LANDSLIDE INITIATION: A UNIFIED GEOSTATISTICAL AND PROBABILISTIC MODELLING TECHNIQUE FOR TERRAIN STABILITY ASSESSMENT

by

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We accept this thesis as conforming to the required standard

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Date Oct. 15, 1996
The terrain attributes data that are described, and for which a discriminant analysis is reported, in Chapter 7 were collected as part of a large study, "Landslide frequencies following logging: Coastal British Columbia (Terrain Attribute Study D33)", by the British Columbia Ministry of Forests (BCMoF). The project leaders were Mr. T.P. Rollerson and Mr. B. Thomson.

A portion of the database was made available to this research project. That portion of the data, for primarily till-dominated open slope landforms, has been used to establish the Terrain Attributes Factor (F) reported in Chapter 7. The data remain the property of the BCMoF, and are confidential. At the request of the BCMoF, an understanding has been given not to publish details of the discriminant analysis reported in Chapter 7, including all text, tables and Figures 7.1, 7.3 and 9.1, and Appendices C and D. The same request is made of the reader. For citation of model development, the reader is also referred to Wilkinson and Fannin (1997).

The reader is cautioned that a subsequent supplemental validation of the database by the BCMoF has revealed errors and omissions in the data made available to this project. Specifically, of the 1526 polygons used in this study, approximately 6% were discovered by the author to have been mistakenly classified as stable or unstable. Any improvement to the segregation of stable and unstable polygons would likely yield a revision to the values of the Terrain Attribute Factor.

The analysis carried out for this project combined terrain attribute data from various regions of Vancouver Island. In making this regional analysis, the reader is also cautioned to interpret and extrapolate the present work with care, and with recourse to local climate, soils and experience.

Reference:

ABSTRACT

Five open slope, translational landslides which have occurred on four separate clear-cut hillslopes in coastal British Columbia are described in terms of their qualitative, geomorphological characteristics and their hydrogeological response to extreme precipitation. The design and operation of a displacement rate controlled, large scale, in-situ direct shear box, which was used to conduct shear strength tests at three of the four locations and on laboratory-reconstituted samples, is described. A mean friction angle of 47° and a root or soil structure cohesion of 1.5 kPa is interpreted from the in-situ and laboratory testing program to be appropriate for the tested shallow, colluvial gravelly sands.

Approximately eight years of near-continuous, shallow piezometric data, collected at the Carnation Creek Experimental Watershed by the Canadian Forest Service, are used to determine the piezometric response of shallow colluvial soils to extreme precipitation events and to determine an appropriate probabilistic distribution for use in Monte Carlo-like simulations of slope stability. An extreme value distribution, which is dependent on the duration of observation, is proposed for this purpose. Piezometric data and the results of infinite slope stability back-analyses indicate the potential for short-term pore pressures which are in excess of hydrostatic pressures, and potentially artesian, to develop during extreme precipitation events.

Optimized terrain attribute data, collected by the British Columbia Ministry of Forests, for a subset of 1,526 mapped terrain polygons are combined with the results of Monte Carlo-like slope stability simulations of the same polygons to create a unified, geostatistical (qualitative terrain attributes) and probabilistic (quantitative slope stability) landslide initiation model. The resulting probabilities of initiation, P(In), for each polygon are compared to existing slope stability assessment techniques used in the forest sector. The proposed assessment technique is intended to represent one component of a multi-disciplinary, quantitative risk assessment approach which considers all hazards to downslope resources and the specific risk of each element at risk.
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NOMENCLATURE AND ACRONYMS

b Location parameter, Gumbel Extreme Value statistic

C_i Terrain Attribute Factor coefficient for ‘i’ ranked terrain attribute

C_c Coefficient of curvature (dimensionless)(C_c = D_{30}^2/D_{60}D_{10})

C_{decay} Relative root cohesion due to decay (0 ≤ C_{decay} ≤ 1)

C_{regrowth} Relative root cohesion due to regrowth (0 ≤ C_{regrowth} ≤ 1)

C_f Root cohesion (over ‘t’ years or at failure) (kPa)

C_s Soil cohesion (real or apparent) (kPa)

C_u Coefficient of uniformity (dimensionless)(C_u = D_{60}/D_{10})

C_{uncut} Root cohesion prior to harvesting (maximum) (kPa)

c_v Coefficient of consolidation (m^2/s)

CCC Cubic Clustering Criterion

CDF Cumulative distribution function, non-exceedence probability, P_n, integral of PDF

D Distance from expected or observed failure plane to ground surface (m)

D_w Distance from expected or observed failure plane to groundwater surface (m)

D_{10} 10% passing particle size (mm)

D_{16} 16% passing particle size (mm)

D_{30} 30% passing particle size (mm)

D_{50} 50% passing particle size (mm)

D_{60} 60% passing particle size (mm)

D_{84} 84% passing particle size (mm)

DEM Digital Elevation Model

e Void ratio (volume of voids / volume of solids)

ESTF Earth Sciences Task Force

F Terrain Attribute Factor

f Partial factor

f_i Partial factor for ‘i’ ranked terrain attribute

f_x(x) Univariate distribution in terms of ‘x’

[f_{ij}(\beta,F)]_{stable} Bivariate distribution of stable polygons

[f_{ij}(\beta,F)]_{unstable} Bivariate distribution of unstable polygons

FPC Forest Practices Code of British Columbia

FS Factor of Safety, ratio of available shear strength to mobilized shear stress

G_s Specific gravity
<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>GIS</td>
<td>Geographic Information System</td>
</tr>
<tr>
<td>GPS</td>
<td>Global Positioning System</td>
</tr>
<tr>
<td>GVRD</td>
<td>Greater Vancouver Regional District</td>
</tr>
<tr>
<td>h</td>
<td>Length of drainage path (m)</td>
</tr>
<tr>
<td>i</td>
<td>Rank number of specific monthly maximum in ordered list of maximums</td>
</tr>
<tr>
<td>k</td>
<td>Hydraulic conductivity (cm/s or m/s)</td>
</tr>
<tr>
<td>M</td>
<td>Constrained modulus (bars)</td>
</tr>
<tr>
<td>m</td>
<td>Scale parameter, Gumbel Extreme Value statistic</td>
</tr>
<tr>
<td>N</td>
<td>Number of periods of observation (months)</td>
</tr>
<tr>
<td>n</td>
<td>Total number of monthly maximums in ordered list of maximums</td>
</tr>
<tr>
<td>n&lt;sub&gt;f&lt;/sub&gt;</td>
<td>Number of polygons in group ‘i’ or exponent to partial factor, f</td>
</tr>
<tr>
<td>n&lt;sub&gt;stable&lt;/sub&gt;</td>
<td>Number of stable polygons</td>
</tr>
<tr>
<td>n&lt;sub&gt;unstable&lt;/sub&gt;</td>
<td>Number of unstable polygons</td>
</tr>
<tr>
<td>P(In)</td>
<td>Probability of initiation</td>
</tr>
<tr>
<td>P&lt;sub&gt;n&lt;/sub&gt;</td>
<td>Non-exceedence probability</td>
</tr>
<tr>
<td>PDF</td>
<td>Probability density function, derivative of CDF</td>
</tr>
<tr>
<td>q&lt;sub&gt;0&lt;/sub&gt;</td>
<td>Surface surcharge (due to trees) (kPa)</td>
</tr>
<tr>
<td>r</td>
<td>Coefficient of correlation</td>
</tr>
<tr>
<td>R&lt;sub&gt;t&lt;/sub&gt;</td>
<td>Return period (months or years)</td>
</tr>
<tr>
<td>RIC</td>
<td>Resources Inventory Committee</td>
</tr>
<tr>
<td>t</td>
<td>Time since harvest (root decay and regrowth) (years)</td>
</tr>
<tr>
<td>t&lt;sub&gt;f&lt;/sub&gt;</td>
<td>Time to failure (minutes or seconds)</td>
</tr>
<tr>
<td>t&lt;sub&gt;lag&lt;/sub&gt;</td>
<td>Time lag between harvest and planting (root regrowth) (years)</td>
</tr>
<tr>
<td>TFL</td>
<td>Tree Farm Licence</td>
</tr>
<tr>
<td>USCS</td>
<td>Unified soil classification system</td>
</tr>
<tr>
<td>w&lt;sub&gt;n&lt;/sub&gt;</td>
<td>Moisture content (%)</td>
</tr>
<tr>
<td>y</td>
<td>Dependent variable</td>
</tr>
<tr>
<td>A</td>
<td>Empirical constant, root regrowth</td>
</tr>
<tr>
<td>α</td>
<td>Slope angle (°) (%)</td>
</tr>
<tr>
<td>B</td>
<td>Empirical constant, root regrowth</td>
</tr>
<tr>
<td>β</td>
<td>Reliability index</td>
</tr>
<tr>
<td>X</td>
<td>Empirical constant, root regrowth</td>
</tr>
<tr>
<td>δ</td>
<td>Shear displacement (mm)</td>
</tr>
<tr>
<td>δ&lt;sub&gt;Error&lt;/sub&gt;</td>
<td>Error of difference between two data sets (used in difference of means test)</td>
</tr>
<tr>
<td>δ&lt;sub&gt;Means&lt;/sub&gt;</td>
<td>Difference of means between two data sets (used in difference of means test)</td>
</tr>
<tr>
<td>φ&lt;sub&gt;cv&lt;/sub&gt;</td>
<td>Constant volume, or large displacement, friction angle (°)</td>
</tr>
</tbody>
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Nomenclature and Acronyms

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
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<tbody>
<tr>
<td>$\gamma$</td>
<td>Moist unit weight (kN/m$^3$)</td>
</tr>
<tr>
<td>$\gamma_{sat}$</td>
<td>Saturated unit weight (kN/m$^3$)</td>
</tr>
<tr>
<td>$\gamma_w$</td>
<td>Unit weight of water (kN/m$^3$)</td>
</tr>
<tr>
<td>$\eta$</td>
<td>Drainage condition parameter (dimensionless) (Bishop and Henkel, 1957)</td>
</tr>
<tr>
<td>$K$</td>
<td>Empirical constant, root regrowth</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>Empirical constant, root decay</td>
</tr>
<tr>
<td>$\mu_i$</td>
<td>Mean of group ‘i’</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Empirical constant, root decay</td>
</tr>
<tr>
<td>$\sigma_i$</td>
<td>Standard deviation of group ‘i’</td>
</tr>
<tr>
<td>$\sigma_n'$</td>
<td>Normal effective stress (kPa)</td>
</tr>
<tr>
<td>$\tau$</td>
<td>Shear stress (kPa)</td>
</tr>
<tr>
<td>$\xi$</td>
<td>Random variable, Extreme Value quantile: (-ln(-ln($P_n$)))</td>
</tr>
<tr>
<td>$\Psi$</td>
<td>Critical state parameter (dimensionless)</td>
</tr>
</tbody>
</table>
ACKNOWLEDGMENTS

Data, bright ideas, encouragement and financial support have been graciously provided by numerous individuals, organizations and agencies.

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1. INTRODUCTION

1.1 Geotechnical Engineering in the Forest Industry

The introduction and ongoing evolution of the Forest Practices Code of British Columbia Act (FPC), regulations, standards and guidebooks has dramatically changed the method by which the forest industry operates in British Columbia, Canada. In addition to the synthesis of many different codes, regulations and bodies of knowledge and experience into a single comprehensive and enforceable act, the FPC has also defined the role and responsibilities of professionals in the forest sector and an acceptable level of performance for forest resources management. The need to quantify the potential damage to human life, non-timber resources such as fisheries habitat, infrastructure and property due to natural hazards has become a problem which spans many aspects of research, community and resource planning and consulting, as well as this new code.

Agency, consulting and industry professionals involved in the forest sector are frequently asked to provide estimates of the potential for mass movement initiation, debris flow magnitude, runout behaviour, routing, travel distance and recurrence, type and quantity of material delivered to downslope life, resources, infrastructure and fish-bearing streams and the potential for and type of damage to these resources due to impact. This is obviously a multidisciplinary question involving the fields of forestry, geomorphology, geotechnology, subsurface and surficial hydrogeology, ecology, biology - and a healthy dose of experience.

The role of a practising geotechnical engineer or geoscientist in the forest sector is to provide expertise concerning terrain mapping and stability assessment, road layout, construction and deactivation, culvert and bridge location and design, and landslide rehabilitation. These duties, and the standard of
practice of these duties, are evolving and being re-defined continuously. Gerath (1995) discusses the importance of studying the cause and effect of past landslides, reviews the development of risk assessment techniques and terminology and states the need to quantify both the estimated risk at a site and the "acceptable risk". Gerath (1995) continues, saying:

"A professional engineer or geoscientist cannot, however, be reasonably expected to certify the non-occurrence of hazards prior to or following tree harvesting. Uncertainties over landslide triggering phenomena also make it unreasonable to assign legal liability on the basis of scientific assurance."

1.2 Statement of Problem and Project Objectives

Public and private concern over the initiation of landslides from forest clearcuts, and the effects of debris in fish-bearing streams, led to the establishment of the 5-year Fish/Forestry Interaction Program in 1981. The objectives of this multidisciplinary research were:

- to study the extent and severity of mass wasting and to assess its impact on fish habitat and forest sites;
- to investigate the feasibility of rehabilitating stream and forest sites damaged by landslides;
- to assess alternative silvicultural treatments for maintaining and improving slope stability; and
- to investigate the feasibility and success of using alternative logging methods, including skylines and helicopters, and by forest development planning to reduce logging related failures.

In addition to the technological advances which have resulted from these broad research studies, significant pressure for the development and implementation of new techniques has arisen from the introduction of the FPC. In particular, fines of up to $100,000 under the April 1995 Forest Road and Timber Harvesting Practices Regulations (BCMOF, 1995a) for excessive soil disturbance or failure to construct or modify forest roads appropriately (amongst others) underscore the need for rational and defensible tools for terrain stability analysis. The need for formal risk assessment techniques will likely be further emphasized by the move toward an explicit statement, quantification and adoption of
acceptable levels of risk within the forest resource sector. Levels of acceptable risk will likely evolve from the continued dialogue between government agencies, environmental groups, industry representatives, researchers and the public, and the consulting professional’s willingness to assume liability.

The purpose of this work is to provide a unified geostatistical and numerical modelling technique for the probabilistic assessment of terrain stability on open, planar slopes. Such a technique could be used in the forest sector as a stand-alone planning and index tool, or as one component of an assessment of risk to downslope resources, such as timber, fish habitat, infrastructure right-of-ways and private property.
Chapter 2. Slope Stability Assessment: An Introduction

2. SLOPE STABILITY ASSESSMENT: AN INTRODUCTION

Landslide hazard mapping is currently used as one component of multidisciplinary 'cumulative impact' assessments. These broader scope assessments attempt to aid investigators who are involved in land use, project and forest operations planning. The objectives of such an investigation are to make use of the landslide, erosion, wildlife, vegetation, fisheries and fire hazard mapping components in order to provide protection of life, and public and private property and to provide for protection of sensitive areas against environmental damage.

The state-of-the-art in hazard mapping has reached a turning point where digital analysis techniques, using the combined database and spatial capabilities of Geographic Information Systems (GIS) as well as Global Positioning (GPS) technologies, are used in conjunction with conventional interpretive mapping techniques to provide descriptive (qualitative) and quantitative measures of hazard over a mapped region. A review of several current techniques (Thurber Engineering Ltd., 1994) has revealed the importance of avoiding the potential for oversimplification with descriptive/subjective techniques and the need to incorporate judgment and experience in quantitative techniques. Achieving the appropriate balance and mix of these two different approaches seems to be a future challenge.

The most thorough account of current and proposed landslide hazard mapping techniques in British Columbia is in draft by the 'Landslide Task Group' of the Resources Inventory Committee (RIC) (VanDine, Hungr and Gerath, 1996 draft). The intent of this sub-committee of the Earth Sciences Task Force (ESTF), which is itself a sub-committee of the RIC, is to establish guidelines and standards for procedures, levels of effort and required expertise for the mapping of landslide hazards in British Columbia. Other objectives of the task force include the development of common terminology, symbols and classifications and to encourage and prepare for the wide-spread application
VanDine, Hungr and Gerath (ESTF, RIC, 1996) have discussed the options which are open to a professional who is responsible for a detailed or on-site field terrain stability assessment. These options include the selection of a type of hazard mapping, such as qualitative (relative), semi-qualitative or quantitative (probabilistic) mapping. Nine different landslide hazard mapping methods have been identified, ranging from the subjective and qualitative ‘Subjective Geomorphic’ method to the objective and quantitative ‘Probabilistic Multivariate’ and ‘Slope Stability’ methods. It has been recognized that no single method is likely to be the most appropriate for every project and data set, and that clear advantages and disadvantages exist for each method. VanDine, Hungr and Gerath (ESTF, RIC, 1996) also list three components of an ideal hazard mapping technique:

1. Identification of the process or predominate type and mechanism of a potential landslide.
2. An estimate of the magnitude (volume or area) of a potential landslide.
3. An estimate of the temporal and/or spatial frequency of a potential landslide, such as a landslide density (number per unit area), proportion of unstable polygons or annual probability of initiation or occurrence.

A further refinement to this ideal hazard mapping method would allow for adjustment of hazard ratings according to the type of proposed activity, such as harvesting, road building, road deactivation and site rehabilitation.

The advantages of a quantitative approach, involving a well-understood and rigorous scientific method, include the provision of a definitive and tangible factor of safety or probability of failure and a
defensible or rational method. The obvious disadvantages include the prohibitive expense of conducting a thorough investigation of an entire watershed. In contrast, the attractions of qualitative, geostatistical methods include the exploitation of prior experience (in a rigorous and statistical approach) concerning conditions which lead to failure, the ability to include intangible field evidence and qualitative descriptions in an assessment and their suitability for use at a watershed scale. The classic and most profound disadvantage of a qualitative approach is the inevitable subjective nature of the hazard rating, resulting primarily from slight variations in definitions and concepts of risk amongst professionals. Successful attempts to incorporate the positive features of both approaches would result in methods suitable for use on a watershed planning scale or for a single polygon, incorporating past experience and using a balance of well-defined and readily measured physical inputs and qualitative descriptors.

Various sedimentation and mass wasting estimation techniques have been proposed and described by others (Swanston and Marion, 1991; WEST Consultants, Inc., 1996; Thurber Engineering Ltd., 1983). A definitive relationship between logging and the incidence of open slope landslides is, however, difficult to establish. For instance, although it has been established that most landsliding events are related to extreme precipitation, a distinct relationship between logging and increased runoff or piezometric pressures has not been established. Jackson et al. (1985) and Thurber Engineering Ltd. (1983) state that the combination of repeated winter storm cycles and low evapotranspiration lead to antecedent moisture conditions and thoroughly saturated soils, which outweigh the effects of forest cover on runoff. A study of the relationship between flood frequencies and clearcutting by Harr and McCorison (1979) in Oregon resulted in the counter-intuitive observation of decreased flood frequencies in clearcut areas. They postulated that rapid melting of snow caught in trees led to increased overall runoff flows and the ensuing floods. Similarly, Thurber Engineering Ltd. (1983) observed Charles Creek, despite its relative lack of logging, to be one of the largest debris flow
sources on the east side of Howe Sound, and further, that slides within four other drainages had originated from completely unlogged areas.

Swanson and Marion (1991) used 1963 and 1983 aerial photography to develop average mass wasting frequency estimates for this observation period, over an area of approximately 42,000 km$^2$ of the Tongass National Forest in southeast Alaska. One objective of the study was to statistically compare the occurrence of landslides in logged areas versus unlogged areas. 1,277 natural landslides, over an area of 41,503 km$^2$, were observed to have occurred over this period, resulting in an average landslide frequency of 0.0015 landslides/km$^2$/year, or 0.00031 landslides/ha during this twenty years. In contrast, the average landslide frequency for logged areas was 0.0053 landslides/km$^2$/year (103 landslides over 980 km$^2$), or 0.0011 landslides/ha during these twenty years. Swanston and Marion (1991) also observed only about 15% of all debris flows and 34% of all debris torrents to reach perennial streams directly.

In this study four landslides which have occurred in coastal British Columbia are described in terms of their qualitative, geomorphological characteristics, their shear strength characteristics, and their hydrogeological response to extreme precipitation. In particular, the objectives of this research are to:

- Characterize and document several polygons from which open slope failures have initiated, using procedures and categories described by Fannin and Rollerson (1993), the ‘Mapping and Assessing Terrain Stability Guidebook’ (BCMOF, 1995b), and the BC Terrain Classification System (Howes and Kenk, 1988).

- Augment shear strength testing results, published by Skermer and Hillis (1970) and Leps (1970), for coarse granular material by conducting in-situ and laboratory reconstituted direct shear tests on typical hillslope materials at low confining stresses.
Chapter 2. Slope Stability Assessment: An Introduction

- Report and comment on the results of statistical analyses of near-continuous piezometric data from a coastal Vancouver Island watershed in order to determine appropriate probability density functions for use in quantitative slope stability analyses.

- Augment and confirm studies completed by Hammond et al. (1992), amongst others, to verify the applicability of the infinite slope model to forest hillslope stability and to confirm the appropriateness of proposed piezometric probability density functions and shear strength characteristics.

- Develop and report, after Rollerson and Sondheim (1985) and Rollerson (1992), a subset of statistically significant terrain attributes, which, when partial factors are assigned to each significant attribute code, best distinguish between the groups of stable and unstable polygons within a set of 1,526 mapped polygons.

- Conduct and report the results of Monte Carlo - like slope stability simulations of the same 1,526 mapped polygons, using reported shear strength and piezometric distributions and mapped polygon characteristics, to determine the likelihood that the factor of safety against translational sliding of an open slope is less than or equal to unity for each polygon.

- Define a unified geostatistical and probabilistic modelling technique for terrain stability assessment by combining the qualitative results of the terrain attribute study and the quantitative results of the slope stability simulations into one rigorous and defensible technique.
Chapter 3. Test Site Characterization

3. TEST SITE CHARACTERIZATION

3.1 General

The use of a physically-based numerical model for slope stability analysis requires a reliable estimate of input parameters for each site. A clear understanding of the probable failure mechanisms, and the geological and physical characteristics of a site, are essential to such a reliable stability assessment. In particular, estimates of depth to the failure plane, shear strength of the failed material, slope angle and groundwater regime at the time of failure are required.

This chapter introduces five landslides which have occurred at four separate locations in coastal British Columbia and are used to validate the infinite slope model and to bound the input parameters. Each validation site is described with respect to an assessment of slope stability, using current techniques, and the slope geology and morphology. Results of strength and soil classification tests performed at each validation site are reported in Chapter 4, including a complete summary of the site characteristics. The infinite slope model, its assumptions, input parameters and sensitivity are discussed in Chapter 6, in addition to the results of detailed back-analyses of the four landslide validation sites.

3.2 Validation Sites

Four clearcut sites in southwestern, coastal British Columbia were chosen for the purpose of back-analysis and validation of the infinite slope model. The criteria by which these four sites were chosen were:

- clearcut polygon,
- site of open slope failure,
- relatively little influence from road building and windthrow,
Chapter 3. Test Site Characterization

- failure well-approximated by infinite slope model (plan dimensions many times greater than surficial material thickness),
- colluvial or morainal surficial materials with well-defined failure plane and without excessive boulder content, and
- reasonable vehicle and foot access for transportation of field shearbox equipment.

The extent to which most landslide sites fit these criteria is variable and rarely complete, and, to some extent, the four landslide sites which were used for back-analysis only partially meet these criteria as discussed below.

The approximate location of each site is illustrated in Fig. 3.1: three are located on Vancouver Island and the fourth is in the Seymour Watershed of the Greater Vancouver Regional District (GVRD). A detailed site investigation was performed at each site; it included terrain stability mapping in accordance with the state-of-the-practice (Ryder, 1994) techniques, completion of the Ministry of Forests Landslide Data Card (Appendix A), soil sampling and shear strength testing (as described in Chapter 4). Terrain mapping showed all four sites to have slopes between 27° (51%) and 36° (73%) and to be either Class III or Class IV terrain according to the terrain stability classification system used within the Vancouver Forest Region (BCMOF, 1995b), see Appendix A.

All four sites are located in the Coastal Western Hemlock or Douglas-fir biogeoclimatic zones (Valentine, Sprout, Baker and Lavkulich, 1986; Acres International, 1993 and BCMOF, 1992) (see Fig. 3.2), which are characterized by ferro-humic or humo-ferric podzol soil landscapes and an average annual precipitation in excess of 2000 mm.

The general geology of the mainland Coast Mountains is granitic (primarily granodiorite and quartz diorite) rocks with minor gneiss and schist, while the Vancouver Island Mountains are primarily
folded sedimentary and metamorphic rocks with granitic batholiths (Fig. 3.3) (Muller, 1977; Valentine et al., 1986).

3.2.1 Seymour Watershed - Jamieson Creek Landslide

The Seymour Watershed is one of several watersheds operated and maintained by the GVRD which provide drinking water for Vancouver. The landslide studied within the Seymour Watershed is commonly referred to as the Jamieson Creek landslide due to its proximity to Jamieson Creek. The position of the Seymour Watershed and the Jamieson Creek landslide site are shown in Figs. 3.4 and 3.5 respectively. The site is at an elevation of approximately 850 m and located within the Very Wet Maritime (CWHvm) subzone of the Coastal Western Hemlock biogeoclimatic zone (Acres International, 1993).

The Jamieson Creek landslide occurred during or following the second of two large consecutive rain-on-snow events of the 1990-1991 winter (Thurber Engineering, 1991). Mean annual precipitation at this location and elevation is reported to be approximately 3300 mm, with a maximum 24-hour precipitation in excess of 300 mm (Acres International, 1993).

3.2.1.1 Site Geology and Slope Morphology

The plutonic bedrock at the Jamieson Creek landslide is comprised primarily of diorite with minor amounts of gabbro, quartz diorite, leucocratic quartz diorite and granodiorite of the Cretaceous period. The exposed bedrock in the area of the initiation zone is relatively planar, with small undulations, and slightly weathered (Plate 3.1). Surficial materials are typically highly weathered morainal and colluvial sandy gravels with frequent sub-rounded to angular cobbles and boulders and discontinuous areas of hard, unweathered morainal material, overlain by ferro-humic podzols and a relatively dense root mat of approximately 0.5 m thickness. Discontinuous seepage points and zones are located
predominantly around the head and east side scarps of the landslide, within and slightly below the root mat level. The terrain polygon from which the landslide initiated has a slope angle of approximately 36° (73%) and a southerly aspect. The initiation zone is observed to be on a ridge of convex curvature but located within a slight natural drainage depression.

The headscarp and initiation zone of the failure are well suited to the infinite slope model, with the initiating length and width being many times greater than the depth of the surficial deposits and a relatively planar shear surface (Plate 3.1). The shear surface is estimated to have been between 0.7 m and 1.5 m below ground surface, with an average depth of approximately 1.25 m, and located at the interface of the weathered colluvial and morainal deposits with the unweathered bedrock and till. The shear surface was also located below the root mat, limiting root cohesion contributions to the periphery of the initiation zone.

Upon initiation, the sliding volume of approximately 5500 m$^3$ liquefied and flowed over an access road, was confined slightly by the lower slope morphology and finally entered Jamieson Creek as a debris torrent (Plate 3.2). The debris was transported a further 1500 m, past the confluence of Jamieson and Orchid creeks (Thurber Engineering, 1991).

3.2.1.2 Stability Assessment

Terrain mapping revealed the polygon from which the landslide initiated to be one of stability class IV. The Acres International report to the GVRD reports the same area, along with 38% of the study area shown in Fig. 3.5 to be of stability class V. In general, this difference may be attributed to minor, site and mapper specific differences between descriptions of class IV and V stability classifications, such as the necessary and sufficient presence of natural landslides for the assignment of class V, and relative estimates of the likelihood of landslide initiation. Specifically, however, this
difference may be attributed to the importance placed on much smaller, surficial slumping to the immediate west of the Jamieson Creek landslide and on significant tension cracks, located upslope of the existing headscarp. At least two continuous tension cracks within 50 m upslope of the existing headscarp indicate the potential for further instability at this site. This difference between classifications again demonstrates the potential benefits of a more objective and defensible landslide hazard rating technique.

### 3.2.2 Carnation Creek - ‘Eugene’ and ‘Bob’ Landslides

Carnation Creek (see Figs. 3.1, 3.2 and 3.6) is located on the central west coast of Vancouver Island, approximately 20 km east of the town of Bamfield. This small (10 km$^2$) watershed drains into Barkley Sound and is part of Tree Farm Licence (TFL) 44. Hetherington (1982) reports an annual precipitation of 2100 mm to 4800 mm, 75% of which falls between October and March, and 95% of which falls as rain. The watershed topography consists of steep slopes to 700 m elevation with a narrow valley bottom. Carnation Creek is also located within the Very Wet Maritime subzone of the Coastal Western Hemlock biogeoclimatic zone.

As well as being part of TFL 44, Carnation Creek has also served as a multi-disciplinary and long-term experimental watershed since 1970. Organizations involved in this watershed study include the BC Ministry of Forests, BC Ministry of Environment, Canadian Forest Service, Environment Canada, the Universities of Victoria, British Columbia and Simon Fraser, and MacMillan Bloedel Limited. Amongst other studies, a series of precipitation stations and a nest of hillside piezometers were installed at Carnation Creek between 1975 and 1982. Several of the precipitation stations and seventeen piezometers were monitored automatically and nearly continuously over a period of approximately eight years. The use of these data in this study is described further in Chapter 5.
Chapter 3. Test Site Characterization

Two landslides in Carnation Creek, referred to as 'Eugene' and 'Bob' (Plates 3.3 and 3.4), were visited and studied during the summer of 1994. Both landslides and all of the piezometer installations are located within a 0.5 km radius of precipitation station E (Fig. 3.6). Both landslides initiated from polygons at an elevation of approximately 200 m, and occurred during the January 23, 1982 rain-on-snow storm event.

3.2.2.1 Site Geology and Slope Morphology

The Carnation Creek watershed area was heavily glaciated during the Pleistocene period. Bedrock is mainly of volcanic origin and from the Bonanza group and the early Jurassic period (Fig. 3.3) (Scrivener, 1987). Bedrock outcrops in the area of the two landslides are observed to be highly fractured and weathered, with irregular joint spacings. Surficial slope materials are shallow, coarse textured and highly organic with a relatively thin root mat of 0.25 m to 0.50 m, whereas valley bottom deposits are derived from recent alluvium, underlain by gravel deposits, bedrock and some silty clay deposits.

The presence of a highly weathered and permeable bedrock layer which surfaces near the original failure headscarp of the 'Bob' landslide is thought to be a contributing factor to this failure. Due to the retrogressive nature of the 'Bob' landslide and continuous undercutting of the existing headscarp, this highly weathered zone 'daylights' below the existing scarp, leaving a relatively smooth and unweathered bedrock contact at the current headscarp location. A distinct seepage zone was observed between the surficial materials at the 'Bob' landslide site and the underlying, relatively unweathered bedrock at the current headscarp. The polygons from which the 'Eugene' and 'Bob' landslides originated have minimal downslope and cross-slope curvature, have slope angles of 34° (67%) and 35° (70%) and southerly and southeasterly slope aspects respectively. Depths to the planes of rupture are
approximately 1.1 m and 1.15 m respectively. Upon initiation, 'Eugene' flowed all the way to the creek whereas, due to the small initial volume, 'Bob' stopped short of the creek.

In contrast to the other three back-analysis sites, the Carnation Creek landslides are thought to have been affected to some extent by harvesting techniques and upslope road building; however, this site is of interest due to the failure mechanisms which are common to all four sites and the availability of a significant amount of piezometric and precipitation data. As will be shown in Chapter 6, back-analysis results for these two landslides are not significantly different to those from the landslides at Seymour Watershed (Jamieson Creek landslide), Holberg Inlet (Holberg Inlet landslide) and Sand River (Slide 32A).

### 3.2.2.2 Stability Assessment

Terrain mapping revealed the polygons from which the two landslides initiated to be stability class IV. Revegetation of the areas surrounding the landslides and natural stabilization of access roads during the thirteen years following these failures suggest the two areas are relatively stable. However, poor revegetation of the landslide scars and likely continued erosion and undercutting of the headscarsps suggest a potential for retrogressive failures and continued minor instabilities.

### 3.2.3 Holberg Inlet - Holberg Inlet Landslide

Holberg Inlet is an east-west oriented inlet off Quatsino Sound near the northwest coast of Vancouver Island. The inlet spans the distance between the towns of Holberg and Coal Harbour, and has a long history in the forest industry. The terrain to the north and south of Holberg Inlet and Quatsino Sound is known collectively as the Quatsino region and is part of TFL 6. The Holberg Inlet landslide site is shown in Fig. 3.7. It is located at an elevation of approximately 800 m and within the Coastal Western Hemlock biogeoclimatic zone (BC Ministry of Forests, 1992) (Plate 3.5).
Discussions with individuals familiar with this site and inspection of recent aerial photography and forest cover maps suggest this landslide occurred during the winter of 1989-1990.

3.2.3.1 Site Geology and Slope Morphology

Bedrock to the north of Holberg Inlet is mainly of volcanic origin and from the Bonanza group and the early Jurassic period (Fig. 3.3) (Scrivener, 1987). Bedrock outcrops within the landslide scar, and those exposed by road cuts are observed to be highly fractured and weathered with an irregular joint spacing and well-defined, but irregular, seepage zones. The general stratigraphy of the site is a discontinuous and highly variable stratum of very hard, unweathered morainal till, overlain continuously by highly weathered and rapidly-drained colluvial sands and gravels with frequent angular cobbles and boulders. In the discontinuous areas of very hard morainal till, there is a distinct, less-weathered contact with the underlying bedrock, whereas the colluvium/bedrock contact is less distinct and highly weathered. The original initiation zone and the path of the ensuing flow demonstrated a strong preference for these highly weathered colluvial areas. With exception to existing landslide scars, the surrounding cutblock has a good surface root mat of approximately 0.3 m thickness and developing regrowth. Slight groundwater seepage was observed from two locations on the headscarp at the sand and gravel colluvium and morainal till contact.

The headscarp and initiation zone are again well-suited to the infinite slope model, with the initiating length and width many times greater than the depth of the surficial materials. The average overburden thickness at the landslide site is approximately 1.2 m, but the near vertical headscarp is closer to 1.8 m high. The slope angle and aspect of the initial failure zone are $27^\circ$ (51%) and southeasterly respectively. There is no vertical or horizontal curvature and the failure is located on a wide, open slope approximately half-way between an access road and the upper cutting boundary. There is
limited evidence of a slight depression and change to natural drainage immediately upslope of the point of initiation due to yarding with a back spar.

3.2.3.2 Stability Assessment

Despite evidence of good revegetation and overall stability, there are several much smaller surficial slumping and sloughing failures over the width of this slope. The polygon is mapped as terrain stability class III according to existing guidelines, but a better understanding of local small scale drainage patterns and the extent and continuity of the morainal till stratum would be required to make an accurate estimate of the potential for, and location of, future instabilities.

3.2.4 Sand River - Slide 32A

Sand River, a part of TFL 44, is a tributary to the north side of Kennedy Lake, on the central west coast of Vancouver Island (Figs. 3.1 and 3.8). The landslide site is located on the west side of the Sand River valley and is commonly referred to as slide 32A (Plate 3.6). The site is at an elevation of approximately 380 m, and located within the Very Wet Maritime subzone of the Coastal Western Hemlock biogeoclimatic zone (BC Ministry of Forests, 1992).

The climate and precipitation are very similar to those reported for Carnation Creek, on the other side of Barkley Sound from Sand River, in section 3.2.2.

3.2.4.1 Site Geology and Slope Morphology

Bedrock outcrops at road cuts immediately south of slide 32A and local geology reported by Muller (1977) and Valentine et al. (1986) indicate plutonic intrusions of the Tertiary period are most common in the Sand River area. Exposed bedrock in the initiation zone of slide 32A is quite planar with few benches and undulations. Surficial materials around the head and side scarps of the slide are comprised predominantly of large cobbles and boulders in a sandy gravel matrix with very few fines.
As with Holberg Inlet, a discontinuous very hard, unweathered morainal till stratum exists immediately downslope of the initiation zone. Very few seepage points and zones are observed within the headscarp area, however, exposed colluvial and morainal materials over the entire headscarp appear to have a relatively high moisture content, as confirmed and described in Section 4.2.2.

The depth of surficial materials in the initiation zone is approximately 1 m and the initial slope angle and aspect are 30° (58%) and easterly respectively. The landslide is located immediately adjacent to the lateral cutting boundary of the cutblock. Windthrow immediately adjacent to the landslide and across the existing scar supports the suspicion that windthrow played a role in the initiation of the slide. Upon initiation, the sliding debris proceeded across two access roads, split into several distinct channels to pass through uncut timber and joined an existing gully approximately 50 m upslope of Sand River. Despite the presence of significant debris levees, and scouring within the lower reaches of the debris flow path upstream of its confluence with the gully, there is evidence of recurring torrents and debris from other landslides in the gully which makes it difficult to assess the extent to which slide 32A affected Sand River.

With exception to the potential role of windthrow in the initiation of this landslide, the geomorphological and rheological characteristics of slide 32A at Sand River are remarkably similar to those of the Jamieson Creek landslide in the Seymour Watershed.

