Rock slopes and reservoirs - lessons learned

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Abstract: Many of the slopes along British Columbia's hydroelectric reservoirs have been investigated, monitored and, in a few cases, stabilized during the last four decades. This has provided an opportunity to compare early interpretations of geology and slope performance with later interpretations derived from long-term monitoring and additional investigations. These comparisons have provided some lessons and cautions, in particular regarding interpretation of geological and groundwater conditions from limited information. These lessons are illustrated through short case histories of a few of the more important slopes: Little Chief Slide, Dutchman's Ridge, Downie Slide, Checkerboard Creek and Wahleach.

Introduction

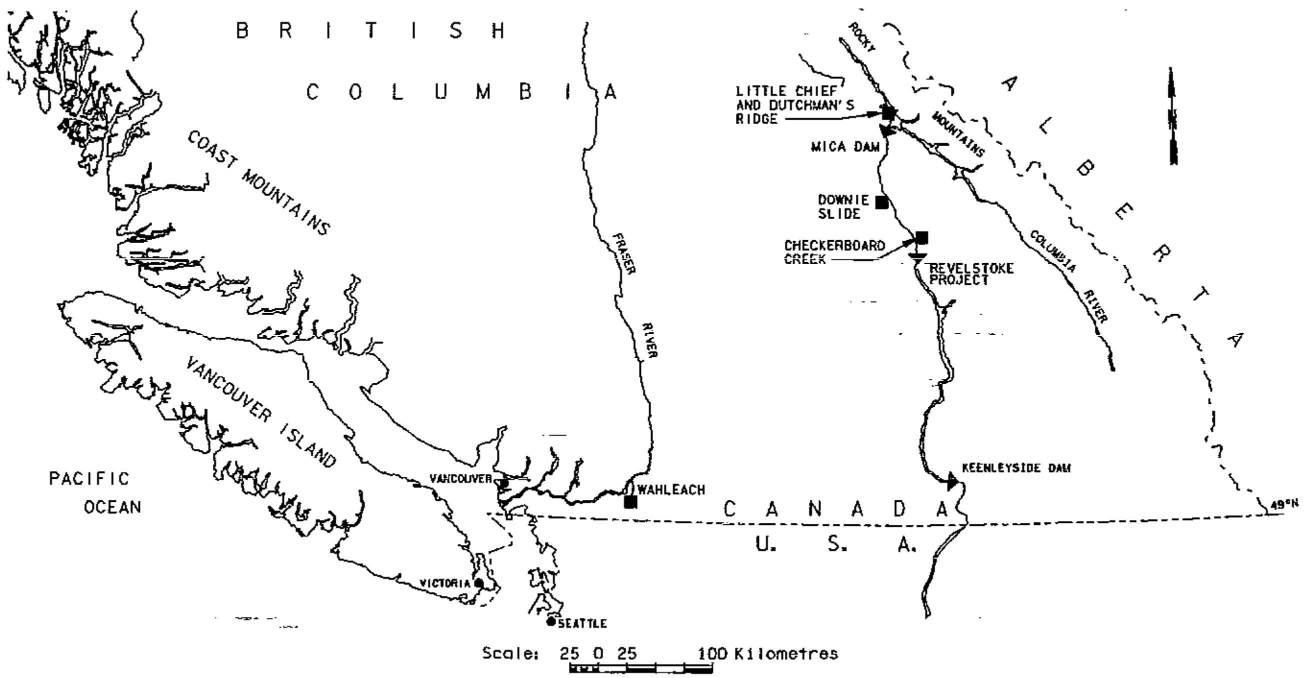
As part of BC Hydro's management of over 2000 km of reservoir shoreline, four rock slides or potential slide areas have been intensively investigated and monitored for up to four decades. Many other slopes have been investigated and monitored to a lesser degree as part of the same program. This investment of time and money has been justified by the large potential consequences to life safety and capital investments and to the long life span associated with hydroelectric projects. Involvement with these slopes over a long period of time and during successive phases of investigation and monitoring and, in some cases, remediation has provided a perspective of rock slopes unavailable to many practitioners who have to contend with similar problems with much smaller budgets and shorter time frames.

The main case histories described are: Dutchman's Ridge and the Little Chief Slide area beside Mica Reservoir, Downie Slide and Checkerboard Creek slope beside Revelstoke Reservoir and the Wahleach slope near Hope, B.C. (Fig. 1 and Table 1).

Table 1. Summary of main case histories.

SLOPE NAME	DESCRIPTION	VOLUME ($x10^6m^3$)	MOVEMENT RATES (mm/year)	CURRENT STATUS
Little Chief Slide	Existing rock slide	800	0.7 - 8	Pressures and movement monitored.
Little Chief Ridge	Rock slope adjacent to slide. Crossed by many linear troughs and ridges.		Zero	Monitored visually and with surface surveys.
Dutchman's Ridge	Adjacent to Little Chief Ridge. Underlain by a fault.	115	20 during reservoir filling. 1-2 since stabilization.	Drained, and thoroughly monitored continuously.
Downie Slide	Existing rock slide.	1500	10 before stabilization with portions of the toe at 300. Less than 1 since stabilization with portions of the toe at 250.	Drained and thoroughly monitored continuously.
Checkerboard Creek	Steep rock slope with renewed movement on ancient tension cracks.	2	5 to 10	Thoroughly monitored, next steps under consideration.
Wahleach	Steep rock slope with slow deformation, both ancient and current. Lining of power conduit was ruptured.	20	0 - 30, some local surface areas greater.	Portion of Power Conduit relocated. Slope thoroughly monitored continuously

Fig. 1. Location plan of five slide areas in southern British Columbia, Canada



Except for Checkerboard Creek, many aspects of these slopes have been described elsewhere in the literature and the references are cited below in the appropriate section. For the purpose of the present paper much use has been made of information from the earlier papers but specific observations regarding geology, groundwater flow, pressures and response to drainage and regarding measured deformations have been added or highlighted. Based on these observations, general statements are made regarding geological and groundwater models and movement mechanisms and some lessons that have been learned or relearned are discussed. Emphasis is placed on the uncertainties that must be dealt with and the judgments that have to be made to solve many rock slope engineering problems. Reduction of these uncertainties usually results from better data rather than better analysis.

The geological and groundwater models referred to are models of the physical attributes of these slopes from which mathematical models have been developed. Mathematical models are beyond the scope of this paper and outside of the expertise of the writer.

Little Chief Slide and Dutchman's Ridge

Little Chief Slide and Dutchman's Ridge are on the west side of Mica Dam reservoir (Kinbasket Lake) just upstream of the dam. Mica Dam is a 245 m high, zoned, earthfill dam built in the late 1960's. The area came to the attention of the designers of the dam and investigations which included an airphoto study by J.D. Mollard, began in about 1961. The first hole was drilled in 1968 and extensive investigations and monitoring have been carried out since then. One slope, Dutchman's Ridge was stabilized through drainage (Lewis and Moore, 1988; Moore and Imrie, 1992).

Investigation and long term monitoring of the Dutchman's Ridge and Little Chief Slide area provide several important lessons for rock slope stability practitioners as follows:

- subsurface interpretations of geology and groundwater conditions based solely on surface and/or limited subsurface information can be significantly wrong.
- long term monitoring can reveal conditions which probably could not be determined from short term studies and monitoring.

During the investigations for Mica Dam and reservoir, the designers (a consortium of the engineering firms of Crippen, Acres and Shawinigan) were alerted to the possibility of rockslides by the discovery of a large existing slide, Little Chief Slide (Fig. 2), a few kilometres upstream from the dam (Mylrea, et al 1978). The two adjacent ridges, Little Chief Ridge and Dutchman's Ridge, were examined using airphotos and surface mapping. Both ridges were heavily forested, steep and crossed by numerous linear, trough-ridge features which trend across the slopes more or less parallel to this reach of the valley. There is evidence that these "linears" were

Fig. 2. Plan view of Little Chief Slide, Little Chief Ridge and Dutchman's Ridge and Mica Dam and reservoir. The "linears" are elongated troughs a few metres deep which generally follow the surface contours and the "linear/faults" are similar topographic features that are known to follow tectonic faults.

