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TECHNICAL TOURS GUIDEBOOK, MAY 7, 1992

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COVER PHOTO:

Atwell Peak (2650 m) in the core complex of the Mt. Garibaldi volcanic edifice. Stratified pyroclastic breccia in the foreground was formed in a series of explosive eruptions during the climax of the late-Pleistocene Fraser Glaciation.

These uncemented granular deposits are the source of 300 to 500 million cubic metres of debris in a series of terraces and fans along the lower Cheekeye River. A need for expansion of the community of Brackendale prompted comprehensive study of natural hazards in the Cheekeye Basin (Technical Tour 1). Photo by J. F. Psutka, 1991.

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TABLE OF CONTENTS

INTRODUCTION

British Columbia's variety of terrain, climate and ground conditions generate the most challenging engineering and geoscience work imaginable. Over much of the last 100 years, civil engineers have focused their attention on hazards requiring flood control works and protection of dam, highway and railroad structures.

Municipal law requiring residential avoidance or protection from hazards other than flooding has only existed since the mid-1970s. This law is framed in British Columbia's Municipal Act which provides terms of reference for increasing development interest in formerly marginal or inaccessible sites.

Development interest has turned from the greater Vancouver area north along steep Howe Sound slopes and toward hazard-prone elements of the Garibaldi Volcanic Complex (Tour 1). Interest has also turned east toward high-relief slopes at the margins of protected agricultural land in the Fraser Valley (Tour 2). There is continuing development infill on flood-protected and seismically sensitive areas of the Fraser Delta (Tour 3).

Geotechnical practice required by the Municipal Act is evolving with specialist skill in hazard recognition, analysis and careful judgement. There is trend toward risk assessment procedures whereby the nature and estimated probability of hazards are first defined then followed by reasoned evaluations of risk. Geotechnical and other professionals are becoming more acutely aware of the variety of natural circumstance and human endeavor which leads to significant risk.

Bedrock landslides up to 200 million cubic metres in volume have been recently discovered; heavy seasonal rains have triggered a number of important soil landslides in the past few years. Property owners and purchasers, developers, building inspectors and planners are increasingly, and sometimes abruptly, engaged by actual or probable hazard occurrences.

These perspectives indicate the GeoHazards '92 technical tours are remarkably timely. The tours draw attention to diverse conditions which are expected to involve non-geotechnical professionals in the context of conference sessions which precede and follow. We thank the trip leaders for their planning, guidebook preparation and field presentations!

R. F. Gerath, P.Geo., Thurber Engineering Ltd.
Technical Tour Coordinator

Technical Tour No. 1

Debris Torrents & Rockslides, Howe Sound to Whistler Corridor

Oldrich Hungr, P.Eng.
Nigel A. Skermer, P.Eng.

Thursday, May 7, 1992

This tour guide is humbly dedicated to the memory of John Price,
a brilliant structural engineer and the chief designer
of the Howe Sound debris control structures.

INTRODUCTION

This trip is along a route fraught with geotechnical hazards. Since Highway 99 was opened to Squamish in 1958 well over 100 rockfall, debris torrent and debris flood events have struck the lower section. It may seem benign as we make this trip today, but lethal events have taken place here in the past.

The trip starts at Horseshoe Bay, which marks the western terminus of the Trans Canada Highway, and it finishes just north of Whistler by the spectacular Green River rock avalanche. The route is shown on the sketch map on Figure 1.

A very brief history of development on the route will help to set the stage for the trip.

Prior to the first European explorers, miners, and loggers struggling through the southern Coast Mountains, valleys and coastal fans and deltas were inhabited by the Indian people. During and after the 19-th century gold rushes, trails were blazed across the mountains to provide access to the interior region.

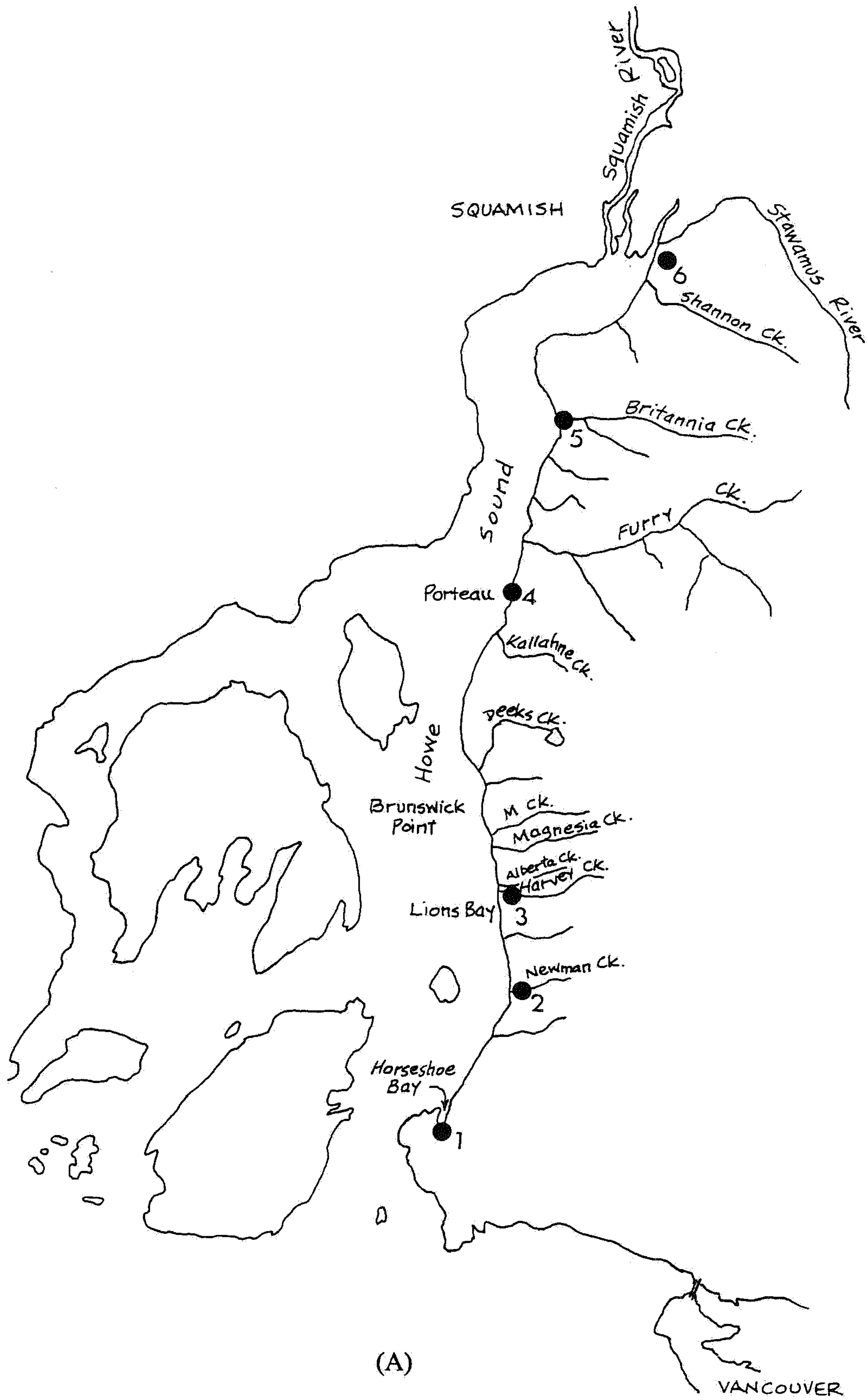
About a hundred years ago a Doctor Forbes discovered the Britannia copper ore deposits while out deer hunting in the mountains not far north of Vancouver. At that time, however, it was a remote area and nobody was too interested in his discovery. Ten years later trapper Oliver Furry, (we will cross the creek bearing his name) grubstaked by Vancouver storekeeper Mr. Clark, staked seven claims. This was the nucleus around which the Britannia mine was built. In 1905 Anaconda Copper began to exploit this rich polymetalic copper deposit. The hazard-plagued community of Britannia Beach was established at that time. The community was serviced by boat until the mid-1950s, and thereafter by road. The mine closed in 1974 and is now a mining museum.

Since 1914 the rail has connected the harbour town of Squamish with Lillooet on the Fraser River, and until the 1950s Squamish used to be the point of shipping for resources from the Interior.

In 1920, to attract tourists and to protect the scenic volcanic peaks from the gradually expanding logging and mining exploration, Garibaldi Provincial Park was established along the eastern uplands of the Cheakamus River valley.

In 1956 the road and rail beds were carved from the steep bedrock cliffs rising on the east side of Howe Sound, finally connecting Squamish with Vancouver. Cheakamus hydroelectric dam was constructed in Cheakamus Valley soon thereafter. Karl Terzaghi took part in its design. Transmission lines were installed in the corridor to transfer power from other generating plants farther north in the Interior to the urban centres of southwestern British Columbia.

In 1965 Highway 99 was extended from Squamish to Pemberton opening this part of the route to tourists, principally skiers. In 1966 ski lift facilities were built at Whistler Mountain serving the expanding recreational needs of Vancouver, and in the late 1970's the Resort



(A)

Figure 1 Sketch map of the route of the trip: (a) Horseshoe Bay to Squamish; (b) Squamish to Whistler.

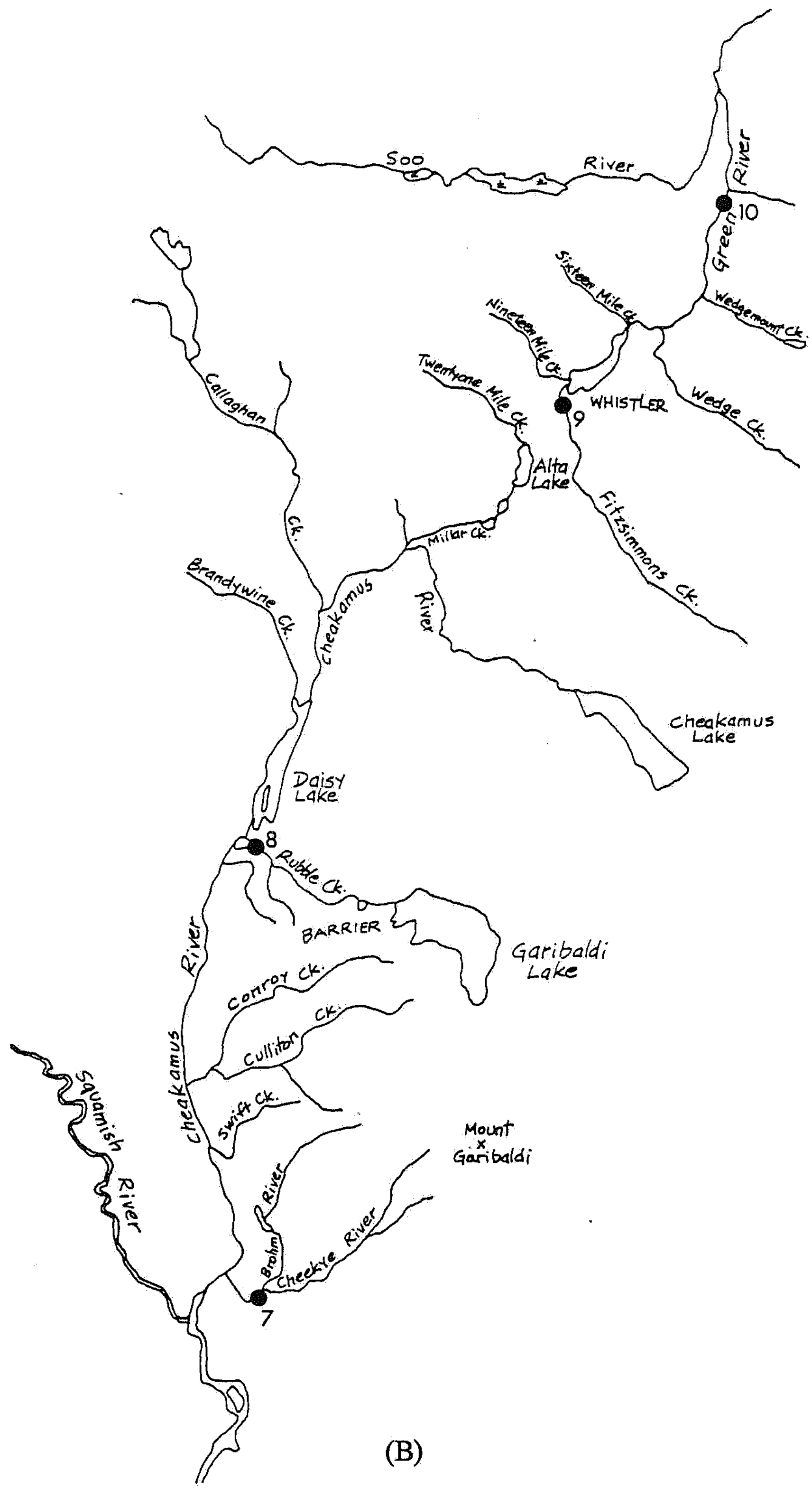


Figure 1 Sketch map of the route of the trip: (a) Horseshoe Bay to Squamish; (b) Squamish to Whistler (continued).

Municipality of Whistler was established on Fitzsimmons Creek fan, and Blackcomb Mountain was developed.

In the 1970s both logging and recreational activities advanced into hitherto inaccessible mountain valleys. Real estate development, continued logging, a need for an expanded highway corridor and intensified tourism presently pose considerable challenges to resource managers of the Vancouver Pemberton corridor.

Upgrading of the existing Howe Sound route - the Sea to Sky Highway - will continue into the mid 1990s.

PHYSIOGRAPHY AND GEOLOGY ALONG THE ROUTE

Howe Sound is a north-south trending fjord about 45 km long. During the Pleistocene, ice in two main valleys was channelled south through Howe Sound. Geophysical investigations indicate the maximum depth of water to be at least 300 m and depth to bedrock to be a further 500 m.

The southern Coast Mountains are underlain mainly by the rocks of the Coast Plutonic Complex, quartz diorite, granodiorite, and diorite. K-Ar dates of plutonic rocks range from Jurassic to Eocene, broadly decreasing from southwest to northeast.

The plutonic rocks intrude a variety of sedimentary and volcanic successions ranging in age from Late Palaeozoic to Tertiary. Remnants of the sedimentary-volcanic assemblages are preserved as metamorphic septa and roof pendants between the intrusive complexes and juxtaposed along major northwest-trending fault zones (see Figure 2). Metamorphic foliation in sedimentary, volcanic, and older intrusive rocks shows predominant northwesterly trends paralleling the overall topographic grain of the Coast Mountains.

During the last 2 Ma several volcanic complexes along what is known as the Garibaldi Volcanic Belt were superimposed onto the rising pedestal of the southern Coast Mountains, in particular Mount Garibaldi. The eruptive centres within each complex trend slightly west of north and appear to be controlled by fracture zones separating valleys from mountain ridges of possibly pre-Pleistocene age. The Garibaldi Volcanic Belt is part of a Pliocene-Recent volcanic province that extends from British Columbia southwards into the northwestern United States. The most recent eruptions in the southern Coast Mountains occurred about 2350 and 200 years ago in the Mt. Meager area.

Neotectonic history and seismicity of the southern Coast Mountains are broadly linked to an offshore ridge-transform system with predominant north-directed right-lateral displacement of the Pacific Plate. Most historic seismicity has been offshore. The largest onshore earthquake in recent history had its epicentre on central Vancouver Island at Campbell River and occurred on June 23, 1946. Shaking related to this seismic event caused numerous slope failures and extensive ground subsidence. Prehistoric fault scarps are in evidence in the area, but so far none have been proven to be of postglacial origin.

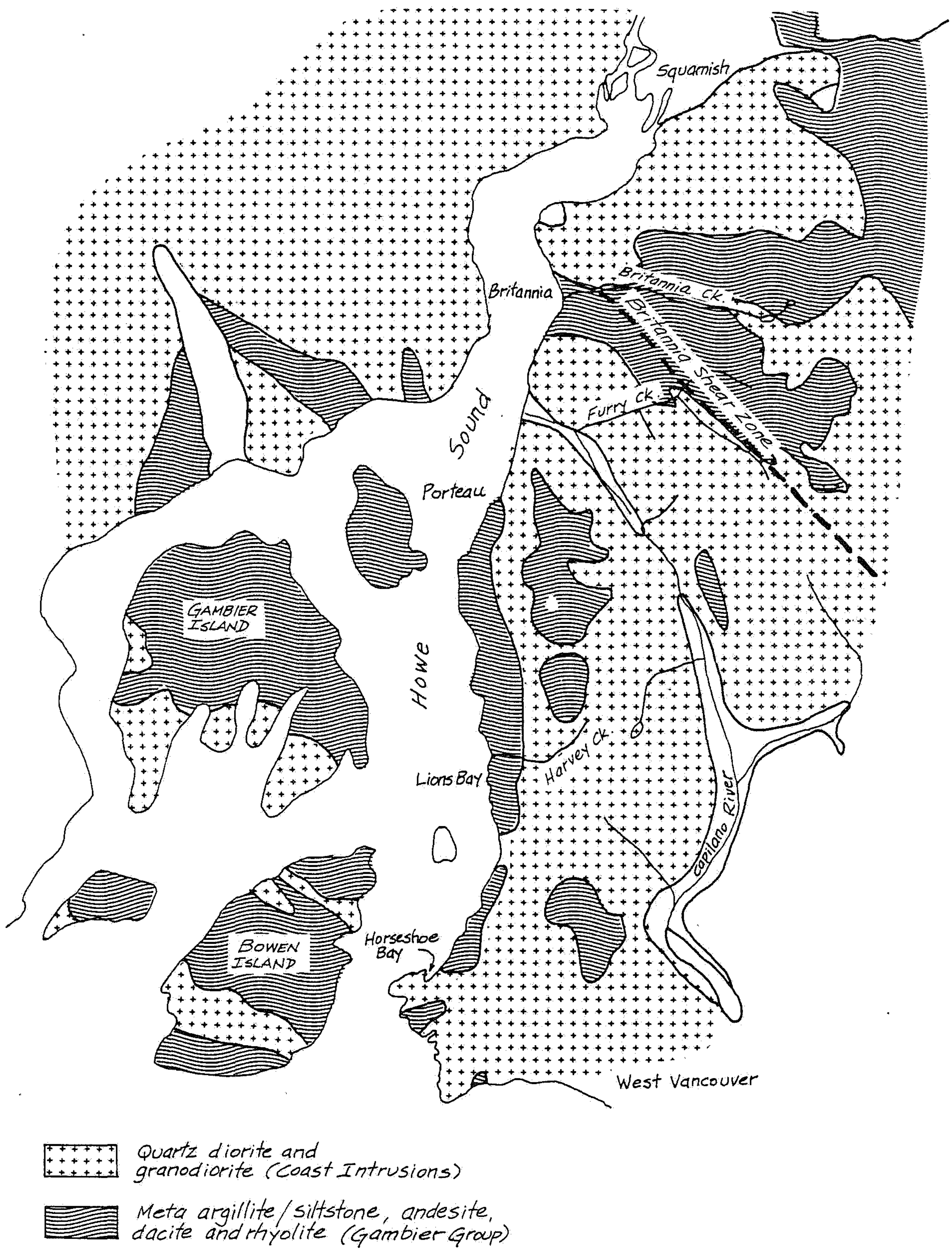


Figure 2 Simplified bedrock geology of the area.

During the Pleistocene the continental shelf of southwestern British Columbia was covered by glaciers. At its full development the vast Cordilleran ice sheet reached an elevation of 2000 metres in the southern Coast Mountains. Much of the unconsolidated deposits filling the valleys and lowlands of the region were laid down during the last major glaciation and during the immediately preceding nonglacial period. In the latest stages of glaciation, a large valley glacier from the Squamish and Cheakamus Valleys projected a floating ice front into the waters of Howe Sound. Deglaciation was accompanied by resurgent volcanic activity, particularly in the Garibaldi area. High volcanic cliffs that had formed at the contact with the glacier ice became unstable and retreated by sporadic large scale mass movements, eg. the Barrier.

Following deglaciation, flanking creek channels have discharged their bedload from steep catchment basins onto adjacent valley floors. Sporadic debris flows channelled down the creeks have been a significant mechanism for debris transport from the heights of the southern Coast Mountains to low-lying terrain.

The climate is significant in the context of geotechnical hazards. Moist air rising along the western slope of the Coast Mountains yields periods of heavy precipitation. Most precipitation falls in autumn and winter, as summarized in the precipitation normals for several typical locations adjacent to Howe Sound, shown as Figure 3. Mean annual precipitation ranges from approximately 1500 mm in Vancouver to 3500 mm in the high mountains. Superimposed on the strong precipitation gradient is the temperature-sensitive control on the position of the winter snow line along slopes facing the sea. Winter snowpacks of 300 to 400 cm may accumulate above 1000 m, while no snow at all may remain on the lowlands. Sudden rises of the freezing level by as much as 2,000 m accompanied by heavy precipitation enhance torrential runoff. On such occasions, failure of saturated soil veneers and embankment erosion along channels may combine into potentially destructive debris torrents.

Slide-triggering rainstorms often occur when a pronounced athmospheric depression is lodged in the Gulf of Alaska causing strong southwesterly gusts, rising air temperatures, and local storm cells. Failure of gully walls and upland rims saturated with overland runoff tends to occur during such times. Rainfall intensities are characteristically uneven, so that major runoff events in small drainages often occur during times when only moderate rainfalls are recorded in the nearest climatic stations. The 1981-1984 period was particularly bad for the Howe Sound Creeks and was also characterized by unusually high annual precipitation.

DEBRIS TORRENT HAZARDS ALONG HOWE SOUND

During deglaciation of the Howe Sound area, a period of rapid isostatic rebound occurred, and stream delta surfaces rose far above present sea level. The elevated relict fans were deeply entrenched by the mountain creeks which continued to build new fans and deltas into the sea. The present-day creeks derive much of their sediment from banks along these relict fluvio-deltaic deposits and from pockets of Pleistocene ice-contact debris and Holocene rock fall talus along the crest of steep catchment areas.

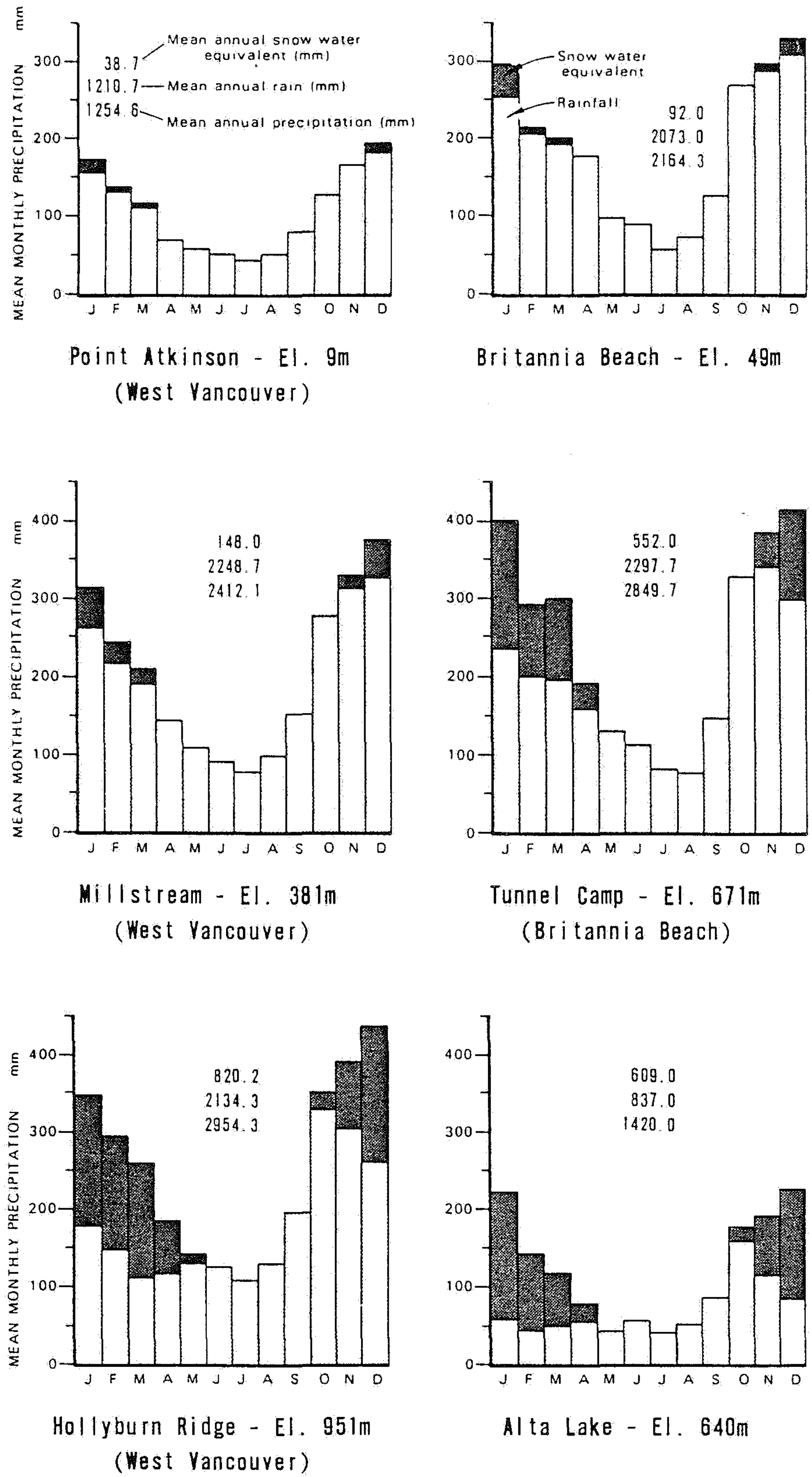


Figure 3 Monthly precipitation normals for the Howe Sound Area (shaded areas represent snow).

"Debris torrent" is a term applied to a special type of debris flow characteristic of the coastal areas of the Pacific Northwest in Canada and U.S.A. These debris flows mobilize coarse bouldery debris derived from resistant unweathered plutonic or metamorphic rocks. Almost completely devoid of fines in the silt and clay sizes, the debris moves in a liquefied condition only thanks to steep slopes and abundant water. Clast support is derived largely from turbulence and the debris tends to deposit out when the surge flows reduce their speed on moderate slopes (less than 10 to 15 degrees, see Hungr et al., 1984). Debris torrents often contain high percentages of timber fragments, often as much as 40 %. Their total volumes in the Howe Sound area have been of the order of 20,000 m^3.

Debris torrent surges tend to be short in duration, but their peaks can attain discharges more than an order of magnitude greater than those of a largest flood. Transitional events, containing heavy sediment loads but lacking the high discharge and peak flow depth are termed "debris floods". While able to transport large amounts of moderately coarse debris and travel on flat angles, debris floods tend to be much less destructive than debris torrents, except where they occur on large streams such as the Britannia Creek mentioned below.

We start our trip by viewing the general aspect of the east site of Howe Sound from the vantage point of Horseshoe Bay. The mountain front has a relief of up to 1400 m. The creek incisions can be seen on the mountain flanks together with a number of the high-level, unstable rock debris sources that cause debris torrents on the creeks. In particular the major source area for the steep channel of the Charles Creek can be seen in the near distance. The 1983 debris torrent on Charles Creek had a peak discharge of 300 m^3/sec, which was recorded by a video camera. The 90 ton concrete deck of the B.C.R. bridge was hurled to the bottom of Howe Sound by the event. Nowadays to protect the highway bridge and the dozen or so expensive homes located on the fan below, a debris catch basin is located upstream of the highway.

Newman Creek the next major creek also has a history of repeated debris torrent activity. Extensive mitigative works have been constructed here too, although of a different concept. There is little storage room just upstream of the highway, and in fact the creek descends over a small waterfall. The approach to managing debris flows here, therefore, is to keep it moving by directing it to the ocean in a smooth well-aligned channel. The channel is lined with steel-fibre reinforced shotcrete, and bridges for both the road and rail are provided with ample clearance to allow debris to pass without blocking. By locating the channel on the norther margin of the fan, bridge crossings for local traffic are eliminated thus minimizing bridges to just the two for the railway and highway. Since the debris torrent hazard is now managed with minimal risk of spillage it has been possible to redevelop the Newman Creek fan. The 18-home subdivision is currently under construction. We will stop here to view the channelization works.

M Creek was the scene of a horrible disaster in 1981 when the wooden trestle highway bridge was washed out. Unsuspecting motorists plunged into the void and 9 people died. The disaster is described in Skermer and Russell (1988). A photograph of the bridge washout on the day of the disaster is shown on Figure 4. The bridge collapsed like matchsticks under the impact of 1 m sized boulders travelling at more than 5 m/sec. An unsuccessful lawsuit was launched by the family residing on the fan whose house was swept into the Sound. The Ministry of Highways were charged together with B.C. Rail and MacMillan Bloedel, who had the timber

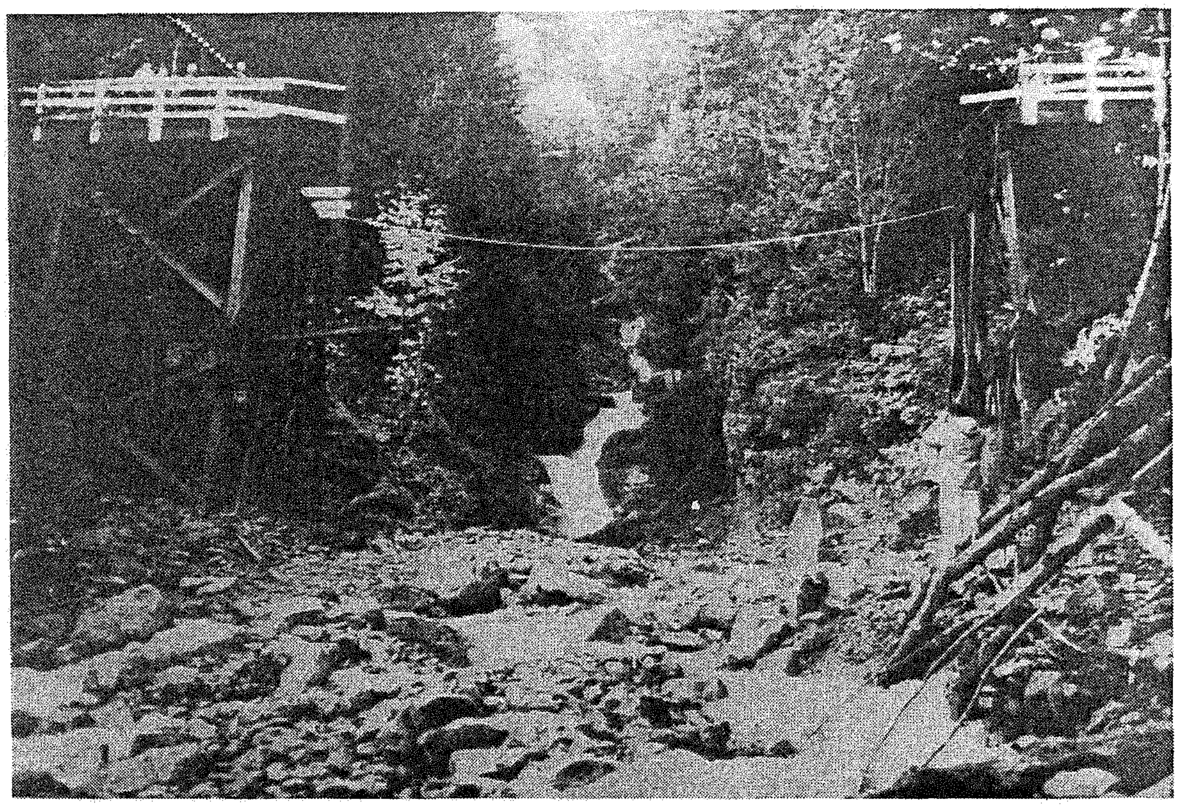


Figure 4 Highway bridge over M-Creek, shortly after its destruction by the October, 1981 debris flow.

licences uphill. The bulk of the debris in this drainage is derived from a naturally occurring rockfall source in an area that has never been harvested.

THE VILLAGE OF LIONS BAY

The village of Lions Bay (pop. approx. 1,500) is built on a coalescent fan of Harvey and Alberta Creeks. Earthwork carried out in Lions Bay revealed that the fan is, in fact, a large delta of glacio-fluvial origin, the largest proportion of which contains dense cross-bedded deposits of sand and gravel many tens of metres thick. Only a veneer of 10 to 20 metres on the surface is formed of recent coarse debris flow and flood deposits.

Harvey Creek has a drainage area of 7 km^2, extending from the sea level to the summit of the Lions at 1646m. The main branch rises from the apex of the alluvial fan at 13 to 14 degrees, to drain a large couloir with a fairly flat floor at Elevation 900m. Four tributary branches drain steep headwall slopes, some of which were clearcut logged in the sixties. Basin geology is mainly quartz diorite.

The creek has no known history of debris flows, although heavy flooding occured in 1969, 1972, 1973 and 1981. Past debris flow activity is indicated by coarse bouldery deposits on the surface of the fan, by the steepness of both the fan and the channel upstream and by the availability of debris sources on the slopes and in the channels. Based on such evidence, the creek's potential for producing a major debris flow event was estimated in a 1983 study as "moderately high" (Thurber Consultants, 1983), meaning that it should be assumed capable of occuring during the lifetime of a house. A major debris flow would endanger approximately 30 residences on the fan and would most probably entail loss of life. In addition, a debris flow could interrupt Highway 99.

In order to prevent the above hazards, a large retention structure was built at the head of the Harvey Creek fan in 1985 (Figure 5). It has a retention capacity of 70,000 m^3, which exceeds the estimated volume of available debris. The barrier was built as a zoned earthfill dyke, capable of retaining water to full height, should the drainage openings be completely plugged. A reinforced concrete decant structure on the upstream face is intended to retain debris, while draining water through two precast concrete outlet conduits. A concrete spillway is constructed over the downstream face of the barrier, to protect the earthfill from erosion in the event of overtopping.

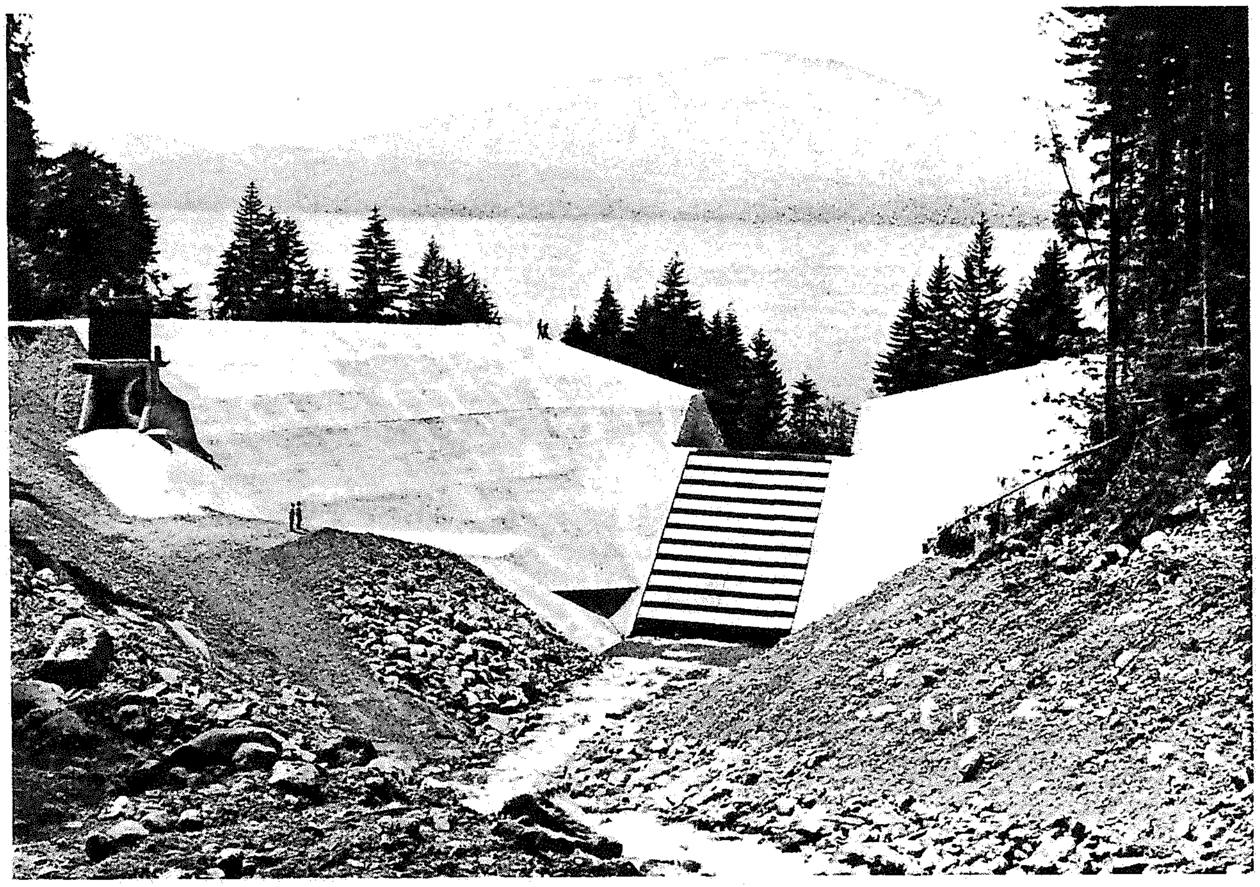
The barrier was dimensioned using principles of debris flow dynamics derived from observations of previous events on other creeks (Hungr et al., 1984 and 1987). For example, the height of the barrier shoulders is such that overtopping should only occur at the spillway. The capacity of the retention space was calculated by analysing runout of debris on the surface of preceding deposits.

Downstream of the barrier, a smoothly curving boulder lined channel was built to control flood flows, as well as debris flow discharges which could result from overtopping of the barrier.

Given the low estimated probability of the design debris flow event on Harvey Creek, the barrier may not be tested to full capacity within our lifetime. If it is, however, it will serve to prevent a major natural disaster.

Alberta Creek is a small stream draining 1.2 km^2 of steep mountain slopes, underlain by sheared and fractured metavolcanics of the Gambier Formation. The stream channel has a steady slope of 16 to 24 degrees above the fan and is deeply incised, with steep unstable banks. Airphotos show that the lower part of the creek was swept by a debris flow in the early thirties. Neither Lions Bay, nor the road or railway were in existence at that time.

By 1980, an established community grew on the alluvial fan on both sides of Alberta Creek. Fifteen houses were built within 30 metres of the creek banks. One house was only 3m from the creek and 0.5m above it. In addition to a wooden trestle bridge for Highway 99 and a concrete railway bridge, five culverted crossings were built for subdivision access. During the early morning hours of February 11, 1983, a debris flow with an estimated magnitude of 20,000 m^3 came down the creek. The highway bridge, all the culverts and fills and three houses



Upstream Face

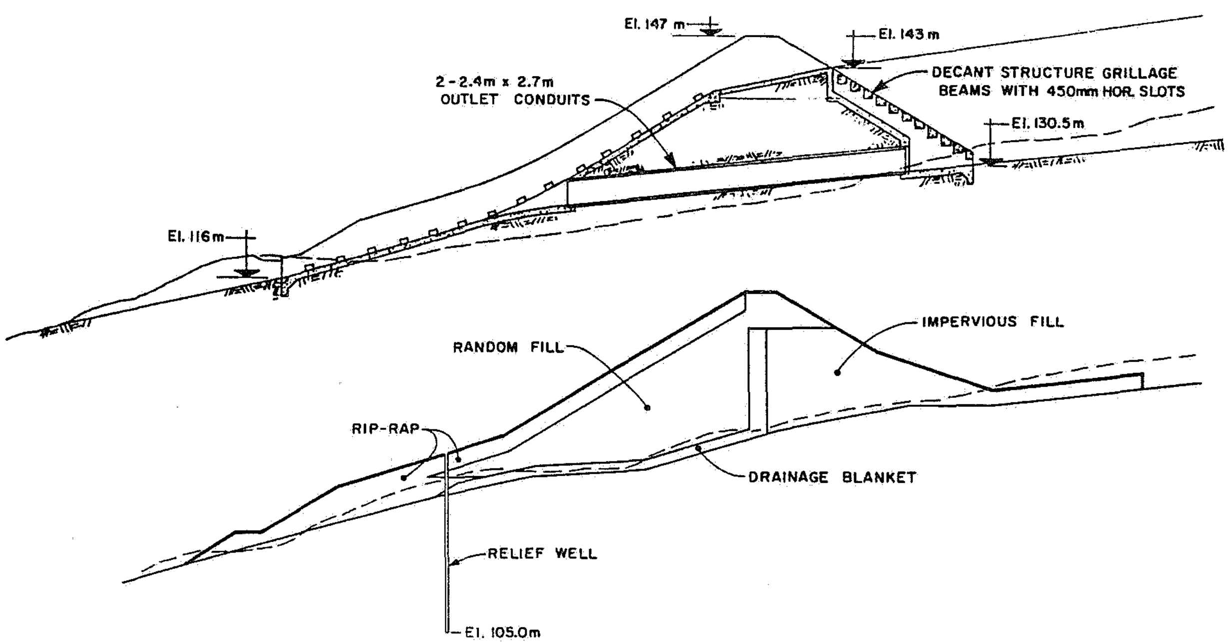


Figure 5 Harvey Creek debris retention basin in Lion's Bay. (From a sketch drawing by Ker Priestman Associates Ltd.)

were completely destroyed. The railway bridge and another house were damaged. Two teenage residents lost their lives.

The event probably started by a shallow slide of steep bedrock-derived colluvium undercut by the creek, on the right bank some 200m upstream from a logging road crossing at Elevation 700m (an unlogged area). The initial failure probably contained less than 100 cubic metres of soil and rock fragments. Most of the debris load of the event was derived by mobilization of channel infilling and undercutting of unstable banks over the 1.3 km length of channel between the initial slide and the fan. About 10-20% of the debris was organic (timber remains). The flow occured in six surges spaced over a half-hour interval. The largest surge may have been as much as 4m deep at its peak with a maximum discharge of 250 to 350 m^3/sec. It was fortunate that the flow avoided spilling out of the established channel at an unprotected bend near the fan apex. Should this have happened, the damage may have been even more serious.