3.2.4.2 Stability Assessment

The polygon from which slide 32A initiated is mapped as terrain class IV and is flanked to the south by class V terrain which has experienced natural failures and to the north by similar clearcut areas with very high landslide frequencies. In addition to these open slope instabilities, at least two gully systems on the same side of the Sand River valley are observed to be very active with frequent
torrents. The east side of Sand River has also experienced a significant number of open slope and road-related failures since harvesting and is one of eight study areas included in Rollerson’s (1984) geostatistical terrain stability study of TFL 44.
Chapter 3. Test Site Characterization

FIG. 3.1 Test site locations (Armstrong, 1990)

1. Seymour Watershed, Greater Vancouver Regional District
2. Carnation Creek, Vancouver Island (central coast)
3. Holberg Inlet, Vancouver Island (north)
4. Sand River, Vancouver Island (central coast)
Chapter 3: Test Site Characterization

FIG. 3.2 Soil landscapes and biogeoclimatic zones across the southern coast along latitude 49.5 degrees N (Valentine et al., 1986)

**FIG. 3.2** Soil landscapes and biogeoclimatic zones across the southern coast along latitude 49.5 degrees N (Valentine et al., 1986)

<table>
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<th>Soil Landscape</th>
<th>Ferro-Humic Podzol</th>
<th>Ferro-Humic Podzol</th>
<th>Humo-Ferric Podzol</th>
<th>Dystric Brunisol</th>
<th>Humo-Ferric Podzol</th>
<th>FHP</th>
<th>FHP</th>
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<th>Humo-Ferric Podzol</th>
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<td>CWH</td>
<td>Coastal Western Hemlock</td>
<td>Coastal Douglas-fir</td>
<td>CWH</td>
<td>Mountain Hemlock</td>
<td>MH</td>
<td>HFM</td>
<td>Coastal Western Hemlock</td>
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<td>Coastal</td>
</tr>
<tr>
<td>Forest Region</td>
<td>Coastal</td>
<td>Subalpine</td>
<td>Coastal</td>
<td>Subalpine</td>
<td>Coastal</td>
<td>Subalpine</td>
<td>Coastal</td>
<td>Subalpine</td>
<td>Coastal</td>
<td>Subalpine</td>
<td>Coastal</td>
<td>Subalpine</td>
<td>Coastal</td>
</tr>
<tr>
<td>Precipitation (mm/yr)</td>
<td>2500-5000</td>
<td>5000+</td>
<td>1250-2500</td>
<td>3000-5000</td>
<td>2500-5000</td>
<td>1250-2500</td>
<td>900-1200</td>
<td>5000-7000</td>
<td>900-1200</td>
<td>5000-7000</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**LEGEND**

1. Seymour Watershed, Greater Vancouver Regional District
2. Carnation Creek, Vancouver Island (central coast)
3. Sand River, Vancouver Island (central coast)
INDEX OF GEOLOGICAL MAPPING ON VANCOUVER ISLAND

LEGEND

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- TERTIARY INTRUSIONS
- TERTIARY VOLCANICS
- LATE MESOZOIC SEDIMENTS
- LEECH RIVER FORMATION
- ISLAND INTRUSIONS
- BONANZA GROUP
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- SICKER GROUP
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NOOTKA SOUND, 92 E (OPEN FILE 344)

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FIG. 3.4 Seymour Watershed, North Shore mountains, Lower Mainland (Acres International, 1993)
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FIG. 3.6 Carnation Creek back-analysis site location (Hetherington, 1994)
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FIG. 3.8 Sand River back-analysis site location (Rollerson, 1984)

4) Sand River, Vancouver Island (central coast)

FIG. 3.8 Sand River back-analysis site location (Rollerson, 1984)
Plate 3.6  Slide 32A, Sand River Valley
Chapter 4. Soil Classification and Shear Strength Testing

4. SOIL CLASSIFICATION AND SHEAR STRENGTH TESTING

4.1 General

With several spatially and temporally varying input parameters to the infinite slope model, such as root cohesion and piezometric response to precipitation events, it is important to obtain reliable information for the other parameters. The four landslide back-analysis sites are described in Chapter 3 in terms of site geology, morphology and stability. This chapter augments these descriptions by means of moisture content, grain size analyses and shear strength testing of the surficial deposits. The sampling, moisture content and grain size analysis techniques for soil classification are discussed, followed by the design, operation and results of a series of in-situ and laboratory reconstituted, large scale direct shear tests. The last section of this chapter summarizes the findings from Chapters 3 and 4 for all four back-analysis sites.

4.2 Soil Classification Testing

Soil classification relies on a series of index and physically relevant tests of a particular sample. The methods of classification employed in this work are:

* visual and textural identification in the field and laboratory
* moisture content testing at various locations around each headscarp
* mechanical sieving for grain size analysis greater than 100 μm
* x-ray diffraction for grain size analysis less than 100 μm

Care was taken while sampling from each head and sidescarp to ensure a representative sample; however, in all cases, the sample obtained was representative of the matrix material only. The frequency and angularity of cobbles and boulders within this matrix are thus only qualitatively described, as reported in Chapter 3, and the following descriptions are valid only for the matrix
Chapter 4. Soil Classification and Shear Strength Testing

material. The cobble and gravel content of the surficial materials observed at these sites is quite variable and estimated to be between 10% and 25% by volume.

4.2.1 Methodology

Moisture content samples were taken from the head and sidescarps of each site from materials representative of the sliding mass. Several samples were also obtained directly from the shear plane of the in-situ direct shear tests, as described in section 4.3.3. Samples were placed in air-tight and taped containers for transport back to the laboratory.

For the purpose of grain size analysis, several large grab samples from each site were combined and split to yield a sample size of approximately 1.5 kg, with particles greater than 25 mm. It was mechanically sieved using a gradation of sieves with openings between 25 mm and 105 μm. Material passing the 105 μm sieve was retained on a pan and used to continue the gradation analysis from 100 μm to 1 μm by means of x-ray diffraction. Samples for use in the MicroMeritics Sedigraph 5100 x-ray diffraction particle size analyzer were prepared by mixing a representative sample of approximately 1.5 g with 80 ml of water. For the first test, two samples from the same site were prepared using the same technique, with one sample containing deflocculant and the other without. The deflocculant was prepared by mixing 21 g of sodium hexametaphosphate, \text{(NaPO}_3)_6, with 1 l of water, and adding it to the sample in the proportions of: 10 ml deflocculant per litre of sample mixture. A comparison of the two curves (see Fig. 4.1) shows no significant effect of the deflocculant on the grain size distribution. All other gradation tests were consequently performed without it.

4.2.2 Results

Results of the combined grain size analyses are provided in Fig. 4.2. Although four distinct sites were visited during this field investigation, seven grain size curves are provided in Fig. 4.2, including three
from Carnation Creek (‘Bob’ landslide, ‘Eugene’ landslide and the road cut at which in-situ direct shear tests were performed; see section 4.3.5) and two from different locations around the headscarp of the Seymour Watershed Jamieson Creek landslide (sites A and B). Three horizontal scales are provided: the BC Terrain Classification System gradation scale, the grain size in millimetres and the grain size in phi units (Krumbein and Monk, 1942). The phi scale is determined by taking the base two logarithm of a particular grain size in millimetres, and is provided for the purpose of estimating hydraulic conductivity, as discussed below.

The sampled material is derived from in-situ weathering of bedrock and subsequent downslope movement as colluvium. These matrix materials are a gravelly sand to sandy gravel with little to some silt and a trace of clay. A description of the entire sliding mass would include this description and an estimate of the frequency of cobbles and boulders within the matrix. Inspection of the curves from all four sites shows them to be parallel and located within a relatively narrow band, with all samples having less than 2.5% clay fraction and all samples having a Unified Soil Classification System (USCS) descriptor of SW or GW. It would appear that despite the geographical separation between sites, the genesis of these materials and the mechanical processes by which these materials have been deposited and sorted are very similar.

It is well-established that the shear strength and behaviour of a cohesionless soil are controlled primarily by the amount and type of inter-particle contacts. Up to a certain frequency of cobbles and boulders within a sandy gravel to gravelly sand matrix, it is reasonable to assume that the behaviour is controlled primarily by the nature of the particle contacts and angularity within the matrix alone, and hence, the mechanical behaviour of the cobble/boulder/matrix mixture is that of the matrix alone. Plate 4.1, a photograph from the ‘Bob’ landslide headscarp, demonstrates the concept of larger cobbles and boulders dispersed in a less coarse, sandy gravel to gravelly sand matrix. As expected of
materials derived from in-situ weathering and colluvial processes, particles from all four sites are angular to sub-angular and irregularly shaped (Plate 4.2).

A summary of average moisture contents is provided in Table 4.1. Given the duration of field work (May to August, 1994), the variability from site to site and antecedent precipitation, a wide range of average moisture contents is exhibited between and within sites.

### Table 4.1 Average moisture contents

<table>
<thead>
<tr>
<th>Location</th>
<th>Average moisture content, w_n (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jamieson Creek landslide, site A, Seymour Watershed</td>
<td>51</td>
</tr>
<tr>
<td>Jamieson Creek landslide, site B, Seymour Watershed</td>
<td>15</td>
</tr>
<tr>
<td>'Bob' landslide, Carnation Ck.</td>
<td>56</td>
</tr>
<tr>
<td>'Eugene' landslide, Carnation Ck.</td>
<td>51</td>
</tr>
<tr>
<td>Road cut, Carnation Creek</td>
<td>43</td>
</tr>
<tr>
<td>Holberg Inlet landslide, Holberg Inlet</td>
<td>36</td>
</tr>
<tr>
<td>Slide 32A, Sand River</td>
<td>85</td>
</tr>
</tbody>
</table>

The hydraulic conductivity of the surficial materials from all four sites is estimated using Hazen's equation for uniform, loose sands (after Craig, 1987) and the method proposed by Krumbein and Monk (1942):

\[
k = D_{10}^2
\]  \hspace{1cm} (4.1)  

where ‘k’ is hydraulic conductivity (cm/s) and ‘D_{10}’ is the 10 % passing particle size (mm), and

\[
k = 0.734(D_{50})^{2} \left( \frac{D_{16}}{D_{84}} \right)^{0.945}
\]  \hspace{1cm} (Krumbein and Monk, 1942) (4.2)  

where ‘D_{50}’, ‘D_{84}’ and ‘D_{16}’ are the 50%, 84% and 16% passing particle sizes (mm) respectively. Table 4.2 provides estimates of matrix hydraulic conductivity using both methods and the coefficients of uniformity, $C_u$, and curvature, $C_c$, for material sampled from the four back-analysis sites.
### Chapter 4. Soil Classification and Shear Strength Testing

#### Table 4.2  Material uniformity, curvature and hydraulic conductivity

<table>
<thead>
<tr>
<th>Location</th>
<th>Coefficient of uniformity $C_u$</th>
<th>Coefficient of curvature $C_c$</th>
<th>Hydraulic conductivity (Hazen) (cm/s)</th>
<th>Hydraulic conductivity (Krumbein and Monk) (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jamieson Creek landslide, site A, Seymour Watershed</td>
<td>8.6</td>
<td>0.73</td>
<td>1.0E-02</td>
<td>1.2E-02</td>
</tr>
<tr>
<td>Jamieson Creek landslide, site B, Seymour Watershed</td>
<td>16.7</td>
<td>0.59</td>
<td>5.6E-03</td>
<td>7.7E-03</td>
</tr>
<tr>
<td>‘Bob’ landslide, Carnation Ck.</td>
<td>47.0</td>
<td>1.03</td>
<td>3.3E-03</td>
<td>3.3E-02</td>
</tr>
<tr>
<td>‘Eugene’ landslide, Carnation Ck.</td>
<td>16.7</td>
<td>1.04</td>
<td>9.0E-02</td>
<td>2.6E-01</td>
</tr>
<tr>
<td>Road cut, Carnation Creek</td>
<td>46.7</td>
<td>1.61</td>
<td>2.0E-03</td>
<td>3.2E-02</td>
</tr>
<tr>
<td>Holberg Inlet landslide, Holberg Inlet</td>
<td>16.4</td>
<td>0.94</td>
<td>2.0E-02</td>
<td>7.1E-02</td>
</tr>
<tr>
<td>Slide 32A, Sand River</td>
<td>11.1</td>
<td>0.87</td>
<td>1.4E-01</td>
<td>1.9E-01</td>
</tr>
</tbody>
</table>

The proximity of the two hydraulic conductivity estimates for each site is best when the coefficient of uniformity is smallest. In other words, while Hazen’s method provides a good estimate of hydraulic conductivity for poorly graded (clean) materials, the method proposed by Krumbein and Monk is better suited to materials with a wider range of grading. According to the latter method, the range of matrix hydraulic conductivities is 0.0077 cm/s to 0.26 cm/s. This range of values is in good agreement with the compiled and reported range of hydraulic conductivities by Freeze and Cherry (1979) for clean sands to gravels.

As seen in Plate 4.1, a typical stratigraphy at the four back-analysis sites is one of frequent cobbles and boulders dispersed in a matrix of sandy gravel to gravelly sand. Another observation, common to most other coastal clearcut sites, is the presence of small and large decayed root holes which act as macro-pore features within the surficial materials. It has been argued that this macro-pore structure may either increase or decrease the drainage of a slope, and hence improve or diminish slope stability. Amidst these differing beliefs, however, is the common recognition that these decayed root holes or macro-pore features, rather than the conductivity of the host matrix, control the apparent hydraulic conductivity and hence the piezometric response of shallow surficial materials on a slope. With this in
mind, it would be useful to assess the drainage and piezometric conditions of a site based on density and connectivity of macro-pore features and slope morphology rather than on matrix hydraulic conductivity and pedological descriptions alone. Determining the density and connectivity of macro-pore features within a well-defined stratigraphy or in the area of an existing headscarp is very difficult. It is practically impossible over a larger area.

4.3 Direct Shear Strength Testing

A large scale, computer controlled, air-actuated and portable direct shear box was designed and built to determine the in-situ, undisturbed strength of the soil matrix. With respect to fundamental soil mechanics and the elegance of simple shear, and cyclic and monotonic triaxial testing, direct shear testing seems primitive and in some ways empirical. As stated in section 1.3 of ASTM Designation: D 5321 - 92: “The test method is not suited for the development of exact stress-strain relationships within the test specimen due to the non-uniform distribution of shearing forces and displacement”. The non-uniform stress distribution and mechanism of progressive failure severely limit the interpretation of results from direct shear testing of any kind, whether they be carefully controlled reconstituted laboratory tests or large scale, in-situ tests. Stress rotation and progressive development of the shear plane during shearing are typically considered to represent significant limitations of the direct shear test. Moreover, the stress condition at failure and on the plane of rupture may be assumed to be the condition of maximum shear stress or, alternatively, the condition of maximum principal stress ratio. Despite these apparent limitations, the direct shear test remains to be the most appropriate undisturbed, in-situ shear strength test method for these materials.

Despite these drawbacks, the attractions of large scale, in-situ direct shear testing include the indisputable similarities of failure mechanism between landsliding and direct shear testing, the lack of
disturbance to in-situ structure, the accommodation of large particle sizes and the relatively simple control and interpretation.

### 4.3.1 Principles and Standards of the Direct Shear Test

ASTM designations D 5321 - 92 and D 3080 - 90 address the determination of shear strength and coefficient of friction of soils under consolidated drained conditions and in contact with geosynthetics. These two designations provide guidance concerning shear box and sample dimensions, rate of shear, interpretation of results and boundary conditions as follows:

- effort should be made to identify the sheared area and failure mechanism of the specimen
- minimum shear box container dimensions should be the greater of 300 mm, or 15 times the ‘D₈₅’ dimension of the coarsest soil
- minimum container height should be the greater of 50 mm or 6 times the maximum particle size of the coarsest soil
- device capable of maintaining normal force to within ±1% of the specified force
- box containing specimen sufficiently rigid to prevent distortion during shearing
- capable of shearing specimen at uniform rate of displacement with less than ±5% deviation
- weight of top shear box constitutes less than 1% of applied normal force
- rate of displacement controlled so that no excess pore pressure exists at failure
- adjust shear strength estimates appropriately to account for the shear resistance of the empty shear box.

Interpretation of the direct shear test results involves the correction of measured shear forces for the shear resistance of the empty shear box, and division by the applied normal force. In this manner the need to correct shear and normal stresses for a continuously changing contact area is precluded and the
only assumption to be made in interpretation is that the ratio of rotated principal stresses on the plane of rupture is similar to the ratio of boundary stresses at failure.

In consideration of ASTM Designations D 3080 - 90 and D 5321 - 92, care was taken to control boundary stresses and rate of displacement and to account for secondary effects, such as pore pressure development, stress rotation and box rotation by maintaining an adequate height to width ratio for the shear box and by limiting (and verifying) strain rates during shear. It has been stated that one of the benefits of in-situ testing is that the failure mechanism, structure and stress conditions are preserved during the test. Hence, an effort was made in both in-situ and laboratory testing to determine the material shear strength within the range of normal stresses typical of field conditions. With only 0.5 m to approximately 2 m or 3 m of surficial material above most failure planes, an appropriate normal effective stress range (assuming near-saturated conditions) is approximately 5 kPa to 25 kPa. This stress range is far below the range of typical geotechnical strength testing and the range for which typical material strengths are reported.

Pore pressure development and dissipation due to strain rate effects, degree of saturation, grain size and stress range are all important factors which determine the drained shear strength of these materials. If the method proposed by Bishop and Henkel (1957) for determining time to failure, and hence strain rate, in a saturated triaxial test is extended to direct shear testing, a very conservative estimate of the required time to failure can be made:

\[
t_f = \frac{20h^2}{\eta c_v}
\]  

(4.3)

where 't_f' is the time to failure, 'h' is the drainage path length (approximately 0.12 m for this device), '\(\eta\)' represents the drainage conditions during the test (\(\eta = 3.0\) for drainage to both sides)(Bishop and
Henkel, 1957) and \( c_v \) is the coefficient of consolidation; \( c_v = kM/\gamma_{\text{water}} \). Using very conservative estimates of hydraulic conductivity, \( k \), and constrained modulus, \( M \), of \( 2.0 \times 10^{-5} \) m/s and 0.1 bar respectively, an estimate of time to failure, \( t_f \), of 78 min. is obtained. ASTM Designation D 3080 - 90 suggests a time of failure for this sandy material of 60 min. With shear displacement to failure (as discussed below) being approximately 40 mm and a strain rate of 0.5 mm/minute being adopted for all tests, both of these strain rate criteria are satisfied.

Having undergone large strains and continuous shearing, it is likely that these colluvial materials are at or near the critical void ratio, and as such, would experience little contraction (leading to the development of excess positive pore pressures during partially drained shear) or dilation (leading to excess negative pore pressures during partially drained shear). Considering the relatively high hydraulic conductivity of these materials, it is believed that pore pressure development during shear exerts a negligible effect on measured shear strengths.

### 4.3.2 Field Direct Shear Box Design and Data Acquisition

The direct shear device was designed to be portable, self-reacting and, to some extent, automatic. It comprises a large square two-piece box, reaction frame and servo controlled actuator (see Fig. 4.3). A unique feature of the design is the use of an air-actuated cylinder; the advantages of using air are reduced weight, resulting in increased portability, and the use of air instead of environmentally hazardous hydraulic fluid in a natural environment. The principle of operation involved assembly of the shear box around an undisturbed column of soil and shearing at a controlled rate of displacement.

#### 4.3.2.1 Physical Characteristics

Physical dimensions and the arrangement of transducers, valves, reaction arms and the loading cylinder are shown in Fig. 4.3. The sample is approximately 300 mm x 300 mm in plan, and 250 mm
Chapter 4. Soil Classification and Shear Strength Testing

high. The device is built from 6 mm and 13 mm aluminum plate and square bar. For portability, the loading cylinder, all of the transducers, the reaction arms and both halves of the shear box are easily detached and rebuilt around the undisturbed column of soil to be tested in the field. Compressed air is supplied to the loading cylinder by means of an electric air compressor, which, along with a notebook computer and the data acquisition unit, is powered by a gasoline powered generator. The force exerted by the loading ram on the top half of the shear box is the result of a differential air pressure between the front and back of the loading cylinder. The shear box, data acquisition unit and computer are illustrated in Plates 4.3 and 4.4.

4.3.2.2 Control, Feedback and Data Acquisition Systems

The method of operation and control of the direct shear box involve the near-continuous determination and provision of a ‘demand’ and hence ‘error’ signal (which represents the difference between the desired and the actual displacement of the top half of the shear box) to the servo controller, which in turn pressurizes the appropriate end of the loading ram in order to maintain the desired rate of displacement.

More specifically, control of the shear box device is provided by a signal conditioner, ramp generator and servo controller which are incorporated into one data acquisition unit (Fig. 4.4). The data acquisition unit accepts analog input from seven channels, including shear position feedback and produces digital output to the motion controller and the computer. Eight channels of digital output to the computer are stored every 2 seconds and displayed continuously on-screen as three different plots of loading ram pressures, shear resistance and vertical displacement (indicative of shear box rotation) versus shear displacement. Digital output from the ramp generator, within the data acquisition unit, is sent directly to and converted to analog by the motion controller. This analog position command is received in the form of a current by the voice coil servo valve, which then supplies compressed air to
the appropriate end of the loading cylinder. The magnitude and sign of the current ('error' signal) which is received by the servo valve are determined by means of a feedback loop which compares the actual to the expected cylinder positions.

In an attempt to minimize the time required to correct any difference between the actual and the expected cylinder positions and to dampen the feedback system, the operating air pressure of the entire system is raised by placing a back pressure of 150 kPa on the servo valve exhaust (Fig. 4.5).

4.3.3 In-situ Test Method

Each in-situ direct shear test was performed according to a consistent method which was followed with very little variance between sites:

- one to three accessible and representative locations around or within the landslide failure scar are located;
- stratigraphy, gradation and frequency of cobbles and boulders as well as evidence of a unique shear zone or plane of weakness are observed at each proposed test location;
- once one or more of the selected locations is deemed satisfactory for testing, a free standing, undisturbed block, with near vertical sides and relatively few exposed cobbles and boulders is excavated, with approximate dimensions: 0.25 m x 0.25 m x 0.25 m;
- perimeter of the block at the location of the shear plane within the direct shear box is measured to the nearest centimetre;
- top and bottom halves of the shear box are separated from the rest of the device and an open ended plastic bag is attached to each half with duct tape, as shown in Fig. 4.6;
- WD40 compound is sprayed around the interior four walls of both shear box halves in order to facilitate cleaning after each test;
• bottom half of the shear box is lowered over the undisturbed soil block so that an approximately 2.5 cm gap exists between the shear box and soil around the entire perimeter;

• shear box is leveled and stabilized using four adjustable support legs;

• gap between plastic sheet and wall at rear of bottom shear box is filled with a 1:2:1, water:grout:plaster, mixture, to a finished surface that is level and below the top edge of the lower half of the shear box;

• remaining three gaps (around sides and front of bottom half of shear box) are filled with coarse sand (Plate 4.5);

• four acetate strips, approximately 1 mm thick by 5 cm wide are placed around all four sides of bottom half of shear box, leaving sufficient material to allow removal prior to testing;

• top half of shear box is lowered over soil column so that edges of top and bottom halves are aligned;

• gap between plastic sheet and wall at front of top shear box is filled with the same 1:2:1 mixture;

• lower half of gap between plastic sheet and wall at rear of top shear box is filled with the same 1:2:1 mixture (to prevent loss of sand and sample during shear) (Fig. 4.6);

• remaining gaps are filled with coarse sand;

• top of soil block is leveled off with coarse sand and 20 cm square loading plate is centred over soil column;

• acetate strips are removed once grout and plaster mixture has hardened (approximately 10 min.);

• reaction arms and loading cylinder are attached to bottom shear box;

• specified normal load (steel plates) is placed on loading plate (Plate 4.6);
displacement, pressure and temperature transducers, load cell and position command input are attached to shear box and data acquisition unit, which is connected to notebook computer and power supply;

level and orientation of shear box are checked and compressed air supply is connected to servo valve to allow back pressure to build;

computer program (Appendix B) is started to allow storage every 2 seconds and real time graphing of eight channels in three on-screen plots;

shear displacement at a constant rate of 0.5 mm/min. started, and test is terminated at approximately 60 mm displacement (25% strain);

shear box is disassembled and shear plane is inspected, sampled and photographed.

Plates 4.7 and 4.8 show two typical shear planes, and the directions of shear, which were photographed and sampled following shear testing. In several instances a test would have to be discarded due to the presence of a cobble within the shear plane, as in test ‘A0806’ (Plate 4.8 and Table 4.3).

4.3.4 Laboratory Reconstituted Test Method

A series of laboratory tests was also performed on oven-dried, reconstituted samples taken from each of the four locations, using the same large direct shear box and test method. The objectives of these tests were to confirm or support results from in-situ direct shear tests using sampled soils in a controlled environment, to assess the influence of soil structure and any root mat on undisturbed shear strengths, and to confirm the negligible role of shear induced pore pressure development during the in-situ tests described above.

In order to validate in-situ test results and to discount the potential effect of soil matrix suction, or negative pore pressures, on test results, a series of large scale direct shear tests were performed in the
laboratory on reconstituted samples obtained from the initiation zone of the studied landslides. Representative grab samples were obtained and sealed in air tight bags for transport to the laboratory where they were oven dried on baking trays at 110 °F (49 °C). The sample masses were recorded before and after drying in order to obtain further moisture content data, as reported in Table 4.1. The dried samples were then scalped to obtain samples with a maximum particle size of 25 mm and placed into air-tight containers for later testing.

The same shear box, instrumentation and data acquisition system were used in the laboratory as were used for the in-situ testing program, with the exception that two 25 cm square rigid wood boxes (top and bottom halves) were placed inside the original 30 cm square aluminum boxes in order to contain the dry sample and test a sample with similar dimensions to those in the field. Teflon tape and acetate strips were used in the same manner as in the field in order to reduce friction and to provide a gap between the containing boxes. The dry laboratory samples were prepared by means of thorough mixing, followed by placement by hand with zero drop height in order to avoid segregation. The 24 cm high samples were placed in 3 tamped lifts, forming samples of moderate density with level surfaces for normal (vertical) load application. Sample density was not carefully controlled, due first, to the highly variable nature of in-situ density within and between sites, and second, to the interest in large displacement resistance only, which is independent of initial density. Reconstituted tests were also performed on a medium coarse sand for purposes of benchmark comparison.

4.3.5 Direct Shear Strength Test Results

The testing conditions and interpreted large displacement stress state of each in-situ and laboratory direct shear test are summarized, with comments, in Table 4.3.
### Table 4.3 Summary of in-situ and reconstituted laboratory direct shear testing

<table>
<thead>
<tr>
<th>Test Identification Code and Location</th>
<th>Normal Stress (kPa)</th>
<th>Shear Stress at Large Displacement (kPa)</th>
<th>Displacement Rate (mm/min.)</th>
<th>Comments and Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Seymour Watershed - In-situ</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A10628</td>
<td>5.5</td>
<td>6.4</td>
<td>1.0</td>
<td>- vertical displacement measurement device not installed</td>
</tr>
<tr>
<td>A20629</td>
<td>8.2</td>
<td>-</td>
<td>1.0, 0.5</td>
<td>- test aborted</td>
</tr>
<tr>
<td>A30705/6</td>
<td>8.3</td>
<td>10.4</td>
<td>1.0, 0.5, 1.0</td>
<td>- load, unloaded then reloaded</td>
</tr>
<tr>
<td>A40706</td>
<td>10.9</td>
<td>12.0</td>
<td>0.5, 1.0, 0.5</td>
<td>- displacement rate decreased then increased</td>
</tr>
<tr>
<td>A50707</td>
<td>13.3</td>
<td>-</td>
<td>0.5</td>
<td>- located immediately below A10628 (same column)</td>
</tr>
<tr>
<td>B10708</td>
<td>5.9</td>
<td>-</td>
<td>0.25, 0.5, 0.25, 0.5</td>
<td>- data unreliable due to large cobbles and rootlets</td>
</tr>
<tr>
<td>B20711</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>- test aborted</td>
</tr>
<tr>
<td>B20712</td>
<td>8.6</td>
<td>8.6</td>
<td>0.5</td>
<td>- abandoned (cobbles)</td>
</tr>
<tr>
<td>B30713A</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>- abandoned (cobbles)</td>
</tr>
<tr>
<td>B30713B</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>- abandoned (cobbles)</td>
</tr>
<tr>
<td>B30713C</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>- abandoned (cobbles)</td>
</tr>
<tr>
<td>B30713</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B40713</td>
<td>17.9</td>
<td>17.9</td>
<td>0.5</td>
<td>- multistage test</td>
</tr>
<tr>
<td><strong>Carnation Creek - In-situ</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A10805</td>
<td>5.5</td>
<td>7.4</td>
<td>0.5</td>
<td>- abandoned (cobbles)</td>
</tr>
<tr>
<td>A0806</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>- abandoned (cobbles)</td>
</tr>
<tr>
<td>A20806</td>
<td>8.0</td>
<td>11.1</td>
<td>0.5</td>
<td>- broken regulator after 25 mm displacement</td>
</tr>
<tr>
<td>A30808A</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>- broken regulator after 25 mm displacement</td>
</tr>
<tr>
<td>A30808</td>
<td>13.1</td>
<td>-</td>
<td>0.5</td>
<td>- test aborted</td>
</tr>
<tr>
<td>A40809</td>
<td>10.5</td>
<td>14.7</td>
<td>0.5</td>
<td>- vertical displacement transducers inoperational</td>
</tr>
<tr>
<td>A50810</td>
<td>13.1</td>
<td>18.9</td>
<td>0.5</td>
<td>- vertical displacement transducers inoperational</td>
</tr>
<tr>
<td><strong>Holberg Inlet - In-situ</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A10816</td>
<td>5.7</td>
<td>9.2</td>
<td>0.5</td>
<td>- air compressor failed</td>
</tr>
<tr>
<td>A20816</td>
<td>7.9</td>
<td>10.6</td>
<td>0.5</td>
<td>- valve failed after ~1 mm displacement</td>
</tr>
<tr>
<td>A30817</td>
<td>10.6</td>
<td>13.2</td>
<td>0.5, 0.25, 0.5</td>
<td>- visible tensioning then failure of root, causing unload then reload</td>
</tr>
</tbody>
</table>

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Chapter 4. Soil Classification and Shear Strength Testing

<table>
<thead>
<tr>
<th>Test Identification Code and Location</th>
<th>Normal Stress (kPa)</th>
<th>Shear Stress at Large Displacement (kPa)</th>
<th>Displacement Rate (mm/min.)</th>
<th>Comments and Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>A40818</td>
<td>13.1</td>
<td>17.0</td>
<td>0.5</td>
<td>- cobbles and rootlets observed</td>
</tr>
<tr>
<td>A50819</td>
<td>5.7 10.8</td>
<td>inconclusive 15.4</td>
<td>0.5</td>
<td>- multistage test - inconclusive results</td>
</tr>
<tr>
<td>A60819</td>
<td>8.2 13.2</td>
<td>12.2 15.4</td>
<td>0.5</td>
<td>- multistage test</td>
</tr>
<tr>
<td>Sand River - Laboratory</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>3.9 5.0</td>
<td></td>
<td>0.5</td>
<td>- reconstituted, dry - test halted after ~ 40 mm displacement due to loss of material from shear box</td>
</tr>
<tr>
<td>2</td>
<td>6.4 8.0</td>
<td></td>
<td>0.5</td>
<td>- reconstituted, dry - test halted after ~ 40 mm displacement due to loss of material from shear box</td>
</tr>
<tr>
<td>3</td>
<td>8.9 10.7</td>
<td></td>
<td>0.5</td>
<td>- reconstituted, dry - test halted after ~ 40 mm displacement due to loss of material from shear box</td>
</tr>
<tr>
<td>4</td>
<td>11.4 13.1</td>
<td></td>
<td>0.5</td>
<td>- reconstituted, dry - vertical displacement transducers disturbed after ~ 14 mm displacement - sheared to ~ 53 mm displacement</td>
</tr>
<tr>
<td>Seymour Watershed - Laboratory</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>11.6 12.8</td>
<td></td>
<td>0.5</td>
<td>- reconstituted, dry</td>
</tr>
<tr>
<td>3</td>
<td>6.7 7.2</td>
<td></td>
<td>0.5</td>
<td>- reconstituted, dry</td>
</tr>
<tr>
<td>Holberg Inlet - Laboratory</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>6.7 7.5</td>
<td></td>
<td>0.5</td>
<td>- reconstituted, dry</td>
</tr>
<tr>
<td>2</td>
<td>11.6 13.1</td>
<td></td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Coarse Sand - Laboratory</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>15.7 10.0</td>
<td></td>
<td>0.5</td>
<td>- dry</td>
</tr>
</tbody>
</table>

Of approximately 35 attempted in-situ and reconstituted laboratory direct shear tests, approximately 30% were discarded or abandoned due to the presence of excessively large particles within the shear zone, failed equipment, human error or inconclusive data.

Direct shear test results are presented in the form of normalized shear stress ratio ($\tau/\sigma_v'$) and average vertical displacement versus shear displacement ($\delta$). The results of both in-situ and laboratory tests are presented in Figs. 4.7 to 4.10, with one figure for each site. While both in-situ and reconstituted laboratory test results are available for the Jamieson Creek landslide and Holberg Inlet slide sites...
Chapter 4. Soil Classification and Shear Strength Testing

(Figs. 4.7 and 4.9), results from testing at ‘Bob’ and ‘Eugene’ landslides at Carnation Creek and Sand River slide 32A are less complete. At Carnation Creek the lack of reasonable accessibility and the over abundance of large cobbles and boulders in the headscarp stratigraphy precluded in-situ testing within the slide initiation zones. As an alternative, a road cut within 0.5 km of ‘Eugene’ landslide and 1 km of ‘Bob’ landslide was located and used for a series of in-situ tests (Fig. 4.8). As well as being more accessible, the material and stratigraphy of this cut was observed to have a very similar matrix gradation (as seen in Fig. 4.2), yet a lower frequency of cobbles and boulders in comparison to the ‘Bob’ and ‘Eugene’ landslide sites. At Sand River, in-situ testing was, again, hindered by the excessive frequency of cobbles and boulders within the initiation zone stratigraphy. Following unsuccessful efforts to locate a nearby road cut or area with a similar matrix material yet fewer cobbles and boulders, large samples were obtained from the initiation zone of slide 32A for a series of reconstituted laboratory tests (Fig. 4.10). For comparison purposes, results of a direct shear test performed on a medium coarse, dry, sub-rounded, narrowly graded sand of known behaviour, using the same apparatus and sample preparation technique are also presented in Figs. 4.7 through 4.10.

Regarding the in-situ direct shear tests, most stress ratio versus displacement curves reveal an early rapid mobilization of shear resistance at displacements less than 5 mm. Thereafter some curves contain a single peak in the region of 5 mm to 12 mm shear displacement, while others show no discernible peak stress ratio. While the existence of a single peak is common amongst the in-situ test results and indicative of a dense material, the magnitude of these peaks is questioned due to the strong possibility that tiny roots are being tensioned and pulled in this region of displacement. This phenomenon is best demonstrated by the occurrence of two distinct peaks in one of the in-situ curves of Fig. 4.7 (Jamieson Creek landslide). Regarding the reconstituted laboratory tests, the observed peak stress ratio in these tests is likely related to density alone, with no contribution from roots and rootlets.
If a material is cohesionless, its mechanical behaviour may be described based primarily on its density, the resulting presence or lack of a peak stress ratio and the existence of a unique stress ratio at a critical or constant volume at large displacements. If cohesionless, a unique stress ratio should be observed at large displacement, regardless of initial density and normal effective stress, $\sigma_n$. This large displacement stress ratio is equal to the tangent of the constant volume friction angle, $\phi_{cv}$, assuming zero cohesion. A horizontal line may be drawn on each of the stress ratio versus shear displacement figures at a level consistent with the average stress ratio at large displacement for all of the curves, so as to intersect the non-linear friction angle axis on the right. In other words, a stress ratio value which is representative of the shear behaviour over a stable portion of the final 30 to 60 mm of shear displacement has been determined for each stress ratio curve. For example, such a line could be drawn at the convergence of all of the stress ratio curves of Fig. 4.7 in the range of 40 to 60 mm shear displacement, indicating a constant volume; or large displacement, friction angle, $\phi_{cv}$, of 45° to 50°.

Inferences concerning the peak friction angle of these materials cannot be drawn in the same manner due to the spatial variability of material density and the potential contribution of root strength at these low to intermediate shear displacements. During several in-situ tests (e.g. A30705/6, A40706), the rate of displacement was increased and decreased by a factor of two for a duration corresponding to approximately 5 mm of displacement in order to verify the independence of rate and resistance under drained conditions. In a similar manner, several test specimens were unloaded and reloaded during displacement (see Table 4.3).

Average vertical displacement is obtained by averaging the readings of the two vertical displacement transducers (see Fig. 4.3 and Plate 4.6) to obtain a relative measure of shear induced volume change between different materials. Although true shear induced volume change measurements would be
useful for a discussion of critical state mechanics and behaviour, unlike the triaxial test, the direct shear test is ill-suited to these precise observations. Because these displacement measurements are non-uniform over the height and area of the sample and taken at the boundaries of the sample, all that can be deduced from two different average vertical displacement versus shear displacement curves is that one material is more dilatant or contractive than another. The extent to which any one material is dilatant or contractive (critical state parameter, $\Psi$) remains undetermined from this series of tests. Despite these inherent interpretation problems, a general average vertical displacement behaviour consists of an initial reduction in height, followed by a gradual increase in height (loosely referred to as slight contraction, followed by gradual dilation). Under ideal testing conditions, it would be expected that the convergence of stress ratio curves in the range of 40 to 60 mm shear displacement coincides with approximately horizontal average vertical displacement curves, signifying constant volume and the achievement of a critical void ratio. Also, the point of inflection of the average vertical displacement curves (immediately following an initial reduction in sample height, or contraction) indicates a state of maximum dilatancy, and should in turn coincide with the maximum or peak stress ratio. Considering the limitations of boundary displacement measurements and the existence of large, irregular particle sizes, the observed behaviour of Figs. 4.7 to 4.10 are in relatively good agreement with accepted material behaviour theory. In general there is a consistent range of shear stress to normal stress ratios at large displacements observed from Figs. 4.7 to 4.10. The best example of agreement is the observed behaviour of reconstituted dry material from Sand River slide 32A in Fig. 4.10.