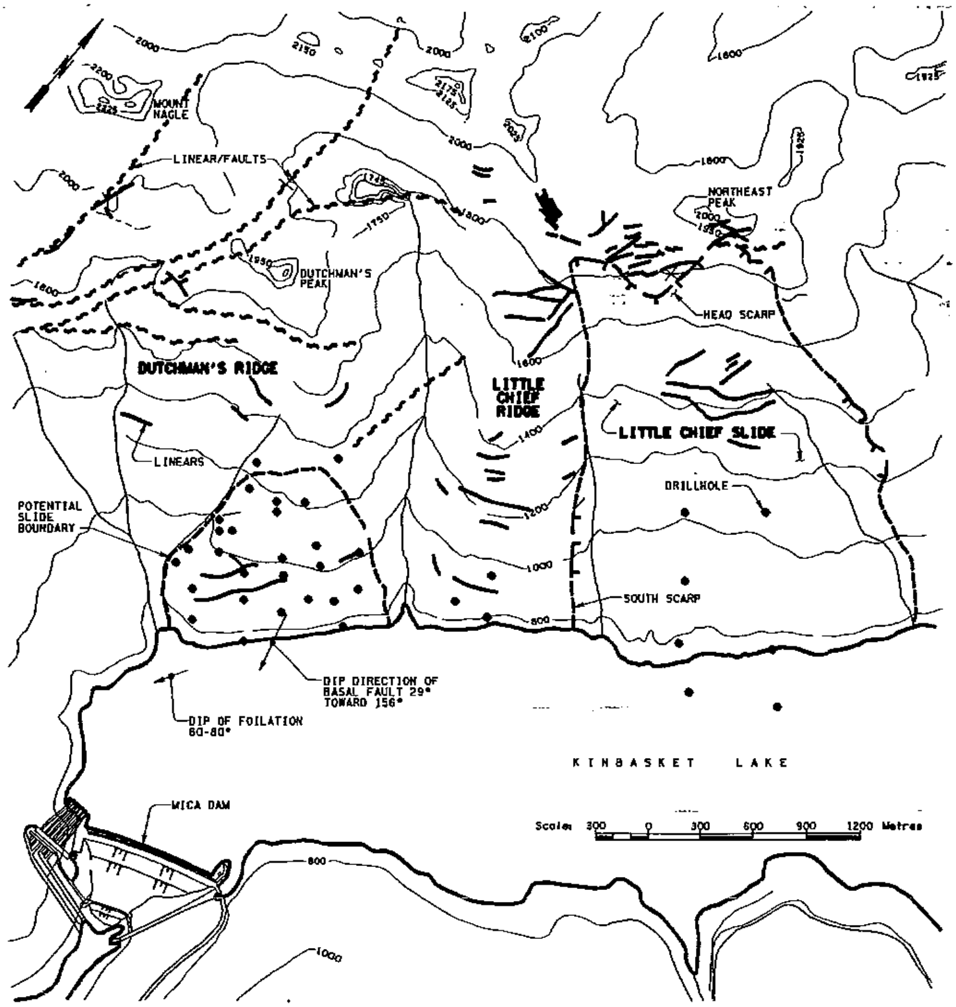
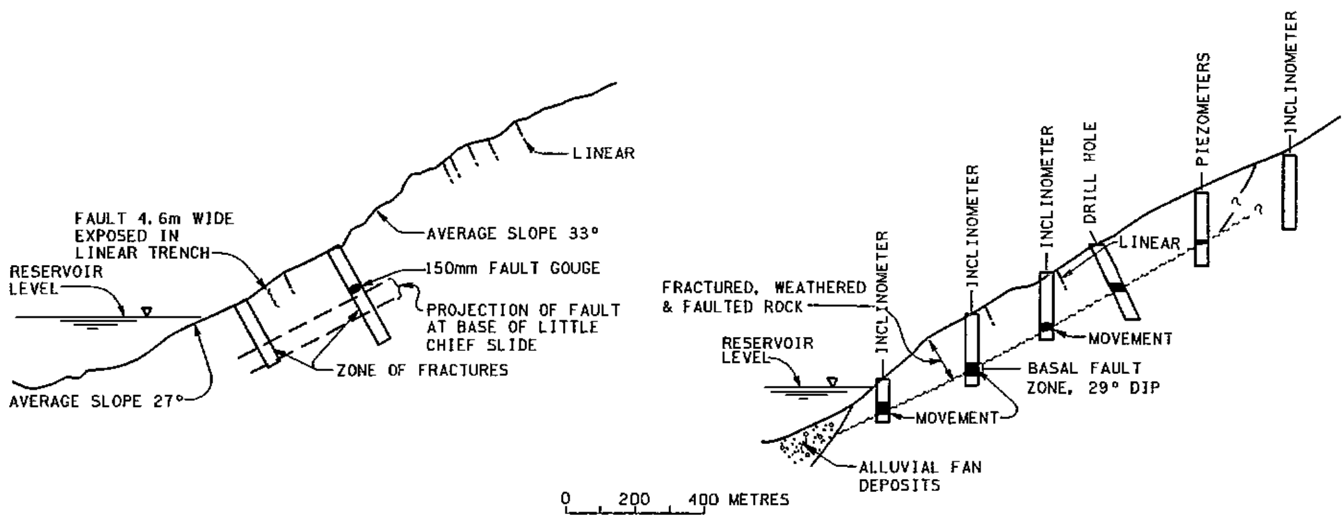


Fig. 3. Cross sections through Little Chief Ridge on the left and Dutchman's Ridge on the right. In spite of very similar topography and surface geology, the probabilities of rock slides from these ridges are judged to be very different due to the absence of an underlying fault zone at Little Chief Ridge and its presence at Dutchman's Ridge.



a result of rock movement along pre-existing faults or fracture zones rather than differential erosion and that this movement had been post-glacial, at least in part. Bedrock is interbedded gneiss and minor schist with foliation which strikes into the slope and dips steeply i.e. not adversely oriented with respect to slope stability. Given this surface information, the original investigators concluded that the slide potentials of both ridges were similar. Given the same information, most rockslide practitioners would conclude the same today. Now consider how this perception changed as the actual subsurface conditions were revealed.

Investigation of Little Chief and Dutchman's Ridges

In view of the proximity of these ridges to the proposed dam, the existence of the adjacent Little Chief Slide and the 1963 Viaont disaster, subsurface investigations of both ridges were considered prudent. These investigations included drilling, geological mapping, seismic refraction surveys and movement monitoring at both ridges and at the existing slide.

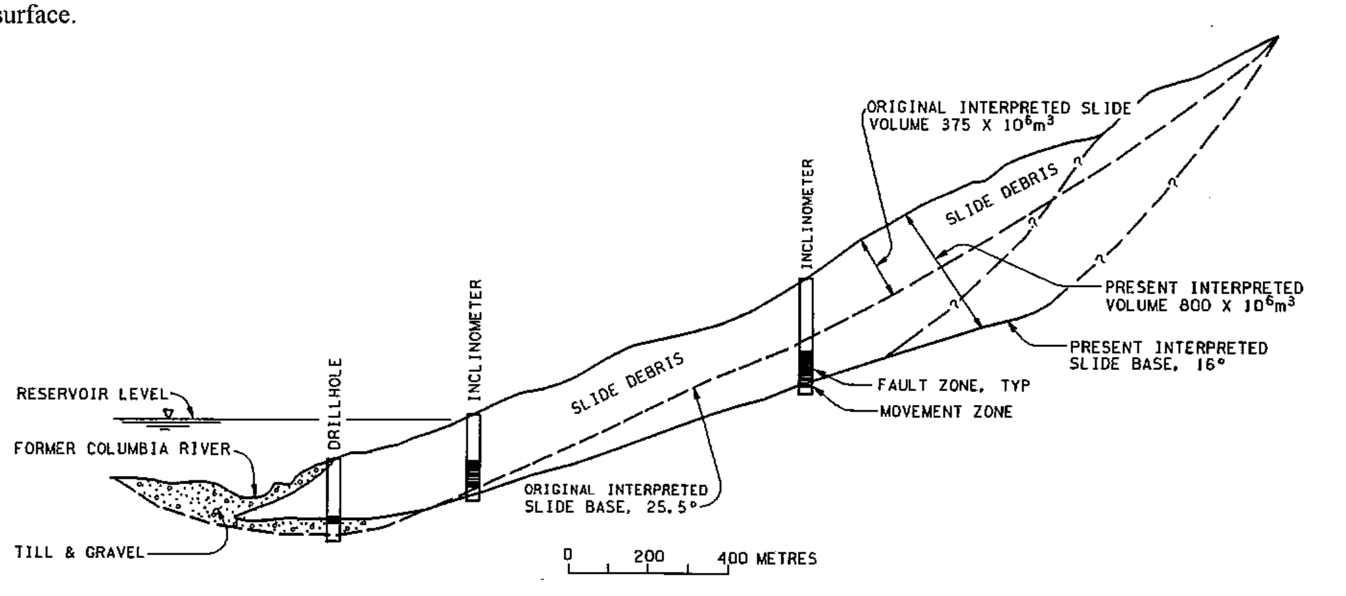
Three holes were drilled and several exploratory trenches were dug on Little Chief Ridge (Fig. 3). The rock is generally unaltered and not closely fractured, although there are some zones of intense fracturing. Some fractures were rusty at depths to several hundred metres. Faults that were determined to exist beneath the adjacent Little Chief Slide did not exist at their projected locations beneath Little Chief Ridge. Trenches excavated along some of the linears indicated that there had been some post-glacial movement but that this movement was localized along pre-existing fracture zones which dipped steeply into the slope. Based on this surface and subsurface information the designers concluded that the investigation had not shown very convincing evidence of the presence of an adverse, continuous weak plane which would be required for rapid sliding of the rock mass. Therefore, the possibility of the entire mass moving as a high velocity rockslide was considered remote.

Thus, the subsurface information at Little Chief Ridge considerably lessened the concern raised by the surface linears and by the adjacent existing slide. In contrast, was the result from Dutchman's Ridge (Fig. 3).

At Dutchman's Ridge, holes drilled in 1969 intersected highly fractured, rusty and decomposed rock averaging 110 m thick overlying a major fault zone. Core recovery was much less than that for Little Chief Ridge, averaging about 70% compared to 95%. After the fourth hole was drilled at Dutchman's Ridge it was recognized that the fault intersections were coplanar, that the plane dipped about 29° downslope, and that the toe of the overlying mass of about $115 \times 10^6 m^3$ of rock would be inundated by the reservoir. This fault was not seen at the surface until it was exposed in 1986 in an extensive bulldozer trench. Thus, two adjacent ridges with very similar surface appearance presented widely disparate risks. Not until the subsurface was investigated did this become apparent.

As a result of the investigations, the original designers proposed about 450 m of tunneling to provide additional geological information, access for instrumentation and drainage. This work was not carried out. Instead, slope monitoring using surface surveys, inclinometers and near-surface tiltmeters was instituted and Mica Dam was built to withstand overtopping waves

Fig. 4. Cross section through Little Chief Slide, an existing slide about 3 km upstream from Mica Dam. The initial subsurface interpretation based on drill core and seismic refraction surveys indicated a slide of $375 \times 10^6 m^3$ on a 25.5^0 surface compared to the current interpretation based also on monitoring results which indicate a slide of $800 \times 10^6 m^3$ on a 16^0 surface.



If movements were detected during filling or future operation, the stability of the slope was to be reviewed. This long-term monitoring proved invaluable as renewed movement along the fault was triggered by reservoir filling and a drainage system was subsequently constructed to stabilize the slope as discussed below.