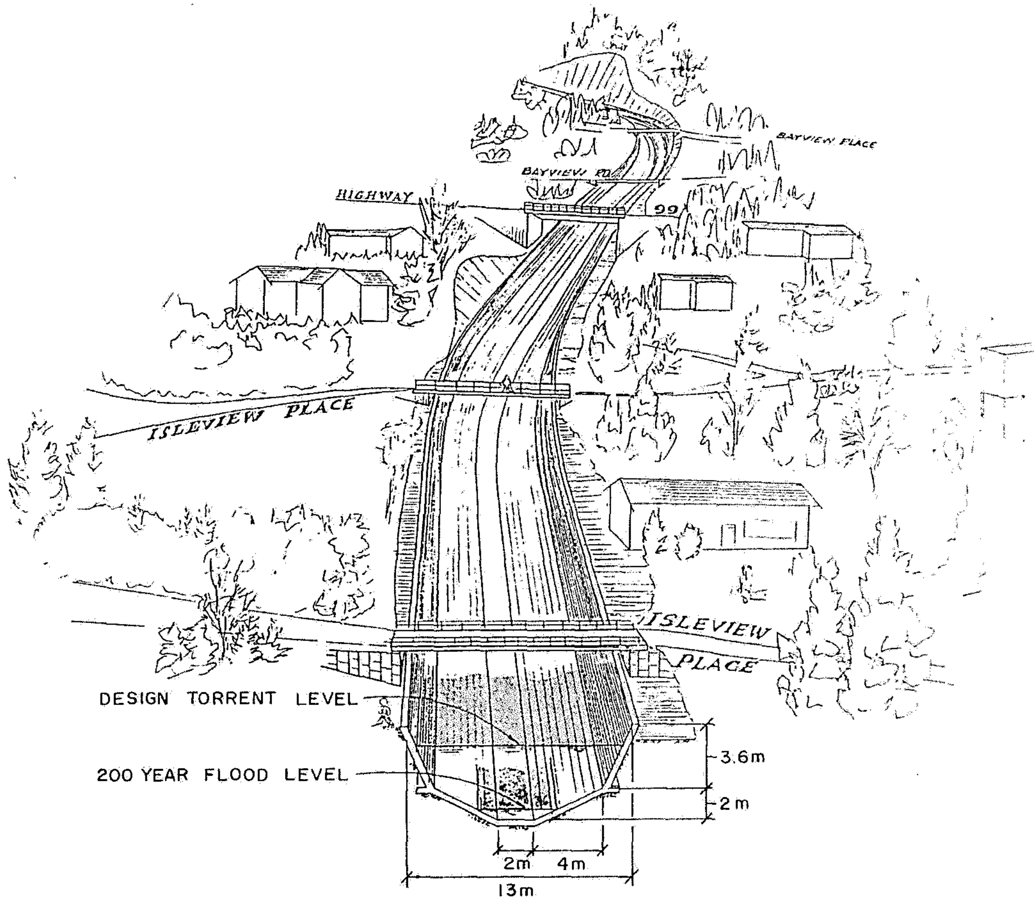
Alberta Creek offers no suitable sites for a retention structure, due to steep slopes and dense population. Consequently, a 700m long "shooting channel" (from German "schussrinne") was built, to deliver flood or debris flow discharges to the toe of the fan in Howe Sound (Figure 6). The design of the channel was based on semi-empirical formulas for debris flow hydraulics, developed by back-analysis of observed events (Hungr et al., 1984). Two main considerations in the design were a) to prevent spillover at bends by providing a uniform, gently curving channel and b) to prevent depositional plugging by ensuring secure confinement of the flow at a range of discharge rates. The lined channel was also used to stabilize steep eroding banks which threatened a number of houses upstream of the fan.

A short distance north of Lions Bay, Highway 99 traverses extremely steep slopes in fractured metavolcanic rocks. A moderately large rockfall here in January, 1991 impacted the fill roadbed and closed the highway for two weeks, effectively cutting Squamish and the communities further to the north from the rest of the world. Figure 7 shows the slide source area and path in 1987 (note small initial rockslide) and in 1991 following the main slide. The hazardous stabilization operation was successfuly completed by the Ministry of Highways. A secondary fall which occurred during the scaling operation injured two crew members. Following the slide, B.C. Government decided to provide a port for large car ferries at the north end of Howe Sound, which is presently under construction.

ROCKFALL NEAR PORTEAU BLUFFS

Since the opening of Highway 99 there have been a number of large rockfalls that have closed the road to travel for days at a time. One slide at Porteau Bluffs occurred in November 1964 and is shown in the sketch on Figure 8 (the same rockfall appears on the cover of the well known rock mechanics textbook by Hoek and Brown, 1977). On this occasion both the highway and the rail were blocked. Another rockfall occurred in February 1969. A relatively small block impacted the side of a car, flying horizontally, and killed three people inside. A house size block landed on the highway in July 1970. The most recent rockfall fatality here occurred in the spring of 1991.

Destruction resulting from
1983 Debris Torrent



Proposed Channelization

Figure 6 Alberta Creek debris channel. (From a sketch drawing by Ker Priestman Associates Ltd. Photo D.F. VanDine)

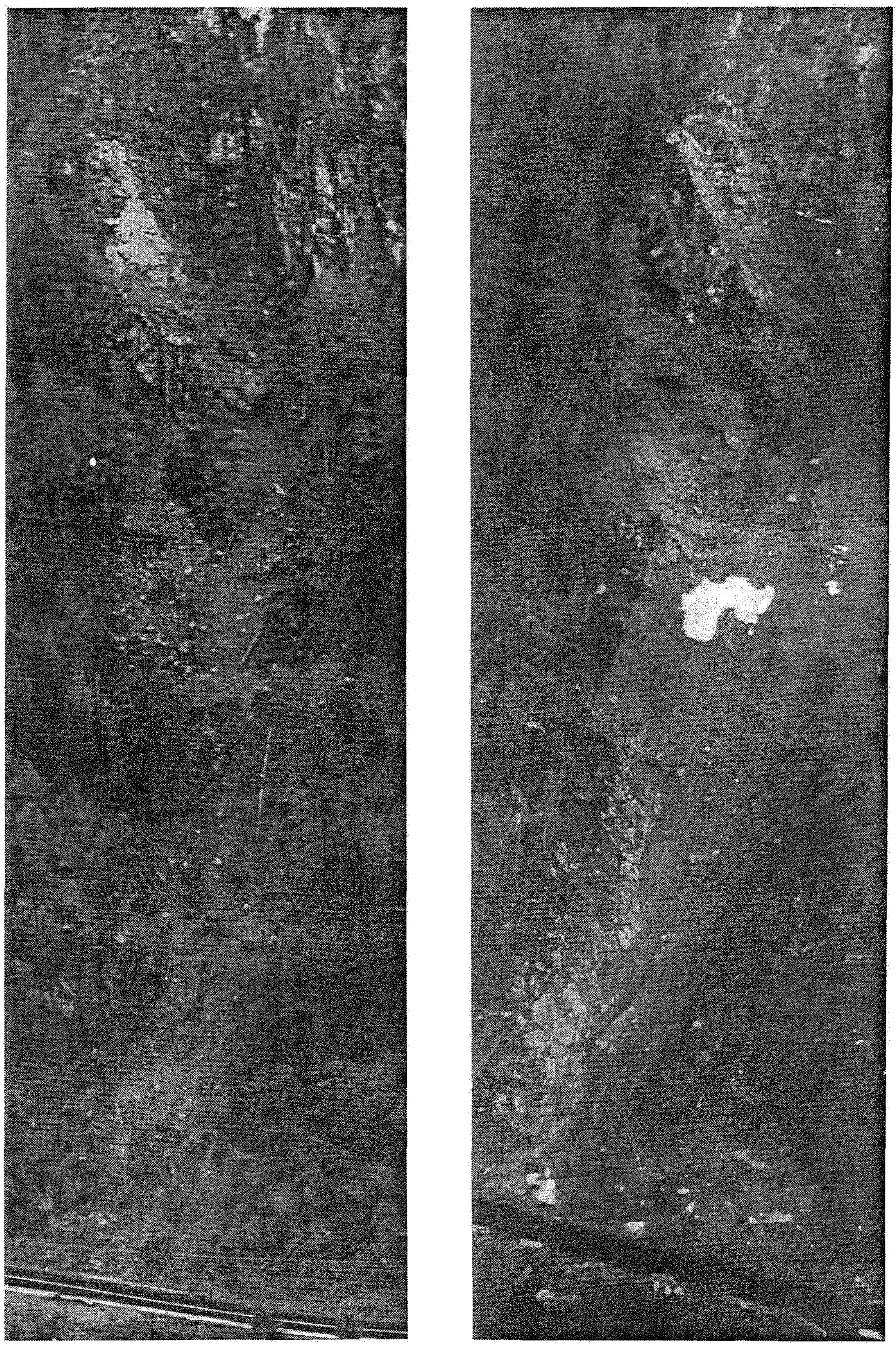


Figure 7 Rockslide near Loggers Creek, north of Lions Bay. a) in 1987, b) following the main slide in 1991. (Photos courtesy of Mr. H. Bartle, P.Eng. of B.C. Ministry of Transportation and Highways.

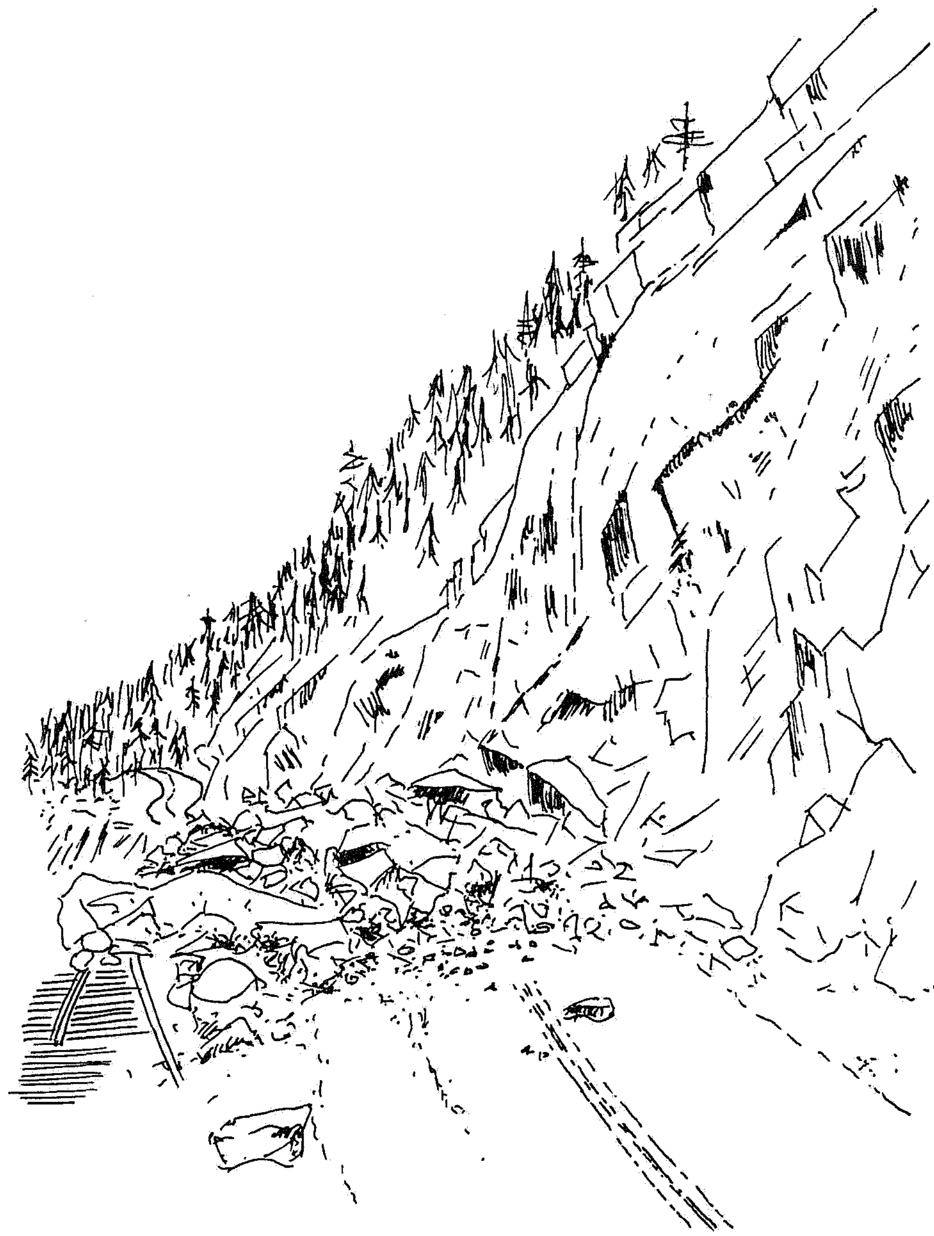


Figure 8 The 1964 rockfall at Porteau Bluffs (Drawing by N. Skermer).

A near vertical face of quartz diorite shows well-developed sheet jointing striking parallel to the highway. The rock dips westerly towards the road at approximately 50 degrees, which is steeper than the angle of friction. These joints, with an average interval of 3 to 8 feet, are cut by widely spaced, east-west trending joints which dip vertically. The two critical joint sets are roughly parallel to the surface.

The slides are usually triggered by repeated freeze-thaw cycles during winter months, sliding occurring on pre-existing planar surfaces. The wide spacing of the joints adds to the severity of the slides, because large rock blocks are involved. South of this area, where the rock is closely jointed volcanic or metasediments, the amount of rock that falls at any one time is relatively small.

A major scaling and stabilization program was completed at Porteau Bluffs by the Ministry of Transportation and Highways in 1991. The cliffs were cleared of vegetation and thoroughly scaled. Some 3,000 metres of galvanized steel resin grouted threadbar anchors were installed, some as long as 10 m. Permanent displacement prisms were attached for long term monitoring. Figures 9 and 10 show scaling operations in progress on the cliffs.

Just north of Porteau in a rock cut, a minor rockfall occurred on Saturday 16th January, 1982. A large boulder fell from the high cliffs on the east side of the highway and struck a stationary car on the northbound lane. The passenger was killed and the driver badly injured. The car was in a line of traffic stalled by heavy wet snow on the road.

The boulder was pried off the rock face about 40 m above road level. A fir tree with its roots in a crack between the intact face and the rock that fell, appears to have toppled under the weight of heavy snow that was falling that morning. The tree prised two boulders loose. Both fell onto a bench below, and one slid off crashing onto the car on the roadway. The other boulder was later pushed off the bench by highway officials. The boulders would have been about to fall for a long time, and critical conditions just happened to arise that particular morning. The point from where the boulder fell is a natural rock face covered with clearly defined glacial striae. The rockfall was the first from this particular segment of rock slope in the time since deglaciation.

The highway cut in weaker argillite rock below is about 25 m high, and the face has been treated with shotcrete. The boulder seems to have fallen from a point above the cut made for the highway.

One of our tour guides (Skermer) gave evidence at the Coroner's inquest regarding the cause of the falling rock block, while our other guide (Hungr) analyzed the trajectory of the fall.

The analysis of the rockfall, summarized in Figure 11 uses impact parameters typical of small fragments. This is justified not so much by the size of the block, as the limited velocities developed in such a short path. A small initial drop is specified, to account for the velocity gained in toppling failure. As would be expected, the path is dominated by the bouncing mode and free flight. The simulation indicates that the boulder struck the car with a velocity of approximately 28 m/s (100 km/hr).

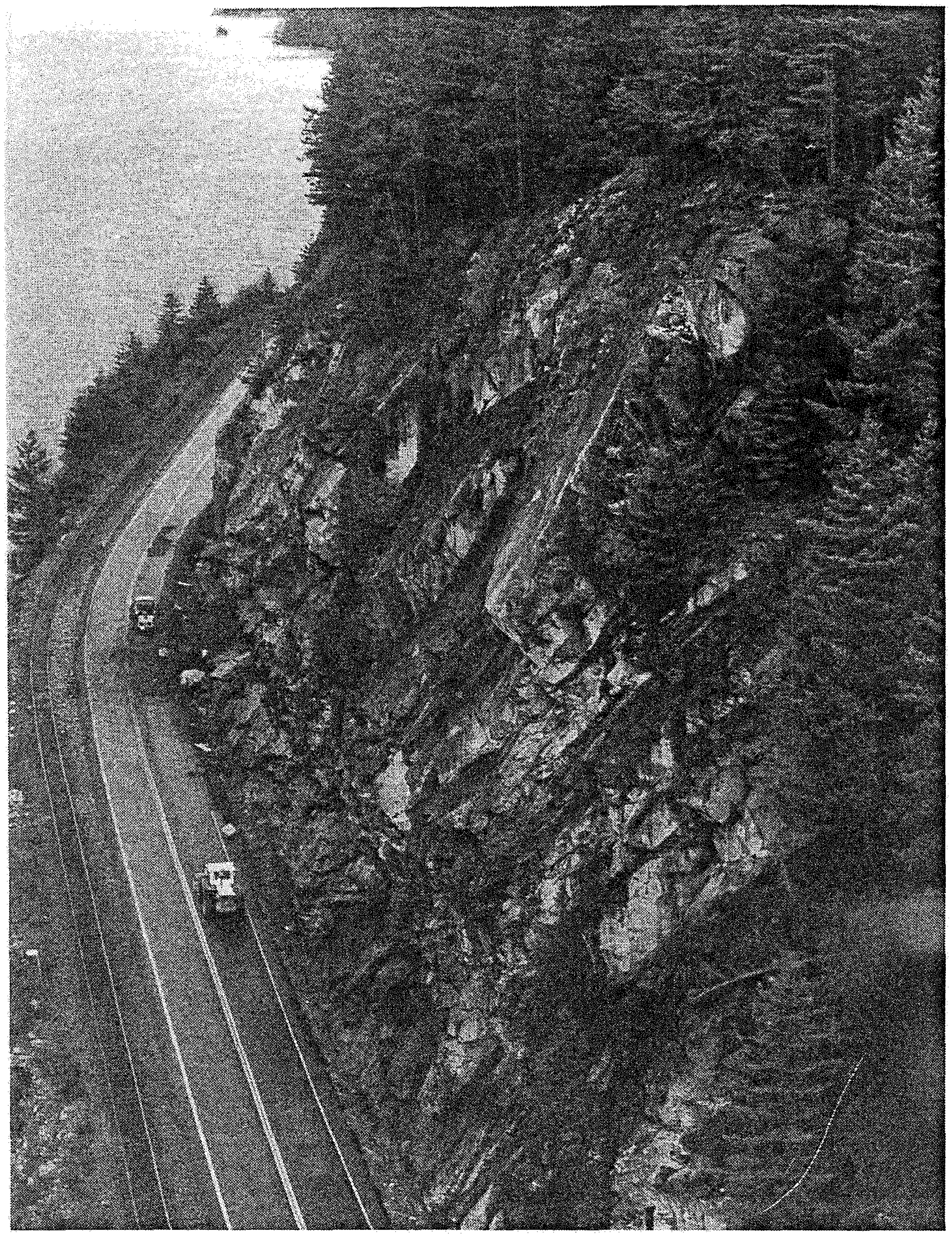


Figure 9 Porteau Bluffs during scaling operations (Photo courtesy B.C. Ministry of Transportation and Highways).

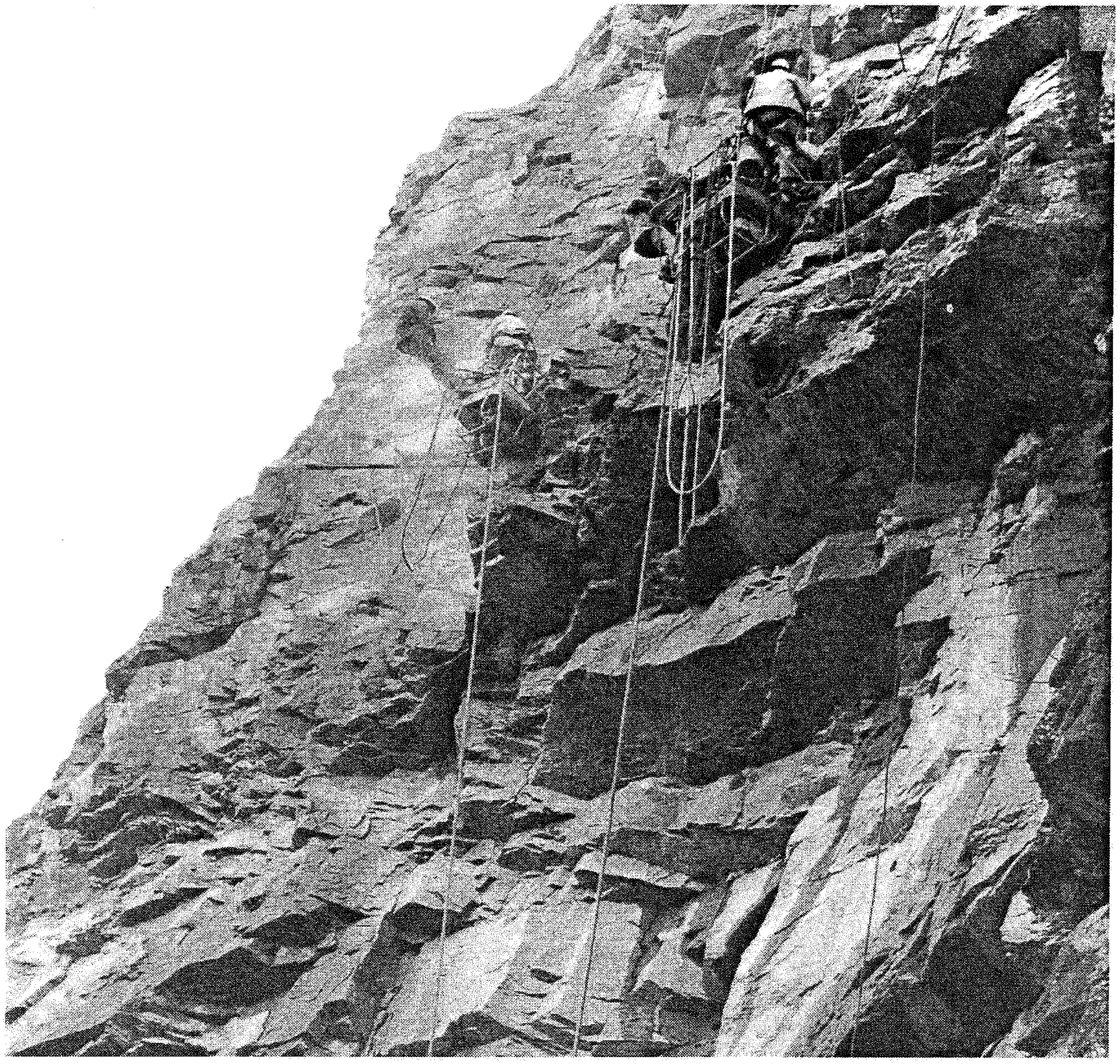


Figure 10 Rock bolting with a bencher drill on the Squamish Highway (Photo courtesy B.C. Ministry of Transportation and Highways).

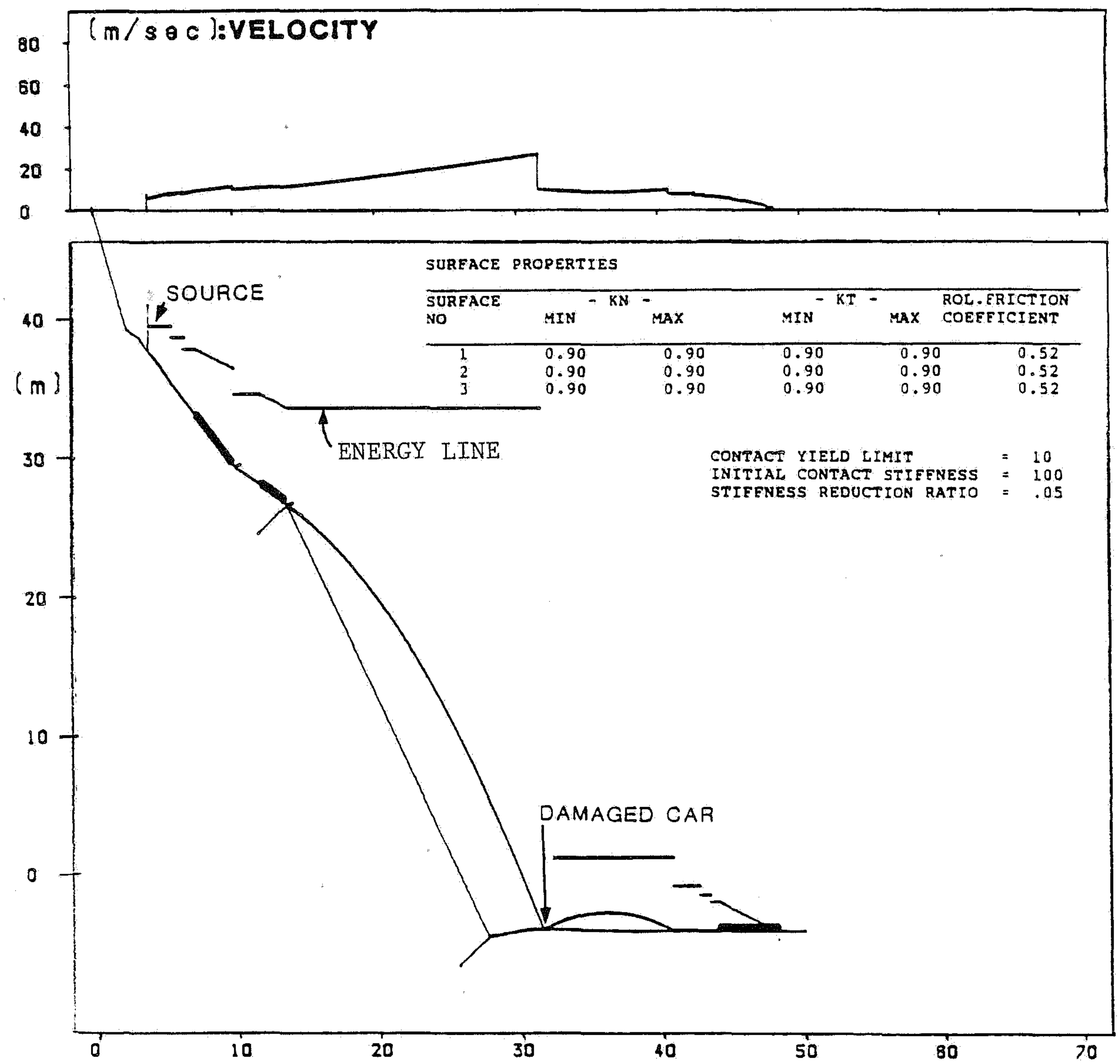


Figure 11 Dynamic analysis of the trajectory of the 1982 rockfall accident north of Porteau.

As a result of this accident the driver of the car sued the Ministry of Highways for damages. The first action was brought in 1985 in the British Columbia Supreme Court, but was dismissed by the judge on the grounds that the decision not to scale the slope above the cut was a policy decision by the Ministry for which they cannot be held liable in a court of law. They can only be held liable for operational functions if negligence is established. The policy decision not the scale the loose blocks was said to involve ordering of priorities based on allocation, province-wide, of limited resources. The decision was appealed to the Court of Appeal in 1986, but was dismissed by all three appeal judges. They said that the danger of the rock becoming dislodged was not apparent from the highway, and therefore the Ministry employees responsible for inspection were not negligent. Finally, however, on appeal to the Supreme Court of Canada a retrial was ordered on the grounds that the Ministry owed a duty of care to the plaintiff. The B.C. Supreme Court concluded that the Ministry crew failed to meet the standard of care by not conducting a climbing inspection. The Court ordered more than $1 million to the plaintiff in November, 1991.

JANE CAMP ROCKSLIDES, BRITANNIA CREEK VALLEY

With 56 fatalities, the Jane Camp rockslide of 1915 is the second most destructive landslide in Canadian history (after the Frank Slide of 1903). Unfortunately, very few details exist concerning the technical aspects of this disaster. Even the site of the rockslide source no longer exists, having been removed by open pit mining.

Jane Camp is the site of the original mine above Britannia Beach, where mining started in 1905 (Ramsay, 1967). The camp was located on a bench in the south wall of the Britannia Creek valley, at Elevation 1000m above the sea level. By 1915 there was a considerable number of mine buildings as well as miners' cottages, bunkhouses and a school located in the camp. An aerial tramline connection was provided with the beach community.

The landslide struck at midnight, March 22, 1915. The anonymous composite photograph kept in the B.C. Mining Museum at Britannia (Figure 12) does not show the source area clearly. It appears to be several hundred metres above the camp, on the south-east side of the large niche surrounding the camp bench. The initial path appears to have been not more than 50m wide and very steep. The slide entered a gully leading towards the camp from its left side, ran up the opposite wall of the gully to a height of about 30m, then descended towards the camp on a slope of less than 30 degrees, incorporating talus and rock waste in its path. On reaching the camp bench, the slide tongue widened to over 100 metres, destroying most buildings on the bench. The maximum thickness of the debris was 15m (Ramsey, 1967) and the overall volume is estimated at $100 \times 100 \times 10 = 100{,}000\ m^3$.

The detachment mechanism is unknown, but sheared rock left in the area of failure after mining has a schistosity dipping steeply into the mountain, suggesting that toppling may have been involved. Cracks open tens of centimeters were observed in the crest area of the slide and were inspected by company geologists two days prior to failure, but the danger was not recognized.

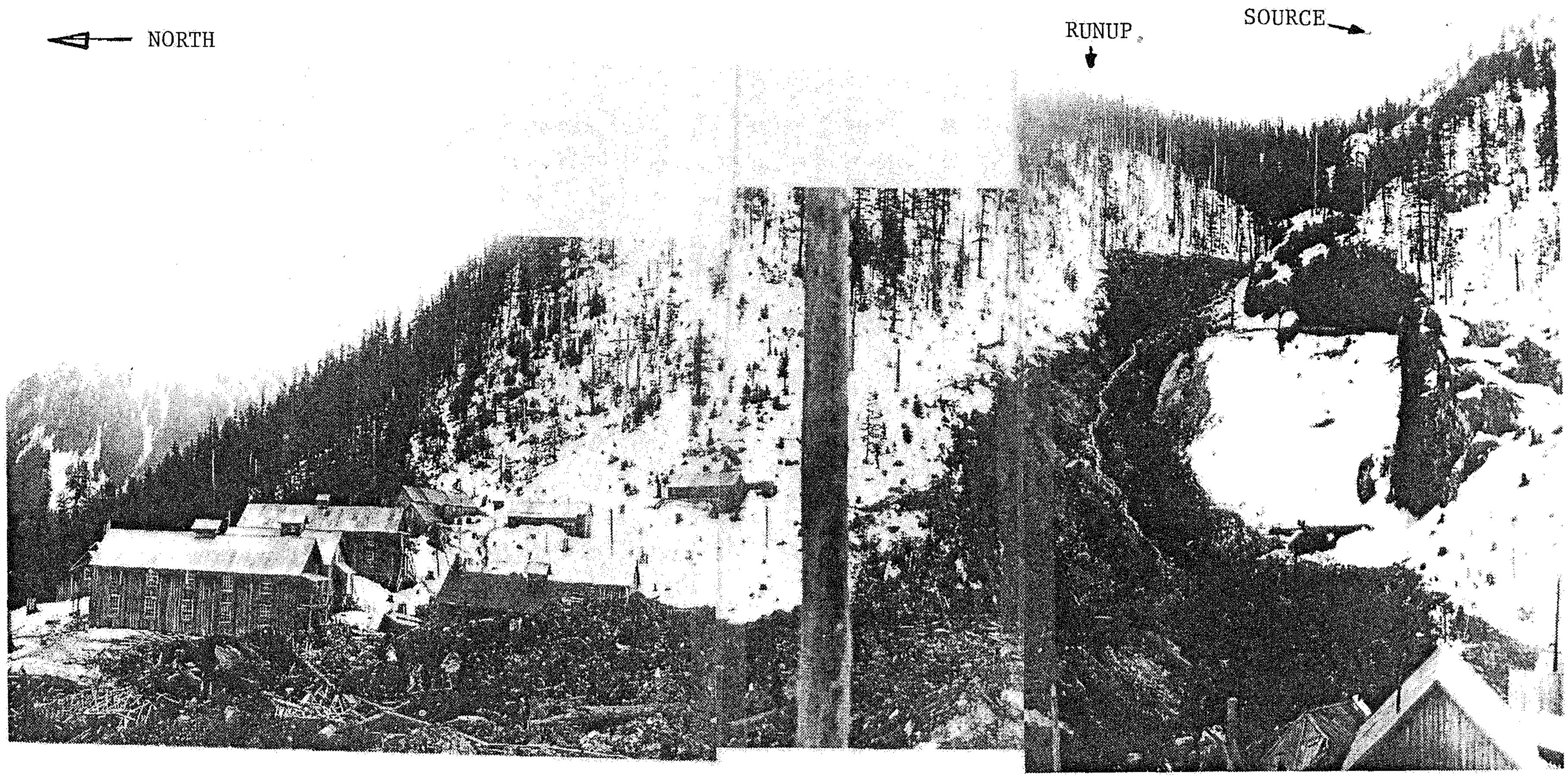


Figure 12 The 1915 rockslide at Jane Camp (photo courtesy B.C. Museum of Mining).

The mobility of the rockslide appears excessive, given its moderate magnitude and coarse dry material. It may have been mobilized by snow accumulated in the gully leading towards the camp.

The mountainside on the west side of the Jane Camp bench, opposite the source of the 1915 disaster is traversed by a series of large scarps and open cracks, suggesting displacements of over 10 metres. These displacements are probably partly due to subsidence caused by ore drawing operations underground some 50 years ago. However, given the steepness of the slopes below the disturbed area, the posibility of a major rockslide must also be considered. The disturbed area is presently being monitored by means of permanent survey prisms, observed from a fixed instrument base in Jane Camp.

THE BRITANNIA BEACH FLOOD OF 1921

The Britannia polymetallic mine produced, over its active life from 1905 to 1974, in excess of one billion pounds of copper as well as lead, zinc, gold and silver. The orebodies extended in a zone 3 km long southeast of the mill on the Britannia shear zone, see Figure 2. After the disaster of 1914, a new townsite was developed on the fan of Britannia Creek.

The Britannia Creek basin had lost most of its forest cover due to the initial mineral exploration and development work. The creek is 8.5 km long with a drainage area of 28.5 km^2 and gradients in the range of 3° to 11°. A sudden outburst debris flood occurred at 9:30 a.m. on the evening of Friday, 28 October 1921. At first it was believed to be the result of the collapse of natural dam of timber and soil debris that had slid from the banks of the creek and temporarily ponded water. Later at the inquest civil engineers testified that, rather than a natural dam, the cause lay in the collapse of a culverted fill that carried the mine's railway across the creek. The 2.5 m wooden culvert became plugged with debris, ponding 60,000 m^3 of water before it failed. When the fill collapsed the surge ("a wall of water 70 ft. wide by 3 to 5 ft. high") destroyed 60 houses in the mining community built on the fan, see Figure 13 from the Province newspaper at the time. Thirty-seven people perished. The flood occurred after days of heavy rain culminating in a cloudburst that yielded an "unprecented" 146 mm in the 24-hour period immediately prior to the flood. Conditions were made worse because "a warm Chinook wind was blowing, which melted the snow in the mountains, adding their quota of water to the already swollen stream." The coroner's inquest found the mining company criminally negligent. Further details are given in Skermer and Russell (1988).

The next incident in 1957 was a submarine landslide on the edge of Britannia Creek fan when a portion of the newly constructed embankment slid out from beneath the B.C. Rail line grade and left the rails hanging over a 50 m wide chasm.

When the beach community was destroyed in 1921, the company developed yet another townsite at Mount Sheer, below Jane Camp. The townsite no longer exists. It is disturbing to note, however, that a large debris avalanche swept over the townsite in 1990, knocking out the abandoned basement walls of several miners' cottages.

Figure 13 The 1921 flood damage on the fan of Britannia Creek (photo courtesy Vancouver Public Library).

The latest acts in the drama have been debris laden floods on Britannia Creek. The first was a controlled flood in 1990 when an old concrete dam upstream was purposely breached for safety reasons. The resulting flood swept a large volume of logs and boulders down the channel into Howe Sound. The debris remained in the confines of the creek channel and little damage occurred. An interesting video recording of this event was taken from a helicopter, and it is available for viewing through the Ministry of Environment. A completely uncontrolled event, however, occurred a year later, at the end of August, 1991, when unusually high summer rainfall struck the Lower Mainland. The flood yielded large quantities of sand, gravel, cobble and boulder debris and a large portion of the fan was overran. The highway plugged off with debris, and the elevated railway embankment then acted as a dyke, with the result that the fan was flooded with up to a metre or so of water. Much sand and silt settled out covering the buildings yards, gardens and services areas and the highway.

STAWAMUS CHIEF

The Chief is a large batholith of quartz diorite rounded by glaciers into an imposing cliff, standing 652 metres above the Squamish River estuary at the north end of Howe Sound. The 600m high massive, near vertical and partly overhanging west face of the mountain is favoured with rock climbers.

Rockfall is not frquent on Stawamus Chief, due to the massive nature of the rock. The occasional falls result from toppling or sliding of near-vertical slabs, separated from the cliffs by sheeting joints. Professor W.H. Mathews was working on the Mamquam River fan on June 23, 1946 and heard a number of rockfalls on surrounding mountain slopes triggered by the magnitude 7.2 Vancouver Island Earthquake. Fresh scars confirmed that blocks several metres in diameter detached from the north-west face of the Chief during the quake.

As a result of low frequency of rockfalls, the talus deposits surrounding the mountain are of modest size. It is interesting to note that the reach of falling fragments relative to the toe of the slope is also very limited despite the phenomenal height of the source cliff. This is illustrated in Figure 14, where the runout profile of the most mobile fragments from the highest part of the west face of the Chief are compared with similar profiles measured elsewhere in British Columbia (Hungr and Evans, 1987).

Another empirical rule proposed to predict the extreme runout of rockfall suggests that the most mobile fragments reach as far as a line drawn at 32 degrees from the cliff top (Toppe, 1987). Were this applied to Stawamus Chief, the predicted runout hazard zone would be nearly 600 m wide and would cover large parts of the town of Squamish. This method obviously cannot be applied here.

The limited horizontal reach of boulders from the Chief confirms that the initial momentum of fragments falling from a vertical source is practically all lost in the first series of impacts. Long horizontal reach is achieved only where fragments are able to build up sufficient horizontal and rotational momentum by travelling over a long inclined segment of talus slope.

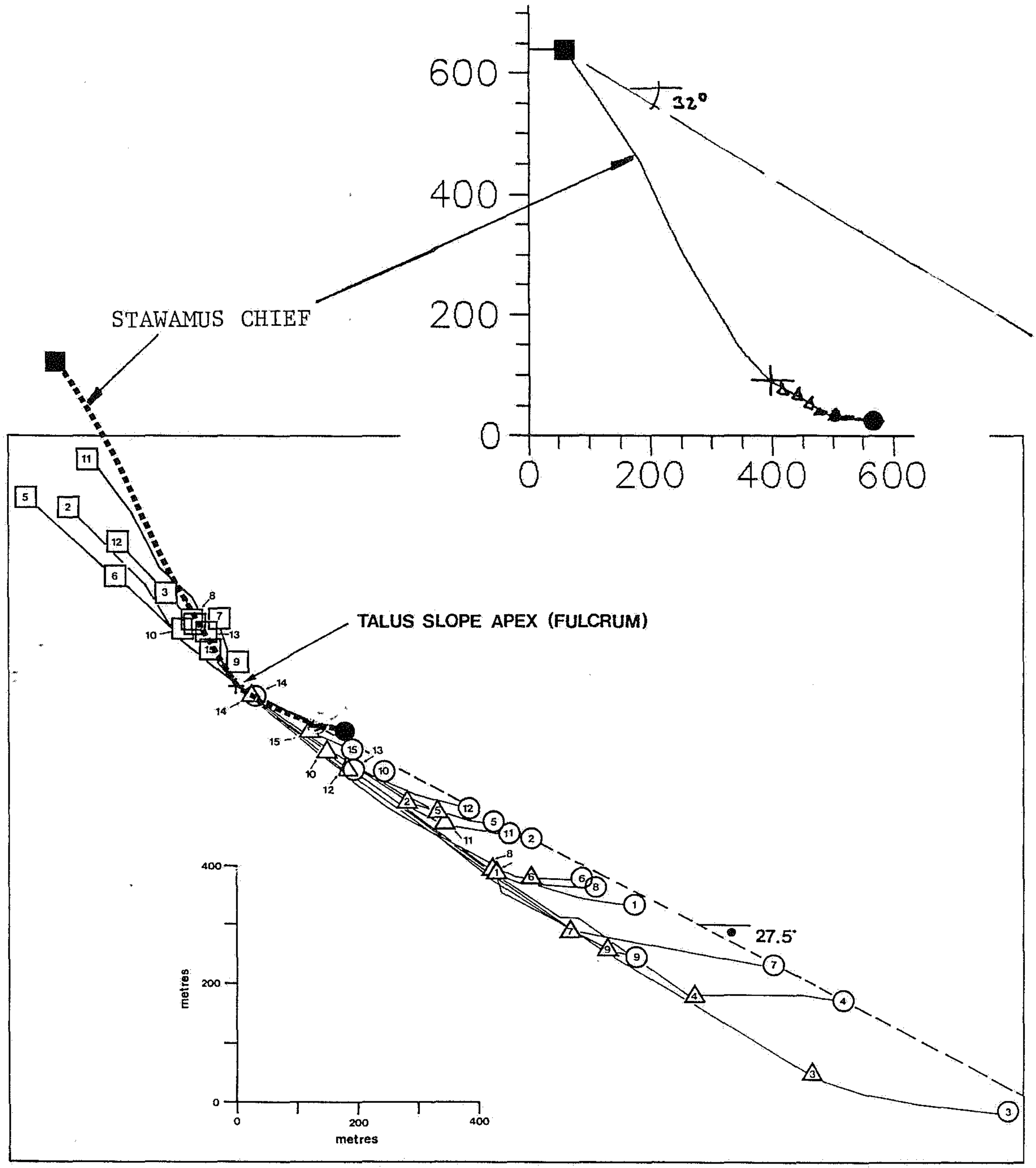


Figure 14 The limits of rockfall from the highest section of the west face of Stawamus Chief, compared with maximum runout profiles from other rockfall sites in B.C. (after Hungr and Evans, 1987 and 1988). The profiles have been plotted with the talus apex (cross) as the common point. Square: top of cliff, triangle: toe of talus deposits, circle: end of extreme boulder runout.

MT. GARIBALDI, CHEEKYE FAN

Mount Garibaldi (2678 m) is the centre of a quarternary volcanic field covering an area about 30km long and 15 wide, with nearly a dozen major vents (Figure 15). Deposits of lava, pyroclastic breccias and tuffs ranging in thickness from a few metres to over 1 km result from periods of volcanic activity spanning the last million years (Mathews, 1952, Green, 1990). Exceptionally vigorous series of eruptions coincided with the last (Fraser) glacial advance, 26,000 to 11,000 years before the present.

Post-glacial erosion deeply incised the volcanic edifice. The head of the Cheekye Valley in particular, advanced eastward to the centre of the main vent of the volcanic field, producing a unique cross-sectional view of the volcano.

