INVESTIGATION OF ROCK SLOPE DEFORMATION
AT THE WAHLEACH HYDROELECTRIC PROJECT
USING THE FLAC COMPUTER CODE

by

THOMAS W.G. STEWART
B.A.Sc., The University of British Columbia, 1987

A THESIS SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE OF
MASTER OF APPLIED SCIENCE

in

THE FACULTY OF GRADUATE STUDIES

(Department of Civil Engineering)
(Geotechnical Engineering Program)

We accept this thesis as conforming
to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

February 1997
© Thomas W.G. Stewart
In presenting this thesis in partial fulfilment of the requirements for an advanced
degree at the University of British Columbia, I agree that the Library shall make it
freely available for reference and study. I further agree that permission for extensive
copying of this thesis for scholarly purposes may be granted by the head of my
department or by his or her representatives. It is understood that copying or
publication of this thesis for financial gain shall not be allowed without my written
permission.

Department of CIVIL ENGINEERING

The University of British Columbia
Vancouver, Canada

Date 23-APRIL-1997.
ABSTRACT

Progressive deformation of the large natural rock slope at British Columbia Hydro's Wahleach hydroelectric project is an ongoing phenomenon. Slope movements have caused significant operational problems in the original power tunnels located within portions of the deforming slope, and pose a recognized hazard to property and facilities located at the base of the slope.

Developing a more comprehensive understanding of the rock slope deformation mechanics has been fundamental in evaluating the potential risk to facilities located at the base of the slope. This has been achieved through an extensive investigation and monitoring program, and supported by detailed numerical modelling studies presented in this report.

Numerical modelling with the finite difference FLAC computer code has utilized the extensive slope monitoring history developed between 1989 and 1994 to evaluate the model response. This has allowed the model to be used to evaluate the possible failure modes in the slope and to investigate the effects of future loading conditions such as earthquakes and extreme precipitation events, in addition to the potential long term evolution of the slope movement process.

Application of the FLAC code enabled a representative simulation of observed slope conditions to be made. Modelling results yielded excellent comparison with slope instrumentation data, developing confidence in the modelling capabilities to carry out predictive analyses of potential future loading conditions. The modelling indicated the importance of transient groundwater flow to the deformation process in the upper 40 to 70 metres of the rock mass.

B.C. Hydro recognized the potentially critical impact of adverse slope movement on the collection of hydroelectric, transportation and communication facilities located at the base of the Wahleach slope. Comprehensive investigative work, undertaken following the rupture of the steel lining of the power conduit, concluded that the nature of the slope movements indicated that a rapid failure mechanism was unlikely to develop. The modelling studies described herein provided further support that rapid slope failure is unlikely. Moreover, the numerical modelling provided important insight into the slope deformation mechanics, enhancing the understanding of this regionally important phenomenon.
TABLE OF CONTENTS

Abstract ........................................................................................................................................... ii
Acknowledgements .......................................................................................................................... xiv
Table of Contents ............................................................................................................................ iv
List of Figures .................................................................................................................................. viii
List of Tables .................................................................................................................................... xiii

Chapter 1  Introduction

1.1 Problem Description ................................................................................................................. 1
1.2 Project Justification .................................................................................................................. 1
1.2 Project Setting .......................................................................................................................... 3
1.2.1 Project Location .................................................................................................................... 3
1.2.2 Project History ...................................................................................................................... 3
1.4 Project Objectives .................................................................................................................... 4
1.5 Methodology ............................................................................................................................ 5

Chapter 2  Review of Literature

2.1 Introduction ............................................................................................................................... 9
2.2 Process Definition ...................................................................................................................... 10
2.3 History of Slow Deformation of Large Natural Mountain Slopes ........................................... 12
2.4 Origins of Large Scale Mountain Slope Deformation ............................................................... 14
2.4.1 Gravitational Forces ............................................................................................................ 15
2.4.2 Seismic Shaking .................................................................................................................... 15
2.4.3 Tectonic Origin .................................................................................................................... 16
2.4.4 Rebound/Stress Relief .......................................................................................................... 17
2.4.5 Physical Erosion Processes ................................................................................................. 18
2.4.6 Combined/Interacting Effects ............................................................................................. 19
2.5 Discussion of Deformation Mechanics .................................................................................... 20
2.5.1 Creep - Slow Plastic Flow ..................................................................................................... 22
2.5.2 Sliding Failure ....................................................................................................................... 23
2.5.3 Toppling Failure .................................................................................................................... 24
2.5.4 Buckling Failure .................................................................................................................... 27
2.5.5 Block Flow ............................................................................................................................ 28
2.5.6 Slope Deformation Involving Combined Mechanisms ......................................................... 29
2.6 Effects of Structural .................................................................................................................. 30
2.7 Deformation Rates ..................................................................................................................... 30
2.8 Numerical Modelling ................................................................................................................ 32
2.9 Uncertainties Surrounding Sackung Processes ........................................................................ 35
2.9.1 Mechanics of Deformation ................................................................................................. 35
<table>
<thead>
<tr>
<th>Chapter 2</th>
<th>Review of Literature (Continued)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.9.2</td>
<td>Depth of Deformation</td>
</tr>
<tr>
<td>2.10</td>
<td>Seismic Effects</td>
</tr>
<tr>
<td>2.11</td>
<td>Groundwater Effects</td>
</tr>
<tr>
<td>2.12</td>
<td>Potential for Sackung to Evolve Into Rapid Movement Processes</td>
</tr>
<tr>
<td>2.13</td>
<td>Deformation Chronology</td>
</tr>
<tr>
<td>2.14</td>
<td>Slope Monitoring</td>
</tr>
<tr>
<td>2.15</td>
<td>Trenching Investigations</td>
</tr>
<tr>
<td>2.16</td>
<td>Summary</td>
</tr>
<tr>
<td>2.17</td>
<td>Potential Study Areas</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Chapter 3</th>
<th>Geologic Setting</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Introduction</td>
</tr>
<tr>
<td>3.2</td>
<td>Regional Geology</td>
</tr>
<tr>
<td>3.3</td>
<td>Tectonic Setting</td>
</tr>
<tr>
<td>3.4</td>
<td>Site Geology</td>
</tr>
<tr>
<td>3.4.1</td>
<td>Rock Types</td>
</tr>
<tr>
<td>3.4.2</td>
<td>Structural Geology</td>
</tr>
<tr>
<td>3.4.3</td>
<td>Surficial Geology</td>
</tr>
<tr>
<td>3.5</td>
<td>Geomorphology</td>
</tr>
<tr>
<td>3.5.1</td>
<td>Physiography</td>
</tr>
<tr>
<td>3.5.2</td>
<td>Slope Morphology</td>
</tr>
<tr>
<td>3.6</td>
<td>Seismicity</td>
</tr>
<tr>
<td>3.7</td>
<td>Geologic Model for Numerical Modelling</td>
</tr>
<tr>
<td>3.7.1</td>
<td>Rock Mass Quality</td>
</tr>
<tr>
<td>3.7.2</td>
<td>Rock Mass Weathering</td>
</tr>
<tr>
<td>3.7.3</td>
<td>Rock Mass Structure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Chapter 4</th>
<th>Hydrogeology</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1</td>
<td>General Conditions</td>
</tr>
<tr>
<td>4.2</td>
<td>Piezometric Monitoring</td>
</tr>
<tr>
<td>4.3</td>
<td>Structural Control on Groundwater Flow</td>
</tr>
<tr>
<td>4.4</td>
<td>Conceptual Hydrogeologic model for modelling analysis</td>
</tr>
<tr>
<td>4.4.1</td>
<td>Slope Phreatic Surface</td>
</tr>
<tr>
<td>4.4.2</td>
<td>Perched Groundwater Conditions</td>
</tr>
<tr>
<td>4.4.3</td>
<td>Transient Groundwater Flow</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Chapter 5</th>
<th>Slope Movement Behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1</td>
<td>Introduction</td>
</tr>
<tr>
<td>5.2</td>
<td>Summary of Slope Deformation Behavior</td>
</tr>
<tr>
<td>5.2.1</td>
<td>Long Term Slope Behavior</td>
</tr>
<tr>
<td>5.2.2</td>
<td>Short Term Slope Behaviour (1952-1993)</td>
</tr>
</tbody>
</table>
Table of Contents (continued)

Chapter 5  Slope Movement Behaviour (continued)

5.2.3 Recent Slope Behavior (1989-1994) .............................................................. 79
5.3 Interpretation of Slope Deformation Mechanics.............................................. 80

Chapter 6  Numerical Modelling

6.1 Introduction ........................................................................................................ 87
6.2 Wahleach Rock Slope Numerical Model ............................................................ 87
   6.2.1 Continuum vs. Discontinuum Approach ................................................... 88
   6.2.2 Two-Dimensional vs. Three-Dimensional Approach ............................... 89
6.3 Material Behaviour ............................................................................................ 91
   6.3.1 Mechanics of Deformation of Rock Slopes .............................................. 91
   6.3.2 Stress-Strain Relationships .................................................................. 94
   6.3.3 Material Strength .................................................................................. 96
6.4 Available Numerical Methods ......................................................................... 97
   6.4.1 Limit Equilibrium Techniques ................................................................ 97
   6.4.2 Implicit Methods ................................................................................... 98
   6.4.3 Explicit Methods .................................................................................. 100
   6.4.4 Overview .............................................................................................. 101
6.5 FLAC Computer Code ...................................................................................... 101
   6.5.1 General ................................................................................................. 101
   6.5.2 Details of Numerical Code .................................................................... 102
   6.5.3 Verification of FLAC Code .................................................................... 104
   6.5.4 Material Behaviour ............................................................................... 105
6.6 Dynamic Modelling ........................................................................................ .. 106
   6.6.1 Dynamic Formulation .......................................................................... 106
   6.6.2 Dynamic Boundary Conditions ............................................................. 108

Chapter 7  Wahleach FLAC Model

7.1 Introduction ....................................................................................................... 116
7.2 General Modelling Methodology ..................................................................... 116
7.3 Model Calibration and Evaluation .................................................................. 118
7.4 Construction of FLAC Model ......................................................................... 121
7.5 Boundary Conditions ...................................................................................... 123
7.6 Loading Sequence for Calibrative Modelling .................................................. 125
   7.6.1 Initial Conditions ................................................................................... 126
   7.6.2 Slope Formation .................................................................................... 129
   7.6.3 Rock Mass Weathering ......................................................................... 129
   7.6.4 Transient Groundwater Conditions ....................................................... 130
   7.6.5 Lining Break Event ............................................................................. 133
   7.6.6 Post-Lining Break Behaviour ............................................................... 135
7.7 Predictive Modelling ........................................................................................ 136
   7.7.1 Seismic Loading .................................................................................... 136
   7.7.2 Extreme Groundwater Conditions ....................................................... 139
Table of Contents (continued)

Chapter 7  Wahleach FLAC Model (continued)

7.7.3 Long Term Evolution of the Slope .............................................. 140

Chapter 8  Discussion of Modelling Results

8.1 Overview .................................................................................. 160
8.2 Initial Conditions ...................................................................... 161
8.3 Slope Formation ........................................................................ 161
8.4 Rock Mass Weathering .............................................................. 163
8.5 Transient Groundwater Conditions .......................................... 164
8.6 Lining Break Event ................................................................. 167
8.7 Post-Lining Break Behaviour ................................................... 168
8.8 Predictive Modelling ............................................................... 172
  8.8.1 Seismic Loading ................................................................. 172
  8.8.2 Progressive Increase in Phreatic Surface .............................. 176
  8.8.3 Long Term Effects of Transient Groundwater Conditions .... 177
8.9 Summary of Modelling Analyses .............................................. 179

Chapter 9  Conclusions .................................................................... 241

References ..................................................................................... 243

Appendices

A Rock Mass Characterization (Hoek-Brown, 1996)
B Rock Structure Characterization (Barton, 1987)
C Affliction Creek Modelling Summary Report
D Modelling Details - Input Data Files (FLAC, UDEC)
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>Project Location Plan.</td>
</tr>
<tr>
<td>1-2</td>
<td>Project cross section showing general arrangement and original (1951) tunnel alignment.</td>
</tr>
<tr>
<td>3-1</td>
<td>Geological Interpretation Plan.</td>
</tr>
<tr>
<td>3-2</td>
<td>Slope cross section showing original tunnel alignment and important geologic features.</td>
</tr>
<tr>
<td>3-4</td>
<td>Summary drillhole log DH89-S1. Note the progressive improvement in rock mass conditions below the surface. From B.C. Hydro Report No. H2780.</td>
</tr>
<tr>
<td>3-5</td>
<td>Idealized stress-strain response for intact rock and jointed rock.</td>
</tr>
<tr>
<td>3-6</td>
<td>Distribution of rock mass conditions in upper slope.</td>
</tr>
<tr>
<td>4-1</td>
<td>Phreatic surface used in FLAC model for calibrative modelling sequence.</td>
</tr>
<tr>
<td>4-2a</td>
<td>Anisotropic groundwater conditions created by rock mass structure.</td>
</tr>
<tr>
<td>4-2b</td>
<td>Structurally controlled porosity/permeability in rock mass leading to large variation in groundwater levels following significant precipitation.</td>
</tr>
<tr>
<td>4-3</td>
<td>Lowered phreatic surface due to drainage into unlined section of intermediate tunnel, corresponding to post 1992 conditions.</td>
</tr>
<tr>
<td>4-4</td>
<td>Perched groundwater conditions registered in MP89-S4.</td>
</tr>
<tr>
<td>4-5</td>
<td>Simulation of transient groundwater conditions, for model input.</td>
</tr>
<tr>
<td>5-1</td>
<td>Site geology plan, showing slope morphology (linears), and instrumentation layout.</td>
</tr>
<tr>
<td>5-2</td>
<td>Upper tunnel and incline offset survey, showing accumulated deformation of the steel lining in the original tunnel.</td>
</tr>
<tr>
<td>5-3</td>
<td>Slope cross section showing distribution of current movements.</td>
</tr>
<tr>
<td>5-4</td>
<td>Plot of inclinometer deformation rates.</td>
</tr>
<tr>
<td>5-5</td>
<td>Movement time plot for slope extensometers in region of lining break.</td>
</tr>
<tr>
<td>6-1</td>
<td>The basic types of movement of materials.</td>
</tr>
<tr>
<td>6-2</td>
<td>Simplified models for mathematical simulation of material behaviour.</td>
</tr>
<tr>
<td>6-3</td>
<td>Strength envelope for the Wahleach rock mass based on the Hoek-Brown criterion.</td>
</tr>
<tr>
<td>6-4a</td>
<td>Simple shear loading response of single zone model (ubiquitous joint model).</td>
</tr>
</tbody>
</table>
6-4b Stress-strain response of single zone ubiquitous joint model, plotted as shear stress, $\tau$, versus shear strain, $\gamma$.

6-5 Plot of acceleration response spectrum for the Northridge (1994) event, as evaluated by the FLAC analysis.

6-6 Fast Fourier Transform plot of Northridge (1994) event, plotted as energy versus frequency.

7-1a Plot of principal stresses in model showing successive equilibrium states in response to slope formation.

7-1b Model stress response during rock mass weathering, with stress reductions related to material yield and associated stress redistribution.

7-2 Plot of model maximum unbalanced force in response to change in material properties during rock mass weathering.

7-3a First generation grid (March, 1995).

7-3b Second generation grid (August, 1995).

7-3c Third generation grid with sloping base.

7-3d Fourth generation grid.

7-3e Fifth generation grid used for detailed modelling study.

7-4 Plot of model interfaces.

7-5a Short, coarse grid used for comparison of boundary effects.

7-5b Long, coarse grid used for comparison of boundary effects.

7-6 Recommendations for model slope set-up.

7-7 Plot of fixed boundary conditions.

7-8 Rock mass conditions for which the Hoek-Brown failure criterion can be applied.

7-9 Representation of material dilation angle as determined from shear testing.

7-10 Plot of current slope profile (slope formation).

7-11 Plot of transient groundwater effects, modelled as seepage forces, in the vadose zone.

7-12 Model simulation of lining break event, showing localized increase in groundwater level and applied seepage force distribution.

7-13 Phreatic surface in slope, reflecting drainage into downstream end of lower tunnel, following 1992 realignment.

7-14 Response spectra comparison for Northridge Earthquake, 1994.

7-15 Probabilistic estimates of peak ground accelerations for Wahleach area.
Comparison of vertical stress levels ($\sigma_{yy}$) with major principal stress levels ($\sigma_1$) prior to slope formation in the initial block model.

Stress levels at centre of model following equilibration of initial conditions.

Plot of maximum unbalanced model force during equilibration of initial conditions.

Horizontal model velocity along slope surface in initial block.

Attenuation of model displacements (upper slope area) in initial block, as stable equilibrium conditions are established.

Model displacement vectors developed during consolidation of initial stress levels to the initial equilibrium state.

Pattern of principle stress vectors in model slope at end of slope formation.

Plot of principal stress vectors following slope formation.

Model displacement vectors in elastic model, showing upward rebound in response to overburden removal.

Model displacements vectors in ubiquitous joint model, showing upward rebound in response to overburden removal.

Formation of shear stresses in model during slope formation.

Formation of zone of tensile stress at ridge crest.

Plasticity indicator in model slope, highlighting the area where temporary yield occurred in conjunction with rock mass weathering.

Reduction in shear ($\tau_{xy}$) and maximum principal stress ($\sigma_1$) levels close to the slope surface due to rock mass weathering.

Shear stress redistribution corresponding to rock mass weathering and related material yield.

Surficial slope movements during rock mass weathering.

Horizontal displacement contours during rock mass weathering, outlining the region of yield related to reduced strength values.

Maximum unbalanced force in model during rock mass weathering.

Horizontal velocity along model slope surface (S7 area) during rock mass weathering.

Out-of-plane stress ($\sigma_{zz}$) correction during rock mass weathering, maintaining the out-of-plane stresses as the intermediate stress.

Model plasticity plot during application of transient groundwater effects, highlighting the zone where yield and flow (deformation) occurred.
Detailed model plasticity plot following application of one cycle of transient groundwater conditions, illustrating that a stable equilibrium (elastic) condition was reestablished.

History of model surface velocity during transient groundwater conditions.

Plot of model grid (magnified) highlighting the formation of slope deformation features, following successive cycles of transient groundwater conditions.

Contour plot of model shear strain increment developed during the application of transient groundwater flow cycles.

Model displacement vectors resulting from 38 cycles of transient groundwater conditions.

History of model surficial displacement (S1 area), highlighting the episodic/cyclic deformation process triggered by the transient groundwater conditions.

History of model surficial displacement (S1 area), plotted with a wider recording interval that masks the cyclic movement pulses, suggesting creep.

Model displacement vectors developed by the transient groundwater conditions in the vicinity of the upper tunnel steel lining break.

History of maximum unbalanced force in model, displaying the large disturbance created by the lining break event.

Model surface velocity history showing the larger response to the lining break event conditions relative to the "seasonal" transient groundwater conditions.

Comparison of measured lining break displacements with model simulation.

Model displacement vectors developed during the lining break event.

Model velocity vectors during lining break event.

Model plasticity indicator during lining break event.

Model shear strain increment during lining break event.

Model gridpoint velocity history along slope surface covering lining break and post-lining break conditions.

Plot of horizontal displacement contours covering the post-lining break response.

Model displacements in relation to original upper power tunnel and the break in the steel lining.

Model inclinometer S-10, showing post lining break response.

Model inclinometer S-7, showing post lining break response.
8-38 Model inclinometer S-1, showing post lining break response.
8-39 Model simulation of strain meter located at lining break (UTSM-9).
8-40 Post-lining break model displacements in the vicinity of the upper tunnel and inclined shaft.
8-41 Magnified grid plot following lining break simulation.
8-42 Detail of surface displacement response during post-lining break simulation, highlighting the upslope propagation of yield and plastic deformation.
8-43 Acceleration time history for 0.01g event.
8-44 Surface displacement response in middle portion of deforming zone (S7 area) in response to Duvall earthquake.
8-45 Acceleration time history for 0.19g case (1/1000 year event).
8-46 Model displacement histories along slope surface in response to 0.19g case.
8-47 Horizontal displacement contours in model generated during 0.19g event.
8-48 Distribution of shear strain increment resulting from 0.19g case.
8-49 Displacement response to 0.41g event at S1 area.
8-50 Acceleration time history for 0.41g event.
8-51 Plot of ubiquitous joint angle following 0.41g event, highlighting the extent of the disturbed zone.
8-52 Model shear stress response in deforming zone (S10 area, 50-70 m depth), during 0.41g event.
8-53 Model surface velocity history at S7 during 0.41g shaking.
8-54 Model horizontal displacement contours generated during 0.41g shaking.
8-55 Model acceleration levels along the slope surface during 0.41g event, illustrating the amplification of ground motions towards the upper slope area.
8-56 Onset of steady state plastic flow in model, corresponding to a 58 metre increase in phreatic surface above currently monitored levels.
8-57 Model plasticity plot corresponding to critical increase in phreatic surface.
8-58 Model displacement vectors during long term evaluation (1000 cycles of transient groundwater conditions in the vadose zone).
8-59 Incremental displacements along model slope surface in S10 area, outlining the incremental plastic deformation developed with each loading cycle of transient flow.
8-60 Model displacement response (S10 surface area) following 1000 cycles of transient groundwater conditions.

8-61 Rotation of ubiquitous joint structure in deforming section of slope during long term transient conditions (1000 cycles).

8-62 Plot of shear strain increment developed during 1000 cycles of transient groundwater conditions.

List of Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Table Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-1</td>
<td>Occurrences of toppling failures and Sackung cases.</td>
<td>25</td>
</tr>
<tr>
<td>2-2</td>
<td>Reported deformation rates in Sackung type cases.</td>
<td>31</td>
</tr>
<tr>
<td>3-1</td>
<td>Rock discontinuity characteristics at surface and 200 m depth.</td>
<td>50</td>
</tr>
<tr>
<td>7-1</td>
<td>Comparison of model response - Different Boundary Locations.</td>
<td>124</td>
</tr>
<tr>
<td>7-2</td>
<td>Rock mass properties for model input.</td>
<td>127</td>
</tr>
<tr>
<td>7-3</td>
<td>Rock joint shear strength properties for model input.</td>
<td>128</td>
</tr>
<tr>
<td>7-4</td>
<td>Details of Northridge Earthquake (1994), at Pacoima Dam.</td>
<td>138</td>
</tr>
<tr>
<td>8-1</td>
<td>Earthquake acceleration levels for the Wahleach Slope area.</td>
<td>174</td>
</tr>
<tr>
<td>8-2</td>
<td>Summary of FLAC modelling analyses.</td>
<td>179</td>
</tr>
</tbody>
</table>
ACKNOWLEDGEMENTS

This project was made possible through support from B.C. Hydro, and the efforts of many people. I would like to thank Dr. Peter Byrne for providing the opportunity to undertake this project through the Department of Civil Engineering, and for his excellent teaching. Dr. K. Wayne Savigny provided much helpful advice, encouragement and perspective for the project work. Large thanks are due to Dr. Michael J. Bovis for providing the opportunity to become involved in his on-going research of large natural slopes, and for the opportunity to visit the Affliction Creek and Mt. Currie sites.

At B.C. Hydro I would like to thank Al Imrie and Dennis Moore for support and encouragement in getting the project started. Dr. Bruce Ripley played a huge role in the project work, and was always available to provide discussion and answer on-going questions, for which I am extremely thankful. Dr. Evert Hoek was involved in the project review, but went way beyond the call of duty by providing many hours of his extremely busy schedule to answer questions and provide interesting discussion.

Sincere thanks are due to my families (Stewart and Willis) for their unwaivering understanding, patience and support along the way. To my wife Sally, an acknowledgement here does not do justice for the support she has provided; her name belongs on the front page.
Chapter 1 - Introduction

1.1 Problem Description

The large natural rock slope above the Wahleach Generating Station has experienced a slow, progressive mass movement phenomenon now considered to occur in many orogenic belts. Geomorphic evidence indicates that progressive slope movements have been active for a minimum of several hundred years, and more likely throughout the Holocene. An extensive instrumentation array, installed to monitor slope behaviour following the 1989 rupture of a steel lined power tunnel, has confirmed the ongoing nature of the slope movements. The successful completion of the hydroelectric project, and the fact that nearly forty years passed before the significant failure of the power conduit lining, attests to the slow nature of the deformation process. The original selection of tunnel alignment reflects an unfamiliarity with the deformation process and the symptomatic landforms present along the slope surface. This is not meant to be critical of the initial design process, instead it is intended to indicate the relatively recent recognition of this mass movement process.

It was only several years prior to the initial 1951 project construction that Stini (1942) presented some of the earliest discussions of this process, although it took more than 30 years before an English translation of this work was available (Müller, 1974). A regional awareness of this slow slope movement process only became apparent following the works of Radbruch-Hall et al. (1976) and Bovis (1982), despite the fact that many civil engineering projects were built on or in slopes where this process has been, or is currently active. The Wahleach project presents an excellent opportunity to investigate this mass movement process in detail, to incorporate sophisticated numerical modelling techniques to develop a more comprehensive understanding of this process, and to develop predictive capabilities of future loading conditions and potential slope behaviour.

1.2 Project Justification

The safety implications of potential large rock slope failure at a number of hydroelectric projects in British Columbia has given cause for B.C. Hydro to undertake comprehensive studies of this phenomenon. These studies have included extensive surface and subsurface investigation and monitoring programs of several large natural rock slopes undergoing active but slow
deformation. The information obtained during these studies, particularly if integrated with numerical modelling studies, could provide a unique analytical approach for further investigation of this phenomena. The need for such an approach can be realized in light of the fact that no detailed study of this large scale process has incorporated detailed geologic information and slope monitoring data, in conjunction with numerical modelling studies. The Wahleach project is but one of several cases related to hydroelectric projects, but provides the most comprehensive investigation and instrumentation information on which to base an investigation of this phenomenon.

Previous studies of large scale rock slope deformation have generally adopted a qualitative approach. Much of this work has involved the recognition and description of geomorphic features considered to be representative of the slow deformation of natural mountain slopes. However, despite these research efforts this slope movement process remains poorly understood, and in particular, the nature of initiating and driving forces, kinematics, material behaviour and long term development remain unresolved.

A more comprehensive understanding of this slope movement process has been hindered by a number of contributing factors. Foremost is the characteristic slow rate of the process, typically considered to be only millimetres to centimetres per year. Consequently, it may take many years for visual changes to manifest themselves. Therefore, observational studies must adopt a patient, long term approach, and must additionally include some means of distinguishing whether the movements occur in a steady state fashion, or are episodic in nature. Secondly, the large scale of the process, frequently distributed over several square kilometres, has precluded comprehensive investigative efforts and consequently limited most studies to surficial investigations of geologic and geomorphic features. Subsurface information from tunnels, excavations or boreholes has rarely been available, therefore, important details of the subsurface geologic and hydrogeologic conditions can only be inferred from surficial observations.

Developing a clearer picture of the process requires the integration of detailed instrumentation of slope movement behaviour. Effective instrumentation must be suitably extensive and carried out for long enough periods to establish important trends. With some knowledge of the historical slope behaviour, appropriate numerical analyses can then be utilized to further investigate the potential material behaviour under a range of loading conditions. The choice of numerical analysis method is considered to be fundamental to the success of the study,
since the use of inappropriate techniques preclude a representative evaluation of the material behaviour. Specifically, the use of simplistic limit equilibrium techniques would not be effective in capturing the complex deformation mechanisms of these large processes.

To date, no investigation of this mass movement process has been able to assimilate the required input to treat this problem in detail. The Wahleach project, however, presents the unique opportunity to integrate a detailed surface and subsurface investigation program, including details of slope movement and groundwater behaviour, with state-of-the-art numerical modelling. This combination will allow a new perspective to be taken in the analysis of this problem.

1.3 Project Setting
1.3.1 Project Location

The Wahleach rock slope is located along the southeastern side of the Fraser River, approximately halfway between the towns of Chilliwack and Hope, British Columbia, some 120 kilometres east of Vancouver, as shown on Figure 1-1.

1.3.2 Project History

The original power conduit for the project was constructed between 1951 and 1952, and operated without incident until 1981 at which time leakage from the upper tunnel was traced to drainage pipes connected to the upper tunnel rock trap (Fig. 1-2). It was presumed that the problem resulted from corrosion of the pipes, and no connection between the leakage and slope movements was made at that time. This condition was successfully repaired, and the tunnels operated successfully until January 1989. Around January 20th flows were observed at the base of the slope that were traced to leakage from the upper tunnel access adit. Upon inspection, a break in the steel lining was observed approximately 30 metres above the downstream end of the upper tunnel (Fig. 1-2). A detailed field investigation began immediately that included the installation of instrumentation providing continuous monitoring of the slope. Conclusions of the investigative work concluded that the lining break, in addition to several other deformed sections of the steel lining, were the result of slope movements. The power conduit has subsequently been relocated to a deeper alignment in the slope, well removed from the deforming rock mass. Continued monitoring of the slope instruments has confirmed the ongoing nature of the deformation process.
1.4 Project Objectives

The primary objective of this project was to develop a more comprehensive understanding of the deformation behaviour and failure mechanisms prevalent in the Wahleach rock slope, through the use of detailed numerical modelling. This knowledge would be valuable in the investigation of other slowly deforming rock slopes. Incorporated within this overall goal were several specific objectives, related to the use of numerical modelling studies, but also focused towards addressing several outstanding issues concerning this poorly understood natural process:

1/ Could the numerical modelling be used to investigate the sensitivity of the slope to the various initiating and driving forces such as the effects of transient groundwater flow or seismic loading?

2/ Could a further understanding of the deformation modes and mechanisms be gained by the modelling? Do model observations provide insight into presently occurring slope behaviour with subtleties that preclude direct observation? For example, do particular sections of the slope display more acute instability conditions that result in subsequent deformation of adjacent areas through the process of stress-strain redistribution?

3/ Could the relationship between the widespread geomorphic features and these slow mass movement process be established with more certainty?

4/ Could the long term slope behaviour be investigated further? Do these slow movement processes maintain current movement trends, self-stabilize, or evolve into potentially rapid movement mechanisms?

5/ Can the modelling approach be used to investigate other similarly behaving rock slopes?

To address these questions, detailed numerical modelling studies using the finite difference computer code FLAC, were designed and carried out for the Wahleach rock slope. Specific objectives related to the numerical modelling were developed to accommodate the specific conditions prevalent in large natural rock slopes such as Wahleach. Foremost among these objectives was demonstrating that the FLAC program was well suited to the specific analytical requirements unique to large rock slopes. Subsequently, the project centred around interpretation of the FLAC modelling results to investigate the issues outlined above, and for the evaluation of future slope behaviour.
1.5 Methodology

The project methodology was developed around the construction of a numerical model of the Wahleach rock slope to be analyzed with the FLAC computer code. The general framework developed for this investigative modelling work included the following:

1/ Development of a practical and representative procedure for modelling large rock slopes under a range of loading conditions.

2/ Incorporation of all available geologic, geomorphic, hydrologic and slope movement data from the Wahleach project into a conceptual model that could be effectively used in the modelling analysis.

3/ Investigation of the model deformation behaviour during static loading of the model slope, including the following cases:
   (a) Construction of a representative model based on physically appropriate initial conditions.
   (b) Progressive development of the current slope profile, through a simplified "erosion" process.
   (c) Progressive weathering of the rock mass to model the physical and chemical degradation of the rock mass, and to establish a state of marginal instability reflective of the current slope behaviour.
   (d) Investigation of variable groundwater conditions, with particular focus on groundwater flow in the vadose zone, where slope movements have been recorded.

4/ Development of an appropriate technique for modelling:
   (a) The effects of transient groundwater conditions.
   (b) The lining break event and post-lining break behaviour.

5/ Calibration of the model slope response against the monitored slope behaviour, to provide a quantitative evaluation of the model's effectiveness in capturing the salient features of the slope deformation process.

6/ Dynamic analyses of the calibrated model to investigate the effects of earthquake loading on the slope. This was based on the selection of an appropriate earthquake time history for the project area.
7/ Investigation of extreme groundwater conditions and long term evolution of the slope.
8/ Interpretation of the model response to the above conditions, in order to establish a more
detailed understanding of the slope deformation behaviour.
9/ Investigation of long term loading of the model slope to investigate the potential
evolutionary development of the slope behaviour.
Figure 1-1: Project Location Plan. From B.C. Hydro Report No. H2135.
Figure 1-2: Project cross section showing original (1951) tunnel alignment. From B.C Hydro Report No. H2135.
Chapter 2 - Review of Literature

2.1 Introduction

Slow deformation of large natural rock slopes has been identified as a regionally significant phenomenon throughout mountainous areas of the world, occurring in a wide variety of geologic settings. Considerable investigation of this diverse process has been undertaken during the last thirty years, yet a deficient understanding of the origins and governing deformation mechanics still persists.

A wide range in terminology has been used to describe the general process, but "sackung," translated as sagging, proposed by Zischinsky (1966) has been the most widely used term. Henceforth, this term will be used in a general connotation without rigorous specification to the actual mechanics and mode of origin.

Recognition of sackung processes has chiefly been based on the discovery of anomalous geomorphologic features on slopes. These geomorphic features typically have been considered symptomatic of internal rock slope deformation, however details regarding the timing and mode of development have been enigmatic due to the characteristic slow process rates and notable lack of quantitative evidence.

The morphological expression of slope movements have long been recognized, described and analyzed. Additionally, many of the fundamental slope movement processes, such as sliding, have undergone extensive field investigation, analysis and model study. This has developed a basic understanding of the deformation behaviour governing these mass movement processes, and with it, the capacity to utilize representative analytical methods for investigation and design. The classification schemes presented by Sharpe (1960), Varnes (1975), Nemčok (1972) and Hutchinson (1988) reflect the progressional understanding of slope movement processes, and the associated geomorphology.

The slow deformation of large mountain slopes stands out in that it is perhaps the least understood slope movement phenomenon. Despite some thirty years of general study a sketchy understanding of this general phenomenon still persists (Terzaghi, 1950; Radbruch-Hall, 1978; McAlpin and Irvine, 1994). The obscurity surrounding this process undoubtedly reflects the large scale, complex behaviour and slow rate of the phenomenon, which, until now, has precluded a comprehensive and quantitative study at an appropriate field scale.

Early research efforts generally focused on the identification and description of the
anomalous slope landforms, and their presumed correlation with large scale slope deformation processes. Consequently, a wide range of literature exists outlining the descriptive aspects of this process. The work of Radbruch-Hall (1978) provided the most comprehensive presentation of this topic to date.

Although this review of literature addresses both the qualitative and quantitative aspects of large scale mountain slope deformation, emphasis was placed on the comparatively undeveloped quantitative aspects. Specifically, this involved a search for cases presenting subsurface investigations, slope instrumentation details, including surface or subsurface methods, the use of detailed surface trenching investigations, or cases involving computer modelling analyses. In addition to a review of documented studies, information and experience has also been acquired during field studies at the following sites over the course of this thesis project: Wahleach rock slope (Agassiz, B.C.); Checkerboard Creek (Revelstoke, B.C.); Affliction Creek and Mt. Currie (Pemberton, B.C.).

2.2 Process Definition

In the context of this discussion, large scale mountain slope deformation refers to the slow, progressive deformation of large rock slopes, wherein the movements are characteristically dispersed throughout the deforming material, and a combination of deformation mechanisms may have prevailed. This definition follows that proposed by Zischinsky (1966,1969), who used the term "sackung" to describe this slow deformation process, and is considered comparable to the slow mountain slope deformation phenomena described by Stini (1941) as "slope creep" or "valley closure". Zischinsky's description of the process outlined movement along a complex of shear zones extending into a central zone dominated by slow dispersed plastic deformation. In this model the extent of internal deformation and development of associated surficial distortion significantly exceeded deformation along any discrete shearing surface within the rock mass. Zischinsky's general concept has been considered in many of the slope deformation studies following his work (Radbruch-Hall, 1978; Bovis, 1982,1990; Beget, 1985; Varnes et al., 1989; McAlpin and Irvine, 1994).

Large scale mountain slope deformation has been termed "valley creep" (Stini, 1942); "deep-seated rock mass creep" (Terzaghi, 1950), "sackung" (Zischinsky, 1966), "deep-seated continuous creep" (Hutchinson, 1968), "depth creep" (Ter-Stepanian, 1969), "deep-seated creep"
(Nemčok 1972), "mass rock creep" (Radbruch-Hall, 1977; and Chigira, 1992), and "rock mass creep" (Beget, 1985). "Creep" has been incorporated in most descriptions of the process, however, without appropriate monitoring of in situ conditions it would difficult to ascertain whether true creep processes were predominant.

A variety of surficial morphological features have been identified which are considered to be characteristic "signatures" of sackung. "Doppelgrat" (or double ridges), "ridge trenches" and "slope trenches" (Jahn, 1964), "sackungen" (Zischinsky, 1966), "troughs" and "benches" (Tabor, 1971), "uphill facing scarps" (Radbruch-Hall et al., 1976), "anti-slope scarps" (Bovis, 1982), and "linears" (Moore et al., 1992) comprised some of the many names used to describe the anomalous slope morphology commonly associated with this slow deformation process.

Zischinsky (1966, 1969) considered gravitational stresses to provide the primary driving force for this mass movement process; a concept further presented in the majority of other prominent research efforts (Ter-Stepanian, 1969; Tabor, 1971; Nemčok 1972; Radbruch-Hall et al., 1976; Mahr, 1977; Bovis, 1982; Varnes et al., 1989). Other driving forces for the origin of this mass movement process have been proposed, including tectonic faulting (Solonenko, 1972; Plafker, 1967; Eisbacher, 1983); seismic triggering (Beck, 1968; Dowrenwend et al., 1978; Clague, 1979); isostatic rebound/strain recovery (Jackli, 1965; Patton and Hendron, 1974; and Bordonau and Vilaplana, 1986); and physical erosion (Paschinger, 1928; Mylrea, 1969).

Slow deformation of large natural slopes is a widespread process, and case examples have been reported from most of the mountainous regions of the world. The correspondence of the major mountain belts with current tectonic plate boundaries, where seismic activity has prevailed, provided some justification for the proposed seismic initiation of sackung processes. However, cases have been reported in regions where seismicity is not considered to be currently significant (ie. Scotland, Holmes and Jarvis, 1985).

A range of potential deformation modes have been proposed to account for the observed slope deformation morphology, including; creep, plastic flow, sliding, toppling, buckling and various combinations of the above. Many studies have included a postulated relationship between the deformation mechanism(s) and the observed geologic/geomorphic conditions. Developing a clearer understanding of the potential material behaviour requires support from instrumented slopes to clarify the nature of the processes. Until such cases are reported, a speculative presentation of the slope behaviour will be the standard.
2.3 History of Slow Deformation of Large Natural Mountain Slopes

A written history of this process has been traced back to 1807 when Zay provided a discussion of the large catastrophic Goldau rockslide that occurred in the Swiss Alps. Creep movements were reported to have occurred in the twenty years prior to the slide. Of particular importance in this case was the recognition of slow progressive mountain slope deformation preceding the catastrophic failure. Similar creep type behaviour was also reported to have preceded the great Frank slide of 1903 in the Crownsnest Pass based on the 1904 study by McConnell and Brock, as summarized by Terzaghi (1950).

Investigation of this phenomenon over the last 50 years has largely grown from European research. Stini (1942-1952) published several accounts of this phenomenon, stressing the wide spread occurrence and range of potential mechanisms. Jahn (1964) provided a discussion of the geomorphic features that he considered to be associated with large scale slope deformation in the Tatras and Carpathian mountains of eastern Europe. Of particular importance in Jahn's work was the proposed relationship between certain geomorphic features and structural geology in relation to the slow mountain slope deformation. Zischinsky (1966) presented an investigation of large scale slope deformation in the Austrian Alps, that proposed a correlation between slope geomorphic features, a causative process (gravity) and deformation mechanisms. This work has frequently been referenced in subsequent studies as it introduced discussion of the governing deformation mechanics of the process, included a descriptive discussion of the material behaviour, and provided illustrative cross section diagrams of the general process. Subsequent works have often adopted Zischinsky's terminology and hypothesized deformation mechanisms.

Beck (1968) discussed deep-seated gravitational deformation of mountain slopes in the New Zealand Alps, and presented a series of schematic cross sections to explain the potential mode(s) of deformation. Although Beck's proposed model of "gravitational-faulting" has not been generally adopted in subsequent studies, his schematic drawings have often been included in summary works of this process.

The first North American documentation of this process was reported by Tabor (1971). Geomorphic features, described as "ridge-top depressions" and "troughs", observed in the Olympic Range (Washington State), were attributed to large scale gravitational creep. Tabor's discussion emphasized the relationship between geologic conditions (lithology and structure) and the resultant geomorphic features developed by the structurally controlled flexural deformation.
Radbruch-Hall et al. (1976, 1977) and Radbruch-Hall (1978) reported cases of large scale mountain slope deformation throughout the western United States. Continued studies in Washington State were motivated by investigations for the nuclear energy industry, with detailed summary works reported by Dohwenrend et al. (1978), and Ertec Northwest Inc., (1981).

Radbruch-Hall (1978) provided a comprehensive summary of previously reported field studies involving rock slope deformation by "creep" behaviour. This presentation grouped examples according to general geologic settings, and identified four general environments conducive to the development of slow mass movement processes and the associated morphology:

1/ Creep in flat-lying, interbedded hard and soft rocks (examples include the lateral creep of valley side rocks due to overburden weight, as described by Lapworth, 1911; and similar phenomena, also reported in England, by Hollingworth et al., 1943).

2/ Creep of rigid blocks over soft rocks (cases from Slovakia are reported by Pasek, 1974, and Zaruba and Mencl, 1969).

3/ Creep in rock with inclined bedding or discontinuities (a wide variety of examples are presented, including the well documented Goldau, Frank and Vaiont slides; early work by Lugeon and Oulianoff, in 1922, who described downslope "bending" of bedding; and the noteworthy work of Zischinsky in 1966).

4/ Creep in mixed rocks or rocks with random discontinuity patterns (several examples are described in the works of Nemčok 1972, and Nemčok and others, 1977).

A focus on large scale mountain slope deformation persisted in eastern Europe research during the 1970's. Nemčok (1972) discussed deep-seated gravitational creep in the Carpathian Mountains. Pasek (1974), Nemčok and Baliak (1977), Malgot (1977), Nemčok (1977), and Mahr (1977) continued this trend of research, with a particular focus on the geomorphology and geologic settings where this process was identified. In these studies general modes of deformation were implied through schematic cross sections of the slopes, however, little supportive data in the form of observed subsurface conditions or measured slope movements was presented.

The first presentation of sackung type features in Canada was reported by Clague (1979), who mapped a series of anomalous slope scarps and trenches in close proximity to the Denali Fault system in the Yukon Territory. The development of these features was attributed to seismic activity along the fault system. Bovis (1982) presented the initial findings of a long
term investigation of slope movements at Affliction Creek in the Coast Mountains of southwestern British Columbia. This site featured an extensive network of slope deformation features representative of the large scale slope deformation. A thorough investigation of the slope morphology and geologic structure, supported by the results of long term, repeat surveying of surficial slope monuments, underscored the relationship between slow, structurally controlled deformation and the formation of the distinctive surficial morphology. Noteworthy in this investigation was the establishment of a survey system used to document ongoing slope behaviour. Results of this survey work, reported in 1990, have provided strong support for a slow gravitationally driven failure process.

2.4 Origins of Large Scale Mountain Slope Deformation

The nature of the forces responsible for initiating and driving the deformation of mountain slopes persists as the most significant controversy in the study of this process. This position was clearly expressed by Terzaghi (1962) in his discussion of the mechanism of deep-seated landslides. Radbruch-Hall (1978) maintained a similar point of view in stating that the mechanisms producing deformation of mountain ridges "are still not completely understood". Recent work by McAlpin and Irvine (1994) concluded that this uncertainty still persists.

A variety of causitive forces have been linked to this process, and the development of the related geomorphology. The following have been reported in the literature as the initiating and/or driving forces behind this process:

1/ Gravitational forces.
2/ Seismic shaking.
3/ Tectonic faulting.
4/ Rebound/stress release.
5/ Erosional forces.

This variability presumably stems from the wide range of geologic environments in which the process has been investigated, and the possibility that several different, and potentially interacting mechanisms may be involved. In this discussion it has so far been presumed that the anomalous surficial morphology is reflective of a large scale slope movement process. However, it should be noted that similar slope features have also been attributed to surficial erosional processes unrelated to slope mass movements. A discussion of the causitive forces follows.
2.4.1 **Gravitational Forces**

Gravitational forces are incessant and ubiquitous, and therefore, must play some role in the overall deformation of large slopes. The potential energy gradient present in any slope, a corollary of gravity, provides a natural tendency for downslope movement. This gradient is resisted/balanced by the existing strength within any sloping rock mass.

Gravitational forces have been widely acclaimed as the fundamental cause of large scale mountain slope deformation, as presented by Zischinsky (1966), Ter-Stepanian (1969), Nemčok (1972), Radbruch-Hall et al (1976), Bovis (1982,1990), Beget (1985) and Moore et al. (1992). The investigations by Bovis (1982,1990) at Affliction Creek, B.C., and by Moore et al. (1992) at Wahleach, B.C., continue to be supported by instrumentation data that strongly supports the occurrence of slow, ongoing, gravitationally driven slope movements considered to be independent of current seismic or tectonic activity.

2.4.2 **Seismic Shaking**

Seismic forces generated by earthquakes exert time-varying shear stresses capable of triggering gravitational mass movements. A seismic trigger was proposed by Beck (1968) as the required catalyst to initiate the deep-seated failure mechanism presented in his gravity-faulting model. Radbruch-Hall (1978) cited two locations in California where linear ridge-top depressions have possibly formed due to recent earthquake activity. Clague (1979) attributed the development of mountain slope scarps and trenches to seismic activity along the Denali fault system, but acknowledged the similarities with gravitationally developed features described by Radbruch-Hall. Dohrenwend et al. (1978) reported a relationship between sackung features and the Straight Creek fault zone in Washington State which implied the potential for seismic triggering. Knitter and Fuller (1982) reported the potential for seismic forces to initiate slope movements responsible for the development of sackung features seen in Washington State. Guerricchio et al. (1988) instrumented sackung features and included seismographs to monitor possible seismic contributions to slope deformation. They reported a "possible" connection between seismic activity and slope movements.
2.4.3 Tectonic Origin

Surface rupture related to tectonic faulting has been proposed as a cause of prominent linears and scarps observed on slopes. Proponents for a tectonic origin have typically emphasized the geographic correlation of these features with active tectonic fault zones. The possibility for earthquake forces, either through surface faulting or seismic shaking, to initiate new slope deformation exists, however, there have been no documented cases of instrumented slopes recording this behaviour. Radbruch-Hall (1978) outlined the difficulty in distinguishing gravitationally developed features from tectonic scarps in light of the fact that gravitationally derived scarps are often developed along pre-existing tectonic features such as faults and shear zones.