Little Chief Slide

Long term monitoring also has proven to be valuable at Little Chief Slide. This slide (Fig. 2) was investigated in 1968-69: six holes were drilled and four of these were instrumented with inclinometers. Surface monument surveys were also instituted. The original investigators were concerned that the toe of the slide might rest on liquefiable sand or that strength might be lost for some other reason, resulting in a rapid acceleration of the slide mass. As part of the evaluation of the hazard this slide presented to the proposed Mica Dam, geological cross-sections were developed (Fig. 4). A choice was made based on one hole to select the upper of two possible slide bases because of its correlation with a seismic refraction boundary. These cross-sections indicated that the original dip of the slide surface was about 25° and the volume of the slide was about $375 \times 10^6 m^3$. The geological investigations showed that rapid acceleration was unlikely. Nevertheless, knowing the uncertainties of subsurface interpretations, the original designers had the foresight to install one inclinometer to a much greater depth than the interpreted slide base.

Monitoring of the inclinometers at Little Chief Slide was carried out and the results were reviewed periodically. Eventually downslope creep movements were detected which showed that the actual slide base was the lower of the two possibilities. This realization resulted in a new cross-section with a dip of the base of about 16° and a volume about 800×10^6 m^3. This is equivalent to a 40% decrease in the ratio of shear to normal forces at the slide base and a 113% increase in the estimated volume. Both the original and the current interpretation are reasonably consistent with measured residual frictional strengths which, as is often the case for large rockslides, varied widely. Similarly, water levels varied from hole to hole and were subject to considerable interpretation. Fortunately this new information did not change the original conclusion that the possibility of a rapid acceleration of Little Chief Slide was unlikely. Nevertheless monitoring is to be continued.

Thus, long term monitoring such as that instituted at Little Chief Slide can provide a valuable check on the subsurface interpretation. There is no doubt that such monitoring could lead to fundamentally different subsurface interpretations at other sites as well.

Stabilization of Dutchman's Ridge

Design and construction of the drainage system to stabilize Dutchman's Ridge also led to some conclusions that could be important to rock slope stability practitioners, namely:

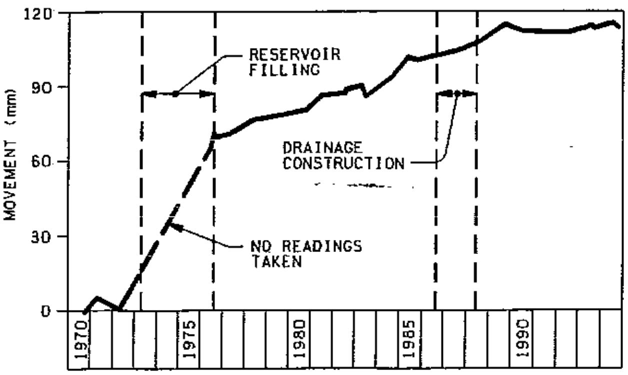
- deciding a course of action is largely based on a prediction of future slope performance which is extremely uncertain in most cases.
- the observational method is probably the only way and certainly the most economical way to design drainage systems for large complex rock masses.
- instrumentation must be in place to allow thorough, accurate and timely observations to be made which in turn can lead to improved design and economic gains.
- extreme contrasts in permeability can compartmentalize groundwater regimes in fractured and sheared rock.
- small increases in factor of safety can provide adequate safety and can significantly decrease deformation rate.

This potential slide consists of 115×10^6 m^3 of gneiss overlying a fault zone which dips 29° towards the reservoir (Fig. 2 and 3). The surface slope of the ridge is about 35° and it is only 1.5 km upstream from the dam so the impact of a rapid slide from this slope could be severe. The designers recommended a short adit to provide further information and some drainage, but this was not agreed to by BC Hydro's Advisory Board at the time. Instead, the crest of the dam was widened to provide increased erosion resistance and a monitoring program for the slope was instituted. Should movement be detected, the stability of the slope was to be re-examined.

No movements were detected during the few years prior to reservoir filling, although the loosened, weathered nature of the rock above the fault indicated that there had been a long history of downslope creep. Movement was triggered during reservoir filling at a rate of about 20 mm/year while filling was in progress. These slowed to a steady movement of about 10 mm/year during normal operation of the reservoir (Fig. 5).

This movement prompted an investigation program which eventually amounted to 41 drill holes instrumented with 267 piezometers, 10 inclinometers and a continuous surveillance system (Fig. 6). The investigation confirmed that a large mass of rock was creeping into the reservoir along the fault zone. Since there was no evidence of very large displacements it was considered to be a first time slide with some potential for rapid acceleration.

Fig. 5. A record of the movement along the fault zone underlying Dutchman's Ridge during reservoir filling and stabilization by drainage. Movement currently continues at the post-drainage rate of one or two millimetres per year.



At this point, BC Hydro, the owner of the dam was faced with the difficult decision of whether to continue monitoring or to proceed with stabilization. Opinions ranged from "Why didn't we stabilize this threat sooner?" to "Why waste any more money on something moving so slowly? To assist our decision making we retained a panel of advisors, Dr. R.B. Peck, Dr. E. Hoek and Dr. C. Allen. The arguments in favour of continued monitoring were quite persuasive as were those in favour of stabilization (Table 2). The decision had to be made when B.C. was in a severe recession: revenues were down and all capital expenditures not absolutely necessary were to be deferred.

Weighing against this was the remote possibility of loss of life and billions of dollars in damage. Even disrupting operation of the reservoir would be costly; for example, holding the reservoir down 30 m for a single year could result in $36 million in lost revenues. In addition, the costs due to diminished public confidence in the corporation and increased concern over public safety could have been very large. Although some considered that a rapid slide was not at all likely, all agreed that any large slide movement would be extremely disruptive to the operation of the generating station. Therefore, it was decided to proceed with stablization.

Before the drainage began, a comprehensive system of piezometers was installed to monitor the construction and thereby allow appropriate changes to the layout to be made as actual conditions were revealed. For this purpose it was useful to have the multiple piezometer system of Westbay Instruments available which allowed installation of piezometers every few metres along a drill hole. Packers used to seal between the piezometer completion zones proved to be very successful in the highly fractured rock, although this was uncertain at the start (Meidal and Moore, 1996).

It is known to be extremely important to have short completion zones right at the features of interest in layered rocks because abrupt and often unpredictable changes in water pressure can occur over very short distances. For example, through use of the Westbay System a pressure head of 75 m was discovered trapped beneath a 150 mm gouge layer at the base of the slide in an area that was formerly thought to be dry. Subsequent drainage of these pressures allowed deletion of almost 3/4 of a million dollars worth of construction from elsewhere on the job and still achieved the objective of offsetting the effect of the reservoir. Thus the cost of the closely spaced piezometers was easily justified by this one observation alone.

Closely spaced instrumentation and readings are required in these complex slopes otherwise important, but isolated, zones and/or transient peaks of water pressure can remain unobserved. Multiple piezometer zones and closely spaced instrumentation locations complete with continuous data acquisition systems on critical instrumentation are now commonly used in recognition of this situation.

At Dutchman's, an 800 m adit, mainly just below the basal fault zone, and 17,000 m of drainholes were constructed to reduce the average head at the base from 36 m to 10 m. There was a rapid response to drainage (Fig. 6) and having proper instrumentation in place allowed us to adjust the numbers and locations of drill holes and the layout and length of the adit to achieve our objective in the most economic manner. No reasonable amount of pre-construction information could have allowed an efficient fixed design to be determined. The observational method had to be used to minimize the number of drain holes and length of adit.

Widespread reduction in water pressure was achieved both above and below the basal fault zone although significant pressure differentials remain across the basal fault (Fig. 7 and 8). The movement along the fault zone has almost stopped (Fig. 5) even though the increase in factor of safety was only in the order of 5%.

Downie Slide

Downie Slide is a $1.5 \times 10^9 m^3$ existing rockslide which was stabilized by drainage prior to filling the reservoir behind Revelstoke Dam in 1983-84 (Enegren and Imrie, 1995; Imrie et al, 1992). Downie is so large that it was not recognized as a slide for many years and the first drilling was done to investigate the suitability of its toe for a dam site. The first investigations of it as a slide began in 1965 as part of the design work for Mica Dam which is located some 50 km upstream (Piteau et al, 1978). Downie has been monitored continuously, investigated in phases and was stabilized. Much has been learned about Downie during that time, some of which is applicable to other rockslides, namely:

Table 2. A synopsis of some of the considerations necessary to decide whether or not Dutchman's Ridge needed to be stabilized. Most major decisions in rock engineering rely heavily on judgments which can vary over a wide range between practitioners even when there is extensive data.

CONTINUE TO MONITOR	OR	STABILIZE
The critical period, during reservoir filling and the first few years of operation, was over.	OR	The critical period could be a combination of groundwater level, reservoir level and mechanical deterioration not yet experienced.
Before a catastrophic movement, there would be lots of warning signs which our monitoring would reveal.	OR	One of the most likely "triggers" is an earthquake which could initiate rapid movement without warning.
There had been substantial movement, the slip surface was at "residual" strength, so no acceleration should be anticipated.	OR	The rock ridge is not a simple soil where the "residual" argument could be more persuasive. Vaiont accelerated after undergoing as much movement as Dutchman's Ridge.
The nearby Little Chief Slide is similar - since it slid slowly, Dutchman's would as well.	OR	Little Chief Slide's movement history is not known with certainty and it might not be as similar to Dutchman's as it first appeared.
Almost all of the existing slides in this metamorphic terrain had been slow.	OR	One or two of the existing slides slid rapidly.
The dam had already been designed to withstand overtopping.	OR	A wave could damage the spillway gates, releasing the top 10 m of reservoir, even though the dam would not be breached.

Fig. 6. Plan view of Dutchman's Ridge showing the extensive investigations and the drainage system installed to increase the stability of the ridge to at least its pre-reservoir condition.

