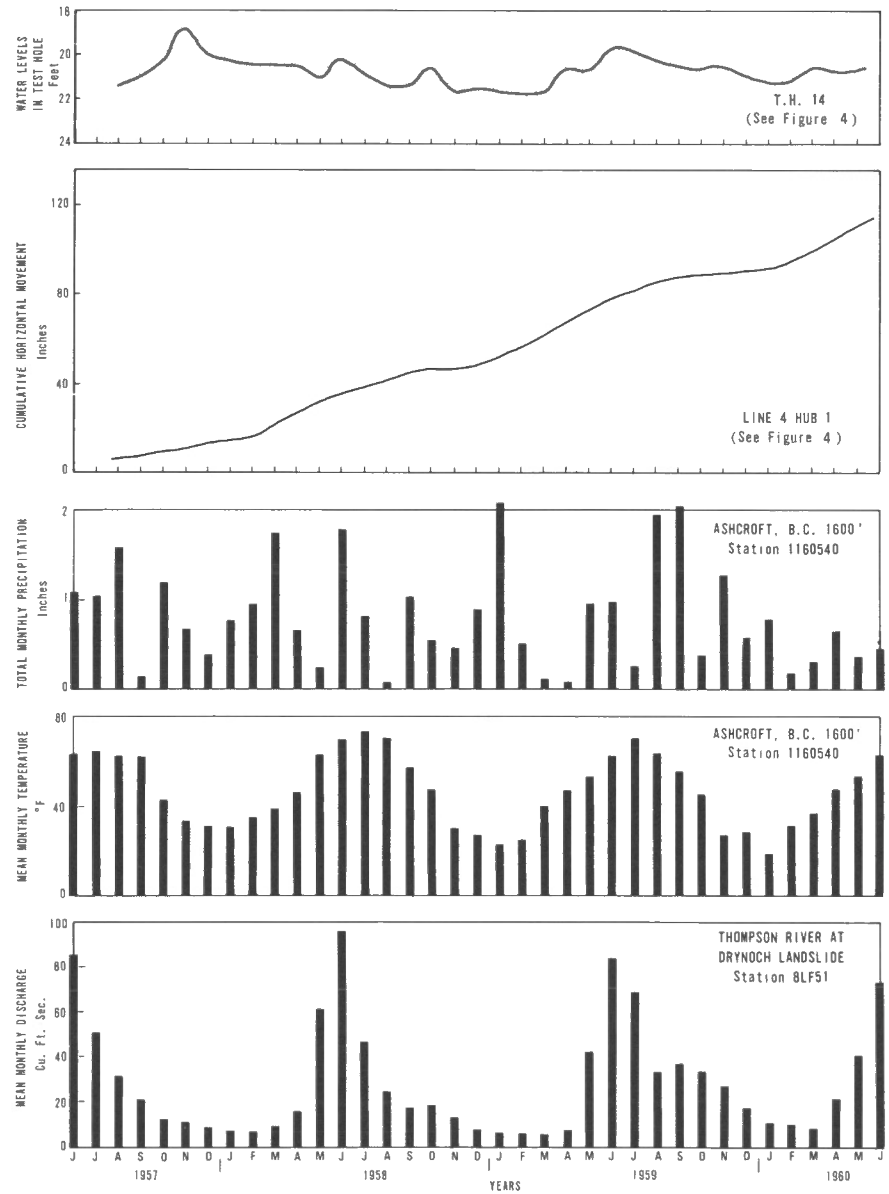


T.H.11; Line 2, Hub 1, June 1957 - June 1960

Appendix 6 cont'd.



T.H.13; Line 3, Hub 2, June 1957 - June 1960



T.H.14; Line 4, Hub 1, June 1957 - June 1960

APPENDIX 7

Slope stability calculations

Assumptions:

i. $c' = 0$ Both Bishop and Morgenstern (1960) and Skempton and Delory (1957) have shown that $c' = 0$ for an infinite slope failure.

ii. $r_u = 0.50$ Bishop (1955) defines r_u as a nondimensional pore water pressure parameter equal to $h_w \gamma_w / h_s \gamma$ where:

h_w = height of water in analyzed slice

h_s = height of regional groundwater table

γ_w = unit weight of water

γ = unit weight of soil

Therefore $r_u = 0.50$ considers the water table to be very close to the surface. The corresponding parameter in Skempton and Delory's analysis is $m = 1.0$

iii. $\gamma = 125.4$ lb/cu ft

From Ripley and Associates (1960):

γ_d = 99 lb/cu ft

w = 26.6%

$$\therefore \gamma = \gamma_d \left(1 + \frac{w}{100}\right)$$

$$= 99 \left(1 + \frac{26.6}{100}\right)$$

$$= 125.4 \text{ lb/cu ft}$$

where t_d = dry unit weight of soil

w = water content

iv. $\beta = 9.0°$ = angle of lower slope

v. $h = 35$ ft = approximate depth to failure plane

These figures are taken from Figure 5 and corresponds to the average slope of the lower portion of the slide and the approximate depth to the failure plane, respectively.

I. From Bishop (1955)

$$F = \text{Factor of safety} = \frac{\tan \phi'}{\tan \beta} (1 - r_u \sec^2 \beta) + \frac{2c'}{h\gamma \sin 2\beta}$$

For infinite sliding analysis with F = 1.0

$$1.0 = \frac{\tan \phi'}{\tan \beta} \left(1 - \frac{\gamma w}{\gamma}\right) \sec^2 \beta$$

$$\therefore \tan \phi' = \frac{\tan \beta}{\sec^2 \beta} \frac{\gamma}{\gamma - \gamma w}$$

$$= \frac{0.158}{1.024} \left(\frac{125.4}{63.0}\right)$$

$$= 0.307$$

$$\therefore \phi' = 17.1°$$

II. From Skempton and Delory (1957)

$$\text{F.S.} = \frac{\gamma - m\gamma w}{\gamma} \frac{\tan \phi'}{\tan \beta}$$

For infinite sliding analysis with F.S. = 1.0

$$1.0 = \frac{\gamma - m\gamma w}{\gamma} \frac{\tan \phi'}{\tan \beta}$$

$$\therefore \tan \phi' = \frac{\gamma}{\gamma - \gamma w} (\tan \beta)$$

$$= \frac{125.4}{63.0} (0.158)$$

$$\therefore \phi' = 17.4°$$