Solonenko (1977) introduced the term "tectonic scarps" which he attributed to "seismotectonic" processes, large scale crustal movements along faults independent of gravitational forces. Solonenko distinguished these from "gravitational-seismotectonic" processes which have been associated with collapse along faults and toppling of mountain peaks, which were presumably gravitationally driven following a seismic trigger. The former have been documented in association with the 1957 Gobi-Altai magnitude 8.6 earthquake (in Mongolia).

Tectonic faulting was considered by Plafker (1967) to account for the scarps found on Montague Island related to the 1964 Alaska earthquake. Surface rupture along reverse faults developed a series of scarps and cracks which resembled sackung features reported in other areas. The tectonic origin for these scarps was supported by their distribution along the zones of rupture, the oblique relationship of the scarps in relation to the surrounding slope topography, and from post-earthquake air photos.

Eisbacher (1983) proposed a tectonic origin for the prominent scarp that can be traced along the summit ridge of Mount Currie, British Columbia. He calculated that a Richter magnitude 7.0 earthquake event was required to create the observed offset along the scarp, and further suggested that the Mystery Creek rock avalanche may have been triggered by the same event. This site has received further study by Evans (1987), and Bovis and Evans (1996) who have presented comprehensive arguments in support of gravitationally driven slope movements to account for the scarp and associated linear features. In addition to structural analyses which outlined the kinematic feasibility for toppling and sliding mechanisms to occur, surficial monitoring of tension cracks by repeat survey methods along the main area of surface
disturbance has indicated progressive slope movements. The slow but ongoing movements recorded along the upper ridge of Mt. Currie indicate that neither tectonic faulting nor seismic shaking are required to maintain this slow deformation process, although it does not rule out their potential role in the initiation stages of the deformation process.

An argument against the tectonic origin of sackung features has been based on the lack of observed slickensided surfaces developed on joints or shears associated with antislope scarps. Slickensided surfaces are often developed along tectonic faults and exposed in scarp faces, particularly in soft, altered rocks associated with fault zones (i.e. fault gouge). Bovis (1982) attributed low confining stresses, prevalent in the dilating rock mass at Affliction Creek to account for the lack of slickensides on the scarp faces caused by the slow gravitationally driven slope movements.

Further support for the above argument has been observed in an excavated linear trough at B.C. Hydro's Checkerboard Creek site in the Columbia River valley (ref. 7). Fault gouge forming the core of a near vertical tectonic fault was observed at the base of the trench. Near horizontal slickensides on the fault indicated right lateral strike-slip offset which was considered to predate the development of the sackung-like feature. If tectonic faulting had created the linear feature, vertically oriented slickensides might be expected to overprint the older tectonic slickensides seen in the soft fault gouge.

2.4.4 Rebound/Stress Relief

Elastic strain recovery or rebound derived from stored strain energy, as a result of previous geologic loading conditions, has been considered in several studies as a mode of origin for sackung type slope features. Two types of cases have been outlined:

1. The development of sackung features such as antislope scarps from rebound alone.
2. Rebound effects that have initiated a state of instability in large natural slopes which have subsequently deformed under gravitational forces forming the ensuing sackung morphology.

Patton and Hendron (1974) outlined a glacial rebound mechanism to account for differential slope displacements found in glaciated valleys. Their illustrated models indicated structurally controlled movements along foliation and joints, and implied the potential for sackung formation from rebound alone, or in association with subsequent gravitational slope
deformation. Bovis (1982) reviewed the glacio-isostatic rebound hypothesis, but indicated three generally observed conditions incompatible with this mechanism:

1. Antislope scarp development should be possible on both sides of a glaciated valley, but are more typically seen on one side only.

2. Antislope scarps developed by isostatic rebound should be more prominent along lower valley slopes, where rebound effects are more likely to be greatest. This contrasts with many studies that have reported sackung features concentrated in the upper reaches of slopes and ridges.

3. Glacio-isostatic rebound should potentially produce more widespread occurrence of antislope scarps than the more localized cases actually observed in glaciated areas.

Another potential sackung origin involving stress relief and associated strain recovery has been attributed to rapid fluvial downcutting releasing locked in tectonic stresses. Reimer et al. (1988) proposed that the release of high lateral stresses, developed during folding of metasedimentary rocks in the Ecuadorean Andes lead to the loosening of the rock mass along the Rio Paute. This initiated the incipient slope instabilities and associated sackung features.

In a similar fashion, the debuttressing of slopes due to the retreat of valley glaciers has also been proposed as an initiating cause of slope movements and sackung features. Two factors have been highlighted in this proposed mechanism. First, glacial scour of slopes and valley bottoms frequently develops an oversteepened slope profile. Subsequent retreat/melting of the valley glacier then leaves the oversteepened slopes unsupported and potentially primed for gravitational mass movements. The correlation of large scale slope deformation morphology with glaciated valleys has been widely documented. de Frietas and Watters (1973) discussed glacial undercutting/debuttressing as a destabilizing mechanism leading to toppling failures observed in Scotland. Bovis (1982,1990) discussed the potential causal relationship between glacial debuttressing and slope deformation at the Affliction Creek site, British Columbia.

2.4.5 Physical Erosion Processes

Physical erosion of surficial slope materials has been reported to have developed sackung type features. A clear distinction is necessary between features developed by erosional processes and those that reflect subsurface deformation processes. Erosionally developed geomorphic
features should be considered "cosmetic" as they do not reflect internal deformation of the underlying rock slope material, but are strictly a surficial sculpting of the slope.

The following geomorphic processes have been presented in mountain slope as having developed anomalous linear troughs:

1/ Wind and frost action, as proposed by Paschinger (1928).

2/ Glacial scour of weak structural zones, as proposed by Mylrea (1969) for the features observed in the vicinity of Mica Dam, British Columbia.

3/ Fluvial erosion, as proposed by Clague (1979) who outlined the differences between seismically and fluvially developed troughs/linears.

A recently documented case investigated a prominent linear trough (antislope scarp), displaying typical sackung morphology, upstream of the Revelstoke Dam, in south-central British Columbia (B.C. Hydro, Ref. 7). The linear feature is approximately 8 to 10 metres in height, 20 to 30 metres in width and traces some 400 metres across the mountain slope sub-parallel to the topography. The linear feature is situated 350 metres above the original valley floor, at approximately 800 metres elevation, well below the local ridge tops (which are around 2000 m). Detailed trenching investigations exposed a faulted dyke at the core of the linear. Glacial scour is believed to have preferentially eroded along the softer fault material, as the surrounding bedrock surface shows strong evidence of polishing. The condition of the infilling overburden materials do not support significant postglacial offset, arguing against a gravitational origin for the linear. Noteworthy in this case was the proximity of this erosion linear to other scarps and tension cracks that have been clearly associated with slow deformation of the slope.

2.4.6 Combined/Interacting Effects

Sackung features and slow deformation of large slopes have also been attributed to interacting causative forces. Noteworthy were the combinations of seismic-gravitational forces suggested by Beck (1968), Dohwenwend et al. (1978), Anderson et al. (1980) and Guerrecchio et al. (1988), rebound-gravitational effects proposed by Patton and Hendron (1974), and glacial debuttressing-gravitational effects proposed by Bovis (1982).
2.5 Discussion of Deformation Mechanics

Large scale mountain slope deformation has been described as a complex process still poorly understood. Many investigators have presented schematic illustrations of particular cases with inferred deformation mechanism(s) compatible with observed surficial geomorphic features. The scarcity of subsurface information from documented studies and/or instrumentation data recording active slope movement processes, has limited the understanding of the actual rock slope behaviour, and any evaluation of active deformation mechanisms. This has lead to the rather speculative treatment of this process, and a weak correlation between slope deformation behaviour and observed slope morphology.

In the discussion of large scale mountain slope deformation, it is important to identify the fundamental differences between rapid slope failures and "creep" type deformation. Beyond the obvious difference in movement rates, there exists a fundamental distinction in the material behaviour of these failure modes. Terzaghi (1950) discussed this distinction by analyzing the deformation patterns developed in laboratory tested asphalt samples. The heterogeneous nature of asphalt was used to represent the heterogeneity characteristic of most rock masses. Terzaghi outlined two key differences that were observed in the deformation patterns of the tested material:

1/ Proportion of loading: Application of a load exceeding the yield strength caused instantaneous failure along discrete shear surfaces. The material behaved as a solid. Short term loading below the yield strength resulted in imperceptible changes.

2/ Period of loading: Long term application of loading below the yield strength, but above the fundamental strength of a material deformed the weaker sections of the material much like a very viscous liquid.

Terzaghi stated that the different deformation patterns reflected the widely different laws governing material behaviour of solids and fluids. Stress levels exceeding the creep strength of a material, but below the yield strength resulted in material creep (analogous to slope creep) if the load was maintained; whereas stress levels exceeding the yield strength resulted in rapid shear failure (likened to rapid landsliding).

The rheological behaviour of rock is a very scale dependent phenomenon. Therefore, it is critical to appreciate the differences between the response of a small sample under laboratory testing and that of a large rock mass. The larger the scale of the rock mass under consideration,
the more significant the effect of the discontinuities (joints, bedding, foliation, shears and faults) on the material behaviour. At the scale of large natural slopes, deformation mechanisms are strongly influenced by the presence and conditions of discontinuities within the rock mass. Therefore, an understanding of the strength and deformation characteristics of both discontinuities and the overall rock mass is essential in any study of this phenomenon.

An important discussion of surficial deformation characteristics, and their potential relationship to slope deformation mechanics, was presented by Terzaghi (1950). He proposed that surficial deformation patterns reflected the nature of subsurface deformations. A wider, more dispersed zone of deformation would presumably have developed more extensive surficial disturbance, typical of sackung type cases. Alternatively, a more concentrated and localized deformation pattern, such as that corresponding to a distinct head scarp, would more likely correspond to sliding along a discrete basal shear zone. This observation is consistent with the current view of sackung deformation mechanisms.

Following the classification schemes mentioned in the introduction (section 2.1), which were process based, slow, large scale mountain slope deformation processes have been included in recent, and widely recognized slope movement classification schemes as follows:

1/ Nemčok et al. (1972) classified these movements as "Creep".
2/ Varnes (1978) described slow deformations in rock as "Rock Flow (deep creep)".
3/ Hutchinson (1988) grouped a number of slow deformation processes under the term "Sagging" (similar to Zischinsky's "Sackung").

Within this classification framework, a number of fundamental deformation mechanisms have been implicated in the sackung process. Creep (Ter-Stepanian,1969), sliding (Beck,1968), toppling (Tabor,1971; Bovis,1982; Pritchard and Savigny,1991), buckling (Reimer et al.,1988) and plastic flow (Pasek,1974; Radbruch-Hall,1978) have all been suggested as potential deformation modes in large scale mountain slope deformation. Mahr (1977) provided a summary of the different failure mechanics reported in Sackung type studies. Hoek and Bray (1981) provided a good summary of the individual deformation processes. The following sections provide a basic description of each of the basic movement types and their proposed occurrence in documented cases of slow large mountain slope deformation.
2.5.1 Creep - Slow Plastic Flow

The inference of creep type behaviour with many studies of large scale mountain slope deformation has been implied through the descriptive terminology (outlined in section 2.2). The term creep has been used to collectively define a range of time-dependent deformation processes. Additionally, the definition of creep generally implies the development of plastic strain at constant levels of stress. In this discussion slow plastic flow is considered synonymous with creep.

Radbruch-Hall (1978) outlined the difficulty in accurately defining creep behaviour at the scale of a large rock slope at which heterogeneity and structural discontinuities strongly influence deformation behaviour. Her description of the term "creep" broadly included very slow outward and downslope deformation, without a discrete, through-going failure surface, and included the potential to evolve into modes of more rapid slope deformation. Radbruch-Hall classified five styles of gravitational slope movements on the basis of different geologic conditions rather than on different deformation mechanisms, although she used the term "mass rock creep" to cover the overall process. A review of the field examples revealed the gradational, or overlapping nature between the various cases.

Creep and plastic-flow mechanisms have been proposed to explain the deformation patterns in a variety of different settings. Flow of relatively rigid rock blocks on an underlying plastically deforming (flowing) material have been proposed by Zaruba and Mencl (1969), Pasek (1974), and Radbruch-Hall (1978).

A theoretical point that merits discussion is the treatment of constant stress conditions implied in creep phenomenon. Although this time dependent behaviour can occur under conditions of varying stress, few studies have highlighted the potential variability in stress conditions, implicitly assuming creep at constant stress levels. In a large rock slope several factors can contribute to changing stress conditions. Examples include fluctuating ground water levels, the continual action of physical erosion, such as fluvial undercutting at the toe or along incised drainage channels in a slope, and/or deposition of material. This suggests that constant stress levels may be an idealized situation in large mountain slopes, and consequently, the treatment of rock slope behaviour as creep alone may not be fully appropriate. The slow deformation rates associated with this process has probably been the reason that creep type behaviour has been used to describe this process.
Terzaghi (1936, 1962) proposed that effective stress, and not total stress, governs material behaviour. The evaluation of effective stress conditions within a large rock slope would require, among other things, a knowledge of the time-varying groundwater profile. Piezometric monitoring of large natural slopes has rarely been reported, and until such time that detailed field investigations provide this information, the treatment of creep hypotheses remains relatively unsubstantiated.

2.5.2 Sliding Failure

Sliding mass movements involve failure along a through-going shear surface. In rock slopes this generally involves movement along rock mass discontinuities. The deformation behaviour of individual discontinuities under simple shear testing in the laboratory has been intensively investigated, however, the complex interaction of numerous discontinuities at the scale of a large rock slope is more problematic.

The lack of a through-going, failure/shear surface has been considered implicit to the description of sackung processes. This condition has generally been implied through a consideration of surficial slope morphology and slow movement rates. Both the pattern of internal deformation and the arrangement of surficial landforms created in sliding dominated mass movements are incompatible with sackung characteristics.

Most research efforts have differentiated sackung morphology from those developed in more rapid landslide processes (Radbruch-Hall, 1978). For example, Holmes and Jarvis (1986) suggested that the close spacing of scarplet features at Ben Attow, Scotland, (less than 40 metre spacing between scarps or tension cracks) was not considered compatible with a sliding type movement. Bulging at the toe of deforming slopes, and arcuate head scarps, considered characteristic of rotational sliding, have rarely been associated with sackung type deformation. Conversely, rock mass dilation in the middle to upper slope areas, with considerable surface deformation (antislope scarps, tension cracks) has more commonly been reported in sackung processes.

The models of rock slope deformation proposed by Zischinsky (1966), Beck (1968), and Nemčok (1972) included planes of sliding, or shear failure, within the deforming rock masses. The shear failure surfaces in these models formed part of a more complex deformation mechanism that satisfied the overall deformation geometry and the nature of the surficial slope
morphology, but did not suggest that the failure mechanics were completely controlled by failure along a single shear surface.

It has also been suggested that a through-going basal shear feature represents the final evolutionary stage of an initially slow, dispersed failure mechanism (Pritchard and Savigny, 1990; Dramis and Sorriso-Valvo, 1994; McAffee and Cruden, 1996). Several examples of large catastrophic failures were cited by Radbruch-Hall (1978) as having displayed creep movements prior to failure (Goldau, Switzerland; Vaiont, Italy), however, in these cases the geologic conditions were more compatible with sliding dominated behaviour than with internally dispersed deformation that evolved into rapid failure.

2.5.3 Toppling Failure

Toppling failure has been identified as a common and widespread mass movement process, and has received considerable study over the last 25 years (de Frietas and Watters, 1973; Goodman and Bray, 1976; Wylie, 1980; Martin, 1990). Toppling failure has also been associated with slow slope deformation processes, and has been considered to be a component of many sackung cases.

de Frietas and Watters (1973) completed the first comprehensive presentation of toppling as a failure mechanism in rock slopes. The failure mechanism involves downslope rotation of blocks or columns of rock, and flexural slip along closely spaced discontinuities (Goodman and Bray, 1976). Structurally controlled behaviour along rock mass discontinuities (joints, bedding, foliation, cleavage, shears and faults) governs the process. Gravitational forces have been considered to drive the failure mechanics, although groundwater and thermal expansion (McAffee and Cruden, 1996) have been considered as contributory effects. Goodman and Bray (1976) stated that toppling also occurred in fractured soils, although this discussion deals exclusively with rock slopes.

The importance of toppling failure mechanisms in relation to large scale slope deformation has been made apparent in the conspicuously similar morphological features developed. The clear resemblance of toppling morphology to the features described as classical sackungen, has been emphasized in the identical descriptive terminology, such as antislope scarps, tension cracks and linear troughs. These similarities have suggested that toppling is potentially a prominent failure mechanism in sackung type mountain slope deformation (Bovis,

Beyond the noted morphological similarities between toppling and sackung, several other notable similarities have been identified between the two phenomena, including:

1/ Occurrence in a wide variety of geologic settings; including all the major rock types, as outlined in Table 2.1:

<table>
<thead>
<tr>
<th>Geologic Environment:</th>
<th>Toppling Failures:</th>
<th>Sackung:</th>
</tr>
</thead>
</table>

2/ The presence of a discrete basal shear surface has not been considered integral to either toppling or sackung processes. Internal deformation patterns have generally been described dispersed throughout the afflicted slope. Deformation and movements have been noted as most pronounced along the surface and upper regions of the deforming rock mass, and progressively diminished with depth.
Slow, progressive movements have been considered characteristic of both toppling and sackung. Rates of rock mass failure attributed to toppling have been considered slow, particularly in comparison to rock falls and slides (Goodman and Bray, 1976). Slow deformation rates in the order of several millimetres to several centimetres per year have been suggested for sackung processes (Radbruch-Hall, 1978, see also Table 2.1).

Both failure processes can involve large volumes of material. de Frietas and Watters (1973) estimated the Glen Pean toppling failure to comprise some 30 million cubic metres. The current volume of slowly failing material at the Wahleach slope, based on detailed subsurface monitoring has been estimated to be in the order of twenty million cubic metres (Moore et al., 1992). The possibility of paleo-slope movements at Wahleach involving up to 60 million cubic metres has also been considered, based on the rock mass quality observed to depths of 150 to 200 metres. The rock to these depths was described as having a "loosened appearance", suggestive of movement, however currently monitored movements were confined to the upper 60 to 120 metres of the slope. The authors stated that current movements at the deeper level could be occurring at rates below the sensitivity of the instruments which would, therefore, require a longer monitoring period to detect.

Sackung processes have most commonly been reported in the upper elevations of ridges and mountain slopes, where the greatest evidence of surface distortion exists. In toppling failures, deformation can extend to large distances upslope from the zone of initiation, or most critical area. Goodman and Bray (1976) suggested that movement can extend well above the slope crest to distances of up to four times the slope height, and be more pronounced in the upper regions of afflicted slopes.

Similar initiating mechanisms have been proposed for the two processes. Specific examples include the following:

a/ Toe erosion of slopes due to fluvial or marine erosion processes; for example, de Frietas and Watters (1973) illustrated a toppling case where instability was initiated by marine erosion and weathering along the Devon coast.

b/ Glacial undercutting/oversteepening and subsequent debuttressing, leading to sackung type slope deformation. This was proposed by Bovis (1982,1990) at Affliction Creek, British Columbia. Similar causes were put forth by de Frietas
and Watters (1973) to account for the initiation of a large toppling failure at Glen Pean, Scotland.

c/ Stress release and rebound (from elastic strain recovery). Reimer et al. (1988) attributed the rapid downcutting of the Rio Paute in the Ecuadorian Andes, with subsequent rebound, as the initiating mechanism for large slope instabilities. This process was considered to have loosened the rock mass to considerable depths, allowing gravitationally driven toppling movements to further deform the slope.

7/ Structurally controlled deformation along rock mass discontinuities has been emphasized in both toppling and sackung. The association between critically oriented structure and toppling failure has been outlined by both de Frietas and Watters (1973) and Goodman and Bray (1976). The importance of rock mass structure in the deformation mechanics of sackung has been emphasized by Jahn (1964), Zischinsky (1966), Tabor (1971), Radbruch-Hall et al. (1976), Bovis (1982), and Pritchard and Savigny (1991). The clear relationship between rock mass structure and the orientation of surficial deformation features identified in both toppling and sackung cases can not be overlooked.

A final analogy made between these two phenomena revolves around the recent nature of their recognition and analysis. Toppling and sackung have only received widespread recognition and concentrated study within the last twenty-five to thirty years. In light of all the similarities described above, the hypothesis that toppling failure mechanisms are prominent in the general framework of sackung processes appears well founded. Further detailed investigations in the field, supported by subsurface instrumentation and analyses using appropriate numerical modelling techniques should provide support for this hypothesis.

2.5.4 Buckling Failure

Buckling failure involves the bending and tensile failure of laminated or closely jointed rocks, in which the failure behaviour is structurally controlled along rock mass discontinuities oriented parallel to the dip of the slope. Hoek and Bray (1981) outlined the details of the process.

Buckling failure mechanisms have been ascribed to a number of cases displaying morphology typical of sackung type slope deformation. Radbruch-Hall (1978) described a
collection of slope trenches and ridge-top depressions associated with surficial buckling of an interbedded sandstone and shale sequence in the Santa Ynez Mountains in southern California. Surficial buckling of the more rigid sandstone layers, with folding and contortion of the shale layers justified the slope trenches. However, the deformation mechanisms occurring at depth and causing large ridge top depressions (up to 15 metres deep, 30 metres wide and over 1.5 kilometres in length) could not be confirmed due to lack of subsurface information. Evidence suggested slow, ongoing movements of the surficial materials.

Reimer et al. (1988) attributed a number of cases involving large scale rock slope deformation to buckling failures. They investigated the La Letra slope along the Rio Paute in the Ecuadorian Andes for the purpose of locating a dam site. Well developed slope features and distortion of regular foliation indicated significant internal deformation of the rock mass. Two notable observations from this example were:

1/ The strong support for structurally controlled deformation within the well foliated rock mass (predominantly phyllite), and;

2/ the evolution of complex deformation patterns, involving flow ("soil-type") and gravitational faulting (described as "glides along the axes of the buckling faults") from initial movements ascribed to buckling failure mechanics.

2.5.5 Block Flow

Block flow refers to a specific deformation process whereby blocks of rigid rock move down slope due to plastic deformation in softer underlying material. This general mechanism has been differentiated from other variations of sackung, based recognized geologic conditions and deformation evidence. However, it shares a number of similarities that merit its inclusion in discussion of this overall process. Block flow can potentially involve large masses of material, with slow movement rates in relation to landsliding, and be characterized by surface morphology similar to other types of slow mountain slope deformation, rather than more rapid sliding type features. Most significantly, the investigated field examples have been described without a discrete failure surface.

Zaruba and Mencl (1969) proposed a general mechanism of plastic flow in underlying material, with downslope progression of overlying rigid rock blocks by rotation and gliding/flow, and illustrated examples observed in the Carpathian mountains. Pasek (1974) described this
phenomenon as a widespread process, and provided a series of diagrams outlining their origin and subsequent development. Similar examples in the western United States have been presented by Radbruch-Hall et al. (1976) as conforming to sackung style behaviour. These examples were similar in that they all involved extensive fragmentation of rigid intrusive rocks overlying plastically flowing weak shales. The development of distinctive sackung morphology was key to the identification of these processes.

2.5.6 Deformation Involving Combined Mechanisms

Many of early investigations proposed a deformation mechanism incorporating several interacting types of material behaviour. For example, Zischinsky's (1966) inferential proposal of the sackung process implied the occurrence of at least two distinct rheologic responses, which he attributed to the distribution of stress and the mechanical properties of the rock mass. Zischinsky described "internal rotation" within a dispersed zone forming the base of a large deformed rock slope as a plastic, flow process. He distinguished this viscous behaviour from the shear failure planes which cut through the upper portions of the deformed mass.

Moore et al. (1992) discussed the nature of at least three distinct deformation mechanisms in the Wahleach slope. Detailed surface and subsurface instrumentation of the Wahleach slope revealed flow, sliding and block rotation to be contemporaneous. An important feature of this behaviour manifests itself in the interacting effects between different zones within the slope. Flow and sliding type movements occurring in the steeper portions of the slope appeared to be having two related effects. First, flow/sliding movements in the middle to upper portion of the slope undermined higher portions of the slope which responded by sagging, toppling or downdropping to catch up with the flowing middle portion of the slope. Secondly, the flowing portion of the slope appeared to be generating movements in the downslope area, presumably due to increased loading from the deforming upslope material. Recognition of this complex and progressive deformation behaviour at Wahleach was significant in that it provided quantitative evidence, and therefore precedent, for similar behaviour to occur in other large rock slopes.

Reimer et al. (1988) presented three case examples of large scale, complex slope deformations in the Ecuadorian Andes. A complex mass movement process that could not be explained by a single deformation style, was rationalized by incorporating toppling, shearing and extensional deformation mechanisms.
Heterogeneous lithologic conditions and strength anisotropy related to structural features are common to most rock slopes. The combined effects of lithologic and structural conditions in a large rock slope should allow for the potential development of complex failure mechanisms involving more than one mode of deformation. When the variety of different driving forces, such as gravitational, seismic shaking and stress rebound are also considered, complex failure mechanisms could be expected to prevail in large mountain slopes susceptible to this process.

The large scale of most sackung phenomena and the wide variety of geologic environments in which they have been described, suggests that any one single failure mechanism has probably not been responsible for all observed phenomena. The incorporation of complex and interacting deformation mechanisms in the description and analysis of the process is probably more appropriate.

2.6 Effects of Structural Geology

Essentially all research efforts have suggested a relationship between sackung type deformation and structural geology. Jahn (1964) outlined the clear relationship observed between the orientation of bedrock structure and slope features attributed to gravitational deformation. Later works also emphasized the connection between structural geology, slope morphology and the governing deformation mechanisms, particularly the detailed investigations of Bovis (1982) and Chigira (1992).

Terzaghi (1962) discussed the association between the low strength of rock mass discontinuities, in relation to intact rock, and the governing material behaviour. This appears to be a logical correlation in light of the strength anisotropy that inherently exists in rock masses containing pervasive structural discontinuities. Exceptions to this general condition may exist in extremely low strength rocks such as poorly indurated argillaceous sediments, and tectonically deformed and altered rocks that persist over a broad distance (such as in major tectonic fault zones).

2.7 Deformation Rates

One of the few widely accepted facts involved in this process is the recognition of a slow surficial deformation rate. This general similarity has been one reason that the wide variety of deformation styles are often grouped into a common mass movement process, despite the fact
that the material behaviour may be significantly different. Quoted rates of deformation range from 0.4 millimetres per year (McAlpin and Irvine, 1994) to 6 metres per year (Chigira, 1992). Terzaghi (1950) quoted a movement rate of one foot per ten years as a general value. Geologic conditions in conjunction with prevailing stress conditions (ie. stress ratios of strength to stress) are believed to govern the deformation rates. Table 2.1 below presents a collection of quoted deformation rates found in the literature.

Table 2-2: Reported deformation rates in sackung type cases

<table>
<thead>
<tr>
<th>Deformation Rate:</th>
<th>Author(s):</th>
<th>Comments/Location:</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4 mm/yr(^1)</td>
<td>McAlpin and Irvine (1994)</td>
<td>Aspen Mtn., Colorado</td>
</tr>
<tr>
<td>1 foot/10 yrs (=3mm/yr)</td>
<td>Terzaghi (1950)</td>
<td>General slope creep</td>
</tr>
<tr>
<td>5 mm/yr</td>
<td>Bovis and Evans (1996)</td>
<td>Mt. Currie, B.C.</td>
</tr>
<tr>
<td>8-12 mm/yr</td>
<td>B.C. Hydro (ref. 7)</td>
<td>Checkerboard Creek, B.C.</td>
</tr>
<tr>
<td>1 cm/yr</td>
<td>Malgot (1977)</td>
<td>Slovakia</td>
</tr>
<tr>
<td>2.4-12 mm/yr</td>
<td>Guerricchio et al (1988)</td>
<td>Maratea Valley, Italy</td>
</tr>
<tr>
<td>15 mm/yr</td>
<td>Moore et al (1992)</td>
<td>Wahleach, B.C.</td>
</tr>
<tr>
<td>80 mm/yr</td>
<td>Bovis (1982,1990)</td>
<td>Affliction Creek, B.C.</td>
</tr>
</tbody>
</table>

Notes:

1/ These near zero (and probably undetectable) rates were based on detailed trenching work in connection with radiocarbon dating. The calculated movement rate was established by dividing the width of the investigated surficial sackung feature by a maximum age of movement from dating work.
2/ Moore et al. (1992) presented the only known, detailed subsurface investigations, based on inclinometer and extensometer data. This average movement rate reflected the overall slope values which ranged between 4-40 mm/yr.

2.8 Numerical Modelling

Numerical modelling techniques have advanced dramatically in the last 20 years, with recent developments leading to the ability to model both stress distribution and deformation patterns within large rock masses. Finite element, finite difference and distinct element programs have been applied to suit different geologic conditions. The distinct element method has been shown by Pritchard and Savigny (1990) to provide a suitable approach for modelling structurally controlled failure mechanisms. Finite difference continuum methods such as that developed by Cundall (Fast Lagrangian Analysis of Continuum, Itasca, 1992) have provided an appropriate continuum approach for modelling slopes in which the rock block size is small in proportion to the scale of the slope. The ability of this program to model progressive deformation has provided a state-of-the-art approach for evaluating the evolution of slope movement processes. Within this program is the ability to simulate other important geologic processes such as groundwater, erosion or loading (glacial or seismic).

Despite development of this analytical capacity, little attention has been directed towards the investigation of large natural rock slope behaviour. The work of Pritchard (1989,1990) and Pritchard and Savigny (1991) represents some of the few reported cases in which numerical modelling techniques have been applied to the deformation analysis of large natural slopes. The following discussion outlines examples in which numerical modelling techniques have been employed, and illustrates the increasing capability of these methods to analyze the significant factors in slope deformation processes.

Sturgul et al. (1976) carried out a finite element analysis of a mountain ridge in the Austrian Alps to investigate the distribution of principle stresses within the mountain mass. This analysis did not include an investigation of the deformation potential of the mountain slopes, but outlined the distribution of gravitationally derived stresses. This modelling work established several important conclusions:

1/ The major principal stress in the vicinity of the slope surface was shown to be essentially parallel to the topography.
With increasing distance away from the free slope surface, the major principal stress was shown to progressively rotate into a vertical orientation.

Tensile stresses developed in the upper slope areas, corresponding to the potential for tensional failure in these regions.

The calculated stress field agreed with values obtained from in situ testing within a nearby underground mine, supporting the hypothesis that the stress state in the slope was compatible with gravitational self-weight stresses.

Savage and Varnes (1987) investigated self-weight (gravitational) stresses in a symmetric ridge established from an elastic solution (two-dimensional, plane strain analysis). These stresses were then applied to a Coulomb plasticity solution to investigate the potential for failure and plastic deformation in the ridge. Their analyses predicted the development of "upward-facing scarps" along the upper portions of the simplified ridges, compatible with that observed in sackung cases. This simplified analysis acknowledged the lack of groundwater effects in the presentation, and commented on the potential limitations. Nevertheless, the authors indicated the potential for plastic flow to occur throughout significant portions of the ridge.

The importance of groundwater in rock mass behaviour has been frequently stressed (Terzaghi, 1950; Hoek, 1968; Patton and Hendron, 1974). The relationship of groundwater pressures and effective stress is fundamental in any deformation or stability analyses. Despite this, the treatment of groundwater conditions has often been neglected or grossly simplified in numerical studies of large rock slopes. There is a need to incorporate more realistic representative groundwater conditions in future modelling studies. An important capability of state-of-the-art numerical methods is their flexibility to model different groundwater conditions, ranging from simple hydrostatic to transient conditions (for example the Universal Distinct Element Code, UDEC, and FLAC, both marketed by Itasca Corp.).

Numerical modelling of open pit mining slopes has been reported from several areas of the world (Martin, 1990; Du Plessis and Martin, 1991; Orr et al., 1991, ). The economic drive behind the analysis of open pit slopes has unfortunately not been translated to large natural slopes, which understandably have received very limited focus.

The experience gained from modelling of open pit mining slopes and other excavated slopes has been valuable as the stress and deformation responses to the excavation activity can
be compared to the geomorphic processes operating in natural mountain slopes and valleys. Although similar processes may operate in mined and natural slopes, rates of deformation in man made slopes can be several orders of magnitude faster. Many researchers have implicated the potential of glacial debuttressing effects, somewhat analogous to open pit excavation, in the initiation of sackung type behaviour (Tabor, 1971; Radbruch-Hall, 1978; Bovis, 1982, 1990; Beget, 1985; and McAlpin and Irvine, 1994). Reimer et al. (1988) accounted for similar stress relief behaviour due to rapid fluvial downcutting.

Rapid stress changes created by open pit mining operations have been observed to develop extensive slope failures, often of a toppling nature, which have been modelled by Orr et al. (1991). They reported that recently developed finite difference techniques, such as the FLAC code, provided a solution much more compatible with observed deformation geometries than previously employed limit equilibrium approaches. Martin (1991) and Martin and Mehr (1993) employed finite difference and distinct element techniques to investigate the deformation behaviour of open pit mine slopes in several North American locations. The importance of these examples was revealed in the ability to experiment with varying geologic conditions (ie. strength parameters of the rock mass and discontinuities, hydrogeology) to simulate the movement patterns established by slope monitoring; a process referred to as "model calibration". Having established a reasonable agreement with monitored slope movements, these modelling studies provided considerable insight into the slope deformation behaviour, including the pattern (depth and distribution) and evolution of movements.

Modelling studies of excavated highway slopes have also been reported. Kalkani and Piteau (1976) applied the finite element method to investigate stress conditions and deformation potential of the Hell's Gate Bluffs in British Columbia. This method provided an evaluation of the observed toppling failure, and included experimentation with fluctuating groundwater levels. The modelling work was able to approximate the monitored slope movements, although more importantly it provided insight into the conditions and probable mechanism of deformation within the slope. The modelling methodology used was well founded in respect of the "data-limited" condition that typifies most geologic problems, as emphasized by Starfield and Cundall (1988).

Modelling of the Heather Hill landslide in British Columbia by Pritchard and Savigny (1991) provided one of the few documented applications of numerical modelling in the
deformation analysis of large natural slopes. The modelling work employed the finite difference, distinct element code, UDEC, to investigate the effects of geologic structure (bedding foliation) on the deformation potential of the slope. The success of this modelling work was seen in the ability to match observed field conditions. Specifically, the evidence of structurally controlled toppling failure along foliation was reproduced by the model. Moreover, the modelling provided insight into the pattern of deformation within the slope, and the progression of toppling movement into a deformed rock mass with a potential deep-seated basal surface. The importance of this work with respect to the sackung process was illustrated in the similarity of the deformation morphology and behaviour in both the model and field conditions. The conclusion that toppling failure mechanisms have the potential to evolve into deep-seated landslide processes addressed one of the fundamental concerns and unknowns surrounding the sackung process.

2.9 Uncertainties Surrounding Sackung Processes

The degree of uncertainty regarding many aspects of the sackung process reflects the lack of subsurface and quantitative information in reported field cases. This section discusses aspects of the problem that are incompletely understood and areas that have yet to receive adequate study.

2.9.1 Mechanics of Deformation

A more thorough understanding of actual failure mechanics, and the influence of seismicity, groundwater and time dependent effects will develop only when quantitative data from instrumented rock slopes provides factual evidence to support the current hypotheses. Hypotheses from previous studies have typically proposed general modes of origin and mechanics of deformation, but have rarely supported the presentation with quantitative details of the process. This has left a deficiency in the understanding of the process mechanics. The following sections expand on particular aspects of the process where an insufficient understanding persists.
2.9.2 Depth of Deformation

A wide range of deformation depths have been proposed in sackung type studies. Deformation depth has often been described as "deep" or "deep-seated", although this term appears to have been used rather loosely, involving considerable interpretation. Investigation of this detail has been restricted by the fact that few studies have experienced the luxury of detailed subsurface investigations to provide such information. Detailed field instrumentation could provide details in the resolution of this depth issue. At present only the detailed investigations related to hydroelectric projects in British Columbia and New Zealand have acquired this level of field study.

In light of the variety of geological environments in which sackung cases have been reported, and the different origins and mechanisms involved, there is probably no definitive depth at which "typical" failure occurs. In a general sense, the depth of deformation can be grouped into two broad ranges:

1/ "Deep-seated" movements which extend to depths greater than approximately 200 metres; and;

2/ "Shallow" deformation phenomena typically occurring at depths of 100 metres or less.

Deep style deformation:

1/ Zischinsky (1966) presented a series of cross sections to accompany his description of sackung, that indicated plastic flow occurring throughout a broad zone to a depth of approximately 400 metres.

2/ Beck's (1968) discussion of "gravity faulting" in the New Zealand Alps outlined a proposed failure mechanism that extended to depths in the range of 700 to 800 metres. Beck explained the inconsistency between the shear strength and shear stress required to cause failure at this depth by suggesting the influence of a seismic trigger.

3/ Zaruba and Mencl (1969), Nemčok (1972), Nemčok and others (1977), and Mahr(1977) extensively investigated large scale slope deformations in regions of eastern Europe, particularly the Carpathian and Tatra mountains of Czechoslovakia. A general agreement that these gravitationally attributed processes extended to substantial depth was held. Nemčok (1972) described three variations of "deep-seated creep" that occurred to depths of 250-300 metres.
Shallow style deformation:

1/ Several authors (Terzaghi, 1950; Ter-Stepanian, 1969; Radbruch-Hall, 1978) emphasized that creep movements of surficial materials, caused by seasonal fluctuations in temperature and moisture (solifluction, gelifluction, soil flow) were a distinct phenomenon from large scale slope deformation.

2/ Bovis (1982), in his initial report on rock slope deformation at the Affliction Creek site in southwest British Columbia, proposed that post glacial ridge deformation was a structurally controlled process that did not extend to the depths suggested by European researchers. Subsequent surficial monitoring of deformation features, and kinematic analyses supported the concept of a failure zone in the range of 70 to 80 metre depth (Bovis, 1990).

3/ Three examples of large scale slope deformation were reported in the Ecuadorian Andes (Reimer et al, 1988). The depth of deformation in these cases ranged from 50 to 100 metres. Supporting information was obtained from subsurface investigations (seismic refraction surveys, drillholes and adits) carried out in conjunction with a dam site selection.

4/ Chigira (1992) provided a very detailed investigation of the subsurface structures associated with long-term gravitational deformation of rock slopes. An important aspect of Chigira's work was the correlation between surface and subsurface features that he attributed to "mass rock creep", following a similar definition to that proposed by Zischinsky (1966) and Radbruch-Hall (1978). Several examples, reflecting different geologic conditions were presented showing deformation depths ranging from 10 metres (in massive rocks, i.e. sandstone and granite) up to 50 metres in strongly foliated rocks. He did, however, cite one example of a large scale gravitational fold in slate that potentially extended as deep as 500 metres, more in line with the deeper seated phenomena reported from eastern Europe.

5/ Deformation of the Wahleach slope has been observed to extend to depths of 60 to 120 metres beneath the slope surface (Moore et al., 1992). The depth of deformation in this slope has been established by detailed monitoring of inclinometers and borehole extensometers, providing information that has not previously been available from slopes of this size.
An interesting case study presented by Norton and Redden (1960) detailed field evidence of shallow deformation from a site in the Black Hills of South Dakota. Distinct zones of deformation, attributed to gravitational creep, were observed in numerous quarried locations. These mass movements occurred in quartz-mica schists, with a zone of flexure and fracturing cross-cutting the distinct foliation. The observed depth of deformation ranged from 4 to 10 feet, and formed parallel to the ground surface. This region has not been glaciated so the cause of deformation requires some explanation other than glacial loading.

An important point to consider regarding the depth of deformation concerns the frictional component of rock material strength. Experimental results from shear strength testing of both intact and jointed rock have yielded strength envelopes that reflect frictional character. Exceptions to this would involve rock slope movement governed by failure along a discrete shear surface comprised of cohesive material such as clay gouge. Field and laboratory experience have shown the Mohr-Coulomb criterion to provide a reasonable relationship for the analysis of rock mass strength. Frictional behaviour, based on the Mohr-Coulomb relationship, dictates that rock mass strength progressively increases with depth (ie. increasing confining stress). Therefore, a threshold depth where shear strength overcomes shear stresses, will preclude deeper failure. This depth will vary for different materials, but in general materials with higher frictional resistance should have a shallower threshold depth than lower frictional strength materials.

The above discussion of material strength is applicable to instantaneous loading, but may not be valid in the case of time dependent behaviour intrinsic to slope creep. The role of time dependent behaviour in slow deformation processes, although implicated in many studies, is poorly known. This reflects the lack of knowledge of in situ conditions and the long term behaviour of large slopes.

2.10 Seismic Effects

To date reports of monitored rock slopes having experienced seismic shaking sufficient to trigger failure or large deformation have not been documented. This would provide precedent setting information regarding the behaviour of large natural slopes under seismic loading, and could answer some of the uncertainty regarding the necessity of seismic triggering to initiate sackung deformation.
The geographic relationship between reported cases of sackung features and active seismic zones has been frequently documented (Beck, 1968; Radbruch-Hall, 1978; Dohrenwend et al., 1978; Clague, 1979; Knitter and Fuller, 1982; Guerricchio and Melidoro, 1988). The overlap between seismically active zones and mountainous regions, where suitable topographic and geomorphic conditions exist to create potential gravitational processes, precludes a conclusion as to the trigger for sackung initiation.

While many studies have suggested the role of earthquake activity in sackung development, certain studies have argued that seismicity is not required to drive the process, or is not involved in active deformation processes. Analysis of the instrumentation data from the Wahleach project has indicated that slope deformation is an ongoing process, in the order of 4 to 40 millimetres per year. This has provided conclusive support for a gravitationally driven process at this site. Slope movement data from the Affliction Creek (Bovis 1982, 1990) and Mt. Currie (Bovis and Evans, 1995) sites have shown progressive surficial movements providing further support for a gravitational origin.

An alternative approach to evaluating the seismic contribution to the sackung process has been based on a detailed study of volcanic ash layers by Beget (1985). Beget investigated the stratigraphic relationships of tephra exposed in prominent antislope scarps near Glacier Peak, in the Cascade Range of Washington State. The lack of growth faults and the uniform distribution of tephra were considered evidence that the scarps did not form during periods of volcanic eruption, when seismicity would presumably have been most active. Gravitationally driven slope deformation, possibly in connection with deglaciation, remained as the proposed origin for these slope scarps.

The fact that evidence was found in support of gravitationally driven slope movements does not preclude seismicity as a potential contributory cause. However, until a well instrumented slope is subjected to a significant earthquake, this correlation remains unsubstantiated.

2.11 Groundwater Effects

In his presentation of landslide mechanisms, Terzaghi (1950) discussed the importance of groundwater on slope behaviour and instability. Terzaghi stated that the majority of landslides could be attributed to abnormal increases in piezometric pressures. Although sackung processes
have been morphologically and mechanically differentiated from rapid slope movement processes, it is likely that groundwater plays a significant factor in both.

Despite Terzaghi's emphasis on groundwater, relatively few studies of sackung have included a discussion of hydrogeologic conditions, or their potential role in the deformation process. The difficulty in acquiring piezometric data was acknowledged as the primary reason why groundwater effects were not discussed. Without detailed subsurface data the hydrogeologic conditions, and their contribution to rock slope deformation behaviour, can only be considered in a speculative framework. The following examples represented the few cases that have provided some discussion of groundwater in consideration of the sackung process.

Holmes and Jarvis (1985) suggested that elevated groundwater levels, presumably coincident with deglaciation, may have been a significant factor in the large scale toppling failures investigated in the Scottish highlands. Their study reports uphill-facing scarps as evidence of large scale toppling. Groundwater pressures were inferred to have facilitated the toppling process by reducing shearing resistance along rock mass discontinuities. As their analysis considered movements to be post glacial and presently dormant, current piezometric data would not confirm their hypothesis unless contemporary movements were also being monitored.

The potential for slope deformation features such as uphill-facing scarps to act as localized catchment areas for precipitation and snow melt was introduced by Savage and Varnes (1987). The corresponding increase in groundwater levels was suggested as a means of developing seasonal slope movements. The computer modelling work that formed the basis of their slope analysis did not include groundwater, so the analysis was in terms of total stress only. They stated that pore water pressures would increase the depth of failure.

Bovis (1990) implicated ground water effects in the slope deformation process at Affliction Creek. A correlation between periods of accelerated slope movements and higher snowpack recharge was qualitatively expressed. Modelling of large scale toppling phenomena in natural slopes by Pritchard and Savigny (1991) addressed groundwater conditions by assuming a fully saturated slope. This approach was taken to simulate conditions prevalent during deglaciation when slope instability, due to glacial oversteepening, may have initiated slope movements.

The Wahleach rock slope, investigated by B.C. Hydro, represents one of the few sackung type cases with detailed piezometric data. The groundwater information, when combined with
the extensive surface and subsurface movement data, has provided a unique opportunity to evaluate groundwater effects on sackung type behaviour. Groundwater monitoring has indicated that the long term piezometric surface lies below the zone of deformation. The potential for transient groundwater pressures, due to significant precipitation events was noted by the authors, although a direct correlation between precipitation and slope movements was not initially established (Moore et al. 1992).

Selby (1993) illustrated the condition whereby piezometric fluctuations in jointed rock can greatly exceed that in soils, as earlier proposed by Terzaghi (1962). This condition highlights the potentially important impact that transient groundwater conditions can impart on rock slopes during periods of significant precipitation or snowmelt. Transient groundwater pressures have the potential to develop incremental creep movements, as suggested by Bovis (1990), although Radbruch-Hall (1978) outlined the difficulty in distinguishing incremental from continuous creep due to the slow rates of movement. Ideally a combination of detailed slope movement and piezometric data, with support from climatic records, would provide the required input to address the role of groundwater in sackung processes.

2.12 Potential for Sackung to Evolve Into Rapid Movement Processes

Several researchers have addressed the potential for sackung behaviour to evolve into rapid failure types. Terzaghi (1950) stated that nearly all landslides have been preceded by creep type movements prior to failure, reflecting the changing ratio of strength to stress. A number of large catastrophic failures have been investigated which display evidence of prefailure creep behaviour, including:

1/ Goldau, Switzerland (Zay, 1807, reported by Terzaghi 1950 and Radbruch-Hall, 1978).
2/ Frank Slide/Turtle Mountain (McDonnell and Brock, 1904; Terzaghi, 1950).
3/ Vaiont Slide, Italy (Patton, 1974).