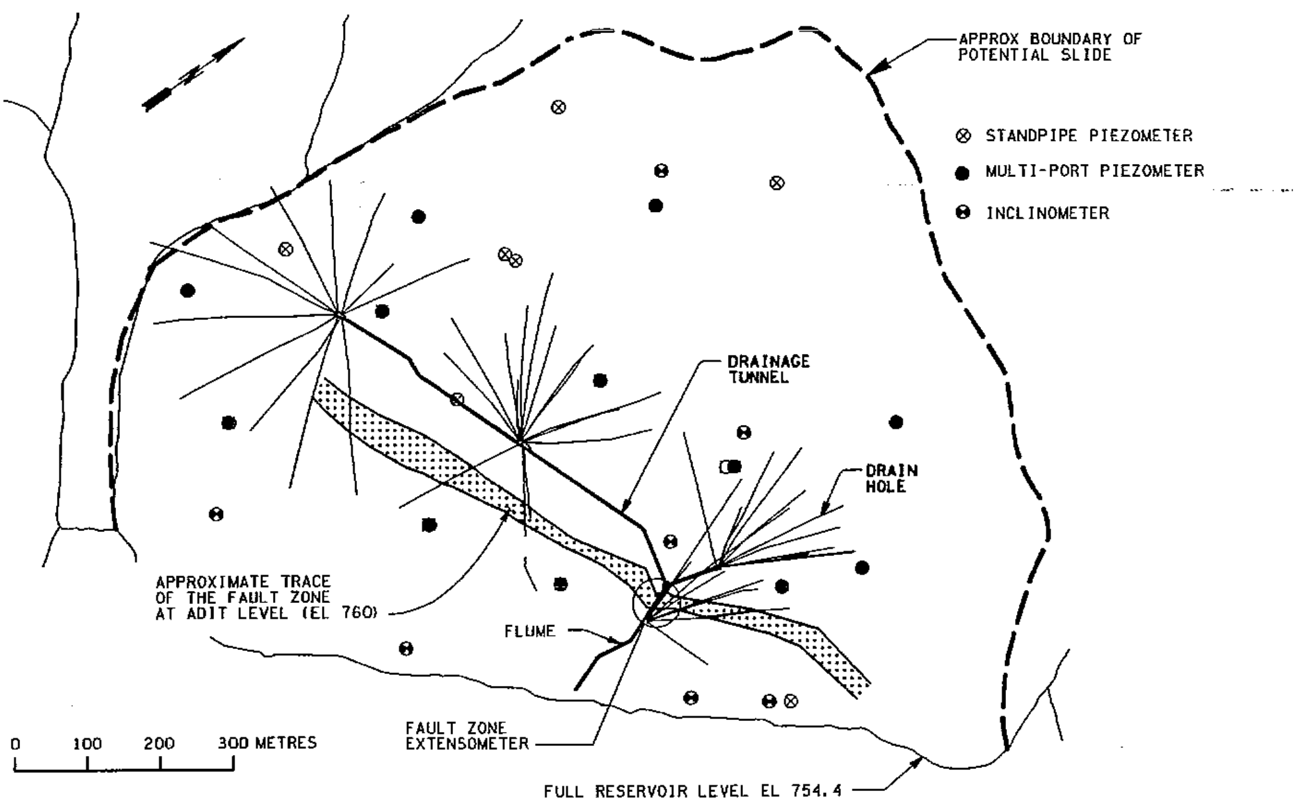


Fig. 7. Typical drop in piezometric levels at Dutchman's Ridge when drainage paths were intersected. Even after drainage, significant pressure differences can exist across less permeable barriers such as fault zones which divide the rock mass into isolated compartments which have to be individually drained.

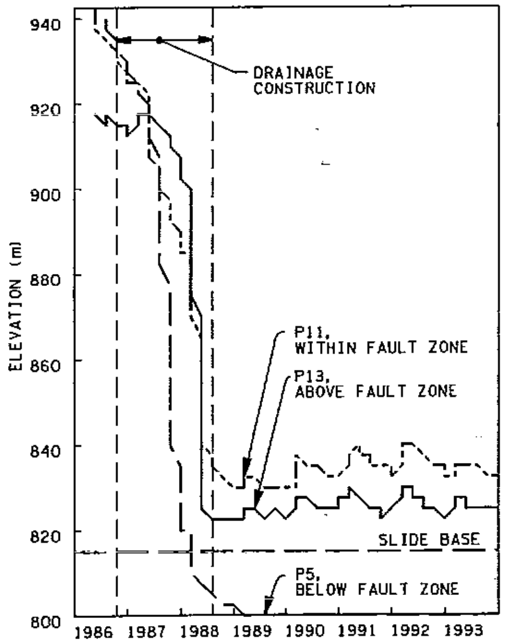
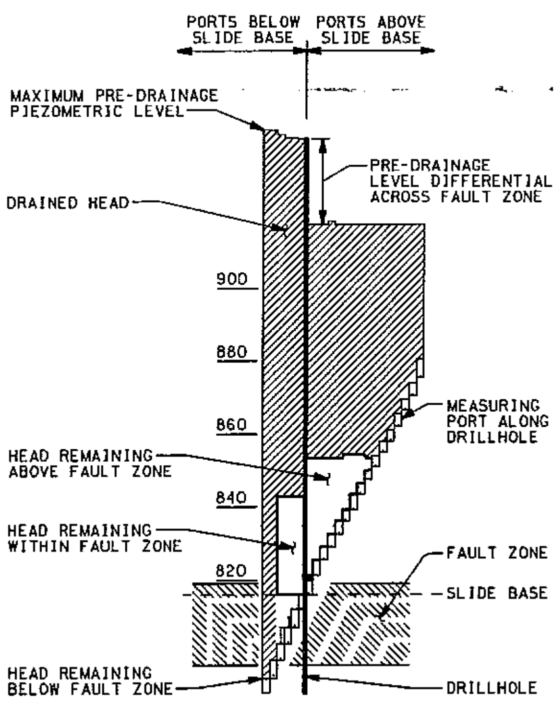


Fig. 8 A graphic log of multiple piezometers installed in a drill hole crossing the basal fault zone at Dutchman's Ridge illustrates the pressure differential which can still exist across faults in spite of extensive drainage. At one location a 75 m differential existed across 150 mm of gouge.



- even some huge rockslides can be stabilized economically using drainage.
- small changes in safety factor can result in marked changes in behaviour although deformation can continue over a significant range of safety factors.
- it is very useful to understand existing groundwater conditions and deformations before they are disturbed.
- under certain conditions, possibly resulting from slide movement, permeable openings ("windows") can exist which allow drainage to occur across thick extensive fault zones
- drainage systems in this type of rock and monitoring systems deteriorate with time and significant maintenance costs should be expected.

Downie Slide covers about 9 km^2 and extends 2.4 km along the west bank of the Columbia Valley (Fig. 9). The slide material extends 3300 m up from the river to a 125 m high headscarp. Generally the surface slope of the slide is 18° except near its toe where the slope is closer to 40° . The slide has a maximum thickness of 245 m and is considered to have failed on pre-existing foliation shears undercut by river or ice erosion. It is thought that slide movement began in early post glacial time (9,000 - 10,000 years ago) when pore pressures in the bedrock were high following glacial melt. No high velocity sliding has occurred (Imrie and Bourne, 1981) but total movement is in the range of 250 - 300 m.

At Downie Slide the stratigraphy consists of interlayered mica schists, mica gneisses and quartzite dipping easterly towards the Columbia river. The slide mass is a series of large intact blocks separated by open steep fractures. The lower shear zone (Fig. 10) which forms the base of the slide ranges from 15 to 20 m thick and consists of broken rock, crushed mica schist and gneiss, and clay gouge. An upper shear zone was encountered in several drill holes but this zone is narrow and discontinuous and is likely to have contributed little to the total downslope displacements.

During the application for a license to build Revelstoke Dam, Downie Slide had considerable attention from the media and became one of the issues of concern to the public. It was agreed as a condition of the license that a drainage system would be constructed to stabilize the slide and that a comprehensive monitoring system would be installed to evaluate the effects of the drainage system and the reservoir which would flood the toe of the slide by about 70 m.

From the start of investigation in 1965 to 1974, there was widespread sliding at the toe of the slide and movement at depth averaged about 10 mm/year. In 1974-75, an exploratory adit was driven into the toe of the slide mainly to investigate the feasibility of tunnelling in rock slide debris. Considerable water flow was intersected (up to approximately 4500 l/min) and at the same time movement at depth slowed to about 2 mm/year and the toe sliding was noticeably reduced.

Unfortunately very little information was available on the pre-adit water pressures in the slope so it was not possible to calculate the change in stability with any accuracy. As river flows began to be regulated by Mica Dam almost at the same time as the adit was driven, it could be argued that the reduction in movement at the toe was due to reduced river erosion along the toe. In many cases it is important to obtain background piezometric and movement information before a slope is disturbed so that cause-effect relationships remain clear and stabilization can be demonstrated through calculations based on before and after data.

The drainage program was continued in several phases prior to reservoir filling which began in 1983. Eventually 2430 m of adit and 13,600 m of drainholes were constructed for a cost of about $25 million (Cdn. 1984).

The Columbia Valley near Revelstoke has a temperate climate with an average annual precipitation of 1500 mm, much of which falls as snow. Downie Slide is a highly fractured layered rock mass which has a water table parallel to the ground surface and an average hydraulic conductivity of 10^{-5} cm/s. Although there are two main shear zones parallel to the layering which are of lower conductivity, there are a sufficient number of higher conductivity "windows" across them that the mass can be considered homogenous. These windows in the gouge layers are probably the result of disruption caused by the significant slide movement. This groundwater regime is in direct contrast to the situation at Dutchman's Ridge (Moore and Imrie, 1991) where the groundwater is far more compartmentalized, presumably because of significantly less movement and numerous low permeability fault zones. At Downie, tests showed that reducing flows from two drainage holes in the original drainage adit caused an increase in piezometric pressure over 600 m away.