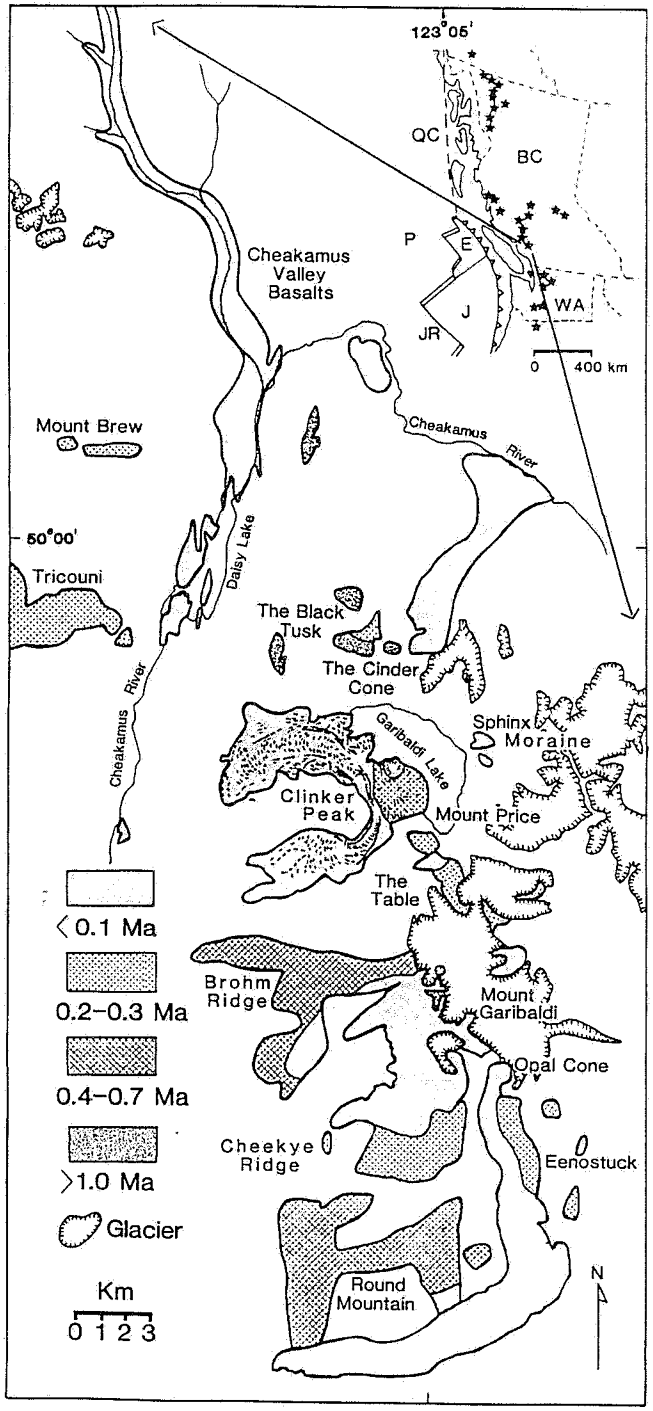
In Figure 16, the pyramid-shaped Atwell Peak on the right represents the centre of a sequence of eruptions which coincided with the middle to later stage of the Fraser Glaciation. Vertically structured lavas and pyroclastics of the central core of the vent form the steep cliffs of the peak. The eruptions were of an explosive Pelean type, expelling flows of hot fragmented material from the vent. Surrounding slopes are formed of a thick cone of faintly bedded pyroclastic breccias concentric on the peak and dipping outward at 10 to 12 degrees. These "Atwell breccias" are well exposed in the 800 m high gullied headwall sloping at 43 degrees to the west and north beneath Diamond Head and Cheekye Ridge.

The pyroclastic breccias are uncemented, dense and free-draining. They consist of a poorly sorted mixture of angular dacitic lava fragments ranging from silt to blocks several metres in diameter. Up to 60 percent by volume is sand and silt. It is this granular nature of the volcanic deposits which is responsible for the rapid headward erosion of the Cheekye Valley. Fragment fall, sheet erosion, gullying and shallow sliding are processes releasing quantities of granular debris from the headwall into the Cheekye drainage.

The top surface of the Atwell breccia cone has never been glaciated, even at altitudes as low as 1400 m asl., far below the local upper limits of glaciation (Figure 17). Brohm Ridge on the opposite side of the Cheekye Valley, in contrast, is covered by glacial drift to at least Elevation 1800 m. A few isolated glacial eratics and other signs of contact with Pleistocene ice, exist at the distal edge of the breccia cone on Cheekye Ridge. This evidence places the time of deposition of the uppermost breccias into the climax, or later part of the Fraser glacial period.

The contact between the Quarternary volcanics and the underlying Mesozoic metamorphic and plutonic rocks is exposed on the walls of the Cheekye Valley, as shown on Figure 17. The uneven topography of the contact, with the volcanics draping down the valley walls in several places, shows that some deeply incised form of the valley existed already prior to the Atwell eruptions. It follows, therefore, that a part of the cone of breccias deposited on top of glacial ice which infilled the ancestral valley at the time of the eruptions. On retreat of Fraser ice, the supraglacial deposits slumped into the Cheekye Valley (Mathews, 1952).

A persistent zone of alteration up to several tens of metres thick occurs in the basement rocks immediately below the contact with the overlying volcanics. Its origin is suspected to be



Timing and distribution of Pleistocene-Holocene volcanic rocks in the dominantly dacitic Mount Garibaldi and andesitic Garibaldi Lake volcanic fields. Inset illustrates the configuration of the Juan de Fuca ridge system (JR), Queen Charlotte fault system (QC), and Juan de Fuca (J), Explorer (E), and Pacific (P) plates along the continental margin of British Columbia (BC) and Washington (WA). Stars show Quaternary volcanic centres.

Figure 15 Map of the Garibaldi volcanic field (from Green 1991, courtesy of Geoscience Canada).

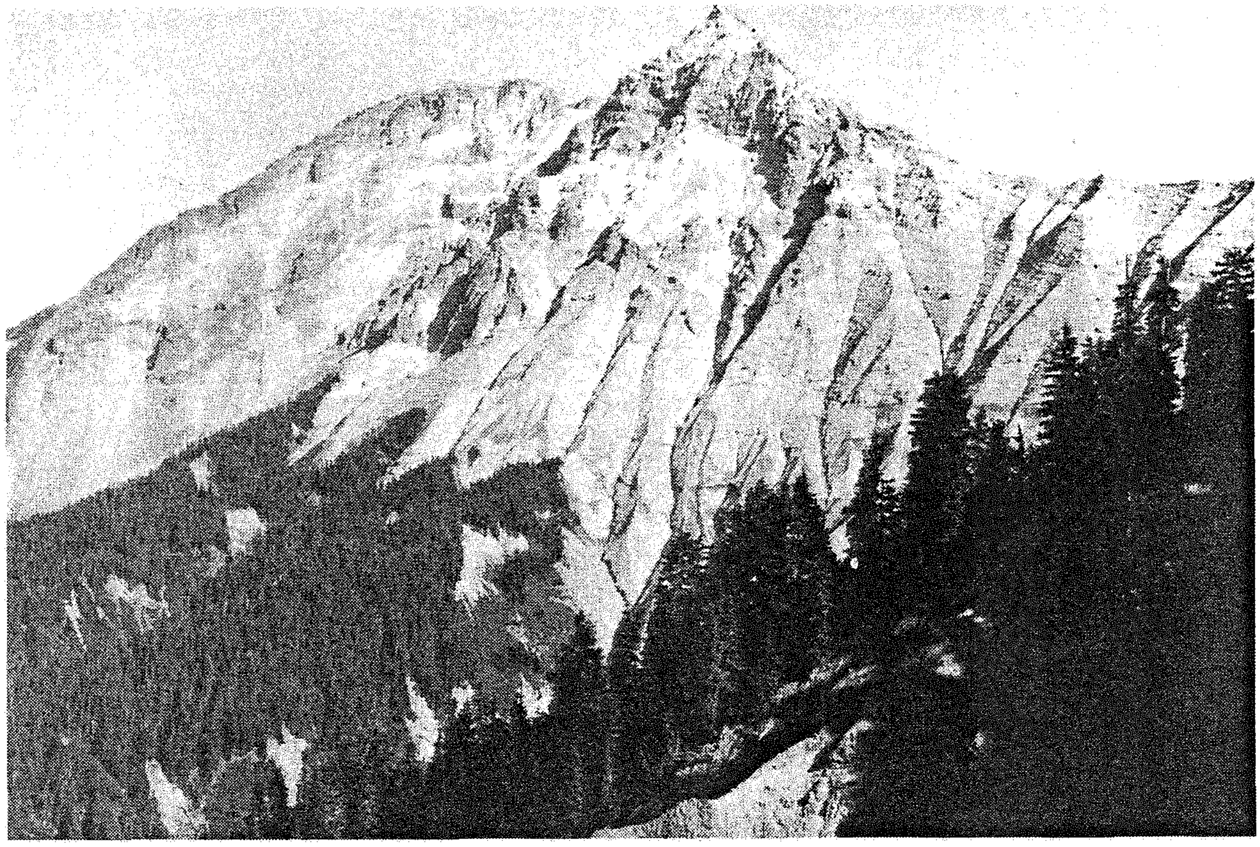


Figure 16 The west face of Atwell Peak (right) and Dalton Dome (left), taken from Cheekye Ridge.

connected with the emplacement of the hot pyroclastics (Mathews, 1952). Should this hypothesis be true, some of the lower units in the pyroclastic sequence must predate the Fraser period, as the altered zone can be found as low as Elevation 800 m. The effect of the alteration is to weaken the strong green parent metamorphics and produce a zone of orange and white - coloured, highly fractured weak rock mass with thick joint infillings of silt and clay.

The rounded peak of Dalton Dome, screening the main summit of Mt. Garibaldi on the left side of Figure 16, is another vent of volcanic activity, which produced a 150 m thick sequence of massive lavas, welded tuffs and pyroclastic breccias. Its layers slope towards the Cheekye Valley at about 32 degrees over a distance of more than a kilometer. Since they cover the erosional scarp left by the slumping of the Atwell cone, they must date after the retreat of ice from the valley. Windblown silt was deposited presumably under periglacial conditions on the lower part of the ridge, indicating that this last eruption in the Cheekye Valley was complete in late Pleistocene.

Mount Garibaldi is dormant at present. Studies by the Geological Survey of Canada revealed no geothermal activity in the area. The region is transparent on maps of earthquake epicentres.

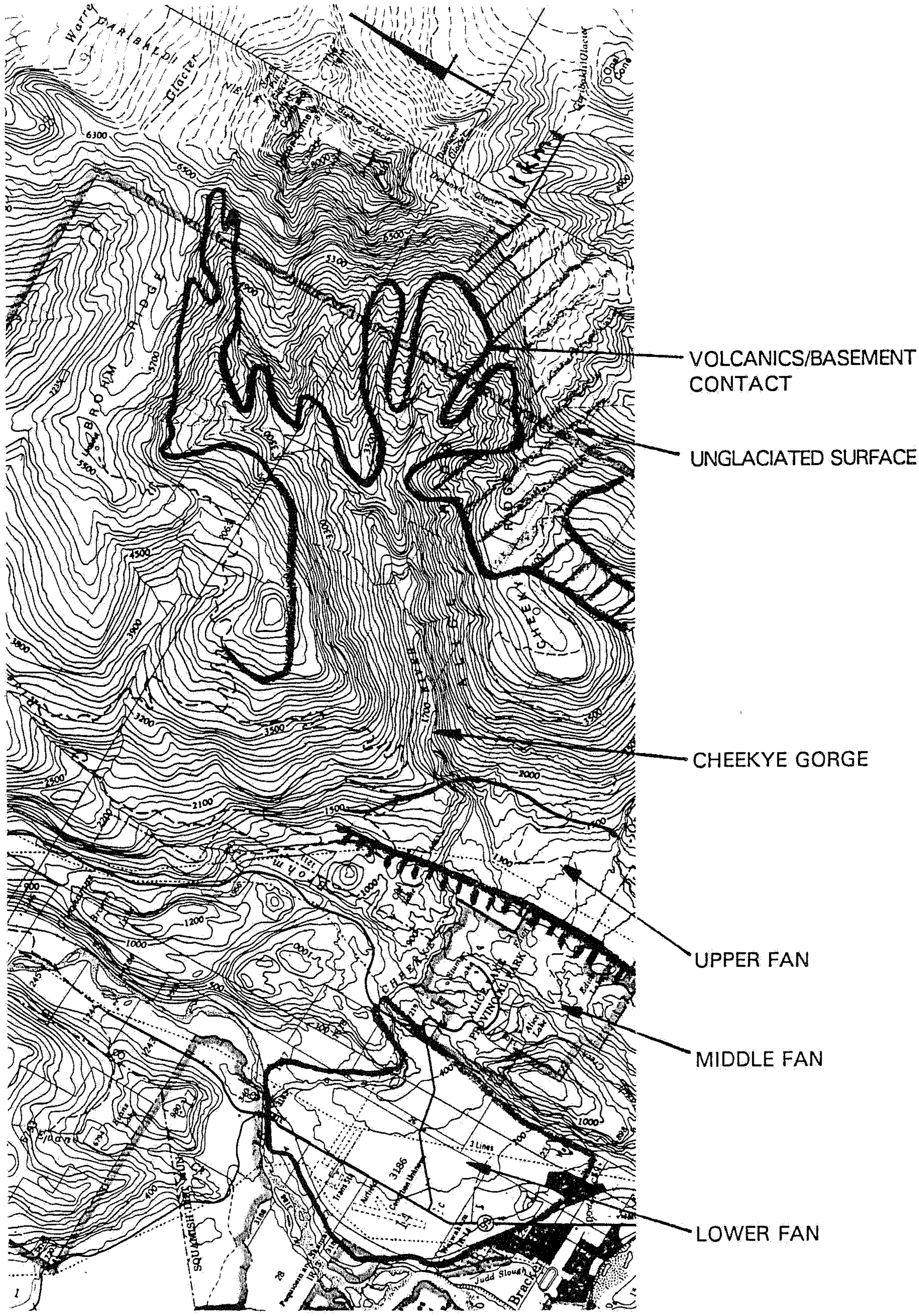


Figure 17 Schematic map of the Cheekye Valley and fans.

CHEEKYE FANS

A sequence of terraced fans formed at the outlet of the Cheekye Gorge, depositing over and around glacially rounded bedrock knobs (Figure 17). The largest terrace (called "Upper Fan" by F. Baumann, unpublished) extends along the foot of the Squamish River valley slope just outside the mouth of the Cheekye Gorge. Its top surface is that of a gently sloping fan, concentric on the gorge. Its western margin is a somewhat irregular linear scarp trending parallel with the Squamish Valley. The Cheekye has cut a 100 m deep funnel-like breach in the upper fan, separating it in two segments.

The Upper Fan deposits are mostly unsorted sands and gravels of volcanic origin with varying proportion of boulders, similar to the pyroclastic breccias from which this debris originated. The deposits are faintly bedded parallel with the upper surface of the fan. The estimated volume of these deposits is 150 million m^3.

Mathews (1952) identified the Upper Fan deposits as the accumulation of detritus from the collapse of the supraglacial pyroclastic cone. The smooth sloping surface, faint but regular bedding and lack of large block concentrations suggest that the material deposited in the form of a long succession of debris flows, rather than a major dry rock or debris avalanche.

A band of irregular topography with relict channels, kettle lakes and hummocks extends west of the Upper Fan. All of the hummocks in this "Middle Fan" are composed of bedrock. The area between them is infilled by gravelly debris of volcanic origin, similar to the Upper Fan deposits. The present channel of the Cheekye River is bordered by a belt of relatively recent debris flow deposits. In several places it is controlled by bedrock. A line of bedrock hills defines the western margin of the Middle Fan.

The "Lower Fan" Is the present active fan of the Cheekye, extending radially from a gap in the bedrock hills out to the braided channels of the Squamish and Cheakamus Rivers. Its surface totals 7 square km and is relatively featureless, smooth, sloping radially at 4 to 6 degrees.

Much of the Lower Fan stratigraphy consists of stream gravels and cobbles similar to the bedload of the present Cheekye channel. The fan sequence does, however, contain periodic units of diamicton - an unsorted mixture of silty sand, gravel and boulders, similar to the Upper Fan deposits. The most prominent diamicton unit was found by F. Baumann in 1990 directly on the surface of the fan. At the Squamish refuse dump in the middle of the fan, the surficial diamicton reaches over 5 metres in thickness and covers buried soil, large tree trunks and upright stumps rooted in an underlying sand unit. The wood remains were radiocarbon dated at 1010 to 1390 years B.P. Other, shallower sections of the same diamicton were uncovered by test pits over as much as 4 square km in the central part of the fan. This diamicton is thought to have originated as one or several major debris flows from the Cheekye drainage.

Another major diamicton is found in the toe of an exposure at the bank of the Squamish River, where it is about 2 metres thick. This was dated by the Geological Survey of Canada at 5890 B.P. (Eisbacher, 1983). Correlating the entire stratigraphic sequence, it appears that there are two clusters of diamictons each containing one major and one or two smaller debris flow units, separated by a thick sequence of cobbly stream gravels.

A reconstructed depositional history of the Cheekye Fans is shown schematically in Figure 18. At the end of the Pleistocene, the Upper Fan deposits were formed during a period of extremely vigorous debris flow activity, occupying a narrow space between the kame terrace-fringed valley wall and stagnant ice, standing in the centre of the Squamish Valley. The ice tongue eventually retreated and the Upper Fan deposits slumped towards the west, forming the marginal scarp and the ablation topography of the Middle Fan. Meltwater channels traversed the fan. On final retreat of the ice, the Cheekye rapidly cut through the upper terraces and built its present floodplain and the Lower Fan with alternating stream gravel and debris flow deposits.

Both ridges bordering the Cheekye drainage have large fields of ancient cracks and scarps, indicative of slope instability (Figures 19 and 20). The scarps stand at the angle of repose (40 degrees) where formed in the pyroclastics or vertical in lava. They are generally 2 to 5 m high, with a few ranging as high as 20 metres. The major scarps have steep sides dipping downslope, indicating normal displacements. Many of the smaller scarps indicate reverse movement, often combining with the former to form grabens.

A test pit excavated at the foot of one of the normal scarps on Cheekye Ridge revealed the existence of a near vertical shear plane (Thurber Eng. and Golder Assoc., 1992). A colluvial wedge formed in front of the shear covers a pocket of peat, which presumably formed at the foot of the scarp shortly after its formation. This was radiocarbon dated at 3220 years B.P.

Seismic refraction profiles and detailed geological mapping showed that the contact between the volcanics and the basement with the underlying zone of alteration exists at a depth of approximately 60 to 100 m. The contact dips towards the Cheekye Valley as a result of the buried ancestral topography.

Our present interpretation of the Cheekye cracks is that they represent moderately shallow translational sliding displacements seated in the altered zone. The translational geometry explains the presence of reverse scarps and graben structures. The directions of the cracks are controlled by the topography of the contact, more than the topography of the present ground surface. The crack fields are at present dormant, but it is likely that detachments of debris from their frontal edges occurred in the past. A network of precise triangulation survey monuments was installed on Cheekye Ridge in the summer of 1991 and will be monitored during subsequent years.

The mechanism responsible for the emplacement of the Lower Fan diamicton units is not known. It is, however, presumably analogous to large modern debris flows described from similar geological environments at Mtns. Cayley and Meager to the north (Jordan 1992, Evans and Jordan, 1991) and at a number of other volcanic centres in the Western American Cordillera. The highly mobile debris flows begin as landslides high in the erosion-prone headwaters of streams similar to the Cheekye. Saturation of the moving debris occurs by entrainment of saturated detritus from the base of gullies (Gallino and Pierson, 1985),

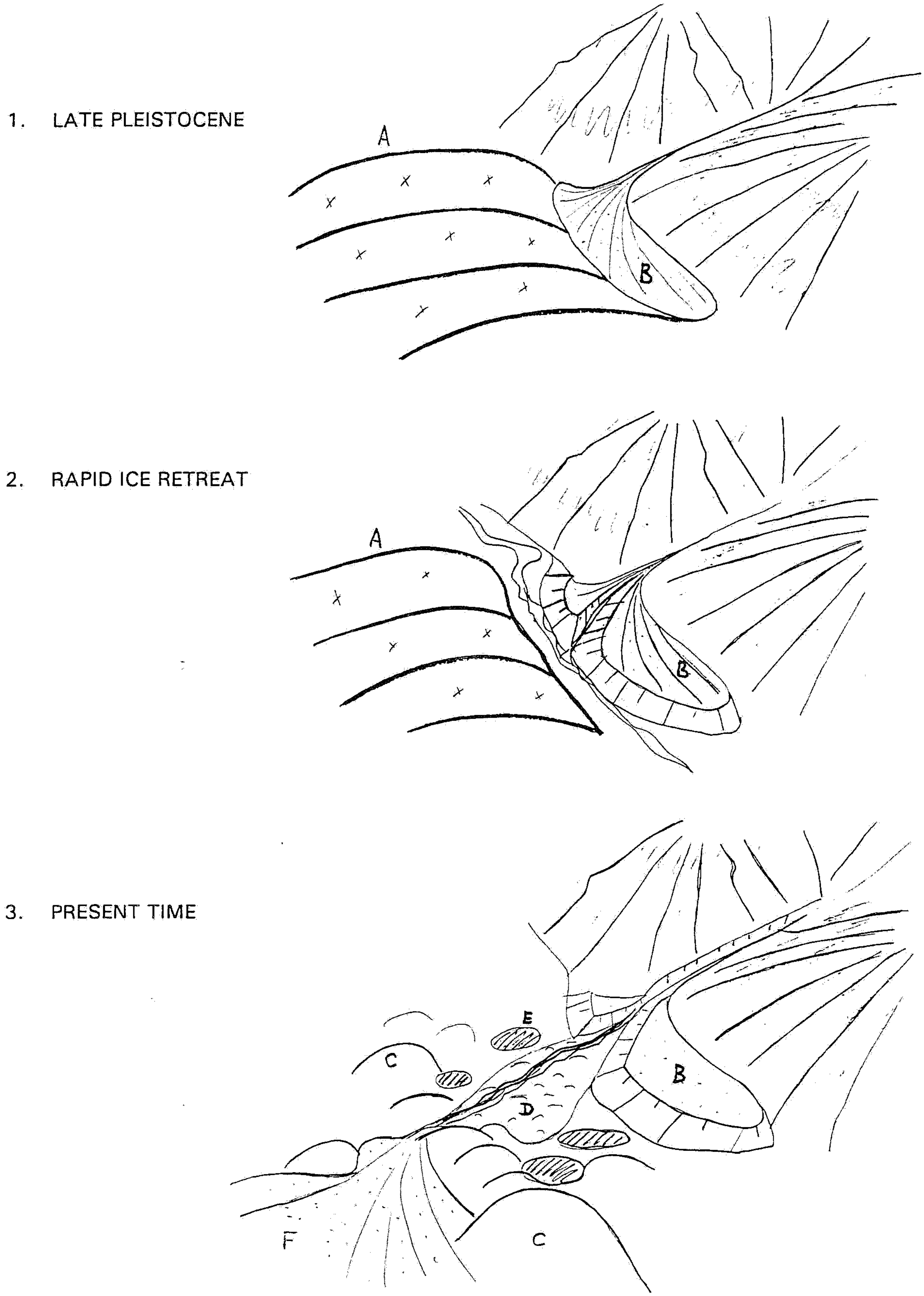


Figure 18 Schematic evolution of the Cheekye Fans: A-Waning Fraser ice; B-Upper Fan deposits; C-Bedrock knolls; D-Debris flow deposits; E-Lakes; and F-Lower Fan.

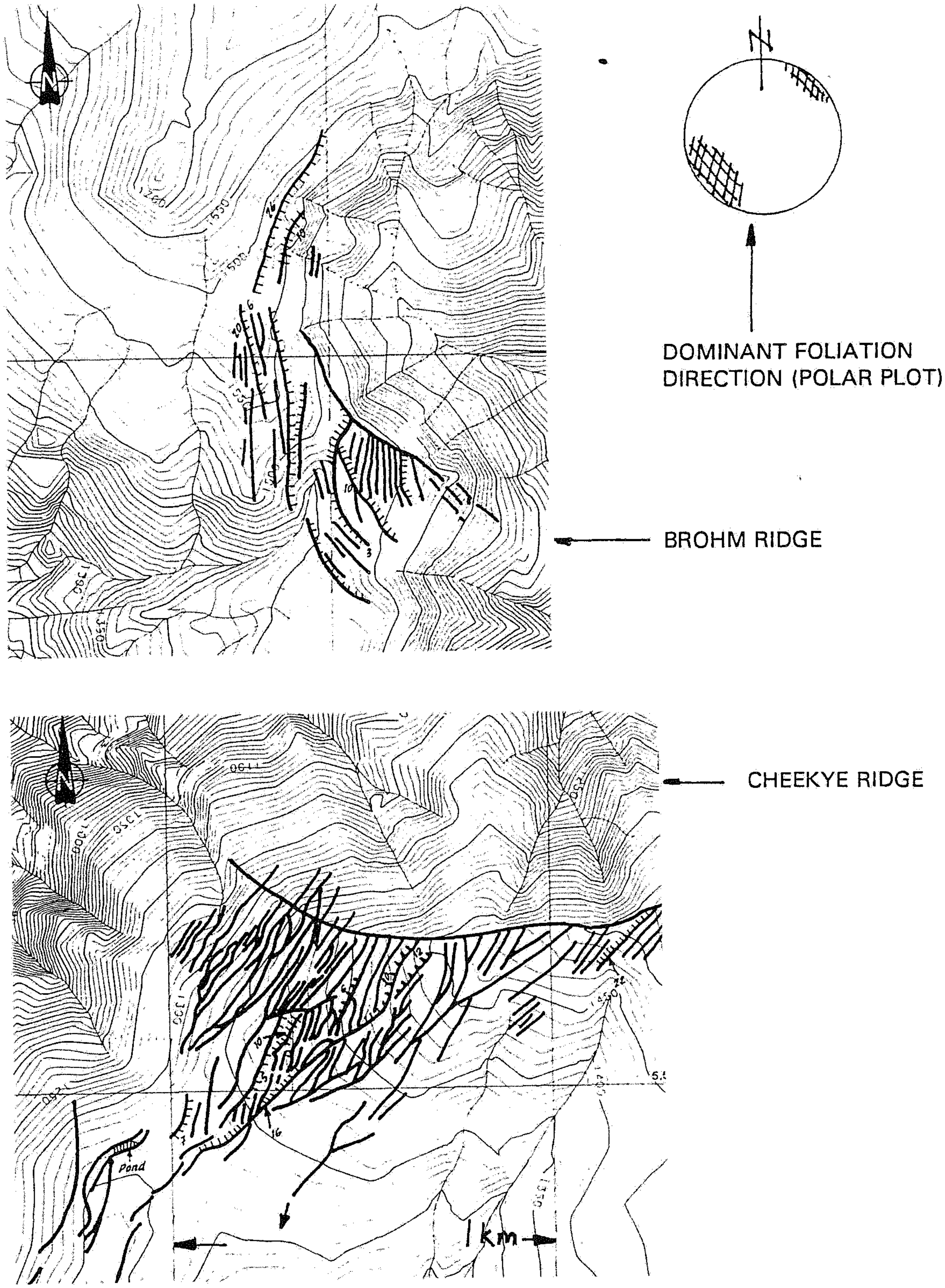


Figure 19 Distribution of scarps and cracks due to slope deformations on Cheekye and Brohm Ridges. Tics show downward displacements.

Figure 20 Linear scarps on Cheekye Ridge. The valley is to the left.

incorporation and melting of snow or ice (Cruden and Lu, 1991) or the formation and subsequent breach of slide dams in the upper or middle reaches of the drainage.

All of these saturation mechanisms are rather limited in terms of total volume of liquefied material produced. A few millions of cubic metres is considered a maximum magnitude for any individual event of this type. Greater magnitudes could have been created in the past by outburst of glacial lakes or subglacial water reservoirs. We have no evidence, however, of significant recent glaciers in the Cheekye drainage.

CHEEKYE FAN TERRAIN HAZARD STUDY

The Howe Sound Region and the Coast Range valleys as far as Whistler are undergoing quite rapid development, resulting from their closeness to Vancouver and from the growth of tourism. The Lower Fan of the Cheekye River is a large parcel of potential development land in Crown ownership, which could serve for an expansion of the town of Brackendale. Provincial government- sponsored plans for housing first appeared in the mid-seventies but were cancelled, partly as a result of uncertainty concerning the risk of landslides and debris flows affecting the fan.

In 1991, a consortium of Provincial ministries led by Lands, Parks and Housing commissioned a major study, to determine the geotechnical hazards on the Lower Fan and recommend feasible forms of development. The study, being completed by Thurber Engineering in association with Golder Associates of Vancouver, is one of the largest projects ever undertaken in B.C. in connection with assessment of natural hazards and their effects on housing development.

The Terms of Reference set by the Government for the consultants required that the full range of hazards be defined, from volcanic eruptions, lahars and rock avalanches to debris flows and flooding. The potential hazards were to be quantitatively described in order to estimate their reach and effects on the fan. The results are presented in probabilistic terms, using a procedure similar to that described by Morgan et al. (1992):

1) The frequency-magnitude relationship is estimated by a combination of stability analyses of slopes in the headwaters and a reconstruction of the depositional history of the fan.

2) With the help of runout analyses, zones of the fan exposed to various magnitudes of flow-sliding or floods with their associated probabilities are defined.

3) A limited range of potential land use types (e.g. high or low density residential, parks, golf courses, forest) are suggested by a professional planner.

4) "Severity" of the natural hazards is expressed as the rate of death, injury or material damage suffered by each alternative land use type due to each of the hazard categories.

5) The significance of the combination of risks expressed under 4) and probabilities determined under 1) and 4) are assessed by comparison with criteria of risk acceptability developed in connection with other types of hazards.

The above procedure is rationally structured, but many steps in it require subjective judgement. For example, the interpretation of fan stratigraphy is difficult due to a lack of natural exposures, except at the distal margins of the fan. Test pits and deep trenches were used to provide exposures higher on the fan, but these are limited in depth (the deepest trench dug was 13 metres). The use of boreholes was rejected, due to the difficulty of identification of stratigraphic units. Even in the test pits and exposures, the complexity of the deposits and lack of marker horizons made correlations difficult.

One surprising aspect of the investigation was the persistent lack of organic material in the stratigraphic column, with the notable exception of large tree trunks under the thickest surficial diamicton deposit at the refuse dump. In other locations, only rare and small fragments of datable material were found. The most credible explanation for this is that the organic material disintegrated completely under the aerobic conditions of the unsaturated granular deposits. Even wood casts are, however, rare. In contrast, the recent gravel deposits forming in the present bed of the Cheekye River, contain an abundance of wood fragments.

A further element of uncertainty connected with the use of stratigraphic evidence is that the glaciation of the upper basin may have changed over the last few centuries, due to neoglacial retreat as observed elsewhere in B.C. Some of the large glaciers presently bordering the Cheekye basin may have projected branches into the valley. Evidence of this may have been obscured by the vigorous erosional activity. The presence of valley glaciers could have provided conditions for triggering major flow slides and floods, which do not exist at present.

The development of magnitude-frequency relationships was therefore approached by combining the stratigraphic evidence on the fan with an analysis of a range of failure scenarios in the headwaters, based on detailed geological and geophysical investigations, stability analyses and comparisons with precedent case histories of landslides in other locations.

RUBBLE CREEK SLIDE

During the late winter of 1855-56 about 25,000,000 cu m of volcanic rock slid from the high cliff known as The Barrier, near Garibaldi, B.C. The debris travelled along Rubble Creek Valley to its confluence with Cheakamus River about 6.5 km from the Barrier and about 1035 m lower (Figure 21). This description of the slide is based on the work of Moore (1976) and the report by Hardy et al. (1978)

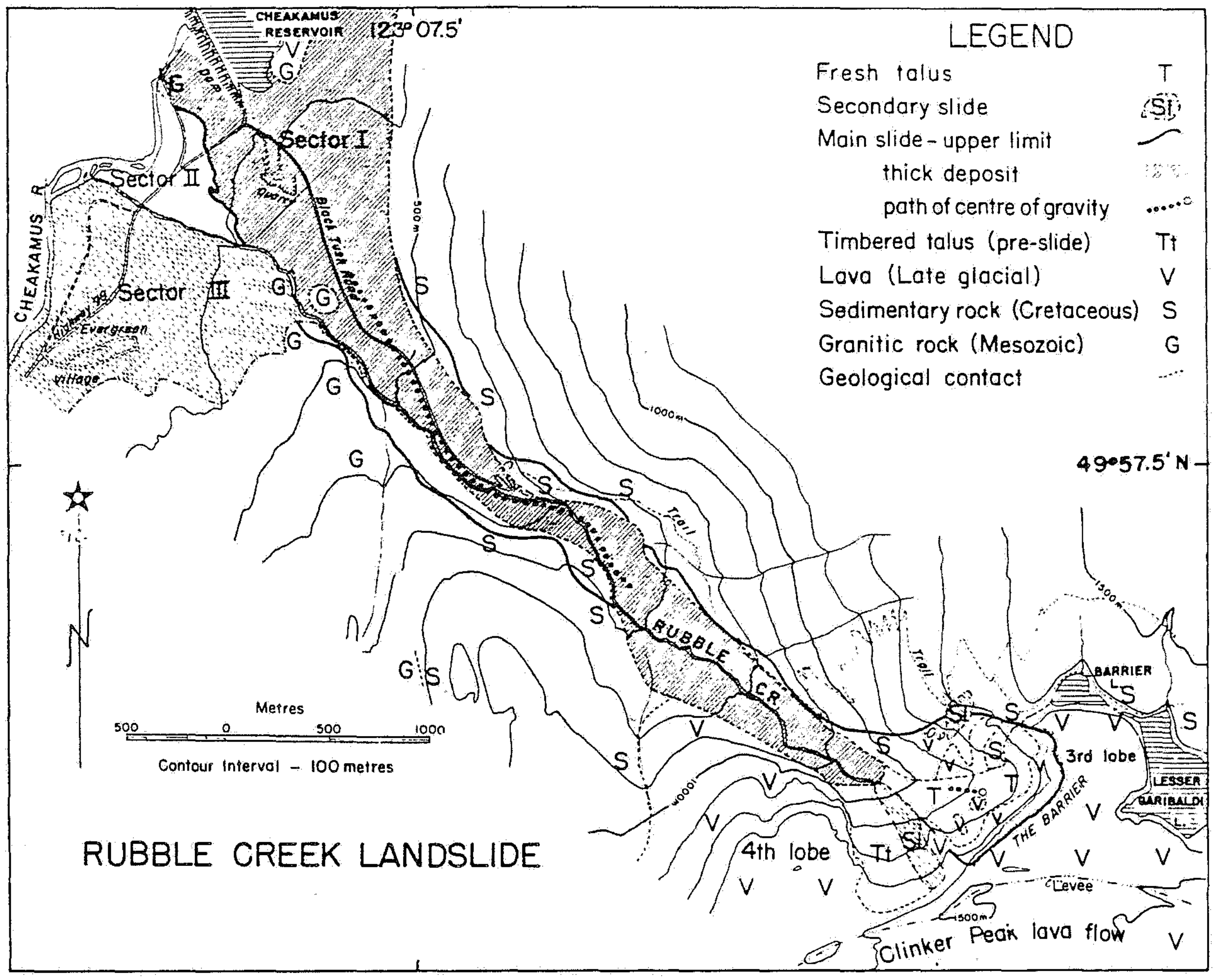
The source of the rockslide was the steep slope known as "The Barrier". This cliff was originally formed during the late stages of glaciation when lava formed precipitous terminal faces where it was ponded against ice remaining in the valleys below.

The mechanism that triggered the landslide is unknown, but blockage of a subsurface drainage system, which drains the area behind the Barrier and escapes as springs at its toe, could have raised groundwater pressures enough to trigger the slide. In addition, as the area is one of recent volcanic activity a local earthquake may have occurred.

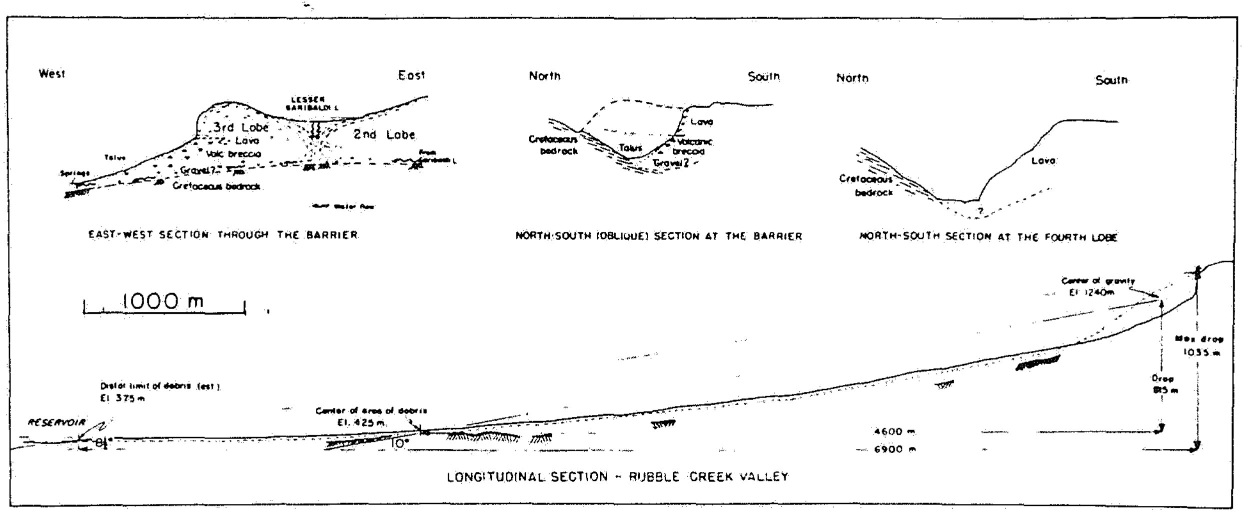
The material appears initially to have travelled as a high velocity tongue of debris, which swept from one side of the valley to the other as the debris stream rounded curves eventually to be deposited on Rubble Creek fan. Calculations indicate that the debris was moving about 30 m per sec. All of the trees in the path of this slide were uprooted and carried away. The trees adjacent to the slide were scarred and bruised by moving debris.

The initial high velocity tongue was apparently followed by debris floods that deposited large rounded boulders and poorly sorted, volcanic debris on an area of the fan not covered by the initial slide. This material was apparently slow moving, as it piled up on the uphill side of some trees that later died and fell on the surface of the debris.

The slide deposit is formed of angular poorly sorted volcanic casts from clay size up to about 250 t. The slide debris can be distinguished from underlying fan deposits by the lack of fine gravel and silt sized particles in the fan material.



Plan of the Rubble Creek landslide.



Sections, Rubble Creek landslide.

Figure 21 Plan of the 1855 rock avalanche at Rubble Creek (from Moore and Mathews, 1978, courtesy Can. Journal of Earth Sciences)

Large parts of the fan are covered by rounded boulders devoid of matrix. Beneath this surface layer is poorly sorted mudflow material. These boulders either could have been segregated to the surface of the mudflow during movement or the matrix could have been removed by subsequent erosion.

The extreme efficiency with which the slide travelled can be seen in the runnout angle. The angle between the top of the head scarp and the distal end of the debris is only 8½° from horizontal. The angle between the estimated position of the center of gravity of the slide mass before the slide and after the slide is 10°. This is consistent with the motion of other rock avalanches the courses of which were confined in narrow valleys.

The Rubble Creek landslide became a subject of controversy in 1972 after the second phase of a proposed subdivision to be built on the fan was refused government approval, because, in the opinion of the Senior Approving Officer, it would be against the public interest to allow the development to proceed. Two reasons were given: first, that there was danger of local flooding and second, that there was a danger of a catastrophic slide. As the first stage of the development had been approved just a few months previously, the developer proceeded with several hundred thousand dollars worth of work on the second phase prior to obtaining the necessary approval. Subsequently the administrative decision was appealed to the Supreme Court of British Columbia. During the court hearings evidence was given by expert witnesses concerning the causes of the massive slide that had occurred in 1855 and the likelihood of a repetition of such an event.

The Judge (Thomas Berger) sided with the Approving Officer to withhold approval. He said there is a sufficient possibility of a catastrophic slide during the life of the community to justify refusal to approve the subdivision.

After the law suit the government commissioned a major study of the slide area under the direction of Drs Frank Patton, Bob Hardy and Norbert Morgenstern. The resulting study became known as the Garibaldi Advisory Panel Report. It identified a number of earlier major debris avalanche deposits off the Barrier. Following this work the government bought out the existing community and moved them away from Geribaldi. It also imposed restrictions on the recreational use of Daisy Lake behind Cheakamus Dam. B.C. Hydro later argued for relaxation of these shoreline restrictions. Small rockfalls occur off the Barrier from time to time.

FITZSIMMONS CREEK

Our next stop is at Whistler, the year-round recreational resort. The village is built on the alluvial fan formed by Fitzsimmons Creek, which drains the valley between the two major ski areas of Whistler and Blackcomb. The village is protected by dykes on the left flank built by the Ministry of Environment prior to the village construction. Uphill of the village the drainage area of Fitzsimmons Creek is about 70 sq km. The creek has a flat gradient midsection steepening to about 11°as it approaches the apex of the fan.

Of some concern has been a slide in glacial till on the Whistler flank of the creek about 2.5 km upstream. This has moved from time to time probably as a result of high pore pressures during wet times and aided by undercutting of the toe by the creek. There seems little real danger, however, of the slide blocking the creek and leading to a washout flood.

A geotechnical study carried out for Whistler in 1987 identified debris laden floods as being the most serious natural hazard posed by Fitzsimmons Creek. A major flood on Boxing Day in 1980 moved large amounts of debris onto the fan, and a more serious repetition occurred in the summer of 1991. This is described in more detail in an article by Ward, Skermer and LaCas (1991 and 1992). About a quarter of million cubic metres of debris were transported onto the fan, and the river shifted course at a number of locations. Most of the bridges were either damaged or destroyed. Before and after photographs in the canyon just upstream of the village, showed that during the flood Fitzsimmons Creek had rafted thicknesses of debris of about 4 to 5 m down the water course. It subsequently cut down through these deposits, although in a number of places the boulder and gravel bed is 3 or 4 m higher than it was prior to the event. Obviously this is a source of material to be carried in future debris floods. Some excavation of the material has been carried out to minimize the amount that can be transported in future. In addition new dykes have been built on the fan to protect development on White Gold Estates on the right bank of the creek.

GREEN RIVER ROCK AVALANCHE

Our final stop, if time permits, will be at the Green River Rock Avalanche about 12 km north of Whistler.

The prehistoric rock avalanche is the largest landslide in the Green River valley. The surface of detachment of the rock avalanche is a fracture in diorite extending from 1200 m to 1600 m elevation. The rupture plan dips about 28° to 30° to the west, a few degrees less than the inclination of the mountainside immediately south of the breakaway zone. After failure, the disintegrating slab of diorite descended about 1000 m to the foot of the slope and crossed the channel of the Green River. The front of the rubble stream climbed about 140 m up a bedrock ridge in the centre of the valley, overtopped its crest, and swept down to the present location of Highway 99 and just over into the Soo Valley (Figure 22). During its ascent of this ridge, the stream of angular blocks left a 30-metre levee along its southern border. Blocks up to 15 m in size are found in this levee and on the surface of the main slide mass. The roadcut at the front of the lobe suggests that the block size in the basal zone is possibly smaller.