These examples involved geologic conditions (structure and/or lithology) that were favourably oriented for sliding type processes. As a result, "true" sackung behaviour may not have been a stage in the development of these failures. The major issue in the potential evolution of slow creep behaviour into rapid sliding revolves around the development of a basal failure surface upon which rapid failure becomes possible. Sackung processes by definition do
not involve movement along a single discrete shearing surface, however with progressive
deformation and rock mass fragmentation comes the potential for concentrated failure surfaces
to develop.

Evidence for the development of large rock slides (or avalanches) from precursory
sackung type deformation has been presented by Chigira and Kiho (1994). They investigated a
number of large slides in sedimentary rocks in Japan which displayed classical sackung features
above existing slide scars. Slow, progressive downslope bending of steeply dipping bedding in
sedimentary rocks developed extensive internal deformation and fragmentation within the rock
mass, facilitating the condition for rapid sliding.

McAffee and Cruden (1996) presented evidence supporting the formation of surfaces of
rupture, along which sliding is possible, from slow precursor toppling failure processes in the
Canadian Rocky Mountains. A similar scenario was considered by Pritchard and Savigny (1991)
to explain the circular failure scar observed at the Heather Hill landslide in British Columbia.

2.13 Deformation Chronology

Many documented cases, in describing surficial deformation features on large natural
slopes, have noted a lack of fresh or recent signs of movement, implying a current state of
dormancy (Holmes and Jarvis, 1985; Radbruch-Hall, 1978; McAlpin and Irvine, 1994). The slow
nature of the process may account for the obscuring of deformation evidence, leading to an
erroneous interpretation that active processes are quiescent. It is possible that the surficial
environments on affected slopes have the ability to mask the internal slope deformation by
adapting to the slow changes. An example is the Wahleach slope on which several well
developed linear troughs were identified that contained trees 100 to 200 years old. The troughs
did not display "fresh" signs of slope movements in the form of tension cracks, nor did the
vegetation show obvious signs of disturbance, yet detailed monitoring of subsurface instruments
(inclinometers and extensometers) revealed movement rates of up to 40 millimetres per year
(Moore et al., 1992).

Distinguishing the activity of sackung cases has been approached in two general fashions;
either by direct monitoring to determine if movements are current, or through the use of indirect
dating techniques such as dendrochronology or tephrochronology.
2.14 **Slope Monitoring**

A number of instrumented sites have been reported. Studies by Rabruch-Hall et al. (1976) and Varnes et al. (1990) included discussion of surficial wire extensometers to monitor potential activity of sackung features on Bald Eagle Mountain, Colorado. Guerrecchio et al. (1988) also used wire strain gauges to monitor displacements of sackung features in the Italian Alps, and also incorporated seismographs to evaluate seismic effects. B.C. Hydro has employed extensive instrumentation arrays in the evaluation of movements at several large rock slopes in British Columbia such as the Downie Slide, Dutchman's Ridge, Wahleach and Checkerboard Creek, the latter two of which conform to sackung type phenomenon. Data from these B.C. Hydro sites has supported active gravitationally driven processes. Bovis (1982, 1990) employed repeat survey techniques to monitor displacements across open tension cracks at Affliction Creek, B.C. Similar survey techniques have been used by Bovis and Evans (1995) to investigate movements along the Mt. Currie scarp. A new survey network has recently been installed across sackung type features above the Mystery Creek slide scar in southwest British Columbia, the site of a large, rapid prehistoric rock slide (Bovis, personal communication).

These direct measuring techniques have been shown to be effective in the evaluation of sackung activity and development. Due to the slow nature of sackung processes a considerable monitoring period (3 to 5 years) has often been required to develop a reasonable understanding of the movement rate patterns, such as at the Wahleach rock slope (B.C. Hydro, ref. 5).

2.15 **Trenching Investigations**

Several North American studies have involved trenching investigations of sackung features, providing an instructive perspective on sackung development. Trenching operations allow investigation of several details, including the timing of movements (Beget, 1985) and the relationship to structural features (McAlpin and Irvine, 1994).

Detailed trenching work across sackung features near Glacier Peak in Washington State was reported by Beget (1985). This work focused on the stratigraphic relationship of tephra deposits to faults and sackung features. Knowledge of tephra deposit ages was used to argue that the development of the sackung features was post glacial, and probably episodic, but considered essentially dormant at present. The stratigraphy of the tephra deposits provided evidence, through the lack of growth faults in the ash layers, that was interpreted as support for gravitationally
driven processes, and evidence against seismically initiated slope movements.

McAlpin and Irvine (1994) documented a field study of sackung features that included the trenching and mapping of an upslope-facing scarp on the upper slopes of the Aspen Highlands Ski Area in Colorado. Dating of organic material within the trough provided age estimates indicating a post glacial origin for the sackung features. The infilling of the trough with progressively younger deposits was considered to represent the slow progression of the sackung deformation at an averaged rate of 0.4 mm/yr since deglaciation. Rotated bedrock structure across the trench implied flexural toppling as the prominent deformation mechanism.

2.16 Summary

Slow deformation of large natural rock slopes is a phenomenon seen throughout mountainous regions of the world, in a wide range of geologic settings. A general agreement has developed among many researchers that these processes are gravitationally driven, although the role of seismicity as a potential trigger remains in question.

A limited understanding of this diverse phenomenon currently exists, primarily due to the lack of field studies which have been able to incorporate actual quantitative measures of the process. Although some studies have provided excellent data of surficial deformation a more complete understanding of the process requires detailed subsurface investigations.

Long term monitoring of slowly deforming rock slopes should eventually record the effects of a significant seismic shaking, although actual "seismic triggering" would be more appropriately addressed in previously inactive slopes. The wealth of data acquired in several short moments of intense seismic shaking could surpass that developed in many years of study, particularly if several monitored slopes representing different geologic conditions were affected.

Numerical modelling appears to be an investigative method with large potential for exploring aspects of this deformation process. Integration of numerical modelling studies with comprehensive field investigation programs appears to be the most likely approach to further develop the understanding of this process.

This investigative approach could help evaluate the evolutionary nature of this phenomena, and help answer the following outstanding question: Do these slow slope movements accelerate with time and develop the potential for rapid mass movements (i.e. rock slides/avalanches); or do they self-stabilize and cease to deform, leaving their characteristic
signature of slope deformation features behind for us to ponder?

2.17 Potential Study Areas

Specific deficiencies in the understanding of sackung processes have been identified. A more comprehensive knowledge of sackung requires a more quantitative approach, most notably in the following areas:

1/ Integration of subsurface deformation studies with surficial geomorphology and slope movement data. This information would provide a clearer understanding of the depth of movement and the actual failure mechanics.

2/ Incorporation of groundwater effects on slope deformation (including both piezometric data and detailed weather records). This would provide some appreciation for the effects of transient groundwater pressures, and the potential for these climatically related conditions to initiate incremental deformation. This would also allow a means of determining whether steady state conditions existed in slowly deforming slopes, implying actual creep type processes, or alternatively, if variable stress conditions existed related to fluctuating groundwater pressures that were responsible for the movements. In this case, "creep" would not be an accurate physical description of the process.

3/ The use of numerical modelling to investigate stress and deformation patterns, and for investigating various loading conditions (ie. seismicity, flooding).

a/ Numerical modelling would be instructive in developing the understanding of the most appropriate rheological relationships in sackung processes. Comparative modelling work would allow analysis of the deformation patterns to determine if the representative in situ conditions were being captured by the model. This could also provide an understanding of the deformation mechanics, and hence a better understanding of the origins of the process. Accordingly, various stress-strain relationships, such as elastic, elastic-plastic or time dependent creep (viscoplastic) could be investigated to determine which provided the most accurate representation of observed field conditions.

b/ Numerical modelling would also be instructive for investigating the significance of various external or internal effects on the sackung process (ie. as an experimental "what-if" tool). External effects such as extreme precipitation events
and elevated groundwater conditions, seismic events and natural slope erosion/undercutting could be modelled, as could internal effects such as weathering, strength reduction and the effects of progressive deformation/fragmentation associated with ongoing slope movements.

4/ Incorporation of detailed field trenching investigations across significant sackung features to:

a/ Develop a better understanding of the relationship between these features and geologic structure. For example, determining if a causative relationship between sackung morphology and pre-existing structural features, such as fault zones, was likely.

b/ For further investigating sackung origins. This could aid in the understanding of the timing of the process in relation to seismic events and deglaciation, which would thus allow an evaluation of sackung development (i.e. is an additional trigger required to initiate the process, or can develop under normal gravitational stresses.

This project aims to focus on the aspects described above by carrying out a detailed analysis of the Wahleach rock slope. The geologic, hydrogeologic and slope movement data available for this case allow for the formation of a detailed model to address these issues. This representative model can then be analyzed with a state-of-the-art computer analysis program (FLAC) for investigating the large disturbance and currently active behaviour of the slope. The lessons learned in the modelling of the Wahleach slope should be instructive in developing a more complete understanding of these sackung processes.
Chapter 3.0 - Geologic Setting

3.1 Introduction

Detailed geologic investigations of the Wahleach slope were carried out following substantial leakage from the ruptured power conduit in 1989. Earlier studies were conducted during the initial construction of the project (1951-52), and in 1981, following an earlier leakage event from the upper tunnel access adit. Details of these investigative works have been documented in a series of B.C. Hydro Engineering Department reports (refs. 1-4).

It was not the intent of this project work to carry out a detailed re-evaluation of the geologic conditions in the Wahleach rock slope. However, it was important to summarize the significant geologic details consistent with conditions in the slope, for the purpose of constructing an appropriate numerical model for the study. The fundamental concern in developing this model was to construct the simplest model possible consistent with the important geological features. This approach intended to dismiss minor details considered to be irrelevant to the overall deformation process. The incorporation of extensive detail into a complex model was found to limit the model's practicality, primarily by causing the modelling analyses to run at increasingly slower speeds, and by complicating the interpretation of the output.

3.2 Regional Geology

The Fraser River valley, west of Hope, B.C., forms the physiographic margin between the Coast Mountains and Cascade Range, which are characterized by different basement rocks. However, for a distance of up to 25 kilometres west of Hope, B.C., including the vicinity of the Wahleach slope, intrusive rocks typical of the southern Coast Mountains traverse the Fraser River to the south and east (Monger and Journeay, 1994).

Plutonic rocks mapped as the Mount Barr Batholith (Monger, 1970) form a collection of intrusive bodies around Wahleach. Two phases within the Mount Barr complex, termed the Mount Barr and Conway phases, occur within the area of the hydroelectric project that spans from Wahleach Lake to the Fraser River. These two phases, comprised chiefly of Miocene granodiorite, dominate the steep eastern slopes of the Fraser Valley.
3.3 Tectonic Setting

Major structural features within the region consist of Tertiary faults considered to be currently inactive (Monger and Journeay, 1994). The majority of these faults have a northwest to northeasterly trend, which was important when considering the associated structural fabric, such as joints and shears, developed in the surrounding rock mass.

The northeasterly trending Vedder fault has been mapped immediately south of the Wahleach site, but its potential projection through the Wahleach project area has not been confirmed. The Vedder fault was active during the Miocene, approximately 25-14 million years before present (Monger and Journeay, 1994), hence it may be possible that the Mount Barr Batholith could truncate the fault. Coish and Journeay (1992) discussed the role of these northeasterly-trending faults as magma conduits and their control on the emplacement of plutonic rocks. This provided a possible explanation for the lack of evidence or projection of the Vedder fault through the Wahleach site, as no regional scale faults have been locally mapped (Moore et al., 1992).

3.4 Site Geology

3.4.1 Rock Types

The detailed investigative work conducted in 1989, supplemented with 1951/52 and 1981 investigations, provided extensive coverage of the site geology, both surficially and underground. Detailed surface mapping was carried out during 1989 across the main region of the deforming slope and was extended to the adjacent slope areas to delineate the geology and potential surficial expression of slope movements. Mapping of tunnel exposures and detailed core logging of the drill holes provided considerable information on the subsurface geology. The primary rock unit encountered throughout the site was a medium to coarse grained granodiorite with frequent dioritic xenoliths and minor felsic and mafic dykes, which were not considered to affect the rock mass quality.

A prominent feature of the rock was the localized occurrence of inter/intra-crystalline microfracturing. Where present, the pervasive microfracturing generated friable, incompetent conditions in an otherwise hard and strong rock mass. Much of the friable rock conditions occurred above the actively deforming section of the slope (ie. above elev. 850-900 metres), but limited occurrences were also seen at depth in drill core bordering fractures and shears. Thin
section analysis determined that chemical weathering was not the origin of the microfracturing, although it may subsequently alter the afflicted rock. This suggested a mechanical origin for the microfracturing and diminished rock quality. A number of possible causes for this could arise from rapid tectonic uplift and stress relief, tectonic faulting, and slope movements (D.P. Moore, personal communication, 1996). Similar friable rock mass conditions have been observed at several other sites in B.C. where large scale slope movements have occurred, or are presently active, such as at Dutchman's Ridge near the Mica Dam, or the Checkerboard Creek rock slope near the Revelstoke Dam.

Weathering within the rock mass was generally confined to discontinuity surfaces and was not pervasive through the intact rock. Weathering was typically manifested by the oxidative degradation of iron bearing minerals such as biotite, producing the characteristic brown "iron stain" on continuous discontinuity surfaces. The degree of weathering generally progressed from slightly to moderately weathered conditions in surficial sections of the slope to fresh conditions at depths in the order of 200 metres or more, although localized occurrences of fresh rock at surface and slightly weathered conditions at depth exist. The terminology used to describe weathering was that typically used by the B.C. Hydro Geotechnical Department, and was generally consistent with the ISRM terminology (Brown, 1993).

3.4.2 Structural Geology

The structural geology is dominated by joints and shears within the Wahleach rock mass. As mentioned in Section 3.3 no regional faults have been observed within the confines of the investigated area.

Joints, defined as fractures or cracks without observable displacement parallel to the plane of the fracture, exist throughout all levels of the rock mass. No distinct joint sets were recognizable, but general trends for steeply dipping, north-south and east-west orientations were seen (Fig. 3-1). It should be noted that the stereonet plot represented structural data from surficial and tunnel mapping data, in addition to borehole televiewer survey work, thereby presenting a cross section of structural fabric from all depths within the slope.

Table 3-1 outlines the important characteristics of the joint structure as determined from the 1989-90 investigation work:
Table 3-1: Discontinuity characteristics at surface and 200 metre depth:

<table>
<thead>
<tr>
<th>Joint Characteristic</th>
<th>Near Surface</th>
<th>200 metre depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathering</td>
<td>slight to moderate</td>
<td>fresh</td>
</tr>
<tr>
<td>Discontinuity Spacing</td>
<td>10 - 20 cm</td>
<td>&gt; 1.0 m</td>
</tr>
<tr>
<td>Orientations</td>
<td>highly variable</td>
<td>predominantly steep</td>
</tr>
</tbody>
</table>

Shear zones were observed to be well distributed throughout the rock mass, with spacing in the range of 20 to 40 metres, and lengths of up to several hundred metres. Shear zone orientations were typically sub-parallel to the strike of the slope (within 45°) with steep dip angles (greater than 45°). Development of gouge material and crushed or decomposed rock was typically less than 50 mm in thickness, with associated zones of increased fracturing creating overall shear zone thickness in the range of 0.5 to 5.0 metres. Shear zones typically displayed iron staining, chloritization and slickensides, and where discernible, offsets of less than 2 metres.

A number of the more significant shear zones were identified by specific names during the field investigation work (Fig. 3-2). Several prominent shears were associated with significant groundwater flows when encountered during tunnel excavation. Differential hydraulic heads across several shear zones, of up to 20 metres, were revealed during piezometric testing of the 1989/90 drill holes. The important characteristic of these zones is their potential to present continuous low permeability barriers to groundwater flow in the slope. These low permeability zones can obstruct downslope groundwater flow, maintaining higher upslope hydraulic heads.

Prominent among the shear zones is the "concrete lining fault" (ref. 5), a substantial structural feature that projects for up to several hundred metres through the rock slope. The strike of this zone parallels slope contours and dips 45° to 50° downslope. It projects to the surface near the break in slope gradient at 950 metres elevation. A general trend of reduced rock mass quality downslope of the concrete lining fault was noted.
3.4.3 Surficial Deposits

Thin, discontinuous deposits of locally derived ablation till and colluvium, typically less than 3 metres in thickness, cover the slope. The presence of these surficial deposits was not considered to have any significant impact on the slope deformation behaviour, except perhaps by channelling surface runoff in more concentrated patterns.

3.5 Geomorphology

3.5.1 Physiography

The deeply incised creek channels along the north and south margins of the slope create a prominent convexity in the Wahleach slope. The main section of the slope displays a modified "U" shaped geometry, with steep slope angles of up to 40 degrees below 650 metres elevation, and a progressive flattening towards the upper ridge crest (Figs. 3-2, 1-2).

3.5.2 Slope Morphology

The Wahleach slope displays a series of well developed antislope scarps (linears), similar to sackung type features that have been documented at many other sites around the world. The larger antislope scarps traverse the slope between elevation 400 and 950 metres, and are up to 10 metres in width. The growth of large trees in the scarps, estimated at over 100 years in age, suggest a long, but slow history of development (ref. 1).

3.6 Seismicity

Southwestern British Columbia is located over the active Cascadia subduction zone, creating a seismic climate similar in nature to Japan and the west coasts of South and Central America (Rogers, 1994). Despite geometric similarities, historical seismicity along the Cascadia subduction zone itself has not produced the large magnitude earthquakes characteristic to other subduction boundaries. Considerable research (Atwater, 1987; Clague and Bobrowsky, 1994) has been collected to support the potential for large damaging earthquakes, although much uncertainty exists regarding the extent of fault rupture and the potential for damaging effects to be felt as far inland as Vancouver, or the Lower Fraser Valley. Generally soft soil sites are more susceptible to longer period earthquake energy that can propagate substantial distances. Therefore, the Wahleach rock slope, located some 250 to 300 kilometres east of the potential
Subduction fault is probably too remote to receive substantial damaging energy from the large subduction event.

Convergence of the Juan de Fuca and North American plates generates frequent seismic activity in two source areas; shallow (< 30 km depth) earthquakes in the over-riding North American continental crust, and deeper (45-65 km depth) earthquakes in the subducting Juan de Fuca oceanic plate (Rogers, 1994). Microseismicity defines the general outlines of these active areas (Fig. 3-3). Historic earthquakes as large as magnitude 7.4 and 7.0 have occurred within the North American plate and the subducting oceanic plate, respectively.

3.7 Geologic Model for Numerical Modelling

The following discussion summarizes the important geologic features that were integrated into the model for numerical modeling. These were considered to be rock mass quality, rock mass weathering and rock mass structure.

3.7.1 Rock Mass Quality

The Wahleach rock mass quality has been characterized by transitional rather than abrupt changes (Moore et al., 1992). A gradual progression from poor/fair rock mass quality near surface (i.e. upper 60 to 100 metres) to good rock mass quality at depth appears to be consistent throughout the slope, and particularly in the deforming or disturbed regions. Rock mass quality for the geologic model was based on important rock mass features, including joint spacing, joint conditions (infilling, roughness, aperture), weathering, and strength values determined from point load and unconfined compressive strength (UCS) testing. A log of drill hole DH89-S1 highlights the transitional nature of the rock mass quality (Fig. 3-4), a trend also seen in holes S2, S4, and S7.

The Wahleach rock slope geologic model represented this improving trend in rock mass quality with depth by imparting a progressive strength increase, and reduced deformability below the slope surface. The empirical Hoek-Brown rock mass strength criterion provided a widely accepted means of estimating the strength of jointed rock masses (Hoek et al., 1995). The 1995 and 1996 updates of the Generalized Hoek-Brown classification included an estimate of rock mass deformability, based on estimates of the rock's elastic modulus.
An important concept in the overall framework of the geologic model was the treatment of the deteriorated rock mass conditions that are, in part, due to slope movements. An early component of the diminished rock mass conditions were certainly related to previous tectonic activity during the Tertiary. The presence of preexisting tectonic structures has been considered to facilitate slope movement processes when subjected to the stress conditions prevalent in large natural rock slopes (Naumann and Savigny, 1991). A positive feedback mechanism involving the formation of prominent tectonic structures, with associated deterioration of rock mass quality, and the development of slope instability was considered in the model. The geologic model constructed for the Wahleach slope attempted to account for this process directly by introducing a progressive reduction in rock mass properties, derived from the Hoek-Brown method, that were consistent with observed conditions in the upper section of the rock slope.

3.7.2 Rock Mass Weathering

This section presents a discussion of weathering effects in jointed rock masses, and provides the background for the modelling of rock mass weathering in the Wahleach slope model, described in section 7.6.3. In the context of this discussion weathering refers to the degradation in rock mass properties due to a combination of physical (mechanical) and chemical processes. The integration of diminishing strength and increasing deformability during rock mass weathering was considered to be essential in developing representative model behaviour. The modelling of weathering effects in this project represent a unique approach to the treatment of rock mass conditions in slope stability analysis.

Of prime concern to slope stability is the reduction in shear strength and the change in deformability and permeability of a rock mass, that accompanies weathering. Mechanical effects include the development of jointing, shear and fault zone formation, the shearing of asperities (roughness) along the discontinuities and the formation of fracture infilling. Chemical processes involve the alteration of original mineral composition, such as the alteration of feldspar into kaolinite or sericite, with limonite. In near surface environments, applicable to the Wahleach slope, chemical alteration is frequently a solution process, in which groundwater generates acidic conditions conducive for mineral alteration.

Physical and chemical weathering mechanisms generally act in unison, although Selby (1993) suggested mechanical degradation, such as joint formation, must initially develop
conditions suitable for chemical alteration, since the low permeability of fresh intact rock precludes the chemical attack of aqueous solutions. Consequently, the presence of fractures, both at the microscopic (intracrystalline microcracks) and macroscopic (joints) scale, are a necessary requirement for water to migrate into a crystalline rock mass, and initiate the chemical breakdown of the mineral components. Groundwater flow then plays another important role in the weathering process through the removal of weathered products, thereby enhancing the capacity for further chemical alteration. These factors act in unison throughout the near surface vadose zone, where fracture density is typically highest, leading to an enhanced weathering profile above the water table (Goodman, 1993), a condition consistent with the upper 50 to 150 metres of the Wahleach rock mass.

The development of rock joints/fractures is generally attributed to stress release, such as by the removal of overburden loads which can lead to the formation of exfoliation, or sheeting, joints; or by the release of locked in tectonic forces during faulting. Stress release processes, and the associated fracturing, are most prevalent in the surficial 50 to 150 metres of a rock, and typically show a progressive increase in fracture spacing and rock mass quality with increasing depth. Tectonic faulting, manifested in localized rock mass fragmentation and fault/shear zone formation, exerts a significant role in the distribution of variable rock quality. Investigative work at Wahleach revealed the importance of both stress relief and faulting mechanisms in the development of the existing rock mass structure.

It is also likely that subsequent slope movements have modified the observed rock mass conditions. Important observations included the deterioration in rock mass quality downslope of the Concrete Lining fault (ref. 5), and the general improving trend of rock mass quality with increasing depth. The coalescing of several fault zones, including Kim's, Seki's and the Concrete Lining Fault near the break in slope gradient at elevation 950 metres, corresponds to reduced rock mass conditions downslope of these features. These conditions were observed to extend downslope to approximately 400 metres elevation, just upslope of drill hole DH89-S9, so that a significant portion of the main slope between elevation 400 and 950 metres comprises a comparatively poor rock mass quality in relation to that observed outside these limits. These reduced conditions are considered to have a significant effect on the potential development of slope movements. Surficial evidence of slope movement has not been identified above the point in the slope where these faults project to the surface, whereas movements at surface and in the
downhole instruments have been conclusively observed downslope of this point. This section of reduced rock mass properties extends downslope to approximately 400 metres. All the large antislope scarps are located between these two points.

Two fundamental changes within the Wahleach slope rock mass were considered to take place during evolution of the rock slope. With unloading, due to erosion of overburden material and glacial withdrawal, a rebound response would occur. The combined effect of elastic and plastic rebound deformation would be expected to change the nature of the rock mass, and in particular, the conditions of the joint surfaces. Movement would be accompanied by a shearing of asperities along the joint surfaces, resulting in reduced shearing resistance. Rock slope movements associated with unloading would subsequently increase the exposure of the rock mass to chemical weathering. A positive feedback mechanism is believed to occur in the rock mass initiated by the fracturing and fragmentation developed during tectonic faulting and slope formation. The fractured rock mass is then primed for chemical weathering processes, and potential slope movements that in turn facilitate the weathering-movement cycle. The modelling strategy incorporated this feedback mechanism.

The overall weathering effect was simulated in the modelling analysis as a progressive reduction in strength values, with a corresponding decrease in deformation moduli. In essence this is analogous to a "strain-softening" response progressively developed throughout the afflicted portion of the rock slope. It could be argued that to present a realistic picture of the material stress-strain behaviour, the results of representative testing (either lab or in situ) are required. Certainly this is valid for capturing the short term behaviour of earth materials, as both rocks and soils generally display a markedly different behaviour after limited levels of strain. Extensive laboratory testing of both intact rock and rock joints has revealed a characteristic strain-softening behaviour with overloading of material strength. This material response has been described as brittle due to the rapid drop in peak shear resistance with limited strain (Brown, 1987) as shown in an idealized stress-strain curve (Fig. 3-5). Barton and Bakhtar (1987) showed that residual strengths on jointed rocks are developed with shear displacements in the order of several millimetres to centimetres. This short term strain-softening represents the mechanical weathering component, whereas the chemical processes represent a more continuous long term response. When combined these effects result in a progressive reduction of peak shear resistance towards residual values, which reflects the approach taken in this analysis.

- 55 -
Two factors have been considered to control the distribution of the weathered rock mass in the Wahleach slope. First, the geometry of the major structural features, including the fault zones, shear zones and associated fracturing create an important framework of mechanically broken rock that allows for accelerated weathering; and secondly, the progressive penetration of weathering effects extending from the surface downwards to depths in excess of 150 metres. The weathering sequence adopted in the modelling analysis included both these effects (Fig. 3-6).

3.7.3 Rock Mass Structure

The rock mass structure at Wahleach was most appropriately characterized by steeply dipping fabric, which included the general trends of both joints and shears. A vertical orientation was chosen to represent the rock mass fabric along which structurally controlled behaviour would be most likely to occur. The potential for rock mass deformation behaviour along other, more random orientations, was incorporated into the overall rock mass strength conditions based on the Hoek-Brown classification described above. This was considered to be consistent with the observed rock mass conditions, including the following points:

1/ The orientation of the predominant structural fabric was considered to be parallel to the slope contours, with steep dips clustered on both sides of vertical (Fig. 3-1).

2/ No major downslope dipping joint sets have been recognized in surface and tunnel mapping, or drill core;

3/ No continuous, downslope-dipping basal shear feature has been observed in the extensive investigation work to date. This condition has been supported by the slope movement data.
Figure 3-3: Seismicity in the Vancouver region from 1980 to 1991 inclusive. Dot size is proportional to event magnitude. The smallest earthquakes are magnitude 1 and the largest is an offshore event of magnitude 5.2. Earthquake data from the Geological Survey of Canada and the University of Washington have been combined to produce this image. Stars are Quaternary volcanoes of the Cascade magmatic arc. (Drawing is taken from Geology of the Vancouver Area, Geological Survey of Canada, Bulletin 481; ref. 99).
Figure 3-4: Summary drillhole log DH89-S1. Note the progressive improvement in rock mass conditions below the surface. From B.C. Hydro Report No. H2780.
Figure 3-5: Idealized stress-strain response for intact rock and jointed rock, illustrating the "brittle" character. From Selby (1993).
Figure 3-6: Distribution of rock mass conditions in upper slope, incorporating effects of rock mass structures (i.e. Concrete Lining Fault), and rock mass "weathering" profile, based on extensive mapping and drilling information.
Chapter 4 - Hydrogeology

4.1 General Conditions

Groundwater conditions in the Wahleach slope have been well documented (Refs. 1-6), based primarily on initial tunnel construction in 1951-52, from 1989/90 investigations and subsequent instrumentation results. Similar to the discussion of geologic conditions in Chapter 3, it was not the intent of this study to re-evaluate the hydrogeology; but instead to develop a conceptual model, consistent with observed, and/or reasonable estimates of in situ conditions, for use in constructing a representative numerical model of the slope.

In this discussion the groundwater regime has been separated into saturated and unsaturated (vadose) zones, which show natural hydraulic connection, but distinctly different flow behaviour. The long term phreatic surface was used to differentiate lower regions of the rock mass where continuous saturation has prevailed, from the upper rock mass where unsaturated and seasonal flow conditions have prevailed.

4.2 Piezometric Monitoring

Instrumentation results from the array of slope piezometers has defined the general range in position of the phreatic surface (Fig. 4-1). A smooth groundwater profile was adopted for the model, and considered representative, although in reality localized irregularities likely exist throughout the rock slope. Rock mass structure can impart significant anisotropy and heterogeneity with respect to groundwater behaviour (Fig. 4-2) and therefore, the complex structure within the Wahleach slope was expected to preclude a smooth groundwater profile that would prevail in a homogeneous porous material. However, for the analysis presented in this project the smooth profile was considered to provide a reasonable representation of conditions below the phreatic surface.

The most important hydrogeologic observation made from the investigation work was that the zone of progressive slope deformation lay almost entirely above the long term phreatic surface, as determined from reliable instrumentation results between 1989 and 1995. A small region in the toe of the deforming mass, near drill hole S1, registered groundwater levels that coincided with the lower limit of the movement zone. These conditions prevailed during operation of the original power conduit. Following realignment of the power conduit outside
of the zone of deforming rock, significant reductions in piezometric levels in the lower regions of the slope have occurred (Fig. 4-3). Draw down of the piezometric surface has resulted from drainage into the downstream end of the intermediate tunnel.

The piezometers have generally been monitored on a monthly basis, although monitoring frequency has been reduced to a quarterly basis since mid 1994. Several piezometers located below the phreatic surface were connected to data-loggers and therefore, provided near continuous real-time groundwater information. A comprehensive understanding of the groundwater response, below the long term phreatic surface, to seasonal variations and operation of the power conduit(s) has been developed (ref. 5). This behaviour was characterized by 10 to 30 metre seasonal increases in phreatic surface corresponding to late autumn-early winter increased precipitation levels; and fluctuations caused by operation of the original and replacement power conduits. At no time during the monitoring history has a continuous groundwater profile extended upwards into the zone of progressively deforming rock.

Knowledge of the groundwater behaviour in the vadose zone is limited. Spatial and temporal limitations of piezometers located in the vadose zone have precluded a more thorough understanding of groundwater conditions in the upper rock mass. Observations during initial construction of seepage into sections of the original upper tunnel and inclined shaft located above the phreatic surface, indicated the potential for transient groundwater conditions to develop in the vadose zone following heavy rainfall. Furthermore, conclusions drawn from piezometric data (ref. 4) indicated regions of the slope adjacent to the downstream end of the original upper power tunnel were influenced more by climatic effects (precipitation) than operation of the power conduit. These two points indicated that transient groundwater flow, related to precipitation events, dominated the groundwater behaviour in the vadose zone where slope movements have been observed.

An important feature of groundwater flow behaviour in the vadose zone was the potential for development and persistence of localized differential pressure heads. Perched and/or compartmentalized groundwater conditions in the vadose zone were considered in this study to create significant changes in effective stress conditions in the upper rock mass where slope movements have been recorded. Confirmation of the potential for these conditions were noted in drill hole S4 following heavy precipitation in late 1989. Precipitation in November 1989 represented the 25-50 year return period rainstorm, and provided a reasonable estimate of
maximum transient pore water flows and pressures in the vadose zone. Figure 4-4 illustrates two piezometric ports in drill hole S4 located above the long term phreatic surface. Piezometric port 52, at elevation 556.5 metres, was separated from port 49, at elevation 549 metres by a shear zone at elevation 551.5 metres. During November 1989, pressure heads of 10 to 15 metres were recorded in port 52 while port 49 remained dry. Such a large rapid piezometric response is characteristic of low porosity, high permeability conditions encountered in fractured rock masses such as the Wahleach slope (see Figure 4-2b).

4.3 Structural Control on Groundwater Flow

Primary intercrystalline porosity in plutonic rocks, such as the granodiorite comprising the Wahleach slope, are typically less than two percent. These low porosities are frequently associated with low permeability due to a lack of inter-pore connectivity, rendering unfractured crystalline rock essentially impermeable (Freeze and Cherry, 1979). Fracture porosity and permeability provides the principal capacity for groundwater flow in plutonic rock masses. The large volumes of seepage into portions of the Wahleach power conduit highlighted the important relationship between rock mass structure and groundwater flow capacity within the rock slope. This condition was noted during initial tunnel excavation and remains currently valid, although at diminished rates from 1951-52 observations, due to drainage into unlined portions of the power conduit.

The two predominant rock mass structures, joints and shear zones, have exerted significant, but sharply contrasting effects on the groundwater flow behaviour within the slope. The shear zones have generally acted as thin aquicludes presenting barriers to groundwater flow. Conversely, joints have created zones of increased permeability, providing considerable fracture flow capacity, particularly in the upper sections of the rock mass where slope movements have generated a dilated condition with increased joint apertures.

Shears within the Wahleach slope typically have an associated zone of fractured rock bounding their margins. The low permeability silt to clay size material forming the core of the shear zones is therefore "sandwiched" by adjacent, more permeable fracture zones. This combination accounts for the observed groundwater flow behaviour along the steeply dipping structure, with higher flow capacity parallel to the general trends of the major structure, and the tendency for differential groundwater conditions to occur across the lower permeability shear
features. Based on geologic mapping and borehole televiewer investigations, the dominant structural fabric was roughly parallel to the strike of the slope face with a general trend for steep orientations. Piezometric response, particularly in reaction to tunnel operation, supported this general pattern of higher flow capacity in a cross slope direction relative to the down slope direction.

Flow in the vadose zone was expected to conform to these observed trends, although in a transient manner following significant precipitation events. Obstruction of flow, caused by the shears, was responsible for the development of perched and compartmentalized conditions, as discussed in section 4.2.

4.4 Conceptual Hydrogeologic Model for Numerical Analysis

A conceptual model of groundwater behaviour in the fractured rock mass was essential for developing suitable input for the numerical analyses. Similar to the geologic model, the hydrogeologic model was intended to be as simple as possible, but also consistent with observed conditions, and capable of capturing the significant features of the groundwater behaviour. The extensive piezometric information provided the necessary detail of the groundwater regime to develop this conceptual model. Based on the discussion above, three basic features of the groundwater system were considered important for input to the numerical model: 1) the position of the phreatic surface in the slope; 2) the development of perched conditions in the vadose zone; and 3) a means of characterizing the transient nature of the groundwater flow in the vadose zone that accompanies prolonged periods of substantial precipitation. These three features will be discussed in the following paragraphs.

4.4.1 Slope Phreatic Surface

The general position of the phreatic surface in the slope was reasonably well defined, sub-parallel to the slope surface at a depth of 80 to 160 metres. The combination of seepage observations in tunnel exposures, piezometric profile testing during drilling of the 1989/90 investigation drill holes, and monitoring of the array of piezometric ports provided a good indication of long term trends and range in observed behaviour. Two groundwater profiles, representing seasonal maximum and minimum, were required for model input (Fig. 4-1). Also required were the maximum and minimum groundwater profiles that have developed following
completion of the replacement power conduit. Substantial drawdown of the phreatic surface by as much as 110 metres have been recorded.

4.4.2 Perched Groundwater Conditions

Perched groundwater conditions have been observed in piezometers above the phreatic surface, as discussed in Section 4.2. Shear zones impart sufficient anisotropic character to the rock mass that substantial differential pressure heads have developed during periods of significant transient flow. This condition has been observed both above and below the phreatic surface, although only the development of perched conditions in the vadose zone were considered to have influence on the slope deformation process. Consequently, only perched zones in the upper sections of the rock mass were considered for the conceptual model to be used in the analytical work.

The regularity of the perched zones was considered to correspond to the spacing between mapped shear zones which was in the range of 30 to 100 metres. An average spacing of 50 metres was considered reasonable for the hydrogeologic model.

The magnitude of hydraulic heads developed in the perched zones were considered to range between 5 and 17 metres, based on piezometric observations. These large fluctuations were considered reasonable, based on the capacity for low porosity (structurally controlled) rock masses to experience large fluctuations in response to significant precipitation (Fig. 4-2b).

Piezometric data has revealed a relationship between the development of perched groundwater conditions and periods of prolonged heavy rainfall, typical of the Pacific storm systems that occur in late October through February, such as that occurring in November, 1989. Substantial rainfall in isolated storm events, that occur periodically throughout any typical annual cycle, were not considered to be capable of producing the perched conditions described above, based on observed piezometric response. The volume of precipitation that enters the slope as infiltration during isolated storm events was considered to be readily accommodated by the high hydraulic conductivity conditions prevalent in the loosened, highly fractured rock mass in the near surface zone, without the development of significant differential heads across the low permeability zones. In contrast, infiltration volumes that confronted the slope during prolonged storm events were considered to exceed the flow capacity of the rock mass, with notable perched conditions developing along zones of low permeability.
4.4.3 Transient Groundwater Flow

Transient flow conditions refer to the time-dependent behaviour in a groundwater system. Transient conditions develop predominantly in response to changes that occur surficially, such as increased infiltration accompanying precipitation and snowmelt (Freeze and Cherry, 1979). Therefore, the effects of transient conditions were expected to be most significant in the near surface vadose zone.

Significant transient groundwater conditions developed with the steel lining rupture in the upper tunnel in late January 1989. These conditions were sufficient to cause accelerated slope movements, based on a projection of overall slope displacement since the end of tunnel construction (see section 5.2), and the progressive reduction in monitored slope movements following the 1989 event. Estimates of the peak pressure heads released into the rock mass surrounding the ruptured steel lining were in the order of 25 to 30 metres. Elevation difference between reservoir level and the upper tunnel lining break yielded hydrostatic pressure heads of nearly 50 metres \((641.6 - 592.0 = 49.6 \text{ m})\). Head losses of approximately 12 metres accompanied operation of the power conduit, based on the change in observed water levels in the surge shaft. This corresponded to maximum hydraulic heads of approximately 36 metres that were released into the surrounding rock mass during the lining break. Head losses of 5 to 10 metres related to the flow conditions were estimated, resulting in the 25-30 metre pressure heads. Dewatering of the power conduit would have established diminishing hydraulic pressures, however, the presence of low permeability shear zones was expected to develop and maintain differential hydraulic heads and perched zones as discussed above.

Other sources of transient groundwater conditions in the slope were developed by climatic effects and operation of the power conduit. Climatic effects such as heavy rainfall and rain on snow have shown a greater influence response in piezometric ports in the surface drill holes, relative to operation of the original power conduit. This indicates that groundwater behaviour in the upper rock mass, and particularly the vadose zone, has been dominated by transient flow conditions generated during significant precipitation events. The magnitude of the transient forces in the vadose zone were discussed above with respect to the development of perched conditions (section 4.4.2).

Treatment of transient groundwater conditions in the conceptual model are schematically shown on Figure 4-5. Hydraulic heads were considered to develop behind the steeply dipping
shear zones with gradients corresponding to the 5 to 15 metre differential heads. Flow in response to these gradients was expected to be structurally controlled along the persistent steeply dipping joints. Vertical seepage forces were expected to develop in association with the near vertical, structurally controlled flow. Consequently, downward seepage forces were applied to the model in combination with buoyant unit weights in the transient flow regions. The differential hydraulic heads also developed horizontal force imbalance across the shear zones, where perched conditions developed. Applied horizontal seepage forces were used to model this effect through the distribution of horizontal forces. The horizontal seepage forces were applied at 50 m spacing in the model, corresponding to the distribution of the shear zones. The vertical and horizontal seepage forces were calculated as follows:

\[ i/ \text{Vertical Seepage Force} = \gamma_w i_{\text{vert}} A \]
\[ ii/ \text{Horizontal Seepage Force} = \gamma_w i_{\text{horz}} A \]

where

- \( i = \text{hydraulic gradient} = \Delta h/\Delta l \)
- \( h = \text{hydraulic head (metres)} \)
- \( l = \text{dimension of gradient (horizontal or vertical, metres)} \)
- \( \gamma_w = \rho_w g \)
- \( \rho_w = \text{mass density of water (kg/m}^3\) \)
- \( g = \text{gravitational acceleration (m/sec}^2\) \)
- \( A = \text{area of force application (m}^2\) \)

These forces were applied in the vadose zone as shown in Figure 4-5. With infiltration rates, shear zone permeability and orientation assumed to be constant throughout the slope, higher differential heads could be expected in areas with steeper topography. Level ground conditions would result in minimal differential heads, so transient horizontal groundwater forces were neglected in areas of low gradient (ie. less than 5° slope). This seepage force approach was similar to that used by Garga et al. (1995) for analyzing fluid flow through rockfill structures, and consistent with that proposed by Cedergren (1989).
Figure 4-1: Phreatic surface used in FLAC model for calibrative modelling sequence. Water levels based on long-term piezometric monitoring.
Figure 4-2a: Anisotropic groundwater conditions created by rock mass structure (from Patton and Deere, 1971).

Figure 4-2b: Structurally controlled porosity/permeability in rock mass leading to large variation in groundwater levels following significant precipitation (from Selby, 1993).
Figure 4-3: Lowered phreatic surface due to drainage into unlined section of intermediate tunnel, corresponding to post 1992 conditions.
**Figure 4-4:** Perched groundwater conditions registered in MP89-S4. Note the 12-15 metre increase in piezometer port P52, following significant precipitation in November, 1989. This increase persists for almost two months, while P49 remains dry.
Figure 4-5: Simulation of transient groundwater conditions, including both vertical and horizontal seepage forces. The vertical forces are uniform throughout the vadose zone, reflecting the transient flow along the near vertical joint sets. The horizontal forces represent the substantial gradients considered to persist across the low permeability shear zones. The combination of the horizontal and vertical forces coincident at the model gridpoints plot as a downwardly inclined vector.
Chapter 5 - Slope Movement Behaviour

5.1 Introduction

Evidence of progressive deformation of the Wahleach slope has been clearly supported by the following: 1) instrumentation data, 2) deformation of the original power conduit steel lining, 3) the disturbed nature of the upper rock mass, and 4) the morphological features along the slope surface. The pattern of slope movements, supported by observed slope conditions, indicates that the rock prominence between Ted and South creeks has undergone a deformation process mechanically distinct from adjacent slope areas. Therefore, slope movements that have occurred in the central portion of the deforming slope have been considered an independent process and analyzed as such.

5.2 Summary of Slope Deformation Behaviour

For the purpose of modelling, Wahleach slope movement behaviour can be considered in three periods, of markedly different time span:

1/ Long term behaviour covering post-glacial to near-present conditions. Evaluation of this period relied chiefly on geomorphic evidence, such as slope surface landforms, and on rock mass conditions within the rock mass, as described in section 5.2.1.

2/ Short term behaviour corresponding to the period from tunnel construction to the present. This period was quantitatively evaluated based on deformation of the steel lining in the upper portions of the original power conduit, as described in section 5.2.2.

3/ Current slope behaviour corresponding to the period following rupture of the upper tunnel steel lining. Detailed monitoring of slope behaviour was provided by the array of slope instruments, as described in section 5.2.3.

5.2.1 Long Term Slope Behaviour

On a large scale the present slope geometry was considered to have been sculpted by the conclusion of Pleistocene glaciation. At the Wahleach site, no major Holocene landslides have occurred, although large post glacial landslides have been documented in the near vicinity, most notably at the Cheam, Katz, Lake of the Woods and Hope sites (Savigny and Evans, 1994). Considerable down-cutting of the tributary creeks into the slope represents the most significant
post-glacial morphological change in the slope geometry. These ongoing processes were considered to have occurred in conjunction with localized surficial instability and debris flows.

Gravitational mass movements have probably been active in the slope throughout much of the Holocene, as evidenced by the significant morphological features. The large linears (antislope scarps) that exist between Ted and South Creeks (Fig. 5-1) were considered to be definitive evidence of long-term gravitational slope movements (ref. 1). Bovis and Evans (1995b) have investigated numerous sites in similar intrusive rocks in the Pacific Ranges of southwestern British Columbia, and argue in favour of a slow gravitational origin for antislope scarps and associated morphological features. It is possible that Wahleach slope movements date from pre/inter glacial periods, with current activity representing a continuing trend. If so, the large antislope scarps likely represent the post-glacial movement component, since glacial scour could be expected to preferentially erode the surficial protuberances of loose, dilated, highly fractured rock comprising the antislope scarps. It is also possible that the current movement zone does not coincide with previous movement patterns, due to different slope conditions, including different groundwater levels, and changing material properties. Moore et al. (1992) suggested that the lower extent of loosened and weathered rock extending to approximately 150 to 200 m depth may have represented the outline of a prehistoric movement zone.

Prehistoric slope behaviour can only be discussed in a speculative and general context. However, despite the limiting knowledge of previous slope behaviour, a number of conclusive statements can be formulated:

1/ If current surface movement rates of 5 to 10 mm/year are projected over the Holocene (ie. 10,000 years) then cumulative displacements in the order of 50 to 100 metres result. These large values may be possible, but seem excessive in light of the current slope morphology. This suggests that if current movement rates have been consistent, then slope movements may be more recent than 10,000 years.

2/ Based on the dimensions of the linear troughs (antislope scarps) cumulative slope movements in the order of metres to tens of metres have been estimated (ref. 1).

3/ Based on the geological conditions and slope morphology, the movement pattern has been developed by a complex deformation mechanism involving toppling and slip along numerous discontinuities with zones of more concentrated shear failure; in sharp contrast to a mechanism dominated by sliding along a discrete basal failure surface.
5.2.2 Short Term Slope Behaviour (1952-1993)

The deformation behaviour of the rock slope from initial project construction has been quantitatively evaluated based on measured deformations of the power conduit steel lining (steel lining offset survey). Additional information was obtained by noting discrete deformation features along portions of the original upper section of the conduit, including both the upper tunnel and incline.

Displacements measured in the offset survey were established on a comparison of the measured alignment with the projected original alignment extending from regions of stable ground. Figure 5-2 illustrates the movement vectors derived from the offset surveys carried out between 1989 and 1993. The movement vectors revealed total cumulative displacement and average displacement rates between 1952 and 1993. Both the magnitude and pattern of displacements, revealed by offset surveys of the steel lining, provided a picture of the deformation process, but did not reveal details on the precise timing of movements. The leakage observed in 1981, traced to the rock trap drains in the upper tunnel, indicates that slope movements were active at that time.

