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SLOPE STABILITY OF NEMO AND WEE SANDY CREEK BASINS NEAR SLOCAN LAKE, BRITISH COLUMBIA

by

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ABSTRACT

In order to determine the possible impacts of forest engineering on landslide occurrence in four eastward-draining basins near Slocan Lake, southeastern British Columbia, slopes are evaluated according to a landslide hazard classification scheme based on natural terrain subdivisions, a stochastic geotechnical model, and past engineering experience. Mass wasting processes currently at work in the area include shallow debris avalanches, debris flows, rockslides and rockfalls, and involve complex glacial and colluvial deposits overlying coarse grained plutonic and high grade metamorphic bedrock. Primary factors known to influence landslide occurrence in the region include slope angle, soil shear strength, tree root strength, groundwater, and shear plane geometry. A stochastic geotechnical model can only be applied to uniform slopes mantled with surficial material because of inherent assumptions and requires quantitative estimates of parameters for broad slope units. Ranges are estimated for values of angle of internal soil friction, soil cohesion, root cohesion, piezometric head, depth to shear plane, soil bulk density, tree surcharge weight, and slope angle. From the model it is possible to explain the observed distribution of many landslides in the study area and surrounding region in terms of expected factor of safety and probability of failure. However, the probabilities cannot actually predict the number of landslides likely to occur on a particular slope, nor the likelihood of a landslide occurring

within a certain time period. Accurate, quantitative predictions of landslide occurrence can be made only where model variables are less subjectively determined, and where probabilities are calibrated and compared with observed events. These semi-quantitatively determined indices of stability are best used to compare the stability of slopes in the study area with slopes that have responded unfavorably to forest engineering in other areas. From such comparisons the indices can be grouped to form hazard classes of use to forest managers and engineers.

In areas where the geotechnical model does not apply, hazards are assigned according to past engineering experience in natural terrain units similar to those in the study area. These units include colluvial fans and aprons, debris fans, and steep rocky terrain. Slopes classed in the 'very high hazard' group include those slopes which show signs of active landsliding as indicated by morphology or vegetation, and steep rocky terrain dominated by gravity processes. Slopes classed in the 'high hazard' group include colluvial fans, upper parts of debris fans, and slopes mantled with surficial material having probabilities of failure greater than 10%. Slopes classed in the 'moderate hazard' group include lower parts of debris fans and slopes mantled with surficial material having probabilities of failure less than 10% but expected factors of safety less than 1.6. 'Low hazard' slopes include gently sloping exposed bedrock and slopes mantled with surficial material having expected factors of safety greater than 1.6. The landslide hazard classification scheme has practical merits for use in planning road alignments and logging systems.

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CHAPTER 1 INTRODUCTION

1.1 The Landslide Problem

Landsliding in its various forms is a dominant erosional process in removing and transporting soil and rock debris from steep mountainous slopes of the western Cordillera. As larger demands are placed upon the valuable forest resources of British Columbia, the steeper, more difficult watersheds are being developed. Various economic and environmental impacts resulting from such development are now demanding that slope hazards be evaluated and understood prior to development.

Past experience in various parts of the world indicates that both deforestation and road building may have marked impacts on landslide occurrence (Swanston 1974, Zeimer 1981, Froehlich, 1979, Dale and James 1977, and Takeda 1976). In the West Kootenay Region of British Columbia, the focus of this study, marked increases in stream sediment load have been linked with logging operations (Chamberlain and Jeffrey 1968). Such increases are attributed to soil disturbance from skidder logging systems which have led to both landsliding and surface erosion. Sedimentation in streams has a negative effect on fish populations and occasionally on local municipal water supplies.

In the Slocan Valley of the West Kootenay Region, a local study has documented numerous examples of both slope and stream degradation resulting from road building on steep slopes near stream channels (Slocan Valley Community 1974). Such occurrences are not unique to this region. In the Coast Mountains north of Vancouver, B.C., O'Loughlin (1973) estimated that logging roads are responsible for up to 47% of of landslides which run directly into stream channels. This can be attributed to the fact that in most areas main access roads are close to major streams.

Studies to the east of Slocan Valley indicate that severe soil disturbance may have an adverse effect on forest productivity, particularly at high elevations (Utzig and Herring 1975). Productivity reductions are greatest where soil disturbance is 'severe' or deep, that is, where (1) the forest litter, A-horizon, and a portion of the B-horizon are removed; (2) the soil surface is buried by .25 m or more of debris; or (3) the A and B mineral horizons are severely compacted. To date, the long-term effects of landsliding on forest productivity in British Columbia have not been assessed quantitatively.

Road damage caused by landsliding may not only damage the environment, but may also significantly increase road maintenance costs and costs incurred by transport delays. Forest companies are now realizing that either avoiding road construction on unstable slopes or engineering roads for them , though costly at the outset, is often to their advantage financially (Gardner 1979).

As landslides are becoming an increasingly widespread problem both financially and environmentally in British Columbia, the land manager is faced with the need for a more comprehensive understanding of the nature and extent of the problem. The three most commonly asked questions by land managers about potential landslides are, according to Burroughs (1980), "(1) Where are they?, (2)How bad are they?, and (3)What can be done about them?

In recent years, several methods of slope stability analysis have been developed for evaluating landslide potential in forested watersheds (Foggin and Rice 1979, Swanston 1980 and Simons et al. 1978). There are several limitations in these analyses: (1) the heterogeniety of landslide controlling factors in the natural setting leads to uncertainties in a stability analysis, (2) data collection is difficult in large inaccessible areas, (3) broad interpretations are often based on inadequate data, and (4) knowledge of the processes involved is usually incomplete. If these limitations are overcome and slope stability is determinable, assumptions as to the kind, intensity, and quality of engineering to be imposed upon the terrain leads to the assignment of relative hazard ratings. It is these hazard ratings that are of interest to the land manager.

1.2 Scope Of Study

This thesis attempts to evaluate the stability of slopes in four eastward-draining basins in the Valhalla Mountains west of Slocan Lake, southeastern British Columbia (117° 22'-38'W; 49° 54'-50° 01'N; see Figure 1.1). It includes the major drainage basins of Wee Sandy Creek and Nemo Creek and the two minor basins of Hoben Creek and Sharp Creek which drain the east-facing Slocan Lakefront between the two major drainages. The

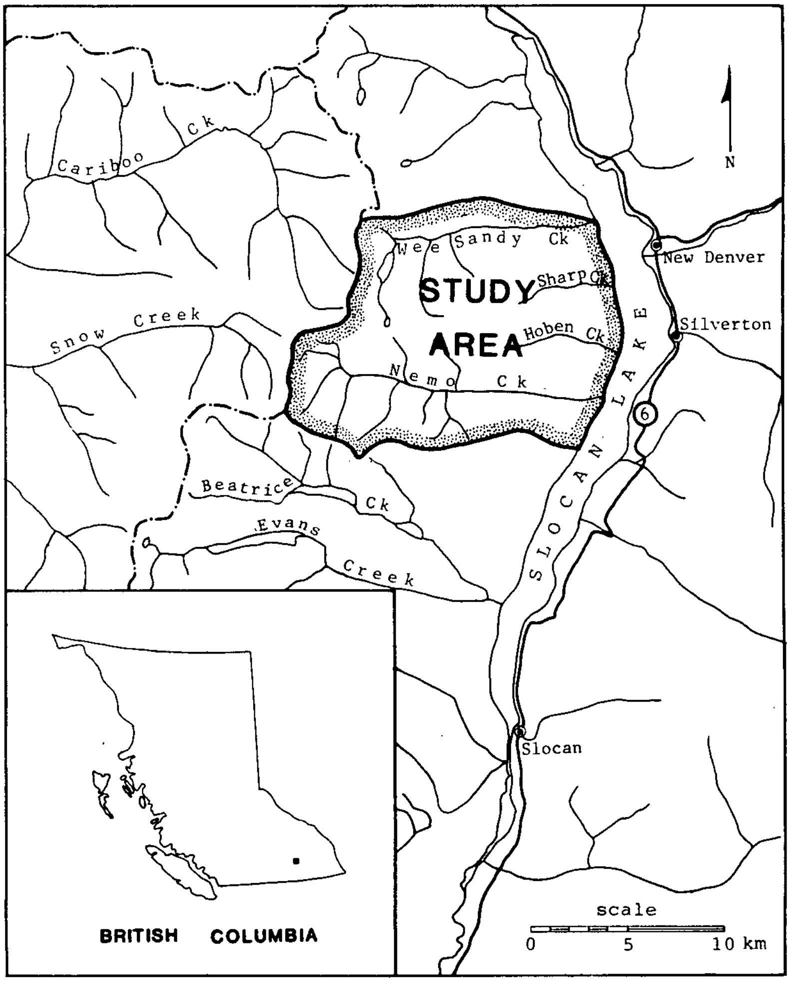

Figure 1.1 Index map.

area encompasses some 160 km² of which 66 km² have potentially harvestable timber. This area was chosen because it includes many steep, potentially unstable slopes, merchantable timber of value to the forest industry, and some high aesthetic-recreational values. Potential land-use conflicts in this area are of public concern and studies are needed to determine what impact, if any, logging operations will have on the environment. No logging or development has occurred in the study area for at least 30 years.

The first of two objectives is to determine, map, and describe the fundamental factors controlling the stability of slopes in the study area. The factors include geologic structure, soil properties, root strength, groundwater conditions, slide geometry and slope angle. This is the first step in assessing the reliability of conventional slope stability analyses in determining hazard ratings. This objective also includes the examination of factor interactions leading to natural instability.

The second objective is to determine how landslide controlling factors interact with particular engineering practices to produce landslides. Landslides caused by engineering activities on slopes similar to those of the study area are a basis for the hazard rating system in the study area. A rational method of landslide prediction is therefore based on stability analysis, the engineering behaviour of similar slopes in other areas, and certain assumptions as to the type and quality of engineering alterations to be imposed upon slopes in the study area. The final result is a map of slope stability hazards.

This study does not attempt to predict site-specific occurrences of landslides and is designed only to delineate and rate areas which are likely to produce slope stability problems. In some cases, failed slopes are described individually, particularly where they are critical to the watershed development scheme.

Potential problem areas associated with low-volume forest roads, as well as slopes that have or will be logged are emphasized. Primary and secondary highways are also examined but are not considered to be as relevant to forest engineering problems. All areas above timberline which constitute the divides between watersheds are included in the landform mapping but are less important to land-use conflicts, and are, therefore, not considered in detail.

Only landsliding, i.e. the downslope movement of rock, soil and debris under the influence of gravity largely independent of contributing forces from agencies such as flowing water or wind (Leopold et al. 1964) will be considered. Other geomorphic hazards, such as snow avalanching, will not be discussed. The term 'landsliding' will be used as an equivalent to 'mass-wasting' or 'soil mass-movement' forthwith, for simplicity.

1.3 Previous Work In The Study Area

Reconnaissance soil surveys were begun in 1980 by the Castlegar Forest District of B.C. Ministry of Forests (Ministry of Forests 1981a). Their preliminary report includes brief descriptions of the geologic environment and soil moisture regimes encountered during a two-day traverse along Nemo Creek. Remarks regarding slope stability and road building are made but are sketchy and of little use to this study. Wee Sandy Creek was not ground checked by the survey.

Tentative road locations in both Nemo and Wee Sandy Creeks were established in 1980 by the Nelson Regional Engineer of B.C.

Ministry of Forests. A report contains brief descriptions of potential slope stability problems along the proposed road alignments (Ministry of Forests 1981b).

CHAPTER 2 STUDY AREA DESCRIPTION

2.1 Physiography

The study area lies in the high, steep-walled, serrated, east-west trending ridges of the northern Valhalla Range of southeastern British Columbia. Local relief varies from 900 to 1350 meters, but the total relief is about 2200 meters and elevations range from 535 meters at Slocan Lake to 2743 meters at Mount Denver 6 kilometers to the west.

The upper end of Nemo Creek is dominated by Mount Meers to the north, Hela Peak to the south, and a series of cirque basins with floors at elevations between 1850 and 2150 meters. Lower Nemo Creek is flanked to the north by rugged cliffs, which rise some 1060 meters from the valley bottom. Nemo Creek has numerous small tributaries that enter from cirque basins on both the north and south sides of the upper valley and a few minor triburaries which feed from the straight, steep slopes of the lower valley. Valley geometry is typically U-shaped in the upper reaches but becomes dominately V-shaped in the lower valley. Figure 2.1 illustrates the U-shaped geometry of the upper Nemo Creek Basin.

The physiography of Wee Sandy Creek Basin is similar to that of Nemo Creek. The upper valley is U-shaped in cross-section but, remarkably, has no tributary cirque basins to the north. Wee Sandy Creek originates at Wee Sandy Lake which occupies a north-south trending hanging tributary valley at the

Figure 2.1. Aerial view of upper Nemo Creek Basin.

head of the basin. The lower valley again assumes a V-shaped cross-section, as does Nemo Creek, at the 1370 meter level. The steepest slopes and cliff faces are consistently found on the northern sides of both valleys.

Both Nemo and Wee Sandy Creek have relatively gentle stream gradients in the upper valley portions, which then steepen abruptly at mid-valley to descend via rapids and cascades at an average 15% gradient to Slocan to Slocan Lake below. One notable cascade 1 kilometer long occurs approximately 5 km up Wee Sandy Creek and has an average gradient of 27%.

Hoben and Sharp Creeks occupy hanging cirque valleys which drain into Slocan Lake between Nemo and Wee Sandy Creek. Both Creeks descend a series of glacially formed steps, some of which are occupied by small tarns, and then drop abruptly into the main Slocan Valley. Neither stream has incised significantly into bedrock and therefore have not developed V-shaped canyons as have Nemo and Wee Sandy Creeks. New Denver Glacier is perched at the head of Sharp Creek Basin is the only representative of once extensive valley glaciers.

2.2 Bedrock Geology

The study area is situated between two distinct geologic features: (1) the Slocan syncline to the north and (2) the domal Valhalla Gneiss Complex to the south. The Valhalla Gneiss Complex is centered at the core of the Valhalla Dome near Gladsheim Peak, approximately 15 km to the south, and includes the southern half of the study area (Parrish 1982 and Reesor 1965, see Figure 2.2). Foliations of the gneiss dome dip quaquaversally and are reflected by a series of inward facing cliffs which rise steeply to gently curving ridges entirely surrounding the central gneiss core. The high cliffs of lower Nemo Creek are an expression of northward dipping foliations of the Valhalla Dome incised by erosion. The north-facing valley slopes of Nemo Creek Basin more closely approach dip-slope geometry and are consequently less steep. Gneiss foliations dip between 15° and 25° to the NNE throughout the southern half of the study area and can be clearly observed on the headwall of the upper Hoben Creek cirque basin.

At the head of Nemo Creek, the monzonitic Nemo Lake Stock intrudes the gneiss complex. A mixture of metamorphic and plutonic rocks constitute a zone of "mixed gneiss" at the

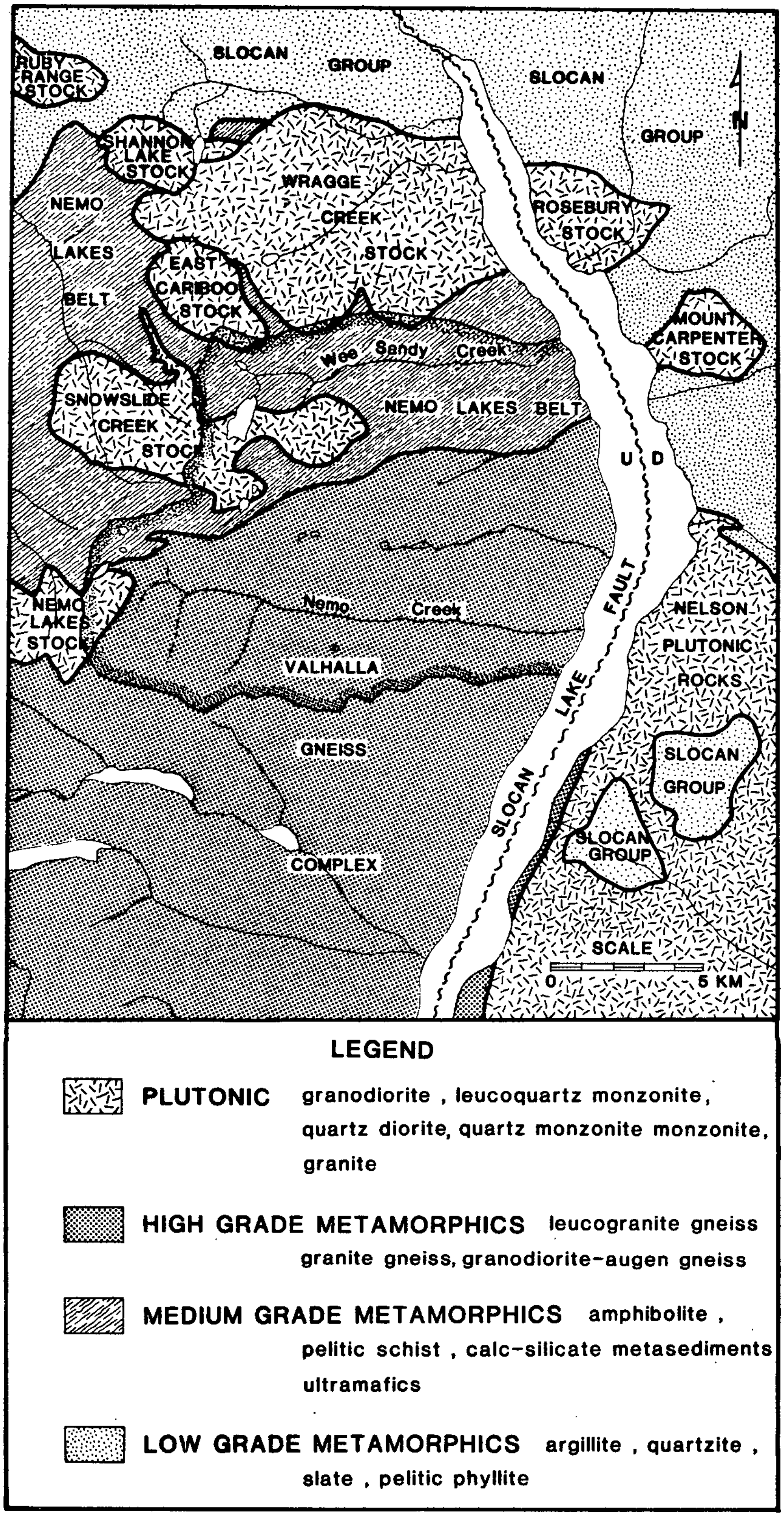

Figure 2.2 Geology of the study area.

intrusion boundary (Reesor 1965) Both the granitic and gneissic rocks are consistently coarse-grained and are of similar composition.

North of the study area, an assemblage of low to medium grade metamorphic rocks form the structurally complex Slocan Syncline on the northern boundary of the Nelson Batholith and Valhalla Dome. The topography formed on these mechanically weaker rocks is more subdued than that found farther to the south. A belt of medium grade intermediate metamorphic rocks, termed the Nemo Lakes Belt, is exposed principally in the drainage of Wee Sandy Creek and farther to the west. These rocks possibly grade continuously from the pelitic phyllites and slates to the north to the leucogranite gneisses to the south (Parrish 1982). The position of the contact between the pelitic schists and amphibolites of the Nemo Lakes Belt and the leucogranite gneisses of the Valhalla Complex of the south is uncertain but is occurs somewhere within the Sharp Creek drainage area.

The Wragge Creek Stock and East Cariboo stock lie immediately to the north of Wee Sandy Creek. These mechnically strong, coarse-grained granitic rocks have resisted glacial erosion and thus explain the absence of cirque basins on that side of the basin. The Snowslide Creek Stock, similar in composition to the Wragge Creek Stock, occurs to the west and south of upper Wee Sandy Creek Basin (see Figure 2.2).

The Slocan Lake Fault bounds the study area to the east and precludes the correlation of rocks across Slocan Lake (Parrish 1982). The granites (some porphyritic) of the Nelson Batholith lie immediately to the east of the Valhalla Gneiss Dome as does the Slocan Group (Little 1952).

2.3 Surficial Geology

The West Kootenay region has undergone multiple glaciations (Holland 1976) giving rise to complex distributions of genetic materials including morainal (basal and ablation), glaciofluvial, fluvial and colluvial deposits in the study area.

A main trunk glacier once occupied the Slocan Valley to at least the 1200 meter level during the last glaciation as evidenced by ice-marginal glaciofluvial deposits and glacial flutes between Hoben and Sharp Creek on east-facing slopes. The tributary glaciers occupying Nemo, Sharp, Hoben and Wee Sandy Creek Basins probably receeded up-valley prior to the disapperarance of the trunk glacier during final ablation stages, as seen in a modern example of the deglaciation stages of a similar valley in Alaska (Figure 2.3). Fluted knobs to the south of the confluence of Wee Sandy Creek with Slocan Lake suggest that a reentrant of the main trunk glacier entered into the lower Wee Sandy Creek Basin. Lower tributary valleys are dominated by both tributary and trunk glacial deposits.

Figure 2.3 Salmon Glacier, near Stewart, B.C. showing the retreat of a tributary glacier prior to the disappearance of the main trunk glacier. The tributary valley occupies a valley similar in geometry to that of both Nemo and Wee Sandy Creeks. (Photo taken by W.H. Mathews).

2.3.1 Morainal Deposits

Ablation morainal blankets and veneers[1] are abundant throughout the study area. Comminution of coarse grained bedrock in Nemo, Hoben and portions of Sharp and Wee Sandy Creek Basins has produced gravelly to sandy morainal deposits with silt

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[1]The term 'ablation' refers to the wastage of glacial ice by melting and evaporation leading to depostion of englacially and/or supraglacially transported debris, the term 'blanket' means a mantle of unconsolidated materials thick enough to mask minor irregularities in the underlying unit, but which still conforms to the general underlying topography (generally greater than 1 m thick), and the term 'veneer' means a layer of unconsolidated materials too thin to mask the minor irregularities of the underlying unit surface (between 10 cm and 1 m thick).

fractions generally less than 20% and only minor clay. Ablation morainal deposits in Wee Sandy Creek basin tend to have finer sand components resulting form comminution of finer grained schists. Figure 2.4 illustrates the obvious difference between

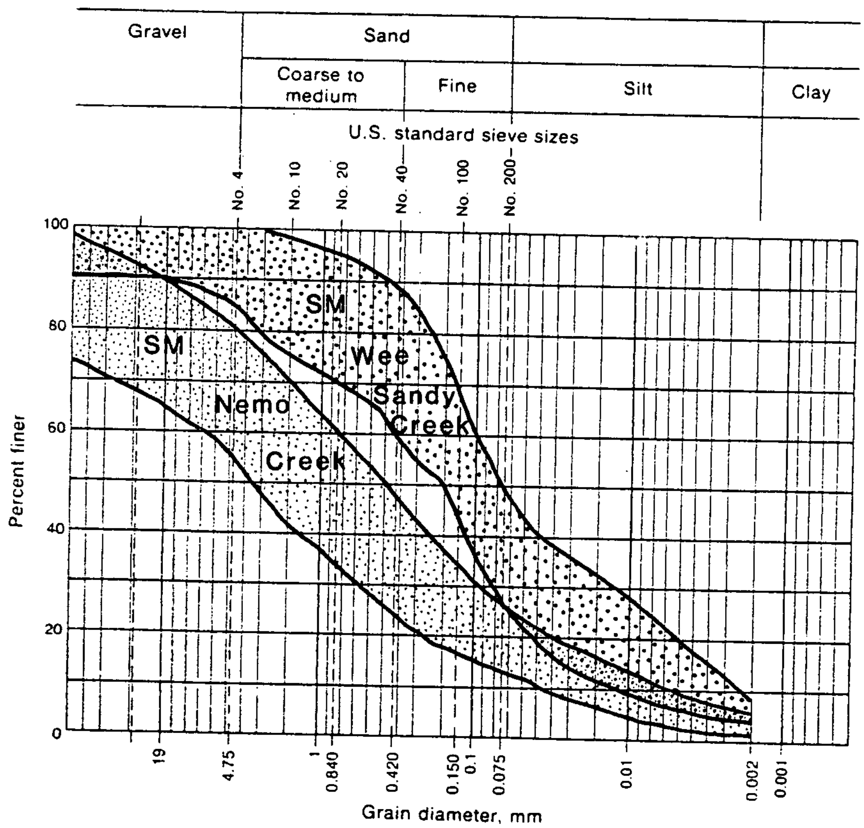

Figure 2.4 Grain-size distributions for morainal SM soils sampled in Nemo and Wee Sandy Creek Basins.

the grain-size distributions of morainal silty sand deposits (SM in the Unified Soil Classification) sampled in Wee Sandy Creek

Basin versus Nemo Creek Basin[1].

These materials, although widely distributed, were seldom observed in cross-section in gullies or on stream banks in the study area. Consequently, observations were mostly limited to soil pits where the lateral continuity or stratification of a deposit is not observable. Englacial and supraglacial materials within ablation moraine are complex and frequently include glaciofluvial lenses and pockets in other areas (Embleton and King 1968). In one instance, a marked variation in texture was observed within 1 meter laterally where a tree had overturned exposing underlying ablation moraine associated with a glaciofluvial pocket (see samples N19-1and N19-2 in Appendix A). Such complexities make positive identification of ablation moraine problematic as most observations are limited to soil pits.

Morainal deposits associated with cirque basins at higher elevations are typically blocky or rubbly with fewer fines and in places exhibit terminal of lateral moraine morphology. In come cases, talus aprons grade gradually into ablation moraine near cirque basin walls. A lobate rock glacier exists in one north-facing cirque in Nemo Creek Basin. Lateral or terminal moraines are rare in the lower valleys, as are kame terraces.