4.4 Synthesis of Results

A plot of large displacement shear stress versus normal effective stress is presented in Fig. 4.11. The three solid lines represent Mohr-Coulomb failure envelopes, being fits to all of the in-situ test data only, to all of the laboratory test data only, and to all of the test data. The slope of each line at any
given normal effective stress is a direct measure of the constant volume friction angle, $\phi_{cv}$, of the materials. The large displacement stress state of the medium coarse, sub-rounded sand is also plotted for comparison as a single point in Fig. 4.11.

The heaviest line represents a power-fit to all of the data with a slightly curved surface and zero cohesion. The equation of this line is:

$$\tau = 1.503 \cdot \sigma_n^{0.913}$$  \hspace{1cm} (4.4)

Although atypical for interpretation of shear strength data, this equation is used in order to effectively describe the curvature of the failure surface at very low normal stresses. Equation 4.4 can be differentiated to give the slope (friction angle) at any normal effective stress, $\sigma_n$. For example, at normal stresses of 4.5 kPa and 25 kPa, the corresponding friction angles are 50° and 46° respectively. The two lighter solid lines represent linear fits to only the in-situ data and only the laboratory data, both possessing a slope of 46° and intercepts of 2.5 kPa and 1.0 kPa respectively. Given the assumption that the reconstituted, oven-dried coarse soils are cohesionless, the 1 kPa intercept is attributed to the linear representation of a curved failure envelope. The difference of 1.5 kPa between the two linear representations is attributed to the presence of rootlets in the undisturbed structure of the in-situ samples, and is deemed a combined root/structure cohesion.

Several premises and some supporting evidence have been put forth regarding the behaviour of these materials. Amongst the most influential of these premises are the statements that, up to a limiting cobble and boulder content, it is the sandy gravel to gravelly sand and fines matrix which controls strength behaviour; it is the macro-pore features, material variability and micro-topography which control hydraulic conductivity and piezometric response of the surficial soils; it is the cobble and
boulder content which controls the rheological behaviour of surficial materials, once mobilized; and finally, matric suction in partially saturated sandy gravels and gravelly sands has a negligible effect on strength behaviour.

Assuming a typical range of overburden stresses between 10 kPa and 25 kPa, and using the derivative of the power curve represented by Equation 4.4, an average friction angle of approximately $47^\circ$ is estimated for the tested surficial materials. Although apparently very high in comparison to friction angles of more typical geotechnical materials at higher stresses, these values are in agreement with findings for similar materials and stress levels, as discussed below.

With the primary transport and deposition mechanism of the tested materials being a colluvial process, it is reasonable to assume that these weathered bedrock derived materials have undergone large strains and shearing. Skermer and Hillis (1970) discuss the concept of an ideal gradation required for optimum compaction (Fig. 4.2) (after Fuller and Thompson, 1907). Drained triaxial tests performed by Skermer and Hillis (1970) have demonstrated several interesting results:

- a poorly-graded fine to coarse gravel sheared under drained conditions sustained particle crushing and degradation which caused the grain size curve after shearing to be similar to Fuller's optimum compaction curve. In other words, mechanical reworking and shearing of uniform, coarse materials often results in more broadly-graded samples with gradations similar in shape to Fuller's optimum compaction curve;
- very high peak angles of friction ($45^\circ$ to $50^\circ$) at moderate confining pressures;
- high dilatancy modified (Rowe, 1962) angles of friction ($43^\circ$) at moderate confining pressures; and
- superior stress-strain behaviour of near-optimum gradations, with less volume change.
Chapter 4. Soil Classification and Shear Strength Testing

Using gradations 1 and 2 of Fig. 4.12 (Skermer and Hillis, 1970), which are similar to the soils in this study, reasonable extrapolation of these results to confining stresses of less than 25 kPa would result in friction angles in the range of 45° to 47°. The solid lines of Fig. 4.12 reflect Skermer and Hillis' test data, while the dashed lines reflect the same data corrected for dilatancy as described by Rowe (1962). Referring, again, to Fig. 4.2, grain size curves for sampled material are observed to have a similar shape and gradation to Fuller's optimum compaction curve with maximum particle size of 10 mm.

Results of shear strength testing of rockfill by Leps (1970) and other in-situ and laboratory shear strength testing of steepland granular soils by Wu, McKinnell and Swanston (1979), Chandler, Parker and Selby (1981) and Sidle and Swanston (1982), amongst others, suggest friction angles in the range of 40° to more than 50°, and the variation of friction angle with normal stress (see Fig. 4.13). The strength criteria for coarse, durable material, used by the U.S. Forest Service is also shown as a stepped function of normal stress on Fig. 4.13, with recommended friction angles between approximately 43° and 47°.
FIG. 4.1 Comparison of x-ray diffraction results: with and without deflocculant
FIG. 4.2 Grain size distribution curves
Plate 4.1 Headscarp stratigraphy at ‘Bob’ landslide, Carnation Creek (note matchbox for scale)
Plate 4.2  Typical grain angularity ('Bob' landslide, Carnation Creek Experimental Watershed)
FIG. 4.3 Large scale direct shear box

Vertical displacement transducers (2)

Normal load

Low-friction plastic

Undisturbed soil column

Shear plane

1 mm gap and teflon tape

Load cell

Feedback displacement transducer

Voice coil servo valve

Air filter

Compressed air supply

Air actuated 12.7 cm bore X 15.25 cm stroke loading cylinder

Pressure transducers (2)

Temperature transducer

Reaction frame (two sides)

Plaster / cement / water mixture (load bearing)

Coarse sand

SCALE approximately 1:5.5
transducers, sand, plaster/cement mixture, teflon and plastic excepted
Plate 4.3 Large scale direct shear box and data acquisition system
Plate 4.4 Direct shear apparatus immediately prior to shearing
Chapter 4. Soil Classification and Shear Strength Testing

real time shear stress vs. displacement plot

Shear load

load cell temperature

air pressure (front)

air pressure (rear)

vertical displacement (front)

vertical displacement (rear)

notebook field computer

data acquisition unit
signal conditioner
ramp generator
servo controller

position feedback

position command

position feedback

analog motion controller

current feedback

current command

pressure transducer

air actuated cylinder

pressure transducer

voice coil air servo valve

Supply (compressed air)

exhaust A

exhaust B

digital or analog signal

actual position

desired position

FIG. 4.4 Data acquisition and feedback control flowchart
FIG. 4.5 Compressed air loading cylinder supply and exhaust schematic
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FIG. 4.6 Direct shear box cross section: In-situ test method

- Undisturbed soil
- Coarse sand
- Aluminum shear box (13mm plate)
- Teflon (12 mm wide by 0.5 mm thick)
- Acetate strips (1.0 mm thick, all four sides, removed prior to test)
- Plaster/cement/water mixture (load bearing)

SCALE approximately 1:3 except teflon, acetate and separations
Plate 4.5  Bottom half of shear box surrounding undisturbed soil block

Plate 4.6  Normally loaded and instrumented shear box prior to shearing
Plate 4.7  Typical acceptable shear zone
Plate 4.8 Typical unacceptable shear zone due to presence of cobble
FIG. 4.7 Stress ratio and average vertical displacement versus shear displacement, Jamieson Creek landslide, Seymour Watershed
FIG. 4.8 Stress ratio and average vertical displacement versus shear displacement, road cut near ‘Bob’ and ‘Eugene’ landslides, Carnation Creek Experimental Watershed
**FIG. 4.9** Stress ratio and average vertical displacement versus shear displacement, Holberg Inlet slide, Coal Harbour
FIG. 4.10 Stress ratio and average vertical displacement versus shear displacement, Sand River slide 32A, Kennedy Lake
FIG. 4.11 Interpreted shear strength, all back-analysis sites and medium coarse sub-rounded uniform sand
### Gradation and compaction properties

<table>
<thead>
<tr>
<th>Gradation</th>
<th>Gradation 1</th>
<th>Gradation 2</th>
<th>Gradation 3</th>
<th>Gradation 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum particle size, in. (mm)</td>
<td>1-1/2 (38.10)</td>
<td>1-1/2 (38.10)</td>
<td>1-1/2 (38.10)</td>
<td>3/16 (4.76) No. 4 sieve</td>
</tr>
<tr>
<td>Gravel content, %</td>
<td>37</td>
<td>65</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>Effective grain size, $D_{10}$, mm</td>
<td>0.18</td>
<td>0.53</td>
<td>5.83</td>
<td>0.12</td>
</tr>
<tr>
<td>Uniformity coefficient $C_u = \frac{D_{60}}{D_{10}}$</td>
<td>20.6</td>
<td>26.0</td>
<td>2.9</td>
<td>4.2</td>
</tr>
<tr>
<td>Curvature coefficient $C_c = \frac{D_{30}^2}{D_{10}D_{10}^2}$</td>
<td>0.6</td>
<td>1.6</td>
<td>0.8</td>
<td>1.2</td>
</tr>
<tr>
<td>Unified soil classification</td>
<td>SP-GW</td>
<td>GW</td>
<td>GP</td>
<td>SP</td>
</tr>
<tr>
<td>Maximum dry density, p.c.f. (kg/m$^3$)</td>
<td>134 (2146)</td>
<td>139 (2226)</td>
<td>115 (1842)</td>
<td>119 (1906)</td>
</tr>
<tr>
<td>Minimum dry density, p.c.f. (kg/m$^3$)</td>
<td>112 (1794)</td>
<td>114 (1826)</td>
<td>97 (1554)</td>
<td>87 (1393)</td>
</tr>
</tbody>
</table>

**FIG. 4.12** Results of drained triaxial tests on well and poorly graded sands and gravels (after Skermer and Hillis, 1970)
FIG. 4.13 Variation of rockfill friction angle with applied normal stress (Leps, 1970)
5. PRECIPITATION AND PIEZOMETRIC DATA

5.1 Subsurface Flow and Response to Precipitation

The stability of granular materials is known to be governed by effective stresses, which are in turn related to pore water pressures within the material. Consequently, a good understanding of the pore water pressure response within hillslopes is very important to assessing stability of the slope. The importance of piezometric response to slope stability in coastal settings cannot be understated, and it is for this reason that a careful prediction and understanding of its role and variability is of paramount importance to a meaningful stability prediction. In addition to the magnitude and frequency of occurrence of a given response, Iverson and Major (1986 and 1987) have astutely demonstrated the influence of seepage direction on slope stability and are skeptical of typical seepage-parallel-to-the-slope assumptions. It has been shown that the effect of non-parallel seepage is to impose an upward or downward gradient on surficial materials, which in turn decreases or increases effective stresses and hence shear strengths of frictional materials. It has also been demonstrated that due to the varying degree of weathering, fracturing, conductivity and topography of underlying ‘impermeable’ bedrock, the seepage-parallel assumption is rarely accurate.

In addition to similar strength and mechanical behaviour characteristics, the steep slopes with thin soil cover of coastal British Columbia are also exposed to similar climatic stresses and exhibit a characteristic hydrological response to precipitation events. Haneberg (1991) and Wieczorek (1987) observed that thin soil instability is usually related to short storms of high intensity (i.e. less dependent on antecedent moisture and precipitation) while thicker hillside soil instability is usually related to prolonged storms of moderate intensity. Jackson and Cundy (1992) and Sidle et al. (1985) have suggested that the hydrologic response of these slopes is influenced strongly by the presence of very
thin soils, a dense network of roots and other macro-pore structures and highly divergent or convergent steep slopes. Despite observations of an extreme spatial variation in piezometric responses of a hillslope, many attempts to model or predict these responses have been made (Sidle and Terry, 1992; Buchanan et al., 1990; Jackson and Cundy, 1992; Reddi and Wu, 1991; Freeze, 1971 and Pierson, 1980). The common objective of these models is to predict groundwater levels, based on a number of input parameters, including antecedent moisture conditions, total precipitation and intensity, soil porosity and hydraulic conductivity, infiltration, evaporation, slope morphology, or position, catchment area and several others. These models are typically empirical in nature or based on 1, 2 or 3-dimensional finite difference methods, or, in the case of the model proposed by Reddi and Wu (1991), based on a simplified probabilistic lumped-parameter model which includes Bayesian updating. Of the input parameters mentioned above, it seems intuitive that slope morphology and the upslope catchment area would play key roles in predicting response differences, yet these two parameters seem quite difficult to include in a model. Of special note is the ability of Jackson and Cundy’s (1992) 2-dimensional, finite difference, saturated subsurface flow model to consider varied topography such as divergent and convergent slope positions and local depressions.

It is evident that several approaches to obtaining piezometric input to stability analyses are available. The empirical and physically based modelling techniques described above are supplemented by observations of meteorological and moisture antecedents to documented debris flows as discussed by Church and Miles (1987), Johnson and Sitar (1989 and 1990) and Wieczorek (1987). A third approach is to use a series of relatively long term piezometric records from a given site in order to obtain a characteristic response, independent of specific precipitation events, which can be presented as a time series of groundwater levels with corresponding statistical return periods. Provided the areas of interest have similar meteorological and surficial hydrological characteristics, it is argued that the results of such an approach can be used across a series of sites with confidence. It is this approach,
based on piezometric data from the Carnation Creek Experimental Watershed, which is used here. The objective of this chapter is to present these precipitation and piezometric data and to place them in a context which will be of use as input to a probabilistic translational slope stability model. The spatial and temporal variation of these data are addressed and a suitable probability density function which accommodates these features will be proposed.

5.2 Carnation Creek Precipitation Gauges and Piezometer Nest

As described in section 3.2.2, the Carnation Creek Experimental Watershed is one of four locations included in the program of in-situ field testing and mapping. Government/industry/university sponsored research conducted at this watershed since 1972 has included, amongst many other studies (Chamberlin, 1987), a long term, nearly continuous piezometer nest installation and several precipitation stations. The data from this study, provided by Dr. Eugene Hetherington, through the Canadian Forest Service, are of particular value due to its long and nearly-continuous duration, the variability of piezometer positions and the proximity of this nest of installations to both the 'Eugene' and 'Bob' landslides.

Amongst several precipitation gauges located within the watershed, Station E (Fig. 3.6) is used to demonstrate the meteorological characteristics of the area. This gauge was moved twice during the 19 years of record (October 1, 1972 to September 30, 1991) in order to accommodate harvesting activities in the area. A plot of hourly precipitation data collected within Carnation Creek is provided as Fig. 5.1, in which the seasonal variation of precipitation events is evident. Figure 5.2 shows the precipitation intensity / duration / frequency curves for the 19 year record of Station E while Fig. 5.3 provides the total precipitation / duration / frequency curves for both Station E of the Carnation Creek Experimental Watershed and the Seymour Falls weather station, near the Jamieson Creek landslide. The records from both stations are provided in order to show the similarity between the two sites and
support the use of observations from Carnation Creek, on the west coast of Vancouver Island, at the Jamieson Creek, Holberg and Sand River landslides.

With an objective of studying the hydrologic response of forest slopes to precipitation events prior to and following harvesting, 77 piezometers were installed within approximately 1 km of precipitation Station E between 1975 and 1982. Of these 77, most were monitored continuously using an analog drum system and pressurized nitrogen while the remainder were visited on a nearly monthly basis in order to manually record the monthly maximum from a crest tube. Of those monitored continuously, 17 drum records were digitized at 15 minute intervals and made available for this study. The piezometers were installed in a variety of slope positions and aspects, including divergent and convergent curvatures, steep-to-shallow and shallow-to-steep slope breaks, open slopes and, as a means of control and for the purpose of comparison, in areas which were and were not harvested. The installation depths ranged from 26 to 216 cm, to the interpreted depth to bedrock and the most likely plane of rupture, and the method of installation involved hand excavation and placement with as little upslope disturbance as possible. With this range of installation depths in mind, piezometric response is hereafter referred to as a normalized groundwater ratio, \( \frac{D_w}{D} \), in which \( D_w \) is the vertical height from the potential or existing failure plane (assumed to be near the base of the piezometer) to the groundwater surface and \( D \) is the vertical height from this plane to the ground surface. The practical range of this ratio is from 0 to 1.0, with \( \frac{D_w}{D} = 1.0 \) representing a fully saturated soil horizon. However, it is recognized that a ratio in excess of 1.0 could occur due to a transient positive pore pressure wave or pulse, an upward gradient, as alluded to by Iverson and Major (1987), or, less likely, surface flow. A typical piezometer installation (P847), with provision for continuous drum logging, prior to harvesting, is illustrated in Plate 5.1. Table 5.1 provides the details of soil depth, slope aspect, uphill, local and downhill slope angle, elevation and slope morphology for each of the
continuously recorded and digitized piezometer records. The recording periods of each of these installations are shown in Fig. 5.4, which represents approximately 1.35 million individual readings.

Table 5.1 Piezometer installation depth and slope morphology details (continuously recorded and digitized piezometers only)

<table>
<thead>
<tr>
<th>Piezometer number</th>
<th>Soil depth (cm)</th>
<th>Slope aspect (°)</th>
<th>Uphill slope (° (%))</th>
<th>Local slope (° (%))</th>
<th>Downhill slope (° (%))</th>
<th>Elevation (masl)</th>
<th>Slope morphology</th>
</tr>
</thead>
<tbody>
<tr>
<td>P803</td>
<td>88</td>
<td>13</td>
<td>18 (32)</td>
<td>40 (84)</td>
<td>18 (32)</td>
<td>211</td>
<td>C</td>
</tr>
<tr>
<td>P814</td>
<td>163</td>
<td>29</td>
<td>29 (55)</td>
<td>20 (36)</td>
<td>20 (36)</td>
<td>259</td>
<td>B</td>
</tr>
<tr>
<td>P816</td>
<td>85</td>
<td>50</td>
<td>20 (36)</td>
<td>20 (36)</td>
<td>20 (36)</td>
<td>243</td>
<td>B</td>
</tr>
<tr>
<td>P820</td>
<td>123</td>
<td>20</td>
<td>22 (40)</td>
<td>22 (40)</td>
<td>22 (40)</td>
<td>245</td>
<td>A</td>
</tr>
<tr>
<td>P823</td>
<td>178</td>
<td>333</td>
<td>19 (34)</td>
<td>19 (34)</td>
<td>19 (34)</td>
<td>248</td>
<td>A</td>
</tr>
<tr>
<td>P825</td>
<td>128</td>
<td>26</td>
<td>38 (78)</td>
<td>23 (42)</td>
<td>38 (78)</td>
<td>232</td>
<td>A</td>
</tr>
<tr>
<td>P845</td>
<td>96</td>
<td>45</td>
<td>32 (62)</td>
<td>23 (42)</td>
<td>23 (42)</td>
<td>186</td>
<td>C</td>
</tr>
<tr>
<td>P847</td>
<td>105</td>
<td>45</td>
<td>31 (60)</td>
<td>31 (60)</td>
<td>32.5 (64)</td>
<td>200</td>
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<tr>
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<td>100</td>
<td>45</td>
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<td>21 (38)</td>
<td>21 (38)</td>
<td>208</td>
<td>D</td>
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<tr>
<td>P856</td>
<td>103</td>
<td>187</td>
<td>22 (40)</td>
<td>22 (40)</td>
<td>22 (40)</td>
<td>232</td>
<td>B</td>
</tr>
<tr>
<td>P858</td>
<td>130</td>
<td>163</td>
<td>34.5 (69)</td>
<td>29 (55)</td>
<td>29 (55)</td>
<td>259</td>
<td>C</td>
</tr>
<tr>
<td>P859</td>
<td>107</td>
<td>215</td>
<td>29 (55)</td>
<td>29 (55)</td>
<td>29 (55)</td>
<td>175</td>
<td>A</td>
</tr>
<tr>
<td>P863</td>
<td>125</td>
<td>215</td>
<td>15 (27)</td>
<td>15 (27)</td>
<td>15 (27)</td>
<td>163</td>
<td>A</td>
</tr>
<tr>
<td>P872</td>
<td>110</td>
<td>115</td>
<td>32 (62)</td>
<td>32 (62)</td>
<td>32 (62)</td>
<td>193.5</td>
<td>A</td>
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<tr>
<td>P875</td>
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<td>115</td>
<td>32 (62)</td>
<td>32 (62)</td>
<td>32 (62)</td>
<td>193.5</td>
<td>A</td>
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<td>P883</td>
<td>118</td>
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<td>24 (45)</td>
<td>24 (45)</td>
<td>191.4</td>
<td>A</td>
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<tr>
<td>P886</td>
<td>94</td>
<td>185</td>
<td>24 (45)</td>
<td>24 (45)</td>
<td>24 (45)</td>
<td>191.4</td>
<td>A</td>
</tr>
</tbody>
</table>

A: Open slope
B: Topographical depression (water accumulator)
C: Steep to shallow slope break
D: Shallow to steep slope break

5.3 Spatial and Temporal Variation of Piezometric Response

A 15 year research project in the Carnation Creek Experimental Watershed was developed and completed as described by Hartman and Scrivener (1990). The project was divided into 3 distinct, 5 year stages: pre-logging, logging and post-logging, and was intended to study the impacts of forestry practices on the coastal stream ecosystem of Carnation Creek. Approximately 41% of the watershed was clearcut using both cable and grapple yarding systems over a period of approximately 6 winters. The cutblock surrounding the piezometer nest discussed in this chapter was harvested during the winter
Chapter 5. Precipitation and Piezometric Data

of 1977-78. The majority of access roads throughout the watershed were constructed using Porclain shovels and D-8 caterpillars with full bench construction (Hartman and Scrivener, 1990).

A preliminary analysis of the digitized piezometer records has been completed in an attempt to establish the existence or lack of any long-term trends between pre- and post-logging piezometric responses. For each of the 17 digitized piezometer records, the pre- and post-harvest maximum, mean, range and standard deviation of the groundwater ratio were extracted for comparison. Figs. 5.5a through 5.5d show the pre- versus post-harvest responses for each of the records, in which the dashed lines represent no observable difference and the solid lines represent the linear regression through all of the data points. As the two sets of lines are almost coincident in all cases, it is concluded, with exception to piezometers 803 and 847, that no significant pre- versus post-harvest difference was measured. In the case of piezometers 803 and 847, it is believed (Hetherington, 1994) that these two sites experienced some surficial yarding disturbance during logging, which altered surficial drainage paths and the resulting piezometric responses. It would appear from field evidence and observation that the most significant effects of logging on slope hydrology are:

- altered drainage paths and a resulting concentration of hydrological stresses due to road building,
- surficial yarding disturbance and the resulting channelization of surficial runoff, and
- the development of a macro-pore network by means of root degradation.

Similar plots of mean and maximum groundwater ratios versus soil depth and slope angle in Figs. 5.6a through 5.6d also demonstrate a lack of conclusive evidence for the variation of groundwater ratio with soil depth and slope angle. These results do not disprove the possibility of a pre- versus post-harvest relationship or a dependence on soil depth or slope angle, they merely show the great variability of piezometric response over a number of slope positions, aspects, depths and periods, and suggest the need for a statistically significant and independent number of piezometers of each type in
order to draw general conclusions. The immense value of these data is clearly their temporal variation or the ability to determine the frequency of groundwater ratios of a certain magnitude within a given period of observation. These observations and the resulting conclusions are presented in the following sections.

These digitized data are used to study the variation of piezometric response with slope position. Other studies and validations of assumed behaviour include the expected difference between forested slope and clearcut responses and the effect of soil depth and slope angle on piezometric response. Although it can be argued that a point located within a converging slope with a large catchment area will experience more exaggerated peak groundwater ratios than a similar point on a divergent slope, in reality, it seems that local variations in macro-pore features and soil type obscure this difference and prevent validation of this expected behaviour. In a similar fashion, it is not counter intuitive that a point located at a steep-to-shallow slope break should experience an exaggerated response due to the higher potential of the uphill groundwater and the potential for bedrock seepage points at these locations. Again, probably due to local macro-pore variations and irregularities, this expected behaviour has not been confirmed nor disproved by the analysis of the Carnation Creek piezometric data. Careful attempts to group these piezometer records into classes, according to slope position, aspect, elevation, soil depth and slope angle have proven to be fruitless. What has been determined is a generalized piezometric response and a concept of statistical frequency of this response.

5.4 Piezometric Data Analysis

5.4.1 Data Filtering and Organization

For the purpose of analysis, all year, month, day and time fields of the original records were converted to years since January 1, 1974. This allowed comparison between piezometers over the
same time periods and facilitated division of the decade between January 1, 1974 and January 1, 1984 into monthly segments. The periods of record are illustrated in Fig. 5.4, starting in July, 1975. After compiling all data into two files for each piezometer, one each for pre- and post-harvesting data, erroneous entries were removed and the largest groundwater ratio, $D_w/D$, in each month was extracted to create a monthly maximum series. It is recognized that certain statistical criteria must be met for the results of a frequency analysis to be valid: randomness, independence, stationarity and homogeneity. Although not all of these criteria are completely satisfied, the natural variation of these monthly maximums and the duration and quality of these piezometric records lend confidence to the frequency analysis presented in the following sections.

### 5.4.2 Extreme Value (Gumbel) Analysis

The recurrence of natural events with greater than normal magnitudes is well-described by extreme value statistics. Common examples of the use of extreme value statistics include the recurrence of rock and snow avalanches, floods and high river levels, large waves, precipitation and record temperatures. Extreme value statistics have been used and reported by several government agencies and researchers (BC Ministry of Environment precipitation records; Church and Miles, 1987; McClung and Mears, 1991).

Three different forms of extreme value functions exist: Type I, the Gumbel distribution, for which the probability density function, PDF, is non-zero over the entire real number range and Types II and III (Weibull), for which an additional parameter, $k$, introduces further non-linearity and causes the PDF to have a finite intercept. Type I, the Gumbel distribution, is used to represent these piezometric data.
The Gumbel distribution is described by a straight line on a plot of groundwater ratio, \( \frac{D_w}{D} \), versus a transformed axis of the non-exceedence probability, \( P_n \). The form of the equation describing this distribution is:

\[
y = \frac{D_w}{D} = m \cdot \xi + b \quad (5.1)
\]

where the x-axis is a transformation of \( P_n \):

\[
\xi = -\ln[-\ln(P_n)] \quad (5.2)
\]

and, \( P_n \) is determined for each monthly maximum using either Weibull’s or Hazen’s plotting position method. Preliminary analysis revealed no significant difference between the two approaches, hence the Weibull method was adopted. In order to determine Weibull’s plotting position, the monthly maximums for each piezometer record are placed in order from smallest to largest and given a rank number, ‘i’, from 1 to ‘n’, where ‘n’ is the number of months in a particular record. The non-exceedence probability for each groundwater ratio value is given by:

\[
P_n = \frac{i}{n + 1} \quad (5.3)
\]

The Gumbel distribution has the following format:

\[
y = \frac{D_w}{D} = m \cdot \left[ -\ln(-\ln(P_n)) \right] + b \quad (5.4)
\]

where ‘m’ and ‘b’ are referred to as the scale and location parameters respectively, and represent the slope and intercept of the linear regression line through the Weibull plotting positions. Figure 5.7 shows the plotting positions for each of the monthly maximums of each piezometer record, bounded by piezometers P859 and P825 (on open slope locations, see Table 5.1).
If, for each of the 120 months between January 1974 and January 1984, the monthly maximums across all piezometer records are averaged and placed in a ranked list as described above, the plotting positions may be determined and plotted as the open diamond symbols of Fig. 5.7. The plotted average monthly maximums approximate a bi-linear curve, as do the individual maximums of each record. It is believed that the steep linear section of this plot corresponds to relatively high frequency precipitation events and is a characteristic of the moisture retention capacity at this particular site rather than an extreme hydrological response to intense and relatively infrequent precipitation events. The extent of this higher frequency, steeper, section is somewhat dependent on the period over which maximum ratios are recorded, which is one month in this analysis. Given an ideal, long-term piezometric record of 30 to 50 years, a more appropriate period for recording maximum ratios would be one year, as opposed to one month, which would likely result in only one linear trend through the plotting positions since the yearly maximum event is likely to be more representative of an extreme hydrological event. Consequently, the flatter, linear section is believed to represent the true, extreme hydrological response through which the Gumbel fit and scale and location parameters are determined to be $m=0.043$ and $b=0.639$ (see Fig. 5.7).

Another common interpretation of non-exceedence probability is the concept of return period. The return period, $R_t$, of any groundwater ratio value with discrete monthly ‘trials’ may be calculated from the plotting position or non-exceedence probability for one exceedence in any one month:

$$P_n = \left( 1 - \frac{1}{R_t} \right)$$  \hspace{1cm} (5.5)

and for a period of observation of ‘$N’ months:
Chapter 5. Precipitation and Piezometric Data

\[ [P_n]_N = \left(1 - \frac{1}{R_t}\right)^N \]  \hspace{1cm} (5.6)

If Equation 5.4 is rearranged to isolate the non-exceedence probability, and if this probability is also recognized to be the value of the cumulative distribution function, CDF, for a particular value of groundwater ratio, then

\[ F(x) = P_n = \left(1 - \frac{1}{R_t}\right) = \exp\left[- \exp\left(- \frac{x - b}{m}\right)\right] \]  \hspace{1cm} (5.7)

where \( x = \frac{D_n}{D} \). The cumulative distribution function (CDF) of Equation 5.7 must be differentiated in order to obtain the groundwater ratio probability density function (PDF):

\[ f(x) = \frac{dF(x)}{dx} = \frac{1}{m} \cdot \exp\left[- \frac{x - b}{m} - \exp\left(- \frac{x - b}{m}\right)\right] \]  \hspace{1cm} (5.8)

Recalling that the non-exceedence probability for a certain magnitude of groundwater ratio is dependent on the length of the observation period (as demonstrated in Equation 5.6), different cumulative distribution and probability density functions should be calculated for each different duration of observation, ‘N’. The density functions of Equations 5.7 and 5.8 are based on any one month of observation. To account for different observation periods, the cumulative distribution function of Equation 5.7 is rewritten:

\[ F(x)^N = [P_n]_N = \left(1 - \frac{1}{R_t}\right)^N = \left[\exp\left(- \exp\left(- \frac{x - b}{m}\right)\right)\right]^N \]  \hspace{1cm} (5.9)

and differentiated to obtain the general groundwater ratio probability density function (PDF), \( f(x)^N \):

\[ f(x)^N = \frac{dF(x)^N}{dx} = \frac{N}{m} \cdot \exp\left[- \frac{x - b}{m} - N \cdot \exp\left(- \frac{x - b}{m}\right)\right] \]  \hspace{1cm} (5.10)
Four probability density functions (PDF’s), suitable for input to a probabilistic stability analysis, corresponding to 1 month and 1, 10 and 20 year assumed observation periods are presented in Fig. 5.8. The 20 year observation period is commonly considered to be the approximate cycle time or critical period following harvesting for slope stability, after which natural or planted revegetation tends to return the site to pre-harvest stability levels. For this reason, a PDF which represents approximately 20 years of ‘observation’ or simulation is required to provide an estimate of the likelihood of occurrence of extreme hydrological events during the 20 year period. The PDF’s are based on the synthetic plotting positions of average monthly maximums across all piezometers, as illustrated in Fig. 5.7. Inspection of the 1 year density function reveals a very small probability of \( \frac{D_w}{D} \) exceeding a value of 1.0, while observation over longer periods of 10 or 20 years results in a relatively significant probability of exceeding a value of 1.0. These expected, but relatively rare, values of \( \frac{D_w}{D} \) greater than 1.0 are confirmed by 33 such observations in three piezometers of the Carnation Creek Experimental Watershed. The range of \( \frac{D_w}{D} \) plotting positions in Fig. 5.7 is approximately 0.0 to 1.28, with the maximum return period calculated to be approximately 5 years.

5.5 Summary of Piezometric Data and Implications

Amidst several theories regarding the hydrological response of hillslopes to precipitation events, the understanding that groundwater ratio plays a key, if not paramount, role in slope stability is well accepted. For this reason, any attempt to develop a rigorous translational slope stability model should address expected groundwater ratios, \( \frac{D_w}{D} \), at the time of failure and the similarity of responses between sites. With this objective, precipitation data for Carnation Creek Experimental Watershed and Seymour Falls have been presented in this chapter while biogeoclimatic descriptions of all four
study sites were provided in Chapter 3. The Carnation Creek Experimental Watershed piezometric data were described and quantified by means of the Extreme Value Type I (Gumbel) distribution, which is well-suited to naturally recurring events and a synthetic (averaged over all digitized piezometer records) groundwater ratio probability density function (PDF), suitable for use as input to a probabilistic stability analysis, is proposed. The proposed model is well supported by observed maximum groundwater ratios and qualitative descriptions of hydrological response (Hetherington, 1994-1995).
FIG. 5.1 Hourly precipitation record, prior to and following harvest
FIG. 5.2 Precipitation intensity / duration / frequency curves: Carnation Creek, Station E; for period October 1, 1972 to September 30, 1991
FIG. 5.3 Total precipitation / duration / frequency curves: Carnation Creek, Station E and Seymour Falls
Plate 5.1 Piezometer installation with continuous drum recorder, Carnation Creek Experimental Watershed (Hetherington)
FIG. 5.4 Piezometer installation recording periods
FIG. 5.5  Pre- versus post-harvest piezometric response (maximum, mean, range and standard deviation of Dw/D ratio)
FIG. 5.6 Morphological variation of piezometric response (slope angle and installation depth)
Chapter 5. Precipitation and Piezometric Data

Non-exceedance probability, $P_n$ (for 1 month observation period)

<table>
<thead>
<tr>
<th>$P_n$</th>
<th>0.1</th>
<th>0.25</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
<th>0.95</th>
<th>0.99</th>
<th>0.995</th>
</tr>
</thead>
</table>

Return period, $R_t$

<table>
<thead>
<tr>
<th>$R_t$</th>
<th>1.25 mth.</th>
<th>2 mth.</th>
<th>6 mth.</th>
<th>1 yr.</th>
<th>2 yr.</th>
<th>5 yr.</th>
<th>10 yr.</th>
<th>15 yr.</th>
<th>20 yr.</th>
</tr>
</thead>
</table>

Piezometer P859

Piezometer P825

Individual piezometer data

Average monthly maximum across all piezometers

Extreme value fit ($m = 0.043, b = 0.639$)

Groundwater ratio, $D/D$

Extrem Value (Gumbel) transformed plotting position

$-\ln(-\ln(P_n)) = -\ln(-\ln(1-1/R_t))$

$P_n$: Non-exceedance probability

$R_t$: Return period

FIG. 5.7 Extreme Value (Gumbel) piezometric plotting positions and synthetic relationship
Chapter 5. Precipitation and Piezometric Data

FIG. 5.8 Groundwater ratio probability density functions for periods of 1 month and 1, 10 and 20 years

\[ f(x)^N = \frac{dF(x)^N}{dx} = \frac{N}{m} \cdot \exp\left[ -\frac{x-b}{m} \right] \cdot \exp\left( 1 - \frac{x-b}{m} \right) \]

Where \( x = \frac{D_w}{D} \), \( N \) = study period (months),
\( m = 0.043 \) and \( b = 0.639 \).
6. INFINITE SLOPE MODEL VALIDATION - BACK-ANALYSIS

6.1 General

The use of back-analysis techniques in geotechnical engineering is common, and based on a desire to calculate one or several geotechnical, hydrological or physical parameters and further obtain a sense of the most likely failure mechanism or range of parameters. The back-analysis is, of course, predicated on a prior knowledge of failure and the assumption that conditions at the time of failure can be estimated during a forensic survey of the slide area. The advantages and limitations of performing a back-analysis in comparison to laboratory testing are well described by Duncan and Stark (1992), who also point out that in theory, back-analysis should allow for the determination of friction angle and cohesion if the depth to failure is known. This is only true if the failure plane is not controlled by a hard or impermeable boundary, as is the case for typical forested slope failures. It is therefore reasonable to perform a back-analysis of those sites which were presented in Chapter 3 in order to determine any one, less known, parameter, such as groundwater ratio, $\frac{D_w}{D}$.

The most notable comments made by Duncan and Stark (1992) include the observations that back-analysis provides estimates of parameters over the entire area of failure rather than the much smaller, and locally variable, failure planes of laboratory and in-situ tests, with no effects of disturbance. It has also been argued that back-analysis gives an investigator an estimate of the average mobilized shear stress over the entire failure plane instead of the single maximum or large strain shear strength which is obtained from smaller scale testing and typically applied over the entire failure plane during conventional analysis. While a potential sliding mass must remain kinematically feasible, certain segments of the failure surface must endure greater strains than others due to slightly varying topography, which leads to progressive failure of the sliding mass. However, recognizing the
geological processes (colluvial) to which these materials are subjected, it is felt that at the moment of full scale initiation, a sliding mass has undergone substantial strain over its entire rupture surface and is thus mobilizing the large strain shear strength over this entire surface.

This chapter presents the infinite slope model as it is proposed by Hammond et al. (1992) and applied to open slope failures in coastal British Columbia. The model is based on several substantial, yet reasonable, assumptions and makes use of nine input parameters. The sensitivity of the model and the variation in sensitivity of the model with different central values are discussed, followed by a deterministic back-analysis of the groundwater ratio to cause failure at each of the four sites examined in this study.