Notable conclusions from the offset survey include the following (ref. 5):

1/ Deformations were confined to a 100 metre section at the downstream end of the upper tunnel, and the upper 250 metres of the inclined shaft.

2/ Displacement vectors diminished with increasing depth below the slope surface, with maximum displacements of 650 millimetres in the vicinity of the upper tunnel and inclined shaft intersection.

3/ Displacement vectors were approximately downslope with azimuth 260°, and plunged 0 to 35° horizontal, with an average of 20 to 25° below horizontal for the larger displacement values near the downstream end of the original upper tunnel.

Although this section has been entitled "short term slope behaviour", the forty years of deformation history represented in the offset survey has provided a detailed record unparalleled in any other investigation of large natural slope deformation. The "short term" label was used in reference to the overall time frame for these natural phenomena.
5.2.3 Recent Slope Behaviour (1989-1994)

Recent slope behaviour was based on the results of instrument monitoring established in conjunction with the 1989 investigation program. As discussed previously, the instrumentation program in the Wahleach rock slope has provided a level of detailed information rare for such a large case\(^1\), and therefore, provides a unique opportunity to investigate the fundamental nature of these progressively deforming rock slopes.

The arrangement of the various instruments within the slope are shown on plan (Fig. 5-1) and cross section (Fig. 5-3). A variety array of different instrumentation methods have been used to monitor slope movements, including the following: inclinometers, borehole extensometers, strainmeters, surface cable extensometers and monument surveys (refs. 5,6).

The inclinometers have defined the depth and upslope-downslope extent of the slope movements, as shown on Figure 5-3. The inclinometer profiles clearly reveal the distribution of deformation with depth, and highlight distinctive deformation behaviour in different areas of the deforming slope. In the lower section of the slope, inclinometer S9 has revealed that no measurable deformation has extended to this level in the slope (Fig. 5-4). Inclinometer S1 has shown a uniformly dispersed deformation profile extending to a depth of approximately 120 metres, indicating that no concentrated zones of deformation project through the lower portion of the slope. This profile is consistent with a toppling mode of failure in the lower movement area. Concentrated zones of deformation, up to 10 metres in thickness, have projected through inclinometers S7, S8 and S10, indicating a movement trend in the upper section of the deforming slope dominated by shear failure along a correlative zone. There is no conclusive evidence to date indicating that this concentrated zone of deformation extends as far downslope as either S1 or S9. S5 shows essentially zero displacement, constraining the upslope projection of slope deformation. Figure 5-3 indicates that the movement zone probably does not extend above the concentration of shear zones centred by the "concrete lining fault", and most likely is progressively dispersed throughout the poorer rock mass conditions noted downslope of the concrete lining fault. A dispersed deformation profile supports the fact that a definitive headscarp has not developed in the upper section of the Wahleach rock slope.

\(^1\) B.C. Hydro is currently monitoring several other large natural rock slopes (Downie Slide, Dutchmen's Ridge and Checkerboard Creek, all along the Columbia River valley) undergoing slow deformation phenomenon, although only Checkerboard Creek appears to be characterized by similar dispersed deformation mechanics.
Non-uniform deformation rates were noted in the inclinometer plots, however, the nature of these instruments and their reading frequency have limited the sensitivity of these measurements. Consequently, periods of acceleration and deceleration were difficult to distinguish conclusively, although S7, S8 and S10 showed some correlation to autumn precipitation periods.

The strain meter located at the lining break (UTSM-9) has provided the most detailed movement record. The reason for this higher sensitivity is that UTSM-9 has measured extensional displacement across a zone of concentrated deformation that projects upslope through the deformation zones in inclinometers S7 and S10. In contrast, the other strain meters, located on the buckle zones in the upper tunnel steel lining (Fig. 5-3) have measured compressional deformation distributed over broader zones. Therefore, these instruments provide a less sensitive measure of the displacement history.

Movements recorded in UTSM-9 illustrate clearly that recent slope deformation has not been consistent with time, and equally important, has been marked by periods of acceleration corresponding to periods of prolonged, significant precipitation (Fig. 5-5). Also clearly apparent in this plot is the reduction in movement rate following the lining break event of 1989. The deformation pattern and history in UTSM-9 establishes the correlation between accelerated slope movement and transient groundwater conditions associated with significant precipitation periods, and therefore, provides the key for comparing the numerical model response to recorded slope movements.

5.3 Interpretation of Slope Deformation Mechanics

Having established significant details of the recorded slope movements, the following summary comments regarding general slope behaviour, which are consistent with instrumentation results and general slope conditions, are provided below:

1/ Zones of relatively concentrated shearing were noted in the middle to upper areas (S7 to S10) of the deforming mass.

2/ The areas of concentrated shear deformation were confined to 5-10 metre wide zones.

3/ Deformation patterns were generally diffuse in the lower area of the deforming mass (S1), indicating a general rotational behaviour suggestive of toppling.
Slope movements were negligible in the vicinity of inclinometers S5 and S9, which provided the respective upslope and downslope extent of the deformation zone.

The deformation history of the Wahleach rock slope reflects a delicate balance between material strength and *in situ* stress conditions. Natural slope angles in excess of 40° exist, with average angles in the order of 35°. Intrinsic shear strength parameters for the rock mass and prominent structural features (i.e., steeply dipping joint sets) are considered sufficient to sustain the shear stresses within the slope. This condition has been confirmed by simple static limit equilibrium analyses, which yielded factors of safety in the order of 1.2 to 1.8 (ref. 107). Furthermore, the potential for shear failure along hypothetical, daylighting and through-going discontinuities, based on monitored piezometric levels, would only be feasible if continuous, low strength infilling material were present. Since no evidence of a continuous through-going shear surface was found in the extensive investigation work, an alternative explanation for the slope movements must be found. The following scenarios provide two possibilities:

1/ Time dependent behaviour or other non obvious mechanisms are occurring.

2/ Additional loading conditions are responsible for driving the slope movements, such as transient groundwater flow.

The first scenario is inconsistent with observed conditions and monitored behaviour. Previous evaluations of the slope behaviour have referred to the process as steady "creep" (ref. 1), and little emphasis has been placed on the evaluation of changes in movement rates. Creep behaviour is not consistent with observations in the slope instruments, and particularly with UTSM-9. This instrument provides a level of detail and sensitivity that can be used to base a representative evaluation of the deformation behaviour. Treatment of the slope movements strictly as a time dependent creep process does not address the important changes in effective stress conditions that occur seasonally, and which correspond to changes in the slope movement rates.

This implicates transient loading conditions, presumably related to groundwater flow, as the most likely cause for triggering the periodic accelerations in the Wahleach slope movements. Consequently, the modelling work will focus attention on modelling changes in effective stress related to varying groundwater conditions.
Figure 5-1: Site geology plan, showing slope morphology (linears), and instrumentation layout. From B.C. Hydro Report No. H2153.
Figure 5-2: Upper tunnel and incline offset survey, showing accumulated deformation of the steel lining in the original tunnel. Displacement values are shown in millimetres. From B.C. Hydro Report No. H2780.
Figure 5.6
Adverse rate plot for slope measurements in section of lining break LTSM.

9 spans across the lining break from B.C. Hydco Report No. 92780.
6.1 Introduction

Numerical modelling is a mathematical simulation of physical processes, used to gain an understanding of the response of a system to specified loading conditions. Selecting the most appropriate mathematical model requires an understanding of the physics and representative mathematics, a knowledge of the conditions under investigation and a well defined idea of the modelling objective(s).

Numerical modelling of the Wahleach slope comprised the core of this project, and therefore, a presentation of the fundamental components of the modelling work shall be included in this chapter. This section begins with a general discussion of the important components comprising the Wahleach numerical model. The purpose of this discussion is to elucidate the most appropriate numerical method for analyzing the large displacements and complex material behaviour characteristic of the Wahleach slope, as well as other large, and potentially similarly deforming slopes.

6.2 Waheach Rock Slope Numerical Model

The Wahleach rock slope was modelled as a continuum material using a two dimensional, plane strain approach, based on a Mohr-Coulomb, elastic-plastic formulation, that is dilational upon yield. Additionally, a slip plane surface, termed the ubiquitous joint model, was incorporated in the numerical formulation to account for the anisotropy imparted in the rock slope by the predominant joint set(s). This modelling approach was considered a reasonable representation of rock behaviour, based on testing experience, which will be described further in section 6.3. The numerical analysis was carried out using a finite difference formulation provided in the FLAC computer code (commercially marketed by the Itasca Consulting Group, 1993).
6.2.1 Continuum vs. Discontinuum Approach

There has been considerable debate concerning the suitability of modelling earth materials as a continuum. In general, similar arguments apply to both soils and rock. Soils are particulate in nature and most rock masses contain a variety of discontinuities such as joints, shears and faults. The argument for modelling soils as a continuum material rests on the issue of scale. The problem of analyzing each individual soil grain is prohibitive, and when the size of the problem considered is large in comparison to the size of individual soil grains, the material can be satisfactorily treated as an equivalent, porous continuum (Chen and Baladi, 1985).

A similar argument has been considered for the treatment of rock as a continuum. Where the spacing of the rock mass discontinuities is small in comparison to the scale of the problem under consideration, a continuum approach again may become appropriate, as suggested by Brown (1993). In the case of the Wahleach rock slope, average discontinuity spacing in the deforming zone was observed to be in the range of 5 to 15 centimetres. For the 1100 metre height of the slope, this average rock block size was considered analogous to fine grained sand (grain size of 0.250 mm) in a 5.5 metre high embankment. Consequently, modelling of the Wahleach rock slope as a continuum was deemed to be reasonable.

The above argument can potentially break down in rock masses containing discontinuities that exist in sets with preferential orientations, a common condition in rock masses. In this case numerical methods require a means of representing the structural fabric of the rock mass, as there typically exists a significant strength anisotropy related to the discontinuity pattern. The FLAC computer code embodies such a numerical capacity which is provided in the "Ubiquitous joint" model. This model incorporates a separate numerical (Mohr-Coulomb) formulation to capture the weaker and more deformable nature of the rock mass structure, in comparison to the intact rock material. Details of the Ubiquitous joint model are provided in section 7.5.4.

Discontinuum modelling analyses of the Wahleach rock slope, using the Universal Distinct Element Code (UDEC, marketed by Itasca Corp.) were carried out as part of this study. The objectives of this work were to investigate the effects of varying rock mass structure on the modelled rock slope behaviour, and to compare the discontinuum (UDEC) and continuum (FLAC) approaches. The UDEC and FLAC codes are based on similar numerical formulations, however, UDEC provides the capability of modelling numerous intersecting discontinuities. The key difference between these modelling approaches lies in the fact that the displacement and
stress fields in the discontinuum need not be continuous over the problem domain. The selection of which approach to employ requires an evaluation and judgment by the modeller.

Pritchard (1989) and Everard (1994) reported successful results modelling rock slopes with the UDEC code. UDEC modelling of the Wahleach rock slope yielded mixed results, based on comparison with the monitored slope movements. Experimentation with different joint orientations, relative to the slope, indicated that the most representative model response was achieved when vertical joint orientations were input to the UDEC model. Modelling this geometry revealed a toppling dominated failure mechanism, which showed agreement with deformation profiles in the lower portion of the unstable rock mass at Wahleach, but did not provide agreement with deformation profiles higher up in the slope (inclinometer S7 and S10 areas, Fig. 5-2). This observation supported the fact that deformation mechanics have been partially controlled by the dominant, steeply dipping joint sets. In contrast, poor agreement between model and observed slope movements was achieved when outwardly dipping structure was modelled, supporting the case that outwardly dipping structures are not continuous within the Wahleach rock slope. It was concluded that the highly fractured condition and the high relief of the Wahleach slope created limitations in using the UDEC discontinuum approach. Specifically, it was not possible to simulate the intricate scale of the rock mass structure, which led to the unrepresentative model responses obtained.

6.2.2 Two-Dimensional vs. Three-Dimensional Approach

The two dimensional plane strain analysis used in the Wahleach model was justified in the following manner. Plane-strain analysis is appropriate for structures with sizable dimension perpendicular to the plane of the section, and where deformation and principal stresses lie within the plane of section, such as wide slopes, long tunnels and large foundations. Plane-strain conditions apply to cases in which deformation is restricted to the plane of the cross section; out-of-plane deformation is not considered.

Structural discontinuities often form a complex three-dimensional geometry in rock masses, potentially requiring corresponding three-dimensional analysis methods. Pande et. al. (1990) presented several key arguments in support of treating some rock mechanics problems in a three-dimensional formulation. They highlighted that the orientation and geometry of the rock mass structure may be unsuitable for two-dimensional analysis, and the orientation of the
principal stresses may not be parallel to the main axis of the problem geometry (ie. tunnel axis or slope section). These arguments are probably more valid for underground applications where \textit{in situ} principal stresses are less likely to be aligned with potential excavation directions. A summary of case histories investigating in situ stresses in steep rock slopes by Ripley and Brawner (1990) concluded that sloping topography develops near surface principal stress conditions that are aligned with the slope axis. These stress conditions are considered to be suitable for two-dimensional plane strain analyses.

In the case of the Wahleach rock slope, three factors suggested that a two-dimensional analysis was appropriate:

1/ Extensive geological mapping from tunnel and surface exposures, and borehole televiewer analysis of the drill core, established that the structural discontinuities displayed general trends that were steeply dipping (greater than 45-50°) and approximately parallel and perpendicular to the slope strike.

2/ Instrumentation results indicated that slope movements were predominantly parallel to the dip direction of the slope, as discussed in section 5.

3/ Allignment of the major principal stresses were assumed to lie within the plane of the slope section. Support for such a condition was generated from the results of hydraulic jacking tests carried out during the 1989/90 investigation, and from modelling experience established by Kohlbeck et al. (1979) and Ripley and Brawner (1990). These hydraulic jacking tests concluded that the lower bound envelope of jacking stresses was approximately equal to 0.38 of the theoretical vertical stress, consistent with values obtained in similar geologic settings (ref. 3). These lower bound stresses were considered to represent minimum confining stresses acting normal to the predominant discontinuity set. Structural analyses revealed that the primary joint set was steeply dipping and striking parallel to the overall trend of the slope, thus the minimum principal stress was concluded to act perpendicular to this orientation.

Results from previous analyses of the slope, based on limit equilibrium analyses carried out shortly after the break in the tunnel lining, concluded that there was no significant difference in the results of two and three-dimensional models (ref. 2). Duncan (1996), summarizing work from numerous researchers, concluded that two dimensional limit equilibrium analyses
consistently returned equivalent or lower factors of safety than comparable three dimensional
evaluation. Although this work was based on soil slopes, side effects generated by typical non-
planar rock mass structure would suggest that this observation would pertain to rock slopes as
well. As such, a two dimensional analysis was not considered to return a relatively under-
conservative evaluation.

A further modelling concern associated with dimensionality was related to earthquake
loading conditions. For two-dimensional analysis the earthquake loading is assumed to act in
the plane of the selected section, which may not represent either the best or worst case scenario,
but avoids the complexity of evaluating the required input, and subsequent evaluation, for three
dimensional loading conditions.

6.3 Material Behaviour

The Wahleach numerical model employed an elastic-plastic stress-strain relationship
based on a Mohr-Coulomb yield strength criterion to simulate the material behaviour of the
fractured rock mass. The strength criterion is based on yield in shear, but includes a tension cut-
off at low stress levels. The following discussion examines the validity of this particular
(simplified) approach.

In this discussion, rock mass behaviour, including intact rock and discontinuities, will be
evaluated in three fundamental ways:
1/ Mechanics of Deformation of Rock Slopes
2/ Stress-strain relationships.
3/ Material strength.

6.3.1 Mechanics of Deformation of Rock Slopes

The behaviour of earth materials is strongly influenced by internal fabric characteristic
of soil and rock masses. This is certainly true of soils, as residual soils often retain relict fabric
from the protolith rock mass, and transported soils typically develop a deposition fabric.
Extensive laboratory testing has shown the strength and deformation anisotropy developed in
deposited soils (Vaid and Chern, 1985).

The effect of fabric is even more pronounced in rock (Gerrard, 1977). Two varieties of
rock mass fabric can be distinguished; 1) "small scale" fabric such as mineral grain orientation
(lineation, schistosity) and intra and inter-crystalline microfracturing; and 2) "large scale" fabric including joints, bedding, foliation, cleavage, shears and fault zones. Small scale features are considered to be most significant to intact rock behaviour. Although the mineralogy of the Wahleach rock mass is reasonably uniform, the friable conditions described in Section 3.2 constitute small scale fabric that must be considered in the modelling analysis. The large scale fabric, collectively referred to as rock mass discontinuities, develops behaviour that is distinct from intact rock.

Rock mass behaviour at a field scale is frequently dominated by the presence of structural discontinuities (Piteau, 1973). This is due to the relatively lower strength and greater deformability of discontinuities in comparison to intact rock. The contribution of discontinuities to rock mass behaviour becomes even more significant in near surface conditions, such as rock slopes, where discontinuity frequency, spacing and conditions facilitate greater effect. Goodman (1993) highlighted the general trend of increased joint frequency in the upper 60 to 100 metres of many rock masses. This condition is applicable to the Wahleach rock slope, although the affected depths extend up to 150 metres, as discussed in Section 3.2. Stress relief from overburden removal, glacial effects and slope movements, in association with gradual weathering were considered responsible for much of this increased fracture distribution.

The strength anisotropy created by rock mass structure dictates that a thorough understanding of the rock discontinuities be developed for any representative analysis. Methods of structural evaluation have been referred to as kinematic analysis. Kinematics refers to the description of motion, and addresses the geometric factors of motion without regard for the causative forces. Motion can be defined as translational, rotational, dilational and/or distortional (Fig. 6-1). Kinematic analysis of rock slopes is based on an evaluation of significant geometric features and their potential role in deformation mechanisms. With respect to in situ conditions, kinematic effects can be assessed in two perspectives: 1) Examination of rock mass structure to evaluate potential deformational mechanisms. This assumes that material failure and deformation is structurally controlled and does not occur through intact rock material; a reasonable assumption in near surface environments where stress levels generally do not exceed intact rock strength. 2) Inspection of the rock mass structure and conditions to evaluate the nature of previous deformation mechanisms.
The first approach requires sufficient structural data throughout the rock mass to enable a determination of representative/potential deformational mechanisms. The most widely used kinematic analysis methods involve the use of stereographic presentation, as discussed by Hoek and Bray (1981). Bovis (1982, 1990) used these methods to evaluate potential failure mechanisms in the Affliction Creek rock slope, which is currently experiencing a potentially similar, slow deformation process as the Wahleach slope. Results from detailed numerical modelling of the Affliction Creek rock slope using the FLAC and UDEC computer codes (Bovis and Stewart, unpublished) support the deformation mechanisms revealed by kinematic analyses. Structural data from the Wahleach rock slope (Fig. 3-1) indicated several potential deformation mechanisms, including toppling, sliding and dispersed flow in areas of highly fractured rock.

Toppling mechanisms, involving rotation and/or flexure of rock blocks are possible along steeply dipping joints and shears striking within 20 to 30° of the slope contours. Simple kinematic analysis, which represents total stress conditions without consideration of pore water pressures, indicate potential for toppling on structures dipping greater than approximately 60° into the Wahleach slope. Stereonet plots of the Wahleach structure indicated a large concentration of joints with this orientation.

Stereonet analysis of rock mass structural data indicated limited potential for sliding along down slope dipping (westerly dipping) structure. It was concluded from the detailed investigations (ref. 5) that no evidence of a well defined joint set or shear feature compatible with simple translational sliding existed. The presence of the antislope scarps did not support a sliding dominated mechanism, that more likely would develop a large headscarp in the upper regions of the slope; a feature clearly absent at Wahleach. Moreover, instrumentation data has indicated that sliding movements alone are not responsible for the current deformation process. In conclusion, kinematic analysis indicates that the deformation mechanism active in the Wahleach slope has most likely comprised a combination of toppling and dispersed shear failure in highly fractured zones to account for the diffuse deformation profile.

The second (and post-mortem) approach to kinematic analysis involves a distinction between disturbed structure in deformed regions, and initial structure in areas where deformation has been relatively minimal. In upper sections of the Wahleach slope considerable sections of outcropping rock clearly display distorted fabric as a result of rotational and translational movements. A comparison of this evidence with undisturbed fabric observed at depth in the
tunnels provided a means of evaluating the possible deformation mechanisms. The disturbed structure, further supported by the presence of the large linears, indicated toppling behaviour near the slope surface. The correlation between toppling movements and sackung features (linears) was discussed in detail in section 2.5.3.

6.3.2 Stress - Strain Relationships

The rheology of rock masses is a complex phenomenon not yet fully understood (Selby, 1993). Analysis of load-deformation behaviour in earth materials is effectively based upon simplified models that can mathematically express observational response (from lab or field testing) considered to be representative of in situ conditions. Idealized models for representing rock behaviour have been founded on simple mechanics, through the use of springs, dashpots and sliding blocks. Springs simulate instantaneous elastic response to loading below material yield limits for which a direct correlation exists between load and displacement, with displacements fully recoverable upon removal of load (Fig. 6-2a). Elastic behaviour in rocks is frequently idealized as a linear relationship, although actual laboratory testing typically displays some curvilinear response below the elastic yield point. Elastic models alone are considered to be acceptable approaches for modelling rock where stress levels are clearly below material failure. Dashpots have been used to idealize time dependent behaviour. Sliding blocks have been used as idealized models of plastic behaviour to represent shear failure conditions. For block movement (plastic deformation) to occur, frictional resistance along the block base, representing intrinsic material strength, must be exceeded. If shear stresses reach the yield strength, non-recoverable deformation occurs. Plastic deformation is further idealized as "perfect" if a plot of stress versus strain shows as a linear relationship parallel to the horizontal strain axis (Fig. 6-2b). This infers elastic conditions below the yield limit, and material flow upon reaching yield. A combination of linear elastic - perfectly plastic models, also referred to as St. Venant material, represents a spring and sliding block in series, allowing for elastic behaviour below yield limits (Fig. 6-2c).

Slope movements at Wahleach required a stress-strain relationship with plastic potential, capable of accommodating permanent slope deformation, as evidenced by the slope linears, the disturbed/dilated nature of rock mass, the lining break/power tunnel deformations and the currently monitored slope movements. Elastic constitutive relations alone could model neither
the current movements observed in the slope instruments, nor any potential plastic deformation of the slope due to future loading conditions. In addition, simulation of the energy dissipation accompanying the permanent deformations was considered a physically significant process, and was therefore important to the numerical capabilities of the chosen analysis method.

The slow progressive deformation of large rock slopes has been described by many researchers as a "creep" process (see Section 2.2). Such a description clearly implies time dependent behaviour, with "creep" referring to deformation in earth materials at stress levels below the instantaneous yield strength (Beer and Johnston, 1981). This term is frequently incorporated in descriptions of slow movement processes, however, without detailed monitoring of \textit{in situ} conditions it is difficult to ascertain whether true creep processes prevail.

Instrumentation results from Wahleach slope inclinometers and strainmeters have revealed that accelerated movements occur in response to changes in slope conditions. In particular, accelerated slope movements have occurred in response to changes in the groundwater flow regime in the upper section of the rock mass during periods of prolonged and heavy rainfall and, therefore, correspond to changing effective stress conditions. This indicates that constitutive relationships formulated strictly around a time-dependent formulation would be inappropriate for the analysis of the Wahleach slope. It was considered likely that some component of time-dependent behaviour prevailed within the overall slope deformation process, but this contribution was greatly exceeded by deformation driven by changes in effective stress levels. Conversely, it was concluded that a plasticity model capable of capturing the yield conditions in response to the changing effective stress levels was the most appropriate approach for simulating the Wahleach slope behaviour.

It should be emphasized that the Wahleach numerical model was intended to provide a reasonable approximation of the material behaviour in the slope for the loading conditions considered. Although detailed load-deformation testing of representative samples from the Wahleach slope were not available, and probably could not be realistically obtained, a linear elastic perfectly-plastic model was considered to be a reasonable representation of material behaviour in the actual slope (Fig. 6-2c). This simplified model was used to prescribe the behaviour of the overall rock mass and the well developed rock mass discontinuities. Further discussion of material behaviour with respect to modelling input, particularly with respect to post yield behaviour, strain influenced response and weathering is discussed in section 7.6.3.
6.3.3 Material Strength

At the field scale rock strength is most effectively described in terms of "rock mass strength" which incorporates the effects of both intact rock and discontinuities. The strength of intact rock and rock discontinuities have received extensive investigation in both the laboratory and field. This has formed the basis for the wide acceptance of the Coulomb equation:

$$\tau = c' + \sigma' \tan \phi'$$

defining the stress conditions at failure. Jaeger and Cook (1969) suggested that the Coulomb equation provides a reasonable estimate of failure stresses in jointed and intact rock.

The use of the Mohr circle provides a technique for expressing the stress state at a point for all conditions, including failure. The integration of the Mohr circle expression into the Coulomb equation yields the widely used Mohr-Coulomb failure criterion:

$$(\sigma'_1 - \sigma'_3) = (\sigma'_1 + \sigma'_3) \sin \phi' + 2c' \cos \phi'$$

where $\sigma'_1$ and $\sigma'_3$ represent the major and minor principal effective stresses, respectively, $c'$ represents the material "cohesion" (shear stress intercept), and $\phi'$ represents the effective material angle of internal friction. The incorporation of effective stresses is in accordance with Terzaghi's fundamental concept.

The Mohr-Coulomb failure criterion above describes a linear failure envelope. Testing of rocks often reveals slight curvature of the failure envelope, particularly if the tests cover a wide range of confining stress conditions. In practice, idealized linear approximations of $c'$ and $\phi'$ are selected for the stress range appropriate to the problem (Fig. 6-3). This approach was adopted in the modelling of the Wahleach rock slope.

Estimates of the two input parameters in the Mohr-Coulomb failure criterion, the cohesion, $c'$, and friction angle, $\phi'$, of the material, can be established from tests results, or alternatively, from empirical methods. Hoek and Brown (1980) proposed an empirically based determination of rock mass strength that has been used widely in practice. The original Hoek-Brown criterion has been modified as recently as 1996 (Hoek, 1996) to accommodate a wider range of rock conditions. The most recent version of the Hoek-Brown rock mass strength evaluation method has been incorporated into the Wahleach rock slope analysis to provide estimates of rock mass friction, cohesion and deformability. Details of the Hoek-Brown rock mass strength determination are provided in Appendix A.

The strength of the prominent joint set(s) in the slope have been evaluated independently.
from the rock mass strength. This has been done in order to capture the significant deformation component potentially occurring along the dominant structure. Estimates of rock joint strength are based on an empirically derived method proposed by Barton and Choubey (1977). Details of the rock joint strength estimate and characterization are provided in Appendix B.

6.4 Available Numerical Methods

This section describes three viable methods of numerical slope analysis; limit equilibrium, implicit (finite element) and explicit (finite difference) formulations. The objective of this discussion is to investigate the relative advantages and disadvantages of each method with respect to modelling the Wahleach rock slope.

6.4.1 Limit Equilibrium Techniques

Limit equilibrium methods of analysis are widely practiced in rock and soil mechanics problems. Limit equilibrium methods of analysis are based on the laws of mechanics, satisfying certain conditions of equilibrium, involving combinations of moment and/or horizontal and vertical force equilibrium according to the particular approach; however, material compatibility is not considered. The basic principle involves the following simplifying assumptions:

1/ Failure occurs along a discrete surface.
2/ Material behaviour is simulated as a rigid block above the failure surface.
3/ Plastic material behaviour is assumed along the failure surface, without reliance on stress-strain relationships.
4/ Material response at failure is restricted to slip (shear failure) along the discrete failure plane/surface.
5/ A limiting equilibrium condition implies total slope collapse (as opposed to some finite displacement).

Equilibrium methods are evaluated on a comparative ratio of material strength (ie. stabilizing forces) to driving forces (destabilizing forces), typically presented as a "Factor of Safety". Factor of safety values of unity represent the "limiting" condition indicating failure.

Certain conditions prevailing within the Wahleach slope are incompatible with the above assumptions, primarily the following observations:
1/ Deformation patterns reflect a complex behaviour involving both shear and toppling mechanisms, rather than sliding along a single, well defined continuous shear failure surface.

2/ Strain is distributed throughout the deforming mass, therefore analysis of the unstable section of the slope as a solid block is highly unrealistic.

3/ The assumption that the entire deforming section of the slope reaches a critical condition simultaneously is unlikely, based on recognition of differential movement rates and variation in observed rock mass conditions.

4/ Large slope deformations have progressively occurred without total slope failure.

5/ Limit equilibrium methods that use a pseudo-static approach for the analysis of earthquake loading, through the application of constant loads in the downslope direction, present an overly conservative approach by simplifying the time varying and directional varying forces that are characteristic of seismic shaking.

6/ Finally, limit equilibrium formulations do not include material stress-strain relationships, and therefore, cannot model the post-failure material behaviour. As this was identified as a key modelling objective, limit equilibrium methods were considered inappropriate for detailed analysis of the Wahleach rock slope.

6.4.2 Implicit Methods

Finite element methods, which are frequently based on an implicit formulation, are the most widely used technique for solving deformation problems in geotechnical engineering (Duncan, 1996). Finite element methods are suitable for both rock and soil under either static or dynamic loading conditions, and are versatile enough to solve both mechanics (stress, strain) and flow (groundwater, heat) problems. The finite element method was first used for geotechnical applications in the late 1960's by Clough and Woodward (1967), but originated in structural engineering analysis.

Finite element methods are more rigorous than the limit equilibrium methods discussed in section 6.4.2, as they not only satisfy equilibrium, but also consider material compatibility, and are capable of incorporating a full range of material stress-strain relationships, including nonlinear behaviour. Equilibrium can be satisfied in either a static or dynamic state. For static equilibrium the following condition applies: $\Sigma F = 0$
Dynamic equilibrium requires the use of Newton's Second Law: $\Sigma F = ma$

Either case involves solution of the following differential equation to derive an approximation for model displacements:

$$\Sigma F = [M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\} = \{R(t)\}$$

$u$ represents displacement vectors, with $\ddot{u}$ and $\dot{u}$ the respective first and second derivatives with respect to time. $[M]$, $[C]$ and $[K]$ represent the mass, damping and stiffness matrices, respectively, for the modelled elements, and $\{R(t)\}$ includes any time dependent loading conditions. Solution details are provided by Desai and Christian (1977).

Duncan and Goodman (1968) outlined details of the method and techniques for setting up analyses. The basic construction of a finite element analysis involves the discretization of the particular problem geometry into a finite number of deformable elements. These elements are connected at a finite number of nodal points at which approximate solutions for force and displacement are obtained. Governing equations are employed to describe the relationship between force and displacement forming a "stiffness" matrix for each element, which are integrated to form a global stiffness matrix for all elements. Solution of the simultaneous equations forming this matrix yield the nodal point displacements and associated element stresses.

Finite element methods have been more commonly applied to continuum problems, but also have been successfully applied to jointed rock masses by modelling the structural discontinuities as infinitely thin elements with different properties and potentially different stress-strain relationships than the intact rock material (Duncan and Goodman 1968; Kalkani and Piteau, 1976). This approach of modelling rock mass structure in an otherwise continuum approach is generally limited by two factors:

1/ A restricted number of discontinuities can be realistically modelled.

2/ The numerical calculation is often restricted to small deformation along the discontinuities (Itasca Consulting Group Inc., 1992). The reason for this is that the stiffness matrices, inherent to the solution, must be reformed following any sizable displacement along the modelled discontinuity surfaces.

These limitations in modelling large, structurally controlled deformations discouraged the use of finite element methods for the deformation analysis of the Wahleach rock slope.
6.4.3 Explicit Methods

In this discussion explicit numerical methods will be considered synonymous with finite difference methods. These general methods are based on solutions of sets of differential equations. The governing equations are used to represent the pertinent physics of a simulated problem, whether mechanical and/or flow. The finite difference methods are formulated on similar physics as used in the finite element method. The laws of mechanics, expressed as static or dynamic equilibrium, are respected, with material compatibility and suitable boundary conditions included. Solution methods are based on replacing the governing differential equations with algebraic expressions at discrete points within the modelled domain. Explicit methods can be adapted to both continuum and discontinuum approaches.

Explicit methods rely on a sequential formulation that incrementally approach a static or dynamic solution state from given initial and boundary conditions. The explicit terminology implies a precisely defined formulation at discrete points in a modelled domain, that can be solved at each stage of the incremental solution process. Like the implicit formulation described above, explicit methods are based on differential equations that provide an approximate solution for stresses and/or strains throughout a specified problem domain. The key advantage of using the incremental (time-stepping or time-marching) approach inherent to the explicit method is a simple formulation that does not require the construction and solution of sets of simultaneous equations at each time step. Significantly, the explicit formulation approach is well suited to the analysis of large strain, unstable, and nonlinear problems (Coetzee et al., 1993). This results from the fact that the governing equations can be easily reformulated and material properties (i.e. deformation moduli) updated with the development of model strain.

Another strength of explicit methods, shared equally by implicit methods, but not limit equilibrium methods, is the capability to simulate changing loading conditions and material properties. The use of an analysis method capable of capturing the changing conditions in the Wahleach slope was considered essential.

It was important to select a numerical method that provided the most representative dynamic modelling capabilities. For the modelling of the Wahleach slope, where complex failure mechanisms prevail, a physically representative method for applying and evaluating the effects of dynamic loading was considered essential. Principally, this involved the capability of modelling the effects of earthquake loading on the Wahleach slope. Widely used pseudo-static...
methods associated with limit equilibrium analysis methods were considered inappropriate, partly for the reasons discussed in section 6.4.1, but also because the slope has shown the capability to withstand significant deformation without total or rapid collapse. Alternatively, implicit and explicit methods provide a more suitable technique for modelling the dynamic loading conditions associated with earthquakes.

The capability of finite difference methods to model both the non-linear and large strain conditions inherent to the Wahleach slope made it the most appropriate numerical modelling method for the analytical work.

6.4.4 Overview

An assimilation of the arguments and discussion in Section 6.4 lead to the selection of an explicit finite difference numerical method to model the natural mass movement processes in the Wahleach rock slope. The FLAC computer code (Itasca Consulting Group, Inc. 1993) was selected from the available numerical methods for the following reasons:

1/ The ability to model the large strain conditions.
2/ The ability to simulate changing loading conditions and material properties.
3/ The capacity to simulate dynamic loading conditions such as earthquakes and transient groundwater flow.
4/ And most importantly, the representative manner in which the material behaviour and overall physics are treated in the numerical formulation.

Details of the FLAC numerical formulation are introduced in section 6.5.2.

6.5 FLAC Computer Code

This section provides an introduction to the FLAC computer code. The important aspects and capabilities of the code are discussed, but a full presentation of the process, including details of the numerical formulation are beyond the scope of this project. Details can be found in the FLAC User Manuals (ref. 59a).

6.5.1 General

The preceding section outlined the important features in the selection of a representative modelling method for the analysis of large rock slope deformation. The FLAC computer code
was selected as the most appropriate numerical modelling tool for the active slope processes at the Wahleach hydroelectric project.

The FLAC acronym stands for Fast Lagrangian Analysis of Continua. The FLAC computer code was developed in 1986 by Dr. Peter Cundall, and is commercially marketed by the Itasca Consulting Group, Inc., of Minneapolis, Minnesota. The program has evolved through a series of upgrades to the current edition, FLAC Version 3.30, released in September 1995. Initial modelling of the Wahleach rock slope was undertaken with FLAC version 3.22, but all subsequent efforts reported in this presentation have been carried out with the updated Version 3.30. The numerical analyses were run on a 66 Megahertz, 486, IBM compatible personal computer with 8 Megabytes Ram storage. Typical run times for the detailed model required approximately 1 to 2 seconds per model timestep, depending on the material state of the model. The basic static analysis required approximately 100,000 to 120,000 solution steps to attain an equilibrium state, and required approximately 36 hours to run. Dynamic and predictive static analyses required 300,000 to 400,000 solution steps corresponding to approximately four and a half days solution time.

6.5.2 Details of Numerical Formulation

FLAC employs an explicit finite difference formulation to solve two-dimensional, mechanics and flow (fluid, thermal) problems. The following discussion, as it applies to the project work, will be restricted to the numerical formulation of mechanical problems in plane-strain.

The finite difference approach used in FLAC involves substituting the governing differential equations with algebraic expressions, which furnish estimates of stress and displacement at distinct positions within the modelled domain. The FLAC numerical formulation is centred around Newton's second law of motion, force equals mass times acceleration. The significance of incorporating the dynamic equation(s) of motion allows for the treatment of physical instability and associated effects such as energy dissipation during material flow, while maintaining numerical stability in the mathematical formulation process (Coetzee et al., 1993). The ability to model instability and resultant material flow makes FLAC well suited for simulating the slope movements observed at Wahleach.

The numerical method follows the time-stepping sequence described below, in which
updated model values are based on prescribed initial conditions. The calculation sequence is described as "explicit" in that each component within the cycle is clearly defined by the finite difference expression of the field equations, as follows:

1/ Model velocities are derived from known stresses and forces based on Newton's second law. Velocities are evaluated at model gridpoints by summing the force contribution from adjoining zones. The zones within the discretized grid are subdivided into pairs of overlaid triangles, with constant strain conditions characterizing each triangular sub-element. A calculation scheme using these overlapping triangular elements avoids geometric difficulties encountered with quadrilateral elements, in addition to the problem of overconstrained elements. Marti and Cundall (1982) described this method as "mixed discretization". The solution of velocities from initial stresses is based on the following equations:

i/ \[ F = m \cdot a \]

where \( F \) = sum of acting forces = \( \sigma \cdot A + m \cdot g \)

\( \sigma \) = stress = \( F/A \)

\( A \) = area (based on zone length and unit width)

\( m \) = mass; \( g \) = gravitational acceleration

\( a \) = acceleration = \( \frac{df}{dt} \), \( \frac{d\mathbf{u}}{dt} = velocity \); \( t \) = time

\( \rho \) = mass density = \( m/V \); \( V \) = volume

ii/ \[ \frac{(\sigma A)}{V} + (m \cdot g)/V = (m \frac{d\mathbf{u}}{dt})/V \]

iii/ \[ \sigma/dx + \rho g = \rho \frac{d\mathbf{u}}{dt} \]

2/ Strain rates are derived from velocities. Velocity gradients, \( \frac{d\mathbf{u}}{dx} \), at the model gridpoints are averaged to yield the strain rate:

\[ \dot{\varepsilon}_{ij} = \frac{1}{2}\{\frac{d\mathbf{u}}{dx_j} + \frac{d\mathbf{u}}{dx_i}\} \]

3/ The material constitutive laws are then consulted to calculate zone stresses from strain rates. The calculated zone stresses are then evaluated with respect to the yield criteria. If zone stresses exceed levels stipulated by the yield function, then plastic deformation occurs, and stresses are corrected to maintain levels compatible with the specified yield function (FLAC manual, version 3.3, p. D-11).
This sequence completes a FLAC timestep during which the calculated variables are held fixed. As subsequent timesteps are executed the stress and force variation is progressively addressed until a static solution is achieved where the force imbalance at model gridpoints approaches an acceptably low value. To physically justify a timestep with fixed conditions, a suitably small timestep must be selected, such that the calculation speed exceeds the speed of propagation of any disturbing effect. Details for calculating this value are provided in the FLAC manual.

The calculation cycle follows a Lagrangian method that allows the modelling of large strain conditions accompanying material yield and flow. In this mode the coordinates of the gridpoint are updated as strains occur, enabling the grid to move with the deforming material. To calculate the model displacements the finite difference form of Newton's second law is incorporated, enabling displacements to be evaluated from model velocities:

i/ Model velocities are updated in the following fashion:

\[
m a = \sum F
\]
\[
m (\Delta v/\Delta t) = \sum F
\]
\[
\Delta v = (1/m) \sum F \Delta t
\]
\[
\theta_i(t+\Delta t/2) = \theta_i(t-\Delta t/2)
\]
\[
\theta_i(t+\Delta t/2) = \theta_i(t-\Delta t/2) + (1/m) \sum F \Delta t
\]

ii/ Model velocities then used to update the model coordinates:

\[
x_i(t+\Delta t) = x_i(t) + \theta_i(t+\Delta t/2) \Delta t
\]

A full discussion of the numerical formulation, with calculation details is provided in the FLAC Manual, section 3.0.

6.5.3 Verification of FLAC Code

Verification of the FLAC code, through comparison with closed form solution evaluations has yielded successful results. A presentation of the verification details are beyond the scope of this project, but are well documented in the User Manual accompanying the FLAC code. An example slope stability problem was outlined in the FLAC manual (Vol. IV, Section 10-1) providing a comparison of a simplified FLAC analysis with a more conventional limit equilibrium analysis.
6.5.4 Material Behaviour

A range of material constitutive relationships are available in the FLAC code for selection by the user. For the modelling of the Wahleach slope, elastic and elastic-perfectly plastic (also described as elasto-plastic) models were used, although the significant aspects of the slope behaviour were modelled with the latter. The elastic model was used in initial trial runs to ensure the model set up was acceptable, and to develop an understanding of the stress distribution in the slope. This was done to provide a more representative input for the determination of stress level dependent material properties.

The FLAC plasticity models rely on two mathematical expressions, a material strength expression (yield function), and a flow rule, to provide a simulation of the material behaviour. Following the discussion in section 6.3.3, the material strength is described by the linear Mohr-Coulomb failure envelope, which defines the states of stress corresponding to onset of material shear and/or tensile failure. The shear flow rule is non-associated in the FLAC Mohr Coulomb plasticity model, indicating that the potential and yield surfaces are not coincidental, whereas the tensile flow rule is associated, with coincidental potential and yield surfaces. The flow rules are incorporated to specify the direction of the plastic strain increment, and to maintain stresses on the yield surface if the calculated stresses violate the strength criteria.

To accommodate the strength anisotropy related to the rock mass structure, an additional Mohr-Coulomb yield criterion and flow rules can be incorporated for the prominent rock mass structure. These separate yield functions are part of the ubiquitous joint model available in the FLAC code, which was used to model the prominent joint structure in the Wahleach slope. This model provides for material shear and tensile failure along the defined structure. The ubiquitous joint formulation is secondary to the general Mohr Coulomb formulation, as it is consulted only after stresses have been evaluated, and potentially corrected, in the general formulation.

Important factors in the formulation include the following:

1/ The failure criterion is evaluated in terms of effective stresses.

2/ The shear flow rules are non-associated. Non-associated shear flow rules are consistent with observations from lab testing of soils and rock, whereas associated flow rules lead to excessively large dilational behaviour (Brown, 1987).

Details of the mathematical expressions can be found in Appendix D of the FLAC Version 3.3 user manuals.
As a test of the Ubiquitous joint model formulation, a single element model was analyzed to evaluate the response to simple shear loading conditions. This was done to illustrate the fundamental stress-strain behaviour of the model material. Material in the single zone was represented by the ubiquitous joint plasticity model. This incorporated a potential plane of yield in addition to the yield of the general material according to the Mohr-Coulomb failure criterion. The ubiquitous joint formulation was utilized to represent the natural strength anisotropy of the geologic structure. The ubiquitous joint plane was oriented vertically (similar to the predominant rock mass fabric in the Wahleach slope) and the model was loaded in simple shear until yield and flow occurred (Fig. 6-4a). Shear yield occurred along the ubiquitous joint and not through the stronger "rock mass" material. The response of the material illustrated a linear elastic, perfectly-plastic response (Fig. 6-4b). This conformed to the expected response of the material based on the numerical formulation, and provided a simplistic, but representative simulation of the structural behaviour.

6.6 Dynamic Modelling

The FLAC code includes the capability to analyze time varying forces based on Newtonian mechanics. This dynamic capability was used to apply seismic loading conditions to the Wahleach rock slope model in order to evaluate the susceptibility of the slope to earthquake loading. The simulation of earthquake loading can be dealt with in several ways, including application of appropriately selected velocity or acceleration time histories; or alternatively, through cyclic force application. In this analysis the earthquake energy was simulated through the application of an appropriately selected acceleration time history, as discussed in section 7.7.1. Earthquake loading of the Wahleach slope was modelled by applying time varying acceleration to the model base resulting in the upward propagation of shear waves through the medium, a technique commonly employed in dynamic modelling studies.

6.6.1 Dynamic Formulation

The formulation used in the FLAC code for dynamic analysis follows a similar approach to that outlined in the static analysis formulation (section 6.5.2), by solving the full dynamic equations of motion. One notable exception differentiating the dynamic formulation involves the treatment of the model mass. A lumped mass approach is used to represent the model
medium, based on the specified material density. The lumped mass approach involves assigning one third of the triangular subzone mass to each of the three gridpoint corners, then dividing the gridpoint masses by two to account for the overlapping subzones utilized in the mixed discretization scheme.

Another essential feature of the dynamic formulation that differs from the quasi-static formulation involves the treatment of material damping. Damping refers to the dissipation of mechanical energy and is the material property that prevents unlimited oscillation in a disturbed system. Damping is attributed to energy dissipation, possibly generated from internal friction effects during solid body deformation, and through frictional resistance between moving solid bodies (i.e. soil particles or rock on opposing sides of a joint/shear zone), with the ultimate transfer of kinetic energy into thermal energy. The cyclic loading of earth materials characteristically displays hysteretic behaviour when viewed with respect to stress-strain behaviour (Byrne, 1996). Many constitutive models do not simulate this energy dissipation adequately, and so require some additional energy dissipation to account for the damping effects. The Mohr-Coulomb and Ubiquitous joint plasticity models provided in the FLAC code fall within this group of models that require additional damping consideration for dynamic loading analyses.

A form of frequency dependent, viscous damping, referred to as Rayleigh damping, is provided in the FLAC code that is considered to be essentially frequency independent for a range of specified frequencies. This frequency range is related to the natural vibrational frequency of the model material, and the predominant frequency range carrying most of the input energy. The Rayleigh damping in FLAC requires as input the fraction of critical damping, and the centre of the frequency range over which the damping is essentially frequency independent.

Rayleigh damping was used in the dynamic loading of the Wahleach rock slope. The range of important frequencies was determined from both the input earthquake record and the undamped response of the slope following excitation. The earthquake input was specified in terms of an acceleration time history, for which the predominant frequency range carrying the significant energy was determined from a detailed analysis of the time history. Based on the input time history response spectrum (Fig. 6-5), the highest acceleration levels occurred in the period range of 0.3 to 0.75 seconds. This corresponded to a frequency range of 1.33 to 3.33 Hz. A Fast Fourier Transform (FFT) evaluation of the input time history, which was run within a
separate FLAC "FISH" routine, indicated that the frequency range containing most of the input energy was approximately 1.5 to 3.0 Hz, agreeing with period range noted above (Fig. 6-6). The undamped response of the numerical model was observed following excitation with the input time history to evaluate the value of the fundamental frequency of the model slope. This exercise was run with an elastic model, for which frequency of shaking was found to be in the range of 1.25 to 1.5 Hz. Combining the above observations resulted in a value of 1.5 Hz used for the for the central frequency range specified for the Rayleigh damping to provide near frequency independent damping. The level of critical damping has been considered to be in the order of 2-10 percent for earth materials. A range of 2 to 5 % was considered appropriate for the rock mass conditions in the Wahleach slope (based on personal communication with Dr. Peter Cundall). For the final analysis a damping value of 5% was used.