Compact basal morainal deposits are observed only where

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[1]The percentages of coarse fragments greater than approximately 2 cm were estimated in the field and discarded from the sample. Samples were then air-dried and sieved with U.S. standard mesh sieves according to ASTM D1140-54 specifications. Size fractions less than .425 mm (# 40 sieve) were determined by the standard hydrometer method (Bowles 1978). Sample locations are shown on Map A (filed separately).

morainal blankets associated with the main trunk glacier are deeply incised by stream erosion. Sample N-0+80 taken from near lower Nemo Creek yields percentages of silt and clay of 43% and 12% respectively. Unlike ablation moraine in the area, these compact deposits can in places include pockets of pure clay.

In general, ablation moraine and glaciofluvial materials blanket underlying basal moraine. This may explain the surprising absence of observable basal moraine in the study area.

2.3.2 Glaciofluvial Deposits

Glaciofluvial deposits are characterized by rounded to subangular, well sorted sands and gravels with little silt or clay. These deposits occur throughout the study area but are particularly common on the east-facing slopes of the main Slocan Valley. Morphologic features associated with these deposits include small terraces, ridges, blankets, and veneers.

Glaciofluvial deposits in places grade into, or are mixed with, ablation morainal materials of similar texture and angularity, and can be distinguished only by the absence of silt and clay. Relatively short transport distances within the tributary basins frequently result in subangular cobbles and gravels associated with ice-marginal or englacial sorting by glacial meltwater.

2.3.3 Fluvial Deposits

Fluvial deposits are characterized by well to moderately well sorted sands and gravels associated with present-day creeks and streams on flat or terraced floodplains and fans. These deposits are usually confined to narrow floodplains and small fans within 50 m of active streams and are areally limited.

2.3.4 Colluvial Deposits

Colluvial deposits are characterized by poorly sorted, blocky to rubbly materials on steep slopes overlying bedrock or accumulated on or at the base of slopes by gravity-induced movement. These deposits occur throughout the study area and dominate the steeper terrain of higher elevations where they are mostly derived from bedrock. The blocky, rubbly texture characteristic of colluvium is strongly influenced by the composition and competence of the granites, gneisses and schists from which it is derived.

Colluvium occurs most frequently as veneers and/or blankets mantling steep terrain in excess of 30° on upper slopes. Thick fans and aprons are common along the toe slopes of steep rock escarpments at any elevation. Colluvium derived from bedrock along the upper parts of valley sides in places overlie ablation moraine or glaciofluvial deposits.

2.3.5 Weathering

Weathering by biological and physical agents has altered only slightly the near-surface physical properties of surficial materials. Humo-ferric Podzolic soils are common throughout the area where inactive geomorphic processes allow soil development. These soils typically occur in coarse to medium textured, acid parent materials, under forest or heath vegetation in cool to very cold humid to perhumid climates (Canada Soil Survey Committee 1978).

Humo-ferric Podzols were never observed deeper than 1 m and Podzolic soil development has had little effect on the bulk physical characteristics of surficial materials in the area.

Mechanical weathering of coarse lithic fragments in glacial materials was occasionally observed in Wee Sandy Creek Basin where mica-rich schists have broken down, in places preferentially to more gneissic rock fragments. However, most surficial materials have not been significantly weathered since deposition.

2.4 Geomorphic Processes

Geomorphic processes including debris avalanches, debris flows, rockslides, rock falls, snow avalanches, water-born erosion and flooding are active throughout the steep, recently glaciated terrain of the study area. Each type of process has played a role in modifying slope morphology since the last glaciation. The following discussion will be limited to those processes which directly affect or are themselves affected by the activities of man.

2.4.1 Debris Avalanche - Debris Flows

Debris avalanches are rapid, shallow failures from steep slopes involving sliding, bouncing and rolling of cohesionless surficial material along a relatively impermeable, mechanically strong bedrock or compact till shear surface (Swanston 1979 and Burroughs 1980).[1] When the surficial material is nearly saturated at time of failure, debris avalanching may revert to debris flowage resulting in the rapid downslope transport of a slurry of soil, rocks, and organic debris directly to stream channels. The combined term 'debris avalanche - debris flow' is used where debris avalanches are initiated by partial soil saturation and almost immediately revert to debris flows after initial failure.

In the study area, debris avalanches occur on long uniform slopes with continuous blankets or veneers of surficial material, on steep gully side walls undercut by recurrent debris flows or continuous surface erosion, or on steep terrace faces adjacent to stream channels. The largest avalanches occur on long uniform slopes where ample supplies of material are available for transport. Figure 2.5 is an example of a major debris avalanche on a north-facing slope with a smooth

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[1]This definition of debris avalanche includes the landslide type commonly refered to as 'debris slide' by Varnes (1958,1978). The distinction is problematic as the two processes are closely related and virtually indistinguishable in many environments (Blong 1973).

unweathered granite gneiss shear surface inclined at 30° in lower Nemo Creek Basin.

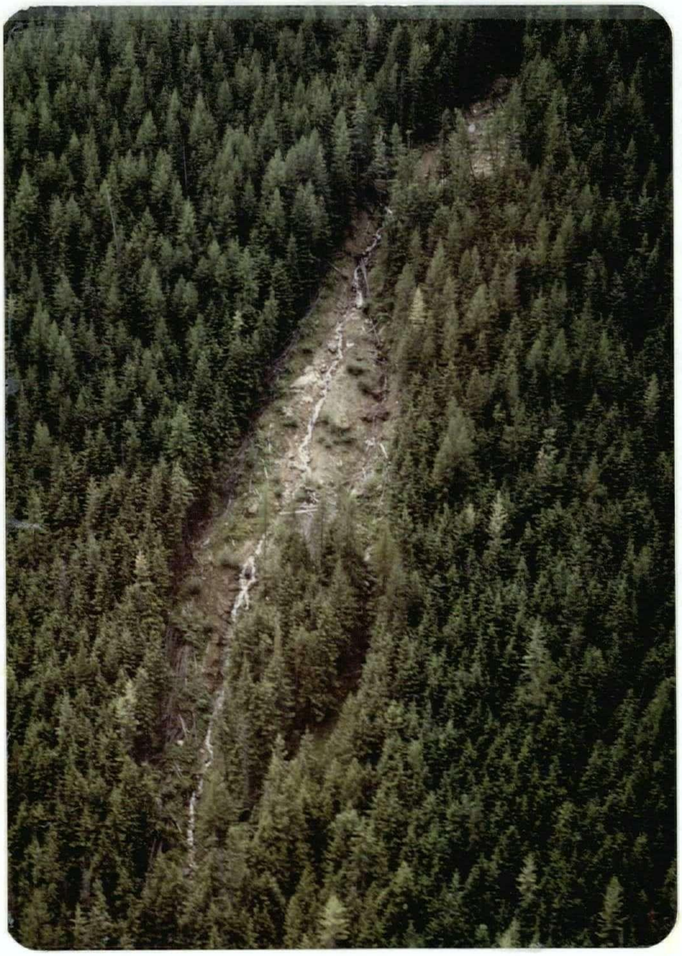

Figure 2.5. Debris avalanche on a north-facing slope of the lower Nemo Creek Basin.

Many debris avalanche - debris flows occur adjacent to gullies and are subsequently confined to a previously scoured channel. With time, numerous small landslides may accumulate significant amounts of debris in the gully bottoms only to be mobilized later by a major debris flow from above or by excessive storm flow during an extreme storm event.[1]

Debris flows initiated by small spoon-shaped debris avalanches in linear depressions are more common than simple planar debris avalanches. South-facing slopes of lower Wee Sandy Creek Basin inclined between 30° and 40° show evidence of numerous V-notch gullies which have, at some time, given rise to debris avalanche - debris flows. These chutes can be distinguished from ordinary erosional gullies by the presence of levee deposits on debris fans at the gully mouth and/or large transported boulders on gully side walls or in the channel. These gullies may also serve as avalanche paths when they develop in alpine areas.

Many gullies have not had debris flows for at least 150 years as indicated by old growth forest stands growing on levee deposits and in gullies. Figure 2.6 is an example of an old debris avalanche - debris flow path subsequently reforested in lower Nemo Creek Basin.

The relative dormancy or activity of a debris flow is determined by (1) the area of catchment basin feeding into the gully, (2) the stability of slopes in the debris source area, (3) the gradient of the gully where the flow gains destructive momentum and (4) the size and gradient of the debris fan at the gully mouth (Eisbacher 1982). On the north-facing slopes of lower Wee Sandy Creek, source materials for debris flows are derived largely from glaciofluvial and morainal blankets. Figure

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[1]Such an occurrence is refered to by many authors as a 'debris torrent' (Wilford and Schwab 1982 and Miles and Kellerhals 1981).

Figure 2.6 Debris avalanche - debris flow path subsequently reforested on the north-facing slope of lower Wee Sandy Creek Basin.

2.7 is the plan view of a particular gully network incised into a glaciofluvial blanket that serves as a debris source area for a recurrent debris flow system. The size of the debris fan at the base suggests that this debris flow system has been only slightly active since the last glaciation. There is little evidence of fluvial erosion at the toe of the fan.

Debris fans at the mouths of gullies on north-facing slopes of both Nemo and Wee Sandy Creek Basins are relatively small in relation to the fans on the south-facing slopes. These larger fans develop from more frequent debris flows originating in long, linear, rock-walled gullies inclined in excess of 40° on steep rock cliffs and benches where colluvial materials

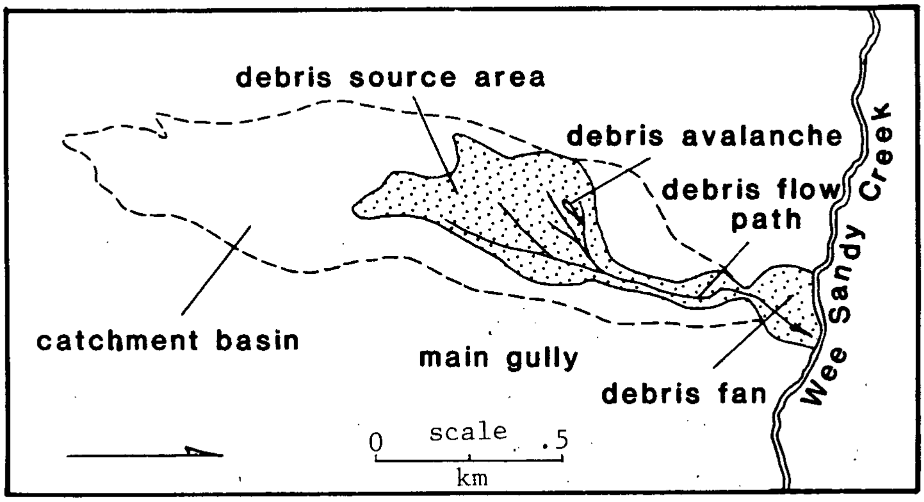

Figure 2.7 Debris flow system on the north-facing slope of the lower Wee Sandy Creek Basin showing the debris source, the main gully, and the debris fan.

accumulate. Heavy rainfalls, perhaps coupled with rapid snowmelt, periodically flush the accumulated debris from gullies resulting in deposition on the debris fan. Colluvial materials on the south-facing slopes are continuously accumulating and serve as excellent debris source areas. Figure 2.8 shows a recent colluvially derived debris flow bifurcated on a debris fan. This particular flow continued to transport debris on slopes as low as 6° into Nemo Creek at the toe of the fan. Figure 2.9 is a diagram showing a typical profile of a debris flow path measured on the south-facing slope of Nemo Creek Basin and the size of the largest boulder deposited on each segment of the fan. Debris fans may have toe slopes as low as 5° or as high

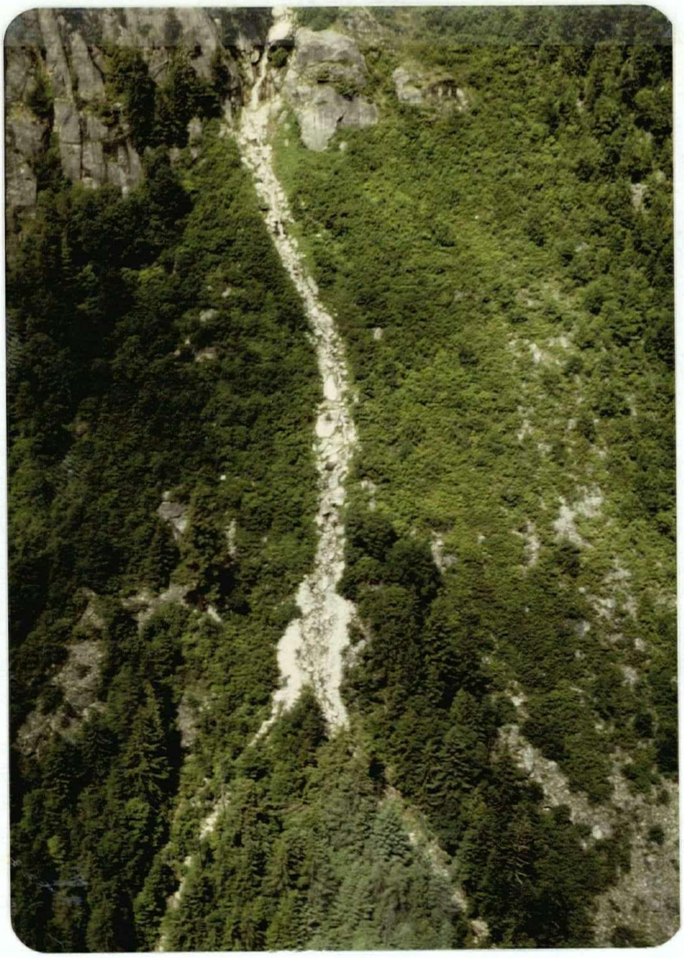

Figure 2.8 Debris flow that has bifurcated on a debris fan of upper Nemo Creek Basin.

as 20° depending on the width of the valley into which the flow descends. The texture of a debris fan is similar to that of a colluvial apron or fan.

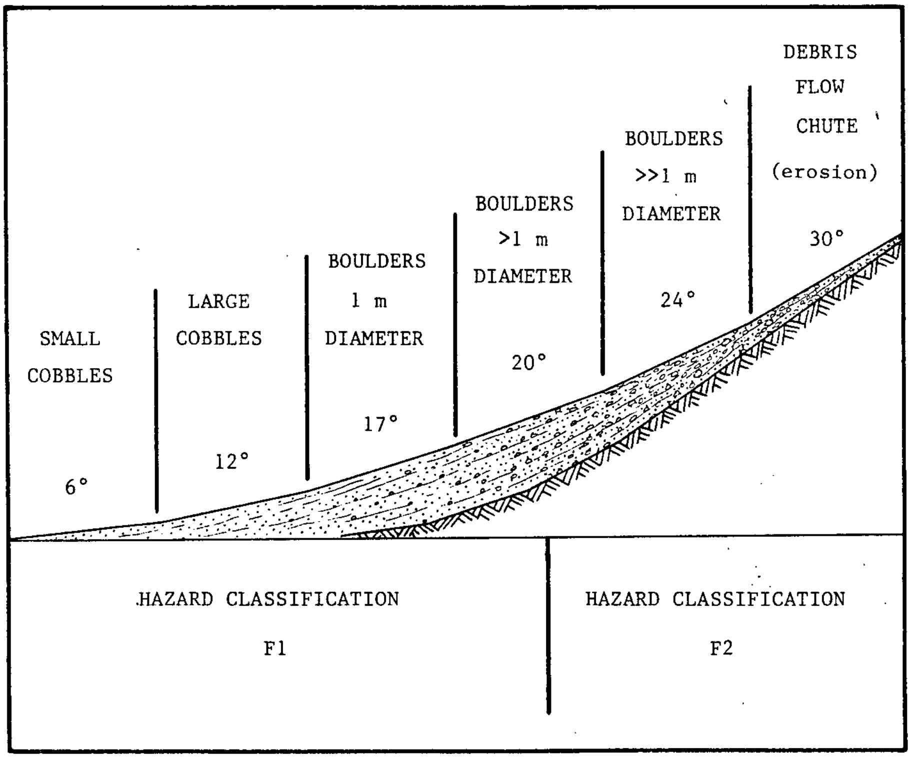

Figure 2.9 Profile of a debris flow on a debris fan in upper Nemo Creek Basin showing the sizes of the largest rock fragments deposited as levees along the flow path. Included are the hazard subdivisions for debris fans discussed in Chapter 5.

2.4.2 Rockslides

A rockslide is a rapid downslope movement of rock, either as an incoherent mass or as a large unbroken block detached from bedrock, and may have either a curvilinear or planar shear surface, depending on the nature and orientation of controlling joints, bedding planes, foliations or other discontinuities. Several large, deep-seated, rotational rockslides were identified on the north-facing slopes of both Nemo and Wee Sandy Creek Basins. These slides are characterized by well defined arcuate headwall scarp areas associated with altered valley-side forms below. Near lower Nemo Creek, the toe of a large rotational rock slide is undergoing toppling failure as evidenced by deep cracks shown in Figure 2.10. It is supposed

Figure 2.10 Toppling rock failure in lower Nemo Creek Basin.

that northward dipping foliations, coupled with glacially oversteepened slopes are contributing to these slides. They are large enough to constitute mappable rock units, but because they either (1) retain their mantle of original surficial materials or (2) form blocky colluvial slopes similar to other non-sliding areas, they have not been mapped individually. Small rockslides are confined to areas of steep rock and colluvium where failure is controlled by exfoliation jointing and perhaps initiated by frost-wedging or seismic activity.

2.4.3 Rockfalls

Colluvial aprons and fans at the base of virtually all steep rock cliffs attest to the frequency of rockfalls in the study area. The wide joint spacings and high competence of these metamorphic and plutonic rocks result in the detachment of large blocks several meters in diameter. In general, rockfalls are known to be most frequent during earthquake events and during freeze-thaw periods. No falls were observed during the course of this study.

2.4.4 Erosion

Erosional processes are closely associated with mass-wasting processes as both are mutually interdependent. Erosion may produce local slope oversteepening which increases instability which, in turn, contributes material to be further eroded. Gullies formed solely by surface erosional processes occasionally occur throughout the study area but are most commonly associated with sandy glaciofluvial terraces and deeper morainal blankets. Gravelly-to-rubbly, shallow, well-drained soils have retarded natural erosional processes in most other areas.

2.4.5 Soil Creep

Trees tipped or bowed along their entire length, indicative of incipient landsliding or soil creep, were rarely observed in the study area. The non-viscous properties of sandy to gravelly soils have limited soil creep processes to those associated with the incremental movement of discrete particles accelerated by tree rooting and tree overthrow. Only on slopes inclined in excess of 35° was strong evidence of soil creep observed. Figure 2.11 shows the buttressing effects of a tree on a 40° slope subject to soil creep and perhaps some minor sloughing.

2.4.6 Snow Avalanching

Linear scars and vegetation patterns on steep forested slopes that display sharp trimlines indicate that snow avalanches frequently occur in the area. Many avalanche paths reach the valley bottom, particularly on the south-facing slopes of both Nemo and Wee Sandy Creek Basins. Snow avalanches most commonly start in steep rocky terrain, then become concentrated in chutes or gullies. Colluvial aprons or veneers, and/or debris fans usually serve as run-out zones.

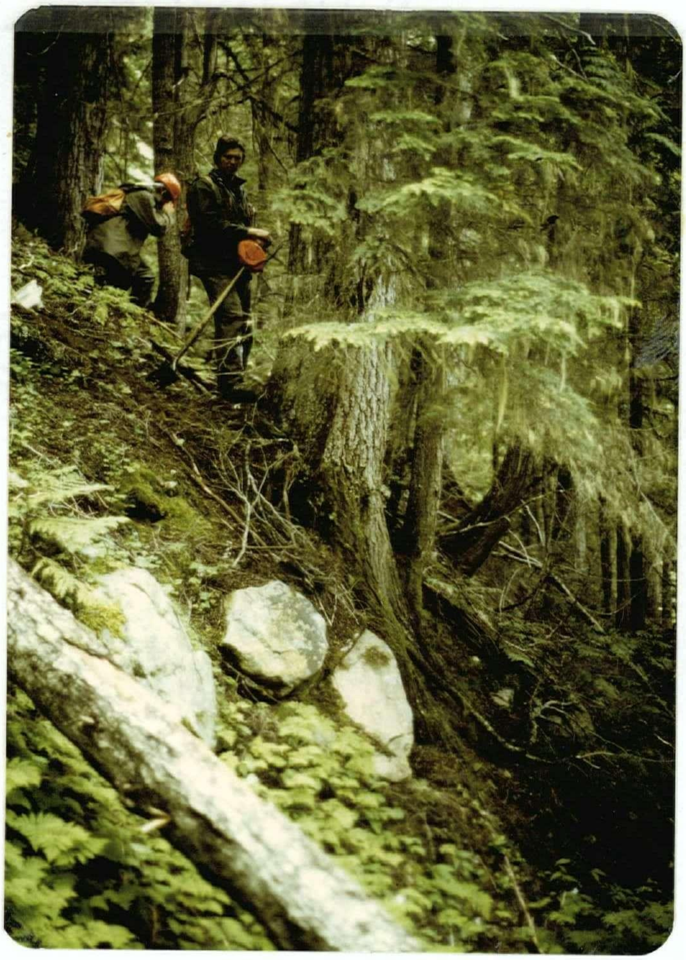

Figure 2.11 Buttressing effect of tree roots resisting soil creep.

2.5 Climate

The southern Selkirk mountains are influenced by both maritime and continental air masses. The climate is predominately moist at lower elevations, e.g. 574 mm/yr at Fauquier (elev. 472 m), and increases to wet at upper elevations, e.g. 1055 mm/yr at Sandon (elev. 1067 m). Localized variations in regional weather patterns are significant in the mountainous terrain of the study area and are influenced by local aspect, elevation, relative topographic position, and the effects of local bodies of water or ice.

The seasonal variations in total monthly precipitation for 5 local weather stations from 1941 to 1970 (see Figure 2.12 for station locations) indicate that precipitation patterns in

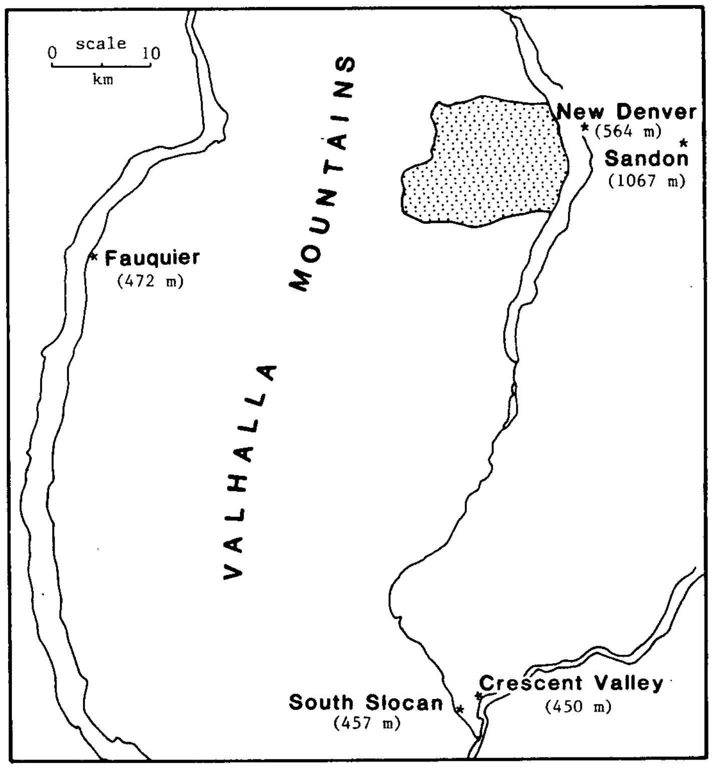

Figure 2.12 Index map showing locations of principal weather stations.

Slocan Valley proper are similar from station to station at the lower elevations of the valley bottom and that precipitation increases significantly with elevation in the New Denver - Sandon area (see Figure 2.13). Extreme 24-hour precipitation intensities between 1941 and 1970 are shown in Figure 2.14 for New Denver, Fauquier and Sandon. At all stations, the most

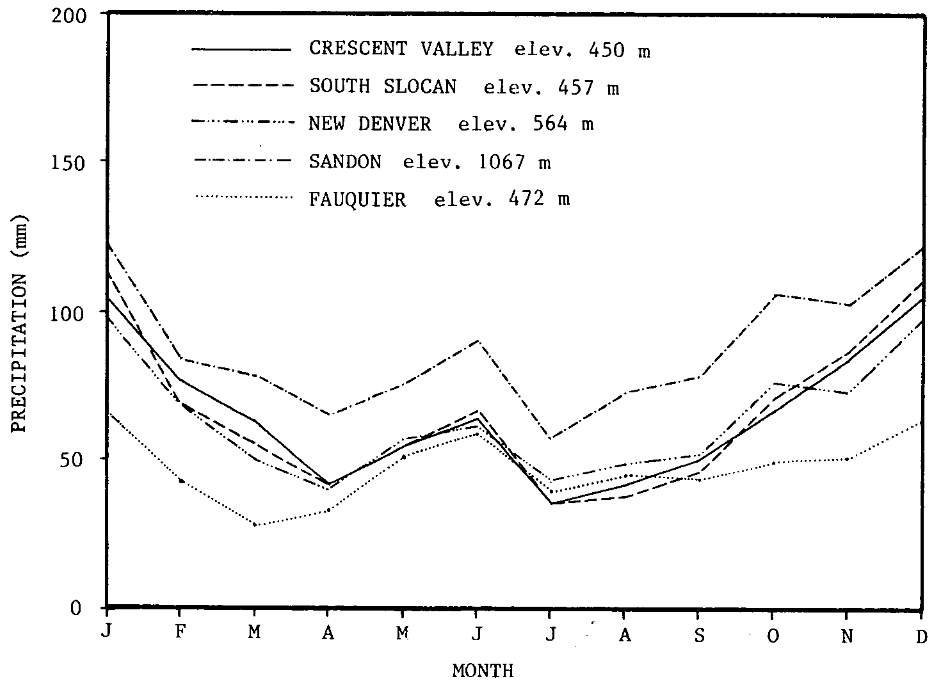

Figure 2.13 Mean total monthly precipitation for selected weather stations near the study area (Air Studies Branch, B.C. Ministry of Environment).

intense storms occurred during the months of June and September with intensities ranging between 41 and 56 mm/day. The unstable air patterns which dominate the summer season give rise to large thunder cells that affect localized areas only. The random nature of these storm events precludes the determination of any areal distribution pattern. However, it has been suggested that summer thunderstorms may increase in frequency and intensity at upper elevations (Utzig 1978). The 48-hour precipitation extremes for New Denver are given in Figure 2.15 showing a maximum of 77 mm per 48-hour period.