Monitoring during reservoir filling indicated the pressure increases were less widespread than considered during design and that movement rates remained low. The overall factor of safety has improved in the order of 7-11% and movement has remained slow but still continues. It is clear that movements can occur over a range of calculated safety factors of at least 10%, thus the common assumption that the factor of safety is 1.0 when movement is occurring is not strictly true.

Fig. 9 Plan view of the 1.5 x 10^9 m^3 Downie Slide showing the original depressurization done using drainage adits and drain holes and the deterioration in the efficiency of this drainage system during the next ten years. Similar deterioration has been noted at Dutchman's Ridge and at the Revelstoke Damsite.

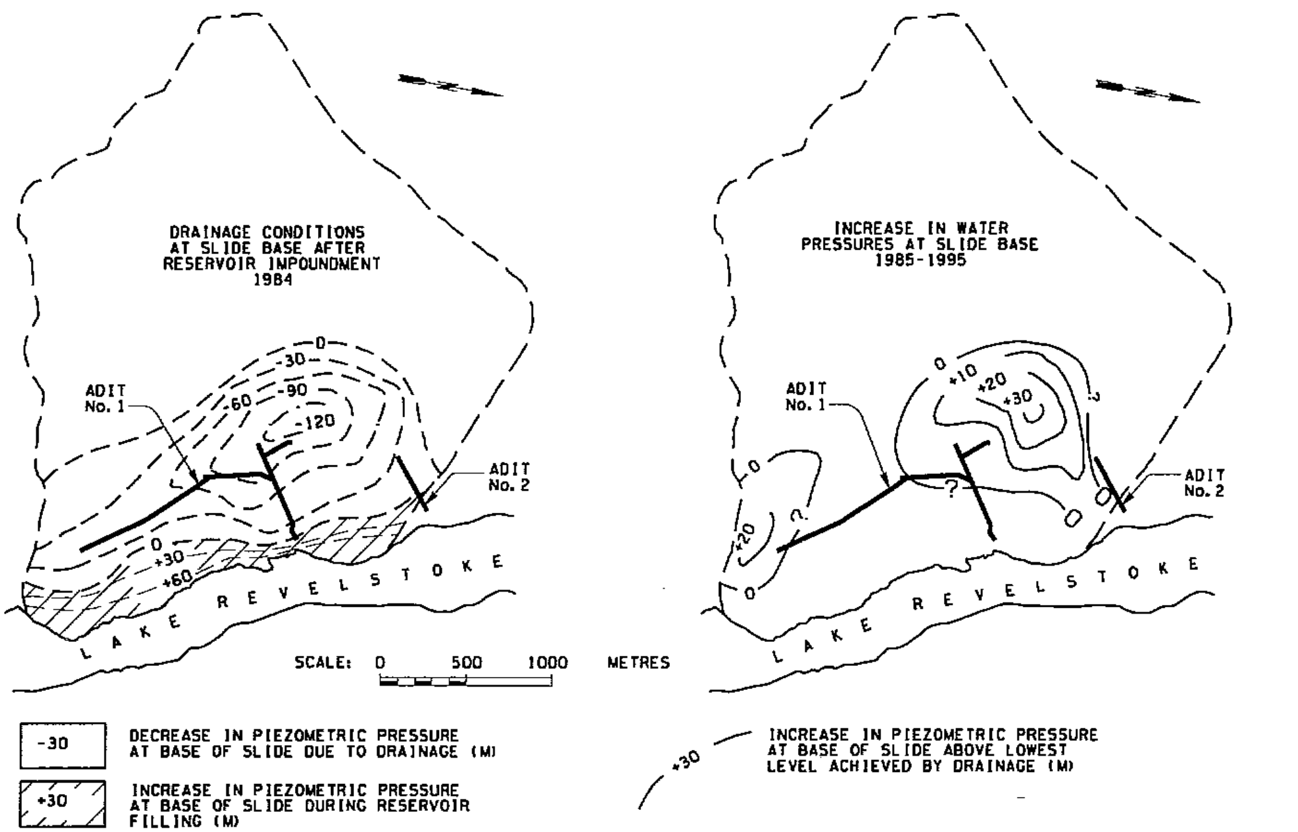
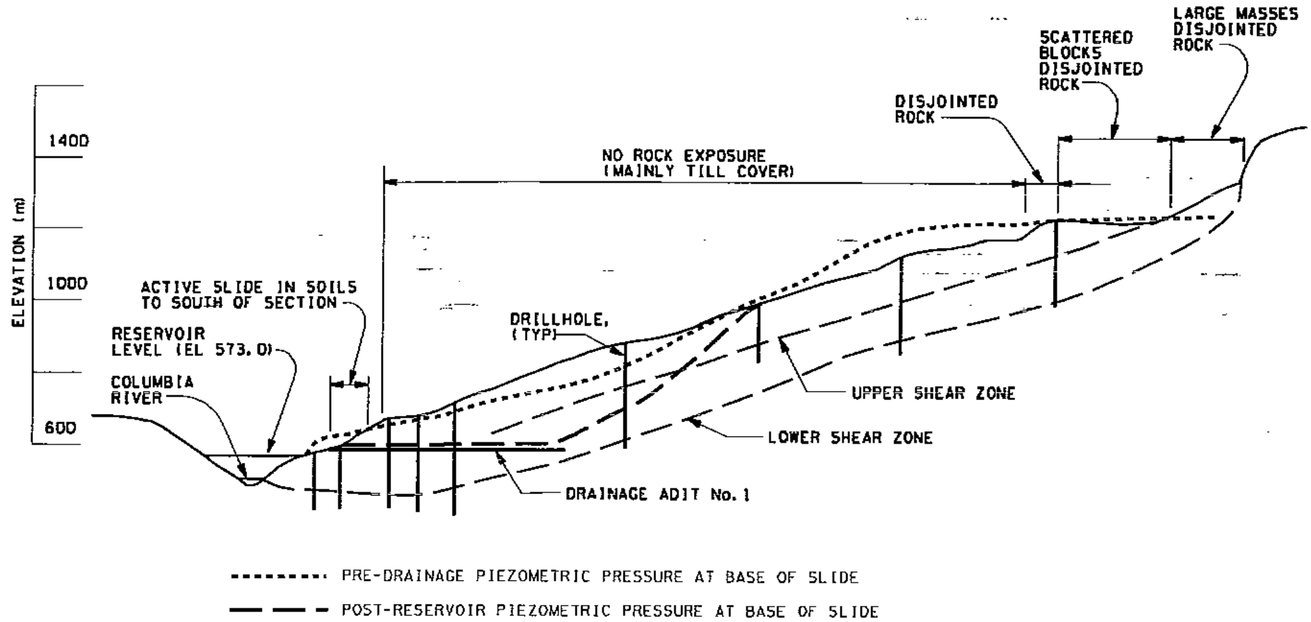


Fig. 10 A cross section of Downie Slide showing the lower shear zone developed parallel to the foliation in the schists and gneisses which form the slope. The overall slope of the surface and the lower shear zone is about 18° and the movement has been about 300 m.



Although the cost of stabilization was considerable, it is still relatively efficient since it is less than 2 cents per cubic metre stabilized. This is due in part to the high water pressures that were available for drainage. At Dutchman's Ridge where pressures were much lower on average, the equivalent cost was closer to 10 cents.

One aspect of drainage systems, such as this one, which first became apparent at Downie is the gradual decrease in efficiency that occurs over time (Fig. 9). Piezometric pressures at Downie and elsewhere have gradually increased from the drained minimums. The cause of these increasing pressures is not known. Possibilities include blockage of drainholes by hole collapse, calcite, eroded debris, algae or movement, as well as, blockage of fractures by calcite, redistribution of fine materials, closing of fractures due to increased effective stress or gas bubbles forming due to the decreased water pressures. Cleaning of the existing drainholes with air and water jets has had little effect. The decrease in the factor of safety over about the last 15 years has been about 1% which still leaves a sufficient margin, but future drainage could be required.