6.6.2 Dynamic Boundary Conditions

The boundaries of a numerical model are a non-physical condition required to provide finite extent to the modelled domain. Surface modelling applications, such as the Wahleach slope, are essentially a semi-infinite half space. Therefore some method must be included in the numerical formulation to represent the continuity of material beyond the model boundaries. For cases involving dynamic loading conditions these boundaries can be problematic due to the reflection of wave energy back into the model that would otherwise propagate away from the area of interest. The FLAC code includes a method, referred to as "free field" boundary conditions, for representing non-reflecting conditions along the imposed boundaries.

In FLAC free field boundaries involve the placement of a single column of finite difference zones attached in parallel to the lateral boundaries of the main model by viscous dashpots. During dynamic loading the dashpots allow the single zone column of free field material to move in coordination with the main finite difference grid under the influence of primary, non-reflected wave energy. However, when the upwardly propagating waves reflect back from the free (slope) surface, a real physical phenomenon, the main and free field grids respond differently, causing the viscous dashpots to absorb the reflected wave energy. This prevents the reflection of energy back into the main grid from the non-real lateral boundaries. The advantage of using the free field boundaries, in contrast to "quiet" or viscous boundaries schemes alone, is that propagating wave energy is not distorted along the boundaries. The
FLAC model provides the capacity to view and evaluate the response of the free field zones, thereby ensuring that the appropriate non-reflective conditions, representing an infinite material, are respected.
Figure 6-1: The basic types of movement of materials: (A) Dilation, involving a change in volume; (B) Translation, involving a change in position; (C) Rotation, involving a change in orientation; and (D) Distortion, involving a change in shape. Adapted from Davis (1984).
Rheological models.

Perfect plastic solid

(a) Elastic (Hookean solid) behaviour, representing material such as intact rock.

(b) Perfectly plastic solid behaviour, representing material such as soft, undrained clays.

(c) Elasto-plastic solid behaviour, representing a range of soil and rock conditions.

(d) Brittle behaviour, without the post failure response, which typically illustrates weaker conditions.

Figure 6-2: Simplified models for mathematical simulation of material behaviour. Adapted from Selby (1993).
Figure 6-3: Strength envelope for the Wahleach rock mass based on the Hoek-Brown criterion. Note the curvature in the failure envelope, indicating the dependence on confining stress levels. Linearized estimates of rock mass "cohesion" and frictional strength can be selected for chosen ranges of confining stress. Details of the Hoek-Brown criterion are included in Appendix A.
Job Title: Ubiquitous Joint Model - Material response to shear loading
From File: ut-2.dat

FLAC 3.22

Step 5000
Exaggerated Grid Distortion
Magnification = 1.905E+02
Max Disp = 5.000E-04
Displacement vectors
Max Vector = 5.000E-04
Plasticity Indicator
\( \text{\textcircled{\text{A}}} \) slip along ubiq. joints.

Figure 6-4a: Simple shear loading response of single zone model (ubiquitous joint model). Model was loaded until yield and flow occurred; refer to Fig. 6-4b for stress-strain response.
Job Title: Ubiquitous Joint Model - Material response to shear loading
From File: ut-2.dat

FLAC 3.22

Step 1000

HISTORY PLOT
Y-axis:
a_tau (FISH)
X-axis:
a_str (FISH)

Figure 6-4b: Stress-strain response of single zone ubiquitous joint model, plotted as shear stress, $\tau$, versus shear strain, $\gamma$. 
Figure 6-5: Plot of acceleration response spectrum for the Northridge (1994) event, as evaluated by the FLAC analysis.
Figure 6-6: Fast Fourier Transform plot of Northridge (1994) event, plotted as energy versus frequency. This illustrates that the significant energy of the earthquake was centred around 2 Hz.
7.1 Introduction

This chapter presents the details of the Wahleach FLAC model, including a description of the modelling methodology, construction of the model layout and treatment of the chosen loading conditions. A discussion of the key geologic and hydrogeologic factors directly related to the modelling input is integrated with this section. A more comprehensive discussion of the geologic and hydrogeologic slope conditions can be found in Chapters 3 and 4, respectively. Details of the FLAC input files for the Wahleach model can be found in Appendix E.

7.2 General Modelling Methodology

The modelling procedure outlined in section 1.4 was designed to recreate the most representative *in situ* slope conditions, while developing the simplest model consistent with observed conditions.

The basic design of the Wahleach FLAC model involved starting with very simple tests of elemental behaviour and progressing towards a level of complexity consistent with the knowledge of site conditions. Progressive modelling in this fashion was supported by the confidence developed at each stage. Model evaluation in these initial stages was based on the expected physical response of rock material established from observed field or laboratory behaviour.

Modelling philosophy for the investigation of geomechanics problems requires a different approach than conventional techniques employed in other fields of engineering. Starfield and Cundall (1988) presented a series of conceptual arguments aimed at developing a suitable approach for numerical modelling in rock and soil mechanics. This approach underscored the "data-limited" condition in geologic systems. A limited knowledge of spatial and/or temporal data is one of two significant factors clearly differentiating rock mechanics problems from those in structural or electrical engineering; the second relates to the complex and variable nature of geologic materials. Rock and soil behave much differently than the predictable elastic behaviour of steel or concrete. Non-linear, inelastic behaviour is characteristic of earth materials, as is the heterogeneity and anisotropy nature of most geologic deposits.

Despite the complexity inherent to most geologic environments, Starfield and Cundall
(1988) encouraged the development and use of simplistic models. Their argument was directed towards the use of numerical modelling in a conceptual manner for the investigation of processes and mechanisms, rather than with the intent of determining precise estimates of stresses and displacements. Precise modelling was deemed inappropriate in light of the uncertainty and wide range of material properties and behaviour potentially existing within a large rock slope. A modelling methodology consistent with Starfield and Cundall's suggestions was adopted for the numerical analysis of the Wahleach slope. Although the Wahleach FLAC model was calibrated against the recorded slope movement data, the objective was directed towards developing a more comprehensive understanding of the material behaviour, rather than an attempt to duplicate precise quantities of the slope movements.

A modelling strategy was developed for the Wahleach slope that systematically addressed the identified objectives. This modelling strategy was separated into two general phases, each comprising a sequence of linked modelling operations. Progression of the overall modelling sequence was based on the success of each preceding sequence. The two general modelling phases were:

1/ **Calibration Phase**: This model sequence involved formation of the current slope conditions, based on the following loading conditions:

   The "calibration" modelling sequence comprised a six step process, outlined as follows:
   
i/ Model construction and development of initial stress distribution in block model.
   ii/ Slope formation through removal of overburden layers in block model.
   iii/ Introduction of rock mass weathering profile.
   iv/ Modelling of groundwater effects, including transient flow in the upper rock mass.
   v/ Simulation of the lining break event (January 1989).

   Integration of the known geologic conditions and slope movement behaviour was essential in building and evaluating the steps of the calibrative phase.

2/ **Predictive Phase**: The predictive work involved subjecting the calibrated model to forecasted loading conditions, such as seismic shaking, extreme transient groundwater conditions and long term slope development/evolution. The objective was to establish a feel for the sensitivity of the slope to these forecasted loading conditions, and to develop a better understanding of the hazard of the Wahleach slope to the variety of engineering
structures at the toe of the slope. This predictive modelling was initiated only when the model calibration phase yielded a satisfactory response.

7.3 Model Calibration and Evaluation

Model calibration represents a "measuring stick" for comparison of model response with known conditions or behaviour. The importance of calibrating model response to observed behaviour in the analysis of geological systems is paramount, for without it any response is equally possible and valid. Calibration does not necessarily imply absolute model accuracy, and can be used to determine if the model response is physically representative, thereby allowing the modelling work to progress into predictive scenarios with some credibility. The wide variability inherent in most geologic systems, and the typically limited spatial and temporal data further underscore the importance of utilizing known conditions to evaluate model response prior to undertaking any predictive work.

The extensive instrumentation data from the Wahleach slope can be well matched to numerical methods that provide the ability to continuously record model response. The time stepping formulation of the FLAC code includes this capability through the recording of model response (stress, velocity, strain) at chosen model locations, allowing direct comparison to instrumentation data.

The Wahleach slope represents one of the most extensively instrumented large rock slopes anywhere, certainly outside of open pit mining slopes, which tend to have more rapid deformation behaviour. The detail and quality of slope movement data from both surface and underground render the Wahleach slope an excellent candidate for numerical modelling studies, since model input and calibration can be relatively well defined. Additionally, conditions observed in the Wahleach slope provide three different time scales upon which model comparison can be based. Long term model behaviour can be evaluated in relation to the large scale deformation features observed across the slope such as the linears and "hillside loosening", presuming these features reflect slope movement processes. Short term model behaviour can be compared against the instrumentation data covering the period after the 1989 lining break. Model behaviour between these extremes can be compared to the results of the steel lining offset survey which represents over 40 years of cumulative slope deformation. The combination of these three calibration scales constitute an unparalleled case upon which to investigate this slow
mass movement phenomena.

For the calibrative work the model was advanced to a steady solution state, representing either equilibrium or steady state flow, following any change in conditions. A steady solution state corresponded to the condition at which force imbalance throughout the model was dissipated to an insignificant level. Two widely different scenarios may have developed during the solution of unbalanced forces; i) the solution of model force imbalance and dissipation of kinetic energy, representing the attainment of static equilibrium conditions, or; ii) solution of model force imbalance without dissipation of kinetic energy, indicating a state of steady material flow under failure conditions.

Evaluation of the Wahleach model was an essential component of both the calibrative and predictive phases. Five general techniques were used to evaluate the model response, described as follows:

1/ The direct relationship of velocity to kinetic energy provided the most obvious means of evaluating the model state. Gridpoint velocity histories along the slope surface, boundaries and interior were monitored to evaluate the range of potentially different conditions within the model. Plots of the model velocities (velocity versus time-step) revealed the condition of the model; with a pattern of convergence to near zero values indicating static equilibrium, and convergence to non zero values revealing steady state material flow. An alternative method of evaluating model conditions involved viewing plots of gridpoint velocities over the whole domain. A random pattern of velocity vectors of low magnitude (ie. less than 1.0x10^-6 metres/timestep) revealed a static equilibrium state; whereas a defined pattern of velocity vectors with significant magnitude indicated on-going deformation. An assessment of "significant" magnitude was based on displacement values and the number of time-steps required to achieve such a value. To determine if the deformation represented elastic response to loading conditions or plastic flow the plasticity indicator was consulted.

2/ Similar assessment of gridpoint displacement histories were used as indicators for the model response to changes in conditions.

3/ Histories of zone stresses were recorded to monitor the progress of the model to the sequential loading conditions, and to detect conditions of plastic flow and corresponding load distribution. Apparently similar changes in stress levels can represent significantly
different behaviour, so it was important to evaluate stresses in conjunction with other indicators. For example, similar stress level reductions were noted during the slope formation, which highlighted an elastic response to the removal of overburden load; and during the reduction in material strength associated with rock mass weathering, that represented a plastic response (Fig. 7-1).

4/ Continuous recording of the maximum model force imbalance was utilized to assess the reaction of the model to changes in either loading conditions or material properties. The maximum unbalanced force represents the algebraic sum of forces acting at a model gridpoint due to stresses in surrounding zones, and any additional applied loads. When the numerical response to any change has numerically progressed (time-stepped) through the model and steady state conditions are reestablished, the unbalanced force converges to a low value. A "low" value is measured in relation to gridpoint forces derived from model zone stresses in the region of interest (the region of slope movements in this case). A plot of the maximum unbalanced force in the model was found to be a sensitive indicator of model behaviour. For example, the response of the model to a change in material strength in the upper section of the rock mass was accompanied by yield and plastic flow. As stress levels were redistributed and progressed to steady state values, the unbalanced force diminished in unison (Fig. 7-2).

5/ Plots of model plasticity state were used to assess when and where material yield was occurring and what specific type of yield had occurred (ie. slip along ubiquitous joints or shear yield in the rock mass).

An integrated approach utilizing all five techniques above was consistently used to evaluate the model behaviour. The information obtained from combinations of the five methods yielded considerable more information regarding model response than any one alone. It should be noted that precise equilibrium can be approached but never completely attained in this time-stepping numerical formulation, particularly for a large grid layout with irregular (ie. non-equilateral) zone geometries. However, the attainment of an acceptably close approximation was used as a reasonable solution, referred to as a "quasi-static" condition. It was important to follow the model to this quasi-static state following any alteration of loading conditions or material properties during the static analysis, otherwise, the response of the system could not be distinguished from subsequent disturbances to the slope model.
7.4 Construction of FLAC Model

The construction of a functional numerical model requires detailed planning in order to arrive at acceptable and meaningful results. In the case of the finite difference grid used in FLAC, several conditions must be satisfied in establishing the model geometry. The discretized model grid must resemble the prototype geometry to a level of accuracy compatible with the modelling objectives, and must be numerically capable of addressing the anticipated loading conditions. Additionally, representative boundary conditions must be established along the model sides to simulate the physical continuity of material and represent stress levels in the prototype.

Discretization of the FLAC model grid involves dividing the model domain into adjoining polygons. The polygons are referred to as zones; the corners of which are called gridpoints. Zone interiors represent constant stress and strain conditions, with single estimates of stress and strain determined for each zone, and corresponding model displacements evaluated at each zone gridpoint. Consequently, a larger number of zones will yield a more uniform and accurate distribution of stress and strain. The cost of higher accuracy is reflected in longer calculation times and increasing computer memory. To approach a compromise providing acceptable accuracy and calculation speed, a number of general guidelines for grid construction were followed for the Wahleach FLAC model:

1/ To focus the modelling attention towards the active slope movement process the finest (smallest) zone sizes were established within the zone of observed deformation.

2/ Throughout the upper 80 metres of the rock slope, zone dimensions were 10 metres high by 15 metres wide.

3/ Zones sizes were progressively coarsened below this depth, and also in the stable rock mass in the upslope and downslope directions.

4/ Increase in zone dimension did not exceed a 5% gradient.

5/ The largest zone dimension was 65 metres, occurring along the base and sides of the model.

Based on the numerical formulation process, in which model zones are subdivided into two sets of overlapping triangles, the most accurate solutions are obtained when aspect ratios are maintained as close to unity as possible. Aspect ratio refers to the relation of zone width to length. In the Wahleach model aspect ratios did not exceed 1.9.
The nature of model loading conditions plays a significant role in determining grid dimensions. If dynamic loading conditions, such as earthquakes, blasting or other vibrational input, are planned, then the maximum zone dimension will be dictated by the material wave speed and the input energy frequency. Studies by Kuhlemeyer and Lysmer (1973) have shown that for representative modelling of wave energy through a numerical model, zone size must not exceed 10 to 12 percent of the predominant input energy wavelength. The wavelength corresponding to the highest frequency component of the input wave can be estimated from:

\[ \lambda = \frac{v}{f} \]

where \( v \) is the material wave speed, generally the shear wave velocity, and \( f \) is the frequency of the input wave containing the predominant energy. Preliminary versions of the Wahleach grid were constructed that violated the limiting zone dimension, yielding questionable results. Problems with these early grids were noted by the appearance of high frequency oscillations imprinted upon the displacement and velocity responses of the model slope, a condition advised against in the FLAC manual (Volume III, Appendix K, p. K-25).

In the course of the Wahleach numerical modelling analyses five different grid geometries were experimented with. The final grid geometry developed complied with wave transmission limitations, provided suitable accuracy in the region of interest, and addressed the geometric recommendations described above. The first generation grid, was too coarse in the area of interest (25 m high by 50 m wide), and relatively, too fine in distal regions (Fig. 7-3a). The second grid provided finer zone dimensions in the zone of interest (20 m high by 25 m wide), and incorporated two subhorizontal interfaces to separate regions of different zone dimension. The major problem with this grid geometry was the large zone size in the base of the model (Fig. 7-3b) which exceeded the wave transmission capacity of the material. A third geometry experimented with a sloping model base (Fig. 7-3c). The sloping base provided a smooth transition from fine (10 m high by 15 m wide) zones in the deforming region to coarser zones along the model boundaries, with the advantage of having the smallest number of overall zones in any of the model grids. However, difficulties with this geometry were encountered due to high static shear stresses along the model base. This resulted in extensive, and non-physical, yield conditions and model displacement during dynamic analyses of seismic loading. Additionally, the sloping base configuration created difficulties with representing the upward propagation of shear waves. A hybrid geometry from second and third generation grids was then
utilized (Fig. 7-3d). However, in attempting to economize on the total number of model zones, this grid slightly exceeded maximum zone dimensions for accurate wave propagation, and could not properly address the dynamic loading requirements.

A fifth, and final grid was established that overcame the geometric limitations encountered in the previous models. This fifth model geometry, shown on Figure 7-3e, formed the basis for the detailed modelling studies described below. Several key features of the grid included:

1/ Vertical model grid columns provided a clear comparison with slope instrumentation (i.e. inclinometers), an important feature in the model calibration.

2/ Grid rows were aligned parallel to the slope surface. This configuration simulated the geomorphic slope processes in the most representative fashion.

3/ The orientation of the grid rows provided the most representative weathering profile in the rock mass (i.e. variation in material properties with depth, as discussed in section 7.6.3).

4/ The alignment of the grid rows provided the best representation of increasing stress level with depth below the slope surface. Modelling studies of large slopes have indicated the parallel arrangement of the major principal stress and the slope in proximity to the ground surface (Sturgul et al., 1976; Savage and Varnes, 1987; and Pan et al., 1995). The arrangement of the model zones parallel to the major principal stress direction enabled a more accurate determination of the stress levels in the zone of interest. Furthermore, this grid arrangement provided the potential for developing the most realistic strain localization in the model, if it was to occur.

To develop the zone geometry shown on Figure 7-3e, several interface surfaces were required to separate portions of the model grid having different zone numbers and sizes (Fig. 7-4. A gradation in zone size was still maintained, as discussed above, however, the interfaces allowed the zone aspect ratios to be kept below a 2:1 value.

7.5 Boundary Conditions

Boundary conditions for surface modelling studies should represent semi-infinite material such that the chosen boundaries exert minimal effect on the zone of interest. Model boundaries
for the Wahleach slope were selected so as to not influence the distribution of stress and
deformation in the region of slope movement.

The Wahleach FLAC model contained two types of boundaries: natural and artificial. The
slope surface represented a natural boundary, whereas the model sides and base were strictly
figments of the modelling process. These artificial boundaries were required to provide a finite
size to the model domain. The objective was to locate the artificial boundaries a sufficient
distance from the zone of interest so they did not influence the model results, measured in terms
of stresses, displacements and material behaviour.

Conceptually, the ridge crest of Four Brother's Mountain, above the Wahleach slope,
represented a plane of negligible lateral displacement. This did not preclude downslope
movements away from the ridge crest, but assumed the ridge did not move appreciably in a
global reference. In a similar fashion the Fraser River valley floor was also considered a
suitable lateral boundary. Consequently, these positions were selected to represent the model
boundaries using fixed horizontal conditions with constant zero velocity.

A comparative study to investigate model behaviour with different boundary locations
was carried out. Simple grids with coarse zonation were used to evaluate the sensitivity of the
model response (Figs. 7-5a,b). Table 7-1 summarizes the difference in model response with
these different boundary locations.

Table 7-1: Comparison of model response with different boundary locations.

<table>
<thead>
<tr>
<th>Model Description</th>
<th>Loading Conditions</th>
<th>Horizontal Displacements</th>
<th>Vertical Displacements</th>
<th>General Model Response</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Upper Position</td>
<td>Lower Position</td>
<td>Upper Position</td>
</tr>
<tr>
<td>Long profile extending to Wahleach Lake</td>
<td>Slope formation</td>
<td>-134</td>
<td>43</td>
<td>2605</td>
</tr>
<tr>
<td>to Four Brothers Mtn.</td>
<td>Slope weathering</td>
<td>20</td>
<td>280</td>
<td>-15</td>
</tr>
<tr>
<td>Short profile extending to Wahleach Lake</td>
<td>Slope formation</td>
<td>-127</td>
<td>35</td>
<td>2650</td>
</tr>
<tr>
<td>to Four Brothers Mtn.</td>
<td>Slope weathering</td>
<td>20</td>
<td>275</td>
<td>-13</td>
</tr>
</tbody>
</table>

Notes: 1. Displacements are reported in millimetres.
2. Positive vertical displacement is upwards.
3. Positive horizontal displacement is to the right.

A number of conclusions were drawn from this boundary comparison:
1/ Displacements at identical monitoring locations in the two cases were comparable
(approximately 1% difference at the second slope break).
2/ The distribution of plastic behaviour, representing temporary material yield during the loading and re-establishment of equilibrium conditions, was nearly identical in the two cases.

3/ A history of the model unbalanced force was remarkably similar for the two cases, suggesting similar model response.

Based on this comparison, it was concluded that locating the upslope model boundary at the ridge crest of Four Brother's Mountain was acceptable for the numerical analysis.

Establishing the level of the lower model boundary was based on the assumption that at a certain depth below ground surface, horizontal and vertical displacements were insignificant. A combination of factors was used in determining this level. General guidelines for numerical modelling of slopes, recommended by Itasca (1996), suggested an overall model dimension of twice the slope height and three times the slope setback (Fig. 7-6). Also important in establishing the model base was maintaining an acceptable model geometry that did not violate the wave transmission criterion, the zone aspect ratio and gradation in zone size.

The chosen lower model boundary was considered to have negligible movements and was therefore fixed in the vertical direction; horizontal confinement being provided by the lateral boundaries. Figure 7-7 indicates the fixity conditions applied along the artificial model boundaries.

As discussed in section 7.4, a number of interface surfaces were utilized in the model to establish the desired geometry. Dynamic analysis required special boundary conditions to prevent non-physical energy reflection back into the model by the artificial boundaries. Interface surfaces in the model cannot adjoin or intercept these dynamic boundary conditions. Consequently, additional zones were required along the sides of the model to isolate the interfaces from the artificial boundaries. Discussion of the dynamic boundary conditions will be discussed in Section 7.7.1.

7.6 Loading Sequence for Calibrative Modelling

The following sections describe the details of the loading conditions for the calibrative modelling sequence. This sequence was designed to simulate the known conditions through the most realistic yet simple process, by simulating the principal geologic and geomorphic processes in the formation of the slope.
7.6.1 Initial Conditions

The initial state of the Wahleach model was constructed to be most representative of *in situ* conditions, including confining stresses, groundwater conditions and material properties.

The distribution of confining stresses in the Wahleach slope for initial model input was considered to be governed by the existing gravitational field, consistent with the studies reported in Section 2.8. Vertical stresses were established from the weight of overburden material assuming an average rock mass density of 2700 kg/m$^3$. Horizontal stresses were initialized at one half the vertical gradient, based on the results of hydraulic jacking tests. These tests indicated confining stresses in the range of 40 to 60 percent of the theoretical total vertical stress. These values were considered representative of minimum confining stresses normal to the steeply dipping discontinuities in the rock mass; thus horizontal stresses ($\sigma_{xx}$ and $\sigma_{zz}$, the out-of-plane stress) of approximately fifty percent of the vertical gravitational stresses were considered reasonable for model input. Although this initial stress distribution was specified for the model domain, it was necessary to allow the model stresses to adjust to the gravitational field, due to the variation in zone size, shape and the sloping model surface (Fig. 7-3e).

Groundwater conditions were described in detail in Chapter 4. Measured piezometric levels in the slope (Fig. 4-1) were used to establish the static groundwater level in the model. Hydrostatic groundwater conditions were inferred below the water table, based on long term trends observed in the slope piezometers.

Material properties required for input into the numerical model were based on empirically derived methods that have been widely used in rock mechanics applications. For the rock mass properties, the updated Hoek-Brown (1996) method was employed, using input parameters based on Point Load and Unconfined Compressive Strength (UCS) tests. The method is based on extensive field and lab experience, and provides strength estimates for jointed rock masses that can not be practically tested in laboratory settings. The curvilinear Hoek-Brown failure envelope provided extrapolated estimates of the rock mass strength, including cohesion, friction angle and tension cut-off, for use in the FLAC Mohr-Coulomb plasticity model. The use of the Hoek-Brown rock mass strength criterion was considered appropriate for evaluating strengths for the Wahleach rock mass based on the scale of the discontinuities with respect to the overall size of the slope (Fig. 7-8).

The range in value of the material properties used was an attempt to capture the
curvilinear nature of the failure envelope (in an otherwise linear Mohr-Coulomb formulation) at different confining stresses in the slope, in addition to the changing rock mass conditions. These strength values are stress level dependent, so it was necessary to establish estimates of the range of principal stresses in key areas of the slope model. Stresses developed in initial runs of the elastic and ubiquitous joint models were used as a range of input values in the determination of the stress level dependent material properties, such as the empirically derived rock mass (Hoek-Brown) and joint (Barton) shear strengths.

An example derivation outlying the estimate of rock mass strength is provided in Appendix A. Table 7-2 highlights the range in rock mass properties used in the modelling. Initial rock mass properties for the model correspond to "fresh" rock conditions, and reflect the good quality rock mass conditions observed at depth in the slope. "Weathered" conditions correspond to the fair-poor quality rock mass observed in the upper sections of the surface drill holes and outcrop. These weathered properties were progressively introduced into the model following the formation of the slope geometry, as described in section 7.6.3.

Table 7-2: Rock mass properties for model input:

<table>
<thead>
<tr>
<th>Rock Mass Property</th>
<th>Initial Conditions (fresh rock mass)</th>
<th>Weathered Rock Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction (degrees)</td>
<td>54 - 62</td>
<td>50 - 54</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>2.4 - 4.5</td>
<td>0.03 - 1.53</td>
</tr>
<tr>
<td>Tension (MPa)</td>
<td>0.5 - 1.0</td>
<td>0.0 - 0.2</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>2700</td>
<td>2700</td>
</tr>
<tr>
<td>Dilation Angle (degrees)</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>Bulk Modulus (MPa)</td>
<td>12250 - 25100</td>
<td>700 - 7500</td>
</tr>
<tr>
<td>Shear Modulus (MPa)</td>
<td>8400 - 17500</td>
<td>500 - 5150</td>
</tr>
</tbody>
</table>

Estimates of the rock mass deformability (shear and bulk moduli) were derived indirectly from the Hoek-Brown method, using values of the elastic modulus, E, and Poisson's ratio, ν. These values were determined in part from lab strength testing, but were also based on previous
experience (Goodman, 1993). The deformation moduli, used in the constitutive relations, and the dilation angle, important in the post failure material response, are typically difficult to establish at the rock mass scale, due to difficulties in representing appropriate field conditions in lab scale tests. The dilation angle can be determined from lab strength testing (ie. direct shear testing, Fig. 7-9). Without suitable tests to derive a site specific value of the dilation angle, the values chosen were based on reported literature, and from experience (Dr. Evert Hoek, personal communication).

Rock joint properties were established using the empirical method proposed by Barton (1987). In a similar fashion to that employed for the rock mass, a range of joint strength properties were input to the model, as shown in Table 7-3. An example of the technique used to estimate the joint shear strength properties is presented in Appendix B. Material properties were based on observations from drill holes, tunnel and surface exposures.

<table>
<thead>
<tr>
<th>Joint Strength Properties</th>
<th>Initial (fresh)</th>
<th>Weathered</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Angle (degrees)</td>
<td>36 - 38</td>
<td>30 - 32</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>1.0 - 1.2</td>
<td>0.0 - 0.05</td>
</tr>
<tr>
<td>Dilation Angle (degrees)</td>
<td>5.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Tension (MPa)</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

The initial model stage was run under two scenarios, first with elastic material and subsequently with elasto-plastic material (Ubiquitous joint model). A comparison of these two runs indicated similar response in terms of stress distribution, and displacement pattern. Both alternatives required approximately 12000 steps to reach a quasi-static solution state. To simplify the modelling sequence and avoid changing the model "material" prior to subsequent modelling stages, the ubiquitous joint model was used for the complete analysis reported below.

The initial block model was run to a steady state condition such that model stresses, displacements, velocities, plasticity conditions and unbalanced forces reflected the development of a new equilibrium state. This condition was checked through a collection of monitoring points established throughout the model, as discussed in section 7.3.
7.6.2 Slope Formation

The actual history of the slope formation can only be considered in a speculative framework, as complex interactions of various geomorphic processes tend to overprint earlier activity. Consequently, the geomorphic process of denudation forming the current slope geometry, considered to include; weathering, erosion, mass wasting and transportation of rock material, was modelled in a simplistic manner. The effects of glacial loading/unloading, tectonic faulting and uplift, and seismic shaking were not included in the modelled slope formation. Modelling of glacial loading and unloading effects on jointed rock slopes has been investigated by Stewart (1995, unpublished report, Appendix C).

Slope formation involved the removal of overburden layers from the initial block model in a progressive fashion. The current slope profile was reached following six excavation stages (Fig. 7-10). The model was run to a quasi-static equilibrium condition following each excavation stage, resulting in modified stress distributions governed by the new model geometry and groundwater conditions. During the slope formation the phreatic surface was lowered in conjunction with each slope surface profile, maintaining a depth approximately 50 metres below the slope surface.

Rock mass properties were held constant during the slope formation sequence. This was justified by considering that a "weathering" front progressing into the rock mass would move at a rate similar to the removal of overburden material. This was obviously a simplistic approach, but considered reasonable for this stage of the modelling, since details of the model movements during slope formation were not the main objective. Additionally, there was no historical evidence available to evaluate the model response during this stage.

7.6.3 Rock Mass Weathering

Rock mass weathering was simulated in the modelling analysis as a progressive reduction in strength values, with a corresponding decrease in deformation moduli. During this stage, the rock mass properties in the afflicted zone were reduced from fresh to weathered values, as outlined in Tables 7-2 and 7-3. A detailed description of the rock mass weathering effects related to the Wahleach slope was presented in section 3.7.2.

The weathering profile was simulated as a "front", which progressively advanced the reduced rock mass properties from the surface into the upper 150 metres of the rock mass. This
gradational contrast in rock properties, shown on Figure 3-6, was based on observed conditions in the drill core, tunnel exposures and surficial outcrops. This profile of material properties provided the additional benefit of addressing the variation in strength and deformation properties with changing confining stress, an inherent characteristic of earth materials (Byrne, 1996).

The weathered rock mass properties were gradually introduced to the model, during which the model was allowed to reestablish a new equilibrium condition. Reduction in frictional strength values were done in 0.5° increments or less, with similarly small reductions in all other properties. These gradual changes were implemented in order to simulate the progressive degradation in rock mass quality, without creating a significant "shock" to the numerical model.

Weathering effects were considered to be more significant along discontinuities when compared to the intact rock. Based on general acknowledgment that rock structure controls rock deformation, strength and permeability (Hoek and Bray, 1981; Brown, 1987; Bandis, 1993), a more significant weathering effect was introduced to the ubiquitous joint structure than the general rock mass. The weathered rock properties were derived from the Hoek-Brown (1996) and Barton (1978) relationships for rock mass and joint properties respectively, as described in section 7.6.1.

7.6.4 Transient groundwater conditions

Groundwater behaviour in the Wahleach rock slope has been observed to fluctuate between near steady-state and transient flow conditions. In general steady-state conditions prevail during the late spring to mid-autumn when water levels reach seasonal lows while transient conditions are most significant during late autumn storms with heavy precipitation and winter storms with rain on snow conditions. Modelling of the groundwater conditions included both seasonal fluctuations in the phreatic surface and the effects of transient flow in the vadose zone.

Instrumentation results have shown a relationship between accelerated slope movements and periods of prolonged, heavy precipitation when transient groundwater flow would be most significant. The fact that Wahleach slope movements occur in the vadose zone implicates the role of transient groundwater conditions in the slope deformation process. Therefore a means of effectively modelling these transient flow conditions in the vadose zone was therefore considered imperative.
Rock mass structure has been shown to exert significant effect on the groundwater flow. The predominant steeply dipping joint set carries much of the downwards seepage, while the shear and fault zones with low permeability material have the capacity to act as retardants to flow. The combined effect of surface infiltration, seepage into the vadose zone and the lower permeability shear zones have been observed to develop transient perched groundwater conditions. Piezometric data has revealed the development of perched conditions, with differential heads ranging up to 10 to 20 metres. The range in spacing of the significant shear features, at 20 to 50 metres, provides some basis for the potential regularity of the perched conditions.

A number of alternatives for modelling the transient groundwater conditions were considered. One approach was to consider the increase in weight with partial saturation of the rock mass above the phreatic surface. Assuming bulk porosity values in the disturbed rock mass of approximately 5 to 10 percent, no discernible effect could be observed in the model. A second approach considered modelling the transient flow as equivalent seepage forces, but this necessitated an assumption of Darcian flow in the rock mass that was not considered representative. Clear evidence of structurally controlled fracture flow along discrete steeply dipping joints was observed throughout the length of the unlined power conduits, thereby invalidating the assumption of Darcian flow, particularly above the phreatic surface where transient flow is most significant. A third approach was based on modelling the interpreted differential heads in the vadose zone as equivalent lateral forces, and applying these forces along the low permeability structural features (shear zones). This technique was partially successful, but resulted in slope movements that were almost entirely horizontal in nature. This pattern disagreed with results of the steel lining offset survey (Fig. 5-3) so the treatment of transient flow required further consideration. A final approach was developed integrating the following conditions, as schematically shown on Figure 4-5:

1/ Vertical seepage forces, based on 5 to 10 metre hydraulic gradients, were applied throughout the vadose zone in conjunction with buoyant unit weights for the rock mass. The seepage forces were calculated as a volumetric force in the following manner:

$$\text{Seepage Force} = i_v \gamma_w V$$

where $i_v$ = vertical hydraulic gradient

$\gamma_w$ = unit weight of water
V = volume through which flow is occurring

The seepage forces were assumed to act vertically downwards, based on the observed structurally controlled flow along the steep dipping joint set. These forces were uniformly distributed at all model gridpoints throughout the vadose zone.

Lateral forces were applied to model gridpoints in the vadose zone at an average of 50 metre intervals. These forces were designed to simulate the perched conditions, with associated differential hydraulic heads developed behind the low permeability shear features, and corresponding hydraulic gradient. The perched conditions developed unbalanced pressures, from which forces were derived, as described in section 4.4.3 and shown on Figure 4-5. These unbalanced forces were considered to generate a net downslope "drag" on the rock mass, analogous to a series of retaining walls impounding differential water levels, thereby justifying the lateral force application. The magnitude of the lateral forces was based on 5 to 15 metre differential heads, consistent with piezometric observations in drill hole DH89-S2 (Fig. 4-3). The net effect of the transient groundwater conditions is a distribution of applied forces in the vadose zone, as shown on Figure 7-11. A seepage force approach was utilized recently by Garga et al. (1995) for modelling flow through rockfill embankments, although this approach has been known for some time, as described by Cedergren (1989).

Following application of the transient groundwater forces, the disturbance to the model was monitored until the time-stepping formulation had progressed to a state reflecting quasi-static equilibrium, based on model unbalanced forces. The transient groundwater effects were then removed and the model allowed to retain a quasi-static equilibrium state. Model evaluation was carried out as described in section 7.3. This modelling approach considered one cycle of transient groundwater conditions to represent the annual effects of significant late autumn to early winter precipitation. Thirty-eight cycles of these transient groundwater sequence were applied in succession to simulate the 1951 to 1989 period, and compared to the results of the steel lining offset survey. Details of the FLAC input data file for the modelling of the transient groundwater flow conditions are included in Appendix D.
7.6.5 Lining Break Event

The final rupture of the steel lining occurred after a period of high precipitation, thus the lining break water release was most likely superimposed on existing transient flow conditions, accounting for the large destabilizing effect on the slope. This event induced a variety of significant changes to the slope conditions that were sufficient to initiate a marked change in deformation behaviour. This fact was based on the instrumentation response following the lining break that illustrated a progressive reduction in deformation rate with time (Fig. 5-4). The maximum deformation rates observed immediately following the lining break also exceeded, and were not consistent with, average movement rates determined from the offset survey. Additionally, the post-lining break deformation rates were considered to be far in excess of projected average annual rates required to develop the observed surficial slope deformation features.

Changes in slope conditions immediately following the break in the steel lining were attributed to the following influences:

1/ Localized increases in groundwater levels and related pore water pressures in the discontinuities in the vicinity of the lining break. The magnitude of this increase was estimated to reach a maximum of approximately 25 to 30 metres, based on the following details:

- The elevation of the tunnel crown was approximately 594.6 metres at the lining break.
- Maximum hydrostatic water levels in the power tunnel during conditions of full pressurization, but no flow, corresponded to 641.6 metres.
- The drop in water level observed in the surge shaft during full operation (i.e. power generation of 60 MW) was approximately 12.7 metres (ref. 2). This corresponded to an estimated head level above the tunnel crown of approximately 34.3 metres.
- Accounting for head losses of 5 to 10 metres, due to seepage through the rock mass, resulted in hydraulic heads in the range of 25 to 30 metres in the vicinity of the lining break.

2/ The change in rock mass conditions due to removal of joint infilling and related strain-induced behaviour, modelled as a localized reduction in strength and deformability.
Transient groundwater conditions in the vicinity of the lining break exceeded regular seasonal conditions described in section 7.6.4. For the modelling work, it was assumed that the lining break occurred simultaneously with a seasonal pulse of transient flow in the upper rock mass. This implied that movements initiated by the effects of heavy seasonal precipitation, and subsequent transient flow in the upper rock mass may have triggered the final failure of the steel lining. The time of occurrence of the lining break was compatible with this scenario.

Assimilating the three conditions described above, the lining break event was modelled in the following manner: Initially, an increase in localized hydrostatic groundwater levels, not exceeding 25 to 30 metres above the break in the lining was introduced to the model (Fig. 7-12). Secondly, a distribution of transient groundwater forces was applied to the model in the manner described in section 7.6.4, with the addition of localized pore water forces around the lining break area corresponding to 5 metres of pressure head. The level of these transient forces was progressively reduced (respecting numerical solution requirements) to reflect the diminishing pressure heads following dewatering of the power conduit. And finally, a minor, localized reduction in joint friction angles (from 32 to 30 degrees) and joint cohesive strengths (from 0.05 to 0.00 MPa), in addition to reduced deformation moduli, were incorporated to simulate the strain-softening effect created by the considerable movements occurring during the initial lining break. The model response to this simulated lining break event was compared to the measured deformations of the steel lining, where approximately 150 mm displacement was measured across the actual tensional rupture.

It should be noted that the model was allowed to respond to these sequential changes prior to the addition of subsequent disturbances. This allowed the model to numerically evaluate and solve for each disturbing effect. Simplicistically what this means was that the model was allowed to numerically solve for the change in conditions. Since the time-stepping formulation incorporates a high degree of viscous damping (80%) in static solution mode, it was imperative to allow for the disturbance to be numerically propagated throughout the model, just as it would be physically propagated throughout the actual slope. Evaluating the state of the model with respect to solving for the change in conditions is best measured through an assessment of the force imbalance throughout the model, termed the maximum unbalanced force. This was a critical factor in the modelling, since even though the transient forces represent a short-lived
phenomenon, the model still was still required to respond numerically to their effect before they could be removed or changed. This particular approach was followed in the treatment of the lining break conditions. For specific details of the FLAC input data file refer to Appendix D.

7.6.6 Post-Lining Break Behaviour

The lining break simulation covered the initial, short-term response of the model slope to the conditions that occurred during January, 1989. These conditions were transient in nature, lasting some days to weeks at full capacity. Thereafter, the elevated groundwater conditions developed in the slope progressively diminished towards seasonal levels. The post-lining break analysis was designed to simulate the slope behaviour for the period 1989 to 1994. Although real time does not figure in the FLAC calculation sequence for static solution conditions, time can be simulated through representative loading conditions, and the associated model response. Accordingly, the post-lining break slope behaviour was simulated through the application of representative loading conditions corresponding to five annual cycles. The behaviour of the slope during this period was well documented by the comprehensive instrumentation, which provided a record of comparison for calibration of the model.

The modelling methodology employed for the post-lining break behaviour involved fluctuating levels of the phreatic surface at depth, in addition to transient groundwater forces in a manner similar to that outlined in section 7.6.4. The first three cycles of this pattern were intended to capture the slope conditions prevailing from mid-1989 through to early 1992. During mid-1992 construction of the realigned intermediate tunnel resulted in a significant drainage effect in the lower portion of the slope, as shown in Figure 7-13. Despite the lowered position of the long-term phreatic surface, the development of the transient groundwater forces, based on perched groundwater levels, was considered to persist in the vadose zone during heavy precipitation periods.

The lining break and post-lining break behaviour comprised the key for calibration of the model. The model response had to be recorded and interpreted in detail to determine if the observed slope conditions were simulated to any degree of realism. It was during these stages that the model had to show reasonable representation of actual conditions in order for the model to be reliably used for the subsequent predictive analyses.
7.7 Predictive Modelling

Having subjected the Wahleach model to the calibrative modelling sequence, considered to represent past loading conditions, the model was subjected to a series of forecasted loading conditions designed to simulate future events. All available evidence suggests that the Wahleach slope exists in an inherent state of instability. Consequently, it was deemed important to investigate the sensitivity of the slope to certain loading conditions that might instigate a change in the current behaviour. Specifically, the objective of the predictive loading analysis was to determine if the slow deformation mechanism could be triggered into a more rapid failure process, thereby posing a hazard to life, facilities and property below the slope. The scenarios investigated in the predictive modelling included seismic shaking related to a large earthquake event, the increase in static groundwater levels and the long term behaviour of the slope under current conditions.

7.7.1 Seismic Loading

The dynamic analysis was intended to evaluate the susceptibility of the Wahleach slope to earthquake loading. To simulate these dynamic loading conditions, it was required to assess the earthquake potential for the Wahleach area, and to select an appropriate time acceleration history from previous events considered representative of the earthquake hazard in the eastern Fraser valley. Ideally these time histories should be derived from previous earthquake events occurring within the surrounding seismogenic zone. Based on the absence of strong local earthquakes to provide such records, an appropriate time history had to be chosen from recorded events in other regions.

In selecting an appropriate earthquake time history it is important to consider conditions that are most representative of the site and estimated seismic activity. This involves matching as closely as possible the following factors between the project site under analysis, and the location of the recorded earthquake event:

1/ The site ground conditions should be similar to those at the recorded location of the time history. For seismic modelling of the Wahleach slope a time history recorded in crystalline rock should be selected if possible. The time history should not be from a site located on thick overburden deposits, as these have the capability of changing/amplifying
the recorded acceleration levels.

2/ Selection of similar tectonic setting, including style of faulting and epicentre to site distance should be matched as closely as possible. For the three earthquake scenarios considered possible for the lower mainland region of British Columbia (see section 3.6) selection of a time history from an earthquake along southern California's San Andreas fault would be considered an inappropriate choice, since these events often include surface rupture along a major strike-slip fault system, quite dissimilar from the seismicity potential in southwestern B.C..

3/ Earthquake magnitude and peak ground motion, in terms of velocity or acceleration, should be representative of the expected levels outlined in section 3.6. This would include earthquake magnitudes in the range of 5.5 to 7.0 for the shallow and deeper events, and 7.5 to 8.5 for the larger subduction event. The subduction event may not be significant at the 280 to 300 kilometre distance between the Wahleach site and the location of the locked-in subduction plate boundary described by Rogers (1995).

4/ Matching of the target response spectrum, developed from a probabilistic analysis of the site, to the available earthquake response spectra provides an acceptable method of selecting an appropriate earthquake time history. Scaling of a time history, with suitable shape but different magnitude, to the target response spectrum is a commonly used technique, although caution should be exercised in scaling time histories either up or down by factors in excess of two.

Selecting an earthquake record that meets all these criteria is often a challenging task, and involves a considerable amount of judgement. Based on the factors described above, a record of the Northridge 1994 earthquake, recorded at the Pacoima Dam, was selected for the modelling of the slope under dynamic loading conditions. The Pacoima Dam time history was recorded on gneissic rock, with a peak ground horizontal acceleration of 0.43 g, notably close to the 0.41g acceleration level established in the probabilistic analysis described below. Details of the Northridge event, as recorded at the Pacoima Dam are summarized in Table 7-4 below:
Table 7-4: Details of Northridge (1994) earthquake, recorded at Pacoima Dam.

<table>
<thead>
<tr>
<th>Peak Horizontal Ground Acceleration (%g)</th>
<th>Epicentral Distance (km)</th>
<th>Magnitude</th>
<th>Fault Mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.43</td>
<td>19</td>
<td>M(L) = 6.4</td>
<td>thrust fault</td>
</tr>
</tbody>
</table>

For the dynamic modelling exercise, a Uniform Hazard Response Spectrum (UHRS) was developed following the Cornell-McGuire method used by the B.C. Hydro Geotechnical Department. This approach is centred on a probabilistic determination that provides a range of spectra at selected probability levels. The UHRS shown in Figure 7-14 was developed by B.C. Hydro geotechnical staff, for the Wahleach dam site, located approximately 4 km northeast of the slope. A summary of the probabilistic estimates of peak ground accelerations for the Wahleach area is presented on Figure 7-15.

It should be recognized that there exists a large degree of uncertainty in the development of these target response spectra, and consequently, the selection of the most representative earthquake records requires experience and judgment. It should also be emphasized that the objective in analyzing seismic loading of the Wahleach site was not to provide precise estimates of slope displacement values, but rather, to investigate the susceptibility of the slope to seismic shaking. To achieve this objective it was considered important to select an earthquake record with seismic shaking levels corresponding to a large potential event. This corresponded to the one in ten thousand year event, with estimated peak ground acceleration levels of 0.41 g. This would provide an evaluation of the seismic sensitivity of a slope with limiting stability conditions such as Wahleach.

Strain-softening behaviour was not considered to be important for the Wahleach slope during the earthquake loading, since the rock mass properties were already considered to be close to residual strength values. Additional strain of a short term mechanical nature was not considered to create a further reduction in the rock mass strength properties based on the large strain conditions already developed.
The modelling work was supplemented with an additional means of calibrating the slope behaviour by considering the effects of a recent earthquake occurring some distance from the Wahleach site. The Duvall, Washington earthquake that occurred in May, 1996, was rather fortuitous in that it provided a lower bound condition on the level of shaking required to initiate movements in the Wahleach slope instruments. This Magnitude 5.5 event was considered to create ground motion values in the order of 0.01g in the Wahleach area, and a thorough check of slope movement instruments immediately following the event indicated that no perceptible displacements were generated by the event (personal communication with Mr. E. G. Enegren, B.C. Hydro Geotechnical Dept.). This provided a further means of calibrating the FLAC model by determining if seismic shaking at a level of 0.01g would create any notable effects. The combination of rock mass and groundwater conditions established in the model to simulate the observed slope behaviour should not be susceptible to collapse, or even notable deformation levels at ground motion levels of 0.01g.