Rainfall occurs during every month of the year while snowfall is limited to November through April. Unfortunately, no temperature data is available for the New Denver - Sandon area.

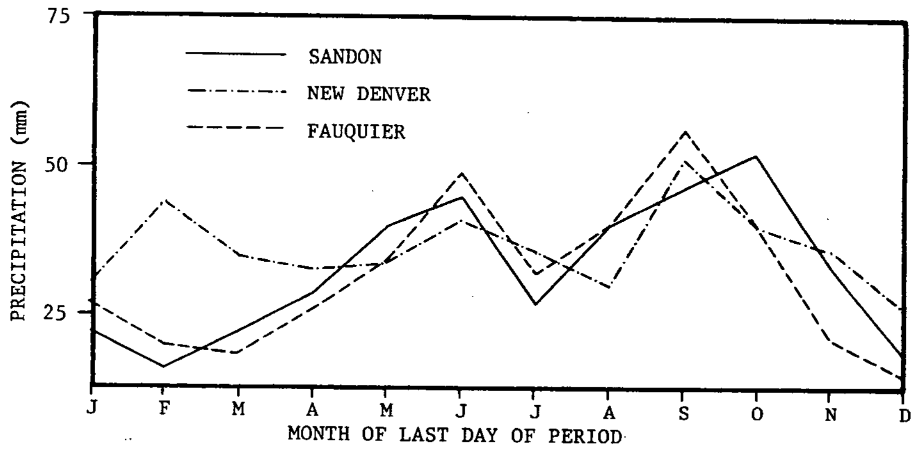

Figure 2.14 Twenty-four hour precipitation extremes for New Denver, Sandon and Fauquier from 1924 to 1979 (Air Studies Branch, B.C. Ministry of Environment).

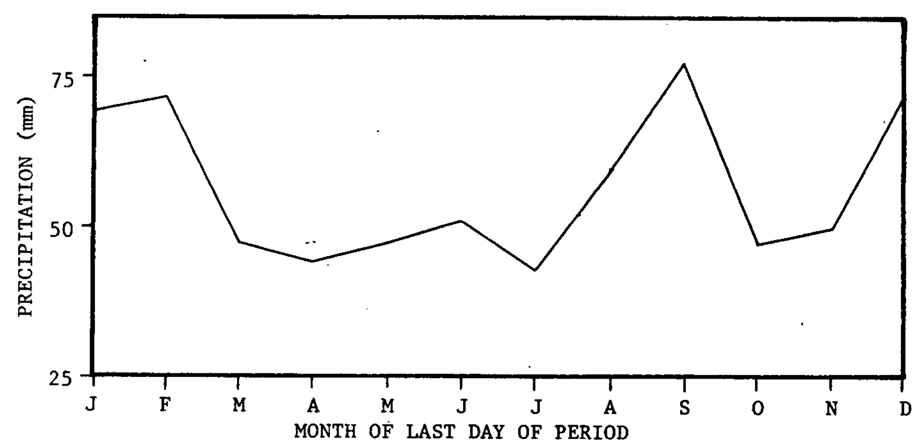

Figure 2.15 Fourty-eight hour precipitation extremes for New Denver, B.C. from 1924 to 1979 (Air Studies Branch, B.C. Ministry of Environment).

2.6 Vegetation

The study area is dominated by the Engelmann Spruce - Subalpine Fir (ESSF) and the Interior Western Red Cedar - Western Hemlock (ICH) Biogeoclimatic Zones according to Krajina and Brooke (1969). The ESSF zone occurs above the 1400 meter level on the upper slopes of all basins within the study area. The ICH zone dominates lower slopes, particularly those near Slocan Lake. Timberline is at approximately 1700 meters.

Plant and shrub communities are often indicative of certain climatic and hydrologic variables. Some species are restricted to a particular habitat while others occupy a broad range of habitats. The dominant plant and shrub species occurring on drier, well-drained soils throughout the study area include Paxistima myrsinites , Vaccinium membranaceum, Spiraea betulifolia , Shepherdia canadensis , Mahonia aquifolium , Chimaphila umbellata , Linnaea borealis , Fragaria vesca , Clintonia uniflora , and Goodyera oblongifolia . In somewhat wetter areas where water is not removed from the soil quite as rapidly, Gaultheria ovatifolia , Vaccinium membranaceum , Pyrola chlorantha , Tiarella unifoliata , Cornus canadensis , Gymnocarpium dryopteris , and Smilacina racemosa become more prevelant. In areas where the groundwater table approaches the soil surface during certain times of the year, Oplopanax horridus , Ribes lacustre , Rubus parviflorus ; Streptopus amplexifolius , Athyrium filix-femina , Equisetum arvense and Carex disperma are more abundant. A complete description of plant associations found within the ICH and ESSF biogeoclimatic zones is found in Utzig et al (1978). In areas where the forest canopy in particularly dense, understory vegetation can be sparse.

Geomorphic processes play a major role in determining vegetation cover. Those slopes influenced by snow avalanching and colluvial activity support shrub communities only, while those subject to only periodic disturbances permit mature forest development. Forest cover is typically dense where elevation limitations, thin soils, or geomorphic processes do not inhibit tree growth. Variation in forest cover type is largely governed by climate and available moisture. In certain areas *Pinus contorta* (lodgepole pine) and *Pseudotsuga menziesii* (Douglas fir) stands were found to prefer sunnier, drier south aspects at lower elevations. Otherwise, forest type varies largely with temperature - moisture changes associated with elevation.

CHAPTER 3 SLOPE STABILITY IN THE STUDY AREA

3.1 Approaches To Slope Stability Assessment

Despite recent advances in slope stability analysis and knowledge of soil and rock properties, the complexity and heterogeneity of most natural slopes prevent accurate determination of stability conditions and a complete understanding of causes of failure (O'Loughlin 1981). Soil mechanics theory, applied to site specific analysis of stability, is accurate in assessing the strength-stress relationships in a small area. However, past experience has demonstrated that this technique requires considerable geotechnical expertise, accurate measurement of the engineering properties of soils involved, and a specific knowledge of the geology and groundwater hydrology at a site. Where the slopes of an entire watershed are being analysed, such techniques are costly and impractical. Consequently, a number of approaches to analysing factors controlling slope stability at the reconnaissance level have been developed by various researchers.

Remote sensing coupled with pattern recognition techniques have been used to examine terrain for features distinctive of landslide hazards in many areas (Foggin and Rice 1979). This approach is effective if landslide hazards manifested in surface characteristics can be photographed or otherwise detected. A second approach, involving empirical models developed through statistical analyses of measurable field and photogrammetric data, attempts to provide a numerical index of slope stability (Furbish 1981 and Pillsbury 1976). Multiple regression and discriminant function analyses are common techniques for developing such relationships. A more common approach is the stability factor technique. Factors related to landslide occurrence are individually delineated on separate maps then superimposed. Factor combinations found to be coincident with known landslide hazards on the map are identified. Identical factor combinations in other areas on the map are then assigned an appropriate hazard rating. Computer techniques are now available that facilitate manipulation and weighting of factors so that hazard maps can be quickly produced from digitized factor maps (VanDriel 1980).

Unfortunately, each of these approaches necessarily makes gross assumptions about the operation of the physical system leading to landsliding. Even in statistically rigorous studies, assumptions are often implicit in the analysis. This has a number of consequences. Evaluations can be expected to be operator dependent due to the difficulties in weighting the importance of various classification indices. If the weighting is wrong or if certain factors are neglected, eg. loss of root strength following harvesting, the resulting evaluation may be very inaccurate. Beven (1981) points out that, "Without some underlying physically based structure on which to base this type of (slope stability) analysis, we may be very wrong in interpreting the success or failure of our evaluation....whenever possible, we should base our models on well-defined physical principles, rather than on empirical relationships that, to a large extent, obscure cause and effect relationships."

Recently, geotechnical models that take into account variability of soil properties and groundwater conditions have been developed in order to extend physical principles of stability to landslide hazard mapping systems (Simons et al. 1978 and Wu and Swanston 1980). Slope equilibrium can be determined in terms of 'expected factor of safety' and 'probability of failure', depending on the amount of uncertainty involved in either the measurement or estimation of slope properties. This approach has the advantage of being more easily transferred from one region to another, but like the other approaches, has several assumptions limiting its universal applicability. Moreover, what at the outset may seem like a completely objective method may actually involve many subjective estimates of model variables that render it as subjective as any other approach. A physical model does, however, provide the basis for recognizing where man has least understanding and opens the way for further studies which can strengthen model weaknesses.

This study is a first attempt at applying a physically-based model to landslide hazard mapping in British Columbia. It is hoped that, if proven successful, the technique will be useful in other parts of the province.

3.2 The Stochastic Geotechnical Model

In simplest terms, a landslide occurs if the shear stress acting on the slope is greater than or equal to the shear strength or internal resistance to shear stress of the slope material. The factor of safety (FS) of the slope is the ratio of the shear strength (s) to shear stress (τ), so that

$$FS = s/\tau \tag{3.1}$$

In traditional soil mechanics, soil shear strength is represented by the Coulomb equation

$$s = C' + \sigma' \tan \phi' \tag{3.2}$$

where c' is effective[1] soil cohesion, σ' is effective normal stress, and ϕ' is effective angle of internal friction. However, in forested watersheds, factors including root cohesion and tree surcharge weight should aslo be included in equation 3.2 (Brown and Sheu 1975 and O'Loughlin 1973). This has lead to an expanded version of the shear strength equation represented by

$$s = C' + Cr + H \cos^2\beta \; [qo/H + (\gamma sat - \gamma wet)M + \gamma(1-M)] \tan\phi' \tag{3.3}$$

for forested slopes as formulated by Simons et al. (1978).

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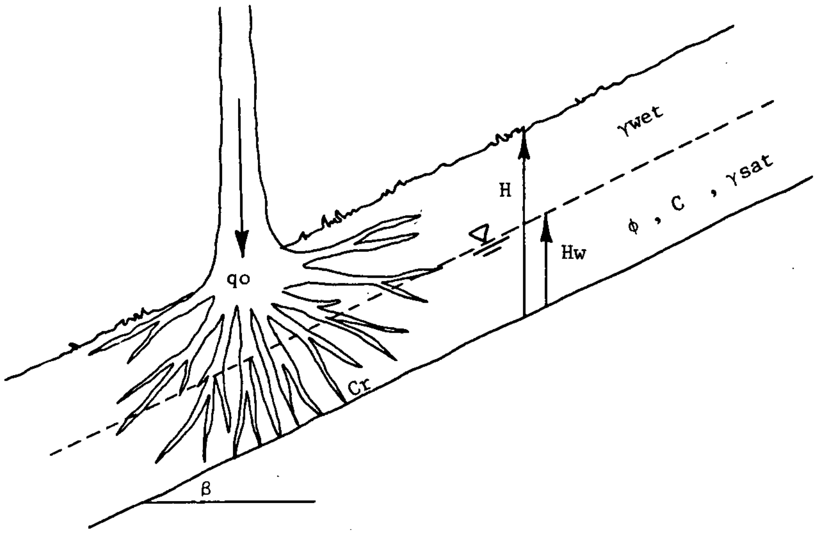
[1]The term "effective" refers to measurements that take into account pore water pressure effects.

Symbols are defined according to Figure 3.1.

Similarly, the simple version of shear stress due to the tangential component of gravitational stress along the basal



β = slope inclination

ϕ = range of angles of internal friction of soil

C = soil cohesion

Cr = root cohesion

qo = tree surcharge load

γwet = unit weight of soil

γsat = unit weight of saturated soil

γ = unit weight of water

H = height of soil mantle above shear surface

Hw = height of water table above shear surface

M = Hw/H

Figure 3.1. Definitions of input variables used in the deterministic geotechnical model.

zone of sliding is expressed as

$$\tau = W \sin\beta \tag{3.4}$$

where β is slope angle and W is weight of the soil mass above the shear plane; but with the addition of the effects of tree surcharge load and groundwater, the equation expands to

$$\tau = H(qo/H + \gamma sat(M) + \gamma(1-M)) \sin\beta \cos\beta \tag{3.5}$$

It is assumed in this equation that the soil is homogeneous and isotropic, the piezometric surface is parallel to a planar shear surface, and that the effects of wind stress are negligible. If equations 3.3 and 3.5 are divided directly to determine factor of safety, it is also assumed that the slope is planar and semi-infinite.

As discussed in Chapter 2, much of the study area has only thin deposits of surficial materials subject to shallow planar-type failures on a bedrock shear surface and debris flows are generally confined to narrow gullies that cross the broader slopes. Many of the slopes generally satisfy the geometrical assumptions of the infinite slope model.[1] In its expanded form, the infinite slope model is expressed as

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[1]It may be noted that glacial deposits in the study area are generally inhomogeneous. This difficulty is partially addressed by Lumb(1970) who found that an inhomogeneous, multilayer soil can often be represented as a homogeneous soil if the soil properties are averaged properly.

$$FS = \frac{c' + Cr + H\cos^2\beta\,[qo/H + (\gamma sat - \gamma wet)M + (1-M)]\tan\phi'}{H(qo/H + \gamma sat\,(M) + \gamma(1-M))\sin\beta\cos\beta} \tag{3.6}$$

The model includes terms for soil (C', γsat, γ, ϕ'), vegetation (Cr, qo), topography (β) and groundwater (M) as shown in Figure 3.1.

As with all mathematical models, analysis of model sensitivity helps define which values must be carefully collected and which can be roughly estimated. Realistic ranges of values for FS variables are first selected and the midpoint for each value is used to compute FS. By altering the value of one variable at a time across its range of values, the percent change in FS can be calculated. The results of the analysis are shown in Figure 3.2. From the figure it is evident that slope equilibrium is highly sensitive to slope angle, soil internal friction, relative piezometric surface, soil cohesion and root cohesion. Variation of factors such as soil density and tree surcharge load is obviously less important. These results coincide with general observations that landslides are associated with steep slopes, seepage, and soils with low shear strength.

It has been argued that the infinite slope model fails to account for many environmental factors known to influence slide location (Blong 1981). Many such factors, however, directly influence the values of fundamental variables in the model (Table 3.1). The determination of the cause effect relationships shown in Table 3.1 is the source of much subjectivity in the geotechnical approach.

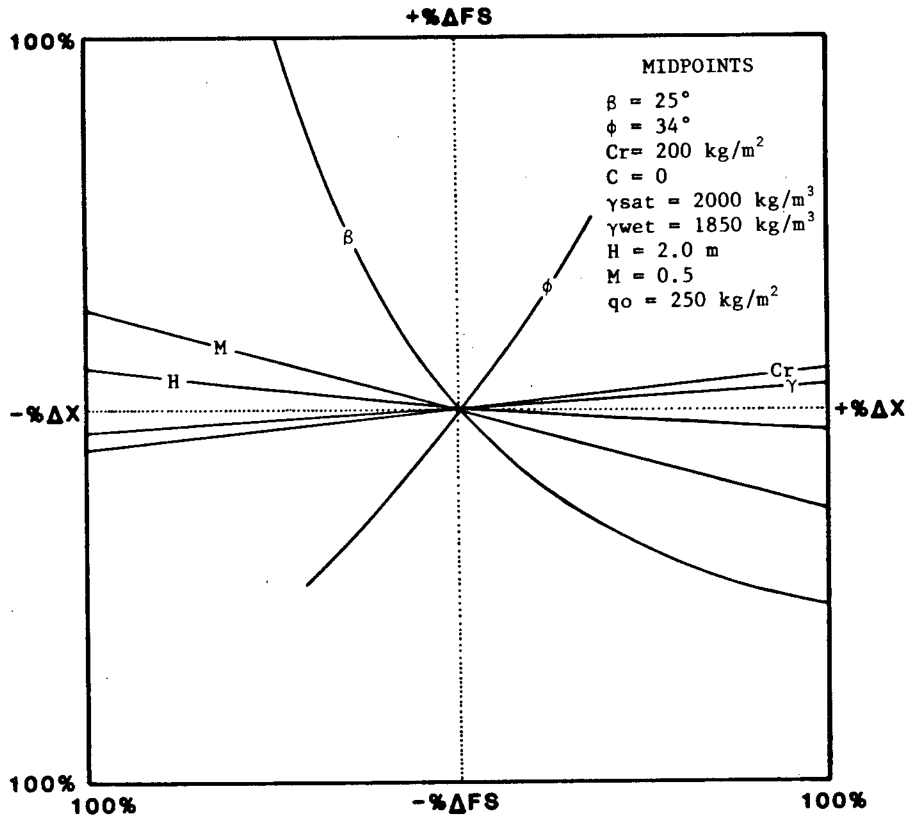

Figure 3.2 Diagram showing the sensitivity of the factor of safety FS to variations in model variables.

Many of the factors influencing the model variables are time dependent. Figure 3.3 illustrates the cycle involved in reaching slope equilibrium. Slopes can conceiveably follow any evolutionary path within the cycle, the end result being long-term stability. The values of model variables are determined by the severity and number of cycles a particular slope has undergone since glaciation. For example, one slope may have been continuously subjected to local steepening by gully action and subsequently burned by a forest fire. A particularly intense rainfall two years later then triggers a landslide. An adjacent slope may have escaped any or all of the above events and thus

IMPORTANT FACTORS IN PREVIOUS STUDIES		FUNDAMENTAL VARIABLES IN MODEL								
	FACTOR INCREASE	FS	M	ϕ	C	Cr	β	γ	qo	REFERENCES
	RAINFALL	-	+	0	0	0	0	+	0	Cleveland 1973
	PREVIOUS LANDSLIDE	-	±	-	-	0	±	0	0	Terzaghi and Peck 1967
	DEFORESTATION	-	±	0	0	-	0	+	-	Brown and Sheu 1975
	ROAD BUILDING	-	±	±	±	-	±	±	-	Prellwitz 1975
	TREE CANOPY DENSITY	+	-	0	0	+	0	0	+	Li 1974
	PROXIMITY TO DRAINAGE DEPRESSION	-	+	0	0	0	+	0	0	O'Loughlin 1973
	WET CLIMATE	±	±	±	±	±	0	0	±	Schumm 1968
	SLOPE FLOODING	-	+	0	0	0	0	0	0	Megahan 1972
	EARTHQUAKE	-	+	-	-	0	0	0	0	Youd 1973
	SHADED ASPECT	±	+	0	0	±	0	0	±	Lee 1963

Table 3.1 Effects of various factors on the variables fundamental to the infinite slope model.

remained stable. From Figure 3.3 it is apparent that road building is the most significant factor as it can alter the model variables ϕ, C, Cr, M, and β simultaneously.

A stochastic version of the deterministic infinite slope model has been developed to allow for the input of a realistic range of values for ϕ, C, and Cr. Monte Carlo testing of various statistical distributions of these values by Ward et al. (1978) has determined that assumed uniform (rectangular) distributions of input variables result in the most conservative estimates of FS, and that the FS distribution is best described as Gaussian (normal). The assumed uniform distribution is particularly convenient because an upper and lower limit (or discrete range) can be assigned without the need for statistically derived standard deviations. The derivation of the stochastic geotechnical model is described in detail in Appendix B.

The methodology first involves the determination of a

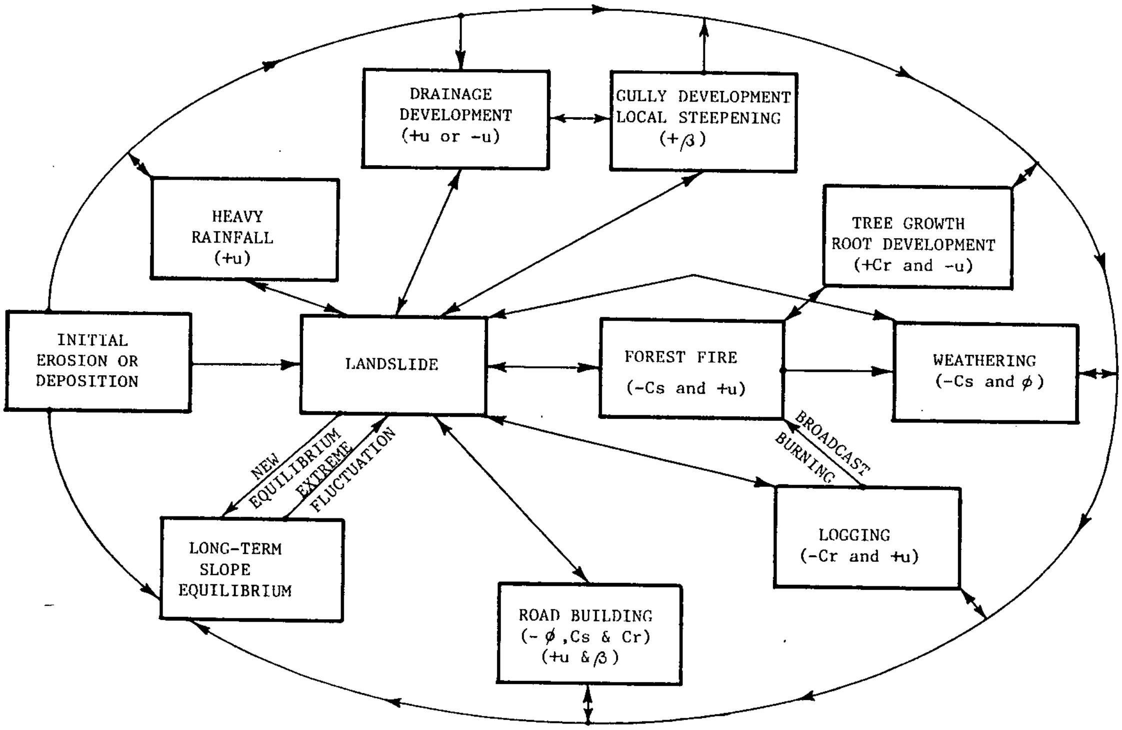

Figure 3.3 Alterations to slope equilibrium subsequent to glaciation leading to long-term stability.

realistic range of values for ϕ, Cr and C over the entire slope unit. Single values of β, M, H, γ, and qo are then estimated as either average or 'worst case' values for the slope. Whereas the variables β and M are probably the most critical to the resulting value of FS, the assignment of a single value, assumed representative of an entire slope, is sometimes problematical. This difficulty will be discussed in sections 3.5 and 3.6. It is important at this point to decide whether or not all model assumptions are satisfied for the slope being analysed. If not, an alternative method of stability analysis is required. The values assigned to the model variables are then run through a series of equations which result in the output of an expected distribution of FS for the slope. The value of FS at the peak of the frequency distribution curve is the 'expected value of factor of safety' E[FS]. The area under that portion of the curve with values of FS ≤ 1.0 yields the 'probability of failure' P, so that failure P so that

$$P = p[FS \leq 1.0]. \tag{3.7}$$

Values of E[FS] and P can then be used to describe the states of equilibrium of various slope units throughout the area. The model, though extremely simplified, demonstrates in a semi-quantiative sense[1], the behaviour of a slope having certain factor combinations. It explains in real physical terms some of the intuitive judgements made qualitatively by various researchers and provides a framework on which to base relative hazard classification.

The following sections deal with the measurement and delineation of the fundamental variables found in the study area. Slopes with characteristics which do not allow the application of the model are treated separately.

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[1]The term 'semi-quantitative' suggests that many of the input variables are somewhat subjectively determined.

3.3 Soil Shear Strength

Soil shear strength is a function of effective angle of internal friction (ϕ'), effective cohesion (C') and the effective stress (σ') as described by equation 3.2. The value of ϕ depends on grain size distribution, particle shape, particle interlocking, and soil density. Larger values of ϕ are associated with dense, well graded, angular materials (Rahn 1969). The value of c' is a reflection of the attractive forces between fine soil particles. The value of σ' is a function of the soil density γ and pore water pressure u, so that

$$\sigma' = \sigma - u \tag{3.8}$$

where $\sigma = \gamma H\cos^2\beta$.

3.3.1 Estimation Of Soil Shear Strength

Traditionally, values of ϕ and C are determined by directly testing soils in a mechanical shear apparatus. Large in-situ shear testing equipment, designed for soil-root networks, has been developed for use by O'Loughlin (1973) and Gray (1970) but is somewhat unreliable, costly, and time consuming. These and other types of in-situ apparatuses are also very difficult to use in reconnaissance surveys because roads are required for their transportation and installation.

Shear boxes or triaxial shear testing equipment can be used in the laboratory as a second alternative. However, depending on the size of the apparatus, ϕ and C values obtained can be inaccurate when gravelly soils with large coarse clasts are tested. Moreover, large samples are required if coarse grained soils are to be accurately represented. Where logistics are difficult and soils are gravelly, the testing of a large number of representative samples can be problematic.

Alternative methods for shear strength determination include the inference of reasonable ranges of ϕ and C from either slope morphology or from past experience with soils of an identical engineering classification. Although such inferences are highly subjective and much less accurate than actual shear testing, the methods offer the advantage of being better able to account for local variations in ϕ and C resulting from the variability of glacial and fluvioglacial depositional conditions.

Erosional slopes on cohesionless soils sometimes reflect the ϕ parameter if factors such as root cohesion and pore water pressures have no influence on the soil mechanics (Krynine and Judd 1957). Slopes subject to dry ravel generally behave according to the equation $\tan\beta = \tan\phi$, allowing the inference of ϕ values for soils with 'loose' relative densities and no cohesion. These ϕ values for loose materials are especially useful when predicting the stability of road cuts or fills (Wilson 1976). Unfortunately, erosional slopes clearly uninfluenced by the effects of vegetation, groundwater, or snow avalanching are extremely rare in the study area. Moreover, the majority of slopes to be critically evaluated for impacts of forest harvesting are necessarily forested! This method is, therefore, of limited utility.