6.2 Infinite Slope Model Assumptions

A schematic diagram showing the model parameters and general equation is provided as Fig. 6.1. Hammond et al. (1992) have provided a good summary of the assumptions of this infinite slope model:

- groundwater surface and failure plane assumed parallel to the ground surface,
- failure plane assumed to be of infinite extent, with no side-effects or contributions to strength from lateral boundaries,
- only one soil layer is assumed and the layer through which failure occurs should be the one for which strength parameters are specified in the model, and
- the infinite slope model is two-dimensional and is thus well-suited to open slope, or planar, failure surfaces with no confining or divergent effects.

The effects of non-parallel groundwater seepage and magnitude of the hydraulic gradient are discussed by Iverson and Major (1986, 1987), who show that stability is minimized when the seepage direction is closer to horizontal than the slope angle by an amount equivalent to the friction angle of the
material. In contrast, stability is maximized when the seepage direction is closer to vertical than the slope angle by an equivalent amount.

### 6.3 Model Sensitivity and Input Parameters

The infinite slope equation is expressed as a ratio of available strength, or resistance, to mobilized stress:

$$FS = \frac{C_r + C_s + \cos^2 \alpha \cdot \left[ \gamma_{o} + \gamma \cdot \left( D - \frac{D_w}{D} \cdot D \right) + \left( \gamma_{sat} - \gamma_w \right) \cdot \frac{D_w}{D} \cdot D \right] \cdot \tan \phi_{cv}}{\sin \alpha \cdot \cos \alpha \cdot \left[ \gamma_{o} + \gamma \cdot \left( D - \frac{D_w}{D} \cdot D \right) + \gamma_{sat} \cdot \frac{D_w}{D} \cdot D \right]} \quad (6.1)$$

Equation 6.1 has nine input parameters and one constant ($\gamma_w$ = unit weight of water, 9.807 kN/m$^3$), as described in Table 6.1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_r$</td>
<td>Root cohesion (kPa) (approximated by difference in apparent cohesion between linear regression through in-situ direct shear strength data and linear regression through reconstituted laboratory (Fig. 4.11))</td>
</tr>
<tr>
<td>$C_s$</td>
<td>Soil cohesion (kPa) (assumed to be zero (Fig. 4.11))</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Slope angle (°)</td>
</tr>
<tr>
<td>$\gamma_{o}$</td>
<td>Surface surcharge (kPa)</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Moist unit weight (calculated using average moisture content over all sites, specific gravity, $G_s = 2.60$ and void ratio, $e = 1.35$)</td>
</tr>
<tr>
<td>$\gamma_{sat}$</td>
<td>Saturated unit weight (kN/m$^3$) (calculated as above with saturation, $S_r = 100%$)</td>
</tr>
<tr>
<td>$D$</td>
<td>Depth from ground surface to failure plane (m) (assumed constant over failure plane)</td>
</tr>
<tr>
<td>$D_w / D$</td>
<td>Groundwater ratio (also assumed constant over failure plane)</td>
</tr>
<tr>
<td>$\phi_{cv}$</td>
<td>Friction angle (°) (Fig. 4.11)</td>
</tr>
</tbody>
</table>

Sensitivity studies of Equation 6.1 reveal, as expected, a decrease in factor of safety with decreasing friction angle and soil and root cohesion, and with increasing slope angle and groundwater ratio. Of
more interest is the observation that stability is most sensitive to slope and groundwater ratio, and least sensitive to moist and saturated unit weights and tree surcharge. Any sensitivity study is dependent on the central values which are chosen for each of the parameters for the study, and as such, it is worth noting that the sensitivity to soil and root cohesion increases, while the sensitivity to groundwater ratio and friction angle decreases, with decreasing depth. Hammond et al. (1992) and Sidle (1984) have observed that sensitivity of soil and root cohesion is very high in the case of shallow soils on steep slopes when the soils are nearly saturated. Inspection of the Mohr-Coulomb strength envelope for all in-situ data and a linear fit (see Figs. 4.11 and 6.4) indicates the apparent cohesive component of mobilized shear strength is greater at lower normal stresses while the frictional component of mobilized shear strength is greater at higher normal stresses.

Simons and others (1978) have shown that tree surcharge is a marginal benefit to stability. More detailed studies of the effects of tree surcharge on slope stability have been reported by Gray and Leiser (1982), and provide further evidence that slope stability is relatively insensitive to this parameter.

From this brief introduction to the infinite slope model, its input parameters and its sensitivity to those parameters, it is evident that, given limited time and resources, the most valuable information for stability analysis includes careful measurements of slope angle and depth to the expected failure plane and estimates or measurements of potential groundwater ratios and shear strength parameters.

6.3.1 Measured, Estimated and Temporally Varying Parameters

Having introduced the role of each input parameter in the infinite slope model it is useful to divide the parameters into three distinct categories: measured, estimated and temporally varying parameters. Measured parameters include those which are site specific and obtained during a field survey of a
terrain polygon or landslide site. These measured parameters include slope angle, α, depth to failure plane, D, and shear strength parameters, φ_{cv} and C_s, if measured using a direct shear device as described in Chapter 4 or another suitable method. Estimated parameters include those for which reasonable values are chosen, based on reported values for similar soils and sites. The majority of estimated parameters, such as surface surcharge, q_0, and moist and saturated unit weights, γ and γ_{sat}, play a relatively insignificant role in stability analyses, as discussed in the previous section. An exception is the relative importance of root cohesion, C_r, and the need to estimate it.

If one considers the geological time scale, the temporal variation of slope stability could be related to changes in material properties such as friction angle and cohesion due to weathering and mechanical alterations, such as cementation, or related to changes in site characteristics such as slope angle and groundwater ratio due to extensive erosion or climatic changes. Although interesting, this time scale and the potential variations in stability are not applicable to the management of vegetated slopes and risk of damage to downslope resources. Considering, now, a time scale in the order of 5 to 25 years, which is more applicable to forest resources management and the risk to downslope resources due to harvesting, the temporal variation of slope stability is likely related to changes in root cohesion due to root degradation and regrowth and the frequency of extreme groundwater ratios over this period. For instance, over the first ten years following harvesting of a given cutblock, the non-exceedence probability, P_n, for an extreme value of \( \frac{D_w}{D} \) may be calculated from Equation 5.9 with ‘N’ equal to 120 months. As the value of ‘N’ increases the non-exceedence probability for a specific groundwater ratio decreases.

The enhancing role of roots in the shear strength of a soil mass is well documented by several researchers (Gray and Leiser, 1982; Greenway, 1987; O'Loughlin and Ziemer, 1982; Gray and
Chapter 6. Model Validation - Back-Analysis

Ohashi, 1983; Wu et al., 1979; Buchanan and Savigny, 1990). Gray and Ohashi (1983) and O'Loughlin and Ziemer (1982) have demonstrated that root reinforcement does not affect the friction angle of sand but does act to provide an apparent cohesive component to the shear strength. Other observations have included the fact that maximum tensile resistance of each root in the area of a slope failure is mobilized at a different time and amount of strain. Also, the maximum tensile resistance of roots may not be mobilized because the roots may pull out prior to breaking in tension (Gray and Ohashi, 1983). A collection of eleven root cohesion studies reviewed by Hammond et al. (1992) shows the majority of values of root cohesion lie between approximately 2 kPa and 7 kPa, with a range of nearly 0 kPa to 20 kPa. The contribution of root cohesion to shear strength is further complicated by effects of root morphology, density and distribution. Four classes of root morphology are described by Tsukamoto and Kusakabe (1984), depending on species, soil thickness, substratum penetration and the resulting likelihood of intersection of roots and the failure plane. For the purposes of model validation and back-analysis, a value of 1.5 kPa has been adopted for root cohesion, which is the observed difference between linear fits to the failure envelopes of in-situ direct shear tests performed on undisturbed columns and laboratory tests performed on dry, reconstituted samples (see Fig. 4.11).

Studies by Burroughs and Thomas (1977), Ziemer (1981) and O'Loughlin (1974) have demonstrated the decay of the net available root reinforcement following harvesting, reaching a minimum after approximately 3 to 5 years, with a return to pre-harvest levels after approximately 10 to 20 years of regrowth. Sidle (1991) presented a method of simulation for changes in root cohesion following harvesting. The proposed method involves two functional relationships: (i) root deterioration as described by an exponential decay function and, (ii) regrowth of newly planted or invading vegetation as described by a sigmoid relationship. The exponential relative root strength deterioration curve is approximated by:

\[ R(t) = R_0 e^{-kt} \]
where \( t \) is time since vegetation removal in years and \( \kappa \) and \( \nu \) are empirical constants. The sigmoid relative root strength regrowth curve is described by:

\[
C_{\text{regrowth}} = \left( A + B e^{-\kappa (t - t_{\text{lag}})^{-1}} \right) + X \tag{6.3}
\]

where \( A \), \( B \), \( K \) and \( X \) are empirical constants as discussed and derived by Sidle (1991) and \( t_{\text{lag}} \) represents the number of years between harvest and planting. At any year, \( t \), following harvest, \( C_r \) can be calculated as: \( C_r = C_{r, \text{pre-harvest}} + (C_{\text{decay}} + C_{\text{regrowth}}) \), where \( C_{r, \text{pre-harvest}} \) is the value of root cohesion prior to harvesting. Reasonable values for each of these six empirical constants have been chosen from Sidle (1991), who has fitted data from several other researchers to Equations 6.2 and 6.3. The selected constants and a plot of the resulting relative root cohesion curve for a period of 0 to 25 years following logging are provided in Table 6.2 and Fig. 6.2. The selected values for root regrowth constants are based partly on data for coastal Douglas Fir. Root decay constants have been selected from those presented by Sidle (1991) which result in a typical root decay-regrowth curve shape.

\[ C_{\text{decay}} = e^{-\kappa t} \] \hspace{1em} (6.2)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Root decay</th>
<th>Root regrowth</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A )</td>
<td>-</td>
<td>0.952</td>
</tr>
<tr>
<td>( B )</td>
<td>-</td>
<td>19.05</td>
</tr>
<tr>
<td>( X )</td>
<td>-</td>
<td>-0.05</td>
</tr>
<tr>
<td>( K )</td>
<td>-</td>
<td>0.25</td>
</tr>
<tr>
<td>( \kappa )</td>
<td>0.3</td>
<td>-</td>
</tr>
<tr>
<td>( \nu )</td>
<td>0.8</td>
<td>-</td>
</tr>
</tbody>
</table>

It is noted from Fig. 6.2 that if root regrowth were to be postponed by five years due to a lag period before replanting, leading to an offset of the root regrowth curve, the sum of the decay and regrowth curves (the relative root cohesion curve) would reach a significantly lower minimum value a few years later.
Chapter 6. Model Validation - Back-Analysis

after the existing minimum. The choice of values for the empirical constants of Equations 6.2 and 6.3 is, of course, debatable and based on no testing or validation by the author. The purpose of these equations and the resulting ‘relative root cohesion’ curve of Fig. 6.2 is to introduce the temporal variation of the ‘C,’ infinite slope input parameter and to demonstrate its use in stability analyses. There is a clear need for more research and validation in this area.

Recognizing that the ‘relative root cohesion’ curve of Fig. 6.2 may be sampled or ‘observed’ over any time period or window within the 25 years or more since harvesting, it is useful to create a probability density function, similar in form and purpose to those for groundwater ratio, for each of several observation periods of root cohesion. These probability density functions (PDF’s) are obtained by horizontally sampling the relative cohesion curve over the period of interest for the particular PDF. Having sampled the curve, the resulting distributions are normalized to have a unit area and plotted for two conditions in Fig. 6.3. The significance of a 5 year lag period between harvesting and replanting, as discussed in the previous paragraph, is most evident in these two figures, with Fig. 6.3a having no lag period and Fig. 6.3b having a five year lag period prior to replanting. The distributions show relatively little reduction in relative root cohesion within the first year, but a significantly depressed range of values over 10 and 20 year periods. The actual value of root cohesion which is used in a particular stability calculation is obtained by multiplying the value of relative root cohesion sampled from a distribution of appropriate duration by the pre-harvest maximum value of root cohesion, \( C_{r, pre-harvest} \).

6.4 Back-Analysis of In-situ Test Sites

Having identified and discussed the measured, estimated and temporally varying input parameters to the infinite slope model, it is desirable to validate the model itself through back-analysis of surveyed landslide sites and to gain insight to the range and most likely value of the parameters. In particular, it
is interesting to observe the effect of using high values, by previous standards, for the frictional shear strength of these materials while observing the range of groundwater ratio, $D_w / D$, and root cohesion, $C_r$, values required for incipient failure over all five documented landslides (4 sites).

The measured and estimated parameters for each of the five landslides are located in Table 6.3, followed by an estimated (and temporally varying) value for root cohesion at the time of failure which is applied to all five landslides. The chosen value of 1.50 kPa is based on several previous observations:

- the vertical distance between linear regression lines through all laboratory direct shear test data and all field direct shear test data is approximately 1.5 kPa (Fig. 4.11), indicating a cohesive strength of 1.5 kPa due to in-situ structure and/or root strength, and
- cohesive root strengths of 1.5 kPa correspond well with relative root strengths of 25% to 50% (as predicted minimums from Fig. 6.2) of pre-harvest root cohesion, $C_r_{pre-harvest}$, values, reported by others to be approximately 3 to 6 kPa.

A simultaneous back-analysis of all five landslides is performed by setting a Factor of Safety, FS, goal of 1.000 for each site while allowing the assumed groundwater ratio, $D_w / D$, specific to each site, to vary. This determination of groundwater ratio to cause failure over all five landslides serves to confirm the reasonable prediction of stability by the infinite slope model and ensures values of groundwater ratio to cause failure which are comparable to those discussed in Chapter 5.
### Table 6.3 Results of simultaneous back-analysis of five landslides

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Data Source</th>
<th>Back-analysis sites</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Seymour Watershed</td>
</tr>
<tr>
<td>Depth to failure surface, D (m)</td>
<td>M</td>
<td>1.25</td>
</tr>
<tr>
<td>Slope angle, α, degrees (%)</td>
<td>M</td>
<td>36 (73)</td>
</tr>
<tr>
<td>Surcharge, q_s (kPa)</td>
<td>A</td>
<td>0</td>
</tr>
<tr>
<td>Soil cohesion, C_s (kPa)</td>
<td>E</td>
<td>0</td>
</tr>
<tr>
<td>Moist unit weight, γ (kN/m³)</td>
<td>E</td>
<td>16.9</td>
</tr>
<tr>
<td>Buoyant unit weight, (γ阮-γw) (kN/m³)</td>
<td>E</td>
<td>8.0</td>
</tr>
<tr>
<td>Friction angle, φ_cv (°)</td>
<td>M</td>
<td>47</td>
</tr>
<tr>
<td>Root cohesion, C_r (kPa)</td>
<td>M</td>
<td>1.50</td>
</tr>
<tr>
<td>D_w / D (ratio)</td>
<td>C</td>
<td>0.75</td>
</tr>
<tr>
<td>Factor of Safety</td>
<td>A</td>
<td>1.000</td>
</tr>
</tbody>
</table>

M: Measured  
A: Assumed or Specified  
E: Estimated  
C: Calculated

#### 6.4.1 Implications for Piezometric Response

Based on summary Table 6.3, it is noted that a Factor of Safety, FS, of 1.00 was achieved for each of the five sites with common shear strength parameters, φ_cv and C_r, of 47° and 1.50 kPa, respectively.

For incipient failure, groundwater ratios between 0.75 at Jamieson Landslide, Seymour Watershed and 1.07 at Holberg Inlet, Coal Harbour were required. As discussed in Chapter 5, it is believed that, although rare, short-term groundwater ratios in excess of unity, without surface flow, are possible given some hydrogeological conditions. Groundwater ratios in this range also indicate the possibility that yarding- or operations-related ground disturbances have caused local groundwater concentrations and stressful hydrological situations. Ideally, the two back-calculated groundwater ratios for the ‘Bob’ and ‘Eugene’ landslides of 0.81 and 0.93, respectively, would be consistent with the observed piezometric responses of the piezometer installations which were closest to these two landslides (see Fig. 5.7). These back-calculated values of 0.81 and 0.93 (and those of the other three landslides) are well-represented by the cluster of measured data and the synthetic groundwater ratio curve shown on Fig. 5.7.
6.5 Summary of Model Validation and Back-Analysis

The assumptions, input parameters and sensitivity to these parameters have been discussed in context of the infinite slope model. It is observed that open slope stability in clearcut areas of coastal British Columbia is well-suited to analysis using the infinite slope model. A study of model sensitivity to input parameters by Hammond et al. (1992) has revealed a subset of three to four parameters, namely slope angle and approximate depth to the failure plane, groundwater ratio and shear strength, which are critical to a reliable estimate of slope stability. The temporally variable nature of at least two parameters, groundwater ratio and root cohesion, has been discussed. A constant value of root cohesion, $C_r$, has been estimated and used in the back-analyses and the required groundwater ratios to cause failure of all five landslides have been calculated. These back-analyses have provided insight into the range and appropriate use of these measured, estimated and temporally varying parameters. Values of groundwater ratio were allowed to vary until all five landslides were calculated to be in a state of incipient failure (F.S. = 1.0).

While the infinite slope model provides some insight into the mechanism of failure and sensitivity of the model to various physical phenomena and characteristics, it clearly stops short of fully characterizing the special, non-quantitative attributes of a polygon which may play a stabilizing or instigating role in the slopes behaviour. It is the objective of Chapter 7 and the remaining chapters of this thesis to study the role and influence of these qualitative terrain attributes in slope stability and to propose, calibrate and test a unified slope stability model which draws on the benefits of these two approaches.
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\[ FS = \frac{C_r + C_s + \cos^2 \alpha \left( q_o + 2(\frac{D_w}{D}) + (\gamma_{sat} - \gamma_w) \cdot \frac{D_w}{D} \right) \cdot \tan \phi_{cv}}{\sin \alpha \cdot \cos \alpha \left( q_o + 2 \left( \frac{D_w}{D} \right) + \gamma_{sat} \cdot \frac{D_w}{D} \right)} \]

FIG. 6.1 Schematic infinite slope model and stability equation
Chapter 6. Model Validation - Back-Analysis

FIG. 6.2 Relative root cohesion due to root mass decay and regrowth (0 to 25 years) (after Sidle, 1991)

<table>
<thead>
<tr>
<th>Parameter</th>
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<th>Root regrowth</th>
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</thead>
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<tr>
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<td>0.3</td>
<td>-</td>
</tr>
<tr>
<td>ν</td>
<td>0.8</td>
<td>-</td>
</tr>
</tbody>
</table>
FIG. 6.3 a) Relative root cohesion probability density functions (PDF) for 1, 10 and 20 years of observation and no lag period prior to replanting, b) with 5 year lag period prior to replanting
FIG. 6.4  Interpreted shear strength for model validation and back-analysis
7. TERRAIN ATTRIBUTE STUDY

7.1 Geostatistical Approach and Benefits

One of several objectives of forest terrain evaluation has, since the early 1980’s, and will continue to be the identification of polygons which will be subject to landslide activity following harvesting and the potential for damage to downslope resources. Tools designed to aid a professional during these evaluations generally fit into two categories:

- quantitative, site-specific slope stability modelling, and
- qualitative, geostatistical approaches used to characterize general locations, frequencies and magnitudes of landslides on a watershed scale.

Geostatistical techniques have been used successfully to classify terrain according to single or groups of descriptive parameters on analog (paper) and digital (GIS) base maps (Rollerson, 1984; Rollerson and Sondheim, 1985; Howes, 1987; Rood, 1990; Niemann and Howes, 1992 and Rollerson, 1992). Rollerson (1984) made use of histograms and frequency plots followed by Kruskal-Wallis and Spearman Rank procedures in order to obtain an ordered list of individual parameters for landslide prediction in TFL 44, on the west coast of Vancouver Island. Rollerson and Sondheim (1985) proposed a method by which several stability statistics are calculated for each polygon, including the probability of post-logging failure. Clearcut and road-related failures were studied separately using three different classifications for clearcut failure and two for road-related failures. Considering only the clearcut case, the three classifications are referred to as ‘remote sensing’, ‘geomorphic’ and ‘geomorphic plus aspect’, depending on the source and kind of information used. Howes (1987) identified a set of factors influencing clearcut failures and defined a set of terrain classes, based on these factors, such that any one polygon could be assigned to only one class. These classes were then
grouped into four stability ratings according to the ratio of the number of observed slides in each class to the number of clearcut hectares. The use of a Digital Elevation Model (DEM) as part of a GIS approach has been presented by Niemann and Howes (1992) for the Norrish and Cascade watersheds, in which a Cubic Clustering Criterion (CCC) is used to arrive at two classification schemes: a morphological classification and a clearcut classification. A comparison between the results of their proposed technique and the results of a conventional mapping exercise was made, and, despite the relatively high resolution 50 m raster size of the proposed DEM technique, it was concluded that a significantly finer resolution would be required to extract information on the smaller escarpments and gullying, which have a critical role in stability. With an objective of developing multifactor classifications for slope hazard mapping, Rollerson (1992) and Fannin and Rollerson (1993) studied a database of landslides in the Queen Charlotte Islands and addressed landslide frequency as related to individual and groups of landscape attributes. Some of this research work now underpins several of the guidebooks to the Forest Practices Code of British Columbia (FPC).

It is proposed to develop a slope stability model comprising the unification of a qualitative terrain attribute technique with a quantitative probabilistic (Monte Carlo simulation) stability analysis technique. The objective of this chapter is to present a subset of terrain attribute data concerning open slope failure occurrences and frequencies. This subset of data is to be used as input to both the qualitative and quantitative elements of the proposed slope stability model. It is the qualitative element, or the descriptive terrain attributes, which is addressed and reported in this chapter.

### 7.2 Data Collection and Analysis

With an objective of characterizing steepland terrain types which are susceptible to post-clearcutting and road building landslides and to better reflect the landbase truly available for harvest, Terrain Attribute Study - D33 was proposed and carried out by several forestry companies and the BC
Chapter 7. Terrain Attribute Study

Ministry of Forests during the late 1980’s and early 1990’s. This more objective and quantitative approach, which requires the collection of landslide frequency data within different terrain types, was intended to improve the reliability of the criteria used by mappers for slope stability assessments. A further objective of this data collection program was to develop a multi-factor terrain-based stability classification system which would be capable of predicting the likelihood and frequency of landslides following conventional clearcutting and road building (Thomson and Rollerson, 1992).

The proposed study area of Terrain Attribute Study - D33 covered areas within the Coast Ranges of the British Columbia mainland and the Insular Mountains of Vancouver Island. The study areas are grouped into seven distinct regions:

- Quatsino
- Gordon
- Cameron
- Clayoquot
- Kyuquot
- Klanawa
- Nootka

Within these regions, logged areas ranging in age from 6 to 15 years following harvesting were selected randomly for study. The lower age limit for the study areas was imposed to ensure the exposure of each area to several severe precipitation events and to allow time for any potential loss of net root strength to occur. Areas exhibiting several different bedrock formation types were stratified into different groups, from which a subset of logged areas from each formation was randomly selected for further study (Thomson and Rollerson, 1992).

The selected areas were mapped at 1:15,000 and 1:20,000 scales using the BC Terrain Classification System (Howes and Kenk, 1988) and aerial photography, after which all polygons were field checked. Terrain attribute data describing the landscape position, slope, aspect, slope morphology, curvature, soil type, drainage, surficial materials, bedrock type and the presence or absence of natural and post-logging landslides were also recorded for each polygon according to a coding system developed by the
Forest Sciences Section of the Vancouver Forest Region. Due to practical limitations to the use of aerial photography, landslides smaller than approximately 0.05 ha were excluded from the data set. Polygons with slopes of less than 20° (36%) were also excluded from the data set due to the relative rarity of landslides in these polygons.

To date, this terrain attribute database is composed of 2,698 individually mapped and field-checked polygons. The data were collected over a period of two field seasons by a group of approximately 6 experienced terrain specialists under contract or employment with the BC Ministry of Forests and under the supervision of Messieurs T. Rollerson (Vancouver Forest Region, Ministry of Forests, Nanaimo) and B. Thomson (Ministry of Environment, Lands and Parks). Details of the terrain attribute study, including the mapped attributes and coding, objectives, study areas, methodology, data collection techniques and proposed analyses are included in the form of the original research proposal as Appendix C: Terrain Attribute Study - D33.

7.3 Classification and Filtering of Terrain Attributes and Polygons

As a complement to Appendix C, Table 7.1 provides a summary of some 52 individual terrain attributes and the classification (one of six) to which they belong. In the field and during air photo interpretation, these attributes and the associated codes and notes for each polygon were recorded on a Ministry of Forests Terrain Data Card. A copy of this card is included at the end of Appendix C.

<table>
<thead>
<tr>
<th>MORPHOLOGY</th>
<th>SOIL</th>
<th>MATERIAL</th>
<th>STABILITY</th>
<th>SLOPES</th>
<th>ROADS</th>
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<td></td>
<td>Material type</td>
<td>Natural failures</td>
<td>Minimum slope</td>
<td>Road number</td>
</tr>
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<td></td>
<td>• colluvial</td>
<td>• number</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>• weathered bdrk</td>
<td></td>
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<td>• colian</td>
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</tr>
<tr>
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<td>• lacustrine</td>
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<td>Polygon location</td>
<td>Soil texture</td>
<td>Texture</td>
<td>Clearcut failures</td>
<td>Maximum slope</td>
<td>Construction method</td>
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<td>• blocks</td>
<td>• number</td>
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<td></td>
</tr>
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<td>• pebbles</td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>• Gordon</td>
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<td>• sand</td>
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<tr>
<td>• Nootka</td>
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<td>• silt</td>
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<tr>
<td>Slope Position</td>
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<td>Average depth</td>
<td>Cut block failures</td>
<td>Average slope</td>
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<td>• escarpment</td>
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<td></td>
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<tr>
<td>• headwater</td>
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</tr>
<tr>
<td>Hillslope configuration</td>
<td>Soil percentage</td>
<td>Material percentage</td>
<td>Road cut failures</td>
<td>Aspect</td>
<td>Revegetation</td>
</tr>
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<td></td>
<td>• number</td>
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<td></td>
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<tr>
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<td></td>
<td></td>
<td>no</td>
</tr>
<tr>
<td>• dissected</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>partial</td>
</tr>
<tr>
<td>• faceted</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• irregular</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• single gully</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hillslope curvature</td>
<td></td>
<td>Road fill failures</td>
<td>Elevation</td>
<td>Ditches</td>
<td></td>
</tr>
<tr>
<td>• concave</td>
<td></td>
<td>• number</td>
<td></td>
<td></td>
<td>yes</td>
</tr>
<tr>
<td>• convex</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>no</td>
</tr>
<tr>
<td>• straight</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>partial</td>
</tr>
<tr>
<td>• complex</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
As seen from Table 7.1, of the 2,698 polygons which were mapped and field-checked according to these attributes, some polygons would be inappropriate for the application of the infinite slope model due to the presence of gullies, the influence of roads, predominant material types or the type of documented failure. With one of the objectives of this research being to identify an improved terrain stability assessment technique which involves both objective (quantitative) stability modelling and less objective (qualitative) terrain attribute mapping, it is natural to limit, or filter, this database to polygons which are well-suited to these two techniques.
The need for such filtering is further discussed as follows. Although a good statistical and qualitative segregation of failed and stable polygons could be based purely on the number, slope, channel gradient and depth of gullies within the polygon, it would be difficult, if not impossible, to conduct a reliable stability analysis for a particular polygon given our lack of knowledge of any landslide location within the gully or open slope systems and the resulting appropriate slope angles, soil depths and groundwater regimes for use in the analysis. For example, a polygon might be described in the database as follows: uniform hillslope configuration, 60% colluvial, 40% morainal material types with two gullies and one reported clearcut failure. For the purposes of a stability analysis it is unknown if the reported failure is associated with one of the gully side walls (with appropriate slope angles of approximately 35° (70%)), gully headwalls (with different hydrogeological conditions) or within the open slope portion of the polygon. In the absence of a specific failure location, this polygon must be excluded from the subset of polygons which are used to develop the improved stability assessment technique using a translational failure model. Another polygon filtering criterion may be the reported material type for the polygon. Because the large scale, in-situ direct shear tests (see Chapter 4) were performed only within morainal and colluvial veneers and blankets over bedrock, it would be inappropriate to apply friction angles deduced from this testing program to failed and stable polygons reported to be composed of, say, glaciomarine or eolian material types. Furthermore, in the case of polygons reported to be a single gully, it is difficult to reliably predict groundwater ratios, $D_{w}/D$, appropriate slope angles and overburden depths for use in the quantitative, or stability assessment, component of the improved assessment technique.

Following these examples, a very conservative and strict set of criteria for inclusion of any of these 2,698 polygons in the subset of data for model development was adopted. Although this approach limits the applicability of the resulting model (unless it is used as an index tool only) to polygons and regions which are similar to those described below, these criteria also ensure the use of appropriate
geotechnical parameters as input to the quantitative infinite slope model, and confidence that any reported failures are indeed related to the attributes which are being used to describe and differentiate between the failed and stable polygons.

In summary, the original 2,698 polygons available for this model development were reduced to 1,526 according to the following criteria:

- only polygons with reported material types of 1 (colluvial) or 8 (morainal), with or without some exposed bedrock were included in the model development data subset, and
- all polygons with type 6 (single gully) hillslope configurations (see Table 7.1 and Appendix C) were excluded from the data subset, and
- only those polygons which had complete data fields (overburden thickness and slope angle) for use in the quantitative stability analyses were included, and
- only those polygon records with reasonable slope angle entries were included (e.g. only those polygons with reported average slope angles between the reported minimum and maximum slope angles were included), and
- only those polygons which were reported to contain no (zero) gullies were included.

To counter initial concerns that these criteria will effectively narrow the applicability of the proposed model to a select and relatively rare class of terrain polygon it is noted that of the 13 potential material types (i.e. colluvial, eolian, glaciolacustrine, marine, etc.) which could be coded on the Terrain Data Card (Table 7.1, Appendix C), only 5 are used: colluvial, fluvioglacial, morainal, bedrock and marine. The majority of those polygons which are filtered from the original 2,698 are removed due to the recorded presence of gullies within the polygon. It is due only to a slight ambiguity in the mapping technique which prevents the inclusion of these polygons in the data set. Unless a reported failure is coded specifically as a 'minor gully wall failure', the interpreter is unsure whether that failure occurred within the typically steeper gully area, which should be reflected in the quantitative
stability analysis as a different set of input parameters, or from the open slope area. Although this problem could be alleviated by re-locating each of those filtered polygons on aerial photography in order to determine if a reported failure originated from a gully side or head wall or from an open slope, the need to include these polygons is diminished by the fact that a more than statistically adequate number of polygons (1,526) remain for model development.

### 7.4 Qualitative Model Development

A geostatistical (qualitative) terrain attribute-based model has been developed for unification with a physical (quantitative) stability analysis model. The qualitative model component was developed by combining categorical and descriptive terrain attribute data, in terms of partial factors, into a ‘Terrain Attribute Factor’ which has been optimized to differentiate between the group of polygons exhibiting no instability and those exhibiting at least one open slope failure.

The remaining subset of 1,526 filtered polygons were split into two groups: those polygons which have remained stable following harvesting and those polygons which have experienced at least one open slope type failure since harvesting. In order to ensure that application of the infinite slope model is appropriate, those polygons which have experienced only road cut, road fill or minor gully wall failures were excluded from the unstable open slope population and are considered to be members of the stable group. In other words, this stability assessment tool is applicable to predictions of open slope type failures, not road related failures. A flow chart representing the filtering and grouping of polygons into stable and unstable groups is presented in Fig. 7.1. The remaining steps of the flow chart, concerning the ranking, combination and optimization of these partial factors to create a Terrain Attribute Factor, F, are discussed in the following sections.
As described in Section 7.1, reasonably successful attempts to cluster coded terrain attributes into groups which, together, act as good predictors of slope stability have been made by others. In this study, terrain attributes are kept separate, and a partial factor, \( f \), is assigned to each statistically significant attribute code (potentially more than 160 different partial factors, see Table 7.1 and Appendix C). For example, bedrock lithology is one terrain attribute which is part of the 'morphology' class of attributes (Table 7.1). Within the bedrock lithology terrain attribute there exist 9 different categorical codes (granite (code 02), granodiorite (code 04), diorite (code 06), volcanic breccia (code 19), etc.); one of which could be used by the terrain mapper to describe the bedrock lithology of that polygon. If it is shown (statistically) that there is a significantly higher proportion of unstable polygons with volcanic breccia bedrock lithology than stable polygons with the same lithology then a partial factor, \( f \), which is less than zero could be assigned to volcanic breccia. In contrast, if it is shown from the database that polygons of andesitic lithology have a significantly lower likelihood of experiencing a failure then a partial factor, \( f \), which is greater than zero could be assigned to andesite.

Partial factors are determined for each terrain attribute code and combined in a function which best distinguishes between the groups of unstable and stable polygons. This function of partial factors constitutes a unique Terrain Attribute Factor, \( F \), for each terrain polygon. The sign and magnitude of each partial factor, \( f \), and the function in which these partial factors are combined to create the Terrain Attribute Factor, \( F \), are manipulated iteratively until the statistical difference (i.e. the ratio the of difference of means to the standard error of difference and results of the chi-squared test) between the groups of unstable and stable polygons is maximized. Selection of the terrain attributes to be included in the function to calculate ‘\( F \)’ is determined by means of a discriminant analysis of the terrain attributes.
7.4.1 Determination of Partial Factors, f

Of the more than 50 terrain attributes for which information is collected on the Ministry of Forests Terrain Data Card, only some would be considered qualitative and descriptive of the terrain polygon to be studied. Several of the attributes are quantitative, such as minimum, average and maximum slope angles, material depths and road gradient while others are of a forensic nature, such as the observed number of clearcut failures and the number of road fill failures. Since observations of landslide occurrences are of limited use for the purpose of development of the proposed model, all attributes describing the number and types of failures are not considered while formulating the partial and terrain attribute factors, f and F, respectively. With the two distinct components of this model being a qualitative terrain attribute study method and a quantitative stability assessment which, together, are combined on orthogonal axes to create a unified predictive model, it is desirable to maintain independence and to avoid duplication of variables between the qualitative and quantitative axes (see Fig. 7.2). For example, because the minimum, average and maximum measured slope angles of a polygon are used as input to the quantitative stability assessment (see Equation 6.1), these terrain attributes are purposefully excluded from those which could be included in a proposed Terrain Attribute Factor, F.