The modelling approach used in the earthquake analysis utilized the Northridge earthquake time history scaled to 0.41g, corresponding to the one in ten thousand, or maximum probable event. For comparison, the same time history was scaled to 0.19g to evaluate the potential effects of earthquake loading corresponding to the one in one thousand year event. Both events were additionally evaluated for post-seismic behaviour to determine if slope movements progressed to a new static equilibrium condition, or to a steady state failure condition. As discussed above, seismic loading corresponding to the 0.01g level of the Duvall earthquake was applied to the model for calibration purposes.

7.7.2 Extreme Groundwater Conditions

This predictive modelling stage investigated the range of groundwater conditions required to initiate steady state slope collapse. This modelling was carried out by raising the phreatic surface in one metre increments from the observed groundwater profile used in the earlier modelling runs. This approach only considered hydrostatic conditions, and did not incorporate transient groundwater effects. To integrate the two in this investigative phase would not allow for clear interpretation of which groundwater condition was generating the critical conditions. The objective of this exercise was to establish an idea of the slope's sensitivity to groundwater levels, and to determine if large increases in groundwater levels could be accommodated by the...
7.7.3 Long-term Evolution of the Slope

This modelling phase investigated the long term development of the slope movement process through the application of repeated cycles of transient groundwater flow. These cycles were identical to those described in section 7.6.4, and did not include any further weathering effects in the rock mass, since it was considered that the rock mass properties were already approaching residual strength values for the hard, jointed rock conditions. Further reduction in material strength properties could only be justified if weak zones of gouge were to develop along a rupture surface. Considering that slope movements have likely been active for much of the Holocene, without the formation of a recognizable, continuous rupture surface, the modelling did not address such a condition.

The primary objective in this modelling stage was to evaluate if the slope maintained a similar deformation behaviour; or evolved into either a more stable configuration, or a potentially accelerating slope movement process, ultimately leading to a rapid failure mechanism.
Figure 7-1a: Plot of principal stresses in model showing successive equilibrium states in response to slope formation. The small spikes seen in the $\sigma_1$ response reflect non-physical transient effects related to the removal of model overburden layers.

Figure 7-1b: Model stress response during rock mass weathering, with stress reductions related to material yield and associated stress redistribution. Note the similar stress pattern in 7-1a, which corresponded to elastic rebound behaviour.
FLAC (Version 3.30)

LEGEND

2/19/1997 10:26
step 52449

HISTORY PLOT
Y-axis:
Max. unbal. force
X-axis:
Number of steps

Figure 7-2: Plot of model maximum unbalanced force in response to change in material properties during rock mass weathering. Yield conditions were reached initially, but stresses were subsequently reduced with the material flow, leading to a new equilibrium condition (levelling off of UBF towards a near zero value).
Figure 7-3a: First generation grid (March, 1995).

Figure 7-3b: Second generation grid (August, 1995).
Figure 7-3c: Third generation grid with sloping base.
Job Title: Wahleach Rock Slope Deformation Analysis - File: WF-DS-2B
From File: wf-ds-2b.sav

FLAC 3.3B

Step 18500
Grid plot
Mover Table

Figure 7-3d: Fourth generation grid.
Figure 7-3e: Fifth generation grid used for detailed modelling study.
Figure 7-4: Plot of model interfaces. Layout was chosen so that interfaces did not intersect the lateral model boundaries which would have interfered with the dynamic boundary conditions.
Figure 7-5b: Long, coarse grid used for comparison of boundary effects.
MODEL SET-UP OVERVIEW

Dimensions of Model

Figure 7-6: Recommendations for model slope set-up.
FLAC 3.38

Step 14181
Fixed Gridpoints
X X-direction
B Both directions
Boundary plot

Figure 7-7: Plot of fixed boundary conditions.
Intact rock-use
Equation 8.1

Single joint set-criterion
applicable to intact rock
components only-use shear
strength criterion for joints.

Two joint sets-use criter-
on with extreme care.

Many joint sets-use
Equation 8.3.

Heavily jointed rock mass-
use Equation 8.3.

Figure 7-8: Rock mass conditions for which the Hoek-Brown failure criterion can be
applied. The Wahleach rock mass falls into the lower two cases, for which the
Hoek-Brown failure criterion has been deemed appropriate, as discussed by
For soils, rocks and concrete, the dilation angle is generally significantly smaller than the friction angle of the material. Vermeer and de Borst (1984) report the following typical values for $\psi$:

<table>
<thead>
<tr>
<th>Material</th>
<th>Typical Dilation Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>dense sand</td>
<td>15°</td>
</tr>
<tr>
<td>loose sand</td>
<td>&lt; 10°</td>
</tr>
<tr>
<td>normally consolidated clay</td>
<td>0°</td>
</tr>
<tr>
<td>granulated and intact marble</td>
<td>12° - 20°</td>
</tr>
<tr>
<td>concrete</td>
<td>12°</td>
</tr>
</tbody>
</table>

Vermeer and de Borst observe that values for the dilation angle are approximately between 0° and 20° whether the material is soil, rock, or concrete. The default value for dilation angle is zero for all the constitutive models in FLAC.

Dilation angle can also be prescribed for the joints in the ubiquitous joint model. This property is typically determined from direct shear tests, and common values can be found in the references discussed in Appendix F.4.3.
Figure 7-10: Plot of current slope profile (slope formation).
Figure 7-11: Plot of transient groundwater effects, modelled as seepage forces, in the vadose zone.
Figure 7-12: Model simulation of lining break event, showing localized increase in groundwater level and applied seepage force distribution.
Figure 7-13: Phreatic surface in slope, reflecting drainage into downstream end of lower tunnel, following 1992 realignment.
NOTES:
1. Response spectra plotted for damping value of 5% of critical damping.
2. Unsealed spectrum is from earthquake time history.

Figure 7-14: Response spectra comparison for Northridge Earthquake, 1994.
NOTE:
Curves marked "Shallow" and "Deep" show accelerations due to shallow and deep seismogenic zones only, respectively. Curve labelled "Combined" shows accelerations due to all zones.

<table>
<thead>
<tr>
<th>Annual Probability</th>
<th>0.01</th>
<th>0.005</th>
<th>0.0021</th>
<th>0.001</th>
<th>0.0005</th>
<th>0.0001</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deep</td>
<td>2.5</td>
<td>4.5</td>
<td>7.4</td>
<td>10.4</td>
<td>12.9</td>
<td>23.5</td>
</tr>
<tr>
<td>Shallow</td>
<td>4.5</td>
<td>7.0</td>
<td>11.6</td>
<td>16.8</td>
<td>22.8</td>
<td>37.9</td>
</tr>
<tr>
<td>Combined</td>
<td>5.8</td>
<td>8.7</td>
<td>13.5</td>
<td>18.9</td>
<td>24.6</td>
<td>40.8</td>
</tr>
</tbody>
</table>

Figure 7-15: Probabilistic estimates of peak ground accelerations for Wahleach area.
Chapter 8 - Discussion of Modelling Results

8.1 Overview

Overall, the modelling work was successful in addressing the objectives set forth in the introduction. A marginally stable condition was developed in the FLAC slope model considered to be representative of current conditions within the Wahleach rock slope. Subsequently, the FLAC model provided good agreement with observed field conditions and monitored slope behaviour in response to the imposed loading conditions.

The numerical analysis revealed the development of episodic model slope movements due to material yield under gravitational stresses and seasonal groundwater conditions; without the need to appeal to seismic or tectonic forces to initiate and drive deformation. It should be emphasized that gravitational stresses alone were not sufficient to drive slope deformation processes, based on a reasonable range of material properties and groundwater levels. The additional effects created by modelling transient groundwater flow through the vadose zone induced conditions of instability and the development of slope movements in good agreement with observed conditions.

The pattern of slope deformation observed in the model displayed a good correlation with the documented slope movement pattern, established in the instruments installed during the 1989/90 investigative work. Additionally, the magnitude and pattern of movements occurring since the construction of the Wahleach project (1951-1993) was well simulated by the modelling analysis. Moreover, the observed surficial landforms often qualitatively linked to slow slope movement processes, were developed in the model, providing meaningful support from the numerical analysis that the geomorphic features seen on the Wahleach slope, and possibly elsewhere, are a manifestation of a progressive, groundwater driven gravitational process. Strong seismic shaking was found to enhance the slope deformation process, and in the extreme case initiate localized flow of near surface material. However, this modelling study established that seismic shaking was not necessarily a required input to drive the deformation process observed in the Wahleach slope, or for the genesis of the anomalous slope morphological features.

Details of model evaluation techniques were described in detail in section 7.3. Model evaluation was extremely important for interpreting the progress of the model with respect to
correspondence with physically acceptable behaviour, and for comparison with monitored slope movements and observed slope conditions, including rock mass conditions and distribution of surficial slope deformation features. Key results from the modelling work are presented in the same sequence as the modelling operations.

8.2 Initial Conditions

An initial state of stress, representing equilibrium conditions, was developed in the detailed slope model. This state of stress reflected a gravitational gradient under a uniform material density of 2700 kg/m³. Vertical stress levels corresponded approximately with the major principal stresses throughout the initial block model (Fig. 8-1). Horizontal in-plane stress levels ($\sigma_{xx}$) corresponded to the minimum principal stress, at roughly 40 to 45 percent of the initial vertical stress levels, consistent with the results of the hydraulic jacking tests described in section 7.6.1. Horizontal out-of-plane stress levels ($\sigma_{zz}$) equilibrated at roughly 55 to 60 percent of the vertical stress levels, corresponding to intermediate stress values. Under this stress regime the out-of-plane stresses ($\sigma_{zz}$) did not enter into the Mohr-Coulomb yield criterion, and the assumption of plane strain conditions was respected. Equilibrium stress levels in the model displayed a minor deviation from the initial input stress values, due to the variable zone geometry (Fig. 8-2). The development of shear stresses ($\tau_{xy}$) reflected the shallow gradient along the top surface of the initial model grid. Plots of the maximum model unbalanced force (Fig. 8-3), model velocities (Fig. 8-4) and model displacements (Fig. 8-5) illustrated the development of an initial equilibrium condition in the model. The above model variables displayed a progression towards level values, indicating the approach towards a balanced numerical solution in the model. A plot of model displacement vectors (Fig. 8-6) indicated that the initialized stress values were not precisely in balance with the gravitational gradient. As a corollary, elastic strains were developed in establishing a new equilibrium stress distribution in the model.

8.3 Slope Formation

This phase of the modelling sequentially developed the present slope geometry (Fig. 7-10). For comparison this modelling sequence was conducted using both Elastic and Ubiquitous joint plasticity models. Essentially little difference was noted between the two cases for the chosen
material properties. Both cases developed equilibrium stress distributions related to the slope geometry, with maximum principal stress trajectories parallel to the topography in the vicinity of the slope surface. With increasing depth below the slope surface the maximum principal stress trajectories rotated towards a vertical orientation (Fig. 8-7), consistent with reported results from other slope analyses (Sturgul et al., 1976; Savage and Varnes, 1987, Ripley and Brawner, 1991). Both models yielded vertical rebound movements in response to the removal of overburden material. Rebound movements of 25.5 cm and 34.5 cm were observed in the Elastic and Ubiquitous joint models, respectively (Fig. 8-8a,b). For the Ubiquitous joint model case, intentionally high strength values were utilized to prevent the development of excessive material failure and plastic strain due to inertial effects during the slope formation. The 10 cm difference in observed displacements between the two models reflected the development of plastic strain in the Ubiquitous joint model during the slope formation. This plastic strain was developed as a result of shear failure along the ubiquitous joint planes. As the excavation of overburden layers was carried out in a coarse manner (for economy of computer effort) unrealistically large impacts to the model were generated. By employing high strength values, the ubiquitous joint model was capable of limiting material yield conditions resulting from inertial effects associated with rapid rebound behaviour. Although the model slope formation was overly abrupt to represent the geomorphic denudation process, the key objective of this stage was to develop a stress distribution in the model slope representing a quasi-static equilibrium condition.

The development of shear stresses within the model accompanied the slope formation (Fig. 8-9). The stepped shear stress profile indicates the rapid slope formation, but highlights the attainment of equilibrium during each excavation sequence. Noteworthy in the shear stress distribution was the development of higher shear stress values in the lower, steeper portion of the slope in comparison to the upper region.

An interesting observation during this stage was the formation of a tensile stress zone in the upper region of the slope (Fig. 8-10). The combination of characteristically low tensile rock mass strengths and prevailing tensile stresses would provide suitable conditions for the development of plastic behaviour in the upper regions of large slopes. Consistent with this observation, many reported cases of sackung type features have been from the upper regions of large slopes. The fact that sackung features and slope movements have not been observed in the
upper area of the Wahleach slope is consistent with the lower slope gradients and the higher rock mass quality in the upper section of the slope.

8.4 Rock Mass Weathering

This modelling stage introduced a progressive reduction in rock mass properties throughout the middle to upper section of the slope to simulate the natural geologic and geomorphic processes. The observed effect could be described as a progressional sequence of "brittle" responses to the diminished (weathered) rock strength properties, which appeared analogous to strain-softening behaviour.

The progressive reduction of rock mass strength led to recurrent yield conditions in the upper 100 metres of the model, as shown on Figure 8-11. Yield conditions were both temporary and periodic during this process, reflecting the capacity of the model to attain successive equilibrium conditions, as stress levels were redistributed throughout the afflicted rock. The regions of yield occurred at low confining stress levels, and corresponded to near surface locations where yield in shear along the ubiquitous joint structure was observed. Numerical solution for the states of stress exceeding the yield condition accounted for the stress redistribution. The stress redistribution process was observed through a comparison of model zone stresses along the slope surface versus stresses at a depth of 50 metres (Fig. 8-12). Both maximum principal stress and shear stress progressively diminished at shallow depth in the rock, whereas stresses below the 50 metre depth increased accordingly. No additional loading conditions were imposed during this model sequence, therefore the behaviour reflected the tendency of the model to "shed" stresses to deeper, less weathered material, a process considered to be representative of natural slope behaviour.

Slope movements associated with the "strain-softening" weathering process were developed in the model, as shown on Figure 8-13. Model displacements were largest in the central portion of the slope and diminished in upslope and downslope directions. Additionally, model slope deformation was confined to the upper 70 metres of the rock mass above the long term phreatic surface (Fig. 8-14). The slope movements occurred as displacement pulses driven by the temporary material yield conditions, during which stress levels temporarily exceeded material strengths. As stresses were redistributed, compatible with current rock mass strengths,
the movements dissipated and the model retained a quasi-static equilibrium condition. Histories of the maximum unbalanced force (Fig. 8-15) and slope surface velocities (Fig. 8-16) provide confirmation of the episodic yield conditions and the reattainment of static equilibrium conditions within the model slope. Noteworthy in the history of model surface velocity (inclinometer S7 area) was the increasing trend in velocity magnitude, suggesting that the slope was progressing towards a more limiting balance between strength and stress conditions, reflective of the marginally stable conditions.

An important observation during initial runs of this modelling sequence was the transformation of the out-of-plane stresses ($\sigma_{zz}$) from the intermediate principal stress into the maximum principal stress. This phenomenon was observed in the middle section of the model slope where shear strength reduction was maximized. This effect was due to the fact that out-of-plane stresses were essentially "locked in", and did not reduce in conjunction with stresses in the plane of the section as the model material was progressively "weathered". This effect was considered to be non-physical, as out-of-plane stresses in the Wahleach slope were judged to undergo similar stress reductions in response to both overburden removal and rock mass weathering. Furthermore, the plane-strain assumption in the FLAC formulation would be violated if the out-of-plane stresses entered into the yield criterion. To prevent this occurrence, a "FLACish" program called Stress_Balance was written that could be imported into the FLAC numerical formulation. The Stress_Balance program maintained the level of the out-of-plane stresses between the maximum and minimum principal stress levels throughout the whole model domain, as shown for a near surface model zone in Figure 8-17. This had the effect of providing out-of-plane stress relief in the model. Details of the Stress_Balance program are included in Appendix E.

8.5 Transient Groundwater Conditions

The response of the FLAC model to the imposed transient groundwater conditions calibrated convincingly with observed slope behaviour. These transient groundwater conditions were imposed on the model both prior to, and following, the lining break event. This section provides discussion of model response prior to the lining break. Calibration of the pre-lining break FLAC model response was possible in two manners. First, the model was compared to the
observed slope deformation features, which provided a comparison in a general geomorphic perspective. Secondly, the overall slope deformation pattern was compared to that measured in the steel lining offset survey.

Deformation of the model slope, as a result of the transient groundwater conditions, occurred in a well defined zone above the phreatic surface (Fig. 8-18a). At the particular stage in the modelling sequence shown in Figure 8-18b, the model material had progressed into a quasi-static equilibrium condition indicated by the elastic state throughout the model domain. However, a plot of slope surface velocity (Fig. 8-19) highlights the cyclic failure conditions and associated model deformation that occurred during the repeated application of the transient groundwater conditions. While displaying the sensitivity of the model slope to these conditions, and the marginally stable conditions developed, the velocity history also demonstrates that an appropriate numerical solution state was achieved during each modelling sequence.

Evaluating the general nature of the model deformation, particularly with respect to the development of geomorphic features, was most effectively achieved by viewing a magnified model grid plot (Fig. 8-20). A comparison of the magnified model grid with the observed slope deformation features yielded two striking similarities:

1/ An extensional deformation zone in the upper area of the model, corresponding to the features described as "hillslope loosening" in the vicinity of drill holes DH89-S2 and DH90-S10 (Fig. 2 from Slope Performance Review).

2/ The development of surficial slope morphology analogous to antislope scarps (linears) in the lower and central region of the model, which were clearly identified in the 1989/90 investigation.

The model grid plot also revealed several important features with respect to the slope deformation mechanics. Deformation was concentrated in 10 to 20 metre thick shear zones in the middle to upper area of the deforming mass, and comprised shear failure both along the steep ubiquitous joints and the overall rock mass, resulting in an overall dilation of the rock mass. Behaviour in the lower portion of the unstable mass was dominated by dispersed, rotational deformation, controlled by toppling failure mechanisms along the steep ubiquitous joints. The overall pattern reflected a complex deformation mechanism that was confined above the long term phreatic surface, as shown in a plot of maximum shear strain increment developed during
the application of the transient groundwater conditions (Fig. 8-21).

An evaluation of the long term deformation potential assumed that each application of transient groundwater conditions corresponded to a seasonal cycle. A projection of the cumulative slope deformation, based on the 38 cycles modelled (Fig. 8-22), resulted in downslope movements in the order of 50 to 80 metres over the Holocene. This agreed with a previous estimate (ref. 1, pp 8-1) which suggested that overall slope deformations in the order of "10's of metres" would be required to develop the large slope linears.

The deformation response observed during this sequence highlighted the sensitivity of the model to the effects of the imposed transient groundwater conditions. The fluctuation of the model between conditions of instability and equilibrium underscored a non-steady state deformation process incompatible with the notion of creep behaviour. A plot of surficial deformation in the model revealed the non-steady nature of this groundwater driven process (Fig. 8-23). A view of the same displacement plot recorded at a wider spaced interval masked the small deformation pulses, and suggested a steady state creep process (Fig. 8-24), despite the fact that important changes in effective stress conditions were occurring in the model.

Although this modelling sequence predated the lining break event, which marked the onset of slope instrumentation, the pattern of model slope deformation (Fig. 8-25) was observed to be remarkably similar to that observed in the steel lining offset survey measured in 1989 (Fig. 5-2). This comparison was considered valid in that the offset survey represented the cumulative displacement in the steel lining, and hence the adjacent rock mass, in the 38 year period following the tunnel construction. Additionally, the magnitude of the model slope movements occurring during this loading sequence provided an estimate of surficial slope movements of up to 10 millimetres per year. This was based on 38 cycles of transient groundwater conditions representing 38 "years" of seasonal conditions (ie. 1951-1989). Maximum model movements occurred along the slope surface, with gradual reduction of displacements below the slope surface. In the vicinity of the steel lining break, model displacements were approximately 40 to 50 percent of the surficial values, corresponding well with the initial measurements of the lining break separation of 150 millimetres.

The observations described above suggest that the procedure used to simulate the transient groundwater conditions provided a good representation of the slope deformation processes.
Furthermore, modelling of this process emphasized the requirement for transient groundwater flow to provide the additional driving force, on top of ambient gravitational stresses, to cause the observed accelerated slope movements related to the prolonged, heavy precipitation events.

8.6 Lining Break Event

The lining break event was effectively modelled following the approach described in section 7.6.5. The simulation of the sudden release of pressurized water into the slope, coupled with a significant transient flow condition, generated a significant response in the model slope, as highlighted by the plot of maximum unbalanced gridpoint forces (Fig. 8-26). The combination of localized seepage forces and elevated water levels generated a model disturbance notably greater than the previously modelled cyclic transient groundwater conditions, and resulted in a significantly larger velocity response (Fig. 8-27). The model simulation was considered to be compatible with the conditions prevailing in the Wahleach slope in late January, 1989.

Estimates of actual movements during the lining break, based on initial separation around the steel lining circumference, were in the order of 150 mm (ref. 1). The response of the FLAC model compared favourably, with horizontal displacements recorded at the lining break location of approximately 170 mm (Fig. 8-28). A distinctive similarity was noted in the trend of diminishing displacement rates and progression towards a more stable condition following the lining break. More important than a discrete comparison of the absolute displacement values, was a comparison of the overall displacement pattern between observed slope conditions and the model response (Fig. 8-29). An overview of the model displacements displayed a range from 200 mm along the ground surface to 100 mm at the 70 metre depth level. These model displacements were roughly parallel to the slope gradient in the upper area of the deforming mass, with a transition to near horizontal displacement towards the toe of the unstable rock mass. The model velocities, captured during the height of the lining break instability, confirmed this movement pattern, and further highlighted the progressive deepening of the failure zone towards the toe area (Fig. 8-30).

A plot of the plasticity condition in the model during the lining break revealed the nature of the yield conditions throughout the deforming zone (Fig. 8-31). Material failure in the lower area was characterized by slip along the vertical ubiquitous joints and was dispersed throughout
the upper 100 metres. In the upslope direction yield conditions were confined within a 10 to 20 metre thickness, as shown on a plot of accumulated shear strain (Fig. 8-32), and comprised both failure along the ubiquitous structure and general shear failure within the modelled rock mass. These observations supported the concept of mixed sliding/flow behaviour in the upper area of the deforming region, and the transition to a more rotational (toppling) failure mechanism in the lower region of the marginally mass. This model behaviour was very similar to that observed in the pre-lining break analyses, suggesting that, although driving forces differed, similar failure mechanisms prevailed.

8.7 Post-Lining Break Behaviour

The FLAC post-lining break slope model provided strong agreement with the slope instrumentation results in terms of general deformation patterns and overall displacement magnitude. This developed a level of confidence in the representative capacity of the model which could be utilized in the subsequent predictive modelling.

Prior to comparing the FLAC model response and slope instrumentation results for calibration, it was important to demonstrate that an acceptable numerical solution was developed in the model. A plot of the model gridpoint velocity in the deformed section indicated that a numerical solution to the applied loading conditions was appropriately achieved (Fig. 8-33). The initial and largest velocity pulse corresponded to the lining break event, during which considerable yield and plastic deformation occurred. The re-establishment of near zero velocity levels indicated the return to a quasi-stable condition with stress redistribution in the deforming zone. The minor, but more prolonged velocity response represented the removal of the lining break loading conditions, and subsequent strain-softening effect around the lining break included in the model sequence. This illustrated that each disturbance to the model was numerically resolved prior to the application of a further loading condition. Noteworthy in the velocity profile was the larger and more prolonged response to lining break loading conditions in comparison to post-lining break transient groundwater conditions. Also evident was the trend of progressively diminishing velocity responses following the lining break, representing the smaller transient effects in the model. This velocity response was consistent with observed conditions in the Wahleach slope, and will be supported by the following discussion of the deformation response.
Evaluation of the post-lining break model response was carried out in the following ways:

1/ Through comparison with slope instrumentation results.
2/ Through comparison with the steel lining offset survey results.
3/ Through an evaluation of the general deformation response of the model, and particularly with respect to the formation of geomorphic features.

Each of these evaluation procedures reflected distinctly different time references. The detailed instrumentation data represented the 1989 to 1994 period (5 years), the offset survey represented the 1951-1993 period (42 years) and the slope deformation features provided a comparative time frame in the order of hundreds of years, indicating an order of magnitude increase between each successive comparison. Evaluating the model response over this range reduced the likelihood that the model response yielded a "fluke" match with the known conditions.

The short term deformation pattern in the slope, representing 1989 to present conditions, was most evident in the inclinometers and the strainmeter located across the steel lining break (Fig. 5-3). This figure outlined the general deformation profile recorded in the instruments and emphasized the general depth of the movement zone. These observations can be compared to the general deformation pattern developed in the model, as shown in the contoured plot of horizontal displacements (Fig. 8-34). Prominent features of the model deformation pattern displayed clear similarity with the monitored slope behaviour, including the following observations:

1/ Model displacements were greatest along the surface and diminished with depth.
2/ Model displacements occurred in the vadose zone, well above the long term phreatic surface.
3/ Model "inclinometers" S5 and S9 established the respective upslope and downslope limits of the deformation zone.
4/ Model deformation was characterized by concentrated bands of shear strain, with thicknesses ranging from 5 to 20 metres, in the upper to middle portions of the deforming slope. Deformation in the lower portion of the movement zone was more gradually dispersed, indicating that a through-going failure surface was not developed in the model.
5/ Horizontal component of the slope movements exceeded the vertical component, resulting
Overall displacements sub-parallel to the slope profile (Fig. 8-35). Consequently the overall movement trends are well represented by the horizontal displacement contours. These model observations were consistent with monitored slope behaviour (ref. 5), and substantiate the effectiveness of the model.

As discussed in Section 7.4, the columns of the model grid were aligned vertically to provide a direct comparison to the slope inclinometers. Five "inclinometer locations" in the FLAC model were chosen to simulate the deformation profiles measured along the cross section used in the analysis. These model inclinometers were instrumental in establishing both the limits and overall displacement patterns for comparison with the actual slope behaviour.

Details of the post-lining break model behaviour (displacement response) are well represented in the model inclinometers. Figures 8-36 to 8-38 present the model inclinometer profiles for comparison to those observed in the respective slope instruments. In general, both deformation patterns (distribution, depth) and magnitudes were comparable for the model and actual slope. It was noted that the model profiles appeared more stepped, relative to the smoother displacement profiles actually measured. Two explanations are provided for this condition. First, the model profiles represented displacement at discrete points in space, whereas the slope inclinometers measured average changes in instrument inclination over two foot (60 cm) intervals, and were therefore less sensitive to small movements. And secondly, the model response represented a continuous record of displacement versus model time, and therefore the changes in movement rate generated by the vadose zone flow were recorded in this cyclic manner. In contrast, the slope inclinometer data was obtained from readings spaced at two to three month intervals. At this frequency, ephemeral fluctuations in movement rate would most likely go undetected, resulting in a smoothed profile, as generally observed. It was seen that model displacement recorded at wider spaced intervals presented a steady-state deformation record analogous to creep (Fig. 8-24). Viewing the model data in this fashion obscured the subtle cyclic movements created by the transient groundwater conditions. This suggested that the monitoring frequency of the slope inclinometers may not capture this cyclic deformation response. Moreover, the description of the slope deformation process as a "creep" process may not reflect the subtle variation in movement rate that accompanies this groundwater driven deformation process. Provenance for a non-steady state process was seen in the strainmeter located across
the steel lining break (UT-SM9, Fig 5-5). This instrument measured real time deformation of the slope with sufficient sensitivity to capture the transient groundwater effects on the slope. Data from this instrument provided justification for the notion that transient groundwater flow may supply the driving forces for the deformation process. A comparable plot of model deformation demonstrated the capability of the model to simulate these conditions (Fig. 8-39).

The post-lining break model was further calibrated against the results of the steel lining offset survey, which provided a forty-two year record of cumulative slope deformation in the steel lining of the original upper power tunnel and inclined shaft. The model response provided a good comparison with the offset survey results, based on a plot of model displacement vectors (Fig. 8-40). These displacement vectors represented the lining break effects in addition to five cycles of transient groundwater conditions, simulated as the 1989-1993 period. Noteworthy observations from this displacement profile included:

1/ The orientation of the model displacement vectors were approximately 25 to 30 degrees below horizontal, with an average inclination of 28 degrees in the vicinity of the lining break. Results from the August, 1993 steel lining offset survey indicated displacement vectors dipping approximately 25 degrees below horizontal in the vicinity of the lining break (Fig. 5-2).

2/ Post-lining break deformation magnitudes of 270 mm were developed in the model. When added to the deformation vectors from pre-lining break conditions (max. disp 384 mm, Fig. 8-25), total model displacements of 655 mm resulted. These displacements provided an exceptionally close match to the August 1993 steel lining offset survey results which reported maximum steel lining displacements of 658 mm. In total, the model simulated 43 cycles ("seasons") of transient groundwater conditions, 38 from the pre-lining break (1951-1988), and 5 (1989-1993) following the lining break, effectively covering the 43 year (1951-1993) history of the Wahleach project.

The post-lining break model response was also compared with the Wahleach slope in a general geomorphic sense. A magnified plot of the model grid following the post-lining break behaviour, clearly showed the development of surface morphology compatible with observed features on the slope (Fig. 8-41). Most obvious were the development of several antislope scarps
along the model surface. Also evident was the extensional zone in the upper area of the slope where numerous tensional features were mapped during the 1989/90 investigation. This figure also highlighted the distribution of deformation at depth in the model. Subsurface deformation behaviour in the model was characterized by zones of concentrated shear in the upper to middle portion of the unstable mass, and a zone of more uniformly distributed deformation in the toe area, suggestive of toppling failure mechanisms along the vertical ubiquitous fabric. This deformation pattern conformed very closely with that observed during the pre-lining break model simulation, implying that failure mechanics were fundamentally similar. This led to the conclusion that the general failure behaviour did not change appreciably with the lining break occurrence, and similar processes prevailed both before and afterwards.

Based on the success of the model in simulating the observed deformation behaviour in the slope, the model response was evaluated in further detail to determine if a more comprehensive understanding of the failure process could be developed. One significant discovery involved the sequence of failure initiation in the model. Analyzing the surface displacement response, at the onset of transient groundwater loading, revealed that plastic deformation initiated in the lower section of the slope, and propagated in the upslope direction (Fig. 8-42). Failure initiated through antithetic shearing along the ubiquitous joints (toppling failure), leading to the progressive undermining of the material upslope. As material in the upper area of the disturbed mass (S10 area) was destabilized, inertial effects transferred load back towards the lower area of the slope, causing a secondary acceleration in the displacement response. This mechanism indicated that the most acute balance of strength and stress( ratio) existed in the toe area of the slope. Therefore, if any remedial measures were considered to mitigate the destabilizing effects of transient groundwater flow, they would be most effective in this critical area.

8.8 Predictive Modelling

8.8.1 Seismic Loading

Seismic loading of the calibrated model slope provided a unique opportunity to investigate the sensitivity of the model slope to a range of earthquake shaking intensities. Additionally, the May 1996 Duvall, Washington earthquake afforded an additional calibration procedure for the
slope modelling, since the low levels of shaking generated by this earthquake event did not induce any discernible response in the slope instruments. Exposure of the model to seismic acceleration levels compatible with the Duvall event passed without measurable effect in the model, lending further support to the effectiveness of the model. However, the model displayed significant response to earthquake shaking at levels corresponding to the one in one thousand (0.19g) and one in ten thousand year (0.41g) events.

Earthquake loading of the FLAC model was based on application of an appropriate earthquake acceleration record to the calibrated slope. The 1994 Northridge event was selected as the most appropriate earthquake record, as described in section 7.7.1. The time acceleration history for the Northridge event was scaled to conform to the acceleration levels predicted for the Wahleach area based on a probabilistic analysis. These acceleration levels are summarized in Table 8-1 below:
Table 8-1: Earthquake acceleration levels for the Wahleach Slope area.

<table>
<thead>
<tr>
<th>Earthquake Case</th>
<th>Peak Horizontal Acceleration (g)</th>
<th>Loading Conditions</th>
</tr>
</thead>
</table>
| I               | 0.01                             | Duvall, Washington event
|                 |                                  | $M_L = 5.5$                             |
| II              | 0.19                             | One in one thousand year event.         |
| III             | 0.41                             | One in ten thousand year event - maximum credible event. |

The Duvall earthquake was estimated to have generated acceleration levels of approximately 0.01g in the Wahleach area (T.E. Little, personal communication). Input acceleration levels corresponding to this acceleration level were applied laterally along the model base (Fig. 8-43). This level of shaking had essentially no effect on the model, based on the displacement response in the lower portion (S1 to S7 inclinometer locations) of the unstable mass (Fig. 8-44). If model deformations were to have occurred under these dynamic loading conditions, they most likely would have developed in this region of the model, as this corresponded to the area displaying the most acute instability condition in the previous static analyses. The displacement profile highlighted zero net displacement, indicating elastic behaviour prevailed throughout the 0.01g seismic loading case.
Following the successful modelling of the Duvall earthquake, the model was subjected to loading conditions representing the one in one thousand year event, corresponding to an annual probability of exceedance of 0.001. Peak ground motions for this event were 0.19g, as shown in Figure 8-45. Yield conditions were developed in the 60 to 80 metres of the rock mass adjacent to the slope surface, similar in distribution to the yield pattern observed in the static loading cases. Model slope movements were generated during the larger acceleration pulses of this earthquake time history (Fig. 8-46). This seismically driven instability was observed to be short lived, as the model retained a quasi-stable condition with attenuation of applied acceleration levels. Elastic behaviour and damping effects were clearly evident in the latter 10 seconds of the earthquake loading, as dynamic energy levels within the model were dissipated. Displacements of approximately 0.3 metres were developed along the slope surface, with localized maximum values around elevation 700 metres (Fig. 8-47). This displacement profile displayed similar trends to the pattern developed during the lining break simulation and transient groundwater conditions; dominated by relatively uniform displacement gradients in the lower section of the slope (S1 area), and more concentrated zones of shear in the upper deforming zone (S7 to S10 area). A plot of the shear strain increment developed during the earthquake loading in the model supported this general deformation pattern (Fig. 8-48). However, displacements were largest in the lower portion of the unstable mass, suggesting that a higher instability potential prevailed in this area. This observation supported conditions in the post-lining break model. In general, the levelling off of model displacements, in conjunction with the pattern of model shear strain increment, indicated that a rapid failure mechanism occurring along a concentrated, through-going shear failure zone did not ensue.

Modelling of the one in ten thousand year event displayed a markedly different result, with extensive deformation occurring throughout the upper 60 to 100 metres of the rock mass, and estimated displacements exceeding several ten's of metres. The displacement history revealed a period of steady state material flow in the surficial 60 to 70 metres of the disturbed slope (Fig. 8-49). The instability was initiated during the 3.75 to 13 second interval, during which peak horizontal accelerations of up to 0.41g were applied to the model base (Fig. 8-50). The full duration of the earthquake event was 20 seconds, at which time the material flow conditions noted above had dissipated, and the unstable rock mass developed a new static equilibrium
condition. With development of large displacements, the vertical ubiquitous joints had rotated into a less critical and stronger orientation (Fig. 8-51). This reflected the tendency for these structurally controlled mechanisms to self-stabilize with increasing deformation, and further indicated the non-catastrophic potential of these toppling failure mechanisms. This observation assumed that no further reduction in strength occurred during the short term earthquake loading. Additional evidence for this strengthening and self-stabilizing response at large strain, was seen in model shear stress histories (Fig. 8-52).

Despite these observations, widespread steady state flow conditions were noted after 15 seconds of seismic shaking, producing model velocities of up to 5 metres per second (Fig. 8-53). Mobilization of this volume of rock on the steep slope gradients at Wahleach was considered to represent the possible development of a more rapid slope movement phenomenon, such as a rock avalanche derived from the surficial "skin" of mobilized rock (Fig. 8-54). Under these conditions small volumes of surficial rock blocks would possibly be mobilized and rapidly transported downslope into the Ted and South Creek channels, creating the potential for debris flow events.

Amplification of surface ground motions has been noted during large earthquakes, notably the 1985 Michoacana event that damaged large portions of Mexico City. This phenomenon has been widely documented, particularly in sites where thick deposits of soft soil occur surficially. Ground motion amplification can also be caused by sloping topography. Finn and Matsunaga (1994), summarizing the results of both theoretical studies and field observations, concluded that the free field ground motions are amplified by topography in relation to motions on level ground. The FLAC model revealed that a similar condition occurred during the seismic loading analyses. In relation to surface accelerations along the valley floor, acceleration levels progressively increased towards the model ridgecrest, with maximum amplification of model acceleration levels of up to 50 percent (Fig. 8-55). This indicates that several effects combined to promote greater deformation potential in the middle to upper regions of the slope; the steeper gradients, the lower rock mass quality and the amplified levels of seismic shaking.

8.8.2 Progressive Increase in Phreatic Surface

This case investigated the potential for a change in slope deformation behaviour with the gradual rising of the phreatic surface. As such this case was based on a hydrostatic groundwater
condition, and therefore considered a highly conservative evaluation of slope instability potential.

Raising of the phreatic surface led to the development of a steady state failure condition within the model slope (Fig. 8-56). Failure conditions, represented by steady state material flow, were initiated when the phreatic surface was raised uniformly a height of 58 metres above the currently monitored levels. This indicated that the calibrated model slope could accommodate a large increase in hydrostatic groundwater levels prior to the development of critical instability. The position of the critical water level was approximately 20 metres above the zone of failure, which was located at a depth of 80 to 90 metres in the central section of the failed mass (Fig. 8-57). Both model observations were contradictory to observed slope conditions, indicating that these high hydrostatic groundwater conditions have likely not played a role in the slope deformation process.

8.8.3 Long Term Effects of Transient Groundwater Conditions

Long term model slope behaviour was investigated through the repeated application of 1000 cycles of transient groundwater conditions. Each loading cycle simulated the seasonal increase in vadose zone flow conditions that has been correlated with the accelerated movement rates in the slope instruments. The model response to these repeated cycles was evaluated to determine if any notable changes developed in the deformation mechanisms or movement rates.

Large strain conditions were developed in response to the transient flow conditions, with maximum displacements in the range of 30 to 40 metres (Fig. 8-58). These large displacements were generated by the incremental accumulation of small displacements with each transient groundwater flow cycle. (Fig. 8-59). One thousand cycles (1,500,000 FLAC model steps) were required to generate this level of displacement. The long term displacement response revealed a gradual attenuation, indicating progression towards a more stable slope condition (Fig. 8-60). The attainment of a more stable configuration occurred as the critically oriented fabric rotated downslope into a flatter orientation (Fig. 8-61). This observation supports the notion that these structurally controlled failure mechanisms progressively self-stabilize. This modelling approach assumed that no further material strain-softening ensued, and that no development of a continuous basal failure surface occurred with the large displacement of the model slope. This assumption was supported by a plot of the shear strain developed under these long term loading conditions,
which showed that a continuous zone of concentrated shearing was not developed in the model (Fig. 8-62). Extensive subsurface information from within the deforming zone would be required to evaluate the potential for developing these lower strength basal shear features.
8.9 Summary of Modelling Analyses

The following table presents a summary of the general model response during each of the main modelling phases:

Table 8-2: Summary of FLAC model behaviour.

<table>
<thead>
<tr>
<th>Modelling Stage</th>
<th>Model Response</th>
<th>Resultant Model State</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Conditions</td>
<td>Stress redistribution</td>
<td>Equilibrium</td>
</tr>
<tr>
<td>Slope Formation</td>
<td>Upward elastic rebound (0.3 m)</td>
<td>Elastic rebound</td>
</tr>
<tr>
<td>Rock Mass Weathering</td>
<td>Progressive (incremental)</td>
<td>Successive quasi-static/marginally stable conditions</td>
</tr>
<tr>
<td></td>
<td>displacements</td>
<td></td>
</tr>
<tr>
<td>Transient Groundwater</td>
<td>Incremental slope movements</td>
<td>Marginally stable conditions</td>
</tr>
<tr>
<td>Seismic loading (scaled Northridge events)</td>
<td>0.01g 0.19g 0.41g</td>
<td>Negligible disp. 0.35 metres disp. Large displacement</td>
</tr>
<tr>
<td>Predictive modelling</td>
<td>Long term behaviour High phreatic surf.</td>
<td>Self-stabilization Failure Condition</td>
</tr>
</tbody>
</table>
Figure 8-1: Comparison of vertical stress levels ($\sigma_{yy}$; in upper figure) with major principal stress levels ($\sigma_1$, in lower figure) prior to slope formation in the initial block model.
Figure 8-2: Stress levels at centre of model following equilibration of initial conditions. The level values reflect numerical "quasi-static" equilibrium and are indicative of stable conditions.
Figure 8-3: Plot of maximum unbalanced force in model during equilibration of initial conditions. The dissipation of the model force imbalance towards near zero values indicates the development of quasi-static equilibrium at this stage of the modelling sequence.
Figure 8-4: Horizontal model velocity along slope surface in initial block. Velocity levels converging towards zero magnitude indicate stable equilibrium conditions.
Figure 8-5: Attenuation of model displacements (upper slope area) in initial block, as stable equilibrium conditions are established.
**FLAC (Version 3.30)**

**LEGEND**

8/09/1996 10:46  
step 12000  
-5.567E+02 < x < 3.377E+03  
-1.817E+03 < y < 2.117E+03  

Displacement vectors  
Max Vector = 9.949E-02

<table>
<thead>
<tr>
<th>Units</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2E -1</td>
</tr>
</tbody>
</table>

**KNIGHT PIESOLD**

Figure 8-6: Model displacement vectors developed during consolidation of initial stress levels to the initial equilibrium state.
Figure 8-7a: Pattern of principle stress vectors in model slope at end of slope formation. The highlighted box is shown in more detail on Fig. 8-7b.
Figure 8-7b: Plot of principal stress vectors following slope formation. Note that \( \sigma_1 \) is oriented parallel to the slope close to the ground surface, with gradual rotation towards a vertical alignment with increasing depth.
Figure 8-8a: Model displacement vectors in elastic model, showing upward rebound in response to overburden removal. Maximum displacements of 25.5 cm were recorded along the ground surface in the toe area of the slope.
Figure 8-8b: Model displacements vectors in ubiquitous joint model, showing upward rebound in response to overburden removal. Maximum displacements of 34.5 cm were recorded along the ground surface in the toe area of the slope.
Figure 8-9: Formation of shear stresses in model during slope formation. Note the levelling off of shear stress values following each excavation sequence, reflecting stable equilibrium conditions. Note also the higher shear stress levels in the lower, steeper slope area (history 92) relative to the upper area.
Figure 8-10: Formation of zone of tensile stress at ridge crest.
Figure 8-11: Plasticity indicator in model slope, highlighting the area where temporary yield occurred in conjunction with rock mass weathering. Note that the slope re-attained a stable, elastic state following stress redistribution (shown in 8-12).
Figure 8-12a: Reduction in shear ($\tau_{xy}$) and maximum principal stress ($\sigma_1$) levels close to the slope surface due to rock mass weathering, which appears similar to strain softening behaviour.