If a surficial deposit is classified according to its genesis, alteration, and texture, its range of properties should be similar to those found in another surficial deposit having the same genesis, alteration and texture (Wilson 1976 and Wilson et al. 1982).[1] Whereas soil physical properties, including shear strength, are largely dependent on grainsize distribution and the plasticity of the fine fraction, the Unified Classification System (USC) has been developed on this basis for engineering purposes (see Appendix C for definitions). Engineering experience with the USC throughout the world has allowed engineers to compile expected ranges of ϕ and C as shown in Table 3.2. Unfortunately, delineating the areal distribution of USC soil classes is difficult because deposits are generally distributed according to genesis, not grain size distribution. It is therefore necessary to characterize mappable genetic materials (i.e. ablation moraine, etc.) according to their typical USC classification. Such a characterization provides the link between past engineering experience in determining ϕ and C with mappable soil units in the study area. Sources of uncertainty with this method include the unreliability or lack of universal applicability of ranges of ϕ and C reported in the literature, the problems involved with accurately classifying and mapping materials according to the USC and genesis, and the wide range of values resulting from the identification of more

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[1]Wilson and his co-authors develop this concept into an approach to the areal assessment of soil engineering properties termed 'pedotechnical engineering'.

CLASSIFICATION		AVERAGE ϕ VALUE RANGES		REFERENCES[2]
USC	SUBGROUP	LOOSE	DENSE	
GW	---	36-40	40-45	BA,BO,M
	GW-GC	31-38		BA,M
	SANDY	33-36	36-42	BA,H,M
GB	---	34-36	36-38	BO,M
GM	---	34 est.		M
GC	---	31 est.		M
SW	---	33-36	36-41	BA,H,M
SP	DRY COARSE	32-35	35-38	BA,BO,M
	WET COARSE	31-34	34-37	BA,M
	MEDIUM	31-34	34-39	BA,BO,M
	FINE	28-32	32-37	BA,BO,M,H
SM	MOIST	29-33	30-34	BA,BO,M
	SATURATED	25-29	29-32	BA,BO,M
SC	---	28-34		M
[2]BA = BAZANT(1979) BO = BOWLES(1979) H = HOUGH(1957) M = MOORE(1969)				

Table 3.2 Estimated angles of internal friction (ϕ) for cohesionless soils

than one USC class within a single complex genetic material.[3]

Values of ϕ and C also vary according to the relative density encountered in the field. This can be crudely estimated with a reinforcing rod and hammer, a method devised by the USDA (1975), and should be considered when determining appropriate ϕ and C values (see Appendix D for complete description of methodology).

[3]This is necessarily the case with complex glacial materials that have ϕ and C values which vary over a broad range.

3.3.2 Range Of Soil Shear Strength Values

In order to determine the distribution of genetic materials and hence ϕ and C in the study area, the Terrain Classification System (TCS) was employed in mapping units on the basis of texture, genetic material, surface expression and modifying process. Developed by the Province of British Columbia, the system is designed to be especially useful in recently glaciated terrain (E.L.U.C. 1976). The results of the mapping project are found as Map B (terrain map filed separately). An explanation of the symbols and terminology are found on the legend of the map, and in Appendix E.

The terrain map demonstrates that the surficial materials in the study area are extremely complex with many map units requiring composite symbols. During the course of the mapping, samples were taken of the characteristic genetic materials and later classified according to USC. Where sampling was not possible, USC classes were estimated in the field according to standard criteria. It was found that the USC classes are governed to a large extent by mode of deposition and can be inferred from knowledge of soil texture and genesis.[1] Indeed, the particle size and gradation requirements of coarse grained USC classes are very similar to the criteria applied in inferring genesis.

Terrain units recognised in the field were found to include

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[1]Unfortunately, the texture designation of TCS does not relate directly to the USC.

a predictable number of Unified Soil classes and are accordingly grouped in Figure 3.4. The range of ϕ and C values for various genetic materials were calculated and ranked according to relative strength. As one would expect, colluvial and morainal soils have higher ϕ values with a greater range than silty sandy fluvial and fluvioglacial deposits. Silty sandy (SM) soils, with silt percentages of less than 20% by weight, occur in a variety of genetic material types. Silty (ML) soils of low plasticity rarely occur as pockets within complex ablation morainal and fluvioglacial deposits, but are not continuous enough to add a significant weak component. Morainal soils have ranges of ϕ of up to 16° for 'loose' to 'firm' deposits.[1] Relatively homogeneous sandy fluvial deposits, on the other hand, have ϕ values which may vary within a 5° range. Thus the complexity or unpredictibility of the soil is reflected in the range of probable ϕ values in Figure 3.4.

Surficial deposits observed in soil pits within 1 meter of the surface almost always exhibit firm relative densities as determined by multiple probings with a reinforcing rod. It is possible that materials are denser deeper within the deposit. However, the predominance of thin blankets or veneers, giving rise to shallow planar failures in the study area, suggest that the firm ϕ values are most applicable to stability calculations. Disturbance by road building may loosen the material and cause a decline in ϕ. The loose ϕ values can be used in calculations of

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[1]In areas where compact tills are common, the range of possible ϕ values can be much higher than 16°. In-situ compact tills are much stronger than remolded or disturbed materials.

STRENGTH CLASS	TCS	USC	LOOSE φ VALUES	FIRM φ VALUES
			25° — 45°	25° — 45°
1	$\not{s}$sF^G sM $\not{s}$sM $\not{s}$gF	SM	27° 32°	29° 34°
2	$\not{s}$gF^G fgF^G frM fgM $\not{s}$rM	SM GM	27° 34°	29° 36°
3	gF^G gkF^G kgF^G bF^G sF^G	SP GP SM GM	27° 36° * ** * ** * * * *	29° 38° * *
	sF^G	SP SM		
4	sF	SP	30° 34°	32° 36°
5	ksF kF gF	GP SP	30° 35°	31° 37°
6	rM gM bM srM	SW SM GW GM	27° 40° *** * *	29° 43°
7	aC rC arC	GW	35° 40° * * **	38° 43° * *

Figure 3.4. Ranges of φ values for both loose and firm surficial materials in the study area. The asterisks represent plots of φ values inferred from angles of repose on road cuts and fills near the study area.

possible road cut or road fill stability.

The estimated ranges of φ given in Figure 3.4 appear reasonable when compared with the repose angles of cut and fill slopes on roads built in similar materials to the south of the study area. In all but one case, the assumed range encompasses the ϕ values inferred from the measured slopes. The plots of these measurements are shown as asterisks on Figure 3.4. Further work is needed to cross-check the assumed ranges with actual shear test data and repose angles measured in similar materials.

Estimates of soil cohesion are more difficult to determine in the field. Atterberg limit tests were performed on all samples collected in order to determine the plasticity and cohesion characteristics of the fine fraction. In almost all cases, silty sands (SM) and silty gravels (GM) did not have measurable plasticity and exhibited little cohesion. Similar results from Atterberg tests have been reported by Swanston (1970) who determined by shear testing cohesion values approaching 0 for silty-loam soils derived from ablation moraine in southeast Alaska. With the exception of compact till and the occasional silt or clay lens in a morainal or glaciofluvial deposit, all soils in the study area are assumed to have negligible cohesion.

3.4 Root Strength

Cohesion imparted to soils by tree roots has an important effect on soil strength within the root zone. In the study area, this zone is confined to the upper meter of soil. Roots can contribute strength to the soil by binding and reinforcing soil particles, anchoring to the underlying bedrock surface or laterally to adjacent root networks, transfering surcharge loads to the substratum, and inducing negative pore pressures by root capillary tension (O'Loughlin 1973). The apparent root cohesion Cr is a term which describes the combined effects of roots on soil strength and is one of the most difficult parameters in the geotechnical model to accurately determine.

3.4.1 Estimation Of Root Strength

Brown and Sheu (1975) and Wu et al. (1979) describe five possible ways of estimating root cohesive strength including (1) back-analysis of a slope failure; (2) analysis of soil creep; (3) analysis of individual tree overthrows; (4) in-situ measurements such as those by O'Loughlin (1973); and (5) soil-root stress-field modelling with known root tensile strengths. Method 2 is impractical and can only be applied to cohesive soils having viscous behaviour. Method 3 is also impractical as such a computation requires a known wind velocity imparting the overturning moment and ignores the possible presence of root-rot leading to the overthrow. Method 4 requires in-situ shear testing and was deemed impractical because of the inaccessibility of many parts of the study area.

Method 1, like the other methods, has many possible sources of error because many input values assumed correct in a back analysis are as difficult to accurately determine as root strength, such as seepage conditions at time of failure or soil strength. Consequently, studies of slope failures by different researchers have produced conflicting root strength values for identical failures (see O'Loughlin 1973 versus Morton 1975). Method 5 can be employed when data on root tensile strength are available. This may prove to be an extremely useful method as research into the strength and rooting habits of various tree species develops.

3.4.2 Range Of Root Cohesion Values

Forest cover maps compiled by B.C. Ministry of Forests, when supplemented with data from aerial photos, provide the basis for determining the distribution of slopes likely to be influenced by root cohesion. Values of Cr vary according to forest type, rooting depth, soil depth, tree density and tree size. No methodology currently exists for accurately delineating the distribution of Cr values. A certain realistic range of Cr values can only be calculated from site-specific landslide back analyses, or estimated from previous work in similar forests elsewhere.

A debris avalanche found within 10 km of the study area fits the criteria for root cohesion determination by back-analysis. Landslide D-14 occurred in a shallow colluvial soil on a planar shear surface at the colluvium-bedrock interface where water diverted by a logging road completely saturated the slope (see Figure 3.5). The fact that the failure occurs on a uniform planar slope and has a length to depth ratio greater than 10 allows the use of the infinite slope model. Input values of C = 0 and ϕ = 38°-43° are taken from Table 3.3, qo = 250 kg/m² is estimated from the local forest, γsat = 1960 kg/m³ can be

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[1]The values qo and γsat, though somewhat inaccurately estimated, should not introduce too much error, as FS is much less sensitive to these than the other parameters (see Figure 3.2).

estimated for firm GW soil[1] , H = 1.0 meters and β = 35° were directly measured and γw = 1000 kg/m³ is a physical constant. Firsthand observations of water issuing from the head scarp

Figure 3.5. Landslide D-14 showing the failure of a thin veneer of soil on a planar bedrock shear surface.

strongly suggest that the soil was saturated at the time of failure, i.e. M = 1.0 (refer to Figure 3.1). Assuming that the factor of safety was 1.0 at the time of failure, back-analysis yields root cohesion values ranging between 301 kg/m² and 409 kg/m² for ϕ values of between 38° and 43°.

In-situ shear testing by Endo and Tsuruta (1968) in birch forests produced values of root cohesion ranging between 200 and 1200 kg/m². Using a similar technique, O'Loughlin (1973) produced lower values ranging between 8 and 186 kg/m² for selected sites in southwestern British Columbia. O'Loughlin's tests were conducted in coastal cedar-hemlock forest soils similar to those encountered in the study area. Unfortunately, O'Loughlin's shear tests only included roots with diameters less than 3 to 4 cm, which only partially contribute to the overall root cohesion.

Doing further work with back analysis techniques, O'Loughlin (1973) found that root cohesion values more realistically fall within the 160 to 300 kg/m² range. Swanston (1970) derived values ranging from 350 to 450 kg/m² in mountain till soils of southeastern Alaska using the same technique. These values, though fraught with assumptions in their calculation, do provide some estimate of the magnitude of reinforcement trees could impart to soils.

It appears that in view of the foregoing, Cr could conceivably have values any where between 0 and 450 kg/m², depending on the orientation of the shear plane relative to the rooting zone. Endo and Tsuruta's 1200 kg/m² value for birch forests is disregarded as being unique to hardwood forests.

With only 1 data point for Cr values in the region of study available, it is virtually impossible to delineate any areal distribution of Cr. It is therefore necessary to subjectively assume that in all forested areas of the study area values of Cr will range between 0 and 450 kg/m². In non-forested areas, the value of Cr is assumed to be 0.

3.5 Groundwater

Piezometric pressures induced by groundwater lower the effective normal stress on a slope and thereby reduce soil strength. In addition, groundwater can contribute to surface erosion, subsurface piping, reduction of cohesive strength, seepage pressure, or liquifaction during earthquakes.

Chamberlain (1972) introduced an 'interflow' model of groundwater flow applicable to forest soils typical of mountainous terrain in British Columbia. The model describes the process of water movement parallel to the soil surface caused by boundary conditions, such as impermeable bedrock, sufficiently restrictive to prevent normal vertical infiltration and pecolation to the water table. During an intense rainfall, water is transmitted via the extremely permeable surface organic horizon (duff layer) to individual tree root channels. The water is then conveyed through these highly conductive conduits to a basal zone composed of matted roots at the interface with bedrock or compact till. The basal zone drains rapidly with high hydraulic conductivity and does not allow a true watertable or piezometric surface to form. The soil is termed 'open' as it does not allow the complete saturation of soil matrix between root channels. The soil has a natural drainage network.

Chamberlain (1972) further suggests that root development relative to soil depth is the most important of the soil forming factors with respect to the 'openness' of a soil. If roots penetrate to the first impermeable layer, it is likely that an open soil will develop; if not, infiltrating water will be transmitted to the less permeable soil matrix between root conduits and result in a closed soil of lower hydraulic conductivity (deVries and Chow 1978).

These considerations are important in predicting the probable groundwater positions on a slope. Some of O'Loughlin's (1973) work illustrates this point. A steep convex slope with a 1 meter mantle of bouldery sandy loam over compact till was monitored with piezometers. Measurements of piezometric head during rainfalls of up to 80 mm/day indicated a maximum rise of only 10 cm. It is inferred that rapid conduction of water through the root mat at the compact till surface did not allow piezometric pressures to rise significantly.

Deeper soils, other than tough impermeable compact tills, allow full root penetration and minimal mat formation. Water is rapidly transmitted to the base of the root zone but is subsequently incorporated into a deeper flow path. In this case, the groundwater table position is governed largely by the hydraulic conductivity of the soil below the root zone. Depending on the relative slope position of the site, the groundwater table is more likely to reach steady-state equilibrium. At the toe of a slope, groundwater input from upslope coupled with rainfall infiltration can cause the piezometric surface to rise to the surface and create temporary or permanent seepage (Freeze 1980). Piezometric pressures are more likely to influence slope stability in these areas.

Areas of permanent seepage or elevated piezometric surfaces can also occur in open soils where drainage depressions concentrate interflow. Piezometric studies by O'Loughlin (1973) in steep linear drainage depressions in shallow soils approximately 1 meter deep indicated 70 to 90 cm rises in piezometric head given an 80 mm/day maximum rainfall input. Likewise, linear depressions in shallow permeable soils of the study area occasionally show signs of ephemeral runoff indicating complete soil saturation. These localized seepage areas, though not areally extensive, have been significant to landslide occurrence in many areas (Bishop and Stevens 1967 and O'Loughlin 1973).

3.5.1 Estimation Of Piezometric Pressures

The traditional method of monitoring slopes with piezometers was not feasible in the study area. Where possible, surface indicators are used to decipher subsurface groundwater conditions. Swamps, springs and seeps were mapped as discharge areas, i.e. where the water table is at or near the ground surface. Mottled soils or free water observed in soil pits also indicate a periodic attainment of a near-surface water table.

Where more direct groundwater indicators are not available, vegetation types have been used to infer groundwater conditions with some success (Pole and Satterlund 1978). Certain plant species may occupy a broad range of habitats while others may be restricted to, and thus indicative of, specific moisture conditions. When plants with habitats restricted to seepage areas are observed on steep slopes, slope instability can be suspected.

The use of plant indicators for assessing ecological moisture regimes requires an existing vegetation classification scheme for the biogeoclimatic subzone under consideration or a reconnaissance of the area to establish relationships between vegetation indicators and the range of moisture and nutrient conditions. Such a scheme exists for southeastern British Columbia (Comeau et al. 1982).

When using plant indicators, the first step is to determine the subzone by refering to biogeoclimatic maps published by B.C. Ministry of Forests. Southeastern B.C. has been divided into three climatic regions, each encompassing areas of similar climatic patterns and each including a number of subzones. In the study area, the subzones include the Kootenay-Columbia Moist Southern Interior Cedar-Hemlock Subzone (ICHa1) and the Moist Southern Engelmann Spruce-Subalpine Fir Subzone (ESSFc). Plant species for a given moisture regime in the subzone can then be compared with plants observed at a particular site in the study area. Moisture regime definitions are given in Table 3.3.

The absence of wet-site plant indicators does not guarantee the absence of occasional near-surface soil saturation. On the other hand, the presence of water-loving plant associations can usually guarantee that the water table is near-surface at least sometime during the year.

By definition, mesic sites are in a state of equilibrium between precitation input and subsurface outflow. A mesic site in a wet biogeoclimatic subzone recieves a much higher precipiation input than a mesic site in a dry subzone. Moisture regimes are therefore not quantitatively equivalent from subzone to subzone and must be scrutinized with respect to groundwater on a subzonal basis. The ICHa1 and ESSFc subzones average precipitation inputs ranging between 70 and 150 cm/yr

VERY XERIC	Water removed extremely rapidly in relation to supply; soil is moist for a negligible time after ppt.
XERIC	Water removed very rapidly in relation to supply; soil is moist for brief periods following ppt.
SUBXERIC	Water removed rapidly in relation to supply; soil is moist for short periods following ppt.
SUBMESIC	Water removed readily in relation to supply; water available for moderately short periods following ppt.
MESIC	Water removed somewhat slowly in relation to supply; soil may remain moist for a small, but significant period of the year.
SUBHYGRIC	Water removed slowly enough to keep the soil wet for a significant part of the growing season; some temper-ary seepage and possibly mottling below 20 cm.
HYGRIC	Water removed slowly enough to keep the soil wet for most of the growing season; permanent seepage and mottling present; possibly weak gleying.
SUBHYDRIC	Water removed slowly enough to keep the water table at or near the surface for most of the year; gleyed mineral or organic soils; permanent seepage less than 30 cm below the surface.

Table 3.3. Definitions of moisture regimes (from Walmsley et al. 1980).

respectively. Whereas plants do not reflect rapid changes in piezometric head caused by intense rainfall, drainage depressions with subxeric to mesic plant indicators could have piezometric surfaces at or near the ground surface at some brief time during the the year. Dynamic fluctuations are critical to slope stability and in these areas, plants fail to accurately indicate critical subsurface groundwater conditions.

Examination of soil pits or exposed soil profiles offers an opportunity to verify the determination of moisture regime made using plant indicators. A coarse textured soil may have a water table just below the rooting zone that is, therefore, not indicated by the vegetation, which can be detected only by excavating a soil pit. On disturbed sites, more emphasis should be placed on results gained by examining the soil, as many of these indicators may invade drier, disturbed sites or may be displaced by other invader species (Walmsley et al. 1980). Even though the use of plant indicators can be a valuable tool for recognizing moisture regimes, it cannot completely replace the examination of soil materials.

Of primary interest to slope stability analysis is the ratio of the vertical distance between the shear surface and the water table to the vertical soil thickness above the potential shear plane. This ratio M can be generalized for different moisture regimes if the depth to failure plane is estimated. It is important to do this, as reconnaissance mapping of groundwater in the study area is best accomplished by delineating moisture regime distributions and relating them to values of M.

3.5.2 Estimated Effects Of Groundwater In Study Area

Surface drainage resulting from heavy rainfall is only evident in gullies scoured to bedrock in the study area. Highly permeable sandy to gravelly soils, interlaced with highly conductive root networks are evidently capable of handling high rainfall inputs without reaching full saturation in most areas. Such 'open' soil hydrologic behaviour only allows soils at receiving sites, i.e. drainage depressions, toe slopes, gullies, etc., to reach saturation as has been observed in similar soils elsewhere (Chamberlain 1972, deVries and Chow 1978, Patric and Swanston 1968 and Swanston 1967). The variation of the parameter M with respect to rainfall input on similar steep soils approximately 1 meter deep in southwestern B.C. is shown in Figure 3.6 for comparison. For these data, the shear surface is assumed to be at the soil-bedrock or soil-compact till interface. Figure 3.6 shows that convex shedding slopes do not have appreciable rises in M with less than 50 mm of rainfall. The record 56 mm/day precipitation extreme recorded at Fauquier would have only raised M to 0.04, assuming study area soils behave similarly. On the other hand, given the same precipitation extreme on concave receiving slopes, the variable M could conceivably rise to between 0.4 and 0.9 with the given precipitation input.

These trends can be correlated with moisture regimes frequently observed in the study area. Planar to convex slopes in shallow soils are generally subxeric to submesic and grade to mesic where soils deepen or on shaded aspects. The subxeric to mesic moisture regimes dominate the steeper slopes where only a

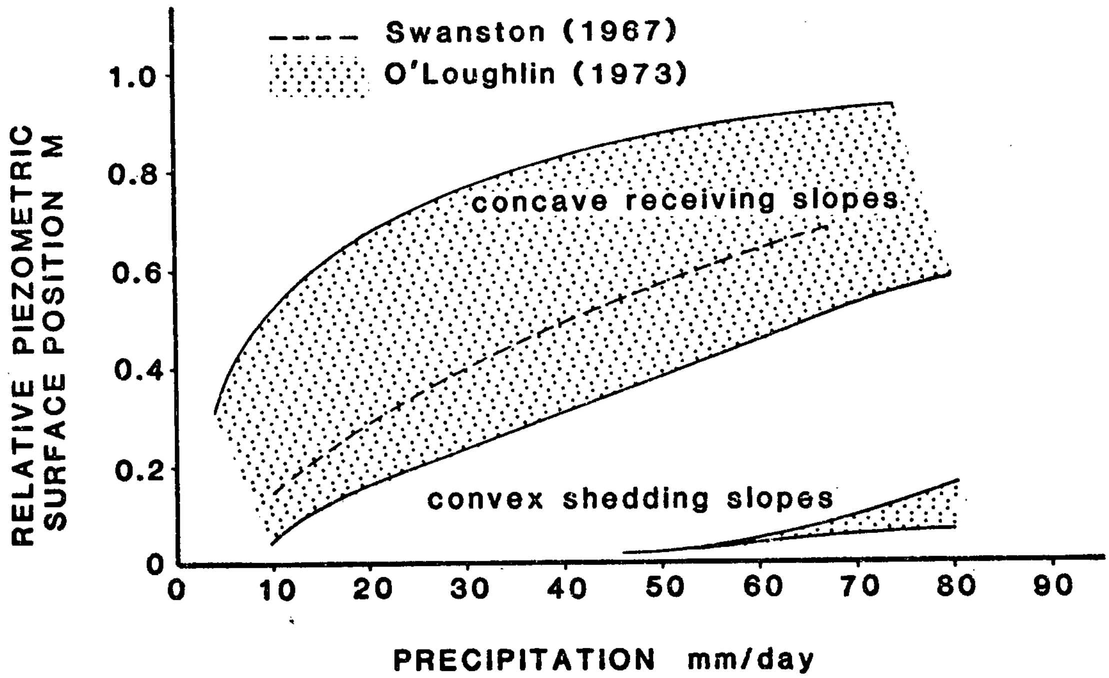


Figure 3.6. Variation of the relative piezometric head M with respect to 24 hr rainfall inputs as determined by Swanston (1967) and O'Loughlin (1973).

slight rise in piezometric head would be expected during a heavy rainstorm. On the other hand, drainage depressions and subdued gullies observed on the same slopes sometimes support subhygric to hygric vegetation, particularly on the lower portions of the slope. In these areas, relative piezometric head is much more likely to reach values above 0.4. A rapid rise in piezometric head can also be expected in gullies not supporting subhygric to hygric plants as moisture fluctuations may be too short-lived to allow phreatophytes to develop.

On lower slopes where surficial deposits are generally thicker and flatter, subhygric and hygric moisture regimes are more common. Figure 3.7 is a schematic cross-section of a typical slope in the study area showing the relationship between slope, soil depth, terrain class, and moisture regime. Hygric sites are almost entirely confined to areas where seepage is evident at the base of locally steepened terrace faces and

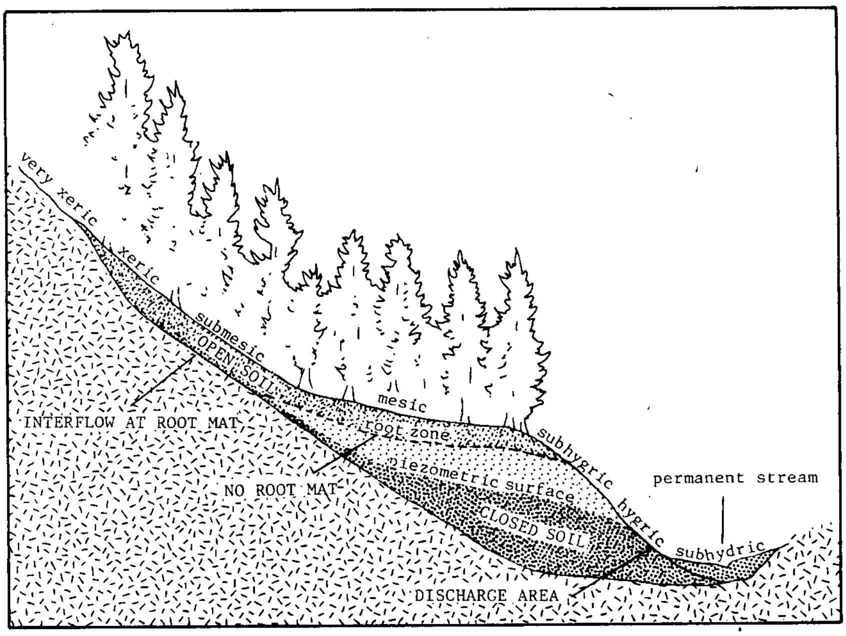


Figure 3.7 Typical profile of an idealized hillslope in the study area showing moisture regimes, probable groundwater table positions and surficial material types.

floodplains near streams.