The lateral block levee suggests that the inner part of the lobe was propelled forward relative to the southern flank of the avalanche after the latter had come to a halt during it ascent of the bedrock ridge in the valley. The rockslide shows up very well on airphotos. Assuming an average thickness of about 20 to 30 metres for the landslide debris its approximate volume is estimated at 30 to 40 million m^3.

The age of the rock avalanche is less than 2,000 years (S.G. Evans, pers. comm.). The presence of very large Douglas firs and cedars on the floodplain aggraded upstream from the rock avalanche suggests that failure of the cliff and blockage of the Green River channel occurred at least a few hundred years ago. It may have been triggered by an earthquake.

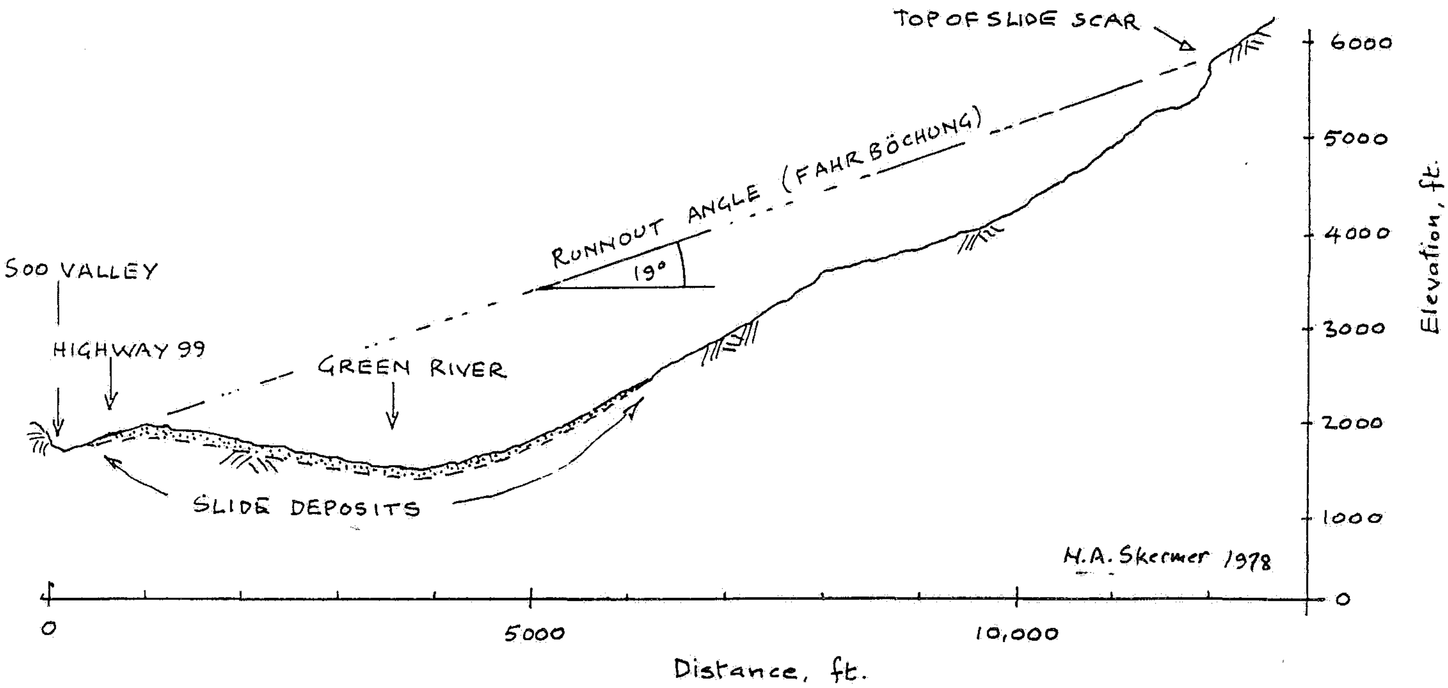


Figure 22 Central profile of the Green River rock avalanche.

LIST OF STOPS (see Figure 1)

1. Horseshoe Bay, beachfront.
2. Newman Creek bridge.
3. Lions Bay, Harvey Creek structure.
4. Porteau.
5. Britannia Beach.
6. Stawamus Chief.
7. Cheekye Fan.
8. Rubble Creek.
9. Fitszimmons Creek, Whistler Village.
10. Green River slide.

REFERENCES

1) Cruden, D.M., and Lu, Z.Y., 1991. The Rockslide and Debris Flow From Mt. Cayley, B.C., June 1984. manuscript in preparation.

2) Eisbacher, G., 1983. Slope Stability and Mountain Torrents, Fraser Lowland and Southern Coast Mountains. B.C. Guidebook, GAC-MAC Annual Meeting, Vol. 2, Victoria, B.C.

3) Evans, S.G., and Jordan, P., 1991. The 1984 Debris Avalanche and Associated Debris Flow From Mt. Cayley Volcano. Manuscript in preparation.

4) Gallino, G.L. and Pierson, T.C., 1985. Pollalie Ck. Debris Flow and Subsequent Dam Break Flood of 1980, East Fork Hood River Basin, Oregon. U.S. Geological Survey, Water Supply Paper 2273, 22 p.

5) Green, N.L., 1990. Late Cenozoic Volcanism in the Mt. Garibaldi Area, Southwestern B.C. Geoscience Canada, Vol. 17, pp. 171-175.

6) Hardy, R.M., Morgenstern, N.R. and Patton, F.D., 1978. Report of the Garibaldi Advisory Panel. B.C. Dept. of Highways, Victoria, 3 vols.

7) Hungr, O., Morgan, G.C., and Kellerhals, R., 1984. Quantitative Analysis of Debris Torrent Hazards for Design of Remedial Measures. Can. Geotech. Journal, Vol. 21, pp. 663-677.

8) Hungr, O., Morgan, G.C., VanDine, D.F. and Lister, D.R., 1987. Debris Flow Defenses in British Columbia. Reviews in Engineering Geology, G.S.A., Vol. 7, pp. 201-223.

9) Hungr, O., and Evans, S.G., 1987. Engineering Evaluation of Fragmental Rockfall Hazards. Procs., 5th Int. Symp. on Landslides, Lausanne, Vol. 1., pp. 685-690.

10) Hungr, O., and Evans, S.G., 1989. Engineering Aspects of Rockfall Hazards in Canada. G.S.C. Open File No. 2061, 100pp.

11) Jordan, P., 1991. Dynamic Behaviour and Material Properties of Debris Flows in the Southern Coast Mountains, B.C. In Evans, S.G. Ed., Landslide Hazards in the Canadian Cordillera, GAC Special Paper (in press).

12) Moore, D.P. and Mathews, W.H., 1978. The Rubble Creek Landslide, B.C. Can. Journal of Earth Sciences, Vol. 15, pp. 1039-1052.

13) Morgan , G.C., Rawlings, G. and Sobkowicz, J., 1992. Risk Assessment for Debris Flow Hazards. Procs., Geohazards 92.

14) Mathews, W.H., 1952. Mt. Garibaldi, a Supra-Glacial Volcano in South-Western B.C. American Journal of Science, Vol. 249, pp. 830-841.

15) Thurber Consultants Ltd., 1983. Debris Torrent and Flooding Hazards Along Highway 99. Report to B.C. Dept. of Highways.

16) Thurber Engineering Ltd. and Golder Associates Ltd., 1992. Cheekye River Terrain Study. Unpublished report to B.C. Ministry of Lands, Parks and Housing.

17) Toppe, R., 1987. Terrain Models, a Tool for Natural Hazard Mapping. In Avalanche Formation, Movement and Effects, IAHS Publ. No. 162, pp. 629-638.

18) Ramsey, B., 1967. Britannia, The Story of a Mine. Britannia Beach Community Association , 175 pp.

19) Skermer, N., and Russell, D., 1988. 28th of October. B.C. Professional Engineer, May 1988.

20) Ward, P.R.B., Skermer, N.A. and La Cas, B.D., 1991. The 50 Year Flood in Fitzsimmons Creek, Whistler. B.C. Professional Engineer, Dec. 1991.

21) Ward, P.R.B., Skermer, N.A. and La Cas, B.D., 1992. The 50 Year Flood in Fitzsimmons Creek, Whistler. B.C. Geotechnique and Natural Hazards. First Canadian Symposium, Vancouver, B.C.

Technical Tour No. 2

Fraser Valley and Fraser Canyon Areas

Wayne Savigny, P.Eng.
Dr. John J. Clague

Thursday, May 7, 1992

INTRODUCTION

The lower Fraser Valley is one of the most strategically important transportation corridors in Canada. Almost all lifelines that sustain the conjoint economies of the Lower Mainland and the resource-rich western provinces utilize it. This technical tour guidebook has been prepared to afford first-hand exposure to natural hazards affecting the corridor, with emphasis on landslides. The specific objectives are:

- to show the variety of landslide types;
- to illustrate the vulnerability of the corridor to landslide hazards;
- to provide information on attendant risks;
- to outline some of the risk management strategies adopted by users of the corridor; and,
- to form the basis of a one-day technical tour originating and ending in Vancouver.

The geological setting of the area is briefly outlined in the next sections. This is followed by the road log with expanded descriptions for features of interest, several of which are planned stops. Figure 1 is a map of the route with the features of interest located. Acknowledgements and references are provided after the road log.

GEOLOGICAL SETTING

Physiography

Fraser Lowland forms the southwestern corner of the British Columbia mainland and the adjoining northwestern corner of Washington state (Holland, 1964; Mathews, 1986). It is a triangular-shaped area of about 3000 km^2 bounded on the west by the Strait of Georgia and Puget Sound, on the north by the Coast Mountains, and on the south and east by the north Cascade Mountains. The Coast and Cascade Mountains are major mountain systems with many peaks in excess of 2500 m in southwestern British Columbia and northwestern Washington. In contrast, Fraser Lowland consists of flat-topped and gently rolling hills, most of which are below 150 m in elevation, separated by wide valleys. The largest of the valleys in Fraser Lowland is occupied by Fraser River which drains most of the southern and central interior of British Columbia and which terminates in a large delta prograding westward into the Strait of Georgia.

Geology

The Coast Mountains consist of Upper Cretaceous and lower Tertiary granitic rocks (100 to 45 Ma) and a variety of metasedimentary and metavolcanic rocks of subgreenschist to amphibolite facies, ranging in age from late Paleozoic to Cretaceous (Fig. 2). The mountains have undergone several episodes of uplift. It is estimated that there has been as much as 4 km of uplift in the last 10 million years (Parrish, 1983), and that the present rate is between 1 and 4 mm/year (Monger, 1992). The rocks are cut by regional, generally north-trending thrust and strike-slip faults. The eastern boundary of the Coast Mountains is marked by the late Eocene (46 to 35 Ma) Fraser fault system along Fraser Valley. Hope Fault (Fig. 3), one of the faults

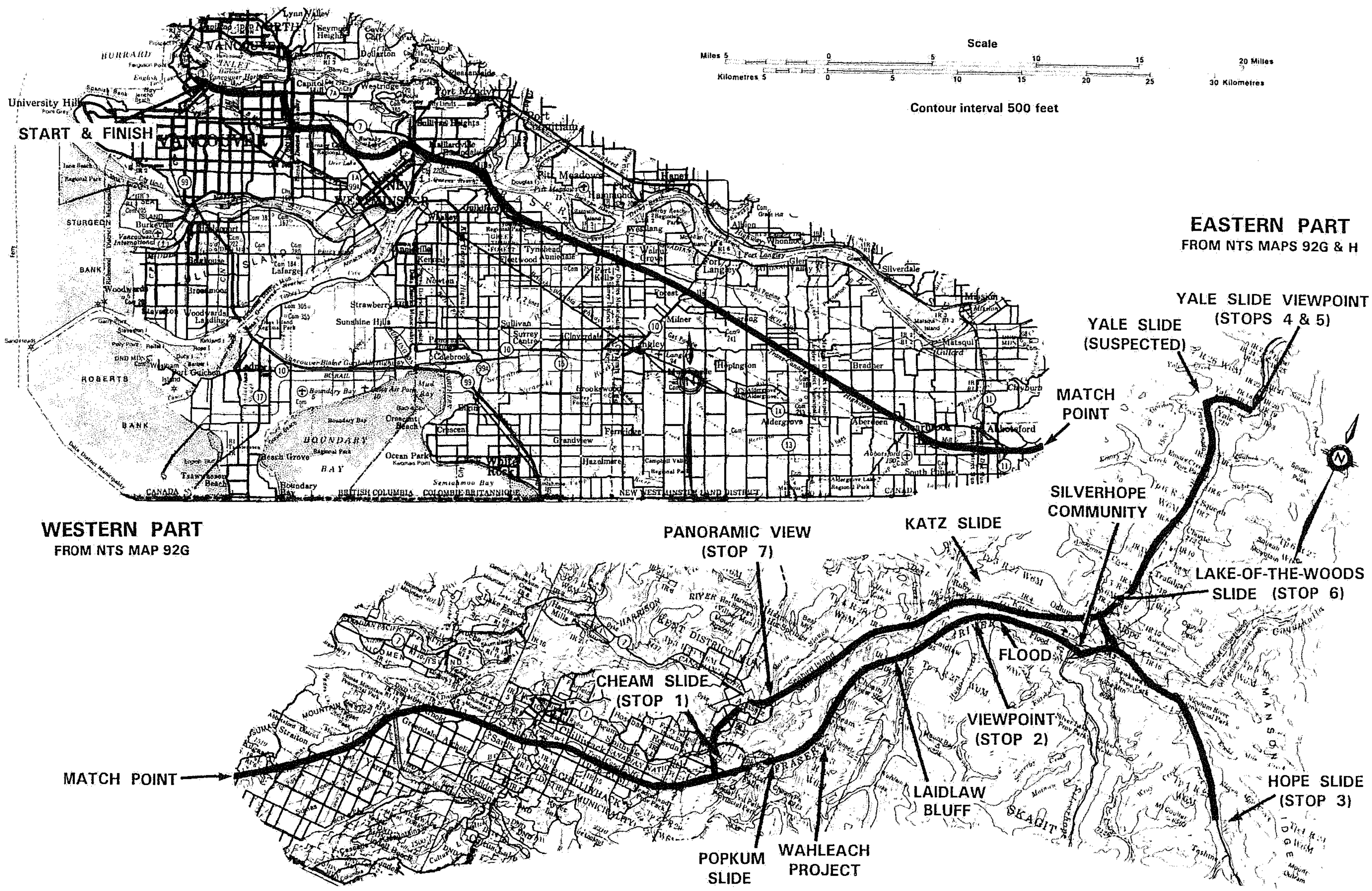


Figure 1: Route map.

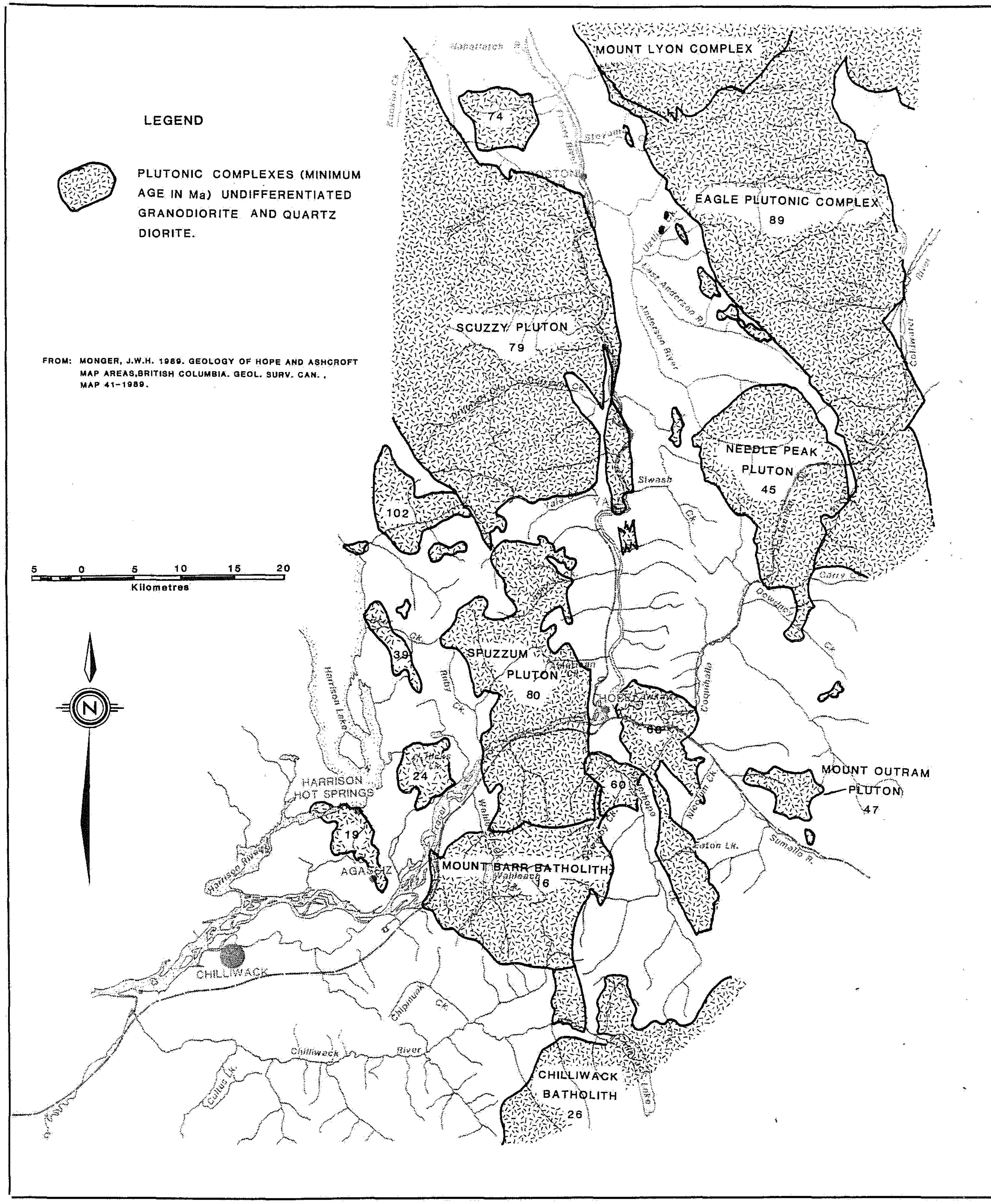


Figure 2: The distribution and age of intrusive plutonic rocks in the lower Fraser River valley transportation corridor (from Savigny, 1992).

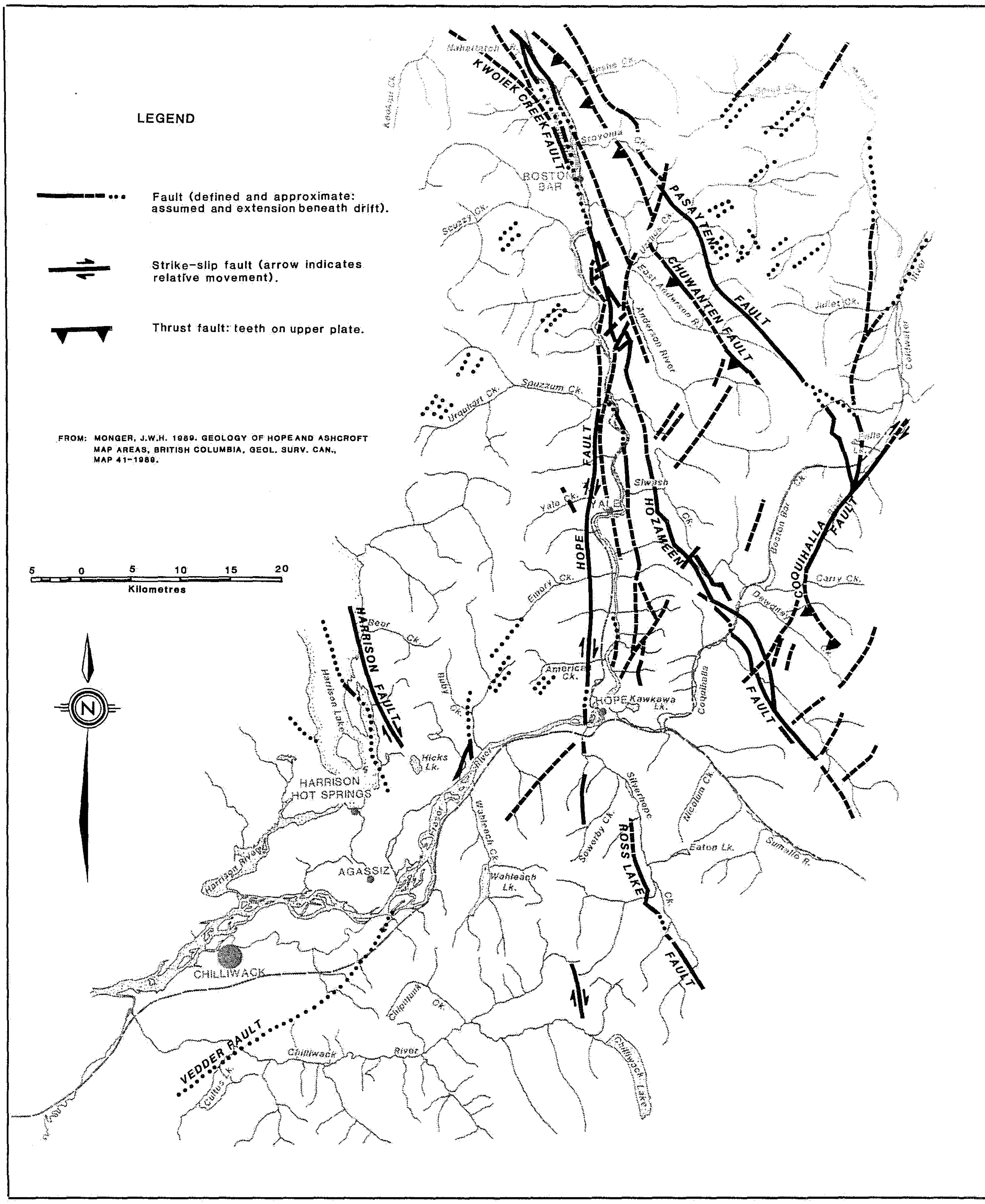


Figure 3: Regional faults in the lower Fraser River valley transportation corridor (from Savigny, 1992).

in this system will be seen at several locations on the tour. The southern boundary is the Neogene (25 to 16? Ma), probably extensional Vedder Fault (Fig. 3; Monger, 1992).

The north Cascade Mountains consist of Devonian to Cretaceous, greenschist-amphibolite facies metamorphic and granitic rocks. Rates of uplift, although similar to those in the Coast Mountains during the Cretaceous (100 to 84 Ma), have been lower than in the Coast Mountains throughout the Neogene (last 23 Ma). Regional, west-vergent thrust faults and north-trending dextral strike slip faults are similar to those in the Coast Mountains (Monger, 1992).

Fraser Lowland is part of a major structural trough which has subsided repeatedly since the Late Cretaceous and into which more than 4000 m of sediments, eroded from the adjacent uplands and mountains, have been deposited (Mathews, 1972).

The present physiography of the region was produced largely during late Tertiary and Quaternary time when rivers and glaciers dissected the rapidly rising region, shedding detritus into the lowlands and Pacific Ocean (Mathews, 1972; Ryder, 1981; Parrish, 1983).

Quaternary Tectonic Setting

The present tectonic regime of southern British Columbia is controlled by the motions of four lithospheric plates which are in contact in the Pacific Ocean west of the Canadian continental margin (Fig. 4). North of Vancouver Island, the two largest plates (Pacific and America) are separated by the Queen Charlotte Fault, a right-lateral transcurrent fault (Keen and Hyndman, 1979). To the south, two secondary plates (Juan de Fuca and Explorer) occur between the Pacific and America plates. A system of spreading ridges and short transform faults separates the Juan de Fuca and Explorer plates from the Pacific plate to the west (Riddihough *et al.*, 1983). The boundary between the Juan de Fuca and America plates is thought to be a zone of convergence or subduction (Riddihough and Hyndman, 1976).

The distribution of earthquakes, active and recently active faults, and Quaternary volcanoes in southern British Columbia (Fig. 4) is related to the tectonic regime outlined above. The main earthquake areas in this region are the Queen Charlotte Fault, offshore spreading ridges and associated fracture zones, and the southern Strait of Georgia (Milne *et al.*, 1978). The first two areas are seismically active because they are plate boundaries. The last area may correspond to the zone beneath which the Juan de Fuca plate remains essentially coherent as it is subducted beneath North America.

On the basis of high seismicity, the Queen Charlotte Fault and some transform faults connecting segments of Juan de Fuca and Explorer ridges are known to be active at present (Milne *et al.*, 1978). In addition, many minor and some major faults on land have Quaternary displacements.

Quaternary volcanoes in southern British Columbia occur in two linear belts (Fig. 4; Souther, 1977). The north-trending Garibaldi belt in the southern Coast Mountains includes about 32 Quaternary cones and domes of andesite, dacite, and minor basalt which may have formed in response to subduction of the Juan de Fuca and Explorer plates. The Anahim volcanic belt extends east-west across British Columbia at approximately 52N latitude and includes about

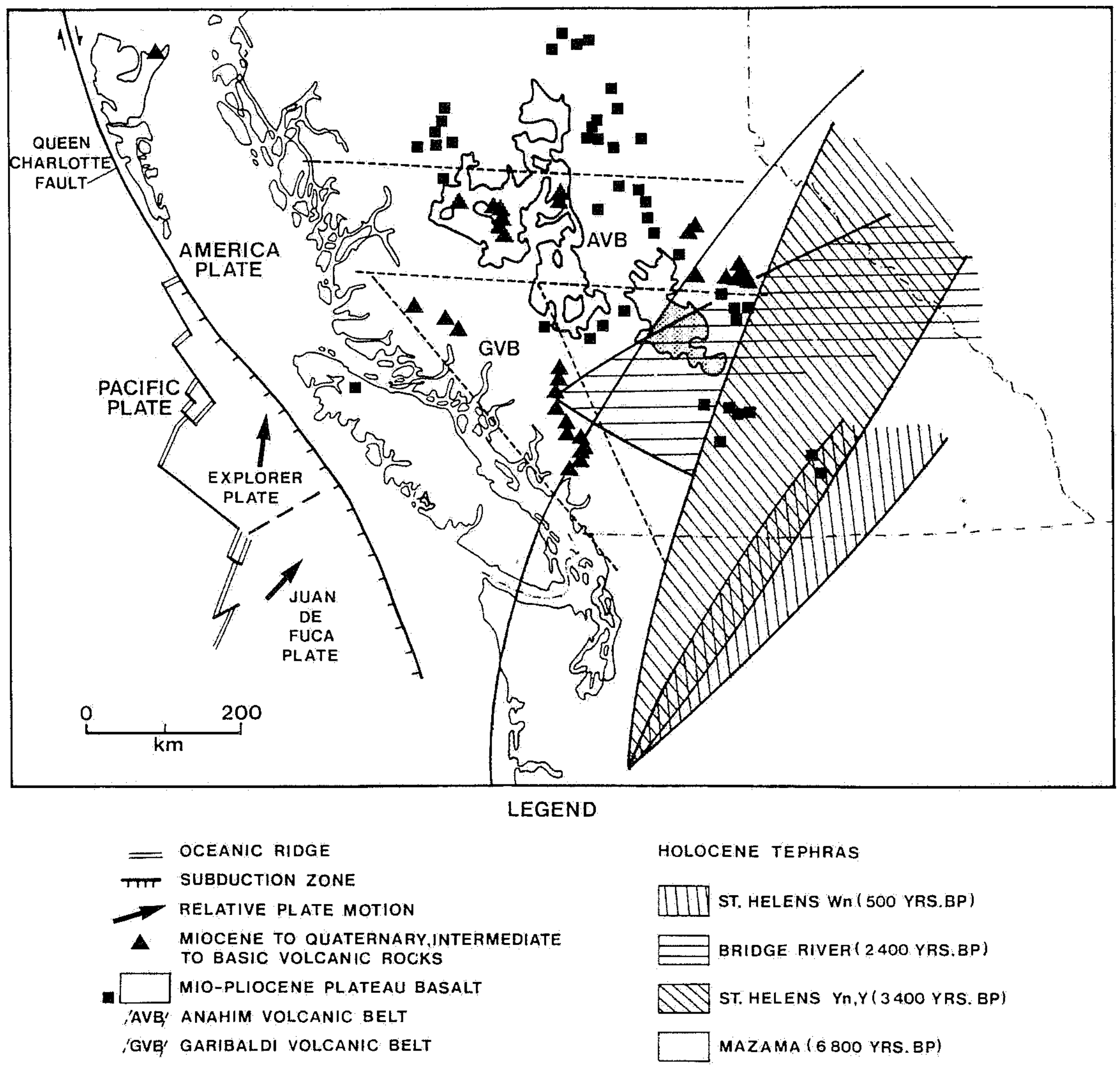


Figure 4: Lithospheric plates, late Tertiary and Quaternary volcanoes, and Holocene tephras in southern British Columbia (from Clague *et al.*, 1987).

37 Quaternary volcanoes of mainly basaltic composition. This belt may, in part, be the product of progressive movement of the America plate over one or more hot spots in the mantle (Bevier *et al.*, 1979).

Quaternary Deposits, and Glaciation

Quaternary deposits up to 300 m thick underlie most of Fraser Lowland, with bedrock hills projecting through the Quaternary cover in only a few localities (Armstrong, 1977a and 1977b). There is little or no relationship between the present land surface and the surface upon which the Quaternary sediments rest. Rather, the present surface is primarily a product of depositional processes operative during final Pleistocene deglaciation of the area 13,500 to 11,000 years ago and, to a lesser extent, during Holocene time.

The Quaternary succession in Fraser Lowland consists of sediments deposited during alternating glaciations and interglaciations (Fig. 5). The lowland was repeatedly invaded by glaciers from the adjacent high mountains during the Pleistocene. At times, ice in Fraser Lowland thickened to over 1500 m and coalesced with a lobe of the Cordilleran Ice Sheet that extended southward along the Strait of Georgia into Washington. Thick complex drift sequences deposited in association with glaciers in Fraser Lowland were eroded by overriding ice and by fluvial, marine, and mass-wasting processes operative during subsequent interglaciations. As a result of these processes, the Quaternary fill in Fraser Lowland consists of several drift packages separated by unconformities and by nonglacial deposits.

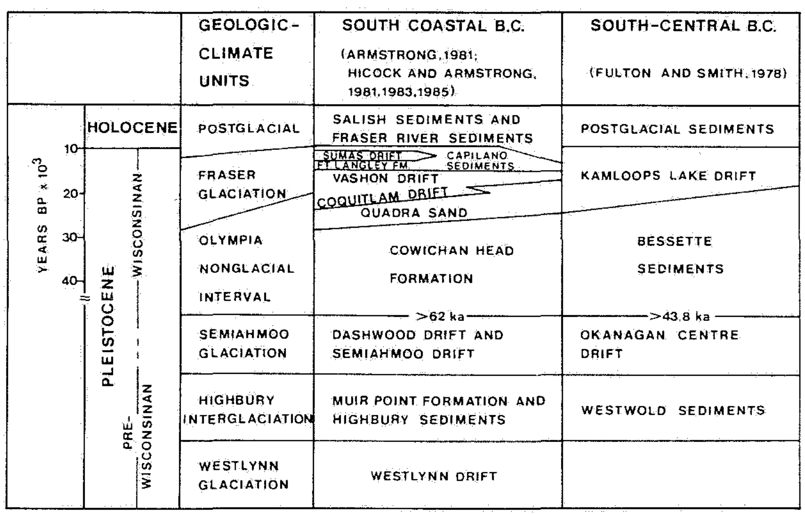


Figure 5: Subdivisions of late Quaternary deposits and events in southern British Columbia (from Clague *et al.*, 1987).

Each major glaciation was accompanied by isostatic adjustments related to glacier build-up and subsequent decay. Combined with eustatic changes in sea level, these glacio-isostatic adjustments produced vertical fluctuations in shoreline positions of up to 200 m in Fraser

Lowland (Mathews *et al.*, 1970; Clague *et al.*, 1982). As a consequence, low-lying land areas were repeatedly transgressed and regressed by the sea during the Pleistocene, and marine, glaciomarine, and deltaic sediments were deposited in complex associations with glacial, glaciofluvial, and ice-contact materials.

Large Landslides

The distribution of landslides and factors controlling slope stability along part of the Fraser Valley transportation corridor were considered by Piteau (1977). He evaluated a wide range of de-stabilizing factors before concluding that fluvial erosion and deposition had the greatest effect on slope stability, and that regional faulting, climate and the human effects were of secondary importance. A somewhat more restricted study by Bell (1980) reached similar conclusions. Both studies provide valuable engineering geology data bases for natural and engineered slopes at low elevations near Fraser River.

Savigny (1992) completed a study of large landslides (greater than approximately 1×10^6 m^3) and major lineaments in a 40 to 60 km wide corridor centred along Fraser Valley from Chilliwack to Boston Bar. Thirty five landslides and landslide complexes ranging in size up to approximately 500×10^6 m^3 are identified in Figure 6. Their areal extent is approximately 190 km^2 or about 4% of the study area. This represents a density of approximately one slide for every 140 km^2. Savigny (1992) suggested that these statistics are conservative, however, because the study area includes many landslides smaller than 1×10^6 m^3 (Piteau, 1977) which were not accounted for because of scale limitations. The large number of major lineaments shown in Figure 6 represent pervasive and contiguous, subvertical regional structures, many of which are shown by name in Figure 3.

The causative factors and processes can be summarized as follows. Some of these are well documented from case histories in the western Canadian Cordillera, but others have been given little or no consideration.

1. Human development

2. Climate and hydrologic conditions

3. Engineering geology:
   - geomorphology of slopes
   - lithology
   - structure
   - glaciation
   - neotectonics

Factors (1) and (2) tend to have the greatest affect on small landslides and are the focus of most geotechnical investigations. Large landslides are more likely to be influenced by regional engineering geology conditions listed under (3) (Savigny, 1992).

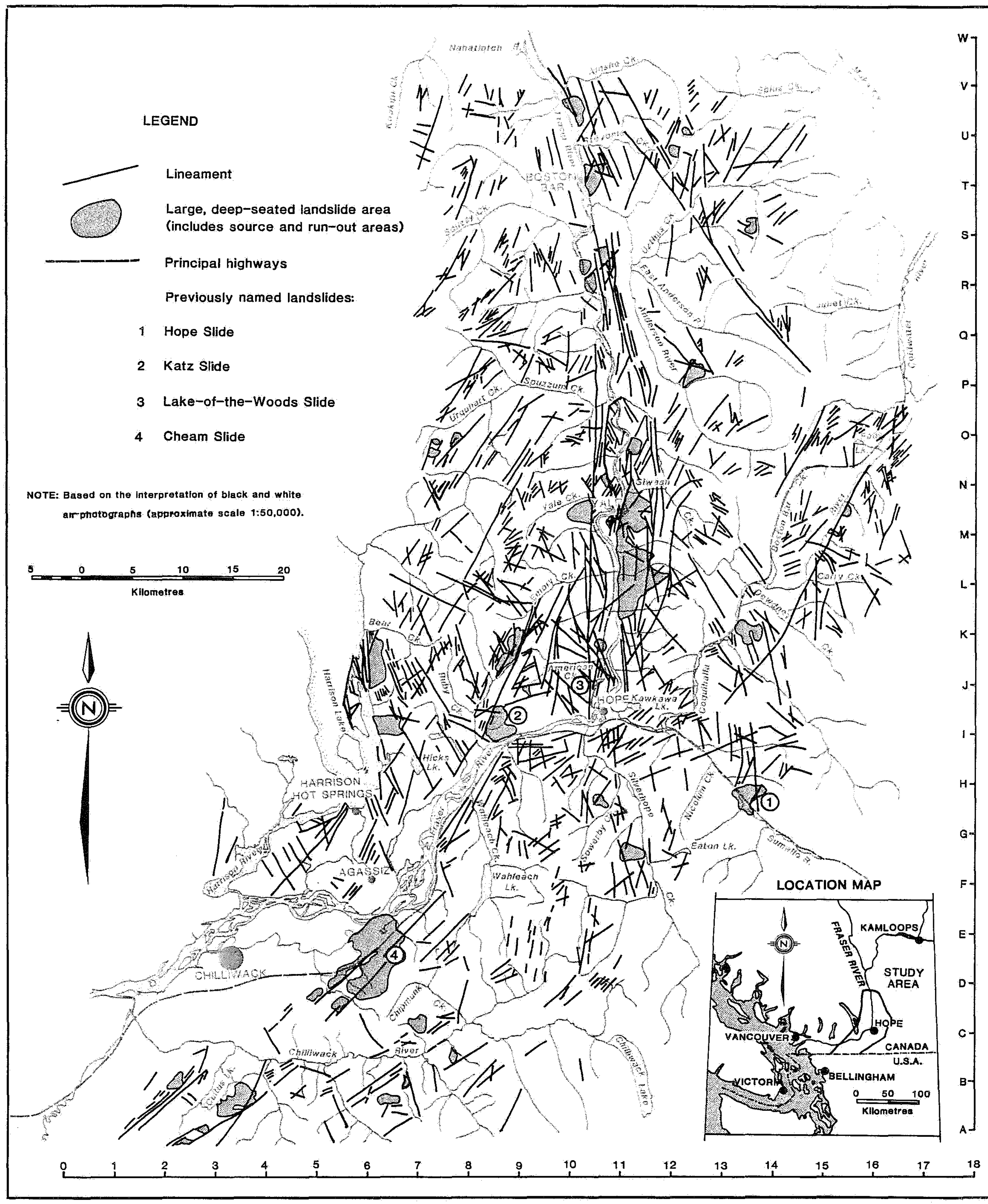


Figure 6: The distribution of large landslides and major lineaments in the lower Fraser River valley transportation corridor (from Savigny, 1992).

ROAD LOG

Introduction

The technical tour is organized on a kilometre by kilometre basis beginning and ending at the Ramada Rennaissance Hotel in downtown Vancouver. There is a 90 minute drive to the first stop. The tour guides will provide a commentary along the way.

km 0, Ramada Rennaissance Hotel

km 7, junction of East Hastings and the Trans Canada Highway

km 26, junction of Highway 7 and the Trans Canada Highway

km 35, junction of 176th Street and the Trans Canada Highway

km 57, view south to Mount Baker

Mount Baker is an impressive snow- and ice-covered peak situated a short distance south of the 49th parallel in Washington. Its summit is over 3200 m in elevation.

The active volcano is perched on a high bedrock ridge and is not as large as it appears. Despite this, Mount Baker is a potential hazard to nearby major population centres in Canada and United States. Eruptions of ash could seriously disrupt air traffic operations at Vancouver or Seattle-Tacoma international airports, and summit eruptions would melt large volumes of ice and snow creating a significant hazard to downstream areas from floods and mudflows. Historic eruptions date back to 1843; the most recent activity was emission of steam from March 1975 to early 1976 (Hickson, 1992).

km 74, junction of Highway 11 and the Trans Canada Highway

km 78 to 91, Sumas Lake drainage and Fraser River dyking

Early farmers in the lower Fraser Valley worked land that was seasonally flooded by the Fraser River. A great flood in 1894 caused severe damage and prompted the Provincial government to improve and extend private dyke systems (Siemens, 1968).

Prior to 1924, Sumas Lake covered the Sumas Valley Lowland between Sumas Mountain on the left (north) and Vedder Mountain on the right (south; Fig. 1). The lake and its wetland margins restricted access to Chilliwack and areas to the east (Roy, 1968). The agricultural potential of its soil had long been recognized, leading to proposals to drain the lake. Intensive drainage construction began in 1920 and pumping of remnant lake water began in July 1923. The lake was completely drained by 1924. Approximately 11,736 ha (29,000 acres) of new agricultural land was made available (Siemans, 1968).

The Sumas Lake area is now drained by Sumas Canal and is protected by 26 km of dykes. The tour crosses Sumas Canal near the Barrowtown Pump Station (left or north side of the highway at km 89) and another canal which diverts Vedder River directly into Fraser River (km 91).

A distributary of the alluvial system of the Nooksack River forms the head of the Sumas Valley. This river originates in northern Washington and flows into Bellingham Bay south of White Rock (Fig. 1). In November 1990, Nooksack flooding affected the Sumas Valley Lowland in Canada. The geologic and geomorphic relationship between alluvial fans of the Nooksack and Vedder Rivers in the Sumas Valley is documented by Cameron (1990).

District governments and the Water Management Division of the B.C. Ministry of Environment, Lands and Parks maintain lowland flood and drainage control works as far upstream as Agassiz, about 30 km east of Sumas Canal. About 505 km of dykes and 94 pump stations protect 74,000 ha of floodplain land (see Armstrong, 1984, p. 33-34). These works form the most extensive and highly maintained natural hazard protection system in the province.

km 80, Sumas Mountain clay quarries

Clay quarries are visible on the side of Sumas Mountain to the left (north) of the bus. Clay is mined from the Tertiary sedimentary beds at the base of the mountain, just behind the plant. The beds dip about 10 southwest. The northeastern part of the mountain is made up of Chehalis volcanic rocks (Jurassic), and the southwestern tip of granitic rocks.

km 83, Trans Mountain Pipe Line Ltd. oil pipeline crossing of the Trans Canada Highway

km 102, junction of main Chilliwack access with the Trans Canada Highway

km 117, junction of Highway 9 and the Trans Canada Highway; stay right and follow signs to Agassiz and Harrison Hot Springs

km 120, turn left into parking area next to the fruit stand. <u>Alternatively, if approval has been obtained from the local Band Council</u>, turn right onto the gravel road and follow the tour guides' instructions to the Band borrow pit (commonly referred to as the CN Rail gravel pit).

STOP 1 - CHEAM SLIDE

Location and Background

The Cheam Slide is a prehistoric rock avalanche located on the southeast (left) side of Fraser River, 15 km east of Chilliwack and 125 km east of Vancouver (Fig. 1). The landslide debris forms a hummocky landform, which contrasts sharply with the flat alluvial plain of the Fraser River and the steep slopes of the north Cascade Mountains (Fig. 7). Armstrong (1980) and Smith (1971) and several consultant reports describe the landform as a colluvial deposit but only speculate on the source area. Naumann (1990) provided the first comprehensive study of both the source and depositional areas.