Figure 8-12b: Shear stress redistribution corresponding to rock mass weathering and related material yield. As surficial material weakens (weathers) yield and flow occurs, with redistribution of shear stress levels to deeper (fresher) zones of the rock.
Figure 8-13: Surficial slope movements during rock mass weathering. Note the episodic displacement pulses representing successive quasi-static equilibrium states.
Figure 8-14: Horizontal displacement contours during rock mass weathering, outlining the region of yield related to reduced strength values.
Figure 8-15: Maximum unbalanced force in model during rock mass weathering. Note the diminishing unbalanced force during each successive weathering stage, indicating appropriate numerical solution, and the reattainment of a stable equilibrium state.
Figure 8-16: Horizontal velocity along model slope surface (S7 area) during rock mass weathering. Note the increasing trend in velocity response to the incremental strength reduction, indicating the development of the marginally stable state.
Figure 8-17: Out-of-plane stress ($\sigma_{out}$) correction during rock mass weathering, maintaining the out-of-plane stresses as the intermediate stress, consistent with the plane strain approach.
Figure 8-18a: Model plasticity plot during application of transient groundwater effects, highlighting the zone where yield and flow (deformation) occurred. Note that yield is confined to the zone above the long term phreatic surface.
Figure 8-18b: Detailed model plasticity plot following application of one cycle of transient groundwater conditions, illustrating that a stable equilibrium (elastic) condition was reestablished.
Figure 8-19: History of model surface velocity during transient groundwater conditions, illustrating the cyclic yield and deformation conditions, associated with the marginally stable conditions, followed by the return to successive equilibrium states prior to subsequent cycles.
Figure 8-20: Plot of model grid (magnified) highlighting the formation of slope deformation features, following successive cycles of transient groundwater conditions. Note the region of extensional deformation in the upper slope, and the formation of "antislope scarps".
Figure 8-21: Contour plot of model shear strain increment developed during the application of transient groundwater flow cycles. Note the concentrated deformation bands in the upper area of the slope, with shear displacement distributed over 10 to 20 metre thickness, which translate downslope into a more dispersed deformation pattern distributed over 50 to 100 metres.
Job Title: Wahleach Rock Slope Deformation Analysis - File: WFDG-3hb
From File: wfdg-3h.sav

FLAC 3.38

Step 107300
Displacement vectors
Max Vector = 3.91E-01
Water Table

Figure 8-22: Model displacement vectors resulting from 38 cycles of transient groundwater conditions:
0.3910 m/38 cycles => 10.3 mm/cycle => 50 - 80 m during Holocene
Figure 8-23: History of model surficial displacement (S1 area), highlighting the episodic/cyclic deformation process triggered by the transient groundwater conditions.
Figure 8-24: History of model surficial displacement (S1 area), plotted with a wider recording interval that masks the episodic/cyclic movement pulses, that suggests a steady state creep process.
Figure 8-25: Model displacement vectors developed by the transient groundwater conditions in the vicinity of the upper tunnel steel lining break.
Job Title: Wahleach Rook Slope Deformation Analysis - File: WFDG-LB
From File: wfdg-lb.sav

FLAC 3.30

Step 111288

HISTORY PLOT
Y-axis:
Max. unbal. force
X-axis:
Number of steps

Figure 8-26: History of maximum unbalanced force in model, displaying the larger disturbance created by the lining break event in relation to the previously simulated groundwater effects.
Figure 8.27: Model surface velocity history showing the larger response to the lining break event conditions relative to the "seasonal" transient groundwater conditions.
Figure 8.28: Comparison of measured lining break displacements (a), with model simulation (b). Model displacements adjacent to lining break area were approximately 170 millimetres.
Figure 8-29: Model displacement vectors developed during the lining break event. Note the overall displacement pattern approximately parallel to the slope, and above the phreatic surface.
Figure 8-30: Model velocity vectors during lining break event. Note the deepening of the movement zone towards the lower section of the unstable mass, and the transition to near horizontal movement in the lower area, suggesting a transition to toppling type deformation.
Figure 8.31: Plasticity indicator during lining break event. Note the predominance of yield along the vertical joint fabric in the toe area, dispersed throughout the upper 70 to 90 metres of the model slope, implying toppling dominated behaviour. Yield in the upper deforming section was more concentrated and comprised tensile, general shear and shear along ubiquitous joints, compatible with a sliding mechanism.
Figure 8-32: Model shear strain increment during lining break event. Note the pattern of shear strain is compatible with the deformation mechanism noted in 8-31. Note also that the zone of concentrated deformation does not extend through to the toe of the unstable mass, consistent with field observations.
Figure 8-33: Model gridpoint velocity history along slope surface covering lining break and post-lining break conditions. Velocity profile indicates appropriate numerical solution, and highlights the sensitivity of the slope to the seasonal transient groundwater conditions.
Figure 8-34: Plot of horizontal displacement contours covering the post-lining break response. Note the overall pattern: concentrated deformation in the upper area, and dispersed deformation in the lower area. Note the locations of the model "inclinometers", with negligible deformation in S5 and S9, consistent with instrumentation results.
Figure 8-35: Model displacements in relation to original upper power tunnel and the break in the steel lining. Note the transition of displacements in the upper area (generally parallel the slope surface) to that in the lower section of the unstable mass (containing a more horizontal component, and diminishing with depth).
Figure 8-36: Model inclinometer S-10.
FLAC 3.30

Step 111297

HISTORY PLOT

Y-axis: Depth
X displacement (67, 48) 0 m
X displacement (67, 45) 30 m
X displacement (67, 42) 60 m

X-axis:
Number of steps

Figure 8-37: Model inclinometer S-7.
Figure 8-38: Model inclinometer S-1.
Figure 8-39: Model simulation of strain meter located at lining break (UTSM-9).
Figure 8-40: Post-lining break model displacements in the vicinity of the upper tunnel and inclined shaft. Compare displacement vectors to steel lining offset survey (Fig. 5-2).
Figure 8-41: Magnified grid plot following lining break simulation. Note the following: 1) "antislope scarps"; 2) extensional zone in upper area; 3) concentrated shear deformation in middle to upper portion of slope; 4) dispersed rotational deformation in lower area; and 5) the lowered phreatic surface.
Figure 8-42: Detail of surface displacement response during post-lining break simulation, highlighting the upslope propagation of yield and plastic deformation (slope movements), which are initiated in the S1 area and move progressively to S7 and S10.
Figure 8-43: Acceleration time history for 0.01g event (based on scaling the Northridge event). Vertical axis is horizontal acceleration (units = m/sec$^2$ x 10$^{-2}$); horizontal axis is time (seconds).
Figure 8-44: Surface displacement response in middle portion of deforming zone (S7 area) in response to Duvall earthquake. Note zero net displacement, and elastic behaviour. Vertical axis is displacement in millimetres; horizontal axis records number of steps during dynamic loading.
Figure 8-45: Acceleration time history for 0.19g case. This represents the 1/1000 year event, and uses the scaled Northridge 1994 earthquake record for model input.
Figure 8-46: Model displacement histories along slope surface in response to 0.19g case. Note largest displacements (~ 0.35 m) in the S1 area of unstable mass. Also note elastic behaviour beyond 10 seconds of shaking indicating a new quasi-static equilibrium state, without slope collapse.
Figure 8-47: Horizontal displacement contours in model generated during 0.19g event, showing general deformation pattern. Largest displacements were recorded in the S1 area.
Figure 8.48: Distribution of shear strain increment resulting from 0.19g case. Note the concentrated shear strain in the central portion of the slope that becomes dispersed in lower region.
Figure 8-49: Displacement response to 0.41g event at S1 area. Note the plastic flow occurring between 4 and 20 seconds, leading to substantial displacements (tens of meters), followed by dissipation of movements and new equilibrium state.
Figure 8-50: Acceleration time history for 0.41 g event. Vertical axis is acceleration (metres/sec²); horizontal axis is time (seconds).
Figure 8-51: Plot of ubiquitous joint angle following 0.41g event, highlighting the extent of the disturbed zone. Note the regions of concentrated shear developing in the upper area, with dispersed, rotational deformation in the toe.
Figure 8-52: Model shear stress response in deforming zone (S10 area, 50-70 m depth), during 0.41g event. Note the increase in shear stress levels after 15 seconds of seismic shaking, reflecting the rotation of the joint structure into a more stable configuration (a self-stabilizing process).
Figure 8-54: Model horizontal displacement contours generated during 0.41g shaking. Note the large displacements, particularly in the lower, steeper portion of the slope.
Figure 8-55: Model acceleration levels along the slope surface during 0.41g event, illustrating the amplification of ground motions towards the upper slope area.
Figure 8-56: Onset of steady state plastic flow in model, corresponding to a 58 metre increase in phreatic surface above currently monitored levels.
Figure 8-57: Model plasticity plot corresponding to critical increase in phreatic surface.

Note the location of the concentrated deformation below the phreatic surface at depths of 80 to 100 metres below ground surface.
Figure 8-58: Model displacement vectors following 1000 cycles of transient groundwater conditions in the vadose zone. Note the large (35-40 m) displacements.
Figure 8-59: Incremental displacements along model slope surface in S10 area, outlining the incremental plastic deformation developed with each loading cycle of transient flow.
Figure 8-60: Model displacement response (S10 surface area) following 1000 cycles of transient groundwater conditions. Note the progression towards a stable condition.
Figure 8-61: Rotation of ubiquitous joint structure in deforming section of slope during long term transient conditions (1000 cycles). Rotation of joints into a flatter orientation accompanied self-stabilization of the model slope.
Figure 8-62: Plot of shear strain increment developed during 1000 cycles of transient groundwater conditions. Note that a continuous zone of concentrated shear failure was not developed at these large strain values.
Chapter 9 - Conclusions

The Wahleach rock slope has undergone large scale movements, and is currently experiencing a slow, complex deformation process. This slope deformation phenomenon has only recently been recognized as a regionally important, but poorly understood mass movement process.

The characteristic slow deformation rates, large strain conditions and complex material behaviour have exceeded the capabilities of most commonly used analytical methods, such as limit equilibrium methods. This project has investigated the potential for employing a recently developed, state-of-the-art, finite difference numerical code (FLAC) for modelling of the complex geotechnical conditions in the Wahleach slope.

Numerical modelling of the Wahleach rock slope presented a novel approach in the analysis of large natural rock slopes, achieved by incorporating detailed geologic and slope movement data from a large actively deforming slope into a numerical analysis method capable of modelling the large scale deformation process. Another significant feature of this modelling study was that it involved analysis of an actively deforming slope prior to collapse, a contrast from many studies that involve post-mortem analyses of failed slopes.

A geologic model, representative of the known slope conditions, was developed for analysis with the FLAC code. A FLAC model was designed, constructed and run under a sequence of loading conditions simulating observed conditions. The response of the model was calibrated against an extensive slope monitoring history, that afforded a unique opportunity to investigate this complex deformation mechanism.

The modelling study simulated several significant geologic and geomorphic processes in the slope formation, including valley development and stress rebound, rock mass structure and weathering, and fluctuating groundwater conditions. The FLAC code was found to be suitably versatile in the physical representation of the stress-displacement material behaviour under these conditions.

The model response to imposed loading conditions compared favourably with the observed slope movement history. The success of the modelling study was measured partly in terms of the ability to reproduce the monitored slope movement pattern, but more importantly, in the capability of the model to elucidate fundamental, and previously unresolved behavioural
factors in this process. Notably, the modelling study illustrated the role of transient groundwater conditions, in unison with gravitational stresses, to potentially drive this slow, incremental, large scale deformation process. Previous studies of similarly afflicted rock slopes have often attributed this deformation process to time dependent "creep" behaviour. This study has demonstrated that time varying deformation behaviour can be overlooked in slow deformation processes. The scarcity of detailed instrumentation has led to this creep classification in many afflicted slopes that may really be dominated by temporal instability and incremental deformation behaviour as a result of changing in situ stress conditions. Detailed slope movement data, in conjunction with supportive modelling studies, could advance the knowledge of large rock slope behaviour, as has been the case with the Wahleach Project.

This study does not dismiss the potential for time dependent behaviour to comprise a component of these characteristically slow deformation processes. However, in crystalline rock slopes that do not contain appreciable amounts of soft material, such as at Wahleach, deformation behaviour may be more related to time varying stress fluctuations. Changing effective stress conditions associated with seasonal groundwater flow has been shown to be a potential driving force in the inherent instability conditions observed. This may apply equally to other similar slopes.

The modelling established a more comprehensive understanding of the deformation mechanics considered to be prevalent in the Wahleach slope. A complex deformation mechanism, involving structurally controlled toppling failure along steeply dipping fabric in the lower region of the slope, was observed to propagate upslope into a highly altered, disturbed rock mass where more concentrated shear failure predominated. The reduced rock quality in the middle to upper slope area, considered to have an earlier tectonic origin, contributed to the marginal stability of the slope. Additionally, more limiting stability conditions were observed in the lower area of the deforming mass. This condition resulted in the initiation of slope movements in the lower (S1) area, undermining upslope material with the propagation of movements towards the upper region of the marginally stable mass.

The modelling demonstrated that seismic shaking was not a required input for generation of slope movements, as has been proposed in several investigations of this process. Despite this observation, dynamic analyses indicated the sensitivity of the slope to large earthquake events, in the form of large surficial deformation, suggesting that significant seismic shaking has
probably not occurred in the immediate vicinity of the slope in the recent geologic past. Seismic shaking corresponding to the estimated one in ten thousand year event (0.41 g), triggered large surficial displacements, but did not result in large scale slope collapse. These displacements may be problematic in relation to surficial type slope processes, such as rock falls and small volume rock avalanches, and may facilitate heightened potential for debris flow events in the existing creek channels adjacent to the slope.

The modelling study revealed a clear relationship between the observed slope deformation features on the Wahleach slope and the long-term deformation process. The presence of sackung type features should therefore act as a warning against development on or in slopes displaying these geomorphic features, prior to developing a more detailed understanding of the slope conditions and behaviour.

The modelling indicated that rapid failure involving large slope volumes was unlikely, based on an understanding of the current deformation mechanisms. A significant change in behaviour would be expected if a zone of concentrated shear failure propagated through the toe of the disturbed rock mass. Further long term study would be required to evaluate the potential for shear failure to propagate through the lower section of the disturbed mass.

Modelling of the long term slope behaviour indicated that current trends in the order of 5 to 10 mm/yr, and including seasonal variations in movement rates, are likely to continue for some time. If the deformation mechanics do not change appreciably, then the toppling failure prevalent in the lower section of the slope, is projected to develop a self-arresting condition.
References


APPENDIX A

HOEK-BROWN ROCK MASS CLASSIFICATION
**GENERALIZED HOEK-BROWN (1995) ROCK MASS STRENGTH**

**Project:** Wahleach Rock Slope - DH90-S10

**Rock Mass:** Granodiorite - (generally hard and strong, with zones of weak friable material)

**Structure:** Generally highly fractured, with zones of sheared and crushed rock; improving to blocky at depth.

**Surface Condition:** Typically slightly weathered, frequent altered joint walls and thin infilling materials.

### Input:

<table>
<thead>
<tr>
<th>GSI</th>
<th>65 Based on RQD&gt;50</th>
</tr>
</thead>
<tbody>
<tr>
<td>sigci (MPa)</td>
<td>100</td>
</tr>
<tr>
<td>mi</td>
<td>28</td>
</tr>
<tr>
<td>Z (m)</td>
<td>100</td>
</tr>
<tr>
<td>Dr (MPa/m)</td>
<td>0.0265</td>
</tr>
</tbody>
</table>

### Output:

| Sigm | 2.650 |
| Sigt | -0.255 |
| mb | 8.022 |
| s | 0.0205 |
| a | 0.50 |
| mu | 0.28 |
| E | 23714 |
| B | 17965 |
| phi | 65.68 |
| S | 9263 |
| coh | 1.198 |
| sigcm | 11.120 |

### Table:

<table>
<thead>
<tr>
<th>sig3</th>
<th>sig1</th>
<th>ds1ds3</th>
<th>sign</th>
<th>tau</th>
<th>sigtau</th>
<th>signsq</th>
<th>lin. plot</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.234</td>
<td>3.841</td>
<td>99.423</td>
<td>-0.194</td>
<td>0.405</td>
<td>-0.078</td>
<td>0.038</td>
<td>0.769</td>
</tr>
<tr>
<td>-0.214</td>
<td>5.550</td>
<td>70.596</td>
<td>-0.133</td>
<td>0.676</td>
<td>-0.090</td>
<td>0.018</td>
<td>0.903</td>
</tr>
<tr>
<td>-0.172</td>
<td>7.978</td>
<td>50.212</td>
<td>-0.013</td>
<td>1.128</td>
<td>-0.015</td>
<td>0.000</td>
<td>1.169</td>
</tr>
<tr>
<td>-0.090</td>
<td>11.437</td>
<td>35.798</td>
<td>0.224</td>
<td>1.874</td>
<td>0.419</td>
<td>0.050</td>
<td>1.693</td>
</tr>
<tr>
<td>0.076</td>
<td>16.377</td>
<td>25.606</td>
<td>0.689</td>
<td>3.100</td>
<td>2.136</td>
<td>0.474</td>
<td>2.722</td>
</tr>
<tr>
<td>0.407</td>
<td>23.461</td>
<td>18.399</td>
<td>1.596</td>
<td>5.097</td>
<td>8.134</td>
<td>2.546</td>
<td>4.729</td>
</tr>
</tbody>
</table>

**SUMS:**

| 5.517 | 20.595 | 38.351 | 14.344 |

**TAU vs. SIGMA(N)**

![Graph of TAU vs. SIGMA(N)](image)

WAH-RM1.XLS

TWStewart - 2/19/97
**Project:** Wahleach Rock Slope - DH90-S10  
**Rock Mass:** Granodiorite (generally hard and strong, with zones of weak, friable material)  
**Structure:** Generally highly fractured with zones of sheared and crushed rock; improving to massive at depth  
**Surface Condition:** Typically slightly weathered, frequent altered joint walls and thin infilling  

<table>
<thead>
<tr>
<th>Input</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>80</td>
</tr>
<tr>
<td>sigc1 (MPa)</td>
<td>125</td>
</tr>
<tr>
<td>m</td>
<td>30</td>
</tr>
<tr>
<td>Z (m)</td>
<td>500</td>
</tr>
<tr>
<td>Dr (MPa/m)</td>
<td>0.0285</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Sigm</td>
<td>13.250</td>
</tr>
<tr>
<td>Sigt</td>
<td>-0.922</td>
</tr>
<tr>
<td>mb</td>
<td>14.686</td>
</tr>
<tr>
<td>s</td>
<td>0.1084</td>
</tr>
<tr>
<td>a</td>
<td>0.50</td>
</tr>
<tr>
<td>E</td>
<td>6287.2</td>
</tr>
<tr>
<td>phi</td>
<td>61.02</td>
</tr>
<tr>
<td>coh</td>
<td>4.236</td>
</tr>
<tr>
<td>sigcm</td>
<td>32.778</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>sig3</th>
<th>sig1</th>
<th>ds1ds3</th>
<th>sign</th>
<th>tau</th>
<th>signtau</th>
<th>signsq</th>
<th>lin. plot</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.819</td>
<td>12.966</td>
<td>67.585</td>
<td>-0.618</td>
<td>1.652</td>
<td>-1.021</td>
<td>0.382</td>
<td>3.121</td>
</tr>
<tr>
<td>-0.715</td>
<td>18.780</td>
<td>48.093</td>
<td>-0.318</td>
<td>2.754</td>
<td>-0.876</td>
<td>0.101</td>
<td>3.662</td>
</tr>
<tr>
<td>-0.508</td>
<td>27.062</td>
<td>34.293</td>
<td>0.273</td>
<td>4.575</td>
<td>1.248</td>
<td>0.074</td>
<td>4.729</td>
</tr>
<tr>
<td>-0.094</td>
<td>38.896</td>
<td>24.541</td>
<td>1.432</td>
<td>7.562</td>
<td>10.832</td>
<td>2.052</td>
<td>6.822</td>
</tr>
<tr>
<td>0.734</td>
<td>55.875</td>
<td>17.646</td>
<td>3.691</td>
<td>12.422</td>
<td>45.852</td>
<td>13.624</td>
<td>10.899</td>
</tr>
<tr>
<td>2.390</td>
<td>80.371</td>
<td>12.771</td>
<td>8.053</td>
<td>20.237</td>
<td>162.965</td>
<td>64.850</td>
<td>18.774</td>
</tr>
<tr>
<td>5.703</td>
<td>115.984</td>
<td>9.323</td>
<td>16.386</td>
<td>32.619</td>
<td>534.484</td>
<td>268.486</td>
<td>33.816</td>
</tr>
</tbody>
</table>

**SUMS =** 28.899 81.822 753.485 349.571

**Reference:**  
Hoek, E (1995)  
Estimating the strength and deformability of very poor quality rock masses.  
Draft Paper, 10 March 1995
<table>
<thead>
<tr>
<th>Project:</th>
<th>Wahleach Rock Slope - DH89-S7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock Mass:</td>
<td>Granodiorite (generally hard and strong, with zones of weak, friable material)</td>
</tr>
<tr>
<td>Structure:</td>
<td>Generally highly fractured with zones of sheared and crushed rock; improving to massive at depth</td>
</tr>
<tr>
<td>Surface Condition:</td>
<td>Typically slightly weathered, frequent altered joint walls and thin infilling</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Input:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>15</td>
</tr>
<tr>
<td>$\mu$</td>
<td>0.35</td>
</tr>
<tr>
<td>$\sigma_{cg}$</td>
<td>25</td>
</tr>
<tr>
<td>$\mu_1$</td>
<td>18</td>
</tr>
<tr>
<td>$Z$ (m)</td>
<td>65</td>
</tr>
<tr>
<td>$D_r$ (MPa/m)</td>
<td>0.0265</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{gt}$</td>
<td>1.723</td>
</tr>
<tr>
<td>$\sigma_{gm}$</td>
<td>0.000</td>
</tr>
<tr>
<td>$\sigma_{m}$</td>
<td>0.965</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>0.0000</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>0.58</td>
</tr>
<tr>
<td>$E$ (GPa)</td>
<td>667</td>
</tr>
<tr>
<td>$\phi$(deg)</td>
<td>39.37</td>
</tr>
<tr>
<td>$\sigma_{coh}$</td>
<td>0.111</td>
</tr>
<tr>
<td>$\sigma_{cm}$</td>
<td>0.468</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\sigma_3$</th>
<th>$\sigma_1$</th>
<th>$\Delta \tau_{d1d3}$</th>
<th>$\sigma_{in}$</th>
<th>$\tau_{i}$</th>
<th>$\sigma_{itau}$</th>
<th>$\sigma_{isq}$</th>
<th>$\alpha_{in}$ plot</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.013</td>
<td>0.317</td>
<td>13.963</td>
<td>0.034</td>
<td>0.076</td>
<td>0.003</td>
<td>0.001</td>
<td>0.138</td>
</tr>
<tr>
<td>0.027</td>
<td>0.479</td>
<td>10.655</td>
<td>0.066</td>
<td>0.127</td>
<td>0.008</td>
<td>0.004</td>
<td>0.165</td>
</tr>
<tr>
<td>0.054</td>
<td>0.727</td>
<td>8.191</td>
<td>0.127</td>
<td>0.210</td>
<td>0.027</td>
<td>0.016</td>
<td>0.215</td>
</tr>
<tr>
<td>0.108</td>
<td>1.111</td>
<td>6.356</td>
<td>0.244</td>
<td>0.344</td>
<td>0.084</td>
<td>0.060</td>
<td>0.311</td>
</tr>
<tr>
<td>0.215</td>
<td>1.709</td>
<td>4.990</td>
<td>0.465</td>
<td>0.557</td>
<td>0.259</td>
<td>0.216</td>
<td>0.492</td>
</tr>
<tr>
<td>0.431</td>
<td>2.656</td>
<td>3.972</td>
<td>0.878</td>
<td>0.892</td>
<td>0.784</td>
<td>0.771</td>
<td>0.831</td>
</tr>
<tr>
<td>0.861</td>
<td>4.177</td>
<td>3.213</td>
<td>1.648</td>
<td>1.411</td>
<td>2.325</td>
<td>2.716</td>
<td>1.463</td>
</tr>
</tbody>
</table>

SUMS = 3.462 3.615 3.488 3.785

Reference:
Estimating the strength and deformability of very poor quality rock masses.
Draft Paper, 10 March 1995
APPENDIX B

BARTON JOINT STRENGTH PROPERTIES
### Project: Wahleach Rock Slope

**Joint Shear Strength Values - For fresh, intact rock**

#### Details:

**Input parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic friction angle (PHIB) - degrees</td>
<td>31</td>
</tr>
<tr>
<td>Joint Roughness Coefficient (JRC)</td>
<td>10</td>
</tr>
<tr>
<td>Joint Compressive Strength (JCS)</td>
<td>125</td>
</tr>
<tr>
<td>Minimum normal stress (SIGNMIN)</td>
<td>0.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Normal stress (SIGN)</th>
<th>Shear strength (TAU)</th>
<th>Friction angle (PHI)</th>
<th>Cohesive strength (COH)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.066</td>
<td>0.134</td>
<td>1.641</td>
<td>56.64</td>
<td>0.026</td>
</tr>
<tr>
<td>0.199</td>
<td>0.331</td>
<td>1.378</td>
<td>54.02</td>
<td>0.057</td>
</tr>
<tr>
<td>0.331</td>
<td>0.505</td>
<td>1.274</td>
<td>51.87</td>
<td>0.084</td>
</tr>
<tr>
<td>0.463</td>
<td>0.669</td>
<td>1.211</td>
<td>50.45</td>
<td>0.106</td>
</tr>
<tr>
<td>0.596</td>
<td>0.827</td>
<td>1.166</td>
<td>49.37</td>
<td>0.132</td>
</tr>
<tr>
<td>0.782</td>
<td>1.039</td>
<td>1.119</td>
<td>48.22</td>
<td>0.164</td>
</tr>
<tr>
<td>0.861</td>
<td>1.127</td>
<td>1.103</td>
<td>47.81</td>
<td>0.177</td>
</tr>
<tr>
<td>0.993</td>
<td>1.271</td>
<td>1.080</td>
<td>47.20</td>
<td>0.199</td>
</tr>
<tr>
<td>1.159</td>
<td>1.448</td>
<td>1.055</td>
<td>46.54</td>
<td>0.225</td>
</tr>
<tr>
<td>1.358</td>
<td>1.656</td>
<td>1.031</td>
<td>45.87</td>
<td>0.256</td>
</tr>
<tr>
<td>1.854</td>
<td>2.155</td>
<td>0.994</td>
<td>44.54</td>
<td>0.330</td>
</tr>
<tr>
<td>2.152</td>
<td>2.444</td>
<td>0.962</td>
<td>43.90</td>
<td>0.374</td>
</tr>
<tr>
<td>2.483</td>
<td>2.760</td>
<td>0.942</td>
<td>43.29</td>
<td>0.421</td>
</tr>
<tr>
<td>4.635</td>
<td>4.685</td>
<td>0.858</td>
<td>40.62</td>
<td>0.710</td>
</tr>
</tbody>
</table>

T.W. Stewart - 1/29/96
## Project: Wahleach Rock Slope

Joint Shear Strength Values - For weathered, slightly disturbed rock

### Details:

### Input parameters

- **Basic friction angle (PHIB) - degrees**: 31
- **Joint Roughness Coefficient (JRC)**: 5
- **Joint Compressive Strength (JCS)**: 50
- **Minimum normal stress (SIGNMIN)**: 0.1

### Normal stress, Shear strength, Friction angle, Cohesive strength

<table>
<thead>
<tr>
<th>Normal stress (SIGN)</th>
<th>Shear stress (TAU)</th>
<th>dTAU</th>
<th>Friction angle (PHI)</th>
<th>Cohesive strength (COH)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.066</td>
<td>0.067</td>
<td>0.937</td>
<td>43.13</td>
<td>0.005</td>
<td>5</td>
</tr>
<tr>
<td>0.199</td>
<td>0.186</td>
<td>0.862</td>
<td>40.75</td>
<td>0.014</td>
<td>15</td>
</tr>
<tr>
<td>0.331</td>
<td>0.297</td>
<td>0.829</td>
<td>39.65</td>
<td>0.023</td>
<td>25</td>
</tr>
<tr>
<td>0.463</td>
<td>0.405</td>
<td>0.808</td>
<td>38.92</td>
<td>0.031</td>
<td>35</td>
</tr>
<tr>
<td>0.596</td>
<td>0.511</td>
<td>0.792</td>
<td>38.38</td>
<td>0.039</td>
<td>45</td>
</tr>
<tr>
<td>0.782</td>
<td>0.657</td>
<td>0.775</td>
<td>37.79</td>
<td>0.051</td>
<td>55</td>
</tr>
<tr>
<td>0.861</td>
<td>0.718</td>
<td>0.770</td>
<td>37.58</td>
<td>0.055</td>
<td>65</td>
</tr>
<tr>
<td>0.993</td>
<td>0.819</td>
<td>0.761</td>
<td>37.27</td>
<td>0.063</td>
<td>75</td>
</tr>
<tr>
<td>1.159</td>
<td>0.944</td>
<td>0.752</td>
<td>36.93</td>
<td>0.073</td>
<td>87.5</td>
</tr>
<tr>
<td>1.358</td>
<td>1.093</td>
<td>0.742</td>
<td>36.59</td>
<td>0.085</td>
<td>102.5</td>
</tr>
<tr>
<td>1.654</td>
<td>1.457</td>
<td>0.724</td>
<td>35.92</td>
<td>0.114</td>
<td>140</td>
</tr>
<tr>
<td>2.152</td>
<td>1.671</td>
<td>0.716</td>
<td>35.59</td>
<td>0.131</td>
<td>162.5</td>
</tr>
<tr>
<td>2.483</td>
<td>1.907</td>
<td>0.708</td>
<td>35.28</td>
<td>0.150</td>
<td>187.5</td>
</tr>
<tr>
<td>4.535</td>
<td>3.388</td>
<td>0.673</td>
<td>33.93</td>
<td>0.270</td>
<td>350</td>
</tr>
</tbody>
</table>
### Barton Shear Failure Criterion

**Project:** Wahleach Rock Slope  
**Joint Shear Strength Values - For weathered, slightly disturbed rock**

#### Input Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic friction angle (PHIB)</td>
<td>28</td>
</tr>
<tr>
<td>Joint Roughness Coefficient (JRC)</td>
<td>5</td>
</tr>
<tr>
<td>Joint Compressive Strength (JCS)</td>
<td>50</td>
</tr>
<tr>
<td>Minimum normal stress (SIGNMIN)</td>
<td>0.1</td>
</tr>
</tbody>
</table>

#### Normal and Shear Stress Values

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Normal stress (MPa)</th>
<th>Shear strength (MPa)</th>
<th>Friction angle (deg)</th>
<th>Cohesive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.066</td>
<td>0.060</td>
<td>0.843</td>
<td>40.14</td>
</tr>
<tr>
<td>15</td>
<td>0.199</td>
<td>0.167</td>
<td>0.775</td>
<td>37.76</td>
</tr>
<tr>
<td>25</td>
<td>0.331</td>
<td>0.267</td>
<td>0.744</td>
<td>36.66</td>
</tr>
<tr>
<td>35</td>
<td>0.463</td>
<td>0.364</td>
<td>0.725</td>
<td>35.93</td>
</tr>
<tr>
<td>45</td>
<td>0.596</td>
<td>0.459</td>
<td>0.710</td>
<td>35.38</td>
</tr>
<tr>
<td>55</td>
<td>0.782</td>
<td>0.590</td>
<td>0.695</td>
<td>34.79</td>
</tr>
<tr>
<td>65</td>
<td>0.861</td>
<td>0.645</td>
<td>0.659</td>
<td>34.59</td>
</tr>
<tr>
<td>75</td>
<td>0.993</td>
<td>0.735</td>
<td>0.682</td>
<td>34.28</td>
</tr>
<tr>
<td>87.5</td>
<td>1.159</td>
<td>0.847</td>
<td>0.673</td>
<td>33.94</td>
</tr>
<tr>
<td>102.5</td>
<td>1.358</td>
<td>0.981</td>
<td>0.664</td>
<td>33.60</td>
</tr>
<tr>
<td>140</td>
<td>1.854</td>
<td>1.306</td>
<td>0.648</td>
<td>32.92</td>
</tr>
<tr>
<td>162.5</td>
<td>2.152</td>
<td>1.497</td>
<td>0.640</td>
<td>32.60</td>
</tr>
<tr>
<td>187.5</td>
<td>2.483</td>
<td>1.708</td>
<td>0.632</td>
<td>32.29</td>
</tr>
<tr>
<td>350</td>
<td>4.635</td>
<td>3.029</td>
<td>0.599</td>
<td>30.94</td>
</tr>
</tbody>
</table>
### BARTON SHEAR FAILURE CRITERION

**Project:** Wahleach Rock Slope

**Joint Shear Strength Values - For weathered, displaced rock**

**Details:**

**Input parameters**

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic friction angle (PHIB) - degrees</td>
<td>28</td>
</tr>
<tr>
<td>Joint Roughness Coefficient (JRC)</td>
<td>1</td>
</tr>
<tr>
<td>Joint Compressive Strength (JCS)</td>
<td>25</td>
</tr>
<tr>
<td>Minimum normal stress (SIGNMIN)</td>
<td>0.1</td>
</tr>
</tbody>
</table>

**Normal Shear strength, Friction angle, Cohesive strength**

<table>
<thead>
<tr>
<th>Normal stress (SIGN) (MPa)</th>
<th>Shear strength (dTAU) (MPa)</th>
<th>Friction angle (dSIGN) (deg)</th>
<th>Cohesive strength (DTDS) (MPa)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.066</td>
<td>0.039</td>
<td>0.561</td>
<td>30.14</td>
<td>5</td>
</tr>
<tr>
<td>0.159</td>
<td>0.115</td>
<td>0.570</td>
<td>29.66</td>
<td>15</td>
</tr>
<tr>
<td>0.331</td>
<td>0.190</td>
<td>0.564</td>
<td>29.44</td>
<td>25</td>
</tr>
<tr>
<td>0.463</td>
<td>0.264</td>
<td>0.561</td>
<td>29.30</td>
<td>35</td>
</tr>
<tr>
<td>0.596</td>
<td>0.339</td>
<td>0.559</td>
<td>29.19</td>
<td>45</td>
</tr>
<tr>
<td>0.782</td>
<td>0.443</td>
<td>0.556</td>
<td>29.07</td>
<td>55</td>
</tr>
<tr>
<td>0.861</td>
<td>0.486</td>
<td>0.555</td>
<td>29.03</td>
<td>65</td>
</tr>
<tr>
<td>0.993</td>
<td>0.560</td>
<td>0.554</td>
<td>28.96</td>
<td>75</td>
</tr>
<tr>
<td>1.159</td>
<td>0.651</td>
<td>0.552</td>
<td>28.90</td>
<td>87.5</td>
</tr>
<tr>
<td>1.358</td>
<td>0.761</td>
<td>0.550</td>
<td>28.83</td>
<td>102.5</td>
</tr>
<tr>
<td>1.854</td>
<td>1.033</td>
<td>0.547</td>
<td>28.69</td>
<td>140</td>
</tr>
<tr>
<td>2.152</td>
<td>1.196</td>
<td>0.546</td>
<td>28.63</td>
<td>162.5</td>
</tr>
<tr>
<td>2.483</td>
<td>1.377</td>
<td>0.544</td>
<td>28.57</td>
<td>187.5</td>
</tr>
<tr>
<td>4.635</td>
<td>2.541</td>
<td>0.538</td>
<td>28.30</td>
<td>350</td>
</tr>
</tbody>
</table>
APPENDIX C

Affliction Creek Modelling Summary Report
Investigation of Large Scale Mountain Slope Deformation: Affliction Creek Rock Slope -

Numerical Modelling Analysis Using FLAC and UDEC

By: Thomas W.G. Stewart
83574814
Course: Geology 595A - Directed Study
Prof: Dr. Michael J. Bovis
Dr. K. Wayne Savigny

Date: 24 October, 1995
File: Geol595A/Affliction Creek-1.mem
Synopsis

Detailed modelling studies of the Affliction Creek rock slope indicate a gravitationally driven failure mechanism that is structurally controlled by a steep, inwardly dipping joint set. Toppling failure along this primary joint set dominates the deformation mechanism, and explains the development of the extensive network of linear features that are a manifestation of the slope movements. The modelling studies show excellent agreement with the field evidence and long term slope movement data, which are used to calibrate and evaluate the modelling.

Modelling highlights the sensitivity to cyclic variations in effective stress, accounted for by groundwater fluctuations. These fluctuating stress conditions drive a cyclical deformation process that resembles a long term creep process. A viscous creep model is not considered appropriate since the changing stress conditions nullify the definition of a creep process.

Metastable conditions in the model appear compatible with field conditions supported by long term movement data. Strong seismic shaking would generate significant deformation in the model, but is not a required catalyst to initiate slope movements, or the formation of the mountainslope linears.

Section 1.0 - Introduction

Rock slope deformation at the Affliction Creek site, in the Coast Mountains of southwest British Columbia, has been investigated and reported by Bovis (1982, 1990, 1995). This rock slope is dominated by a collection of surficial linear features that attest to a large degree of slope mass movement. Bovis (1982) highlighted the parallelism of the structurally controlled linear features to the slope contours and proposed a gravitational origin for the features. Long-term monitoring of the slope, dating from 1979, has provided an important insight into the nature of the slope deformation.

Numerous sites in mountainous areas of the world are characterized by similar landforms, as reported by Zischinsky, (1966), Beck (1968), Tabor (1971), Nemcok (1972), Radbruch-Hall et al. (1976), and Beget (1985). Recent description of these slope deformation features from a broad region in the British Columbia cordillera, as evidence for an important and widespread slope movement phenomenon (Bovis and Evans, 1995) emphasizes the importance of developing a clearer understanding of this slow but large scale process. The computer modelling described in this report is an attempt to gain such an understanding.
Few investigations of mountain slope linears have included quantitative evidence, such as slope movement data, to support the generally accepted view that the genesis of these features is gravitationally driven. Further, no published report of large scale mountain slope deformation has incorporated both numerical modelling and slope movement data in conjunction with the geological (structure and lithology) and geomorphological (slope deformation features) site characteristics.

This report presents the results of numerical modelling of the Affliction Creek rock slope which were used to investigate the stress-strain (deformation) relationship (deformation) in the slope. The modelling work incorporates the extensive site information compiled and presented by Bovis (1982,1990), and in particular, makes use of the long-term slope movement data. The magnitude and pattern of the slope movement data are instrumental in evaluating the ability of the model to capture the significant behaviour of the slope. Additionally, the morphology and distribution of the slope deformation features provides an important means of calibrating the model response. The intent of the modelling extends beyond a simulation of the slope movements, and proposes to address a number of the fundamental characteristics of the slope movements, including the following:

i. An investigation of the potential driving mechanism(s) for the slope movements (ie. gravitational, glacial unloading, and seismic shaking forces).
ii. An examination of the failure mechanism(s) characterizing the slope movements.
iii. The sensitivity of the slope to various loading conditions, such as glacial debuttressing, groundwater fluctuations, and earthquakes.
iv. The long-term evolution, or progression of the slope movements, with particular focus on the attainment of a stable slope configuration or development of an inherently unstable slope prone to more rapid or catastrophic failure.

The modelling work employs two distinct numerical techniques; a discontinuum approach using the UDEC computer code, and the continuum FLAC code to undertake an analysis of the rock slope. Although the two methods differ in their numerical formation and consideration of material continuity, it will be shown that the methods they can be employed in a complimentary fashion. A comparison of their results provides a means of evaluating the effectiveness of the modelling techniques.
Section 2.0 - Numerical Modelling

2.1 - Modelling Techniques

The modelling work has incorporated two distinctly different methods of analysis. One approach has been to model the slope as a discontinuum material, representing the structural fabric of the rock mass with a finite number of discrete discontinuities (UDEC). The second approach has involved a continuum approach that incorporates a strength anisotropy to capture the importance of the structural fabric in the overall slope behaviour. Both models have the ability to represent the intact rock and the structural fabric, such as joints or shear zones, independently. This is a significant feature as the intact rock and discontinuities can be expected to behave quite differently. More specifically, the intact rock in competent igneous lithologies, comprising much of the Coast Mountain range, is significantly stronger than joint or shear features within the rock mass. Consequently, the rock slope deformation behaviour can be expected to show clear structural control. This fact is supported by the results of both modelling approaches.

Similarities between the models include a two-dimensional plane strain analysis; an explicit, finite difference time-stepping scheme to solve the equations of motion; a lagrangian framework that enables the calculation of large strains (displacements); the incorporation of non-linear material behaviour; and the ability to solve static or dynamic loading cases. In general, both codes employ Newton's second law (F=ma) and a material constitutive relation (stress-strain relationship) to solve for the particular loading conditions defined by the slope geometry and specified state.

Both models have used a Mohr-Coulomb constitutive relationship to capture the material behaviour. The stress-strain relationship uses effective stresses in the numerical formulation, which provides for the consideration of groundwater in the rock slope. As suggested by Bovis (1990) the effects of groundwater conditions in the rock slope have significant effect on the overall behaviour, which will be demonstrated by the modelling results.

A presentation of the numerical formulation of the two computer codes is beyond the scope of this presentation. Details of the UDEC code can be found in the manuals accompanying UDEC version 1.83, and the corresponding manuals for FLAC version 3.22, both available through the Itasca Consulting Group, Inc., of Minneapolis, Minnesota (see reference list).
2.2 - General Model Approach

A similar modelling methodology was used in both the Discontinuum and Continuum analyses. The conceptual idea behind the modelling methodology was to recreate a state most representative of the in situ slope conditions. This involved a five step process; the first four steps of which are termed the "calibration" phase and include the formation of the slope through to the current conditions. The "predictive" phase that follows investigates ongoing and future conditions such as rainfall/groundwater fluctuations and earthquake loading. It could be argued that groundwater fluctuations and earthquake events have occurred in the past, as indeed they most certainly have, however, attempting to reconstruct these conditions simply introduces more uncertainty and complexity into the model. For example, a question could arise as to what would be the effect of an earthquake at an earlier stage of the model. Many variables could have changed significantly since that time such as the slope geometry, the groundwater conditions and the position of the ice. Hypothetical modelling can be undertaken to investigate these issues, however, it is more effective to carry out predictive modelling work for the slope under the conditions that we have the most complete knowledge, which is the current state. Based on this approach the general modelling methodology followed is outlined below.

2.2.1. Initial Block Consolidation

In this step the slope geometry, boundary and initial conditions were specified (Fig. 1a,b). A large block with representative material was consolidated under a gravitational stress field to establish the initial stress distribution. Horizontal stresses (Sxx) were set equal to the out of plane stresses (Szz) at 50% of the vertical stresses (Syy). Higher values of horizontal stress were not considered to be appropriate in the tectonic setting of the Affliction Creek area.

Rock mass properties were based on the Hoek-Brown General Rock Mass Classification (1995). The method provides strength values for use in the Mohr-Coulomb yield function, and deformation moduli for use in the material flow rule. The method is an empirically derived technique for establishing properties of jointed rock masses, and is based on extensive field experience with wide acceptance in applied rock mechanics applications. Table 1 provides a range of the rock mass properties used in the analyses. Laboratory and field experience has shown these properties to be a function of the relevant effective stress level. The range of values used is an attempt to capture the non-linear Mohr-Coulomb and stress-strain behaviour characteristic of jointed rock masses.
Table 1: Rock Mass Properties

<table>
<thead>
<tr>
<th>Rock Mass Property</th>
<th>Initial (fresh)</th>
<th>Weathered</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction (degrees)</td>
<td>54 - 62</td>
<td>50 - 54</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>2.4 - 4.5</td>
<td>0.03 - 1.53</td>
</tr>
<tr>
<td>Tension (MPa)</td>
<td>0.5 - 1.0</td>
<td>0.0 - 0.2</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>2700</td>
<td>2700</td>
</tr>
<tr>
<td>Dilation Angle (degrees)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Bulk Modulus (MPa)</td>
<td>12250 - 25100</td>
<td>700 - 7500</td>
</tr>
<tr>
<td>Shear Modulus (MPa)</td>
<td>8400 - 17500</td>
<td>500 - 5150</td>
</tr>
</tbody>
</table>

Rock joint properties were established using the method proposed by Barton (1987). In a similar fashion to that employed for the rock mass, a range of joint strength properties were input to the model, as shown in Table 2:

Table 2: Rock Joint Shear Strength Properties

<table>
<thead>
<tr>
<th>Joint Strength Properties</th>
<th>Initial (fresh)</th>
<th>Weathered</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Angle</td>
<td>36 - 38</td>
<td>31 - 33.5</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>1.0 - 1.2</td>
<td>0.0 - 0.05</td>
</tr>
<tr>
<td>Tension (MPa)</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Boundary conditions were established such that boundary location did not influence the area of interest in the rock slope. The model boundaries were assumed to represent a fixed position such that movements perpendicular to the boundaries were not possible, representing the field conditions.

The block was considered initially saturated providing an effective stress analysis under hydrostatic conditions.

The models were then run to a steady state condition such that model stresses, displacements and unbalanced forces reflected the development of a new equilibrium state. This condition was checked through a collection of monitoring points established throughout the model (Figs. 2a,b,c).
2.2.2. - Slope Formation

This model stage involved the removal of overburden material in a progressive manner to the approximate profile of the present day slope. At this stage rock material still occupied the extent of the future glacially (and fluvially) incised Affliction Creek valley (Fig. 3a,b). Again, the models were run to an equilibrium condition, allowing a new stress distribution to be established according to the new model geometry and conditions. During the slope formation the phreatic surface was lowered in conjunction with the ground surface, maintaining a profile some 20 to 25 metres below the slope profile (Fig. 3b).

Both models used an identical cross section profile to define the slope geometry, based on the site map of Affliction Creek compiled by Dr. M.J. Bovis

2.2.3. - Glacier Formation and Downwasting

This stage modelled the formation of the Affliction glacier, to elevation 1675 metres, representing a condition during retreat of the main Affliction valley glacier towards the later stages of Fraser glaciation (Fig. 4). From an equilibrium condition the glacial level was progressively dropped to model the conditions during glacial downwasting. Movements, stresses and the distribution of surficial distortion to the model grid were monitored throughout this sequence to capture the slope behaviour under these changing conditions. Results of this stage are presented in Section 3.0

2.2.4. - Rock Mass Weathering

Two fundamental changes to the rock mass were considered to take place during evolution of the rock slope. With the unloading of overburden material (slope/valley formation) and glacial downwasting, an anticipated rebound response was observed with the diminishing overburden stresses. The effect of combined elastic and plastic rebound deformation would be expected to change the nature of the rock mass, and in particular, the conditions of the joint surfaces. Movement would be accompanied by a shearing of asperities along the joint surfaces, resulting in a reduced shearing resistance. The second component of rock mass weathering would be developed through the combined exposure to physical and chemical weathering agents. Rock slope movements associated with unloading would increase the exposure of the rock mass to weathering thereby altering the fresh rock mass conditions to varying degrees of weathered material. Obviously this weathering is a positive feedback process; whereby movement allows for increased penetration of weathering agents that in turn facilitate increased movement.

The overall weathering effect was seen as a "strain softening" response in the weathered portion of the rock slope. Figure 5 illustrates the reduced resistance to shear stress
(ie. reduced shear strength) as deformation occurs. This plot represents the material response in a zone located near the slope surface where weathering effects are considered important. This strain softening behaviour is analogous to the "brittle" material behaviour that is considered to accompany the development of strain in rock masses.

The initial distribution of this weathering profile was based on the extent of the area of slope movement developed during glacial downwasting (Fig. 6). During the modelling it was tempting to integrate the weathering response with the unloading (glacial debuttressing) sequence, however, confusing model response arises when a complexity of disturbances are introduced to the model simultaneously. For that reason, the effects were separated. The weathering profile was progressively introduced to the rock mass, which is believed to represent the physical process that has taken place during the evolution of the slope. It is believed that much of the weathering and strength reduction would occur in the stage corresponding to the most significant slope movements, that is during and following the glacial downwasting.

The weathered rock mass properties are believed to affect the condition of the joints more significantly than the intact rock. Therefore, a larger strain softening response was introduced to the structure than intact material. The weathered rock properties were based on the Hoek-Brown general and Barton relationships as discussed above.

2.2.5. - Predictive Model Stages

At the completion of the weathering stage the rock slope was found to be in a metastable condition. This is acceptable and justified by the knowledge of current slope movements, and the "fresh" surficial activity of the slope. Both these observed field conditions reflect the state of incipient large scale instability.

Conditions were considered to be representative of the present state of the rock slope, based on all available information. These model conditions were then subjected to the predictive modelling which involved the application of groundwater fluctuations and seismic loading to the slope.

The groundwater conditions were based on the profile presented by Bovis (1990) which appears to be the best estimate based on limited field information. Variations from this initial groundwater condition have been investigated in the modelling work. These fluctuating groundwater conditions are believed to represent the conditions prevalent during periods of significant infiltration associated with precipitation, snowmelt or a combination of the two. The model treated these conditions through a rise in overall phreatic surface, and the application of transient seepage forces acting in the direction of hydraulic gradient, but controlled by the structural fabric.
Seismic loading was carried out by applying a recorded acceleration time history from the 1994 Northridge earthquake event. This was considered the most representative earthquake record and was selected with the following points in consideration:

i. The acceleration time history was recorded at a station located on similar bedrock conditions (i.e. not from a site located on softer rock or soil foundation conditions where amplification of ground motions has been observed to be a significant factor).

ii. The peak ground motion recorded at the site was approximately 0.3 g, considered an appropriate peak ground acceleration for the Affliction Creek location. Consequently, the earthquake record did not require scaling to provide sufficient seismic loading.

iii. The earthquake event was of a magnitude (Richter M6.5) considered possible/probable for this site.