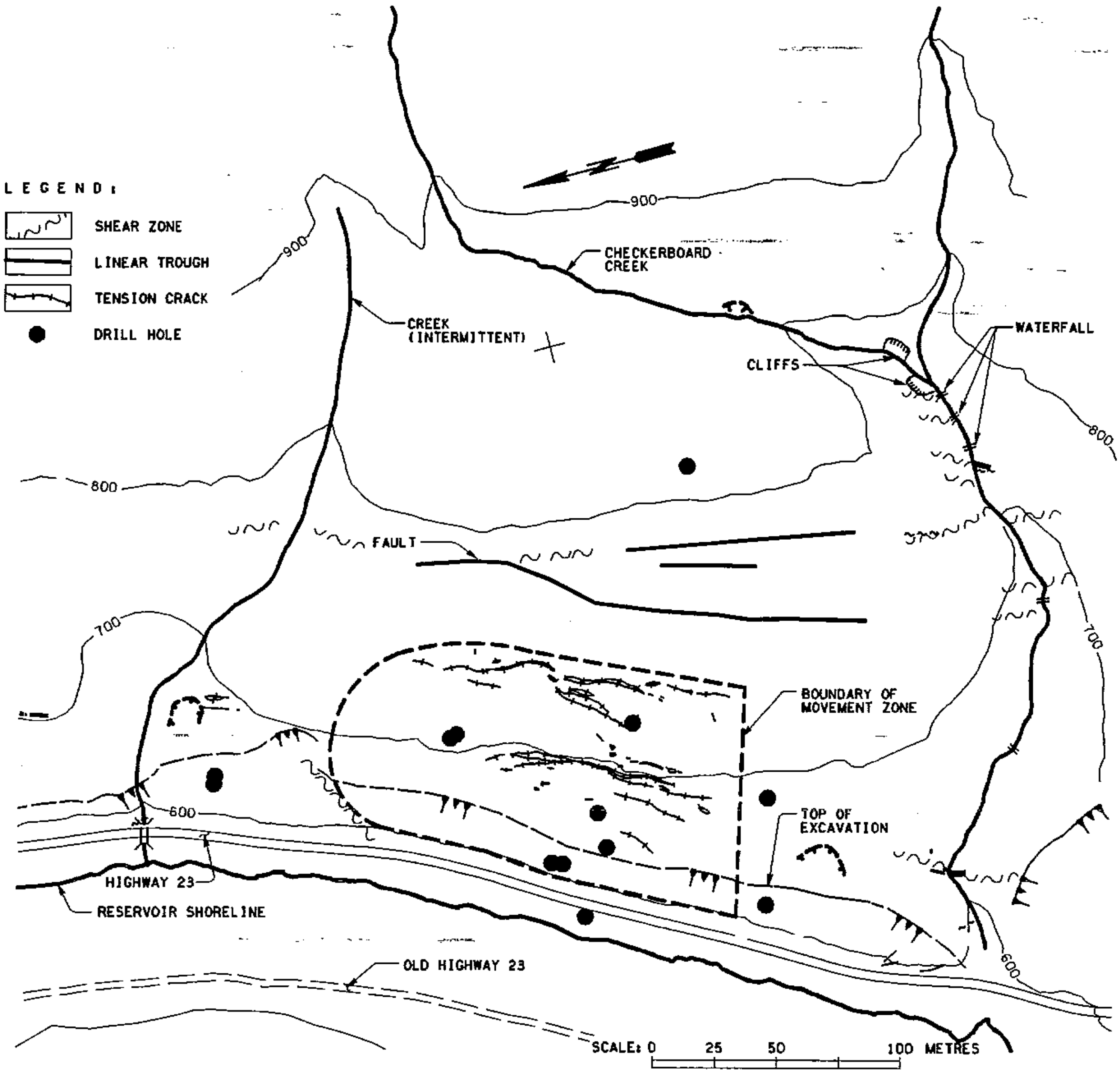
Checkerboard Creek

Not far downstream from Downie Slide and almost 400 times smaller in volume, the Checkerboard Creek rock slope is nevertheless a potential problem because it is only 1500 m from Revelstoke Dam. This slope was of interest during the late 70's when a highway was relocated across the toe of the slope. Interest was renewed in 1984 during the filling of Revelstoke Reservoir when fresh tensions cracks were discovered above the highway excavation. Several lessons were learned or relearned during the investigations and monitoring that have been carried out at Checkerboard, namely:

- investigations should cover areas well beyond the excavation and/or the potential slide boundaries.
- airphotos and/or helicopter reconnaissance are no substitute for walking.
- rock above the saturated zone possibly can be affected by transient pressures as groundwater percolates downward.

Investigations prior to highway relocation consisted of the usual surface mapping. This mapping was restricted to the steep slope into which the excavation for the highway had to be made. The rock which cropped out was hard, strong granodiorite and the excavation slopes were designed accordingly. No one saw, or at least no one recognized the importance of a series of post glacial tension cracks that were in the forest above the rock excavation (Fig. 11). Almost as soon as excavation began in 1978, the true nature of the rock was revealed - the only hard, strong rock was the exposed rock, the rest was fractured, weathered and loose. The excavated slopes were stabilized by adding vast amounts of rock support and flattening them as much as was possible. Several

Fig. 11. Plan view of the Checkerboard Creek rock slope which is about 1.5 km upstream of Revelstoke Dam. Movement on ancient tension cracks was renewed when the excavations to relocate Highway 23 were carried out.



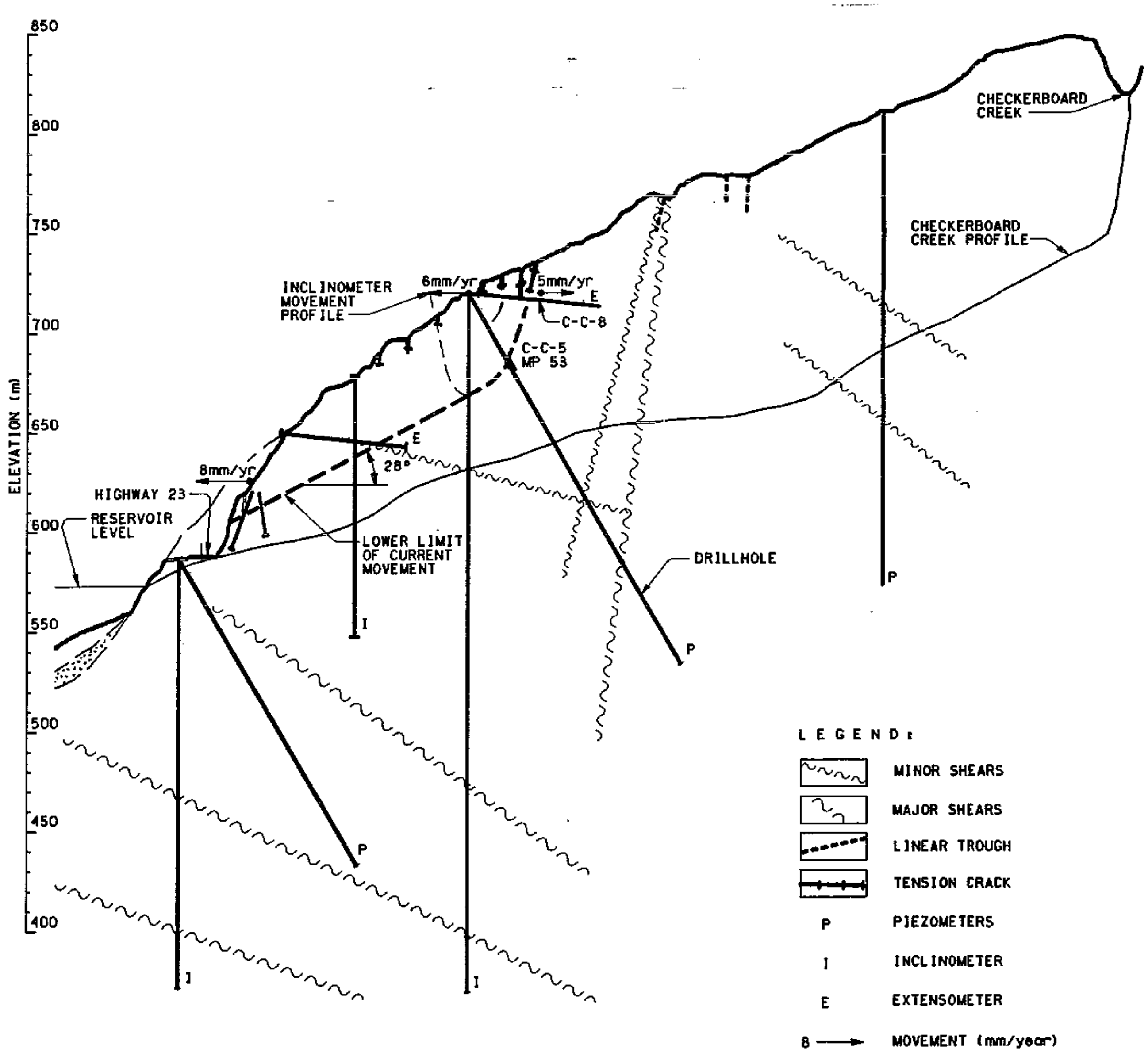
exploratory holes were drilled to provide information for the redesign during the first winter when excavation had stopped. Even then, no one had made note of the tension cracks in the forest above the excavation. The excavation was completed and the highway was relocated. Surface raveling continued and eventually it has amounted to the equivalent to about half a metre thickness of rock. The pieces were generally 0.1 to 0.3 m in diameter.

The next phase of the Checkerboard Creek investigation began during Revelstoke Reservoir filling in 1984. During filling, the entire reservoir was inspected by helicopter several times a week by slope stability practitioners to help ensure that no slope movements went unnoticed. Also, monitoring of previously identified potential problems was intensified and the highway cuts and fills were inspected regularly during filling. None of this work alerted designers to fresh movement along the old tension cracks in the forest above the rock cut. Although some of the fresh cracks crossed the access road to the top of the excavation, and some were open wide enough to walk into, they were not seen from the helicopter until they were identified on the ground. Clearly, important features can be missed if only air photos and aerial reconnaissance are used in forested areas.

Monitoring results indicated that the slope was deforming at a rate of 5 to 10 mm/year. This situation is not too serious above a highway excavation but the possibility of a wave overtopping Revelstoke Dam has justified several phases of investigation and monitoring which have provided excellent information regarding this slope. Initially, Checkerboard Creek was taken to be the uphill boundary of the moving area. This creek takes a very unusual south turn into a steep-sided, deeply-incised canyon parallel to the overall slope contours near El. 900. The creek outlines what looked like a classic, horseshoe-shaped, slide boundary. Drilling at the toe of the slope intersected major fault zones (Fig. 12) which could have been the base of a 20 to 55 x 10^6 m^3 slide, with the volume depending upon how this information was interpreted.

Additional drilling, mapping, instrumentation and monitoring has been done which shows that the faults dip moderately into the slope and that the Creek could have been diverted to the south by glacial deposits. Also, it has been conclusively shown that the current movements and post glacial tension cracks are restricted to a much smaller area and depth near the center of the slope. The volume that is currently moving is about 2 x 10^6 m^3: less than 10% of the volume interpreted from surface observations.

Fig. 12. Cross section of Checkerboard Creek showing the limits of the current creep movements. The cause of the linear troughs further up the slope is not known. The phreatic surface is irregular but generally parallel to the slope and always below the limit of current movement.