Ten terrain attributes of the more than 50 which are presented in Table 7.1 were selected as potential candidates for inclusion in the Terrain Attribute Factor, F. These ten attributes for further study are:

- Terrain Unit
- Hillslope Configuration
- Slope Aspect
- Bedrock Formation
- Bedrock Structure
- Slope Position
- Hillslope Curvature
- Polygon Elevation
- Bedrock Lithology
- Drainage Class

Each of these attributes has a set of codes which may be used by the mapper to describe a given polygon. If the filtered polygons are split (Fig. 7.1) into groups of unstable and stable polygons it is
possible to count the number of occurrences observed for the code of each terrain attribute in the two groups. These observed frequencies can then be compared to frequencies which would be expected if there was no relationship between the occurrence of failures and a terrain attribute. It is from this comparison between observed and expected (if no relationship were to exist) occurrences that partial factors, $f$, may be determined for the code of each included terrain attribute. The following example for bedrock lithology (Table 7.2) clarifies this procedure:

Table 7.2 Development of partial factors, $f$: Bedrock Lithology example

<table>
<thead>
<tr>
<th>Bedrock Lithology</th>
<th>Attribute Code</th>
<th>Code Descriptor</th>
<th>Number of Stable Polygons (Expected)</th>
<th>Number of Unstable Polygons (Expected)</th>
<th>Number of Stable Polygons (Observed)</th>
<th>Number of Unstable Polygons (Observed)</th>
<th>Initial Partial Factor, $f$</th>
<th>Initial Rank</th>
<th>Cumulative Chi-squared Statistic</th>
<th>Initial Ratio of Diff. of Means to Std. Error</th>
<th>Adjusted Partial Factor, $f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td>F</td>
<td>G</td>
<td>H</td>
<td>I</td>
<td>J</td>
<td>K</td>
<td>J</td>
</tr>
<tr>
<td>22</td>
<td>Phyllite</td>
<td>25.68</td>
<td>6.32</td>
<td>19</td>
<td>13</td>
<td>-1.32</td>
<td>1</td>
<td>-</td>
<td>-1.32</td>
<td>-</td>
<td>-1.32</td>
</tr>
<tr>
<td>19</td>
<td>Volcanic Breccia</td>
<td>12.04</td>
<td>2.96</td>
<td>9</td>
<td>6</td>
<td>-1.28</td>
<td>2</td>
<td>0.00037</td>
<td>2.802</td>
<td>-1.28</td>
<td></td>
</tr>
<tr>
<td>58</td>
<td>Limestone</td>
<td>12.04</td>
<td>2.96</td>
<td>14</td>
<td>1</td>
<td>0.83</td>
<td>3</td>
<td>0.00078</td>
<td>3.086</td>
<td>0.83</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>Andesite</td>
<td>66.61</td>
<td>16.39</td>
<td>77</td>
<td>6</td>
<td>0.79</td>
<td>4</td>
<td>0.00005</td>
<td>4.377</td>
<td>0.79</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Quartz Diorite</td>
<td>11.24</td>
<td>2.76</td>
<td>10</td>
<td>4</td>
<td>-0.56</td>
<td>5</td>
<td>0.00012</td>
<td>4.459</td>
<td>-0.56</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>Rhyolite</td>
<td>32.10</td>
<td>7.90</td>
<td>35</td>
<td>5</td>
<td>0.46</td>
<td>6</td>
<td>0.00017</td>
<td>4.611</td>
<td>0.46</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Granite</td>
<td>187.00</td>
<td>46.00</td>
<td>176</td>
<td>57</td>
<td>-0.30</td>
<td>7</td>
<td>0.00010</td>
<td>5.162</td>
<td>-0.30</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>Basalt</td>
<td>195.82</td>
<td>48.18</td>
<td>202</td>
<td>42</td>
<td>0.16</td>
<td>8</td>
<td>0.00016</td>
<td>5.138</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Diorite</td>
<td>114.77</td>
<td>28.23</td>
<td>116</td>
<td>27</td>
<td>0.05</td>
<td>9</td>
<td>0.00034</td>
<td>5.122</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Granodiorite</td>
<td>80.26</td>
<td>19.74</td>
<td>81</td>
<td>19</td>
<td>0.05</td>
<td>10</td>
<td>0.00068</td>
<td>5.113</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Quartz Monzonite</td>
<td>1.61</td>
<td>0.39</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>11</td>
<td>0.00084</td>
<td>5.113</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>53</td>
<td>Greywacke</td>
<td>4.82</td>
<td>1.18</td>
<td>4</td>
<td>2</td>
<td>0</td>
<td>12</td>
<td>0.00121</td>
<td>5.113</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>38</td>
<td>Quartzite</td>
<td>7.22</td>
<td>1.78</td>
<td>8</td>
<td>1</td>
<td>0</td>
<td>13</td>
<td>0.00186</td>
<td>5.113</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Syenite</td>
<td>0.80</td>
<td>0.20</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>14</td>
<td>0.00078</td>
<td>5.113</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

In this example the ‘expected stable’ entry for code 22 (phyllite, column C) is calculated by multiplying the number of observed phyllite polygons (19 + 13) by the ratio of the number of stable polygons to the total number of polygons (1,225/1,526). The remaining ‘expected’ entries are calculated following a similar pattern and represent the number of observed stable and unstable polygons which would be expected for each attribute code if the given attribute code had no effect on likelihood of failure. The initial partial factor is a measure of how far the observed frequency of
Chapter 7. Terrain Attribute Study

stable and unstable polygons is from the expected frequency for each terrain attribute code. In order to ensure that partial factors are significant, to be assigned a non-zero initial partial factor, an attribute code must have a number of observations (stable or unstable) which is greater than an arbitrarily set lower limit of 1% of the total number of coded polygons. In this example, because there are only 6 observed polygons with the greywacke bedrock lithology attribute code (code 53), which is less than 1% of the total number of polygons for which bedrock lithology has been recorded (942), a partial factor, \( f \), of zero is assigned to the greywacke attribute code. In the case of the attribute codes for which there are sufficient stable or unstable observations, the initial partial factor, \( f \), is calculated according to the following relationship:

\[
 f = \frac{E-C}{C} + \frac{D-F}{D}
\]  

(7.1)

The letters in Equation 7.1 refer to the column letters of Table 7.2. A calculated partial factor, \( f \), of zero would indicate that a given code of this terrain attribute makes no contribution to determining whether a polygon belongs to the stable or unstable group. A calculated value which is less than zero indicates an increased likelihood that a polygon with this attribute code belongs to the unstable group of polygons. The opposite holds for a partial factor, \( f \), calculated to be greater than zero.

Two general statistical tests are used throughout this research to test the goodness-of-fit between actual and fitted data and to test for association (or lack of) between two sets of data. The 'chi-squared' test is well-suited to categorical data, such as those presented in Table 7.2. For example, the chi-squared test is well-suited to testing if the observed groups of stable and unstable polygons are significantly (statistically) different from the equivalent expected groups. The chi-squared test is typically used to compare a single pair of expected and observed data at a predetermined level of significance. An alternative, used in this analysis, is to consider the chi-squared test in the form of a contingency table
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(Table 7.2). Discussion of this method is beyond the scope of this thesis but is well-presented by Spiegel (1961), Benjamin and Cornell (1970), Ang and Tang (1975) and St. John and Richardson (1990). The chi-squared statistics reported in Table 7.2 approximate the likelihood that any difference between the observed and expected frequencies of attribute codes is due to chance alone. Therefore, a chi-squared significance test value signifies a high degree of confidence in a proposed relationship between attribute codes rather than a chance occurrence. For example, if the result of a chi-squared test between the observed and expected groups is greater than a calculated critical level (typically corresponding to the 0.01 or 0.05 significance levels), it can be said that there is less than a 1% or 5% chance that the observed data and expected data vary due to chance alone.

A second statistical test is the simple ‘comparison of two means’ test, which is well-suited to approximately normally distributed interval data of sample sizes greater than 30. If, for example, the difference between the means of two samples is more than twice the calculated standard error of the difference, it is generally accepted that the two samples are from different populations. The comparison of two means test is conducted by calculating the standard error of difference and the difference of means as follows:

$$\delta_{\text{Error}} = \sqrt{\frac{\sigma_1^2}{n_1} + \frac{\sigma_2^2}{n_2}}$$  \hspace{1cm} (7.2)

and,

$$\delta_{\text{Means}} = \mu_1 - \mu_2$$  \hspace{1cm} (7.3)

where $\mu_1$, $\sigma_1$ and $n_1$ are the mean, standard deviation and sample size of the first set of data being compared, respectively, and $\mu_2$, $\sigma_2$ and $n_2$ are the mean, standard deviation and sample size of the second set of data being compared. Whereas the chi-square test is best-suited to categorical data, this
comparison of two means procedure is best-suited to approximately normally distributed interval data. In this manner, each statistical test is a good complement to the other.

There remain three steps to the determination of the statistically significant partial factors for combination and use in the Terrain Attribute Factor, F. First, the tables of observed and expected occurrences and initial partial factors are created for each of the remaining 9 terrain attributes, in a similar form to Table 7.2. Each attribute code (for each terrain attribute) is given an initial rank, based on the magnitude (absolute value) of the calculated initial partial factors. The magnitude of an initial partial factor provides a preliminary indication of the predictive ability of that attribute code.

Second, a chi-squared test of significance, using the contingency table method, is performed on each of the sets of observed and expected data for each of the 10 terrain attributes in order to determine the likelihood that any observed differences between the observed and expected frequencies are due to chance alone. Using bedrock lithology as an example (Table 7.2), the chi-squared statistic for the top three ranked codes (phyllite, volcanic breccia and limestone) is calculated to be 0.00078, indicating only a 0.078% likelihood that the observed and expected frequencies are unrelated to the phyllite, volcanic breccia and limestone codes.

As the third (and last) step to the determination of statistically significant partial factors for combination and use in the Terrain Attribute Factor, F, each of the ranked attribute codes (for each of the 10 terrain attributes) is sequentially added to the list of attribute codes which will be used to characterize the terrain attribute. This sequential adding of attribute codes is a form of discriminant analysis. The difference of means test is used to test if the addition of the next, lower ranked, attribute code helps to distinguish between the groups of stable and unstable polygons or, rather, 'clouds' the distinction between these two groups. Each subsequent attribute code is added to the list of significant codes until the difference of means test results in a lower ratio, indicating a less
significant difference between the two groups of polygons. Again, using bedrock lithology as an example (Table 7.2), if all stable and unstable polygons with bedrock lithology codes of 22 and 19 (phyllite and volcanic breccia) are assigned values of -1.32 and -1.28 respectively and all other polygons (with different lithology codes) are assigned a value of zero, the difference in means of these assigned values between the stable and unstable groups may be compared to the standard error of the difference. This ratio, currently with only phyllite and volcanic breccia included on the list of significant attribute codes, is provided in column ‘J’ of Table 7.2: 2.802. This ratio is observed to increase with each subsequent addition of an attribute code until the basalt code (17) and its initial partial factor (0.16) is added to the list and the difference of means test is observed to decrease from a maximum of 5.162 to a lower value of 5.138. The list of initial partial factors is then adjusted, as reported in column ‘K’ of Table 7.2, to assign the non-zero values to those attribute codes (phyllite, volcanic breccia, limestone, andesite, quartz diorite, rhyolite and granite) which improve the distinction between the groups of stable and unstable polygons; all other codes, which neither increase nor decrease predicted likelihoods of instability, are assigned a value of zero. These steps to determine partial factors for inclusion in a Terrain Attribute Factor, F, are also illustrated in Fig. 7.1. The results of the same procedure, conducted for the other 9 terrain attributes, are provided in Appendix D.

The maximum difference of means test results for each of the 10 terrain attributes are then used to rank the terrain attributes. The results of the above three steps to determine partial factors and the ranking of the attributes are summarized in Table 7.3.
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Table 7.3 Summary of studied terrain attributes and Partial Factors, \( f \)

<table>
<thead>
<tr>
<th>Terrain Attribute</th>
<th>Number of Codes Used</th>
<th>Percentage of Mapped (and Filtered) Polygons for Which Attribute is Coded</th>
<th>Initial Number of Significant Codes</th>
<th>Chi-squared Significance Statistic</th>
<th>Initial Ratio of Diff. of Means to Std. Error</th>
<th>Rank</th>
<th>Final Number of Significant Codes (used in Terrain Attribute Function, ( F ))</th>
<th>Optimized Ratio of Diff. of Means to Std. Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td>F</td>
<td>G</td>
<td>H</td>
<td>I</td>
</tr>
<tr>
<td>Terrain Unit</td>
<td>17</td>
<td>100%</td>
<td>10</td>
<td>1.51e-4</td>
<td>5.467</td>
<td>1</td>
<td>10</td>
<td>5.563</td>
</tr>
<tr>
<td>Bedrock Lithology</td>
<td>14</td>
<td>61%</td>
<td>10</td>
<td>1.02e-4</td>
<td>5.162</td>
<td>2</td>
<td>7</td>
<td>5.381</td>
</tr>
<tr>
<td>Hillslope Configuration</td>
<td>5</td>
<td>100%</td>
<td>4</td>
<td>3.88e-3</td>
<td>3.884</td>
<td>3</td>
<td>4</td>
<td>3.940</td>
</tr>
<tr>
<td>Hillslope Curvature</td>
<td>4</td>
<td>99%</td>
<td>4</td>
<td>1.37e-3</td>
<td>3.423</td>
<td>4</td>
<td>3</td>
<td>3.430</td>
</tr>
<tr>
<td>Slope Position</td>
<td>6</td>
<td>99%</td>
<td>4</td>
<td>1.27e-1</td>
<td>2.705</td>
<td>5</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>Slope Aspect</td>
<td>8</td>
<td>99%</td>
<td>8</td>
<td>5.56e-3</td>
<td>2.526</td>
<td>6</td>
<td>8</td>
<td>-</td>
</tr>
<tr>
<td>Drainage Class</td>
<td>4</td>
<td>100%</td>
<td>3</td>
<td>1.36e-1</td>
<td>1.877</td>
<td>7</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>Bedrock Structure</td>
<td>4</td>
<td>39%</td>
<td>3</td>
<td>2.91e-1</td>
<td>1.429</td>
<td>8</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>Bedrock Formation</td>
<td>6</td>
<td>80%</td>
<td>5</td>
<td>2.62e-3</td>
<td>5.095</td>
<td>9</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>Polygon Elevation</td>
<td>11</td>
<td>99%</td>
<td>9</td>
<td>3.55e-3</td>
<td>5.062</td>
<td>10</td>
<td>8</td>
<td>-</td>
</tr>
</tbody>
</table>

Column ‘I’, of Table 7.3 is discussed in the following section. A complete listing of initial partial factors for all attribute codes of the 10 terrain attributes, prior to combination into a Terrain Attribute Factor, \( F \), is also provided in Appendix D. The relatively low ranking of the drainage class terrain attribute within the list of potential attributes given in Table 7.3, and its insignificant difference of means and chi-squared test results (1.877 < 2.0 and 13.6% respectively) are consistent with observations made by Fannin and Rollerson (1993) regarding the poor predictive ability of drainage class as an indicator of potential instability.

Despite the relatively high difference of means test results for the bedrock formation and polygon elevation terrain attributes (5.10 and 5.06 respectively), these two attributes have been ranked ninth and tenth for the following reasons:

- The coefficient of correlation between partial factors of the bedrock formation and bedrock lithology terrain attributes for the group of stable polygons is 0.68 and 0.45 for the group of unstable polygons. These relatively high coefficients indicate a lack of independence between these two terrain attributes and, consequently, the need for only one to be included. Bedrock
lithology has been chosen over bedrock formation as the preferred terrain attribute due to the superior chi-squared and difference of means test results for bedrock lithology (Table 7.3) and due to the more general (less regional) nature of the bedrock lithology attribute codes in comparison with those of bedrock formation.

- Polygon elevation is believed to be overly site-specific or ‘local’ to the coastal Vancouver Island mapping sites. As a result, inclusion of polygon elevation would inappropriately bias results of mapping efforts in other areas. Also, polygon elevation is not independent of orographic effects such as the likelihood of frost or differing precipitation intensities and quantities at different elevations, both of which may be more appropriate indicators of potential instability and may be included in a similar stability assessment model in the future.

### 7.4.2 Development of Terrain Attribute Factor, F

In order to develop a function which will allow for the calculation of a Terrain Attribute Factor, F, for each polygon, based on partial factors from the previous section, a simplified form of a discriminant analysis is adopted. The discriminant analysis was performed, using Microsoft Excel ver 5.0 spreadsheet software, by starting with the initial partial factors for the highest, or best, ranked terrain attribute from Table 7.3 and adding the second best terrain attribute. The partial factors for the ‘terrain unit’ terrain attribute, followed by ‘bedrock lithology’ then ‘hillslope configuration’ were first added to the following function:

\[
F = f_1 + f_2 + \ldots + f_i
\]  

(7.4)

In this equation, \(f_i\) through \(f_1\) represent the initial partial factors determined in Section 7.4.1, where ‘i’ represents the rank of the last terrain attribute added to the function during this discriminant analysis. In this case, \(f_i\) will be the corresponding optimized partial factor for the Terrain Unit attribute of each polygon, while \(f_2\) will be the corresponding optimized partial factor for the Bedrock Lithology attribute.
of each polygon. Following each addition of a terrain attribute to the function, the mean and standard deviation of the stable and unstable groups of polygons were calculated in order to determine the ratio of the difference of means to the standard error. This process is continued until the difference of means test result (or the result of another statistical test) for the function is maximized or until the incremental change becomes insignificant.

This initial function (Equation 7.4) defining the Terrain Attribute Factor, F, was then optimized by sequentially and iteratively multiplying each partial factor in the function by a constant coefficient and raising the partial factor to an exponent. In this manner, a pair of coefficients and exponents was determined for each terrain attribute code in the function. Each attribute code of a given terrain attribute was multiplied by the same coefficient and raised to the same exponent. In summary, a function of the following form was optimized for its ability to distinguish between the groups of stable and unstable polygons by iteratively adjusting coefficients $C_1$ to $C_i$ and exponents $n_1$ to $n_i$, which were added to Equation 7.4 as follows:

$$F = C_1 f_1^{n_1} + C_2 f_2^{n_2} + \ldots + C_i f_i^{n_i}$$

(7.5)

Table 7.4 lists the calculated difference of means test results from this discriminant analysis. The ratio of the difference of means to the standard error (column 'C') continues to increase as each of the 10 terrain attributes are added to Equation 7.5. The incremental benefit of adding further terms to Equation 7.5 is, however, very low after the fourth term, hillslope curvature, has been included. Supporting this observation that little is gained from adding a fifth terrain attribute to the equation after terrain unit, bedrock lithology, hillslope configuration and hillslope curvature, is the observation that chi-squared statistics for the remainder of the terrain attributes (polygon elevation and bedrock formation excepted as discussed above) are very high (poor) in comparison with the same statistics for the first four terms in the equation. The functions defined in Equations 7.4 and 7.5 should, therefore,
be limited to four terms: terrain unit, bedrock lithology, hillslope configuration and hillslope curvature.

Table 7.4  Terrain Attribute Factor, F, discriminant analysis results

<table>
<thead>
<tr>
<th>Terrain Attribute</th>
<th>Rank</th>
<th>Optimized Ratio of Diff. of Means to Std. Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Terrain Unit</td>
<td>1</td>
<td>5.58</td>
</tr>
<tr>
<td>Bedrock Lithology</td>
<td>2</td>
<td>8.85</td>
</tr>
<tr>
<td>Hillslope Configuration</td>
<td>3</td>
<td>9.68</td>
</tr>
<tr>
<td>Hillslope Curvature</td>
<td>4</td>
<td>10.27</td>
</tr>
<tr>
<td>Slope Position</td>
<td>5</td>
<td>10.31</td>
</tr>
<tr>
<td>Slope Aspect</td>
<td>6</td>
<td>10.33</td>
</tr>
<tr>
<td>Drainage Class</td>
<td>7</td>
<td>10.36</td>
</tr>
<tr>
<td>Bedrock Structure</td>
<td>8</td>
<td>10.42</td>
</tr>
<tr>
<td>Bedrock Formation</td>
<td>9</td>
<td>10.57</td>
</tr>
<tr>
<td>Polygon Elevation</td>
<td>10</td>
<td>10.76</td>
</tr>
</tbody>
</table>

Coefficients $C_1$ to $C_{10}$ and exponents $n_1$ to $n_{10}$ were determined iteratively (5,000 iterations) and are also provided for each terrain attribute in Appendix D. Because only one coefficient and one exponent was determined for each terrain attribute, the initial partial factor of each of the four included terrain attributes was multiplied by coefficient $C_i$ and raised to exponent $n_i$ in order to determine a set of 'optimized' partial factors from the initial partial factors. These optimized partial factors are listed for each of the included terrain attributes in Table 7.5:
Table 7.5  List of terrain attribute ranking and partial factors

<table>
<thead>
<tr>
<th>Terrain Attribute</th>
<th>Rank</th>
<th>Attribute Code</th>
<th>Code Descriptor</th>
<th>Optimized Partial Factor, f</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terrain Unit</td>
<td>1</td>
<td>1</td>
<td>Deep, pure Mb, Mh</td>
<td>-1.28</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2</td>
<td>Deep/Shallow, pure Mb / or // Mv</td>
<td>8.37</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>3</td>
<td>Shallow, pure Mv, Mv / or // Mb, Mbv</td>
<td>-0.45</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>4</td>
<td>Deep, pure Cb, Ca, Cf</td>
<td>3.84</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>6</td>
<td>Shallow, pure Cv, Cv/Cb or //Cb, Cbv</td>
<td>-2.19</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>10</td>
<td>Deep, mixture of MbCb, CbMb, / or // or stratigraphic combinations</td>
<td>3.99</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>11</td>
<td>Deep/Shallow, mixtures of Mb / or // Cbv or Cv, Cb / or // Mbv or Mv or stratigraphic combinations, minor FGbv or Fbv</td>
<td>-2.09</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>12</td>
<td>Shallow, mixtures of MvCv, CvMv, / or // combinations, Mbv / or // Cbv or Cv, Cbv / or // Mbv or Mv, minor FGv or Fv</td>
<td>-1.16</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>15</td>
<td>Shallow, mixtures of Mv / or // R, Mbv / or // R, MvCv / or // R</td>
<td>5.72</td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>17</td>
<td>Shallow, mixtures of Cv / or // R, Cbv // R</td>
<td>-8.23</td>
</tr>
<tr>
<td>Bedrock Lithology</td>
<td>2</td>
<td>2</td>
<td>Granite</td>
<td>-4.50</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>5</td>
<td>Quartz Diorite</td>
<td>-6.73</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>13</td>
<td>Rhyolite</td>
<td>5.93</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>15</td>
<td>Andesite</td>
<td>8.44</td>
</tr>
<tr>
<td></td>
<td>19</td>
<td>19</td>
<td>Volcanic Breccia</td>
<td>-11.51</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>22</td>
<td>Phyllite</td>
<td>-11.73</td>
</tr>
<tr>
<td></td>
<td>58</td>
<td>58</td>
<td>Limestone</td>
<td>8.68</td>
</tr>
<tr>
<td>Hillslope Configuration</td>
<td>4</td>
<td>1</td>
<td>Uniform</td>
<td>-1.55</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2</td>
<td>Benched</td>
<td>4.77</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>3</td>
<td>Dissected</td>
<td>-6.96</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>5</td>
<td>Irregular</td>
<td>5.35</td>
</tr>
<tr>
<td>Hillslope Curvature</td>
<td>5</td>
<td>1</td>
<td>Concave</td>
<td>-4.66</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2</td>
<td>Convex</td>
<td>2.42</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>4</td>
<td>Complex</td>
<td>-2.53</td>
</tr>
</tbody>
</table>

The optimized partial factors of Table 7.5 have been applied to each of the 1,526 mapped polygons, resulting in a Terrain Attribute Factor, F, for each polygon. The average Terrain Attribute Factor, over all of these polygons was calculated to be -1.285. Because insignificant partial factors and missing (uncoded) terrain attributes have been assigned a value of zero, the average Terrain Attribute Factor, over all mapped polygons, should also be shifted to have a value of zero in order to ensure that a statistically insignificant attribute code, such as granodiorite (see Table 7.2), for example, does not bias a calculated Terrain Attribute Factor excessively towards the stable (positive) or unstable (negative) groups of polygons. This shift is accomplished through the inclusion of a constant term to
the above equations, resulting in a final, optimized function defining the Terrain Attribute Factor as follows:

\[ F = f_{\text{TerrainUnit}} + f_{\text{BedrockLithology}} + f_{\text{HillslopeConfiguration}} + f_{\text{HillslopeCurvature}} + 1.285 \]  

(7.6)

If the Terrain Attribute Factor is calculated for all 1,526 mapped polygons using Equation 7.6, the median value is observed to be approximately 2.55. If no information exists regarding any of the terrain attributes included in Equation 7.6, the Terrain Attribute Factor would be correctly calculated to be 1.285. This is in good agreement with the observation that approximately 79% of the stable polygons are observed to have Terrain Attribute Factors which are greater than 1.285, while 93% of the unstable polygons were scored below 1.285. Additionally, considering the combined group of stable and unstable polygons, approximately 18% of all polygons scored below 1.285, which is in excellent agreement with the prior knowledge that 301 (19.7%) of the 1,526 mapped polygons experienced at least one significant open slope failure. In other words, without any terrain attribute information for a polygon, all that may be said regarding the likelihood of open slope failures within the polygon is that approximately 18% of all mapped polygons used in this study experienced open slope failures, which would be reflected by a Terrain Attribute Factor of 1.285.

Where mapping of a polygon results in a terrain attribute for which an attribute code is not found in Table 7.5, a value of zero is assigned to the partial factor of that attribute. The ratio of difference of means to the standard error, using the optimized partial factors of Table 7.5, are provided in column ‘I’ of Table 7.3 in order to ensure that non-zero partial factors are indeed statistically significant indicators of polygon stability.


Chapter 7. Terrain Attribute Study

7.5 Terrain Attribute Study: Discussion

The proposed probabilistic stability assessment model is illustrated in Fig. 7.1. Chapter 7 has described the process by which the qualitative component this model has been developed and tested. The average and standard deviation of the calculated (Equation 7.6) Terrain Attribute Factors for the 1,226 mapped stable polygons and 301 mapped unstable polygons are provided in Table 7.6.

Table 7.6 Summary of Terrain Attribute Factor, F, statistics for stable and unstable polygons

<table>
<thead>
<tr>
<th>Summary Statistics</th>
<th>Stable Polygons (1.225)</th>
<th>Unstable Polygons (301)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>0.73</td>
<td>-2.98</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>6.67</td>
<td>5.95</td>
</tr>
</tbody>
</table>

The formula used to calculate the Terrain Attribute Factor, F, and provided as Equation 7.6, is the result of statistical fitting and efforts to optimize the segregation of observed groups of stable and unstable polygons. It is reasonable to question how well this proposed function achieves the stated goal. The optimized difference of means test result for the above data is calculated to be 9.46, which is well in excess of the required value of 2.0 in order to claim a statistically significant difference between two groups. This value of 9.46 is slightly below the value of 10.27, expected from Table 7.4. The reason for this difference is that the selection and optimization of partial factors (Tables 7.1 to 7.5 and Appendix D) is based on estimates of population statistics, using the data found in Appendix D, whereas the values reported in Table 7.6 are true population means and standard deviations, calculated from 1,526 polygons for which actual Terrain Attribute Factors have been calculated. For the purpose of graphical comparison, the distributions of calculated Terrain Attribute Factors for each of the stable and unstable polygons are plotted in Fig. 7.3.

As the existing database of mapped polygons is augmented by future terrain attribute studies the occurrence of no statistically significant partial factor existing for a given attribute code will be reduced and the resulting segregation of stable and unstable polygons will concurrently improve. A
truly versatile form of the function defining this proposed Terrain Attribute Factor would be one in which the partial factors of Table 7.6 are continuously updated and augmented with the addition of new polygons to the database. This potential updating system is analogous to Bayesian updating in that a prior estimate is upgraded and improved by each new piece of information or observation.
Chapter 7. Terrain Attribute Study

Regions (7):
- Quatsino
- Gordon
- Cameron
- Clayoquot
- Kyuquot
- Klanawa
- Nootka

Filter Polygons (2,698 to 1,526)

Selected Categorical Terrain Attributes (10):
- Terrain Unit
- Hillslope configuration
- Slope Aspect
- Bedrock Formation
- Bedrock Structure
- Slope Position
- Hillslope Curvature
- Slope Elevation
- Bedrock Lithology
- Drainage Class

Table 7.1

Unstable Polygons (301)

Stable Polygons (1,225)

Initial Partial Factors, f (Eqn. 7.1)

Unstable Polygons (301)

Contingency Tables (10)
- (Chi-Squared Test)
- (Table 7.2 and Appendix D)

Calculate Ratio of Difference of Means to Standard Error
- (Eqns 7.4, 7.5)

Optimized Partial Factors (24)
- for Statistically Significant Attributes (4):
  - Terrain Unit
  - Bedrock Lithology
  - Hillslope Configuration
  - Hillslope Curvature
  - (Table 7.3)

Calculate Ratio of Difference of Means to Standard Error for Each Significant Terrain Attribute
- (Eqns 7.2, 7.3)
  - (Table 7.3)

Calculate Ratio of Difference of Means to Standard Error for Interim Terrain Attribute Function
- (Equations 7.2, 7.3, 7.5)
  - (Table 7.4)

Optimize Partial Factors and Terrain Attribute Factor, F
- (Table 7.5, Eqn. 7.6)

Calculate Ratio of Difference of Means to Standard Error for Optimized Function, F
- (Eqns 7.2, 7.3)
  - (Fig. 7.4)

Statistically Insignificant Attributes (6):
- Slope Aspect
- Bedrock Formation
- Bedrock Structure
- Slope Position
- Polygon Elevation
- Drainage Class

Optimized Partial Factors (24)
- for Statistically Significant Attributes (4):
  - Terrain Unit
  - Bedrock Lithology
  - Hillslope Configuration
  - Hillslope Curvature
  - (Table 7.3)

Build Terrain Attribute Function / Factor
- Using Discriminant Analysis

Iterate (4 times):
- Adjust Function Coefficients and Exponents
  - (Eqns. 7.6)

FIG. 7.1 Flow chart: development of Terrain Attribute Factor, F, from mapped polygons
Chapter 7. Terrain Attribute Study

Qualitative Rating
Terrain Attribute Factor, F (see Chpt. 7)
(from Terrain Attribute Study)

Probability of Initiation
P(ln)

Independent Axes

Quantitative Rating
Reliability Index (see Chpt. 8)
(from Probabilistic Slope Stability Assessment)

FIG. 7.2 Schematic of proposed terrain stability assessment model
FIG. 7.3  Distributions of Terrain Attribute Factor, F, for stable and unstable polygons
8. \textit{PROBABILISTIC MODELLING}

8.1 \textit{Probabilistic Approach: Benefits and Drawbacks}

As the role of resource managers, owners, consultants, contractors, government agencies and the public in the technical, financial, social and operational aspects of many projects increasingly overlap, the need to effectively communicate the variability and potential risks of each project also increases. Rarely can a professional offer advice based purely on experience and a personal disposition towards risk. The experience must be documented or confirmed by case histories and the risk must be quantified. The risk that an individual is willing to adopt often depends on the potential severity of a ‘failure’ and the individual’s willingness to balance risk against cost. In a similar manner, the forest industry is challenged by the task of assigning value to harvested timber (in terms of timber value, value-added products and employment), downslope resources and infrastructure (including other timber resources, fish habitats, transportation and service corridors, tourism and wildlife) and, less often, to human life and property. The severity and the potential for damage, combined with the cost of harvesting must be offset against the value of that timber. Although a formal risk assessment can often be a time consuming and laborious approach to problem solving, we all use varying levels of risk assessment to make and rationalize decisions which involve potentially negative outcomes.

A formal risk assessment varies from decision making in that each possible outcome is assigned (based on judgment or computation) a probability of occurrence, given certain conditions, and the cost of each outcome, including the initial costs and the potential costs or benefits, is considered. Simple decision making may involve an unstated pre-disposition to risk and a certain outcome, without quantifying the probability of occurrence of a given outcome or its severity.
Chapter 8. Probabilistic Modelling

This chapter presents the concept of Monte Carlo simulation as applied to the infinite slope equation. This simulation technique is used in many disciplines and is well-described for the infinite slope equation with application to forest hillslope stability by Hammond et al. (1992). From simulation, an output distribution is collected, from which the probability that the factor of safety is less than unity, \( P(\text{FS} < 1) \), and the reliability index are calculated. The potential to consider the variation of risk with time is discussed and the results of a simulation of the 1,526 polygons discussed in Chapter 7 are presented.

8.2 Concept of Probability of Failure and Reliability Index

While a single factor of safety for a given material strength, system loading and geometry is typically calculated, if it is accepted that a distribution of potential factors of safety, such as the examples in Fig. 8.1 a) and b), is available, it is then possible to calculate a probability that the factor of safety will be less than unity, based on the variability of the input parameters.

In Fig. 8.1 a), a normal distribution of calculated factors of safety (FS), with a mean of 1.2 and a standard deviation of 0.2 is shown. From the figure, it is seen that although the most likely factor of safety may have been calculated in a deterministic approach to be 1.2, a significant number of outcomes may be expected to fall below unity due to input uncertainty. The area beneath the distribution curve and to the left of a vertical line drawn through FS=1 represents the proportion of calculations which resulted in a factor of safety less than unity, or the probability that factor of safety will be less than unity. In the case of Fig. 8.1 a), this area is equal to 15.7% of the entire area.

Other methods of demonstrating the potential for an outcome to be less than unity include the calculation of a reliability index, \( \beta \). The reliability index, in its simplest form, is calculated from the
first and second moments of the calculated output. For the factor of safety case, reliability index, $\beta$, is given by:

$$\beta = \frac{\mu - 1}{\sigma} \quad \text{ (8.1a)}$$

where $\mu$ is the mean factor of safety of the given output and $\sigma$ is the standard deviation of the given output. In the case of Fig. 8.1 a), the mean and standard deviation of the output distribution are 1.2 and 0.2 respectively, and the reliability index is therefore equal to unity. If the reliability index is negative then the mean factor of safety is less than unity. Conversely, if the reliability index is positive, the mean factor of safety is greater than unity. As the standard deviation of the output distribution decreases, or, in other words, as the uncertainty in the output distribution decreases, the absolute value of the reliability index increases.

As shown for the example in Fig. 8.1b, the output distribution of factors of safety may not be normally distributed. In this case the probability of factor of safety being less than unity is a more exact measure of the potential for failure than the reliability index. In reality, however, a reliability index does provide a reasonable measure given only two statistics of the output distribution rather than the entire distribution.

As an alternative to Equation 8.1a, the reliability index may also be calculated using the probability of factor of safety being less than unity as:

$$\beta = -\Phi^{-1}(P_f) \quad \text{ (8.1b)}$$

where $\Phi^{-1}$ is the inverse of the standard normal distribution and $P_f$ is the probability of factor of safety being less than unity as determined from the output distribution. The reliability index has been
calculated for all 1,526 polygons using both approaches of Equations 8.1a and 8.1b and are observed
to give similar results. The mean and standard deviation of reliability indices calculated using
Equations 8.1a and 8.1b are $\mu = 0.359, \sigma = 1.43$ and $\mu = 0.396, \sigma = 1.49$, respectively.

A further point regarding the shortcomings of a deterministic analysis in comparison to a Monte Carlo
type simulation is the observation from Fig. 8.1 that although the factor of safety is 1.2 in both cases,
the probabilities of factor of safety being less than unity are 15.7% and 27% respectively, which
represents a significant difference. This classic example of misleading deterministic results is taken to
an extreme in Fig. 8.2 a), where the deterministic factors of safety of cases ‘A’ and ‘B’ are 1.2 and
1.45 respectively but the probabilities of factors of safety being less than unity are 9.1% and 14.8%
respectively (reliability indices 1.33 and 1.05). In both cases, conservative or ‘worst case’
deterministic analyses would have resulted in unacceptable factors of safety less than unity.

In order to better demonstrate the relationship between reliability index and probability of factor of
safety being less than unity, Fig. 8.2b shows their unique relationship for any normally distributed
variable with any mean and standard deviation. In other words, for a normally distributed
distribution, such as those of Fig. 8.2a, reliability index is an adequate and more simply calculated
method of measuring potential for failure. In fact, as seen from Fig. 8.2b, as probability of factor of
safety being less than unity approaches zero or unity, the absolute value of reliability index increases
greatly. It is often difficult to discern the significance of a small shift in probability of factor of safety
being less than unity, such as a shift from 0.0001 to 0.001, even though such a shift represents a
tenfold increase in risk or hazard. This same shift in terms of reliability index would be from
approximately 3.72 to 3.09, a more tangible difference (Fig. 8.2b). The relationship between
reliability index and probability of factor of safety being less than unity is further demonstrated by
Fig. 8.3, showing contours of reliability index with variations of mean and standard deviation for the normal distribution.

8.3 Simulation of the Infinite Slope Model

A validation of the infinite slope model for the four coastal British Columbia test sites was presented in Chapter 6. The model assumptions, sensitivities, input parameter types (measured, estimated and temporally varying) and interpreted implications were presented for five deterministically back-analyzed failures. The purpose of this chapter is to demonstrate the same approach to the problem of infinite slope failures, but to extend the deterministic analysis to a modified Monte Carlo simulation of all 1,526 mapped polygons which were described and categorized in Chapter 7. Reference should be made to the schematic representation and equation of the infinite slope model shown in Fig. 6.1.

In extending the deterministic analysis of a particular slope to a Monte Carlo simulation, one is recognizing the inherent spatial and temporal variability of all geo-physical properties, even over a single, relatively uniform site. For instance, even though a single, most likely estimate of each input parameter to the infinite slope equation was used for the model validation exercises of Chapter 6, the parameters, such as root cohesion, depth to failure plane and slope angle, are known to vary. Given that the data collected for each of the 1,526 polygons discussed in Chapter 7 were collected for the purposes of a qualitative terrain attribute study rather than quantitative and site specific slope stability analyses, only some of the input parameters shown in Fig. 6.1 were sufficiently described to determine a suitable input distribution.

Those parameters for which sufficient data were collected during the terrain attribute study (slope angle and depth to failure plane), for which laboratory and in-situ testing provides a distribution of inputs (friction angle and root/structure cohesion), for which piezometric records have allowed the
determination of Gumbel distribution inputs (groundwater ratio) and for which published data and the recognition of its temporal variation has allowed for the postulation of input distributions (root/structure cohesion) are used as variable inputs to the infinite slope equation and are sampled continuously during the Monte Carlo simulation. Those parameters to which slope stability is less sensitive and for which site specific information is not readily available, such as saturated and moist unit weights, tree surcharge and specific gravity, are kept constant (as specified in Chapter 6).

Fig. 8.4 presents the distributions of minimum, average and maximum slope angles for all mapped stable and unstable polygons. It is reassuring to note that the unstable distributions of slope angles (minimum, average and maximum alike) are, for the most part, greater than the corresponding stable distributions. For each mapped polygon, the minimum, average and maximum slope angles were measured and recorded on the field data card. These data are most appropriately represented as an input to the Monte Carlo simulation as triangular distributions with minimum, most likely and maximum values taken directly from the data cards. Those mapped polygons for which the minimum, average and maximum slope angles were not in ascending order, for which one or more of the three measurements were missing or for which there was another obvious data entry error were removed from the data set. The distributions of measured or estimated overburden depths, which were inferred to be equivalent to the depths to the failure plane for all unstable polygons, are presented in Fig. 8.5. The difference between the most likely depths for the stable and unstable polygons is insignificant, as determined using the difference of means test described in Chapter 7. The significance of two truncated normal distributions of depth is the physical limitation of depth to values which are greater than or equal to zero. It is interesting to note that the most commonly mapped depth over 1,526 coastal British Columbia polygons is approximately 1.5 to 2 m. For the purposes of Monte Carlo simulation, the single mapped value for overburden thickness is varied by ±50% to determine the minimum and maximum values of an assumed uniform input distribution for depth to failure plane.
The concept of ±50% of the mapped value is based on general field observations of typical spatial variations of overburden depth in mountainous regions of coastal British Columbia and on the practical observation that 50% of any reported positive depth remains a positive number, and is thus adequate for use in the infinite slope equation.