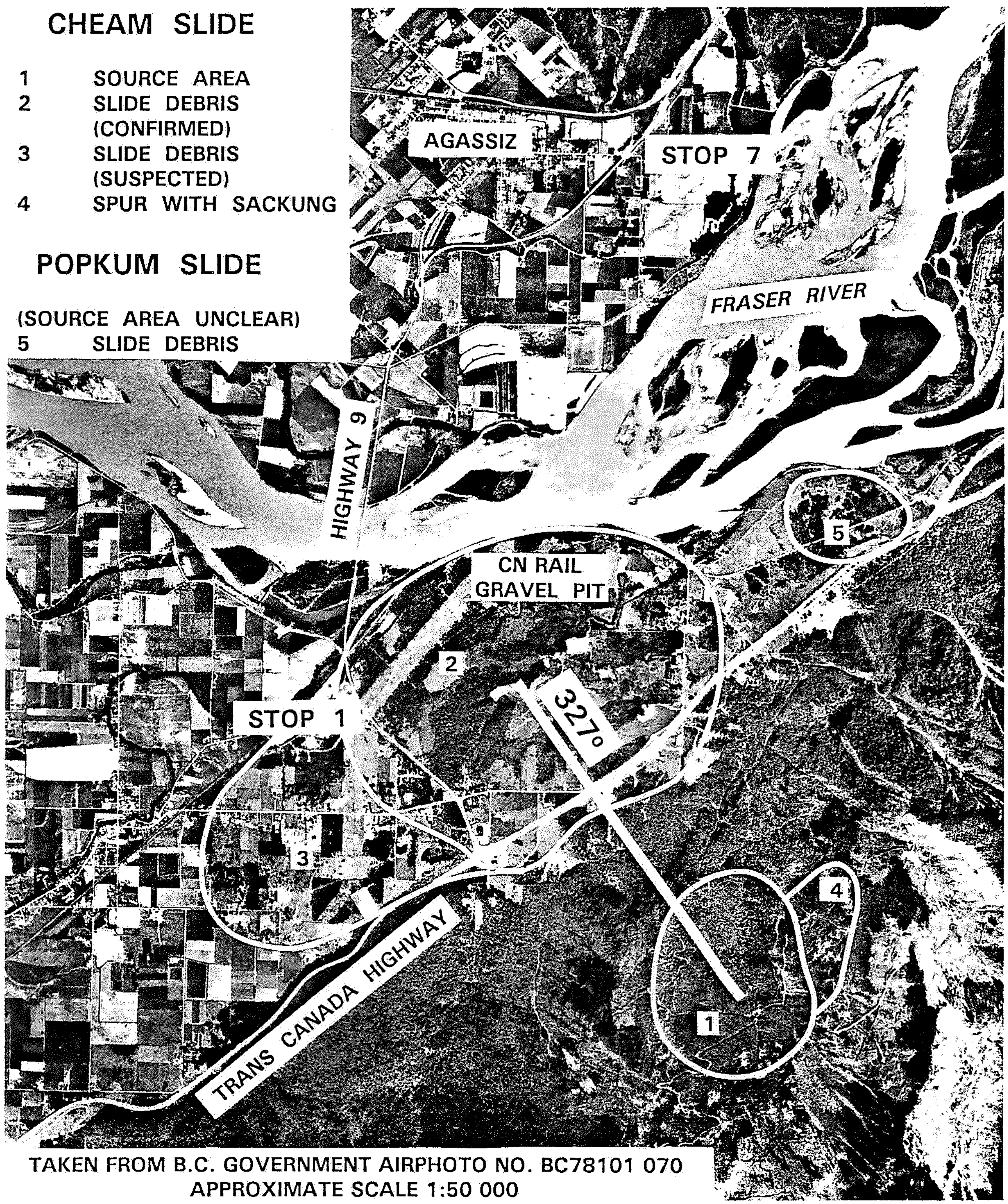


Figure 7: Air photo of the Cheam and Popkum slide areas.

Geological Setting of the Cheam Slide

Rock Units

The geological map in Figure 8 shows that the north Cascade Mountains in the vicinity of the Cheam Slide consist of a granitic core with pendants of weakly metamorphosed sedimentary and volcanic rocks. The core is represented by the Chilliwack and Mount Barr batholiths (Oligocene and Miocene ages, respectively) which consist of granodiorite. The Devonian to Permian Chilliwack Group includes fine- to coarse-grained volcanic arenites, argillites, cherty or argillaceous limestones, and local conglomerates and tuffs. Karst features are common in the limestones and at one nearby location karst contributed to a large landslide (Savigny, 1992). Stratigraphically above, and lying unconformably on the Chilliwack Group, is the Upper Triassic and Lower Jurassic Cultus Formation, comprising fine- to medium-grained volcanic arenites, argillites and slates (Monger, 1989; as reported by Naumann, 1990).

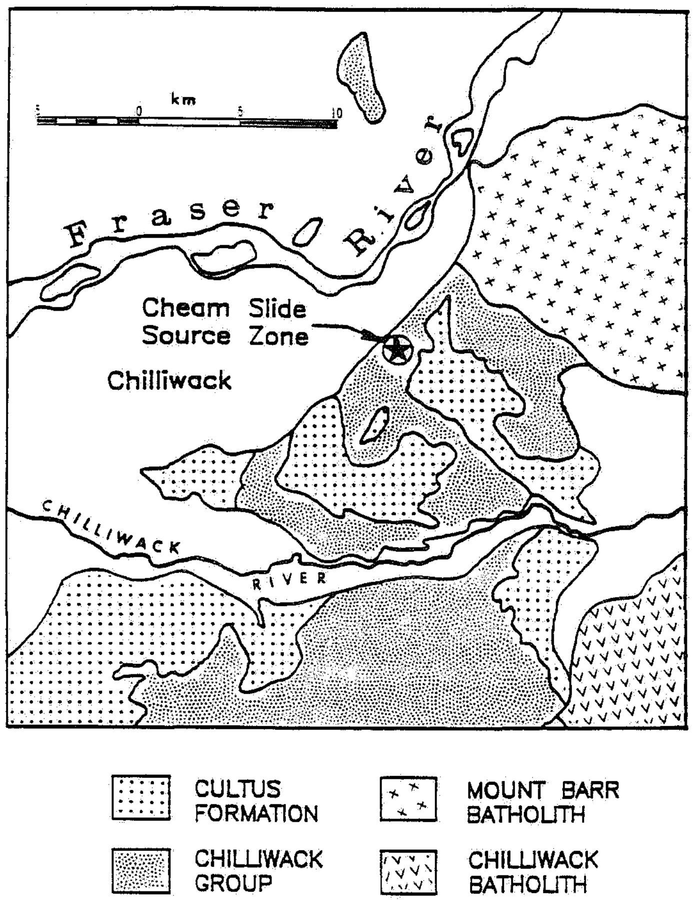


Figure 8: Bedrock geology of the Cheam Slide area (from Naumann, 1990).

Structure

Northeast-trending folds began to develop in the mid-Cretaceous. These were later overturned to the northwest, and southeast-dipping thrust faults developed. A ductile deformation style characterizes this phase of deformation. A second, brittle phase of deformation took place prior to the emplacement of the Oligocene Chilliwack Batholith. This is marked by open, northwest-trending folds and northeast-dipping thrust faults (Monger 1966, as reported by Naumann, 1990).

Stratigraphy of Surficial Materials in the Deposition Area

The stratigraphy of surficial materials in the depositional zone was described by Naumann (1990), based on measurement of exposures in the CN Rail gravel pit (Fig. 7). This site is believed to be near the northeastern limit of the landslide. Figure 9 is a plan view of the pit showing the measurement locations. Stratigraphic sections of the pit walls are illustrated in Figure 10.

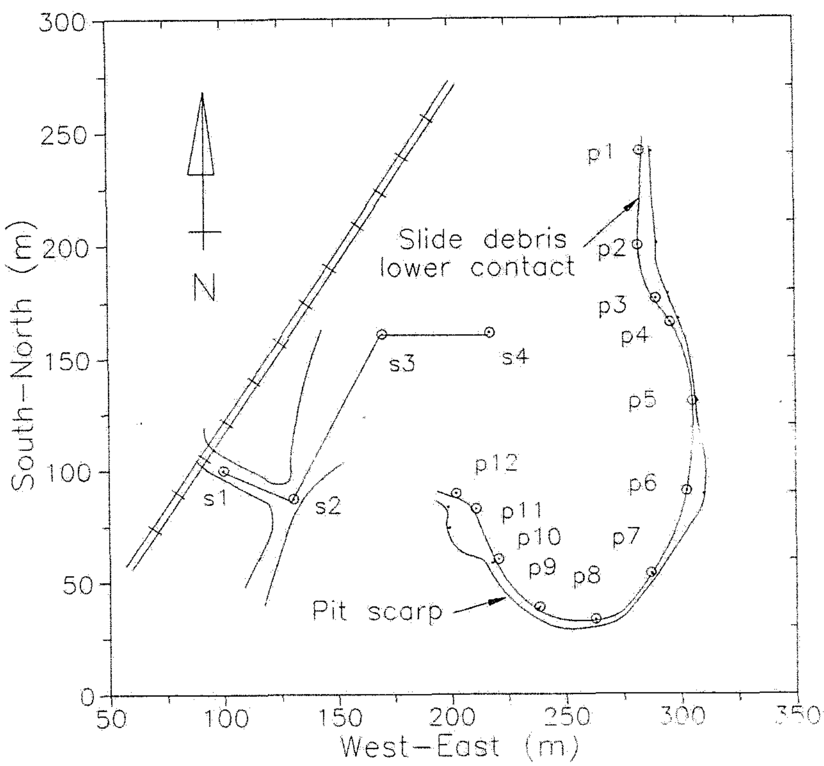


Figure 9: Map of the CN Rail gravel pit showing survey stations at the lower slide debris contact (coded 'P') in the working face exposures, and the survey line used to bring elevation control from the CN Rail tracks (coded 'S'). Corresponding vertical sections are shown in Figure 10. Note, the position of the working face is depicted as of August 1989. The location of the pit in the Cheam Slide debris is shown on Figure 7 (from Naumann, 1990).

The landslide debris rests on thick fluvial and glaciofluvial deposits. The basal sediments consist of more than 40 m of what are assumed to be glaciofluvial sand and gravel (unit 1, Fig. 10). These are overlain by a blanket of fluvial sand usually less than a metre thick (unit 2, Fig. 10). In the east pit wall the sand grades upward into a 1 to 2 m thick silt bed, and then a

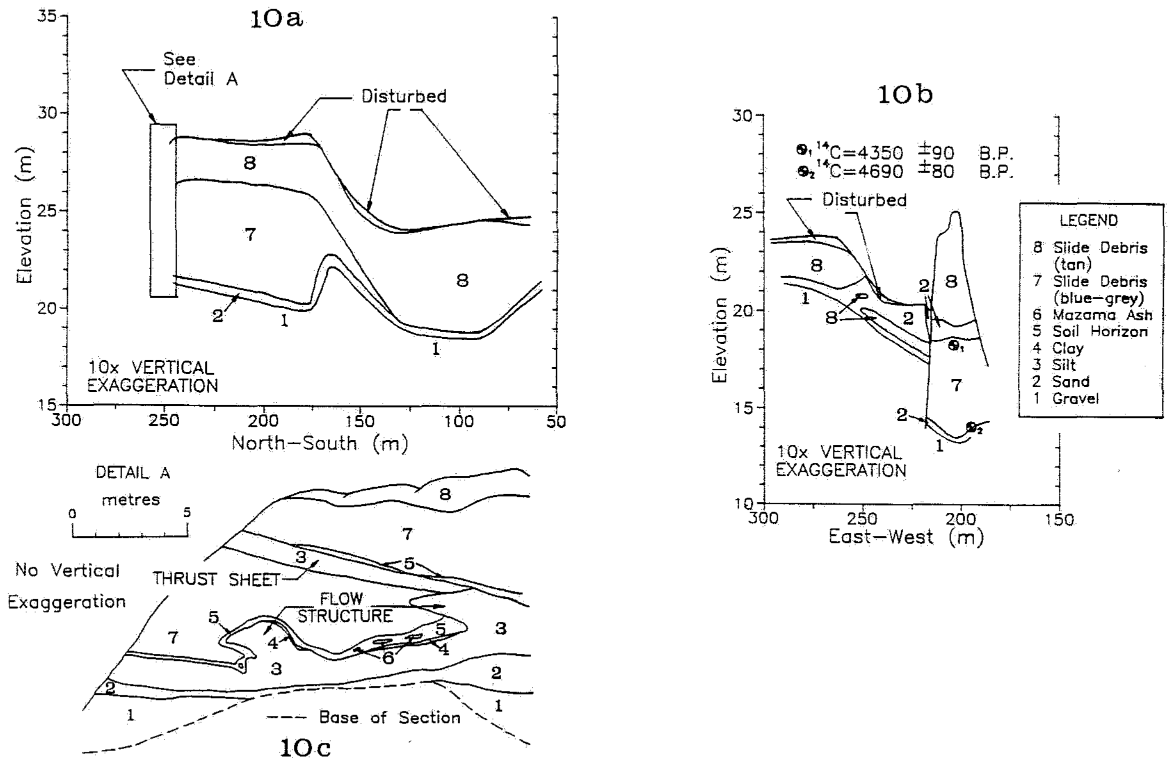


Figure 10: Vertical sections in the working face of the CN Rail gravel pit as of August 1989: a) looking east; b) looking south; and, c) detail of fault structures (from Naumann, 1990).

discontinuous clay layer up to 0.15 m thick (units 3 and 4, respectively, Fig. 10c). A soil horizon (unit 5, Fig. 10c) marks the top of this sequence. Mount Mazama ash (Nasmith *et al.*, 1967; Bacon, 1983) found in the soil horizon (unit 6, Fig. 10c) means the landslide debris at this site is younger than 6800 years BP (Naumann, 1990).

The landslide debris is represented by two units, one blue-grey in colour and the other tan (units 7 and 8, respectively, Fig. 10c). The tan colour is believed to be a result of near-surface weathering. The two units cannot be distinguished on the basis of texture or plasticity, but X-ray diffraction analyses show the tan-coloured unit has a higher illite content.

Grain size analyses reported by Naumann (1990) demonstrate that the landslide debris has a different texture than the local glacial deposits. Clast characteristics are also dissimilar, with the slide debris consisting of predominantly angular clasts, the largest of which are derived from the Chilliwack Group, the dominant rock unit in nearby slopes (Fig. 8). The landslide debris is classified as SM (sand with low plastic, silty fines) to SC (sand with low plastic, clayey fines) according to the Unified Soil Classification system. The debris ranges up to 9 m in thickness, but the top of the section is disturbed by grading activity and so the maximum thickness must have been somewhat greater (Naumann, 1990).

Two diapirs and a thrust structure in the east pit wall (Fig. 10c), another thrust structure in the south pit wall, and plastic deformation of gravel and sand (unit 1) at measurement point P3 (Fig. 9) are interpreted to be evidence of undrained loading imposed by the landslide event. The diapirs in Figure 10c indicate liquefaction of the fluvial deposits, particularly unit 3. The associated thrust structures show that excess pore pressures in unit 3 facilitated faulting when a secondary pulse of debris overtook the slowing debris front.

Two wood samples were collected for dating from the locations shown in Figure 10a. The first was from a log entrained in the slide debris. The second was a rootlet sampled immediately below the basal contact. The uncorrected radiocarbon ages are 4360±90 years BP (lab no. SFU W-04) for the log and 4690±80 years BP (lab no. SFU W-03) for the rootlet (Naumann, 1990). A third date of 5010±70 years BP (GSC ________) was obtained on a log in the debris by Clague (1989).

Source Area

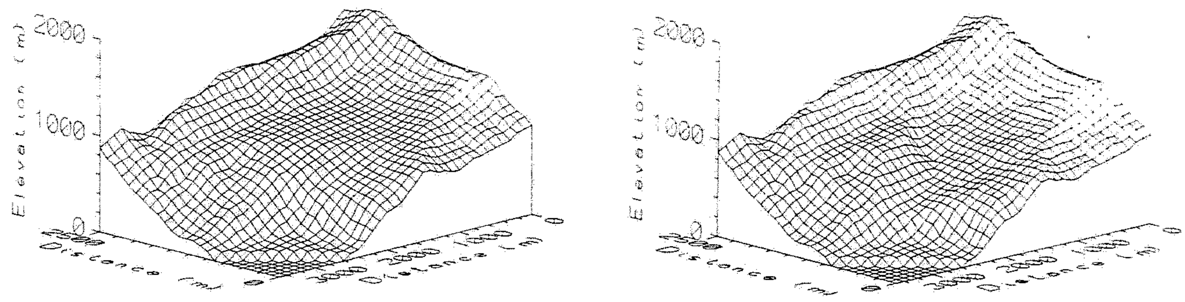
Naumann (1990) theorized that the source of the landslide must be the mountain slopes overlooking Bridal Falls (Fig. 1), based on the direction of flow structures in the debris (Fig. 10c) and the arcuate alignment of hummocks in the landform (Fig. 7). On the basis of airphoto interpretation, he identified a large asymmetrical wedge depression on the mountain side. The line of intersection of the two main surfaces forming the wedge daylights about 400 m above the Fraser Valley floor.

Field checking confirmed that one of the surfaces is a thrust fault trending 340 and dipping 21 northeast. The fault is characterized by a 1 m thick shear zone which contains slickensided clay gouge having a Unified Soil Classification of CL (clay of low plasticity) and clay minerals consisting of chlorite and illite. No single extensive plane representing a second surface was found in outcrop. Naumann proposed that a joint set striking 100 and dipping 49 south controlled the second surface, which he estimated to strike 85 and dip 39 south.

Slide Volume

The pre- and post-slide topographies are compared in Figure 11. A source volume of 150 x 10^6 m^3 is determined by comparing these surfaces. The volume of landslide debris was estimated by Smith (1971) to be 56 x 10^6 m^3. Assuming a 15% bulking, Smith's estimate would reduce to 49 x 10^6 m^3, which is less than one-third the volume determined from the source area reconstruction (Naumann, 1990).

There are several possible explanations for this volume discrepancy. Fraser River, which turns sharply to the west as it flows past the landslide (Fig. 7), may have originally flowed against the mountain slope. The large volume of debris required to fill such a channel was not considered by Smith. Other possible explanations are that the pre-slide topographic reconstruction is wrong, or that the slide scar developed in response to more than one landslide



(a) (b)

Figure 11: Topographic configuration of Cheam Slide source area: a) before slide; and, b) after slide.

event. Colluvium from one or more earlier events may have been removed by glacial or fluvial processes.

Slide Mobility

The line of intersection between the two surfaces of the asymmetric wedge trends 327. This is the same as the trend of the line between the centre of gravity of the reconstructed source area wedge and the centre of the landslide debris as shown on Figure 7. The angle of inclination from the top of the slide headscarp to the distal margin of the debris is 11.2. The calculated F value is 0.2 (see inset, Fig. 12). This is consistent with the behaviour of other landslides in the Canadian Cordillera, and with the predicted mobility from the Scheidegger relationship (Scheidegger, 1973).

Re-evaluation of the Cheam Slide by Robert Gerath

Recent investigation by Thurber Engineering Ltd. (1991) and Gerath (in progress) has found evidence which prompts a re-evaluation of the Cheam Slide interpretation of Naumann (1990).

The landslide debris is rubble and diamicton composed of distinctive rocks from mountain slopes to the south. The lithologies contrast with drift and alluvium from other regional sources. Rubbly surface material extends 5 km between Popkum and Rosedale (Fig. 1). The western limit of concentrated rubble is a 2 km long, curved landform near Rosedale. Rubble ridges, up to 50 m high, and intervening elongate depressions are visible from the Trans Canada Highway and Highway 9.

Rubble and diamicton overlie thick Fraser Valley gravel and sand in the northwestern half of the landslide deposit. This is retreat outwash deposited at the end of Fraser Glaciation between ice in the central Fraser Valley and older drift and other ice on the south valley side. Hummocky topography and deformed and faulted outwash indicate burial and collapse of dead ice. Most debris post-dates the outwash but diamicton contained in gravel indicates local syndeposition.

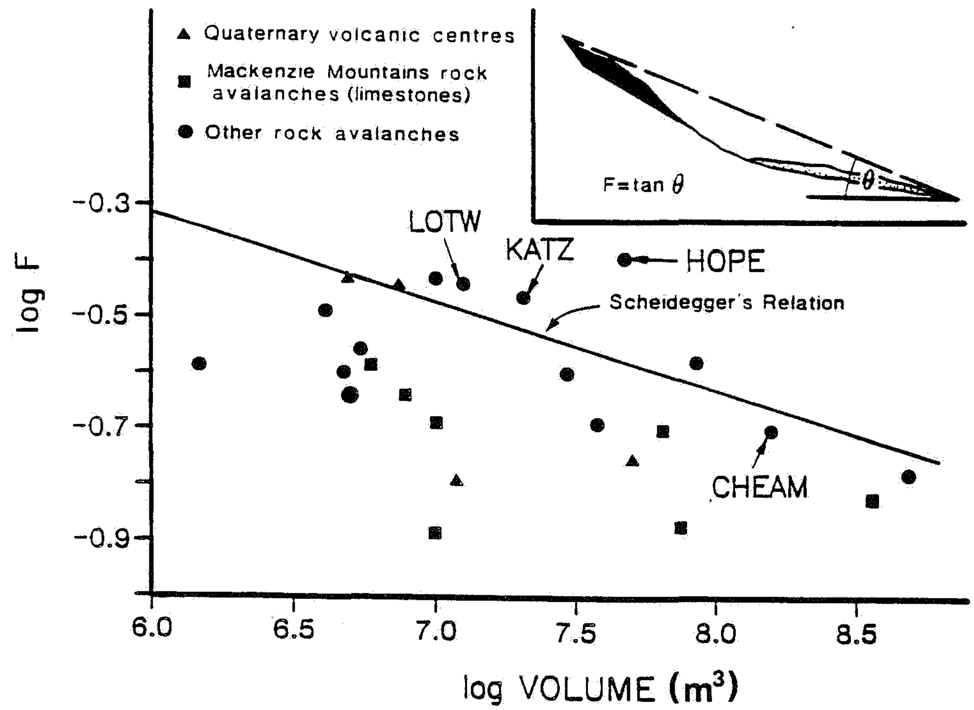


Figure 12: Mobility of the Cheam, Hope, Katz and Lake-of-the-Woods (LOTW) slides compared with other rock avalanches in the western Canadian Cordillera and the Scheidegger (1973) empirical relation (from Naumann, 1990; modified from original by Evans *et al.*, 1989).

Surface diamicton exposed in pits along the northwestern margin of the landslide deposit is till-like with some textural and colour banding. If it is flow material, a slide occurred while there was subaerial, but waning, outwash deposition. There was probably relatively thick, stagnant ice to the north, south and the east. Alternatively, if the diamicton and rubble are lodgement and ablation till facies, Cheam Slide must have ran out onto glacier ice which had re-advanced over the outwash.

Meltwater discharging from remnant ice in the central valley cut hanging channel inlets and eroded complex channel systems through plateau debris, dead ice and Fraser Valley outwash. Reworked and mixed material was re-deposited on top of slide rubble on portions of the plateau west of Highway 9.

The landform near Rosedale probably marks the limit of slide debris run out on glacier ice. There may have been more than one slide event. Debris ran out adjacent to relatively thick masses of stagnant ice near Cheam Lake. Ice decay in this area formed topographically reversed, elongate depressions which were partially filled with Holocene fan colluvium, alluvium and Cheam Lake marl.

The Cheam Slide is radiocarbon dated at 5000 years BP. The dated samples were collected in diamicton near a Fraser River erosional slope on the landslide-distal side of the meltwater channel systems. This suggests the samples may be collected in colluvial earth material reworked from more ancient Cheam Slide debris.

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km 0, Oedometers should be reset at the fruit stand on the west side of Highway 9. Continue south on Highway 9, returning to the Trans Canada Highway, and follow signs west to Hope.

km 6, POPKUM SLIDE, by Drum Cavers

Approximately 2 km east of the Cheam Slide debris, another area of hummocky terrain can be seen on the left (northwest) side of the highway. Like the Cheam Slide, this landform contrasts sharply with the flat floodplain of the Fraser River and the nearby steep mountain slopes. The landform was first recognized in October 1992 and is currently being investigated. Several test pits excavated in the hummocky landform reveal massive, till-like material consisting of a heterogeneous mixture of silt, sand, gravel and cobbles. The material is tentatively interpreted as debris flow deposits from the north-facing slopes of Mount Cheam situated on the right (southeast) side of the highway.

A large tension crack or graben is present near the summit of Mount Cheam. The date of the last movement is unclear, but the feature is estimated to be at least several tens of years, and likely several hundred years old or more. The width of the graben is estimated to be 20 to 40 m, and the depth is as great as 16 m. A large number of cracks and holes are associated with the graben, and toppling is evident on the outer (northwest) side. There is no evidence of a "toe" downslope, hence the depth of movement is unknown.

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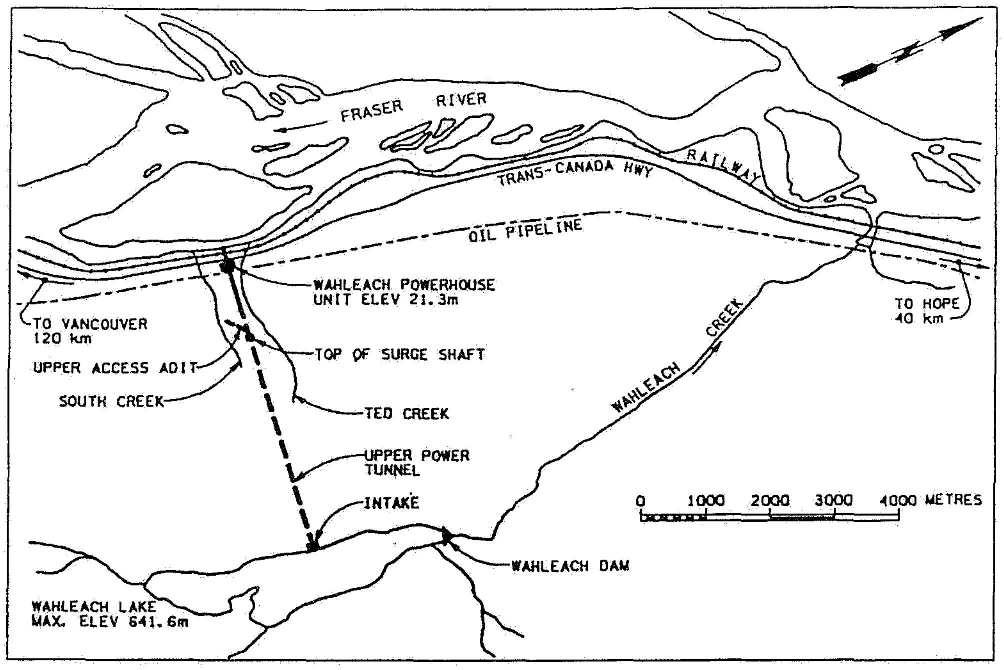
km 11, WAHLEACH PROJECT, by Dennis Moore, Alan Imrie and Doug Baker

The Wahleach hydroelectric development is located about 10 km east of the Cheam Slide and immediately southeast of the Trans Canada Highway (Fig. 1). Water flows from a reservoir at Wahleach Lake through a 3-m-diameter, 3500-m-long upper tunnel, a 600 m shaft inclined at 48, a 300 m lower tunnel, and a 485 m surface penstock to the powerhouse located on the right side (southeast) of the highway (Fig. 13a). The conduit is lined with concrete-encased steel below a point 235 m upstream of the intersection of the upper tunnel and the inclined shaft (Fig. 13b). The steel lining is 2 m in diameter and ranges from 10 mm thick in the upper tunnel to 38 mm thick in the lower tunnel. A total of 620 m of head is developed.

The tunnels were excavated mainly in granodiorite, which ranges from massive and fresh to highly fractured and moderately weathered. Subvertical joint sets strike both parallel and perpendicular to the slope, and a third set dips 40 subparallel to the slope. Shear zones strike parallel to the trend of the slope, are subvertical, and are more widely spaced and persistent than joints. Thorough investigations indicated that no extensive shear zones are oriented in a direction conducive to downslope sliding.

(a)



(b)

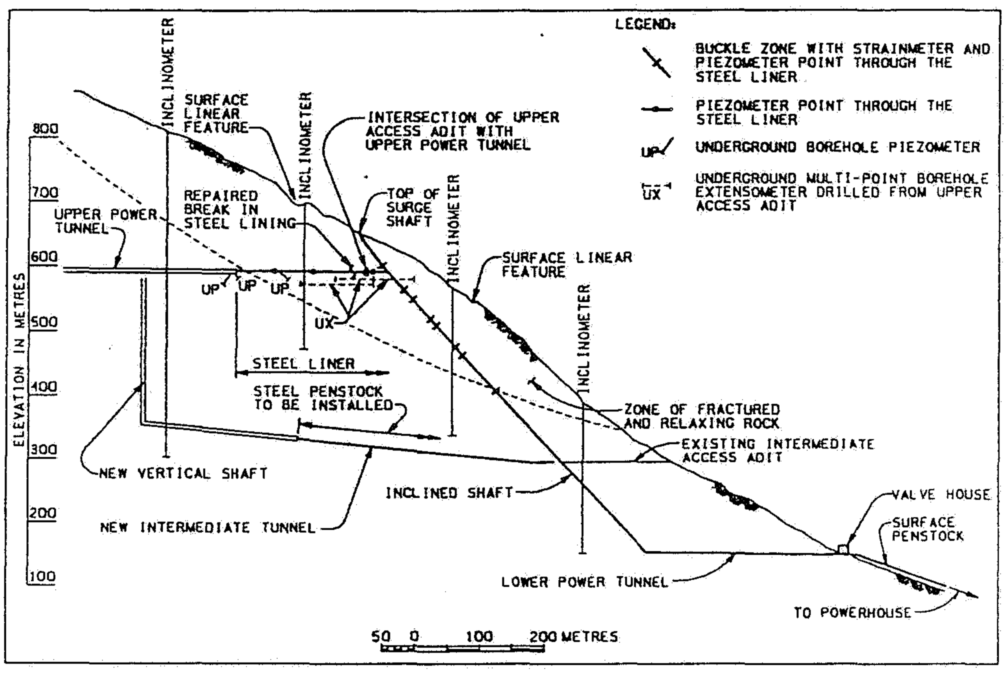


Figure 13: Wahleach Project: a) Layout. b) Cross-section looking west. The dashed line is the approximate location of a gradational boundary between loosened, fractured and weathered rock and more intact rock. Downslope movement - currently being monitored - is generally well above this boundary (from Moore *et al.*, 1991).

The ridge in which the tunnels are located is crossed by sackung. Old trees growing in the sackung features indicate a history of downslope movements spanning at least several hundred years.

On January 25, 1989, highway maintenance crews discovered water flowing from the upper access adit down the slope to the highway near the powerhouse. When the tunnels were dewatered, a 15 mm wide circumferential tension crack was found in the steel lining of the upper power tunnel about 30 m from its intersection with the inclined shaft (Fig. 13b). Subsequent investigations showed that this tension crack and eight, largely compressional, buckle zones in the shaft were the result of downslope creep of the rock slope, involving as much as 60×10^6 m^3 of rock. The creep movement was several centimetres per year at the surface during 1989-90 and appears to be ongoing. Savigny and Rinne (1991) suggested that the mechanism of deformation is toppling on the basis of numerical simulation of observed displacement magnitudes and directions.

The possibility of a large rockslide from this slope was a serious concern because of the presence of the Fraser River, the Trans Canada Highway, a railway, an oil pipeline, a fibre optic telephone line, and the powerhouse at the toe of the slope. After carrying out a very extensive investigation and monitoring program, B.C. Hydro in conjunction with their consultants, and with the agreement of Hydro's regulator, the Comptroller of Water Rights, decided that the portion of the power tunnel and shaft within the moving rock mass should be replaced (Ripley *et al.*, 1991) as shown in Figure 13b. Also, it was decided that the existing water conduit could be operated in the interim provided a predictive monitoring system and an emergency response plan were established.

The monitoring system required a major commitment of technical and financial resources by B.C. Hydro. The conduit was re-watered in September 1989 after an eight month shutdown and has been operating since then while the replacement conduit is being constructed (Baker, 1991; Tatchell, 1991). This replacement conduit is scheduled to be put into service in May 1992.

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km 19, LAIDLAW BLUFF ROCK SLIDE, by Michael Oliver

Laidlaw Bluffs are located approximately 18 km northeast of Cheam Slide along the Trans Canada Highway (Fig. 1). The orientation of jointing here, and along several other nearby highway cuts, is particularly adverse. Rough, irregular but pervasive joints dip out of the slope toward the highway at angles of 60 to 75. A smooth, near-vertical joint set, which in places causes overhangs, facilitates separation as sliding occurs along the inclined joint set. The cut was designed with a vertical backslope and wide ditch (4 m), and was constructed in 1985.

Instability was noted during construction, and as a consequence bolts, rock dowels and shotcrete were installed. Ongoing rockfall necessitated additional scaling and installation of mesh in 1987 and 1989. Small rockfalls were reported on November 22 and 28, 1991, prompting inspections which revealed long open tension cracks. Unstable zones were identified, and highways officials decided to implement a monitoring program. On December 6, 1991, access was established, ropes installed and installation of a surface monument began. This was completed on December 11, 1991.

A rock slide with an estimated volume of 6000 m^3 occurred at 0330 on December 12, 1991. The slide debris formed a fan approximately 8 m in height and 27 m in width which blocked the eastbound lanes of the Trans Canada Highway and extended into the westbound lanes. A westbound truck narrowly avoided the debris as the slide occurred by veering to the right. The operator lost control of the vehicle and proceeded down the embankment, coming to rest within 2 m of an eastbound train. The driver sustained only minor injuries and no other injuries were reported. The highway was closed and traffic routed along Highway 7.

Heavy rainfall was reported during the period from December 5 to 12, 1991, and freeze-thaw cycles occurred each of the five days prior to the slide. Rainfall was heavy at the time of the slide and vibrations from the passing train may have triggering it.

Stabilization work was required before the highway could be reopened. Scaling and isolated trim blasts were performed over a period of five days by highway crews before the clean-up was started. Additional scaling and trimming, and installation of bolts, mesh and horizontal drains were carried out through March 1992, with the eastbound lanes closed to traffic. The highway was restored to normal operation on March 23, 1992.

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km 26, take the exit marked Hunter Creek Road and St. Elmo Road. Turn right onto Hunter Creek Road, left at the T-intersection toward the rest area, and left into the rest area at km 27. Follow the tour guides instructions to the vantage point that affords a view northwest across Fraser River, then to the second point where Hunter Creek can be seen.

STOP 2 - KATZ SLIDE VIEWPOINT

Introduction

This stop has two purposes. First, it affords an overview of the Katz Slide located across the valley on the north side of the Fraser River. The tour will pass closer to the slide this afternoon on the return to Vancouver, but it will not be possible to stop the buses. Second, streams draining the mountain slopes to the south are prone to debris torrents. An unnamed stream a short distance ahead (northeast) along the Trans Canada Highway is the location of a recent destructive debris torrent which is discussed below (km 3, Flood). We will consider some of the details of this event before driving past the site, because there will not be an opportunity to stop.

Location and Background

Katz Slide is located on the northwest side of Fraser River, 35 km northeast of Chilliwack and 13 km southwest of Hope (Fig. 1). Large blocks of rock on the floodplain more than 1 km from the bottom of the mountain slope were long suspected of being landslide debris. The slide was confirmed in 1988 in the course of land-use studies (Thurber Consultants Ltd., 1988a). Naumann (1990) undertook a limited study to determine the age of the slide, but much work remains to be done before a definitive interpretation of the landslide hazard can be made and the attendant risks assessed.

Geological Setting of the Katz Slide Area

Bedrock in the source area is part of the Spuzzum Pluton (Fig. 2) of Late Cretaceous age, which here consists of quartz diorite interbedded with bands of dark micaceous schist (Thurber Consultants Ltd., 1988a; Monger, 1989). A regional fault, believed to be the northeast extension of the Vedder Fault, passes through the summit area of the unnamed mountain which is the source of the slide (Figs. 3 and 6; Savigny, 1992).

Source Area

The source areas of the Katz Slide are shown on Figure 14. The larger main area on the left is described below. Little is known about the area on the right.

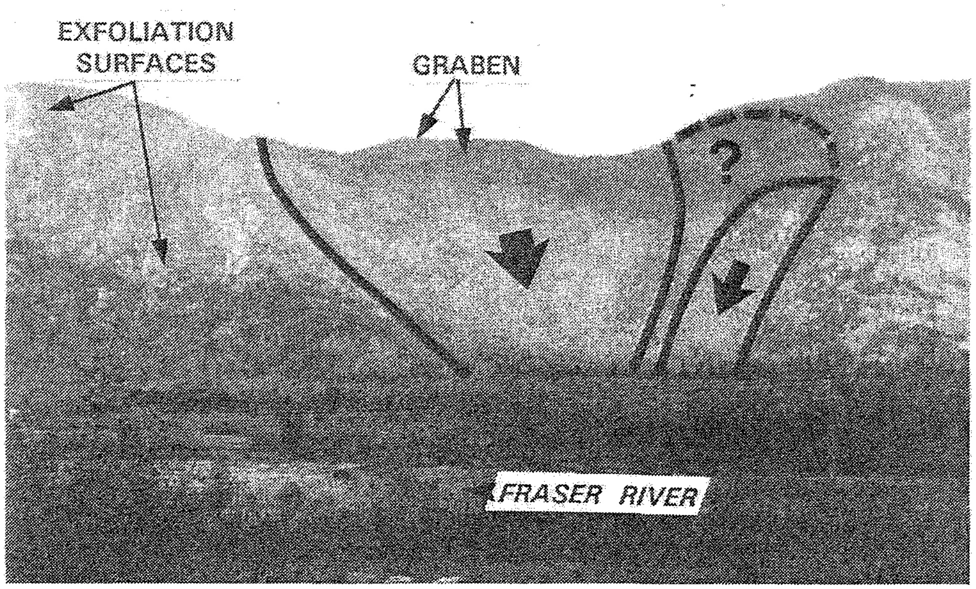

Figure 14: Katz Slide scars as viewed from Stop 2. The main scar is the large area on the left. Little is known about the smaller area on the right or the possibility that the two areas are connected by one headscarp. Fraser River is visible because the photograph was taken from a slightly higher vantage point than at the rest stop

The failure surface controlling the main source area (on the left, Fig. 14) appears to be planar and to dip toward the Fraser Valley. Planar exfoliation surfaces with similar orientations are prominent on both sides of the scar, but particularly on the left or southwest side. The surface of sliding is believed to have formed along exfoliation surfaces.

The mountain shows evidence of lateral spreading along the fault passing through the summit of the main source area. A large graben, approximately 120 m wide and at least 35 to 45 m deep, has formed along the spreading centre. This configuration, which is illustrated schematically in Figure 15, is similar to the model proposed by Beck (1968). How it may have contributed to the Katz Slide and how it should be considered in hazard and risk assessment are unclear.

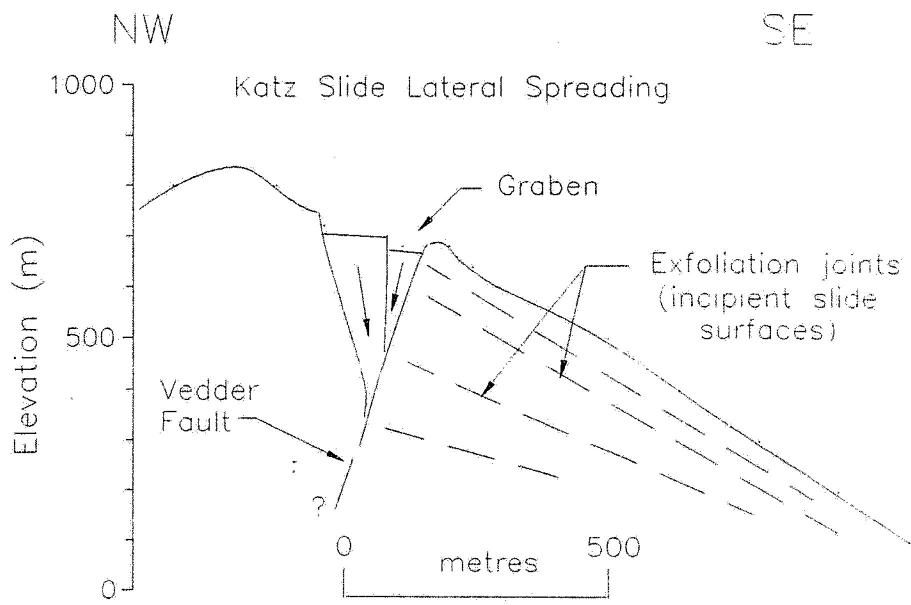

Figure 15: Schematic illustration of the lateral spreading model believed to be occurring at the graben in the headscarp area of Katz Slide (from Naumann, 1990; model from Beck, 1968).

Deposition Area

Airphoto interpretation, helicopter reconnaissance and field traverses in the depositional area provide evidence that the Katz Slide occurred as at least two rock avalanches separated by a period of hundreds to thousands of years (Savigny, 1992).

The first is believed to have extended across the Fraser Lowland as shown in Figure 16, creating a landslide dam and a small lake. A fan delta formed at the head of the lake and quickly prograded through it, eventually covering all but the largest blocks of the landslide dam. At the time of the second rock avalanche, broad, shallow channels carried the flow of Fraser River along the northwest (right) and southeast (left) sides of the valley (Fig. 16).

The second rock avalanche extended about half-way across the valley, blocked several channels on the northwest side, and diverted all flow to the southeast side (Fig. 16). Naumann (1990) cored at one of the abandoned northwest channels. An organic sample from near the base of the channel-fill sequence yielded a radiocarbon age of 3260±70 years BP (SFU W-02). This is a minimum age for the second rock avalanche.

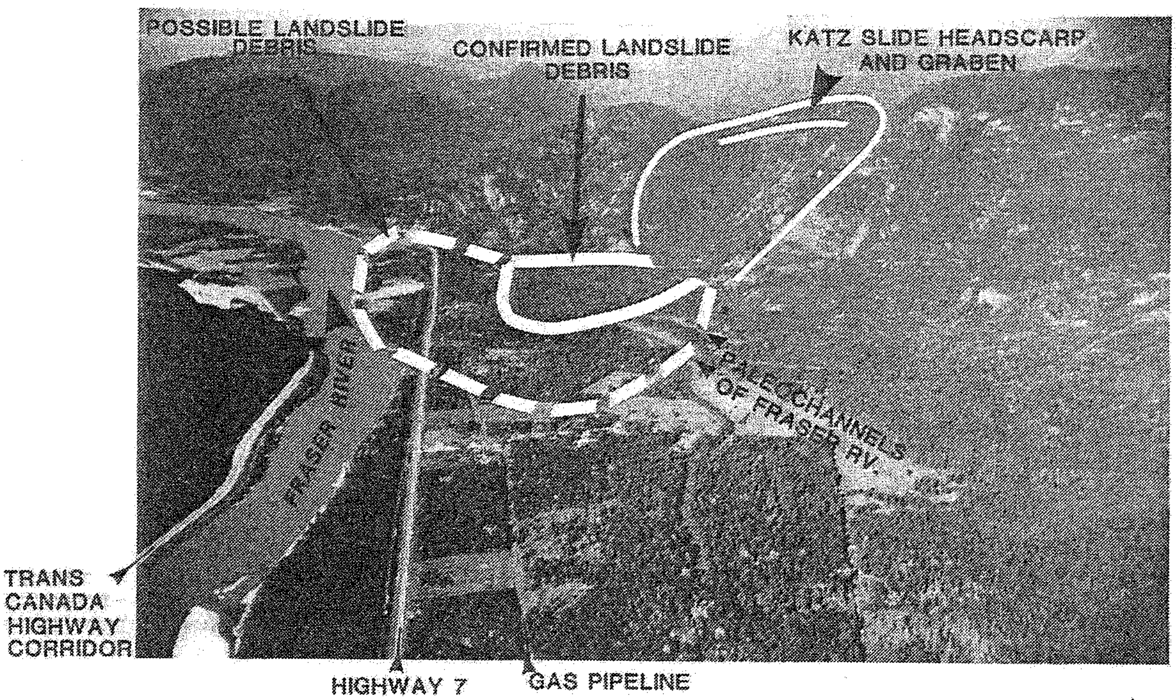

Figure 16: An oblique air photo looking downstream along the lower Fraser River valley transportation corridor in the vicinity of Katz Slide (from Savigny, 1992).