Using both surface and plant indicators, the areal distribution of groundwater was roughly delineated. Areas not traversed during the course of the survey were subjectively delineated on the basis of slope position and morphology. Much of this work was done concurrently with Greg Utzig, a soils and plant ecologist also working in the study area.

The results of the moisture regime mapping project are to be included in the report entitled "Terrain and Ecological Classification of the Valhalla Mountains Study Area" by G.F. Utzig, of file with B.C. Ministry of Forest, Arrow District Office. Moisture regime delineations often correspond to terrain units with the exception of linear gullies, drainage depressions and terrace fronts.

Observations of groundwater table positions in relation to moisture regimes adjacent to the study area were made at road cuts where possible. This, coupled with observations in the study area and previously published data lead to the following generalizations for both ICHa1 and ESSFc biogeoclimatic subzones:

(1) Xeric to mesic sites on veneers and thin blankets of surficial material generally exhibit interflow at the bedrock interface. This flow is sometimes rapid in drainage depressions after storms.

(2) Xeric to mesic sites on thicker blankets of surficial material may have water tables approaching the surface in drainage depressions but are usually deeper than 1 meter.

(3) Subhygric to hygric sites are most commonly found in soils deeper than 1 meter and usually have water tables within 1 meter of the surface

(4) Occasionally subhygric to hygric plant indicators occur on shallow well-drained slopes where plant roots are able to tap permanent interflow in minor depressions.

(5) Subhydric to hydric sites always have watertables at the surface.

On straight slopes where groundwater is not concentrated and where potential planar debris slides are less than 2 m thick, the maximum value of M for various moisture regimes is estimated to be those shown in Table 3.4. These values do not apply to

MOISTURE REGIME	M	MOISTURE REGIME	M
Very Xeric	0	Subhygric	.5
Xeric	.1	Hygric	1.0
Subxeric	.1	Subhydric	1.0
Submesic	.1	Hydric	1.0
Mesic	.2		

Table 3.4. Maximum values of M for various moisture regimes in ESSFa1 and ICHa1 subzones where shear planes are less than 2 m deep.

deep rotational slides nor to drainage depressions. The hydrology of large slopes must not be generalized without treating locallized concentrations of groundwater seperately.

3.6 Slope Angle

The geotechnical model is highly sensitive to the inclination of the shear plane β as shown in Figure 3.2. Whereas potential slides to be analysed are of the shallow planar variety, the slope angle is assumed to be identical to the slope of the shear plane.

3.6.1 Measurement Of Slope Angle

Maps delineating units with slopes occurring within a certain interval can be produced by three methods including: (1) photogrammetric measurement of aerial photos; (2) analysis of topographic maps produced either by ground surveys or photogrammetry; and (3) actual field measurements.

3.6.2 Distribution Of Slope Angles In Study Area

Slope classes are best subdivided into intervals which, when delineated, have boundaries coincident with natural breaks in slope found in the study area. One natural break occurs at between 33° and 35° above which slopes are dominated by gravity processes, that is, colluvial fans and cliffs with pockets of colluvium. However colluvial fans are sometimes affected by avalanches originating from steeper slopes above and are altered to more concave, flatter slope profiles (Caine 1969). Young (1972) mentions that for a slope with continuous vegetation and soil cover, without rock outcrops or scars in the root mat, the upper limit of the slope angle is typically 30° to 36°. Most uniform forested slopes of primary interest to this study are inclined at less than this limit. Concave till mantled slopes typical of glaciated valleys generally have slope inclinations falling within the 15° to 35° range. Slopes of less than 15° are found either on either the east-facing slopes of the main Slocan Valley or on cirque basin floors and U-shaped valley bottoms. Though usually not dominated by gravity processes, these slopes sometimes serve as run-out zones for snow avalanches and landslides in the steeper valleys, particularly on the lower debris fans.

On the basis of these general observations, the slopes of the study area are somewhat arbitrarily grouped into the 4 classes shown in Figure 3.8.

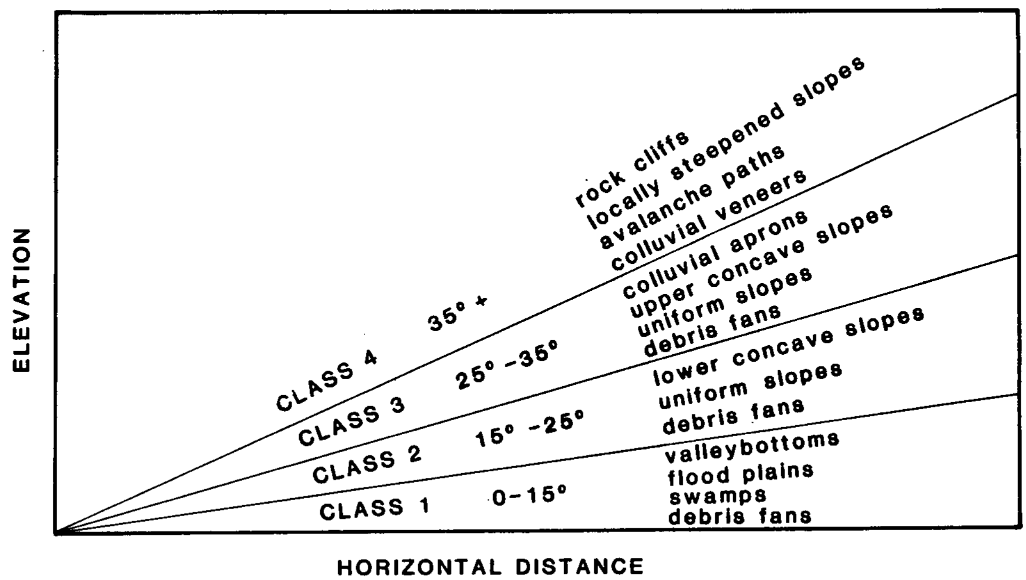


Figure 3.8. Slope class intervals used for the study area showing the types of slopes occurring within each class.

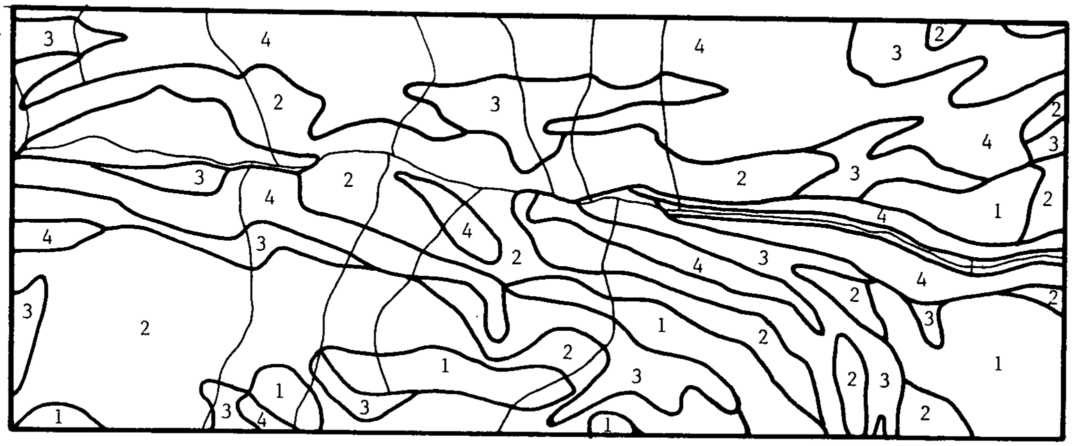
Wherever possible, slopes were measured during ground traverses and plotted directly on aerial photos. Study area size did not allow a complete survey of all slope angles in the study area; therefore aerial photos and topographic maps were employed to interpolate from known to unknown slopes.

Enlargement of 1:50,000 topographic maps (100 ft contour interval) to 1:20,000 permitted slope units to be defined by the bar-template method of Chapman (1952). It was found that 10° to

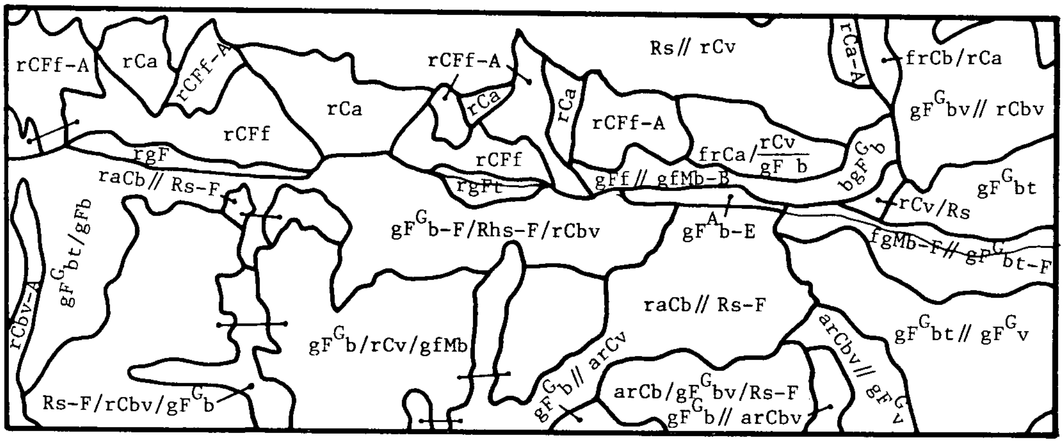
15° slope class intervals were required to delineate cartographically readable units in view of the extremely complex topography encountered. The end result of this slope survey is Map C (slope map filed separately). The unit boundaries are largely artificial and subject to inaccuracies introduced by: machine error in producing the 1:50,000 map and then enlarging it to 1:20,000; operator error in using the bar-template method; and the averaging effect of slope classes which do not account for variations in topography within the map unit. Ground truth measurements indicate that slope unit boundaries are artificial and somewhat erroneous in many areas.

Aerial photos can be used to delineate more natural and accurate slope class boundaries when coupled with slope the terrain unit maps previously discussed. Terrain mapping indicated that breaks in slope are often associated with changes in material type, eg. a talus slope below a rock face or a steep colluvial slope below a flatter bench mantled with ablation moraine. By comparing the terrain map with the slope map, many boundaries are seen to roughly correspond. Where major discrepancies appear, comparison with aerial photos enables a decision to be made as to which boundary is most accurate. Where possible, the results of this process are compared with ground measurements. An example from lower Nemo Creek Basin is shown in Figure 3.9. The final slope map resembles both the terrain map and the topographically derived map and is a more accurate representation of slope inclination than the topographically derived slope map when compared with ground measurements.

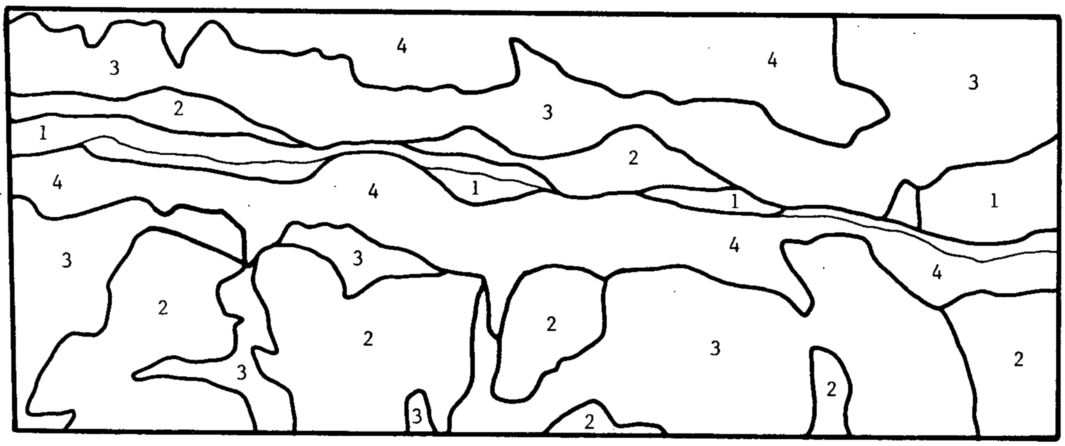
In units with multimodal slope class distributions, e.g.



(a) slope map derived from 1:50,000 topographic contours



(b) terrain map



(c) slope map interpreted from maps (a) and (b)

Figure 3.9 Three maps of a portion of lower Nemo Creek Basin showing (a) the original slope map derived from a topographic map; (b) the original terrain map; and (c) the more accurate slope map resembling the first two maps.

terraced or hummocky terrain, the steepest mode is assumed to be the most critical and is, consequently, used as the basis for slope class designation. Map (c) of Figure 3.9 has a somewhat larger area covered by class 3 and class 4 slopes than map (a). This is because slope units are assigned to the critical (or steepest) slope class; unmappable flatter sections such as benches, etc. within the unit are necessarily ignored.

3.7 Miscellaneous Factors

Tree surcharge weight and soil bulk density[1] are the last two factor variables which require consideration in the geotechnical model. The model is less sensitive to these variables and they will, therefore, not be considered in as much detail as the other variables.

Expected bulk densities of cohesionless soils in the Unified Classification System are given in Table 3.5. Bulk density and porosity for firm soils are assumed to lie midway between the loose and dense values.

Time and access limited in-situ bulk density determinations in the study area. Those values determined, using a modified volume measure technique (Utzig and Herring 1975), fall within the upper limits of expected ranges for gravelly soils with firm to dense relative densities, and are given in Table 3.6. These values can be combined to derive values of γwet and γsat for the different genetic materials as was done for ϕ values in section

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[1]This term is not equivalent to 'unit weight' used in some texts. Bulk density is mass per unit volume (kg/m^3) while unit weight is force per unit volume (kN/m^3).

3.3.2.

Brown and Sheu (1975) found that surcharge loads due to forests are typically 250 kg/m² with extremes as high as 500

USC	LOOSE dry*	sat	n	FIRM dry	sat	n	DENSE dry	sat	n
GP	1400	1420	---	1860	1940	---	2320	2460	---
GW	1425	1440	.46	1880	1960	.29	2340	2480	.12
GM	1600	2000	.41	1985	2250	.26	2370	2500	.11
GC	1600	2000	.41	1985	2250	.26	2370	2500	.11
SP	1330	1345	.50	1610	1760	.40	1890	2180	.29
SW	1360	1380	.75	1740	1875	.55	2115	2370	.35
SM	1390	1410	.47	1710	1840	.35	2035	2275	.23
SC	1390	1410	.47	1710	1840	.35	2035	2275	.23

* kg/m^3

Table 3.5. Average bulk densities for different Unified Soil Classes (from Bowles 1979, Sowers 1979 and Hough 1957).

kg/m². These values vary according to the spacing and size of trees and can be derived from forest mensuration data, if available. Forest cover maps do exist for the study area, but fail to provide enough data for surcharge load determination. A value of 250 kg/m² is assumed as a estimate of the average surcharge load for forested slopes in the study area.

3.8 Slope Equilibrium In The Study Area

It is now important to examine how the various model

NO.	REL. DENS.	USC	n%	w%	dry (kg/m^3)	in-situ (kg/m^3)	sat (kg/m^3)
D-1	FIRM	SW	35.1	11.6	1720	1921	2324
N1+100	FIRM	SM	38.5	9.0	1630	1781	2257
R-1	FIRM	SM	42.6	17.2	1520	1790	2158
N-0	DENSE	SM	20.0	9.3	2120	2327	2544
N1+100-2	DENSE	SM	20.4	8.8	2110	2304	2540
N1+55	DENSE	SM	14.3	7.7	2270	2455	2594

Table 3.6. In-situ bulk densities determined in the study area.

variables delineated in the previous sections combine to affect slope stability. The profile of a typical slope in the study area is shown in Figure 3.10. Model parameters, when assigned to various segments of the slope and combined according to Figure 3.3, yield the expected factors of safety E[FS] and probabilities of failure P shown, for both forested and non-forested slope segments. Some interesting interrelationships become apparent. First, the steeper slopes are usually drier and the flatter slopes are usually wetter. These two variables compensate for each other and result in less extreme FS and P values. The exception to this rule is, of course, the terrace front where wet conditions and steep slopes combine to produce probabilities of 100%. Second, the effects of root cohesion are more critical to stability on the steeper slopes. This point agrees with observed increases in landslide occurrence following loss of root strength after logging on similar slopes in Alaska

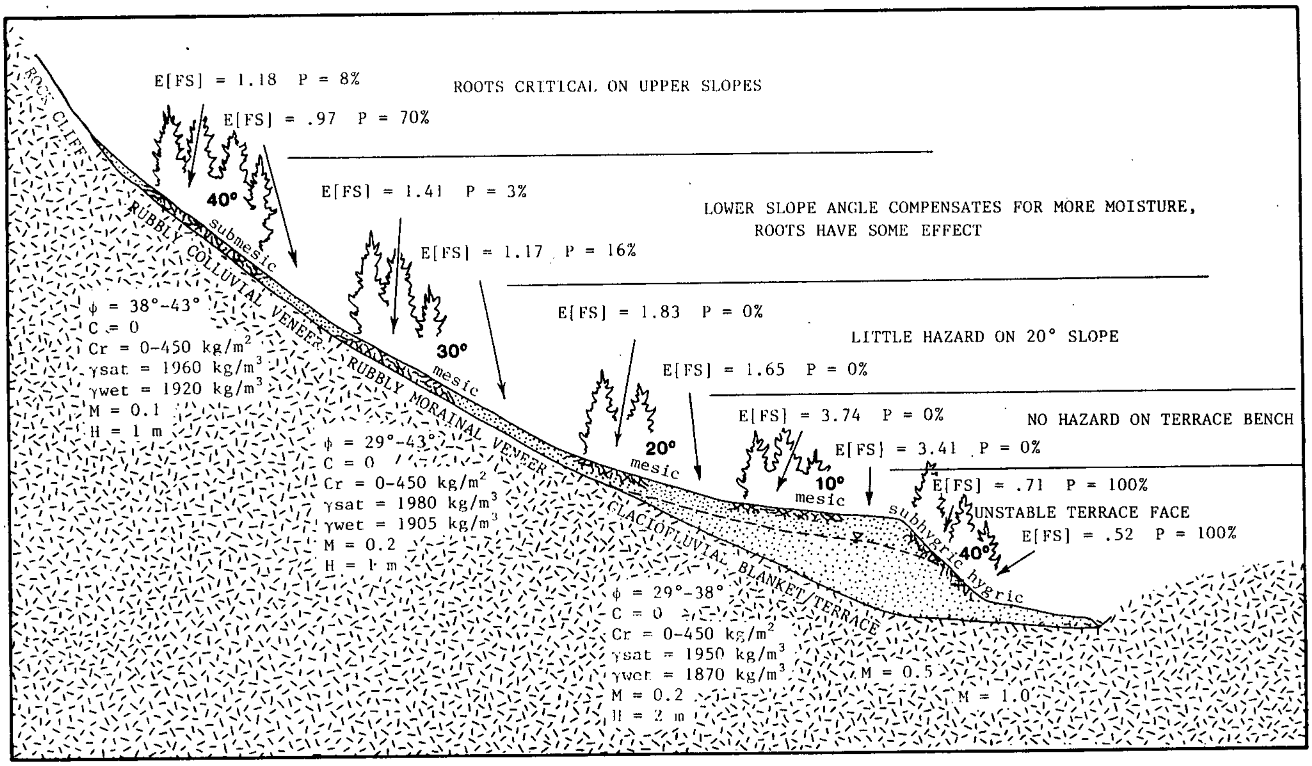

Figure 3.10. Profile of a typical slope in the study area showing the relative stability resulting from various combinations of quantified variables of the stochastic geotechnical model.

and S.W. British Columbia (Swanston 1974 and O'Loughlin 1973). Third, slopes inclined at less than 20° have high FS values and low probabilities of failure.

From the model it is possible to explain in semi-quantitative terms the observed distribution of many landslides is the study area and surrounding region. However, it is somewhat unclear as to what the stability values really mean in terms of forest management and proposed engineering. The probabilities cannot be used to actually predict the number of landslides likely to occur on a particular slope, nor the likelihood of a landslide occurring within a certain time period. Such quantitative predictions can only be made where model variables are less subjectively determined and when probabilities are calibrated and compared with observed events.

The values of E[FS] and P are best used as relative indices of stability. These indices can be used to compare the equilibrium of slopes in the study area with slopes that have responded unfavorably to forest engineering in other areas. From such comparisons, the indices can be grouped to form hazard classes of use to forest managers and engineers.

Many slopes do not satisfy the assumptions of the stochastic geotechnical model. They include: rocky slopes and cliffs with little surficial material; linear or site-specific gullies and terrace fronts which cannot be delineated as map polygons; debris fans dominated by flow processes; and talus fans formed by discrete rockfall and minor rockslides. The probable behaviour of these slopes must also be predicted by comparison with experience on similar slopes elsewhere. However, rather than develop individual schemes for the determination of slope equilibrium for these slope classes, it is assumed that the behaviour is homogeneous throughout the unit. In other words, in the case of steep rocky slopes, it is assumed that similar engineering problems are likely to be encountered anywhere within the unit; likewise on terrace fronts and debris fans.

CHAPTER 4 LANDSLIDE HAZARD CLASSIFICATION

4.1 Hazards On Slopes Mantled With Surficial Material

A reconnaissance survey of landslides associated with forest engineering on slopes mantled with surficial materials was conducted near the study area in order to: (1) relate pre-existing slope conditions and relative stability indices to resulting impacts, (2) identify those engineering practices most likely to promote failures and (3) develop a hazard classification system based on relative stability which can stratify slopes in the study area according to likely impacts, given certain assumptions as to the kind, intensity, and quality of engineering to be imposed on the slope.

4.1.1 Engineering Problems Near Study Area

Landslides associated with forest roads near the study area initiate either at the road cut or road fill and may continue progressively downslope, or retrogressively upslope. These slides are generated by changes in values of β, M, ϕ, Cr, and H caused by steepening slopes with cuts and fills, intercepting subsurface groundwater flow, loosening surficial materials, loading the slope with sidecast material, or destroying cohesion by tree root removal.

Cut slope failures are documented in areas where the road prism intercepts subsurface flow allowing seepage pressures to reduce the effective normal stress and induce failure.

Landslides D19, D20 and S1 are examples of this type of failure (see Appendix F for landslide data and locations). Two of the three slides were relatively minor, involving less than 72 m^3, and of little consequence to the road. Landslide D19, on the other hand, blocked approximately 25 meters of road with 225 m^3 of saturated sandy morainal debris (see Figure 4.1). Evidence of

Figure 4.1 Cut-slope failures caused by seepage on the cut face of a logging road in the Cariboo Creek area (Landslide D19 - see Appendix F).

piping along root conduits can be seen as holes on the cut face associated with free water within 0.7 meters of the ground surface. Vegetation at this particular site indicates subhygric to hygric moisture conditions inferring a near-surface groundwater table. Failures of this type occur on wet sites where soil is sufficiently deep to allow the development of an elevated piezometric surface in a closed soil, but were never observed in shallow soils where interflow processes predomiate.

Minor ravelling of material from cut slope faces is a much more common phenomenon near the study area. The usual road construction practice is to bulldoze cut slopes to near vertical angles with the allowance that the surficial material will eventually readjust according to its angle of internal friction. Minor cut slope failures have been known to cause water drainage problems in other regions where sloughing material chokes inside drainage ditches and diverts water over the road bed onto unprotected fill slopes (Burroughs et al. 1976 and O'Loughlin 1973). This, however, was not found to be a problem near the study area.

Shallow colluvial soils may in places ravel retrogressively long distances upslope from a rock cut as observed near landslide D4 where 0.5 to 1.0 meters of colluvium continues to ravel from more than 50 meters upslope. The volumes of material involved in ravelling are largely a function of the continuity of the colluvial veneers between rock buttresses and the ability of tree roots to anchor soil material to bedrock. The role of rapid subsurface water interflow in initiating these failures is uncertain.

Where slope angles approach the loose angles of internal friction of side cast material, fill slopes may fail progressively, either to the base of the slope (often a creek bed), or until some obstruction such as a rock outcrop or tree stump buttresses the fill. Slides of this type were in all cases restricted to slopes inclined in excess of 34° near the study area, similar to trends observed in other regions of the Pacific Northwest (O'Loughlin 1973, Utzig and Herring 1975 and Swanston 1969). Failures are induced by: fill saturation due to the obstruction of subsurface or surface water flow; erosion due to water running uncontrolled down the fill slope; or deterioration of organic debris buttressing fill slopes.

Landslide D1 is an example of a large debris avalanche - debris flow triggered by both surface and subsurface water flow interception by an old skidder road near Wragge Creek. A spoon-shaped shear surface at the zone of initiation occured within the fill material but, upon incorporation of water, transformed into a V-shaped surface involving approximately 9800 m^3 of morainal blanket material (see Figure 4.2). Recurrent failures along this gully now require annual maintenance at three main haul road crossings along the slide path and contribute sediment to Wragge Creek.

Where water is allowed to run uncontrolled down fill slopes, the resulting erosion will eat away at the road surface and undercut the embankment sufficiently to cause landsliding. Landslide D3 on Shannon Creek Road is an example of where the improper placement of a culvert has led to erosion which in turn promoted accelerated ravelling (Figure 4.3). Water diverted onto natural slopes can also initiate landslides such as D14 described in section 3.4.2 (see Figure 3.5). Landslides initiated solely due to loss of root cohesion following logging were never observed on uniform slope mantled with surficial material near the study area.