As discussed in Chapter 4, extensive (26 successful tests) large scale, in-situ, undisturbed direct shear testing and reconstituted, large-scale laboratory direct shear testing (8 successful tests) at typical sites and of typical overburden materials have been completed and summarized in Figs. 4.11 and 6.4. A triangular distribution with minimum, most likely and maximum values of friction angle of 45°, 47° and 49° respectively, as shown on Figure 8.6, has been chosen to be appropriate for use as input to these simulations. A triangular distribution is selected over a normal distribution due to limitations of the available data, with knowledge from Fig. 4.11 of a most likely value of approximately 47° and a reasonable upper and lower bound of ±2°. A more sophisticated and complicated normal distribution of friction angles is neither warranted nor desired. As discussed in Chapter 6, soil cohesion is taken as zero for the purpose of simulation.

Reference should be made to Figs. 5.8 and 6.3 a) for the temporally varying groundwater ratio and root cohesion input distributions. The groundwater ratio distributions are extreme value (Gumbel) distributions with parameters as discussed in Chapter 5, while the root cohesion distributions are 'general' in that they are individually sampled from the plot of relative root cohesion versus time (Fig. 6.2, after Sidle, 1991) and are thus unique. Development of these root cohesion input distributions is presented in Chapter 6.
A summary of the various infinite slope equation input distributions is provided as Fig. 8.8, in which schematic distributions for each varied input parameter are shown. Similar to the distinction between temporally varying and constant input parameter distributions, another logical distinction between site specific and regionally constant distributions can be made as shown on Fig. 8.8. Specifically, the friction angle, root cohesion and groundwater ratio distributions are applied on a regional basis while the overburden depth, or inferred depth to the failure plane, and slope angle distributions are applied on a site specific basis, with mapped observations forming the minimum, most likely and maximum limits to the input distributions. Ideally, input distributions for friction angle, root cohesion (dependent on species, biogeoclimatic zone, replanting lag amongst other factors) and groundwater ratio (dependent on local geology, morphology, surficial deposits and climate amongst other factors) would be determined on a site specific basis. Given the colossal amount of field work and investigations which would be required for this level of sophistication, regional distributions have been adopted. These regional distributions are likely to become more accurate and site specific with the continued influx of soils testing, climatological, hydrogeological and silvicultural data.

8.4 Sampling Issues and Simulation Technique

Given five temporally varying and temporally constant, regional and site specific input distributions (Fig. 8.8), plus four input parameters for which single (deterministic) values are specified, a Monte Carlo simulation proceeds by repetitively sampling each input distribution for one value, combining these sampled inputs with the other deterministic inputs (moisture content, surcharge, unit weights and specific gravity) and calculating a single value of factor of safety from the infinite slope equation for that specific set of inputs. If repeated a sufficient number of times and if a random number is used at the beginning of each calculation to ensure adequate representation of the full range (especially extreme events, such as extreme groundwater responses to extreme precipitation events) of each distribution, a representative distribution of calculated factors of safety (outputs) may be obtained for
plotting and statistical calculations. The add-in program ‘@RISK’ by Palisade Corporation (1994) was used with Microsoft Excel Ver. 5.0 to perform the required Monte Carlo simulations of the 1,526 mapped (and filtered) polygons.

Although a simple concept, there are a number of items which must be addressed in order to ensure meaningful, complete and timely results from the simulation. These items include, but are not limited to, the following concerns:

- model stability (indeterminant or undefined calculations due to input samples; negative groundwater ratio samples for example),
- correlation of input variables,
- representative sampling from input distributions; especially of extreme events,
- convergence of output to a stable distribution, and
- computational and memory requirements.

Most statistical distributions may be described using one or two parameters, such as mean and standard deviation for the normal distribution. If, as an example, overburden depth was specified to be a normal distribution with mean $\mu = 1.5 \, \text{m}$ and standard deviation $\sigma = 0.25 \, \text{m}$, it would be possible to inappropriately sample a negative depth from this input distribution for use in the infinite slope equation. In addition to being physically impossible, a negative number or some other sampled number may cause a calculation to return an undefined result due to division by zero or some other instability. For this reason, care must be taken to specify distributions which are physically possible and stable over their entire range. Triangular distributions are limited to the range of samples between the specified minimum and maximum values and are therefore not a concern in this case. Because 50% of a positive number is still always positive, the uniform distribution used for overburden (depth
to failure plane) is also stable over its entire range. The root cohesion and groundwater ratio distributions are also stable over their entire range because the ‘general’ distribution is fully specified to be within a stable and reasonable range and although there is no upper limit to an extreme sample from the extreme value (Gumbel) distribution the lower limit is fully specified and thus stable.

Correlation of input variables is an important consideration when the various input variables to a equation are known not to be independent. An example of this, in the case of the infinite slope equation, is the friction angle and soil cohesion input variables. A negative correlation is known to exist between these two variables, which should be reflected in the sampled values. When a relatively high value for friction angle is sampled, a relatively low value of soil cohesion should also be sampled to be realistic. A correlation of variables may be incorporated by specifying a correlation coefficient, r, between -1 and +1, between each pair of correlated variables. For example, correlation coefficient, r, values of -0.2 to -0.85 have been reported (Cherubini et al., 1983) between friction angle and soil cohesion. Because soil cohesion is taken to be constant (and zero), a correlation of these two parameters is not necessary for this study. It would be reasonable to expect a slight negative correlation between overburden depth and measured minimum, average and maximum slope angles. This expectation is confirmed by calculated correlation coefficients of approximately -0.07 to -0.12 between overburden depth and minimum and maximum slope angles respectively. Because slope angle and overburden depth are site specific input parameters, the need for a correlation coefficient in this case is minimal. If, however, these parameters were estimated on a regional basis, a deeper overburden thickness would be expected on shallower slopes and the use of a correlation coefficient would be advised.

As in this case, when a large number of simulations are required to accurately simulate output variables, computational efficiency and efficient sampling of input distributions are required.
Sampling efficiency refers to the number of iterations or samples which must be taken in order to adequately represent the original input distribution. The original sampling method, referred to as Monte Carlo sampling, simply uses a random number generator to obtain a sample from each input distribution. Using this method, it may take many iterations to sample relatively extreme events and thus many samples to adequately represent the original distribution. An advanced sampling technique, referred to as 'Latin Hypercube' sampling, and available in the @RISK add-on, is a more efficient sampling technique which breaks a range of potential inputs into equal-sized 'bins', which are all equally sampled, resulting in an adequate and faster representation of extreme events and distributions.

As a further computational efficiency, the statistical stability of the output distribution may be monitored on a regular basis with the simulation halting once the objective stability, or convergence, is attained. For example the percent change in mean, standard deviation and range of the output distribution after every 100 iterations will demonstrate the number of iterations at which the output distribution is considered stable. Several trial simulations with a tolerance for less than 1.5% change in mean, standard deviation and range after every 100 iterations indicated a typical requirement for approximately 1500 iterations per simulation.

Fig. 8.8 provides a graphical representation of the output distribution for an example polygon with a relatively high probability of factor of safety less than unity. As seen from the plot, the number of calculations which resulted in factors of safety being less than or equal to unity may be accumulated and divided by the total number of iterations to determine the probability of factor of safety being less than unity. In Fig. 8.8, the resulting probability is approximately 87%. The slight right-skew of the output histogram is due to the combined effects of non-symmetrical input distributions for groundwater ratio, slope angle and root cohesion and the non-linearity of the infinite slope equation.
Chapter 8. Probabilistic Modelling

8.5 Summary and Implications of Probabilistic Modelling

For each of the mapped 1,526 polygons, a simulation, as described above was completed with the site specific and regional input distributions. Because the groundwater ratio and root cohesion parameters are both temporally variable and because input distributions have been generated for 1, 10 and 20 year simulations, it would be possible to study the results of simulation of the mapped polygons over various time intervals. For instance, the probability of attaining a given value of groundwater ratio is higher if observed (simulated) over a 20 year period than over a shorter period, such as 10 years. The probability of factor of safety being less than unity should be greater (i.e. lower reliability index) for a 20 year observation (simulation) period than for a 10 year simulation period. This effect of decreasing reliability index with greater groundwater ratios over longer observation periods is slightly offset by the effects of root regrowth between 10 and 20 years since clearcutting (Fig. 6.2). Due to computational and time limitations, however, only the 20 year simulation results are discussed during the remaining sections. An ideal, however impractical, approach to this question of simulation duration would be to simulate each polygon for the actual duration between the year of harvesting and the year during which the polygon was mapped, thereby accounting for the increased likelihood of instability for a polygon which is mapped after 15 years over a polygon which is mapped after 6 years. The 20 year simulation duration is also selected due to the belief that the effects of logging on slope stability (in terms of root decay and regrowth, re-establishment of natural drainage patterns, and the achievement of substantial regrowth) are significant for a period of approximately 20 years. This is supported, according to the simulation results at least, by the observations of Fig. 8.9, in which the average probabilities of factor of safety being less than unity for the stable and unstable groups of polygons are plotted for 1, 10 and 20 years of simulation. These averages are observed to have reached a nearly constant value by approximately 20 years, indicating the appropriate use of 20 years as an indicative simulation period. Also, for simplicity and for improved distinction between polygons which have probabilities of factor of safety being less than unity which approach zero or unity, the
reliability index is calculated from the first and second moments of the output data according to Equation 8.1. The distributions of simulated reliability indices are plotted for the groups of unstable and stable polygons in Fig. 8.10. The two equivalent distributions of the probability of factors of safety being less than unity would not, however, be normally distributed due to the concentration of data and indiscernible differences at probabilities near zero and unity. It is again encouraging to note the significant difference between the stable and unstable distributions of reliability index. The mean and standard deviation of the output distribution of stable reliability indices are approximately 0.7 and 1.76, and -0.12 and 1.53 for the unstable output distribution.

This quantitative component of the proposed terrain stability assessment model (Fig. 7.2) is based on the infinite slope equation with measured material strengths, slope angles and groundwater ratios and several other assumed input parameters. The parameters were assumed to be either regionally constant or site specific and either temporally variable or temporally constant. This categorization of input parameters is demonstrated in Fig. 8.7.

Upon completion of many iterative factor of safety calculations for each of the 1,526 mapped polygons, statistics for each output distribution of factors of safety were collected in order to determine the reliability index, $\beta$, of each polygon. The reliability index was calculated using Equation 8.1a.

As with the resulting difference between the stable and unstable distributions of terrain attribute factor, $F$, (Fig. 7.2), although the difference between the stable and unstable distributions of reliability index, $\beta$, is statistically significant (ratio of difference of means to standard error of difference in means equal to approximately 8.1), it is recognized that there may be some benefit to combining both the
qualitative and quantitative results discussed in Chapters 7 and 8 to form a unified and more powerful predictive tool.
FIG. 8.1 a) Hypothetical normal distribution of factors of safety; deterministic and probabilistic measures of stability. b) Hypothetical irregular distribution.
FIG. 8.2 a) Misleading comparison of deterministic factors of safety. b) Unique relationship between Reliability Index, $\beta$, and Probability, $P(\text{FS} < 1)$. 
FIG. 8.3 Variation of Reliability Index, $\beta$, with standard deviation and mean factor of safety for the normal distribution.
FIG. 8.4 a) Distributions of mapped minimum slope angles, b) average slope angles, and, c) maximum slope angles.
FIG. 8.5 Distribution of mapped overburden thicknesses.
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FIG. 8.6 Interpreted shear strength distribution for Monte Carlo simulation input.
FIG. 8.7 Site specific, regional, temporally varying and constant input parameter distributions for simulation.
FIG. 8.8 Typical histogram of calculated factors of safety (1,000 trials).

\[ P(\text{FS} < 1.0) = \frac{873}{1000} = 87.3\% \]

\[ t = 20 \text{ years} \]
FIG. 8.9  Average probability of factor of safety less than unity over 1, 10 and 20 years of simulation for stable and unstable groups of polygons.
FIG. 8.10 Distributions of reliability index, $\beta$, for stable and unstable polygons.
9. **UNIFIED MODEL**

9.1 **Interpretation of Qualitative and Quantitative Data**

Chapters 7 and 8 have presented the findings of a qualitative, terrain attribute based study and the results of a quantitative simulation of the infinite slope model for all 1,526 mapped polygons. In Chapter 7, a Terrain Attribute Factor, \( F \), composed of five different morphological and drainage related partial factors was identified and described. The ability of this combined factor to distinguish between the unstable subset of 301 polygons and the stable subset of 1,225 polygons was assessed by comparing the statistical difference between two normal distributions which were fitted to the data subsets. A visual comparison can be made from Fig. 7.3. The ratio of the difference in means to the standard error for the stable versus unstable distributions is approximately 9.5, with a minimum of 2.0 required for a statistically significant difference between groups.

In a similar fashion, Chapter 8 describes a probabilistic modelling, or quantitative approach to distinguishing between the stable and unstable groups of mapped polygons. This Monte Carlo - like simulation of each polygon incorporated the inherent measurement uncertainty and natural variability of several input parameters in order to obtain a distribution of possible outputs (factors of safety) for each polygon.

As discussed in Chapter 8, the advantage of the reliability index is its ability to reflect both the mean and standard deviation of the output distribution in a single number. The variation of reliability index with sample mean and standard deviation was demonstrated in Fig. 8.3. As the Terrain Attribute Factors, \( F \), of the stable and unstable groups of polygons were plotted as separate distributions, so were the distributions of reliability index, \( \beta \), for the same two groups. Figure 8.10 shows the normal
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distributions of simulated stable and unstable reliability factors. The ratio of the difference in means to the standard error for these two distributions was calculated to be approximately 8.1, also demonstrating a statistically significant difference between groups.

The following table summarizes the population mean and standard deviation for stable and unstable groups of Terrain Attribute Factor, F, and reliability index, \( \beta \):

Table 9.1 Mean and standard deviation of reliability index and Terrain Attribute Factor for unstable and stable groups of polygons.

<table>
<thead>
<tr>
<th></th>
<th>Reliability index, ( \beta )</th>
<th>Terrain Attribute Factor, F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unstable polygons (301)</td>
<td>Mean, ( \mu_{\text{unstable}} )</td>
<td>-0.121</td>
</tr>
<tr>
<td></td>
<td>Standard deviation, ( \sigma_{\text{unstable}} )</td>
<td>1.529</td>
</tr>
<tr>
<td>Stable polygons (1,225)</td>
<td>Mean, ( \mu_{\text{stable}} )</td>
<td>0.704</td>
</tr>
<tr>
<td></td>
<td>Standard deviation, ( \sigma_{\text{stable}} )</td>
<td>1.765</td>
</tr>
</tbody>
</table>

Figs. 9.1 and 9.2 show the observed variation of open slope failure frequency (in terms of the ratio of the number of landslides to the polygon area) with Terrain Attribute Factor, F, and reliability index, \( \beta \), for the unstable polygons. The number of landslides and the polygon area are both obtained from the database of mapped polygons. Although these plots show some clustering and general behaviour amongst the unstable polygons, it would be difficult to use these two plots with confidence in practice.

A natural progression from the two approaches described in Chapters 7 and 8, one qualitative, the other quantitative and probabilistic, is to unify the two indicators, F and \( \beta \), into one more powerful or reliable predictor, referred to as the probability of initiation, \( p(\text{In}) \) (Fig. 7.2). Unified, the two indicators may provide greater insight into the likelihood of failure for a given polygon. Successful unification of these two approaches would result in exploitation of the advantages of the two methods while mostly avoiding, or protecting against, their drawbacks.
The following sections of this chapter describe an attempt to unify the qualitative and quantitative predictive methods described in Chapters 7 and 8 and comment on the interpretation and performance of this unified method.

9.2 **Unification of Terrain Attribute Factor and Reliability Index**

Figs. 7.3 and 8.10 present typical probability density functions (PDF's) for the stable and unstable groups of polygons, with the Terrain Attribute Factor, F, or reliability index, $\beta$, on the x-axis and probability density (relative frequency) on the y-axis. All four plotted distributions are unit distributions, with a cumulative area under the distributions of unity. Sections 9.2.1 and 9.2.2 discuss the unification of these four unit distributions into two bivariate normal distributions and into two kriged surface distributions. The bivariate normal distributions, one representing the unstable group of polygons, the other representing the stable group, are statistical fits to the true 3-dimensional distribution of polygons. The kriged surface distributions are, however, approximations of the real data distributions, and, despite being more difficult to work with, should provide improved performance over the bivariate normal equivalents.

9.2.1 **Bivariate Normal Distributions**

A bivariate normal distribution is a joint probability density function (PDF) which is represented by a 3-dimensional surface and, when viewed in section, provides the marginal probability density function of one of the two variables given a constant value for the other variable.

The univariate normal distribution, such as those shown in Figs. 7.3 and 8.10, is defined by the following equation:
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\[ f_x(x) = \frac{1}{\sigma_x \sqrt{2\pi}} \exp \left[ -\frac{1}{2} \left( \frac{x - \mu_x}{\sigma_x} \right)^2 \right] \]  \hspace{1cm} (9.1)

where \( \mu_x \) and \( \sigma_x \) are the mean and standard deviation of the population and \( x \) is the variable for which the pdf is required (Terrain Attribute Factor, \( F \), or reliability index, \( \beta \)). The bivariate normal distribution is 3-dimensional and thus has two variables, \( x \) and \( y \), which in this case are reliability index, \( \beta \), and Terrain Attribute Factor, \( F \), respectively.

The general equation defining the bivariate normal distribution is as follows:

\[ f_{\beta, F}(\beta, F) = \frac{1}{2\pi \sigma_\beta \sigma_F \sqrt{1-r^2}} \exp \left\{ \frac{1}{2(1-r^2)} \left[ \left( \frac{\beta - \mu_\beta}{\sigma_\beta} \right)^2 - 2r \left( \frac{\beta - \mu_\beta}{\sigma_\beta} \right) \left( \frac{F - \mu_F}{\sigma_F} \right) + \left( \frac{F - \mu_F}{\sigma_F} \right)^2 \right] \right\} \]  \hspace{1cm} (9.2)

where \( \mu_\beta, \sigma_\beta \) and \( \mu_F, \sigma_F \) represent the mean and standard deviation of the distributions of reliability index and Terrain Attribute Factor respectively, and \( r \) represents the coefficient of correlation between \( \beta \) and \( F \). The correlation coefficient, \( r \), reflects any dependence of reliability index on the Terrain Attribute Factor (or vice versa) for any one polygon. The value of the coefficient could be positive or negative, and must be between -1 and +1. A highly positive coefficient would indicate that for the majority of mapped polygons, a relatively large reliability index is usually observed in conjunction with a relatively large and equally signed Terrain Attribute Factor. Conversely, a negative coefficient of correlation is indicative of two variables which are inversely related. The calculated coefficients of correlation are 0.012 for stable polygons and 0.093 for unstable polygons. Given their proximity to zero, the coefficient has, for simplicity, been assumed to be zero in both cases, indicative of complete independence between axes (reliability index, \( \beta \) and Terrain Attribute Factor, \( F \)).
With reliability index, $\beta$, plotted on the x-axis and Terrain Attribute Factor, $F$, plotted on the y-axis, this results in a bivariate normal surface which is symmetrical around both a vertical line at $\beta = \mu_\beta$ and a horizontal line at $F = \mu_F$. Equation 9.2 is therefore simplified as follows:

$$f_{\beta,F}(\beta, F) = \frac{1}{2\pi\sigma_\beta\sigma_F}\exp\left\{-\frac{1}{2}\left[\left(\frac{\beta - \mu_\beta}{\sigma_\beta}\right)^2 + \left(\frac{F - \mu_F}{\sigma_F}\right)^2\right]\right\} \quad (9.3)$$

Figure 9.3 demonstrates the schematic unification of the two univariate normally distributed predictive variables (plots 'A' and 'B') into one bivariate distribution (plot 'C'). The bold lines of all three plots represent the distributions of unstable polygons. Inspection of plot 'C' reveals a progression towards less stability as both reliability index and Terrain Attribute Factor decrease, and vice versa as they increase.

Plotting the same distributions but in terms of the bivariate normal distributions results in Fig. 9.4, which provides the bivariate ($\beta, F$) normal distributions of the unstable ('A') and stable ('B') groups of polygons, and an overlay ('C') of both distributions for comparison. The oval shaped lines of Fig. 9.4 represent contours of constant relative frequency (probability density). The volume contained within each bivariate normal distribution, between the distribution surface and a plane at zero elevation, is equal to unity. As can be observed from plot 'C', there is an offset in both the x ($\beta$) and y ($F$) directions of the two distributions. Two sections, as shown in plot 'C', represent the shapes of the univariate distributions of Figs. 7.3 and 8.10.

### 9.2.2 Kriged Surface Distributions

As an alternative to fitting bivariate normal distributions to the groups of unstable and stable polygons as discussed in Section 9.2.1, the real distribution of data in three dimensions may be contoured and
plotted as shown in Fig. 9.5. Again, plots ‘A’ and ‘B’ represent the surface frequency contours of the groups of unstable and stable polygons respectively. In order to obtain the z (frequency), or out-of-page, value of each β, F location in plots ‘A’ and ‘B’, a visual basic routine for the Windows environment was written. The routine performs the function of a 2-dimensional histogram routine, but in three dimensions, by dividing the x, y (β, F) surface into evenly spaced segments and accumulating the number of observations in each segment in order to provide the z (frequency) value of each segment. The sets of x, y, z (β, F, frequency) points were then kriged in order to obtain contours of the joint probability density functions. The kriged surfaces of plots ‘A’ and ‘B’ have also been normalized (cumulative volumes of unity) and overlain in plot ‘C’ for comparison. A greater x (β) and y (F) offset between stable and unstable surfaces is observed in plot ‘C’ of Fig. 9.5 (kriged surface distribution) than in plot ‘C’ of Fig. 9.4 (bivariate normal distribution), and is attributed to the inability of the bivariate normal distribution to represent slight left- and right-skews to the real (kriged) data.

9.3 Probability of Initiation

In Chapters 7 and 8 and in Sections 9.1 and 9.2, four methods for the statistical separation of unstable polygons from stable polygons have been presented and discussed. It is the fourth, unified approach using surface kriging of observations, and as shown in Fig. 9.5, which offers the most promising distinction between these two polygon groups; both statistically and visually. The purpose of this section is to discuss how this established distinction between stable and unstable polygons may be used as a predictive tool and to discuss the results and performance of this unified approach.

Ideally, a group of qualitative partial factors, f, and measured (quantitative) parameters would be identified which are always indicative of either an unstable or a stable polygon. Unfortunately, and as expected from a highly variable natural system, four thorough attempts to make this clear distinction
have resulted, at best, in some overlap between the groups of stable and unstable polygons. For example, a polygon with a calculated reliability index, $\beta = -1.5$ and Terrain Attribute Factor, $F = 2.5$ would leave an interpreter using Fig. 9.5 with a problem: a similar frequency of stable and unstable polygons is observed at the given $(\beta, F)$ location.

Without this ideal set of distinguishing attributes and parameters, it appears that the best that can be done at present is to comment on the likelihood, or probability, that a polygon with a given $(\beta, F)$ pair will be a part of the stable or unstable group of polygons. This concept is demonstrated in two dimensions in Fig. 9.6. Given overlapping stable and unstable (univariate) distributions as shown, and a polygon with a parameter value $(\beta$ or $F$) of ‘$x$’, the likelihood, or probability, that the polygon belongs to the unstable group is equal to the ratio of ‘$a$’ to ‘$a+b$’. In three dimensions now, the probability of initiation, $p(\text{In})$ at any location on the $x$-$y$ $(\beta$-$F)$ plane is the ratio of the distance from the $x$-$y$ plane to the unstable bivariate normal or kriged surface distribution to the sum of this distance and the distance to the stable surface.

For the bivariate normal distributions presented in Section 9.2.1 and Fig. 9.4, this probability of initiation, $p(\text{In})$, is calculated as follows:

$$p(\text{In}) = \frac{n_{\text{unstable}} \left[f_{\beta,F}(\beta,F)\right]_{\text{unstable}}}{n_{\text{unstable}} \left[f_{\beta,F}(\beta,F)\right]_{\text{unstable}} + n_{\text{stable}} \left[f_{\beta,F}(\beta,F)\right]_{\text{stable}}} \quad (9.4)$$

where $[f_{\beta,F}(\beta,F)]_{\text{unstable}}$ represents the value of Equation 9.3 with $\mu_\beta, \sigma_\beta, \mu_F$ and $\sigma_F$ equal to the values presented in Table 9.1 for unstable polygons and to the similar values for stable polygons for $[f_{\beta,F}(\beta,F)]_{\text{stable}}$. $n_{\text{stable}}$ and $n_{\text{unstable}}$ represent the total observed number of stable (1,225) and unstable (301) polygons in the data set of 1,526 polygons.
Equation 9.4 is a form of Bayes theorem given two hypotheses: either the slope in question is unstable or it is stable. The probability of instability, \( P(\text{In}) \), is determined given the evidence of a pair of \( F \) and \( \beta \). This probability is calculated, according to Bayes theorem, by knowing the probabilities of \( F \) and \( \beta \) occurring if the slope is either stable or unstable, and the observed probabilities (prior knowledge) of instability (e.g. numberable / Number of observations).

Unlike the bivariate normal case, no analytical solution for \( p(\text{In}) \) exists for the kriged distributions presented in Section 9.2.2 and Fig. 9.5. The probability of initiation, surface may, however, be calculated using the gridded data files which are used to produce contour plots ‘A’ and ‘B’ of Fig. 9.5.

The results of the transformation described by Equation 9.4 are presented for the bivariate normal case (plots ‘A’ and ‘B’ of Fig. 9.4) and for the kriged surface case (plots ‘A’ and ‘B’ of Fig. 9.5) as plots ‘A’ and ‘B’ of Fig. 9.7. The plots provide the probability that a given polygon with parameters \( \beta \) and \( F \) belongs to the unstable group of polygons. As predicted from Fig. 9.3 during the original unification of the reliability index and Terrain Attribute Factor axes, the probability of initiation, \( p(\text{In}) \), is seen to increase towards the lower left and decrease towards the upper right of the plot area. The contours should not logically be permitted to decrease as one moves downwards or towards the left axis of the plot area. This anomaly is a weakness of the bivariate normal distribution method, which, with reference to plot ‘B’, is less prevalent in the real, kriged data than is suggested by plot ‘A’. For this, and other reasons, the kriged surface methodology presented in Fig. 9.5 and plot ‘B’ of Fig. 9.7 is preferred.
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It is possible, using the partial factors presented in Chapter 7 and the infinite slope modelling procedure presented in Chapter 8, to obtain a $\beta$, F pair which specifies a location beyond the axis range of plots 'A' and 'B' of Fig. 9.7. Due to the lack of data in these outer regions and the limitations of kriging near data boundaries and statistical fits using few data points, the reliability of Fig. 9.7 decreases at and beyond the plot boundaries. The actual number of plotted ($\beta$, F) pairs for the combined group of unstable and stable polygons is provided in Fig. 9.8, and may be considered to be indicative of the reliability of an estimate of $p(\text{In})$ for a mapped polygon.

9.4 Landslide Frequency, Unified Model Performance and Summary

The propensity for failure of mapped terrain polygons is occasionally described in terms of landslide frequency (the number of open slope landslides per unit area) and in terms of the percentage of polygons historically experiencing open slope failures. The probability of initiation, calculated using both the bivariate normal and kriged surface distributions, of each unstable polygon is plotted against open slope landslide frequency (number per hectare) and percentage of unstable polygons in Figs. 9.9 and 9.10 respectively. The percentage of polygons with landslides is calculated by dividing the number of observed unstable polygons within a specific range of $P(\text{In})$, as depicted by the horizontal error bars on Fig. 9.10, by the total number of polygons within the same range of $P(\text{In})$ values. The ranges of values in both the $x$ and the $y$ directions are shown as error bars in both figures. As expected, the number of open slope landslides per hectare and the percentage of unstable polygons both increase as the probability of initiation increases. The relationships between landslide frequency or percentage of unstable polygons and the probability of initiation, $p(\text{In})$, are appropriate for the studied data only and within the plotted limits of Fig. 9.8.

frequencies for Terrain Stability Classes I through V. The table is based on generalized data reported by Howes (1987), Rollerson (1992) and Rollerson and Sondheim (1985), and, as with the data reported here, may not apply to climatic regions other than coastal British Columbia or to longer time periods than 6 to 15 years.

The data provided in Table A2 of the guidebook are plotted as landslide frequency versus percentage of unstable polygons, with error bars representing the range of data on both axes, in Fig. 9.11. The current data, for the 1,526 modelled polygons, are also plotted for comparison. The estimated terrain stability classes associated with four ranges of landslide frequency and percentages of unstable polygons are provided along the bottom and right axes of the same figure. The two data sources compare very well considering the immense variability of survey sites, climatic regions, mapping techniques and interpretation, periods of observation or simulation, and the fact that road and gully related failures have been excluded from the data used in this study and included in the data reported in the BCMOF Guidebook. Of particular interest, however, is the observation that significant landslide frequencies (density) may be associated with terrain of low (III) to moderate (IV) stability class.
Open slope failure frequency (failures/hectare) versus terrain attribute factor, F: unstable polygons.

FIG. 9.1 Failure frequency versus terrain attribute factor, F: unstable polygons.
FIG. 9.2 Failure frequency versus reliability index, $\beta$: unstable polygons.
FIG. 9.3 Unification of terrain attribute factor, $F$, and reliability index, $\beta$: stable and unstable polygons.
FIG. 9.4 Bivariate normal distributions of stable and unstable polygons.
FIG. 9.5 Kriged surface distributions of stable and unstable polygons.
FIG. 9.6 Concept of probability of initiation, $p(\text{In})$: univariate case.

Probability of initiation, $p(\text{In}) = \frac{a}{a+b}$
FIG. 9.7 Transformed bivariate normal and kriged surface distributions of probability of initiation, $p(\ln)$. 

Chapter 9. Unified Model
FIG. 9.8 Data density (reliability) over adopted ranges of Terrain Attribute Factor, $F$ and reliability index, $\beta$. 
FIG. 9.9 Failure frequency versus probability of initiation, p(In): bivariate normal and kriged surface distributions.
FIG. 9.10 Percentage of polygons with landslides versus probability of initiation, \( p(\ln) \): bivariate normal and kriged surface distributions.
Chapter 9. Unified Model

Proposed unified model analysis results of Terrain Attribute Study D33 data
For open slopes only and for 20 year simulation period.

Generalized relationship between terrain stability class, frequency and likelihood of landslides after harvesting and road construction for several coastal study areas of British Columbia and for a period of 5 to 15 years after logging
(after Howes, 1987; Rollerson and Sondheim, 1985; and 'Mapping and Terrain Stability Guidebook', BCMOF, 1995)

FIG. 9.11 Results of application of proposed unified model to Terrain Attribute Study D33 data and comparison with previously published data.
10. SUMMARY AND CONCLUSIONS

10.1 General Observations and Conclusions

Figure 10.1, adapted from Wolff and Harr (1987), demonstrates the large variation in probability of failure amongst loading conditions and failure mechanisms of seemingly similar factors of safety. The shaded region towards the upper left corner of the plot represents the range of data obtained using the proposed technique. The region delimiting the variation of probability of failure and factor of safety for sudden drawdown of an embankment is a good example of how relatively high factors of safety against failure (3 to 5) do not necessarily result in very low probabilities of failure.

The data used in developing this model are presented in the same form as the shaded region of Fig. 10.1. Of the 1,526 mapped polygons used during the development of the proposed model, 301 polygons experienced open slope failures. This is consistent with the extension of the shaded region to the left of the vertical line defining a factor of safety of unity, and to probabilities of initiation near or equal to unity. Because many terrain polygons studied by specialists in the forestry sector plot in this general vicinity: relatively low factors of safety, high degrees of uncertainty and consequently, significant probabilities of initiation, the need to estimate even a relative likelihood of landslide initiation and the consequences of such a failure (environmental impacts, damage to other resources or property and safety issues) becomes very important.

The publication of many Forest Practices Code guidebooks, case history papers and conference proceedings and the development of a 'Division of Engineers and Geoscientists in the Forest Sector' as part of the Association of Professional Engineers and Geoscientists of B.C. will continue to better
Chapter 10. Summary and Conclusions

define standards of practice and suggested methods for the specialist. The objectives of the unified geostatistical and probabilistic terrain stability assessment technique presented here are to:

1. Provide a practitioner with another rational and defensible tool for use in the broader picture of forest resources management and risk assessment.
2. Provide and describe a framework for the unification of qualitative and quantitative terrain stability assessment techniques, which may be incorporated into a larger risk assessment approach to integrated resource management.
3. Provide a rational and systematic approach to determine site specific recommendations, using real probabilities, or to rank-prioritize sites for harvesting investment or remedial works.
4. Demonstrate the need for continued regional terrain attribute studies.
5. Describe the potential for use of piezometric and precipitation data to obtain temporally variable estimates of the probability of initiation or occurrence at a particular site.

This unified method has been developed in a modular format, such that new piezometric or terrain attribute data and quantitative slope stability methods may be incorporated as they are improved or become available. A modular format also allows for the future adaptation of its various components to allow for its use in different biogeoclimatic and geomorphological settings.

10.2 Model Summary: Theory and Practice

The need for a terrain stability assessment tool which adequately exploits a vast and growing base of knowledge and experience while, at the same time, incorporating the indicators provided by natural laws and observations has been identified in Chapters 1 and 2. Public and private concern regarding forestry practices, sustainable development, and environmental impacts is gaining momentum through
public demonstrations and media coverage, and has been addressed in the 5-year Fish / Forestry Interaction Program, the findings of the ESSA Report on Integrated Resource Management and, the development of the B.C. Forest Practices Code and associated guidebooks.

The unified method proposed here attempts to incorporate observed and subjective field data with a site specific stability analysis technique to obtain a unified terrain stability assessment method. By studying the temporal variation of a relatively long-term set of piezometric records and their extreme or episodic behaviour, and by considering the decay and regrowth of roots within the failure surface, consideration for the temporal variation of probability of initiation or occurrence has been included.

Five different open slope landslides, located at four sites on Vancouver Island and within the Seymour Watershed of the Greater Vancouver Water District (GVWD), have been studied and described in Chapter 3. All five landslide sites have been mapped according to the B.C. Terrain Stability Classification System, and classified according to ‘Event Type’, as described by Fannin and Rollerson (1992). Four of the five open slope landslides were of Type I, characterized by initiation and travel down a relatively uniform slope, while the Jamieson Creek landslide, in the Seymour Watershed, was of Type III, characterized by initiation and travel down an open slope followed by confinement and travel for relatively long distances along gentle (5° to 15°), confined channels.

Soil classification and shear strength testing were performed at each of these landslide locations in order to characterize the in-situ and reconstituted mechanical behaviour of the landslide materials and to determine a range of realistic inputs to the infinite slope model. Qualitative observations, such as the density of root matting and its effect on mechanical reinforcement and hydrology within the overburden horizon, the role of over-sized (cobble to boulder size) particles in finer, sand, gravel, silt and clay matrices, evidence of groundwater response to extreme precipitation events and, potential
Chapter 10. Summary and Conclusions

mechanisms of failure were noted and discussed in Chapters 3, 4 and 5. A series of large scale, in-situ and reconstituted direct shear tests were performed at low normal stresses on materials from the five landslide sites using an air-actuated shear box, designed and built at the University of British Columbia. In-situ direct shear tests were performed on exposed, but undisturbed, blocks of overburden at their natural moisture content and with the fine root system, if any, intact. Additionally, disturbed bulk samples materials from all sites were lab-dried, sieved to 25 mm minus and dry tamped into the shear box for testing. Repeatability of testing results was established by comparing measured shear strength parameters from several tests at the same normal stresses and on both the same materials and on materials from different landslide sites. As a benchmark, against which all in-situ and laboratory reconstituted test results could be compared, the same sample preparation and shearing technique was used in order to determine the shear response of a known, sub-rounded quartz sand. Results of tests on this benchmark sand compared well with previously published strength parameters for the same material, and lend confidence to the shear strength parameters reported in Chapter 4. Strength parameters reported in Chapter 4 also compare well with parameters reported by others (Skemter and Hillis, 1970 and Leps, 1970)

The quantitative component of the unified model involves a Monte Carlo-like simulation of the infinite slope equation for each of the polygons which were included in the Terrain Attribute Study. Because of the physical assumptions made in using the infinite slope equation and because of the limited use of the proposed model to the analysis of open slope failures, the original data set of approximately 2700 polygons was reduced to a subset of 1,526 polygons. As described in Chapter 6, the infinite slope model is composed of geomorphological data such as hillslope angle and depth to the failure plane, hydrological data such as depth to groundwater, and material properties such as moist and saturated unit weights, soil and root cohesion, and friction angle. Uncertainty in the various model input parameters is incorporated by assigning a distribution of likely values to these parameters. For
instance, based on field and laboratory testing of typical colluvial materials at four study sites in coastal British Columbia, a range of most likely values of friction angle was determined for use as the minimum, maximum and most likely friction angle values in a triangular input distribution. The variation of groundwater levels within the modelled hillslope section is accommodated temporally by means of an extreme value distribution specific to the duration of time being simulated. For instance, three extreme value distributions, based on near-continuous piezometric data from Carnation Creek Experimental Watershed are presented for 1, 10 and 20 years of simulation (Chapter 5). In a similar manner, three different distributions of root cohesion are used to represent 1, 10 and 20 years of observation. With a distribution of input parameters, a distribution of output factors of safety is expected and accumulated over many sampled iterations. From this distribution of outputs, the portion of factors of safety which are calculated to be less than or equal to unity is determined for each polygon and recorded as the probability of factor of safety being less than unity of a period of 1, 10 or 20 years. Also, provided the output distribution is not excessively skewed, the mean and standard deviation of the output are used to calculate the reliability index, $\beta$.