Slide Volume and Mobility

The first event was the larger, but no volume estimate has been made because almost all of the debris is buried. Slide debris from the second event covers an area of 1.1 km^2 and has a volume of about 15 x 10^6 m^3 (Thurber Consultants Ltd., 1988a).

The angle of inclination from the far edge of the slide debris emplaced during the second event to the top of the main slide scar is about 18.3. The calculated F value is 0.33 (see inset, Fig. 12). Like Cheam, this is consistent with the behaviour of other landslides in the Canadian Cordillera, and with the predicted mobility from the Scheidegger relationship (Scheidegger, 1973; Naumann, 1990).

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km 0, Oedometers should be reset before leaving the rest area. Return to the Trans Canada Highway access and and follow signs west to Hope.

km 3, DEBRIS TORRENT NEAR FLOOD, by Anthony H. Rice

An intense rainstorm in southwestern British Columbia between November 8 and 10, 1989, caused several debris torrents in the Hope area. One occurred near Flood, which is situated on the south (left) side of Fraser River approximately 9 km west of Hope (Fig. 1). Since 1939, as many as seven separate debris torrent/scour events have affected this site, and these are only a few of the many that have contributed to development of a large colluvial fan here.

The event described here occurred in the early morning hours of Friday, November 10, 1989. It originated in steep terrain south of the highway where peaks rise to elevations of between 750 and 1000 m. Heavy rainfall caused debris avalanches on steep vegetated colluvial slopes in the headwaters of the unnamed stream, and triggered a debris torrent which eroded additional bed and bank material as it descended. Depositional lobes developed in a few locations along the confined channel, but most deposition occurred on the fan. Debris initially came to rest just upslope of the Trans Mountain Pipe Line Company Ltd. right-of-way where a 610 mm diameter high pressure oil pipeline is located, but it was quickly remobilized due to a sharp drop immediately downstream of the right-of-way. Fill was scoured to an estimated depth of 3 m, leaving less than 1 m of cover on the buried pipeline. Much of the debris was deposited on the eastbound lanes of the Trans Canada Highway, closing the road between Flood and Hunter Creek (Stop 2) for three days. An estimated 5000 m^3 of debris, with blocks up to 3 m in size was transported during the event. Blocks up to 10 m have been transported during previous events.

Debris torrents like this are typically triggered by concentrated "cells" of high intensity precipitation which occur during storms with return periods of 2 to 5 years (Miles and Kellerhals, 1981). Daily precipitation values for the period November 7 to 11, 1989, for four meteorological stations in the vicinity of Hope are provided in Table I. The two closest stations are Jones Lake and Hope airport. Table II provides a preliminary analysis of return periods for the five-day storm. Return periods have not been calculated for the Jones Lake station since the length of the record is only six years, insufficient for statistical analysis. The cumulative precipitation at the Hope airport station up to the early morning hours of November 10, 1989, had a return period greater than 25 years. All other return periods are less than 25 years.

The Trans Mountain pipeline supplies almost all crude oil to the Lower Mainland refineries which produce gasoline and other products for the local market. The economic importance of this facility and the potential environmental implications of damage to the pipeline led Trans Mountain to undertake a project to deepen and protect the pipeline from inevitable future debris torrents. This work was completed during the summer of 1990.

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km 8, take exit right to Silverhope

km 9 to 10, SILVERHOPE AREA

The community of Silverhope is located along the Trans Canada Highway 3 km west of Hope (Fig. 1). The residential area is surrounded by, and built amongst, isolated large blocks,

Table I

Daily Precipitation Values Observed in the Vicinity of Hope, B.C.
(from Golder Associates, 1990)

Location	Station Operated By *1	24-hour Precipitation (mm) on November Data					Data *2 Collection Period
		7	8	9	10	11	
Abbotsford	AES	0.4	39.0	70.8	45.4	14.4	1
Jones (Wahleach) Lake	BCHPA	10.5	64.3	113.4	77.6	12.6	1
Chlliwack	AES	6.3	51.0	76.1	19.7	0.7	2
Hope Airport	AES	13.6	51.9	104.9	93.4	7.0	1

Notes: *1 AES is atmospheric Envir. Service. BCHPA is BC Hydro & Power Authority.
*2 1: 24-hour period starting at 22:00 hrs on the day preceeding date indicated.
2: Climate day starting at 08:00 on the day shown above.

as can be seen on the right (south) side of the bus from the highway. There is an obvious landslide hazard from the 400 to 1000 m high cliffs formed in conglomerate hornfels and quartz diorite which border the community. The runout of large boulders beyond the limits of talus deposits is confined within a narrow "rockfall shadow" band, conforming to an empirical criterion, a 27.5 angle projected from the apex of the talus cones (Hungr and Evans, 1989). The substrate of the rockfall shadow includes a deltaic sand terrace with foreset bedding, which must date from the end of glaciation as there was no standing water in this area during the Holocene.

A hazard study carried out for the Regional District of Fraser-Cheam used such observations, in addition to a computer rockfall dynamics model, to draw hazard boundaries and estimate probabilities. The study defined three hazard zones (Thurber Consultants Ltd., 1986). The first is a roughly 50-m-wide rockfall zone beyond the talus slope. This is estimated to have annual probabilities of rockfall occurrence of 1:100 or greater. A large rock fall in 1991 was contained on the talus slope. The second hazard zone relates to four potential landslide sources on the high, precipitous rock slopes. It is not possible to estimate probabilities of occurrence for these areas because no landslides of this type are believed to have occurred in the area since deglaciation about 11 000 years ago. The area of potential slide runout, however, is a designated caution zone, and extends roughly 500 m from the base of the slope. The third hazard zone is defined for debris flow/flood hazards along Silverhope Creek.

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km 11.5, stop light on highway immediately east of Hope

Table II

Frequency Analysis of Multi-Day Precipitation at Abbotsford, Chilliwack and Hope Airport (from Golder Associates, 1990)

Location	Number of Days	Cumulative Precipitation (mm)	Return Period (Years)
Abbotsford	1	70.8	< 5
	2	116.2	< 10
	3	155.2	< 25
	4	169.6	< 20
	5	170.0	< 10
Chilliwack	1	76.1	2 to 5
	2	127.1	5
	3	146.8	2 to 5
	4	153.1	< 2
	5	153.8	< 2
Hope Airport	1	104.9	< 10
	2	198.3	25
	3	250.2	25 to 50
	4	263.8	< 25
	5	270.8	10 to 25

* Preliminary data supplied by Earl Coatta and Colin di Cenzo, Scientific Servcies, Atmospheric Environment Services, Vancouver.
* Return periods for Chilliwack based on Coligado (1982); others provided by Earl Coatta.

km 11.8, follow signs to highways 5 and 3 to Merritt and Princeton

km 18, junction of Highway 5, continue on Highway 3 toward Princeton

km 29, turn left into Hope Slide viewpoint and rest area

STOP 3 - HOPE SLIDE

Introduction

On January 9, 1965, a large rock avalanche occurred in the north Cascade Mountains approximately 160 km east of Vancouver (Fig. 1). The Hope Slide involved more than 48 x 10^6 m^3 of rock, which descended the southwestern slope of Johnson Peak, inundated several kilometres of the Hope-Princeton transportation corridor (Figs. 17 and 18), buried three vehicles and claimed four lives. The reconstructed sequence of events begins with a snow avalanche at about 0400. Vehicles stopped by this obstruction were buried at 0700 by the rock avalanche. Two small earthquakes of local magnitudes (M_L) 3.2 and 3.1, were recorded at locations throughout southwestern British Columbia at 0356 and 0658, respectively. The highway was closed for 21 days while a temporary route was constructed atop the colluvium at the approximate location of the highway today (Fig. 17), but on the opposite side of the valley from the original highway.

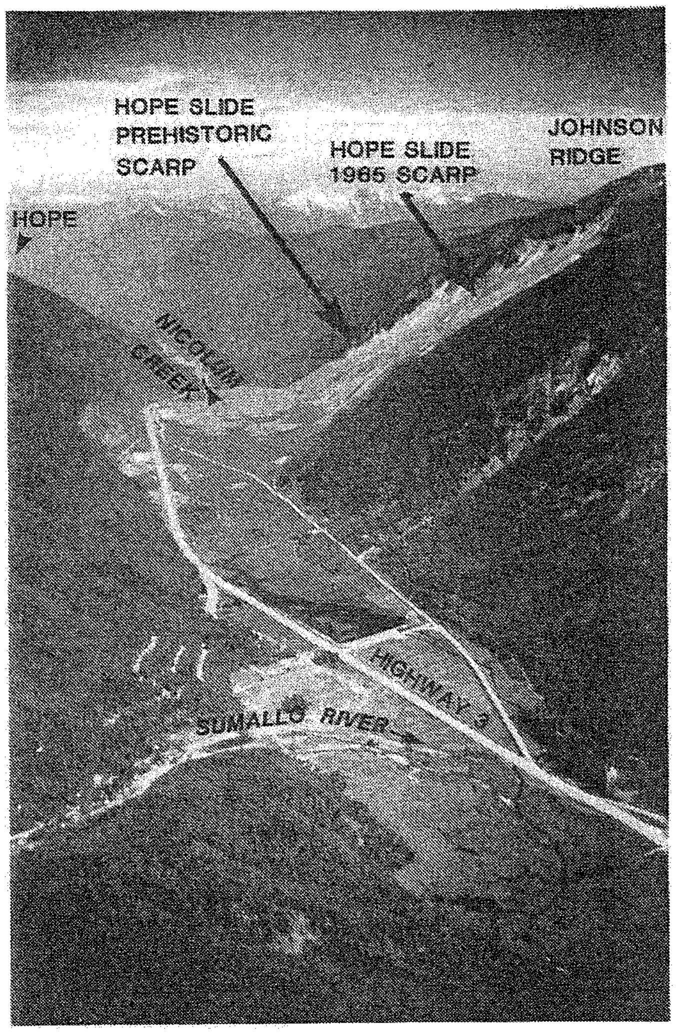

Figure 17: Hope Slide is the only historical rock avalanche in the vicinity of the lower Fraser River valley transportation corridor. On January 9, 1965, approximately 50 million cubic metres of rock and snow inundated the Nicolum-Sumallo valley, burying Provincial Highway 3. The slide occurred in the headscarp of a prehistoric slide of similar magnitude (from Savigny, 1992).

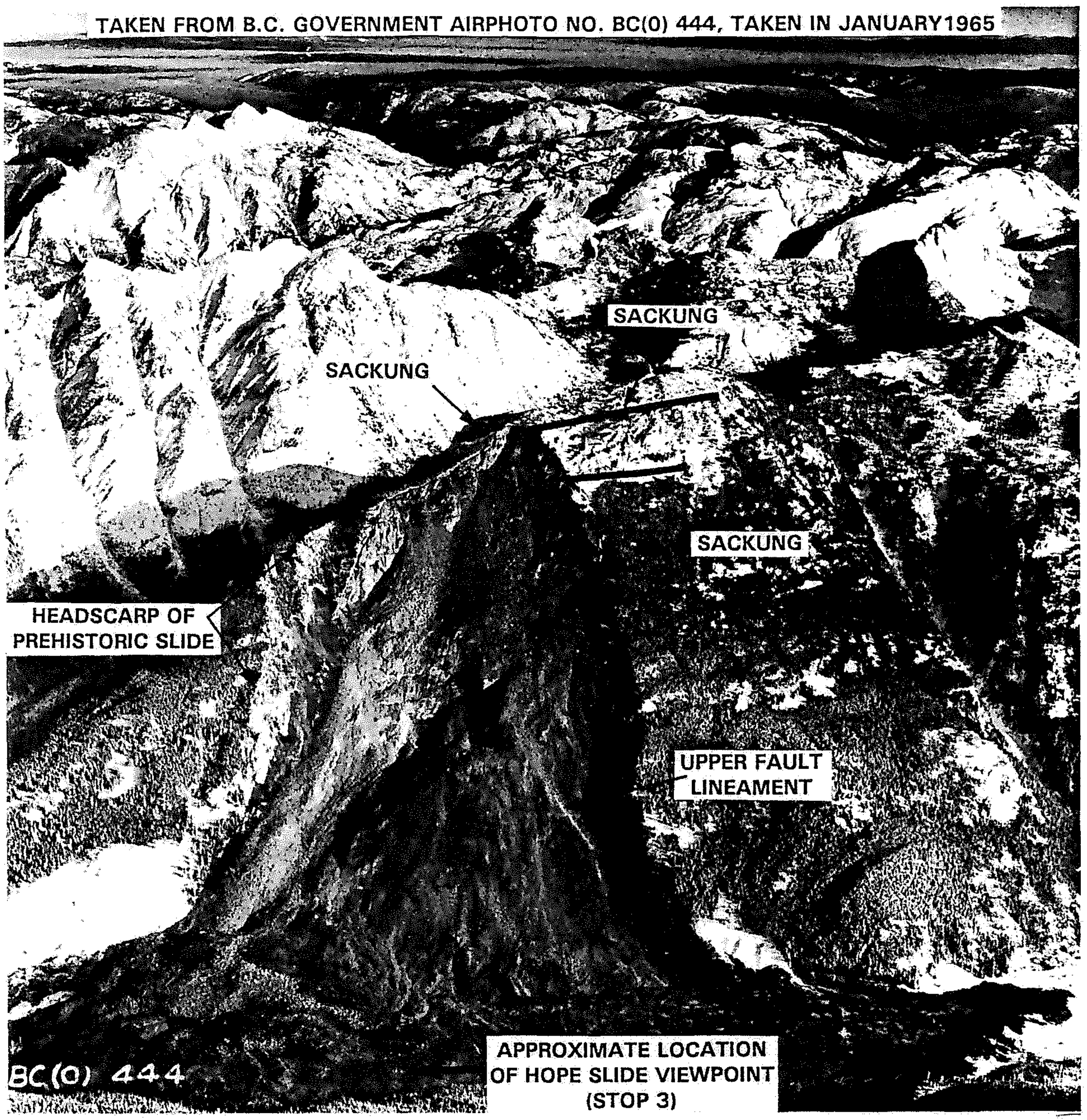

Figure 18: High level oblique aerial photograph of the 1965 Hope Slide, looking east.

An initial evaluation of the event was reported by Mathews and McTaggart (1969, 1978). The findings can be summarized as follows:

1. The rock avalanche involved greenstone and intrusive felsite sheets.
2. Pre-existing planes of weakness were located along or adjacent to the felsite sheets, which were roughly parallel to the 1965 rupture surface.
3. Four joint sets were found, three of which dip steeply.
4. The greenstone exhibits slight schistosity with local serpentinization along joints.
5. A landslide of comparable size occurred at the same location about 10 000 years ago (Figs. 17 and 18).

6. A small incipient landslide was identified north of the 1965 slide scar.
7. Seasonal, sub-zero temperatures were recorded for several days prior to the event.
8. The factor of safety was near unity, and the small earthquakes triggered the failure.

Wetmiller and Evans (1989) re-evaluated the seismograph data and concluded that a seismic trigger could not be confirmed or discounted. Weichert *et al.* (1990) proposed that the slide may not have been triggered by the two earthquakes, but rather two rock avalanches occurred and induced seismic shocks equivalent to the two recorded earthquakes. Work by von Sacken (1991) and von Sacken *et al.* (1992) lends support to this theory. These authors concluded that the upper and lower portions of the 1965 failure surface were controlled by different mechanisms, providing a possible explanation for two different events. Moreover, back analysis demonstrated that failure of the lower portion would leave the upper portion at a critical factor of safety. The following brief summary of the 1965 Hope Slide is based on the recent work of von Sacken (1991) and von Sacken *et al.* (1992).

Geological setting of the Hope Slide area

Rock Units

The geology of the 1965 Hope Slide is shown in Figure 19. Greenstone which underlies most of the slide area belongs to the Hozameen Complex of Permian to Jurassic age. Outcrops are widespread at the headscarp, along the mountain ridge to the south, and along the northern flank of the slide scar. Felsite intrusions occur in sheets within the greenstone; contacts are commonly marked by a weathered gouge-filled surface. X-ray diffraction analysis of the gouge shows it is made up of clinochlore (a chlorite mineral) and fibrous actinolite. The sheets, which are commonly subparallel to the slope, are most prevalent along the northern part of the slide scarp (left looking upslope) and in exposures near the centre of the detachment surface. Their age is unknown. Granodiorite of the Eocene Mount Outram Pluton outcrops a short distance southeast of the headscarp and is believed to occur at depth below Johnson Ridge (Fig. 2; von Sacken, 1991).

Structure

Three major joint sets are present in each of two domains in the slide area. The orientations of joints in Domain 1, the headscarp and ridge area (Fig. 19), are shown in Figure 20a. The orientations of joints in Domain 2, the central detachment surface and precipitous cliffs on the north side of the slide, are shown in Figure 20b. These orientations are somewhat different (von Sacken, 1991).

Two conspicuous faults, termed the upper and lower faults (UF and LF, respectively), cut across the detachment surface (Fig. 19). The most conspicuous is the UF which strikes between 30 and 35 and has a near vertical dip. It is well exposed at the base of the precipitous northern slide margin where highly slickensided gouge, up to 0.25 m thick, was found. X-ray diffraction analysis showed the clay gouge to be made up of clinochlore, kaolinite, montmorillonite and actinolite. The LF is less obvious and is identified on the basis of a prominent north-trending lineament (von Sacken, 1991).

A number of large open tension cracks have been found near the headscarp of the Hope Slide. These are subparallel to two of the joint sets. Movement hubs have been installed across

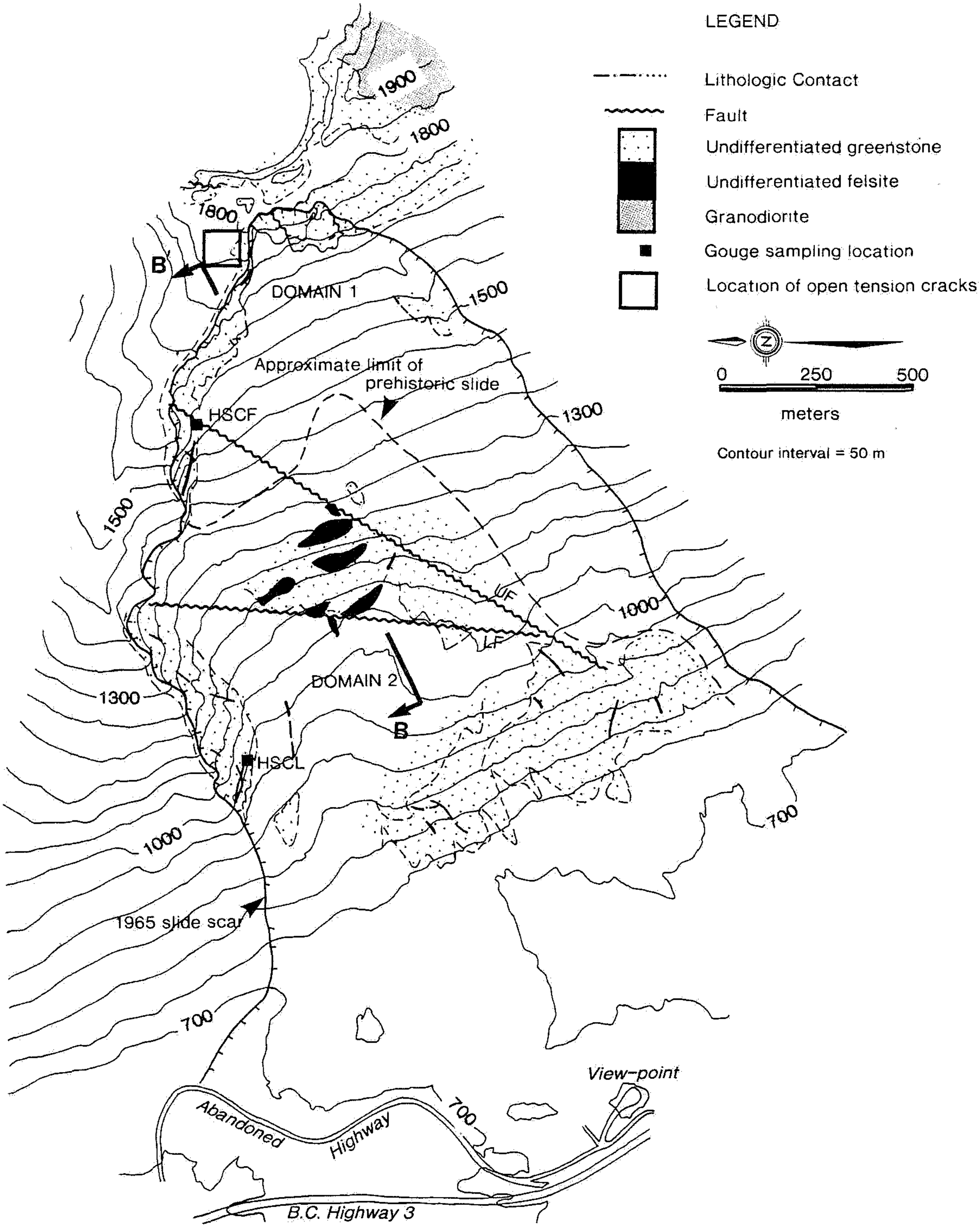

Figure 19: Geology of the Hope Slide area (UF - upper fault; LF - lower fault; HSCF - fault gouge sample location; HSCL - lithotectonic gouge sample location). See general profile in Figure 22 and cross-section BB' in Figure 23 (from von Sacken *et al.*, 1992).

these and regular measurements are being taken. The cracks are at the same elevation as prominent sackung features that can be traced along contour lines several kilometres to the southeast (Fig. 18), where they merge with moraines believed to mark the maximum local limit of late Wisconsin glaciation.

Geological Control of the 1965 Failure Surface

The failure surface in Domain 1 is controlled by joints. J1 is the only set that could have facilitated translational sliding (Fig. 20a). Although the mean orientation of J1 does not daylight on the upper slope, approximately 10% of the J1 surfaces measured have dip angles that are shallower than the slope, with the same mean strike (JS on Fig. 19a). A profile through the headscarp area (Fig. 21) shows the stair-step configuration created by the juxtaposition of these two surfaces (von Sacken, 1991; von Sacken *et al.*, 1992).

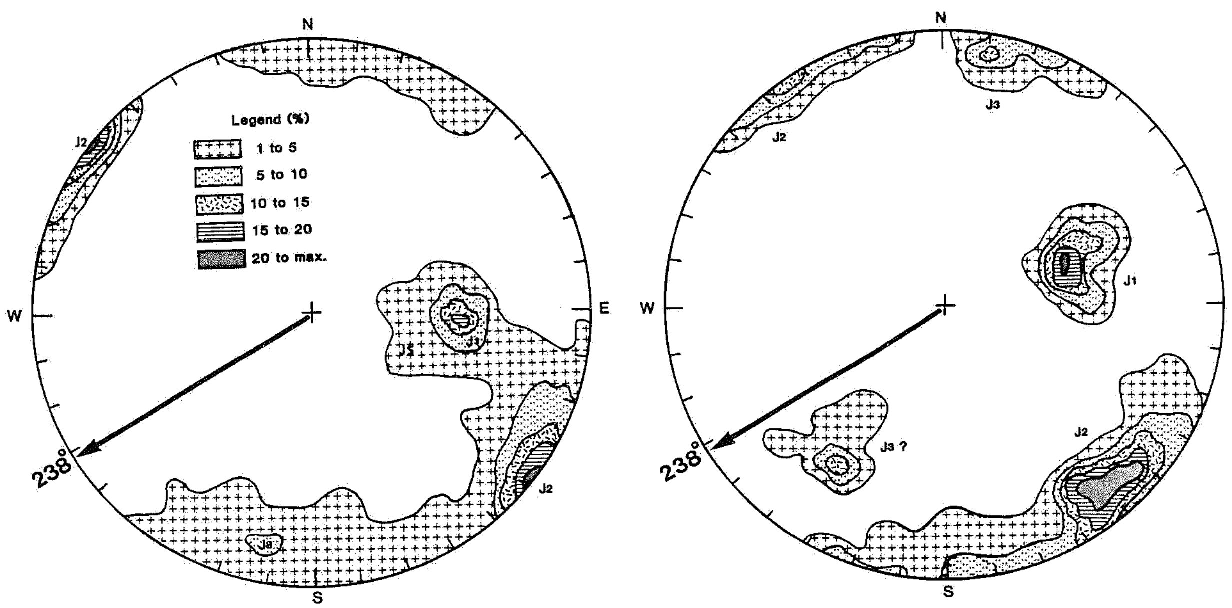

Figure 20: Lower hemisphere projections of the three major joint sets, J1, J2, and J3 in a) Domain 1 (1910 data points; from von Sacken *et al.*, 1992), and b) Domain 2 (5904 data points; from von Sacken, 1991), on an equal area stereonet and a 1% counting area. See Figure 19 for location of domains.

The failure surface in Domain 2 is controlled by the felsite sheets (von Sacken, 1991; von Sacken *et al.*, 1992).

Back Analysis

Pre- and post-1965 topographic profiles are displayed in Figure 22. Several cross-sections of the slope were analyzed by von Sacken (1991); the worst case scenario is shown in

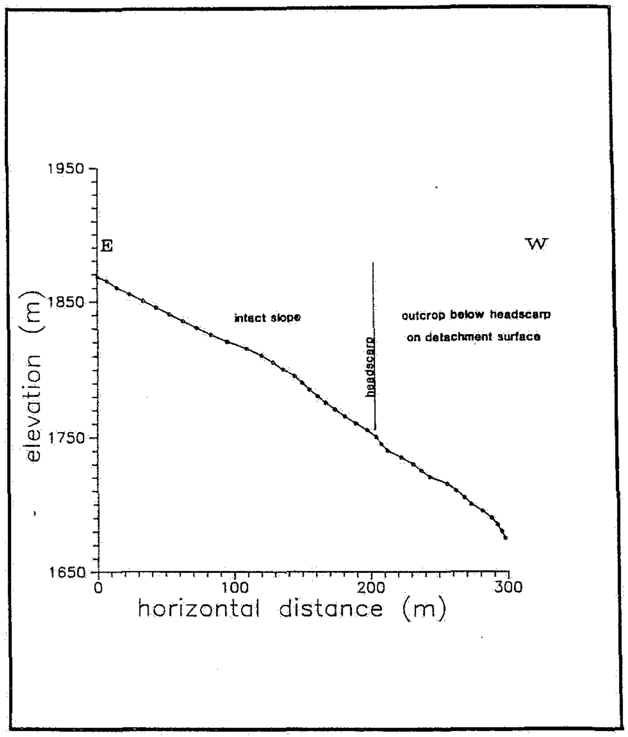

Figure 21: Profile showing the step-like failure surface in Domain 1. The dip angles of the steeper and shallower surfaces range from 42 to 51 and 20 to 31, respectively, based on 1:1000 topographic map (from von Sacken, 1991).

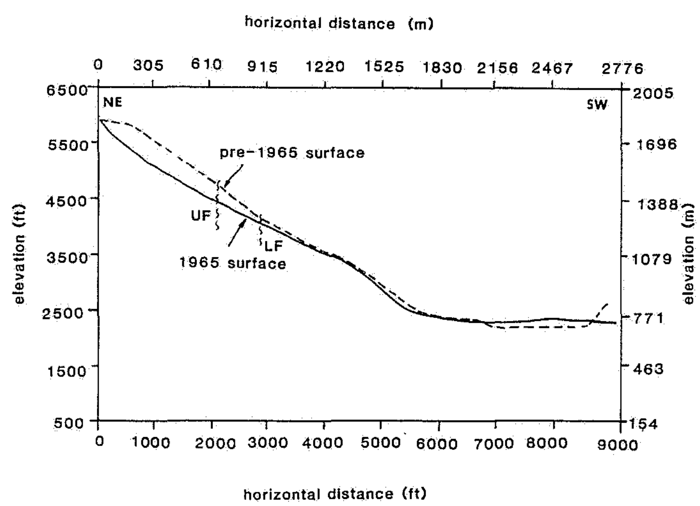

Figure 22: Pre- and post-1965 profiles based on 1965 topographic maps, taken parallel to the direction of longest runout (238, Fig. 19). Note minimal rock was removed in the lower slope (below the lower fault - LF). Most came from the upper slope (above the upper fault - UF; from von Sacken *et al.*, 1992).

Figure 23. The slope is divided into two parts at the prominent UF (Fig. 23). The stability of the upper part is controlled by the strength of greenstone-greenstone contacts across joint surfaces, corrected for field roughness. The stability of the lower part is controlled, to an extent, by similar contacts, but also by the contacts between felsite sheets and greenstone. Stability analyses were performed for two conditions, with and without gouge infill along the felsite-greenstone contacts. Although many assumptions were necessary, the results leave little doubt that the existence of gouge along the felsite-greenstone contacts is the single most important factor in reducing the stability of the slope to a critical level.

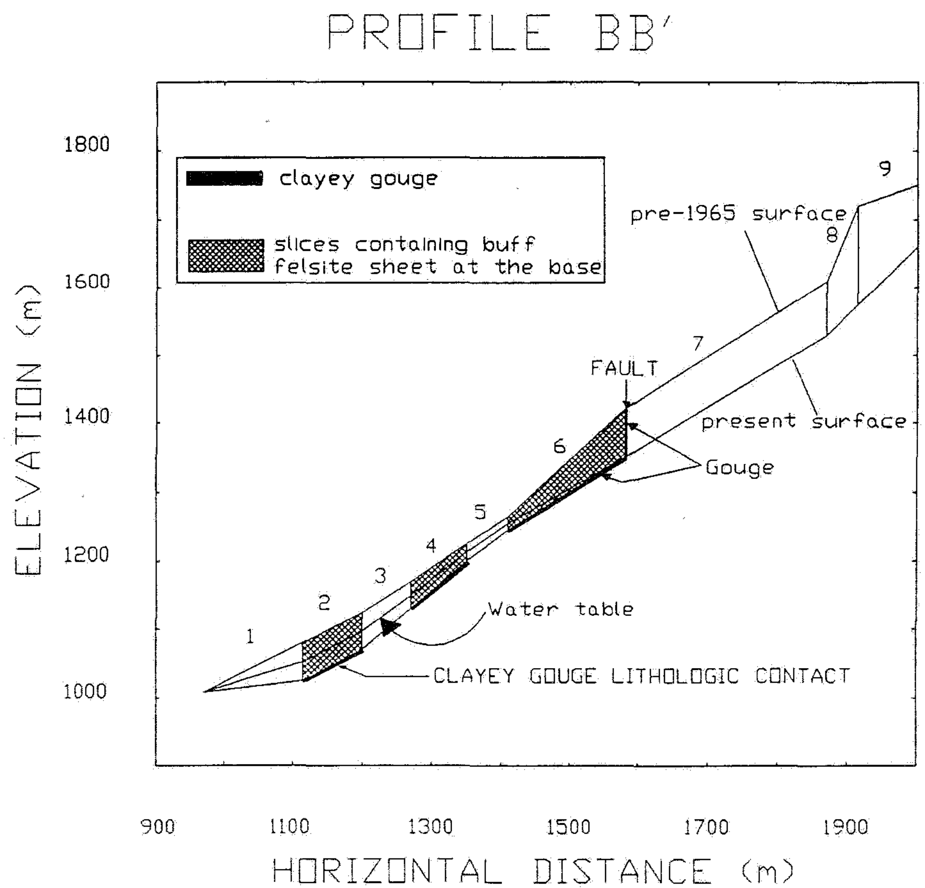

Figure 23: Cross-section BB' used in back analysis (from von Sacken, 1991).

Possibility of Two Separate Slide Events

Keefer (1984) studied 40 earthquake-induced landslides throughout the world, and found that none, with the exception of the Hope Slide, was triggered by an earthquake of magnitude less than 4. The hypothesis of Weichert *et al.* (1990) that two rock avalanches may have induced the seismic shocks, together with von Sacken's (1991) and von Sacken et al.s' (1992) recognition of two distinctive domains caused the latter authors to analyze the stability of the upper portion of the slope, above the fault in Figure 23, with the lower portion removed. Under these conditions, the upper portion of the slope had a critical factor of safety.

On the basis of the detailed geological mapping and back analysis, von Sacken (1991) proposed the following revised account of events for the early morning hours of January 9, 1965. The first blockage of the highway at about 0400 was caused by a rockslide from the lower portion of the slope (Fig. 18). Detachment occurred along a failure surface controlled in part by the litho-tectonic contacts between felsite and greenstone and extending as far upslope as the UF (Figs. 18 and 23). Because it was dark, a rockslide could have easily been mistaken

for a snow avalanche. Oversteepening of the toe coupled with pervasive jointing and cracking at the top of the ridge caused the failure of the upper portion of the ridge at approximately 0700.

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km 0, Oedometers should be reset before leaving the Hope Slide viewpoint and rest area. Turn right onto Highway 3 toward Hope.

km 10, ROCKSLIDES ON MOUNT HOPE

Two rockslide scars can be seen on Mount Hope directly ahead of the bus. These slides occurred in 1979. Rubbly debris reached the old highway bridge and partially blocked the span.

km 11, junction of Highway 5, continue toward Hope

km 14, take exit 173 to Hope

km 16, keep right following the sign to Hope town centre

km 17, turn left following the sign to Hope town centre

km 17.7, turn right at T-intersection following the sign to Highway 1 east

km 18.6, bridge across Fraser River

km 20, junction of Highway 7, continue on Highway 1 north toward Yale

km 22.6, Lake-of-the-Woods Slide. The tour stops here on the return to Vancouver.

km 43, pull off to the right before entering the Yale Tunnel

STOP 4 - YALE SLIDE VIEWPOINT

Introduction

The community of Yale is situated on the west side of Fraser River approximately 25 km upstream of Hope. Ahead is the Fraser Canyon, a deeply insised, narrow valley which extends 70 km to the north. This is the most confined reach of the Fraser Valley transportation corridor, hence the most vulnerable to landslides.

Yale originated as a placer gold mining centre in the last century. At the peak of mining activity in 1858 more than 30 alluvial bars and terraces along Fraser River between Hope and Yale were mined by 9000 men. The town was the upstream limit of river boat navigation for men and supplies destined for the Caribou gold fields in central British Columbia later in the 1800s. The Canadian Pacific Railway was built in the 1880s, the Canadian National Railway was constructed in the early part of this century, and the Trans Canada Highway was built in the 1960s. Presently, there are also power transmission lines located in the Canyon, and the river itself has a major salmon run. This concentration of facilities and the importance of the river to the west coast fishing industry make the economy of the whole Lower Mainland region particularly vulnerable to landslides in the canyon.

There are two stops in close proximity. Stop 4 provides a panorama of the Fraser Canyon and is the best point to view the suspected Yale Slide scar. After a few minutes the tour will move to Stop 5, located below and a short distance upstream. Stop 5 affords an appreciation for the confinement of the Canyon and the unavoidable interaction of linear facilities and the river.

Yale Slide

Figure 24 shows the suspected Yale Slide (Savigny, 1992). One continuous headscarp defines a large slide area situated immediately west of the Hope Fault (Fig. 3) The headscarp shows evidence of north-northeast trending open cracks and displaced scarps. The slide area can be divided into north and south parts. The south part is believed to be the scar of an ancient rockslide that failed catastrophically into Fraser River, probably creating a short-lived landslide dam and lake. The north part translated as a rock block slide or rock slump until it encountered

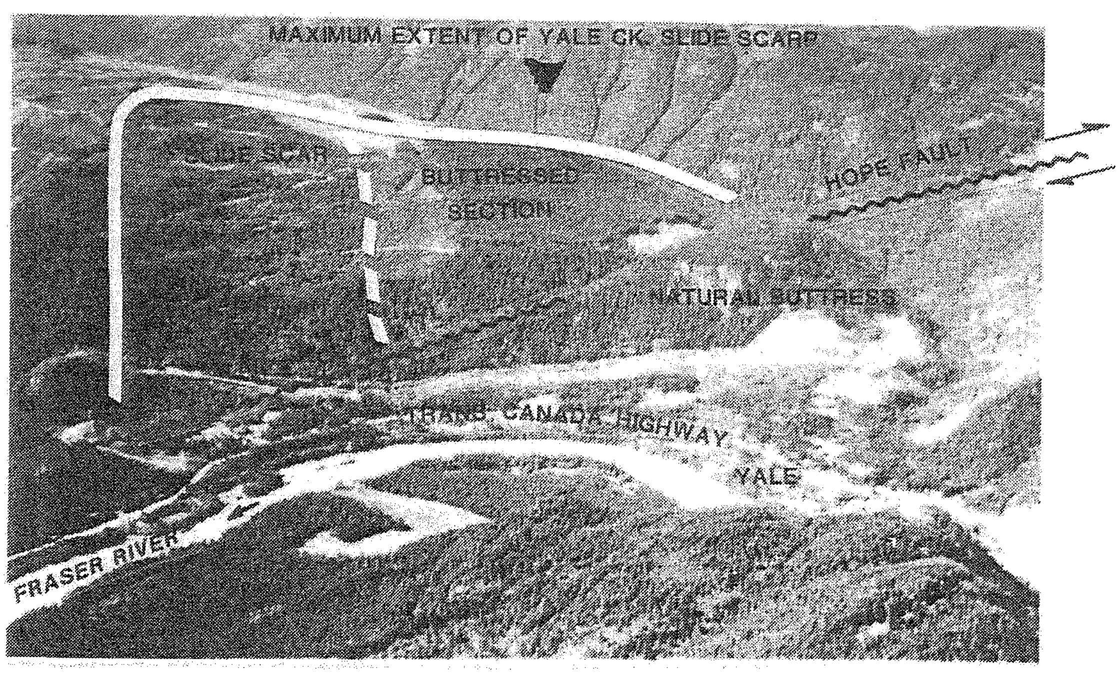

Figure 24: Low level oblique aerial photograph of the suspected Yale Slide (from Savigny, 1992).

a natural buttress on the east side of the Hope Fault. The buttress apparently prevented catastrophic failure. A small amount of colluvium is present on the valley floor below the scar. It consists of high grade schistose and ultramafic debris which is expected since the Settler Schist outcrops in the source area, but it also includes granodiorite clasts of unknown origin. It cannot be said with certainty that this colluvium is landslide debris. Movement of the north and south parts is inferred to be late glacial or Holocene because the north flank has not been modified by Late Wisconsin glacial erosion which cut deeply into the Hope Fault shear zone nearby.

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km **0, Oedometers should be reset before leaving Stop 5. Turn left onto Highway 1 toward Hope.**

km **0.6, turn left and immediately cross CP Rail tracks, continue along old highway**

km **1.9, This stop is near the end of a now-abandoned section of the old highway which was used before the Trans Canada Highway was constructed. The buses will be parked on the river side of the road between two CP Rail tunnels. <u>Under no circumstances should delegates approach the CP Rail tracks!</u> Trains frequently pass this point at high speed and, because of the tunnels, there is little warning of their approach. The best viewpoint is reached by continuing on foot up the hill and around the corner to the end of the road, approximately 150 m from where the buses are parked.**

STOP 6 - OLD HIGHWAY, EAST OF YALE, by Duncan Wyllie

Introduction

All of the linear facilities in this section of the Fraser Canyon, the river, and examples of the many landslide hazards are visible from this vantage point. Looking up the steep slope from the parking area, the Trans Canada Highway can be seen above the CP Rail grade. The CN Rail grade and a major B.C. Hydro power grid are visible on the opposite side of Fraser River which flows swiftly past the toes of both slopes. The tour guides will point out examples of debris avalanches, debris torrents, rockfalls, colluvial cones and aprons, and several examples of recessive weathering along fault zones.

Engineering Geology

Geology

The major rock type in the Fraser Canyon is a very strong granite. The rock is jointed, with joints generally being planar, oriented in orthogonal sets, and ranging from about 0.5 m to as much as 10 m in length. Excavations are generally stable, but rock bolting is required where potentially unstable blocks are formed by natural and blast-induced fractures. Installation of major support is limited to faulted and weathered zones.

Slope Stability Conditions

The construction of the highway and railways required the excavation of a large number of rock cuts and tunnels, most of which are unsupported. Blasting was required for almost all the rock excavation, and the blasting methods used often resulted in substantial overbreak and shattering of the rock behind the face. As a result, most of the rock in exposed faces is highly fractured. The combination of the shattered rock and the weather results in loosening and failure of blocks on the faces. Most of these failures occur in the spring and fall when precipitation levels are high and freeze/thaw cycles occur. Figure 25 shows one such failure which occurred in November 1990 following a period of heavy rain.

There have, unfortunately, been a number of accidents on both the railways and the highway as the result of rock falls. The consequences of these falls range from minor damage to vehicles, to injuries and, occasionally, death. As a result of these hazards, systematic rock slope and tunnel stabilization programs have been implemented over about the past 20 years. The objective of these programs has been to identify potentially hazardous sites and then carry out stabilization work before accidents occur. This is termed a "proactive" approach, in contrast to a "reactive" approach in which work is carried out after a rock fall has occurred.