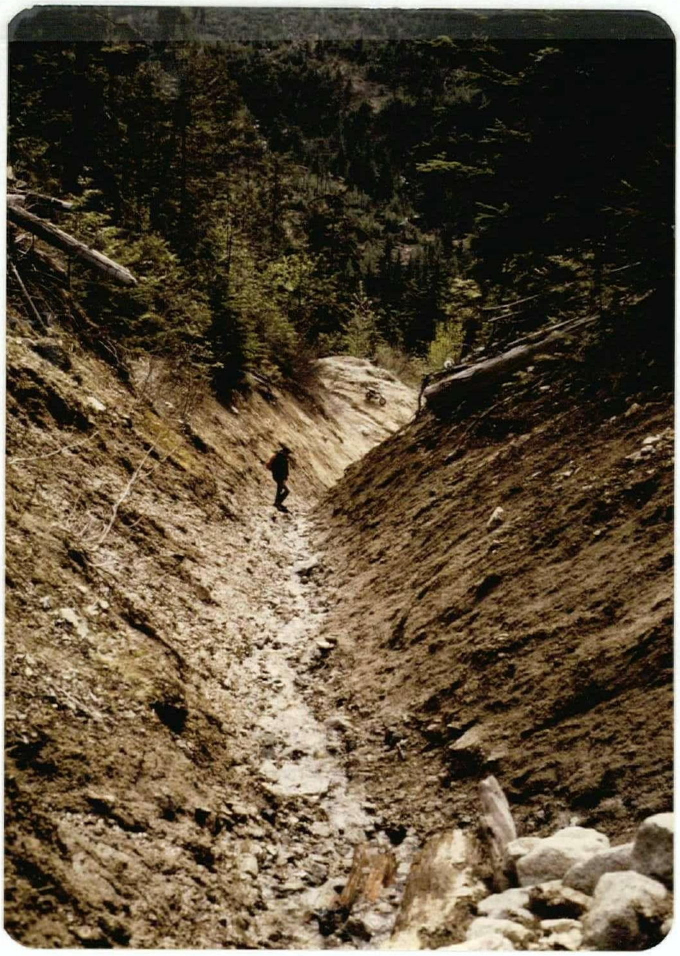

Figure 4.2. V-shaped profile of the path of a debris avalanche-debris flow caused by the saturation of road fill material near Wragge Creek Road.

4.1.2 Hazard Classes

Stability indices, when calculated for slopes adjacent to landslides, can be used for direct comparison with slopes to be developed in the study area. Values of E[FS] and P calculated for natural slopes adjacent to 14 landslides initiated by engineering are given in Table 4.1. The values of model variables were inferred in the same fashion as slopes in the study area. Results indicate that 11 of 14 sites had

Figure 4.3. Fill slope erosion and ravelling resulting from improper water control at the culvert exit.

probabilities of failure greater than 4% prior to engineering and that at the 3 more stable sites, obvious engineering errors, such as use of organic debris in road fills, flooding by water diversion, or improper switchback layout were responsible for landsliding. If the small cut slope failure occurring on the P = 4% slope listed in Table 4.1 is ignored, landslides of major consequence all occur on slopes with natural probabilities of failure greater than 10%.

The stability indices for natural slopes are used for

				BEFORE ENGINEERING								AFTER ENGINEERING						
NO	β	M.R.*	TCS	USC	φ°	H	Cr	Cs	M	E[FS]	P	Cr	M	β	H	φ°	E[FS]	P
D1	32°	2-3	fgMb	GW-GM	29-36	2.0	0-450	0	.1	1.10	17%	0	.2	34°	5.0	29-36	.85	99%
D2	30°	3	rCv	GM	38-43	1.0	0-450	0	.1	1.66	0%	0	.1	40°	3.0	35-40	.87	100%
D3	44°	4	rCa	GW	38-43	2.0	0-450	0	.2	.91	87%	0	.2	44°	2.0	35-40	.72	100%
D5	26°	3	rMb	GW	29-43	2.0	0-450	0	.1	1.59	0%	0	.1	35°	.5	27-40	.92	73%
D11b	36°	2	$gF^{G}b$	GP	29-38	2.0	0-450	0	.1	.99	52%	0	.1	38°	.5	27-36	.76	100%
D12	39°	3	$sF^{G}b$	SP	29-38	2.0	0-450	0	.1	.90	83%	0	.1	42°	2.0	27-36	.65	100%
D14	35°	3	rCv	GW	38-43	1.0	0-450	0	.1	1.40	0%	0-450	1.0	35°	1.0	38-43	.88	82%
D15	36°	3	$sF^{G}b$	SC	29-38	2.0	0-450	0	.1	.99	52%	0	.2	65°	1.0	29-38	.28	100%
D17	34°	3-4	sMb	SW	34-38	2.0	0-450	0	.2	1.09	13%	--	--	--	--	--	--	--
D18	38°	3	gMt	SW	34-38	2.0	0-450	0	.1	1.00	48%	0-450	.2	45°	4.0	34-38	.71	100%
D19	28°	5	sMb	SP-SW	32-38	2.0	0-450	0	.5	1.12	11%	0	.5	62°	1.5	32-38	.28	100%
D20	30°	3	fsMb	SW-SM	29-38	2.0	0-450	0	.1	1.23	4%	0	.1	52°	2.0	29-38	.49	100%
S1	36°	2-3	$gF^{G}t$	GP	29-38	2.0	0-450	0	.1	.99	52%	0	.1	75°	.5	29-38	.17	100%
A1	40°	3-4	$gF^{G}t$	GW-GP	29-38	2.0	0-450	0	.2	.87	89%	0	.2	40°	.5	27-36	.67	100%

*Moisture Regime

Table 4.1 Values of E[FS] and P calculated for natural slopes adjacent to 14 landslides.

comparison between areas because, depending on the type of failure involved, the values used in calculating the stability of a particular landslide may be quite different from those used in natural slope stability calculations. For example, landslide D2 occurred on a natural slope with E[FS] = 1.66 and P = 0%. However, taking into account the loosening of surficial material by disturbance, the steepening of the slope angle by road prism construction, and the destruction of stabilizing tree roots caused by bulldozing results in E[FS] = .87 and P = 100% for the actual landslide (see Table 4.1).[1] The prediction of stability conditions at the time of failure can only be made with site

[1]The infinite slope model, though not usually applied to short fill slopes, does give an approximate estimate of stability.

specific knowledge of the type of engineering alteration to be imposed upon the natural slope. It is assumed that engineering practices common in the region of study will be applied to slopes of the study area in determining the hazards on slopes with certain natural stability indices.

High P values and low E[FS] values sometimes calculated for slopes showing no evidence of natural instability prior to road construction reflect the conservative estimates of input value ranges and simplifying assumptions used in the model. Unfortunately, time did not permit the inverse approach of analysing slopes for stability indices in areas where landslides have not occurred. It is possible that, given better forest engineering practices, a large percentage of the slides occurring on slopes with P values greater than 10% would not have occurred. Indeed, in many areas, slides do not occur on slopes that are extremely steep and appear to be potentially unstable. However, it seems reasonable that slopes with probabilities of failure greater than 10% in the study area be assigned to the 'high hazard' class, as a single landslide event, though occurring only once on an areally extensive slope, may have critical environmental and financial implications. Moreover, there is no guarantee that landslide preventative techniques will be financially feasible or technically possible on these slopes. The four slides which occurred on more stable slopes had a larger component of human error responsible for their initiation, and landslides occurring on slopes with E[FS] greater than 1.6 were not observed. The 'moderate hazard' class is therefore assigned to the stability index interval P < 10%

and E[FS] < 1.6, and the 'low hazard' class is assigned to all slopes with E[FS] values greater than 1.6.

Roads developed on slopes which show signs of incipient failure, as indicated by slide morphology, exposed tree roots, recently buried soil horizons, etc. prior to road construction, were not observed near the study area. It is assumed that slopes fitting this criteria in the study area are of the 'very high hazard' class, as the process is already active.

For mapping purposes, the landslide hazard classes discussed above are given the symbols S4, S3, S2, and S1 corresponding to the very high, high, moderate, and low hazard classes respectively, and delineated on the slope stability map (Map D filed separately). The letter 'S' designates the terrain subdivision including slopes mantled with surficial material, and the numbers designate the relative hazard within the terrain unit. The 'S' terrain subdivision only includes those slopes which can be analysed with the stochastic geotechnical model, i.e. slopes with more than 70% cover of blankets (b) or veneers (v) of unconsolidated surficial material (see Map B). Slopes with fan (f) or apron (a) surface expressions are not included.

4.1.3 Preventative And Remedial Engineering Techniques

A number of engineering techniques would have helped prevent some of the landslides that occurred on S3 and S2 hazard slopes near the study area. The failure to control both surface and subsurface water on the road prism is a recurring problem in the region. Some solutions have been presented by various authors including Enberg (1963) who demonstrates that solutions to drainage problems have two parts: (1) longitudnal drainage (drainage parallel to the road) and (2) lateral drainage (drainage at right angles to the road). Haupt et al. (1963) found that road surfaces cambered toward the cut slope with drainage ditches adjacent to the cut slope are the most successful in controlling erosion on erodible granitic soils. However, this technique has contributed to a number of landslides in the study area as it tends to concentrate water in areas where ponding and road fill saturation is possible. The concentrated water, when drained laterally and disposed of downslope, can further aggravate slope stability. In areas where soils are highly resistant to rill erosion, cambering toward the sidecast is often the best solution as it avoids the problems associated with water concentration and the danger of uncontrolled flooding of unstable slopes below.

Where soils are too erodible to allow cambering toward the sidecast, certain provisions can be made for protecting fill slopes from concentrated lateral drainage, and road beds from ponding and saturation. First, slopes can be protected with riprap or culverts. Where possible, culverts should be placed in areas where natural drainage channels exist or where slopes are as flat as possible. Second, culverts should be spaced as frequently as possible in order to minimize the amount of concentrated water handled by any one culvert. Third, inside ditches should be properly pitched and regularly maintained.

Areas where surface and plant indicators of subhygric to hygric moisture conditions exist should be avoided wherever possible. Where excessive seepage on the cut face is encountered, perforated pipe can be installed to help lower the phreatic surface and carry water to the inside ditch. If slumping continues, gabion or log crib structures can be constructed to help buttress the slope. Cutslopes can also be stabilized by a variety of bio-mechanical techniques (Schiechtl 1980).

Where possible, road widths and cut and fill slope lengths should be minimized (Gardner 1979). Useful methods for road prism design on potentially unstable forested slopes have been developed by Prellwitz (1975) and Hendrickson and Lund (1974) based on stability analyses. Minimizing soil disturbance by limiting skidder road density and haul road mileage will significantly decrease the risk of landslide occurrence on both S3 and S2 slopes in the study area.

4.2 Hazards On Steep Rocky Slopes

This terrain subdivision constitutes the major portion of slopes in the study area where rockslides or rockfalls resulting from frost wedging and other mechanical weathering processes dominate. Stability is largely controlled by the mechanical properties of the coherent in-situ rock mass.

4.2.1 Engineering Problems Near The Study Area

Roads on steep rocky slopes of the highly competent granitic terrain in the study area invariably require blasting during their construction. Unfavourable discontinuity orientations leading to wedge failures from rock cuts were only noted in areas where 'overbreak' due to excessive dynamite charges has extensively fractured the cut face, leading to periodic rockfalls triggered by joint water pressures or frost wedging (see Figure 4.4).

Damage to productive soils, timber destruction and debris avalanche initiation where rock is thrown downslope can also result from overblasting on steep slopes. Because slope angles are generally greater than the angle of internal friction of sidecast material, large scars descending long portions of the slope frequently result. Landslides D9, D10, D13 and A5 in Appendix F are all examples of rock cuts where sidecast material has descended into the adjacent creek bed. Environmental damage can be significant in these cases. Near site D4, a large scar visible from at least 25 km away has resulted from the failure of rock sidecast.

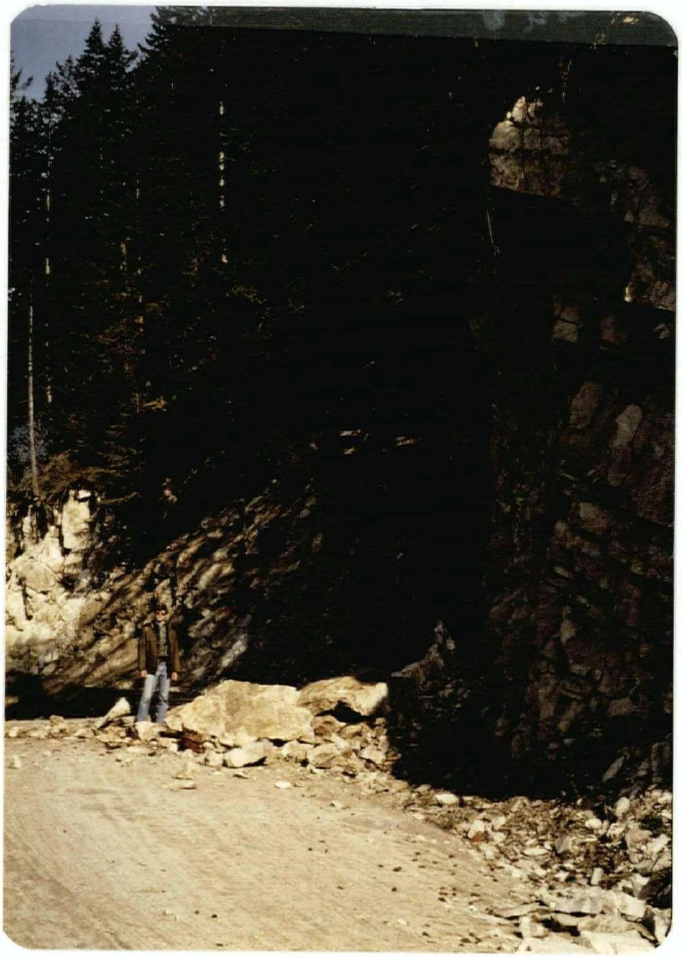

Figure 4.4. Rock failure on lower Shannon Creek Road.

4.2.2 Hazard Classes

High impacts resulting from engineering on steep rocky slopes near the study area will undoubtedly occur on similar slopes in the study area. All steep rocky slopes delineated as Rs on the terrain map are therfore designated as being of the 'very high hazard' class and delineated as R3 on the slope stability map (Map D). Composite units on the terrain map, according to this designation, must include at least 30% Rs in order to be assigned to the R3 hazard class.

Rock slopes inclined at less than 35° do not fit the criteria for this hazard class. The high mechanical strength of bedrock coupled with slope angles below the angle of repose of material being mechanically weathered make these slopes fairly stable, depending on the orientation and inclination of discontinuities within the rock mass. Blasting is usually required in road construction except in flatter areas where subgrade material can be imported and placed over the bedrock surface. Because blasted materials will usually rest in place on sidecast slopes and will not threaten the downslope areas, these slopes are designated as being in the 'low hazard' class and are delineated as R1 on the slope stability map (Map D).

4.2.3 Preventative And Remedial Engineering Techniques

Burroughs et al. (1976) emphasizes the need for proper blasting techniques when constructing roads through hard bedrock in steep terrain. The objective is to minimize cut slope overbreak, and prevent the throwing of material downslope. 'Presplitting' the cut slope , followed by controlled production blasting will usually accomplish both objectives thus allowing fractured rock to be trucked to safer areas without damaging slopes below.

4.3 Hazards On Colluvial Aprons And Fans

The colluvial apron and fan terrain subdivisions include those slopes which serve zones of accumulation of rock debris falling from cliffs above. Morphology is governed largely by the mechanical properties of the colluvial material, rockfall frequency, and rockfall height (Carson 1977).

4.3.1 Engineering Problems Near The Study Area

A recently constructed road on a 40° colluvial apron in the Wragge Creek drainage provides the only example of the engineering behaviour of colluvial aprons near the study area. At sites W1, W2, and W3, minor ravelling of rubble and blocks from the road cut onto the road bed suggests that material has some incipient instability. However, the road is largely stable due to the positive effects of particle angularity, tree root cohesion, and the fact that colluvial aprons derived from high cliffs are naturally more stable than those derived from low cliffs. Because the road was constructed only one year prior to the survey, it is possible that, with time, problems associated with continual ravelling from cut slopes or fill slopes following the deterioration of buttressing trees will require annual attention. In other regions, roads constructed on colluvial aprons frequently involve the ravelling of large volumes of material from long distances upslope (Baily 1971 and Burroughs et al. 1976).

4.3.2 Hazard Class

Landslide problems on colluvial aprons and fans in the study area are likely to occur in view of past experience with similar slopes elsewhere. Where infrequent rockfalls have allowed forest development and where aprons lie at the base of high cliffs, slopes can be expected to be more stable, albeit subject to rockfall hazards. In general, colluvial aprons and fans, identified as Ca and Cf on Map B, should be given a 'high hazard' rating delineated by the symbol R2 on the slope stability map.

4.3.3 Preventative And Remedial Engineering Techniques

Colluvial aprons and fans should be avoided wherever possible unless other alternatives are not available. The most common and successful control measure for preventing cut slope ravel is buttressing by gabion structures. 'End-hauling' excavated material to safe areas will also help alleviate the downslope ravelling problem.

4.4 Hazards On Debris Fans

Slopes formed by debris flow deposition have morphologies similar to both fluvial and colluvial fans and are inclined anywhere between 5° and 35°. Where depositional processes are still active, some particular engineering problems exist.

4.4.1 Engineering Problems In Other Regions

Roads constructed on active or recently active debris fans near the study area were not encountered during this study. However, experience with these terrain features elsewhere can be used to predict the types of hazards likely to be associated with them.

Debris fans have material properties similar to those of colluvial fans. However, because they are formed by slurry deposition, they are generally more stable than gravity dominated colluvial slopes of the R2 hazard classification. Roads which cross active debris flow channels are threatened by periodic flows which can cause severe damage. Slurries with boulders in excess of one meter in diameter have been known to descend these channels with little forewarning and great destructiveness (Nasmith and Mercer 1979, Miles and Kellerhals 1981 and Eisbacher 1982).

The flatter portions of the lower fans pose little hazard to engineering except at the isolated channel to which a debris flow may be confined. However, areas where channels are not well incised, flows may affect a much larger area. On the upper fans, engineering properties of the substratum are similar to those of colluvial fans, particularly on debris fans at the base of high cliffs.

4.4.2 Hazard Class

Because slope angles on debris fans vary over a broad range, it is useful to somewhat arbitrarily divide the terrain subdivision into two hazard classes at the 20° slope angle. It is felt that slopes inclined in excess of 20° (upper fans) are much more likely to involve hazards associated with cut and fill slope ravelling. Hazards associated with lower fans are confined to areas where periodic debris flows are likely to run. Steeply inclined upper debris fan areas are assigned to the 'high hazard' class and are delineated on the slope stability map (Map D) with the F2 symbol. Lower debris fans have a 'moderate hazard' rating and are delineated with the F1 symbol. It is noted that the F1 class is similar to the S1 class as they are frequently found adjacent to one other in the valley bottom. However, the debris flow hazard causes the F1 class to be given a 'moderate' rather than 'low' hazard rating.

4.4.3 Preventative And Remedial Engineering Techniques

Certain protective measures have been successfully employed in other areas where roads cross debris torrent paths. They include masonry or rock fill checkdams and sills to retain debris material above the road bed, large corrugated steel sheet culverts used with concrete bridges, and stone and log crib-type retaining walls on channel embankments (Heinrich 1978, Hattinger 1978 and Gagoshidze 1969). These measures reduce water velocity by obstructing debris channels, increase bridge strength enabling debris pressure to be withstood, and protect channel slopes from erosion near road crossings. Culverts should be large enough to carry periodic influxes of flowing debris with little damage to road embankments. A rule-of-thumb is to design the approximate cross-section of the culvert according to the approximate cross-section of the debris channel (Hattinger 1978). Upper debris fans should be treated like colluvial fans in areas other than debris flow paths.

4.5 Hazards On Terraces And Gullies

Steep linear terrace fronts and gully slopes are critical features found within the hazard classes described in the previous sections. Many landslides occurring naturally in the study area and associated with roads in the region are concentrated in the areas where slopes are locally steepened and in groundwater discharge areas. Terraces and gullies are usually too small to be mapped as units at the 1:20,000 scale and can only be represented as linear symbols within the other hazard units.

4.5.1 Engineering Problems Near The Study Area

Engineering problems associated with terrace fronts and gully slopes are identical to those described for steep slopes mantled with surficial material. Various factors combine to cause landslides which may effect not only the road prism itself, but also the stream channels at the base of the slope.

These slopes are affected by loss of root cohesion following deforestation. Since logging occurred about 15 years ago, numerous slides have developed on terrace fronts near Shannon Creek, one of which is shown in Figure 4.5. These slides

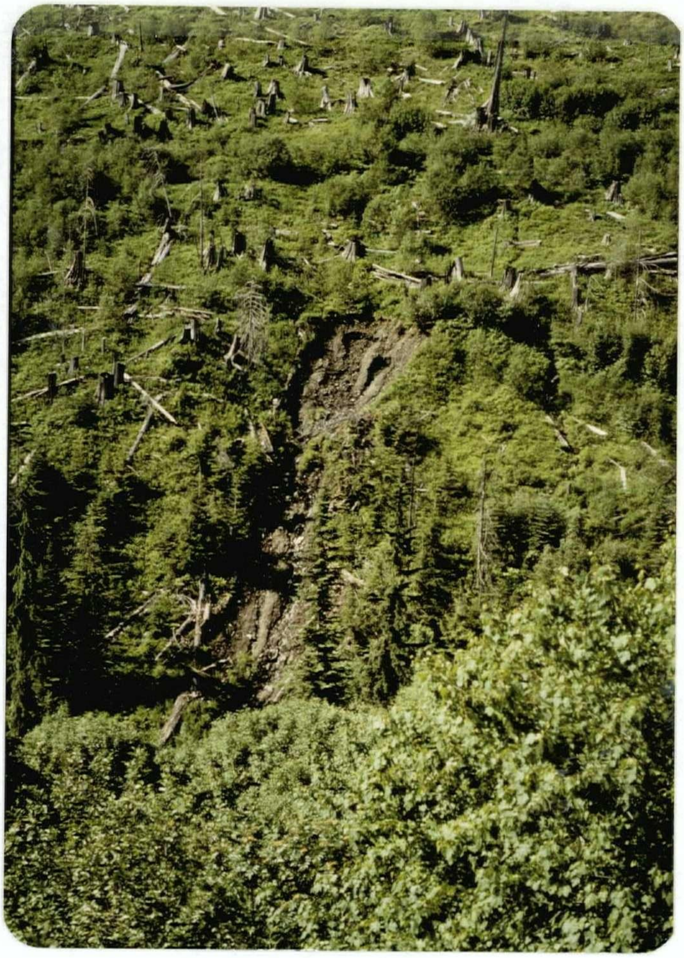

Figure 4.5. Landslide on a terrace slope near Shannon Creek initiated by loss of root cohesion (Landslide D3 - see Appendix F).

are not related to skidder roads nor to flooding from water diversion. The occurrence of these landslides 10 years or more after logging suggests that the slow deterioration of tree roots was responsible. It is also possible that decreased evapotranspiration may have allowed piezometric pressures to rise.[1] Where sidecast materials choke steep gullies at a road crossing, periodic influxes of concentrated surface runoff following a heavy rainfall may mobilize the debris and cause a large-scale debris flow (Ziemer 1981). However, problems of this nature were not found near the study area.

4.5.2 Hazard Class

Because landslides are frequently associated with gullies and terraces, the 'very high hazard' class is assigned to these features. No distinction is made between gullies and terrace fronts on the slope stability map (Map D), as problems associated with each are similar.

4.5.3 Preventative And Remedial Engineering Techniques

Engineering techniques given for S4 and S3 slopes can be applied to these linear features. Where roads cross gullies, special attention should be given to the proper containment of debris by correctly placing culverts, trucking excavated material to safe areas, and buttressing road cut slopes. Provisions for drainage of both subsurface and surface water should be conservatively designed to allow for runoff surges during storm events due to subsurface flow concentration.

Roads located on terrace benches should not divert water

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[1]The effect of deforestation on piezometric pressures is uncertain, as researchers have presented conflicting theories that require further testing (deVries and Chow 1978, Chamberlain 1972, O'Loughlin 1973 and Linsley 1975).

over the break in slope and seepage areas on terrace faces should be avoided, particularly near streams. Trees should not be harvested on these slopes as root cohesion may be critical to slope stability.

4.6 Summary Of The Hazard Classification System

In summary, hazard classes in the study area are divided primarily according to natural terrain subdivisions, then further subdivided according to the observed engineering behaviour of similar slopes in other areas. Figure 4.6 illustrates the use of the hazard classification system on a hypothetical slope, and can be used as a quick reference to the defining criteria of each class. It is noted that many land-use planners prefer to use a 3 to 4 class hazard rating system for decision making purposes, as more complex systems tend to confuse and complicate the issue, particularly when multiple-use decisions are being made. For this reason, the four ratings: low, moderate, high, and very high were ascribed to the nine more specific hazard classes discussed in this chapter. Thus, both the engineer and the planner can more easily utilize the information.