The qualitative component of the unified model is described in Chapter 7 and involves a Terrain Attribute Study of the subset of 1,526 mapped polygons, each with approximately 50 terrain attributes. The 'difference of means' and chi-squared tests were used to determine partial factors, $f$, for each terrain attribute. These partial factors were ranked by significance and by their ability to distinguish between the groups of stable and unstable polygons. The ranked partial factors are then accumulated into a Terrain Attribute Factor, $F$, using a discriminant analysis with the chi-squared test and a contingency table. Four terrain attributes: terrain unit, bedrock lithology, hillslope configuration, and hillslope curvature were determined to provide significant distinction between the groups of stable and unstable polygons for this data subset.
Unification of the subjective and qualitative Terrain Attribute Factor, F, and the quantitative reliability index, \( \beta \), is accomplished by combining these two indicators on a 3-dimensional plot with the observed frequency of the stable and unstable polygons. Bivariate normal and kriged surfaces are fitted to the stable and unstable distributions and then normalized to provide unit volumes under each surface (Chapter 9). For any newly mapped polygon with indicator parameters, \( \beta \) and F, the height to the unstable distribution surface is compared to the similar height to the stable distribution surface in order to determine the probability of the newly mapped polygon belonging to the stable or unstable group of polygons. This probability is referred to as the probability of initiation: it is compared with other published landslide frequencies and hazard ratings for all polygons included in this study.

### 10.3 Model and Biogeoclimatic Limitations: Implementation

Although an original data base size of 1,526 mapped polygons provides a significant source of field data for development of a model, all of the field data are from one region of coastal British Columbia (Vancouver Island) and from a limited number of biogeoclimatic sub-zones. Similarly, all piezometric observations, and laboratory and in-situ field testing of material strengths and properties, are based on sites representing a relatively small portion of the province. For these reasons, this model is intended, at best, to provide real, temporal probabilities of initiation or occurrence at sites similar to those used during the model development, and provide a relative comparison between sites with similar characteristics to the original database.

An inherent error which is introduced into any predicative model which is developed partially using observed field data is the case in which a polygon which is observed to be stable at the time of mapping and was harvested between 6 and 15 years previously experiences on open slope landslide after mapping. Further potential inaccuracies are introduced by extending limited in-situ field and
laboratory strength testing results to an entire collection of polygons as a triangular distribution of friction angles for input into the Monte Carlo like simulation. An attempt has, however, been made to minimize this effect by using only those polygons which are reported to have one of 5 (of the possible 19) specific Terrain Unit types, which approximately span the range of laboratory or field tested material types. Other inaccuracies are introduced due to the subjective and qualitative nature of the mapping and differences in interpretation amongst mappers, which may influence the calculated Terrain Attribute Factor despite the relatively small group of experienced individuals who collected the data used in this study. As new terrain attribute and landslide mapping data become available, it would be possible and recommended to incorporate these data into the model, thereby improving its precision by virtue of a larger statistical database.

Likely the most significant deviation from reality for each polygon is the use of a single distribution of possible groundwater rations, $D_w/D$, across all simulated polygons for calculation/simulation of the 1, 10 or 20 year reliability index. In addition to being one of the most sensitive input parameters for most slope stability calculations, including the infinite slope model, groundwater ratio was observed, as reported in Chapter 5, to be one of the most variable and unpredictable characteristics of a polygon. Again, despite these uncertainties, careful back-analyses of 5 open slope failures across the 4 different field tested sites, and analysis of almost 8 years of piezometric records from 14 geomorphologically different installations within Carnation Creek has bounded the range of expected piezometric responses and allowed for the development of appropriate extreme value distributions for three different study durations (1, 10 and 20 years).

It is possible, given the current partial terrain attribute factors and the range of possible input parameters to the Monte Carlo-like infinite slope simulation, to obtain a pair of indicators, ($\beta$, $F$), which plot beyond the limits of the 3-dimensional distributions presented in Figs. 9.7 a) and b).
signifies another limitation of the proposed model due to the relative scarcity of observed data in these regions. For example, although it may be determined that a mapped polygon plots in the region to the right of the right axis of these two figures (9.7a and b), suggesting a polygon with a very low probability of initiation, it is possible that stability of the polygon in question is controlled more dramatically by a failure mechanism which was not considered during development of the model and which is not, therefore, necessarily well represented by either or both indicators.

10.4 Future Research

Despite all of the above qualifying comments, the proposed model does outline a framework for a unified qualitative and quantitative hazard mapping tool which has been demonstrated to be feasible for use in southwestern coastal British Columbia. This study has demonstrated the need to continue and expand the collection of terrain attribute data within other biogeoclimatic regions using accepted, yet subjective, classification systems and a standardized data acquisition technique. Also, as discussed above, given the variability of observed piezometric responses and the relatively high sensitivity of slope stability calculations to piezometric assumptions, the need for further studies of typical and extreme groundwater responses to precipitation events is clear. A promising approach for the extension of this model into other biogeoclimatic zones may involve the correlation of the existing piezometric data to existing precipitation records from Carnation Creek. Once a relationship has been established, based on observed (recorded) piezometric levels and historical precipitation, the extreme value distributions of $D_w/D$ for a newly studied region may be shifted according to the regional precipitation relative to Carnation Creek.

Common to other resource sectors, such as mineral and oil and gas exploration, the need to balance real and perceived environmental risks and hazards with costs and economic returns is becoming standard practice in British Columbia forest resource management. The socio-economic and
environmental impacts of exploiting or completely banning any one resource without consideration for coordinated management with other industries such as mining, tourism and fisheries have made it necessary to include a broader suite of disciplines in resource management decisions. An example in the forestry sector is the increased use of geotechnical and geoscience specialists, who are charged with providing comments regarding the stability of proposed and existing cutblocks and roads under a variety of hydrometeorological and operational conditions. As such, specialists are expected to use limited and often subjective or incomplete information to formulate opinions, which may consequently expose the specialist to excessive liability risks. The development and adoption of further tools which enable a practitioner to more confidently provide advice to resource managers continues to be a challenging and important step in the sustainable development of forestry and other resource practices in British Columbia.
FIG. 10.1 Effect of failure mechanism on likelihood of failure (after Wolff and Harr, 1987)
11. REFERENCES


References


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*Deterministic Level I Stability Analysis Ver. 1.02 (DLISA 1.02)*. May 1991. US Department of Agriculture, Forest Service, Intermountain Research Station, Engineering Technology Research Work Unit, Moscow, ID.
References


References

Hartman G. F. and Scrivener J. C., 1990. *Impacts of forestry practices on a coastal stream ecosystem, Carnation Creek, British Columbia*. Canadian Bulletin of Fisheries and Aquatic Sciences 223, Department of Fisheries and Oceans, Biological Sciences Branch, Pacific Biological Station, Ottawa, Ont.


Level I Stability Analysis Ver. 2.0 (LISA 2.0). January 1991. US Department of Agriculture, Forest Service, Intermountain Research Station, Engineering Technology Research Work Unit, Moscow, ID.


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References


References


APPENDIX A

TERRAIN STABILITY CLASSIFICATION
### Table 3. Terrain stability classification

<table>
<thead>
<tr>
<th>Terrain stability class</th>
<th>Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>• No significant stability problems exist.</td>
</tr>
<tr>
<td>II</td>
<td>• There is a very low likelihood of landslides following timber harvesting or road construction.</td>
</tr>
<tr>
<td></td>
<td>• Minor slumping is expected along road cuts, especially for one or two years following construction.</td>
</tr>
<tr>
<td>III</td>
<td>• Minor stability problems can develop.</td>
</tr>
<tr>
<td></td>
<td>• Timber harvesting should not significantly reduce terrain stability. There is a low likelihood of landslide initiation following timber harvesting.</td>
</tr>
<tr>
<td></td>
<td>• Minor slumping is expected along road cuts, especially for one or two years following construction. There is a low likelihood of landslide initiation following road-building.</td>
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<tr>
<td></td>
<td>• A field inspection by a terrain specialist is usually not required.</td>
</tr>
<tr>
<td>IV</td>
<td>• Expected to contain areas with a moderate likelihood of landslide initiation following timber harvesting or road construction. Wet season construction will significantly increase the potential for road-related landslides.</td>
</tr>
<tr>
<td></td>
<td>• A field inspection of these areas is to be made by a qualified terrain specialist prior to any development, to assess the stability of the affected area.</td>
</tr>
<tr>
<td>V</td>
<td>• Expected to contain areas with a high likelihood of landslide initiation following timber harvesting or road construction. Wet season construction will significantly increase the potential for road-related landslides.</td>
</tr>
<tr>
<td></td>
<td>• A field inspection of these areas is to be made by a qualified terrain specialist prior to any development, to assess the stability of the affected area.</td>
</tr>
</tbody>
</table>

Modified from: *Land Management Handbook 18* (1994). The classification addresses landslides greater than 0.05 ha. in size.
<table>
<thead>
<tr>
<th>COMPONENT</th>
<th>MATERIAL</th>
<th>TEXTURE</th>
<th>AVERAGE DEPTH (m)</th>
<th>STRUCTURE</th>
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| TERRAIN COMPONENTS |

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</table>

**SLOPE POSITION:** Macro ap up mid lo sec hd

**HILLSLOPE CONFIGURATION:** un be di loc ir sg  
**CURVATURE:** concave convex straight

**SLOPE CONDITION:** mnl mccl mgvif

**FORESTED TERRAIN:** js pb lt tm cpl

**DRAINAGE CLASS:** r w m l p wp

**SOIL CLASSIFICATION**

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<tr>
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<th>AVG. DEPTH</th>
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**BEDROCK:** from a b c d e f g h i j k l m n o p

**ASPECT:**

**ELEVATION:**

**SLOPE ANGLES:** minimum average maximum

**GULLY:** number depth gradient gully wall

**LANDSLIDES:**

**ROADS:**

**RECORDED**

DATE Y M D

FS 124 HRE 92/10
APPENDIX B

QUICKBASIC DATA ACQUISITION CODE
DSHEAR.BAS QuickBasic data acquisition code:

********* UBC Civil Engineering - Direct Shear Testing Program *********
'BY:  John M.T. Wilkinson and John Y.K. Wong
'Date:  March/1994, revision June/94
'Hardware: Remote Data Acquisition Modules
'Software: RDAS.lib
'  Revision 1.01 March/1994
***************************************************************

'uses "dshear.ini" initiation file which contains i lines with
'the calibration factor and channel name (in double quotes) on
'each line; where i is the number of channels, followed by initial offsets and axis dimensions
'DECLARE SUB RDAS0 (comm%)
'DECLARE SUB RDAS1 (comm%, firstchan%, lastchan%, COMMAND%)
'DECLARE SUB RDAS2 (comm%, firstchan%, lastchan%, dat!, er$())
'$INCLUDE: 'jgrfx.j.bi'

COMMAND1% = &HE  'CONVERSION COMMAND \SINGLE,5 1/2,COMP
COMMAND2% = &H1E  'CONVERSION COMMAND \CONTINUOUS,5 1/2,COMP
COMMAND3% = &H6  ' \single,4 1/2,comp
COMMAND4% = &H16  ' \continuous,4 1/2,comp

DIM dat!(31), er$(31), x1!(2), x2!(2), x3!(2), posi!(2), press!(2), load!(2), shear!(2)

plotting arrays x1, x2 and x3 and engineering variables for position, pressure, load, shear stress
'demand, feedback and pressure difference declared

graphics subroutines called to initialize graphics and view windows
CALL jlNITGRFX(screen.vga%, clr.black%, 0, clr.bwhite%)
CALL jlNITWNDW(1,51,100,0,50, clr.black%, clr.bwhite%, clr.bwhite%)
CALL jlNITWNDW(2,51,100,50,100, clr.black%, clr.bwhite%, clr.bwhite%)
CALL jlNITWNDW(3,0,51,78,100, clr.black%, clr.bwhite%, clr.bwhite%)

**************************************************************

CLS
'specify which comm port to use (specify 0 if none connected or port will "timeout")
LOCATE 13, 15: PRINT "Enter number of COM port to be used (0 for none)"
tryagain:
m% = jb.iskey%
IF m% = 0 THEN GOTO tryagain
IF m% = 49 THEN comm% = 1: GOTO start
IF m% = 50 THEN comm% = 2: GOTO start
IF m% = 48 THEN comm% = 0: GOTO start
GOTO tryagain

start:
'initialize variables, read channel labels, calibration factors and axis defaults from "dshear.ini"
CLS
COMMAND% = COMMAND1%  'set conversion type (1 to 4)
firstchan% = 0
lastchan% = 7
starttime! = 0
OPEN "dsshear.ini" FOR INPUT AS #2
INPUT #2, message$(0), k!(0)
k!(0) = k!(0) * .00005
INPUT #2, message$(1), k!(1)
k!(1) = k!(1) * .00005
INPUT #2, message$(2), k!(2)
k!(2) = k!(2) * .0001
INPUT #2, message$(3), k!(3)
k!(3) = k!(3) * .01
INPUT #2, message$(4), k!(4)
k!(4) = k!(4) * .01
INPUT #2, message$(5), k!(5)
k!(5) = k!(5) * .00005
INPUT #2, message$(6), k!(6)
k!(6) = k!(6) * .00005
INPUT #2, message$(7), k!(7)
k!(7) = k!(7) * .001
INPUT #2, offset!(0), offset!(1), offset!(2), offset!(3), offset!(4), offset!(5), offset!(6), offset!(7)
offset!(0) = offset!(0) / .00005: offset!(1) = offset!(1) / .00005
offset!(2) = offset!(2) / .0001
offset!(3) = offset!(3) / .01: offset!(4) = offset!(4) / .01
offset!(5) = offset!(5) / .00005: offset!(6) = offset!(6) / .00005
offset!(7) = offset!(7) / .001
INPUT #2, cyc1!, y1min!, y1max!
INPUT #2, cyc2!, y2min!, y2max!
INPUT #2, cyc3!, y3min!, y3max!
CLOSE #2
checkflag% = 0
ccount% = 0
zeros! = 0
xmin! = 0
xmax1! = cyc1
xmax2! = cyc2
xmax3! = cyc3
y3min! = -y3max!
'specify titles for graph axes
xtitle$ = "TIME (minutes)"
y1titlea$ = "COMMAND (mm)"
y1titleb$ = "FEEDBACK (mm)"
y1titlec$ = "SHEAR STRESS (kPa x10)"
y2titlea$ = "LOAD (kg)"
y2titleb$ = "PRESSURE 1 (kPa)"
y2titlec$ = "PRESSURE 2 (kPa)"
y3titlea$ = "POS.DIFF.(mm)"
y3titleb$ = "PRESS.DIFF.(atm)"
IF comm% = 1 OR comm% = 2 THEN CALL RDAS0(comm%) 'setup serial communication port

*************** main program ***************

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CLS
CALL jSCALEWNDW(1, 18!, 6!, 10!, 5!, xmin!, xmax!, y1min!, y1max!)
CALL jSCALEWNDW(2, 18!, 6!, 10!, 5!, xmin!, xmax2!, y2min!, y2max!)
CALL jSCALEWNDW(3, 18!, 6!, 6!, 6!, xmin!, xmax3!, y3min!, y3max!)
CALL jPLACEWNDW(1)
CALL jPLACEWNDW(2)
CALL jPLACEWNDW(3)

main:
COLOR 15
GOSUB menu1
ON KEY(1) GOSUB quit
KEY(1) ON
ON KEY(2) GOSUB storerate
KEY(2) ON
ON KEY(3) GOSUB offset
KEY(3) ON
ON KEY(4) GOSUB initialize
KEY(4) ON
ON KEY(9) GOSUB cycle
KEY(9) ON
ON KEY(10) GOSUB startlog
KEY(10) ON
message$ = "Please select Function key."
LOCATE 23, 10: PRINT STRING$(31, "")
LOCATE 23, 10: PRINT message$
oldlog! = TIMER
oldtime1! = TIMER
oldtime2! = TIMER
oldtime3! = TIMER
ipts% = 2
GOSUB text
GOTO again

text:
t% = jb.gmsg%(xtitle$, 490, 465, gfont.ct%, gfont.normal%, gfont8x8%, clr.bwhite%, -1)
t% = jb.gmsg%(y1titlea$, 333, 110, gfont.ct%, gfont.left%, gfont8x8%, clr.bwhite%, -1)
t% = jb.gmsg%(y1titleb$, 347, 110, gfont.ct%, gfont.left%, gfont8x8%, clr.bwhite%, -1)
t% = jb.gmsg%(y1titlec$, 361, 110, gfont.ct%, gfont.left%, gfont8x8%, clr.white%, -1)
t% = jb.gmsg%(y2titlea$, 333, 350, gfont.ct%, gfont.left%, gfont8x8%, clr.bwhite%, -1)
t% = jb.gmsg%(y2titleb$, 347, 350, gfont.ct%, gfont.left%, gfont8x8%, clr.bwhite%, -1)
t% = jb.gmsg%(y2titlec$, 361, 350, gfont.ct%, gfont.left%, gfont8x8%, clr.white%, -1)
t% = jb.gmsg%(y3titlea$, 6, 424, gfont.ct%, gfont.left%, gfont6x6%, clr.bwhite%, -1)
t% = jb.gmsg%(y3titleb$, 16, 424, gfont.ct%, gfont.left%, gfont6x6%, clr.white%, -1)
t% = jb.gmsg%(xtitle$, 490, 225, gfont.ct%, gfont.normal%, gfont8x8%, clr.bwhite%, -1)
t% = jb.gmsg%("0", 381, 225, gfont.ct%, gfont.normal%, gfont8x8%, clr.bwhite%, -1)
t% = jb.gmsg%("0", 381, 465, gfont.ct%, gfont.normal%, gfont8x8%, clr.bwhite%, -1)
RETURN

again:
IF comm% = 1 OR comm% = 2 THEN CALL RDAS1(comm%, firstchan%, lastchan%, COMMAND%)
IF comm% = 1 OR comm% = 2 THEN CALL RDAS2(comm%, firstchan%, lastchan%, dat!, er$())
GOSUB display
IF fileflag = 1 AND ABS(TIMER - oldlog!) >= store! THEN GOSUB datalog
IF ABS(TIMER - oldtime1!) >= .1 THEN GOSUB upgraph1
IF x1(2) >= xmax1 THEN JPLACEWNDW (1): x1(1) = 0: GOSUB text
IF ABS(TIMER - oldtime2!) >= .1 THEN GOSUB upgraph2
IF x2(2) >= xmax2 THEN JPLACEWNDW (2): x2(1) = 0: GOSUB text
IF ABS(TIMER - oldtime3!) >= .1 THEN GOSUB upgraph3
IF x3(2) >= xmax3 THEN JPLACEWNDW (3): x3(1) = 0: GOSUB text
GOTO again

offkey:
KEY(1) OFF: KEY(2) OFF: KEY(3) OFF: KEY(4) OFF: KEY(9) OFF: KEY(10) OFF
RETURN

onkey:
KEY(1) ON: KEY(2) ON: KEY(3) ON: KEY(4) ON: KEY(9) ON: KEY(10) ON
RETURN

***************end of main program***************

datalog:
ON KEY(1) GOSUB quit
ON KEY(2) GOSUB storerate
ON KEY(3) GOSUB offset
ON KEY(4) GOSUB initialize
ON KEY(9) GOSUB cycle
ON KEY(10) GOSUB startlog
COLOR 15
message$ = "Running and Logging... "
LOCATE 23, 10: PRINT STRING$(31, " ")
LOCATE 23, 10: PRINT message$
GOSUB offkey
OPEN file$ FOR APPEND AS #10
WIDTH #10, 126
PRINT #10, (TIMER - startime!) / 60; dat!(0); dat!(1); dat!(2); dat!(3); dat!(4); dat!(5); dat!(6); dat!(7)
CLOSE #10
GOSUB onkey
datacount% = datacount% + 1
LOCATE 5, 33: PRINT datacount%
oldlog! = TIMER
RETURN

upgraph1:
ON KEY(1) GOSUB quit
ON KEY(2) GOSUB storerate
ON KEY(3) GOSUB offset
ON KEY(4) GOSUB initialize
ON KEY(9) GOSUB cycle
ON KEY(10) GOSUB startlog

\[ x_1(2) = x_1(1) + \frac{(\text{TIMER} - \text{oldtime}1)}{60} \]

\[ \text{demand}(2) = (\text{dat}(0) - \text{offset}(0)) \times k(0) \]

\[ \text{feedbk}(2) = (\text{dat}(1) - \text{offset}(1)) \times k(0) \]

\[ \text{shear}(2) = (\text{dat}(2) - \text{offset}(2)) \times k(2) \times 9.807 / 1000 \times 0.3048 / (0.3048 - (\text{feedbk}(2) / 1000)) \times 10 \]

IF demand(2) > ylmax OR demand(2) <= ylmin OR feedbk(2) > ylmax OR feedbk(2) < ylmin
THEN \( t\% = \text{jmsg}("\text{POSITION OUT OF RANGE}", 452, 6, \text{gfont.ct\%}, \text{gfont.normal\%}, \text{gfont6x6\%}, \text{clr.bwhite\%}, -1) \) ELSE \( t\% = \text{jmsg}("\text{POSITION OUT OF RANGE}", 452, 6, \text{gfont.ct\%}, \text{gfont.normal\%}, \text{gfont6x6\%}, \text{clr.black\%}, -1) \)

IF shear(2) > ylmax OR shear(2) <= ylmin THEN \( t\% = \text{jmsg}("\text{SHEAR OUT OF RANGE}", 578, 6, \text{gfont.ct\%}, \text{gfont.normal\%}, \text{gfont6x6\%}, \text{clr.white\%}, -1) \) ELSE \( t\% = \text{jmsg}("\text{SHEAR OUT OF RANGE}", 578, 6, \text{gfont.ct\%}, \text{gfont.normal\%}, \text{gfont6x6\%}, \text{clr.black\%}, -1) \)

CALL jPLOT(1, x1(), demand(), ipts\%, \text{clr.bwhite\%})
CALL jPLOT(1, x1(), feedbk(), ipts\%, \text{clr.bwhite\%})
CALL jPLOT(1, x1(), shear(), ipts\%, \text{clr.white\%})

demand(1) = demand(2)
feedbk(1) = feedbk(2)
shear(1) = shear(2)

\[ x_1(1) = x_1(2) \]

\[ \text{oldtime}1! = \text{TIMER} \]

RETURN

upgraph2:
ON KEY(1) GOSUB quit
ON KEY(2) GOSUB store rate
ON KEY(3) GOSUB offset
ON KEY(4) GOSUB initialize
ON KEY(9) GOSUB cycle
ON KEY(10) GOSUB startlog

\[ x_2(2) = x_2(1) + \frac{(\text{TIMER} - \text{oldtime}2)}{60} \]

\[ \text{load}(2) = (\text{dat}(2) - \text{offset}(2)) \times k(2) \]

\[ \text{press1}(2) = (\text{dat}(3) - \text{offset}(3)) \times k(3) \]

\[ \text{press2}(2) = (\text{dat}(4) - \text{offset}(4)) \times k(4) \]

IF press1(2) > y2max OR press1(2) <= y2min OR press2(2) > y2max OR press2(2) <= y2min
THEN \( t\% = \text{jmsg}("\text{PRESSURE OUT OF RANGE}", 565, 245, \text{gfont.ct\%}, \text{gfont.normal\%}, \text{gfont6x6\%}, \text{clr.white\%}, -1) \) ELSE \( t\% = \text{jmsg}("\text{PRESSURE OUT OF RANGE}", 565, 245, \text{gfont.ct\%}, \text{gfont.normal\%}, \text{gfont6x6\%}, \text{clr.black\%}, -1) \)

IF load(2) > y2max OR load(2) <= y2min THEN \( t\% = \text{jmsg}("\text{LOAD OUT OF RANGE}", 440, 245, \text{gfont.ct\%}, \text{gfont.normal\%}, \text{gfont6x6\%}, \text{clr.bwhite\%}, -1) \) ELSE \( t\% = \text{jmsg}("\text{LOAD OUT OF RANGE}", 440, 245, \text{gfont.ct\%}, \text{gfont.normal\%}, \text{gfont6x6\%}, \text{clr.black\%}, -1) \)

CALL jPLOT(2, x2(), load(), ipts\%, \text{clr.bwhite\%})
CALL jPLOT(2, x2(), press1(), ipts\%, \text{clr.white\%})
CALL jPLOT(2, x2(), press2(), ipts\%, \text{clr.white\%})

load(1) = load(2)
p ress1(1) = press1(2)
p ress2(1) = press2(2)

\[ x_2(1) = x_2(2) \]

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oldtime2! = TIMER
RETURN

upgraph3:
ON KEY(1) GOSUB quit
ON KEY(2) GOSUB storerate
ON KEY(3) GOSUB offset
ON KEY(4) GOSUB initialize
ON KEY(9) GOSUB cycle
ON KEY(10) GOSUB startlog

x3(2) = x3(1) + (TIMER - oldtime3) / 60
posi(2) = (dat!(1) - offset!(1)) * k!(1) - (dat!(0) - offsetl(O)) * k!(O)
pressd(2) = ((dat!(3) - offset!(3)) * k!(3) - (dat!(4) - offset!(4)) * k!(4)) / 101.325

IF posi(2) > y3max OR posi(2) < y3min THEN t% = jb.gmsg%("POSITION DIFFERENCE OUT OF RANGE", 180, 378, gfont.ct%, gfont.normal%, gfont6x6%, clr.bwhite%, -1) ELSE t% = jb.gmsg%("POSITION DIFFERENCE OUT OF RANGE", 180, 378, gfont.ct%, gfont.normal%, _
gfont6x6%, clr.black%, -1)

IF pressd(2) > y3max OR pressd(2) < y3min THEN t% = jb.gmsg%("PRESSURE DIFFERENCE OUT OF RANGE", 180, 470, gfont.ct%, gfont.normal%, gfont6x6%, clr.white%, -1) ELSE t% = jb.gmsg%("PRESSURE DIFFERENCE OUT OF RANGE", 180, 470, gfont.ct%, gfont.normal% _
, gfont6x6%, clr.black%, -1)

CALL jPLOT(3, x3(), posi(), ipts%, clr.bwhite%)
CALL jPLOT(3, x3(), pressd(), ipts%, clr.white%)
posi(1) = posi(2)
pressd(1) = pressd(2)
x3(1) = x3(2)
oldtime3! = TIMER
RETURN

menu1:
COLOR 7
LOCATE 1, 4: PRINT "UBC - Civil Engineering Department"
LOCATE 2, 7: PRINT "Direct Shear Testing Program"
LOCATE 4, 1: PRINT "Date:"
LOCATE 5, 1: PRINT "Time:"
LOCATE 6, 1: PRINT "Zeros:"
LOCATE 4, 20: PRINT "Data Rate:"
LOCATE 5, 20: PRINT "Data Stored:"
LOCATE 6, 20: PRINT "File:"
LOCATE 8, 20: PRINT "File:"
LOCATE 8, 18: PRINT "ACTUAL"
LOCATE 8, 30: PRINT "ENGINEERING"
LOCATE 9, 18: PRINT "READING"
LOCATE 9, 33: PRINT "UNITS"
COLOR 15
LOCATE 10, 4: PRINT message$(0)
COLOR 7
LOCATE 10, 38: PRINT "mm"
COLOR 15
LOCATE 11, 4: PRINT message$(1)
COLOR 7
LOCATE 11, 38: PRINT "mm"
COLOR 15
LOCATE 12, 4: PRINT message$(2)
COLOR 7
LOCATE 12, 38: PRINT "kg"
COLOR 15
LOCATE 13, 4: PRINT message$(3)
COLOR 7
LOCATE 13, 38: PRINT "kPa"
COLOR 15
LOCATE 14, 4: PRINT message$(4)
COLOR 7
LOCATE 14, 38: PRINT "kPa"
COLOR 15
LOCATE 15, 4: PRINT message$(5)
COLOR 7
LOCATE 15, 38: PRINT "mm"
COLOR 15
LOCATE 16, 4: PRINT message$(6)
COLOR 7
LOCATE 16, 38: PRINT "mm"
COLOR 15
LOCATE 17, 4: PRINT "Cel"
COLOR 15
LOCATE 17, 38: PRINT "Shear Stress"
COLOR 7
LOCATE 18, 4: PRINT "kPa"
LOCATE 20, 1: PRINT "<F1> Quit/Suspend"
LOCATE 20, 20: PRINT "<F2> File Name/Rate"
LOCATE 21, 1: PRINT "<F3> Set Offsets"
LOCATE 21, 20: PRINT "<F4> Initialize"
LOCATE 22, 1: PRINT "<F9> Graph Cycle"
LOCATE 22, 20: PRINT "<F10> Log Data"
LOCATE 23, 1: PRINT "MESSAGE: *
GOSUB display
RETURN

display:
COLOR 7
j = 0
FOR i = firstchan% TO lastchan%
LOCATE 10 + j, 1: PRINT **
LOCATE 10 + j, 1: PRINT i
j = j + 1
NEXT
j = 0
actual$ = "###.####"
FOR i = firstchan% TO lastchan%
COLOR 7
LOCATE 10 + j, 16: PRINT STRINGS$(10, "")
IF i = 0 OR i = 1 OR i = 5 OR i = 6 THEN LOCATE 10 + j, 16: PRINT USING actual$; datl(i) * 0.00005: LOCATE 10 + j, 24: PRINT " V"
IF i = 2 THEN LOCATE 10 + j, 16: PRINT USING actual$; datl(i) * .00001: LOCATE 10 + j, 24: PRINT " mV"
IF i = 3 OR i = 4 THEN LOCATE 10 + j, 16: PRINT USING actual$; datl(i) * .000001 * 1000: LOCATE 10 + j, 24: PRINT " mV"
COLOR 15
LOCATE 10 + j, 27: PRINT STRINGS$(10, "")
LOCATE 10 + j, 27: PRINT USING eng$; (datl(i) - offsetl(i)) * kl(i)
j = j + 1
NEXT
COLOR 15
LOCATE 18, 27: PRINT STRINGS$(10, "")
LOCATE 18, 27: PRINT USING eng$; shear(2) / 10
LOCATE 4, 7: PRINT DATES
LOCATE 5, 7: PRINT TIME$
LOCATE 4, 31: PRINT USING "###.#", store!
LOCATE 6, 9: PRINT USING "##", zeros!
LOCATE 6, 26: PRINT file$
LOCATE 7, 1: PRINT STRINGS$(40, "-")
LOCATE 19, 1: PRINT STRINGS$(40, "-")
RETURN

offset:
IF storeflag = 0 THEN offfile% = 1: GOTO storerate
offfile% = 0
COLOR 15
GOSUB offkey
LOCATE 23, 1: PRINT STRINGS$(40, "")
LOCATE 23, 1: PRINT "Zero Command Position ?"
commoff:
chan% = 0: aa% = jb.iskey%
IF aa% = 0 THEN GOTO commoff
IF aa% = 121 OR aa% = 89 THEN GOSUB yes
GOSUB menu1
LOCATE 23, 1: PRINT STRINGS$(40, "")
LOCATE 23, 1: PRINT "Zero Feedback Position ?"
feedoff:
chan% = 1: z% = jb.iskey%
IF z% = 0 THEN GOTO feedoff
IF z% = 121 OR z% = 89 THEN GOSUB yes
GOSUB menu1
LOCATE 23, 1: PRINT STRINGS$(40, "")
LOCATE 23, 1: PRINT "Zero Load Cell ?"
loadoff:
chan% = 2: y% = jb.iskey%
IF y% = 0 THEN GOTO loadoff
IF y% = 121 OR y% = 89 THEN GOSUB yes
GOSUB menu1
LOCATE 23, 1: PRINT STRINGS$(40, "")
LOCATE 23, 1: PRINT "Zero Pressure Transducer 1 ?"
presstooff:
chan% = 3: x% = jb.iskey%
IF x% = 0 THEN GOTO pressoff
IF x% = 121 OR x% = 89 THEN GOSUB yes
GOSUB menu1
LOCATE 23, 1: PRINT STRINGS$(40, "")
LOCATE 23, 1: PRINT "Zero Pressure Transducer 2 ?"
presstooff:
chan% = 4: w% = jb.iskey%
IF w% = 0 THEN GOTO press2off
IF w% = 121 OR w% = 89 THEN GOSUB yes
GOSUB menu1
LOCATE 23, 1: PRINT STRINGS$(40, "")
LOCATE 23, 1: PRINT "Zero Vertical Displacement 1 ?"
vertloff:
chan% = 5: v% = jb.iskey%
IF v% = 0 THEN GOTO vertloff
IF v% = 121 OR v% = 89 THEN GOSUB yes
GOSUB menu1
LOCATE 23, 1: PRINT STRINGS$(40, "")
LOCATE 23, 1: PRINT "Zero Vertical Displacement 2 ?"
vertloff:
chan% = 6: u% = jb.iskey%
IF u% = 0 THEN GOTO vertloff
IF u% = 121 OR u% = 89 THEN GOSUB yes
GOSUB menu1
GOSUB onkey
GOTO main

yes:
LOCATE 23, 1: PRINT STRINGS$(40, "")
LOCATE 23, 1: INPUT : "Enter offset or <CR>: ": off$
IF off$ = "" THEN offset(chan%) = datl(chan%) ELSE IF VAL(off$) > -1000000 AND VAL(off$) < 1000000 THEN offset(chan%) = VAL(off$)
OPEN file$ FOR APPEND AS #10
PRINT #10, message$(chan%); chan%; datl(chan%)
CLOSE #10
RETURN

storerate:
COLOR 15
IF storeflag = 1 THEN GOTO newfile ELSE GOTO origfile
origfile:
GOSUB offkey
LOCATE 23, 10: PRINT STRING$(31, "")
LOCATE 23, 10: INPUT; "Enter file name: ", a$
IF a$ = "" THEN GOSUB menu1: GOSUB onkey: GOTO main
file$ = a$
LOCATE 6, 26: PRINT STRING$(12, "")
retry:
LOCATE 23, 10: PRINT STRING$(31, "")
LOCATE 23, 10: INPUT; "Enter data rate (seconds): ", b$
IF VAL(b$) >= .5 AND VAL(b$) <= 3600 THEN store! = VAL(b$) ELSE GOTO retry
LOCATE 23, 10: PRINT STRING$(31, "")
LOCATE 23, 10: PRINT "Enter No. of weights: ".
retryb:
bb% = jb.iskey%
IF bb% = 0 THEN GOTO retryb
IF bb% = 49 THEN weights% = 1: GOTO nexta
IF bb% = 50 THEN weights% = 2: GOTO nexta
IF bb% = 51 THEN weights% = 3: GOTO nexta
IF bb% = 52 THEN weights% = 4: GOTO nexta
GOTO retryb
nexta:
retryc:
LOCATE 23, 10: PRINT STRING$(31, "")
LOCATE 23, 10: INPUT; "Enter strike azimuth: ", strikeazi%
IF strikeazi% < 0 OR strikeazi% > 360 THEN GOTO retryc
retryd:
LOCATE 23, 10: PRINT STRING$(31, "")
LOCATE 23, 10: INPUT; "Enter strike inclination: ", strikelat%
IF strikelat% < -90 OR strikelat% > 90 THEN GOTO retryd:
retrye:
LOCATE 23, 10: PRINT STRING$(31, "")
LOCATE 23, 10: INPUT; "Enter dip azimuth: ", dipazi%
IF dipazi% < 0 OR dipazi% > 360 THEN GOTO retrye
retryf:
LOCATE 23, 10: PRINT STRING$(31, "")
LOCATE 23, 10: INPUT; "Enter dip inclination: ", diplat%
IF diplat% < -90 OR diplat% > 90 THEN GOTO retryf
retryg:
LOCATE 23, 10: PRINT STRING$(31, "")
LOCATE 23, 10: INPUT; "Enter perimeter (m): ", peri!
IF peri! < .5 OR peri! > 1.3 THEN GOTO retryg
OPEN file$ FOR APPEND AS #10
WIDTH #10, 126
PRINT #10, "File: "; file$
PRINT #10, "Date: "; DATE$
PRINT #10, "Time: "; TIME$
PRINT #10, "Rate: "; b$
PRINT #10, "Weights: "; weights%
PRINT #10, "Strike Az.: "; strikeazi%
PRINT #10, "Strike Lat.: "; strikelat%
PRINT #10, "Dip Azi: "; dipazi%
PRINT #10, "Dip Lat: "; diplat%
PRINT #10, "Perimeter: "; peri!
PRINT #10, "Time(min.): "; Time(min.)
PRINT #10, "Demand: "; Demand
PRINT #10, "Feedback: "; Feedback
PRINT #10, "Load: "; Load
PRINT #10, "Press.1: "; Press.1
PRINT #10, "Press.2: "; Press.2
PRINT #10, "Vert.1: "; Vert.1
PRINT #10, "Vert.2: "; Vert.2
PRINT #10, "Temperature"
CLOSE #10
storeflag = 1
GOSUB menu1
GOSUB onkey
IF ccount% = 1 THEN GOSUB initialize
IF checkflag% = 1 THEN GOTO startlog
IF offile% = 1 THEN GOTO offset ELSE GOTO main
newfile:
GOSUB offkey
oldfile$ = file$
over:
LOCATE 23, 10: PRINT STRING$(31,"")
LOCATE 23, 10: PRINT "Specify new file or rate ?"
setup:
q% = jb.iskey
IF q% = 0 THEN GOTO setup
IF q% = 121 OR q% = 89 THEN GOTO continue ELSE GOSUB menu1: GOSUB onkey: GOTO main
continue:
LOCATE 23, 10: PRINT STRING$(31,"")
LOCATE 23, 10: INPUT; "Enter file name: ", h$
IF h$ = "" THEN GOTO reenter
file$ = h$
OPEN oldfile$ FOR APPEND AS #10
PRINT #10, "File: "; oldfile$; " closed properly by operator."
PRINT #10, "New file: "; file$; "opened at "; TIMES$; " on "; DATE$
CLOSE
LOCATE 6, 26: PRINT STRING$(12,"")
reenter:
LOCATE 23, 10: PRINT STRING$(31,"")
LOCATE 23, 10: INPUT; "Enter data rate (seconds): ", g$
IF VAL(g$) >= .5 AND VAL(g$) <= 3600 THEN store! = VAL(g$) ELSE GOTO reenter
OPEN file$ FOR APPEND AS #10
WIDTH #10, 126
PRINT #10, "File: "; file$
PRINT #10, "Date: "; DATE$
PRINT #10, "Time: "; TIME$
PRINT #10, "Rate: "; g$
PRINT #10, "Weights: "; weights%
PRINT #10, "Strike Azi: "; strikeazi%
PRINT #10, "Strike Lat: "; strikelat%
PRINT #10, "Dip Azi: "; dipazi%
PRINT #10, "Dip Lat: "; diplat%
PRINT #10, "Time(min.): "; Time(min.)
PRINT #10, "Demand: "; Demand
PRINT #10, "Feedback: "; Feedback
PRINT #10, "Load: "; Load
PRINT #10, "Press.1: "; Press.1
PRINT #10, "Press.2: "; Press.2
PRINT #10, "Vert.1: "; Vert.1
PRINT #10, "Vert.2: "; Vert.2
PRINT #10, "Temperature"
CLOSE #10
storeflag = 1
GOSUB menu1
GOSUB onkey
GOTO main