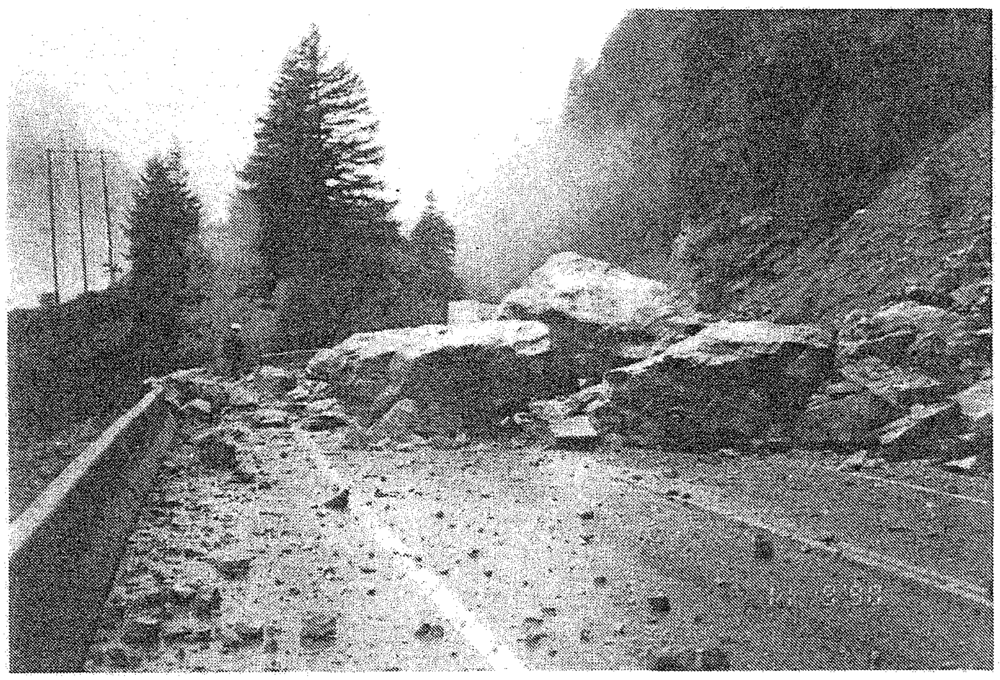


Figure 25: Highway blocked by rockfall in very strong, massive granite that originated from approximately 300 m above the highway and destroyed a 3 m high concrete wall.

Inventory of Slope Stability Conditions

There are many hundreds of rock cuts along the Fraser Canyon with heights ranging from a few metres to as much as 120 m. The rockfall hazard on these slopes is highly variable, depending not only on slope geometry and geological conditions, but also on the presence of a catch ditch at the toe of the cut, and the ability of drivers to observe and avoid rockfalls.

There are obviously insufficient funds to stabilize all the rock slopes in the Canyon. However, by concentrating on the most hazardous slopes a significant improvement in safety can be achieved, making the most efficient use of the available funds. In order to identify the most hazardous slopes, an inventory system has been developed. This system involves the assignment of point scores in the range 3 to 81 to eleven categories describing the condition of the slopes (Table III). By adding the point scores for each cut it is possible to rank the slopes, the most hazardous being the one with highest total score. Once the inventory has been completed, it is possible to schedule stabilization work; those slopes with the highest hazard rating are stabilized early in the program. The inventory system shown in Table III was developed for CP Rail in the early 1970s and is now being used on highways in many U.S. States (Brawner and Wyllie, 1975; Wyllie, 1987; Pierson *et al.*, 1990).

Stabilization Measures

Stabilization measures for rock slopes can be divided into three classes as shown in Figure 26 (Wyllie, 1991).

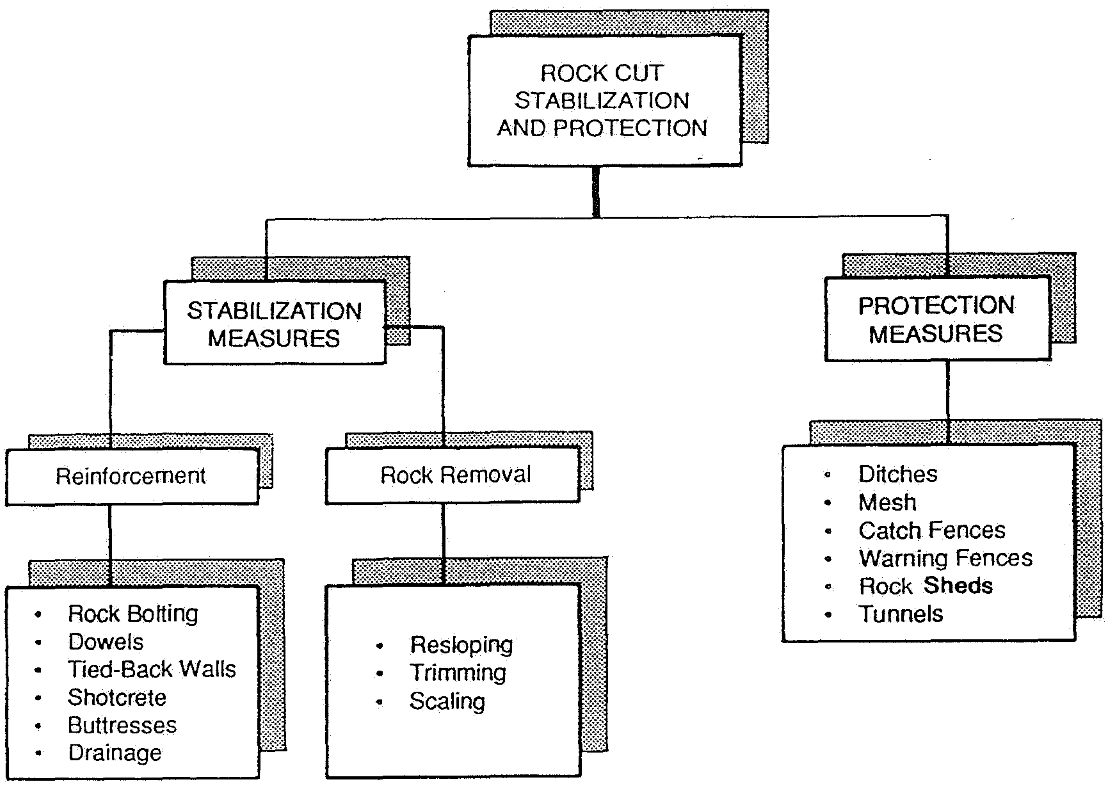


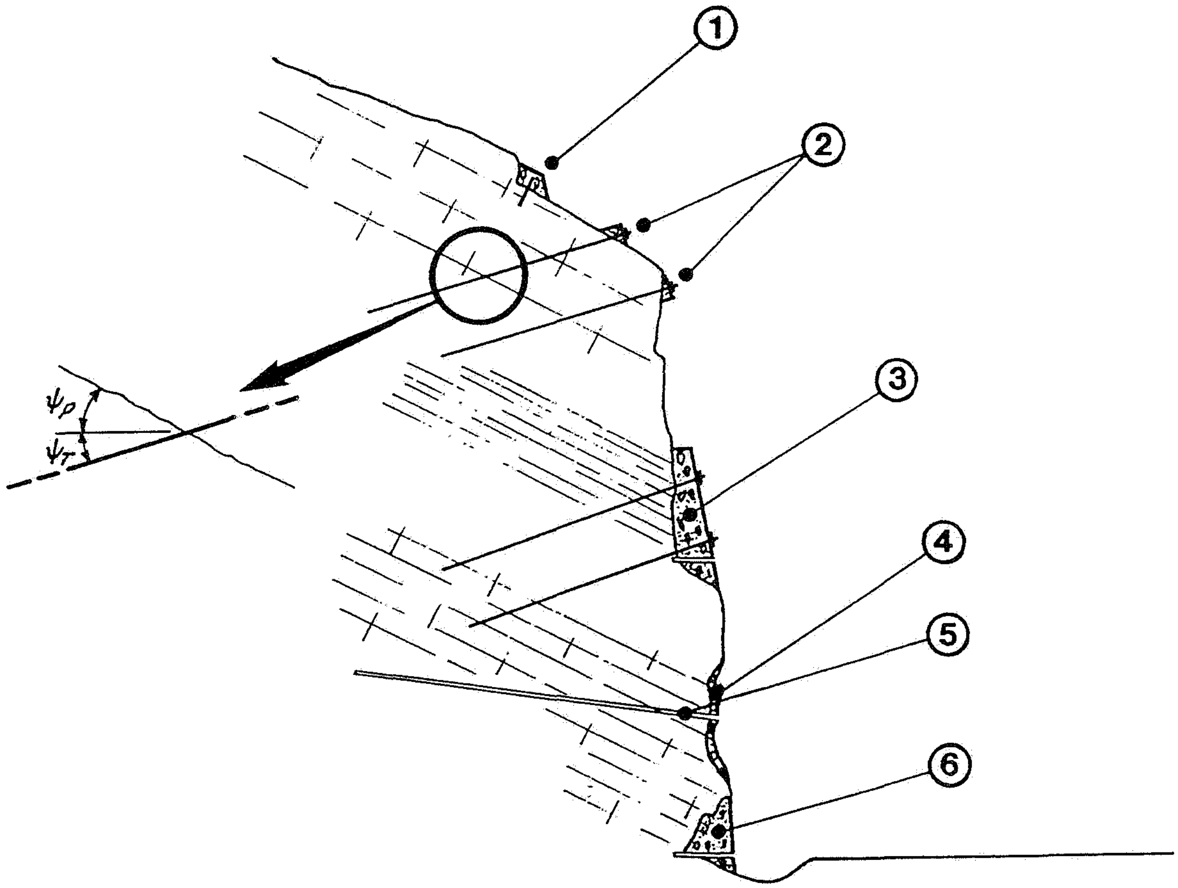
Figure 26: Classes of rock slope stabilization measures.

Rock Reinforcement

A number of reinforcement techniques may be implemented to secure potentially loose rock on a face (Fig. 27). All of these techniques minimize relaxation and loosening of the rock mass which may take place when rock fractures open. Once relaxation has taken place, there is a reduced contact between blocks, and a significant decrease in shear strength.

Table III
Rockfall Hazard Rating System

CATEGORY	RATING CRITERIA AND SCORE			
	POINTS 3	POINTS 9	POINTS 27	POINTS 81
SLOPE HEIGHT	25 FT	50 FT	75 FT	100 FT
DITCH EFFECTIVENESS	Good catchment	Moderate catchment	Limited catchment	No catchment
AVERAGE VEHICLE RISK	25% of the time	50% of the time	75% of the time	100% of the time
PERCENT OF DECISION SITE DISTANCE	Adequate site distance, 100% of low design value	Moderate site distance, 80% of low design value	Limited site distance 60% of low design value	Very limited site distance 40% of low design value
ROADWAY WIDTH INCLUDING PAVED SHOULDERS	44 feet	36 feet	28 feet	20 feet
GEOLOGIC CHARACTER — CASE 1 — STRUCTURAL CONDITION	Discontinuous joints, favorable orientation	Discontinuous joints, random orientation	Discontinuous joints, adverse orientation	Continuous joints, adverse orientation
GEOLOGIC CHARACTER — CASE 1 — ROCK FRICTION	Rough, Irregular	Undulating	Planar	Clay infilling, or slickensided
GEOLOGIC CHARACTER — CASE 2 — STRUCTURAL CONDITION	Few differential erosion features	Occasional erosion features	Many erosion features	Major erosion features
GEOLOGIC CHARACTER — CASE 2 — DIFFERENCE IN EROSION RATES	Small difference	Moderate difference	Large difference	Extreme difference
BLOCK SIZE QUANTITY OF ROCKFALL/EVENT	1 FT 3 cubic yards	2 FT 6 cubic yards	3 FT 9 cubic yards	4 FT 12 cubic yards
CLIMATE AND PRESENCE OF WATER ON SLOPE	Low to moderate precipitation; no freezing periods; no water on slope	Moderate precipitation or short freezing periods or intermittent water on slope	High precipitation or long freezing periods or continual water on slope	High precipitation and long freezing periods or continual water on slope and long freezing periods
ROCKFALL HISTORY	Few falls	Occasional falls	Many falls	Constant falls



1. Reinforced concrete dowel to prevent loosening of slab at crest.

2. Tensioned rock anchors to secure planar type failure along crest.

3. Tied back wall to prevent sliding failure on fault zone.

4. Shotcrete to prevent ravelling of zone of fractured rock.

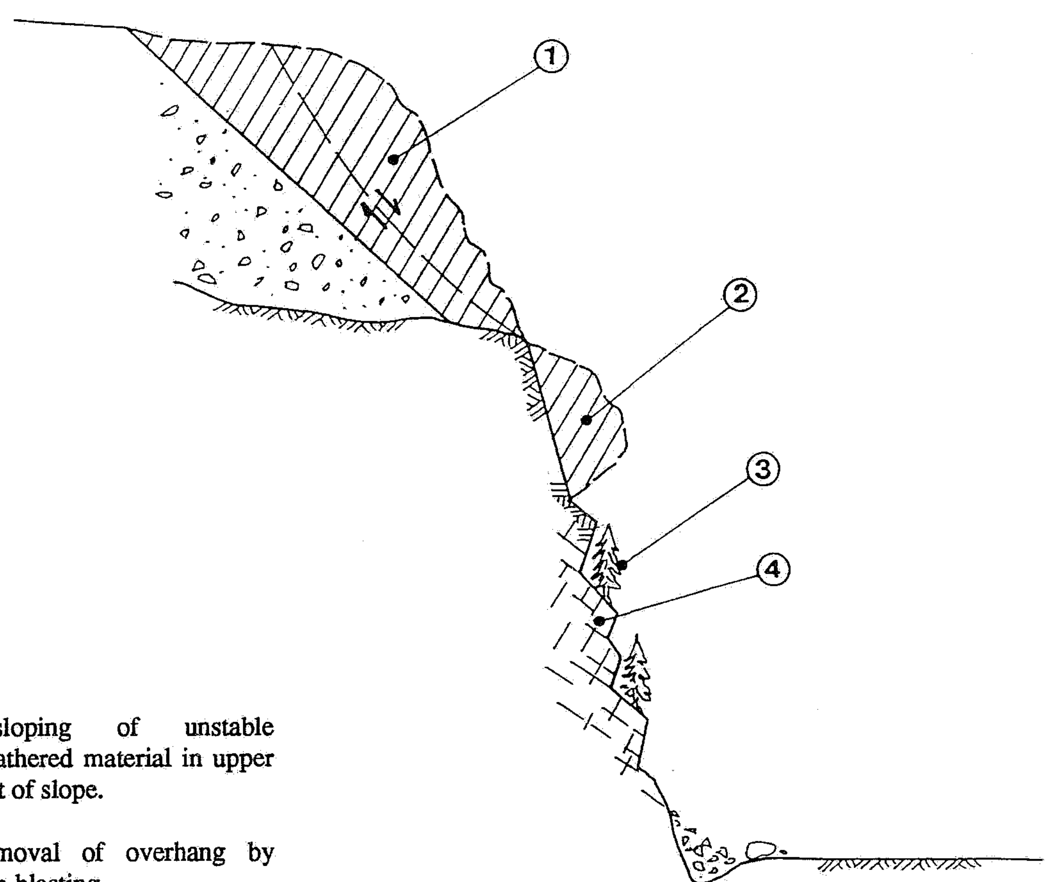
5. Drain hole to reduce water pressure within slope.

6. Concrete buttress to support rock above cavity.

Figure 27: Rock slope reinforcement methods.

Rock Removal

Stabilization of rock slopes can be accomplished by removing potentially unstable rock. Figure 28 illustrates typical removal methods; resloping zones of low strength rock, trim blasting of overhangs, and scaling of individual blocks of rock.



1. Resloping of unstable weathered material in upper part of slope.

2. Removal of overhang by trim blasting.

3. Removal of trees with roots growing in cracks on face.

4. Hand scaling of loose blocks in shattered rock.

Figure 28: Rock removal methods for slope stabilization.

The advantages of removing unstable rock, in comparison to installing tensioned rock anchors for example, is that removal can be a permanent stabilization measure and the construction work is usually less expensive. However, removal of loose rock on the face of a slope will only be effective if there is no risk of undermining the upper part of the slope and if the rock forming the new face is sound. An example of where rock removal should be carried out with care is area 4 in Figure 28. It would be safe to remove the outermost loose rock, provided that the fracturing was caused by blasting and only extends to shallow depth. However, if the rock mass is deeply fractured, continued scaling would soon develop a cavity that would undermine the upper part of the slope. The removal of loose rock on the face of a slope is also ineffective where the rock is incompetent or highly weathered. In these circumstances, the exposure of a new face will start a new cycle of weathering and instability. For both of the conditions described above, a more appropriate stabilization method would be protection with shotcrete, if adequate adhesion to the face can be achieved, or excavation of a ditch at the toe of the slope.

Protection Measures

Protection can be provided by a variety of structures placed on and at the toe of the slope. These structures include catchment ditches, mesh, fences and rock sheds. A common feature of all these protection structures is their energy-absorbing characteristics: the rockfall is either stopped over some distance or deflected away from the facility that is being protected. It is possible, by using appropriate techniques, to control rocks as much as 2 to 3 m in diameter, falling from heights of several hundred metres. In contrast, rigid structures, such as reinforced concrete walls, are rarely effective for catching rockfalls; the rock fall in Figure 25 completely destroyed a 3 m high concrete wall.

Figure 29 shows a ditch design chart from which the appropriate ditch width and depth can be determined. A protection system utilized by the railways consists of wires strung along the toe of very steep rock cuts. If a rock fall breaks a wire, a light signal is activated that warns engineers to stop trains or proceed with caution. Such systems are not suitable for highways.

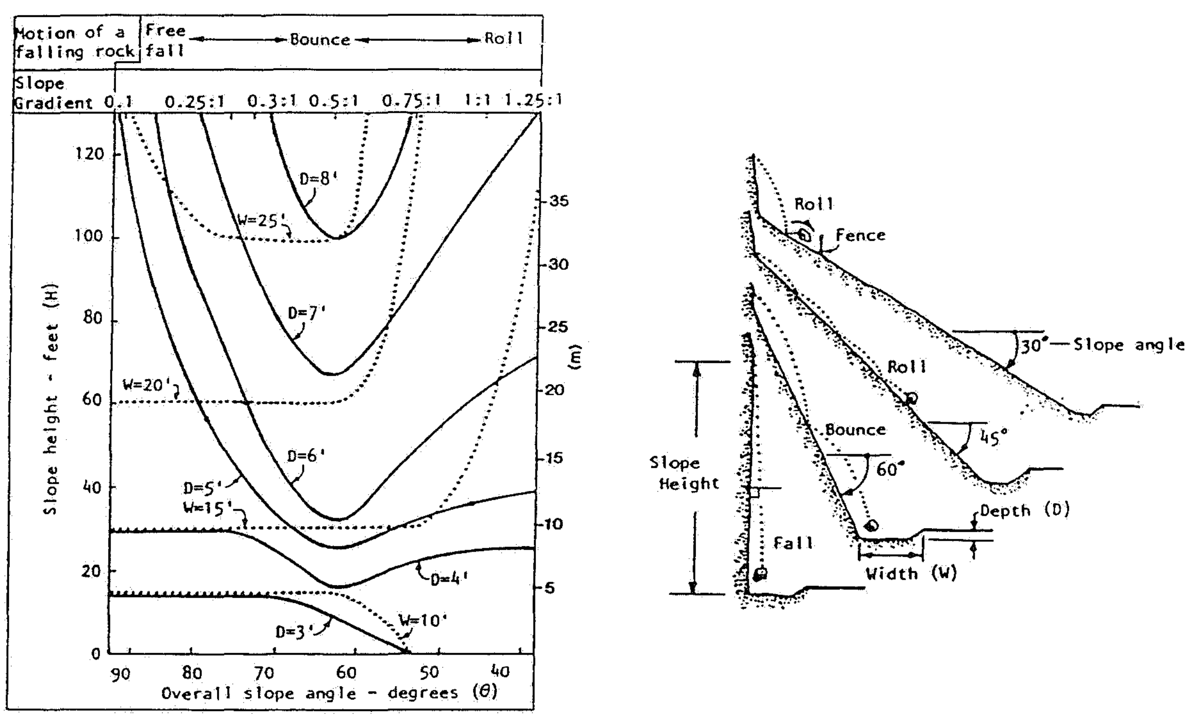


Figure 29: Design chart to determine the required width and depth of rock catch ditches in relation to the height and face angle of the slope (from Ritchie, 1963).

km 3.1, junction of old highway and Trans Canada Highway, turn left toward Hope

km 22, turn right into parking area at Lake-of-the-Woods

STOP 7 - LAKE-OF-THE-WOODS SLIDE

Lake-of-the-Woods Slide is located 4 km north of Hope on the west side of Fraser Valley (Fig. 1). The scar of the rock avalanche extends from an elevation of 500 to 800 m (Fig. 30). Bedrock in the source area is quartz diorite which is part of the Spuzzum Pluton (Fig. 2). The Hope Fault extends north across the lower part of the slide scar (Fig. 3). The orientation of the failure planes, which are joint-controlled, gives the scar a distinctive asymmetric wedge shape. The steep west side of the wedge is visible from this stop. The slide volume has been estimated at 20 x 10^6 m^3 (Thurber Consultants Ltd., 1988b).

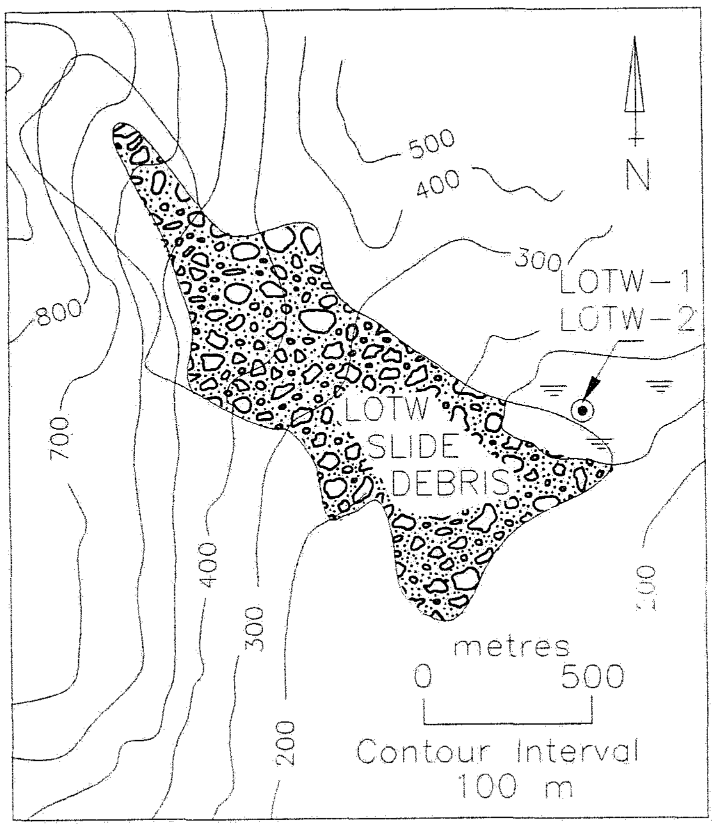


Figure 30: Location map of the Lake-of-the-Woods Slide showing location of piston core sampling (from Naumann, 1990).

The slide debris appears very blocky, but because it impounds a lake, the interior of the deposit must contain very fine material or organics in the basin act as an impermeable liner. The level of the lake is controlled by seepage through the debris.

Bottom samples were collected for dating using a piston corer and aluminum casing lowered from a floating platform at the location shown in Figure 30. A typical core log is shown in Figure 31. Mount Mazama ash (6800 years BP) was found 0.25 m above mineral soil at the bottom of the core, providing a minimum age for the lake. Two samples of gyttja (soft organic ooze) from the lowest 20 cm of the lake sequence were radiocarbon dated at 8260±70 (Isotrace LW-PS1A) and 8430±60 (Isotrace LW-PS3A) years BP. Thus the rock avalanche is at least 8500 years old.

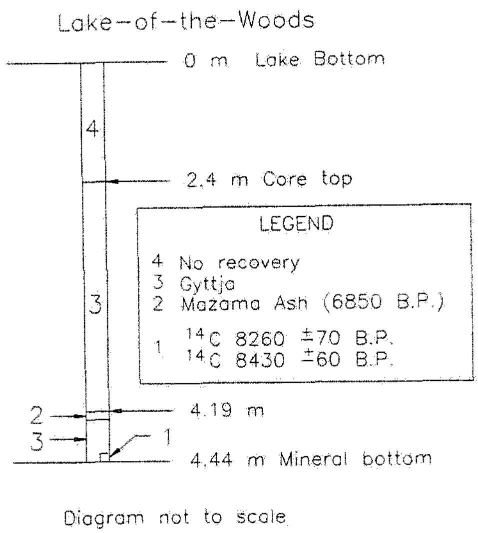


Figure 31: Representative section of piston cores LOTW-1 and LOTW-2 taken from the lake bottom sediments at Lake-of-the-Woods (see Fig. 30 for core locations; from Naumann, 1990).

Turn right onto Trans Canada Highway toward Hope

km 25, junction of Highway 7, keep right onto Highway 7 toward Agassiz

km 29, Westcoast Energy Inc. aerial pipeline crossing of Fraser River

This structure was built in 1957. The pipeline is 76 mm in diameter and operates at pressures of 6.5 MPa, delivering natural gas from northeast British Columbia to markets in the Lower Mainland and United States. The aerial crossing spans 448 m from tower to tower and is maintained through annual engineering inspections, tensioning, painting and other refurbishing as needed. Most natural gas pipelines built today are buried at river and stream crossings and thus out of sight. Because the knowledge of river hydraulics and engineering was not as advanced in the 1950s as it is today, the original pipeline crossings of the Fraser, Peace and other major rivers were aerial structures in contrast to the present-day conventional buried structures.

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km 33, Katz Slide scar visible to the right (northwest) side of bus. Note a few large blocks on the left side of the bus.

km 53, junction of Highway 9, continue over overpass to Highway 9

km 54, turn right onto road beside Highland Helicopter hanger. The tour guides will summarize the day from this vantage point which provides a panoramic view of the Cheam and Popkum slide scars, Mount Cheam and Wahleach

km 0, Oedometers should be reset before leaving the Highland Helicopter hanger. Turn left and continue southeast toward the Trans Canada Highway

km 4, Fraser River bridge

km 8, junction with the Trans Canada Highway. Retrace the route from this morning to point of origin

km 125, Ramada Rennaissance Hotel and end of tour!

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REFERENCES

1) Armstrong, J.E. 1977a. Quaternary Stratigraphy of the Fraser Lowland. Geological Association of Canada, Annual Meeting, Vancouver, B.C., Guidebook, Trip 10.

2) ___ 1977b. Quaternary Geology of the Fraser Lowland (Field Trip No. 6). *In* Geological Excursions in the Pacific Northwest. *Edited by* E.H. Brown and R.C. Ellis. Western Washington University, Department of Geology, Bellingham, Washington, pp. 204-226.

3) ___ 1980. Surficial Geology, Chilliwack, British Columbia. Geological Survey of Canada, Map 1487A, scale 1:50,000.

4) ___ 1984. Environmental and Engineering Applications of the Surficial Geology of the Fraser Lowland, B.C. Geological Survey of Canada, Paper 83-23, 54 p.

5) Bacon, C.R. 1983. Eruptive History of Mount Mazama and Crater Lake Caldera, Cascade Range, U.S.A. Journal of Volcanology and Geothermal Research, 18: 57-115.

6) Baker, D.G. 1991. Wahleach Power Tunnel Monitoring. *In* Proceedings of the Third International Symposium on Field Measurements in Geomechanics, Oslo, Norway.

7) Beck, A.C. 1968. Gravity Faulting as a Mechanism of Topographic Adjustment. New Zealand Journal of Geology and Geophysics, 11: 191-199.

8) Bell, A. 1980. Terrain Mapping and Regional Slope Stability Evaluation in the Fraser Canyon, British Columbia. Unpublished M.Sc. Thesis, Department of Geography, University of British Columbia, Vancouver, 178 p.

9) Bevier, M.L., Armstrong, J.E., and Souther, J.G. 1979. Miocene Peralkaline Volcanism in West-Central British Columbia -- Its Temporal and Plate Tectonics Setting. Geology, 7: 389-392.

10) Cameron, V.J. 1990. Late Quaternary Geomorphic History of the Sumas Valley. Unpublished Master of Science Thesis, Department of Geography, Simon Fraser University, Burnaby, British Columbia, 154 p.

11) Brawner, C.O. and Wyllie, D.C. 1975. Rock Slope Stability on Railway Projects. American Railway Engineering Society, Regional Meeting, Vancouver, B.C., 9 p.

12) Clague, J. 1989.

13) Clague, J.J., Evans, S.G., Fulton, R.J., Ryder, J.M., and Stryd, A.H. 1987. Quaternary Geology of the Southern Canadian Cordillera. Twelfth International Quaternary Association Congress Field Excursion A-18, 67 p.

14) Clague, J., Harper, J.R., Hebda, R.J., and Howes, D.E. 1982. Late Quaternary sea Levels and Crustal Movements, Coastal British Columbia. Canadian Journal of Earth Sciences, 19: 597-618.

15) Evans, S.G., Clague, J.J., Woodsworth, G.J., and Hungr, O. 1989. The Pandemonium Creek Rock Avalanche, British Columbia. Canadian Geotechnical Journal, 26: 427-446.

16) Holland, S.S. 1964. Landforms of British Columbia, a Physiographic Outline. British Columbia Department of Mines and Petroleum Resources, Bulletin 48 (reprinted 1976).

17) Hickson, C. 1992. Personal Communication.

18) Hungr, O. and Evans, S.G. 1989. Engineering Aspects of Rockfall Hazards in Canada. Geological Survey of Canada, Open File 2061, 102 p.

19) Keefer, D.K. 1984. Landslides Caused by Earthquakes. Geological Society of America Bulletin, 95: 406-421.

20) Keen, C.E., and Hyndman, R.D. 1979. Geophysical Review of the Continental Margins of Eastern and Western Canada. Canadian Journal of Earth Sciences, 16: 712-747.

21) Mathews, W.H. 1972. Geology of Vancouver Area of British Columbia. 24th International Geological Congress, Montreal, Quebec, Guidebook, Field Excursion A05-C05.

22) ___ 1986. Physiographic Map of the Canadian Cordillera. Geological Survey of Canada, Map 1701A.

23) Mathews, W.H., Fyles, J.G., and Nasmith, H.W. 1970. Postglacial Crustal Movements in Southwestern British Columbia and Adjacent Washington State. Canadian Journal of Earth Sciences, 7: 690-702.

24) Mathews, W.H. and McTaggart, K.C. 1969. The Hope Landslide, British Columbia. *In* Proceedings of the Geological Association of Canada, 20: 65-75.

25) ___1978. Hope Rock Slides, British Columbia, Canada. *In* Rockslides and Rock Avalanches. *Edited by* B. Voight. Elsevier, 259-275.

26) Milne, W.G., Rogers, G.C., Riddihough, R.P., McMechan, G.A., and Hyndman, R.D. 1978. Seismicity of Western Canada. Canadian Journal of Earth Sciences, 15: 1170-1193.

27) Miles, M.J. and Kellerhals, R. 1981. Some Engineering Aspects of Debris Torrents. *In* Proceedings of the Fifth Canadian Hydrotechnical Conference, sponsored by the Canadian Society for Civil Engineers, Fredericton, N.B., 395-420.

28) Monger, J.W.H. 1966. The Stratigraphy and the Type Area of the Chilliwack Group, Southwestern British Columbia. Unpublished Ph.D. thesis, Department of Geological Sciences, University of British Columbia, 173 p.

29) Monger, J.W.H. 1989. Geology of Hope and Ashcroft Map Areas, British Columbia. Geological Survey of Canada, Maps 41-1989 and 42-1989.

30) Monger, J.W.H. 1992.

31) Moore, D.P., Imrie, A.S., and Baker, D.G. 1991. Rockslide Risk Reduction Using Monitoring. *In* Proceedings of the Third Annual Canadian Dam Safety Conference, Whistler, British Columbia.

32) Nasmith, H., Mathews, W.H., and Rouse; G.E. 1967. Bridge River Ash and Some Other Recent Ash Beds in British Columbia. Canadian Journal of Earth Sciences, 4: 163-170.

33) Naumann, C.M. 1990. The Cheam Slide: A Study of the Interrelationship of Rock Avalanches and Seismicity. Unpublished M.A.Sc. Thesis, Geological Engineering Program, Department of Geological Sciences, University of British Columbia, Vancouver, 203 p.

34) Naumann, C.M. and Savigny, K.W. 1992. Large Rock Avalanches and Seismicity in Southwestern British Columbia, Canada. In Landslides, Proceedings of the Sixth International Symposium, Volume 2. Edited by David H. Bell. A.A. Balkema, pp. 1187-1192.

35) Parrish, R.R. 1983. Cenozoic Thermal Evolution and Tectonics of the Coast Mountains of British Columbia, 1. Fission Track Dating, Apparent Uplift Rates, and Patterns of Uplift. Tectonics, 2: 601-631.

36) Pierson, L., Davis, S., and Van Vickle, R. 1990. The Rockfall Hazard Rating System Implementation Manual. Technical Report #FHWA-OR-EG-90-01, Washington, D.C., 40 p.

37) Piteau, D.R. 1977. Regional Slope-Stability Controls and Engineering Geology of the Fraser Canyon, British Columbia. *In* Landslides, Reviews in Engineering Geology, Vol. III. *Edited by* D.R. Coates. Geological Society of America, Boulder, Colorado, pp.85-111.

38) Riddihough, R.P., and Hyndman, H.D. 1976. Canada's Active Western Margin -- The Case for Subduction. Geoscience Canada, 3: 269-278.

39) Riddihough, R.P., Beck, M.E., Chase, R.L., Davis, E.E., Hyndman, R.D., Johnson, S.H., and Rogers, G.C. 1983. Geodynamics of the Juan de Fuca Plate. *In* Geodynamics of the Eastern Pacific Region, Caribbean and Scotia Arcs. *Edited by* R. Cabre. American Geophysical Union, Geodynamics Series, 9, pp. 5-21.

40) Ripley, B.D., Banks, P.A. and Imrie, A.S. 1991. Design Aspects for the Wahleach Conduit Replacement. *In* Proceedings, Canadian Electrical Association; Hydraulic Power Section, Toronto.

41) Ritchie, A.M. 1963. Evaluation of Rockfall and Its Control. Highway Research Board, Highway Research Record 17, 13-28.

42) Roy, P.E. 1968. The Changing Role of Railways in the Lower Fraser Valley. *In* The Lower Fraser Valley, Evolution of a Cultural Landscape. *Edited by* A.H. Siemans. Tantalus Press, Vancouver, pp. 51-68.

43) Ryder, J.M. 1981. Geomorphology of the Southern Part of the Coast Mountains of British Columbia. Zeitschrift für Geomorphologie, 37: 120-147.

44) Savigny, K.W. 1992. Engineering Geology of Large Landslides in the Lower Fraser River Valley Transportation Corridor, Southwestern Canadian Cordillera. *In* Landslide Hazards in the Canadian Cordillera. *Edited by* S.G. Evans. Geological Association of Canada, Special Publication in press.

45) Savigny, K.W. and Rinne, N.F. 1991. Assessment of Landslide Hazards Along a Pipeline Corridor in Mountainous Terrain of Southwestern British Columbia. Text distributed at the 44th Canadian Geotechnical Conference, sponsored by the Canadian Geotechnical Society, Calgary, Alberta, 19 p.

46) Scheidegger, A.E. 1973. On the Prediction of the Reach and Velocity of Catastrophic Landslides. Rock Mechanics, 5: 231-236.

47) Siemans, A.H. 1968. The Process of Settlement in the Lower Fraser Valley. *In* The Lower Fraser Valley, Evolution of a Cultural Landscape. *Edited by* A.H. Siemans. Tantalus Press, Vancouver, pp. 27-50.

48) Smith, R.H. 1971. A Study of a Prehistoric Landslide Surface at Cheam Lake, East of Chilliwack, British Columbia. Unpublished B.A.Sc. thesis, Department of Geological Sciences, University of British Columbia, 24 p.

49) Souther, J.G. 1977. Volcanism and Tectonic Environments in the Canadian Cordillera -- A Second Look. *In* Volcanic Regimes in Canada. *Edited by* W.R.A. Baragar, L.C. Coleman, and J.M. Hall. Geological Association of Canada, Special Paper 16, pp. 3-24.

50) Tatchell, G.E. 1991. Automatic Data Acquisition Systems for Monitoring Dams and Landslides. *In* Proceedings of the Third International Symposium on Field Measurements in Geomechanics, Oslo, Norway.

51) Thurber Consultants Ltd. 1986. Geotechnical Hazard Assessment Silverhope, B.C. Report to the Regional District of Fraser-Cheam (File No. 14-50-2, dated December 1, 1986).

52) Thurber Consultants Ltd. 1988a. Report to the Seventh Day Adventist Church on the Chawuthen Area Subdivision (File No. 19-567-1, dated September 6, 1988).

53) Thurber Consultants Ltd. 1988b. Report to the Regional District of Fraser-Cheam on a Geotechnical Hazard Investigation, Ross Road/Lake-of-the-Woods area, Electoral Area B (File No. 14-50-6, dated October 11, 1988).

54) Thurber Engineering Ltd. 1991. Re-evaluation of the Cheam Slide. Draft report to Regional District of Fraser-Cheam.

55) von Sacken, R.S. 1991. New Data and Re-evaluation of the 1965 Hope Slide, British Columbia. Unpublished M.Sc. Thesis, Department of Geological Sciences, University of British Columbia, Vancouver, 196 p.

56) von Sacken, R.S., Savigny, K.W., and Evans, S.G. 1992. The Detachment Surface of the 1965 Hope Slide, British Columbia, Canada. *In* Landslides, Proceedings of the Sixth International Symposium, Volume 1. *Edited by* David H. Bell. A.A. Balkema, pp. 249-254.

57) Weichert, D.H., Horner, R.B. and Evans, S.G. 1990. Earthquakes and the 1965 Hope landslide? Abstracts, American Geophysical Union Pacific Northwest Regional Meeting, September 13-14, 1990, Seattle, Washington.

58) Wetmiller, R.J. and Evans, S.G. 1989. Analysis of the Earthquakes Associated with the 1965 Hope Landslide and Their Effects on Slope Stability at the Site. Canadian Geotechnical Journal, Vol. 26, No. 3, pp. 484-490.

59) Wyllie, D. 1987. Rock Slope Inventory System. *In* Proceedings of the Federal Highway Administration Rockfall Mitigation Seminar, F.H.W.A. Region 10, Portland, Oregon, 22 p.

60) Wyllie, D. 1991. Rock Slope Stabilization and Protection Measures. *In* Proceedings of a National Symposium on Highway and Railroad Slope Maintenance. Sponsored by the Association of Engineering Geologists, Chicago, 41-63.

Technical Tour No. 3

Fraser River Delta

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Thursday, May 7, 1992

INTRODUCTION

The coastal area of southern British Columbia lies within an active seismic region. Seven earthquakes in the magnitude range M5-7 have occurred in the past 100 years. Geological evidence suggests that very large subduction earthquakes of the order M8+ have occurred in the past. The recurrence period of these earthquakes is thought to be about every 700 years on average, with the most recent such event having occurred about 300 years ago.

The Fraser Delta lies within this region and is particularly prone to damage in the event of a major earthquake. This is because it is underlain by deep deposits of relatively loose or soft soils. The presence of such soils can: amplify the intensity of shaking; lengthen the predominant period of the motion, and cause strength loss or liquefaction of saturated sandy soils.

Experience at Mexico City during the 1985 earthquake showed that a major cause of damage was the very high amplification of acceleration that occurred as the motion propagated upward through the soft clay lacustrine deposits. A similar amplification occurred in the San Francisco Bay muds and caused much of the damage in San Francisco and Oakland during the 1989 Loma Prieta earthquake. In addition, liquefaction of loose sand fill placed on top of the Bay mud greatly added to the damage where it was present, as for example in the Marina district.

In much of the Fraser Delta, natural deposits of loose to medium dense sands overlie deep silt and clay deposits, so that the combined effects of both amplification and liquefaction are a possibility in the event of a major earthquake, i.e. the amplified motions are more likely to trigger liquefaction.

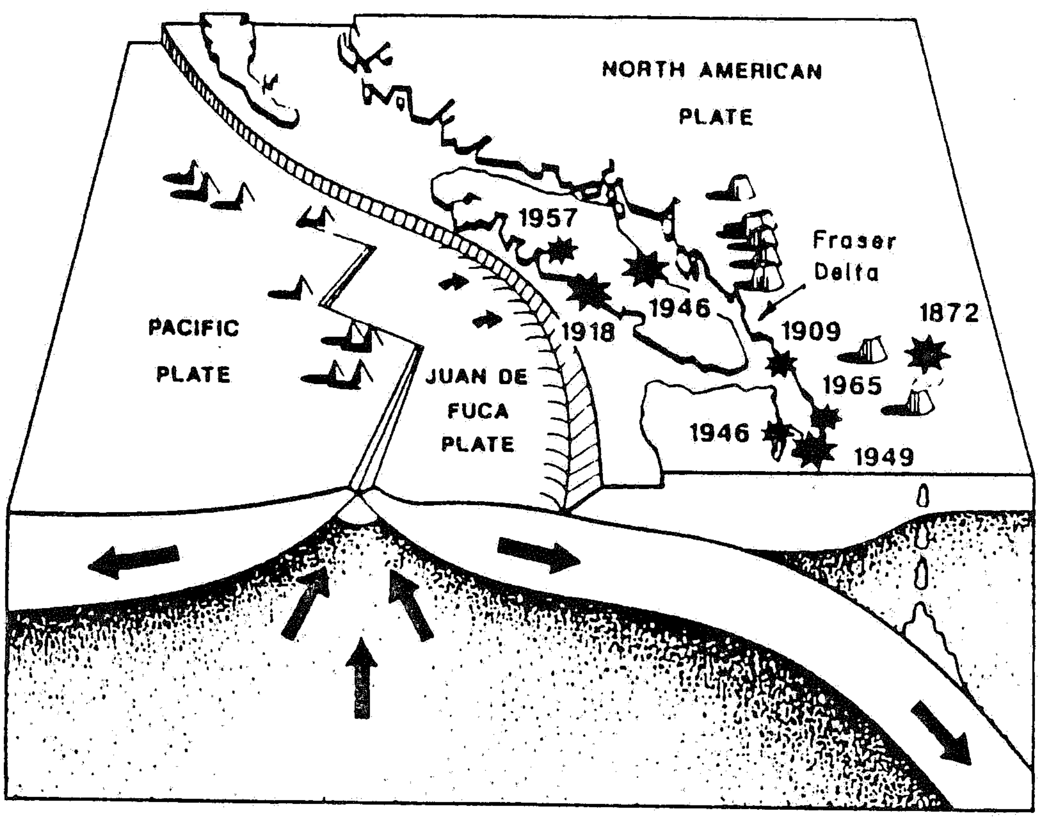
Geological evidence has recently come to light indicating that liquefaction has occurred in the delta in the recent past. Excavations for the foundations of Kwantlen College in Richmond have revealed cracks in the surface crust that are filled with loose sand. Such features are consistent with the loose sand underlying the crust having liquefied and flowed upward into cracks formed by differential movements. Similar features were produced during the 1988 Saguenay earthquake in Quebec.

It is not a question then of whether liquefaction will occur in the Fraser Delta, but rather when it will occur and what is the likely damage.

This guide book is intended to give a background in the seismic setting, the soil conditions, the geologic evidence of liquefaction, the history of seismic design of building foundations, some structures of interest and their foundations, and the likely damage in the event of a major earthquake.