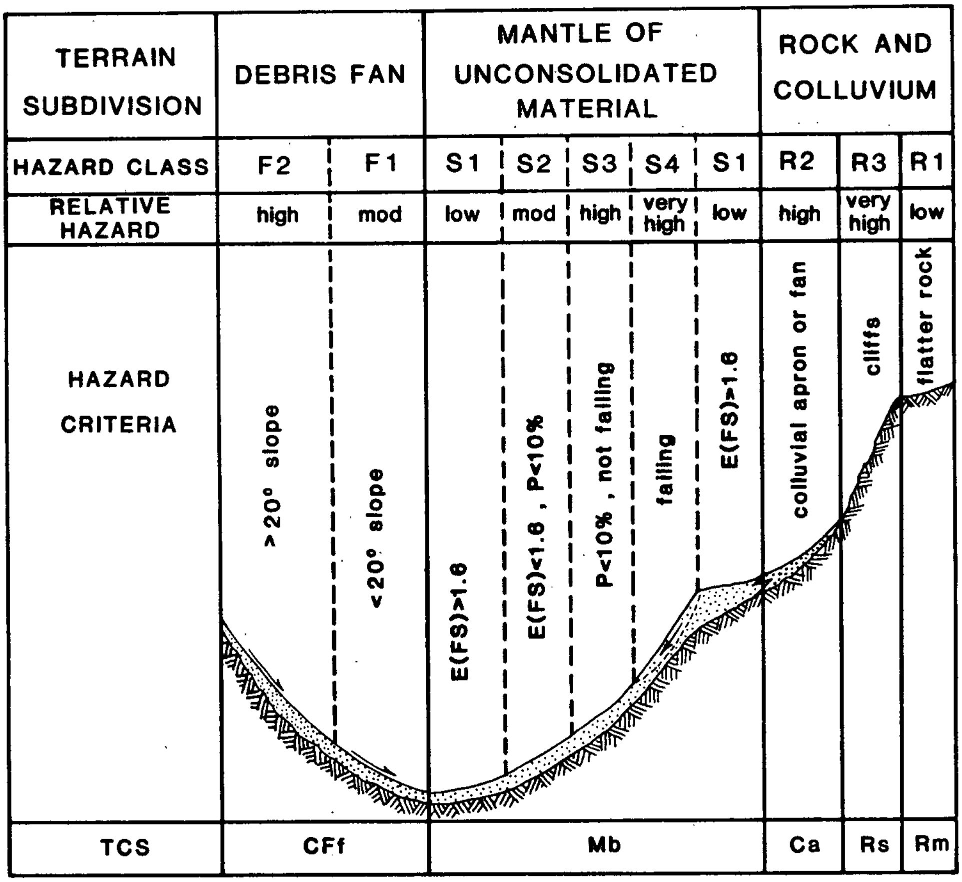


Figure 4.6. A hypothetical slope illustrating the use of the landslide hazard classification system.

4.7 Distribution Of Hazard Classes

High hazard R3 and R2 slopes are by far the most areally extensive of any hazard class in the study area as it includes the rugged, recently glaciated topography of the alpine areas between all four drainage basins. In places, rock ribs are exposed on the lower slopes of the valleys and pose formidable barriers to road construction. One such rib occurs to the southeast of the mouth of Wee Sandy Creek where R3 slopes encroach on both sides of the valley. Farther upvalley, R3 slopes again occur adjacent to one another on oposite sides of the valley in the vicinity of the large cascade described in Chapter 2.

In the lower portions of the valleys, R2 slopes occur most frequently at the base of the high south-facing cliffs of both Nemo and Wee Sandy Creek Basins. It is sometimes difficult to distinguish between colluvial blankets and colluvial aprons, in which case the composite symbol R2-S3 is used. Difficulties also arise where steep rocky terrain units include numerous but discontinuous colluvial aprons, such as slopes to the southeast of Wee Sandy Lake. In these areas the composite symbol R3-R2 is employed.

In the upper Wee Sandy Creek Basin, S4 slopes occur where streams have incised into morainal blankets or debris fans. Farther downvalley, failures are occurring on steep uniform slopes adjacent to R3 and S3 units, and on the upper slopes of gully networks feeding debris flow channels. In places, terrace fronts are extensive enough along lower Wee Sandy Creek to be mappable as S4 hazard units. Some of the most extensive and obviously unstable S4 slopes are found in lower Nemo Creek Basin. The lower north-facing slopes along approximately 5 km of the lower valley show evidence of recent landslide activity, including the large debris avalanche shown in Figure 2.5.

High hazard S3 slopes include a major portion of forest slopes of both Nemo and Wee Sandy Creek Basins. They frequently exist as the transition unit between R3 or R2 hazard classes upslope and flatter S2 or S1 hazard units downslope, along the concave profile typical of glaciated valleys (see Figure 4.6).

Moderate to low hazard S2 and S1 slopes are largely confined to the valley bottoms or cirque basin floors of the study area, except for the long uniform S2 and S1 slopes on the north-facing sides of both lower Nemo and Wee Sandy Creek Basins, and the east-facing slopes of the main Slocan Valley. In alpine areas where glacial materials have been deposited on relatively flat rock benches, areally extensive S2 and S1 slopes occur closely associated with glacially scoured R1 rock benches.

High to moderate hazard F2 and F1 debris fan units are widely distributed along the base of the south-facing cliffs of the major basins as discussed in Chapter 2, and also in the upper Sharpe Creek Drainage. F1 slopes are confined to the valley bottoms and are commonly adjacent to S2 and S1 slopes in these areas.

Gullies incised into surficial materials or bedrock commonly occur with R3 and S3 slopes on the south sides of both Nemo and Wee Sandy Creek Basins. Fluvial terraces are only found along the major or tributary creek channels of Wee Sandy and Nemo Creek.

CHAPTER 5 ROAD CORRIDOR ASSESSMENTS

5.1 General

The utility of the hazard classification system presented in Chapter 4 can be demonstrated by looking in detail at main logging road alignments in Nemo and Wee Sandy Creek Basins. The primary objective is to delineate the most environmentally sound yet economically feasible road alignments.

Several road options were tentatively located and flagged by B.C. Ministry of Forests during the summer of 1981. These road locations, shown on the slope stability map, were engineered to meet grade and location requirements for summer logging operations. What impact, if any, these roads will have on the stability of slopes can be determined by identifying the hazard classes traversed by each. The five separate road options will be discussed in detail.

5.2 Nemo Creek Road Options

Three road options were tentatively located for access into Nemo Creek Basin. The longest of the three is option A and begins at Slocan Lake approximately 2 km south of the mouth of Wee Sandy Creek. The nature of the hazards associated with this road location can be interpreted from the slope stability map.

For the first 4 km, the road traverses low hazard S1 slopes of the lower Sharp Creek Basin, crosses a rock rib to the north of Hoben Creek via a 'notch' easily discernable on the map, then unavoidably traverses some shallow soils requiring intermittent blasting. At km 6.7, the road begins to traverse potentially hazardous R3 and S3 slopes which in some areas may require blasting. These slopes are unavoidable because of the road grades needed to attain a bench adjacent to steep R3 slopes near Nemo Creek at km 9.0. The road then descends, via the bench, into the Nemo Creek Basin proper. At km 8.9, the road crosses a gully which will require special treatement in order to prevent debris avalanching and subsequent stream siltation.

Continuing up the basin on the north side of Nemo Creek, the road traverses a series of moderately hazardous S2 and F1 slopes with scattered boulders several meters in diameter derived from the high cliffs to the north. The substratum is mostly stable in these areas. However, a total of 5 active debris flow paths are crossed which would require special precautionary engineering. The road then crosses Nemo Creek at km 12.3 in order to avoid a particularly threatening snow avalanche path on the north side of the creek. Terrace fronts occurring on the south side of Nemo Creek demand particular care in locating a bridge crossing where engineering techniques can maintain stability.

Continuing to the west on the south side of Nemo Creek, the road traverses both S3 and F2 slopes that have been incised to form terrace fronts near the creek. The road should not be located near these terrace fronts. At km 14.0, the road enters the broader upper Nemo Creek Basin where the main haul road is terminated.

A summary of total lengths of road corridor A traversing each hazard class is given in Table 5.1. Included in the Table is a tally of the total number of gullies, debris flow channels, and terrace fronts traversed, and the number of kilometres

		KILOMETRES OF EACH HAZARD CLASS TRAVERSED														KILOMETRES OF EACH RELATIVE HAZARD								
ROAD OPTION	TOTAL LENGTH	S1	S2	S3	S4	R1	R2	R3	F1	F2	S3-R3	S1-R1	S2-R1	R3-R1	S2-R3	VERY HIGH	HIGH	MODERATE	LOW	INTERMITTENT BLASTING	CONTINUOUS BLASTING	GULLIES CROSSED	DEBRIS FLOWS CROSSED	TERRACES CROSSED
A	14.0	5.6	2.6	1.5	0	0	.3	0	1.8	.2	.9	.9	.2	0	0	0	2.9	4.6	6.5	2.0	0	1	5	1
B1	9.4	.3	3.6	1.6	.3	0	.3	0	1.8	.2	0	0	1.1	.2	0	.3	2.3	6.5	.3	1.1	.2	3	5	1
B2	9.4	.8	3.2	1.1	.3	0	.3	.2	1.8	.2	0	0	1.1	.4	0	.3	2.2	6.1	.8	1.1	.6	3	5	2
C	9.0	.9	2.5	.4	.6	0	.1	.6	1.3	1.5	0	0	0	.1	1.0	.6	4.2	3.8	.9	1.0	.7	2	3	1
D	11.0	2.6	2.6	1.0	.6	.3	.1	.8	1.3	1.5	0	0	0	0	0	.8	3.4	3.9	2.9	0	1.1	6	3	1
S4, R3 - VERY HIGH S3, R2, F2 - HIGH S2, F1 - MODERATE S1, R1 - LOW																								

Figure 5.1. Hazards traversed by various proposed road corridors in Nemo and Wee Sandy Creek Basins.

likely to require either intermittent or continuous blasting during road construction.

Nemo Creek road option B begins on Slocan Lake approximately 1.8 km south of the mouth of Nemo Creek. It is approximately 4.6 km shorter than option A and provides a more direct access to the Nemo Creek Basin. Traversing southwest from Slocan Lake, the road encounters within 0.5 km a major gully apt to cause stability problems if the road is not properly engineered. The road then switches back, again crosses the gully, and traverses S2 slopes likely to require some minor blasting because of shallow surficial materials. At km 1.9, the road begins to descend towards Nemo Creek via a locally steepened S4 slope showing evidence of recent landsliding. In this area, slopes are steeper than 35° and include gullies with spoon-shaped scarp faces at their heads. At km 2.4, the road emerges from the high risk area onto a low hazard S1 slope but soon encounters shallow soils likely to require intermittent blasting during road construction.

At km 3.3, road option B splits into (1) an upper route labeled B1 which links up with road option A at km 4.3, and (2) a lower route with a more favourable grade that will, unfortunately, require blasting during road construction for 0.1 to 0.2 km on a R3 slope immediately adjacent to Nemo Creek. The lower route, labeled B2, intersects an active slide at km 4.6, then joins road option A at km 5.0. From this point, options B1 and B2 follow the same route as option A.

Table 5.1 summarizes the total length of road corridors B1 and B2 traversing each hazard class and the total number of linear hazard features crossed.

Road options A, B1 and B2 each have economic and environmental advantages and disadvantages. When and where environmental considerations are critical to the development scheme, the road option likely to minimize landslide occurrence should be chosen, which in this case would be option A. However, if financial considerations dominate the picture, and if the financial and environmental consequences of landsliding are acceptable, perhaps option B1 should be chosen because of its lower construction costs.

5.3 Wee Sandy Creek Road Options

Two corridor options were tentatively located for access into Wee Sandy Creek Basin. The longest of the two (option D, see slope stability map and Table 5.1) begins at Slocan Lake where road option A begins 2 km south of Wee Sandy Creek.

For the first three kilometres, the road traverses low hazard S1 and S2 slopes of the lower Sharp Creek Basin. At km 3.0, the road encounters a steep R3 slope veneered with colluvium which will require continuous blasting for approximately 0.7 km. At kilometre 3.7, the road enters a notch on a bedrock dominated ridge, then descends into the Wee Sandy Creek Basin proper intersecting three major gullies, all of which have at some time involved debris flows. Once near Wee Sandy Creek, the road descends a terrace front and then crosses the creek at km 6.6 in order to avoid steeper slopes and snow avalanche paths on the south side.

Continuing to the west, the road traverses a series of F1 and F2 slopes, crosses three recently active debris flow paths, and takes a double switch-back on a debris fan slope in order to gain elevation and maintain proper grade. At this point, the road crosses an unavoidable series of high hazard S3 and R3 slopes likely to have some impact on Wee Sandy Creek immediately adjacent to the south. Actively failing S4 slopes are virtually unavoidable between km 9.0 and 10.2 because of the extreme steepness of the canyon in this area. At km 11.0, the road enters the broader upper Wee Sandy Creek Basin where the main road is terminated. A summary of the total lengths of road corridor D traversing each hazard class and the number of critical linear features crossed is given in Table 5.1.

Wee Sandy Creek road option C begins at the same point as option D then traverses directly to the north towards the mouth of Wee Sandy Creek. This more direct route crosses moderate to steep S2 and R3-S2 slopes, some of which have shallow veneers of surficial material over competent bedrock which would require intermittent blasting during road construction. Approaching Wee Sandy Creek at km 2.0, the road unavoidably encounters extremely steep R3 slopes likely to produce unstable debris which will descend to the creek below. If a maximum allowable favourable grade is maintained, the road can emerge onto a terrace bench at km 2.5 and avoid a highly unstable terrace front showing evidence of active landsliding directly into the creek. Any attempt to locate a road across this terrace front would almost certainly result in landslides causing environmental damage.

Farther to the west, road option C crosses two debris flow gullies before descending a terrace front and crossing Nemo Creek at km 4.3. At km 5.0, the road intersects an follows road option D to upper Wee Sandy Creek Basin.

A summary of the total lengths of road option C traversing each hazard class and number of critical linear features crossed is given in Table 5.1.

Road options C and D attempt to minimize cost and environmental impacts. From Table 5.1 and the above description, it is evident that portions of road requiring blasting and involving environmental damage due to landsliding are unavoidable with either option. Financial and environmental considerations suggest that option D is a better choice. However, because a host of considerations other than landslide potential affect the choice of final road location, the ultimate decision is left to the land planner.

CHAPTER 6 SUMMARY AND CONCLUSIONS

The first of two objectives of this thesis was to map, describe and determine the predictibility of fundamental factors controlling the stability of slopes in the study area. Factors deemed important in landslide occurrence in other regions include geologic, geomorphologic, hydrologic, pedologic and vegetative variables in addition to engineering considerations. However, from observations of the nature and type of dominant landslide processes currently active in the study area, it was possible to infer which factors are likely to affect landsliding. The general observation that initial failures occur on planar shear surfaces within 2 meters of the ground surface partially satisfies the assumptions of a physically based geotechnical model. The model greatly simplifies the number of factors to be considered but can only be applied to certain terrain units. Advantages of the physical model are that it provides a framework on which to build a hazard classification system for uniform slopes mantled with surficial material, the slopes most likely to be affected by forest engineering. Moreover, because it is less operator-dependent than other empirical models which are based on the statistical analysis of subjectively chosen and evaluated factors, it has the advantage of being more easily transferred from region to region. However, some difficulties were encountered in accurately quantifying the fundamental variables of the model in the study area.

It was first necessary to delineate those slopes in the study area which could be evaluated according to the geotechnical model. This was done using the Terrain Classification System (TCS) to map slope units primarily on the basis of genetic material and surface expression. It was found that less than one half of the slopes in the study area could be evaluated according to the model. Of the slopes that could be evaluated, many were complex assemblages of genetic material, variable topography, and groundwater conditions. In addition, parameters such as soil shear strength, root strength, piezometric pressure, and depth to shear plane were difficult to measure in the field due to logistical and time constraints. Therefore, it was necessary to develop the means to roughly estimate the likely values of the fundamental landslide controlling factors over these slopes. The uncertainties involved in the estimates can be partially accounted for in a stochastic version of the geotechnical model.

Genetic materials were found to be extremely complex leading to difficulties in characterizing shear strength properties. The method of estimating ϕ values developed in this thesis fails to accurately determine absolute values at a particular site. However, it does appear successful in determining the relative values associated with different genetic materials. For example, even though the absolute ranges of ϕ values for colluvial and morainal soils could not be determined very accurately, the fact that colluvial soils are generally stronger than less angular fluvioglacial materials, and that morainal soils are more unpredictible than either colluvial or fluvioglacial materials is born out in the analysis.

A particular difficulty encountered was that of accurately delineating on the map the distribution of slope angles, the factor most fundamental to slope stability. In many areas, slopes are obscured by the tree canopy and cannot be accurately evaluated with aerial photographs. Moreover, topographic complexity demanded that slope classes be given 10° intervals, an interval over which a fairly wide range of slope equilibrium values can be calculated. This, however, is a difficulty that is not unique to this methodology and is one of the primary reasons why areal assessments of stability cannot replace site-specific evaluations.

Semi-quantitative variables in this thesis affect or are themselves affected by many of the factors described by authors in other regions. For example, empirical evaluations in many regions have drawn a direct relationship between deforestation and landslide occurrence, sometimes with little explanation as to why this is the case. The physical model, on the other hand, clarifies the fact that loss of root cohesion following deforestation is, in fact, the factor directly influencing landsliding. Similarly, in the study area, the stochastic model demonstrates not only the fact that landslides are commonly associated with terrace fronts, but also that elevated piezometric pressures and locally steepened slopes are contributing to them.

Factors controlling stability in areas not evaluated according to the geotechnical model were inferred from knowledge of slope morphology and genesis. For example, hazards due to debris flows could be inferred from evidence of past debris flow activity such as large boulders deposited on levees adjacent to channels.

The second objective of this thesis was to examine landslides initiated by engineering activities in environments similar to those of the study area and develop a hazard rating system based on past experience. Stability indices, when calculated for slopes adjacent to landslides induced by engineering activities, were used for comparison with slopes to be developed in the study area. The results indicated that due to natural factors and engineering misjudgement, landslides tend to occur primarily on slopes with probabilities of failure greater than 10% as determined by the stochastic geotechnical model. Engineering problems associated with these failures include inadequate provisions for water drainage, improper fill slope construction and incorporation of organic debris in fill materials. High probabilities of failure were also calculated for slopes showing no evidence of instability either before or after road construction, indicating that conservative estimates of model input values lead to higher than expected probabilities of failure. Fortunately, this is of little consequence if the stability indices are grouped to form hazard classes according to engineering experience. From the fact that slopes with probabilities of failure greater than 10% near the study area host a wide variety of landslide problems, it can be inferred that slopes with greater than 10% values in the study area will likewise involve similar types of problems, regardless of what the the actual 10% value may mean in real physical terms. In the absence of more accurately determined model input values and statistical analyses of probability distributions for slopes of the study area, the relative hazard rating system is the only recourse available. In areas where the geotechnical model does not apply because of limiting assumptions, hazards are assigned according to past engineering experience in similar natural terrain units. The engineering behaviour is then assumed to be homogeneous throughout the unit.

Units classed as 'very high hazard' include those slopes which show signs of active failure as indicated by morphology and vegetation, as well as steep rocky slopes. 'High hazard' slopes include colluvial fans, upper parts of debris fans, and slopes mantled with surficial materials having probabilities of failure greater than 10%. Slopes with 'moderate' landslide hazards include lower debris fans and and slopes mantled with surficial materials having probabilities of failure less than 10% and expected factors of safety less than 1.6. 'Low hazard' slopes include slopes mantled with surficial materials having expected factors of safety greater than 1.6 and gently sloping bedrock dominated terrain.

The uncertainties involved in characterizing natural slope conditions over large areas where available data are limited, have no doubt led to some broad generalizations which, at the local level, are in error. However, as a broad reconnaissance-level tool, the hazard unit delineations of the slope stability map and the interpretations and recommendations for engineering proceedures contained in this thesis should help the land manager to make wiser planning decisions.

BIBLIOGRAPHY

Bailey, R.G. 1971. Landslide hazards related to land-use planning in Teton National Forest, northwest Wyoming. USDA Forest Service, Intermountain Region, Ogden, Utah, 131p.

Beven, K. 1981. Impacts and management of steeplands erosion: reviews on theme 4. In: Symposium on Erosion and Sediment Transport in Pacific Rim Steeplands. Jour. of Hydrology (NZ), v.20, n.1, p.107-117.

Bishop, D.M. and Stevens, M.E. 1964. Landslides on logged areas in southeast Alaska. USDA Forest Service, Research Paper NOR-1.

Blong, R.J. 1973. A numerical classification of selected landslides of the debris slide-avalanche-flow type. Engineering Geology, n.7, p.99-114.

Blong, R.J. 1981. Methods of assessing slope erosion and non-channel sediment sources in upland regions: review of theme 1. In: Sympodium on Erosion and Sediment Transport in Pacific Rim Steeplands, Jour. of Hydrology (NZ), v.20, n.1, p.80-89.

Bowles, J.E. 1978. Engineering Properties of Soils and Their Measurement. McGraw-Hill Book Co., 213p.

Bowles, J.E. 1979. Physical and Geotechnical Properties of Soils. McGraw- Hill Book Co.

Brown, C. and Sheu, M.S. 1975. Effects of deforestation on slopes. Jour. Soil Engineering Div., American Society of Civil Engineers, v.101, n.GT2.

Burroughs, E.R., Chalfant, G.R. and Townsend, M.A. 1976. Slope stability in road construction. Bureau of Land Management, Oregon State Office, 102p.

Burroughs, E.R. 1980. Proposal to develop guidelines for management of unstable lands in the northwestern United States. USDA Forest Service, Intermountain Forest and Range Experiment Station, Forest Sciences Laboratory, Bozeman, Montana.

Caine, T.N. 1969. A model for alpine talus slope development by slush avalanching. Jour. Geology, v.77, p.92-100.

Canada Soil Survey Committee. 1978. The Canadian System of Soil Classification. Research Branch of Canada Department of Agriculture, Pub. 1646, 164p.

Carson, M.A. 1977. Angles of repose, angles of shearing resistance and angles of talus slopes. Earth Surface Processes, v.2, p.363-380.

Chamberlain, T.W. and Jeffrey, W.W. 1968. Soil movement and water quality deterioration associated with timber harvesting in the West Kootenay Area, B.C., University of British Columbia Faculty of Forestry, Vancouver, B.C.

Chamberlain, T.W. 1972. Interflow in the mountainous forest soils of coastal British Columbia. In: Mountain Geomorphology: Geomorphological Processes in the Canadian Cordillera, B.C. Geographical Series, n.4.

Chapman, C.A. 1952. A new quantitative method of topographic analysis. American Jour. of Science, v.250.

Cleveland, G.B. 1973. Fire + rain = mudflows, Big Sur 1973. California Geology, v.26, n.6.

Comeau, P.G., Comeau, M.A. and Utzig, G.F. 1981. A guide to plant indicators of moisture for southeastern British Columbia, with engineering interpretations. B.C. Ministry of Forests Land Management Rept., n.5, 117p.

Dale, R.W. and James, I.L. 1977. Forest and environment in the Kaimai Ranges. New Zealand Forest Service, Forest Research Institute Technical Paper, n.65, 34p.

DeVries, J. and Chow, T.L. 1978. Hydrologic behaviour of a forested mountain soil in coastal British Columbia. Water Resource Research, v.14, n.5, p.935-942.

Eisbacher, G.H. 1982. Slope stability and land use in mountain valleys. Geoscience Canada, v.9, n.1, p.14-27.

E.L.U.C. Secretariat. 1976. Terrain classification system. Environment and Land-use Committee Secretariat, Parliament Buildings, Victoria, B.C.

Embleton, C. and King, C.A.M. 1968. Glacial Geomorphology. Edward Arnold Ltd., London, England. 573p.

Enberg, P. 1963. Design of roadway drainage to prevent erosion on forested watersheds. Symposium of Forest Watershed Management, Society of American Foresters, Columbia River Section and Oregon State University.

Endo, T. and Tsuruta, T. 1968. On the effect of tree roots upon the shearing strength of soil. Annual Report of the Hokkaido Branch Forest Esperiment Station.

Foggin, G.T.III and Rice, R.M. 1979. Predicting slope stability form aerial photos. Jour. of Forestry, v.77, n.3, p.152-155.

Freeze, R.A. 1980. A stochastic-conceptual analysis of rainfall-runoff processes on a hillslope. Water Resource Research, v.16, n.2, p.391-408.

Froehlich, H.A. 1979. The influence of clearcutting and road building activities on landscape stability in Western United States. In: Proceedings of Fifth North American Forest Soils Conference, Colorado State University, Fort Collins, Colorado, p.165-171.

Furbish, D.J. 1981. Debris slides related to logging of streamside hillslopes in northwestern California. M.Sc thesis, Humbolt State Unversity.

Gagoshidze, M.S. 1969. Mudflows and floods and their control. Soviet Hydrology: Selected Papers, Publ. by American Geophysical Union. Issue 4.

Gardner, R.B. 1979. Some environmental and economic effects of alternate forest road designs. Transactions of the American Society of Agricultural Engineers, v.22, n.1, p.63-68.

Gray, D.H. 1970. Effects of forest clearcutting on the stability of natural slopes. Bulletin of the Association of Engineering Geologists. v.7, n.1 and 2, p.45-66.

Hattinger, H. 1976. Torrent control in the mountains with reference to the tropics. In: Hydrologic Techniques for Upstream Conservation, FAO Conservation Guide 2, p.119-134.

Haupt, H.F., Rackard, H.C. and Finn, L.E. 1963. Effect of severe rainstorms on insloped and outsloped roads. USDA Forest Service, Research Note INT-1.

Heinrich, R. 1978. Protection of forest roads using biological and engineering methods. In: Mountain Forest Roads and Harvesting Technical Report of 2nd FAO/Austria training course on forest roads and harvesting in mountainous forests, p. 79-92.

Hendrickson, L.G. and Lund, J.W. 1974. Highway cut and fill slope design guide based on engineering properties of soils and rock. Twelfth Annual Symposium on Engineering Geology and Soils Engineering, Boise, Idaho, p.71-95.

Holland, S.S. 1964. Landforms of British Columbia, A Physiographic Outline. B.C. Department of Mines and Petroleum Resources, Victoria, B.C.

Hough, B.K. 1957. Basic Soils Engineering. Ronald Press Company, New York.

Krajina, V.J. and Brooke, R.C. 1969. Ecology of Western North America. Department of Botany, University of British Columbia.

Krynine, D.P. and Judd, W.R. 1957. Principles of Engineering Geology and Geotechnics. McGraw-Hill, New York, 730p.

Lee, R. 1963. Evaluation of solar beam irradiation as a climatic parameter of mountain watersheds. Hydrologic Paper n.2, Colorado State University, Fort Collins, Colorado.

Leopold, L.B., Wolman, M.G. and Miller, J.P. 1964. Fluvial Processes in Geomorphology. W.H. Freeman and Co., San Francisco, 522p.

Li, R.M. 1974. Mathematical modeling of response from small watersheds. PhD Dissertation, Colorado State University, Fort Collins, Colorado.

Linsley, R.K., Kohler, M.A. and Paulhus, J.L. 1975. Hydrology for Engineers. McGraw-Hill Series in Water Resources and Environmental Engineering. 482p.