initialize:
GOSUB offkey
ccount% = 1
COLOR 15
IF storeflag <> 1 THEN GOTO storerate ELSE GOTO init
init: OPEN file$ FOR APPEND AS #10
WIDTH #10, 126
PRINT #10, "Initialization: ", TIMES
PRINT #10, (TIMER - starttime!) / 60; datl(0); datl(1); datl(2); datl(3); datl(4); datl(5); datl(6); datl(7)
CLOSE #10
zeros! = zeros! + 1
oldlog! = TIMER
GOSUB menu1
GOSUB onkey
RETURN

startlog:
checkflag% = 1
IF storeflag <> 1 THEN GOTO storerate ELSE fileflag = 1
IF starttime! = 0 THEN starttime! = TIMER
GOTO main

cycle:
COLOR 15
GOSUB offkey
LOCATE 23, 10: PRINT STRING$(31, " ")
LOCATE 23, 10: PRINT "Adjust graph 1, 2 or 3 ?"
0 cycleagain:
c% = jb.iskey%
IF c% = 0 GOTO cycleagain
IF c% = 121 OR c% = 89 THEN GOTO adjusty
IF c% = 120 OR c% = 88 THEN GOTO adjustx
IF c% = 13 THEN GOSUB onkey: GOTO main
GOTO cycle

graph:
LOCATE 23, 10: PRINT STRING$(31, " ")
LOCATE 23, 10: PRINT "Adjust X or Y axis ?"
axisover:
b% = jb.iskey%
IF b% = 0 THEN GOTO axisover
IF b% = 121 OR b% = 89 THEN GOTO adjusty
IF b% = 120 OR b% = 88 THEN GOTO adjustx
IF b% = 13 THEN GOSUB onkey: GOTO main
GOTO graph

adjustx:
LOCATE 23, 10: PRINT STRING$(31, "")
IF c% = 49 THEN LOCATE 23, 10: INPUT; "Enter graph 1 x-max (min): ", cyc1$: c% = 49:
GOTO xscale
IF c% = 50 THEN LOCATE 23, 10: INPUT; "Enter graph 2 x-max (min): ", cyc2$: c% = 50:
GOTO xscale
IF c% = 51 THEN LOCATE 23, 10: INPUT; "Enter graph 3 x-max (min): ", cyc3$: c% = 51:
GOTO xscale
GOTO cycle

xscale:
IF c% = 49 AND VAL(cyc1$) >= .1 AND VAL(cyc1$) <= 1000 THEN xmax1 = VAL(cyc1$): x1(1) = 0: c% = 49: GOTO xplace: ELSE IF c% = 49 GOTO adjustx
IF c% = 50 AND VAL(cyc2$) >= .1 AND VAL(cyc2$) <= 1000 THEN xmax2 = VAL(cyc2$): x2(1) = 0: c% = 50: GOTO xplace: ELSE IF c% = 50 GOTO adjustx
IF c% = 51 AND VAL(cyc3$) >= .1 AND VAL(cyc3$) <= 1000 THEN xmax3 = VAL(cyc3$): x3(1) = 0: c% = 51: GOTO xplace: ELSE IF c% = 51 GOTO adjustx
GOTO cycle

xplace:
IF c% = 49 THEN CALL jSCALEWNDW(1, 18!, 6!, 10!, 5!, xmin!, xmax1!, ylmin!, ylmax!):
CALL jPLACEWNDW(1)
IF c% = 50 THEN CALL jSCALEWNDW(2, 18!, 6!, 10!, 5!, xmin!, xmax2!, y2min!, y2max!):
CALL jPLACEWNDW(2)
IF c% = 51 THEN CALL jSCALEWNDW(3, 18!, 6!, 6!, 6!, xmin!, xmax3!, y3min!, y3max!):
CALL jPLACEWNDW(3)
GOSUB text
GOTO cycle

adjusty:
LOCATE 23, 10: PRINT STRING$(31, "")
IF c% = 49 THEN LOCATE 23, 10: INPUT; "Enter graph 1 y-max: ", y1maxtry$: c% = 49:
GOTO yscale
IF c% = 50 THEN LOCATE 23, 10: INPUT; "Enter graph 2 y-max: ", y2maxtry$: c% = 50:
GOTO yscale
IF c% = 51 THEN LOCATE 23, 10: INPUT; "Enter graph 3 y-max: ", y3maxtry$: c% = 51:
GOTO yscale
GOTO cycle

yscale:
IF c% = 49 AND VAL(y1maxtry$) > 0 AND VAL(y1maxtry$) <= 200 THEN y1max = VAL(y1maxtry$):
x1(1) = 0: c% = 49: GOTO yplace: ELSE IF c% = 49 GOTO adjusty
IF c% = 50 AND VAL(y2maxtry$) > 0 AND VAL(y2maxtry$) <= 2000 THEN y2max = VAL(y2maxtry$):
x2(1) = 0: c% = 50: GOTO yplace: ELSE IF c% = 50 GOTO adjusty
IF c% = 51 AND VAL(y3maxtry$) > 0 AND VAL(y3maxtry$) <= 2000 THEN y3max = VAL(y3maxtry$):
y3min = -VAL(y3maxtry$): x3(1) = 0: c% = 51: GOTO yplace: ELSE IF c% = 51 GOTO adjusty
GOTO cycle

yplace:
IF c% = 49 THEN CALL jSCALEWNDW(1, 18!, 6!, 10!, 5!, xmin!, xmax1!, y1min!, y1max!):
CALL jPLACEWNDW(1)
IF c% = 50 THEN CALL jSCALEWNDW(2, 18!, 6!, 10!, 5!, xmin!, xmax2!, y2min!, y2max!):
CALL jPLACEWNDW(2)
IF c% = 51 THEN CALL jSCALEWNDW(3, 18!, 6!, 6!, xMin!, xMax3!, y3Min!, y3Max!):
CALL jPLACEWNDW(3)
GOSUB text
GOTO cycle

quit:
GOSUB offkey
LOCATE 23, 10: PRINT STRING$(31, " ")
COLOR 15
LOCATE 23, 10: PRINT "Q": LOCATE 23, 18: PRINT "S"
COLOR 7
LOCATE 23, 11: PRINT "uit or ": LOCATE 23, 19: PRINT "uspend logging ? 
suspend:
s% = jb.iskey%
IF s% = 0 THEN GOTO suspend
IF s% = 113 OR s% = 81 THEN GOTO byebye
IF s% = 115 OR s% = 83 THEN fileflag = 0
GOSUB onkey
GOTO main
byebye:
IF storeflag = 1 THEN GOTO quit1 ELSE GOTO quit2
quit1:
CLS
OPEN file$ FOR APPEND AS #10
PRINT #10, "File:; file$; " - was closed properly by operator"
PRINT #10, "Date closed: "; DATE$
PRINT #10, "Time closed: "; TIMES
CLOSE
CLS
LOCATE 1, 1: PRINT "Program terminated by user... 
LOCATE 2, 1: PRINT "File:; file$; " - successfully closed."
END
quit2:
CLS
LOCATE 1, 1: PRINT "Program terminated by user...
END

DSHEAR.INI file containing calibration factors and channel labels:

"Command" 15.24
"Feedback" 15.24
"Load" 44.0736188099
"Press.1" 31.6243857725
"Press.2" 31.3333605995
"Vert.1" 1
"Vert.2" 1
"Temp." 1
0
0
-1082
10.54
10.93
0
0
0
140 0 200
1 0 600
1 -.2 .2

format:
Calib0 Label0
Calib1 Label1
Calib2 Label2
Calib3 Label3
Calib4 Label4
Calib5 Label5
Calib6 Label6
Calib7 Label7
off0
off1
off2
off3
off4
off5
off6
off7
x1max y1min y1max
x2max y2min y2max
x3max y3min y3max
APPENDIX C

TERRAIN ATTRIBUTE STUDY - D33
Title: Landslide Frequencies Following Logging: Coastal British Columbia
(Terrain Attribute Study - D33)

Project Leaders:

Bruce Thomson - Forest Sciences Section, Vancouver Forest Region
Terry Rollerson - Forest Sciences Section, Vancouver Forest Region

Contacts/Client Groups:

G. Sutherland - Planning and Inventory Section - Vancouver Forest Region
L. Leroux - Timber Harvesting Section - Vancouver Forest Region
P. Lewis - Fisheries Branch - Ministry of Environment, Lands and Parks
J. Lamb - Department of Fisheries and Oceans
Inventory Branch - Ministry of Forests
Timber Harvesting Branch - Ministry of Forests

Introduction

The identification of terrain that will be subject to landslides following logging or road building is a high priority for forest management in coastal British Columbia. This information can be used at the planning level to ensure that annual allowable cut (AAC) calculations reflect the landbase truly available for harvest, and at the development stage to ensure that environmentally sensitive areas are not damaged.

A program of slope stability mapping suitable for forest development purposes has been carried out by several forest companies, and the BC Ministry of Forests for about the last 15 years. Slope stability mapping is based on the production of terrain maps, usually at a scale of 1:20,000. Each terrain polygon is assigned an estimated stability class based on slope, surficial material, and evidence of previous instability. Additionally, the Ministry requires reconnaissance level ESA mapping to identify areas vulnerable to post-logging landslide activity for purposes of preliminary planning and AAC netdown calculations. These assessments are often very subjective, and are highly dependent on the local knowledge and experience of the mapper.

There is a need for more objective and quantitative approach for the prediction of post-logging stability. This approach requires the collection of data on the frequency of slope failures following logging, in order that the reliability of the criteria used by mappers for slope stability assessments can be improved.

We feel that an empirical approach, applied to a representative sample of landscape units over a wide geographical area, has potential for quantifying the likelihood of landslide occurrence. Work with limited data from the southwest coast of Vancouver Island (Rollerson and Sondheim, 1985) and the southern Coast Mountains (Howes, 1987), and a larger data set from the Queen Charlotte Islands (Rollerson, 1992), indicates that this approach viable.
Objectives

To characterize steepland terrain types which are subject to landslides following conventional clearcutting and road building, and those which are not.

To develop a multi-factor terrain-based stability classification system which estimates the likelihood and frequency of landslides occurring following conventional clearcutting and road building.

To provide data so that appropriate AAC netdown allowances can be made for the ES1 and ES2 categories of the Ministry of Forests ESA mapping program.

Study Areas

The study will be conducted across a number of large contiguous land units within Coast Ranges of the BC mainland and the Insular Mountains of Vancouver Island. Areas where work has already taken place on Vancouver Island include: the area north of the Brooks Peninsula and west of Quatsino Sound, the Kyuquot area, the area to the immediate east of Ucluelet, and logged areas within the Klanawa and Cameron River drainages.

Methodology

Within each main study area, a subset of logged areas is randomly selected (alternately all logged areas meeting the following criteria are sampled). The logged areas range in age from 6 to 15 years following logging. The lower age limit is imposed to ensure that the study areas have experienced a number of large storms, and to give time for loss in root strength to occur. If bedrock in the study areas varies significantly (e.g. sedimentary versus intrusive rocks), then clearcut areas of the appropriate age range are stratified by underlying bedrock formation and numbered in a systematic fashion. A subset of logged areas from each formation is then randomly selected for further study.

Each selected area is mapped at a scale of 1:20,000 using the B. C. Terrain Classification System (Howes and Kenk, 1988), and 1:15,000 to 1:20,000 scale aerial photography. Each terrain polygon is verified in the field. For the sake of efficiency, terrain polygons with slopes less than 20 degrees are excluded from the study, because they rarely show evidence of post-logging failure. Each terrain polygon constitutes a single sample. Data for a minimum of 2000-3000 terrain polygons, will be collected within each major physiographic region.

Data Collection

For each map polygon, terrain attribute data such as landscape position, slope, aspect, slope morphology, slope curvature, soil type, drainage class, surficial materials, bedrock type and the presence or absence of natural and post-logging landslides are recorded. For post-logging landslide frequency estimation, landslides smaller than 0.05 hectares are excluded, as they cannot reliably be identified at the air photo scales used. Landslides identified in the field or on aerial photography, which are 0.05 hectares or larger are included in the data set.
Data analysis

Non-parametric statistical tests will be applied to identify relationships between landslide frequency and terrain attributes. Individual terrain polygons will be grouped into a limited number of multi-factor terrain categories having similar landslide frequencies, and likelihood of post-logging failure. These categories will form the basis of a terrain stability classification which estimates the likelihood and frequency of landslides occurring following conventional clearcutting and road building.

Products

Research papers or land management reports, workshop and conference proceedings. The results will enable criteria to be defined for the assignment of terrain stability classes within the study areas. The data will facilitate identification of those ES1 and ES2 units which will experience excessive landslide activity following logging. These relationships can then be used to develop appropriate AAC netdown factors for landslide prone terrain.
**MORPHOLOGY**

<table>
<thead>
<tr>
<th>Polygon #</th>
<th>(5 fields)</th>
</tr>
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<tbody>
<tr>
<td>C - Clayquot</td>
<td></td>
</tr>
<tr>
<td>F - Cameron, China Creek area</td>
<td></td>
</tr>
<tr>
<td>K - Kyuquot</td>
<td></td>
</tr>
<tr>
<td>Q - Quatsino</td>
<td></td>
</tr>
<tr>
<td>A - Klamath</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Terrain Unit</th>
<th>Not recorded on form see last pg - Variable Terrain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope</td>
<td>(1 field)</td>
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<tr>
<td>1 - Apex (crest)</td>
<td></td>
</tr>
<tr>
<td>2 - Upper Slope</td>
<td></td>
</tr>
<tr>
<td>3 - Mid Slope</td>
<td></td>
</tr>
<tr>
<td>4 - Lower Slope</td>
<td></td>
</tr>
<tr>
<td>5 - Escarpment</td>
<td></td>
</tr>
<tr>
<td>6 - Headwater Drainage</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Hillslope Configuration</th>
<th>(1 Field)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - Uniform</td>
<td></td>
</tr>
<tr>
<td>2 - Benched</td>
<td></td>
</tr>
<tr>
<td>3 - Dissected</td>
<td></td>
</tr>
<tr>
<td>4 - Faceted</td>
<td></td>
</tr>
<tr>
<td>5 - Irregular</td>
<td></td>
</tr>
<tr>
<td>6 - Single Gully</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Hillslope Curvature</th>
<th>(1 field)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - Concave</td>
<td></td>
</tr>
<tr>
<td>2 - Convex</td>
<td></td>
</tr>
<tr>
<td>3 - Straight</td>
<td></td>
</tr>
<tr>
<td>4 - Complex</td>
<td></td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Bedrock Formation</th>
<th>(2 fields)</th>
</tr>
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<tbody>
<tr>
<td>1 - Island Intrusives</td>
<td></td>
</tr>
<tr>
<td>2 - Bonanza</td>
<td></td>
</tr>
<tr>
<td>3 - Karmutsen</td>
<td></td>
</tr>
<tr>
<td>4 - Other (Quatsino)</td>
<td></td>
</tr>
<tr>
<td>5 - Sicker</td>
<td></td>
</tr>
<tr>
<td>6 - Leech River</td>
<td></td>
</tr>
</tbody>
</table>

1
Bedrock Lithology (2 fields) - Codes for bedrock lithology follows the coding numbers as presented on Lithology pg. 47 of Describing Ecosystems in the Field - MOE Manual 11, 1990.

02 - granite
04 - granodiorite
06 - diorite
13 - rhyolite
15 - andesite
17 - basalt
19 - volcanic breccia
53 - greywacke
58 - limestone

Competence (1 field)
1 - High
2 - Medium
3 - Low

Structure (1 field)
1 - Massive
2 - Fractured
3 - Sheared
4 - Bedded

A.B.D. (3 fields) - Recorded as 0.0m

Drainage Class (1 field) - right justified to the decimal point.
1 - Rapid
2 - Well
3 - Moderately well
4 - Imperfect
5 - Poor
6 - Very poor

Watershed Code Not recorded on form
SOIL

| Soil Classification | (1 field) | 1 - Brunisol
|                    |           | 2 - Podzol
|                    |           | 3 - Gleyed Podzol
|                    |           | 4 - Gleyed Brunisol
|                    |           | 5 - Gleysol
|                    |           | 6 - Regosol

| Soil Texture       | (3 fields) - See Material Texture |

| Soil Depth         | (3 fields) - Recorded as 0.0m |

| %                  | (3 fields) - Recorded as %age |
MATERIAL

Material (2 fields) - As in Terrain Classification System

1 - Colluvial
2 - weathered bdrk
3 - Eolian
4 - Fluvial
5 - Fluvio-glacial
6 - Lacustrine
7 - Glaciolacustrine

- 8 - Morainal
- 9 - Organic
- 10 - Bedrock
- 11 - Volcanic
- 12 - Marine
- 13 - Glaciomarine

Texture (3 fields) - As Terrain Classification System

1 - blocks (a)
2 - boulders (b)
3 - cobbles (k)
4 - pebbles (p)
5 - sand (s)

- 6 - silt (s)
- 7 - clay (c)
- 8 - gravel (g)
- 9 - angular fragments (x)
- 10 - rubble (r)

Average Depth (3 fields) - Recorded as 0.0m

% (3 fields) - Recorded as % age

As per Terrain Classification System.

Examples of component % ages:

= 5:5  / = 6:2:2
/ 6:4  / / 5:3:2
// 8:2  // // 6:3:1
= = 4:3:3  // = 8:1:1
= / 4:4:2  // / 7:2:1
= // 5:4:1  // // 7:2:1
STABILITY

#nf, #ccf, #cbf, #rcf, #rff (1 field each)
0 - If not present
1 - If present

mnf, js, pb, lt, mccf, mgwf, tc, crp, (1 field each)
1 - if present
No entry if not present
### SLOPES

<table>
<thead>
<tr>
<th>Field</th>
<th>Description</th>
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<tbody>
<tr>
<td>Minimum slope</td>
<td>(2 fields) - Recorded in degrees</td>
</tr>
<tr>
<td>Maximum slope</td>
<td>&quot;</td>
</tr>
<tr>
<td>Average slope</td>
<td>&quot;</td>
</tr>
<tr>
<td>Aspect</td>
<td>(3 fields) - Recorded in degrees</td>
</tr>
<tr>
<td>Elevation</td>
<td>(4 fields) - Recorded in meters asl</td>
</tr>
<tr>
<td>Av. gully gradient</td>
<td>(2 fields) - Recorded in degrees</td>
</tr>
<tr>
<td>Av. gully wall</td>
<td>&quot;</td>
</tr>
<tr>
<td>Gully #</td>
<td>(2 fields) - Number of gullies in polygon</td>
</tr>
<tr>
<td>Av. gully depth</td>
<td>(2 fields) - Recorded in meters</td>
</tr>
<tr>
<td>Polygon area</td>
<td>Not recorded on form</td>
</tr>
<tr>
<td>Road length</td>
<td>Not recorded on form</td>
</tr>
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</table>

Not recorded on form
<p>| | |</p>
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<td>Road#</td>
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<td>Construction</td>
<td>Not recorded on form</td>
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<tr>
<td><strong>Type</strong></td>
<td>(1 field)</td>
</tr>
<tr>
<td></td>
<td>1 - full benched</td>
</tr>
<tr>
<td></td>
<td>2 - end haul</td>
</tr>
<tr>
<td></td>
<td>3 - back hoe</td>
</tr>
<tr>
<td><strong>Revegetation</strong></td>
<td>(1 field)</td>
</tr>
<tr>
<td></td>
<td>1 - yes</td>
</tr>
<tr>
<td></td>
<td>2 - no</td>
</tr>
<tr>
<td></td>
<td>3 - partial</td>
</tr>
<tr>
<td><strong>Ditches</strong></td>
<td>(1 field)</td>
</tr>
<tr>
<td></td>
<td>1 - yes</td>
</tr>
<tr>
<td></td>
<td>2 - no</td>
</tr>
<tr>
<td></td>
<td>3 - partial</td>
</tr>
<tr>
<td><strong>Berms</strong></td>
<td>(1 field)</td>
</tr>
<tr>
<td></td>
<td>1 - yes</td>
</tr>
<tr>
<td></td>
<td>2 - no</td>
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<td><strong>Culverts</strong></td>
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<tr>
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</tr>
<tr>
<td></td>
<td>2 - no</td>
</tr>
<tr>
<td></td>
<td>3 - partial</td>
</tr>
<tr>
<td></td>
<td>4 - squamish</td>
</tr>
<tr>
<td></td>
<td>5 - wood</td>
</tr>
<tr>
<td><strong>Surface erosion</strong></td>
<td>(1 field)</td>
</tr>
<tr>
<td></td>
<td>1 - neg</td>
</tr>
<tr>
<td></td>
<td>2 - rill</td>
</tr>
<tr>
<td></td>
<td>3 - gullied</td>
</tr>
<tr>
<td></td>
<td>4 - washed out</td>
</tr>
<tr>
<td><strong>Fillslope recovery</strong></td>
<td>(1 field)</td>
</tr>
<tr>
<td></td>
<td>1 - yes</td>
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<tr>
<td></td>
<td>2 - no</td>
</tr>
<tr>
<td><strong>Road gradient</strong></td>
<td>(2 fields) - Recorded in degrees</td>
</tr>
<tr>
<td>X Drains</td>
<td>(1 field)</td>
</tr>
<tr>
<td>----------</td>
<td>-----------</td>
</tr>
<tr>
<td>1 - yes</td>
<td></td>
</tr>
<tr>
<td>2 - no</td>
<td></td>
</tr>
<tr>
<td>3 - rare</td>
<td></td>
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</table>

<table>
<thead>
<tr>
<th>mrcf, mrf, tc</th>
<th>(1 field each)</th>
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<tbody>
<tr>
<td>0 - not present</td>
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</tr>
<tr>
<td>1 - present</td>
<td></td>
</tr>
<tr>
<td>VARIABLE: TERRAIN</td>
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</tr>
<tr>
<td>-------------------</td>
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</tr>
<tr>
<td><strong>DEEP</strong></td>
<td><strong>DEEP-SHALLOW</strong></td>
</tr>
<tr>
<td><strong>PURE</strong></td>
<td></td>
</tr>
<tr>
<td>Mb, Mh.</td>
<td>Mb / or // Mv</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Cb, Ca, Cf</td>
<td>Cb / or // Cv</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
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<td></td>
</tr>
<tr>
<td><strong>MIXTURES</strong></td>
<td></td>
</tr>
<tr>
<td>MbCb</td>
<td>Mb / or // Cv or Cv</td>
</tr>
<tr>
<td>CbMb</td>
<td>Cb / or // Mb or Mv</td>
</tr>
<tr>
<td>/ or // combinations</td>
<td>or stratigraphic combinations</td>
</tr>
<tr>
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<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>MbFG or MbF</td>
<td>Mb / or // R</td>
</tr>
<tr>
<td>FGMb or FMb</td>
<td>MbR,</td>
</tr>
<tr>
<td>/ or // combinations</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>stratigraphic combinations</td>
<td></td>
</tr>
<tr>
<td>minor components of Cv or Mv</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>note:</strong> 2 &amp; 3 may be combined</td>
<td></td>
</tr>
<tr>
<td>5 &amp; 6 may be combined</td>
<td></td>
</tr>
<tr>
<td>the same for 11 &amp; 12, 14 &amp; 15, 16 &amp; 17 and perhaps 18 &amp; 19, etc.</td>
<td></td>
</tr>
</tbody>
</table>

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APPENDIX D

INITIAL PARTIAL FACTOR, f

CONTINGENCY TABLES
### Terrain Unit

<table>
<thead>
<tr>
<th>Stable (Expected)</th>
<th>Unstable (Expected)</th>
<th>Stable (Observed)</th>
<th>Unstable (Observed)</th>
<th>Initial 'f'</th>
<th>Score</th>
<th>Attribute Code</th>
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<td>35.359</td>
<td>8.661</td>
<td>42</td>
<td>2</td>
<td>0.958</td>
<td>0.958</td>
<td>2</td>
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<tr>
<td>114.049</td>
<td>27.951</td>
<td>93</td>
<td>49</td>
<td>-0.936</td>
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<tr>
<td>141.356</td>
<td>34.644</td>
<td>154</td>
<td>22</td>
<td>0.454</td>
<td>0.454</td>
<td>15</td>
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<tr>
<td>24.898</td>
<td>6.102</td>
<td>26</td>
<td>5</td>
<td>0.225</td>
<td>0.225</td>
<td>10</td>
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<tr>
<td>48.993</td>
<td>12.007</td>
<td>51</td>
<td>10</td>
<td>0.208</td>
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<td>123.587</td>
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<td>122</td>
<td>32</td>
<td>-0.069</td>
<td>0.069</td>
<td>6</td>
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<td>23.292</td>
<td>5.708</td>
<td>23</td>
<td>6</td>
<td>-0.064</td>
<td>0.064</td>
<td>11</td>
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<tr>
<td>244.161</td>
<td>59.839</td>
<td>243</td>
<td>61</td>
<td>-0.024</td>
<td>0.024</td>
<td>1</td>
</tr>
<tr>
<td>128.506</td>
<td>31.494</td>
<td>128</td>
<td>32</td>
<td>-0.030</td>
<td>0.030</td>
<td>12</td>
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<tr>
<td>301.185</td>
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<td>301</td>
<td>74</td>
<td>-0.003</td>
<td>0.003</td>
<td>3</td>
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<td>2.409</td>
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### Chi-squared Significance

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### Number of Codes Added to Partial Factor, f

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<th>Cumulative Std. Dev. (Stable)</th>
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<th>Initial Ratio of Diff. of Means to Std. Error</th>
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### Coefficient Exponent

| Coefficient | Exponent | 85.54 | 0.511 | 80%    | 20%    | 100%   | 5.467 |

### Rank Amongst Other Attributes

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<th>Average Partial Factor, f</th>
<th>Average Partial Factor, f</th>
<th>Std. Deviation of Stable Polygons</th>
<th>Std. Deviation of Unstable Polygons</th>
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## Bedrock Lithology

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### Average Partial Factor, \( f \) of All Polygons and Standard Deviation

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<th>Unstable Percentage of All Coded Polygons</th>
<th>Coded Percentage of All Mapped Polygons</th>
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### Rank Amongst Other Attributes

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<th>Std. Deviation of Stable Polygons</th>
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## Hillslope Curvature

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<th>Cumulative Std. Dev. (Unstable)</th>
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<tr>
<th>Coefficient Exponent</th>
<th>Stable Percentage of All Coded Polygons</th>
<th>Unstable Percentage of All Coded Polygons</th>
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### Chi-squared Significance

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### Initial Ratio of Diff. of Means to Std. Error

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### Number of Codes Added to Partial Factor, f

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### Optimized Partial Factor, f

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### Optimized Ratio of Difference of Means to Std. Error

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242
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### Number of Codes Added to Partial Factor, f

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<th>Cumulative Mean (Stable)</th>
<th>Cumulative Mean (Unstable)</th>
<th>Cumulative Std. Dev. (Stable)</th>
<th>Cumulative Std. Dev. (Unstable)</th>
<th>Initial Ratio of Diff. of Means to Std. Error</th>
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### Coefficient Exponent

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<tr>
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<th>Stable Percentage of All Coded Polygons</th>
<th>Unstable Percentage of All Coded Polygons</th>
<th>Coded Percentage of All Mapped Polygons</th>
<th>Initial Ratio of Difference of Means to Std. Error</th>
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<td>20%</td>
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### Rank Amongst Other Attributes

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<th>Average Partial Factor, f, of All Unstable Polygons</th>
<th>Std. Deviation of Stable Polygons</th>
<th>Std. Deviation of Unstable Polygons</th>
<th>Optimized Ratio of Difference of Means to Std. Error</th>
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Drainage Class

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<th>Unstable</th>
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<th>Score</th>
<th>Attribute</th>
<th>Code</th>
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Chi-squared Optimized Partial Factor, f

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Number of Codes Added to Partial Factor, f

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<th>Cumulative Mean (Unstable)</th>
<th>Cumulative Std. Dev. (Stable)</th>
<th>Cumulative Std. Dev. (Unstable)</th>
<th>Initial Ratio of Diff. of Means to Std. Error</th>
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Coefficient Exponent

<table>
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<tr>
<th>Stable Percentage of All Coded Polygons</th>
<th>Unstable Percentage of All Coded Polygons</th>
<th>Coded Percentage of All Mapped Polygons</th>
<th>Initial Ratio of Difference of Means to Std. Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.000</td>
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<td>20%</td>
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Rank Amongst Other Attributes

<table>
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<tr>
<th>Average Partial Factor, f, of All Polygons</th>
<th>Average Partial Factor, f, of All Stable Polygons</th>
<th>Average Partial Factor, f, of All Unstable Polygons</th>
<th>Std. Deviation of Stable Polygons</th>
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<th>Optimized Ratio of Difference of Means to Std. Error</th>
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<th>Stable Percentage of All Coded Polygons</th>
<th>Unstable Percentage of All Coded Polygons</th>
<th>Coded Percentage of All Mapped Polygons</th>
<th>Initial Ratio of Difference of Means to Std. Error</th>
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<th>Average Partial Factor, ( f ), of All Stable Polygons</th>
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### Bedrock Formation

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</tbody>
</table>

### Chi-squared Optimized Attribute Significance Code

<table>
<thead>
<tr>
<th>Factor, f</th>
<th>Expected</th>
<th>Observed</th>
<th>Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.20E-03</td>
<td>0.8405</td>
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</tr>
<tr>
<td>2.62E-03</td>
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<tr>
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<tr>
<td>7.03E-03</td>
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<td>1</td>
<td></td>
</tr>
<tr>
<td>2.76E-05</td>
<td>0.0000</td>
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</table>

### Number of Codes Added to Partial Factor, f

<table>
<thead>
<tr>
<th>Average Partial Factor, f, of all Polygons</th>
<th>Cumulative Mean (Stable)</th>
<th>Cumulative Mean (Unstable)</th>
<th>Cumulative Std. Dev. (Stable)</th>
<th>Cumulative Std. Dev. (Unstable)</th>
<th>Initial Ratio of Diff. of Means to Std. Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.0679</td>
<td>0.0785</td>
<td>0.0219</td>
<td>0.2435</td>
<td>0.1333</td>
</tr>
<tr>
<td>3</td>
<td>0.0994</td>
<td>0.1115</td>
<td>0.0471</td>
<td>0.2470</td>
<td>0.1488</td>
</tr>
<tr>
<td>4</td>
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<td>0.0322</td>
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<td>0.2285</td>
<td>0.1877</td>
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<tr>
<td>5</td>
<td>0.0266</td>
<td>0.0409</td>
<td>-0.0351</td>
<td>0.2266</td>
<td>0.1904</td>
</tr>
<tr>
<td>6</td>
<td>0.0266</td>
<td>0.0409</td>
<td>-0.0351</td>
<td>0.2886</td>
<td>0.1904</td>
</tr>
<tr>
<td>7</td>
<td>0.0266</td>
<td>0.0409</td>
<td>-0.0351</td>
<td>0.2886</td>
<td>0.1904</td>
</tr>
<tr>
<td>8</td>
<td>0.0266</td>
<td>0.0409</td>
<td>-0.0351</td>
<td>0.2886</td>
<td>0.1904</td>
</tr>
<tr>
<td>9</td>
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<td>0.0409</td>
<td>-0.0351</td>
<td>0.2886</td>
<td>0.1904</td>
</tr>
<tr>
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<td>0.0266</td>
<td>0.0409</td>
<td>-0.0351</td>
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<td>0.1904</td>
</tr>
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</tr>
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<td>0.1904</td>
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<tr>
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<td>0.0409</td>
<td>-0.0351</td>
<td>0.2886</td>
<td>0.1904</td>
</tr>
</tbody>
</table>

### Coefficient Exponent

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Exponent</th>
<th>Stable Percentage of All Coded Polygons</th>
<th>Unstable Percentage of All Coded Polygons</th>
<th>Coded Percentage of All Mapped Polygons</th>
<th>Initial Ratio of Difference of Means to Std. Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.000</td>
<td>1.000</td>
<td>81%</td>
<td>19%</td>
<td>80%</td>
<td>5.095</td>
</tr>
</tbody>
</table>

### Rank Amongst Other Attributes

<table>
<thead>
<tr>
<th>Average Partial Factor, f, of All Polygons</th>
<th>Average Partial Factor, f, of All Stable Polygons</th>
<th>Average Partial Factor, f, of All Unstable Polygons</th>
<th>Std. Deviation of Stable Polygons</th>
<th>Std. Deviation of Unstable Polygons</th>
<th>Optimized Ratio of Difference of Means to Std. Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.099</td>
<td>0.111</td>
<td>0.047</td>
<td>0.247</td>
<td>0.149</td>
</tr>
</tbody>
</table>
### Polygon Elevation

<table>
<thead>
<tr>
<th>Stable (Expected)</th>
<th>Unstable (Expected)</th>
<th>Stable (Observed)</th>
<th>Unstable (Observed)</th>
<th>Initial 'T' Score</th>
<th>Score</th>
<th>Attribute Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>43.272</td>
<td>10.728</td>
<td>50</td>
<td>4</td>
<td>0.783</td>
<td>0.783</td>
<td>10</td>
</tr>
<tr>
<td>92.954</td>
<td>23.046</td>
<td>102</td>
<td>14</td>
<td>0.490</td>
<td>0.490</td>
<td>9</td>
</tr>
<tr>
<td>238.379</td>
<td>56.621</td>
<td>210</td>
<td>75</td>
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<tr>
<td>33.656</td>
<td>8.344</td>
<td>31</td>
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<tr>
<td>75.323</td>
<td>18.675</td>
<td>71</td>
<td>23</td>
<td>-0.289</td>
<td>0.289</td>
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<tr>
<td>10.332</td>
<td>4.768</td>
<td>20</td>
<td>4</td>
<td>0.201</td>
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</tr>
<tr>
<td>201.134</td>
<td>49.866</td>
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<td>44</td>
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<tr>
<td>208.346</td>
<td>51.634</td>
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<tr>
<td>115.391</td>
<td>28.609</td>
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<tr>
<td>177.094</td>
<td>43.096</td>
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<td>0.083</td>
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</tr>
<tr>
<td>11.219</td>
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<td>14</td>
<td>0</td>
<td>0.000</td>
<td>0.000</td>
<td>12</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Chi-squared</th>
<th>Optimized Partial Factor, f</th>
<th>Attribute Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.85E-03</td>
<td>0.7826</td>
<td>10</td>
</tr>
<tr>
<td>1.90E-04</td>
<td>0.4898</td>
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</tr>
<tr>
<td>4.01E-04</td>
<td>-0.4051</td>
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<tr>
<td>6.42E-04</td>
<td>-0.3972</td>
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<tr>
<td>1.48E-03</td>
<td>-0.2890</td>
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<td>2.29E-03</td>
<td>0.2010</td>
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<tr>
<td>3.35E-03</td>
<td>0.1468</td>
<td>8</td>
</tr>
<tr>
<td>6.05E-03</td>
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</tr>
<tr>
<td>9.92E-03</td>
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</tr>
<tr>
<td>5.05E-03</td>
<td>0.0000</td>
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</tr>
</tbody>
</table>

### Summary

- **Stable Unstable Coded Initial Ratio**
  - Percentage of Stable Polygons: 80%
  - Percentage of Unstable Polygons: 20%
  - Optimized Ratio of Difference of Means to Std. Error: 5.062

### Coefficient

- **Exponent:**
  - Stable Percentage of All Coded Polygons: 100%
  - Unstable Percentage of All Coded Polygons: 99%
  - Optimized Ratio of Difference of Means to Std. Error: 5.062

### Rank Amongst Other Attributes

- **Average Partial Factor, f, of All Polygons:**
  - Average Partial Factor, f, of All Stable Polygons: -0.035
  - Average Partial Factor, f, of All Unstable Polygons: -0.017
  - Std. Deviation of Stable Polygons: 0.300
  - Std. Deviation of Unstable Polygons: 0.262
  - Optimized Ratio of Difference of Means to Std. Error: 5.062

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