TECTONIC SETTING AND SEISMICITY

The present tectonic regime of western North America is controlled by the motions of the Pacific, America, Juan de Fuca, and Explorer plates (Fig. 1). Of particular importance in the context of this field trip is the boundary between the America and Juan de Fuca plates. This boundary is thought to be a zone of convergence or subduction. It is well established that subduction has occurred along the coasts of British Columbia, Washington, and Oregon during the last several million years, but there has been some debate as to whether or not it is continuing at present. The doubt arises primarily from the absence of a deep marginal trench characteristic of most active subduction zones, the lack of deep or thrust-type earthquakes in an eastward-dipping Benioff zone, and the relatively low level of historic volcanism in the mountains bordering the Pacific Ocean. Most workers, however, are still of the opinion that subduction is occurring south of 50°N latitude, although at a very low rate in southwestern British Columbia.



(Energy, Mines and Resources Canada)

Figure 1. Tectonic Setting.

This question aside, the distribution of earthquakes, active and recently active faults, and young volcanoes in the region clearly is controlled by plate motions. Most large earthquakes and active faults are associated with offshore spreading ridges and transform faults. Earthquakes are also common in the Vancouver Island - Strait of Georgia - Puget Sound region, and this seismicity may be related to lateral compression or overlapping in the sinking Juan de Fuca plate due to the major change in trend of the North American continental margin north of Washington State. Earthquake activity in the Strait of Georgia decreases northward and is very low north of the presumed Explorer-America plate boundary at about 51°N. There has been no significant historic seismicity along the interface between the Juan de Fuca and America plates (Cascadia subduction zone), either because the plates are sliding past one another aseismically or because they are locked. If the latter is true, it is likely that there will be a very large earthquake (M8+) somewhere along the Cascadia subduction zone within the next several hundred years. Geologic evidence suggests that such 'megaquakes' have occurred in the past, the most recent about 300 years ago.

Reports of seismic activity in southwestern British Columbia date back to 1872 when a large earthquake of estimated Richter magnitude 7-7.5 was felt throughout most of the settled Pacific Northwest. The epicentre of this event is poorly located, but is known to have been somewhere in north-central Washington or southernmost British Columbia. The most damaging historic earthquake in western Canada occurred in 1946 on central Vancouver Island. This earthquake had a magnitude of 7.2 and caused liquefaction in coastal areas up to 100 km from the epicentre. Although there apparently was no damage or significant liquefaction on the Fraser Delta, the local newspaper reported that runways at Vancouver Airport "rolled like waves." Several other earthquakes have been felt in the region; the most significant events are listed in Table 1.

SETTING AND GEOLOGY OF THE FRASER DELTA

The Fraser River delta, south of Vancouver (Fig. 2), is the largest and most important delta on the west coast of Canada. It is an important agricultural and waterfowl area and a vital link in the Fraser River salmon fishery; it is also an area of explosive urban and industrial growth, with almost 150,000 people now living on the deltaic plain.

The Fraser Delta extends 15-23 km west and south from a narrow gap in the Pleistocene uplands at New Westminster to meet the sea along a perimeter of about 40 km. The sub-aerial, inhabited portion of the delta is dyked and lies 0-2 m above mean sea level, with the water table within 2 m of the surface. Very gently sloping tidal flats extend up to 9 km west and south from the protective dykes to the sub-tidal delta foreslope.

Table 1. Significant Historic Earthquakes Affecting the Fraser Delta[1]

Date	Epicentre	Region	Magnitude	Comment
14 Dec. 1872	4.83°, 120.3°	Northern Washington	Est. 7.4	A large earthquake felt through most of settled B.C.
12 Dec. 1880		Puget Sound	6.0	Felt from Victoria to Portland
11 Jan. 1909	48.7°, 122.8°	Puget Lowland	6.0	Felt over 64,000 km^2
18 Aug. 1915	49°, 122°	Fraser Lowland		Felt from Victoria to the Okanagan and to Oregon
6 Dec. 1918	49.6°, 125.9°	Vancouver Island	7.0	Felt at Victoria, Vancouver, Kelowna and Seattle
17 Sept. 1926	50.0°, 123.0°	Coast Mountains	5.5	
23 June 1946	49.5°, 125.3°	Vancouver Island	7.2	Most damaging earthquake in history of western Canada
13 Apr. 1949	47.2°, 122.6°	Puget Lowland	7.1	Extensive damage in southern Puget Lowland, felt throughout southwestern B.C.
29 Apr. 1965	47.4°, 122.3°	Puget Lowland	6.5	Damage in Puget Lowland, felt throughout southwestern B.C.
16 May 1976	48.8°, 123.3°	Gulf Islands	5.4	Felt on Vancouver Island, B.C., Mainland and Washington

[1]Sources: Milne et al. (1978, Table 2) & Earthquake data file of the Geological Survey, Canada

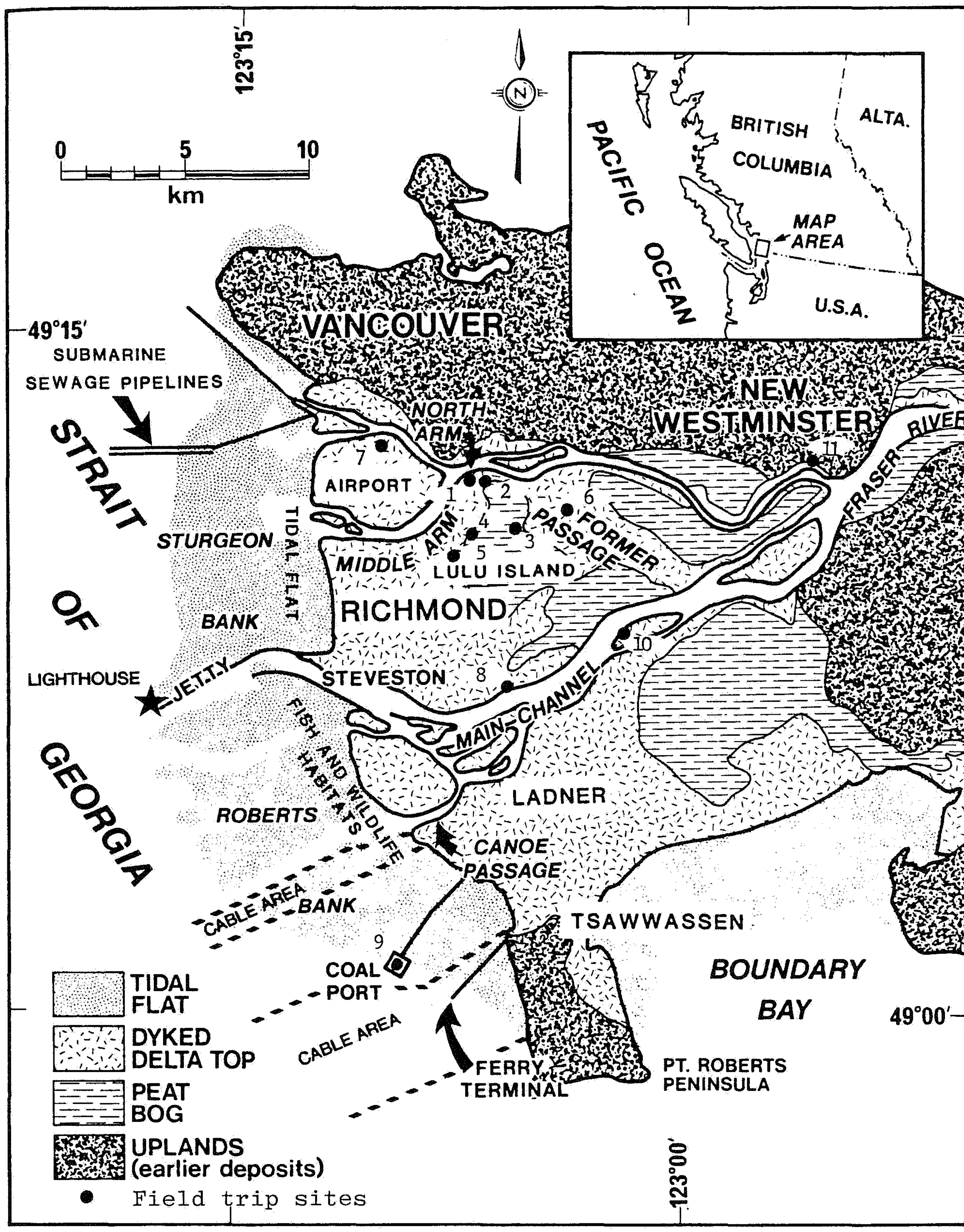


Figure 2. Fraser Delta.

The delta is geologically young, having formed since the disappearance of the last Cordilleran Ice Sheet 13,000-11,000 years ago. At the maximum of the last glaciation, ca. 15,000-14,000 years ago, a lobe of this ice sheet about 1500 m thick filled the Strait of Georgia and Fraser Lowland and flowed south across the area now occupied by the Fraser Delta.

A thick sequence of postglacial delta deposits unconformably overlies Pleistocene till and stratified glaciomarine sediments, and comprises three main stratigraphic units (Fig. 3):

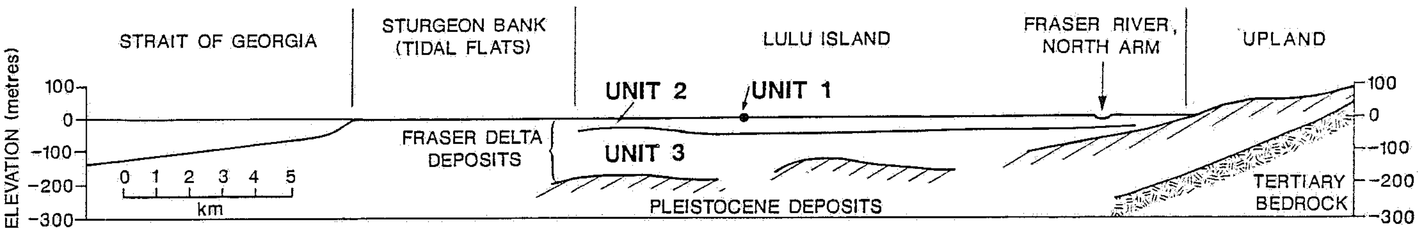


Figure 3. Fraser Delta Stratigraphic Section.

Unit 1. The uppermost unit consists of horizontally bedded mud and sand deposited in intertidal and fluvial environments, and peat deposited in swamps and bogs. Locally, distributary channels filled with sand extend several metres down into unit 2, giving rise to an irregular surface. Unit 1 thins and decreases in age towards the west and southwest, in the direction of delta growth. On the western and southern parts of the dyked delta plain, it is typically 2-5 m thick; to the east, nearer the apex of the delta, it may attain thicknesses of 20 m, the uppermost few metres of which are peat. In areas without peat, the surface sediments have been tilled to a depth of 0.2-0.3 m. Locally, in developed areas, these sediments have been stripped and replaced by 0.5-2 m of sand fill. The uppermost 1-1.5 m of undisturbed mud is stiff, due to desiccation, and may contain shrinkage cracks. Mud underlying the desiccated crust is soft to firm in consistency.

Unit 2. Unit 1 is underlain by a succession of sand beds that commonly dip gently seaward. These sediments are thought to record the seaward advance of the delta slope, although some may have been deposited in intertidal and distributary channel environments. The sands typically are well sorted and fine- to medium-grained. They range from loose to very dense over relatively short distances. Unit 2 is 15-30 m thick on the northern Fraser Delta in the vicinity of Richmond, but is as much as 150 m thick on the southern delta between the Middle Arm of Fraser River and Boundary Bay.

Unit 3. The lowest unit consists of mud and fine sand deposited in ancestral Strait of Georgia in lower foreslope and basin environments.

ENGINEERING CONCERNS AND FOUNDATION DESIGN IN THE FRASER DELTA

Traditional Foundation Design

Prior to 1968 to 1970, soil liquefaction concerns were not addressed in foundation design. Light one to three storey structures were generally founded on spread footings bearing on the stiff to firm silt crust common to much of the delta. Heavier structures were founded on either timber or expanded base concrete (Franki) pilings. Preloading was often used to reduce building settlements.

Between 1970 and approximately 1985, liquefaction was addressed, but using a design acceleration, varying from 0.08 to 0.21 g. For level ground conditions, piles taken through the liquefied soil were assumed stable and generally little consideration was given to pile deformations or ductility. A few foundation designs included soil densification to reduce liquefaction risk.

Between 1985 to approximately 1990, liquefaction was generally considered in the design, using the design maximum acceleration recommended by Byrne and Anderson, i.e., 0.17 g in 1982 and 0.20-0.22 g in 1987. Some sites, generally for larger high-rise structures were densified using either compaction piles or vibro-replacement. Toward the end of this period concern was being raised that, for level ground conditions, piles passing through liquefied soil might not survive the induced lateral displacements, and soil densification became more common. In 1982 and 1987, Byrne and Anderson, suggested a lateral displacement of 0.3 m be used for design in level ground conditions.

Present Foundation Design

Current foundation designs consider the potential amplification of ground motions, the potential for soil liquefaction and related lateral and vertical movements within the soil profile. Foundation designs are generally conducted using the recommendations of the National Building Code of Canada (NBCC), and the 1991 Task Force (Earthquake Design In the Fraser Delta) chaired by Byrne and Anderson. The NBCC specifies a minimum design earthquake which buildings must survive without collapse causing significant loss of life. This design earthquake is a 475 year return period event based on a probabilistic assessment of historical earthquake records. For the Fraser Delta the NBCC design earthquake has a hard ground maximum acceleration of 0.21 g and maximum particle velocity of 0.21 m/sec., with the predominant risk coming from magnitude 6.3 to 7.3 events. The 1991 Task Force recommended that for the Fraser Delta the foundation factor for determining building base shear should be increased from 1.5 to 2 to allow for ground motion amplification and period shift. It recommended that, for liquefaction assessment, that the NBCC design earthquake maximum acceleration should be amplified from 0.21 g (base rock motion) to 0.3 g (at the ground surface). It also suggested procedures for estimating liquefaction induced settlement and lateral displacements (estimated to possibly exceed 1 m for some near level ground soil profiles).

Pile load tests have recently been conducted in the Fraser Delta to assess the lateral deformations (or curvatures) which the piles could sustain. These tests indicated that pipe, timber, and specially designed concrete piles may tolerate relatively large lateral deformations (and curvatures) without collapse. The tests also indicated that the traditional concrete piles with relatively light spiral reinforcement would tolerate significantly less deformation and might collapse.

Light structures (one to three storey wood frame) are usually built without soil densification. They are founded on spread foundations bearing on the surficial clay/silt crust which must have sufficient strength so that the footings will not fail into the underlying liquefied soil. Structurally the buildings must tolerate liquefaction induced vertical settlements without collapse. Preloading is often conducted to mitigate non-seismic settlement concerns.

Heavier high-rise structures are generally founded on expanded base piles. Potentially liquefiable ground around and below the pile foundations is normally densified to reduce the liquefaction risk. Preloading may still be conducted in order to mitigate non-seismic settlement concerns.

Several alternative foundation designs have been used, particularly for mid-size structures, including:

- Pipe piles designed to tolerate large lateral displacements without collapse, no densification;

- Spread foundations bearing on the stone columns resulting from vibro-replacement densification;

- Removal of the clay/silt crust, replacement with sand, and densification to reduce liquefaction risk. The structure is founded on spread footings foundations.

FIELD TRIP SITE DESCRIPTIONS

In driving through the Fraser Delta we will discuss seismic design aspects of the foundations of several structures. These structures are designated by numbers and are located on Fig. 2.

1. Oak Street Bridge

The Oak Street Bridge spans the North Arm of the Fraser River and provides the main access south through the Fraser Delta. The bridge consists of simply supported concrete spans and was constructed between 1955 and 1957. The three central spans are supported on bridge piers in the river. The north approaches consist of 27 simply supported spans (Piers N2 to N29); and the south approaches consists of 54 simply supported spans (Piers S2 to S58) as shown in Fig. 4.

Soil Conditions

Glacial deposits outcrop on the north side and dip to the south. They are overlain by delta deposits. The central spans and the south approaches are located on significant thicknesses of alluvial deposits underlain by dense glacial deposits. The alluvial deposits consist of a loose to compact sand strata varying from 7 to 20 m thick at the surface, underlain by a soft to firm silty clay strata over 46 m in thickness.

On the north side, a soft organic silt stratum up to 7 m in thickness is present close to the river bank. This is underlain by dense sand and gravels and glacial deposits that gradually rise to the ground surface towards the north approaches.

Foundations

Oak Street bridge is supported on four types of foundations; footings, caissons, steel H-piles, and timber pile groups.

Spread footings: The north approaches, where the dense glacial deposits exists at shallow depth, the piers (N8 to N29 and N38 to N44) are supported on spread footings.

Caissons foundations: The two major river piers (N1 and S1) that support the central span are founded on caisson foundations. The foundations were constructed on the dense glacial strata by excavating the soft alluvial strata using a prefabricated cofferdam. At Pier N1, additional steel H piles were driven beneath the foundation level into the glacial materials.

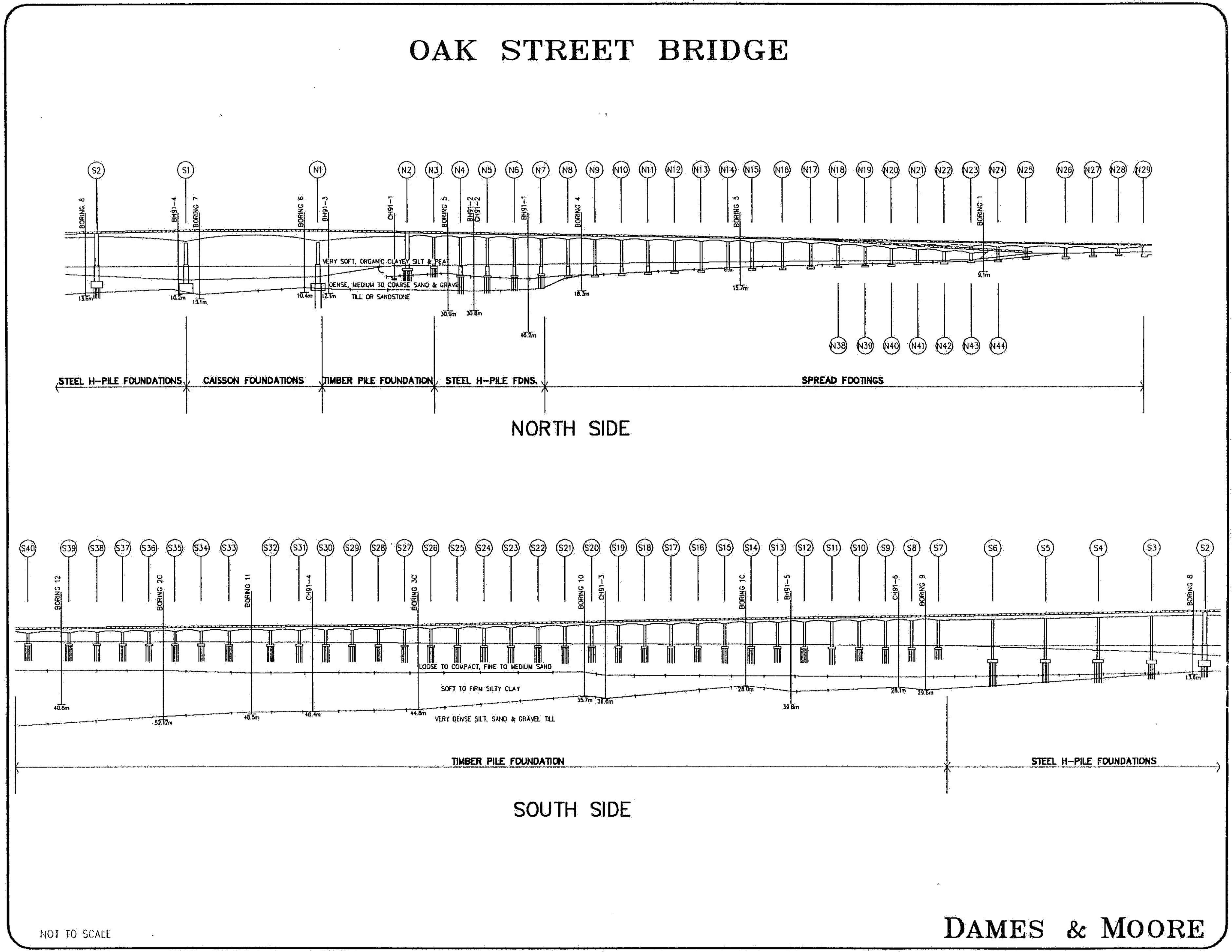


Figure 4. Foundations for Oak Street Bridge.

Steel H-Pile Foundations: Steel H-pile foundations are used adjacent to the main piers on both the north and south sides of the bridge. On the south side, the piers (S2 to S6 inclusive) were constructed by excavating through the alluvial strata using a cofferdam to depths of about 9 to 12 m below the ground surface and driving steel H-piles below these depths into the underlying dense glacial strata to depths of 8 to 16 m.

On the north side, the piers (N4 to N7, inclusive) were constructed by excavating through the soft swampy soils using sheeting to depths of about 7 m below existing ground surface and by driving steel H-piles to penetrate into the underlying dense sand and gravel strata. The piles varied in length from 9 to 12 m below the pile cap elevation.

Timber Pile Supported Foundations: At the south approach piers (S7 to S56 and S64 to S73, inclusive) are supported on groups of driven untreated timber piles with 250 mm tip diameter. The piles within the foundation group were driven at a spacing of 0.9 m to depths of about 7 to 10 m below the pile cap. The pier foundations (N2 and N3) at the river bank of the north abutment were also constructed on driven timber piles. These piles were driven through soft soils meeting refusal at about 4 to 6 m below pile cap elevation.

Recent engineering studies indicate that the tips of the timber piles on the south side are generally founded below the depth of liquefaction and that the driving of the piles has densified the soil within the pile groups. However, the sand adjacent to the group and to a depth of about 8 m is predicted to liquefy and undergo significant horizontal movements which could displace the foundations and result in damage to the bridge structure.

2. Transmission Towers

Electrical power to the delta is provided by BC Hydro via high level transmission lines supported on towers. Major crossings of the North Arm occur near the Oak and Queensborough Bridges. A major crossing of the South Arm occurs adjacent to the George Massey tunnel and there are three crossings on or adjacent to Annacis Island. The towers for these main power lines generally rest on timber pile supported footings. Liquefaction of the underlying sands could cause significant movement of the tower foundations and perhaps failure of the piles in some cases.

Preliminary analyses indicated that timber pile foundations could not withstand the estimated liquefaction induced lateral displacements. However, experience in Japan during the 1964 Niigata earthquake indicated that transmission towers had not toppled in zones of severe liquefaction.

B.C. Hydro have undertaken a detailed study of their tower foundations involving field and laboratory testing of the timber piles. Key factors are the magnitude of the liquefaction induced lateral displacements, and the ability of timber piles to resist axial loads after subjection to large lateral movement cycles. The preliminary results suggest that the piles have significantly greater resistance to lateral movements than expected, and this could account for the observed "good" behaviour in Japan.

3. Sand Dyke Exposures

In 1991, numerous sand dykes were discovered in a recently cleaned, 2 m deep, drainage ditch along Highway 99. These dykes are believed to be indicative of past earthquake induced liquefaction in the delta. A portion of the ditch has been cleaned again by the Department of Highways for this field trip.

The exposed soil profile consists of approximately 1.2 m of peat over silty clay or clayey silt. Although not exposed, loose to medium-dense fine- to medium-grained sand underlies the silty clay and is believed to be the source zone for the sand in the dykes. Figure 5 shows an elevation of some of the dyke exposures in the ditch. The dykes generally dip steeply (35 to 90 degrees) through the silty clay and then flatten to form horizontal sills near the base of the peat layer. The dykes appear to penetrate the peat no more than 0.3 m. This is believed to be the result of low specific gravity and high tensile strength of the upper fibrous peat, making it easier to spread laterally than to fracture vertically. The dykes range in thickness from less than 1 mm to approximately 300 mm, sometimes varying in thickness over short distances. In some places a distinct bedding off-set can be seen where the intruded dykes have displaced the soil.

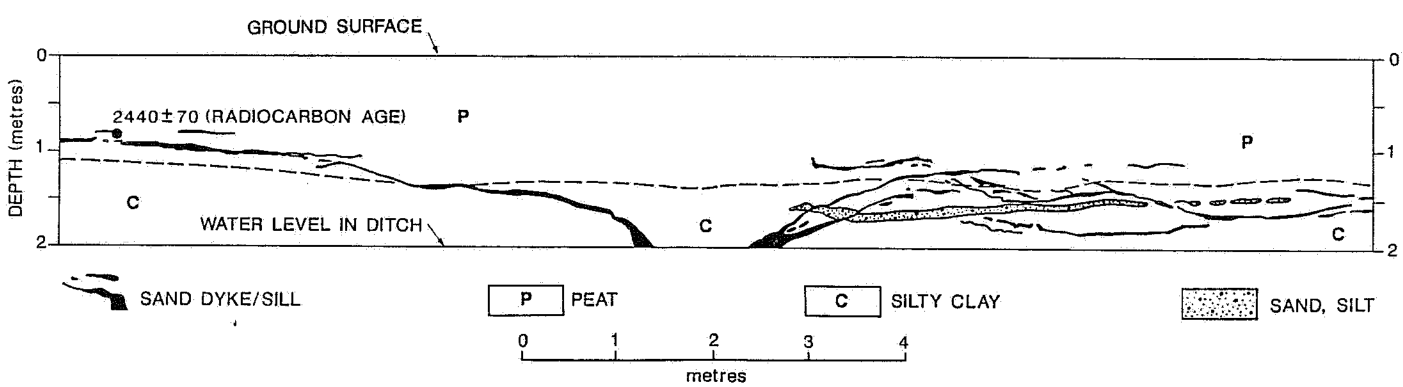


Figure 5. Sand Dykes and Silts (after Clague et al.)

Similar sand dykes have been observed in six other sites in the Fraser Delta area and are expected to be relatively common.

4. Kwantlen College

The college is a three storey structure with approximately 10,000 square metre footprint. Site preparation work commenced in 1990. The surficial clay/silt crust was removed and replaced with sand fill. Then the site was densified, to mitigate liquefaction risks, using relatively economic dynamic compaction procedures (Fig. 6). A 16 ton weight dropping 23 metres was used on a 10 metre grid pattern in order to densify to a depth of 10 metres. The structure was then founded on spread footings bearing on the densified sand. The procedure was more economical than vibro-replacement and eliminated the need for pile foundations.

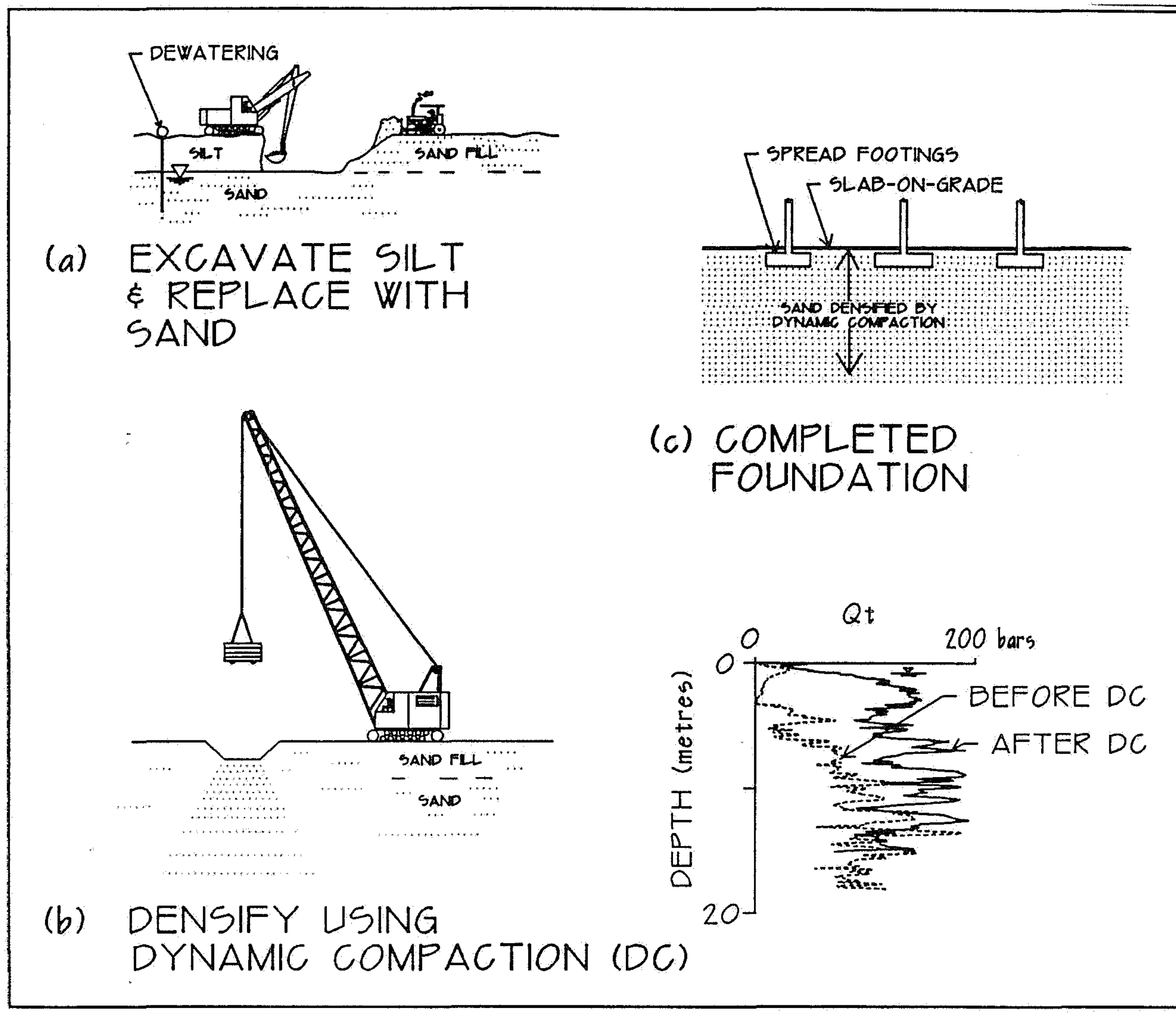


Figure 6. Kwantlen College Site Preparation Procedures

Numerous prehistorical sand dyke and some sand boil features which are indicative of past earthquake induced soil liquefaction were observed in the excavation during the Kwantlen college site preparation work.

5. Westminster Highway & No. 3 Road

The building is a 16 storey high-rise constructed in 1988/1989. It is founded on expanded base concrete piles bearing in the sand strata at approximately 9 metres depth. The building footprint was densified using vibro-replacement procedures to reduce liquefaction risk (Fig. 7). No preloading was carried out.

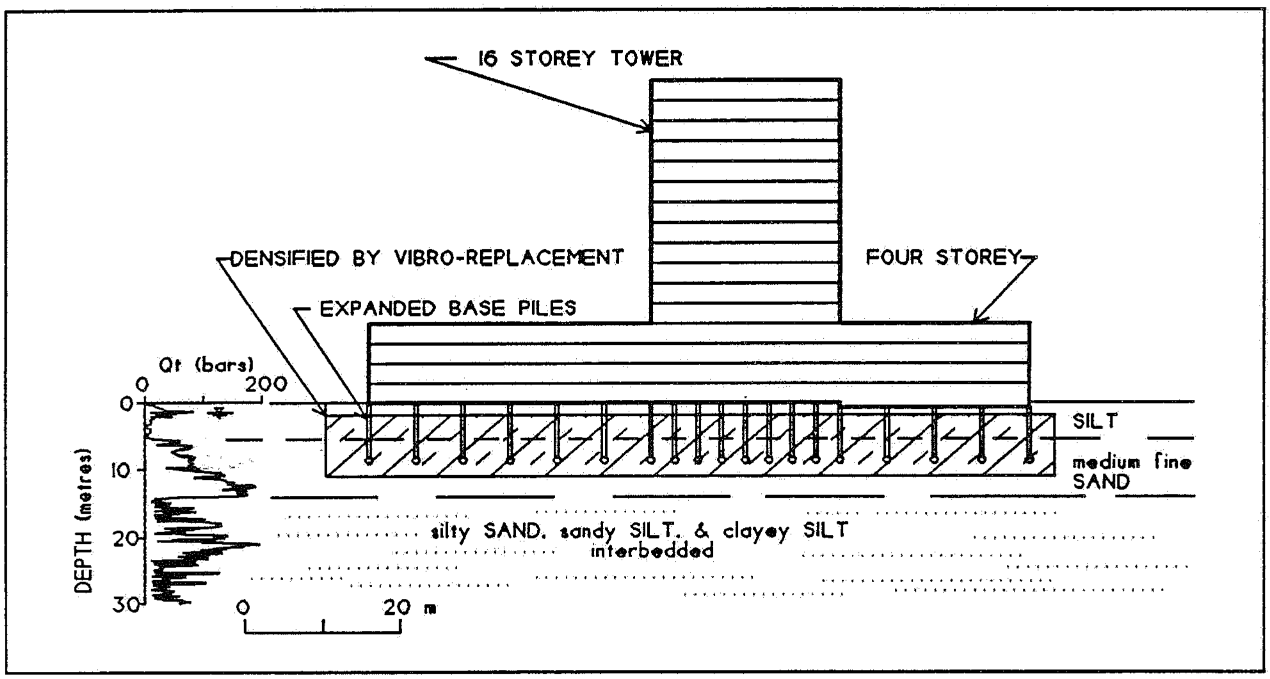


Figure 7. Typical Section (Westminster Highway and No. 3 Road)

6. Richmond No. 7 Firehall (No. 6 Road & Westminster Hwy.)

This structure, built in 1991, is a designated post disaster building and as such was designed to higher standards of earthquake resistance than required for ordinary structures under the National Building Code of Canada (NBCC).

The firehall is founded on deep Fraser River Delta deposits. Results of the geotechnical site investigation and seismic risk analysis indicated that the foundation soils at this site were potentially liquefiable under the NBCC design earthquake motions. Ground improvement using excavation and replacement and vibro-replacement was used to prevent liquefaction.

This vibro-replacement work consisted of installation of 168 stone columns at 2.56 m centres to a depth of about 18 m. Ground improvement to the density required to prevent liquefaction was confirmed by piezocone penetration testing, which showed increased cone tip resistance in the native sand between stone columns.

Within the building area, a surficial layer of compressible silts and organic soils was replaced with a compacted clean granular fill. Building column loads and the floor slab loads are supported on shallow footings in the fill.

7. McDonald's Park Site

Lunch stop, in-situ testing and drilling demonstration, and pile load test poster presentation.

Seismic Cone Penetration Test and Seismic Surface Wave Technique

Considerable progress has been made in the last 25 years in recognizing liquefaction hazards, understanding liquefaction phenomena, and analyzing and evaluating the potential for liquefaction at a site. Recent findings related to the application of the seismic cone penetration test (SCPT) for the evaluation of liquefaction potential will be demonstrated and discussed. The SCPT provides independent measurements of penetration resistance, pore pressures and shear wave velocity in a fast, continuous and economic manner. Recent developments in techniques for shear wave velocity measurement to evaluate liquefaction potential will be demonstrated using the seismic cone penetration test (SCPT) and the surface wave technique (SASW). Limited case history data using the two techniques will be presented.

Sonic Drill

The sonic drill, which provides continuous core in most types of soil and rock will be demonstrated. A variable frequency vibrator is used to drive a drill casing. Fast drill rates are achieved by operating near the resonance frequency of the drill casing. The vibratory action causes the soil particles surrounding the drill bit and pipe to liquefy, thereby giving fast effortless drill rates. A continuous core with little disturbance of the stratification is obtained as the drilling progresses.

Pile Lateral Load Tests

A poster presentation describing recent full scale pile load tests conducted at this site will be given. The tests were conducted to examine the lateral deformation characteristics of the piles which simulate liquefied soil and very large strain conditions. Expanded base concrete, pipe, and timber piles were tested. Lateral deformations of up to 1 metre (and pile curvatures of 0.5 rad/m) were obtained in some of the tests.

8. George Massey Tunnel

Access to Richmond from the south is via the George Massey Tunnel and the Annacis Island bridge. The tunnel lies in a trench excavated in the river bottom of the South Arm of the Fraser River and is covered by about 6 metres of rockfill and sand. The tunnel is composed of reinforced concrete segments which are rectangular in section (24 metres by 8 metres) and about 105 metres long. These segments were constructed in dry docks, floated to location and sunk. The segments were then joined under water and pumped out. The sand which was jetted into position beneath the base of the tunnel and the underlying sand, together with the or sand fill at the sides of the tunnel could liquefy during a major earthquake. The concern is that liquefaction induced displacements of these soils could overstress the segments causing cracking and possible misalignments at the joints which could hinder traffic and result in severe leakage.

9. Roberts Bank Coal Port

The Roberts Bank Coal Port is located within the Municipality of Delta, in the Strait of Georgia, at the southwest of the Fraser Lowland.

Glacial pleistocene deposits are present in the Point Roberts Bluffs which rise to about 60 m above sea level and are visible southeast of the Port facilities. The glacial soils slope down from the beach at Point Roberts and are at a depth of about 100 m at the Port location. Within the last 10,000 years, Fraser River sediments consisting of sand and silt have been deposits over the glacial soils. The recently deposits sea bed slopes down gradually to form a large tidal flat that extends towards the Port. Immediately seaward of the Port, the slope of the sea bed is a maximum of about 10H:1V to a depth of about 60 m, and then becomes flatter, reaching depths of about 140 m near the centre of the Strait.

The Port consists of a 100 hectare man-made island that is connected to the mainland by a 4.5 km long causeway. The island was constructed using sand dredged from adjacent areas. In general, the island and the core of the causeway were formed using clean sand. Silty materials were used on the side slopes of the causeway. The thickness of the dredged sand at the island ranges from about 6 to 9 m. The settlement due to the weight of this sand fill has been more than 1 m. Side slopes are flatter than 10H:1V on the causeway, and as steep as 2.5H:1V on the island, depending on the requirements for ship berthing and erosion protection.

The Port development includes coal stockpiles as well as loading and conveyor facilities, with docks supported on both steel and pre-cast concrete piles more than 40 m long and 600 to 900 mm in diameter. The dumper pit construction included excavation and dewatering to a 20 m depth in the dredged fill. Geotechnical considerations associated with the development of the site include settlement due to the weight of fills and stockpiles, seismic stability as well as foundation bearing, pile capacity and slope stability.

10. LNG Tank

This project involved soil improvement for a new concrete impoundment dyke which encircles an existing LNG tank. The dyke requires protection against possible soil liquefaction.

The underlying soils generally comprise of approximately 6 m of sandy silt overlying 4.5 m of loose silty sands. Below depths of 10.5 m the sands become less silty (with silt contents of less than 5%) and are also somewhat coarser. The soils under the existing LNG tank had been compacted using timber piles driven to depths of approximately 15 m and terminated in medium dense, well-graded sand.

The client required sufficient safety against liquefaction under the proposed dyke from an earthquake with a mean recurrence interval of 10,000 years.

The solution involved the use of vibro-techniques to provide non-liquefiable, highly densified soil under the new impoundment dyke. In addition, this would improve lateral support to the soil under the existing tank during an earthquake. Treatment was required to a depth of 23 m below existing grade and extended 30 m from the perimeter of the tank.

11. New Westminster Waterfront Development

The development includes a two-storey public market building, three-storey townhouse/apartments, and 20-storey high-rises. Prior to development the site was relatively flat with existing old bulkheads and docks along the waterfront. The site is underlain by loose sand fill and interlayered fine sands and silts to approximately 12 m depth. Below this loose zone are medium dense to dense medium fine sands.

It was judged that the loose sands and silty sands in the upper 12 m were potentially liquefiable. From simple pseudo-dynamic analyses based on Newmark, and from review of the reports from the Niigata earthquake and others, it was judged that the existing bulkhead along the waterfront would fail into the Fraser River if the soils behind it liquefied and that the failure could possibly extend well back into the site.

Remedial measures involved densifying a zone along the waterfront using vibro-replacement procedures to prevent liquefaction and to develop sufficient shear strength to retain the soils on the landward side. Stone columns approximately 1 m in diameter were placed on a 2.75 to 3 m equilateral spacing. In addition to densifying the soil the stone columns increased the shear strength of the soil and provided drainage. The drainage would help prevent liquefaction by reducing the pore pressure rise during earthquake shaking. Two dimensional dynamic finite element simulation of the densified section during the design earthquake was

conducted to assist in assessing the adequacy of the design and the potential effect on structures adjacent to the waterfront.

The more economic dynamic compaction procedure was used back from the water edge to allow some of the smaller buildings to be founded on spread footings and to provide lateral resistance for the foundations. 20 tonne weights were used to give a compactive effort in the order of 3200 kJ/m^2.

Low-rise buildings within the development are generally founded on spread footing foundations bearing on the dynamically compacted soil. High-rise structures are founded on expanded base piles driven through dynamically compacted ground to bear on the underlying compact sands. The earthquake induced building base shear of the structures was transferred to the soil by the passive pressure of the soil against the perimeter basement walls and grade beams, and by the lateral resistance of the pile foundations.

DAMAGE SUMMARY

Based on experience during past earthquakes, the likely damage in the delta in the event of a major earthquake may be as follows:

- Most buildings in Richmond will suffer some damage due to uneven ground movements as a result of liquefaction of the underlying soils. Cracks in the ground surface will be common, and where such cracks run beneath buildings, severe damage will occur. Taller buildings are supported on piles and may suffer damage if the soil surrounding these piles liquefies. In the event that liquefaction does not occur, damage to high-rise structures may result from high lateral forces induced by resonance if they are not adequately designed.

- The highway system may be disrupted due to lateral spreading and fissuring, resulting in vertical faults of up to 0.5 metres. Liquefaction induced damage could occur for some of the older major bridges. In addition, the approaches may suffer some damage, and lateral spreading of the abutment fills may disrupt access to the bridges.

- The dykes will be severely damaged by cracking. Serious flooding could occur unless these are quickly repaired.

- The water, sewer, gas, electrical power and telephone services in the area will likely suffer severe damage.

- Fires will be initiated by the earthquake. The disruption of water lines together with damage to the transportation system will hamper fire-fighting so that fires may get out of control, and a conflagration is a possibility.

- A damaging tsunami or earthquake generated tidal wave resulting from a slump at the Delta face is a possibility.

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REFERENCES

1) Byrne, Peter M. and D.L. Anderson, Earthquake Design in Richmond, B.C., Version II, Soil Mechanics Series No. 109, Dept. of Civil Engineering, University of British columbia, Vancouver, B.C., Canada, 1987.

2) Clague, J., E. Naesgaard and A. Sy, Liquefaction Features in the Fraser Delta - Evidence for Prehistoric Earthquakes?, accepted for publication by the Canadian Journal of Earth Sciences.

3) Naesgaard, E., A. Sy and J. Clague, Liquefaction Sand Dykes at Kwantlen College, Richmond, B.C., First Symposium on Geotechnique and Natural Hazards, Vancouver, B.C., Canada, 1992.

4) Task Force Report, Earthquake Design in the Fraser Delta, Co-Chair: P.M. Byrne and D.L. Anderson; Soil Mechanics Series No. 150, Dept. of Civil Engineering, University of British Columbia, Vancouver, B.C., Canada, June 1991.