2.3 - Discontinuum Analyses

The numerical analyses began by using the "Discontinuum" UDEC code to investigate the importance of the geologic structure on the slope behaviour and deformation mechanics. In particular the two predominant joint sets were modelled to determine if one joint set was more significant in the control of the rock slope behaviour. The results were compared with respect to the observed slope movement behaviour and the surficial morphology.

In modelling the structural fabric of the rock slope a number of key assumptions were made based on field observations. Of primary importance was the consideration of the joint set orientations. The "disturbed" state of the structural fabric has been observed and measured along much of the slope surface. The "undisturbed" in situ structure, representing initial conditions prior to slope movement, has to be distinguished from the disturbed structure for input into the initial model state.

Evaluation of the undisturbed structural fabric requires some judgement on the nature of the slope deformation mechanics. If block sliding along the downslope dipping joint set was the dominant failure mechanism then there would be minimal disturbance to the joint orientations. Conversely, if toppling failure was predominant, then rotation of the structure would be manifested in outcrop. Direct observation of the bent and rotated joint surfaces, particularly on the downslope flank of several prominent antislope scarps, strongly supports the fact that toppling mechanisms play a significant factor in the slope movements, at least in the near surface movements. Joints dipping into the slope in the order of 50 to 55 degrees represent the disturbed joint fabric. It was estimated that the undisturbed fabric had undergone rotation in the order of 10 to 20 degrees. This would correspondingly transform the dip angle of the second joint set into a shallower orientation of 35 to 45 degrees downslope.
Mapping of the structural fabric has indicated that the inwardly dipping set is pervasive (over metres to tens of metres). The downslope dipping set is considerably less continuous, forming a cross-cutting relationship with the primary set. This combination of structure provided three possible scenarios in the modelling of the joint structure:

Case I: Modelling of a single joint structure dipping into the slope (using an average dip of 70 degrees to model the undisturbed fabric). In this case toppling of the block columns was observed during the glacial downwasting, slope weathering and fluctuating groundwater conditions.

Case II: The joint structure was modelled as a single set dipping 35 to 45 degrees downslope ("undisturbed" fabric). This case makes the very liberal assumption that the downslope dipping set is continuous at the scale of the slope. In this case large scale block sliding is possible under various combinations of joint shear strength and groundwater levels. However, the pattern of model slope movements does not reflect the monitored behaviour. Most importantly there is essentially no internal deformation of the moving mass and no development of surficial landforms within the body of the moving mass. This contradicted the observed conditions at Affliction Creek, therefore, this model was not developed further.

Case III: The structural fabric was represented by both joint sets, with the inwardly dipping set continuous and the downslope joint set forming a cross-cutting, but less continuous pattern.

Cases I and III yielded the most representative development of slope movement patterns, suggesting that the failure mechanism is predominantly of a toppling nature. Figure 7a illustrates the structure along which shear movements occurred outlining an antithetic sense of displacement, confirming the toppling mechanism. The clear development of a tension crack zone, with the formation of downslope antislope scarps is developed during this slope deformation.

2.4 - Continuum Analysis (FLAC)

The UDEC modelling highlighted the important structural control of rock slope movements by the inwardly dipping joint set. This fact was used to establish the anisotropic strength fabric used in the continuum analysis. The rock mass material in the FLAC model
was modelled as Mohr-Coulomb non-linear material with a ubiquitous structure representing the inwardly dipping joints. This provided the capacity to model the intact material and primary joint structure independently.

Due to the faster numerical formulation and solution (several hours versus several days) the FLAC code was chosen to model the predictive phase of the analysis.

The model grid was carefully constructed to provide the greatest model accuracy in the zone of interest corresponding to the movement zone. The grid geometry was aligned to allow the following capabilities:

i) The rows in the model grid were oriented parallel to the slope surface to facilitate the most representative slope formation and variation in glacial level. This set-up also yielded the most realistic weathering profile in the rock slope.

ii) The columns in the model grid were aligned in a vertical orientation to provide a clear representation of the potential rotation of the material during deformation.

The behaviour of the model was continuously monitored throughout the analysis, in terms of model stresses, displacements and forces to ensure that the results were realistic (ie. physically representative of the geometry and material conditions). This model monitoring provided the means to interpret the results and allowed a comparison with the observed field conditions.

Section 3.0 - Modelling Results

The key points of the modelling work will be summarized below. The modelling work was successful in answering the questions/objectives that were set forth in the introduction. Most importantly, the modelling work was able to provide good agreement with the observed field conditions and monitored slope movements. The modelling revealed that progressive slope movements are developed under gravitational stresses, without the need to appeal to seismic or tectonic forces to drive deformation. Furthermore, the observed surficial landforms that are often qualitatively related to the deformation process have received quantitative support from the model stress-strain analyses. This provides significant support to the argument that the linear features seen on numerous mountain slopes are a manifestation of a progressive, gravitationally driven process. Strong seismic shaking will no doubt enhance the slope deformation process, and in some cases initiate complete slope failure, but it is not necessarily a required input to drive the deformation process and the genesis of the anomalous linear features.

The two available means by which the model behaviour can be evaluated is through a comparison with the monitored slope movements and with the observed distribution of
surficial slope deformation features.

3.1 - Comparison of model with monitored slope movements

The distribution of slope movements in the UDEC and FLAC models can be seen in Figures 8 and 9, respectively. Figure 10 shows a slope cross section illustrating the observed surficial landforms and slope movement vectors measured during the 1982 to 1986 period (from Bovis, 1990). The displacement patterns in both Figures 8 and 9 indicate a zone of deformation compatible with the region of the Affliction Creek slope that has undergone movement, based on the distribution of monitored slope movements.

A comparison of the model slope movement vectors (Fig. 11) with the monitored slope movements (Fig. 10) shows a striking similarity in overall pattern. This pattern reveals the largest movements at the crest of the steep slope section above the Affliction Glacier. Movement magnitude progressively decreases in the upslope direction towards the level ground at elevation 1700 metres. The movement vectors for the model and slope also show a similar pattern of diminutive but more vertical movements in the upper (back) areas of the deforming zone.

Another relevant comparison can be seen in the magnitude of slope movements. Figure 11 indicates maximum movements along the slope surface in the order of 30 metres. Actual slope movements based on surficial deformation features would be in the order of tens of metres depending on the location of cross section. This represents a good agreement of movement magnitude, pattern and distribution, indicating that the model is capturing the important characteristics of the actual slope behaviour.

3.2 - Comparison of model with observed slope features

The UDEC model highlights the formation of tension cracks and antislope scarps which develop as a result of the antithetic movements (Fig. 7). The FLAC model shows the formation of similar features (Fig. 12), although the continuum formulation presents a more uniform distribution based on the requirement of continuity within the model material.

The rotation of the structure (bending of the joint surfaces) is a condition clearly evident in the outcrop exposure of several of the large antislope scarps. Figures 12a and b illustrate that the model captured this structural rotation. Figure 12c shows the distribution of structural rotation from the initial value of 70 degrees. The range of 40 to 65 degrees shows reasonable agreement with the observed joint surfaces.

The "bulging" of the lower slope seen in Figures 12a-c reflects the dilational nature of the slope movement. The continuum analysis illustrates the distortion of the material, and although the model "material" does not progressively "ravel", it indicates the process of active
rockfall that would accompany this style of slope deformation. Field evidence supports this process in the form of active talus cones at the base of the slope (Bovis, 1982).

Section 4.0 - Discussion

A comparison of the model behaviour with the measured and observed slope behaviour indicates the capability of the modelling process to capture the salient features of the material behaviour in the Affliction Creek rock slope. The calibrated behaviour of the model and slope allows the model to be used to investigate phenomena not directly observable in the field and to predict response to future events such as long term groundwater fluctuations and earthquake loading.

Based on an evaluation of the surface movement vectors Bovis (1990) projected a depth of deformation in the order of 45 to 75 metres. Acknowledged as a simplistic ("Brunton-and-boot-kicking-approach") analysis, this estimate shows remarkable agreement with the results obtained from the UDEC and FLAC models as outlined below.

<table>
<thead>
<tr>
<th>Estimate</th>
<th>Estimated depth of deformation (metres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bovis (1990)</td>
<td>45 - 75</td>
</tr>
<tr>
<td>UDEC</td>
<td>30 - 80</td>
</tr>
<tr>
<td>FLAC</td>
<td>30 - 55</td>
</tr>
</tbody>
</table>

The style of material behaviour throughout the deformation zone is revealed in Figure 12a,b. Both models indicate that toppling behaviour, facilitated by slip along inwardly dipping joints, is the predominant deformation mechanism. Although there is no subsurface data from the site to confirm this hypothesis, the large scale deformation throughout the surface area strongly suggests that this toppling movement, with significant internal deformation prevails in the rock slope.

If sliding was the dominant failure mechanism then surficial morphology would favour the development of a significant headscarp structure, without the development of significant disruption of the slope surface in the central area of the movement zone. Neither the model results nor the field evidence support a sliding dominant failure mechanism.

The analyses have indicated a metastable slope condition, sensitive to changes in loading (stress) conditions. An interesting characteristic of the model behaviour is the capability of the slope to adjust to changing stress conditions, with the development of
significant strains, but without reaching a condition of steady state flow indicative of more rapid failure behaviour. Simplistically, the slope has the ability to "catch" itself in response to the movements driven by changing stress conditions. This is an important feature that is believed to represent actual conditions within the rock slope. Fluctuating groundwater levels and the transient seepage forces associated with periods of high infiltration would be sufficient to generate these changing effective stress conditions. The models display sensitivity to these varying conditions (Fig. 13). This figure illustrates the cyclic deformations that result from the combination of a 2 to 8 metre head increase in groundwater pressures and the application of seepage forces in the upper regions of the rock slope due to infiltration from significant precipitation or snowmelt. The seepage forces are based on a 2 to 5 head of water that would dissipate as groundwater levels reacted to the change in flow conditions. As is evident from the plot of movement versus model "time", the changing stress conditions initiate plastic deformation that attenuates as the disturbing stress is redistributed throughout the model. A new balance is achieved at the cost of cumulative displacement. Deformations in the range of several millimetres to centimetres accompany each cycle; the accumulation of several cycles represents a "typical" season that would yield movements of several centimetres to fractions of a metre. These magnitudes compare favourably with the measured movement rates that indicate relative movements in the order of 0.1 to 0.6 metres (10 to 60 centimetres) over the period of 3 to 4 years (the period of time between slope surveying). Viewed over a long term these cyclic deformations appear as slow "creep" type behaviour, which is often the term used to describe the slow deformation of large rock slopes, many of which display these linear surficial landforms.

The question of earthquake loading has been addressed in the modelling exercise. The response of the slope to a significant event (described in Section 2.2.5) is shown in Figure 16. Large displacements in the order of 1 to 8 metres result from the loading. The pattern of the displacement curves indicates movement initiating at the surface, where actual flow (steady state deformation) of the material occurs. Movements are progressively smaller at depth and show a stabilizing trend. The large surface movements and steady state nature of the deformation indicate considerable yield occurring in the upper 10 metres of the slope. The large movements and steady state flow indicate this material would most probably be transported rapidly down to the valley bottom through a combination of sliding, falling and possibly a larger flow type process such as rockslide avalanche. This could lead to the development of more significant flow type phenomena, particularly if ice or water from the valley bottom were entrained into the moving material with important implications for downstream facilities in the Lillooet River valley (this hypothetical scenario is clearly beyond the scope of this presentation).
It must be recognized that there is a large degree of uncertainty in the application of seismic loading. The potential variation in earthquake magnitude, frequency, duration of loading, and number of significant cycles of strong shaking reveals the wide range of earthquake responses that could be modelled. Despite this uncertainty, a dynamic analysis is a significant improvement on more simplistic pseudo-static analyses that apply overly conservative loading conditions. The important point to recognize in the dynamic modelling work is the sensitivity of the slope to significant deformations, including the potential for steady state flow behaviour.

5.0 - Conclusions

The modelling analyses have displayed the potential for the Affliction Creek rock slope to undergo progressive, gravitationally driven deformation. Model results show the deformation to be governed by a predominantly toppling type failure mechanism, structurally controlled by the primary joint set that dips steeply into the slope. The modelling shows the surficial linear features to be genetically related to the slope failure mechanism. Excellent agreement with the observed field conditions and compatibility with the monitored slope movements were achieved, thereby providing support for the gravitational hypothesis proposed by Bovis (1990).

A metastable condition was found to exist in the model displaying sensitivity to variations in effective stress levels. Groundwater fluctuations and associated seepage forces were shown to provide these stress variations, and yielded cyclic movements resembling a longer term creep behaviour.

Strong seismic shaking was found to generate significant model slope movements, but not to the point of large scale slope collapse. Seismic forces were not required to initiate or drive subsequent slope movements, nor were they an essential ingredient in the formation of slope movement landforms.

6.0 - Acknowledgements

The permission of B. C. Hydro to use the FLAC computer code is gratefully acknowledged. Similarly, the permission of Dr. K.W. Savigny of the Department of Geology, University of British Columbia, to use the UDEC code provided the opportunity to carry out the modelling studies. The modelling work received helpful input from Dr. B.D. Ripley and T.E. Little of B.C. Hydro, and Dr. K.W. Savigny.

Most importantly, I am grateful to Dr. M.J. Bovis for providing the opportunity to participate in the long term study, the chance to visit the Affliction Creek site and his enthusiastic support in the study of this interesting phenomenon.
Figure 1a: Initial UDEC block.
Figure 1b: Initial FLAC grid.
Job Title: Affliction Creek Rock Slope Deformation Analysis - File: Aff-6A.
From File: aff-6a.sav

FLAC 3.22
Step 3000

HISTORY PLOT
Y-axis: Max. unbal. force
X-axis: Number of steps

Figure 2a: Plot of unbalanced force during consolidation of FLAC model. Note the attenuation of the unbalanced force indicating an equilibrium condition at the completion of consolidation.
Figure 2b: Plot of horizontal and vertical displacements at the model centre during consolidation. The levelling off of the vertical indicates an equilibrium state.
Job Title: Affliction Creek Rock Slope Deformation Analysis - File: Aff-6A.
From File: aff-6a.sav

FLAC 3.22

Step 3000

HISTORY PLOT
Y-axis:
Ave. SXY (36, 20)
X-axis:
Number of steps

Figure 2c: Plot of shear stress at centre of model. The attainment of zero shear stress is predicted in the block at equilibrium.
Figure 3a: UDEC grid during slope formation.
Figure 3b: FLAC grid during slope formation, showing groundwater level.
Figure 5a: Plot of principal and shear stresses in a near surface zone. The reduction of the shear stress and the maximum principal stress (compressive stresses are negative, following the structural convention) reflect the weakening of the material during weathering.
Figure 5b: Plot of shear stress versus displacement illustrating the strain softening behaviour.
Figure 6a: Displacement contours indicate the zone and magnitude of movement associated with glacial downwasting. This zone was used to define the extent on initial weathering.
Figure 6b: Displacement vectors illustrating movement due to glacial downwasting.
Figure 7: Block rotation (toppling) leading to the formation of tension cracks and a series of antislip scarps.
Figure 8: Area of deformation in the UDEC analysis. Note the structural features along which the movements are occurring, indicating a toppling type failure mechanism.
Figure 9a: Deformation zone in the FLAC model illustrated by the displacement contours (see detail Fig. 9b).
Job Title: Affliction Creek Rock Slope Deformation Analysis - File: Aff-3WA
From File: aff-3wa.sav

FLAC 3.22

Step 48000
X-displacement contours
Contour interval = 1.00E-01
Minimum: 1.00E-01
Maximum: 1.00E+00
Boundary plot

Figure 9b: Detail of deformation zone in FLAC model. See Figure 13 for illustration of material behaviour.
Figure 10: Cross section of Affliction Creek slope showing measured displacement vectors and slope deformation features. (after Bovis, 1990).
Job Title: Affliction Creek Rock Slope Deformation Analysis - File: Aff-3WB
From File: aff-3wc.sav

FLAC 3.22

Step 50000
Displacement vectors
Max Vector = 2.835E+01 (m)

Figure 11: Plot of displacement vectors from FLAC model.
Job Title: Affliction Creek Rock Slope Deformation Analysis - File: Aff-3WB
From File: aff-3wb.sav

FLAC 3.22

Step 49000
Exaggerated Grid Distortion
Magnification = 5.000E+00
Max Disp = 8.929E+00

Figure 12a: Deformation features in the FLAC model.
Figure 12b: Formation of "antislope scarps" in FLAC model as a result of progressive gravitational deformation.
Figure 12c: FLAC model showing the rotation of the structural fabric in the deforming zone. Values are in degrees compared to the initial joint angles of 70 degrees.
Figure 13a: UDEC model showing the joint surfaces along which movements are occurring. This outlines a zone in which toppling failure controls the material behaviour.
Job Title: Affliction Creek Rock Slope Deformation Analysis - File: Aff-3WB
From File: aff-3wc.sav

FLAC 3.22

Step 50000
Plasticity Indicator
* at yield
X elastic, at yield in past
a uniaxial tension failure
+ yield & tension failure
^ slip along ubiq. joints
. ubiq. joints slip in past
v tens. fail. ubiq. joints

Boundary plot

Figure 13b: FLAC model showing the material behaviour in the deforming zone. Movements are accomodated by slip along the ubiquitous joints.
Figure 14: Plot of movements in the FLAC model in response to the application of cyclic groundwater fluctuations and transient seepage forces. Note the longterm trend of this behaviour in analogy with "creep" phenomena.
Figure 15: Acceleration time history of applied earthquake.
Figure 16a: FLAC model grid during earthquake loading, illustrating displacement primarily occurring in the upper zone of the slope.
Figure 16b: FLAC model displacement vectors in response to earthquake loading.
Job Title: Affliction Creek Rock Slope - Dynamic Analysis - File: Aff-EQ6
From File: anp-1.dat

Figure 16c: FLAC model displacement profile at crest of steep slope section. Note the attenuation of movements below 10 metre depth, and the steady state "flow" of the surficial zone.
APPENDIX A

HOEK-BROWN ROCK MASS CLASSIFICATION

**Project:** Affliction Creek Rock Slope  
**Rock Mass:** Quartz Monzanite  
**Structure:** Blocky  
**Surface Condition:** Good - Slightly weathered

<table>
<thead>
<tr>
<th>Input</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>55</td>
</tr>
<tr>
<td>sigc (MPa)</td>
<td>75</td>
</tr>
<tr>
<td>mi</td>
<td>28</td>
</tr>
<tr>
<td>Z (m)</td>
<td>50</td>
</tr>
<tr>
<td>Dr (MPa/m)</td>
<td>0.0265</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sigm</td>
<td>1.325</td>
</tr>
<tr>
<td>Sigt</td>
<td>-0.090</td>
</tr>
<tr>
<td>mb</td>
<td>5.613</td>
</tr>
<tr>
<td>s</td>
<td>0.0067</td>
</tr>
<tr>
<td>a</td>
<td>0.50</td>
</tr>
<tr>
<td>E</td>
<td>11549</td>
</tr>
<tr>
<td>phi</td>
<td>65.95</td>
</tr>
<tr>
<td>coh</td>
<td>0.522</td>
</tr>
<tr>
<td>sigcm</td>
<td>4.905</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>sig3</th>
<th>sig1</th>
<th>ds1ds3</th>
<th>sign</th>
<th>tau</th>
<th>signtau</th>
<th>signsq</th>
<th>lin. plot</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.080</td>
<td>2.008</td>
<td>101.830</td>
<td>-0.059</td>
<td>0.205</td>
<td>-0.012</td>
<td>0.004</td>
<td>0.389</td>
</tr>
<tr>
<td>-0.069</td>
<td>2.883</td>
<td>72.298</td>
<td>-0.029</td>
<td>0.342</td>
<td>-0.010</td>
<td>0.001</td>
<td>0.457</td>
</tr>
<tr>
<td>-0.049</td>
<td>4.126</td>
<td>51.415</td>
<td>0.031</td>
<td>0.571</td>
<td>0.018</td>
<td>0.001</td>
<td>0.592</td>
</tr>
<tr>
<td>-0.007</td>
<td>5.897</td>
<td>36.649</td>
<td>0.150</td>
<td>0.949</td>
<td>0.142</td>
<td>0.022</td>
<td>0.858</td>
</tr>
<tr>
<td>0.076</td>
<td>8.426</td>
<td>26.208</td>
<td>0.382</td>
<td>1.571</td>
<td>0.601</td>
<td>0.146</td>
<td>1.380</td>
</tr>
<tr>
<td>0.241</td>
<td>12.050</td>
<td>18.824</td>
<td>0.837</td>
<td>2.584</td>
<td>2.163</td>
<td>0.700</td>
<td>2.398</td>
</tr>
<tr>
<td>0.572</td>
<td>17.272</td>
<td>13.604</td>
<td>1.716</td>
<td>4.218</td>
<td>7.238</td>
<td>2.945</td>
<td>4.367</td>
</tr>
</tbody>
</table>

**SUMS =**  
3.028   10.441  10.139  3.819

**Reference:**  
Estimating the strength and deformability of very poor quality rock masses.  
Draft Paper, 10 March 1995
APPENDIX B

BARTON JOINT STRENGTH PROPERTIES
BARTON SHEAR FAILURE CRITERION

Project: Affliction Creek Rock Slope

Details:

Input parameters

Basic friction angle (PHIB) - degrees 28
Joint Roughness Coefficient (JRC) 5
Joint Compressive Strength (JCS) 30
Minimum normal stress (SIGNMIN) 0.1

<table>
<thead>
<tr>
<th>Normal stress (SIGN) MPa</th>
<th>Shear strength (TAU) MPa</th>
<th>dTAU</th>
<th>Friction angle (PHI) deg</th>
<th>Cohesive strength (COH) MPa</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.066</td>
<td>0.058</td>
<td>0.811</td>
<td>39.04</td>
<td>0.004</td>
<td>5</td>
</tr>
<tr>
<td>0.199</td>
<td>0.161</td>
<td>0.744</td>
<td>36.65</td>
<td>0.012</td>
<td>15</td>
</tr>
<tr>
<td>0.331</td>
<td>0.257</td>
<td>0.715</td>
<td>35.55</td>
<td>0.020</td>
<td>25</td>
</tr>
<tr>
<td>0.464</td>
<td>0.350</td>
<td>0.696</td>
<td>34.82</td>
<td>0.028</td>
<td>35</td>
</tr>
<tr>
<td>0.596</td>
<td>0.441</td>
<td>0.682</td>
<td>34.28</td>
<td>0.035</td>
<td>45</td>
</tr>
<tr>
<td>0.728</td>
<td>0.530</td>
<td>0.671</td>
<td>33.84</td>
<td>0.042</td>
<td>55</td>
</tr>
<tr>
<td>0.860</td>
<td>0.618</td>
<td>0.681</td>
<td>33.48</td>
<td>0.049</td>
<td>65</td>
</tr>
<tr>
<td>0.993</td>
<td>0.706</td>
<td>0.654</td>
<td>33.17</td>
<td>0.057</td>
<td>75</td>
</tr>
<tr>
<td>1.126</td>
<td>0.792</td>
<td>0.647</td>
<td>32.90</td>
<td>0.064</td>
<td>85</td>
</tr>
<tr>
<td>1.258</td>
<td>0.877</td>
<td>0.641</td>
<td>32.66</td>
<td>0.071</td>
<td>95</td>
</tr>
<tr>
<td>1.987</td>
<td>1.335</td>
<td>0.617</td>
<td>31.67</td>
<td>0.109</td>
<td>150</td>
</tr>
<tr>
<td>3.311</td>
<td>2.133</td>
<td>0.590</td>
<td>30.56</td>
<td>0.178</td>
<td>250</td>
</tr>
<tr>
<td>6.622</td>
<td>4.023</td>
<td>0.556</td>
<td>29.06</td>
<td>0.344</td>
<td>500</td>
</tr>
<tr>
<td>13.224</td>
<td>7.567</td>
<td>0.522</td>
<td>27.56</td>
<td>0.665</td>
<td>1000</td>
</tr>
</tbody>
</table>

T.W. Stewart - 10/23/95
APPENDIX D

Modelling Details - Input Data Files

(FLAC, UDEC)
*Elemental shear test - File: UT-2.dat

title
Ubiquitous Joint Model - Material response to shear loading
*1 x 1 grid
grid 1,1
*Check the ubiquitous joint model
model ubiquitous
*Set the material properties E=30 GPa \( \mu=0.25 \)
prop d=2700 b=20 e9 s=12 e9 f=45 d=5 c=1 e6 t=1 e6 ja=0 jf=35 jc=1 e5 jt=0.5 e5
set pltc 1 e6
set pltf 45
set pltt 5 e6
*Set the initial stress conditions - Isostatic consolidation
*Set the stress level to an equivalent 10m depth level
ini sxx=-264870
ini syy=-264870
ini szz=-264870
pl sig1 hold
pl syy hold
pl sxx hold
pl sxy hold
pl plas hold
*Set the boundary conditions
fix x y j=1
fix x y j=2
ini xvel=1 e-7 i=1,2 j=2
ini xvel=-1 e-7 i=1,2 j=1
pl xv fill hold
pl xd fill hold
pl yv fill hold
pl yd fill hold
pl fix hold
*def output
while_stepping
a_sx=-sxx(1,1)
a_sd=-syy(1,1)+sxx(1,1)
a_str=xdisp(2,2)
a_ev=-ydisp(2,2)-xdisp(2,2)
a_tau=sxy(1,1)
end
hist unbal
his a_sx
his a_sd
his a_str
his a_ev
his a_tau
his sxy i=1 j=1
his xd i=2 j=2
his sig1 i=1 j=1
his sig2 i=1 j=1
his syy i=1 j=1
his sxx i=1 j=1
step 100
pl sxy fill hold
pl sig1 hold
pl sig2 hold
pl plas hold
pl xd fill hold
pl ssi hold
pl his 6 vs 4 hold
step 900
pl sxy fill hold
pl plas hold
pl xd fill hold
pl sigl fill hold
pl his 6 vs 4 hold
pl his 7 vs 8 hold
pl fail hold
pl ssi hold
*End of file
*Wahleach Rock Slope Deformation Analysis - File: WFDG-3A.dat

*Model geometry includes interfaces to separate fine and coarse sections of grid at depth.
*File written: 09-May-96 By: TWS
*File revised: 13-May-96 By: TWS (2nd revision)
*Allow dynamic loading
config dynamic
set dynamic off

Title
Wahleach Rock Slope Deformation Analysis - File: WFDG-3A

*Generate slope grid layout, with finest grid mesh in deforming zone of slope.
*Grid includes six interfaces; 4 sub-horizontal, 3 vertical.
*Initial stress distribution developed with hydrostatic groundwater present.
grid 127,53
model elastic
*Consolidate model with rock mass properties corresponding to 500 m depth
prop d=0.0027 b=32000 s=24000
model null
model null
model null
model null
model null

j=52,53
j=i,23
j=16,23
j=l,23

i=7
i = l, 6
i=36
i=37,47
i=48

model null i=49,124 j=9,23
model null i=8,123 j=24
model null i=8,123 j=31
model null i=93,123 j=25,30
model null i=125,127 j=34,53

model null i=49,124 j=9,23
model null i=125,127 j=34,53

*Zone 1 - Left of 1st vertical interface

<table>
<thead>
<tr>
<th>gen</th>
<th>-360, -1100</th>
<th>-360,250</th>
<th>0, 250</th>
<th>0,-1100</th>
<th>i=1,7</th>
<th>j=1,24</th>
<th>ratio 1.00, 0.99</th>
</tr>
</thead>
<tbody>
<tr>
<td>gen</td>
<td>-360,250</td>
<td>-360,500</td>
<td>0,500</td>
<td>0,250</td>
<td>i=1,7</td>
<td>j=24,30</td>
<td>ratio 1.00, 0.99</td>
</tr>
<tr>
<td>gen</td>
<td>-360,500</td>
<td>-360,1080</td>
<td>0,1140</td>
<td>0,500</td>
<td>i=1,7</td>
<td>j=30,46</td>
<td>ratio 1.00, 1.00</td>
</tr>
<tr>
<td>gen</td>
<td>-360,1080</td>
<td>-360,1340</td>
<td>0,1400</td>
<td>0,1140</td>
<td>i=1,7</td>
<td>j=46,52</td>
<td>ratio 1.00,1.02</td>
</tr>
</tbody>
</table>

*Zone 2 - below 1st interface

<table>
<thead>
<tr>
<th>gen</th>
<th>0, -1100</th>
<th>0,250</th>
<th>850,400</th>
<th>850,-1100</th>
<th>i=8,22</th>
<th>j=1,24</th>
<th>ratio 0.99, 0.98</th>
</tr>
</thead>
<tbody>
<tr>
<td>gen</td>
<td>850, -1100</td>
<td>850,400</td>
<td>1080,350</td>
<td>1100,-1100</td>
<td>i=22,26</td>
<td>j=1,24</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>1100, -1100</td>
<td>1080,350</td>
<td>1550,80</td>
<td>1550,-1100</td>
<td>i=26,33</td>
<td>j=1,24</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>1550, -1100</td>
<td>1550,80</td>
<td>1710,-20</td>
<td>1710,-1100</td>
<td>i=33,36</td>
<td>j=1,24</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>1710, -1100</td>
<td>1710,-20</td>
<td>1830,-100</td>
<td>1830,-1100</td>
<td>i=37,39</td>
<td>j=1,16</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>1830, -1100</td>
<td>1830,-100</td>
<td>1950,-180</td>
<td>1950,-1100</td>
<td>i=39,41</td>
<td>j=1,16</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>1950, -1100</td>
<td>1950,-180</td>
<td>2200,-340</td>
<td>2200,-1100</td>
<td>i=41,45</td>
<td>j=1,16</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>2200, -1100</td>
<td>2200,-340</td>
<td>2390,-460</td>
<td>2390,-1100</td>
<td>i=45,48</td>
<td>j=1,16</td>
<td>ratio 0.98, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>2390, -1100</td>
<td>2390,-460</td>
<td>2620,-610</td>
<td>2620,-1100</td>
<td>i=49,53</td>
<td>j=1,9</td>
<td>ratio 0.98, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>2620, -1100</td>
<td>2620,-610</td>
<td>2860,-760</td>
<td>2860,-1100</td>
<td>i=53,58</td>
<td>j=1,9</td>
<td>ratio 0.98, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>2860, -1100</td>
<td>2860,-760</td>
<td>3000,-850</td>
<td>3000,-1100</td>
<td>i=58,61</td>
<td>j=1,9</td>
<td>ratio 0.98, 0.98</td>
</tr>
</tbody>
</table>

*Zone 3 - Right side of 4th vertical interface

<table>
<thead>
<tr>
<th>gen</th>
<th>3000, -1100</th>
<th>3000,-850</th>
<th>3180,-850</th>
<th>3180,-1100</th>
<th>i=125,128</th>
<th>j=1,6</th>
<th>ratio 1.0, 0.98</th>
</tr>
</thead>
<tbody>
<tr>
<td>gen</td>
<td>3000,-850</td>
<td>3000,-610</td>
<td>3180,-610</td>
<td>3180,-850</td>
<td>i=125,128</td>
<td>j=6,12</td>
<td>ratio 1.0, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>3000,-610</td>
<td>3000,20</td>
<td>3180,20</td>
<td>3180,-610</td>
<td>i=125,128</td>
<td>j=12,28</td>
<td>ratio 1.0, 0.99</td>
</tr>
<tr>
<td>gen</td>
<td>3000,20</td>
<td>3000,500</td>
<td>3180,500</td>
<td>3180,20</td>
<td>i=125,128</td>
<td>j=28,34</td>
<td>ratio 1,1.04</td>
</tr>
</tbody>
</table>

*Zone 4 - above 1st interface and below 2nd interface

<table>
<thead>
<tr>
<th>gen</th>
<th>0.250</th>
<th>0,500</th>
<th>930,670</th>
<th>850,400</th>
<th>i=8,30</th>
<th>j=25,31</th>
<th>ratio 0.95, 0.98</th>
</tr>
</thead>
<tbody>
<tr>
<td>gen</td>
<td>850,400</td>
<td>930,670</td>
<td>1110,590</td>
<td>1080,350</td>
<td>i=30,38</td>
<td>j=25,31</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>1080,350</td>
<td>1110,590</td>
<td>1550,340</td>
<td>1550,80</td>
<td>i=38,56</td>
<td>j=25,31</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>1550,80</td>
<td>1550,340</td>
<td>1710,220</td>
<td>1710,-20</td>
<td>i=56,63</td>
<td>j=25,31</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>1710,-20</td>
<td>1710,220</td>
<td>1830,100</td>
<td>1830,-100</td>
<td>i=63,67</td>
<td>j=25,31</td>
<td>ratio 1.00, 0.98</td>
</tr>
<tr>
<td>gen</td>
<td>1830,-100</td>
<td>1830,100</td>
<td>1950,20</td>
<td>1950,-180</td>
<td>i=67,71</td>
<td>j=25,31</td>
<td>ratio 1.00, 0.98</td>
</tr>
</tbody>
</table>
gen 2620, -380 2620, -310 2620, -310 2620, -240 2620, -300 2620, -300 2620, -240 2620, 20 2620, 20 2620, 20 2860, -450 2860, -370 2860, -370 2860, -300 2860, -300 2860, -240 2860, -240 2860, 20 2860, 20 2860, 20

• Interface Surfaces - 6 interfaces in current grid
  interface 1 aside from 7,1 to 7,24 bside from 8,1 to 8,24
  interface 2 aside from 7,24 to 7,30 bside from 8,25 to 8,31
  interface 3 aside from 7,30 to 7,46 bside from 8,32 to 8,54
  interface 4 aside from 36,1 to 36,24 bside from 37,1 to 37,16
  interface 5 aside from 48,1 to 48,16 bside from 49,1 to 49,9
  interface 6 aside from 61,1 to 61,9 bside from 125,1 to 125,6
  interface 7 aside from 93,25 to 93,31 bside from 125,6 to 125,12
  interface 8 aside from 124,32 to 124,54 bside from 125,12 to 125,34
  interface 9 aside from 8,24 to 36,24 bside from 8,25 to 63,25
  interface 10 aside from 37,16 to 48,16 bside from 63,25 to 81,25
  interface 11 aside from 49,9 to 61,9 bside from 81,25 to 93,25
  interface 12 aside from 8,31 to 93,31 bside from 8,32 to 124,32

• Set boundary and initial conditions
  fix x i=1 j=1, 52
  fix x i=128 j=1, 34
  fix x y j=1

• Initial stress field based on horizontal stresses equivalent to 50% of vertical stresses (established from hydraulic jacking testing in S10).
  * Vertical stresses set to a depth of 1400 metres
  ini sxx=-33.108 var 11.92, 33.108 i=1,127 j=1,53
  ini syy=-66.217 var 23.83, 66.217 i=1,127 j=1,53
  ini szz=-33.108 var 11.92, 33.108 i=1,127 j=1,53
  * Set small strain logic for consolidation of initial stress distribution
  set small.
  set grav=9.81

* Set up history plots
  his unbal

* Monitor stresses and displacements at drill hole locations
  * DH89-65

* Stresses
  his sig1 i=26 j=47
  his sig2 i=26 j=47
  his szz i=26 j=47
  his sig1 i=26 j=46
his sig2 i=26 j=46
his szz i=26 j=46
his sig1 i=26 j=45
his sig2 i=26 j=45
his szz i=26 j=45

*Displacements
his xd i=27 j=48
his xd i=27 j=45
his yd i=27 j=48
his yd i=27 j=45

*DH90-S10

*Stresses
*Surface zone
his sig1 i=52 j=47
his sig2 i=52 j=47
his sxy i=52 j=47
his szz i=52 j=47

*40-50 metre depth
his sig1 i=52 j=43
his sig2 i=52 j=43
his sxy i=52 j=43
his szz i=52 j=43

*40-50 m depth
his sx x i=52 j=43
his syy i=52 j=43
his esxx i=52 j=43
his esyy i=52 j=43

*Displacements
his xd i=53 j=48
his xd i=53 j=45
his xd i=53 j=43
his xd i=53 j=41
his yd i=53 j=48
his yd i=53 j=45
his yd i=53 j=43
his yd i=53 j=41

*DH89-S7

*Stresses
his sig1 i=66 j=47
his sig2 i=66 j=47
his esyy i=66 j=47
his esxx i=66 j=47
his szz i=66 j=47
his angle i=66 j=47
his sig1 i=66 j=41
his sig2 i=66 j=41
his sig2 i=66 j=41
his sig1 i=66 j=40
his sig2 i=66 j=40
his szz i=66 j=40

*Monitor stress-strain behavior in deforming region of slope
*At DH89-S7

his sxy i=66 j=42
his sxy i=66 j=41
his sxy i=66 j=40

*Displacements
his xd i=67 j=48
his xd i=67 j=45
his xd i=67 j=42
his xd i=67 j=40
his xd i=67 j=39
his yd i=67 j=48
his yd i=67 j=45
his yd i=67 j=42
his yd i=67 j=40
his yd i=67 j=39
his xv i=67 j=48
his yv i=67 j=48

*DH89-S1

*Stresses

*Surface zone

his sig1 i=80 j=47
his sig2 i=80 j=47
his sxy i=80 j=47

*40-50 metre depth

his sig1 i=80 j=43
his sig2 i=80 j=43
his sxy i=80 j=43

*95-110 metre depth

his sig1 i=80 j=38
his sig2 i=80 j=38
his sxy i=80 j=38

*130-150 metre depth

his sig1 i=80 j=36
his sig2 i=80 j=36
his sxy i=80 j=36

*Displacements

his xd i=81 j=48
his xd i=81 j=45
his xd i=81 j=42
his xd i=81 j=39
his xd i=81 j=37
his yd i=81 j=48
his yd i=81 j=45
his yd i=81 j=42
his yd i=81 j=39
his yd i=81 j=37

*DH89-S9

*Stresses

his sig1 i=92 j=47
his sig2 i=92 j=47
his sxy i=92 j=47

*130-150 metre depth

his sig1 i=92 j=45
his sig2 i=92 j=45
his sxy i=92 j=45

*Displacements

his xd i=93 j=48
his xd i=93 j=46
his xd i=93 j=44
his yd i=93 j=48
his yd i=93 j=46
his yd i=93 j=44

*Set the hydrostatic groundwater level (approx. 1989 levels)

<table>
<thead>
<tr>
<th>Table 1</th>
<th>1950,240 2200,130 2390,80 2620,45 2860,19 3000,17 3180,17</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table 1</td>
<td>-360,1020 0,1080 750,900 1175,680 1375,590 1595,465 1785,360</td>
</tr>
</tbody>
</table>

water dens=0.001 table=1

*Establish initial stress distribution in model (equilibrium condition)
step 12000
save wfdg-3a.sav
call wfdg-3b.dat

*end of file
Wahleach Rock Slope Deformation Analysis - File: WFDG-3B.dat
*Slope formation - Removal (erosion) of slope overburden
*Model geometry includes 2 interfaces - detailed grid geometry.
*File written: 11-May-96  By: TWS
*File revised: 13-May-96  By: TWS (1st revision)
restore wfdg-3a.sav

Title
Wahleach Rock Slope Deformation Analysis - File: WFDG-3B
*Current cycle @ N=12000
*Reset slope displacements
ini xd=0.0 yd=0.0 xv=0.0 yd=0.0
*Remove slope overburden progressively
model null i=8,123 j=53
model null i=1,6  j=51
model null i=125,127 j=33
step 4400
model null i=8,123 j=52
model null i=1,6  j=50
model null i=125,127 j=32
step 4400
model null i=8,123 j=51
model null i=1,6  j=49
model null i=125,127 j=31
step 4400
model null i=8,123 j=50
model null i=1,6  j=48
model null i=125,127 j=30
step 5000
.model null i=8,123 j=49
model null i=1,6  j=47
model null i=125,127 j=29
step 6100
model null i=8,123 j=48
model null i=1,6  j=46
model null i=125,127 j=28
step 6200
*Slope excavated to current 'post-glacial' profile
save wfdg-3b.sav
call wfdg-3c.dat
Wahleach Rock Slope Deformation Analysis
*File: W-U-la.dat
*File written: 11 January 1996 By: TWS
*File revised: 25 January 1996 (revision 1)
*Consolidation of initial block using elastic constitutive relation

Heading

Wahleach Rock Slope Deformation Analysis - File: W-U-la

round 0.05
Set edge 2.0

*Set the block size
block 0,0 0,1140 3000,1140 3000,0

*Set the slope surface
crack -1,1140 930,940
crack 930,940 1555,605
crack 1555,605 1950,290
crack 1950,290 2200,150
crack 2200,150 2500,75
crack 2500,75 2750,50
crack 2750,50 3000,50

*Set the subsurface structure
crack 930,-1 930,940
crack 1555,-1 1555,605
crack 1950,-1 1950,290
crack 2200,-1 2200,150
crack 2500,-1 2500,75
crack 2750,-1 2750,50
crack 0,1000 930,750

*Set sequential slope overburden layers
crack 930,940 3000.5,500
crack 1555,605 3000.5,250

*Set the two principal joint sets
*Major set dipping into the slope @ 85 degrees
*Secondary joint set dipping downslope @ 30 degrees

jregion 0,1000 0,1140 930,940 930,750
jset 85,0 300,0 0,0 40,0
jset -30,0 500,0 0,0 40,0

jregion 930,750 930,940 1555,605 1555,450
jset 85,0 300,0 0,0 25,0
jset -30,0 500,0 0,0 25,0

jregion 1555,450 1555,605 1950,290 1950,175
jset 85,0 250,0 0,0 25,0
jset -30,0 300,0 0,0 25,0

jregion 1950,175 1950,290 2200,150 2200,100
jset 85,0 100,0 0,0 50,0
jset -30,0 100,0 0,0 50,0
pbl bl num hold

320
*Discretize the deformable blocks

```
gen  region 0,0 0,1140 930,940 930,0 edge 200
  gen  region 930,0 930,750 1555,450 1555,0 edge 150
  gen  region 930,750 930,940 1555,605 1555,450 edge 100
  gen  region 1555,0 1555,450 1950,175 1950,0 edge 150
  gen  region 1555,450 1555,605 1950,290 1950,175 edge 100
  gen  region 1950,0 1950,175 2200,100 2200,0 edge 150
  gen  region 1950,175 1950,290 2200,150 2200,100 edge 100
  gen  region 2200,0 2200,150 2500,75 2500,0 edge 150
  gen  region 2500,0 2500,75 2750,50 2750,0 edge 150
  gen  region 2750,0 2750,50 3000,50 3000,0 edge 150
```

*Set the slope material properties

*Slope overburden is modelled as simple linear, elastic material

```
prop   mat=1 d=2700 k=10e9 g=7e9 jkn=15e9 jks=10e9
```

*For the lower rock mass apply the following rock mass properties

```
change region 0,0 0,1140 930,940 930,0 cons=3 mat=2
change region 930,0 930,940 1555,605 1555,0 cons=3 mat=2
change region 1555,0 1555,605 1950,605 1950,0 cons=3 mat=2
change region 1950,0 1950,605 2200,150 2200,0 cons=3 mat=2
change region 2200,0 2200,150 2500,75 2500,0 cons=3 mat=2
change region 2500,0 2500,75 2750,50 2750,0 cons=3 mat=2
change region 2750,0 2750,50 3000,50 3000,0 cons=3 mat=2
prop mat=2 d=2700 k=10e9 g=7e9 fr=65 coh=5e6 di=5 t=4e6
prop mat=2 jkn=15e9 jks=10e9
pl model hold
```

*Set the joint material properties

```
prop  jmat=1 jkn=15e9 jks=10e9 jfric=60 jd=5 jcoh=5e6 jten=1e6
prop  jmat=1 jp=300 ar=0.001 az=0.005
prop  jmat=2 jkn=15e9 jks=10e9 jfric=60 jd=5 jcoh=5e6 jten=1e6
prop  jmat=2 jp=300 ar=0.001 az=0.005
change angle 80,88 jmat=3
prop  jmat=3 jkn=15e9 jks=10e9 jfr=35 jd=5 jcoh=1e6 jten=1e6
prop  jmat=3 jp=300 ar=0.01 azero=0.005
change angle -35,-25 jmat=5
prop  jmat=5 jkn=15e9 jks=10e9 jfric=35 jd=5 jcoh=1e6 jten=1e6
prop  jmat=5 jp=300 ar=0.01 azero=0.005
plot material joint hold
```

*Set initial and boundary conditions

```
in 0 3000 0 1140 st -15.1e6 0.0 -30.2e6 yg 13243.5 0.0 26487
insitu 0 3000 0 1140 szz -15.1e6 zgrad 0 13243.5
*insitu ywtable 2000
*fluid dens=1000 bulkw=2e9
*set flow off
boundary -.1 .1 0 1140 xvel=0.0
boundary 2999.49 3000.5 0 1139.9 xvel=0.0
boundary -.1 3000.1 -0.1 0.1 xvel=0.0
boundary -.1 3000.1 -0.1 0.1 yvel=0.0
pl bound xcon hold
pl bound ycon hold
```

*Set up instrumentation points

```
hist unbal
```
hist sxx 1500,0 syy 1500,0 szz 1500,0
hist sxx 1555,600 syy 1555,600 sxy 1555,600 szz 1555,600
hist xdis 0,1120
hist ydis 0,1120
hist xdis 930,940
hist ydis 930,940
hist xv 930,940
hist yv 930,940
hist xdis 1555,605
hist ydis 1555,605
hist xdis 1555,570
hist ydis 1555,570
hist xdis 1950,290
hist ydis 1950,290
hist xdis 2750,50
hist ydis 2750,50

set gravity 0.0 -9.81
damp auto

*Establish initial stress distribution in block model
cycle 4000
save w-u-la.sav

call w-u-1b.dat

*End of file
*Wahleach Rock Slope Deformation Analysis - File: W-U-1b.dat
*File written: 3-Jan-96 By: TWS
*File revised: 23-Jan-96
*Remove overburden layers progressively
   restore w-u-la.sav

Heading
Wahleach Rock Slope Deformation Analysis - File: W-U-1b
reset damp
damp auto
prop mat=2 cch=5e6 t=1e6
prop jmat=1 jc=10e6 jt=1e6
prop jmat=2 jc=10e6 jt=1e6
prop jmat=3 jc=10e6 jt=1e6
prop jmat=5 jc=5e6 jt=1e6
*Reset displacements
reset disp
reset jdisp
reset rota
*Remove layer #2
del bl 2542
cycle 3000
del bl 2723
cycle 3000
*Remove layer #3
*Reset displacements
reset disp
reset jdisp
reset rota
del bl 242
cycle 3000
save w-u-1b.sav
call w-u-1wat.dat