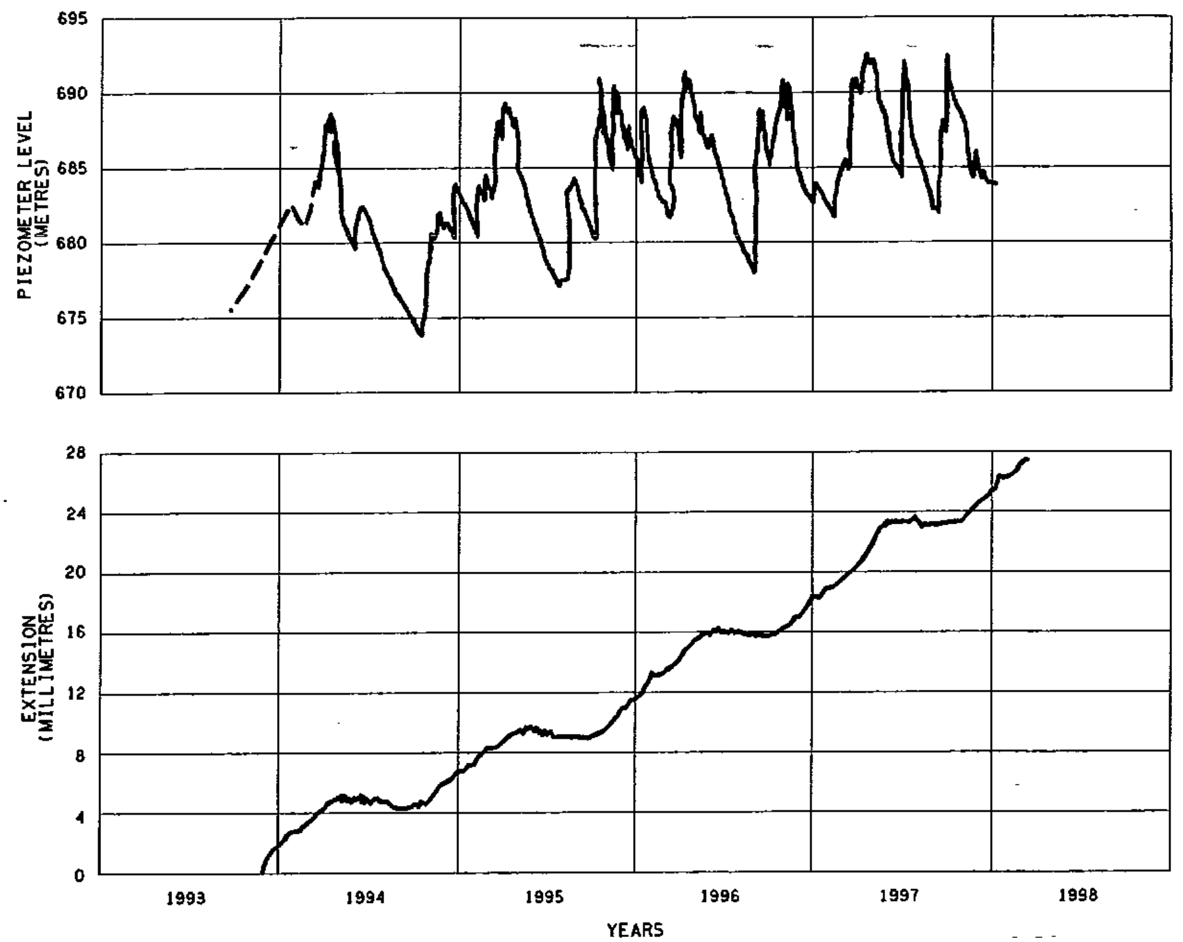


Movement of the Checkerboard Creek slope is now monitored using inclinometers; borehole extensometers, cable extensometers on the surface, a very sensitive crack meter and surveys of many surface monuments. Water pressures are monitored using multiple piezometers and were measured between downhole packers during drilling. Visual inspections are carried out regularly and a lot of the instrumentation is recorded on data loggers. The behaviour of the slope is reviewed regularly.

Current movement is diffuse and all above the phreatic surface. The displacement is greatest at surface and decreases gradually with depth. With the exception of one inclinometer, the direction of movement is directly downslope. The exception is an inclinometer near the upstream boundary which shows movements 45^0 to the south of downslope. The rate of movement is consistent between the various monitoring devices and amounts to about 5 to 10 mm/year (Fig. 13). An annual cycle is recorded showing most movement in the Fall to Spring period with little movement in the Summer. Curiously, the start of the movement cycle is associated more with the increasing groundwater levels and the end with decreasing levels, rather than with any particular level.

An assessment is now being carried out to decide what if any risk this slope poses. Difficult questions include: What could be the ranges of volumes and velocities? Could a slide be triggered by an earthquake, if so what earthquake? Would there be a warning period long enough to mitigate the consequences? What is the probability associated with these combinations? What would be the consequences on the dam, the highway and the reservoir? Are these risks acceptable? If not, what should be done?

Fig. 13. Correlation of movements measured at Checkerboard Creek above the phreatic surface and pressures nearby but below this surface. Movement occurs in the Fall-Winter period and is associated more with increasing pressures than with any particular pressure level.



Wahleach

The Wahleach hydroelectric project was built in 1951-52 to develop the power potential between Wahleach Lake and Fraser River near Hope, B.C. (Fig.14). Tunnelling for the incline and connecting tunnels was found to be much more difficult at the Fraser River end than expected but the underlying cause of these poor ground conditions was not recognized. Even in 1981, when greatly increased leakage from the power conduit into an upper access tunnel was remedied, the real problem was not recognized. Investigations following another large leak in January 1989, this time from a ruptured steel lining of the power conduit, finally revealed the underlying cause of the problem. The intensive investigation and monitoring program and, later the relocation of a section of the power conduit provided several insights into this rock slope which may be pertinent to other slopes, namely:

- unless the underlying problem is recognized, the sympton can be cured instead of the problem.
- monitoring is a powerful tool to provide insights into geological conditions and slope processes.
- deformation styles can vary from place to place in a single rock mass at a single time

The western end of the original power conduit included a nearly horizontal upper tunnel, a surge shaft, an inclined shaft and a lower, nearly horizontal tunnel (Fig. 15) excavated in a rockslope which rises from the Fraser River at El. 21 m to a ridge crest at El. 1100 m. Linear, ridge-trough couplets up to 10 m deep trending along the contours cross the steeper portions of the slope.

During construction of the incline and western end of the upper tunnel, unexpected conditions, including large water inflows, open rusty fractures and shears, and seepage soon after rainfalls, were encountered at depths greater than predicted. Even though the ridge-trough couplets which cross the slope had been seen, the concept that this slope had undergone downslope deformations at depth was not recognized. The tunnelling problems were overcome using increased rock support and other traditional methods.

The project performed satisfactorily until 1981 when a sudden increase in leakage from the upper access tunnel was observed. When this incident was investigated, two steel, drainage pipes embedded in the concrete surrounding the main lining were thought to have rusted through, allowing water from the unlined tunnel to escape to the access adit. The drainage pipes were plugged and the leakage stopped. This solution was considered successful.

In January 1989 another larger leak was detected from the upper access adit. When the conduit was dewatered the 2 m diameter steel lining was found to be ruptured about 30 m upstream the intersection of the incline and the upper tunnel and buckled at 8 other locations. Extensive investigations and monitoring were carried out and the underlying cause of both leakages and the poor tunnelling conditions was determined to be slope movement which had amounted to almost a metre since construction. The section of power conduit that could be damaged by further slope movement was relocated (Ripley and Beaulieu, 1992) and the slope is thoroughly and continuously monitored because of the threat to surface facilities.

The geology, groundwater and movement of the slope are now well understood. The granodiorite rock forming the slope is generally hard and strong but locally breaks easily into sand-sized material when squeezed or lightly struck due to a myriad of micro fractures. Rusty fractures, often with soft coatings or granular infillings, are spaced an average of 0.2 m apart near the ground surface. A gradual trend to fewer fractures, less rust, less soft or granular material, less core loss and tighter interlocking of the fracture surfaces is evident with depth where the average block size is closer to 0.5 to 1.5 m on edge. The general rock quality is characterized by transitional rather than abrupt changes with few if any exceptions.

The fractures have a wide range of orientations with two vague concentrations: one strikes roughly across the slope, the other strikes roughly downslope and both dip within 20° of vertical (Fig.14).

Shear zones spaced 20 to 40 m apart are generally less than a few metres in thickness with little or no clay gouge. They can be continuous for hundreds of metres. The shear zones strike roughly across the slope (+-45°) and dip steeply. Of note is the lack of through going discontinuities with downslope dips less than 45°.

Piezometric measurements show that the phreatic surface is generally below the zone of slope movements, but local transient pressures build up within the zone during rainfalls and possibly also during snowmelt.

The slope movements at Wahleach fall into three general categories: near-surface sliding of local areas, current movement, and ancient movement. The current movement which involves about 20 million cubic metres of rock to depths of about 60 - 120 m has been monitored by a variety of methods since 1989.

Inclinometers provide the best overall picture of the current movements and show the following characteristics (Fig. 15):

1. Generally within the moving rock mass the movements are diffuse but at a few locations they are concentrated.
2. Towards the uphill part of the slope the movements tend to be more concentrated within zones and the greater rates of deformation tend to be at the base of the moving rock mass.
3. The movements generally are not associated with any notable or unusual geological features in the drill core.