Little, H.W. 1952. Geology of the west half of the Nelson map sheet, Kootenay and Similkameen Districts, British Columbia. GSC Memoir 308.

Lumb, P. 1970. Safety factors and probability distribution of soil strength. Can. Geotech. Jour., v.17, n.3.

Megahan, W.F. and Kidd, W.J. 1972. Effects of logging and logging roads on erosion and sediment deposition from steep terrain. Jour. of Forestry, v.70, p.136-141.

Miles, M.J. and Kellerhals, R. 1981. Some engineering aspects of debris torrents. Fifth National Hydrotechnical Conference, Canadian Society for Civil Engineers.

Ministry of Forests, 1981a. Reconnaissance survey of the Valhallas. Report on file at B.C. Ministry of Forests, Arrow District Office, Castlegar, B.C.

Ministry of Forests, 1981b. Valhalla area forest development proposal. Report on file at B.C. Ministry of Forests, Nelson Regional Office, Nelson, B.C.

Morton, C.K. 1975. Root contribution to slope stability. B.S.F. Thesis, University of British Columbia.

Nasmith, H.W. and Mercer, A.G. 1979. Design of dykes to protect against debris flows at Port Alice, British Columbia. Can. Geotech. Jour., v.16, n.4, p.748-757.

O'Loughlin, C.L. 1973. An investigation of the stability of the steepland forest soils in the coast mountains, southwest British Columbia. PhD Dissertation, University of British Columbia, Vancouver, B.C., 147p.

O'Loughlin, C.L. 1981. Methods of assessing slope erosion and non-channel sediment sources in upland regions: reviews of theme 1. In: Symposium on Erosion and Sediment Transport in Pacific Rim Steeplands, Jour. of Hydrology (NZ), v.20, n.1, p.80-89.

Parrish, R.R. 1982. Geology of the Nemo Lakes Belt, northern Valhalla Range, southeastern British Columbia. Can. Jour. E. Sci., v.18, n.5, p.944-958.

Patric, J.H. and Swanston, D.N. 1968. Hydrology of a slide-prone glacial till soil in southeast Alaska. Jour. of Forestry, v.66, n.1, p.62-66.

Pillsbury, N.H. 1976. A system for landslide evaluation on igneous terrain. PhD Dissertation, Colorado State University, Fort Collins, Colorado, 109p.

Pole, M.W. and Satterlund, D.R. 1978. Plant indicators of slope instability. Jour. of Soil and Water Conservation, Sept.-Oct., p.230-232.

Prellwitz, R.W. 1975. Simiplified slope design for low standard roads in mountainous areas. Report on file, USDA Forest Service, Northern Region, Missoula, Montana, 20p.

Rahn, R.H. 1969. The relationship between natural forested slopes and angles of repose for sand and gravel. GSA Bull., v.80, n.10, p.2123-2128.

Reesor, J.E. 1965. Structural evolution and plutonism in Valhalla Gneiss Complex, British Columbia. GSC Bull., n.129, 128p.

Schumm, A. 1968. Speculations concerning paleohydrologic controls of terrestrial sedimentation. GSA Bull., v.79, n.11.

Simons, D.B., Li, R.M. and Ward, T.J. 1978. Mapping of potential landslide areas in terms of slope stability. Colorado State University Engineering Research Center Report CER78-79DBS-RML-TJW19, Fort Collins, Colorado.

Slocan Valley Community. 1974. Forest management project, final report. Valley Resource Society, General Delivery, Winlaw, B.C.

Sowers, G.F. 1979. Introductory Soil Mechanics and Foundations: Geotechnical Engineering. Fourth Edition, Macmillan, New York, 621p.

Swanston, D.N. 1967. Soil-water piezometry in a southeast Alaska landslide area. USDA Forest Service, Research Note PNW-68, 17p.

Swanston, D.N. 1969. Mass wasting in coastal Alaska. USDA Forest Service, Research Paper PNW-83.

Swanston, D.N. 1970. Mechanics of debris avalanching in shallow till soils of southwest Alaska. USDA Forest Service, Research Paper PNW-103, 17p.

Swanston, D.N. 1974. Slope stability problems associated with timber harvesting in mountainous regions of the western United States. USDA Forest Service, General Techincal Report PNW-21, 14p.

Swanston, D.N. 1979. Effect of geology on soil mass movement activity in the Pacific Northwest. In: Forest Soils and Land Use, Proceedings of the Fifth North American Forest Soils Conference, Colorado State University, Fort Collins, Colorado, Aug 1978, p.89-116.

Swanston, D.N. 1980. Interpreting stability problems for the land manager. Background paper prepared for B.C. Forest Service Public Involvement Conference. Nov. 14-15, Victoria, B.C.

Takeda, Y. 1976. A study on the transition of landslides in the mountain and the movement in future, through the decipherment of aerial photograph. Bull. of Nagoya University Forests, n.6, p.77-156.

Terzaghi, K. and Peck, R.B. 1967. Soil Mechanics in Engineering Practice. Second Edition, John Wiley, New York, 729p.

U.S.D.A. Soil Conservation Service. 1975. Engineering Field Manual for Conservation Practices. U.S.D.A. Soil Conservation Service, Washington, D.C.

Utzig, G. and Herring, L. 1975. Forest harvesting impacts at high elevations: five case studies. B.C. Forest Service Research Note, n.72, 85p.

Utzig, G., MacDonald, D. and Comeau, P. 1978. Ecological Classification for the Nelson Forest Region--Second Approximation. B.C. Ministry of Forests, Nelson Region.

Utzig, G. Classification and detailed mapping of soil and terrain features in two mountainous watersheds of southeastern B.C. M.Sc. Thesis, University of British Columbia, Vancouver, B.C. 160p.

VanDriel, J.N. 1980. Computer-composite mapping for geologists. Environmental Geology, v.3, p.151-157.

Varnes, D.J. 1957. Landslide types and processes. In: Landslides and Engineering Practice, E.B. Eckel, ed. NAS-NRC Publication 54, p.20-47.

Varnes, D.J. 1978. Slope movement types and processes. In: Landslides: Analysis and Control. Special Report 176, Transportation Research Board, Washington, D.C., p.11-33.

Walmsley, M., Utzig, G., Vold, T., Moon, D. and VanBarneveld, J. 1980. Describing Ecosystems in the Field. R.A.B. Technical Paper 2, B.C. Ministry of Environment and B.C. Ministry of Forests.

Ward, T.J., Li, R.M. and Simons, D.B. 1978. Landslide potential and probability considering randomness of controlling factors. Report on file at Engineering Research Center, Colorado State University, Fort Collins, Colorado.

Wilford, D.J. and Schwab, J.W. 1982. A summary report on mass wasting in Rennel Sound. Report in progress, B.C. Ministry of Forests, Research Branch.

Wilson, G. 1976. Engineering pedology and its application to slope stability problems in British Columbia. 29th Canadian Geotechnical Conference on Slope Stability, 13-16 October, 1976, p.v-12 - v-37.

Wilson, G., McCormack, D.E. and Moon, D.E. 1982. Pedotechnical aspects of terrain analysis. Jour. of Geotech. Eng. Div., American Society of Civil Engineers, in press.

Wu, T.H., McKinnel, W.P.III, and Swanston, D.N. 1979. Strength of tree roots and landslides on Prince of Wales Island, Alaska. Can. Geotech. Jour., v.16, n.1, p.19-33.

Wu, T.H. and Swanston, D.N. 1980. Risk of landslides in shallow soils and its relation to clearcutting in southeastern Alaska. Forest Science, v.26, n.3, p. 495-510.

Youd, T.L. 1973. Liquifaction, flow and associated ground failure. USGS Circular 688.

Young, A. 1972. Slopes. Oliver and Boyd, Edinburgh, 288p.

Ziemer, R.R. 1981. Management of steepland erosion: an overview. Jour. of Hydrology (NZ), in press.

APPENDIX A

SOIL TEST DATA

					SAND					
NO	P.I.	STONES	COBBLES	GRAVEL	COARSE	MEDIUM	FINE	SILT	CLAY	USC
WS1	--	0	10	20	14.1	25.1	24.9	4.8	1.1	SP-SM
WS2	--	0	5	50	4.4	7.2	13.7	18.3	1.4	GM
WS3	15.3	0	0	0	7.7	33.5	14.1	34.4	8.5	SM
WS4	--	5	20	45	7.4	10.0	7.4	4.7	0.4	GW
WS5	--	3	40	20	5.6	7.5	11.2	9.6	3.0	GM
WS6	--	1	10	20	12.7	16.0	24.2	16.0	0.1	SM
WS7	--	0	0	8	4.9	19.9	30.2	34.9	2.0	SM
WS20	--	0	5	25	13.9	21.2	23.7	9.7	1.5	SP-SM
WS21	--	1	10	5	10.0	12.6	28.2	30.0	3.1	SM
WS23	--	5	5	5	6.4	10.7	42.1	23.4	2.4	SM
WS25	--	5	20	30	12.6	12.0	12.6	7.2	0.6	GP-GM
WS26	--	2	25	25	7.3	10.6	12.9	15.9	1.2	GM
WS27	--	0	0	0	2.2	8.9	40.0	44.2	4.6	SM
N10	--	0	0	20	10.8	20.2	22.0	24.0	2.9	SM
N11	--	30	15	20	5.5	11.6	7.1	1.0	1.1	GP
N12	--	10	10	15	9.9	20.9	15.7	15.9	2.6	SM
N13	--	10	15	20	8.8	15.8	11.6	16.0	2.8	GM
N14	--	5	5	20	10.3	25.4	15.4	15.4	3.4	SM
N15	--	0	0	0	0.6	1.2	32.4	53.9	11.9	ML
N17	--	0	10	40	12.3	20.7	14.8	1.9	0.3	GP
N18	--	5	10	30	10.1	14.0	13.6	15.4	1.9	GM
N19-1	--	5	15	20	4.8	16.6	20.8	15.9	1.8	GM
N19-2	--	5	15	20	15.6	29.9	12.3	1.4	0.7	SP
N20	--	5	20	15	15.0	22.8	10.2	10.5	1.4	SW-SM
N21	--	5	10	20	17.1	25.9	15.1	6.3	0.5	SW-SM
N22	--	0	2	7	3.4	11.4	32.0	36.3	7.8	SM
N0+80	2.7	0	1	5	4.4	13.2	21.4	42.8	12.2	ML
N1+100	--	1	5	35	10.3	15.8	13.2	18.1	1.7	SM
N1+100-2	5.6	1	5	35	8.0	19.1	12.7	15.7	3.4	SM
N2+40	2.8	5	5	15	9.3	22.8	16.3	19.3	7.2	SM
N3+25	--	5	25	35	8.0	11.2	8.0	6.6	1.1	GW-GM
N4+07	--	--	--	25.5	15.0	26.2	14.9	16.9	1.3	SM
R1	--	1	10	30	10.8	15.9	13.9	16.6	1.5	SM
R2	3.7	--	--	7.6	7.4	15.5	22.4	34.4	13.2	SM
R3	--	--	--	5.9	4.3	23.4	45.8	17.4	3.1	SM
R4	5.4	1	5	5	15.2	23.5	18.1	21.1	11.1	SM
R5	--	1	5	20	17.0	20.6	12.7	19.9	3.8	SM
R6	--	5	5	30	13.5	22.1	17.1	6.9	0.4	SW-SM
1	--	5	5	15	17.2	27.4	16.4	11.6	2.4	SM
2	--	--	--	43	14.5	19.7	12.0	9.5	1.5	SW-SM
3	11.0	--	--	19.2	13.5	26.1	11.9	20.1	9.1	SM

APPENDIX B

STOCHASTIC GEOTECHNICAL MODEL

The stochastic geotechnical model used in this study represents the summary and refinement of ideas presented by Swanston et al (1973), O'Loughlin (1974), Brown and Sheu (1975) and Simons et al (1976), as developed by Simons et al (1978). It assumes the hillside is of the 'infinite slope' variety. It determines an expected factor of safety, E[FS], and probability of failure, P. Input variables are those shown in Figure 3.1, each of which were discussed in Chapter 3. The deterministic model from which the probabilistic model is developed is equation 3.6.

The expected factor of safety is expressed as

$$E[FS] = L1(E[C] + E[Cr]) + L2(E[\tan\phi]) \tag{1}$$

The variance of the factor of safety is formulated as

$$\begin{aligned} VAR[FS] = {} & L1(VAR[C] + E[C] + 2E[C]\,E[Cr] + VAR[Cr] + E[Cr]) \\ & + L2(VAR[\tan\phi] + E[\tan\phi]) + 2L1\,L2\,E[\tan\phi]\,(E[C] + E[Cr]) \\ & - E[FS] \end{aligned} \tag{2}$$

In equations (1) and (2), the symbols E[ ] and VAR[ ] are the expected values and variances of the variable inside the brackets respectively. Assuming uniform distributions for input values, the expected value of a random value X is expressed as

$$E[X] = (Xmax + Xmin)/2 \tag{3}$$

and the variance as

$$VAR[X] = (Xmax - Xmin)/12 \tag{4}$$

where Xmax and Xmin are the upper and lower limits of the random input value respectively. The constants L1 and L2 are

$$L1 = \frac{2}{\gamma H \sin 2\beta (qo/\gamma H) + [(\gamma sat/\gamma)M] + (\gamma wet/\gamma)(1-M)} \tag{5}$$

and

$$L2 = \frac{(qo/\gamma H) + (\gamma sat/\gamma - 1)M + [\gamma wet/\gamma(1-M)]}{[(qo/\gamma H) + (\gamma sat/\gamma)M + (\gamma sat/\gamma(1-M))]\tan\beta} \tag{6}$$

Values of E[FS] and VAR[FS] computed from equations 1 and 2 can be used to estimate probability of failure. By definition, probability of failure is

$$p[FS \leq 1] = P \tag{7}$$

where P is the probability of failure and $p[FS \leq 1]$ is the cumulative probability that FS is less than or equal to 1.0. Ward (1976) found that a reasonable distribution of failure probability is the normal or Gaussian distribution which allows the computation of an approximate value of P by first determining the value of the non-dimensional variate U by using the equation

$$U = (1-E[FS])/(VAR[FS])^{0.5} \quad (8)$$

The value of U is then used to compute the cumulative failure 'k' by the expression

$$k = 0.4|U| \text{ if } |U| \leq 0.13 \quad (9)$$

or

$$k = -0.01314 + 0.49494|U| - 0.15804|U|^2 + 0.01661|U|^3$$
$$\text{if } |U| > 0.13$$
(10)

Equations 9 and 10 are approximations with errors of less than 1 percent. From U and k, the probability of failure P is found as

$$P = 0.5 + k \quad \text{if} \quad U>0$$
(11)

$$P = 0.5 - k \quad \text{if} \quad U<0$$
(12)

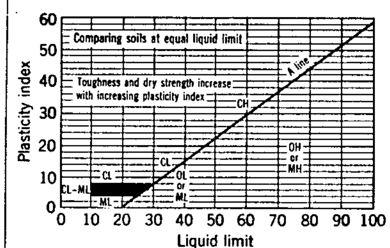
$$P = 0.5 \quad \text{if} \quad U=0$$
(13)

APPENDIX C

UNIFIED SOIL CLASSIFICATION

Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)				Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils More than half of material is *larger* than No. 200 sieve size[b] (The No. 200 sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than No. 4 sieve size (For visual classification, the ¼ in. size may be used as equivalent to the No. 4 sieve size)	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine, about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols $C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures		Atterberg limits below "A" line, or *PI* less than 4 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than No. 4 sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7

Identification Procedures on Fraction Smaller than No. 40 Sieve Size		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols	Typical Names	Information Required for Describing Soils
Fine-grained soils More than half of material is *smaller* than No. 200 sieve size	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

From Wagner, 1957.
[a] *Boundary classifications*. Soils possessing characteristics of two groups are designated by combinations of group symbols. For example *GW–GC*, well graded gravel-sand mixture with clay binder.
[b] All sieve sizes on this chart are U.S. standard.

Field Identification Procedure for Fine Grained Soils or Fractions

These procedures are to be performed on the minus No. 40 sieve size particles, approximately $\frac{1}{64}$ in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (Reaction to shaking):
After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.
Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.
Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (Crushing characteristics):
After removing particles larger than No. 40 sieve size, mould a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.
High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (Consistency near plastic limit):
After removing particles larger than the No. 40 sieve size, a specimen of soil about one-half inch cube in size, is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eight inch in diameter. The thread is then folded and re-rolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.
After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.
The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.
Highly organic clays have a very weak and spongy feel at the plastic limit.

APPENDIX D

RELATIVE DENSITY DETERMINATION TECHNIQUE

CONSISTENCY	q_u (Tsf)[1]	RULE-OF-THUMB	BLOWS PER FOOT[2]
Very soft	0.25	Core (Height = twice the diameter) sags under own weight	0 - 1
Soft	0.25 - 0.50	Can be pinched in two between thumb and forefinger	2 - 4
Firm	0.50 - 1.00	Can be imprinted easily with fingers	5 - 8
Stiff	1.00 - 2.00	Can be imprinted with considerable pressure from fingers	9 - 15
Very stiff	2.00 - 4.00	Barely can be imprinted by pressure from fingers	16 - 30
Hard	4.00+	Cannot be imprinted by fingers	Over 30

[1] q_u is unconfined compressive strength in tons/sq.ft.

[2] Blows as measured with 2-in. OD, 1 3/8-in. ID sampler driven 1 ft by 140-lb hammer falling 30 in. See Tentative Method for Penetration Test and Split-Barrel Sampling of Soils, ASTM Designation: D1586-58T.

APPENDIX E

TERRAIN CLASSIFICATION SYSTEM

Texture

	SIZE mm / ROUNDNESS	> 256	256–64	64–2	2–.062	.062–.0039	< .0039
specif	ROUNDED	BOULDERY b	COBBLY k	PEBBLY p			
specif	ROUND OR ANGULAR				SANDY s	SILTY $\not{s}$	CLAYEY c
common	ROUNDED	GRAVELLY g					
common						FINES f	
common	ANGULAR	BLOCKY a	RUBBLY r				

Genetic Material

Anthropogenic -- A
Colluvial ------ C
Eolian --------- E
Fluvial -------- F
Ice ------------ I
Lacustrine ----- L
Morainal ------- M
Organic ------- O
Bedrock -------- R
Saprolite ------ S
Volcanic ------- V
Marine --------- W

Qualifying Descriptor

Bog ------------ B
Fen ------------ F
Swamp ---------- S
Glacial -------- G
Active --------- A
Inactive ------- I

Surface Expression

Apron ---------- a
Blanket -------- b
Fan ------------ f
Hummocky ------- h
Level ---------- l
Subdued -------- m
Ridged --------- r
Steep ---------- s
Terraced ------- t
Veneer --------- v

Modifying Process

Avalanched --------- A
Bevelled ----------- B
Cryoturbated ------- C
Deflated ----------- D
Channeled ---------- E
Failing ------------ F
Kettled ------------ H
Karst modified ----- K
Nivated ------------ N
Piping ------------- P
Solifluncted -------- S
Gullied ------------ V
Washed ------------- W

Texture — g
Genetic Material — F
Qualifying Descriptor — G
Surface Expression — b
Modifying Process — F

gF^{G}b – F

APPENDIX F

LANDSLIDE DATA AND LOCATIONS

LANDSLIDES ASSOCIATED WITH ROADS ON SLOPES MANTLED WITH SURFICIAL MATERIAL

		SLIDE DIMENSIONS[2]								BED-			
NO.	TYPE[1]	L	W	D	VOL.	β_i^3	β_s	TCS	USC	ROCK		M	IMPACT
D1	DA-DF	390	10	5	9800	34°	33°	fgMb	GW-GM	GRAN	2-3	.2	road washouts and stream siltation
D2	DA	6	20	3	360	40°	30°	rCv	GM	SS	3	--	fill slope debris in gully
D3	R	50	20	2	2000	44°	44°	rCb	GW	PHY	4	--	fill slope faiulure causing stream siltation
D5	SF	2	15	.5	15	35°	26°	rMb	GW	PHY	3	--	damage to road bed from organic debris deter.
D11b	R	--	--	.5	--	38°	36°	$gF^{G}b$	GP	ARG	2	--	fill slope damage and stream siltation
D12	R	75	25	2	3750	42°	39°	$sF^{G}b$	SP	GRAN	3	--	fill slope damage and stream siltation
D14	DA-DF	1500	6	1	2700	32°	35°	rCb	GW	GRAN	3	1.0	debris flow blocked major highway
D15	SC	9	100	1	900	65°	36°	$sF^{G}b$	SC	--	3	--	water diversion leading to road blockage
D17	DA	15	7	1	105	--	34°	sMb	SW	--	3-4	--	damage to road bed
D18	DA-DF	60	15	4	3600	45°	38°	sMb	SW	PHY	3	.1	road washout stream siltation
D19	SC	6	25	1.5	225	62°	28°	sMb	SP-SW	MONZ	5	.5	road blockage from fill slope failures
D20	SC	6	6	2	72	52°	30°	fsMb	SW-SM	ARG	3	--	road blockage from fill slope failures
S1	R	------ravel-------				75°	36°	$gF^{G}b$	GP	--	2-3	--	road blockage
A1	DA	40	30	.5	600	40°	40°	$gF^{G}b$	GW-GP	PHY	3-4	--	damage to road
A6	SC	20	10	10	200	50°	50°	rMb	--	--	--	--	

LANDSLIDES ASSOCIATED WITH ROADS IN STEEP ROCKY TERRAIN

NO.	TYPE[1]	SLIDE DIMENSIONS[2] L	W	D	VOL.	β_i^3	β_s	TCS	USC	BED-ROCK	MR	M	IMPACT
D4	RF	----- rockfall -----				84°	44°	rCv/R	GW	SYE	2	--	Rockfall on road causing blockage
D9	DA	90	10	1.5	1400	47°	47°	rCv=R	GW	PHY	3	--	Damage to creek by blasted rock
D10	RF-DA	30	10	2	600	65°	41°	rCv=R	GW	DIOR	2	--	Road blockage, damage to creek
D13	RF-DA	--- rock failure ---				66°	43°	R/rCv	rubb	GRAN	3	--	Road blockage, damage to creek
A5	DA	40	8	3	960	80°	39°	rCv=R	rubb	GRAN	2	--	Damage to creek, road bed threatened

LANDSLIDES ASSOCIATED WITH ROADS ON COLLUVIAL APRONS

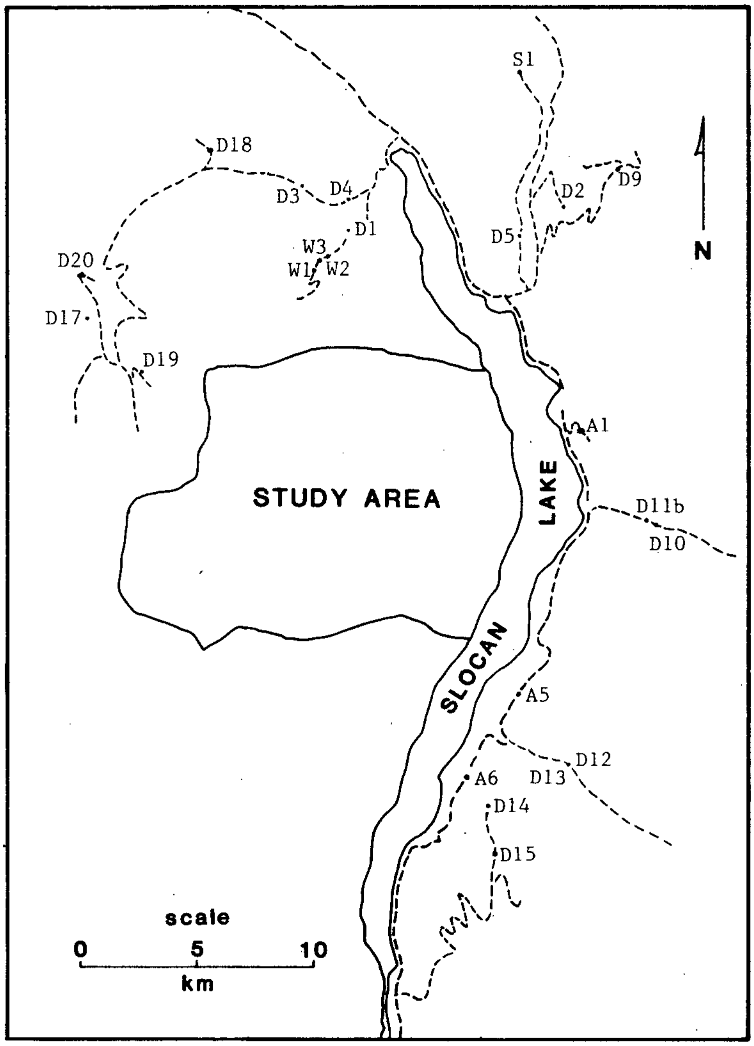
NO.	TYPE	L	W	D	VOL.	β_i	β_s	TCS	USC	BED-ROCK	MR	M	IMPACT
W1	R	-- discrete ravel --				54°	40°	rCa	GW	GRAN	2	--	Road blockage
W2	R	-- discrete ravel --				58°	42°	rCa	GW	GRAN	2	--	Road blockage
W3	R	-- discrete ravel --				46°	41°	rCa	GW	GRAN	2	--	Road blockage

[1]R-Ravel DA-Debris avalanche RF-Rock failure SF-Fill slope failure SC-Cut slope failure DF-Debris flow

[2]in meters

[3] β_i-slope inclination at zone of slide initiation β_s-inclination of entire hillslope

TCS-Terrain Classification USC-Unified Soil Classification MR-Moisture Regime M-Relative height of water table with respect to the shear plane (see Chapter 3)


S1
D18
D3
D4
D1
W3
W1
W2
D20
D17
D19
D2
D9
D5
N
A1
STUDY AREA
LAKE
SLOCAN
D11b
D10
A5
D12
D13
A6
D14
D15
scale
0
5
10
km