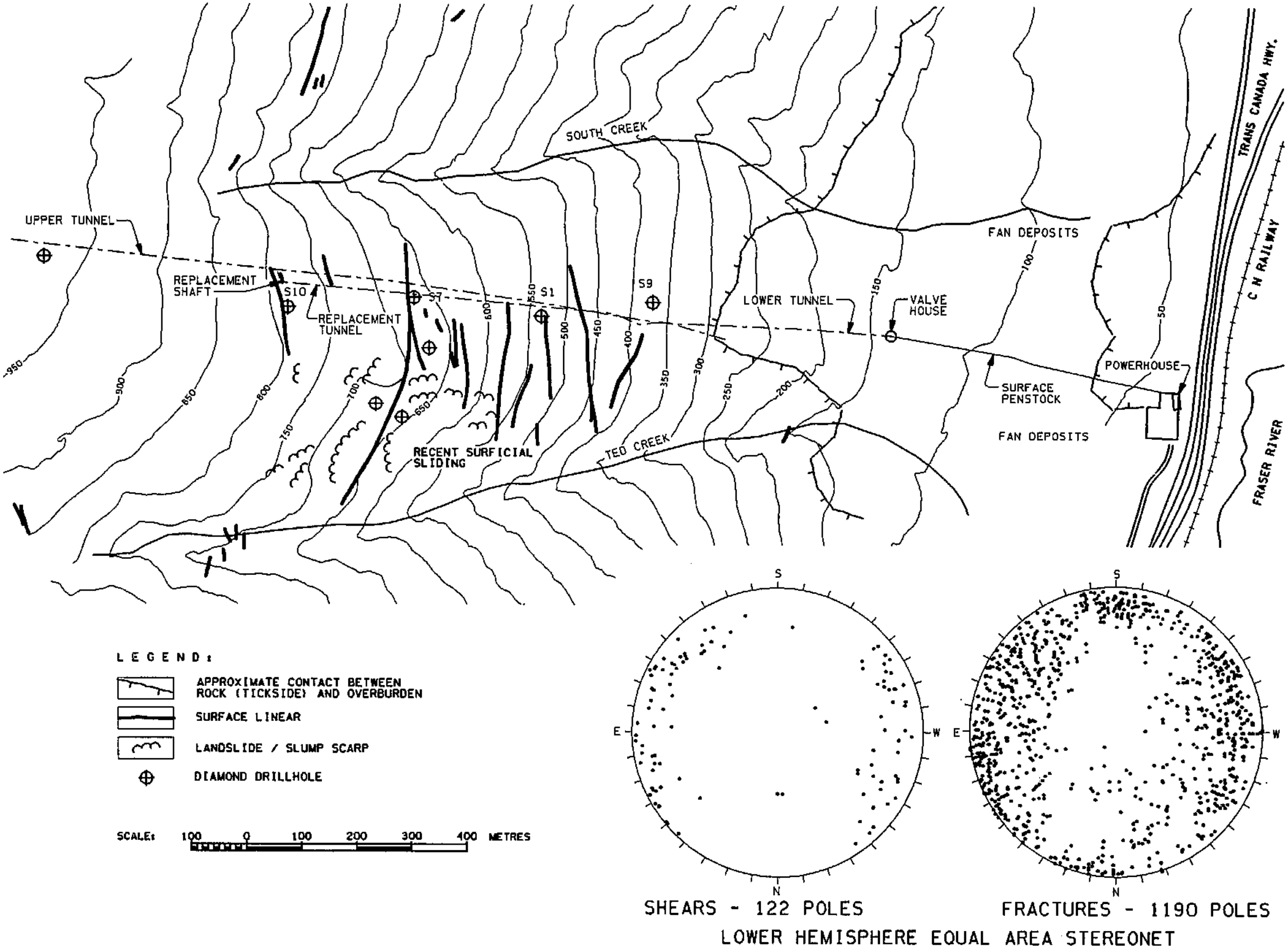


Fig. 14. Plan view of the Wahleach slope showing linear troughs crossing the prominence between the two creeks. Stereonet plots of fractures and shears show steeply dipping fractures and shears with widespread strikes.

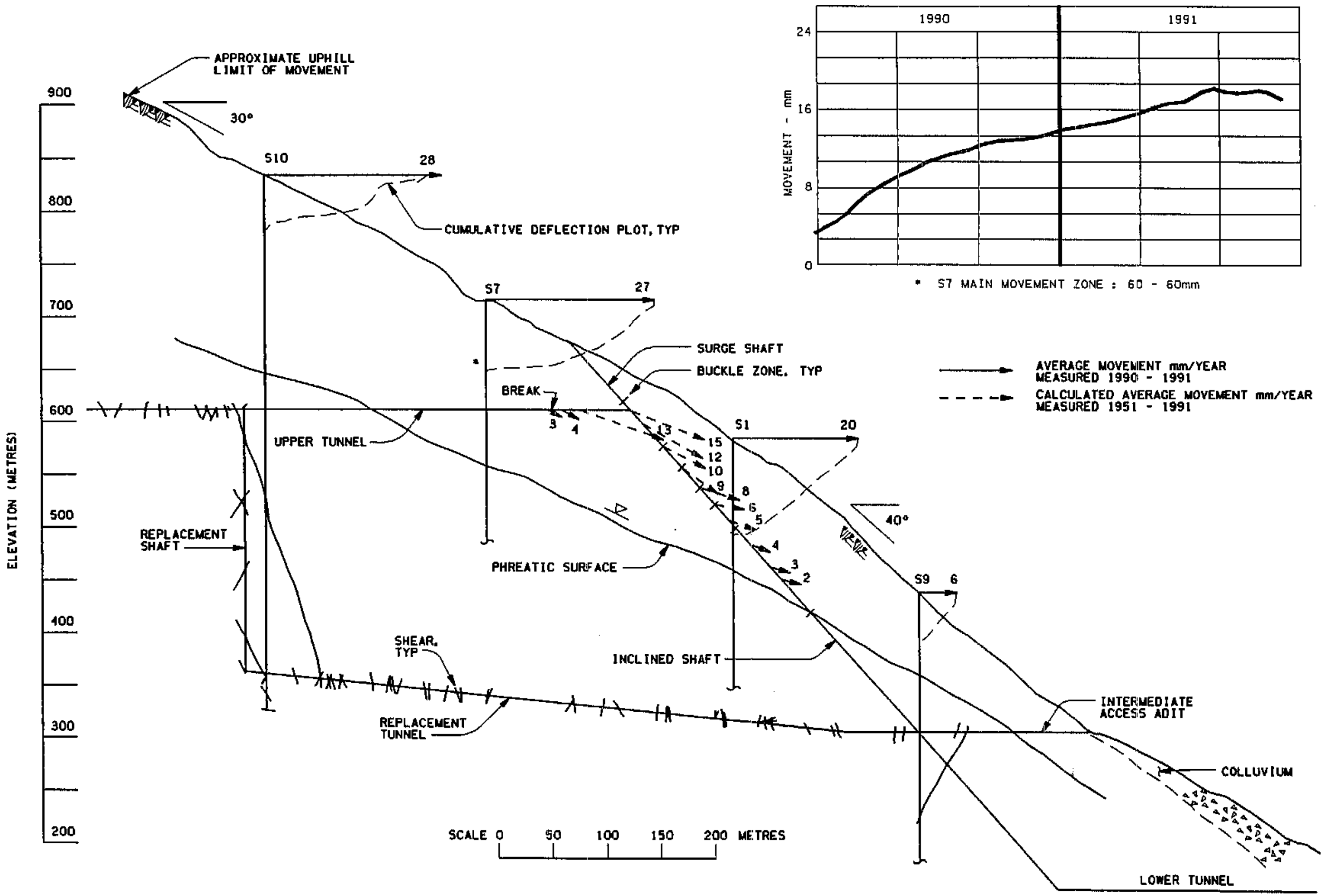


Fig. 15. Cross section of the Wahleach slope showing movement measured in inclinometers and a survey of the position of the steel lining in 1990 compared to its 1951-52 position.

4. The movements are seldom, if ever, associated with downslope dipping discontinuities.
5. The movements at depth have resulted in the larger cumulative displacements being at the ground surface and near the uphill boundary of the moving rock mass.
6. The rates of movement have been relatively constant during 1990 to 1998 and are slower than they were in 1989.
7. The movements in 1990-1998 have resulted in surface displacements ranging from 4 to 40 mm/year.
8. The movements in the inclinometers do not exhibit an annual cycle but the extension of the break in the steel lining in the upper tunnel does. Fall, Winter and Spring rates are more rapid than Summer rates.
9. The movements are above the long term phreatic surface, but are within an area where transient water pressures exist locally during rainfalls.
10. Dewatering of the replacement water conduit typically triggers a small extension at the lining break; rewatering does not.

The movement of the steel lining between construction and 1990 was determined by surveying the position in 1990 and comparing it to the as-built position. This survey showed the following:

1. The largest measured movement of the steel lining of the inclined shaft is near its intersection with the upper tunnel and is about 600 mm, which is equivalent to an average of about 15 mm/year and an average of about 22 mm/year if extrapolated to the surface.
2. The movement is generally downslope toward the Fraser Valley at azimuths between 250° and 270° and inclinations of -20° to -30°.

A major component of the movement at Wahleach is diffuse and not associated with through going weaknesses. This is consistent with mass rock creep (Chigira, 1992) more akin to "flow" than to movement of large intact blocks relative to each other by rotation or sliding. This "creep" is facilitated by shear stresses due to the steep topography; by the large number of individual fractures and shears, most of which are much steeper than the ground surface; and by the lack of through going discontinuities weaker than the rock mass and oriented conducive to sliding.

The evidence from the upper part of the slope of more concentrated movements is consistent with sliding, but clearly no through going displacement surface has pre-existed or developed as a result of these movements. The poorer quality of the rock and the relatively more concentrated movements near the top of the slope indicate that the failure has progressed further in this area and probably began there rather than at the toe of the slope.

The series of ridge-trough couplets and the abrupt change in the inclination of the movements in the inclined shaft are consistent with relative movement of large blocks along steep, through going discontinuities. This could result from rotation or from downdropping of wedges combined with dilation of the rock mass.

The diffuse movement and the lack of any concentrated zones of movement in drill hole S1 which is part way down the slope is considered to be a very important observation implying no sliding surface has been developed in this area.

At Wahleach, all three types of movement; mass rock creep, sliding, and block rotation or downdropping, are contemporaneous. This could well be true for many other similar slopes throughout the world and could explain part of the long running controversy between "sagging" and "sliding" (e.g. Noverraz, 1996). The conflicting observations could be due to deformation styles differing from place to place in a single rock mass at the same time.

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